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FEASIBILITY REPORT AND ENVIRONMENTAL IMPACT STATEMENT: FARGO-MOORHEAD METROPOLITAN AREA FLOOD RISK MANAGEMENT, JULY 2011

COMMUNICATION

FROM

THE ASSISTANT SECRETARY OF THE ARMY, CIVIL WORKS, THE DEPARTMENT OF DEFENSE

TRANSMITTING

THE CORPS FINAL FEASIBILITY REPORT AND ENVIRONMENTAL IMPACT STATEMENT

PART 2 OF 7

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INFORMATION
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Final Feasibility Report and Environmental Impact Statement

Fargo-Moorhead Metropolitan Area Flood Risk Management

Volume 2
Appendix A, Attachment 5 continued

July 2011

US Army Corps of Engineers ®

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RED RIVER DIVERSION

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

APPENDIX C – HYDRAULICS WITH–PROJECT CONDITIONS

ATTACHMENT C1 – EXCERPT FROM APPENDIX A-HYDROLOGY FROM THE JULY 30, 2010 REPORT

Report for the US Army Corps of Engineers, and the cities of Fargo, North Dakota & Moorhead, Minnesota

FINAL: February 28, 2011

ATTACHMENT C1

A1.0 BACKGROUND

A1.1 Need for Study. The Red River has posed a repeated threat to the communities of Fargo, ND and Moorhead, MN. A flood has occurred at least once each year since 1993. Although the communities have demonstrated significant resilience in defending against the floods, the efforts can be massive and highly disruptive to the communities and the people. Various plans have been formulated that address portions of the communities and have been developed to varying degrees. This study looks at the potential to develop a single comprehensive system to address flooding for the entire community.

A1.2 Related Studies and Project Evolution. Since the beginning of this study, several reports have been produced. Although this report is meant to stand alone, the other reports are listed below for completeness. Sufficient data from the other reports will be included in this report to allow the reader an understanding of the evolution of the project hydrology. However, not all details from the previous report will be copied into this report. In order to clarify the progression of the phases of this project and how they relate to the evolution of the hydrology; Table A1 provides a summary of Reports, Phases and Hydrology. Hydrology phases have progressed from Phase 1 to Phase 3.1.

Table A1 - Project Phases as Related to Hydrology

Study Phase	Report Date	Hydrology Phase	Notes
Phase 1	March 2009	Phase 1	Draft Report March 2009 by US Army Corps of Engineers
Phase 2	August 2009	Phase 1	Initial work by Moore Eng. and others
Phase 2	October 2009	Phase 2	Low Flow Analysis
Phase 2 Part 2	December 2010	Phase 2	Updated in January 2010
Phase 2 Part 2	January 2010	Phase 2	Northwest Diversion
Phase 3	May 2010	Phase 3	Hydrology updated for Wet and Dry cycles
Phase 3 (3.1)	July 2010	Phase 3.1	Hydrology amended, study limited to LPP and FCP

A1.2.1 Ongoing FEMA Restudy. Since prior to the 1997 flood, FEMA has been in the process of updating the Flood Insurance Rate Map (FIRM) for the area of Cass and Clay Counties in the vicinity of Fargo and Moorhead as part of its Map Modernization Program. Two studies in particular have defined much of the area of the Red River adjacent to the two cities. They include the "Flood Insurance Restudy for Southern Cass County, North Dakota, and Clay County, MN" (Reference A), which defines the hydrology for the area south of Fargo and the "City of Fargo CTP Project, Clay County/Oakport Township," (Reference B). CTP is short for Coordinating Technical Partner. Table A2 is an abbreviated listing of the Red River hydrology used in these studies. Flows are in terms of cubic feet per second (cfs).

Table A2 - FEMA Hydrology

River Station	10 year	50 year	100 year	500 year
Hickson	7,648	12,307	14,173	21,818
Fargo	10,300	22,300	29,300	50,000

A1.2.2 August 31, 2009 Report. In August 2009, a report entitled *"Fargo - Moorhead Metro Flood Risk Management Project, Feasibility Study Report for the US Army Corps of Engineers, and the Cities of Fargo, ND and Moorhead, MN, Phase 2"* (Reference C), was completed that looked at a variety of alternatives for a Red River Diversion. Specifically,

four alignments were considered around the FM Metro Area. The hydrology was based on Phase 1 Hydrology developed by the US Army Corps of Engineers (USACE) and presented in USACE report of *"Fargo - Moorhead Metro Flood Risk Management Project Red River of the North, Fargo, ND and Moorhead, MN"* (Reference D). Table A3 provides an abbreviated listing of the Red River hydrology used in these studies. Flows are in terms of cfs.

Table A3 - USACE Phase 1 Hydrology

River Station	10 year	50 year	100 year	500 year
Hickson	6,764	12,174	17,879	34,445
Fargo	12,700	23,000	31,000	57,400

A1.2.3 Low Flow Hydrology. In October 2009, a report on low flow conditions entitled *"Appendix H - Red River Diversion Hydraulic Structure Velocities"* (Reference E), was completed. This report was written in terms of an additional appendix to the report provided in Reference C. The purpose of this portion of the overall study was to gain a better understanding of the Red River under frequent flooding events smaller than the 2-year event. This analysis was particularly important in looking at fish Table A4 provides an abbreviated listing of the Red River hydrology used in this analysis. Flows are in terms of cfs. The low flow hydrology provided is based on USACE Phase 2 Hydrology of the Red River. It should be noted that the flows for the 99.99% through 80% exceedance frequencies were not changed in Phase 3 or 3.1 Hydrology. The flow exceedance of 99.99% is an estimate of the exceedance related to 50 cfs of flow. 50 cfs of flow was set as a boundary on the hydraulic modeling. Below this amount of flow, there were concerns about both the relevance of the data to the project and the ability to accurately model the Red River.

Table A4 - Red River Low Flow Phase 2 Hydrology

River Station / Flow Exceedance	99.99% est.(cfs)	99% (cfs)	95% (cfs)	90% (cfs)	80% (cfs)	50% (cfs)	20% (cfs)
Hickson	50	251	547	809	1,250	2,550	5,900
Fargo	50	251	550	829	1,360	3,500	9,600

A1.2.4 December 31, 2009 (updated January 6, 2010) Report. In December 2009, a report entitled, *"Fargo - Moorhead Metro Flood Risk Management Project, Feasibility Study Report for the US Army Corps of Engineers, and the Cities of Fargo, ND and Moorhead, MN, Phase 2 - Part 2"* (Reference F), was completed that updated the progress of the study and updated the hydrology to Phase 2 Hydrology. Phase 2 Hydrology was developed to update the Red River hydrology to incorporate the 2009 flood event. This report was further updated January 6, 2010. Table A5 provides an abbreviated listing of the Red River hydrology used in this phase of the study. Flows are in terms of cfs.

Table A5 - USACE Phase 2 Hydrology

River Station	2 year	5 year	10 year	20 year	50 year	100 year	200 year	500 year
Hickson	3,139	6,160	9,555	13,729	19,709	23,757	26,164	30,016
Fargo	4,352	10,608	15,394	20,345	27,441	32,921	42,242	57,641

A1.2.5 Northwest Diversion. In January 2010, a report entitled "*Northwest Diversion, Fargo - Moorhead Metro Flood Risk Management Project, Feasibility Study Report, Phase*

2 - Part 2, for the US Army Corps of Engineers, and the Cities of Fargo, ND and Moorhead, MN," (Reference G) was completed. The Northwest Diversion Alternative was created as a means to deal with flooding north of Fargo and West Fargo from the Sheyenne River and its tributaries if only a Minnesota Diversion Alternative was constructed. This diversion needed to consider not only a Red River flood but also local flooding from the Sheyenne River. Table A6 provides a summary of the local flood hydrology for the Sheyenne River. The hydrology was developed from the existing Flood Insurance Study (FIS) and modified to take into account additional modeling done in the area.

Table A6 - Northwest Diversion Reference Hydrology

Location	10 year	50 year	100 year	500 year
Sheyenne River south of Horace, ND	3,400	4,500	4,600	4,600
Local Input, Distributed flow from U/S end to I-94 on the Sheyenne R. / Div	200	750	750	3,100
Split to the H to WF reach of the Sheyenne R. Div (1,2)	1,660	2,510	2,580	2,580
Sheyenne River D/S of Diversion Inlet (2)	1,740	1,990	2,020	2,020
West Fargo reach of Sheyenne R. Div D/S of outlet from H to WF reach (2)	3,400	4,500	4,600	4,600
West Fargo reach of Sheyenne R. Div at I-94	3,600	5,250	5,350	7,700
Local Input, Overland flooding assumed concentrated at Drain 14	0	1,950	1,960	7,600
Maple River Inflow	2,390	5,170	7,940	9,550
Lower Rush River Inflow	40	410	750	1,350
Rush River Inflow	510	820	1,500	2,700
Sheyenne R. at Red R.(3)	6,750	13,600	17,500	28,900

1 - Split is estimated. The hydraulic model conducts an energy balance at this location to determine the actual split.
2 – The listed flow does not include the distributed flow that is applied from the upstream end of the Sheyenne River to Interstate 94.
3 – The listed flow represents the total Sheyenne River system flow at the Red River. A portion of this flow would break out to Drains 45 and 13.

A1.2.6 May 17, 2010 Report. In May 2010 a report entitled, *"Fargo - Moorhead Metro Flood Risk Management Project, Feasibility Study Report for the US Army Corps of Engineers, and the Cities of Fargo, ND and Moorhead, MN, Phase 3"* (Reference H), was completed that updated the study progress and hydrology. Phase 3 Hydrology was developed as part of a need to look at the Red River hydrology as two dissimilar periods of record. Additional discussion on this aspect of the hydrology is provided below. Table A7, provides an abbreviated listing of the Red River hydrology used in this phase of the study. Flows are in terms of cfs.

Table A7 - USACE Phase 3 Hydrology

River Station	2 year	5 year	10 year	20 year	50 year	100 year	200 year	500 year
Hickson	4,000	7,000	10,500	14,800	21,000	25,000	28,500	32,000
Fargo	5,600	12,150	17,000	22,000	29,300	34,700	46,200	61,700

A1.3 Need for Phase 3 Hydrology. It has been noted that the past few decades represent most of the significant floods on record in the FM Metro Area. A review of the period of record for the Red River gages illustrates two dissimilar hydrology patterns. The first portion of the 20th Century is characterized by many of the lower flows in the Red River including the drought years of the 1930's. The latter portion of the 20th Century and the past decade represent a time of larger flows including the larger floods of 1969, 1993, 1997, and 2009. To characterize this as simply a shift in climate and would be an over simplification considering the flood of 1897. To address the concern with dissimilarity in the period of record, the St. Paul District of the USACE contracted with the USACE Hydrologic Engineering Center (HEC) to regenerate the discharge frequency curves for the Red River.

A1.4 Overview of Phase 3 Hydrology. Phase 3 Hydrology re-characterizes the project discharge frequency curve as three separate curves based on a "Wet" condition and a "Dry" condition. The start of the project is referred to as Year 0 which uses the "Wet" condition as the hydrology. Over time, the hydrology is expected to shift and Year 25 of the project is characterized by a discharge frequency curve that is prorated between the "Wet" and "Dry" curves with an 80% and 20% split. Finally, at Year 50 is the final discharge frequency curve used is a 65/35 split of the "Wet" and "Dry" curves.

A1.5 Phase 3.1 Hydrology, Update to Phase 3. It was noted that Phase 3 Hydrology increased the flows in Fargo significantly. However, the flows further downstream at locations such as Halstad and Grand Forks did not increase significantly and were similar to values used in Phase 2 Hydrology. This in effect reflected an increase in contribution of drainage areas south of Fargo and reduced the overall contribution of watersheds north of Fargo. This effect was also largely confined to events larger than the 50-year event. The Sheyenne River contribution is not calculated through an independent statistical analysis of gage data but rather is derived based on drainage area transfer after accounting for locations such as Fargo, Halstad, and the contributions of the Buffalo River and the Wild Rice River of Minnesota. (Drainage area transfer is a method of calculating flow upstream or downstream of a given location or to another watershed based on a previously determined flow, empirical formulas, and the drainage area).This method of calculation on the Red River Watershed allowed for an anomaly to occur in the data where the 100-year flow contribution from the Sheyenne River could exceed the 200-year or 500-year flow contribution from the Sheyenne River. Phase 3.1 Hydrology was developed to address this anomaly in the data. The flows at Fargo and Halstad were unchanged but the intermediate points along the Red River were adjusted to allow for a more conventional progression of flow from each watershed with increasing return period. Table A8, provides a summary of the contribution from the Sheyenne River. As Phase 3 Hydrology is characterized by the project year, only the Year 0 is provided in this table.

Table A8 - Coincidental Sheyenne River Flows, Phase 3.1 Hydrology (Year 0)

River Station	2 year	5 year	10 year	20 year	50 year	100 year	200 year	500 year
Upstream of Sheyenne R.	5,908	12,618	17,616	22,791	30,340	35,970	47,541	63,141
Downstream of Sheyenne R.	8,857	16,795	23,062	29,776	39,503	47,212	59,029	75,188
Diff (Sheyenne R.)	2,949	4,177	5,446	6,985	9,163	11,242	11,488	12,047

RED RIVER DIVERSION

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

APPENDIX C – HYDRAULICS WITH–PROJECT CONDITIONS

ATTACHMENT C2 – RED RIVER DIVERSION, FARGO-MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 3 (PHASE 3.1 HYDROLOGY), REPORT FOR THE US ARMY CORPS OF ENGINEERS, AND THE CITIES OF FARGO, NORTH DAKOTA & MOORHEAD, MINNESOTA, APPENDIX B-HYDRAULICS, AUGUST 11, 2010 VERSION

Report for the US Army Corps of Engineers, and the cities of Fargo, North Dakota & Moorhead, Minnesota

FINAL: April 11, 2011

RED RIVER DIVERSION

FARGO - MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 3 (PHASE 3.1 HYDROLOGY)

Report for the US Army Corps of Engineers, and the cities of Fargo, North Dakota & Moorhead, Minnesota

APPENDIX B - HYDRAULICS

By: MOORE ENGINEERING

Aug 11, 2010

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ATTACHMENTS

Attachment B1 - Moore Engineering Memorandum 2009 Calibration of HEC-RAS Steady State Model (04/11/2010)

Attachment B2 - Internal Review, QA/QC Model Review Comments

B1.0 RED RIVER DIVERSION - BACKGROUND AND OVERVIEW OF ALIGNMENTS

B1.1 Need for Study. The Red River has posed a repeated threat to the communities of Fargo, ND and Moorhead, MN. Since 1993, a flood has occurred at least once each year. Although the communities have demonstrated significant skill in defending themselves against floods, the efforts can be massive and highly disruptive to the communities and the people. Various plans have been formulated that address portions of the communities and have been developed to varying degrees. This study looks at the potential to develop a single comprehensive system to address flooding for the entire community. Appendix A - Hydrology covered the hydrology of the project area both with and without a project in place. This appendix, Appendix B – Hydraulics, looks at the hydraulic modeling of the Red River and its tributaries in conjunction with a Red River Diversion.

B1.2 Related Studies and Project Evolution. Since the beginning of this study, several reports have been produced. Although this report is meant to stand alone, the other reports are listed below for completeness. Sufficient data from the other reports will be included in this report to allow the reader an understanding of the evolution of the project, but not all details from the previous report will be copied into this report. In order to clarify the progression of the phases of this project and how they relate to the evolution of the hydrology and the hydraulics; a copy of Table A1 from Appendix A – Hydrology is provided below. Hydrology has progressed from Phase 1 to Phase 3.1.

Copy of Table A1 – Project Phases as Related to Hydrology (from Appendix A)

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Phase 3	May 2010	Phase 3	Hydrology updated for West and Dry cycles
Phase 3 (3.1)	July 2010	Phase 3.1	Hydrology amended, study limited to LPP and FCP

B1.2.1 Ongoing FEMA Restudy. Since prior to the 1997 flood, FEMA has been in the process of updating the Flood Insurance Rate Map (FIRM) for the area of Cass and Clay counties in the vicinity of Fargo and Moorhead as part of its Map Modernization program. Two studies in particular have defined much of the area of the Red River adjacent to the two cities. They include the "*Flood Insurance Restudy for Southern Cass County, North Dakota, and Clay County, MN*" (Reference A), which defines the hydrology for the area south of Fargo and the "*City of Fargo CTP Project, Clay County/Oakport Township,*" (Reference B) (Note: CTP is short for Coordinating Technical Partner). The FEMA model served as the basis for hydraulic modeling on the Red and Wild Rice Rivers during the initial phase of the FM Metro study.

B1.2.2 August 31, 2009 Report. In August 2009, a report entitled *"Fargo - Moorhead Metro Flood Risk Management Project, Feasibility Study Report for the US Army Corps of Engineers, and the Cities of Fargo, ND & Moorhead, MN, Phase 2"* (Reference C), was

completed that looked at a variety of alternatives of sizing and flows for a Red River Diversion. Specifically, four alignments were considered around the FM Metro Area. Two of the alignments were considered on the Minnesota side. Minnesota Long Diversion is the same alignment as Minnesota Short Diversion downstream of a point west of Sabin, MN. Upstream of that location, the Minnesota Long Diversion alignment extends farther south to intersect the Red River at a location approximately 3 miles to the south. In a similar fashion, the two North Dakota alignments are also similar. The primary difference between the two alignments, North Dakota West Diversion and North Dakota East Diversion, is the portion of the channel between Horace and West Fargo. As a result of public input, the East alignment was developed so that it closely followed the current Sheyenne River Diversion alignment within this area. Figure B1 provides a layout of the four alignments from this phase of the study. Although these alignments are not current, they serve to show some of the alignments considered. In addition to the four alignments, nine (9) options of flow in the Red River Diversion were considered. They include:

1. MN Short Alignment 45 Kcfs Alternative,
2. MN Short Alignment 35 Kcfs Alternative,
3. MN Short Alignment 25 Kcfs Alternative,
4. MN Long Alignment 45 Kcfs Alternative,
5. MN Long Alignment 35 Kcfs Alternative,
6. MN Long Alignment 25 Kcfs Alternative,
7. ND West Alignment 45 Kcfs Alternative,
8. ND West Alignment 35 Kcfs Alternative,
9. ND East Alignment 35 Kcfs Alternative.

B1.2.3 Low Flow Modeling. In October 2009, a report on low flow conditions entitled "*Appendix H - Red River Diversion Hydraulic Structure Velocities*" (Reference D), was completed. This report was written in terms of an additional appendix to Reference C. The purpose of this portion of the overall study was to gain a better understanding of the Red River under frequent flooding events smaller than the 2-year event. This analysis was particularly important in looking at fish passage. It should be noted that with the Red River 2-year and smaller events the Red River Diversion would not be operating. Therefore, the analysis only involved the main channel and the Red River Control Structure. The low flow events below the 2-year event considered were:

1. 80% Exceedance or a 1.25-year recurrence,
2. 90% Exceedance or a 1.11-year recurrence,
3. 95% Exceedance or a 1.05-year recurrence,
4. 99% Exceedance or a 1.01-year recurrence,
5. 50 cfs which was estimated at 99.99% Exceedance.

B1.2.4 December 31, 2009 (Updated January 6, 2010) Report. In December 2009, a report entitled *"Fargo - Moorhead Metro Flood Risk Management Project, Feasibility Study Report for the US Army Corps of Engineers, and the Cities of Fargo, ND & Moorhead, MN, Phase 2 - Part 2"* (Reference E), was completed that updated the progress of the study and updated the hydrology to Phase 2 Hydrology. Phase 2

Hydrology was developed to update the Red River hydrology to incorporate the 2009 flood event. This report was further updated in January of 2010. As part of this phase of the study, the number of alignments was reduced to two, the Minnesota Short Alignment and the North Dakota East Alignment. No significant changes were made to the alignments from the previous phase, therefore a separate set of figures are not provided. This phase looked to determine the National Economic Development (NED) Alternative. Therefore several flow options were considered. These flow options were primarily for the Minnesota Short Alignment as it was determined that this alignment, of the two considered in this phase, would be the NED alignment. The flow options considered were:

1. MN Short Alignment 10 Kcfs Alternative,
2. MN Short Alignment 15 Kcfs Alternative,
3. MN Short Alignment 20 Kcfs Alternative,
4. MN Short Alignment 25 Kcfs Alternative,
5. MN Short Alignment 30 Kcfs Alternative,
6. MN Short Alignment 35 Kcfs Alternative,
7. ND East Alignment 30 Kcfs Alternative,
8. ND East Alignment 35 Kcfs Alternative.

B1.2.5 Northwest Diversion. In January 2010, a report was completed on the Northwest Diversion Alternative. The report entitled "*Northwest Diversion, Fargo - Moorhead Metro Flood Risk Management Project, Feasibility Study Report, Phase 2 - Part 2, for the US Army Corps of Engineers, and the Cities of Fargo, ND & Moorhead, MN*," is listed as Reference F. The Northwest Diversion Alternative was created as a means to deal with flooding north of Fargo and West Fargo from the Sheyenne River and its tributaries if a Minnesota Diversion Alternative was constructed. This diversion needed to consider not only a Red River flood but also local flooding from the Sheyenne River itself. The hydraulic model used for this part of the study used an unsteady model of the Red River and its tributaries, which was under development to look at downstream impacts of the project. The model was reduced to the portion dealing with the Sheyenne River system. The local flood events considered for this part of the study were the 10-, 50-, 100- and 500-year events. Figure B2 provides a plan view of this alternative.

B1.2.6 May 17, 2010 Report. In May 2010, a report entitled *"Fargo - Moorhead Metro Flood Risk Management Project, Feasibility Study Report for the US Army Corps of Engineers, and the Cities of Fargo, ND & Moorhead, MN Phase 3"* (Reference G), was completed that updated the study progress and hydrology. Phase 3 Hydrology was developed as part of a need to look at the Red River hydrology as two dissimilar periods of record. One period of record was characterized as "Wet" while the other was characterized as "Dry". From these two records, hydrology was developed for the project year zero, project year 25, and project year 50. Project year zero has the greatest flow through the FM Metro with decreasing flows in the subsequent two project year time periods. Because of this, it was necessary to run models for each alternative and flow option for three project time periods. Additional discussion on this aspect of the hydrology is provided in Appendix A – Hydrology. As part of this phase, only two Red River

Diversion alignments were considered, Minnesota Short Alignment and North Dakota East Alignment. The May 17th report was the first version of the Phase 3 report and eventually evolved into this current report, which is identified as Phase 3 with Phase 3.1 Hydrology. Changes to the alignments were being considered with the May 17th version and were implemented with the current phase. Figures showing the current alignments are provided later in the report. With new hydrology, it was necessary to revise the NED analysis. The NED analysis focused on the Minnesota Short Alignment. The North Dakota East Alignment 35 Kcfs alternative was also considered due to being chosen as the Locally Preferred Plan (LPP). The flow options for the Red River Diversion considered during this phase were:

1. MN Short Alignment 20 Kcfs Alternative,
2. MN Short Alignment 25 Kcfs Alternative,
3. MN Short Alignment 30 Kcfs Alternative,
4. MN Short Alignment 35 Kcfs Alternative,
5. MN Short Alignment 40 Kcfs Alternative,
6. MN Short Alignment 45 Kcfs Alternative,
7. ND East Alignment 35 Kcfs Alternative.

B1.3 Phase 3.1 Hydrology, Update to Phase 3. It was noted that Phase 3 Hydrology increased the flows in Fargo significantly. Conversely, the flows further downstream at locations such as Halstad and Grand Forks did not increase significantly and were similar to values used in Phase 2 Hydrology. As a result, some inconsistencies were noted, particularly with the Sheyenne River coincidental flows. Given the importance of the Sheyenne River on project parameters, a revision to the Phase 3 Hydrology was developed. As this new hydrology did not represent a fundamental update but rather a smaller change, it was referenced as Phase 3.1 for the study. For more details on Phase 3 and Phase 3.1 Hydrology, see Appendix A - Hydrology. The current phase of the project is still considered Phase 3, but it now references Phase 3.1 hydrology. This is the fundamental difference between this report and the May 17th version. At this point, the alternatives have still been narrowed down to two alternatives. These two alternatives represent the Federally Comparable Plan (FCP) and the Locally Preferred Plan (LPP) plan which are:

1. MN Short Alignment 35 Kcfs Alternative - FCP
2. ND East Alignment 35 Kcfs Alternative - LPP

B1.4 Overview of Channel Alignments. The two channel alignments still under consideration are the Minnesota Short Diversion and the North Dakota East Diversion. The Minnesota Short Diversion diverts water from the Red River just downstream of the Wild Rice River mouth to just downstream of the Sheyenne River mouth. The North Dakota East Diversion diverts water from both the Red River and Wild Rice River to a location farther downstream of the Sheyenne River mouth. Figure B3 shows the location of the two alignments. A more detailed map for each channel alignment is provided in Figures 1 and 2 of the Main Report.

B2.0 MINNESOTA DIVERSION - ALIGNMENT DESIGN CONSTRAINTS

B2.1 Initial Limitations (Minnesota Short and Long Alignments). As discussed above, the alignment on the Minnesota side- referred to as the Minnesota Short Alignment- was developed during the initial phase of the study as one of two potential alignments on the Minnesota side of the Red River. Both Minnesota alignments were restricted by a few key elements. The first objective was that no rivers be crossed. This meant that the downstream outlet had to enter the Red River prior to the Buffalo River and that the alignment could not go east of the Buffalo River. Another condition on the outlet was that it needed to be located such that the diversion flow entered the Red River parallel to a river tangent rather than at a right angle to minimize river channel erosion and to provide better outflow conditions. Another objective was to have the diversion pass east of the urban area as far as possible. This objective was balanced with the need to remain west of the Buffalo Aquifer. Some existing features were avoided due to the higher cost of replacement, such as highway bridge interchanges and the Moorhead Airport. The impact to the Dilworth Rail yard was also considered. The overall length was reduced by using a centerline not parallel to section lines to reduce the overall cost.

B2.2 Extension Channels.

B2.2.1 Minnesota Long Alignment Connecting Channel. The Minnesota Long Diversion intersects the Red River south of Cass County Highway 16 and has an extension channel that extends from the Red River to the Wild Rice River near Cass County Drain 47. This extension channel if referred to as the Minnesota Long Diversion Connecting Channel as it connects the Wild Rice and Red Rivers. Because Red River is intercepted so far to the south and the channel connects to the Wild Rice River, the Minnesota Long Alignment could pick up flows from the two rivers before they would cause the breakouts that occur between the two rivers and the breakouts into the Rose Coulee Watershed. See Figure B1 for additional detail on the Minnesota Long Diversion Alignment.

B2.2.2 Minnesota Short Alignment Extension and Breakout Channels. The Minnesota Short Alignment intersects the Red River north (downstream) of the confluence with the Wild Rice River. This location allows it to pull flow into the Red River Diversion from the total flow of both rivers. By itself; however, this alignment does not prevent the exchange of flows between the two rivers and it does not prevent the breakout flows into the Rose Coulee Watershed. An extension channel added to the Minnesota Short Alignment extends from near the inlet of the diversion south to intersect the Red River near Cass County Highway 16. This extension channel allows the Minnesota Short Alignment to pick up additional flow from the Red River at the location where breakouts from the Wild Rice River enter the Red River. This is of particular importance because a tie-back levee associated with the Minnesota Short Alignment would prevent flow into Rose Coulee. This would likely force more water from the Wild Rice to the Red River in this breakout location; thus causing stage increases on the Red River. A second extension channel with the Minnesota Short Alignment connects an existing breakout location at Cass County Highway 16 and west of the I-29 interchange to

the St. Benedict Coulee where the coulee intercepts the Wild Rice River. This extension channel is referred to as the Wild Rice River Breakout Channel. This breakout channel is in place to mimic an existing breakout location and prevent stage increases on the Wild Rice River. See Figure B3 for additional detail on the Minnesota Short Diversion Alignment.

B2.3 Tie-back Levees. Both Minnesota Alignments require a tie-back levee to tie into high ground on the North Dakota side of the Red River. The Minnesota Long Alignment tie-back levee continues from the extension channel between the Red and Wild Rice Rivers and then proceeds west along Cass County Drain 47 to tie into higher ground near the Sheyenne River. The Minnesota Short Alignment requires a longer tie-back levee, which extends from the control structure on the Red River; then west and south near the Wild Rice River to the extension channel at St. Benedict; then west on the north side of Cass County Highway 16 to tie into high ground near the Sheyenne River. The tie-back levee not only ties the system into high ground, but also serves to prevent breakouts into the Rose Coulee Watershed.

B2.4 Alignment Changes. During Phase 3 of the study, issues related to right-of-way, geotechnical considerations and other factors came to light. These issues resulted in changes to the diversion alignment, particularly in the reach north of Dilworth, MN, and the location of the Red River Control Structure was moved north approximately one river mile. Finally, the turn radii were increased to three-times the channel width (top flow width for 500-year event), which allows for better hydraulic performance and reduces erosion potential. These changes are reflected within this report.

B3.0 NORTH DAKOTA DIVERSION - ALIGNMENT DESIGN CONSTRAINTS

B3.1 Initial Limitations (North Dakota East and West Alignments). As discussed above, the alignment on the North Dakota side- referred to as the North Dakota East Alignment- was developed during the initial phase of the study as one of two potential alignments on the North Dakota side of the Red River. Both North Dakota alignments were restricted by a few key elements. On the North Dakota side, river crossings could not be avoided, but the diversion outlet was placed downstream of the Sheyenne - Red River confluence to minimize the number of crossings. Similar to the Minnesota alignments, the North Dakota outlet was located so that the diversion flow would enter the Red River parallel to a river tangent rather than at right angle to minimize river channel erosion. Cass County Drain 13 is a major drainage area north and west of Fargo, ND. The crossing of this drain under Interstate 29 is only a couple of miles north of the Sheyenne River and formed a logical crossing place for the diversion under Interstate 29. In order to minimize the length of the Red River Diversion, and in turn its cost, the alignment was situated close to the Sheyenne River. The crossing points of the other river systems, such as the Rush River and the Maple River, are within a mile or two of their confluences with the Sheyenne River. Similar to the Minnesota alignment, the alignment was designed to avoid, as much as possible, significant urban areas. As a result, the alignment passes south of Argusville, west of West Fargo, and south of

Horace. The inlet to the Red River Diversion was placed at the Wild Rice River and Red River at a location that would ensure sufficient was flow available to divert. (See Appendix A - Hydrology.) Furthermore, by using a location south of Cass County 16, the diversion is upstream of the existing breakouts into the Rose Coulee Watershed.

B3.2 West versus East Alignments. The North Dakota West and East alignments vary only between West Fargo and Horace. In this area, the west alignment goes roughly due north whereas the east alignment follows the existing Sheyenne River (Horace) Diversion. The east alignment was created at the request of the public, whom expressed a desire to utilize the existing Sheyenne River Diversion right-of way-corridor within the proposed Red River Diversion. As a result, the Red River Diversion - North Dakota East Alignment incorporates a portion of the Sheyenne River Diversion between Horace and West Fargo but preserves the West Fargo leg of the Sheyenne River Diversion.

B3.3 Alignment Changes. There have been no significant alignment changes to the North Dakota East Alignment from the alignment initially proposed at the start of this study to the current alignment; other than increasing curve radii similar to the Minnesota Short Alignment and the extension of the tie-back levee in Minnesota. Similar updates have not been incorporated into the North Dakota West Alignment because it is not currently under consideration as the Federally Comparable Plan or Locally Preferred Plan.

B3.4 Tie-back Levee. Phase 3.1 Hydrology represents a significant increase in Red River flows above that presented in Phase 1 Hydrology (see Appendix A - Hydrology). As a result, the height of the tie-back levees needed to be increased and the tie-back levee for the North Dakota East Alignment had to be extended to higher ground. Three alignments for the levee were considered, as discussed below. Figure B4 provides a plan layout of the three alignments considered. Figure B5 provides a layout of the Wolverton Creek crossing with the overflow channel (discussed later).

B3.4.1 North Dakota East Alignment Tie-Back Levee, South Option. This alignment starts at the Red River Control Structure and terminates just south of Comstock, Minnesota in Section 27 of Holy Cross Township. This option is intended to eliminate the need for a control structure on Wolverton Creek since the alignment stays west of Wolverton Creek. The disadvantage of this option is that there is not sufficient high ground west of Wolverton Creek other than the existing high ground on the left (west) bank of Wolverton Creek. While the ground might appear adequate on the profile, the tie-in is only marginal and would not be adequate if there are any future increases in the height of the proposed tie-back levee. There also is a concern with water entering Wolverton Creek upstream of the tie-back levee. This could circumvent the levee system and result in additional flow to Wolverton Creek.

B3.4.2 North Dakota East Alignment Tie-Back Levee, East Option 1. This alignment starts at the Red River control structure and terminates northeast of Comstock in Section 22 of Holy Cross Township. This alignment crosses Wolverton Creek so that a control structure is required in addition to the tie-back levee. As with South Option, the ground

might appear adequate on the profile; however, the tie-in is only marginal and would not be adequate if there are any future increases in the height of the proposed tie-back levee. There is also a concern with water entering Wolverton Creek upstream of the tie-back levee, similar to the South Option. This could circumvent the levee system and result in additional flow to Wolverton Creek.

B3.4.3 North Dakota East Alignment Tie-Back Levee, East Option 2. This alignment is similar to East Option 1; however, instead of extending south from Section 10 of Holy Cross Township, the tie-back levee is extended to a high ridge located just west of the Buffalo River. This option is the preferred option since there is sufficient high ground for a good tie-in with the existing ground, even if the levee is raised in the future. As with East Option 1, a control structure is required for Wolverton Creek.

B3.4.4 Hydraulic Sizing of the Wolverton Creek Control Structure. Hydraulic sizing of the Wolverton Creek Control Structure is discussed below as part of Section B6 - North Dakota East Alignment Conceptual Design.

B4.0 RED RIVER DIVERSION - MODEL CONSTRUCTION

B4.1 Overview. During the initial phase of the study, many of the channel design parameters were determined as part of establishing the feasibility of the project and establishing an estimated cost. Hydraulic modeling of the Red River Diversion was accomplished using the steady state routine within the HEC-RAS software developed by the US Army Corps of Engineers, Hydrologic Engineering Center. The hydraulic models of the Red River were originally developed as part of ongoing Flood Insurance Studies for the local area as provided in References A and B. The models have since been recalibrated for the 2009 flood event and the location and number of lateral weirs representing breakout flow locations has been adjusted. The models for the Red River Diversion were initially developed as separate models, which allowed for more efficient optimization of the channels and then were merged with the Red River model. Although some attempts were made to add the Wild Rice River to the model, it generally was found to be unnecessary and the Wild Rice is indirectly modeled through flow inputs. In addition, the Wild Rice River Breakout Channel that allows for breakouts from Cass County Highway 16 to St. Benedict Coulee was modeled in the earlier phases to optimize the size. This modeling has not been updated with the later phases.

B4.2 Modeling Assumptions. Although several assumptions were made early in the modeling process, many of these assumptions have since been verified or removed. Some of the remaining assumptions are provided below.

1) *A single "n" value for the diversion channel is assumed (0.03).* A single "n" value for all overbank areas are assumed (0.04). The "n" value for the channel was discussed at the kickoff meeting and is based on data from the Winnipeg Floodway. On the North Dakota side, "n" values have been adjusted to reflect the concrete and riprap channel linings associated with the hydraulic structures at the Sheyenne and Maple Rivers. Much of the area adjacent to the channel is

farmland allowing use of a single value in the overbanks. The actual value is based on past modeling experience.

2) *MINNESOTA ALIGNMENTS: Tributary flows entering the diversion were assumed to be inconsequential.* The upstream flow is the same as the downstream flow in the diversion. This assumption is consistent with the discussion presented in Appendix A - Hydrology.
3) *The Dilworth railroad yard may be treated as a single bridge.* This assumption will be refined with additional bridge design if necessary. However, the gaps between individual bridges would likely be small. Modeling as a single bridge is conservative. This issue is left pending final design. This same assumption was used where parallel roads and railroads are very close.
4) *Features in a reach could be used to prevent counting conveyance areas twice in two adjacent reaches.* Ineffective flow areas, levees, and blocked obstructions were used near the inlet and outlet of the diversion to account for the control structure and spoil banks. With the Minnesota Short Alignment, the effective flow limits for the Red River and the extension channel were adjusted to that they did not overlap as the two alignments are parallel and in close proximity to each other.
5) *Containment levees on the diversion above existing ground were modeled with levee cards.* The location of these levee cards were offset 50 feet from bank stations, which were placed where the channel excavation daylighted with existing ground. The 50 feet represents the required geotechnical bench as the diversion geometry daylights into existing ground. The location of these levee cards would then represent the toe of the diversion containment levees above existing ground.

B4.3 Modeling North Dakota Tributary Flows. The tributary rivers, such as the Sheyenne, Maple, Lower Rush, and Rush Rivers, are not directly modeled in the North Dakota East Alignment model. Rather, a change in flow is made at these locations. Early in the modeling process, it was necessary to determine through hydrology the flow input at each tributary river (see Appendix A – Hydrology). However, a significant amount of work has taken place to refine this method of accounting for the tributary rivers. As a conceptual design for the river crossings at the diversion was developed, these crossings were inserted into the diversion models as bridge structures. This allowed for direct determination of the head loss at each structure. It was also possible to refine the flow input at each structure. Initially, the flow input was based on Appendix A; however, with separate modeling associated with the design of the structures, an actual flow was determined. This separate modeling effort used rating curves developed for each tributary for both local and Red River coincidental flooding at the location of the hydraulic structure and the flows presented in Appendix A as target design flows. Based on this analysis, the actual performance of the hydraulic structures was determined and then these calculated flows were put back into the diversion models. Expanded discussion of the details of construction and operation of the various channel structures is found within Appendix F - Hydraulic Structures.

B4.4 Recalibration of the Red River Model. In Phase 2 of this study, (Phase 1 Hydrology) the Red River model was based on the preliminary FIS model. This model was calibrated for the 1997 flood. During Phase 2 Part 2 of this study, the Red River

model was modified and recalibrated for the 2009 event. A summary of study phases was provided above in Section B1.2. Since Phase 2 Part 2 of this study, the USACE Agency Technical Review (ATR) committee has reviewed the models. In response to their comments, additional extensive re-calibration was done during this phase. The changes to the Red River model are summarized in a Moore Engineering Memorandum attached as Attachment B1 to this report. Many of the changes focused on two areas. The first area was the cross section resistance (Manning's "n" values) and bank station locations. The second area was the location of the flow optimization points in the model, namely lateral weirs and junctions.

B4.4.1 Calibration of Red River Bank Stations and Flow Resistance. The previous version of the Red River model often had the channel subdivided into different "n" values. When looking at these values in aggregate, the values tended to be larger than expected. To deal with this issue, the bank locations were adjusted with the aid of aerial views of the river corridor. The channel "n" values were then aggregated into a single value between the banks. By utilizing the discharge-elevation curve at the Fargo gage, the model channel "n" values were calibrated for low flows. Having calibrated the channel resistance "n", it was then necessary to also calibrate it for large flows, which involves extensive flow in the overbank areas. With the aid of 50 observed high water mark elevations from the 2009 flood throughout the Fargo-Moorhead area, the overbank "n" values were calibrated to best fit the observed data. Figure B6 is a Discharge - Elevation Curve for the recalibrated model at the Fargo gage. Figure B7 provides a modeled profile of the Red River versus the observed water surface elevations. Upon completion of this calibration, the values for channel resistance tended to vary from 0.038 to 0.045 with overbank values of 0.10 to 0.16, which are within the range of expected values.

B4.4.2 Model Optimization Locations- Weirs and Junctions. The Red River model has several weirs and junctions, which allow the model to split flows based on energy and momentum equations between separate channels. On the south side of Fargo, these channels are the Red and Wild Rice Rivers. On the north side of Fargo, the channels are the Red River and the Oakport Coulee. On the south side of Fargo, the weirs were turned off. This approach was followed because the Wild Rice River reach is not included in the model and the weirs did not accurately reflect breakout flows associated with modeling a single river system. It was still possible to model the effect of exchange of flows in terms of flow attenuation. Significant attenuation due to storage occurs for large flows along the Red and Wild Rice Rivers. The hydrologic analysis conducted by the USACE provided the necessary increase or reduction of flow along the Red River from Hickson to the confluence of the Wild Rice River. On the north side of Fargo, several issues with balancing flows at junctions and weirs led to the inability for the model to converge in the Oakport region. One issue had to do with a connecting reach that allowed for flow from Oakport Coulee to the Red River near the Highland Park subdivision. It was noticed that, given the difference in water surface elevations between the Red River and Oakport Coulee, this channel could flow in either direction; however, the limits of the model did not properly allow for that interaction. With regards to the junctions, at very low flow conditions water should not flow into the Oakport Coulee, but without a separate geometry file this could not be modeled with a junction. It was decided to replace several

of the junctions in the Oakport area with weirs. This method is also more similar to that used in the unsteady flow modeling, which removes the Oakport Coulee and replaces it with weirs and storage cells. Several weirs connect the Red River with the Oakport Coulee. To ensure sufficient tailwater on the weirs, the location of the tailwater was moved upstream in some cases. A discussion was had with regard to possibly simplifying this reach by removing the Oakport reach altogether and replacing it with longer Red River cross sections; however, this in itself would present problems with the flow balancing routines between cross sections. Barr Engineering conducted a sensitivity analysis of the weirs and their initial estimated flows within the model. It was determined that even if the model was still unable to converge in this region the changes in water surface elevations dependent on input values did not change significantly. Therefore these results had little impact upstream where the economic analysis was conducted. Further discussion of the sensitivity analysis can be found in the Moore Engineering Responses to the Barr Engineering Memorandum on the 2009 HEC-RAS Calibration Model, Fargo Moorhead Metro Study, included as Reference H. Additional discussion on use of an equivalent reach can be found in a Barr Engineering Memorandum, which is included as Reference I.

B4.4.3 Emergency Levees. Several emergency levees were in place during the 2009 flood. The locations of these levees were digitized and incorporated into the calibration model. During another large flood, these levees would likely be reconstructed, but since they were temporary and not certified, they were not used in the reference or project models.

B4.5 Team Development of the Models. Given the time constraints of the study, it was necessary to conduct the modeling using a team of three engineering firms. Moore Engineering developed the reference models and completed modeling of several of the alternatives. Barr Engineering modeled the hydraulic structures. Additional modeling of the alternatives and project time frames during Phase 3 was conducted by Barr Engineering and Houston Engineering. To ensure consistent quality work, the models developed by one engineering firm were reviewed by one or both of the other engineering firms listed above. The reviews conducted were in-depth, with comments passed verbally and in writing. A set of the comments is provided as Attachment B2.

B5.0 MINNESOTA SHORT ALIGNMENT - CONCEPTUAL DESIGN

B5.1 Minnesota Long Alignment. The Minnesota Long Alignment had many of the same design features as the Minnesota Short Alignment such as cross section, depth, outlet and inlet design, channel grade, and more. Where they differed was in the alignment, the location of tie-back levees, and auxiliary channels. As the Minnesota Long Alignment is no longer under consideration, no further discussion on its conceptual design will be included.

B5.2 Channel Cross Section. During the initial phase it was determined that the most cost beneficial channel geometry (in terms of excavation volume) incorporated a deeper

channel as opposed to a wider channel having the same capacity. This led to consideration of a "V" notch channel. It was further considered that an even more economical section may be parabolic or a semicircle. The determination was made that channel designs of this fashion were not practical and a trapezoidal channel with a minimum channel bottom width of 50 to 100 feet was established. This bottom width was further modified by other design constraints. The next parameter in the channel cross section was the channel side slope. Based on guidance received at the Project Kickoff meeting, the side slopes were to be 7:1 horizontal to vertical. The 7:1 slope was selected to ensure stability of the channel in lieu of conducting soils testing at that phase of the study. At bridges, the 7:1 slope was reduced to a 5:1 slope to reduce bridge span lengths. This side slopes have been modified in the current phase as a result of geotechnical considerations and the cross sections now include benches and flatter slopes in some areas. Table B1 provides a list of the various cross sections used in the channel and their locations.

B5.3 Geotechnical Considerations. One of the early concerns with the channel alignment and channel cross section was the avoidance of the Buffalo Aquifer. Even if the channel did not impact the aquifer, it was still possible for ground water to impact the stability of the channel. Early geotechnical testing concluded that ground water and soil stability would preclude maximizing the depth of the channel. Discussions with the US Army Corps of Engineers concluded that a channel excavation deeper than 35 feet on the Minnesota alignments may be problematic in terms of underground aquifers. To ensure additional safety on this issue, a limit of approximately 30 feet was suggested and this limit was incorporated into the design. Since then, additional geotechnical work has placed additional constraints on the channel. These include a change made in the alignment during the current phase and a change in cross section as discussed above. Another recent change was to raise the bottom an additional four feet in the area south of approximately Station 566+58. Due to concern with the effects of uplift, on the channel bottom, as a result of potential hydrostatic water pressure; it was determined that the channel should be raised at this point and to the south for several miles. However, given the nearly flat channel slope, it was not possible to simply raise a portion of the channel. Therefore the entire channel south of approximately Station 566+58 was raised the four feet. At Station 566+58, a sheetpile and riprap drop structure is provided to ensure channel stability.

B5.4 Channel Depth versus Vegetative Requirements. Maintaining vegetation on the diversion channel was identified as an initial project feature. In order to accomplish this goal, a criteria was set that the minimum elevation at the outlet should not fall below the 75% flow duration for the month of May on the Red River. Figure B8 provides guidance for setting the channel minimum elevation at the outlet of the Minnesota Short Alignment at the Red River.

B5.5 Channel Slope. Channel slope is governed by two factors. The first is to pursue a sufficient slope to allow water in the channel to drain in sufficient time to preserve vegetation on the channel bottom. The second factor is to ensure adequate depth of the channel at the upstream end to provide the required channel capacity. Based on these

competing requirements, a channel slope of 0.01% was used for the downstream half of the channel and 0.02% for the upstream half of the channel.

B5.6 Auxiliary Channels, Tie-back Levees, and Channel Breakouts. The Minnesota Short Alignment has the advantage of both the Red and Wild Rice Rivers having already converged upstream of the diversion to provide sufficient flow to the diversion. However, this also means that breakout flows have also occurred prior to the diversion. These breakouts flow into the Rose Coulee Watershed and would impact Fargo. An initial concept was to add a tie-back levee extending from the Red River Control Structure west to high ground near Horace, ND. With this levee, it was also necessary to construct a side channel to return the breakout flows to the rivers. After several hydraulic computations, it was determined that after the breakout flows had reached the side channel, insufficient hydraulic grade remained to return the flow to the rivers and that the side channel then actually presented a potential path for breakout from the river to areas to the west. As a result, it was necessary to look at moving the tie-back levee upstream. The east - west leg of the levee was finally moved to a location just north of Cass County Highway 16, with a north - south leg adjacent to the Wild Rice River, which tied the east - west leg back to the Red River Control Structure and the Red River Diversion. Refer to Figure B3 for more details on the exact alignment of the final tie-back levee. The tie-back levee prevents breakouts from the Wild Rice River north of Cass County Highway 14 from entering the Rose Coulee Watershed. As a result, more water remains in the river. To offset the potential for stage increase an auxiliary channel is added to the Red River Diversion that extends from a point north of the Wolverton Creek confluence with the Red River to the diversion northward on the east side of the Red River. This channel then pulls off over 14,000 cfs on the 500-year, project year zero, Red River event. In the model, this auxiliary channel is referred to as the Extension Channel. A second smaller channel is constructed to allow water that breaks across Cass County Highway 16 on the west side of Interstate 29 to return to the Wild Rice River at the St Benedict Coulee (1 mile north or Cass Hwy 16). This channel is referred to in the hydraulic model as the Wild Rice River Breakout Channel since it deals with the Cass County Highway 16 breakout. The combination of the two auxiliary channels then effectively eliminates the potential stage increases that would be a result of the prevention of breakouts due to project tie-back levees. In the hydraulic model, the Extension Channel parallel to the Red River is included as a reach to allow direct modeling of its impact.

B5.7 Minnesota Short Diversion Weirs. Initially it was decided that the Red River Diversion should not operate prior to the Red River exceeding the 5-year flood event. During Phase 2 Hydrology, it was determined that the 5-year event equated to approximately 9,600 cfs on the Red River downstream of the Wild Rice River confluence. As part of Phase 3 Hydrology, the flow objective was kept at 9,600 cfs rather than strictly a 5-year event. With Phase 3.1 Hydrology, 9,600 cfs equates to approximately a 3.6-year event. Two weirs are used in conjunction with the Red River Diversion with the Minnesota Short alignment. The weirs control when the Red River Diversion begins to flow and how much flow is directed to the channel. The weir on the Extension Channel is set at approximately the 3.6 year event plus one and a half feet. The weir on the Diversion Channel is set at approximately the 3.6 year event plus one half foot. Both of these weirs

ensure the Red River Diversion begins operation soon after a 3.6 year event. The use of one half foot and one and a half feet above the 3.6 year level for the two weirs is driven by hydraulic balancing of the diversion with the Red River Control Structure. It is not intended to establish a starting operation point of the diversion other than above a 3.6 year event. The minimum flow in the Red River prior to operation of the Red River Diversion is based on environmental concerns

B5.8 Red River and Diversion Control Structures. The Red River Control Structure is a combination earthen berm and concrete structure with three gated openings. Initial concepts of this structure included several culvert openings. To increase the efficiency of the structure, to address velocity concerns, and to deal with ice issues, those openings have been consolidated into three larger openings. The gates for the three openings will not be allowed to close off flow within two feet of the gate invert elevation. Appendix F discusses the purpose of this design. The operation of the gates will allow normal flows during non-flood conditions and floods less than 9,600 cfs to pass unimpeded at this structure. The upper portion of the openings are gated to control flow downstream. The gates, along with the Red River Diversion Inlet Weir, also control head conditions upstream of the structure. The control structure is modeled in HEC-RAS as an inline structure with gates. The inlet weir to the Red River Diversion is envisioned as a sheetpile and rock weir. It is also modeled using the inline structure routine as a compound weir embankment. The compound weir allows better response for intermediate flows. For more information on this or other hydraulic structures, see Appendix F - Hydraulic Structures.

B5.9 Maintaining the Upstream Water Surface Profile. Phase 3 of the study involved changing hydrology during different time periods for the project (see Appendix A – Hydrology). As the flows for the different time periods, project years 0, 25, and 50, would vary, the design of the inlet weir may also need to vary. Since the weir is designed for the project year 0 condition and no changes to its design are made in the future, then it may pass insufficient flows on the later years due to insufficient head on the weir (because the flows decreased). Because of this concern, it was initially decided that the weir should be designed for Year 0 and that to ensure sufficient head on subsequent hydrologic time periods, the Year 0 upstream water surface would be allowed for all project time frames. This was later modified into 3 priorities; 1) In all cases, 9,600 cfs should be allowed to pass downstream of the Red River closure structure toward Fargo; 2) At the Year 25, it would first be attempted to operate the system such that the water surface is not raised above the existing condition for Year 25; 3) If necessary the water surface upstream of the weir would be allowed to rise above the existing Year 25 water surface, but not above the existing Year 0 water surface.

B6.0 NORTH DAKOTA EAST ALIGNMENT - CONCEPTUAL DESIGN

B6.1 North Dakota West Alignment. The North Dakota West Alignment has many of the same design features as the North Dakota East Alignment such as cross section, depth, outlet and inlet design, channel grade, and more. Where they differ is in the

alignment between Horace and West Fargo. As the North Dakota West Alignment is not currently the Local Preferred Plan or the Federal Comparable Plan, no further discussion on its conceptual design will be included.

B6.2 Channel Cross Section. During the initial phase it was determined that the most cost beneficial channel geometry (in terms of excavation volume) incorporated a deeper channel as opposed to a wider channel having the same capacity. This led to consideration of a "V" notch channel. It was further considered that an even more economical section may be parabolic or a semicircle. The determination was made that channel designs of this fashion were not practical and a trapezoidal channel with a minimum channel bottom width of 50 to 100 feet was established. This bottom width was further modified by other design constraints. The next parameter in the channel cross section was the channel side slope. Based on guidance received at the Project Kickoff meeting, the side slopes were to be 7:1 horizontal to vertical. The 7:1 slope was selected to ensure stability of the channel in lieu of conducting soils testing at that phase of the study. At bridges, the 7:1 slope was reduced to a 5:1 slope to reduce bridge span lengths. This side slopes have been modified in the current phase as a result of geotechnical considerations and the cross sections now include benches and flatter slopes in some areas. Table B2 provides a list of the various cross sections used in the channel and their locations.

B6.3 Channel Depth versus Vegetative Requirements. As with the Minnesota Short Alignment, maintaining vegetation on the diversion channel was identified as an initial project feature. In order to accomplish this goal, a criteria was set that the minimum elevation at the outlet should not fall below the 75% flow duration for the month of May on the Red River. Figure B9 provides guidance for setting the minimum channel elevation at the outlet of the North Dakota East Alignment at the Red River.

B6.4 Channel Slope. The North Dakota East Diversion Alignment channel bottom slope is 0.02% and has remained unchanged with each phase.

B6.5 Red River to Wild Rice River Connecting Reach. With the North Dakota East Alignment, it is necessary to start the diversion channel at the Red River to pull water from both the Red and Wild Rice Rivers. However, the design of the connecting reach between these two rivers is different from the rest of the diversion. This unique design is to ensure proper balance of flow is maintained between the two rivers. Among the design features, the channel is graded at a 0.02% slope but sloped from the Wild Rice River to the Red River. This allows the channel to drain to the Red River. It also provides that the channel will fill from back water from the Red River prior to actually flowing. The invert of the channel at the Red River is set at the Phase 2 Hydrology 2-year water surface elevation on the Red River. Section B6.9 North Dakota East Diversion Weirs, provides a detailed discussion of the weirs that govern the operation of this connecting reach.

B6.6 Tie-Back Levees. Tie-back levees extend from the Red River closure structure east until hitting high ground in Minnesota. No other tie-back levees were added to this alignment for the area south of Fargo. The initial tie-back levee extended east across

Wolverton Creek and approximately two to three miles into Minnesota to find high ground. See Figure B1 for the exact line of the levee. With Phase 3.1 Hydrology it was necessary to update the tie-back levee with the increased height of the Red River. Three alignments have been proposed. See Section B3.4 for additional details on the alignments and which is preferred.

B6.7 Wolverton Creek Crossing. As the preferred alignment for the North Dakota East Alignment tie-back crosses Wolverton Creek, it was necessary to start a discussion of how to size the crossing structure. This structure was provided in the initial reports, however, with the preliminary FIS hydrology (See Appendix A - Hydrology) and Phase 1 Hydrology, this crossing represented a flow of less than 2,500 cfs at the 500-year Red River event. With the current hydrology, the 500-year, project year zero flow could be as much as 9,200 cfs. Some of the ongoing discussion on this crossing is provided below.

B6.7.1 Unimpeded 500-year flow. The current hydraulic modeling assumes that this crossing allows the coincidental flow for a 500-year Red River event to proceed unimpeded. However, sizing a structure large enough to allow that amount of flow within the provided hydraulic head constraints may be as large as four (4) 11' wide by 22' high box culverts. It should be noted that the road crossing at this location has two (2) 10' x 10' box culverts today and that the road submerges under many flood events. The invert of the levee crossing box culverts would be set at approximately 895.0 to closely match the invert of the existing box culverts under the road. This is not a preferred solution.

B6.7.2 Gated Flow. If four (4) 11' wide by 22' high box culverts were used at this crossing, there would likely be a need to place gates on this crossing. This is to ensure that such a crossing could not pose a risk of too much flow through the line of protection. A large gated structure at this location is not a preferred solution.

B6.7.3 Fish Passage. With placing a crossing on Wolverton Creek the issue of fish passage is raised. Any crossing should be designed to minimize velocities and to allow for fish passage. Minimizing velocities could entail larger boxes or roughening of the lower box to provide areas of lower velocity. It should be noted that with the existing road crossing and the preliminary FIS hydrology, a 10-year local flood event would see a velocity over 8 fps within the existing road crossing. Velocities as high as 12 fps are evident at this crossing on a 100 or 500-year local flood event. The 10-year preliminary FIS event flows as an open channel, or non-pressurized, event through the existing crossing. The 50-year preliminary FIS event on Wolverton Creek overtops the existing road at this location.

B6.7.4 Auxiliary Channel. The preferred solution to placing large box culverts at this crossing would be to size culverts to match the road culverts or to move this portion of the levee onto the road with a road raise and use the existing culverts. With these options, an auxiliary channel would be constructed to divert larger flows to the Red River, to ensure stage impacts are not seen upstream of the crossing. This approach would reduce the risk of large openings on a tie-back levee while preventing stage increases. Under this scenario, provisions could be made to even enhance fish passage on

Wolverton Creek by starting to divert water on smaller floods; however, a minor adjustment in the operation of the Red River Control Structure may be necessary to deal with additional water upstream.

B6.8 North Dakota Tributary Hydraulic Structures. Hydraulic structures were necessary at the North Dakota tributaries. These hydraulic structures allow lower flows to continue downstream in the rivers while diverting a majority of all river flows into the Red River Diversion during larger flood flows.

B6.8.1 Minimum Downstream Flows. An objective established by the USACE stipulated that a minimum flow at each river be allowed to pass the Red River Diversion and enter the downstream reach of the River. This flow was set as the 2-year local flood flow at the Sheyenne River and the Maple River. Under project conditions, the Lower Rush River and Rush Rivers are diverted completely to the Red River Diversion, so the 2-year flow amount for these two rivers was added to the amount set on the Maple River. In other words, the minimum amount to pass through to the downstream reach of the Maple River would be the sum of the 2-year local flood flows of the Maple, Lower Rush, and Rush Rivers. A minimum flow objective was also set for the Wild Rice River. The Red River minimum flow was set as 9,600 cfs during Phase 2 Hydrology, which was a 5-year event flow downstream of the Wild Rice River. With Phase 3 and 3.1 Hydrology, the 9,600 cfs flow amount was maintained which is equivalent to approximately a 3.6 year event on the Red River. The Wild Rice River minimum flow was set as the coincidental flow associated with 9,600 cfs on the Red River. This flow amount for the Wild Rice River is approximately 2,350 cfs.

B6.8.2 Sheyenne River Hydraulic Structure. The Red River Diversion flows under the Sheyenne River at this crossing. The crossing exists as a series of bays similar to a row of precast box culverts through which the diversion flows. On top of the boxes is an open concrete channel through which the Sheyenne River flows perpendicular to the Diversion Channel. The sizing of the top channel dictates the amount of flow that passes the diversion and reaches the downstream reach of the Sheyenne River. Flows above this amount are directed into the Red River Diversion. For more details on this structure, see Appendix F - Hydraulic Structures.

B6.8.3 Maple River Hydraulic Structure. The Red River Diversion flows under and over the Maple River at this crossing. The crossing exists as a series of bays similar to a row of precast box culverts through which the diversion flows. On top of the boxes is an open concrete channel through which the Maple River flows perpendicular to the Diversion Channel. When the diversion flows exceed what will pass underneath the river open channel, additional flows will flow over the top and continue downstream in the diversion. The sizing of the concrete open channel dictates the amount of flow that passes the diversion and reaches the downstream reach of the Maple River. As the diversion can flow over the top of this structure, it was also necessary to gate the downstream end of the open river channel to control flows into the Maple River downstream of the crossing. For more details on this structure, see Appendix F - Hydraulic Structures.

B6.8.4 Rush and Lower Rush Rivers. The Rush and Lower Rush Rivers have separate outlets to the Red River Diversion. However, there is not a crossing as 100% of both rivers are diverted into the diversion. To maintain adequate flows in the Sheyenne River at this location, additional water, was allowed to enter the Sheyenne River from the Maple River crossing. See Section B6.8.1 for additional discussion on downstream flows for the Maple River.

B6.8.5 Wild Rice Hydraulic Structure. The Wild Rice River hydraulic structure has changed in concept since the first phase of this study. With Phase 1 Hydrology, a structure was envisioned to allow small flows in the Wild Rice River to proceed downstream in the river. Under larger floods, the entire river would be diverted to the diversion channel but some water would be pulled from the Red River Diversion to enter the Wild Rice River to maintain flow in the downstream reach of the river. During Phase 2 Hydrology, this crossing was simplified. It now consists of two weirs on the diversion and a control structure on the Wild Rice River downstream of the diversion.

B6.9 North Dakota East Diversion Weirs. Two weirs are used on the Red River Diversion with the North Dakota East Alignment. The weirs control when the Red River Diversion begins to flow and how much flow is directed to the channel. The east weir is located on the diversion channel on the east side of the Wild Rice River, which is referred to as the "Red River to Wild Rice River Connection Reach." The west weir is located on the diversion channel on the west side of the Wild Rice River. During Phase 2 Hydrology, the east weir was set at the Red River 5-year flood elevation and the west weir was set at the Red River 5-year flood elevation plus one foot. This ensured that the diversion was not put into use prior to exceeding a Phase 2 Hydrology 5-year event on the Red River. The flow that was associated with the 5-year event was 9,600 cfs downstream of the confluence of the Red and Wild Rice Rivers. This flow level was established as a minimum flow to maintain in the rivers for environmental considerations. During Phase 3 Hydrology, rather than establishing minimum flows based on local flow events, the 9,600 cfs flow was maintained as the benchmark. This flow now equates to approximately a 3.6- year flow on the Red River downstream of the Wild Rice River confluence with the Phase 3.1 Hydrology. Based on this target flow, the east weir was reset to the elevation of a 3.6-year flood elevation. The west weir was set one foot higher than the east weir (3.6-year flood elevation plus one foot).

B6.10 Red River Control Structure. The Red River Control Structure is a combination earthen berm and concrete structure with three gated openings. Initial concepts of this structure included several culvert openings. To increase the efficiency of the structure, to address velocity concerns for fish passage and to deal with ice issues, those openings have been consolidated into three larger openings. The gates for the three openings will not be allowed to close off flow within two feet of the gate invert elevation. Appendix F discusses the purpose of this design. The operation of the gates will allow normal flows during non-flood conditions and floods less than 6,100 cfs to pass unimpeded at this structure. The 6,100 cfs is the flow at the control structure, which when added together with additional flow inputs downstream equates to 9,600 cfs through Fargo-Moorhead. The upper portion of the openings are gated to control flow downstream. The gates,

along with the North Dakota East Diversion Inlet weirs, also control head conditions upstream of the structure.

B7.0 ADDITIONAL CHANNEL FEATURES

B7.1 Channel Outlet. During Phase 1 of this study, several project personnel had an opportunity to visit the Winnipeg Floodway project. That project has a significant structural outlet to reduce outlet velocities by taking up grade. However, in the FM Metro area, there is significantly less overall drop in the Red River than in Winnipeg. Further, given the depth of the channels, water will back into the channel several feet deep prior to water entering upstream. As a result, it was decided that an exit structure is not necessary on the Red River Diversion beyond general riprap erosion protection.

B7.2 Low Flow Channel. Given the large width of the channel bottom of both alignments, there is concern with maintaining vegetation on the bottom. This is particularly important as the Red River Diversion will ultimately have drainage systems entering it and thus will have low flows in it frequently. To address this issue, a low flow channel is proposed. The low flow channel would convey most of the base flow level of drainage flows, which would keep the remaining portion of the Red River Diversion Channel bottom dry allowing vegetation to grow. Flows from larger rain events would likely overflow the low flow channel. Initial designs of the low flow channel consist of a trapezoidal channel with 4:1 horizontal to vertical side slopes, a 10 foot bottom width, and a depth of 3 feet. Detailed design of this feature will be conducted later in the design process. A conceptual design of the low flow channel has been incorporated into the hydraulic models. Although a low flow channel is incorporated into the diversion, actual non-flood flows may fill more than the low flow channel. This is particularly true downstream of the Lower Rush and Rush Rivers on the North Dakota East Diversion Alignment.

B8.0 DISCUSSION OF MODELING RESULTS

B8.1 Low Flow Cases. The Red River Diversion is designed primarily for the Red River 500-year, project year zero, event, but to achieve benefits on smaller events, significant design has gone into the process to ensure the system performs through the full range of flood events down to the 3.6 year Red River event. As mentioned above, the system is designed to first start operating when the discharge at Fargo is above 9,600 cfs. Thus, for the Red River 2-year flood events, the diversion system would not be in operation. However, design for these smaller events was conducted to ensure the Red River Control Structure does not adversely impact the Red River, even on these small events. Fish passage is maintained even on low flow events, as well as the use of the river by boats.

B8.2 Model Geometry. As discussed under Section B4 - Red River Model Construction, the model geometry was originally based on the 500-year Red River model developed

by the preliminary Flood Insurance Studies cited in References A and B and later modified as discussed above. The Red River Diversion reaches were developed using HEC-GeoRAS from the LIDAR developed by the International Water Institute and provided by the US Army Corps of Engineers. Additional cross sections were added to the Red River adjacent to the Red River Control Structure. These additional cross sections were generated using the LIDAR and merging existing channel bathometry from the existing model. The Red River Control Structure is modeled as an inline structure with gates. The North Dakota tributary crossings were modeled as bridges. The Sheyenne River and Maple River aqueducts, which is the portion of the hydraulic structure connecting the upstream and downstream reaches of the rivers above the diversion channel, are modeled as a bridge deck. The tributary rivers, namely the Wild Rice, Sheyenne, Maple, Lower Rush and Rush Rivers, were not modeled directly but are implied as flow changes. Separate modeling of the hydraulic crossings was conducted to refine the flow inputs. Two dimensional modeling has also been conducted on selected features, however, that modeling effort is not part of this appendix. Figure B10 provides a layout of the stationing on the Minnesota Short Alignment used in the model. Figure B11 provides a layout of the stationing on the North Dakota East Alignment used in the model.

B8.3 Hydrology Inputs. Appendix A - Hydrology provides the hydrology used for this study. This hydrology was further refined as input into the models. These inputs are summarized in Tables B3 through B11 for the reference condition and two alternatives with three project time periods each. Table B3 provides the extreme events of the 1,000-year and 10,000-year Red River events. The tables are placed in order of:

1. Reference Condition Year 0
2. Reference Condition Year 25
3. Reference Condition Year 50
4. Minnesota Short Alignment Year 0
5. Minnesota Short Alignment Year 25
6. Minnesota Short Alignment Year 50
7. ND East Alignment Year 0
8. ND East Alignment Year 25
9. ND East Alignment Year 50

The top section of the table provides flow values representing the local contributions at the north end of the study area. The middle portion of the table incorporates initial flows within the area of Oakport, MN based on upstream flows on the Red River and local flows. The bottom portion of the table contains local contributions for the area south of Fargo. The bottom portion of the table also provides values inputted into the HEC-RAS flow files where a direct input is used for the Red River Diversion. The amount diverted into the diversion channel then dictates the corresponding flow through the Fargo-Moorhead Metro area. Descriptions are provided on both sides of the table indicating the purpose of each flow change. These tables are initial flows throughout the project area. The actual flows are calculated by the hydraulic model.

B8.4 Plan Layout. Table B12 provides a listing of the HEC-RAS project and plan files that model the reference conditions and the two diversion alternatives. Typically, four plan files are provided for each project directory. These plan files take the project from very small floods to greater than the 500-year flood. Two of the files represent the existing conditions. The remaining two files represent the "with project" condition. One file represents the 2-year and smaller flood, while the other file represents larger floods.

B8.5 Red River Control Structure and Diversion Inlet Weir. During the conceptual design of the diversions, it was necessary to optimize the operation of the Red River Control Structure to maintain upstream head conditions. However, final operation of the gates would likely be provided in a Operation and Maintenance Plan. For additional data on the gates, see Appendix F - Hydraulic Structures.

B8.6 Red River Diversion Typical Sections and Profiles. Figures providing typical cross sections of the Red River Diversion and its extension channels are provided. Also provided are profile plots of the Red River Diversion. The typical cross sections are for project year zero unless otherwise specified. The profile plots are for project years 0, 25, and 50. Figure B12 provides a typical cross section of the Wild Rice River Breakout Channel used with Minnesota Short Diversion for the breakout flow from Cass County Highway 16 to St. Benedict. The water levels on this typical cross section are not from the current hydrology. Figure B13 through B16 provide a typical cross section and profile plots for the Minnesota Short Diversion Extension Channel. Figure B17 provides a sample cross section of the Minnesota Short Diversion (see Table B1 for additional cross sections). Figures B18 through B20 provide profile plots for the Minnesota Short Diversion. Figure B21 provides a typical section for the North Dakota East Diversion Connection Channel. Figure B22 provides a sample cross section for the North Dakota East Diversion (see Table B2 for additional cross sections). Figures B23 to B25 provide profile plots for the North Dakota East Diversion.

B8.7 Red River Profiles, Tables and Graphs. Tables B13 through B15 provide a summary of results from the HEC-RAS files for the existing condition for project year 0, project year 25, and project year 50, respectively. Table B16 provides a similar table for the two existing extreme events considered; the 1,000 year and the 10,000 year Red River flood events. Tables B17 through B22 provide the summary of results from the HEC-RAS files for the two alternatives in the order provided above in Section B8.3. Similarly, Figures B26 through B31 provide a summary of the same results in the form of water surface profiles.

B8.8 Stage Frequency Curve at the Fargo USGS Gage. Figures B32 through B37 provide a frequency distribution of elevation and stage for the range of flood events studied at the Fargo USGS Gage. The figures provide both existing and with-project conditions to allow for comparison. As expected, the project significantly lowers the elevation, or stage, at each flood event above a 9,600 cfs flood event on the Red River.

B8.9 Discharge Frequency Curve at the Fargo USGS Gage. Figures B38 through B40 provide discharge frequency curves for the Fargo USGS Gage for the existing and with project conditions for the project Year 0, 25, and 50.

B8.10 Minnesota Diversion versus North Dakota Diversion River Profiles. The Minnesota Short Diversion and the North Dakota East Diversion have different starting and ending locations where they intercept the Red River. Not only does this affect the total square miles that are protected but it also has an impact in tailwater through Fargo. The Minnesota Short and North Dakota East Alignments were compared for the 35 Kcfs alternatives. Figures B41 through B46 provide a comparison of the water surface profiles on the Red River through the FM Metro area for the 100 and 500-year events for the project years of 0, 25, and 50. Although the two alternatives have the same design capacity, the points of intercept on the Red River have an impact on the water surface profile.

B9.0 REFERENCES

A - Flood Insurance Restudy for Southern Cass County, North Dakota, and Clay County, MN, Dated: ongoing

B - City of Fargo CTP Project, Clay County / Oakport Township, Dated: ongoing

C - Fargo - Moorhead Metro Flood Risk Management Project, Feasibility Study Report for the US Army Corps of Engineers, and the Cities of Fargo, ND & Moorhead, MN, Moore Engineering, Etal., August 31, 2009.

D - Fargo - Moorhead Metro Flood Risk Management Project, Feasibility Study Report for the US Army Corps of Engineers, and the Cities of Fargo, ND & Moorhead, MN, Moore Engineering, Etal., Appendix H- Red River Hydraulic Structure Velocities, October 15, 2009.

E - Fargo - Moorhead Metro Flood Risk Management Project, Feasibility Study Report, Phase 2 - Part 2, for the US Army Corps of Engineers, and the Cities of Fargo, ND & Moorhead, MN, Moore Engineering, Etal., December 31, 2009, with a Revised Addition on January 6, 2010.

F - Northwest Diversion, Fargo - Moorhead Metro Flood Risk Management Project, Feasibility Study Report, Phase 2 - Part 2, for the US Army Corps of Engineers, and the Cities of Fargo, ND & Moorhead, MN, Moore Engineering, January 27, 2010.

G - Fargo - Moorhead Metro Flood Risk Management Project, Feasibility Study Report, Phase 3, for the US Army Corps of Engineers, and the Cities of Fargo, ND & Moorhead, MN, Moore Engineering, Etal. May 17, 2010.

H - Memorandum, Moore Engineering Responses to April 12,2010 Comments by Barr Engineering, 2009 HEC-RAS Calibration Model, Fargo Moorhead Metro Study, Economics Profiles for the NED Plan Determination with the Phase 3 Hydrology, April 23, 2010.

I - Memorandum, Barr Engineering, May 5, 2010

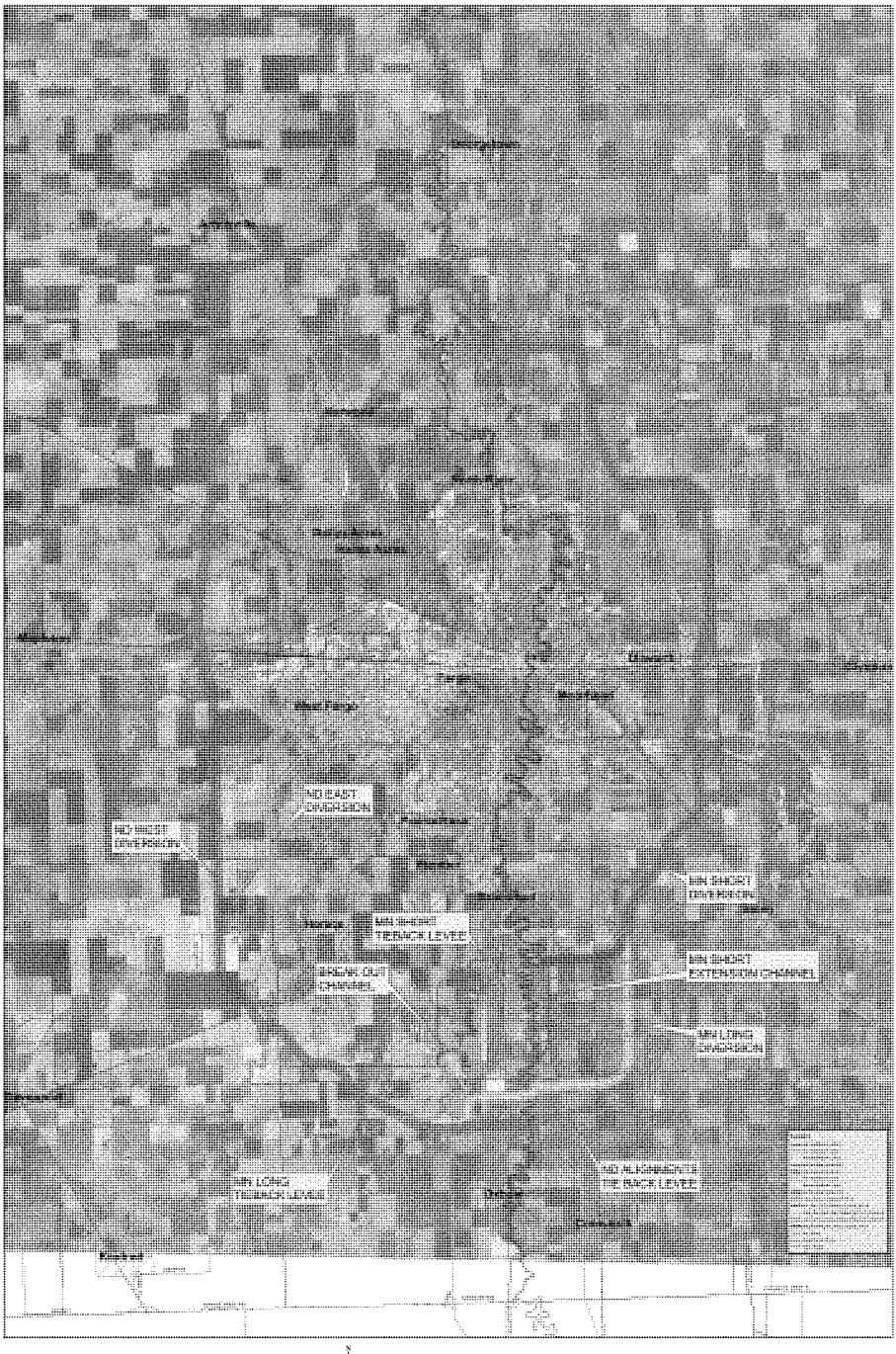

Figure B1 - Original Red River Diversion Alignments

NORTHWEST DIVERSION
500 YEAR DESIGN EVENT

Figure 1

Drawn By: GZ
Date: 01-22-15
T:\Murrenad Projects\449\NW_ALIGNMENT_DESIGN_EVENT_FLOWS_rl_plot.mxd

moore engineering, inc.
Consulting Engineering – Land Surveying

Figure B2 - Northwest Diversion Channel Alignment

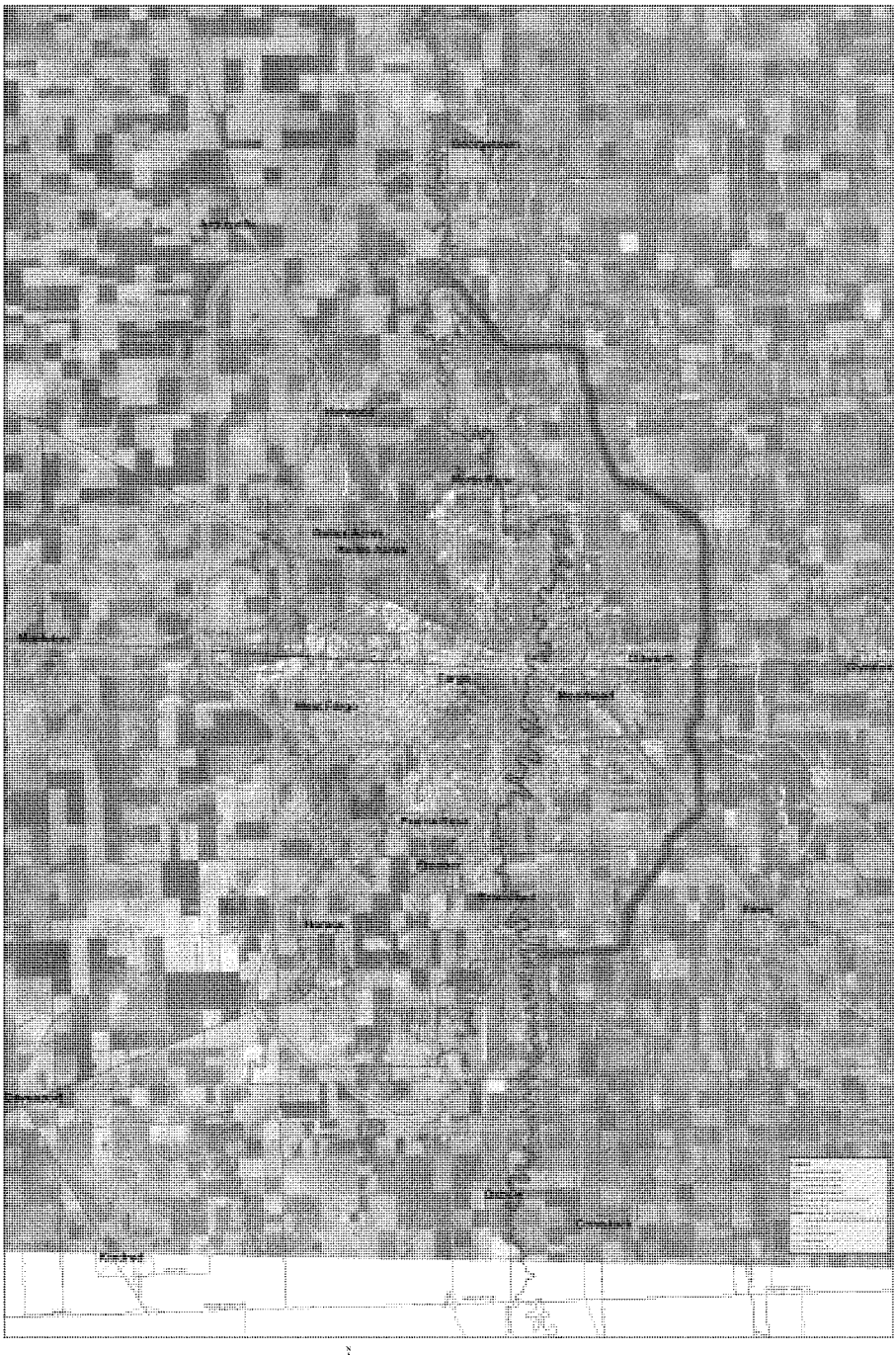

Figure B3 - Current Red River Diversion Alignments

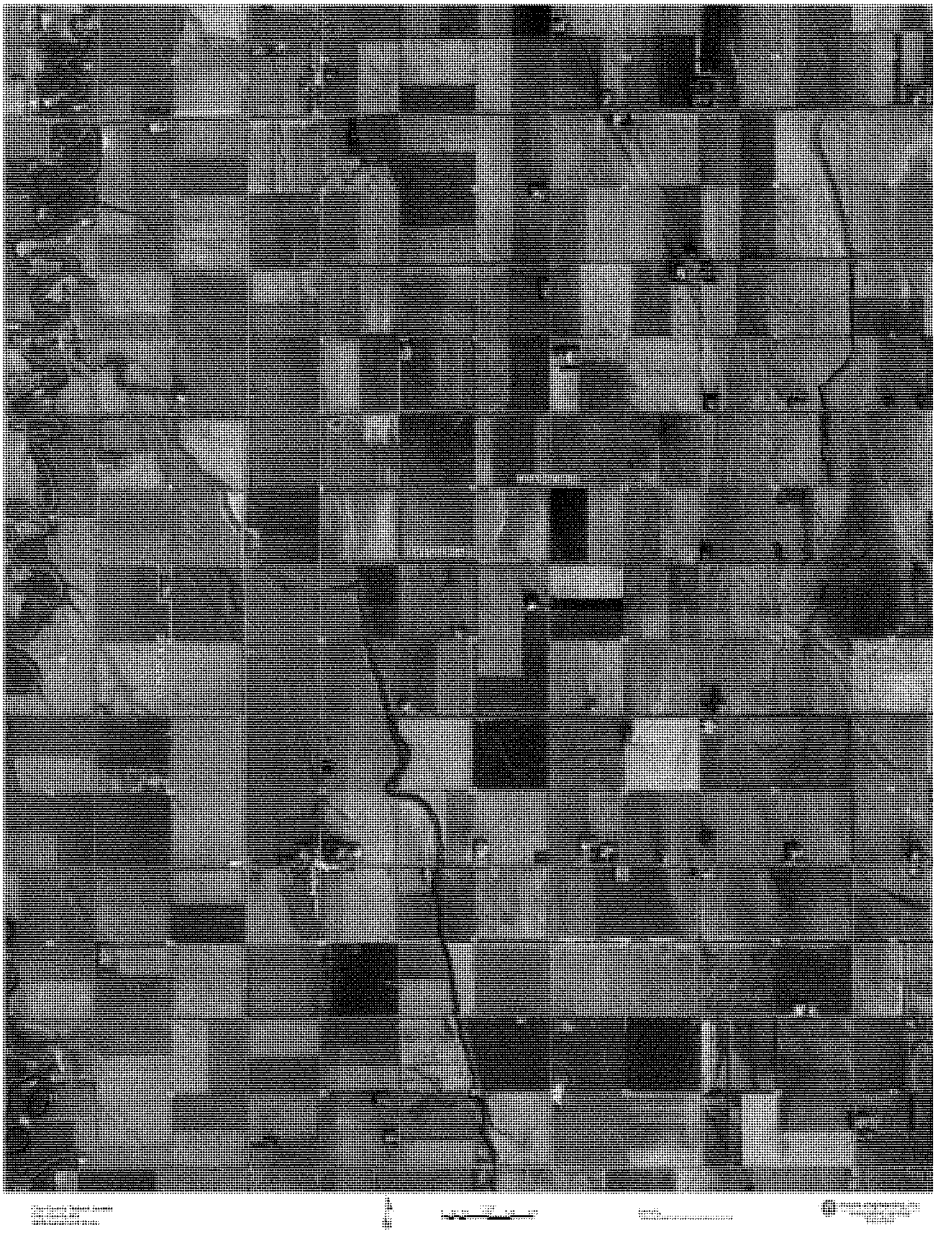

Figure B4 - ND East Diversion Alignment, Tie-Back Levee Options

Figure 8 - Wildwood Creek Crossing and
Detention Channel Layout

Figure B10 - HEC-RAS Georeferenced Cross-sections for the MN Short Alignment.

Figure B11 - HEC-RAS Georeferenced Cross-sections for the ND East Alignment

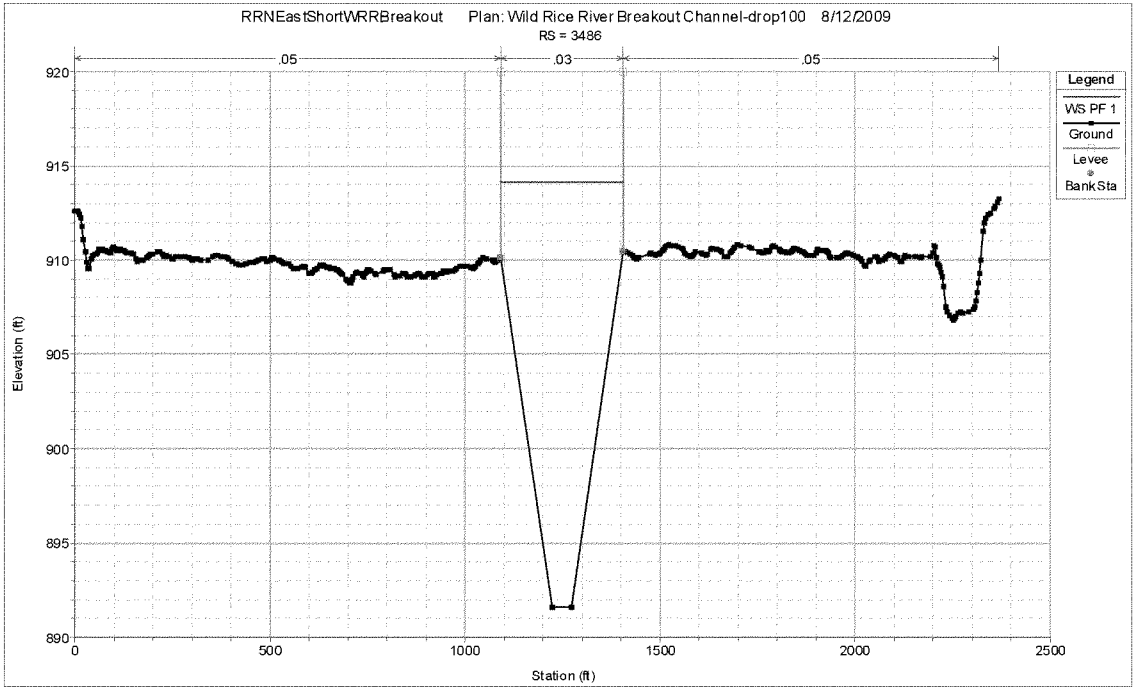

Figure B12 - MN Short Diversion Wild Rice River Breakout Channel - HEC-RAS Typical Cross Section

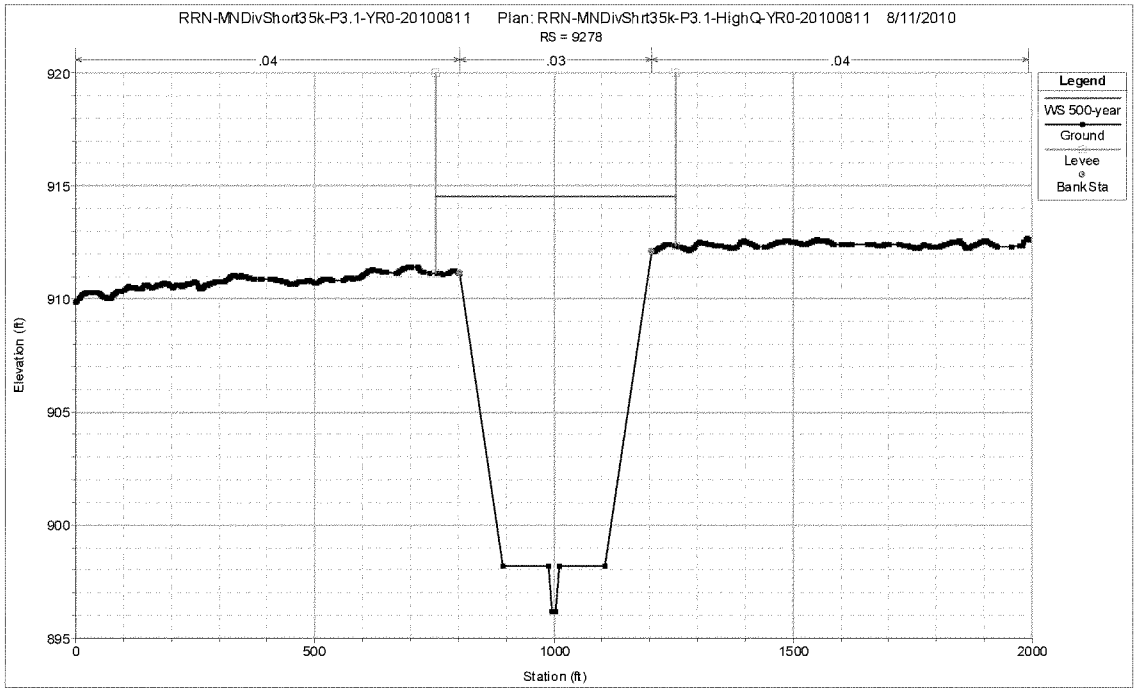

Figure B13 - MN Short Diversion Extension Channel - HEC-RAS Typical Model Cross Section

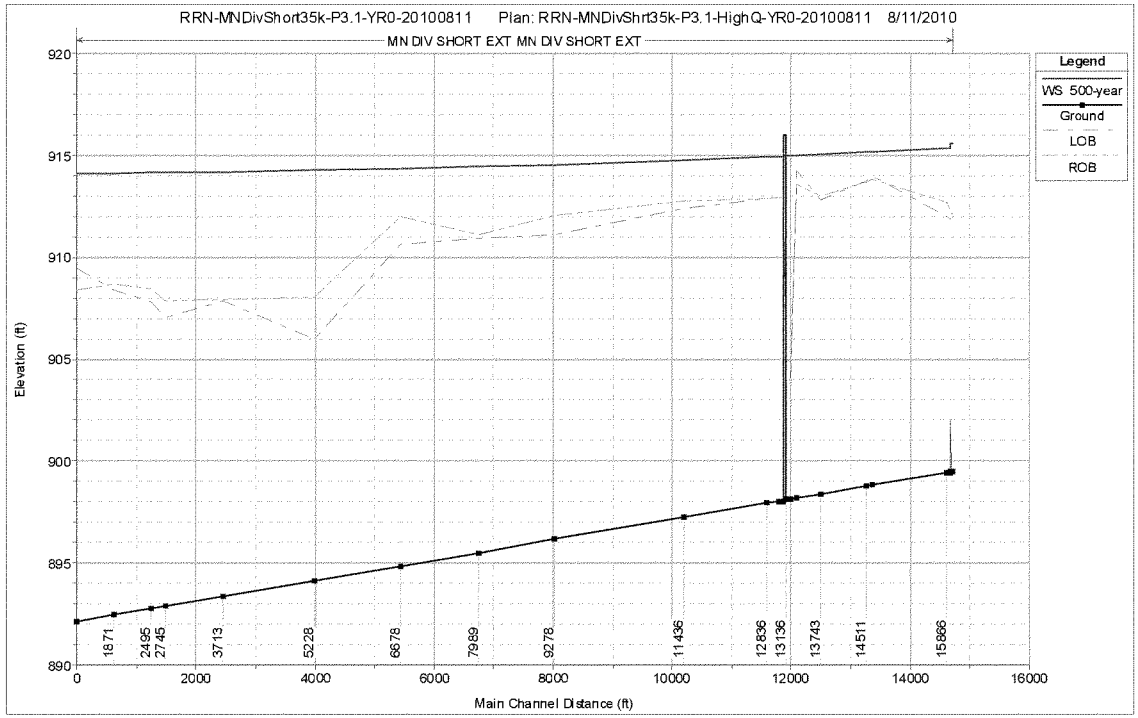

Figure B14 - MN Short Diversion Extension Channel - HEC-RAS Profile, Year 0

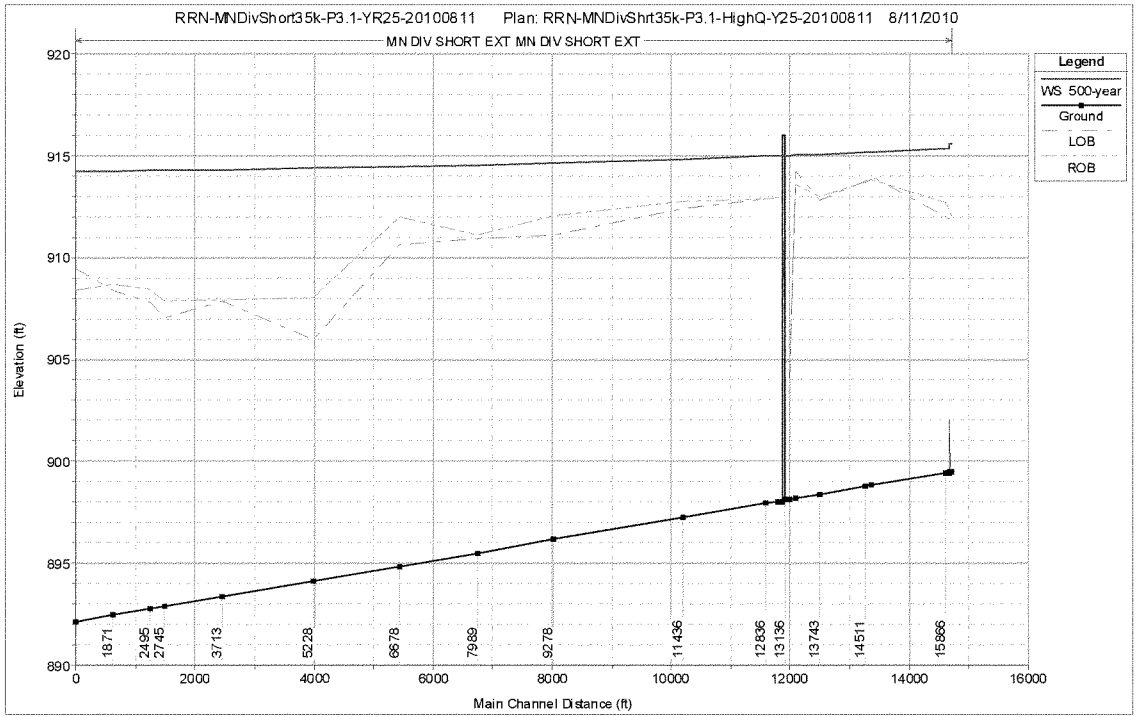

Figure B15 - MN Short Diversion Extension Channel - HEC-RAS Profile, Year 25

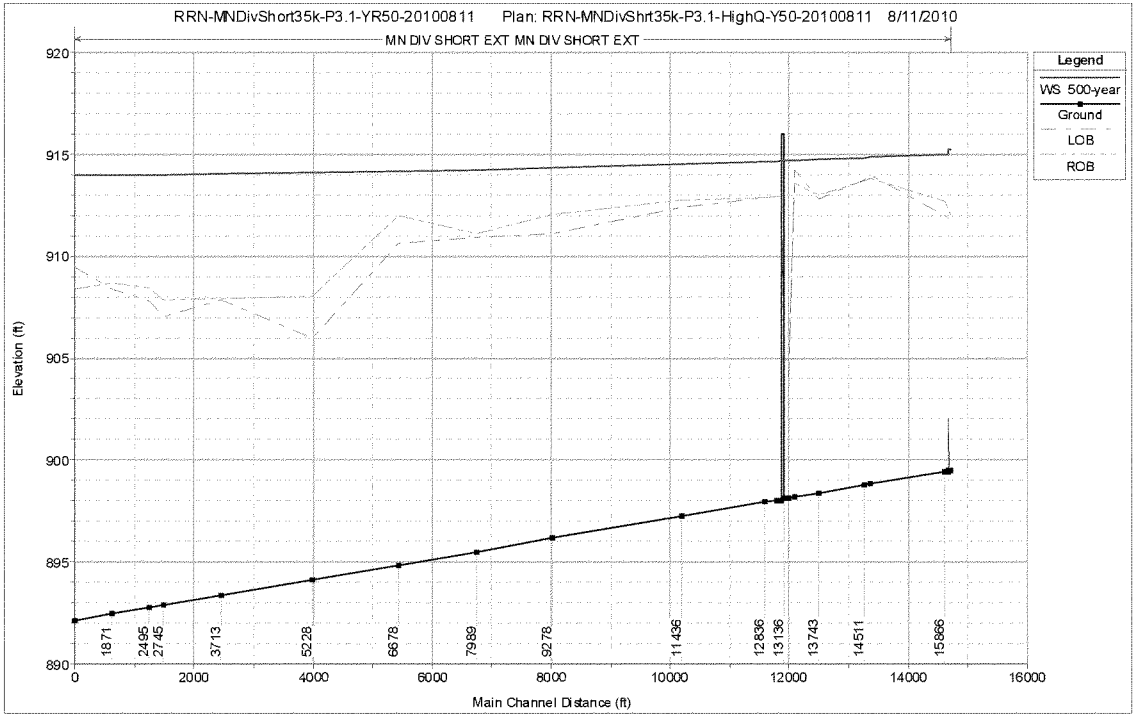

Figure B16 - MN Short Diversion Extension Channel - HEC-RAS Profile, Year 50

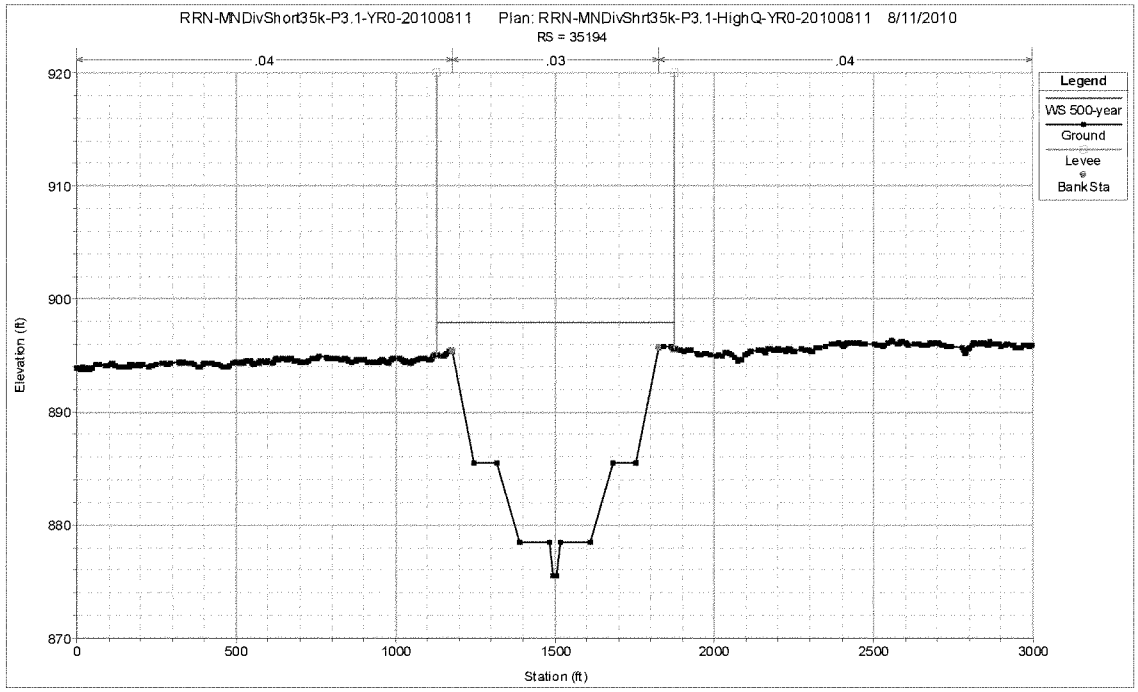

Figure B17 - MN Short Diversion 35 Kcfs Capacity - HEC-RAS Sample Model Cross Section

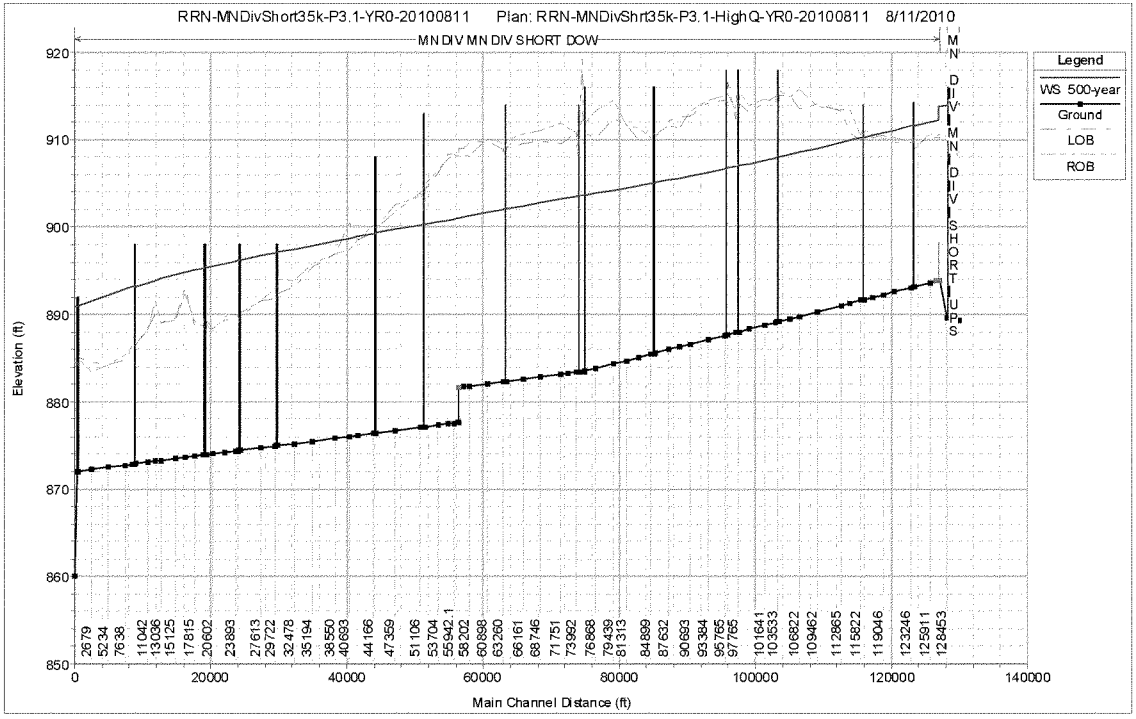

Figure B18 - MN Short Diversion 35 Kcfs Capacity - HEC-RAS Profile, Year 0

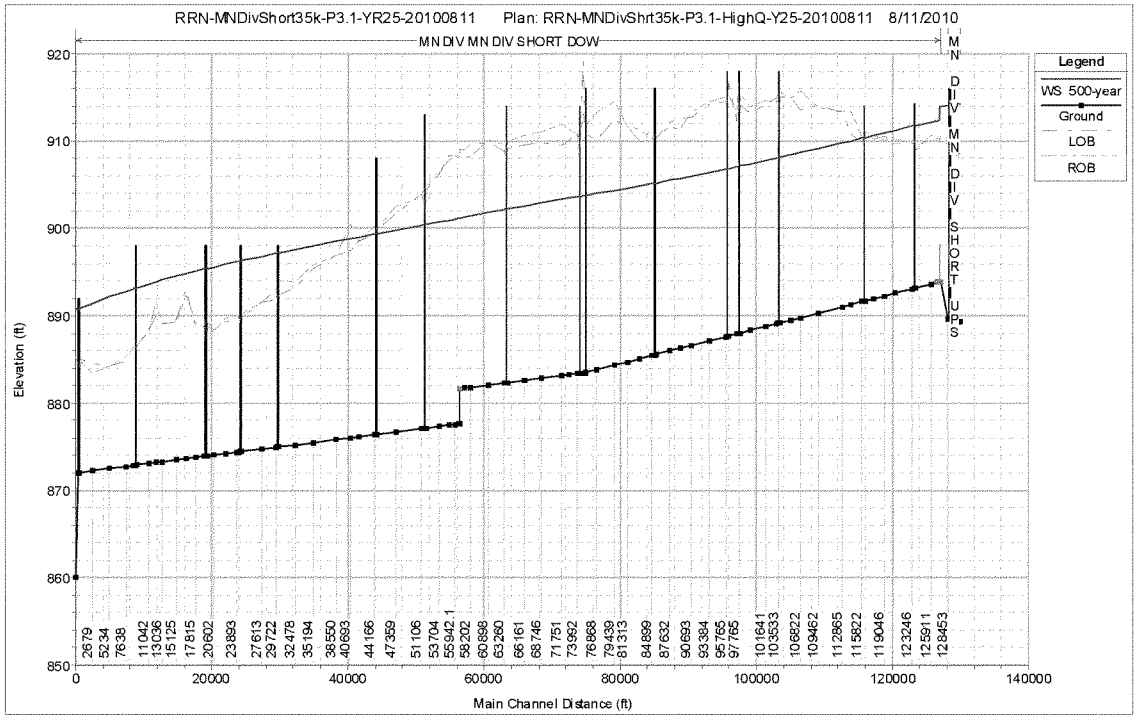

Figure B19 - MN Short Diversion 35 Kcfs Capacity - HEC-RAS Profile, Year 25

50

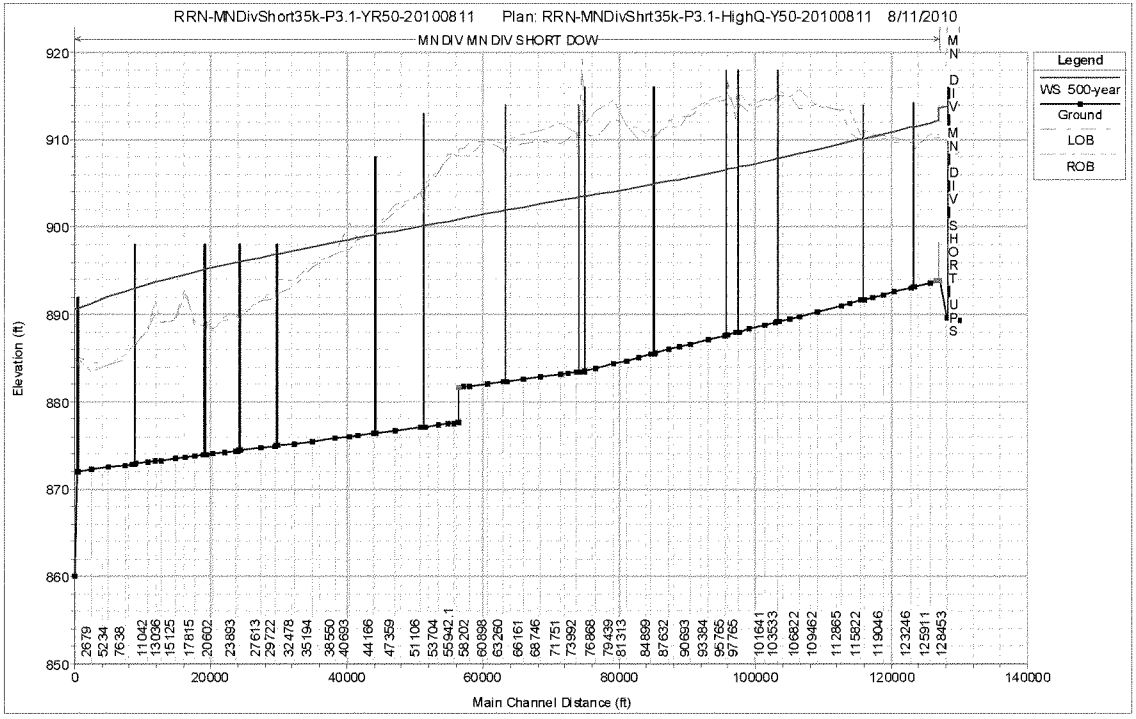

Figure B20 - MN Short Diversion 35 Kcfs Capacity - HEC-RAS Profile, Year 50

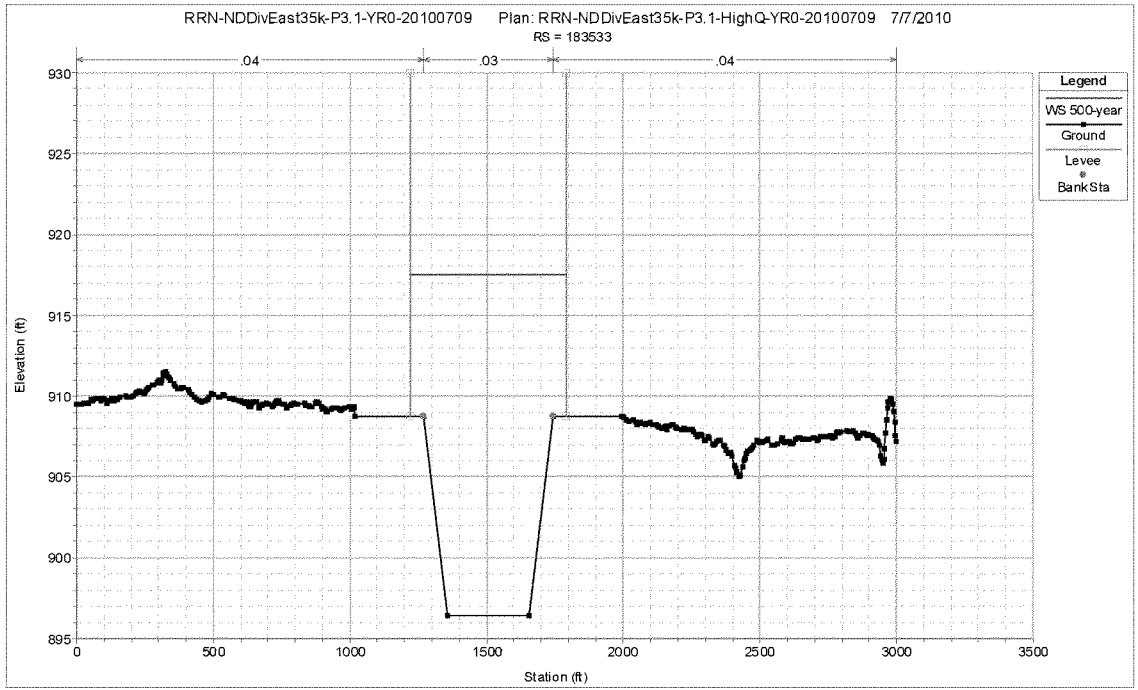

Figure B21 - ND East Alignment Connection Channel - HEC-RAS Typical Model Cross Section

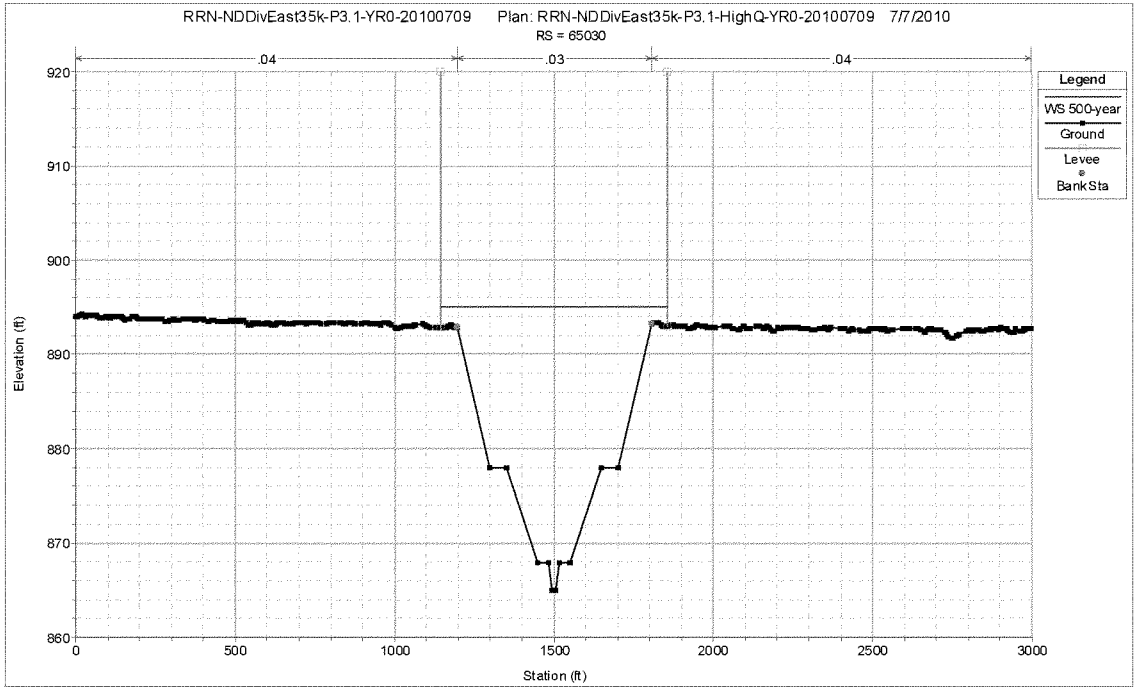

Figure B22 - ND East Diversion 35 Kcfs Capacity - HEC-RAS Sample Model Cross Section

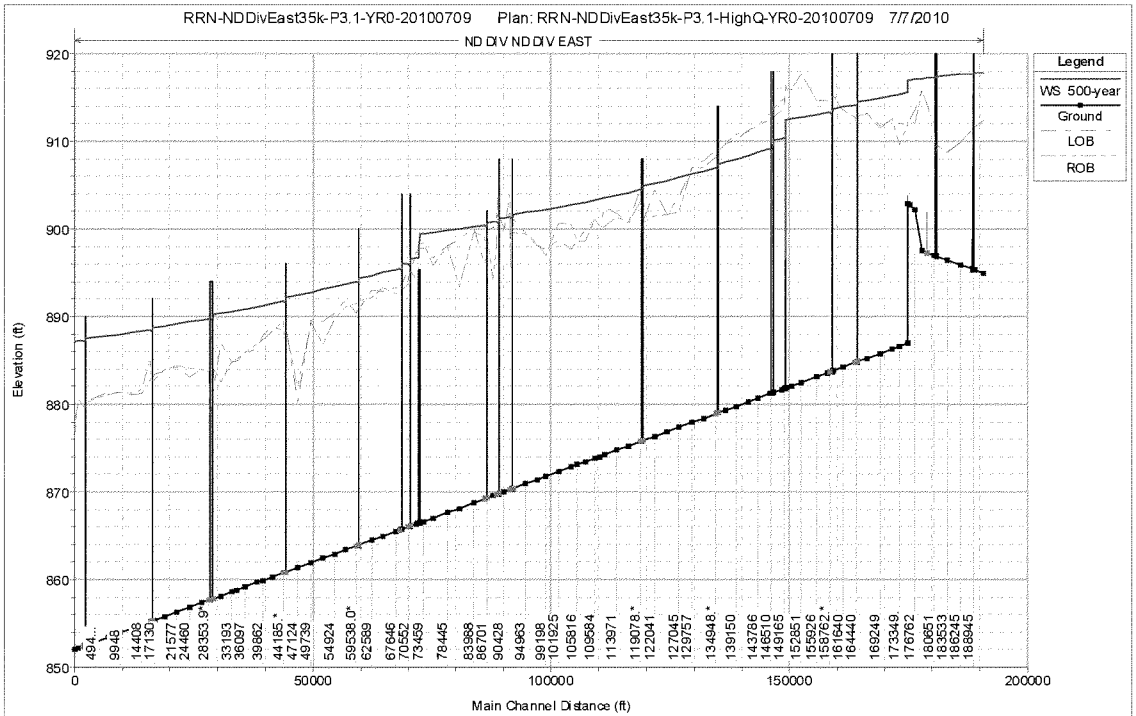

Figure B23 - ND East Diversion 35 Kcfs Capacity - HEC-RAS Profile, Year 0

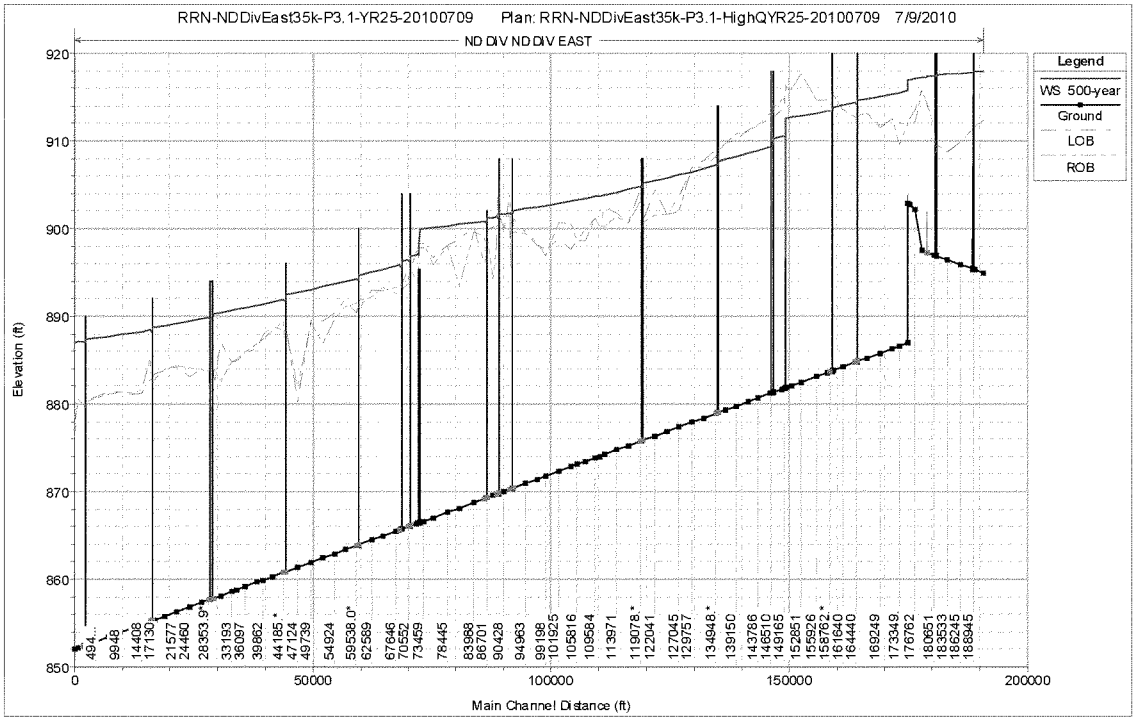

Figure B24 - ND East Diversion 35 Kcfs Capacity - HEC-RAS Profile, Year 25

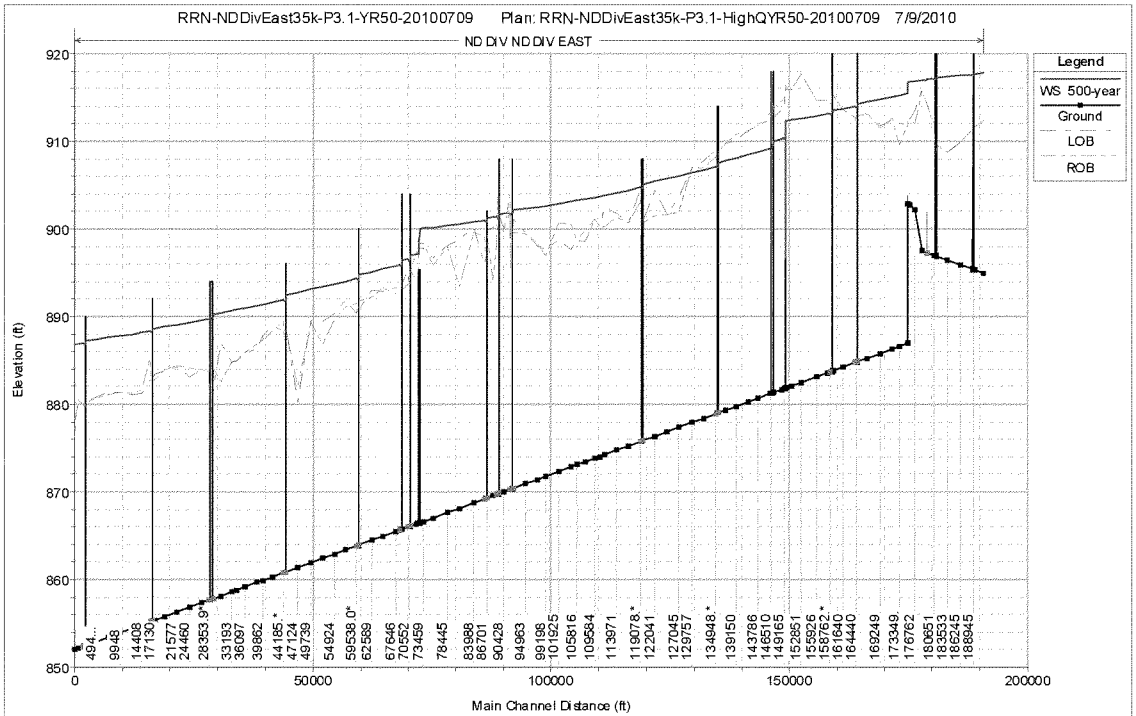

Figure B25 - ND East Diversion 35 Kcfs Capacity - HEC-RAS Profile, Year 50

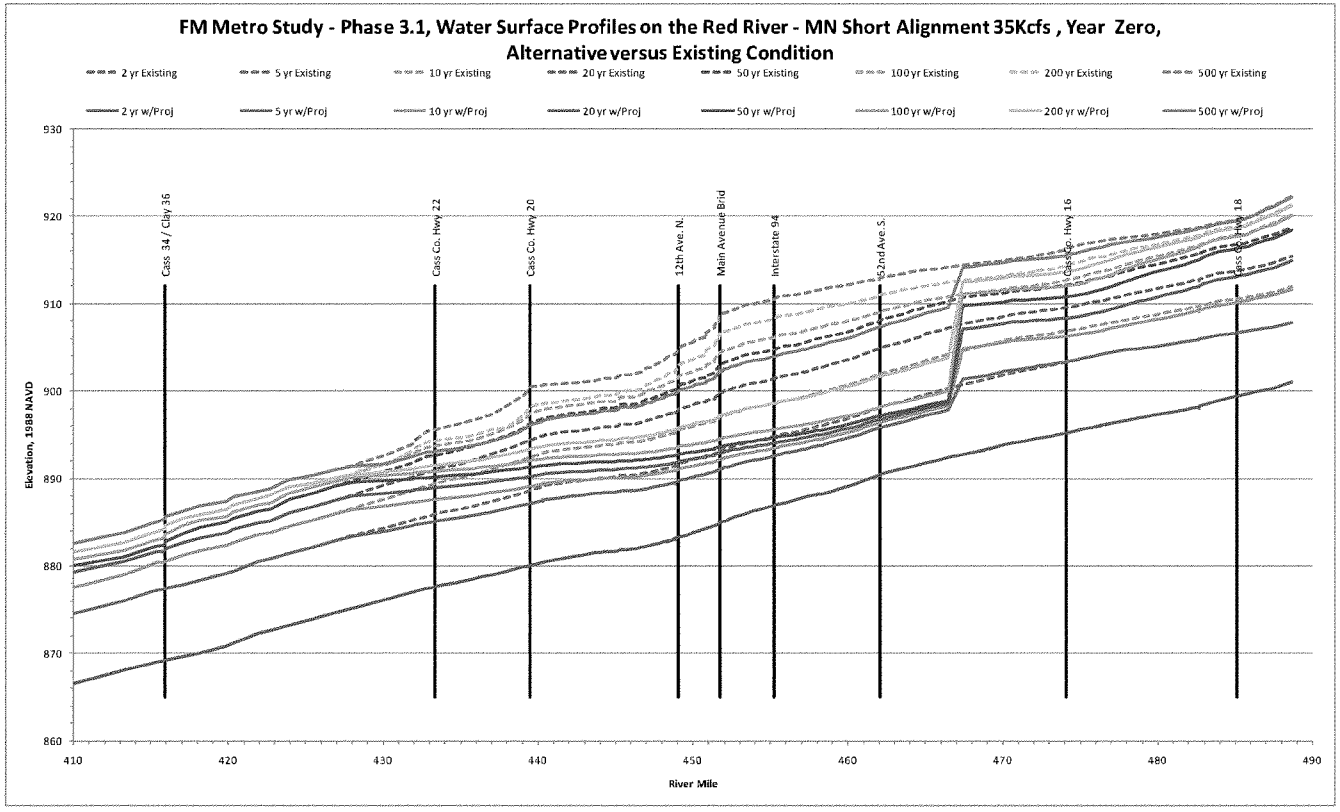

Figure B26 - Water Surface Profiles on the Red River – MN Short Alignment 35 Kcfs vs. Existing Condition, Year 0

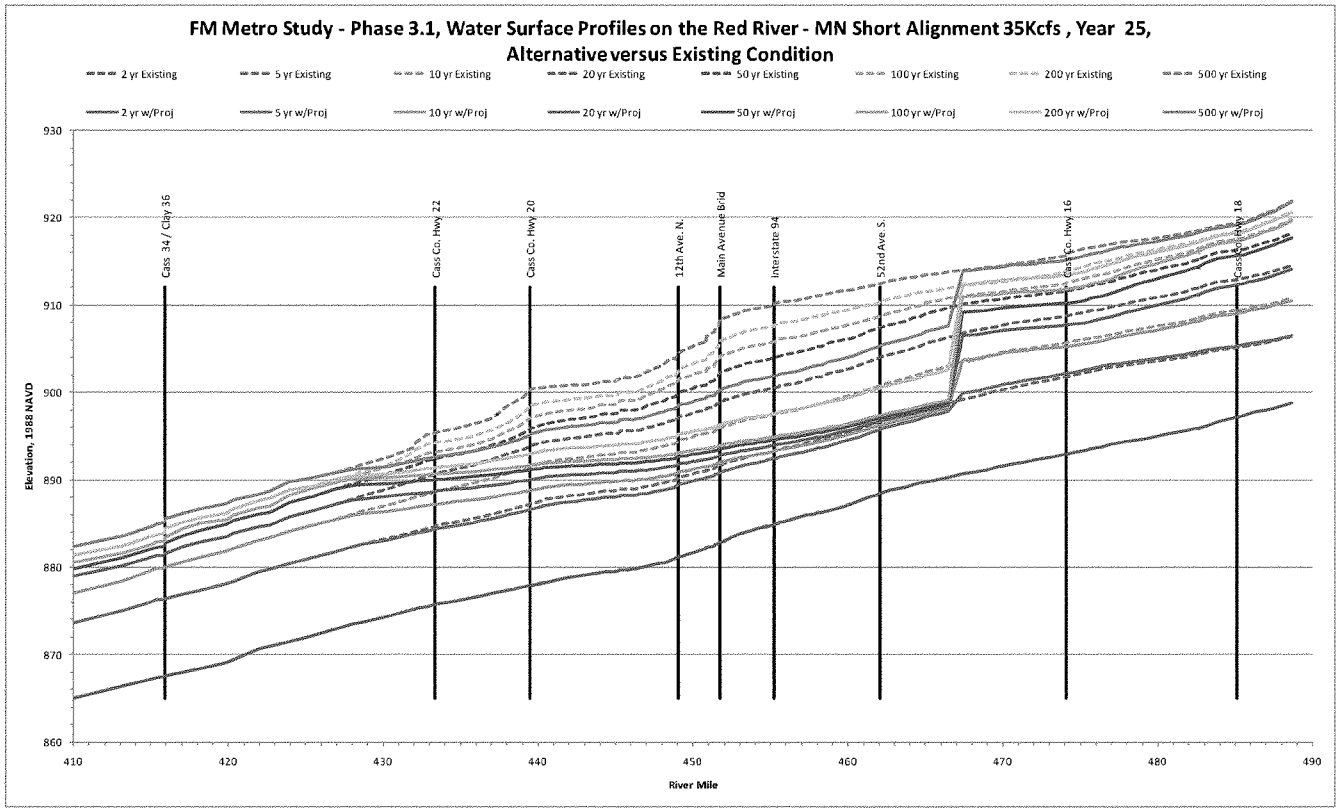

Figure B27 - Water Surface Profiles on the Red River – MN Short Alignment 35 Kcfs vs. Existing Condition, Year 25

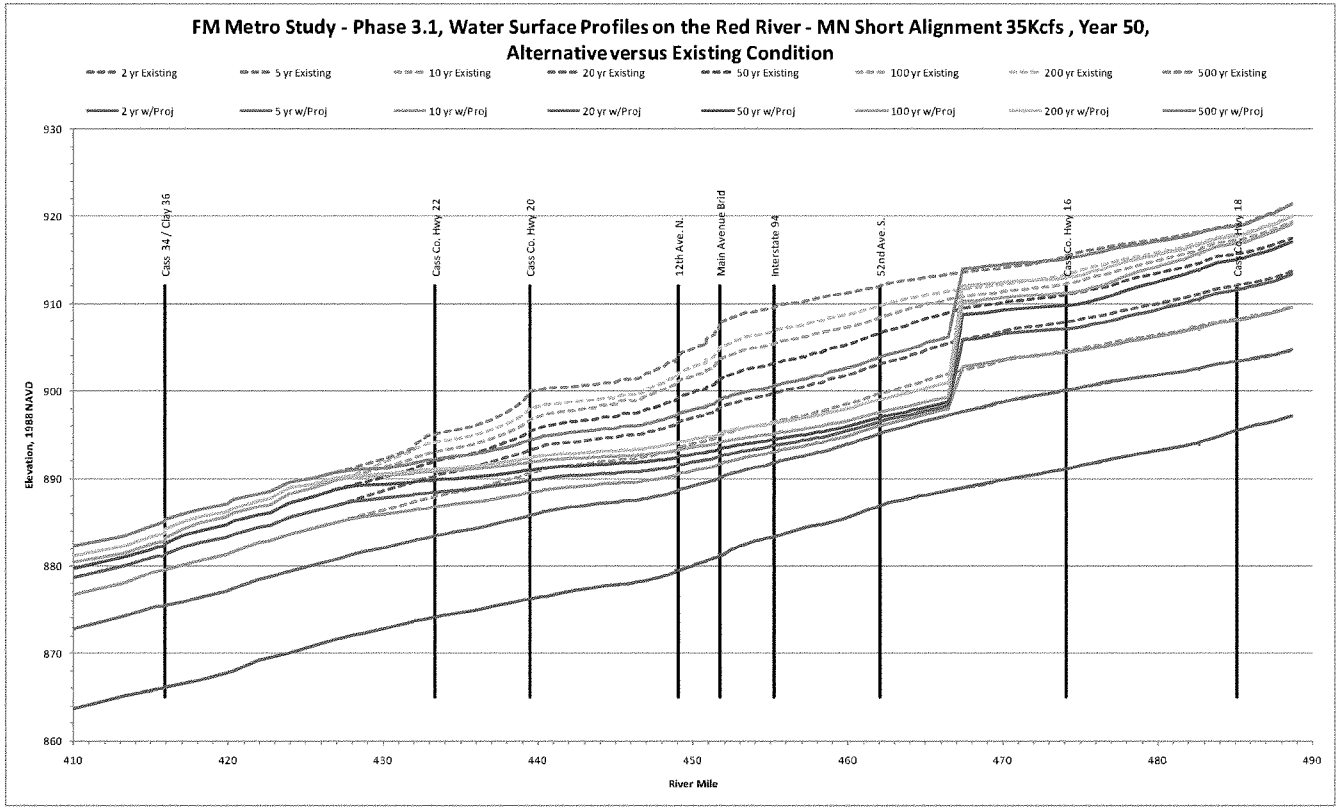

Figure B28 - Water Surface Profiles on the Red River – MN Short Alignment 35 Kcfs vs. Existing Condition, Year 50

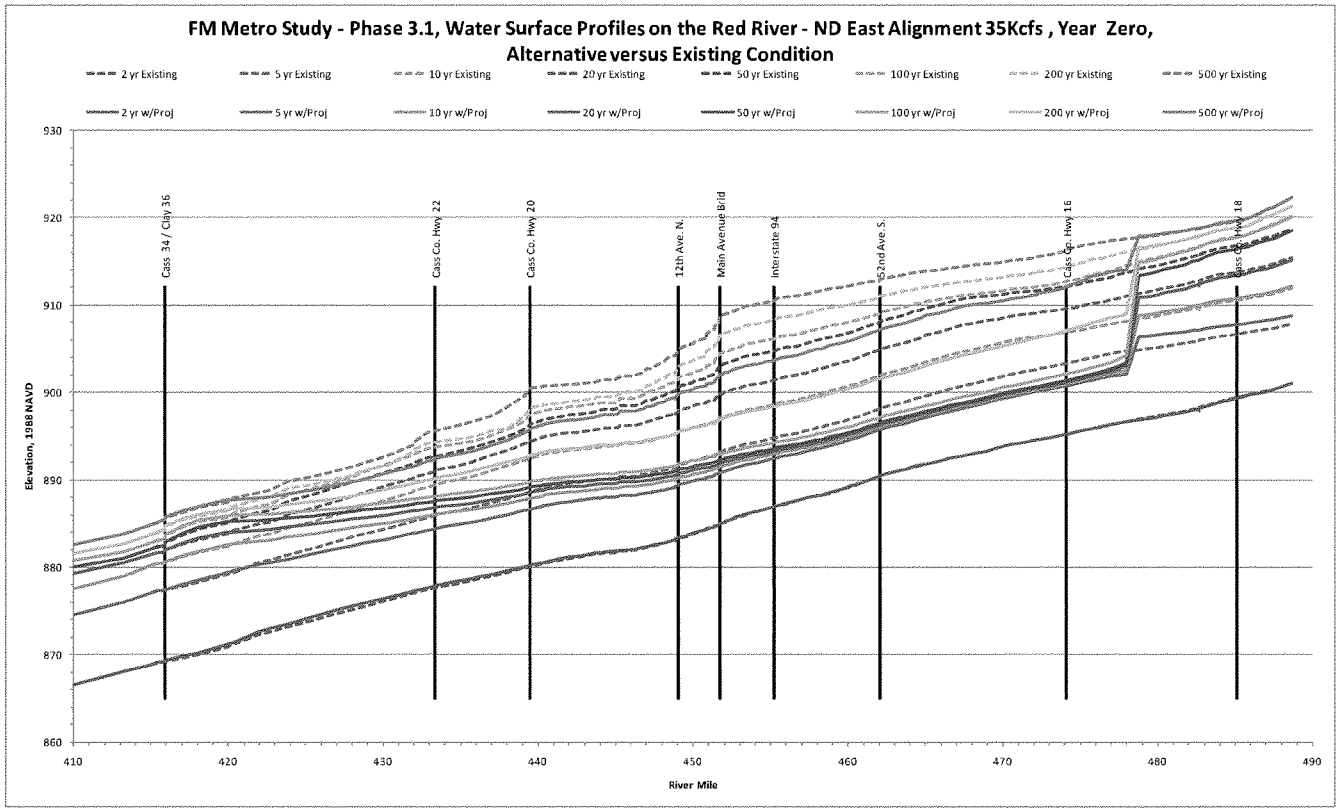

Figure B29 - Water Surface Profiles on the Red River – ND East Alignment 35 Kcfs vs. Existing Condition, Year 0

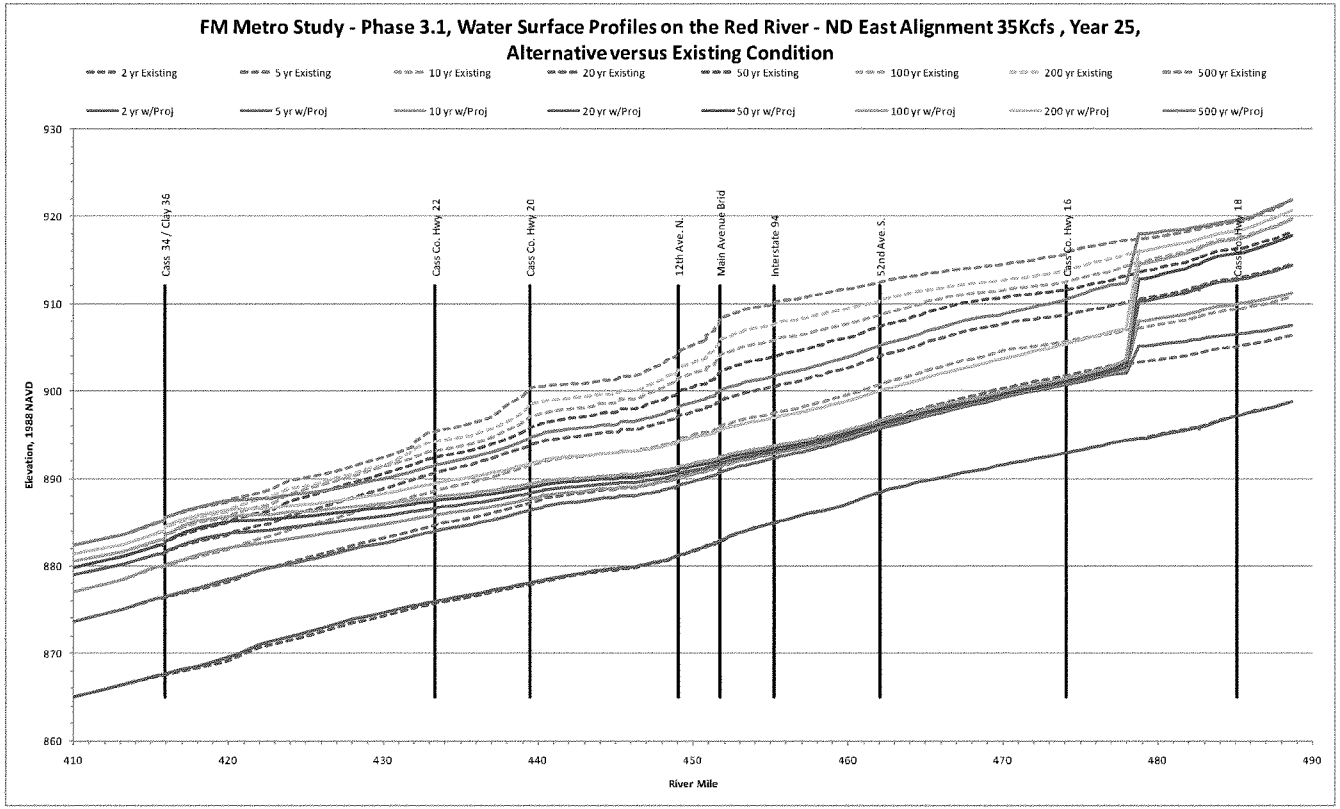

Figure B30 - Water Surface Profiles on the Red River – ND East Alignment 35 Kcfs vs. Existing Condition, Year 25

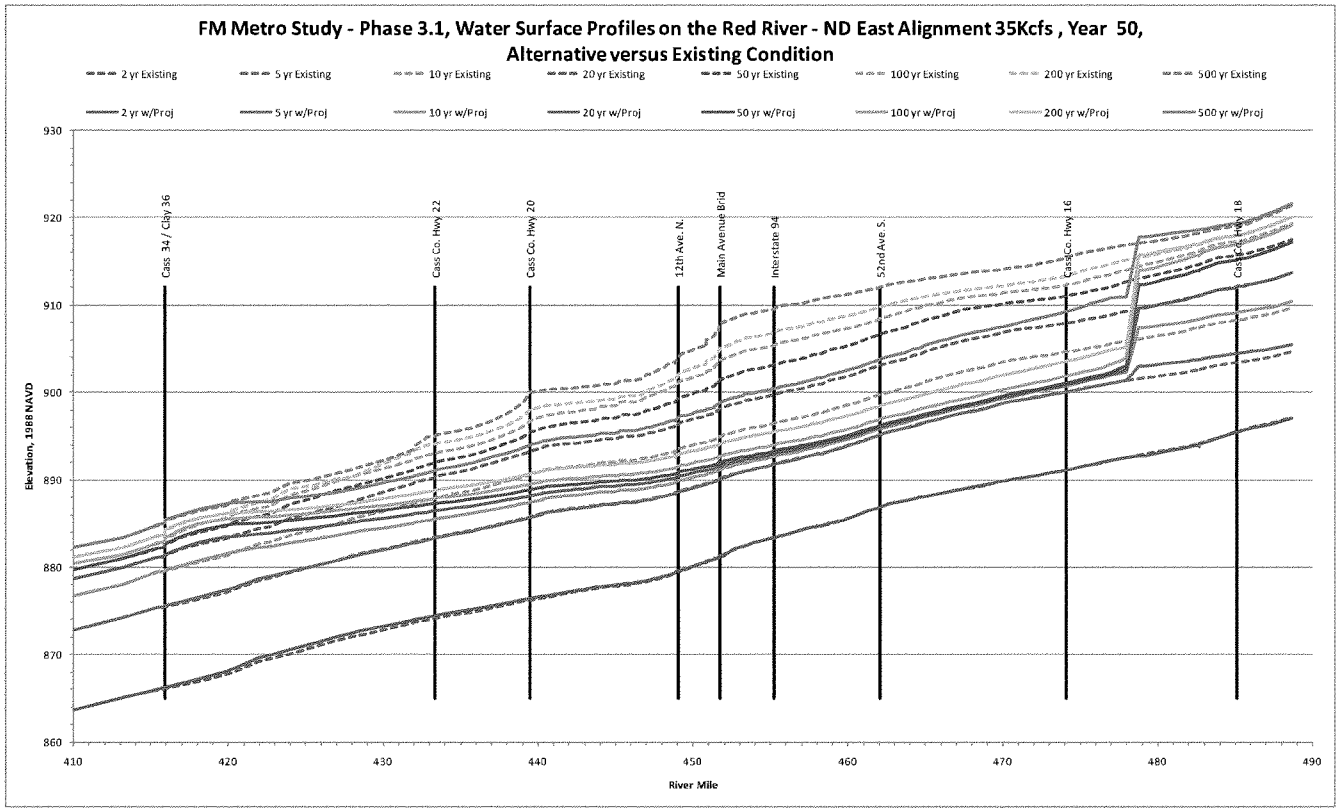

Figure B31 - Water Surface Profiles on the Red River – ND East Alignment 35 Kcfs vs. Existing Condition, Year 50

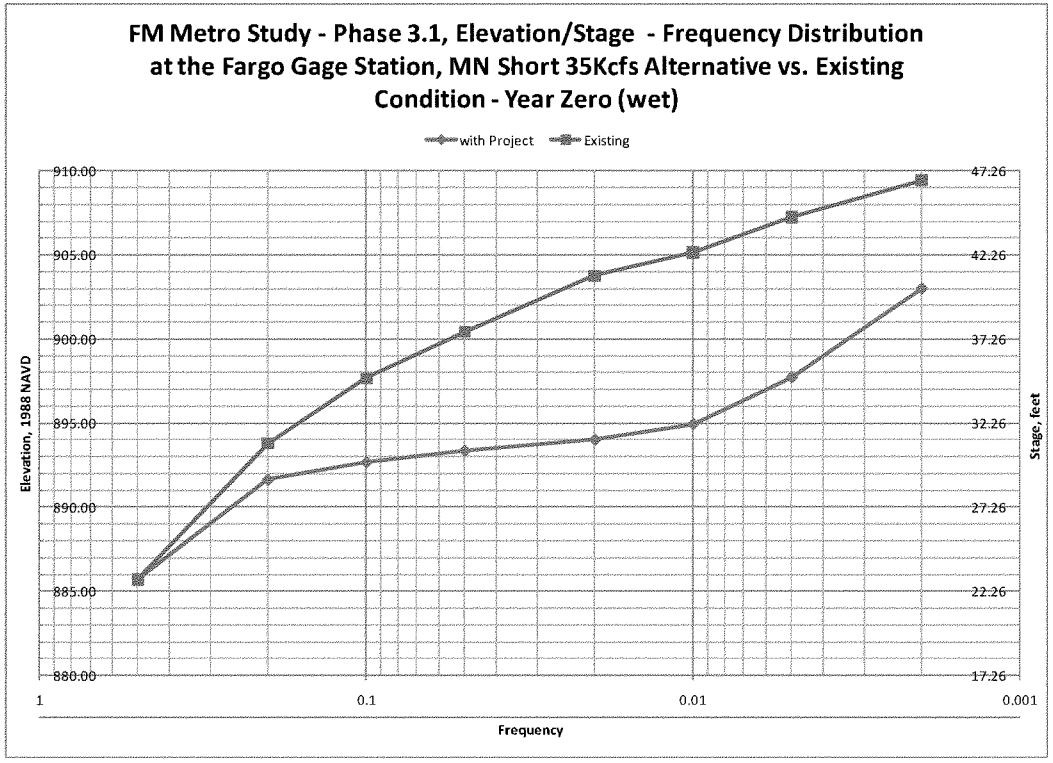

Figure B32 - Elevation/Stage - Frequency Distribution at the Fargo Gage Station, MN Short Align 35 Kcfs vs. Existing , Year 0

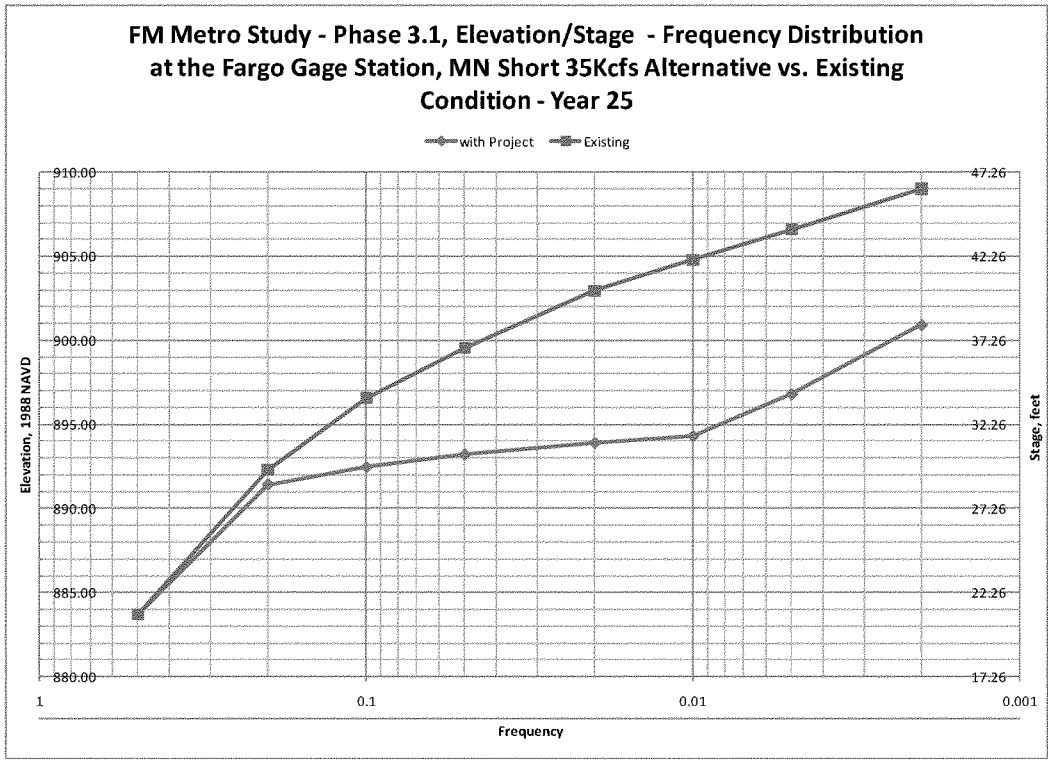

Figure B33 - Elevation/Stage - Frequency Distribution at the Fargo Gage Station, MN Short Align 35 Kcfs vs. Existing , Year 25

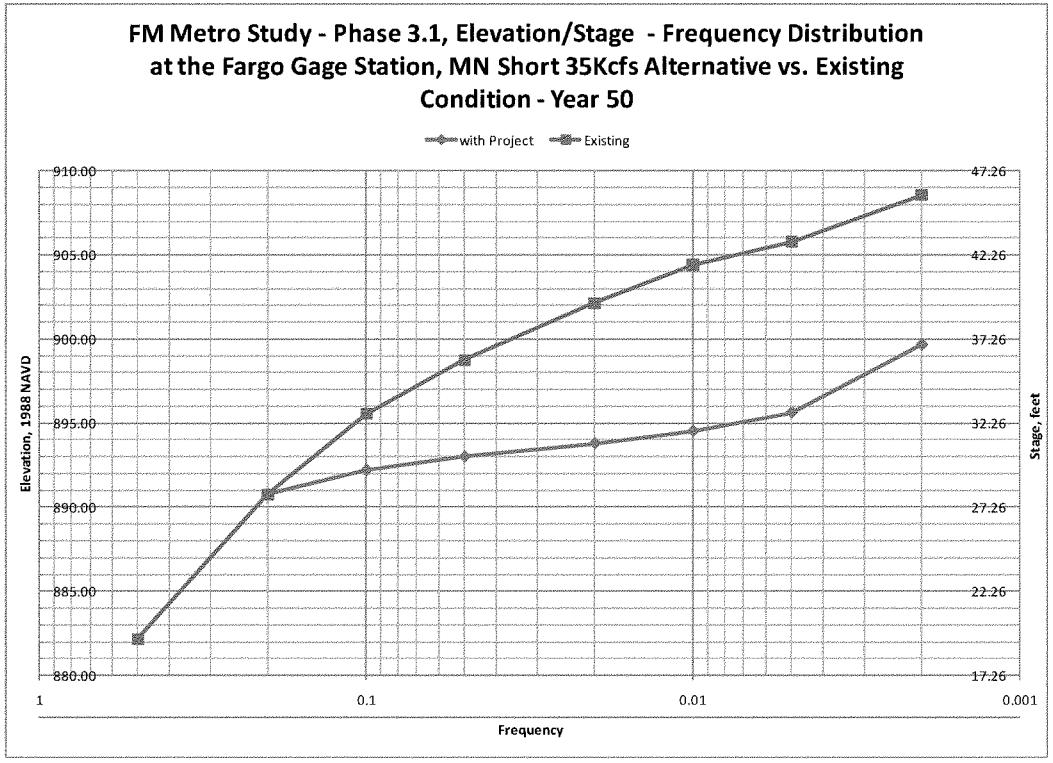

Figure B34 - Elevation/Stage - Frequency Distribution at the Fargo Gage Station, MN Short Align 35 Kcfs vs. Existing , Year 50

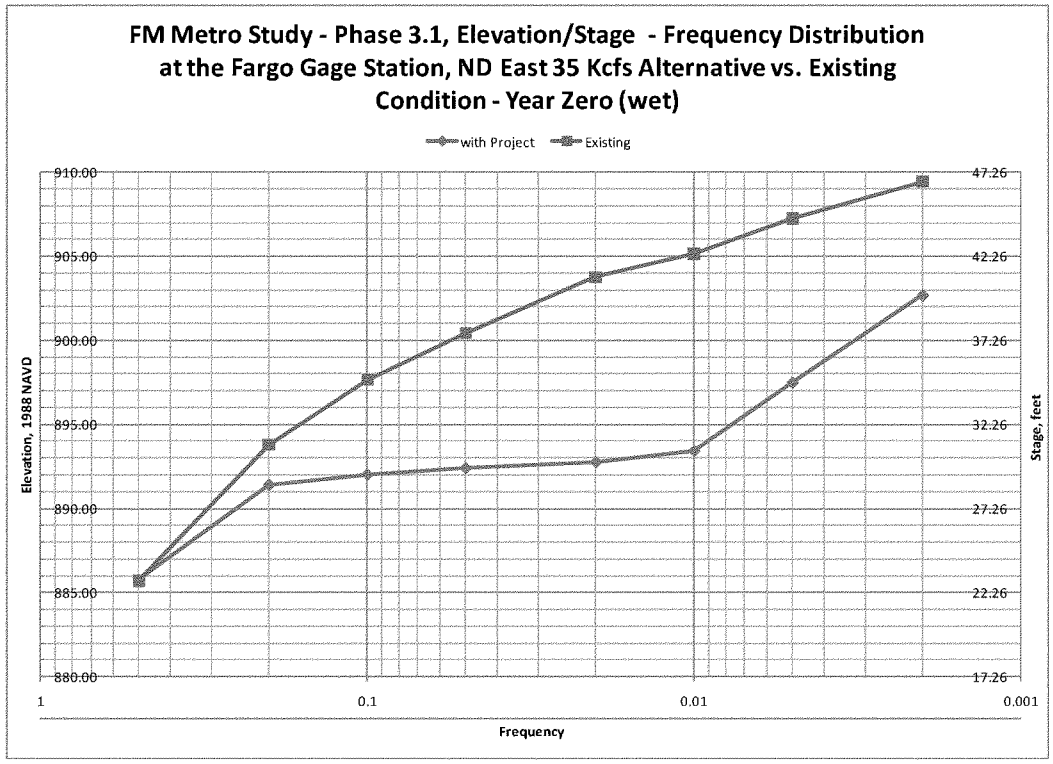

Figure B35 - Elevation/Stage - Frequency Distribution at the Fargo Gage Station, ND East Align 35 Kcfs vs. Existing , Year 0

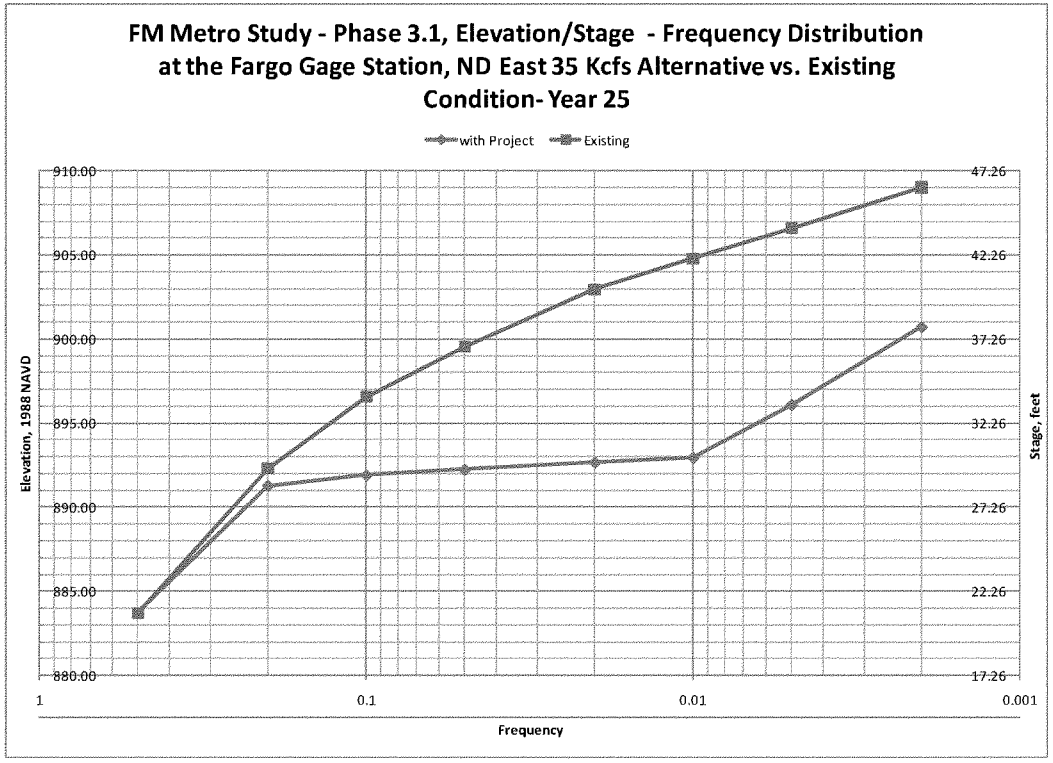

Figure B36 - Elevation/Stage - Frequency Distribution at the Fargo Gage Station, ND East Align 35 Kcfs vs. Existing , Year 25

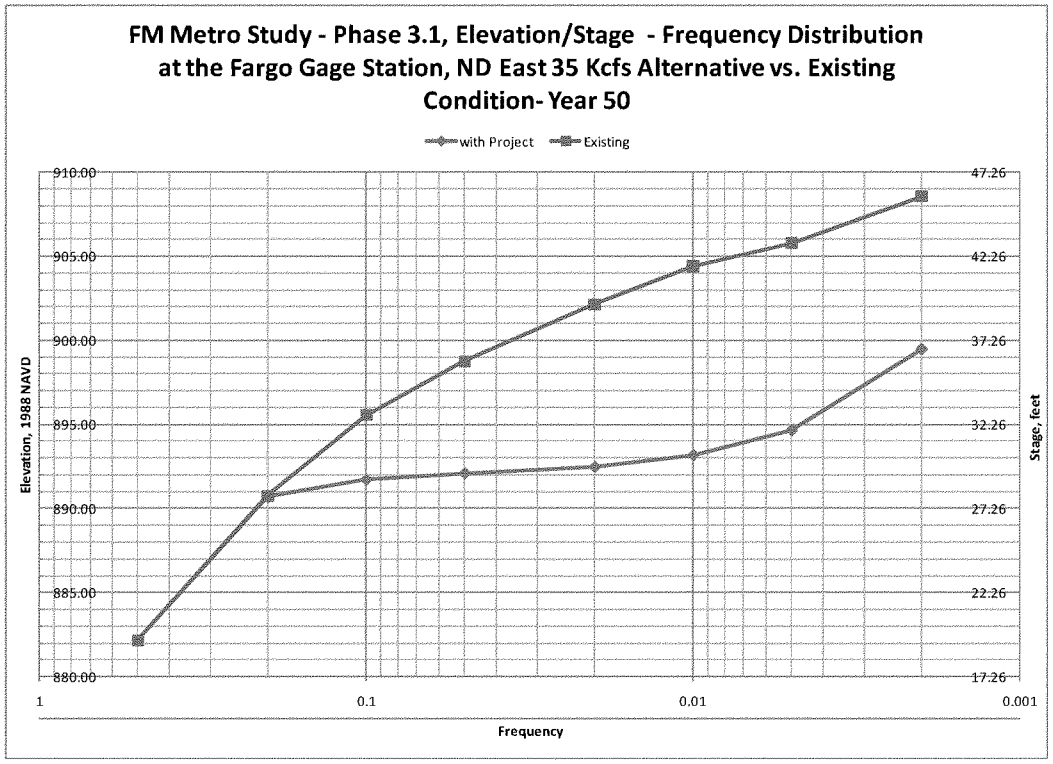

Figure B37 - Elevation/Stage - Frequency Distribution at the Fargo Gage Station, ND East Align 35 Kcfs vs. Existing , Year 50

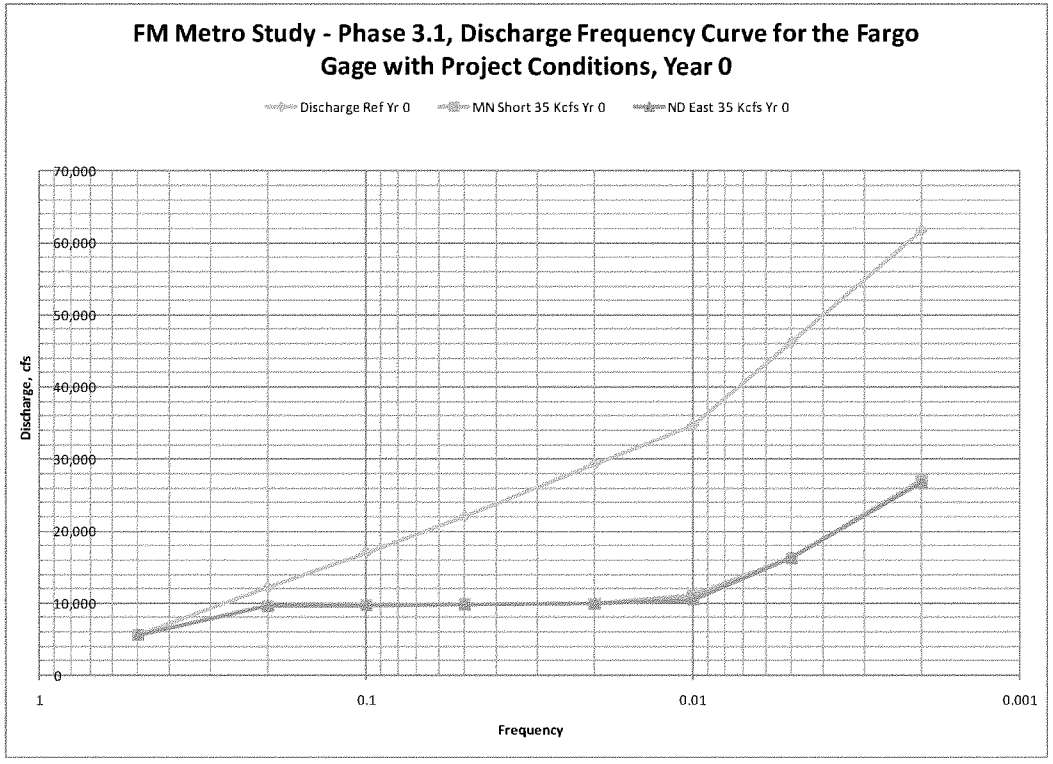

Figure B38 - Discharge Frequency Curve for the Fargo Gage with Project Conditions, Year 0

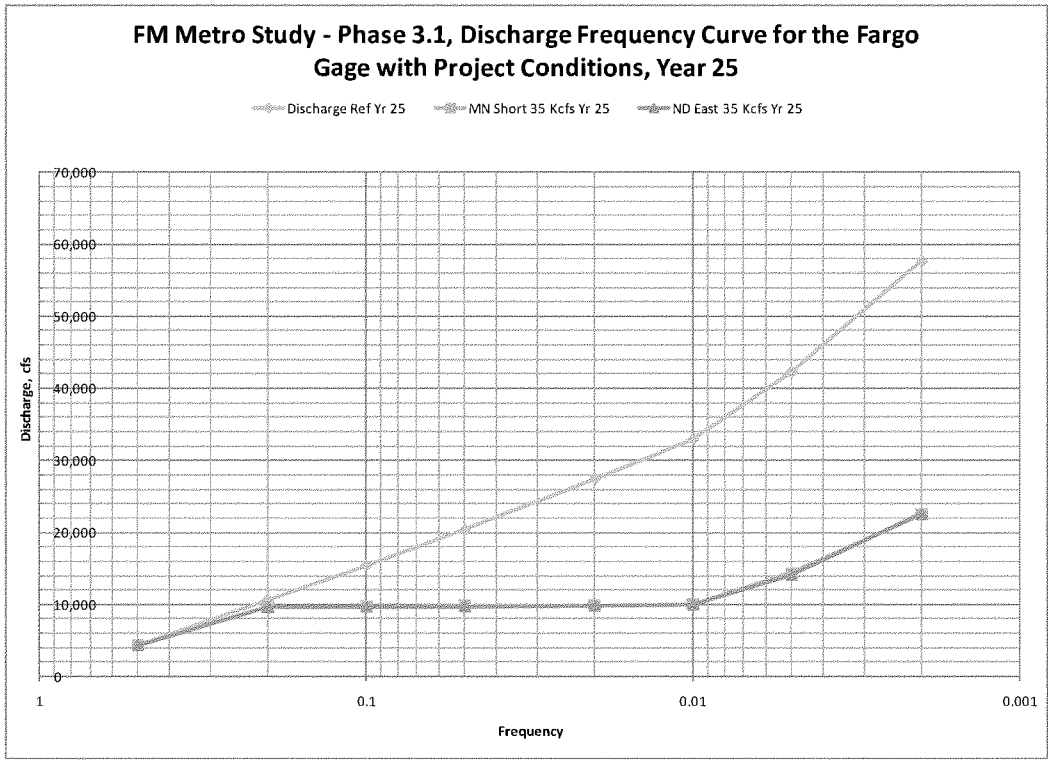

Figure B39 - Discharge Frequency Curve for the Fargo Gage with Project Conditions, Year 25

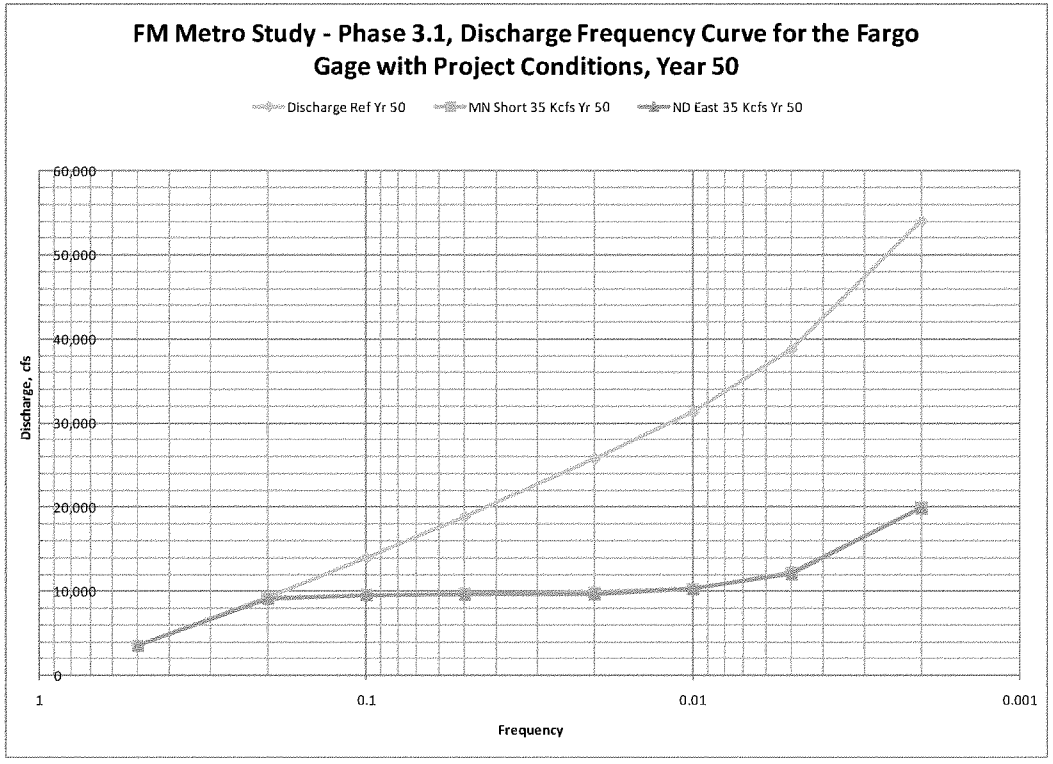

Figure B40 - Discharge Frequency Curve for the Fargo Gage with Project Conditions, Year 50

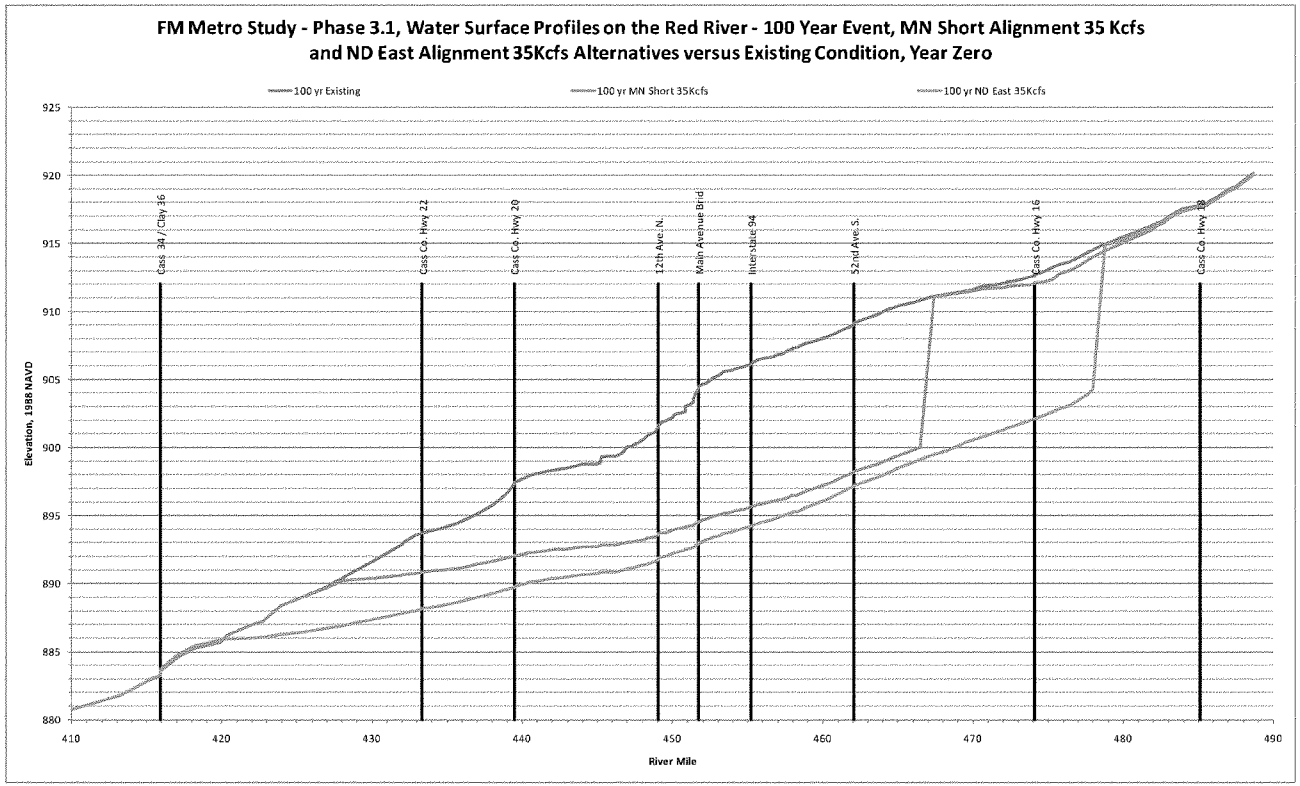

Figure B41 - Water Surface Profiles on the Red River - 100-Year Event, MN Short Align35 Kcfs and ND East Align 35 Kcfs Alternatives vs. Existing Condition, Year 0

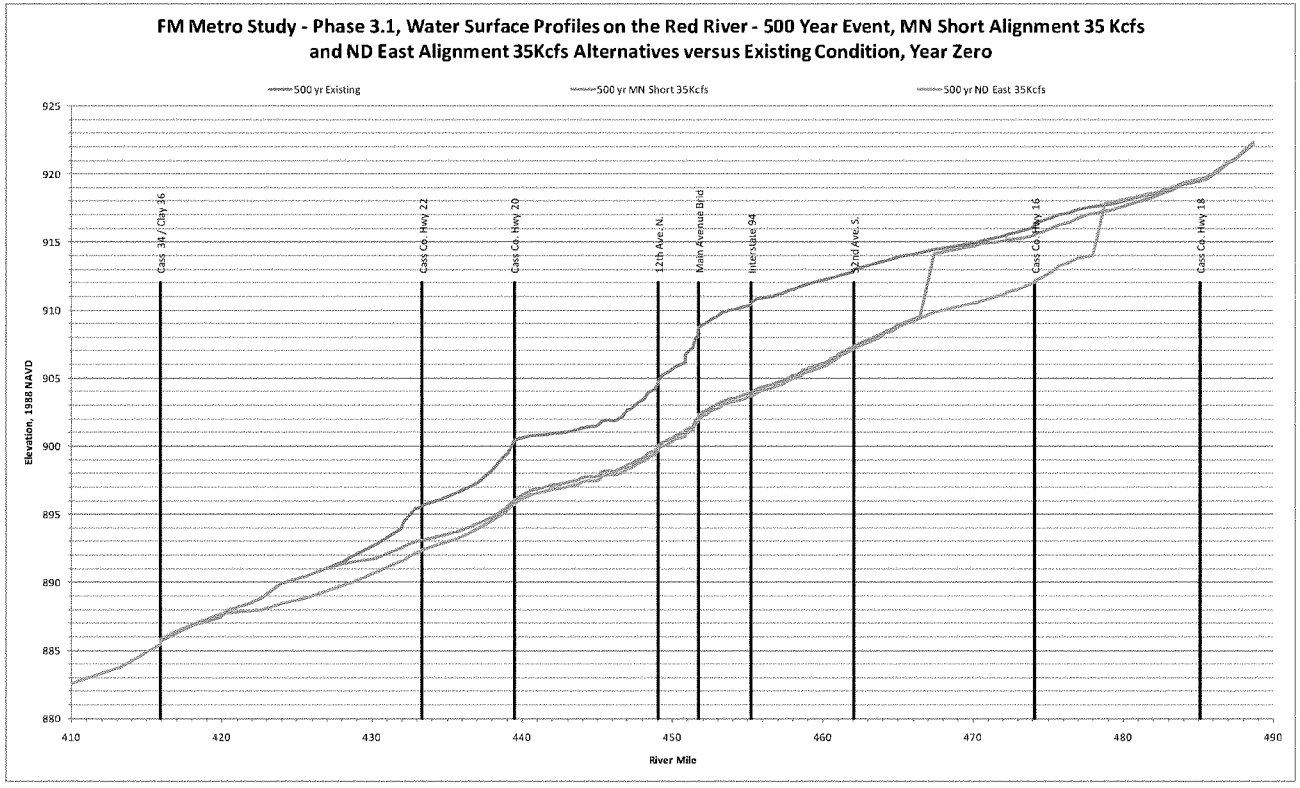

Figure B42 - Water Surface Profiles on the Red River - 500-Year Event, MN Short Align35 Kcfs and ND East Align 35 Kcfs Alternatives vs. Existing Condition, Year 0

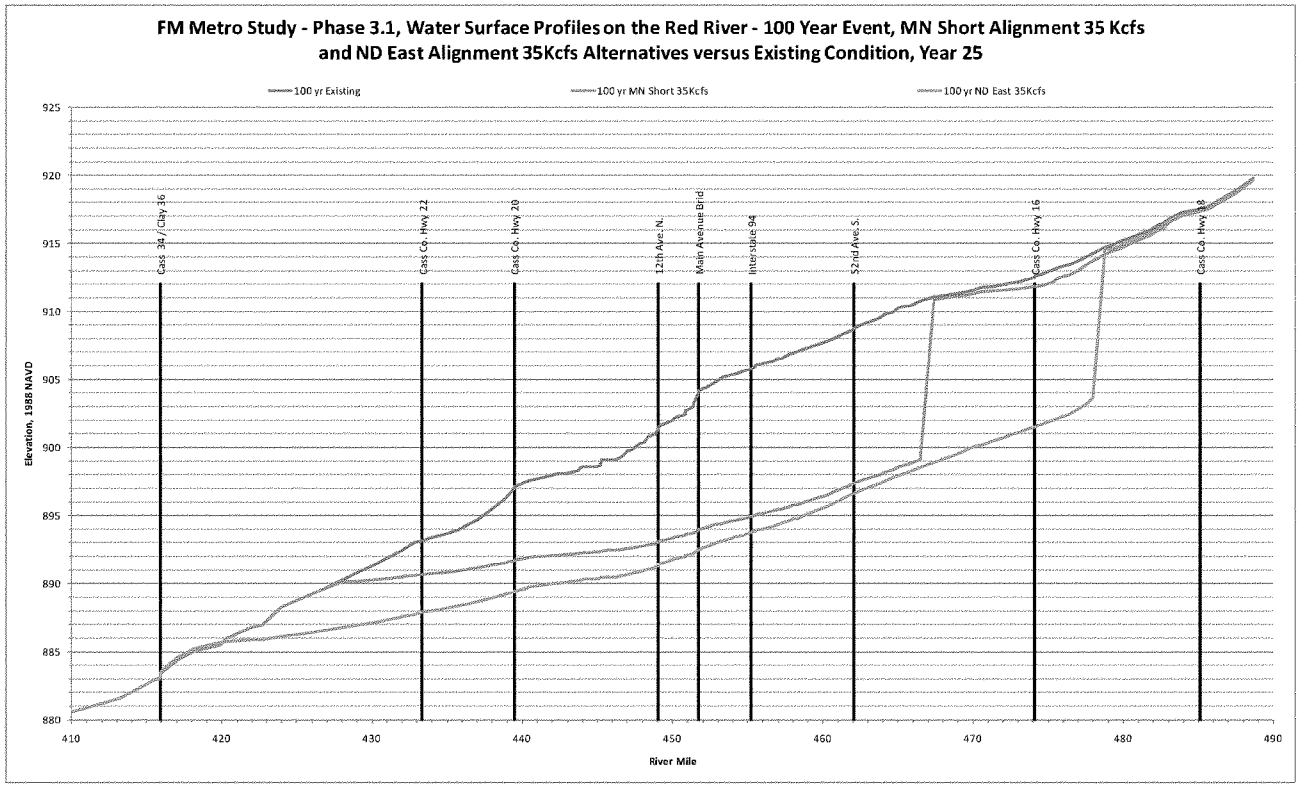

Figure B43 - Water Surface Profiles on the Red River - 100-Year Event, MN Short Align35 Kcfs and ND East Align 35 Kcfs Alternatives vs. Existing Condition, Year 25

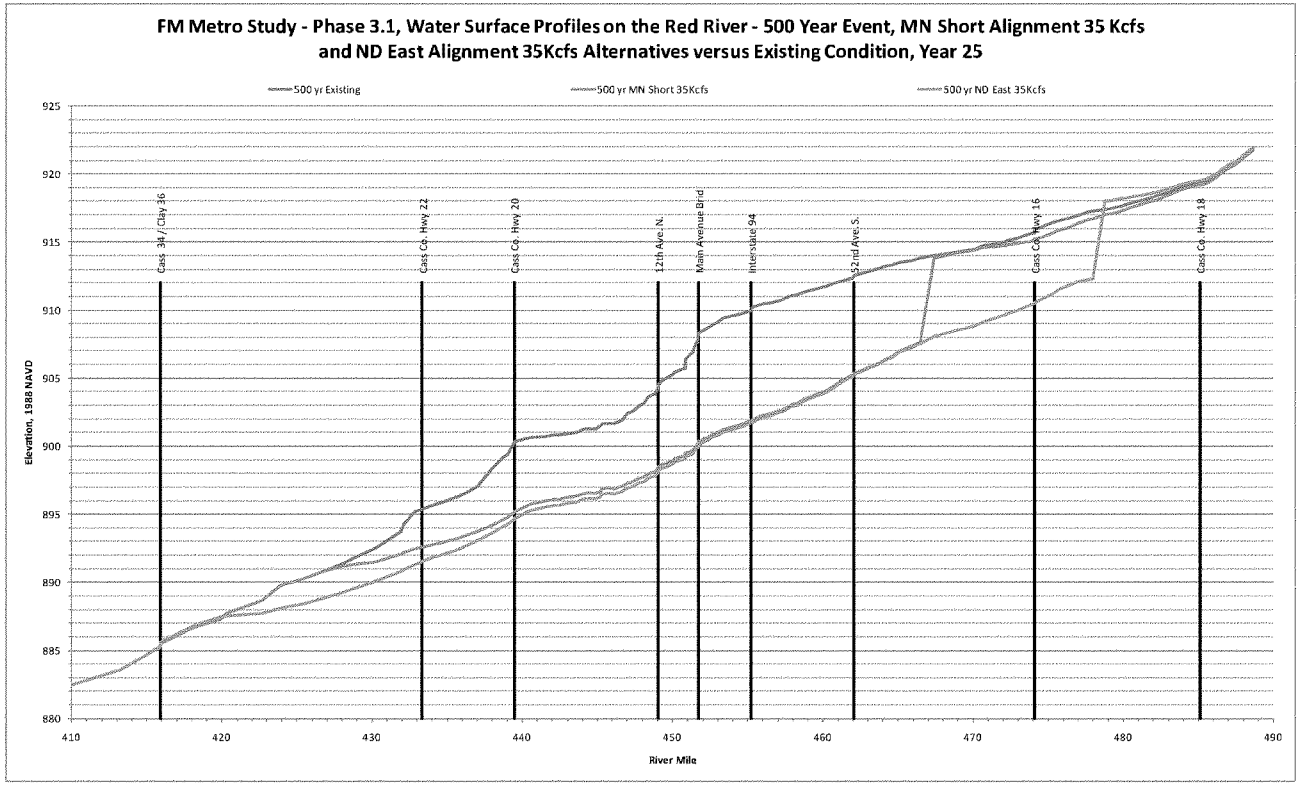

Figure B44 - Water Surface Profiles on the Red River - 500-Year Event, MN Short Align35 Kcfs and ND East Align 35 Kcfs Alternatives vs. Existing Condition, Year 25

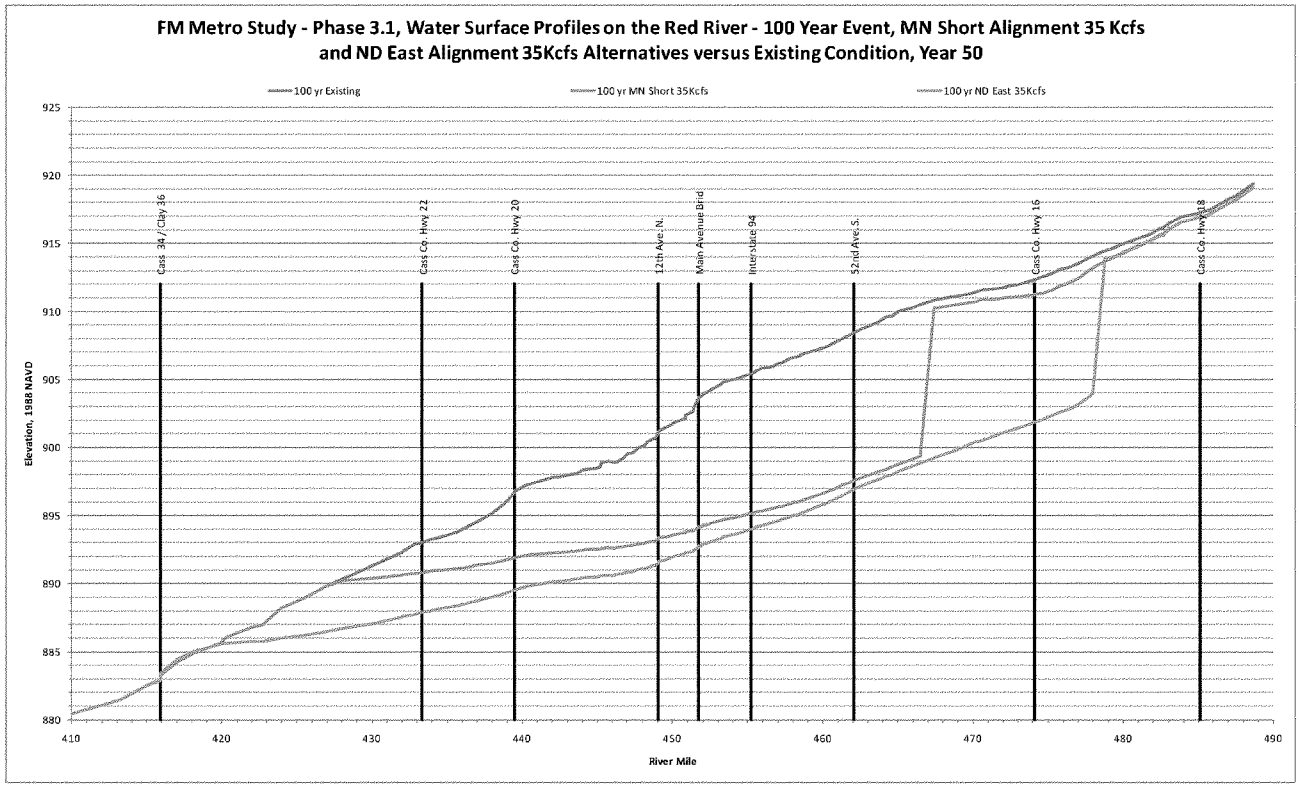

Figure B45 - Water Surface Profiles on the Red River - 100-Year Event, MN Short Align35 Kcfs and ND East Align 35 Kcfs Alternatives vs. Existing Condition, Year 50

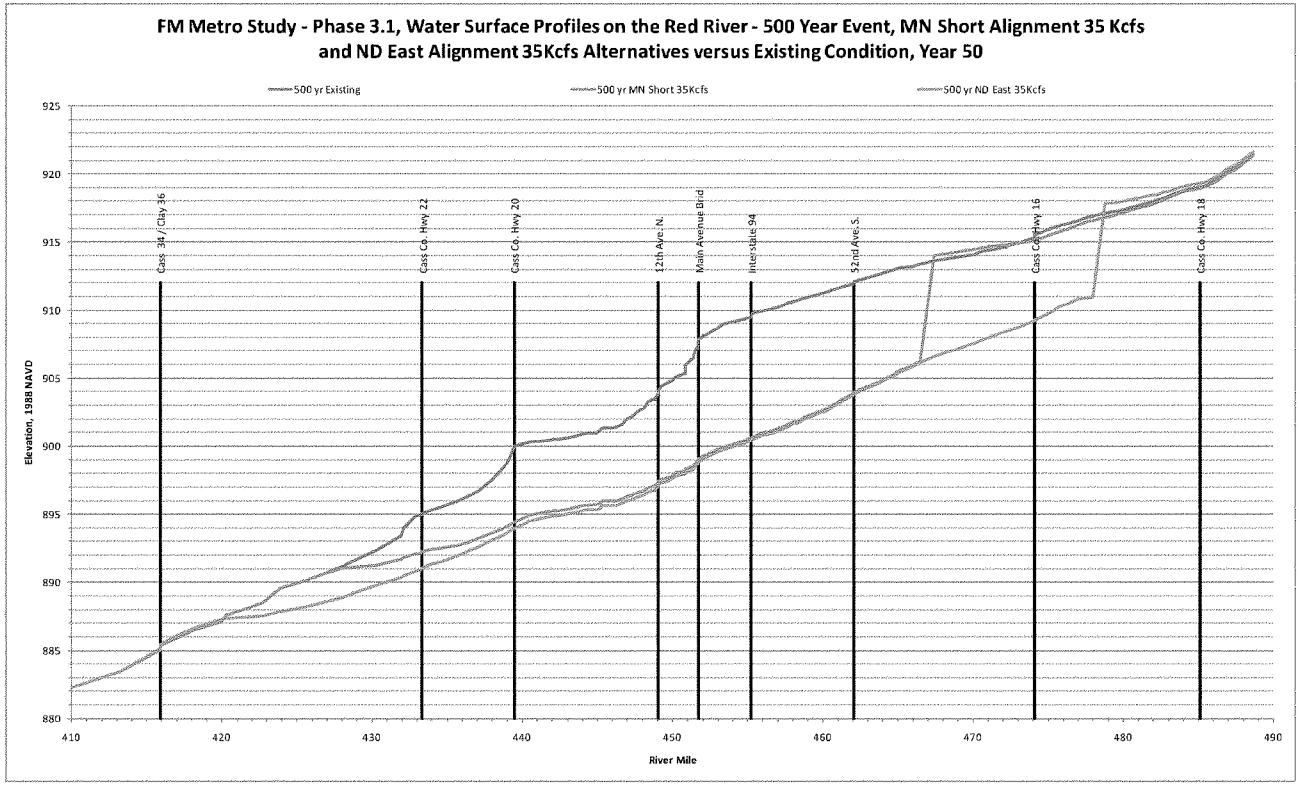

Figure B46 - Water Surface Profiles on the Red River - 500-Year Event, MN Short Align35 Kcfs and ND East Align 35 Kcfs Alternatives vs. Existing Condition, Year 50

Table B-3
Red River of the North - Fargo-Moorhead Metro Area to Perley
Initial HEC-RAS Discharges (Year Zero)

Location/Source	Fargo-Moorhead Metro Study, Phase 3 Hydrologs																				Location/Source
	2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year		1,000-Year		10,000-Year		
Hickson Gage	4,000		7,000		10,500		14,500		21,000		25,000		28,500		32,000		35,000		77,714		Hickson Gage
Phase 3.1 Drainage Area Analysis_Fargo Gage	6,000		13,100		17,600		22,000		28,500		34,000		40,000		52,000		74,000		220,000		Phase 3.1 Drainage Area Analysis_Fargo Gage
XS 283	10,380		20,140		27,800		36,050		47,720		57,070		69,040		85,540		98,600		147,070		XS 283
Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River Contributions	4,775		7,040		10,200		14,050		19,420		22,970		28,540		33,940		23,660		36,070		Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River Contributions
Drains 3 & 49	30	0.0054	45	0.0049	55	0.0047	65	0.0041	90	0.0054	70	0.0030	75	0.0031	90	0.0032	95	0.0032	100	0.0032	Drains 3 & 49
Drain 10	195	0.0403	280	0.0327	370	0.0307	395	0.0264	430	0.0226	440	0.0198	475	0.0206	510	0.0193	520	0.0193	535	0.0193	Drain 10
Drain 40/45, Sheyenne River & Buffalo River Contributions	4,555		7,065		10,435		13,590		17,910		21,625		22,290		23,240		24,025		25,415		Drain 40/45, Sheyenne River & Buffalo River Contributions
Buffalo River	1,312	Note 1	3,061	Note 1	4,431	Note 1	5,809	Note 1	7,604	Note 1	9,100	Note 1	9,275	Note 1	9,630	Note 1	9,850	Note 1	10,430	Note 1	Buffalo River
Drain 40/45	293	0.0391	424	0.0490	534	0.0448	791	0.0529	1,166	0.0611	1,320	0.0684	1,325	0.0642	1,590	0.0601	1,655	0.0601	1,765	0.0601	Drain 40/45
Sheyenne River	2,930	Note 1	4,180	Note 1	5,450	Note 1	6,990	Note 1	9,160	Note 1	11,240	Note 1	11,490	Note 1	12,050	Note 1	12,530	Note 1	13,200	Note 1	Sheyenne River
Check Total for Drain 40/45, Sheyenne & Buffalo	4,535		7,065		10,435		13,590		17,930		21,605		22,290		23,240		24,025		25,415		Check Total for Drain 40/45, Sheyenne & Buffalo
Sheyenne River Flow																					Sheyenne River Flow
75% enters at Sheyenne, XS 299	2,210		3,140		4,090		5,240		6,870		8,430		8,620		9,040		9,400		9,900		75% enters at Sheyenne, XS 299
25% enters at Drain 13, XS 283	740		1,040		1,360		1,750		2,290		2,810		2,870		3,010		3,130		3,300		25% enters at Drain 13, XS 283

Location/Source	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	Location/Source
XS 415	5,690		12,150		17,000		22,000		28,320		34,200		40,200		46,200		74,000		121,000		**XS 415**
Oakport Breakout LS, XS 221		1		1		1		1		1		1		1		1		1		1	Oakport Breakout LS, XS 221
Drains 3 & 49 Flow Enters		30		45		55		65		90		70		75		90		95		100	Drains 3 & 49 Flow Enters
XS 326	5,630		12,105		17,053		22,065		28,360		34,270		40,275		46,290		74,095		121,100		**XS 326**
Drain 10 Flow Enters		195		280		370		395		430		440		475		510		530		555	Drain 10 Flow Enters
LOB_RRN breakout LS, XS 2.89		1		1		1		1		1		1		1		1		1		1	LOB_RRN breakout LS, XS 2.89
XS 311	5,632		12,475		17,425		22,460		28,750		34,710		40,750		46,790		74,625		121,655		**XS 311**
XS 303.5	5,825		12,475		17,425		22,460		28,750		35,210		40,750		46,790		74,625		121,655		**XS 303.5**
Drain 40/45		293		424		534		791		1,166		1,320		1,325		1,590		1,655		1,765	Drain 40/45
XS 303	6,118		12,899		17,959		23,251		29,916		36,530		42,075		48,380		76,280		123,420		**XS 303**
Sheyenne River flow		5,210		3,140		4,090		5,240		6,870		8,430		8,620		9,040		9,400		9,900	Sheyenne River flow
XS 299	6,118		16,039		22,049		28,491		37,876		45,160		56,895		75,930		85,680		133,320		**XS 299**
Buffalo River flow		1,312		3,061		4,431		5,809		7,604		9,100		9,275		9,630		9,850		10,430	Buffalo River flow
XS 291	9,648		19,100		26,500		34,300		45,410		54,260		66,170		85,210		94,530		143,770		**XS 291**
Drain 13/Sheyenne River flow		740		1,040		1,360		1,750		2,290		2,810		2,870		3,010		3,130		3,300	Drain 13/Sheyenne River flow
XS 283	10,380		20,140		27,800		36,050		47,720		57,070		69,040		85,540		98,600		147,070		**XS 283**

River	Reach	RS	HEC-RAS Flow File Input- Existing Conditions- Wet Scenario- Year 0																		River	Reach	RS
			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year		1,000-Year		10,000-Year		
lob_rrn	overland	2.89	1		1		1		1		1		1		1		1		1		1	lob_rrn	overland	2.89
Oakport_SI	Oakport	221	1		1		1		1		1		1		1		1		1		1	Oakport_SI	Oakport	221
RRN	Cass_Clay_Fargo	488.68	4000		7000		10500		14500		21000		25000		28500		32000		35000		77714	RRN	Cass_Clay_Fargo	488.68
RRN	Cass_Clay_Fargo	477	5021		7227		10537		13918		20568		24672		28720		33777		36820		78578	RRN	Cass_Clay_Fargo	477
RRN	Cass_Clay_Fargo	476.4	4968		8052		10793		13455		17872		21395		26755		42395		52104		93574	RRN	Cass_Clay_Fargo	476.4
RRN	Cass_Clay_Fargo	470.1	5600		12140		17000		22066		29200		34500		40200		61900		74000		121000	RRN	Cass_Clay_Fargo	470.1
RRN	Cass_Clay_Fargo	326	5630		12105		17053		22065		28360		34270		40275		61900		74095		121100	RRN	Cass_Clay_Fargo	326
RRN	Cass_Clay_Fargo	311	5825		12475		17425		22460		28750		35210		40750		62300		74625		121655	RRN	Cass_Clay_Fargo	311
RRN	Above_Sheyenne	303.8	5825		12475		17425		22460		28750		35210		40750		62300		74625		121655	RRN	Above_Sheyenne	303.8
RRN	Above_Sheyenne	303	6118		12899		17959		23251		29916		36530		42075		63400		76280		123420	RRN	Above_Sheyenne	303
RRN	Sheyenne_R_to_P	299	6118		16039		22049		28491		37876		45160		56895		72930		85680		133320	RRN	Sheyenne_R_to_P	299
RRN	Sheyenne_R_to_P	291	9648		19100		26500		34300		45410		54260		66170		82330		94530		143770	RRN	Sheyenne_R_to_P	291
RRN	Sheyenne_R_to_P	283	10380		20140		27800		36050		47720		57070		69040		85540		98600		147070	RRN	Sheyenne_R_to_P	283

Note 1 Discharge values based on coincidental frequency analysis.

Table B-4
Red River of the North - Fargo-Moorhead Metro Area to Perley
Initial HEC-RAS Discharges (Year 25)

Location/Source	Fargo-Moorhead Metro Study, Phase 3.1 Hydrology																Location/Source
	2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year		
Hickson Gage	3,139		6,160		9,555		13,729		19,709		23,757		26,164		30,016		Hickson Gage
Phase 3.1 Drainage Area Analysis_Fargo Gage	4,352		10,608		15,994		20,345		27,441		32,921		42,242		57,641		Phase 3.1 Drainage Area Analysis_Fargo Gage
XS 283	8,950		18,490		26,120		34,240		45,780		55,200		66,150		82,930		XS 283
Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River	4,598		7,882		10,726		13,895		18,339		22,279		23,908		25,289		Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River
Drains 3 & 49	20	0.0054	45	0.0049	55	0.0047	60	0.0041	65	0.0034	70	0.0030	75	0.0031	95	0.0032	Drains 3 & 49
Drain 10	200	0.0403	295	0.0327	390	0.0307	415	0.0264	440	0.0226	450	0.0198	535	0.0206	575	0.0193	Drain 10
Drain 40/45, Sheyenne River & Buffalo River Contributions	4,378		7,542		10,281		13,420		17,834		21,759		23,298		24,619		Drain 40/45, Sheyenne River & Buffalo River Contributions
Buffalo River	1,076	Note 1	2,719	Note 1	4,036	Note 1	5,385	Note 1	7,149	Note 1	8,623	Note 1	8,846	Note 1	9,213	Note 1	Buffalo River
Drain 40/45	292	0.0591	443	0.0490	565	0.0448	815	0.0529	1,195	0.0611	1,556	0.0684	1,682	0.0642	1,786	0.0601	Drain 40/45
Sheyenne River	3,010	Note 1	4,380	Note 1	5,680	Note 1	7,220	Note 1	9,490	Note 1	11,580	Note 1	12,770	Note 1	13,620	Note 1	Sheyenne River
Check Total for Drain 40/45, Sheyenne & Buffalo	4,378		7,542		10,281		13,420		17,834		21,759		23,298		24,619		Check Total for Drain 40/45, Sheyenne & Buffalo
Sheyenne River Flow																	Sheyenne River Flow
75% enters at Sheyenne, XS 299	2,260		3,290		4,260		5,420		7,120		8,690		9,580		10,220		75% enters at Sheyenne, XS 299
25% enters at Drain 13, XS 283	750		1,090		1,420		1,800		2,370		2,890		3,190		3,400		25% enters at Drain 13, XS 283

Location/Source	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	Location/Source
XS 415	4,352		10,608		15,994		20,345		27,441		32,921		42,242		57,641		**XS 415**
Oakport Breakout LS, XS 221		1		1		1		1		1		1		1		1	Oakport Breakout LS, XS 221
Drains 3 & 49 Flow Enters		20		45		55		60		65		70		75		95	Drains 3 & 49 Flow Enters
XS 326	4,372		10,653		15,449		20,105		27,506	0	32,991		42,317		57,736		**XS 326**
Drain 10 Flow Enters		200		295		390		415		440		450		535		575	Drain 10 Flow Enters
LOB_RRN breakout LS, XS 2.89		0		0		0		0		0		0		0		0	LOB_RRN breakout LS, XS 2.89
XS 311	4,572		10,948		15,839		20,520		27,946		33,441		42,852		58,311		**XS 311**
XS 303.8	4,572		10,948		15,839		20,520		27,946		33,441		42,852		58,311		**XS 303.8**
Drain 40/45		292		443		565		815		1,195		1,556		1,682		1,786	Drain 40/45
XS 303	4,864		11,391		16,404		21,335		29,141		34,997		44,534		60,097		**XS 303**
Sheyenne River flow		3,260		5,090		4,260		5,420		7,120		8,690		9,580		10,220	Sheyenne River flow
XS 299	7,124		14,681		20,664		27,055		36,261		43,687		54,114		70,317		**XS 299**
Buffalo River flow		1,076		2,719		4,036		5,385		7,149		8,623		8,846		9,213	Buffalo River flow
XS 291	8,200		17,400		24,700		32,440		43,410		52,310		62,960		79,530		**XS 291**
Drain 13/Sheyenne River flow		750		1,090		1,420		1,800		2,370		2,890		3,190		3,400	Drain 13/Sheyenne River flow
XS 283	8,950		18,490		26,120		34,240		45,780		55,200		66,150		82,930		**XS 283**

River	Reach	RS	HEC-RAS Flow File Input- Existing Conditions- Year 25												River	Reach	RS
			2-Year	5-Year	10-Year	20-Year	50-Year	100-Year	200-Year	500-Year							
lob_rrn	overland	2.89	1	1	1	1	1	1	1	1					lob_rrn	overland	2.89
Oakport_SI	Oakport	221	1	1	1	1	1	1	1	1					Oakport_SI	Oakport	221
RRN	Cass_Clay_Fargo	488.68	3139	6160	9555	13729	19709	23757	26164	30016					RRN	Cass_Clay_Fargo	488.68
RRN	Cass_Clay_Fargo	477	3147	6349	9577	13547	19388	23742	26339	31095					RRN	Cass_Clay_Fargo	477
RRN	Cass_Clay_Fargo	476.4	3198	7772	9722	12188	16690	20053	27549	39492					RRN	Cass_Clay_Fargo	476.4
RRN	Cass_Clay_Fargo	470.1	4352	10608	15994	20345	27441	32921	42242	57641					RRN	Cass_Clay_Fargo	470.1
RRN	Cass_Clay_Fargo	326	4372	10653	15449	20405	27506	32991	42317	57736					RRN	Cass_Clay_Fargo	326
RRN	Cass_Clay_Fargo	311	4572	10948	15839	20820	27946	33441	42852	58311					RRN	Cass_Clay_Fargo	311
RRN	Above_Sheyenne	303.8	4572	10948	15839	20820	27946	33441	42852	58311					RRN	Above_Sheyenne	303.8
RRN	Above_Sheyenne	303	4864	11391	16404	21335	29141	34997	44534	60097					RRN	Above_Sheyenne	303
RRN	Sheyenne_R_to_P	299	7124	14681	20664	27055	36261	43687	54114	70317					RRN	Sheyenne_R_to_P	299
RRN	Sheyenne_R_to_P	291	8200	17400	24700	32440	43410	52310	62960	79530					RRN	Sheyenne_R_to_P	291
RRN	Sheyenne_R_to_P	283	8950	18490	26120	34240	45780	55200	66150	82930					RRN	Sheyenne_R_to_P	283

Table B-5
Red River of the North - Fargo-Moorhead Metro Area to Perley
Initial HEC-RAS Discharges (Year 50)

Location/Source	Fargo-Moorhead Metro Study, Phase 3.1 Hydrology																Location/Source
	2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year		
Hickson Gage	2,550		5,366		8,710		12,762		18,543		22,626		24,116		28,246		Hickson Gage
Phase 3.1 Drainage Area Analysis Fargo Gage	3,506		9,161		13,965		18,855		25,764	25,764	31,304		38,787		54,054		Phase 3.1 Drainage Area Analysis Fargo Gage
XS 283	7,890		17,020		24,610		32,650		44,080		53,560		63,590		80,610		XS 283
Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River	4,384		7,859		10,645		13,795		18,316		22,256		24,803		26,376		Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River
Drains 3 & 49	20	0.0054	45	0.0049	60	0.0047	65	0.0041	70	0.0034	75	0.0030	90	0.0031	110	0.0032	Drains 3 & 49
Drain 10	195	0.0403	310	0.0327	405	0.0307	420	0.0264	450	0.0226	465	0.0198	580	0.0206	625	0.0193	Drain 10
Drain 40/45, Sheyenne River & Buffalo River Contributions	4,169		7,504		10,180		13,310		17,796		21,716		24,133		25,841		Drain 40/45, Sheyenne River & Buffalo River Contributions
Buffalo River	903	Note 1	2,413	Note 1	3,684	Note 1	5,004	Note 1	6,738	Note 1	8,190	Note 1	8,456	Note 1	8,861	Note 1	Buffalo River
Drain 40/45	286	0.0591	461	0.0490	586	0.0448	846	0.0529	1,258	0.0611	1,596	0.0684	1,807	0.0642	1,960	0.0601	Drain 40/45
Sheyenne River	2,980	Note 1	4,630	Note 1	5,910	Note 1	7,460	Note 1	9,820	Note 1	11,930	Note 1	13,870	Note 1	15,020	Note 1	Sheyenne River
Check Total for Drain 40/45, Sheyenne & Buffalo	4,169		7,504		10,180		13,310		17,796		21,716		24,132		25,841		Check Total for Drain 40/45, Sheyenne & Buffalo
Sheyenne River Flow																	Sheyenne River Flow
75% enters at Sheyenne, XS 299	2,246		3,479		4,430		5,600		7,370		8,950		10,400		11,270		75% enters at Sheyenne, XS 299
25% enters at Drain 13, XS 283	740		1,160		1,480		1,860		2,450		2,980		3,470		3,750		25% enters at Drain 13, XS 283

Location/Source	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	Location/Source
XS 415	3,506		9,161		13,965		18,855		25,764		31,304		38,787		54,054		**XS 415**
Oakport Breakout LS, XS 221		-1		-1		-1		-1		-1		-1		-1		-1	Oakport Breakout LS, XS 221
Drains 3 & 49 Flow Enters		20		45		60		65		70		75		90		110	Drains 3 & 49 Flow Enters
XS 326	3,526		9,206		14,025		18,920		25,834		31,379		38,877		54,144		**XS 326**
Drain 10 Flow Enters		195		310		405		420		450		465		580		625	Drain 10 Flow Enters
LOB_RRN breakout LS, XS 289		-1		-1		-1		-1		-1		-1		-1		-1	LOB_RRN breakout LS, XS 289
XS 311	3,721		9,516		14,430		19,340		26,284		31,844		39,457		54,769		**XS 311**
XS 303.8	3,721		9,516		14,430		19,340		26,284		31,844		39,457		54,769		**XS 303.8**
Drain 40/45		286		461		586		846		1,258		1,596		1,807		1,960	Drain 40/45
XS 303	4,007		9,977		15,016		20,186		27,542		33,440		41,264		56,729		**XS 303**
Sheyenne River flow		2,240		3,479		4,430		5,600		7,370		8,950		10,400		11,270	Sheyenne River flow
XS 299	6,247		13,447		19,446		25,786		34,892		42,390		51,664		67,999		**XS 299**
Buffalo River flow		903		2,413		3,684		5,004		6,738		8,190		8,456		8,861	Buffalo River flow
XS 291	7,150		15,860		23,130		30,790		41,630		50,580		60,120		76,860		**XS 291**
Drain 13/Sheyenne River flow		740		1,160		1,480		1,860		2,450		2,980		3,470		3,750	Drain 13/Sheyenne River flow
XS 283	7,890		17,020		24,610		32,650		44,080		53,560		63,590		80,610		**XS 283**

River	Reach	RS	HEC-RAS Flow File Input- Existing Conditions- Year 50												River	Reach	RS
			2-Year	5-Year	10-Year	20-Year	50-Year	100-Year	200-Year	500-Year							
lob_rrn	overland	2.89	1	1	1	1	1	1	1	1					lob_rrn	overland	2.89
Oakport_SI	Oakport	221	1	1	1	1	1	1	1	1					Oakport_SI	Oakport	221
RRN	Cass_Clay_Fargo	488.68	2550	5366	8710	12762	18543	22626	24116	28246					RRN	Cass_Clay_Fargo	488.68
RRN	Cass_Clay_Fargo	477	2550	5485	8718	12882	18734	22925	24358	29256					RRN	Cass_Clay_Fargo	477
RRN	Cass_Clay_Fargo	476.4	2548	6358	8774	14434	15609	19019	25243	30927					RRN	Cass_Clay_Fargo	476.4
RRN	Cass_Clay_Fargo	470.1	3506	9161	13965	18855	25764	31304	38787	54034					RRN	Cass_Clay_Fargo	470.1
RRN	Cass_Clay_Fargo	326	3526	9206	14025	18929	25834	31379	38877	54144					RRN	Cass_Clay_Fargo	326
RRN	Cass_Clay_Fargo	311	3721	9516	14430	19340	26284	31844	39457	54769					RRN	Cass_Clay_Fargo	311
RRN	Above_Sheyenne	303.8	3721	9516	14430	19340	26284	31844	39457	54769					RRN	Above_Sheyenne	303.8
RRN	Above_Sheyenne	303	4007	9977	15016	20186	27522	33440	41264	56729					RRN	Above_Sheyenne	303
RRN	Sheyenne_R_to_P	299	6247	13447	19446	25786	34892	42390	51664	67999					RRN	Sheyenne_R_to_P	299
RRN	Sheyenne_R_to_P	291	7150	15860	23130	30790	41630	50580	60120	76860					RRN	Sheyenne_R_to_P	291
RRN	Sheyenne_R_to_P	283	7890	17020	24610	32650	44080	53560	63590	80610					RRN	Sheyenne_R_to_P	283

Table B-6
Red River of the North - Fargo-Moorhead Metro Area to Perley
Initial HEC-RAS Discharges (Year Zero)

Location/Source	Fargo-Moorhead Metro Study, Phase 3.1 Hydrology																Location/Source
	2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year		
Hickson Gage	4,000		7,000		10,500		14,800		21,000		25,000		28,500		32,000		Hickson Gage
Phase 3.1 Drainage Area Analysis_Fargo Gage	3,000		5,930		9,658		13,730		19,806		23,941		26,870		29,978		Phase 3.1 Drainage Area Analysis_Fargo Gage
XS 283	10,380		20,140		27,860		36,050		47,720		57,070		69,040		85,540		XS 283
Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River Contributions	4,780		7,990		10,860		14,030		18,420		22,370		27,840		23,490		Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River Contributions
Drains 3 & 49	50	0.0054	45	0.0049	55	0.0047	65	0.0041	60	0.0034	70	0.0030	75	0.0031	90	0.0032	Drains 3 & 49
Drain 10	195	0.0403	280	0.0327	370	0.0307	395	0.0264	430	0.0226	440	0.0198	475	0.0206	510	0.0193	Drain 10
Drain 40/45, Sheyenne River & Buffalo River Contributions	4,535		7,665		10,435		13,590		17,930		21,860		27,290		23,240		Drain 40/45, Sheyenne River & Buffalo River Contributions
Buffalo River	1,312	Note 1	3,061	Note 1	4,431	Note 1	5,809	Note 1	7,604	Note 1	9,100	Note 1	9,275	Note 1	9,600	Note 1	Buffalo River
Drain 40/45	293	0.0591	424	0.0490	554	0.0448	791	0.0529	1,166	0.0611	1,520	0.0684	1,525	0.0642	1,590	0.0601	Drain 40/45
Sheyenne River	2,950	Note 1	4,180	Note 1	5,450	Note 1	6,990	Note 1	9,160	Note 1	11,240	Note 1	11,490	Note 1	12,050	Note 1	Sheyenne River
Check Total for Drain 40/45, Sheyenne & Buffalo	4,555		7,665		10,435		13,590		17,930		21,860		27,290		23,240		Check Total for Drain 40/45, Sheyenne & Buffalo
Sheyenne River Flow																	Sheyenne River Flow
75% enters at Sheyenne, XS 299	2,210		3,140		4,090		5,240		6,870		8,430		8,620		9,040		75% enters at Sheyenne, XS 299
25% enters at Drain 13, XS 283	740		1,040		1,360		1,750		2,290		2,810		2,870		3,010		25% enters at Drain 13, XS 283

Location/Source	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	Location/Source
XS 415	5,600		9,620		9,655		9,770		9,808		10,941		16,370		26,978		**XS 415**
Oakport Breakout LS, XS 221		1		1		1		1		1		1		1		1	Oakport Breakout LS, XS 221
Drains 3 & 49 Flow Enters		30		45		55		65		60		70		75		90	Drains 3 & 49 Flow Enters
XS 326	5,630		9,665		9,710		9,835		9,868		11,011		16,445		27,068		**XS 326**
Drain 10 Flow Enters		195		280		370		395		430		440		475		510	Drain 10 Flow Enters
LOB_RRN breakout LS, XS 2.89		1		1		1		1		1		1		1		1	LOB_RRN breakout LS, XS 2.89
XS 311	5,825		9,945		10,080		10,230		10,298		11,451		16,920		27,578		**XS 311**
XS 303.8	5,823		9,945		10,080		10,230		10,298		11,451		16,925		27,578		**XS 303.8**
Drain 40/45		293		424		554		791		1,166		1,520		1,525		1,590	Drain 40/45
XS 303	6,118		10,369		10,634		11,021		11,464		12,971		18,445		29,168		**XS 303**
Sheyenne River flow		2,210		3,140		4,090		5,240		6,870		8,430		8,620		9,040	Sheyenne River flow
XS 299	8,328		16,039		22,069		28,491		37,826		45,160		56,895		72,930		**XS 299**
Buffalo River flow		1,312		3,061		4,431		5,809		7,604		9,100		9,275		9,600	Buffalo River flow
XS 291	9,640		19,100		26,500		34,300		45,430		54,260		66,170		82,530		**XS 291**
Drain 13/Sheyenne River flow		740		1,040		1,360		1,750		2,290		2,810		2,870		3,010	Drain 13/Sheyenne River flow
XS 283	10,380		20,140		27,860		36,050		47,720		57,070		69,040		85,540		**XS 283**

River	Reach	RS	HEC-RAS Flow File Input- MN Short 35,000 cfs- Wet Scenario- Year 0														River	Reach	RS
			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year				
lob_rrn	overland	2.89	1		1		1		1		1		1		1		1		lob_rrn	overland	2.89
MN DIV	MN DIV_SHORT UPS	130278	0.1		1050		4078		7955		13176		16991		19252		20125		MN DIV	MN DIV_SHORT UPS	130278
MN DIV	MN DIV_SHORT DOW	127463.*	0.2		2530		7345		12530		19492		23959		29830		34722		MN DIV	MN DIV_SHORT DOW	127463.*
MN DIV_SHORT EXT	MN DIV_SHORT EXT	15966	0.1		880		2667		4775		6316		7694		10578		13577		MN DIV_SHORT EXT	MN DIV_SHORT EXT	15966
Oakport_SI	Oakport	221	1		1		1		1		1		1		1		1		Oakport_SI	Oakport	221
RRN	Upstream_Ext	488.68	4000		7000		10500		14800		21000		25000		28500		32000		RRN	Upstream_Ext	488.68
RRN	Upstream_Ext	477	4021		7227		10537		14618		20566		24472		28720		33177		RRN	Upstream_Ext	477
RRN	Upstream_Ext	476.4	4166		8952		10791		13453		17872		21196		30252		42385		RRN	Upstream_Ext	476.4
RRN	Upstream_Div	474.6	4156		8972		9124		9178		11556		13528		19674		27788		RRN	Upstream_Div	474.6
RRN	Upstream_Div	470.6	4130		8349		8147		9076		11227		13250		19819		28259		RRN	Upstream_Div	470.6
RRN	Upstream_Div	470.1	5606		11251		14533		17725		22084		27632		35625		47103		RRN	Upstream_Div	470.1
RRN	Downstream_Div	467.4	5600		9620		9655		9770		9808		10941		16370		26978		RRN	Downstream_Div	467.4
RRN	Downstream_Div	326	5630		9665		9710		9835		9868		11011		16445		27068		RRN	Downstream_Div	326
RRN	Downstream_Div	311	5825		9945		10080		10230		10298		11451		16920		27578		RRN	Downstream_Div	311
RRN	Above_Sheyenne	303.8	5825		9945		10080		10230		10298		11451		16920		27578		RRN	Above_Sheyenne	303.8
RRN	Above_Sheyenne	303	6118		10369		10634		11021		11464		12971		18445		29168		RRN	Above_Sheyenne	303
RRN	Sheyenne_R_to_P	299	8328		16039		22069		28491		37826		45160		56895		72930		RRN	Sheyenne_R_to_P	299
RRN	Sheyenne_R_to_P	291	9646		19100		26500		34300		45430		54260		66170		82530		RRN	Sheyenne_R_to_P	291
RRN	Sheyenne_R_to_P	283	10380		20140		27860		36050		47720		57070		69040		85540		RRN	Sheyenne_R_to_P	283

Table B-7
Red River of the North - Fargo-Moorhead Metro Area to Perley
Initial HEC-RAS Discharges (Year 25)

Location/Source	Fargo-Moorhead Metro Study, Phase 3.1 Hydrology																Location/Source
	2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year		
Hickson Gage	3,139		6,160		9,555		13,729		19,709		23,757		26,164		30,016		Hickson Gage
Phase 3.1 Drainage Area Analysis Fargo Gage	4,352		9,608		9,647		9,738		9,759		9,896		14,255		22,721		Phase 3.1 Drainage Area Analysis Fargo Gage
XS 283	8,950		18,490		26,120		34,240		45,780		55,200		66,150		82,930		XS 283
Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River Contributions	4,598		7,882		10,726		13,895		18,339		22,279		25,908		25,289		Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River Contributions
Drains 3 & 49	20	0.0054	45	0.0049	55	0.0047	60	0.0041	65	0.0034	70	0.0030	75	0.0031	95	0.0032	Drains 3 & 49
Drain 10	200	0.0403	295	0.0327	390	0.0307	415	0.0264	440	0.0226	450	0.0198	535	0.0206	575	0.0193	Drain 10
Drain 40/45, Sheyenne River & Buffalo River Contributions	4,378		7,542		10,281		13,420		17,834		21,759		25,298		24,619		Drain 40/45, Sheyenne River & Buffalo River Contributions
Buffalo River	1,076	Note 1	2,719	Note 1	4,036	Note 1	5,385	Note 1	7,149	Note 1	8,623	Note 1	8,846	Note 1	9,213	Note 1	Buffalo River
Drain 40/45	292	0.0591	443	0.0490	565	0.0448	815	0.0529	1,195	0.0611	1,556	0.0684	1,682	0.0642	1,786	0.0601	Drain 40/45
Sheyenne River	3,010	Note 1	4,380	Note 1	5,680	Note 1	7,220	Note 1	9,490	Note 1	11,580	Note 1	12,770	Note 1	13,620	Note 1	Sheyenne River
Check Total for Drain 40/45, Sheyenne & Buffalo	4,378		7,542		10,281		13,420		17,834		21,759		25,298		24,619		Check Total for Drain 40/45, Sheyenne & Buffalo
Sheyenne River Flow																	Sheyenne River Flow
75% enters at Sheyenne, XS 299	2,260		3,290		4,260		5,420		7,120		8,690		9,580		10,220		75% enters at Sheyenne, XS 299
25% enters at Drain 13, XS 283	750		1,090		1,420		1,800		2,370		2,890		3,190		3,400		25% enters at Drain 13, XS 283

Location/Source	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	Location/Source
XS 415	4,352		9,608		9,647		9,738		9,759		9,896		14,255		22,721		**XS 415**
Oakport Breakout LS, XS 221		1		1		1		1		1		1		1		1	Oakport Breakout LS, XS 221
Drains 3 & 49 Flow Enters		20		45		55		60		65		70		75		95	Drains 3 & 49 Flow Enters
XS 326	4,372		9,653		9,702		9,798		9,824		9,966		14,330		22,816		**XS 326**
Drain 10 Flow Enters		200		295		390		415		440		450		535		575	Drain 10 Flow Enters
LOB_RRN breakout LS, XS 2.89		1		1		1		1		1		1		1		1	LOB_RRN breakout LS, XS 2.89
XS 311	4,572		9,948		10,092		10,213		10,264		10,416		14,865		23,391		**XS 311**
XS 303.8	4,572		9,948		10,092		10,213		10,264		10,416		14,865		23,391		**XS 303.8**
Drain 40/45		292		443	565		815		1,195		1,556		1,682		1,786		Drain 40/45
XS 303	4,864		10,391		10,657		11,028		11,459		11,972		16,547		25,177		**XS 303**
Sheyenne River flow	2,260	2,260	3,290	3,290	4,260	4,260	5,420	5,420	7,120	7,120	8,690	8,690	9,580	9,580	10,220	10,220	Sheyenne River flow
XS 299	7,124		14,681		20,664		27,055		36,261		43,687		54,114		79,317		**XS 299**
Buffalo River flow	1,076	1,076	2,719	2,719	4,036	4,036	5,385	5,385	7,149	7,149	8,623	8,623	8,846	8,846	9,213	9,213	Buffalo River flow
XS 291	8,200		17,400		24,700		32,440		43,410		52,310		62,960		79,530		**XS 291**
Drain 13/Sheyenne River flow	750	750	1,090	1,090	1,420	1,420	1,800	1,800	2,370	2,370	2,890	2,890	3,190	3,190	3,400	3,400	Drain 13/Sheyenne River flow
XS 283	8,950		18,490		26,120		34,240		45,780		55,200		66,150		82,930		**XS 283**

River	Reach	RS	HEC-RAS Flow File Input- MN Short 35,000 cfs- Year 25														River	Reach	RS
			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year				
lob_rrn	overland	2.89	1		1		1		1		1		1		1		1	lob_rrn	overland	2.89
MN DIV	MN DIV SHORT UPS	130278	0.1		890		3527		9837		21860		13545		14368		21130	MN DIV	MN DIV SHORT UPS	130278
MN DIV	MN DIV SHORT DOW	127463.4	0.2		1090		5747		16607		37682		23625		17987		34920	MN DIV	MN DIV SHORT DOW	127463.4
MN DIV SHORT EXT	MN DIV SHORT EXT	15966	0.1		120		2014		3350		5222		7140		3619		13590	MN DIV SHORT EXT	MN DIV SHORT EXT	15966
Oakport_SI	Oakport	221	1		1		1		1		1		1		1		1	Oakport_SI	Oakport	221
RRN	Upstream_Ext	488.68	3139		6160		9555		13729		19709		23757		26164		30016	RRN	Upstream_Ext	488.68
RRN	Upstream_Ext	477	3147		6349		9877		13547		19288		23242		26339		31093	RRN	Upstream_Ext	477
RRN	Upstream_Ext	476.4	3198		7772		9722		12388		16680		20655		27549		39492	RRN	Upstream_Ext	476.4
RRN	Upstream_Div	474.6	3198		7652		9708		8638		10458		12678		17950		25902	RRN	Upstream_Div	474.6
RRN	Upstream_Div	470.6	3203		7797		7721		8537		10638		12695		18014		26785	RRN	Upstream_Div	470.6
RRN	Upstream_Div	470.1	4352		10488		13380		16595		21619		25741		32623		44051	RRN	Upstream_Div	470.1
RRN	Downstream_Div	467.4	4352		9608		9647		9738		9759		9896		14255		22721	RRN	Downstream_Div	467.4
RRN	Downstream_Div	326	4372		9653		9702		9798		9824		9966		14330		22816	RRN	Downstream_Div	326
RRN	Downstream_Div	311	4572		9948		10092		10213		10264		10416		14865		23391	RRN	Downstream_Div	311
RRN	Above_Sheyenne	303.8	4572		9948		10092		10213		10264		10416		14865		23391	RRN	Above_Sheyenne	303.8
RRN	Above_Sheyenne	303	4864		10391		10657		11028		11459		11972		16547		25177	RRN	Above_Sheyenne	303
RRN	Sheyenne_R_to_P	299	7124		14681		20664		27055		36261		43687		54114		79317	RRN	Sheyenne_R_to_P	299
RRN	Sheyenne_R_to_P	291	8200		17400		24700		32440		43410		52310		62960		79530	RRN	Sheyenne_R_to_P	291
RRN	Sheyenne_R_to_P	283	8950		18490		26120		34240		45780		55200		66150		82930	RRN	Sheyenne_R_to_P	283

Table B-8
Red River of the North - Fargo-Moorhead Metro Area to Perley
Initial HEC-RAS Discharges (Year 50)

Location/Source	Fargo-Moorhead Metro Study, Phase 3.1 Hydrology																Location/Source
	2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year		
Hickson Gage	2,550		5,366		8,710		12,762		18,543		22,626		24,116		28,246		Hickson Gage
Phase 3.1 Drainage Area Analysis_Fargo Gage	5,906		9,139		9,546		9,587		9,658		10,191		12,103		19,922		Phase 3.1 Drainage Area Analysis_Fargo Gage
XS 283	7,890		17,020		24,610		32,650		44,080		53,560		61,590		80,610		XS 283
Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River Contributions	4,384		7,859		10,645		13,795		18,316		22,256		24,803		26,376		Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River Contributions
Drains 3 & 49	20	0.0054	45	0.0049	60	0.0047	65	0.0041	70	0.0034	75	0.0030	90	0.0031	110	0.0032	Drains 3 & 49
Drain 10	195	0.0403	310	0.0327	405	0.0307	420	0.0264	450	0.0226	465	0.0198	580	0.0206	625	0.0193	Drain 10
Drain 40/45, Sheyenne River & Buffalo River Contributions	4,169		7,504		10,180		13,310		17,796		21,716		24,133		25,841		Drain 40/45, Sheyenne River & Buffalo River Contributions
Buffalo River	903	Note 1	2,413	Note 1	3,684	Note 1	5,004	Note 1	6,738	Note 1	8,190	Note 1	8,456	Note 1	8,861	Note 1	Buffalo River
Drain 40/45	286	0.0591	461	0.0390	586	0.0448	846	0.0529	1,238	0.0611	1,596	0.0684	1,807	0.0642	1,960	0.0601	Drain 40/45
Sheyenne River	2,980	Note 1	4,630	Note 1	5,910	Note 1	7,460	Note 1	9,820	Note 1	11,930	Note 1	13,870	Note 1	15,020	Note 1	Sheyenne River
Check Total for Drain 40/45, Sheyenne & Buffalo	4,169		7,504		10,180		13,310		17,796		21,716		24,133		25,841		Check Total for Drain 40/45, Sheyenne & Buffalo
Sheyenne River Flow																	Sheyenne River Flow
75% enters at Sheyenne, XS 299	2,240		3,470		4,430		5,600		7,370		8,950		10,400		11,270		75% enters at Sheyenne, XS 299
25% enters at Drain 13, XS 283	740		1,160		1,480		1,860		2,450		2,980		3,470		3,750		25% enters at Drain 13, XS 283

Location/Source	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	Location/Source
XS 415	5,906		9,139		9,546		9,587		9,658		10,191		12,103		19,922		**XS 415**
Oakport Breakout LS, XS 221		1		1		1		1		1		1		1		1	Oakport Breakout LS, XS 221
Drains 3 & 49 Flow Enters		20		45		60		65		70		75		90		110	Drains 3 & 49 Flow Enters
XS 526	5,526		9,204		9,806		9,932		9,728		10,266		12,193		20,032		**XS 526**
Drain 10 Flow Enters		195		310		405		420		450		465		580		625	Drain 10 Flow Enters
LOB_RRN breakout LS, XS 2.89		1		1		1		1		1		1		1		1	LOB_RRN breakout LS, XS 2.89
XS 311	5,721		9,514		10,011		10,072		10,178		10,731		12,773		20,657		**XS 311**
XS 303.8	5,721		9,514		10,011		10,072		10,178		10,731		12,773		20,657		**XS 303.8**
Drain 40/45		286		461		586		846		1,238		1,596		1,807		1,960	Drain 40/45
XS 303	4,007		9,975		10,597		10,918		11,416		12,227		14,580		22,617		**XS 303**
Sheyenne River flow		2,240		3,470		4,430		5,600		7,370		8,950		10,400		11,270	Sheyenne River flow
XS 299	6,247		13,447		19,446		25,786		34,892		42,990		51,664		67,999		**XS 299**
Buffalo River flow		903		2,413		3,684		5,004		6,738		8,190		8,456		8,861	Buffalo River flow
XS 291	7,150		15,860		23,130		30,790		41,630		50,580		60,120		76,860		**XS 291**
Drain 13/Sheyenne River flow		740		1,160		1,480		1,860		2,450		2,980		3,470		3,750	Drain 13/Sheyenne River flow
XS 283	7,890		17,020		24,610		32,650		44,080		53,560		61,590		80,610		**XS 283**

River	Reach	RS	HEC-RAS Flow File Input- MN Short 35,000 cfs- Year 50												River	Reach	RS
			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year				
lob_rm	overland	2.89	1		1		1		1		1		1		1		lob_rm	overland	2.89
MN DIV	MN DIV SHORT UPS	130278	2.1		2		2963		5963		10724		14206		17842		MN DIV	MN DIV SHORT UPS	130278
MN DIV	MN DIV SHORT DOW	127463.7	2.2		2		4419		9268		16108		21115		26684		MN DIV	MN DIV SHORT DOW	127463.7
MN DIV SHORT EXT	MN DIV SHORT EXT	15966	2.1		2		4456		9285		15927		20847		25966		MN DIV SHORT EXT	MN DIV SHORT EXT	15966
Oakport_SI	Oakport	221	1		1		1		1		1		1		1		Oakport_SI	Oakport	221
RRN	Upstream_Ext	488.68	2550		5366		8710		12762		18543		22626		24116		RRN	Upstream_Ext	488.68
RRN	Upstream_Ext	477	2550		5485		8718		12582		18134		22123		24258		RRN	Upstream_Ext	477
RRN	Upstream_Ext	476.4	2548		6358		8774		11434		15609		19019		25243		RRN	Upstream_Ext	476.4
RRN	Upstream_Div	474.6	2548		6357		8718		8149		10227		13172		16401		RRN	Upstream_Div	474.6
RRN	Upstream_Div	470.6	2548		6443		7823		8049		10014		11910		16401		RRN	Upstream_Div	470.6
RRN	Upstream_Div	470.1	2506		9160		12509		15370		20382		24457		29048		RRN	Upstream_Div	470.1
RRN	Downstream_Div	467.4	2506		9159		9546		9587		9658		10191		12103		RRN	Downstream_Div	467.4
RRN	Downstream_Div	326	3526		9204		9606		9652		9728		10266		12193		RRN	Downstream_Div	326
RRN	Downstream_Div	311	3721		9514		10011		10072		10178		10731		12773		RRN	Downstream_Div	311
RRN	Above_Sheyenne	303.8	3721		9514		10011		10072		10178		10731		12773		RRN	Above_Sheyenne	303.8
RRN	Above_Sheyenne	303	4007		9975		10597		10918		11416		12227		14580		RRN	Above_Sheyenne	303
RRN	Sheyenne_R_to_P	299	6247		13447		19446		25786		34892		42990		51664		RRN	Sheyenne_R_to_P	299
RRN	Sheyenne_R_to_P	291	7150		15860		23130		30790		41630		50580		60120		RRN	Sheyenne_R_to_P	291
RRN	Sheyenne_R_to_P	283	7890		17020		24610		32650		44080		53560		61590		RRN	Sheyenne_R_to_P	283

Table B-9
Red River of the North - Fargo-Moorhead Metro Area to Perley
Initial HEC-RAS Discharges (Year Zero)

Location/Source	Fargo-Moorhead Metro Study, Phase 3.1 Hydrology																Location/Source
	2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year		
Hickson Gage	4,000		7,000		10,500		14,800		21,000		25,900		28,500		32,000		Hickson Gage
Phase 3.1 Drainage Area Analysis Fargo Gage	5,600		9,036		9,717		9,834		9,915		10,524		16,291		26,806		Phase 3.1 Drainage Area Analysis Fargo Gage
XS 283	10,380		20,140		27,860		36,050		47,720		57,070		69,040		85,540		XS 283
Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River Contributions	4,780		7,990		10,860		14,050		18,470		22,370		27,840		33,840		Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River Contributions
Drains 3 & 49	30	0.0054	45	0.0049	55	0.0047	65	0.0041	60	0.0034	70	0.0030	75	0.0031	90	0.0032	Drains 3 & 49
Drain 10	195	0.0403	280	0.0327	370	0.0307	395	0.0264	430	0.0226	440	0.0198	475	0.0266	510	0.0193	Drain 10
Drain 40/45, Sheyenne River & Buffalo River Contributions	4,555		7,665		10,435		13,590		17,940		21,860		26,590		32,240		Drain 40/45, Sheyenne River & Buffalo River Contributions
Buffalo River	1,312	Note 1	3,061	Note 1	4,431	Note 1	5,809	Note 1	7,604	Note 1	9,100	Note 1	9,275	Note 1	9,600	Note 1	Buffalo River
Drain 40/45 -Existing Conditions	293	0.0591	424	0.0490	554	0.0448	791	0.0529	1,166	0.0611	1,520	0.0684	1,525	0.0642	1,590	0.0601	Drain 40/45 -Existing Conditions
Drain 40/45 - With Project Conditions	293		300		300		300		300		300		300		300		Drain 40/45 - With Project Conditions
Sheyenne River	2,570	Note 1	3,049	Note 1	3,239	Note 1	3,447	Note 1	3,688	Note 1	3,750	Note 1	3,999	Note 1	4,123	Note 1	Sheyenne River
Check Total for Drain 40/45, Sheyenne & Buffalo	4,555		7,665		10,435		13,590		17,940		21,860		26,590		32,240		Check Total for Drain 40/45, Sheyenne & Buffalo
Sheyenne River Flow																	Sheyenne River Flow
100% enters at Sheyenne, XS 299	2,570		3,049		3,239		3,447		3,688		3,750		3,999		4,123		100% enters at Sheyenne, XS 299
0% enters at Drain 13, XS 283	0		0		0		0		0		0		0		0		0% enters at Drain 13, XS 283

Location/Source	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	Location/Source
XS 415	5,600		9,616		9,717		9,834		9,915		10,524		16,291		26,806		**XS 415**
Oakport Breakout LS, XS 221		1		1		1		1		1		1		1		1	Oakport Breakout LS, XS 221
Drains 3 & 49 Flow Enters		30		45		55		65		60		70		75		90	Drains 3 & 49 Flow Enters
XS 326	5,630		9,661		9,772		9,899		9,975		10,594		16,366		26,896		**XS 326**
Drain 10 Flow Enters		195		280		370		395		430		440		475		510	Drain 10 Flow Enters
LOB_RRN breakout LS, XS 2.89		1		1		1		1		1		1		1		1	LOB_RRN breakout LS, XS 2.89
XS 311	5,825		9,941		10,142		10,294		10,405		11,034		16,841		27,406		**XS 311**
XS 303.8	5,825		9,941		10,142		10,294		10,405		11,034		16,841		27,406		**XS 303.8**
Drain 40/45		293		300		300		300		300		300		300		300	Drain 40/45
XS 303	6,118		10,241		10,442		10,594		10,705		11,334		17,141		27,706		XS 303
Sheyenne River flow		2,570		3,049		3,239		3,447		3,688		3,750		3,999		4,123	Sheyenne River flow
XS 299	8,688		13,290		13,681		14,041		14,393		15,084		21,140		31,829		**XS 299**
Buffalo River flow		1,312		3,061		4,431		5,809		7,604		9,100		9,275		9,600	Buffalo River flow
XS 291	10,380		20,140		27,860		36,050		47,720		57,070		69,040		85,540		XS 291
Drain 13/Sheyenne River flow		0		0		0		0		0		0		0		0	Drain 13/Sheyenne River flow
XS 283	10,380		20,140		27,860		36,050		47,720		57,070		69,040		85,540		**XS 283**

River	Reach	RS	HEC-RAS Flow File Input- ND East 35,000 cfs- Wet Scenario- Year 0														River	Reach	RS
			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year				
lob_rrn	overland	2.89	1	1419	1	3021	1	6185	1	8648	1	11655	1	13780	1	15801	1	18342	lob_rrn	overland	2.89
ND DIV	ND DIV EAST	190867	1	NO	1864	YES	3448	YES	5868	YES	10086	YES	12746	YES	16458	YES	18902	YES	ND DIV	ND DIV EAST	190867
ND DIV	ND DIV EAST	176762	0	1419	2534	330	5783	2350	12166	3350	19385	3350	24176	3450	29909	3256	34894	2580	ND DIV	ND DIV EAST	176762
ND DIV	ND DIV EAST	148766	125	125	3119	585	8405	1122	13985	1819	22199	2814	27069	2893	32902	2893	37787	2893	ND DIV	ND DIV EAST	148766
ND DIV	ND DIV EAST	124541	125	0	3119	0	8405	0	13985	0	22199	0	27569	500	33363	560	38487	700	ND DIV	ND DIV EAST	124541
ND DIV	ND DIV EAST	80945	125	0	3119	0	8405	0	13985	0	22469	261	28959	1370	34837	1478	40252	1785	ND DIV	ND DIV EAST	80945
ND DIV	ND DIV EAST	71959	125	0	3426	301	9259	854	15570	1585	24868	2408	31821	2882	37545	2708	42971	2699	ND DIV	ND DIV EAST	71959
ND DIV	ND DIV EAST	57083	210	85	3543	123	9422	163	15780	210	25153	285	32176	355	37905	360	43351	380	ND DIV	ND DIV EAST	57083
ND DIV	ND DIV EAST	47124	380	170	3789	246	9748	326	16200	420	25722	570	32886	710	38625	720	44111	760	ND DIV	ND DIV EAST	47124
Oakport_SI	Oakport	221	1		1		1		1		1		1		1		1		Oakport_SI	Oakport	221
RRN	Upstream_Div	488.68	4000		7000		10500		14800		21000		25000		28500		32000		RRN	Upstream_Div	488.68
RRN	Downstream_Div	478.81	4000		5137		7052		9531		10923		12754		13042		13098		RRN	Downstream_Div	478.81
RRN	Downstream_Div	477	4021		5164		7089		8750		10486		11726		12262		14275		RRN	Downstream_Div	477
RRN	Downstream_Div	476.4	4166		7089		7343		7525		7792		8450		13794		23483		RRN	Downstream_Div	476.4
RRN	Downstream_Div	470.6	4163		7266		7566		7483		7552		8172		13035		23454		RRN	Downstream_Div	470.6
RRN	Downstream_Div	470.1	5606		9616		9717		9834		9915		10524		16291		26806		RRN	Downstream_Div	470.1
RRN	Downstream_Div	326	5630		9661		9772		9899		9975		10594		16366		26896		RRN	Downstream_Div	326
RRN	Downstream_Div	311	5825		9941		10142		10294		10405		11034		16841		27406		RRN	Downstream_Div	311
RRN	Above_Sheyenne	303.8	5825		9941		10142		10294		10405		11034		16841		27406		RRN	Above_Sheyenne	303.8
RRN	Above_Sheyenne	303	6118		10241		10442		10594		10705		11334		17141		27706		RRN	Above_Sheyenne	303
RRN	Shey_Upstrm_Div	299	8688		13290		13681		14041		14393		15084		21140		31829		RRN	Shey_Upstrm_Div	299
RRN	Shey_Dwnstrm_Div	292	9068		17079		23439		30241		40116		47978		59763		75940		RRN	Shey_Dwnstrm_Div	292
RRN	Shey_Dwnstrm_Div	291	10380		20140		27860		36050		47720		57070		69040		85540		RRN	Shey_Dwnstrm_Div	291
RRN	Shey_Dwnstrm_Div	283	10380		20140		27860		36050		47720		57070		69040		85540		RRN	Shey_Dwnstrm_Div	283

Table B-10
Red River of the North - Fargo-Moorhead Metro Area to Perley
Initial HEC-RAS Discharges (Year 25)

Location/Source	Fargo-Moorhead Metro Study, Phase 3.1 Hydrology																Location/Source
	2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year		
Hickson Gage	3,139		6,160		9,555		13,729		19,709		23,757		26,164		30,016		Hickson Gage
Phase 3.1 Drainage Area Analysis_Fargo Gage	4,352		9,607		9,690		9,702		9,823		9,974		14,133		22,570		Phase 3.1 Drainage Area Analysis_Fargo Gage
XS 283	8,950		18,490		26,120		34,240		45,780		55,200		66,150		82,930		XS 283
Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River Contributions	4,598		7,843		10,736		13,895		18,379		22,979		25,608		28,289		Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River Contributions
Drains 3 & 49	20	0.0054	45	0.0049	55	0.0047	60	0.0041	65	0.0034	70	0.0030	75	0.0031	95	0.0032	Drains 3 & 49
Drain 10	200	0.0403	295	0.0327	390	0.0307	415	0.0264	440	0.0226	450	0.0198	535	0.0206	575	0.0193	Drain 10
Drain 40/45, Sheyenne River & Buffalo River Contributions	4,378		7,542		10,281		13,420		17,834		21,759		25,298		24,619		Drain 40/45, Sheyenne River & Buffalo River Contributions
Buffalo River	1,076	Note 1	2,719	Note 1	4,036	Note 1	5,385	Note 1	7,149	Note 1	8,623	Note 1	8,846	Note 1	9,213	Note 1	Buffalo River
Drain 40/45 -Existing Conditions	292	0.0391	443	0.0490	565	0.0448	815	0.0529	1,195	0.0611	1,556	0.0684	1,682	0.0642	1,786	0.0601	Drain 40/45 -Existing Conditions
Drain 40/45 - With Project Conditions	292		300		300		300		300		300		300		300		Drain 40/45 - With Project Conditions
Sheyenne River	2,611	Note 1	3,083	Note 1	3,268	Note 1	3,475	Note 1	3,713	Note 1	3,938	Note 1	3,778	Note 1	4,187	Note 1	Sheyenne River
Check Total for Drain 40/45, Sheyenne & Buffalo	4,378		7,542		10,281		13,420		17,834		21,759		25,298		24,619		Check Total for Drain 40/45, Sheyenne & Buffalo
Sheyenne River Flow																	Sheyenne River Flow
100% enters at Sheyenne, XS 299	2,611		3,083		3,268		3,475		3,713		3,938		3,778		4,187		100% enters at Sheyenne, XS 299
0% enters at Drain 13, XS 283	0		0		0		0		0		0		0		0		0% enters at Drain 13, XS 283

Location/Source	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	Location/Source
XS 415	4,352		9,607		9,690		9,702		9,823		9,974		14,133		22,570		XS 415
Oakport Breakout LS, XS 221		1		1		1		1		1		1		1		1	Oakport Breakout LS, XS 221
Drains 3 & 49 Flow Enters		20		45		55		60		65		70		75		95	Drains 3 & 49 Flow Enters
XS 326	4,372		9,652		9,745		9,762		9,888		10,044		14,208		22,665		XS 326
Drain 10 Flow Enters		200		295		390		415		440		450		535		575	Drain 10 Flow Enters
LOB_RRN breakout LS, XS 2.89		1		1		1		1		1		1		1		1	LOB_RRN breakout LS, XS 2.89
XS 311	4,572		9,947		10,135		10,177		10,328		10,494		14,743		23,240		XS 311
XS 303.8	4,572		9,947		10,135		10,177		10,328		10,494		14,743		23,240		XS 303.8
Drain 40/45		292		300		300		300		300		300		300		300	Drain 40/45
XS 303	4,864		10,247		10,435		10,477		10,628		10,794		15,043		23,540		XS 303
Sheyenne River flow		2,611		3,083		3,268		3,475		3,713		3,938		3,778		4,187	Sheyenne River flow
XS 299	7,475		13,330		13,703		13,952		14,341		14,732		18,821		27,727		XS 299
Buffalo River flow		1,076		2,719		4,036		5,385		7,149		8,623		8,846		9,213	Buffalo River flow
XS 291	8,950		18,490		26,120		34,240		45,780		55,200		66,150		82,930		XS 291
Drain 13/Sheyenne River flow		0		0		0		0		0		0		0		0	Drain 13/Sheyenne River flow
XS 283	8,950		18,490		26,120		34,240		45,780		55,200		66,150		82,930		XS 283

River	Reach	RS	HEC-RAS Flow File Input- ND East 35,000 cfs- Year 25														River	Reach	RS
			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year				
lob_nn	overland	2.89	0	1148	0	2691	0	5658	0	8957	0	10980	0	13134	0	14577	0	17264	lob_nn	overland	2.89
ND DIV	ND DIV EAST	190867	0	50	668	115	2366	145	4036	175	6045	210	12163	215	13182	215	20127	215	ND DIV	ND DIV EAST	190867
ND DIV	ND DIV EAST	176762	0	1148	1001	2350	5704	2750	10643	3350	17618	3350	27047	3150	28109	3350	35071	3750	ND DIV	ND DIV EAST	176762
ND DIV	ND DIV EAST	148766	144	144	1669	668	6921	1217	12355	1912	20511	2893	29840	2893	31060	2893	37964	2893	ND DIV	ND DIV EAST	148766
ND DIV	ND DIV EAST	124541	144	0	1669	0	6921	0	12355	0	20546	35	29423	585	31887	885	39064	1100	ND DIV	ND DIV EAST	124541
ND DIV	ND DIV EAST	80945	144	0	1669	0	6921	0	12373	18	20871	325	27736	1361	34123	2234	41951	2897	ND DIV	ND DIV EAST	80945
ND DIV	ND DIV EAST	71959	144	0	3057	388	7874	953	14246	1673	23308	2534	30758	2964	37781	3159	44756	2755	ND DIV	ND DIV EAST	71959
ND DIV	ND DIV EAST	57083	229	85	2185	128	8043	169	14465	219	23500	295	31115	365	37681	401	45134	428	ND DIV	ND DIV EAST	57083
ND DIV	ND DIV EAST	47124	395	170	2441	256	8381	338	14903	438	24290	590	31845	730	38483	802	45990	856	ND DIV	ND DIV EAST	47124
Oakport_SI	Oakport	221	1		1		1		1		1		1		1		1		Oakport_SI	Oakport	221
RRN	Upstream_Div	488.68	3139		6160		9555		13729		19709		23757		26164		30016		RRN	Upstream_Div	488.68
RRN	Downstream_Div	478.81	3139		5806		7749		8793		10771		11594		10767		9859		RRN	Downstream_Div	478.81
RRN	Downstream_Div	477	3147		5689		7181		8611		10360		11079		10457		10936		RRN	Downstream_Div	477
RRN	Downstream_Div	476.4	3196		7142		7326		7452		7692		7892		11667		19535		RRN	Downstream_Div	476.4
RRN	Downstream_Div	470.6	3203		7257		7339		7351		7472		7622		11781		20218		RRN	Downstream_Div	470.6
RRN	Downstream_Div	470.1	3352		9607		9690		9702		9823		9974		14133		22570		RRN	Downstream_Div	470.1
RRN	Downstream_Div	326	4372		9652		9745		9762		9888		10044		14208		22665		RRN	Downstream_Div	326
RRN	Downstream_Div	311	4572		9947		10135		10177		10328		10494		14743		23240		RRN	Downstream_Div	311
RRN	Above_Sheyenne	303.8	4572		9947		10135		10177		10328		10494		14743		23240		RRN	Above_Sheyenne	303.8
RRN	Above_Sheyenne	303	4864		10247		10435		10477		10628		10794		15043		23540		RRN	Above_Sheyenne	303
RRN	Shey_Upstrm_Div	299	7475		13330		13703		13952		14341		14732		18821		27727		RRN	Shey_Upstrm_Div	299
RRN	Shey_Dwnstrm_Div	292	7874		15771		23984		28455		39651		46577		57304		73717		RRN	Shey_Dwnstrm_Div	292
RRN	Shey_Dwnstrm_Div	291	8950		18490		26120		34240		45780		55200		66150		82930		RRN	Shey_Dwnstrm_Div	291
RRN	Shey_Dwnstrm_Div	283	8950		18490		26120		34240		45780		55200		66150		82930		RRN	Shey_Dwnstrm_Div	283

Table B-11
Red River of the North - Fargo-Moorhead Metro Area to Perley
Initial HEC-RAS Discharges (Year 50)

Location/Source	Fargo-Moorhead Metro Study, Phase 3.1 Hydrology																Location/Source
	2-Year		5-Year		10-Year		25-Year		50-Year		100-Year		200-Year		500-Year		
Hickson Gage	2,550		5,366		8,710		12,762		18,543		22,626		24,116		28,246		Hickson Gage
Phase 3.1 Drainage Area Analysis Fargo Gage	3,506		9,135		9,550		9,581		9,637		10,287		12,013		19,869		Phase 3.1 Drainage Area Analysis Fargo Gage
XS 283	7,890		17,020		24,610		32,650		44,080		53,560		63,590		80,610		XS 283
Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River Contributions	4,384		7,859		10,645		13,795		18,316		22,356		24,803		36,576		Drains 3, 49, 10, 40/45, Sheyenne River & Buffalo River Contributions
Drains 3 & 49	20	0.0054	45	0.0049	60	0.0047	65	0.0041	70	0.0034	75	0.0030	90	0.0031	110	0.0032	Drains 3 & 49
Drain 10	195	0.0403	310	0.0327	405	0.0307	420	0.0264	450	0.0226	465	0.0198	580	0.0206	625	0.0193	Drain 10
Drain 40/45, Sheyenne River & Buffalo River Contributions	4,169		7,504		10,180		13,310		17,796		21,816		24,133		25,841		Drain 40/45, Sheyenne River & Buffalo River Contributions
Buffalo River	903	Note 1	2,413	Note 1	3,684	Note 1	5,004	Note 1	6,738	Note 1	8,190	Note 1	8,456	Note 1	8,861	Note 1	Buffalo River
Drain 40/45 -Existing Conditions	286	0.0591	461	0.0490	586	0.0448	846	0.0529	1,238	0.0611	1,596	0.0684	1,807	0.0642	1,960	0.0601	Drain 40/45 -Existing Conditions
Drain 40/45 - With Project Conditions	286		300		300		300		300		300		300		300		Drain 40/45 - With Project Conditions
Sheyenne River	2,590	Note 1	3,121	Note 1	3,297	Note 1	3,499	Note 1	3,719	Note 1	3,761	Note 1	3,784	Note 1	4,213	Note 1	Sheyenne River
Check Total for Drain 40/45, Sheyenne & Buffalo	4,169		7,504		10,180		13,310		17,796		21,816		24,133		25,841		Check Total for Drain 40/45, Sheyenne & Buffalo
Sheyenne River Flow																	Sheyenne River Flow
100% enters at Sheyenne, XS 299	2,590		3,121		3,297		3,499		3,719		3,761		3,784		4,213		100% enters at Sheyenne, XS 299
0% enters at Drain 13, XS 283	0		0		0		0		0		0		0		0		0% enters at Drain 13, XS 283

Location/Source	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	RRN	Inflow or Breakout	Location/Source
XS 415	3,506		9,135		9,550		9,581		9,637		10,287		12,013		19,869		XS 415
Oakport Breakout LS, XS 221		1		1		1		1		1		1		1		1	Oakport Breakout LS, XS 221
Drains 3 & 49 Flow Enters		20		45		60		65		70		75		90		110	Drains 3 & 49 Flow Enters
XS 326	3,526		9,180		9,610		9,646		9,707		10,362		12,103		19,979		XS 326
Drain 10 Flow Enters		195		310		405		420		450		465		580		625	Drain 10 Flow Enters
LOB_RRN breakout LS, XS 2.89		1		1		1		1		1				1		1	LOB_RRN breakout LS, XS 2.89
XS 311	3,721		9,490		10,015		10,066		10,157		10,827		12,683		20,604		XS 311
XS 303.8	3,721		9,490		10,015		10,066		10,157		10,827		12,683		20,604		XS 303.8
Drain 40/45		286		300		300		300		300		300		300		300	Drain 40/45
XS 303	4,007		9,790		10,315		10,366		10,457		11,127		12,983		20,904		XS 303
Sheyenne River flow		2,590		3,121		3,297		3,499		3,719		3,761		3,784		4,213	Sheyenne River flow
XS 299	6,597		12,911		13,612		13,865		14,176		14,888		16,767		25,117		XS 299
Buffalo River flow		903		2,413	1,684		5,004		6,738		8,190		8,456		8,861		Buffalo River flow
XS 291	7,890		17,020		24,610		32,650		44,080		53,560		63,590		80,610		XS 291
Drain 13/Sheyenne River flow		0		0	0		0		0		0		0		0		Drain 13/Sheyenne River flow
XS 283	7,890		17,020		24,610		32,650		44,080		53,560		63,590		80,610		XS 283

River	Reach	RS	HEC-RAS Flow File Input- ND East 35,000 cfs- Year 50														River	Reach	RS
			2-Year		5-Year		10-Year		25-Year		50-Year		100-Year		200-Year		500-Year				
lob_rrn	overland	2.89	1	957	1	2375	1	5145	1	7520	1	10367	1	12545	1	13450	1	10299	lob_rrn	overland	2.89
ND DIV	ND DIV EAST	190867	0	NO	1	YES	1180	YES	4104	YES	8110	YES	16832	YES	15674	YES	20216	YES	ND DIV	ND DIV EAST	190867
ND DIV	ND DIV EAST	176762	0	987	26	3350	4115	2350	9774	3450	16127	3350	23017	3450	26774	3350	34165	3580	ND DIV	ND DIV EAST	176762
ND DIV	ND DIV EAST	148766	135	135	796	770	5733	1318	11293	2019	19020	2893	23916	2893	29967	2893	37958	2893	ND DIV	ND DIV EAST	148766
ND DIV	ND DIV EAST	124541	135	0	796	0	5733	0	11293	0	19140	120	24583	975	30927	1160	38508	1450	ND DIV	ND DIV EAST	124541
ND DIV	ND DIV EAST	80945	135	0	796	0	5733	0	11353	40	19660	520	26371	1786	33846	3019	42289	3781	ND DIV	ND DIV EAST	80945
ND DIV	ND DIV EAST	71959	135	0	1288	492	6789	1056	13100	1767	22249	2588	29357	2986	37059	3213	45213	2924	ND DIV	ND DIV EAST	71959
ND DIV	ND DIV EAST	57083	270	85	1424	136	6964	175	13327	227	22554	306	29732	375	37495	436	45886	473	ND DIV	ND DIV EAST	57083
ND DIV	ND DIV EAST	47124	390	170	1696	272	7814	350	13701	454	23166	612	30482	750	38367	872	46632	946	ND DIV	ND DIV EAST	47124
Oakport_SI	Oakport	221	1		1		1		1		1		1		1		1		Oakport_SI	Oakport	221
RRN	Upstream_Div	488.68	2550		5366		8710		12762		18543		22626		24116		28246		RRN	Upstream_Div	488.68
RRN	Downstream_Div	478.81	2550		5365		7130		8658		10433		11804		8442		8030		RRN	Downstream_Div	478.81
RRN	Downstream_Div	477	2550		5484		7138		8478		10024		11301		8584		9020		RRN	Downstream_Div	477
RRN	Downstream_Div	476.4	2548		6357		7194		7130		7499		8197		9569		16711		RRN	Downstream_Div	476.4
RRN	Downstream_Div	470.6	2548		6443		7199		7230		7286		7915		9662		17417		RRN	Downstream_Div	470.6
RRN	Downstream_Div	470.1	3526		9135		9550		9581		9637		10287		12013		19869		RRN	Downstream_Div	470.1
RRN	Downstream_Div	326	3526		9180		9610		9646		9707		10362		12103		19979		RRN	Downstream_Div	326
RRN	Downstream_Div	311	3721		9490		10015		10066		10157		10827		12683		20604		RRN	Downstream_Div	311
RRN	Above_Sheyenne	303.8	3721		9490		10015		10066		10157		10827		12683		20604		RRN	Above_Sheyenne	303.8
RRN	Above_Sheyenne	303	4007		9790		10315		10366		10457		11127		12983		20904		RRN	Above_Sheyenne	303
RRN	Shey_Upstrm_Div	299	6597		12911		13612		13865		14176		14888		16767		25117		RRN	Shey_Upstrm_Div	299
RRN	Shey_Dwnstrm_Div	292	6087		14607		20926		27646		37342		45370		55134		71749		RRN	Shey_Dwnstrm_Div	292
RRN	Shey_Dwnstrm_Div	291	7890		17020		24610		32650		44080		53560		63590		80610		RRN	Shey_Dwnstrm_Div	291
RRN	Shey_Dwnstrm_Div	283	7890		17020		24610		32650		44080		53560		63590		80610		RRN	Shey_Dwnstrm_Div	283

Table B-12

LAYOUT OF HEC-RAS MODELING PLAN FILES

Scenario	Capacity	HEC-RAS Project Filename	HEC-RAS Plan Filename	HEC-RAS Project Description	HEC-RAS Plan Description
Year 0	Existing	RRNandWRRND_Existing.prj	RRNandWRRND_Existing.p19	RRNandWRRND_2009Calibration_20100630	RRNMain_P3.1_Exist_HighQ_Year0_20100630
		RRNandWRRND_Existing.prj	RRNandWRRND_Existing.p20	RRNandWRRND_2009Calibration_20100630	RRNMain_P3.1_Exist_lowQ_Year0_20100630
Year 25	Existing	RRNandWRRND_Existing.prj	RRNandWRRND_Existing.p21	RRNandWRRND_2009Calibration_20100630	RRNMain_P3.1_Exist_HighQ_Year25_20100630
Year 50	Existing	RRNandWRRND_Existing.prj	RRNandWRRND_Existing.p22	RRNandWRRND_2009Calibration_20100630	RRNMain_P3.1_Exist_HighQ_Year50_20100630
Year 0	Existing	RRNandWRRND_Existing.prj	RRNandWRRND_Existing.p23	RRNandWRRND_2009Calibration_20100702	RRNMain_P3.1_Exist_1000_Year0_20100702
Year 0	Existing	RRNandWRRND_Existing.prj	RRNandWRRND_Existing.p24	RRNandWRRND_2009Calibration_20100702	RRNMain_P3.1_Exist_10000_Year0_20100702
Year 0	MN 35K	RRNandMNDivShort35kP3_1YR0.prj	RRNandMNDivShort35kP3_1YR0.p03	RRN-MNDivShort35k-P3.1-YR0-20100811	RRN-MNDivShrt35k-P3.1-HighQ-YR0-20100811
		RRNandMNDivShort35kP3_1YR0.prj	RRNandMNDivShort35kP3_1YR0.p04	RRN-MNDivShort35k-P3.1-YR0-20100811	RRN-MNDivShrt35k-P3.1-LowQ-YR0-20100702
Year 25	MN 35K	RRNandMNDivShort35kP3_1YR25.prj	RRNandMNDivShort35kP3_1YR25.p03	RRN-MNDivShort35k-P3.1-YR25-20100811	RRN-MNDivShrt35k-P3.1-HighQ-Y25-20100811
		RRNandMNDivShort35kP3_1YR25.prj	RRNandMNDivShort35kP3_1YR25.p04	RRN-MNDivShort35k-P3.1-YR25-20100811	RRN-MNDivShrt35k-P3.1-LowQ-Y25-20100702
Year 50	MN 35K	RRNandMNDivShort35kP3_1YR50.prj	RRNandMNDivShort35kP3_1YR50.p03	RRN-MNDivShort35k-P3.1-YR50-20100811	RRN-MNDivShrt35k-P3.1-HighQ-Y50-20100811
		RRNandMNDivShort35kP3_1YR50.prj	RRNandMNDivShort35kP3_1YR50.p04	RRN-MNDivShort35k-P3.1-YR50-20100811	RRN-MNDivShrt35k-P3.1-LowQ-Y50-20100702
Year 0	ND 35K	RRNandNDDivEast35kP3_1YR0.prj	RRNandNDDivEast35kP3_1YR0.p03	RRN-NDDivEast35k-P3.1-YR0-20100709	RRN-NDDivEast35k-P3.1-HighQ-YR0-20100709
		RRNandNDDivEast35kP3_1YR0.prj	RRNandNDDivEast35kP3_1YR0.p04	RRN-NDDivEast35k-P3.1-YR0-20100709	RRN-NDDivEast35k-P3.1-LowQ-YR0-20100709
Year 25	ND 35K	RRNandNDDivEast35kP3_1YR25.prj	RRNandNDDivEast35kP3_1YR25.p03	RRN-NDDivEast35k-P3.1-YR25-20100709	RRN-NDDivEast35k-P3.1-HighQYR25-20100709
		RRNandNDDivEast35kP3_1YR25.prj	RRNandNDDivEast35kP3_1YR25.p04	RRN-NDDivEast35k-P3.1-YR25-20100709	RRN-NDDivEast35k-P3.1-LowQ-YR25-20100709
Year 50	ND 35K	RRNandNDDivEast35kP3_1YR50.prj	RRNandNDDivEast35kP3_1YR50.p03	RRN-NDDivEast35k-P3.1-YR50-20100709	RRN-NDDivEast35k-P3.1-HighQYR50-20100709
		RRNandNDDivEast35kP3_1YR50.prj	RRNandNDDivEast35kP3_1YR50.p04	RRN-NDDivEast35k-P3.1-YR50-20100709	RRN-NDDivEast35k-P3.1-LowQ-YR50-20100709

Table B13

Fargo Moorhead Metro Study - Phase 3.1
Reference, Wet - Year Zero, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
279	403.65	832.3	10380	863.05	20140	871.03	27860	874.18	36050	875.89	47720	877.03	57070	877.85	69040	878.81	85540	879.53
280	404.98	836.6	10380	863.75	20140	871.81	27860	874.78	36050	876.6	47720	877.78	57070	878.66	69040	879.62	85540	880.37
281	406.51	834.9	10380	864.68	20140	872.74	27860	875.86	36050	877.71	47720	878.82	57070	879.66	69040	880.42	85540	881.28
282	410.21	838	10380	866.68	20140	874.62	27860	877.64	36050	879.39	47720	880.1	57070	880.81	69040	881.67	85540	882.67
283	413.23	837.6	10380	868.04	20140	876	27860	879	36050	880.5	47720	881	57070	881.76	69040	882.67	85540	883.75
284	414.96	842.7	9640	868.81	19100	876.99	26500	880.15	34300	881.42	45430	882.01	54260	882.77	66170	883.71	82530	884.82
285	415.38	841.4	9640	869	19100	877.22	26500	880.37	34300	881.61	45430	882.29	54260	883	66170	883.95	82530	885.08
286	415.84	843	9640	869.12	19100	877.29	26500	880.42	34300	881.67	45430	882.39	54260	883.15	66170	884.2	82530	885.35
287	415.90	844.1	9640	869.17	19100	877.37	26500	880.55	34300	881.87	45430	882.54	54260	883.27	66170	884.16	82530	885.41
288 22nd St SE or Cass Co. #34 / 170th Ave NW or Clay Co.36																		
289	415.91	844.1	9640	869.17	19100	877.36	26500	880.54	34300	881.89	45430	882.57	54260	883.47	66170	884.65	82530	885.65
290	415.93	837.5	9640	869.19	19100	877.38	26500	880.55	34300	881.9	45430	882.77	54260	883.58	66170	884.65	82530	885.69
291	417.10	840.8	9640	869.62	19100	877.88	26500	881.21	34300	882.69	45430	883.74	54260	884.6	66170	885.4	82530	886.31
292	418.16	844.4	8328	870.05	16039	878.37	22069	881.72	28491	883.21	37826	884.38	45160	885.2	56895	885.89	72930	886.86
293	419.92	844.7	8328	870.86	16039	879.19	22069	882.39	28491	883.83	37826	885.06	45160	885.67	56895	886.43	72930	887.43
294	420.32	843.8	8328	871.12	16039	879.42	22069	882.67	28491	884.16	37826	885.46	45160	886.16	56895	886.94	72930	887.88
295	421.93	845.3	8328	872.31	16039	880.5	22069	883.55	28491	884.92	37826	886.22	45160	886.93	56895	887.74	72930	888.51
296	422.69	847.4	8328	872.64	16039	880.82	22069	883.82	28491	885.15	37826	886.43	45160	887.18	56895	888.07	72930	888.84
297	423.95	849.5	8328	873.19	16039	881.43	22069	884.56	28491	886.06	37826	887.56	45160	888.33	56895	889.02	72930	889.9
298	425.51	848.7	8328	873.93	16039	882.17	22069	885.27	28491	886.79	37826	888.31	45160	889.05	56895	889.47	72930	890.41
299	426.87	851.1	8328	874.63	16039	882.84	22069	885.93	28491	887.53	37826	889.13	45160	889.67	56895	890.05	72930	890.98
300	428.06	849.6	6119	875.21	12900	883.46	17979.56	886.54	21314.4	888.12	25302.55	889.73	29304.55	890.39	37201.99	890.46	54993.26	891.52
301	428.74	850.3	6119	875.52	12900	883.81	17979.56	886.96	21314.4	888.55	25302.55	890.11	29304.55	890.81	37201.99	891.04	54993.26	891.96
302	430.23	852	6119	876.15	12900	884.41	17979.56	887.7	21314.4	889.32	25302.55	890.92	29304.55	891.71	37201.99	891.64	54993.26	892.76
303	431.94	853.5	6119	877	12900	885.24	17979.56	888.64	21640.56	890.21	29647.61	891.84	36509.41	892.75	41972.05	893.21	59204.82	893.95
303.4	432.07	853.42	5826	877.05	12476	885.32	17425.56	888.79	20743.93	890.34	28331.87	891.9	34610.68	892.83	40178.12	893.43	57406.94	894.24
303.5	432.10	853.46	5826	877.09	12476	885.36	17425.56	888.83	20577.2	890.39	27770.36	891.99	33998.95	892.94	39724.64	893.55	56999.13	894.48
303.8	432.53	854.5	5826	877.28	12476	885.54	17425.56	889.03	20180.08	890.64	26103.22	892.41	32091.27	893.39	38207.77	894.06	55472.32	895.23
304	432.84	855.5	5827	877.38	12477	885.65	17226.11	889.12	19571.74	890.72	21355.76	892.49	24669.57	893.49	26841.65	894.17	38394.39	895.4
305	433.26	854.4	5827	877.57	12477	885.85	17226.11	889.32	19564.56	890.88	22163.24	892.6	21996.86	893.59	22441.65	894.27	34116.52	895.54
306	433.35	855	5827	877.65	12477	885.95	17226.11	889.46	19562.85	891.02	22355.76	892.63	21360.11	893.61	21372.79	894.29	32772.67	895.57
307 76th Ave. No. (Fargo) / 90th Ave NW (Moorhead) Bridge																		
308	433.37	855	5827	877.65	12477	885.96	17226.11	889.47	19562.85	891.04	22355.76	892.68	21360.11	893.66	21372.79	894.32	32772.67	895.63
309	433.83	854.5	5827	877.81	12477	886.15	17226.11	889.69	19562.85	891.27	22355.76	892.88	21360.11	893.8	21372.79	894.45	32772.67	895.86
310	434.61	852.8	5827	878.09	12477	886.4	17226.11	889.97	19562.85	891.54	22355.76	893.14	21360.11	893.97	21372.79	894.59	32772.67	896.08
310.5	435.70	855	5827	878.49	12477	886.79	17226.11	890.38	19562.85	892.03	22355.76	893.61	21360.11	894.33	21372.79	894.91	32772.67	896.59
311	436.34	855	5827	878.74	12477	887.07	16711.44	890.71	19538.96	892.4	21653.78	893.98	22745.29	894.64	24702.53	895.2	33679.75	896.95
312	437.05	863.83	5632	879.01	12197	887.39	16341.44	891.07	19143.96	892.79	20968.85	894.36	23932.2	895.01	11766.78	895.56	37943.85	897.32
313	437.95	856	5631	879.39	12196	887.83	16540.89	891.56	19752.3	893.35	25971.24	894.68	29726.99	895.6	35593.65	895.82	49247.58	898.23
314	438.76	857	5631	879.76	12196	888.23	16540.89	891.98	19752.3	893.84	25971.24	895.51	29726.99	896.31	35593.65	896.79	49247.58	899.21
315	439.05	856	5631	879.86	12196	888.35	16540.89	892.12	19752.3	894.02	25971.24	895.75	29726.99	896.59	35593.65	897.16	49247.58	899.41
316	439.46	855.1	5631	880.04	12196	888.59	16540.89	892.42	19752.3	894.37	23378.22	896.29	24229.16	897.25	24124.94	898.08	26991.72	900.38
317 40th Ave. No. (Fargo) / Wall Street NW (Moorhead) Bridge																		
318	439.47	855.1	5631	880.06	12196	888.6	16540.89	892.42	19752.3	894.37	23378.22	896.29	24229.16	897.28	24124.94	898.12	26991.72	900.4
319	439.50	857	5631	880.07	12196	888.66	16540.89	892.49	19752.3	894.44	23378.22	896.38	24229.16	897.36	24124.94	898.23	26991.72	900.45
320	439.66	856	5631	880.13	12196	888.73	16540.89	892.56	19752.3	894.51	23378.22	896.44	24229.16	897.45	24124.94	898.29	26991.72	900.49
321	440.31	858	5631	880.37	12196	889.02	16540.89	892.88	19752.3	894.86	23378.22	896.83	24229.16	897.75	24124.94	898.54	26991.72	900.68
322	440.32	858.1	5631	880.36	12196	889.01	16540.89	892.87	19752.3	894.86	23378.22	896.83	24229.16	897.75	24124.94	898.53	26991.72	900.68
323 Broadway Street No. (Fargo) / Broadway Street NW (Moorhead) Bridge																		
324	440.33	858.1	5631	880.41	12196	889.12	16540.89	892.94	19752.3	894.91	23378.22	896.87	24229.16	897.78	24124.94	898.56	26991.72	900.7
325	440.34	857	5631	880.41	12196	889.14	16540.89	892.95	19752.3	894.92	23378.22	896.88	24229.16	897.78	24124.94	898.56	26991.72	900.7
326	440.54	856	5631	880.46	12196	889.19	16540.89	893.01	19752.3	894.99	23378.22	896.97	24229.16	897.86	24124.94	898.63	26991.72	900.74
327	441.53	859	5601	880.85	12151	889.47	16485.89	893.26	19687.3	895.25	23292.36	897.25	24208.3	898.11	24131.7	898.84	26869.85	900.86
328	442.14	858	5601	881.05	12151	889.62	16658.77	893.41	20448.98	895.42	24794.7	897.43	29256.02	898.28	33160.77	899	39596.47	900.94
329	442.45	857	5601	881.12	12151	889.64	16989.04	893.42	21904.67	895.43	28860.41	897.44	34010.02	898.31	43641.26	899.03	56214.89	900.97
329.5	442.93	858.06	5600	881.26	12150	889.75	17000	893.51	22000	895.52	29300	897.55	34700	898.37	46200	899.11	61700	901.03
330	443.69	858	5600	881.49	12150	889.95	17000	893.67	22000	895.66	29300	897.73	34700	898.58	46200	899.29	61700	901.2
331	443.99	859	5600	881.58	12150	890.06	17000	893.8	22000	895.84	29300	897.97	34700	898.7	46200	899.46	61700	901.36
332	444.33	856	5600	881.62	12150	890.1	17000	893.84	22000	895.89	29300	898.03	34700	898.74	46200	899.52	61700	901.42
333	444.92	859	5600	881.69	12150	890.14	17000	893.86	22000	895.91	29300	898.04	34700	898.71	46200	899.55	61700	901.47
334	445.18	860	5600	881.78	12150	890.22	17000	893.95	22000	896	29300	898.13	34700	898.81	46200	899.72	61700	901.63
335	445.31	860	5600	881.85	12150	890.33	17000	894.09	22000	896.21	29300	898.45	34700	899.24	46200	899.94	61700	901.81
336	445.36	859	5600	881.85	12150	890.34	17000	894.11	22000	896.23	29300	898.47	34700	899.24	46200	899.96	61700	901.83
336.5	445.76	857.69	5600	881.89	12150	890.36	17000	894.14	22000	896.25	29300	898.52	34700	899.3	46200	900.01	61700	901.89
337	446.09	860	5600	881.91	12150	890.36	17000	894.13	22000	896.25	29300	898.5	34700	899.28	46200	900	61700	901.86
338	446.29	861	5600	881.98	12150	890.39	17000	894.14	22000	896.26	29300	898.5	34700	899.28	46200	900.04	61700	901.94
338.5	446.70	860.9	5600	882.13	12150	890.53	17000	894.29	22000	896.44	29300	898.72	34700	899.53	46200	900.26	61700	902.17
339	446.76	861	5600	882.16	12150	890.55	17000	894.3	22000	896.45	29300	898.76	34700	899.59	46200	900.37	61700	902.27
340	447.04	860	5600	882.25	12150	890.66	17000	894.46	22000	896.67	29300	899.08	34700	899.97	46200	900.77	61700	902.66
341	447.24	859	5600	882.33	12150	890.72	17000	894.5	22000	896.71	29300	899.1	34700	900.05	46200	900.83	61700	902.72
342	447.42	861	5600	882.44	12150	890.8	17000	894.58	22000	896.8	29300	899.21	34700	900.19	46200	901.03	61700	902.92
342.5	447.78	856.99	5600	882.6	12150	890.95	17000	894.74	22000	897	29300	899.49	34700	900.38	46200	901.32	61700	903.24
343	448.13	861	5600	882.72	12150	891.06	17000	894.85	22000	897.13	29300	899.63	34700	900.58	46200	901.53	61700	903.48
344	448.36	859	5600	882.83	12150	891.18	17000	895.02	22000	897.36	29300	899.95	34700	900.89	46200	901.99	61700	903.93
345	448.89	865.3	5600	883.14	12150	891.39	17000	895.2	22000	897.55	29300	900.17	34700	901.15	46200	902.27	61700	904.23
348	448.90	864.3	5600	883.13	12150	891.36	17000	895.16	22000	897.5	29300	900.1	34700	901.08	46200	902.2	61700	904.2
348.4 North Dam																		
348.8	448.90	864.3	5600	883.2	12150	891.42	17000	895.21	22000	897.57	29300	900.17	34700	901.14	46200	902.25	61700	904.23
351	448.91	861	5600	883.21	12150	891.42	17000	895.21	22000	897.57	29300	900.17	34700	901.14	46200	902.26	61700	904.24
351.4	449.06	861	5600	883.3	12150	891.58	17000	895.41	22000	897.81	29300	900.48	34700	901.48	46200	902.73	61700	904.67
351.5 12th Ave. No. (Fargo) / 15th Ave No. (Moorhead) Bridge																		
351.6	449.07	861	5600	883.31	12150	891.71	17000	895.51	22000	897.88	29300	900.53	34700	901.56	46200	902.86	61700	904.8
352	449.09	862	5600	883.32	12150	891.73	17000	895.54	22000	897.92	29300	900.58	34700	901.59	46200	902.86	61700	904.8
353	449.28	863	5600	883.41	12150	891.83	17000	895.67	22000	898.09	29300	900.8	34700	901.83	46200	903.19	61700	905.16
353.5	449.61	860.13	5600	883.54	12150	891.91	17000	895.73	22000	898.16	29300	900.89	34700	901.95	46200	903.35	61700	905.35
354	449.97	864	5600	883.79	12150	892.17	17000	895.96	22000	898.39	29300	901.12	34700	902.18	46200	903.63	61700	905.61
355	450.19	862	5600	883.91	12150	892.27	17000	896.09	22000	898.55	29300	901.31	34700	902.38	46200	903.86	61700	905.83
356	450.84	858	5600	884.26	12150	892.5	17000	896.31	22000	898.78	29300	901.55	34700	902.63	46200	904.17	61700	906.16
357	450.86	863.4	5600	884.27	12150	892.51	17000	896.31	22000	898.79	29300	901.56	34700	902.63	46200	904.14	61700	906.11
358 BNRR Bridge																		
359	450.87	863.4	5600	884.35	12150	892.59	17000	896.36	22000	898.92	29300	901.81	34700	902.96	46200	904.71	61700	906.68
361	451.20	865	5600	884.5	12150	892.71	17000	896.49	22000	899.07	29300	901.98	34700	903.18	46200	905.04	61700	907.11
361.5	451.37	865.2	5600	884.59	12150	892.78	17000	896.56	22000	899.14	29300	902.06	34700	903.28	46200	905.17	61700	907.26
362.5 1st Ave. No. Bridge																		
364	451.39	865.2	5600	884.61	12150	892.8	17000	896.58	22000	899.17	29300	902.2	34700	903.59	46200	905.32	61700	907.48
366	451.41	861	5600	884.66	12150	892.85	17000	896.61	22000	899.21	29300	902.23	34700	903.6	46200	905.32	61700	907.48
367	451.57	867	5600	884.72	12150	892.98	17000	896.82	22000	899.49	29300	902.61	34700	904.01	46200	905.82	61700	907.94
368	451.61	866.8	5600	884.76	12150	893.03	17000	896.87	22000	899.55	29300	902.68	34700	904.07	46200	905.88	61700	907.99
369 NP Ave. (Fargo) / Center Ave. (Moorhead)																		
370	451.63	866.8	5600	884.77	12150	893.04	17000	896.9	22000	899.57	29300	902.76	34700	904.16	46200	905.94	61700	908.07
371	451.65	859	5600	884.79	12150	893.03	17000	896.89	22000	899.55	29300	902.73	34700	904.12	46200	905.89	61700	908.02
372	451.67	867.9	5600	884.78	12150	893.02	17000	896.88	22000	899.55	29300	902.73	34700	904.12	46200	905.86	61700	907.97
373 BNRR Bridge																		
374	451.68	867.9	5600	884.89	12150	893.12	17000	896.95	22000	899.61	29300	902.83	34700	904.17	46200	906.16	61700	908.28
375	451.69	864	5600	884.9	12150	893.12	17000	896.94	22000	899.59	29300	902.81	34700	904.16	46200	906.19	61700	908.42
376	451.71	865.9	5600	884.91	12150	893.15	17000	897.01	22000	899.7	29300	902.98	34700	904.37	46200	906.44	61700	908.63
377 Main Ave. Bridge																		
378	451.72	865.9	5600	884.93	12150	893.16	17000	897.03	22000	899.74	29300	903.02	34700	904.43	46200	906.53	61700	908.74
379	451.73	864	5600	885.01	12150	893.26	17000	897.12	22000	899.82	29300	903.11	34700	904.49	46200	906.57	61700	908.75
380	451.85	864	5600	885	12150	893.23	17000	897.09	22000	899.79	29300	903.08	34700	904.53	46200	906.62	61700	908.82
381	452.03	863	5600	885.12	12150	893.4	17000	897.28	22000	900	29300	903.3	34700	904.66	46200	906.75	61700	908.94
381.5	452.04	863	5600	885.13	12150	893.42	17000	897.29	22000	900.01	29300	903.32	34700	904.67	46200	906.76	61700	908.95
384 Midtown Dam																		
385.5	452.05	867	5600	885.15	12150	893.43	17000	897.3	22000	900.02	29300	903.33	34700	904.68	46200	906.77	61700	908.96
386	452.06	867	5600	885.15	12150	893.43	17000	897.3	22000	900.03	29300	903.34	34700	904.69	46200	906.78	61700	908.96
387	452.25	867	5600	885.26	12150	893.49	17000	897.36	22000	900.09	29300	903.39	34700	904.76	46200	906.86	61700	909.06
388	452.46	869	5600	885.43	12150	893.59	17000	897.46	22000	900.21	29300	903.54	34700	904.88	46200	906.99	61700	909.19
389	452.70	866.08	5600	885.65	12150	893.76	17000	897.63	22000	900.39	29300	903.74	34700	905.1	46200	907.21	61700	909.4
390	452.94	867	5600	885.77	12150	893.85	17000	897.71	22000	900.48	29300	903.84	34700	905.21	46200	907.33	61700	909.53
391	453.16	867	5600	885.9	12150	893.96	17000	897.83	22000	900.61	29300	903.98	34700	905.38	46200	907.51	61700	909.7
392	453.39	865	5600	886	12150	894.05	17000	897.94	22000	900.74	29300	904.15	34700	905.54	46200	907.66	61700	909.84
393	454.02	866	5600	886.34	12150	894.28	17000	898.13	22000	900.92	29300	904.32	34700	905.71	46200	907.83	61700	910.01
394	454.10	865	5600	886.38	12150	894.32	17000	898.16	22000	900.95	29300	904.34	34700	905.73	46200	907.86	61700	910.04
395	454.75	864	5600	886.66	12150	894.54	17000	898.38	22000	901.18	29300	904.56	34700	905.94	46200	908.07	61700	910.25
396	455.16	866	5600	886.86	12150	894.75	17000	898.58	22000	901.37	29300	904.71	34700	906.09	46200	908.21	61700	910.38
397	455.20	866.6	5600	886.9	12150	894.78	17000	898.61	22000	901.41	29300	904.75	34700	906.1	46200	908.21	61700	910.36
399 I-94 Bridge																		
401	455.23	866.6	5600	886.9	12150	894.78	17000	898.61	22000	901.41	29300	904.75	34700	906.1	46200	908.25	61700	910.43
402	455.28	865	5600	886.93	12150	894.8	17000	898.64	22000	901.43	29300	904.77	34700	906.17	46200	908.38	61700	910.64
402.5	455.52	864.58	5600	887.05	12150	894.92	17000	898.75	22000	901.55	29300	904.91	34700	906.32	46200	908.52	61700	910.77
403	455.59	866	5600	887.08	12150	894.96	17000	898.78	22000	901.58	29300	904.98	34700	906.37	46200	908.56	61700	910.8
404	455.92	865	5600	887.22	12150	895.09	17000	898.93	22000	901.77	29300	905.15	34700	906.49	46200	908.65	61700	910.87
405	456.65	865	5600	887.57	12150	895.33	17000	899.14	22000	901.97	29300	905.3	34700	906.64	46200	908.79	61700	911.01
406	457.09	866	5600	887.79	12150	895.52	17000	899.33	22000	902.17	29300	905.51	34700	906.83	46200	908.96	61700	911.17

Table B13

Fargo Moorhead Metro Study - Phase 3.1
Reference, Wet - Year Zero, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
407	457.27	861.18	5600	887.86	12150	895.58	17000	899.38	22000	902.21	29300	905.54	34700	906.86	46200	908.99	61700	911.21
408	457.80	864	5600	888.07	12150	895.83	17000	899.65	22000	902.51	29300	905.88	34700	907.16	46200	909.27	61700	911.46
409	457.96	855.9	5600	888.14	12150	895.93	17000	899.75	22000	902.61	29300	905.96	34700	907.24	46200	909.33	61700	911.51
409.1	457.97	855.9	5600	888.14	12150	895.93	17000	899.75	22000	902.61	29300	905.96	34700	907.25	46200	909.34	61700	911.51
409.5			South Dam															
409.6	457.98	855.9	5600	888.19	12150	895.96	17000	899.78	22000	902.64	29300	905.97	34700	907.26	46200	909.35	61700	911.52
409.7	457.99	855.9	5600	888.19	12150	895.97	17000	899.78	22000	902.65	29300	906	34700	907.26	46200	909.36	61700	911.53
413	458.32	869	5600	888.2	12150	896	17000	899.82	22000	902.69	29300	906.07	34700	907.34	46200	909.45	61700	911.63
414	458.75	866	5600	888.47	12150	896.28	17000	900.13	22000	903.04	29300	906.31	34700	907.58	46200	909.65	61700	911.81
459.64	459.64	864.61	5600	888.9	12150	896.68	17000	900.49	22000	903.39	29300	906.61	34700	907.87	46200	909.92	61700	912.07
460.26	460.26	868.74	5600	889.28	12150	897.01	17000	900.8	22000	903.69	29300	906.87	34700	908.12	46200	910.13	61700	912.25
460.72	460.72	869.04	5600	889.58	12150	897.28	17000	901.08	22000	904	29300	907.19	34700	908.33	46200	910.31	61700	912.42
461.3	461.3	868.04	5600	889.99	12150	897.66	17000	901.49	22000	904.43	29300	907.6	34700	908.63	46200	910.55	61700	912.62
462.03	462.03	864.16	5600	890.38	12150	898.06	17000	901.86	22000	904.8	29300	907.98	34700	908.93	46200	910.8	61700	912.81
462.04	462.04	869.5	5600	890.42	12150	898.09	17000	901.88	22000	904.85	29300	908.01	34700	908.96	46200	910.81	61700	912.82
462.05			52nd Avenue So. (Fargo) / 60th Ave So. (Moorhead) Bridge															
462.06	462.06	869.5	5600	890.42	12150	898.1	17000	901.9	22000	904.93	29300	908.15	34700	909.13	46200	910.96	61700	912.95
462.07	462.07	871.5	5600	890.46	12150	898.13	17000	901.94	22000	904.92	29300	908.13	34700	909.13	46200	910.96	61700	912.95
462.7	462.7	862.82	5600	890.81	12150	898.45	17000	902.24	22000	905.23	29300	908.47	34700	909.41	46200	911.23	61700	913.17
463.7	463.7	864.71	5600	891.18	12150	898.88	17000	902.69	22000	905.67	29300	908.9	34700	909.8	46200	911.58	61700	913.49
464.2	464.2	869.2	5600	891.44	12150	899.17	17000	903.01	22000	906.01	29300	909.25	34700	910.08	46200	911.78	61700	913.65
464.6	464.6	865	5600	891.64	12150	899.37	17000	903.18	22000	906.17	29300	909.4	34700	910.2	46200	911.9	61700	913.76
465.1	465.1	862.9	5600	891.84	12150	899.63	17000	903.51	22000	906.56	29300	909.72	34700	910.42	46200	912.08	61700	913.92
465.9	465.9	865.6	5600	892.14	12150	900	17000	903.89	22000	906.93	29300	910	34700	910.6	46200	912.22	61700	914.04
466.5	466.5	864.9	5600	892.39	12150	900.29	17000	904.19	22000	907.23	29300	910.29	34700	910.82	46200	912.4	61700	914.2
467.4	467.4	864.8	5600	892.77	12150	900.72	17000	904.66	22000	907.71	29300	910.68	34700	911.1	46200	912.65	61700	914.42
468.2	468.2	862.6	5600	893.04	12150	901.01	17000	904.96	22000	907.96	29300	910.87	34700	911.26	46200	912.8	61700	914.56
468.9	468.9	861	5600	893.29	12150	901.31	17000	905.27	22000	908.19	29300	911.03	34700	911.4	46200	912.94	61700	914.69
469.7	469.7	870	5600	893.66	12150	901.71	17000	905.59	22000	908.39	29300	911.14	34700	911.53	46200	913.06	61700	914.81
470.1	470.1	870.9	5600	893.84	12150	901.9	17000	905.78	22000	908.65	29300	911.25	34700	911.62	46200	913.14	61700	914.88
470.6	470.6	869.4	4166	894.02	8952	902.09	10791	905.97	13453	908.76	17872	911.4	21196	911.81	30252	913.35	42385	915.1
471.4	471.4	867.5	4166	894.27	8952	902.36	10791	906.17	13453	908.92	17872	911.5	21196	911.93	30252	913.49	42385	915.25
472.18	472.18	869.9	4166	894.53	8952	902.64	10791	906.37	13453	909.09	17872	911.63	21196	912.09	30252	913.67	42385	915.45
472.2	472.2	870.8	4166	894.53	8952	902.63	10791	906.36	13453	909.08	17872	911.61	21196	912.06	30252	913.63	42385	915.41
472.22	472.22	870.8	4166	894.54	8952	902.65	10791	906.38	13453	909.12	17872	911.68	21196	912.15	30252	913.75	42385	915.54
473	473	871.8	4166	894.79	8952	902.88	10791	906.54	13453	909.25	17872	911.79	21196	912.29	30252	913.91	42385	915.71
473.8	473.8	869	4166	895.06	8952	903.18	10791	906.77	13453	909.47	17872	911.99	21196	912.52	30252	914.18	42385	916.01
474.08	474.08	873.5	4166	895.17	8952	903.29	10791	906.86	13453	909.55	17872	912.08	21196	912.61	30252	914.27	42385	916.1
474.09	474.09	873.5	4166	895.18	8952	903.32	10791	906.89	13453	909.58	17872	912.11	21196	912.66	30252	914.34	42385	916.13
474.1			Cass Co Hwy 16/112th Ave So. (Fargo) / 110th Ave So. (Moorhead) Bridge															
474.11	474.11	874	4166	895.18	8952	903.32	10791	906.89	13453	909.58	17872	912.13	21196	912.66	30252	914.37	42385	916.35
474.12	474.12	874	4166	895.19	8952	903.33	10791	906.89	13453	909.59	17872	912.18	21196	912.71	30252	914.38	42385	916.37
474.6	474.6	873.4	4166	895.39	8952	903.49	10791	907.02	13453	909.72	17872	912.29	21196	912.87	30252	914.56	42385	916.53
475.3	475.3	872.2	4166	895.66	8952	903.78	10791	907.26	13453	909.97	17872	912.57	21196	913.21	30252	914.91	42385	916.84
475.7	475.7	871	4166	895.8	8952	903.95	10791	907.42	13453	910.16	17872	912.81	21196	913.45	30252	915.19	42385	916.98
476.4	476.4	873.7	4166	896.07	8952	904.2	10791	907.62	13453	910.34	17872	912.97	21196	913.64	30252	915.37	42385	917.14
477	477	874	4021	896.34	7227	904.45	10537	907.85	14618	910.57	20566	913.2	24472	913.91	28720	915.6	33177	917.36
477.6	477.6	870.2	4000	896.53	7000	904.61	10500	908.06	14800	910.85	21000	913.55	25000	914.31	28500	915.92	32000	917.5
478	478	873.4	4000	896.67	7000	904.71	10500	908.18	14800	911	21000	913.72	25000	914.51	28500	916.09	32000	917.56
478.8	478.8	874.8	4000	896.91	7000	904.9	10500	908.42	14800	911.31	21000	914.1	25000	914.94	28500	916.41	32000	917.7
479.2	479.2	875.7	4000	897.02	7000	904.99	10500	908.53	14800	911.44	21000	914.25	25000	915.1	28500	916.49	32000	917.78
479.7	479.7	871.5	4000	897.2	7000	905.1	10500	908.67	14800	911.61	21000	914.44	25000	915.33	28500	916.7	32000	917.89
480.5	480.5	874.7	4000	897.48	7000	905.3	10500	908.93	14800	911.9	21000	914.78	25000	915.67	28500	916.99	32000	918.14
481.8	481.8	875.3	4000	897.88	7000	905.64	10500	909.33	14800	912.35	21000	915.28	25000	916.22	28500	917.47	32000	918.48
482.3	482.3	874.2	4000	898.09	7000	905.82	10500	909.57	14800	912.62	21000	915.56	25000	916.5	28500	917.66	32000	918.62
482.698	482.698	869.74	4000	898.2	7000	905.92	10500	909.68	14800	912.75	21000	915.72	25000	916.71	28500	917.84	32000	918.75
482.7	482.7	887.04	4000	898.1	7000	905.86	10500	909.62	14800	912.68	21000	915.67	25000	916.67	28500	917.83	32000	918.75
482.702	482.702	878.54	4000	898.23	7000	905.93	10500	909.69	14800	912.75	21000	915.72	25000	916.71	28500	917.84	32000	918.75
483.1	483.1	879.2	4000	898.47	7000	906.12	10500	909.93	14800	913.04	21000	916.04	25000	917.02	28500	918.05	32000	918.87
483.9	483.9	873	4000	898.87	7000	906.39	10500	910.26	14800	913.45	21000	916.47	25000	917.5	28500	918.47	32000	919.26
485.08	485.08	880.5	4000	899.37	7000	906.62	10500	910.48	14800	913.69	21000	916.74	25000	917.8	28500	918.76	32000	919.55
485.09	485.09	880.5	4000	899.38	7000	906.61	10500	910.48	14800	913.69	21000	916.74	25000	917.8	28500	918.77	32000	919.55
485.1			Cass Co. Hwy 18/52nd St. SE (Fargo) / 160th Ave So. (Moorhead) Bridge															
485.11	485.11	879.4	4000	899.38	7000	906.63	10500	910.56	14800	913.77	21000	916.79	25000	917.84	28500	918.81	32000	919.58
485.12	485.12	879.4	4000	899.39	7000	906.66	10500	910.56	14800	913.77	21000	916.79	25000	917.85	28500	918.82	32000	919.59
485.6	485.6	878.62	4000	899.58	7000	906.77	10500	910.65	14800	913.85	21000	916.88	25000	917.94	28500	918.92	32000	919.69
485.9	485.9	879.5	4000	899.74	7000	906.88	10500	910.77	14800	913.99	21000	917.05	25000	918.15	28500	919.15	32000	919.91
486.3	486.3	872.1	4000	899.9	7000	907	10500	910.91	14800	914.15	21000	917.25	25000	918.39	28500	919.42	32000	920.2
487	487	871.3	4000	900.12	7000	907.18	10500	911.13	14800	914.44	21000	917.66	25000	918.9	28500	919.99	32000	920.83
487.5	487.5	878.8	4000	900.35	7000	907.36	10500	911.33	14800	914.66	21000	917.91	25000	919.19	28500	920.31	32000	921.14
488.68	488.68	873.24	4000	901.04	7000	907.87	10500	911.93	14800	915.39	21000	918.75	25000	920.14	28500	921.33	32000	922.26

Table B14

Fargo Moorhead Metro Study - Phase 3.1
Reference, Year 25, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
279	403.65	832.3	8950	861.67	18490	870.2	26122.73	873.73	34240	875.7	45780	876.86	55200	877.69	66150	878.58	82929.99	879.41
280	404.98	836.6	8950	862.33	18490	870.94	26122.73	874.31	34240	876.38	45780	877.66	55200	878.49	66150	879.38	82929.99	880.24
281	406.51	834.9	8950	863.21	18490	871.84	26122.73	875.33	34240	877.45	45780	878.67	55200	879.48	66150	880.19	82929.99	881.14
282	410.21	838	8950	865.13	18490	873.73	26122.73	877.12	34240	879.09	45780	879.9	55200	880.64	66150	881.44	82929.99	882.51
283	413.23	837.6	8950	866.44	18490	875.11	26122.73	878.49	34240	880.2	45780	881.15	55200	881.69	66150	882.44	82929.99	883.58
284	414.98	842.7	8200	867.17	17400	876.04	24702.73	879.62	32440	881.11	43410	882.04	52310	882.6	62960	883.47	79529.99	884.65
285	415.38	841.4	8200	867.35	17400	876.26	24702.73	879.85	32440	881.31	43410	882.29	52310	882.83	62960	883.72	79529.99	884.9
286	415.84	843	8200	867.46	17400	876.34	24702.73	879.9	32440	881.35	43410	882.38	52310	882.97	62960	883.87	79529.99	885.17
287	415.90	844.1	8200	867.53	17400	876.41	24702.73	880.01	32440	881.55	43410	882.52	52310	883.1	62960	883.98	79529.99	885.23
288			22nd St SE or Cass Co. #34 / 170th Ave NW or Clay Co.36															
289	415.91	844.1	8200	867.53	17400	876.4	24702.73	880	32440	881.67	43410	882.63	52310	883.29	62960	884.42	79529.99	885.46
290	415.93	837.5	8200	867.55	17400	876.42	24702.73	880.02	32440	881.56	43410	882.72	52310	883.39	62960	884.42	79529.99	885.5
291	417.10	840.8	8200	867.97	17400	876.89	24702.73	880.6	32440	882.37	43410	883.63	52310	884.41	62960	885.17	79529.99	886.13
292	418.18	844.4	7124	868.37	14681	877.36	20666.73	881.13	27055	882.89	36261	884.24	43687	885.01	54114	885.66	70316.99	886.67
293	419.82	844.7	7124	869.15	14681	878.16	20666.73	881.86	27055	883.52	36261	884.9	43687	885.49	54114	886.2	70316.99	887.26
294	420.32	843.8	7124	869.42	14681	878.39	20666.73	882.14	27055	883.85	36261	885.3	43687	885.98	54114	886.7	70316.99	887.74
295	421.93	845.3	7124	870.63	14681	879.47	20666.73	883.09	27055	884.63	36261	886.04	43687	886.75	54114	887.57	70316.99	888.36
296	422.69	847.4	7124	870.95	14681	879.8	20666.73	883.37	27055	884.87	36261	886.25	43687	886.97	54114	887.86	70316.99	888.68
297	423.95	849.5	7124	871.49	14681	880.39	20666.73	884.07	27055	885.78	36261	887.36	43687	888.24	54114	888.63	70316.99	889.75
298	425.51	848.7	7124	872.22	14681	881.11	20666.73	884.78	27055	886.49	36261	888.09	43687	889.03	54114	889.27	70316.99	890.25
299	426.87	851.1	7124	872.92	14681	881.76	20666.73	885.43	27055	887.2	36261	888.9	43687	889.62	54114	890.07	70316.99	890.63
300	428.06	849.6	4865	873.48	11392	882.37	16407.73	886.03	20438.81	887.79	24558.68	889.5	25882.98	890.28	36722.73	890.44	51625.63	891.35
301	428.74	850.3	4865	873.76	11392	882.7	16407.73	886.42	20438.81	888.22	24558.68	889.87	25882.98	890.65	36722.73	891.01	51625.63	891.77
302	430.23	852	4865	874.35	11392	883.25	16407.73	887.06	20438.81	888.97	24558.68	890.68	25882.98	891.45	36722.73	891.6	51625.63	892.52
303	431.94	853.5	4865	875.15	11392	884.03	16407.73	887.9	20727.7	889.87	27375.36	891.57	32307.96	892.36	41268.12	893.15	58100.14	893.7
303.4	432.07	853.42	4573	875.2	10949	884.11	15842.74	888.01	19758.38	890.01	26029.58	891.62	30435.12	892.43	39095.07	893.36	55742.19	894.04
303.5	432.10	853.46	4573	875.23	10949	884.14	15842.74	888.05	19522.3	890.06	25786.9	891.7	29928.15	892.52	38244.53	893.5	54815.82	894.26
303.8	432.53	854.5	4573	875.4	10949	884.32	15842.74	888.23	19208.64	890.28	25045.02	892.1	28476.34	892.93	35369.43	893.97	51771.56	895.02
304	432.84	855.5	4574	875.49	10950	884.43	15745.36	888.33	18862.04	890.36	22093.19	892.19	21781.94	893.02	25470.43	894.07	36586.48	895.18
305	433.26	854.4	4574	875.66	10950	884.62	15745.36	888.53	18862.04	890.53	19942.84	892.28	21701.19	893.11	23798.73	894.16	31982.32	895.31
306	433.35	855	4574	875.73	10950	884.71	15745.36	888.64	18862.04	890.67	19430.39	892.4	21681.94	893.14	23403.76	894.19	30686.33	895.33
307			76th Ave. No. (Fargo) / 90th Ave NW (Moorhead) Bridge															
308	433.37	855	4574	875.74	10950	884.72	15745.36	888.66	18862.04	890.65	19430.39	892.42	21681.94	893.18	23403.76	894.23	30686.33	895.39
309	433.83	854.5	4574	875.89	10950	884.88	15745.36	888.86	18862.04	890.9	19430.39	892.62	21681.94	893.35	23403.76	894.39	30686.33	895.6
310	434.61	852.8	4574	876.14	10950	885.13	15745.36	889.12	18862.04	891.18	19430.39	892.83	21681.94	893.55	23403.76	894.56	30686.33	895.82
310.5	435.70	855	4574	876.51	10950	885.51	15745.36	889.51	18862.04	891.6	19430.39	893.21	21681.94	893.95	23403.76	894.94	30686.33	896.3
311	436.34	855	4574	876.73	10950	885.77	15629.2	889.82	18451.33	891.97	21764.75	893.52	22058.02	894.28	25780.14	895.28	32507.69	896.64
312	437.05	863.63	4374	876.98	10655	886.05	15239.2	890.15	18035.33	892.35	21474	893.92	28199.87	894.68	26820.77	895.73	47117.78	897.05
313	437.95	856	4373	877.32	10654	886.48	15336.56	890.64	18382.93	892.89	24276.58	894.49	28302.42	895.46	35164.14	896.38	47117.78	898.33
314	438.78	857	4373	877.66	10654	886.85	15336.56	891.05	18382.93	893.32	24276.58	895.09	28302.42	896.13	35164.14	897.24	47117.78	899.21
315	439.05	856	4373	877.75	10654	886.96	15336.56	891.18	18382.93	893.47	24276.58	895.31	28302.42	896.39	35164.14	897.55	47117.78	899.39
316	439.46	855.1	4373	877.91	10654	887.18	15336.56	891.47	18382.93	893.8	22681.07	895.8	23600.97	897	23276.18	898.41	25520.56	900.26
317			40th Ave. No. (Fargo) / Wall Street NW (Moorhead) Bridge															
318	439.47	855.1	4373	877.92	10654	887.19	15336.56	891.47	18382.93	893.79	22681.07	895.79	23600.97	897.01	23276.18	898.46	25520.56	900.3
319	439.50	857	4373	877.93	10654	887.25	15336.56	891.54	18382.93	893.86	22681.07	895.87	23600.97	897.11	23276.18	898.55	25520.56	900.34
320	439.66	856	4373	877.99	10654	887.31	15336.56	891.6	18382.93	893.93	22681.07	895.94	23600.97	897.15	23276.18	898.61	25520.56	900.39
321	440.31	858	4373	878.21	10654	887.58	15336.56	891.91	18382.93	894.26	22681.07	896.33	23600.97	897.48	23276.18	898.82	25520.56	900.56
322	440.32	858.1	4373	878.21	10654	887.58	15336.56	891.91	18382.93	894.25	22681.07	896.33	23600.97	897.48	23276.18	898.82	25520.56	900.56
323			Broadway Street No. (Fargo) / Broadway Street NW (Moorhead) Bridge															
324	440.33	858.1	4373	878.22	10654	887.69	15336.56	891.98	18382.93	894.31	22681.07	896.37	23600.97	897.51	23276.18	898.84	25520.56	900.58
325	440.34	857	4373	878.23	10654	887.7	15336.56	891.99	18382.93	894.31	22681.07	896.38	23600.97	897.52	23276.18	898.84	25520.56	900.58
326	440.54	858	4373	878.28	10654	887.76	15336.56	892.05	18382.93	894.38	22681.07	896.49	23600.97	897.59	23276.18	898.9	25520.56	900.61
327	441.53	859	4353	878.66	10609	888.04	15281.58	892.31	18322.93	894.63	22627.84	896.76	23579.8	897.85	23269.71	899.09	25685.54	900.73
328	442.14	858	4353	878.86	10609	888.2	15314.36	892.46	19086.22	894.78	24856.78	896.94	28342.62	898.02	31218.69	899.22	40608.48	900.8
329	442.45	857	4353	878.94	10609	888.22	15390.92	892.47	20285.08	894.79	27224.61	896.95	32378.56	898.05	40232.64	899.26	53648.67	900.83
329.5	442.93	858.06	4352	879.08	10608	888.34	15394	892.56	20345	894.89	27441	897.08	32921	898.1	42242	899.32	57641	900.89
330	443.69	858	4352	879.32	10608	888.53	15394	892.72	20345	895.05	27441	897.23	32921	898.29	42242	899.46	57641	901.05
331	443.99	859	4352	879.4	10608	888.64	15394	892.84	20345	895.2	27441	897.47	32921	898.58	42242	899.59	57641	901.2
332	444.33	856	4352	879.45	10608	888.68	15394	892.88	20345	895.25	27441	897.53	32921	898.62	42242	899.64	57641	901.25
333	444.92	859	4352	879.53	10608	888.72	15394	892.91	20345	895.27	27441	897.53	32921	898.59	42242	899.66	57641	901.29
334	445.18	860	4352	879.62	10608	888.8	15394	892.99	20345	895.35	27441	897.63	32921	898.68	42242	899.8	57641	901.44
335	445.31	860	4352	879.67	10608	888.89	15394	893.12	20345	895.55	27441	897.93	32921	899.08	42242	899.98	57641	901.61
336	445.36	859	4352	879.68	10608	888.9	15394	893.13	20345	895.56	27441	897.95	32921	899.08	42242	899.99	57641	901.63
336.5	445.76	857.69	4352	879.71	10608	888.93	15394	893.16	20345	895.59	27441	897.99	32921	899.13	42242	900.04	57641	901.68
337	446.09	860	4352	879.75	10608	888.93	15394	893.15	20345	895.59	27441	897.98	32921	899.11	42242	900.03	57641	901.67
338	446.29	861	4352	879.82	10608	888.96	15394	893.17	20345	895.6	27441	897.98	32921	899.12	42242	900.08	57641	901.73
338.5	446.70	860.9	4352	879.97	10608	889.1	15394	893.31	20345	895.78	27441	898.2	32921	899.35	42242	900.26	57641	901.94
339	446.76	861	4352	879.99	10608	889.12	15394	893.33	20345	895.78	27441	898.22	32921	899.41	42242	900.33	57641	902.04
340	447.04	860	4352	880.08	10608	889.22	15394	893.46	20345	895.97	27441	898.5	32921	899.76	42242	900.67	57641	902.4
341	447.24	859	4352	880.16	10608	889.28	15394	893.51	20345	896.01	27441	898.54	32921	899.79	42242	900.72	57641	902.46
342	447.42	861	4352	880.28	10608	889.37	15394	893.59	20345	896.1	27441	898.64	32921	899.92	42242	900.89	57641	902.65
342.5	447.78	858.99	4352	880.43	10608	889.51	15394	893.74	20345	896.29	27441	898.9	32921	900.24	42242	901.14	57641	902.95
343	448.13	861	4352	880.56	10608	889.62	15394	893.85	20345	896.41	27441	899.04	32921	900.4	42242	901.33	57641	903.17
344	448.35	859	4352	880.66	10608	889.73	15394	894	20345	896.62	27441	899.33	32921	900.77	42242	901.73	57641	903.6
345	448.89	865.3	4352	881.01	10608	889.95	15394	894.18	20345	896.81	27441	899.55	32921	901.01	42242	901.98	57641	903.89
348	448.90	864.3	4352	881	10608	889.93	15394	894.15	20345	896.78	27441	899.49	32921	900.93	42242	901.92	57641	903.85
348.4			North Dam															
348.8	448.90	864.3	4352	881.07	10608	889.99	15394	894.2	20345	896.83	27441	899.56	32921	901.02	42242	901.97	57641	903.88
351	448.91	861	4352	881.09	10608	889.99	15394	894.2	20345	896.84	27441	899.56	32921	901.02	42242	901.97	57641	903.89
351.4	449.06	861	4352	881.17	10608	890.13	15394	894.38	20345	897.06	27441	899.86	32921	901.34	42242	902.4	57641	904.31
351.5			12th Ave. No. (Fargo) / 15th Ave No. (Moorhead) Bridge															
351.6	449.07	861	4352	881.18	10608	890.23	15394	894.49	20345	897.14	27441	899.89	32921	901.4	42242	902.51	57641	904.45
352	449.09	862	4352	881.19	10608	890.25	15394	894.52	20345	897.18	27441	899.94	32921	901.44	42242	902.53	57641	904.45
353	449.28	863	4352	881.27	10608	890.35	15394	894.64	20345	897.33	27441	900.15	32921	901.66	42242	902.82	57641	904.78
353.5	449.61	860.13	4352	881.41	10608	890.44	15394	894.7	20345	897.4	27441	900.23	32921	901.77	42242	902.96	57641	904.97
354	449.97	864	4352	881.65	10608	890.67	15394	894.94	20345	897.63	27441	900.47	32921	901.99	42242	903.22	57641	905.22
355	450.19	862	4352	881.79	10608	890.77	15394	895.06	20345	897.79	27441	900.65	32921	902.17	42242	903.44	57641	905.45
356	450.84	858	4352	882.17	10608	891.02	15394	895.27	20345	898.02	27441	900.89	32921	902.41	42242	903.73	57641	905.76
357	450.86	863.4	4352	882.18	10608	891.02	15394	895.28	20345	898.02	27441	900.9	32921	902.42	42242	903.71	57641	905.71
358			BNRR Bridge															
359	450.87	863.4	4352	882.25	10608	891.12	15394	895.33	20345	898.13	27441	901.1	32921	902.71	42242	904.19	57641	906.35
361	451.20	865	4352	882.4	10608	891.24	15394	895.45	20345	898.27	27441	901.27	32921	902.9	42242	904.48	57641	906.76
361.5	451.37	865.2	4352	882.5	10608	891.31	15394	895.52	20345	898.34	27441	901.35	32921	903.01	42242	904.6	57641	906.89
362.5			1st Ave. No. Bridge															
364	451.39	865.2	4352	882.52	10608	891.33	15394	895.53	20345	898.37	27441	901.48	32921	903.17	42242	904.72	57641	907.1
366	451.41	861	4352	882.56	10608	891.38	15394	895.58	20345	898.41	27441	901.49	32921	903.2	42242	904.72	57641	907.1
367	451.57	867	4352	882.62	10608	891.49	15394	895.75	20345	898.66	27441	901.85	32921	903.64	42242	905.2	57641	907.56
368	451.61	866.8	4352	882.67	10608	891.54	15394	895.8	20345	898.72	27441	901.91	32921	903.71	42242	905.27	57641	907.61
369			NP Ave. (Fargo) / Center Ave. (Moorhead)															
370	451.63	866.8	4352	882.68	10608	891.55	15394	895.81	20345	898.73	27441	901.98	32921	903.82	42242	905.34	57641	907.67
371	451.65	859	4352	882.71	10608	891.55	15394	895.8	20345	898.71	27441	901.95	32921	903.79	42242	905.3	57641	907.63
372	451.67	867.9	4352	882.69	10608	891.54	15394	895.79	20345	898.71	27441	901.95	32921	903.79	42242	905.28	57641	907.59
373			BNRR Bridge															
374	451.68	867.9	4352	882.8	10608	891.64	15394	895.87	20345	898.77	27441	902.01	32921	903.86	42242	905.54	57641	907.87
375	451.69	864	4352	882.82	10608	891.64	15394	895.86	20345	898.76	27441	901.99	32921	903.83	42242	905.54	57641	907.99
376	451.71	865.9	4352	882.82	10608	891.66	15394	895.92	20345	898.86	27441	902.14	32921	904.06	42242	905.6	57641	908.2
377			Main Ave. Bridge															
378	451.72	865.9	4352	882.84	10608	891.67	15394	895.94	20345	898.89	27441	902.2	32921	904.11	42242	905.87	57641	908.3
379	451.73	864	4352	882.92	10608	891.77	15394	896.03	20345	898.98	27441	902.29	32921	904.18	42242	905.92	57641	908.32
380	451.85	864	4352	882.91	10608	891.74	15394	896	20345	898.95	27441	902.27	32921	904.21	42242	905.97	57641	908.39
381	452.03	863	4352	883.02	10608	891.9	15394	896.18	20345	899.15	27441	902.48	32921	904.34	42242	906.1	57641	908.5
381.5	452.04	863	4352	883.03	10608	891.91	15394	896.2	20345	899.16	27441	902.5	32921	904.35	42242	906.11	57641	908.51
384			Midtown Dam															
385.5	452.05	867	4352	883.07	10608	891.92	15394	896.2	20345	899.17	27441	902.51	32921	904.36	42242	906.12	57641	908.52
386	452.06	867	4352	883.08	10608	891.92	15394	896.21	20345	899.18	27441	902.52	32921	904.37	42242	906.13	57641	908.53
387	452.25	867	4352	883.2	10608	891.99	15394	896.26	20345	899.23	27441	902.57	32921	904.43	42242	906.21	57641	908.62
388	452.46	869	4352	883.39	10608	892.09	15394	896.36	20345	899.35	27441	902.72	32921	904.55	42242	906.33	57641	908.75
389	452.70	866.08	4352	883.64	10608	892.26	15394	896.52	20345	899.52	27441	902.91	32921	904.75	42242	906.55	57641	908.95
390	452.94	867	4352	883.77	10608	892.35	15394	896.61	20345	899.61	27441	903.01	32921	904.86	42242	906.67	57641	909.08
391	453.16	867	4352	883.89	10608	892.46	15394	896.72	20345	899.74	27441	903.15	32921	905.04	42242	906.85	57641	909.25
392	453.39	865	4352	884.01	10608	892.55	15394	896.82	20345	899.87	27441	903.31	32921	905.2	42242	907	57641	909.38
393	454.02	866	4352	884.36	10608	892.8	15394	897.02	20345	900.05	27441	903.49	32921	905.37	42242	907.18	57641	909.56
394	454.10	865	4352	884.39	10608	892.84	15394	897.06	20345	900.08	27441	903.51	32921	905.38	42242	907.2	57641	909.58
395	454.75	864	4352	884.68	10608	893.07	15394	897.27	20345	900.31	27441	903.73	32921	905.6	42242	907.42	57641	909.79
396	455.16	866	4352	884.87	10608	893.28	15394	897.47	20345	900.51	27441	903.89	32921	905.75	42242	907.55	57641	909.92
397	455.20	866.6	4352	884.91	10608	893.31	15394	897.5	20345	900.54	27441	903.93	32921	905.76	42242	907.56	57641	909.9
399			I-94 Bridge															
401	455.23	866.6	4352	884.92	10608	893.32	15394	897.5	20345	900.54	27441	903.93	32921	905.76	42242	907.59	57641	909.98
402	455.28	865	4352	884.95	10608	893.33	15394	897.52	20345	900.57	27441	903.95	32921	905.8	42242	907.71	57641	910.16
402.5	455.52	864.58	4352	885.06	10608	893.46	15394	897.64	20345	900.68	27441	904.09	32921	905.94	42242	907.86	57641	910.29
403	455.59	866	4352	885.09	10608	893.5	15394	897.68	20345	900.71	27441	904.16	32921	906.05	42242	907.9	57641	910.32
404	455.92	865	4352	885.23	10608	893.62	15394	897.81	20345	900.89	27441	904.34	32921	906.16	42242	907.99	57641	910.4
405	456.65	865	4352	885.6	10608	893.88	15394	898.02	20345	901.1	27441	904.51	32921	906.32	42242	908.13	57641	910.53
406	457.09	866	4352	885.83	10608	894.07	15394	898.21	20345	901.3	27441	904.73	32921	906.51	42242	908.31	57641	910.69

Table B14

Fargo Moorhead Metro Study - Phase 3.1
Reference, Year 25, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
407	457.27	861.18	4352	885.91	10608	894.13	15394	898.26	20345	901.34	27441	904.75	32921	906.53	42242	908.34	57641	910.73
408	457.80	864	4352	886.1	10608	894.37	15394	898.52	20345	901.63	27441	905.08	32921	906.83	42242	908.62	57641	910.98
409	457.95	855.9	4352	886.15	10608	894.47	15394	898.62	20345	901.73	27441	905.18	32921	906.9	42242	908.69	57641	911.03
409.1	457.97	855.9	4352	886.16	10608	894.47	15394	898.62	20345	901.73	27441	905.18	32921	906.91	42242	908.69	57641	911.04
409.5			South Dam															
409.6	457.98	855.9	4352	886.22	10608	894.5	15394	898.66	20345	901.75	27441	905.21	32921	906.92	42242	908.7	57641	911.05
409.7	457.99	855.9	4352	886.22	10608	894.51	15394	898.66	20345	901.77	27441	905.22	32921	906.93	42242	908.71	57641	911.05
413	458.32	869	4352	886.22	10608	894.53	15394	898.7	20345	901.81	27441	905.26	32921	907	42242	908.8	57641	911.15
414	458.75	866	4352	886.49	10608	894.81	15394	899	20345	902.15	27441	905.63	32921	907.25	42242	909.02	57641	911.33
459.64	459.64	864.81	4352	886.9	10608	895.24	15394	899.37	20345	902.5	27441	905.96	32921	907.54	42242	909.3	57641	911.59
460.28	460.28	868.74	4352	887.28	10608	895.57	15394	899.68	20345	902.8	27441	906.22	32921	907.78	42242	909.55	57641	911.78
460.72	460.72	869.04	4352	887.59	10608	895.84	15394	899.95	20345	903.1	27441	906.54	32921	907.99	42242	909.73	57641	911.94
461.3	461.3	868.04	4352	887.99	10608	896.22	15394	900.34	20345	903.52	27441	906.97	32921	908.3	42242	909.99	57641	912.15
462.03	462.03	864.16	4352	888.36	10608	896.62	15394	900.72	20345	903.9	27441	907.34	32921	908.62	42242	910.24	57641	912.35
462.04	462.04	869.5	4352	888.4	10608	896.66	15394	900.75	20345	903.95	27441	907.38	32921	908.63	42242	910.25	57641	912.36
462.05			52nd Avenue So. (Fargo) / 60th Ave So. (Moorhead) Bridge															
462.06	462.06	869.5	4352	888.41	10608	896.66	15394	900.76	20345	904.03	27441	907.43	32921	908.78	42242	910.43	57641	912.5
462.07	462.07	871.5	4352	888.45	10608	896.69	15394	900.79	20345	904.03	27441	907.41	32921	908.79	42242	910.43	57641	912.5
462.7	462.7	862.82	4352	888.81	10608	897.02	15394	901.11	20345	904.33	27441	907.74	32921	909.07	42242	910.69	57641	912.72
463.7	463.7	864.71	4352	889.14	10608	897.44	15394	901.55	20345	904.77	27441	908.18	32921	909.47	42242	911.04	57641	913.05
464.2	464.2	869.2	4352	889.4	10608	897.72	15394	901.86	20345	905.1	27441	908.54	32921	909.83	42242	911.27	57641	913.22
464.6	464.6	865	4352	889.61	10608	897.91	15394	902.05	20345	905.26	27441	908.7	32921	909.93	42242	911.38	57641	913.32
465.1	465.1	862.9	4352	889.78	10608	898.16	15394	902.35	20345	905.64	27441	909.05	32921	910.27	42242	911.56	57641	913.49
465.9	465.9	865.6	4352	890.05	10608	898.52	15394	902.73	20345	906.03	27441	909.35	32921	910.45	42242	911.72	57641	913.61
466.5	466.5	864.9	4352	890.29	10608	898.79	15394	903.02	20345	906.34	27441	909.65	32921	910.77	42242	911.9	57641	913.78
467.4	467.4	864.8	4352	890.64	10608	899.22	15394	903.48	20345	906.81	27441	910.08	32921	911.03	42242	912.16	57641	914
468.2	468.2	862.6	4352	890.89	10608	899.51	15394	903.76	20345	907.09	27441	910.28	32921	911.18	42242	912.31	57641	914.14
468.9	468.9	861	4352	891.11	10608	899.8	15394	904.08	20345	907.32	27441	910.45	32921	911.31	42242	912.45	57641	914.27
469.7	469.7	870	4352	891.47	10608	900.2	15394	904.41	20345	907.55	27441	910.6	32921	911.44	42242	912.56	57641	914.39
470.1	470.1	870.9	4352	891.63	10608	900.38	15394	904.61	20345	907.74	27441	910.72	32921	911.56	42242	912.66	57641	914.46
470.6	470.6	869.4	3198	891.8	7772	900.58	9722	904.8	12388	907.94	16680	910.88	20055	911.74	27549	912.86	39492	914.68
471.4	471.4	867.5	3198	892.04	7772	900.85	9722	904.99	12388	908.11	16680	910.99	20055	911.85	27549	913	39492	914.83
472.18	472.18	869.9	3198	892.28	7772	901.12	9722	905.19	12388	908.29	16680	911.13	20055	911.99	27549	913.17	39492	915.03
472.2	472.2	870.8	3198	892.29	7772	901.11	9722	905.19	12388	908.28	16680	911.11	20055	911.97	27549	913.14	39492	914.99
472.22	472.22	870.8	3198	892.29	7772	901.12	9722	905.2	12388	908.31	16680	911.17	20055	912.05	27549	913.26	39492	915.12
473	473	871.8	3198	892.53	7772	901.36	9722	905.37	12388	908.45	16680	911.29	20055	912.17	27549	913.41	39492	915.29
473.8	473.8	869	3198	892.79	7772	901.66	9722	905.6	12388	908.67	16680	911.49	20055	912.39	27549	913.68	39492	915.58
474.08	474.08	873.5	3198	892.89	7772	901.76	9722	905.69	12388	908.75	16680	911.57	20055	912.46	27549	913.77	39492	915.68
474.09	474.09	873.5	3198	892.9	7772	901.79	9722	905.72	12388	908.78	16680	911.61	20055	912.52	27549	913.84	39492	915.71
474.1			Cass Co Hwy 16/112th Ave So. (Fargo) / 110th Ave So. (Moorhead) Bridge															
474.11	474.11	874	3198	892.9	7772	901.78	9722	905.72	12388	908.77	16680	911.62	20055	912.54	27549	913.87	39492	915.93
474.12	474.12	874	3198	892.91	7772	901.79	9722	905.72	12388	908.78	16680	911.64	20055	912.57	27549	913.89	39492	915.95
474.6	474.6	873.4	3198	893.12	7772	901.96	9722	905.85	12388	908.91	16680	911.77	20055	912.72	27549	914.06	39492	916.11
475.3	475.3	872.2	3198	893.37	7772	902.24	9722	906.08	12388	909.15	16680	912.04	20055	913.04	27549	914.42	39492	916.43
475.7	475.7	871	3198	893.51	7772	902.41	9722	906.23	12388	909.33	16680	912.27	20055	913.26	27549	914.7	39492	916.57
476.4	476.4	873.7	3198	893.78	7772	902.66	9722	906.43	12388	909.51	16680	912.44	20055	913.44	27549	914.9	39492	916.74
477	477	874	3147	894.04	6349	902.89	9577	906.64	13547	909.74	19288	912.66	23242	913.7	26339	915.14	31093	916.96
477.6	477.6	870.2	3139	894.23	6160	903.05	9555	906.85	13729	910.01	19709	913.02	23757	914.08	26164	915.45	30016	917.19
478	478	873.4	3139	894.37	6160	903.15	9555	906.98	13729	910.17	19709	913.2	23757	914.28	26164	915.61	30016	917.25
478.8	478.8	874.8	3139	894.6	6160	903.34	9555	907.22	13729	910.46	19709	913.59	23757	914.69	26164	915.94	30016	917.39
479.2	479.2	875.7	3139	894.71	6160	903.42	9555	907.33	13729	910.59	19709	913.74	23757	914.85	26164	916.07	30016	917.46
479.7	479.7	871.5	3139	894.9	6160	903.54	9555	907.48	13729	910.76	19709	913.93	23757	915.07	26164	916.26	30016	917.58
480.5	480.5	874.7	3139	895.18	6160	903.75	9555	907.74	13729	911.06	19709	914.25	23757	915.4	26164	916.54	30016	917.83
481.8	481.8	875.3	3139	895.57	6160	904.08	9555	908.16	13729	911.51	19709	914.76	23757	915.97	26164	917	30016	918.24
482.3	482.3	874.2	3139	895.77	6160	904.26	9555	908.39	13729	911.79	19709	915.04	23757	916.28	26164	917.21	30016	918.39
482.698	482.698	869.74	3139	895.88	6160	904.36	9555	908.51	13729	911.93	19709	915.2	23757	916.49	26164	917.39	30016	918.54
482.7	482.7	887.04	3139	895.77	6160	904.3	9555	908.44	13729	911.86	19709	915.13	23757	916.44	26164	917.37	30016	918.54
482.702	482.702	878.54	3139	895.92	6160	904.37	9555	908.51	13729	911.93	19709	915.2	23757	916.49	26164	917.39	30016	918.54
483.1	483.1	879.2	3139	896.16	6160	904.56	9555	908.76	13729	912.22	19709	915.53	23757	916.81	26164	917.62	30016	918.7
483.9	483.9	873	3139	896.59	6160	904.84	9555	909.08	13729	912.62	19709	916.01	23757	917.24	26164	918.03	30016	919.06
485.08	485.08	880.5	3139	897.17	6160	905.1	9555	909.32	13729	912.86	19709	916.27	23757	917.52	26164	918.32	30016	919.33
485.09	485.09	880.5	3139	897.18	6160	905.09	9555	909.29	13729	912.86	19709	916.27	23757	917.52	26164	918.32	30016	919.33
485.1			Cass Co. Hwy 18/52nd St. SE (Fargo) / 160th Ave So. (Moorhead) Bridge															
485.11	485.11	879.4	3139	897.19	6160	905.09	9555	909.35	13729	912.96	19709	916.32	23757	917.57	26164	918.36	30016	919.37
485.12	485.12	879.4	3139	897.19	6160	905.12	9555	909.42	13729	912.96	19709	916.33	23757	917.57	26164	918.37	30016	919.38
485.6	485.6	878.62	3139	897.4	6160	905.23	9555	909.51	13729	913.04	19709	916.4	23757	917.66	26164	918.46	30016	919.47
485.9	485.9	879.5	3139	897.58	6160	905.34	9555	909.64	13729	913.18	19709	916.57	23757	917.86	26164	918.67	30016	919.68
486.3	486.3	872.1	3139	897.75	6160	905.47	9555	909.78	13729	913.34	19709	916.76	23757	918.09	26164	918.92	30016	919.94
487	487	871.3	3139	897.97	6160	905.64	9555	910	13729	913.62	19709	917.14	23757	918.56	26164	919.45	30016	920.52
487.5	487.5	878.8	3139	898.19	6160	905.83	9555	910.2	13729	913.84	19709	917.38	23757	918.84	26164	919.72	30016	920.81
488.68	488.68	878.24	3139	898.83	6160	906.36	9555	910.79	13729	914.55	19709	918.19	23757	919.75	26164	920.67	30016	921.85

Table B15

Fargo Moorhead Metro Study - Phase 3.1
Reference, Year 50, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
279	403.65	832.3	7890	860.38	17020	869.46	24610	873.29	32650	875.42	44080	876.69	53560	877.55	63590	878.37	80610	879.31
280	404.98	836.6	7890	861.02	17020	870.16	24610	874.14	32650	876.09	44080	877.53	53560	878.34	63590	879.15	80610	880.13
281	406.51	834.9	7890	861.88	17020	871.03	24610	875.07	32650	877.15	44080	878.68	53560	879.33	63590	879.96	80610	881.01
282	410.21	839	7890	863.78	17020	872.89	24610	876.78	32650	878.79	44080	879.82	53560	880.48	63590	881.22	80610	882.37
283	413.23	837.6	7890	865.06	17020	874.26	24610	878.09	32650	880.03	44080	881.03	53560	881.43	63590	882.22	80610	883.43
284	414.96	842.7	7150	865.78	15860	875.14	23130	879.17	30790	880.91	41630	881.89	50580	882.45	60120	883.26	76860	884.49
285	415.38	841.4	7150	865.95	15860	875.34	23130	879.4	30790	881.11	41630	882.13	50580	882.67	60120	883.5	76860	884.74
286	415.84	843	7150	866.06	15860	875.42	23130	879.45	30790	881.14	41630	882.22	50580	882.82	60120	883.65	76860	885
287	415.90	844.1	7150	866.12	15860	875.48	23130	879.55	30790	881.37	41630	882.36	50580	882.96	60120	883.76	76860	885.06
288			22nd St SE or Cass Co. #34 / 170th Ave NW or Clay Co.36															
289	415.91	844.1	7150	866.12	15860	875.48	23130	879.54	30790	881.36	41630	882.46	50580	883.12	60120	884.16	76860	885.29
290	415.93	837.5	7150	866.14	15860	875.49	23130	879.55	30790	881.37	41630	882.55	50580	883.23	60120	884.16	76860	885.33
291	417.10	840.8	7150	866.55	15860	875.94	23130	880.11	30790	882.14	41630	883.45	50580	884.24	60120	885	76860	885.96
292	418.18	844.4	6247	866.96	13447	876.38	19446	880.64	25786	882.65	34892	884.04	42390	884.91	51664	885.65	67999	886.5
293	419.92	844.7	6247	867.73	13447	877.16	19446	881.37	25786	883.29	34892	884.7	42390	885.61	51664	886.15	67999	887.09
294	420.32	843.8	6247	868	13447	877.39	19446	881.62	25786	883.61	34892	885.09	42390	886.05	51664	886.61	67999	887.57
295	421.93	845.3	6247	869.23	13447	878.47	19446	882.62	25786	884.4	34892	885.84	42390	886.76	51664	887.44	67999	888.19
296	422.69	847.4	6247	869.55	13447	878.8	19446	882.92	25786	884.65	34892	886.06	42390	886.97	51664	887.73	67999	888.51
297	423.95	849.5	6247	870.08	13447	879.37	19446	883.58	25786	885.52	34892	887.14	42390	888.17	51664	888.91	67999	889.59
298	425.51	848.7	6247	870.82	13447	880.09	19446	884.3	25786	886.22	34892	887.88	42390	888.93	51664	889.3	67999	890.09
299	426.87	851.1	6247	871.54	13447	880.72	19446	884.94	25786	886.92	34892	888.68	42390	889.78	51664	890.02	67999	890.68
300	428.06	849.6	4008	872.07	9978	881.3	15017	885.53	19746.67	887.51	22353.59	889.25	25377.91	890.37	32964.36	890.36	48752.31	891.19
301	428.74	850.3	4008	872.33	9978	881.6	15017	885.89	19746.67	887.94	22353.59	889.59	25377.91	890.69	32964.36	890.89	48752.31	891.56
302	430.23	852	4008	872.9	9978	882.11	15017	886.47	19746.67	888.68	22353.59	890.31	25377.91	891.4	32964.36	892	48752.31	892.3
303	431.94	853.5	4008	873.65	9978	882.84	15017	887.26	19931.73	889.53	25305.74	891.14	30661.32	892.24	38065.91	893.19	54130.79	893.43
303.4	432.07	853.42	3722	873.69	9517	882.9	14431	887.36	18629.62	889.73	23947.99	891.22	28996.08	892.3	35992.23	893.37	51640.3	893.78
303.5	432.10	853.46	3722	873.72	9517	882.94	14431	887.4	18435.61	889.77	23757.49	891.28	28854.42	892.38	35590.23	893.47	50785.38	893.98
303.8	432.53	854.5	3722	873.88	9517	883.11	14431	887.57	17804.72	889.97	23241.29	891.56	28492.7	892.78	34907.98	893.89	48157.75	894.69
304	432.84	855.5	3723	873.96	9518	883.21	14420.55	887.66	17424	890.05	21106.97	891.73	23149.58	892.87	26500.48	893.99	35271.27	894.84
305	433.26	854.4	3723	874.11	9518	883.38	14420.55	887.85	17747.64	890.21	19411.48	891.87	21773.76	892.97	23941.6	894.08	30655.17	894.97
306	433.35	856	3723	874.18	9518	883.46	14420.55	887.95	17826.32	890.33	19006.97	892	21447.1	893.01	23333.11	894.11	29558.07	895
307			76th Ave. No. (Fargo) / 90th Ave NW (Moorhead) Bridge															
308	433.37	855	3723	874.19	9518	883.47	14420.55	887.97	17826.32	890.35	19006.97	892.01	21447.1	893.05	23333.11	894.16	29558.07	895.07
309	433.83	854.5	3723	874.33	9518	883.62	14420.55	888.15	17826.32	890.56	19006.97	892.21	21447.1	893.22	23333.11	894.31	29558.07	895.27
310	434.61	852.8	3723	874.56	9518	883.85	14420.55	888.4	17826.32	890.81	19006.97	892.43	21447.1	893.42	23333.11	894.49	29558.07	895.49
310.5	435.70	855	3723	874.9	9518	884.22	14420.55	888.77	17826.32	891.21	19006.97	892.82	21447.1	893.82	23333.11	894.87	29558.07	895.96
311	436.34	855	3723	875.11	9518	884.46	14383.68	889.06	17366.53	891.56	21464.23	893.13	22720.87	894.16	24054.21	895.21	31326.24	896.3
312	437.05	863.83	3528	875.34	9208	884.73	13978.68	889.36	16946.53	891.92	21071.17	893.55	23144.82	894.56	25426.5	895.6	33917.35	896.69
313	437.95	856	3527	875.85	9207	885.11	13989.13	889.82	17327.25	892.41	23148.55	894.14	27598.99	895.15	31881.74	896.19	43587.72	897.51
314	438.78	857	3527	875.97	9207	885.46	13989.13	890.21	17327.25	892.82	23148.55	894.72	27598.99	895.82	31881.74	896.92	43587.72	898.45
315	439.05	856	3527	876.06	9207	885.57	13989.13	890.33	17327.25	892.95	23148.55	894.94	27598.99	896.08	31881.74	897.21	43587.72	898.77
316	439.46	855.1	3527	876.2	9207	885.76	13989.13	890.59	17327.25	893.25	21617.13	895.4	24018.56	896.68	22677.43	897.94	24179.85	899.95
317			40th Ave. No. (Fargo) / Wall Street NW (Moorhead) Bridge															
318	439.47	855.1	3527	876.21	9207	885.77	13989.13	890.59	17327.25	893.25	21617.13	895.39	24018.56	896.69	22677.43	897.98	24179.85	899.98
319	439.50	857	3527	876.22	9207	885.82	13989.13	890.66	17327.25	893.32	21617.13	895.46	24018.56	896.76	22677.43	898.07	24179.85	900.02
320	439.66	856	3527	876.27	9207	885.88	13989.13	890.72	17327.25	893.38	21617.13	895.53	24018.56	896.84	22677.43	898.12	24179.85	900.07
321	440.31	858	3527	876.48	9207	886.13	13989.13	891.01	17327.25	893.7	21617.13	895.91	24018.56	897.23	22677.43	898.35	24179.85	900.23
322	440.32	858.1	3527	876.48	9207	886.12	13989.13	891	17327.25	893.69	21617.13	895.9	24018.56	897.22	22677.43	898.35	24179.85	900.23
323			Broadway Street No. (Fargo) / Broadway Street NW (Moorhead) Bridge															
324	440.33	858.1	3527	876.49	9207	886.24	13989.13	891.09	17327.25	893.75	21617.13	895.95	24018.56	897.26	22677.43	898.38	24179.85	900.25
325	440.34	857	3527	876.49	9207	886.25	13989.13	891.1	17327.25	893.76	21617.13	895.95	24018.56	897.26	22677.43	898.38	24179.85	900.25
326	440.54	856	3527	876.54	9207	886.3	13989.13	891.15	17327.25	893.82	21617.13	896.03	24018.56	897.34	22677.43	898.44	24179.85	900.29
327	441.53	859	3507	876.9	9162	886.58	13929.13	891.4	17262.25	894.06	21536.03	896.32	23967.18	897.62	22682.97	898.63	24526.17	900.4
328	442.14	858	3507	877.11	9162	886.75	13929.13	891.55	16181.65	894.2	20972.32	896.49	27250.07	897.8	30523.76	898.77	47392.84	900.48
329	442.45	857	3507	877.19	9162	886.78	13929.13	891.56	18725.21	894.21	25353.2	896.5	30858.57	897.81	37506.85	898.8	54034	900.53
329.5	442.93	858.06	3506	877.33	9161	886.89	13965	891.65	18855	894.29	25764	896.59	31304	897.93	38787	898.87	54034	900.6
330	443.69	858	3506	877.57	9161	887.09	13965	891.82	18855	894.46	25764	896.77	31304	898.11	38787	899.01	54034	900.75
331	443.99	859	3506	877.65	9161	887.18	13965	891.93	18855	894.59	25764	896.96	31304	898.36	38787	899.14	54034	900.89
332	444.33	856	3506	877.71	9161	887.22	13965	891.97	18855	894.64	25764	897.02	31304	898.43	38787	899.19	54034	900.94
333	444.92	859	3506	877.8	9161	887.26	13965	892	18855	894.69	25764	897.02	31304	898.44	38787	899.21	54034	900.98
334	445.18	860	3506	877.88	9161	887.34	13965	892.07	18855	894.75	25764	897.13	31304	898.53	38787	899.33	54034	901.13
335	445.31	860	3506	877.93	9161	887.42	13965	892.19	18855	894.92	25764	897.4	31304	898.86	38787	899.52	54034	901.3
336	445.36	859	3506	877.94	9161	887.43	13965	892.2	18855	894.94	25764	897.42	31304	898.9	38787	899.54	54034	901.31
336.5	445.76	857.69	3506	877.98	9161	887.46	13965	892.23	18855	894.97	25764	897.46	31304	898.95	38787	899.56	54034	901.36
337	446.09	860	3506	878.03	9161	887.46	13965	892.22	18855	894.96	25764	897.44	31304	898.93	38787	899.57	54034	901.35
338	446.29	861	3506	878.1	9161	887.5	13965	892.24	18855	894.97	25764	897.45	31304	898.93	38787	899.61	54034	901.41
338.5	446.70	860.9	3506	878.25	9161	887.64	13965	892.38	18855	895.13	25764	897.68	31304	899.15	38787	899.72	54034	901.61
339	446.76	861	3506	878.27	9161	887.66	13965	892.39	18855	895.14	25764	897.67	31304	899.21	38787	899.8	54034	901.7
340	447.04	860	3506	878.36	9161	887.75	13965	892.52	18855	895.32	25764	897.93	31304	899.53	38787	900.3	54034	902.06
341	447.24	859	3506	878.44	9161	887.81	13965	892.56	18855	895.36	25764	897.97	31304	899.57	38787	900.34	54034	902.12
342	447.42	861	3506	878.56	9161	887.9	13965	892.64	18855	895.45	25764	898.08	31304	899.68	38787	900.5	54034	902.31
342.5	447.78	856.99	3506	878.71	9161	888.04	13965	892.79	18855	895.62	25764	898.32	31304	899.99	38787	900.74	54034	902.59
343	448.13	861	3506	878.83	9161	888.14	13965	892.89	18855	895.74	25764	898.45	31304	900.13	38787	900.91	54034	902.8
344	448.36	859	3506	878.94	9161	888.25	13965	893.02	18855	895.92	25764	898.73	31304	900.47	38787	901.29	54034	903.23
345	448.89	865.3	3506	879.32	9161	888.48	13965	893.21	18855	896.11	25764	898.94	31304	900.7	38787	901.53	54034	903.5
348	448.90	864.3	3506	879.32	9161	888.46	13965	893.18	18855	896.07	25764	898.88	31304	900.64	38787	901.46	54034	903.46
348.4			North Dam															
348.8	448.90	864.3	3506	879.42	9161	888.51	13965	893.23	18855	896.14	25764	898.95	31304	900.68	38787	901.51	54034	903.49
351	448.91	861	3506	879.44	9161	888.51	13965	893.23	18855	896.14	25764	898.95	31304	900.68	38787	901.52	54034	903.5
351.4	449.06	861	3506	879.51	9161	888.64	13965	893.4	18855	896.35	25764	899.23	31304	901.04	38787	901.91	54034	903.93
351.5			12th Ave. No. (Fargo) / 15th Ave No. (Moorhead) Bridge															
351.6	449.07	861	3506	879.52	9161	888.7	13965	893.52	18855	896.44	25764	899.29	31304	901.1	38787	902.01	54034	904.06
352	449.09	862	3506	879.53	9161	888.72	13965	893.55	18855	896.48	25764	899.34	31304	901.14	38787	902.04	54034	904.06
353	449.28	863	3506	879.61	9161	888.81	13965	893.66	18855	896.62	25764	899.54	31304	901.35	38787	902.31	54034	904.39
353.5	449.61	860.13	3506	879.75	9161	888.91	13965	893.72	18855	896.69	25764	899.62	31304	901.45	38787	902.45	54034	904.57
354	449.97	864	3506	879.99	9161	889.13	13965	893.96	18855	896.92	25764	899.86	31304	901.7	38787	902.7	54034	904.82
355	450.19	862	3506	880.16	9161	889.22	13965	894.07	18855	897.06	25764	900.03	31304	901.88	38787	902.92	54034	905.04
356	450.84	858	3506	880.54	9161	889.5	13965	894.28	18855	897.28	25764	900.27	31304	902.12	38787	903.19	54034	905.35
357	450.86	863.4	3506	880.55	9161	889.5	13965	894.28	18855	897.28	25764	900.28	31304	902.13	38787	903.18	54034	905.3
358			BNRR Bridge															
359	450.87	863.4	3506	880.62	9161	889.6	13965	894.34	18855	897.38	25764	900.46	31304	902.38	38787	903.59	54034	905.93
361	451.20	865	3506	880.77	9161	889.72	13965	894.46	18855	897.51	25764	900.62	31304	902.56	38787	903.84	54034	906.31
361.5	451.37	865.2	3506	880.86	9161	889.79	13965	894.53	18855	897.58	25764	900.7	31304	902.65	38787	903.95	54034	906.45
362.5			1st Ave. No. Bridge															
364	451.39	865.2	3506	880.9	9161	889.81	13965	894.54	18855	897.61	25764	900.74	31304	902.82	38787	904.03	54034	906.64
366	451.41	861	3506	880.94	9161	889.86	13965	894.58	18855	897.65	25764	900.77	31304	902.85	38787	904.04	54034	906.64
367	451.57	867	3506	881	9161	889.95	13965	894.74	18855	897.88	25764	901.11	31304	903.26	38787	904.51	54034	907.11
368	451.61	866.8	3506	881.04	9161	890	13965	894.79	18855	897.93	25764	901.17	31304	903.33	38787	904.56	54034	907.16
369			NP Ave. (Fargo) / Center Ave. (Moorhead)															
370	451.63	866.8	3506	881.05	9161	890.01	13965	894.8	18855	897.97	25764	901.23	31304	903.43	38787	904.67	54034	907.22
371	451.65	859	3506	881.08	9161	890.01	13965	894.79	18855	897.95	25764	901.2	31304	903.4	38787	904.62	54034	907.17
372	451.67	867.9	3506	881.07	9161	890	13965	894.78	18855	897.95	25764	901.2	31304	903.4	38787	904.62	54034	907.13
373			BNRR Bridge															
374	451.68	867.9	3506	881.17	9161	890.1	13965	894.88	18855	898.01	25764	901.26	31304	903.44	38787	904.71	54034	907.45
375	451.69	864	3506	881.19	9161	890.1	13965	894.87	18855	898	25764	901.24	31304	903.42	38787	904.7	54034	907.55
376	451.71	865.9	3506	881.19	9161	890.11	13965	894.92	18855	898.09	25764	901.38	31304	903.61	38787	904.96	54034	907.77
377			Main Ave. Bridge															
378	451.72	865.9	3506	881.21	9161	890.12	13965	894.93	18855	898.11	25764	901.41	31304	903.65	38787	905.02	54034	907.87
379	451.73	864	3506	881.29	9161	890.22	13965	895.03	18855	898.2	25764	901.49	31304	903.73	38787	905.08	54034	907.89
380	451.85	864	3506	881.28	9161	890.19	13965	894.99	18855	898.17	25764	901.47	31304	903.71	38787	905.13	54034	907.95
381	452.03	863	3506	881.38	9161	890.34	13965	895.17	18855	898.36	25764	901.68	31304	903.93	38787	905.26	54034	908.07
381.5	452.04	863	3506	881.39	9161	890.35	13965	895.18	18855	898.39	25764	901.7	31304	903.95	38787	905.28	54034	908.08
384			Midtown Dam															
385.5	452.05	867	3506	881.46	9161	890.36	13965	895.19	18855	898.39	25764	901.71	31304	903.96	38787	905.29	54034	908.09
386	452.06	867	3506	881.47	9161	890.36	13965	895.19	18855	898.39	25764	901.72	31304	903.96	38787	905.29	54034	908.09
387	452.25	867	3506	881.61	9161	890.43	13965	895.25	18855	898.45	25764	901.77	31304	904.04	38787	905.37	54034	908.18
388	452.46	869	3506	881.83	9161	890.54	13965	895.35	18855	898.56	25764	901.92	31304	904.17	38787	905.5	54034	908.31
389	452.70	866.08	3506	882.1	9161	890.72	13965	895.51	18855	898.73	25764	902.1	31304	904.36	38787	905.73	54034	908.51
390	452.94	857	3506	882.23	9161	890.81	13965	895.59	18855	898.82	25764	902.21	31304	904.47	38787	905.85	54034	908.63
391	453.16	867	3506	882.35	9161	890.92	13965	895.7	18855	898.94	25764	902.34	31304	904.61	38787	906.03	54034	908.81
392	453.39	865	3506	882.48	9161	891.01	13965	895.8	18855	899.06	25764	902.49	31304	904.79	38787	906.2	54034	908.94
393	454.02	866	3506	882.83	9161	891.28	13965	896	18855	899.25	25764	902.67	31304	904.97	38787	906.38	54034	909.11
394	454.10	865	3506	882.87	9161	891.32	13965	896.04	18855	899.28	25764	902.7	31304	904.99	38787	906.41	54034	909.14
395	454.75	864	3506	883.14	9161	891.55	13965	896.25	18855	899.5	25764	902.92	31304	905.21	38787	906.63	54034	909.34
396	455.16	866	3506	883.33	9161	891.76	13965	896.45	18855	899.71	25764	903.09	31304	905.36	38787	906.78	54034	909.47
397	455.20	866.6	3506	883.37	9161	891.8	13965	896.48	18855	899.74	25764	903.14	31304	905.39	38787	906.79	54034	909.45
399			I-94 Bridge															
401	455.23	866.6	3506	883.38	9161	891.8	13965	896.48	18855	899.74	25764	903.14	31304	905.39	38787	906.82	54034	909.5
402	455.28	865	3506	883.41	9161	891.82	13965	896.5	18855	899.76	25764	903.16	31304	905.42	38787	906.92	54034	909.7
402.5	455.52	864.58	3506	883.52	9161	891.94	13965	896.61	18855	899.88	25764	903.31	31304	905.56	38787	907.07	54034	909.83
403	455.59	866	3506	883.54	9161	891.98	13965	896.65	18855	899.91	25764	903.34	31304	905.64	38787	907.12	54034	909.86
404	455.92	865	3506	883.7	9161	892.1	13965	896.78	18855	900.08	25764	903.52	31304	905.79	38787	907.23	54034	909.94
405	456.65	865	3506	884.09	9161	892.38	13965	897.01	18855	900.29	25764	903.7	31304	905.95	38787	907.38	54034	910.07
406	457.09	866	3506	884.33	9161	892.57	13965	897.19	18855	900.48	25764	903.92	31304	906.17	38787	907.57	54034	910.23

Table B15

Fargo Moorhead Metro Study - Phase 3.1
Reference, Year 50, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
407	457.27	861.18	3506	884.4	9161	892.64	13965	897.24	18855	900.53	25764	903.95	31304	906.19	38787	907.6	54034	910.27
408	457.80	864	3506	884.57	9161	892.86	13965	897.49	18855	900.81	25764	904.29	31304	906.5	38787	907.9	54034	910.52
409	457.95	855.9	3506	884.62	9161	892.96	13965	897.59	18855	900.91	25764	904.38	31304	906.57	38787	907.97	54034	910.57
409.1	457.97	855.9	3506	884.62	9161	892.96	13965	897.59	18855	900.91	25764	904.38	31304	906.58	38787	907.98	54034	910.58
409.5			South Dam															
409.6	457.98	855.9	3506	884.7	9161	893	13965	897.63	18855	900.94	25764	904.41	31304	906.59	38787	907.99	54034	910.59
409.7	457.99	855.9	3506	884.7	9161	893	13965	897.64	18855	900.95	25764	904.42	31304	906.6	38787	908	54034	910.6
413	458.32	869	3506	884.7	9161	893.02	13965	897.67	18855	900.99	25764	904.46	31304	906.66	38787	908.08	54034	910.7
414	458.75	866	3506	884.96	9161	893.29	13965	897.96	18855	901.31	25764	904.83	31304	906.9	38787	908.32	54034	910.88
459.64	459.64	864.81	3506	885.35	9161	893.73	13965	898.33	18855	901.67	25764	905.17	31304	907.19	38787	908.61	54034	911.14
460.28	460.28	868.74	3506	885.73	9161	894.07	13965	898.65	18855	901.97	25764	905.45	31304	907.44	38787	908.86	54034	911.33
460.72	460.72	869.04	3506	886.06	9161	894.34	13965	898.91	18855	902.26	25764	905.78	31304	907.66	38787	909.06	54034	911.49
461.3	461.3	868.04	3506	886.45	9161	894.73	13965	899.29	18855	902.67	25764	906.22	31304	907.98	38787	909.35	54034	911.7
462.03	462.03	864.16	3506	886.79	9161	895.12	13965	899.68	18855	903.04	25764	906.62	31304	908.36	38787	909.63	54034	911.91
462.04	462.04	869.5	3506	886.83	9161	895.16	13965	899.7	18855	903.06	25764	906.68	31304	908.4	38787	909.64	54034	911.92
462.05			52nd Avenue So. (Fargo) / 60th Ave So. (Moorhead) Bridge															
462.06	462.06	869.5	3506	886.84	9161	895.17	13965	899.72	18855	903.08	25764	906.71	31304	908.43	38787	909.83	54034	912.07
462.07	462.07	871.5	3506	886.68	9161	895.2	13965	899.75	18855	903.12	25764	906.7	31304	908.41	38787	909.83	54034	912.07
462.7	462.7	862.82	3506	887.24	9161	895.53	13965	900.06	18855	903.43	25764	907.01	31304	908.77	38787	910.1	54034	912.3
463.7	463.7	864.71	3506	887.56	9161	895.93	13965	900.49	18855	903.87	25764	907.46	31304	909.23	38787	910.46	54034	912.62
464.2	464.2	869.2	3506	887.8	9161	896.21	13965	900.79	18855	904.19	25764	907.82	31304	909.58	38787	910.72	54034	912.8
464.6	464.6	865	3506	888	9161	896.4	13965	900.98	18855	904.36	25764	907.99	31304	909.68	38787	910.84	54034	912.91
465.1	465.1	862.9	3506	888.15	9161	896.63	13965	901.26	18855	904.72	25764	908.37	31304	910.02	38787	911.04	54034	913.08
465.9	465.9	865.6	3506	888.41	9161	896.98	13965	901.64	18855	905.12	25764	908.68	31304	910.21	38787	911.2	54034	913.2
466.5	466.5	864.9	3506	888.63	9161	897.25	13965	901.93	18855	905.43	25764	908.99	31304	910.51	38787	911.4	54034	913.38
467.4	467.4	864.8	3506	888.95	9161	897.66	13965	902.36	18855	905.9	25764	909.46	31304	910.8	38787	911.67	54034	913.6
468.2	468.2	862.6	3506	889.18	9161	897.94	13965	902.65	18855	906.17	25764	909.68	31304	911	38787	911.82	54034	913.74
468.9	468.9	861	3506	889.39	9161	898.22	13965	902.96	18855	906.43	25764	909.86	31304	911.14	38787	911.97	54034	913.88
469.7	469.7	870	3506	889.73	9161	898.61	13965	903.32	18855	906.72	25764	910.02	31304	911.26	38787	912.1	54034	914
470.1	470.1	870.9	3506	889.88	9161	898.79	13965	903.51	18855	906.91	25764	910.15	31304	911.38	38787	912.18	54034	914.07
470.6	470.6	869.4	2548	890.04	6358	898.99	8774	903.7	11434	907.1	15609	910.32	19019	911.55	25243	912.38	36927	914.29
471.4	471.4	867.5	2548	890.27	6358	899.23	8774	903.9	11434	907.28	15609	910.44	19019	911.65	25243	912.52	36927	914.44
472.18	472.18	869.9	2548	890.5	6358	899.47	8774	904.1	11434	907.46	15609	910.59	19019	911.79	25243	912.7	36927	914.63
472.2	472.2	870.8	2548	890.51	6358	899.47	8774	904.09	11434	907.45	15609	910.57	19019	911.77	25243	912.66	36927	914.6
472.22	472.22	870.8	2548	890.51	6358	899.47	8774	904.11	11434	907.47	15609	910.62	19019	911.84	25243	912.77	36927	914.72
473	473	871.8	2548	890.74	6358	899.69	8774	904.27	11434	907.62	15609	910.74	19019	911.97	25243	912.93	36927	914.89
473.8	473.8	869	2548	890.99	6358	899.95	8774	904.51	11434	907.83	15609	910.94	19019	912.18	25243	913.19	36927	915.19
474.08	474.08	873.5	2548	891.09	6358	900.05	8774	904.6	11434	907.91	15609	911.02	19019	912.27	25243	913.29	36927	915.28
474.09	474.09	873.5	2548	891.1	6358	900.07	8774	904.62	11434	907.95	15609	911.06	19019	912.31	25243	913.35	36927	915.31
474.1			Cass Co Hwy 16/112th Ave So. (Fargo) / 110th Ave So. (Moorhead) Bridge															
474.11	474.11	874	2548	891.1	6358	900.07	8774	904.62	11434	907.94	15609	911.05	19019	912.33	25243	913.39	36927	915.54
474.12	474.12	874	2548	891.11	6358	900.07	8774	904.62	11434	907.95	15609	911.07	19019	912.36	25243	913.41	36927	915.55
474.6	474.6	873.4	2548	891.32	6358	900.23	8774	904.75	11434	908.07	15609	911.21	19019	912.5	25243	913.58	36927	915.72
475.3	475.3	872.2	2548	891.56	6358	900.49	8774	904.97	11434	908.31	15609	911.47	19019	912.8	25243	913.95	36927	916.04
475.7	475.7	871	2548	891.69	6358	900.64	8774	905.12	11434	908.48	15609	911.68	19019	913.06	25243	914.21	36927	916.19
476.4	476.4	873.7	2548	891.96	6358	900.87	8774	905.31	11434	908.66	15609	911.86	19019	913.24	25243	914.42	36927	916.36
477	477	874	2550	892.23	5485	901.09	8718	905.53	12582	908.89	18134	912.08	22123	913.48	24258	914.66	29236	916.58
477.6	477.6	870.2	2550	892.42	5366	901.25	8710	905.73	12762	909.15	18543	912.45	22626	913.86	24116	914.97	28246	916.81
478	478	873.4	2550	892.55	5366	901.35	8710	905.86	12762	909.3	18543	912.64	22626	914.04	24116	915.12	28246	916.86
478.8	478.8	874.8	2550	892.79	5366	901.54	8710	906.1	12762	909.6	18543	913.03	22626	914.46	24116	915.45	28246	917.11
479.2	479.2	875.7	2550	892.9	5366	901.63	8710	906.21	12762	909.73	18543	913.19	22626	914.6	24116	915.58	28246	917.18
479.7	479.7	871.5	2550	893.1	5366	901.75	8710	906.35	12762	909.9	18543	913.38	22626	914.81	24116	915.76	28246	917.29
480.5	480.5	874.7	2550	893.38	5366	901.96	8710	906.61	12762	910.2	18543	913.7	22626	915.14	24116	916.04	28246	917.53
481.8	481.8	875.3	2550	893.76	5366	902.3	8710	907.05	12762	910.66	18543	914.19	22626	915.69	24116	916.5	28246	917.93
482.3	482.3	874.2	2550	893.97	5366	902.48	8710	907.28	12762	910.94	18543	914.47	22626	915.99	24116	916.73	28246	918.09
482.698	482.698	869.74	2550	894.07	5366	902.57	8710	907.4	12762	911.07	18543	914.63	22626	916.16	24116	916.91	28246	918.24
482.7	482.7	887.04	2550	893.94	5366	902.51	8710	907.33	12762	911	18543	914.57	22626	916.1	24116	916.88	28246	918.23
482.702	482.702	878.54	2550	894.12	5366	902.58	8710	907.4	12762	911.08	18543	914.62	22626	916.16	24116	916.91	28246	918.24
483.1	483.1	879.2	2550	894.35	5366	902.78	8710	907.64	12762	911.36	18543	914.93	22626	916.51	24116	917.16	28246	918.41
483.9	483.9	873	2550	894.81	5366	903.07	8710	907.96	12762	911.75	18543	915.4	22626	916.94	24116	917.59	28246	918.76
485.08	485.08	880.5	2550	895.47	5366	903.38	8710	908.21	12762	912	18543	915.68	22626	917.23	24116	917.88	28246	919.03
485.09	485.09	880.5	2550	895.47	5366	903.37	8710	908.19	12762	912	18543	915.66	22626	917.23	24116	917.86	28246	919.03
485.1			Cass Co. Hwy 18/52nd St. SE (Fargo) / 160th Ave So. (Moorhead) Bridge															
485.11	485.11	879.4	2550	895.48	5366	903.38	8710	908.23	12762	912.11	18543	915.71	22626	917.28	24116	917.9	28246	919.06
485.12	485.12	879.4	2550	895.49	5366	903.4	8710	908.29	12762	912.12	18543	915.72	22626	917.28	24116	917.9	28246	919.07
485.6	485.6	878.62	2550	895.71	5366	903.52	8710	908.39	12762	912.2	18543	915.8	22626	917.37	24116	917.99	28246	919.16
485.9	485.9	879.5	2550	895.92	5366	903.65	8710	908.53	12762	912.34	18543	915.96	22626	917.55	24116	918.18	28246	919.36
486.3	486.3	872.1	2550	896.09	5366	903.77	8710	908.67	12762	912.5	18543	916.15	22626	917.78	24116	918.41	28246	919.62
487	487	871.3	2550	896.32	5366	903.95	8710	908.88	12762	912.77	18543	916.5	22626	918.22	24116	918.86	28246	920.16
487.5	487.5	878.8	2550	896.52	5366	904.15	8710	909.08	12762	912.99	18543	916.74	22626	918.49	24116	919.15	28246	920.44
488.68	488.68	878.24	2550	897.12	5366	904.69	8710	909.67	12762	913.69	18543	917.53	22626	919.36	24116	920.04	28246	921.42

Table B16

Fargo Moorhead Metro Study - Phase 3.1
Reference, Wet - Year Zero, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			0.1%		0.01%	
Recurrence Interval			1,000 -Year		10,000 -Year	
River Sta	River Mile	Invert, Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
279	403.65	832.3	98859.99	880.12	147070	881.65
280	404.86	836.8	98859.99	881.03	147070	882.87
281	406.51	834.9	98859.99	881.98	147070	884.03
282	410.21	838	98859.99	883.49	147070	885.77
283	413.23	837.8	98859.99	884.56	147070	887.02
284	414.98	842.7	95529.99	885.65	143770	888.19
285	415.38	841.4	95529.99	885.91	143770	888.49
286	415.94	842	95529.99	886.2	143770	888.81
287	415.96	844.1	95529.99	886.76	143770	888.86
288						22nd St SE or Cass Co. #64 / 170th Ave NW or Clay Co 36
289	415.91	844.1	95529.99	886.52	143770	889.1
290	415.90	837.5	95529.99	886.56	143770	889.13
291	417.10	840.9	95529.99	887.15	143770	889.67
292	418.18	844.4	85679.99	887.7	133320	890.27
293	419.92	844.7	85679.99	888.27	133320	890.85
294	420.32	843.9	85679.99	888.71	133320	891.28
295	421.83	845.3	85679.99	889.33	133320	891.9
296	422.69	847.4	85679.99	889.7	133320	892.35
297	423.95	849.1	85679.99	890.69	133320	893.102
298	425.51	848.7	85679.99	891.21	133320	893.58
299	426.87	851.1	85679.99	891.74	133320	894.06
300	429.06	849.9	68020.64	892.93	123420	894.47
301	428.74	850.3	68030.64	892.83	123420	894.78
302	430.23	852	68030.64	893.75	123420	895.33
303	431.94	852.5	71326.44	895.03	123420	896.02
303.4	432.07	853.42	69366.71	894.7		
303.5	432.10	853.66	68873.55	895		
303.6	432.53	854.5	67268.64	895.44		
304	432.84	853.5	47592.07	896.15	121655	896.74
305	433.26	854.4	40305.18	896.31	121655	897.34
306	433.35	855	38570.34	896.35	121655	897.47
307						76th Ave. No. (Fargo) / 80th Ave NW (Moorhead) Bridge
308	433.37	855	38570.34	896.42	121655	897.55
309	433.83	854.5	38570.34	896.68	121655	898.03
310	434.81	852.8	38570.34	896.99	121655	898.59
310.5	435.70	855	38570.34	897.53	121655	899.22
311	436.34	850	29706.46	897.91	121655	901.66
312	437.05	863.83	46363.54	898.32	121100	902.5
314	437.95	858	54852.04	899.4	121100	903.4
314	438.78	857	54852.04	900.49	121100	904.03
315	439.06	858	54852.04	900.76	121100	904.29
318	439.46	855.1	33308.34	901.55	121100	904.8
317						40th Ave. No. (Fargo) / Wall Street NW (Moorhead) Bridge
318	439.47	855.1	33308.34	901.58	121100	904.82
319	439.50	857	33308.34	901.64	121100	904.87
320	439.66	858	33308.34	901.7	121100	905.1
321	440.31	858	33308.34	901.91	121100	905.36
322	440.32	853.1	33308.34	901.92	121100	905.38
323						Broadway Street No. (Fargo) / Broadway Street NW (Moorhead) Bridge
324	440.33	858.1	33308.34	901.93	121100	905.47
325	440.34	857	33308.34	901.94	121100	905.42
326	440.54	858	33308.34	901.97	121100	905.49
327	441.53	859	33543.25	902.11	121000	905.76
328	442.14	858	52469.98	902.2	121000	905.92
329	442.45	857	68967.61	902.24	121000	905.99
329.5	442.93	858.06	74000	902.3	121000	906.06
330	443.89	858	74000	902.47	121000	906.24
331	443.99	859	74000	902.62	121000	906.39
332	444.33	858	74000	902.68	121000	906.44
333	444.92	859	74000	902.74	121000	906.52
334	445.18	860	74000	902.9	121000	906.89
335	445.31	860	74000	903.07	121000	906.83
336	445.36	859	74000	903.08	121000	906.84
336.5	445.76	857.88	74000	903.14	121000	906.91
337	446.09	860	74000	903.14	121000	906.92
338	446.29	861	74000	903.21	121000	907.01
338.5	446.70	860.98	74000	903.45	121000	907.27
339	446.76	861	74000	903.53	121000	907.34
340	447.04	860	74000	903.91	121000	907.69
341	447.24	859	74000	903.98	121000	907.79
342	447.42	861	74000	904.18	121000	907.99
342.5	447.73	859.89	74000	904.5	121000	908.33
343	448.13	861	74000	904.75	121000	908.63
344	448.36	859	74000	905.2	121000	909.04
345	449.89	860.3	74000	905.51	121000	909.41
349	448.90	864.3	74000	905.5	121000	909.45
348.4						North Dam
349.8	448.90	864.3	74000	905.53	121000	909.46
351	448.91	861	74000	905.54	121000	909.47
351.4	449.06	861	74000	905.94	121000	909.8
351.5						12th Ave. No. (Fargo) / 15th Ave No. (Moorhead) Bridge
351.6	449.07	861	74000	906.07	121000	909.92
352	449.09	862	74000	906.07	121000	909.93
353	449.28	863	74000	906.41	121000	910.24
353.6	449.61	860.13	74000	906.64	121000	910.64
354	449.97	864	74000	906.88	121000	910.77
355	450.79	862	74000	907.11	121000	910.99
356	450.84	858	74000	907.46	121000	911.38
357	450.86	863.4	74000	907.39	121000	911.33
358						BNRR Bridge
359	450.87	865.2	74000	907.49	121000	911.6
361	451.20	865	74000	908.02	121000	912.15
361.5	451.37	865.2	74000	908.19	121000	912.32
362.5						1st Ave. No. Bridge
364	451.39	865.2	74000	908.46	121000	912.58
366	451.41	861	74000	908.47	121000	912.59
367	451.57	867	74000	908.95	121000	912.98
368	451.61	866.8	74000	909	121000	913.02
369						NP Ave. (Fargo) / Center Ave. (Moorhead)
370	451.63	866.8	74000	909.13	121000	913.16
371	451.65	859	74000	909.08	121000	913.12
372	451.67	867.9	74000	909.03	121000	915.07
373						BNRR Bridge
374	451.68	867.9	74000	909.35	121000	913.6
375	451.69	864	74000	909.52	121000	913.78
376	451.71	863.8	74000	909.73	121000	913.62
377						Main Ave. Bridge
378	451.72	865.9	74000	909.85	121000	914.08
379	451.73	864	74000	909.86	121000	914.08
380	451.85	864	74000	909.94	121000	914.19
381	452.03	863	74000	910.07	121000	914.31
381.5	452.04	863	74000	910.03	121000	914.32
384						Midtown Dam
385.5	452.05	867	74000	910.08	121000	914.33
386	452.06	867	74000	910.09	121000	914.33
387	452.25	867	74000	910.2	121000	914.47
388	452.46	868	74000	910.35	121000	914.62
389	452.70	866.08	74000	910.56	121000	914.83
390	452.84	867	74000	910.71	121000	914.89
391	453.16	867	74000	910.89	121000	915.16
392	453.39	865	74000	911.03	121000	915.29
393	454.02	868	74000	911.22	121000	915.49
394	454.10	865	74000	911.24	121000	915.52
395	454.75	864	74000	911.47	121000	915.76
396	455.16	868	74000	911.6	121000	915.89
397	455.20	868.6	74000	911.56	121000	915.85
398						I-94 Bridge
401	455.23	868.6	74000	911.71	121000	916.05
402	455.28	868	74000	911.92	121000	916.23
402.5	455.52	854.58	74000	912.05	121000	916.36
403	455.59	868	74000	912.08	121000	916.39
404	455.92	865	74000	912.16	121000	916.46
405	456.80	865	74000	912.3	121000	916.61
406	457.09	868	74000	912.46	121000	916.77

Table B16

Fargo Moorhead Metro Study - Phase 3.1
Reference, Wet - Year Zero, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			0.1%		0.01%	
Recurrence Interval			1,000 -Year		10,000 -Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
407	457.27	861.18	74000	912.51	121000	916.83
408	457.80	866	74000	912.76	121000	917.07
409	457.95	855.9	74000	912.8	121000	917.11
409.1	457.97	855.9	74000	912.81	121000	917.11
409.5						South Dam
409.6	457.98	855.9	74000	912.82	121000	917.13
409.7	457.98	855.9	74000	912.82	121000	917.14
413	458.32	869	74000	912.94	121000	917.26
414	458.75	866	74000	913.11	121000	917.43
459.64	459.64	864.81	74000	913.38	121000	917.69
460.28	460.28	868.74	74000	913.56	121000	917.87
460.72	460.72	869.04	74000	913.73	121000	918.02
461.3	461.3	868.04	74000	913.91	121000	918.18
462.03	462.03	864.16	74000	914.06	121000	918.33
462.04	462.04	869.5	74000	914.11	121000	918.33
462.05						52nd Avenue So. (Fargo) / 60th Ave So. (Moorhead) Bridge
462.06	462.06	869.5	74000	914.18	121000	918.39
462.07	462.07	871.5	74000	914.19	121000	918.4
462.7	462.7	862.82	74000	914.4	121000	918.59
463.7	463.7	864.71	74000	914.71	121000	918.88
464.2	464.2	869.2	74000	914.86	121000	919.88
464.6	464.6	865	74000	914.97	121000	919.08
465.1	465.1	862.9	74000	915.12	121000	919.21
465.9	465.9	865.6	74000	915.23	121000	919.3
466.5	466.5	864.9	74000	915.39	121000	919.44
467.4	467.4	864.8	74000	915.6	121000	919.62
468.2	468.2	862.6	74000	915.73	121000	919.74
468.9	468.9	861	74000	915.87	121000	919.87
469.7	469.7	870	74000	915.98	121000	919.97
470.1	470.1	870.9	74000	918.06	121000	920.03
470.6	470.6	869.4	52104	916.28	93574	920.27
471.4	471.4	867.5	52104	916.44	93574	920.45
472.18	472.18	869.9	52104	916.65	93574	920.69
472.2	472.2	870.8	52104	916.82	93574	920.87
472.22	472.22	870.8	52104	916.741	93574	920.78
473	473	871.9	52104	916.92	93574	921
473.8	473.8	869	52104	917.24	93574	921.36
474.08	474.08	873.5	52104	917.32	93574	921.44
474.09	474.09	873.5	52104	917.34	93574	921.44
474.1						Cass Co Hwy 16/112th Ave So. (Fargo) / 110th Ave So. (Moorhead) Bridge
474.11	474.11	874	52104	917.54	93574	921.53
474.12	474.12	874	52104	917.56	93574	921.55
474.6	474.6	873.4	52104	917.72	93574	921.7
475.3	475.3	872.7	52104	918.01	93574	922.02
475.7	475.7	871	52104	918.16	93574	922.13
476.4	476.4	873.7	52104	918.32	93574	922.29
477	477	874	39620	918.53	76876	922.48
477.9	477.9	870.2	35000	918.64	77714	922.6
478	478	873.4	35000	918.66	77714	922.69
478.8	478.8	874.8	35000	918.8	77714	922.85
479.2	479.2	875.7	35000	918.87	77714	922.99
479.7	479.7	871.9	35000	918.96	77714	923.1
480.5	480.5	874.7	35000	919.11	77714	923.3
481.8	481.8	875.3	35000	919.38	77714	923.62
482.3	482.3	874.2	35000	919.49	77714	923.68
482.698	482.698	869.74	35000	919.58	77714	923.76
482.7	482.7	887.04	35000	919.59	77714	923.77
482.702	482.702	878.54	35000	919.59	77714	923.77
483.1	483.1	879.2	35000	919.68	77714	923.84
483.9	483.9	873	35000	919.96	77714	924.06
485.08	485.08	880.5	35000	920.24	77714	924.36
485.09	485.09	880.5	35000	920.24	77714	924.36
485.1						Cass Co. Hwy 16/52nd St. SE (Fargo) / 160th Ave So. (Moorhead) Bridge
485.11	485.11	879.4	35000	920.28	77714	924.41
485.12	485.12	879.4	35000	920.28	77714	924.42
485.8	485.8	878.67	35000	920.38	77714	924.56
485.9	485.9	879.6	35000	920.57	77714	924.71
486.3	486.3	872.1	35000	920.86	77714	925.06
487	487	871.3	35000	921.5	77714	925.58
487.5	487.5	878.8	35000	921.82	77714	925.95
488.68	488.68	878.24	35000	923.01	77714	926.74

Table B17

Fargo Moorhead Metro Study - Phase 3.1
MN Short 35Kcfs, Year Zero - Wet, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
279	403.65	832.3	10380	863.05	20140	871.03	27860	874.18	36050.16	875.86	47720	877.05	57070	877.85	69040	878.81	85540	879.53
280	404.98	836.6	10380	863.75	20140	871.81	27860	874.78	36050.16	876.6	47720	877.78	57070	878.66	69040	879.62	85540	880.37
281	406.51	834.9	10380	864.68	20140	872.74	27860	875.86	36050.16	877.71	47720	878.82	57070	879.68	69040	880.42	85540	881.28
282	410.21	838	10380	866.68	20140	874.62	27860	877.84	36050.16	879.39	47720	880.1	57070	880.81	69040	881.67	85540	882.67
283	413.23	837.6	10380	868.04	20140	876	27860	879	36050.16	880.5	47720	881	57070	881.76	69040	882.67	85540	883.75
284	414.98	842.7	9640	868.81	19100	876.99	26500	880.15	34300.16	881.42	45430	882.01	54260	882.77	66170	883.71	82530	884.82
285	415.36	841.4	9640	868.99	19100	877.22	26500	880.37	34300.16	881.61	45430	882.29	54260	883	66170	883.95	82530	885.08
286	415.64	843	9640	869.12	19100	877.29	26500	880.42	34300.16	881.67	45430	882.39	54260	883.15	66170	884.2	82530	885.35
287	415.90	844.1	9640	869.17	19100	877.37	26500	880.55	34300.16	881.67	45430	882.54	54260	883.27	66170	884.18	82530	885.41
288			22nd St SE or Cass Co. #34 / 170th Ave NW or Clay Co.36															
289	415.91	844.1	9640	869.17	19100	877.36	26500	880.54	34300.16	881.89	45430	882.67	54260	883.47	66170	884.65	82530	885.65
290	416.93	837.8	9640	869.19	19100	877.38	26500	880.56	34300.16	881.9	45430	882.77	54260	883.58	66170	884.65	82530	885.69
291	417.10	840.9	9640	869.62	19100	877.88	26500	881.21	34300.16	882.69	45430	883.74	54260	884.6	66170	885.4	82530	886.31
292	418.18	844.4	8328	870.05	16039	878.37	22069	881.72	28491.16	883.21	37826	884.38	45160	885.2	56895	885.89	72930	886.86
293	419.92	844.7	8328	870.85	16039	879.19	22069	882.39	28491.16	883.83	37826	885.06	45160	885.67	56895	886.43	72930	887.43
294	420.32	843.8	8328	871.12	16039	879.42	22069	882.87	28491.16	884.16	37826	885.46	45160	886.16	56895	886.94	72930	887.88
295	421.93	845.3	8328	872.31	16039	880.5	22069	883.55	28491.16	884.92	37826	886.22	45160	886.93	56895	887.74	72930	888.51
296	422.69	847.4	8328	872.64	16039	880.62	22069	883.82	28491.16	885.15	37826	886.43	45160	887.18	56895	888.07	72930	888.84
297	423.95	849.5	8328	873.19	16039	881.43	22069	884.56	28491.16	886.08	37826	887.56	45160	888.33	56895	889.02	72930	889.9
298	425.51	848.7	8328	873.93	16039	882.17	22069	885.27	28491.16	886.79	37826	888.31	45160	889.05	56895	889.47	72930	890.41
299	426.87	851.1	8328	874.63	16039	882.64	22069	885.93	28491.16	887.53	37826	889.13	45160	889.67	56895	890.05	72930	890.98
300	428.06	849.6	6118	875.2	10378.13	883.41	10737.53	886.43	11044.33	888	9784.71	889.59	10398.12	890.21	14549.75	890.38	26556.14	891.37
301	428.74	850.3	6118	875.52	10378.13	883.64	10737.53	886.59	11044.33	888.13	9784.71	889.65	10398.12	890.26	14549.75	890.48	26556.14	891.48
302	430.23	852	6118	876.15	10378.13	884.04	10737.53	886.86	11044.33	888.36	9784.71	889.79	10398.12	890.4	14549.75	890.74	26556.14	891.75
303	431.94	853.5	6118	877	10378.13	884.62	10737.53	887.24	11044.33	888.67	10817.06	889.98	12896.99	890.6	17911.14	891.09	28630.37	892.52
303.4	432.07	853.42	5825	877.05	9954.13	884.68	10163.53	887.3	10274.01	888.73	9674.05	890.02	11210.65	890.65	16273.81	891.14	26579.11	892.58
303.5	432.10	853.46	5825	877.09	9954.13	884.71	10163.53	887.32	10296.43	888.75	9713.7	890.03	10932.06	890.66	16092.69	891.17	25836.66	892.64
303.8	432.53	854.5	5825	877.28	9954.13	884.84	10163.53	887.4	10296.43	888.81	9797.46	890.08	10226.65	890.73	15535.12	891.34	23565.5	892.94
304	432.84	855.5	5825	877.38	9955.13	884.92	10184.53	887.45	10271.75	888.85	9677.93	890.1	9973.35	890.75	16123.19	891.39	18725.59	892.99
305	433.26	854.4	5825	877.56	9955.13	885.06	10184.53	887.55	10271.75	888.93	10145.3	890.15	10478.9	890.79	13732.16	891.46	18749.24	893.06
306	433.35	855	5825	877.64	9955.13	885.13	10184.53	887.6	10271.75	888.98	10258.29	890.19	10598.92	890.83	13400.69	891.53	18754.87	893.08
307			76th Ave. No. (Fargo) / 90th Ave NW (Moorhead) Bridge															
308	433.37	855	5825	877.65	9955.13	885.14	10184.53	887.61	10271.75	888.98	10258.29	890.2	10598.92	890.83	13400.69	891.53	18754.87	893.12
309	433.83	854.5	5825	877.81	9955.13	885.27	10184.53	887.71	10271.75	889.06	10258.29	890.27	10598.92	890.9	13400.69	891.64	18754.87	893.25
310	434.61	852.8	5825	878.09	9955.13	885.46	10184.53	887.84	10271.75	889.17	10258.29	890.36	10598.92	890.99	13400.69	891.76	18754.87	893.39
310.5	435.70	855	5825	878.49	9955.13	885.75	10184.53	888.04	10271.75	889.34	10258.29	890.5	10598.92	891.13	13400.69	891.98	18754.87	893.71
311	436.34	855	5825	878.74	9955.13	885.96	10184.53	888.2	10257.49	889.48	10242	890.61	11189.51	891.25	14862.29	892.16	20004.9	893.97
312	437.05	863.83	5630	879.01	9675.13	886.19	9814.53	888.37	9862.49	889.62	9812.01	890.75	10749.51	891.41	14394.56	892.4	20158.79	894.29
313	437.95	856	5630	879.39	9674.13	886.53	9813.53	888.63	9889.18	889.84	9931.54	890.94	11002.81	891.62	14799.23	892.74	24334.91	894.77
314	438.76	857	5630	879.76	9674.13	886.84	9813.53	888.87	9889.18	890.04	9931.54	891.11	11002.81	891.81	14799.23	893.03	24334.91	895.34
315	439.05	856	5630	879.86	9674.13	886.93	9813.53	888.94	9889.18	890.1	9931.54	891.16	11002.81	891.87	14799.23	893.12	24334.91	895.56
316	439.46	855.1	5630	880.04	9674.13	887.12	9813.53	889.09	9889.18	890.23	9931.54	891.26	11002.81	892	14799.23	893.34	23141.39	896.03
317			40th Ave. No. (Fargo) / Wall Street NW (Moorhead) Bridge															
318	439.47	855.1	5630	880.05	9674.13	887.12	9813.53	889.09	9889.18	890.23	9931.54	891.26	11002.81	892	14799.23	893.34	23141.39	896.03
319	439.50	857	5630	880.07	9674.13	887.17	9813.53	889.13	9889.18	890.27	9931.54	891.31	11002.81	892.04	14799.23	893.39	23141.39	896.12
320	439.66	856	5630	880.13	9674.13	887.22	9813.53	889.17	9889.18	890.3	9931.54	891.34	11002.81	892.07	14799.23	893.43	23141.39	896.18
321	440.31	858	5630	880.38	9674.13	887.45	9813.53	889.35	9889.18	890.46	9931.54	891.47	11002.81	892.22	14799.23	893.66	23141.39	896.58
322	440.32	858.1	5630	880.36	9674.13	887.44	9813.53	889.34	9889.18	890.45	9931.54	891.47	11002.81	892.22	14799.23	893.66	23141.39	896.57
323			Broadway Street No. (Fargo) / Broadway Street NW (Moorhead) Bridge															
324	440.33	858.1	5630	880.4	9674.13	887.54	9813.53	889.41	9889.18	890.5	9931.54	891.51	11002.81	892.25	14799.23	893.7	23141.39	896.62
325	440.34	857	5630	880.41	9674.13	887.55	9813.53	889.41	9889.18	890.51	9931.54	891.51	11002.81	892.26	14799.23	893.7	23141.39	896.62
326	440.54	858	5630	880.48	9674.13	887.6	9813.53	889.45	9889.18	890.54	9931.54	891.54	11002.81	892.29	14799.23	893.75	23141.39	896.71
327	441.53	859	5600	880.84	9629.13	887.84	9758.53	889.62	9824.18	890.68	9871.54	891.66	10932.81	892.41	14724.56	893.93	23063.66	897.01
328	442.14	858	5600	881.05	9629.13	887.98	9758.53	889.72	9824.18	890.76	9871.54	891.73	10976.09	892.49	15395.08	894.04	25148.86	897.2
329	442.45	857	5600	881.11	9629.13	887.99	9758.53	889.73	9824.18	890.77	9871.54	891.73	11045.61	892.5	16190.31	894.05	26957	897.2
329.5	442.93	868.06	5600	881.26	9628.13	888.09	9757.53	889.8	9837.27	890.82	9909.81	891.77	11044.73	892.54	16248.62	894.11	27125.26	897.31
330	443.69	858	5600	881.49	9628.13	888.27	9757.53	889.93	9837.27	890.92	9909.81	891.86	11044.73	892.63	16248.62	894.24	27125.26	897.48
331	443.99	859	5600	881.58	9628.13	888.35	9757.53	890	9837.27	890.99	9909.81	891.91	11044.73	892.69	16248.62	894.35	27125.26	897.69
332	444.33	856	5600	881.62	9628.13	888.39	9757.53	890.03	9837.27	891.01	9909.81	891.93	11044.73	892.71	16248.62	894.38	27125.26	897.75
333	444.92	859	5600	881.69	9628.13	888.42	9757.53	890.05	9837.27	891.03	9909.81	891.95	11044.73	892.73	16248.62	894.4	27125.26	897.75
334	445.18	860	5600	881.76	9628.13	888.49	9757.53	890.1	9837.27	891.07	9909.81	891.99	11044.73	892.77	16248.62	894.47	27125.26	897.84
335	445.31	860	5600	881.85	9628.13	888.57	9757.53	890.17	9837.27	891.14	9909.81	892.04	11044.73	892.83	16248.62	894.6	27125.26	898.13
336	445.36	859	5600	881.85	9628.13	888.58	9757.53	890.18	9837.27	891.14	9909.81	892.05	11044.73	892.84	16248.62	894.61	27125.26	898.15
336.5	445.76	857.69	5600	881.89	9628.13	888.6	9757.53	890.2	9837.27	891.16	9909.81	892.06	11044.73	892.86	16248.62	894.64	27125.26	898.19
337	446.09	860	5600	881.91	9628.13	888.6	9757.53	890.19	9837.27	891.16	9909.81	892.06	11044.73	892.85	16248.62	894.63	27125.26	898.17
338	446.29	861	5600	881.98	9628.13	888.63	9757.53	890.21	9837.27	891.17	9909.81	892.07	11044.73	892.86	16248.62	894.64	27125.26	898.18
338.5	446.70	860.9	5600	882.13	9628.13	888.75	9757.53	890.31	9837.27	891.25	9909.81	892.14	11044.73	892.94	16248.62	894.76	27125.26	898.37
339	446.76	861	5600	882.15	9628.13	888.77	9757.53	890.32	9837.27	891.26	9909.81	892.15	11044.73	892.95	16248.62	894.77	27125.26	898.41
340	447.04	860	5600	882.24	9628.13	888.86	9757.53	890.39	9837.27	891.33	9909.81	892.22	11044.73	893.02	16248.62	894.91	27125.26	898.67
341	447.24	859	5600	882.33	9628.13	888.91	9757.53	890.43	9837.27	891.36	9909.81	892.24	11044.73	893.05	16248.62	894.94	27125.26	898.71
342	447.42	861	5600	882.44	9628.13	888.99	9757.53	890.49	9837.27	891.41	9909.81	892.28	11044.73	893.09	16248.62	895.01	27125.26	898.81
342.5	447.78	856.99	5600	882.6	9628.13	889.12	9757.53	890.59	9837.27	891.5	9909.81	892.36	11044.73	893.18	16248.62	895.15	27125.26	899.06
343	448.13	861	5600	882.72	9628.13	889.21	9757.53	890.67	9837.27	891.56	9909.81	892.42	11044.73	893.24	16248.62	895.24	27125.26	899.19
344	448.36	859	5600	882.82	9628.13	889.31	9757.53	890.75	9837.27	891.64	9909.81	892.49	11044.73	893.32	16248.62	895.39	27125.26	899.47
345	448.89	865.3	5600	883.13	9628.13	889.51	9757.53	890.9	9837.27	891.76	9909.81	892.59	11044.73	893.43	16248.62	895.54	27125.26	899.68
348	448.90	864.3	5600	883.13	9628.13	889.49	9757.53	890.88	9837.27	891.74	9909.81	892.58	11044.73	893.41	16248.62	895.51	27125.26	899.62
348.4			North Dam															
348.8	448.90	864.3	5600	883.2	9628.13	889.54	9757.53	890.92	9837.27	891.78	9909.81	892.61	11044.73	893.46	16248.62	895.56	27125.26	899.69
351	448.91	861	5600	883.21	9628.13	889.54	9757.53	890.92	9837.27	891.79	9909.81	892.61	11044.73	893.46	16248.62	895.56	27125.26	899.69
351.4	449.06	861	5600	883.3	9628.13	889.66	9757.53	891.03	9837.27	891.88	9909.81	892.7	11044.73	893.56	16248.62	895.73	27125.26	899.97
351.5			12th Ave. No. (Fargo) / 15th Ave No. (Moorhead) Bridge															
351.6	449.07	861	5600	883.31	9628.13	889.74	9757.53	891.12	9837.27	891.97	9909.81	892.78	11044.73	893.63	16248.62	895.81	27125.26	900.02
352	449.09	862	5600	883.32	9628.13	889.76	9757.53	891.14	9837.27	891.98	9909.81	892.79	11044.73	893.65	16248.62	895.84	27125.26	900.07
353	449.26	863	5600	883.41	9628.13	889.85	9757.53	891.21	9837.27	892.04	9909.81	892.85	11044.73	893.72	16248.62	895.96	27125.26	900.27
353.5	449.61	860.13	5600	883.54	9628.13	889.93	9757.53	891.27	9837.27	892.09	9909.81	892.89	11044.73	893.76	16248.62	896.01	27125.26	900.35
354	449.97	864	5600	883.79	9628.13	890.14	9757.53	891.44	9837.27	892.26	9909.81	893.03	11044.73	893.9	16248.62	896.21	27125.26	900.58
355	450.19	862	5600	883.91	9628.13	890.22	9757.53	891.51	9837.27	892.32	9909.81	893.09	11044.73	893.97	16248.62	896.32	27125.26	900.76
356	450.84	858	5600	884.28	9628.13	890.46	9757.53	891.69	9837.27	892.47	9909.81	893.22	11044.73	894.11	16248.62	896.51	27125.26	900.99
357	450.86	863.4	5600	884.27	9628.13	890.46	9757.53	891.7	9837.27	892.48	9909.81	893.23	11044.73	894.11	16248.62	896.51	27125.26	900.99
358			BNRR Bridge															
359	450.87	863.4	5600	884.35	9628.13	890.55	9757.53	891.76	9837.27	892.53	9909.81	893.27	11044.73	894.15	16248.62	896.56	27125.26	901.19
361	451.20	865	5600	884.5	9628.13	890.66	9757.53	891.85	9837.27	892.61	9909.81	893.34	11044.73	894.23	16248.62	896.67	27125.26	901.35
361.5	451.37	865.2	5600	884.59	9628.13	890.73	9757.53	891.91	9837.27	892.66	9909.81	893.38	11044.73	894.27	16248.62	896.73	27125.26	901.43
362.5			1st Ave. No. Bridge															
364	451.39	865.2	5600	884.61	9628.13	890.76	9757.53	891.93	9837.27	892.67	9909.81	893.39	11044.73	894.28	16248.62	896.75	27125.26	901.53
366	451.41	861	5600	884.65	9628.13	890.79	9757.53	891.96	9837.27	892.7	9909.81	893.42	11044.73	894.31	16248.62	896.78	27125.26	901.57
367	451.57	867	5600	884.72	9628.13	890.89	9757.53	892.05	9837.27	892.79	9909.81	893.51	11044.73	894.41	16248.62	896.97	27125.26	901.91
368	451.61	866.8	5600	884.76	9628.13	890.93	9757.53	892.09	9837.27	892.82	9909.81	893.53	11044.73	894.44	16248.62	897.01	27125.26	901.97
369			NP Ave. (Fargo) / Center Ave. (Moorhead)															
370	451.63	866.8	5600	884.77	9628.13	890.94	9757.53	892.09	9837.27	892.83	9909.81	893.54	11044.73	894.44	16248.62	897.04	27125.26	902.04
371	451.65	859	5600	884.79	9628.13	890.94	9757.53	892.09	9837.27	892.83	9909.81	893.54	11044.73	894.44	16248.62	897.03	27125.26	902.01
372	451.67	867.9	5600	884.78	9628.13	890.93	9757.53	892.08	9837.27	892.82	9909.81	893.53	11044.73	894.43	16248.62	897.02	27125.26	902.02
373			BNRR Bridge															
374	451.68	867.5	5600	884.89	9628.13	891.02	9757.53	892.16	9837.27	892.89	9909.81	893.59	11044.73	894.49	16248.62	897.08	27125.26	902.07
375	451.69	864	5600	884.9	9628.13	891.02	9757.53	892.16	9837.27	892.89	9909.81	893.59	11044.73	894.49	16248.62	897.07	27125.26	902.05
376	451.71	865.9	5600	884.9	9628.13	891.04	9757.53	892.18	9837.27	892.9	9909.81	893.61	11044.73	894.52	16248.62	897.14	27125.26	902.2
377			Main Ave. Bridge															
378	451.72	865.9	5600	884.93	9628.13	891.05	9757.53	892.19	9837.27	892.91	9909.81	893.62	11044.73	894.53	16248.62	897.16	27125.26	902.26
379	451.73	864	5600	885.01	9628.13	891.14	9757.53	892.26	9837.27	892.98	9909.81	893.68	11044.73	894.59	16248.62	897.24	27125.26	902.35
380	451.85	864	5600	885	9628.13	891.11	9757.53	892.24	9837.27	892.96	9909.81	893.66	11044.73	894.57	16248.62	897.21	27125.26	902.33
381	452.03	863	5600	885.12	9628.13	891.26	9757.53	892.37	9837.27	893.08	9909.81	893.76	11044.73	894.69	16248.62	897.38	27125.26	902.53
381.5	452.04	863	5600	885.13	9628.13	891.27	9757.53	892.38	9837.27	893.09	9909.81	893.77	11044.73	894.7	16248.62	897.39	27125.26	902.55
384			Midtown Dam															
385.5	452.05	867	5600	885.15	9628.13	891.28	9757.53	892.38	9837.27	893.09	9909.81	893.76	11044.73	894.7	16248.62	897.4	27125.26	902.56
386	452.06	867	5600	885.15	9628.13	891.28	9757.53	892.39	9837.27	893.1	9909.81	893.76	11044.73	894.71	16248.62	897.4	27125.26	902.57
387	452.25	867	5600	885.26	9628.13	891.34	9757.53	892.43	9837.27	893.14	9909.81	893.82	11044.73	894.74	16248.62	897.45	27125.26	902.62
388	452.46	869	5600	885.42	9628.13	891.44	9757.53	892.52	9837.27	893.21	9909.81	893.86	11044.73	894.81	16248.62	897.55	27125.26	902.77
389	452.70	866.08	5600	885.65	9628.13	891.6	9757.53	892.65	9837.27	893.33	9909.81	893.99	11044.73	894.92	16248.62	897.69	27125.26	902.95
390	452.94	867	5600	885.77	9628.13	891.69	9757.53	892.72	9837.27	893.39	9909.81	894.05	11044.73	894.98	16248.62	897.77	27125.26	903.05
391	453.16	867	5600	885.89	9628.13	891.79	9757.53	892.81	9837.27	893.47	9909.81	894.11	11044.73	895.05	16248.62	897.87	27125.26	903.18
392	453.39	865	5600	886	9628.13	891.87	9757.53	892.89	9837.27	893.54	9909.81	894.18	11044.73	895.12	16248.62	897.98	27125.26	903.34
393	454.02	866	5600	886.34	9628.13	892.11	9757.53	893.07	9837.27	893.7	9909.81	894.32	11044.73	895.27	16248.62	898.15	27125.26	903.51
394	454.10	865	5600	886.38	9628.13	892.15	9757.53	893.1	9837.27	893.73	9909.81	894.35	11044.73	895.3	16248.62	898.18	27125.26	903.53
395	454.75	864	5600	886.65	9628.13	892.36	9757.53	893.29	9837.27	893.9	9909.81	894.5	11044.73	895.45	16248.62	898.37	27125.26	903.74
396	455.16	866	5600	886.86	9628.13	892.56	9757.53	893.46	9837.27	894.05	9909.81	894.64	11044.73	895.6	16248.62	898.56	27125.26	903.9
397	455.20	866.6	5600	886.9	9628.13	892.6	9757.53	893.49	9837.27	894.08	9909.81	894.66	11044.73	895.62	16248.62	898.59	27125.26	903.94
399			I-94 Bridge															
401	455.23	866.6	5600	886.9	9628.13	892.6	9757.53	893.49	9837.27	894.08	9909.81	894.66	11044.73	895.62	16248.62	898.59	27125.26	903.94
402	455.26	866	5600	886.93	9628.13	892.62	9757.53	893.5	9837.27	894.09	9909.81	894.67	11044.73	895.63	16248.62	898.61	27125.26	903.96
402.5	455.52	864.58	5600	887.05	9628.13	892.74	9757.53	893.6	9837.27	894.19	9909.81	894.76	11044.73	895.72	16248.62	898.72	27125.26	904.09
403	455.59	866	5600	887.08	9628.13	892.77	9757.53	893.64	9837.27	894.22	9909.81	894.79	11044.73	895.75	16248.62	898.75	27125.26	904.16
404	455.92	865	5600	887.22	9628.13	892.89	9757.53	893.74	9837.27	894.31	9909.81	894.87	11044.73	895.84	16248.62	898.89	27125.26	904.34
405	456.65	865	5600	887.57	9628.13	893.14	9757.53	893.95	9837.27	894.5	9909.81	895.04	11044.73	896.01	16248.62	899.08	27125.26	904.51
406	457.09	869	5600	887.79	9628.13	893.33	9757.53	894.11	9837.27	894.65	9909.81	895.16	11044.73	896.15	16248.62	899.25	27125.26	904.72
407	457.27	861.18	5600	887.88	9628.13	893.38	9757.53	894.16	9837.27	894.69	9909.81	895.22	11044.73	896.18	16248.62	899.29	27125.26	904.74
408	457.80	864	5600	888.07	9628.13	893.61	9757.53	894.36	9837.27	894.88	9909.81	895.39	11044.73	896.37	16248.62	899.54	27125.26	905.08
409	457.95	855.9	5600	888.14	9628.13	893.7	9757.53	894.45	9837.27	894.96	9909.81	895.47	11044.73	896.45	16248.62	899.64	27125.26	905.16
409.1	457.97	855.9	5600	888.14	9628.13	893.7	9757.53	894.45	9837.27	894.96	9909.81	895.47	11044.73	896.45	16248.62	899.64	27125.26	905.17

Table B17

Fargo Moorhead Metro Study - Phase 3.1
MN Short 35Kcfs, Year Zero - Wet, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
409.5			South Dam															
409.6	457.96	855.9	5600	888.19	9628.13	893.74	9757.53	894.48	9837.27	895	9909.81	895.5	11044.73	896.48	16248.62	899.67	27125.26	905.2
409.7	457.99	855.9	5600	888.19	9628.13	893.74	9757.53	894.49	9837.27	895	9909.81	895.5	11044.73	896.48	16248.62	899.67	27125.26	905.2
413	458.32	869	5600	888.2	9628.13	893.76	9757.53	894.51	9837.27	895.02	9909.81	895.52	11044.73	896.51	16248.62	899.71	27125.26	905.24
414	458.75	866	5600	888.47	9628.13	894.02	9757.53	894.74	9837.27	895.24	9909.81	895.73	11044.73	896.72	16248.62	899.99	27125.26	905.61
459.64	459.64	864.61	5600	888.9	9628.13	894.44	9757.53	895.11	9837.27	895.58	9909.81	896.03	11044.73	897.03	16248.62	900.33	27125.26	905.92
460.28	460.28	868.74	5600	889.28	9628.13	894.77	9757.53	895.4	9837.27	895.85	9909.81	896.29	11044.73	897.28	16248.62	900.62	27125.26	906.19
460.72	460.72	869.04	5600	889.58	9628.13	895.02	9757.53	895.64	9837.27	896.06	9909.81	896.49	11044.73	897.49	16248.62	900.88	27125.26	906.5
461.3	461.3	868.04	5600	889.99	9628.13	895.4	9757.53	895.98	9837.27	896.36	9909.81	896.79	11044.73	897.8	16248.62	901.26	27125.26	906.92
462.03	462.03	864.16	5600	890.38	9628.13	895.78	9757.53	896.33	9837.27	896.71	9909.81	897.1	11044.73	898.12	16248.62	901.62	27125.26	907.29
462.04	462.04	869.5	5600	890.42	9628.13	895.82	9757.53	896.36	9837.27	896.75	9909.81	897.12	11044.73	898.14	16248.62	901.64	27125.26	907.33
462.05			52nd Avenue So. (Fargo) / 60th Ave So. (Moorhead) Bridge															
462.06	462.06	869.5	5600	890.42	9628.13	895.83	9757.53	896.37	9837.27	896.75	9909.81	897.13	11044.73	898.15	16248.62	901.66	27125.26	907.37
462.07	462.07	871.5	5600	890.46	9628.13	895.86	9757.53	896.39	9837.27	896.78	9909.81	897.15	11044.73	898.18	16248.62	901.69	27125.26	907.36
462.7	462.7	862.82	5600	890.81	9628.13	896.17	9757.53	896.69	9837.27	897.05	9909.81	897.41	11044.73	898.44	16248.62	901.98	27125.26	907.68
463.7	463.7	864.71	5600	891.18	9628.13	896.57	9757.53	897.06	9837.27	897.41	9909.81	897.76	11044.73	898.8	16248.62	902.41	27125.26	908.11
464.2	464.2	869.2	5600	891.44	9628.13	896.84	9757.53	897.32	9837.27	897.66	9909.81	897.99	11044.73	899.04	16248.62	902.71	27125.26	908.47
464.6	464.6	865	5600	891.54	9628.13	897.03	9757.53	897.5	9837.27	897.82	9909.81	898.15	11044.73	899.21	16248.62	902.88	27125.26	908.63
465.1	465.1	862.9	5600	891.84	9628.13	897.27	9757.53	897.72	9837.27	898.04	9909.81	898.36	11044.73	899.43	16248.62	903.19	27125.26	908.96
465.9	465.9	865.6	5600	892.14	9628.13	897.61	9757.53	898.04	9837.27	898.35	9909.81	898.66	11044.73	899.75	16248.62	903.56	27125.26	909.27
466.4	466.4	865.6	5600	892.35	9628.13	897.63	9757.53	898.26	9837.27	898.56	9909.81	898.86	11044.73	899.96	16248.62	903.79	27125.26	909.51
466.5	466.5	864.9	5600	892.39	9628.13	897.88	9757.53	898.31	9837.27	898.61	9909.81	898.91	11044.73	900.01	16248.62	903.85	27125.26	909.54
466.9	466.9	865	5600	892.45	9628.13	897.96	9757.53	898.38	9837.27	898.68	9909.81	898.97	11044.73	900.07	16248.62	903.92	27125.26	909.58
467			Proposed Red River Closure Structure															
467.1	467.1	865	5600	892.49	9628.13	901.16	9757.53	904.75	9837.27	907.06	9909.81	909.71	11044.73	911.03	16248.62	912.32	27125.26	913.76
467.2	467.2	864.8	5600	892.48	9628.13	901.16	9757.53	904.76	9837.27	907.07	9909.81	909.73	11044.73	911.04	16248.62	912.36	27125.26	913.87
467.3	467.3	864.8	5600	892.52	9628.13	901.18	9757.53	904.77	9837.27	907.08	9909.81	909.73	11044.73	911.05	16248.62	912.37	27125.26	913.91
467.4	467.4	864.6	5600	892.78	9628.13	901.35	9757.53	904.88	9837.27	907.16	9909.81	909.78	11044.73	911.1	16248.62	912.46	27125.26	914.1
468.2	468.2	862.6	5600	893.05	11276.93	901.55	14279.39	905.01	17683.42	907.26	23006.7	909.86	27015.66	911.18	35510.02	912.57	47229.36	914.29
468.9	468.9	861	5600	893.3	11276.93	901.79	14279.39	905.23	17683.42	907.43	23006.7	910	27015.66	911.32	35510.02	912.74	47229.36	914.46
469.7	469.7	870	5600	893.67	11276.93	902.1	14279.39	905.46	17683.42	907.6	23006.7	910.13	27015.66	911.43	35510.02	912.87	47229.36	914.62
470.1	470.1	870.9	5600	893.85	11276.93	902.25	14279.39	905.6	17683.42	907.74	23006.7	910.23	27015.66	911.52	35510.02	912.97	47229.36	914.73
470.6	470.6	869.4	4166	894.03	8255.93	902.41	8093.39	905.74	9034.42	907.89	11349.7	910.38	13233.66	911.66	19707.01	913.12	28885.36	914.9
471.4	471.4	867.5	4166	894.28	8078.93	902.63	8070.39	905.85	9136.42	907.98	11578.7	910.44	13511.66	911.71	19562.01	913.19	27914.36	914.99
472.18	472.18	869.9	4166	894.54	8078.93	902.84	8070.39	905.97	9136.42	908.09	11578.7	910.52	13511.66	911.78	19562.01	913.28	27914.36	915.09
472.2	472.2	870.8	4166	894.54	8078.93	902.84	8070.39	905.97	9136.42	908.08	11578.7	910.51	13511.66	911.77	19562.01	913.27	27914.36	915.08
472.22	472.22	870.8	4166	894.55	8078.93	902.85	8070.39	905.98	9136.42	908.09	11578.7	910.54	13511.66	911.81	19562.01	913.32	27914.36	915.14
473	473	871.8	4166	894.8	8078.93	903.03	8070.39	906.08	9136.42	908.18	11578.7	910.61	13511.66	911.87	19562.01	913.4	27914.36	915.23
473.8	473.8	869	4166	895.07	8078.93	903.27	8070.39	906.22	9136.42	908.3	11578.7	910.72	13511.66	911.98	19562.01	913.54	27914.36	915.39
474.08	474.08	873.5	4166	895.17	8078.93	903.36	8070.39	906.26	9136.42	908.35	11578.7	910.77	13511.66	912.03	19562.01	913.6	27914.36	915.46
474.09	474.09	873.5	4166	895.19	8078.93	903.39	8070.39	906.3	9136.42	908.37	11578.7	910.79	13511.66	912.05	19562.01	913.63	27914.36	915.48
474.1			Cass Co Hwy 16/112th Ave So. (Fargo) / 110th Ave So. (Moorhead) Bridge															
474.11	474.11	874	4166	895.19	8078.93	903.38	8070.39	906.29	9136.42	908.36	11578.7	910.78	13511.66	912.06	19562.01	913.69	27914.36	915.59
474.12	474.12	874	4166	895.2	8078.93	903.39	8070.39	906.3	9136.42	908.37	11578.7	910.79	13511.66	912.08	19562.01	913.7	27914.36	915.6
474.6	474.6	873.4	4166	895.39	8078.93	903.52	8070.39	906.38	9136.42	908.44	11578.7	910.87	13511.66	912.16	19562.01	913.79	27914.36	915.71
475.3	475.3	872.2	4166	895.56	8952	903.78	10791	906.56	13453	908.63	17872	911.07	21196	912.37	30252	914.09	42385	916.05
475.7	475.7	871	4166	895.81	8952	903.95	10791	906.73	13453	908.85	17872	911.35	21196	912.72	30252	914.45	42385	916.25
476.4	476.4	873.7	4166	896.08	8952	904.2	10791	906.95	13453	909.09	17872	911.6	21196	912.95	30252	914.72	42385	916.46
477	477	874	4021	896.35	7227	904.45	10537	907.2	14618	909.37	20566	911.91	24472	913.27	28720	915.02	33177	916.75
477.6	477.6	870.2	4000	896.54	7000	904.61	10500	907.43	14800	909.7	21000	912.34	25000	913.76	28500	915.4	32000	917.02
478	478	873.4	4000	896.68	7000	904.71	10500	907.57	14800	909.89	21000	912.6	25000	913.99	28500	915.6	32000	917.1
478.8	478.8	874.8	4000	896.91	7000	904.9	10500	907.83	14800	910.25	21000	913.1	25000	914.48	28500	915.99	32000	917.26
479.2	479.2	875.7	4000	897.03	7000	904.99	10500	907.95	14800	910.41	21000	913.3	25000	914.67	28500	916.14	32000	917.35
479.7	479.7	871.5	4000	897.21	7000	905.1	10500	908.11	14800	910.61	21000	913.53	25000	914.92	28500	916.36	32000	917.49
480.5	480.5	874.7	4000	897.48	7000	905.3	10500	908.39	14800	910.97	21000	913.93	25000	915.31	28500	916.69	32000	917.78
481.8	481.8	875.3	4000	897.88	7000	905.64	10500	908.84	14800	911.5	21000	914.52	25000	915.95	28500	917.21	32000	918.25
482.3	482.3	874.2	4000	898.09	7000	905.82	10500	909.09	14800	911.83	21000	914.85	25000	916.3	28500	917.43	32000	918.42
482.698	482.698	869.74	4000	898.2	7000	905.92	10500	909.22	14800	911.98	21000	915.04	25000	916.52	28500	917.62	32000	918.59
482.7	482.7	887.04	4000	898.1	7000	905.86	10500	909.15	14800	911.9	21000	914.98	25000	916.47	28500	917.61	32000	918.63
482.702	482.702	878.54	4000	898.23	7000	905.93	10500	909.22	14800	911.99	21000	915.03	25000	916.52	28500	917.62	32000	918.62
483.1	483.1	879.2	4000	898.47	7000	906.12	10500	909.48	14800	912.32	21000	915.43	25000	916.87	28500	917.86	32000	918.76
483.9	483.9	873	4000	898.87	7000	906.39	10500	909.83	14800	912.77	21000	915.98	25000	917.33	28500	918.31	32000	919.15
485.08	485.08	880.5	4000	899.38	7000	906.62	10500	910.08	14800	913.04	21000	916.28	25000	917.65	28500	918.63	32000	919.46
485.09	485.09	880.5	4000	899.38	7000	906.61	10500	910.08	14800	913.04	21000	916.28	25000	917.64	28500	918.63	32000	919.46
485.1			Cass Co. Hwy 18/52nd St. SE (Fargo) / 100th Ave So. (Moorhead) Bridge															
485.11	485.11	879.4	4000	899.39	7000	906.63	10500	910.19	14800	913.15	21000	916.33	25000	917.69	28500	918.66	32000	919.49
485.12	485.12	879.4	4000	899.4	7000	906.66	10500	910.19	14800	913.15	21000	916.34	25000	917.7	28500	918.67	32000	919.5
485.6	485.6	878.62	4000	899.58	7000	906.77	10500	910.29	14800	913.24	21000	916.43	25000	917.79	28500	918.78	32000	919.61
485.9	485.9	879.5	4000	899.75	7000	906.88	10500	910.42	14800	913.4	21000	916.61	25000	918.01	28500	919.01	32000	919.83
486.3	486.3	872.1	4000	899.91	7000	907	10500	910.57	14800	913.58	21000	916.83	25000	918.26	28500	919.29	32000	920.12
487	487	871.3	4000	900.12	7000	907.18	10500	910.8	14800	913.89	21000	917.26	25000	918.77	28500	919.87	32000	920.76
487.5	487.5	878.8	4000	900.36	7000	907.35	10500	911.01	14800	914.13	21000	917.53	25000	919.07	28500	920.2	32000	921.07
488.68	488.68	878.24	4000	901.06	7000	907.67	10500	911.64	14800	914.93	21000	918.42	25000	920.03	28500	921.23	32000	922.21

Table B18

Fargo Moorhead Metro Study - Phase 3.1
MN Short 35Kcfs, Year 25, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
279	403.65	832.3	8950	861.67	18490	870.2	26120	873.73	34240	875.7	45780	876.86	55200	877.69	66177.08	878.58	82930	879.41
280	404.98	836.6	8950	862.33	18490	870.94	26120	874.31	34240	876.38	45780	877.66	55200	878.49	66177.08	879.38	82930	880.24
281	406.51	834.9	8950	863.21	18490	871.84	26120	875.33	34240	877.45	45780	878.67	55200	879.48	66177.08	880.19	82930	881.14
282	410.21	838	8950	865.13	18490	873.73	26120	877.12	34240	879.09	45780	879.9	55200	880.54	66177.08	881.44	82930	882.51
283	413.23	837.6	8950	866.44	18490	875.11	26120	878.49	34240	880.2	45780	881.15	55200	881.59	66177.08	882.44	82930	883.58
284	414.58	842.7	8200	867.17	17400	876.04	24700	879.61	32440	881.11	43410	882.04	52310	882.6	62987.08	883.48	79530	884.65
285	415.38	841.4	8200	867.35	17400	876.26	24700	879.85	32440	881.31	43410	882.29	52310	882.83	62987.08	883.72	79530	884.9
286	415.84	843	8200	867.48	17400	876.34	24700	879.9	32440	881.35	43410	882.38	52310	882.97	62987.08	883.87	79530	885.17
287	415.90	844.1	8200	867.53	17400	876.41	24700	880.01	32440	881.55	43410	882.52	52310	883.1	62987.08	883.90	79530	885.23
288			22nd St SE or Cass Co. #34 / 170th Ave NW or Clay Co.36															
289	415.91	844.1	8200	867.53	17400	876.4	24700	880	32440	881.57	43410	882.63	52310	883.29	62987.08	884.42	79530	885.48
290	415.93	837.5	8200	867.55	17400	876.42	24700	880.02	32440	881.58	43410	882.72	52310	883.39	62987.08	884.43	79530	885.5
291	417.10	840.8	8200	867.97	17400	876.89	24700	880.6	32440	882.37	43410	883.63	52310	884.41	62987.08	885.17	79530	886.13
292	418.18	844.4	7124	868.38	14681	877.36	20664	881.13	27055	882.69	36261	884.24	43687	885.01	54141.08	885.66	70317	886.67
293	419.92	844.7	7124	869.15	14681	878.16	20664	881.86	27055	883.52	36261	884.9	43687	885.49	54141.08	886.2	70317	887.26
294	420.32	843.8	7124	869.42	14681	878.39	20664	882.14	27055	883.85	36261	885.3	43687	885.98	54141.08	886.7	70317	887.74
295	421.93	845.3	7124	870.63	14681	879.47	20664	883.08	27055	884.63	36261	886.04	43687	886.75	54141.08	887.57	70317	888.36
296	422.69	847.4	7124	870.95	14681	879.8	20664	883.37	27055	884.67	36261	886.25	43687	886.97	54141.08	887.86	70317	888.68
297	423.95	849.5	7124	871.49	14681	880.39	20664	884.07	27055	885.78	36261	887.36	43687	888.24	54141.08	888.83	70317	889.75
298	425.51	848.7	7124	872.22	14681	881.11	20664	884.78	27055	886.48	36261	888.09	43687	889.03	54141.08	889.27	70317	890.25
299	426.87	851.1	7124	872.92	14681	881.76	20664	885.43	27055	887.2	36261	888.9	43687	889.62	54141.08	890.07	70317	890.83
300	428.06	849.6	4864	873.40	10430.06	882.35	10704.88	885.95	11103.7	887.69	10146.55	889.36	9483.57	890.12	13805.42	890.37	22390.43	891.19
301	428.74	850.3	4864	873.76	10430.06	882.63	10704.88	886.12	11103.7	887.82	10146.55	889.43	9483.57	890.17	13805.42	890.46	22390.43	891.28
302	430.23	852	4864	874.35	10430.06	883.1	10704.88	886.41	11103.7	888.06	10146.55	889.59	9483.57	890.29	13805.42	890.69	22390.43	891.48
303	431.94	853.5	4864	875.15	10430.06	883.78	10704.88	886.82	11103.7	888.4	10613.41	889.79	11849.72	890.46	16670.2	891.01	24061.47	892.08
303.4	432.07	853.42	4572	875.2	9987.06	883.84	10139.88	886.88	10288.7	888.45	9529.77	889.83	10160.58	890.5	14815.88	891.06	22060.58	892.12
303.5	432.10	853.46	4572	875.23	9987.06	883.87	10139.88	886.9	10288.7	888.46	9721.8	889.84	9921.79	890.51	14528.41	891.09	21718.56	892.17
303.8	432.53	854.5	4572	875.4	9987.06	884.03	10139.88	886.99	10288.7	888.54	10076.98	889.9	9283.7	890.57	13670.71	891.23	20763.56	892.42
304	432.84	855.5	4572	875.49	9988.06	884.12	10140.88	887.04	10278.64	888.57	9986.1	889.92	9058.38	890.59	13472.3	891.25	18155.42	892.47
305	433.26	854.4	4572	875.66	9988.06	884.28	10140.88	887.15	10278.64	888.65	10243.04	889.98	9587.65	890.63	12786.45	891.32	17836.17	892.55
306	433.35	855	4572	875.73	9988.06	884.36	10140.88	887.2	10278.64	888.7	10283.77	890.02	9715.33	890.66	12623.87	891.38	17760	892.57
307			76th Ave. No. (Fargo) / 90th Ave NW (Moorhead) Bridge															
308	433.37	855	4572	875.74	9988.06	884.37	10140.88	887.21	10278.64	888.71	10283.77	890.03	9715.33	890.66	12623.87	891.38	17760	892.6
309	433.83	854.5	4572	875.88	9988.06	884.52	10140.88	887.31	10278.64	888.79	10283.77	890.1	9715.33	890.72	12623.87	891.48	17760	892.73
310	434.61	852.8	4572	876.14	9988.06	884.74	10140.88	887.49	10278.64	888.91	10283.77	890.19	9715.33	890.8	12623.87	891.59	17760	892.9
310.5	435.70	855	4572	876.5	9988.06	885.08	10140.88	887.67	10278.64	889.08	10283.77	890.33	9715.33	890.92	12623.87	891.79	17760	893.21
311	436.34	855	4572	876.73	9988.06	885.31	10140.88	887.83	10268.28	889.22	10253.33	890.45	10251.99	891.03	13816.73	891.96	18262.22	893.46
312	437.05	863.83	4372	876.97	9693.06	885.56	9750.88	888.01	9853.28	889.38	9813.33	890.59	9801.99	891.15	13283.73	892.17	17816.13	893.75
313	437.95	858	4372	877.32	9692.06	885.94	9749.88	888.28	9863.34	889.6	9902.21	890.78	10026.81	891.34	13682.14	892.44	20295.38	894.16
314	438.78	857	4372	877.66	9692.06	886.28	9749.88	888.52	9863.34	889.81	9902.21	890.95	10026.81	891.5	13682.14	892.69	20295.38	894.61
315	439.05	856	4372	877.75	9692.06	886.38	9749.88	888.59	9863.34	889.87	9902.21	891	10026.81	891.55	13682.14	892.78	20295.38	894.78
316	439.46	855.1	4372	877.91	9692.06	886.58	9749.88	888.74	9863.34	890	9902.21	891.12	10026.81	891.67	13682.14	892.97	19900.53	895.13
317			40th Ave. No. (Fargo) / Wall Street NW (Moorhead) Bridge															
318	439.47	855.1	4372	877.92	9692.06	886.58	9749.88	888.75	9863.34	890	9902.21	891.12	10026.81	891.67	13682.14	892.97	19900.53	895.12
319	439.50	857	4372	877.93	9692.06	886.63	9749.88	888.79	9863.34	890.04	9902.21	891.16	10026.81	891.7	13682.14	893.01	19900.53	895.18
320	439.66	856	4372	877.99	9692.06	886.69	9749.88	888.83	9863.34	890.08	9902.21	891.19	10026.81	891.72	13682.14	893.05	19900.53	895.25
321	440.31	858	4372	878.2	9692.06	886.94	9749.88	889.01	9863.34	890.23	9902.21	891.32	10026.81	891.86	13682.14	893.26	19900.53	895.58
322	440.32	858.1	4372	878.2	9692.06	886.93	9749.88	889.01	9863.34	890.23	9902.21	891.32	10026.81	891.85	13682.14	893.26	19900.53	895.58
323			Broadway Street No. (Fargo) / Broadway Street NW (Moorhead) Bridge															
324	440.33	858.1	4372	878.22	9692.06	887.04	9749.88	889.08	9863.34	890.28	9902.21	891.36	10026.81	891.89	13682.14	893.3	19900.53	895.62
325	440.34	857	4372	878.22	9692.06	887.05	9749.88	889.09	9863.34	890.29	9902.21	891.36	10026.81	891.89	13682.14	893.3	19900.53	895.62
326	440.54	858	4372	878.27	9692.06	887.1	9749.88	889.12	9863.34	890.32	9902.21	891.39	10026.81	891.92	13682.14	893.34	19900.53	895.69
327	441.53	859	4352	878.66	9647.06	887.37	9694.88	889.3	9803.34	890.46	9837.21	891.51	9956.81	892.03	13607.14	893.51	19805.53	895.96
328	442.14	858	4352	878.86	9647.06	887.52	9694.88	889.41	9803.34	890.55	9837.21	891.58	9956.84	892.1	13848.37	893.61	21058.39	896.11
329	442.45	857	4352	878.93	9647.06	887.54	9694.88	889.42	9803.34	890.56	9837.21	891.59	9994.46	892.1	14334.29	893.61	22432.78	896.12
329.5	442.93	858.06	4352	879.08	9646.06	887.65	9693.88	889.49	9812.94	890.61	9872.9	891.63	10025.81	892.14	14350.13	893.67	22531.17	896.2
330	443.69	858	4352	879.32	9646.06	887.84	9693.88	889.63	9812.94	890.72	9872.9	891.72	10025.81	892.22	14350.13	893.78	22531.17	896.35
331	443.99	859	4352	879.4	9646.06	887.93	9693.88	889.7	9812.94	890.76	9872.9	891.77	10025.81	892.27	14350.13	893.87	22531.17	896.5
332	444.33	856	4352	879.45	9646.06	887.97	9693.88	889.73	9812.94	890.81	9872.9	891.79	10025.81	892.29	14350.13	893.9	22531.17	896.55
333	444.92	859	4352	879.53	9646.06	888	9693.88	889.76	9812.94	890.83	9872.9	891.81	10025.81	892.31	14350.13	893.92	22531.17	896.55
334	445.18	860	4352	879.61	9646.06	888.08	9693.88	889.81	9812.94	890.87	9872.9	891.85	10025.81	892.34	14350.13	893.98	22531.17	896.64
335	445.31	860	4352	879.67	9646.06	888.16	9693.88	889.88	9812.94	890.94	9872.9	891.91	10025.81	892.4	14350.13	894.08	22531.17	896.85
336	445.36	859	4352	879.68	9646.06	888.17	9693.88	889.89	9812.94	890.95	9872.9	891.91	10025.81	892.41	14350.13	894.09	22531.17	896.87
336.5	445.76	857.69	4352	879.71	9646.06	888.2	9693.88	889.91	9812.94	890.96	9872.9	891.93	10025.81	892.42	14350.13	894.11	22531.17	896.9
337	446.09	860	4352	879.75	9646.06	888.2	9693.88	889.9	9812.94	890.96	9872.9	891.92	10025.81	892.42	14350.13	894.11	22531.17	896.89
338	446.28	861	4352	879.82	9646.06	888.23	9693.88	889.92	9812.94	890.97	9872.9	891.93	10025.81	892.42	14350.13	894.11	22531.17	896.9
338.5	446.70	860.9	4352	879.97	9646.06	888.37	9693.88	890.02	9812.94	891.06	9872.9	892.01	10025.81	892.49	14350.13	894.22	22531.17	897.07
339	446.76	861	4352	879.99	9646.06	888.38	9693.88	890.03	9812.94	891.07	9872.9	892.01	10025.81	892.5	14350.13	894.23	22531.17	897.08
340	447.04	860	4352	880.08	9646.06	888.47	9693.88	890.11	9812.94	891.14	9872.9	892.06	10025.81	892.56	14350.13	894.34	22531.17	897.29
341	447.24	859	4352	880.16	9646.06	888.53	9693.88	890.15	9812.94	891.17	9872.9	892.11	10025.81	892.59	14350.13	894.37	22531.17	897.33
342	447.42	861	4352	880.28	9646.06	888.62	9693.88	890.21	9812.94	891.22	9872.9	892.15	10025.81	892.63	14350.13	894.43	22531.17	897.42
342.5	447.78	856.99	4352	880.43	9646.06	888.75	9693.88	890.32	9812.94	891.31	9872.9	892.23	10025.81	892.7	14350.13	894.55	22531.17	897.62
343	448.13	861	4352	880.58	9646.06	888.86	9693.88	890.4	9812.94	891.38	9872.9	892.28	10025.81	892.76	14350.13	894.63	22531.17	897.74
344	448.36	859	4352	880.69	9646.06	888.96	9693.88	890.48	9812.94	891.45	9872.9	892.35	10025.81	892.83	14350.13	894.75	22531.17	897.96
345	448.89	865.3	4352	881.01	9646.06	889.18	9693.88	890.64	9812.94	891.58	9872.9	892.46	10025.81	892.93	14350.13	894.89	22531.17	898.15
348	448.90	864.3	4352	881	9646.06	889.16	9693.88	890.62	9812.94	891.57	9872.9	892.44	10025.81	892.91	14350.13	894.86	22531.17	898.1
348.4			North Dam															
349.8	448.90	864.3	4352	881.07	9646.06	889.22	9693.88	890.66	9812.94	891.61	9872.9	892.48	10025.81	892.95	14350.13	894.91	22531.17	898.15
351	448.91	861	4352	881.09	9646.06	889.22	9693.88	890.66	9812.94	891.61	9872.9	892.48	10025.81	892.95	14350.13	894.91	22531.17	898.15
351.4	449.06	861	4352	881.17	9646.06	889.35	9693.88	890.77	9812.94	891.71	9872.9	892.57	10025.81	893.04	14350.13	895.05	22531.17	898.38
351.5			12th Ave. No. (Fargo) / 15th Ave No. (Moorhead) Bridge															
351.6	449.07	861	4352	881.17	9646.06	889.43	9693.88	890.86	9812.94	891.79	9872.9	892.65	10025.81	893.11	14350.13	895.13	22531.17	898.44
352	449.09	862	4352	881.19	9646.06	889.45	9693.88	890.88	9812.94	891.81	9872.9	892.66	10025.81	893.12	14350.13	895.16	22531.17	898.48
353	449.28	863	4352	881.27	9646.06	889.54	9693.88	890.95	9812.94	891.87	9872.9	892.72	10025.81	893.18	14350.13	895.25	22531.17	898.65
353.5	449.61	860.13	4352	881.4	9646.06	889.63	9693.88	891.01	9812.94	891.92	9872.9	892.76	10025.81	893.22	14350.13	895.3	22531.17	898.72
354	449.97	864	4352	881.65	9646.06	889.85	9693.88	891.19	9812.94	892.09	9872.9	892.91	10025.81	893.35	14350.13	895.48	22531.17	898.93
355	450.19	862	4352	881.79	9646.06	889.93	9693.88	891.26	9812.94	892.15	9872.9	892.97	10025.81	893.41	14350.13	895.58	22531.17	899.09
356	450.84	858	4352	882.17	9646.06	890.19	9693.88	891.45	9812.94	892.31	9872.9	893.1	10025.81	893.54	14350.13	895.75	22531.17	899.31
357	450.86	863.4	4352	882.18	9646.06	890.19	9693.88	891.45	9812.94	892.32	9872.9	893.1	10025.81	893.54	14350.13	895.75	22531.17	899.31
358			BNRR Bridge															
359	450.87	863.4	4352	882.25	9646.06	890.29	9693.88	891.53	9812.94	892.37	9872.9	893.15	10025.81	893.58	14350.13	895.79	22531.17	899.46
361	451.20	865	4352	882.4	9646.06	890.41	9693.88	891.62	9812.94	892.45	9872.9	893.22	10025.81	893.65	14350.13	895.89	22531.17	899.59
361.5	451.37	865.2	4352	882.5	9646.06	890.48	9693.88	891.68	9812.94	892.5	9872.9	893.26	10025.81	893.69	14350.13	895.94	22531.17	899.66
352.5			1st Ave. No. Bridge															
364	451.39	865.2	4352	882.52	9646.06	890.5	9693.88	891.69	9812.94	892.52	9872.9	893.27	10025.81	893.7	14350.13	895.95	22531.17	899.7
366	451.41	861	4352	882.56	9646.06	890.54	9693.88	891.73	9812.94	892.55	9872.9	893.3	10025.81	893.72	14350.13	895.99	22531.17	899.73
367	451.57	867	4352	882.62	9646.06	890.64	9693.88	891.82	9812.94	892.64	9872.9	893.39	10025.81	893.81	14350.13	896.14	22531.17	900.02
368	451.61	866.8	4352	882.66	9646.06	890.69	9693.88	891.86	9812.94	892.67	9872.9	893.42	10025.81	893.84	14350.13	896.18	22531.17	900.07
369			NP Ave. (Fargo) / Center Ave. (Moorhead)															
370	451.63	866.8	4352	882.68	9646.06	890.69	9693.88	891.86	9812.94	892.68	9872.9	893.42	10025.81	893.84	14350.13	896.18	22531.17	900.1
371	451.65	859	4352	882.7	9646.06	890.69	9693.88	891.86	9812.94	892.67	9872.9	893.42	10025.81	893.84	14350.13	896.18	22531.17	900.08
372	451.67	867.9	4352	882.69	9646.06	890.68	9693.88	891.85	9812.94	892.67	9872.9	893.41	10025.81	893.83	14350.13	896.17	22531.17	900.08
373			BNRR Bridge															
374	451.68	867.9	4352	882.79	9646.06	890.78	9693.88	891.93	9812.94	892.74	9872.9	893.47	10025.81	893.89	14350.13	896.23	22531.17	900.14
375	451.69	864	4352	882.82	9646.06	890.78	9693.88	891.93	9812.94	892.73	9872.9	893.47	10025.81	893.89	14350.13	896.22	22531.17	900.12
376	451.71	865.9	4352	882.82	9646.06	890.79	9693.88	891.95	9812.94	892.75	9872.9	893.49	10025.81	893.91	14350.13	896.27	22531.17	900.23
377			Main Ave. Bridge															
378	451.72	865.9	4352	882.84	9646.06	890.81	9693.88	891.96	9812.94	892.76	9872.9	893.5	10025.81	893.92	14350.13	896.29	22531.17	900.26
379	451.73	864	4352	882.92	9646.06	890.9	9693.88	892.04	9812.94	892.83	9872.9	893.56	10025.81	893.98	14350.13	896.36	22531.17	900.35
380	451.85	864	4352	882.91	9646.06	890.88	9693.88	892.01	9812.94	892.81	9872.9	893.54	10025.81	893.96	14350.13	896.34	22531.17	900.32
381	452.03	863	4352	883.02	9646.06	891.02	9693.88	892.14	9812.94	892.93	9872.9	893.65	10025.81	894.06	14350.13	896.49	22531.17	900.52
381.5	452.04	863	4352	883.03	9646.06	891.04	9693.88	892.15	9812.94	892.94	9872.9	893.66	10025.81	894.07	14350.13	896.5	22531.17	900.53
384			Midtown Dam															
385.5	452.05	867	4352	883.07	9646.06	891.05	9693.88	892.16	9812.94	892.95	9872.9	893.66	10025.81	894.08	14350.13	896.51	22531.17	900.54
386	452.06	867	4352	883.08	9646.06	891.05	9693.88	892.16	9812.94	892.95	9872.9	893.67	10025.81	894.08	14350.13	896.51	22531.17	900.55
387	452.26	867	4352	883.2	9646.06	891.11	9693.88	892.21	9812.94	892.99	9872.9	893.7	10025.81	894.12	14350.13	896.55	22531.17	900.6
388	452.46	869	4352	883.39	9646.06	891.22	9693.88	892.3	9812.94	893.07	9872.9	893.77	10025.81	894.18	14350.13	896.64	22531.17	900.72
389	452.70	866.08	4352	883.64	9646.06	891.39	9693.88	892.43	9812.94	893.19	9872.9	893.88	10025.81	894.28	14350.13	896.77	22531.17	900.89
390	452.94	867	4352	883.76	9646.06	891.48	9693.88	892.51	9812.94	893.25	9872.9	893.93	10025.81	894.34	14350.13	896.85	22531.17	900.99
391	453.16	867	4352	883.89	9646.06	891.59	9693.88	892.59	9812.94	893.33	9872.9	894	10025.81	894.4	14350.13	896.94	22531.17	901.11
392	453.39	865	4352	884.01	9646.06	891.67	9693.88	892.67	9812.94	893.4	9872.9	894.07	10025.81	894.46	14350.13	897.02	22531.17	901.24
393	454.02	866	4352	884.36	9646.06	891.92	9693.88	892.87	9812.94	893.57	9872.9	894.22	10025.81	894.61	14350.13	897.19	22531.17	901.42
394	454.10	865	4352	884.39	9646.06	891.96	9693.88	892.9	9812.94	893.6	9872.9	894.24	10025.81	894.63	14350.13	897.22	22531.17	901.44
395	454.75	864	4352	884.67	9646.06	892.19	9693.88	893.09	9812.94	893.77	9872.9	894.39	10025.81	894.77	14350.13	897.4	22531.17	901.66
396	455.16	866	4352	884.87	9646.06	892.4	9693.88	893.27	9812.94	893.93	9872.9	894.53	10025.81	894.91	14350.13	897.57	22531.17	901.84
397	455.20	866.6	4352	884.91	9646.06	892.43	9693.88	893.3	9812.94	893.96	9872.9	894.56	10025.81	894.93	14350.13	897.59	22531.17	901.88
399			I-94 Bridge															
401	455.23	866.6	4352	884.91	9646.06	892.43	9693.88	893.3	9812.94	893.96	9872.9	894.56	10025.81	894.93	14350.13	897.59	22531.17	901.88
402	455.28	865	4352	884.95	9646.06	892.45	9693.88	893.31	9812.94	893.97	9872.9	894.57	10025.81	894.94	14350.13	897.61	22531.17	901.91
402.5	455.52	864.50	4352	885.06	9646.06	892.57	9693.88	893.42	9812.94	894.06	9872.9	894.65	10025.81	895.02	14350.13	897.71	22531.17	902.02
403	455.59	866	4352	885.09	9646.06	892.61	9693.88	893.45	9812.94	894.09	9872.9	894.68	10025.81	895.05	14350.13	897.74	22531.17	902.05
404	455.92	865	4352	885.23	9646.06	892.73	9693.88	893.56	9812.94	894.19	9872.9	894.77	10025.81	895.14	14350.13	897.86	22531.17	902.23
405	456.65	865	4352	885.6	9646.06	893	9693.88	893.77	9812.94	894.39	9872.9	894.94	10025.81	895.3	14350.13	898.04	22531.17	902.42
406	457.09	866	4352	885.83	9646.06	893.19	9693.88	893.94	9812.94	894.54	9872.9	895.08	10025.81	895.43	14350.13	898.21	22531.17	902.62
407	457.27	861.18	4352	885.91	9646.06	893.25	9693.88	893.99	9812.94	894.58	9872.9	895.12	10025.81	895.47	14350.13	898.25	22531.17	902.65
408	457.80	864	4352	886.1	9646.06	893.48	9693.88	894.19	9812.94	894.77	9872.9	895.3	10025.81	895.64	14350.13	898.47	22531.17	902.96
409	457.96	855.9	4352	886.15	9646.06	893.57	9693.88	894.28	9812.94	894.85	9872.9	895.37	10025.81	895.71	14350.13	898.56	22531.17	903.05
409.1	457.97	855.9	4352	886.15	9646.06	893.57	9693.88	894.28	9812.94	894.85	9872.9	895.37	10025.81	895.72	14350.13	898.56	22531.17	903.05

Table B18

Fargo Moorhead Metro Study - Phase 3.1
MN Short 35Kcfs, Year 25, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	nvert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
409.5			South Dam															
409.6	457.98	855.9	4352	886.22	9646.06	893.61	9693.88	894.32	9812.94	894.89	9872.9	895.41	10025.81	895.75	14350.13	898.6	22531.17	903.08
409.7	457.99	855.9	4352	886.22	9646.06	893.61	9693.88	894.32	9812.94	894.89	9872.9	895.41	10025.81	895.75	14350.13	898.6	22531.17	903.09
413	458.32	869	4352	886.22	9646.06	893.63	9693.88	894.34	9812.94	894.91	9872.9	895.43	10025.81	895.77	14350.13	898.64	22531.17	903.13
414	458.75	866	4352	886.49	9646.06	893.9	9693.88	894.58	9812.94	895.13	9872.9	895.64	10025.81	895.97	14350.13	898.9	22531.17	903.47
459.64	459.64	864.81	4352	886.9	9646.06	894.33	9693.88	894.95	9812.94	895.48	9872.9	895.95	10025.81	896.27	14350.13	899.23	22531.17	903.81
460.28	460.28	868.74	4352	887.28	9646.06	894.67	9693.88	895.25	9812.94	895.75	9872.9	896.2	10025.81	896.51	14350.13	899.51	22531.17	904.1
460.72	460.72	869.04	4352	887.59	9646.06	894.93	9693.88	895.49	9812.94	895.97	9872.9	896.4	10025.81	896.71	14350.13	899.75	22531.17	904.41
461.3	461.3	868.04	4352	887.99	9646.06	895.31	9693.88	895.84	9812.94	896.3	9872.9	896.71	10025.81	897.01	14350.13	900.1	22531.17	904.86
462.03	462.03	864.16	4352	888.36	9646.06	895.71	9693.88	896.19	9812.94	896.63	9872.9	897.02	10025.81	897.31	14350.13	900.45	22531.17	905.22
462.04	462.04	869.5	4352	888.4	9646.06	895.74	9693.88	896.23	9812.94	896.66	9872.9	897.05	10025.81	897.34	14350.13	900.47	22531.17	905.27
462.05			52nd Avenue So. (Fargo) / 60th Ave So. (Moorhead) Bridge															
462.06	462.06	869.5	4352	888.41	9646.06	895.75	9693.88	896.23	9812.94	896.67	9872.9	897.05	10025.81	897.35	14350.13	900.49	22531.17	905.34
462.07	462.07	871.5	4352	888.45	9646.06	895.78	9693.88	896.26	9812.94	896.69	9872.9	897.08	10025.81	897.37	14350.13	900.51	22531.17	905.33
462.7	462.7	862.82	4352	888.81	9646.06	896.1	9693.88	896.55	9812.94	896.97	9872.9	897.34	10025.81	897.62	14350.13	900.8	22531.17	905.63
463.7	463.7	864.71	4352	889.14	9646.06	896.51	9693.88	896.94	9812.94	897.33	9872.9	897.68	10025.81	897.96	14350.13	901.21	22531.17	906.06
464.2	464.2	869.2	4352	889.4	9646.06	896.78	9693.88	897.19	9812.94	897.58	9872.9	897.92	10025.81	898.19	14350.13	901.5	22531.17	906.41
464.6	464.6	865	4352	889.61	9646.06	896.98	9693.88	897.37	9812.94	897.75	9872.9	898.08	10025.81	898.35	14350.13	901.67	22531.17	906.56
465.1	465.1	862.9	4352	889.78	9646.06	897.21	9693.88	897.6	9812.94	897.97	9872.9	898.29	10025.81	898.56	14350.13	901.94	22531.17	906.97
465.9	465.9	865.6	4352	890.05	9646.06	897.56	9693.88	897.93	9812.94	898.28	9872.9	898.59	10025.81	898.85	14350.13	902.3	22531.17	907.32
466.4	466.4	865.8	4352	890.25	9646.06	897.79	9693.88	898.14	9812.94	898.49	9872.9	898.79	10025.81	899.05	14350.13	902.52	22531.17	907.57
466.5	466.5	864.9	4352	890.3	9646.06	897.84	9693.88	898.19	9812.94	898.54	9872.9	898.84	10025.81	899.1	14350.13	902.57	22531.17	907.61
466.9	466.9	865	4352	890.35	9646.06	897.91	9693.88	898.26	9812.94	898.61	9872.9	898.9	10025.81	899.17	14350.13	902.64	22531.17	907.66
467			Proposed Red River Closure Structure															
467.1	467.1	865	4352	890.37	9646.06	899.65	9693.88	903.58	9812.94	906.37	9872.9	909.1	10025.81	910.83	14350.13	912.27	22531.17	913.6
467.2	467.2	864.8	4352	890.36	9646.06	899.65	9693.88	903.59	9812.94	906.37	9872.9	909.11	10025.81	910.85	14350.13	912.3	22531.17	913.68
467.3	467.3	864.8	4352	890.4	9646.06	899.68	9693.88	903.6	9812.94	906.38	9872.9	909.11	10025.81	910.85	14350.13	912.31	22531.17	913.71
467.4	467.4	864.8	4352	890.65	9646.06	899.89	9693.88	903.73	9812.94	906.47	9872.9	909.17	10025.81	910.9	14350.13	912.38	22531.17	913.84
468.2	468.2	862.6	4352	890.9	10487.04	900.13	13380.91	903.87	16559.85	906.58	21661.6	909.25	25791.55	910.97	32667.25	912.48	43804.1	913.99
468.9	468.9	861	4352	891.12	10487.04	900.39	13380.91	904.11	16559.85	906.76	21661.6	909.41	25791.55	911.1	32667.25	912.62	43804.1	914.17
469.7	469.7	870	4352	891.47	10487.04	900.74	13380.91	904.36	16559.85	906.97	21661.6	909.54	25791.55	911.22	32667.25	912.74	43804.1	914.3
470.1	470.1	870.9	4352	891.64	10487.04	900.9	13380.91	904.51	16559.85	907.11	21661.6	909.65	25791.55	911.3	32667.25	912.82	43804.1	914.4
470.6	470.6	869.4	3198	891.81	7796.04	901.07	7721.91	904.66	8501.85	907.25	10680.6	909.8	12655.55	911.43	16088.25	912.96	26538.1	914.56
471.4	471.4	867.5	3198	892.04	7651.04	901.32	7708.91	904.78	8602.85	907.35	10900.6	909.87	12925.55	911.49	17974.25	913.02	25655.1	914.64
472.18	472.18	869.9	3198	892.29	7651.04	901.56	7708.91	904.91	8602.85	907.46	10900.6	909.96	12925.55	911.56	17974.25	913.1	25655.1	914.74
472.2	472.2	870.8	3198	892.29	7651.04	901.55	7708.91	904.91	8602.85	907.45	10900.6	909.95	12925.55	911.55	17974.25	913.09	25655.1	914.73
472.22	472.22	870.8	3198	892.3	7651.04	901.57	7708.91	904.92	8602.85	907.46	10900.6	909.97	12925.55	911.58	17974.25	913.14	25655.1	914.78
473	473	871.9	3198	892.53	7651.04	901.78	7708.91	905.03	8602.85	907.54	10900.6	910.05	12925.55	911.64	17974.25	913.21	25655.1	914.87
473.8	473.8	869	3198	892.8	7651.04	902.05	7708.91	905.19	8602.85	907.66	10900.6	910.16	12925.55	911.75	17974.25	913.33	25655.1	915.03
474.08	474.08	873.5	3198	892.9	7651.04	902.14	7708.91	905.25	8602.85	907.71	10900.6	910.21	12925.55	911.8	17974.25	913.38	25655.1	915.08
474.09	474.09	873.5	3198	892.91	7651.04	902.17	7708.91	905.27	8602.85	907.73	10900.6	910.23	12925.55	911.82	17974.25	913.41	25655.1	915.11
474.1			Cass Co Hwy 18/112th Ave So. (Fargo) / 110th Ave So. (Moorhead) Bridge															
474.11	474.11	874	3198	892.91	7651.04	902.16	7708.91	905.26	8602.85	907.73	10900.6	910.23	12925.55	911.82	17974.25	913.46	25655.1	915.21
474.12	474.12	874	3198	892.92	7651.04	902.17	7708.91	905.27	8602.85	907.73	10900.6	910.23	12925.55	911.84	17974.25	913.47	25655.1	915.22
474.6	474.6	873.4	3198	893.12	7651.04	902.32	7708.91	905.36	8602.85	907.8	10900.6	910.31	12925.55	911.91	17974.25	913.56	25655.1	915.32
475.3	475.3	872.2	3198	893.38	7772	902.59	9722	905.54	12388	907.98	16680	910.5	20055	912.12	27549	913.82	39492	915.65
475.7	475.7	871	3198	893.51	7772	902.74	9722	905.71	12388	908.17	16680	910.77	20055	912.44	27549	914.14	39492	915.86
476.4	476.4	873.7	3198	893.78	7772	902.98	9722	905.92	12388	908.4	16680	911.01	20055	912.67	27549	914.39	39492	916.08
477	477	874	3147	894.05	6349	903.21	9577	906.16	13547	908.67	19288	911.33	23242	912.98	26339	914.68	31093	916.36
477.6	477.6	870.2	3139	894.23	6160	903.36	9555	906.38	13729	908.99	19709	911.75	23757	913.46	26164	915.04	30016	916.63
478	478	873.4	3139	894.37	6160	903.45	9555	906.52	13729	909.17	19709	911.98	23757	913.69	26164	915.22	30016	916.7
478.8	478.8	874.8	3139	894.6	6160	903.63	9555	906.78	13729	909.52	19709	912.47	23757	914.17	26164	915.59	30016	916.99
479.2	479.2	875.7	3139	894.72	6160	903.71	9555	906.9	13729	909.67	19709	912.66	23757	914.35	26164	915.73	30016	917.08
479.7	479.7	871.5	3139	894.91	6160	903.82	9555	907.05	13729	909.87	19709	912.89	23757	914.6	26164	915.96	30016	917.21
480.5	480.5	874.7	3139	895.19	6160	904.02	9555	907.33	13729	910.22	19709	913.3	23757	914.98	26164	916.26	30016	917.49
481.8	481.8	875.3	3139	895.57	6160	904.34	9555	907.79	13729	910.75	19709	913.89	23757	915.61	26164	916.76	30016	917.94
482.3	482.3	874.2	3139	895.78	6160	904.51	9555	908.04	13729	911.06	19709	914.23	23757	915.95	26164	916.99	30016	918.12
482.698	482.698	869.74	3139	895.89	6160	904.6	9555	908.16	13729	911.22	19709	914.41	23757	916.14	26164	917.18	30016	918.28
482.7	482.7	887.04	3139	895.77	6160	904.55	9555	908.09	13729	911.13	19709	914.34	23757	916.05	26164	917.16	30016	918.28
482.702	482.702	878.54	3139	895.92	6160	904.61	9555	908.17	13729	911.22	19709	914.41	23757	916.14	26164	917.18	30016	918.28
483.1	483.1	879.2	3139	896.16	6160	904.8	9555	908.42	13729	911.54	19709	914.77	23757	916.53	26164	917.44	30016	918.47
483.9	483.9	873	3139	896.59	6160	905.06	9555	908.76	13729	911.98	19709	915.31	23757	917	26164	917.89	30016	918.86
485.08	485.08	880.5	3139	897.17	6160	905.31	9555	909.01	13729	912.26	19709	915.8	23757	917.3	26164	918.19	30016	919.16
485.09	485.09	880.5	3139	897.18	6160	905.3	9555	908.99	13729	912.26	19709	915.8	23757	917.3	26164	918.18	30016	919.16
485.1			Cass Co. Hwy 18/52nd St. SE (Fargo) / 160th Ave So. (Moorhead) Bridge															
485.11	485.11	879.4	3139	897.19	6160	905.3	9555	909.06	13729	912.38	19709	915.67	23757	917.35	26164	918.22	30016	919.19
485.12	485.12	879.4	3139	897.19	6160	905.33	9555	909.11	13729	912.38	19709	915.67	23757	917.35	26164	918.23	30016	919.2
485.6	485.6	878.62	3139	897.4	6160	905.43	9555	909.21	13729	912.48	19709	915.77	23757	917.45	26164	918.33	30016	919.3
485.9	485.9	879.5	3139	897.58	6160	905.55	9555	909.35	13729	912.63	19709	915.95	23757	917.65	26164	918.54	30016	919.52
486.3	486.3	872.1	3139	897.75	6160	905.66	9555	909.5	13729	912.81	19709	916.16	23757	917.89	26164	918.79	30016	919.79
487	487	871.3	3139	897.97	6160	905.84	9555	909.72	13729	913.1	19709	916.56	23757	918.38	26164	919.31	30016	920.38
487.5	487.5	878.8	3139	898.19	6160	906.02	9555	909.94	13729	913.34	19709	916.83	23757	918.66	26164	919.61	30016	920.69
488.68	488.68	878.24	3139	898.84	6160	906.52	9555	910.55	13729	914.11	19709	917.71	23757	919.5	26164	920.58	30016	921.75

Table B19

Fargo Moorhead Metro Study - Phase 3.1
MN Short 35Kcfs, Year 50, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
279	403.65	832.3	7890	860.38	17020	869.46	24610	873.29	32650	875.42	44080	876.69	53560	877.55	63590	878.37	80610	879.31
280	404.96	836.6	7890	861.02	17020	870.16	24610	874.14	32650	876.09	44080	877.53	53560	878.34	63590	879.15	80610	880.13
281	406.51	834.9	7890	861.88	17020	871.03	24610	875.07	32650	877.15	44080	878.68	53560	879.33	63590	879.96	80610	881.01
282	410.21	838	7890	863.78	17020	872.89	24610	876.78	32650	878.78	44080	879.82	53560	880.48	63590	881.22	80610	882.37
283	413.23	837.6	7890	865.06	17020	874.26	24610	878.09	32650	880.03	44080	881.03	53560	881.43	63590	882.22	80610	883.43
284	414.98	842.7	7150	865.77	15860	875.14	23130	879.17	30790	880.91	41630	881.89	50580	882.45	60120	883.26	76860	884.49
285	415.38	841.4	7150	865.94	15860	875.34	23130	879.4	30790	881.11	41630	882.13	50580	882.67	60120	883.5	76860	884.74
286	415.84	843	7150	866.08	15860	875.42	23130	879.45	30790	881.14	41630	882.22	50580	882.82	60120	883.65	76860	885
287	415.90	844.1	7150	866.12	15860	875.48	23130	879.55	30790	881.37	41630	882.36	50580	882.96	60120	883.76	76860	885.06
288			22nd St SE or Cass Co. #34 / 170th Ave NW or Clay Co.36															
289	415.91	844.1	7150	866.12	15860	875.48	23130	879.54	30790	881.36	41630	882.48	50580	883.12	60120	884.18	76860	885.29
290	415.93	837.5	7150	866.14	15860	875.49	23130	879.56	30790	881.37	41630	882.56	50580	883.23	60120	884.18	76860	885.33
291	417.10	840.8	7150	866.55	15860	875.94	23130	880.11	30790	882.14	41630	883.45	50580	884.24	60120	885	76860	885.96
292	418.16	844.4	6247	866.96	13447	876.38	19446	880.64	25786	882.66	34892	884.04	42390	884.91	51664	885.65	67999	886.5
293	419.92	844.7	6247	867.73	13447	877.16	19446	881.37	25786	883.29	34892	884.7	42390	885.61	51664	886.15	67999	887.09
294	420.32	843.8	6247	868	13447	877.39	19446	881.62	25786	883.61	34892	885.09	42390	886.05	51664	886.61	67999	887.57
295	421.93	845.3	6247	869.23	13447	878.47	19446	882.62	25786	884.4	34892	885.84	42390	886.76	51664	887.44	67999	888.19
296	422.69	847.4	6247	869.55	13447	878.8	19446	882.92	25786	884.65	34892	886.06	42390	886.97	51664	887.73	67999	888.51
297	423.95	849.5	6247	870.08	13447	879.37	19446	883.58	25786	885.52	34892	887.14	42390	888.17	51664	888.91	67999	889.59
298	425.51	848.7	6247	870.82	13447	880.09	19446	884.3	25786	886.22	34892	887.86	42390	888.93	51664	889.3	67999	890.09
299	426.87	851.1	6247	871.53	13447	880.72	19446	884.94	25786	886.92	34892	888.68	42390	889.78	51664	890.02	67999	890.98
300	428.06	849.6	4007	872.07	9978	881.3	10598	885.47	10994.9	887.41	10244.1	889.14	10162.13	890.24	11631.11	890.29	18390.06	891.02
301	428.74	850.3	4007	872.33	9978	881.6	10598	885.65	10994.9	887.54	10244.1	889.21	10162.13	890.29	11631.11	890.36	18390.06	891.08
302	430.23	852	4007	872.89	9978	882.11	10598	885.95	10994.9	887.8	10244.1	889.38	10162.13	890.42	11631.11	890.53	18390.06	891.23
303	431.94	853.5	4007	873.65	9978	882.64	10598	886.39	10994.9	888.15	10438.72	889.6	12257.95	890.61	14337.65	890.77	22588.62	891.71
303.4	432.07	853.42	3721	873.69	9517	882.9	10012	886.45	10148.9	888.2	9375.91	889.64	10536.74	890.65	12417.95	890.82	20426.39	891.76
303.5	432.10	853.48	3721	873.72	9517	882.94	10012	886.47	10148.9	888.21	9706.64	889.66	10346.61	890.68	12206.51	890.84	20062.66	891.81
303.8	432.53	854.5	3721	873.88	9517	883.11	10012	886.56	10148.9	888.29	10244.77	889.71	9914.28	890.73	11579.62	890.93	19005.26	892.05
304	432.64	855.5	3721	873.96	9518	883.21	10013	886.62	10147.46	888.32	10181	889.73	9690.23	890.75	11299.55	890.95	17802.35	892.09
305	433.26	854.4	3721	874.11	9518	883.38	10013	886.73	10147.46	888.41	10285.2	889.79	9938.39	890.78	11082.17	891	15044.98	892.16
306	433.35	855	3721	874.18	9518	883.46	10013	886.79	10147.46	888.46	10294.47	889.84	9997.93	890.82	11030.36	891.04	14968.65	892.23
307			76th Ave. No. (Fargo) / 90th Ave NW (Moorhead) Bridge															
308	433.27	855	3721	874.19	9518	883.47	10013	886.8	10147.46	888.46	10294.47	889.84	9997.93	890.82	11030.36	891.05	14968.65	892.24
309	433.83	854.5	3721	874.32	9518	883.62	10013	886.9	10147.46	888.55	10294.47	889.92	9997.93	890.89	11030.36	891.12	14968.65	892.36
310	434.61	852.8	3721	874.56	9518	883.85	10013	887.05	10147.46	888.66	10294.47	890.01	9997.93	890.99	11030.36	891.21	14968.65	892.49
310.5	435.70	855	3721	874.9	9518	884.22	10013	887.27	10147.46	888.84	10294.47	890.16	9997.93	891.09	11030.36	891.36	14968.65	892.73
311	436.34	855	3721	875.11	9518	884.46	10013	887.44	10140.39	888.98	10265.56	890.28	10482.44	891.2	12199.26	891.49	17243.31	892.93
312	437.06	863.83	3526	875.33	9208	884.73	9608	887.63	9720.4	889.14	9815.56	890.42	10017.44	891.34	11619.26	891.67	16651.75	893.21
313	437.96	856	3526	875.65	9207	885.11	9607	887.9	9721.84	889.36	9879.34	890.61	10241.49	891.52	11899.32	891.91	17821.22	893.6
314	438.76	857	3526	875.97	9207	885.46	9607	888.15	9721.84	889.57	9879.34	890.79	10241.49	891.68	11899.32	892.12	17821.22	893.99
315	439.05	856	3526	876.06	9207	885.57	9607	888.22	9721.84	889.63	9879.34	890.84	10241.49	891.74	11899.32	892.19	17821.22	894.13
316	439.46	855.1	3526	876.2	9207	885.76	9607	888.37	9721.84	889.77	9879.34	890.97	10241.49	891.86	11899.32	892.34	17821.22	894.41
317			40th Ave. No. (Fargo) / Wall Street NW (Moorhead) Bridge															
318	439.47	855.1	3526	876.21	9207	885.77	9607	888.38	9721.84	889.77	9879.34	890.97	10241.49	891.86	11899.32	892.34	17821.22	894.41
319	439.50	857	3526	876.22	9207	885.82	9607	888.42	9721.84	889.8	9879.34	891	10241.49	891.89	11899.32	892.38	17821.22	894.46
320	439.66	856	3526	876.27	9207	885.88	9607	888.46	9721.84	889.84	9879.34	891.03	10241.49	891.91	11899.32	892.41	17821.22	894.52
321	440.31	858	3526	876.47	9207	886.13	9607	888.65	9721.84	890	9879.34	891.17	10241.49	892.05	11899.32	892.58	17821.22	894.81
322	440.32	858.1	3526	876.47	9207	886.12	9607	888.64	9721.84	889.99	9879.34	891.16	10241.49	892.04	11899.32	892.58	17821.22	894.81
323			Broadway Street No. (Fargo) / Broadway Street NW (Moorhead) Bridge															
324	440.33	858.1	3526	876.48	9207	886.24	9607	888.72	9721.84	890.05	9879.34	891.2	10241.49	892.08	11899.32	892.61	17821.22	894.85
325	440.34	857	3526	876.49	9207	886.25	9607	888.73	9721.84	890.06	9879.34	891.21	10241.49	892.08	11899.32	892.62	17821.22	894.85
326	440.54	856	3526	876.53	9207	886.3	9607	888.76	9721.84	890.08	9879.34	891.24	10241.49	892.11	11899.32	892.65	17821.22	894.92
327	441.53	859	3506	876.9	9162	886.58	9547	888.95	9656.84	890.23	9809.34	891.36	10166.49	892.22	11809.32	892.79	17711.22	895.13
328	442.14	858	3506	877.1	9162	886.75	9547	889.06	9656.84	890.32	9809.34	891.43	10177.6	892.29	11950.05	892.87	18759.55	895.27
329	442.45	857	3506	877.18	9162	886.78	9547	889.07	9656.84	890.33	9809.34	891.44	10233.32	892.29	12219.64	892.88	19890.08	895.27
329.5	442.93	858.06	3506	877.33	9161	886.89	9546	889.15	9662.98	890.38	9838.57	891.48	10249.66	892.33	12228.27	892.93	19959.57	895.36
330	443.69	856	3506	877.57	9161	887.09	9546	889.29	9662.98	890.49	9838.57	891.57	10249.66	892.41	12228.27	893.03	19959.57	895.49
331	443.99	859	3506	877.65	9161	887.18	9546	889.36	9662.98	890.56	9838.57	891.63	10249.66	892.47	12228.27	893.1	19959.57	895.63
332	444.33	856	3506	877.7	9161	887.22	9546	889.4	9662.98	890.58	9838.57	891.65	10249.66	892.49	12228.27	893.12	19959.57	895.67
333	444.92	859	3506	877.8	9161	887.26	9546	889.42	9662.98	890.6	9838.57	891.66	10249.66	892.5	12228.27	893.14	19959.57	895.69
334	445.18	860	3506	877.88	9161	887.34	9546	889.48	9662.98	890.65	9838.57	891.7	10249.66	892.54	12228.27	893.19	19959.57	895.77
335	445.31	860	3506	877.93	9161	887.42	9546	889.55	9662.98	890.71	9838.57	891.76	10249.66	892.6	12228.27	893.27	19959.57	895.94
336	445.36	859	3506	877.94	9161	887.43	9546	889.56	9662.98	890.72	9838.57	891.77	10249.66	892.6	12228.27	893.28	19959.57	895.96
336.5	445.76	857.69	3506	877.98	9161	887.46	9546	889.58	9662.98	890.74	9838.57	891.78	10249.66	892.62	12228.27	893.29	19959.57	895.99
337	446.09	860	3506	878.03	9161	887.46	9546	889.58	9662.98	890.73	9838.57	891.78	10249.66	892.61	12228.27	893.29	19959.57	895.98
338	446.29	861	3506	878.1	9161	887.5	9546	889.6	9662.98	890.75	9838.57	891.78	10249.66	892.62	12228.27	893.3	19959.57	895.99
338.5	446.70	860.9	3506	878.24	9161	887.64	9546	889.7	9662.98	890.83	9838.57	891.87	10249.66	892.69	12228.27	893.39	19959.57	896.14
339	446.76	861	3506	878.26	9161	887.66	9546	889.71	9662.98	890.84	9838.57	891.88	10249.66	892.7	12228.27	893.4	19959.57	896.15
340	447.04	860	3506	878.35	9161	887.75	9546	889.79	9662.98	890.91	9838.57	891.94	10249.66	892.76	12228.27	893.48	19959.57	896.33
341	447.24	859	3506	878.44	9161	887.81	9546	889.83	9662.98	890.95	9838.57	891.97	10249.66	892.79	12228.27	893.51	19959.57	896.36
342	447.42	861	3506	878.55	9161	887.9	9546	889.9	9662.98	891	9838.57	892.01	10249.66	892.83	12228.27	893.56	19959.57	896.45
342.5	447.78	856.99	3506	878.71	9161	888.04	9546	890	9662.98	891.09	9838.57	892.09	10249.66	892.9	12228.27	893.66	19959.57	896.52
343	448.13	861	3506	878.83	9161	888.14	9546	890.08	9662.98	891.16	9838.57	892.15	10249.66	892.96	12228.27	893.73	19959.57	896.73
344	448.36	859	3506	878.94	9161	888.25	9546	890.17	9662.98	891.23	9838.57	892.22	10249.66	893.03	12228.27	893.82	19959.57	896.92
345	448.69	865.3	3506	879.32	9161	888.48	9546	890.33	9662.98	891.36	9838.57	892.33	10249.66	893.13	12228.27	893.94	19959.57	897.1
348	448.90	864.3	3506	879.32	9161	888.46	9546	890.31	9662.98	891.35	9838.57	892.31	10249.66	893.11	12228.27	893.92	19959.57	897.05
348.4			North Dam															
348.8	448.90	864.3	3506	879.42	9161	888.51	9546	890.36	9662.98	891.39	9838.57	892.35	10249.66	893.14	12228.27	893.97	19959.57	897.1
351	448.91	861	3506	879.44	9161	888.51	9546	890.36	9662.98	891.39	9838.57	892.35	10249.66	893.15	12228.27	893.97	19959.57	897.11
351.4	449.06	861	3506	879.51	9161	888.64	9546	890.47	9662.98	891.49	9838.57	892.44	10249.66	893.24	12228.27	894.09	19959.57	897.31
351.5			12th Ave. No. (Fargo) / 15th Ave No. (Moorhead) Bridge															
351.6	449.07	861	3506	879.52	9161	888.7	9546	890.55	9662.98	891.57	9838.57	892.52	10249.66	893.3	12228.27	894.16	19959.57	897.39
352	449.09	862	3506	879.53	9161	888.72	9546	890.57	9662.98	891.59	9838.57	892.53	10249.66	893.32	12228.27	894.18	19959.57	897.42
353	449.28	863	3506	879.61	9161	888.81	9546	890.64	9662.98	891.65	9838.57	892.59	10249.66	893.36	12228.27	894.26	19959.57	897.57
353.5	449.61	860.13	3506	879.75	9161	888.91	9546	890.71	9662.98	891.71	9838.57	892.64	10249.66	893.42	12228.27	894.3	19959.57	897.63
354	449.97	864	3506	879.99	9161	889.13	9546	890.89	9662.98	891.87	9838.57	892.76	10249.66	893.55	12228.27	894.47	19959.57	897.84
355	450.19	862	3506	880.16	9161	889.22	9546	890.96	9662.98	891.93	9838.57	892.84	10249.66	893.61	12228.27	894.54	19959.57	897.99
356	450.84	858	3506	880.54	9161	889.5	9546	891.16	9662.98	892.1	9838.57	892.96	10249.66	893.74	12228.27	894.69	19959.57	898.2
357	450.86	863.4	3506	880.55	9161	889.5	9546	891.16	9662.98	892.1	9838.57	892.98	10249.66	893.74	12228.27	894.7	19959.57	898.2
358			BNRR Bridge															
359	450.87	863.4	3506	880.62	9161	889.6	9546	891.24	9662.98	892.16	9838.57	893.03	10249.66	893.78	12228.27	894.74	19959.57	898.31
361	451.20	865	3506	880.77	9161	889.72	9546	891.33	9662.98	892.24	9838.57	893.1	10249.66	893.85	12228.27	894.82	19959.57	898.44
361.5	451.37	865.2	3506	880.88	9161	889.79	9546	891.39	9662.98	892.29	9838.57	893.15	10249.66	893.89	12228.27	894.87	19959.57	898.51
362.5			1st Ave. No. Bridge															
364	451.39	865.2	3506	880.9	9161	889.81	9546	891.41	9662.98	892.3	9838.57	893.16	10249.66	893.9	12228.27	894.88	19959.57	898.53
366	451.41	861	3506	880.94	9161	889.86	9546	891.44	9662.98	892.33	9838.57	893.18	10249.66	893.93	12228.27	894.91	19959.57	898.57
367	451.57	867	3506	881	9161	889.95	9546	891.53	9662.98	892.42	9838.57	893.27	10249.66	894.01	12228.27	895.03	19959.57	898.81
368	451.61	866.8	3506	881.04	9161	890	9546	891.57	9662.98	892.46	9838.57	893.3	10249.66	894.04	12228.27	895.06	19959.57	898.86
369			NP Ave. (Fargo) / Center Ave. (Moorhead)															
370	451.63	866.8	3506	881.05	9161	890.01	9546	891.58	9662.98	892.46	9838.57	893.3	10249.66	894.05	12228.27	895.07	19959.57	898.88
371	451.65	859	3506	881.08	9161	890.01	9546	891.58	9662.98	892.46	9838.57	893.3	10249.66	894.04	12228.27	895.06	19959.57	898.86
372	451.67	867.9	3506	881.07	9161	890	9546	891.57	9662.98	892.45	9838.57	893.29	10249.66	894.04	12228.27	895.05	19959.57	898.86
373			BNRR Bridge															
374	451.66	867.9	3506	881.17	9161	890.1	9546	891.65	9662.98	892.52	9838.57	893.36	10249.66	894.1	12228.27	895.12	19959.57	898.92
375	451.69	864	3506	881.19	9161	890.1	9546	891.65	9662.98	892.52	9838.57	893.35	10249.66	894.09	12228.27	895.12	19959.57	898.9
376	451.71	865.9	3506	881.19	9161	890.11	9546	891.66	9662.98	892.54	9838.57	893.37	10249.66	894.12	12228.27	895.15	19959.57	898.99
377			Main Ave. Bridge															
378	451.72	865.9	3506	881.21	9161	890.12	9546	891.67	9662.98	892.55	9838.57	893.38	10249.66	894.12	12228.27	895.16	19959.57	899.02
379	451.73	864	3506	881.29	9161	890.22	9546	891.75	9662.98	892.62	9838.57	893.44	10249.66	894.18	12228.27	895.23	19959.57	899.11
380	451.85	864	3506	881.28	9161	890.19	9546	891.73	9662.98	892.59	9838.57	893.42	10249.66	894.16	12228.27	895.21	19959.57	899.08
381	452.03	863	3506	881.38	9161	890.34	9546	891.86	9662.98	892.72	9838.57	893.53	10249.66	894.27	12228.27	895.34	19959.57	899.27
381.5	452.04	863	3506	881.39	9161	890.35	9546	891.87	9662.98	892.73	9838.57	893.54	10249.66	894.28	12228.27	895.35	19959.57	899.28
384			Midtown Dam															
385.5	452.05	867	3506	881.46	9161	890.36	9546	891.88	9662.98	892.73	9838.57	893.55	10249.66	894.29	12228.27	895.36	19959.57	899.29
386	452.06	867	3506	881.47	9161	890.36	9546	891.88	9662.98	892.74	9838.57	893.55	10249.66	894.29	12228.27	895.36	19959.57	899.3
387	452.25	867	3506	881.61	9161	890.43	9546	891.93	9662.98	892.78	9838.57	893.59	10249.66	894.32	12228.27	895.4	19959.57	899.35
388	452.46	869	3506	881.83	9161	890.54	9546	892.02	9662.98	892.86	9838.57	893.66	10249.66	894.39	12228.27	895.48	19959.57	899.46
389	452.70	866.08	3506	882.1	9161	890.72	9546	892.16	9662.98	892.97	9838.57	893.76	10249.66	894.49	12228.27	895.59	19959.57	899.62
390	452.94	867	3506	882.22	9161	890.81	9546	892.24	9662.98	893.04	9838.57	893.82	10249.66	894.54	12228.27	895.66	19959.57	899.71
391	453.16	867	3506	882.35	9161	890.92	9546	892.33	9662.98	893.12	9838.57	893.89	10249.66	894.61	12228.27	895.74	19959.57	899.83
392	453.39	865	3506	882.48	9161	891.01	9546	892.4	9662.98	893.19	9838.57	893.96	10249.66	894.67	12228.27	895.81	19959.57	899.95
393	454.02	866	3506	882.63	9161	891.28	9546	892.61	9662.98	893.36	9838.57	894.11	10249.66	894.82	12228.27	895.97	19959.57	900.13
394	454.10	865	3506	882.87	9161	891.32	9546	892.64	9662.98	893.4	9838.57	894.14	10249.66	894.84	12228.27	896	19959.57	900.16
395	454.75	864	3506	883.14	9161	891.55	9546	892.83	9662.98	893.56	9838.57	894.29	10249.66	894.98	12228.27	896.16	19959.57	900.37
396	455.16	866	3506	883.33	9161	891.76	9546	893.01	9662.98	893.73	9838.57	894.43	10249.66	895.12	12228.27	896.32	19959.57	900.56
397	455.20	866.6	3506	883.37	9161	891.8	9546	893.04	9662.98	893.75	9838.57	894.45	10249.66	895.14	12228.27	896.34	19959.57	900.59
399			I-94 Bridge															
401	455.23	866.6	3506	883.38	9161	891.8	9546	893.04	9662.98	893.75	9838.57	894.46	10249.66	895.14	12228.27	896.34	19959.57	900.59
402	455.28	869	3506	883.41	9161	891.82	9546	893.06	9662.98	893.77	9838.57	894.47	10249.66	895.15	12228.27	896.36	19959.57	900.62
402.5	455.52	864.58	3506	883.52	9161	891.94	9546	893.16	9662.98	893.86	9838.57	894.55	10249.66	895.24	12228.27	896.45	19959.57	900.73
403	455.59	866	3506	883.54	9161	891.98	9546	893.2	9662.98	893.89	9838.57	894.58	10249.66	895.26	12228.27	896.48	19959.57	900.76
404	455.92	865	3506	883.7	9161	892.1	9546	893.3	9662.98	893.99	9838.57	894.67	10249.66	895.35	12228.27	896.58	19959.57	900.93
405	456.65	865	3506	884.09	9161	892.38	9546	893.53	9662.98	894.18	9838.57	894.85	10249.66	895.51	12228.27	896.76	19959.57	901.12
406	457.09	868	3506	884.33	9161	892.57	9546	893.7	9662.98	894.33	9838.57	894.99	10249.66	895.64	12228.27	896.91	19959.57	901.31
407	457.27	861.18	3506	884.4	9161	892.64	9546	893.75	9662.98	894.38	9838.57	895.03	10249.66	895.68	12228.27	896.95	19959.57	901.35
408	457.80	864	3506	884.57	9161	892.86	9546	893.95	9662.98	894.57	9838.57	895.2	10249.66	895.85	12228.27	897.15	19959.57	901.63
409	457.95	855.9	3506	884.62	9161	892.96	9546	894.04	9662.98	894.65	9838.57	895.28	10249.66	895.93	12228.27	897.23	19959.57	901.73
409.1	457.97	855.9	3506	884.62	9161	892.98	9546	894.04	9662.98	894.65	9838.57	895.28	10249.66	895.93	12228.27	897.24	19959.57	901.73

Table B19

Fargo Moorhead Metro Study - Phase 3.1
MN Short 35Kcfs, Year 50, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
409.5			South Dam															
409.6	457.96	855.9	3506	884.7	9161	893	9546	894.08	9662.98	894.69	9838.57	895.32	10249.66	895.96	12228.27	897.27	19959.57	901.76
409.7	457.99	855.9	3506	884.7	9161	893	9546	894.08	9662.98	894.69	9838.57	895.32	10249.66	895.98	12228.27	897.27	19959.57	901.77
413	458.32	869	3506	884.7	9161	893.02	9546	894.1	9662.98	894.71	9838.57	895.34	10249.66	895.98	12228.27	897.29	19959.57	901.81
414	458.75	866	3506	884.96	9161	893.29	9546	894.34	9662.98	894.93	9838.57	895.55	10249.66	896.19	12228.27	897.53	19959.57	902.13
459.64	459.64	864.81	3506	885.35	9161	893.73	9546	894.73	9662.98	895.28	9838.57	895.86	10249.66	896.48	12228.27	897.84	19959.57	902.47
460.28	460.28	868.74	3506	885.73	9161	894.07	9546	895.03	9662.98	895.56	9838.57	896.12	10249.66	896.73	12228.27	898.11	19959.57	902.77
460.72	460.72	869.04	3506	886.06	9161	894.34	9546	895.27	9662.98	895.78	9838.57	896.32	10249.66	896.93	12228.27	898.33	19959.57	903.05
461.3	461.3	868.04	3506	886.45	9161	894.73	9546	895.62	9662.98	896.11	9838.57	896.65	10249.66	897.22	12228.27	898.65	19959.57	903.46
462.03	462.03	864.16	3506	886.79	9161	895.12	9546	895.98	9662.98	896.44	9838.57	896.94	10249.66	897.53	12228.27	898.98	19959.57	903.83
462.04	462.04	869.5	3506	886.83	9161	895.16	9546	896.02	9662.98	896.47	9838.57	896.97	10249.66	897.55	12228.27	899.01	19959.57	903.88
462.05			52nd Avenue So. (Fargo) / 60th Ave So. (Moorhead) Bridge															
462.06	462.06	869.5	3506	886.84	9161	895.17	9546	896.02	9662.98	896.48	9838.57	896.98	10249.66	897.56	12228.27	899.02	19959.57	903.96
462.07	462.07	871.5	3506	886.88	9161	895.2	9546	896.05	9662.98	896.51	9838.57	897	10249.66	897.58	12228.27	899.04	19959.57	903.95
462.7	462.7	862.82	3506	887.24	9161	895.53	9546	896.35	9662.98	896.79	9838.57	897.26	10249.66	897.84	12228.27	899.32	19959.57	904.25
463.7	463.7	864.71	3506	887.58	9161	895.93	9546	896.73	9662.98	897.15	9838.57	897.61	10249.66	898.18	12228.27	899.7	19959.57	904.68
464.2	464.2	869.2	3506	887.8	9161	896.21	9546	896.99	9662.98	897.4	9838.57	897.85	10249.66	898.41	12228.27	899.96	19959.57	905
464.6	464.6	865	3506	888	9161	896.4	9546	897.17	9662.98	897.57	9838.57	898.01	10249.66	898.57	12228.27	900.13	19959.57	905.16
465.1	465.1	862.9	3506	888.15	9161	896.63	9546	897.4	9662.98	897.78	9838.57	898.22	10249.66	898.79	12228.27	900.37	19959.57	905.53
465.9	465.9	865.6	3506	888.41	9161	896.98	9546	897.73	9662.98	898.1	9838.57	898.53	10249.66	899.09	12228.27	900.71	19959.57	905.91
466.4	466.4	865.6	3506	888.6	9161	897.21	9546	897.94	9662.98	898.31	9838.57	898.72	10249.66	899.29	12228.27	900.92	19959.57	906.16
466.5	466.5	864.9	3506	888.64	9161	897.25	9546	897.99	9662.98	898.36	9838.57	898.77	10249.66	899.33	12228.27	900.97	19959.57	906.21
466.9	466.9	865	3506	888.68	9161	897.33	9546	898.06	9662.98	898.42	9838.57	898.84	10249.66	899.4	12228.27	901.04	19959.57	906.26
467			Proposed Red River Closure Structure															
467.1	467.1	865	3506	888.7	9161	897.39	9546	902.66	9662.98	905.75	9838.57	908.68	10249.66	910.19	12228.27	911.94	19959.57	913.8
467.2	467.2	864.8	3506	888.7	9161	897.38	9546	902.67	9662.98	905.75	9838.57	908.69	10249.66	910.21	12228.27	911.96	19959.57	913.86
467.3	467.3	864.8	3506	888.73	9161	897.43	9546	902.69	9662.98	905.77	9838.57	908.69	10249.66	910.21	12228.27	911.97	19959.57	913.88
467.4	467.4	864.8	3506	888.96	9161	897.71	9546	902.82	9662.98	905.86	9838.57	908.75	10249.66	910.26	12228.27	912.02	19959.57	913.98
468.2	468.2	862.6	3506	889.19	9161	897.99	12509	902.98	15532.81	905.97	20409.9	908.84	24432.98	910.34	30003.36	912.1	41338.51	914.11
468.9	468.9	861	3506	889.4	9161	898.27	12509	903.22	15532.81	906.2	20409.9	908.99	24432.98	910.48	30003.36	912.23	41338.51	914.26
469.7	469.7	870	3506	889.74	9161	898.66	12509	903.49	15532.81	906.41	20409.9	909.13	24432.98	910.61	30003.36	912.34	41338.51	914.38
470.1	470.1	870.9	3506	889.89	9161	898.84	12509	903.64	15532.81	906.55	20409.9	909.24	24432.98	910.7	30003.36	912.42	41338.51	914.46
470.6	470.6	869.4	2548	890.05	6444	899.03	7323	903.79	8011.81	906.69	10041.9	909.39	11885.98	910.84	16552.36	912.55	25037.51	914.6
471.4	471.4	867.5	2548	890.28	6358	899.28	7318	903.92	8111.81	906.78	10254.9	909.46	12147.98	910.9	16459.36	912.61	24231.51	914.68
472.18	472.18	869.9	2548	890.51	6358	899.52	7318	904.07	8111.81	906.89	10254.9	909.55	12147.98	910.98	16459.36	912.69	24231.51	914.76
472.2	472.2	870.8	2548	890.52	6358	899.51	7318	904.06	8111.81	906.88	10254.9	909.55	12147.98	910.97	16459.36	912.68	24231.51	914.75
472.22	472.22	870.8	2548	890.52	6358	899.52	7318	904.07	8111.81	906.89	10254.9	909.57	12147.98	910.99	16459.36	912.72	24231.51	914.8
473	473	871.8	2548	890.75	6358	899.73	7318	904.19	8111.81	906.98	10254.9	909.64	12147.98	911.06	16459.36	912.79	24231.51	914.88
473.8	473.8	869	2548	891	6358	900	7318	904.35	8111.81	907.1	10254.9	909.75	12147.98	911.18	16459.36	912.91	24231.51	915.01
474.08	474.08	873.5	2548	891.1	6358	900.09	7318	904.42	8111.81	907.14	10254.9	909.79	12147.98	911.22	16459.36	912.96	24231.51	915.07
474.09	474.09	873.5	2548	891.1	6358	900.11	7318	904.44	8111.81	907.16	10254.9	909.82	12147.98	911.25	16459.36	912.98	24231.51	915.09
474.1			Cass Co Hwy 16/112th Ave So. (Fargo) / 110th Ave So. (Moorhead) Bridge															
474.11	474.11	874	2548	891.1	6358	900.11	7318	904.43	8111.81	907.16	10254.9	909.81	12147.98	911.24	16459.36	913	24231.51	915.18
474.12	474.12	874	2548	891.11	6358	900.11	7318	904.44	8111.81	907.16	10254.9	909.82	12147.98	911.26	16459.36	913.02	24231.51	915.19
474.6	474.6	873.4	2548	891.32	6358	900.27	7318	904.53	8111.81	907.23	10254.9	909.89	12147.98	911.34	16459.36	913.11	24231.51	915.28
475.3	475.3	872.2	2548	891.57	6358	900.54	8774	904.71	11434	907.4	15609	910.07	19019	911.54	25243	913.35	36927	915.56
475.7	475.7	871	2548	891.7	6358	900.68	8774	904.86	11434	907.58	15609	910.32	19019	911.85	25243	913.67	36927	915.76
476.4	476.4	873.7	2548	891.97	6358	900.91	8774	905.07	11434	907.79	15609	910.56	19019	912.09	25243	913.92	36927	915.96
477	477	874	2550	892.24	5485	901.13	8718	905.29	12582	908.05	18134	910.87	22123	912.42	24258	914.25	29236	916.22
477.6	477.6	870.2	2550	892.42	5366	901.28	8710	905.5	12762	908.35	18543	911.27	22626	912.92	24116	914.6	28246	916.48
478	478	873.4	2550	892.56	5366	901.39	8710	905.64	12762	908.52	18543	911.5	22626	913.19	24116	914.78	28246	916.54
478.8	478.8	874.8	2550	892.79	5366	901.58	8710	905.89	12762	908.86	18543	911.95	22626	913.7	24116	915.14	28246	916.81
479.2	479.2	875.7	2550	892.91	5366	901.66	8710	906	12762	909.01	18543	912.13	22626	913.89	24116	915.28	28246	916.88
479.7	479.7	871.5	2550	893.1	5366	901.79	8710	906.15	12762	909.19	18543	912.37	22626	914.13	24116	915.48	28246	917.06
480.5	480.5	874.7	2550	893.38	5366	902	8710	906.42	12762	909.54	18543	912.77	22626	914.53	24116	915.79	28246	917.32
481.6	481.6	875.3	2550	893.76	5366	902.33	8710	906.86	12762	910.07	18543	913.35	22626	915.16	24116	916.29	28246	917.74
482.3	482.3	874.2	2550	893.97	5366	902.51	8710	907.1	12762	910.37	18543	913.69	22626	915.5	24116	916.54	28246	917.92
482.698	482.698	869.74	2550	894.07	5366	902.6	8710	907.22	12762	910.52	18543	913.86	22626	915.69	24116	916.73	28246	918.07
482.7	482.7	887.04	2550	893.94	5366	902.54	8710	907.15	12762	910.44	18543	913.76	22626	915.62	24116	916.69	28246	918.07
482.702	482.702	878.54	2550	894.12	5366	902.61	8710	907.23	12762	910.52	18543	913.86	22626	915.69	24116	916.73	28246	918.07
483.1	483.1	879.2	2550	894.36	5366	902.81	8710	907.47	12762	910.84	18543	914.22	22626	916.06	24116	917.01	28246	918.26
483.9	483.9	873	2550	894.82	5366	903.1	8710	907.8	12762	911.26	18543	914.74	22626	916.55	24116	917.46	28246	918.64
485.08	485.08	880.5	2550	895.47	5366	903.41	8710	908.06	12762	911.53	18543	915.03	22626	916.86	24116	917.74	28246	918.92
485.09	485.09	880.5	2550	895.48	5366	903.4	8710	908.04	12762	911.54	18543	915.03	22626	916.86	24116	917.74	28246	918.92
485.1			Cass Co. Hwy 16/52nd St. SE (Fargo) / 160th Ave So. (Moorhead) Bridge															
485.11	485.11	879.4	2550	895.48	5366	903.4	8710	908.08	12762	911.66	18543	915.11	22626	916.92	24116	917.78	28246	918.95
485.12	485.12	879.4	2550	895.49	5366	903.43	8710	908.14	12762	911.66	18543	915.11	22626	916.92	24116	917.79	28246	918.96
485.6	485.6	878.62	2550	895.71	5366	903.55	8710	908.25	12762	911.76	18543	915.2	22626	917.01	24116	917.88	28246	919.06
485.9	485.9	879.5	2550	895.92	5366	903.67	8710	908.38	12762	911.91	18543	915.38	22626	917.21	24116	918.07	28246	919.27
486.3	486.3	872.1	2550	896.1	5366	903.8	8710	908.53	12762	912.08	18543	915.59	22626	917.44	24116	918.3	28246	919.52
487	487	871.3	2550	896.32	5366	903.98	8710	908.74	12762	912.36	18543	915.96	22626	917.9	24116	918.78	28246	920.08
487.5	487.5	878.8	2550	896.52	5366	904.17	8710	908.96	12762	912.59	18543	916.23	22626	918.18	24116	919.05	28246	920.36
488.68	488.66	878.24	2550	897.12	5366	904.71	8710	909.55	12762	913.34	18543	917.09	22626	919.11	24116	919.96	28246	921.36

Table B20

Fargo Moorhead Metro Study - Phase 3.1
ND East 35Kcfs, Year Zero - Wet, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
279	403.65	832.3	10380	863.05	20140	871.03	27860	874.18	36050	875.88	47720	877.03	57070	877.85	69040	878.81	85540	879.53
280	404.98	836.6	10380	863.75	20140	871.81	27860	874.78	36050	876.5	47720	877.78	57070	878.66	69040	879.62	85540	880.37
281	406.51	834.9	10380	864.68	20140	872.74	27860	875.86	36050	877.71	47720	878.82	57070	879.66	69040	880.42	85540	881.28
282	410.21	838	10380	866.68	20140	874.62	27860	877.64	36050	879.39	47720	880.1	57070	880.81	69040	881.67	85540	882.67
283	413.23	837.6	10380	868.04	20140	876	27860	879	36050	880.5	47720	881	57070	881.76	69040	882.67	85540	883.75
284	414.98	842.7	10380	868.85	20140	877.02	27860	880.18	36050	881.44	47720	882.04	57070	882.8	69040	883.73	85540	884.85
285	415.38	841.4	10380	869.07	20140	877.28	27860	880.42	36050	881.65	47720	882.34	57070	883.06	69040	884	85540	885.12
286	415.84	843	10380	869.21	20140	877.35	27860	880.48	36050	881.72	47720	882.46	57070	883.22	69040	884.26	85540	885.41
287	415.90	844.1	10380	869.27	20140	877.44	27860	880.62	36050	881.93	47720	882.62	57070	883.35	69040	884.24	85540	885.48
288			22nd St SE or Cass Co. #34 / 170th Ave NW or Clay Co.38															
289	415.91	844.1	10380	869.26	20140	877.43	27860	880.6	36050	881.95	47720	882.76	57070	883.59	69040	884.76	85540	885.73
290	415.93	837.5	10380	869.29	20140	877.45	27860	880.62	36050	881.97	47720	882.87	57070	883.69	69040	884.76	85540	885.77
291	417.10	840.8	10380	869.78	20140	878	27860	881.33	36050	882.81	47720	883.9	57070	884.76	69040	885.53	85540	886.42
292	418.18	844.4	9068	870.26	17079	878.54	23429	881.88	30241	883.37	40116	884.57	47970	885.39	59765	886.04	75940	886.98
293	418.92	844.7	8686	871.15	13290	879.4	13680.08	882.53	14039.03	883.96	14391.02	885.2	15081.65	885.86	21139.43	886.6	31807.62	887.65
294	420.32	843.8	8686	871.42	13290	879.55	13680.08	882.63	14039.03	884.04	14391.02	885.25	15081.65	885.91	21139.43	886.66	31807.62	887.74
295	421.93	845.3	8686	872.65	13290	880.3	13680.08	883	14039.03	884.25	14391.02	885.38	15081.65	886.02	21139.43	886.81	31807.62	887.89
296	422.69	847.4	8686	872.99	13290	880.53	13680.08	883.13	14039.03	884.33	14391.02	885.43	15081.65	886.05	21139.43	886.86	31807.62	887.98
297	423.95	849.5	8686	873.56	13290	880.97	13680.08	883.46	14039.03	884.62	14391.02	885.67	15081.65	886.28	21139.43	887.2	31807.62	888.4
298	425.51	848.7	8686	874.32	13290	881.52	13680.08	883.84	14039.03	884.91	14391.02	885.88	15081.65	886.47	21139.43	887.47	31807.62	888.81
299	426.87	851.1	8686	875.03	13290	882.03	13680.08	884.2	14039.03	885.21	14391.02	886.13	15081.65	886.7	21139.43	887.81	31807.62	889.32
300	428.06	849.6	6116	875.6	10242	882.51	10442.08	884.54	10593.03	885.5	10704.02	886.37	11332.31	886.92	16940.61	888.12	22924.77	889.74
301	428.74	850.3	6116	875.9	10242	882.77	10442.08	884.74	10593.03	885.69	10704.02	886.53	11332.31	887.08	16940.61	888.4	22924.77	890.05
302	430.23	852	6116	876.48	10242	883.22	10442.08	885.08	10593.03	885.98	10704.02	886.8	11332.31	887.37	16940.61	888.91	22924.77	890.73
303	431.94	853.5	6116	877.27	10242	883.86	10442.08	885.58	10593.03	886.42	10704.02	887.19	11332.31	887.76	17021.09	889.54	26758.36	891.53
303.4	432.07	853.42	5825	877.32	9942	883.92	10142.08	885.63	10293.03	886.47	10404.02	887.24	11032.31	887.81	16696.73	889.65	26307.2	891.57
303.5	432.10	853.46	5825	877.36	9942	883.95	10142.08	885.66	10293.03	886.5	10404.02	887.25	11032.31	887.83	16655.68	889.66	26063.31	891.65
303.8	432.53	854.5	5825	877.54	9942	884.1	10142.08	885.77	10293.03	886.59	10404.02	887.34	11032.31	887.92	16586.96	889.84	25296.24	892.07
304	432.84	855.5	5825	877.64	9943	884.19	10143.08	885.84	10294.03	886.65	10405.02	887.39	11032.71	887.97	16318.54	889.91	22404.75	892.15
305	433.26	854.4	5825	877.81	9943	884.36	10143.08	885.97	10294.03	886.77	10405.02	887.5	11032.71	888.08	16510.89	890.06	20331.37	892.25
306	433.35	855	5825	877.89	9943	884.43	10143.08	886.03	10294.03	886.83	10405.02	887.56	11032.71	888.14	16674.01	890.17	19637.48	892.38
307			76th Ave. No. (Fargo) / 90th Ave NW (Moorhead) Bridge															
308	433.37	855	5825	877.9	9943	884.44	10143.08	886.04	10294.03	886.84	10405.02	887.56	11032.71	888.14	16574.01	890.18	19637.48	892.4
309	433.83	854.5	5825	878.05	9943	884.58	10143.08	886.16	10294.03	886.95	10405.02	887.67	11032.71	888.25	16574.01	890.37	19637.48	892.61
310	434.61	852.8	5825	878.31	9943	884.8	10143.08	886.33	10294.03	887.1	10405.02	887.8	11032.71	888.39	16574.01	890.6	19637.48	892.83
310.5	435.70	855	5825	878.7	9943	885.13	10143.08	886.59	10294.03	887.34	10405.02	888.02	11032.71	888.61	16574.01	890.95	19637.48	893.22
311	436.34	855	5825	878.94	9943	885.36	10143.08	886.78	10294.03	887.51	10405.02	888.18	11024.92	888.78	15924.61	891.26	21486.16	893.54
312	437.05	853.53	5630	879.2	9663	885.61	9773.08	886.99	9899.03	887.71	9975.02	888.37	10584.92	888.97	15449.61	891.56	21087.35	893.93
313	437.95	856	5630	879.57	9662	885.98	9772.08	887.3	9898.03	888	9974.02	888.63	10584.52	889.25	15717.04	892	23869.86	894.48
314	438.76	857	5630	879.92	9662	886.32	9772.08	887.58	9898.03	888.25	9974.02	888.88	10584.52	889.49	15717.04	892.35	23869.86	895.06
315	439.05	856	5630	880.03	9662	886.42	9772.08	887.66	9898.03	888.33	9974.02	888.95	10584.52	889.57	15717.04	892.47	23869.86	895.28
316	439.46	855.1	5630	880.2	9662	886.61	9772.08	887.83	9898.03	888.49	9974.02	889.1	10584.52	889.73	15717.04	892.73	22545.57	895.75
317			40th Ave. No. (Fargo) / Wall Street NW (Moorhead) Bridge															
318	439.47	855.1	5630	880.21	9662	886.62	9772.08	887.84	9898.03	888.5	9974.02	889.1	10584.52	889.73	15717.04	892.73	22545.57	895.74
319	439.50	857	5630	880.22	9662	886.67	9772.08	887.88	9898.03	888.54	9974.02	889.14	10584.52	889.76	15717.04	892.8	22545.57	895.82
320	439.66	856	5630	880.29	9662	886.72	9772.08	887.93	9898.03	888.59	9974.02	889.19	10584.52	889.82	15717.04	892.85	22545.57	895.89
321	440.31	858	5630	880.52	9662	886.97	9772.08	888.14	9898.03	888.78	9974.02	889.37	10584.52	890	15717.04	893.13	22545.57	896.28
322	440.32	858.1	5630	880.51	9662	886.96	9772.08	888.14	9898.03	888.77	9974.02	889.36	10584.52	890	15717.04	893.13	22545.57	896.27
323			Broadway Street No. (Fargo) / Broadway Street NW (Moorhead) Bridge															
324	440.33	858.1	5630	880.56	9662	887.07	9772.08	888.23	9898.03	888.85	9974.02	889.43	10584.52	890.06	15717.04	893.18	22545.57	896.32
325	440.34	857	5630	880.56	9662	887.08	9772.08	888.23	9898.03	888.86	9974.02	889.44	10584.52	890.07	15717.04	893.19	22545.57	896.32
326	440.54	858	5630	880.61	9662	887.13	9772.08	888.27	9898.03	888.9	9974.02	889.47	10584.52	890.11	15717.04	893.24	22545.57	896.4
327	441.53	859	5600	880.98	9617	887.39	9717.08	888.49	9833.03	889.09	9914.02	889.65	10514.52	890.28	15642.04	893.46	22468.45	895.7
328	442.14	858	5600	881.18	9617	887.54	9717.08	888.61	9833.03	889.2	9914.02	889.75	10514.52	890.38	15866.35	893.59	24473.8	895.88
329	442.45	857	5600	881.24	9617	887.57	9717.08	888.63	9833.03	889.21	9914.02	889.76	10514.52	890.39	16275.85	893.6	26572.38	896.89
329.5	442.93	858.06	5600	881.38	9616	887.67	9716.08	888.72	9832.03	889.29	9913.02	889.83	10521.65	890.46	16290.43	893.68	26784.62	896.99
330	443.69	858	5600	881.61	9616	887.86	9716.08	888.87	9832.03	889.44	9913.02	889.96	10521.65	890.59	16290.43	893.82	26784.62	897.16
331	443.99	859	5600	881.69	9616	887.95	9716.08	888.95	9832.03	889.52	9913.02	890.04	10521.65	890.66	16290.43	893.93	26784.62	897.38
332	444.33	858	5600	881.74	9616	887.98	9716.08	888.99	9832.03	889.55	9913.02	890.08	10521.65	890.69	16290.43	893.97	26784.62	897.44
333	444.92	859	5600	881.81	9616	888.02	9716.08	889.02	9832.03	889.58	9913.02	890.09	10521.65	890.72	16290.43	893.99	26784.62	897.45
334	445.18	860	5600	881.89	9616	888.09	9716.08	889.08	9832.03	889.64	9913.02	890.14	10521.65	890.77	16290.43	894.07	26784.62	897.54
335	445.31	860	5600	881.96	9616	888.18	9716.08	889.16	9832.03	889.71	9913.02	890.21	10521.65	890.84	16290.43	894.2	26784.62	897.83
336	445.36	859	5600	881.96	9616	888.19	9716.08	889.17	9832.03	889.72	9913.02	890.22	10521.65	890.85	16290.43	894.21	26784.62	897.84
336.5	445.76	857.69	5600	881.99	9616	888.21	9716.08	889.19	9832.03	889.74	9913.02	890.24	10521.65	890.87	16290.43	894.24	26784.62	897.89
337	446.09	860	5600	882.01	9616	888.21	9716.08	889.19	9832.03	889.73	9913.02	890.24	10521.65	890.87	16290.43	894.23	26784.62	897.67
338	446.29	861	5600	882.08	9616	888.25	9716.08	889.21	9832.03	889.76	9913.02	890.26	10521.65	890.88	16290.43	894.24	26784.62	897.87
338.5	446.70	860.9	5600	882.24	9616	888.38	9716.08	889.32	9832.03	889.86	9913.02	890.35	10521.65	890.98	16290.43	894.38	26784.62	898.08
339	446.76	861	5600	882.26	9616	888.4	9716.08	889.34	9832.03	889.87	9913.02	890.37	10521.65	890.99	16290.43	894.39	26784.62	898.11
340	447.04	860	5600	882.34	9616	888.49	9716.08	889.42	9832.03	889.95	9913.02	890.44	10521.65	891.06	16290.43	894.53	26784.62	898.37
341	447.24	859	5600	882.43	9616	888.55	9716.08	889.47	9832.03	890	9913.02	890.48	10521.65	891.11	16290.43	894.57	26784.62	898.41
342	447.42	861	5600	882.54	9616	888.63	9716.08	889.54	9832.03	890.08	9913.02	890.54	10521.65	891.17	16290.43	894.64	26784.62	898.52
342.5	447.78	856.99	5600	882.69	9616	888.77	9716.08	889.66	9832.03	890.17	9913.02	890.64	10521.65	891.28	16290.43	894.79	26784.62	898.76
343	448.13	861	5600	882.81	9616	888.87	9716.08	889.75	9832.03	890.25	9913.02	890.72	10521.65	891.35	16290.43	894.89	26784.62	898.9
344	448.36	859	5600	882.91	9616	888.97	9716.08	889.84	9832.03	890.34	9913.02	890.81	10521.65	891.44	16290.43	895.04	26784.62	899.19
345	448.89	865.3	5600	883.22	9616	889.18	9716.08	890.02	9832.03	890.51	9913.02	890.96	10521.65	891.59	16290.43	895.21	26784.62	899.4
348	448.90	864.3	5600	883.21	9616	889.16	9716.08	890	9832.03	890.49	9913.02	890.94	10521.65	891.57	16290.43	895.17	26784.62	899.34
348.4			North Dam															
348.8	448.90	864.3	5600	883.28	9616	889.21	9716.08	890.05	9832.03	890.53	9913.02	890.98	10521.65	891.62	16290.43	895.22	26784.62	899.41
351	448.91	861	5600	883.29	9616	889.22	9716.08	890.05	9832.03	890.54	9913.02	890.98	10521.65	891.63	16290.43	895.22	26784.62	899.41
351.4	449.06	861	5600	883.38	9616	889.34	9716.08	890.17	9832.03	890.65	9913.02	891.09	10521.65	891.74	16290.43	895.4	26784.62	899.7
351.5			12th Ave. No. (Fargo) / 15th Ave No. (Moorhead) Bridge															
351.6	449.07	861	5600	883.39	9616	889.42	9716.08	890.25	9832.03	890.74	9913.02	891.19	10521.65	891.83	16290.43	895.49	26784.62	899.75
352	449.09	862	5600	883.4	9616	889.44	9716.08	890.27	9832.03	890.76	9913.02	891.2	10521.65	891.85	16290.43	895.52	26784.62	899.79
353	449.28	863	5600	883.49	9616	889.53	9716.08	890.35	9832.03	890.83	9913.02	891.27	10521.65	891.93	16290.43	895.64	26784.62	899.99
353.5	449.61	860.13	5600	883.61	9616	889.62	9716.08	890.42	9832.03	890.9	9913.02	891.34	10521.65	891.98	16290.43	895.7	26784.62	900.08
354	449.97	864	5600	883.86	9616	889.83	9716.08	890.62	9832.03	891.08	9913.02	891.51	10521.65	892.17	16290.43	895.91	26784.62	900.31
355	450.19	862	5600	883.98	9616	889.92	9716.08	890.7	9832.03	891.16	9913.02	891.59	10521.65	892.25	16290.43	896.03	26784.62	900.49
356	450.84	858	5600	884.32	9616	890.18	9716.08	890.91	9832.03	891.36	9913.02	891.77	10521.65	892.43	16290.43	896.23	26784.62	900.72
357	450.86	863.4	5600	884.33	9616	890.18	9716.08	890.92	9832.03	891.36	9913.02	891.77	10521.65	892.43	16290.43	896.23	26784.62	900.73
358			BNRR Bridge															
359	450.87	863.4	5600	884.41	9616	890.28	9716.08	891	9832.03	891.44	9913.02	891.84	10521.65	892.49	16290.43	896.28	26784.62	900.93
361	451.20	865	5600	884.56	9616	890.39	9716.08	891.1	9832.03	891.54	9913.02	891.93	10521.65	892.59	16290.43	896.4	26784.62	901.09
361.5	451.37	865.2	5600	884.65	9616	890.46	9716.08	891.17	9832.03	891.59	9913.02	891.99	10521.65	892.64	16290.43	896.46	26784.62	901.17
362.5			1st Ave. No. Bridge															
364	451.39	865.2	5600	884.67	9616	890.49	9716.08	891.18	9832.03	891.61	9913.02	892	10521.65	892.66	16290.43	896.48	26784.62	901.26
366	451.41	861	5600	884.71	9616	890.53	9716.08	891.22	9832.03	891.65	9913.02	892.04	10521.65	892.69	16290.43	896.52	26784.62	901.3
367	451.57	867	5600	884.77	9616	890.63	9716.08	891.32	9832.03	891.74	9913.02	892.13	10521.65	892.79	16290.43	896.71	26784.62	901.65
368	451.61	866.8	5600	884.82	9616	890.67	9716.08	891.36	9832.03	891.78	9913.02	892.17	10521.65	892.83	16290.43	896.75	26784.62	901.71
369			NP Ave. (Fargo) / Center Ave. (Moorhead)															
370	451.63	866.8	5600	884.83	9616	890.68	9716.08	891.37	9832.03	891.79	9913.02	892.18	10521.65	892.83	16290.43	896.78	26784.62	901.77
371	451.65	859	5600	884.85	9616	890.68	9716.08	891.37	9832.03	891.79	9913.02	892.17	10521.65	892.83	16290.43	896.77	26784.62	901.74
372	451.67	867.9	5600	884.84	9616	890.67	9716.08	891.36	9832.03	891.78	9913.02	892.17	10521.65	892.82	16290.43	896.76	26784.62	901.74
373			BNRR Bridge															
374	451.68	867.9	5600	884.94	9616	890.77	9716.08	891.44	9832.03	891.85	9913.02	892.24	10521.65	892.9	16290.43	896.83	26784.62	901.8
375	451.69	864	5600	884.96	9616	890.77	9716.08	891.44	9832.03	891.86	9913.02	892.24	10521.65	892.9	16290.43	896.82	26784.62	901.78
376	451.71	865.9	5600	884.96	9616	890.78	9716.08	891.46	9832.03	891.88	9913.02	892.26	10521.65	892.92	16290.43	896.86	26784.62	901.93
377			Main Ave. Bridge															
378	451.72	865.9	5600	884.98	9616	890.79	9716.08	891.47	9832.03	891.89	9913.02	892.27	10521.65	892.93	16290.43	896.9	26784.62	901.96
379	451.73	864	5600	885.07	9616	890.89	9716.08	891.56	9832.03	891.97	9913.02	892.35	10521.65	893.01	16290.43	896.99	26784.62	902.05
380	451.85	864	5600	885.05	9616	890.86	9716.08	891.53	9832.03	891.95	9913.02	892.33	10521.65	892.98	16290.43	896.96	26784.62	902.02
381	452.03	863	5600	885.17	9616	891.01	9716.08	891.67	9832.03	892.08	9913.02	892.45	10521.65	893.12	16290.43	897.14	26784.62	902.24
381.5	452.04	863	5600	885.18	9616	891.02	9716.08	891.68	9832.03	892.09	9913.02	892.46	10521.65	893.13	16290.43	897.15	26784.62	902.25
384			Midtown Dam															
385.5	452.05	867	5600	885.2	9616	891.03	9716.08	891.69	9832.03	892.1	9913.02	892.47	10521.65	893.14	16290.43	897.16	26784.62	902.26
386	452.06	867	5600	885.21	9616	891.03	9716.08	891.69	9832.03	892.1	9913.02	892.48	10521.65	893.14	16290.43	897.16	26784.62	902.27
387	452.25	867	5600	885.31	9616	891.1	9716.08	891.75	9832.03	892.15	9913.02	892.52	10521.65	893.18	16290.43	897.21	26784.62	902.32
388	452.46	869	5600	885.47	9616	891.2	9716.08	891.84	9832.03	892.24	9913.02	892.61	10521.65	893.27	16290.43	897.31	26784.62	902.47
389	452.70	866.08	5600	885.7	9616	891.37	9716.08	891.98	9832.03	892.38	9913.02	892.74	10521.65	893.4	16290.43	897.46	26784.62	902.66
390	452.94	867	5600	885.81	9616	891.46	9716.08	892.07	9832.03	892.46	9913.02	892.81	10521.65	893.47	16290.43	897.55	26784.62	902.76
391	453.16	867	5600	885.94	9616	891.57	9716.08	892.17	9832.03	892.55	9913.02	892.9	10521.65	893.56	16290.43	897.65	26784.62	902.89
392	453.39	865	5600	886.04	9616	891.66	9716.08	892.25	9832.03	892.63	9913.02	892.98	10521.65	893.64	16290.43	897.76	26784.62	903.05
393	454.02	866	5600	886.38	9616	891.9	9716.08	892.47	9832.03	892.84	9913.02	893.17	10521.65	893.82	16290.43	897.94	26784.62	903.23
394	454.10	865	5600	886.41	9616	891.94	9716.08	892.51	9832.03	892.87	9913.02	893.2	10521.65	893.86	16290.43	897.97	26784.62	903.25
395	454.75	864	5600	886.69	9616	892.17	9716.08	892.71	9832.03	893.06	9913.02	893.39	10521.65	894.04	16290.43	898.17	26784.62	903.47
396	455.16	866	5600	886.89	9616	892.38	9716.08	892.9	9832.03	893.25	9913.02	893.56	10521.65	894.21	16290.43	898.37	26784.62	903.63
397	455.20	866.6	5600	886.93	9616	892.41	9716.08	892.93	9832.03	893.28	9913.02	893.59	10521.65	894.24	16290.43	898.4	26784.62	903.67
399			I-94 Bridge															
401	455.23	866.6	5600	886.94	9616	892.41	9716.08	892.93	9832.03	893.28	9913.02	893.59	10521.65	894.24	16290.43	898.4	26784.62	903.67
402	455.28	865	5600	886.96	9616	892.43	9716.08	892.95	9832.03	893.29	9913.02	893.6	10521.65	894.26	16290.43	898.42	26784.62	903.69
402.5	455.52	864.58	5600	887.08	9616	892.56	9716.08	893.06	9832.03	893.4	9913.02	893.71	10521.65	894.36	16290.43	898.53	26784.62	903.83
403	455.59	866	5600	887.11	9616	892.59	9716.08	893.1	9832.03	893.44	9913.02	893.74	10521.65	894.39	16290.43	898.57	26784.62	903.89
404	455.92	865	5600	887.25	9616	892.71	9716.08	893.21	9832.03	893.54	9913.02	893.84	10521.65	894.5	16290.43	898.71	26784.62	904.08
405	456.65	865	5600	887.59	9616	892.98	9716.08	893.45	9832.03	893.77	9913.02	894.06	10521.65	894.71	16290.43	898.9	26784.62	904.26
406	457.09	866	5600	887.82	9616	893.17	9716.08	893.62	9832.03	893.94	9913.02	894.22	10521.65	894.87	16290.43	899.09	26784.62	904.47

Table B20

Fargo Moorhead Metro Study - Phase 3.1
ND East 35Kcfs, Year Zero - Wet, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
407	457.27	861.18	5600	887.89	9616	893.23	9716.08	893.68	9832.03	893.99	9913.02	894.27	10521.65	894.92	16290.43	899.13	26784.62	904.5
408	457.80	864	5600	888.09	9616	893.45	9716.08	893.89	9832.03	894.2	9913.02	894.47	10521.65	895.12	16290.43	899.39	26784.62	904.84
409	457.95	865.9	5600	888.16	9616	893.55	9716.08	893.98	9832.03	894.29	9913.02	894.56	10521.65	895.21	16290.43	899.48	26784.62	904.92
409.1	457.97	855.9	5600	888.17	9616	893.55	9716.08	893.99	9832.03	894.29	9913.02	894.56	10521.65	895.21	16290.43	899.48	26784.62	904.93
409.5			South Dam															
409.6	457.98	855.9	5600	888.21	9616	893.59	9716.08	894.02	9832.03	894.33	9913.02	894.6	10521.65	895.25	16290.43	899.51	26784.62	904.96
409.7	457.99	855.9	5600	888.21	9616	893.59	9716.08	894.02	9832.03	894.33	9913.02	894.6	10521.65	895.25	16290.43	899.52	26784.62	904.97
413	458.32	869	5600	888.22	9616	893.61	9716.08	894.05	9832.03	894.35	9913.02	894.62	10521.65	895.27	16290.43	899.56	26784.62	905
414	458.75	866	5600	888.5	9616	893.88	9716.08	894.3	9832.03	894.6	9913.02	894.86	10521.65	895.51	16290.43	899.85	26784.62	905.37
459.64	459.64	864.81	5600	888.92	9616	894.31	9716.08	894.7	9832.03	894.98	9913.02	895.23	10521.65	895.88	16290.43	900.2	26784.62	905.7
460.28	460.28	868.74	5600	889.3	9616	894.65	9716.08	895.02	9832.03	895.29	9913.02	895.53	10521.65	896.17	16290.43	900.5	26784.62	905.97
460.72	460.72	869.04	5600	889.6	9616	894.91	9716.08	895.26	9832.03	895.53	9913.02	895.76	10521.65	896.4	16290.43	900.76	26784.62	906.29
461.3	461.3	868.04	5600	890	9616	895.29	9716.08	895.63	9832.03	895.88	9913.02	896.1	10521.65	896.74	16290.43	901.15	26784.62	906.72
462.03	462.03	864.16	5600	890.39	9616	895.68	9716.08	896	9832.03	896.24	9913.02	896.48	10521.65	897.09	16290.43	901.52	26784.62	907.1
462.04	462.04	869.5	5600	890.43	9616	895.72	9716.08	896.04	9832.03	896.28	9913.02	896.49	10521.65	897.12	16290.43	901.54	26784.62	907.14
462.05			52nd Avenue So. (Fargo) / 60th Ave So. (Moorhead) Bridge															
462.06	462.06	869.5	5600	890.44	9616	895.73	9716.08	896.04	9832.03	896.29	9913.02	896.5	10521.65	897.13	16290.43	901.56	26784.62	907.18
462.07	462.07	871.5	5600	890.47	9616	895.75	9716.08	896.07	9832.03	896.31	9913.02	896.52	10521.65	897.16	16290.43	901.59	26784.62	907.18
462.7	462.7	862.82	5600	890.82	9616	896.08	9716.08	896.38	9832.03	896.61	9913.02	896.82	10521.65	897.45	16290.43	901.89	26784.62	907.49
463.7	463.7	864.71	5600	891.19	9616	896.48	9716.08	896.77	9832.03	897	9913.02	897.2	10521.65	897.83	16290.43	902.35	26784.62	907.93
464.2	464.2	869.2	5600	891.45	9616	896.76	9716.08	897.04	9832.03	897.26	9913.02	897.45	10521.65	898.09	16290.43	902.63	26784.62	908.29
464.6	464.6	865	5600	891.66	9616	896.95	9716.08	897.23	9832.03	897.44	9913.02	897.63	10521.65	898.27	16290.43	902.81	26784.62	908.45
465.1	465.1	862.9	5600	891.86	9616	897.19	9716.08	897.46	9832.03	897.67	9913.02	897.86	10521.65	898.5	16290.43	903.12	26784.62	908.81
465.9	465.9	865.6	5600	892.15	9616	897.53	9716.08	897.79	9832.03	898.01	9913.02	898.19	10521.65	898.84	16290.43	903.49	26784.62	909.11
466.5	466.5	864.9	5600	892.4	9616	897.8	9716.08	898.06	9832.03	898.27	9913.02	898.44	10521.65	899.09	16290.43	903.78	26784.62	909.41
467.4	467.4	864.8	5600	892.78	9616	898.21	9716.08	898.46	9832.03	898.65	9913.02	898.84	10521.65	899.49	16290.43	904.24	26784.62	909.86
468.2	468.2	862.6	5600	893.05	9616	898.49	9716.08	898.73	9832.03	898.93	9913.02	899.1	10521.65	899.76	16290.43	904.54	26784.62	910.06
468.9	468.9	861	5600	893.29	9616	898.77	9716.08	899.01	9832.03	899.21	9913.02	899.37	10521.65	900.04	16290.43	904.85	26784.62	910.24
469.7	469.7	870	5600	893.67	9616	899.16	9716.08	899.4	9832.03	899.59	9913.02	899.75	10521.65	900.42	16290.43	905.16	26784.62	910.39
470.1	470.1	870.9	5600	893.84	9616	899.34	9716.08	899.58	9832.03	899.77	9913.02	899.92	10521.65	900.59	16290.43	905.36	26784.62	910.51
470.6	470.6	869.4	4166	894.02	7266	899.54	7365.08	899.76	7461.03	899.95	7581.02	900.11	8169.65	900.77	13938.43	905.55	24432.62	910.68
471.4	471.4	867.5	4166	894.28	7089	899.82	7342.08	900.05	7583.03	900.24	7790.02	900.4	8447.65	901.07	13793.43	905.89	23461.62	910.92
472.18	472.18	869.9	4166	894.54	7089	900.09	7342.08	900.32	7583.03	900.52	7790.02	900.69	8447.65	901.37	13793.43	906.24	23461.62	911.2
472.2	472.2	870.8	4166	894.54	7089	900.08	7342.08	900.32	7583.03	900.52	7790.02	900.68	8447.65	901.37	13793.43	906.22	23461.62	911.16
472.22	472.22	870.8	4166	894.54	7089	900.09	7342.08	900.33	7583.03	900.53	7790.02	900.69	8447.65	901.38	13793.43	906.25	23461.62	911.29
473	473	871.8	4166	894.79	7089	900.33	7342.08	900.57	7583.03	900.78	7790.02	900.95	8447.65	901.65	13793.43	906.52	23461.62	911.51
473.8	473.8	869	4166	895.07	7089	900.63	7342.08	900.88	7583.03	901.09	7790.02	901.27	8447.65	901.98	13793.43	906.9	23461.62	911.88
474.08	474.08	873.5	4166	895.17	7089	900.73	7342.08	900.98	7583.03	901.2	7790.02	901.38	8447.65	902.1	13793.43	907.04	23461.62	912.03
474.09	474.09	873.5	4166	895.19	7089	900.76	7342.08	901.01	7583.03	901.23	7790.02	901.41	8447.65	902.13	13793.43	907.1	23461.62	912.1
474.1			Cass Co Hwy 16/112th Ave So. (Fargo) / 110th Ave So. (Moorhead) Bridge															
474.11	474.11	874	4166	895.18	7089	900.75	7342.08	901.01	7583.03	901.22	7790.02	901.4	8447.65	902.12	13793.43	907.06	23461.62	912.13
474.12	474.12	874	4166	895.19	7089	900.76	7342.08	901.01	7583.03	901.23	7790.02	901.41	8447.65	902.13	13793.43	907.1	23461.62	912.17
474.6	474.6	873.4	4166	895.39	7089	900.93	7342.08	901.18	7583.03	901.41	7790.02	901.59	8447.65	902.31	13793.43	907.3	23461.62	912.41
475.3	475.3	872.2	4166	895.66	7089	901.22	7342.08	901.48	7583.03	901.71	7790.02	901.9	8447.65	902.63	13793.43	907.67	23461.62	912.87
475.7	475.7	871	4166	895.81	7089	901.38	7342.08	901.64	7583.03	901.88	7790.02	902.07	8447.65	902.81	13793.43	907.93	23461.62	913.26
476.4	476.4	873.7	4166	896.08	7089	901.63	7342.08	901.9	7583.03	902.14	7790.02	902.34	8447.65	903.09	13793.43	908.22	23461.62	913.51
477	477	874	4021	896.34	5364	901.86	7088.08	902.16	8748.03	902.43	10484.02	902.67	11723.65	903.44	12261.43	908.54	14253.62	913.81
477.6	477.6	870.2	4000	896.53	5137	901.99	7051.08	902.37	8930.03	902.75	10918.02	903.11	12251.65	903.93	12041.43	908.8	13076.62	913.95
478	478	873.4	4000	896.67	5137	902.08	7051.08	902.52	8930.03	902.97	10918.02	903.42	12251.65	904.27	12041.43	908.94	13076.62	914.01
478.58	478.58	875	4000	896.8	5137	902.16	7051.08	902.68	8930.03	903.2	10918.02	903.75	12251.65	904.63	12041.43	909.12	13076.62	914.11
478.72	478.72	875.38	4000	896.81	5137	902.17	7051.08	902.69	8930.03	903.23	10918.02	903.78	12251.65	904.66	12041.43	909.14	13076.62	914.11
478.76	478.76	875.51	4000	896.82	5137	902.17	7051.08	902.69	8930.03	903.22	10918.02	903.77	12251.65	904.65	12041.43	909.13	13076.62	914.1
478.765	478.765		Proposed Red River Closure Structure															
478.77	478.77	875.51	4000	896.85	5137	906.37	7051.08	908.83	8930.03	910.84	10918.02	913.38	12251.65	914.87	12041.43	916.49	13076.62	917.84
478.81	478.81	875.64	4000	896.85	5137	906.38	7051.08	908.85	8930.03	910.86	10918.02	913.42	12251.65	914.92	12041.43	916.51	13076.62	917.85
479	479	876.35	4000	896.88	7000	906.39	10500	908.86	14800	910.87	21000	913.42	25000	914.92	28500	916.51	32000	917.85
479.2	479.2	875.7	4000	896.9	7000	906.39	10500	908.86	14800	910.88	21000	913.43	25000	914.93	28500	916.52	32000	917.89
479.7	479.7	871.5	4000	897.09	7000	906.48	10500	908.99	14800	911.06	21000	913.66	25000	915.16	28500	916.72	32000	918
480.5	480.5	874.7	4000	897.37	7000	906.65	10500	909.24	14800	911.39	21000	914.05	25000	915.52	28500	917.01	32000	918.24
481.8	481.8	875.3	4000	897.78	7000	906.93	10500	909.62	14800	911.88	21000	914.65	25000	916.1	28500	917.49	32000	918.56
482.3	482.3	874.2	4000	897.99	7000	907.08	10500	909.84	14800	912.18	21000	914.98	25000	916.4	28500	917.68	32000	918.7
482.698	482.698	869.74	4000	898.11	7000	907.16	10500	909.95	14800	912.33	21000	915.16	25000	916.61	28500	917.86	32000	918.82
482.7	482.7	887.04	4000	898	7000	907.11	10500	909.89	14800	912.25	21000	915.08	25000	916.57	28500	917.85	32000	918.83
482.702	482.702	878.54	4000	898.14	7000	907.16	10500	909.96	14800	912.33	21000	915.16	25000	916.61	28500	917.86	32000	918.83
483.1	483.1	879.2	4000	898.38	7000	907.32	10500	910.19	14800	912.65	21000	915.54	25000	916.95	28500	918.07	32000	918.94
483.9	483.9	873	4000	898.79	7000	907.54	10500	910.5	14800	913.08	21000	916.08	25000	917.4	28500	918.49	32000	919.31
485.08	485.08	880.5	4000	899.31	7000	907.72	10500	910.72	14800	913.33	21000	916.37	25000	917.7	28500	918.79	32000	919.6
485.09	485.09	880.5	4000	899.31	7000	907.71	10500	910.72	14800	913.33	21000	916.37	25000	917.7	28500	918.79	32000	919.6
485.1			Cass Co. Hwy 18/52nd St. SE (Fargo) / 160th Ave So. (Moorhead) Bridge															
485.11	485.11	879.4	4000	899.31	7000	907.74	10500	910.77	14800	913.42	21000	916.43	25000	917.75	28500	918.82	32000	919.63
485.12	485.12	879.4	4000	899.33	7000	907.77	10500	910.78	14800	913.43	21000	916.43	25000	917.75	28500	918.83	32000	919.64
485.6	485.6	878.62	4000	899.51	7000	907.85	10500	910.86	14800	913.51	21000	916.52	25000	917.85	28500	918.93	32000	919.74
485.9	485.9	879.5	4000	899.68	7000	907.94	10500	910.98	14800	913.68	21000	916.7	25000	918.06	28500	919.16	32000	919.96
486.3	486.3	872.1	4000	899.84	7000	908.04	10500	911.11	14800	913.84	21000	916.92	25000	918.31	28500	919.43	32000	920.25
487	487	871.3	4000	900.06	7000	908.19	10500	911.33	14800	914.14	21000	917.34	25000	918.82	28500	920	32000	920.87
487.5	487.5	876.8	4000	900.3	7000	908.34	10500	911.52	14800	914.37	21000	917.61	25000	919.11	28500	920.22	32000	921.18
488.68	488.68	878.24	4000	901	7000	908.78	10500	912.1	14800	915.13	21000	918.48	25000	920.07	28500	921.33	32000	922.29

Table B21

Fargo Moorhead Metro Study - Phase 3.1
ND East 35Kcfs, Year 25, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
279	403.65	832.3	8950	861.67	18490	870.2	26120	873.73	34240	875.7	45780	876.86	55200	877.69	66150	878.58	82930	879.41
280	404.98	836.6	8950	862.33	18490	870.94	26120	874.31	34240	875.38	45780	877.66	55200	878.49	66150	879.38	82930	880.24
281	406.51	834.9	8950	863.21	18490	871.84	26120	875.33	34240	877.46	45780	878.67	55200	879.48	66150	880.19	82930	881.14
282	410.21	838	8950	865.13	18490	873.73	26120	877.12	34240	879.09	45780	879.9	55200	880.64	66150	881.44	82930	882.51
283	413.23	837.6	8950	866.44	18490	875.11	26120	878.49	34240	880.2	45780	881.15	55200	881.59	66150	882.44	82930	883.58
284	414.98	842.7	8950	867.22	18490	876.08	26120	879.65	34240	881.14	45780	882.07	55200	882.53	66150	883.51	82930	884.68
285	415.38	841.4	8950	867.43	18490	876.32	26120	879.9	34240	881.36	45780	882.34	55200	882.89	66150	883.77	82930	884.96
286	415.84	843	8950	867.59	18490	876.41	26120	879.96	34240	881.4	45780	882.45	55200	883.04	66150	884.03	82930	885.24
287	415.90	844.1	8950	867.64	18490	876.49	26120	880.08	34240	881.62	45780	882.6	55200	883.18	66150	884.02	82930	885.3
288			22nd St SE or Cass Co #34 / 170th Ave NW or Clay Co.36															
289	415.91	844.1	8950	867.64	18490	876.48	26120	880.07	34240	881.64	45780	882.73	55200	883.4	66150	884.51	82930	885.55
290	415.93	837.5	8950	867.67	18490	876.5	26120	880.1	34240	881.66	45780	882.82	55200	883.51	66150	884.51	82930	885.59
291	417.10	840.8	8950	868.15	18490	877.02	26120	880.73	34240	882.51	45780	883.79	55200	884.59	66150	885.3	82930	886.25
292	418.18	844.4	7874	868.82	15771	877.54	22084	881.31	28855	883.07	38631	884.43	46577	885.21	57304	885.81	73717	886.81
293	419.92	844.7	7475	869.49	13326.77	878.42	13700.4	882.02	13949.59	883.68	14340.77	885.05	14732	885.69	18820.89	886.37	27706.46	887.45
294	420.32	843.8	7475	869.77	13326.77	878.6	13700.4	882.14	13949.59	883.76	14340.77	885.11	14732	885.74	18820.89	886.43	27706.46	887.52
295	421.93	845.3	7475	871.01	13326.77	879.48	13700.4	882.58	13949.59	883.99	14340.77	885.25	14732	885.85	18820.89	886.56	27706.46	887.65
296	422.69	847.4	7475	871.34	13326.77	879.75	13700.4	882.73	13949.59	884.08	14340.77	885.3	14732	885.98	18820.89	886.6	27706.46	887.73
297	423.95	849.5	7475	871.89	13326.77	880.24	13700.4	883.08	13949.59	884.39	14340.77	885.54	14732	886.11	18820.89	886.89	27706.46	888.09
298	425.51	848.7	7475	872.64	13326.77	880.85	13700.4	883.49	13949.59	884.69	14340.77	885.76	14732	886.3	18820.89	887.13	27706.46	888.43
299	426.87	851.1	7475	873.35	13326.77	881.41	13700.4	883.88	13949.59	885.01	14340.77	886.02	14732	886.54	18820.89	887.44	27706.46	888.87
300	428.06	849.6	4864	873.9	10244.77	881.93	10433.4	884.25	10475.59	885.31	10628.77	886.26	10795	886.75	14671.33	887.71	19853.47	889.24
301	428.74	850.3	4864	874.16	10244.77	882.22	10433.4	884.46	10475.59	885.49	10628.77	886.42	10795	886.9	14671.33	887.94	19853.47	889.5
302	430.23	852	4864	874.71	10244.77	882.7	10433.4	884.81	10475.59	885.79	10628.77	886.69	10795	887.16	14671.33	888.36	19853.47	890.09
303	431.94	853.5	4864	875.45	10244.77	883.4	10433.4	885.33	10475.59	886.23	10628.77	887.08	10795	887.53	14673.93	888.9	21626.18	890.77
303.4	432.07	853.42	4572	875.5	9944.77	883.46	10133.4	885.36	10175.59	886.29	10328.77	887.13	10495	887.58	14475.24	889	21491.71	890.86
303.5	432.10	853.46	4572	875.53	9944.77	883.49	10133.4	885.41	10175.59	886.31	10328.77	887.15	10495	887.6	14646.87	889.02	21433.25	890.9
303.6	432.53	854.5	4572	875.69	9944.77	883.66	10130.4	885.53	10175.59	886.41	10328.77	887.24	10495	887.68	14695.19	889.15	21263.31	891.2
304	432.84	855.5	4572	875.77	9945.77	883.76	10134.4	885.6	10176.59	886.47	10329.77	887.29	10496	887.73	14572	889.22	20377.99	891.27
305	433.26	854.4	4572	875.93	9945.77	883.93	10134.4	885.73	10176.59	886.59	10329.77	887.4	10496	887.83	14572.06	889.37	18767.03	891.42
306	433.35	855	4572	876	9945.77	884.01	10134.4	885.8	10176.53	886.65	10329.77	887.45	10496	887.98	14572.06	889.46	18382.9	891.55
307			76th Ave. No. (Fargo) / 90th Ave NW (Moorhead) Bridge															
308	433.37	855	4572	876.01	9945.77	884.02	10134.4	885.81	10176.59	886.65	10329.77	887.46	10496	887.69	14572.06	889.47	18382.9	891.56
309	433.83	854.5	4572	876.14	9945.77	884.17	10134.4	885.93	10176.59	886.76	10329.77	887.56	10496	887.99	14572.06	889.63	18382.9	891.75
310	434.61	852.8	4572	876.39	9945.77	884.41	10134.4	886.11	10176.59	886.92	10329.77	887.7	10496	888.13	14572.06	889.83	18382.9	891.98
310.5	435.70	855	4572	876.74	9945.77	884.76	10134.4	886.36	10176.59	887.16	10329.77	887.92	10496	888.33	14572.06	890.13	18382.9	892.38
311	436.34	856	4572	876.95	9945.77	885.01	10134.4	886.58	10176.59	887.33	10329.77	888.08	10496	888.49	14500.07	890.35	19814.9	892.69
312	437.05	863.83	4372	877.18	9550.77	885.27	9744.4	886.79	9761.59	887.53	9889.77	888.26	10046	888.57	13965.07	890.64	19239.9	893.07
313	437.95	858	4372	877.51	9649.77	885.68	9743.4	887.11	9760.59	887.82	9888.77	888.53	10045	888.93	14088.26	891.03	20125.22	893.6
314	438.78	857	4372	877.84	9649.77	886.01	9743.4	887.39	9760.59	888.07	9888.77	888.77	10045	889.17	14088.26	891.36	20125.22	894.09
315	439.05	856	4372	877.93	9649.77	886.12	9743.4	887.46	9760.59	888.15	9888.77	888.85	10045	889.24	14088.26	891.46	20125.22	894.27
316	439.46	855.1	4372	878.08	9649.77	886.32	9743.4	887.65	9760.59	888.31	9888.77	889	10045	889.39	14088.26	891.7	20125.22	894.62
317			40th Ave. No. (Fargo) / Wall Street NW (Moorhead) Bridge															
318	439.47	855.1	4372	878.09	9649.77	886.33	9743.4	887.66	9760.59	888.31	9888.77	889	10045	889.39	14088.26	891.7	20125.22	894.62
319	439.50	857	4372	878.1	9649.77	886.38	9743.4	887.7	9760.59	888.36	9888.77	889.04	10045	889.43	14088.26	891.75	20125.22	894.69
320	439.66	856	4372	878.16	9649.77	886.44	9743.4	887.75	9760.59	888.4	9888.77	889.08	10045	889.47	14088.26	891.81	20125.22	894.76
321	440.31	858	4372	878.37	9649.77	886.69	9743.4	887.96	9760.59	888.6	9888.77	889.26	10045	889.65	14088.26	892.06	20125.22	895.12
322	440.32	858.1	4372	878.37	9649.77	886.68	9743.4	887.96	9760.59	888.59	9888.77	889.26	10045	889.64	14088.26	892.06	20125.22	895.11
323			Broadway Street No. (Fargo) / Broadway Street NW (Moorhead) Bridge															
324	440.33	858.1	4372	878.39	9649.77	886.8	9743.4	888.05	9760.59	888.67	9888.77	889.33	10045	889.71	14088.26	892.12	20125.22	895.16
325	440.34	857	4372	878.39	9649.77	886.81	9743.4	888.06	9760.59	888.68	9888.77	889.34	10045	889.71	14088.26	892.13	20125.22	895.17
326	440.54	858	4372	878.44	9649.77	886.86	9743.4	888.1	9760.59	888.72	9888.77	889.37	10045	889.75	14088.26	892.18	20125.22	895.25
327	441.53	859	4352	878.81	9604.77	887.14	9688.4	888.32	9700.59	888.91	9823.77	889.55	9975	889.92	14013.26	892.39	20031.6	895.51
328	442.14	858	4352	879	9604.77	887.3	9688.4	888.45	9700.59	889.02	9823.77	889.65	9975	890.01	13980.6	892.51	21152.81	895.67
329	442.45	857	4352	879.07	9604.77	887.32	9688.4	888.46	9700.59	889.04	9823.77	889.66	9975	890.02	14124.71	892.52	22451.36	895.68
329.5	442.93	858.06	4352	879.22	9603.77	887.43	9687.4	888.56	9699.59	889.12	9822.77	889.73	9974	890.09	14132.89	892.59	22549.46	895.77
330	443.69	858	4352	879.44	9603.77	887.63	9687.4	888.71	9699.59	889.26	9822.77	889.86	9974	890.21	14132.89	892.73	22549.46	895.93
331	443.99	855	4352	879.52	9603.77	887.72	9687.4	888.8	9699.59	889.34	9822.77	889.93	9974	890.29	14132.89	892.83	22549.46	896.09
332	444.33	856	4352	879.57	9603.77	887.76	9687.4	888.83	9699.59	889.37	9822.77	889.96	9974	890.31	14132.89	892.87	22549.46	896.15
333	444.92	859	4352	879.65	9603.77	887.8	9687.4	888.86	9699.59	889.4	9822.77	889.99	9974	890.34	14132.89	892.89	22549.46	896.16
334	445.18	860	4352	879.73	9603.77	887.87	9687.4	888.93	9699.59	889.46	9822.77	890.04	9974	890.39	14132.89	892.95	22549.46	896.25
335	445.31	860	4352	879.79	9603.77	887.96	9687.4	889	9699.59	889.53	9822.77	890.11	9974	890.46	14132.89	893.07	22549.46	896.47
336	445.36	859	4352	879.79	9603.77	887.97	9687.4	889.01	9699.59	889.54	9822.77	890.12	9974	890.47	14132.89	893.08	22549.46	896.49
336.5	445.76	857.69	4352	879.83	9603.77	888	9687.4	889.03	9699.59	889.56	9822.77	890.14	9974	890.48	14132.89	893.1	22549.46	896.52
337	446.09	860	4352	879.86	9603.77	888	9687.4	889.03	9699.59	889.56	9822.77	890.14	9974	890.48	14132.89	893.1	22549.46	896.51
338	446.29	861	4352	879.93	9603.77	888.03	9687.4	889.06	9699.59	889.58	9822.77	890.15	9974	890.5	14132.89	893.11	22549.46	896.52
338.5	446.70	860.9	4352	880.08	9603.77	888.17	9687.4	889.17	9699.59	889.69	9822.77	890.25	9974	890.59	14132.89	893.25	22549.46	896.7
339	446.76	861	4352	880.1	9603.77	888.19	9687.4	889.19	9699.59	889.7	9822.77	890.26	9974	890.6	14132.89	893.24	22549.46	896.72
340	447.04	860	4352	880.18	9603.77	888.28	9687.4	889.27	9699.59	889.78	9822.77	890.34	9974	890.68	14132.89	893.36	22549.46	896.94
341	447.24	859	4352	880.26	9603.77	888.34	9687.4	889.32	9699.59	889.83	9822.77	890.38	9974	890.72	14132.89	893.4	22549.46	896.98
342	447.42	861	4352	880.38	9603.77	888.43	9687.4	889.4	9699.59	889.89	9822.77	890.44	9974	890.78	14132.89	893.47	22549.46	897.07
342.5	447.76	856.99	4352	880.53	9603.77	888.57	9687.4	889.51	9699.59	890	9822.77	890.54	9974	890.88	14132.89	893.6	22549.46	897.28
343	448.13	861	4352	880.65	9603.77	888.68	9687.4	889.61	9699.59	890.08	9822.77	890.62	9974	890.95	14132.89	893.69	22549.46	897.4
344	448.36	859	4352	880.75	9603.77	888.78	9687.4	889.7	9699.59	890.17	9822.77	890.7	9974	891.03	14132.89	893.82	22549.46	897.63
345	448.80	865.3	4352	881.09	9603.77	889	9687.4	889.89	9699.59	890.34	9822.77	890.86	9974	891.18	14132.89	893.98	22549.46	897.83
348	448.90	864.3	4352	881.08	9603.77	888.98	9687.4	889.87	9699.59	890.32	9822.77	890.84	9974	891.16	14132.89	893.95	22549.46	897.78
348.4			North Dam															
348.8	448.90	864.3	4352	881.15	9603.77	889.06	9687.4	889.91	9699.59	890.36	9822.77	890.88	9974	891.2	14132.89	894	22549.46	897.85
351	448.91	861	4352	881.17	9603.77	889.05	9687.4	889.91	9699.59	890.37	9822.77	890.88	9974	891.21	14132.89	894	22549.46	897.85
351.4	449.06	861	4352	881.25	9603.77	889.18	9687.4	890.03	9699.59	890.48	9822.77	890.99	9974	891.31	14132.89	894.16	22549.46	898.09
351.5			12th Ave. No. (Fargo) / 15th Ave No. (Moorhead) Bridge															
351.6	449.07	861	4352	881.25	9603.77	889.26	9687.4	890.12	9699.59	890.56	9822.77	891.08	9974	891.4	14132.89	894.26	22549.46	898.16
352	449.09	862	4352	881.27	9603.77	889.27	9687.4	890.13	9699.59	890.58	9822.77	891.1	9974	891.42	14132.89	894.28	22549.46	898.21
353	449.28	863	4352	881.35	9603.77	889.37	9687.4	890.22	9699.59	890.66	9822.77	891.17	9974	891.49	14132.89	894.38	22549.46	898.37
353.5	449.61	860.13	4352	881.49	9603.77	889.46	9687.4	890.29	9699.59	890.73	9822.77	891.24	9974	891.55	14132.89	894.44	22549.46	898.45
354	449.97	864	4352	881.72	9603.77	889.68	9687.4	890.49	9699.59	890.91	9822.77	891.41	9974	891.72	14132.89	894.66	22549.46	898.68
355	450.19	862	4352	881.86	9603.77	889.77	9687.4	890.57	9699.59	890.99	9822.77	891.48	9974	891.79	14132.89	894.75	22549.46	898.84
356	450.84	858	4352	882.23	9603.77	890.04	9687.4	890.79	9699.59	891.19	9822.77	891.67	9974	891.97	14132.89	894.95	22549.46	899.07
357	450.86	863.4	4352	882.24	9603.77	890.04	9687.4	890.8	9699.59	891.19	9822.77	891.67	9974	891.98	14132.89	894.95	22549.46	899.07
358			BNRR Bridge															
359	450.87	863.4	4352	882.31	9603.77	890.14	9687.4	890.88	9699.59	891.27	9822.77	891.74	9974	892.04	14132.89	895	22549.46	899.21
361	451.20	865	4352	882.46	9603.77	890.26	9687.4	890.96	9699.59	891.37	9822.77	891.83	9974	892.13	14132.89	895.11	22549.46	899.36
361.5	451.37	865.2	4352	882.55	9603.77	890.33	9687.4	891.05	9699.59	891.43	9822.77	891.89	9974	892.18	14132.89	895.17	22549.46	899.43
362.5			1st Ave. No. Bridge															
364	451.39	865.2	4352	882.58	9603.77	890.35	9687.4	891.07	9699.59	891.44	9822.77	891.9	9974	892.2	14132.89	895.18	22549.46	899.47
366	451.41	861	4352	882.62	9603.77	890.4	9687.4	891.11	9699.59	891.48	9822.77	891.94	9974	892.23	14132.89	895.22	22549.46	899.5
367	451.57	867	4352	882.68	9603.77	890.49	9687.4	891.2	9699.59	891.57	9822.77	892.03	9974	892.33	14132.89	895.37	22549.46	899.79
368	451.61	866.8	4352	882.72	9603.77	890.54	9687.4	891.25	9699.59	891.61	9822.77	892.07	9974	892.36	14132.89	895.42	22549.46	899.85
369			NP Ave. (Fargo) / Center Ave. (Moorhead)															
370	451.63	866.8	4352	882.73	9603.77	890.55	9687.4	891.25	9699.59	891.62	9822.77	892.07	9974	892.37	14132.89	895.42	22549.46	899.86
371	451.65	859	4352	882.76	9603.77	890.55	9687.4	891.25	9699.59	891.62	9822.77	892.07	9974	892.37	14132.89	895.42	22549.46	899.86
372	451.67	867.9	4352	882.75	9603.77	890.54	9687.4	891.24	9699.59	891.61	9822.77	892.06	9974	892.36	14132.89	895.41	22549.46	899.86
373			BNRR Bridge															
374	451.68	867.9	4352	882.85	9603.77	890.64	9687.4	891.33	9699.59	891.69	9822.77	892.14	9974	892.44	14132.89	895.48	22549.46	899.91
375	451.69	864	4352	882.87	9603.77	890.64	9687.4	891.33	9699.59	891.69	9822.77	892.14	9974	892.43	14132.89	895.48	22549.46	899.9
376	451.71	865.9	4352	882.87	9603.77	890.65	9687.4	891.35	9699.59	891.71	9822.77	892.16	9974	892.45	14132.89	895.52	22549.46	900.01
377			Main Ave. Bridge															
378	451.72	865.9	4352	882.89	9603.77	890.66	9687.4	891.36	9699.59	891.72	9822.77	892.17	9974	892.46	14132.89	895.54	22549.46	900.04
379	451.73	864	4352	882.97	9603.77	890.76	9687.4	891.44	9699.59	891.8	9822.77	892.25	9974	892.54	14132.89	895.62	22549.46	900.13
380	451.85	864	4352	882.96	9603.77	890.73	9687.4	891.42	9699.59	891.78	9822.77	892.22	9974	892.51	14132.89	895.59	22549.46	900.1
381	452.03	863	4352	883.07	9603.77	890.88	9687.4	891.56	9699.59	891.91	9822.77	892.35	9974	892.65	14132.89	895.76	22549.46	900.3
381.5	452.04	863	4352	883.08	9603.77	890.9	9687.4	891.57	9699.59	891.92	9822.77	892.36	9974	892.66	14132.89	895.77	22549.46	900.32
384			Midtown Dam															
385.5	452.05	867	4352	883.12	9603.77	890.9	9687.4	891.58	9699.59	891.93	9822.77	892.37	9974	892.66	14132.89	895.78	22549.46	900.33
386	452.06	867	4352	883.13	9603.77	890.91	9687.4	891.58	9699.59	891.93	9822.77	892.37	9974	892.67	14132.89	895.78	22549.46	900.34
387	452.26	867	4352	883.25	9603.77	890.97	9687.4	891.64	9699.59	891.98	9822.77	892.42	9974	892.71	14132.89	895.83	22549.46	900.39
388	452.46	869	4352	883.44	9603.77	891.08	9687.4	891.73	9699.59	892.07	9822.77	892.51	9974	892.8	14132.89	895.92	22549.46	900.52
389	452.70	866.08	4352	883.69	9603.77	891.25	9687.4	891.88	9699.59	892.21	9822.77	892.64	9974	892.92	14132.89	896.07	22549.46	900.69
390	452.94	867	4352	883.8	9603.77	891.35	9687.4	891.97	9699.59	892.29	9822.77	892.71	9974	893	14132.89	896.15	22549.46	900.79
391	453.16	867	4352	883.93	9603.77	891.46	9687.4	892.07	9699.59	892.38	9822.77	892.8	9974	893.08	14132.89	896.25	22549.46	900.92
392	453.39	865	4352	884.04	9603.77	891.55	9687.4	892.15	9699.59	892.46	9822.77	892.87	9974	893.15	14132.89	896.34	22549.46	901.05
393	454.02	866	4352	884.39	9603.77	891.8	9687.4	892.37	9699.59	892.67	9822.77	893.07	9974	893.34	14132.89	896.53	22549.46	901.23
394	454.10	865	4352	884.42	9603.77	891.84	9687.4	892.41	9699.59	892.71	9822.77	893.1	9974	893.38	14132.89	896.57	22549.46	901.26
395	454.75	864	4352	884.71	9603.77	892.07	9687.4	892.62	9699.59	892.9	9822.77	893.29	9974	893.56	14132.89	896.76	22549.46	901.48
396	455.16	866	4352	884.9	9603.77	892.28	9687.4	892.81	9699.59	893.09	9822.77	893.46	9974	893.73	14132.89	896.94	22549.46	901.67
397	455.20	866.6	4352	884.94	9603.77	892.32	9687.4	892.84	9699.59	893.12	9822.77	893.49	9974	893.76	14132.89	896.97	22549.46	901.71
399			I-94 Bridge															
401	455.23	866.6	4352	884.94	9603.77	892.32	9687.4	892.84	9699.59	893.12	9822.77	893.49	9974	893.76	14132.89	896.97	22549.46	901.71
402	455.28	865	4352	884.97	9603.77	892.33	9687.4	892.86	9699.59	893.13	9822.77	893.5	9974	893.77	14132.89	896.98	22549.46	901.74
402.5	455.52	864.58	4352	885.09	9603.77	892.46	9687.4	892.97	9699.59	893.24	9822.77	893.61	9974	893.87	14132.89	897.1	22549.46	901.86
403	455.59	866	4352	885.12	9603.77	892.5	9687.4	893.01	9699.59	893.28	9822.77	893.64	9974	893.91	14132.89	897.13	22549.46	901.89
404	455.92	866	4352	885.28	9603.77	892.62	9687.4	893.12	9699.59	893.38	9822.77	893.74	9974	894.01	14132.89	897.25	22549.46	902.07
405	456.65	865	4352	885.52	9603.77	892.89	9687.4	893.36	9699.59	893.61	9822.77	893.98	9974	894.21	14132.89	897.46	22549.46	902.27
406	457.09	866	4352	885.89	9603.77	893.08	9687.4	893.54	9699.59	893.78	9822.77	894.12	9974	894.37	14132.89	897.64	22549.46	902.47

Table B21

Fargo Moorhead Metro Study - Phase 3.1
ND East 35Kcfs, Year 25, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
407	457.27	861.18	4352	885.93	9603.77	893.15	9687.4	893.6	9699.59	893.83	9822.77	894.17	9974	894.42	14132.89	897.68	22549.46	902.51
408	457.80	864	4352	886.12	9603.77	893.37	9687.4	893.81	9699.59	894.04	9822.77	894.37	9974	894.62	14132.89	897.92	22549.46	902.82
409	457.95	855.9	4352	886.17	9603.77	893.47	9687.4	893.9	9699.59	894.13	9822.77	894.46	9974	894.71	14132.89	898.02	22549.46	902.91
409.1	457.97	855.9	4352	886.18	9603.77	893.47	9687.4	893.91	9699.59	894.13	9822.77	894.46	9974	894.71	14132.89	898.02	22549.46	902.91
409.5			South Dam															
409.6	457.98	855.9	4352	886.24	9603.77	893.51	9687.4	893.94	9699.59	894.17	9822.77	894.5	9974	894.75	14132.89	898.06	22549.46	902.94
409.7	457.99	855.9	4352	886.24	9603.77	893.51	9687.4	893.94	9699.59	894.17	9822.77	894.5	9974	894.75	14132.89	898.06	22549.46	902.95
413	458.32	869	4352	886.24	9603.77	893.53	9687.4	893.97	9699.59	894.19	9822.77	894.52	9974	894.77	14132.89	898.09	22549.46	902.99
414	458.75	866	4352	886.51	9603.77	893.81	9687.4	894.22	9699.59	894.44	9822.77	894.76	9974	895	14132.89	898.37	22549.46	903.34
459.64	459.64	864.81	4352	886.91	9603.77	894.24	9687.4	894.63	9699.59	894.83	9822.77	895.13	9974	895.37	14132.89	898.72	22549.46	903.69
460.28	460.28	868.74	4352	887.29	9603.77	894.58	9687.4	894.95	9699.59	895.13	9822.77	895.43	9974	895.66	14132.89	899.02	22549.46	903.99
460.72	460.72	869.04	4352	887.61	9603.77	894.84	9687.4	895.2	9699.59	895.37	9822.77	895.68	9974	895.89	14132.89	899.27	22549.46	904.3
461.3	461.3	868.04	4352	888.01	9603.77	895.23	9687.4	895.56	9699.59	895.73	9822.77	896.01	9974	896.23	14132.89	899.64	22549.46	904.76
462.03	462.03	864.16	4352	888.37	9603.77	895.63	9687.4	895.94	9699.59	896.09	9822.77	896.36	9974	896.58	14132.89	900.01	22549.46	905.13
462.04	462.04	869.5	4352	888.41	9603.77	895.66	9687.4	895.96	9699.59	896.13	9822.77	896.39	9974	896.61	14132.89	900.03	22549.46	905.18
462.05			52nd Avenue So. (Fargo) / 60th Ave So. (Moorhead) Bridge															
462.06	462.06	869.5	4352	888.42	9603.77	895.67	9687.4	895.96	9699.59	896.14	9822.77	896.4	9974	896.62	14132.89	900.05	22549.46	905.25
462.07	462.07	871.5	4352	888.46	9603.77	895.7	9687.4	896.01	9699.59	896.16	9822.77	896.43	9974	896.64	14132.89	900.08	22549.46	905.24
462.7	462.7	862.82	4352	888.82	9603.77	896.03	9687.4	896.32	9699.59	896.46	9822.77	896.72	9974	896.93	14132.89	900.37	22549.46	905.54
463.7	463.7	864.71	4352	889.15	9603.77	896.43	9687.4	896.72	9699.59	896.85	9822.77	897.1	9974	897.31	14132.89	900.8	22549.46	905.98
464.2	464.2	869.2	4352	889.41	9603.77	896.71	9687.4	896.98	9699.59	897.11	9822.77	897.36	9974	897.57	14132.89	901.1	22549.46	906.33
464.6	464.6	865	4352	889.62	9603.77	896.9	9687.4	897.17	9699.59	897.3	9822.77	897.54	9974	897.74	14132.89	901.28	22549.46	906.49
465.1	465.1	862.9	4352	889.79	9603.77	897.14	9687.4	897.4	9699.59	897.52	9822.77	897.76	9974	897.97	14132.89	901.56	22549.46	906.88
465.9	465.9	865.6	4352	890.05	9603.77	897.49	9687.4	897.74	9699.59	897.86	9822.77	898.09	9974	898.29	14132.89	901.93	22549.46	907.24
466.5	466.5	864.9	4352	890.3	9603.77	897.76	9687.4	898	9699.59	898.12	9822.77	898.34	9974	898.55	14132.89	902.21	22549.46	907.55
467.4	467.4	864.8	4352	890.54	9603.77	898.17	9687.4	898.41	9699.59	898.51	9822.77	898.74	9974	898.94	14132.89	902.64	22549.46	908.03
468.2	468.2	862.6	4352	890.89	9603.77	898.45	9687.4	898.66	9699.59	898.78	9822.77	899	9974	899.21	14132.89	902.93	22549.46	908.3
468.9	468.9	861	4352	891.12	9603.77	898.73	9687.4	898.96	9699.59	899.05	9822.77	899.27	9974	899.48	14132.89	903.23	22549.46	908.51
469.7	469.7	870	4352	891.47	9603.77	899.13	9687.4	899.34	9699.59	899.44	9822.77	899.65	9974	899.86	14132.89	903.57	22549.46	908.7
470.1	470.1	870.9	4352	891.63	9603.77	899.31	9687.4	899.52	9699.59	899.62	9822.77	899.82	9974	900.03	14132.89	903.76	22549.46	908.85
470.6	470.6	869.4	3198	891.8	7253.77	899.5	7336.4	899.71	7346.59	899.8	7471.77	900.01	7622	900.21	11780.89	903.95	20197.46	909.06
471.4	471.4	867.5	3198	892.04	7108.77	899.79	7323.4	899.99	7449.59	900.09	7691.77	900.29	7892	900.5	11666.89	904.28	19314.46	909.37
472.18	472.18	869.9	3198	892.29	7108.77	900.06	7323.4	900.27	7449.59	900.37	7691.77	900.58	7892	900.79	11666.89	904.61	19314.46	909.68
472.2	472.2	870.8	3198	892.29	7108.77	900.05	7323.4	900.26	7449.59	900.36	7691.77	900.57	7892	900.79	11666.89	904.6	19314.46	909.66
472.22	472.22	870.8	3198	892.29	7108.77	900.06	7323.4	900.27	7449.59	900.37	7691.77	900.59	7892	900.8	11666.89	904.63	19314.46	909.73
473	473	871.8	3198	892.53	7108.77	900.31	7323.4	900.52	7449.59	900.62	7691.77	900.84	7892	901.06	11666.89	904.89	19314.46	909.97
473.8	473.8	869	3198	892.79	7108.77	900.6	7323.4	900.83	7449.59	900.93	7691.77	901.16	7892	901.38	11666.89	905.26	19314.46	910.34
474.08	474.08	873.5	3198	892.89	7108.77	900.71	7323.4	900.93	7449.59	901.04	7691.77	901.27	7892	901.49	11666.89	905.4	19314.46	910.48
474.09	474.09	873.5	3198	892.9	7108.77	900.73	7323.4	900.96	7449.59	901.07	7691.77	901.3	7892	901.52	11666.89	905.44	19314.46	910.54
474.1			Cass Co Hwy 16/112th Ave So. (Fargo) / 110th Ave So. (Moorhead) Bridge															
474.11	474.11	874	3198	892.9	7108.77	900.73	7323.4	900.95	7449.59	901.06	7691.77	901.3	7892	901.52	11666.89	905.43	19314.46	910.52
474.12	474.12	874	3198	892.91	7108.77	900.74	7323.4	900.96	7449.59	901.07	7691.77	901.3	7892	901.52	11666.89	905.44	19314.46	910.55
474.6	474.6	873.4	3198	893.12	7108.77	900.91	7323.4	901.13	7449.59	901.25	7691.77	901.48	7892	901.7	11666.89	905.63	19314.46	910.78
475.3	475.3	872.2	3198	893.38	7108.77	901.2	7323.4	901.43	7449.59	901.54	7691.77	901.79	7892	902.01	11666.89	905.97	19314.46	911.2
475.7	475.7	871	3198	893.51	7108.77	901.36	7323.4	901.59	7449.59	901.71	7691.77	901.96	7892	902.19	11666.89	906.19	19314.46	911.53
476.4	476.4	873.7	3198	893.78	7108.77	901.62	7323.4	901.85	7449.59	901.97	7691.77	902.22	7892	902.45	11666.89	906.47	19314.46	911.8
477	477	874	3147	894.05	5685.77	901.85	7178.4	902.11	8606.59	902.26	10299.77	902.55	11079	902.8	10456.89	906.77	10915.46	912.11
477.6	477.6	870.2	3139	894.23	5496.77	902	7156.4	902.34	8790.59	902.58	10720.77	902.99	11594	903.28	10281.89	907.02	9838.47	912.24
478	478	873.4	3139	894.37	5496.77	902.1	7156.4	902.49	8790.59	902.8	10720.77	903.29	11594	903.62	10281.89	907.16	9838.47	912.3
478.58	478.58	875	3139	894.49	5496.77	902.2	7156.4	902.65	8790.59	903.03	10720.77	903.61	11594	903.98	10281.89	907.33	9838.47	912.37
478.72	478.72	875.38	3139	894.5	5496.77	902.21	7156.4	902.67	8790.59	903.05	10720.77	903.64	11594	904.01	10281.89	907.34	9838.47	912.38
478.76	478.76	875.51	3139	894.51	5496.77	902.21	7156.4	902.66	8790.59	903.05	10720.77	903.63	11594	904	10281.89	907.33	9838.47	912.37
478.765	478.765		Proposed Red River Closure Structure															
478.77	478.77	875.51	3139	894.53	5496.77	905.18	7156.4	908.04	8790.59	910.23	10720.77	912.8	11594	914.51	10281.89	916.07	9838.47	918.01
478.81	478.81	875.64	3139	894.54	5496.77	905.19	7156.4	908.06	8790.59	910.26	10720.77	912.83	11594	914.55	10281.89	916.08	9838.47	918.01
479	479	876.35	3139	894.57	6160	905.2	9555	908.07	13729	910.26	19709	912.84	23757	914.55	26164	916.08	30016	918.01
479.2	479.2	875.7	3139	894.6	6160	905.21	9555	908.07	13729	910.27	19709	912.85	23757	914.56	26164	916.09	30016	918.04
479.7	479.7	871.5	3139	894.79	6160	905.29	9555	908.2	13729	910.45	19709	913.08	23757	914.79	26164	916.28	30016	918.13
480.5	480.5	874.7	3139	895.08	6160	905.45	9555	908.43	13729	910.77	19709	913.47	23757	915.15	26164	916.56	30016	918.28
481.8	481.8	875.3	3139	895.47	6160	905.7	9555	908.8	13729	911.25	19709	914.04	23757	915.75	26164	917.02	30016	918.56
482.3	482.3	874.2	3139	895.66	6160	905.84	9555	909.01	13729	911.54	19709	914.37	23757	916.08	26164	917.22	30016	918.68
482.698	482.698	869.74	3139	895.79	6160	905.92	9555	909.12	13729	911.69	19709	914.55	23757	916.3	26164	917.4	30016	918.79
482.7	482.7	887.04	3139	895.67	6160	905.87	9555	909.06	13729	911.61	19709	914.48	23757	916.24	26164	917.38	30016	918.79
482.702	482.702	879.54	3139	895.83	6160	905.92	9555	909.12	13729	911.69	19709	914.55	23757	916.3	26164	917.4	30016	918.79
483.1	483.1	879.2	3139	896.07	6160	906.07	9555	909.34	13729	911.99	19709	914.9	23757	916.66	26164	917.63	30016	918.9
483.9	483.9	873	3139	896.51	6160	906.29	9555	909.64	13729	912.4	19709	915.43	23757	917.1	26164	918.04	30016	919.23
485.08	485.08	880.5	3139	897.11	6160	906.47	9555	909.85	13729	912.65	19709	915.72	23757	917.4	26164	918.33	30016	919.49
485.09	485.09	880.5	3139	897.11	6160	906.46	9555	909.85	13729	912.65	19709	915.72	23757	917.4	26164	918.33	30016	919.49
485.1			Cass Co. Hwy 18/52nd St. SE (Fargo) / 160th Ave So. (Moorhead) Bridge															
485.11	485.11	879.4	3139	897.12	6160	906.48	9555	909.95	13729	912.76	19709	915.78	23757	917.45	26164	918.37	30016	919.52
485.12	485.12	879.4	3139	897.13	6160	906.5	9555	909.96	13729	912.76	19709	915.78	23757	917.45	26164	918.38	30016	919.53
485.6	485.6	878.62	3139	897.34	6160	906.58	9555	910.04	13729	912.85	19709	915.87	23757	917.54	26164	918.47	30016	919.62
485.9	485.9	879.5	3139	897.53	6160	906.67	9555	910.16	13729	912.99	19709	916.05	23757	917.74	26164	918.68	30016	919.82
486.3	486.3	872.1	3139	897.7	6160	906.77	9555	910.28	13729	913.16	19709	916.26	23757	917.98	26164	918.92	30016	920.07
487	487	871.3	3139	897.92	6160	906.92	9555	910.48	13729	913.44	19709	916.66	23757	918.46	26164	919.44	30016	920.64
487.5	487.5	878.8	3139	898.14	6160	907.06	9555	910.67	13729	913.67	19709	916.92	23757	918.74	26164	919.73	30016	920.93
488.68	488.68	878.24	3139	898.79	6160	907.48	9555	911.22	13729	914.4	19709	917.79	23757	919.65	26164	920.68	30016	921.95

Table B22

Fargo Moorhead Metro Study - Phase 3.1
ND East 35Kcfs, Year 50, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
279	403.65	832.3	7890	860.38	17020	869.46	24610	873.29	32650	875.42	44080	876.69	53560	877.55	63590	878.37	80610	879.31
280	404.98	836.6	7890	861.02	17020	870.16	24610	874.14	32650	876.09	44080	877.53	53560	878.34	63590	879.15	80610	880.13
281	406.51	834.9	7890	861.88	17020	871.03	24610	875.07	32650	877.15	44080	878.68	53560	879.33	63590	879.96	80610	881.01
282	410.21	838	7890	863.78	17020	872.89	24610	876.78	32650	878.79	44080	879.82	53560	880.48	63590	881.22	80610	882.37
283	413.23	837.6	7890	865.06	17020	874.26	24610	878.09	32650	880.03	44080	881.03	53560	881.43	63590	882.22	80610	883.43
284	414.98	842.7	7890	865.83	17020	875.17	24610	879.2	32650	880.94	44080	881.92	53560	882.48	63590	883.29	80610	884.52
285	415.38	841.4	7890	866.04	17020	875.41	24610	879.46	32650	881.16	44080	882.19	53560	882.73	63590	883.56	80610	884.79
286	415.84	843	7890	866.19	17020	875.49	24610	879.52	32650	881.19	44080	882.29	53560	882.89	63590	883.72	80610	885.08
287	415.90	844.1	7890	866.24	17020	875.57	24610	879.63	32650	881.45	44080	882.45	53560	883.04	63590	883.84	80610	885.14
288			22nd St SE or Cass Co. #34 / 170th Ave NW or Clay Co.36															
289	415.91	844.1	7890	866.24	17020	875.56	24610	879.62	32650	881.44	44080	882.56	53560	883.24	63590	884.31	80610	885.39
290	415.93	837.5	7890	866.27	17020	875.58	24610	879.64	32650	881.45	44080	882.66	53560	883.35	63590	884.32	80610	885.43
291	417.10	840.8	7890	866.76	17020	876.09	24610	880.25	32650	882.29	44080	883.62	53560	884.43	63590	885.11	80610	886.1
292	418.18	844.4	6987	867.23	14607	876.59	20926	880.83	27646	882.85	37342	884.25	45370	885.05	55134	885.62	71749	886.66
293	419.92	844.7	6597	868.1	12912	877.47	13607.03	881.58	13865	883.46	14176	884.87	14896.01	885.53	16775.5	886.16	25146.25	887.3
294	420.32	843.8	6597	868.38	12912	877.67	13607.03	881.7	13865	883.54	14176	884.93	14896.01	885.59	16775.5	886.21	25146.25	887.36
295	421.93	845.3	6597	869.65	12912	878.63	13607.03	882.21	13865	883.8	14176	885.08	14896.01	885.71	16775.5	886.32	25146.25	887.47
296	422.69	847.4	6597	869.98	12912	878.92	13607.03	882.38	13865	883.9	14176	885.13	14896.01	885.75	16775.5	886.36	25146.25	887.54
297	423.95	849.5	6597	870.52	12912	879.45	13607.03	882.73	13865	884.2	14176	885.38	14896.01	885.99	16775.5	886.61	25146.25	887.87
298	425.51	848.7	6597	871.28	12912	880.1	13607.03	883.17	13865	884.52	14176	885.61	14896.01	886.19	16775.5	886.82	25146.25	888.17
299	426.87	851.1	6597	872	12912	880.69	13607.03	883.57	13865	884.84	14176	885.87	14896.01	886.44	16775.5	887.09	25146.25	888.57
300	428.06	849.6	4007	872.54	9792	881.22	10311.03	883.96	10367	885.15	10458	886.11	11136	886.67	12992.5	887.33	18573.3	888.9
301	428.74	850.3	4007	872.77	9792	881.52	10311.03	884.17	10367	885.33	10458	886.27	11136	886.83	12992.5	887.52	18573.3	889.18
302	430.23	852	4007	873.28	9792	882.02	10311.03	884.63	10367	885.63	10458	886.54	11136	887.11	12992.5	887.88	18573.3	889.74
303	431.94	853.5	4007	873.98	9792	882.73	10311.03	885.06	10367	886.05	10458	886.93	11136	887.5	12992.5	888.35	19506.04	890.37
303.4	432.07	853.42	3721	874.02	9492	882.79	10011.03	885.12	10067	886.13	10158	886.98	10836	887.56	12692.5	888.42	19218.64	890.46
303.5	432.10	853.46	3721	874.05	9492	882.82	10011.03	885.14	10067	886.15	10158	887	10836	887.58	12692.5	888.44	19239.57	890.5
303.8	432.53	854.5	3721	874.19	9492	883	10011.03	885.26	10067	886.25	10158	887.09	10836	887.67	12692.5	888.55	19292.8	890.72
304	432.84	855.5	3721	874.26	9493	883.1	10012.03	885.34	10068	886.31	10159	887.14	10837	887.72	12645	888.61	18784.4	890.79
305	433.26	854.4	3721	874.41	9493	883.27	10012.03	885.48	10068	886.43	10159	887.24	10837	887.83	12645	888.73	18311.74	890.94
306	433.36	855	3721	874.47	9493	883.35	10012.03	885.54	10068	886.49	10159	887.3	10837	887.89	12645	888.8	18141.81	891.06
307			76th Ave. No. (Fargo) / 90th Ave NW (Moorhead) Bridge															
308	433.37	855	3721	874.46	9493	883.36	10012.03	885.55	10068	886.5	10159	887.3	10837	887.89	12645	888.81	18141.81	891.07
309	433.83	854.5	3721	874.61	9493	883.52	10012.03	885.67	10068	886.61	10159	887.41	10837	888	12645	888.94	18141.81	891.27
310	434.61	852.8	3721	874.83	9493	883.75	10012.03	885.85	10068	886.77	10159	887.54	10837	888.14	12645	889.11	18141.81	891.51
310.5	435.70	855	3721	875.15	9493	884.12	10012.03	886.13	10068	887.01	10159	887.76	10837	888.36	12645	889.36	18141.81	891.93
311	436.34	855	3721	875.36	9493	884.37	10012.03	886.33	10068	887.18	10159	887.92	10834.26	888.53	12613.67	889.57	18192.53	892.25
312	437.05	863.83	3526	875.56	9163	884.63	9607.03	886.55	9648	887.39	9709	888.1	10369.26	888.72	12033.67	889.79	17567.53	892.6
313	437.95	856	3526	875.86	9162	885.03	9606.03	886.87	9647	887.66	9708	888.36	10368.26	888.99	12081.18	890.1	18075.93	893.09
314	438.78	857	3526	876.17	9162	885.38	9606.03	887.16	9647	887.92	9708	888.6	10368.26	889.24	12081.18	890.39	18075.93	893.5
315	439.05	856	3526	876.25	9162	885.48	9606.03	887.24	9647	889	9708	888.68	10368.26	889.32	12081.18	890.47	18075.93	893.64
316	439.46	855.1	3526	876.39	9162	885.68	9606.03	887.42	9647	888.16	9708	888.82	10368.26	889.48	12081.18	890.66	18075.93	893.95
317			40th Ave. No. (Fargo) / Wall Street NW (Moorhead) Bridge															
318	439.47	855.1	3526	876.4	9162	885.69	9606.03	887.42	9647	888.15	9708	888.83	10368.26	889.48	12081.18	890.66	18075.93	893.95
319	439.50	857	3526	876.41	9162	885.73	9606.03	887.47	9647	888.2	9708	888.87	10368.26	889.52	12081.18	890.71	18075.93	894.01
320	439.66	856	3526	876.46	9162	885.8	9606.03	887.52	9647	888.25	9708	888.91	10368.26	889.56	12081.18	890.76	18075.93	894.07
321	440.31	858	3526	876.66	9162	886.05	9606.03	887.73	9647	888.44	9708	889.09	10368.26	889.75	12081.18	890.97	18075.93	894.39
322	440.32	858.1	3526	876.66	9162	886.04	9606.03	887.73	9647	888.44	9708	889.08	10368.26	889.75	12081.18	890.97	18075.93	894.38
323			Broadway Street No. (Fargo) / Broadway Street NW (Moorhead) Bridge															
324	440.33	858.1	3526	876.66	9162	886.16	9606.03	887.82	9647	888.52	9708	889.15	10368.26	889.81	12081.18	891.03	18075.93	894.43
325	440.34	857	3526	876.67	9162	886.17	9606.03	887.83	9647	888.53	9708	889.16	10368.26	889.82	12081.18	891.04	18075.93	894.44
326	440.54	858	3526	876.71	9162	886.22	9606.03	887.87	9647	888.58	9708	889.19	10368.26	889.85	12081.18	891.06	18075.93	894.51
327	441.53	859	3506	877.06	9137	886.51	9546.03	888.09	9582	888.76	9638	889.37	10293.26	890.03	11991.18	891.27	17965.93	894.74
328	442.14	858	3506	877.26	9137	886.67	9546.03	888.22	9582	888.87	9638	889.47	10293.26	890.13	11991.18	891.36	18721.09	894.88
329	442.45	857	3506	877.34	9137	886.7	9546.03	888.24	9582	888.89	9638	889.48	10293.26	890.14	11991.18	891.39	19608.34	894.09
329.5	442.93	858.06	3506	877.48	9136	886.82	9545.03	888.33	9581	888.97	9637	889.55	10295.01	890.21	12021.5	891.46	19698.25	894.98
330	443.69	858	3506	877.71	9136	887.02	9545.03	888.49	9581	889.12	9637	889.69	10295.01	890.34	12021.5	891.59	19698.25	895.13
331	443.99	859	3506	877.78	9136	887.11	9545.03	888.58	9581	889.19	9637	889.76	10295.01	890.41	12021.5	891.68	19698.25	895.28
332	444.33	858	3506	877.84	9136	887.15	9545.03	888.61	9581	889.22	9637	889.79	10295.01	890.44	12021.5	891.71	19698.25	895.32
333	444.92	859	3506	877.93	9136	887.19	9545.03	888.64	9581	889.25	9637	889.81	10295.01	890.47	12021.5	891.73	19698.25	895.34
334	445.18	860	3506	878	9136	887.27	9545.03	888.71	9581	889.31	9637	889.87	10295.01	890.52	12021.5	891.79	19698.25	895.42
335	445.31	860	3506	878.05	9136	887.35	9545.03	888.78	9581	889.38	9637	889.93	10295.01	890.59	12021.5	891.86	19698.25	895.6
336	445.36	859	3506	878.06	9136	887.36	9545.03	888.79	9581	889.39	9637	889.94	10295.01	890.6	12021.5	891.89	19698.25	895.62
336.5	445.76	857.69	3506	878.1	9136	887.39	9545.03	888.81	9581	889.41	9637	889.96	10295.01	890.62	12021.5	891.91	19698.25	895.65
337	446.09	860	3506	878.15	9136	887.39	9545.03	888.81	9581	889.41	9637	889.96	10295.01	890.62	12021.5	891.9	19698.25	895.64
338	446.29	861	3506	878.22	9136	887.43	9545.03	888.84	9581	889.43	9637	889.96	10295.01	890.63	12021.5	891.92	19698.25	895.65
338.5	446.70	860.9	3506	878.36	9136	887.57	9545.03	888.96	9581	889.54	9637	890.07	10295.01	890.73	12021.5	892.03	19698.25	895.81
339	446.76	861	3506	878.38	9136	887.59	9545.03	888.97	9581	889.55	9637	890.09	10295.01	890.75	12021.5	892.04	19698.25	895.82
340	447.04	860	3506	878.46	9136	887.68	9545.03	889.05	9581	889.63	9637	890.18	10295.01	890.83	12021.5	892.13	19698.25	896.01
341	447.24	859	3506	878.54	9136	887.75	9545.03	889.11	9581	889.68	9637	890.2	10295.01	890.87	12021.5	892.17	19698.25	896.04
342	447.42	861	3506	878.66	9136	887.84	9545.03	889.18	9581	889.74	9637	890.26	10295.01	890.92	12021.5	892.24	19698.25	896.13
342.5	447.78	856.99	3506	878.81	9136	887.98	9545.03	889.3	9581	889.85	9637	890.36	10295.01	891.03	12021.5	892.35	19698.25	896.31
343	448.13	861	3506	878.93	9136	888.08	9545.03	889.39	9581	889.94	9637	890.44	10295.01	891.1	12021.5	892.43	19698.25	896.42
344	448.36	859	3506	879.03	9136	888.18	9545.03	889.48	9581	890.02	9637	890.52	10295.01	891.19	12021.5	892.54	19698.25	896.62
345	448.89	865.3	3506	879.41	9136	888.42	9545.03	889.68	9581	890.19	9637	890.68	10295.01	891.34	12021.5	892.69	19698.25	896.81
348	448.90	864.3	3506	879.41	9136	888.4	9545.03	889.66	9581	890.17	9637	890.66	10295.01	891.32	12021.5	892.67	19698.25	896.76
348.4			North Dam															
348.8	448.90	864.3	3506	879.5	9136	888.45	9545.03	889.7	9581	890.22	9637	890.7	10295.01	891.38	12021.5	892.71	19698.25	896.83
351	448.91	861	3506	879.51	9136	888.45	9545.03	889.71	9581	890.22	9637	890.7	10295.01	891.38	12021.5	892.71	19698.25	896.83
351.4	449.06	861	3506	879.58	9136	888.58	9545.03	889.82	9581	890.33	9637	890.81	10295.01	891.49	12021.5	892.85	19698.25	897.05
351.5			12th Ave. No. (Fargo) / 15th Ave No. (Moorhead) Bridge															
351.6	449.07	861	3506	879.59	9136	888.64	9545.03	889.9	9581	890.42	9637	890.9	10295.01	891.59	12021.5	892.95	19698.25	897.13
352	449.09	862	3506	879.6	9136	888.66	9545.03	889.92	9581	890.43	9637	890.91	10295.01	891.6	12021.5	892.97	19698.25	897.17
353	449.28	863	3506	879.68	9136	888.75	9545.03	890	9581	890.51	9637	890.99	10295.01	891.68	12021.5	893.06	19698.25	897.31
353.5	449.61	860.13	3506	879.81	9136	888.84	9545.03	890.08	9581	890.58	9637	891.05	10295.01	891.74	12021.5	893.11	19698.25	897.38
354	449.97	864	3506	880.05	9136	889.07	9545.03	890.28	9581	890.78	9637	891.22	10295.01	891.92	12021.5	893.31	19698.25	897.6
355	450.19	862	3506	880.22	9136	889.16	9545.03	890.36	9581	890.84	9637	891.29	10295.01	891.99	12021.5	893.4	19698.25	897.75
356	450.84	858	3506	880.6	9136	889.44	9545.03	890.59	9581	891.04	9637	891.48	10295.01	892.18	12021.5	893.56	19698.25	897.97
357	450.86	863.4	3506	880.6	9136	889.44	9545.03	890.59	9581	891.05	9637	891.48	10295.01	892.18	12021.5	893.56	19698.25	897.97
358			BNRR Bridge															
359	450.87	863.4	3506	880.67	9136	889.54	9545.03	890.68	9581	891.12	9637	891.56	10295.01	892.24	12021.5	893.64	19698.25	898.08
361	451.20	865	3506	880.82	9136	889.66	9545.03	890.78	9581	891.22	9637	891.64	10295.01	892.34	12021.5	893.74	19698.25	898.21
361.5	451.37	865.2	3506	880.93	9136	889.73	9545.03	890.85	9581	891.28	9637	891.7	10295.01	892.39	12021.5	893.8	19698.25	898.28
362.5			1st Ave. No. Bridge															
364	451.39	865.2	3506	880.95	9136	889.76	9545.03	890.87	9581	891.3	9637	891.71	10295.01	892.41	12021.5	893.81	19698.25	898.3
366	451.41	861	3506	880.99	9136	889.8	9545.03	890.9	9581	891.34	9637	891.75	10295.01	892.44	12021.5	893.84	19698.25	898.34
367	451.57	867	3506	881.05	9136	889.89	9545.03	891	9581	891.43	9637	891.84	10295.01	892.54	12021.5	893.97	19698.25	898.59
368	451.61	866.8	3506	881.09	9136	889.94	9545.03	891.04	9581	891.47	9637	891.88	10295.01	892.58	12021.5	894.01	19698.25	898.64
369			NP Ave. (Fargo) / Center Ave. (Moorhead)															
370	451.63	866.8	3506	881.1	9136	889.95	9545.03	891.05	9581	891.47	9637	891.88	10295.01	892.58	12021.5	894.01	19698.25	898.67
371	451.65	859	3506	881.13	9136	889.95	9545.03	891.05	9581	891.47	9637	891.88	10295.01	892.58	12021.5	894.01	19698.25	898.66
372	451.67	867.9	3506	881.12	9136	889.94	9545.03	891.04	9581	891.45	9637	891.87	10295.01	892.57	12021.5	894	19698.25	898.65
373			BNRR Bridge															
374	451.68	867.9	3506	881.21	9136	890.04	9545.03	891.13	9581	891.55	9637	891.95	10295.01	892.65	12021.5	894.08	19698.25	898.71
375	451.69	864	3506	881.23	9136	890.04	9545.03	891.13	9581	891.55	9637	891.95	10295.01	892.65	12021.5	894.08	19698.25	898.7
376	451.71	865.9	3506	881.23	9136	890.05	9545.03	891.14	9581	891.56	9637	891.97	10295.01	892.67	12021.5	894.11	19698.25	898.79
377			Main Ave. Bridge															
378	451.72	865.9	3506	881.26	9136	890.06	9545.03	891.15	9581	891.57	9637	891.97	10295.01	892.68	12021.5	894.12	19698.25	898.82
379	451.73	864	3506	881.33	9136	890.16	9545.03	891.24	9581	891.65	9637	892.05	10295.01	892.76	12021.5	894.2	19698.25	898.91
380	451.85	864	3506	881.32	9136	890.14	9545.03	891.22	9581	891.63	9637	892.03	10295.01	892.73	12021.5	894.17	19698.25	898.88
381	452.03	863	3506	881.42	9136	890.28	9545.03	891.35	9581	891.76	9637	892.16	10295.01	892.87	12021.5	894.32	19698.25	899.07
381.5	452.04	863	3506	881.43	9136	890.3	9545.03	891.37	9581	891.77	9637	892.17	10295.01	892.88	12021.5	894.33	19698.25	899.09
384			Midtown Dam															
385.5	452.05	867	3506	881.5	9136	890.3	9545.03	891.37	9581	891.78	9637	892.17	10295.01	892.89	12021.5	894.34	19698.25	899.1
386	452.06	867	3506	881.51	9136	890.31	9545.03	891.38	9581	891.78	9637	892.18	10295.01	892.89	12021.5	894.34	19698.25	899.1
387	452.25	867	3506	881.65	9136	890.38	9545.03	891.44	9581	891.84	9637	892.23	10295.01	892.94	12021.5	894.39	19698.25	899.16
388	452.46	869	3506	881.87	9136	890.49	9545.03	891.53	9581	891.93	9637	892.31	10295.01	893.02	12021.5	894.48	19698.25	899.27
389	452.70	866.08	3506	882.13	9136	890.66	9545.03	891.69	9581	892.07	9637	892.44	10295.01	893.15	12021.5	894.62	19698.25	899.44
390	452.94	867	3506	882.26	9136	890.76	9545.03	891.77	9581	892.15	9637	892.52	10295.01	893.22	12021.5	894.69	19698.25	899.53
391	453.16	867	3506	882.38	9136	890.87	9545.03	891.87	9581	892.24	9637	892.6	10295.01	893.31	12021.5	894.78	19698.25	899.65
392	453.39	865	3506	882.5	9136	890.96	9545.03	891.95	9581	892.32	9637	892.68	10295.01	893.39	12021.5	894.86	19698.25	899.77
393	454.02	866	3506	882.86	9136	891.23	9545.03	892.18	9581	892.53	9637	892.87	10295.01	893.58	12021.5	895.05	19698.25	899.95
394	454.10	865	3506	882.89	9136	891.27	9545.03	892.22	9581	892.56	9637	892.91	10295.01	893.61	12021.5	895.09	19698.25	899.99
395	454.75	864	3506	883.17	9136	891.5	9545.03	892.43	9581	892.76	9637	893.09	10295.01	893.79	12021.5	895.27	19698.25	900.21
396	455.16	866	3506	883.35	9136	891.71	9545.03	892.62	9581	892.94	9637	893.27	10295.01	893.97	12021.5	895.45	19698.25	900.4
397	455.20	865.6	3506	883.39	9136	891.75	9545.03	892.65	9581	892.98	9637	893.3	10295.01	894	12021.5	895.48	19698.25	900.43
399			I-94 Bridge															
401	455.23	865.6	3506	883.4	9136	891.75	9545.03	892.65	9581	892.98	9637	893.3	10295.01	894	12021.5	895.48	19698.25	900.43
402	455.28	865	3506	883.43	9136	891.77	9545.03	892.67	9581	892.99	9637	893.31	10295.01	894.01	12021.5	895.49	19698.25	900.46
402.5	455.52	864.58	3506	883.54	9136	891.89	9545.03	892.79	9581	893.1	9637	893.42	10295.01	894.12	12021.5	895.6	19698.25	900.57
403	455.59	866	3506	883.57	9136	891.93	9545.03	892.82	9581	893.13	9637	893.45	10295.01	894.15	12021.5	895.63	19698.25	900.6
404	455.92	865	3506	883.72	9136	892.05	9545.03	892.94	9581	893.24	9637	893.55	10295.01	894.26	12021.5	895.74	19698.25	900.77
405	456.65	865	3506	884.1	9136	892.33	9545.03	893.18	9581	893.47	9637	893.77	10295.01	894.47	12021.5	895.95	19698.25	900.97
406	457.09	866	3506	884.35	9136	892.53	9545.03	893.36	9581	893.64	9637	893.93	10295.01	894.63	12021.5	896.11	19698.25	901.17

Table B22

Fargo Moorhead Metro Study - Phase 3.1
ND East 35Kcfs, Year 50, Water Surface Profiles
ALL WATER SURFACE ELEVATIONS IN NAVD 1988

Exceedance			50%		20%		10%		5%		2%		1%		0.5%		0.2%	
Recurrence Interval			2-Year		5-Year		10-Year		20-Year		50-Year		100-Year		200-Year		500-Year	
River Sta	River Mile	Invert Stage	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)	Q Total (cfs)	W.S. Elev (ft)
407	457.27	861.18	3506	884.42	9136	892.59	9545.03	893.42	9581	893.7	9637	893.98	10295.01	894.68	12021.5	896.16	19698.25	901.21
408	457.80	864	3506	884.59	9136	892.82	9545.03	893.63	9581	893.9	9637	894.18	10295.01	894.88	12021.5	896.38	19698.25	901.49
409	457.96	855.9	3506	884.64	9136	892.91	9545.03	893.73	9581	893.99	9637	894.27	10295.01	894.97	12021.5	896.47	19698.25	901.59
409.1	457.97	855.9	3506	884.64	9136	892.91	9545.03	893.73	9581	893.99	9637	894.27	10295.01	894.97	12021.5	896.47	19698.25	901.6
409.5			South Dam															
409.6	457.98	855.9	3506	884.72	9136	892.96	9545.03	893.77	9581	894.03	9637	894.31	10295.01	895.01	12021.5	896.51	19698.25	901.63
409.7	457.99	855.9	3506	884.72	9136	892.96	9545.03	893.77	9581	894.03	9637	894.31	10295.01	895.01	12021.5	896.51	19698.25	901.63
413	458.32	869	3506	884.72	9136	892.98	9545.03	893.79	9581	894.05	9637	894.33	10295.01	895.03	12021.5	896.53	19698.25	901.67
414	458.75	866	3506	884.98	9136	893.25	9545.03	894.04	9581	894.3	9637	894.57	10295.01	895.27	12021.5	896.79	19698.25	902
459.64	459.64	864.81	3506	885.36	9136	893.69	9545.03	894.45	9581	894.69	9637	894.94	10295.01	895.64	12021.5	897.15	19698.25	902.35
460.28	460.28	868.74	3506	885.74	9136	894.03	9545.03	894.77	9581	895	9637	895.23	10295.01	895.93	12021.5	897.44	19698.25	902.65
460.72	460.72	869.04	3506	886.07	9136	894.3	9545.03	895.02	9581	895.24	9637	895.47	10295.01	896.16	12021.5	897.68	19698.25	902.93
461.3	461.3	868.04	3506	886.46	9136	894.69	9545.03	895.39	9581	895.6	9637	895.81	10295.01	896.51	12021.5	898.03	19698.25	903.35
462.03	462.03	864.16	3506	886.8	9136	895.09	9545.03	895.77	9581	895.96	9637	896.17	10295.01	896.86	12021.5	898.39	19698.25	903.72
462.04	462.04	869.5	3506	886.83	9136	895.12	9545.03	895.81	9581	896	9637	896.2	10295.01	896.89	12021.5	898.42	19698.25	903.77
462.05			52nd Avenue So. (Fargo) / 60th Ave So. (Moorhead) Bridge															
462.06	462.06	869.5	3506	886.85	9136	895.13	9545.03	895.81	9581	896	9637	896.21	10295.01	896.9	12021.5	898.43	19698.25	903.85
462.07	462.07	871.5	3506	886.88	9136	895.16	9545.03	895.84	9581	896.03	9637	896.23	10295.01	896.92	12021.5	898.46	19698.25	903.85
462.7	462.7	862.82	3506	887.25	9136	895.49	9545.03	896.15	9581	896.33	9637	896.53	10295.01	897.22	12021.5	898.75	19698.25	904.15
463.7	463.7	864.71	3506	887.57	9136	895.9	9545.03	896.55	9581	896.72	9637	896.91	10295.01	897.6	12021.5	899.16	19698.25	904.58
464.2	464.2	869.2	3506	887.81	9136	896.17	9545.03	896.82	9581	896.98	9637	897.16	10295.01	897.86	12021.5	899.43	19698.25	904.91
464.6	464.6	865	3506	888.01	9136	896.37	9545.03	897	9581	897.16	9637	897.34	10295.01	898.04	12021.5	899.61	19698.25	905.07
465.1	465.1	862.9	3506	888.16	9136	896.6	9545.03	897.23	9581	897.39	9637	897.56	10295.01	898.27	12021.5	899.86	19698.25	905.43
465.9	465.9	865.6	3506	888.41	9136	896.95	9545.03	897.57	9581	897.72	9637	897.89	10295.01	898.6	12021.5	900.21	19698.25	905.83
466.5	466.5	864.9	3506	888.64	9136	897.22	9545.03	897.84	9581	897.98	9637	898.14	10295.01	898.85	12021.5	900.46	19698.25	906.13
467.4	467.4	864.8	3506	888.96	9136	897.63	9545.03	898.24	9581	898.38	9637	898.53	10295.01	899.25	12021.5	900.9	19698.25	906.6
468.2	468.2	862.6	3506	889.19	9136	897.9	9545.03	898.51	9581	898.64	9637	898.79	10295.01	899.52	12021.5	901.17	19698.25	906.88
468.9	468.9	861	3506	889.4	9136	898.18	9545.03	898.79	9581	898.92	9637	899.07	10295.01	899.8	12021.5	901.46	19698.25	907.12
469.7	469.7	870	3506	889.73	9136	898.58	9545.03	899.17	9581	899.3	9637	899.45	10295.01	900.18	12021.5	901.84	19698.25	907.39
470.1	470.1	870.9	3506	889.89	9136	898.76	9545.03	899.35	9581	899.47	9637	899.62	10295.01	900.35	12021.5	902.01	19698.25	907.57
470.6	470.6	869.4	2548	890.05	6444	898.95	7194.03	899.54	7230	899.66	7286	899.8	7943.02	900.53	9670.5	902.2	17546.25	907.77
471.4	471.4	867.5	2548	890.27	6358	899.2	7189.03	899.82	7330	899.94	7499	900.09	8205.02	900.83	9577.5	902.51	16740.25	908.11
472.18	472.18	869.9	2548	890.51	6358	899.44	7189.03	900.1	7330	900.22	7499	900.37	8205.02	901.12	9577.5	902.82	16740.25	908.44
472.2	472.2	870.8	2548	890.51	6358	899.44	7189.03	900.09	7330	900.21	7499	900.36	8205.02	901.12	9577.5	902.81	16740.25	908.43
472.22	472.22	870.8	2548	890.52	6358	899.45	7189.03	900.1	7330	900.22	7499	900.37	8205.02	901.13	9577.5	902.83	16740.25	908.47
473	473	871.8	2548	890.74	6358	899.67	7189.03	900.35	7330	900.47	7499	900.63	8205.02	901.39	9577.5	903.08	16740.25	908.72
473.8	473.8	869	2548	890.99	6358	899.93	7189.03	900.65	7330	900.78	7499	900.94	8205.02	901.72	9577.5	903.42	16740.25	909.09
474.08	474.08	873.5	2548	891.09	6358	900.02	7189.03	900.76	7330	900.89	7499	901.05	8205.02	901.84	9577.5	903.54	16740.25	909.23
474.09	474.09	873.5	2548	891.1	6358	900.05	7189.03	900.78	7330	900.92	7499	901.08	8205.02	901.87	9577.5	903.57	16740.25	909.29
474.1			Cass Co Hwy 16/112th Ave So. (Fargo) / 110th Ave So. (Moorhead) Bridge															
474.11	474.11	874	2548	891.1	6358	900.04	7189.03	900.78	7330	900.91	7499	901.07	8205.02	901.86	9577.5	903.57	16740.25	909.27
474.12	474.12	874	2548	891.11	6358	900.05	7189.03	900.79	7330	900.92	7499	901.08	8205.02	901.87	9577.5	903.58	16740.25	909.29
474.6	474.6	873.4	2548	891.32	6358	900.21	7189.03	900.96	7330	901.09	7499	901.26	8205.02	902.05	9577.5	903.75	16740.25	909.5
475.3	475.3	872.2	2548	891.56	6358	900.47	7189.03	901.26	7330	901.39	7499	901.56	8205.02	902.36	9577.5	904.07	16740.25	909.9
475.7	475.7	871	2548	891.69	6358	900.62	7189.03	901.42	7330	901.55	7499	901.73	8205.02	902.54	9577.5	904.26	16740.25	910.2
476.4	476.4	873.7	2548	891.97	6358	900.85	7189.03	901.68	7330	901.81	7499	901.99	8205.02	902.81	9577.5	904.53	16740.25	910.48
477	477	874	2550	892.23	5485	901.07	7133.03	901.94	8478	902.11	10024	902.32	11309.01	903.16	8592.5	904.81	9049.25	910.76
477.6	477.6	870.2	2550	892.42	5366	901.23	7125.03	902.17	8658	902.42	10433	902.75	11812.01	903.64	8450.5	905.03	8059.25	910.88
478	478	873.4	2550	892.56	5366	901.33	7125.03	902.32	8658	902.64	10433	903.05	11812.01	903.97	8450.5	905.16	8059.25	910.93
478.58	478.58	875	2550	892.67	5366	901.44	7125.03	902.49	8658	902.87	10433	903.36	11812.01	904.32	8450.5	905.32	8059.25	910.99
478.72	478.72	875.38	2550	892.69	5366	901.45	7125.03	902.5	8658	902.89	10433	903.39	11812.01	904.35	8450.5	905.33	8059.25	910.99
478.76	478.76	875.51	2550	892.69	5366	901.45	7125.03	902.5	8658	902.89	10433	903.38	11812.01	904.34	8450.5	905.33	8059.25	910.99
478.765	478.765		Proposed Red River Closure Structure															
478.77	478.77	875.51	2550	892.72	5366	902.96	7125.03	907.35	8658	909.65	10433	912.28	11812.01	913.91	8450.5	915.75	8059.25	917.82
478.81	478.81	875.64	2550	892.72	5366	902.98	7125.03	907.37	8658	909.66	10433	912.31	11812.01	913.95	8450.5	915.76	8059.25	917.82
479	479	876.35	2550	892.75	5366	902.99	8710	907.38	12762	909.69	18543	912.31	22626	913.95	24116	915.75	28246	917.81
479.2	479.2	875.7	2550	892.78	5366	903	8710	907.39	12762	909.7	18543	912.32	22626	913.96	24116	915.76	28246	917.84
479.7	479.7	871.5	2550	892.98	5366	903.1	8710	907.5	12762	909.86	18543	912.55	22626	914.2	24116	915.94	28246	917.93
480.5	480.5	874.7	2550	893.28	5366	903.27	8710	907.72	12762	910.17	18543	912.93	22626	914.59	24116	916.21	28246	918.12
481.8	481.8	875.3	2550	893.67	5366	903.54	8710	908.07	12762	910.84	18543	913.49	22626	915.21	24116	916.64	28246	918.39
482.3	482.3	874.2	2550	893.87	5366	903.69	8710	908.27	12762	910.91	18543	913.83	22626	915.54	24116	916.85	28246	918.51
482.698	482.698	869.74	2550	893.98	5366	903.77	8710	908.37	12762	911.05	18543	913.99	22626	915.73	24116	917.02	28246	918.65
482.7	482.7	887.04	2550	893.85	5366	903.71	8710	908.31	12762	910.97	18543	913.92	22626	915.67	24116	917	28246	918.65
482.702	482.702	878.54	2550	894.03	5366	903.77	8710	908.38	12762	911.05	18543	913.99	22626	915.73	24116	917.02	28246	918.65
483.1	483.1	879.2	2550	894.27	5366	903.93	8710	908.58	12762	911.34	18543	914.35	22626	916.1	24116	917.26	28246	918.75
483.9	483.9	873	2550	894.74	5366	904.17	8710	908.86	12762	911.73	18543	914.85	22626	916.59	24116	917.67	28246	919.06
485.08	485.08	880.5	2550	895.41	5366	904.41	8710	909.06	12762	911.98	18543	915.14	22626	916.9	24116	917.94	28246	919.31
485.09	485.09	880.5	2550	895.42	5366	904.4	8710	909.04	12762	911.98	18543	915.14	22626	916.9	24116	917.94	28246	919.31
485.1			Cass Co. Hwy 18/52nd St. SE (Fargo) / 160th Ave So. (Moorhead) Bridge															
485.11	485.11	879.4	2550	895.42	5366	904.4	8710	909.09	12762	912.09	18543	915.21	22626	916.95	24116	917.98	28246	919.34
485.12	485.12	879.4	2550	895.43	5366	904.42	8710	909.15	12762	912.1	18543	915.21	22626	916.95	24116	917.98	28246	919.34
485.6	485.6	878.62	2550	895.66	5366	904.52	8710	909.23	12762	912.16	18543	915.3	22626	917.05	24116	918.07	28246	919.43
485.9	485.9	879.5	2550	895.86	5366	904.62	8710	909.35	12762	912.32	18543	915.48	22626	917.24	24116	918.26	28246	919.62
486.3	486.3	872.1	2550	896.05	5366	904.73	8710	909.47	12762	912.48	18543	915.68	22626	917.47	24116	918.48	28246	919.85
487	487	871.3	2550	896.27	5366	904.88	8710	909.65	12762	912.75	18543	916.05	22626	917.93	24116	918.94	28246	920.37
487.5	487.5	878.8	2550	896.48	5366	905.04	8710	909.83	12762	912.97	18543	916.31	22626	918.21	24116	919.21	28246	920.64
488.68	488.68	878.24	2550	897.08	5366	905.51	8710	910.36	12762	913.68	18543	917.17	22626	919.13	24116	920.09	28246	921.59

Attachment B1

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West Fargo • Fergus Falls

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Memorandum

To: File, FM Metro Diversion Alternatives

From: Lyndon Pease and Stuart Dobberpuhl

Date: 04/12/2010

RE: 2009 Calibration of HEC-RAS Steady State Model (04/11/2010)

Work updating the 2009 HEC-RAS Calibration Steady State Model was accomplished in response to the two USACE ATR Comments on the Fargo-Moorhead Metro Feasibility Study. Each comment is flagged as a critical issue. Specifically, the two comments addressed are:

1. Comment 3087524 regarding the bank stations in the HEC-RAS model.

2. Comment 3087545 regarding the calibration of the HEC-RAS model at discharges less than 20,000 cfs.

In addition to the work for the two ATR comments, additional revisions to the 2009 HEC-RAS Calibration Model were made in response to internal review comments.

Below is a condensed summary of the changes for the 2009 Calibration for the Steady State model dated 03-25-2010. The date of this new calibration in the HEC-RAS model is 04-11-2010. Changes to bank stations, n values, and restructuring to the Oakport network were made.

1. Channel overbanks on the RRN were adjusted to match the primary bank elevation for the Red River of the North channel. Initial placement was governed from Houston Engineering's latest Unsteady Model for their 2009 Calibration. Further modifications ensure a channel width of 130-150 ft within the bank stations. This is a typical width in the primary bank for the RRN from Hickson to Perley. One channel n value is now used for the channel. This takes care of

2. known issues with regard to the HEC-RAS Version 4.0 software. The channel n value was then lowered from 0.055 to:

    a. RS 279 – 328 channel n value of 0.042
    b. RS 329 – 401 channel n value of 0.038
    c. RS 402 – 488.68 channel n value of 0.045

3. Overbank n values were initially increased using Houston Engineering's Unsteady Model. Further modifications were made which provided a consistent majority of overbank n values between 0.10 and 0.16.

4. The Oakport network was modified as shown below.

    a. Junction_1 between RRN RS 328 – 329 was removed. Connection to the Oakport_SI reach is now through RRN LS 329.1. The geometry of this lateral structure comes from Upper_Oakport_SE RS 22.15 (MN_Wall_Street/CR22).
    b. Junction_2 between RRN RS 311 – 312 was removed. Connection to the lob_rrn is now through RRN LS 312.1. The geometry of this lateral structure comes from lob_rrn RS 2.9 (Cass County Road 31).
    c. The Connection_Reach was removed.
    d. Junction_3 between RRN RS 310.5 – 311 was removed. Connection to the Oakport_SI reach is now through RRN LS 310.6. The geometry of this lateral structure comes from the now removed Connection_Reach RS 2.15.
    e. Junction_OakptsI was removed. The Upper_Oakport_SI and Lower Oakport_SI reaches were combined into one now called Oakport_SI. The river stations in the Upper_Oakport_SI reach were multiplied by ten to ensure proper stationing after this reach merge.
    f. RRN RS 310.4 was removed and all downstream distances from the FIS model were reinstated.
    g. Oakport_SI BR 16.5 was converted to an inline structure. The culvert at this crossing was removed as field inspection found no such culvert.

5. The benefits of doing this work are:

    a. The modifications will help with the resolution of the two outstanding ATR comments.
    b. The modifications results in a more reasonable composite n value for the channel in the HEC-RAS model.
    c. The 2009 Flood Event is now better calibrated against observed water surface elevations throughout the model.
    d. The modifications allow for a better calibration of the model for the full range of events. This is accomplished by having a lower n value for the channel and higher n values for the overbanks.

RED RIVER DIVERSION

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

APPENDIX C – HYDRAULICS WITH–PROJECT CONDITIONS

EXHIBIT 1 – FCP IMPACTS

Report for the US Army Corps of Engineers, and the cities of Fargo, North Dakota & Moorhead, Minnesota

FINAL: February 28, 2011

FIGURES

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Figure C-E1- 95: 2010 Historical Flood Hydrographs for FCP @ Grand Forks
Figure C-E1- 96: 2010 Historical Flood Hydrographs for FCP @ Drayton

Minnesota Diversion (FCP) - 10% Chance Event								2-28-2011
Location	Station	Existing No Protection		MN Diversion (FCP)		Difference (ft) Project vs. Existing No Protection	
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)
Drayton Gage	1062362	798.53	62917	798.57	63651	0.04	734
ND SH#17/ MN SH317	1223286	800.78		800.84		0.06	
Co. Hwy 15	1315673	802.60		802.65		0.05	
Minimum Impact Location	1410241	809.75		809.78		0.03	
Oslo Gage	1416287	810.51		810.56		0.05	
DS Grand Forks Levees	1533523	823.07		823.16		0.09	
Grand Forks Gage	1558518	825.98	56662	826.10	57258	0.12	596
32nd Ave, Grand Forks	1580152	827.76		827.90		0.14	
Thompson Gage	1667877	837.58	42815	837.82	43590	0.24	775
Co. Hwy 25/ Co. Rd 221	1726274	842.99		843.29		0.30	
DS Sandhill River/ Climax	1763746	845.62		845.93		0.31	
Nielsville	1829877	850.21	39667	850.54	40402	0.33	735
DS Marsh River	1864960	853.10		853.43		0.33	
US Goose River/ Shelly	1891054	855.90		856.24		0.34	
Halstad Gage	1981580	864.55	34653	864.88	35715	0.33	1063
Hendrum	2038409	868.94	32267	869.27	33471	0.33	1204
Perley	2129181	874.83	28448	875.23	29852	0.40	1404
Georgetown	2194021	880.34	29462	880.65	31317	0.31	1856
Maximum Impact Location	2236491	883.37		883.82		0.45	
North River/ Clay Co. Hwy 93	2305647	889.72		887.50		-2.22	
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	894.08		890.23		-3.85	
Fargo Gage (13th Ave S, 12th Ave S)	2388223	897.33 (34.59*)	17024	892.66 (29.92*)	9933	-4.67	-7091
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	901.61		896.31		-5.30	
US FCP Diversion	2470898	904.54		904.71		0.17	
US ND Wild Rice River	2484618	905.37		905.26		-0.11	
Hickson Gage	2563754	910.21	10428	910.27	10459	0.06	31
Abercrombie	2764835	929.05	11278	929.05	11278	0.00	0

* Flood stage at USGS Gaging Station 05054000, Fargo, ND

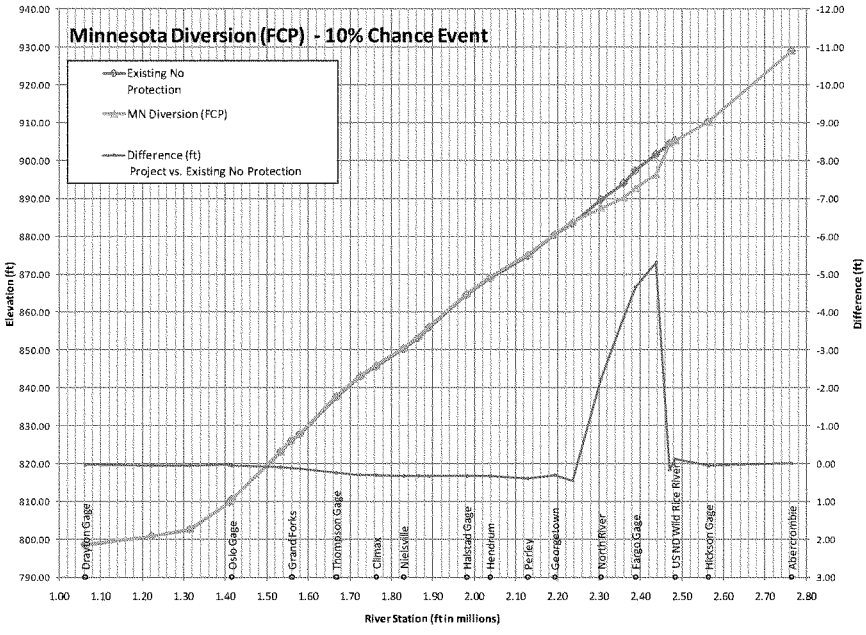

Figure C-E1- 1: 10-Percent Chance Impacts for FCP

Minnesota Diversion (FCP) - 2% Chance Event								2-28-2011
Location	Station	Existing No Protection		MN Diversion (FCP)		Difference (ft) Project vs. Existing No Protection	
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)
Drayton Gage	1062362	800.72	100869	800.83	102845	0.11	1976
ND SH#17/ MN SH317	1223286	803.19		803.30		0.11	
Co. Hwy 15	1315673	804.35		804.44		0.09	
Oslo Gage	1416287	812.53		812.56		0.03	
Minimum Impact Location	1448026	814.89		814.91		0.02	
DS Grand Forks Levees	1533523	826.86		826.94		0.08	
Grand Forks Gage	1558518	831.13	91118	831.26	92141	0.13	1023
32nd Ave, Grand Forks	1580152	833.57		833.83		0.26	
Thompson Gage	1667877	844.83	69367	845.61	73330	0.78	3963
Co. Hwy 25/ Co. Rd 221	1726274	851.40		852.33		0.93	
DS Sandhill River/ Climax	1763746	854.11		855.09		0.98	
Maximum Impact Location	1829650	858.51		859.52		1.01	
Nielsville	1829877	858.57	63543	859.57	67968	1.00	4425
DS Marsh River	1864960	860.76		861.65		0.89	
US Goose River/ Shelly	1891054	863.04		863.85		0.81	
Halstad Gage	1981580	867.99	59416	868.47	65150	0.48	5735
Hendrum	2038409	872.82	56143	873.29	62201	0.47	6058
Perley	2129181	877.75	51248	878.15	56562	0.40	5314
Georgetown	2194021	882.76	53909	883.11	59776	0.35	5866
North River/ Clay Co. Hwy 93	2305647	892.97		889.27		-3.70	
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	898.21		891.75		-6.46	
Fargo Gage (13th Ave S, 12th Ave S)	2388223	902.6 (39.86*)	29167	894.02 (31.28*)	10878	-8.58	-18289
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	906.94		897.41		-9.53	
US FCP Diversion	2470898	909.54		909.40		-0.14	
US ND Wild Rice River	2484618	910.11		910.17		0.06	
Hickson Gage	2563754	916.34	18898	916.37	18925	0.03	27
Abercrombie	2764835	934.48	20726	934.49	20726	0.01	0

* Flood stage at USGS Gaging Station 05054000, Fargo, ND

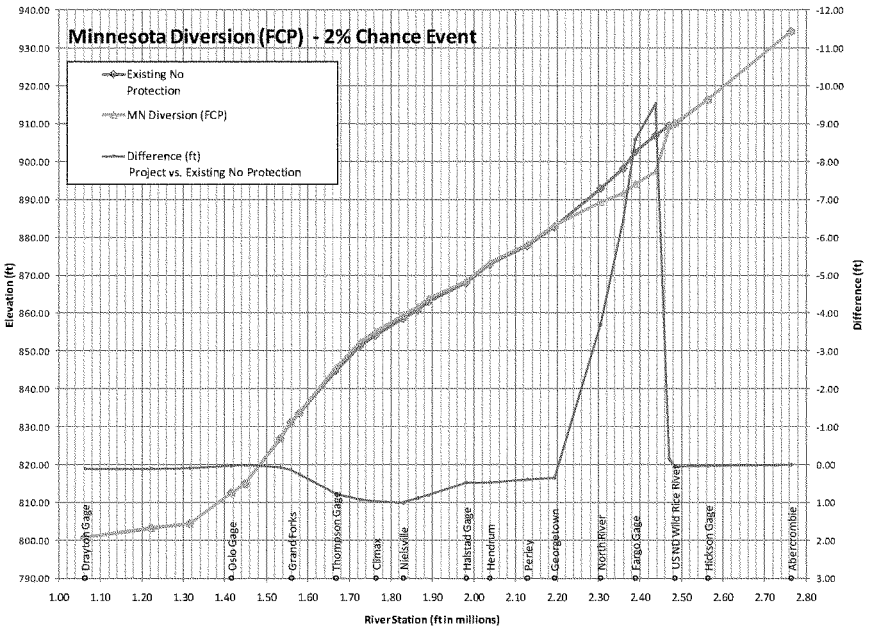

Figure C-E1- 2: 2-Percent Chance Impacts for FCP

Minnesota Diversion (FCP) - 1% Chance Event								2-28-2011
Location	Station	Existing No Protection		MN Diversion (FCP)		Difference (ft) Project vs. Existing No Protection	
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)
Drayton Gage	1062362	801.73	119255	801.92	122945	0.19	3690
ND SH#17/ MN SH317	1223286	804.09		804.27		0.18	
Co. Hwy 15	1315673	805.08		805.23		0.15	
Minimum Impact Location	1408098	811.34		811.39		0.05	
Oslo Gage	1416287	813.01		813.09		0.08	
DS Grand Forks Levees	1533523	827.98		828.21		0.23	
Grand Forks Gage	1558518	832.97	107980	833.35	112047	0.38	4067
32nd Ave, Grand Forks	1580152	835.83		836.37		0.54	
Thompson Gage	1667877	847.35	82926	848.11	88519	0.76	5593
Co. Hwy 25/ Co. Rd 221	1726274	854.46		855.56		1.10	
DS Sandhill River/ Climax	1763746	857.34		858.52		1.18	
Maximum Impact Location	1813905	860.78		862.01		1.23	
Nielsville	1829877	861.66	75745	862.87	82280	1.21	6536
DS Marsh River	1864960	863.43		864.47		1.04	
US Goose River/ Shelly	1891054	865.36		866.25		0.89	
Halstad Gage	1981580	869.09	71581	869.68	80624	0.59	9043
Hendrum	2038409	873.75	67278	874.36	77118	0.61	9839
Perley	2129181	878.50	61693	879.11	71345	0.61	9652
Georgetown	2194021	883.36	63847	883.88	75540	0.52	11692
North River/ Clay Co. Hwy 93	2305647	893.73		890.13		-3.60	
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	899.08		892.65		-6.43	
Fargo Gage (13th Ave S, 12th Ave S)	2388223	903.86 (41.12*)	34875	894.91 (32.17*)	11756	-8.95	-23119
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	907.71		898.26		-9.45	
US FCP Diversion	2470898	910.13		910.71		0.58	
US ND Wild Rice River	2484618	910.70		911.15		0.45	
Hickson Gage	2563754	917.52	21730	917.51	21734	-0.01	3
Abercrombie	2764835	935.62	23000	935.62	23000	0.00	0

* Flood stage at USGS Gaging Station 05054000, Fargo, ND

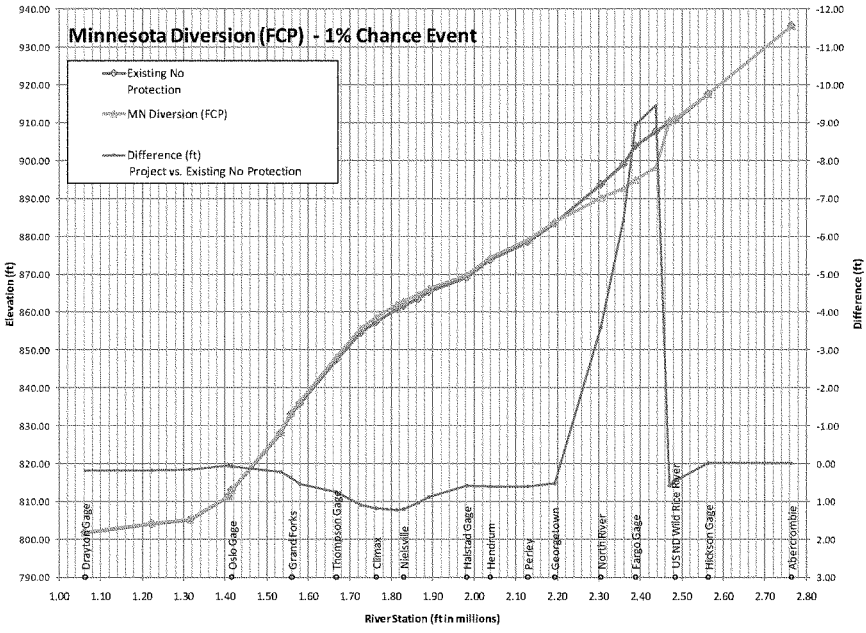

Figure C-E1- 3: 1-Percent Chance Impacts for FCP

Minnesota Diversion (FCP) - 0.2% Chance Event								2-28-2011
Location	Station	Existing No Protection		MN Diversion (FCP)		Difference (ft) Project vs. Existing No Protection	
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)
Drayton Gage	1062362	804.12	168364	804.27	170409	0.15	2045
ND SH#17/ MN SH317	1223286	805.99		806.10		0.11	
Co. Hwy 15	1315673	806.74		806.87		0.13	
Oslo Gage	1416287	813.88		813.95		0.07	
Minimum Impact Location	1416400	814.23		814.29		0.06	
DS Grand Forks Levees	1533523	829.92		830.11		0.19	
Grand Forks Gage	1558518	836.36	146225	836.72	150748	0.36	4523
32nd Ave, Grand Forks	1580152	839.75		840.20		0.45	
Maximum Impact Location	1580152	839.75		840.20		0.45	
Thompson Gage	1667877	850.69	112422	850.93	115330	0.24	2908
Co. Hwy 25/ Co. Rd 221	1726274	859.38		859.71		0.33	
DS Sandhill River/ Climax	1763746	862.75		863.13		0.38	
Nielsville	1829877	867.04	107296	867.45	110696	0.41	3399
DS Marsh River	1864960	868.06		868.43		0.37	
US Goose River/ Shelly	1891054	869.30		869.64		0.34	
Halstad Gage	1981580	871.54	101754	871.72	104334	0.18	2580
Hendrum	2038409	875.77	97650	875.86	98430	0.09	780
Perley	2129181	879.89	90756	879.95	89907	0.06	-849
Georgetown	2194021	884.48	92746	884.46	91337	-0.02	-1409
North River/ Clay Co. Hwy 93	2305647	895.35		893.27		-2.08	
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	900.31		898.40		-1.91	
Fargo Gage (13th Ave S, 12th Ave S)	2388223	905.8 (43.06*)	61717	902.83 (40.09*)	30044	-2.97	-31673
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	909.13		907.28		-1.85	
US FCP Diversion	2470898	910.99		910.81		-0.18	
US ND Wild Rice River	2484618	911.54		911.79		0.25	
Hickson Gage	2563754	919.69	35636	919.67	35565	-0.02	-71
Abercrombie	2764835	940.90	44308	940.90	44308	0.00	0

* Flood stage at USGS Gaging Station 05054000, Fargo, ND

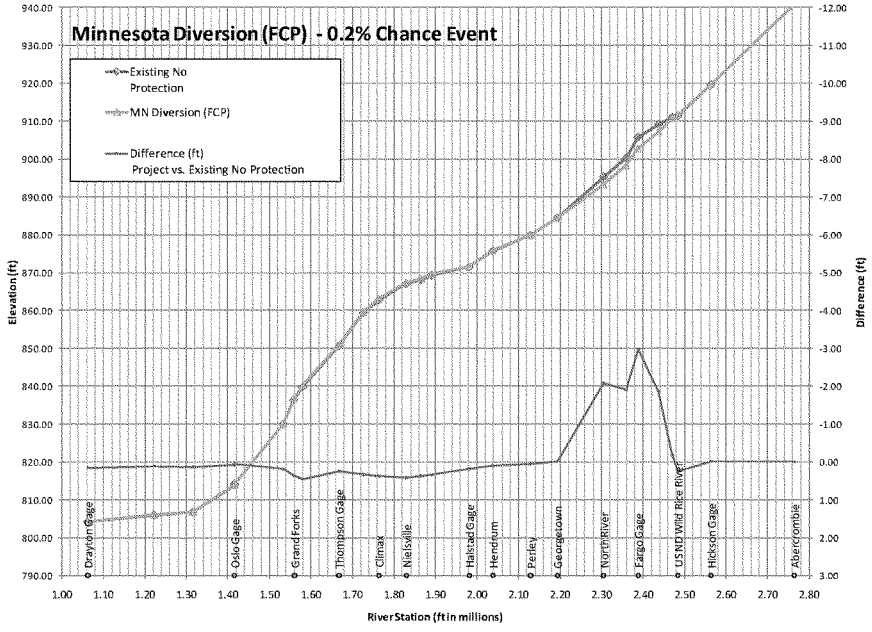

Figure C-E1- 4: 0.2-Percent Chance Impacts for FCP

Minnesota Diversion (FCP) -1997 Event (No Protection)								2-28-2011
Location	Station	Existing No Protection		MN Diversion (FCP)		Difference (ft) Project vs. Existing No Protection	
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)
Drayton Gage	1062362	801.95	123404	802.05	125375	0.10	1971
ND SH#17/ MN SH317	1223286	804.31		804.40		0.09	
Co. Hwy 15	1315673	805.30		805.39		0.09	
Oslo Gage	1416287	813.29		813.34		0.05	
Minimum Impact Location	1425253	814.37		814.40		0.03	
DS Grand Forks Levees	1533523	828.62		828.72		0.10	
Grand Forks Gage	1558518	834.04	119103	834.21	120893	0.17	1790
32nd Ave, Grand Forks	1580152	837.02		837.24		0.22	
Thompson Gage	1667877	847.29	78351	847.66	81143	0.37	2792
Co. Hwy 25/ Co. Rd 221	1726274	853.81		854.36		0.55	
DS Sandhill River/ Climax	1763746	856.49		857.10		0.61	
Maximum Impact Location	1813905	859.97		860.60		0.63	
Nielsville	1829877	860.86	71728	861.48	74715	0.62	2987
DS Marsh River	1864960	862.72		863.26		0.54	
US Goose River/ Shelly	1891054	864.75		865.21		0.46	
Halstad Gage	1981580	868.65	64821	868.92	68476	0.27	3655
Hendrum	2038409	873.24	55101	873.51	58191	0.27	3090
Perley	2129181	878.05	54689	878.28	58214	0.23	3525
Georgetown	2194021	882.92	56376	883.15	59737	0.23	3361
North River/ Clay Co. Hwy 93	2305647	893.04		889.40		-3.64	
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	898.26		892.05		-6.21	
Fargo Gage (13th Ave S, 12th Ave S)	2388223	902.42 (39.68*)	27574	894.1 (31.36*)	9978	-8.32	-17596
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	906.49		896.30		-10.19	
US FCP Diversion	2470898	908.85		908.94		0.09	
US ND Wild Rice River	2484618	909.37		909.60		0.23	
Hickson Gage	2563754	913.85	13729	914.00	13738	0.15	10
Abercrombie	2764835	931.08	13995	931.08	13995	0.00	0

* Flood stage at USGS Gaging Station 05054000, Fargo, ND

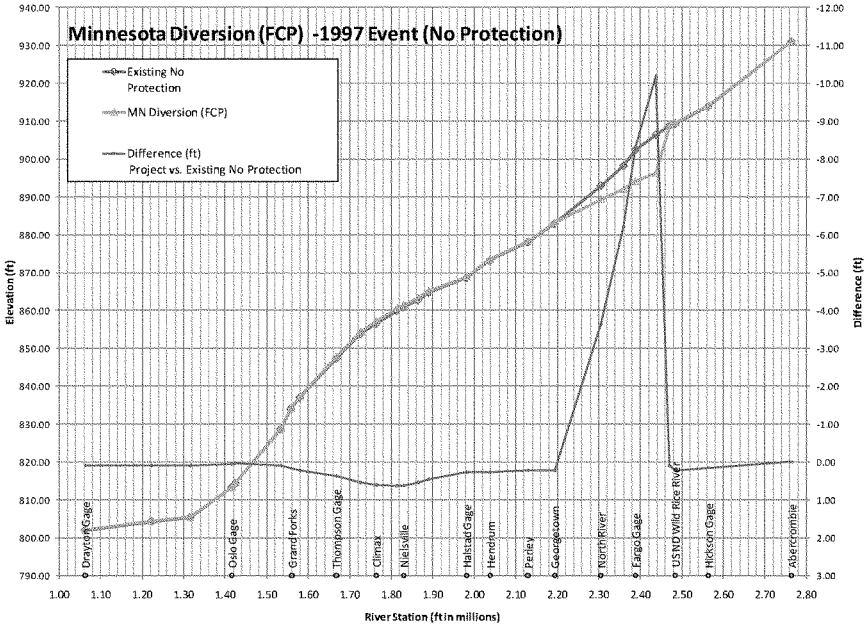

Figure C-E1- 5: 1997 Historical Flood Impacts for FCP

Minnesota Diversion (FCP) -2006 Event (No Protection)								2-28-2011
Location	Station	Existing No Protection		MN Diversion (FCP)		Difference (ft) Project vs. Existing No Protection	
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)
Drayton Gage	1062362	799.44	78252	799.47	78770	0.03	518
ND SH#17/ MN SH317	1223286	801.84		801.87		0.03	
Co. Hwy 15	1315673	803.39		803.42		0.03	
Oslo Gage	1416287	811.58		811.60		0.02	
Minimum Impact Location	1448026	814.15		814.16		0.01	
DS Grand Forks Levees	1533523	825.11		825.15		0.04	
Grand Forks Gage	1558518	828.63	72782	828.69	73160	0.06	378
32nd Ave, Grand Forks	1580152	830.60		830.67		0.07	
Thompson Gage	1667877	840.63	52499	840.84	53450	0.21	951
Co. Hwy 25/ Co. Rd 221	1726274	846.54		846.83		0.29	
DS Sandhill River/ Climax	1763746	849.17		849.45		0.28	
Nielsville	1829877	853.70	48037	854.02	49044	0.32	1007
DS Marsh River	1864960	856.33		856.65		0.32	
US Goose River/ Shelly	1891054	859.08		859.42		0.34	
Halstad Gage	1981580	866.64	43060	866.86	44955	0.22	1895
Hendrum	2038409	870.74	36229	871.09	38142	0.35	1913
Maximum Impact Location	2058853	871.99		872.36		0.37	
Perley	2129181	876.40	36253	876.61	38349	0.21	2096
Georgetown	2194021	881.37	36894	881.59	39723	0.22	2829
North River/ Clay Co. Hwy 93	2305647	891.46		888.25		-3.21	
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	895.97		890.84		-5.13	
Fargo Gage (13th Ave S, 12th Ave S)	2388223	899.57 (36.83*)	21028	893.15 (30.41*)	10078	-6.42	-10950
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	903.97		896.67		-7.30	
US FCP Diversion	2470898	906.81		906.53		-0.28	
US ND Wild Rice River	2484618	907.56		907.21		-0.35	
Hickson Gage	2563754	913.11	14313	913.15	14352	0.04	39
Abercrombie	2764835	931.58	15027	931.58	15027	0.00	0

* Flood stage at USGS Gaging Station 05054000, Fargo, ND

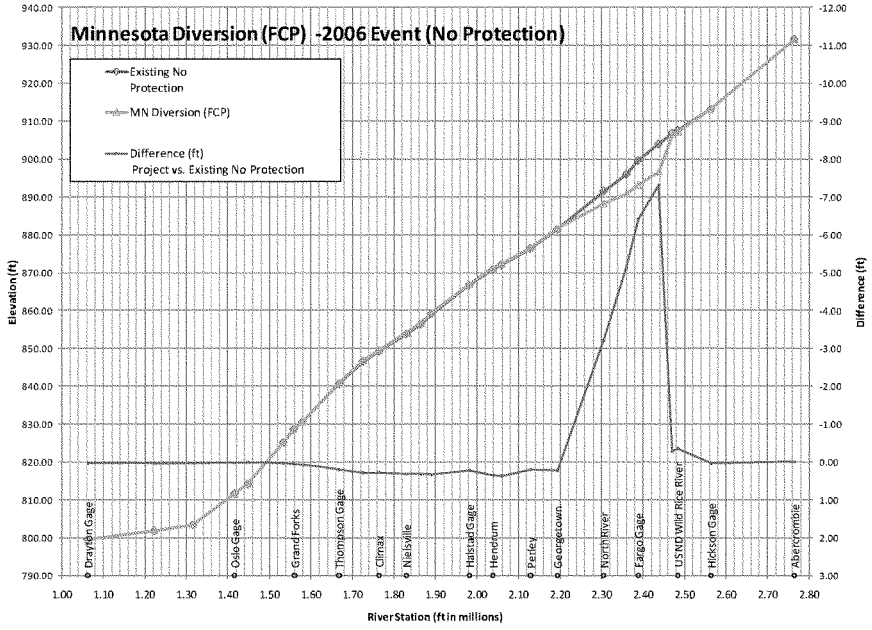

Figure C-E1- 6: 2006 Historical Flood Impacts for FCP

Minnesota Diversion (FCP) -2009 Event (No Protection)								2-28-2011
Location	Station	Existing No Protection		MN Diversion (FCP)		Difference (ft) Project vs. Existing No Protection	
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)
Drayton Gage	1062362	799.85	85308	799.98	87702	0.13	2393
ND SH#17/ MN SH317	1223286	802.36		802.52		0.16	
Co. Hwy 15	1315673	803.81		803.93		0.12	
Minimum Impact Location	1410241	810.81		810.89		0.08	
Oslo Gage	1416287	812.02		812.16		0.14	
DS Grand Forks Levees	1533523	825.65		826.01		0.36	
Grand Forks Gage	1558518	829.33	77165	829.83	80831	0.50	3666
32nd Ave, Grand Forks	1580152	831.80		832.45		0.65	
Thompson Gage	1667877	843.05	61510	843.97	65379	0.92	3869
Co. Hwy 25/ Co. Rd 221	1726274	849.33		850.39		1.06	
DS Sandhill River/ Climax	1763746	851.96		853.06		1.10	
Maximum Impact Location	1789494	853.76		854.88		1.12	
Nielsville	1829877	856.67	58164	857.79	62482	1.12	4318
DS Marsh River	1864960	859.13		860.15		1.02	
US Goose River/ Shelly	1891054	861.73		862.65		0.92	
Halstad Gage	1981580	867.60	55176	868.02	60798	0.42	5622
Hendrum	2038409	872.25	44075	872.88	48595	0.63	4520
Perley	2129181	877.21	45715	877.73	51361	0.52	5645
Georgetown	2194021	882.32	47470	882.82	54928	0.50	7458
North River/ Clay Co. Hwy 93	2305647	892.95		889.08		-3.87	
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	898.28		891.74		-6.54	
Fargo Gage (13th Ave S, 12th Ave S)	2388223	902.66 (39.92*)	29234	894.03 (31.29*)	11964	-8.63	-17270
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	906.96		897.80		-9.16	
US FCP Diversion	2470898	909.61		909.47		-0.14	
US ND Wild Rice River	2484618	910.20		910.22		0.02	
Hickson Gage	2563754	917.76	24393	917.75	24407	-0.01	14
Abercrombie	2764835	937.51	28176	937.51	28176	0.00	0

* Flood stage at USGS Gaging Station 05054000, Fargo, ND

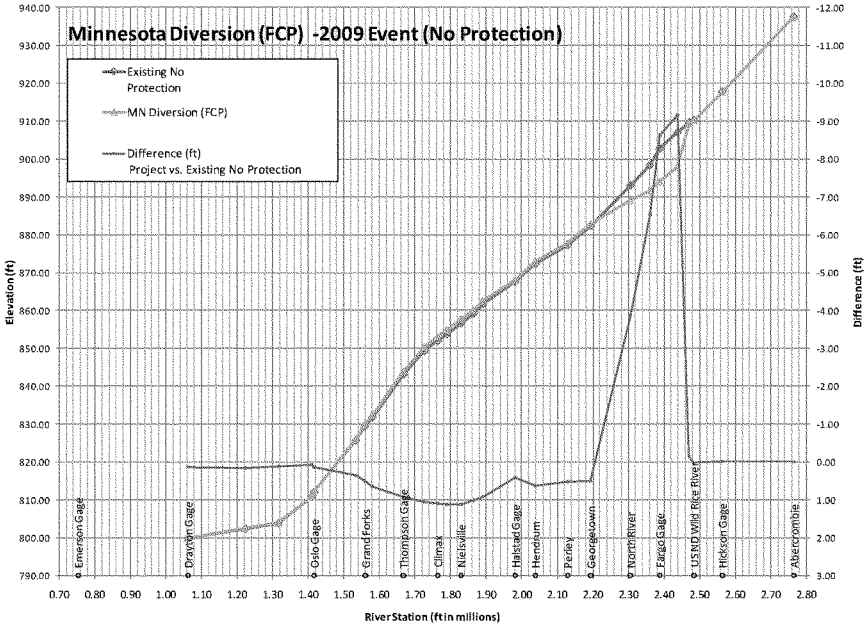

Figure C-E1- 7: 2009 Historical Flood Impacts for FCP

Minnesota Diversion (FCP) -2010 Event (No Protection)								2-28-2011
Location	Station	Existing No Protection		MN Diversion (FCP)		Difference (ft) Project vs. Existing No Protection	
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)
Drayton Gage	1062362	798.71	65928	798.76	66687	0.05	759
ND SH#17/ MN SH317	1223286	801.04		801.10		0.06	
Co. Hwy 15	1315673	802.96		803.00		0.04	
Oslo Gage	1416287	811.09		811.11		0.02	
Minimum Impact Location	1467237	815.28		815.30		0.02	
DS Grand Forks Levees	1533523	824.07		824.12		0.05	
Grand Forks Gage	1558518	827.23	63406	827.29	63783	0.06	377
32nd Ave, Grand Forks	1580152	829.35		829.42		0.07	
Thompson Gage	1667877	840.28	52023	840.55	53139	0.27	1116
Co. Hwy 25/ Co. Rd 221	1726274	846.28		846.61		0.33	
DS Sandhill River/ Climax	1763746	848.95		849.27		0.32	
Maximum Impact Location	1829650	853.73		854.10		0.37	
Nielsville	1829877	853.84	49914	854.20	51122	0.36	1208
DS Marsh River	1864960	856.61		856.95		0.34	
US Goose River/ Shelly	1891054	859.37		859.71		0.34	
Halstad Gage	1981580	866.55	42389	866.76	43888	0.21	1499
Hendrum	2038409	870.62	38264	870.97	40448	0.35	2183
Perley	2129181	876.31	35431	876.53	37676	0.22	2245
Georgetown	2194021	881.30	36134	881.54	39235	0.24	3100
North River/ Clay Co. Hwy 93	2305647	891.54		888.36		-3.18	
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	896.11		891.04		-5.07	
Fargo Gage (13th Ave S, 12th Ave S)	2388223	899.77 (37.03*)	21481	893.37 (30.63*)	10231	-6.40	-11250
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	904.15		896.74		-7.41	
US FCP Diversion	2470898	906.89		906.80		-0.09	
US ND Wild Rice River	2484618	907.59		907.42		-0.17	
Hickson Gage	2563754	912.23	12677	912.42	12697	0.19	20
Abercrombie	2764835	930.57	13236	930.57	13236	0.00	0

* Flood stage at USGS Gaging Station 05054000, Fargo, ND

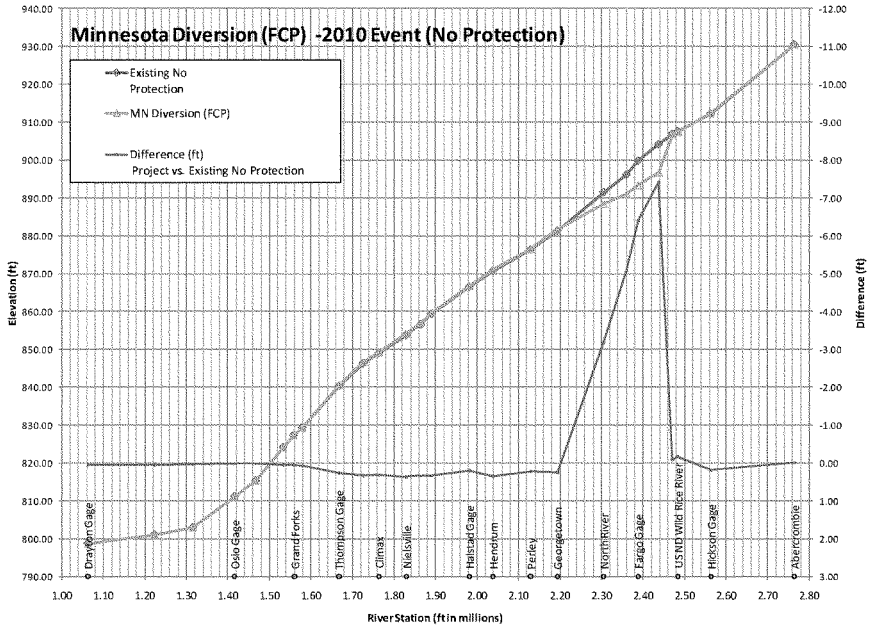

Figure C-E1- 8: 2010 Historical Flood Impacts for FCP

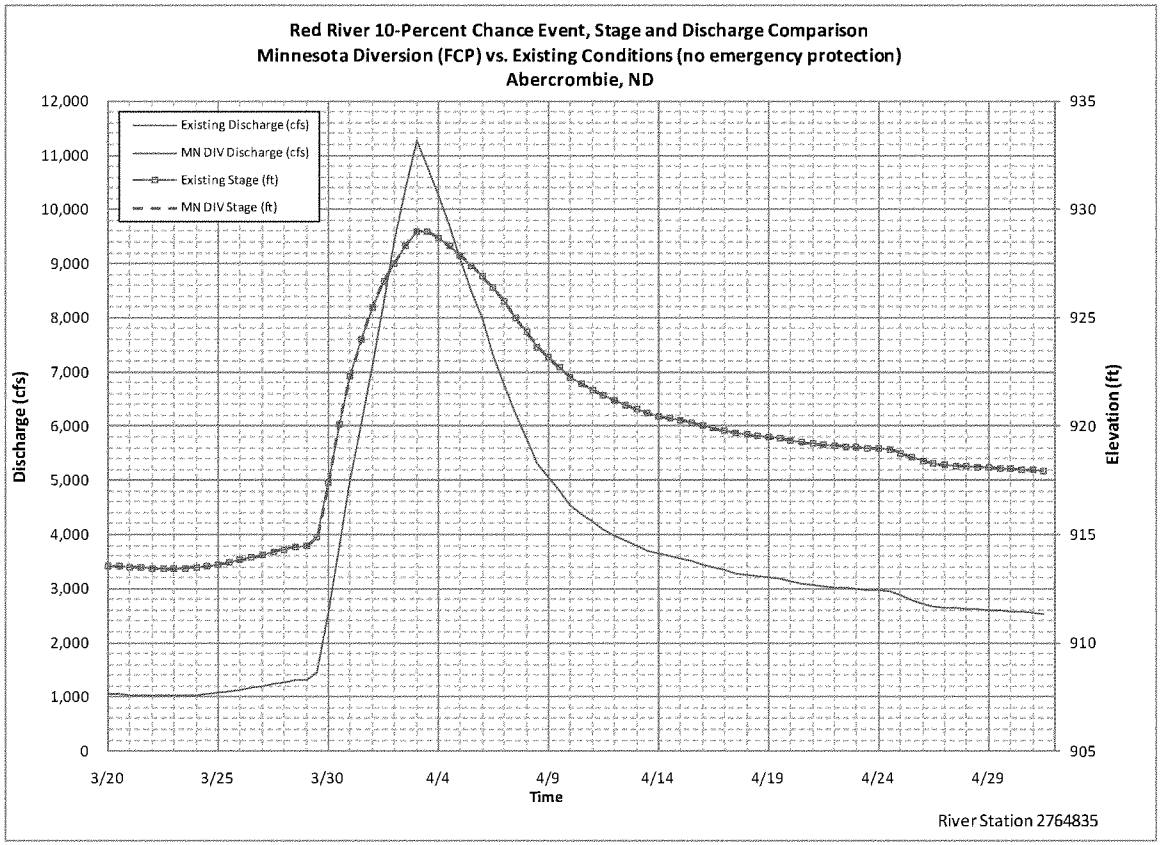

Figure C-E1- 9: 10-Percent Chance Hydrographs for FCP @ Abercrombie

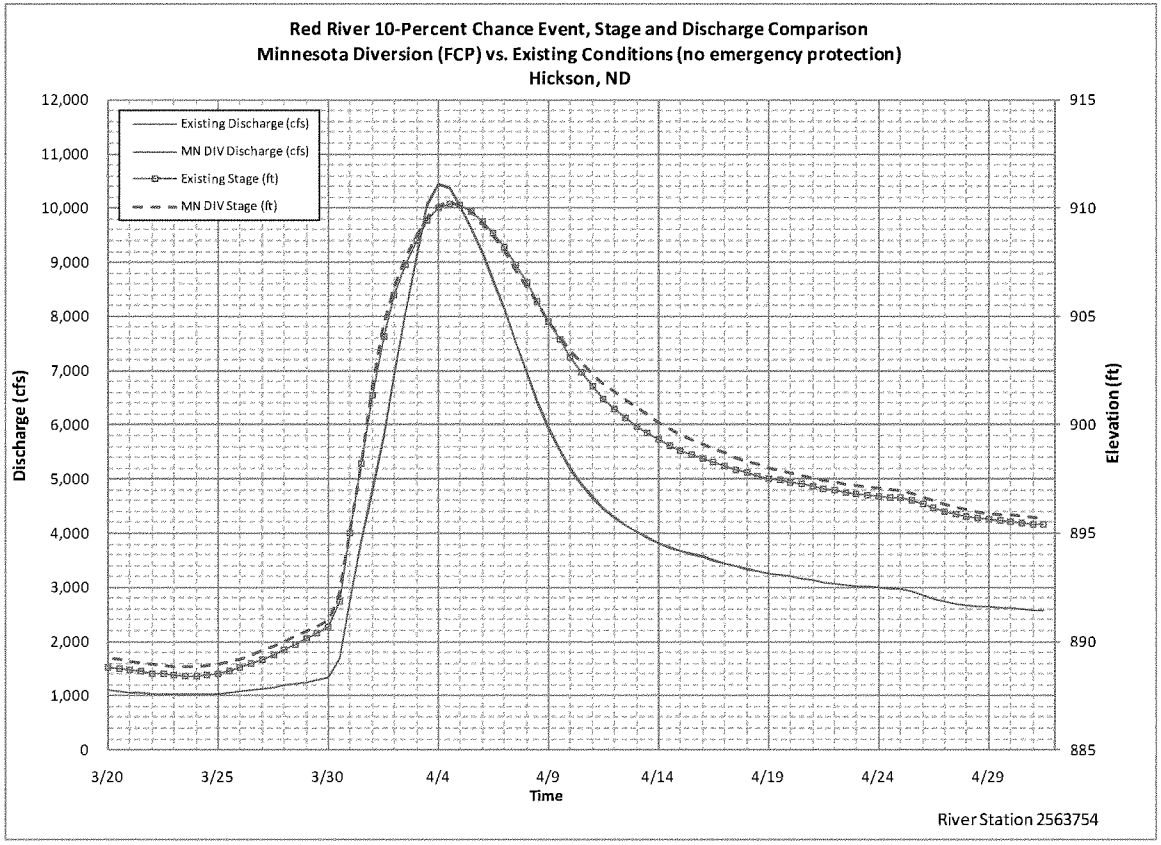

Figure C-E1- 10: 10-Percent Chance Hydrographs for FCP @ Hickson

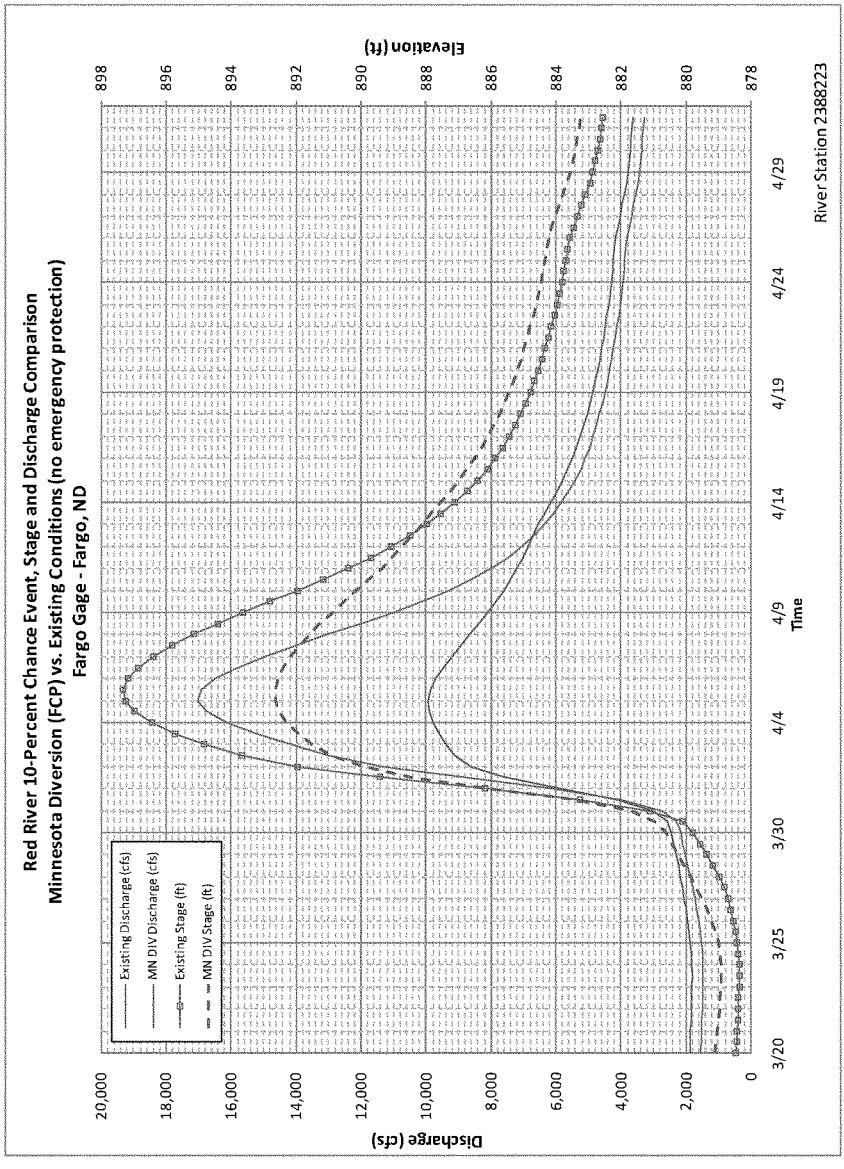

Figure C-E1- 11: 10-Percent Chance Hydrographs for FCP @ Fargo

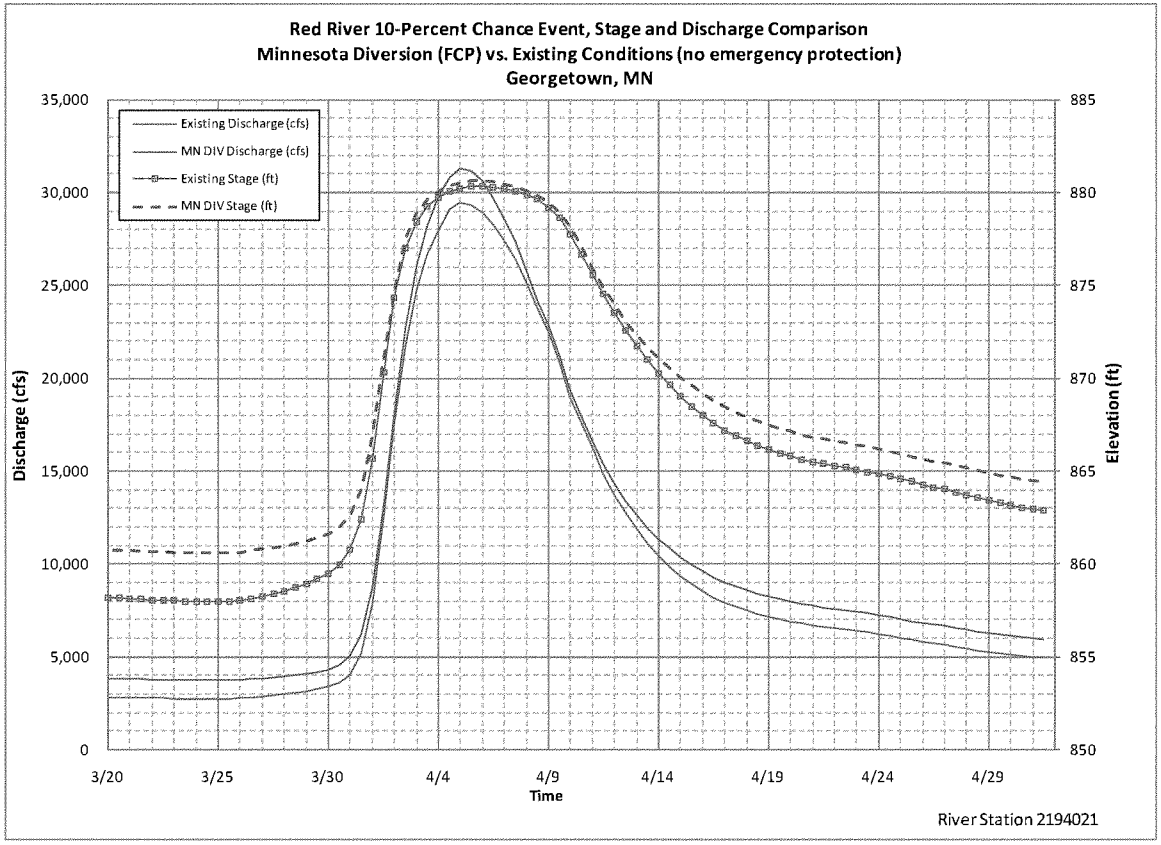

Figure C-E1- 12: 10-Percent Chance Hydrographs for FCP @ Georgetown

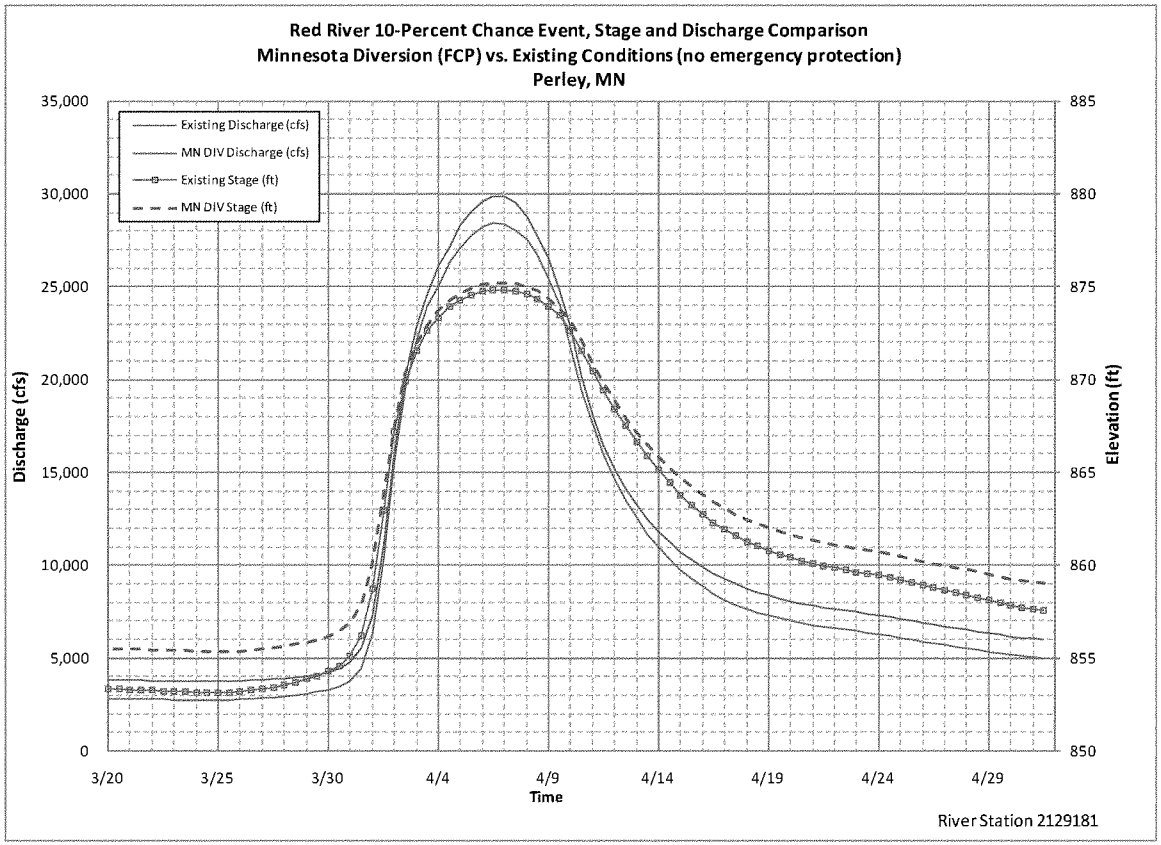

Figure C-E1- 13: 10-Percent Chance Hydrographs for FCP @ Perley

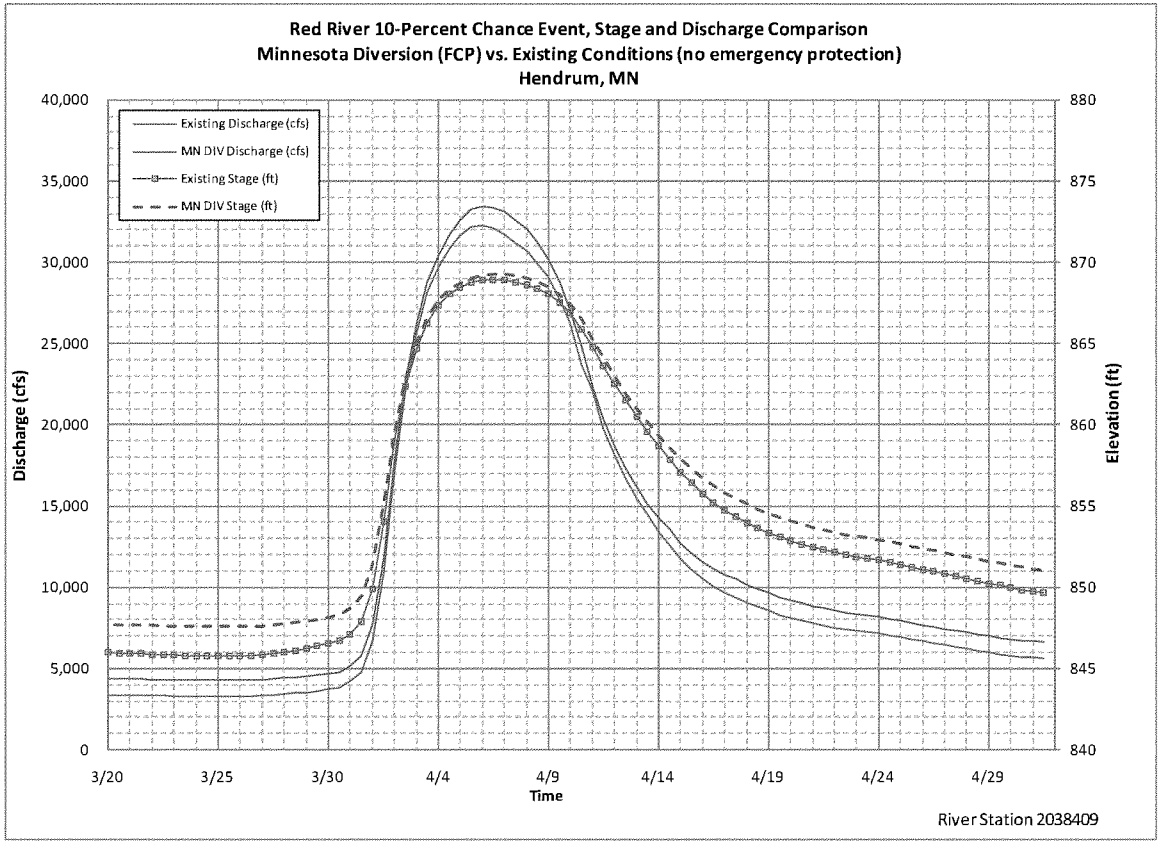

Figure C-E1- 14: 10-Percent Chance Hydrographs for FCP @ Hendrum

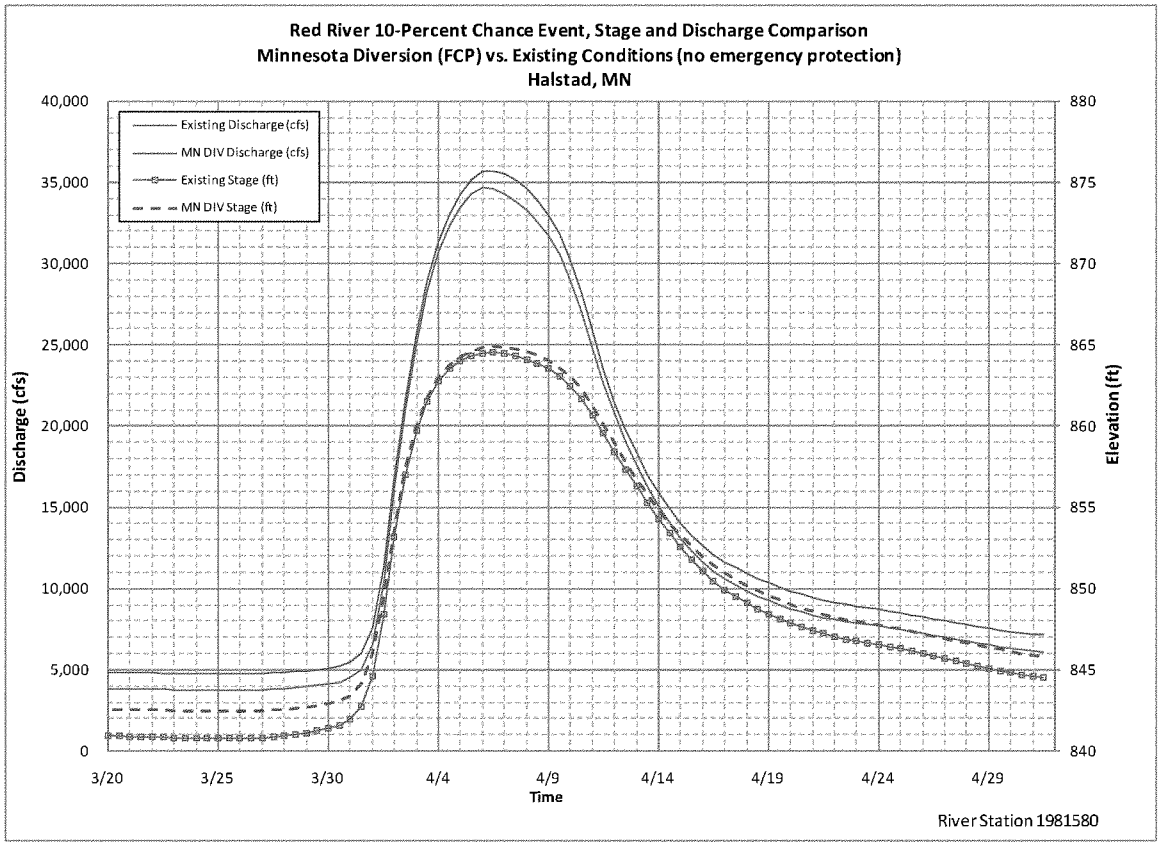

Figure C-E1- 15: 10-Percent Chance Hydrographs for FCP @ Halstad

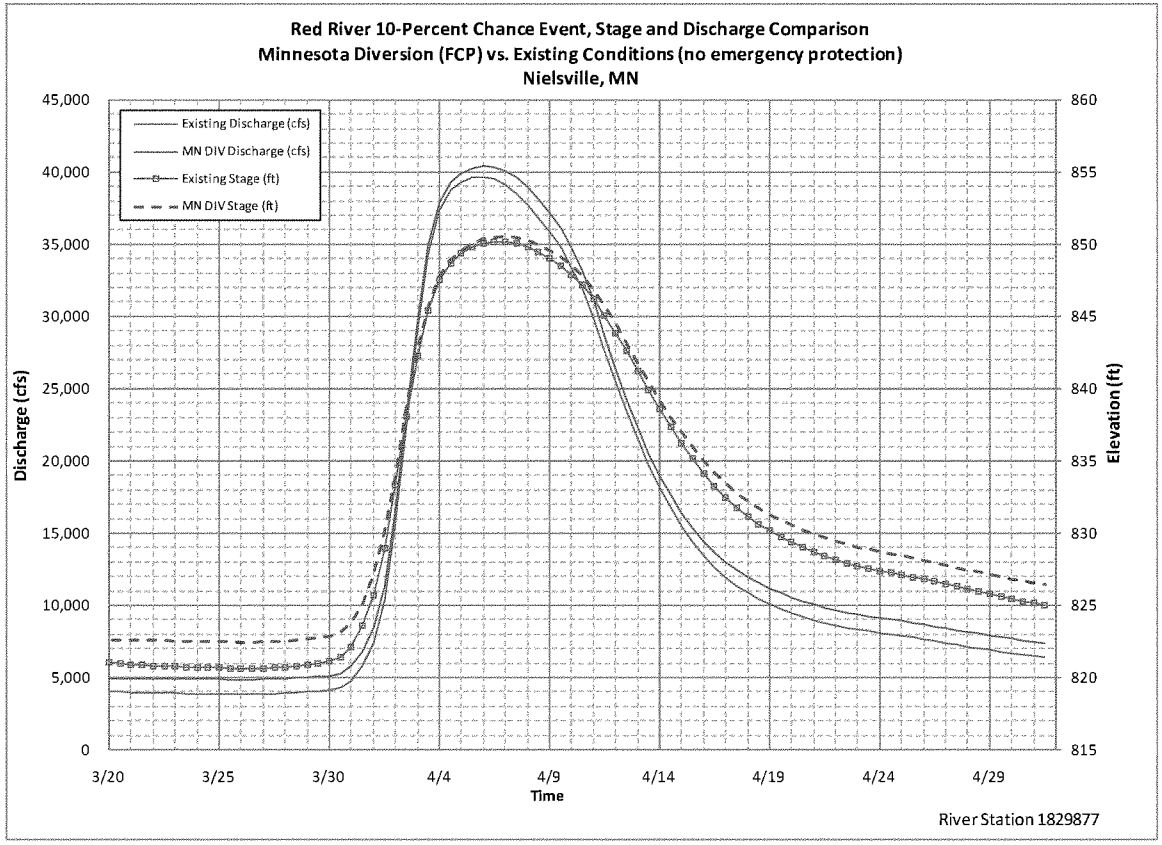

Figure C-E1- 16: 10-Percent Chance Hydrographs for FCP @ Nielsville

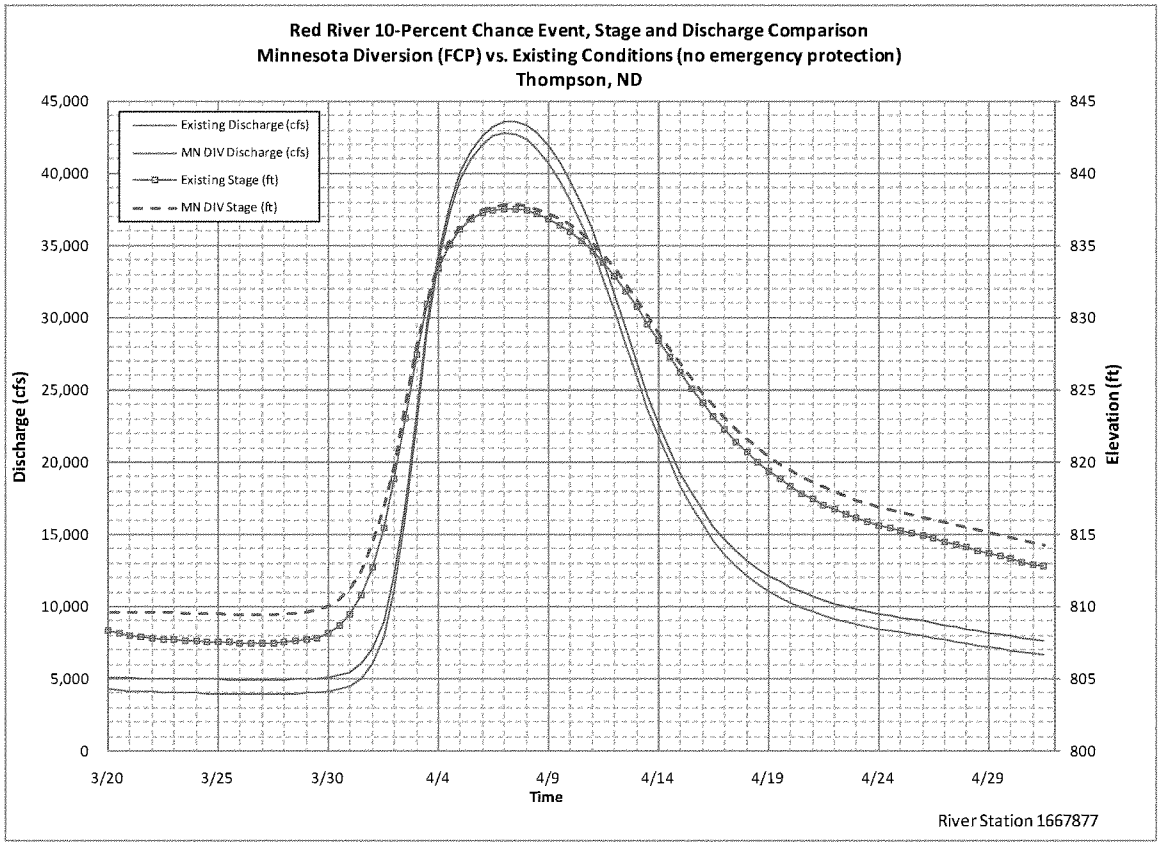

Figure C-E1- 17: 10-Percent Chance Hydrographs for FCP @ Thompson

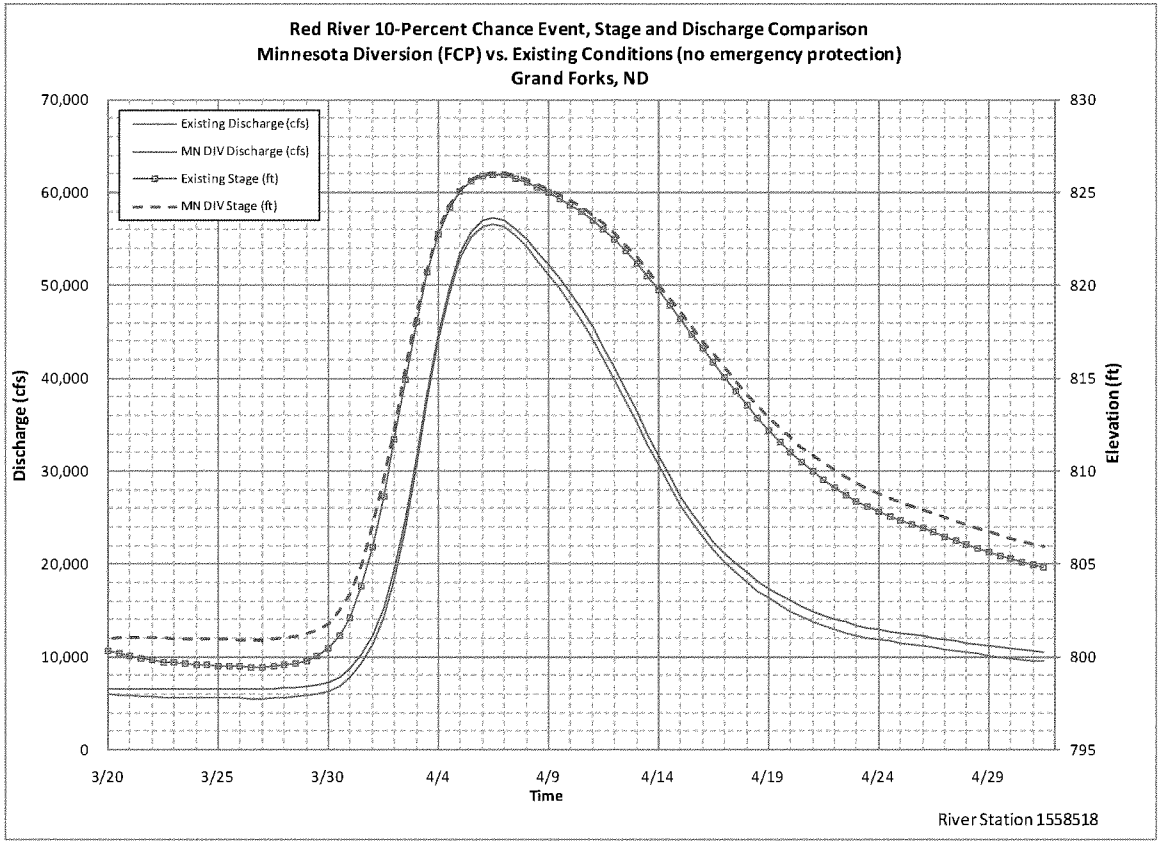

Figure C-E1- 18: 10-Percent Chance Hydrographs for FCP @ Grand Forks

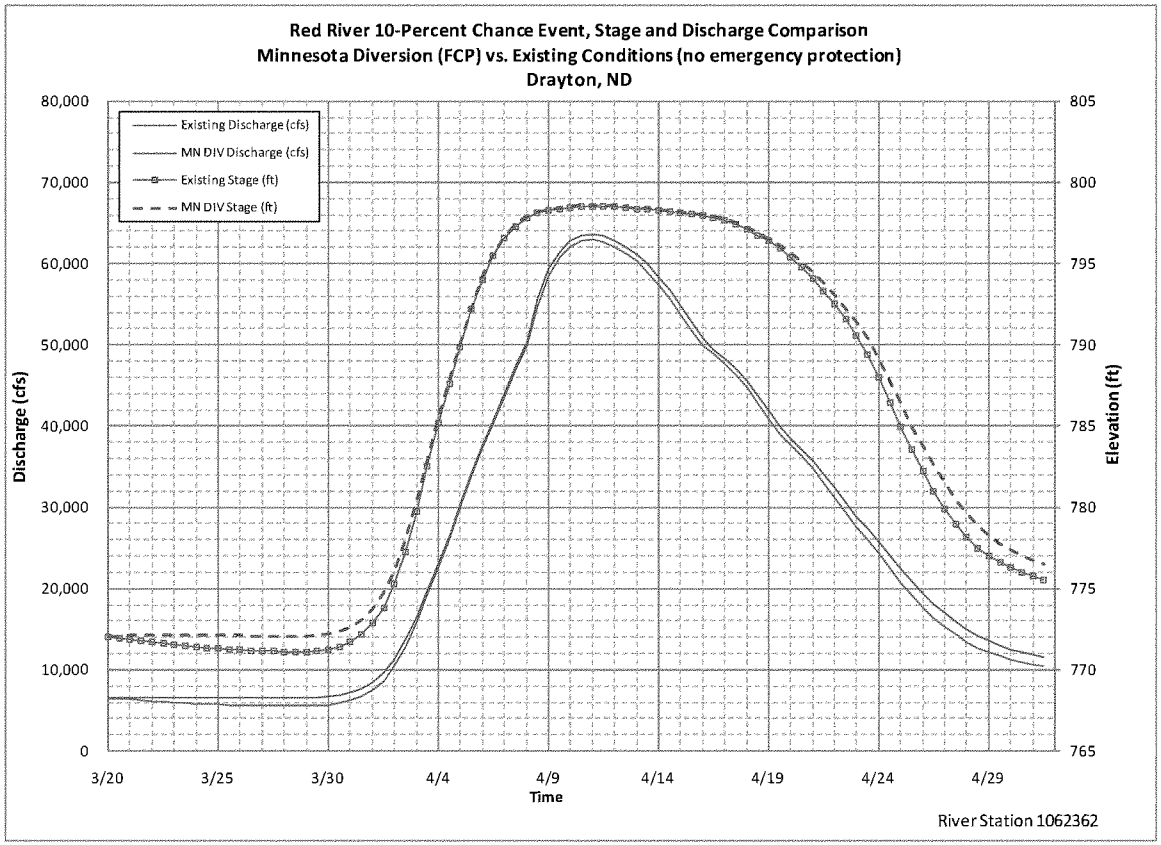

Figure C-E1- 19: 10-Percent Chance Hydrographs for FCP @ Drayton

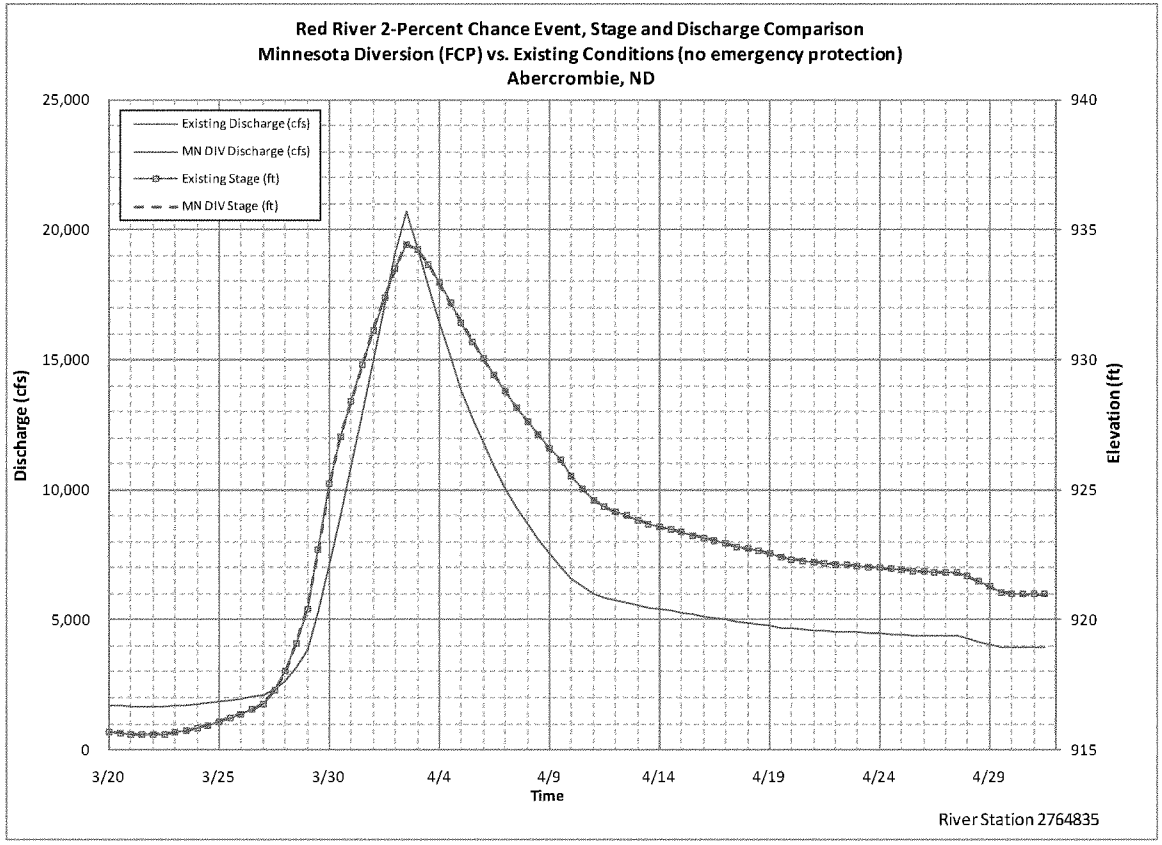

Figure C-E1- 20: 2-Percent Chance Hydrographs for FCP @ Abercrombie

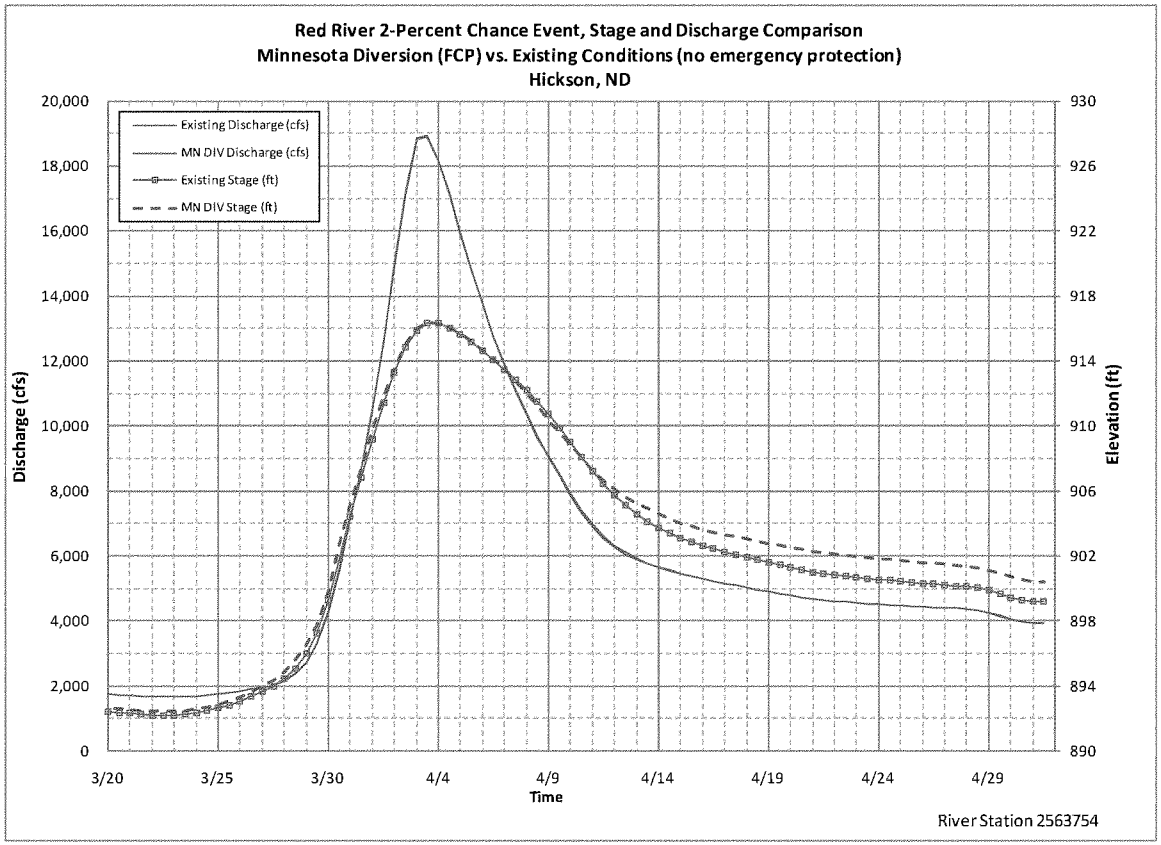

Figure C-E1- 21: 2-Percent Chance Hydrographs for FCP @ Hickson

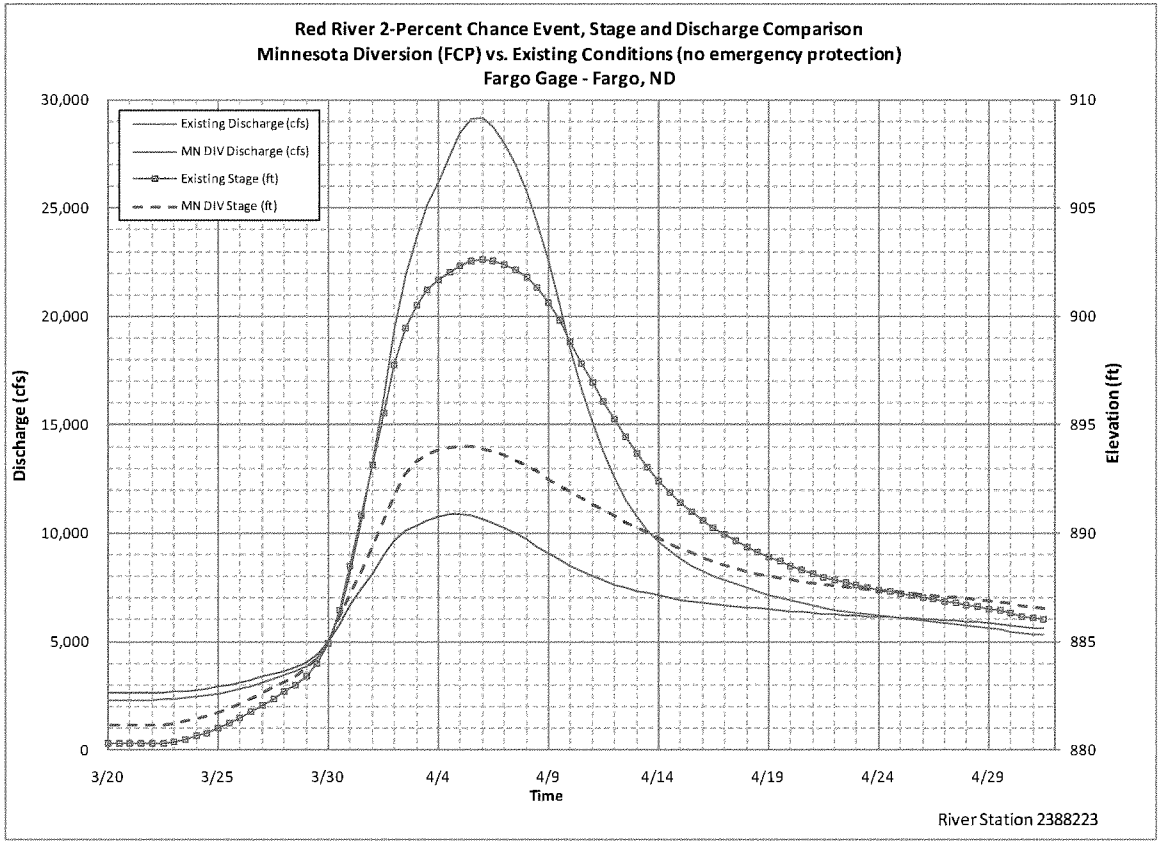

Figure C-E1- 22: 2-Percent Chance Hydrographs for FCP @ Fargo

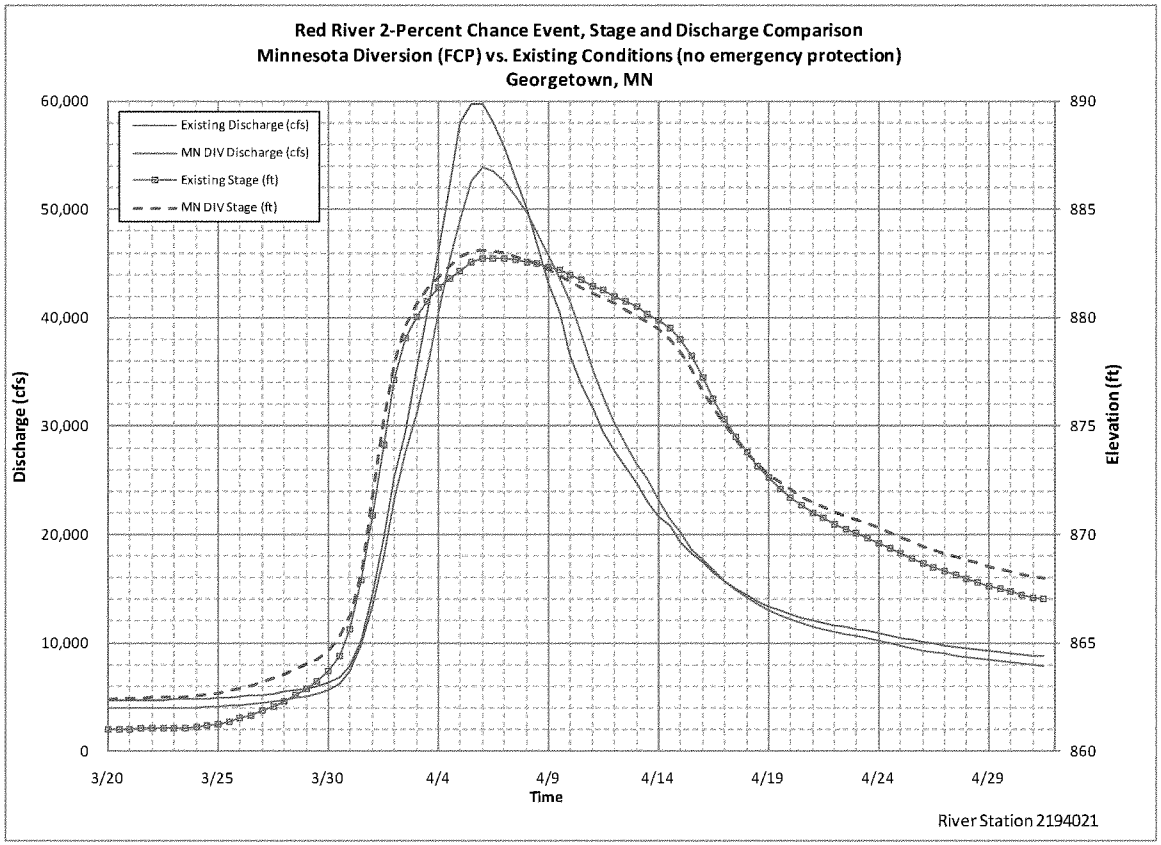

Figure C-E1- 23: 2-Percent Chance Hydrographs for FCP @ Georgetown

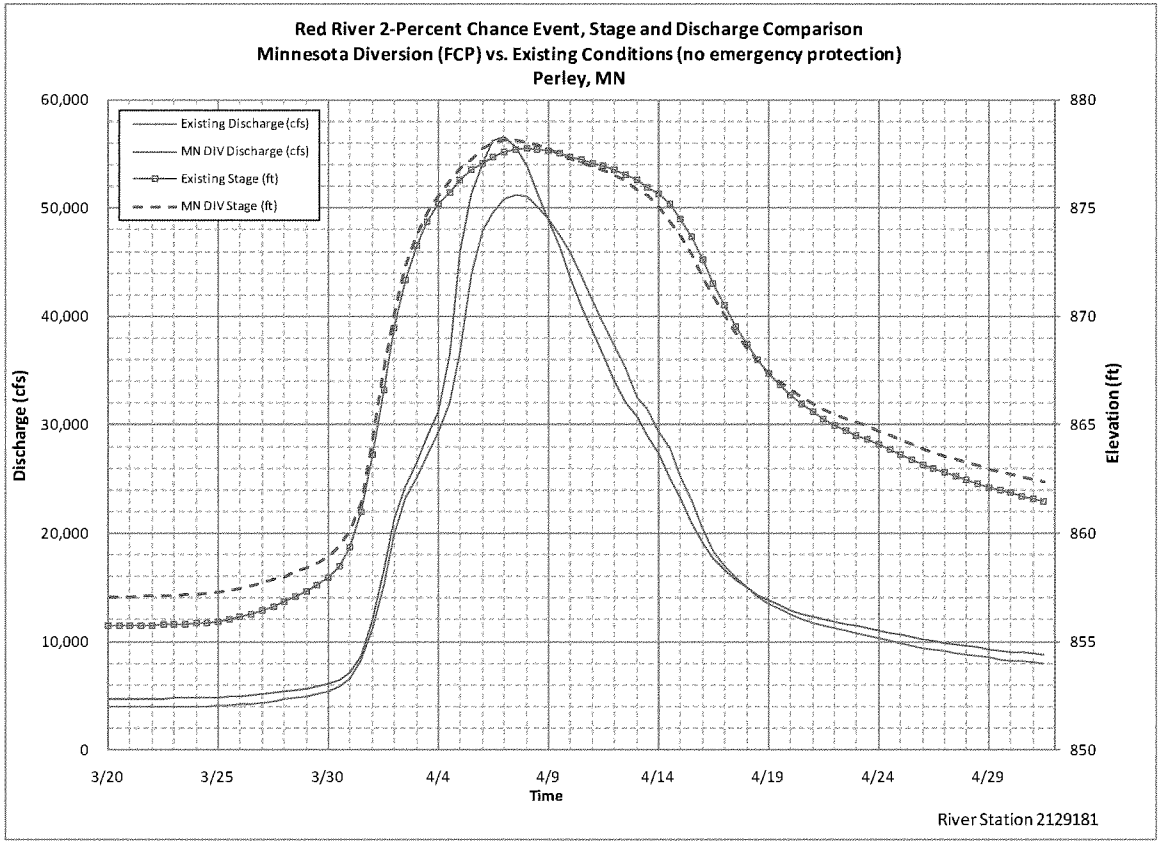

Figure C-E1- 24: 2-Percent Chance Hydrographs for FCP @ Perley

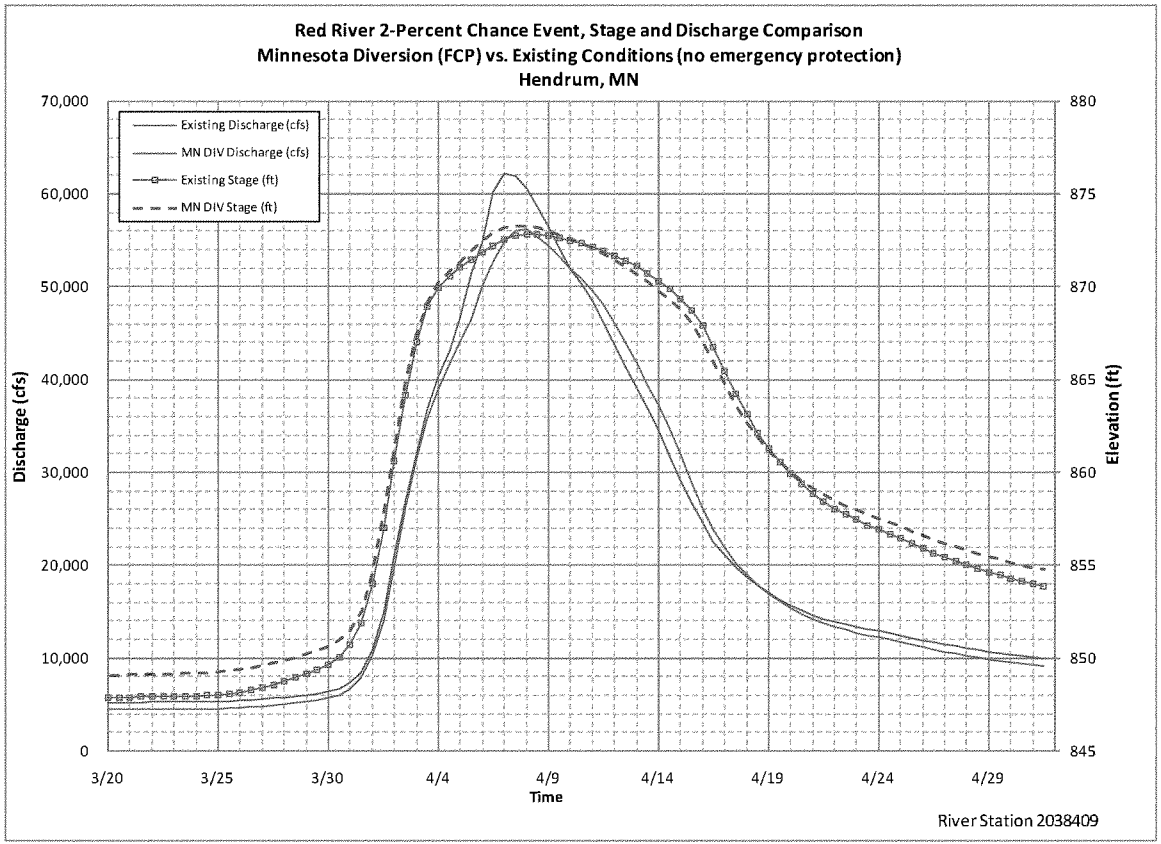

Figure C-E1- 25: 2-Percent Chance Hydrographs for FCP @ Hendrum

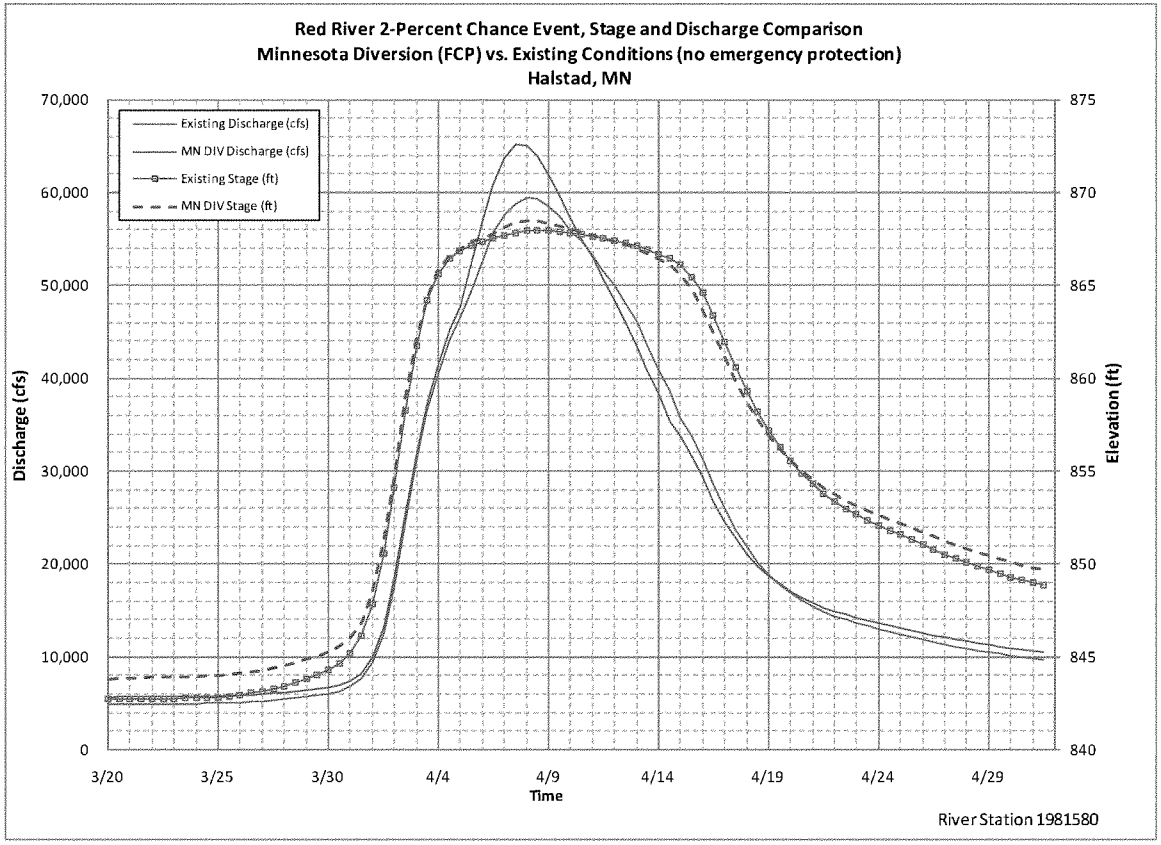

Figure C-E1- 26: 2-Percent Chance Hydrographs for FCP @ Halstad

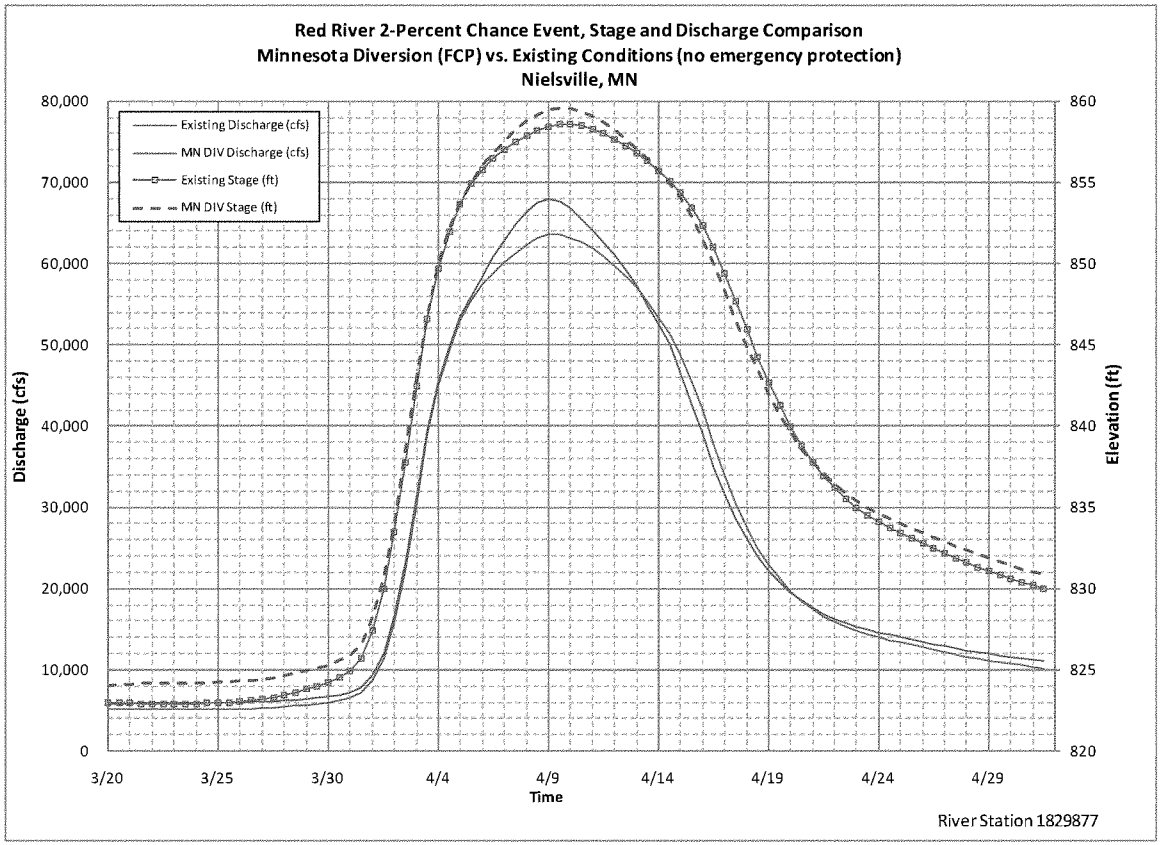

Figure C-E1- 27: 2-Percent Chance Hydrographs for FCP @ Nielsville

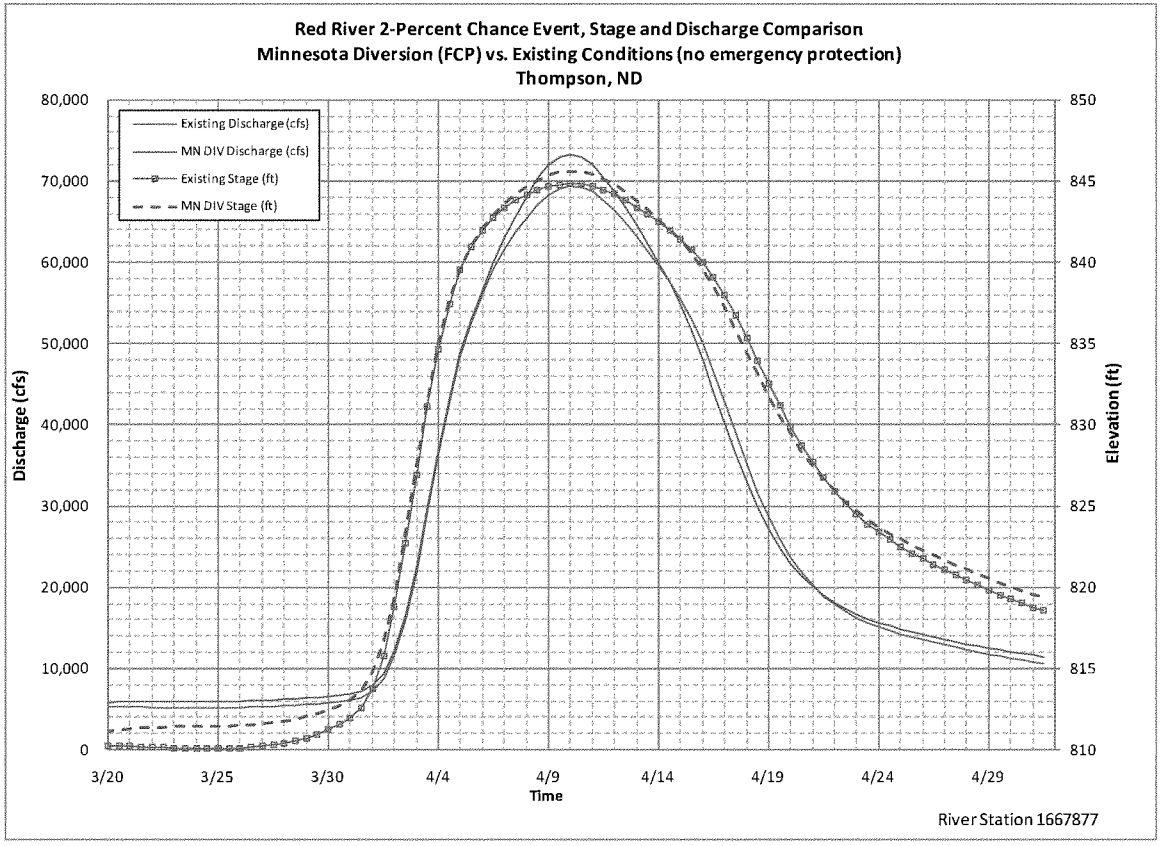

Figure C-E1- 28: 2-Percent Chance Hydrographs for FCP @ Thompson

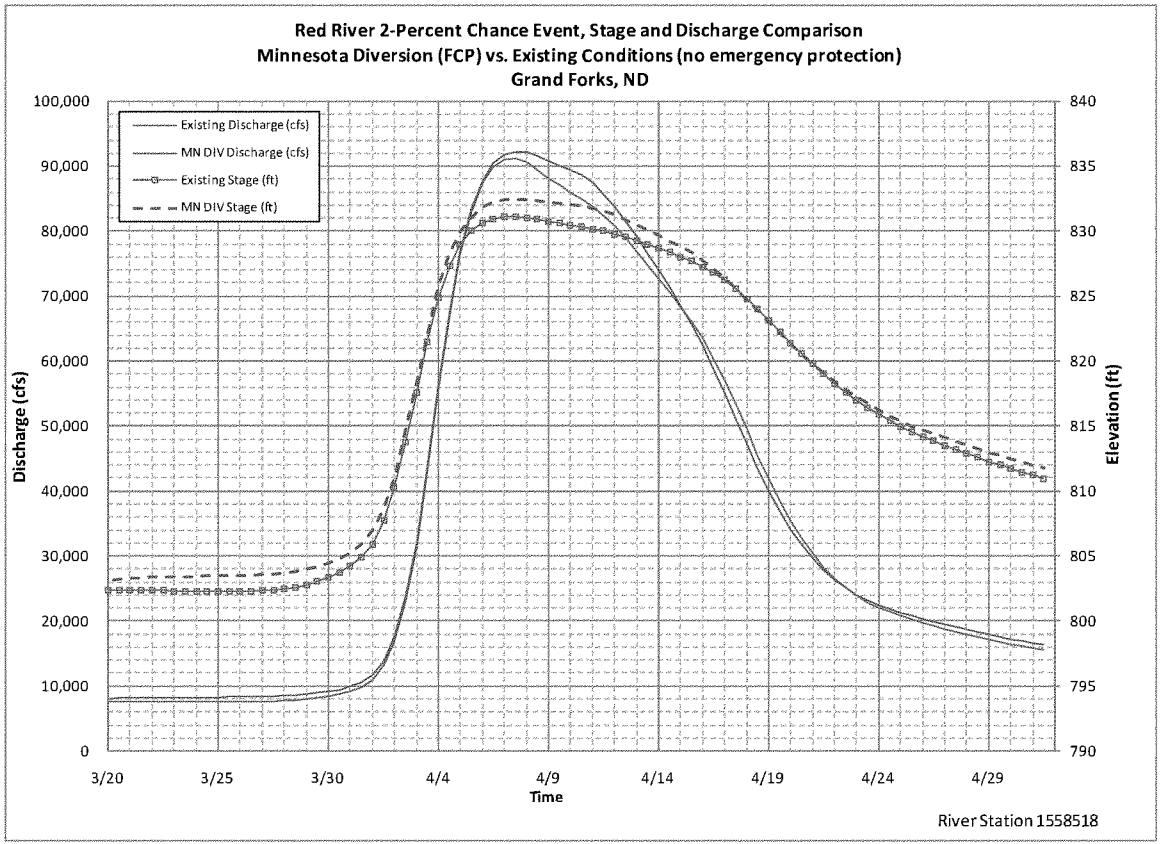

Figure C-E1- 29: 2-Percent Chance Hydrographs for FCP @ Grand Forks

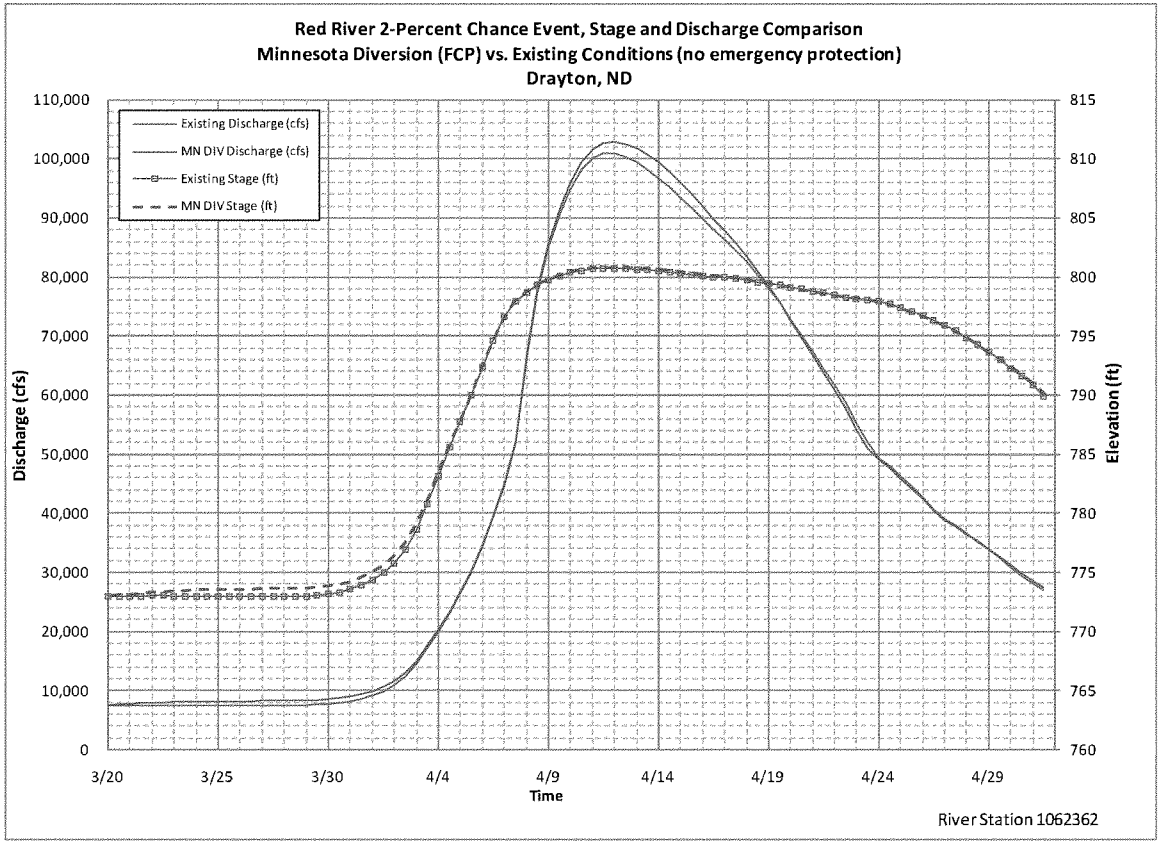

Figure C-E1- 30: 2-Percent Chance Hydrographs for FCP @ Drayton

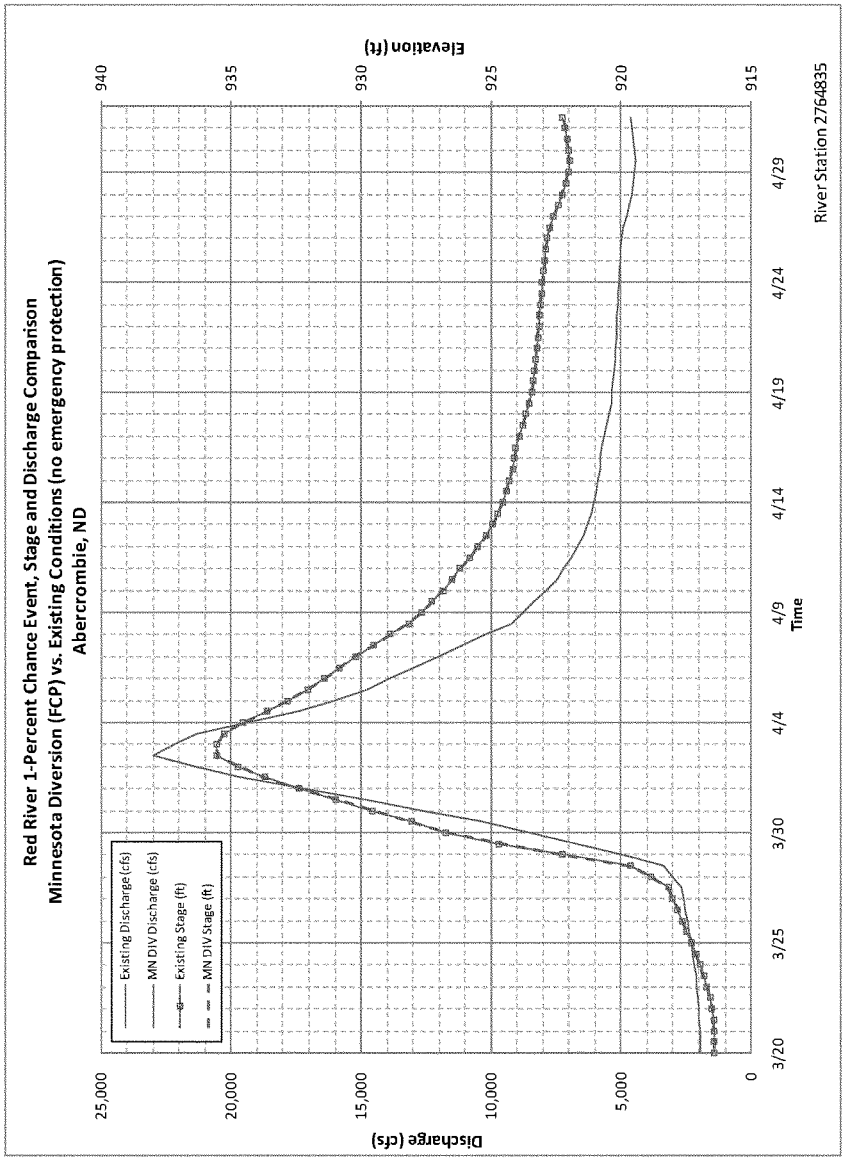

Figure C-E1- 31: 1-Percent Chance Hydrographs for FCP @ Abercrombie

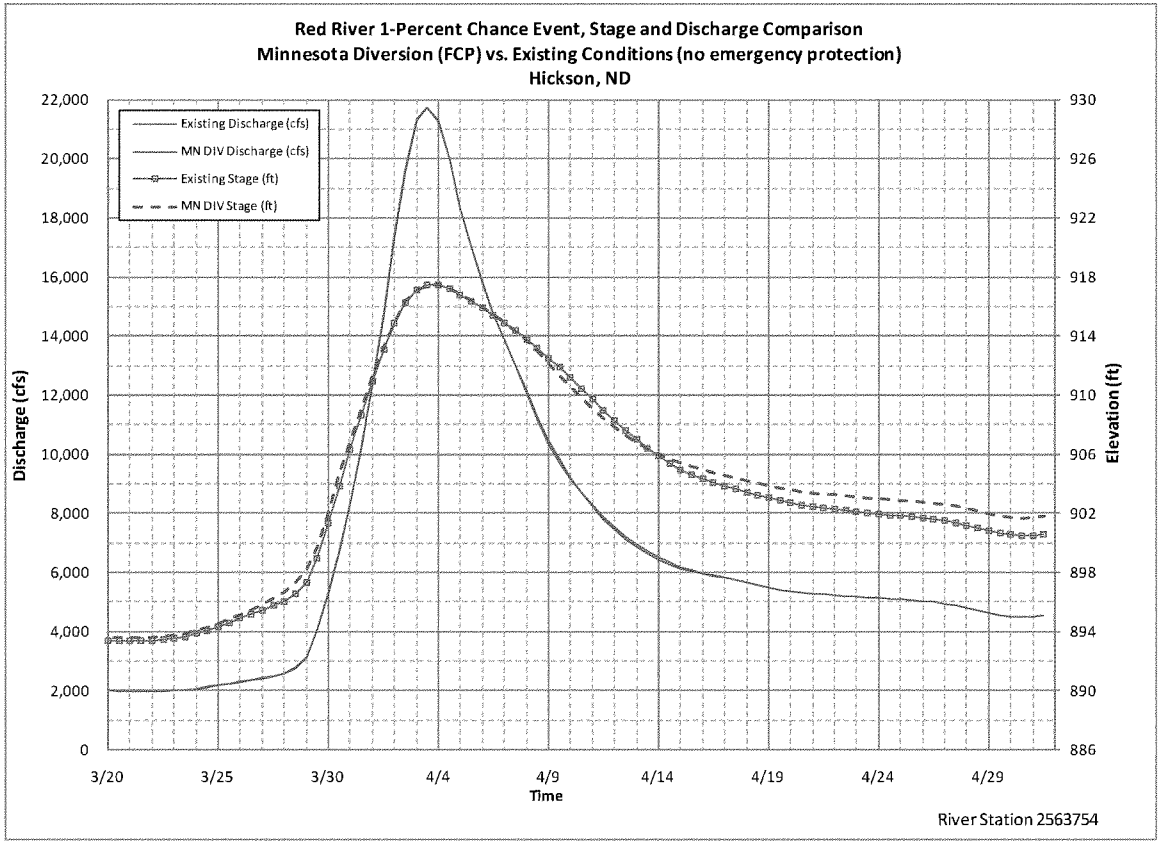

Figure C-E1- 32: 1-Percent Chance Hydrographs for FCP @ Hickson

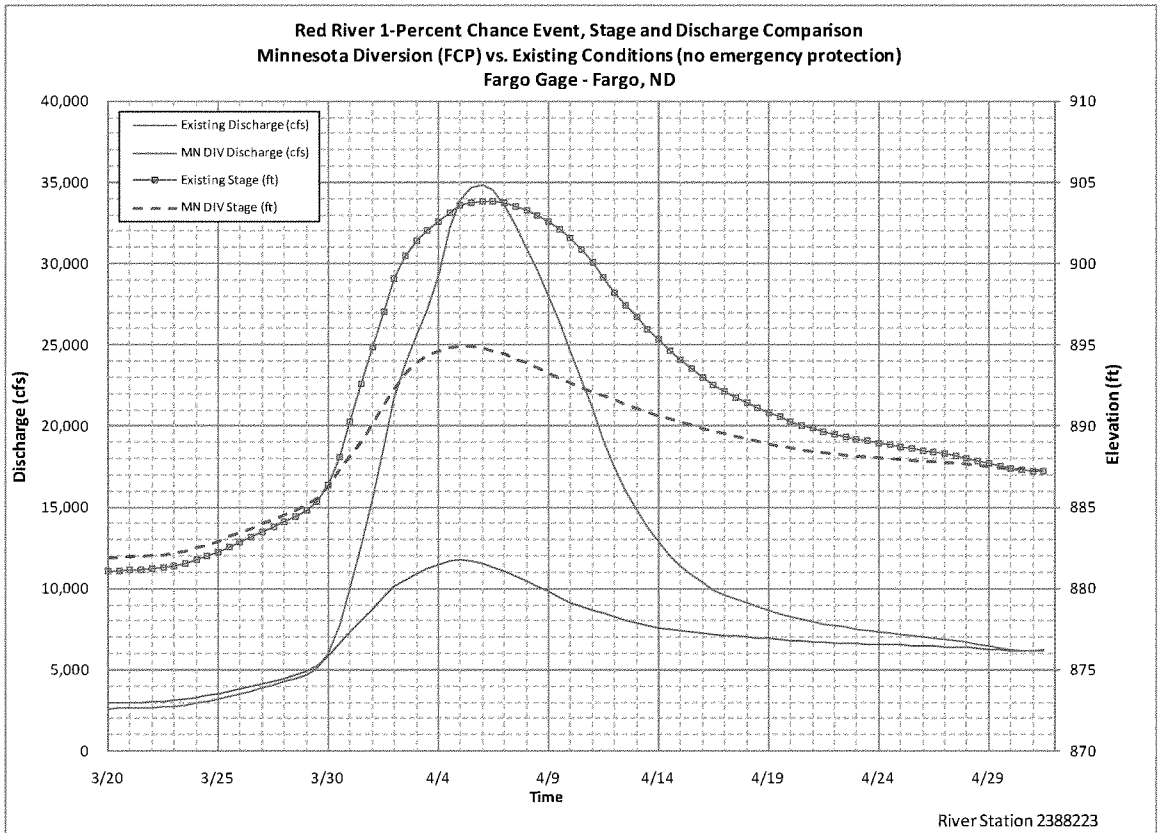

Figure C-E1- 33: 1-Percent Chance Hydrographs for FCP @ Fargo

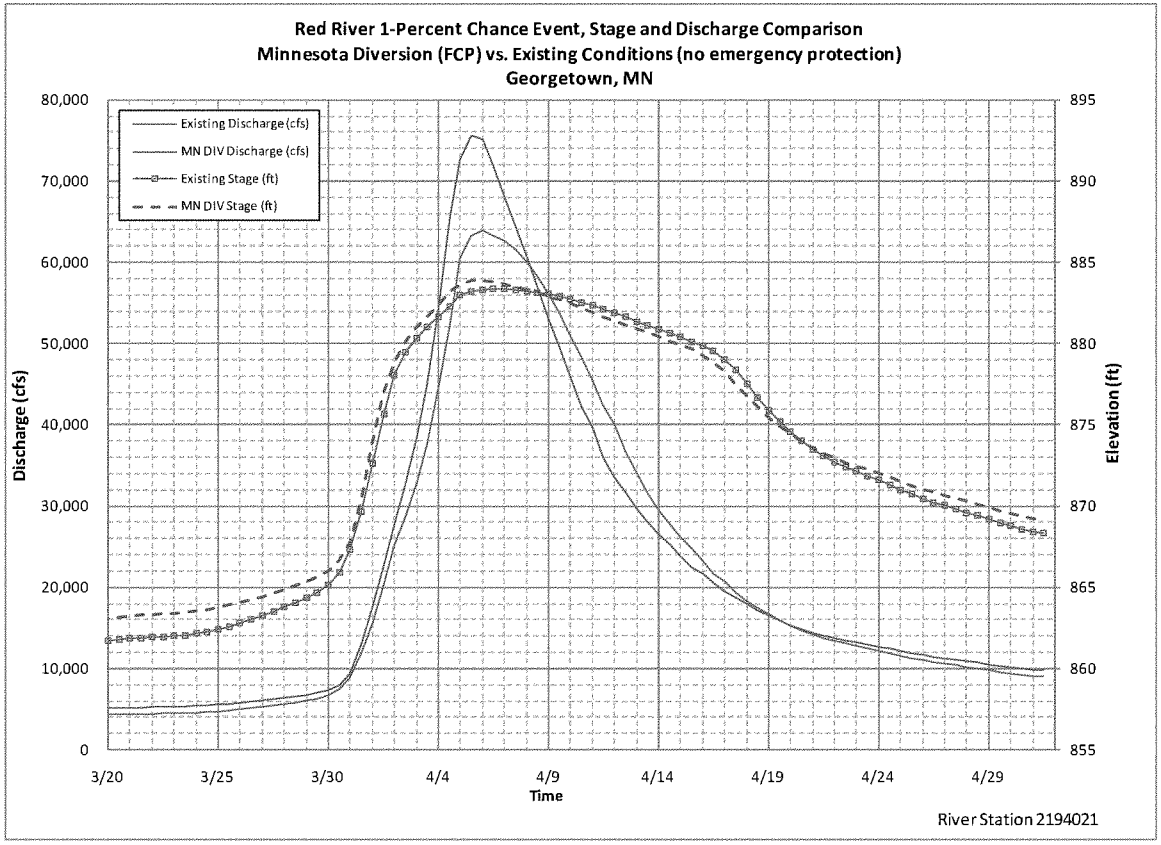

Figure C-E1- 34: 1-Percent Chance Hydrographs for FCP @ Georgetown

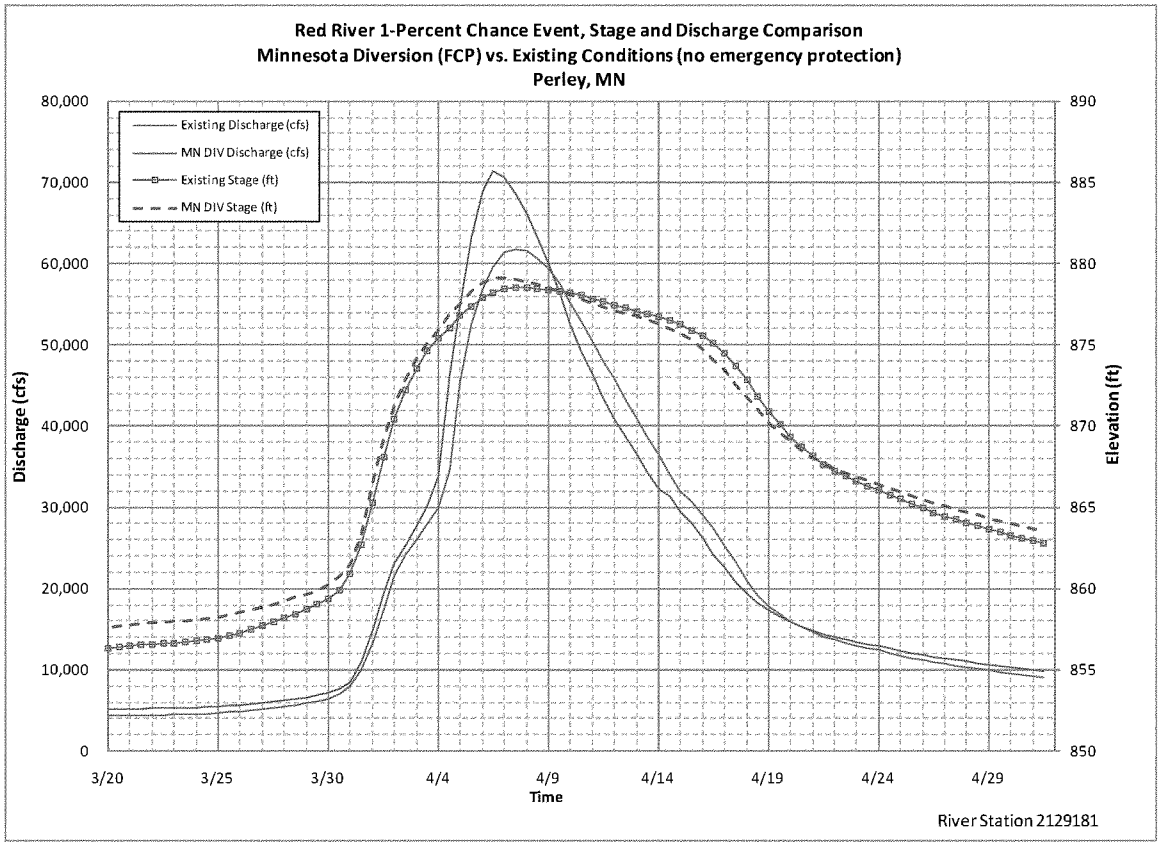

Figure C-E1- 35: 1-Percent Chance Hydrographs for FCP @ Perley

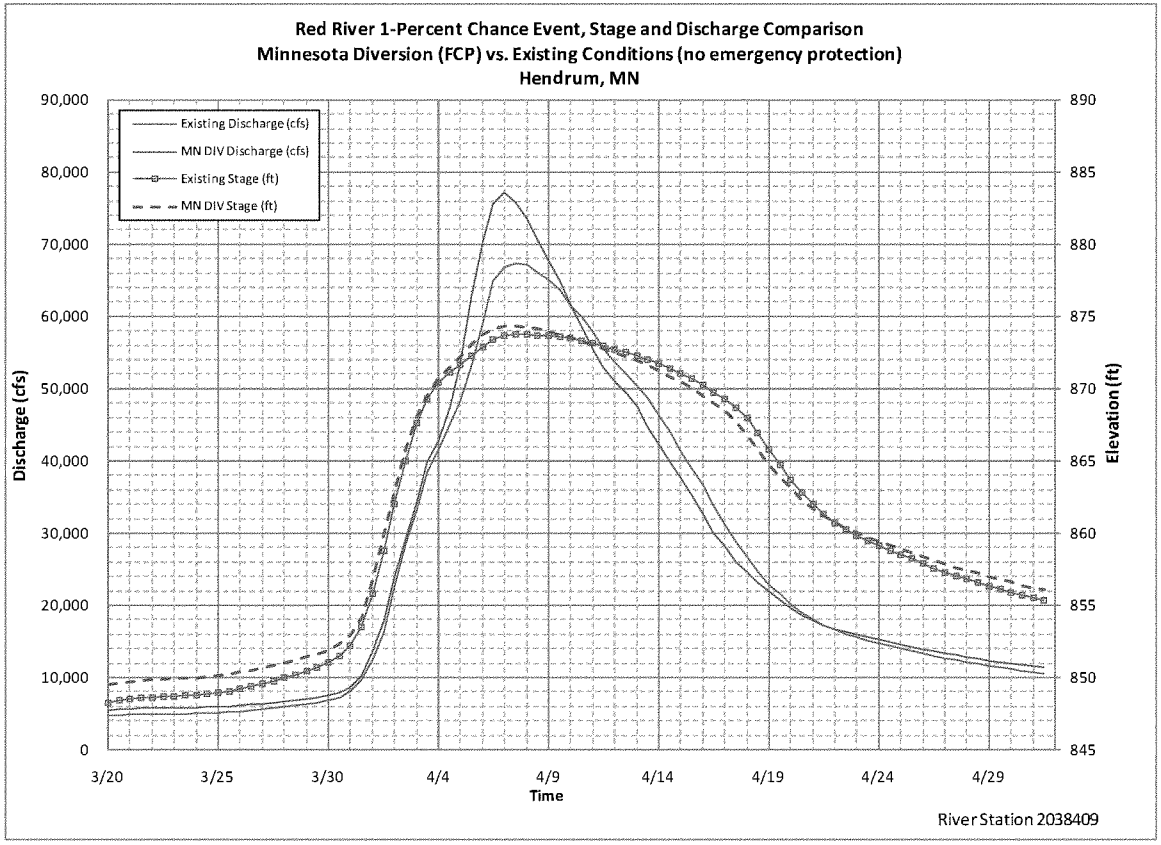

Figure C-E1- 36: 1-Percent Chance Hydrographs for FCP @ Hendrum

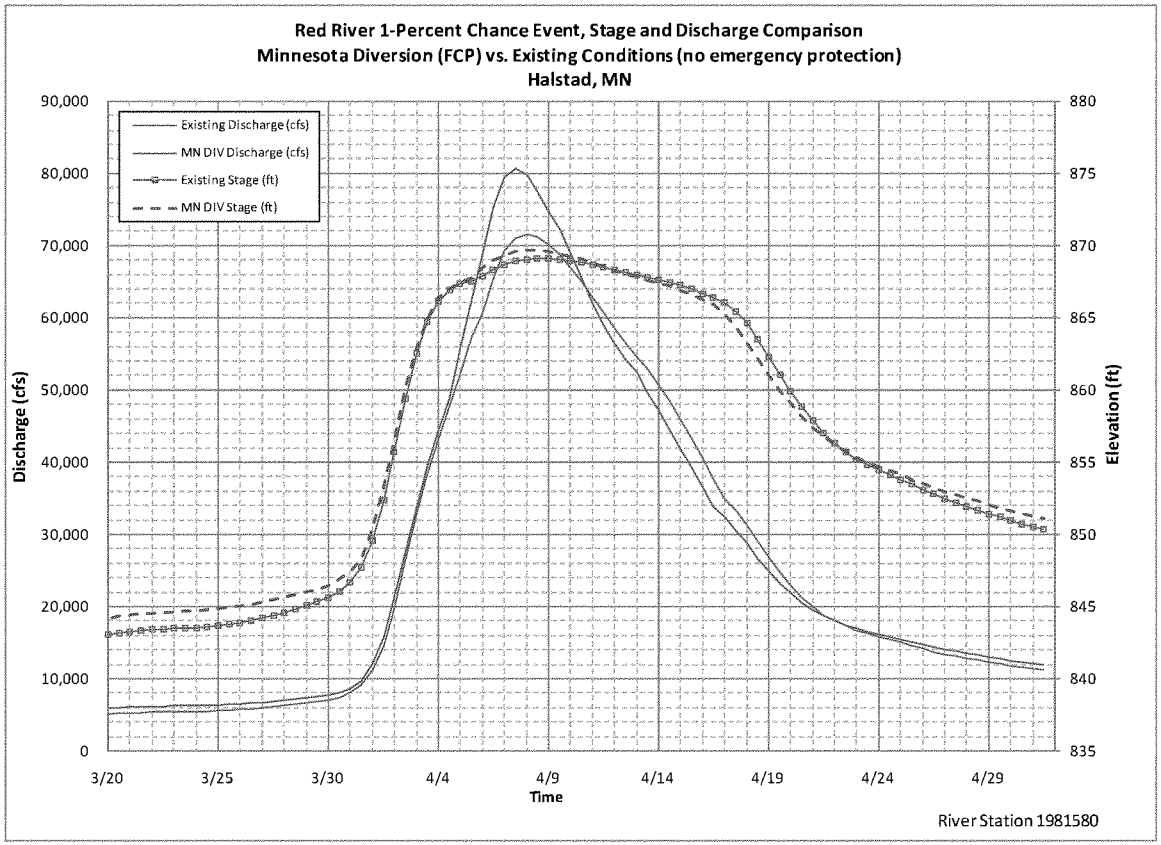

Figure C-E1- 37: 1-Percent Chance Hydrographs for FCP @ Halstad

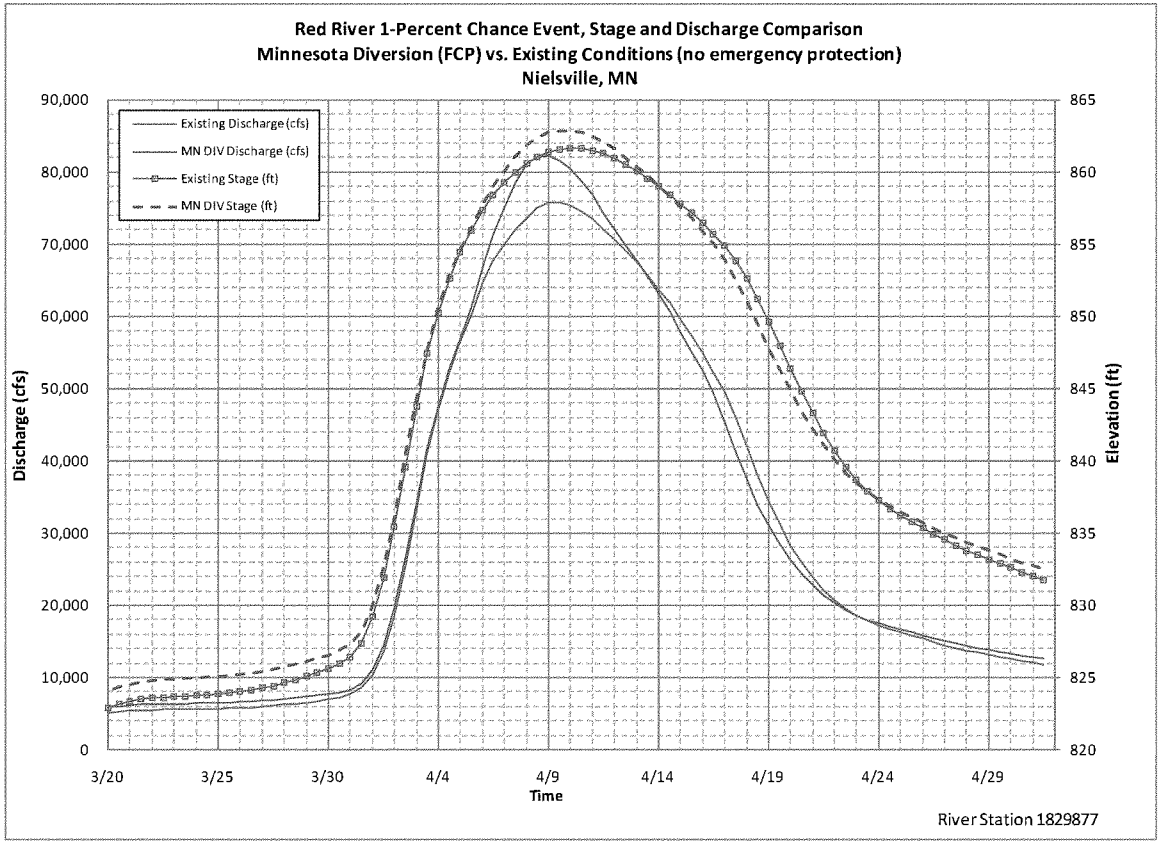

Figure C-E1- 38: 1-Percent Chance Hydrographs for FCP @ Nielsville

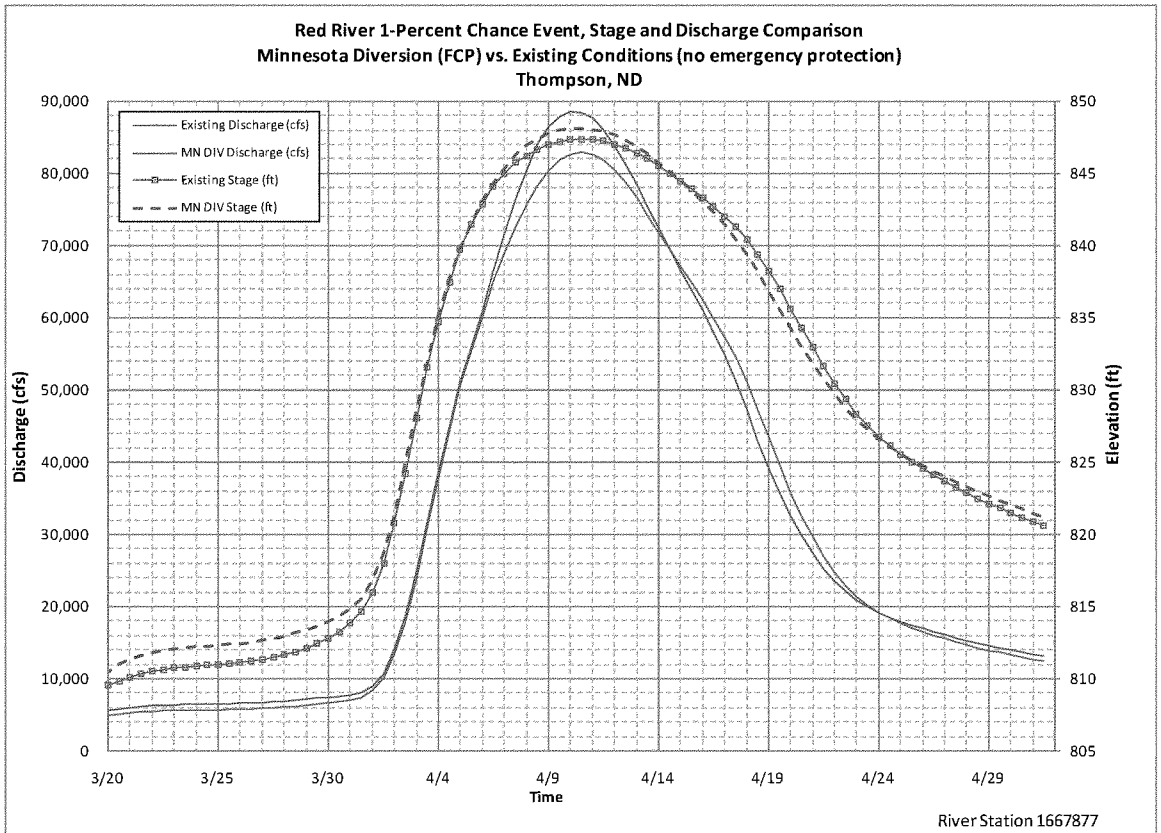

Figure C-E1- 39: 1-Percent Chance Hydrographs for FCP @ Thompson

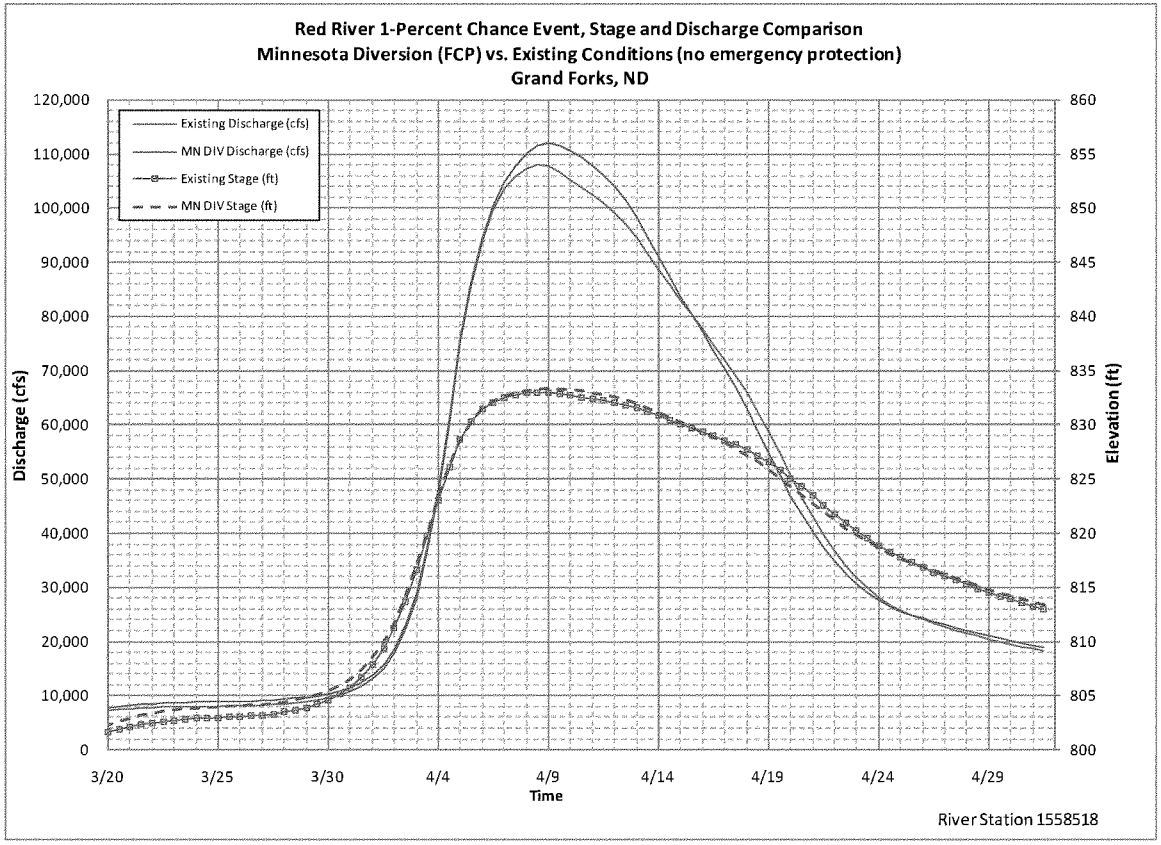

Figure C-E1- 40: 1-Percent Chance Hydrographs for FCP @ Grand Forks

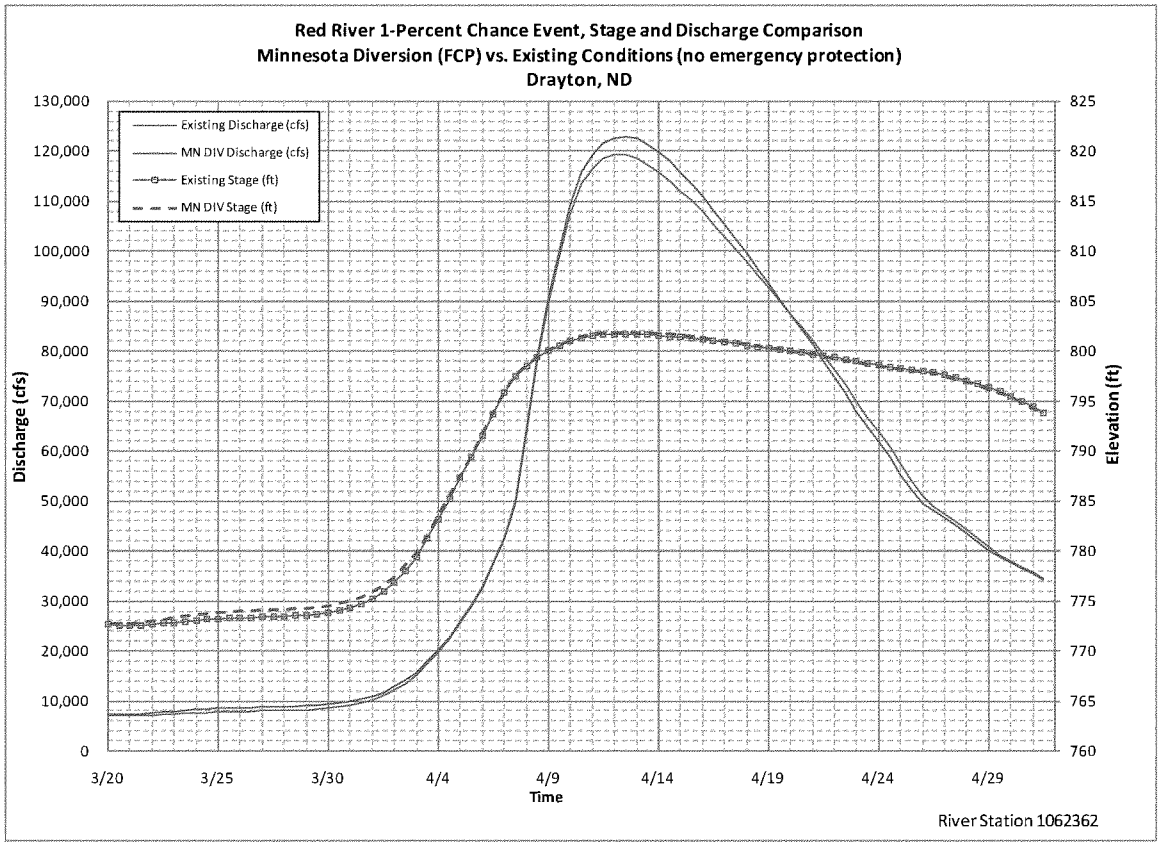

Figure C-E1- 41: 1-Percent Chance Hydrographs for FCP @ Drayton

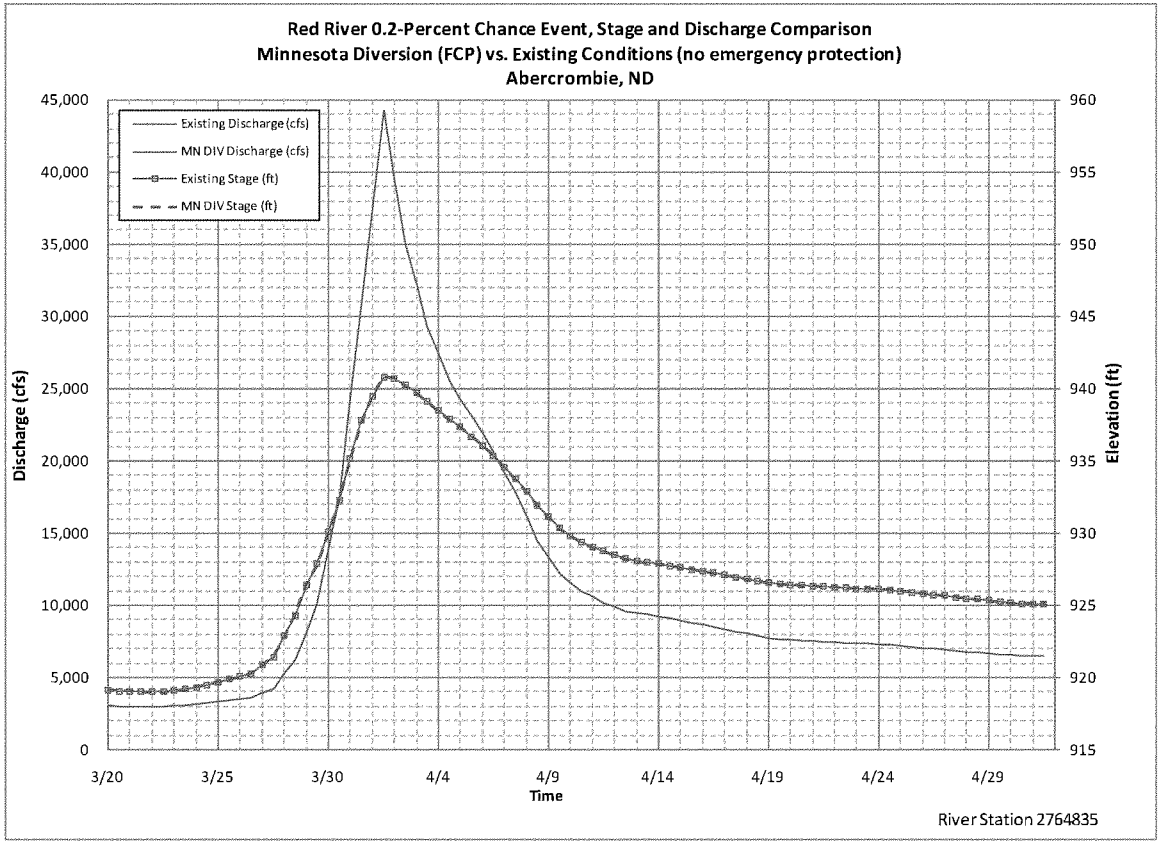

Figure C-E1- 42: 0.2-Percent Chance Hydrographs for FCP @ Abercrombie

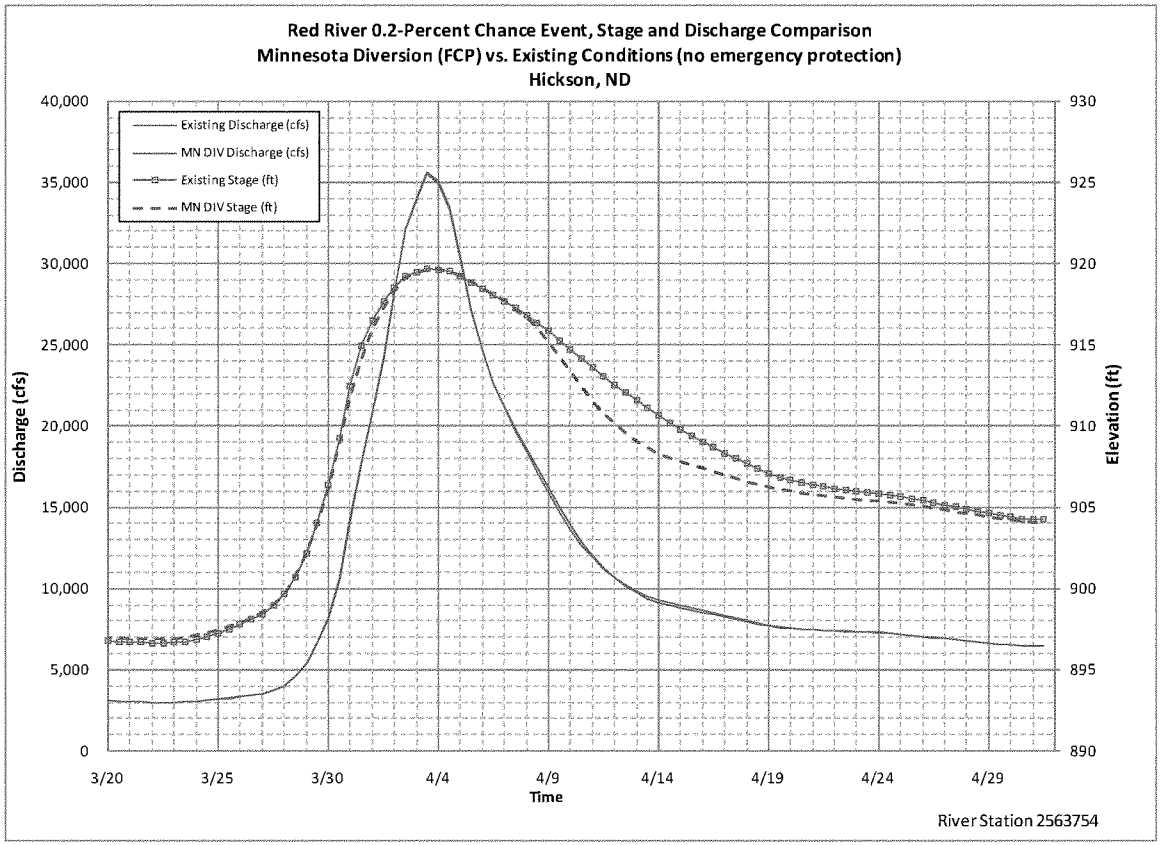

Figure C-E1- 43: 0.2-Percent Chance Hydrographs for FCP @ Hickson

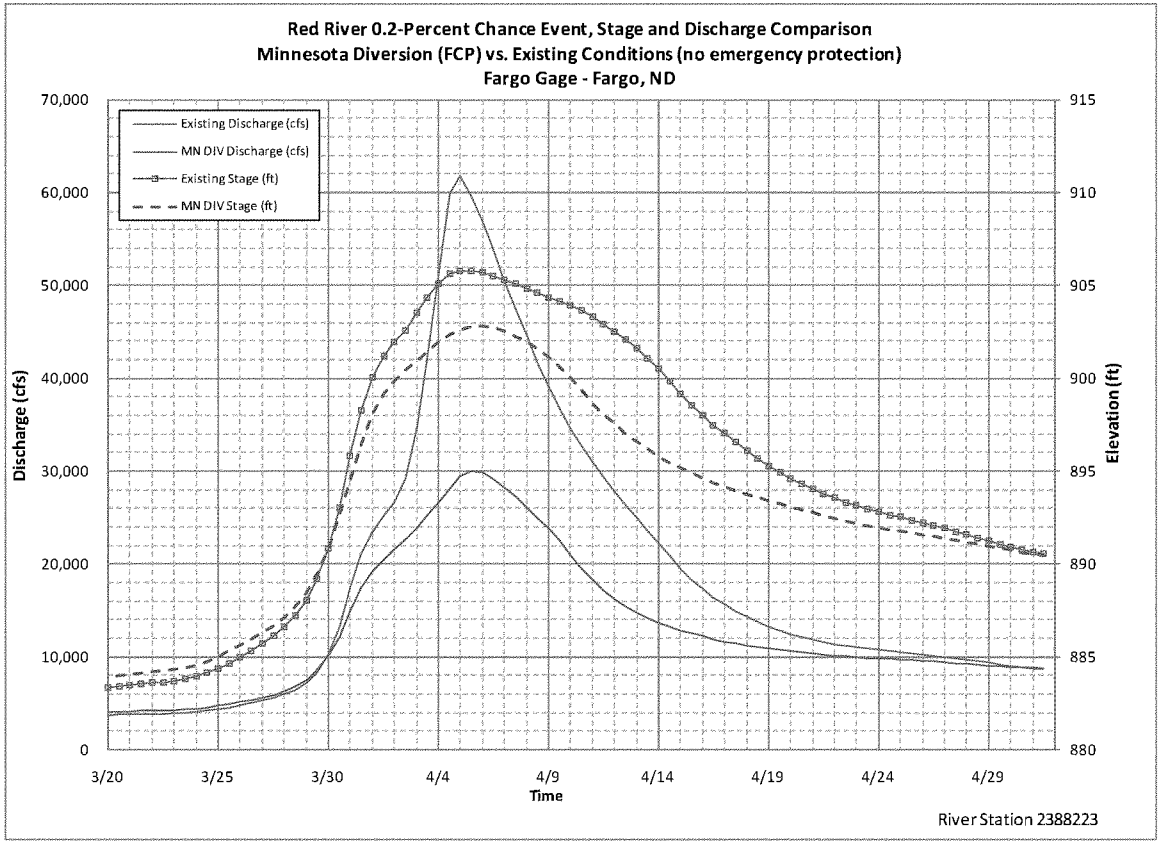

Figure C-E1- 44: 0.2-Percent Chance Hydrographs for FCP @ Fargo

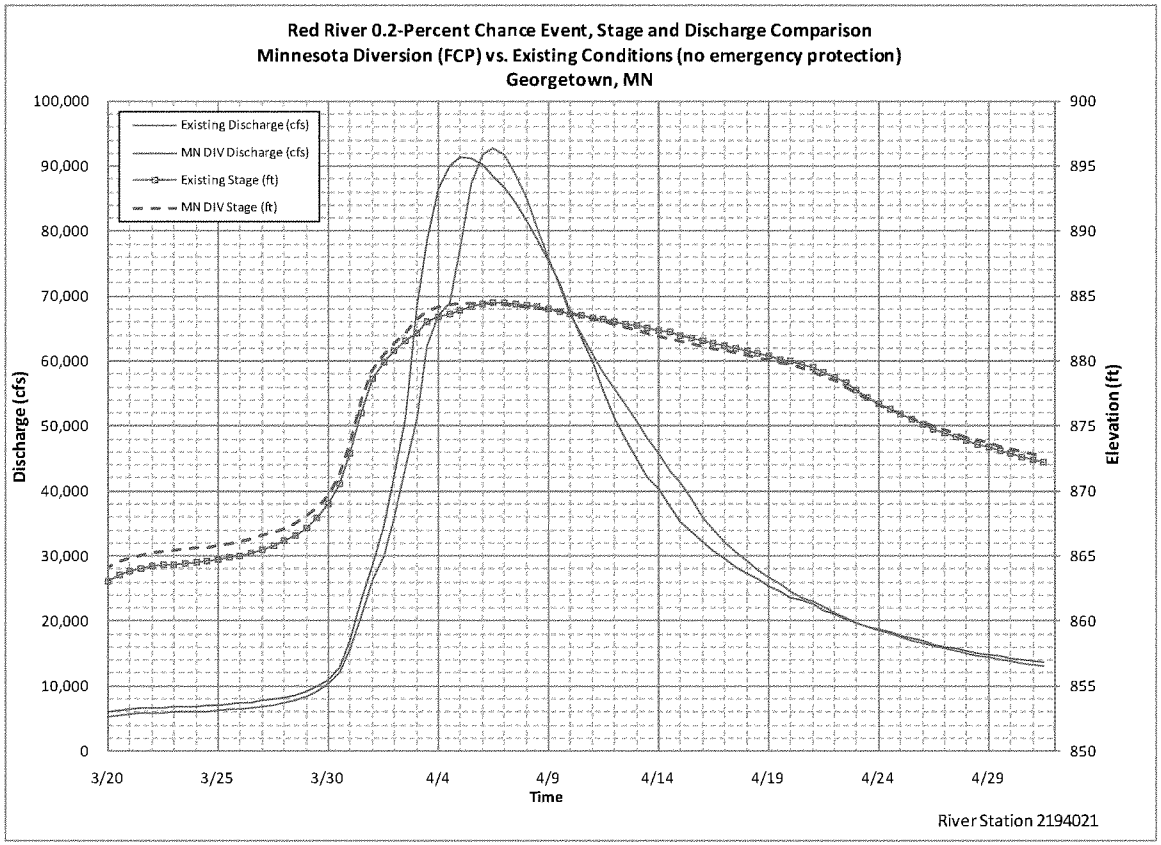

Figure C-E1- 45: 0.2-Percent Chance Hydrographs for FCP @ Georgetown

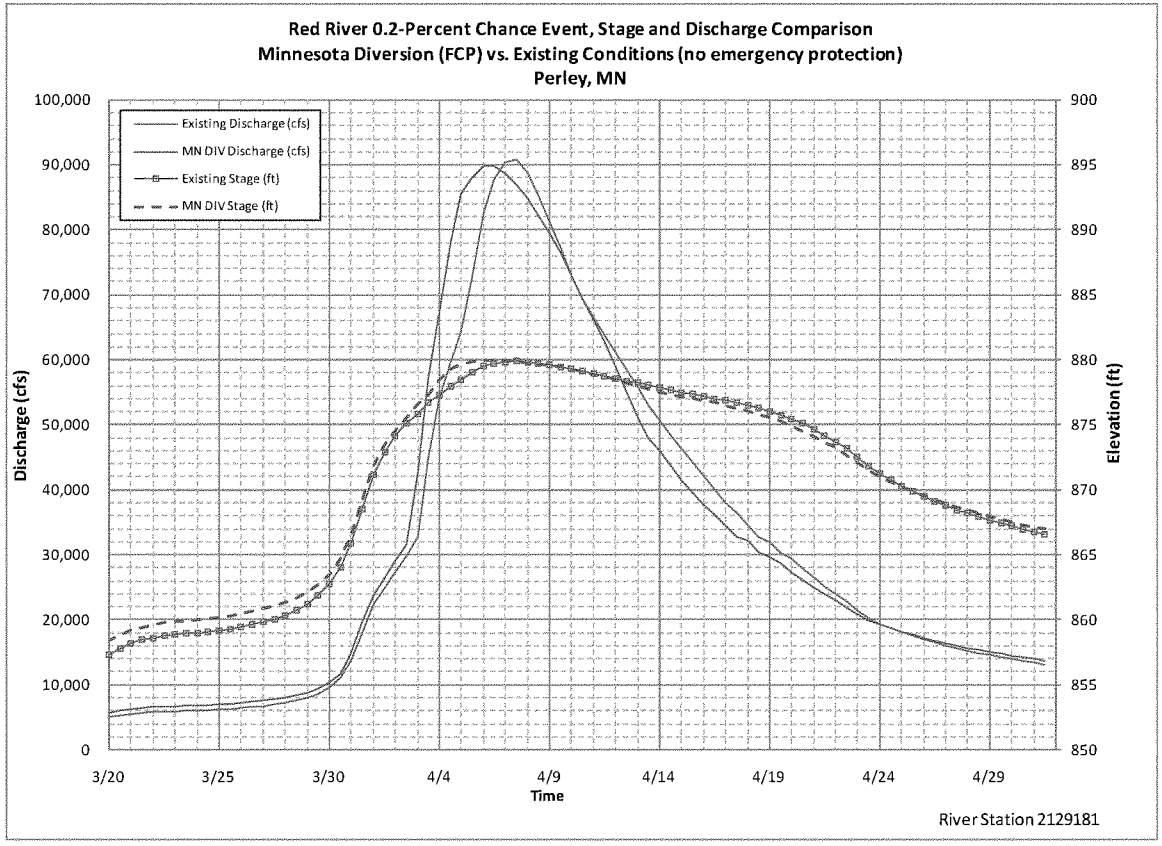

Figure C-E1- 46: 0.2-Percent Chance Hydrographs for FCP @ Perley

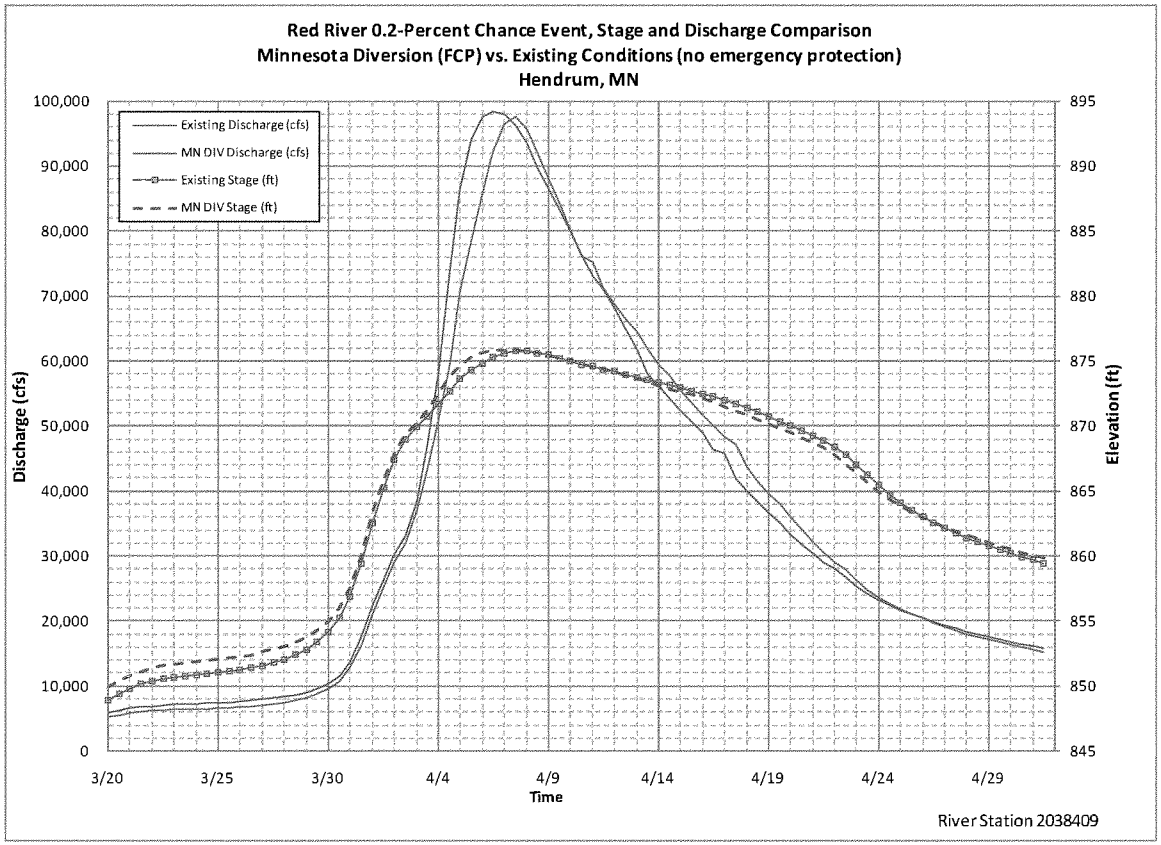

Figure C-E1- 47: 0.2-Percent Chance Hydrographs for FCP @ Hendrum

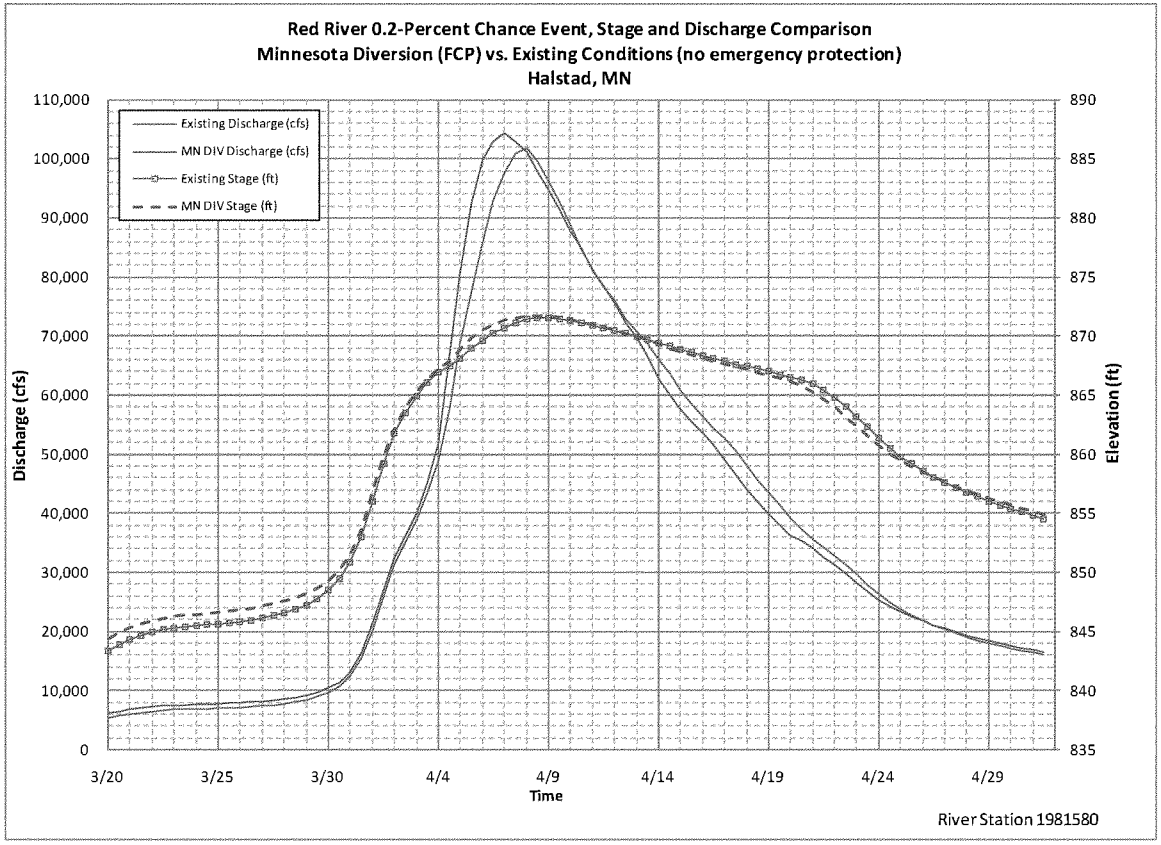

Figure C-E1- 48: 0.2-Percent Chance Hydrographs for FCP @ Halstad

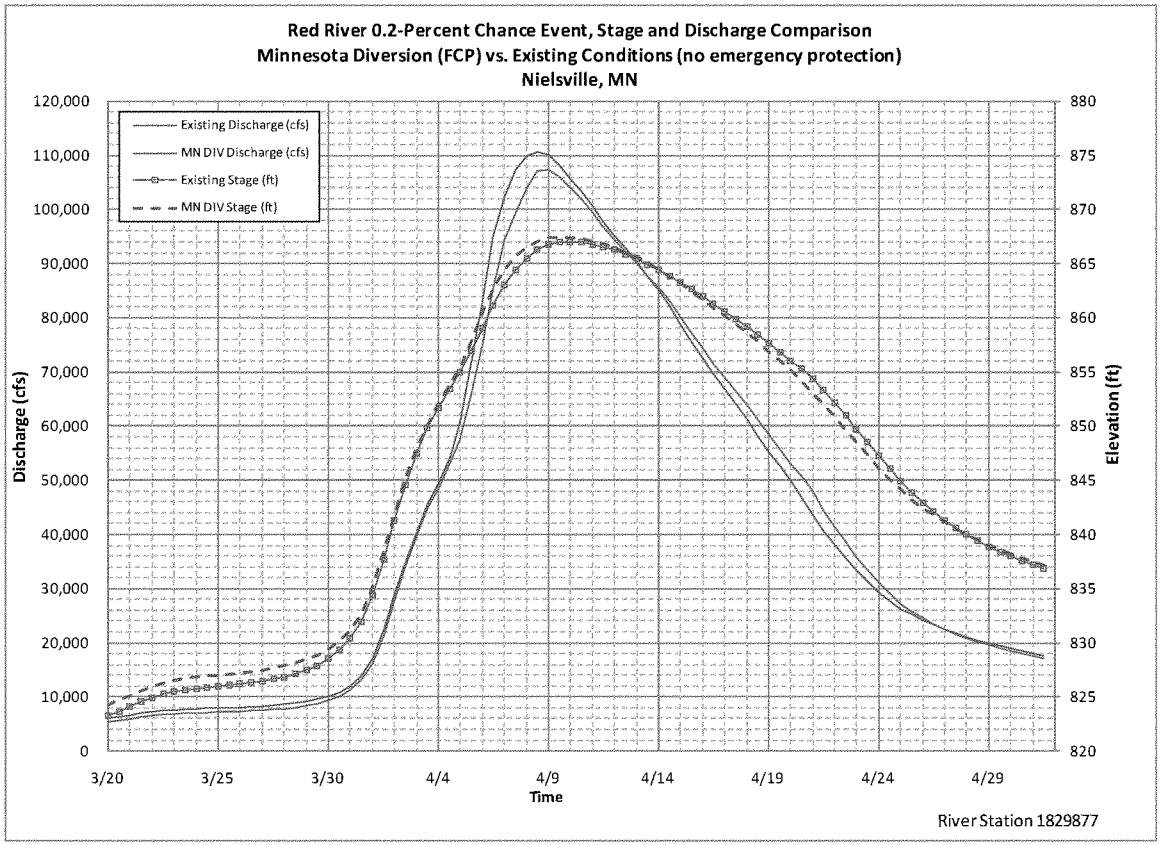

Figure C-E1- 49: 0.2-Percent Chance Hydrographs for FCP @ Nielsville

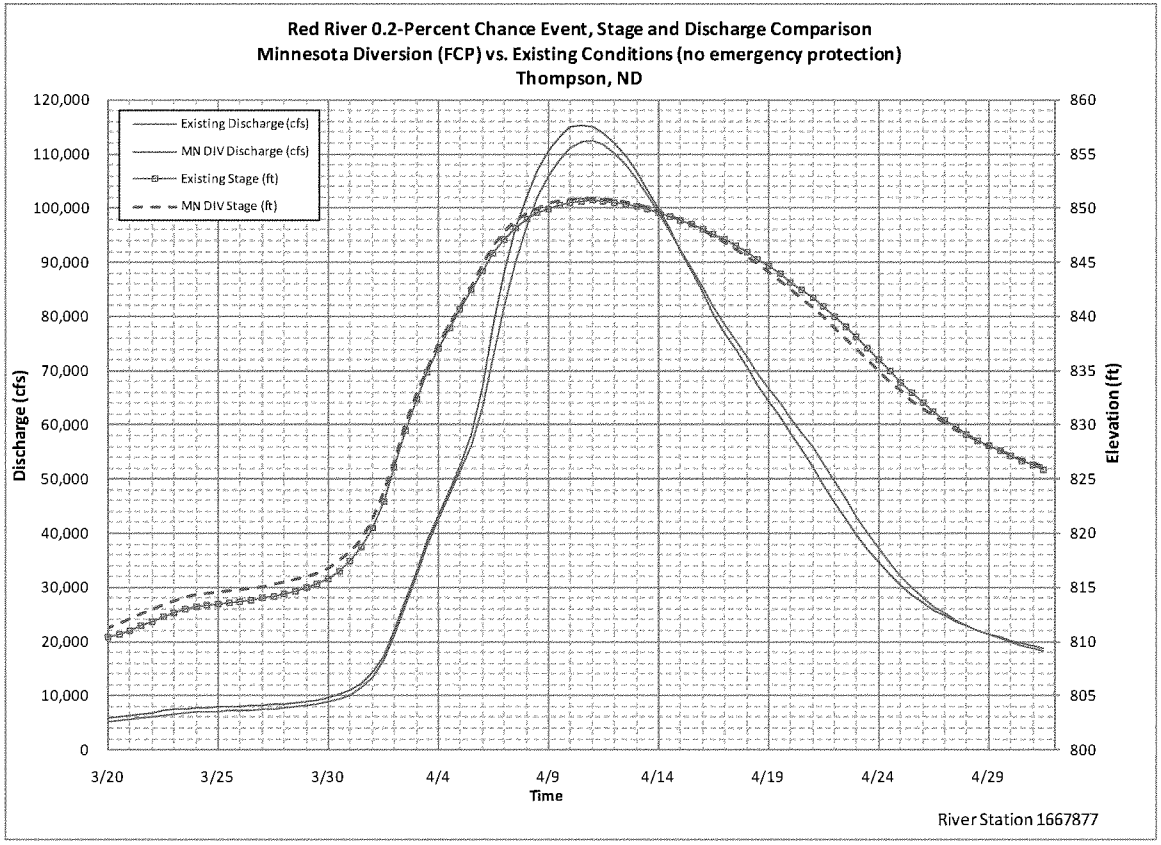

Figure C-E1- 50: 0.2-Percent Chance Hydrographs for FCP @ Thompson

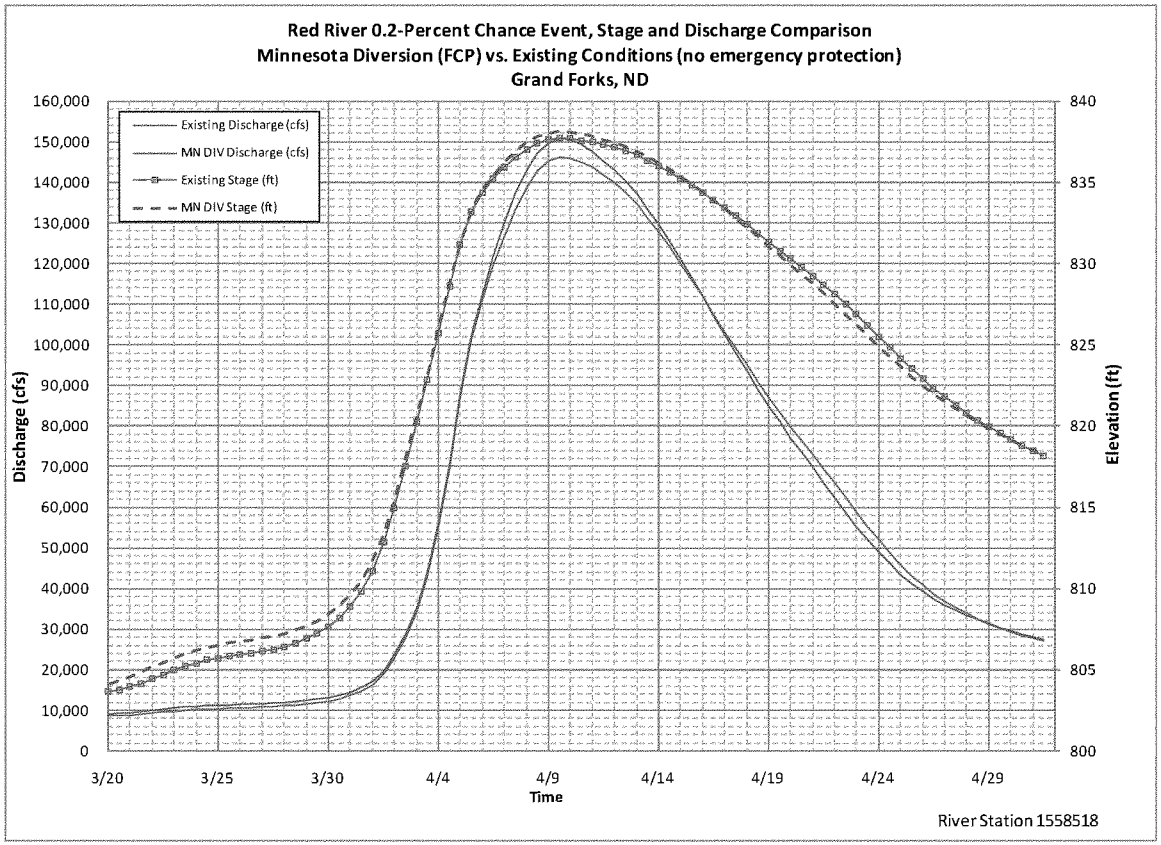

Figure C-E1- 51: 0.2-Percent Chance Hydrographs for FCP @ Grand Forks

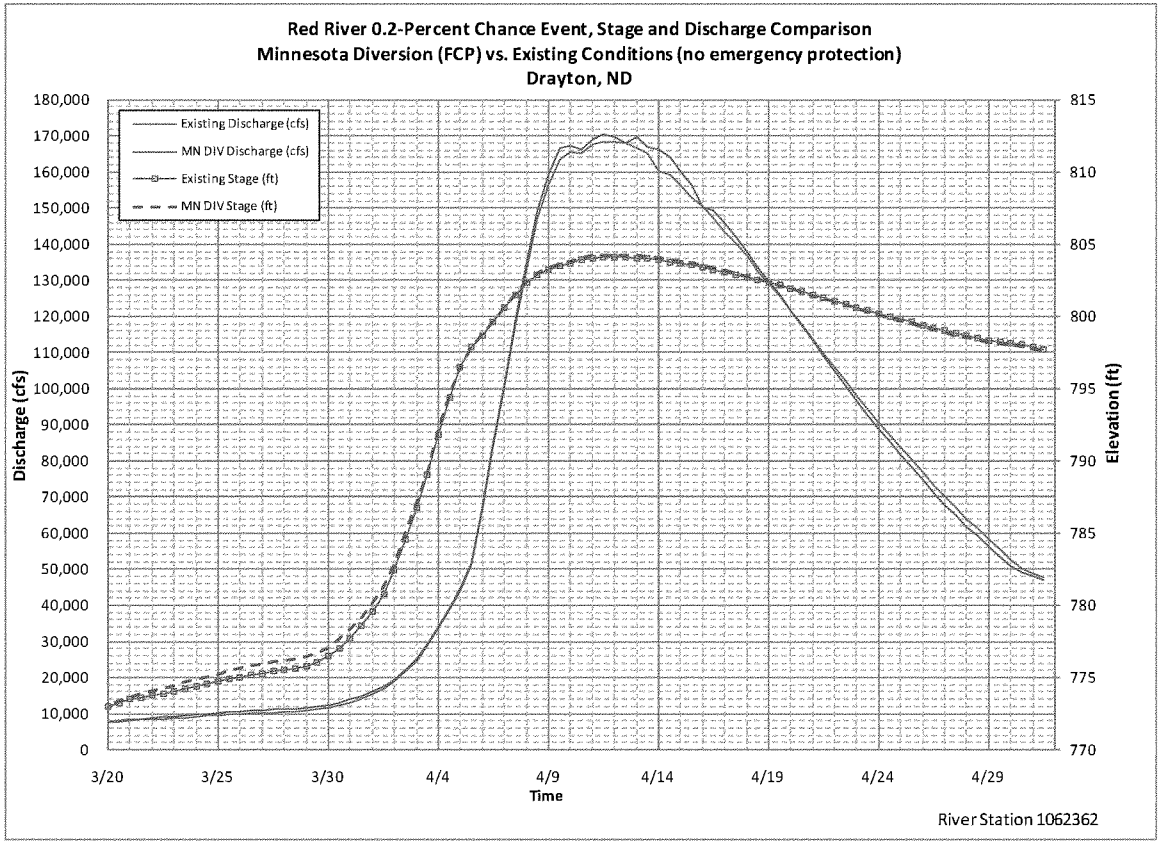

Figure C-E1- 52: 0.2-Percent Chance Hydrographs for FCP @ Drayton

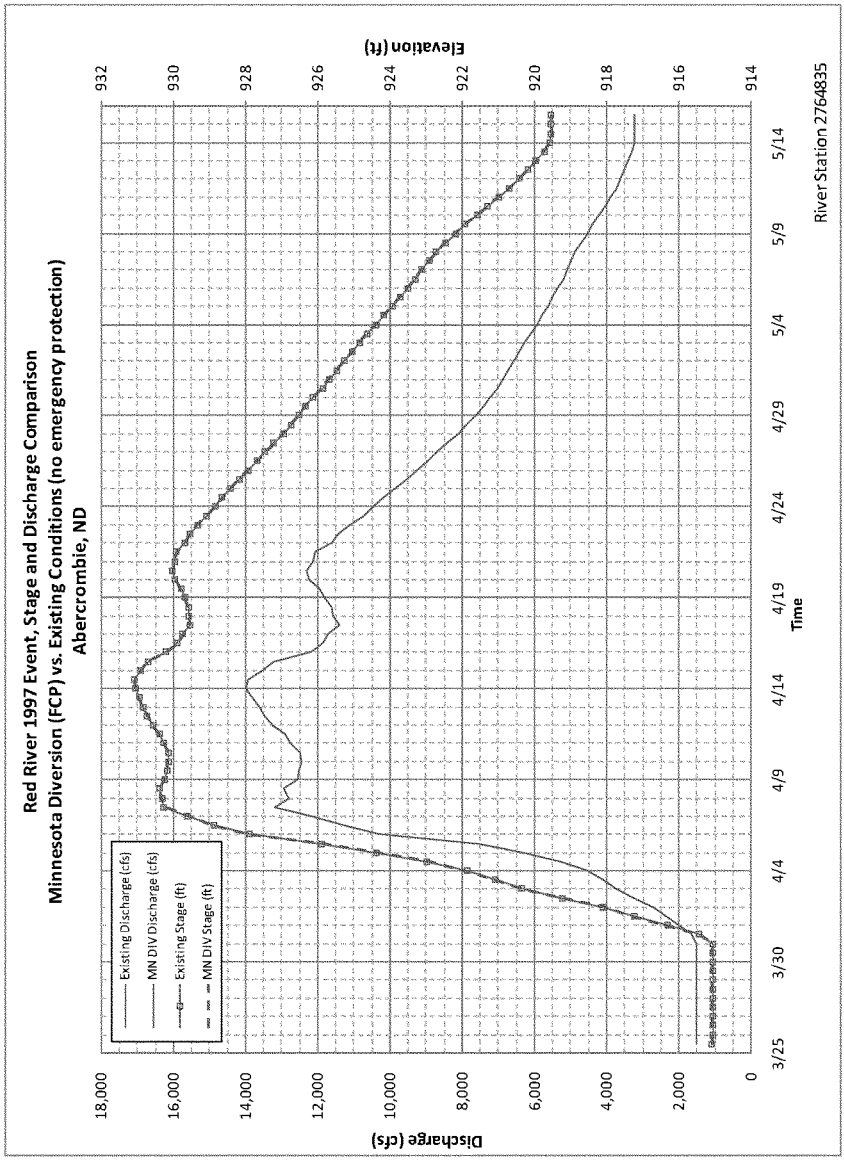

Figure C-E1- 53: 1997 Historical Flood Hydrographs for FCP @ Abercrombie

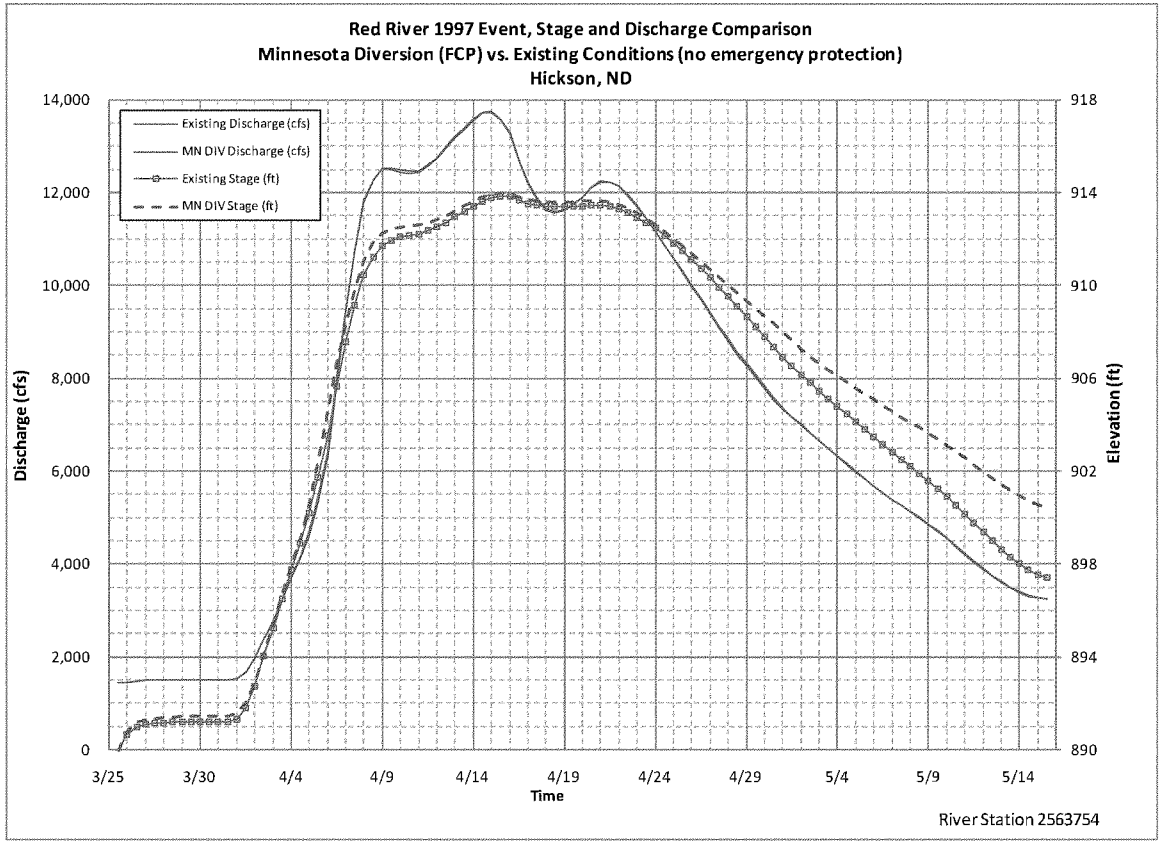

Figure C-E1- 54: 1997 Historical Flood Hydrographs for FCP @ Hickson

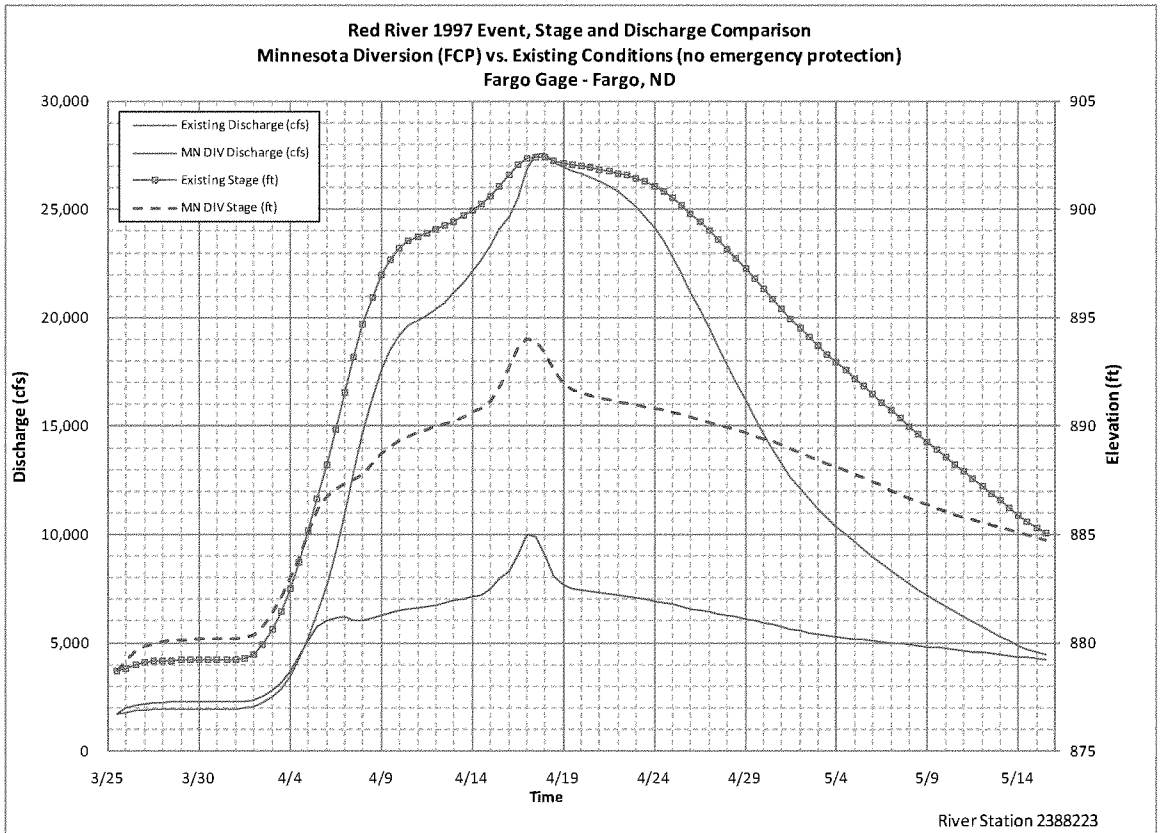

Figure C-E1- 55: 1997 Historical Flood Hydrographs for FCP @ Fargo

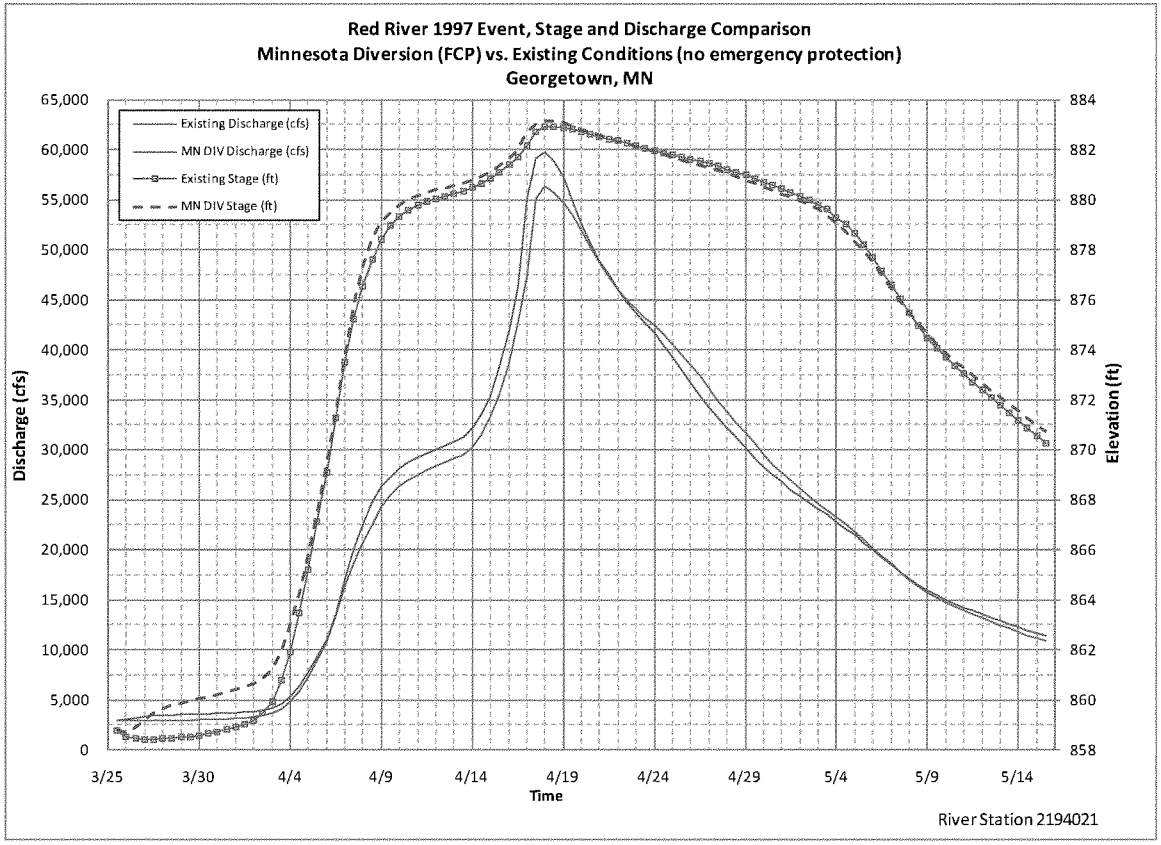

Figure C-E1- 56: 1997 Historical Flood Hydrographs for FCP @ Georgetown

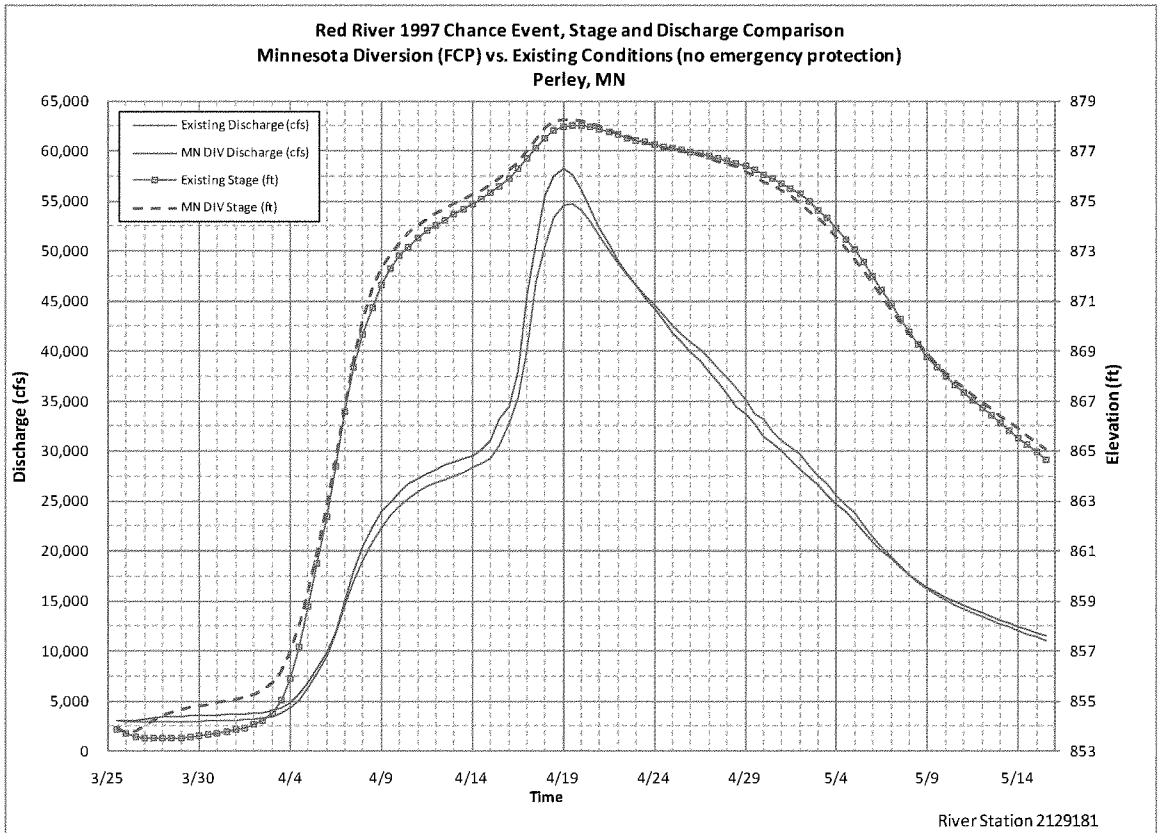

Figure C-E1- 57: 1997 Historical Flood Hydrographs for FCP @ Perley

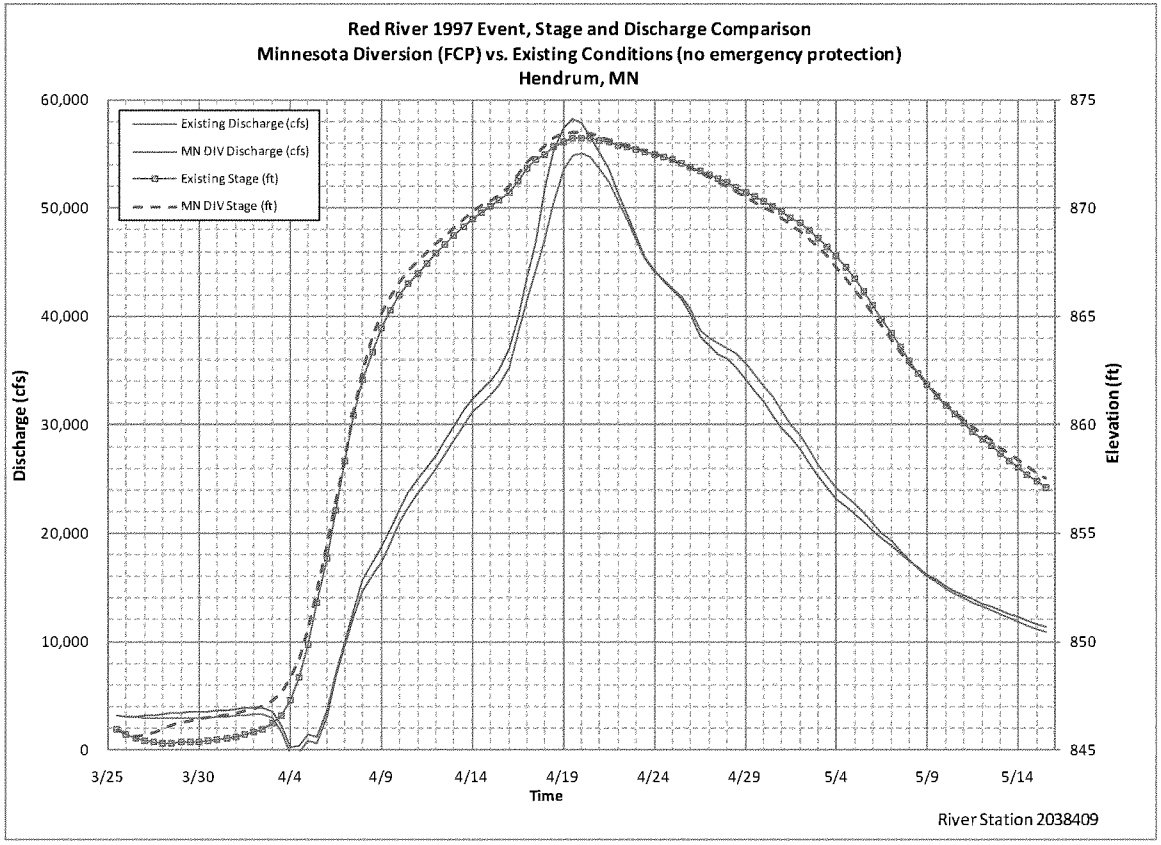

Figure C-E1- 58: 1997 Historical Flood Hydrographs for FCP @ Hendrum

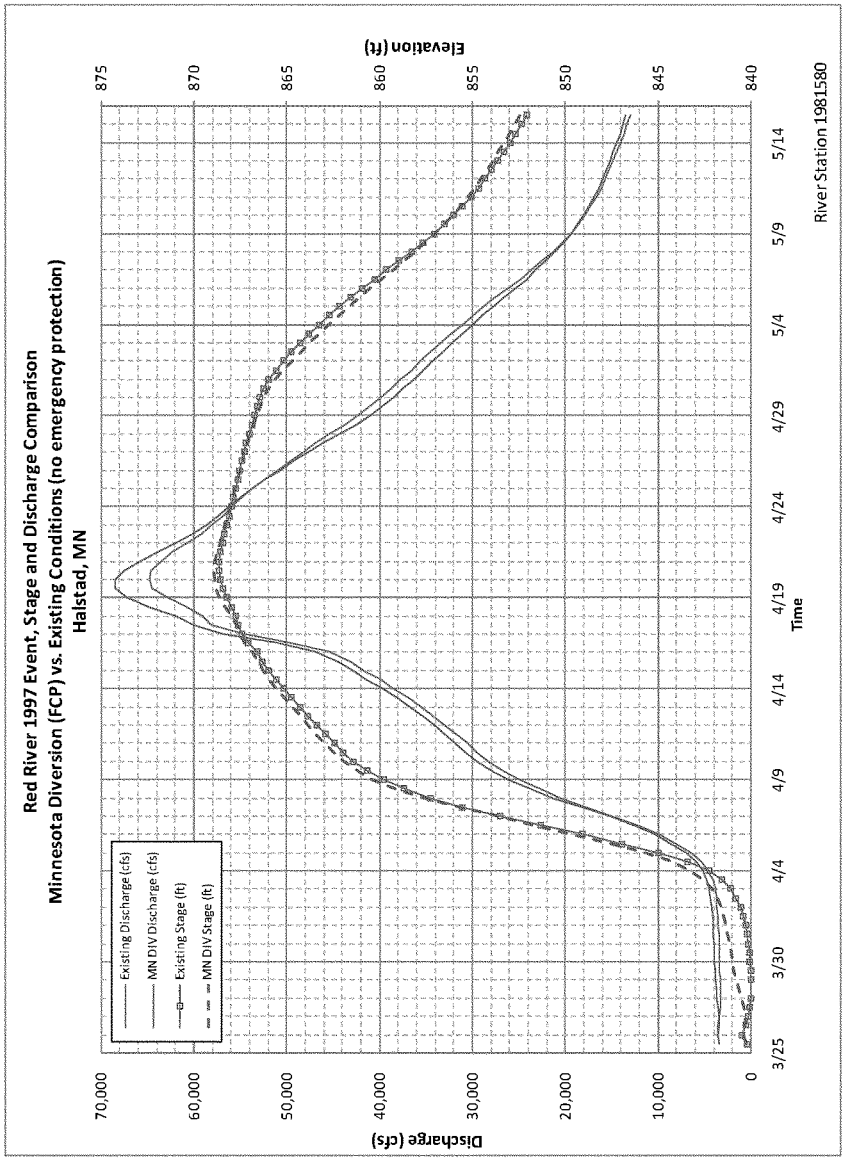

Figure C-E1- 59: 1997 Historical Flood Hydrographs for FCP @ Halstad

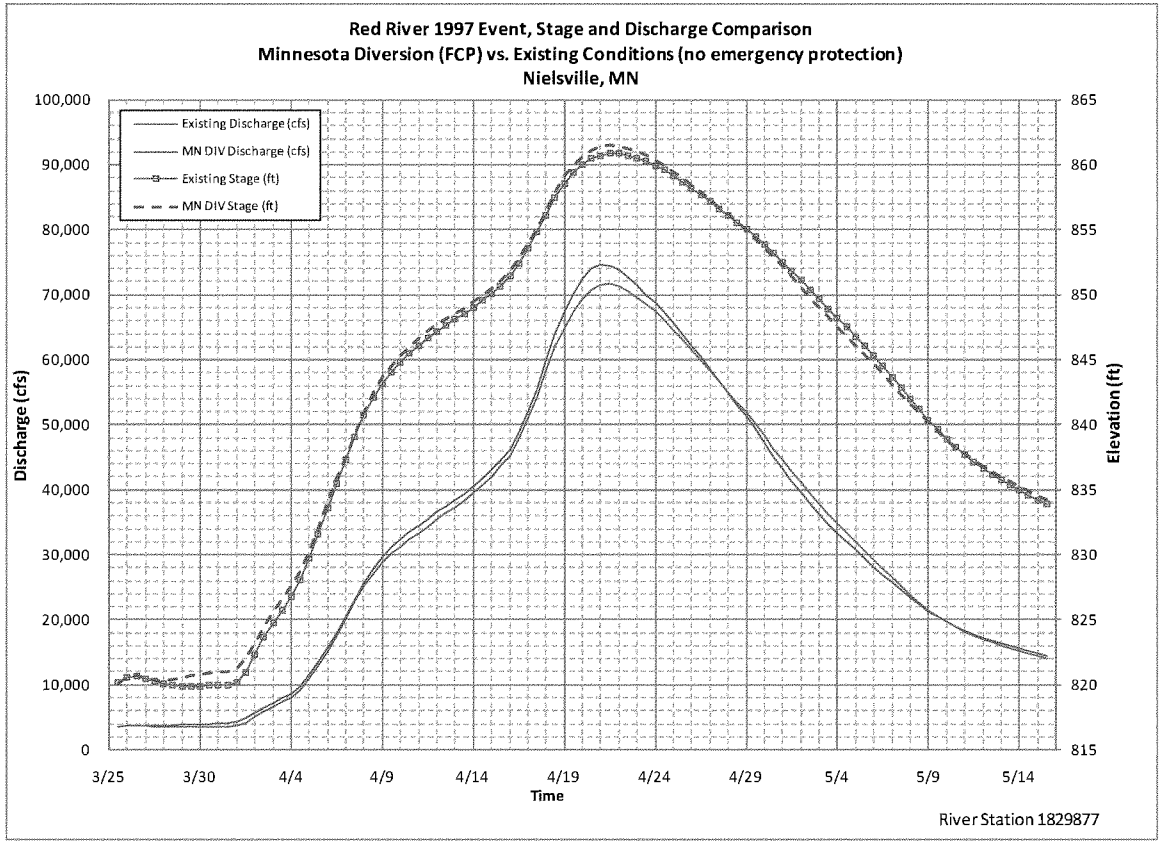

Figure C-E1- 60: 1997 Historical Flood Hydrographs for FCP @ Nielsville

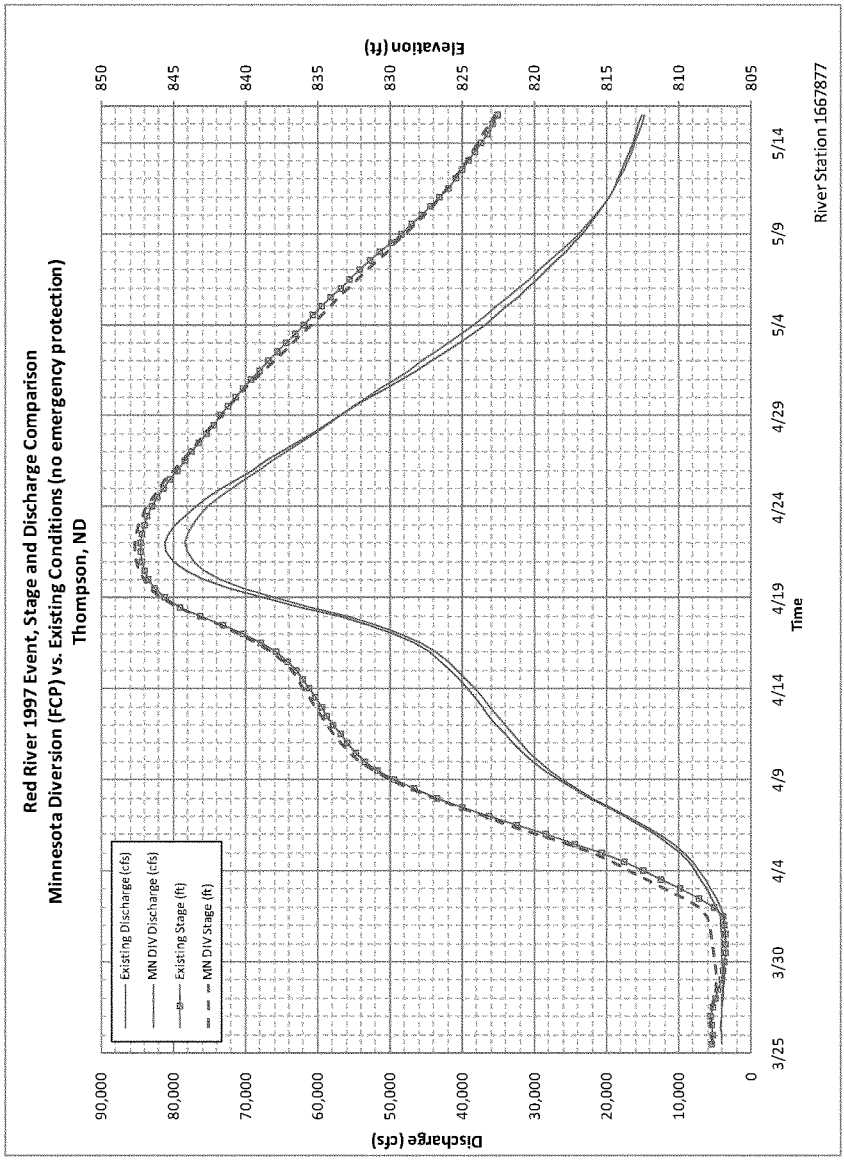

Figure C-E1- 61: 1997 Historical Flood Hydrographs for FCP @ Thompson

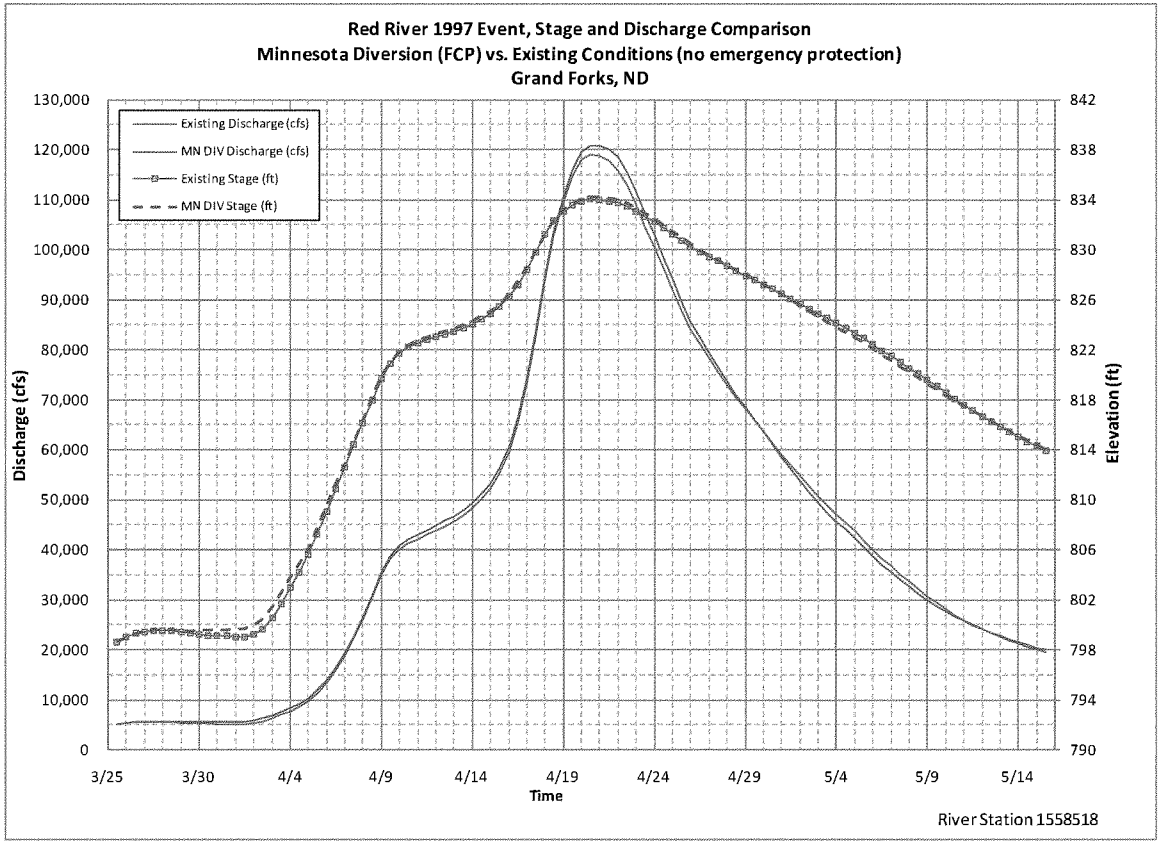

Figure C-E1- 62: 1997 Historical Flood Hydrographs for FCP @ Grand Forks

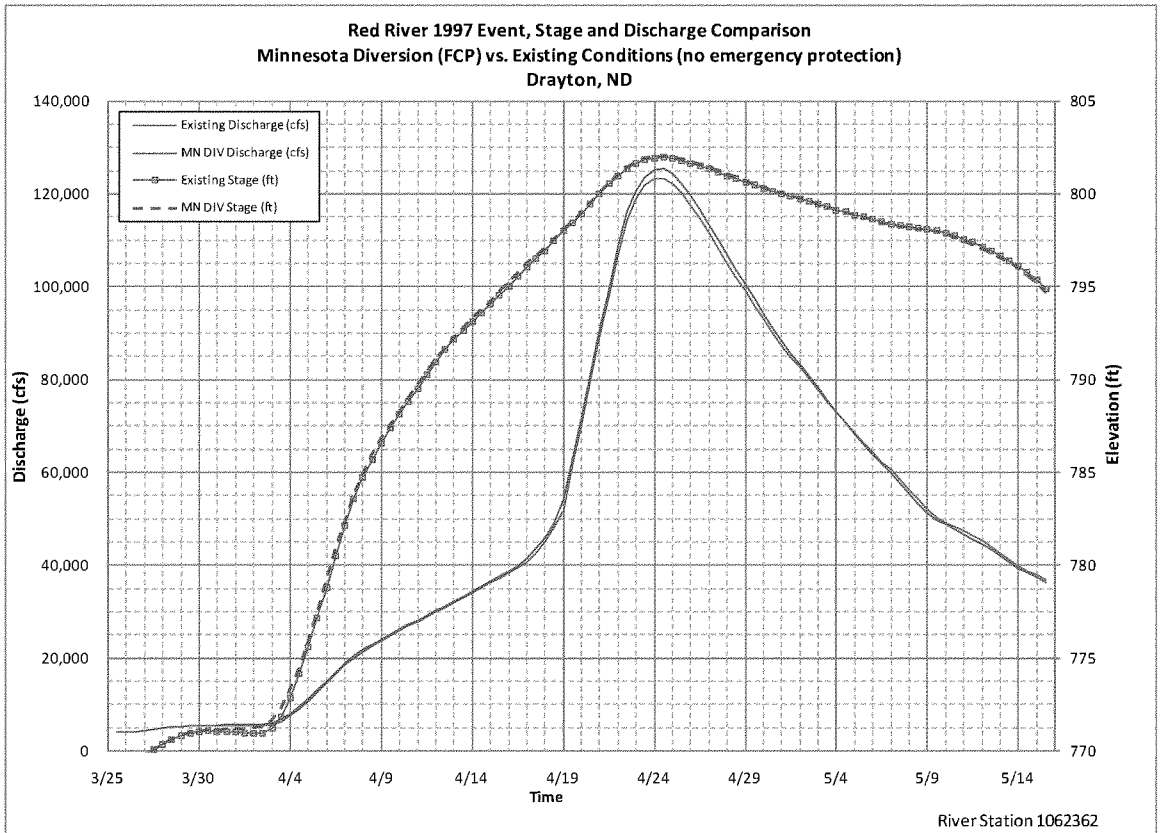

Figure C-E1- 63: 1997 Historical Flood Hydrographs for FCP @ Drayton

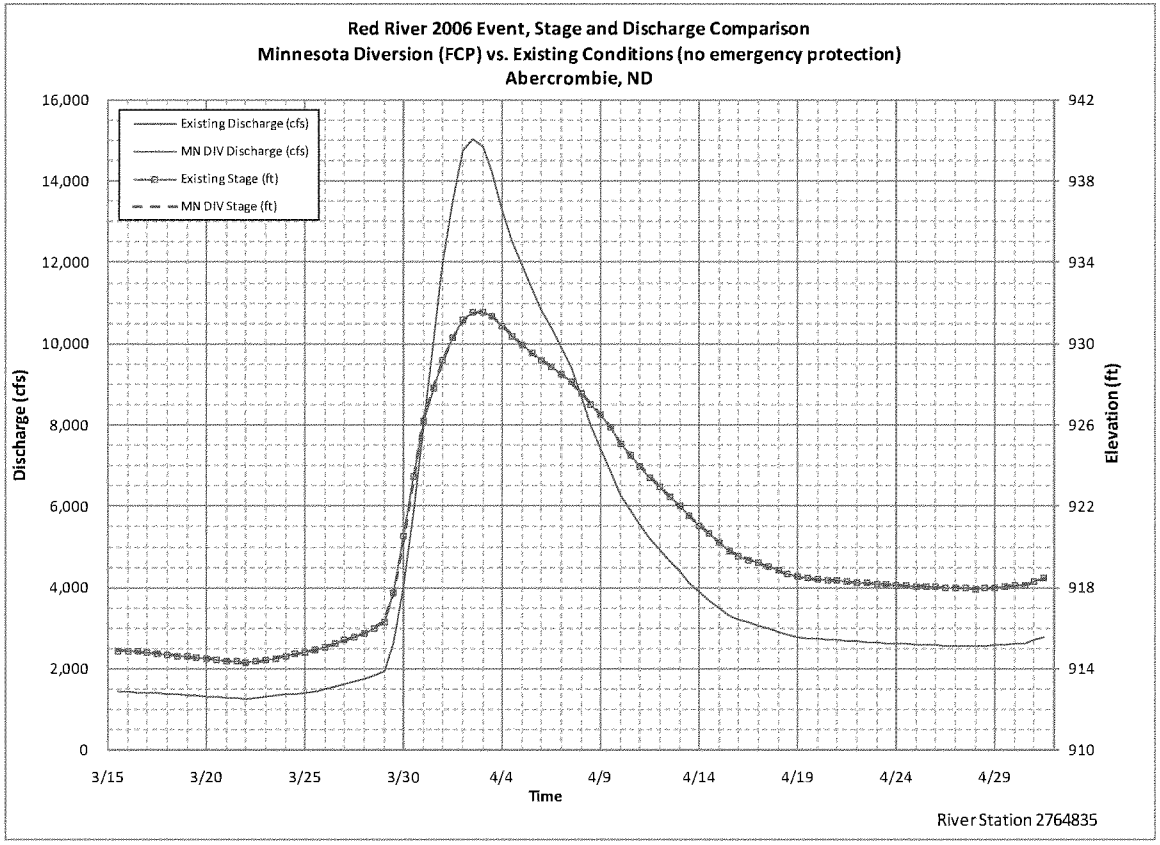

Figure C-E1- 64:2006 Historical Flood Hydrographs for FCP @ Abercrombie

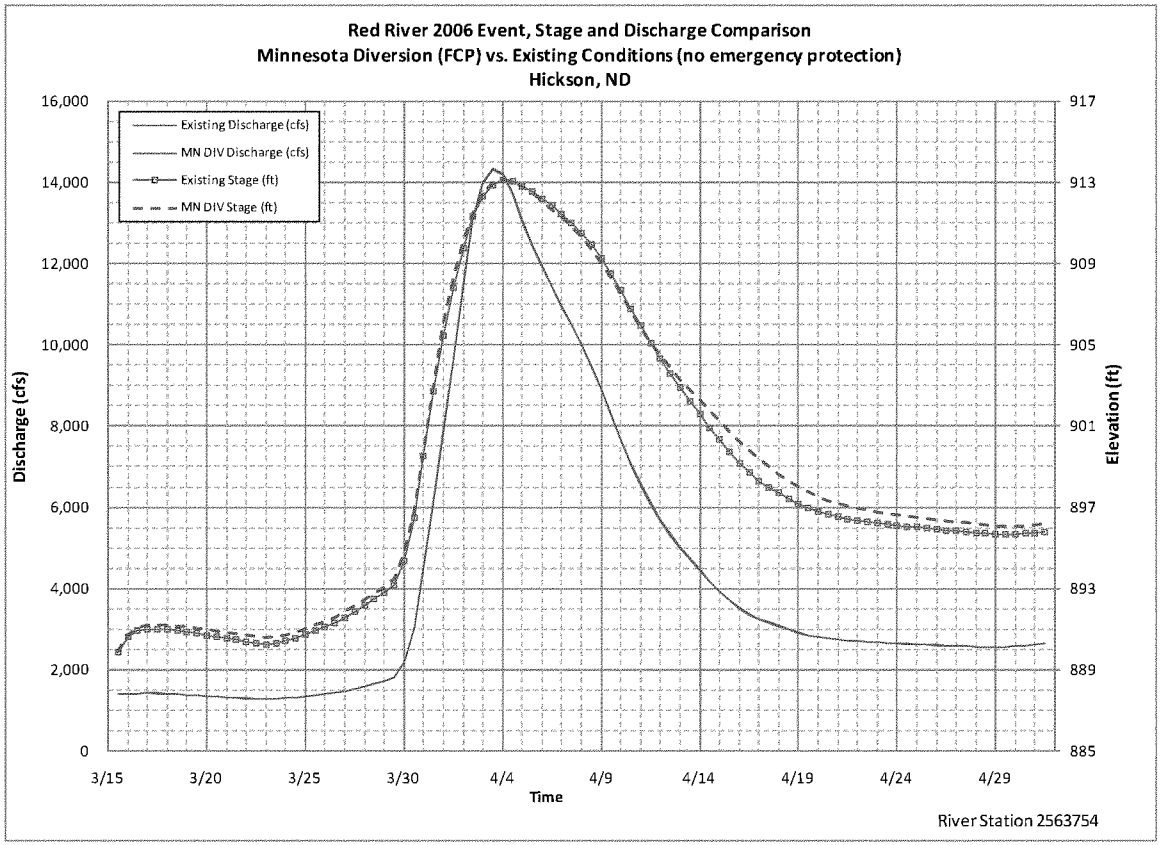

Figure C-E1- 65:2006 Historical Flood Hydrographs for FCP @ Hickson

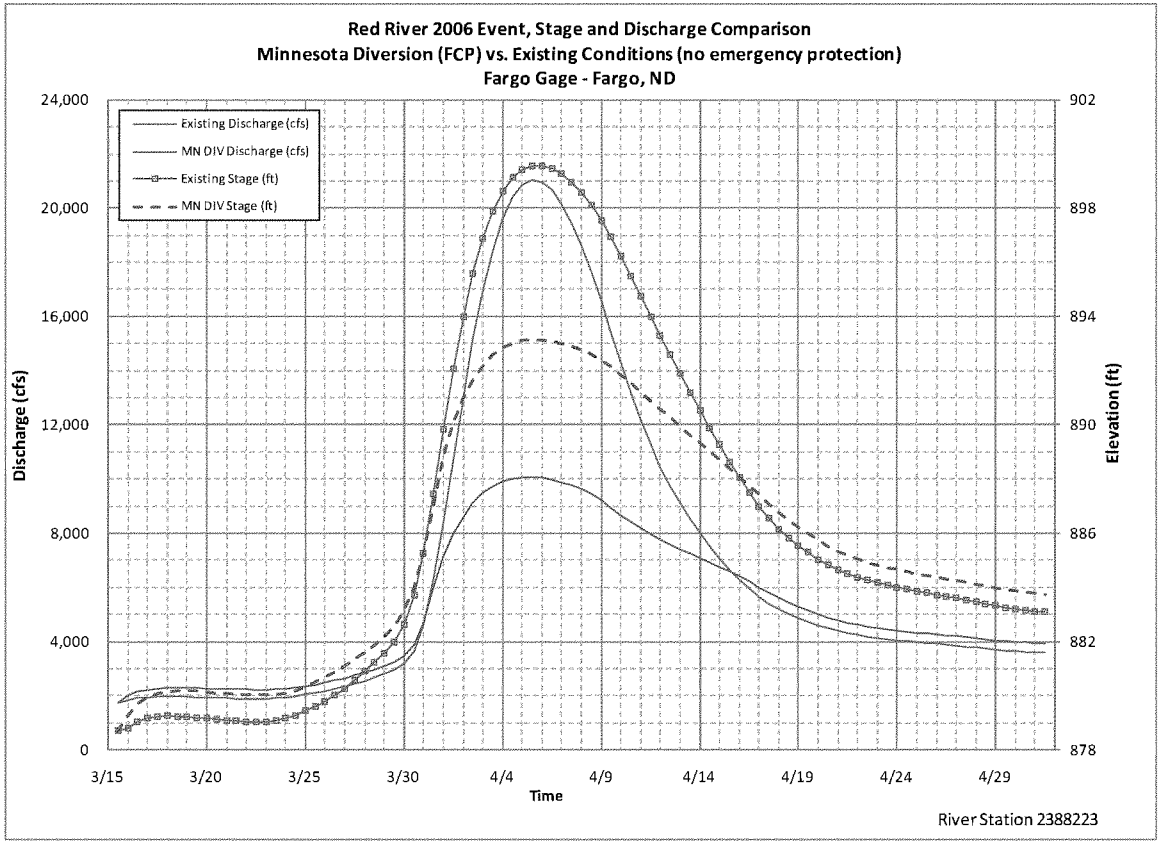

Figure C-E1- 66:2006 Historical Flood Hydrographs for FCP @ Fargo

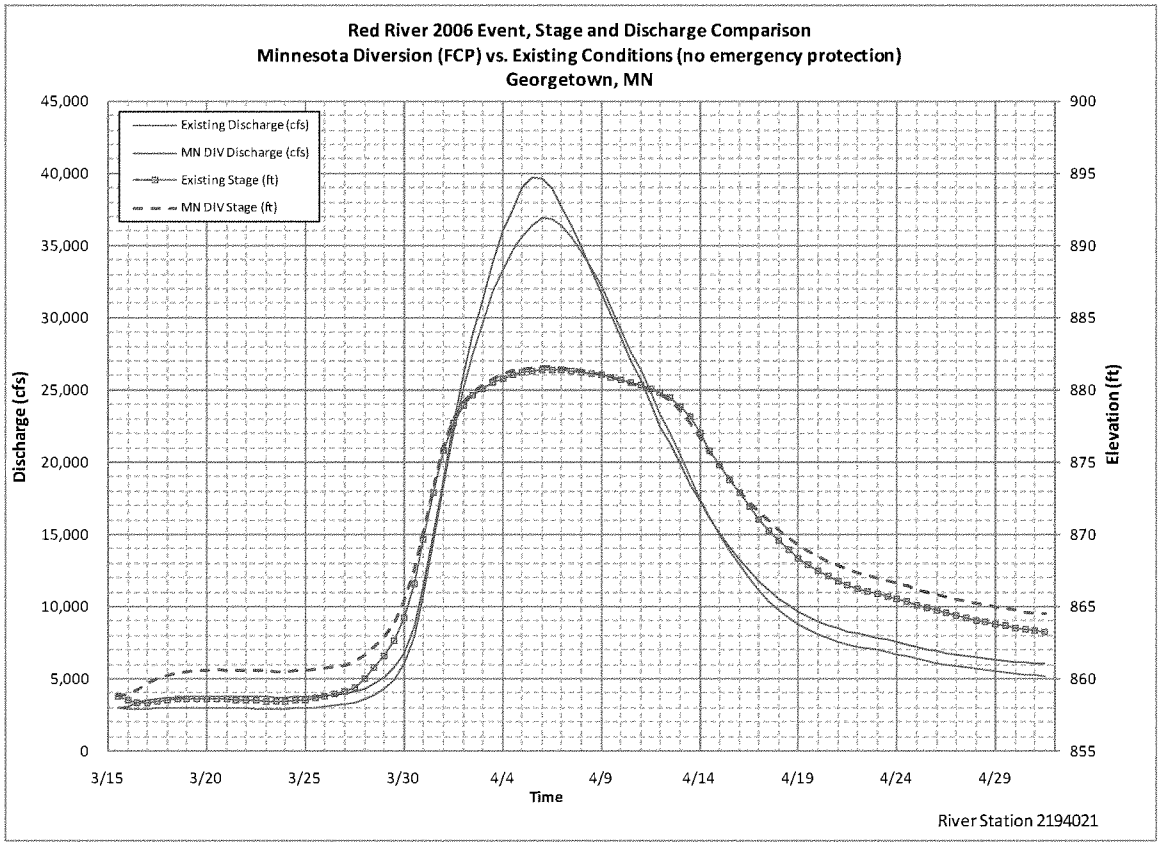

Figure C-E1- 67:2006 Historical Flood Hydrographs for FCP @ Georgetown

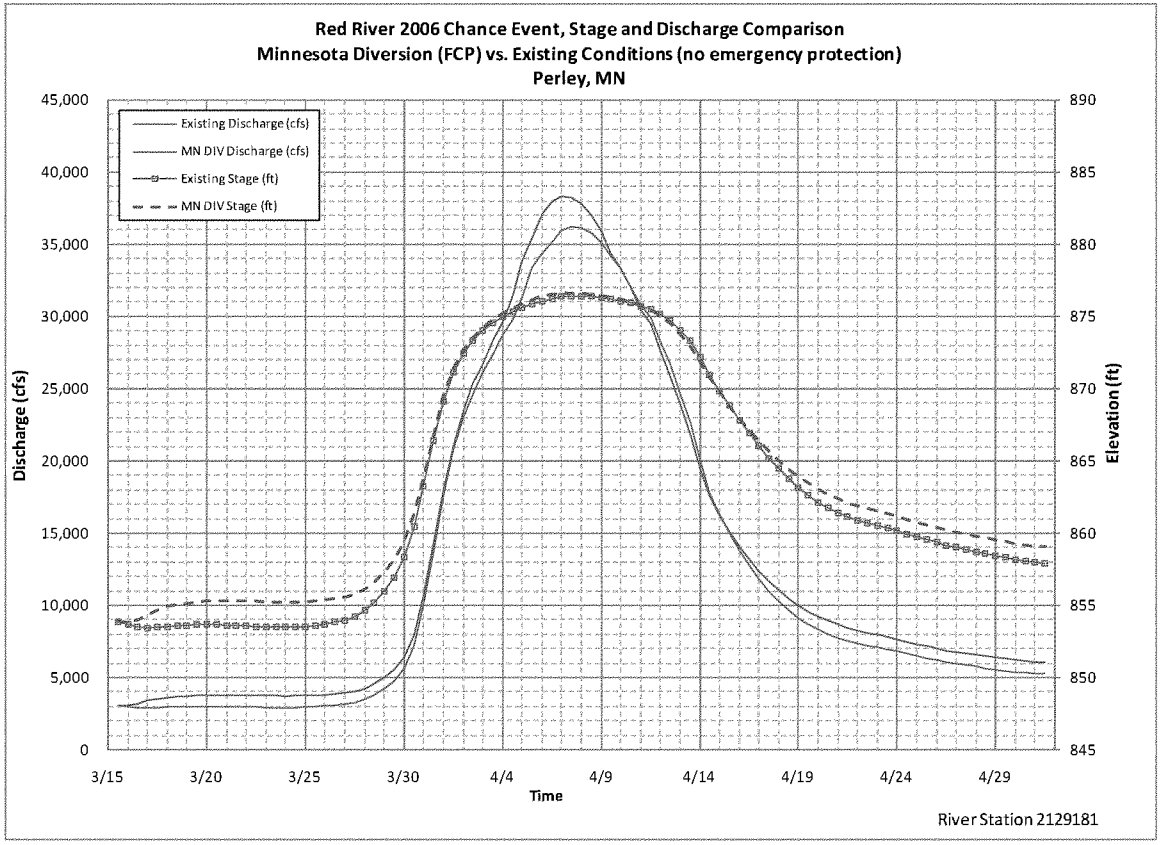

Figure C-E1- 68:2006 Historical Flood Hydrographs for FCP @ Perley

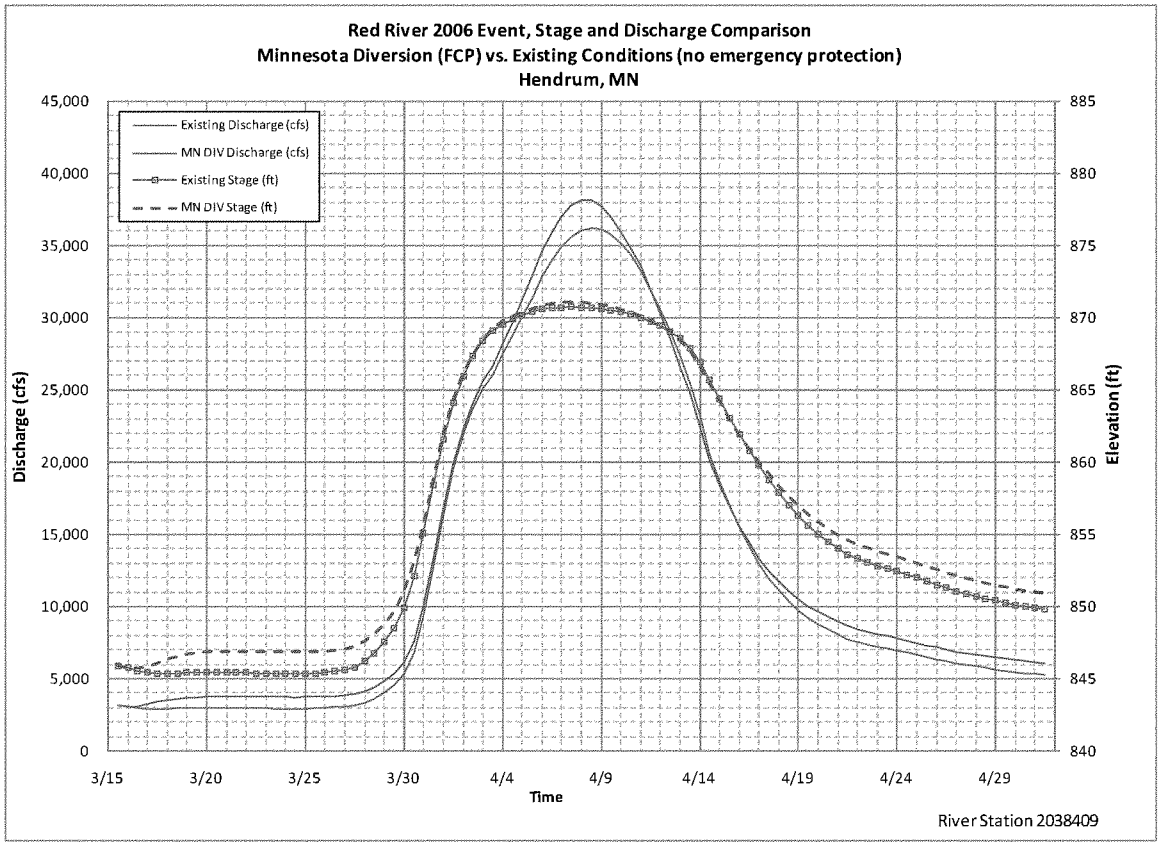

Figure C-E1- 69:2006 Historical Flood Hydrographs for FCP @ Hendrum

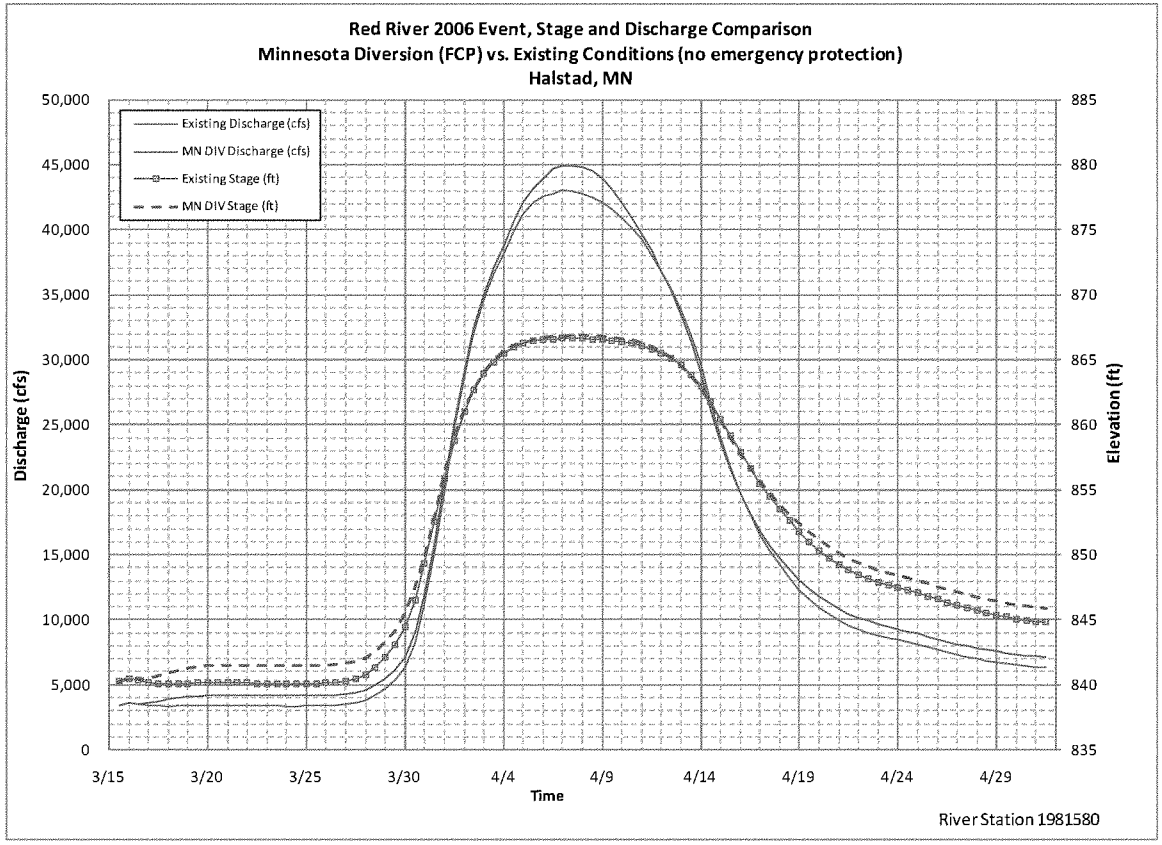

Figure C-E1- 70:2006 Historical Flood Hydrographs for FCP @ Halstad

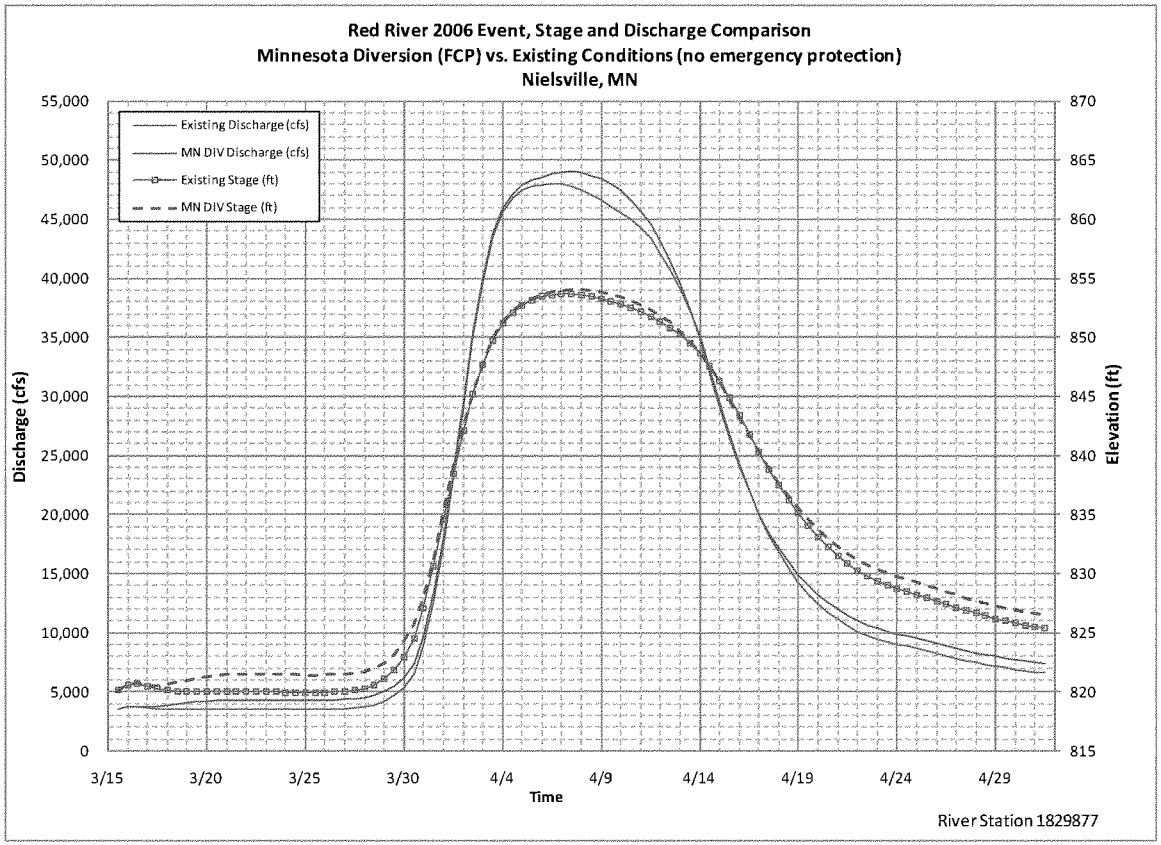

Figure C-E1- 71:2006 Historical Flood Hydrographs for FCP @ Nielsville

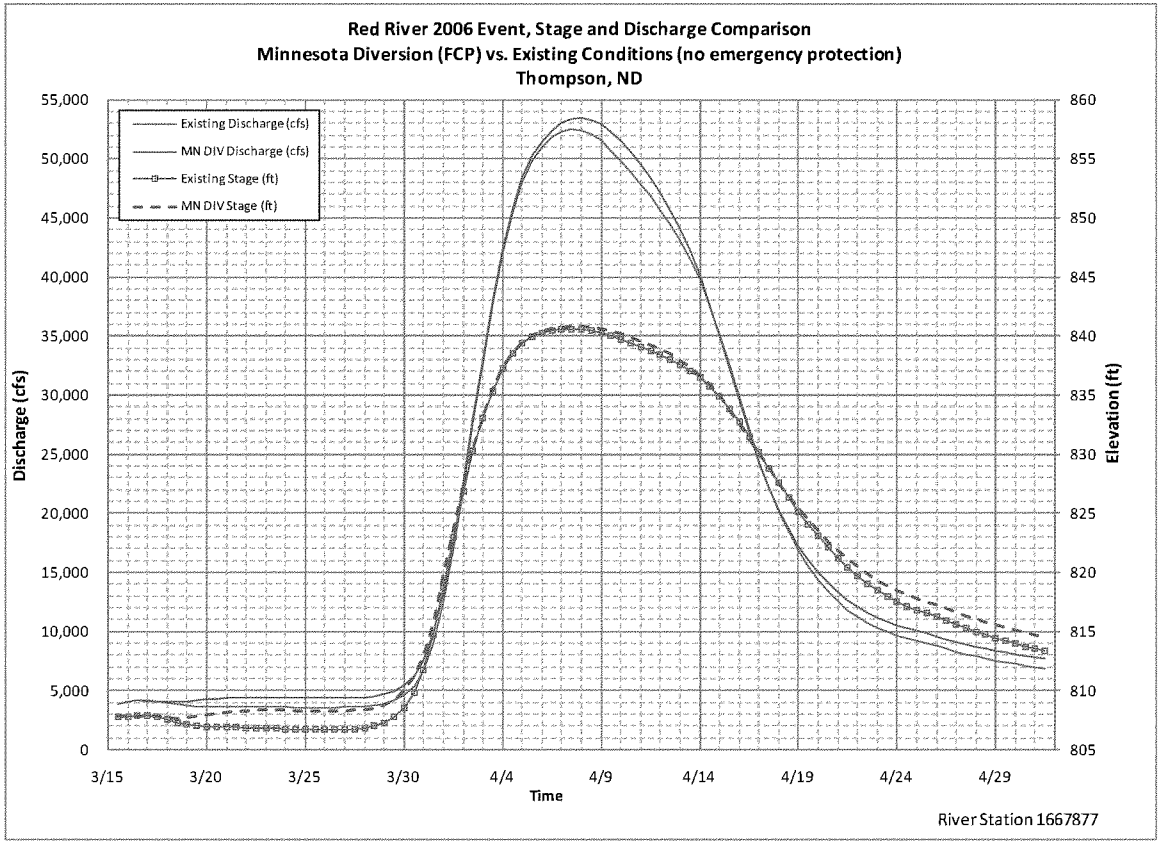

Figure C-E1- 72:2006 Historical Flood Hydrographs for FCP @ Thompson

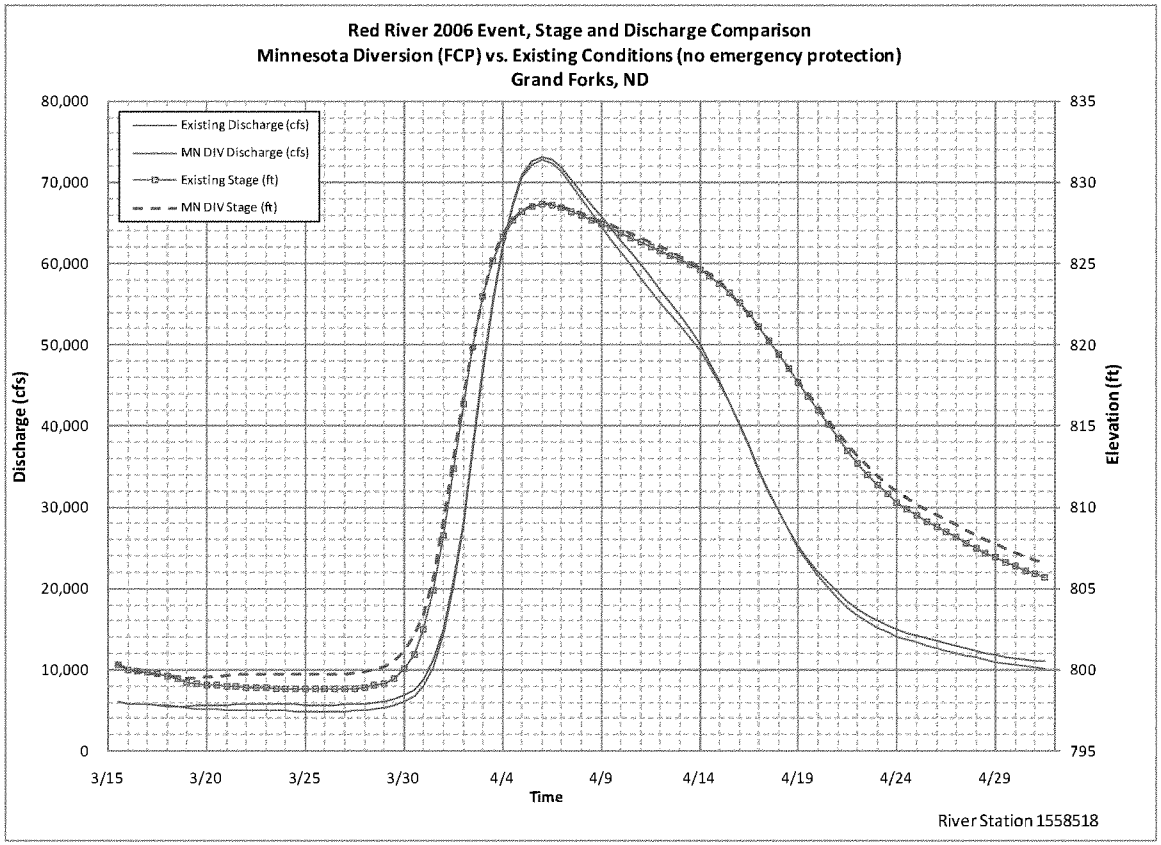

Figure C-E1- 73:2006 Historical Flood Hydrographs for FCP @ Grand Forks

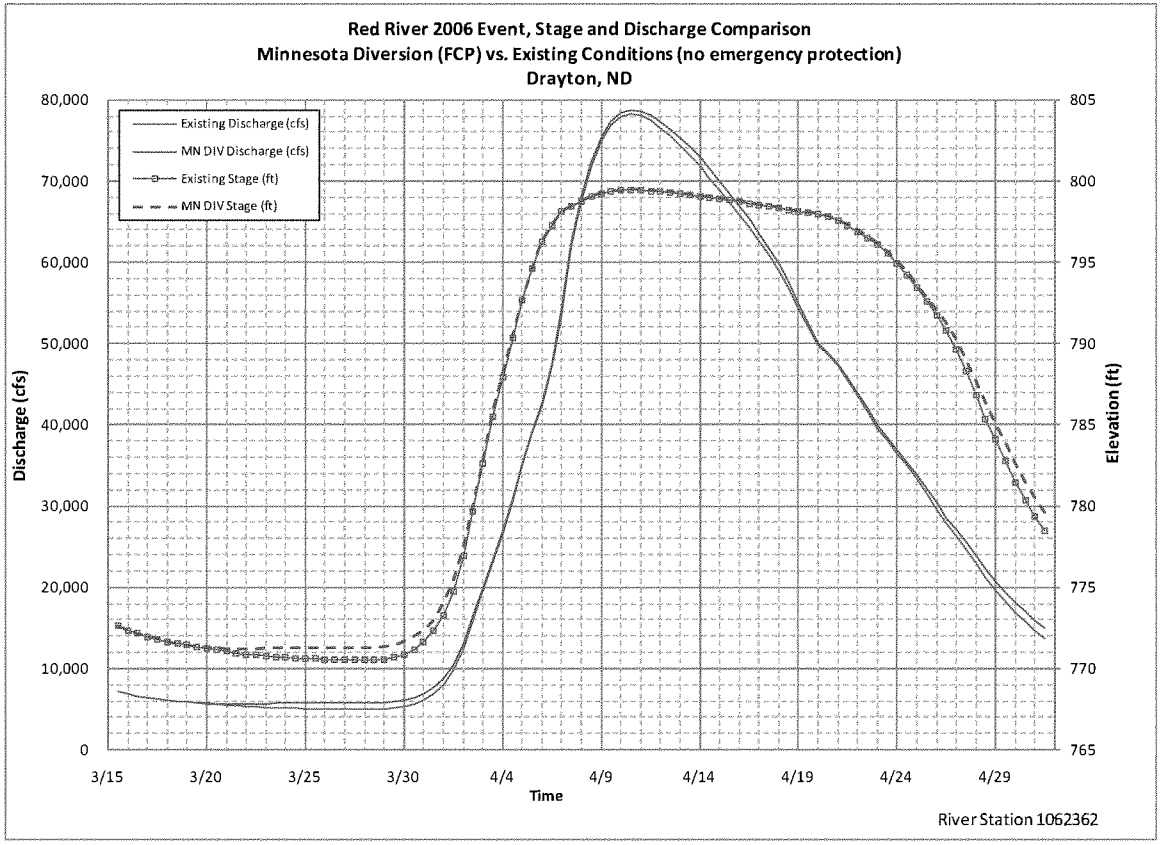

Figure C-E1- 74:2006 Historical Flood Hydrographs for FCP @ Drayton

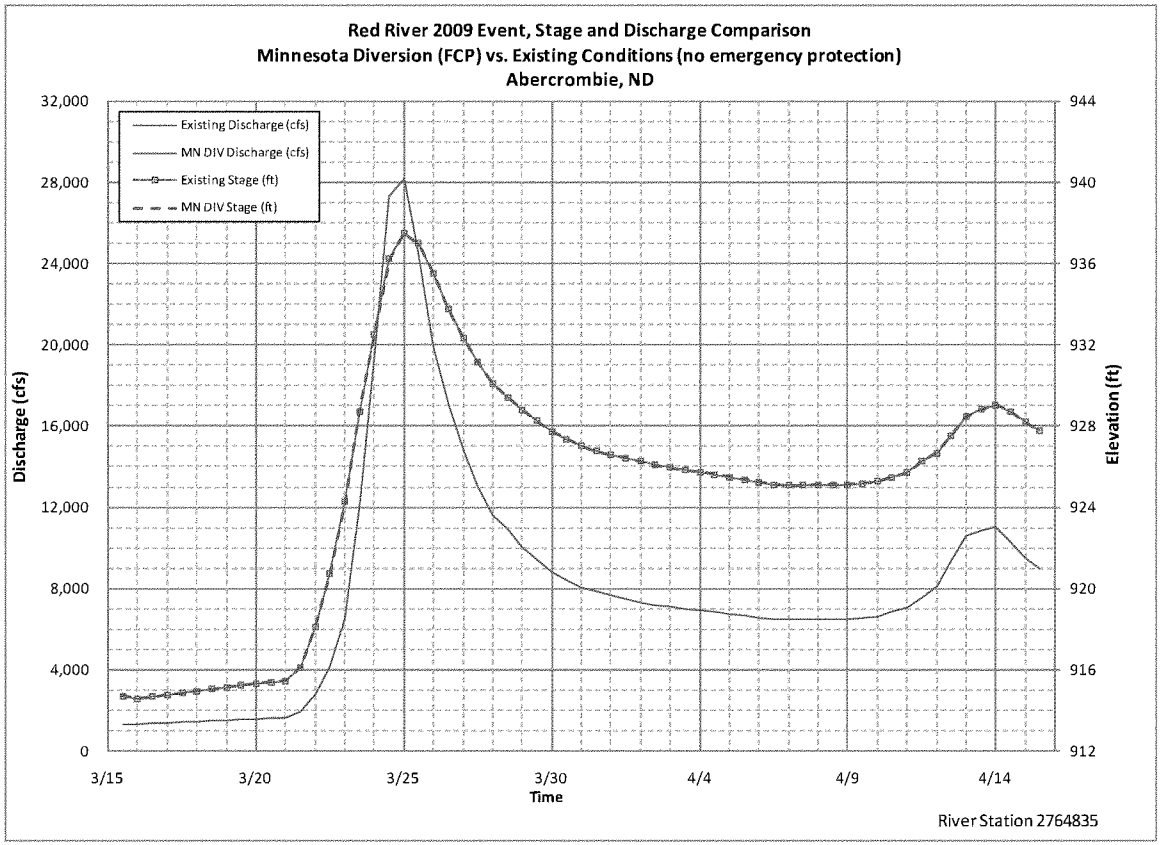

Figure C-E1- 75: 2009 Historical Flood Hydrographs for FCP @ Abercrombie

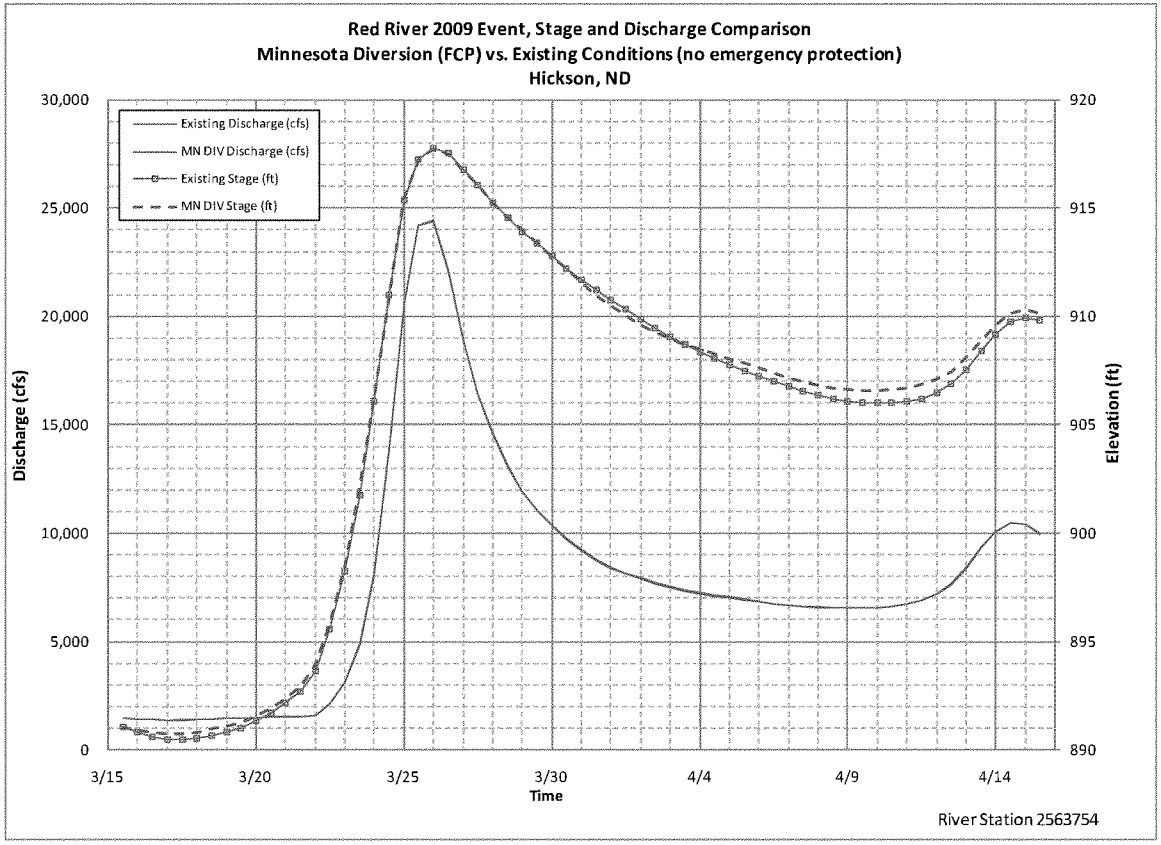

Figure C-E1- 76: 2009 Historical Flood Hydrographs for FCP @ Hickson

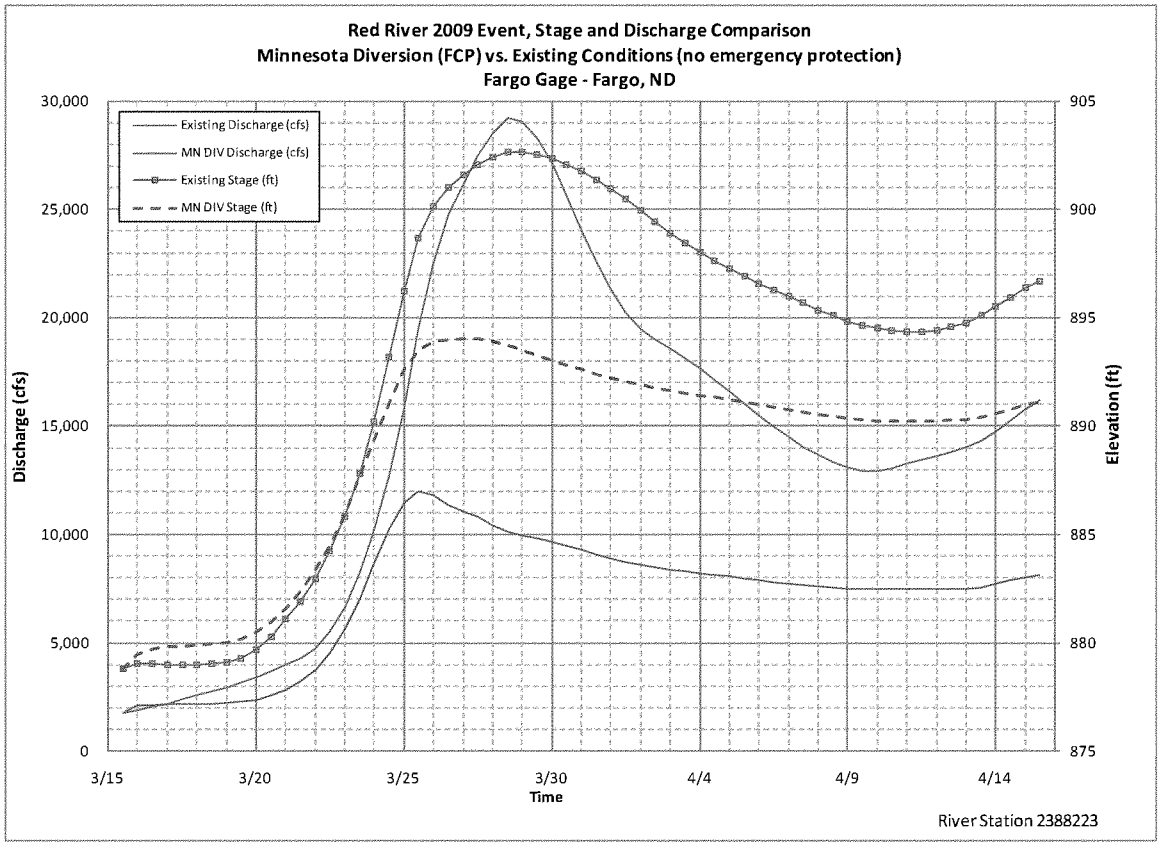

Figure C-E1- 77: 2009 Historical Flood Hydrographs for FCP @ Fargo

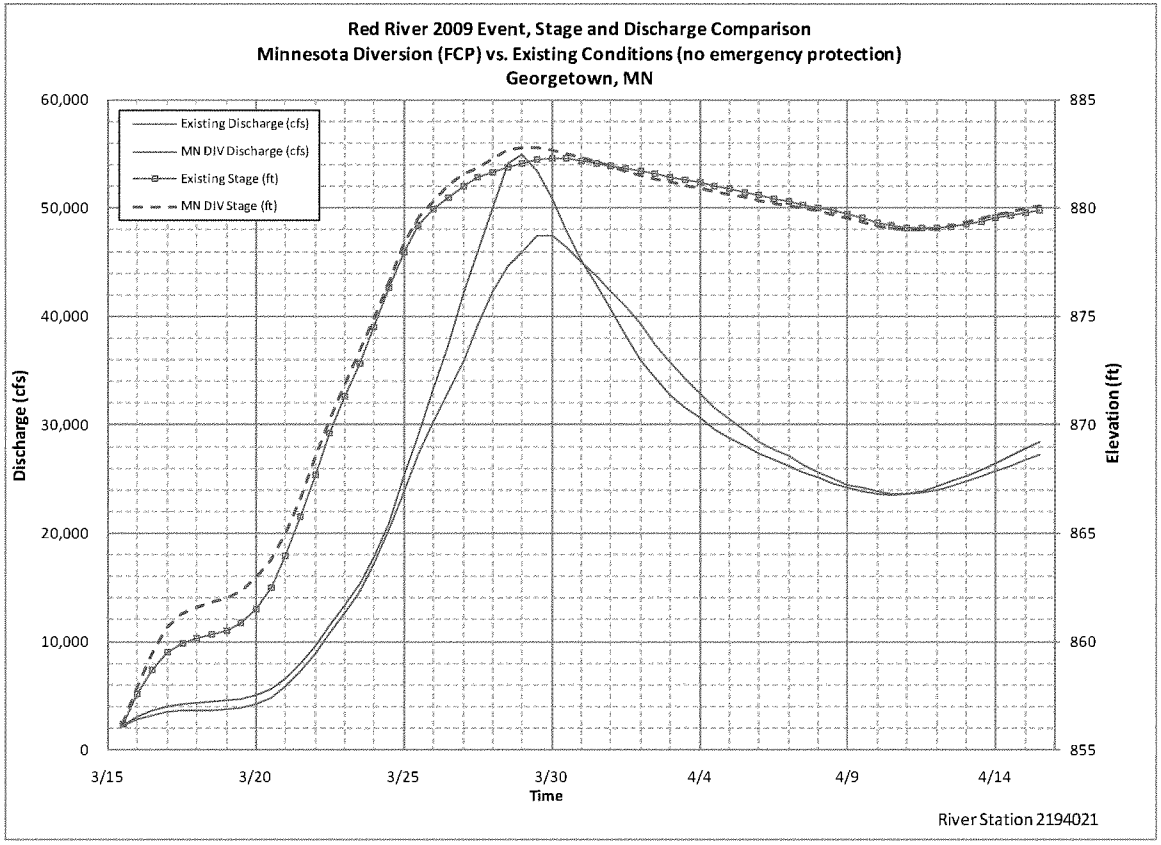

Figure C-E1- 78: 2009 Historical Flood Hydrographs for FCP @ Georgetown

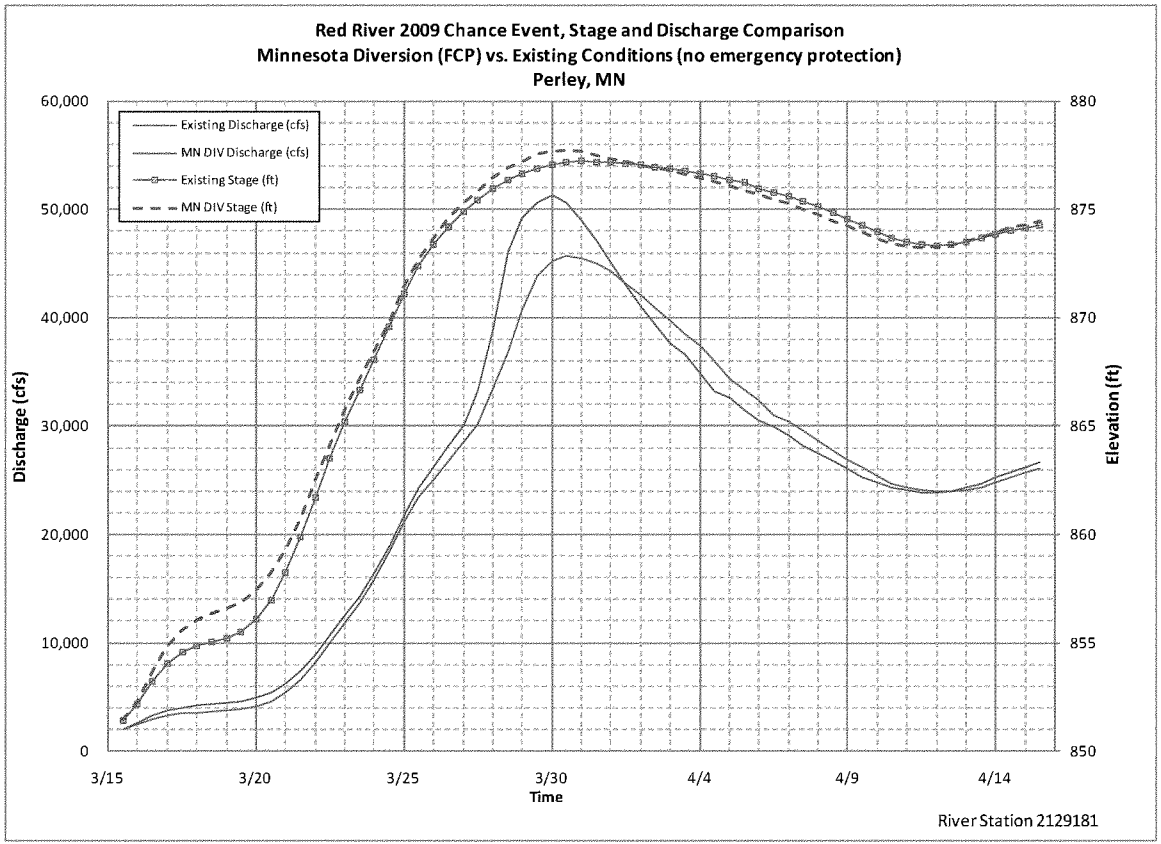

Figure C-E1- 79: 2009 Historical Flood Hydrographs for FCP @ Perley

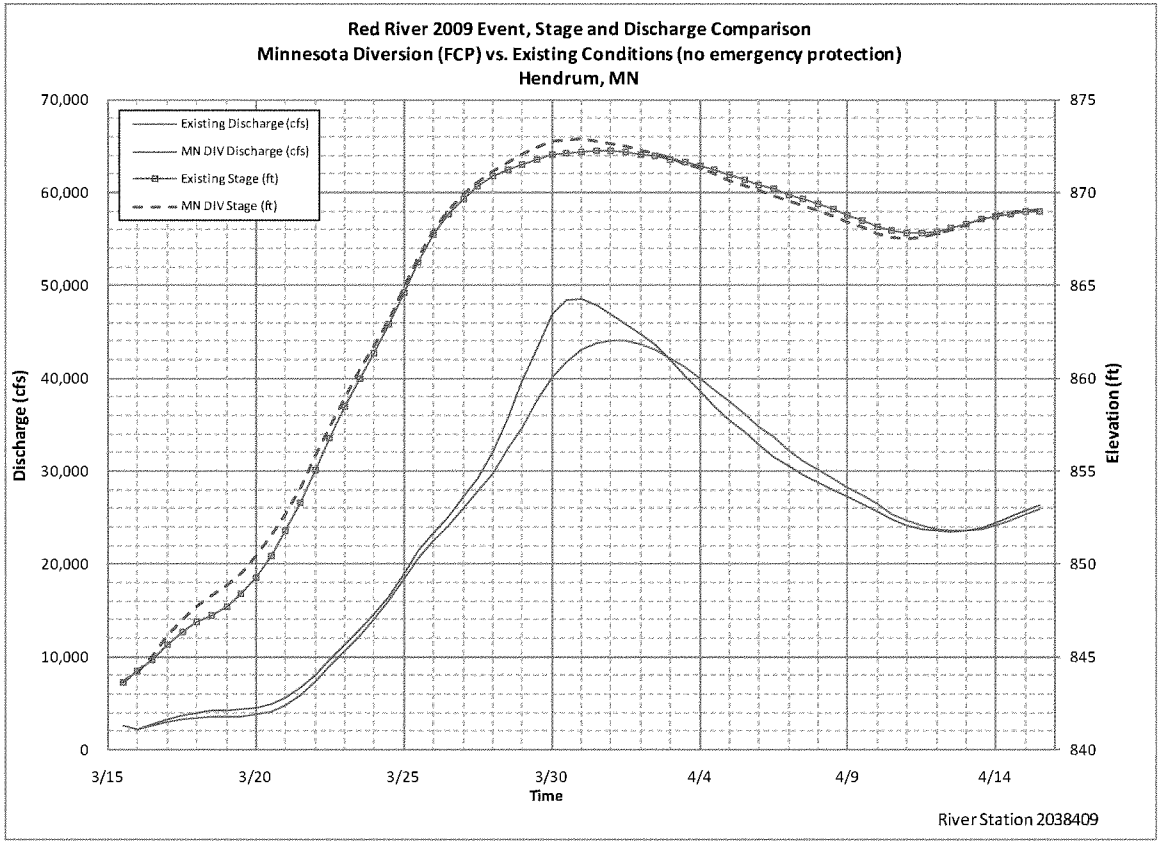

Figure C-E1- 80: 2009 Historical Flood Hydrographs for FCP @ Hendrum

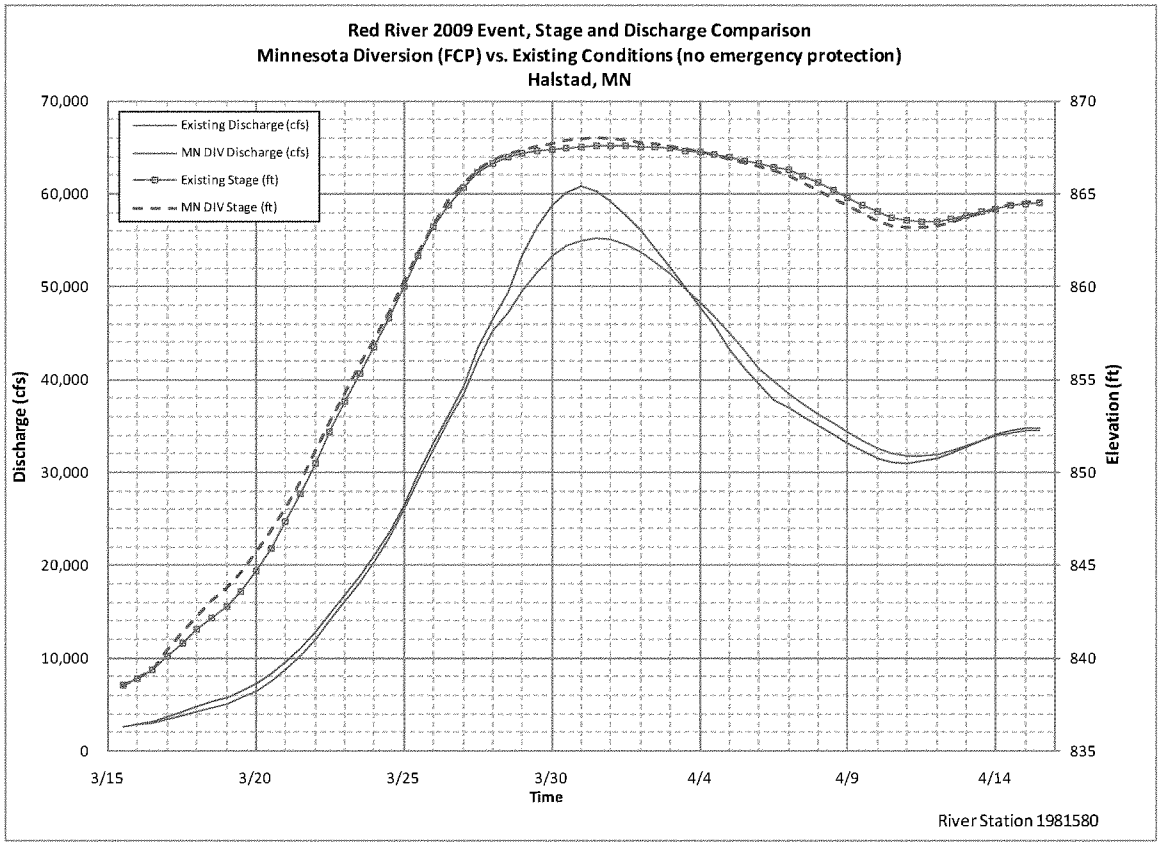

Figure C-E1- 81: 2009 Historical Flood Hydrographs for FCP @ Halstad

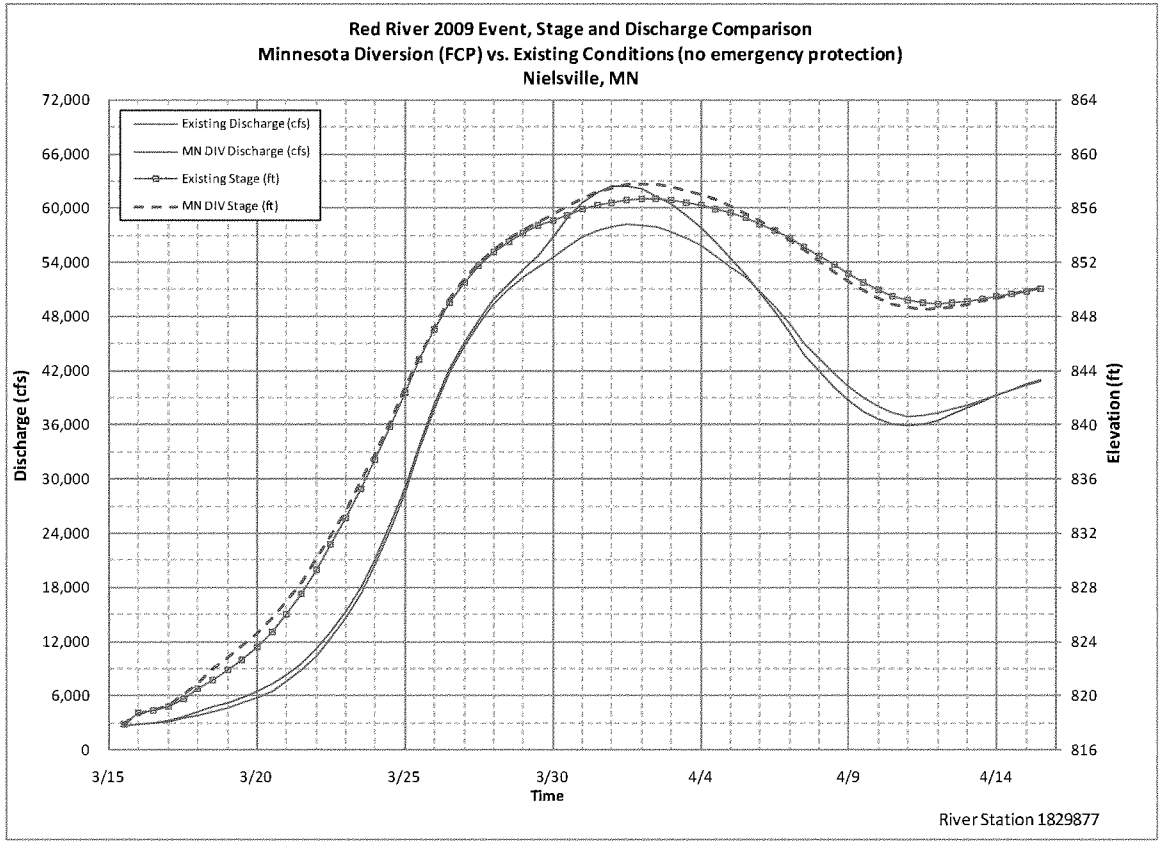

Figure C-E1- 82: 2009 Historical Flood Hydrographs for FCP @ Nielsville

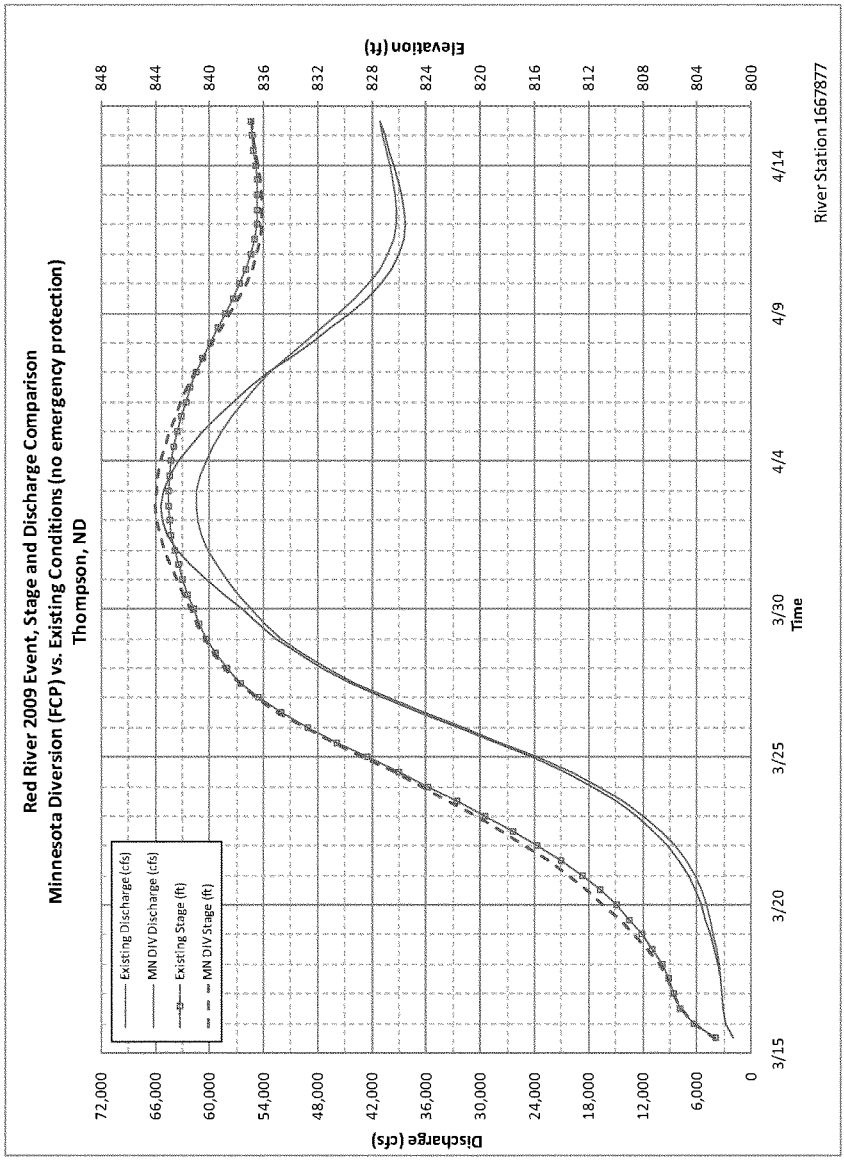

Figure C-E1- 83: 2009 Historical Flood Hydrographs for FCP @ Thompson

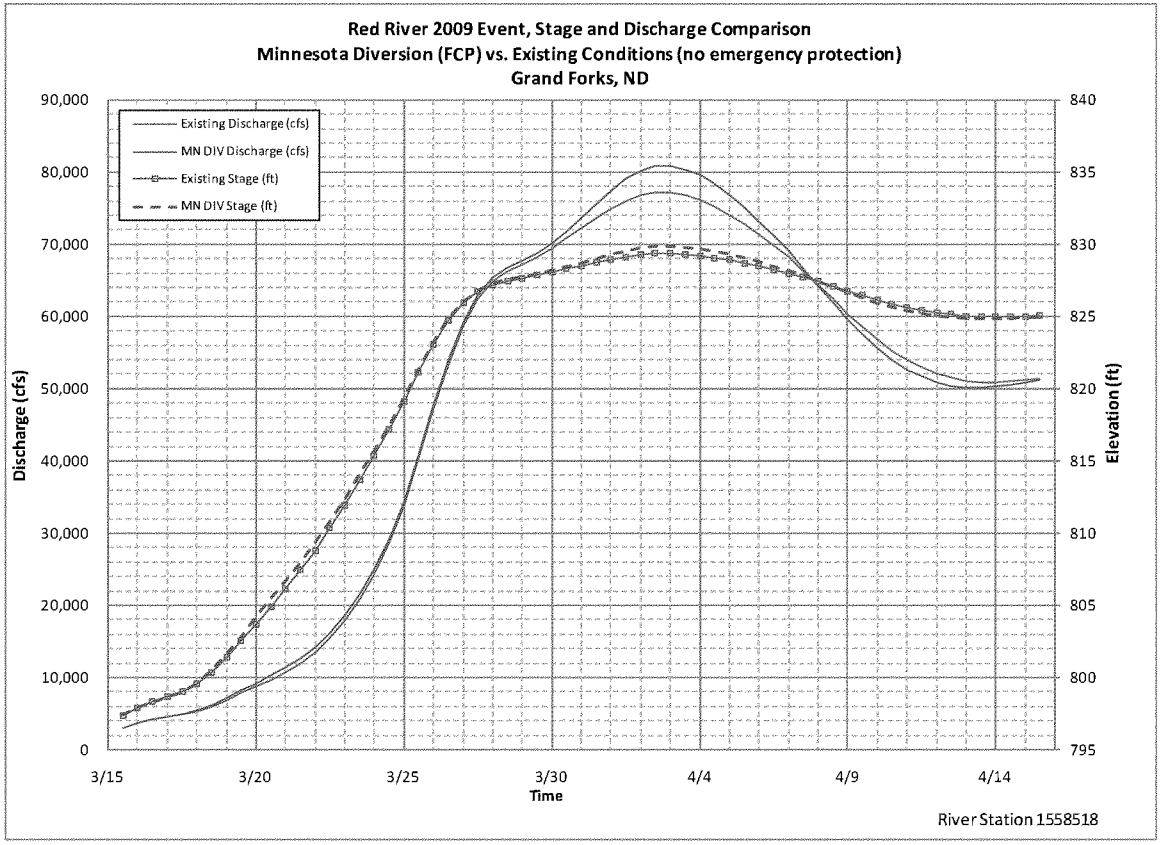

Figure C-E1- 84: 2009 Historical Flood Hydrographs for FCP @ Grand Forks

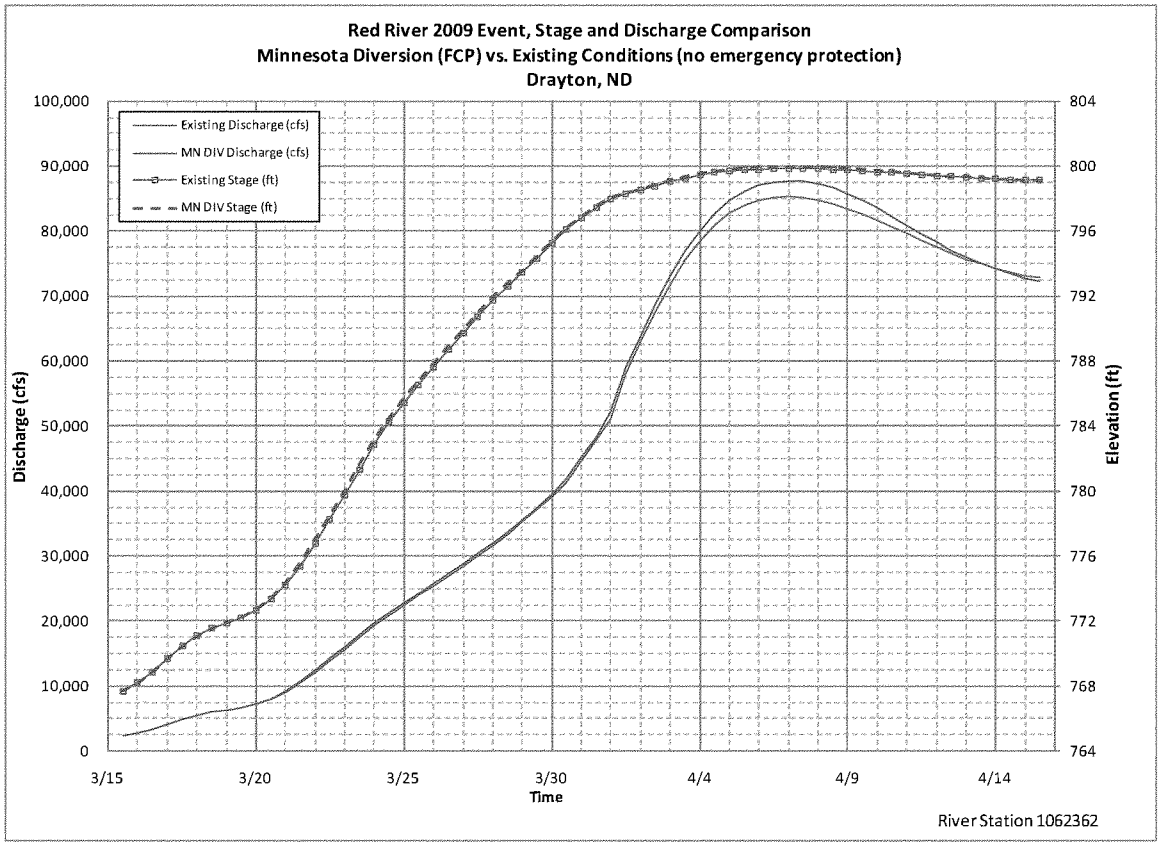

Figure C-E1- 85: 2009 Historical Flood Hydrographs for FCP @ Drayton

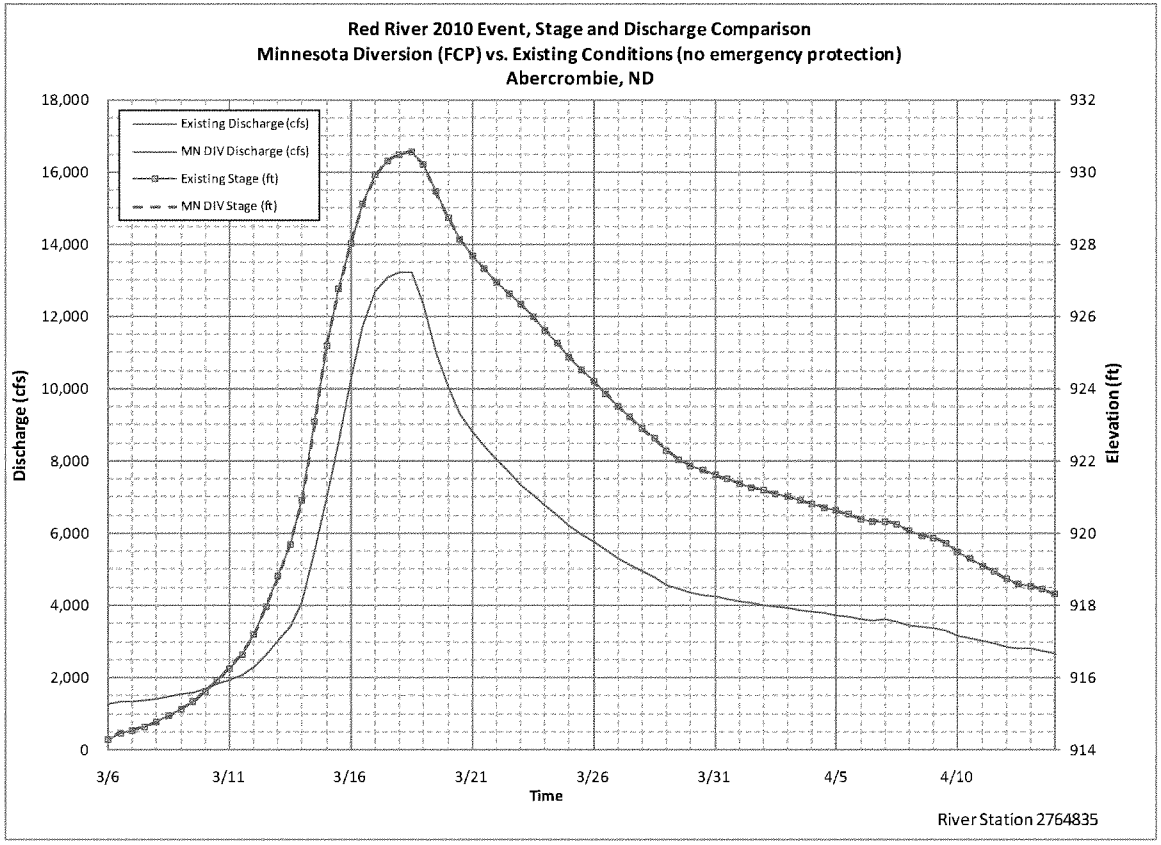

Figure C-E1- 86: 2010 Historical Flood Hydrographs for FCP @ Abercrombie

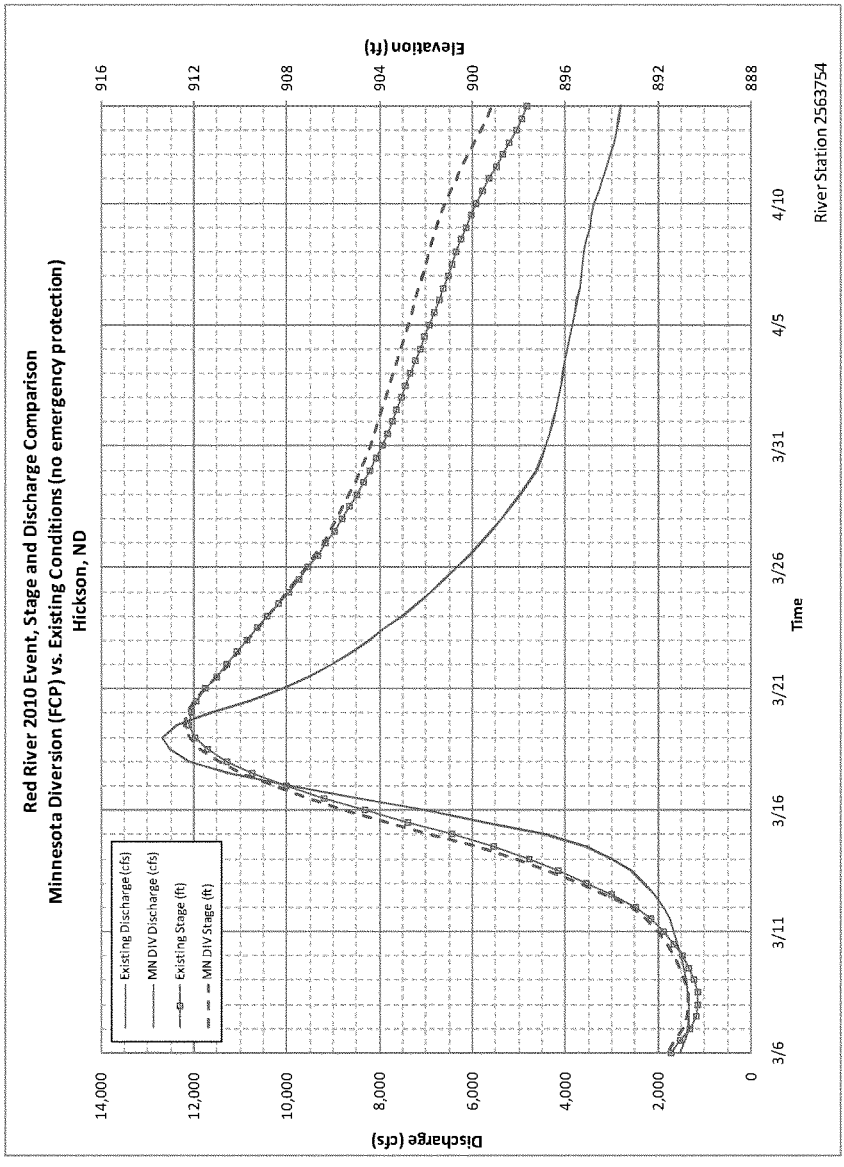

Figure C-E1- 87: 2010 Historical Flood Hydrographs for FCP @ Hickson

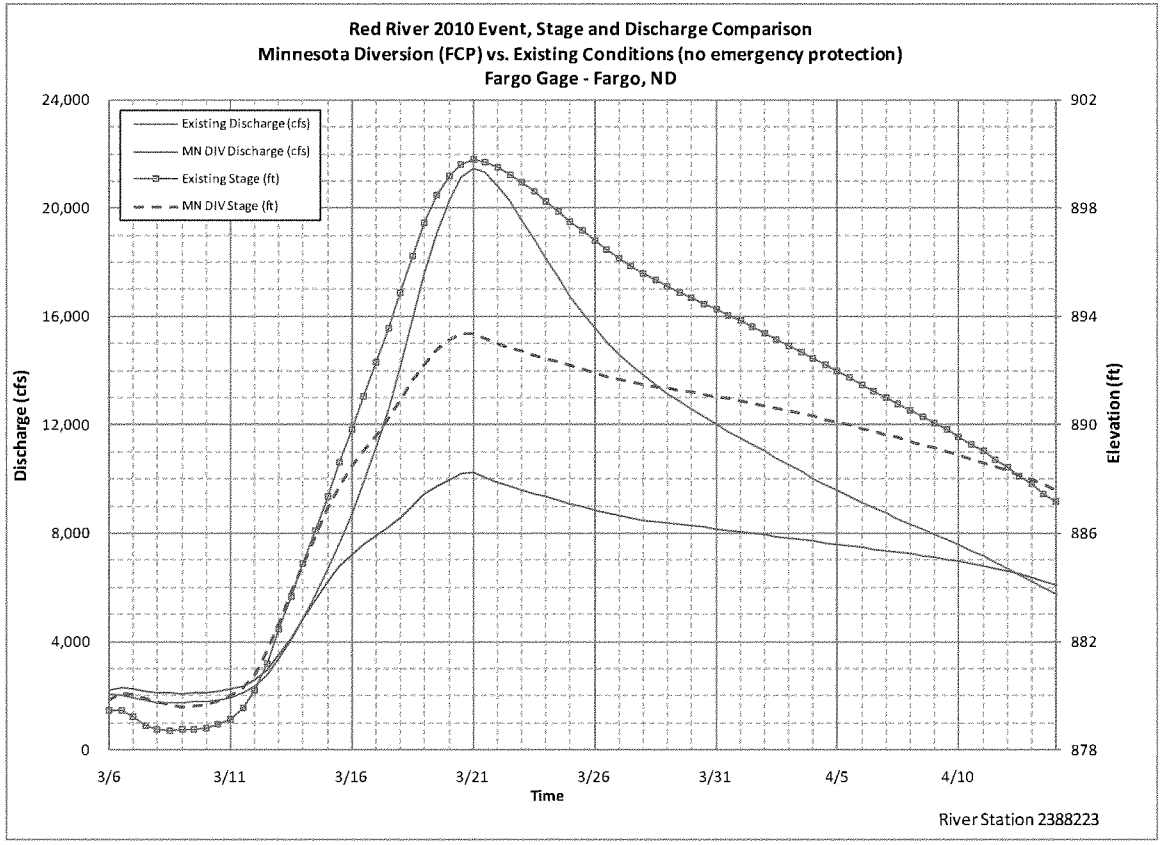

Figure C-E1- 88: 2010 Historical Flood Hydrographs for FCP @ Fargo

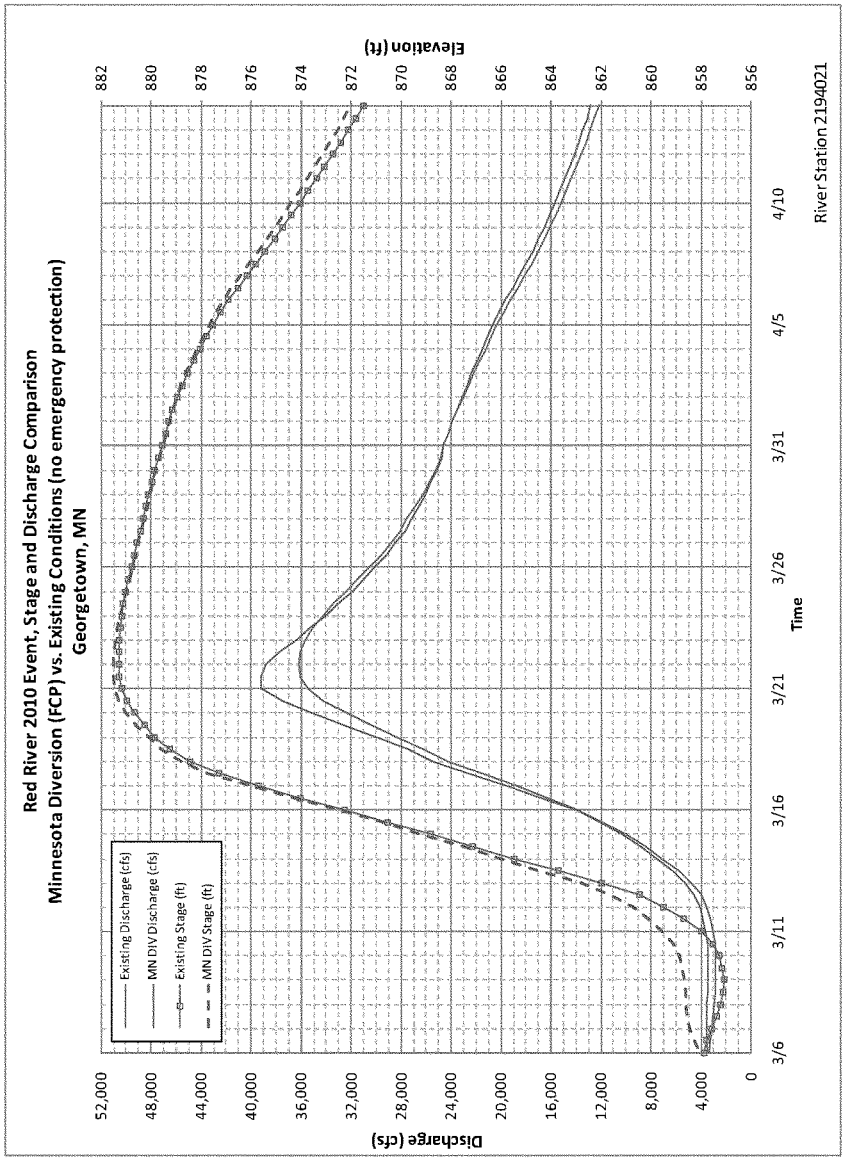

Figure C-E1- 89: 2010 Historical Flood Hydrographs for FCP @ Georgetown

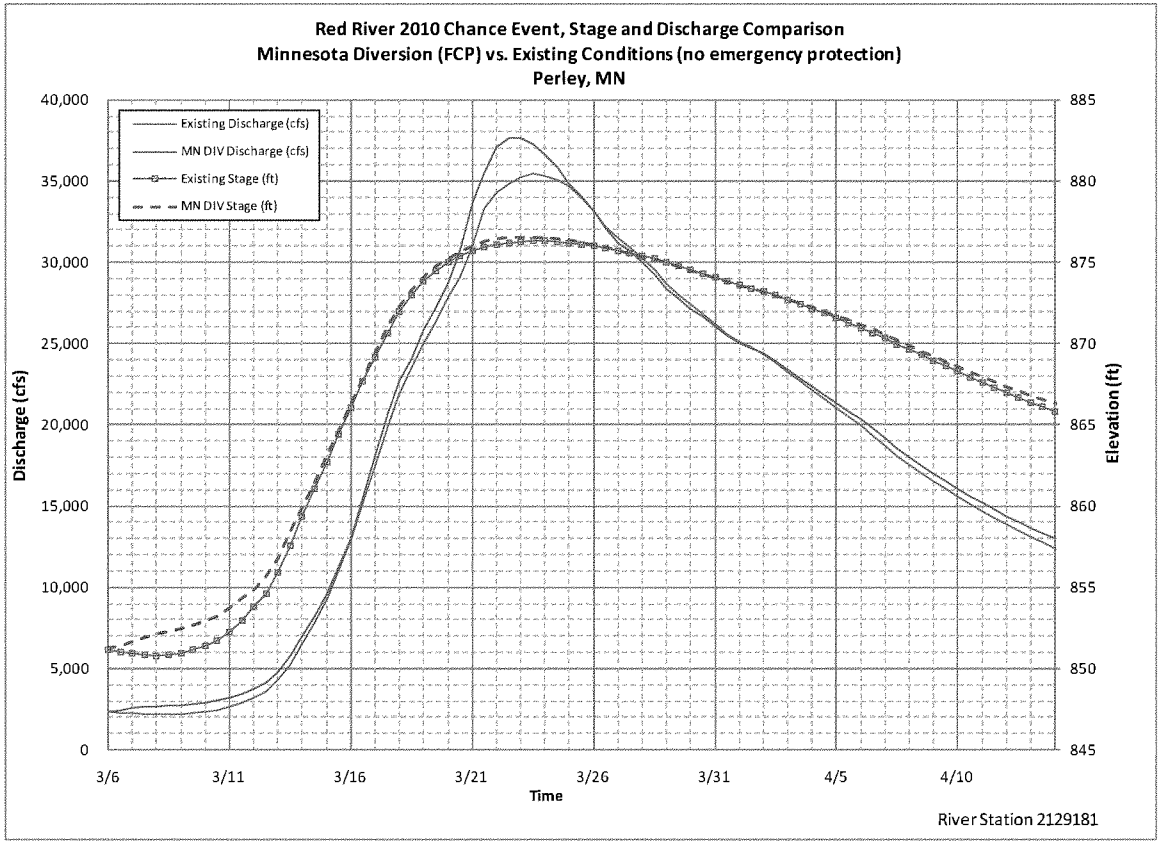

Figure C-E1- 90: 2010 Historical Flood Hydrographs for FCP @ Perley

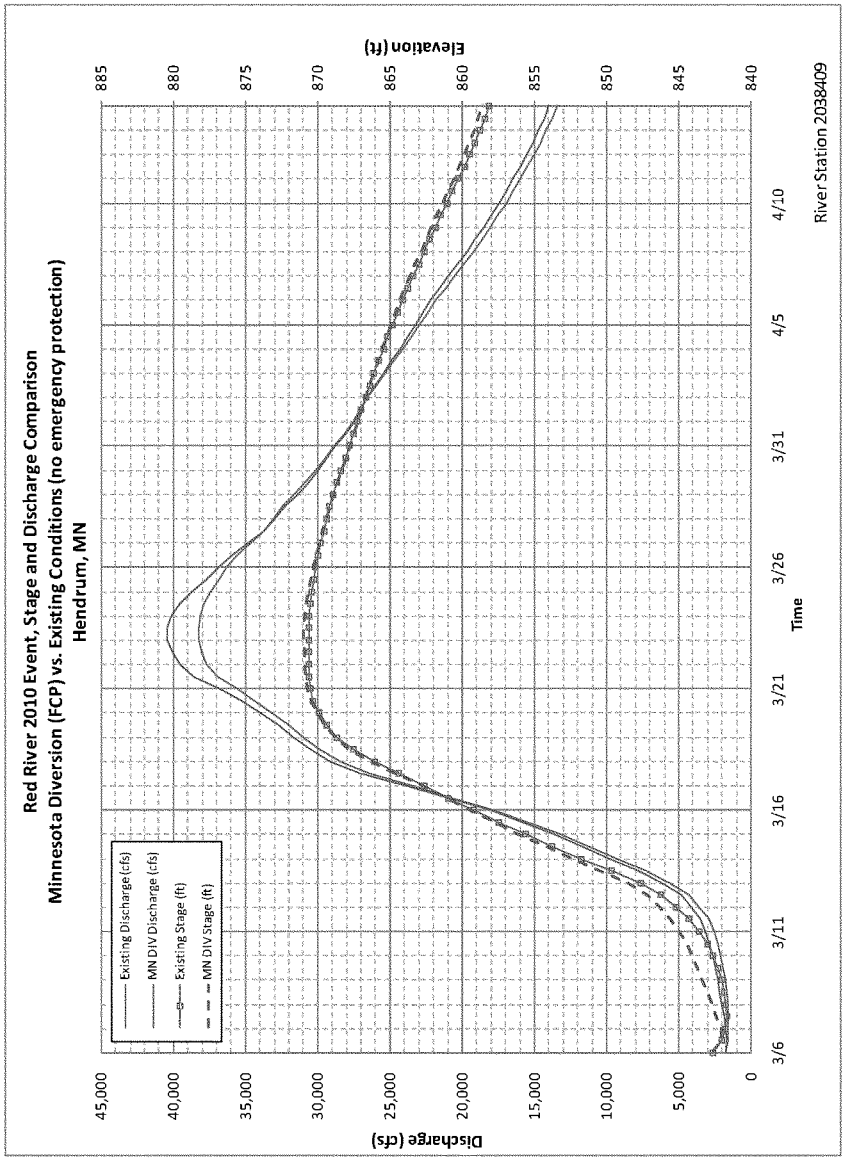

Figure C-E1- 91: 2010 Historical Flood Hydrographs for FCP @ Hendrum

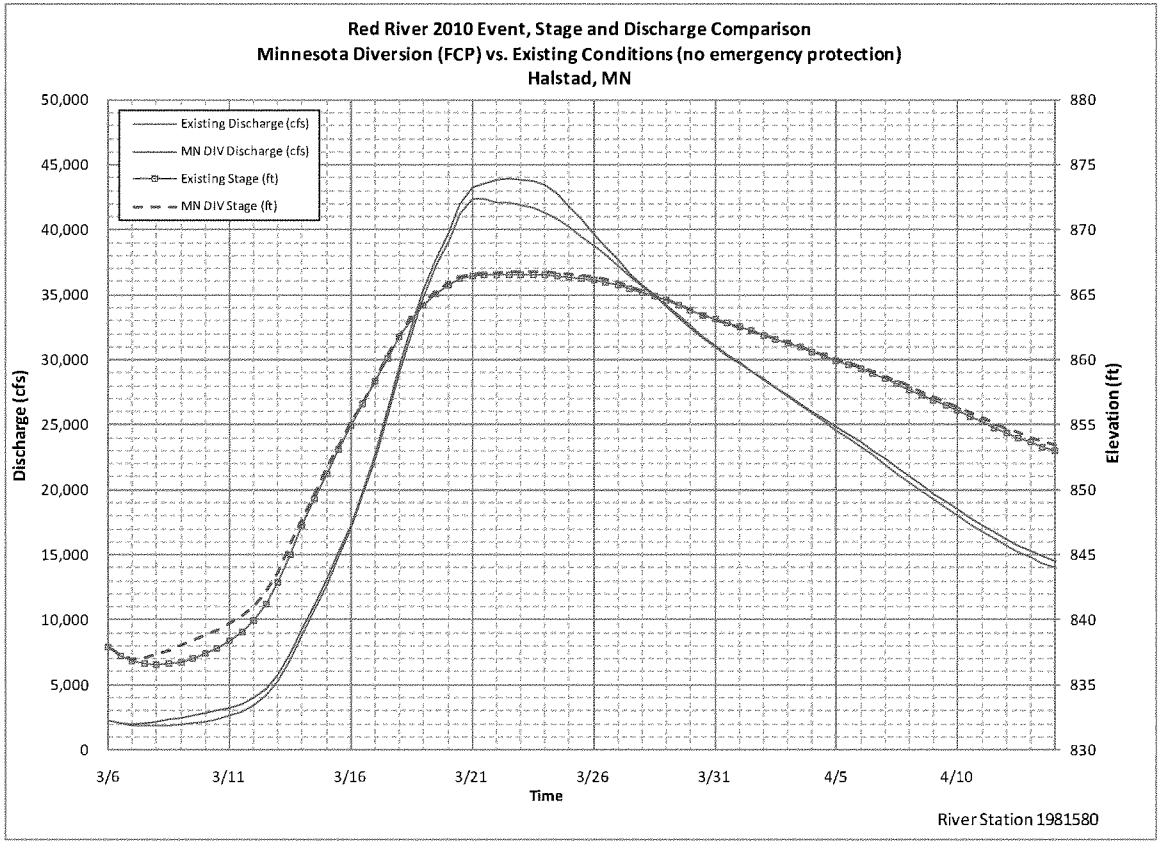

Figure C-E1- 92: 2010 Historical Flood Hydrographs for FCP @ Halstad

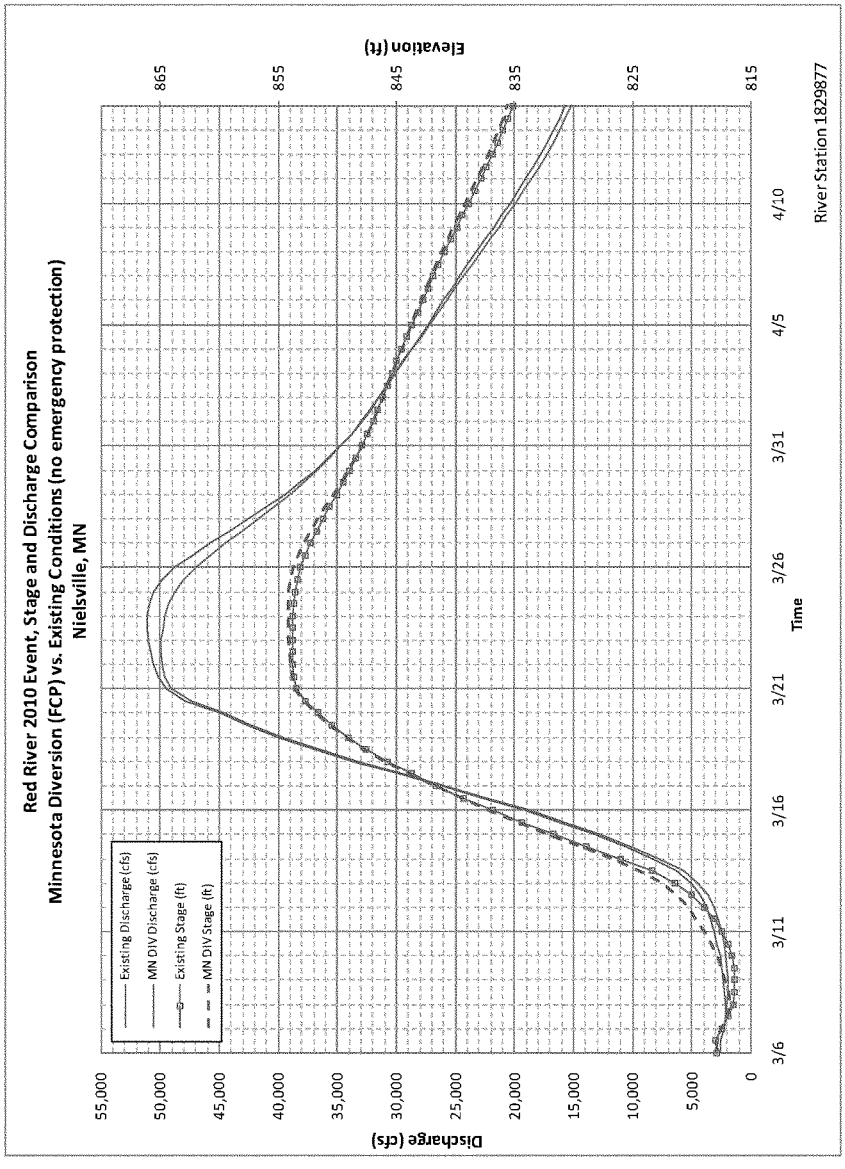

Figure C-E1- 93: 2010 Historical Flood Hydrographs for FCP @ Nielsville

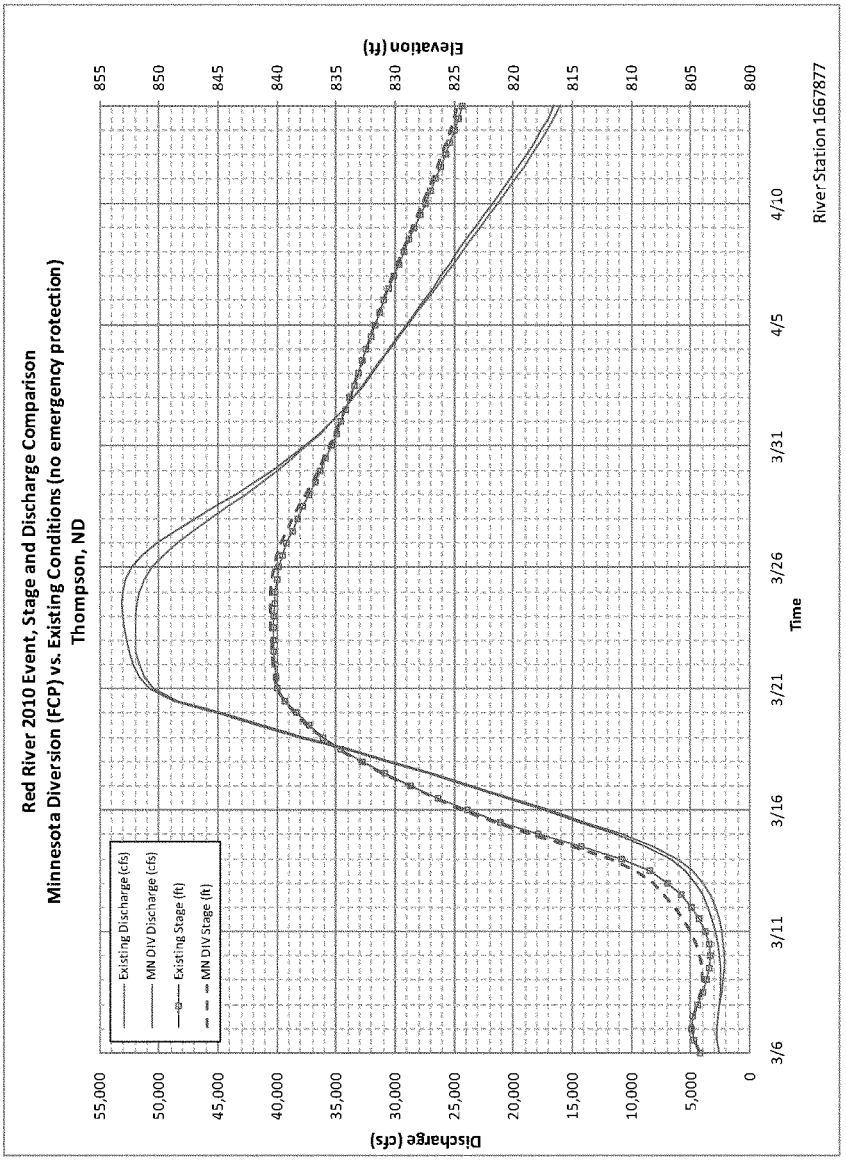

Figure C-E1- 94: 2010Historical Flood Hydrographs for FCP @ Thompson

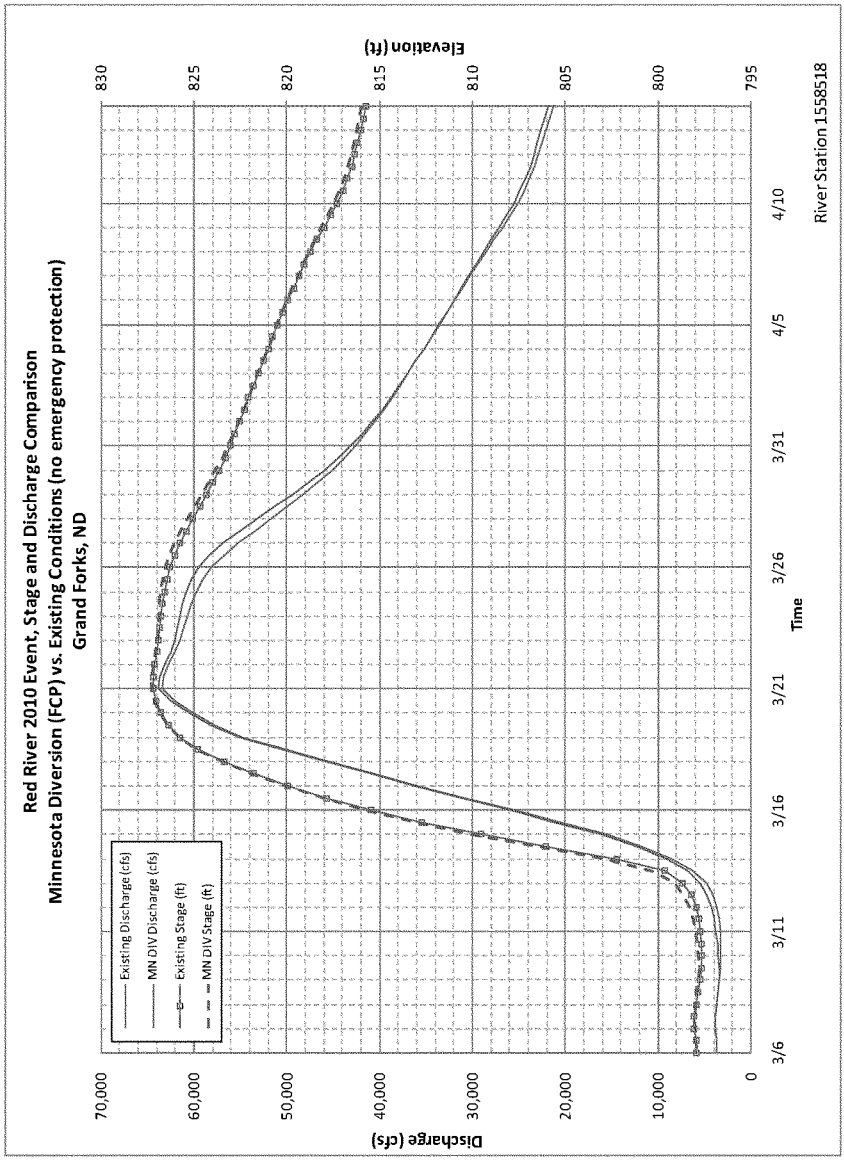

Figure C-E1- 95: 2010 Historical Flood Hydrographs for FCP @ Grand Forks

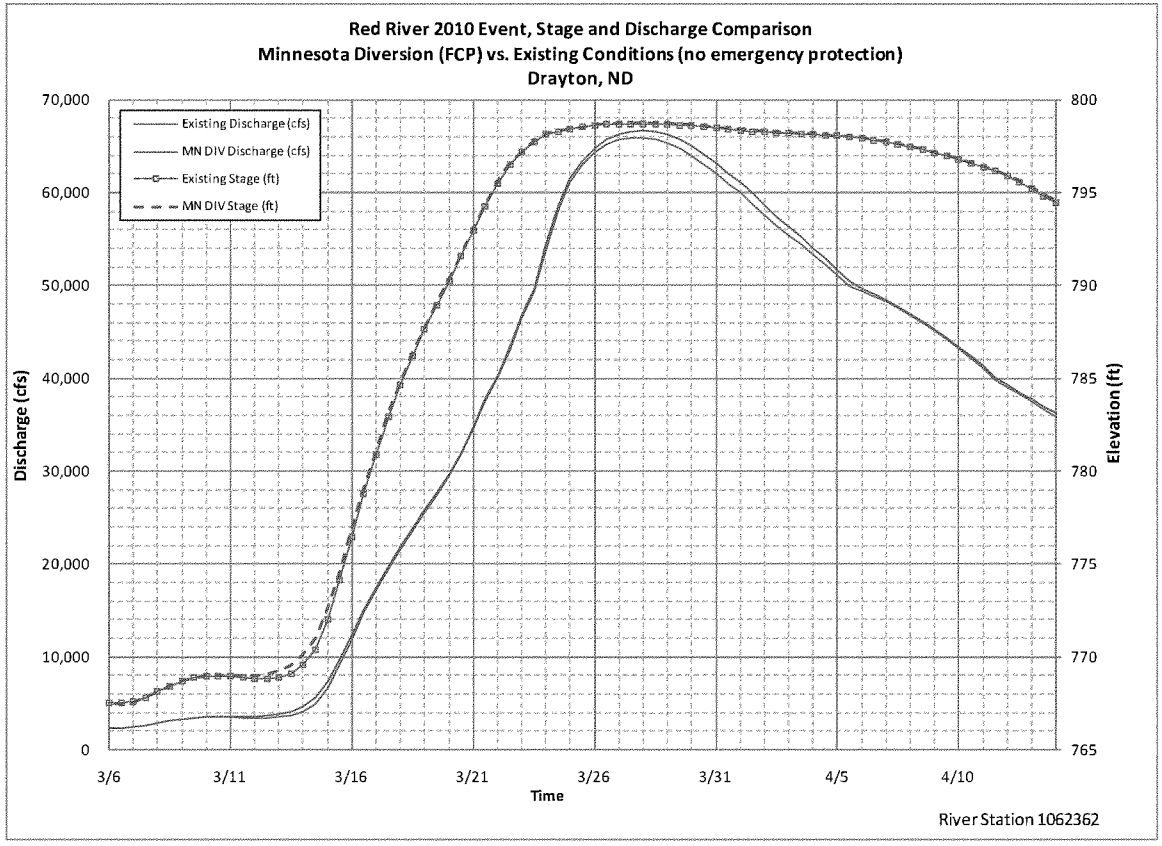

Figure C-E1- 96: 2010 Historical Flood Hydrographs for FCP @ Drayton

RED RIVER DIVERSION

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

APPENDIX C – HYDRAULICS WITH–PROJECT CONDITIONS

EXHIBIT 2 – LPP IMPACTS

Report for the US Army Corps of Engineers, and the cities of Fargo, North Dakota & Moorhead, Minnesota

FINAL: February 28, 2011

FIGURES

Figure C-E2- 1: 10-Percent Chance Impacts for LPP
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Figure C-E2- 44: 0.2-Percent Chance Hydrographs for LPP @ Fargo
Figure C-E2- 45: 0.2-Percent Chance Hydrographs for LPP @ Georgetown
Figure C-E2- 46: 0.2-Percent Chance Hydrographs for LPP @ Perley
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Figure C-E2- 95: 2010 Historical Flood Hydrographs for LPP @ Grand Forks
Figure C-E2- 96: 2010 Historical Flood Hydrographs for LPP @ Drayton

North Dakota Diversion (LPP) - 10% Chance Event								2-28-2011
Location	Station	Existing No Protection		ND Diversion (LPP)		Difference (ft) Project vs. Existing No Protection	
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)
Drayton Gage	1062362	798.53	62917	798.54	63042	0.01	125
ND SH#17/ MN SH317	1223286	800.78		800.80		0.02	
Co. Hwy 15	1315673	802.60		802.61		0.01	
Minimum Impact Location	1327581	803.44		803.45		0.01	
Oslo Gage	1416287	810.51		810.55		0.04	
DS Grand Forks Levees	1533523	823.07		823.15		0.08	
Grand Forks Gage	1558518	825.98	56662	826.09	57169	0.11	507
Maximum Impact Location	1561283	826.49		826.61		0.12	
32nd Ave, Grand Forks	1580152	827.76		827.87		0.11	
Thompson Gage	1667877	837.58	42815	837.62	42843	0.04	28
Co. Hwy 25/ Co. Rd 221	1726274	842.99		843.03		0.04	
DS Sandhill River/ Climax	1763746	845.62		845.65		0.03	
Nielsville	1829877	850.21	39667	850.24	39942	0.03	274
DS Marsh River	1864960	853.10		853.14		0.04	
US Goose River/ Shelly	1891054	855.90		855.93		0.03	
Halstad Gage	1981580	864.55	34653	864.43	34160	-0.12	-493
Hendrum	2038409	868.94	32267	868.69	31580	-0.25	-687
Perley	2129181	874.83	28448	874.29	26476	-0.54	-1972
Georgetown	2194021	880.34		879.91		-0.43	
North River/ Clay Co. Hwy 93	2305647	889.72		884.23		-5.49	
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	894.08		888.72		-5.36	
Fargo Gage (13th Ave S, 12th Ave S)	2388223	897.33 (34.59*)	17024	891.86 (29.12*)	10156	-5.47	-6868
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	901.61		896.12		-5.49	
US ND Wild Rice River	2484618	905.37		900.22		-5.15	
US LPP Diversion	2531315	908.06		916.29		8.23	
Hickson Gage	2563754	910.21	10428	916.80	10077	6.59	-351
Abercrombie	2764835	929.05	11278	929.16	11278	0.11	0

* Flood stage at USGS Gaging Station 05054000, Fargo, ND

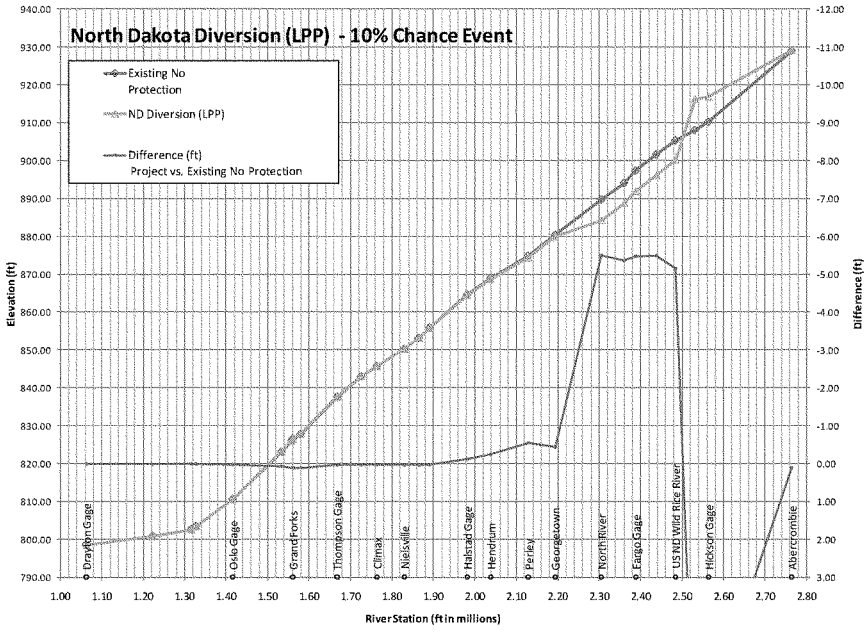


Figure C-E2- 1: 10-Percent Chance Impacts for LPP

North Dakota Diversion (LPP) - 2% Chance Event								2-28-2011
Location	Station	Existing No Protection		ND Diversion (FCP)		Difference (ft) Project vs. Existing No Protection	
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)
Drayton Gage	1062362	800.72	100869	800.80	102165	0.08	1296
ND SH#17/ MN SH317	1223286	803.19		803.26		0.07	
Co. Hwy 15	1315673	804.35		804.40		0.05	
Minimum Impact Location	1410241	811.12		811.15		0.03	
Oslo Gage	1416287	812.53		812.57		0.04	
DS Grand Forks Levees	1533523	826.86		826.97		0.11	
Grand Forks Gage	1558518	831.13	91118	831.31	92619	0.18	1501
32nd Ave, Grand Forks	1580152	833.57		833.85		0.28	
Maximum Impact Location	1602184	836.27		836.65		0.38	
Thompson Gage	1667877	844.83	69367	845.07	70104	0.24	737
Co. Hwy 25/ Co. Rd 221	1726274	851.40		851.61		0.21	
DS Sandhill River/ Climax	1763746	854.11		854.32		0.21	
Nielsville	1829877	858.57	63543	858.75	64333	0.18	791
DS Marsh River	1864960	860.76		860.92		0.16	
US Goose River/ Shelly	1891054	863.04		863.16		0.12	
Halstad Gage	1981580	867.99	59416	867.99	59542	0.00	126
Hendrum	2038409	872.82	56143	872.70	55381	-0.12	-762
Perley	2129181	877.75	51248	877.43	47559	-0.32	-3689
Georgetown	2194021	882.76		882.53		-0.23	
North River/ Clay Co. Hwy 93	2305647	892.97		886.22		-6.75	
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	898.21		889.86		-8.35	
Fargo Gage (13th Ave S, 12th Ave S)	2388223	902.6 (39.86*)	29167	892.72 (29.98*)	10603	-9.88	-18565
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	906.94		896.73		-10.21	
US ND Wild Rice River	2484618	910.11		900.70		-9.41	
US LPP Diversion	2531315	913.76		920.86		7.10	
Hickson Gage	2563754	916.34	18898	920.92	18428	4.58	-470
Abercrombie	2764835	934.48	20726	934.62	20726	0.14	0

* Flood stage at USGS Gaging Station 05054000, Fargo, ND

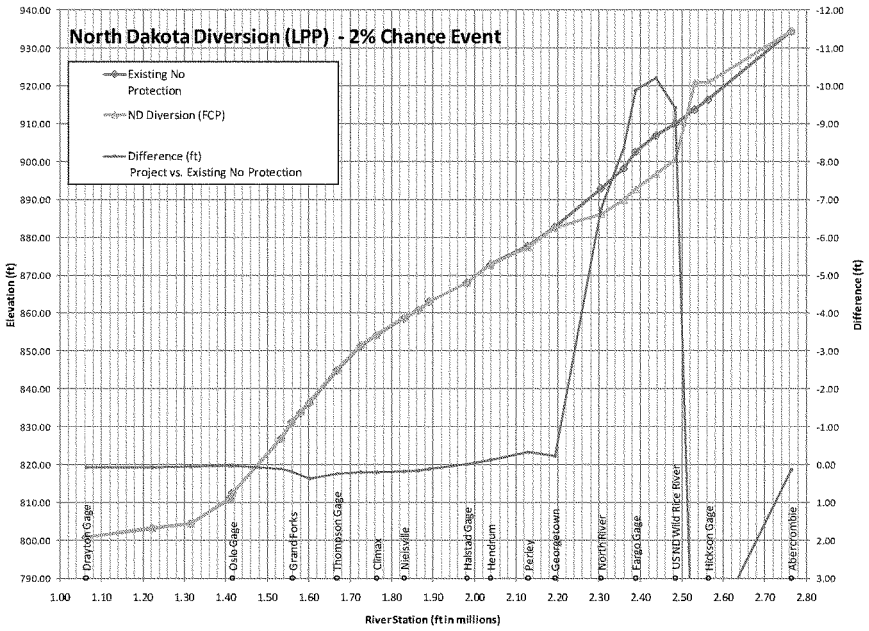


Figure C-E2- 2: 2-Percent Chance Impacts for LPP

North Dakota Diversion (LPP) - 1% Chance Event								2-28-2011
Location	Station	Existing No Protection		ND Diversion (LPP)		Difference (ft) Project vs. Existing No Protection	
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)
Drayton Gage	1062362	801.73	119255	801.81	120751	0.08	1496
ND SH#17/ MN SH317	1223286	804.09		804.16		0.07	
Co. Hwy 15	1315673	805.08		805.13		0.05	
Minimum Impact Location	1410241	811.47		811.51		0.04	
Oslo Gage	1416287	813.01		813.07		0.06	
DS Grand Forks Levees	1533523	827.98		828.13		0.15	
Grand Forks Gage	1558518	832.97	107980	833.21	110497	0.24	2517
Maximum Impact Location	1573768	835.27		835.56		0.29	
32nd Ave, Grand Forks	1580152	835.83		836.11		0.28	
Thompson Gage	1667877	847.35	82926	847.39	82608	0.04	-317
Co. Hwy 25/ Co. Rd 221	1726274	854.46		854.44		-0.02	
DS Sandhill River/ Climax	1763746	857.34		857.30		-0.04	
Nielsville	1829877	861.66	75745	861.62	76038	-0.04	293
DS Marsh River	1864960	863.43		863.40		-0.03	
US Goose River/ Shelly	1891054	865.36		865.32		-0.04	
Halstad Gage	1981580	869.09	71581	869.03	70992	-0.06	-589
Hendrum	2038409	873.75	67278	873.69	66095	-0.06	-1183
Perley	2129181	878.50	61723	878.22	57044	-0.28	-4679
Georgetown	2194021	883.36		883.11		-0.25	
North River/ Clay Co. Hwy 93	2305647	893.73		886.48		-7.25	
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	899.08		890.50		-8.58	
Fargo Gage (13th Ave S, 12th Ave S)	2388223	903.86 (41.12*)	34875	893.54 (30.8*)	11718	-10.32	-23157
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	907.71		897.66		-10.05	
US ND Wild Rice River	2484618	910.70		901.71		-8.99	
US LPP Diversion	2531315	914.65		922.88		8.23	
Hickson Gage	2563754	917.52	21730	922.90	18655	5.38	-3075
Abercrombie	2764835	935.62	23000	935.73	23000	0.11	0

* Flood stage at USGS Gaging Station 05054000, Fargo, ND

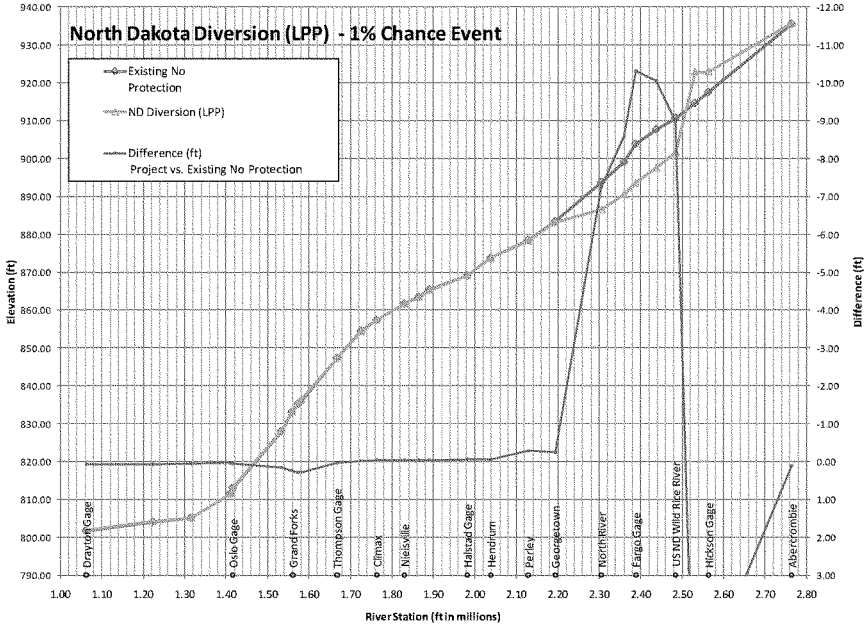


Figure C-E2- 3: 1-Percent Chance Impacts for LPP

North Dakota Diversion (LPP) - 0.2% Chance Event								2-28-2011
Location	Station	Existing No Protection		ND Diversion (FCP)		Difference (ft) Project vs. Existing No Protection	
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)
Drayton Gage	1062362	804.12	168364	804.23	171002	0.11	2638
ND SH#17/ MN SH317	1223286	805.99		806.06		0.07	
Co. Hwy 15	1315673	806.74		806.83		0.09	
Minimum Impact Location	1410241	812.15		812.19		0.04	
Oslo Gage	1416287	813.88		813.93		0.05	
DS Grand Forks Levees	1533523	829.92		830.04		0.12	
Grand Forks Gage	1558518	836.36	146225	836.58	149112	0.22	2887
Maximum Impact Location	1561353	838.53		838.80		0.27	
32nd Ave, Grand Forks	1580152	839.75		839.98		0.23	
Thompson Gage	1667877	850.69	112422	850.64	111394	-0.05	-1027
Co. Hwy 25/ Co. Rd 221	1726274	859.38		859.26		-0.12	
DS Sandhill River/ Climax	1763746	862.75		862.60		-0.15	
Nielsville	1829877	867.04	107296	866.88	105953	-0.16	-1344
DS Marsh River	1864960	868.06		867.92		-0.14	
US Goose River/ Shelly	1891054	869.30		869.17		-0.13	
Halstad Gage	1981580	871.54	101754	871.32	92746	-0.22	-9007
Hendrum	2038409	875.77	97650	875.47	90871	-0.30	-6779
Perley	2129181	879.89	90756	879.53	79857	-0.36	-10899
Georgetown	2194021	884.48		884.15		-0.33	
North River/ Clay Co. Hwy 93	2305647	895.35		892.96		-2.39	
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	900.31		898.32		-1.99	
Fargo Gage (13th Ave S, 12th Ave S)	2388223	905.8 (43.06*)	61717	902.77 (40.03*)	29865	-3.03	-31852
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	909.13		907.08		-2.05	
US ND Wild Rice River	2484618	911.54		910.23		-1.31	
US LPP Diversion	2531315	915.94		922.44		6.50	
Hickson Gage	2563754	919.69	35636	922.54	32491	2.85	-3145
Abercrombie	2764835	940.90	44308	940.91	44308	0.01	0

* Flood stage at USGS Gaging Station 05054000, Fargo, ND

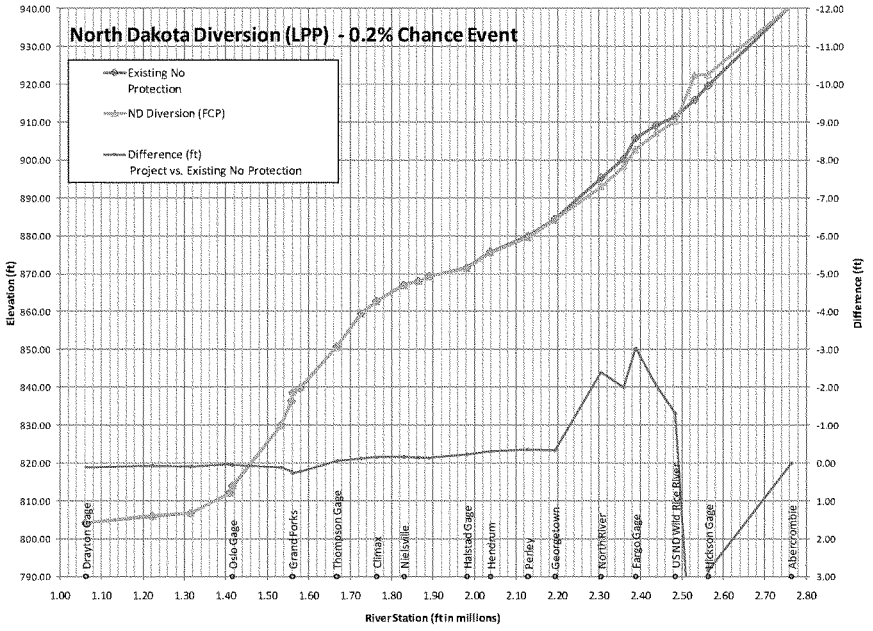


Figure C-E2- 4: 0.2-Percent Chance Impacts for LPP

North Dakota Diversion (LPP) -1997 Event (No Protection)								2-28-2011
Location	Station	Existing No Protection		ND Diversion (LPP)		Difference (ft) Project vs. Existing No Protection	
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)
Drayton Gage	1062362	801.95	123404	801.94	123251	-0.01	-153
ND SH#17/ MN SH317	1223286	804.31		804.30		-0.01	
Co. Hwy 15	1315673	805.30		805.30		0.00	
Oslo Gage	1416287	813.29		813.30		0.01	
Minimum Impact Location	1555329	833.59		833.60		0.01	
DS Grand Forks Levees	1533523	828.62		828.63		0.01	
Grand Forks Gage	1558518	834.04	119103	834.05	119142	0.01	39
32nd Ave, Grand Forks	1580152	837.02		837.05		0.03	
Thompson Gage	1667877	847.29	78351	847.43	79439	0.14	1088
Co. Hwy 25/ Co. Rd 221	1726274	853.81		854.02		0.21	
DS Sandhill River/ Climax	1763746	856.49		856.72		0.23	
Maximum Impact Location	1829877	860.86		861.11		0.25	
Nielsville	1829877	860.86	71728	861.11	72925	0.25	1197
DS Marsh River	1864960	862.72		862.93		0.21	
US Goose River/ Shelly	1891054	864.75		864.93		0.18	
Halstad Gage	1981580	868.65	64821	868.78	66780	0.13	1959
Hendrum	2038409	873.24	55101	873.38	54197	0.14	-903
Perley	2129181	878.05	54689	878.16	56314	0.11	1625
Georgetown	2194021	882.92		883.03		0.11	
North River/ Clay Co. Hwy 93	2305647	893.04		888.08		-4.96	
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	898.26		891.01		-7.25	
Fargo Gage (13th Ave S, 12th Ave S)	2388223	902.42 (39.68*)	27574	893.11 (30.37*)	9968	-9.31	-17606
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	906.49		896.06		-10.43	
US ND Wild Rice River	2484618	909.37		900.01		-9.36	
US LPP Diversion	2531315	911.89		921.60		9.71	
Hickson Gage	2563754	913.85	13729	921.63	13235	7.78	-494
Abercrombie	2764835	931.08	13995	931.36	13995	0.28	0

* Flood stage at USGS Gaging Station 05054000, Fargo, ND

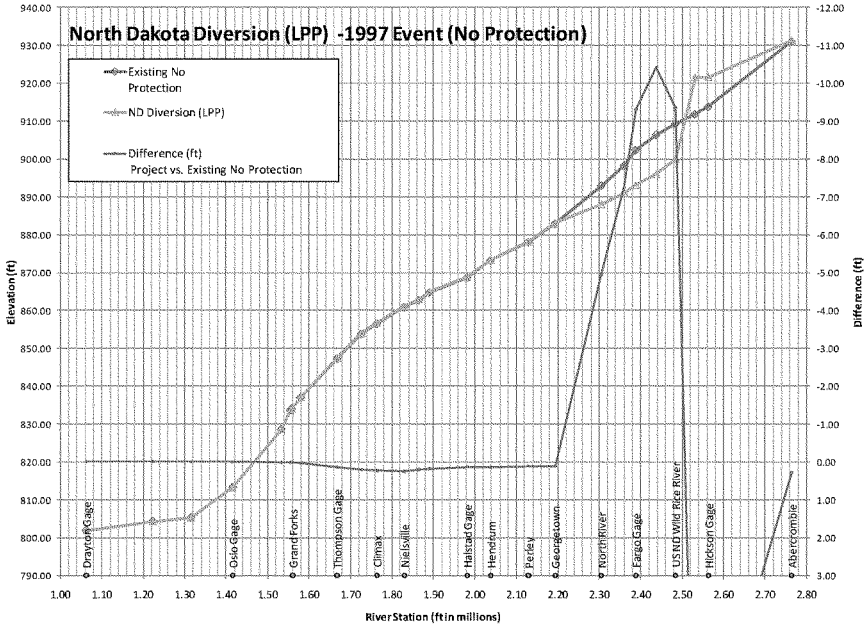


Figure C-E2- 5: 1997 Historical Flood Impacts for LPP

North Dakota Diversion (LPP) -2006 Event (No Protection)								2-28-2011
Location	Station	Existing No Protection		ND Diversion (LPP)		Difference (ft) Project vs. Existing No Protection	
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)
Drayton Gage	1062362	799.44	78252	799.46	78666	0.02	414
ND SH#17/ MN SH317	1223286	801.84		801.87		0.03	
Co. Hwy 15	1315673	803.39		803.41		0.02	
Oslo Gage	1416287	811.58		811.61		0.03	
Minimum Impact Location	1443147	813.86		813.88		0.02	
DS Grand Forks Levees	1533523	825.11		825.18		0.07	
Grand Forks Gage	1558518	828.63	72782	828.72	73387	0.09	605
32nd Ave, Grand Forks	1580152	830.60		830.71		0.11	
Thompson Gage	1667877	840.63	52499	840.84	53273	0.21	775
Co. Hwy 25/ Co. Rd 221	1726274	846.54		846.79		0.25	
Maximum Impact Location	1749702	848.33		848.59		0.26	
DS Sandhill River/ Climax	1763746	849.17		849.41		0.24	
Nielsville	1829877	853.70	48036	853.95	48880	0.25	844
DS Marsh River	1864960	856.33		856.58		0.25	
US Goose River/ Shelly	1891054	859.08		859.31		0.23	
Halstad Gage	1981580	866.64	43060	866.70	43552	0.06	492
Hendrum	2038409	870.74	36229	870.71	33849	-0.03	-2381
Perley	2129181	876.40	36254	876.09	33619	-0.31	-2635
Georgetown	2194021	881.37		881.14		-0.23	
North River/ Clay Co. Hwy 93	2305647	891.46		884.49		-6.97	
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	895.97		888.87		-7.10	
Fargo Gage (13th Ave S, 12th Ave S)	2388223	899.57 (36.83*)	21028	891.96 (29.22*)	10109	-7.61	-10919
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	903.97		896.16		-7.81	
US ND Wild Rice River	2484618	907.56		900.26		-7.30	
US LPP Diversion	2531315	910.60		918.72		8.12	
Hickson Gage	2563754	913.11	14313	918.90	14362	5.79	49
Abercrombie	2764835	931.58	15027	931.74	15027	0.16	0

* Flood stage at USGS Gaging Station 05054000, Fargo, ND

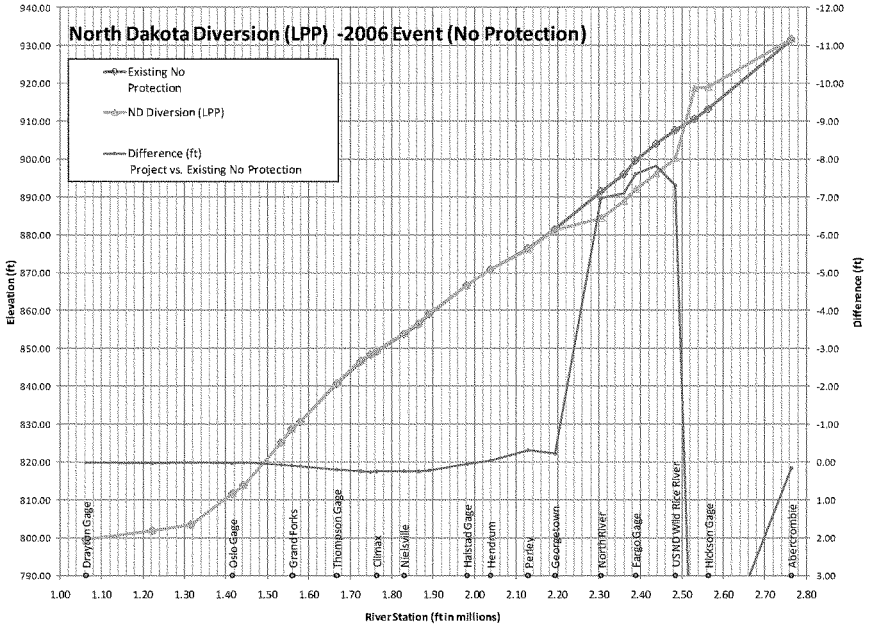


Figure C-E2- 6: 2006 Historical Flood Impacts for LPP

North Dakota Diversion (LPP) -2009 Event (No Protection)								2-28-2011
Location	Station	Existing No Protection		ND Diversion (LPP)		Difference (ft) Project vs. Existing No Protection	
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)
Drayton Gage	1062362	799.85	85308	799.84	85166	-0.01	-143
ND SH#17/ MN SH317	1223286	802.36		802.35		-0.01	
Co. Hwy 15	1315673	803.81		803.80		-0.01	
Minimum Impact Location	1345544	805.87		805.88		0.01	
Oslo Gage	1416287	812.02		812.04		0.02	
DS Grand Forks Levees	1533523	825.65		825.70		0.05	
Grand Forks Gage	1558518	829.33	77165	829.39	77550	0.06	385
Maximum Impact Location	1561353	830.20		830.28		0.08	
32nd Ave, Grand Forks	1580152	831.80		831.86		0.06	
Thompson Gage	1667877	843.05	61510	843.07	61577	0.02	67
Co. Hwy 25/ Co. Rd 221	1726274	849.33		849.35		0.02	
DS Sandhill River/ Climax	1763746	851.96		851.98		0.02	
Nielsville	1829877	856.67	58164	856.67	58406	0.00	242
DS Marsh River	1864960	859.13		859.13		0.00	
US Goose River/ Shelly	1891054	861.73		861.72		-0.01	
Halstad Gage	1981580	867.60	55176	867.56	54910	-0.04	-266
Hendrum	2038409	872.25	44075	872.15	41266	-0.10	-2810
Perley	2129181	877.21	45715	876.99	42804	-0.22	-2911
Georgetown	2194021	882.32		882.00		-0.32	
North River/ Clay Co. Hwy 93	2305647	892.95		886.79		-6.16	
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	898.28		890.52		-7.76	
Fargo Gage (13th Ave S, 12th Ave S)	2388223	902.66 (39.92*)	29234	893.46 (30.72*)	11561	-9.20	-17674
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	906.96		897.50		-9.46	
US ND Wild Rice River	2484618	910.20		901.39		-8.81	
US LPP Diversion	2531315	914.24		921.62		7.38	
Hickson Gage	2563754	917.76	24393	921.64	24562	3.88	170
Abercrombie	2764835	937.51	28176	937.59	28176	0.08	0

* Flood stage at USGS Gaging Station 05054000, Fargo, ND

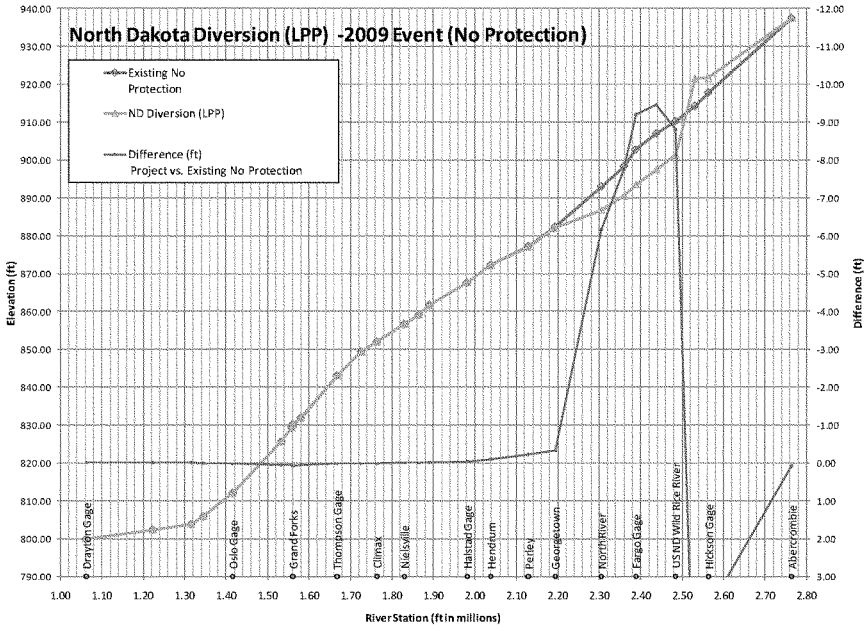


Figure C-E2- 7: 2009 Historical Flood Impacts for LPP

North Dakota Diversion (LPP) -2010 Event (No Protection)								2-28-2011
Location	Station	Existing No Protection		ND Diversion (LPP)		Difference (ft) Project vs. Existing No Protection	
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)
Drayton Gage	1062362	798.71	65928	798.72	66106	0.01	177
ND SH#17/ MN SH317	1223286	801.04		801.05		0.01	
Co. Hwy 15	1315673	802.96		802.97		0.01	
Minimum Impact Location	1327581	803.80		803.81		0.01	
Oslo Gage	1416287	811.09		811.07		-0.02	
DS Grand Forks Levees	1533523	824.07		824.04		-0.03	
Grand Forks Gage	1558518	827.23	63406	827.19	63172	-0.04	-235
32nd Ave, Grand Forks	1580152	829.35		829.31		-0.04	
Thompson Gage	1667877	840.28	52023	840.44	52694	0.16	672
Co. Hwy 25/ Co. Rd 221	1726274	846.28		846.48		0.20	
DS Sandhill River/ Climax	1763746	848.95		849.15		0.20	
Nielsville	1829877	853.84	49914	854.07	50734	0.23	820
DS Marsh River	1864960	856.61		856.83		0.22	
US Goose River/ Shelly	1891054	859.37		859.59		0.22	
Halstad Gage	1981580	866.55	42389	866.70	43585	0.15	1196
Maximum Impact Location	2038409	870.62		870.86		0.24	
Hendrum	2038409	870.62	38264	870.86	39350	0.24	1085
Perley	2129181	876.31	35431	876.36	35863	0.05	433
Georgetown	2194021	881.30		881.41		0.11	
North River/ Clay Co. Hwy 93	2305647	891.54		885.66		-5.88	
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	896.11		889.49		-6.62	
Fargo Gage (13th Ave S, 12th Ave S)	2388223	899.77 (37.03*)	21481	892.38 (29.64*)	10291	-7.39	-11190
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	904.15		896.38		-7.77	
US ND Wild Rice River	2484618	907.59		900.30		-7.29	
US LPP Diversion	2531315	910.17		918.90		8.73	
Hickson Gage	2563754	912.23	12677	918.98	12686	6.75	8
Abercrombie	2764835	930.57	13236	930.74	13236	0.17	0

* Flood stage at USGS Gaging Station 05054000, Fargo, ND

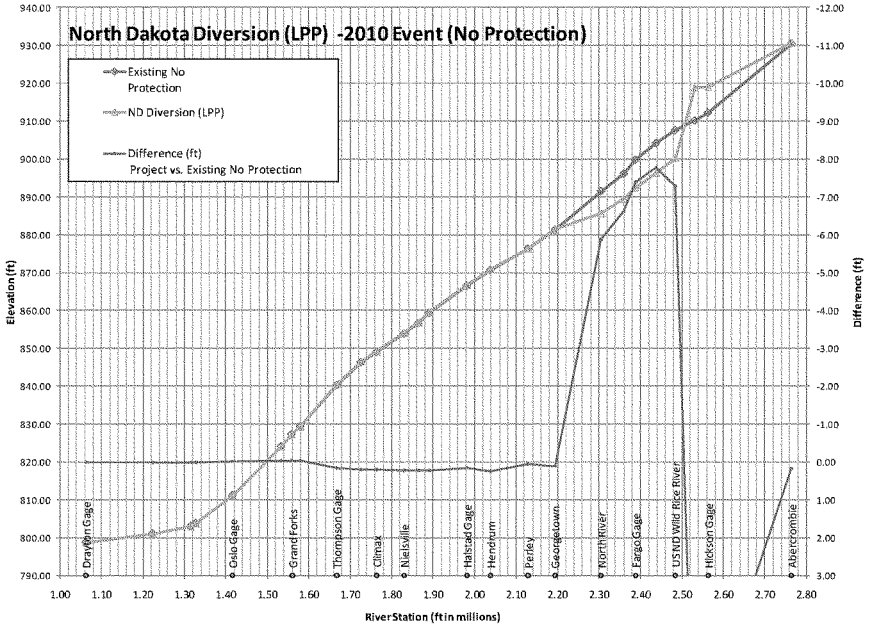


Figure C-E2- 8: 2010 Historical Flood Impacts for LPP

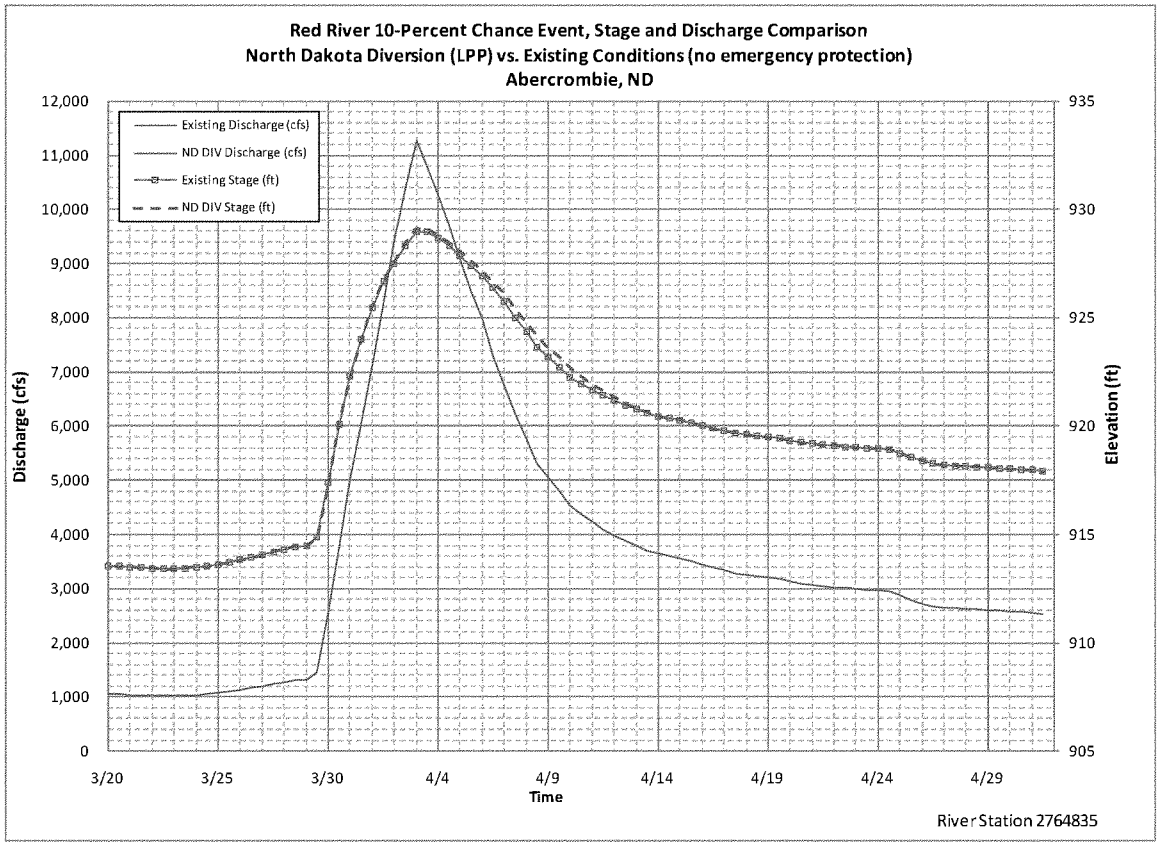


Figure C-E2- 9: 10-Percent Chance Hydrographs for LPP @ Abercrombie

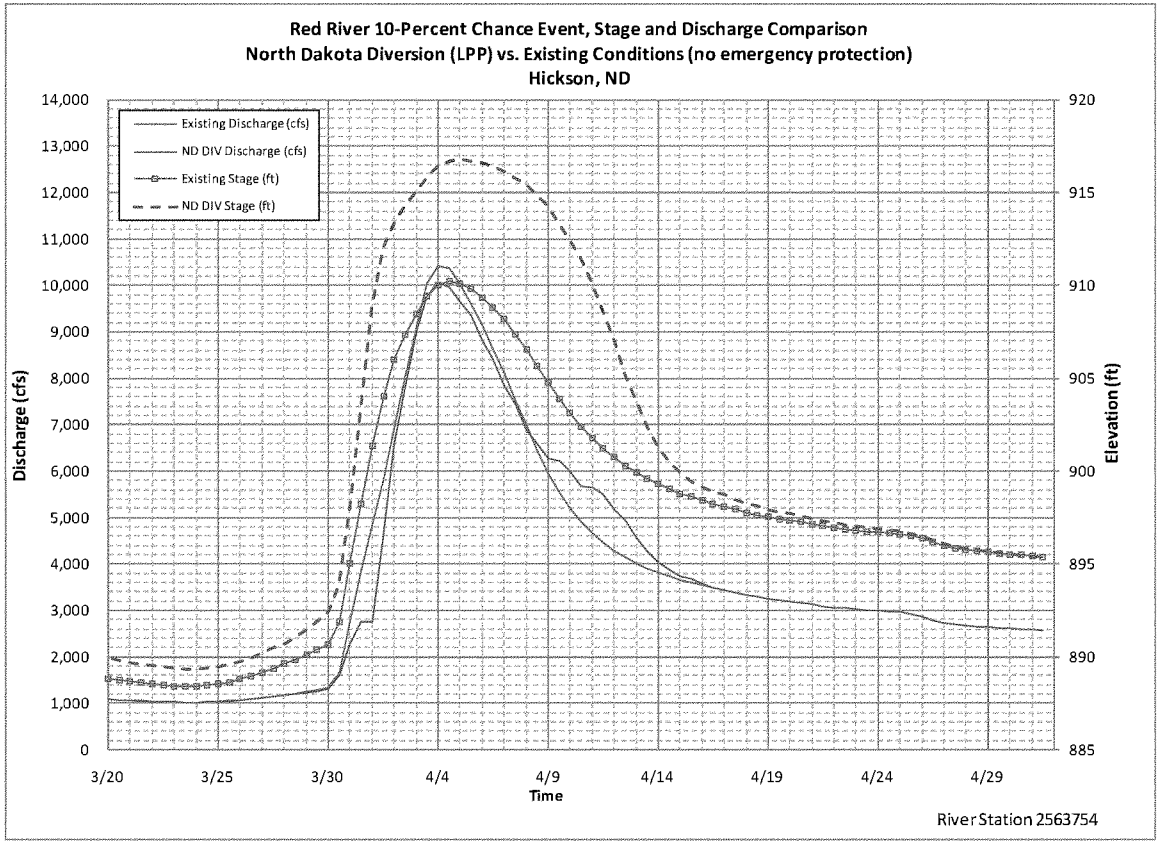


Figure C-E2- 10: 10-Percent Chance Hydrographs for LPP @ Hickson

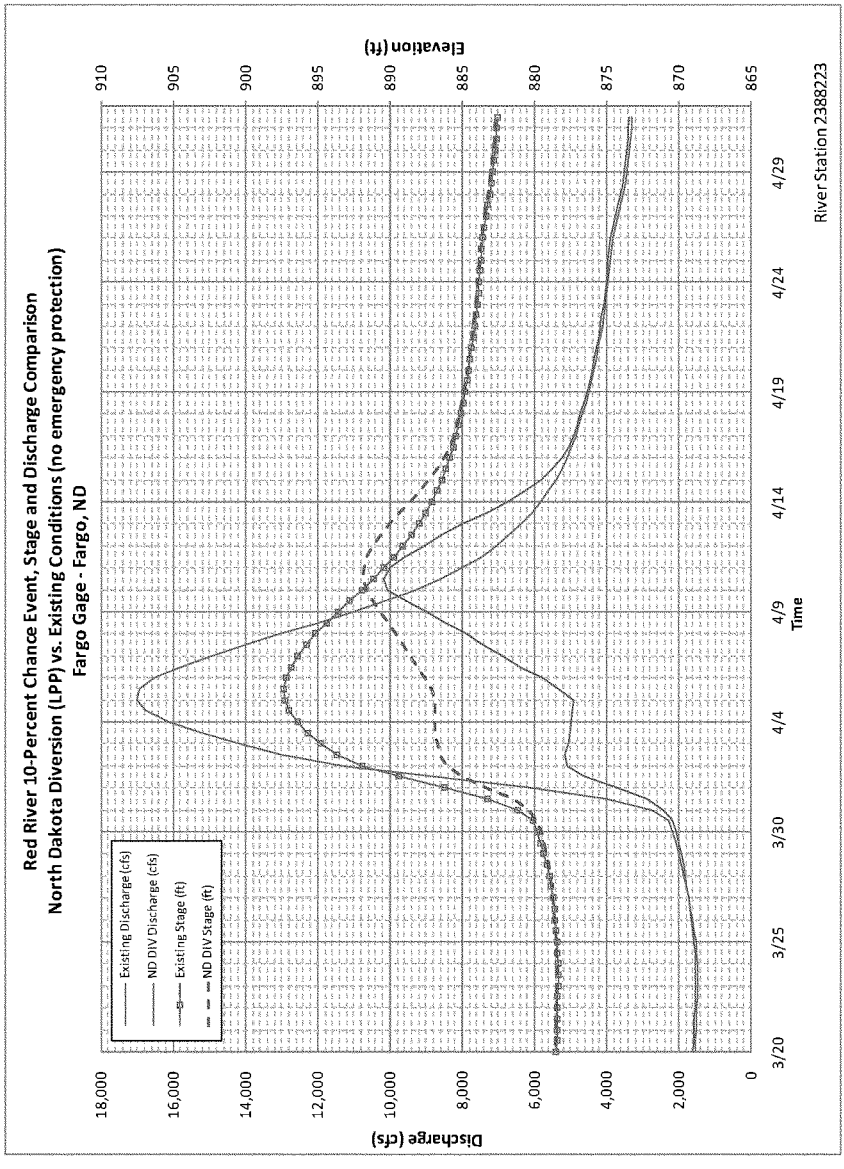


Figure C-E2- 11: 10-Percent Chance Hydrographs for LPP @ Fargo

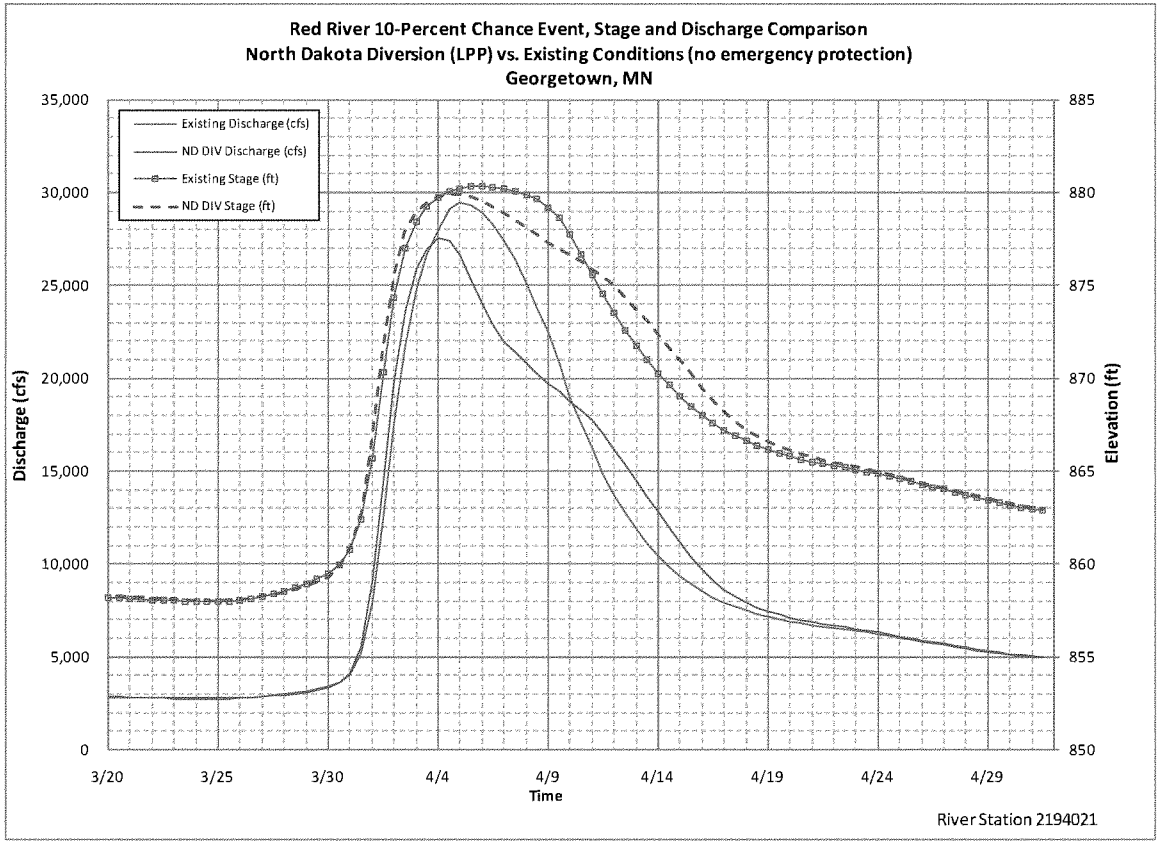


Figure C-E2- 12: 10-Percent Chance Hydrographs for LPP @ Georgetown

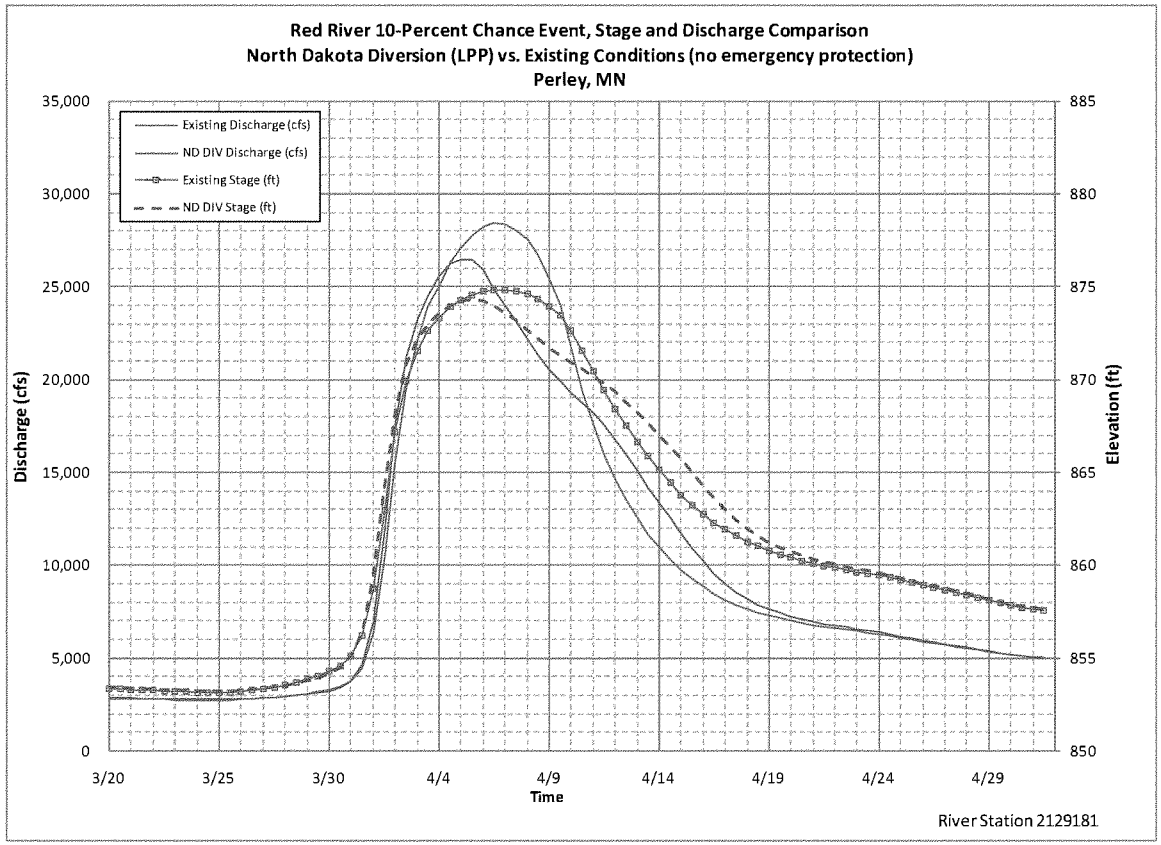
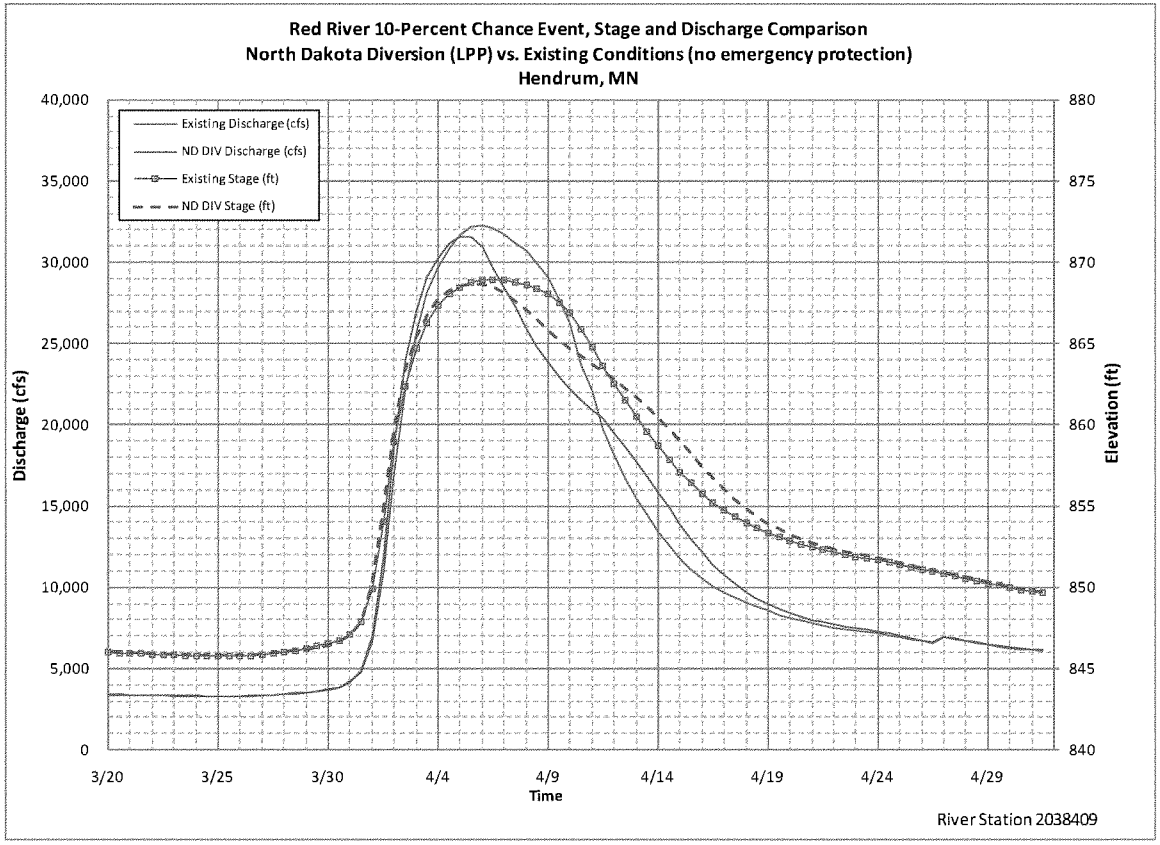


Figure C-E2- 13: 10-Percent Chance Hydrographs for LPP @ Perley



Fignre C-E2- 14: 10-Percent Chance Hydrographs for LPP @ Hendrum

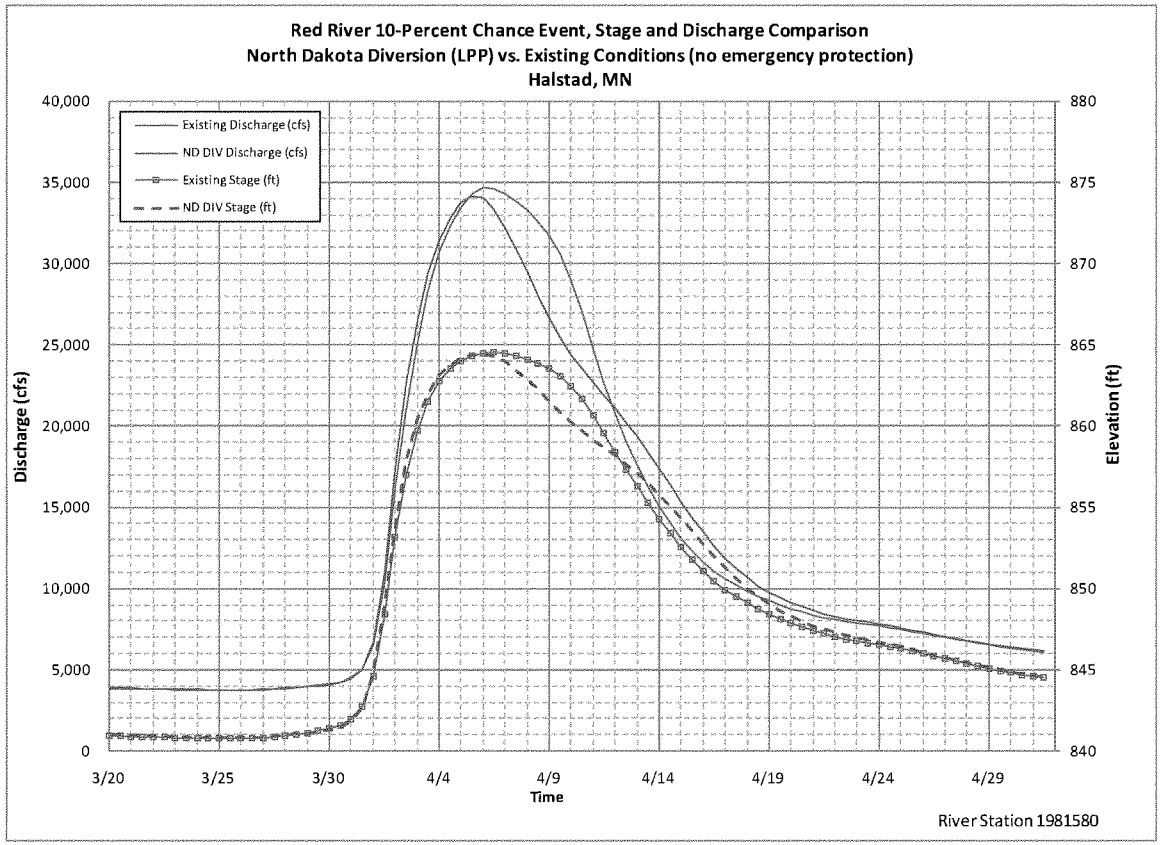


Figure C-E2- 15: 10-Percent Chance Hydrographs for LPP @ Halstad

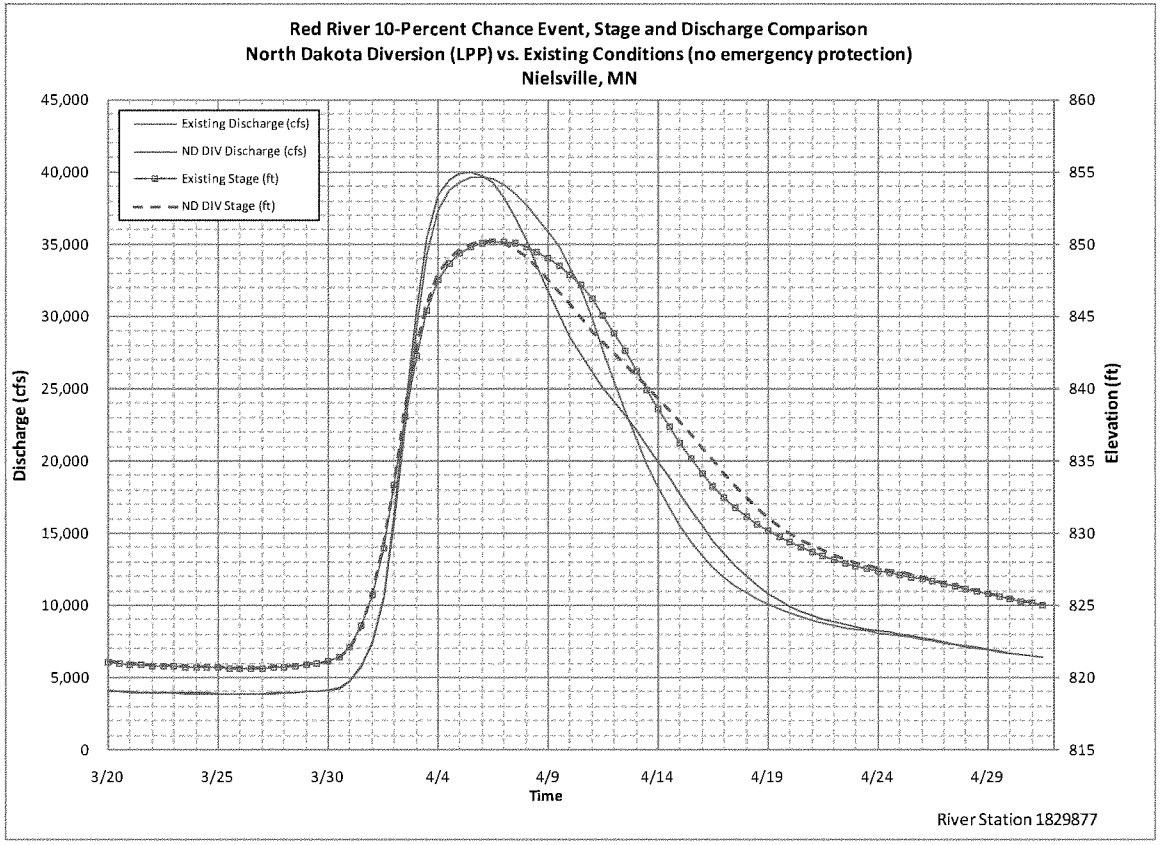


Figure C-E2- 16: 10-Percent Chance Hydrographs for LPP @ Nielsville

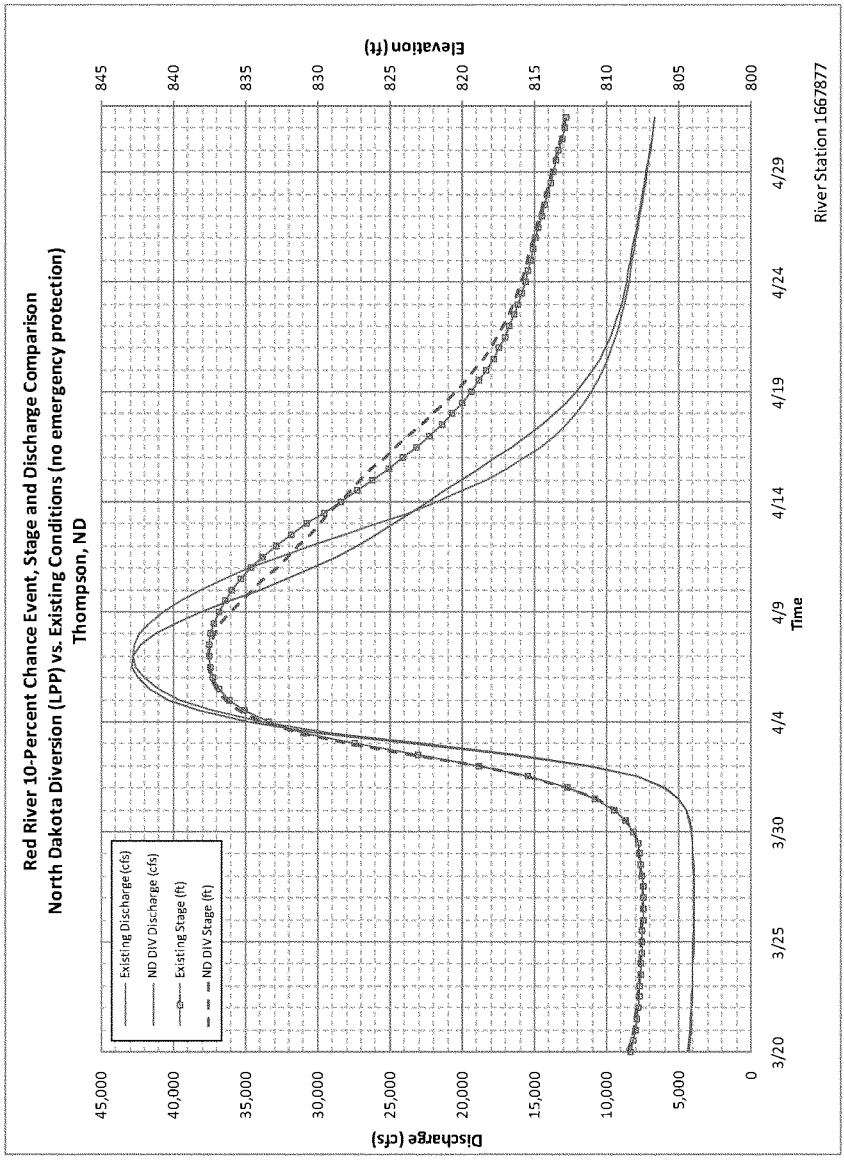


Figure C-E2- 17: 10-Percent Chance Hydrographs for LPP @ Thompson

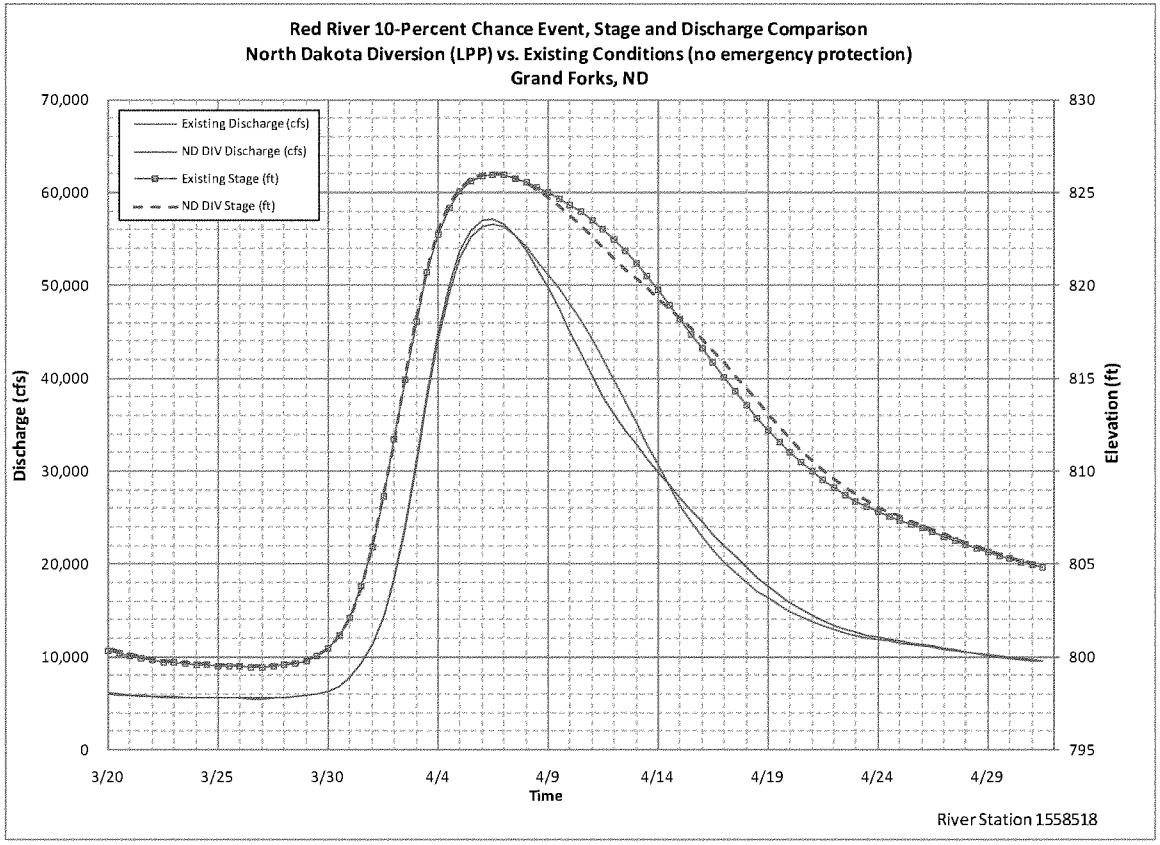


Figure C-E2- 18: 10-Percent Chance Hydrographs for LPP @ Grand Forks

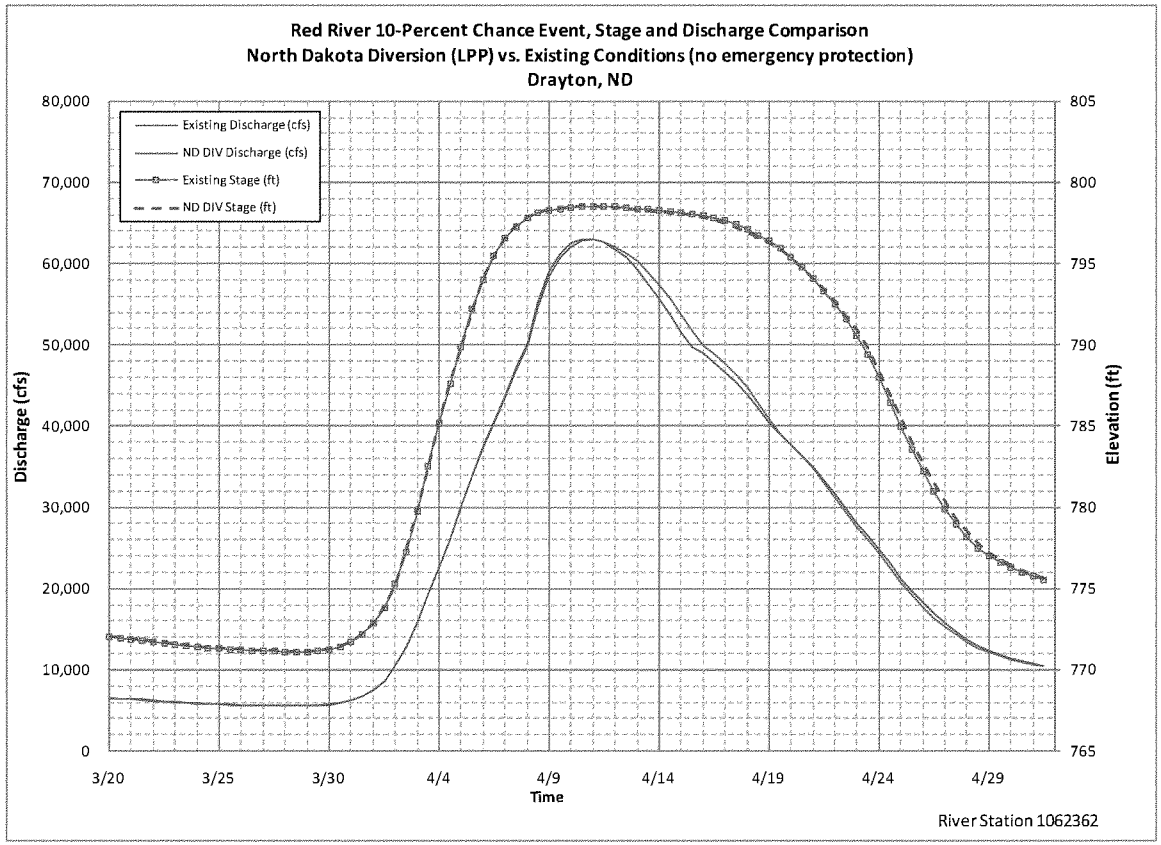


Figure C-E2- 19: 10-Percent Chance Hydrographs for LPP @ Drayton

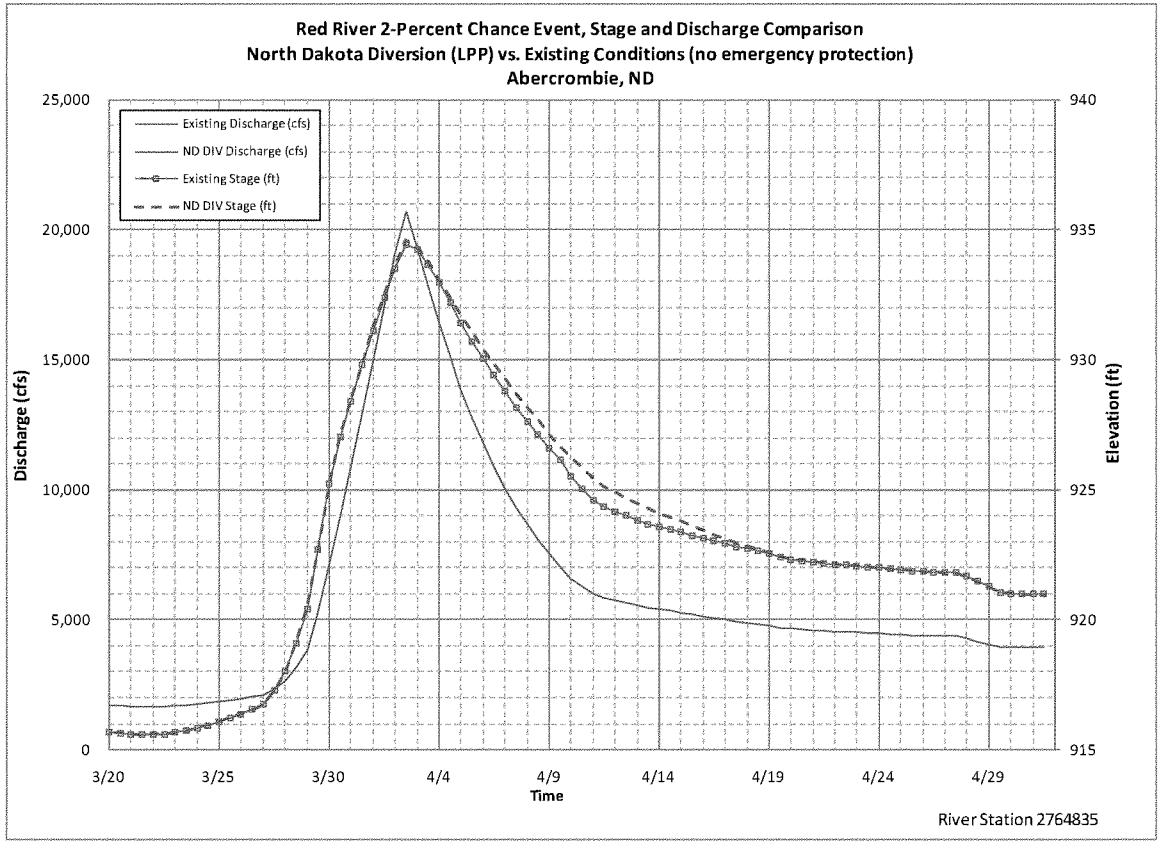


Figure C-E2- 20: 2-Percent Chance Hydrographs for LPP @ Abercrombie

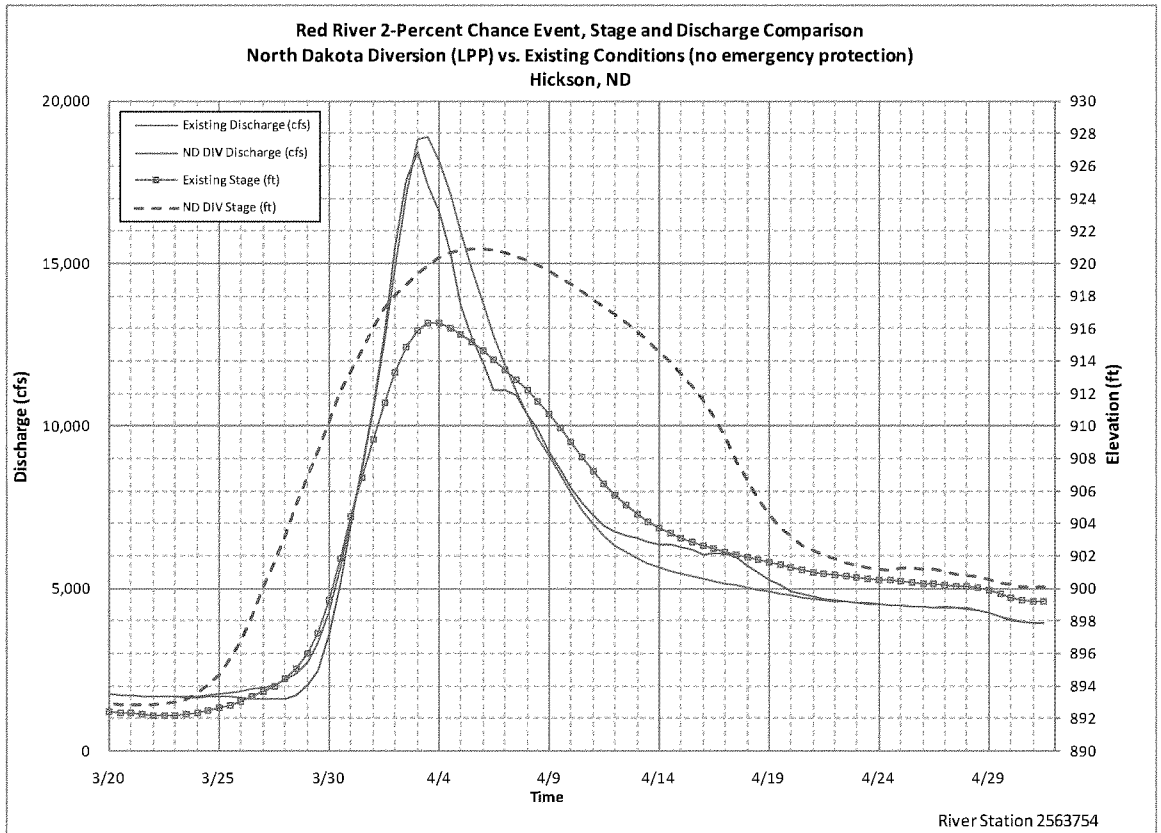


Figure C-E2- 21: 2-Percent Chance Hydrographs for LPP @ Hickson

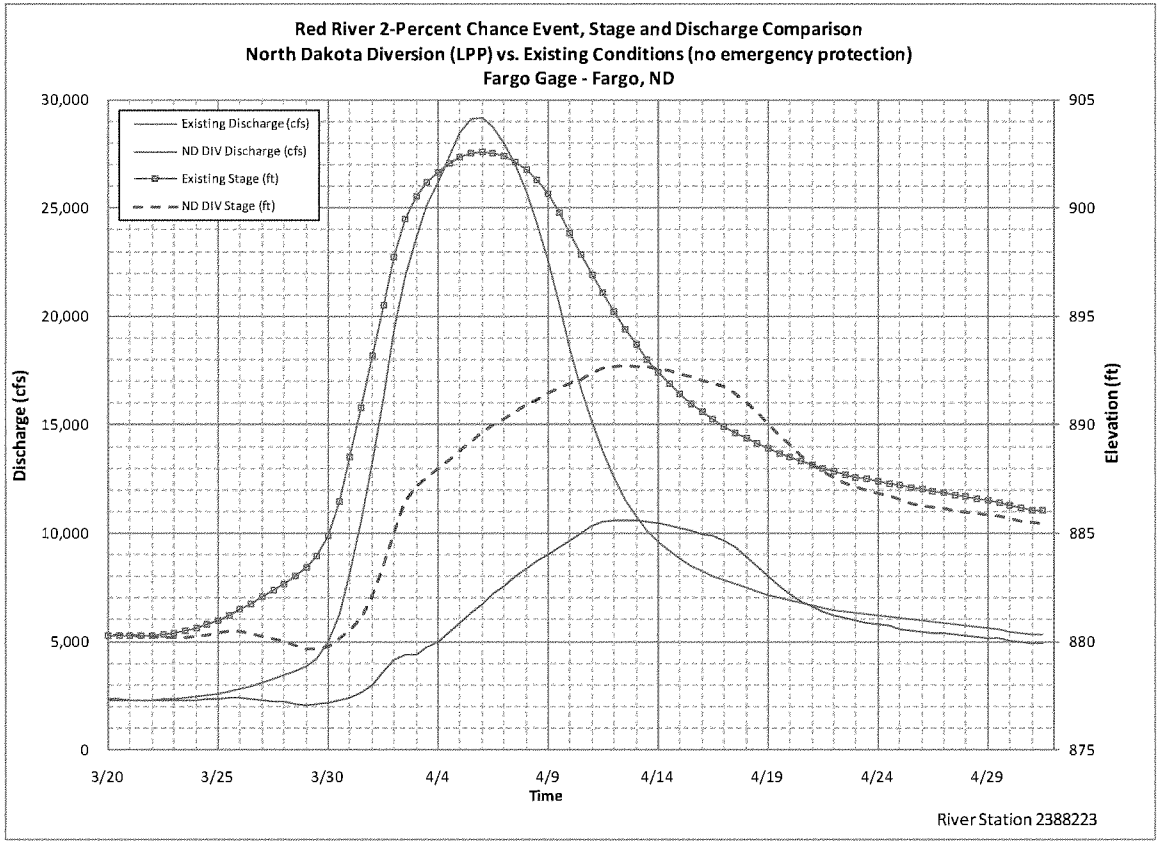


Figure C-E2- 22: 2-Percent Chance Hydrographs for LPP @ Fargo

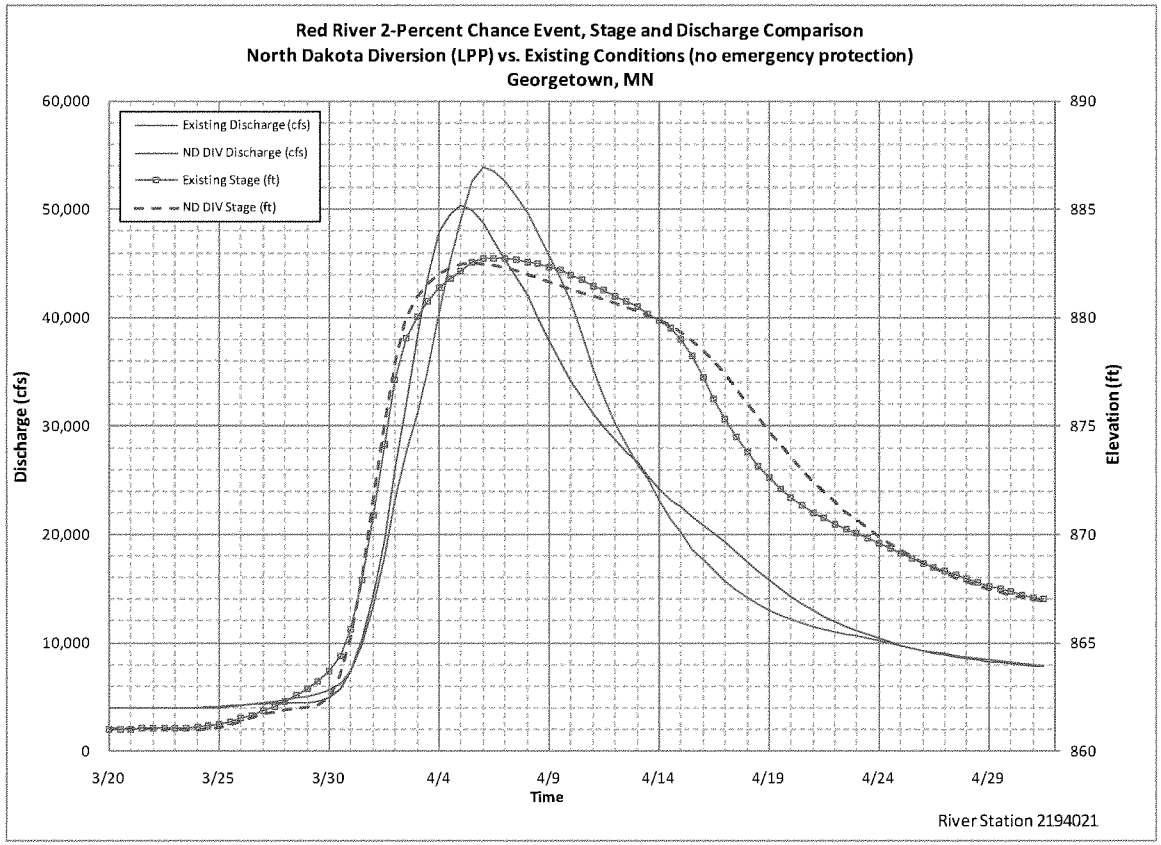


Figure C-E2- 23: 2-Percent Chance Hydrographs for LPP @ Georgetown

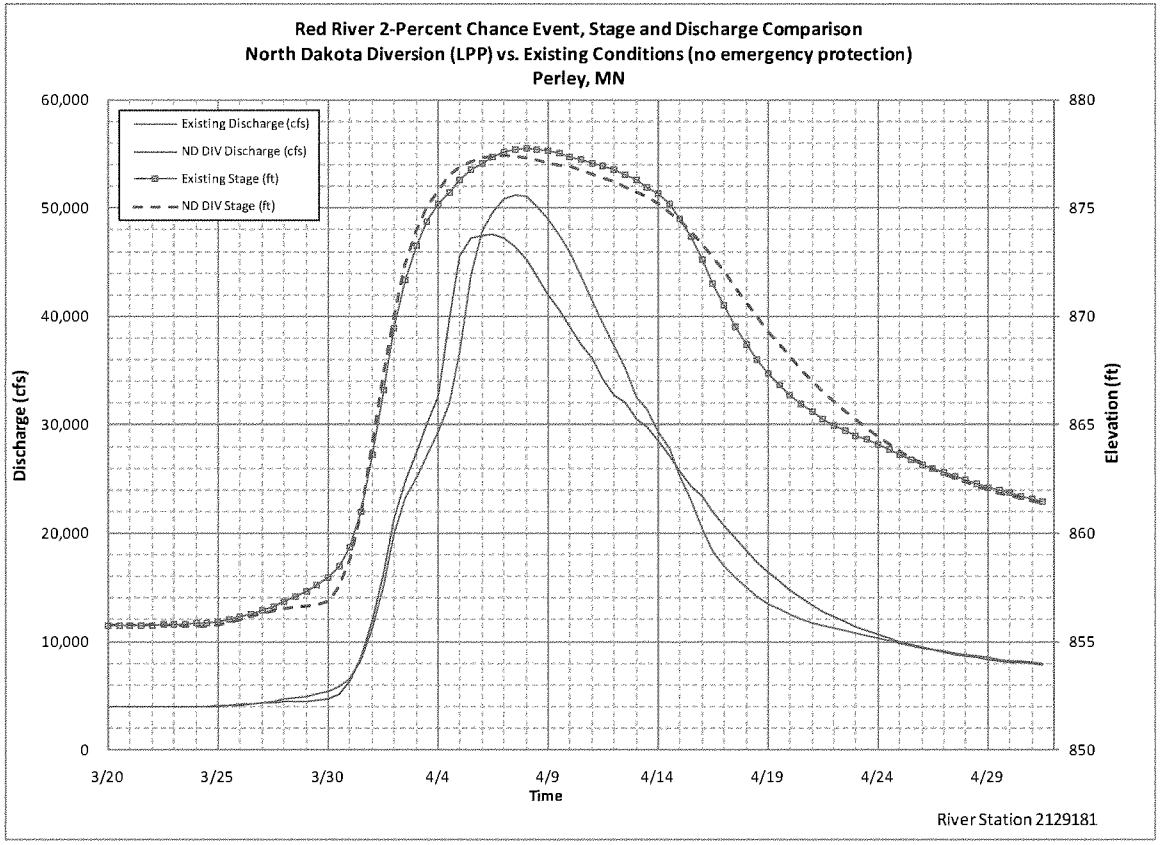


Figure C-E2- 24: 2-Percent Chance Hydrographs for LPP @ Perley

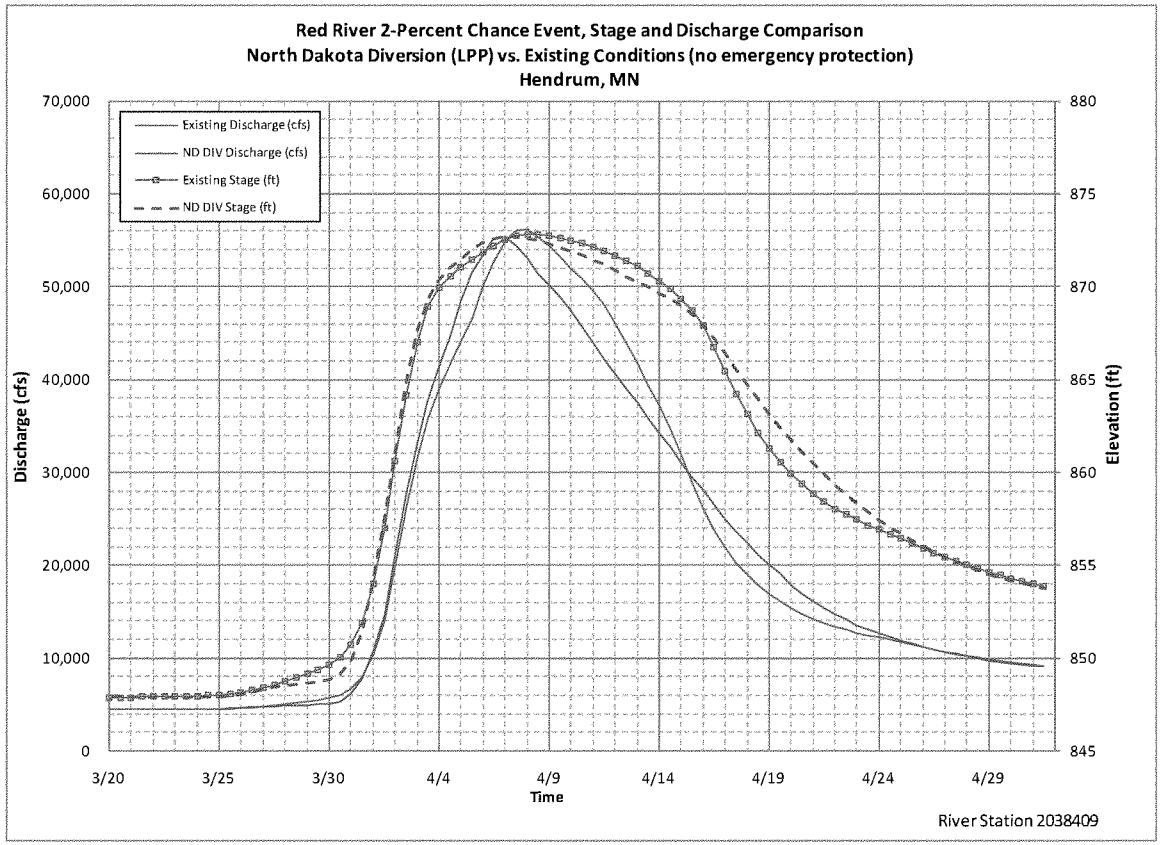


Figure C-E2- 25: 2-Percent Chance Hydrographs for LPP @ Hendrum

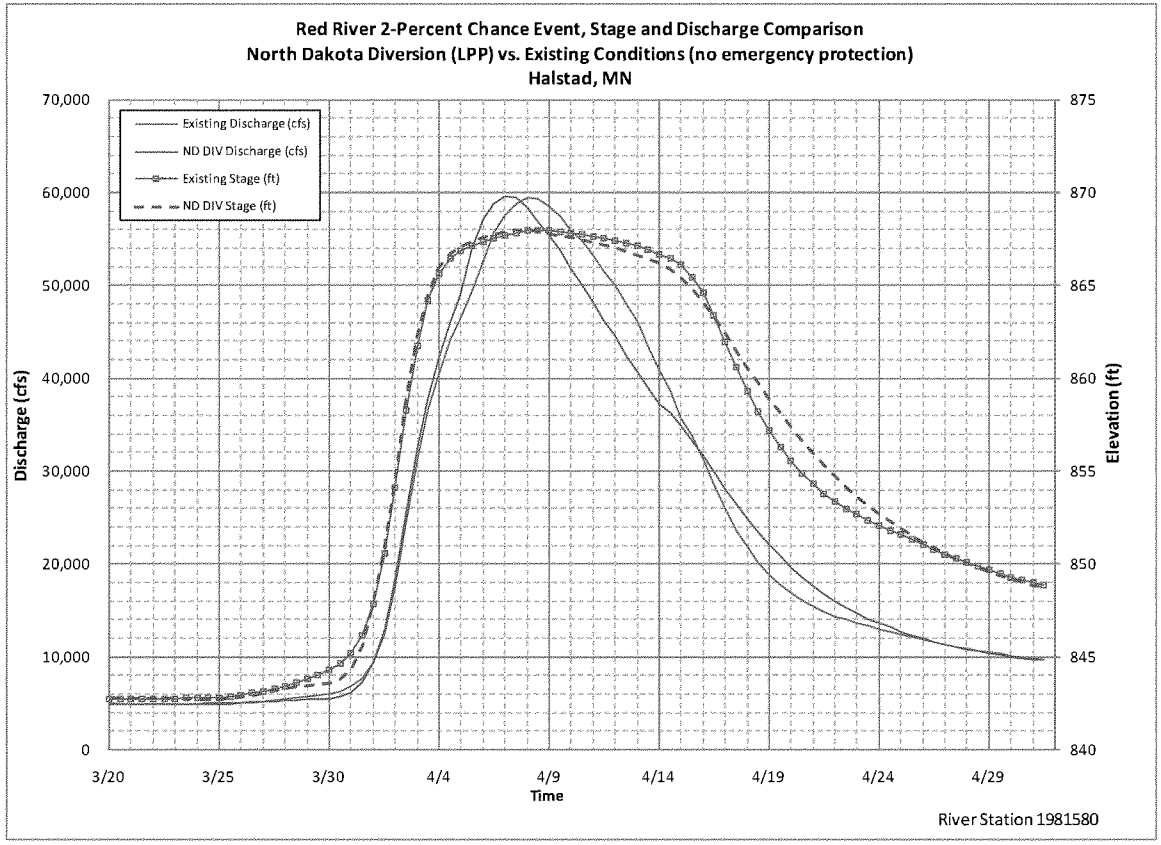


Figure C-E2- 26: 2-Percent Chance Hydrographs for LPP @ Halstad

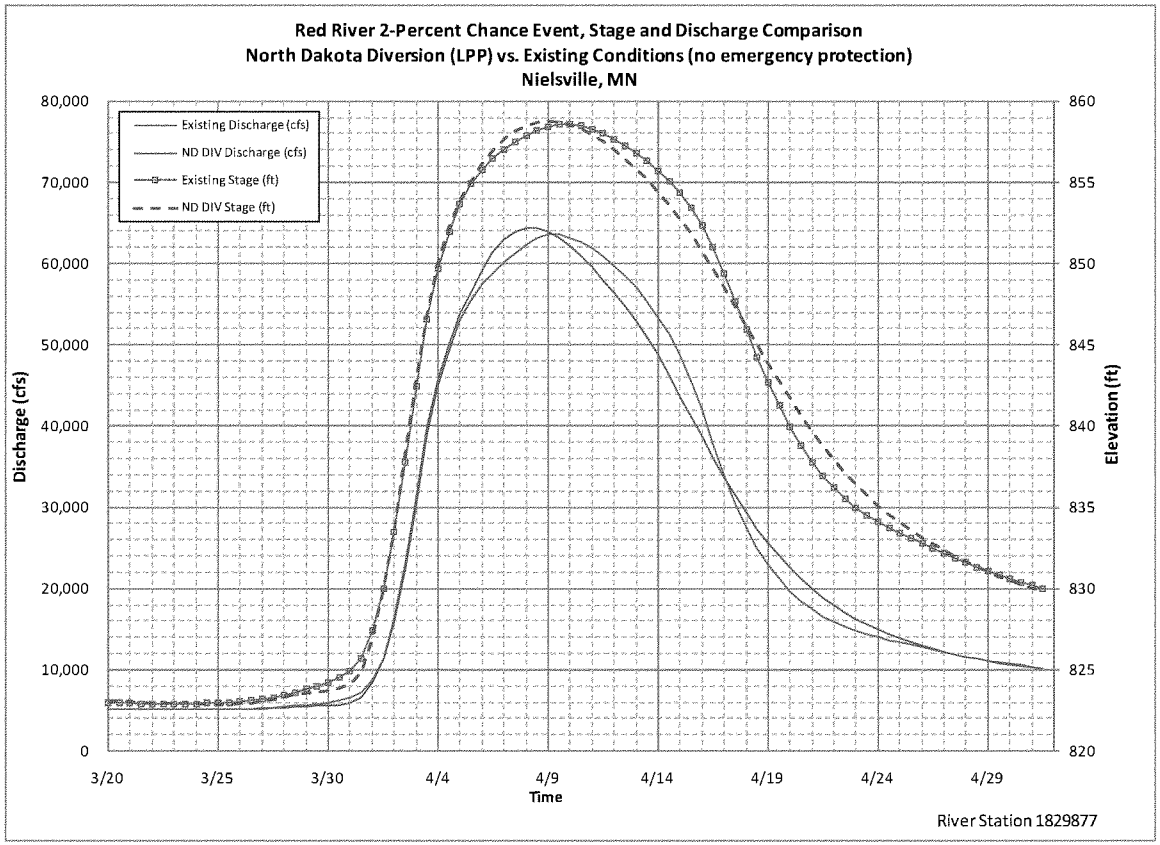


Figure C-E2- 27: 2-Percent Chance Hydrographs for LPP @ Nielsville

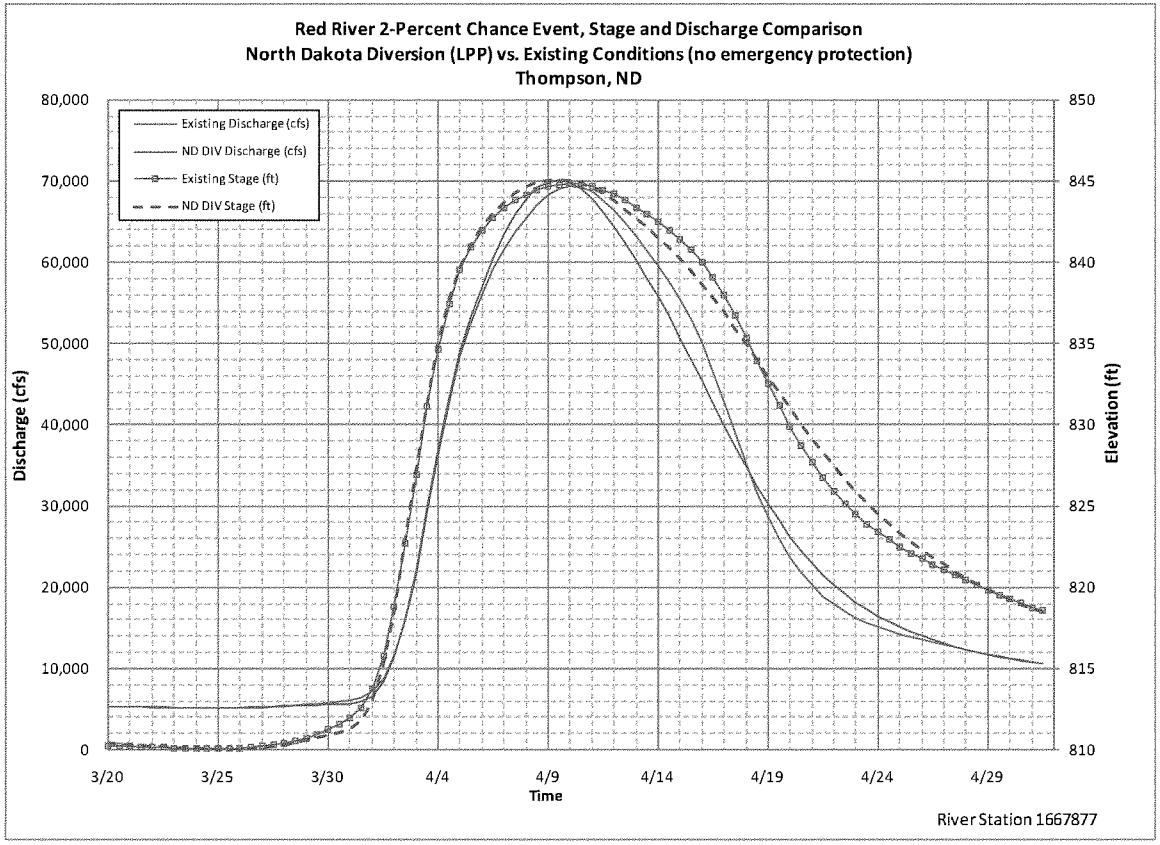


Figure C-E2- 28: 2-Percent Chance Hydrographs for LPP @ Thompson

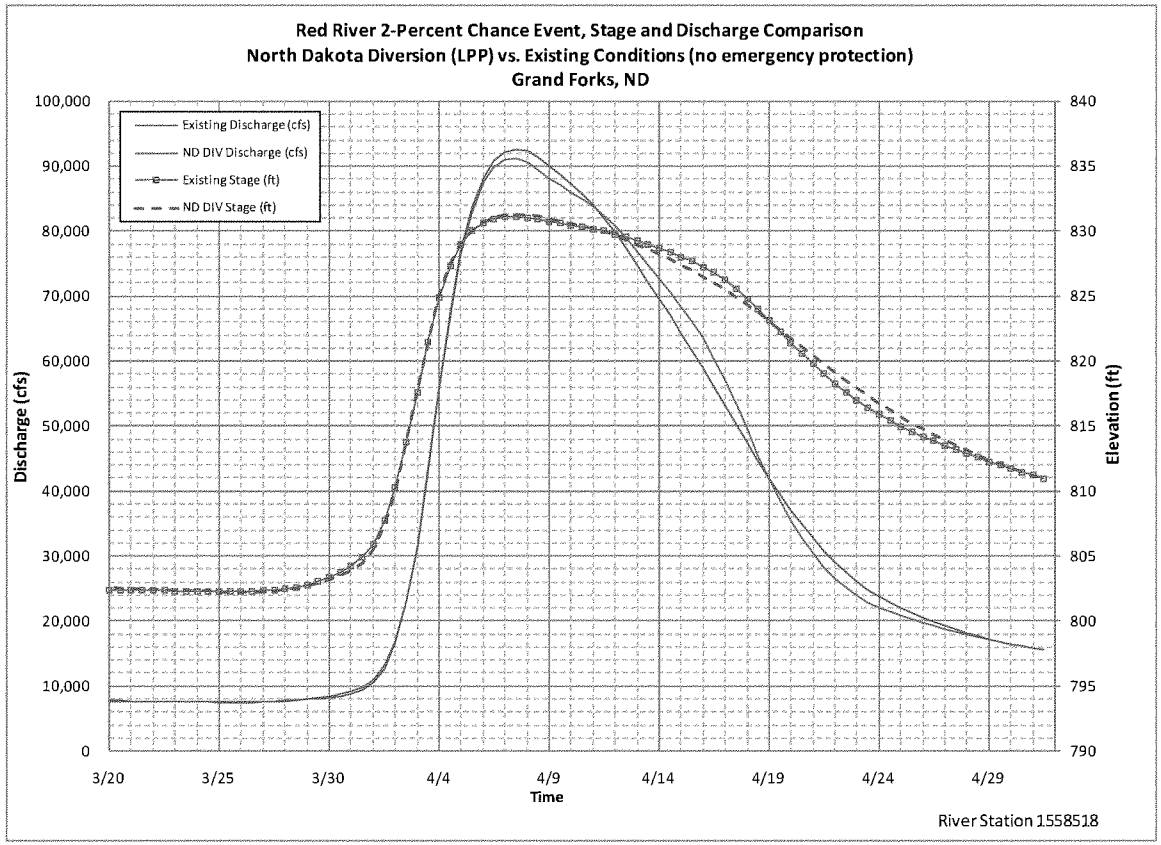


Figure C-E2- 29: 2-Percent Chance Hydrographs for LPP @ Grand Forks

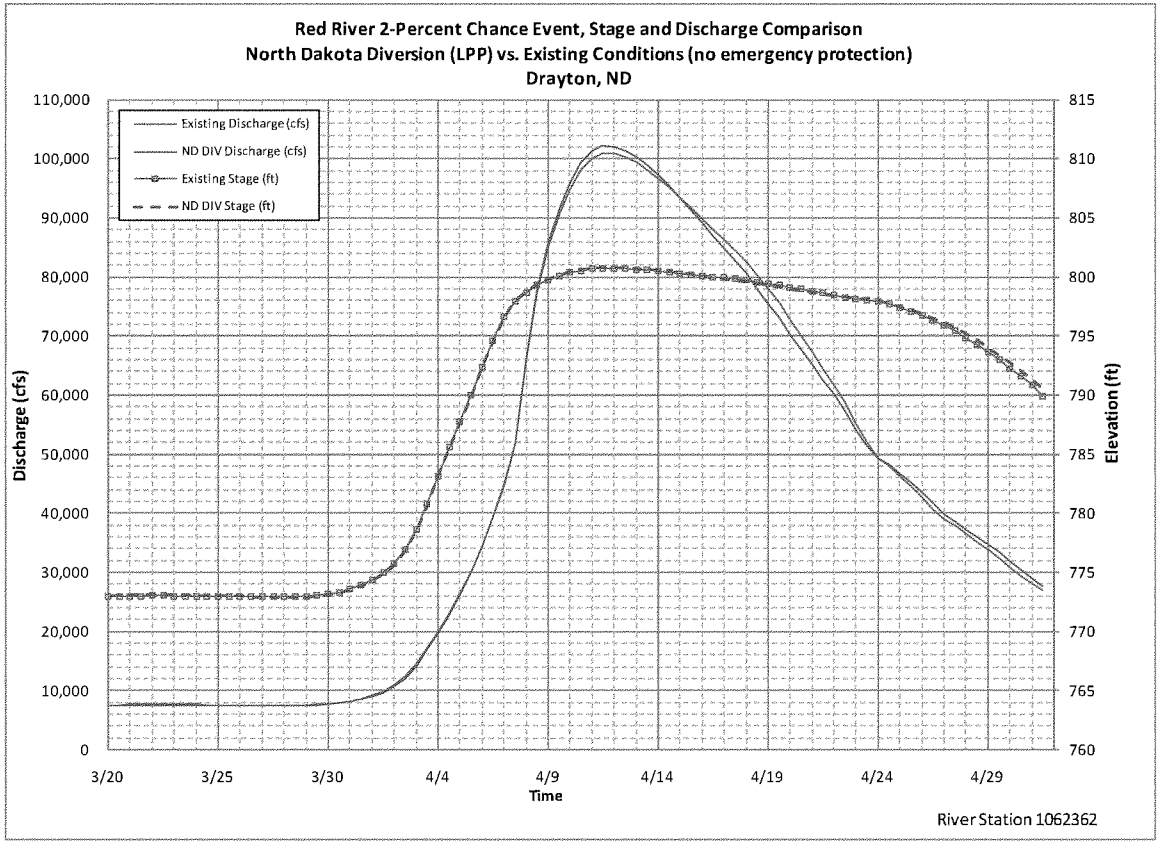


Figure C-E2- 30: 2-Percent Chance Hydrographs for LPP @ Drayton

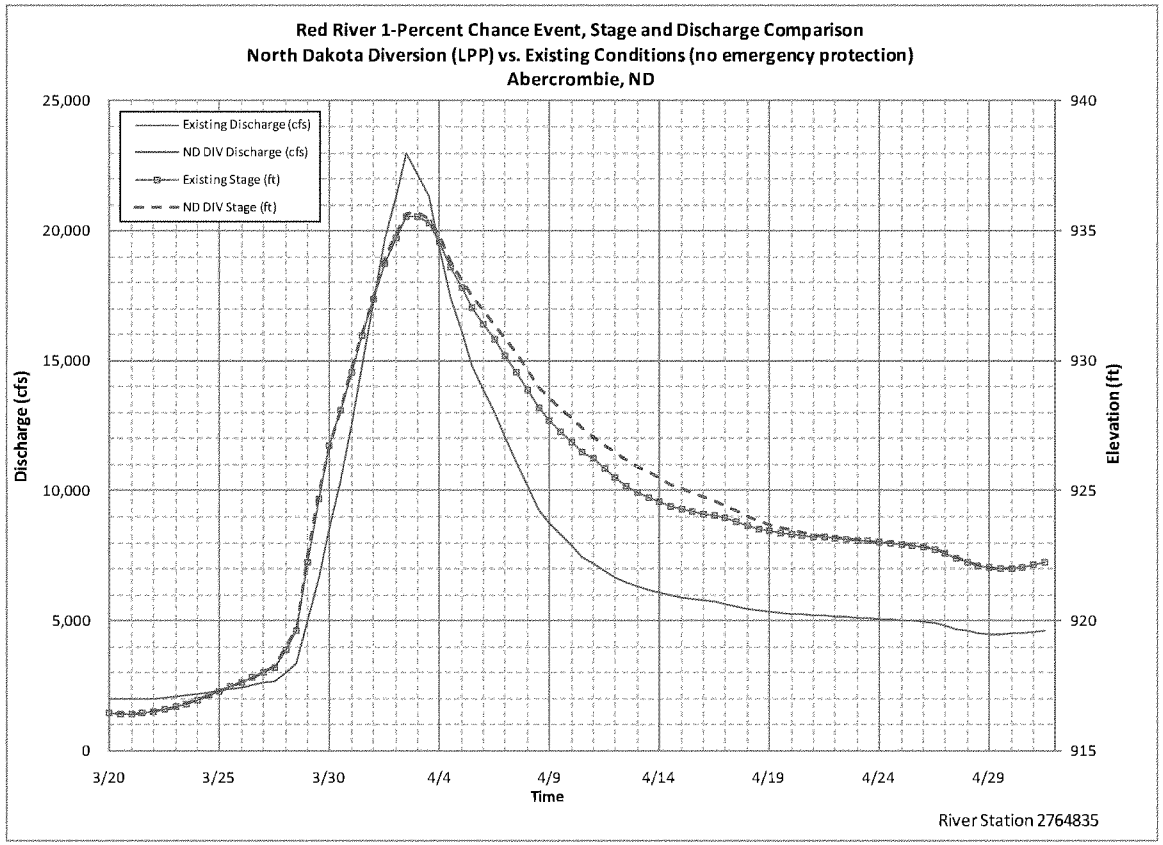


Figure C-E2- 31: 1-Percent Chance Hydrographs for LPP @ Abercrombie

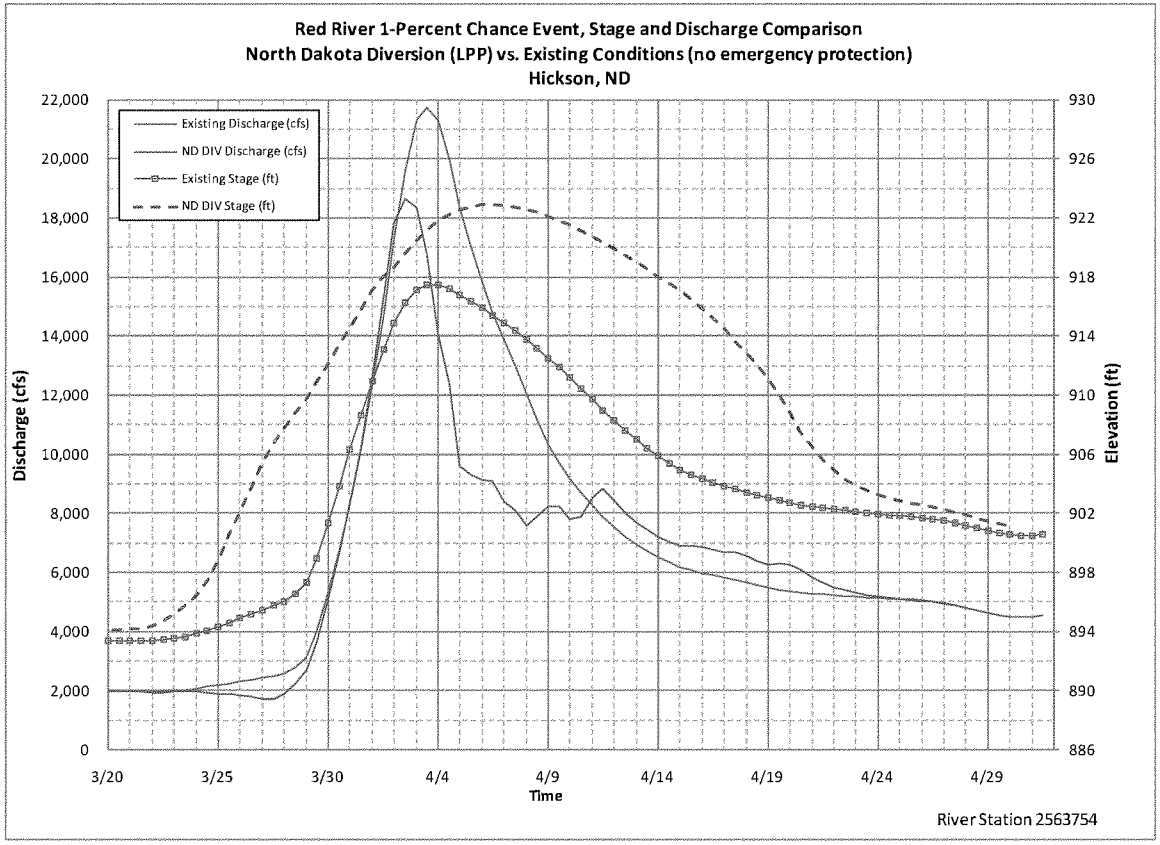


Figure C-E2- 32: 1-Percent Chance Hydrographs for LPP @ Hickson

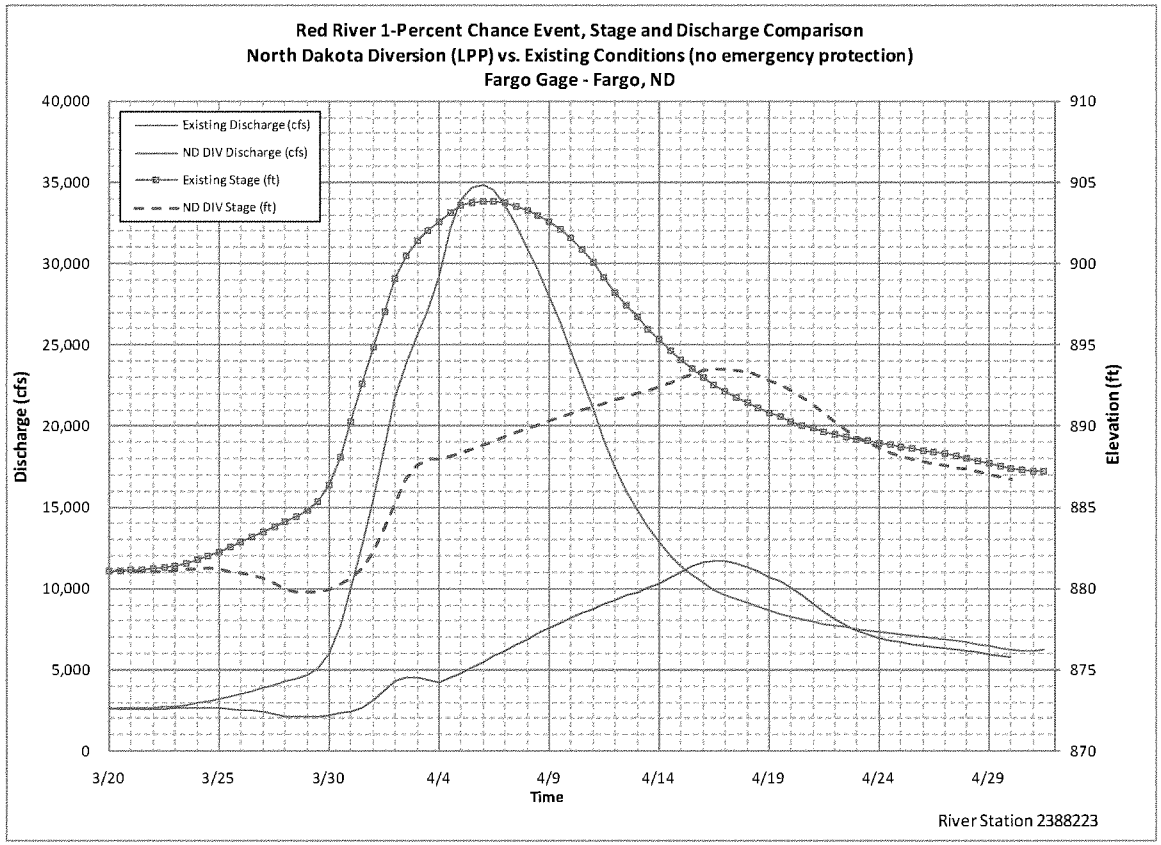


Figure C-E2- 33: 1-Percent Chance Hydrographs for LPP @ Fargo

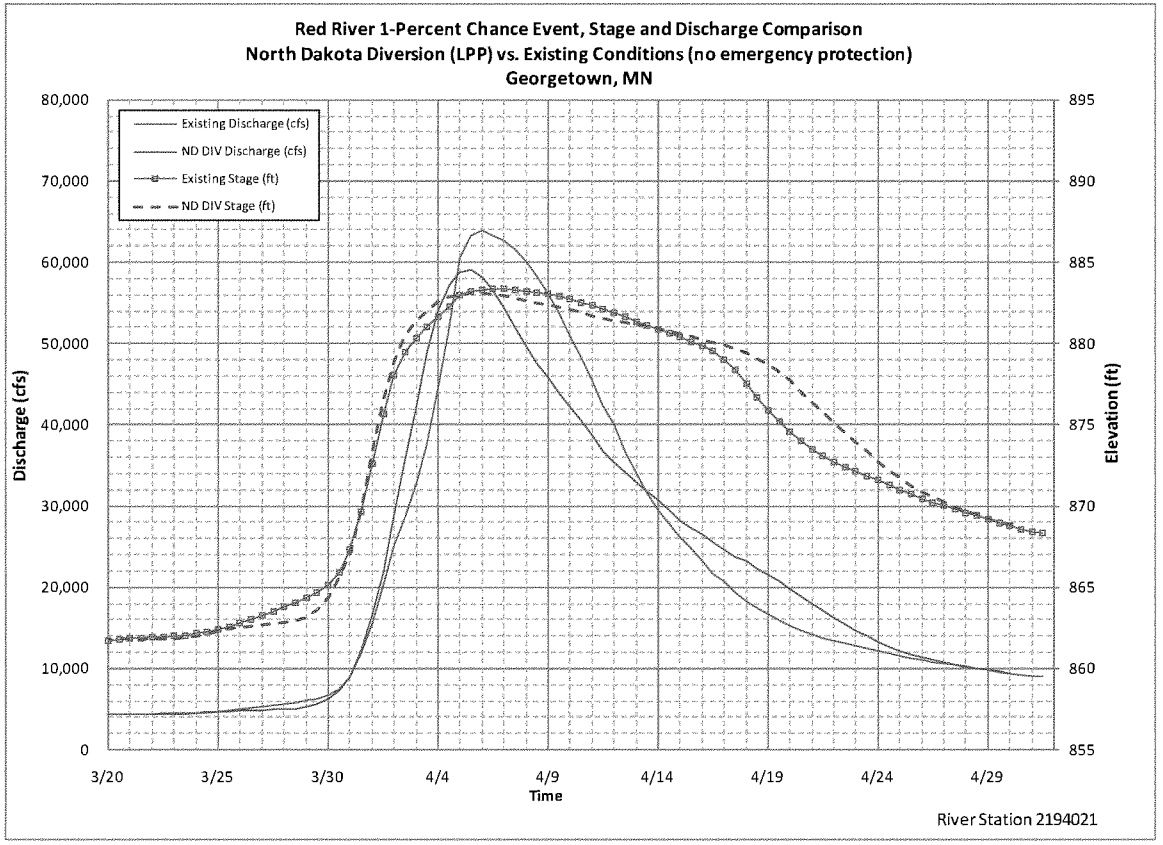


Figure C-E2- 34: 1-Percent Chance Hydrographs for LPP @ Georgetown

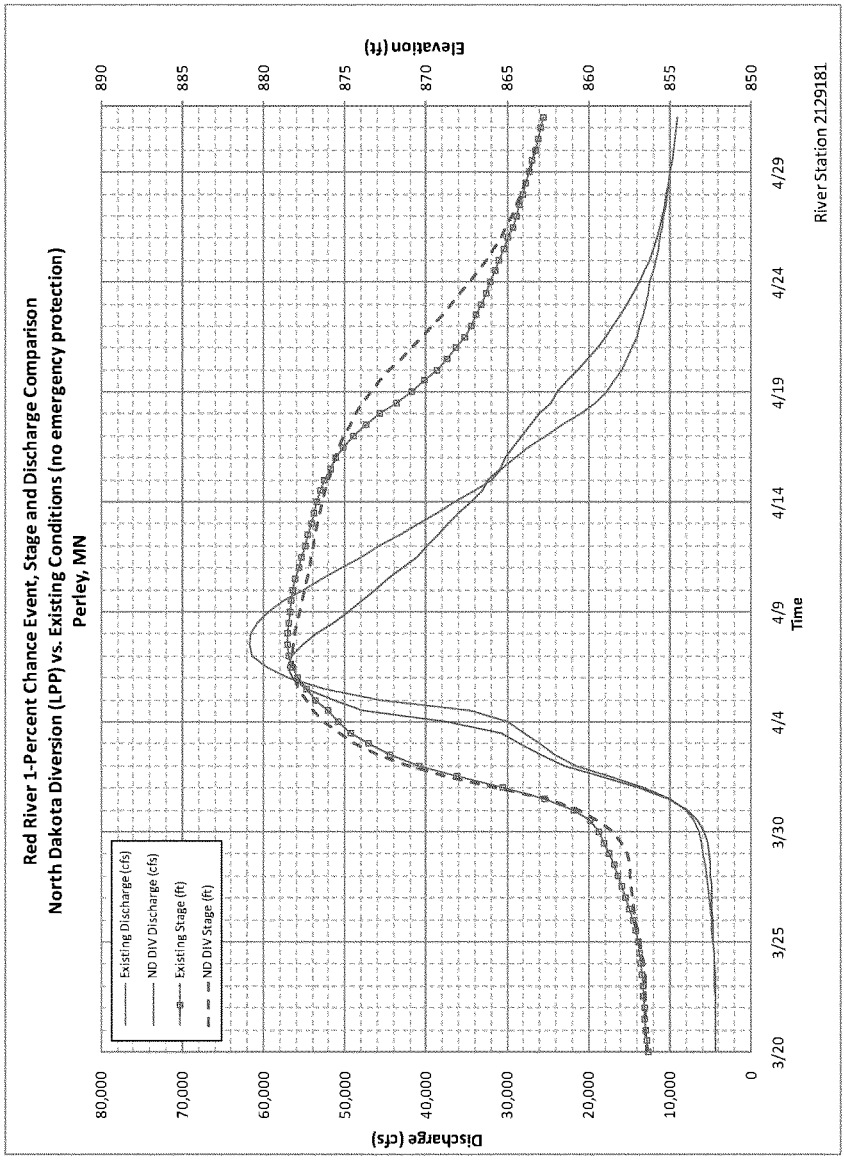


Figure C-E2- 35: 1-Percent Chance Hydrographs for LPP @ Perley

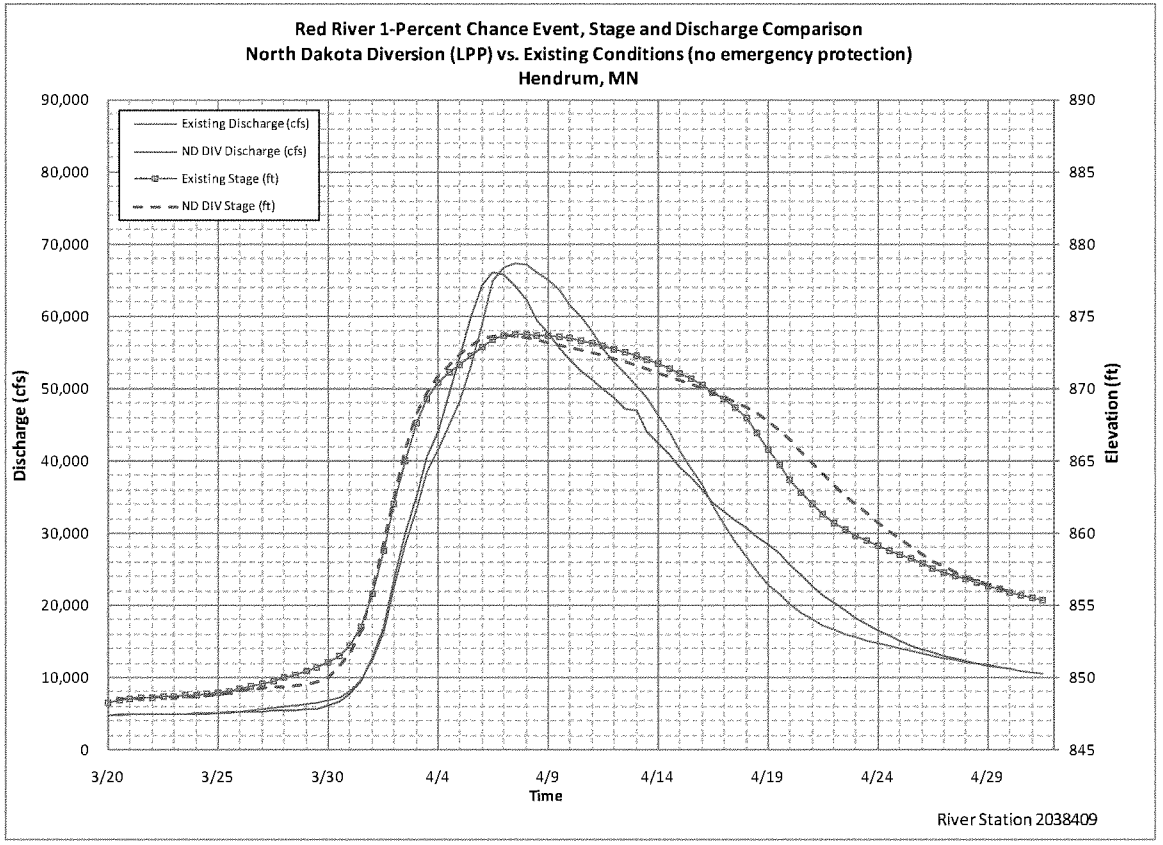


Figure C-E2- 36: 1-Percent Chance Hydrographs for LPP @ Hendrum

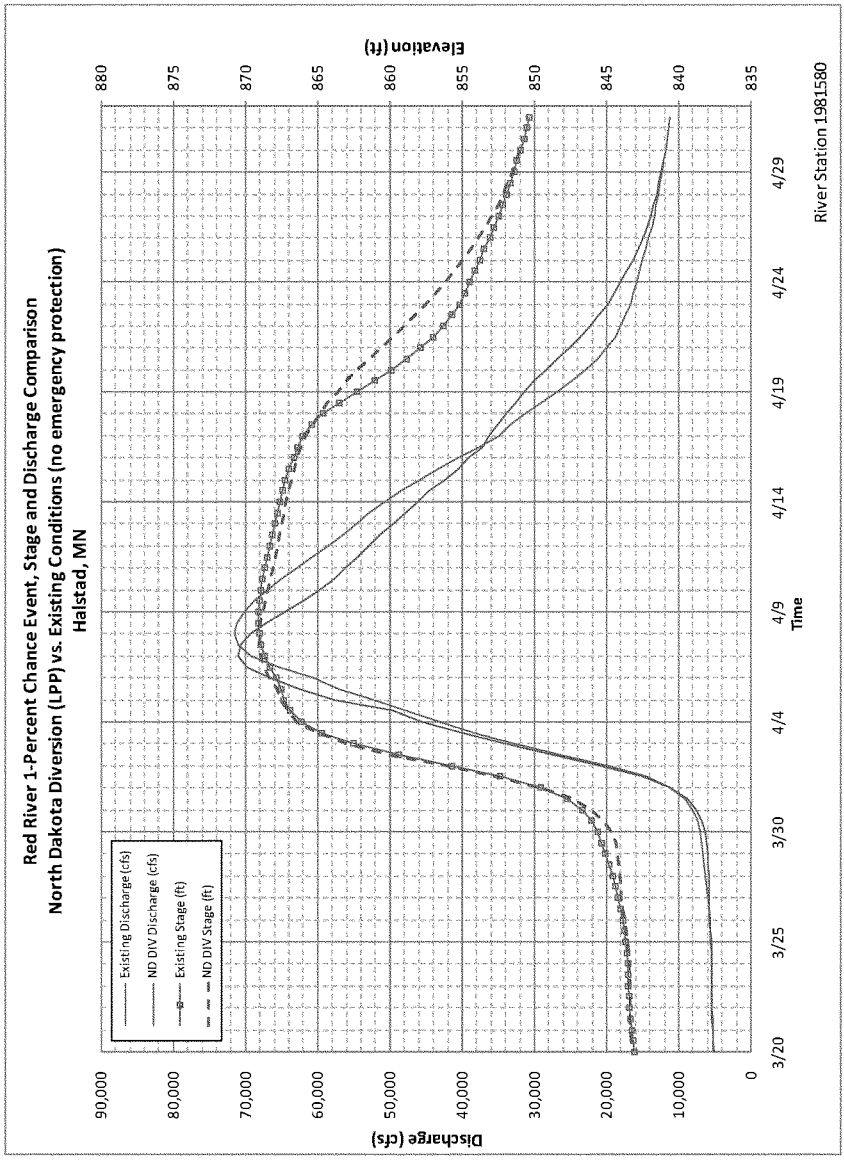


Figure C-E2- 37: 1-Percent Chance Hydrographs for LPP @ Halstad

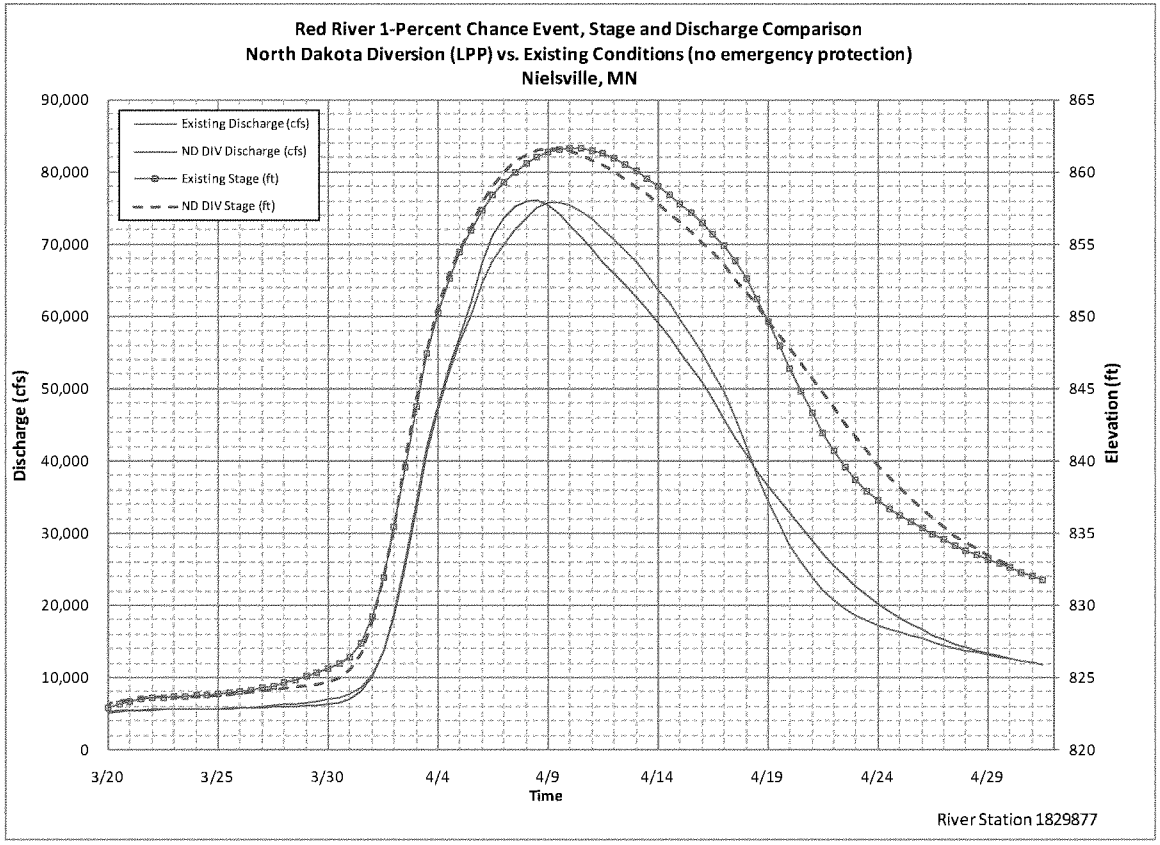


Figure C-E2- 38: 1-Percent Chance Hydrographs for LPP @ Nielsville

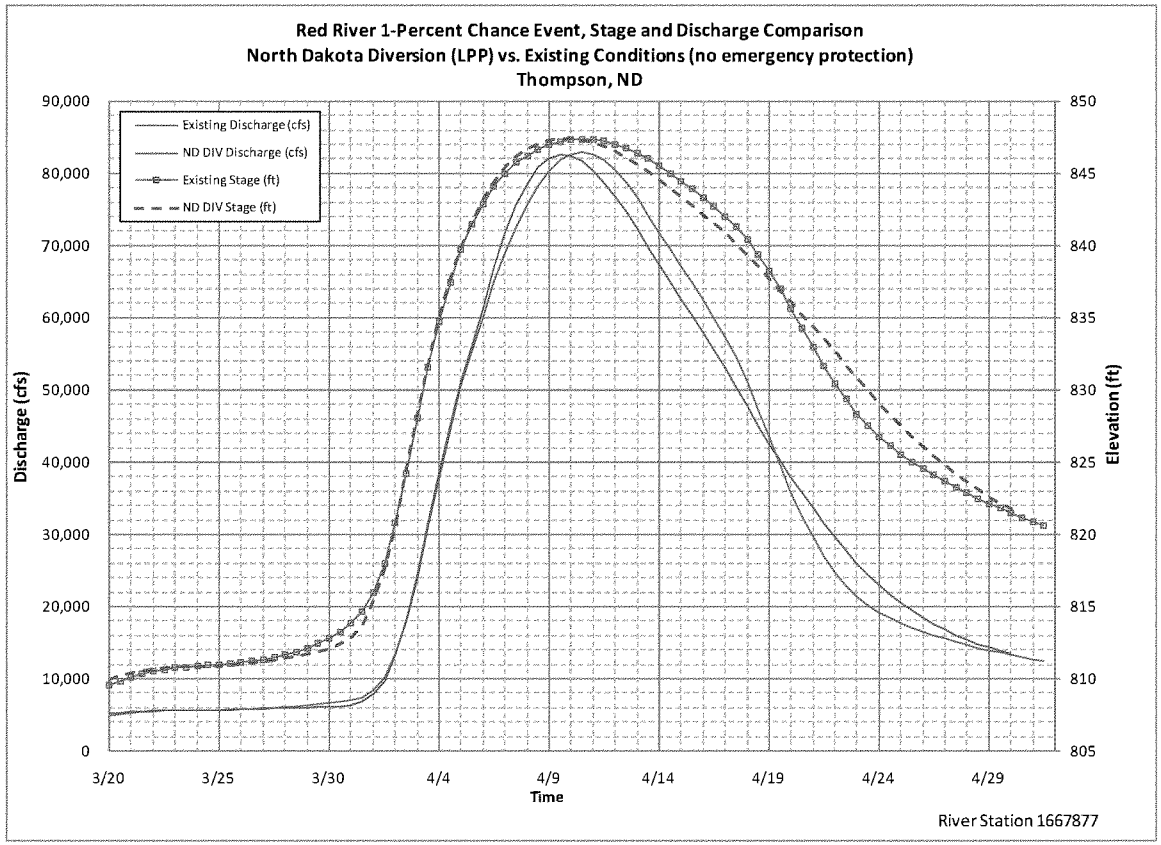


Figure C-E2- 39: 1-Percent Chance Hydrographs for LPP @ Thompson

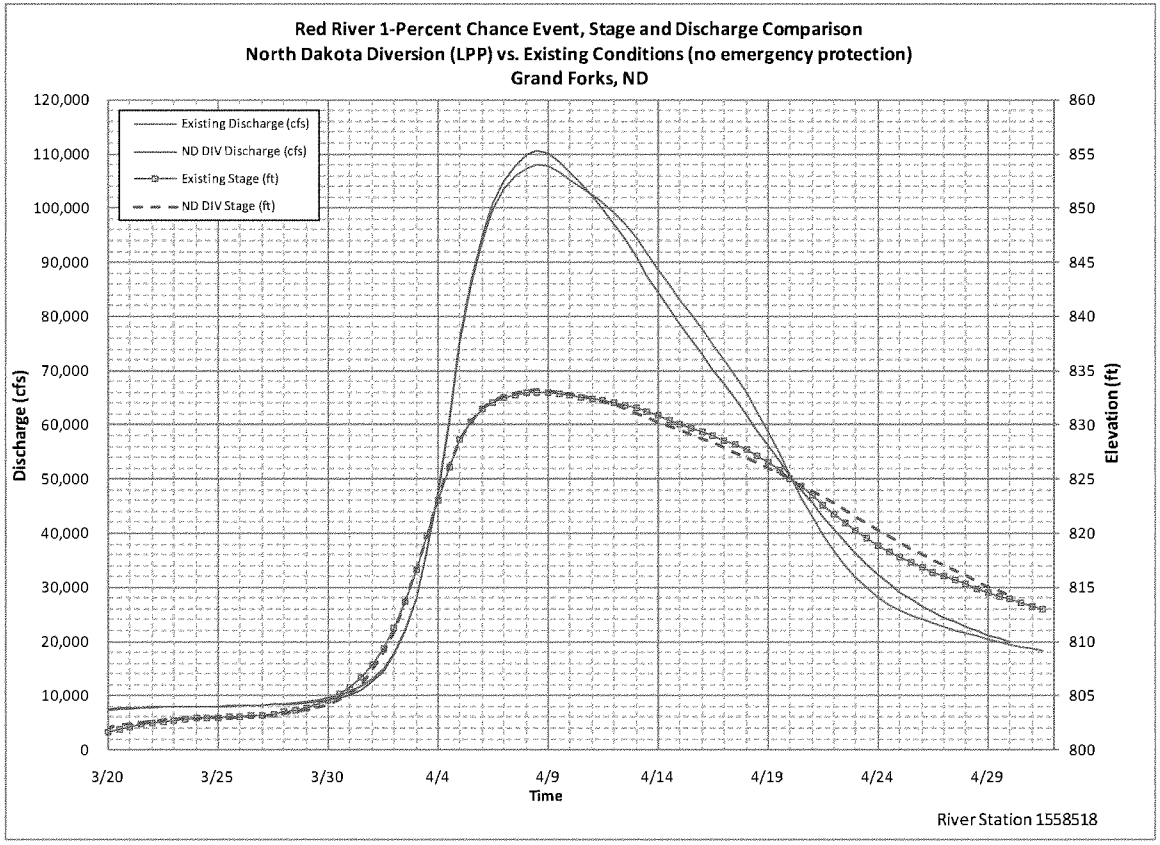


Figure C-E2- 40: 1-Percent Chance Hydrographs for LPP @ Grand Forks

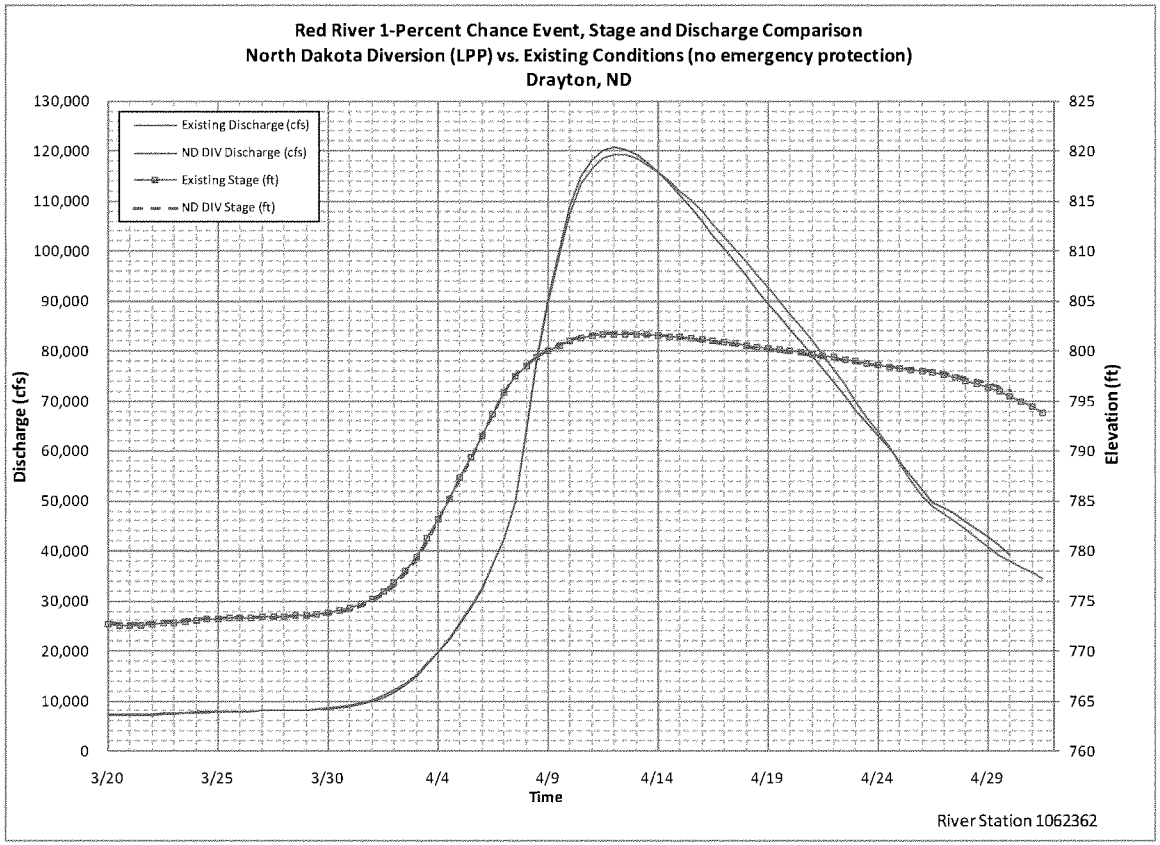


Figure C-E2- 41: 1-Percent Chance Hydrographs for LPP @ Drayton

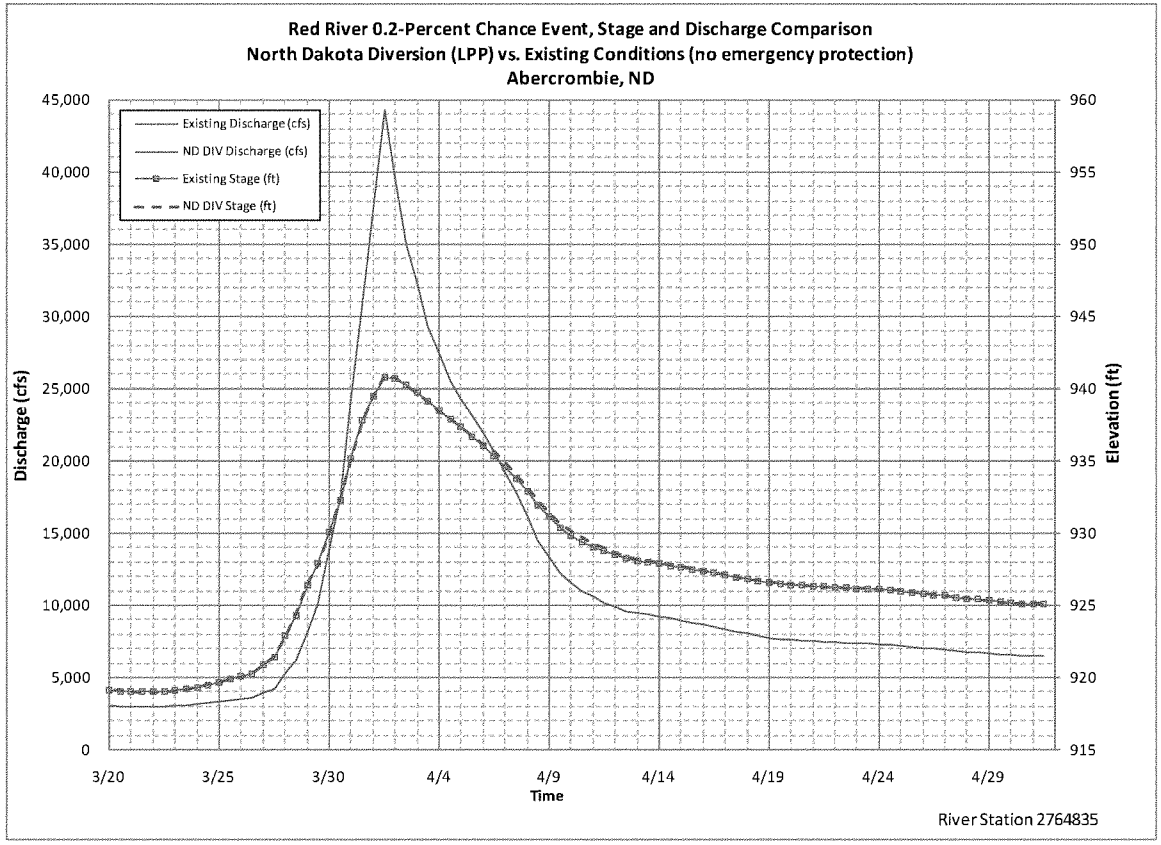


Figure C-E2- 42: 0.2-Percent Chance Hydrographs for LPP @ Abercrombie

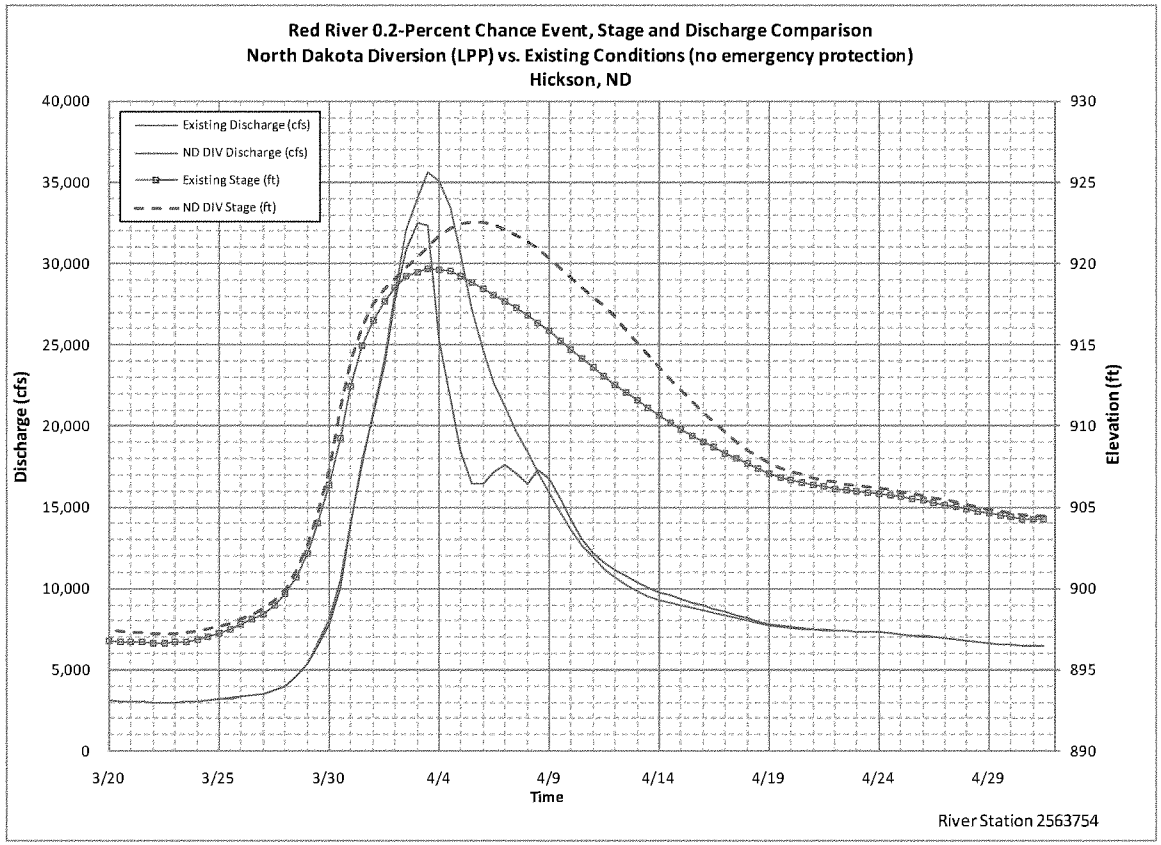


Figure C-E2- 43: 0.2-Percent Chance Hydrographs for LPP @ Hickson

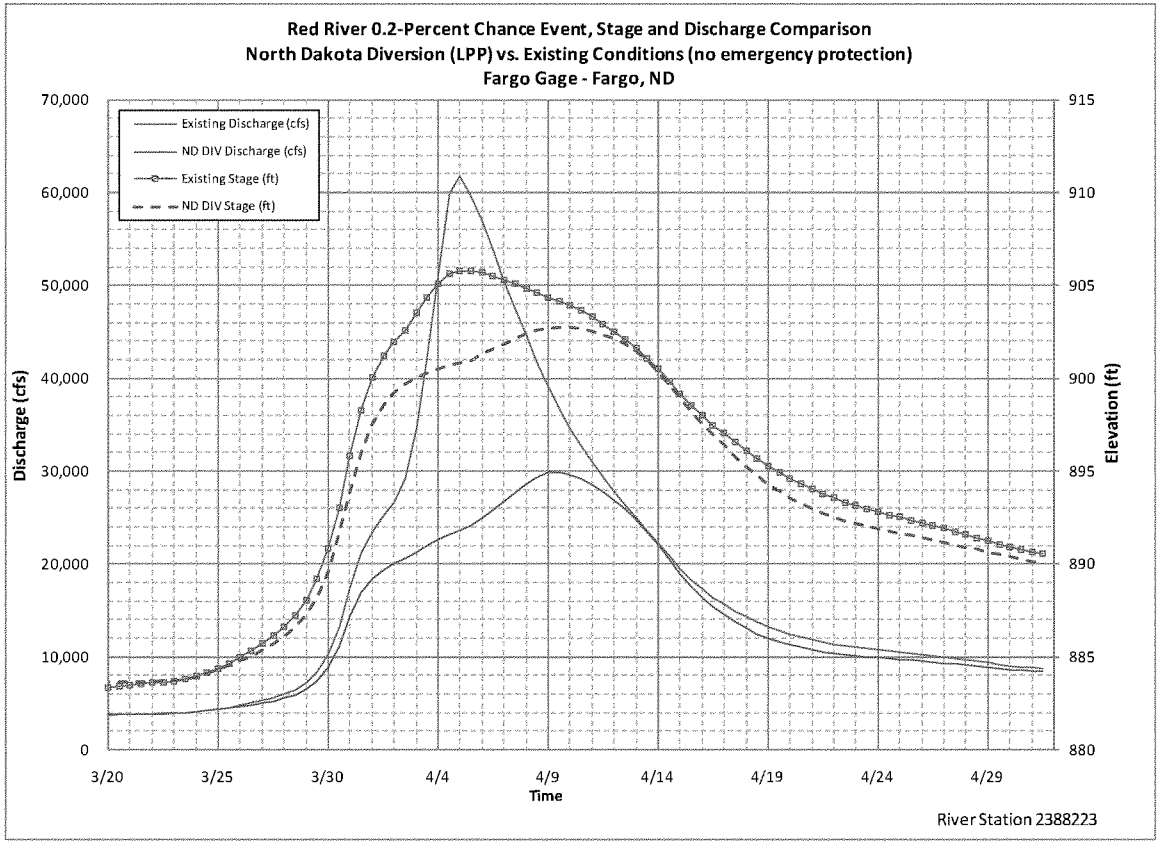


Figure C-E2- 44: 0.2-Percent Chance Hydrographs for LPP @ Fargo

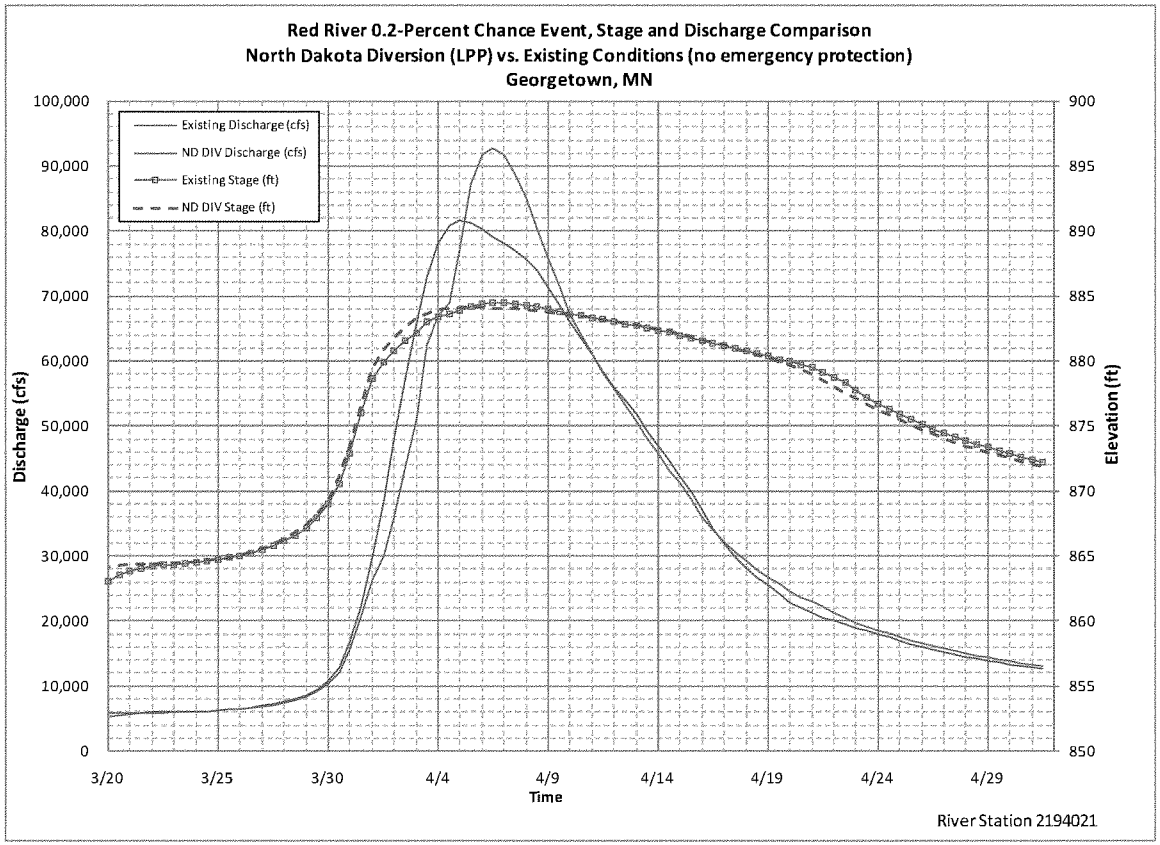


Figure C-E2- 45: 0.2-Percent Chance Hydrographs for LPP @ Georgetown

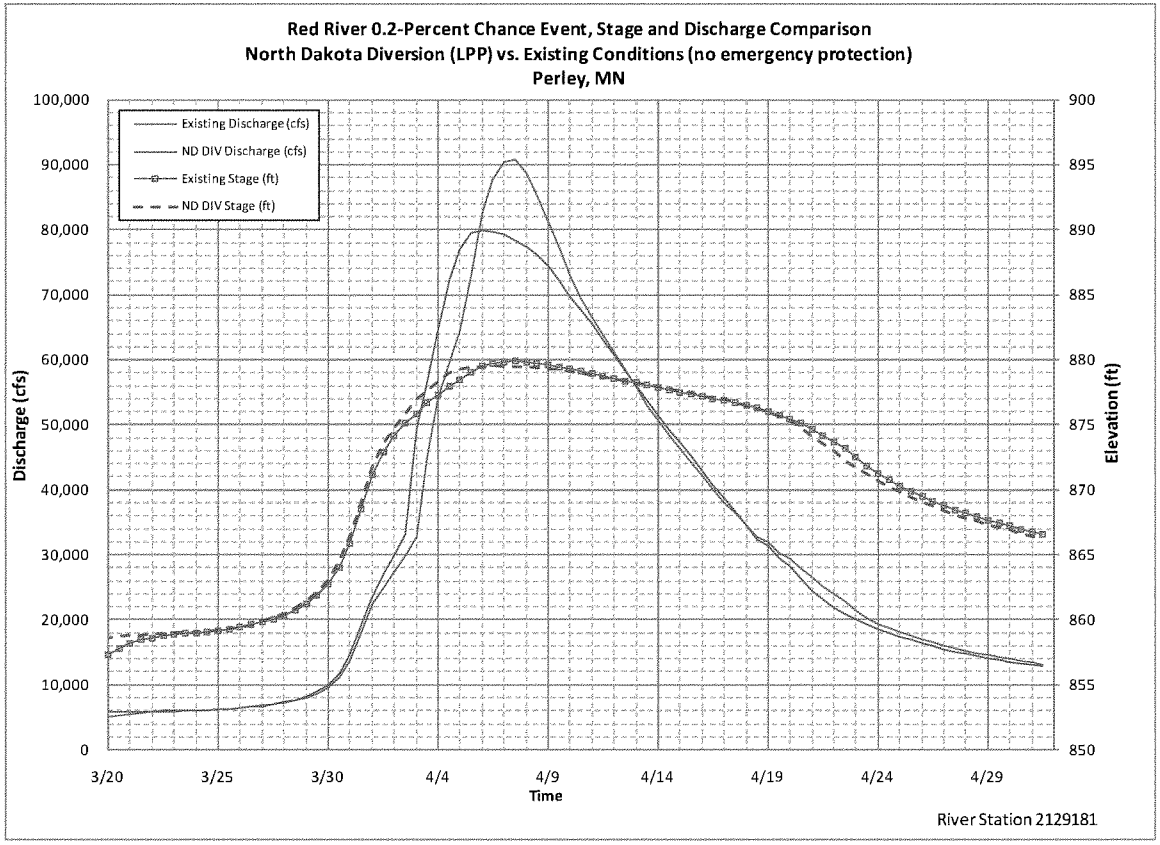


Figure C-E2- 46: 0.2-Percent Chance Hydrographs for LPP @ Perley

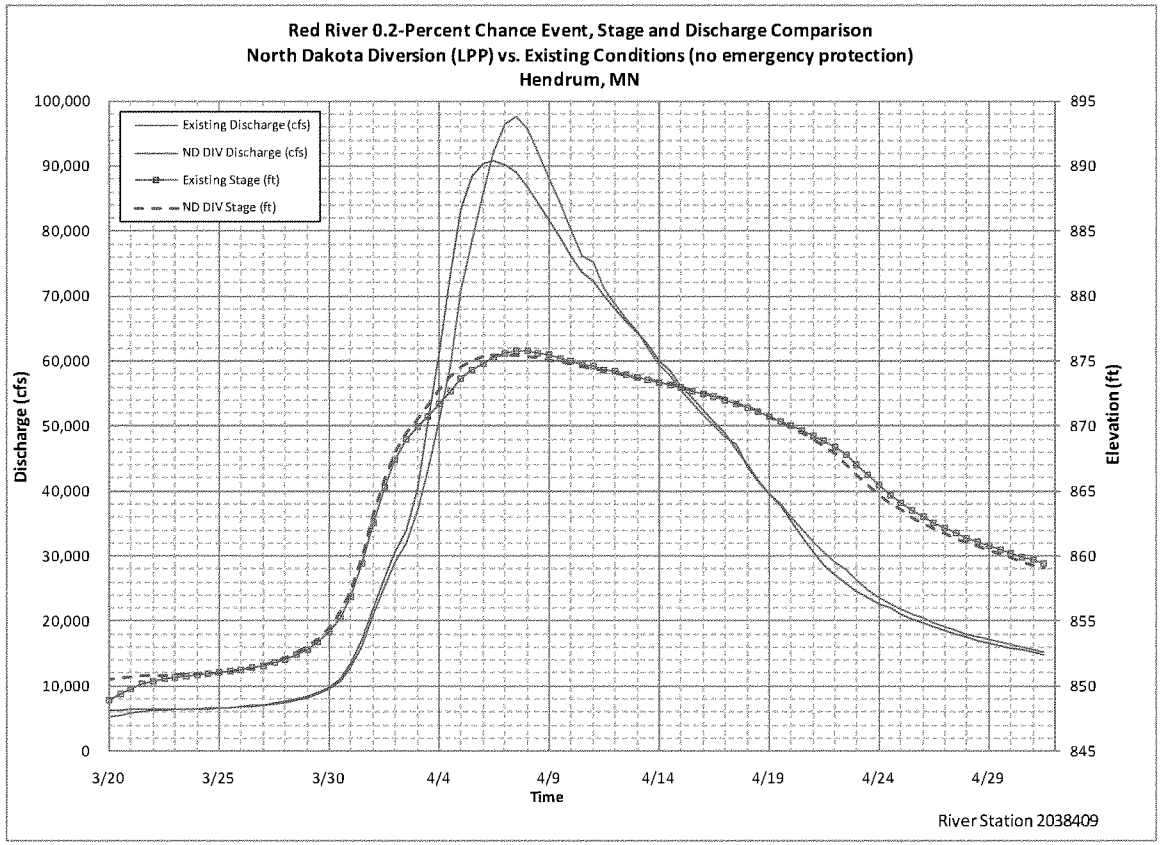


Figure C-E2- 47: 0.2-Percent Chance Hydrographs for LPP @ Hendrum

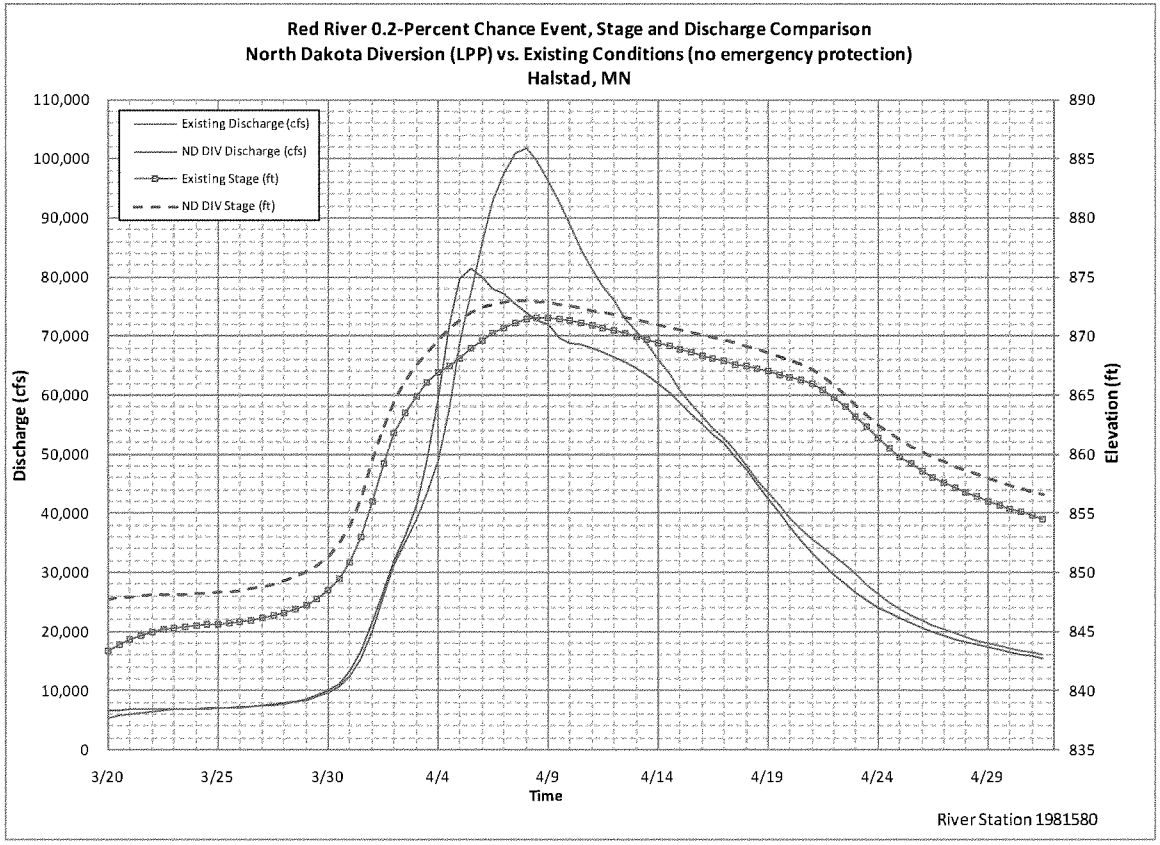


Figure C-E2- 48: 0.2-Percent Chance Hydrographs for LPP @ Halstad

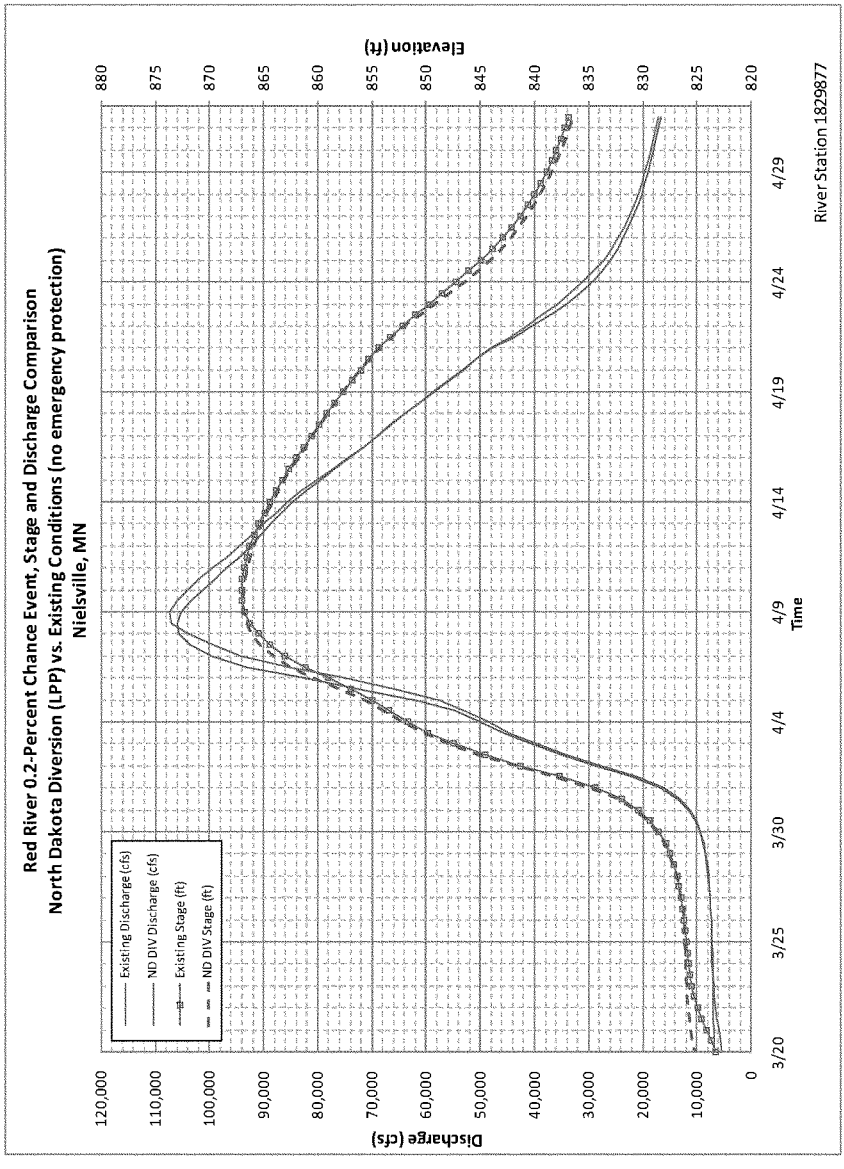


Figure C-E2- 49: 0.2-Percent Chance Hydrographs for LPP @ Nielsville

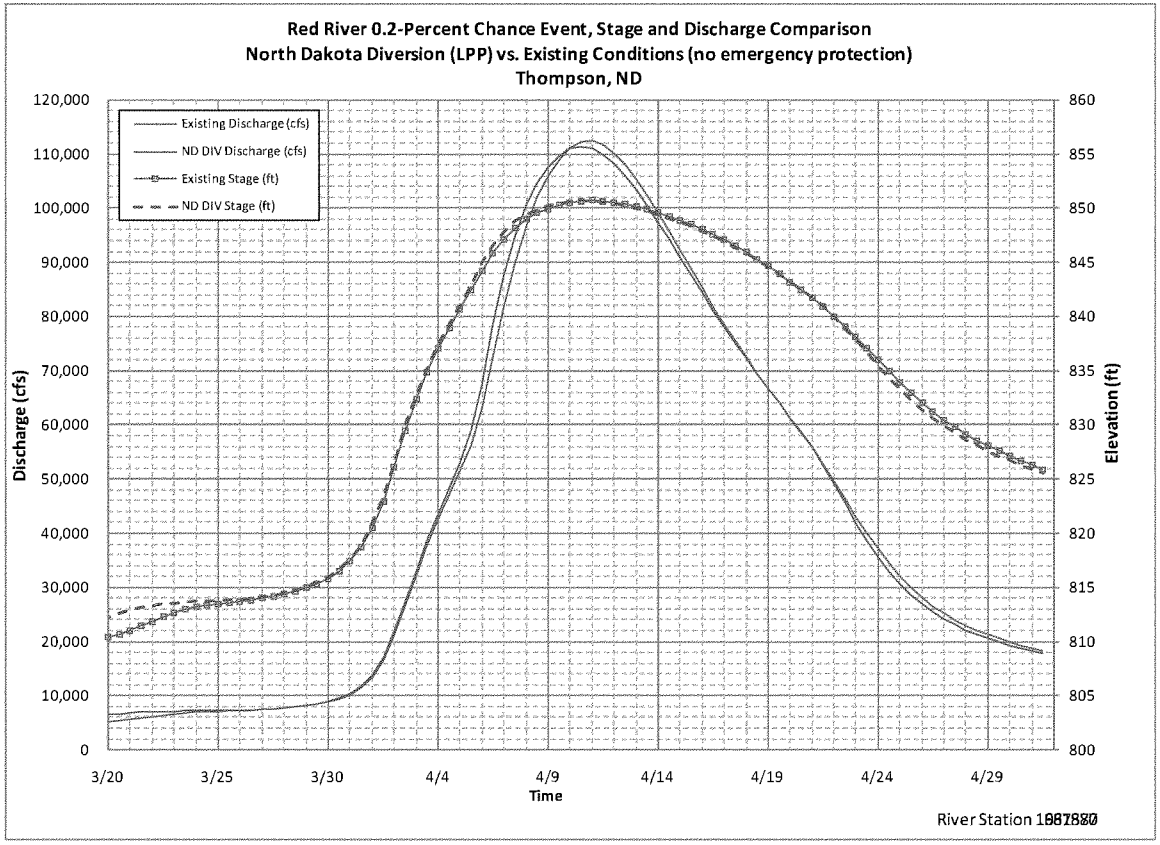


Figure C-E2- 50: 0.2-Percent Chance Hydrographs for LPP @ Thompson

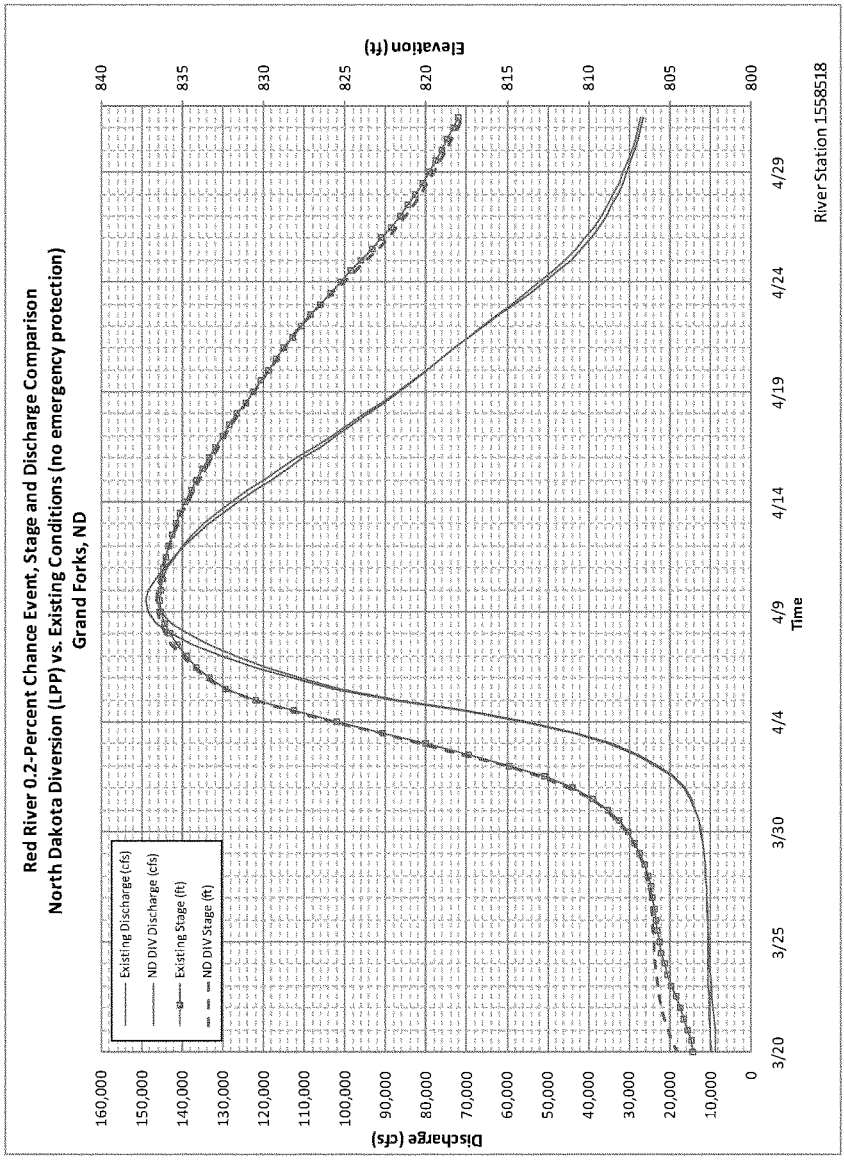


Figure C-E2- 51: 0.2-Percent Chance Hydrographs for LPP @ Grand Forks

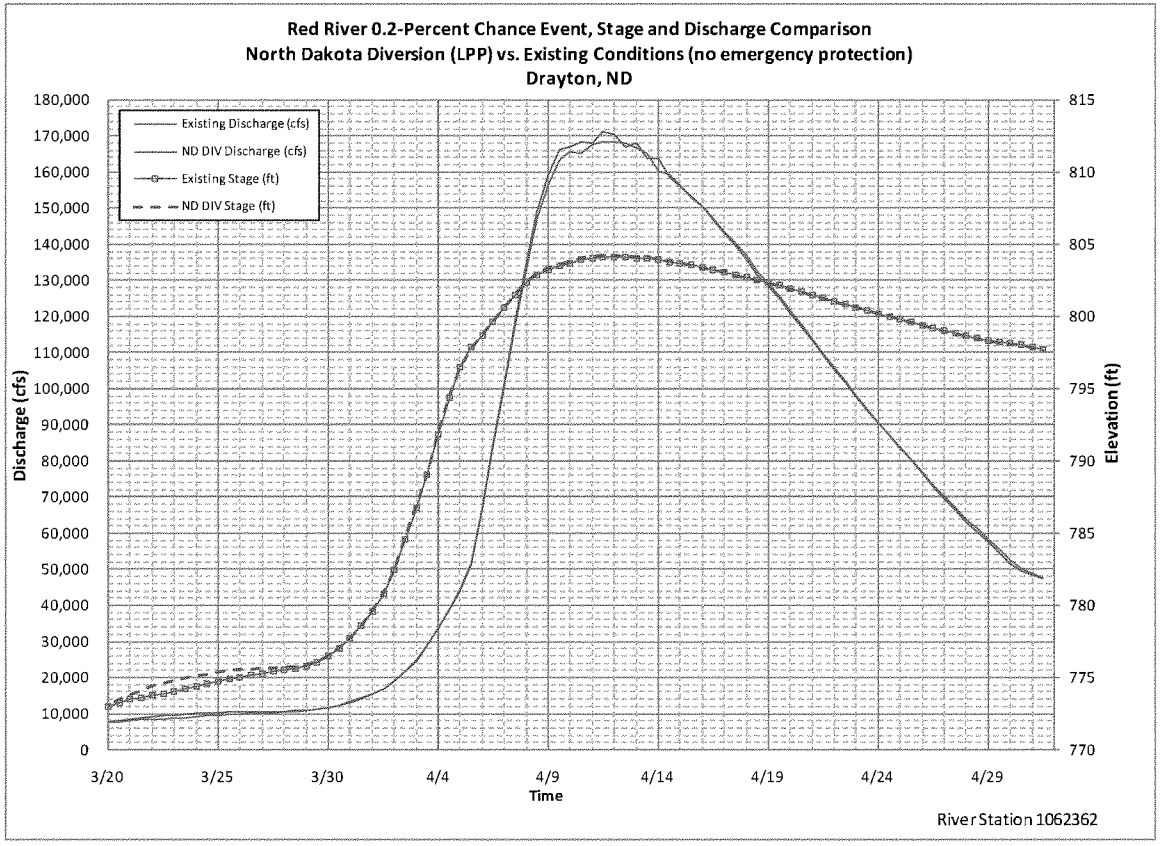


Figure C-E2- 52: 0.2-Percent Chance Hydrographs for LPP @ Drayton

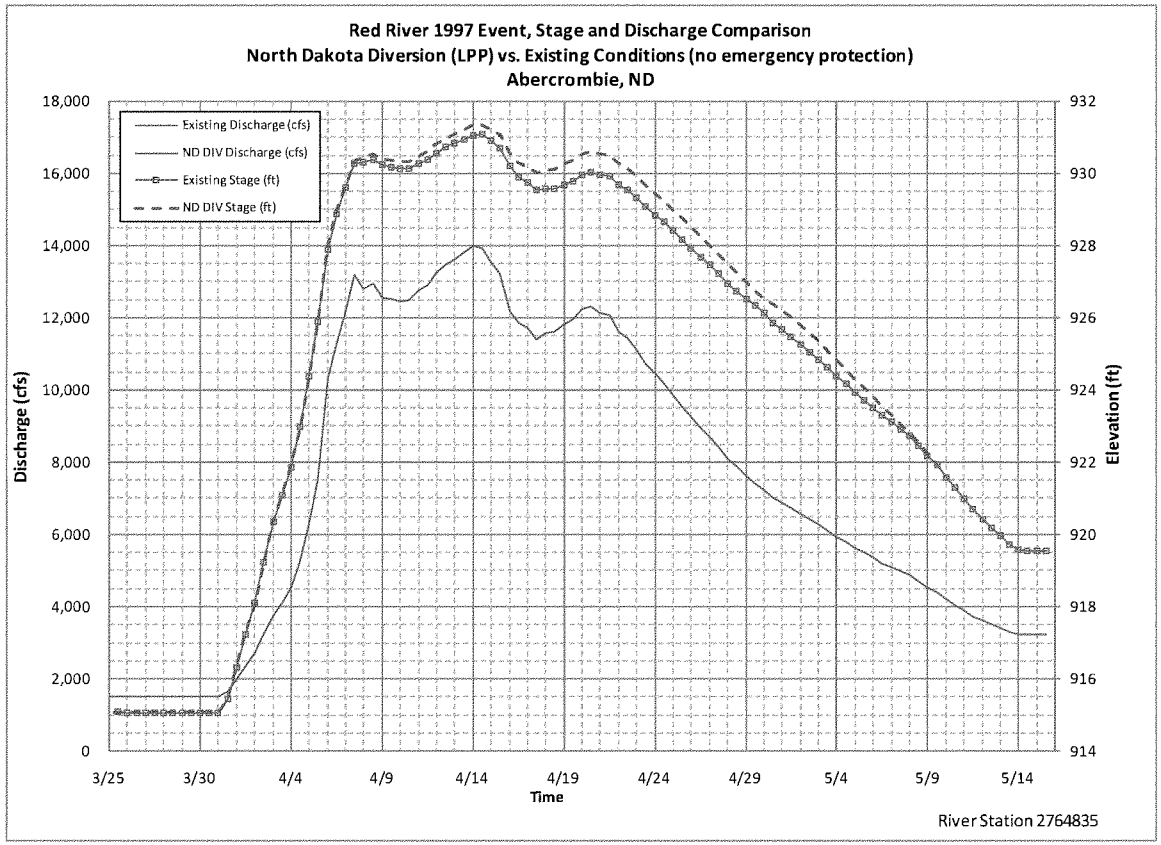


Figure C-E2- 53: 1997 Historical Flood Hydrographs for LPP @ Abercrombie

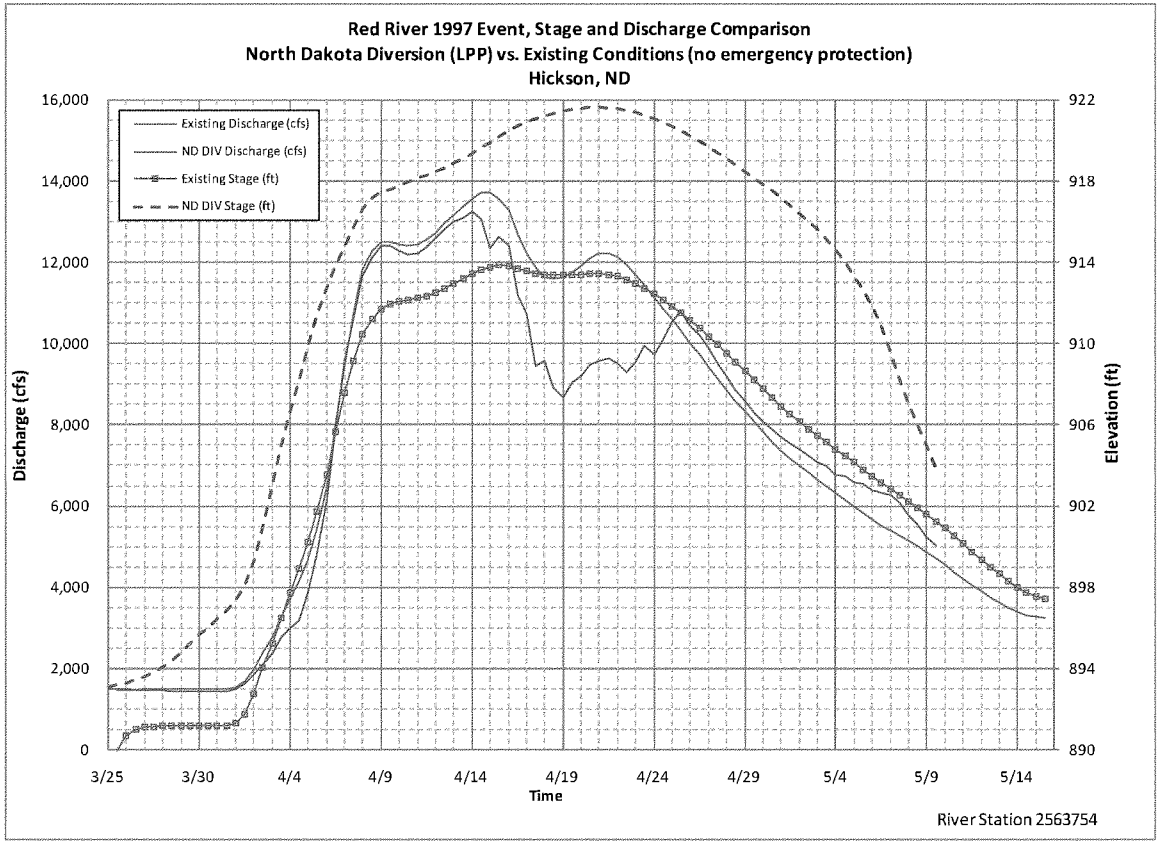


Figure C-E2- 54: 1997 Historical Flood Hydrographs for LPP @ Hickson

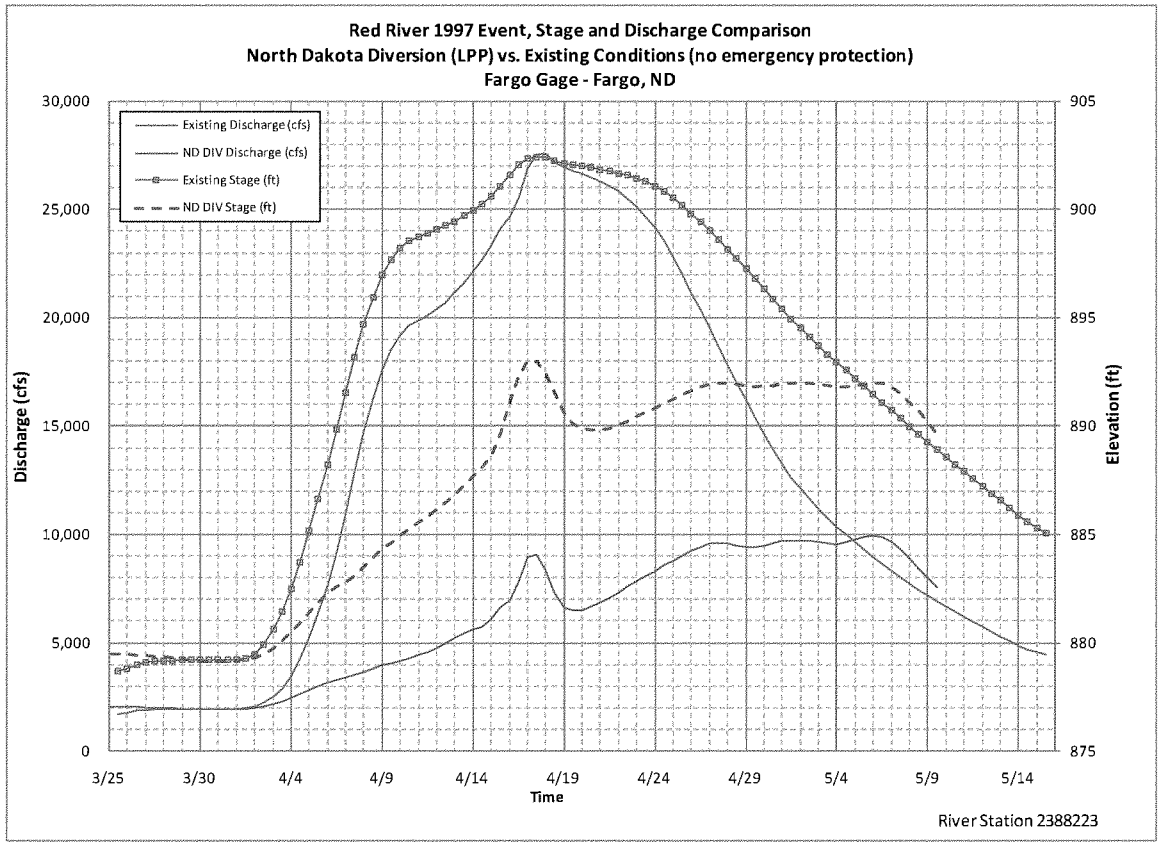


Figure C-E2- 55: 1997 Historical Flood Hydrographs for LPP @ Fargo

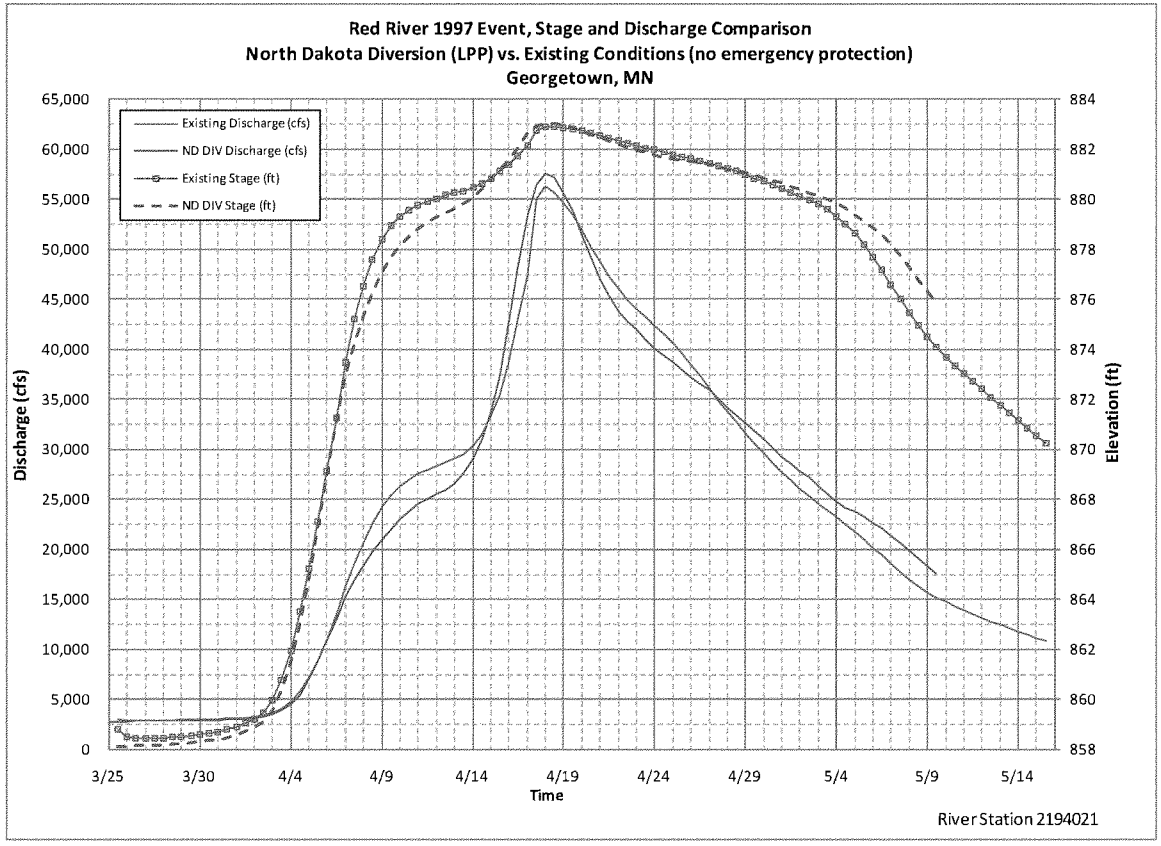


Figure C-E2- 56: 1997 Historical Flood Hydrographs for LPP @ Georgetown

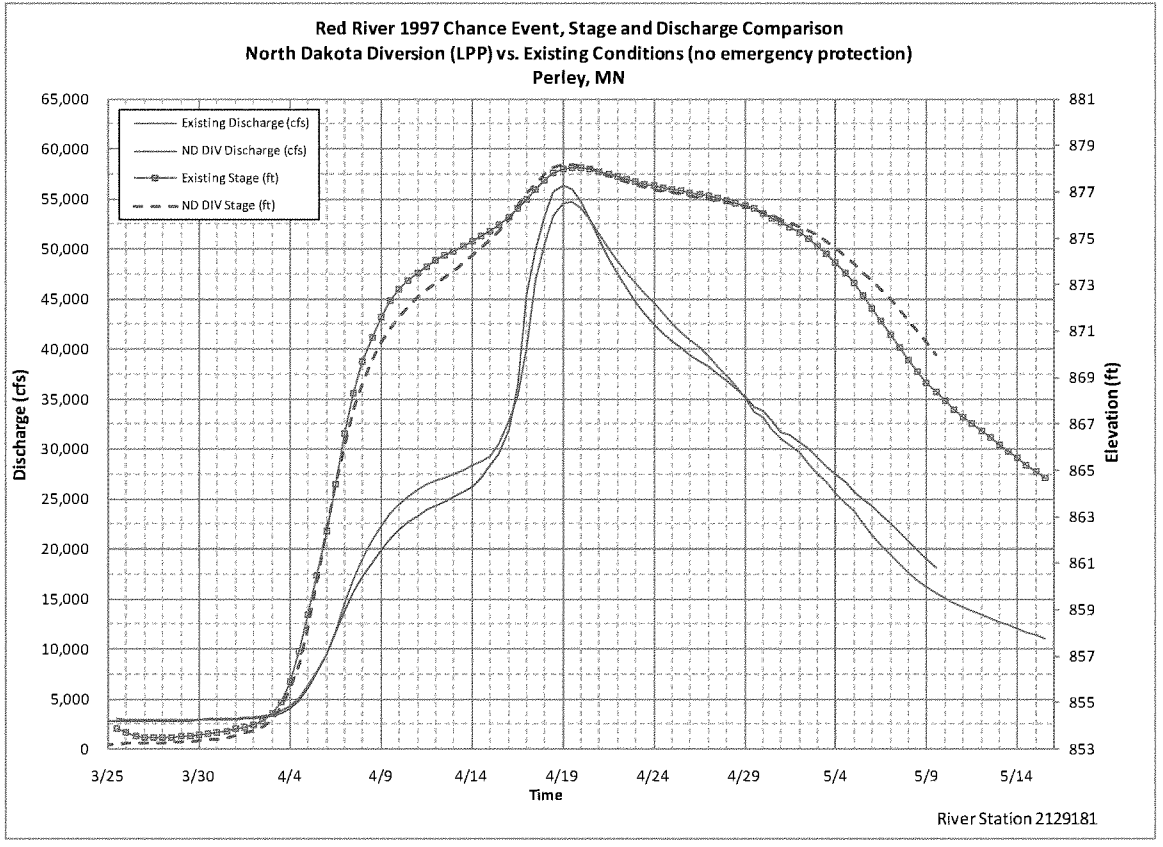


Figure C-E2- 57: 1997 Historical Flood Hydrographs for LPP @ Perley

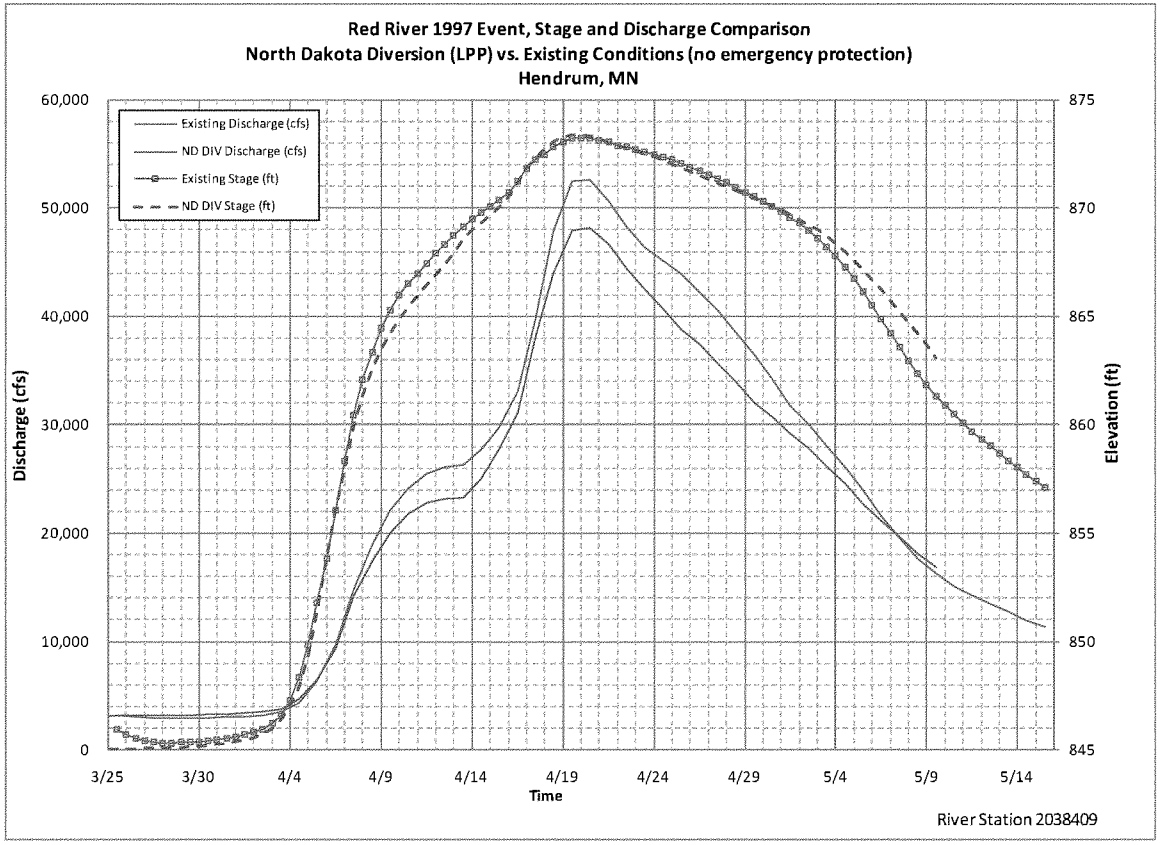


Figure C-E2- 58: 1997 Historical Flood Hydrographs for LPP @ Hendrum

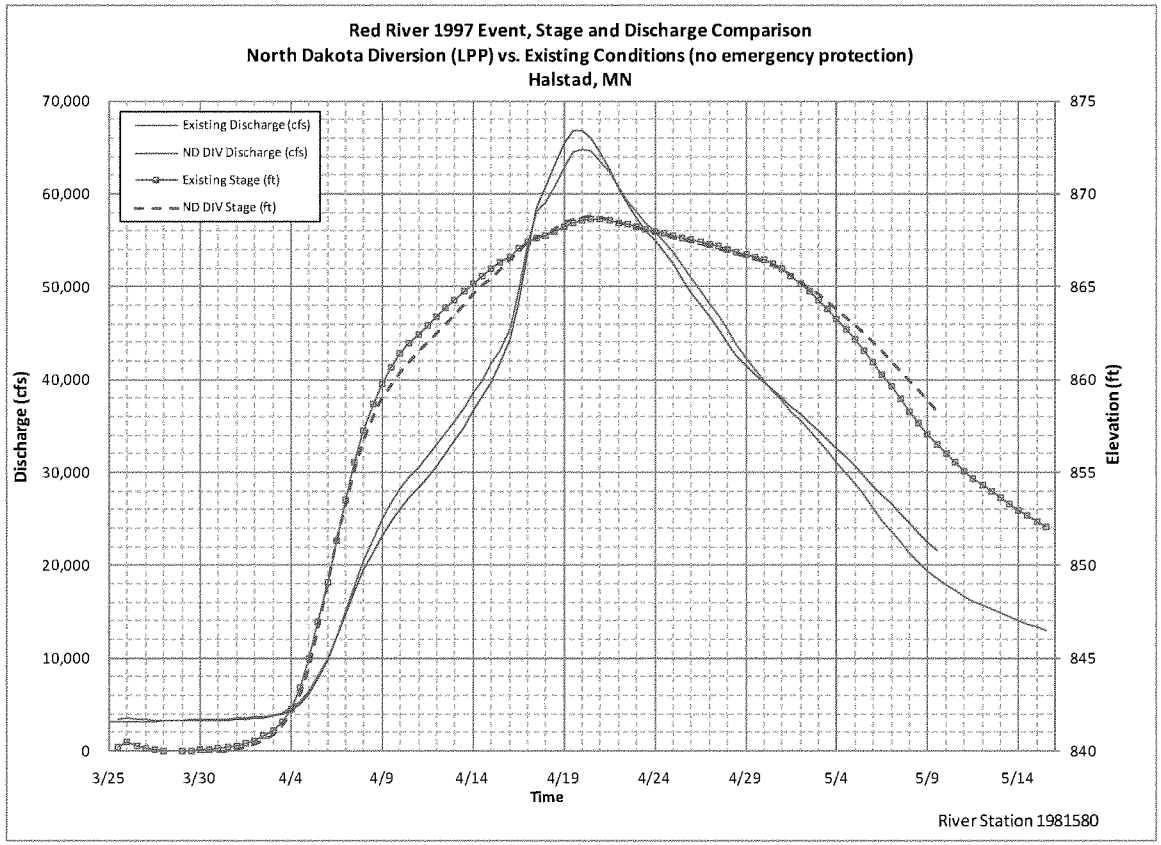


Figure C-E2- 59: 1997 Historical Flood Hydrographs for LPP @ Halstad

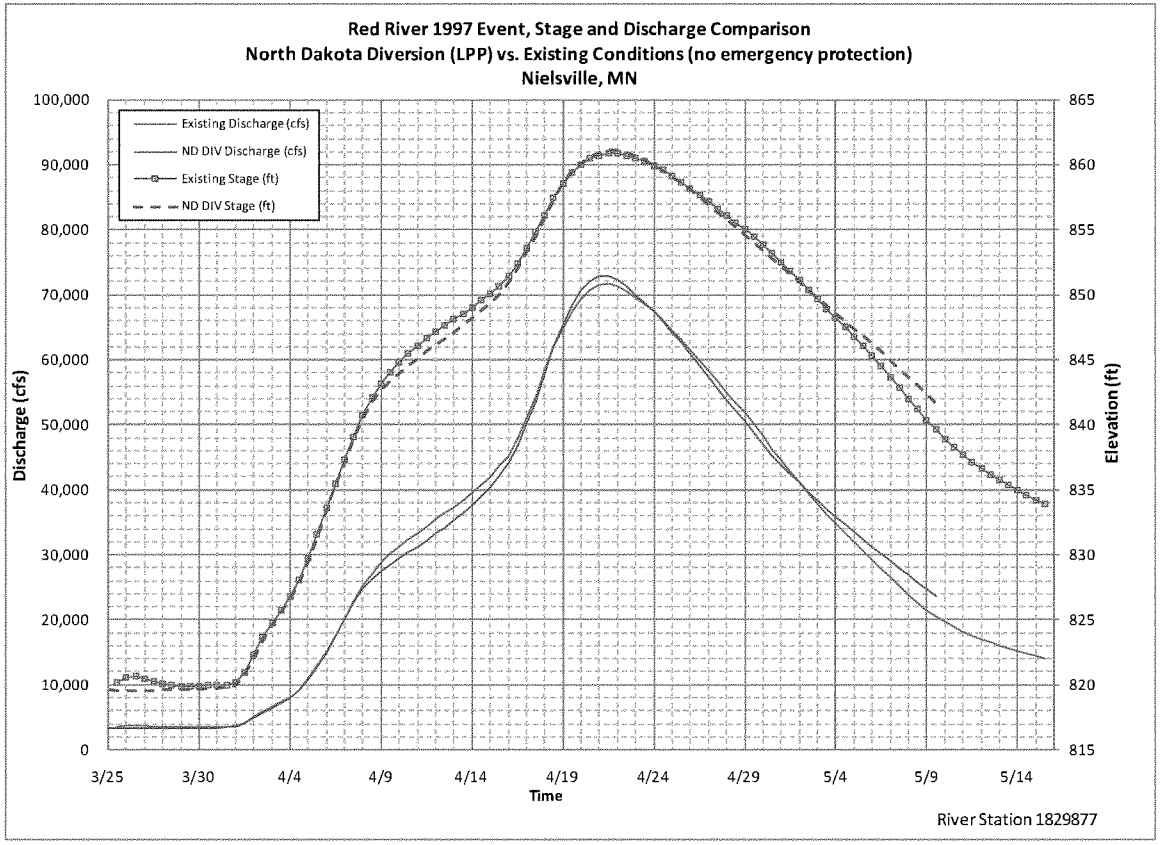


Figure C-E2- 60: 1997 Historical Flood Hydrographs for LPP @ Nielsville

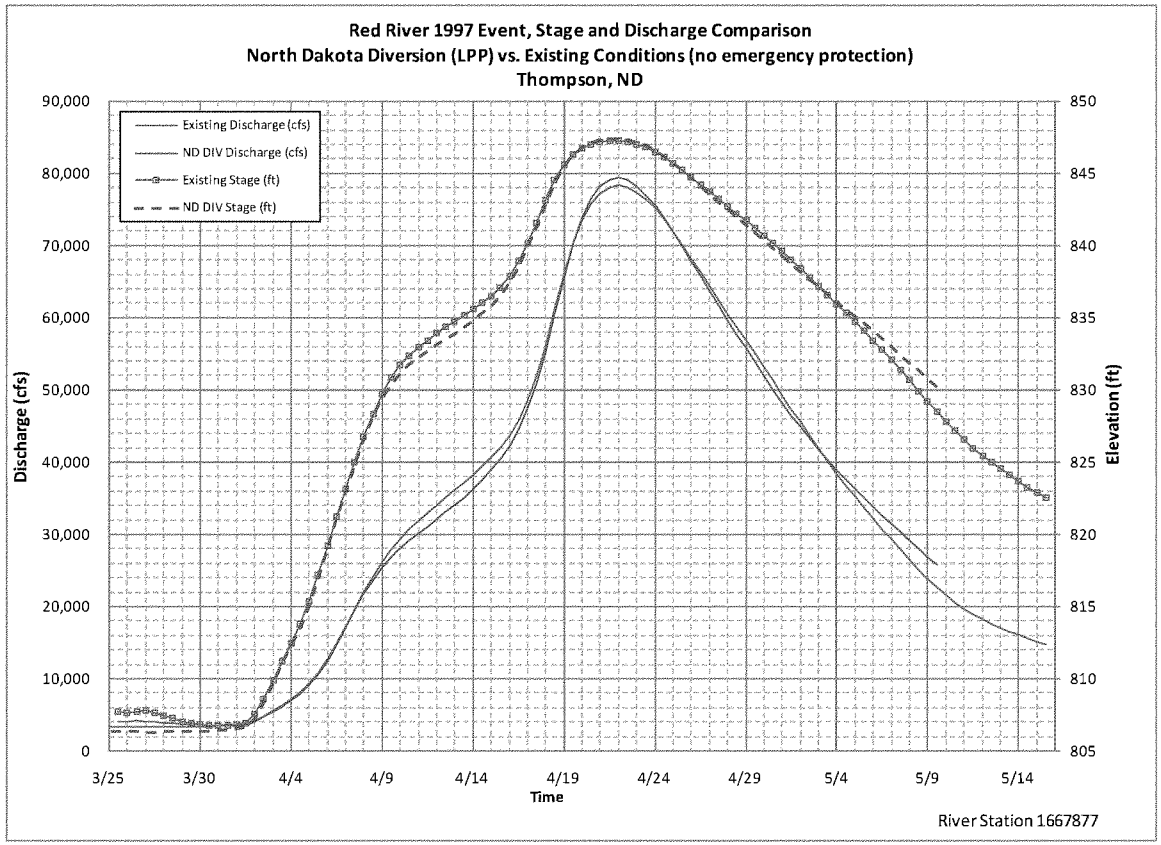


Figure C-E2- 61: 1997 Historical Flood Hydrographs for LPP @ Thompson

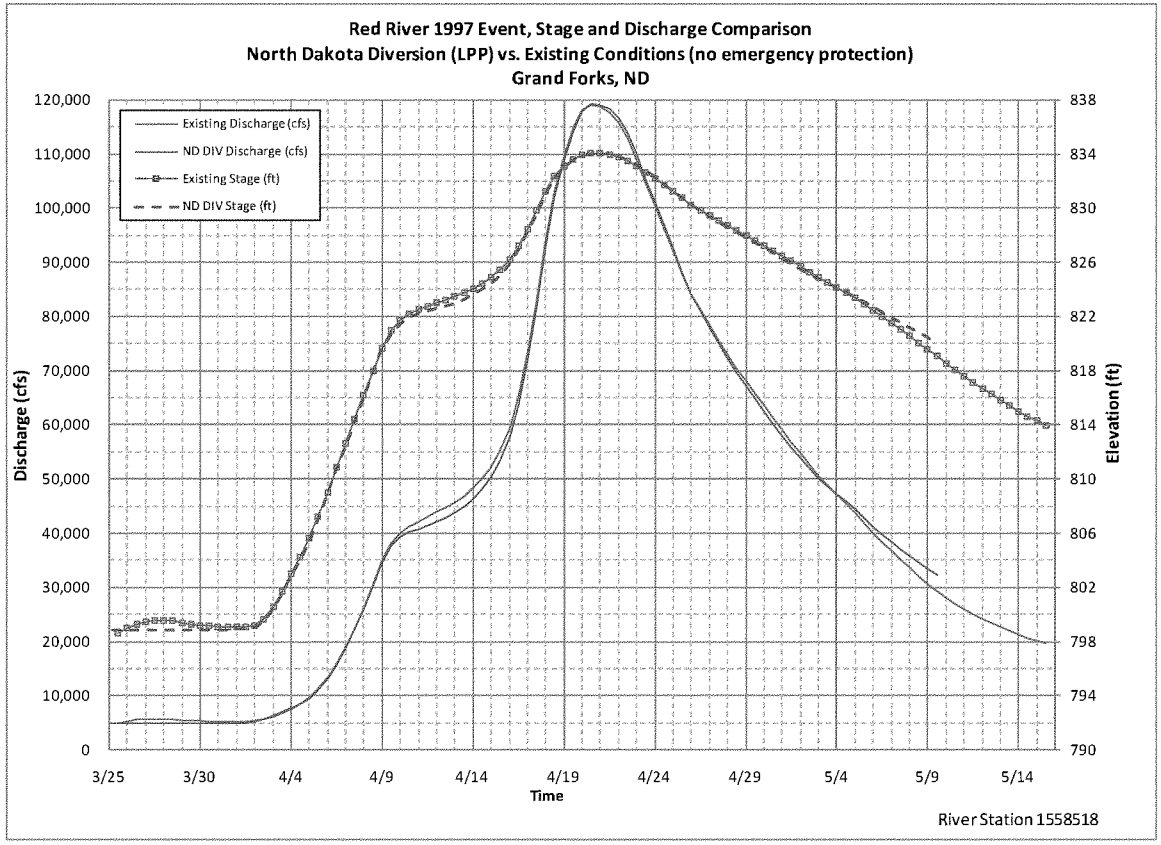


Figure C-E2- 62: 1997 Historical Flood Hydrographs for LPP @ Grand Forks

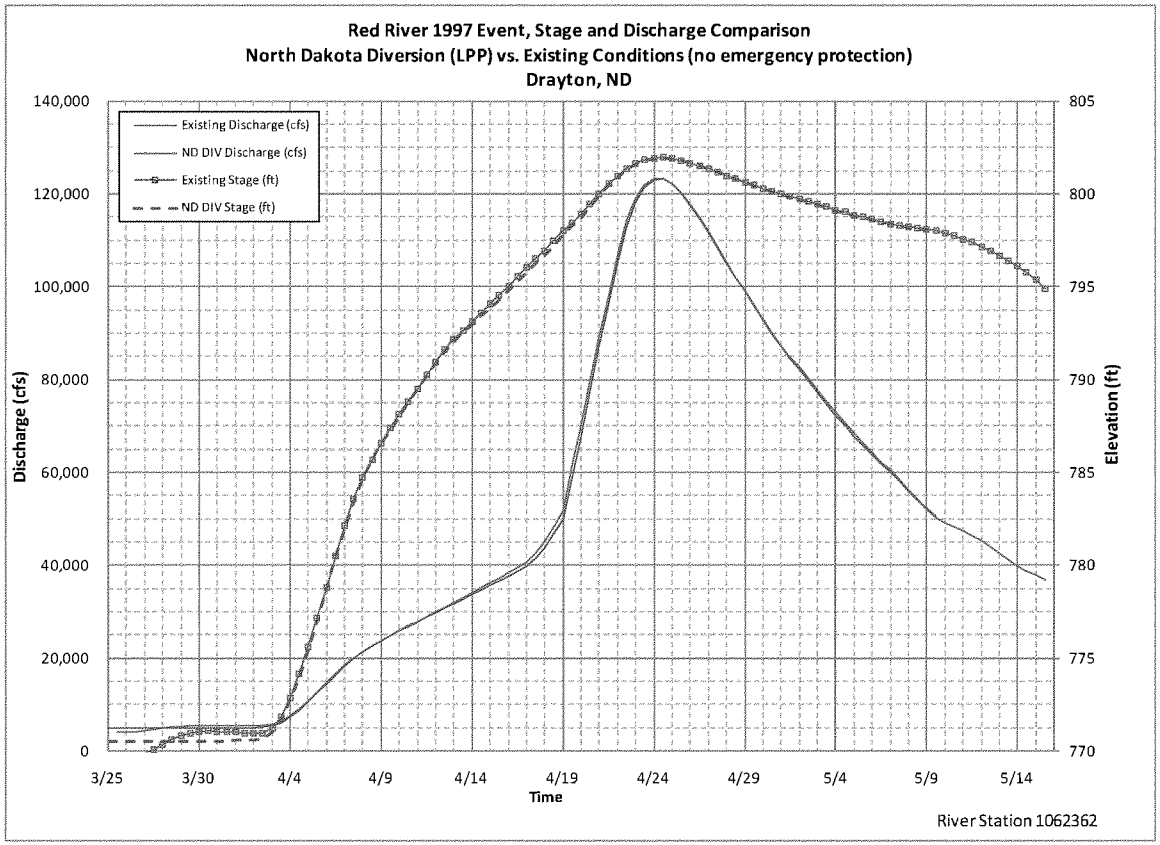


Figure C-E2- 63: 1997 Historical Flood Hydrographs for LPP @ Drayton

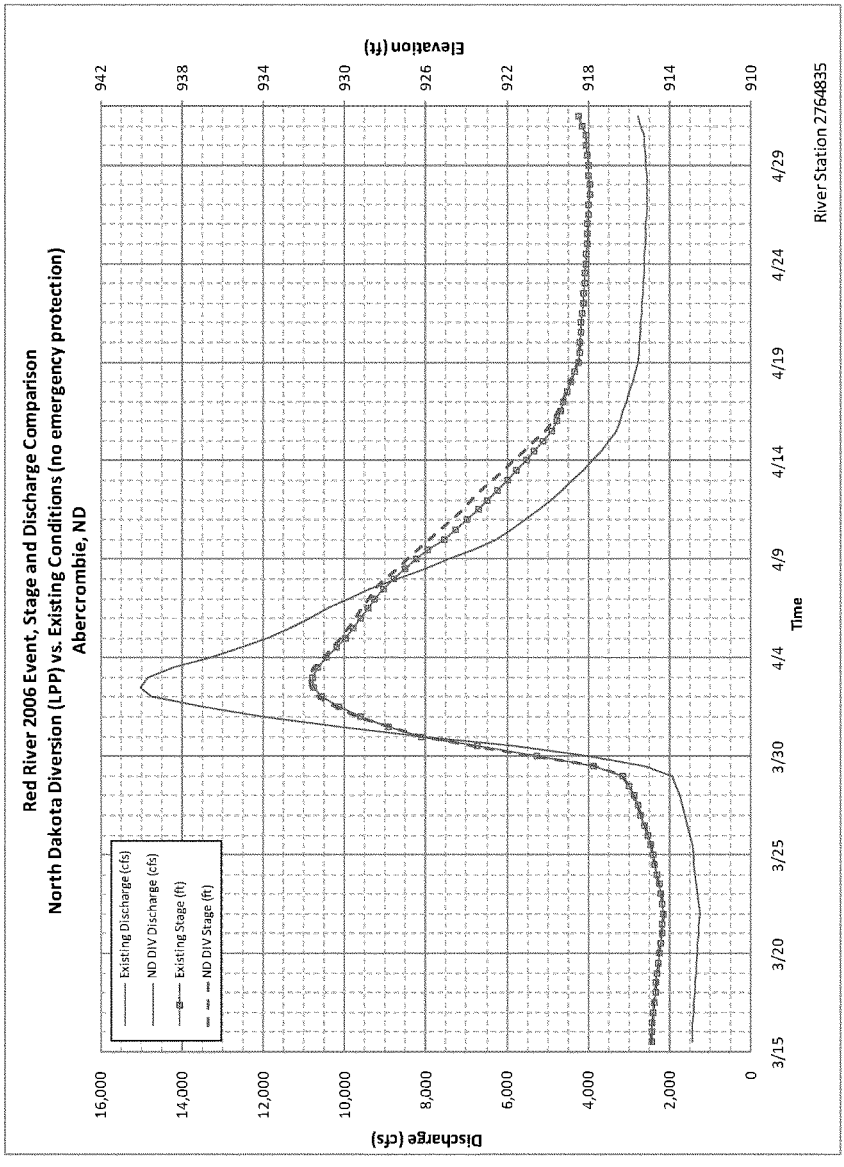


Figure C-E2- 64: 2006 Historical Flood Hydrographs for LPP @ Abercrombie

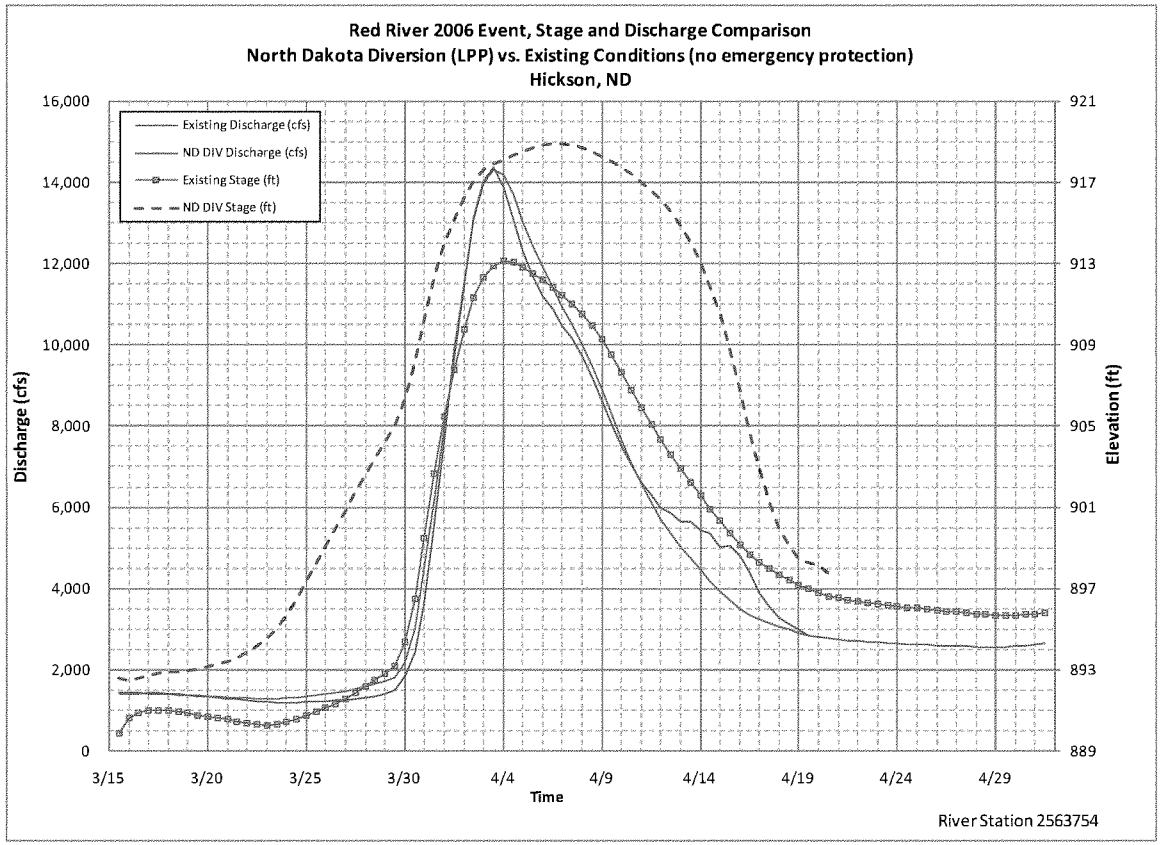


Figure C-E2- 65: 2006 Historical Flood Hydrographs for LPP @ Hickson

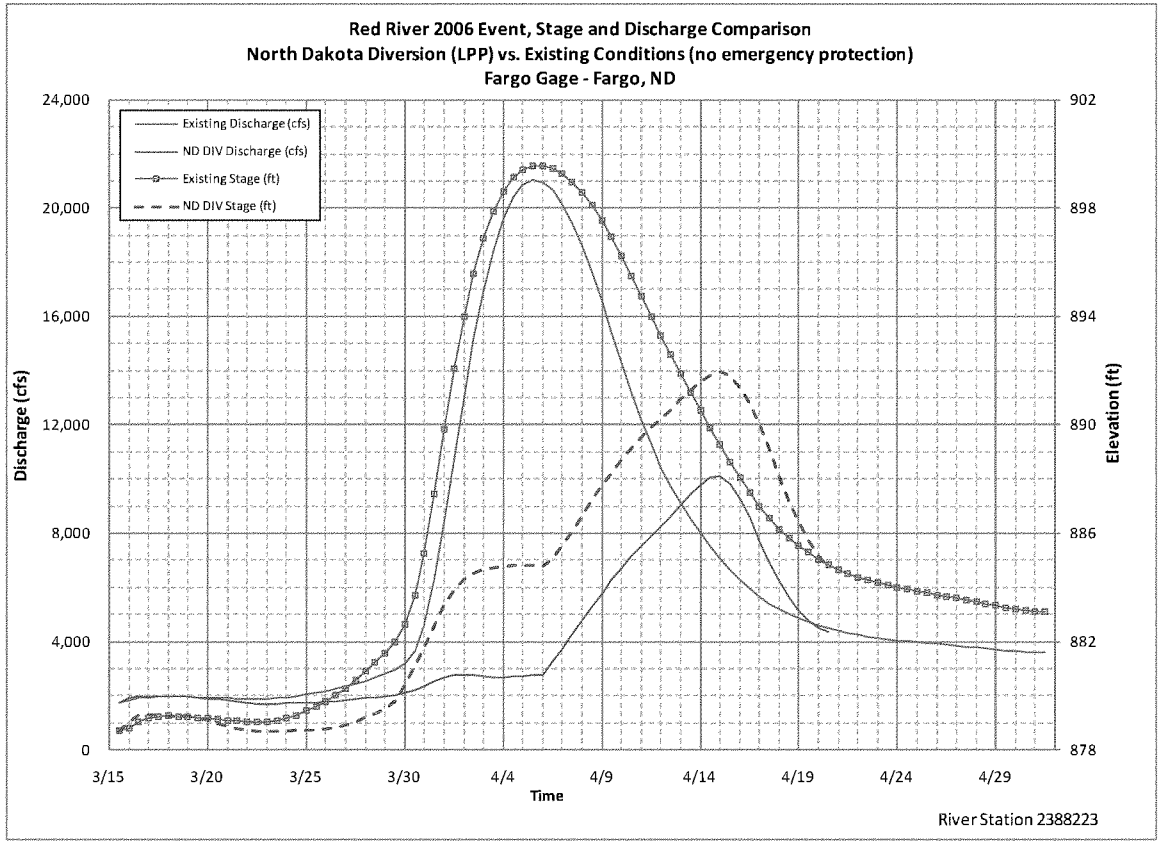


Figure C-E2- 66: 2006 Historical Flood Hydrographs for LPP @ Fargo

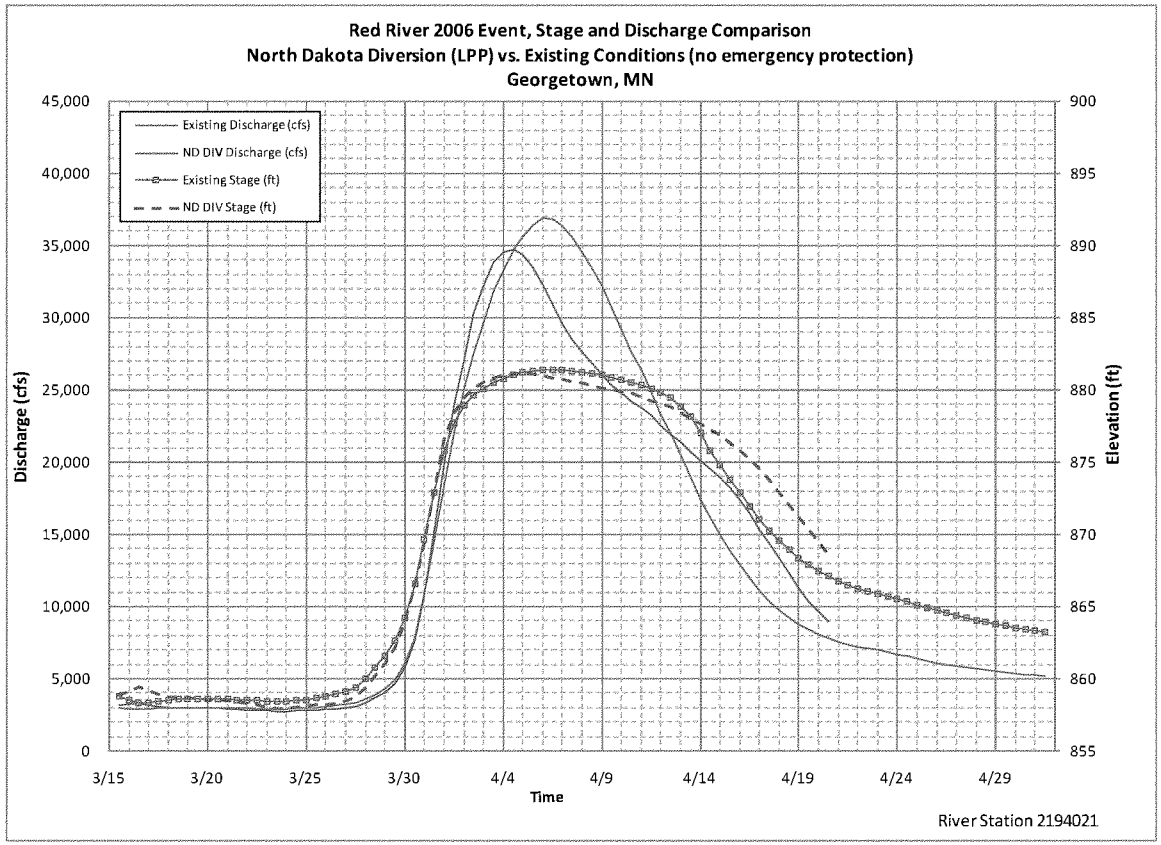


Figure C-E2- 67: 2006 Historical Flood Hydrographs for LPP @ Georgetown

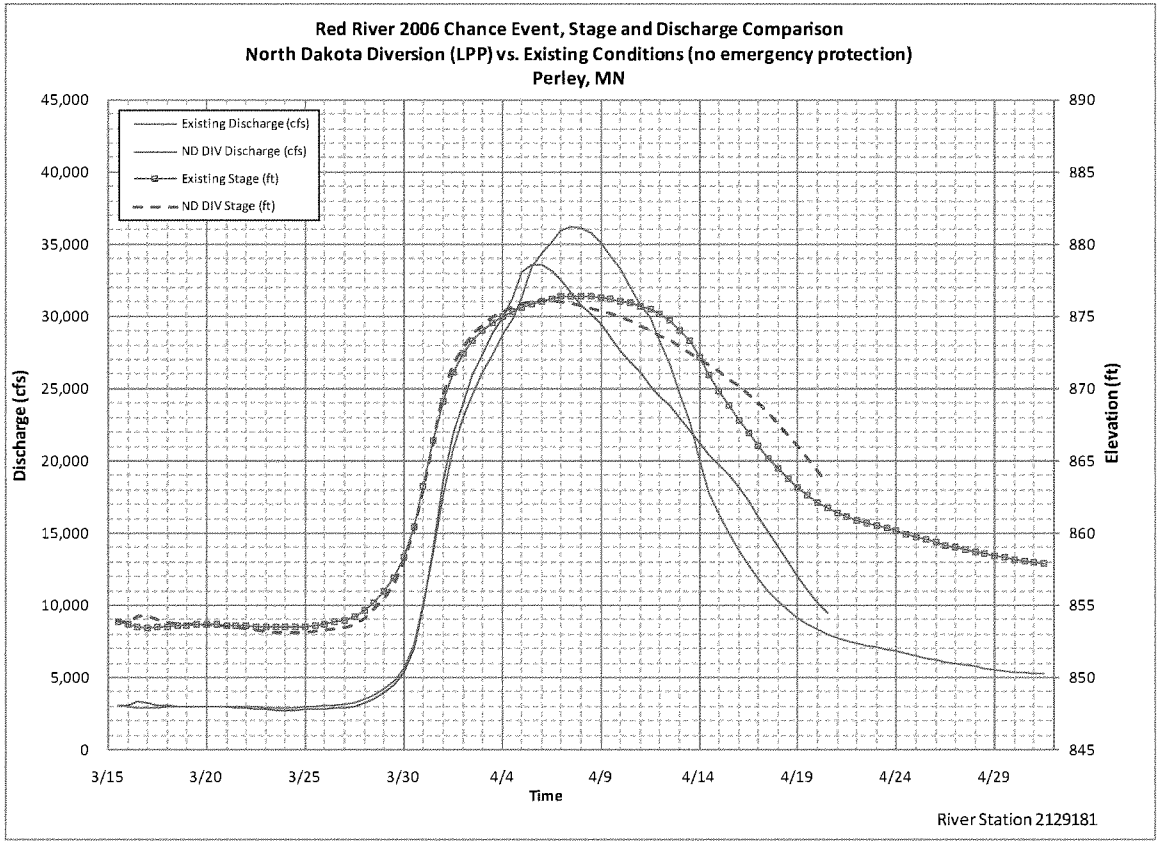


Figure C-E2- 68: 2006 Historical Flood Hydrographs for LPP @ Perley

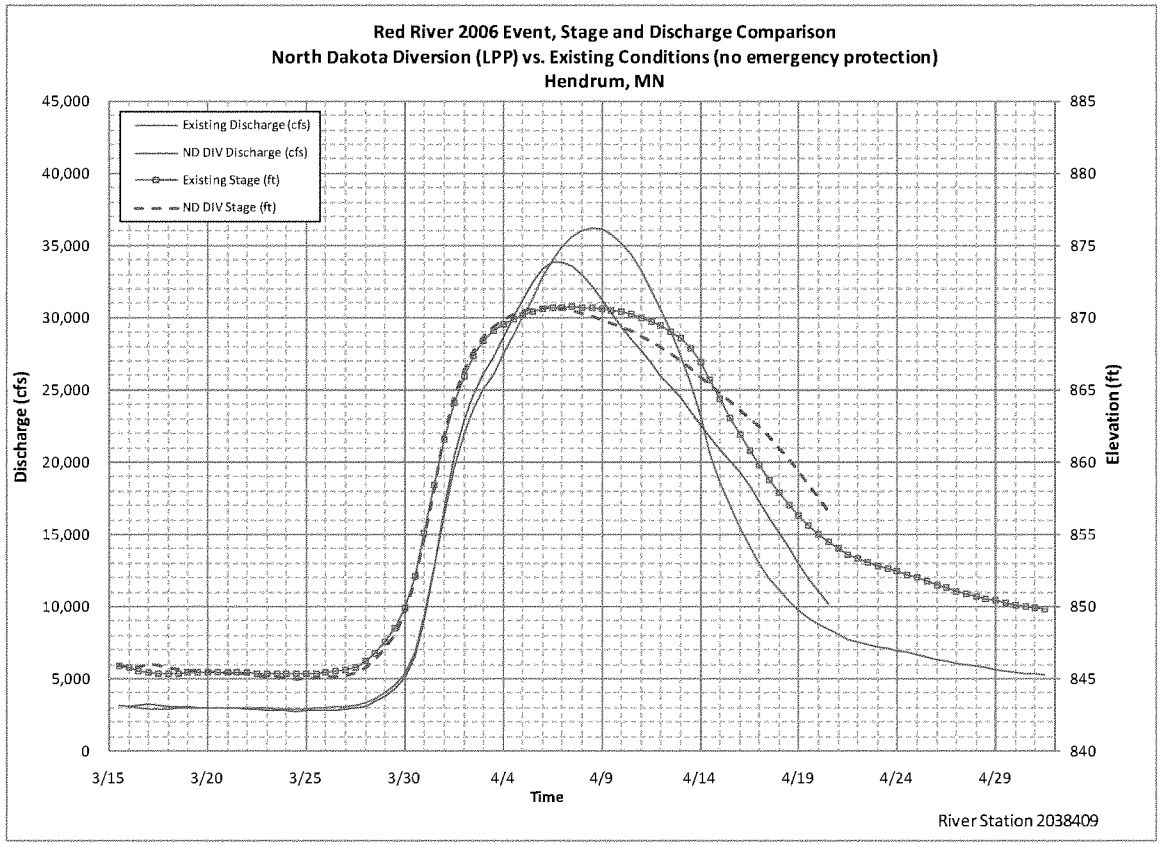


Figure C-E2- 69: 2006 Historical Flood Hydrographs for LPP @ Hendrum

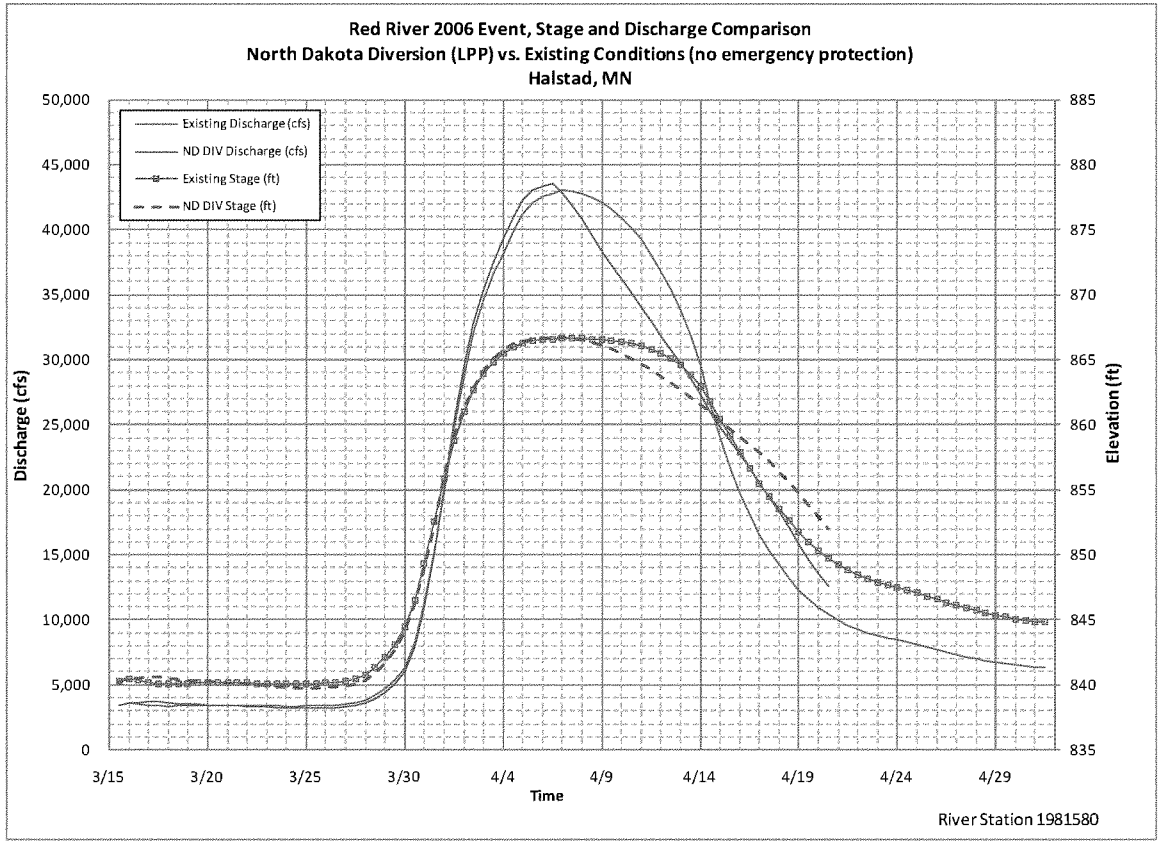


Figure C-E2- 70: 2006 Historical Flood Hydrographs for LPP @ Halstad

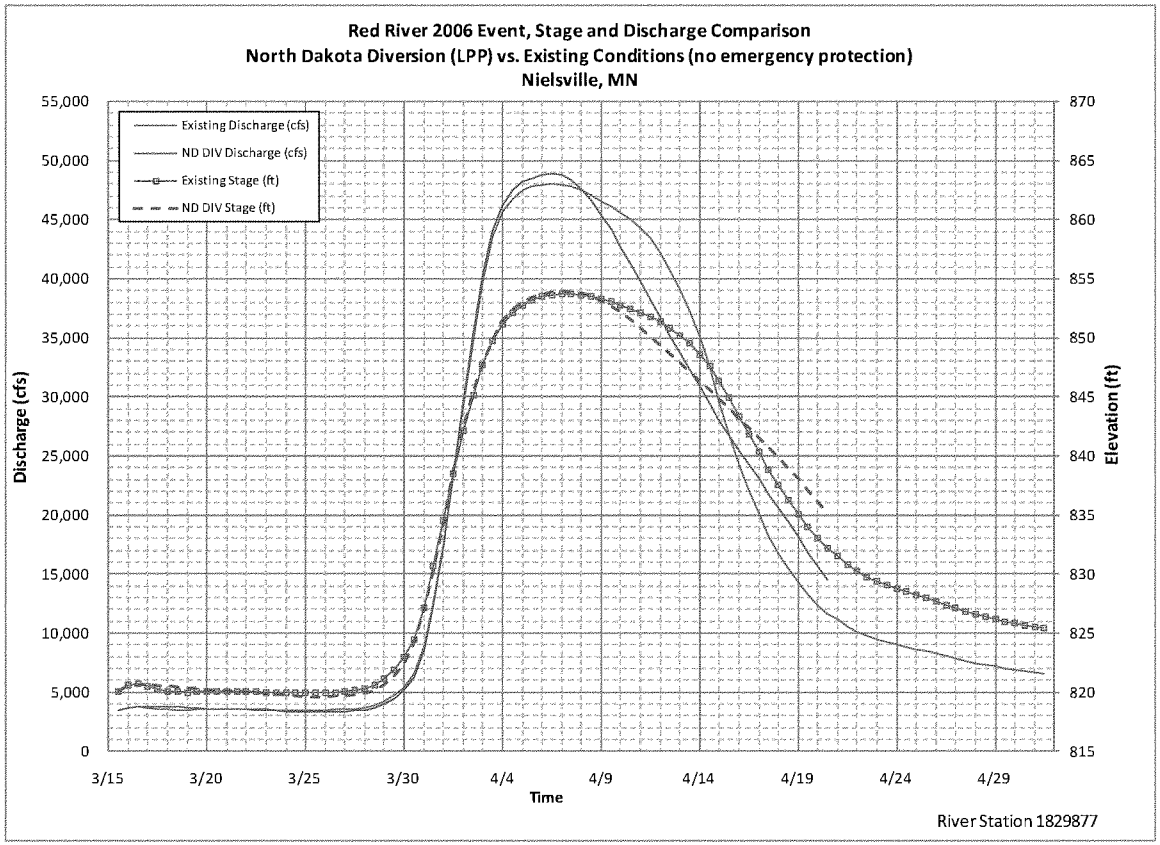


Figure C-E2- 71: 2006 Historical Flood Hydrographs for LPP @ Nielsville

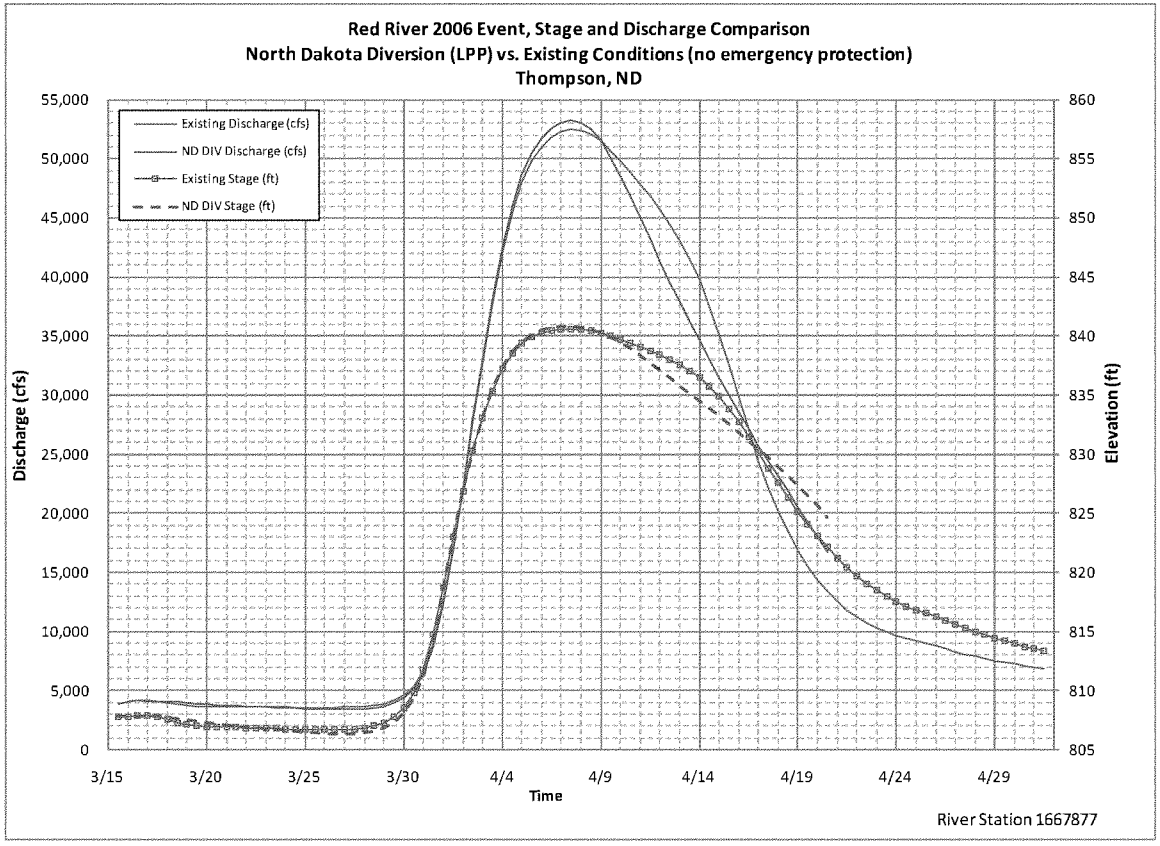


Figure C-E2- 72: 2006 Historical Flood Hydrographs for LPP @ Thompson

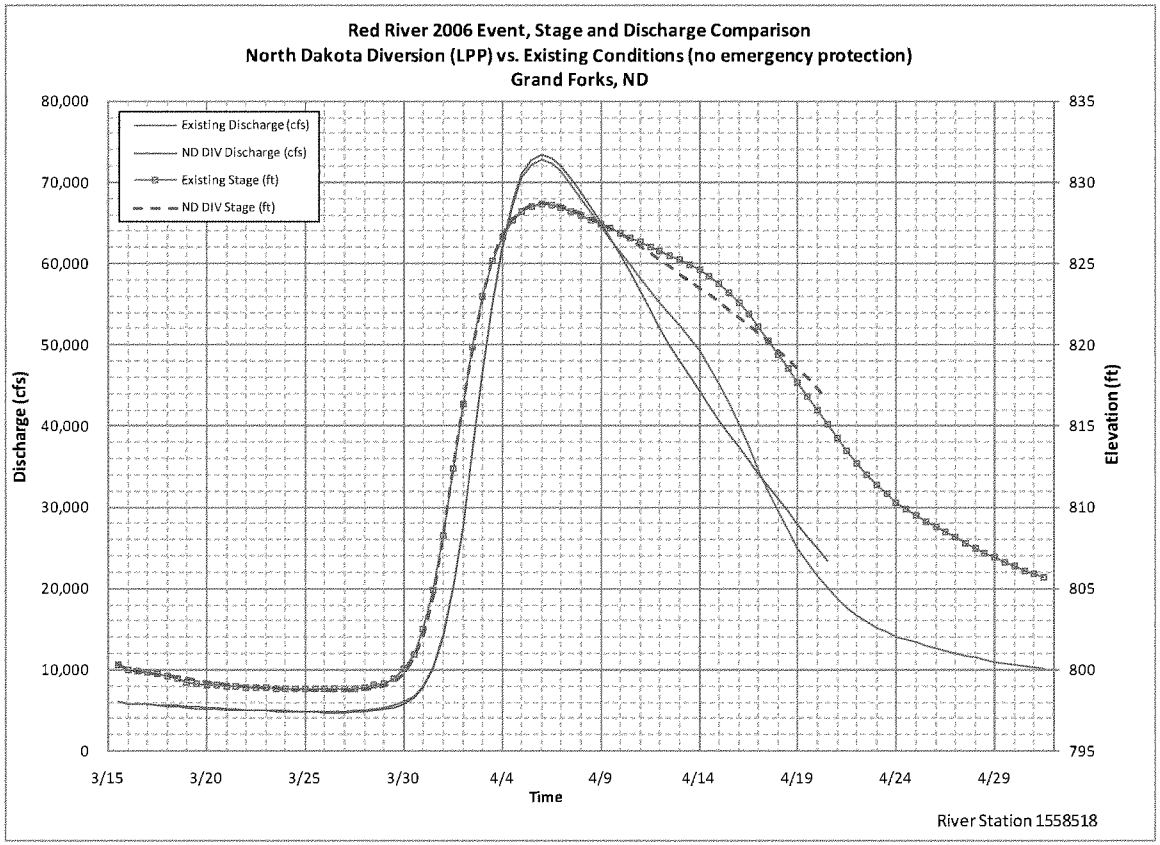


Figure C-E2- 73: 2006 Historical Flood Hydrographs for LPP @ Grand Forks

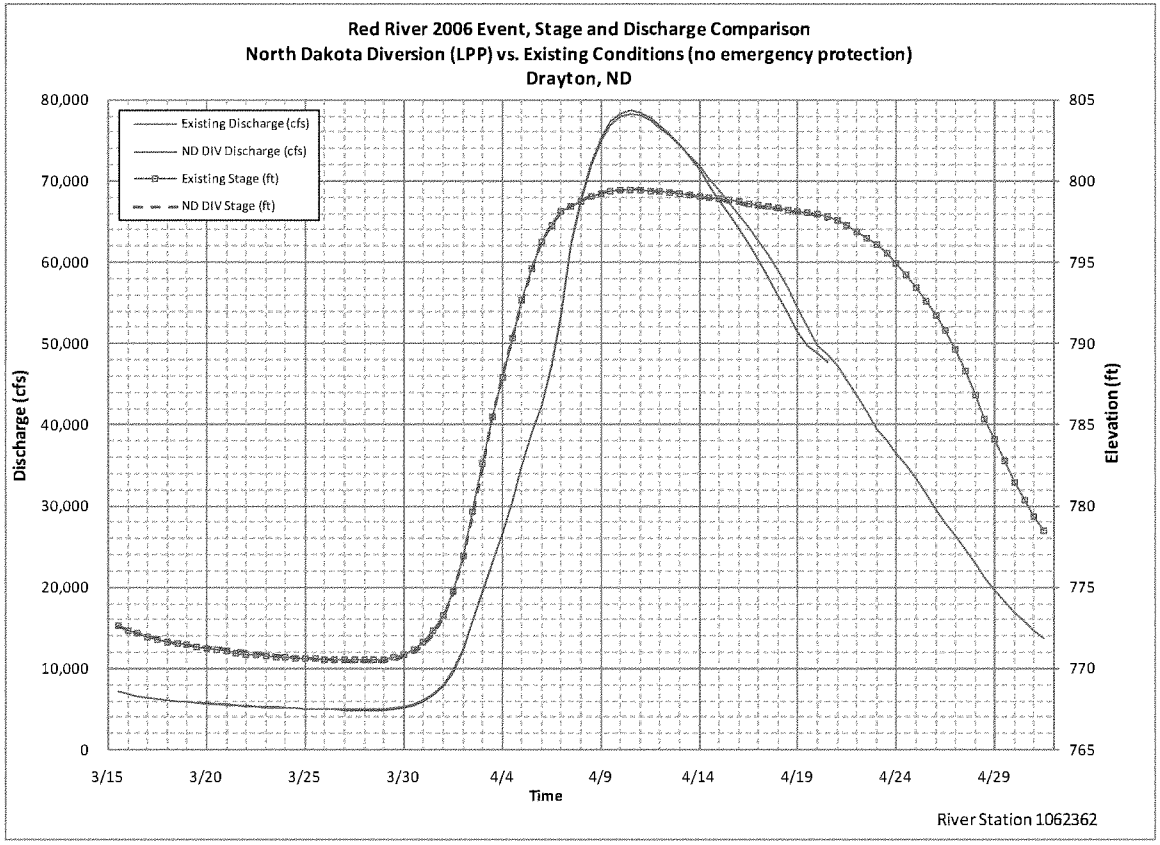


Figure C-E2- 74: 2006 Historical Flood Hydrographs for LPP @ Drayton

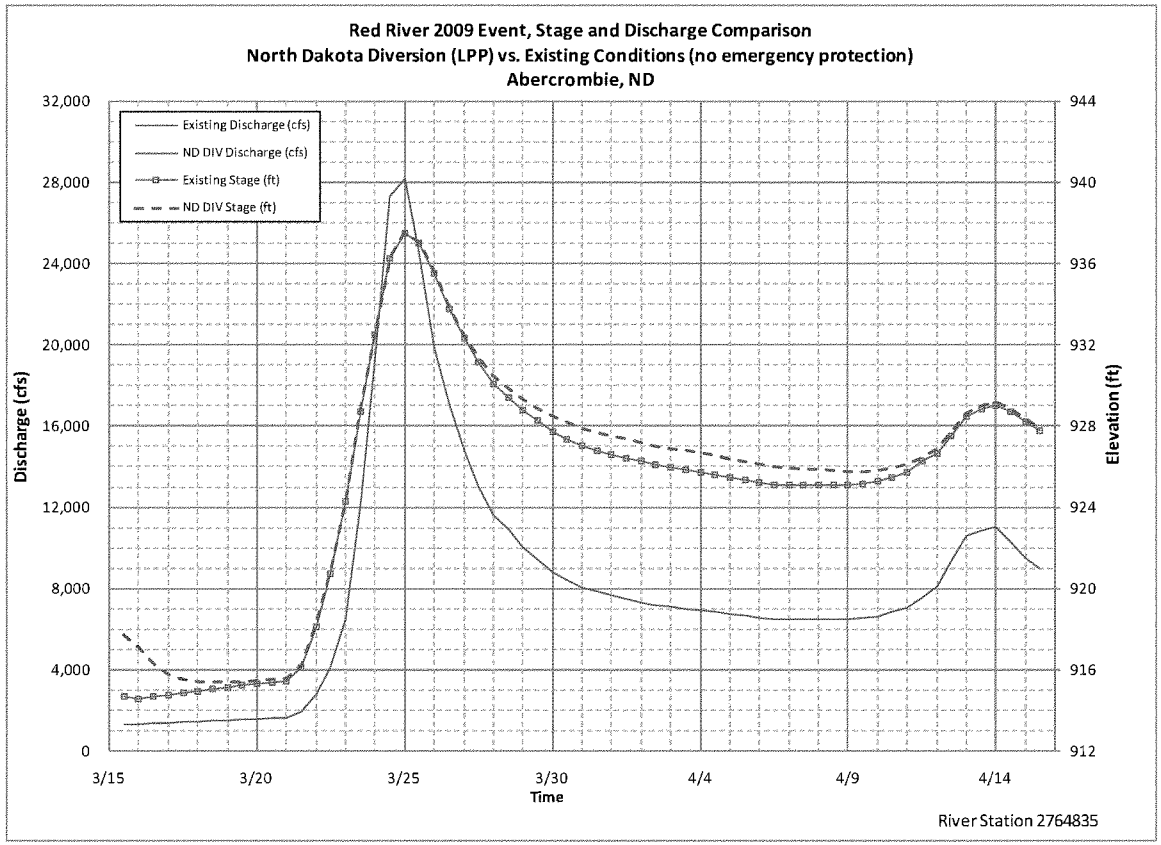


Figure C-E2- 75: 2009 Historical Flood Hydrographs for LPP @ Abercrombie

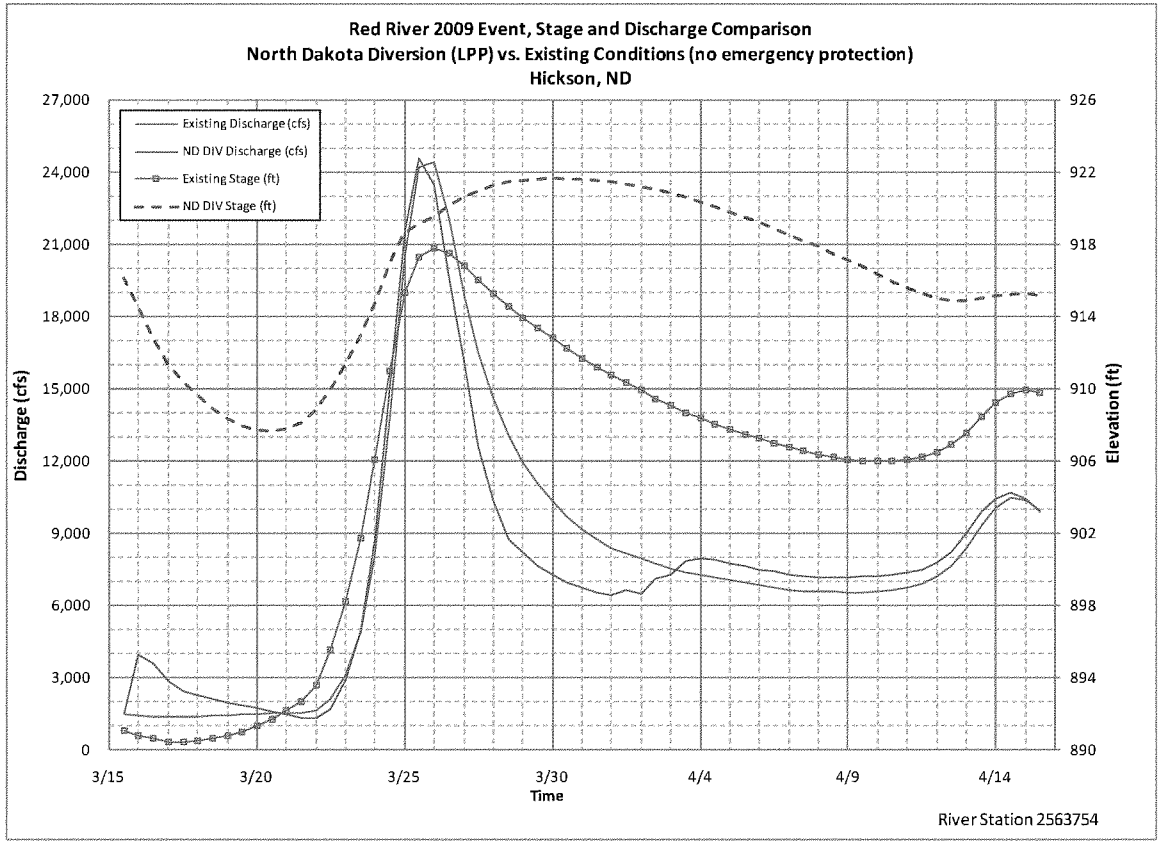


Figure C-E2- 76: 2009 Historical Flood Hydrographs for LPP @ Hickson

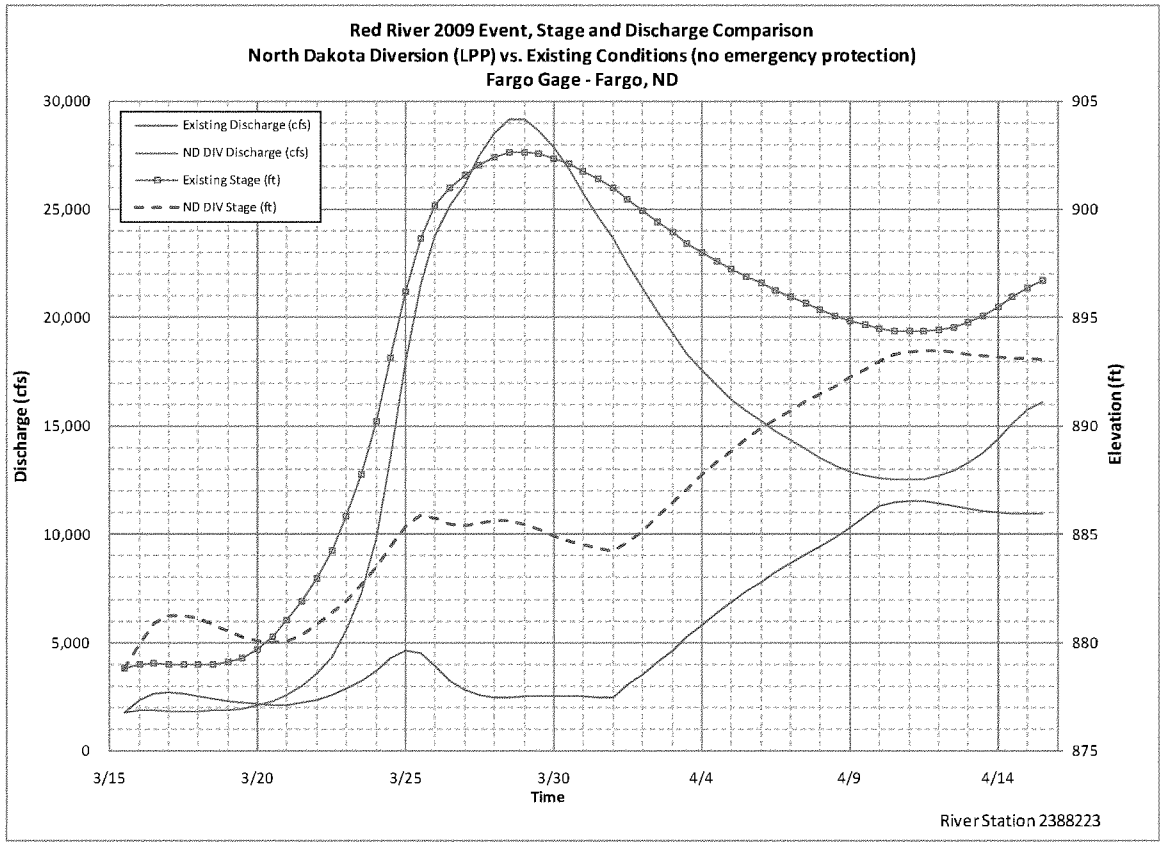


Figure C-E2- 77: 2009 Historical Flood Hydrographs for LPP @ Fargo

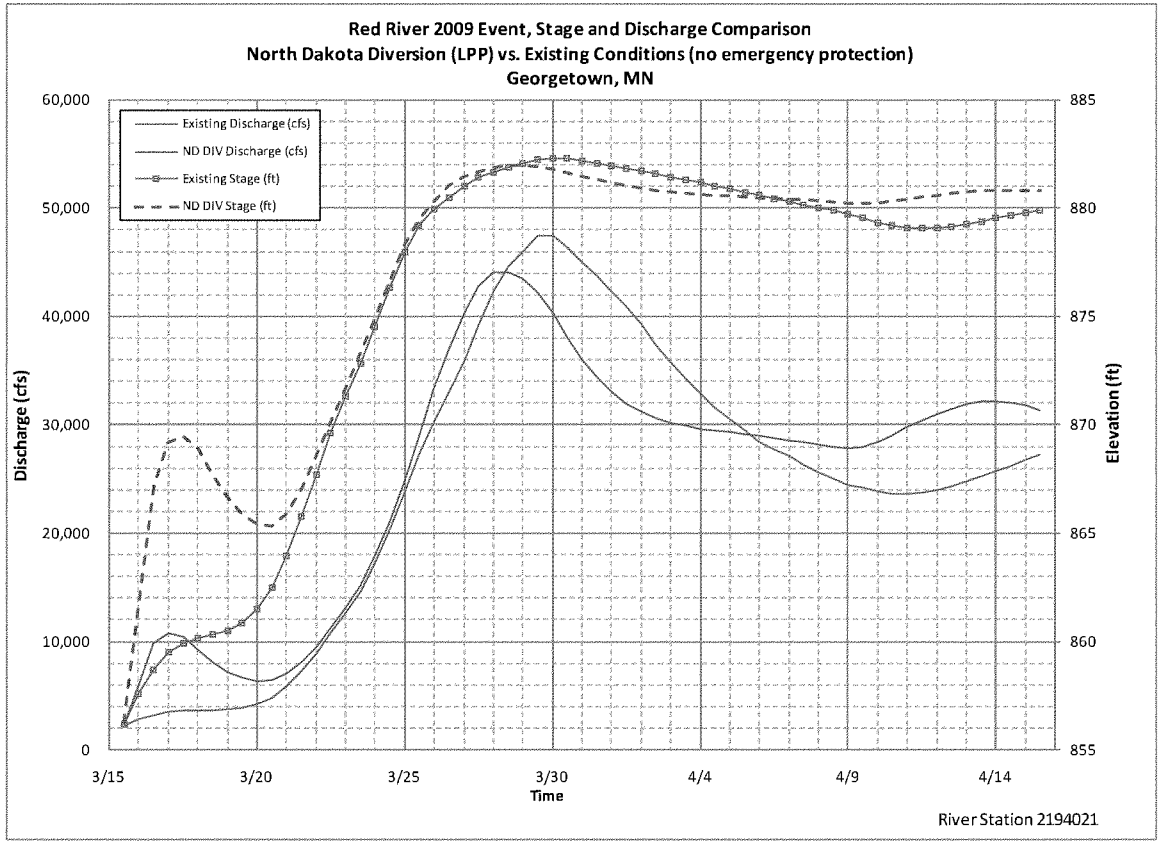


Figure C-E2- 78: 2009 Historical Flood Hydrographs for LPP @ Georgetown

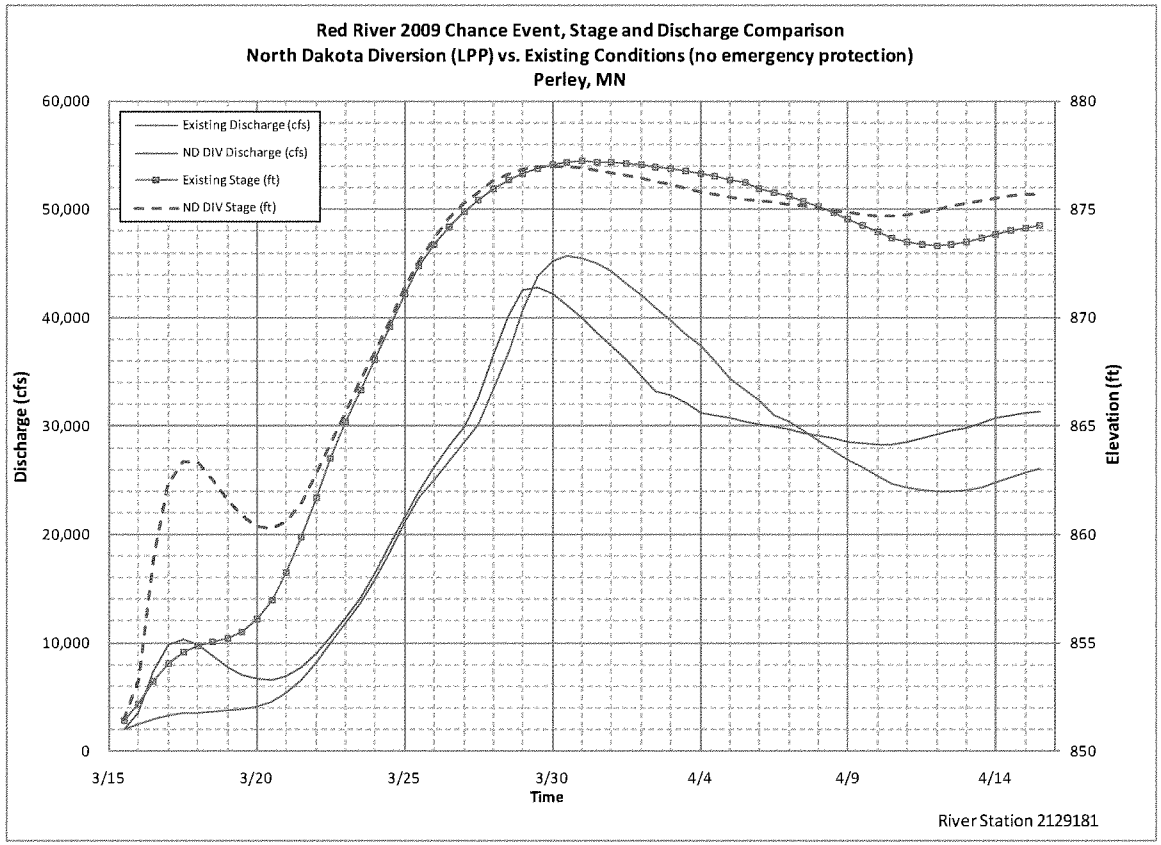


Figure C-E2- 79: 2009 Historical Flood Hydrographs for LPP @ Perley

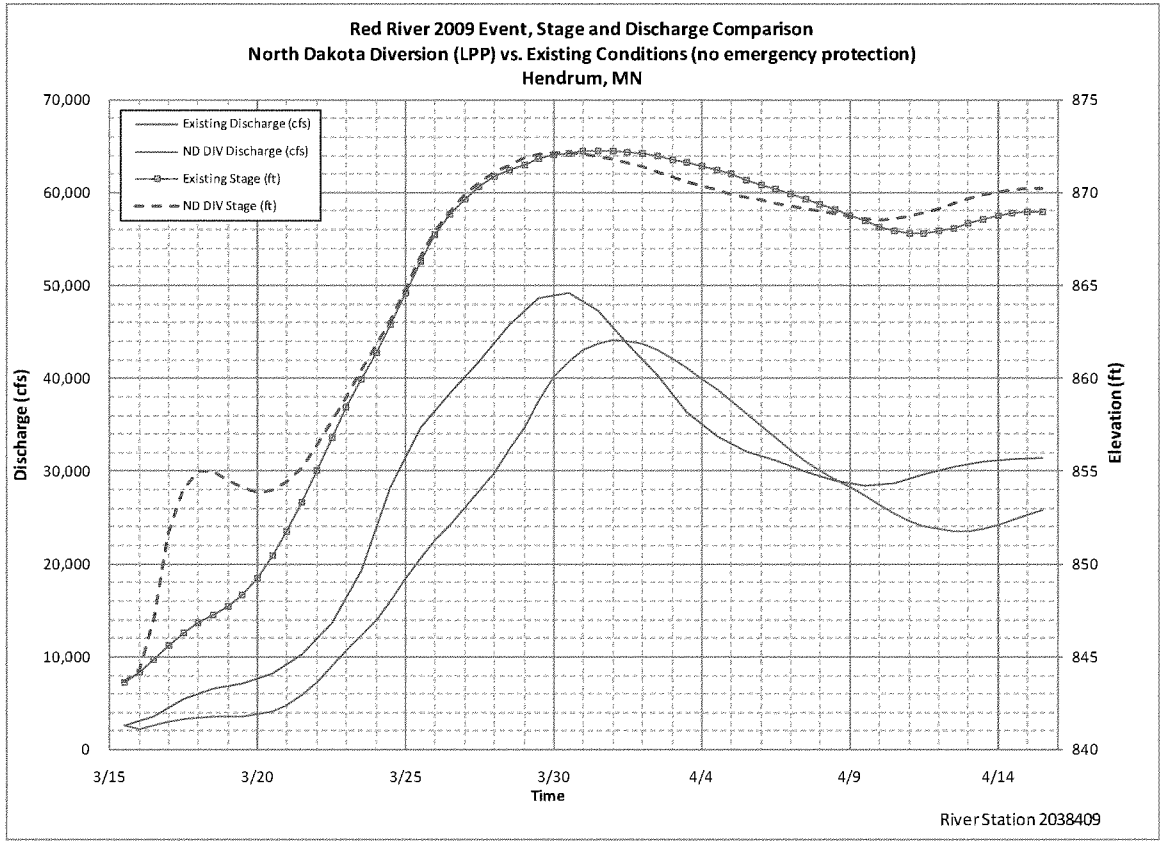


Figure C-E2- 80: 2009 Historical Flood Hydrographs for LPP @ Hendrum

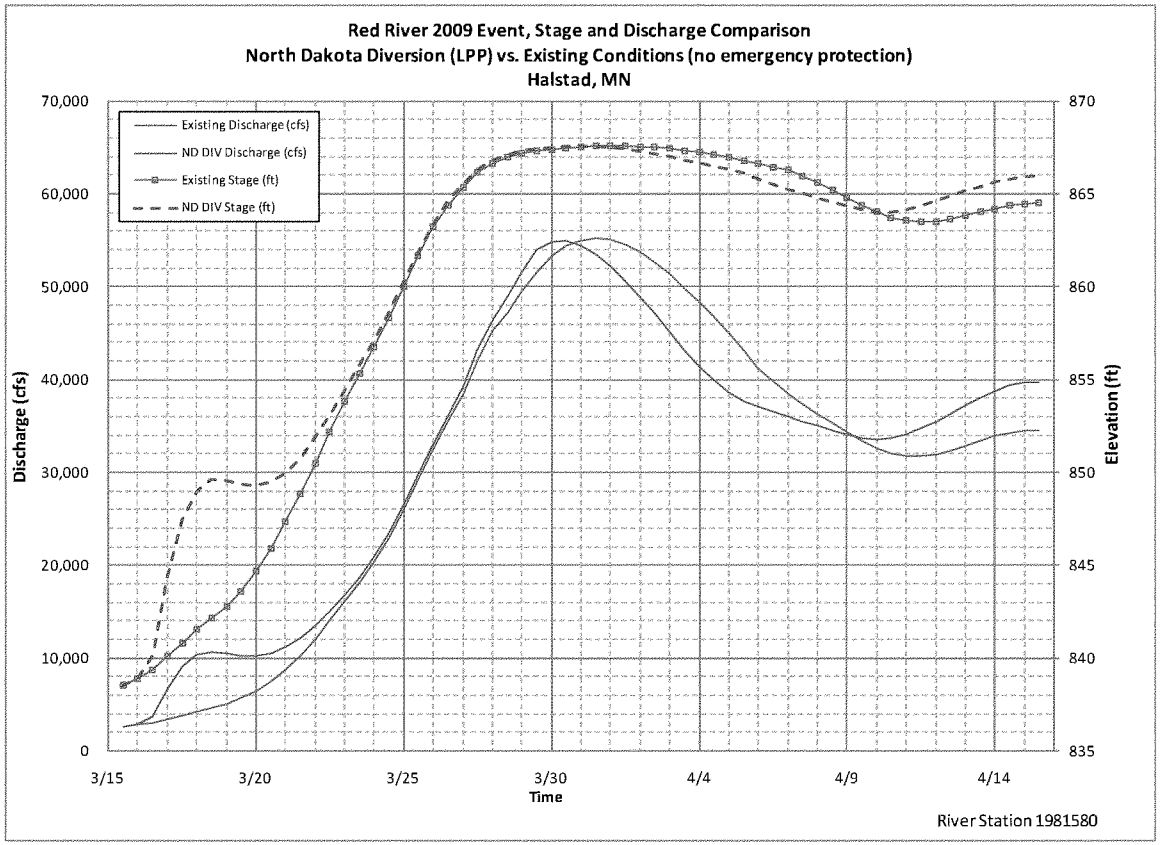


Figure C-E2- 81: 2009 Historical Flood Hydrographs for LPP @ Halstad

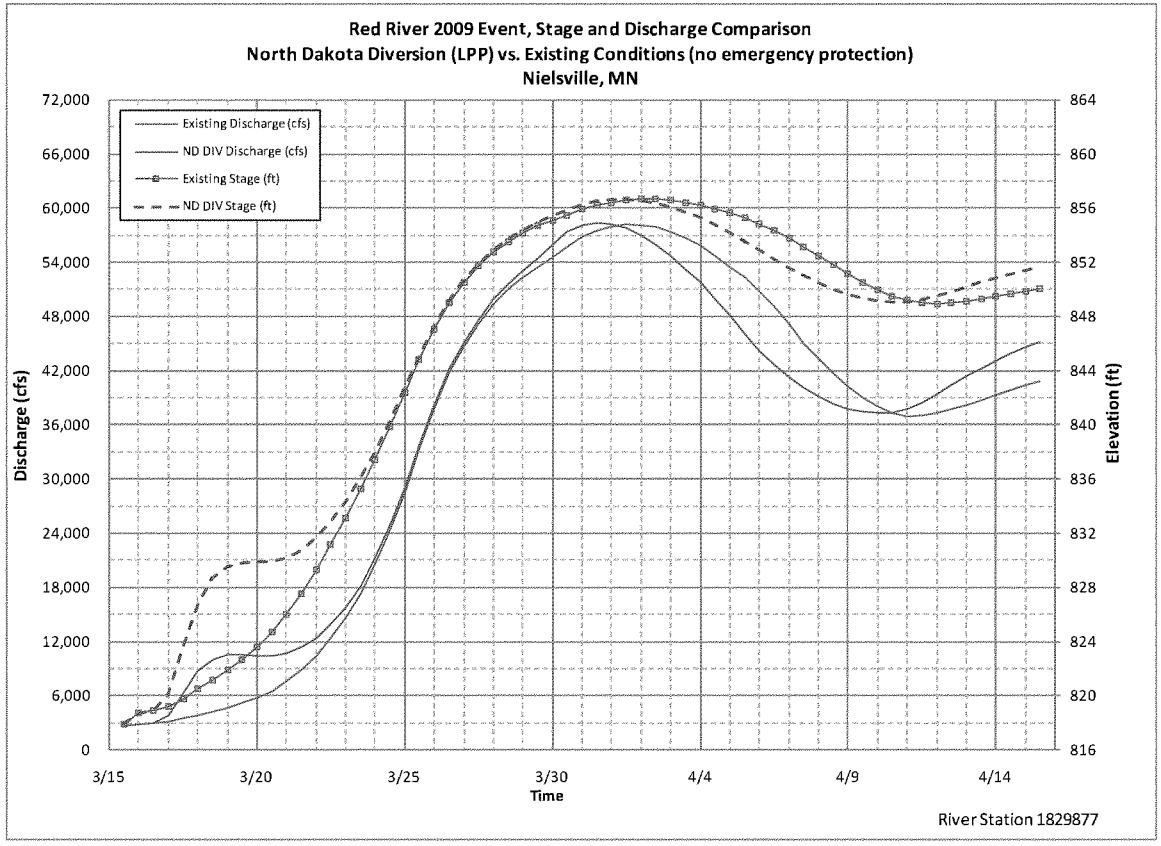


Figure C-E2- 82: 2009 Historical Flood Hydrographs for LPP @ Nielsville

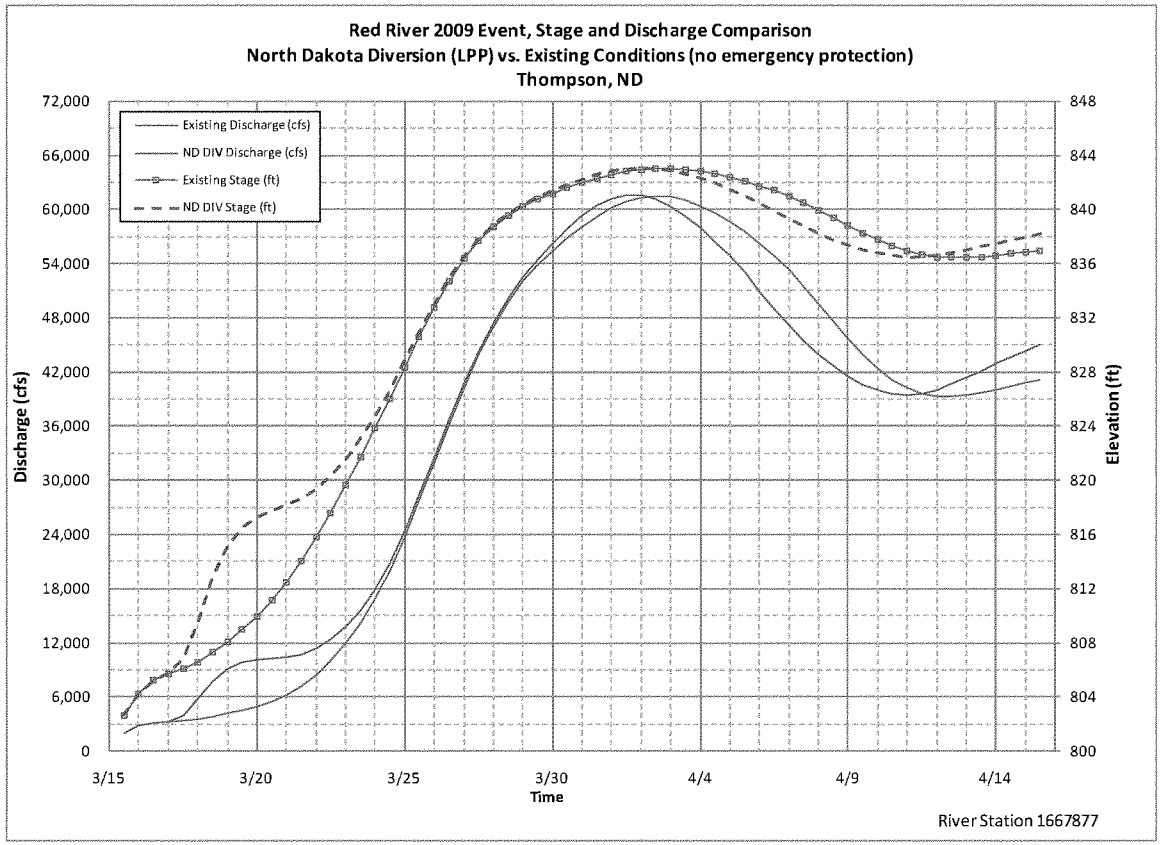


Figure C-E2- 83: 2009 Historical Flood Hydrographs for LPP @ Thompson

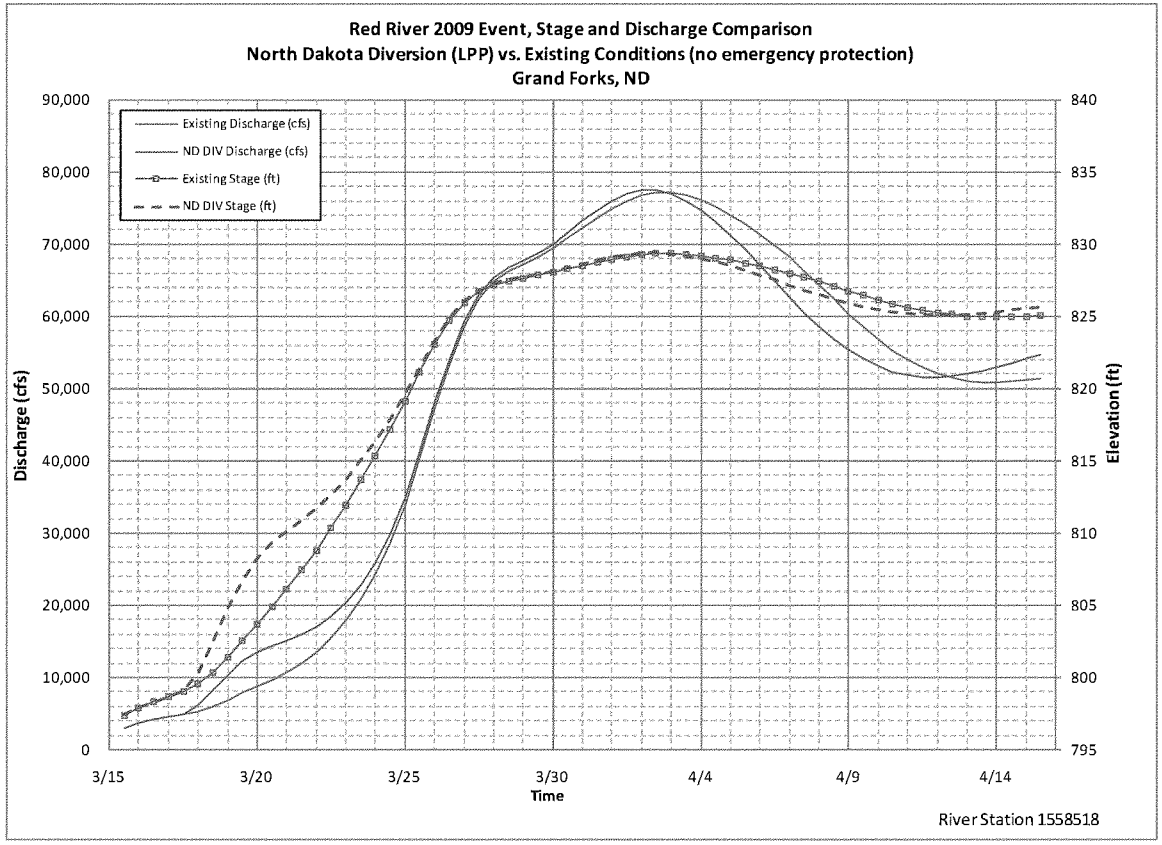


Figure C-E2- 84: 2009 Historical Flood Hydrographs for LPP @ Grand Forks

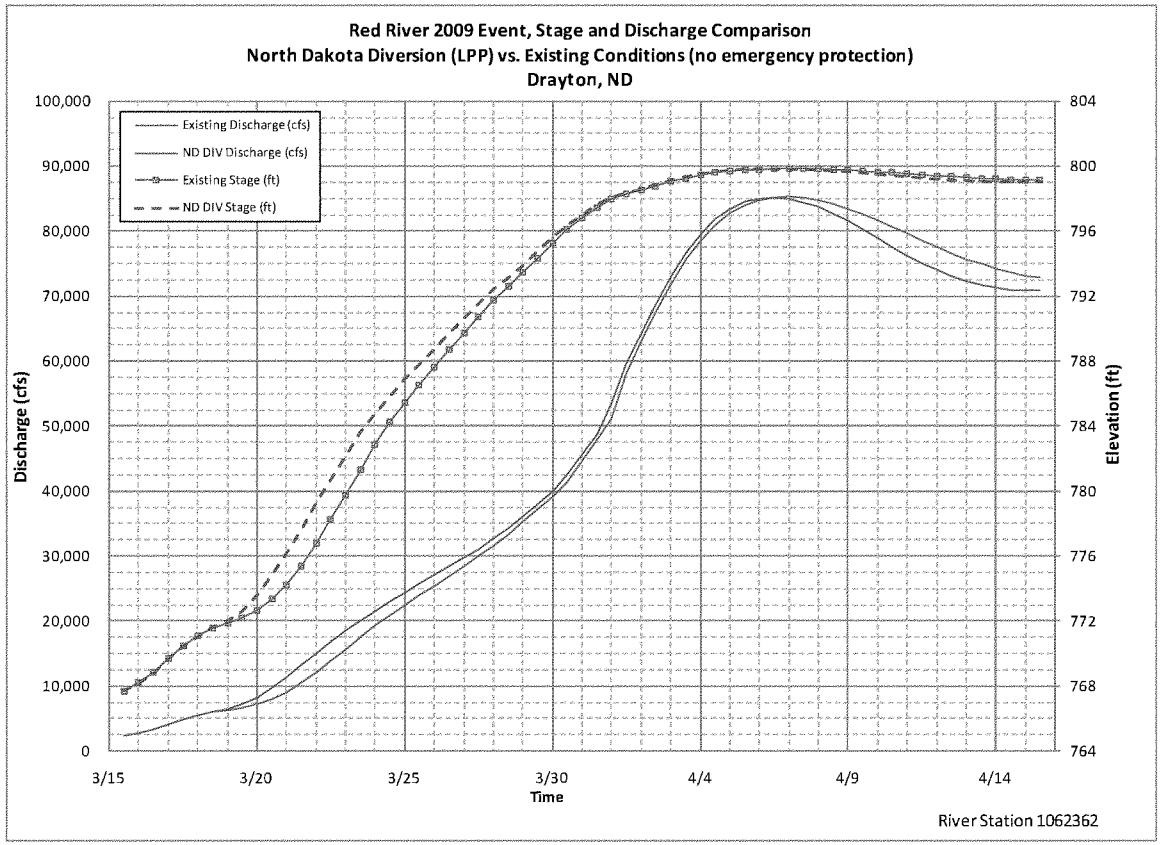


Figure C-E2- 85: 2009 Historical Flood Hydrographs for LPP @ Drayton

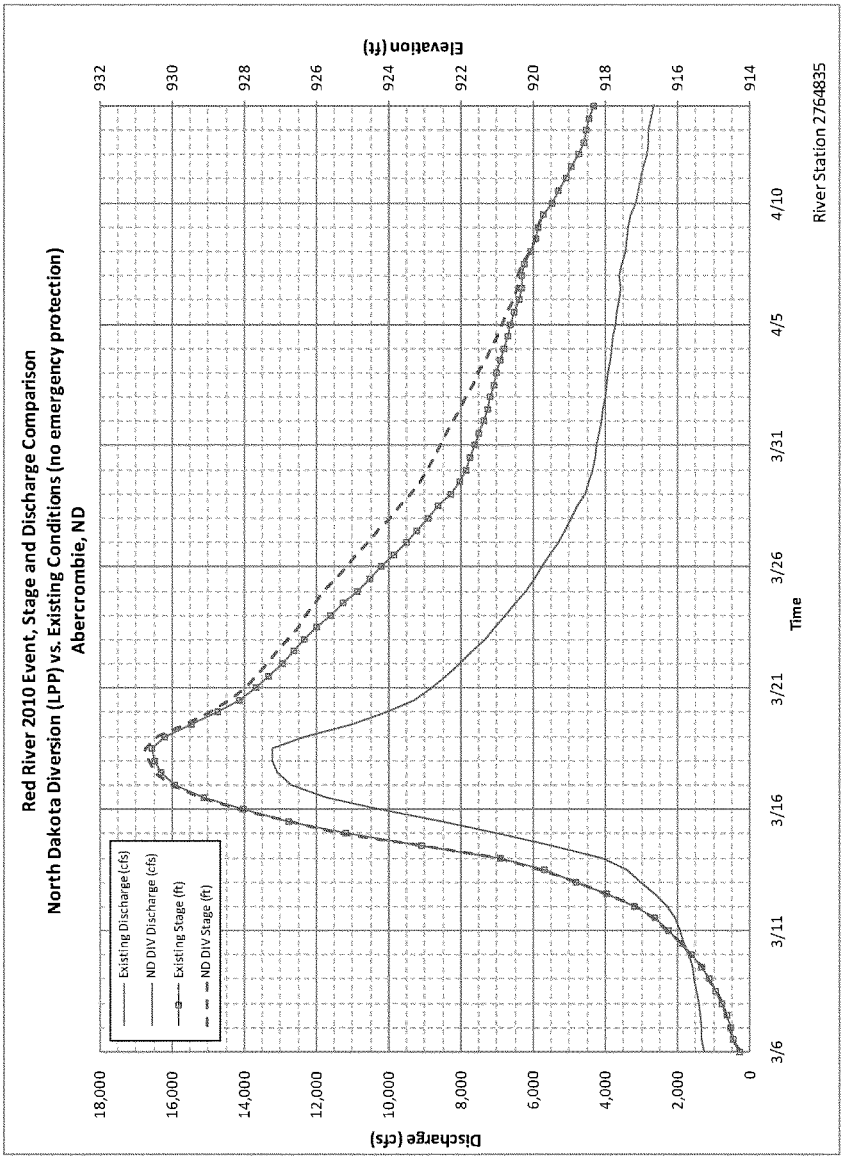


Figure C-E2- 86: 2010 Historical Flood Hydrographs for LPP @ Abercrombie

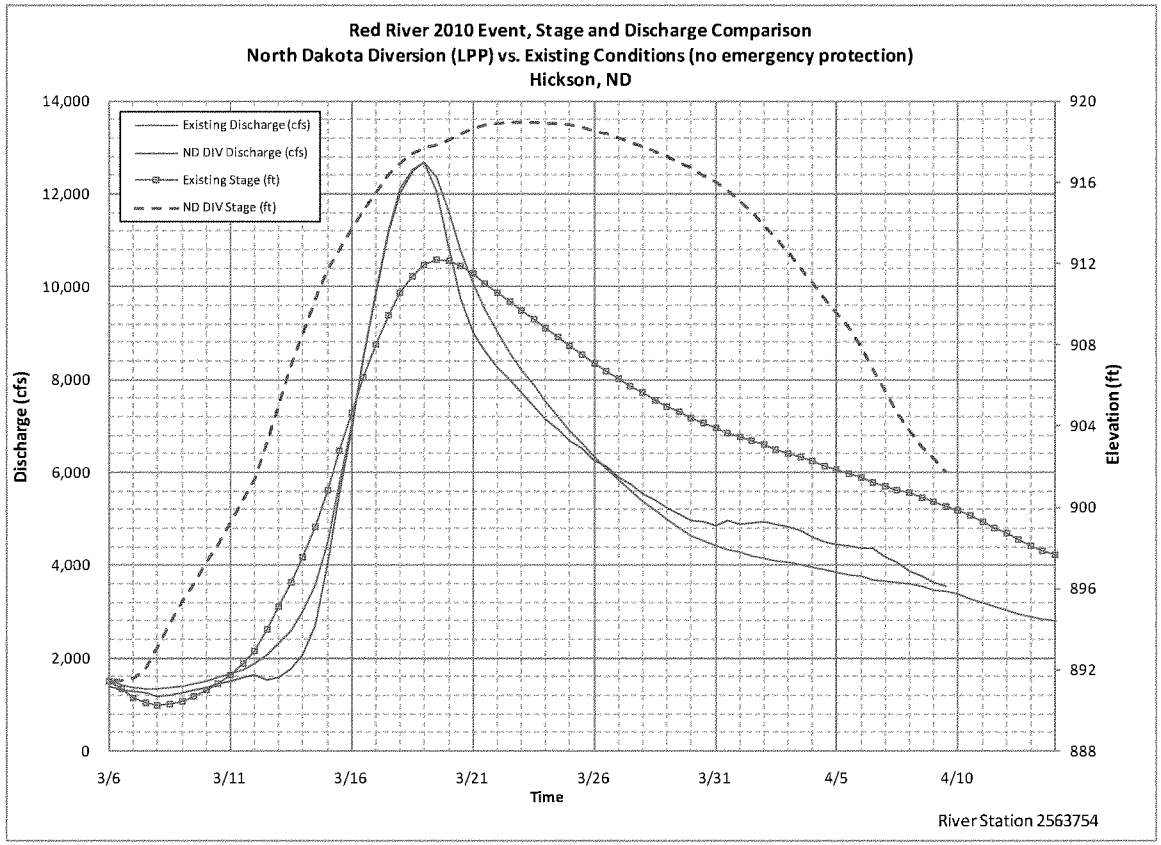


Figure C-E2- 87: 2010 Historical Flood Hydrographs for LPP @ Hickson

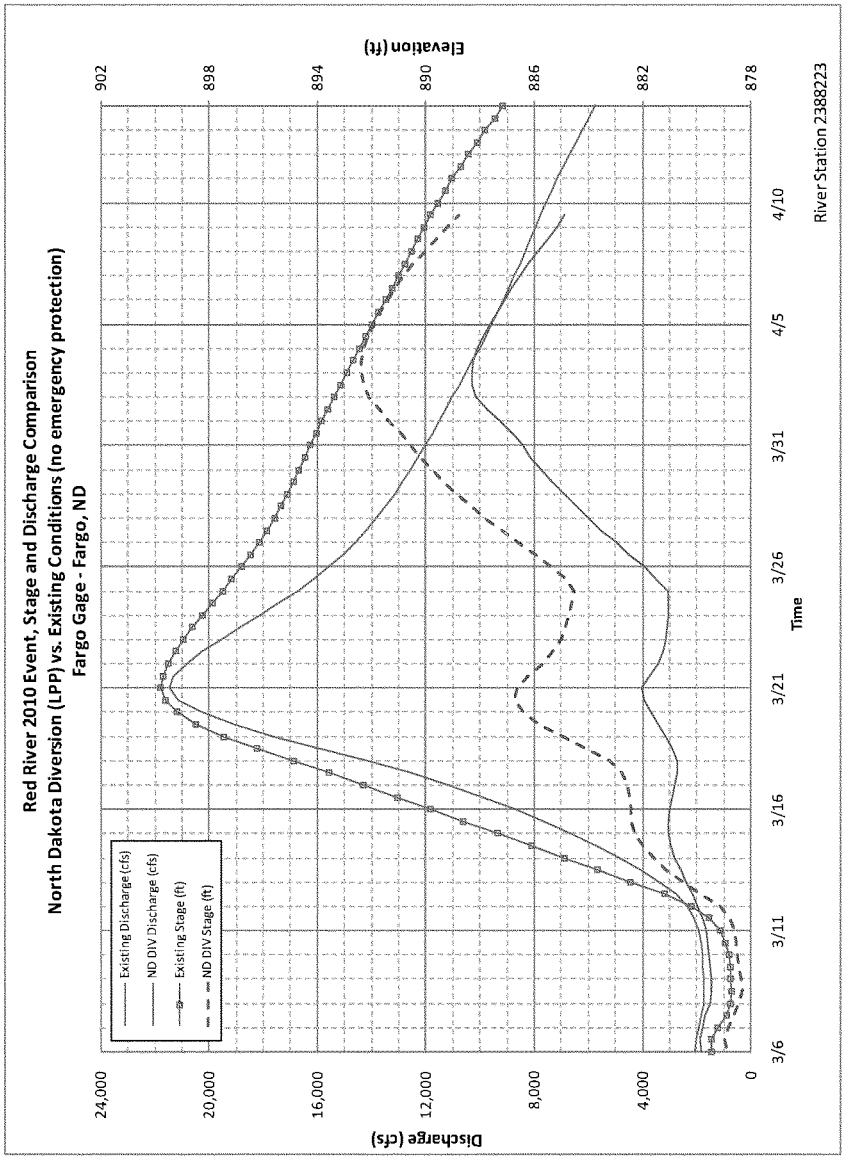


Figure C-E2- 88: 2010 Historical Flood Hydrographs for LPP @ Fargo

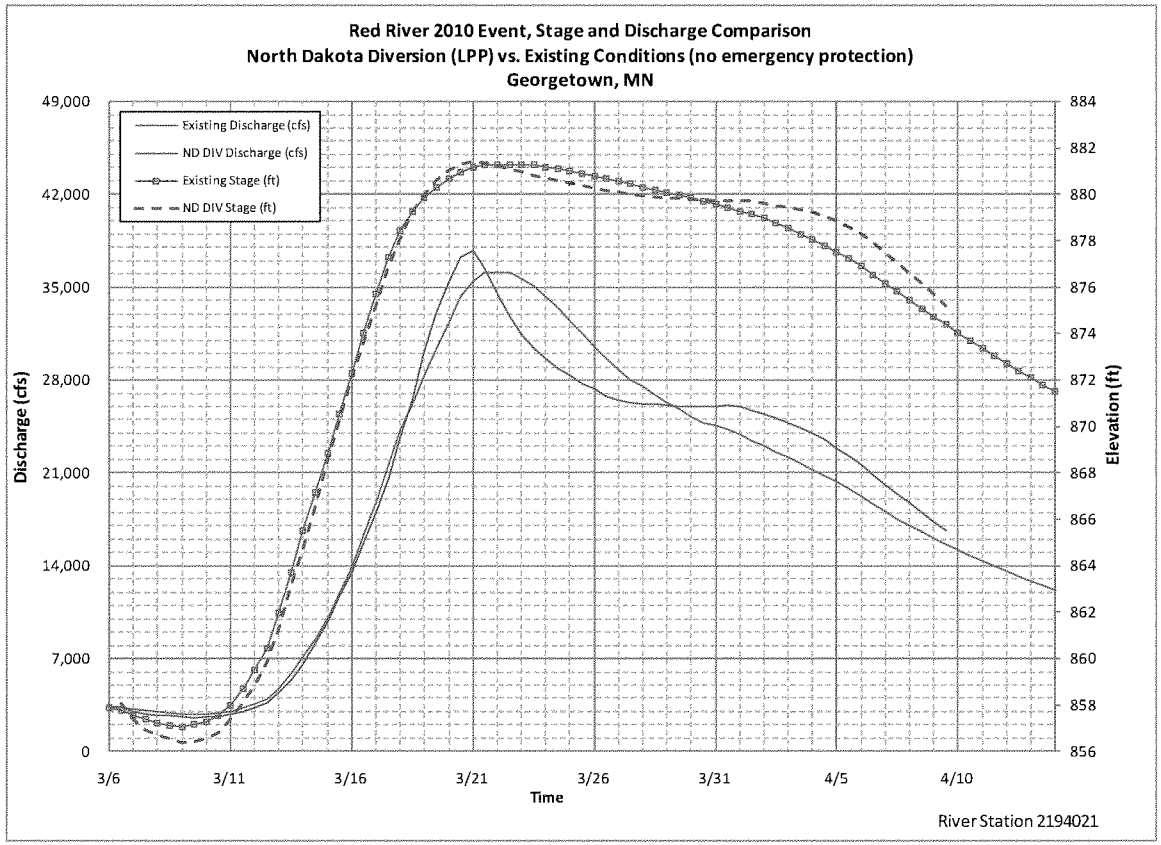


Figure C-E2- 89: 2010 Historical Flood Hydrographs for LPP @ Georgetown

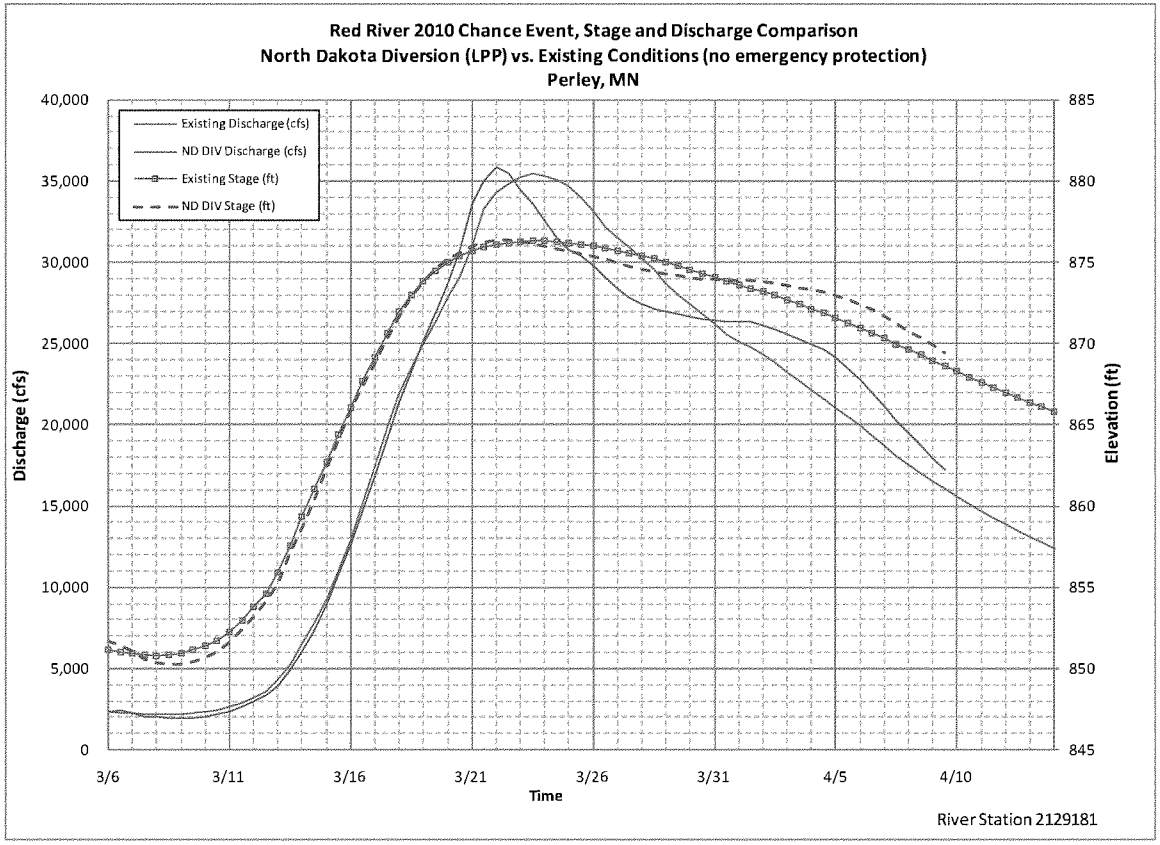


Figure C-E2- 90: 2010 Historical Flood Hydrographs for LPP @ Perley

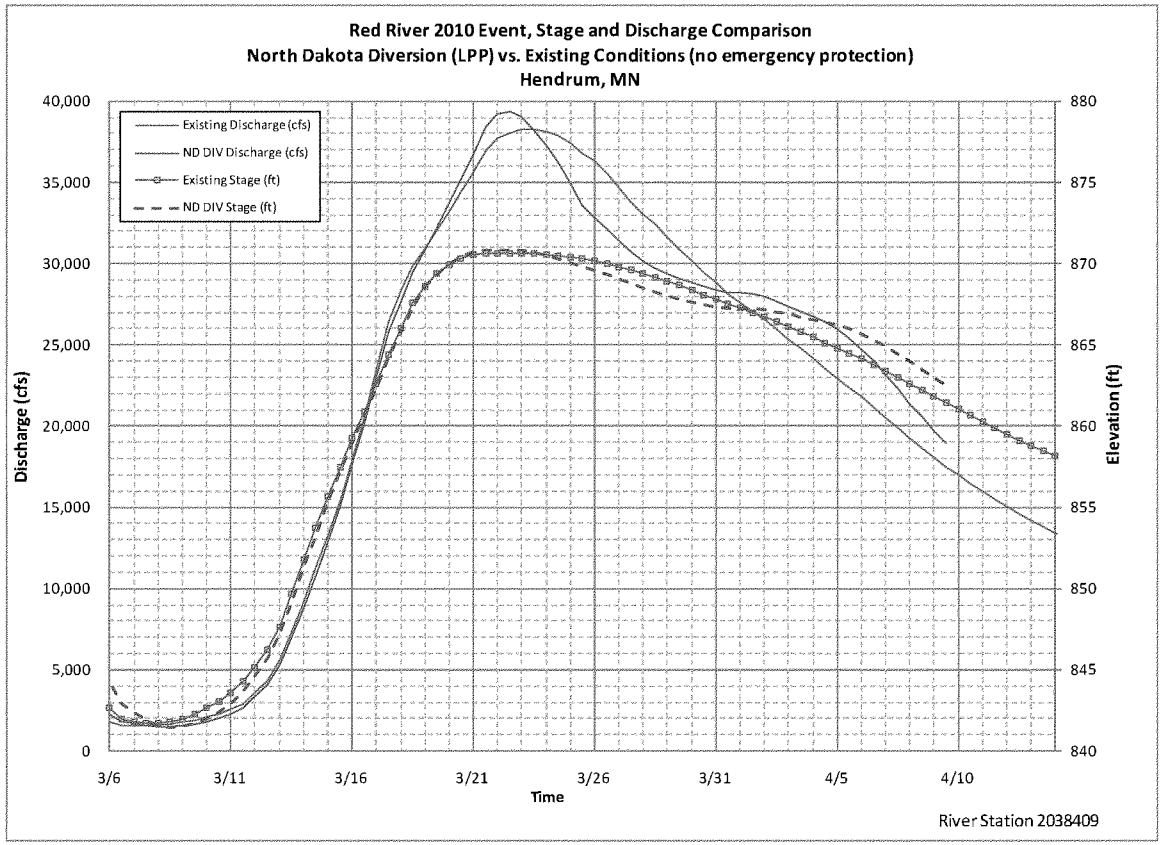


Figure C-E2- 91: 2010 Historical Flood Hydrographs for LPP @ Heudrum

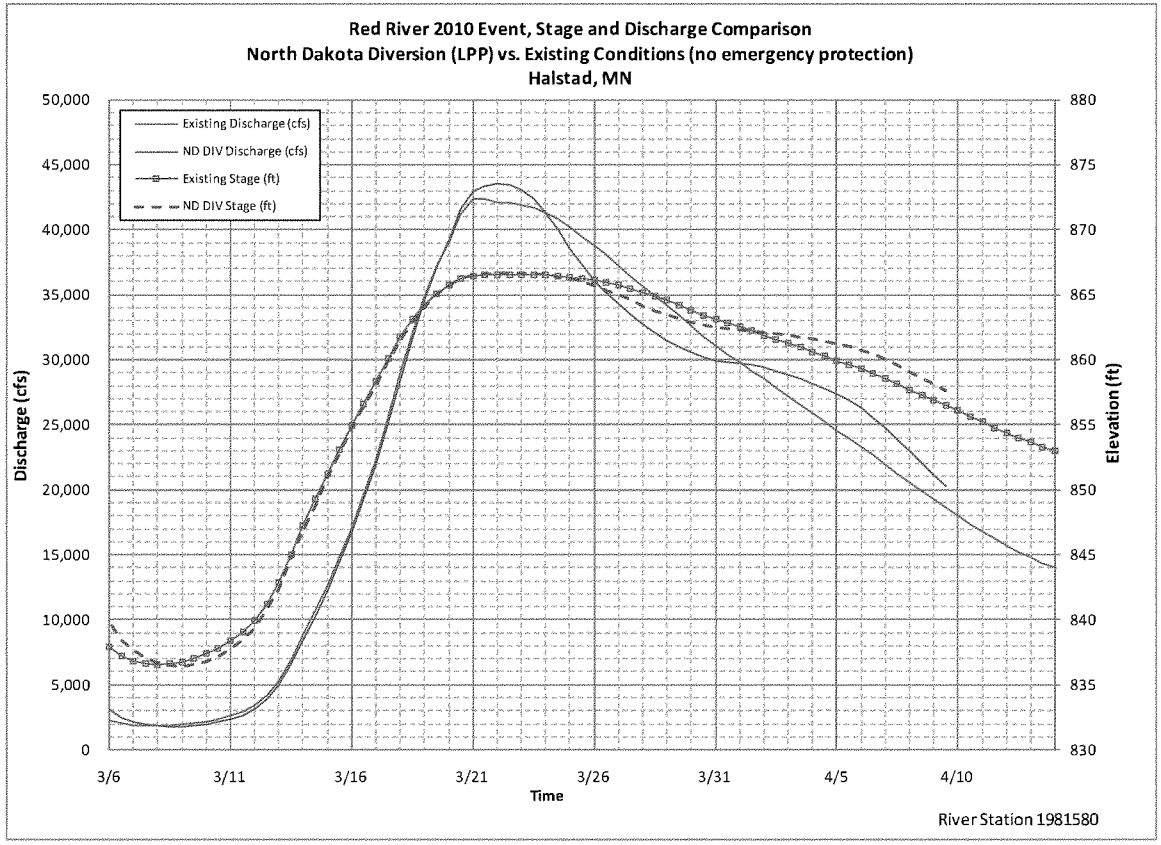


Figure C-E2- 92: 2010 Historical Flood Hydrographs for LPP @ Halstad

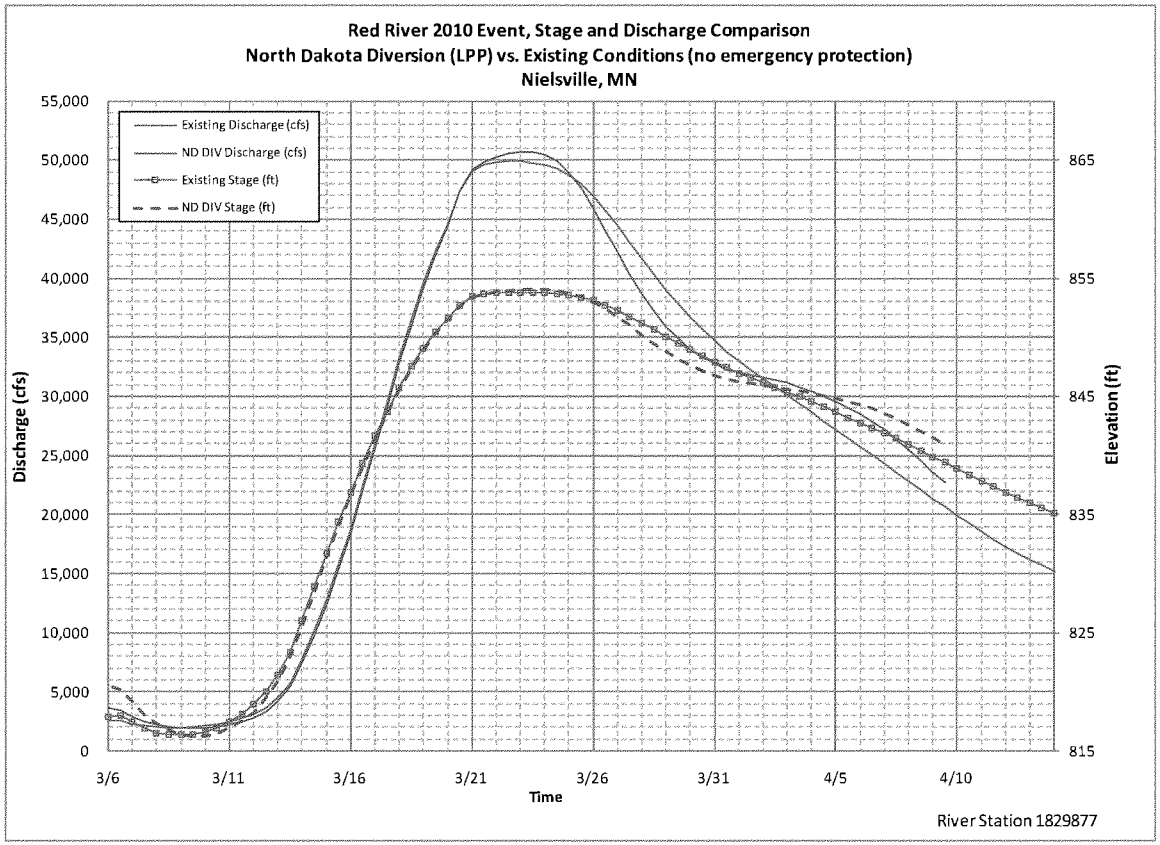


Figure C-E2- 93: 2010 Historical Flood Hydrographs for LPP @ Nielsville

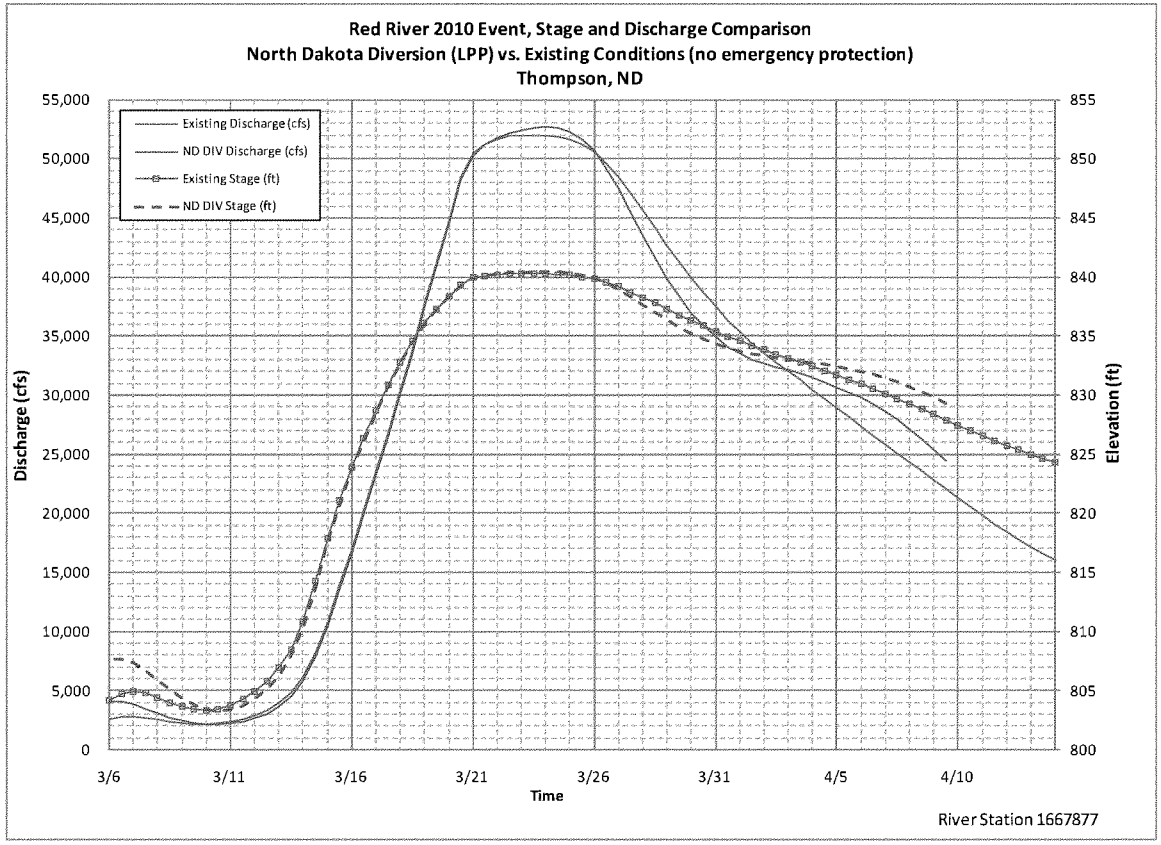


Figure C-E2- 94: 2010Historical Flood Hydrographs for LPP @ Thompson

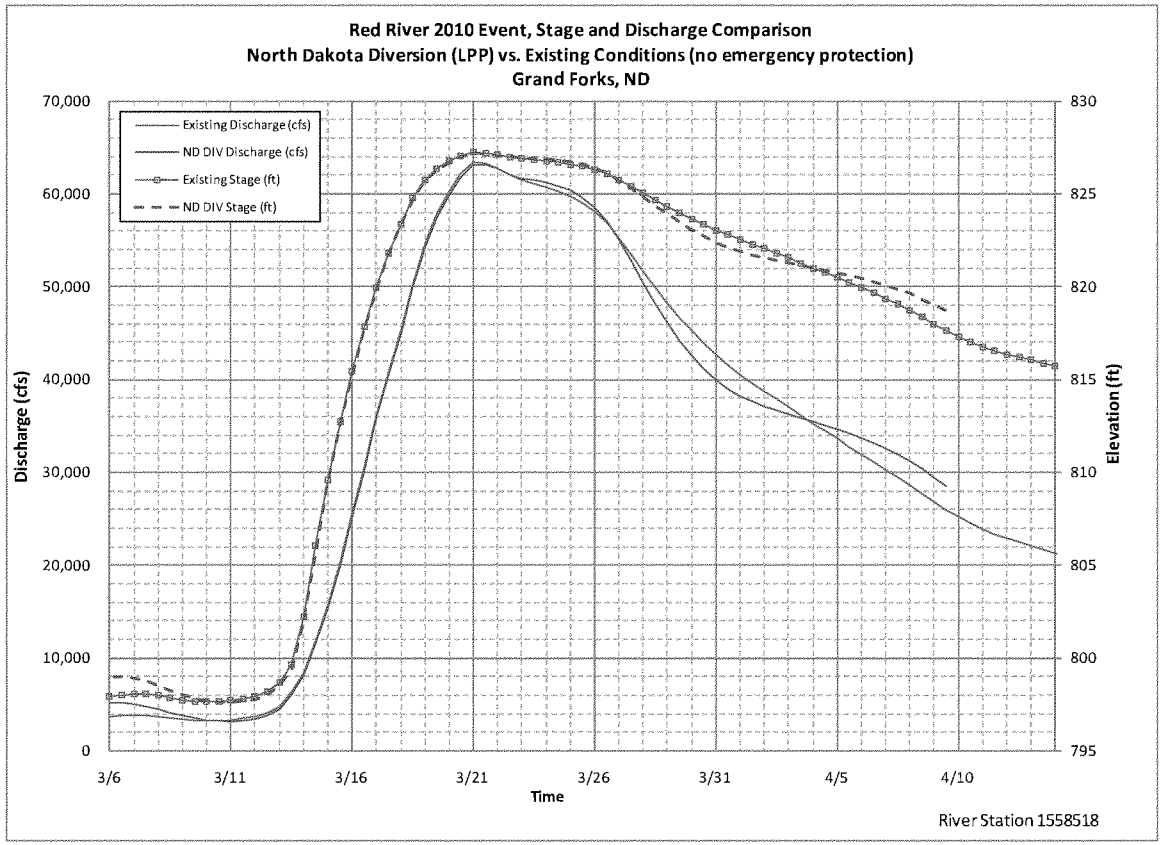


Figure C-E2- 95: 2010 Historical Flood Hydrographs for LPP @ Grand Forks

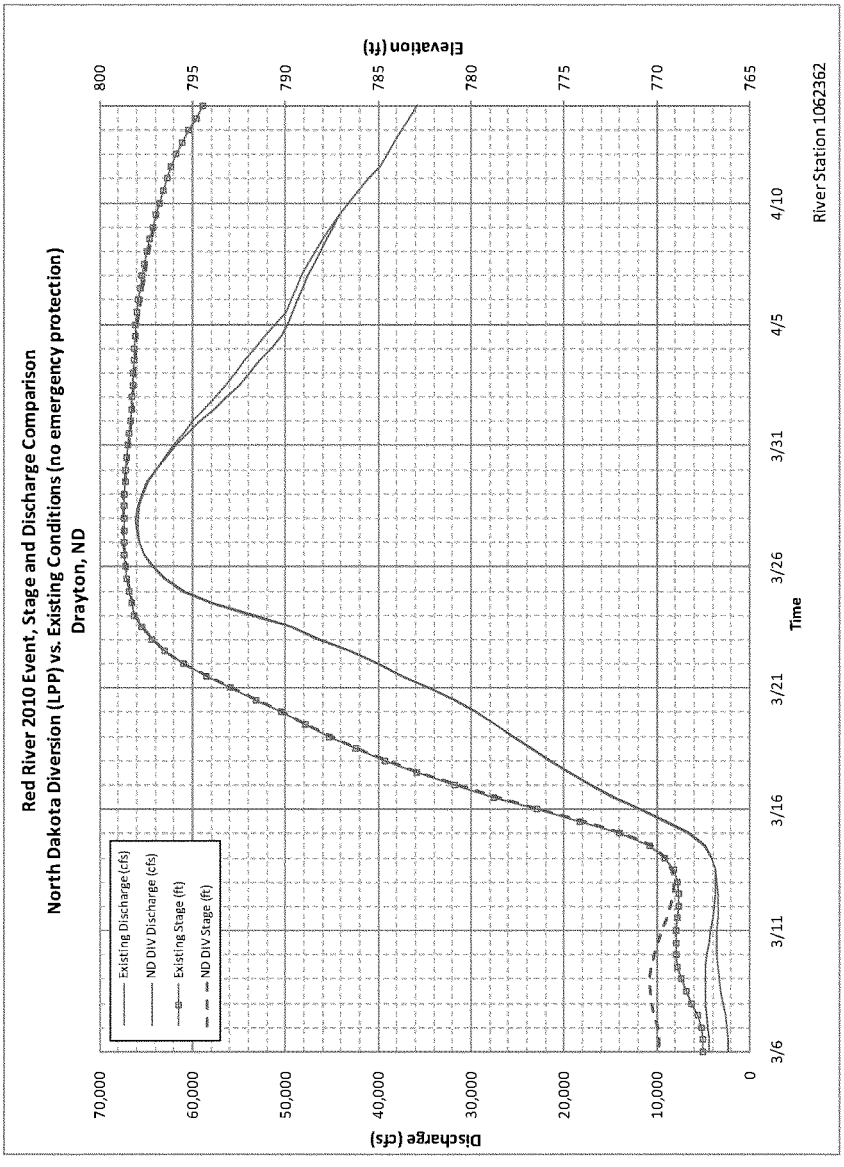


Figure C-E2- 96: 2010 Historical Flood Hydrographs for LPP @ Drayton

RED RIVER DIVERSION

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

APPENDIX C – HYDRAULICS WITH–PROJECT CONDITIONS

EXHIBIT 3 – PROJECT IMPACT MAPS

Report for the US Army Corps of Engineers, and the cities of Fargo, North Dakota & Moorhead, Minnesota

FINAL: February 28, 2011

FIGURES

Figure C-E3- 1 : Change in Maximum Water Surface Elevations Layout
Figure C-E3- 2 : 10-Percent Chance Change in Maximum Water Surface Elevation (Map 1 of 3)
Figure C-E3- 3 : 10-Percent Chance Change in Maximum Water Surface Elevation (Map 2 of 3)
Figure C-E3- 4 : 10-Percent Chance Change in Maximum Water Surface Elevation (Map 3 of 3)
Figure C-E3- 5 : 2-Percent Chance Change in Maximum Water Surface Elevation (Map 1 of 3)
Figure C-E3- 6 : 2-Percent Chance Change in Maximum Water Surface Elevation (Map 2 of 3)
Figure C-E3- 7 : 2-Percent Chance Change in Maximum Water Surface Elevation (Map 3 of 3)
Figure C-E3- 8 : 1-Percent Chance Change in Maximum Water Surface Elevation (Map 1 of 3)
Figure C-E3- 9 : 1-Percent Chance Change in Maximum Water Surface Elevation (Map 2 of 3)
Figure C-E3- 10 : 1-Percent Chance Change in Maximum Water Surface Elevation (Map 3 of 3)
Figure C-E3- 11 : 0.2-Percent Chance Change in Maximum Water Surface Elevation (Map 1 of 3)
Figure C-E3- 12 : 0.2-Percent Chance Change in Maximum Water Surface Elevation (Map 2 of 3)
Figure C-E3- 13 : 0.2-Percent Chance Change in Maximum Water Surface Elevation (Map 3 of 3)

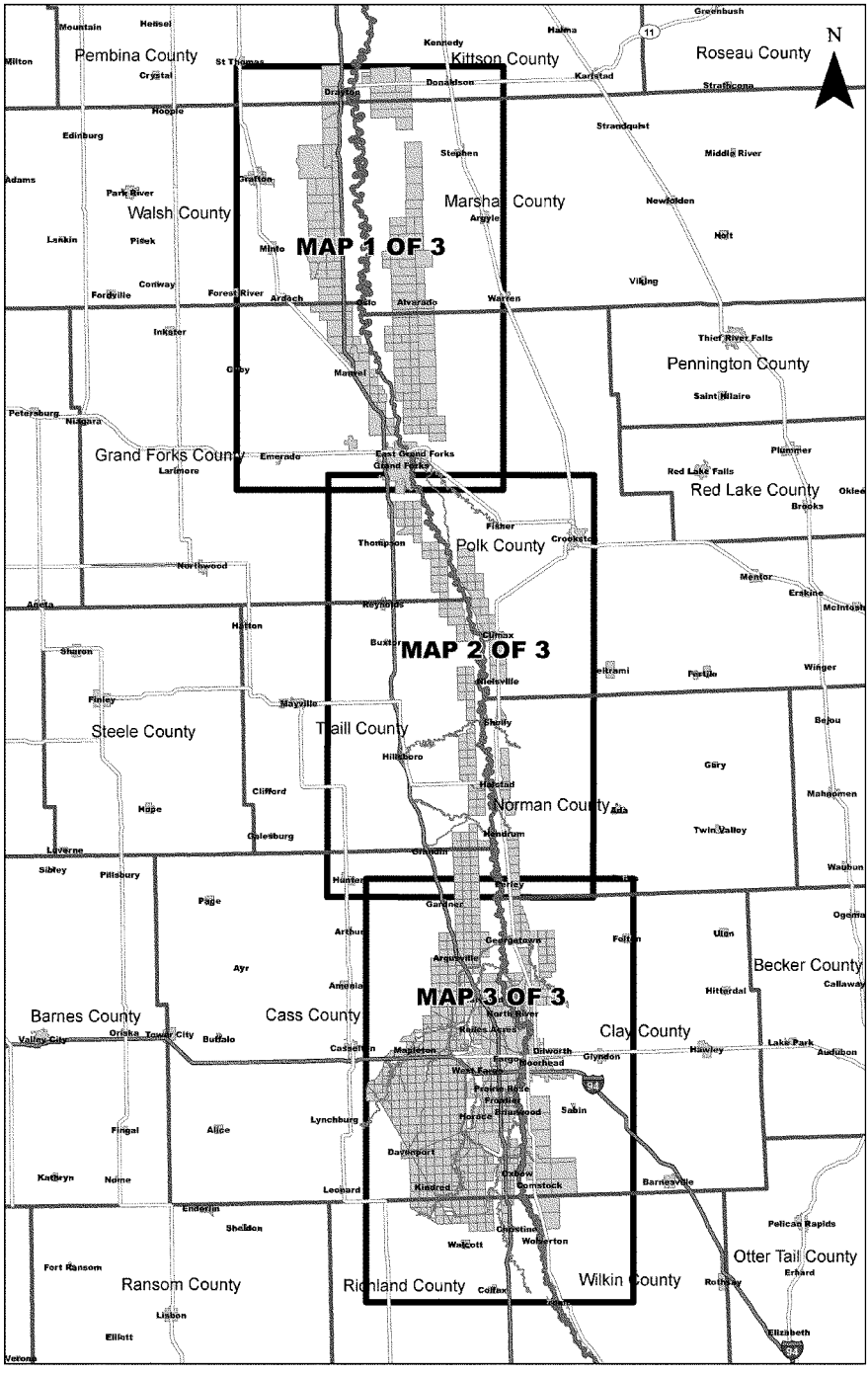


Figure C-E3-1: Change in Maximum Water Surface Elevations Layout

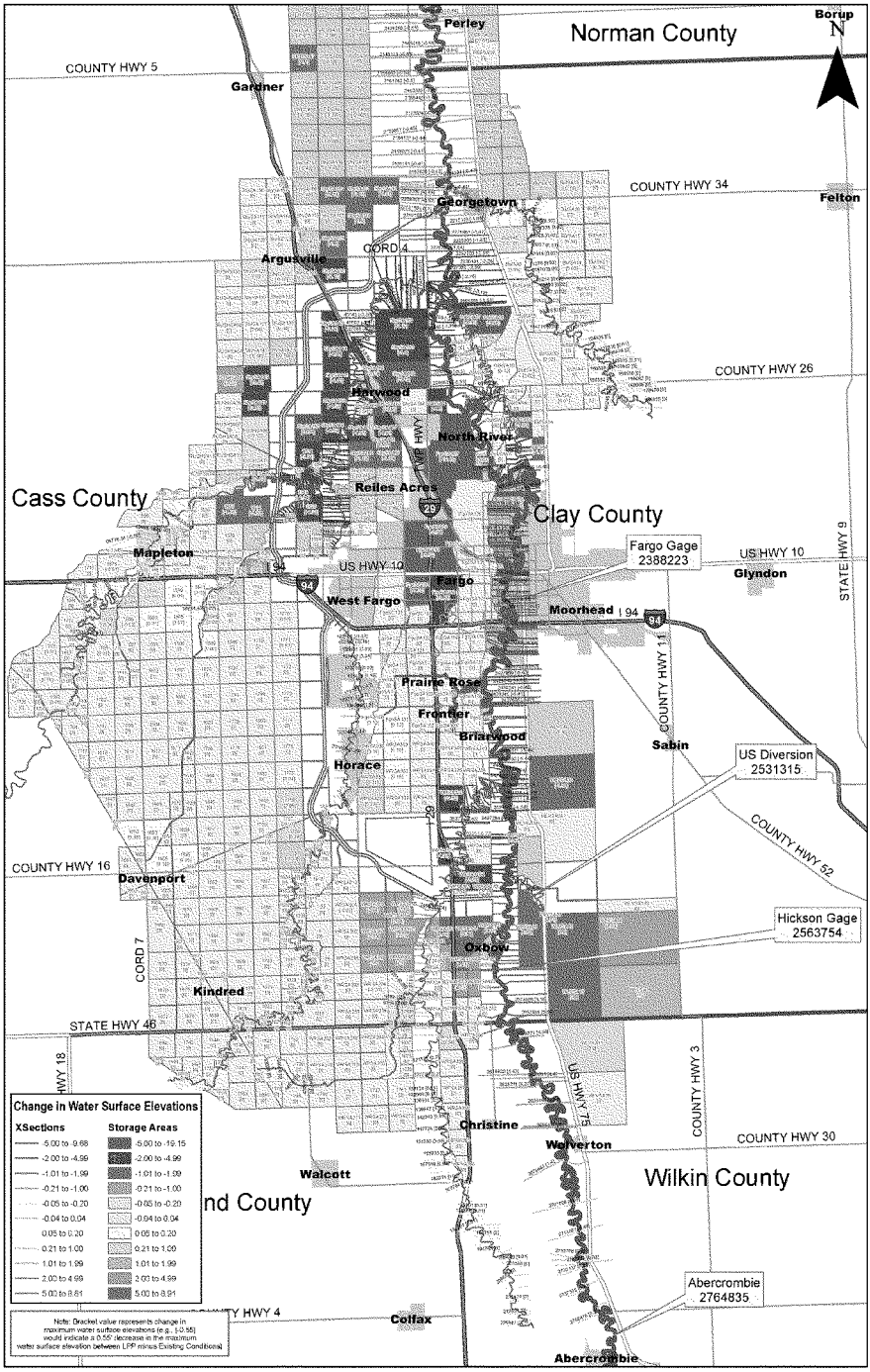


Figure C-E3-2: 10-Percent Chance Change in Maximum Water Surface Elevation (Map 1 of 3)

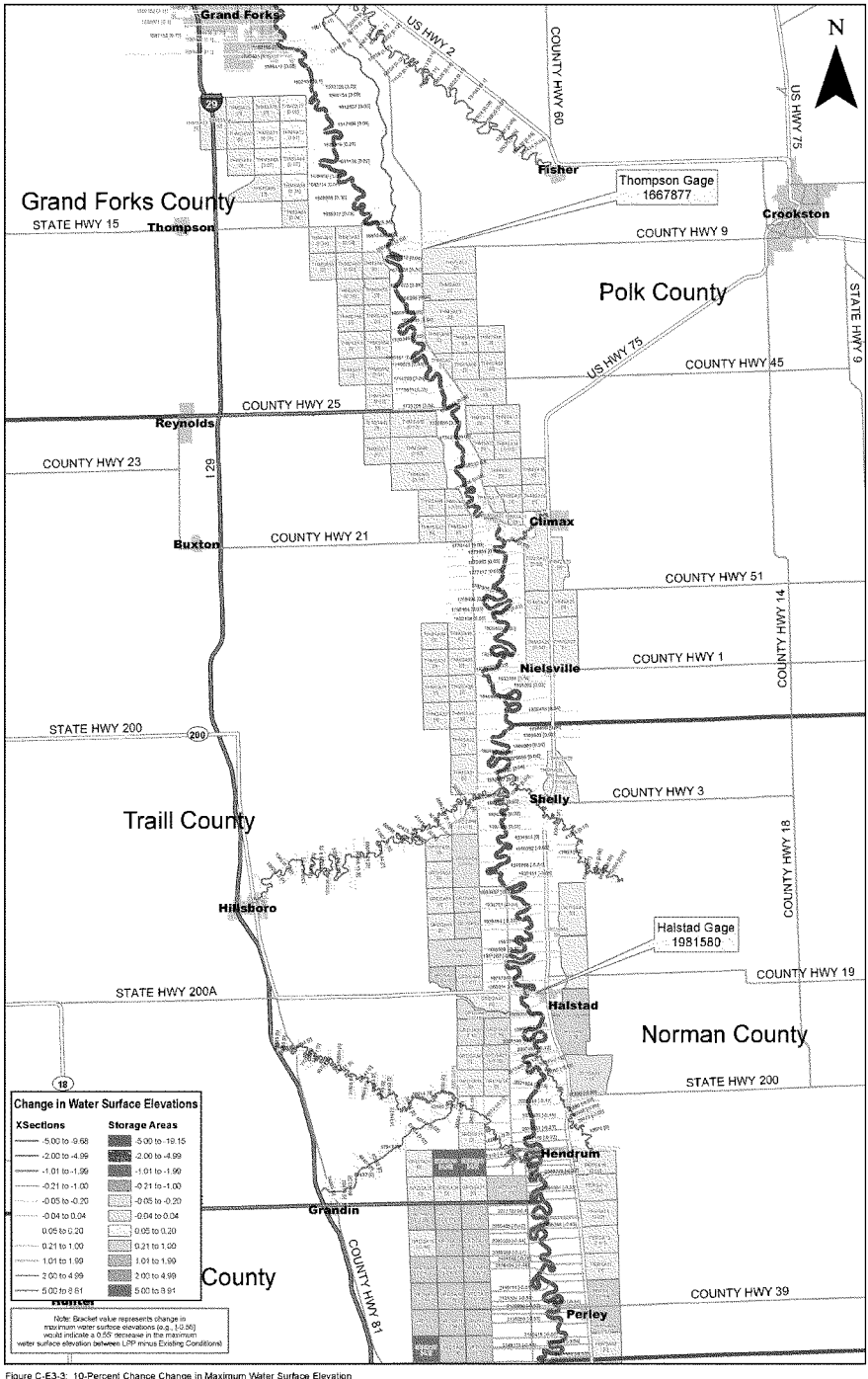


Figure C-E3-3: 10-Percent Chance Change in Maximum Water Surface Elevation
(Map 2 of 3)

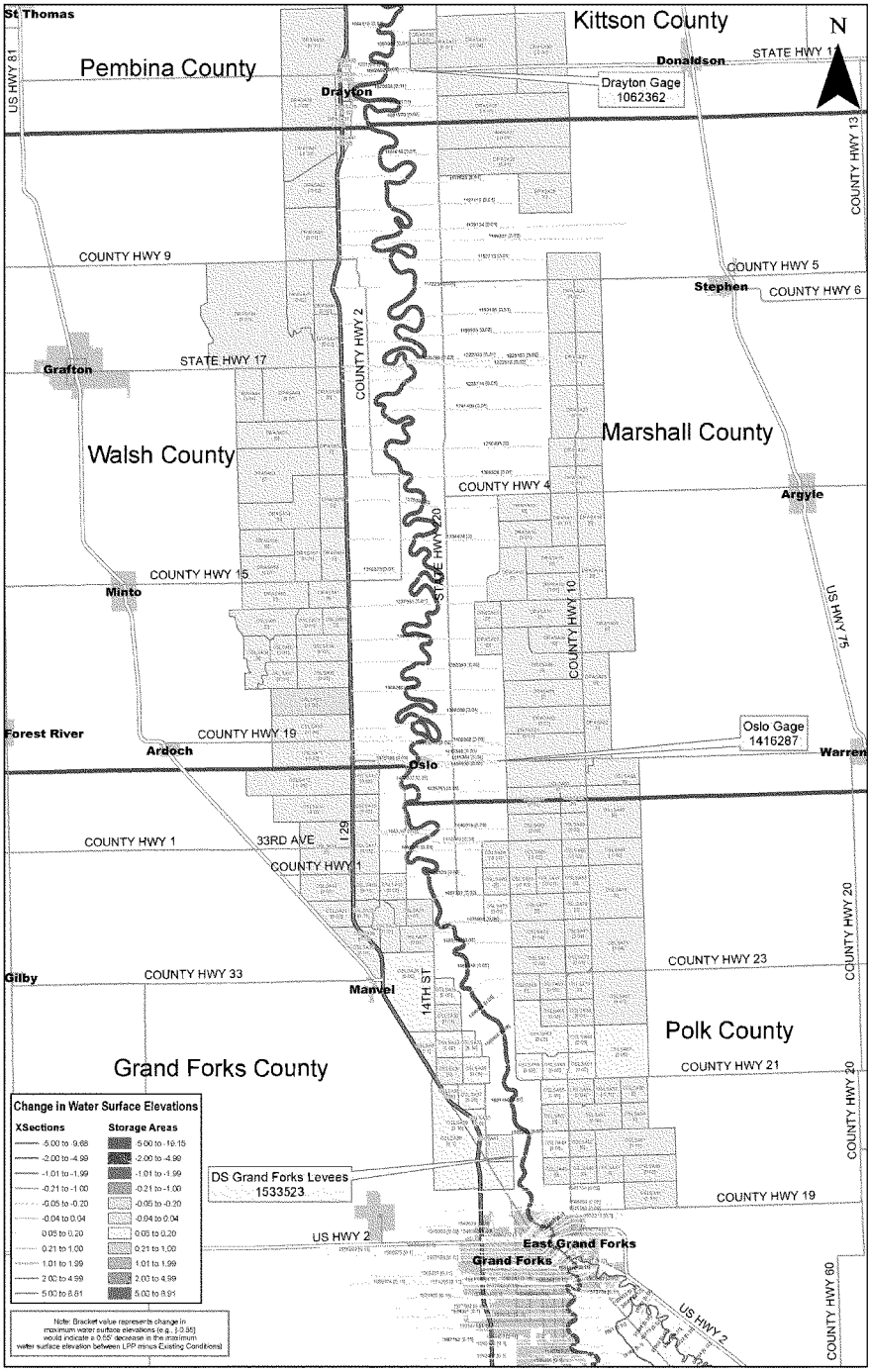


Figure C-E3-4: 10-Percent Chance Change in Maximum Water Surface Elevation (Map 3 of 3)

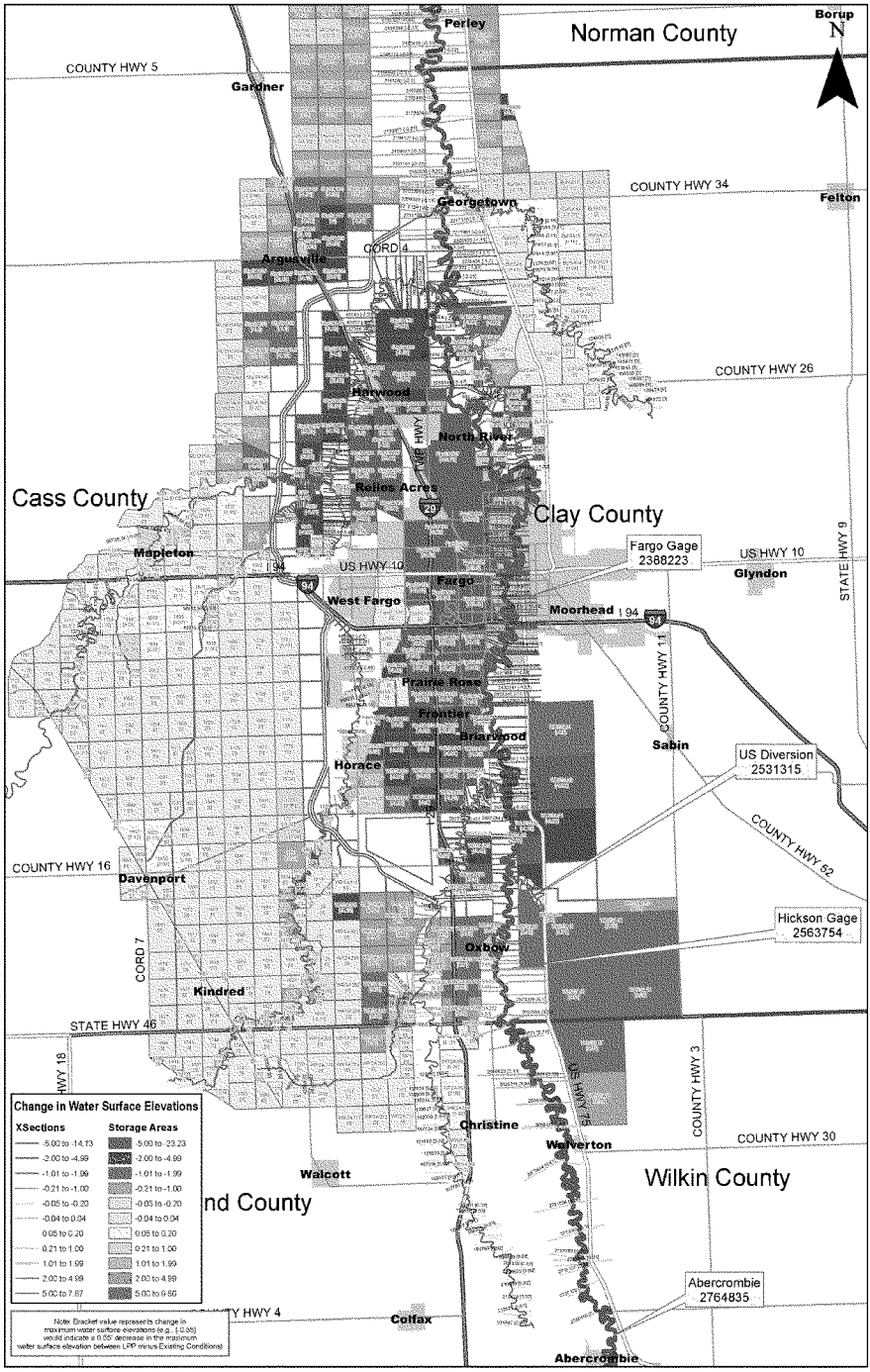


Figure C-E3-5. 2-Percent Chance Change in Maximum Water Surface Elevation (Map 1 of 3)

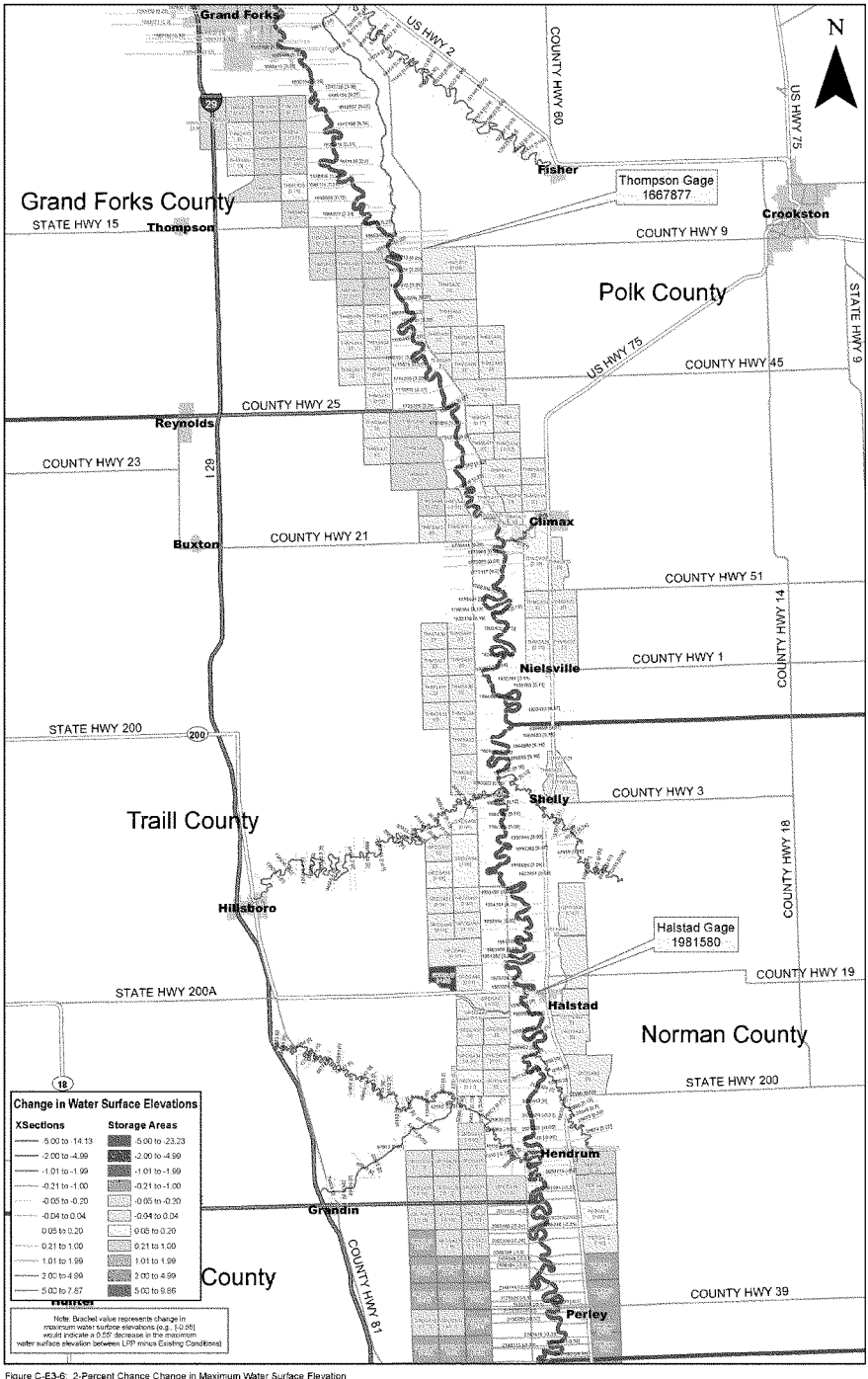


Figure C-E3-6: 2-Percent Chance Change in Maximum Water Surface Elevation
(Map 2 of 3)

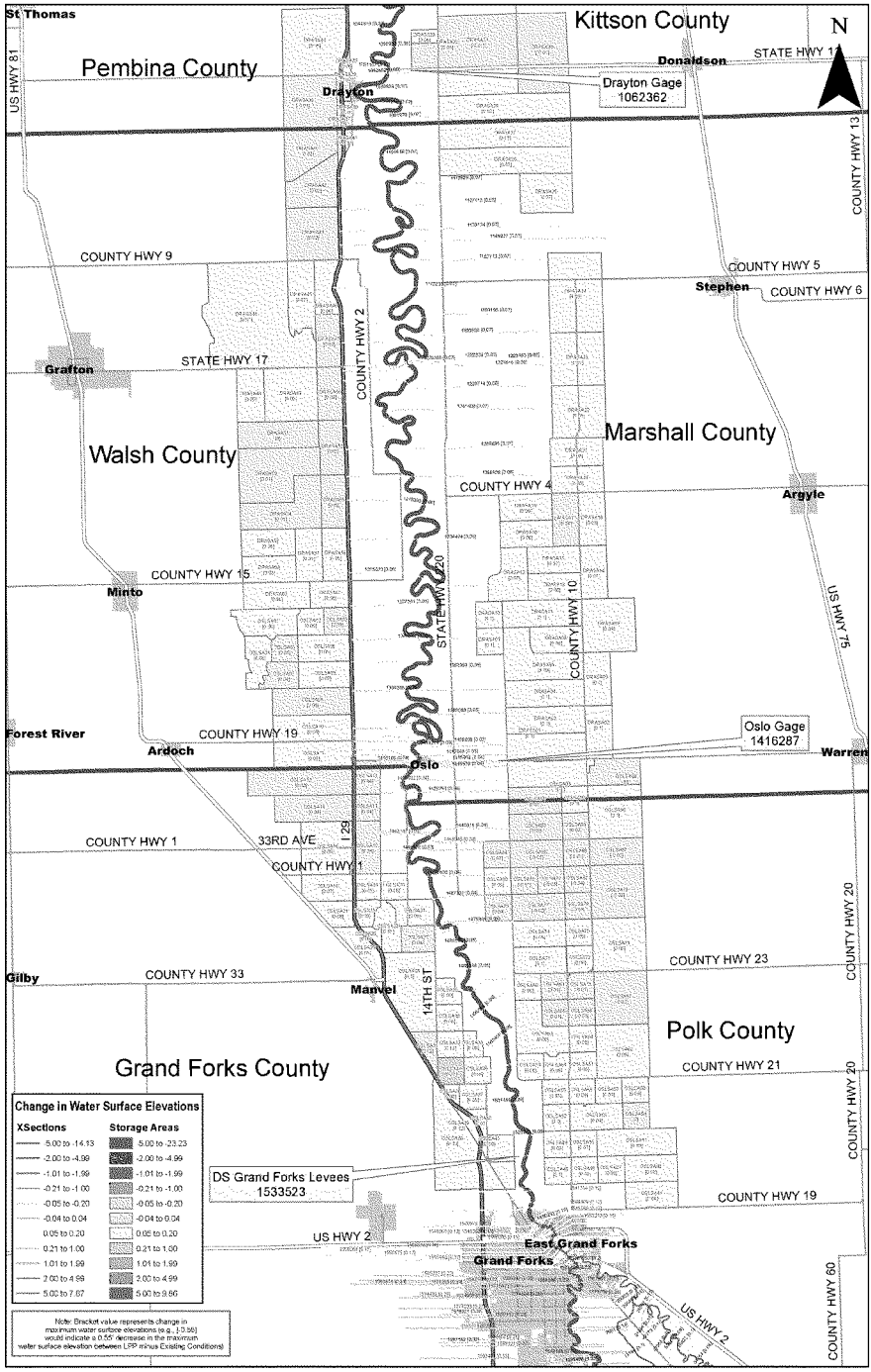


Figure C-E3-7: 2-Percent Chance Change in Maximum Water Surface Elevation (Map 3 of 3)

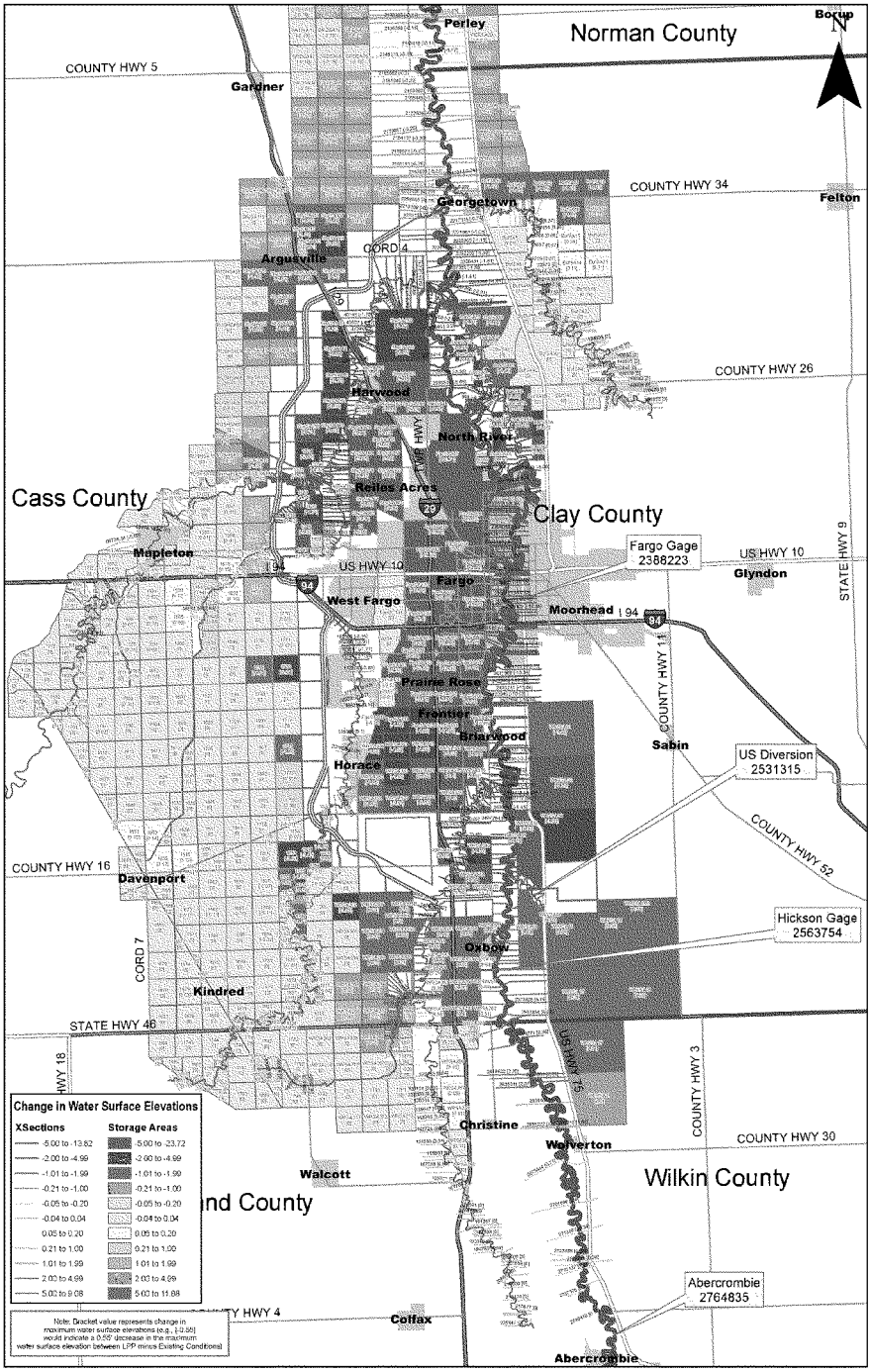


Figure C-E3-8: 1-Percent Chance Change in Maximum Water Surface Elevation (Map 1 of 3)

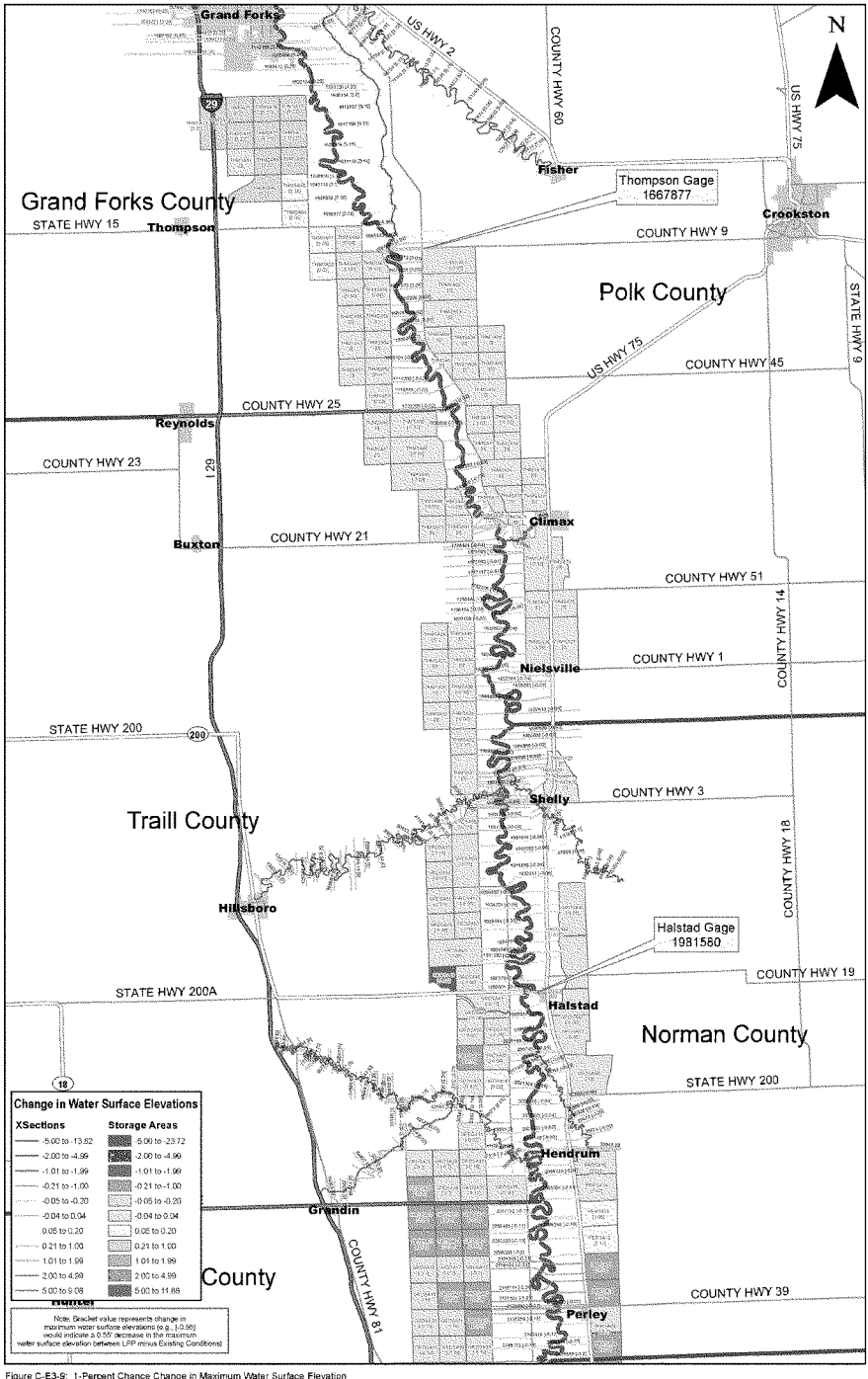


Figure C-E3-9: 1-Percent Chance Change in Maximum Water Surface Elevation (Map 2 of 3)

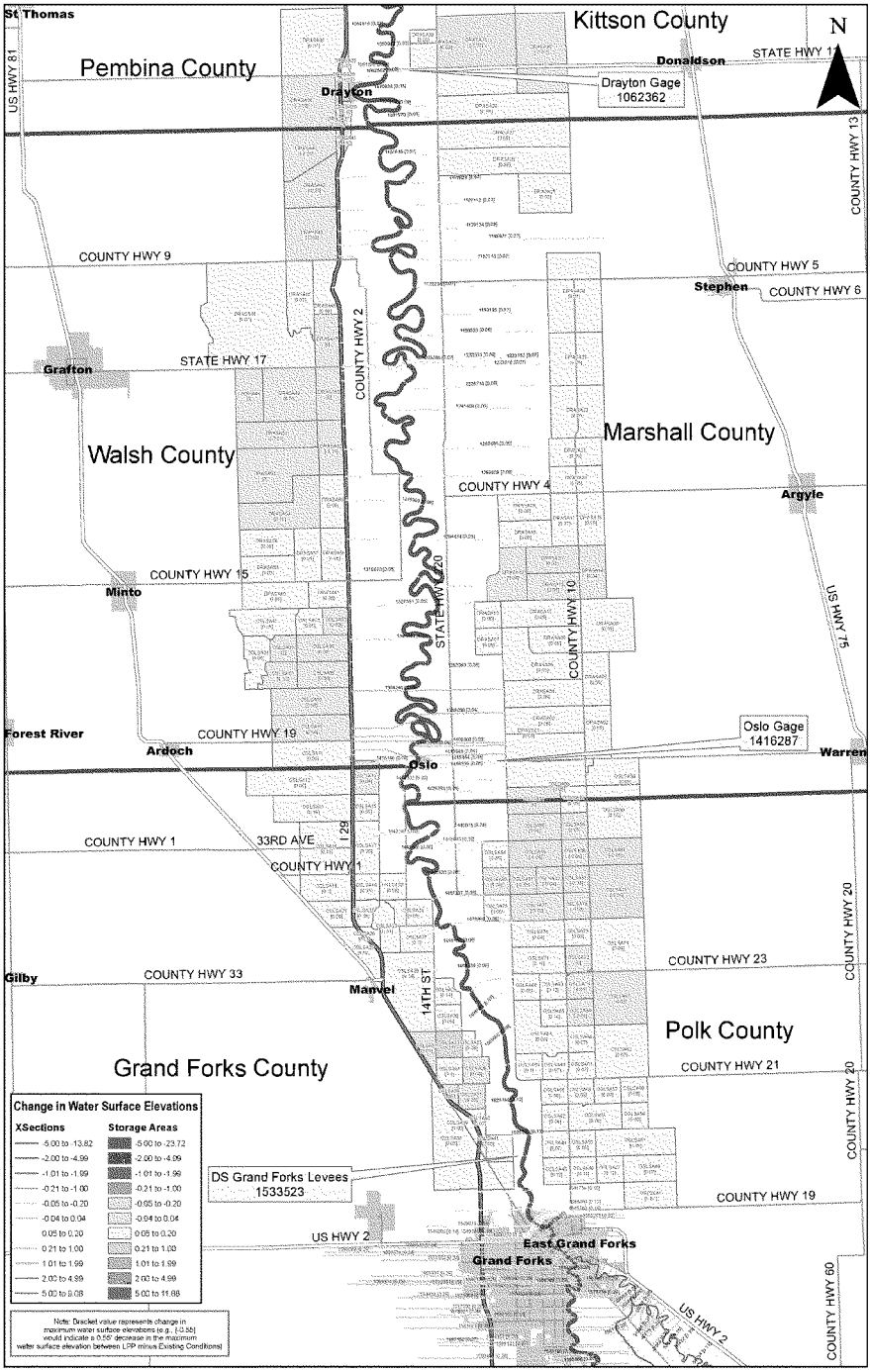


Figure C-E3-10: 1-Percent Chance Change in Maximum Water Surface Elevation (Map 3 of 3)

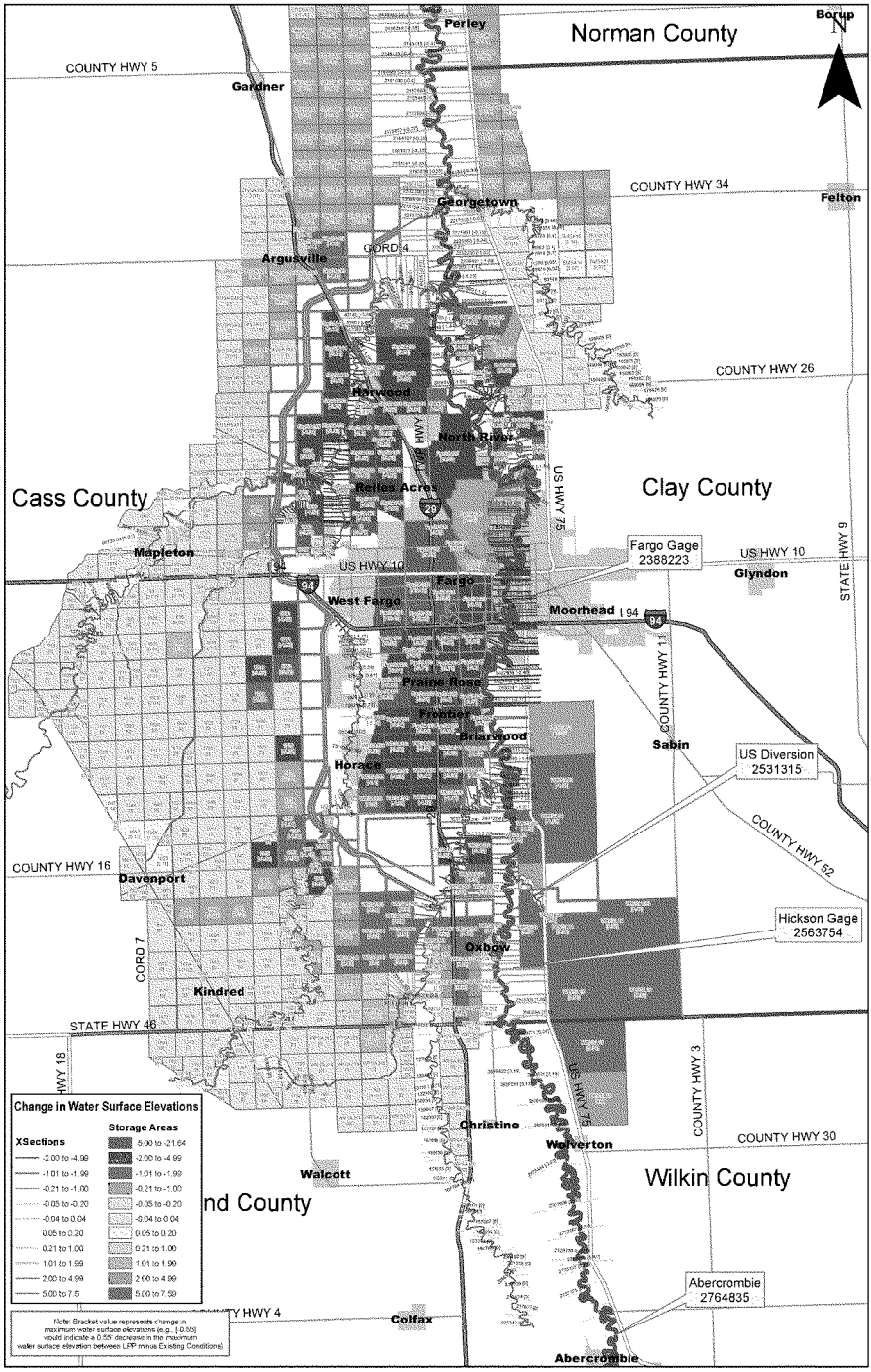


Figure C-E3-11: 0.2-Percent Chance Change in Maximum Water Surface Elevation
(Map 1 of 3)

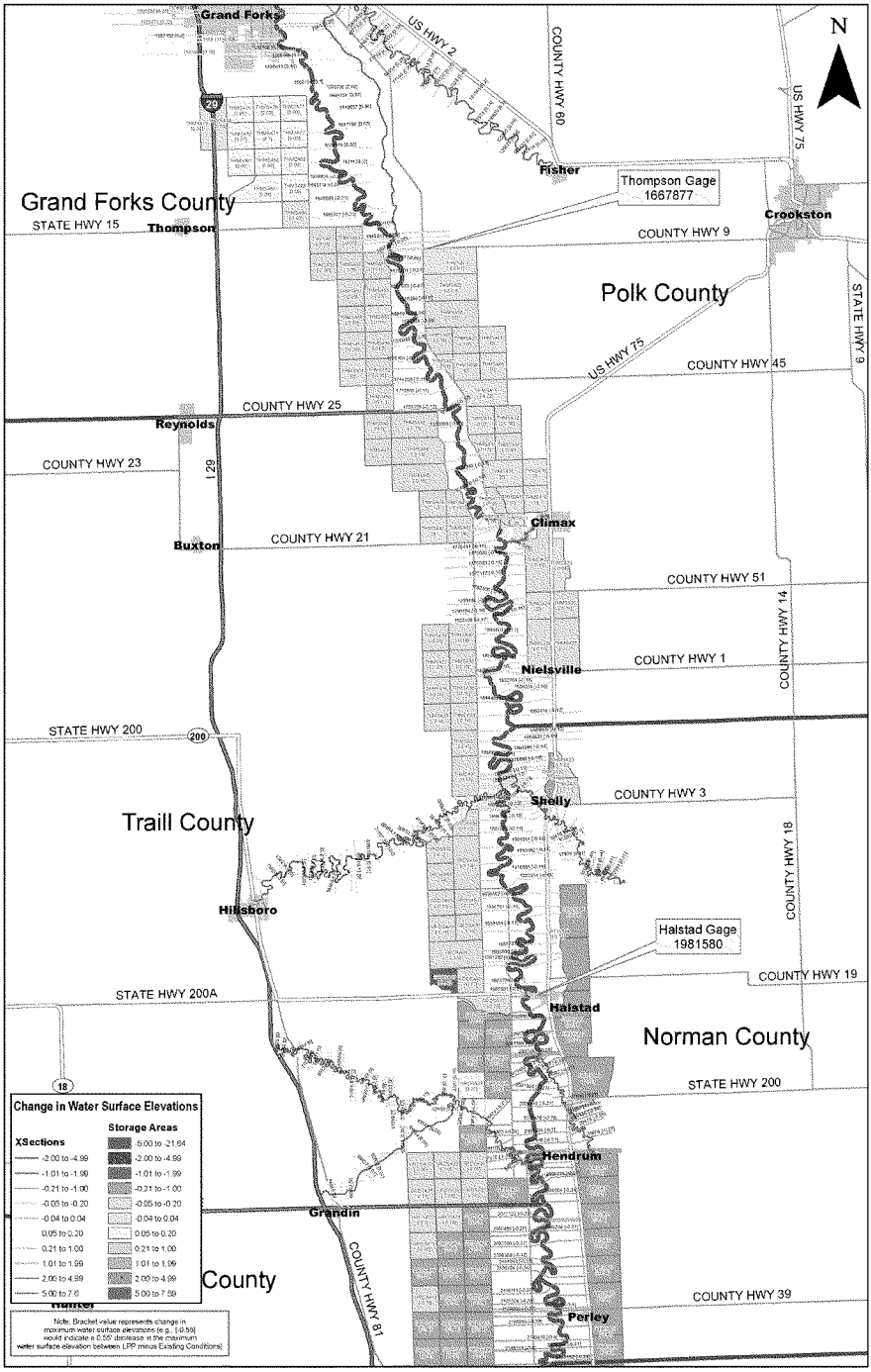


Figure C-E3-12: 0.2-Percent Chance Change in Maximum Water Surface Elevation (Map 2 of 3)

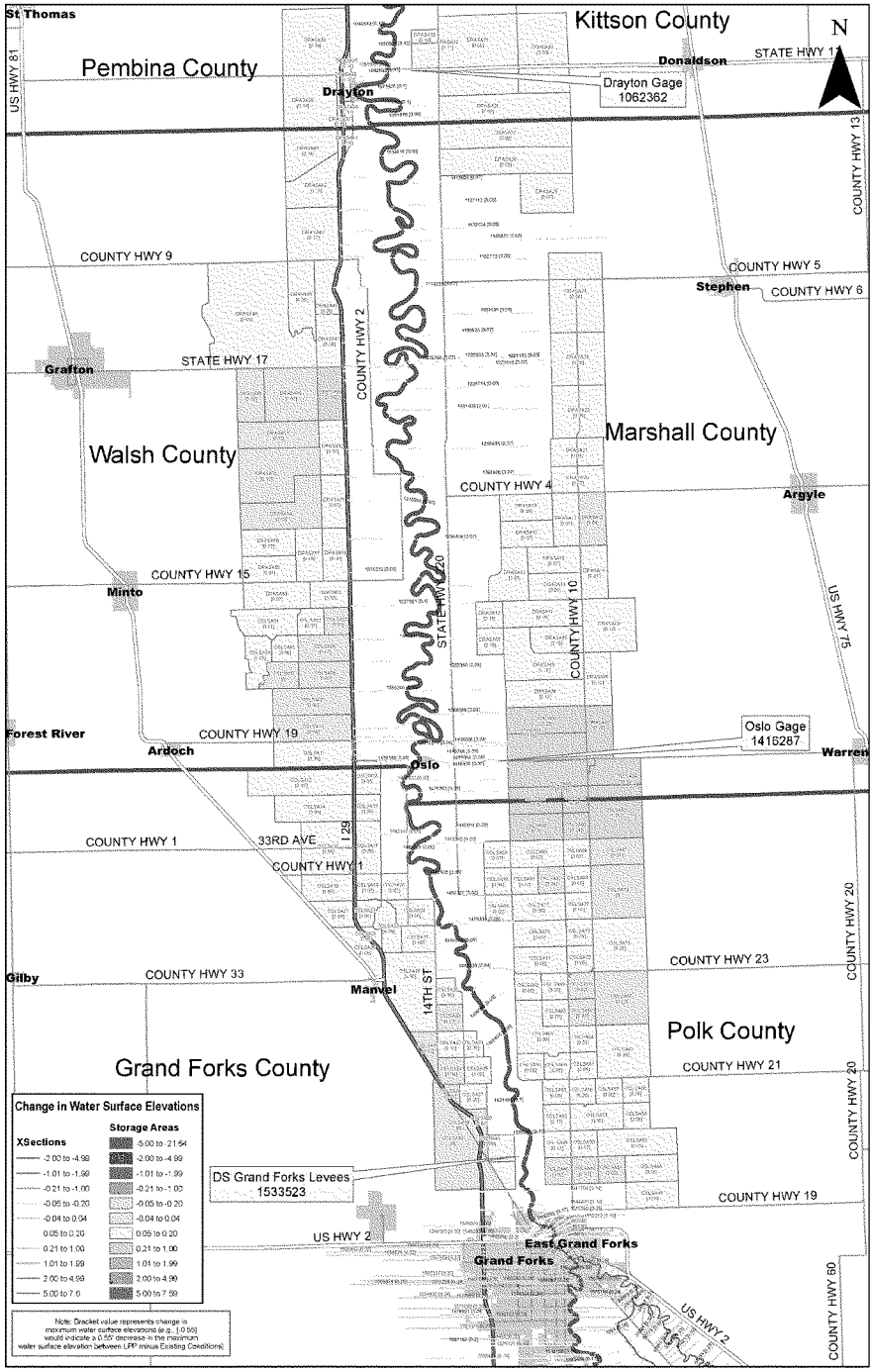


Figure C-E3-13. 0.2-Percent Chance Change in Maximum Water Surface Elevation
(Map 3 of 3)

RED RIVER DIVERSION

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

APPENDIX C – HYDRAULICS WITH–PROJECT CONDITIONS

EXHIBIT 4 – INUNDATION MAPS

Report for the US Army Corps of Engineers, and the cities of Fargo, North Dakota & Moorhead, Minnesota

FINAL: February 28, 2011

FIGURES

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Figure C-E4- 2 : FCP 10-Percent Chance Flood - Grand Forks to Drayton
Figure C-E4- 3 : FCP 10-Percent Chance Flood - Halstad to Grand Forks
Figure C-E4- 4 : FCP 10-Percent Chance Flood - Georgetown to Climax
Figure C-E4- 5 : FCP 2-Percent Chance Flood - Overall Map
Figure C-E4- 6 : FCP 2-Percent Chance Flood - Grand Forks to Drayton
Figure C-E4- 7 : FCP 2-Percent Chance Flood - Halstad to Grand Forks
Figure C-E4- 8 : FCP 2-Percent Chance Flood - Georgetown to Climax
Figure C-E4- 9 : FCP 1-Percent Chance Flood - Overall Map
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Figure C-E4- 19 : FCP 1997 Historic Flood - Halstad to Grand Forks
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Figure C-E4- 25 : FCP 2009 Historic Flood - Overall Map
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Figure C-E4- 33 : LPP 10-Percent Chance Flood - Overall Map
Figure C-E4- 34 : LPP 10-Percent Chance Flood - Grand Forks to Drayton
Figure C-E4- 35 : LPP 10-Percent Chance Flood - Halstad to Grand Forks
Figure C-E4- 36 : LPP 10-Percent Chance Flood - Georgetown to Climax
Figure C-E4- 37 : LPP 2-Percent Chance Flood - Overall Map
Figure C-E4- 38 : LPP 2-Percent Chance Flood - Grand Forks to Drayton
Figure C-E4- 39 : LPP 2-Percent Chance Flood - Halstad to Grand Forks

Figure C-E4- 40 : LPP 2-Percent Chance Flood - Georgetown to Climax
Figure C-E4- 41 : LPP 1-Percent Chance Flood - Overall Map
Figure C-E4- 42 : LPP 1-Percent Chance Flood - Grand Forks to Drayton
Figure C-E4- 43 : LPP 1-Percent Chance Flood - Halstad to Grand Forks
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Figure C-E4- 45 : LPP 0.2-Percent Chance Flood - Overall Map
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Figure C-E4- 47 : LPP 0.2-Percent Chance Flood - Halstad to Grand Forks
Figure C-E4- 48 : LPP 0.2-Percent Chance Flood - Georgetown to Climax
Figure C-E4- 49 : LPP 1997 Historic Flood - Overall Map
Figure C-E4- 50 : LPP 1997 Historic Flood - Grand Forks to Drayton
Figure C-E4- 51 : LPP 1997 Historic Flood - Halstad to Grand Forks
Figure C-E4- 52 : LPP 1997 Historic Flood - Georgetown to Climax
Figure C-E4- 53 : LPP 2006 Historic Flood - Overall Map
Figure C-E4- 54 : LPP 2006 Historic Flood - Grand Forks to Drayton
Figure C-E4- 55 : LPP 2006 Historic Flood - Halstad to Grand Forks
Figure C-E4- 56 : LPP 2006 Historic Flood - Georgetown to Climax
Figure C-E4- 57 : LPP 2009 Historic Flood - Overall Map
Figure C-E4- 58 : LPP 2009 Historic Flood - Grand Forks to Drayton
Figure C-E4- 59 : LPP 2009 Historic Flood - Halstad to Grand Forks
Figure C-E4- 60 : LPP 2009 Historic Flood - Georgetown to Climax
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Figure C-E4- 63 : LPP 2010 Historic Flood - Halstad to Grand Forks
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Figure C-E4- 65 : LPP 10-Percent Chance Flood - Staging Area
Figure C-E4- 66 : LPP 2-Percent Chance Flood - Staging Area
Figure C-E4- 67 : LPP 1-Percent Chance Flood - Staging Area
Figure C-E4- 68 : LPP 0.2-Percent Chance Flood - Staging Area
Figure C-E4- 69 : LPP 1997 Historic Flood - Staging Area
Figure C-E4- 70 : LPP 2006 Historic Flood - Staging Area
Figure C-E4- 71 : LPP 2009 Historic Flood - Staging Area
Figure C-E4- 72 : LPP 2010 Historic Flood - Staging Area

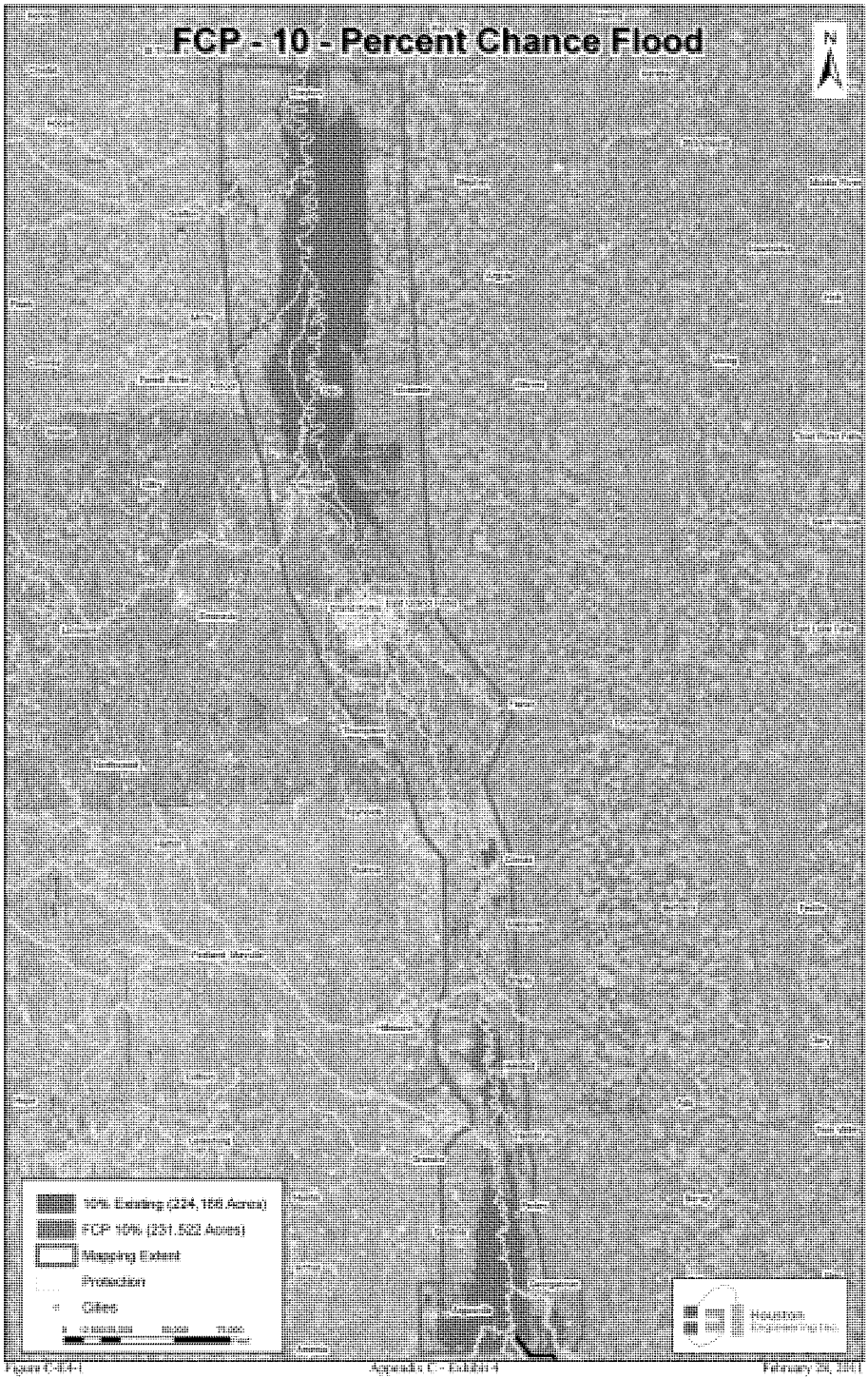


Figure C-3.4-1 Appendix C - Exhibit 4 February 28, 2001

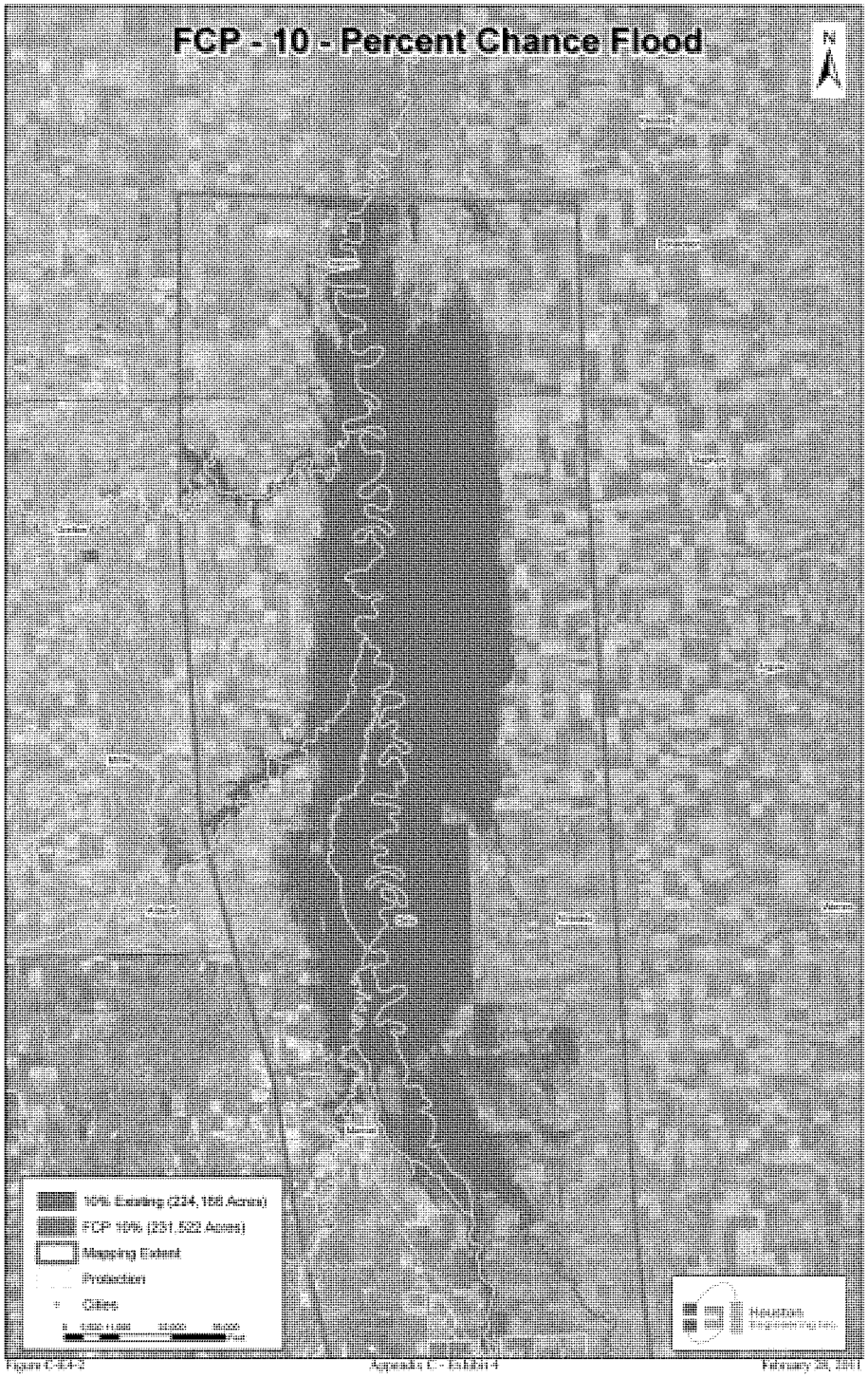


Figure C-3.4-2 Appendix C - Exhibit 4 February 28, 2011

Figure C-B4-3 Appendix C - Exhibit 4 February 28, 2001

Figure 4-4-4 Appendix E – Exhibit 4 February 28, 2011

Figure C-44-3 Appendix C - Exhibit 4 February 28, 2001

Figure 4-4-3 Appendix L - Exhibit 4 February 28, 2011

Figure 4.4-7 Appendix E - Exhibit 4 February 28, 2001

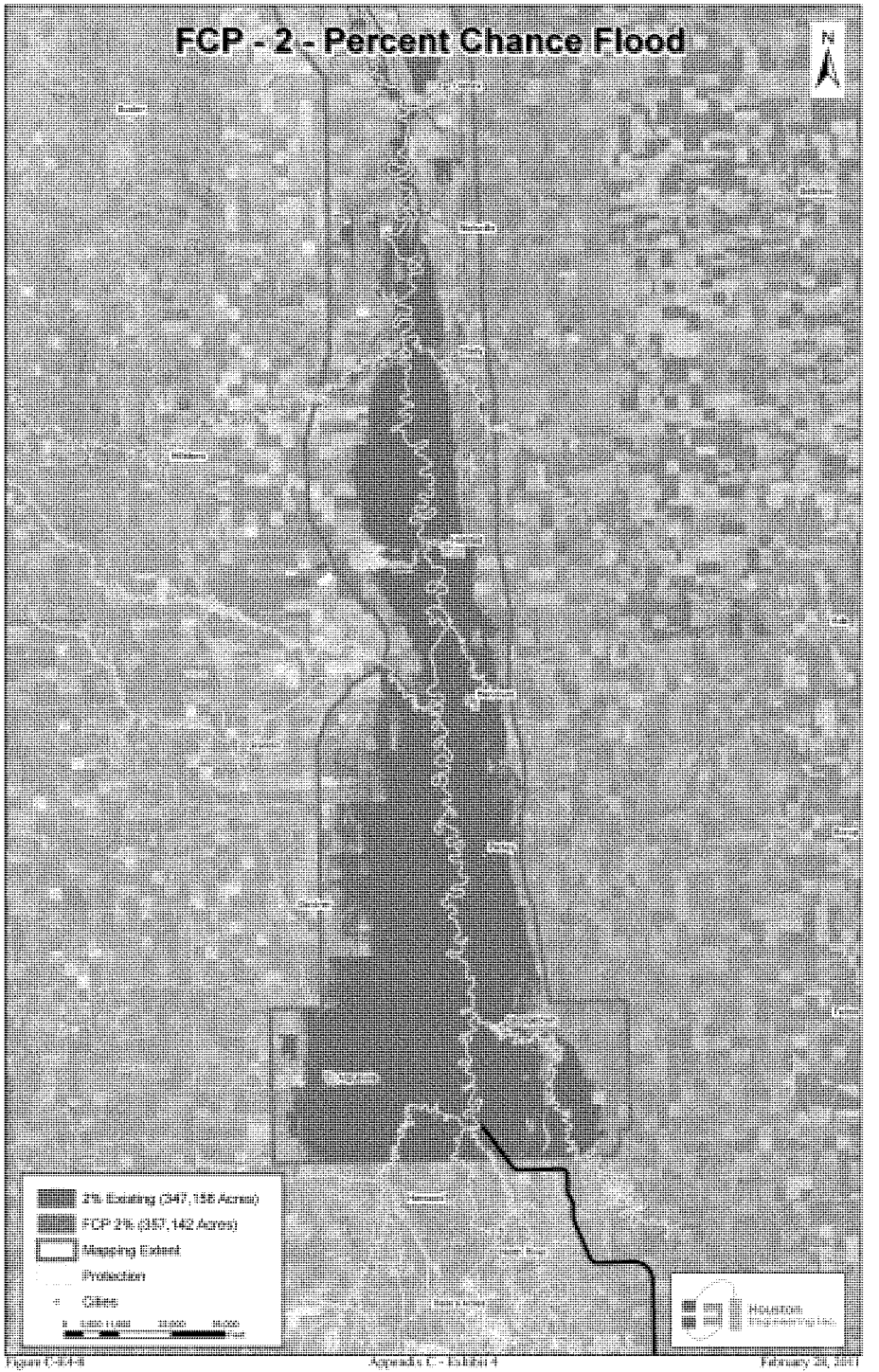


Figure 4-4-8 Appendix E – Exhibit 4 February 28, 2011

Figure C-4.4-2 Appendix C - Exhibit 4 February 28, 2001

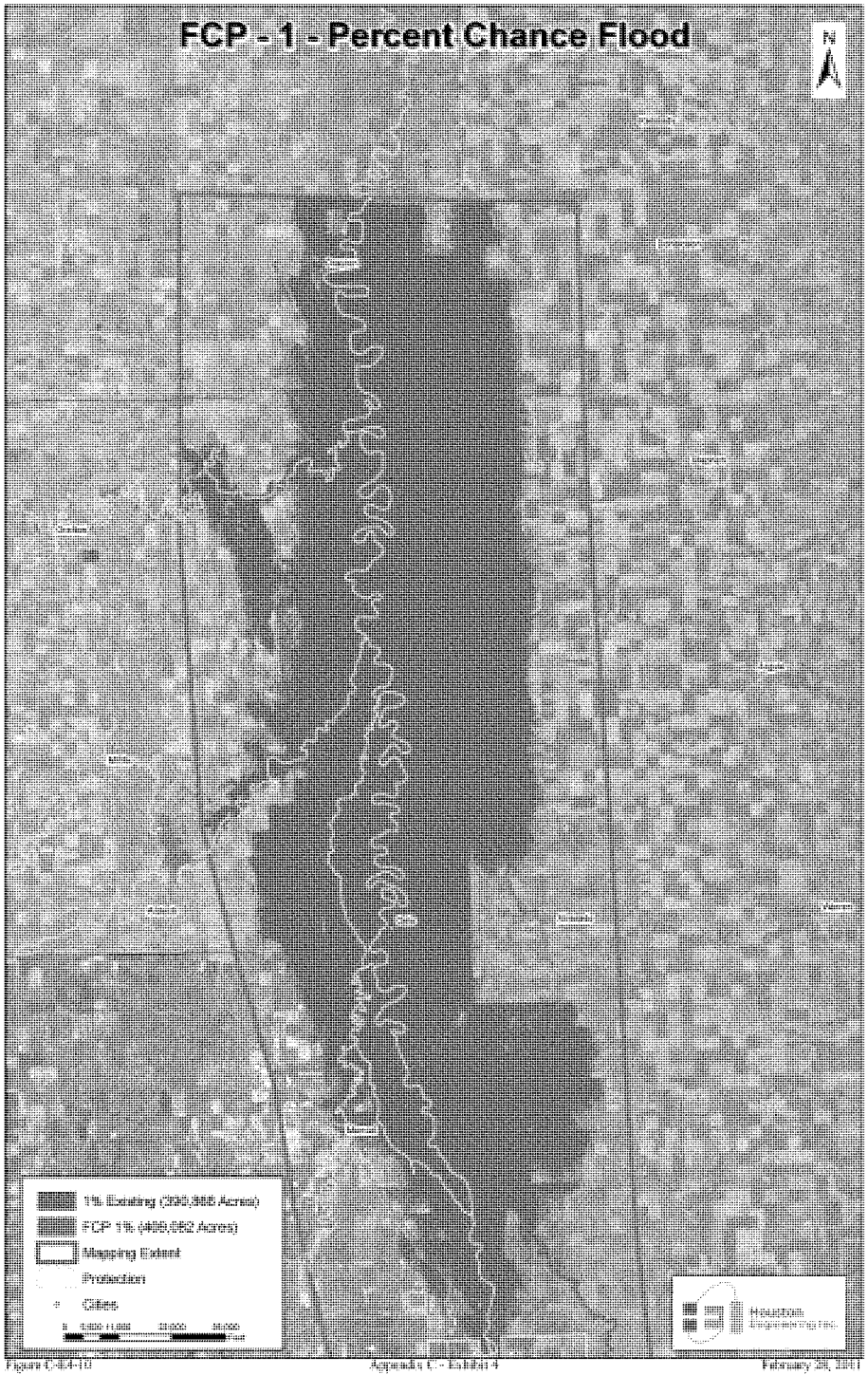

FCP - 1 - Percent Chance Flood
N
1% Existing (280,885 Acres)
FCP 1% (466,662 Acres)
Mapping Extent
Protection
Cities
Houston Engineering

Figure C-8.4-11 Appendix C - Exhibit 4 February 28, 2011

Figure 4-4-12 Appendix C - Exhibit 4 February 28, 2011

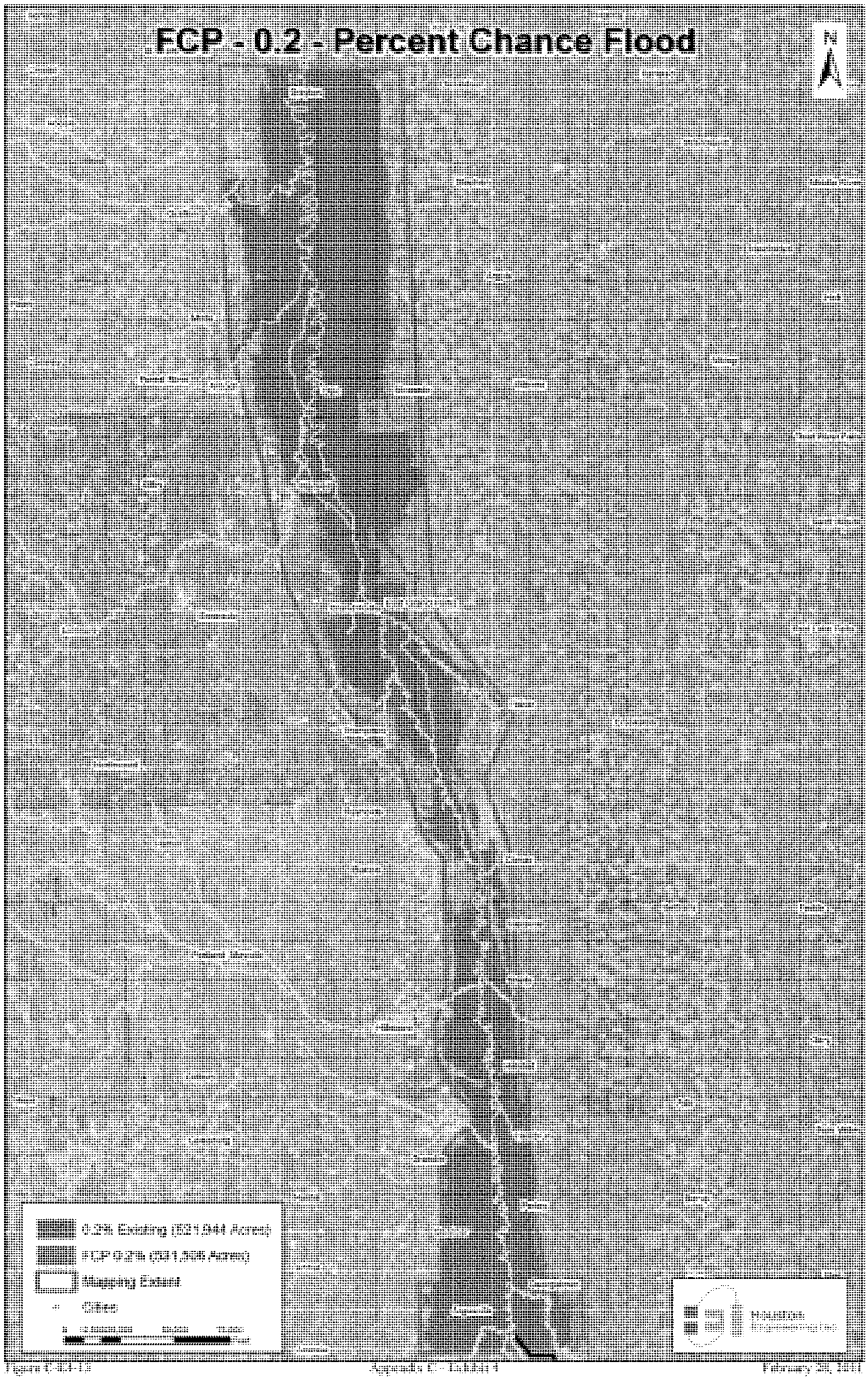
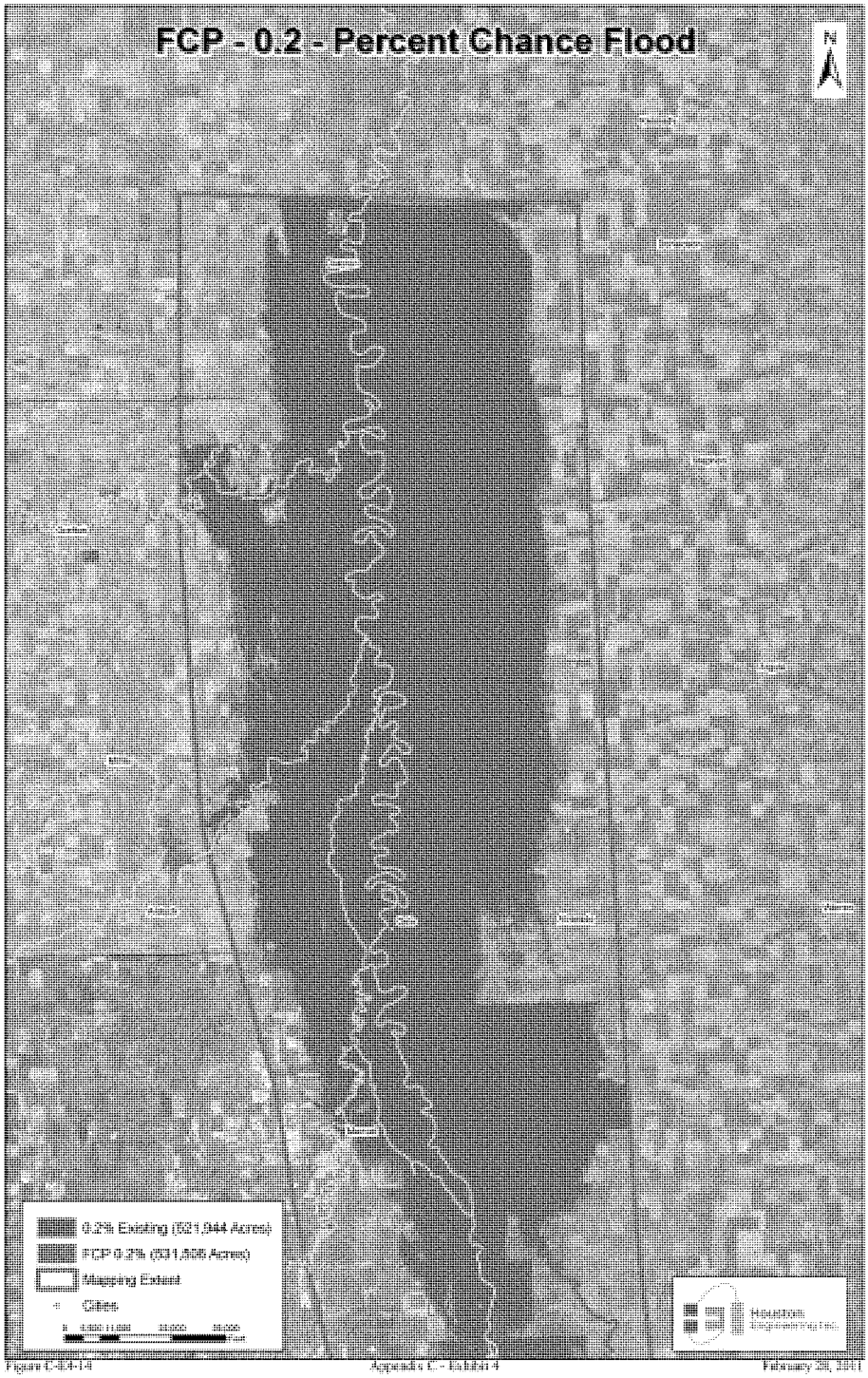


Figure C-4.4-1.0 Appendix C - Exhibit 4 February 28, 2011


FCP - 0.2 - Percent Chance Flood
N
0.2% Existing (621,944 Acres)
FCP 0.2% (631,808 Acres)
Mapping Extent
Cities
Houston Engineering Inc.

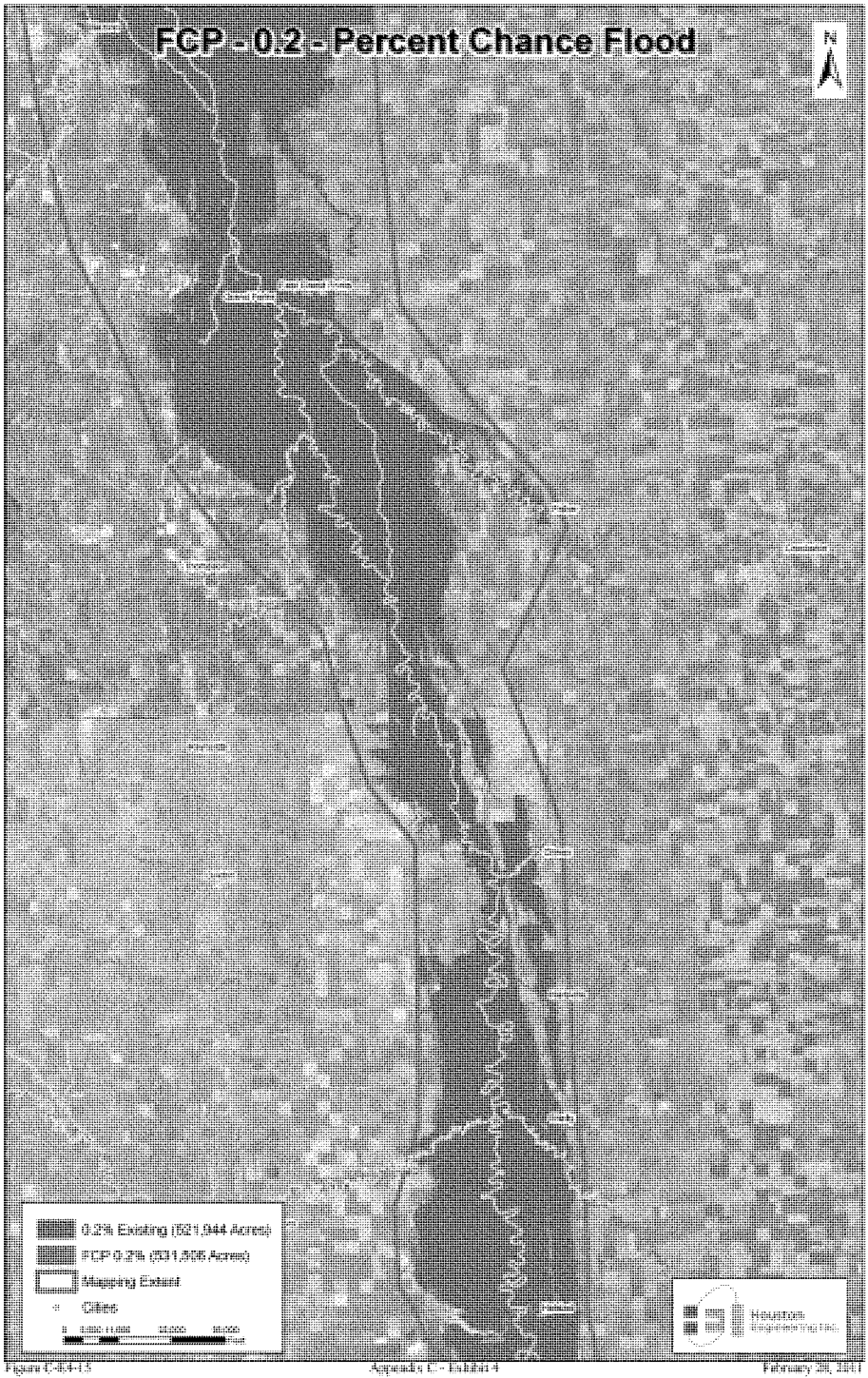


Figure C-44-15 Appendix C - Exhibit 4 February 28, 2001

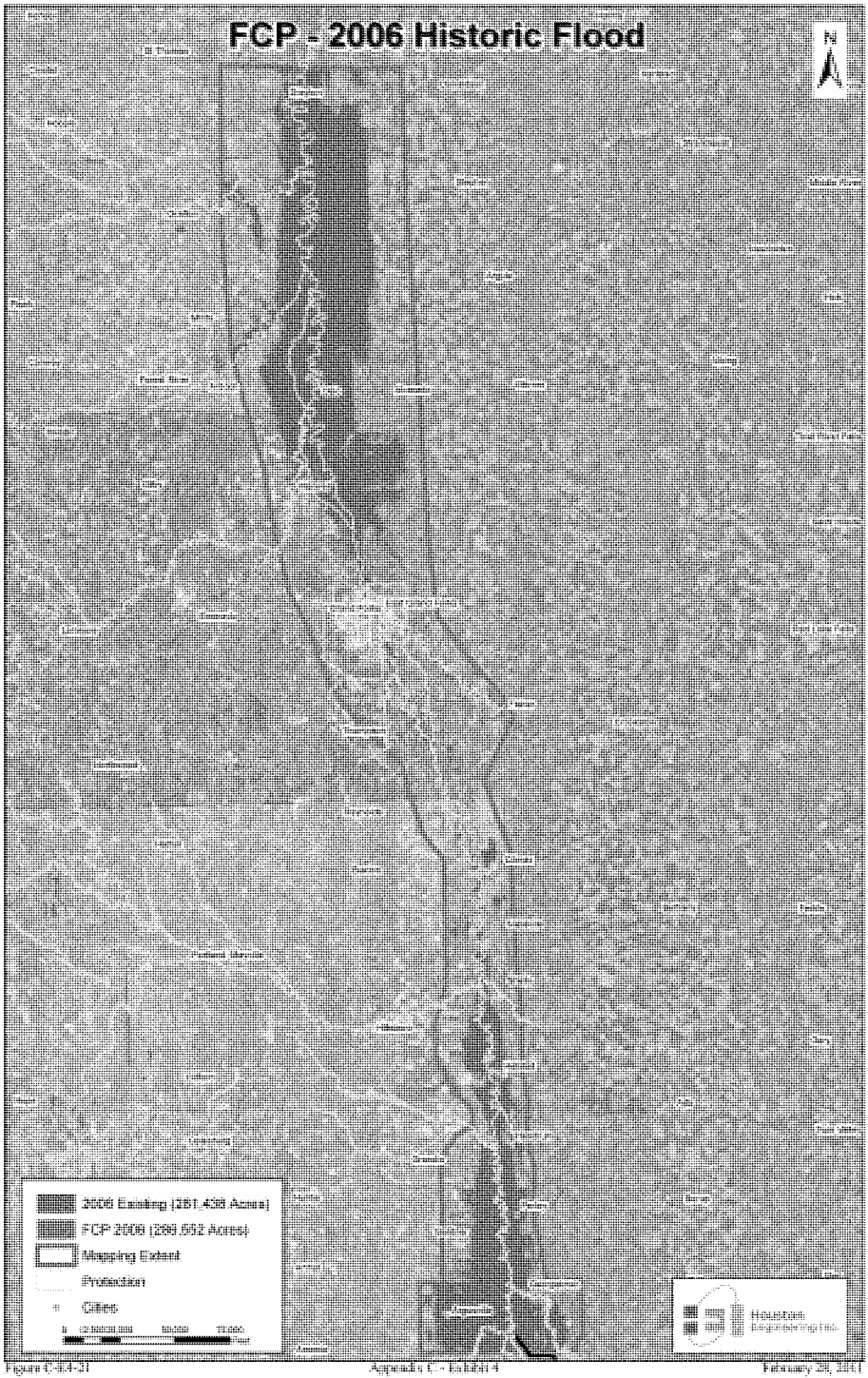
Figure 4-4-16 Appendix E – Exhibit 4 February 28, 2011

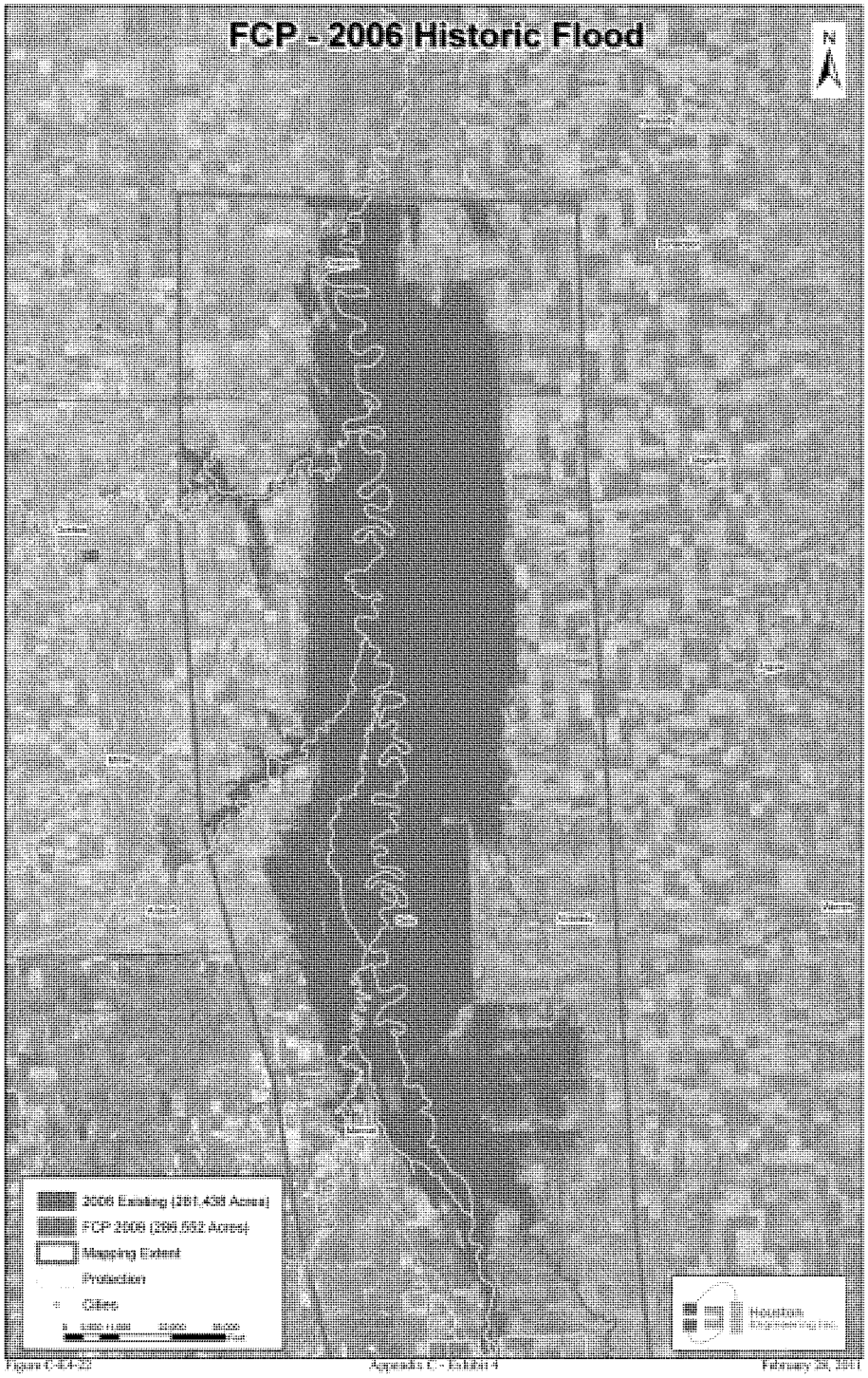
FCP - 1997 Historic Flood
N
1997 Existing (394,020 Acres)
FCP 1997 (402,648 Acres)
Mapping Extent
Protection
Cities
Houston Engineering Inc.
Figure C-4.4-17
Appendix C - Exhibit 4
February 28, 2001

FCP - 1997 Historic Flood
N
1997 Existing (394,020 Acres)
FCP 1997 (402,548 Acres)
Mapping Extent
Protection
Cities
Houston Engineering, Inc.

Figure C-8.4-10 Appendix C - Exhibit 4 February 28, 2001

Figure 4-4-31 Appendix C – Exhibit 4 February 28, 2011


FCP - 2006 Historic Flood
N
2006 Existing (281,438 Acres)
FCP 2006 (280,592 Acres)
Mapping Extent
Protection
Cities
Houston
Figure C-6.4-31
Appendix C - Exhibit 4
February 28, 2011


FCP - 2006 Historic Flood
N
2006 Existing (281,438 Acres)
FCP 2008 (289,552 Acres)
Mapping Extent
Protection
Cities
Houston

FCP - 2006 Historic Flood
N
2006 Existing (281,438 Acres)
FCP 2006 (289,592 Acres)
Mapping Extent
Protection
Cities
Houston
Figure C-44-15
Appendix C - Exhibit 4
February 28, 2011

Figure 4-4-34 Appendix E - Exhibit 4 February 28, 2011

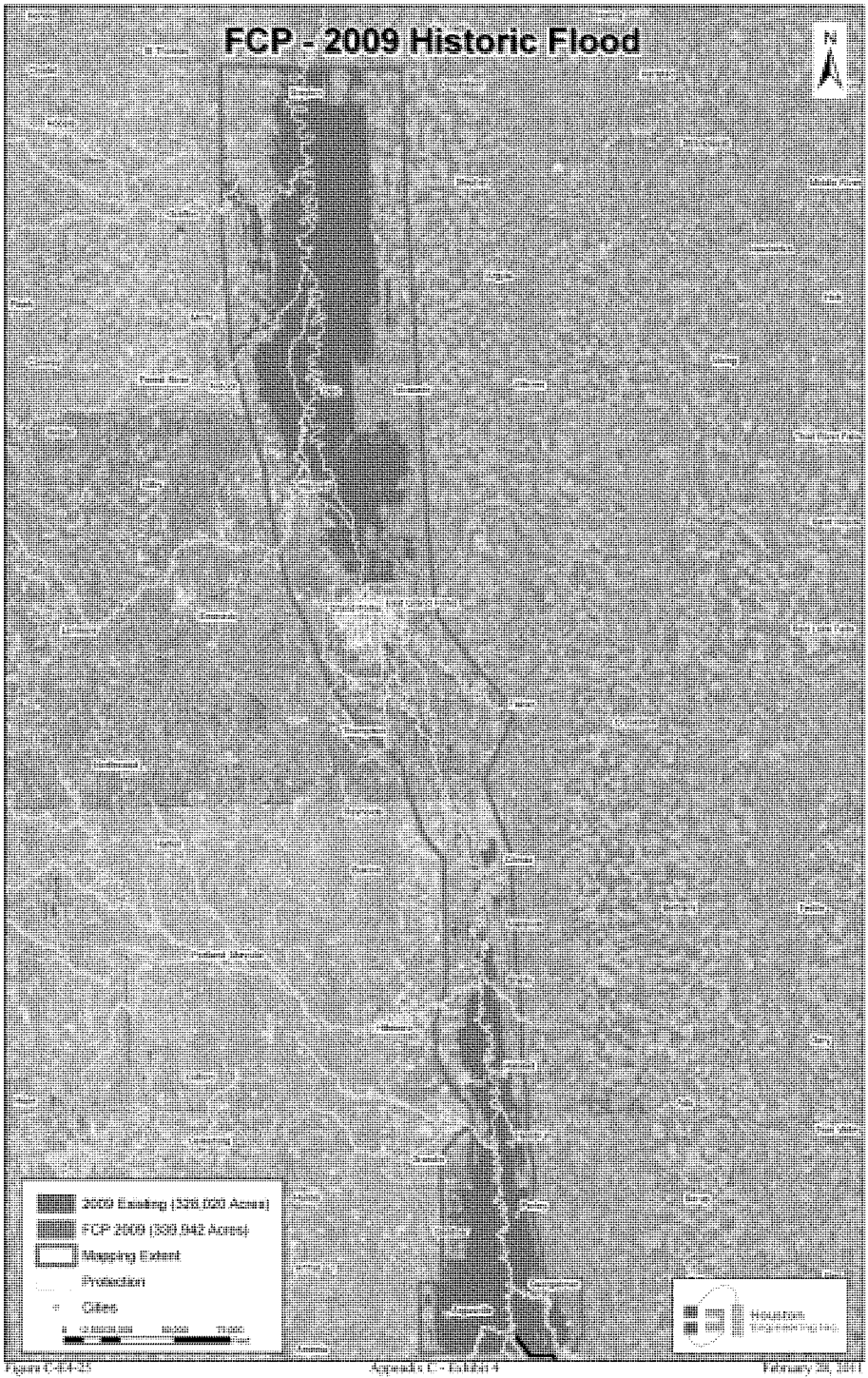
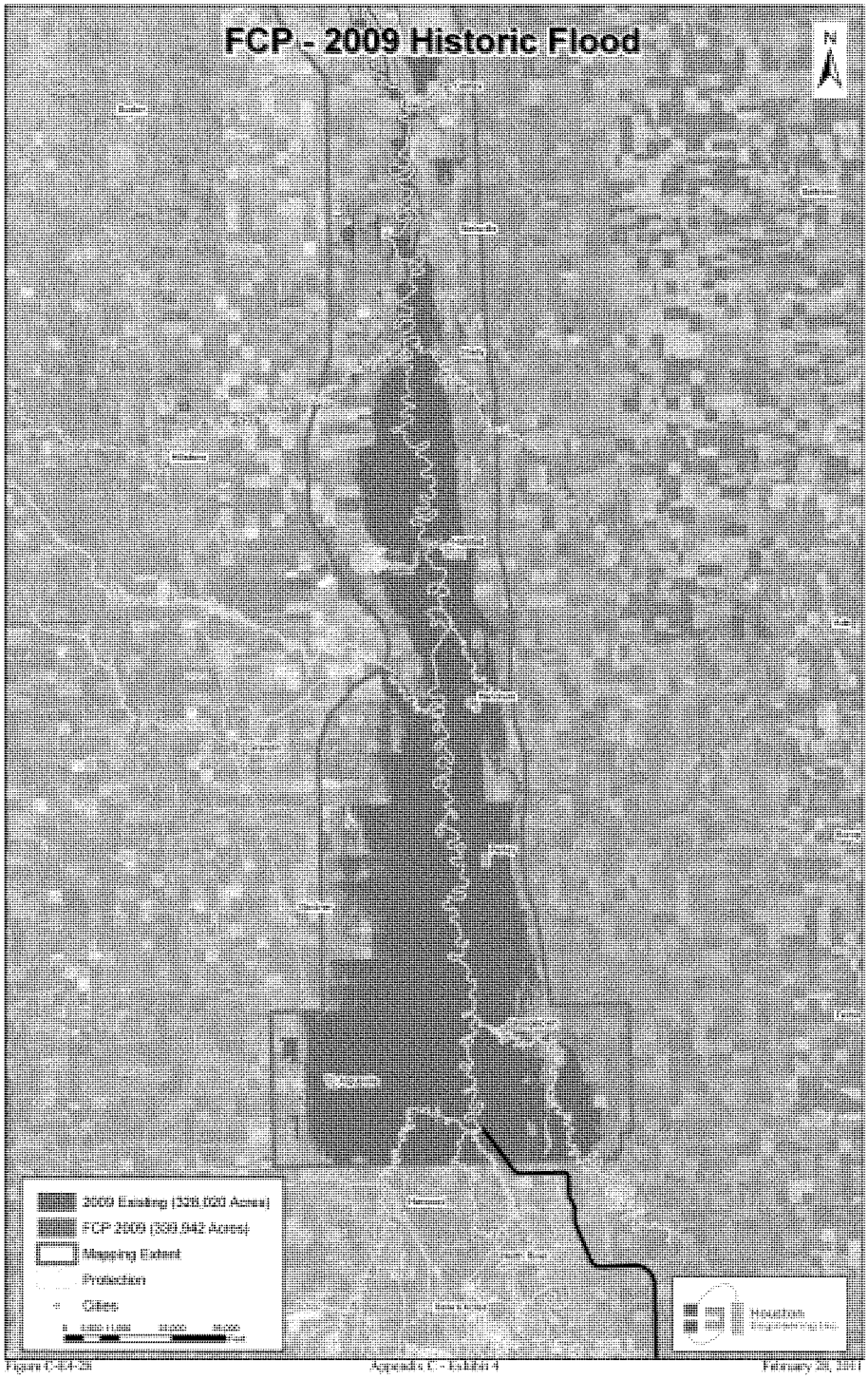


Figure C-44-25 Appendix C - Exhibit 4 February 28, 2011

FCP - 2009 Historic Flood
N
2009 Existing (326,020 Acres)
FCP 2009 (339,942 Acres)
Mapping Extent
Protection
Cities
Houston Engineering Inc.

Figure C-3.4-27 Appendix C - Exhibit 4 February 28, 2001


FCP - 2009 Historic Flood
N
2009 Existing (326,020 Acres)
FCP 2009 (339,942 Acres)
Mapping Extent
Protection
Cities
Houston Engineering

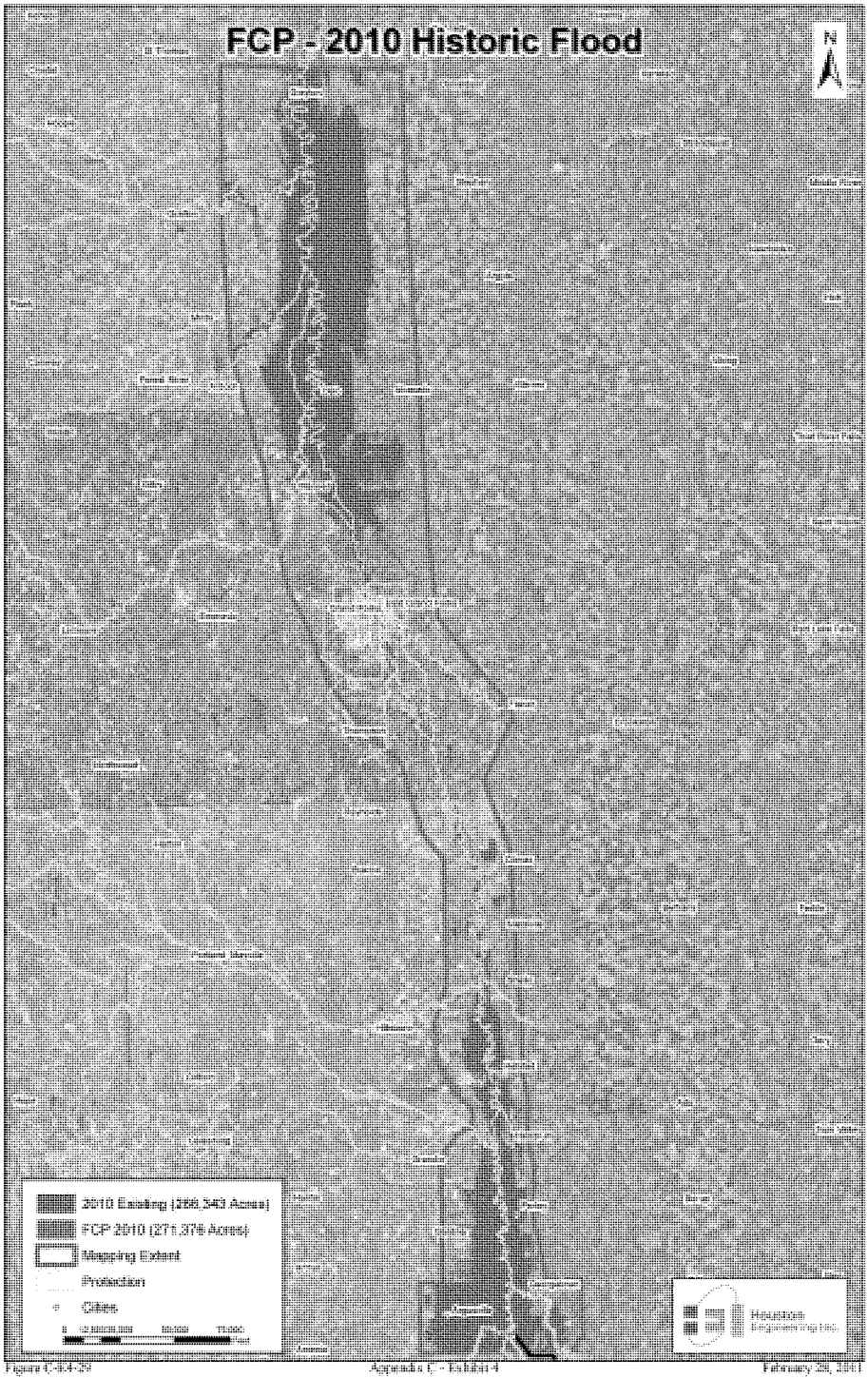


Figure C-4.4-3A Appendix C - Exhibit 4 February 28, 2001

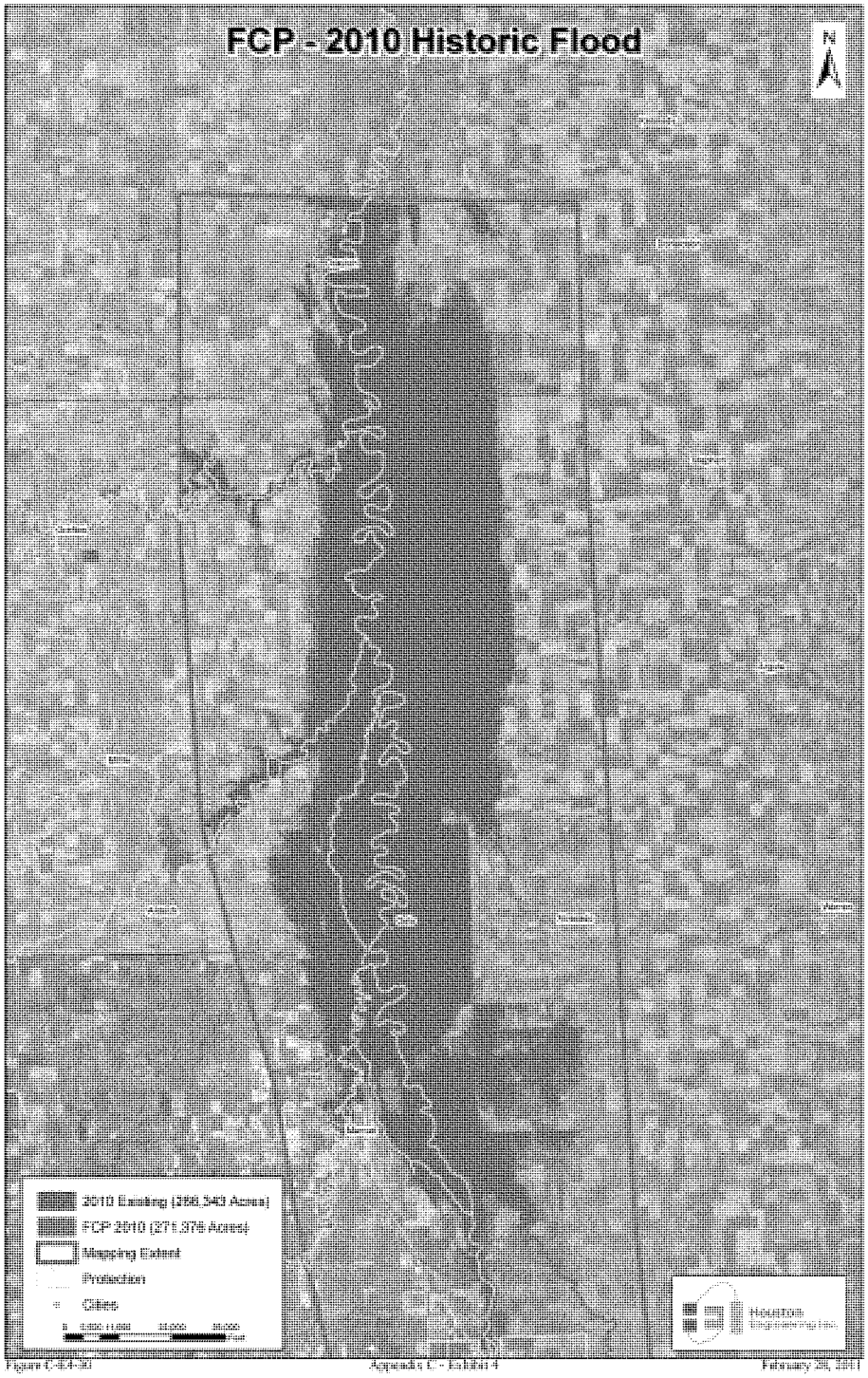

FCP - 2010 Historic Flood
N
2010 Existing (266,340 Acres)
FCP 2010 (271,378 Acres)
Mapping Extent
Protection
Cities
Houston Engineering

Figure C-44-31 Appendix C - Exhibit 4 February 28, 2001

FCP - 2010 Historic Flood
N
2010 Existing (266,340 Acres)
FCP 2010 (271,378 Acres)
Mapping Extent
Protection
Cities
Houston

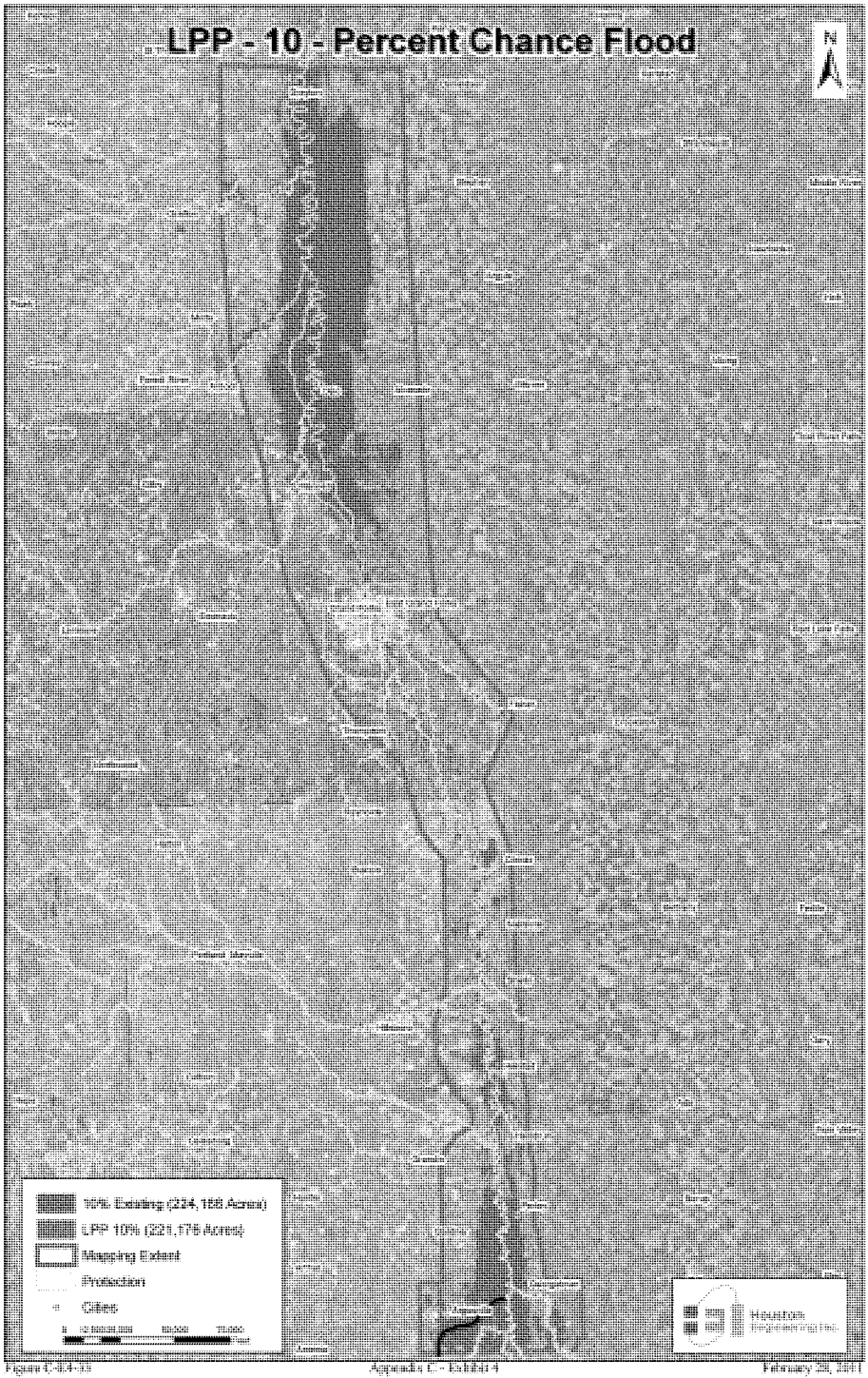


Figure C-3.4-11 Appendix C - Exhibit 4 February 28, 2001

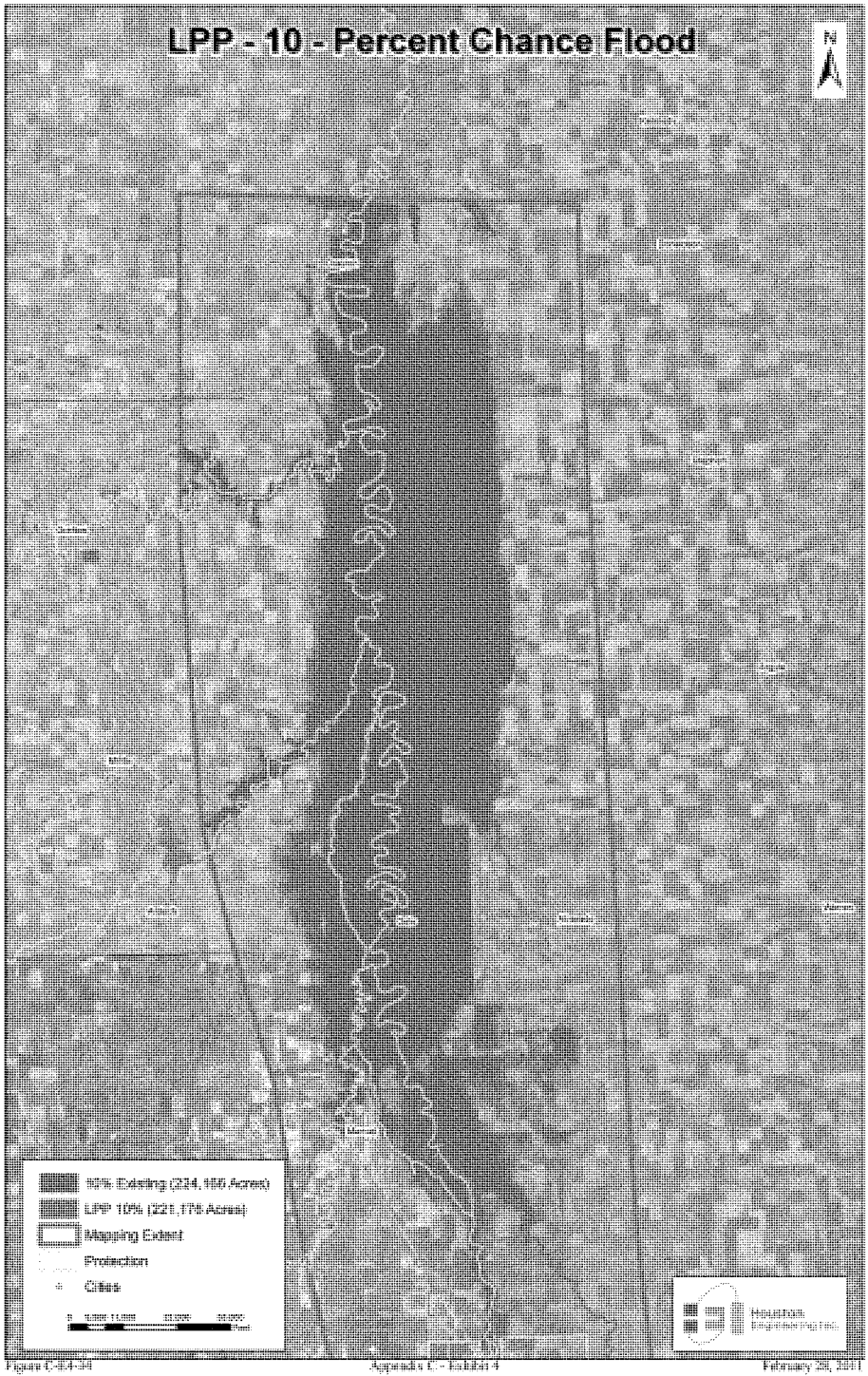


Figure 4-3.4-34 Appendix E - Exhibit 4 February 28, 2011

Figure C-E4-35 Appendix C - Exhibit 4 February 28, 2001

Figure 4-4-10 Appendix E – Exhibit 4 February 28, 2011

Figure C-4.4-37 Appendix C - Exhibit 4 February 28, 2011

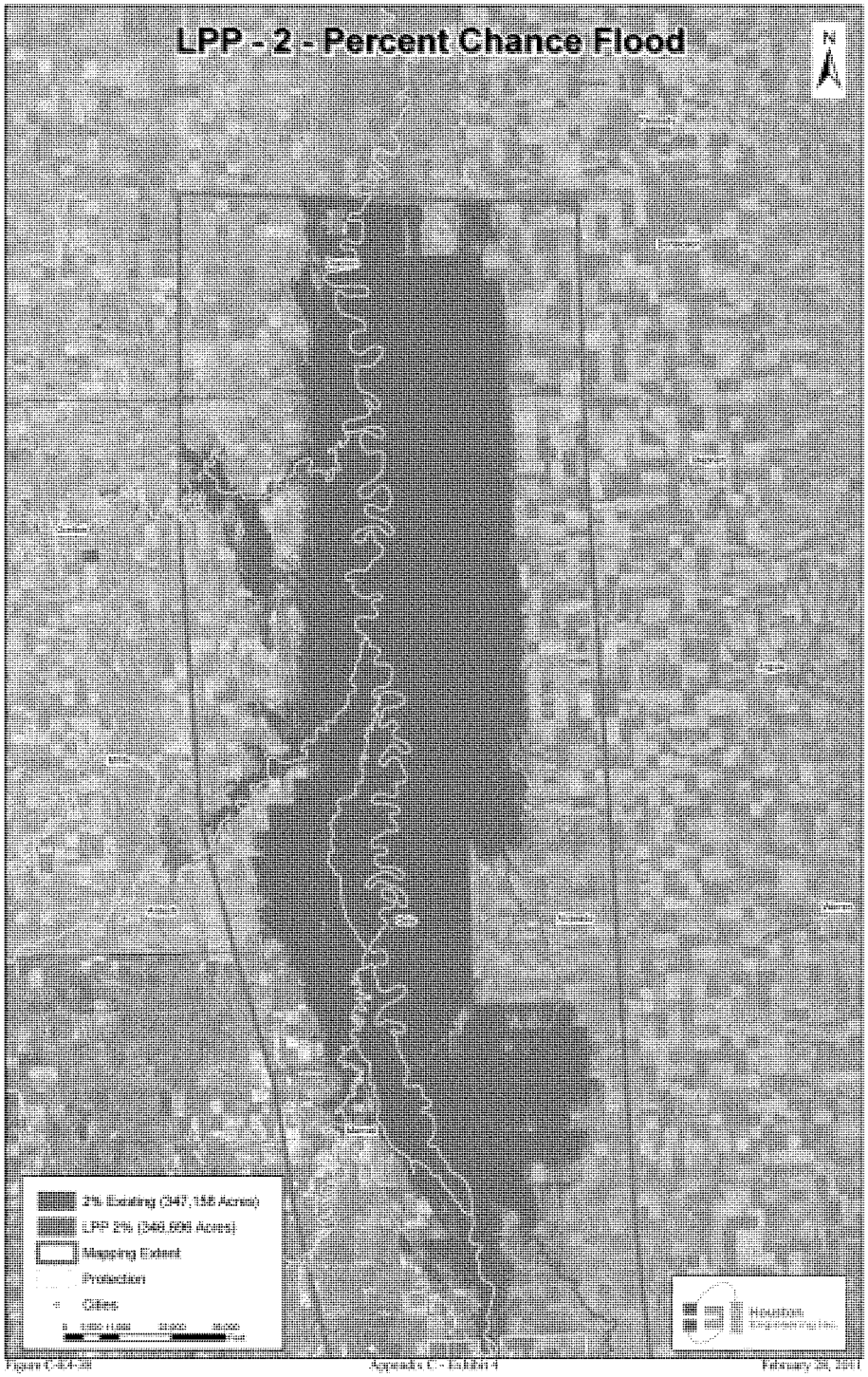

LPP - 2 - Percent Chance Flood
N
2% Existing (347,135 Acres)
LPP 2% (349,999 Acres)
Mapping Extent
Protection
Cities
Houston

Figure C-44-01 Appendix C - Exhibit 4 February 28, 2001

Figure 4-3.4-8J Appendix E – Exhibit 4 February 28, 2011

Figure C-E4-01 Appendix C - Exhibit 4 February 28, 2001

LPP - 1 - Percent Chance Flood
N
1% Existing (295,885 Acres)
LPP 1% (350,557 Acres)
Mapping Extent
Protection
Cities
Houston Engineering Inc.

Figure C-3.4-13 Appendix C - Exhibit 4 February 28, 2011

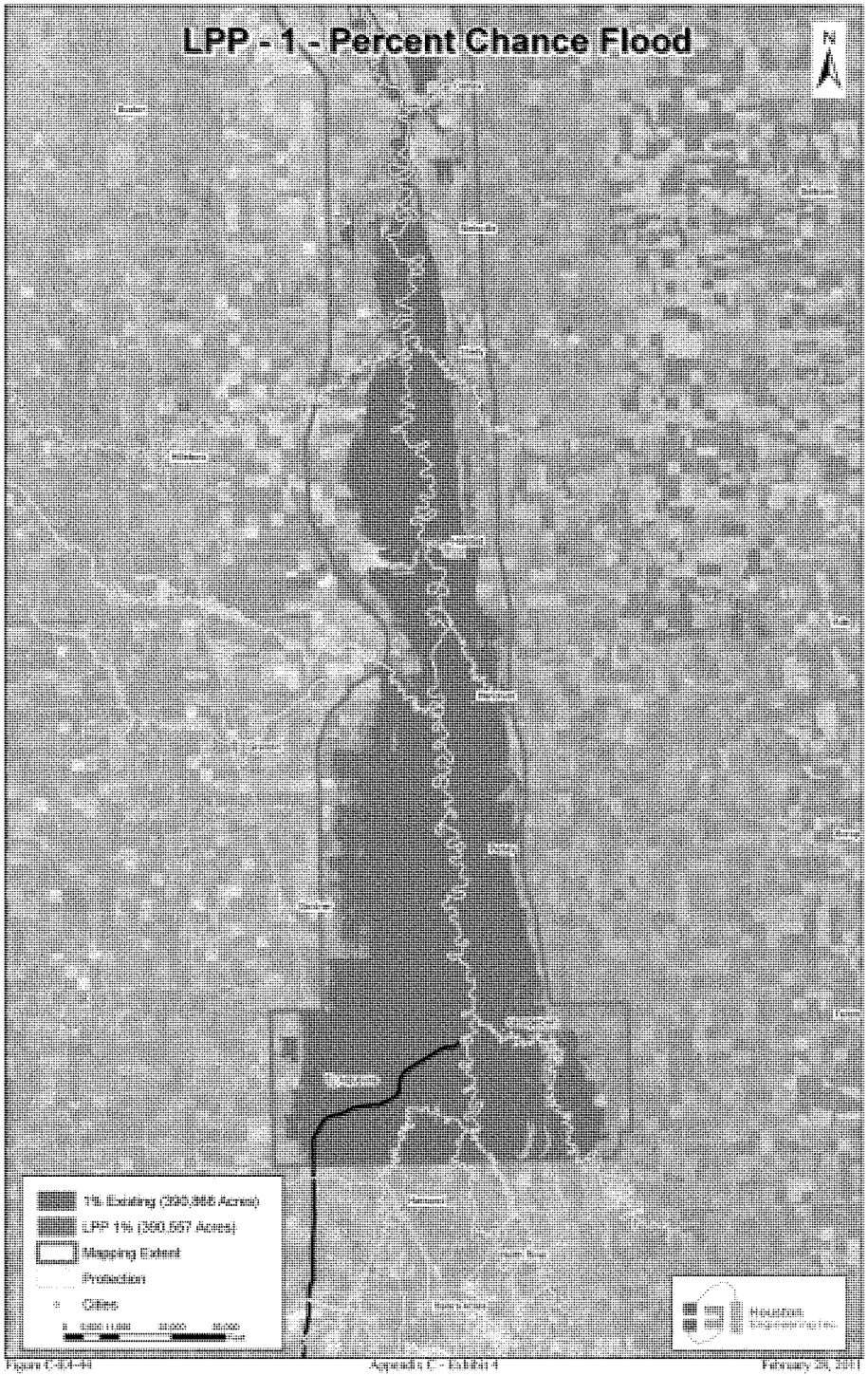
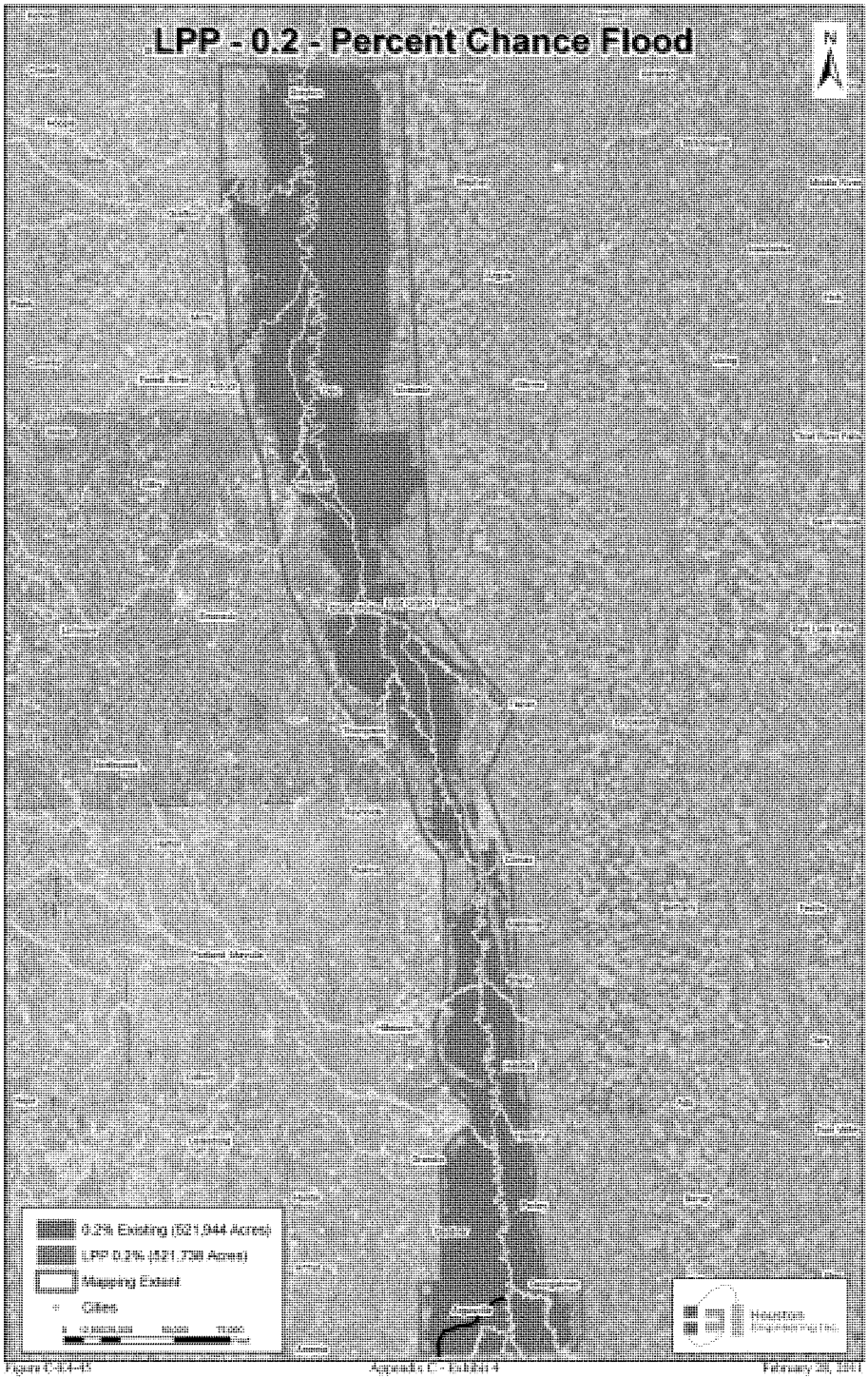
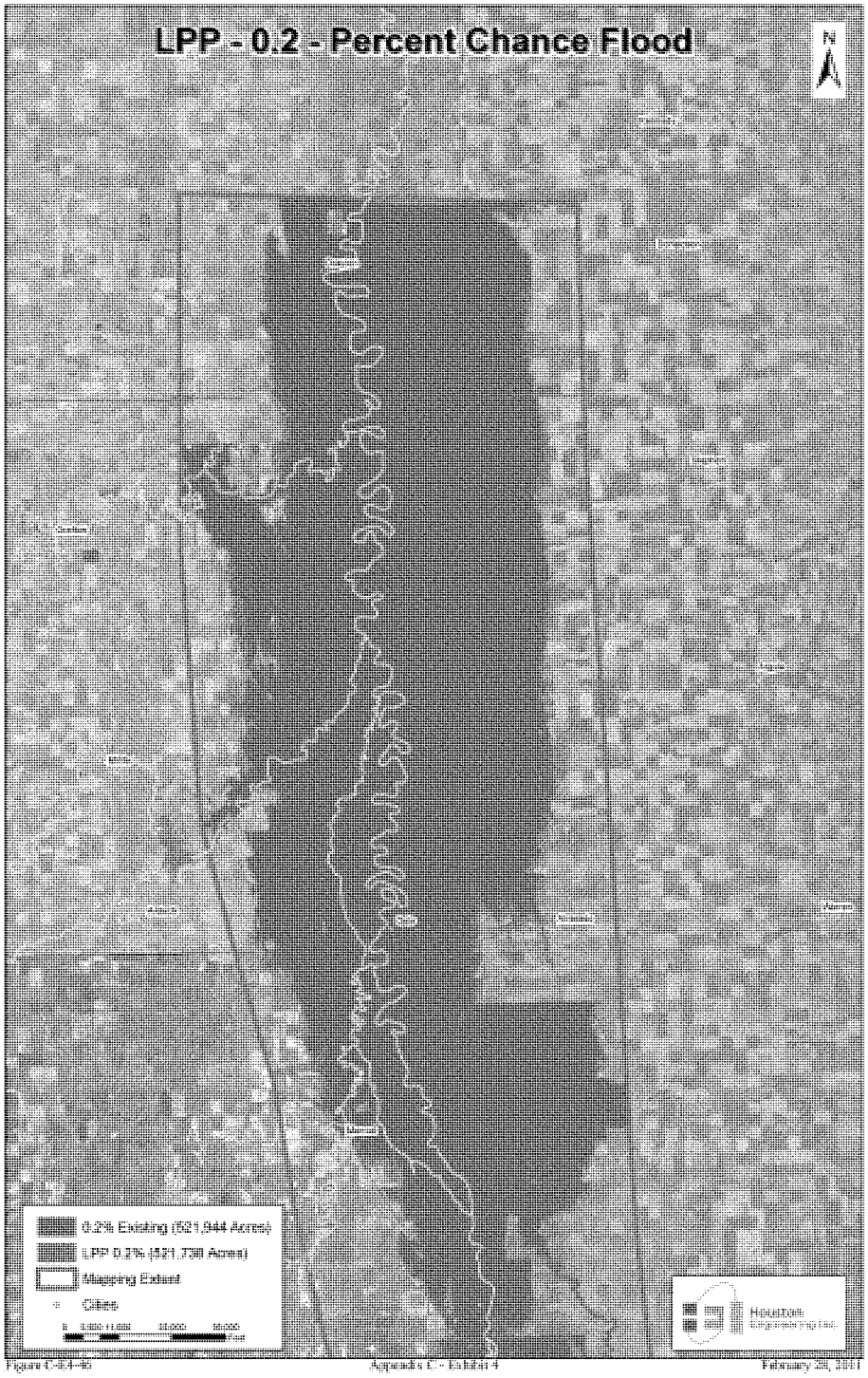


Figure 4-4-44 Appendix E - Exhibit 4 February 28, 2011


LPP - 0.2 - Percent Chance Flood
N
0.2% Existing (521,944 Acres)
LPP 0.2% (521,739 Acres)
Mapping Extent
Cities
Houston
Figure C-4.4-45
Appendix C - Exhibit 4
February 28, 2011


LPP - 0.2 - Percent Chance Flood
N
0.2% Existing (521,944 Acres)
LPP 0.2% (521,738 Acres)
Mapping Extent
Cities
Houston
Engineering Inc.

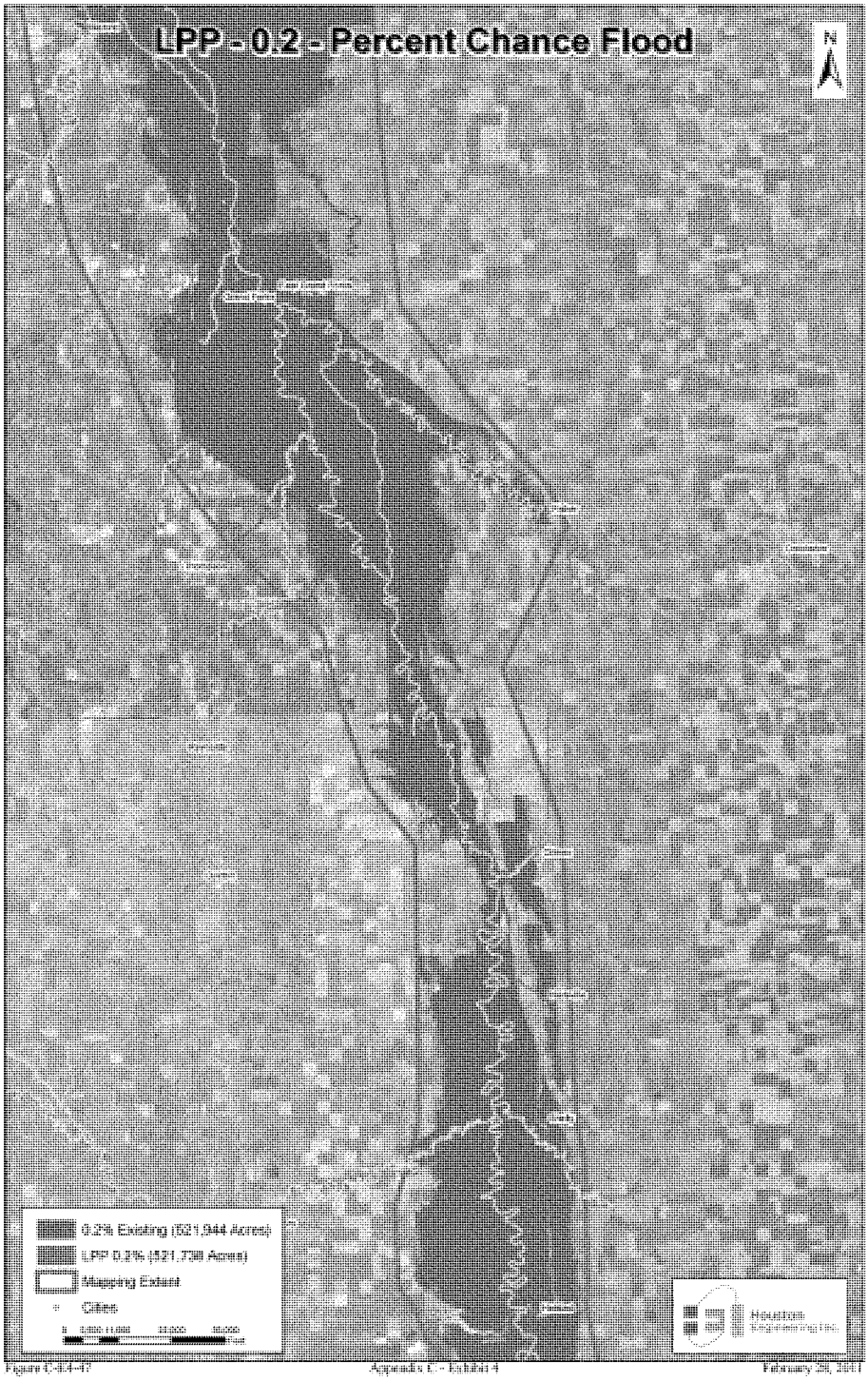


Figure C-4.4-07 Appendix C - Exhibit 4 February 28, 2011

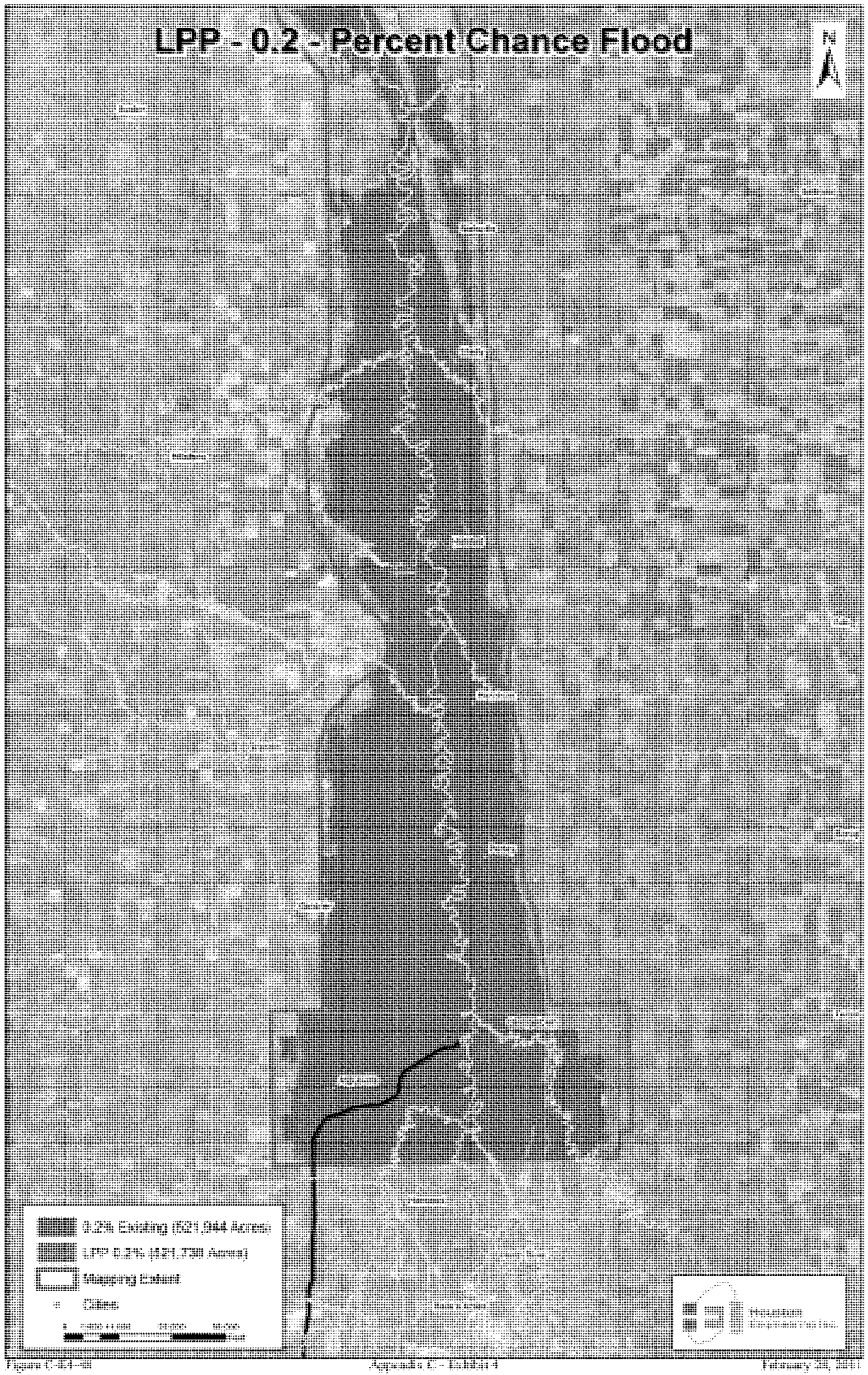

LPP - 0.2 - Percent Chance Flood
N
0.2% Existing (521,944 Acres)
LPP 0.2% (521,738 Acres)
Mapping Extent
Cities
Houston Engineering Inc.

Figure C-4.4-40 Appendix C - Exhibit 4 February 28, 2001

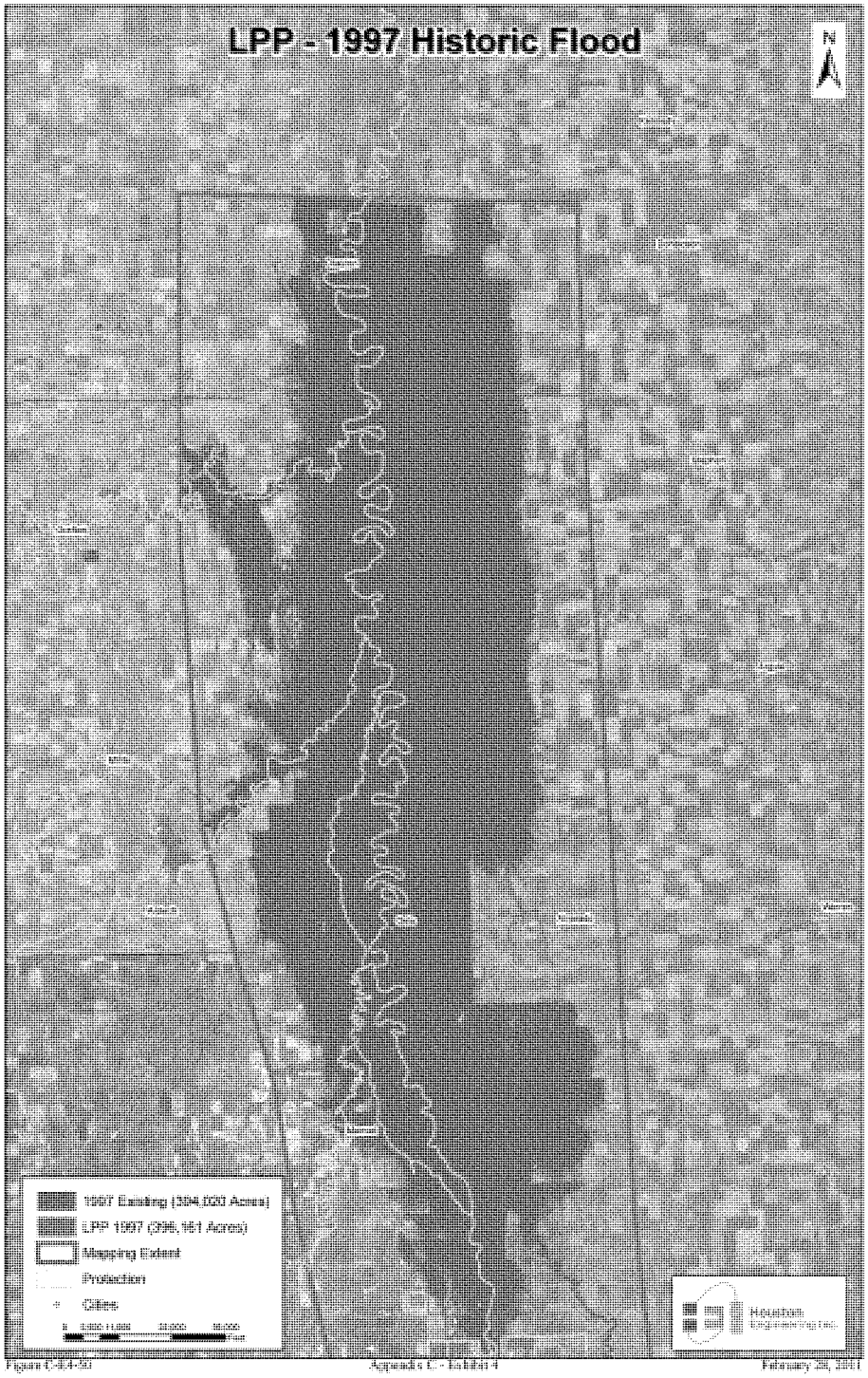

LPP - 1997 Historic Flood
N
1997 Existing (394,020 Acres)
LPP 1997 (396,161 Acres)
Mapping Extent
Protection
Cities
Houston Engineering Inc.

Figure C-4.4-31 Appendix C - Exhibit 4 February 28, 2001

Figure 4-4-52 Appendix E – Exhibit 4 February 28, 2011

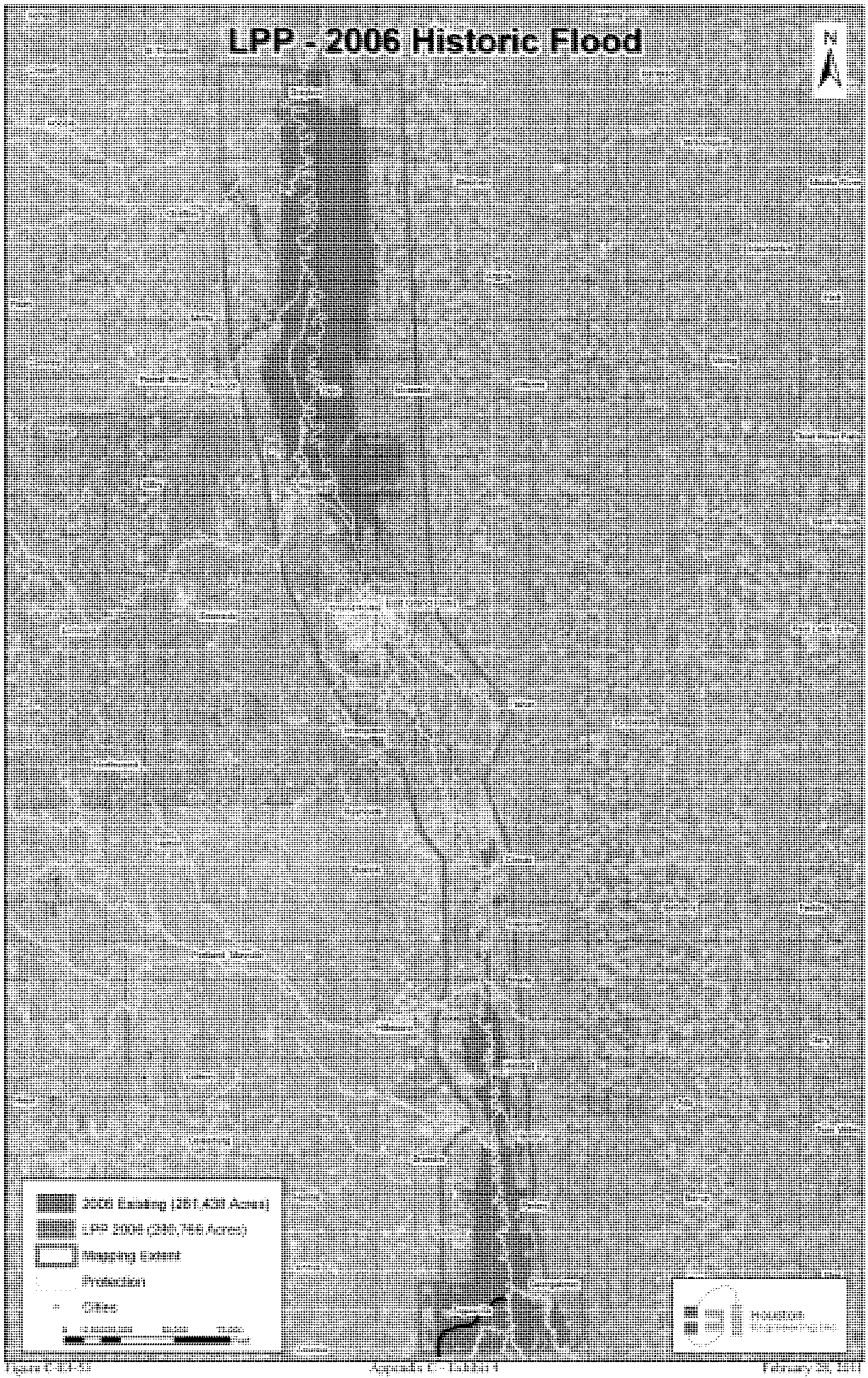


Figure C-8.4-5.1 Appendix L - Exhibit 4 February 28, 2011

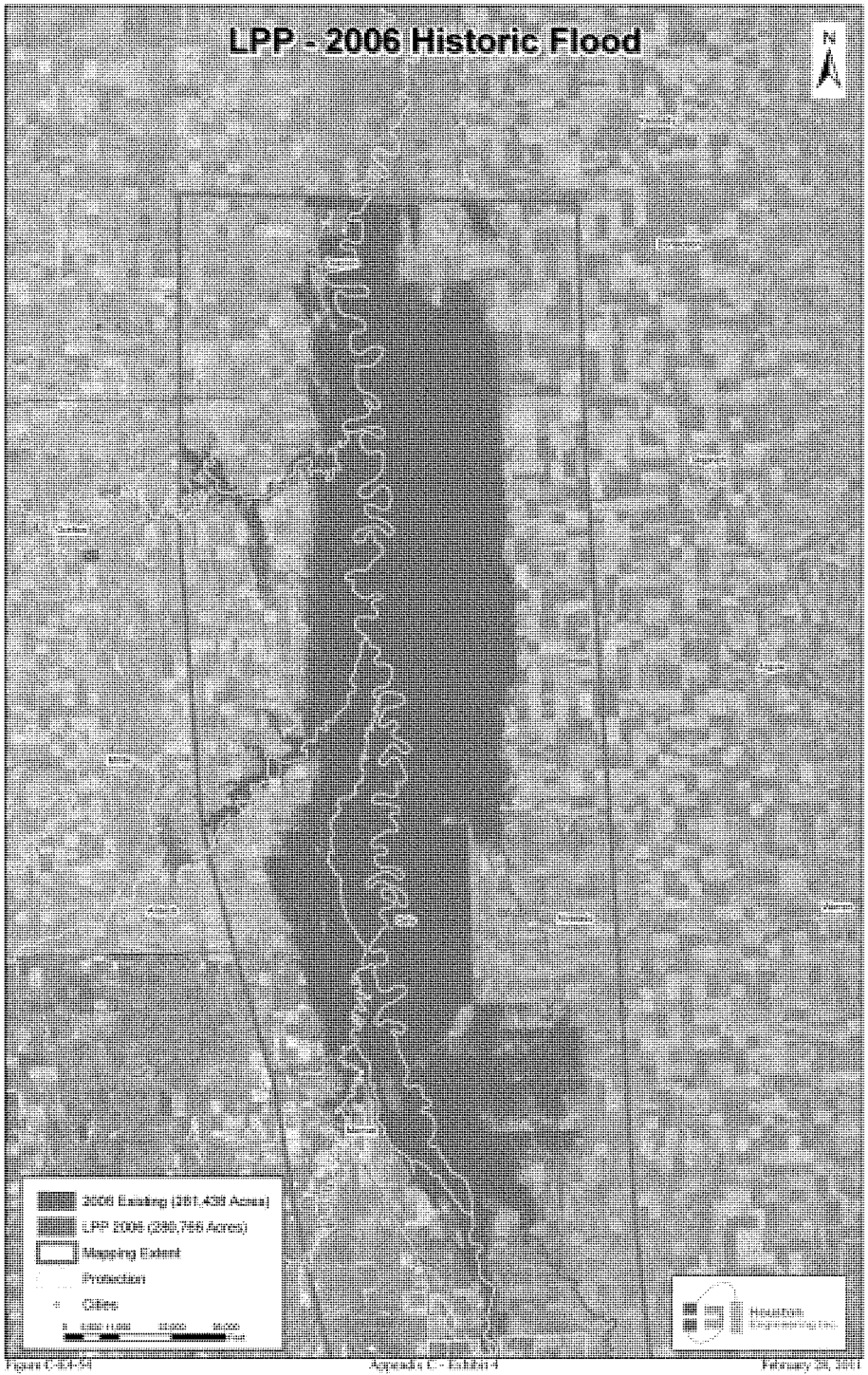


Figure 4-3.4-54 Appendix C - Exhibit 4 February 28, 2011

Figure C-3.4-55 Appendix C - Exhibit 4 February 28, 2011

Figure 4.4-20 Appendix E – Exhibit 4 February 28, 2011

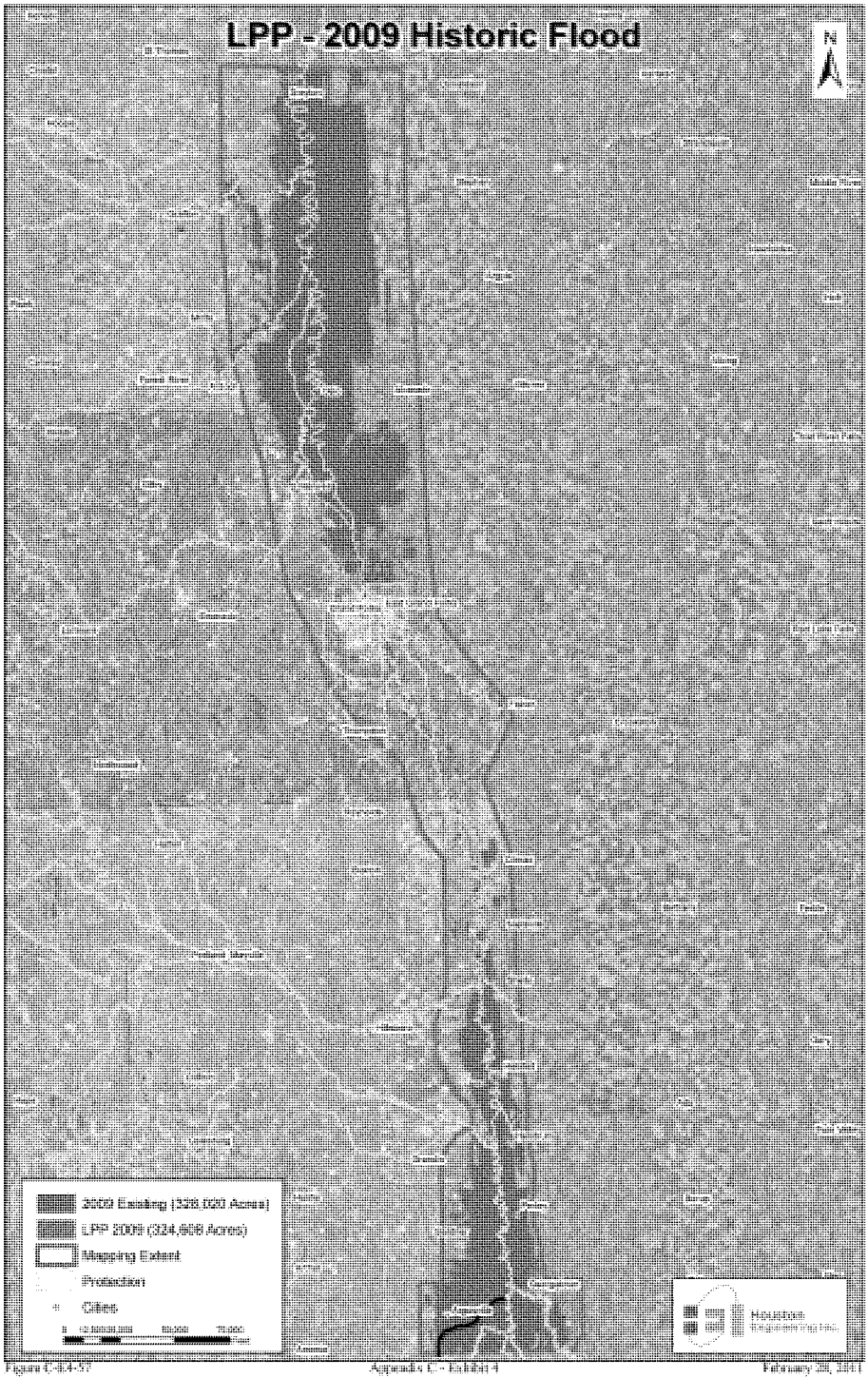


Figure C-4.4-57 Appendix C - Exhibit 4 February 28, 2011

Figure 4-4-5B Appendix C – Exhibit 4 February 28, 2011

Figure C-4.4-59 Appendix C - Exhibit 4 February 28, 2011

LPP - 2009 Historic Flood
N
2009 Existing (336,020 Acres)
LPP 2009 (324,908 Acres)
Mapping Extent
Protection
Cities
Houston Engineering, Inc.

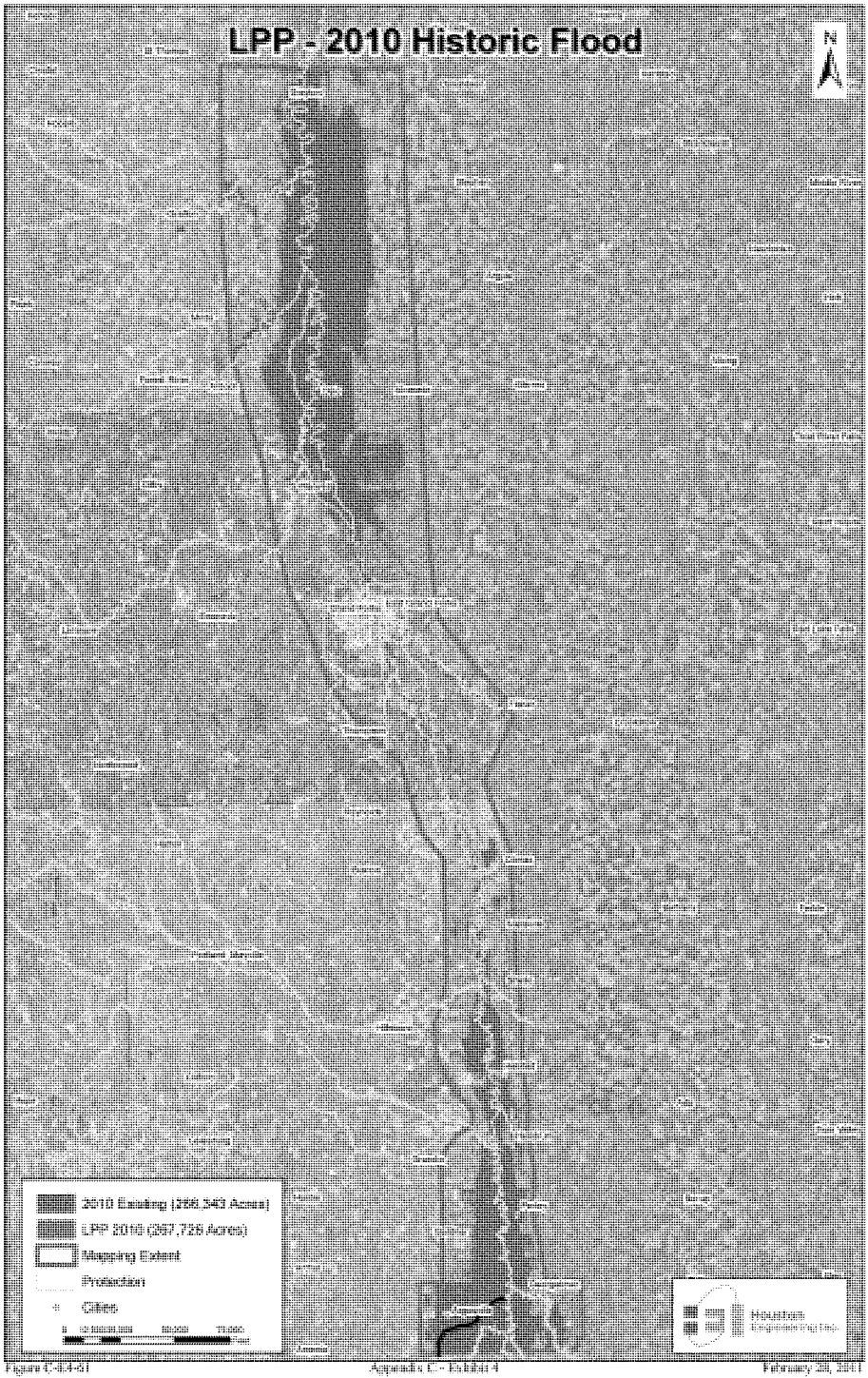


Figure C-4.4-01 Appendix C - Exhibit 4 February 28, 2011

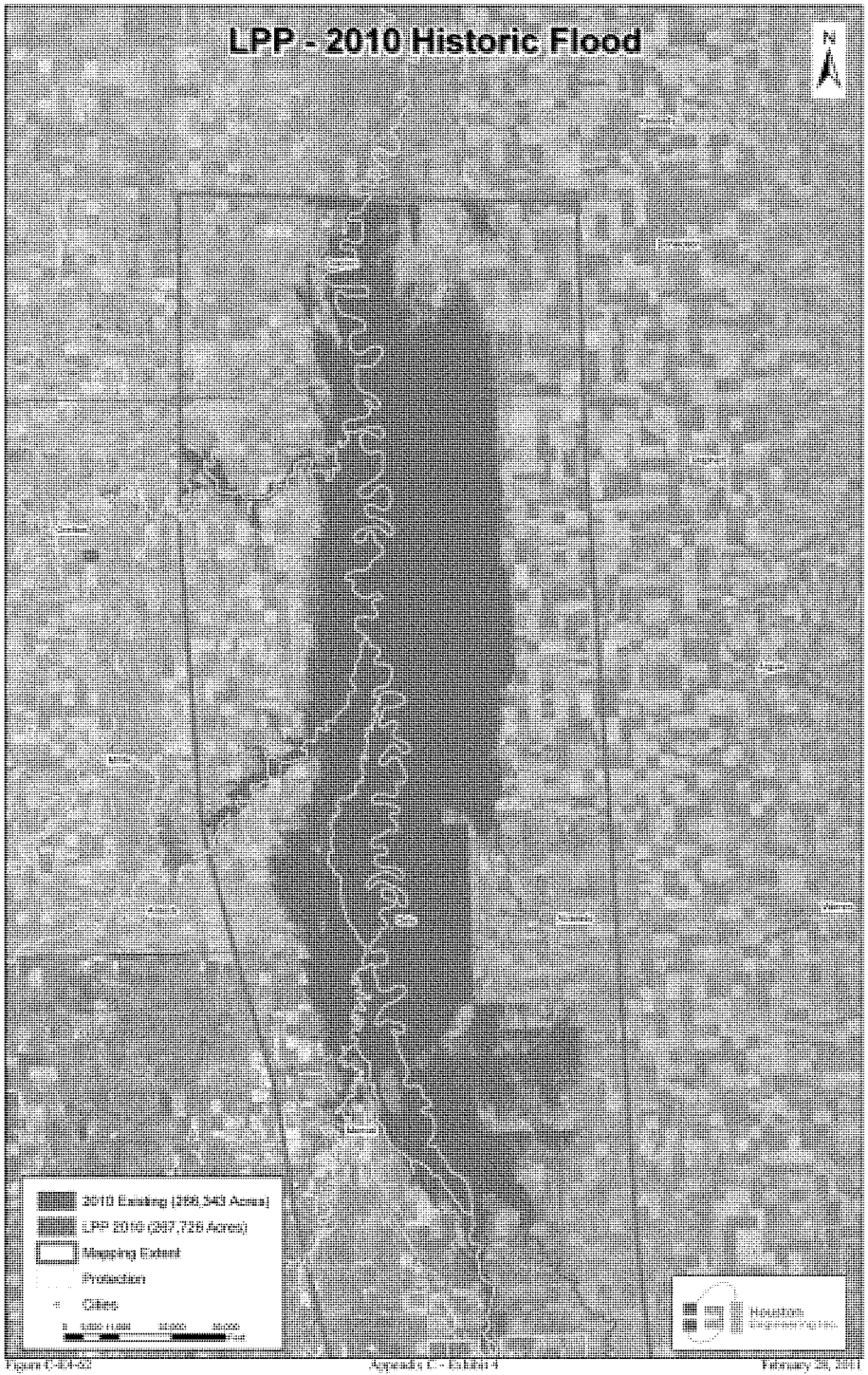
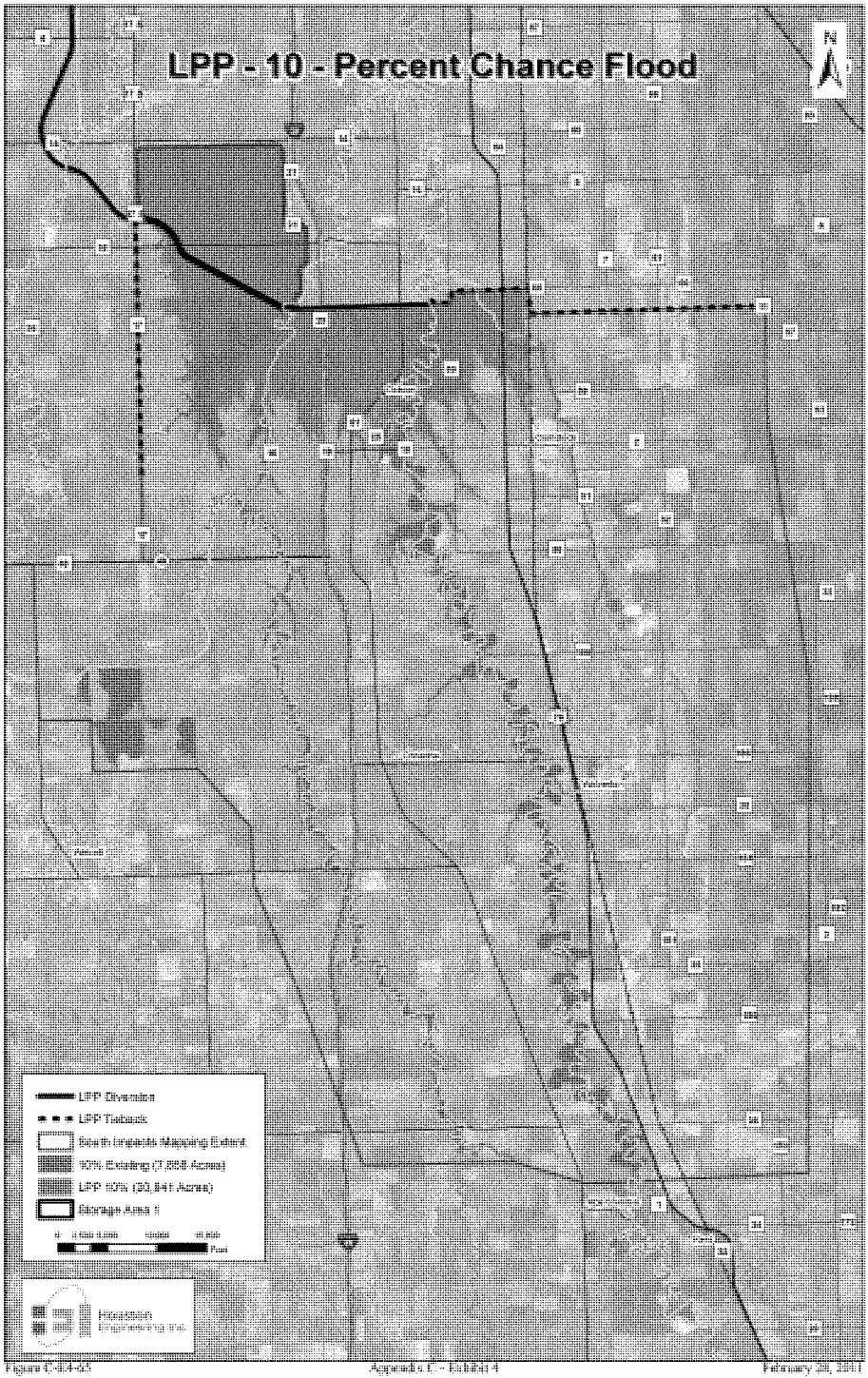


Figure 4-4-22 Appendix E – Exhibit 4 February 28, 2011

Figure C-3.4-3.5 Appendix C – Exhibit 4 February 28, 2011

Figure 4-4-64 Appendix E - Exhibit 4 February 28, 2011


LPP - 10 - Percent Chance Flood
N
LPP Diversion
LPP Tieback
South Irrigation Mapping Extent
10% Existing (7,258 Acres)
LPP 10% (30,841 Acres)
Storage Area 1
Miles
Figure C-4.4-05
Appendix C - Exhibit 4
February 28, 2001

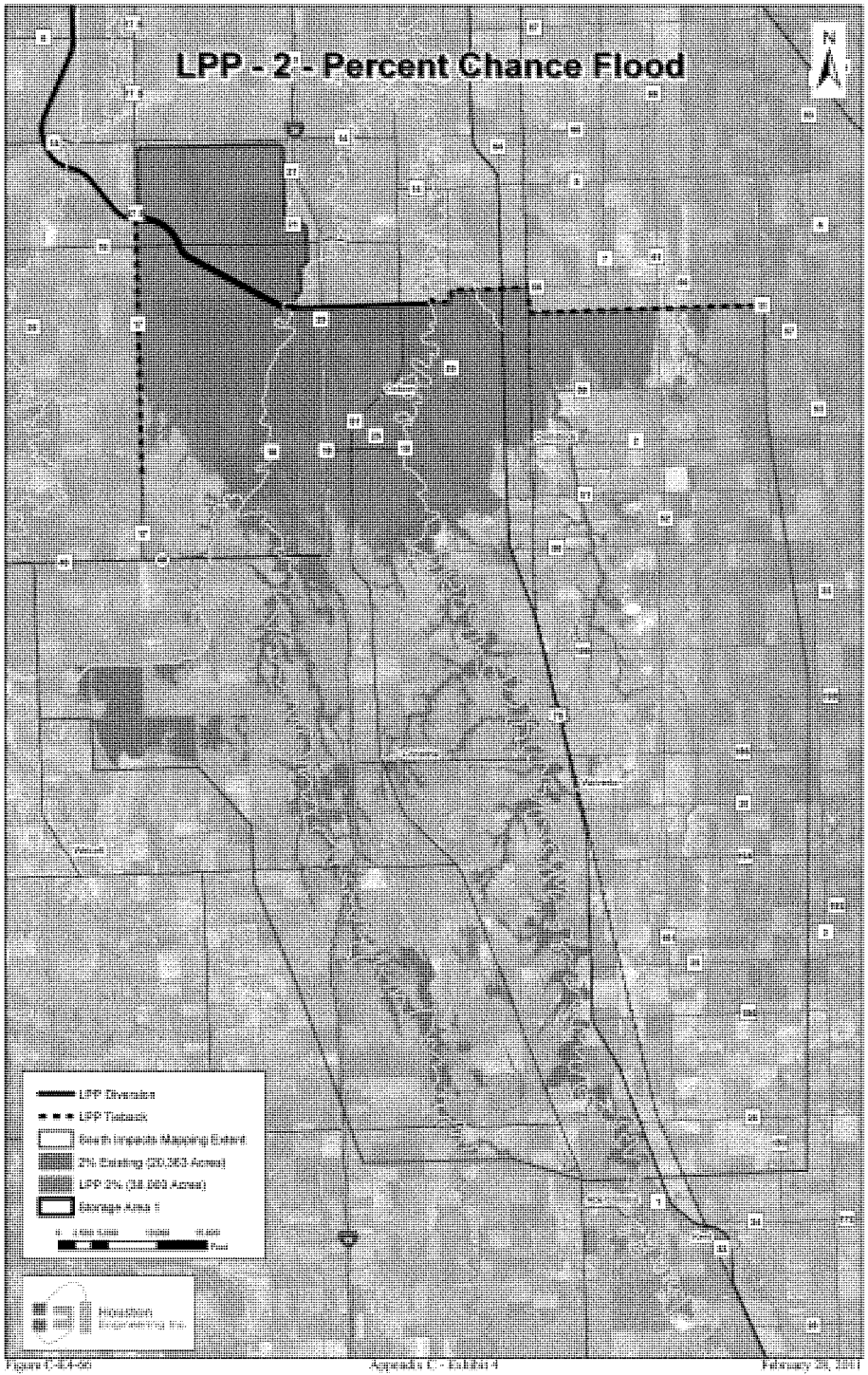


Figure 4-3.4-20 Appendix E - Exhibit 4 February 28, 2011

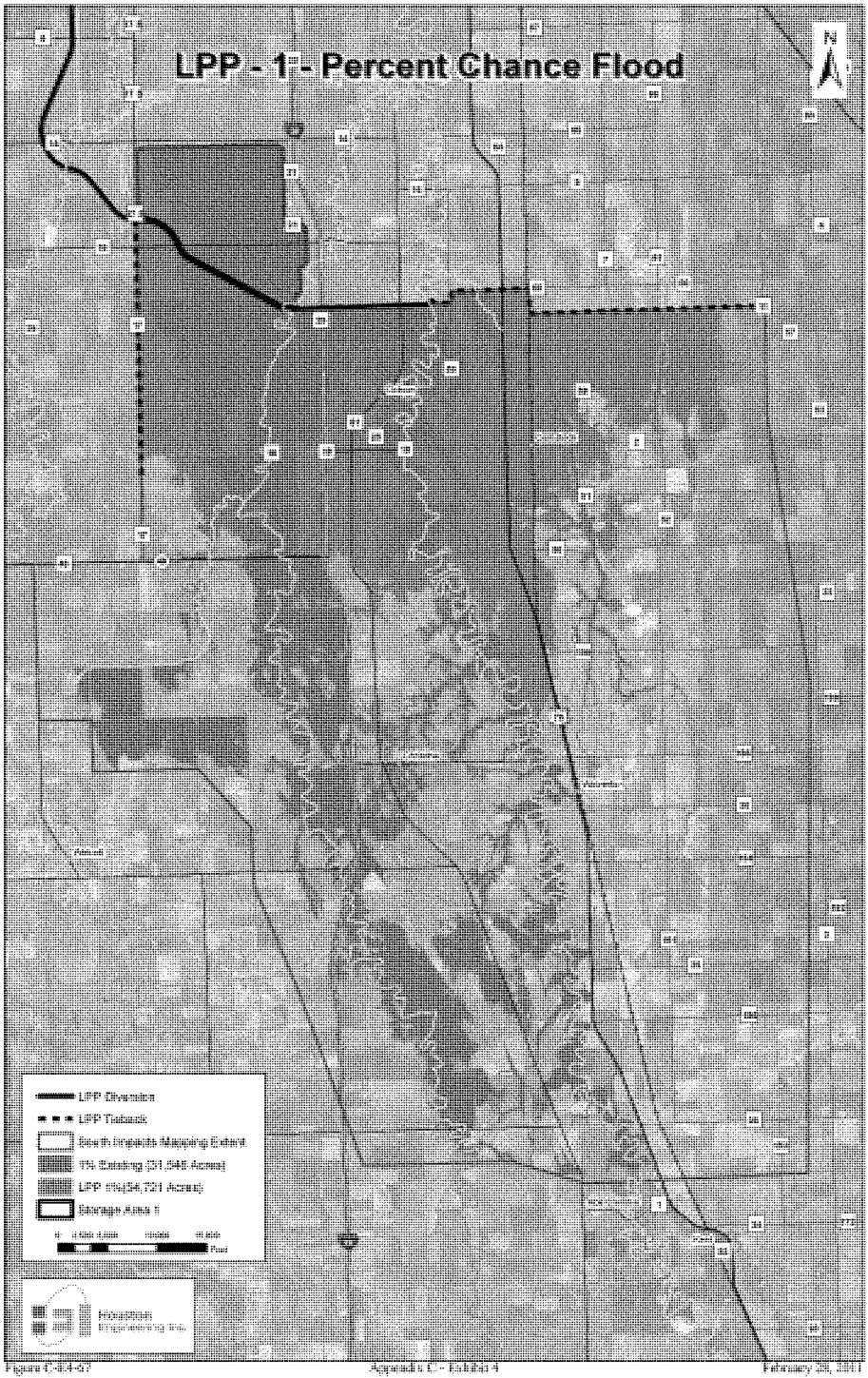
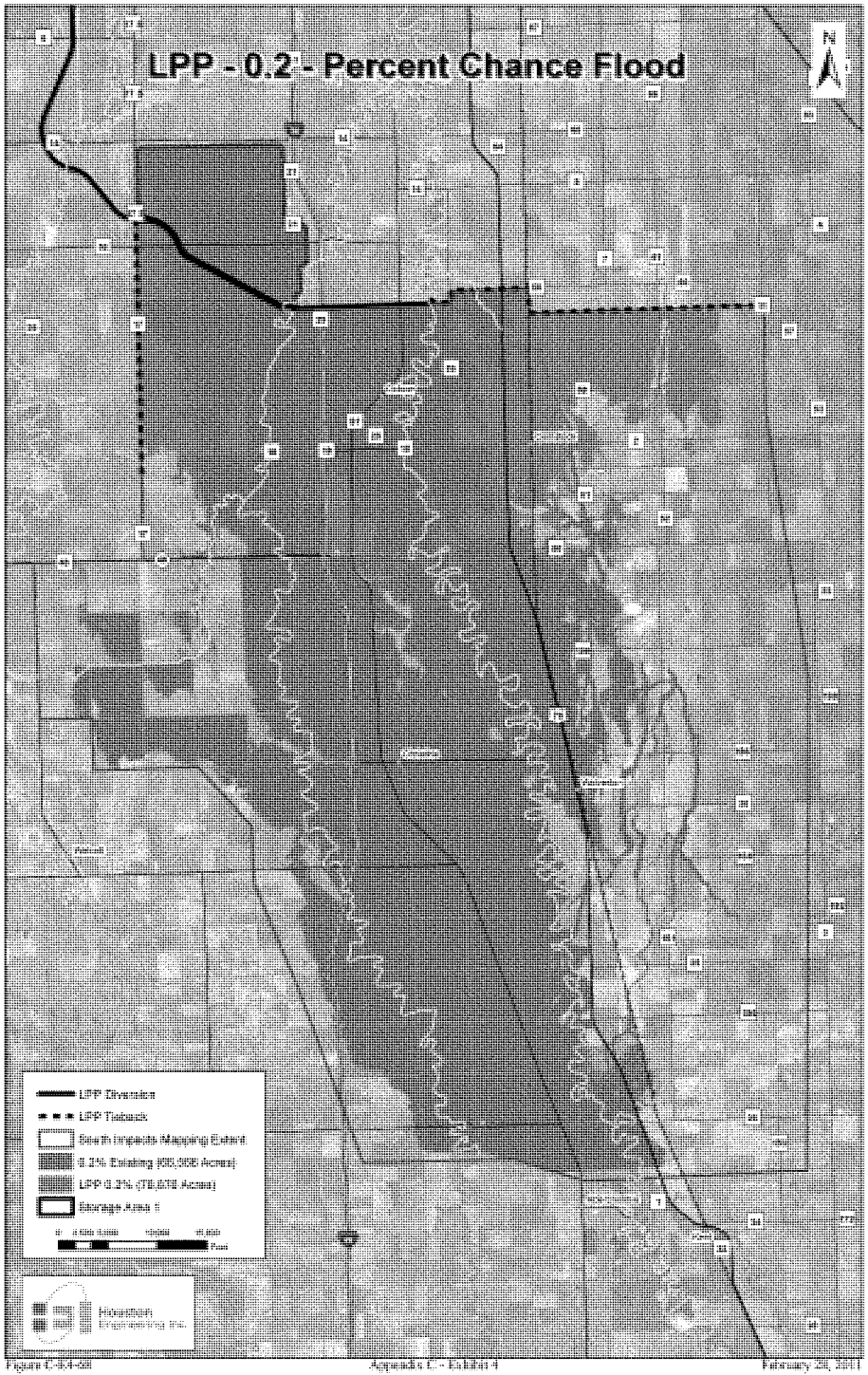
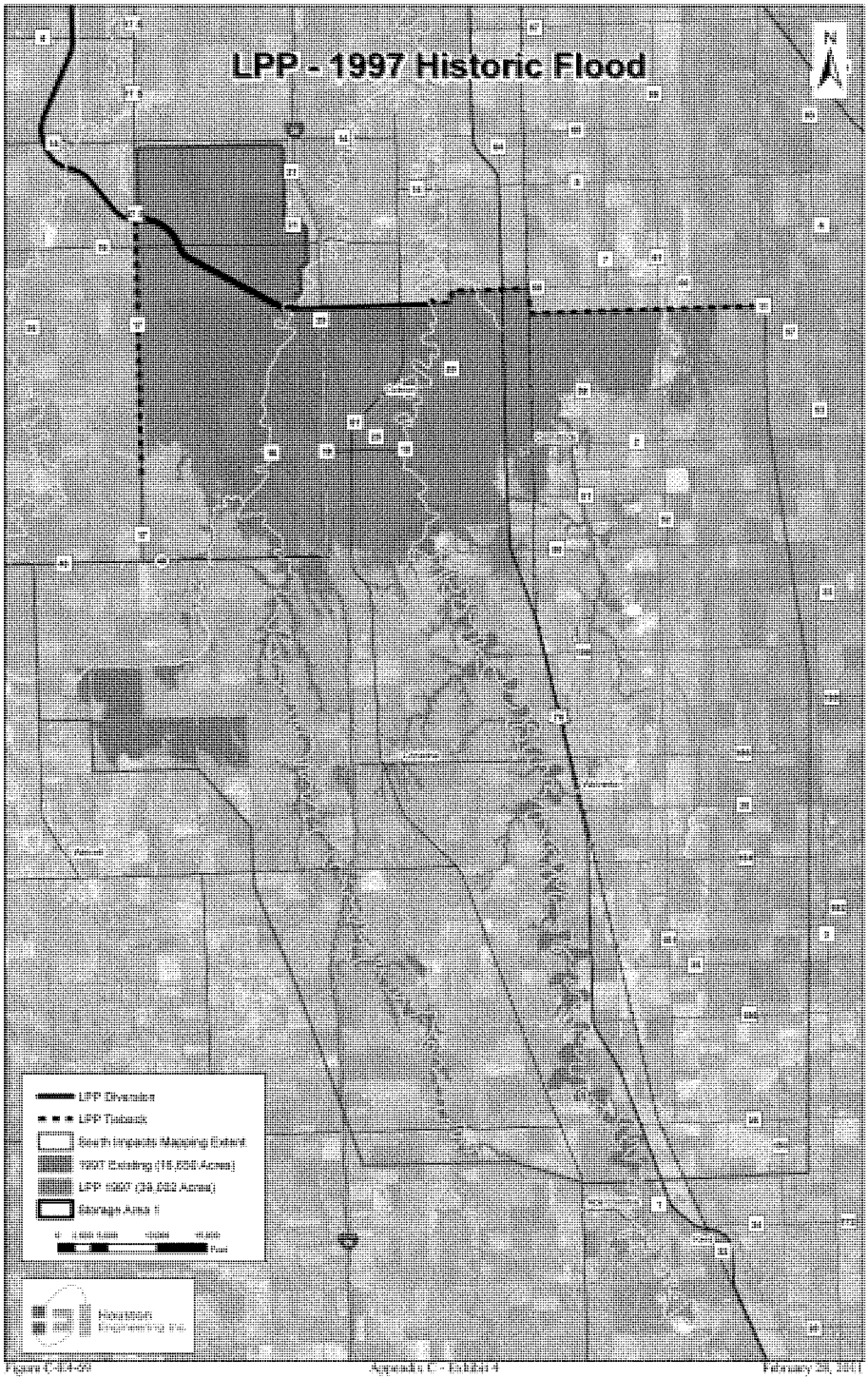
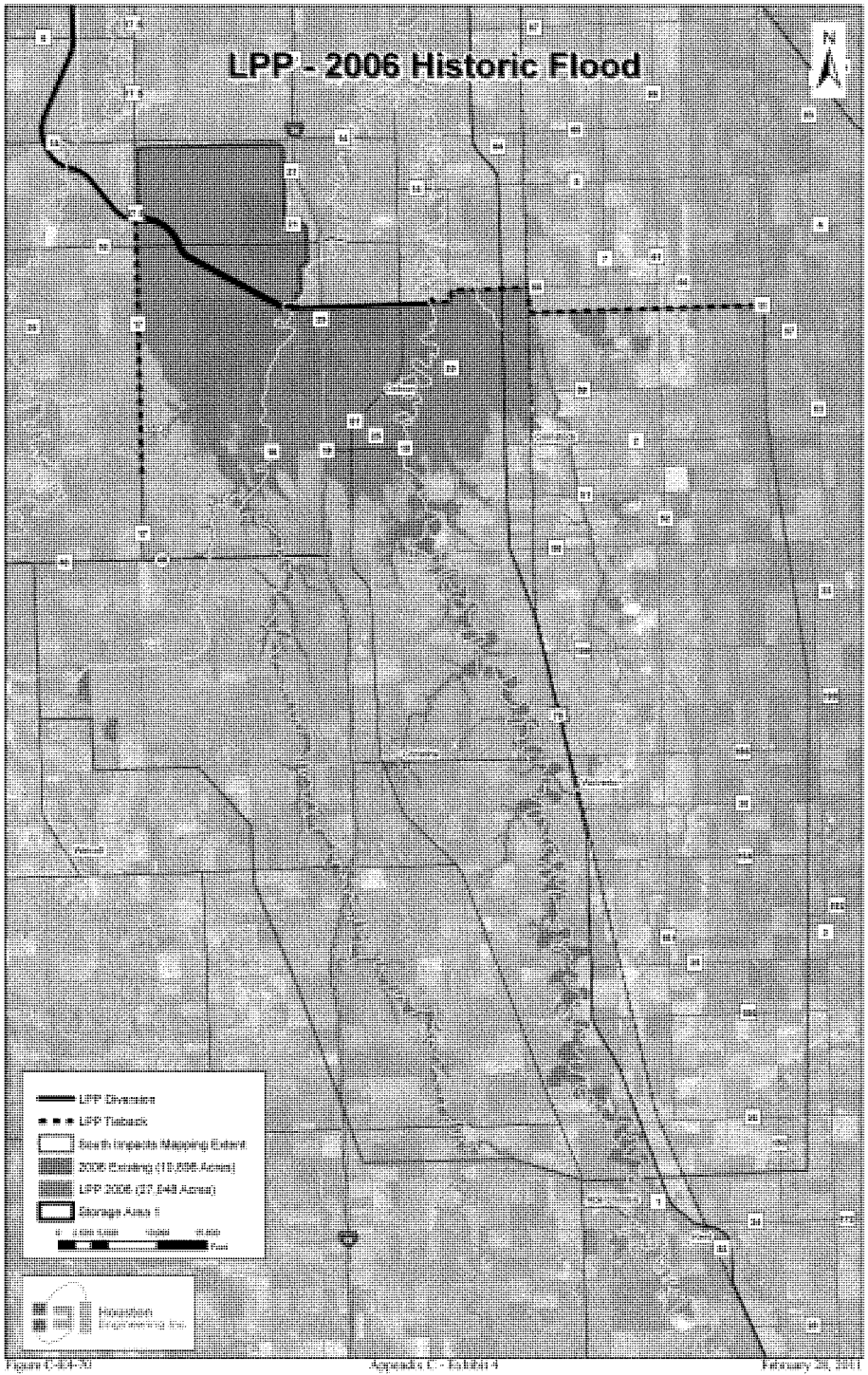
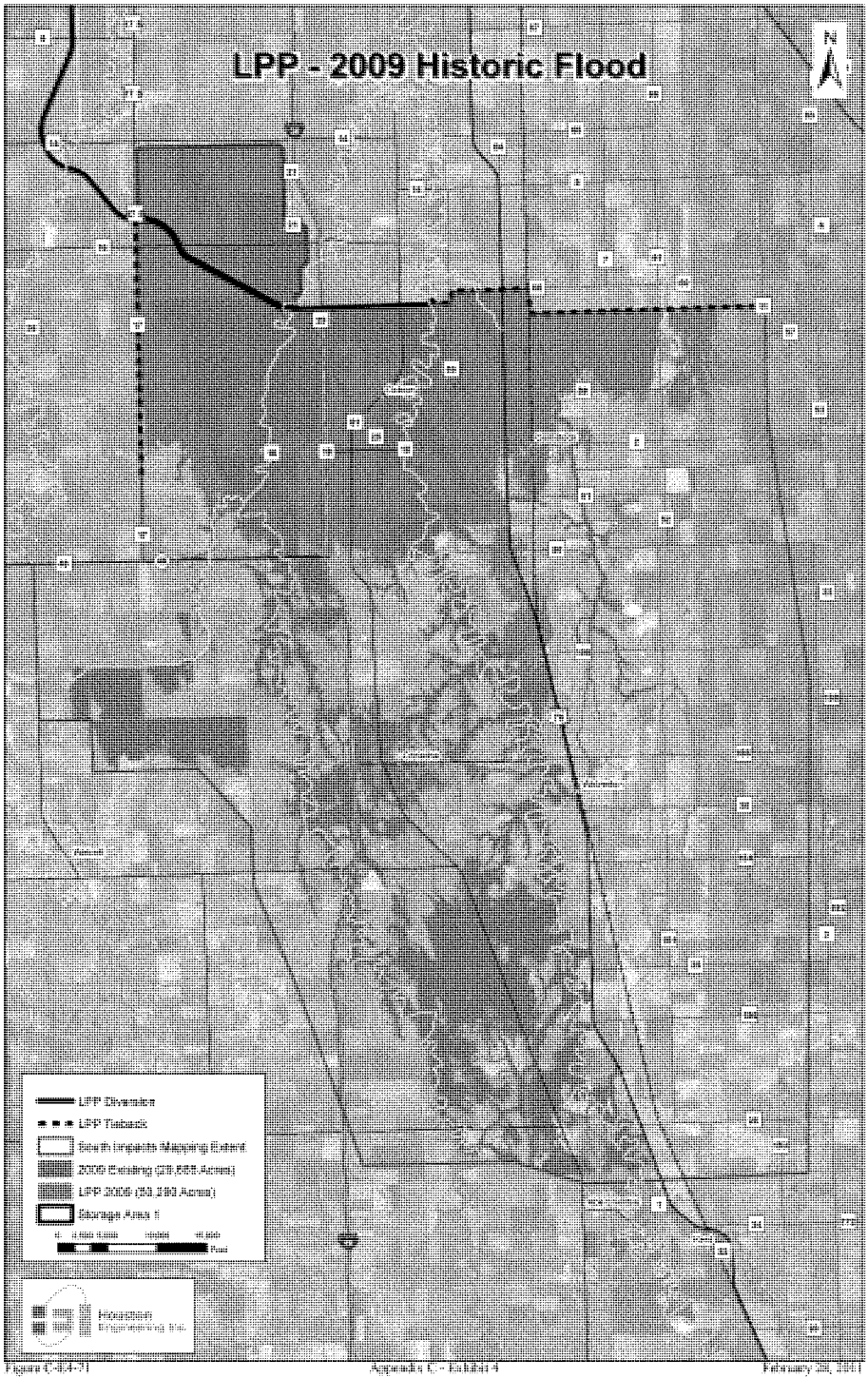


Figure C-8.4-37 Appendix C - Exhibit 4 February 28, 2001


LPP - 0.2 - Percent Chance Flood
N
LPP Diversion
LPP Tieback
Storage Area 1
Halff Associates, Inc.


LPP - 1997 Historic Flood
N
LPP Diversion
LPP Tieback
South Impacts Mapping Extent
1997 Existing (16,656 Acres)
LPP 100Y (39,082 Acres)
Storage Area 1
Houston


LPP - 2006 Historic Flood
N
LPP Diversion
LPP Tieback
2006 Existing (18,696 Acres)
LPP 2006 (27,648 Acres)
Storage Area 1
Houston Engineering Inc.


LPP - 2009 Historic Flood
N
LPP Diversion
LPP Tieback
South Impacts Mapping Extent
2009 Existing (28,685 Acres)
LPP 2009 (53,283 Acres)
Storage Area 1
Houston

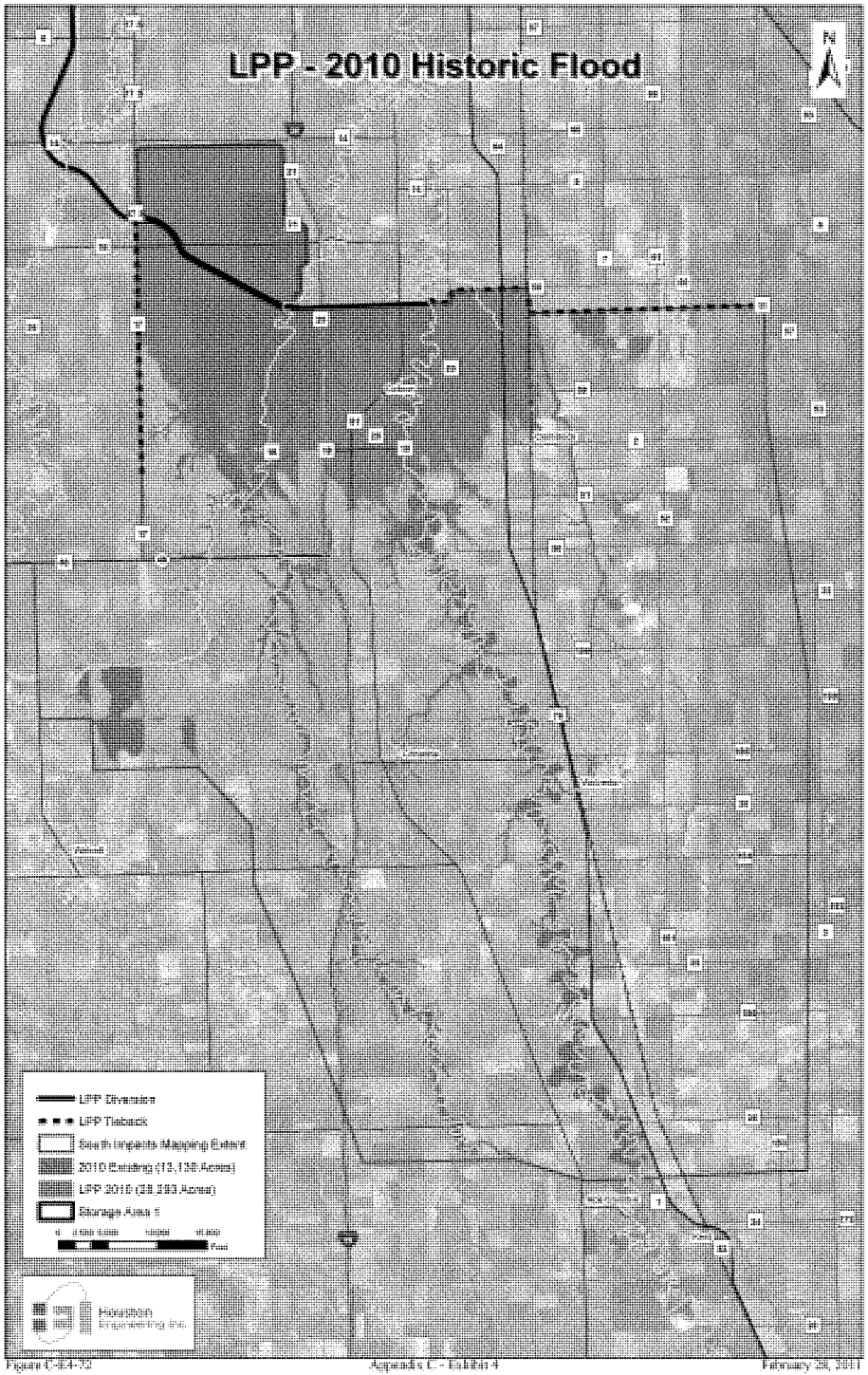


Figure 4-8.4-10 Appendix E – Exhibit 4 February 28, 2011

RED RIVER DIVERSION

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

APPENDIX C – HYDRAULICS WITH–PROJECT CONDITIONS

EXHIBIT 5 – QA/QC COMMENTS AND RESPONSES

Report for the US Army Corps of Engineers, and the cities of Fargo, North Dakota & Moorhead, Minnesota

FINAL: February 28, 2011

moore engineering, inc.
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West Fargo • Fergus Falls

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Memorandum

To: File

From: Stu Dobberpuhl, Mike Opat & Lyndon Pease- Moore Engineering, Inc.

Date: February 24, 2011

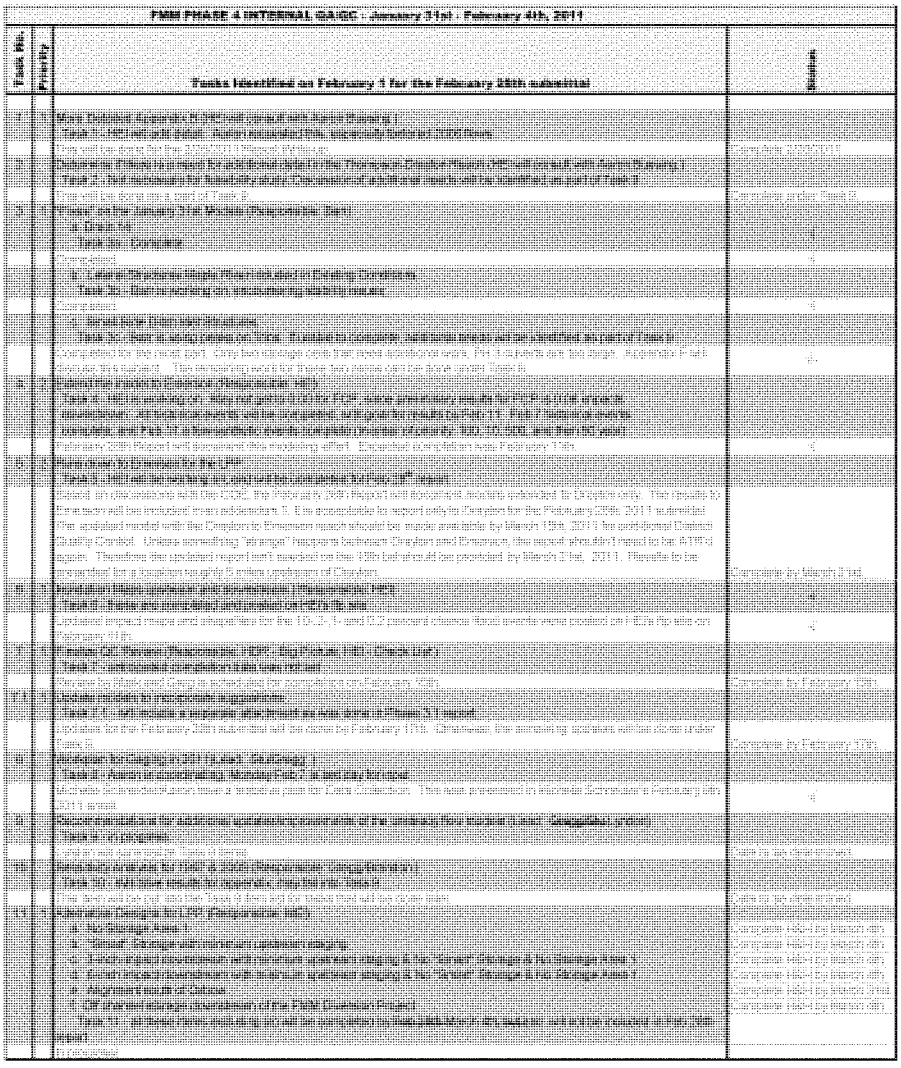
RE: Fargo-Moorhead Metro Feasibility Study- Phase 4

Ref: February 1-2, 2011 QA/QC review of existing conditions and with-project (LPP) models

Introduction:

Following the submittal of the January 31, 2011 report documenting the Phase 4 design of the Fargo-Moorhead Metro Feasibility Study, members of the design team (Moore Engineering, Houston Engineering, Barr Engineering & HDR Engineering) met at Moore Engineering to begin an internal QA/QC review process on the modeling that was completed at that point. With a final report expected to be submitted on February 28, 2011, the goal of the review was to identify any major issues with the models and have them corrected before the final report was released. Through this review, comments on minor issues having no potential impact on the results were also documented. While some of these comments were incorporated into the final models, many of them were deemed to be beyond the feasibility level of the study and many others simply could not be addressed given the timeline for this phase of the study.

The list of comments developed during this review is attached to this memo. Tasks were assigned to members of the design for follow-up on each of the comments. Regular progress meetings were convened to follow up on the status of each task and the attached tables were updated accordingly. Task No. 9 under the February 1st review comments includes a list of items that were identified for future updates and improvements. These tasks were further documented in Appendix C of the February 28, 2011 Phase 4 report.



Task No.	Priority	Tasks Identified on February 1 for the February 28th submittal	Status
1	1	Meet Delaware Approach Plan and consult with Aaron Downey (	
		Task 1 - Meet with Aaron - Action completed this, especially Section 1000 Book	
		Time will be done by Dec 31st, 2011 Wayne Anderson	Done Dec 31st, 2011
2	1	Prepare a Report to meet the requirements of the Delaware Coastal Permit (CP) and consult with Aaron Downey (	
		Task 2 - Not necessary for Submittal since Documentation and Report will be submitted as part of Task 3	
		Time will be done by end of Task 3	Completed as part of Task 3
3	1	Prepare and file January 31st Monitor Progress Report	
		a. Task 3A	
		Task 3A - Complete	
		Completed	
		b. Update Shoreline Study Plan and add in Existing Conditions	
		Task 3b - Start reviewing on shoreline by starting now	
		Completed	
		c. Add in new Data and Analysis	
		Task 3c - Start working please on Data. If unable to complete, comments should still be submitted as part of Task 3	
		Completed for the most part. Only two storage cells that need additional work. PH 1 cells are the large. Appendix F will	
		develop this material. The remaining work for these two areas can be done under Task 5	
4	1	Update the project Schedule (Responsible: MH)	
		Task 4 - MH is working on this and will submit to the PDT some preliminary results for PDT review and discussion. All tasks or events will be completed, with goal for results by Jan 17. Task 7 following events	
		including and Task 10 in the schedule, such as changes in project schedule by Jan 10, 2012, and then to point	
		February 28th Report will incorporate this modeling effort. Presented to and then was February Date	
5	1	Prepare cost for Elements for the CPP	
		Task 5 - MH will be working on this and will be completed by Dec 28th Report	
		Report on the costs are in the CCSC. The February 28th Report will incorporate revisions requested by Delaware only. The results for	
		the report will be included in any submission. It is impossible to report only to Delaware for the February 28th, 2011 submittal.	
		The updated model with the Delaware Breakout report should be made available by March 15th, 2011 for additional Delaware	
		Coastal Control. Unless something "strange" happens between Delaware and Delaware, the report should need to be ATTN'd	
		again. Therefore the updated report will needed no later than be provided by March 31st, 2011. Results to be	
		presented for a maximum of 8 cases (options) of Elevation	Complete by March 31st
6	1	Update the Benefits and Costs (Responsible: MH)	
		Task 6 - These are included and will be updated in Task 5A	
		Updated impact means and adaptations for the 10-25, 1- and 0.2 percent chances flood events were posted on FEMA flip web on	
		February 11th	
7	1	Update QA Review (Responsible: MH - Rod Emslie, MH) - Check List	
		Task 7 - continuous consultation with Rod and staff	
		Plan on to Work and Close to schedule for completion on February 28th	Complete by February 28th
8	1	Update Appendix to incorporate suggestions	
		Task 8 - will include a separate write-up with all the work done in Phase 3 Report	
		Updates for the February 28th submittal will be done by February 17th. Otherwise, the remaining updates will be done under	
		Task 10	Complete by February 17th
9	1	Participate in Charrette (Date: Wednesday)	
		Task 9 - Aaron is coordinating, Monday Feb 7 is date we have	
		Activities to be included were made a matter of point for Dune Construction. This was presented in electronic Screen and in February 9th,	
		2011 email	
10	1	Obtain the costs and the additional updated information of the Delaware Bay Section 1 Level (Responsible: project	
		Task 10 - Engineer	
		Time will be done by Date of Task 10	Date to be determined
11	1	Participate in Public Meeting (Responsible: Craig Mitchell)	
		Task 11 - Will attend, provide the agenda, make the notes, Task 11	
		Time item will be put into the Task 9 item and the Public Hall will be done later	Date to be determined
12	1	Determine Changes to the CPP (Responsible: MH)	
		a. No Change Area	Complete (Mitch) by March 15th
		b. "Minimal" Change area with minor modifications	Complete (Mitch) by March 15th
		c. Major change recommended with minor modifications (Maybe in the "Center" Change to No Change Area)	Complete (Mitch) by March 15th
		d. Major change recommended with minor modifications (Maybe in the "Center" Change to No Change Area)	Complete (Mitch) by March 15th
		e. Major change Area	Complete (Mitch) by March 15th
		f. Off the Island change (Development of the Field Elevation Project)	
		Task 12 - All these items should be put on the map and be completed by March 15th. Mitch will work with Rod Emslie on this	
		report	
		Completed	

FMM PHASE 4 INTERNAL QA/QC - January 31st - February 4th, 2011			
Task No.	Priority	Tasks Identified on Febuary 2nd for the February 28th submittal	Status
1	1	Volume check for storage areas. Maps will be generated for existing and proposed conditions. (Responsible: MEI) Task 1 - will need to verify impact of releasing SA volume from Fargo area. Will see if this affects volume for existing conditions.	
		Completed for previous version of the model. Need to verify that the February 15th models do not have any issues at Georgetown and other locations downstream.	√
2	1	Check culverts on storage areas. (Responsible: MEI Barr) Task 2 - see Task 1 above	
		See Task 1. The remaining work can be done under Task 9.	√
3	1	Sensitivity on Diversion Channel /Staging Interaction. (Responsible: Barr) Task 3 - complete, will make changes to LPP model once Task's 1 and 2 are complete	
		In progress.	Complete by February 15th
4		Diversion Channel Weir - Invert Elevation / Gates ? (Responsible: MEI) Task 4 - raising invert weir to 914.25 had no impact downstream; these findings will need to be documented. Aaron sees advantage of raising weir.	
		Analysis is done. Need to document in Appendix C.	√
5	1	RRN Gates Open at the start of the simulation. (Responsible: Barr) Task 5 - this process has not yet begun	
		This item will be put into the Task 9 item list for tasks that will be done later.	Date to be determined.
6	1	Verify Sheyenne River Pass Through flow (Responsible: Barr) a. "Turn Off" Horace to West Fargo Diversion Task 6a - will wait until Task's 1 and 2 are complete	
		Completed.	√
7		Sensitivity of 2009 Flood Event with current Balanced Hydrographs (Responsible: HEI) Task 7 - this is referred to with Day 1 (Feb 1) Task 10	
		This item will be put into the Task 9 item list for tasks that will be done later.	Date to be determined.
8		Geometry differences between historic and synthetic events - Maple/Sheyenne Rivers. (Responsible: MEI) Task 8 - Greg and Lyndon will look into differences	
		Completed.	√
9	1	Post Processing to completion (Responsible: HEI) Task 9 - working on, Aaron will follow up with HEC	
		Post Processing is an issue yet with RAS 4.1. Models can be post processed with RAS 4.0 or 4.2 if RAS 4.1 doesn't work.	√

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Memorandum

To: Greg Thompson, PE- Houston Engineering

From: Stu Dobberpuhl, Mike Opat & Lyndon Pease- Moore Engineering, Inc.

Date: February 24, 2011

RE: Fargo-Moorhead Metro Feasibility Study- Phase 4

Ref: February 20, 2011 QA/QC review of with-project (LPP) models by Houston Engineering

Introduction:

Moore Engineering (MEI) has reviewed the comments developed by Houston Engineering (HEI) after reviewing the Phase 4 unsteady flow models for the with-project (LPP) conditions. The checklist used by HEI to conduct the review is attached this memo. MEI's responses to these comments are included in red text within the checklist, or by notes added to the column on the right side of the list. Many of the comments were noted, but did not require a response. A large number of the comments were identified as items that should be addressed with future improvements and updates to the models. The timeline associated with this phase of the study would not allow for allow for all of these comments to be addressed and some were not deemed to be applicable to the feasibility level of the study. Any comments that had the potential to measurably impact the results were addressed and incorporated into the final Phase 4 models.

HEC-RAS Modeling Check List

Project Number: 6059-025	
Project Name/Description: Fargo Moorhead Metropolitan Feasibility Study	
Model Name: 100yr.prj	
Model Developed By: Barr Engineering	Model Checked By: Houston Engineering Inc.
Model Date: 2/10/2011	Model Check Date: 2/20/2011
Project Plan: 100yr_20110208_Cstr	
Project Geometry: ND East_20110208_Cstr	
Project Flow: NDE 100yr 20110208_Bstrg	

√ Indicates everything checks ok and no additional action is necessary

#		General Information/Checks	Comments	Note
1	√	What version of HEC-RAS was used?	4.1	NC
2		What datum is the survey data and the HEC-RAS model in (NGVD29 or NAVD88)?	Not documented in the model Response: Model is in NAVD 88	(1)
3		Is the datum noted in the "description" box of the main project screen?	No	(1)
4		Is the model georefenced and if so what is the horizontal projection?	Yes it is geo-referenced. Projection is not indicated within the model.	(1)
5		Is the stationing reference noted in the "description" box of the main project screen?	No	(1)
6		Does the stationing in the HEC-RAS model match the map	Not Checked	NC
7		Is the model in the same datum as the mapping? (mapping should be NAVD88)	Not Checked - No Mapping	NC
8		Are photos attached to cross-sections and structures (i.e. photos facing downstream)?	No	NC
9		Are there any modeling or mapping anomalies that should be pointed out to a reviewer?	Overlapping cross sections may cause mapping issues. Cross sections will need to be adjusted during mapping.	(1)
10		Are the "Plan" files clearly labeled/named?	No. Unsure of naming convention. Response: Table provided in Appendix C describes each plan	(1)
11		Was GeoRAS used to create your geometry schematic?	Yes	NC
12		Were cross-sections created using both survey data and digital topography (i.e. LiDAR, 2ft topo, etc.)?	Yes. Parts of the diversion cross sections were from lidar, it is assumed that other parts were designed with channel modification techniques.	NC
13		Is the unsteady model computation time appropriate?	Yes, Warm up time step is 30 seconds, for 50 time steps, then the time step is 5 minutes. Existing conditions is set at 5 min. Iterations = 40 on some, 20 on others.	NC
14		If there are options checked on the plan file do these make sense?	Yes.	NC
15		If the tolerances have been changed are these appropriate?	Yes. Tolerances are 0.03' for XS and 0.1' for storage areas. Same as Ex Cond.	NC

#		Hydrology Checks	Comments	
16		Has the hydrology been reviewed internally?	Yes, Hydrology was compared to existing conditions for the historic events	NC
17		Has the hydrology been reviewed and approved?	n/a	NC
18		Is a write-up of the hydrology included?	n/a Note: See Appendix A	NC
19		Have the reach boundary conditions been reviewed internally?	n/a	NC
20		Is the boundary condition a "Known W.S."?	n/a	NC
21		Do the 10-, 50-, 100-, and 500-yr flows entered in RAS match the flow hydrographs calculated in your model/hydrologic calculations? Are they labeled in HEC-RAS?	Not Checked	NC
22		Are the 10-, 50-, 100-, and 500-yr flow hydrographs labeled as such in the HEC-RAS flow editor?	Yes, Unsteady model references .dss files with corresponding event frequency.	NC

23		Are there any negative flow hydrographs that may indicate correction hydrographs within HEC-RAS?	Yes, to counter balance inflows required for stability in diversion channel.	(1)
24		Do the output hydrographs from HEC-RAS have any abrupt changes?	Yes, ND DIV US between Red and WRR. However, change in flow direction seems reasonable.	(1)
25		Are storage areas adjacent to river reach cascading flow ahead of the river?	Not Checked	NC
26		Do the profiles through the entire hydrograph make sense?	15MAR2006 2400 to 17MAR2006 2400 profile does not display correctly. Likely due to inverse slope in the upper end of ND DIV, ND DIV EAST reach. Response: This is caused by the presence of a pilot channel included in the model for stability. This is not an issue.	NC
27		Do any river reaches have negative flow?	Yes. Flows on ND DIV, ND DIV US	NC

#		Hydraulics Checks	Comments	
28		Is a write-up of the hydraulics included?	No. Response: See Appendix B and Appendix C	
29		Do discharge values match hydrologic analysis results?	Not Checked	NC
30		Do the locations where discharges change in the model agree with locations on drainage area map?	Yes	NC
31		Do the discharges change along the stream at the appropriate location?	Yes	NC
32		Have the "Summary of Errors, Warnings, and Notes" been reviewed and addressed for each profile and floodway?	It was reviewed. Many "Divided Flow", "Conveyance Ratio" warnings. Divided flow occurs where conveyance is allowed outside of diversion. Levees are not set at top of diversion bank. Example: ND DIV, ND DIV EAST, XS 119218.	(1)
33		Has Check-RAS been run, and each issue addressed?	Not Checked	NC
34		For any remaining Check-RAS issues, has the Check-RAS output been annotated?	Not Checked	NC
35		What is the boundary conditions for the starting water surface elevation?	n/a	NC
36		Have you reviewed the HEC-RAS profiles?	yes, 100-year	NC
37		Do profiles cross for different return periods?	Profiles do not plot correctly for the first few days. It is likely a plotting issue since the Diversion profile is backwards between Sheyenne River and Wild Rice River. Response: This is caused by the presence of a pilot channel included in the model for stability. This is not an issue.	NC
38		Have you checked the water surface elevations to make sure there is no negative water surface slope?	Negative slopes occur in ND DIV, ND DIV EAST, Wild Rice to Sheyenne Area where the profile is backwards.	(1)
39		Have you used the 3D viewer (X-Y-Z plot) to review the 100- and 500-yr floodplains and floodway?	Not Checked	NC
40		Are the WSELs on tributaries lower than the WSELs on the main channel at or near the confluence?	no.	NC
41		Are there drawdowns on the profiles?	Not Checked	NC
42		What is the source for your Manning's N values?	Not documented in model	(1)
43		Has a summary of the range of Manning's N values been created?	Not documented in model	(1)
44		Do the Manning's N values seem reasonable?	Yes. Is there a reason why there is 0.03 and 0.04? Also, there are many horizontally varying breaks where n values remain the same. It would be cleaner if this was fixed.	(1)
45		Do the expansion and contraction coefficients seem reasonable?	Not set for Unsteady flow	NC
46		Are the cross sections lengths appropriate for the model reach?	Length of diversion cross sections are typically 3000 feet long. Will the spoil piles affect conveyance?	(1)
47		Are there interpolated cross sections?	Yes.	NC

48		Have Cross-Sections been placed at least every 1000 feet?	No, there are 72 cross sections in ND DIV, ND DIV East that are greater than 1000' apart Response: With the relatively straight alignment and constant slope, this spacing was deemed to acceptable.	(1)
49		Are the WSELs for every cross section lower than the cross section end points?	No. The cross sections adequately provide the modeled conveyance.	NC
50		Are the cross sections perpendicular to flow?	Yes	NC
51		Do the cross sections locations model constrictions and expansions in the floodplain?	Yes	NC
52		Do the cross section locations model structures in floodplains	Yes	NC
53		Do cross sections overlap storage areas?	Yes, however levees are placed so conveyance does not seem to be double counted.	NC
54		Do the locations of bank stations make sense?	Yes	NC
55		At road crossings, are there actual road names in the "description" box of your structure data?	No Road crossings modeled on diversion	(1)
56		Are all road crossings being modeled?	No. It appears as though bridges are not modeled.	(1)
57		Are you using the correct "bridge modeling approach" for each bridge?	n/a	NC
58		Is the top of bridge roadway being properly defined?	n/a	NC
59		Is the correct top-of-road elevation, low-chord elevation, and deck width being modeled?	n/a	NC
60		Are the proper coefficients being used for weirs, pressure, and/or culvert flows?	ND DIV East Inline structures have coefficient of 2.6, consistent with remainder of model. ND DIV East Lateral Structures have coefficients of 2.6, remainder of 1.	(1)
61		Are levees used, and if so, is a description included?	Levees are used. I haven't seen a description. Most are likely constructed spoil banks. Response: Levees are in fact used to reflect spoil banks. Levees were placed in model at inside toe of the spoil banks.	(1)
62		Are ineffective flow areas modeled correctly?	Not Checked	NC
63		Have ineffective flow area been placed at correct stations/elevations?	Not Checked	NC
64		Do the ineffective flow areas contain any drastic changes in width?	Not Checked	NC
65		Are the ineffective flow areas being modeled correctly at bridges that are not overtopped?	Not Checked	NC
66		Is there ineffective flow areas where high ground prevents flow?	Not Checked	NC
67		Is there appropriate control for split flow?	Not Checked	NC
68		Are split flows at lateral overflow weirs set up correctly?	Not Checked	NC
69		Are all lateral flow structures connected to another reach or storage area?	Yes	NC
70		Are all storage areas connected to another storage area or reach via lateral structure or storage area connection?	Yes	NC

Additional Notes:

Numerous interpolated cross sections on diversion

Junction and cross sections do not line up at Red River/ND DIV US. Cross sections in downstream reach are spatially placed upstream of structure.

Wild Rice River ND LS 75000 conveys water from WRR to Storage Area 1. This essentially bypasses the inline structure on the diversion. Does the inline structure have the additional capacity to convey the water from WRR to storage area? Or is there enough room to construct all of the weirs that are modeled in that location? This is likely a final design consideration.

Wild Rice cross sections downstream of diversion do not follow channel centerline.

Response Notes:

Some responses were provided in 'red' text above.

Comments noted with "NC" were deemed not to be comments or require no response

Comments noted with (1) will be addressed during future improvements and modifications to the models, after the February 28, 2011 submittal.

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Memorandum

To: Mark Forest- HDR

From: Stu Dobberpuhl, Mike Opat & Lyndon Pease- Moore Engineering, Inc.

Date: February 26, 2011

RE: Fargo-Moorhead Metro Feasibility Study- Phase 4

Ref: February 25, 2011 QA/QC review of with-project (LPP) models by Mark Forest- HDR Engineering

Introduction:

Moore Engineering (MEI) has reviewed the comments developed by HDR Engineering (HDR) after reviewing the Phase 4 unsteady flow models for the with-project (LPP) conditions. HDR completed a QA/QC review if the 2009 calibration unsteady flow model, the synthetic event models, and the LPP alternative models and provided one set of combined models. The comments for the calibration model and the synthetic event models are addressed by Houston Engineering in Appendix B of the February 28, 2011 Phase 4 report. The comments on the LPP models were extracted from the combined comment document and included in this memo. The LPP models were developed by Moore Engineering (MEI) and Barr Engineering (Barr) and the responses to HDR's comments were developed by MEI. The responses are noted in red text following each comment.

With the timelines involved with this phase of the study, most of the submitted comments could not be fully addressed in the final models. None the comments appeared to have the potential to significantly affect the results of the final models included in the February 28, 2011 submittal. The most important finding from HDR's review is that the current LPP models are suitable for the feasibility level of this study.

HDR Comments- Proposed Condition for LPP

42. Many of the comments above pertain to the proposed condition model also. Noted.

43. The 100-year model goes unstable near the very end of the simulation. This issue went through extensive debugging, including involvement with programmers at the Corps of Engineers Hydraulic Engineering Center, without resolution. This issue has been identified in Appendix C as a task to be addressed with future improvements and updates to the models.

44. I am assuming that independent verification has been performed of the data inputs to assure that the data represented in the model geometry matches the preliminary plans. I did not perform these checks. Barr Engineering coordinated the designs for the hydraulic structures included in the models to be consistent with what is included on the plans for those structures. The project layout plans developed by Houston were developed from the geometry included in the final models.

45. The design concept requires human intervention based on operational criteria for downstream flows and predicted inflows. Have alternatives been explored that do not require gates? Assuming that other outlet configurations have been explored and found to be deficient, how much risk is associated with the range of potential system flow contributions and variability in how the operating criteria might be employed? The gated structure design selected over other alternatives during earlier phases of the feasibility study. The Operations Plan that will be developed for this project is expected to address the operational criteria for the entire system.

46. Area 1 has a bottom elevation that is 25 to 30 feet higher than the diversion channel. Would additional storage benefit be obtained from lowering the bottom of this storage area to gain additional storage volume? While the elevation differential does not appear to be as large as noted (14-17 feet), the potential for additional storage is noted for further study.

47. Minor differences in flow volume are noted between the existing and proposed condition in the downstream reaches that appear to be due to changes in residual storage that occurs in many of the storage areas. Elimination of overflow into these areas as a result of project improvements would result in reductions to these residual volumes that remain in these storage areas after passage of the peak flow. This would be an anticipated result. How much potential residual storage occurs after an event may be difficult to quantify without more drainage structure data. The issue is addressed in Appendix C. Elimination of the retained storage has also been identified for future resolution.

48. The flow conditions at the diversion structures (Figure 25) are complex and vary with stage. This area would likely benefit from the future two dimensional capabilities of HEC-RAS. Agreed.

49. A very large flow value is reported for several of the storage area connections such as DRASC33 (see Figure 26) and many of the OSLSC connections (Figure 27) that should be checked. This reach has specifically been identified for the addition of greater detail and refinement with future updates and improvements to the models (see Appendix C).

50. Attached are some locations in the post-processing of the results that appear to be suspect. In Figure 28 and 30, the abrupt edges either mean that you have a roadway or other "levee-like" structure that is impounding the water or there is overflow that is not being accounted for. If it is an artificial impoundment, how will we ultimately need to treat an embankment of unknown integrity? This reach has specifically been identified for the addition of greater detail and refinement with future updates and improvements to the models (see Appendix C).

51. In other locations, the mapping shows areas of isolated ponding with no apparent way for water to get there (Figure 29). Those locations should be verified. This appears to be an issue with the software used for the mapping and there should not be any issue with actual modeling results.

52. For the post-project condition upstream of the diversion (Figure 31), we will not be able to consider water contained by "levee-like" structures that are not compliant with 44 CFR 65.10 if we are increasing the water surface against those embankments. If there is a need to constrain the water at those locations, the project costs would need to include constructed levees at these locations that will meet FEMA's minimum standards. Noted.

RED RIVER DIVERSION

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

APPENDIX D – CIVIL AND GEOTECHNICAL DESIGN

Report for the US Army Corps of Engineers, and the cities of Fargo, North Dakota & Moorhead, Minnesota

REVISED: April 11, 2011

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D1.0 PROJECT LAYOUT AND TOPOGRAPHY

D1.1 TOPOGRAPHY

The Fargo-Moorhead area is characterized by relatively flat topography deposited by glacial Lake Agassiz. The flat topography results in the Red River having very little slope, generally less than one half of a foot per mile from south to north. While the flat terrain contributes to the potential for widespread flooding, it also allows for the potential to construct diversion channels around the community either to the east or to the west without creating extremely deep excavations. A more detailed discussion of the alignments follows.

D1.2 CONSIDERATIONS IN ALIGNMENT SELECTION

The general concepts for the alignments were provided by the US Army Corps of Engineers (Corps of Engineers). Modifications to those concepts, done during previous phases of feasibility, were based on a number of considerations. The main considerations used in the selection of alignments follow:

1. Alignments were shortened where reasonable to reduce footprint and cost.
2. Alignments were moved to relatively lower ground to minimize the excavation required.
3. Alignments cross roads and railroads at or as close to 90 degrees as possible to minimize bridge lengths.
4. Alignment avoids populated areas to minimize buyouts where possible.
5. North Dakota alignment crosses rivers at or as close to 90 degrees as possible to minimize hydraulic structure size and cost.
6. Alignments were located in areas that minimize geotechnical concerns related to know aquifers.

D1.3 MINNESOTA SHORT ALIGNMENT (FCP)

The Minnesota Short Alignment serves as the FCP for Phase 4 of this study. The FCP design was not changed from the Phase 3.1 design. However, the FCP was analyzed as part of Phase 4 in order to determine the project impacts based on the updated unsteady flow models. For background information and documentation on the development of the Minnesota Short Alignment, refer to the Attachment C2 in Appendix C.

D1.4 NORTH DAKOTA DIVERSION ALIGNMENT (LPP)

The North Dakota Diversion Alignment (LPP), shown in Figure D1, starts approximately four miles south of the confluence of the Red and Wild Rice Rivers and extends west and north around the Cities of Horace, Fargo, West Fargo, and Harwood and ultimately re-enters the Red River north of the confluence of the Red and Sheyenne Rivers near the City of Georgetown, MN. The alignment is approximately 36 miles long. The background information on the development and previous changes to the alignment can be found in the reports for first three phases of this study. While the alignment has remained largely unchanged from its initial layout, some modifications were made to the North Dakota Diversion (LPP) alignment during Phase 4, although they do not have a sizeable impact on the project. The north end of the alignment was adjusted near

Argusville to avoid interference with Cass County Drain 13. It was determined that Drain 13 was already an efficiently functioning legal drain so it could be utilized for its capacity separately from the diversion. The alignment was shifted to the south and east to accommodate this change. In other areas, minor changes were also incorporated where existing homes and buildings could be reasonably avoided. The LPP diversion channel will incorporate the existing Horace Diversion channel, so the alignment was adjusted so that the east side of the LPP diversion matched the existing Horace Diversion channel.

The Phase 4 LPP design includes storage and staging on the upstream side of the project. This storage reduces the discharge that must pass through the diversion to the downstream end of the project. Thus, the size of diversion channel was reduced to handle the smaller discharges associated with the Phase 4 design. The configuration of the channel cross section was determined through a combination of the hydraulic capacity, geotechnical constraints, and minimum depth constraints related to the tributary hydraulic structures, specifically the Sheyenne and Maple River structures.

A combination of control structures on the Red and Wild Rice Rivers at the south end of the project, along with a weir on the diversion channel, control the flow split between the Red and Wild Rice River channels and the diversion channel and produce the required staging. Additionally, the alignment crosses several rivers, including the Sheyenne, Maple, Lower Rush, and Rush Rivers. Aqueducts are necessary at the point where the diversion channel crosses the Sheyenne and Maple Rivers. The purpose of these aqueducts is to allow a minimum of a 50-percent chance event to continue down the various rivers while diverting excess water during flood events to the diversion channel. The result of this is added flood protection along all of the affected rivers. The Rush and Lower Rush Rivers, which currently consist of constructed trapezoidal channels, would be allowed to flow into the diversion channel resulting in cutting off the downstream portion of these rivers. The tie-back levee associated with this alternative extends east from the Red River control structure to high ground on the Minnesota side. Additionally, tie-back levees on the North Dakota side contain the flows within the designated staging and storage areas. Further discussion of these structures is included in Appendix F.

D1.5 PROJECT LAYOUT

The North Dakota Diversion (LPP) layout for Phase 4 is depicted in the drawings attached to this appendix (Appendix D). For the FCP project layout, refer to Appendix D.

D2.0 UTILITIES

As would be expected with any project of this scale, each of the proposed diversion alternatives impact existing utilities. The utilities impacted by the diversion alignments include electric, natural gas, petroleum transmission, water supply, wastewater transmission, and various communication utilities. An inventory of existing utilities was

obtained from the various providers. The locations of these utilities are included on the project drawings attached to this appendix.

D3.0 OTHER LOCAL IMPACTED STRUCTURES AND FEATURES

D3.1 IMPACTS ON STRUCTURES AND PROPERTY

While efforts were made to minimize impacts to existing homes and other structures, each of the diversion alternative alignments studied impacts existing structures. Any realistic diversion alignments would require acquisition and relocation. The main feature impacted by a diversion alternative besides existing structures is agricultural land. As the proposed diversion alternatives generally avoid the developed metropolitan area, agricultural land would be removed from production to accommodate a diversion.

D3.2 TRIBUTARY AND DRAINAGE INFRASTRUCTURE IMPACTS

As discussed in the description of the North Dakota East Alignment, a North Dakota diversion alternative would impact several other regional rivers, including the Sheyenne, Maple, and Rush and Lower Rush Rivers. Due to the proximity of these rivers to the Fargo Moorhead Metro area, any North Dakota diversion alignment would cross these rivers. The alignment chosen cross each of these rivers a single time. Generally, the diversion alignment crosses the Sheyenne River near Horace and continues northward paralleling the Sheyenne River. The design of these river crossings is discussed in detail in Appendices C and F.

All of the diversion alternatives studied also impact existing local drainage facilities. On both sides of the Red River, a significant drainage system is maintained throughout the region. Impacts to the existing system, generally consisting of open ditches, should be minimal. Existing drains would simply be allowed to flow into the diversion channel rather than continuing into what would be the protected area. A control structure for each significant drainage area intersected by the diversion alignment would be included along with additional collector drainage channels in locations where existing localized drainage is blocked. A more detailed discussion of the side ditch inlets is found in Appendices C and F.

D3.3 TRANSPORTATION INFRASTRUCTURE IMPACTS

The construction of the proposed North Dakota Diversion (LPP) and Minnesota Diversion (FCP) channels will require the construction of bridges at major roadways and railroads. For the LPP, 20 highway and 4 railroad bridges will be constructed, while for the FCP, 19 highway and 4 railroad bridges will be constructed. Design and cost information for the bridge structures is discussed in Appendix E – Bridge Structures. The major impacts as a result of the construction of these bridge structures will involve temporary closures or detours during construction activities.

As part of the Phase 4 feasibility study, upstream staging and off-channel storage features (Storage Area 1) have been added to the project design for the North Dakota Diversion (LPP). The design of these features is discussed in detail in Appendices C and F. The addition of upstream staging and Storage Area 1 (SA1) will cause impacts to transportation routes upstream from the diversion channel and in SA1. Major transportation routes that will be impacted include Interstate 29; U.S. Highway 75; Cass County Highways 16, 18, 21, 25, and 81; Clay County Highways 2, 58, and 59; the Burlington Northern and Santa Fe (BNSF) railroad grade from Breckenridge, MN to Moorhead, MN; and several township roads. It should be noted that several of these transportation routes are impacted and in many cases inundated by floodwaters during existing conditions.

It is anticipated that a majority of the structures in the staging area and SA1 will be purchased and removed as part of the project. Therefore, only critical transportation routes, including Interstate 29, U.S. Highway 75, and the BNSF railway line were included as grade raises through the staging area. For cost estimating purposes, it was assumed the driving lanes of the roads would be raised to the 1-percent chance flood staged level. Similarly, the BNSF railroad grade was elevated above the 1-percent chance flood staged level. Interstate 29 interchange 50 at Cass County Highway 18 will also need to be reconstructed to accommodate the road raise. Grade raises were also included for roads crossing the levees for SA1 at Cass County Highways 16 and 21 along with for all roads crossing the tie-back levees.

Impacts to downstream transportation infrastructure will be minor. For the LPP, the upstream staging and SA1 will mitigate downstream stage increases, however, the duration of flooding for transportation infrastructure that currently floods will be extended. For the FCP, the impacts to stage may increase the frequency of flooding for some transportation infrastructure in areas where downstream impacts are experienced. Information concerning impacts to stage as a result of the LPP and FCP diversion alternatives is presented in Appendix C.

D4.0 GEOTECHNICAL CONSIDERATIONS

With the reduction in the required capacity of the channel due to the upstream staging component, the bottom of the diversion channel was raised from the Phase 3.1 design. Along with this, the raising of the channel bottom also limited the amount of excavation that will be required within the Brenna formation, which will have an impact on the constructability and cost of the project.

The basic cross section geometry for Phase 4 was determined based on hydraulic capacity and then modified based on geotechnical analysis at various reaches along the diversion. A channel with a bottom width of 250 feet was determined to be the appropriate design for Phase 4, based on the following criteria: (1) allowed design to meet goal of zero impacts downstream; (2) allowed design to meet targeted 1-percent chance profile in the

diversion channel; (3) did not increase the volume of excavation with respect to other (wider bottom width) options; (4) resulted in a channel with invert elevations above the Brenna formation for a significant length of the diversion alignment. As was done with the Phase 3.1 design, the proposed channel design for the LPP diversion was provided to the Corps of Engineers for stability analysis during the design. While the stability issues still required benching in some locations, the extent of the benching was far less with the Phase 4 design. The Corps' geotechnical analysis called for benching in five sections along the diversion alignment. The benches were all set at eight feet above the bottom of the main channel and varied in width from 15 to 40 feet. The side slopes above and below the benches all remained at 7:1 (H:V) throughout the length of the diversion. The low flow channel included in Phase 3, which had a depth of three feet, 4:1 (H:V) side slopes and a 10 foot bottom, was also incorporated into the channel geometry. The cross section geometries for the diversion are summarized in Table D2 with the locations shown in Figure D2. The size of the channel in the reach between the Wild Rice River and the diversion inlet weir was reduced for the final Phase 4 design. Within this reach, the channel has a 100 foot bottom and 7:1 (H:V) side slopes.

Construction of a diversion alternative would require considerable space for disposal of excess soil. Excess soil would be disposed of adjacent to the diversion channel. The geotechnical parameters provided by the Corps of Engineers define the side slopes as well as the maximum embankment height of the spoil piles. The spoil slopes are 7:1 (H:V) and 10:1 (H:V) for the diversion side and outside slopes respectively and the maximum spoil height is generally 15 feet. There are areas in which the geotechnical conditions limit the height of spoil immediately adjacent to the diversion channel. The width of the spoil piles is controlled by the total volume of material to be disposed of. The project layout drawings included in this appendix show the spoil pile footprints that factor in all of these design constraints.

D5.0 RIGHT OF WAY

The proposed diversion alternatives will require acquisition of a right of way corridor wide enough to allow for the footprint of the diversion channel as well as the adjacent spoil piles. Additional right of way will also be necessary for tie-back levees. The Corps of Engineers provided the following parameters for right of way acquisition:

1. Permanent easement to 30' outside the toe of spoil or levee
2. Temporary easement 15' beyond the permanent easement for construction limits

The easement area required for the North Dakota Diversion alignment is summarized in Table D1 below. It should be noted that these numbers include the area required for Storage Area 1, which was added to the LPP for Phase 4, so the numbers appear considerably higher than those included for Phase 3.

Table D1- Easement Area Summary for LPP Diversion and Hydraulic Structures

Structure	Easements Areas (Acres)		
	Permanent	Temporary	Total
Red River Control Structure	110	4	114
Wolverton Creek	4	0	4
Wild Rice River	114	2	116
Storage Area 1	4,484	7	4,490
Inlet	13	1	15
Sheyenne River	158	4	162
Maple River	151	4	155
Lower Rush River	122	2	124
Rush River	126	3	129
Outlet	76	2	78
LPP Diversion Channel	5,944	107	6,051
TOTALS=	**11,302**	**136**	**11,438**

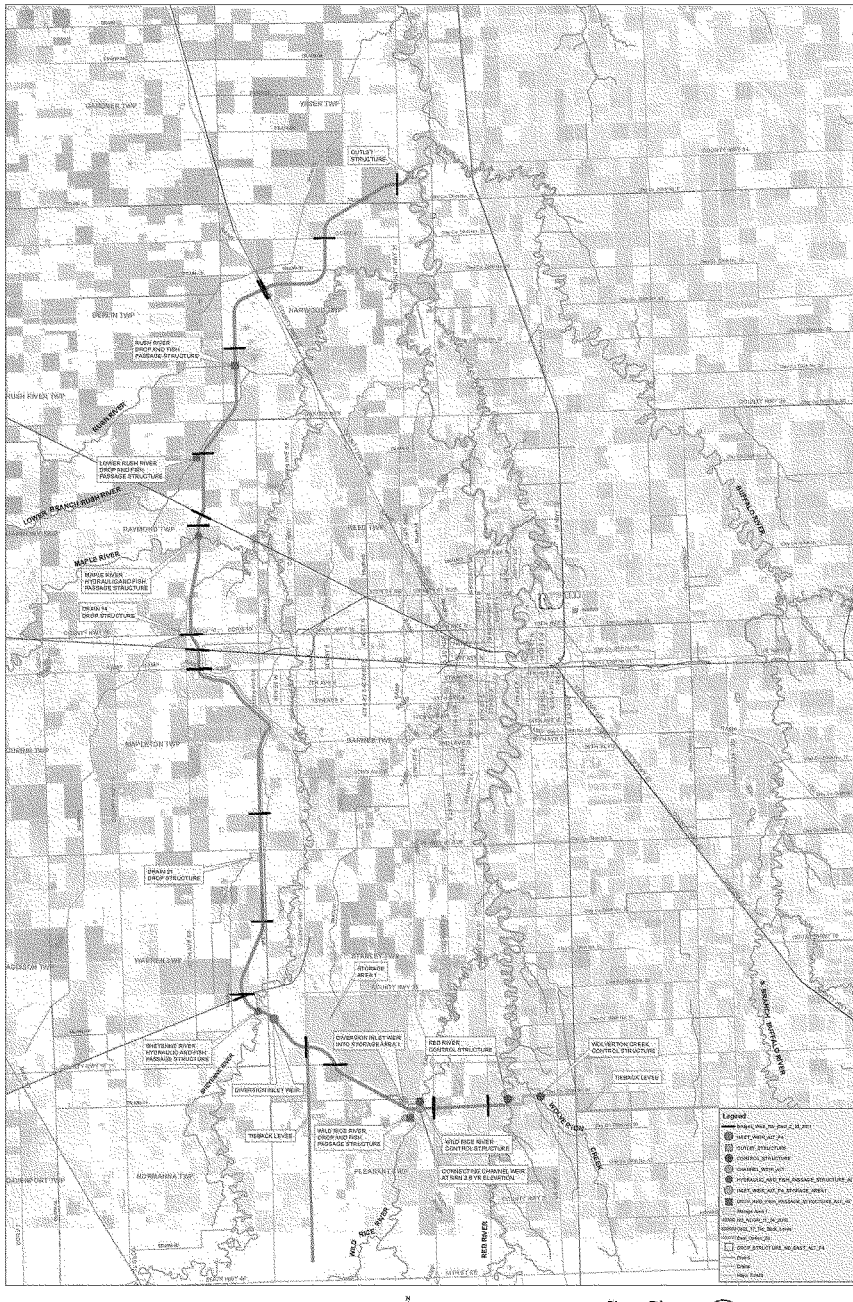


Figure D1- North Dakota Diversion LPP Alignment

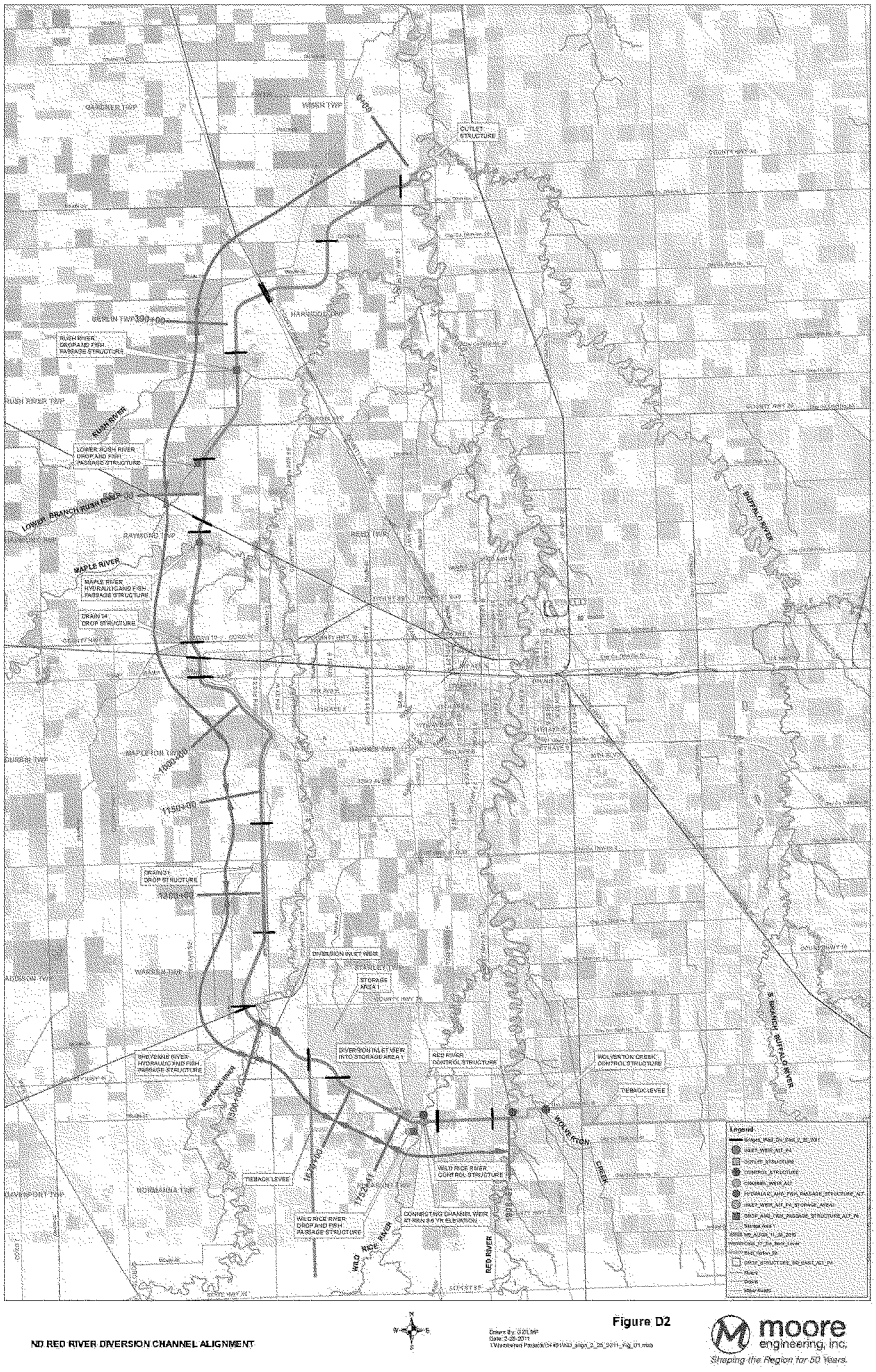


Figure D2- North Dakota Diversion LPP Alignment-Channel Cross Section Locations

Table D2- LPP Cross Sections By Station

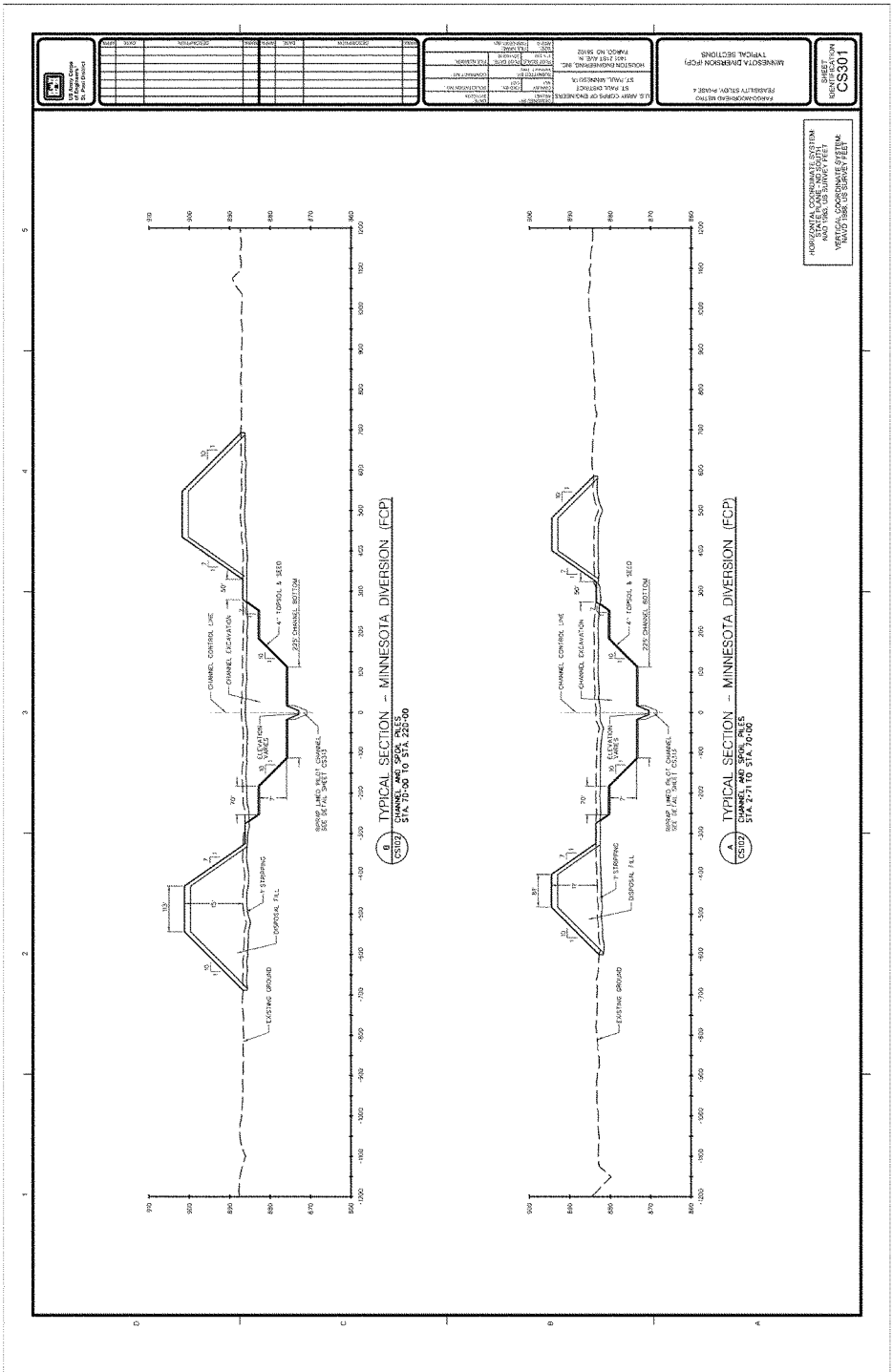
						Channel Configuration					Bench Configuration				Spoil Pile Surcharge		Stability Analysis: Min FS				
Filename	Station	Start	End	Length (ft)	Length (mi)	Ground Surface	Bottom Elev.	Depth	Δ Phase 3.1 vs 4.4	Bottom Width	Bench Width	Bench Height	Bench Slope	Channel Slope	Setback	Height	(A) Global Entry/Exit	(B) Wedge	(C) Lower Localized	(D) Upper Localized	(2) Undrained Global
FM_P4_ND_Div_Sect-01_P4_vr4	STA 120+00	0+00	390+00	39,000	7.4	882	865	17	8	250				7	50	8	1.554	1.558	1.793	2.206	1.345
FM_P4_ND_Div_Sect-02_P4_vr4	STA 545+00	390+00	660+00	27,000	5.1	890	868	22	3	250	15	8	7	7	50	11	1.431	1.437	1.508	1.957	1.315
FM_P3_ND_Div_Sect-03_P4_vr4	STA 940+00	660+00	1000+00	34,000	6.4	900	877	23	4	250	25	8	7	7	50	15	1.421	1.411	1.710	1.794	1.332
FM_P3_ND_Div_Sect-04_P4_vr4	STA 1080+00	1000+00	1150+00	15,000	2.8	900	879	21	3	250	20	8	7	7	50	15	1.429	1.425	1.611	1.882	1.364
FM_P3_ND_Div_Sect-05_P4_vr4	STA 1225+00	1150+00	1300+00	15,000	2.8	905	881	22	2	250	40	8	7	7	50	12	1.411	1.412	1.418	1.640	1.306
FM_P3_ND_Div_Sect-05B_P4_vr4	STA 1445+00	1300+00	1490+00	19,000	3.6	913	885	28	2	250	40	8	7	7	50		1.416	1.415	1.208	1.407	1.307
FM_P3_ND_Div_Sect-06B_P4_vr4	STA 1550+00	1490+00	1550+00	6,000	1.1	920	903	17	18	250				7	50		1.912	1.912	1.782	2.866	1.563
FM_P3_ND_Div_Sect-06C_P4_vr4	STA 1720+00	1670+00	1770+00	10,000	1.9	913	905	8	16	250				7	50		2.353	2.350	2.637	4.330	2.092
FM_P3_ND_Div_Sect-07_P4_vr4	STA 1810+00	1770+00	1922+00	15,200	2.9	912	901	11	5	250				7	50		2.201	2.173			2.000

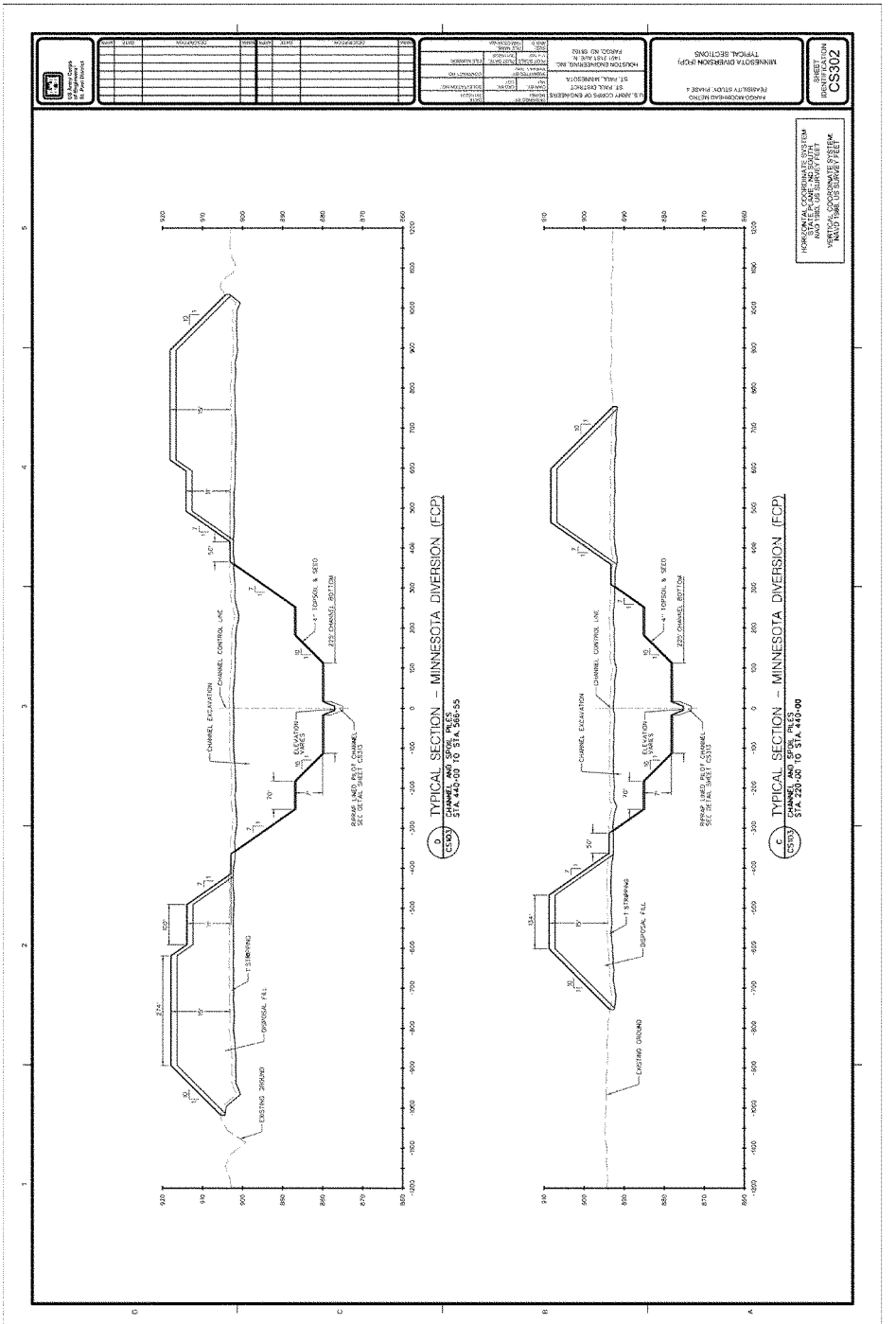
** NOTES **

1) "Bottom Elev." indicates the invert elevation of the main channel bottom. The low flow channel invert elevation is 3 feet less.

2) Low Flow channel analyzed without riprap.

3) Constant channel slope of 1V on 7H was used for both the bench and the slope above the bench.


MINNESOTA DIVERSION (FCP)
TYPICAL SECTIONS
FEASIBILITY STUDY PHASE 4
PHASE/SUBPHASE NUMBER
HOUSTON ENGINEERING, INC.
1401 21ST AVE. N
FARGO, ND 58102
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL, MINNESOTA
SHEET IDENTIFICATION
CS301
HORIZONTAL COORDINATE SYSTEM:
STATE PLANE, NAD 1983(2011)
NORTH DAKOTA, US FEET
VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET
TYPICAL SECTION – MINNESOTA DIVERSION (FCP)
CHANNEL AND SPOIL PILES
STA. 70+00 TO STA. 220+00
TYPICAL SECTION – MINNESOTA DIVERSION (FCP)
CHANNEL AND SPOIL PILES
STA. 2+71 TO STA. 70+00
CHANNEL CONTROL LINE
CHANNEL EXCAVATION
4' TOPSOIL & SEED
225' CHANNEL BOTTOM
EXISTING GROUND
DISPOSAL FILL
3:1 STRIPPING
SPOIL PILES
SEE DETAIL SHEET CS313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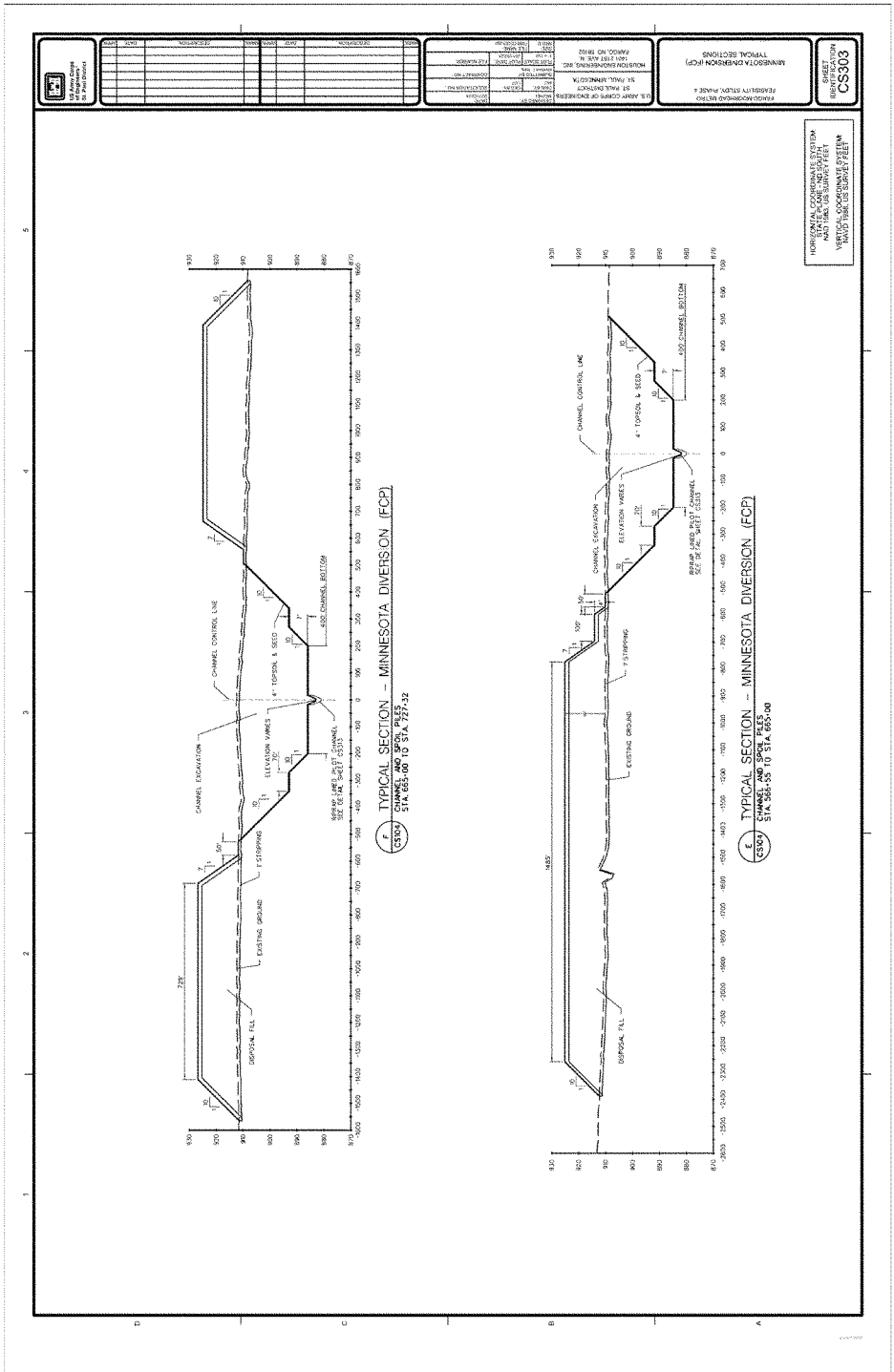
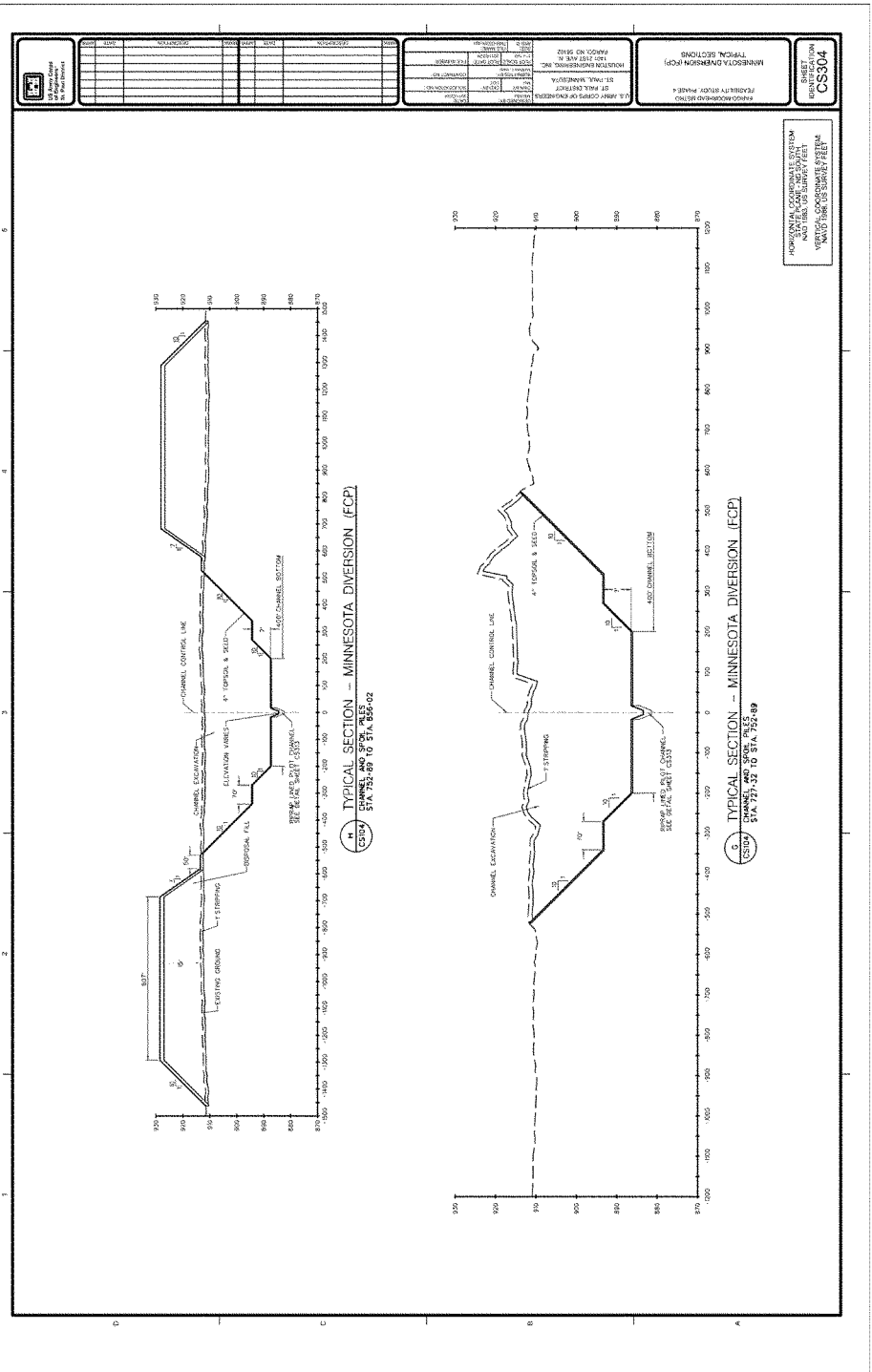
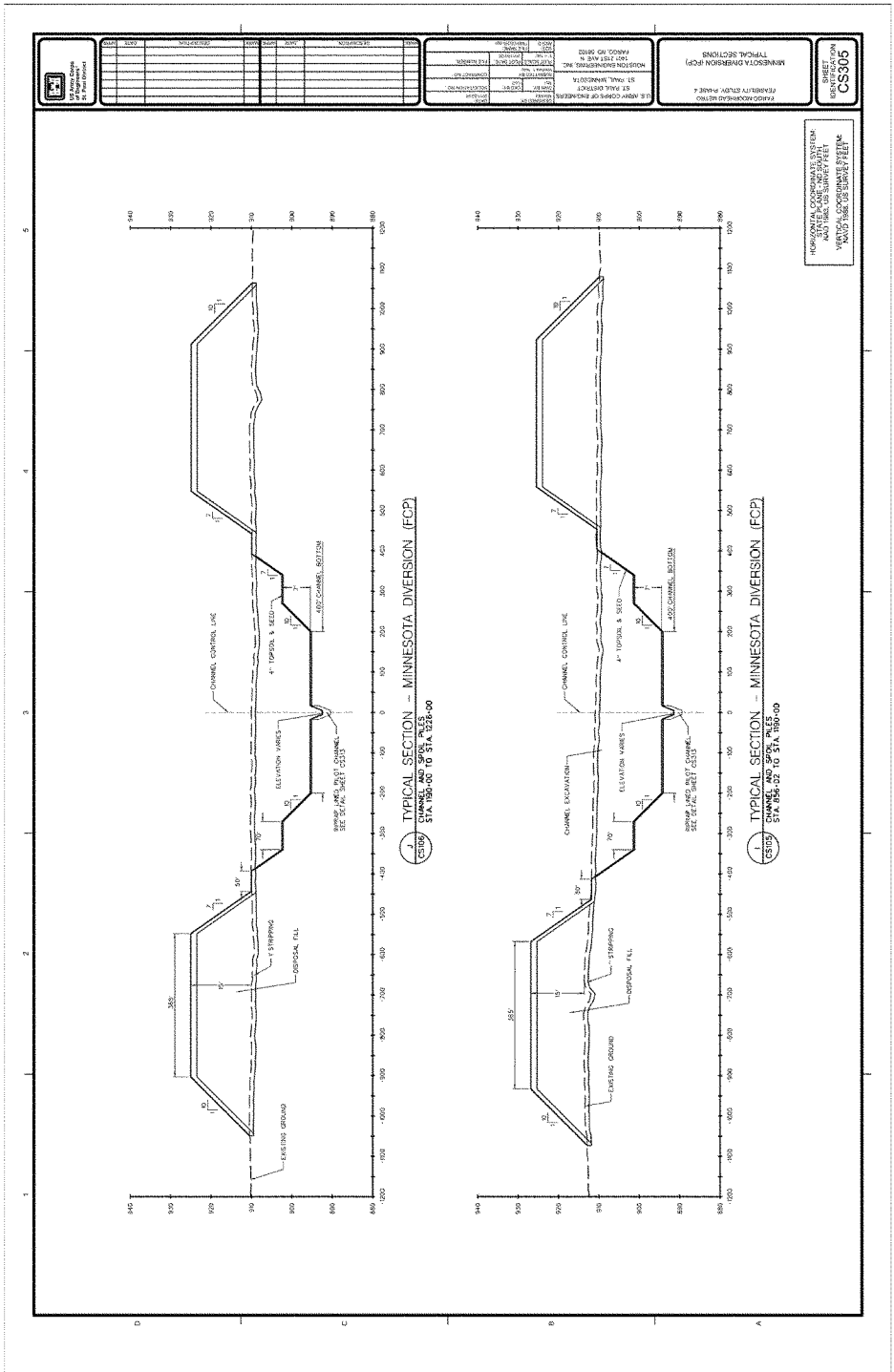
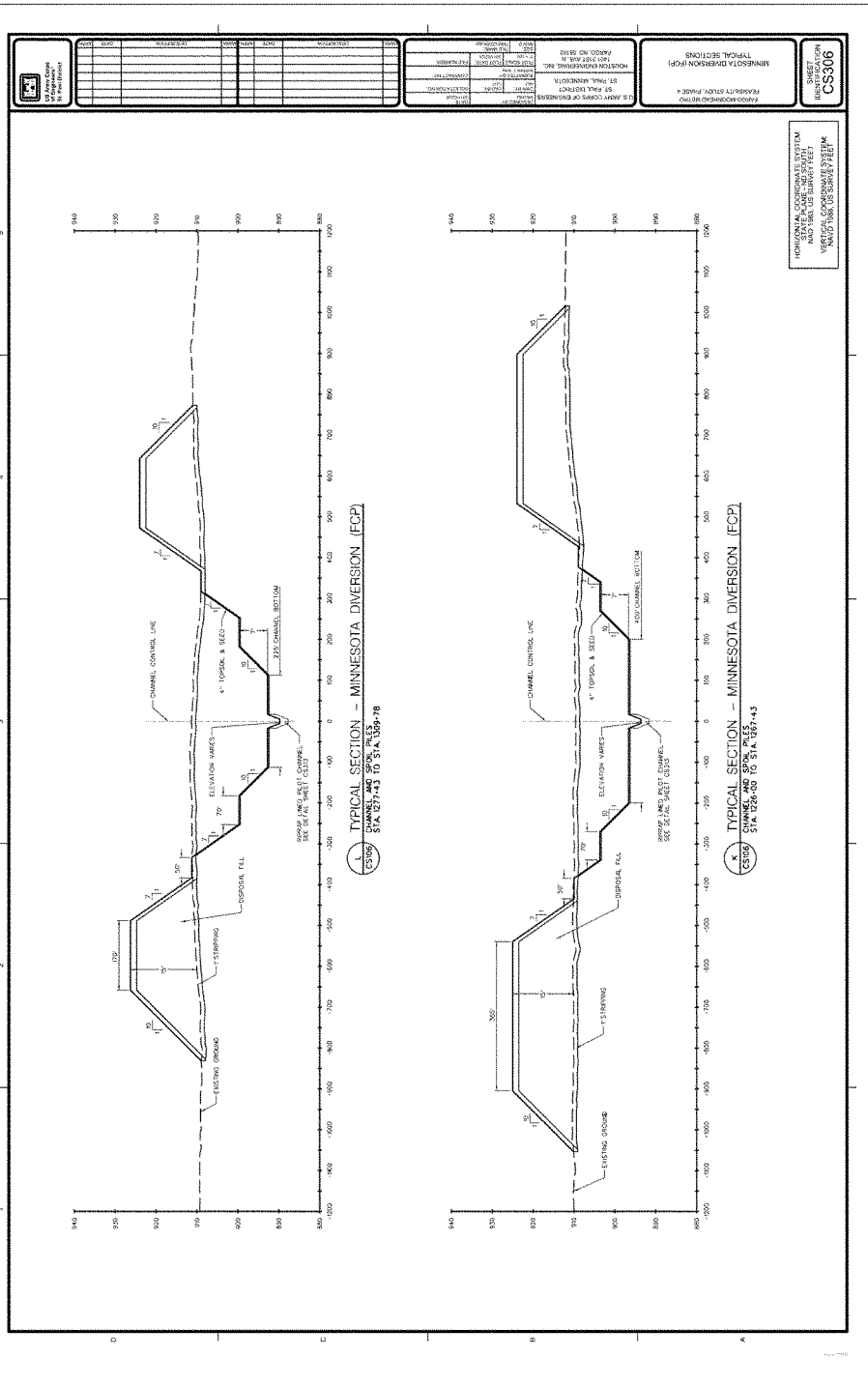
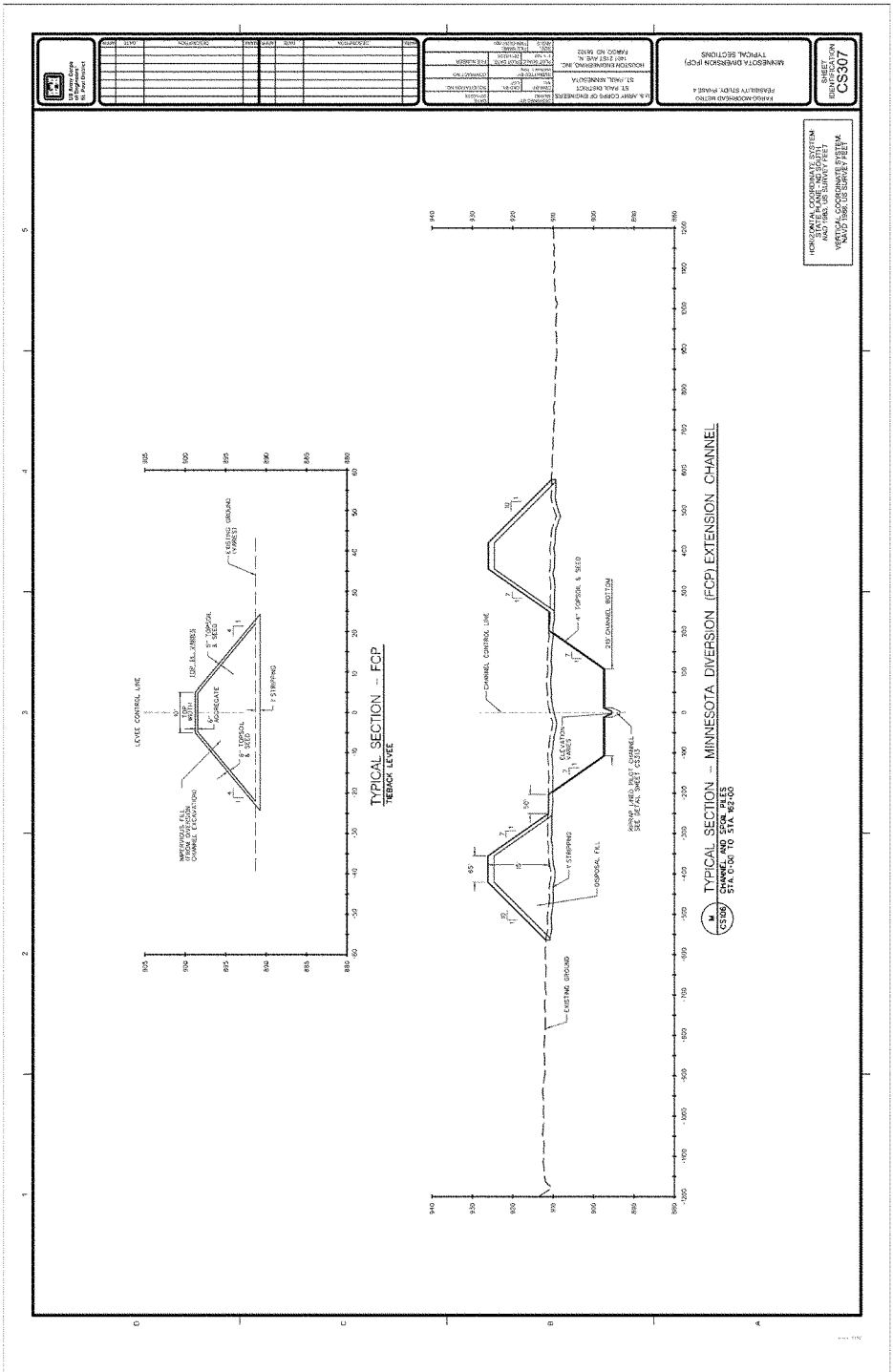
CHANNEL AND SPOIL PILES
STA 440+00 TO STA 550+55
TYPICAL SECTION – MINNESOTA DIVERSION (FCP)
CHANNEL AND SPOIL PILES
STA 250+00 TO STA 440+00
CHANNEL EXCAVATION
CHANNEL CONTROL LINE
EXISTING GROUND
DISPOSAL FILL
1 STRIPPING
RIPRAP LINED PILOT CHANNEL
SEE DETAIL SHEET CS310
4" TOPSOIL & SEED
225' CHANNEL BOTTOM
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
HOUSTON ENGINEERING, INC.
1401 21ST AVE. N.
FARGO, ND 58102
MINNESOTA DIVERSION (FCPP)
TYPICAL SECTIONS
CS302

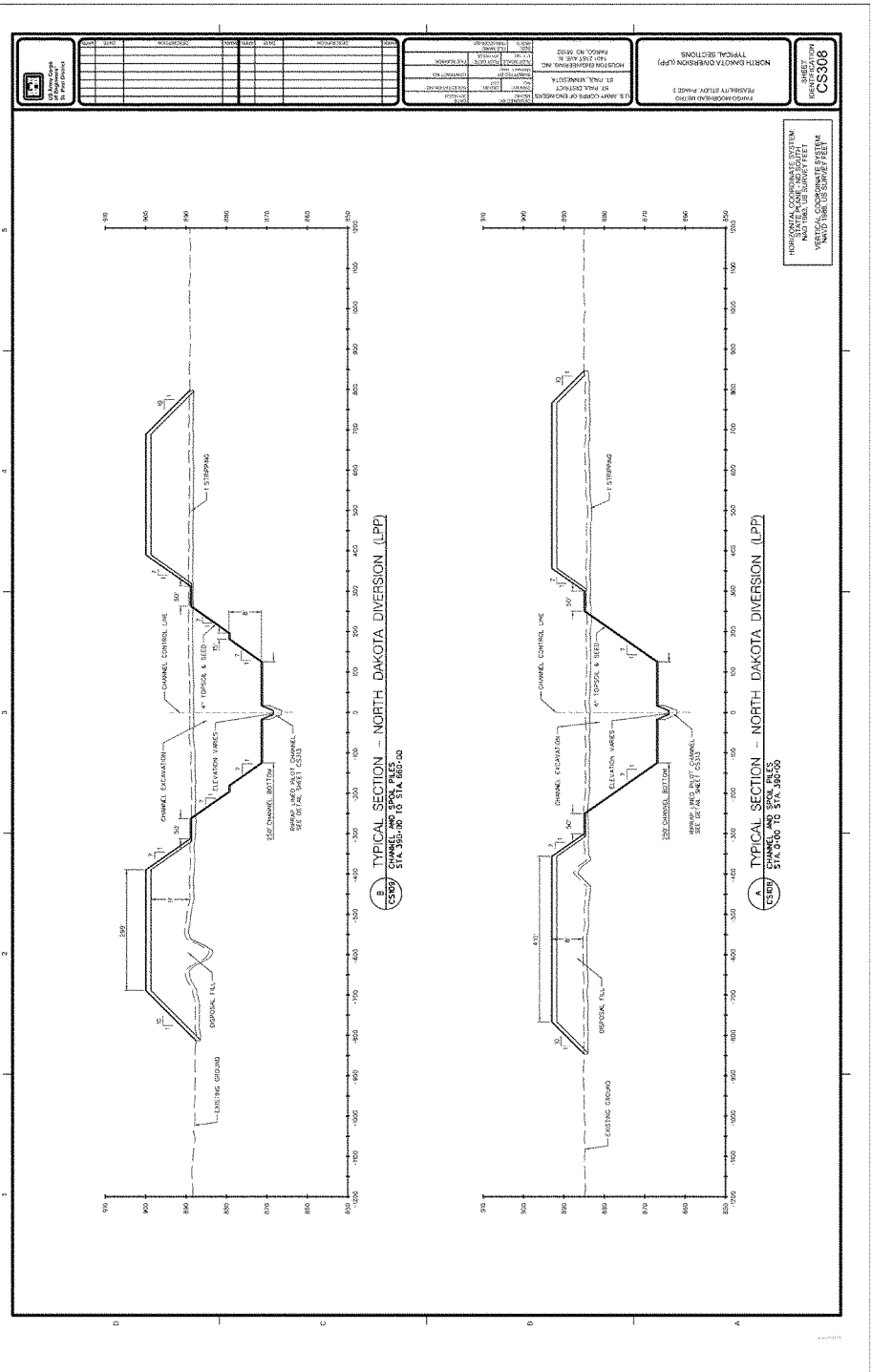

MINNESOTA DIVERSION (FCP)
TYPICAL SECTIONS
FARGO-MOORHEAD METRO FEASIBILITY STUDY, PHASE 4
HOLDING SOMMERS, INC.
1401 21ST AVE. N.
FARGO, ND 58102
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
SHEET IDENTIFICATION
CS303
HORIZONTAL COORDINATE SYSTEM
NAD 1983 US SURVEY FEET
STATE PLANE - ND SOUTH
ZONE 3302, US SURVEY FEET
VERTICAL COORDINATE SYSTEM
NAVD 1988, US SURVEY FEET
TYPICAL SECTION – MINNESOTA DIVERSION (FCP)
CHANNEL AND SPOIL PILES
STA. 665+00 TO STA. 727+32
TYPICAL SECTION – MINNESOTA DIVERSION (FCP)
CHANNEL AND SPOIL PILES
STA. 565+55 TO STA. 665+00
CHANNEL CONTROL LINE
CHANNEL EXCAVATION
ELEVATION VARIES
4' TOPSOIL & SEED
EXISTING GROUND
DISPOSAL FILL
REPAIR LINER PILOT CHANNEL
SEE DETAIL SHEET CS315


MINNESOTA DIVERSION (FCP)
FEASIBILITY STUDY, PHASE 4
TYPICAL SECTIONS
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
HOUSTON ENGINEERING, INC.
SHEET IDENTIFICATION
CS304
TYPICAL SECTION – MINNESOTA DIVERSION (FCP)
CHANNEL AND SPOIL PILES
STA. 752-89 TO STA. 856-02
TYPICAL SECTION – MINNESOTA DIVERSION (FCP)
CHANNEL AND SPOIL PILES
STA. 727-32 TO STA. 752-89
CHANNEL CONTROL LINE
CHANNEL EXCAVATION
4" TOPSOIL & SEED
400' CHANNEL SECTION
EXISTING GROUND
STRIPPING
DISPOSAL FILL
ELEVATION VARIES
SPOIL LINES PILOT CHANNEL
SEE DETAIL SHEET CS313
HORIZONTAL COORDINATE SYSTEM
STATE PLANE, NAD 1983/11
NAD 1988, US SURVEY FEET
VERTICAL DATUM
NAVD 1988, US SURVEY FEET


MINNESOTA DIVERSION (FCP)
TYPICAL SECTIONS
SHEET IDENTIFICATION
CS305
TYPICAL SECTION – MINNESOTA DIVERSION (FCP)
CHANNEL AND SPOIL PILES
STA 1190-00 TO STA 1226-00
TYPICAL SECTION – MINNESOTA DIVERSION (FCP)
CHANNEL AND SPOIL PILES
STA 856-02 TO STA 1190-00
CHANNEL CONTROL LINE
4" TOPSOIL & SEED
455' CHANNEL BOTTOM
ELEVATION VARIES
RIPRAP LINED PILOT CHANNEL
SEE DETAIL SHEET CS313
CHANNEL EXCAVATION
STRIPPING
DISPOSAL FILL
EXISTING GROUND
HORIZONTAL COORDINATE SYSTEM
STATE PLANE, NAD 1983
NORTH DAKOTA SOUTH
VERTICAL COORDINATE SYSTEM
NAVD 1988, US SURVEY FEET


TYPICAL SECTION – MINNESOTA DIVERSION (FCP)
CHANNEL AND SPOIL PILES
STA. 1277+43 TO STA. 1369+78
TYPICAL SECTION – MINNESOTA DIVERSION (FCP)
CHANNEL AND SPOIL PILES
STA. 1226+00 TO STA. 1277+43
CHANNEL CONTROL LINE
4" TOPSOIL & SEED
ELEVATION VARIES
DISPOSAL FILL
STRIPPING
EXISTING GROUND
225' CHANNEL BOTTOM
400' CHANNEL BOTTOM
MINNESOTA DIVERSION (FCP)
TYPICAL SECTIONS
CS306


MINNESOTA DIVERSION (FCP)
TYPICAL SECTIONS
PHASE 4 ADDENDUM METRO
FEASIBILITY STUDY, PHASE 4
SHEET IDENTIFICATION
CS307
HORIZONTAL COORDINATE SYSTEM
NAD 1983 US SURVEY FEET
VERTICAL COORDINATE SYSTEM
NAVD 1988 US SURVEY FEET
TYPICAL SECTION -- FCP
TEBACK LEVEE
LEVEE CONTROL LINE
10' WIDTH
10% EL. ARMIS
5" TOPSOIL & SEED
4" AGGREGATE
EXISTING GROUND
4" STRIPPING
IMPERVIOUS FILL (FROM DIVERSION CHANNEL EXCAVATION)
TYPICAL SECTION -- MINNESOTA DIVERSION (FCP) EXTENSION CHANNEL
CS306
CHANNEL AND SPOIL FILES
STA 0+00 TO STA 45+00
CHANNEL CONTROL LINE
4" TOPSOIL & SEED
210' CHANNEL BOTTOM
EXCAVATION
DISPOSAL FILL
4" STRIPPING
EXISTING GROUND
SPOIL LAND FOR FUTURE CHANNEL
SEE PLAN SHEET CS305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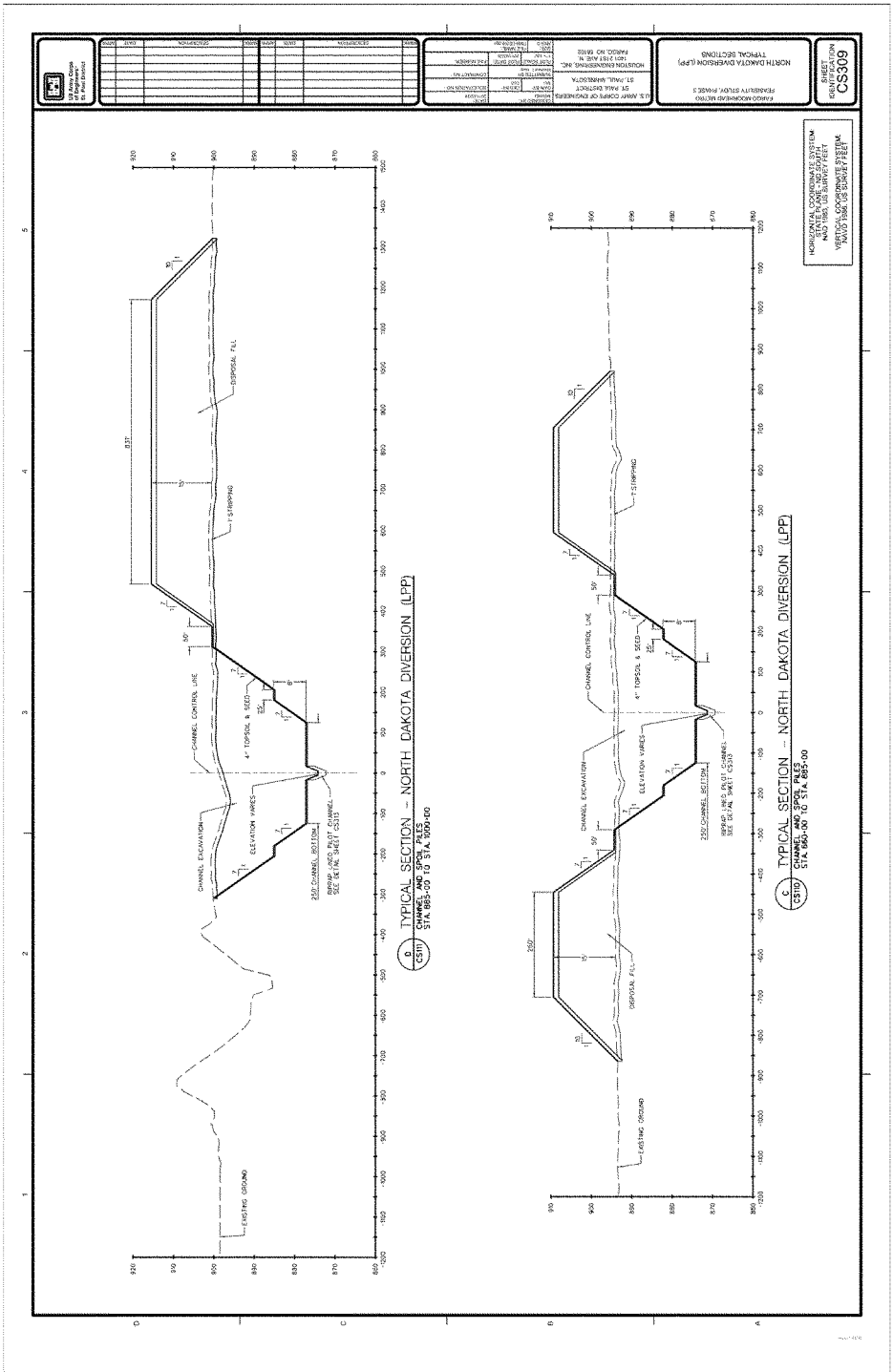
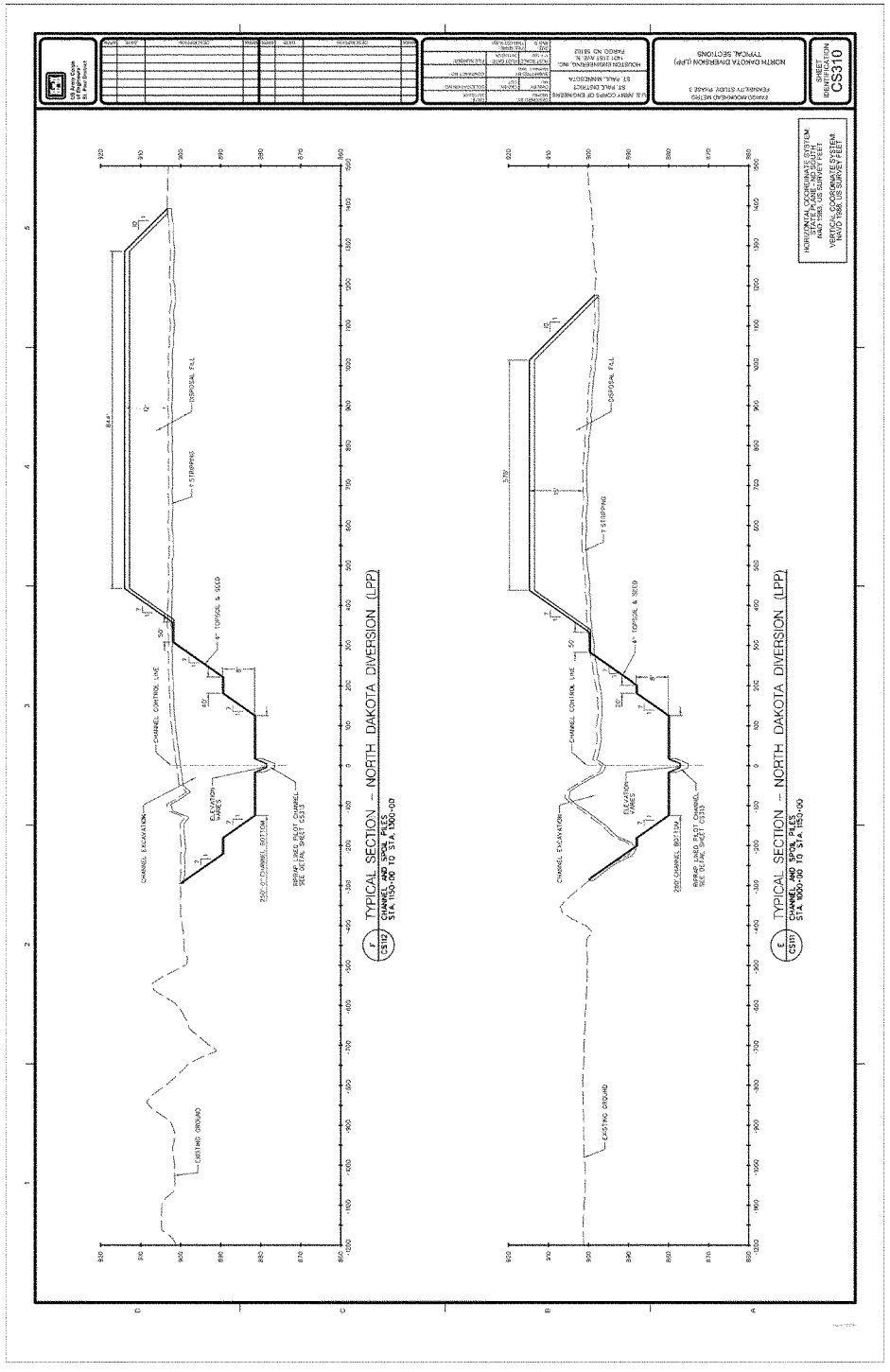
TYPICAL SECTIONS
FARGO-MOORHEAD METRO FEASIBILITY STUDY, PHASE 3
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
HOLSTEIN ENGINEERING, INC.
SHEET IDENTIFICATION
CS308
TYPICAL SECTION – NORTH DAKOTA DIVERSION (LPP)
CHANNEL AND SPOIL PILES
STA 0+00 TO STA 380+00
EXISTING GROUND
DISPOSAL FILL
CHANNEL EXCAVATION
CHANNEL CONTROL LINE
4" TOPSOIL & SEED
STRIPPING
ELEVATION VARIES
RIPRAP LINED PILOT CHANNEL
SEE DETAIL SHEET CS313
HORIZONTAL COORDINATE SYSTEM
STATE PLANE NORTH ZONE (3301)
NAD 1983, US SURVEY FEET
VERTICAL DATUM
NAVD 1988, US SURVEY FEET


NORTH DAKOTA DIVERSION (LPP)
TYPICAL SECTIONS
SHEET IDENTIFICATION
CS309
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
HORIZONTAL COORDINATE SYSTEM
STATE PLANE, NORTH DAKOTA
NORTH ZONE
VERTICAL COORDINATE SYSTEM
NAVD 1988, US SURVEY FEET
TYPICAL SECTION – NORTH DAKOTA DIVERSION (LPP)
CHANNEL AND SPOIL PILES
STA. 885-00 TO STA. 1000-00
TYPICAL SECTION – NORTH DAKOTA DIVERSION (LPP)
CHANNEL AND SPOIL PILES
STA. 660-00 TO STA. 885-00
EXISTING GROUND
CHANNEL EXCAVATION
CHANNEL CONTROL LINE
ELEVATION VARIES
4" TOPSOIL & SEED
DISPOSAL FILL
STRIPPING
250' CHANNEL BOTTOM
RIPRAP AND PILOT CHANNEL
SEE DETAIL SHEET CS315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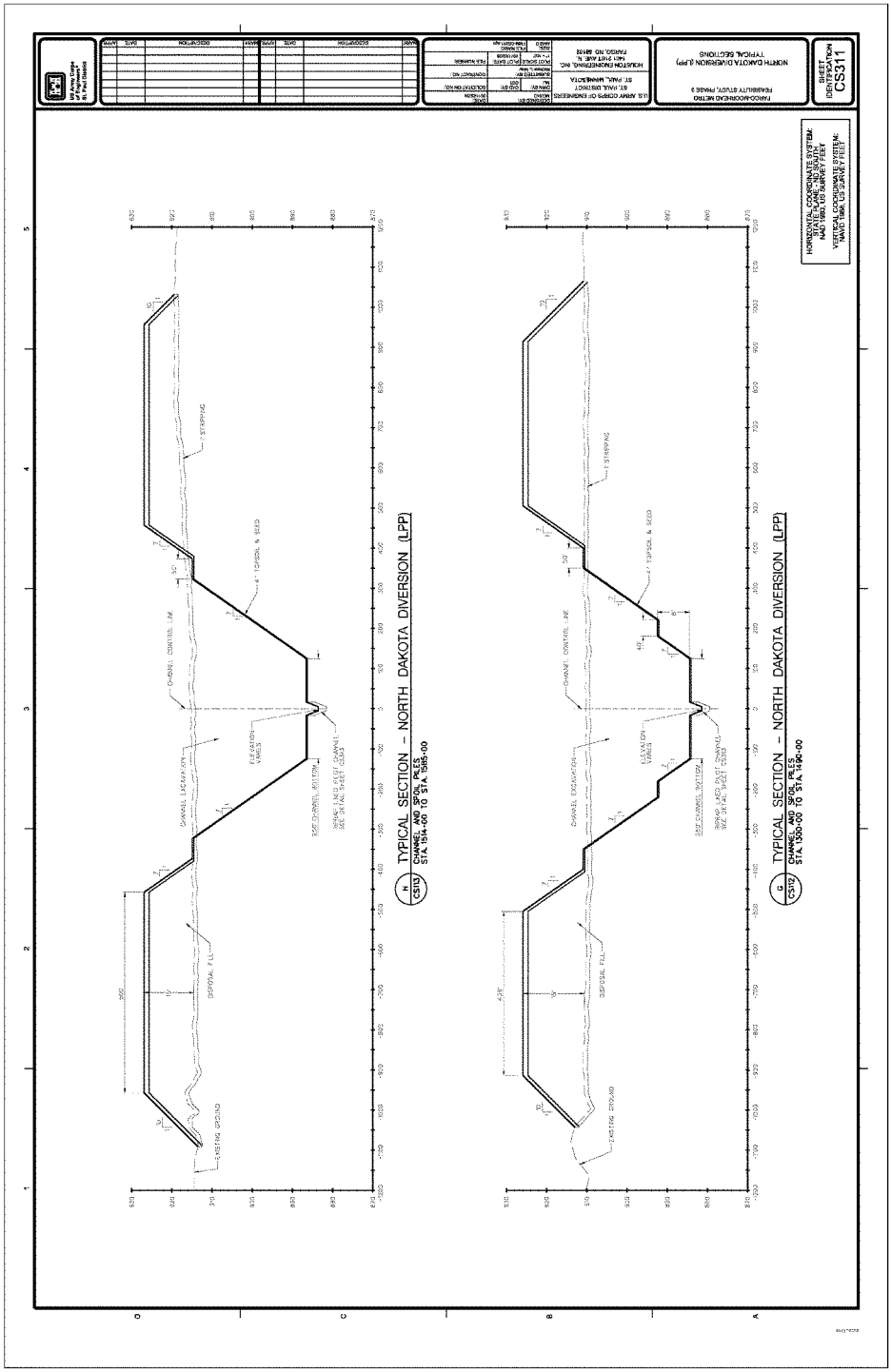
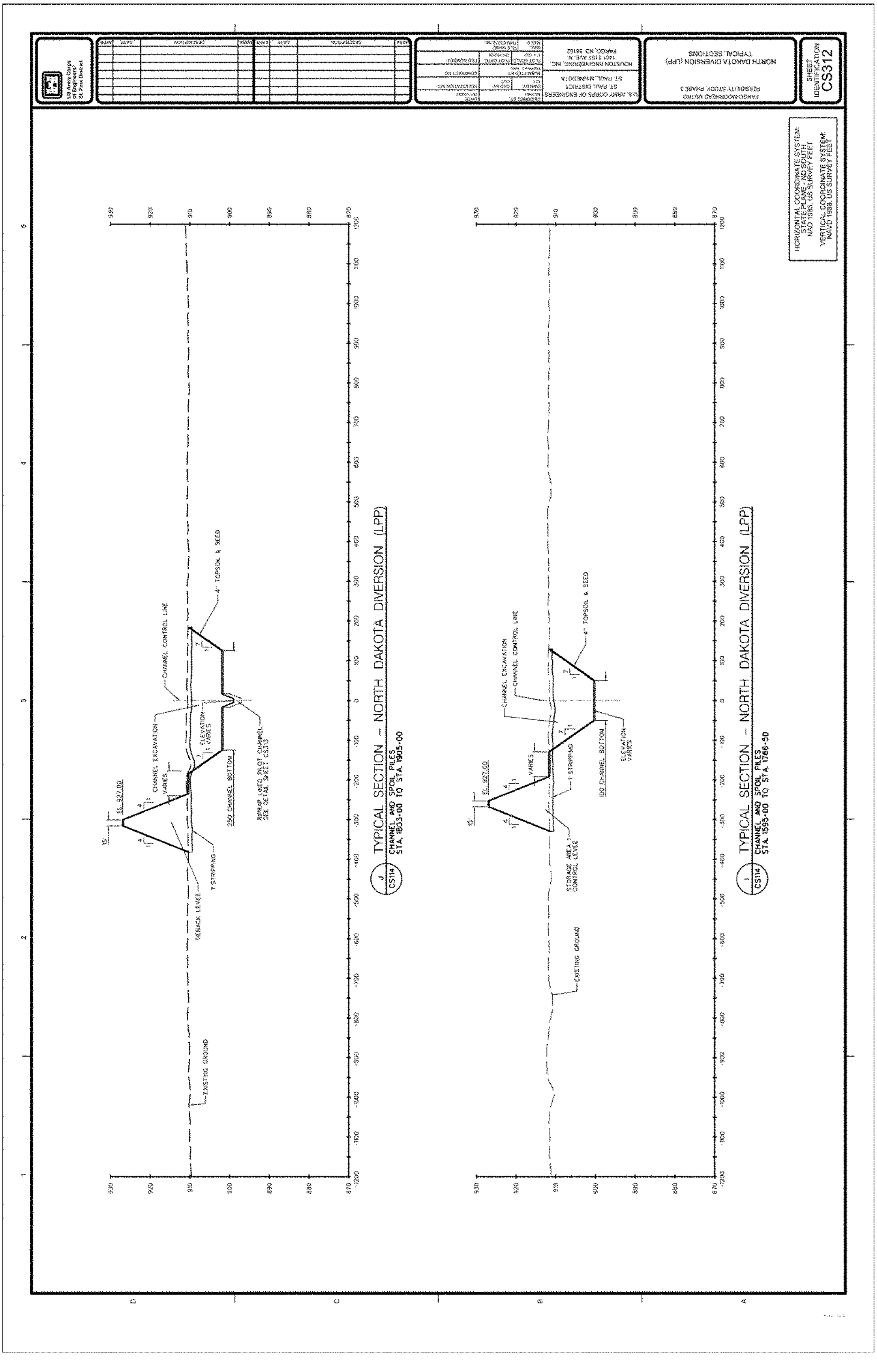
TYPICAL SECTIONS
CS310
F CS310 TYPICAL SECTION – NORTH DAKOTA DIVERSION (LPP)
CHANNEL AND SPOIL PILES
STA 1000+00 TO STA 1150+00
E CS310 TYPICAL SECTION – NORTH DAKOTA DIVERSION (LPP)
CHANNEL AND SPOIL PILES
STA 1000+00 TO STA 1150+00
EXISTING GROUND
CHANNEL EXCAVATION
CHANNEL CONTROL LINE
DISPOSAL FILL
STRIPPING
4" TOPSOIL & SEED
RIPRAP LINED PILOT CHANNEL
SEE DETAIL SHEET CS313
250' PILOT CHANNEL BOTTOM


NORTH DAKOTA DIVERSION (LPP)
TYPICAL SECTIONS
FARGO-MOORHEAD METRO FEASIBILITY STUDY, PHASE II
HOUSTON ENGINEERING, INC.
1401 21ST AVE. N.
FARGO, ND 58102
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
SHEET IDENTIFICATION
CS311
HORIZONTAL COORDINATE SYSTEM:
NAD 83, 1986, ND SOUTH
ZONE, U.S. SURVEY FEET
VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET
TYPICAL SECTION – NORTH DAKOTA DIVERSION (LPP)
CHANNEL AND SPOIL PILES
STA. 1514+00 TO STA. 1565+00
TYPICAL SECTION – NORTH DAKOTA DIVERSION (LPP)
CHANNEL AND SPOIL PILES
STA. 1350+00 TO STA. 1400+00


US Army Corps of Engineers St. Paul District
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
HOUSTON ENGINEERING, INC.
1401 21ST AVE. N.
FARGO, ND 58102
FARGO-MOORHEAD METRO
FEASIBILITY STUDY, PHASE 3
NORTH DAKOTA DIVERSION (LPP)
TYPICAL SECTIONS
SHEET IDENTIFICATION
CS312
HORIZONTAL COORDINATE SYSTEM:
STATE PLANE - ND SOUTH
NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET
EL. 927.00
15'
CHANNEL EXCAVATION
VARIES
CHANNEL CONTROL LINE
ELEVATION VARIES
4" TOPSOIL & SEED
TIEBACK LEVEE
1' STRIPPING
250' CHANNEL BOTTOM
RIPRAP LINED PILOT CHANNEL
SEE DETAIL SHEET CS313
EXISTING GROUND
J
CS114
TYPICAL SECTION – NORTH DAKOTA DIVERSION (LPP)
CHANNEL AND SPOIL PILES
STA. 1803-00 TO STA. 1905-00
STORAGE AREA 1
CONTROL LEVEE
100' CHANNEL BOTTOM
I
CS114
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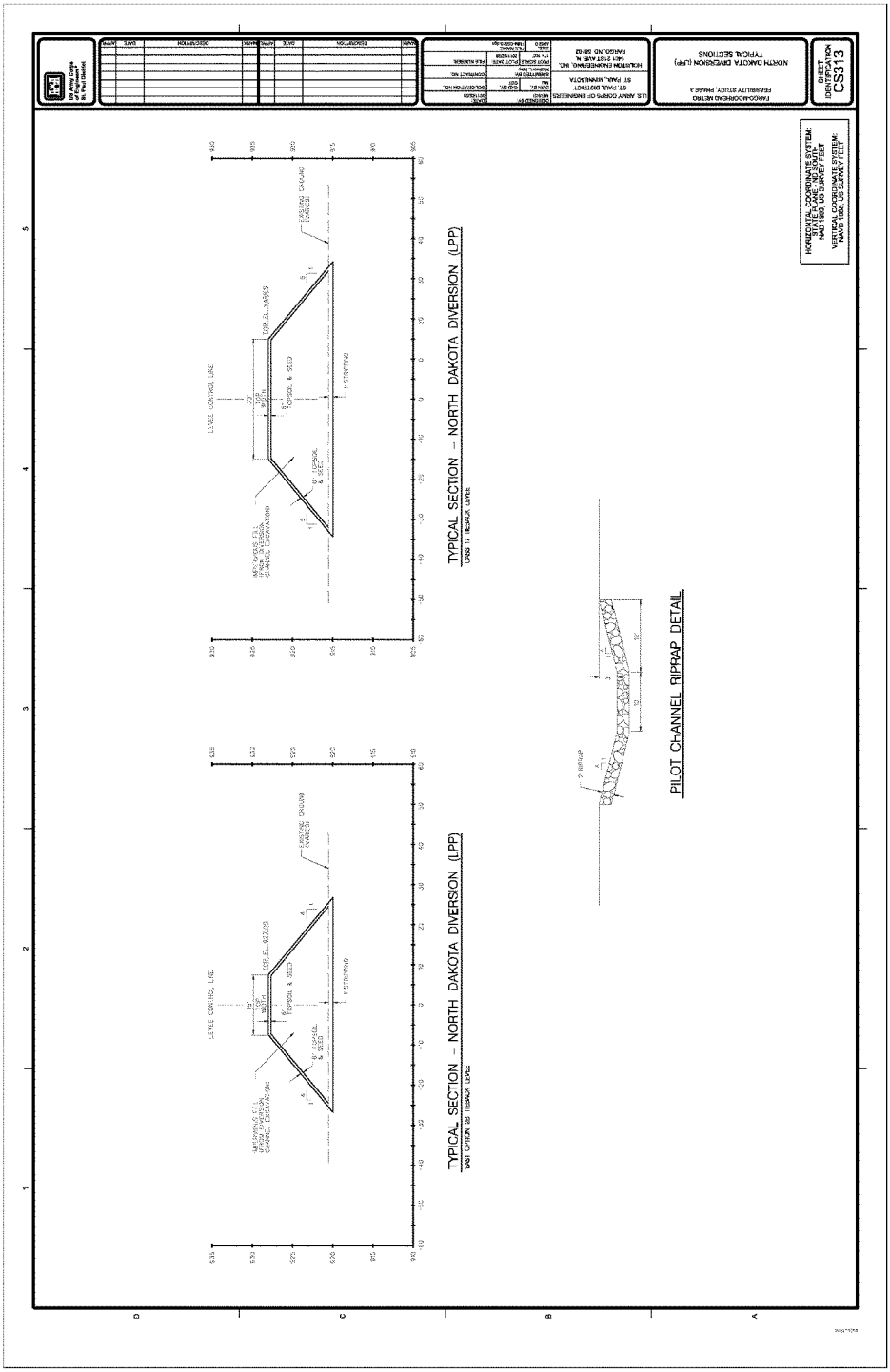
CHANNEL AND SPOIL PILES
STA. 1595-00 TO STA. 1766-50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EAST OPTION 2B TIEBACK LEVEE
TYPICAL SECTION – NORTH DAKOTA DIVERSION (LPP)
OXBOW TIEBACK LEVEE
PILOT CHANNEL RIPRAP DETAIL
NORTH DAKOTA DIVERSION (LPP)
TYPICAL SECTIONS
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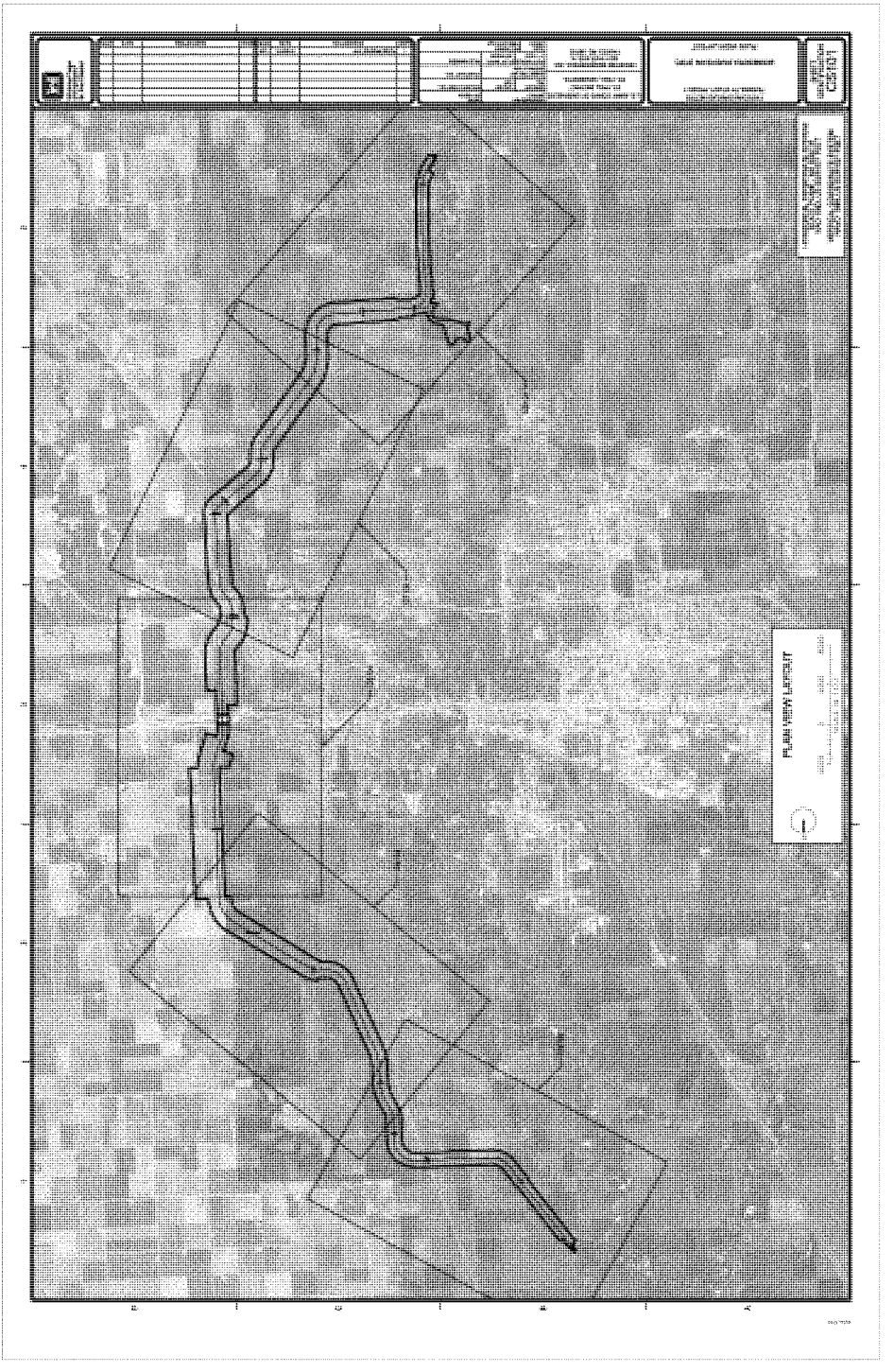
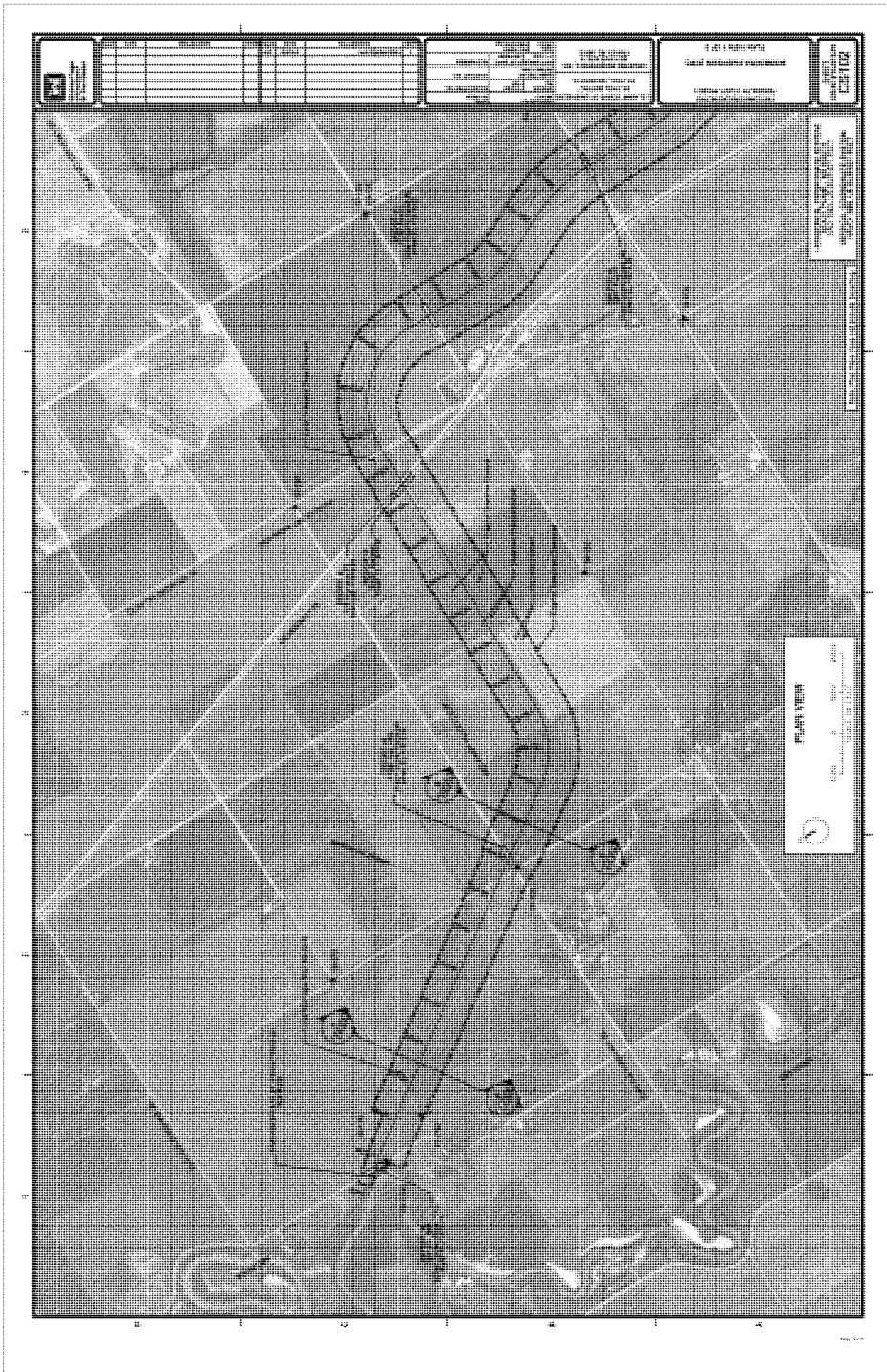
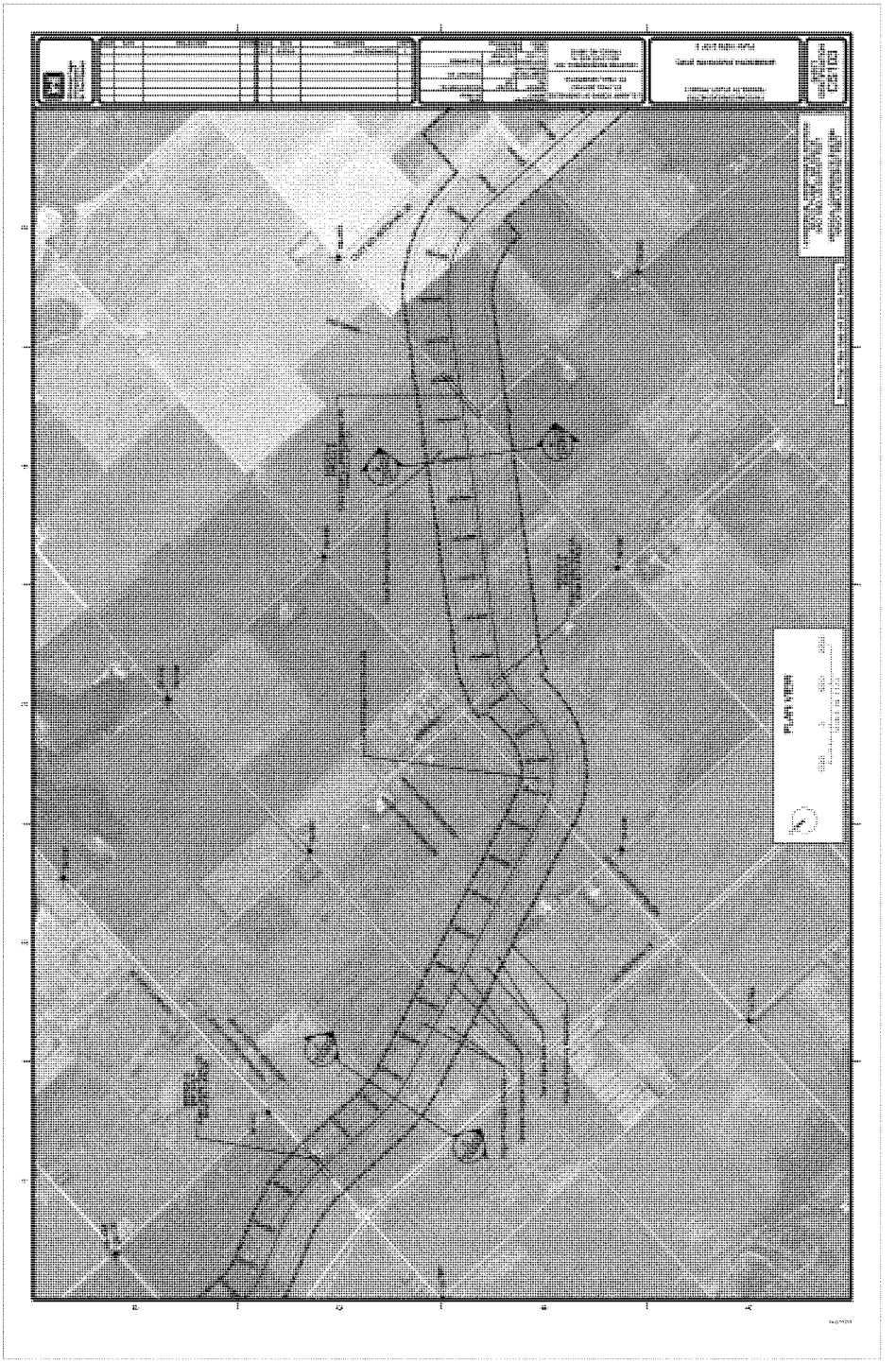
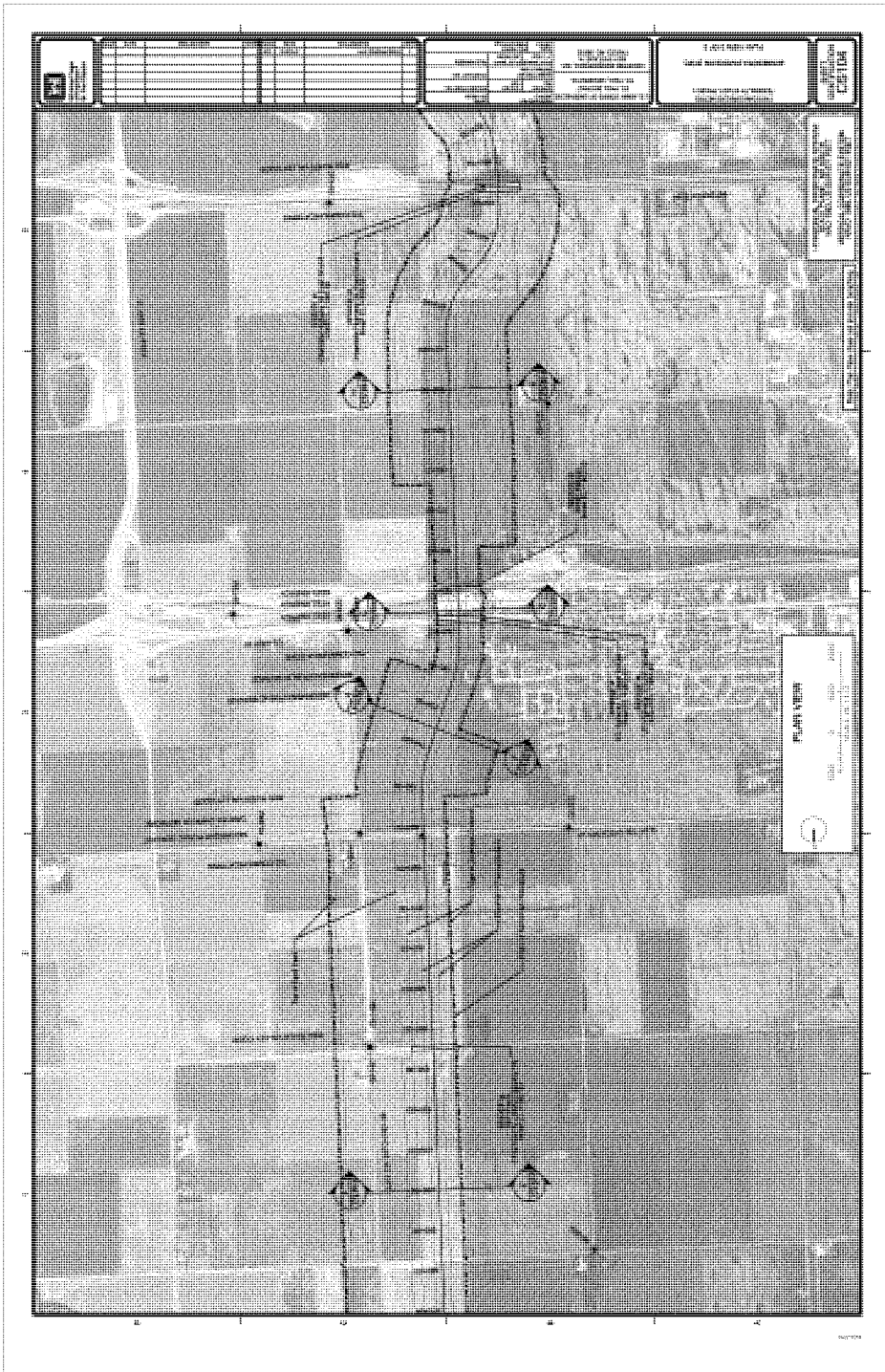
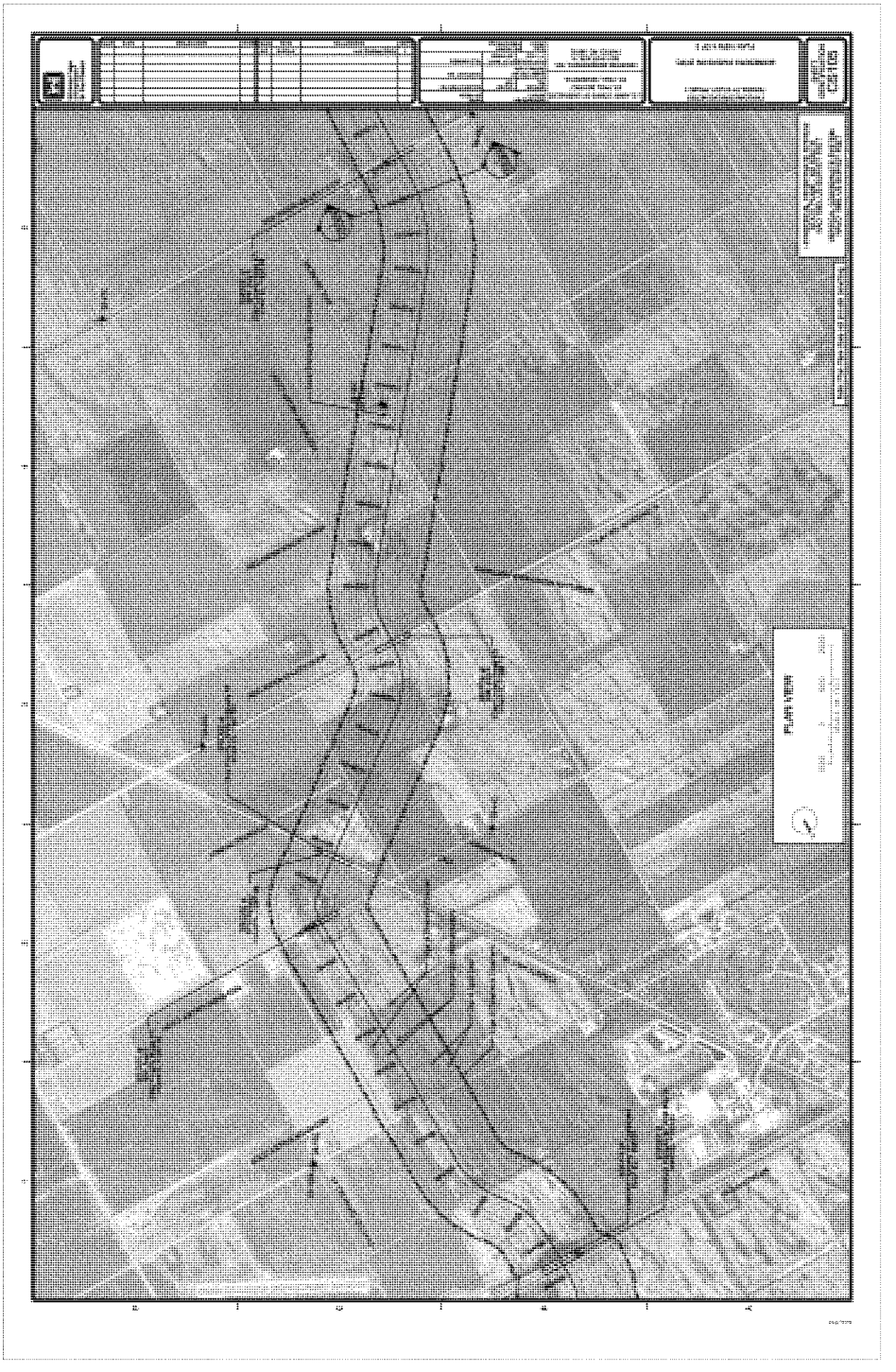
NAD 83, UNIT: US SURVEY FEET
VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET

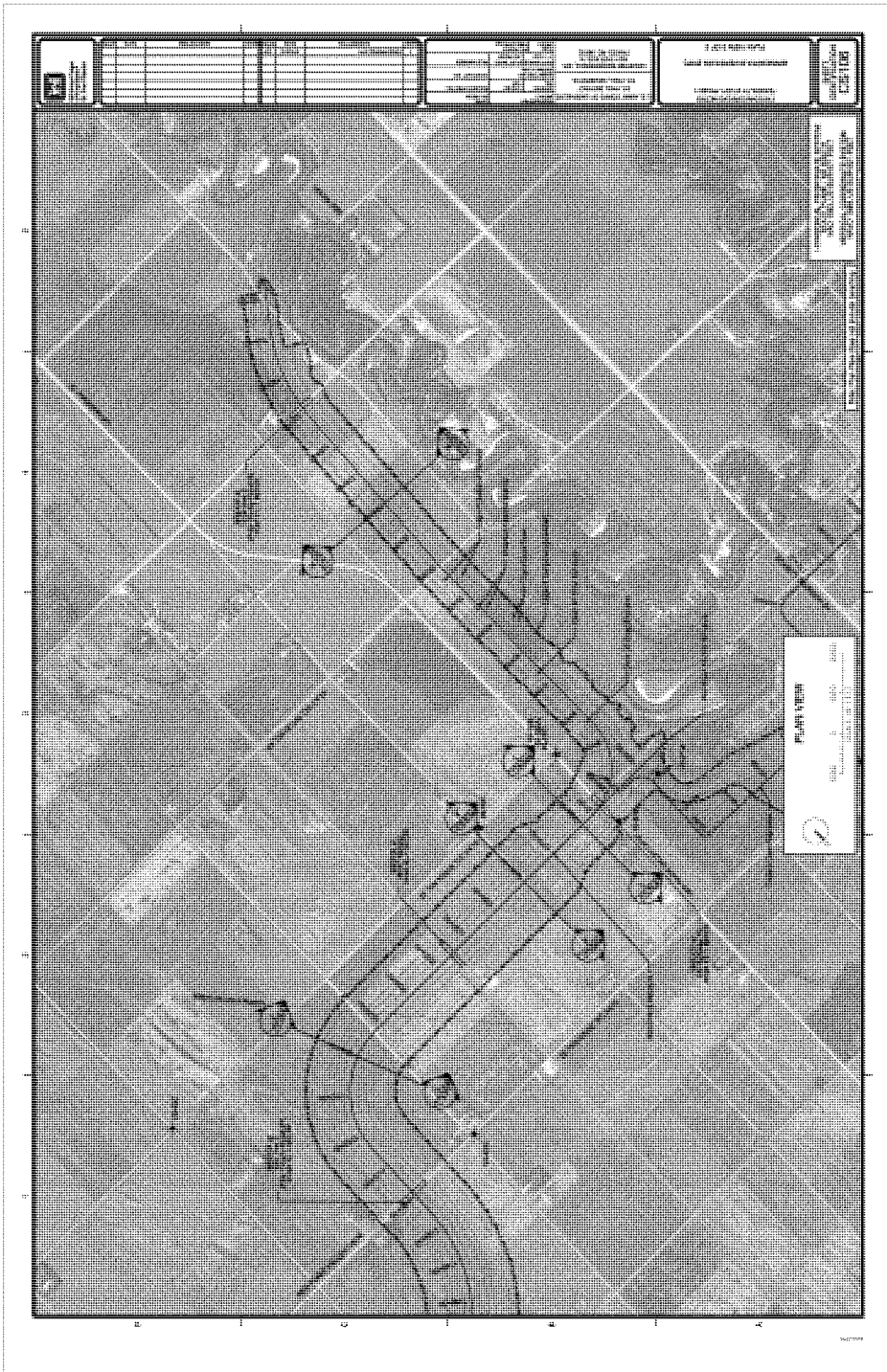


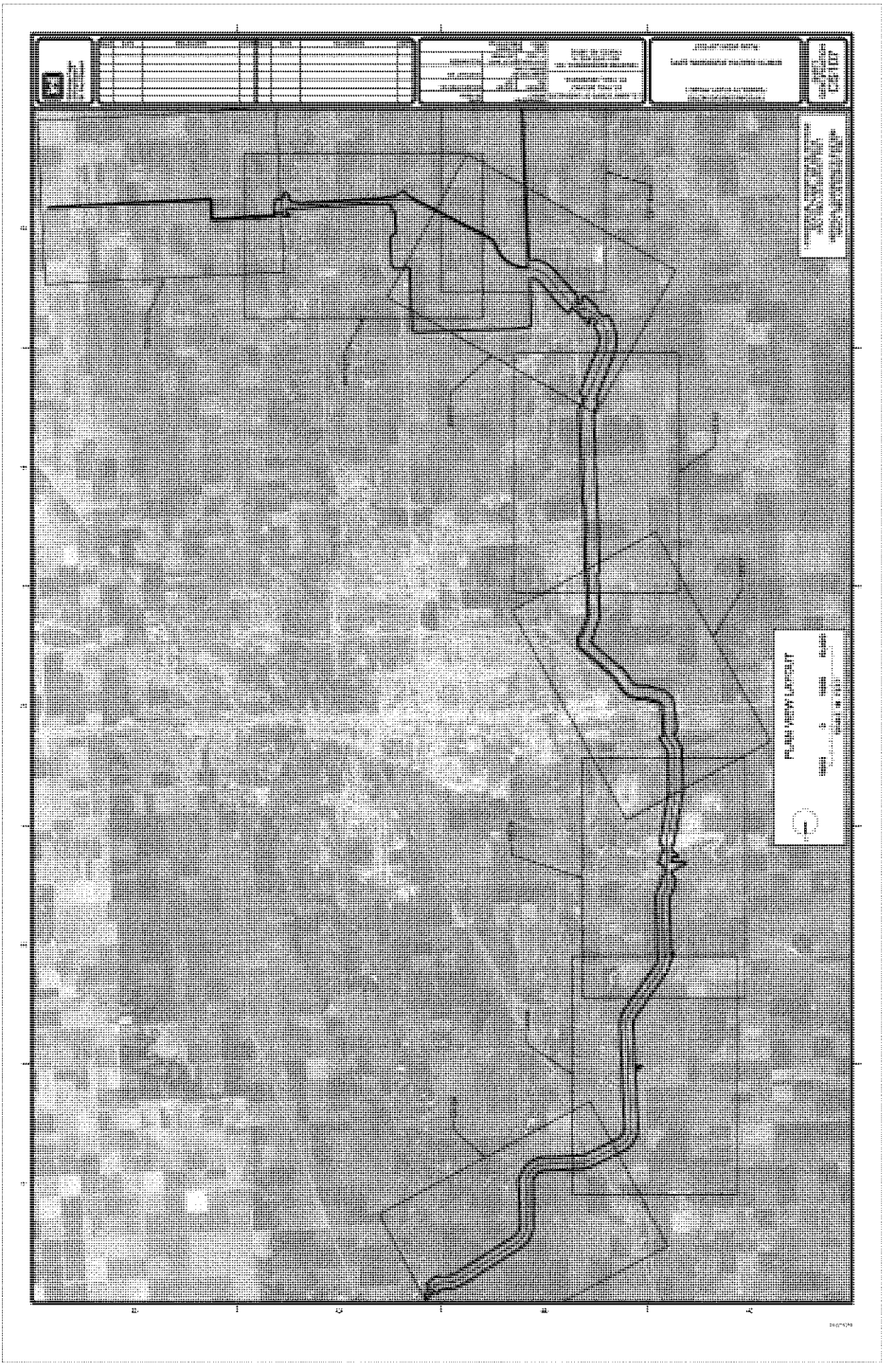


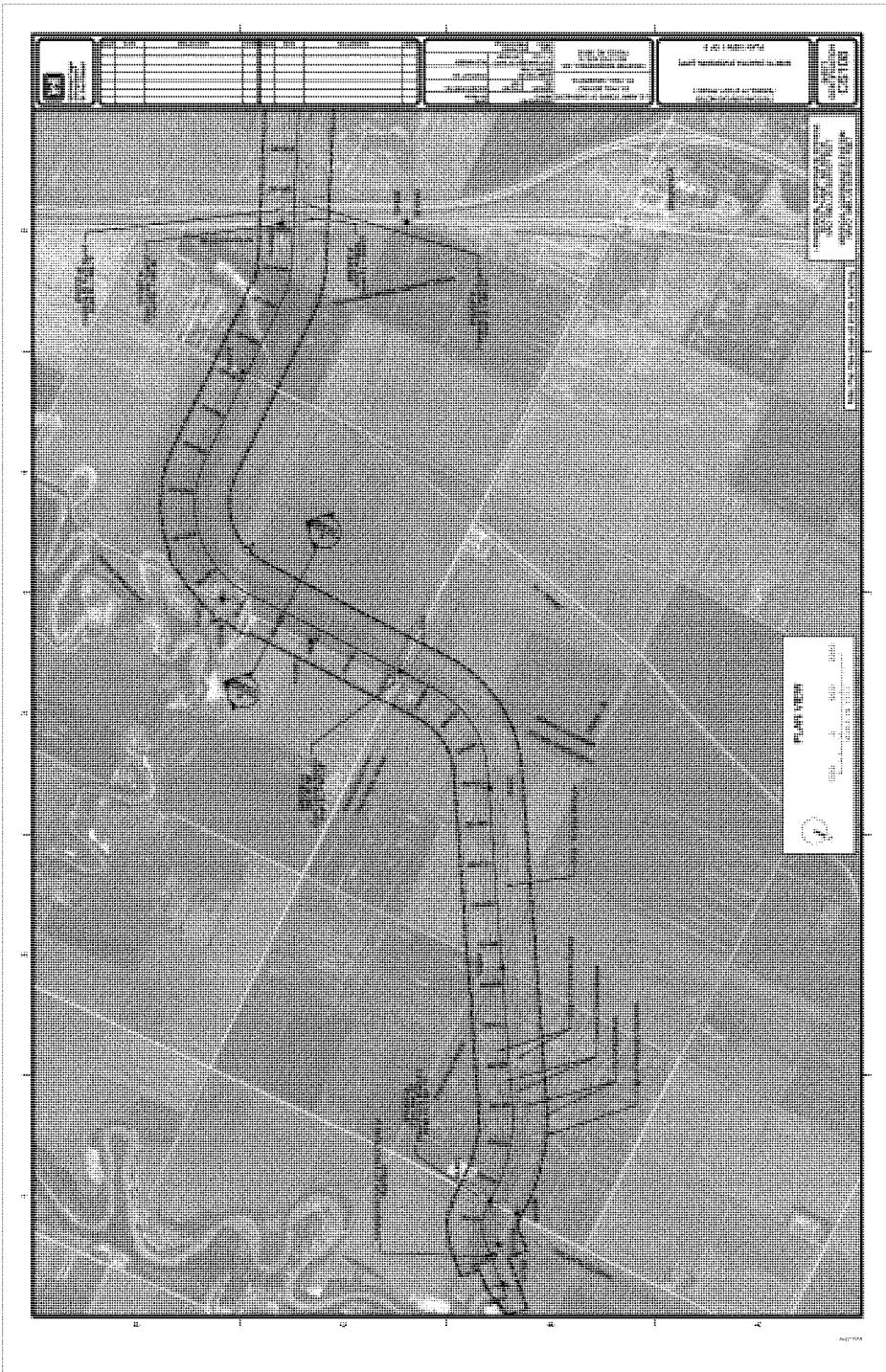


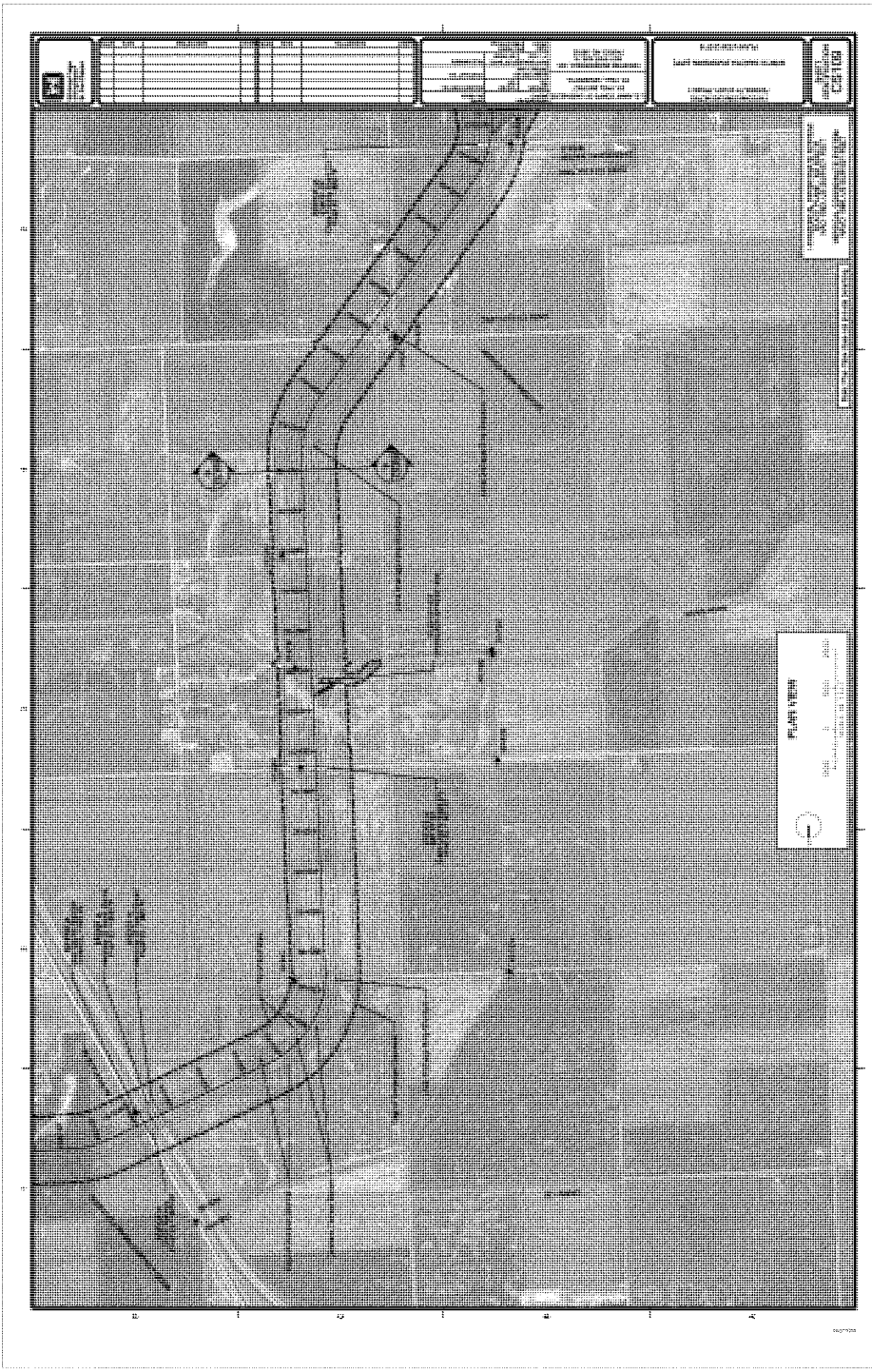


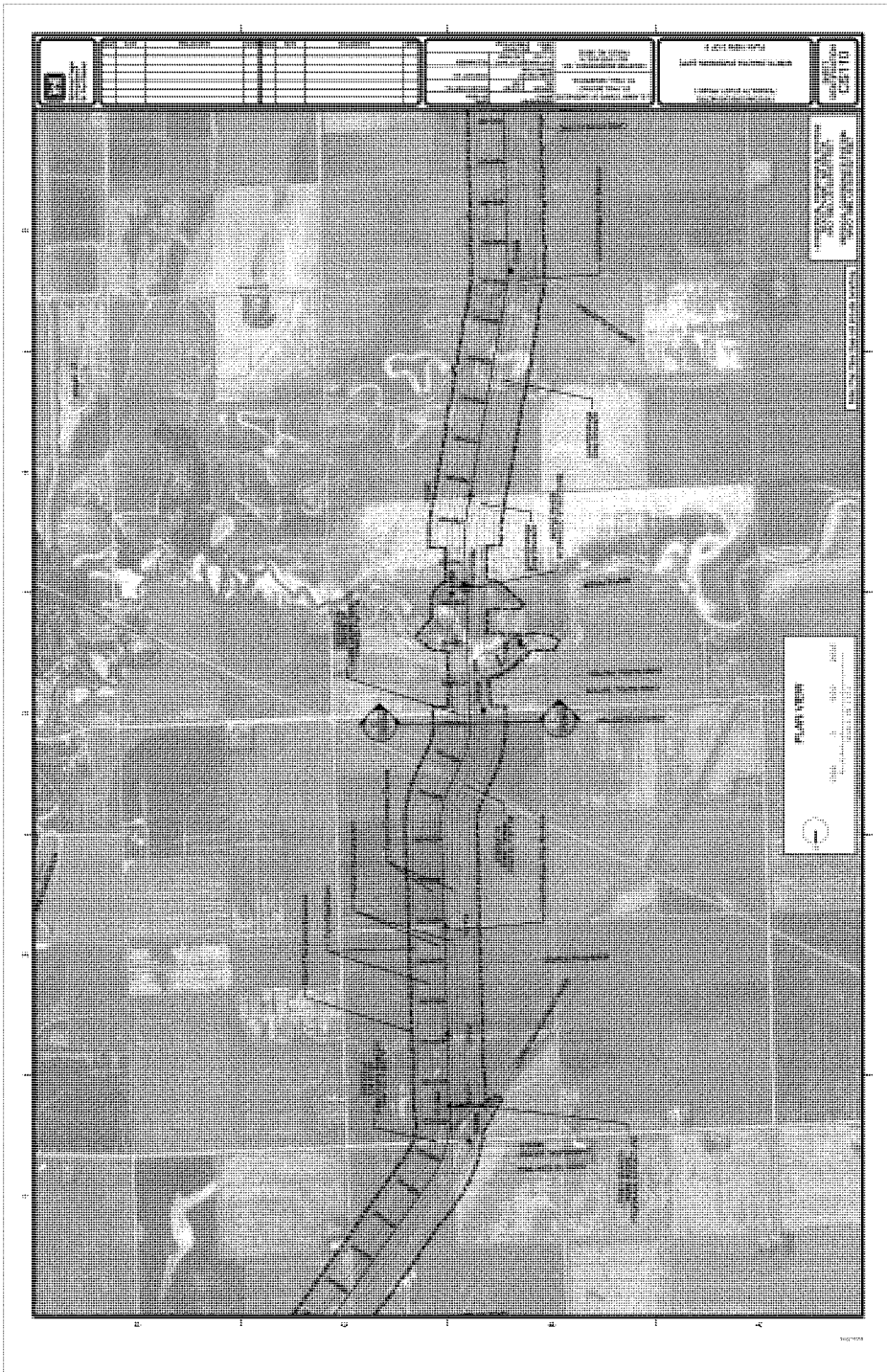


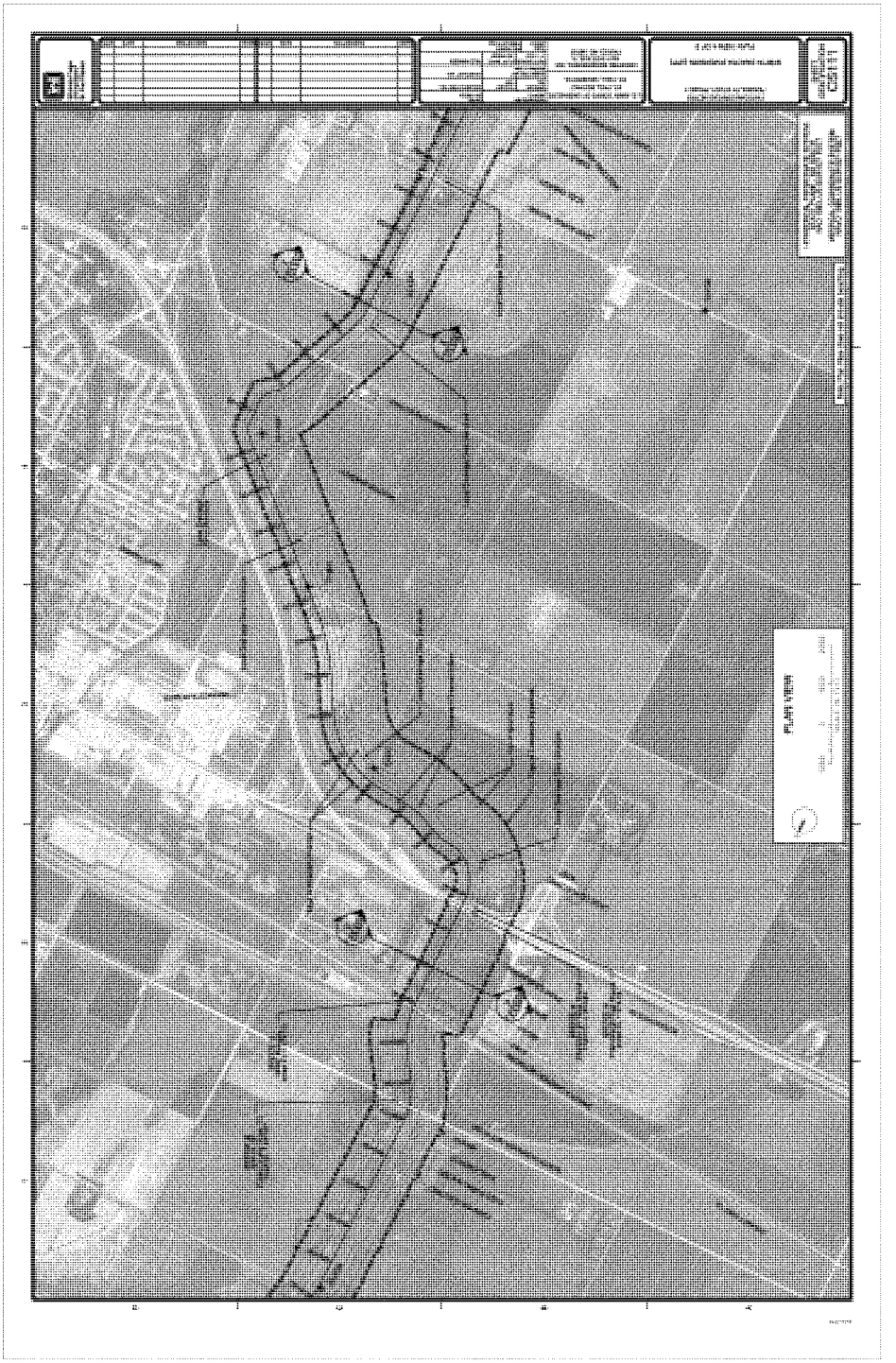


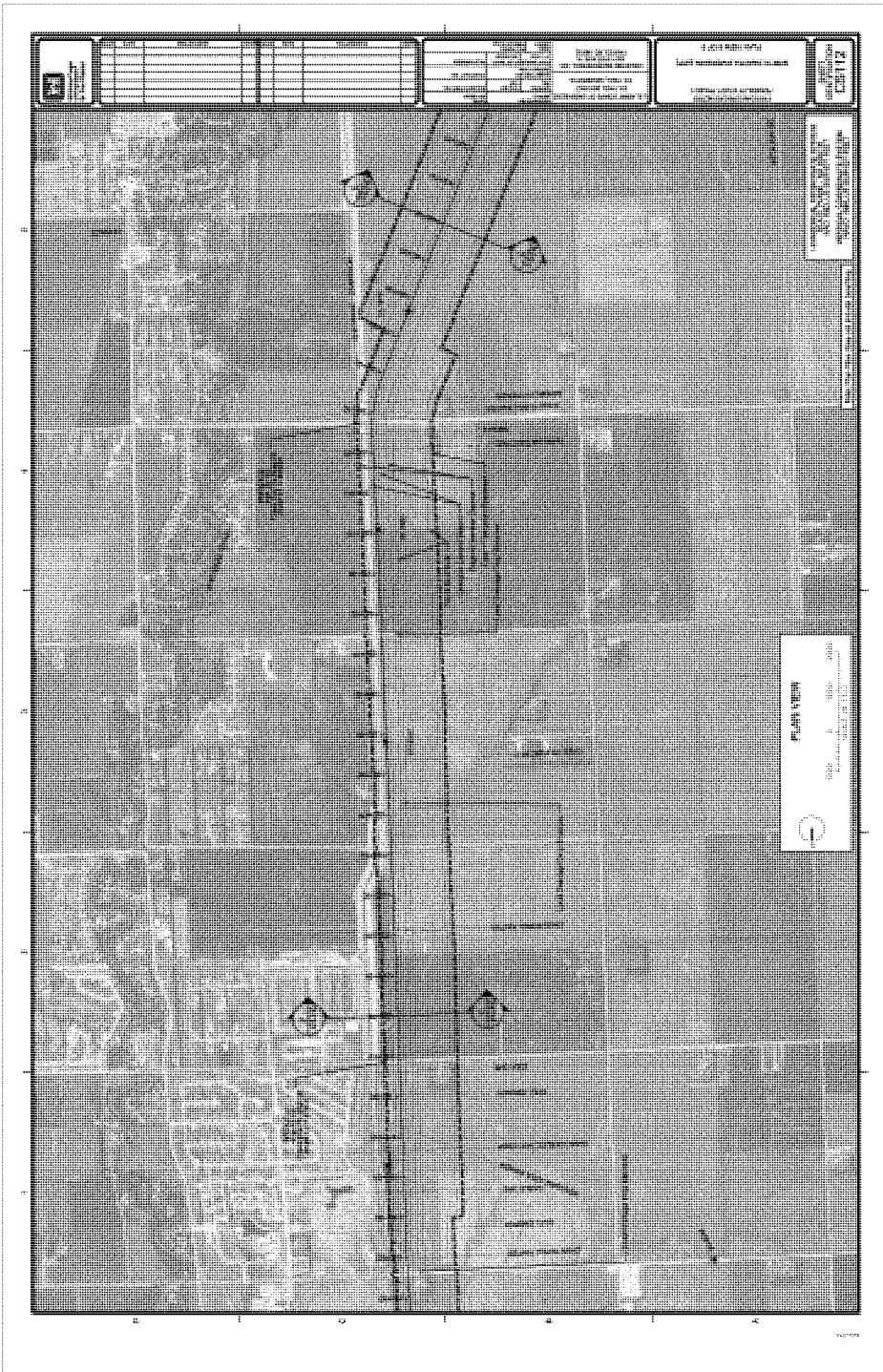


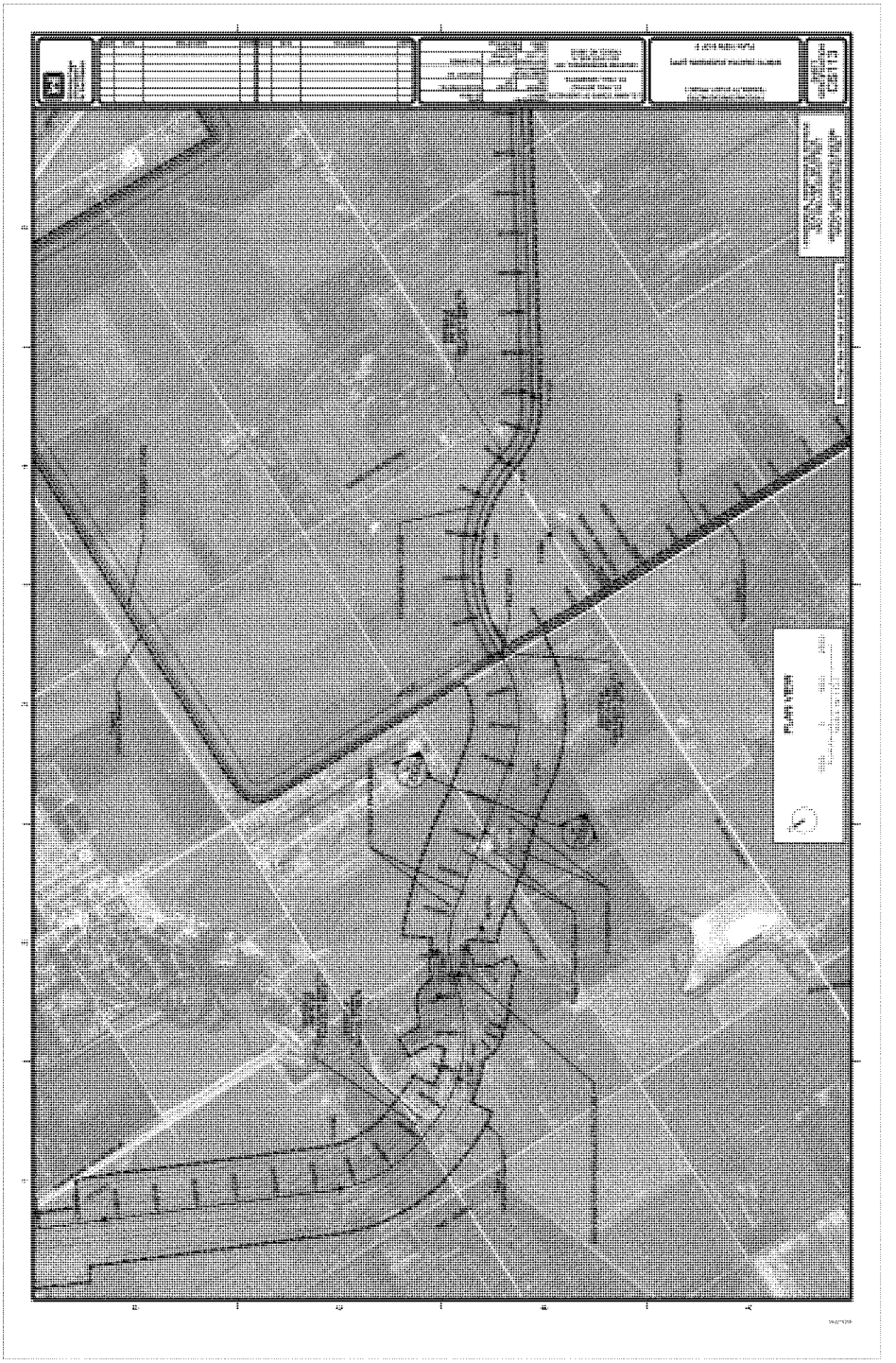


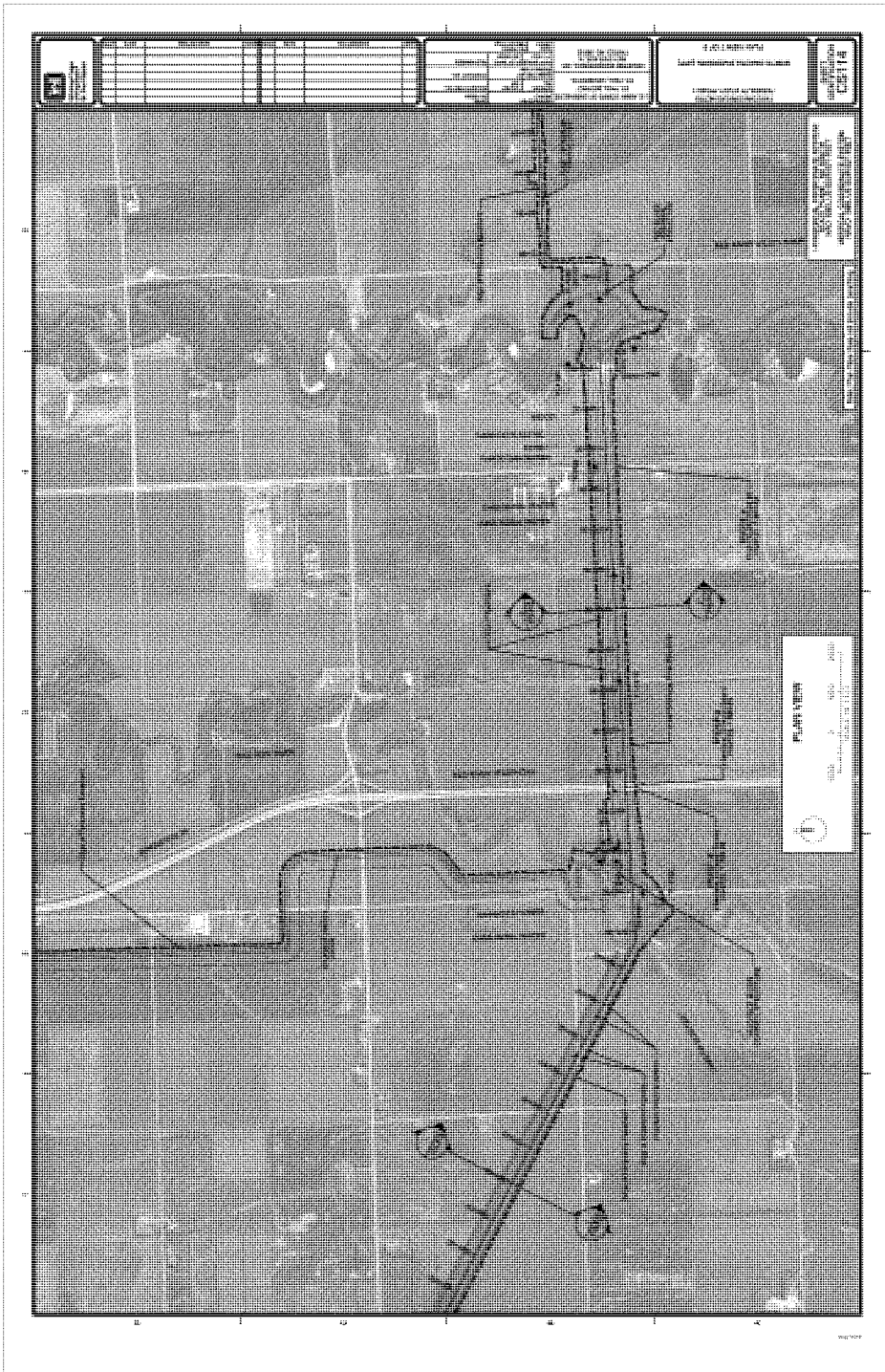


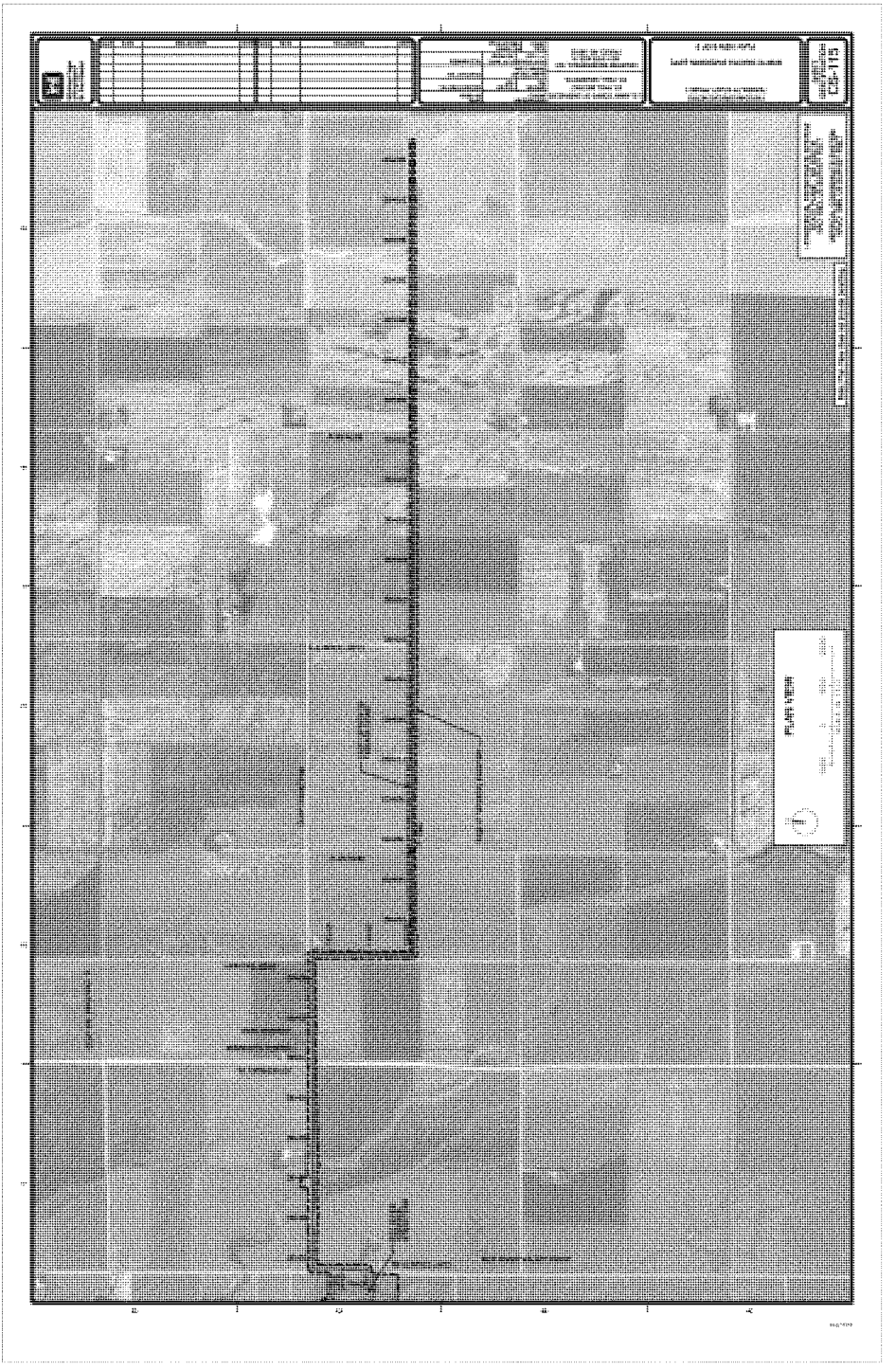


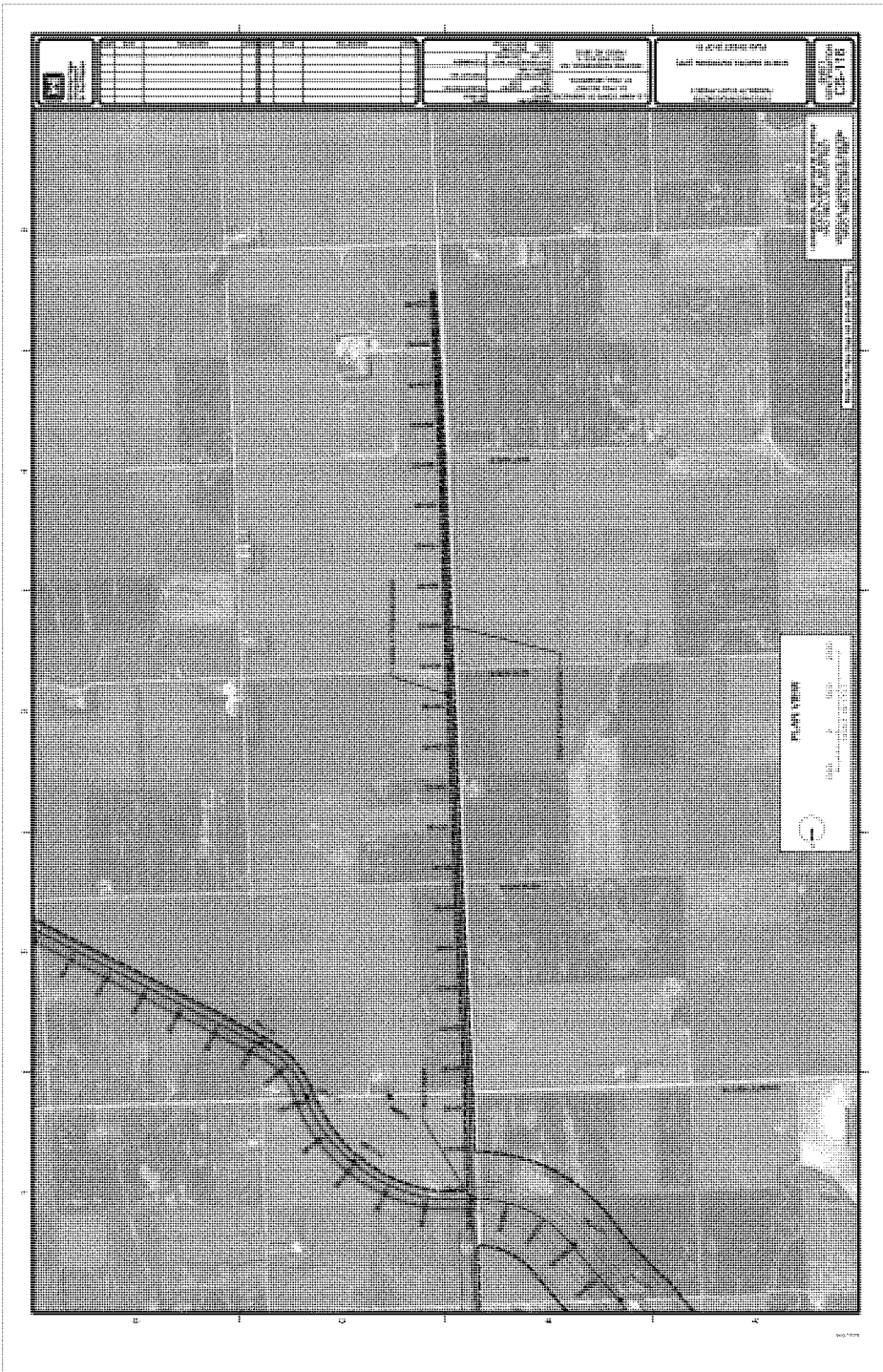












RED RIVER DIVERSION

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

APPENDIX E –BRIDGE STRUCTURES

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

By: HOUSTON ENGINEERING, INC.

FINAL: February 28, 2011

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E2.0 BRIDGE LOCATIONS E-3
E3.0 DESIGN BASIS E-4
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E5.0 COST BASIS E-5
E6.0 RR RELOCATION COSTS E-6

EXHIBITS

A Bridge Length Determination
B Detailed Cost Estimate
C Minnesota Grade Raise Estimate
D Bridge Cost Summary

E1.0 DISCUSSION OF PROJECT FEATURES

The F-M Metro Flood Risk Management Project Feasibility Study, Phase 4 includes the evaluation of two diversion concepts. These include the Minnesota Diversion alternative (Federally Comparable Plan – FCP) and the North Dakota Diversion alternative (Locally Preferred Plan – LPP). This section documents the preliminary bridge design procedure used to develop cost estimates and preliminary bridge layout drawings for the two diversion alternatives under consideration. The two diversion alternatives require differing amounts of bridges as follows:

Alignment	No. of Highway Bridges	No. of RR Bridges
FCP	20	4
LPP	19	4

(Crossings at divided highways such as I-94 are counted as two bridges)

The following table shows the locations of the bridges for each of the alignments.

E2.0 BRIDGE LOCATIONS

BRIDGE LOCATIONS

Minnesota Diversion (FCP)	North Dakota Diversion (LPP)
Location	**Location**
Interstate 29 South Bound	Cass County Road 81
Interstate 29 North Bound	Interstate 29 North Bound
110th Ave. S (C-SAH 8)	Interstate 29 South Bound
US Highway 75	48th St. SE (Cass County Road 16)
BNSF Railway	170th Ave. SE (Cass County Road 17)
80th Ave. S (Clay County Road 67)	RRVW Railway
60th Ave. S (C-SAH 12)	46th St. SE (Cass County Road 14)
County-State Highway 52	44th St. SE (Cass County Road 6)
OTV Railway	41st St. SE (Cass County Road 8)
50th Ave. S (Clay County Road 75)	Interstate 94 East Bound
Interstate 94 East Bound	Interstate 94 West Bound
Interstate 94 West Bound	BNSF Railway
BNSF Railway	36th St. SE (Cass County Road 10)
US Highway 10 East Bound	33rd St. SE (Cass County Road 20)
US Highway 10 West Bound	BNSF Railway
28th Ave. N (C-SAH 18)	31st St. SE (Cass County Road 22)
57th Ave. N (C-SAH 22)	28th St. SE (Cass County Road 32)
40th St. N (Clay County Road 95)	Interstate 29 South Bound
90th Ave. N (Clay County Road 26)	Interstate 29 North Bound
100th Ave. N (C-SAH 5)	BNSF Railway
US Highway 75	County Road 81
BNSF Railway	25th St. SE (Cass County Road 4)
110th Ave. NW (Clay County Road 99)	173rd Ave. SE (Cass County Road 31)
15th St. NW (Clay County Road 100)	

E3.0 DESIGN BASIS

The conceptual bridge designs were developed in accordance with the following specifications and manuals: AASHTO LRFD Bridge Design Specifications, Current Edition; Current MnDOT LRFD Bridge Design Manual ; Current NDDOT LRFD Bridge Design Manual.

The superstructures for all of the bridges are of prestressed concrete girders with cast in place concrete decks. Steel plate girder superstructures were evaluated for comparison, but were found to be more costly than the prestressed concrete bridges, and therefore, are not presented here. The substructures consist of concrete wall piers and concrete abutments supported on steel H-piling. The pile termination elevation was assumed to be approximately 100 feet below existing grade, which is consistent with typical bridges in the area. Adjustments were made in unit prices to reflect the varying heights of the structures thereby accounting for the increased substructure costs based on the height of the bridges.

Approach costs were estimated based on raising the roadways to tie into the proposed bridge elevations. Minimum vertical and horizontal (if applicable) curves were designed and fill quantities were estimated based on the difference between proposed and existing grades. Pavement, guardrail, aggregate base, embankment, and other misc. costs were included in the estimates.

Preliminary designs were performed for each type of bridge (based on roadway classification) for both the FCP and LPP alternatives.

The bridge types and associated width are as follows:

MN Divided Highway (I-94 and US 10)	42' clear roadway
MN State Highway	48' clear roadway
MN Local Roads	32' clear roadway
ND Divided Highway (I-29 and I-94)	40' clear roadway
ND Local Roads	28' to 36' clear roadway

E4.0 BRIDGE LENGTH DETERMINATION

Bridge lengths were calculated for each bridge location for each of the alternatives using Excel spreadsheets. The bridge lengths were based on the channel bottom width and elevation, channel slopes consistent with the rest of the channel geometrics (7:1 slopes, bench, then 7:1 slopes), and the estimated deck elevation. The deck elevation was computed by taking the 500-year water surface elevation at the structure and adding freeboard and the superstructure depth. The freeboard was assumed to be 3'-0" and the superstructure depth was approximately 5'-5". The water surface elevations, channel

bottom elevations, and existing ground elevations were taken from the HEC-RAS models provided by Moore Engineering, Inc.

For some bridge locations, the calculated deck elevation was lower than the existing ground elevation. In those cases, the bridge length was based on matching the bridge deck elevation to the existing ground elevation.

Exhibit A shows the Bridge Length Determination spreadsheets for the Minnesota Diversion (FCP) and North Dakota Diversion (LPP) alternatives. The calculated bridge lengths were averaged and rounded to the nearest 20 feet.

E5.0 COST BASIS

A detailed cost estimate was performed for two bridges of varying length and span counts to establish an average superstructure unit cost per square foot of bridge deck. The superstructure unit cost for each bridge type was then applied to the bridge widths and lengths determined for each bridge location. Site specific pier costs were calculated and added to the superstructure cost to account for the various pier heights and span counts found at each bridge. Pier heights for each bridge were assumed to be the same independent of channel slope to be conservative for estimating. The quantities used in the detailed cost estimates were estimated using an Excel spreadsheet, with estimated dimensions of all of the bridge components. The unit prices used in the detailed cost estimates were based on recent average bid prices obtained from websites of the Minnesota and North Dakota Departments of Transportation. See Exhibit B for a Detailed Cost Estimate example spreadsheet.

In addition to the bridge costs, the detailed cost estimates also include costs to construct a temporary bypass or crossover for the road so that traffic can be maintained during construction of the bridge. The bypass cost estimates assumed that the bypass would be constructed along the existing grade.

The last element included in the cost estimates was the approximate costs for grade raises for the bridge sites that would require building the bridge above the existing ground elevation. The quantities for these estimates were based on a maximum grade of 2.5% for highways. See Exhibit C for an example MN Grade Raise Cost Estimate Spreadsheet.

The results of all of the cost estimates are summarized in Exhibit D.

E6.0 RAILROAD BRIDGE COST ESTIMATES

The railroad bridge locations for both the Minnesota Diversion (FCP) and North Dakota Diversion (LPP) Alternatives are listed in the above table for reference only. The costs estimates associated with these structures were developed separately and are included in Appendix G.

Minnesota Diversion (FCP) Alignment

			Station:	0+00	0+00	131+52	1286+89	1060+72	1035+21	977+56	960+10	854+06	853+15	743+21	742+27	635+18	515+72	444+11	299+74	245+18	195+22	90+36	6+90
			Feature Carried:	Interstate 29 South Bound	Interstate 29 North Bound	110th Ave. S	US Highway 75	80th Ave. S	60th Ave. S	County-State Highway 52	50th Ave. S	Interstate 94 East Bound	Interstate 94 West Bound	US Highway 10 East Bound	US Highway 10 West Bound	28th Ave. N	57th Ave. N	40th St. N	90th Ave. N	100th Ave. N	US Highway 75	110th Ave. NW	15th St. NW
A			Invert Elevation:	898.09	898.09	898.09	889.60	891.66	889.15	887.99	887.65	885.50	885.50	883.40	883.40	882.33	877.13	876.42	874.98	874.43	873.92	872.88	872.04
B			Q 500 WS Elevation	914.87	914.87	914.87	914.02	910.29	908.02	907.04	906.75	905.1	905.1	903.58	903.58	902.12	900.36	899.32	897.12	896.24	895.38	893.24	890.96
C			Approximate Ground Elevation:	912.64	916.03	916.79	913.22	913.2	918.26	917.01	918.4	914.64	914.76	914.23	913.5	913.45	902.11	903.39	895.85	892.52	892.45	888.86	887.37
D			Q 500 Depth (ft):	15.40	15.40	15.40	21.35	16.87	16.70	17.90	18.60	17.72	17.72	19.93	19.93	19.80	22.48	21.80	20.91	20.64	20.38	19.62	17.73
E			Required Freeboard (ft):	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
F			Abutment Setback (ft):	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
G			Abutment Thickness (ft):	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4	4
H			* Skew Angle (degrees):	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0	0
I			Skew Direction:																				
J			Girder Depth (in):	54	54	54	54	54	54	54	54	54	54	54	54	54	54	54	54	54	54	54	54
K			Deck Thickness plus Riser (in):	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11
L			Grade Raise Needed (ft):	10.6	7.3	6.5	9.2	5.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	6.7	4.3	9.7	12.1	13.9	12.8	12.0
M	Channel Dimensions at Crossing		Bottom Width (ft):	100	100	215	225	400	400	400	400	400	400	400	400	400	225	225	225	225	225	225	225
N			Side Slope Below Bench (_H:1V):	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10	10
O			Bench Width (ft):	0	0	0	70	70	70	70	70	70	70	70	70	70	70	70	70	70	70	70	70
P			Bench Height (ft):	0	0	0	7	7	7	7	7	7	7	7	7	7	7	7	7	7	7	7	7
Q			Side slope Above Bench (_H:1V):	7	7	7	7	7	7	7	7	10	10	10	10	10	7	7	7	7	7	7	7
R			Estimated Bridge Length (EWS to EWS) (ft):	300.6	300.6	415.6	690.9	803.2	800.8	817.6	827.4	867.4	867.4	911.6	911.6	889.0	706.7	697.2	684.7	681.0	677.3	666.7	640.2
See Sheet S-201 for details			Bridge Length to Use for Design (ft):	300	300	420	680	800	800	820	820	860	860	900	900	880	700	700	680	680	680	660	640
			Bridge Deck Width (ft):	44.5	44.5	34.5	50.5	34.5	34.5	34.5	34.5	44.5	44.5	44.5	44.5	34.5	34.5	34.5	34.5	34.5	50.5	34.5	34.5
			Deck Surface Area (ft^2):	13350	13350	14490	34340	27600	27600	28290	28290	38270	38270	40050	40050	30360	24150	24150	23460	23460	34340	22770	22080

* Skew Angles are measured for estimating bridge lengths at specific crossings. Actual skews used for final design shall follow MN DOT Design Manual guidelines.

North Dakota Diversion (LPP) Alignment

		Station:	1885+15.00	1806+15.00	1805+05.00	1640+45.00	1587+20.00	1461+20.00	1347+45.00	1188+70.00	917+55.00	916+65.00	863+75.00	702+75.00	595+15.00	425+75.00	316+35.00	315+30.00	311+75.00	167+55.00	25+10
		Featura Carried:	County Road 81	Interstate 29 North Bound	Interstate 29 South Bound	48th St. SE	170th Ave. SE	46th St. SE	44th St. SE	41st St. SE	Interstate 94 East Bound	Interstate 94 West Bound	36th St. SE	33rd St. SE	31st St. SE	28th St. SE	Interstate 29 South Bound	Interstate 29 North Bound	County Road 81	25th St. SE	173rd Ave. SE
A		Invert Elevation:	898.30	899.88	899.90	900.75	883.98	882.31	880.66	878.37	874.44	874.43	873.67	871.34	869.78	867.33	865.74	865.73	865.68	863.59	861.53
B		Q 500 WS Elevation:	921.73	921.72	921.72	920.58	904.45	901.51	900.19	898.58	896.42	896.42	896.09	893.16	891.61	889.34	887.81	887.81	887.81	885.83	884.08
C		Approximate Ground Elevation:	915.32	915.99	916.03	916.98	918.41	915.52	913.30	903.74	908.50	908.50	902.16	897.06	894.44	890.89	888.42	888.29	887.34	885.87	883.21
D		Q 500 Depth (ft):	23.43	21.84	21.82	19.83	20.47	19.20	19.53	20.21	21.98	21.99	22.42	21.82	21.83	22.01	22.07	22.08	22.13	22.24	22.55
E		Required Freeboard (ft):	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3	3
F		Abutment Setback (ft):	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5	2.5
G		Abutment Thickness (ft):	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0	4.0
H		* Skew Angle (degrees):	0	0	0	20	0	21	7	0	5	5	13	0	0	0	0	0	0	0	0
I		Skew Direction:				L		L	R		L	L	L								
J		Girder Depth (in):	54	54	54	54	54	54	54	54	54	54	54	54	54	54	54	54	54	54	54
K		Deck Thickness plus Riser (in):	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11	11
L		Grade Raise Needed (ft):	14.8	14.1	14.1	12.0	0.0	0.0	0.0	3.3	0.0	0.0	2.3	4.5	5.6	6.9	7.8	7.9	8.9	8.4	9.3
M	Channel Dimensions at Crossing	Bottom Width (ft):	250	250	250	100	250	250	250	250	250	250	250	250	250	250	250	250	250	250	250
N		Side Slope Below Bench (_H:1V):	7	7	7	7	7	7	7	7	7	7	7	7	7	7	7	7	7	7	7
O		Bench Width (ft):	0	0	0	0	0	40	40	40	25	25	25	25	15	15	0	0	0	0	0
P		Bench Height (ft):	0	0	0	0	0	8	8	8	8	8	8	8	8	8	0	0	0	0	0
Q		Side slope Above Bench (_H:1V):	7	7	7	7	7	7	7	7	7	7	7	7	7	7	7	7	7	7	7
R		Estimated Bridge Length (EWS to EWS) (ft):	605	583	582	431	641	752	702	640	689	689	659	632	613	615	586	586	587	588	593
See Sheet S-201 for details		Bridge Length to Use for Design (ft):	600	580	580	440	640	760	700	640	680	680	660	640	620	620	580	580	580	580	600
		Bridge Deck Width (ft):	38.5	42.5	42.5	32.5	30.5	32.5	30.5	38.5	42.5	42.5	38.5	38.5	30.5	30.5	42.5	42.5	38.5	32.5	30.5
		Deck Surface Area (ft²):	23100	24650	24650	14300	19520	24700	21350	24640	28900	28900	25410	24640	18910	18910	24650	24650	22330	18850	18300

* Skew Angles are measured for estimating bridge lengths at specific crossings. Actual skews used for final design shall follow ND DOT Design Manual guidelines.

Example Bridge 1

Summary of Quantities: 8-span, 760' bridge MN-Rural

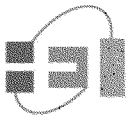
Item:		Excavation		Concrete		Reinforcing Steel		Prestressed Beam	Steel Pile	Bridge Backwall Protection	Abutment Strip Drain	Elastomeric Bearing	Expansion Device	Substructure Water proofing	Approach Pavement (10in)	Approach Pavement Footing
		Class I (dry)	Class II (wet)	4 ksi AE-3	4 ksi AAE-3	Epoxy	Non									
Location		CYD	CYD	CYD	CYD	LBS	LBS	LIN FOOT	LIN FOOT	SQYD	SQYD	EACH	LIN FOOT	SQYD	SQYD	CYD
Substructure	Abut 1	95		64.5		**			630	47	43					
	Pier 1	50	162	130.5			24010		1440							
	Pier 2	40	162	130.5			24010		1440							
	Pier 3	30	162	130.5			24010		1440							
	Pier 4	30	162	130.5			24010		1440			10		24		
	Pier 5	30	162	130.5			24010		1440							
	Pier 6	40	162	130.5			24010		1440							
	Pier 7	50	162	130.5			24010		1440							
	Abut 2	95		64.5		**			630	47	43					
	Totals	460	1134	1042.5			168070		11340	94	86	10		24		
	Unit Price	$12.50	$39.00	$525.00	$625.00	$1.15	$1.10	$210.00	$45.00	$25.00	$31.00	$1,880.00	$940.00	$58.61	$150.00	$287.00
	Total	$5,750	$44,226	$547,313			$184,877		$510,300	$2,350	$2,666	$18,800		$1,407		
Superstructure	Girder							3760								
	Deck				1004.9	232300							68		98	
	Totals				1004.9	232300		3760					68		98	
	Unit Price	$12.50	$39.00	$525.00	$625.00	$1.15	$1.10	$210.00	$45.00	$25.00	$31.00	$1,880.00	$940.00	$58.61	$150.00	$287.00
	Total				$628,063	$267,145		$789,600					$63,920		$14,700	

Example Bridge 2

Summary of Quantities: 5-span , 450' MN-Rural

Item:		Excavation		Concrete		Reinforcing Steel		Prestressed Beam	Steel Pile	Bridge Backwall Protection	Abutment Strip Drain	Elastomeric Bearing	Expansion Device	Substructure Water proofing	Aproach pavement (10in)	Approach pavement Footing
		Class I (dry)	Class II (wet)	4 ksi AE-3	4 ksi AAE-3	Epoxy	Non									
Location		CYD	CYD	CYD	CYD	LBS	LBS	LIN FOOT	LIN FOOT	SQYD	SQYD	EACH	LIN FOOT	SQYD	SQYD	CYD
Substructure	Abut 1	95		17.6		**			630	47	43					
	Pier 1	50	162	130.5			24010		1260							
	Pier 2	40	162	130.5			24010		1260							
	Pier 3	40	162	130.5			24010		1260							
	Pier 4	50	162	130.5			24010		1260							
	Abut 2	95		17.6		**			630	47	43					
	Totals	370	648	557.2			96040		6300	94	86					
	Unit Price	$12.50	$39.00	$525.00	$625.00	$1.15	$1.10	$210.00	$45.00	$25.00	$31.00	$1,880.00	$940.00	$58.61	$150.00	$287.00
	Total	$4,625	$25,272	$292,530			$105,644		$283,500	$2,350	$2,666					
Superstructure	Girder							2275							250	57
	Deck				618	162360										
	Totals				618	162360		2275							250	57
	Unit Price	$12.50	$39.00	$525.00	$625.00	$1.15	$1.10	$210.00	$45.00	$25.00	$31.00	$1,880.00	$940.00	$58.61	$150.00	$287.00
	Total				$386,250	$186,714		$477,750							$37,500	$16,359

** Included in Superstructure quantities.



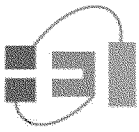
Houston Engineering Inc.

Exhibit - B
Detailed Cost Estimate

Total Structure costs							
Bridge	Super Structure	Substructure	Total Bridge	Length (ft)	Deck width (ft)	Deck area (SQFT)	Unit price ($/SQFT)
760 ft	$1,763,428	$1,317,688	$3,081,116	760.5	34.5	26237	$117.43
450 ft	$1,104,573	$716,587	$1,821,160	460.5	34.5	15887	$114.63

Pier only costs							
Bridge	Class I (dry)	Class II (wet)	4 ksi	Re-Steel 60ksi	Piling	total	per pier
760 ft	$3,375	$44,226	$479,588	$184,877	$453,600	$1,165,666	$166,524
450 ft	$2,250	$25,272	$274,050	$105,644	$226,800	$634,016	$158,504

Superstructure Unit cost without Piers					
Bridge	Total Bridge Cost	Pier total	Superstructure only	Deck area (SQFT)	Unit price ($/SQFT)
760 ft	$3,081,116	$1,165,666	$1,915,450	26237	$73
450 ft	$1,821,160	$634,016	$1,187,144	15887	$75



HoustonEngineering Inc.

Exhibit - C FMM Diversion Feasibility Study - Phase 4
MN Grade Raise Cost Estimate

Example Grade Raise Cost Estimate

	Route:	US Highway 75 Over FCP Alignment	
	Existing Roadway grade Elevation:	892.45	
	Estimated Grade Raise Needed:	13.9	ft
	Existing grade:	0.0	%
	Approach grade:	2.40	%
	Exit Grade:	-2.40	%
crest	VC K:	312	
	Curve Length:	1499.77	
	VC K:	206	
Sag	Curve Length:	495.12	
	Bridge Length:	680	ft
	Calculated appoach Length:	905	ft

Bid Item Quantities

Estimated Approach Fill per abutment:	19000	CYD	(Estimated using Geopak)
Pavement Base Course per abutment:	530	CYD	
Asphalt per abutment:	710	Ton	
Existing Pavement Removal:	13900	Sq. Yd	
W Beam Guard rail:	480	ft	
Guard Rail End Sections:	4		

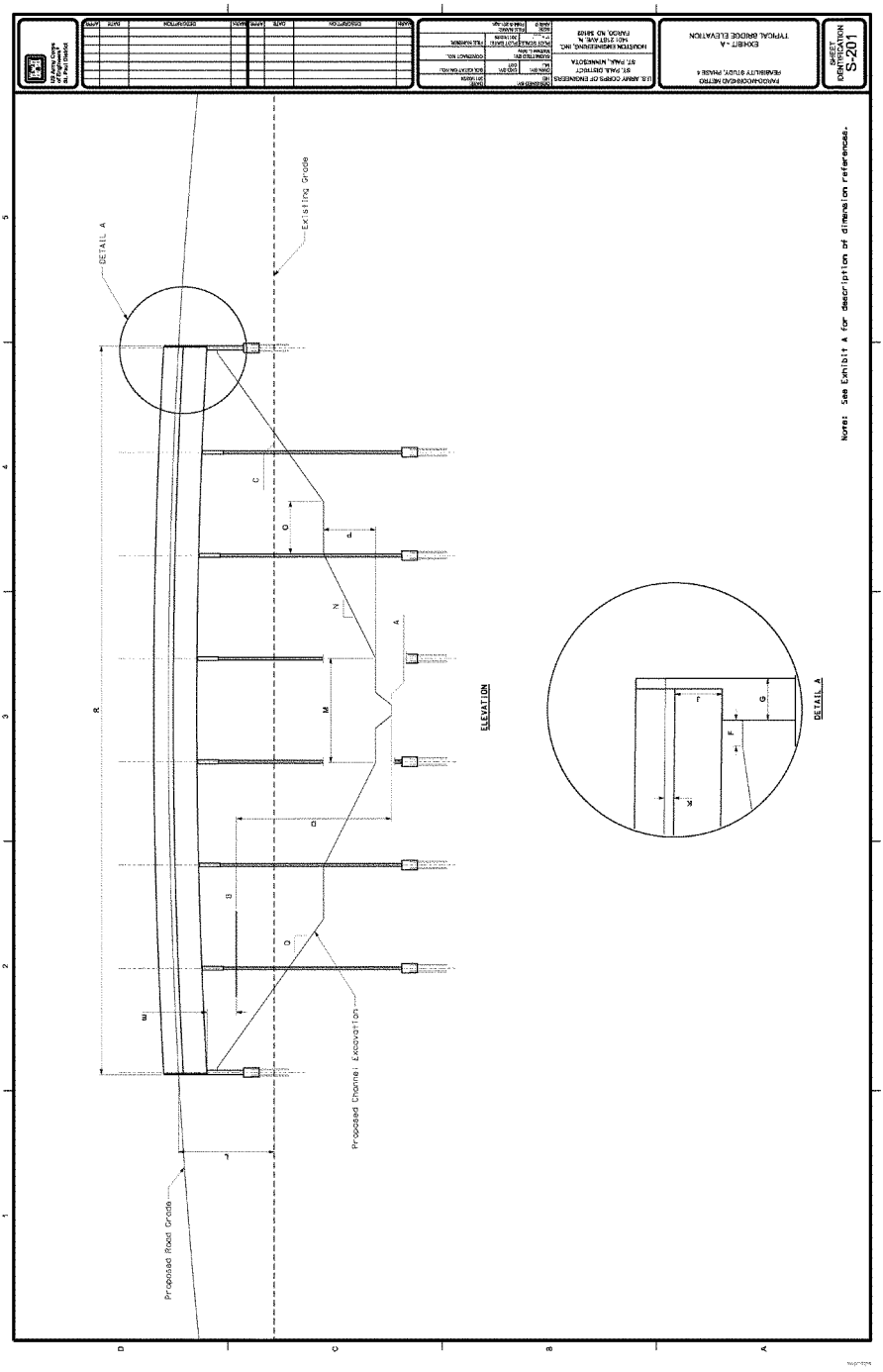
Bid Item	Total Quantity	Unit Price	Subtotal
Approach fill	38000	$10	$380,000
Pavement Base Course	1060	$36	$38,160
Asphalt	1420	$50	$71,000
Existing Pavement Removal	13900	$3	$41,700
W Beam Guard Rail	480	$33	$15,840
Guardrail End Section	4	$2,250	$9,000
		Total Approach Costs =	$555,700

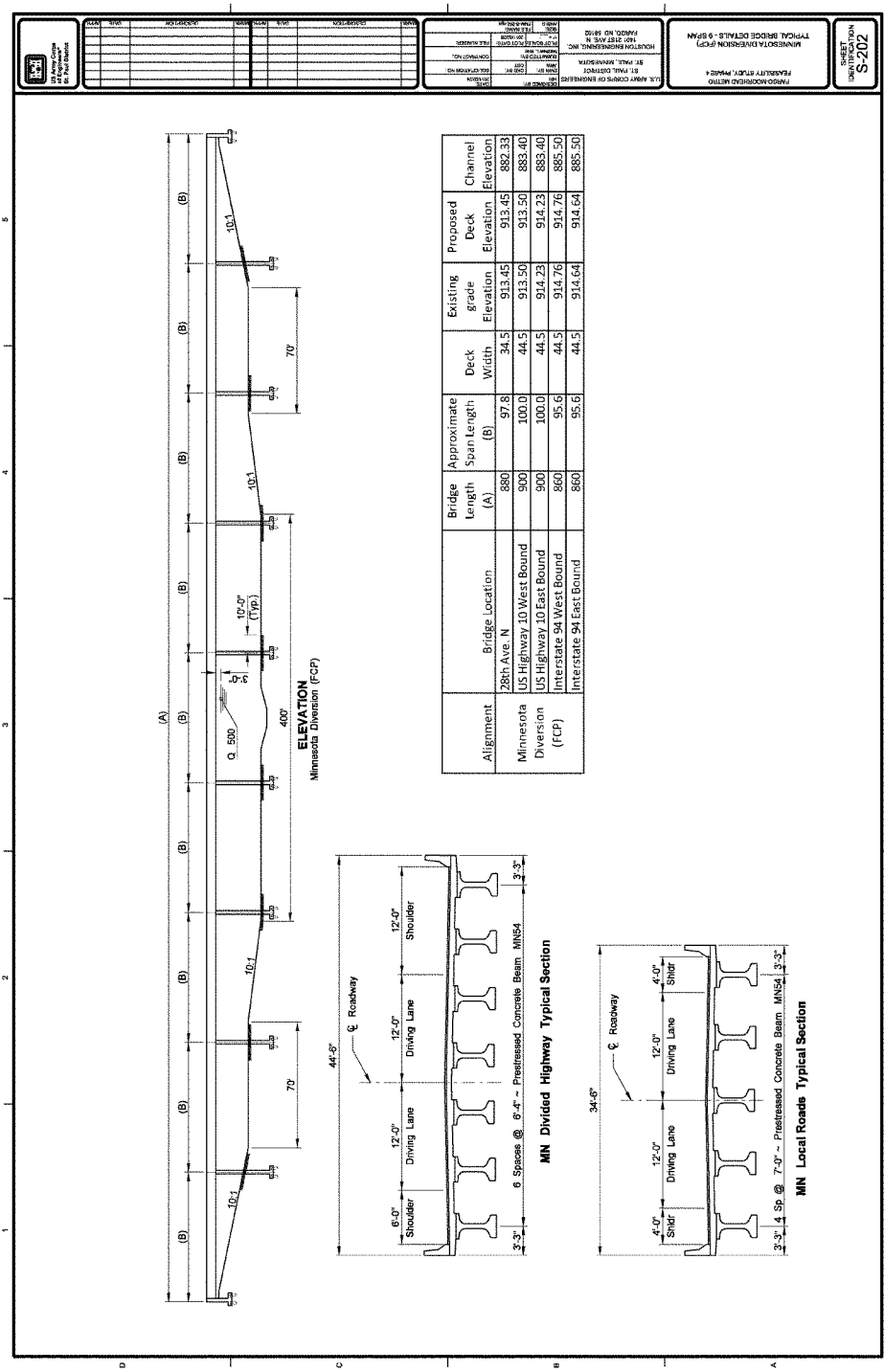
Minnesota Diversion (FCP) Bridge Costs Summary

Location	Bridge Length (ft)	Bridge Deck Area (ft^2)	Cost per ft^2 Deck Area	Bridge Structure Cost	Existing Structure Removal	Approach Road Cost	Temporary Bypass Cost	Total Cost per Bridge
Interstate 29 South Bound	300	13,350	125	$1,670,000	$0	$670,000	$230,000	$2,570,000
Interstate 29 North Bound	300	13,350	125	$1,670,000	$0	$500,000	$230,000	$2,400,000
110th Ave. S	420	14,490	115	$1,670,000	$0	$290,000	$270,000	$2,230,000
US Highway 75	680	34,340	110	$3,780,000	$0	$390,000	$310,000	$4,480,000
80th Ave. S	800	27,600	110	$3,040,000	$0	$260,000	$300,000	$3,600,000
60th Ave. S	800	27,600	115	$3,170,000	$0	$70,000	$250,000	$3,490,000
County-State Highway 52	820	28,290	115	$3,250,000	$0	$70,000	$250,000	$3,570,000
50th Ave. S	820	28,290	115	$3,250,000	$0	$50,000	$230,000	$3,530,000
Interstate 94 East Bound	860	38,270	115	$4,400,000	$0	$160,000	$230,000	$4,790,000
Interstate 94 West Bound	860	38,270	115	$4,400,000	$0	$160,000	$230,000	$4,790,000
US Highway 10 East Bound	900	40,050	115	$4,610,000	$0	$80,000	$260,000	$4,950,000
US Highway 10 West Bound	900	40,050	115	$4,610,000	$0	$80,000	$260,000	$4,950,000
28th Ave. N	880	30,360	120	$3,640,000	$0	$70,000	$260,000	$3,970,000
57th Ave. N	700	24,150	115	$2,780,000	$0	$300,000	$300,000	$3,380,000
40th St. N	700	24,150	115	$2,780,000	$0	$220,000	$280,000	$3,280,000
90th Ave. N	680	23,460	110	$2,580,000	$0	$400,000	$320,000	$3,300,000
100th Ave. N	680	23,460	110	$2,580,000	$0	$480,000	$340,000	$3,400,000
US Highway 75	680	34,340	110	$3,780,000	$0	$560,000	$350,000	$4,690,000
110th Ave. NW	660	22,770	110	$2,500,000	$0	$500,000	$340,000	$3,340,000
15th St. NW	640	22,080	110	$2,430,000	$0	$480,000	$330,000	$3,240,000
							Total	$73,950,000

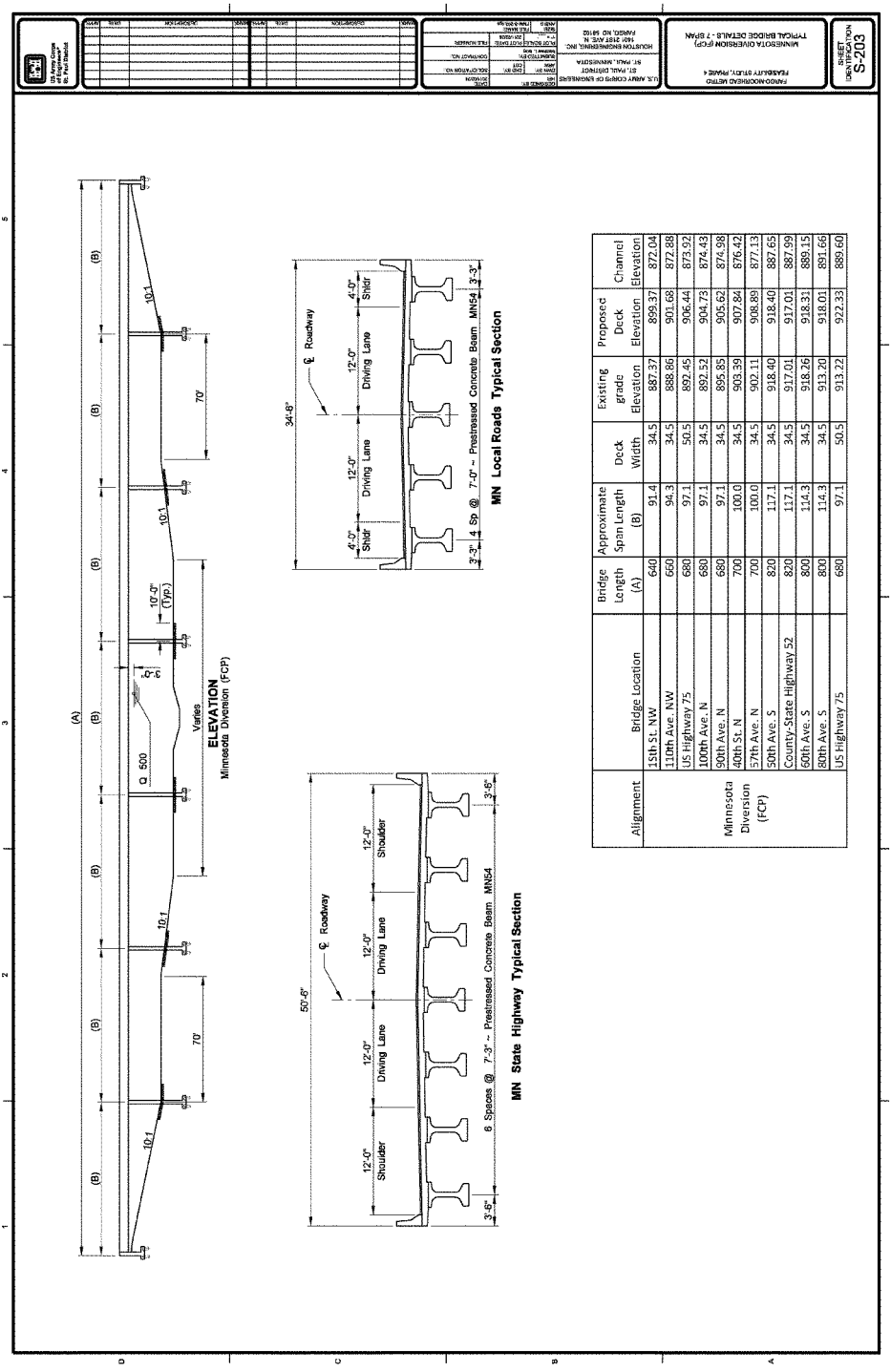
North Dakota Diversion (LPP) Bridge Costs Summary

Location	Bridge Length (ft)	Bridge Deck Area (ft^2)	Cost per ft^2 Deck Area	Bridge Structure Cost	Existing Structure Removal	Approach Road Cost	Temporary Bypass Cost	Total Cost per Bridge
County Road 81	600	23,100	120	$2,770,000	$0	$550,000	$350,000	$3,670,000
Interstate 29 North Bound	580	24,650	105	$2,590,000	$0	$840,000	$230,000	$3,660,000
Interstate 29 South Bound	580	24,650	105	$2,590,000	$0	$830,000	$230,000	$3,650,000
48th St. SE	440	14,300	120	$1,720,000	$0	$460,000	$320,000	$2,500,000
170th Ave. SE	640	19,520	125	$2,440,000	$0	$70,000	$240,000	$2,750,000
46th St. SE	760	24,700	120	$2,960,000	$0	$70,000	$250,000	$3,280,000
44th St. SE	700	21,350	125	$2,670,000	$20,000	$70,000	$250,000	$3,010,000
41st St. SE	640	24,640	125	$3,080,000	$20,000	$160,000	$270,000	$3,530,000
Interstate 94 East Bound	680	28,900	115	$3,320,000	$0	$140,000	$230,000	$3,690,000
Interstate 94 West Bound	680	28,900	115	$3,320,000	$0	$140,000	$230,000	$3,690,000
36th St. SE	660	25,410	115	$2,920,000	$0	$130,000	$260,000	$3,310,000
33rd St. SE	640	24,640	125	$3,080,000	$0	$200,000	$280,000	$3,560,000
31st St. SE	620	18,910	125	$2,360,000	$0	$240,000	$290,000	$2,890,000
28th St. SE	620	18,910	120	$2,270,000	$0	$280,000	$290,000	$2,840,000
Interstate 29 South Bound	580	24,650	120	$2,960,000	$10,000	$520,000	$230,000	$3,720,000
Interstate 29 North Bound	580	24,650	120	$2,960,000	$10,000	$530,000	$230,000	$3,730,000
County Road 81	580	22,330	120	$2,680,000	$10,000	$360,000	$310,000	$3,360,000
25th St. SE	580	18,850	120	$2,260,000	$0	$340,000	$300,000	$2,900,000
173rd Ave. SE	600	18,300	120	$2,200,000	$0	$370,000	$310,000	$2,880,000
							Total	$62,620,000

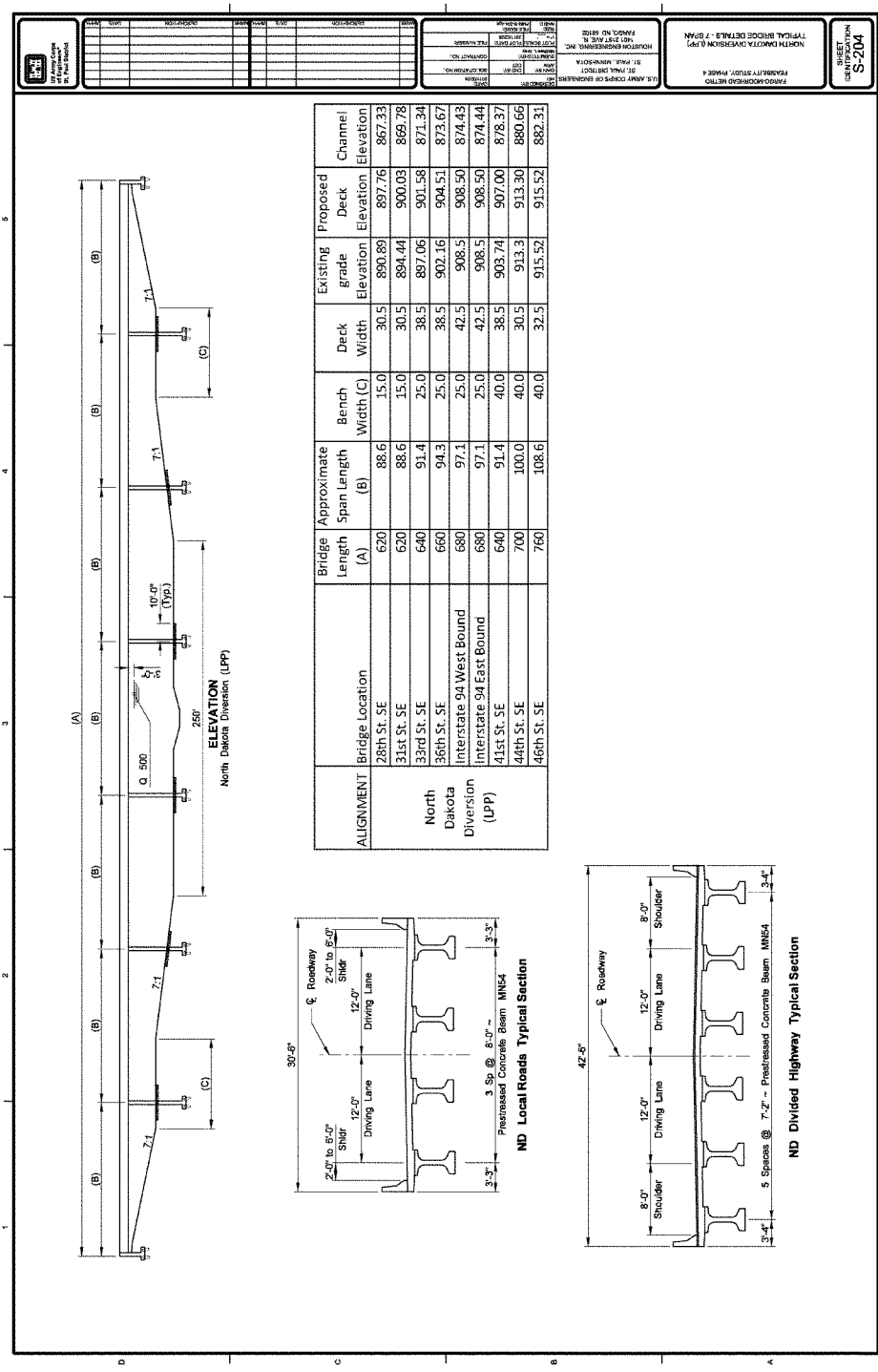

FARGO-MOORHEAD METRO
FEASIBILITY STUDY, PHASE II
EXHIBIT - A
TYPICAL BRIDGE ELEVATION
SHEET IDENTIFICATION S-201
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
HOUSTON ENGINEERING, INC.
MD. 218TH AVE. N.
FARGO, ND 58102
Existing Grade
Proposed Channel Excavation
Proposed Root Grade
DETAIL A
ELEVATION
DETAIL A
Note: See Exhibit A for description of dimension references.



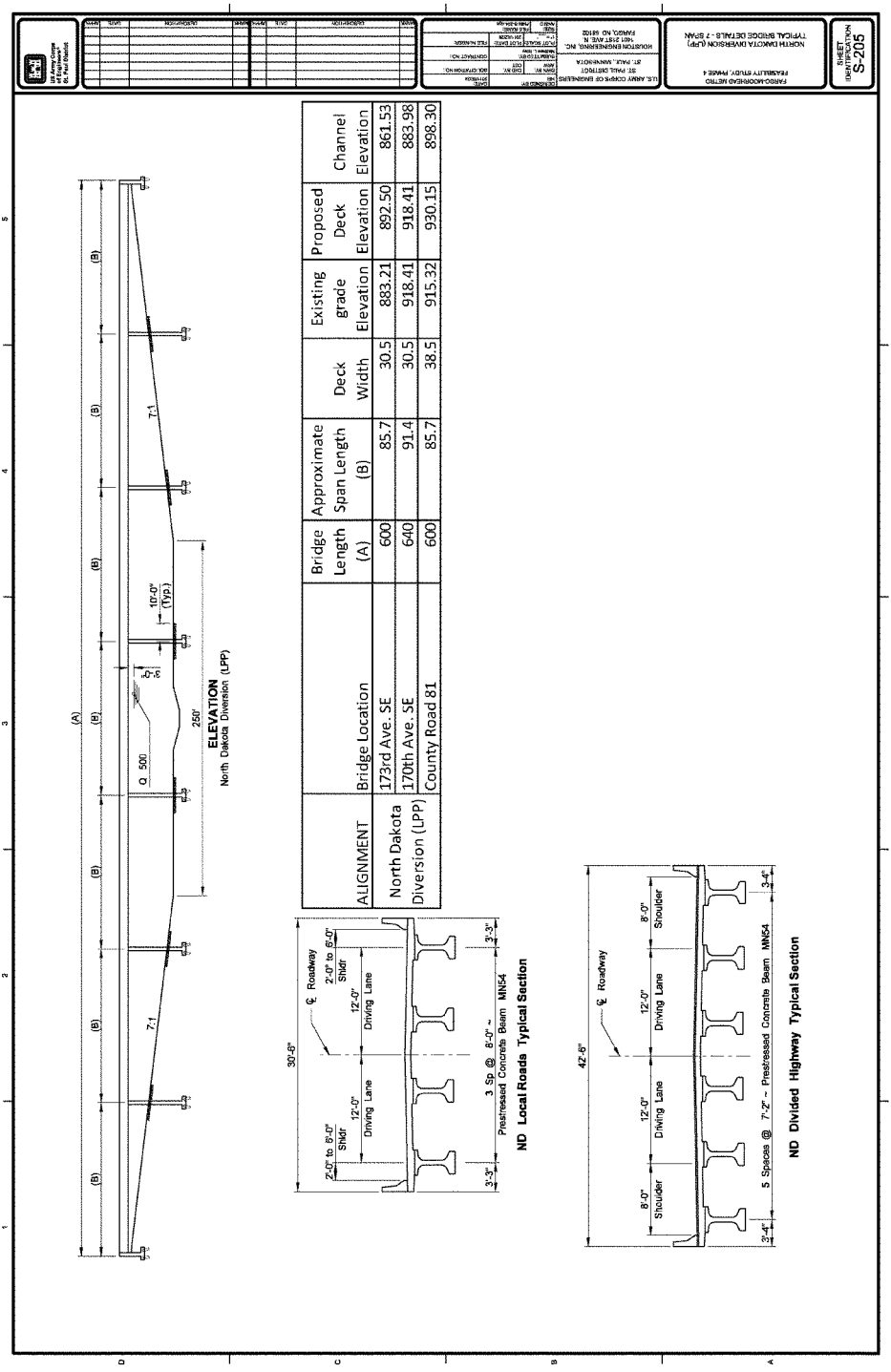
Alignment	Bridge Location	Bridge Length (A)	Approximate Span Length (B)	Deck Width	Existing grade Elevation	Proposed Deck Elevation	Channel Elevation
Minnesota Diversion (FCP)	28th Ave. N	880	97.8	34.5	913.45	913.45	882.33
	US Highway 10 West Bound	900	100.0	44.5	913.50	913.50	883.40
	US Highway 10 East Bound	900	100.0	44.5	914.23	914.23	883.40
	Interstate 94 West Bound	860	95.6	44.5	914.76	914.76	885.50
	Interstate 94 East Bound	860	95.6	44.5	914.64	914.64	885.50

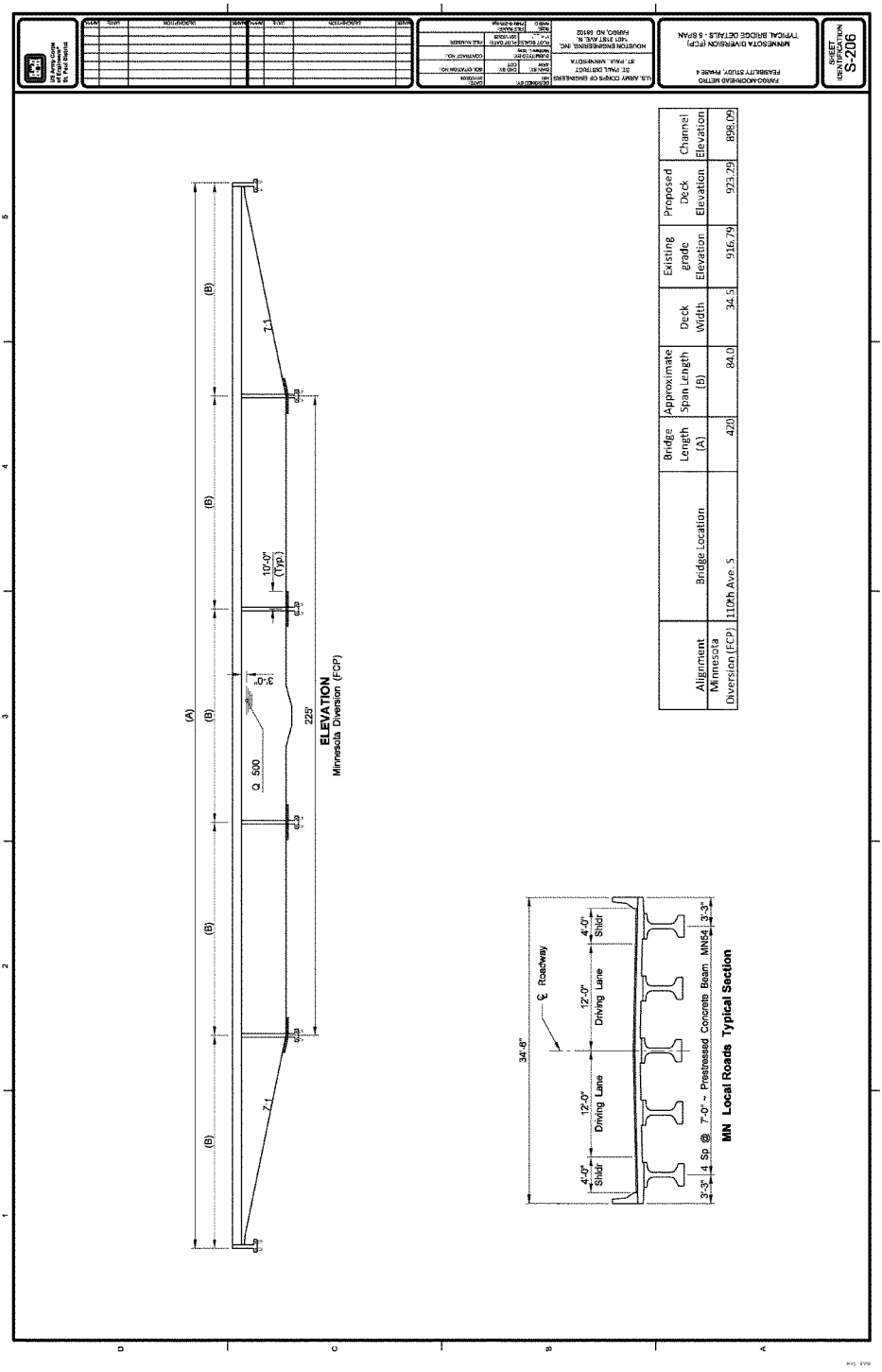


Alignment	Bridge Location	Bridge Length (A)	Approximate Span Length (B)	Deck Width	Existing grade Elevation	Proposed Deck Elevation	Channel Elevation
Minnesota Diversion (FCP)	15th St. NW	640	91.4	34.5	887.37	899.37	872.04
	110th Ave. NW	660	94.3	34.5	888.86	901.68	872.88
	US Highway 75	680	97.1	50.5	892.45	906.44	873.92
	100th Ave. N	680	97.1	34.5	892.52	904.73	874.43
	90th Ave. N	680	97.1	34.5	895.85	905.62	874.98
	40th St. N	700	100.0	34.5	903.39	907.84	876.42
	57th Ave. N	700	100.0	34.5	902.11	908.89	877.13
	50th Ave. S	820	117.1	34.5	918.40	918.40	887.65
	County-State Highway 52	820	117.1	34.5	917.01	917.01	887.99
	60th Ave. S	800	114.3	34.5	918.26	918.31	889.15
	80th Ave. S	800	114.3	34.5	913.20	918.01	891.66
	US Highway 75	680	97.1	50.5	913.22	922.33	889.60

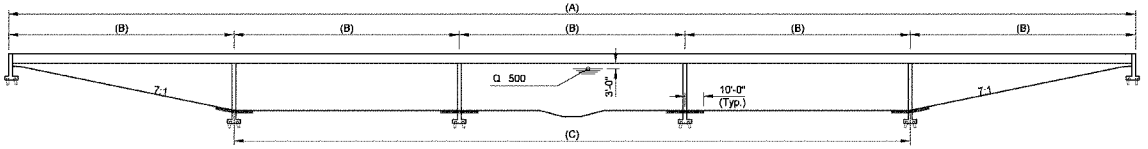


ALIGNMENT	Bridge Location	Bridge Length (A)	Approximate Span Length (B)	Bench Width (C)	Deck Width	Existing grade Elevation	Proposed Deck Elevation	Channel Elevation
North Dakota Diversion (LPP)	28th St. SE	620	88.6	15.0	30.5	890.89	897.76	867.33
	31st St. SE	620	88.6	15.0	30.5	894.44	900.03	869.78
	33rd St. SE	640	91.4	25.0	38.5	897.06	901.58	871.34
	36th St. SE	660	94.3	25.0	38.5	902.16	904.51	873.67
	Interstate 94 West Bound	680	97.1	25.0	42.5	908.5	908.50	874.43
	Interstate 94 East Bound	680	97.1	25.0	42.5	908.5	908.50	874.44
	41st St. SE	640	91.4	40.0	38.5	903.74	907.00	878.37
	44th St. SE	700	100.0	40.0	30.5	913.3	913.30	880.66
	46th St. SE	760	108.6	40.0	32.5	915.52	915.52	882.31

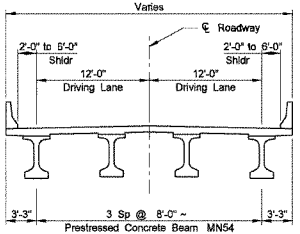

FARGO-MOORHEAD METRO
FEASIBILITY STUDY, PHASE 4
NORTH DAKOTA DIVERSION (LPP)
TYPICAL BRIDGE DETAILS - 7 SPAN
SHEET
S-205
ELEVATION
North Dakota Diversion (LPP)
Q 500
7:1
10'-0" (Typ)
250'
(A)
(B)
ALIGNMENT
Bridge Location
Bridge Length (A)
Approximate Span Length (B)
Deck Width
Existing grade Elevation
Proposed Deck Elevation
Channel Elevation
North Dakota Diversion (LPP)
173rd Ave. SE
600
85.7
30.5
883.21
892.50
861.53
170th Ave. SE
640
91.4
30.5
918.41
918.41
883.98
County Road 81
600
85.7
38.5
915.32
930.15
898.30
30'-6"
℄ Roadway
2'-0" to 8'-0" Shldr
12'-0" Driving Lane
3 Sp @ 8'-0" = Prestressed Concrete Beam MN64
3'-3"
ND Local Roads Typical Section
42'-6"
8'-0" Shoulder
12'-0" Driving Lane
5 Spaces @ 7'-2" = Prestressed Concrete Beam MN64
3'-4"
ND Divided Highway Typical Section



Alignment	Bridge Location	Bridge Length (A)	Approximate Span Length (B)	Deck Width	Existing grade Elevation	Proposed Deck Elevation	Channel Elevation
Minnesota Diversion (FCP)	110th Ave. S	420	84.0	34.5	916.79	923.29	896.09

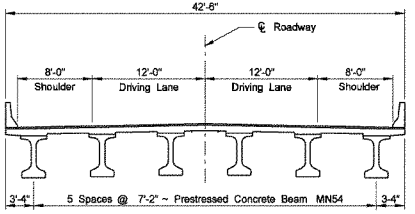


ELEVATION
North Dakota Diversion (LPP)



ND Local Roads Typical Section

ALIGNMENT	Bridge Location	Bridge Length (A)	Approximate Span Length (B)	Channel Bottom Width (C)	Deck Width	Existing grade Elevation	Proposed Deck Elevation	Channel Elevation
North Dakota Diversion (LPP)	25th St. SE	580	116	250	32.5	885.87	894.25	863.59
	County Road 81	580	116	250	38.5	887.34	896.23	865.68
	Interstate 29 North Bound	580	116	250	42.5	888.29	896.23	865.73
	Interstate 29 South Bound	580	116	250	42.5	888.42	896.23	865.74
	48th St. SE	440	88	100	32.5	916.98	929.00	900.75
	Interstate 29 South Bound	580	116	250	42.5	916.03	930.14	899.9
	Interstate 29 North Bound	580	116	250	42.5	915.99	930.14	899.88



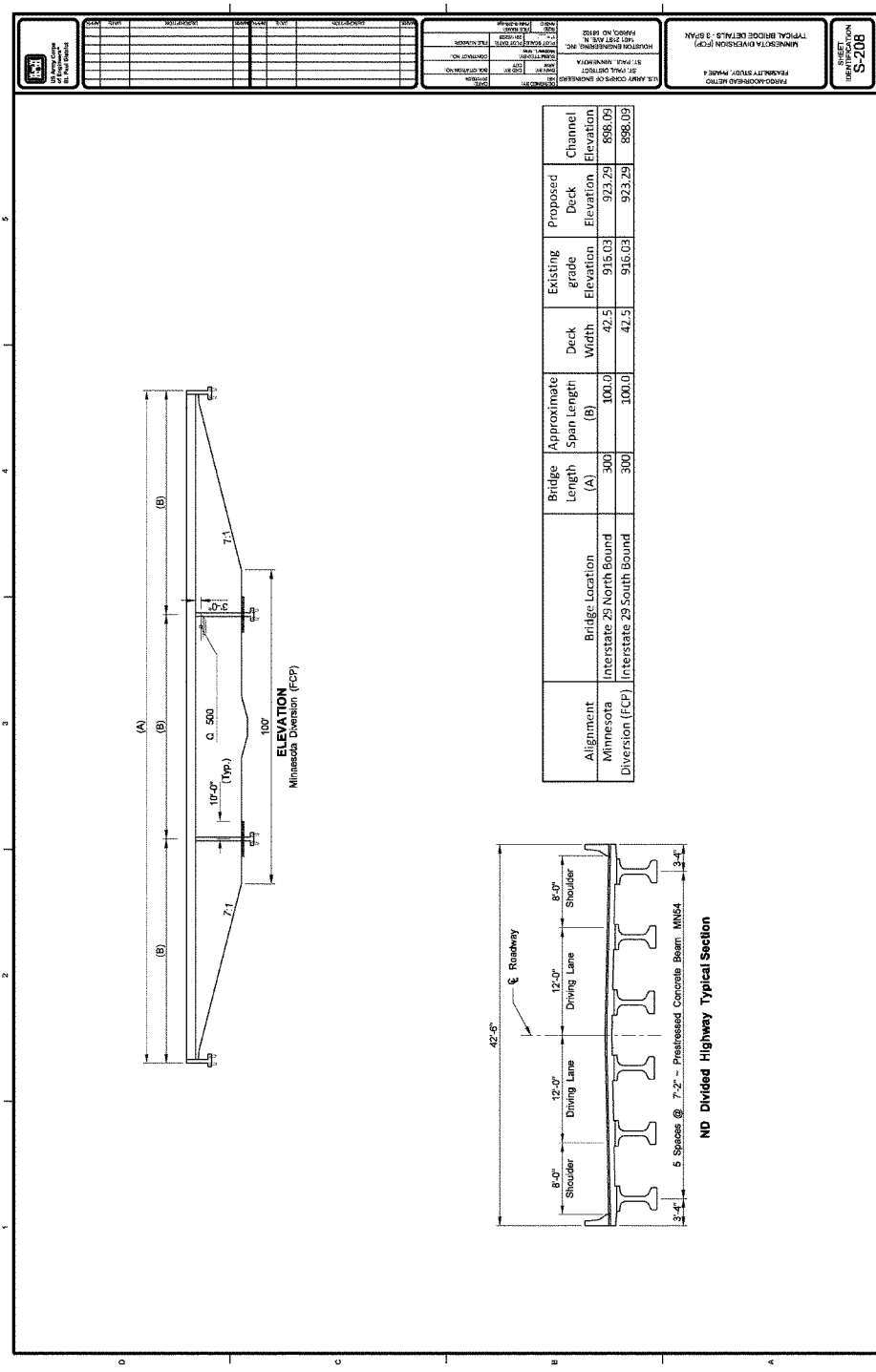
ND Divided Highway Typical Section

US Army Corps of Engineers
St. Paul District

FARGO-MOORHEAD METRO
FEASIBILITY STUDY, PHASE I

NORTH DAKOTA DIVERSION (LPP)
TYPICAL BRIDGE DETAILS - 5 SPAN

SHEET IDENTIFICATION
S-207



Alignment	Bridge Location	Bridge Length (A)	Approximate Span Length (B)	Deck Width	Existing grade Elevation	Proposed Deck Elevation	Channel Elevation
Minnesota	Interstate 29 North Bound	300	100.0	42.5	916.03	923.29	898.09
Diversion (FCP)	Interstate 29 South Bound	300	100.0	42.5	916.03	923.29	898.09

PARSONS/MOORHEAD METRO FEASIBILITY STUDY, PHASE 4
MINNESOTA DIVERSION (FCP)
TYPICAL BRIDGE DETAILS - 3 SPAN

SHEET IDENTIFICATION
S-208

RED RIVER DIVERSION

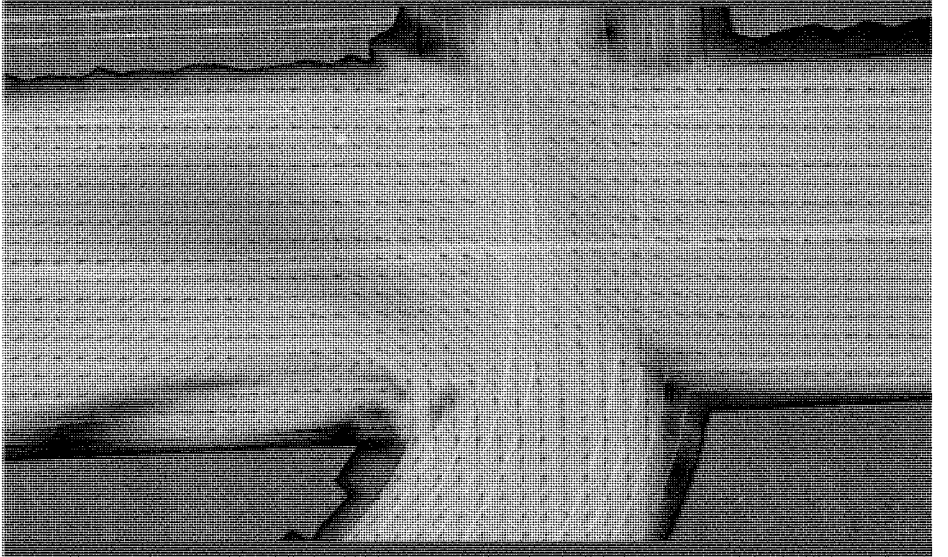
FARGO-MOORHEAD METRO FLOOD

RISK MANAGEMENT PROJECT

FEASIBILITY STUDY - PHASE 4

Volume 4

Appendix F



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

Prepared by:
Moore Engineering, Inc.; Houston Engineering, Inc.;
Barr Engineering Company; and HDR Engineering, Inc.

April 2011

RED RIVER DIVERSION

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

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

Moore Engineering, Inc.; Houston Engineering, Inc.;
Barr Engineering Company; HDR Engineering, Inc.

VOLUME 4

APPENDIX F

FINAL – Version April 14, 2011*
*Contents of this report are the same as the February 28, 2011 submittal unless otherwise specified.

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

By: Barr Engineering Co.

FINAL – February 28, 2011

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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 steady 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 unsteady 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.

Outlet Structure

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.

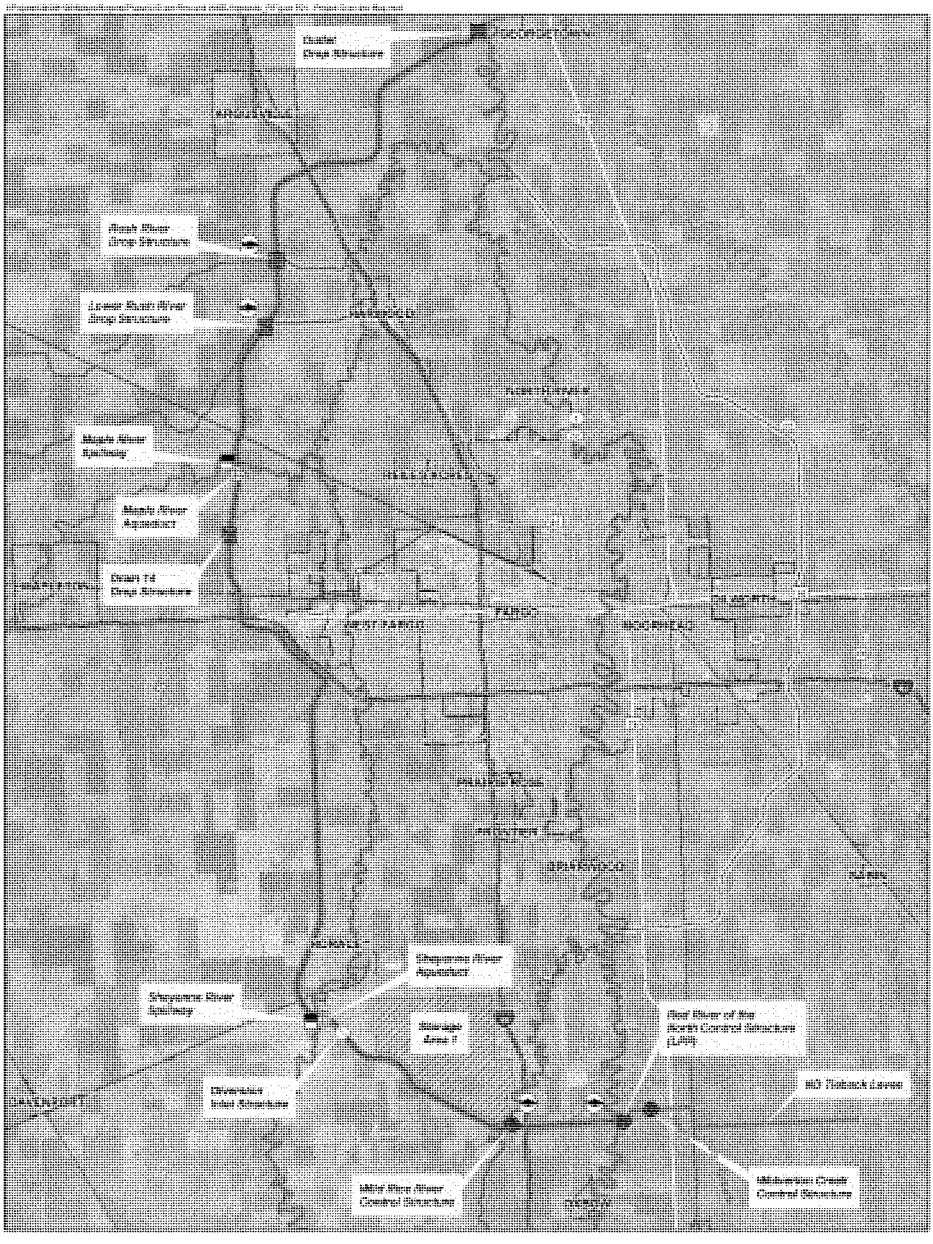
Topsoil stripping, replacing and site restoration is assumed to be required only for those areas disturbed for containment levee construction and borrow and not all areas for which easements are acquired for this facility.

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.


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
0 0.5 1 2 3 4
Miles

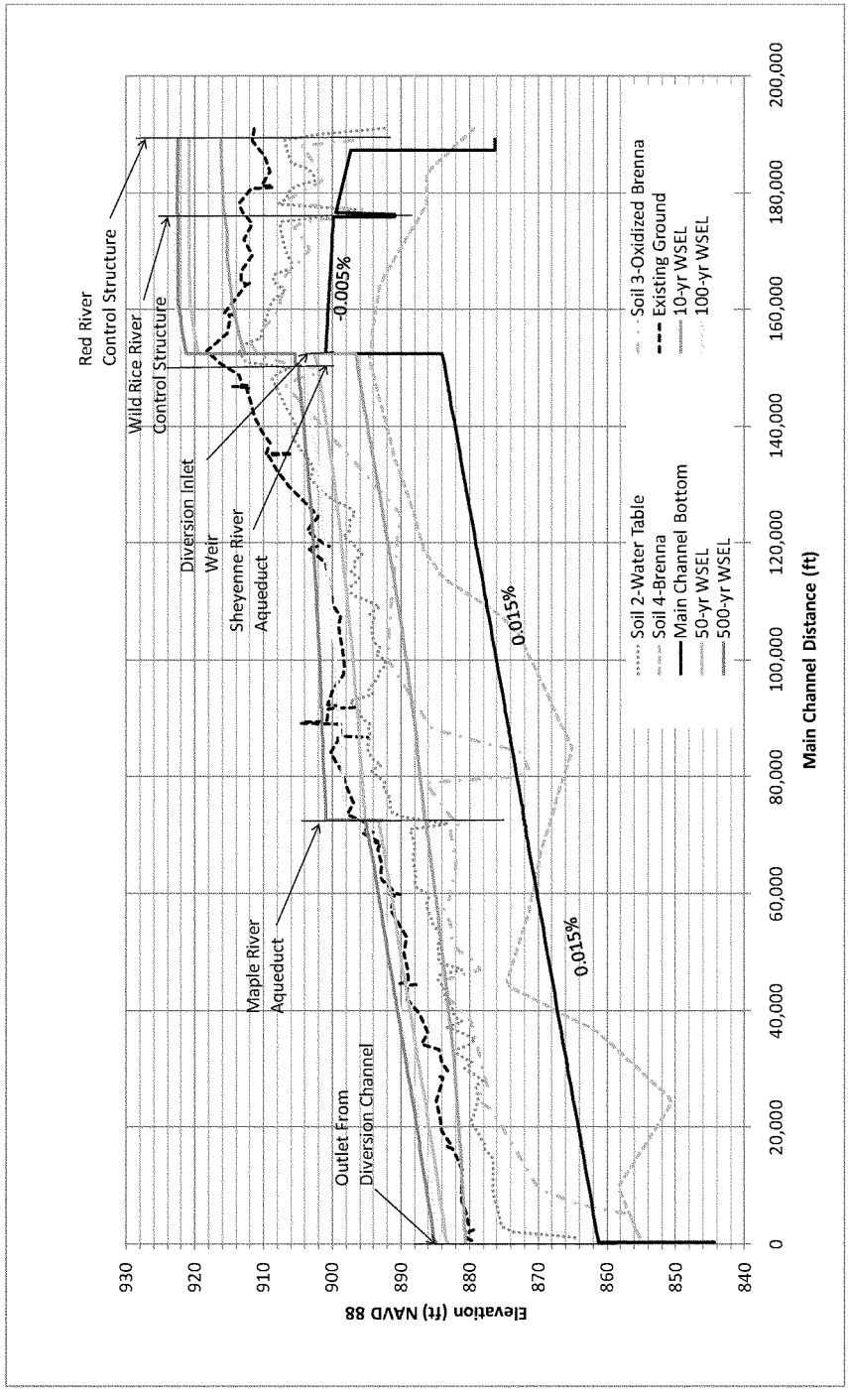
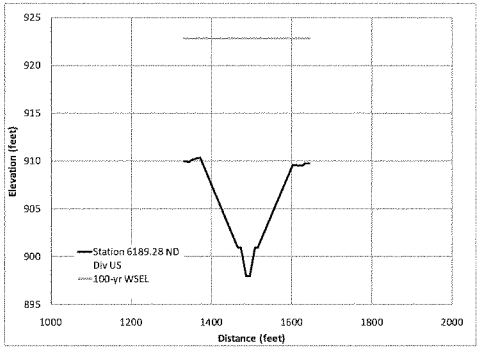


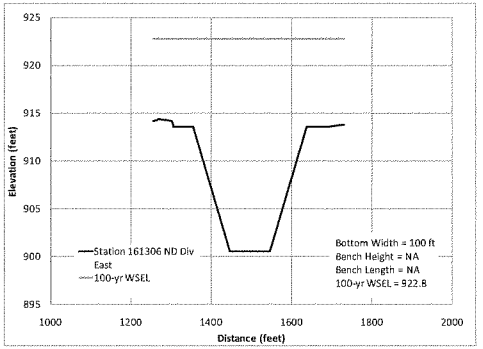
Figure F02 Longitudinal Profile of LPP Diversion Channel

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

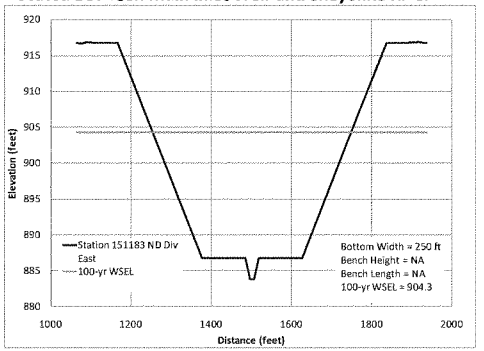
Station 6189.28 LPP ND Div US
Located Between Red River and the Wild Rice River



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

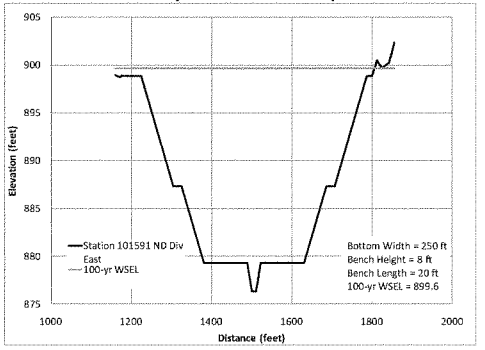
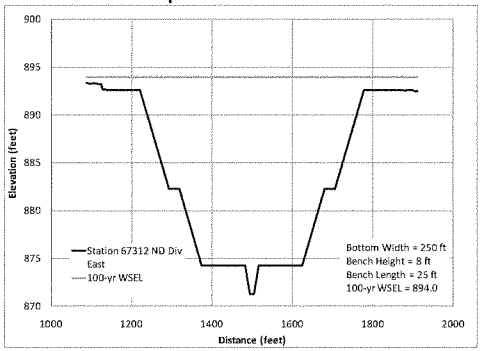
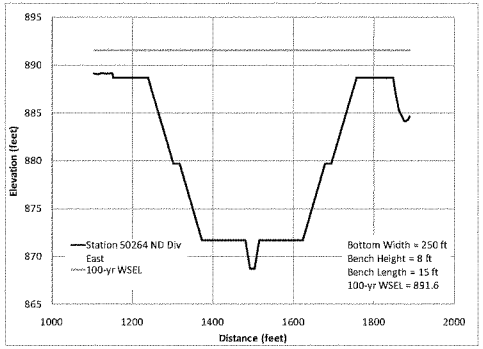


Figure F04 Typical Cross Sections of LPP Diversion Channel – See Note 1 (continued)

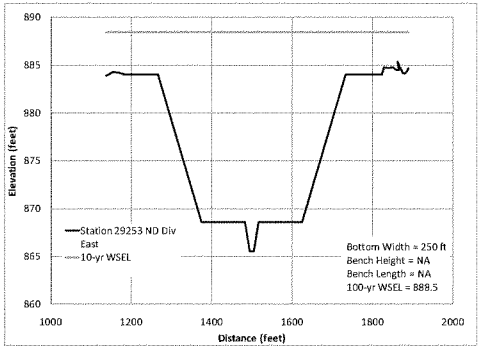
Station 67312 LPP ND Div East
Located Between Maple River and Lower Rush River



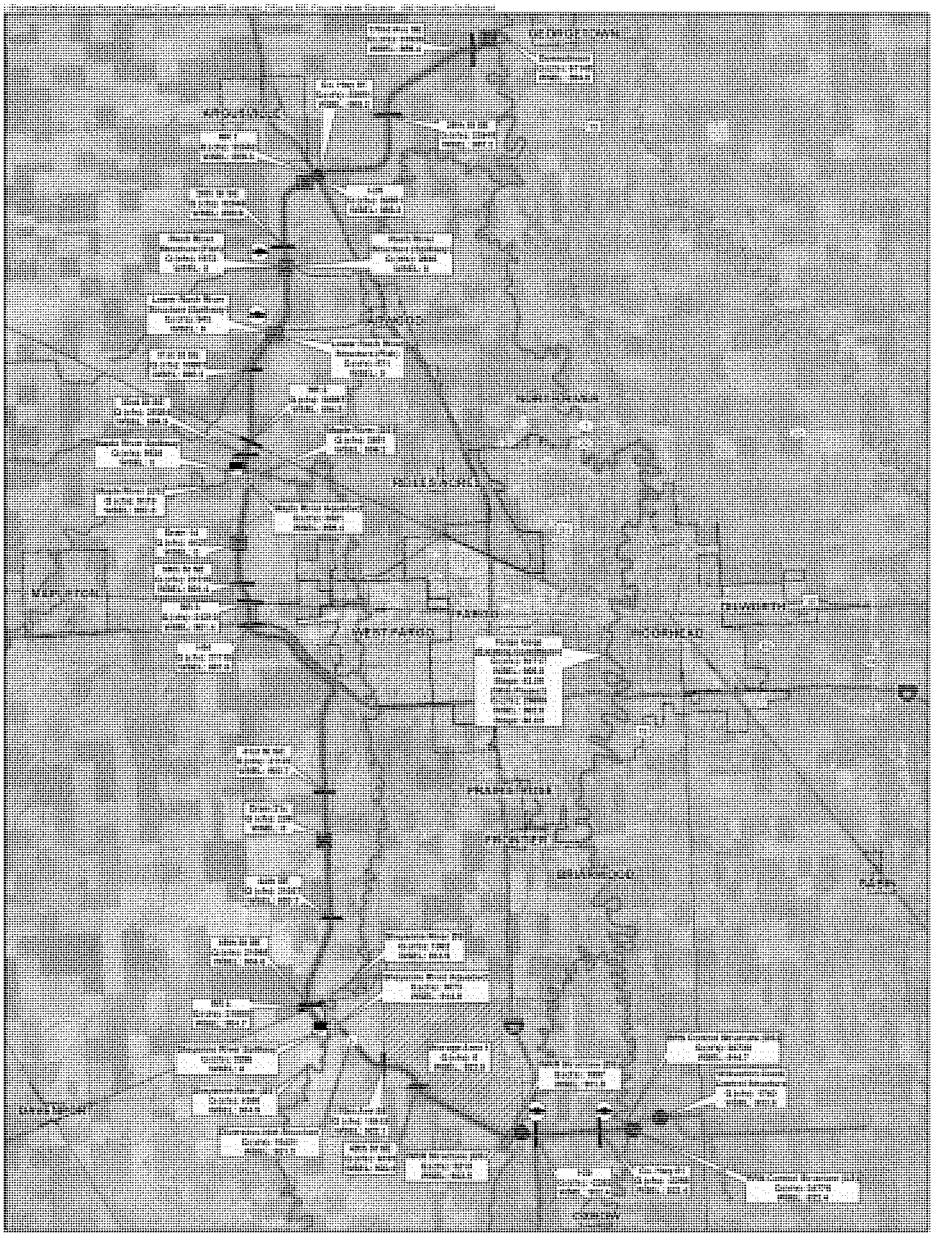
Station 50264 LPP ND Div East
Located Between Lower Rush River and Rush River



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.



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

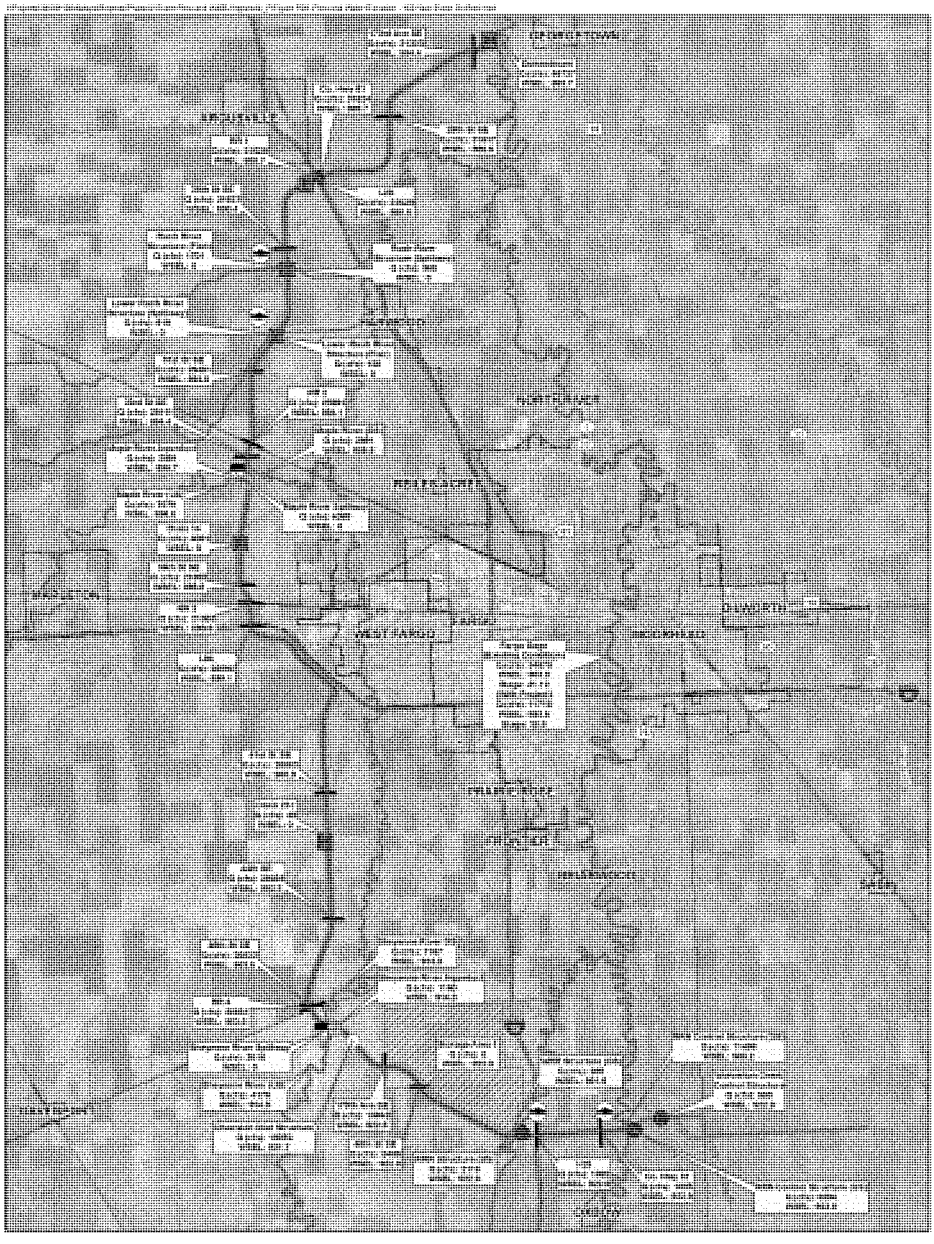
Note: Flows in rivers (US) are in main channel only. Flows in overbanks/floodplain are not reported.

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



Hydraulic Structures

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

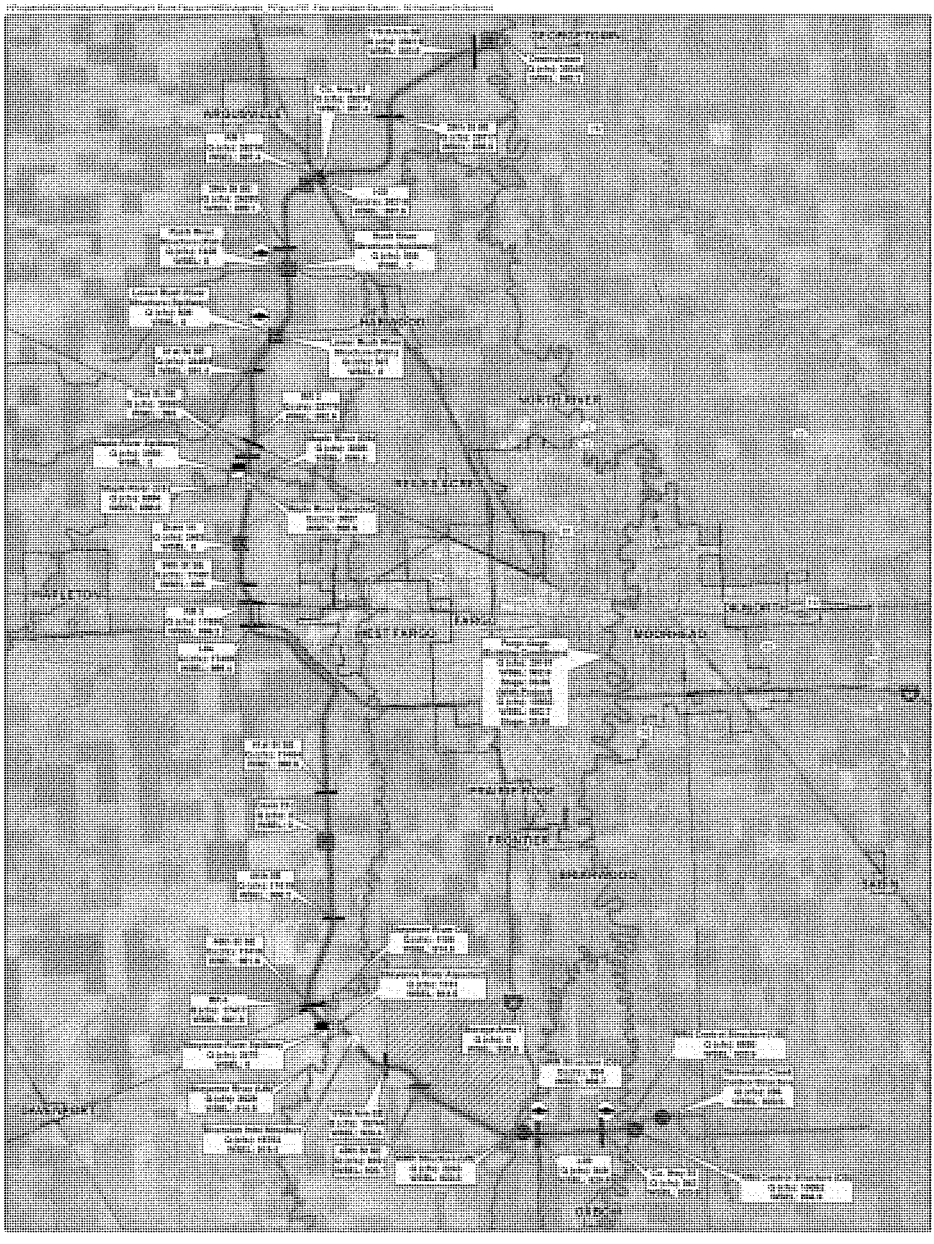
- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

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

0 0.5 1 2 3 4 Miles



Hydraulic Structures

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

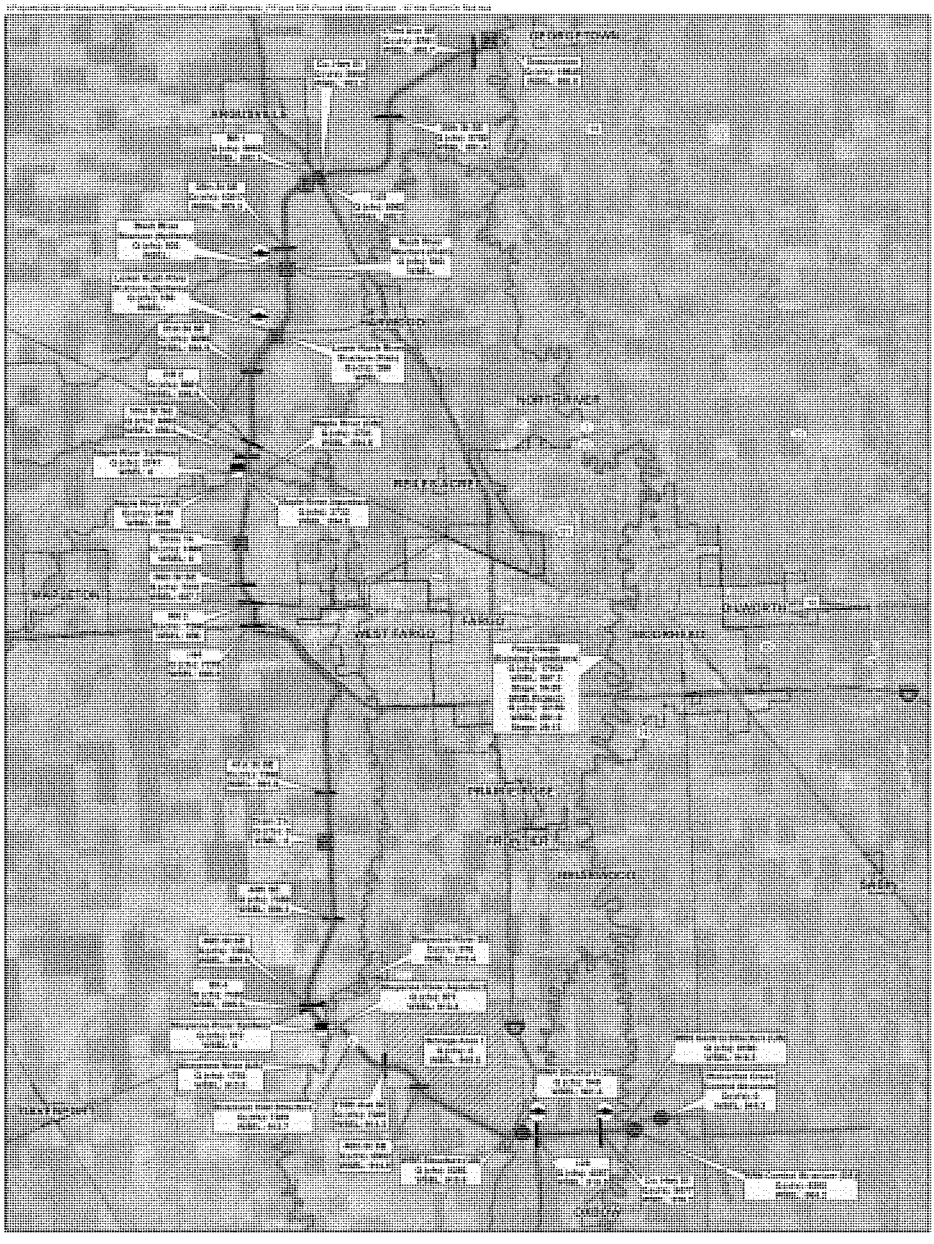
- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

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

0 0.5 1 2 3 4 Miles



Hydraulic Structures

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

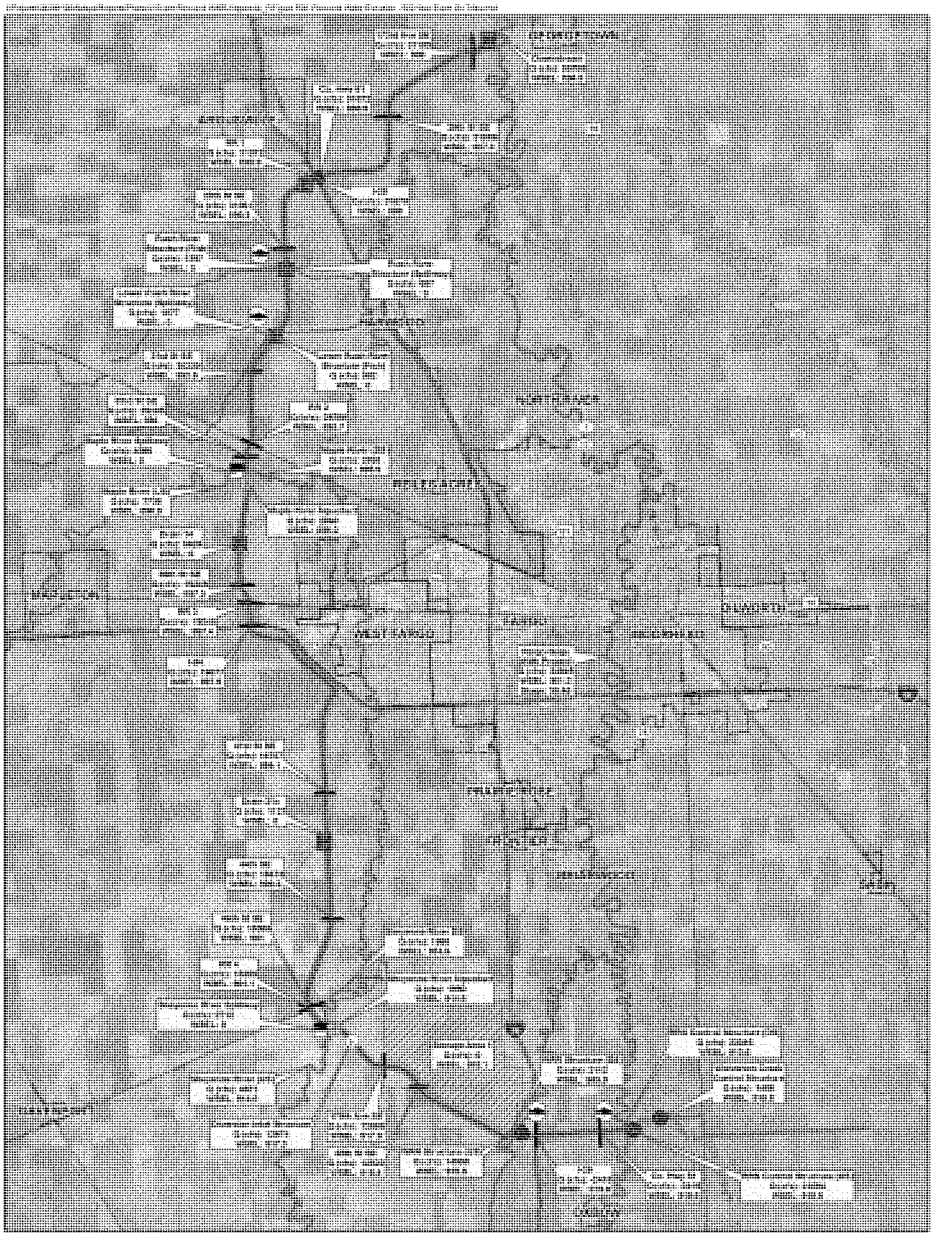
- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

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

0 0.5 1 2 3 4 Miles



Hydraulic Structures

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

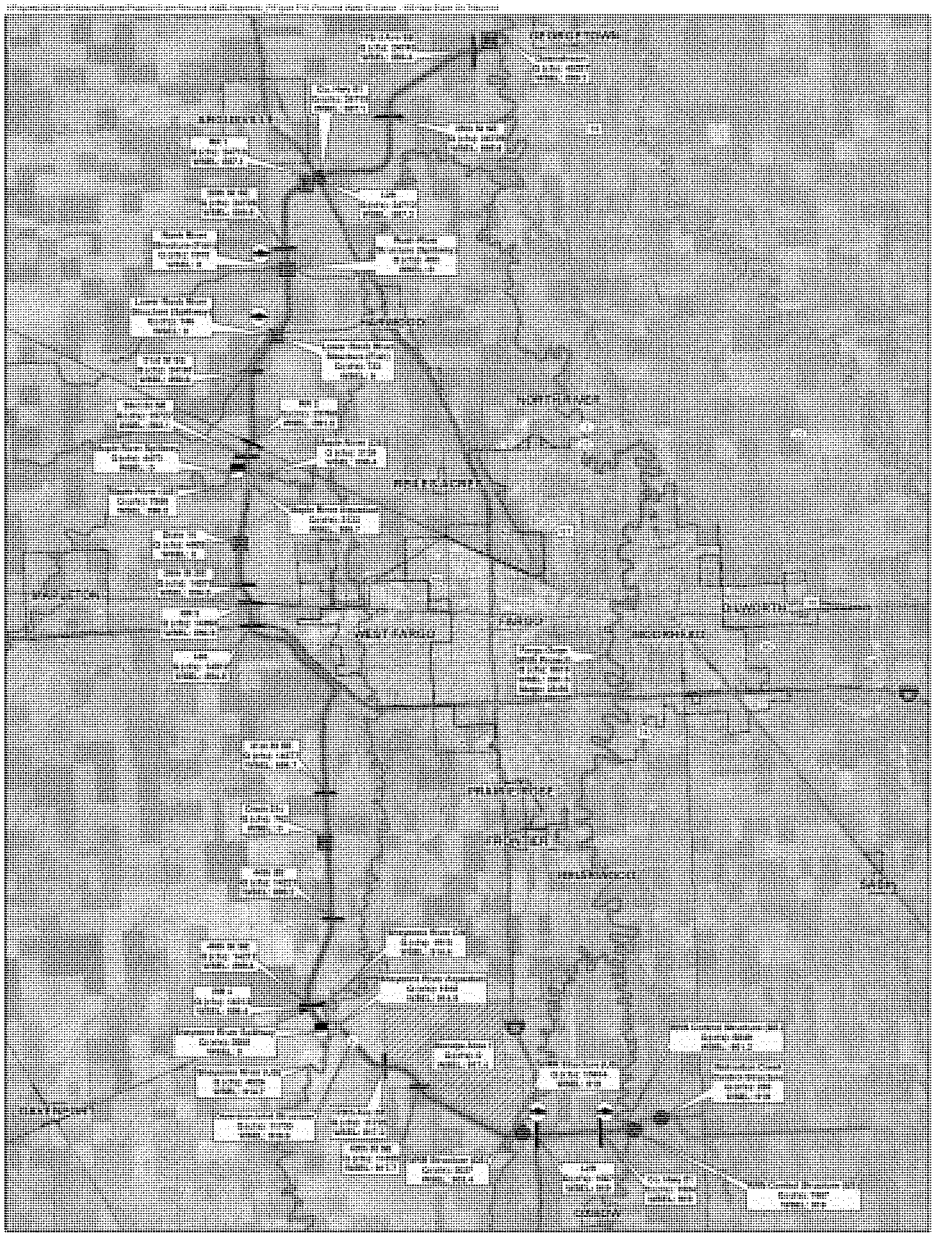
- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

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

0 0.5 1 2 3 4 Miles



Hydraulic Structures

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

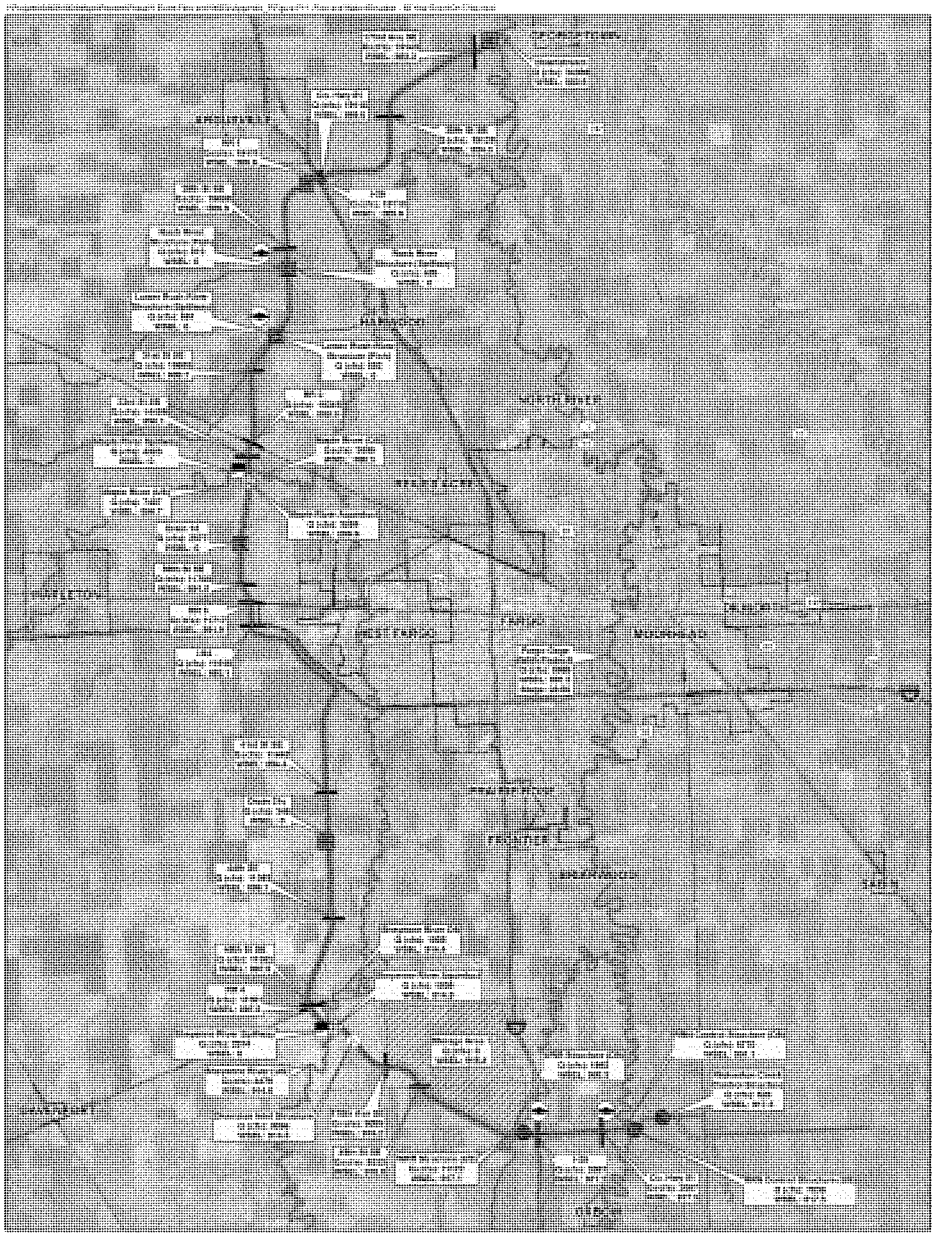
- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

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

0 0.5 1 2 3 4 Miles



Hydraulic Structures

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

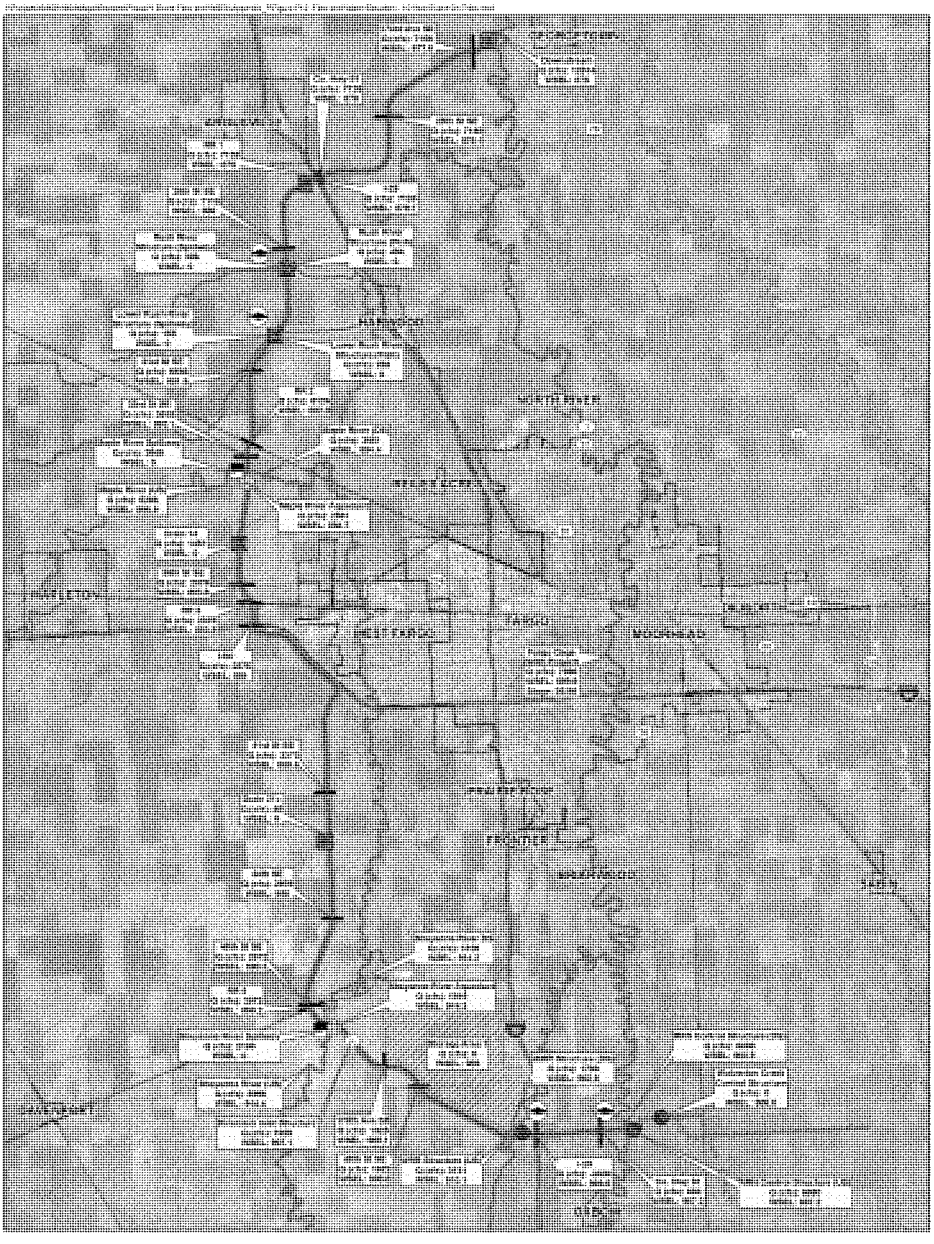
- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

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

0 0.5 1 2 3 4 Miles



Hydraulic Structures

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

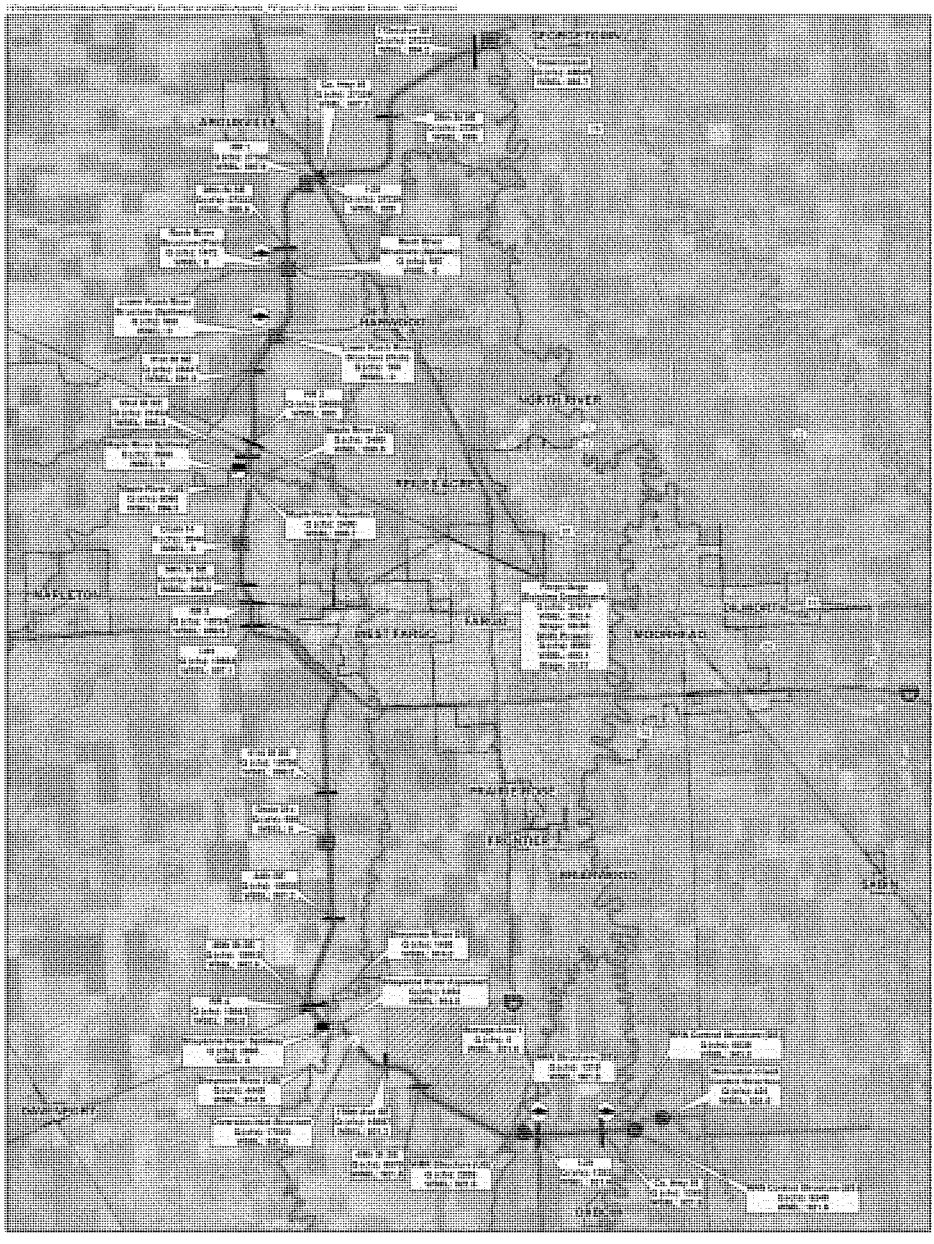
- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

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

0 0.5 1 2 3 4 Miles



Hydraulic Structures

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

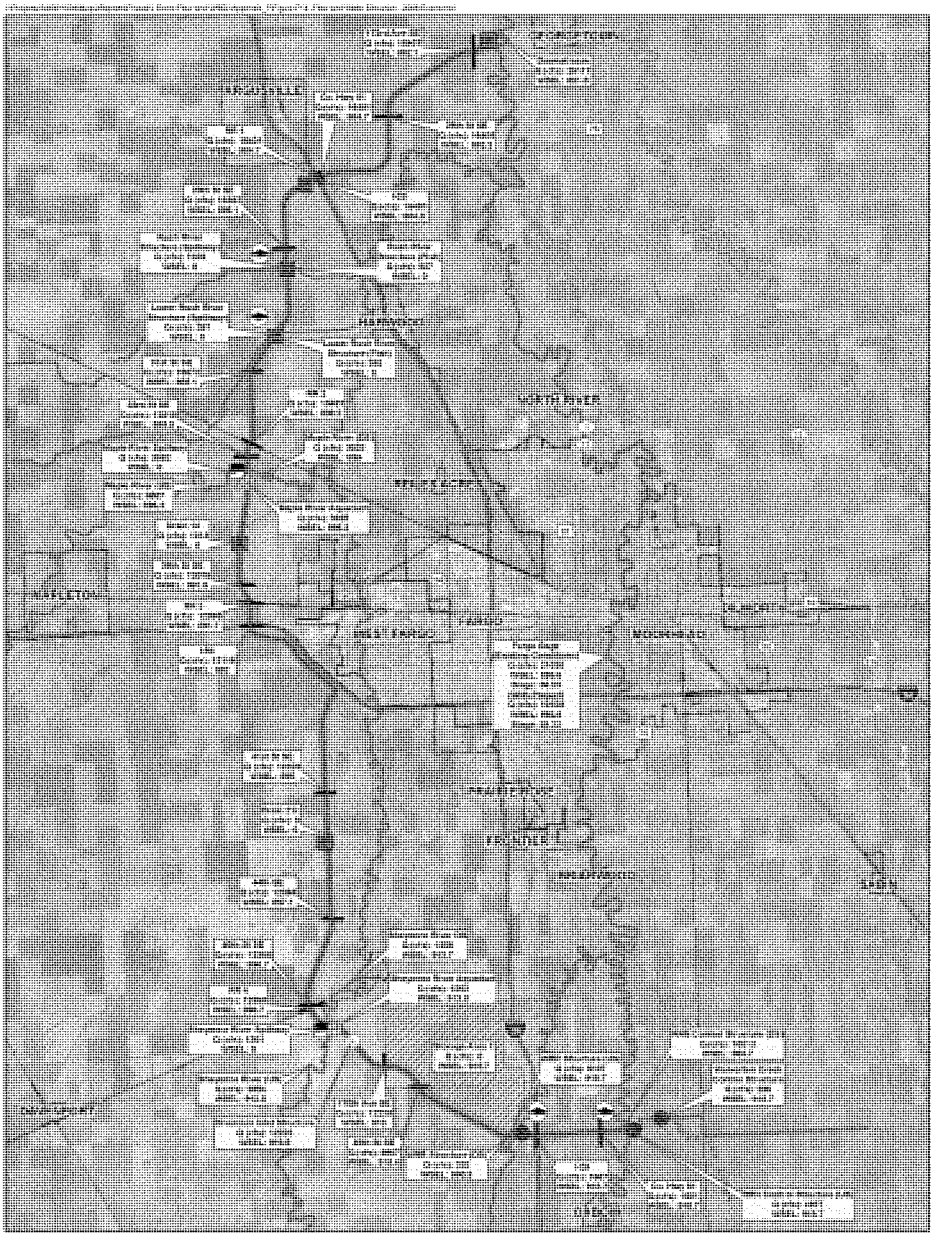
- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

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

0 0.5 1 2 3 4 Miles



Hydraulic Structures

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

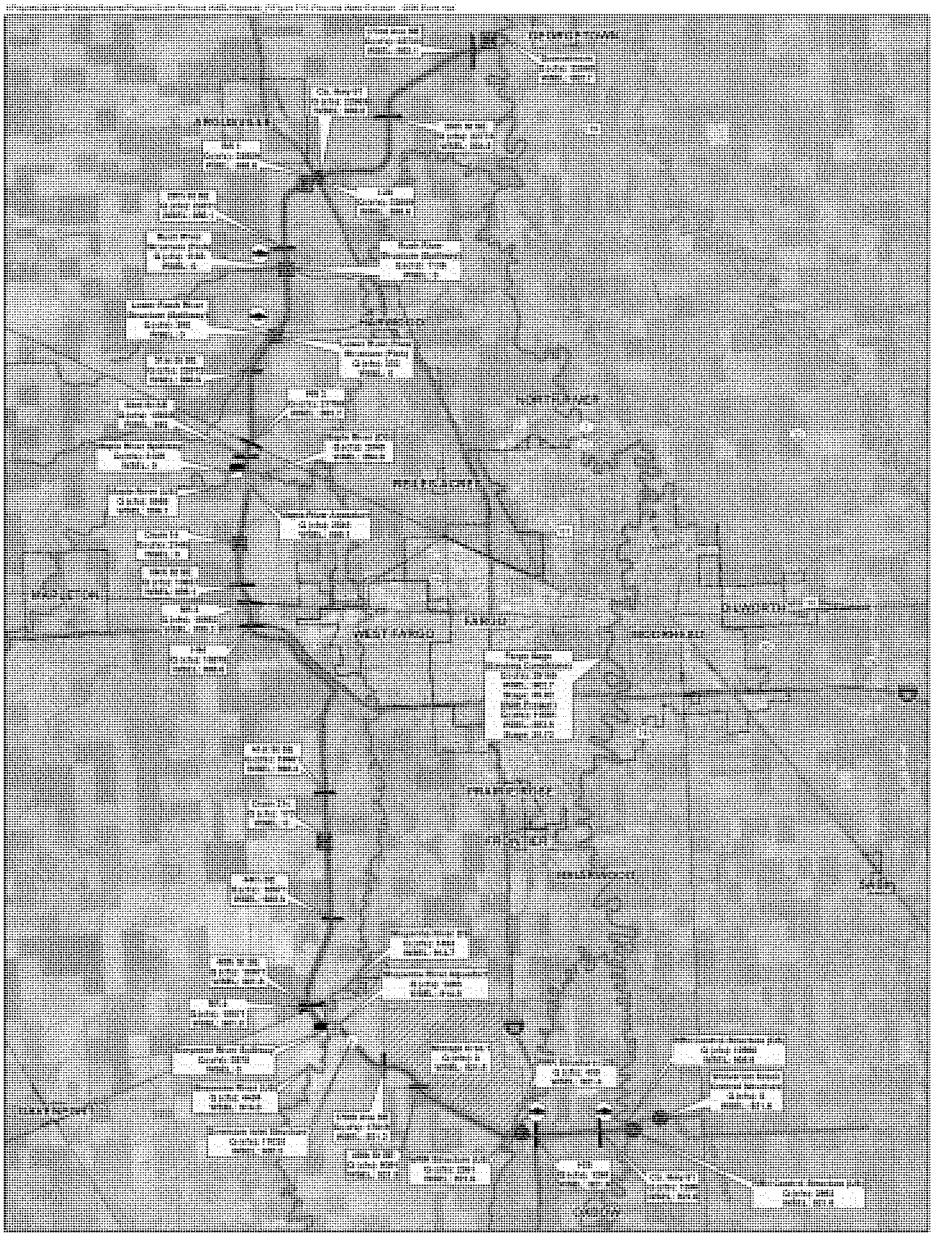
- North Dakota Diversion Locally Preferred Plan (LPP)
- ND Tieback Levee
- Channel Reclamation Reaches
- Bridge Reconstruction
- Storage Area 1

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

0 0.5 1 2 3 4 Miles



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

0 0.5 1 2 3 4 Miles

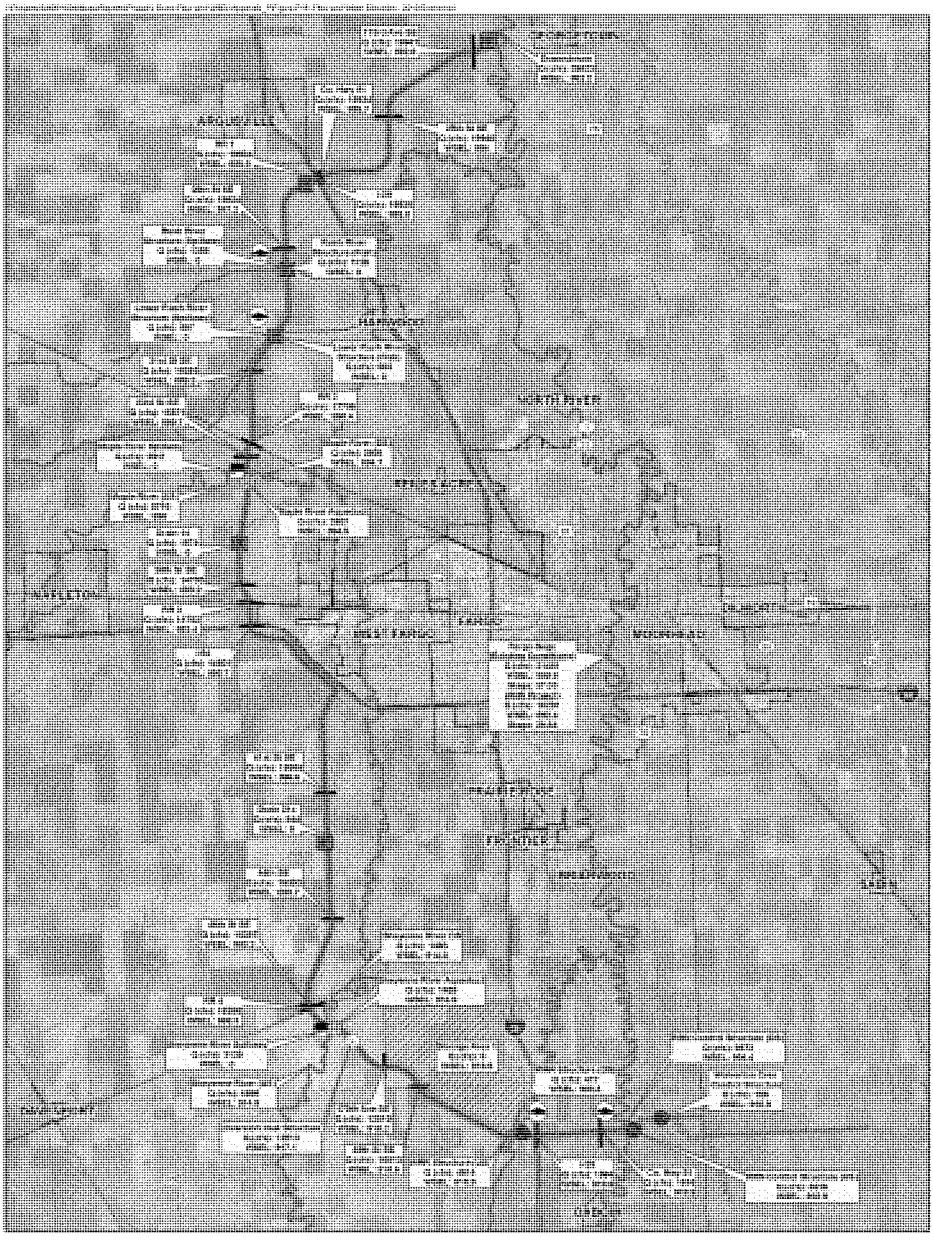


Figure F15

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

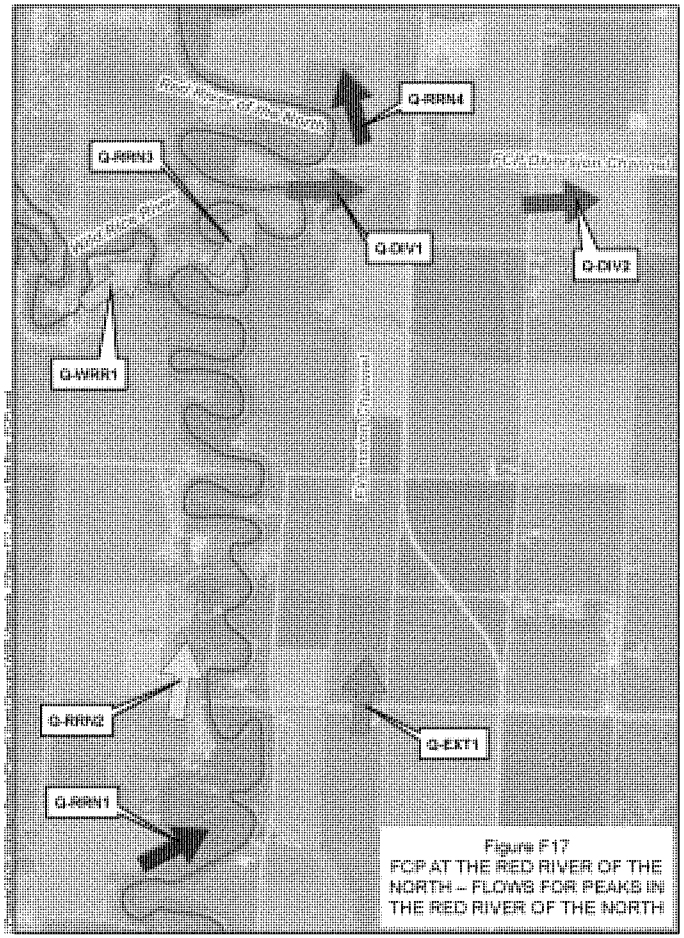


Figure F17
FCP AT THE RED RIVER OF THE NORTH – FLOWS FOR PEAKS IN THE RED RIVER OF THE NORTH

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 F16
LOP AT THE RED RIVER OF THE NORTH
AND WILD RICE RIVER – FLOWS FOR
PEAKS IN THE RED RIVER OF THE NORTH
Flows (cfs)
Q-DIV1
Diversion Inlet Structure
Q-DIV2
Diversion Channel
Q-WR2
Q-CC1
Diversion Channel Confluence
Q-WR1
Q-RRN2
Q-RRN1
Red River of the North
Wild Rice River
0
2 Miles

Red River Control Structure - Flow Scenario 1
LPP ND East 35K
Flow Scenario 1:
average flow
Red River of the North
North Dakota
Minnesota
Wild Rice River
Fish Channel
Fish Passage System
Tintah Levee
Red River
Control Structure
Approach Channel
Protected Side
Lower
Stop Banks
Upper
Embankment
Control Structure
Approach Channel
Red River of the North

Figure F19

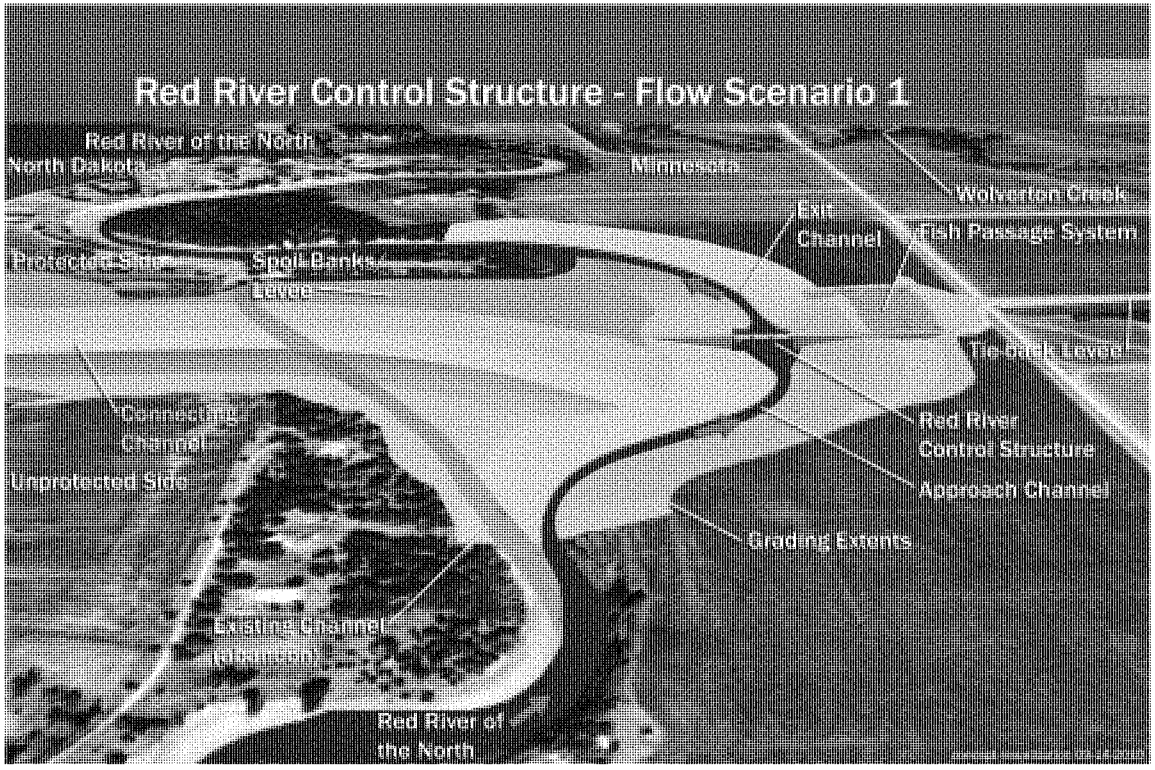
Red River Control Structure - Flow Scenario 2
LPP ND East 35K
Flow Scenario 2:
2-year event
Figure F20

Red River Control Structure - Flow Scenario 3
LPP ND East 35K
Flow Scenario 3:
10-year event
Red River of the North
Minnesota
North Dakota
Wolverton Creek
Bed Channel
Fish Passage System
Traverse Lakes
Steel Barrier
Levees
Red River
Control Structure
Approach Channel
Red River of the North

Figure F21

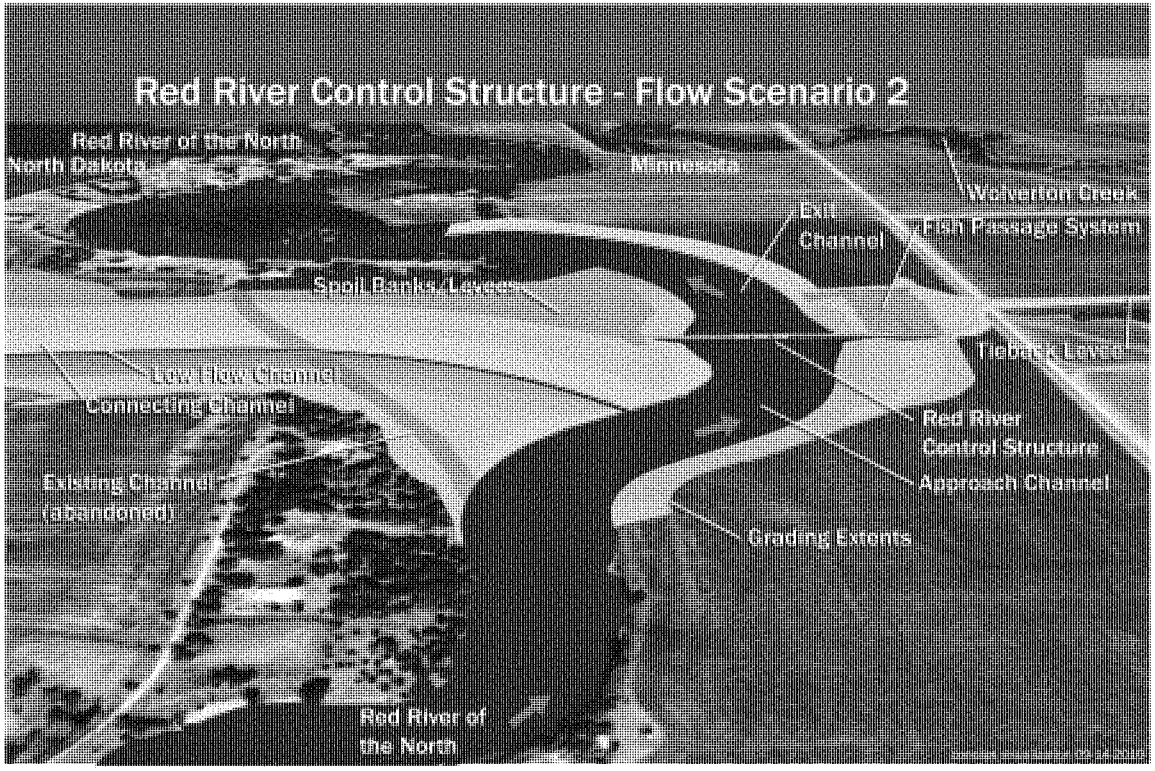
Red River Control Structure - Flow Scenario 4
LPP ND East 35K
Flow Scenario 4:
100-year event
Red River of the North
Minnesota
North Dakota
Wild Rice River
Grading Extents
Exit Channel
Fish Passage System
Tieback Levee
Red River Control Structure
Approach Channel
Connecting Channel
Unprotected side
Red River of the North

Figure F22



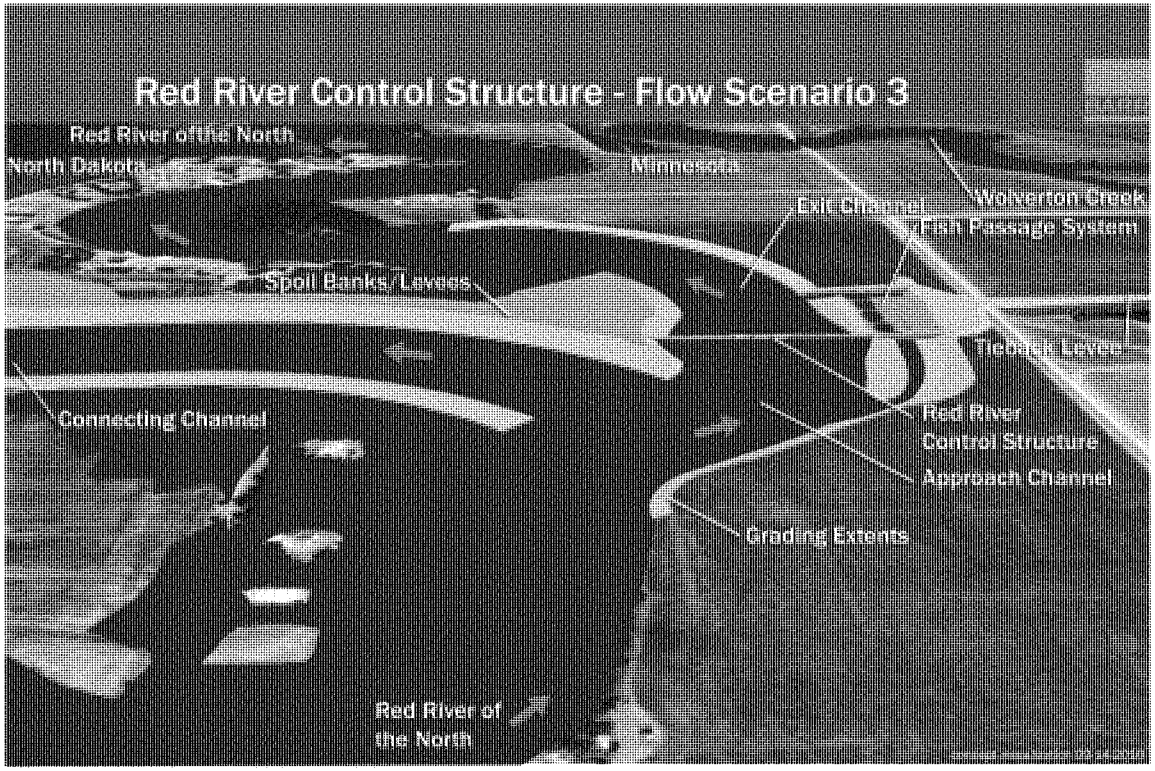
LPP ND East 35K
Flow Scenario 1:
average flow

Figure F23



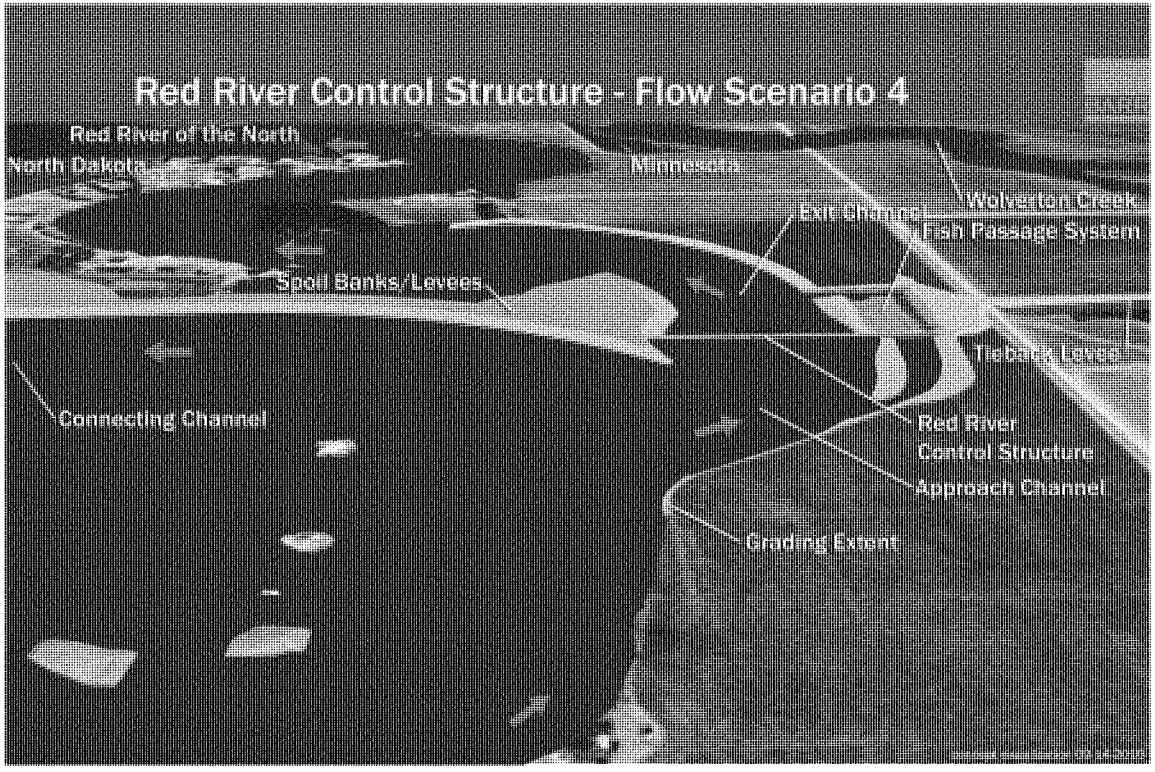
LPP ND East 35K
Flow Scenario 2:
2-year event

Figure F24



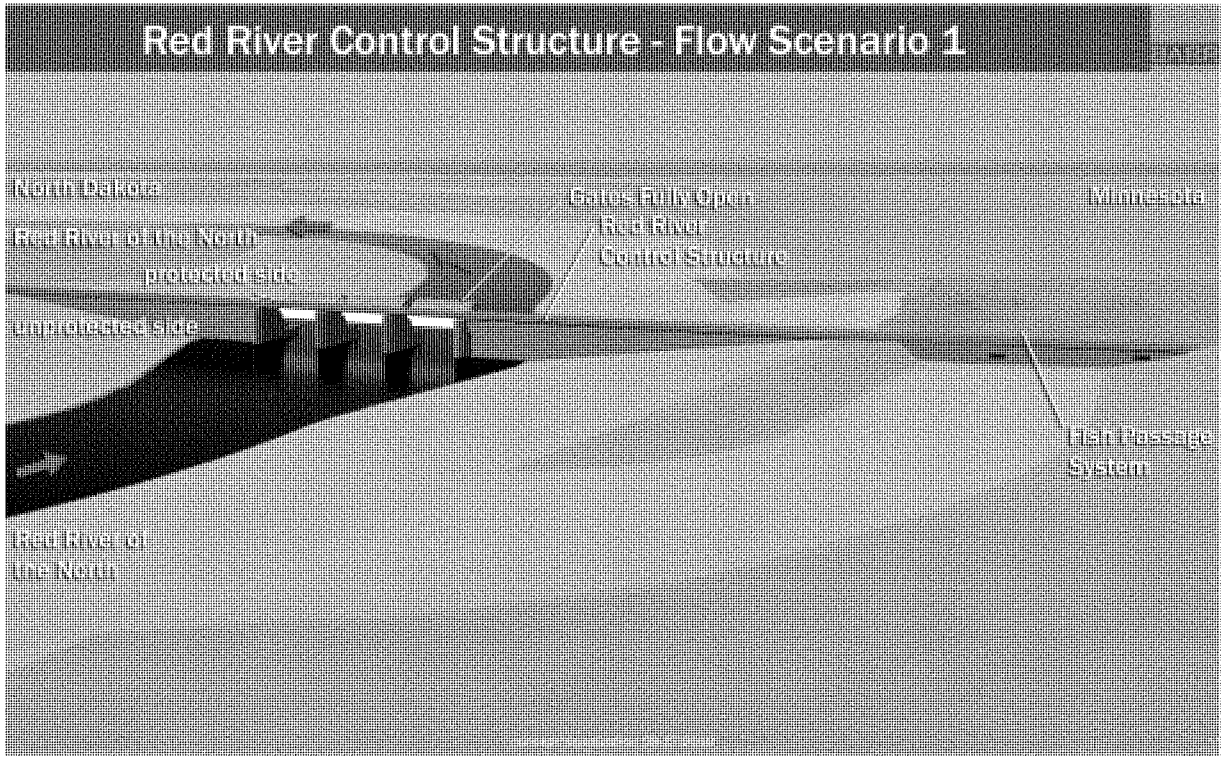
LPP ND East 35K
Flow Scenario 3:
10-year event

Figure F25



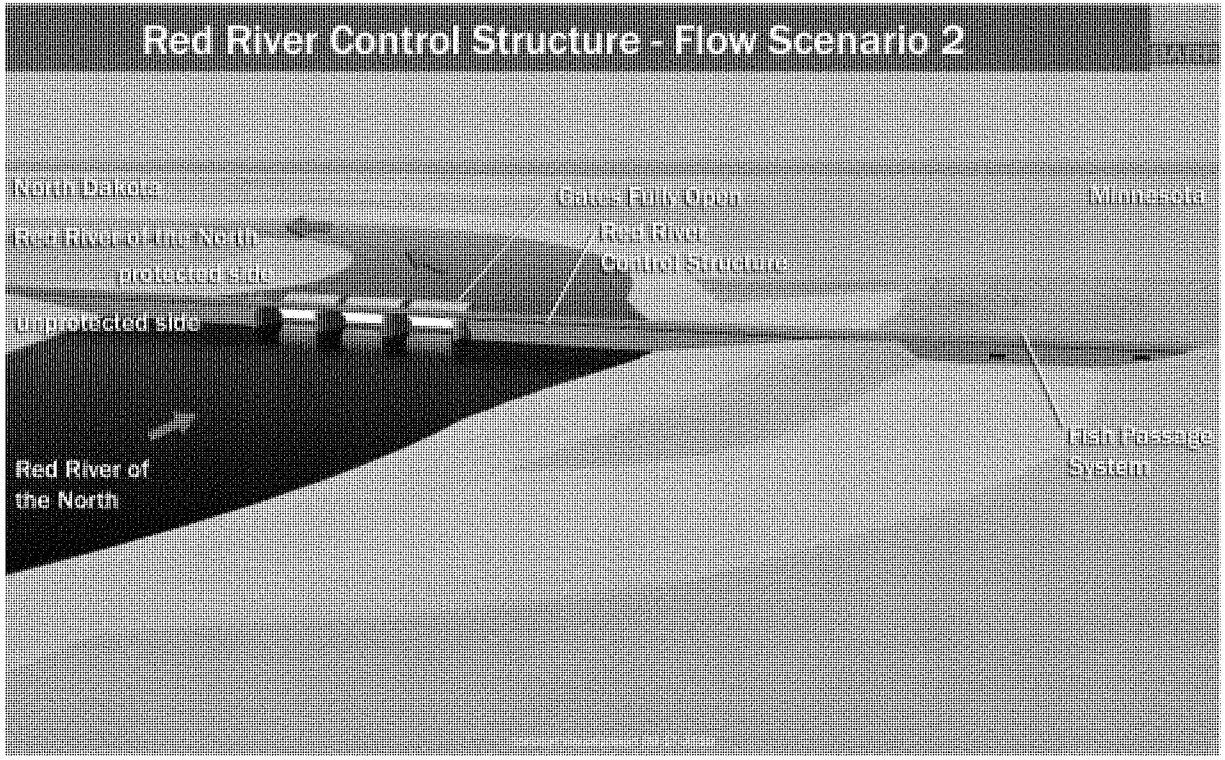
LPP ND East 35K
Flow Scenario 4:
100-year event

Figure F26



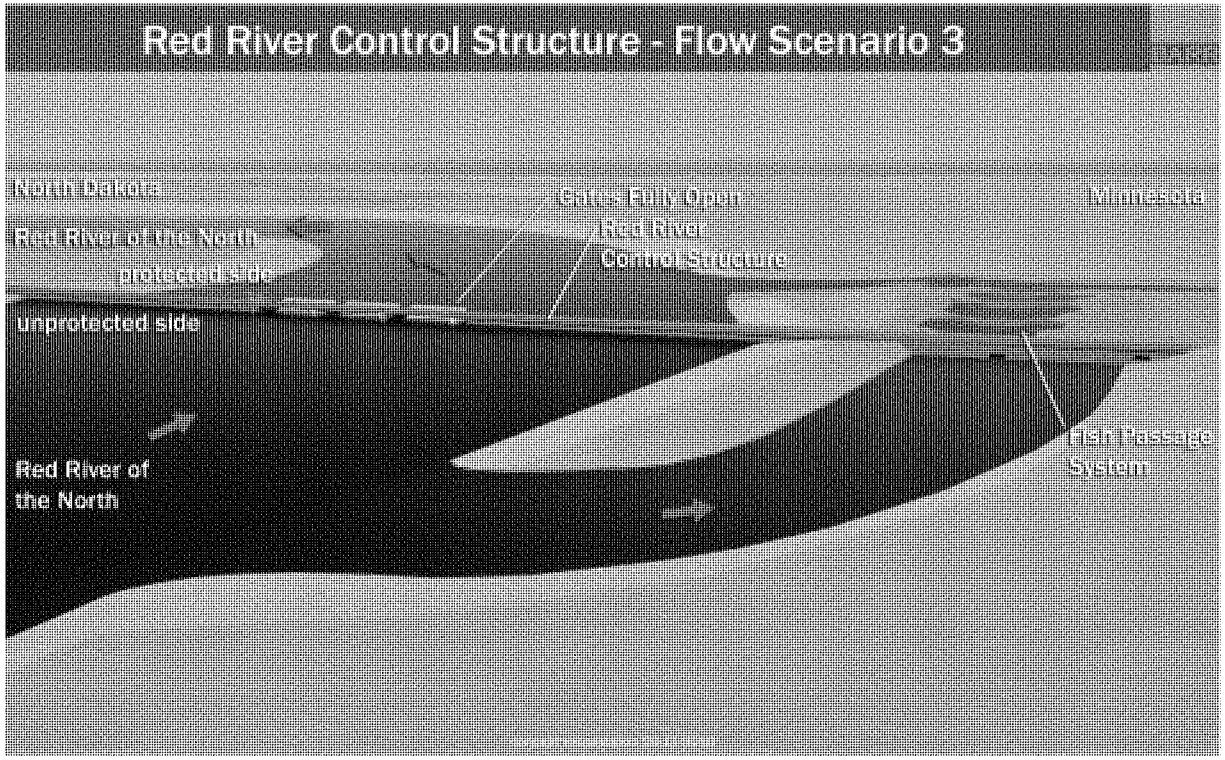
LPP ND East 35K
Flow Scenario 1
average flow

Figure F27



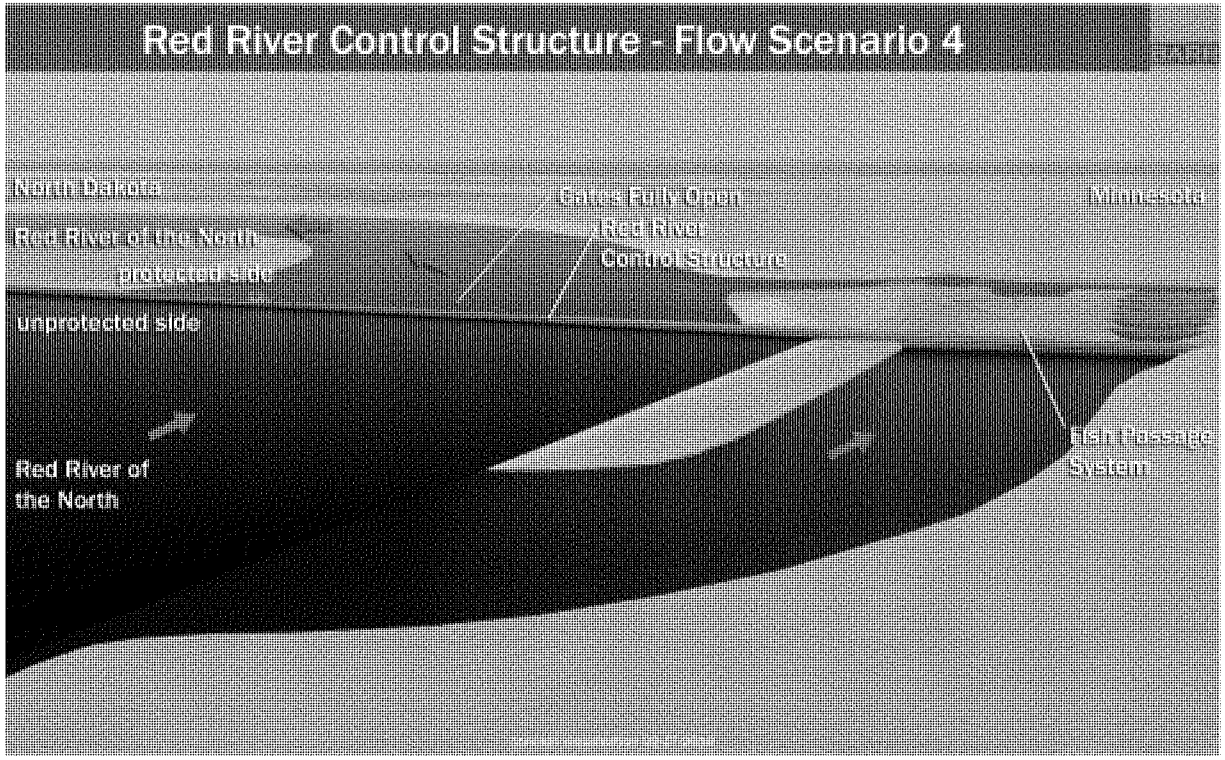
LPP ND East 35K
Flow Scenario 2
2-year flow

Figure F28



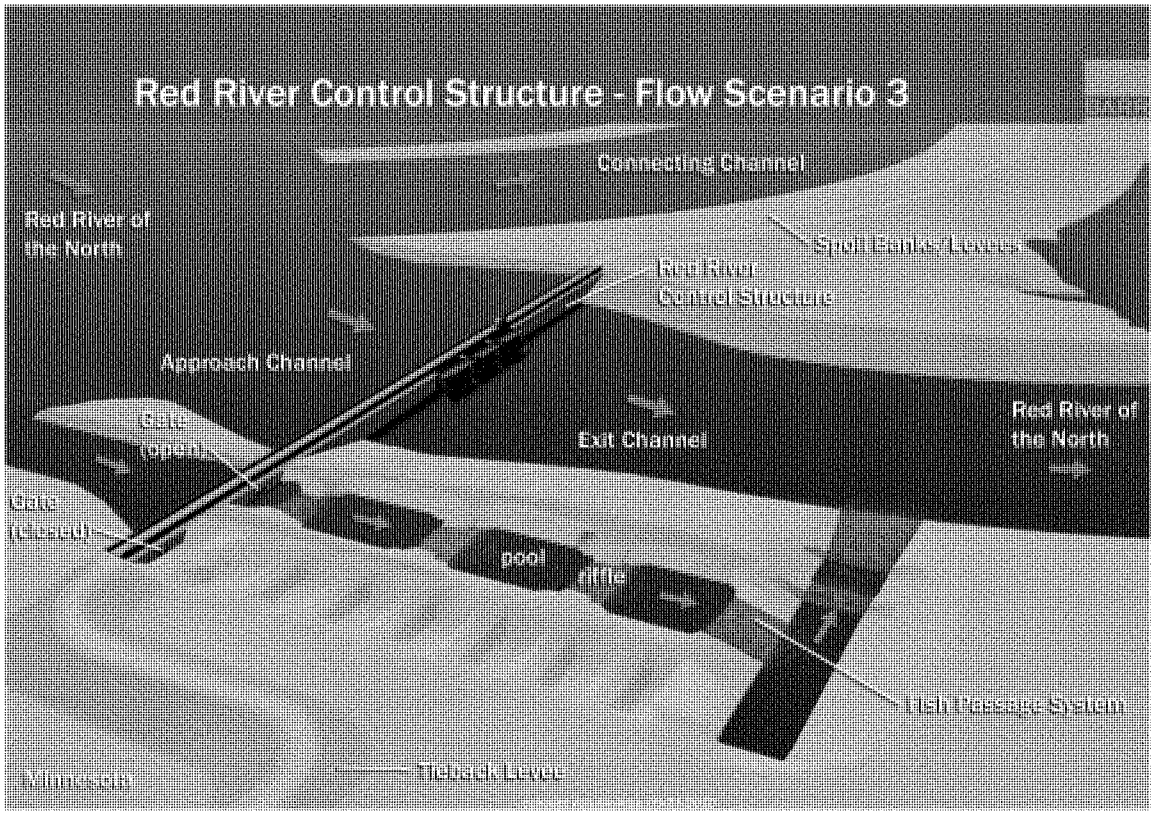
LPP ND East 35K
Flow Scenario 3
10-year flow

Figure F29



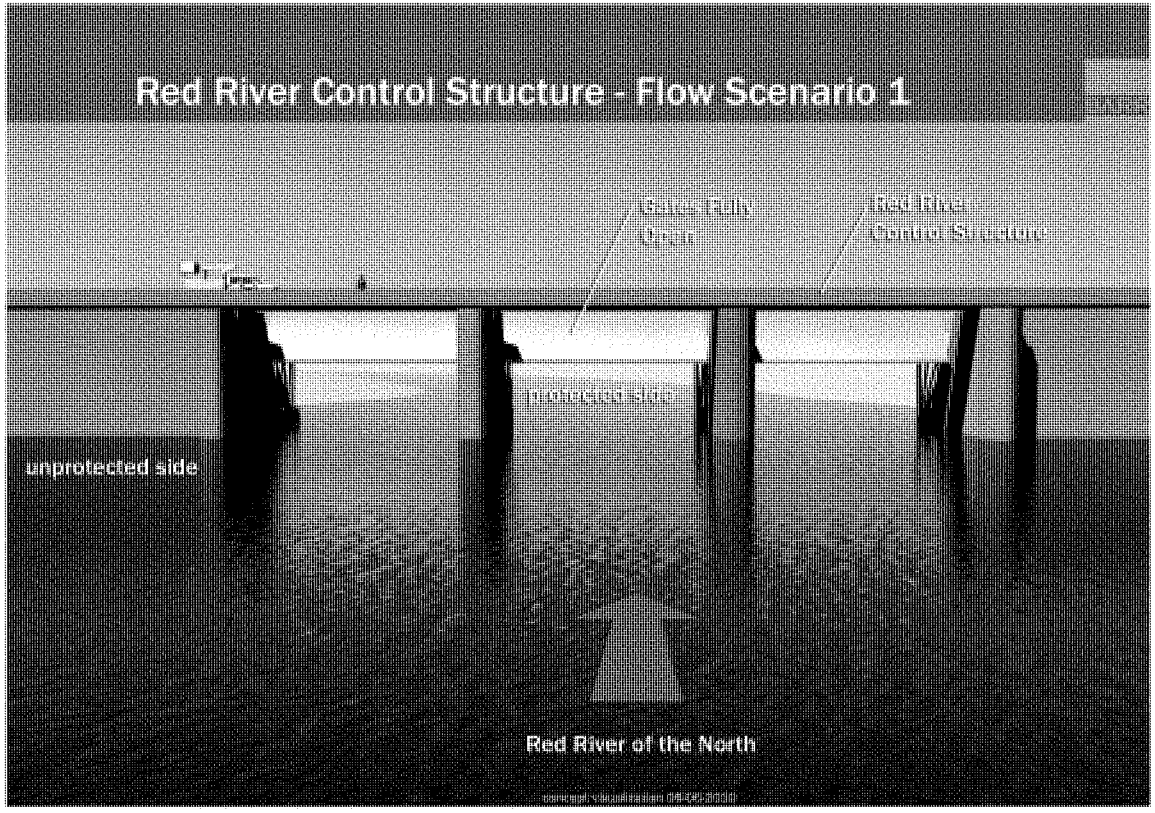
LPP ND East 35K
Flow Scenario 3
100-year flow

Figure F30



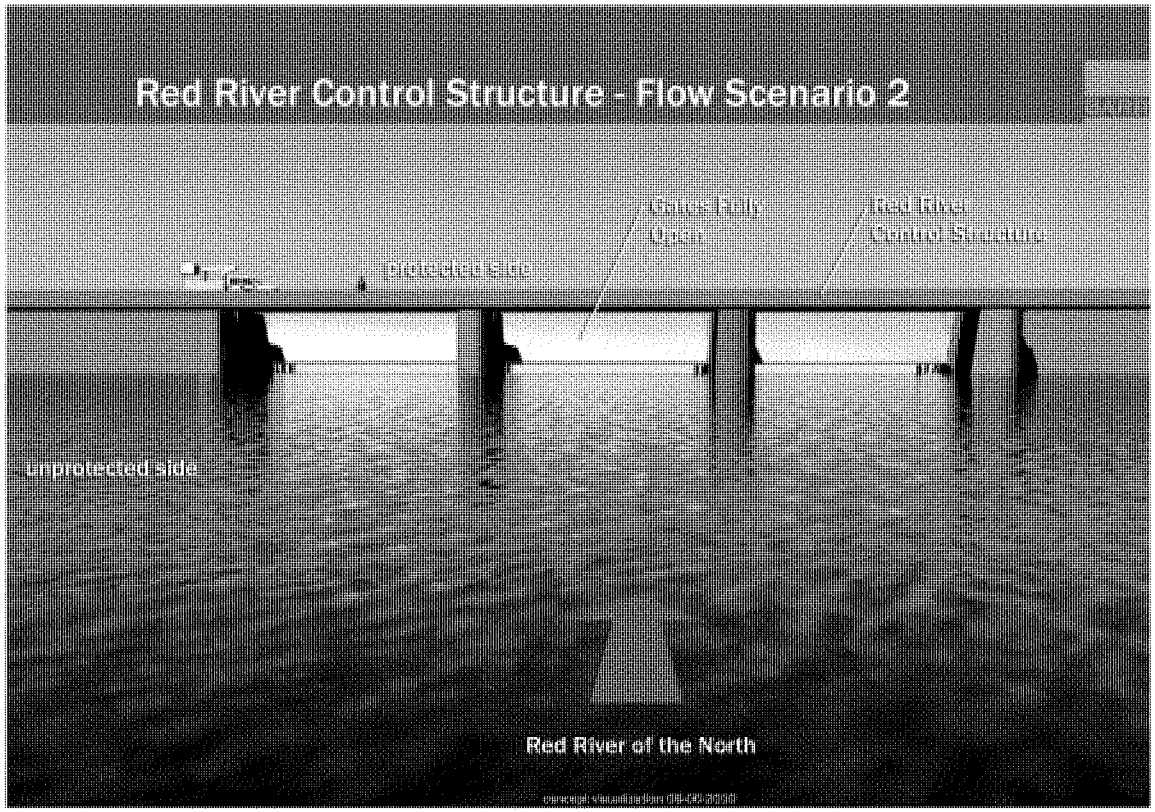
LPP ND East 35K
Flow Scenario 3
10-year flow

Figure F31



LPP ND East 35K
Flow Scenario 1:
average flow

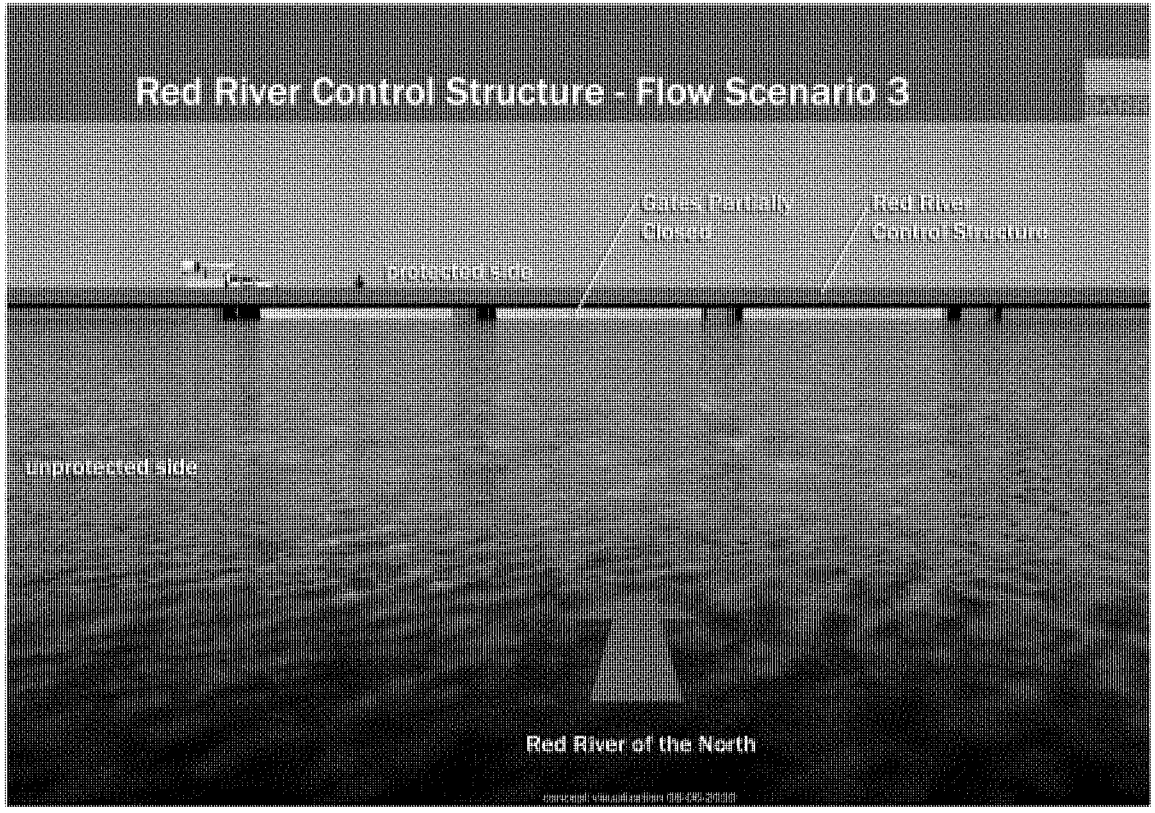
Figure F32



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

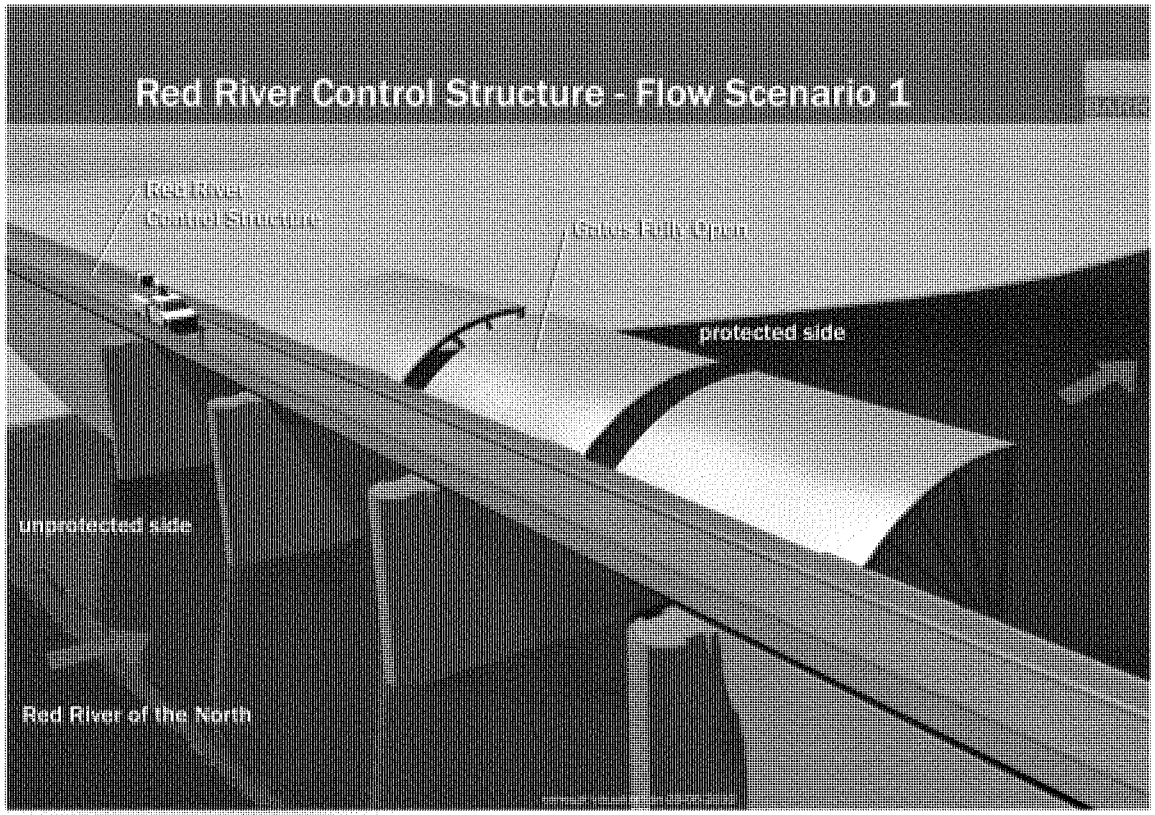
Figure F33

600



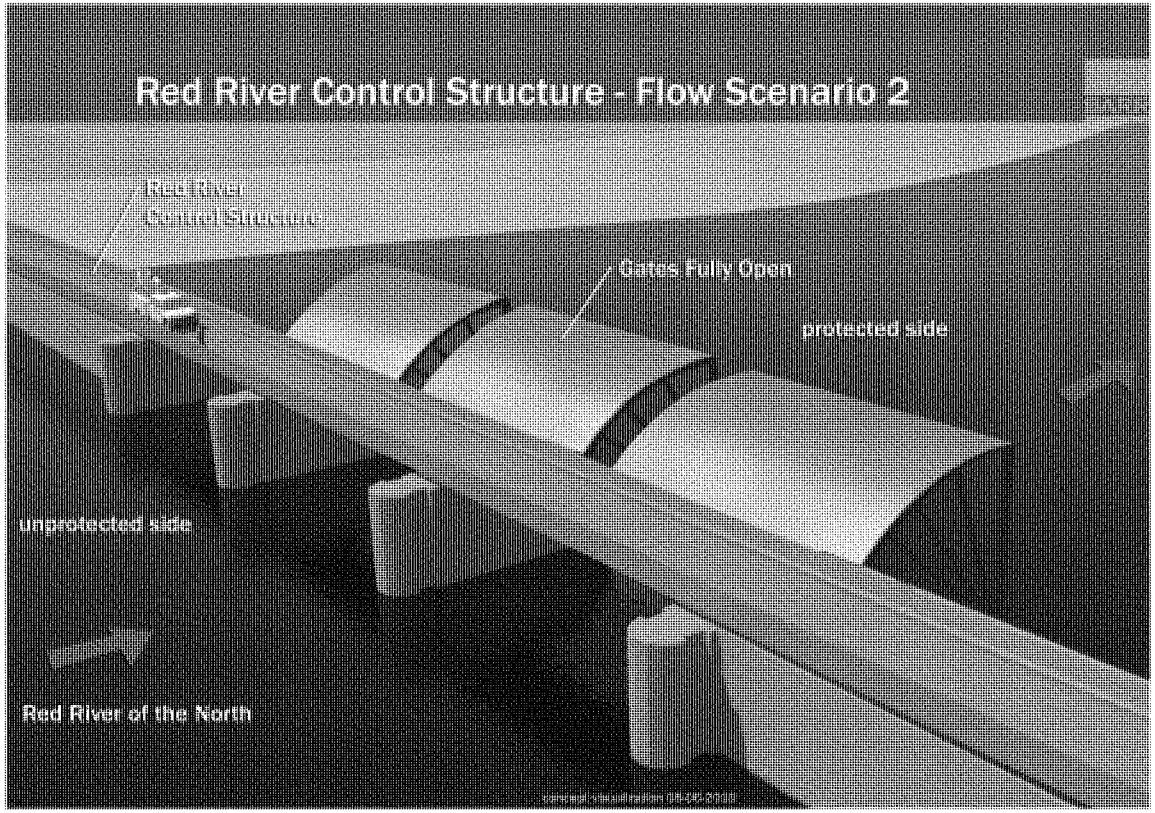
LPP ND East 35K
Flow Scenario 3
10-year flow

Figure F34



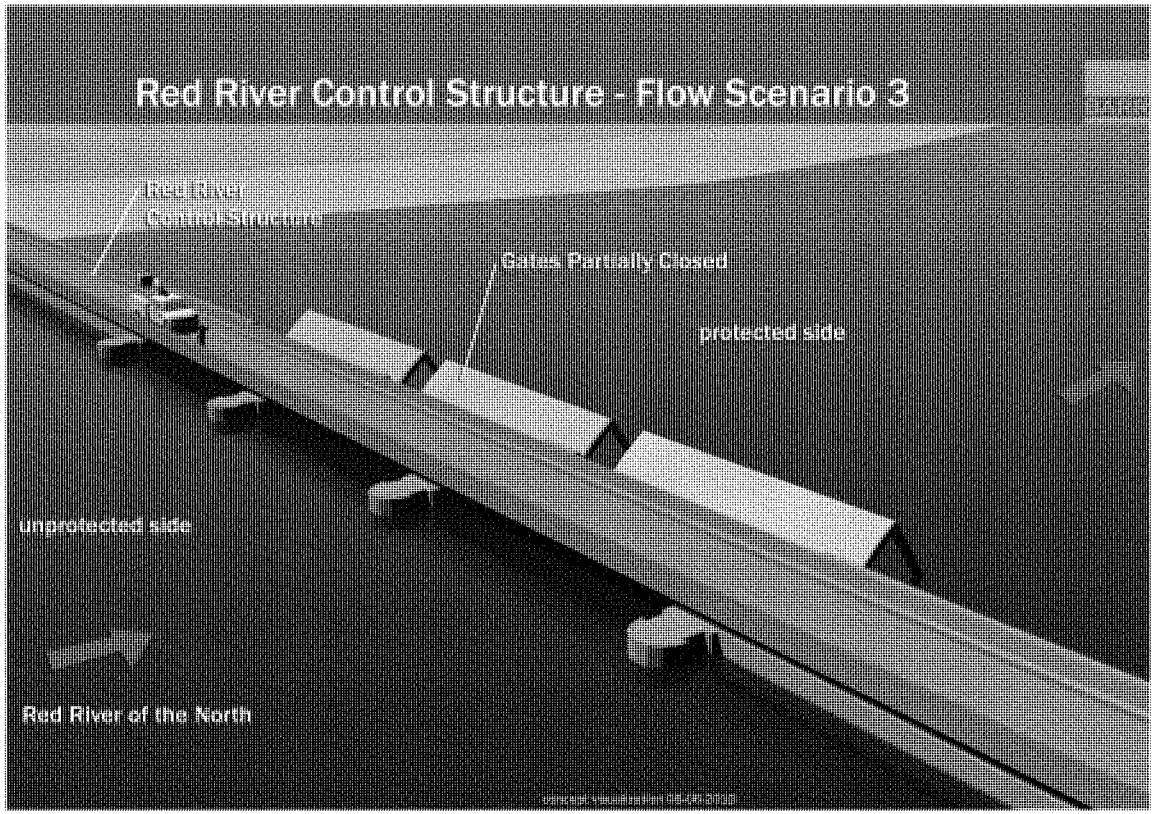
LPP ND East 35K
Flow Scenario 1:
average flow

Figure F35



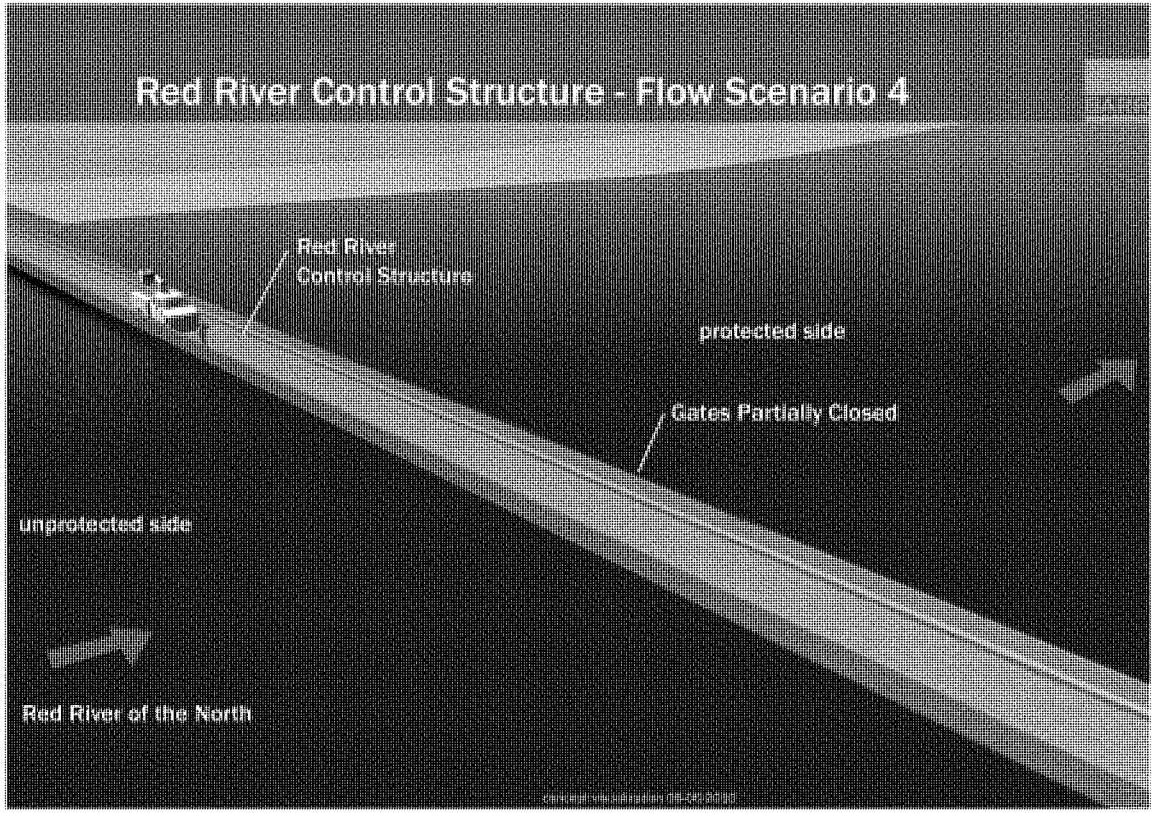
LPP ND East 35K
Flow Scenario 2:
2-year flow

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

Figure F-60
UPP AT SHEYENNE RIVER - SPILLWAY & CROSSING
DESIGN FLOWS FOR CONCURRENTAL EVENTS
TO RELEASE IN THE RED RIVER OF THE NORTH
Q-DC2
Q-SR2
Q-DC1
Q-SR1
Q-SR3
Flows (cfs)

Figure 4-41
LRP AT MAPLE RIVER - SPELLWAY & CROSSWAY
DESIGN FLOWS FOR CONCIDENTAL EVENTS
TO PEAKS IN THE REED RIVER OF THE NORTH
Q-SC1
Q-SW1
Q-WR1
Q-SC1
Q-WR2
Maple River
Reed River of the North

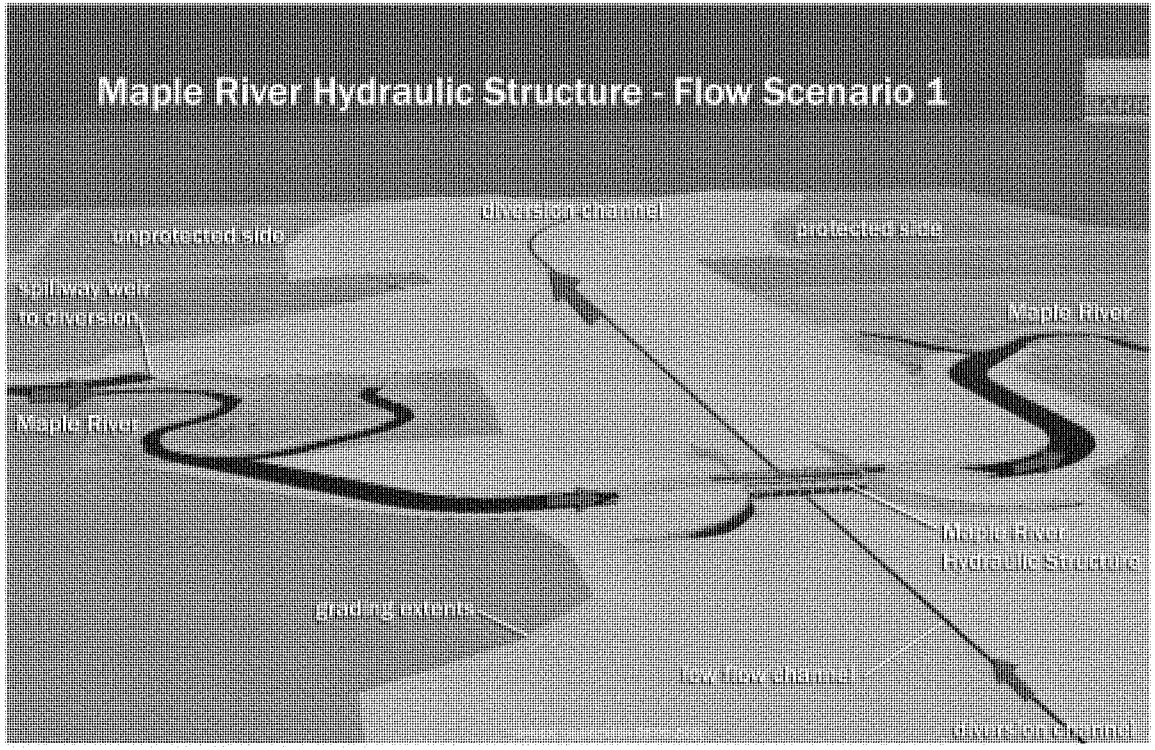
LPP ND East 35M
Flow Scenario 1
low flow channel,
no flow diverted
over spillway

Figure F42

Maple River Hydraulic Structure - Flow Scenario 2
LPP ND East 35K
Flow Scenario 2
unassisted
diversion flow
flow over spillway
Figure F43

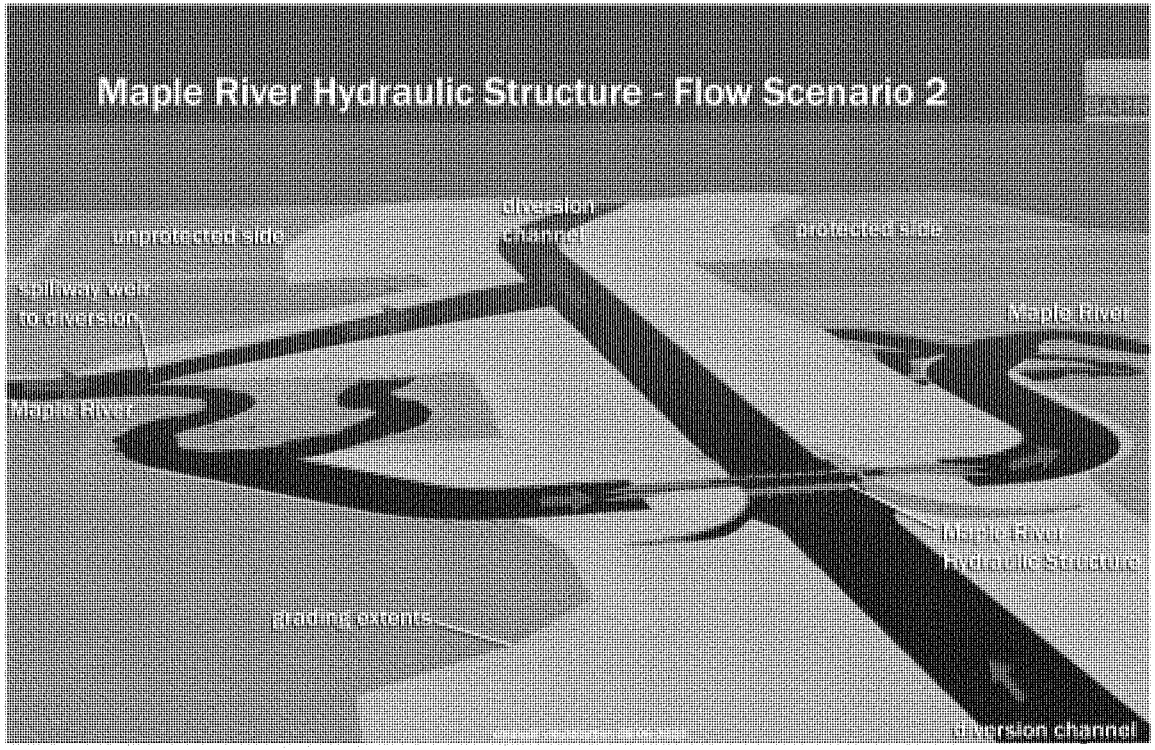
Maple River Hydraulic Structure - Flow Scenario 3
diversion channel
protected side
existing channel
(not to be disturbed)
Maple River
Maple River
hydraulic structure
diversion channel
Maple River
LFP ND East 35K
Flow Scenario 3
pressurized
diversion flow,
flow over spillway

Figure F44



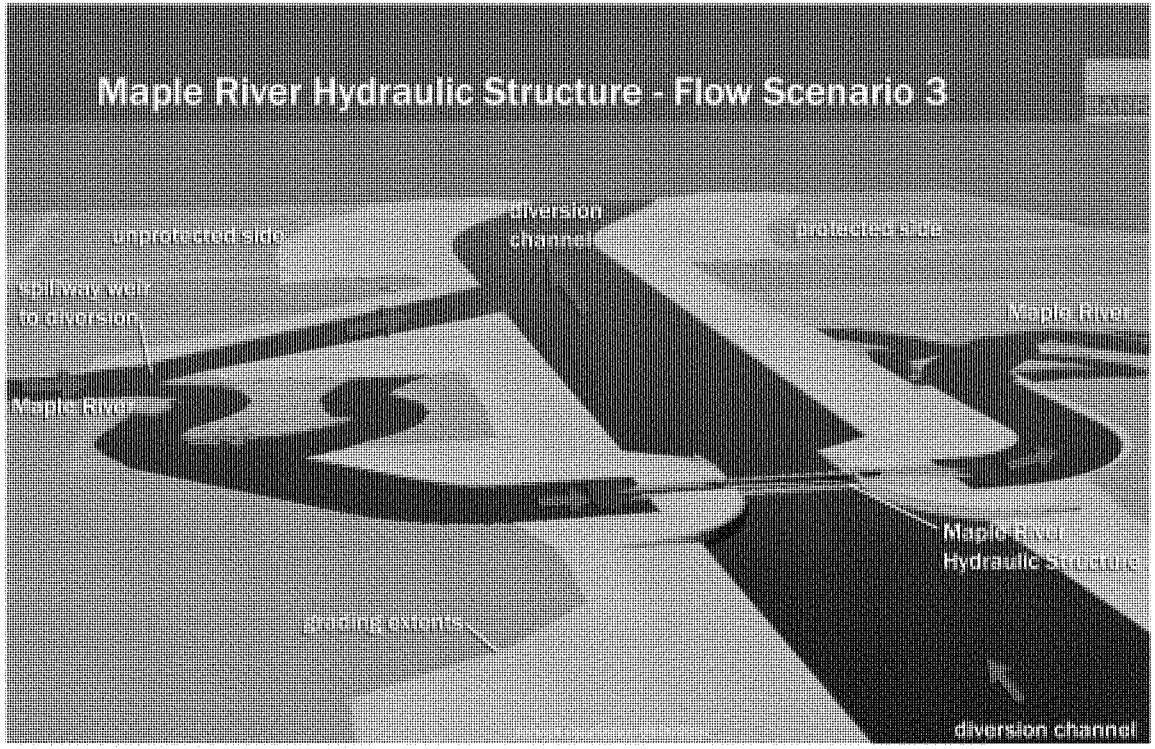
LPP ND East 35K
Flow Scenario 1
low flow channel,
no flow diverted over spillway

Figure F45



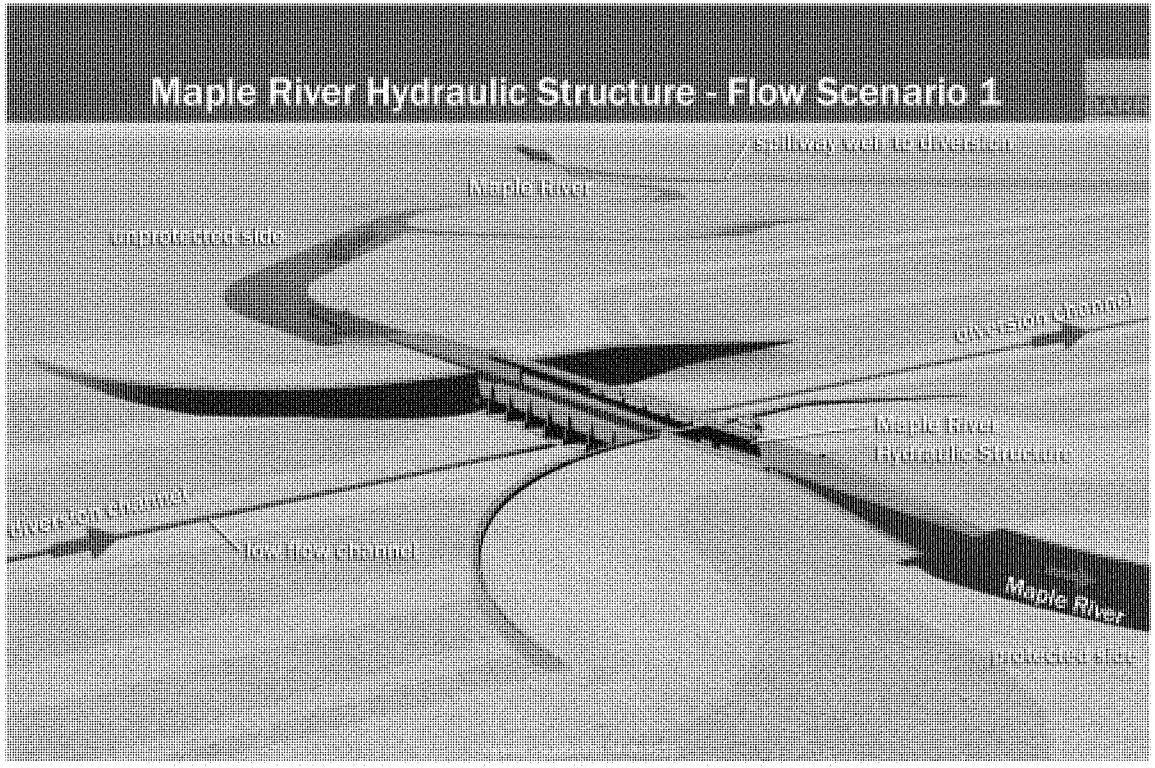
LPP ND East 35K
Flow Scenario 2
unpressurized diversion flow,
flow over spillway

Figure F46



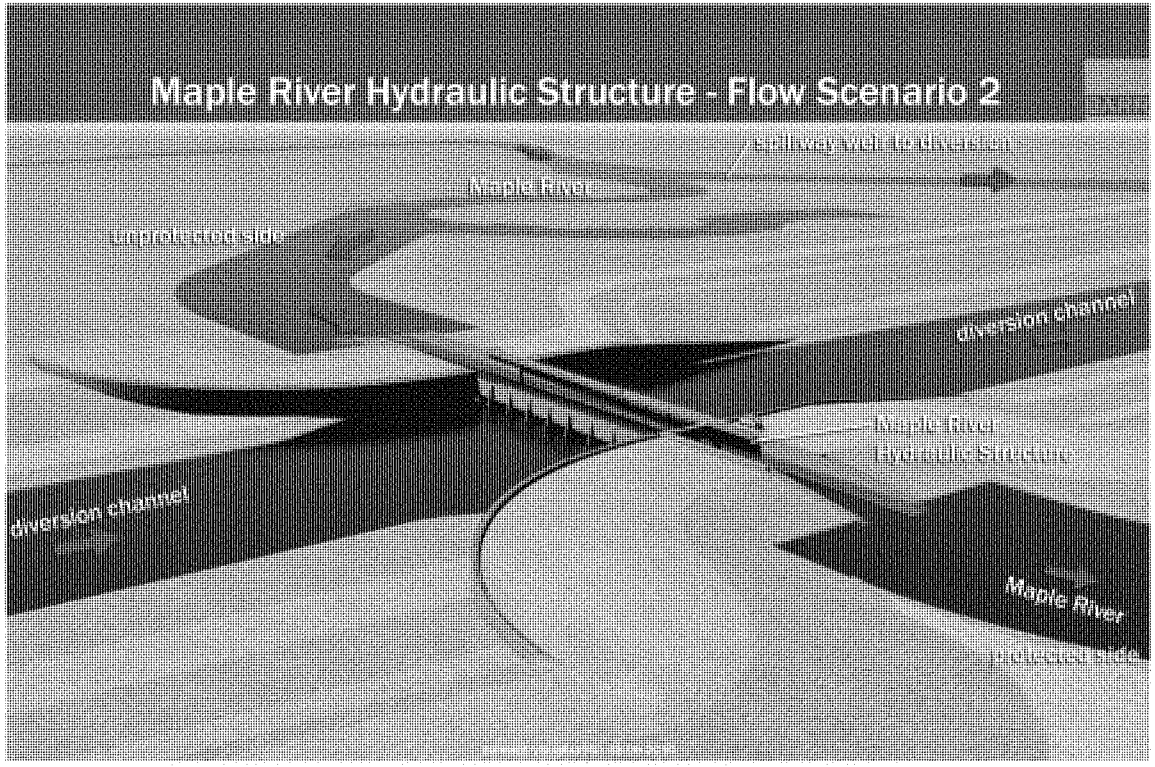
LPP ND East 35K
Flow Scenario 3
pressurized diversion flow,
flow over spillway

Figure F47



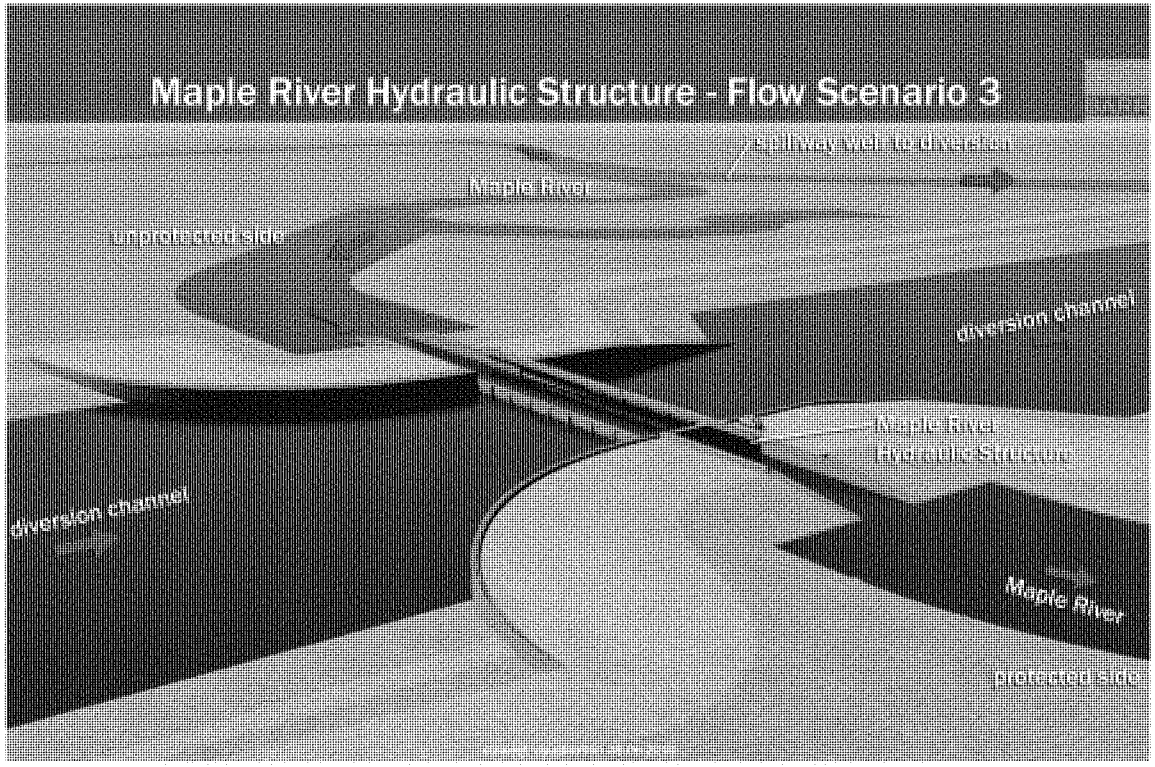
LPP ND East 35K
Flow Scenario 1
low flow channel,
no flow diverted over spillway

Figure F48



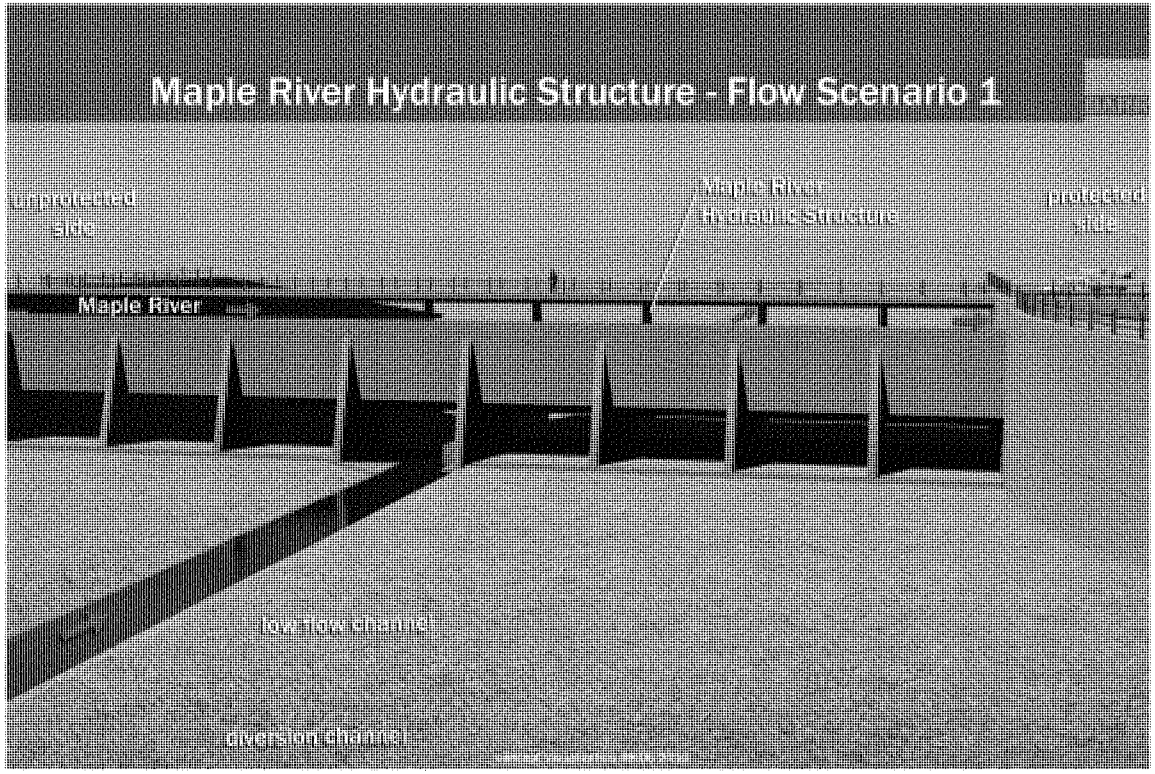
LPP ND East 35K
Flow Scenario 2
unpressurized diversion flow,
flow over spillway

Figure F49



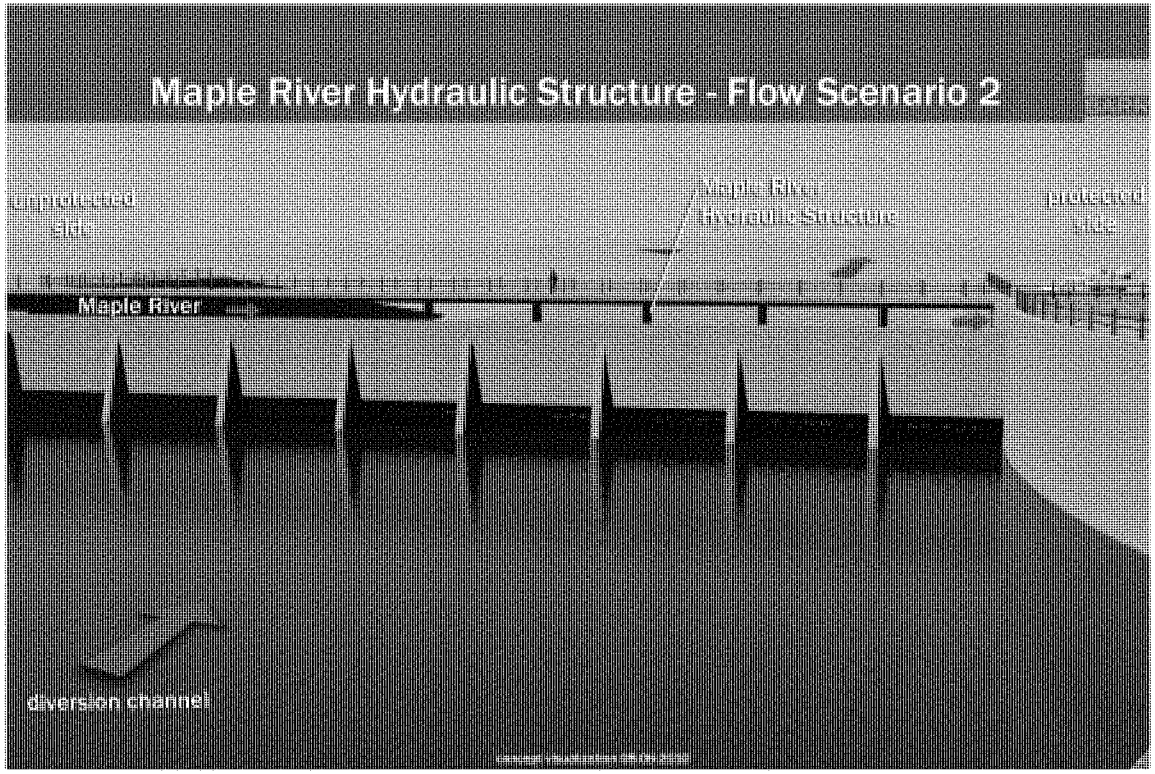
LPP ND East 35K
Flow Scenario 3
pressurized diversion flow,
flow over spillway

Figure F50



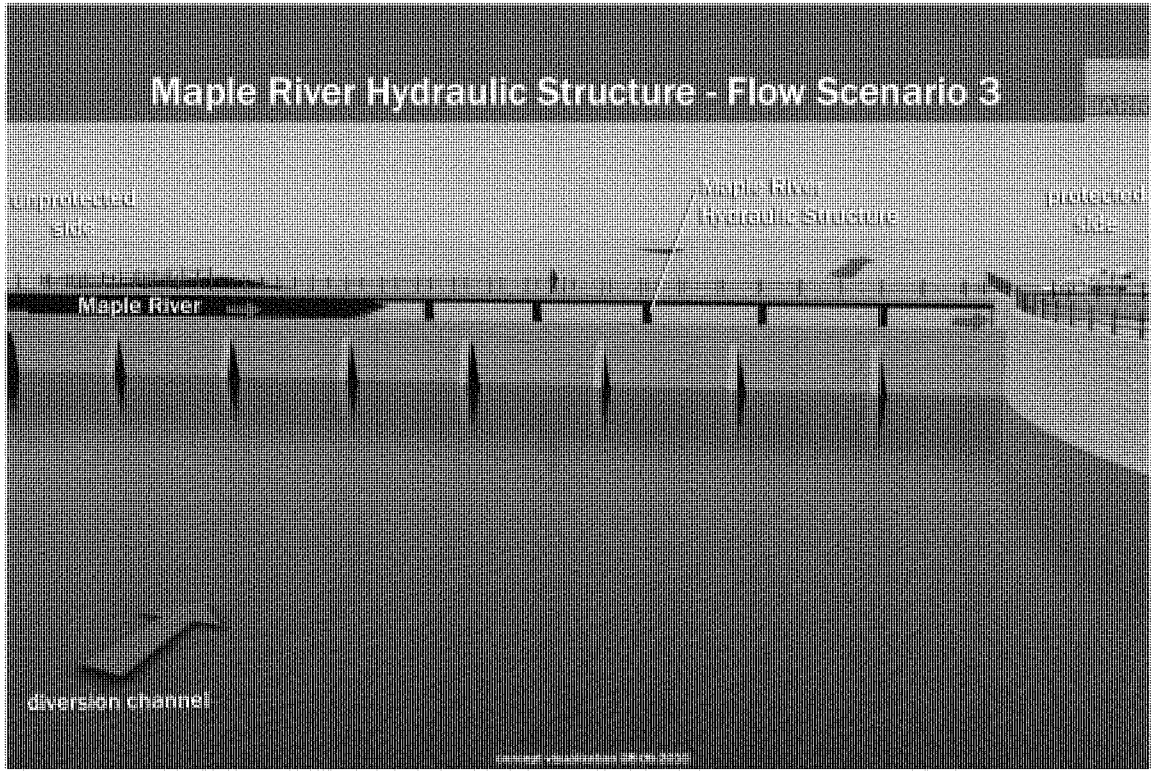
LPP ND East 35K
Flow Scenario 1
low flow channel,
no flow diverted over spillway

Figure F51



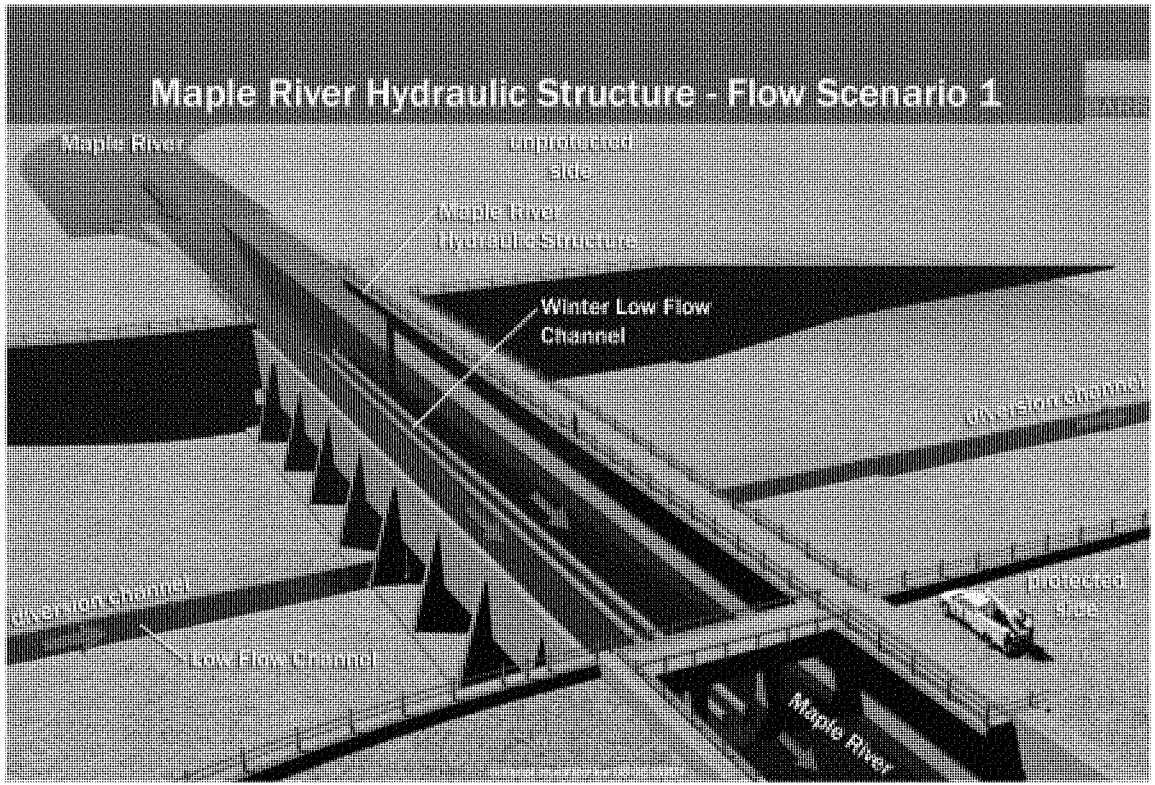
LPP ND East 35K
Flow Scenario 2
unpressurized diversion flow,
flow over spillway

Figure F52



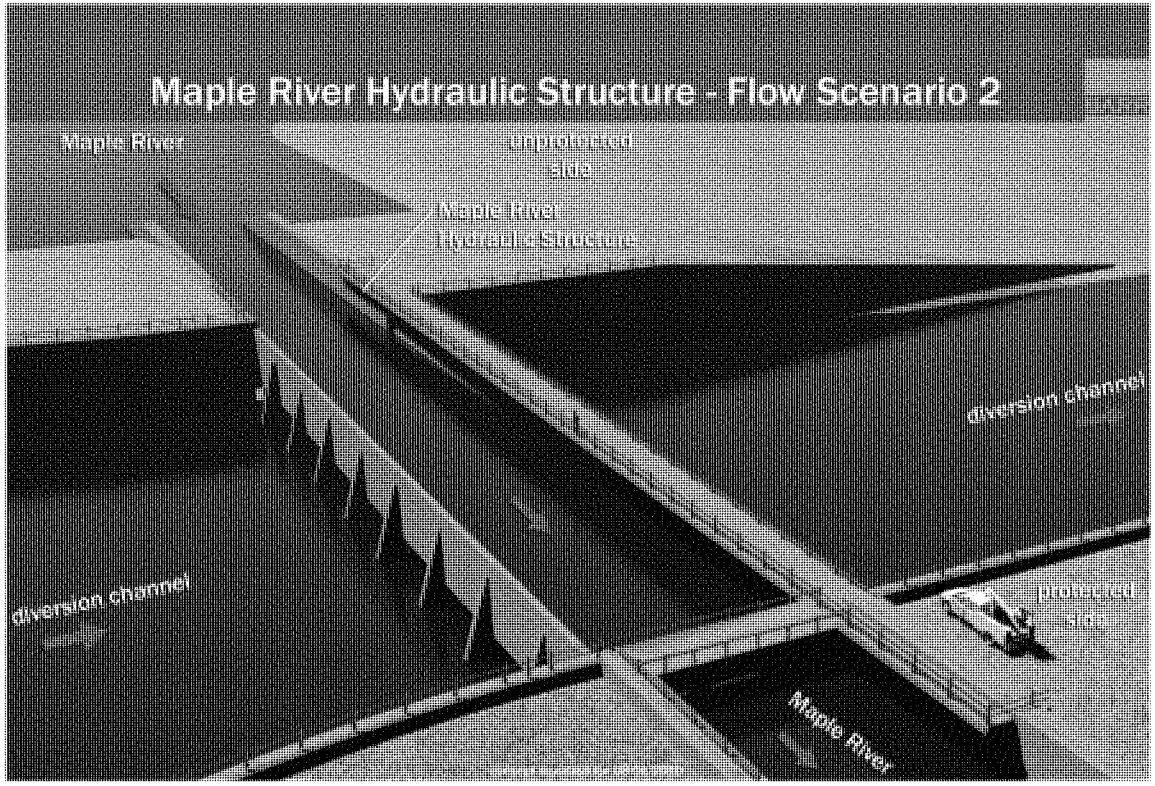
LPP ND East 35K
Flow Scenario 3
pressurized diversion flow,
flow over spillway

Figure F53



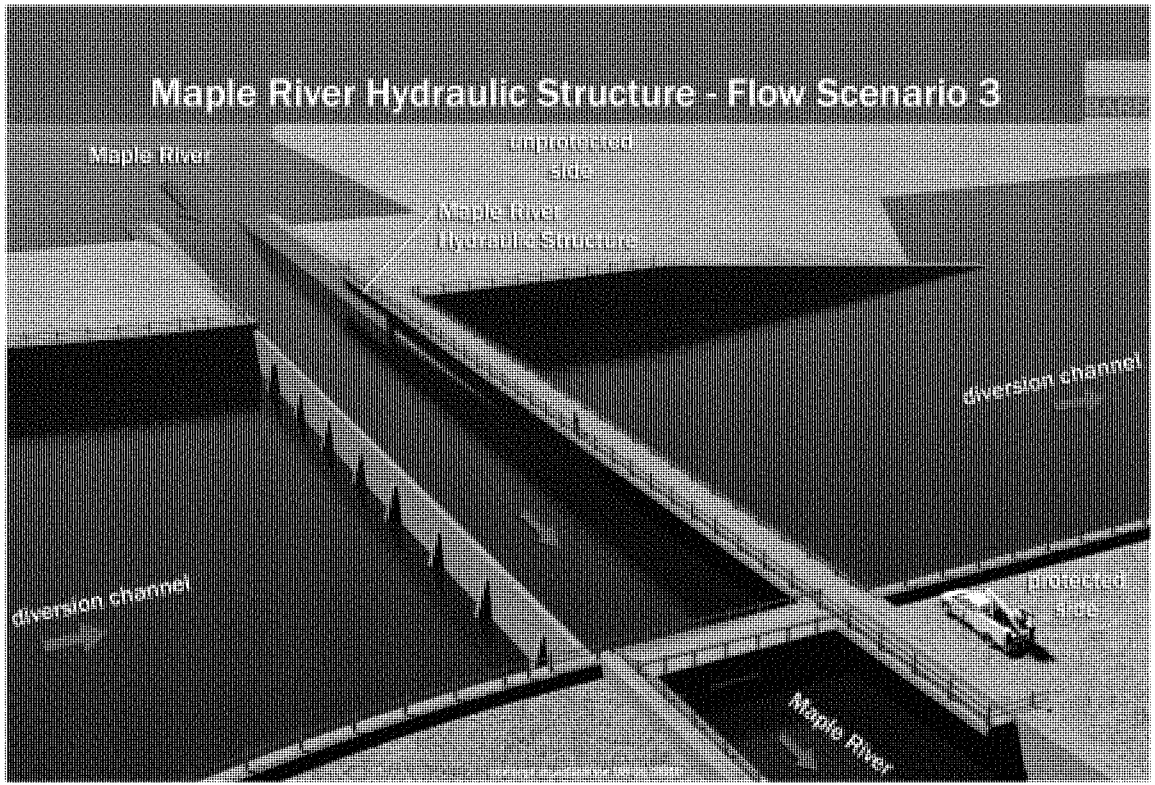
LPP ND East 35K
Flow Scenario 1
low flow channel,
no flow diverted over spillway

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 F31

Figure F32

Figure F33

Figure F34

Figure F35

Figure F36

Figure F37

Figure F38

Figure F39

Figure F41

Figure F42

Figure F43

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

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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.

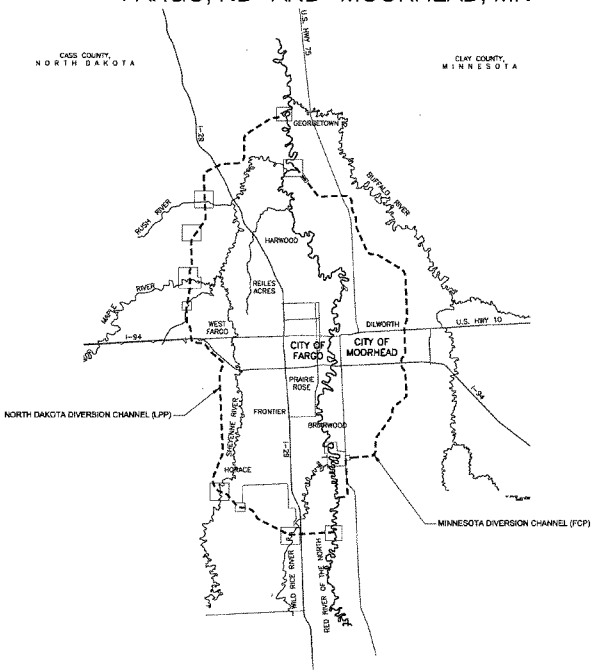
(d) An ice/debris load of 500 PLF along the structure will be used for the wing and retaining wall structures.

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

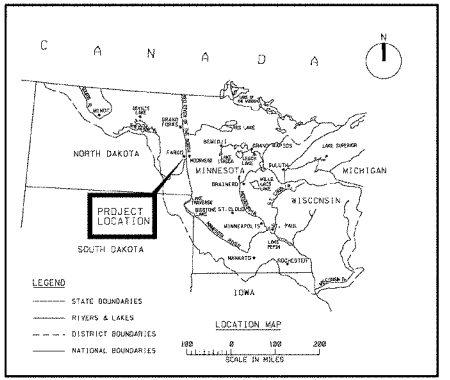
Notes:

- 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



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

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

NO.	DESCRIPTION	DATE	APPROVED
1	75% CONCEPT AND FCP CONSTRUCTION	12/2011	JACK SARGENT

DESIGNED BY: MK
DRAWN BY: MK
CHECKED BY: JS
QC BY: JS
APPROVED BY: JACK SARGENT
DATE APPROVED:
CONTRACT NO.
FILE NUMBER:

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

MARK PADMORE, PROJECT MANAGER
KEVIN SIPPOLA, PROJECT ENGINEER
MINNEAPOLIS, MN 55415

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

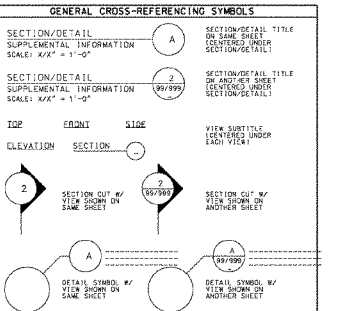
COVER SHEET
VICINITY MAP AND
LOCATION MAP

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
- EXISTING OVERHEAD POWER LINE
- UNDERGROUND POWER
- EXISTING GROUND SURFACE (PROFILE)
- PROPOSED GROUND SURFACE (PROFILE)
- EDGE OF WATER
- WATERMAIN
- TELEPHONE
- ACCESS ROAD
- EXISTING GRADE ELEVATION CONTOUR
- PROPOSED FINISH GRADE ELEVATION CONTOUR

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

US Army Corps of Engineers® St. Paul District

NO.	DESCRIPTION	DATE	BY
	PRELIMINARY SUBMITTAL	10/31/2011	
	FINAL SUBMITTAL	12/30/2011	

DESIGNED BY: DRAWN BY: CHECKED BY: QUALITY CONTROL BY: SUPERVISED BY: DATE: 12/30/2011 CONTRACT NO. DACW37-[illegible] FSA TASKORD. NO. [illegible]

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

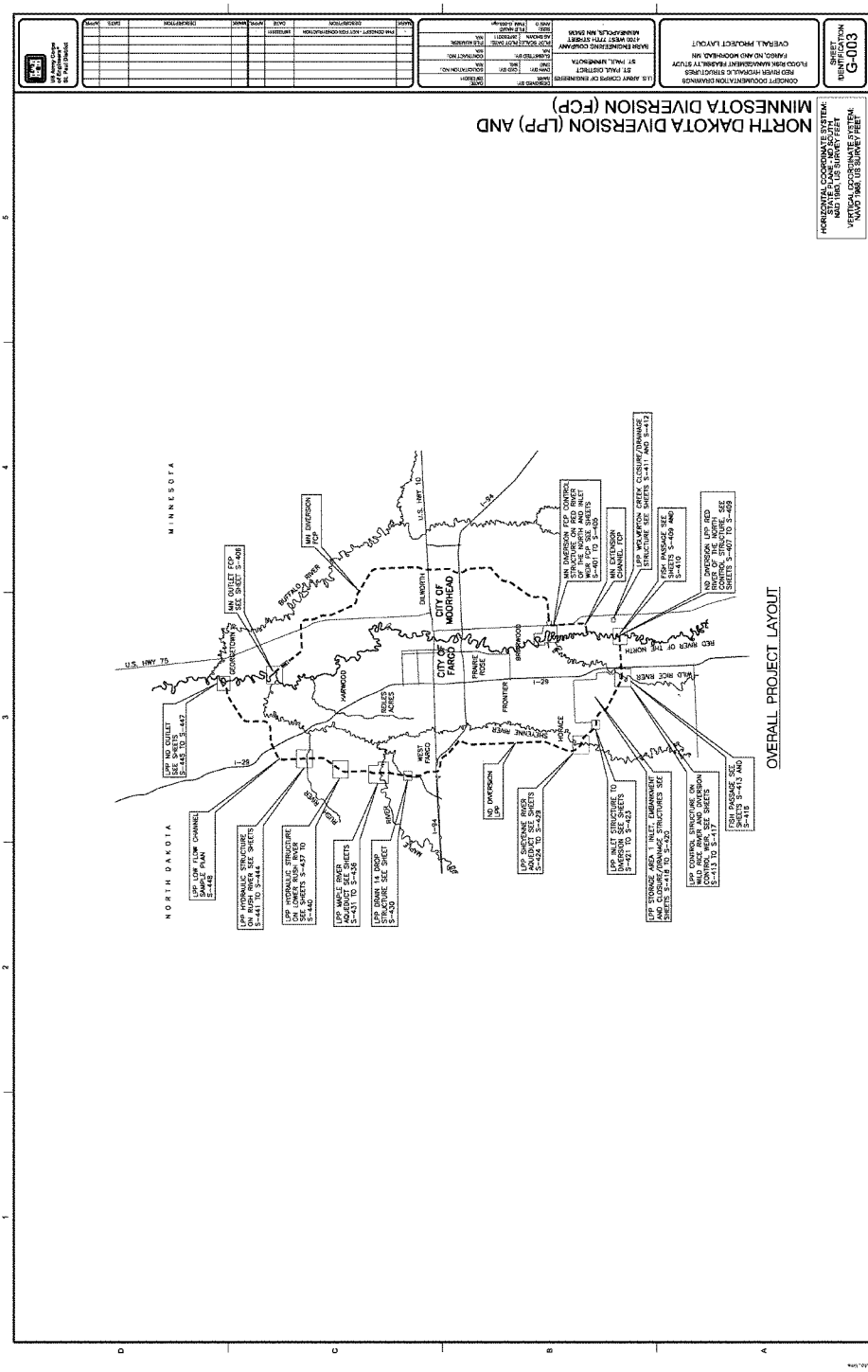
SHAW ENGINEERING COMPANY 1100 NORTH 4TH STREET MINNEAPOLIS, MN 55405

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION INTAKE & STRUCTURES
FLOOD DIVERSION CHANNEL & STRUCTURES
FARGO, ND AND MOORHEAD, MN

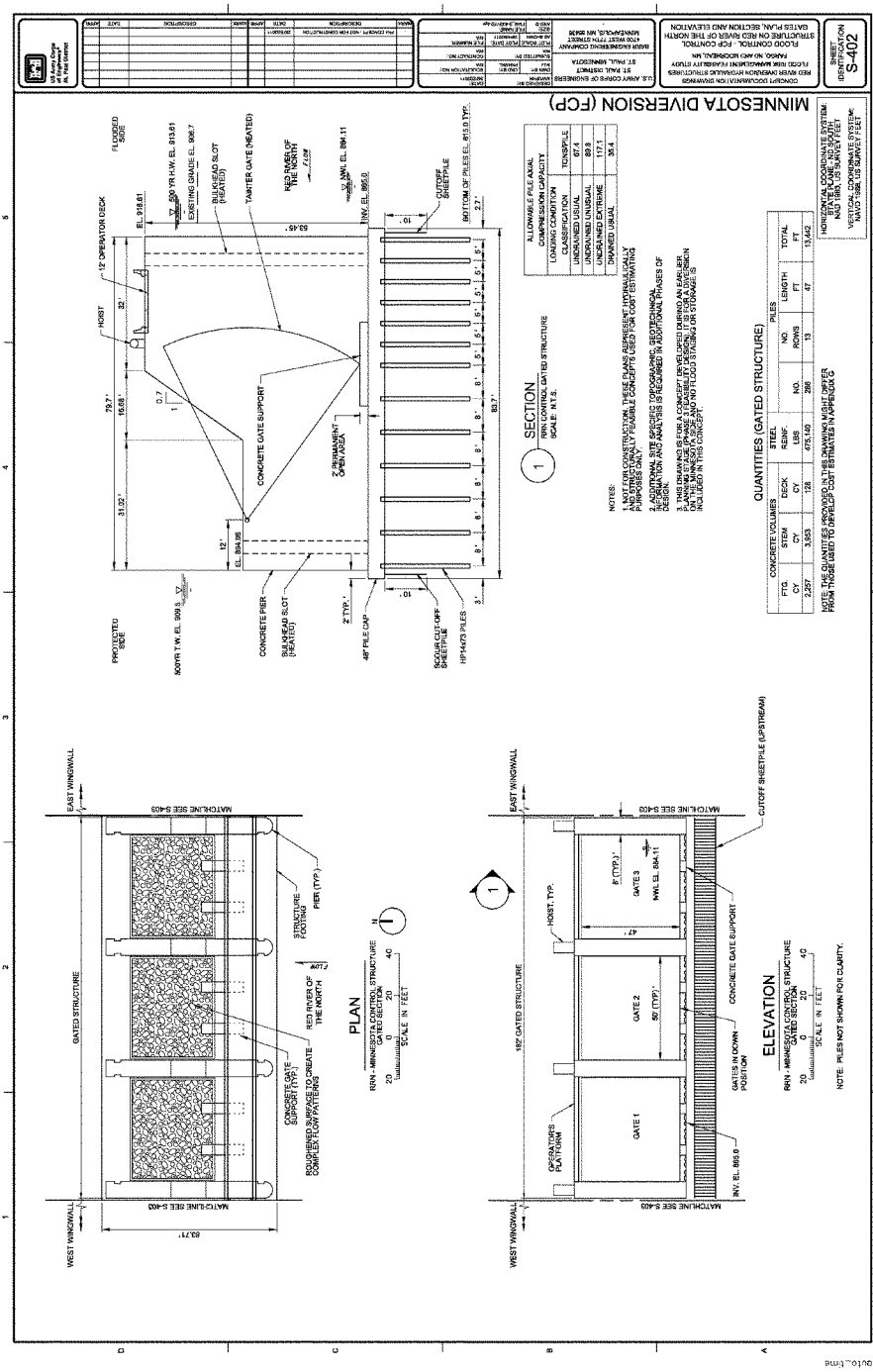
DRAWING INDEX AND GENERAL INFORMATION

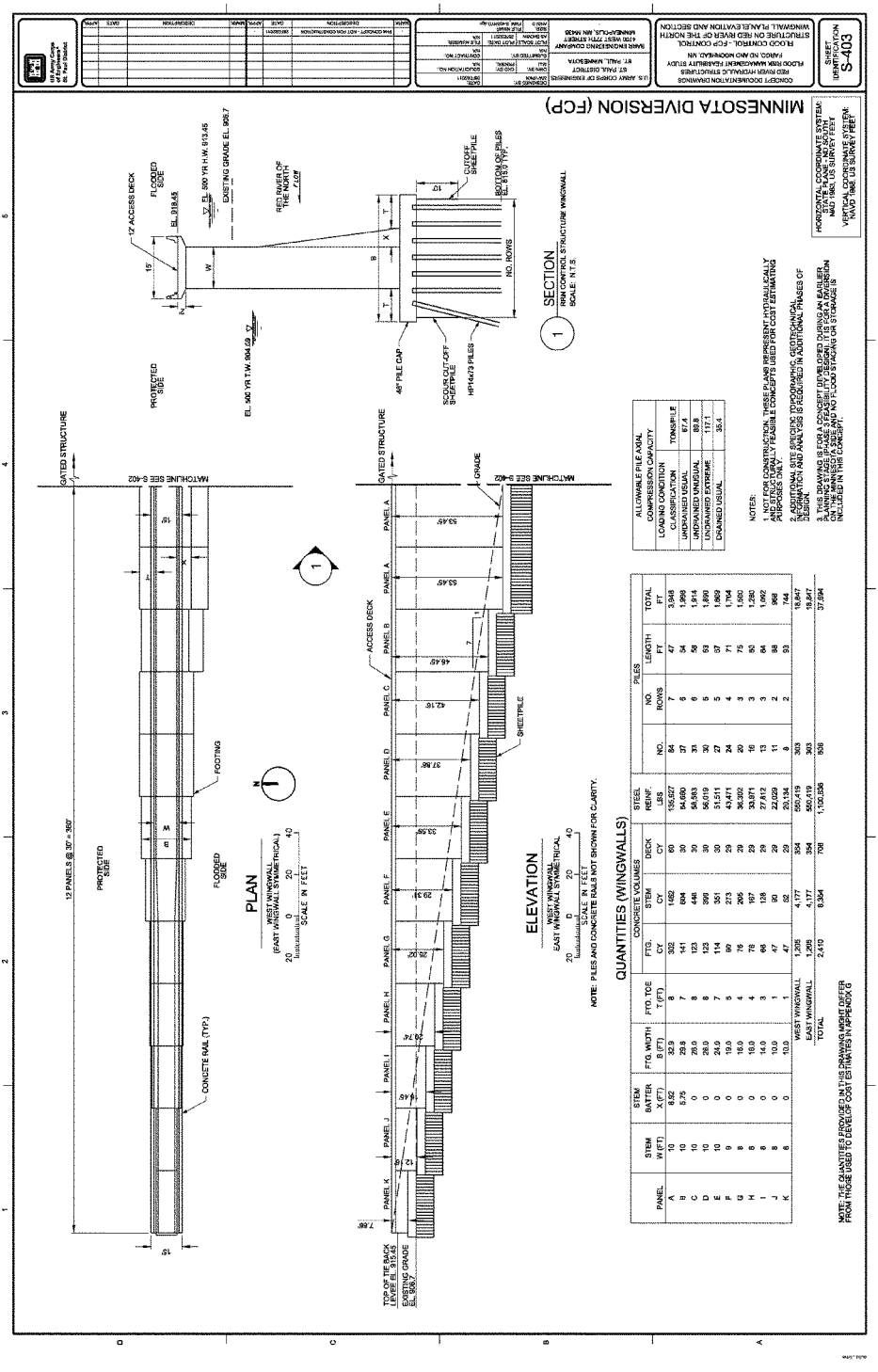
SHEET IDENTIFICATION
G-002

639


NORTH DAKOTA DIVERSION (LPP) AND MINNESOTA DIVERSION (FCP)
OVERALL PROJECT LAYOUT
CONCEPT DOCUMENTATION DRAWINGS
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN
OVERALL PROJECT LAYOUT
SHEET IDENTIFICATION G-003
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
BAKER ENGINEERING COMPANY
4100 WEST 77TH STREET
MINNEAPOLIS, MN 55435
HORIZONTAL COORDINATE SYSTEM: STATE PLANE ZONE 4202 (FT) NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
MINNESOTA
NORTH DAKOTA
RED RIVER OF THE NORTH
BUFFALO RIVER
WILD RICE RIVER
SHEYENNE RIVER
U.S. HWY 75
U.S. HWY 10
I-94
I-29
MOORHEAD
CITY OF FARGO
DILWORTH
HARWOOD
HORACE
WEST FARGO
PRAIRIE ROSE
FRONTIER
BRIDGEWOOD
ARGUSVILLE
REED
LPP NO. OUTLET SEE S-445 TO S-447
LPP LOW FLOW CHANNEL SAMPLE PLAN S-448
LPP HYDRAULIC STRUCTURE ON LPPH RIVER SEE SHEETS S-449 TO S-450
LPP HYDRAULIC STRUCTURE ON LOWER RUSH RIVER SEE SHEETS S-437 TO S-438
LPP MAPLE RIVER AQUEDUCT SEE SHEETS S-431 TO S-436
LPP DRAIN 14 DROP STRUCTURE SEE SHEET S-430
LPP STORAGE AREA 1 INLET, EMBANKMENT AND STRUCTURES SEE SHEETS S-418 TO S-429
LPP CONTROL STRUCTURE ON FORCED CHANNEL AND DIVERSION CONTROL WEIR, SEE SHEETS S-413 TO S-417
FISH PASSAGE SEE SHEETS S-413 AND S-418
LPP INLET STRUCTURE TO S-420 AND S-423 SEE SHEETS S-419 TO S-423
LPP SHEYENNE RIVER AQUEDUCT SEE SHEETS S-424 TO S-429
ND DIVERSION LPP
MN DIVERSION FCP CONTROL STRUCTURE ON RED RIVER AND INLET WEIR FCP SEE SHEETS S-401 TO S-405
MN EXTENSION CHANNEL FCP
LPP WOLVERTON CREEK CLOSURE/DIVERSION STRUCTURE SEE SHEETS S-411 AND S-412
FISH PASSAGE SEE SHEETS S-409 AND S-410
ND DIVERSION LPP RED RIVER OF THE NORTH DIVERSION CHANNEL SEE SHEETS S-407 TO S-409
MN OUTLET FCP SEE SHEET S-406
MN DIVERSION FCP

MINNESOTA DIVERSION (FCP)
PLAN
RRN - MINNESOTA INLET STRUCTURE
SCALE IN FEET
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY FEASIBLE CONCEPTS. GEOTECHNICAL FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INVESTIGATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
3. THIS DRAWING IS FOR A CONCEPT DEVELOPED DURING AN EARLIER PHASE OF THE PROJECT. THE CONCEPT SHOWN IS NOT THE DIVERSION ON THE MINNESOTA SIDE AND NOT LOCD STAGING OR STORAGE IS INCLUDED IN THIS CONCEPT.
FEMA DEEDED PROPERTIES
76TH AVE S.
PROTECTED SIDE
FLOODWALL TO AVOID FEMA DEEDED PROPERTY
FOREST RIVER ROAD
ACCESS ROAD FOR RRN CLOSURE
RAMP OVER TIE BACK LEVEE
TIE BACK LEVEE
UNPROTECTED SIDE
RED RIVER OF THE NORTH
ICE/DEBRIS CONTROL MEASURE
ICE/DEBRIS CONTROL MEASURE AT ENTRANCE TO EXTENSION CHANNEL. ICE/DEBRIS CONTROL MEASURE INCLUDES ACCESS ROAD, POWER AND SCADA
EXTENSION CHANNEL
RED RIVER VALLEY COOP
HIGHWAY 75
BRIDGE OVER DIVERSION
DIVERSION CHANNEL
DIVERSION INLET WEIR SEE DRAWING S-405
SCADA MONITORING POINT (HEAD ACROSS WEIR)
WEIR MAINTENANCE PARKING AREA
PROPOSED ELECTRICAL SERVICE TO WEIR AREA
PROPOSED ELECTRICAL SERVICE TO CLOSURE STRUCTURE
SCADA MONITORING POINT (TAIL ACROSS ICE CONTROL MEASURE)
APPROXIMATE LIMITS OF TEMPORARY FLOOD PROTECTION FOR CONSTRUCTION
ACCESS ROAD FOR RRN, ICE/DEBRIS CONTROL AND DIVERSION INLET WEIR
MAINTENANCE BUILDING AND PARKING
SCADA MONITORING POINT (ACROSS CLOSURE STRUCTURE)
TEMPORARY 1/4 TO 50YEAR EVENT SEE DRAWING S-404
RRN CLOSURE STRUCTURE SEE DRAWING S-402
RIPRAP (TYP.)
CLAY COUNTY ST.
HIGHWAY 75
HIGHWAY 81
30TH ST S.
HORIZONTAL COORDINATE SYSTEM: NAD 1983 (2011) ADJUST NSRS 2007 SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
CONCEPT PDC 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 DESCRIPTION
S-401
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
MAIN SUPERSEDING DRAWING
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
U.S. Army Corps of Engineers
St. Paul District

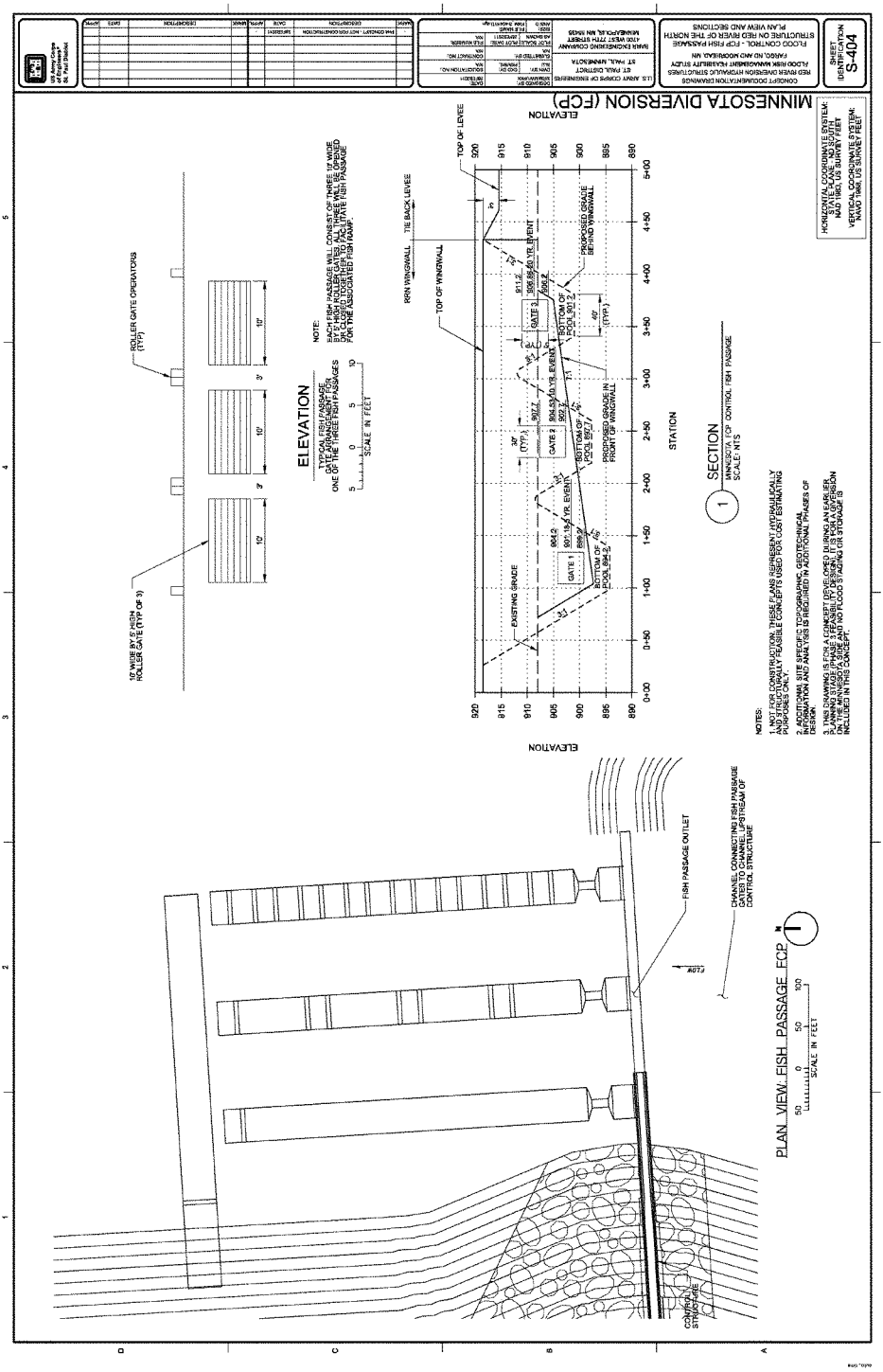

MINNESOTA DIVERSION (FCP)
SECTION
RRN - MINNESOTA CONTROL STRUCTURE GATED SECTION
SCALE: NTS.
PLAN
RRN - MINNESOTA CONTROL STRUCTURE GATED SECTION
SCALE IN FEET
ELEVATION
RRN - MINNESOTA CONTROL STRUCTURE GATED SECTION
SCALE IN FEET
NOTE: PILES NOT SHOWN FOR CLARITY.
QUANTITIES (GATED STRUCTURE)
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES.
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 1) AND IS NOT INTENDED FOR A LATER STAGE OF DESIGN. ALL ITEMS NOT SHOWN OR DESCRIBED IN THIS DRAWING ARE INCLUDED IN THIS CONCEPT.
ALLOWABLE PILE AXIAL COMPRESSION CAPACITY
HORIZONTAL COORDINATE SYSTEM: NAD 1983 (2011) UTM ZONE 14N, US SURVEY FEET
VERTICAL DATUM: NAVD 1988, US SURVEY FEET
SHEET IDENTIFICATION S-402
FLOOD CONTROL - FCP CONTROL STRUCTURES RSM MANDATORY FLOODWAY STUDY
FARGO, ND AND MOORHEAD, MN
GATES SIM ELEVATION AND SECTION
FLOOD CONTROL - FCP CONTROL STRUCTURES RSM MANDATORY FLOODWAY STUDY
FARGO, ND AND MOORHEAD, MN
GATES IN DOWN POSITION
GATED STRUCTURE
CONCRETE GATE SUPPORT
CONCRETE PIER
CUTOFF SHEETPILE
HP14X73 PILES
48' PILE CAP
2' TYP.
2' PERMANENT OPEN AREA
RED RIVER OF THE NORTH
TAINTER GATE (MOUNTED)
BULKHEAD SLOT (MOUNTED)
12' OPERATOR DECK
HOIST
PROTECTED SIDE
FLOODED SIDE
EL. 918.61
EL. 894.98
INV. EL. 884.11
INV. EL. 865.0
BOTTOM OF PILES EL. 815.0 TYP.
63.45'
79.7'
15.68'
31.02'
32'
12'
0.7
1
STRUCTURE FOOTING
PIER (TYP.)
ROUGHENED SURFACE TO CREATE COMPLEX FLOW PATTERNS
WEST WINGWALL
EAST WINGWALL
MATCHLINE SEE S-403
63.71'
192' GATED STRUCTURE
OPERATORS PLATFORM
HOIST, TYP.
GATE 1
GATE 2
GATE 3
85' (TYP.)
47'
CUTOFF SHEETPILE (UPSTREAM)
GATES IN DOWN POSITION
CONCRETE GATE SUPPORT
INV. EL. 865.0

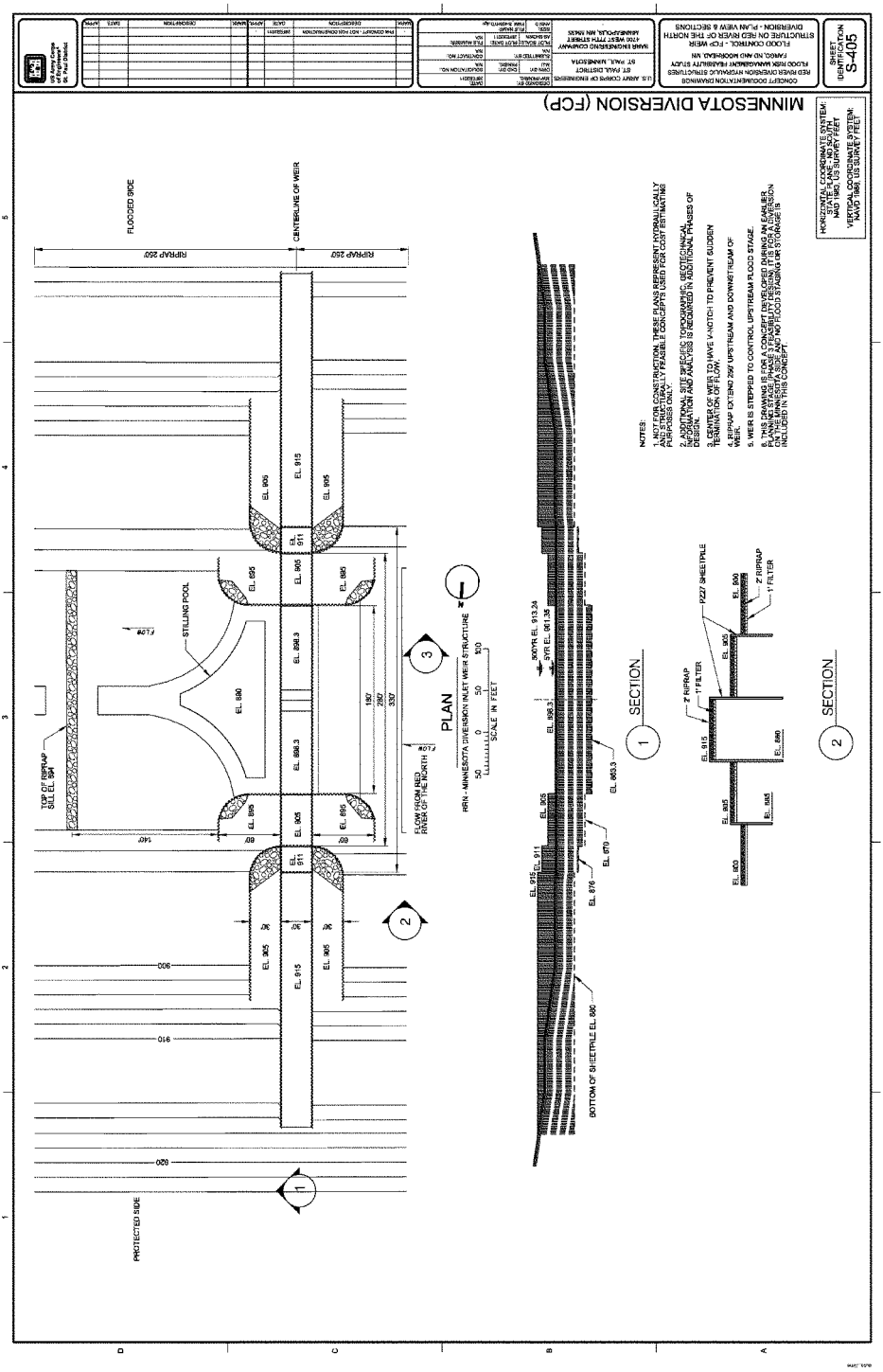


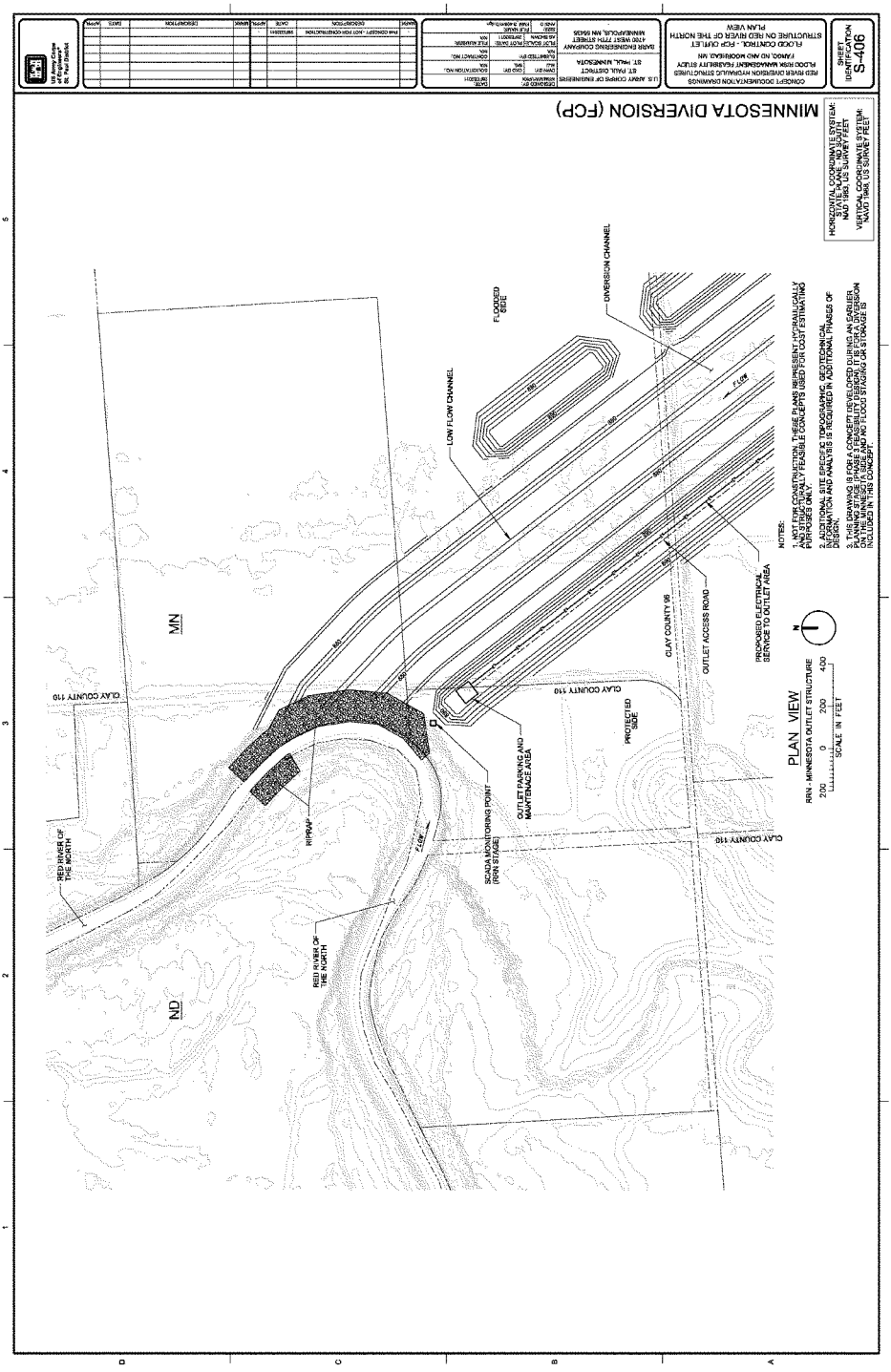
PANEL	STEM W (FT)	STEM BATTER X (FT)	FTG. WIDTH B (FT)	FTG. TOE T (FT)	CONCRETE VOLUMES FTG. CY	STEM CY	DECK CY	REINF. LBS	PILES NO.	NO. ROWS	LENGTH FT	TOTAL FT
A	10	8.92	32.9	8	302	1462	60	135,627	84	7	47	3,948
B	10	5.75	29.8	7	141	694	30	64,660	37	6	54	1,999
C	10	0	28.0	8	123	446	30	56,563	33	6	59	1,914
D	10	0	28.0	8	123	399	30	56,519	30	5	59	1,800
E	10	0	24.5	7	114	351	30	51,515	27	5	57	1,599
F	9	0	19.0	5	90	273	29	43,471	24	4	71	1,764
G	8	0	18.0	4	75	205	29	36,302	20	3	75	1,560
H	8	0	18.0	4	78	167	29	33,871	16	3	80	1,280
I	8	0	14.0	3	66	128	29	27,812	13	3	64	1,092
J	8	0	10.0	1	47	80	29	22,029	11	2	88	968
K	8	0	10.0	1	47	82	29	20,134	9	2	93	744
			WEST WINGWALL		1,206	4,177	354	490,419	303			15,647
			EAST WINGWALL		1,205	4,177	354	490,419	303			15,647
			TOTAL		2,410	8,354	708	1,100,836	606			37,894

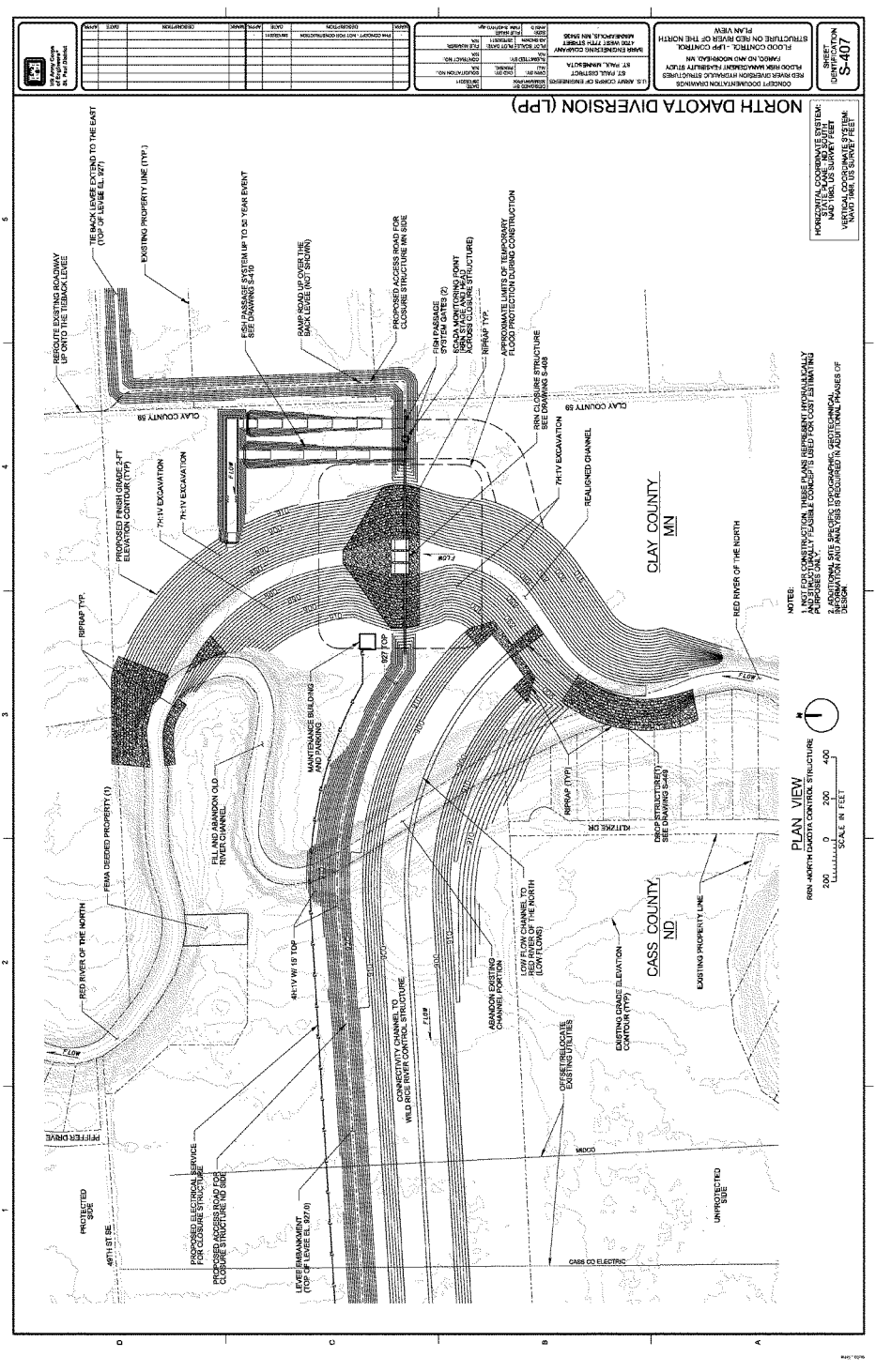
LOADING CONDITION	CLASSIFICATION	TONS/PILE
UNDRAINED USUAL		67.4
UNDRAINED UNUSUAL		89.8
UNDRAINED EXTREME		117.1
DRAINED USUAL		35.4

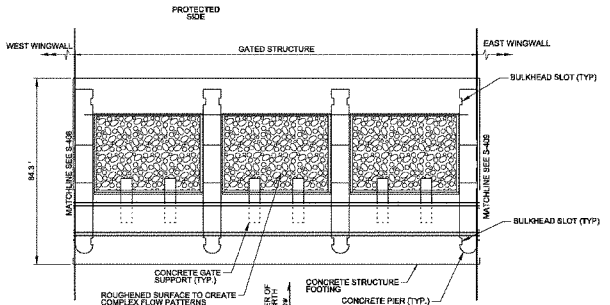
NOTE: THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX B


MINNESOTA DIVERSION (FCP)
PLAN VIEW FISH PASSAGE FCP
ELEVATION
SECTION
STATION
ELEVATION
TOP OF LEVEE
TOP OF WINGWALL
RRR WINGWALL
TIE BACK LEVEE
EXISTING GRADE
PROPOSED GRADE BEHIND WINGWALL
PROPOSED GRADE IN FRONT OF WINGWALL
BOTTOM OF FCP/L 901.2
BOTTOM OF FCP/L 897.7
GATE 1
GATE 2
GATE 3
ROLLER GATE OPERATORS (TYP)
10' WIDE BY 6' HIGH ROLLER GATE (TYP OF 3)
FLOW
FISH PASSAGE OUTLET
CHANNEL CONNECTING FISH PASSAGE GATES TO CHANNEL UPSTREAM OF CONTROL STRUCTURE
CONCRETE STRUCTURE
GRAVEL BEDDING
NOTE:
TYPICAL FISH PASSAGE GATE ARRANGEMENT ONE OF THE THREE FISH PASSAGES
SCALE IN FEET
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL EVALUATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
3. THIS DRAWING IS FOR A CONCEPT DEVELOPED DURING AN EARLIER PHASE OF THE MINNESOTA AND RED RIVER FLOOD DIVERSION PROJECT AND INCLUDED IN THIS CONCEPT.
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD CONTROL - FCP FISH PASSAGE
FARGO, ND AND MOORHEAD, MN
MAIN CONTRACTOR
MINNESOTA DIVERSION AUTHORITY
HORIZONTAL COORDINATE SYSTEM: NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
SHEET IDENTIFICATION
S-404


MINNESOTA DIVERSION (FCP)
PLAN
SECTION
SECTION
SECTION
FLOODED SIDE
CENTERLINE OF WEIR
PROTECTED SIDE
RIPRAP 250'
RIPRAP 250'
STILLING POOL
TOP OF RIPRAP SILL EL. 894
EL. 898.3
EL. 915
EL. 905
EL. 911
EL. 895
EL. 880
EL. 870
EL. 913.24
BOTTOM OF SHEETPILE EL. 880
FLOW FROM RED RIVER OF THE NORTH
ARN - MINNESOTA DIVERSION INLET WEIR STRUCTURE
SCALE IN FEET
PZ27 SHEETPILE
2' RIPRAP
1' FILTER
EL. 920
EL. 905
EL. 880
EL. 895
EL. 890
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, INFILTRATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
3. TOP ELEVATION 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 TO DESCRIBE INTENT AS SHOWN IN THE ENVIRONMENTAL IMPACT STATEMENT (EIS). DESIGN IS NOT TO SCALE AND NO FLOOD STAGING OR STORAGE IS INCLUDED IN THIS CONCEPT.
CONCEPT PLAN DRAWING
RED RIVER DIVERSION
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN
STRUCTURE ON RED RIVER OF THE NORTH
FLOOD CONTROL - FCP WEIR
DIVERSION - PLAN VIEW & SECTIONS
SHEET DESCRIPTION
S-405
HORIZONTAL COORDINATE SYSTEM: NORTH DAKOTA STATE PLANE NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
U.S. ARMY CORPS OF ENGINEERS
ST PAUL DISTRICT
ST. PAUL, MINNESOTA
WARE SURVEYING COMPANY
4700 WEST 77TH STREET
EDINA, MINNESOTA
ARNEP-POLK, MN 56635
US Army Corps of Engineers St. Paul District


CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION PERMANENT EASEMENT STUDY
FARGO, ND AND MOORHEAD, MN
FLOOD CONTROL - PCP CUTLET
STRUCTURE ON RED RIVER OF THE NORTH
PLAN VIEW
SHEET IDENTIFICATION
S-406
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55436
MINNESOTA DIVERSION (FCP)
PLAN VIEW
RRN - MINNESOTA OUTLET STRUCTURE
SCALE IN FEET
200 0 200 400
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING AND STAGE 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 CHANNEL WITH A 20,000 CFS FLOOD STRIDE AND 91 STORAGE IS INCLUDED IN THIS CONCEPT.
HORIZONTAL COORDINATE SYSTEM: NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
MN
ND
CLAY COUNTY 110
RED RIVER OF THE NORTH
RIPRAP
SCADA MONITORING POINT (RRN STAGE)
OUTLET PARKING AND MAINTENANCE AREA
PROTECTED SIDE
FLOODED SIDE
LOW FLOW CHANNEL
DIVERSION CHANNEL
CLAY COUNTY 99
OUTLET ACCESS ROAD
PROPOSED ELECTRICAL SERVICE TO OUTLET AREA


NORTH DAKOTA DIVERSION (LPP)
PLAN VIEW
PLAN VIEW
RRN-NORTH DAKOTA CONTROL STRUCTURE
SCALE IN FEET
CLAY COUNTY
MN
CASS COUNTY
ND
RED RIVER OF THE NORTH
REALIGNED CHANNEL
REMOVE EXISTING ROADWAY UP ONTO THE TIEBACK LEVEE
TIE BACK LEVEE EXTEND TO THE EAST (TOP OF LEVEE EL. 927)
EXISTING PROPERTY LINE (TYP.)
FISH PASSAGE SYSTEM UP TO 50 YEAR EVENT SEE DRAWING S-410
RAMP ROAD UP OVER THE TIEBACK LEVEE (NOT SHOWN)
PROPOSED ACCESS ROAD FOR CLOSURE STRUCTURE MN SIDE
FISH PASSAGE SYSTEM GATES (2)
SIGNAGE MOUNTING POINT (TYP.) (ATTACHED TO ACROSS CLOSURE STRUCTURE)
RIPRAP TYP.
APPROXIMATE LIMITS OF TEMPORARY FLOOD PROTECTION DURING CONSTRUCTION
RRN CLOSURE STRUCTURE SEE DRAWING S-400
7H:1V EXCAVATION
PROPOSED FINISH GRADE 2-FT ELEVATION CONTOUR (TYP.)
FEMA DEERED PROPERTY (1)
FILL AND ABANDON OLD RIVER CHANNEL
MAINTENANCE BUILDING AND PARKING
927 TOP
41H:1V W/ 15' TOP
CONNECTIVITY CHANNEL TO WILD RICE RIVER CONTROL STRUCTURE
ABANDON EXISTING CHANNEL PORTION
LOW FLOW CHANNEL TO RED RIVER OF THE NORTH (LOW FLOWS)
DROP STRUCTURE(1) SEE DRAWING S-440
KUTZKE DR
PROTECTED SIDE
45TH ST SE
PFEIFER DRIVE
PROPOSED ELECTRICAL SERVICE FOR CLOSURE STRUCTURE
PROPOSED ACCESS ROAD FOR CLOSURE STRUCTURE ND SIDE
LEVEE EMBANKMENT (TOP OF LEVEE EL. 927.0)
CASS CO ELECTRIC
UNPROTECTED SIDE
EXISTING GRADE ELEVATION CONTOUR (TYP.)
EXISTING PROPERTY LINE
OVERHEAD UTILITIES
UNDER
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND GEOTECHNICALLY FEASIBLE CONCEPTS AND ARE USED FOR PURPOSES ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
HORIZONTAL COORDINATE SYSTEM: NAD 1983 (2011) ADJUSTMENT UTM ZONE 14N US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988 US SURVEY FEET
SHEET DESCRIPTION
S-407
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN
FLOOD CONTROL - LPP
STRUCTURE ON RED RIVER OF THE NORTH
PLAN VIEW
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
DESIGNED BY
DRAWN BY
CHECKED BY
QUALITY CHK
APPROVED BY
DATE
SUBMITTED
CONTRACT NO.
SOLICITATION NO.
FILE NUMBER
PLOT SCALE
PLOT DATE
AMES ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
DESIGN
DRAWN
CHECKED
APPROVED
DATE
DESCRIPTION
NAME
APVD.
DATE
DESCRIPTION
NAME
APVD.
DATE
DESCRIPTION
NAME
APVD.

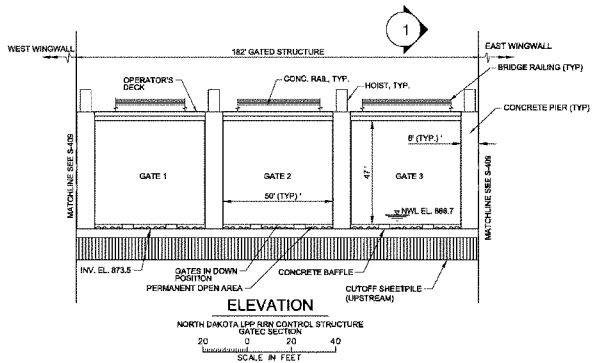


PLAN

NORTH DAKOTA LPP RRN CONTROL STRUCTURE GATED SECTION

20 0 20 40

SCALE IN FEET



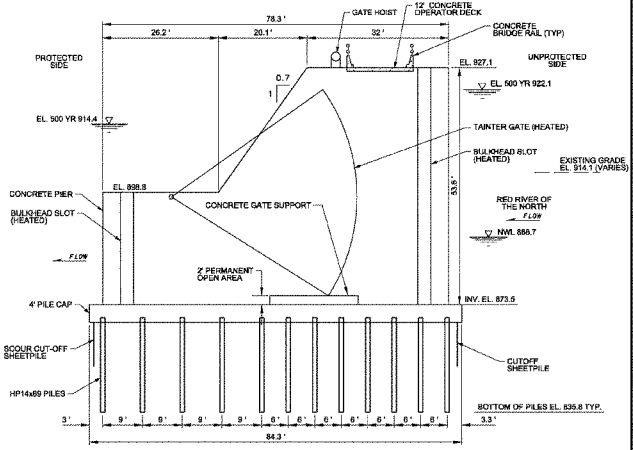
ELEVATION

NORTH DAKOTA LPP RRN CONTROL STRUCTURE GATEC SECTION

20 0 20 40

SCALE IN FEET

NOTE: PILES NOT SHOWN FOR CLARITY.



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	99.8
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.

QUANTITIES (GATED STRUCTURE)

CONCRETE VOLUMES			STEEL	PILES			
FTG. CY	PIER CY	DECK CY	REINF. LBS	NO.	NO. ROWS	LENGTH FT	TOTAL FT
2,273	3,759	128	487,450	316	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

NORTH DAKOTA DIVERSION (LPP)

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD CONTROL, LPP CONTROL STRUCTURE
FARGO, ND AND MOORHEAD, MN

FLOOD CONTROL, LPP CONTROL STRUCTURE PLAN, SECTION AND ELEVATION
GATED PLAN, SECTION AND ELEVATION

US Army Corps of Engineers
St. Paul District

NO.	DESCRIPTION	DATE	APPD
1	FOR CONSTRUCTION AND FOR CONSTRUCTION	08/16/2011	JMP

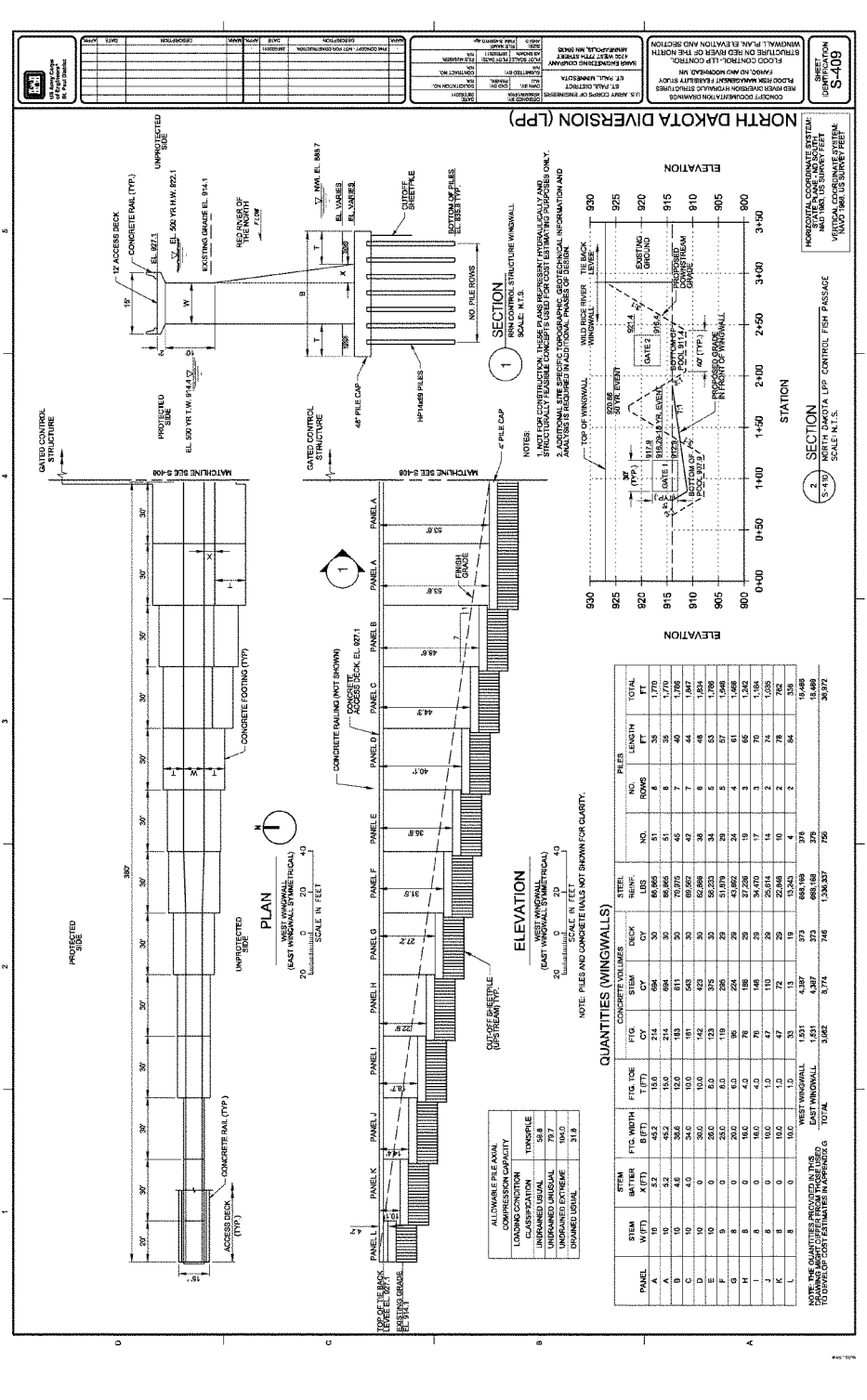
DESIGNED BY: DATE ASSIGNED:
DRAWN BY: DESIGN MANAGER: EDUCATION ADV:
CHECKED BY: CONTRACT NO.:
SUBMITTED BY: PLOT SCALE: PLOT DATE: 08/16/2011
PLOT NAME: S-408.DGN
FILE NAME: S-408.DGN

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

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

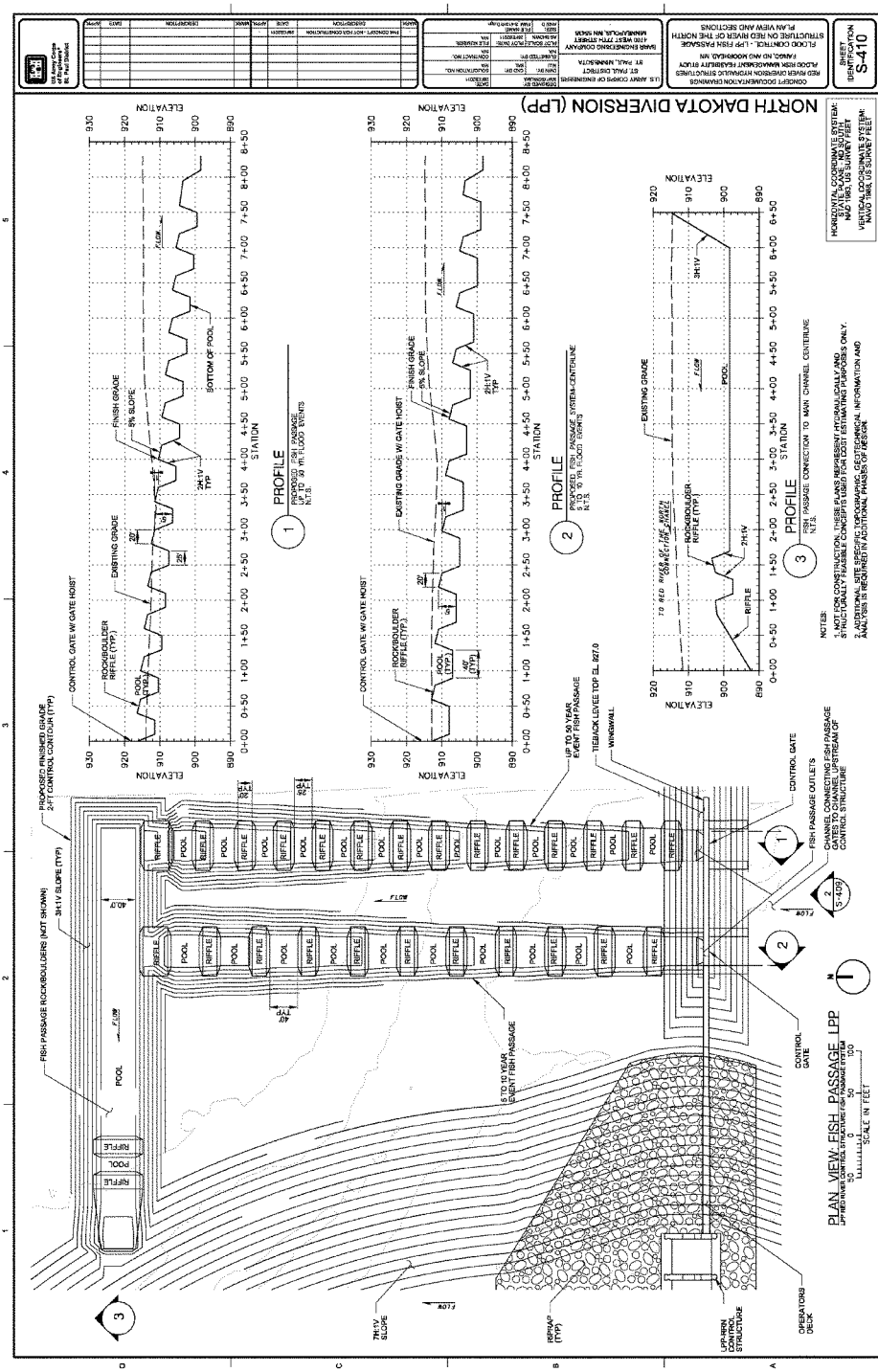
SHEET IDENTIFICATION
S-408

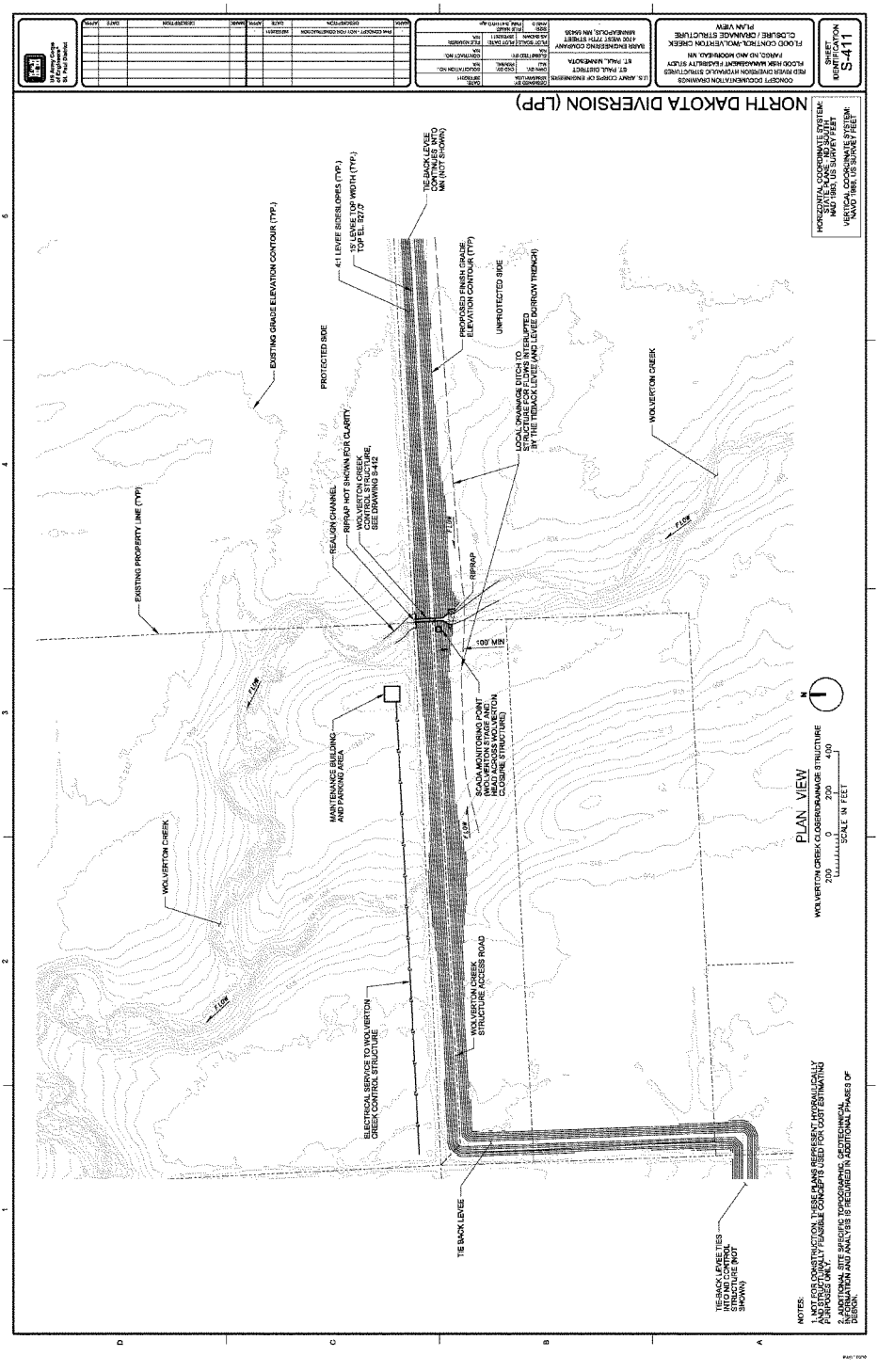
auto_time

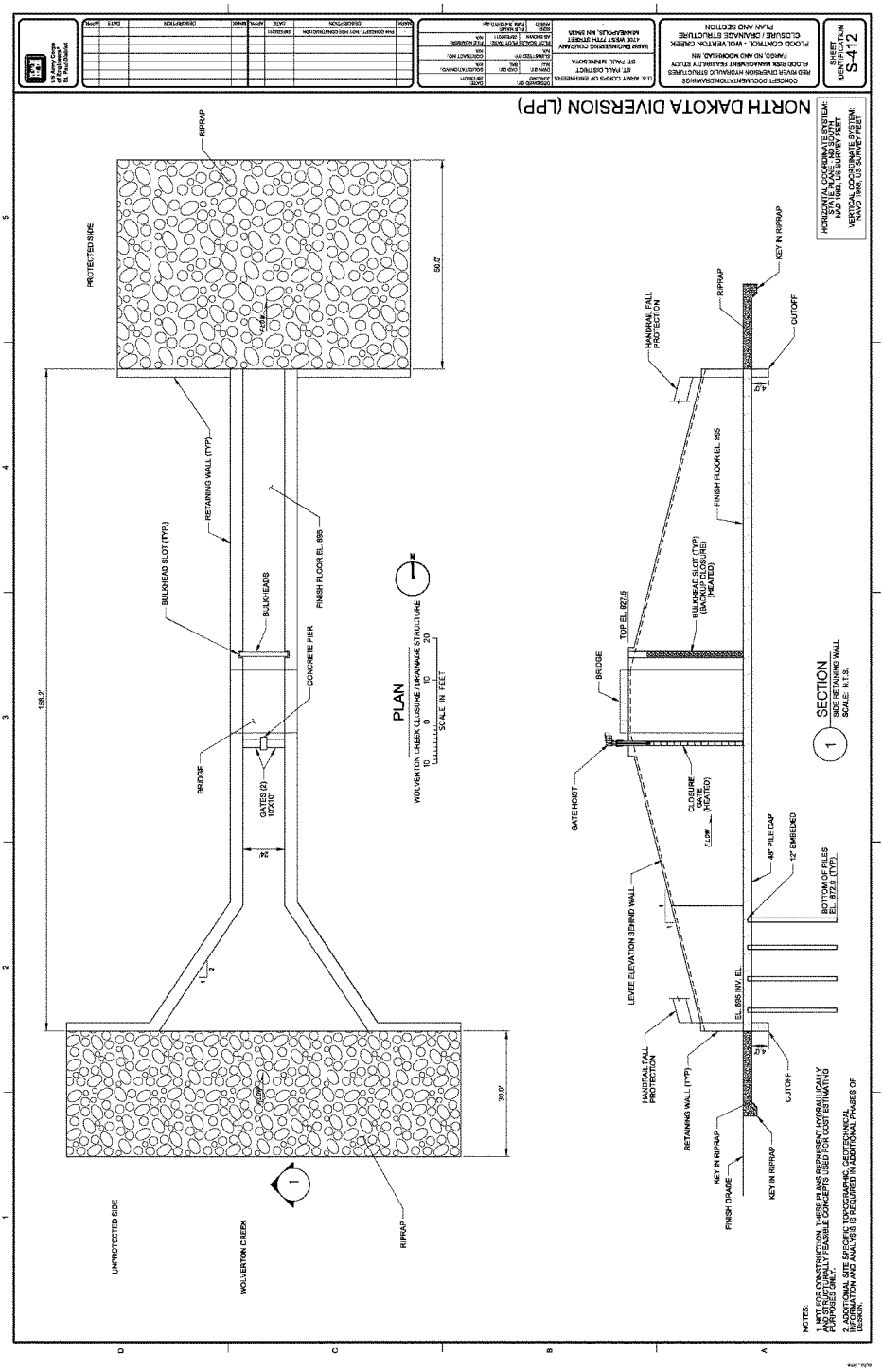


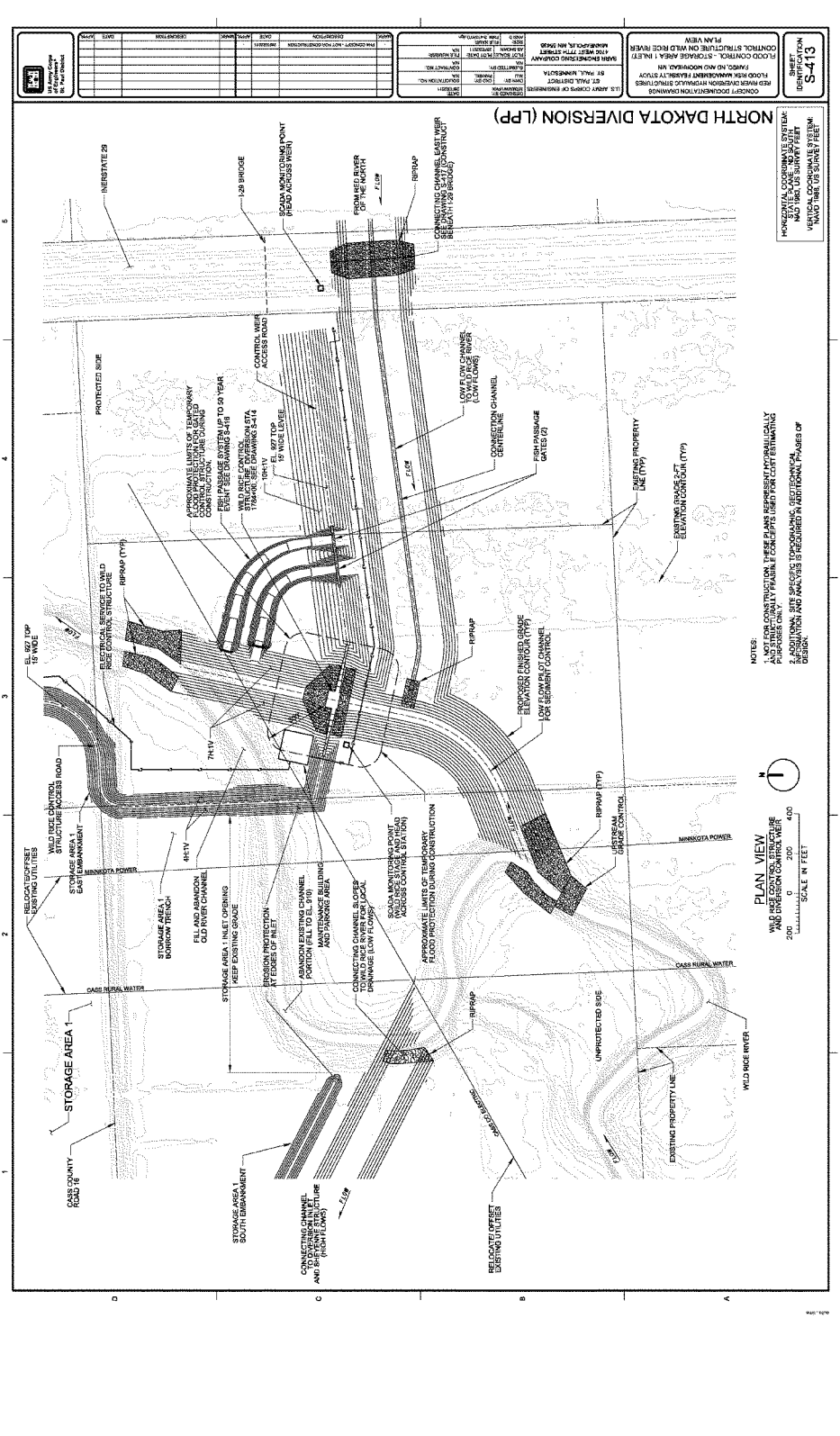
PANEL	STEM W (FT)	STEM BATTER (V:FT)	FTG. WIDTH B (FT)	FTG. TOE T (FT)	FTG. HOE T (FT)	CONCRETE VOLUMES FTG CY	STEM CY	DECK CY	STEEL WEIGHT (LBS)	NO.	NO. ROWS	PILES LENGTH	TOTAL FT
A	10	3:2	45.2	15.0	4.0	214	694	30	86,985	51	8	35	1,770
B	10	4:2	45.2	15.0	4.0	214	694	30	86,985	51	8	35	1,770
C	10	4.6	38.6	12.0	1.0	183	611	30	76,975	45	7	40	1,766
D	10	4.0	34.0	10.0	1.0	161	543	30	69,562	42	7	44	1,847
E	10	0	30.0	10.0	1.0	142	423	30	62,868	38	6	48	1,824
F	10	0	26.0	8.0	1.0	125	375	30	56,233	34	5	53	1,766
G	9	0	23.0	8.0	1.0	110	326	29	49,870	30	5	57	1,648
H	8	0	19.0	6.0	1.0	95	224	29	41,992	24	4	61	1,465
H	8	0	16.0	4.0	1.0	76	186	29	37,258	19	3	65	1,242
I	8	0	16.0	4.0	1.0	76	146	29	34,470	17	3	70	1,164
J	8	0	16.0	1.0	1.0	47	110	29	25,814	14	2	74	1,035
K	8	0	10.0	1.0	1.0	47	72	29	22,848	10	2	78	782
L	8	0	10.0	1.0	1.0	33	13	19	13,243	4	2	84	336
			WEST WINGWALL			1,931	4,397	373	698,168	376			19,489
			EAST WINGWALL			1,131	4,377	373	640,169	379			19,383
NOTE: THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX G			TOTAL			3,062	8,774	746	1,338,337	755			38,872

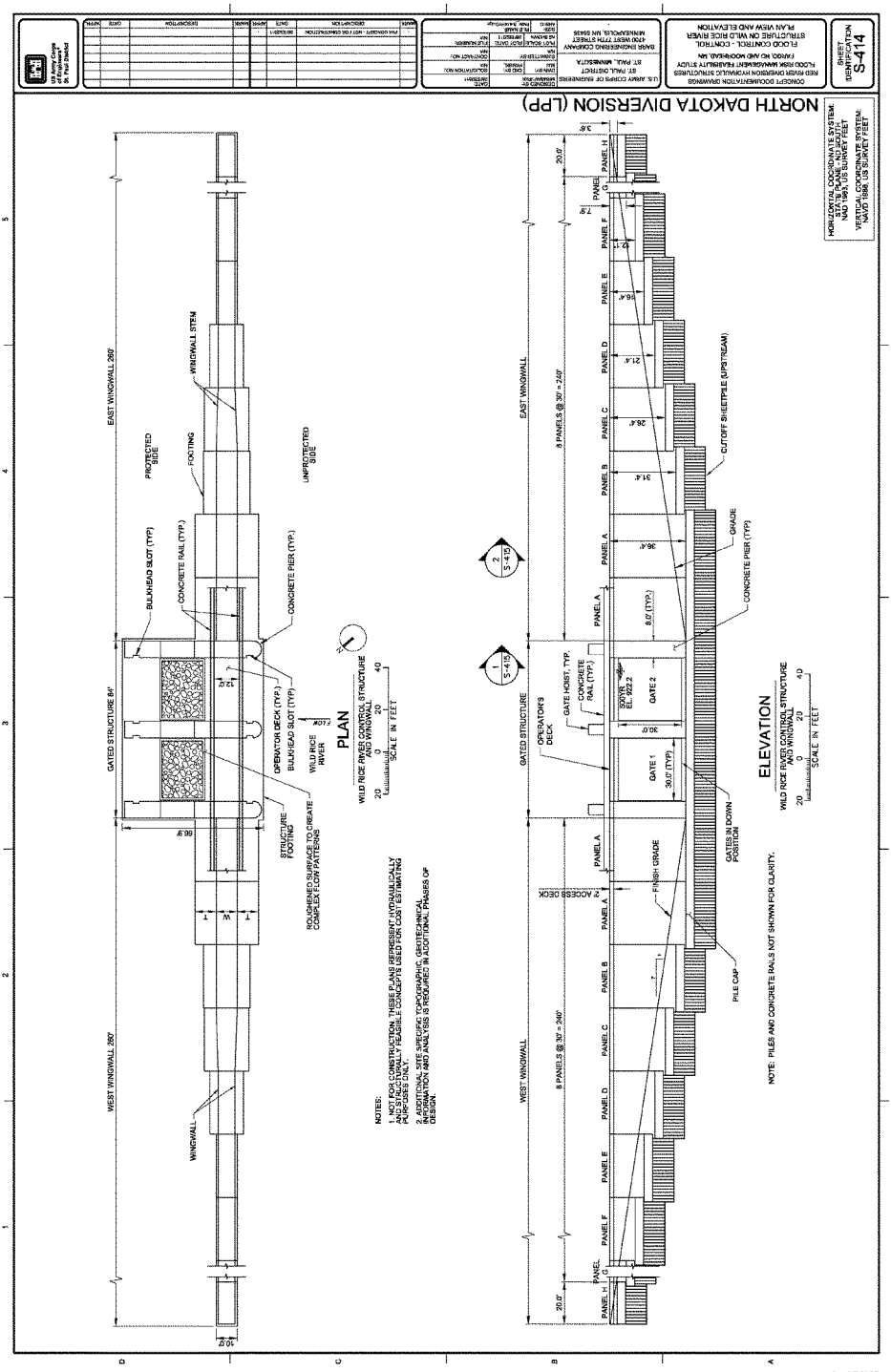
ALLOWABLE PILE AXIAL COMPRESSION CAPACITY	
LOADING CONDITION	TENSRILE
CLASSIFICATION	
UNDRAINED USUAL	39.3
UNDRAINED UNUSUAL	79.7
UNDRAINED EXTREME	104.0
DRAINED USUAL	31.8

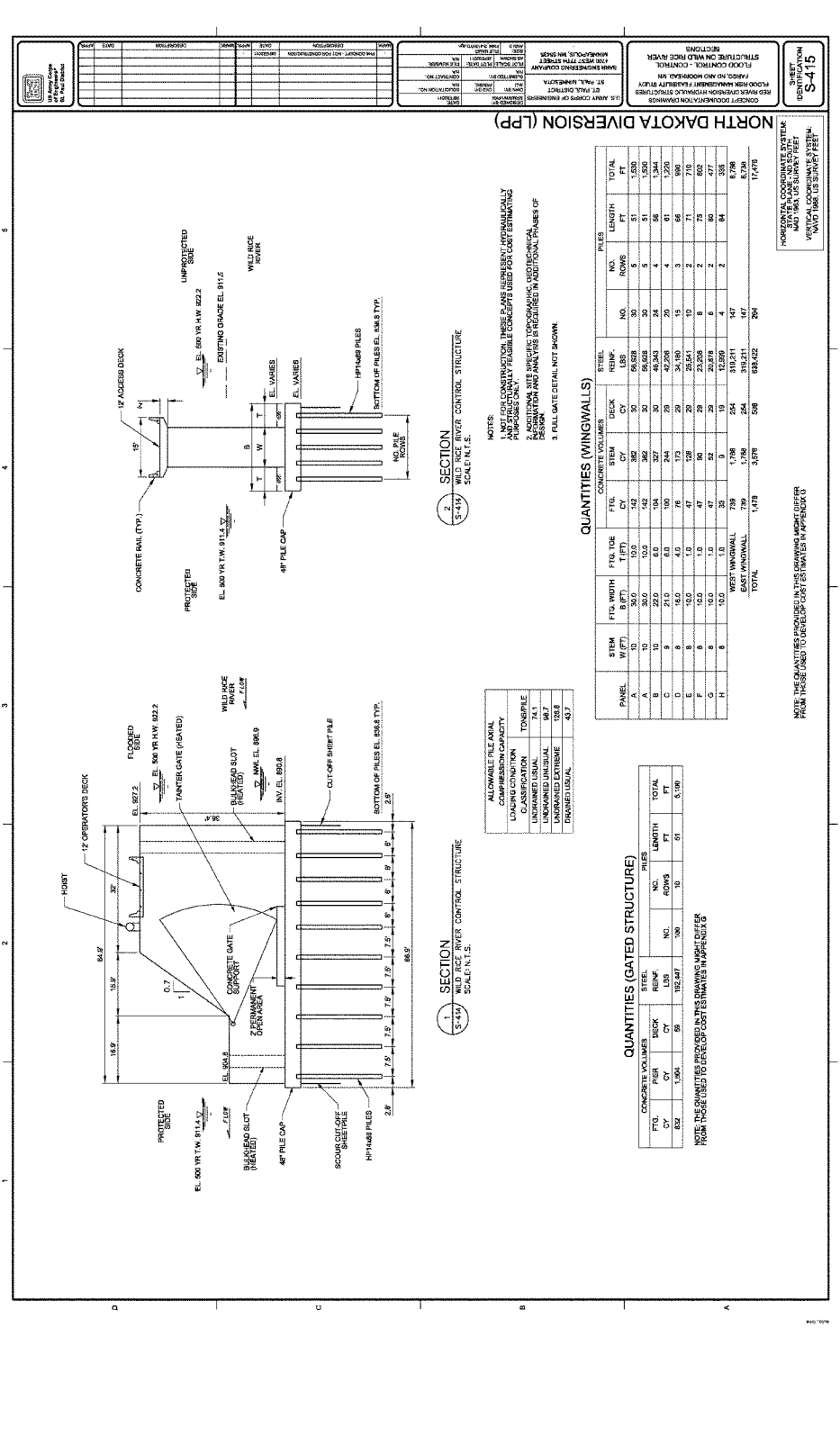

NORTH DAKOTA DIVERSION (LPP)
PLAN VIEW FISH PASSAGE LPP
PROFILE
PROFILE
PROFILE
FISH PASSAGE ROCKBOULDERS (NOT SHOWN)
3H:1V SLOPE (TYP)
PROPOSED FINISHED GRADE
2FT CONTROL CONTOUR (TYP)
CONTROL GATE W/ GATE HOIST
ROCKBOULDER RIFFLE (TYP.)
EXISTING GRADE
FINISH GRADE
5% SLOPE
BOTTOM OF POOL
2H:1V TYP
STATION
ELEVATION
FLOW
POOL
RIFFLE
UP TO 50 YEAR EVENT FISH PASSAGE
TIEBACK LEVEE TOP EL. 927.0
WINGWALL
5 TO 10 YEAR EVENT FISH PASSAGE
CONTROL GATE
FISH PASSAGE OUTLETS
CHANNEL CONNECTING FISH PASSAGE GATES TO CHANNEL UPSTREAM OF CONTROL STRUCTURE
RIPRAP (TYP)
UPSTREAM CONTROL STRUCTURE
OPERATORS DESK
3H:1V SLOPE
PROPOSED FISH PASSAGE SYSTEM-CENTERLINE
5 TO 10 YR FLOOD EVENTS
N.T.S.
PROPOSED FISH PASSAGE
UP TO 50 YR FLOOD EVENTS
N.T.S.
FISH PASSAGE CONNECTION TO MAIN CHANNEL CENTERLINE
N.T.S.
TO RED RIVER OF THE NORTH CONNECTION
EXISTING GRADE
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 FOR ADDITIONAL PHASES OF DESIGN.
HORIZONTAL COORDINATE SYSTEM: NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD CONTROL - LPP FISH PASSAGE
FARGO, ND AND MOORHEAD, MN
STRUCTURES ON RED RIVER OF THE NORTH
PLAN VIEW AND SECTIONS
SHEET IDENTIFICATION
S-410
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
180 EAST FIFTH STREET
ST. PAUL, MINNESOTA 55101-1678
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
DESIGNED BY
DRAWN BY
CHECKED BY
DATE
SUBMITTED BY
DATE
APPROVED BY
DATE
DESCRIPTION
DATE
APPR.
CHK'D
DESCRIPTION
DATE
APPR.
DRAWN
FHM CONCEPT - NOT FOR CONSTRUCTION
08/12/2011
US Army Corps of Engineers St. Paul District

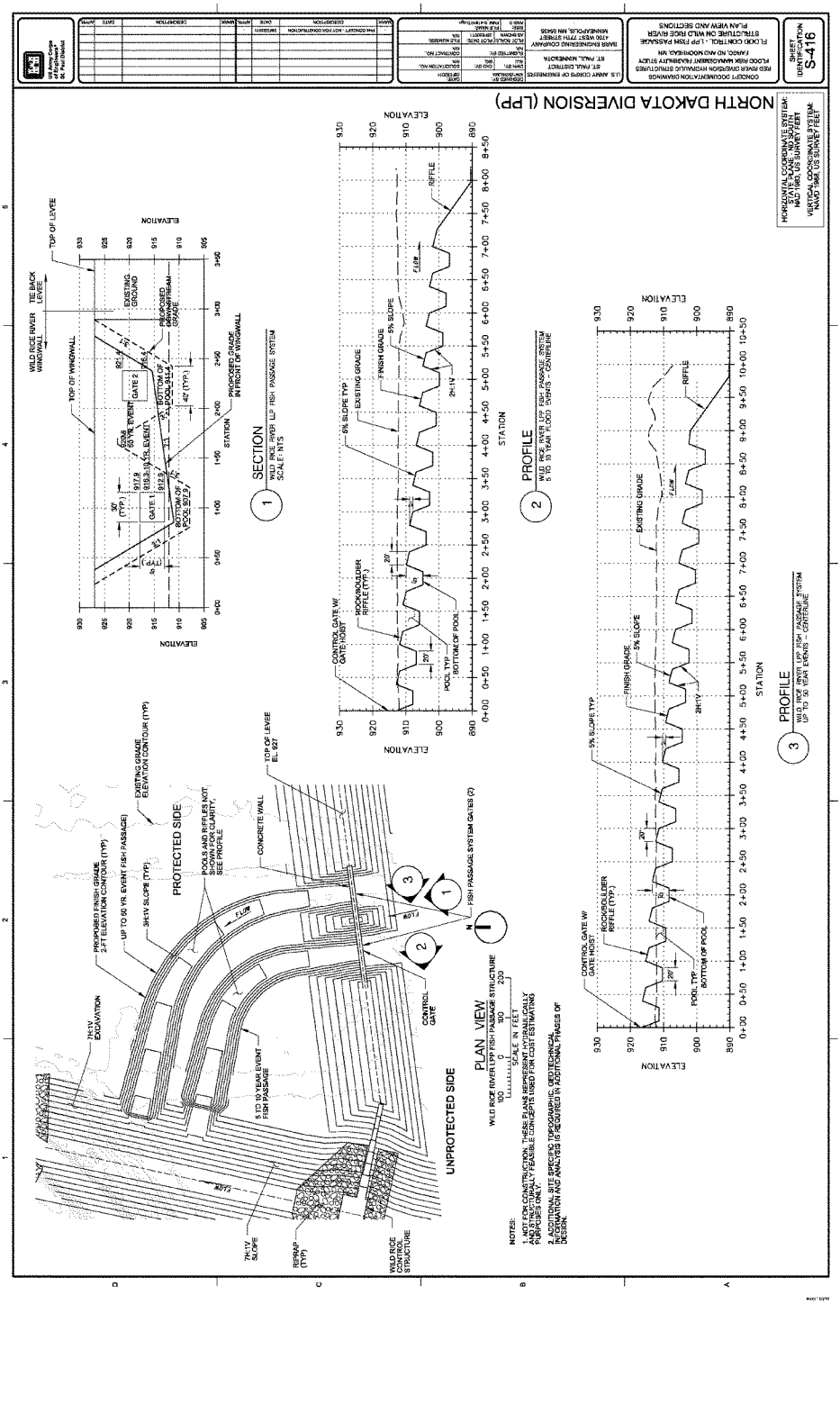

CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN
FLOOD CONTROL, MOUTH OF WOLVERTON CREEK
CLOSURE / DRAINAGE STRUCTURE
PLAN VIEW
SHEET
EXISTING CONDITION
S-411
U.S. ARMY CORPS OF ENGINEERS
ST PAUL DISTRICT
ST. PAUL, MINNESOTA
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
HORIZONTAL COORDINATE SYSTEM:
NAD 1983 MN SOUTH PROJECT
VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET
NORTH DAKOTA DIVERSION (LPP)
EXISTING GRADE ELEVATION CONTOUR (TYP.)
EXISTING PROPERTY LINE (TYP)
PROTECTED SIDE
4:1 LEVEE SIDESLOPES (TYP.)
15' LEVEE TOP WIDTH (TYP.)
TOP EL 1070
TIE-BACK LEVEE CONTINUES INTO NM (NOT SHOWN)
PROPOSED FINISH GRADE ELEVATION CONTOUR (TYP)
UNPROTECTED SIDE
LOCAL DRAINAGE DITCH TO STRUCTURE FOR FLOWS INTERUPTED BY THE TIEBACK LEVEE (AND LEVEE BORROW TRENCH)
WOLVERTON CREEK
REALIGN CHANNEL
RIPRAP NOT SHOWN FOR CLARITY
WOLVERTON CREEK CONTROL STRUCTURE, SEE DRAWING S-412
RIPRAP
100' MIN
FLOW
MAINTENANCE BUILDING AND PARKING AREA
SCADA MONITORING POINT (WOLVERTON STAGE AND VELOCITY FOR GATE ACTION CLOSURE STRUCTURES)
WOLVERTON CREEK
ELECTRICAL SERVICE TO WOLVERTON CREEK CONTROL STRUCTURE
WOLVERTON CREEK STRUCTURE ACCESS ROAD
TIE BACK LEVEE
TIE-BACK LEVEE TIES INTO NO CONTROL STRUCTURE (NOT SHOWN)
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY FEASIBLE CONCEPTS ONLY. FEASIBLE CHANGES TO BE DONE DURING PRELIMINARY DESIGN FOR PURPOSES ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL, ENVIRONMENTAL AND WETLAND INVESTIGATIONS WILL BE REQUIRED IN ADDITIONAL PHASES OF DESIGN.
PLAN VIEW
WOLVERTON CREEK CLOSER/DRAINAGE STRUCTURE
200 0 200 400
SCALE IN FEET
N

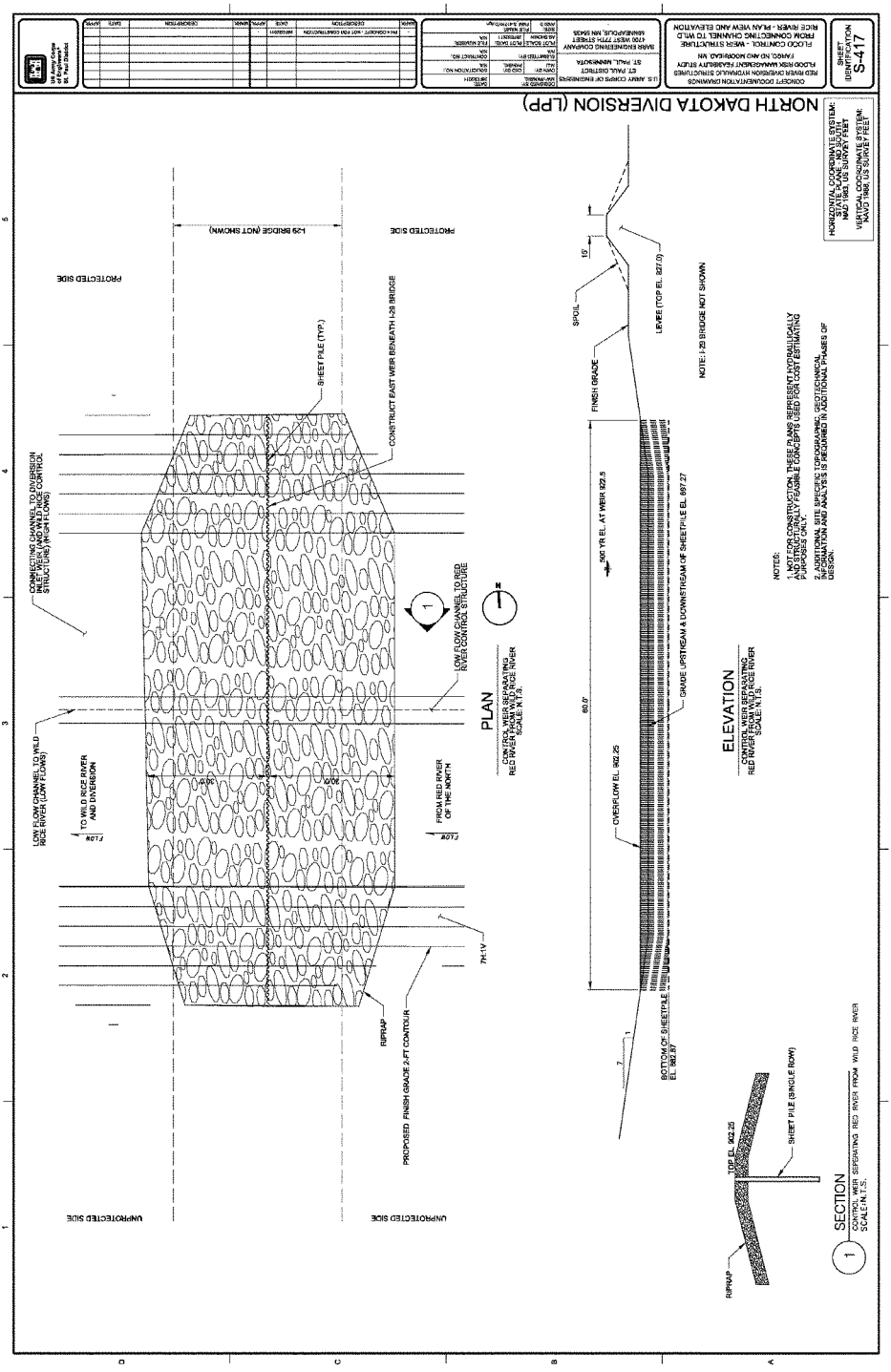

NORTH DAKOTA DIVERSION (LPP)
PLAN
WOLVERTON CREEK CLOSURE / DRAINAGE STRUCTURE
SCALE: IN FEET
SECTION
SIDE RETAINING WALL
SCALE: N.T.S.
PROTECTED SIDE
UNPROTECTED SIDE
WOLVERTON CREEK
RIPRAP
RETAINING WALL (TYP.)
BULKHEAD SLOT (TYP.)
BULKHEADS
FINISH FLOOR EL. 895
CONCRETE PIER
BRIDGE
GATES (2) 12'x12'
158.2'
24'
59.0'
30.0'
BRIDGE
TOP EL. 927.8
GATE HOIST
CLOSURE GATES (HEATED)
BULKHEAD SLOT (TYP) (BACKUP CLOSURE) (HEATED)
FINISH FLOOR EL. 895
HANDRAIL FALL PROTECTION
RIPRAP
KEY IN RIPRAP
CUTOFF
LEVEE ELEVATION BEHIND WALL
4@ PILE CAP
12" EMBEDDED
BOTTOM OF PILES EL. 872.0 (TYP.)
EL. 895 RIV. EL.
FINISH GRADE
NOTES:
1. FOR CONSTRUCTION, THERE IN ARE SUFFICIENTLY HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.
2. ADDITIONAL AND EXPENSIVE EXCAVATION, GEOTECHNICAL INVESTIGATION, AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
HORIZONTAL COORDINATE SYSTEM: NORTH DAKOTA STATE PLANE NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND WOODRUSAG, MN
FLOOD CONTROL - WOLVERTON CREEK
CLOSURE / DRAINAGE STRUCTURE
PLAN AND SECTION
SHEET IDENTIFICATION
S-412
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
MAIN ENGINEERING COMPANY
4100 WEST 77TH STREET
MINNEAPOLIS, MN 55435
DESIGNED BY
DRAWN BY
CHECKED BY
DATE
SUBMITTED
CONTRACT NO.
SOLICITATION NO.
DWG. NO.
SHEET NO.
SCALE
AS SHOWN
DATE SCALE ON PLOT DWG.
FILE NUMBER
Plotted: 24-Jul-2013 09:49
DESCRIPTION
DATE
APPVD
BY
DESCRIPTION
DATE
APPVD
DATE
Has CONCEPT / NOT FOR CONSTRUCTION
DESCRIPTION
DATE
APPVD

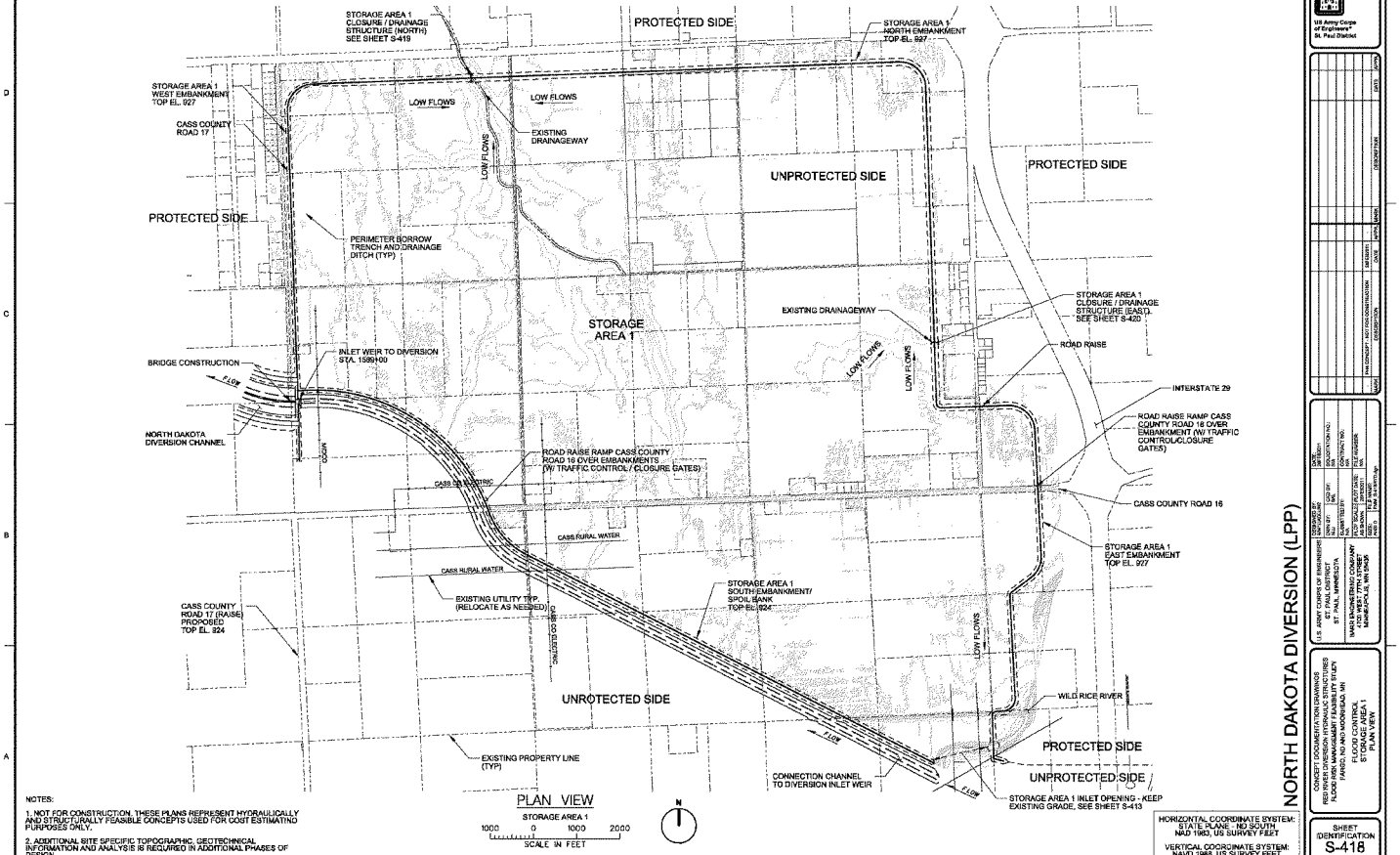

NORTH DAKOTA DIVERSION (LPP)
PLAN VIEW
WILD RICE CONTROL STRUCTURE AND DIVERSION CONTROL WEIR
SCALE IN FEET
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT PRELIMINARY DESIGN AND ARE AVAILABLE FOR USER-DEFINED COST ESTIMATING PURPOSES ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL AND HYDRAULIC INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
STORAGE AREA 1
CASS COUNTY ROAD 16
WILD RICE CONTROL STRUCTURE ACCESS ROAD
EL. 927 TOP 15 WIDE
ELECTRICAL SERVICE TO WILD RICE CONTROL STRUCTURE
RIPRAP (TYP)
STORAGE AREA 1 EMBANKMENT
RELOCATE/OFFSET EXISTING UTILITIES
CASS RURAL WATER
MINNKOTA POWER
STORAGE AREA 1 INLET OPENING
KEEP EXISTING GRADE
EROSION PROTECTION AT EDGES OF INLET
FILL AND ABANDON OLD RIVER CHANNEL
ABANDON EXISTING CHANNEL PORTION AS NEEDED FOR MAINTENANCE BUILDING AND PARKING AREA
CONNECTING CHANNEL SLOPES TO WILD RICE RIVER FOR LOCAL DRAINAGE (LOW FLOWS)
SCADA MONITORING POINT (WILD RICE RIVER STAGE AND DISCHARGE ACROSS CONTROL STATION)
APPROXIMATE LIMITS OF TEMPORARY CONSTRUCTION EASEMENT
RIPRAP
PROTECTED SIDE
APPROXIMATE LIMITS OF TEMPORARY CONSTRUCTION EASEMENT FOR CONTROL STRUCTURE DURING CONSTRUCTION
WILD RICE CONTROL STRUCTURE (WEIR) EL. 927.0 TOP 150' WIDE (SEE DRAWING S-414)
EL. 927 TOP 15 WIDE LEVEE
10H:1V
7H:1V
4H:1V
CONTROL WEIR ACCESS ROAD
INTERSTATE 29
I-29 BRIDGE
SCADA MONITORING POINT (HEAD ACROSS WEIR)
FROM RED RIVER OF THE NORTH
FLOW
RIPRAP
CONNECTING CHANNEL EAST WEIR (LOW FLOWS THROUGH BENEATH I-29 BRIDGE)
LOW FLOW CHANNEL (LOW FLOWS)
CONNECTION CHANNEL CENTERLINE
FISH PASSAGE GATES (2)
EXISTING PROPERTY LINE (TYP)
EXISTING GRADE 4FT ELEVATION CONTOUR (TYP)
PROPOSED FINISHED GRADE ELEVATION CONTOUR (TYP)
LOW FLOW PILOT CHANNEL FOR SEDIMENT CONTROL
RIPRAP (TYP)
DIRECTION OF FLOW
UPSTREAM EDGE OF CONTROL
MINNKOTA POWER
CASS RURAL WATER
UNPROTECTED SIDE
WILD RICE RIVER
EXISTING PROPERTY LINE
RELOCATE/ OFFSET EXISTING UTILITIES
STORAGE AREA 1 SOUTH EMBANKMENT
CONNECTING CHANNEL SLOPES (LOW FLOWS) AND 8H:1V (HIGH FLOWS)
HORIZONTAL COORDINATE SYSTEM: NAD 1983 (NSRS 2007) US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
CONCEPT PLAN
FLOOD CONTROL - STORAGE AREA 1 INLET
CONTROL STRUCTURE ON WILD RICE RIVER
PLAN VIEW
SHEET DESCRIPTION
S-413


NORTH DAKOTA DIVERSION (LPP)
PLAN
ELEVATION
WILD RICE RIVER CONTROL STRUCTURE AND WINGWALL
SCALE IN FEET
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INVESTIGATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN
NOTE: PILES AND CONCRETE RAILS NOT SHOWN FOR CLARITY.
EAST WINGWALL 280'
WEST WINGWALL 280'
GATED STRUCTURE 84'
WINGWALL
PROTECTED SIDE
UNPROTECTED SIDE
FOOTING
WINGWALL STEM
CONCRETE RAIL (TYP.)
BULKHEAD SLOT (TYP)
CONCRETE PIER (TYP.)
OPERATION DECK (TYP.)
STRUCTURE FOOTING
ROUGHENED SURFACE TO CREATE COMPOSITE CONNECTIONS
WILD RICE RIVER
66.9'
10.0'
8 PANELS @ 30' = 240'
PANEL A
PANEL B
PANEL C
PANEL D
PANEL E
PANEL F
PANEL G
PANEL H
20.0'
7.9'
3.9'
GATED STRUCTURE
OPERATOR'S DECK
GATE HOIST, TYP.
CONCRETE RAIL (TYP.)
GATE 1
GATE 2
30.0' (TYP.)
8.0' (TYP.)
FINISH GRADE
GRADE
CUTOFF SHEETPILE (UPSTREAM)
CONCRETE PIER (TYP.)
PILE CAP
GATES IN DOWN POSITION
2 ACCESS DECK
1 S-415
2 S-415
HORIZONTAL COORDINATE SYSTEM: NORTH DAKOTA STATE PLANE NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
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
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
US Army Corps of Engineers St. Paul District


NORTH DAKOTA DIVERSION (LPP)
CONCEPT PROJUMENTATION DRAWINGS
RED RIVER DIVERSION STRUCTURES
FLOOD CONTROL - CONTROL STRUCTURE
FARGO, ND AND MOORHEAD, MN
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
RED RIVER DIVERSION
FLOOD CONTROL - MOD RICE RIVER
STRUCTURE
SECTIONS
SHEET
S-415
US ARMY CORPS OF ENGINEERS
ST. PAUL, MINNESOTA
HORIZONTAL COORDINATE SYSTEM:
NAD 1983 (2011) ADJ
VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET
QUANTITIES (GATED STRUCTURE)
QUANTITIES (WINGWALLS)
ALLOWABLE PILE AXIAL COMPRESSION CAPACITY
SECTION
WILD RICE RIVER CONTROL STRUCTURE
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND GEOTECHNICALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INVESTIGATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
3. FULL GATE DETAIL NOT SHOWN.
NOTE: THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX G
NOTE: THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX G


NORTH DAKOTA DIVERSION (LPP)
PLAN VIEW
WILD RICE RIVER LPP FISH PASSAGE STRUCTURE
SCALE: IN FEET
PROTECTED SIDE
UNPROTECTED SIDE
PROPOSED FINISH GRADE 2-FT ELEVATION CONTOUR (TYP)
UP TO 60 YR EVENT FISH PASSAGE
3H:1V SLOPE (TYP)
EXISTING GRADE ELEVATION CONTOUR (TYP)
POOLS AND RIFFLES NOT SHOWN FOR CLARITY, SEE PROFILE
CONCRETE WALL
TOP OF LEVEE EL. 927
FISH PASSAGE SYSTEM GATES (2)
CONTROL GATE
5 TO 10 YEARS EVENT FISH PASSAGE
3H:1V EXCAVATION
3H:1V SLOPE
RIPRAP (TYP)
WILD RICE CONTROL STRUCTURE
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT PRELIMINARY AND ARE NOT FINAL. ONLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING AND ACCESS ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
1 SECTION
WILD RICE RIVER LPP FISH PASSAGE SYSTEM
SCALE: NTS
WILD RICE RIVER WINGWALL
TIE BACK LEVEE
TOP OF LEVEE
TOP OF WINGWALL
EXISTING GROUND
PROPOSED GRADE
PROPOSED GRADE IN FRONT OF WINGWALL
GATE 1
GATE 2
BOTTOM OF POOL 912.5
BOTTOM OF POOL 914.5
912.5
914.5
916.5
60 YR. EVENT
10 YR. EVENT
2 PROFILE
WILD RICE RIVER LPP FISH PASSAGE SYSTEM
5 TO 10 YEAR EVENTS - CENTERLINE
3 PROFILE
WILD RICE RIVER LPP FISH PASSAGE SYSTEM
UP TO 50 YEAR EVENTS - CENTERLINE
CONTROL GATE W/ GATE HOIST
ROCK/BOULDER RIFFLE (TYP.)
POOL TYP
BOTTOM OF POOL
FINISH GRADE
EXISTING GRADE
5% SLOPE
5% SLOPE TYP
6% SLOPE TYP
RIFFLE
FLOW
20'
50'
2H:1V
ELEVATION
STATION
HORIZONTAL COORDINATE SYSTEM: NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
CONCEPT HOUSING DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD CONTROL - LPP FISH PASSAGE
STRUCTURE ON WILD RICE RIVER
FARGO, ND AND MOORHEAD, MN
PLAN VIEW AND SECTIONS
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
SHEET IDENTIFICATION
S-416


NORTH DAKOTA DIVERSION (LPP)
CONCEPT PLAN
RED RIVER DIVERSION CHANNEL
FARGO, ND AND MOORHEAD, MN
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
RICE RIVER - PLAN VIEW AND ELEVATION
SHEET
S-417
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
HORIZONTAL COORDINATE SYSTEM:
NAD 1983 HARN
NORTH DAKOTA SOUTH
INTERNATIONAL FEET
VERTICAL COORDINATE SYSTEM:
NAVD 1988 US SURVEY FEET
PLAN
CONTROL WEIR SEPARATING RED RIVER FROM WILD RICE RIVER
SCALE: N.T.S.
ELEVATION
CONTROL WEIR SEPARATING RED RIVER FROM WILD RICE RIVER
SCALE: N.T.S.
SECTION
CONTROL WEIR SEPARATING RED RIVER FROM WILD RICE RIVER
SCALE: N.T.S.
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND GEOTECHNICALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL EVALUATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
NOTE: I-29 BRIDGE NOT SHOWN
PROTECTED SIDE
UNPROTECTED SIDE
I-29 BRIDGE (NOT SHOWN)
CONNECTING CHANNEL TO DIVERSION INLET WEIR (AND WILD RICE CONTROL STRUCTURE) (RIGHT FLOWS)
LOW FLOW CHANNEL TO WILD RICE RIVER (CAN FLOW)
TO WILD RICE RIVER AND DIVERSION
FROM RED RIVER OF THE NORTH
FLOW
SHEET PILE (TYP.)
CONSTRUCT EAST WEIR BENEATH I-29 BRIDGE
LOW FLOW CHANNEL TO RED RIVER CONTROL STRUCTURE
RIPRAP
PROPOSED FINISH GRADE 2-FT CONTOUR
7H:1V
SPOIL
FINISH GRADE
LEVEE (TOP EL. 927.0)
50Y YR EL. AT WEIR 922.5
60.0'
OVERFLOW EL. 902.25
GRADE UPSTREAM & DOWNSTREAM OF SHEETPILE EL. 897.27
BOTTOM OF SHEETPILE EL. 882.27
TOP EL. 902.25
SHEET PILE (SINGLE ROW)



PLAN VIEW
STORAGE AREA 1

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

NO.	DESCRIPTION	DATE	APPR'D
1	95% DESIGN MEETING	2/2/2015	
2	95% DESIGN MEETING	2/2/2015	
3	95% DESIGN MEETING	2/2/2015	
4	95% DESIGN MEETING	2/2/2015	
5	95% DESIGN MEETING	2/2/2015	
6	95% DESIGN MEETING	2/2/2015	
7	95% DESIGN MEETING	2/2/2015	
8	95% DESIGN MEETING	2/2/2015	
9	95% DESIGN MEETING	2/2/2015	
10	95% DESIGN MEETING	2/2/2015	

DESIGNED BY	DATE
DRAWN BY	DESIGNATION NO.
CHECKED BY	CONTRACT NO.
SUBMITTED BY	FILE NUMBER
95% SUBMITTAL	FILE NAME

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

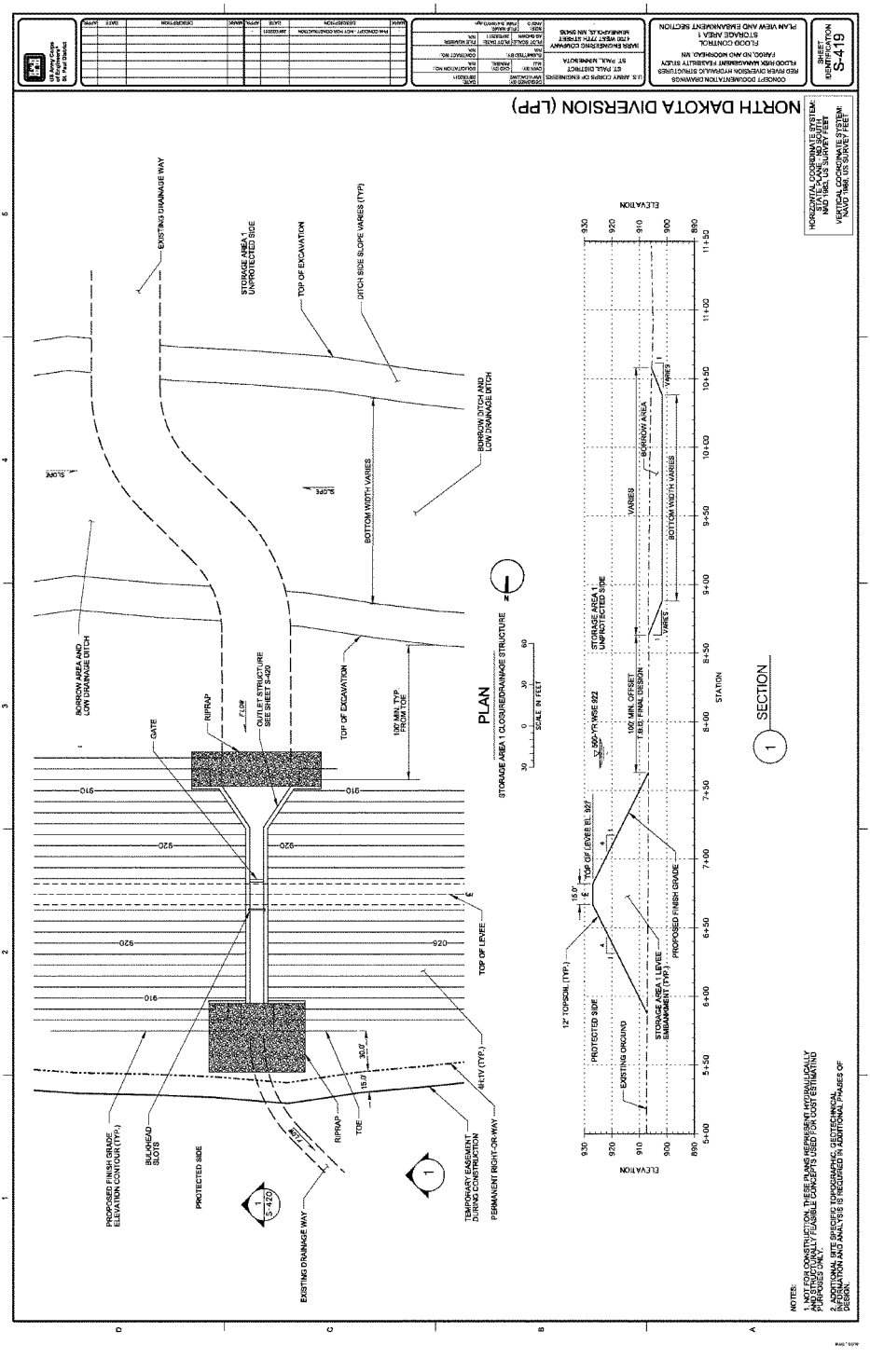
BARR ENGINEERING COMPANY 4300 MARKETPOINTE DRIVE MINNEAPOLIS, MN 55435

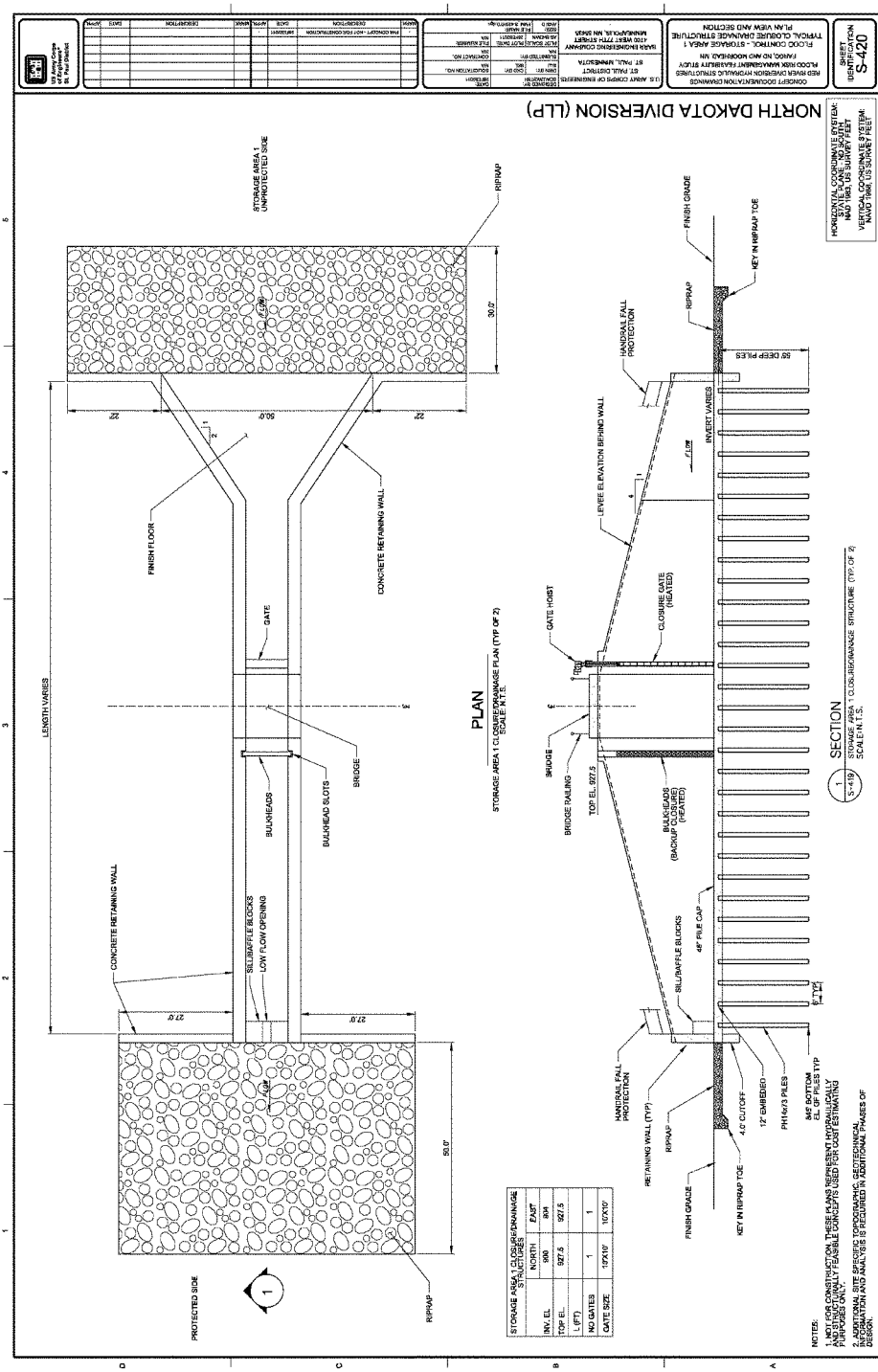
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN

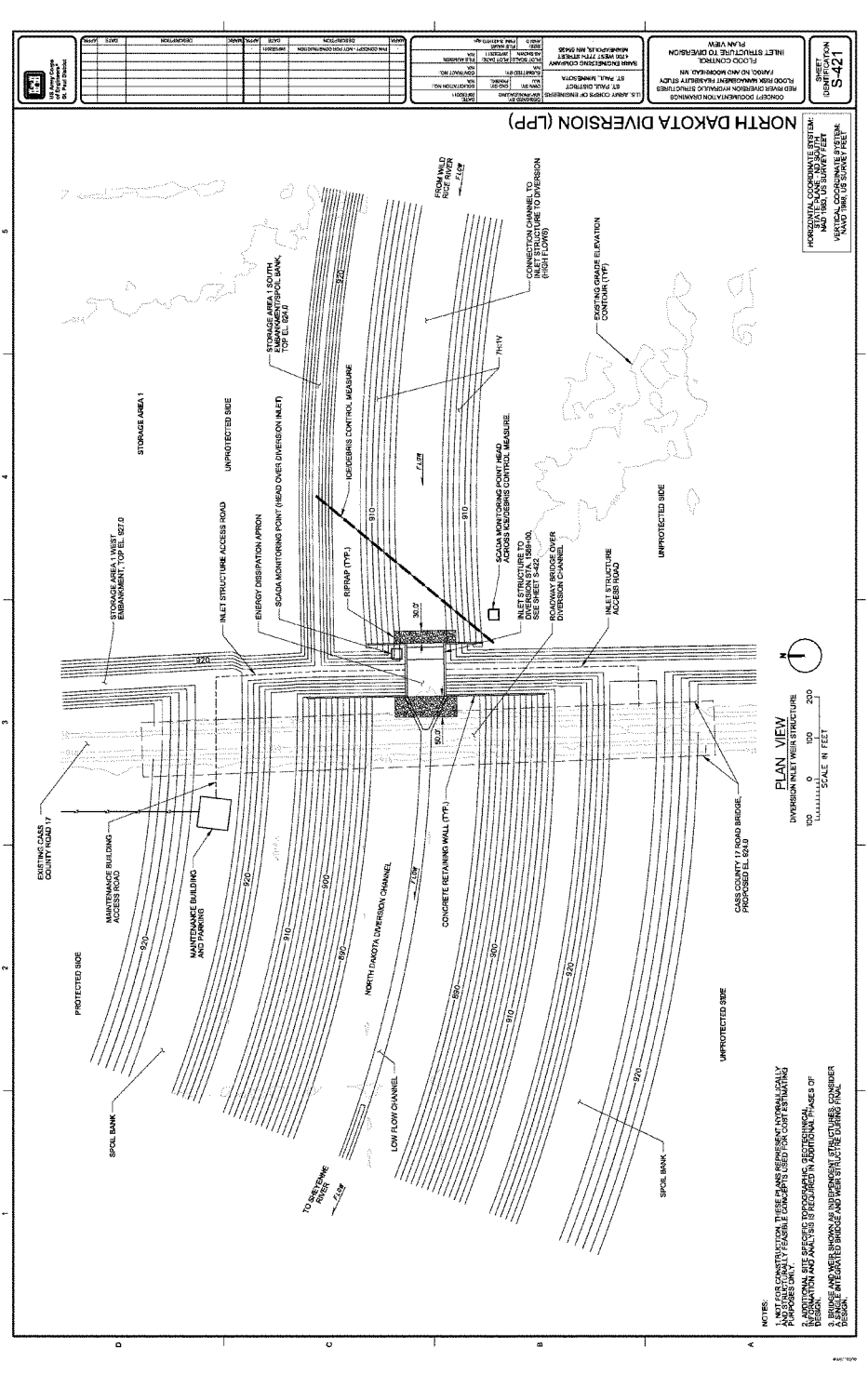
FLOOD CONTROL
STORAGE AREA 1
PLAN VIEW

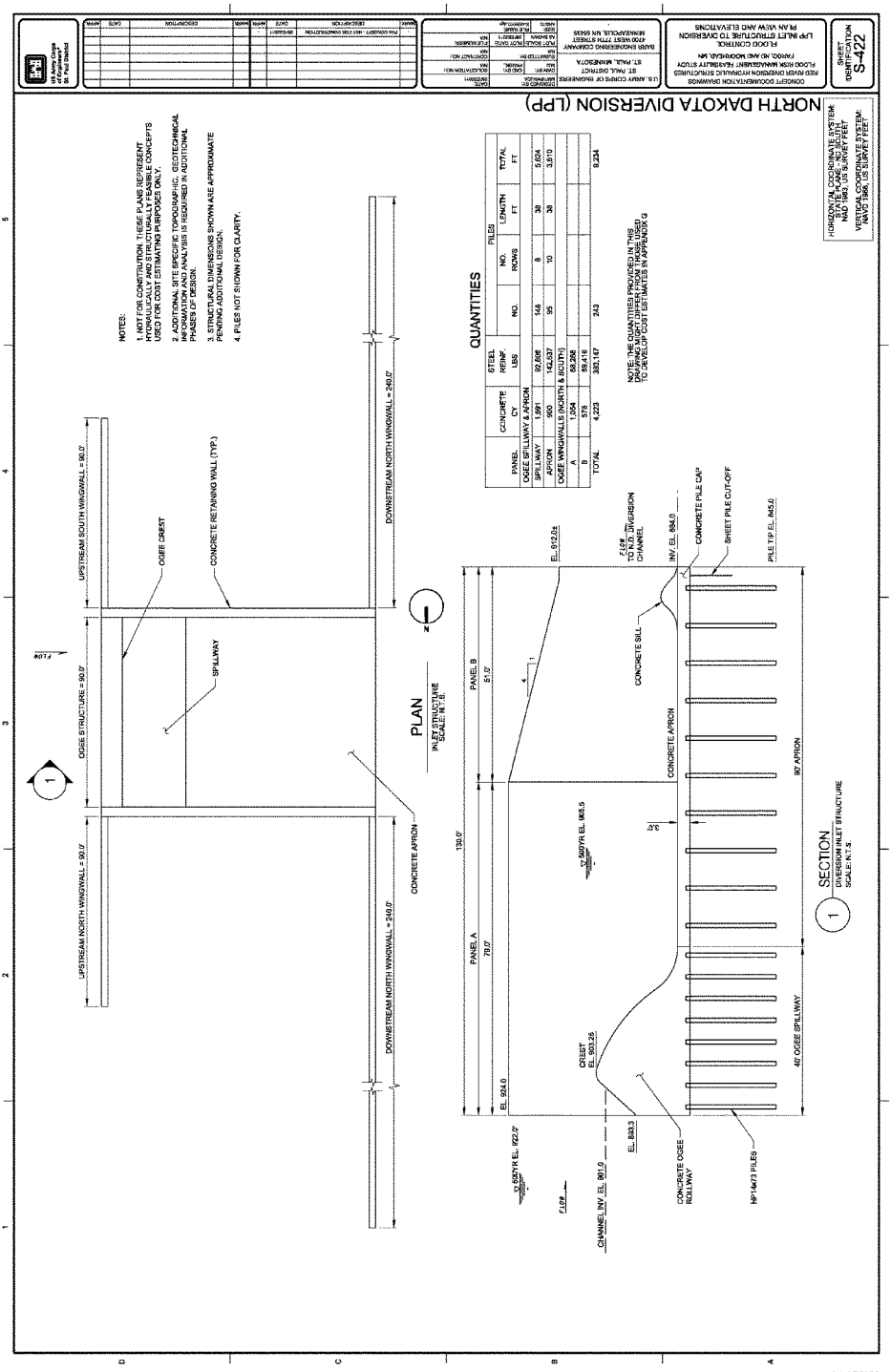
NORTH DAKOTA DIVERSION (LPP)

SHEET IDENTIFICATION
S-418


NORTH DAKOTA DIVERSION (LPP)
PLAN
STORAGE AREA 1 CLOSURE/DRAINAGE STRUCTURE
SCALE IN FEET
SECTION
STORAGE AREA 1 LEVEE EMBANKMENT (TYP.)
PROPOSED FINISH GRADE
EXISTING GROUND
PROTECTED SIDE
STORAGE AREA 1 UNPROTECTED SIDE
TOP OF LEVEE EL. 927
12' TOPSOIL (TYP.)
100' MIN. OFFSET (E.O.S. FINAL DESIGN)
BORROW AREA
BOTTOM WIDTH VARIES
ELEVATION
STATION
TOP OF EXCAVATION
DITCH SIDE SLOPE VARIES (TYP)
BORROW DITCH AND LOW DRAINAGE DITCH
EXISTING DRAINAGE WAY
BORROW AREA AND LOW DRAINAGE DITCH
OUTLET STRUCTURE SEE SHEET S-420
RIPRAP
GATE
FLOW
TOP OF LEVEE
PROPOSED FINISH GRADE ELEVATION CONTOUR (TYP.)
BULKHEAD SLOTS
TOE
TEMPORARY EASEMENT DURING CONSTRUCTION
PERMANENT RIGHT-OF-WAY
4H:1V (TYP.)
100' MIN. TYP. FROM TOE
SLOPE
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND ENVIRONMENTALLY FEASIBLE CHANNELS AND ARE INTENDED FOR PERMITTING PURPOSES ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL, HYDROLOGIC, HYDRAULIC AND ENVIRONMENTAL DATA WILL BE REQUIRED FOR FINAL DESIGN.
3. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL, HYDROLOGIC AND ANALYSIS IS REQUIRED FOR ADDITIONAL PHASES OF DESIGN.
CONCEPT PLAN
RED RIVER DIVERSION
FLOOD RISK MANAGEMENT
FARGO, ND AND MOORHEAD, MN
PLAN VIEW AND EMBANKMENT SECTION
S-419


NORTH DAKOTA DIVERSION (LLP)
PLAN
STORAGE AREA 1 CLOSURE/DRAINAGE PLAN (TYP OF 2)
SCALE: N.T.S.
SECTION
STORAGE AREA 1 CLOSURE/DRAINAGE STRUCTURE (TYP OF 2)
SCALE: N.T.S.
LENGTH VARIES
CONCRETE RETAINING WALL
SILT/BAFFLE BLOCKS
LOW FLOW OPENING
BULKHEADS
BULKHEAD SLOTS
BRIDGE
GATE
FINISH FLOOR
STORAGE AREA 1 UNPROTECTED SIDE
RIPRAP
PROTECTED SIDE
50.0'
27.0'
22'
GATE HOIST
BRIDGE RAILING
TOP EL. 927.5
CLOSURE GATE (HEATED)
LEVEE ELEVATION BEHIND WALL
HANDRAIL FALL PROTECTION
INVERT VARIES
55' DEEP PILES
FINISH GRADE
KEY IN RIPRAP TOE
BULKHEADS (BACKUP CLOSURE) (HEATED)
SILT/BAFFLE BLOCKS
48" PILE CAP
RETAINING WALL (TYP)
4.0' CUTOFF
12' EMBEDED
PH14x73 PILES
94' BOTTOM EL. OF PILES TYP
STORAGE AREA 1 CLOSURE/DRAINAGE STRUCTURES
NORTH
EAST
INV. EL.
TOP EL.
L (FT)
NO. GATES
GATE SIZE
927.5
927.5
1
1
10'X10'
10'X10'
NOTES:
1. PLANS FOR CONSTRUCTION. THESE PLANS REPRESENT INFORMATION 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: NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
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
SHEET IDENTIFICATION
S-420
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
MINNEAPOLIS, MN 55415
MAIN ENGINEERING COMPANY
ST. PAUL, MINNESOTA
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
US Army Corps of Engineers St. Paul District


NORTH DAKOTA DIVERSION (LPP)
PLAN VIEW
DIVERSION INLET WEIR STRUCTURE
SCALE IN FEET
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT PRELIMINARY ANALYSIS AND DESIGN AND ARE PROVIDED TO DEMONSTRATE FEASIBILITY PURPOSES ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL AND HYDRAULIC ANALYSIS IS REQUIRED FOR ADDITIONAL PHASES OF DESIGN.
3. BRIDGE AND WEIR SHOWN AS INDEPENDENT STRUCTURES CONSIDER COMBINING THE BRIDGE AND WEIR STRUCTURE DURING FINAL DESIGN.
EXISTING CASS COUNTY ROAD 17
MAINTENANCE BUILDING ACCESS ROAD
MAINTENANCE BUILDING AND PARKING
PROTECTED SIDE
SPOIL BANK
TO SHEYENNE RIVER
LOW FLOW CHANNEL
NORTH DAKOTA DIVERSION CHANNEL
CONCRETE RETAINING WALL (TYP.)
CASS COUNTY 17 ROAD BRIDGE, PROPOSED EL. 924.0
UNPROTECTED SIDE
SPOIL BANK
STORAGE AREA 1 WEST EMBANKMENT, TOP EL. 927.0
STORAGE AREA 1
INLET STRUCTURE ACCESS ROAD
UNPROTECTED SIDE
ENERGY DISSIPATION APRON
SCADA MONITORING POINT (HEAD OVER DIVERSION INLET)
STORAGE AREA 1 SOUTH EMBANKMENT/SPOIL BANK, TOP EL. 924.0
RIPRAP (TYP.)
IDE/DEBRIS CONTROL MEASURE
30.0
60.0
SCADA MONITORING POINT, HEAD ACROSS IDE/DEBRIS CONTROL MEASURE.
INLET STRUCTURE TO DIVERSION (NOT SHOWN), SEE SHEET S-422
ROADWAY BRIDGE OVER DIVERSION CHANNEL
INLET STRUCTURE ACCESS ROAD
UNPROTECTED SIDE
EXISTING GRADE ELEVATION CONTOUR (TYP)
CONNECTION CHANNEL TO INLET STRUCTURE TO DIVERSION (HIGH FLOWS)
FROM WILD RICE RIVER
TO DIVERSION INLET WEIR STRUCTURE
CONCEPT PLAN
RED RIVER DIVERSION, PERMANENT FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FLOOD CONTROL
FARGO, ND AND MOORHEAD, MN
INLET STRUCTURE TO DIVERSION
PLAN VIEW
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
HORIZONTAL COORDINATE SYSTEM: NAD 1983 (2011) NORTH DAKOTA SOUTH
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
SHEET DESIGNATION
S-421


NORTH DAKOTA DIVERSION (LPP)
PLAN
INLET STRUCTURE
SCALE: N.T.S.
SECTION
OVERFLOW INLET STRUCTURE
SCALE: N.T.S.
QUANTITIES
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 AND HYDRAULIC ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
3. STRUCTURAL DIMENSIONS SHOWN ARE APPROXIMATE PENDING ADDITIONAL DESIGN.
4. PILES NOT SHOWN FOR CLARITY.
OGEE STRUCTURE = 90.0'
UPSTREAM SOUTH WINGWALL = 90.0'
UPSTREAM NORTH WINGWALL = 90.0'
DOWNSTREAM NORTH WINGWALL = 240.0'
OGEE CREST
SPILLWAY
CONCRETE RETAINING WALL (TYP.)
CONCRETE APRON
PANEL A
78.0'
PANEL B
51.0'
130.0'
EL. 912.0±
EL. 924.0
CREST EL. 903.25
EL. 893.3
CHANNEL INV. EL. 901.9
CONCRETE OGEE SPILLWAY
HP14x73 PILES
40' OGEE SPILLWAY
87' APRON
CONCRETE SILL
INV. EL. 894.0
CONCRETE PILE CAP
SHEET PILE CUT-OFF
PILE TIP EL. 845.0
TO N.D. DIVERSION CHANNEL
HORIZONTAL COORDINATE SYSTEM: NAD 1983 (2011), UTM ZONE 14N, NAD 1983, US SURVEY FEET.
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET.
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
CONSTRUCTION DRAWINGS
RED RIVER EMERGENCY HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT, FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN
LPP INLET STRUCTURE TO DIVERSION
FLOOD CONTROL
PLAN VIEW AND ELEVATIONS
SHEET IDENTIFICATION
S-422

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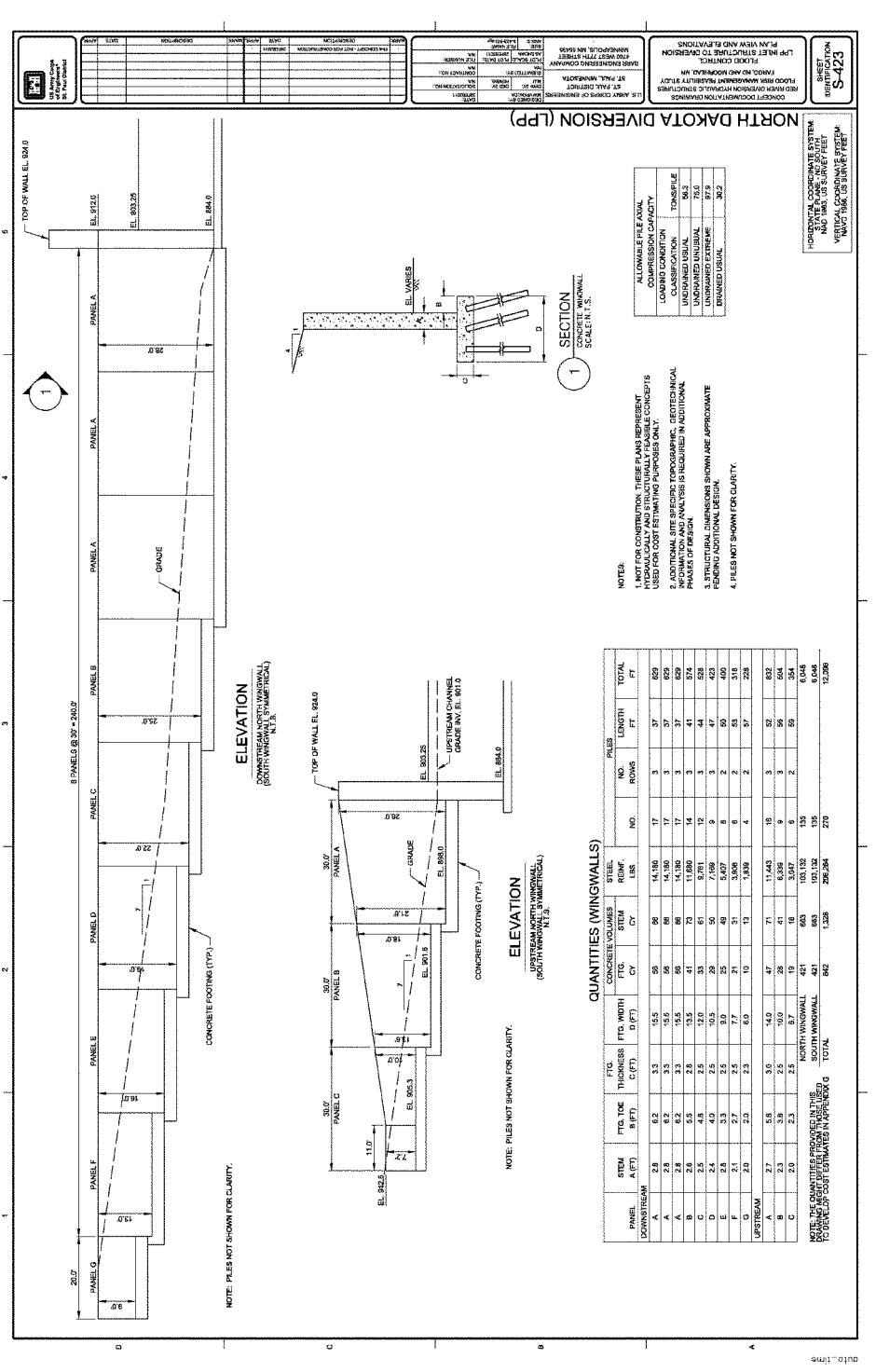
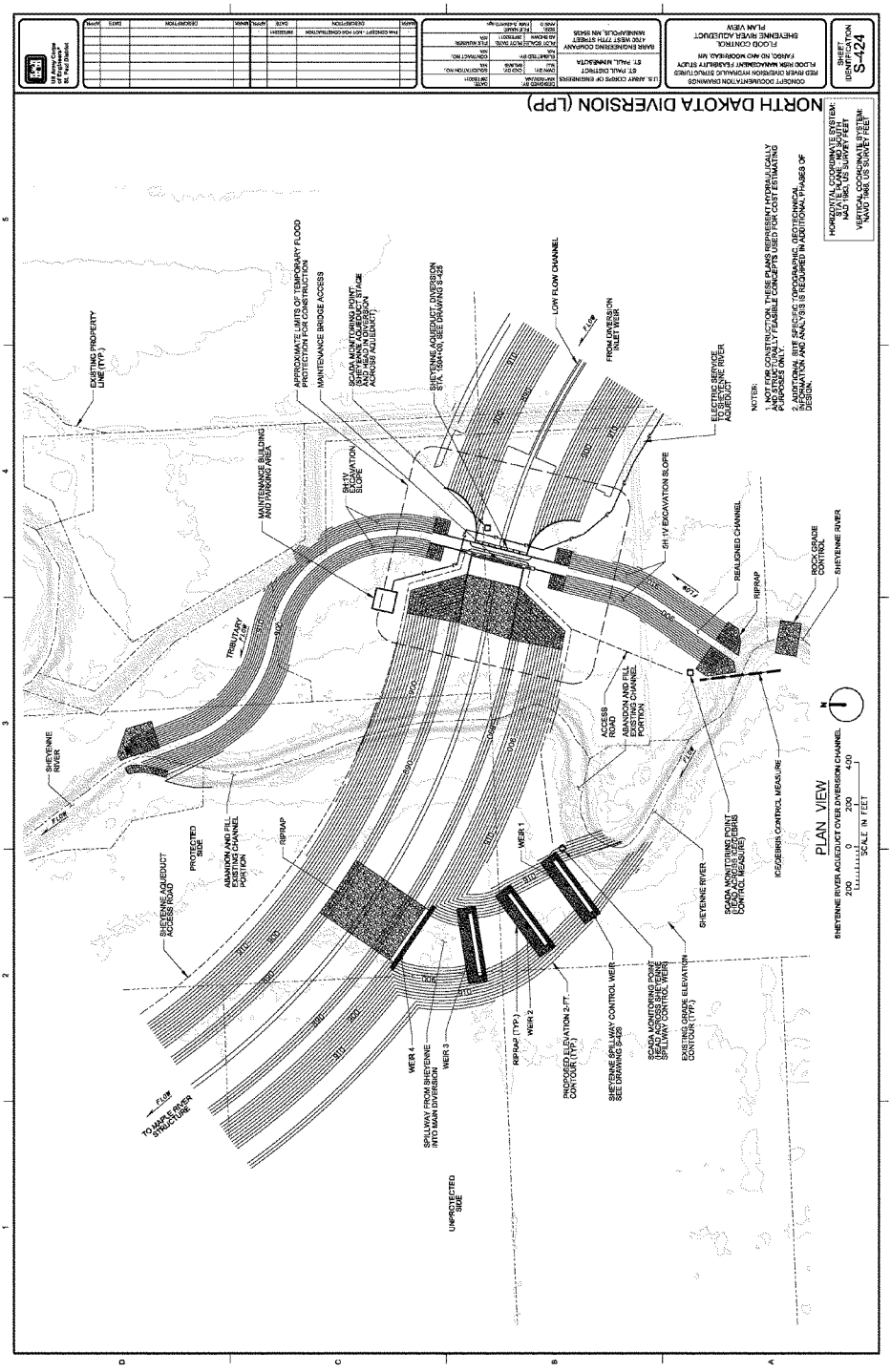
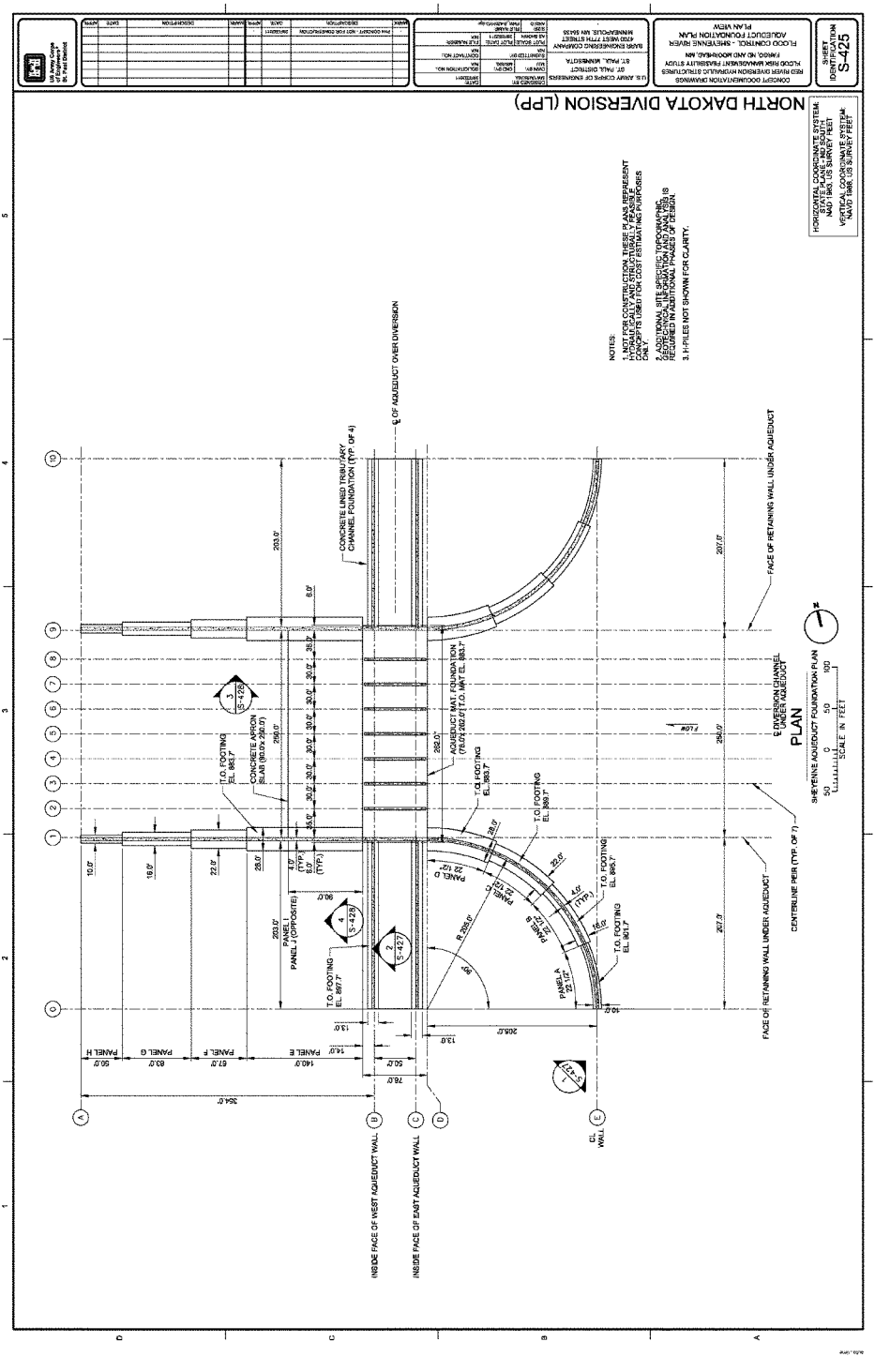
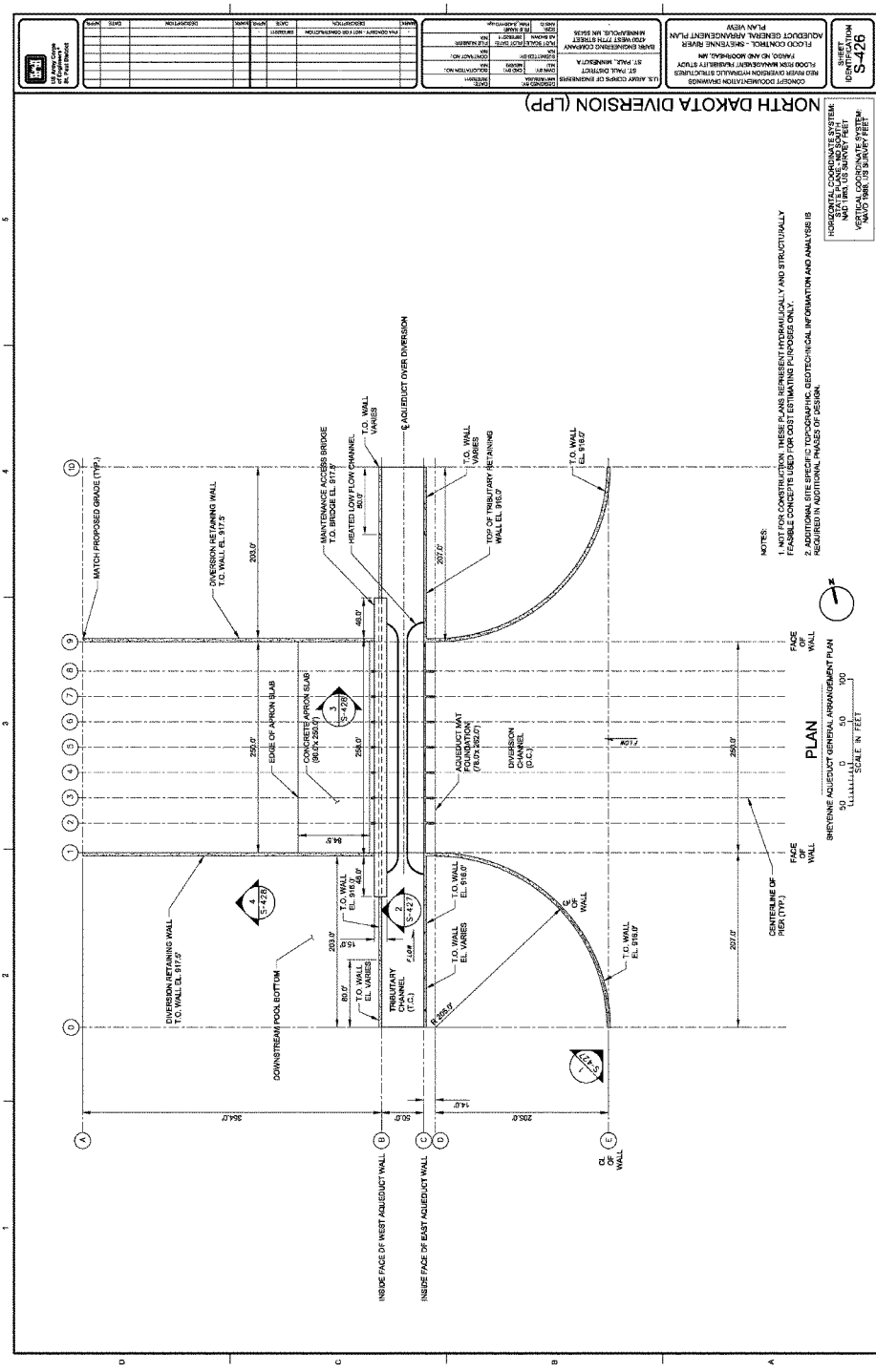
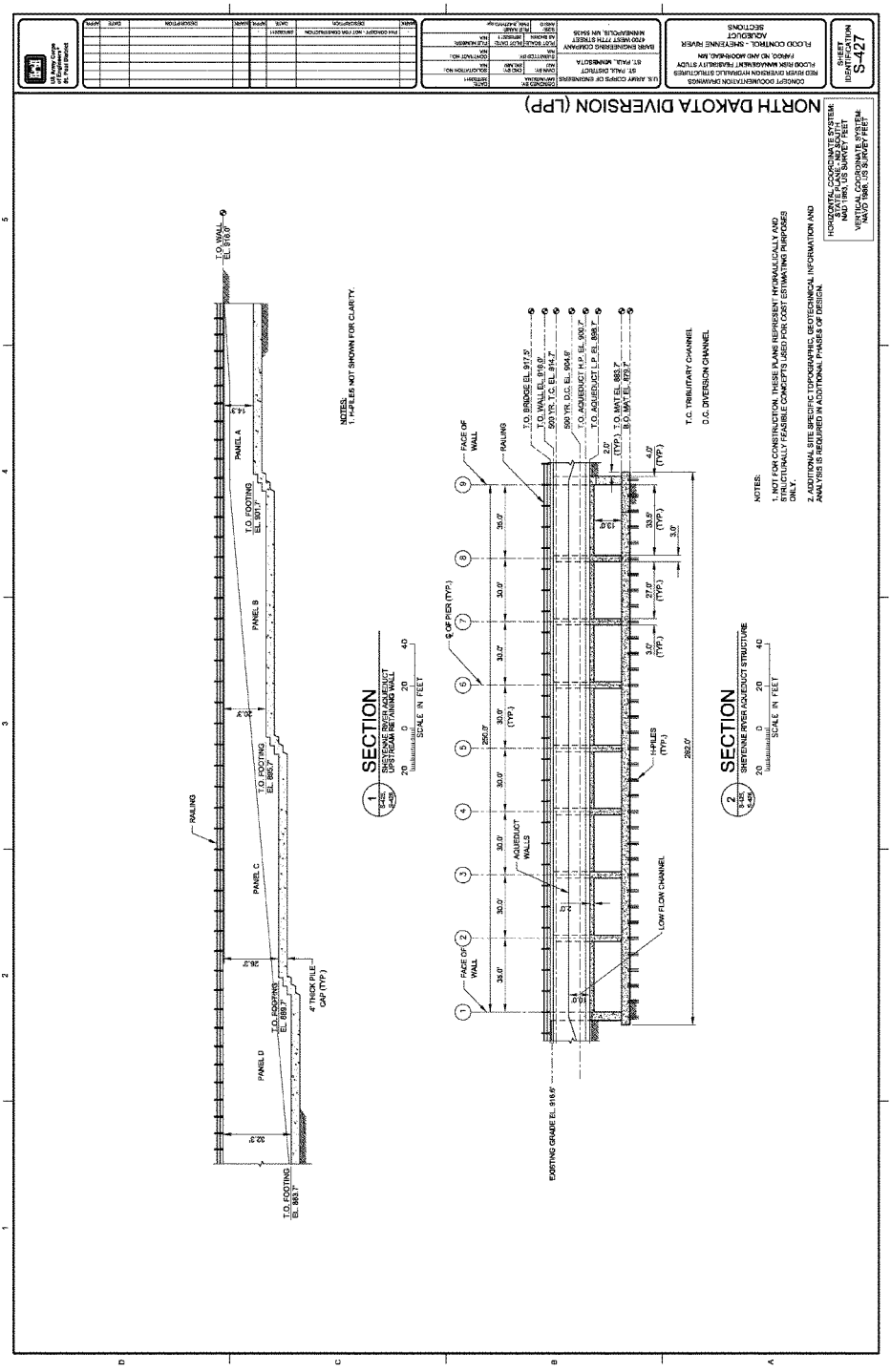

NORTH DAKOTA DIVERSION (LPP)
ELEVATION
SECTION
CONCRETE WINGWALL
SCALE: N.T.S.
QUANTITIES (WINGWALLS)
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT PRELIMINARY LAYOUT AND FEASIBILITY CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
3. STRUCTURAL DIMENSIONS SHOWN ARE APPROXIMATE PENDING ADDITIONAL DESIGN.
4. PILES NOT SHOWN FOR CLARITY.
ALLOWABLE PILE AXIAL COMPRESSION CAPACITY
HORIZONTAL COORDINATE SYSTEM: NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION PROJECT
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN
PLAN VIEW AND ELEVATIONS
FLOOD CONTROL
LPP INLET STRUCTURE TO DIVERSION
SHEET DESIGNATION
S-423

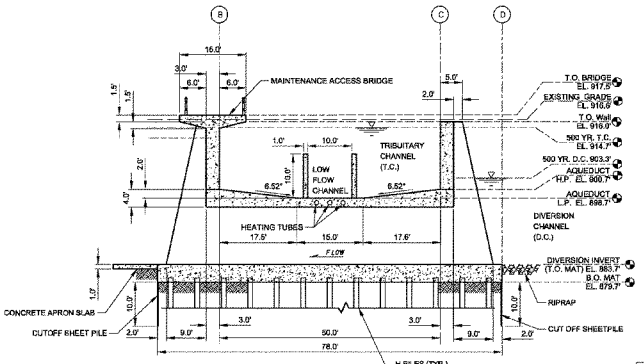

NORTH DAKOTA DIVERSION (LPP)
PLAN VIEW
SHEYENNE RIVER AQUEDUCT OVER DIVERSION CHANNEL
SCALE IN FEET
EXISTING PROPERTY LINE (TYP.)
APPROXIMATE LIMITS OF TEMPORARY FLOOD PROTECTION FOR CONSTRUCTION
MAINTENANCE BRIDGE ACCESS
SCADA MONITORING POINT (HEAD ACROSS SHEYENNE AQUEDUCT AND HEAD ACROSS DIVERSION ACROSS AQUEDUCT)
SHEYENNE AQUEDUCT DIVERSION STA. 1594+00, SEE DRAWING S-425
LOW FLOW CHANNEL
FROM DIVERSION INLET WEIR
MAINTENANCE BUILDING AND PARKING AREA
5H:1V EXCAVATION SLOPE
ELECTRIC SERVICE TO SHEYENNE RIVER AQUEDUCT
5H:1V EXCAVATION SLOPE
REALIGNED CHANNEL
RIPRAP
POST-PROJECT SHEYENNE RIVER
SHEYENNE RIVER
TRIBUTARY FLOW
SHEYENNE AQUEDUCT ACCESS ROAD
PROTECTED SIDE
ABUTMENT AND FILL (EXISTING CHANNEL PORTION)
RIPRAP
WEIR 1
ACCESS ROAD
ABUTMENT AND FILL (EXISTING CHANNEL PORTION)
SHEYENNE RIVER
SCADA MONITORING POINT (HEAD ACROSS WEIR 1)
DEBRIS CONTROL MEASURE
TO MAPLE RIVER SYSTEM
WEIR 4
WEIR 3
SPILLWAY FROM SHEYENNE INTO MAPLE DIVERSION
RIPRAP (TYP.)
WEIR 2
PROPOSED ELEVATION 2-FT. CONTOUR (TYP.)
SHEYENNE SPILLWAY CONTROL WEIR SEE DRAWING S-426
SCADA MONITORING POINT (HEAD ACROSS SHEYENNE SPILLWAY CONTROL WEIR)
EXISTING GRADE ELEVATION CONTOUR (TYP.)
UNPROTECTED SIDE
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: NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
CONSTR'D DOUBLEBARREL DRAINAGE RES RIVER DIVERSION HYDRAULIC STRUCTURES FLOOD CONTROL SHEYENNE RIVER ADJUSTMENT FARGO, ND AND WOORDEAU, MN
PLAN VIEW
SHEET IDENTIFICATION S-424
U.S. ARMY CORPS OF ENGINEERS ST. PAUL DISTRICT ST. PAUL, MINNESOTA
BARR ENGINEERING COMPANY 4700 WEST 77TH STREET MINNEAPOLIS, MN 55435
DESIGNED BY DRAWN BY CHECKED BY APPROVED BY
DATE 08/15/2011
DWG NO. 2121-121
SHEET NO. 1 OF 1 SHEETS
CONTRACT NO. N/A
CONSTRUCTION NO. N/A
SPEC. NO. 23183
DESCRIPTION
DATE
APPR.
BY
DESCRIPTION
DATE
APPR.
BY
US Army Corps of Engineers St. Paul District


NORTH DAKOTA DIVERSION (LPP)
PLAN
SHEET INDEX NUMBER S-425
FLOOD CONTROL - SHEYENNE RIVER
AQUEDUCT FOUNDATION PLAN
PLAN VIEW
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL, MINNESOTA
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
HORIZONTAL COORDINATE SYSTEM: NAD 1983 US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
NOTES:
1. BRIDGE CONSTRUCTION SHOWN IS AS REPRESENT IN PRELIMINARY AND STRUCTURAL FEASIBILITY CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC INFORMATION IS REQUIRED FOR FINAL DESIGN AND IS REQUIRED IN ADDITIONAL PLAN AND PROFILES.
3. H-PILES NOT SHOWN FOR CLARITY.
CONCRETE LINED TRIBUTARY CHANNEL FOUNDATION (TYP. OF 4)
℄ OF AQUEDUCT OVER DIVERSION
AQUEDUCT MAT FOUNDATION (TO EL. 902.0') T.O. MAT EL. 893.7'
CONCRETE APRON SLAB (80.0'x 290.0')
T.O. FOOTING EL. 893.7'
T.O. FOOTING EL. 897.7'
T.O. FOOTING EL. 898.7'
T.O. FOOTING EL. 895.7'
T.O. FOOTING EL. 901.7'
PANEL I
PANEL J (OPPOSITE)
PANEL A
PANEL B
PANEL C
PANEL D
PANEL E
PANEL F
PANEL G
PANEL H
R 206.0'
45°
293.0'
354.0'
207.0'
FACE OF RETAINING WALL UNDER AQUEDUCT
FACE OF RETAINING WALL UNDER AQUEDUCT
CENTERLINE PIER (TYP. OF 7)
INSIDE FACE OF WEST AQUEDUCT WALL
INSIDE FACE OF EAST AQUEDUCT WALL
CL WALL E
℄ DIVERSION CHANNEL UNDER AQUEDUCT
SHEYENNE AQUEDUCT FOUNDATION PLAN
SCALE IN FEET
0 50 100
50 0 50



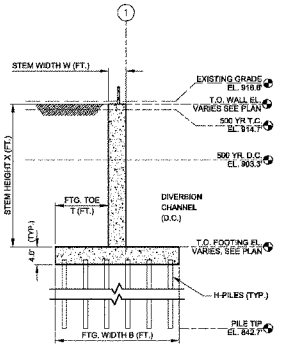
USE Army Corps of Engineers St. Paul District


NORTH DAKOTA DIVERSION (LPP)
SHEET IDENTIFICATION
S-427
SHEYENNE RIVER AQUEDUCT STRUCTURE
SCALE IN FEET
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.
T.O. WALL EL. 916.0'
T.O. FOOTING EL. 893.7'
PANEL A
PANEL B
PANEL C
PANEL D
RAILING
4' THICK PILE CAP (TYP.)
EXISTING GRADE EL. 916.0'
FACE OF WALL
AQUEDUCT WALLS
LOW FLOW CHANNEL
H-PILES (TYP.)
T.O. BRIDGE EL. 917.5'
T.O. WALLS EL. 916.0'
500 YR. T.C. EL. 914.7'
500 YR. D.C. EL. 904.6'
T.O. AQUEDUCT I.P. EL. 900.7'
T.O. AQUEDUCT I.P. EL. 898.7'
T.O. MAT EL. 883.7'
B.O. MAT EL. 878.7'
T.C. TRIBUTARY CHANNEL
D.C. DIVERSION CHANNEL
282.0'
256.0'
30.0' (TYP.)
4.0' (TYP.)
3.0'
2.0' (TYP.)
HORIZONTAL COORDINATE SYSTEM: NAD 1983 US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988 US SURVEY FEET
DOCKET DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN
FLOOD CONTROL - SHEYENNE RIVER
AQUEDUCT
SECTIONS
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
190 FIFTH STREET EAST
ST. PAUL, MINNESOTA 55101-1638
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
MVP
MVD
MVR
MVS
MVK
MVM
US Army Corps of Engineers
St. Paul District



3 S-426, S-428 **SECTION**
SHEYENNE AQUEDUCT
10 0 10 20
SCALE IN FEET

ALLOWABLE PILE CAPACITY AXIAL COMPRESSION	
LOADING CONDITION CLASSIFICATION	TONS/PILE
UNDRAINED USUAL	92.0
UNDRAINED UNUSUAL	92.6
UNDRAINED EXTREME	107.7
DRAINED USUAL	36.5



4 S-425, S-426 **SECTION**
RETAINING WALL
10 0 10 20
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 AND GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.

QUANTITIES (AQUEDUCT)

	CONCRETE VOLUMES (CY.)	STEEL (LBS.)
FOOTINGS	3,781	832,485
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)

PANEL	STEM W (FT)	STEM HEIGHT X (FT)	FTG. WIDTH B (FT)	FTG. TOE T (FT)	CONCRETE VOLUMES FTG. (CY)	CONCRETE VOLUMES STEM (CY)	STEEL REINF. STEM (LBS.)	STEEL REINF. FTG.	PILES NO.	PILES NO. ROWS	PILES LENGTH FT	PILES TOTAL FT
A	4	14.3	10	3	120	171	31,662	21,897	24	12	56	1,344
B	4	20.3	16	8	191	243	44,929	36,035	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	100,824	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	35,122	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	101,158	71,781	72	24	52	3,744
J	3	18.3	13	5	391	413	101,159	71,781	72	24	52	3,744
				SOUTH WALL	2,776	3,436	682,742	506,417	527			24,118
				NORTH WALL	2,776	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

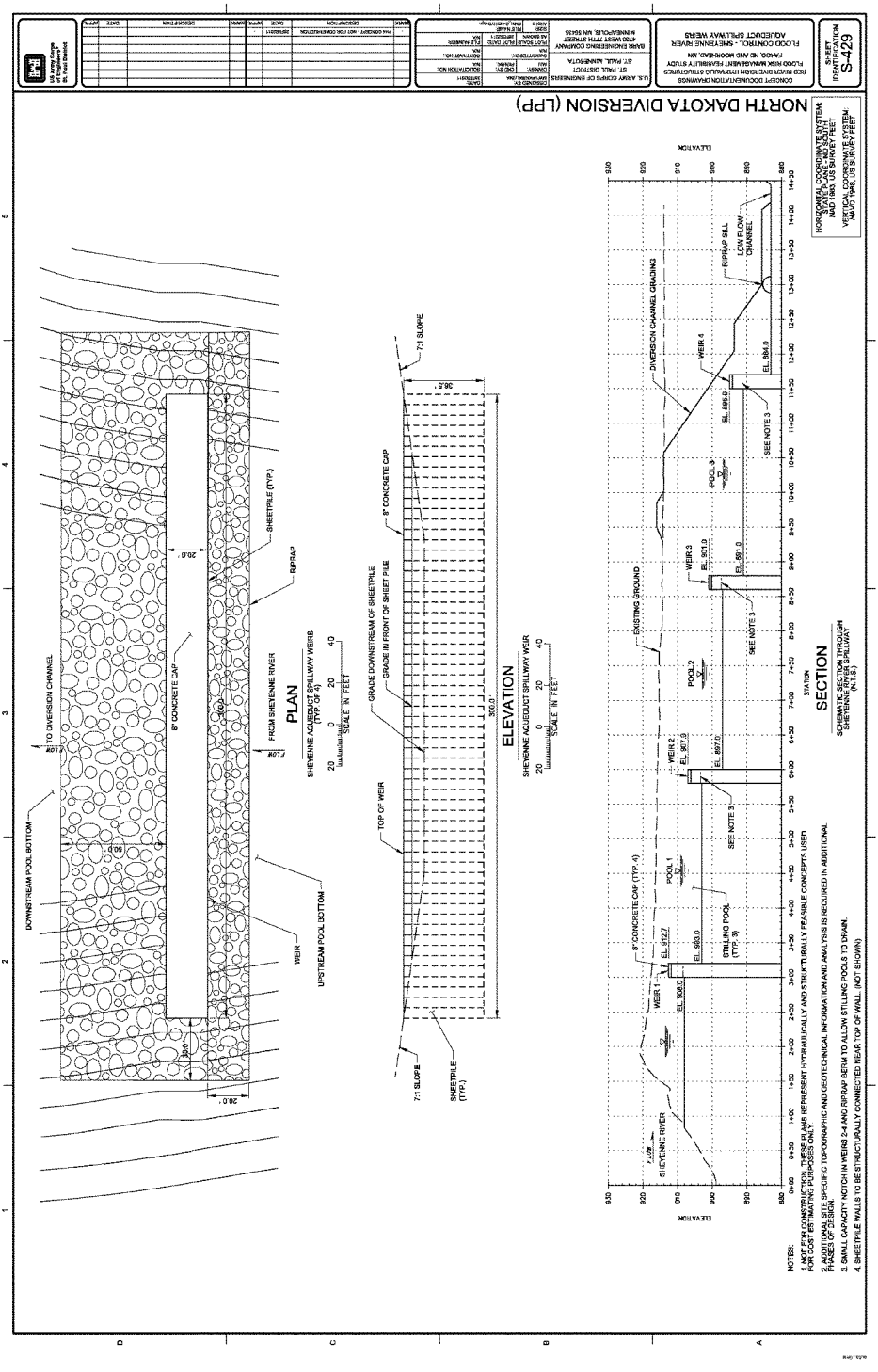
NO.	DESCRIPTION	DATE	APP'D
	PRE-CONSTRUCTION DESIGN		
	30% DESIGN		
	60% DESIGN		
	90% DESIGN		
	FINAL DESIGN		

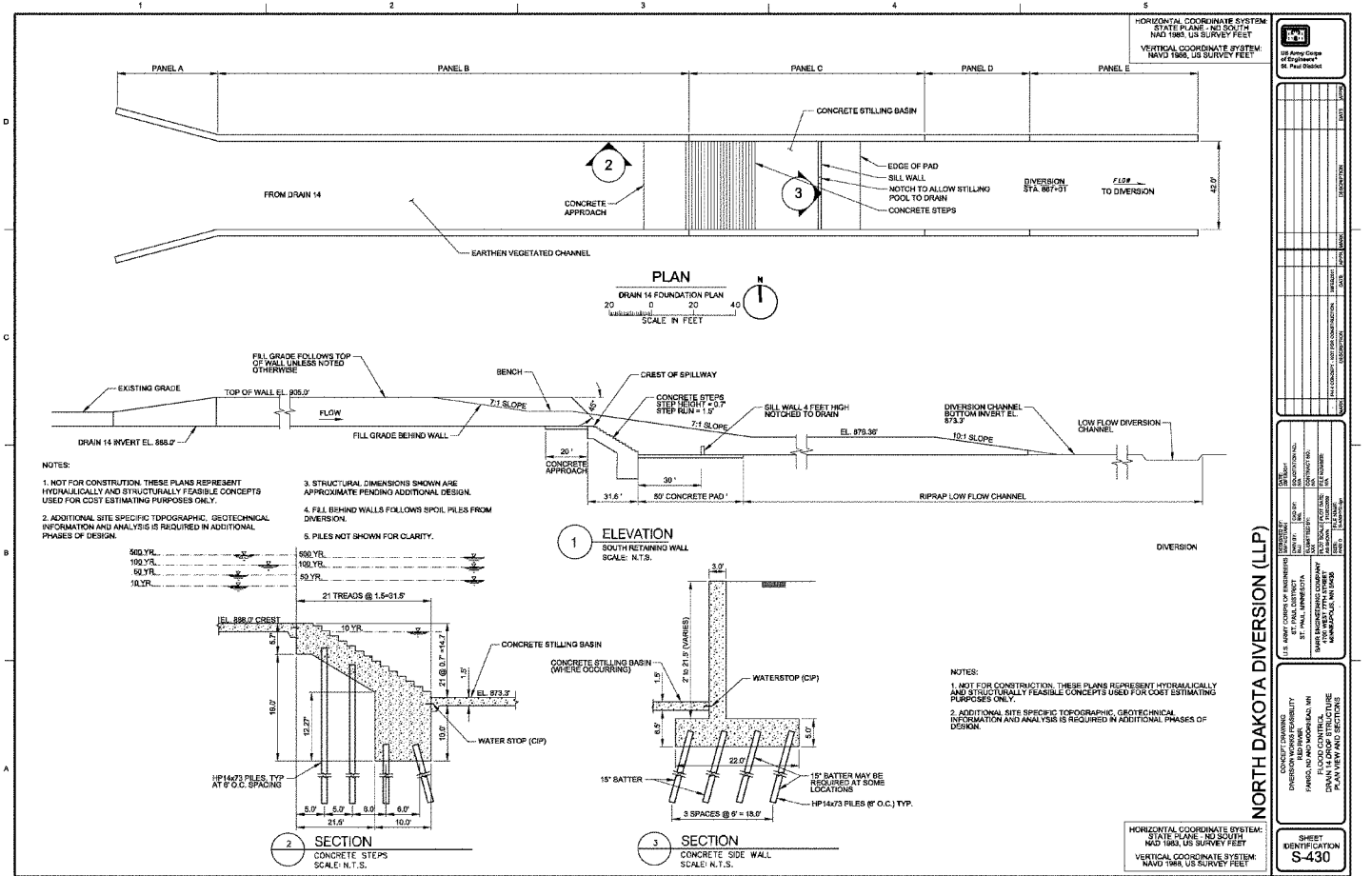
DESIGNED BY: Ma
DRAWN BY:
CHECKED BY:
QA/QC BY:
APPROVED BY:
DATE:
SUBMITTED BY:
DISTRICT
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
BARR ENGINEERING COMPANY
4300 MARKETPOINTE DRIVE
MINNEAPOLIS, MN 55435
EDITION NO.:
CONTRACT NO.:
PROJECT NO.:
FILE NUMBER:
SHEET NO.:
FILE NAME:
MAP LAYOUT:

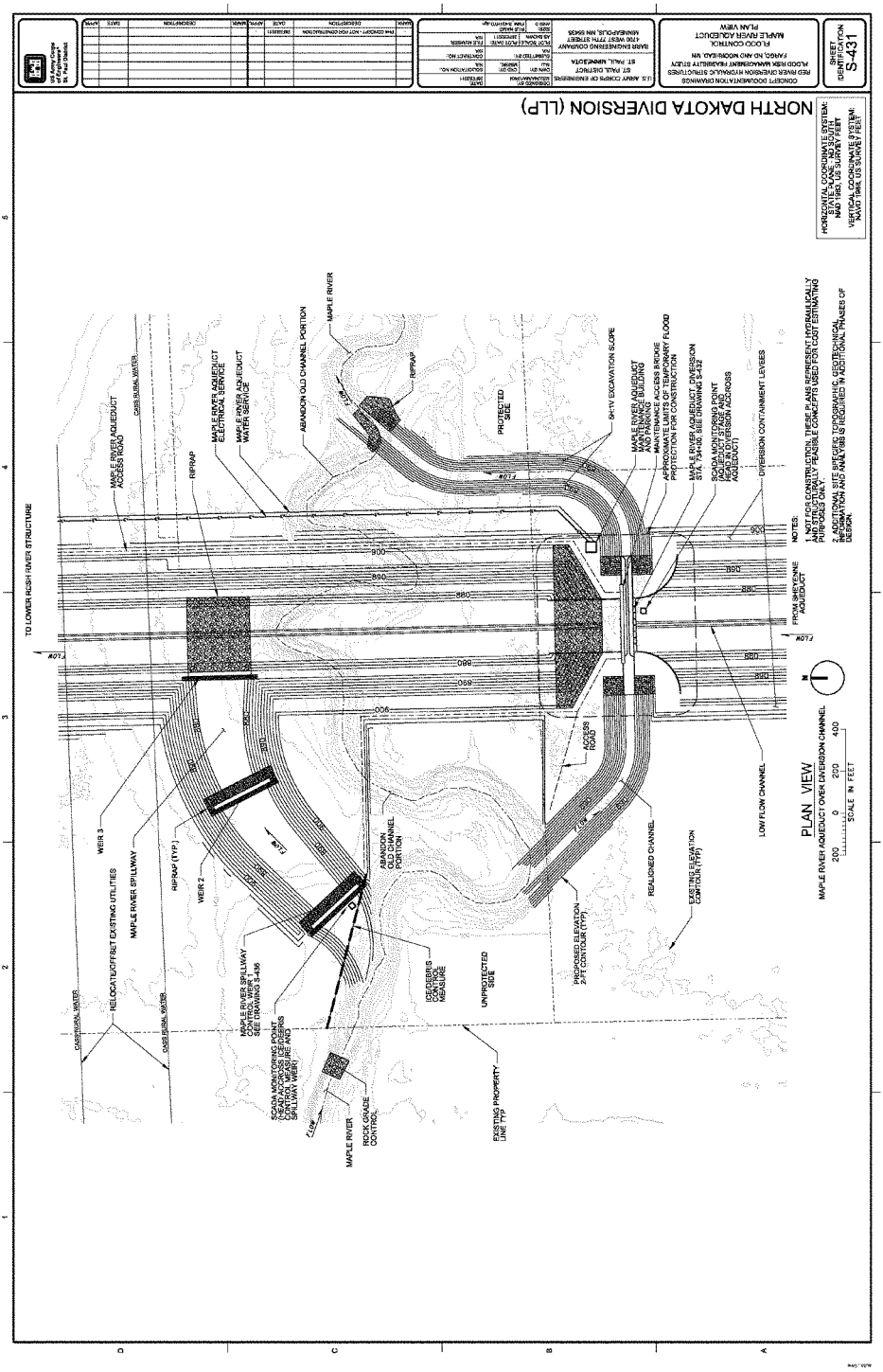
NORTH DAKOTA DIVERSION (LPP)

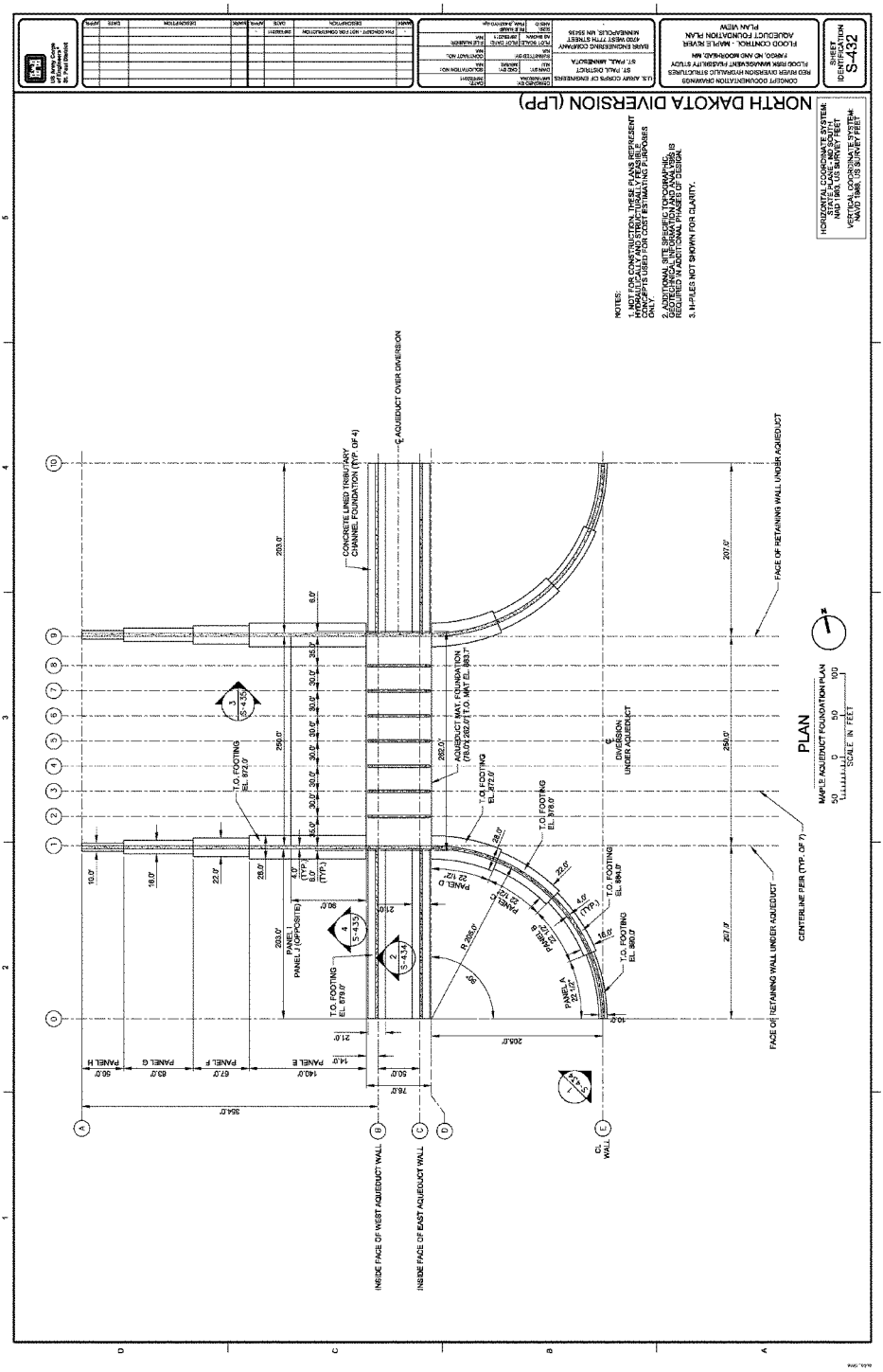
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD CONTROL - MAPLE RIVER DIVERSION
FARGO, ND AND MOORHEAD, MN
FLOOD CONTROL - SHEYENNE RIVER
DIVERSION
AQUEDUCT SECTIONS

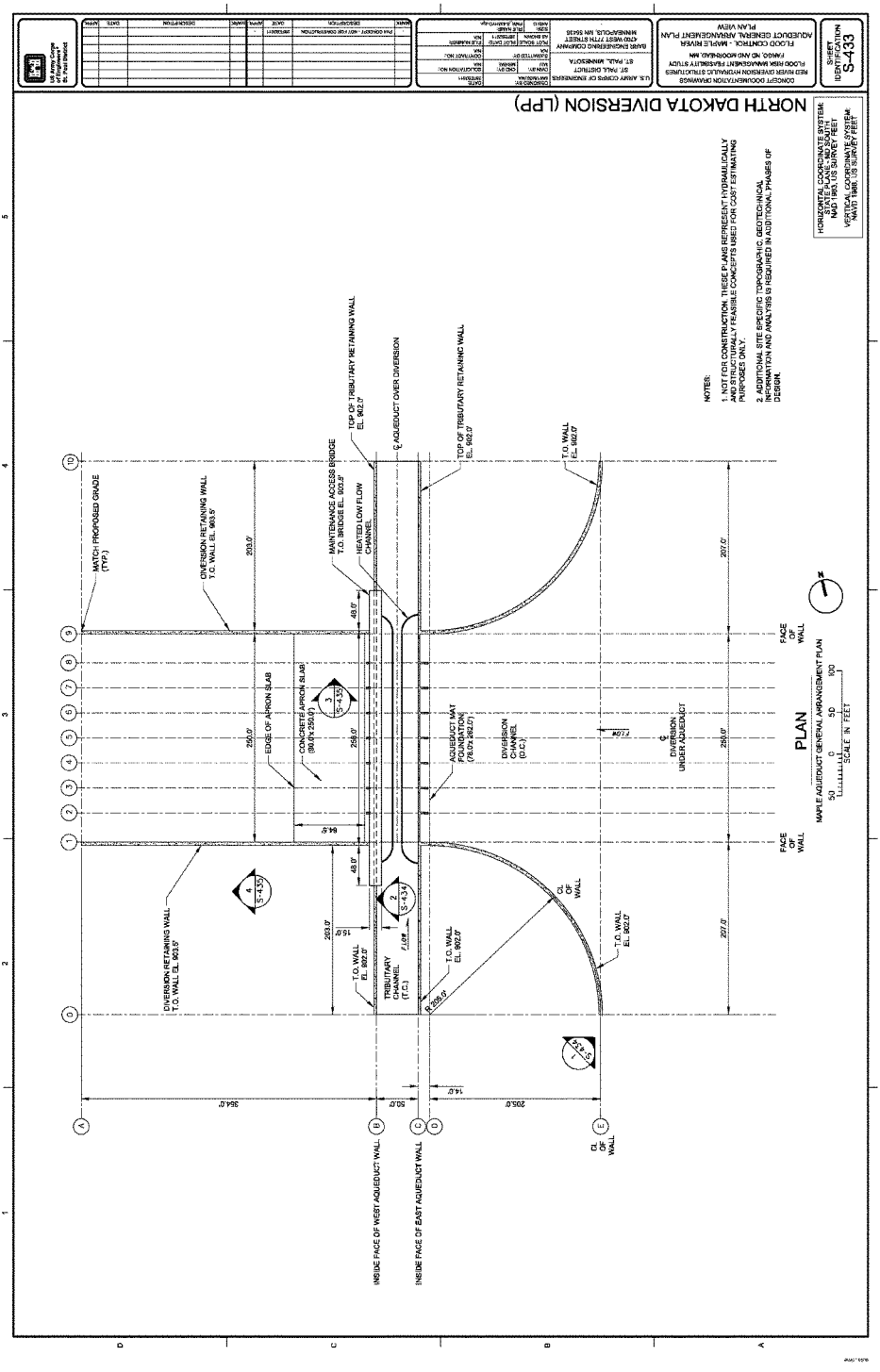
SHEET IDENTIFICATION
S-428

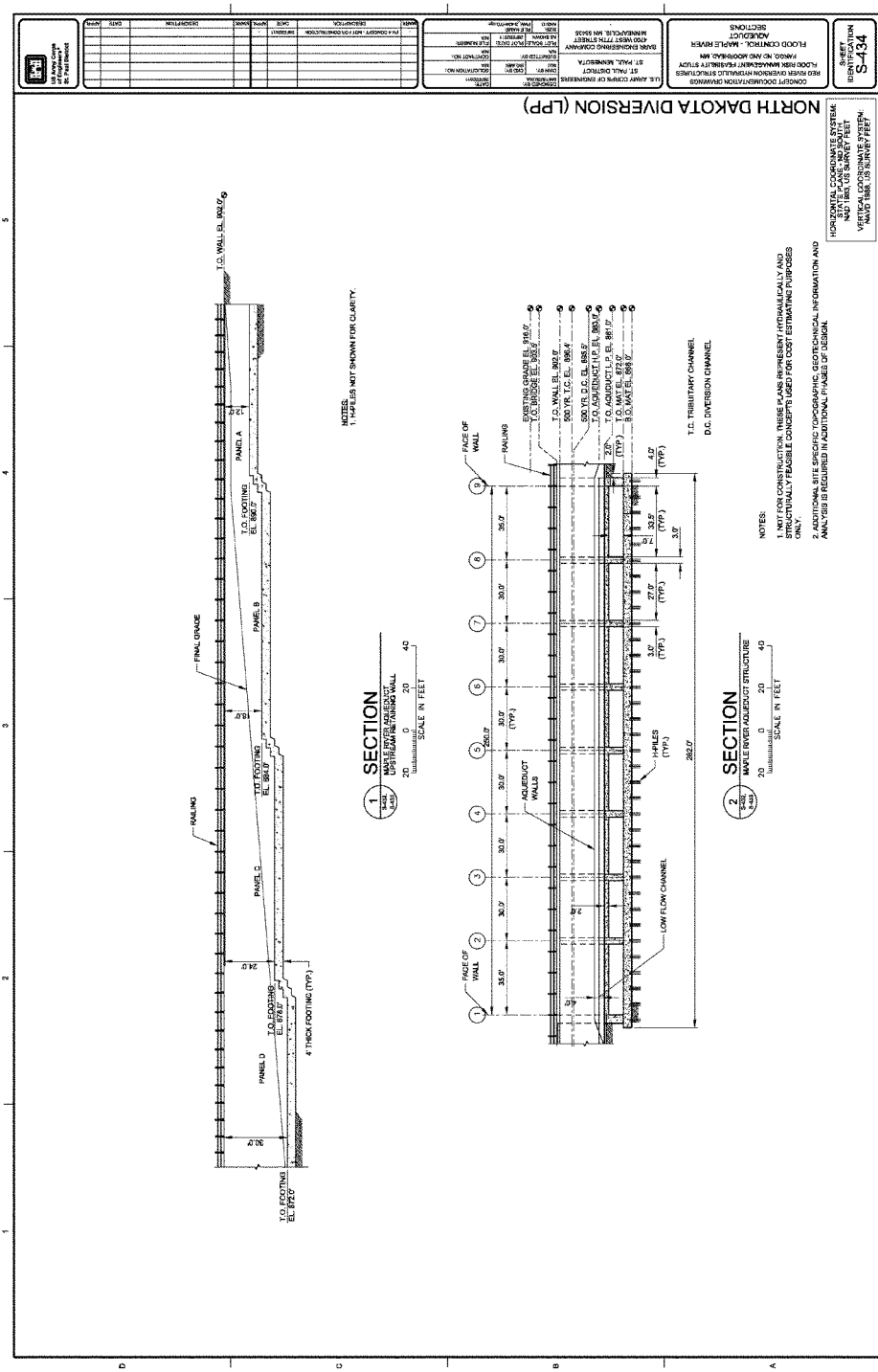

NORTH DAKOTA DIVERSION (LPP)
PLAN
ELEVATION
SECTION
SHEYENNE AQUEDUCT SPILLWAY WEIR (TYP. OF 4)
SCALE IN FEET
DOWNSTREAM POOL BOTTOM
TO DIVERSION CHANNEL
8" CONCRETE CAP
SHEETPILE (TYP.)
RIPRAP
FROM SHEYENNE RIVER
WEIR
UPSTREAM POOL BOTTOM
TOP OF WEIR
GRADE DOWNSTREAM OF SHEETPILE
GRADE IN FRONT OF SHEET PILE
7:1 SLOPE
SHEETPILE (TYP.)
EXISTING GROUND
DIVERSION CHANNEL GRADING
STILLING POOL (TYP. 3)
WEIR 1
WEIR 2
WEIR 3
WEIR 4
POOL 1
POOL 2
POOL 3
SEE NOTE 3
RIPRAP SILL
LOW FLOW CHANNEL
SHEYENNE RIVER
ELEVATION
STATION
SCHEMATIC SECTION THROUGH SHEYENNE AQUEDUCT SPILLWAY (N.T.S.)
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.
2. SEE GEOTECHNICAL AND 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: NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
SHEET IDENTIFICATION S-429
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
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
BRAUN ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
US Army Corps of Engineers St. Paul District

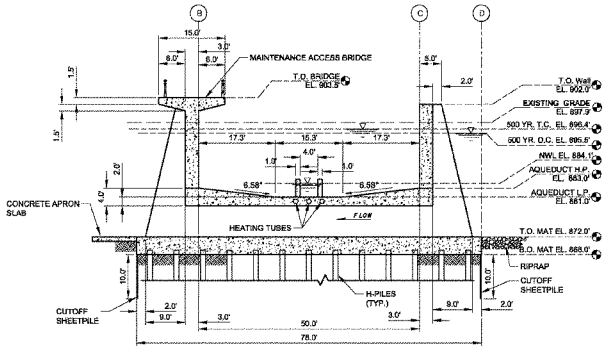

HORIZONTAL COORDINATE SYSTEM: STATE PLANE - ND SOUTH NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
PANEL A
PANEL B
PANEL C
PANEL D
PANEL E
CONCRETE STILLING BASIN
EDGE OF PAD
SILL WALL
NOTCH TO ALLOW STILLING POOL TO DRAIN
CONCRETE STEPS
DIVERSION STA. 667+31
FLOW TO DIVERSION
42.0'
FROM DRAIN 14
CONCRETE APPROACH
EARTHEN VEGETATED CHANNEL
PLAN
DRAIN 14 FOUNDATION PLAN
20 0 20 40
SCALE IN FEET
N
FILL GRADE FOLLOWS TOP OF WALL UNLESS NOTED OTHERWISE
BENCH
CREST OF SPILLWAY
EXISTING GRADE
TOP OF WALL EL. 905.0'
FLOW
7:1 SLOPE
CONCRETE STEPS STEP HEIGHT = 0.7' STEP RUN = 1.5'
SILL WALL 4 FEET HIGH NOTCHED TO DRAIN
7:1 SLOPE
EL. 879.36'
10:1 SLOPE
DIVERSION CHANNEL BOTTOM INVERT EL. 873.3'
LOW FLOW DIVERSION CHANNEL
DRAIN 14 INVERT EL. 885.0'
FILL GRADE BEHIND WALL
20 '
CONCRETE APPROACH
30 '
31.6'
50' CONCRETE PAD '
RIPRAP LOW FLOW CHANNEL
DIVERSION
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. FILL BEHIND WALLS FOLLOWS SPOIL PILES FROM DIVERSION.
5. PILES NOT SHOWN FOR CLARITY.
1 ELEVATION
SOUTH RETAINING WALL
SCALE: N.T.S.
500 YR.
100 YR.
50 YR.
10 YR.
21 TREADS @ 1.5=31.5'
EL. 888.0' CREST
10 YR.
6.3'
21 @ 0.7' =14.7'
1.5'
EL. 873.3'
CONCRETE STILLING BASIN
18.0'
12.27'
10.0'
WATER STOP (CIP)
HP14x73 PILES, TYP AT 6' O.C. SPACING
5.0'
5.0'
9.0'
6.0'
21.5'
10.0'
2 SECTION
CONCRETE STEPS
SCALE: N.T.S.
3.0'
2' to 21.5' (VARIES)
CONCRETE STILLING BASIN (WHERE OCCURRING)
1.5'
WATERSTOP (CIP)
6.5'
4.0'
22.0'
15° BATTER
15° BATTER MAY BE REQUIRED AT SOME LOCATIONS
HP14x73 PILES (6' O.C.) TYP.
3 SPACES @ 9' = 18.0'
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
US Army Corps of Engineers® St. Paul District
NORTH DAKOTA DIVERSION (LLP)
CONCEPT DRAWING
DIVERSION WORKS FEASIBILITY
PHASE
FARGO, ND AND MOORHEAD, MN
FLOOD CONTROL
DRAIN 14 DIVERSION STRUCTURE
PLAN VIEW AND SECTIONS
U.S. ARMY CORPS OF ENGINEERS ST. PAUL DISTRICT ST. PAUL, MINNESOTA
BARR ENGINEERING COMPANY 4700 WEST 77TH STREET MINNEAPOLIS, MN 55435
SHEET IDENTIFICATION
S-430


NORTH DAKOTA DIVERSION (LLP)
PLAN VIEW
MAPLE RIVER AQUEDUCT OVER DIVERSION CHANNEL
SCALE IN FEET
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY FEASIBLE CONCEPTS. FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL, AND HYDRAULIC INFORMATION AND RANKINE IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
TO LOWER RUSH RIVER STRUCTURE
MAPLE RIVER AQUEDUCT ACCESS ROAD
CASS RURAL WATER
RIPRAP
MAPLE RIVER AQUEDUCT ELECTRICAL SERVICE
MAPLE RIVER AQUEDUCT WATER SERVICE
ABANDON OLD CHANNEL PORTION
MAPLE RIVER
PROTECTED SIDE
SH1V EXCAVATION SLOPE
MAPLE RIVER AQUEDUCT MAINTENANCE ACCESS RAMP AND PARKING
APPROXIMATE LIMITS OF TEMPORARY FLOOD PROTECTION FOR CONSTRUCTION
MAPLE RIVER AQUEDUCT, DIVERSION STA. 734+00, SEE DRAWING S-432
SCADA MONITORING POINT (LOCATED AT DIVERSION HEAD IN DIVERSION ACCROSS AQUEDUCT)
DIVERSION CONTAINMENT LEVEES
FROM SHEYENNE AQUEDUCT
LOW FLOW CHANNEL
ACCESS ROAD
REALIGNED CHANNEL
EXISTING ELEVATION CONTOUR (TYP)
PROPOSED ELEVATION 2'VT CONTOUR (TYP)
RELOCATE/OFFSET EXISTING UTILITIES
MAPLE RIVER SPILLWAY
WEIR 3
RIPRAP (TYP.)
WEIR 2
MAPLE RIVER SPILLWAY CONTROL WEIR SEE DRAWING S-435
SCADA MONITORING POINS (LOCATED AT SPILLWAY WEIR AND SPILLWAY)
DEBRIS CONTROL RACKS
UNPROTECTED SIDE
EXISTING PROPERTY LINE TYP
ROCK GRADE CONTROL
MAPLE RIVER
FLOW
HORIZONTAL COORDINATE SYSTEM: NAD 1983 HARN, SOUTH ZONE, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
CONCEPT PLAN DRAWINGS
RED RIVER DIVERSION HYDRAULIC FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN
MAPLE RIVER AQUEDUCT
PLAN VIEW
SHEET DRAWING NUMBER
S-431
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
190 FIFTH STREET EAST
ST. PAUL, MINNESOTA 55101-1638
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
US Army Corps of Engineers St. Paul District

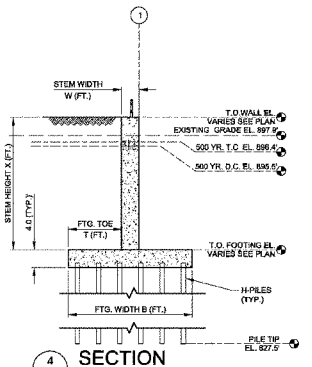

NORTH DAKOTA DIVERSION (LPP)
PLAN
MAPLE AQUEDUCT FOUNDATION PLAN
SCALE IN FEET
FACE OF RETAINING WALL UNDER AQUEDUCT
CENTERLINE PIER (TYP. OF 7)
INSIDE FACE OF WEST AQUEDUCT WALL
INSIDE FACE OF EAST AQUEDUCT WALL
CL WALL
CONCRETE LINED TRIBUTARY CHANNEL FOUNDATION (TYP. OF 4)
AQUEDUCT MAT FOUNDATION (TO EL 282.0/T.O. MAT EL. 883.7
T.O. FOOTING EL. 872.0
T.O. FOOTING EL. 878.0
T.O. FOOTING EL. 880.0
T.O. FOOTING EL. 886.0
PANEL A
PANEL B
PANEL C
PANEL D
PANEL E
PANEL F
PANEL G
PANEL H
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 AND HYDRAULIC DATA WILL BE REQUIRED IN ADDITIONAL PHASES OF DESIGN
3. H-PILES NOT SHOWN FOR CLARITY.
HORIZONTAL COORDINATE SYSTEM: STATE PLANE, NAD 83 (96) NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
CONSEPT OODWIST/ATION DRAWINGS RED WARN DIVERSION HYDRAULIC STRUCTURES FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FLOOD CONTROL - MAPLE RIVER AQUEDUCT FOUNDATION PLAN
PLAN VIEW
S-432


NORTH DAKOTA DIVERSION (LPP)
PLAN
MAPLE AQUEDUCT GENERAL ARRANGEMENT PLAN
SCALE IN FEET
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND GEOTECHNICALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
MATCH PROPOSED GRADE (TYP.)
DIVERSION RETAINING WALL T.O. WALL EL. 903.5'
MAINTENANCE ACCESS BRIDGE T.O. BRIDGE EL. 903.5'
HEATED LOW FLOW CHANNEL
TOP OF TRIBUTARY RETAINING WALL EL. 902.5'
℄ AQUEDUCT OVER DIVERSION
T.O. WALL EL. 902.5'
EDGE OF APRON SLAB
CONCRETE APRON SLAB (80.0'X 250.0')
AQUEDUCT MAT FOUNDATION (78.0'X 248.0')
DIVERSION CHANNEL (T.C.)
℄ DIVERSION UNDER AQUEDUCT
FACE OF WALL
CL OF WALL
TRIBUTARY CHANNEL (T.C.)
R 209.0'
INSIDE FACE OF WEST AQUEDUCT WALL
INSIDE FACE OF EAST AQUEDUCT WALL
CL OF WALL
364.0'
50.0'
14.0'
205.0'
250.0'
258.0'
207.0'
203.9'
203.0'
84.0'
48.0'
15.0'
1.08'
#071
#072
S-433
S-434
S-435
HORIZONTAL COORDINATE SYSTEM: NAD 1983 US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988 US SURVEY FEET
SHEET IDENTIFICATION
S-433
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
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
DESIGNED BY
DRAWN BY
CHECKED BY
APPROVED BY
DATE
DESCRIPTION
MARK
DATE
APPVD
DESCRIPTION
MARK
DATE
APPVD
DESCRIPTION


NORTH DAKOTA DIVERSION (LPP)
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION PERIMETER STRUCTURES
FLOOD CONTROL - MAPLE RIVER
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN
SHEET IDENTIFICATION
S-434
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
HORIZONTAL COORDINATE SYSTEM:
NORTH DAKOTA STATE PLANE
NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY 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.
T.O. WALL EL. 902.0'
FINAL GRADE
RAILING
PANEL A
PANEL B
PANEL C
PANEL D
T.O. FOOTING EL. 896.0'
T.O. FOOTING EL. 884.0'
T.O. FOOTING EL. 878.0'
T.O. FOOTING EL. 872.0'
4' THICK FOOTING (TYP.)
18.0'
24.0'
30.0'
12'
1 SECTION
MAPLE RIVER DIVERSION
UPSTREAM RETAINING WALL
SCALE IN FEET
NOTES:
1. H-PILES NOT SHOWN FOR CLARITY.
FACE OF WALL
EXISTING GRADE EL. 918.0'
T.O. BRIDGE EL. 905.0'
T.O. WALL EL. 902.0'
500 YR. T.C. EL. 896.4'
500 YR. D.C. EL. 895.5'
T.O. AQUEDUCT H.P. EL. 883.0'
T.O. AQUEDUCT L.P. EL. 881.0'
T.O. MAT EL. 872.0'
B.O. MAT EL. 869.0'
AQUEDUCT WALLS
H-PILES (TYP.)
LOW FLOW CHANNEL
T.C. TRIBUTARY CHANNEL
D.C. DIVERSION CHANNEL
282.0'
30.0' (TYP.)
35.0'
27.0' (TYP.)
33.5' (TYP.)
3.0' (TYP.)
4.0' (TYP.)
2 SECTION
MAPLE RIVER AQUEDUCT STRUCTURE
SCALE IN FEET



3 SECTION
S-432, S-433
MAPLE RIVER AQUEDUCT
10 0 10 20
SCALE IN FEET



4 SECTION
S-432, S-433
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	932,483
WALLS	1,298	310,733
DECK ELEVATED SLAB	1,932	336,022
PIERS	531	137,333
TOTAL	7,542	1,816,571

PILES				
NO.	NO. ROWS	NO. COLUMNS	LENGTH (FT.)	TOTAL (FT.)
455	35	13	42	19,110

QUANTITIES (RETAINING WALLS)

PANEL	STEM W (FT)	STEM HEIGHT X (FT)	FTG. WIDTH B (FT)	FTG. TOE T (FT)	CONCRETE VOLUMES FTG. CY	CONCRETE VOLUMES STEM CY	STEEL (LBS.) FTG.	STEEL (LBS.) STEM	PILES NO.	PILES NO. ROWS	PILES LENGTH FT	PILES TOTAL FT
A	4	11.94	10	3	126	143	21,897	26,400	24	12	60	1,440
B	4	17.94	16	6	191	214	35,035	39,698	36	12	54	1,944
C	4	23.94	22	9	253	286	48,173	52,933	52	13	48	2,496
D	4	29.94	28	12	334	418	61,311	77,316	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

NORTH DAKOTA DIVERSION (LPP)

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

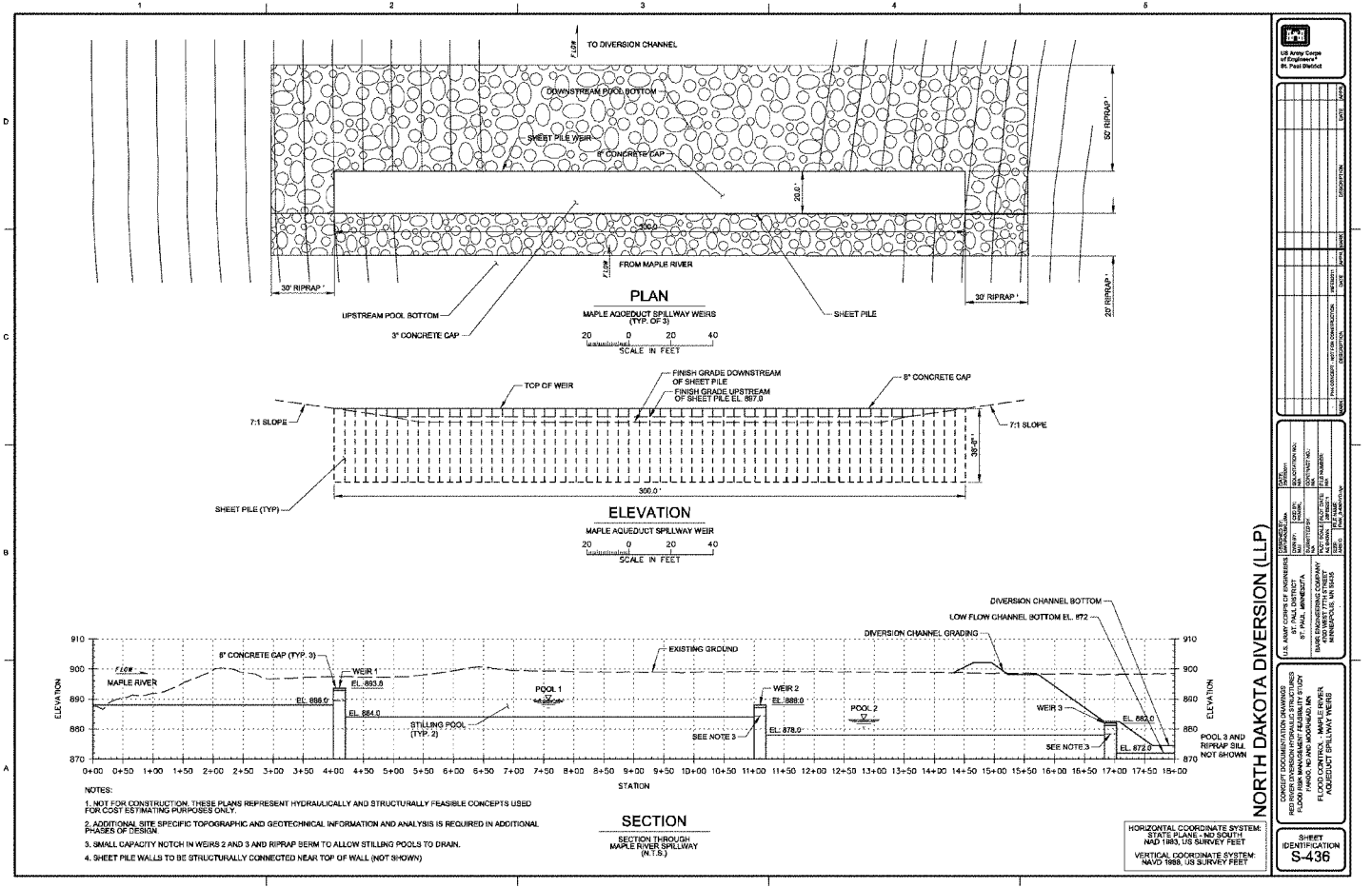
FLOOD CONTROL - MAPLE RIVER
AQUEDUCT SECTIONS

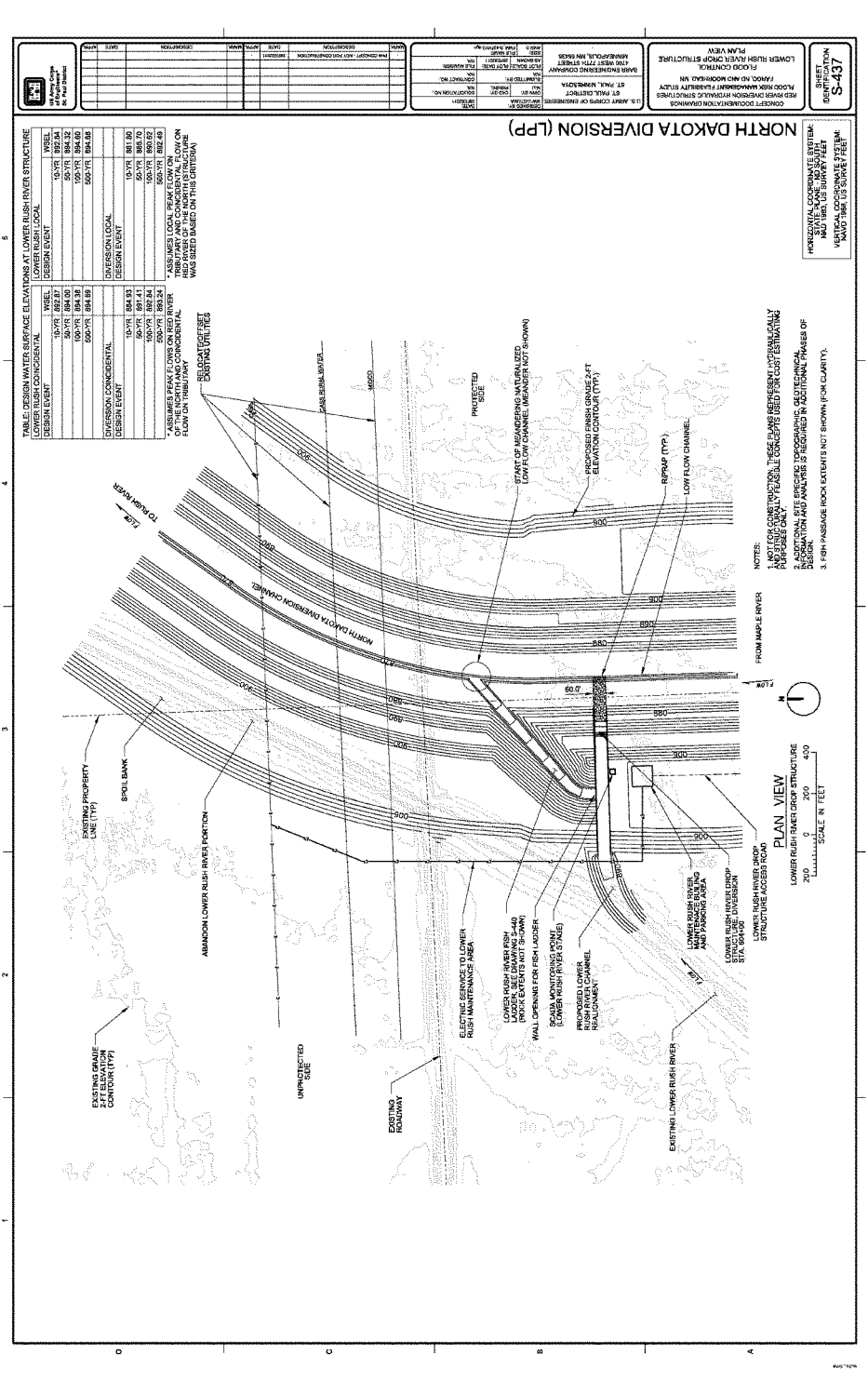
SHEET IDENTIFICATION
S-435

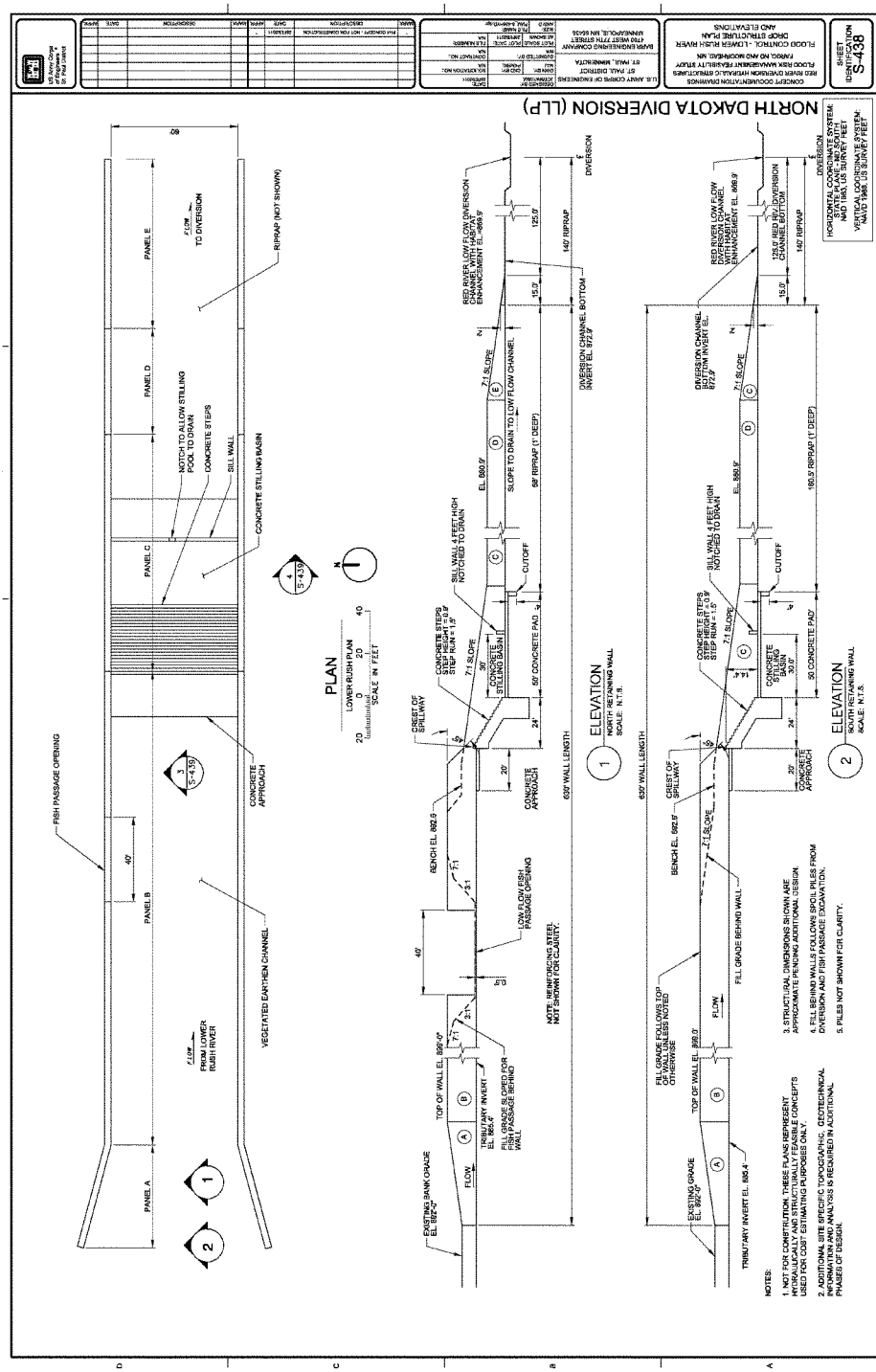
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA

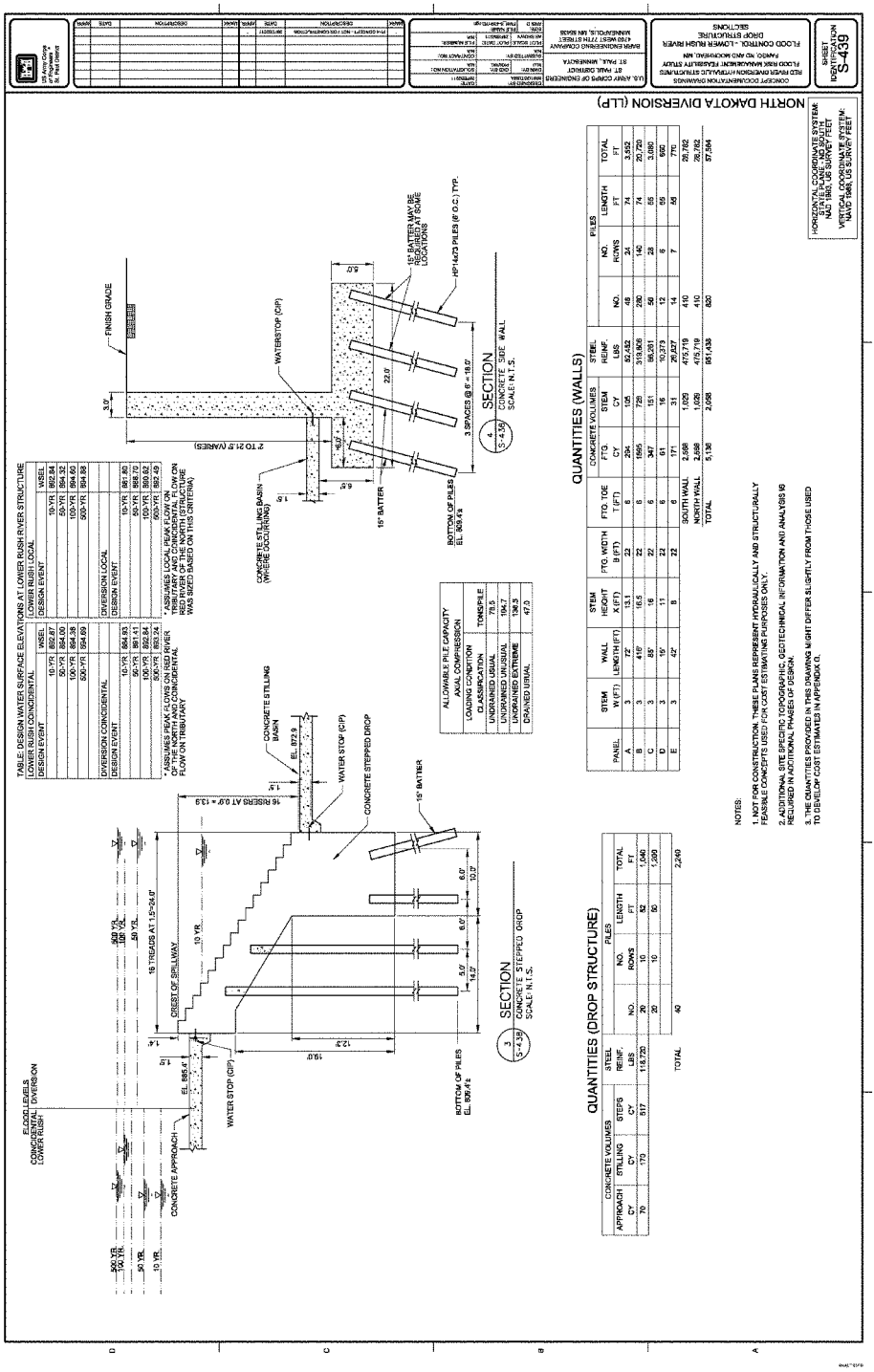
BARR ENGINEERING COMPANY
4300 MARKETPOINTE DRIVE
MINNEAPOLIS, MN 55435

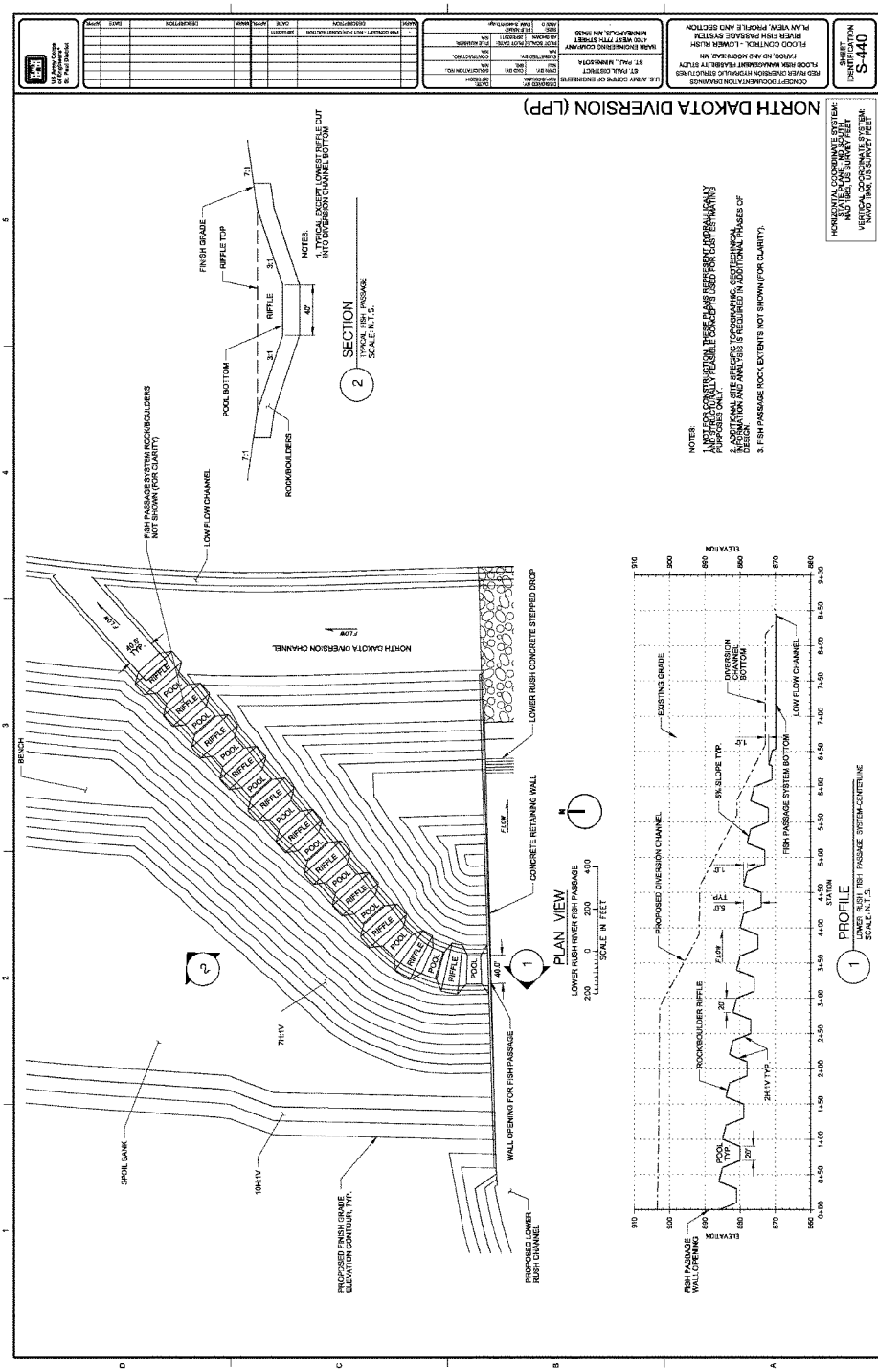
675


TO DIVERSION CHANNEL
FLOW
DOWNSTREAM POOL BOTTOM
SHEET PILE WEIR
8" CONCRETE CAP
25.0'
300.0'
52' RIPRAP *
20' RIPRAP *
30' RIPRAP *
30' RIPRAP *
FLOW
FROM MAPLE RIVER
PLAN
MAPLE AQUEDUCT SPILLWAY WEIRS
(TYP. OF 3)
20 0 20 40
SCALE IN FEET
UPSTREAM POOL BOTTOM
3" CONCRETE CAP
SHEET PILE
TOP OF WEIR
FINISH GRADE DOWNSTREAM OF SHEET PILE
FINISH GRADE UPSTREAM OF SHEET PILE EL. 867.0
8" CONCRETE CAP
7:1 SLOPE
7:1 SLOPE
35'-0"
300.0'
SHEET PILE (TYP)
ELEVATION
MAPLE AQUEDUCT SPILLWAY WEIR
20 0 20 40
SCALE IN FEET
DIVERSION CHANNEL BOTTOM
LOW FLOW CHANNEL BOTTOM EL. 872
DIVERSION CHANNEL GRADING
EXISTING GROUND
8" CONCRETE CAP (TYP. 3)
FLOW
MAPLE RIVER
WEIR 1
EL. 893.5
EL. 886.0
EL. 884.0
POOL 1
STILLING POOL
(TYP. 2)
WEIR 2
EL. 888.0
EL. 878.0*
SEE NOTE 3
POOL 2
WEIR 3
EL. 882.0
SEE NOTE 3
EL. 872.0
POOL 3 AND RIPRAP SILL NOT SHOWN
ELEVATION
ELEVATION
910
900
890
880
870
0+00 0+50 1+00 1+50 2+00 2+50 3+00 3+50 4+00 4+50 5+00 5+50 6+00 6+50 7+00 7+50 8+00 8+50 9+00 9+50 10+00 10+50 11+00 11+50 12+00 12+50 13+00 13+50 14+00 14+50 15+00 15+50 16+00 16+50 17+00 17+50 18+00
STATION
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
NORTH DAKOTA DIVERSION (LLP)
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT, FARGO-MOORHEAD AREA
FARGO, ND AND MOORHEAD, MN
FLOOD CONTROL - MAPLE RIVER
AQUEDUCT SPILLWAY WEIRS
SHEET IDENTIFICATION
S-436
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
BAKER ENGINEERING COMPANY
ARCHITECTS ENGINEERS
MINNEAPOLIS, MN 55435
PROJECT NO.
DRAWING NO.
FILE NUMBER
SHEET NAME
PLAN AND SECTION
DESIGNED BY
DRAWN BY
CHECKED BY
DATE
SUBMITTED
APPROVED
DESCRIPTION
DATE
APPR.
CHECK
PRELIMINARY - NOT FOR CONSTRUCTION
DESIGNED
DRAWN
CHECKED
SUBMITTED
APPROVED
DATE
SUBMITTED
APPROVED
DESCRIPTION
DATE
APPR.
CHECK


NORTH DAKOTA DIVERSION (LPP)
PLAN VIEW
LOWER RUSH RIVER DROP STRUCTURE
SCALE IN FEET
TABLE: DESIGN WATER SURFACE ELEVATIONS AT LOWER RUSH RIVER STRUCTURE
LOWER RUSH COINCIDENTAL
LOWER RUSH LOCAL
DIVERSION COINCIDENTAL
DIVERSION LOCAL
* ASSUMES PEAK FLOWS ON RED RIVER AT NORTH AND COINCIDENTAL FLOW ON RUSH RIVER (NOT THE LOCAL PEAK FLOW WAS SIZED BASED ON THIS CRITERIA)
* ASSUMES LOCAL PEAK FLOW ON RED RIVER AT HARWOOD AND COINCIDENTAL FLOW ON RUSH RIVER (THIS CRITERIA)
BELOW/OFFSET EXISTING UTILITIES
TO RUSH RIVER
FLOW
NORTH DAKOTA DIVERSION CHANNEL
ABANDON LOWER RUSH RIVER PORTION
EXISTING PROPERTY LINE (TYP)
SPOIL BANK
EXISTING GRADE 2-FT ELEVATION CONTOUR (TYP)
UNPROTECTED SIDE
EXISTING ROADWAY
ELECTRIC SERVICE TO LOWER RUSH MAINTENANCE AREA
LOWER RUSH RIVER FISH LADDER (SEE SHEET S-440 FOR ADDITIONAL DETAILS) (ROCK EXTENTS NOT SHOWN)
WALL OPENING FOR FISH LADDER
SCADA MONITORING POINT (LOWER RUSH RIVER STAGE)
PROPOSED LOWER RUSH RIVER CHANNEL REALIGNMENT
EXISTING LOWER RUSH RIVER
LOWER RUSH RIVER MAINTENANCE BUILDING AND PARKING AREA
LOWER RUSH RIVER DROP STRUCTURE DIVERSION STA. 904+00
LOWER RUSH RIVER DROP STRUCTURE ACCESS ROAD
START OF MEANDERING NATURALIZED LOW FLOW CHANNEL (MEANDER NOT SHOWN)
PROPOSED FINISH GRADE 2-FT ELEVATION CONTOUR (TYP.)
RIPRAP (TYP.)
LOW FLOW CHANNEL
PROTECTED SIDE
FROM MAPLE RIVER
NOTES:
1. NOT FOR CONSTRUCTION - THESE PLANS REPRESENT HYDRAULICALLY 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).
U.S. ARMY CORPS OF ENGINEERS
ST PAUL DISTRICT
ST. PAUL, MINNESOTA
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
CONCEPT DESIGN FOR REDISTRIBUTION OF FLOWS FROM THE RED RIVER BASIN DURING EXTREME FLOOD EVENTS
FLOOD CONTROL
LOWER RUSH AREA DROP STRUCTURE
PLAN VIEW
HORIZONTAL COORDINATE SYSTEM: NAD 1983 HARN RESTRAINED MN SOUTH ZONE
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
SHEET DESIGNATION
S-437


NORTH DAKOTA DIVERSION (LLP)
PLAN
ELEVATION
ELEVATION
FISH PASSAGE OPENING
CONCRETE APPROACH
VEGETATED EARTHEN CHANNEL
RIPRAP (NOT SHOWN)
CONCRETE STILLING BASIN
SILL WALL
CONCRETE STEPS
NOTCH TO ALLOW STILLING POOL TO DRAIN
PANEL A
PANEL B
PANEL C
PANEL D
PANEL E
FLOW FROM LOWER RUSH RIVER
FLOW TO DIVERSION
TOP OF WALL EL. 898'-0"
TRIBUTARY INVERT EL. 886.4'
EXISTING BANK GRADE EL. 902'-0"
BENCH EL. 892.8
CREST OF SPILLWAY
CONCRETE APPROACH
LOW FLOW FISH PASSAGE OPENING
NOTE: REINFORCING STEEL NOT SHOWN FOR CLARITY.
639' WALL LENGTH
FILL GRADE FOLLOWS TOP OF WALL UNLESS NOTED OTHERWISE
FILL GRADE BEHIND WALL
SILL WALL 4 FEET HIGH NOTCHED TO DRAIN
CONCRETE STEPS STEP HEIGHT 1.5' STEP RUN = 1.5'
CONCRETE STILLING BASIN
CUTOFF
50' CONCRETE PAD
24'
20'
7:1 SLOPE
3:1
2:1
EL. 880.9'
SLOPE TO DRAIN TO LOW FLOW CHANNEL
68' RIPRAP (1' DEEP)
180.5' RIPRAP (1' DEEP)
DIVERSION CHANNEL BOTTOM INVERT EL. 872.9'
DIVERSION CHANNEL BOTTOM WIDTH 30.0'
RED RIVER LOW FLOW DIVERSION CHANNEL ENHANCEMENT EL. 888.9'
125.9'
15.0'
140' RIPRAP
DIVERSION
12'H X 8'W RCP DIVERSION CULVERT
NORTH RETAINING WALL
SCALE: N.T.S.
SCALE IN FEET
0 20 40
LOWER RUSH PLAN
2D
1
2
3
4
S-435
S-439
N
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT INTRA-LOCALLY 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 SHOW PILES FROM DIVERSION AND FISH PASSAGE EXCAVATION.
5. PILES NOT SHOWN FOR CLARITY.
CONCEPT COORDINATION 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
HORIZONTAL COORDINATE SYSTEM: NORTH DAKOTA STATE PLANE NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
MINNEAPOLIS, MN 55415
HDR ENGINEERING, INC.
8404 INDIANA AVENUE, SUITE 201
ST. PAUL, MINNESOTA 55114
BRAUN INTERTEC CORPORATION
11001 HAMPSHIRE AVENUE SOUTH
MINNEAPOLIS, MN 55438
DESIGNED BY
DRAWN BY
CHECKED BY
DATE
SUBMITTED
PROJECT MANAGER
E.J. HAMMERS
P.E. NUMBER
DESIGNED BY
DRAWN BY
CHECKED BY
DATE
SUBMITTED
PROJECT MANAGER
K.E. BROWN
P.E. NUMBER
DESIGNED BY
DRAWN BY
CHECKED BY
DATE
SUBMITTED
PROJECT MANAGER
M.E. MILLER
P.E. NUMBER
DESIGNED BY
DRAWN BY
CHECKED BY
DATE
SUBMITTED
PROJECT MANAGER
K.E. BROWN


NORTH DAKOTA DIVERSION (LLP)
QUANTITIES (WALLS)
QUANTITIES (DROP STRUCTURE)
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. THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER SLIGHTLY FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX E.
FLOOD CONTROL - LOWER RUSH RIVER
DROP STRUCTURE
SECTIONS
S-439


NORTH DAKOTA DIVERSION (LPP)
PLAN VIEW
LOWER RUSH RIVER FISH PASSAGE
SCALE IN FEET
PROFILE
LOWER RUSH FISH PASSAGE SYSTEM-CENTERLINE
SCALE: N.T.S.
SECTION
TYPICAL FISH PASSAGE
SCALE: N.T.S.
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING AND PERMITTING 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: NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
SHEET IDENTIFICATION
S-440
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN
RIVER FISH PASSAGE SECTION
PLAN VIEW, PROFILE AND SECTION
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
MAIN ENGINEERING COMPANY
4100 WEST 77TH STREET
MINNEAPOLIS, MN 55435
NORTH DAKOTA DIVERSION CHANNEL
FLOW
FISH PASSAGE SYSTEM ROCK/BOULDERS NOT SHOWN (FOR CLARITY)
LOW FLOW CHANNEL
BENCH
POOL
RIFFLE
7H:1V
10H:1V
SPOIL BANK
PROPOSED FINISH GRADE ELEVATION CONTOUR, TYP.
PROPOSED LOWER RUSH CHANNEL
WALL OPENING FOR FISH PASSAGE
CONCRETE RETAINING WALL
LOWER RUSH CONCRETE STEPPED DROP
FINISH GRADE
RIFFLE TOP
POOL BOTTOM
ROCK/BOULDERS
NOTES:
1. FILL EXCEPT LOWEST RIFFLE CUT INTO DIVERSION CHANNEL BOTTOM
EXISTING GRADE
PROPOSED DIVERSION CHANNEL
DIVERSION CHANNEL BOTTOM
6% SLOPE TYP.
FISH PASSAGE SYSTEM BOTTOM
LOW FLOW CHANNEL
ROCK/BOULDER RIFFLE
POOL TYP.
2H:1V TYP.
FISH PASSAGE WALL OPENING
ELEVATION
STATION

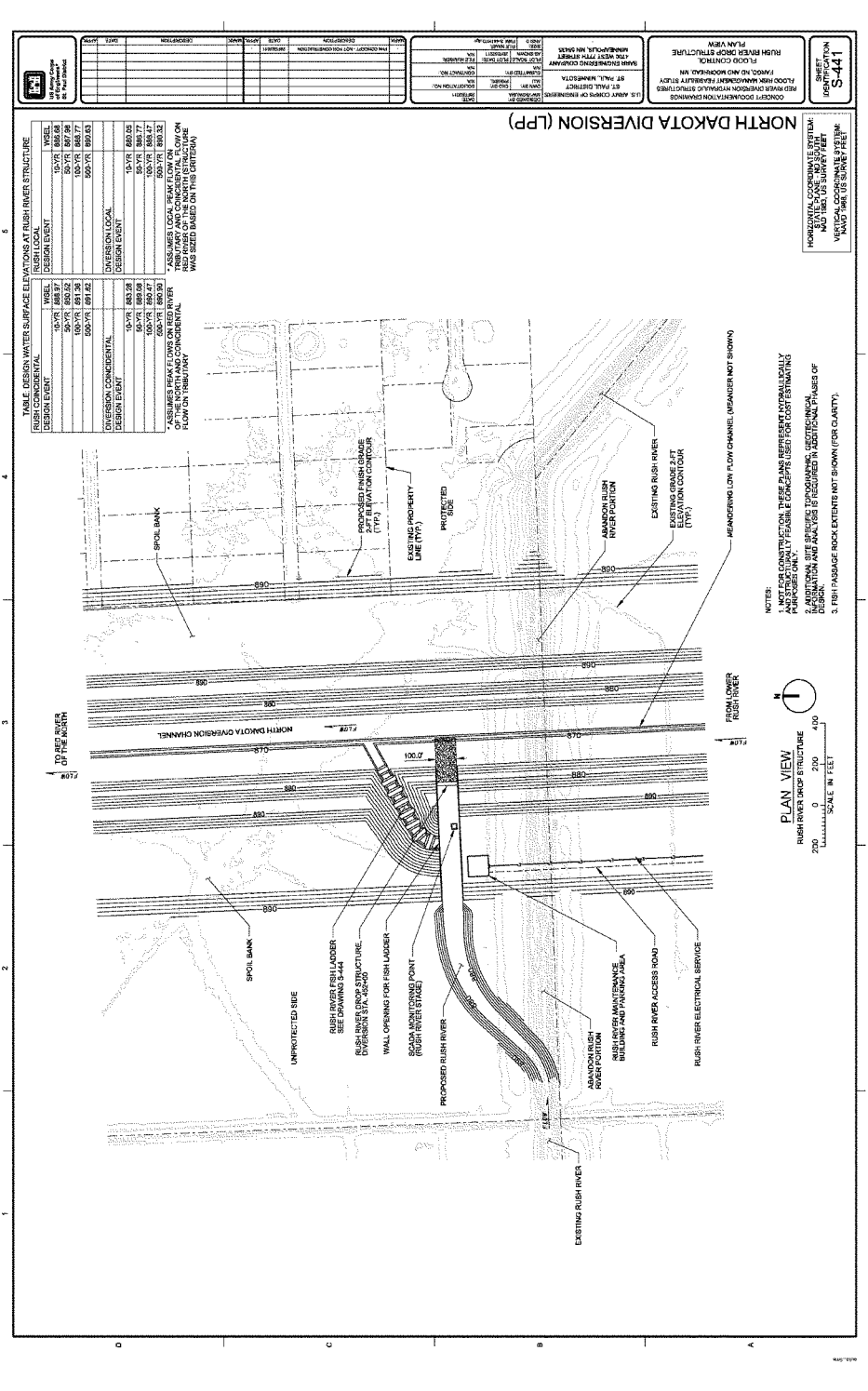
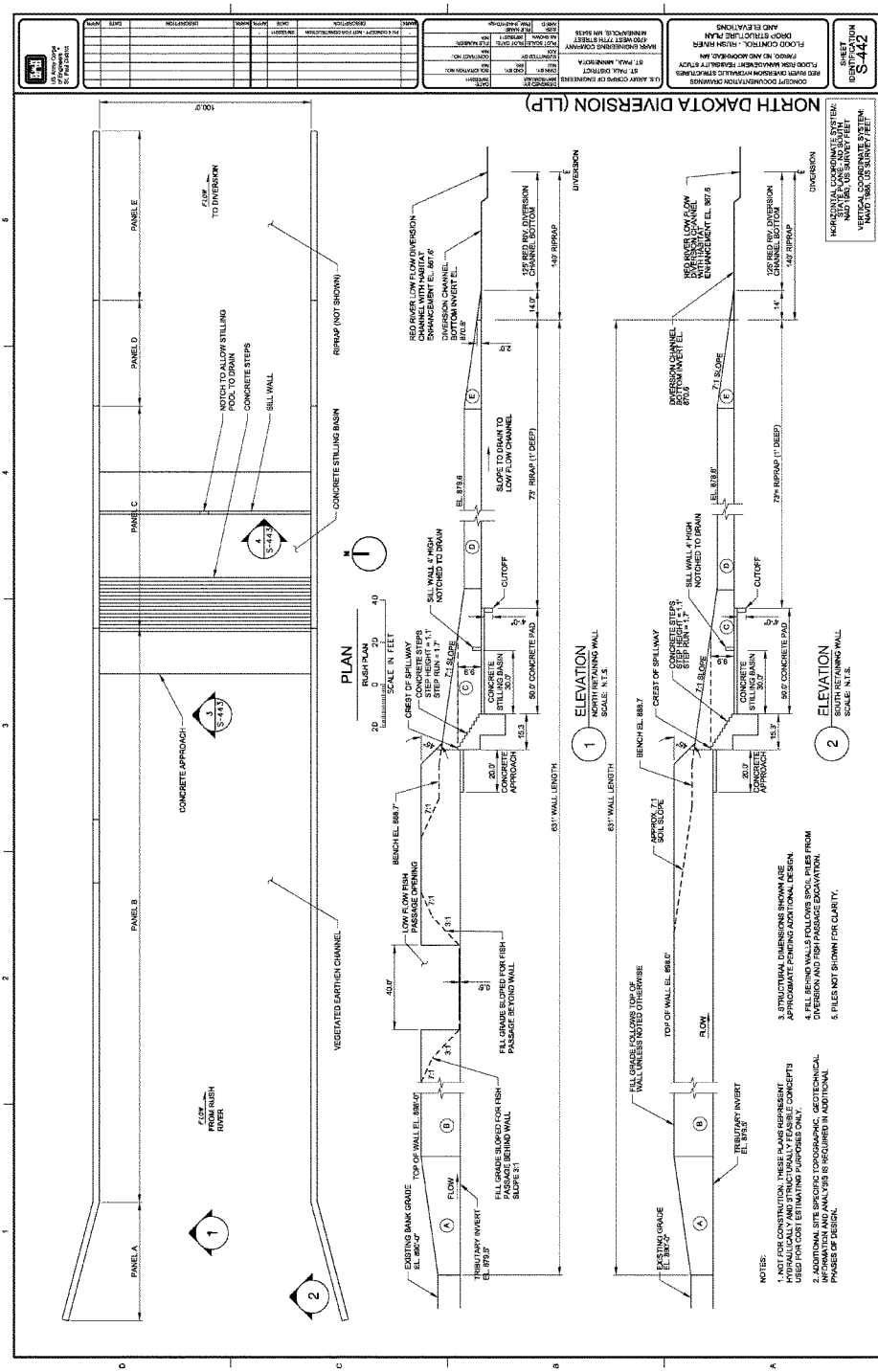
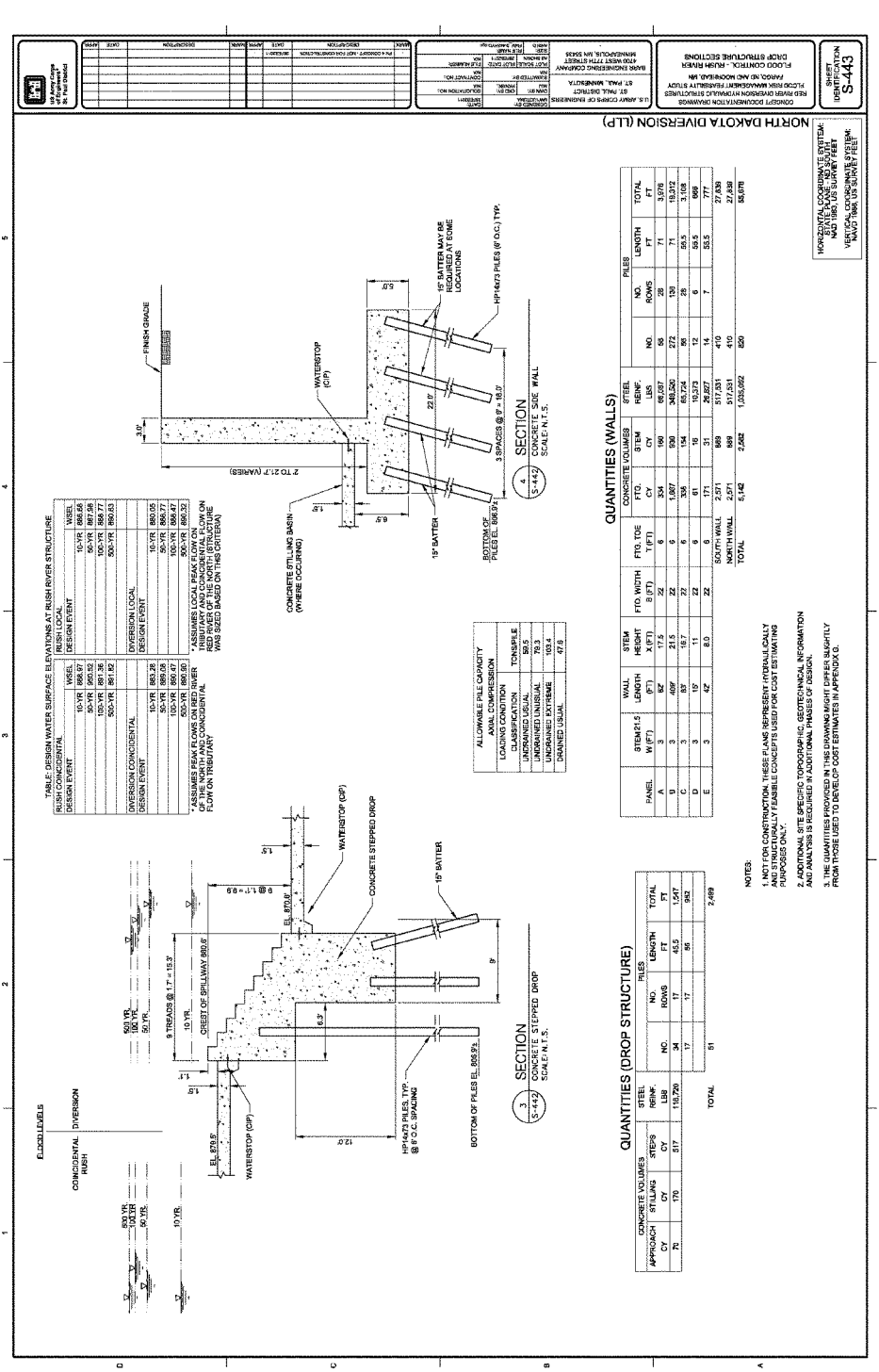
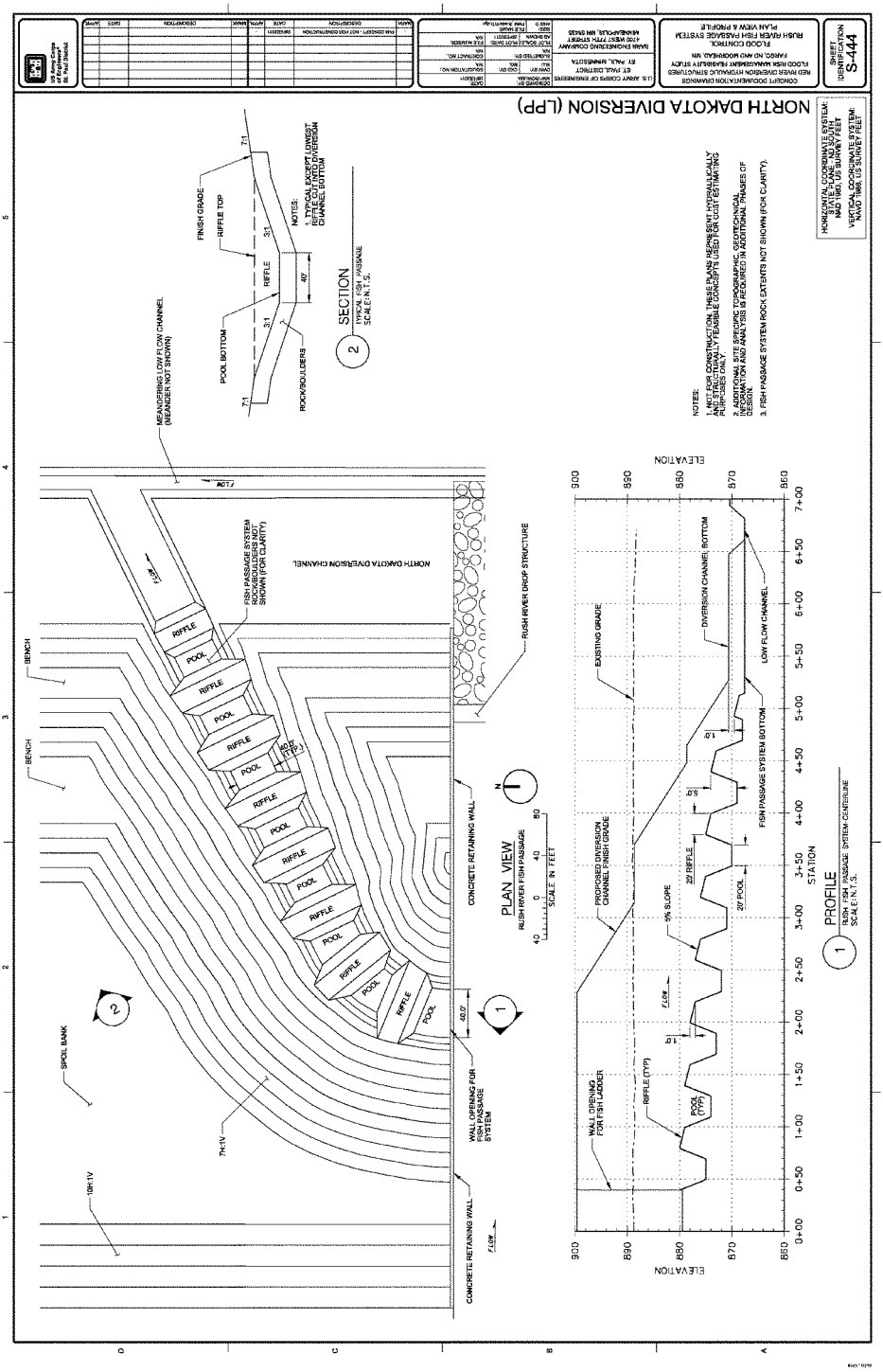
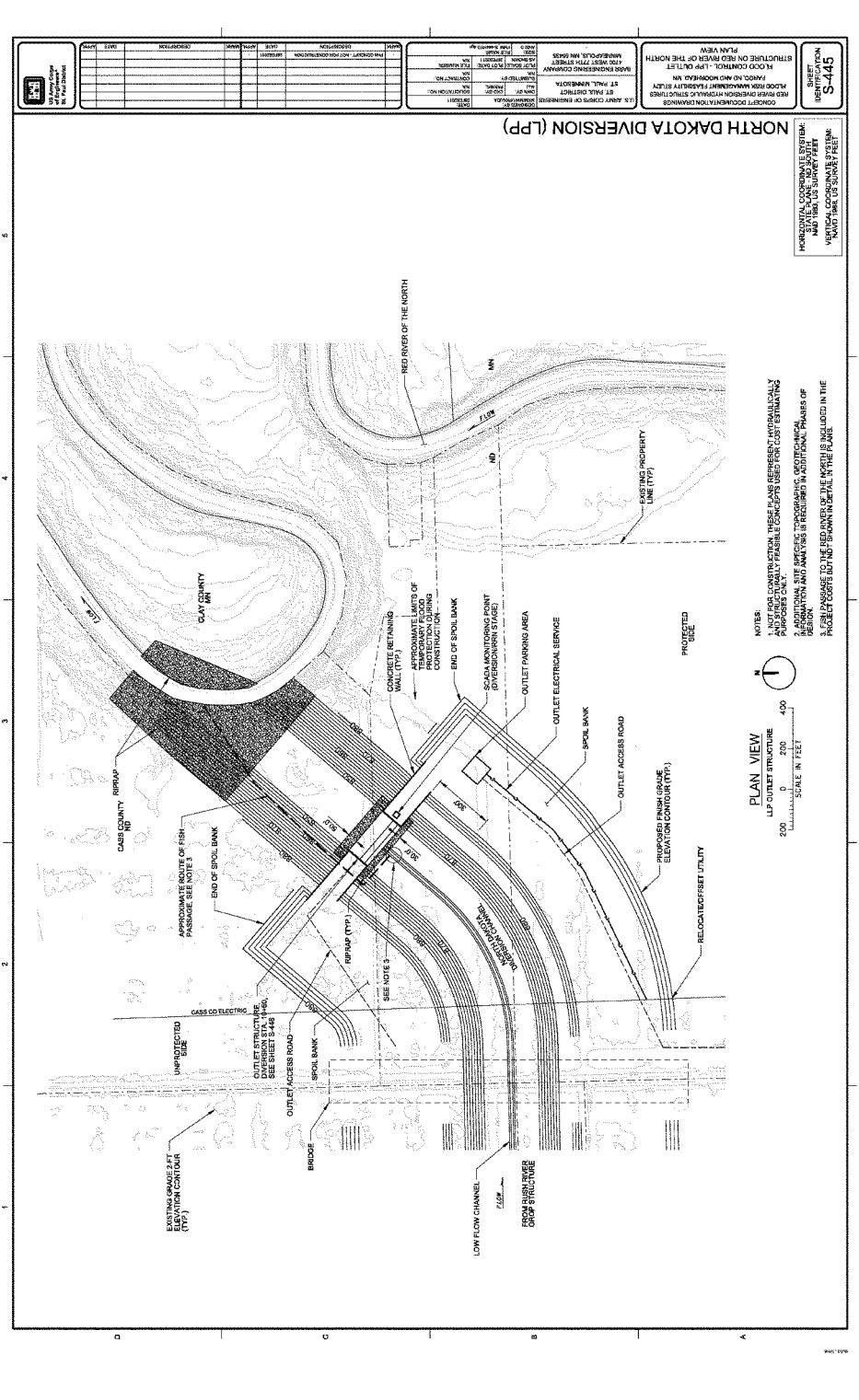


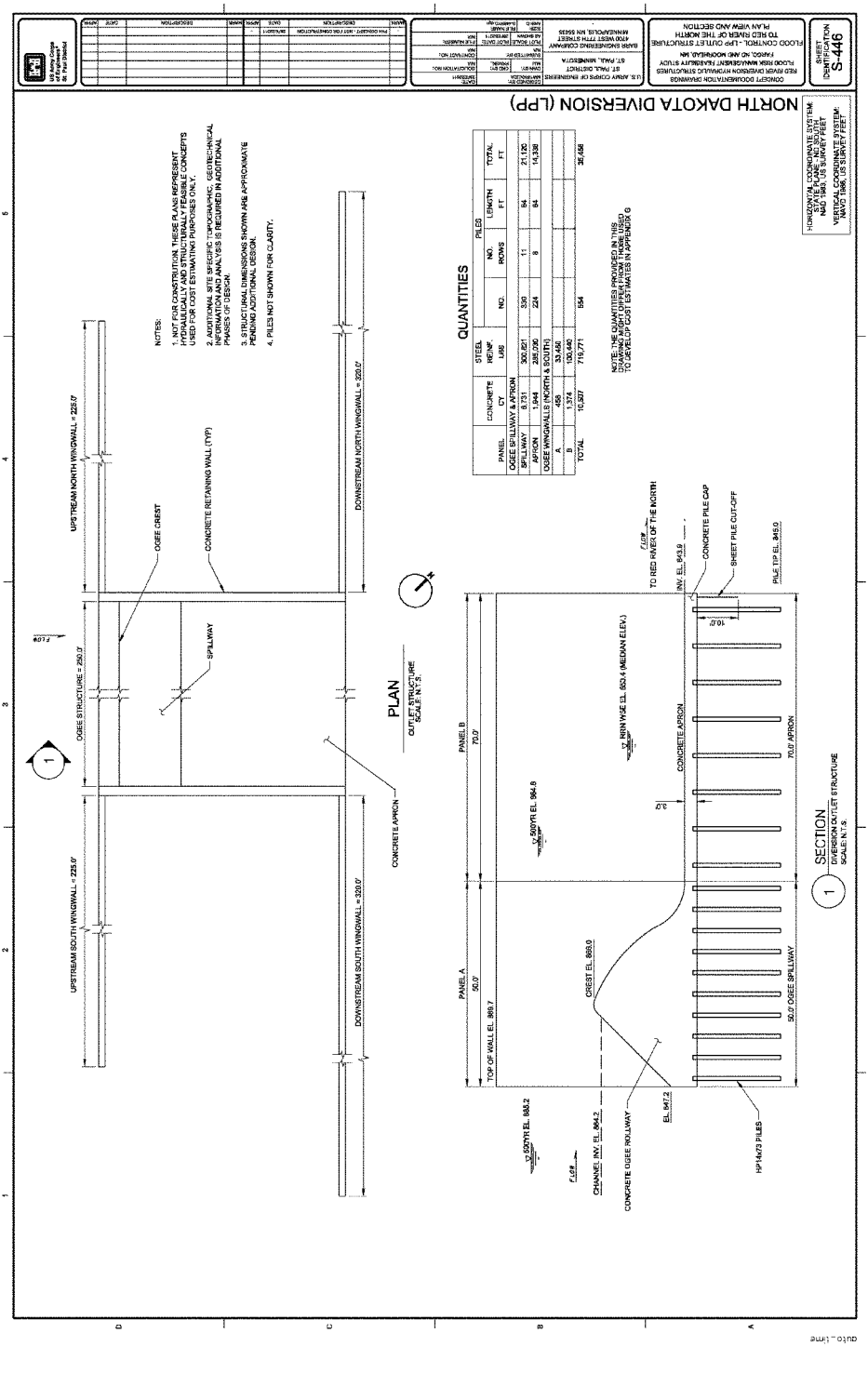
TABLE: DESIGN WATER SURFACE ELEVATIONS AT RUSH RIVER STRUCTURE
RUSH COINCIDENTAL
DESIGN EVENT
WSEL
10-YR
885.37
50-YR
890.22
100-YR
891.36
500-YR
891.82
DIVERSION COINCIDENTAL
DESIGN EVENT
10-YR
883.28
50-YR
889.99
100-YR
890.97
500-YR
890.90
* ASSUMES PEAK FLOWS ON RED RIVER FROM NORTH AND COINCIDENTAL FLOW ON RUSH RIVER
RUSH LOCAL
DESIGN EVENT
WSEL
10-YR
885.68
50-YR
897.98
100-YR
898.77
500-YR
899.63
DIVERSION LOCAL
DESIGN EVENT
10-YR
860.05
50-YR
869.77
100-YR
870.47
500-YR
890.32
* ASSUMES LOCAL PEAK FLOW ON TRIB #1 AND CONCIDENTAL FLOW ON RED RIVER WAS SIZED BASED ON THIS CRITERIA)
NORTH DAKOTA DIVERSION (LPP)
PLAN VIEW
RUSH RIVER DROP STRUCTURE
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 INVESTIGATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
3. FISH PASSAGE ROCK EXTENTS NOT SHOWN (FOR CLARITY).
TO RED RIVER OF THE NORTH
NORTH DAKOTA DIVERSION CHANNEL
FROM LOWER RUSH RIVER
MEANDERING LOW FLOW CHANNEL (MSANDER NOT SHOWN)
EXISTING RUSH RIVER
ABANDON RUSH RIVER PORTION
EXISTING GRADE AT ELEVATION (FOOT DUR (TYP.)
PROPOSED FINISH GRADE ELEVATION CONTOUR
EXISTING PROPERTY LINE (TYP.)
PROTECTED SIDE
SPOIL BANK
UNPROTECTED SIDE
RUSH RIVER FISH LADDER SEE DRAWING S-444
RUSH RIVER DROP STRUCTURE DIVERSION STA. 452+00
WALL OPENING FOR FISH LADDER
SCADA MONITORING POINT (RUSH RIVER STAGE)
PROPOSED RUSH RIVER
ABANDON RUSH RIVER PORTION
RUSH RIVER MAINTENANCE BUILDING AND PARKING AREA
RUSH RIVER ACCESS ROAD
RUSH RIVER ELECTRICAL SERVICE
EXISTING RUSH RIVER
CONCEPT PDC/ENGINEERING DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD CONTROL
FARGO, ND AND MOORHEAD, MN
RUSH RIVER DROP STRUCTURE
PLAN VIEW
U.S. ARMY CORPS OF ENGINEERS
ST PAUL DISTRICT
ST. PAUL, MINNESOTA
SHAW ENGINEERING COMPANY
1706 WEST 7TH STREET
MINNEAPOLIS, MN 55455
HORIZONTAL COORDINATE SYSTEM:
NORTH DAKOTA STATE PLANE
NAD 83 1996 US SURVEY FEET
VERTICAL COORDINATE SYSTEM:
NAVD 1988 US SURVEY FEET
SHEET
S-441

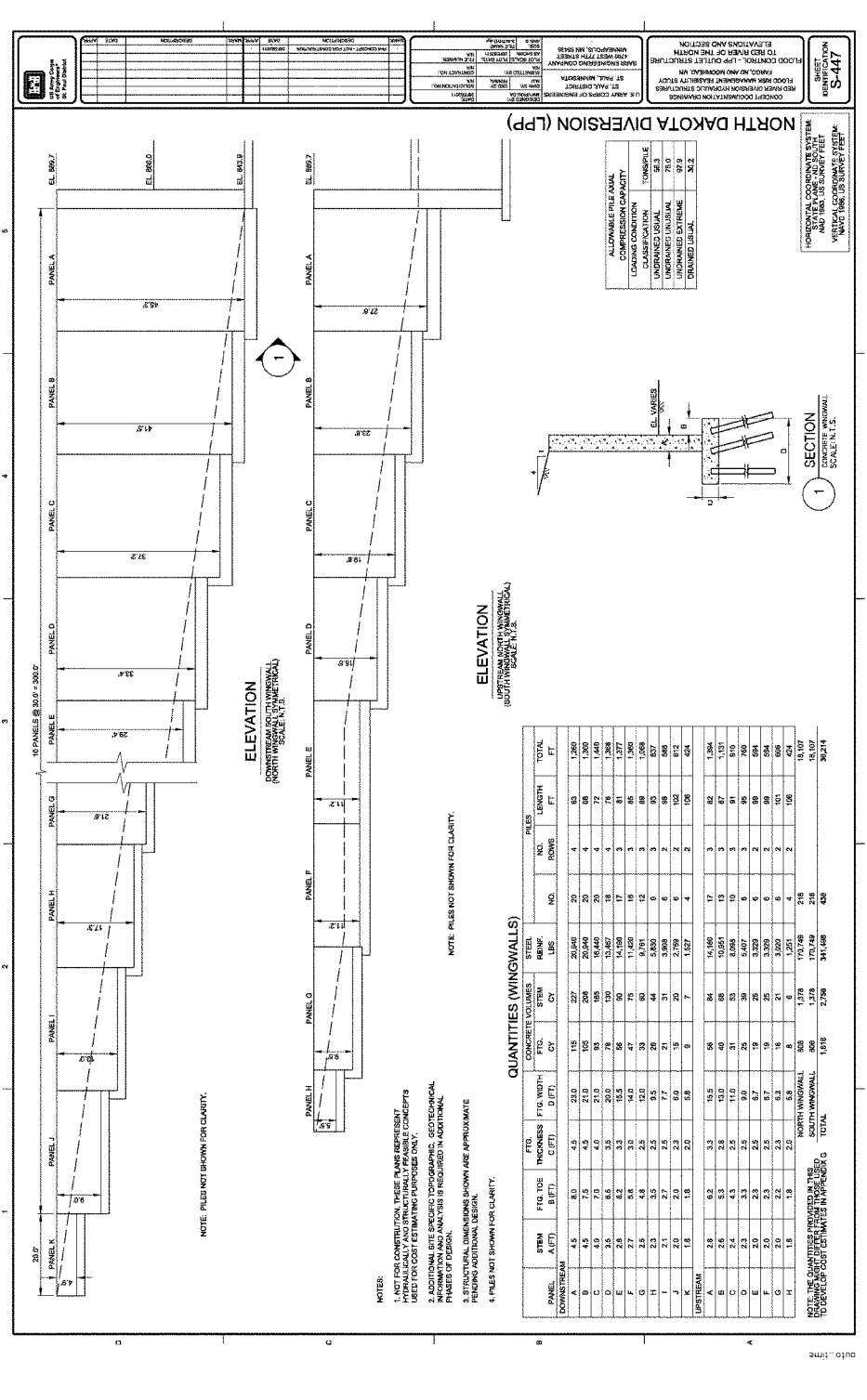

NORTH DAKOTA DIVERSION (LLP)
PLAN
ELEVATION
NORTH RETAINING WALL
SOUTH RETAINING WALL
CONCRETE STILLING BASIN
CONCRETE APPROACH
CONCRETE STEPS
SILL WALL
CUTOFF
RIPRAP (NOT SHOWN)
VEGETATED EARTHEN CHANNEL
LOW FLOW FISH PASSAGE OPENING
FILL GRADE SLOPED FOR FISH PASSAGE BEYOND WALL
FILL GRADE SLOPED FOR FISH PASSAGE BEHIND WALL
FILL GRADE FOLLOWS TOP OF WALL UNLESS NOTED OTHERWISE
CREST OF SPILLWAY
BENCH EL. 888.7'
TOP OF WALL EL. 896.0'
EXISTING BANK GRADE EL. 890.0'
EXISTING GRADE EL. 880.0'
TRIBUTARY INVERT EL. 879.5'
TRIBUTARY INVERT EL. 876.2'
631' WALL LENGTH
50.0' CONCRETE PAD
20.0' CONCRETE APPROACH
15.3'
40.0'
PANEL A
PANEL B
PANEL C
PANEL D
PANEL E
100.0'
FLOW FROM RED RIVER
FLOW TO DIVERSION
NOTCH TO ALLOW STILLING POOL TO DRAIN
RED RIVER LOW FLOW DIVERSION CHANNEL WITH HABITAT ENHANCEMENT EL. 887.6'
DIVERSION CHANNEL BOTTOM INVERT EL. 885.6
SLOPE TO DRAIN TO LOW FLOW CHANNEL
73' RIPRAP (1' DEEP)
125' RED RIV. DIVERSION CHANNEL BOTTOM
14.0'
14'
14.0'
147' RIPRAP
DIVERSION
DIVERSION CHANNEL BOTTOM INVERT EL. 870.6
EL. 878.6'
7:1 SLOPE
APPROX. 7:1 SOIL SLOPE
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS SUBJECT FOR FURTHER REFINEMENT DURING PED.
2. ADDITIONAL SITE SPECIFIC TOPOGRAPHIC, GEOTECHNICAL AND ENVIRONMENTAL DATA 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 AND FISH PASSAGE EXCAVATION.
5. PILES NOT SHOWN FOR CLARITY.
US Army Corps of Engineers St. Paul District
DESCRIPTION
DATE
APVD
ST. PAUL DISTRICT
U.S. ARMY CORPS OF ENGINEERS
DESIGNED BY
DRAWN BY
CHECKED BY
SUBMITTED BY
DATE
FILE NAME
CONSTRUCTION OVERSIGHT BY
DATE
CONTRACT NO.
DACW37-1
DATE
APVD
MINNEAPOLIS, MN 56455
MAIN ENGINEERING COMPANY
4700 WEST 77TH STREET
FARGO, ND AND MOORHEAD, MN
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
RED RIVER DIVERSION CHANNEL STRUCTURES
DRAWING NO.
S-442
SHEET IDENTIFICATION
HORIZONTAL COORDINATE SYSTEM: NORTH DAKOTA STATE PLANE NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET

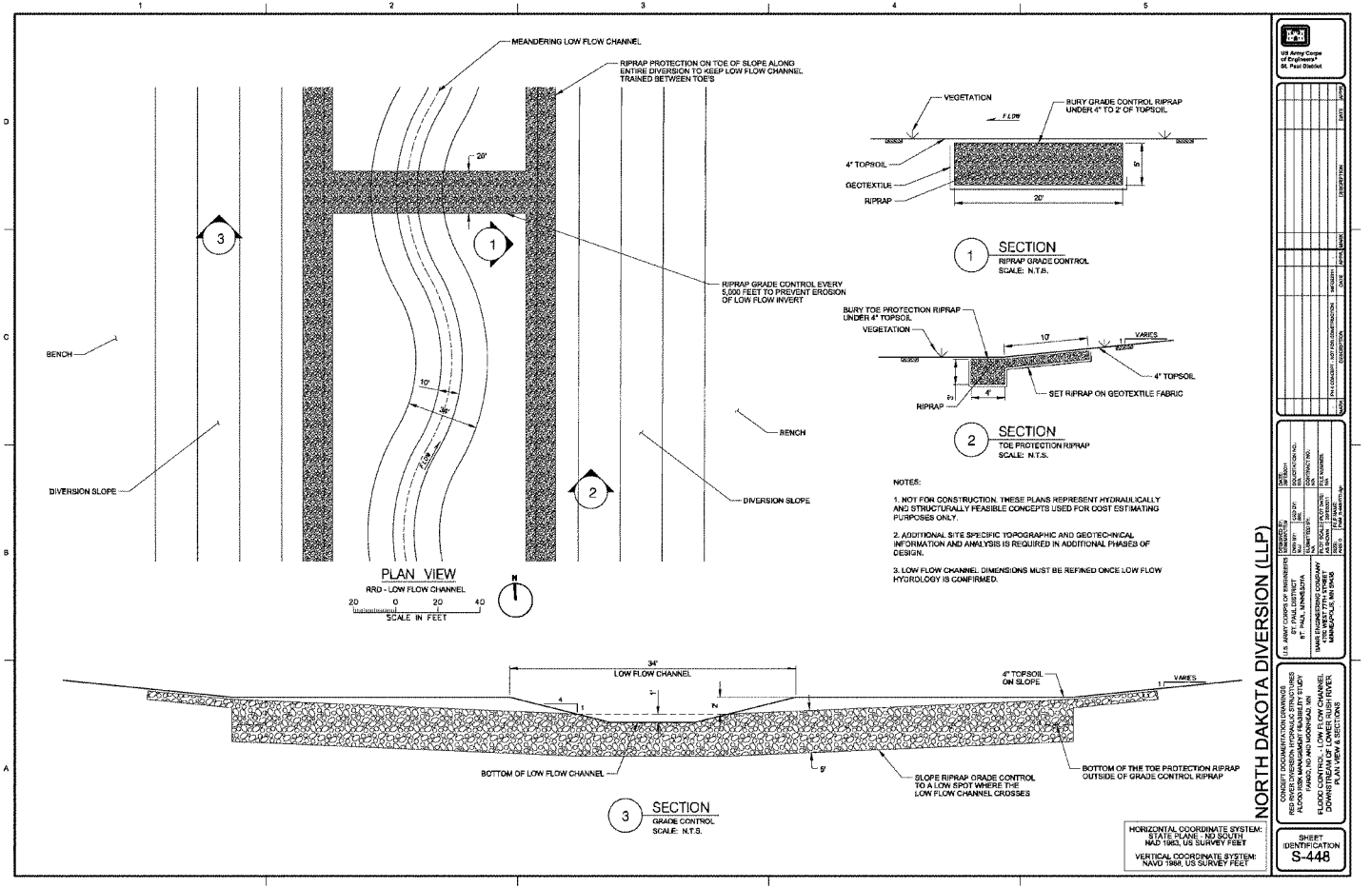

NORTH DAKOTA DIVERSION (LLP)
FLOOD CONTROL - RUSH RIVER
DIVERSION STRUCTURE
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN
BASE ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
ST PAUL DISTRICT
ST PAUL, MINNESOTA
U.S. ARMY CORPS OF ENGINEERS
CONSIDERED DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
S-443
SHEET
DESIGN DOCUMENTATION
HORIZONTAL COORDINATE SYSTEM:
NAD 1983 (2011) US SURVEY FEET
VERTICAL COORDINATE SYSTEM:
NAVD 1988 US SURVEY FEET
FLOOD LEVELS
CONCRETE STEPPED DROP
CONCRETE STILLING BASIN
(WHERE OCCURRING)
WATERSTOP (CIP)
FINISH GRADE
15° BATTER
BOTTOM OF PILES EL. 896.94
HP14x73 PILES @ 6' O.C. SPACING
SECTION
CONCRETE SIDE WALL
SCALE: N.T.S.
QUANTITIES (DROP STRUCTURE)
QUANTITIES (WALLS)
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 FOR ADDITIONAL PHASES OF DESIGN.
3. THE QUANTITIES PROVIDED IN THIS DRAWING MIGHT DIFFER SLIGHTLY FROM THOSE USED TO DEVELOP COST ESTIMATES IN APPENDIX D.

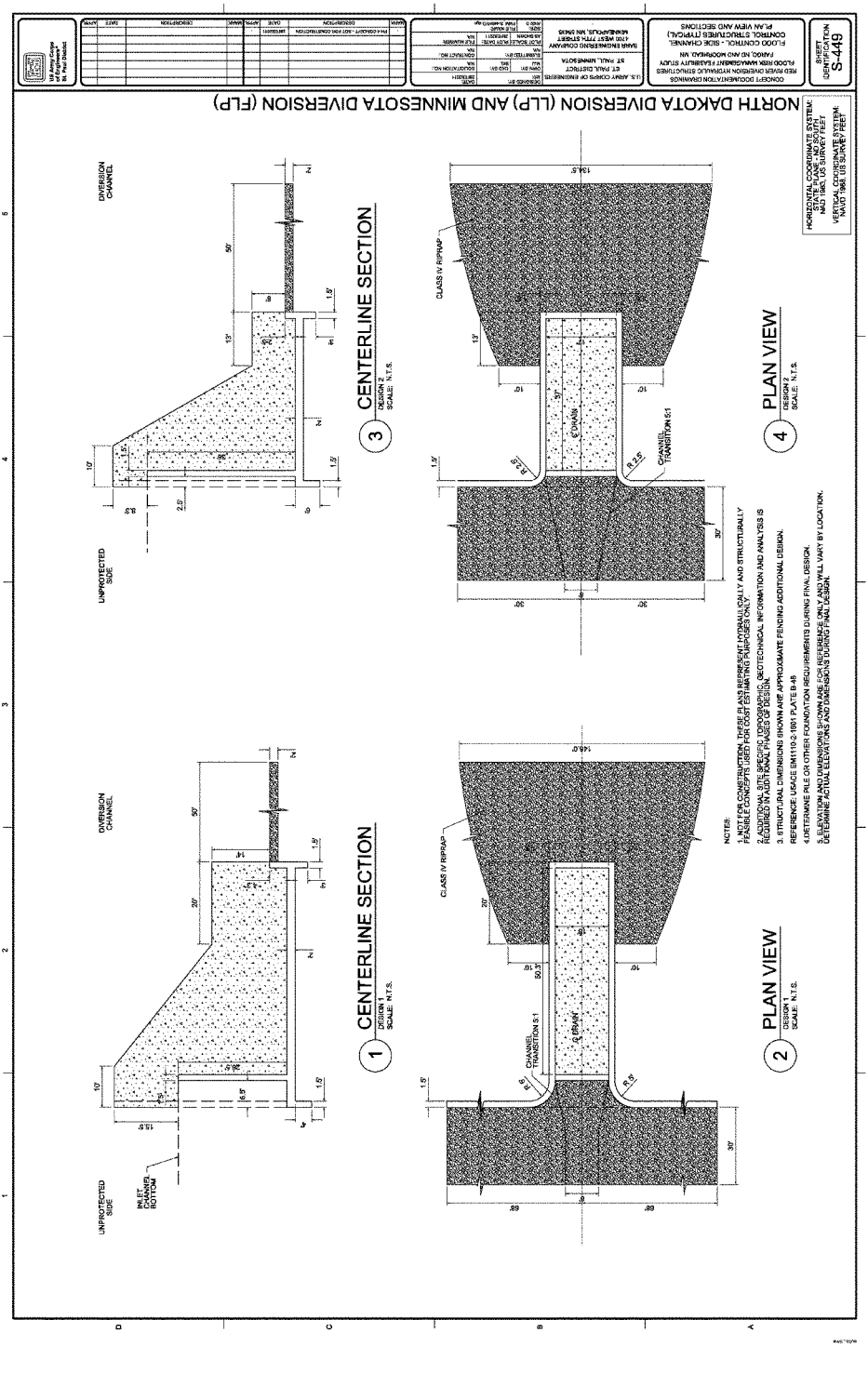

NORTH DAKOTA DIVERSION (LPP)
PLAN VIEW
RUSH RIVER FISH PASSAGE
SCALE IN FEET
PROFILE
RUSH RIVER FISH PASSAGE SYSTEM CENTERLINE
SCALE IN FEET
SECTION
TYPICAL FISH PASSAGE
SCALE IN FEET
NORTH DAKOTA DIVERSION CHANNEL
MEANDERING LOW FLOW CHANNEL (MEANDER NOT SHOWN)
FISH PASSAGE SYSTEM ROCK/BOULDERS NOT SHOWN (FOR CLARITY)
RUSH RIVER DROP STRUCTURE
CONCRETE RETAINING WALL
WALL OPENING FOR FISH PASSAGE SYSTEM
SPOIL BANK
BENCH
RIFFLE
POOL
FLOW
10H:1V
7H:1V
45.0'
FINISH GRADE
RIFFLE TOP
POOL BOTTOM
ROCK/BOULDERS
NOTES:
1. TYPICAL EXCEPT LOWEST RIFFLE AT DIVERSION CHANNEL BOTTOM
WALL OPENING FOR FISH LADDER
RIFFLE (TYP)
POOL (TYP)
5% SLOPE
22' RIFFLE
20' POOL
PROPOSED DIVERSION CHANNEL FINISH GRADE
EXISTING GRADE
DIVERSION CHANNEL BOTTOM
LOW FLOW CHANNEL
FISH PASSAGE SYSTEM BOTTOM
ELEVATION
STATION
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).
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER BASIN MINNESOTA
FLOOD RISK MANAGEMENT MEASURES STUDY
FARGO, ND AND MOORHEAD, MN
RUSH RIVER FISH PASSAGE SYSTEM
PLAN VIEW & PROFILE
HORIZONTAL COORDINATE SYSTEM:
NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET
SHEET IDENTIFICATION
S-444
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
MAIN ENGINEERING COMPANY
4100 WEST 77TH STREET
MINNEAPOLIS, MN 55435
US Army Corps of Engineers St. Paul District

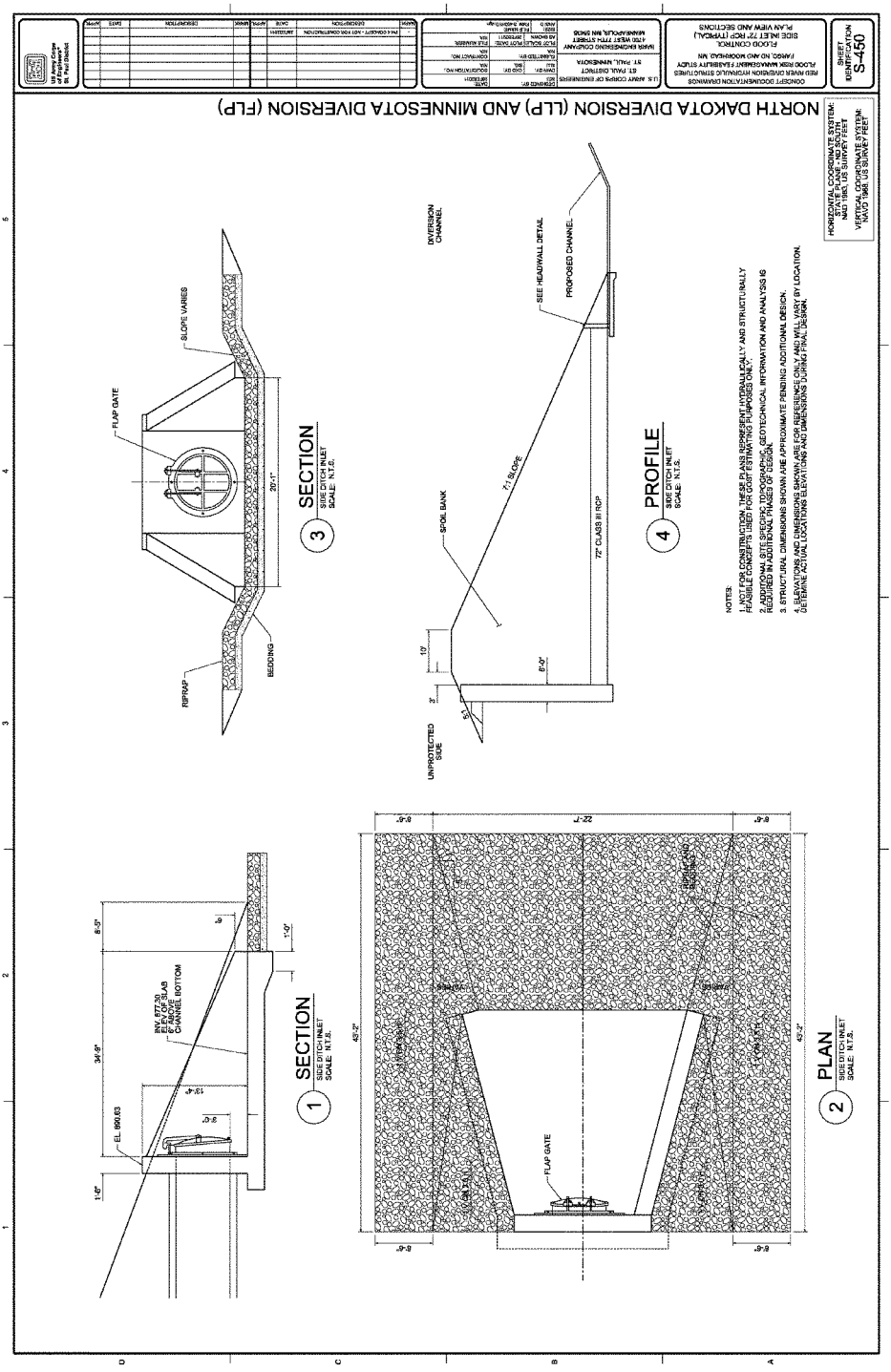

NORTH DAKOTA DIVERSION (LPP)
PLAN VIEW
LLP OUTLET STRUCTURE
SCALE IN FEET
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY FEASIBLE CONCEPTS ONLY. FINISHED CHANNEL SECTION, COST ESTIMATING PURPOSES ONLY.
2. ADDITIONAL SITE SURVEYS, TOPOGRAPHIC, GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
3. FISH PASSAGE TO THE FIXED RIVER OF THE NORTH IS INCLUDED IN THE PROJECT COSTS BUT NOT SHOWN IN DETAIL IN THE PLANS.
RED RIVER DIVERSION CHANNEL
RED RIVER OF THE NORTH
CASS COUNTY ND
CLAY COUNTY MN
RIPRAP
APPROXIMATE ROUTE OF FISH PASSAGE SEE NOTE 3
END OF SPOIL BANK
OUTLET STRUCTURE (SEE NOTE 3 ABOVE) SEE SHEET S-446
OUTLET ACCESS ROAD
SPOIL BANK
BRIDGE
CASS CO ELECTRIC
UNPROTECTED SIDE
EXISTING GRADE 2 FT ELEVATION CONTOUR (TYP.)
LOW FLOW CHANNEL
FLOW
FROM RED RIVER DROP STRUCTURE
CONCRETE RETAINING WALL (TYP.)
APPROXIMATE LIMITS OF TEMPORARY FLOOD PROTECTION DURING CONSTRUCTION
SCADA MONITORING POINT (OPERATIONAL INFO ONLY)
OUTLET PARKING AREA
OUTLET ELECTRICAL SERVICE
SPOIL BANK
OUTLET ACCESS ROAD
PROPOSED FINISH GRADE ELEVATION CONTOUR (TYP.)
RELOCATE/OFFSET UTILITY
PROTECTED SIDE
EXISTING PROPERTY LINE (TYP.)
HORIZONTAL COORDINATE SYSTEM: NAD 1983 (96) US FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
SHEET IDENTIFICATION
S-445
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD CONTROL - LPP OUTLET
STRUCTURE ON RED RIVER OF THE NORTH
PLAN VIEW
U.S. ARMY CORPS OF ENGINEERS
ST PAUL DISTRICT
ST PAUL, MINNESOTA
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
US Army Corps of Engineers St. Paul District


NORTH DAKOTA DIVERSION (LPP)
PLAN
OUTLET STRUCTURE
SCALE: N.T.S.
SECTION
DIVERSION OUTLET STRUCTURE
SCALE: N.T.S.
QUANTITIES
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 AND HYDRAULIC ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
3. STRUCTURAL DIMENSIONS SHOWN ARE APPROXIMATE PENDING ADDITIONAL DESIGN.
4. PILES NOT SHOWN FOR CLARITY.
UPSTREAM NORTH WINGWALL = 225.0'
UPSTREAM SOUTH WINGWALL = 225.0'
OGEE CREST
CONCRETE RETAINING WALL (TYP)
SPILLWAY
OGEE STRUCTURE = 250.0'
DOWNSTREAM NORTH WINGWALL = 320.0'
DOWNSTREAM SOUTH WINGWALL = 320.0'
CONCRETE APRON
PANEL A
50.0'
PANEL B
70.0'
TOP OF WALL EL. 868.7
500YR EL. 864.8
100YR EL. 863.2
CREST EL. 863.0
MIN WSE EL. 863.4 (MEDIAN ELEV.)
CONCRETE APRON
5.0'
TO RED RIVER OF THE NORTH
INV. EL. 843.9
CONCRETE PILE CAP
SHEET PILE CUT-OFF
10.0'
PILE TIP EL. 845.0
CHANNEL INV. EL. 864.2
CONCRETE OGEE ROLLWAY
EL. 847.2
HP14x73 PILES
50.0' OGEE SPILLWAY
70.0' APRON
PANEL
CONCRETE CY
STEEL REINF LBS
PILES NO.
PILES NO. ROWS
PILES LENGTH FT
PILES TOTAL FT
OGEE SPILLWAY & APRON
SPILLWAY 8,751 300,821 336 11 84 21,120
APRON 1,944 235,090 224 8 84 14,336
OGEE WINGWALLS (NORTH & SOUTH)
A 4585 33,480
B 4,774 310,180
TOTAL 15,307 719,771 784 35,456
NOTE: THE QUANTITIES PROVIDED IN THIS TABLE ARE THE QUANTITIES THAT WERE USED TO DEVELOP COST ESTIMATES IN APPENDIX G
HORIZONTAL COORDINATE SYSTEM: NORTH DAKOTA STATE PLANE NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM: NAVD 1988, US SURVEY FEET
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER EMERGENCY HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN
FLOOD CONTROL - LPP OUTLET STRUCTURES
TO RED RIVER OF THE NORTH
PLAN VIEW AND SECTION
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
BARR ENGINEERING COMPANY
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
SHEET IDENTIFICATION
S-446


NORTH DAKOTA DIVERSION (LPP)
ELEVATION
DOWNSTREAM WINGWALL
(NORTH WINGWALL SYMMETRICAL)
SCALE: N.T.S.
ELEVATION
UPSTREAM NORTH WINGWALL
(SOUTH WINGWALL SYMMETRICAL)
SCALE: N.T.S.
SECTION
CONCRETE WINGWALL
SCALE: N.T.S.
NOTE: PILES NOT SHOWN FOR CLARITY.
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. PILES NOT SHOWN FOR CLARITY.
QUANTITIES (WINGWALLS)
ALLOWABLE PILE AXIAL COMPRESSION CAPACITY
LOADING CONDITION
CLASSIFICATION
TONS/PILE
UNDRAINED LRFD
UNDRAINED LRS (ASD)
UNDRAINED EXTREME
DRAINED LRS (ASD)
HORIZONTAL COORDINATE SYSTEM:
NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET
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
EXTENSION
S-447


MEANDERING LOW FLOW CHANNEL
RIPRAP PROTECTION ON TOE OF SLOPE ALONG ENTIRE DIVERSION TO KEEP LOW FLOW CHANNEL TRAINED BETWEEN TOE'S
RIPRAP GRADE CONTROL EVERY 5,000 FEET TO PREVENT EROSION OF LOW FLOW INVERT
BENCH
DIVERSION SLOPE
PLAN VIEW
RRD - LOW FLOW CHANNEL
SCALE IN FEET
VEGETATION
BURY GRADE CONTROL RIPRAP UNDER 4" TO 2' OF TOPSOIL
4" TOPSOIL
GEOTEXTILE
RIPRAP
SECTION
RIPRAP GRADE CONTROL
SCALE: N.T.S.
BURY TOE PROTECTION RIPRAP UNDER 4" TOPSOIL
SET RIPRAP ON GEOTEXTILE FABRIC
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.
LOW FLOW CHANNEL
BOTTOM OF LOW FLOW CHANNEL
SLOPE RIPRAP GRADE CONTROL TO A LOW SPOT WHERE THE LOW FLOW CHANNEL CROSSES
BOTTOM OF THE TOE PROTECTION RIPRAP OUTSIDE OF GRADE CONTROL RIPRAP
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
NORTH DAKOTA DIVERSION (LLP)
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION (HYDRAULIC STRUCTURES ALONG MAINSTEM RED RIVER AND DIVERSION CUT)
FARGO, ND AND MOORHEAD, MN
FLOOD CONTROL - LOW FLOW CHANNEL DOWNSTREAM OF DIVERSION INLET ON RED RIVER, PLAN VIEW & SECTIONS
SHEET IDENTIFICATION
S-448


NORTH DAKOTA DIVERSION (LLP) AND MINNESOTA DIVERSION (FLP)
CONCEPT MODIFICATION 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
DRAWING DESIGNATION
S-449
HORIZONTAL COORDINATE SYSTEM:
NAD 1983 (NSRS 2007)
MN TM, US SURVEY FEET
VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET
U.S. ARMY CORPS OF ENGINEERS
ST PAUL DISTRICT
ST. PAUL, MINNESOTA
SHAW ENGINEERING COMPANY
1100 WEST 79TH STREET
BLOOMINGTON, MN 55431
US Army Corps of Engineers
St. Paul District
UNPROTECTED SIDE
DIVERSION CHANNEL
1 CENTERLINE SECTION
DESIGN 1
SCALE: N.T.S.
2 PLAN VIEW
DESIGN 1
SCALE: N.T.S.
3 CENTERLINE SECTION
DESIGN 2
SCALE: N.T.S.
4 PLAN VIEW
DESIGN 2
SCALE: N.T.S.
CLASS IV RIPRAP
CHANNEL TRANSITION 5:1
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTUAL DESIGNS FOR FEASIBILITY LEVEL ANALYSIS.
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 EM1110-2-1901 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.


NORTH DAKOTA DIVERSION (LLP) AND MINNESOTA DIVERSION (FLP)
1 SECTION
SIDE DITCH INLET
SCALE: N.T.S.
2 PLAN
SIDE DITCH INLET
SCALE: N.T.S.
3 SECTION
SIDE DITCH INLET
SCALE: N.T.S.
4 PROFILE
SIDE DITCH INLET
SCALE: N.T.S.
FLAP GATE
SLOPE VARIES
RIPRAP
BEDDING
SPOIL BANK
DIVERSION CHANNEL
PROPOSED CHANNEL
SEE HEADWALL DETAIL
72" CLASS III RCP
UNPROTECTED SIDE
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 FOR FINAL DESIGN.
3. STRUCTURAL DIMENSIONS SHOWN ARE APPROXIMATE PENDING ADDITIONAL DESIGN.
4. ELEVATIONS AND DIMENSIONS SHOWN ARE FOR REFERENCE ONLY AND WILL VARY BY LOCATION. DETERMINE ACTUAL LOCATIONS ELEVATIONS AND DIMENSIONS DURING FINAL DESIGN.
HORIZONTAL COORDINATE SYSTEM:
ND STATE PLANE SYSTEM
NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET
CONCEPT DOCUMENTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD CONTROL
SIDE INLET 72" RCP (TYPICAL)
PLAN VIEW AND SECTIONS
SHEET IDENTIFICATION
S-450
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
ST. PAUL, MINNESOTA
MAIN DIMENSIONING COMPANY
4150 WEST 77TH STREET
MINNEAPOLIS, MN 55435
DESIGNED BY
DRAWN BY
CHECKED BY
QUALITY CHECK BY
SUBMITTED BY
DATE
SUBMITTED
CONTRACT NO.
SCALE
AS SHOWN
PLOT SCALE (PLOT DATE)
FILE NUMBER
DESIGNATION NO.
CONTRACT NO.
DATE
ISSUED
Date 2-dd-mm-yy
Appr.
Date
DESCRIPTION
No.
INITIATED
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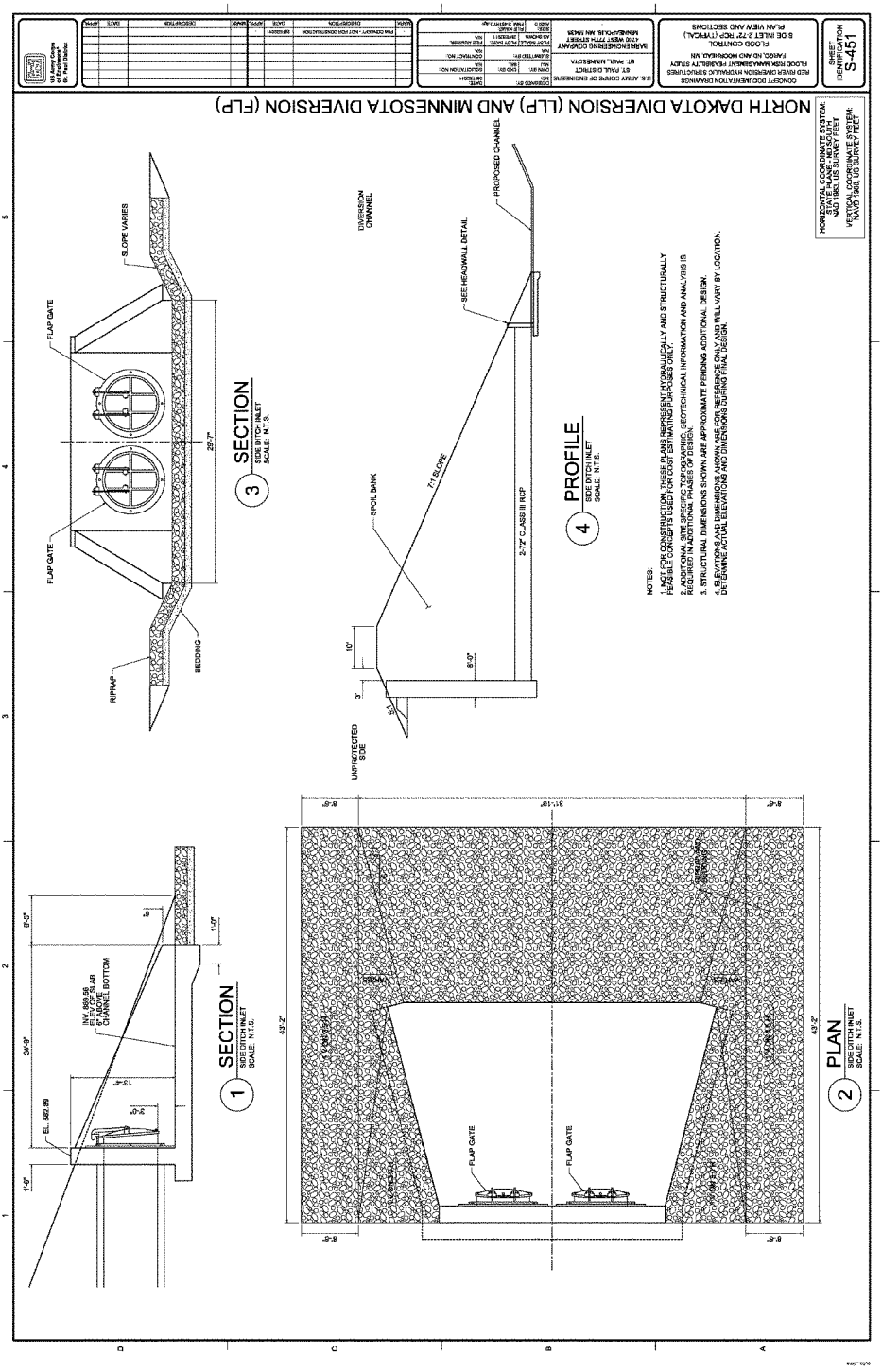
NORTH DAKOTA DIVERSION (LLP) AND MINNESOTA DIVERSION (FLP)
1 SECTION
SIDE DITCH INLET
SCALE: N.T.S.
2 PLAN
SIDE DITCH INLET
SCALE: N.T.S.
3 SECTION
SIDE DITCH INLET
SCALE: N.T.S.
4 PROFILE
SIDE DITCH INLET
SCALE: N.T.S.
FLAP GATE
SLOPE VARIES
RIPRAP
BEDDING
SPOIL BANK
DIVERSION CHANNEL
PROPOSED CHANNEL
SEE HEADWALL DETAIL
7:1 SLOPE
2-72" CLASS III RCP
UNPROTECTED SIDE
NOTES:
1. NOT FOR CONSTRUCTION. THESE PLANS REPRESENT HYDRAULICALLY AND STRUCTURALLY FEASIBLE CONCEPTS USED FOR COST ESTIMATING PURPOSES ONLY.
2. ADDITIONAL GEOTECHNICAL INFORMATION AND ANALYSIS IS REQUIRED IN ADDITIONAL PHASES OF DESIGN.
3. STRUCTURAL DIMENSIONS SHOWN ARE APPROXIMATE PENDING ADDITIONAL DESIGN.
4. ELEVATIONS AND DIMENSIONS SHOWN ARE FOR REFERENCE ONLY AND WILL VARY BY LOCATION. DETERMINE ACTUAL ELEVATIONS AND DIMENSIONS DURING FINAL DESIGN.
HORIZONTAL COORDINATE SYSTEM:
NAD 1983, US SURVEY FEET
VERTICAL COORDINATE SYSTEM:
NAVD 1988, US SURVEY FEET
CONCEPT PODUNERTATION DRAWINGS
RED RIVER DIVERSION HYDRAULIC STRUCTURES
FLOOD RISK MANAGEMENT FEASIBILITY STUDY
FARGO, ND AND MOORHEAD, MN
SIDE INLET 2.72" REP (TYPICAL)
FLOOD CONTROL
PLAN VIEW AND SECTIONS
SHEET
DESIGNATION
S-451
U.S. ARMY CORPS OF ENGINEERS
ST PAUL DISTRICT
ST. PAUL, MINNESOTA
DESIGNED BY
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CONTRACT NO.
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US Army Corps of Engineers
St. Paul District

RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT A – BACKGROUND HYDROLOGIC INFORMATION

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

By: BARR ENGINEERING

FINAL: February 28, 2011

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT A – BACKGROUND HYDROLOGIC INFORMATION

F-A1.0 INTRODUCTION

The background hydrology presented in this exhibit summarizes the existing hydrology (without project impacts) for the Red River of the North and the five major tributaries in North Dakota within the Project area (Wild Rice River, Sheyenne River, Maple River, Lower Rush River, and Rush River). The hydrology is summarized for the 10-, 50-, 100-, and 500-year events at the locations where the proposed Diversion Channel would intersect the rivers, unless otherwise noted. The additional hydrology information presented here includes water surface elevations, average channel velocities and measured velocity distributions. More details about background hydrology can be found in Appendix A.

The scenarios summarizing the "coincidental tributary hydrology" for the North Dakota tributaries takes into account the hydrology on the Red River of the North and can be thought of as the expected flow or water surface elevation in a tributary when a flood of a certain event occurs on the Red River of the North. All hydrology presented in this exhibit is Phase 4 hydrology unless noted otherwise, and a definition of this hydrology can be found in Appendix A.

The scenarios summarizing the "local tributary hydrology" for the North Dakota tributaries are based on when the peak flows and water surface elevations occur on the Maple River. The local tributary hydrology has been analyzed for both existing and with-project conditions. Although the analysis of the local peak tributary flows was only extended as far as Pearly downstream of the Project for this phase.

F-A2.0 BACKGROUND HYDROLOGY OF THE RED RIVER OF THE NORTH

F-A2.1 BACKGROUND HYDROLOGY OF THE RED RIVER OF THE NORTH AT THE FCP CONTROL STRUCTURE

The following information regarding the FCP was previously presented as part of Appendix F Exhibit A in the Phase 3 report submitted on August 6th, 2010, and is included here for completeness. All of the following FCP information is based on Phase 3 hydrology and the HEC-RAS steady flow model developed during Phase 3.

Existing flows, water surface elevations, flow areas, and average velocities for the Red River of the North at the location of the proposed FCP Control Structure (Station 467.4) for the 2-, 5-,

10-, 20-, 50-, 100-, 200-, and 500-year events are presented for the Year 0, 25 and 50 hydrology in Tables FA-1, FA-2, and FA-3, respectively. Existing water surface elevations in the Red River of the North at the FCP Control Structure (Station 467.4) for the flood events listed above, plotted against the cross section data, are presented for the Year 0, 25 and 50 hydrology in Figures FA-1, FA-2, and FA-3.

Flows and water surface elevations decrease from Year 0 to Year 25 to Year 50 hydrology. The decrease in flows from Year 0 to Year 50 hydrology at the FCP Control Structure is more significant for the more frequent events (37 percent for 2-year events) than for less frequent events (12 percent for 500-year events).

Table F-A1 Background Existing Year 0 Hydrology of the Red River of the North at the Approximate Location of the FCP Control Structure (Station 467.4)

Hydrology	Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
Phase3.1	Median*	360	880.02	1,041	0.34
Phase3.1	2	5,600	892.77	3,868	1.84
Phase3.1	5	12,150	900.72	9,031	2.44
Phase3.1	10	17,000	904.66	14,955	2.78
Phase3.1	20	22,000	907.71	24,913	2.91
Phase3.1	50	29,300	910.68	45,927	2.84
Phase3.1	100	34,700	911.10	69,644	2.72
Phase3.1	200	46,200	912.65	89,961	2.85
Phase3.1	500	61,700	914.42	113,230	2.97

*Note: Value extrapolated from existing data. Estimate of median flow is available for Year 0 Hydrology only.

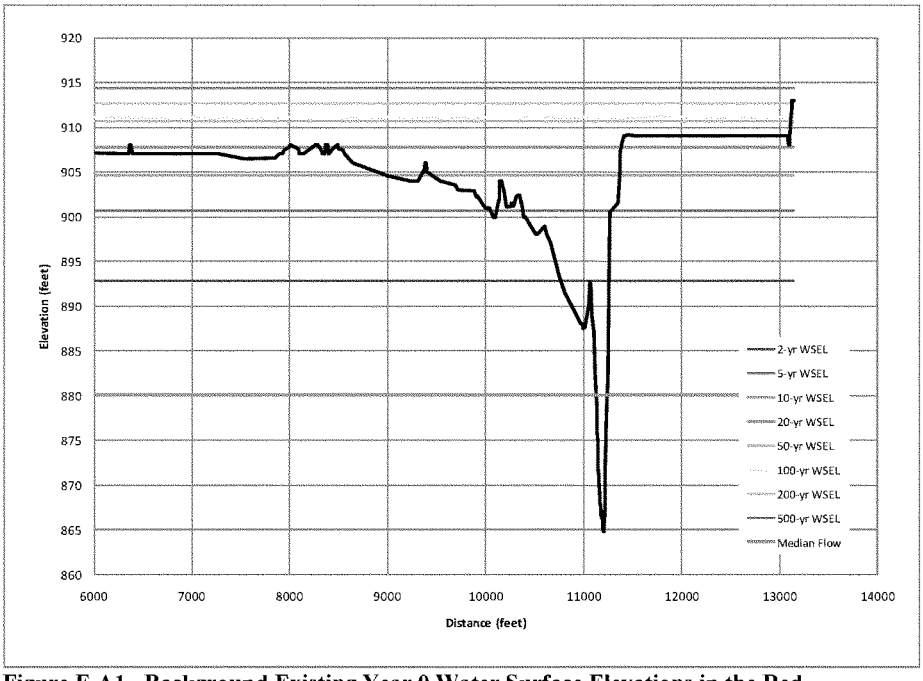

Figure F-A1 Background Existing Year 0 Water Surface Elevations in the Red River of the North at the Approximate Location of the FCP Control Structure (Station 467.4)

Table F-A2 Background Existing Year 25 Hydrology of the Red River of the North at the Approximate Location of the FCP Control Structure (Station 467.4)

Hydrology	Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
Phase3.1	2	4,352	890.64	2,928	1.68
Phase3.1	5	10,608	899.22	7,704	2.33
Phase3.1	10	15,394	903.48	12,683	2.64
Phase3.1	20	20,345	906.81	20,712	2.91
Phase3.1	50	27,441	910.08	41,191	2.86
Phase3.1	100	32,921	911.03	68,760	2.61
Phase3.1	200	42,242	912.16	83,548	2.80
Phase3.1	500	57,641	914.00	107,699	2.93

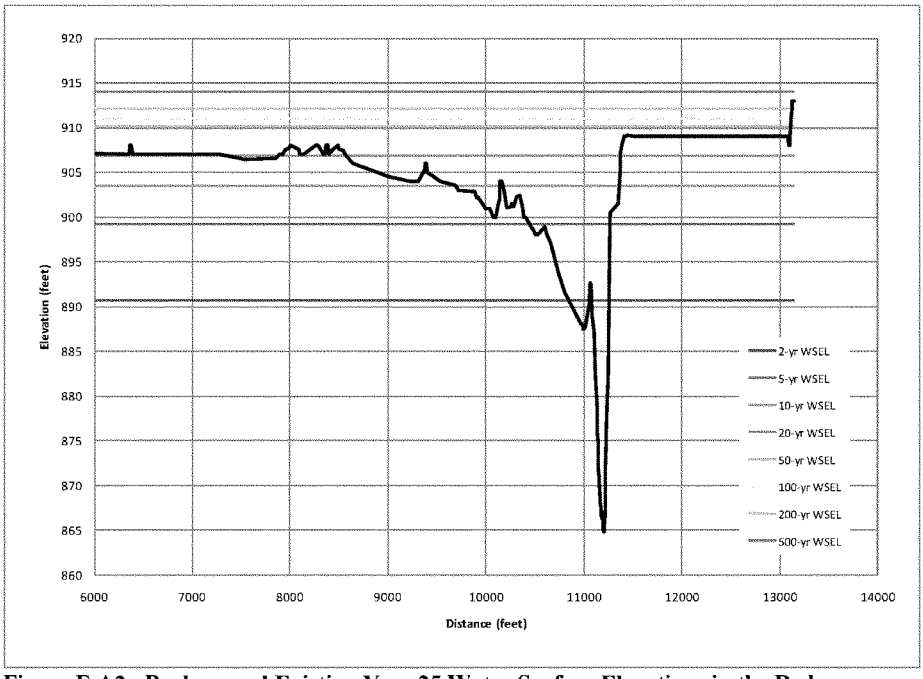

Figure F-A2 Background Existing Year 25 Water Surface Elevations in the Red River of the North at the Approximate Location of the FCP Control Structure (Station 467.4)

Table F-A3 Background Existing Year 50 Hydrology of the Red River of the North at the Approximate Location of the FCP Control Structure (Station 467.4)

Hydrology	Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
Phase3.1	2	3,506	888.95	2,372	1.53
Phase3.1	5	9,161	897.66	6,572	2.18
Phase3.1	10	13,965	902.36	11,003	2.54
Phase3.1	20	18,855	905.90	18,066	2.82
Phase3.1	50	25,764	909.46	36,310	2.90
Phase3.1	100	31,304	910.80	46,858	2.99
Phase3.1	200	38,787	911.67	77,099	2.78
Phase3.1	500	54,034	913.60	102,503	2.90

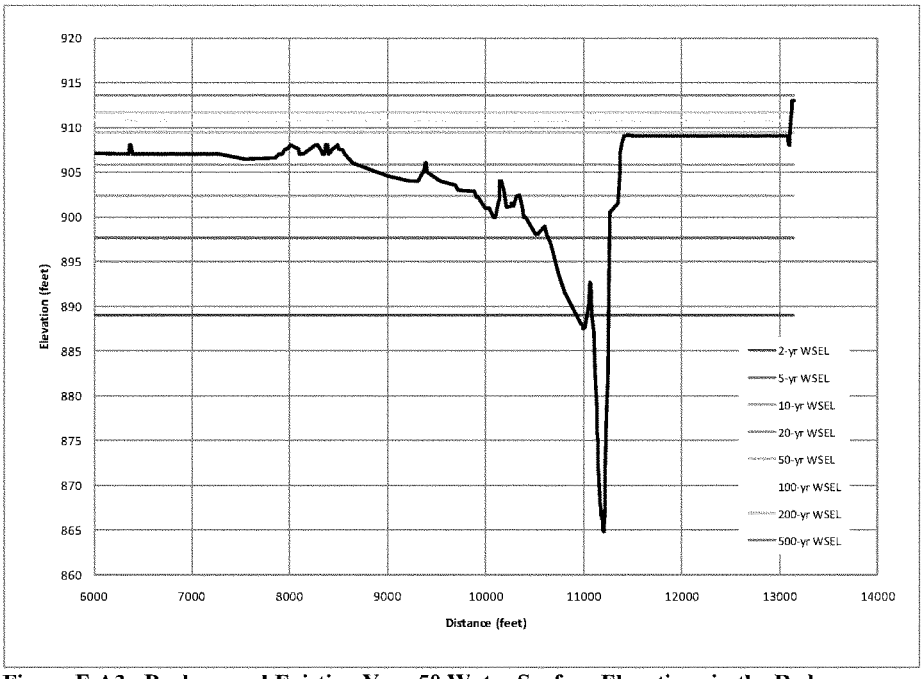

Figure F-A3 Background Existing Year 50 Water Surface Elevations in the Red River of the North at the Approximate Location of the FCP Control Structure (Station 467.4)

F-A2.2 BACKGROUND HYDROLOGY OF THE RED RIVER OF THE NORTH AT THE FCP OUTLET

The following information regarding the FCP was previously presented as part of Appendix F Exhibit A in the Phase 3 report submitted on August 6th, 2010, and is included here for completeness. All of the following FCP information is based on Phase 3 hydrology and the HEC-RAS steady flow model developed during Phase 3.

Existing flows, water surface elevations, flow areas, and average velocities for the Red River of the North at the proposed location of the FCP Diversion Channel Outlet (Station 299) for the 2-, 5-, 10-, 20-, 50-, 100-, 200-, and 500-year events are presented for the Year 0, 25 and 50 hydrology in Tables FA-4, FA-5, and FA-6, respectively. Existing water surface elevations in the Red River of the North at the proposed FCP Diversion Channel Outlet (Station 299) for the flood events listed above, plotted against the cross section data, are presented for the Year 0, 25 and 50 hydrology in Figures FA-4, FA-5, and FA-6.

Table F-A4 Background Existing Year 0 Hydrology of the Red River of the North at the Approximate Location of the FCP Diversion Channel Outlet (Station 299)

Hydrology	Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
Phase3.1	median	876	858.07	657	1.32
Phase3.1	2	8,328	874.63	4,404	2.56
Phase3.1	5	16,039	882.84	10,452	3.26
Phase3.1	10	22,069	885.93	29,360	3.36
Phase3.1	20	28,491	887.53	44,094	3.45
Phase3.1	50	37,826	889.13	58,887	3.66
Phase3.1	100	45,160	889.67	63,892	4.07
Phase3.1	200	56,895	890.05	149,180	3.03
Phase3.1	500	72,930	890.98	171,235	3.29

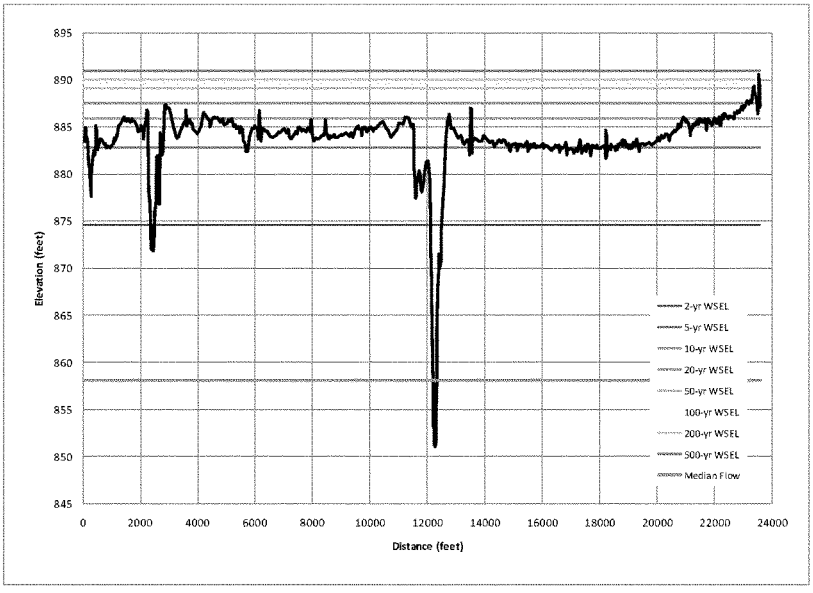

Figure F-A4 Background Existing Year 0 Water Surface Elevations in the Red River of the North at the Approximate Location of the FCP Diversion Channel Outlet (Station 299)

Table F-A5 Background Existing Year 25 Hydrology of the Red River of the North at the Approximate Location of the FCP Diversion Channel Outlet (Station 299)

Hydrology	Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
Phase3.1	2	7,124	872.92	3,752	2.42
Phase3.1	5	14,681	881.76	8,959	3.16
Phase3.1	10	20,667	885.43	24,889	3.35
Phase3.1	20	27,055	887.20	41,079	3.43
Phase3.1	50	36,261	888.90	56,774	3.62
Phase3.1	100	43,687	889.62	63,410	3.96
Phase3.1	200	54,114	890.07	149,695	2.87
Phase3.1	500	70,317	890.83	167,563	3.26

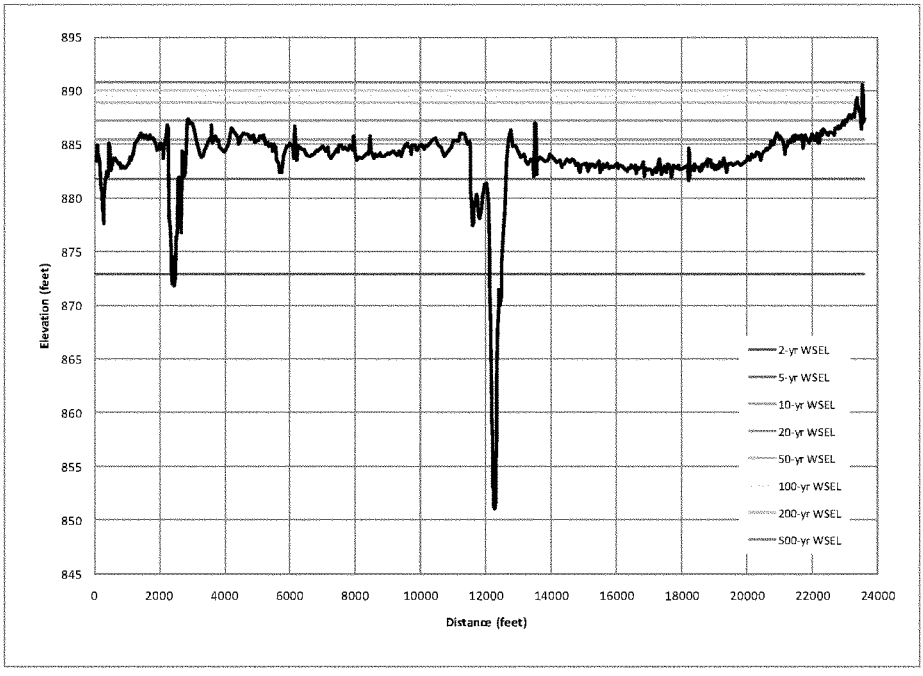

Figure F-A5 Background Existing Year 25 Water Surface Elevations in the Red River of the North at the Approximate Location of the FCP Diversion Channel Outlet (Station 299)

Table F-A6 Background Existing Year 50 Hydrology of the Red River of the North at the Approximate Location of the FCP Diversion Channel Outlet (Station 299)

Hydrology	Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
Phase3.1	2	6,247	871.54	3,253	2.31
Phase3.1	5	13,447	880.72	7,896	3.03
Phase3.1	10	19,446	884.94	20,728	3.33
Phase3.1	20	25,786	886.92	38,506	3.41
Phase3.1	50	34,892	888.68	54,729	3.59
Phase3.1	100	42,390	889.78	64,865	3.76
Phase3.1	200	51,664	890.02	148,529	2.76
Phase3.1	500	67,999	890.68	164,052	3.23

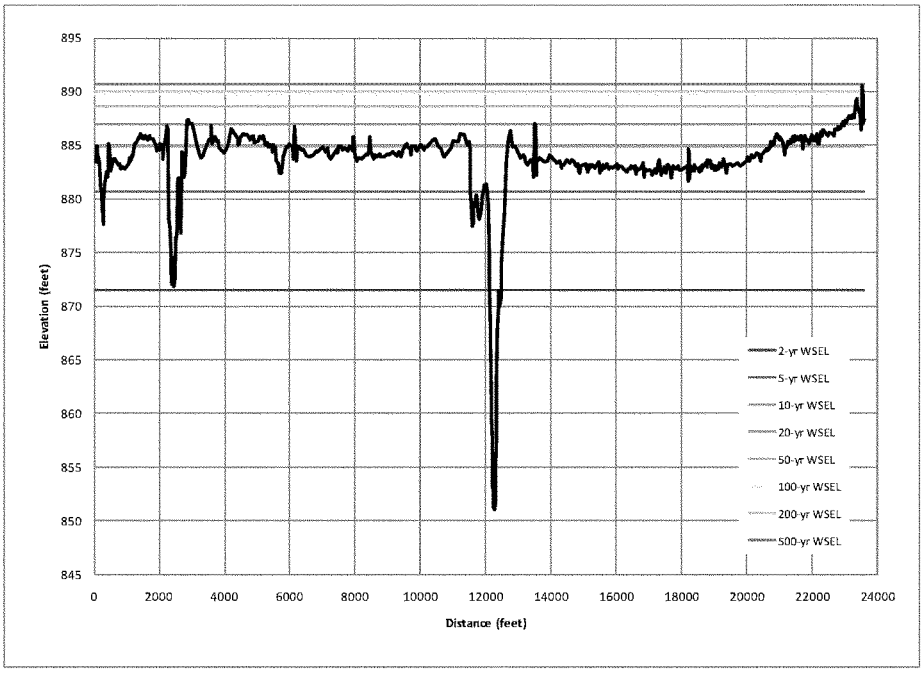

Figure F-A6 Background Existing Year 50 Water Surface Elevations in the Red River of the North at the Approximate Location of the FCP Diversion Channel Outlet (Station 299)

Flows and water surface elevations decrease from Year 0 to Year 25 to Year 50 hydrology. The decrease in flows from Year 0 to Year 50 hydrology at the FCP Diversion Channel Outlet is more significant for the more frequent events (25 percent for 2-year events) than for less frequent events (seven percent for 500-year events).

F-A2.3 BACKGROUND HYDROLOGY OF THE RED RIVER OF THE NORTH AT THE LPP CONTROL STRUCTURE

Existing flows, water surface elevations, and average velocities for the Red River of the North at the approximate location of the LPP Control Structure (Phase 3 Station 478.8, Phase 4 Station 2531315) for the 10-, 50-, 100-, and 500-year events are presented in Table FA-7. The LPP Control Structure will be constructed in a proposed channel off of the existing river channel. Phase 3 Station 478.8 and Phase 4 Station 2531315 are at approximately the same location in the existing river channel as the LPP Control Structure will be in the proposed channel. Existing water surface elevations in the Red River of the North at the approximate location of the LPP Control Structure (Phase 3 Station 478.8, Phase 4 Station 2531315) for the flood events listed above, plotted against the cross section data, are presented in Figure FA-7.

Table F-A7 Background Existing Hydrology of the Red River of the North at the Approximate Location of the LPP Control Structure (Phase 3 Station 478.8, Phase 4 Station 2531315)

Event (yr)	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)*	WSEL (ft)	Velocity (ft/s)*
	Phase 3			Phase 4		
2	4,000	896.91	1.57	-	-	-
5	7,000	904.9	1.77	-	-	-
10	10,500	908.42	2.2	10,271	908.06	1.12
50	21,000	914.1	3.05	18,207	913.76	0.89
100	25,000	914.94	3.33	21,458	914.65	0.90
500	32,000	917.7	2.46	28,623	915.94	0.98

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
Note: Phase 3 values based on steady flow analysis
Note: Phase 4 values based on unsteady flow analysis
*Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

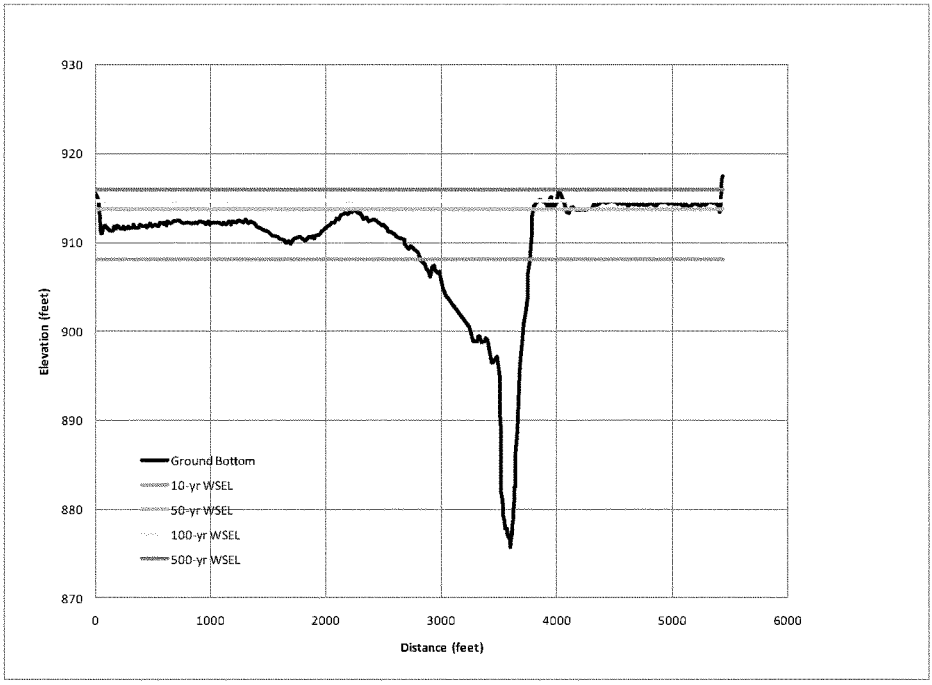

Figure F-A7 Background Existing Water Surface Elevations in the Red River of the North at the Approximate Location of the LPP Control Structure (Station 2531315)

Flows and water surface elevations decrease from Phase 3 to Phase 4 due to changes from the Phase 3 hydrology discussed in Appendix A, and a change in methodology for computing water surface elevations for Phase 4. Phase 4 water surface elevations were calculated using a HEC-RAS unsteady flow model that accounts for flow through the floodplain and off channel storage areas, compared to Phase 3 that used a combination of a HEC-RAS steady flow model for the Red River and rating curves for the tributaries. The Phase 4 model includes storage areas and storage area connections to provide an accurate depiction of how flows enter and exit the main channel and accounts for conveyance of flow through the floodplain.

F-A2.4 BACKGROUND HYDROLOGY OF THE RED RIVER OF THE NORTH AT THE LPP OUTLET

Existing flows, water surface elevations, and average velocities for the Red River of the North at the proposed location of the LPP Diversion Channel Outlet (Phase 3 Station 292, Phase 4 Station 2208555) for the 10-, 50-, 100-, and 500-year events are presented in Table FA-8. Existing water surface elevations in the Red River of the North at the proposed LPP Diversion Channel Outlet (Phase 3 Station 292, Phase 4 Station 2208555) for the flood events list above, plotted against the cross section data, are presented in Figure FA-8.

Table F-A8 Background Existing Hydrology of the Red River of the North at the Approximate Location of the LPP Diversion Channel Outlet (Phase 3 Station 292, Phase 4 Station 2208555)

Event (yr)	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)*	WSEL (ft)	Velocity (ft/s)*
	Phase 3			Phase 4		
2	8,328	870.05	2.57	-	-	-
5	16,039	878.37	3.35	-	-	-
10	22,069	881.72	3.3	23,069	881.12	0.87
50	37,826	884.38	3.75	40,743	883.33	0.54
100	45,160	885.20	3.18	47,522	883.92	0.56
500	72,930	886.86	3.92	62,085	885.03	0.61

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
Note: Phase 3 values based on steady flow analysis
Note: Phase 4 values based on unsteady flow analysis
*Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

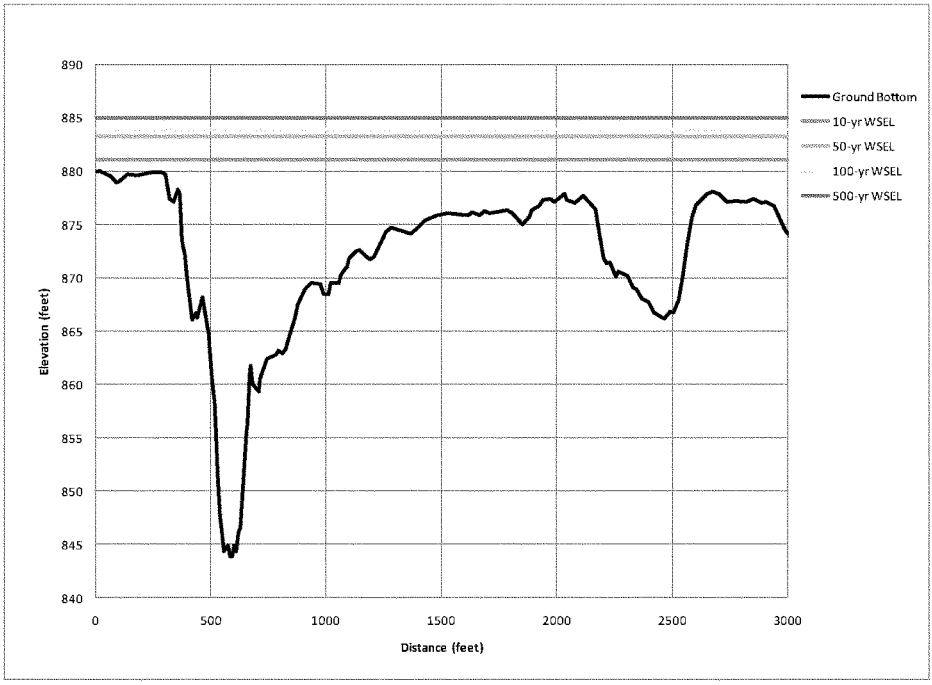

Figure F-A8 Background Existing Water Surface Elevations in the Red River of the North at the Approximate Location of the LPP Diversion Channel Outlet (Station 2208555)

Flows and water surface elevations in Phase 3 and Phase 4 can be different. Phase 4 water surface elevations were calculated using a HEC-RAS unsteady flow model that accounts for flow through the floodplain and off channel storage areas, compared to Phase 3 that used a combination of a HEC-RAS steady flow model for the Red River and rating curves for the tributaries. The Phase 4 model includes storage areas and storage area connections to provide an accurate depiction of how flows enter and exit the main channel and accounts for conveyance of flow through the floodplain.

F-A2.5 MEASURED VELOCITIES IN THE RED RIVER OF THE NORTH

Actual velocities in the Red River of the North have been measured at two US Geological Survey (USGS) gaging stations: Fargo (USGS Gage 05054000) and Hickson (USGS Gage 05051522). Figure F-A9 presents maximum channel velocities in the Red River of the North at Fargo plotted against flow. Figure F-A10 presents velocity distributions in the Red River of the North at Fargo measured on July 7, 2007 and March 28, 2009. Figures F-A11 and F-A12 present average channel velocities plotted against flow in the Red River of the North at Fargo and Hickson, respectively.

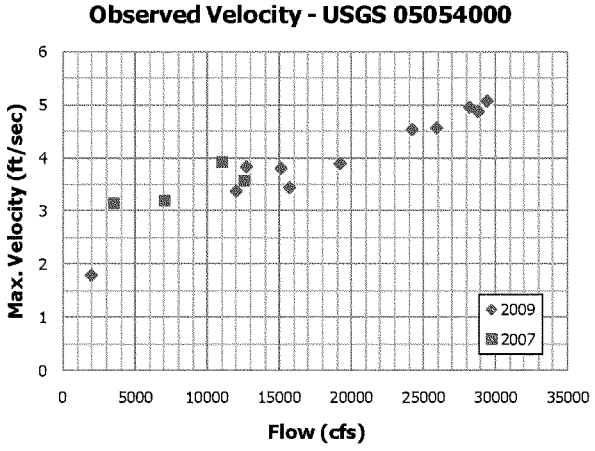

Figure F-A9 Maximum channel velocities in the Red River measured at USGS gage 05054000 (Fargo, ND)

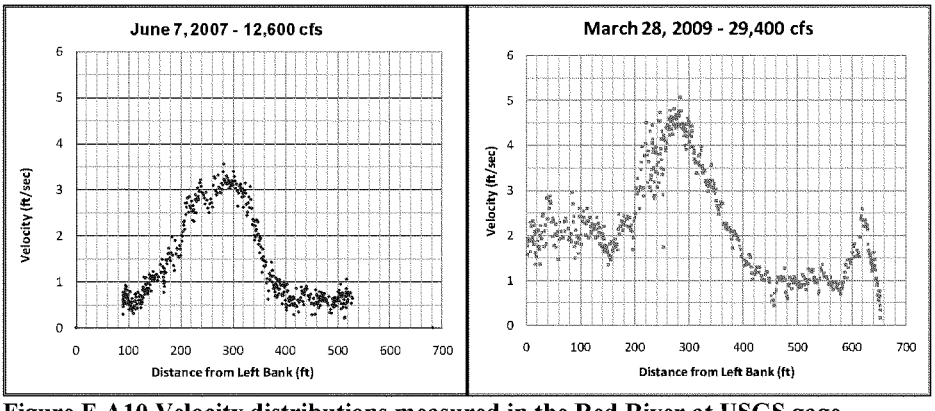

Figure F-A10 Velocity distributions measured in the Red River at USGS gage 05054000 (Fargo, ND)

12,600 cfs corresponds to approximately a 5-year event based on Phase 3 hydrology (low flow events were not updated during Phase 4), and 29,400 cfs between a 10-year and 50-year event based on Phase 4 hydrology.

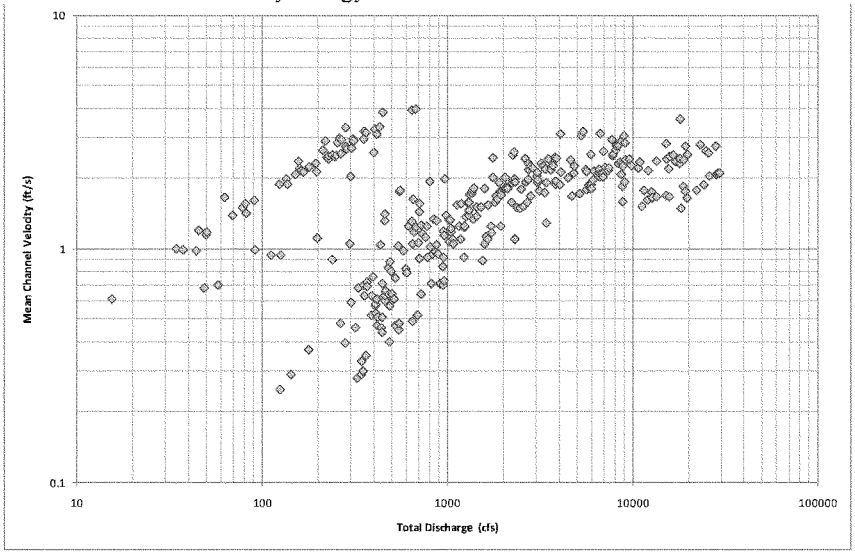

Figure F-A11 Average channel velocities versus total discharge in the Red River measured at USGS gage 05054000 (Fargo, ND)

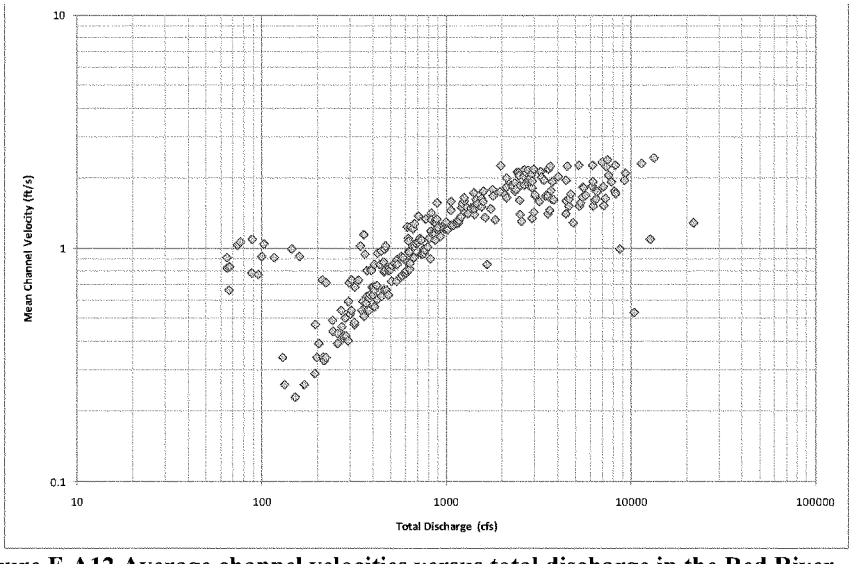

Figure F-A12 Average channel velocities versus total discharge in the Red River measured at USGS gage 05051522 (Hickson, ND)

F-A3.0 BACKGROUND HYDROLOGY OF THE WILD RICE RIVER

Flows, water surface elevations, and average velocities for the Wild Rice River for the 10-, 50-, 100-, and 500-year events are presented for the local tributary hydrology in Table FA-9. Water surface elevations in the Wild Rice River for the flood events listed above, plotted against the cross section data, and are presented for local tributary hydrology in Figure FA-13.

Table F-A9 Background Existing Local Hydrology of the Wild Rice River at the Approximate Location of the LPP Diversion Channel

Event (yr)	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)**	WSEL (ft)	Velocity (ft/s)**
	Phase 3			Phase 4		
2	1,388	902.22	1.16	-	-	-
5	3,996	908.94	1.36	-	-	-
10	6,593	912.69	1.47	5,444	910.59	1.76
50	14,639	916.97	0.62	8,824	915.15	0.58
100	19,016	917.92	0.66	8,688	915.84	0.45
500	31,107	920.22	0.76	9,565	916.99	0.35

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
Note: Phase 3 values based on steady flow analysis
Note: Phase 4 values based on unsteady flow analysis
**Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

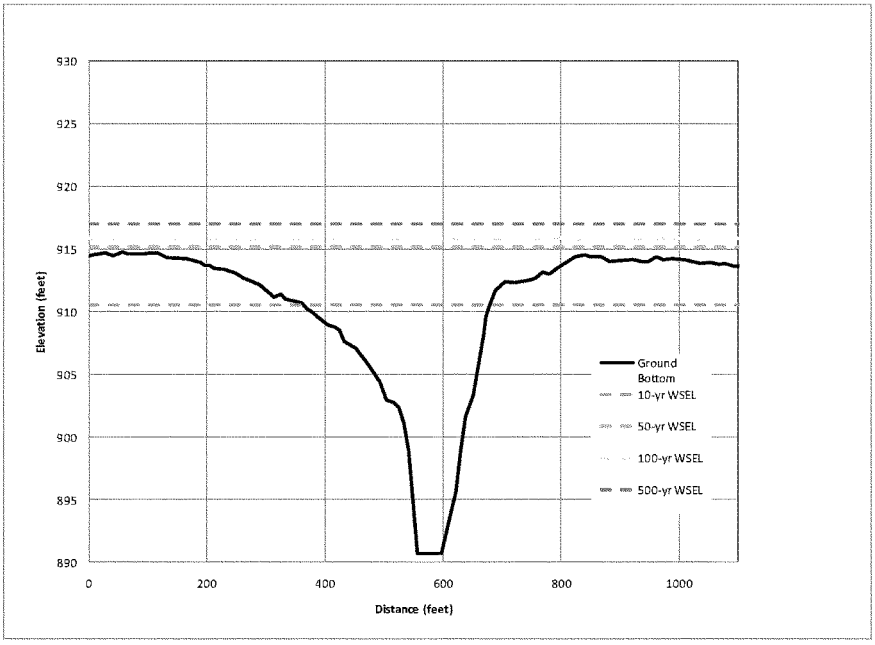

Figure F-A13 Background Existing Local Hydrology Water Surface Elevations in the Wild Rice River at the Approximate Location of the LPP Diversion Channel (Station 69855)

Flows, water surface elevations, flow areas, and average velocities for the Wild Rice River for the 10-, 50-, 100-, and 500-year events are presented coincidental tributary hydrology in Table FA-10. Water surface elevations in the Wild Rice River for the flood events listed above, plotted against the cross section data, and are presented coincidental tributary hydrology in Figure FA-14.

Table F-A10 Background Existing Coincidental Hydrology of the Wild Rice River at the Approximate Location of the LPP Diversion Channel

Event (yr)*	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)**	WSEL (ft)	Velocity (ft/s)**
	Phase 3			Phase 4		
2	1,419	902.31	1.16	-	-	-
5	3,021	906.76	1.35	-	-	-
10	6,185	912.18	1.47	6,393	912.13	1.60
50	11,655	916.10	0.61	8,641	915.82	0.45
100	13,780	916.74	0.61	8,503	916.31	0.39
500	18,342	917.77	0.66	8,767	916.88	0.35

*Event recurrence interval refers to the Red River flow record.
Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
Note: Phase 3 values based on steady flow analysis
Note: Phase 4 values based on unsteady flow analysis
**Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

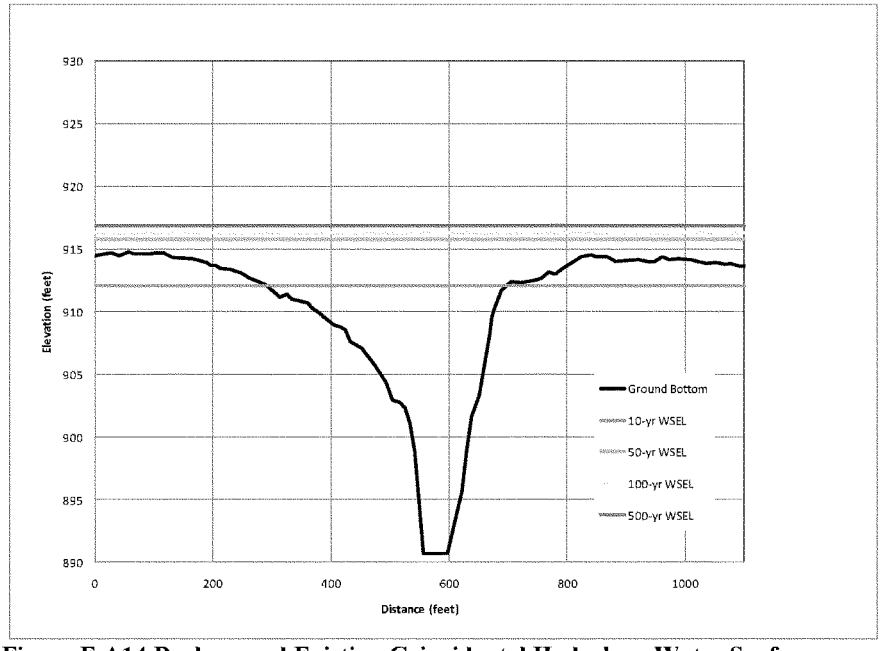

Figure F-A14 Background Existing Coincidental Hydrology Water Surface Elevations in the Wild Rice River at the Approximate Location of the LPP Diversion Channel (Station 69855)

Flows and water surface elevations in Phase 3 and Phase 4 can be different. Phase 4 water surface elevations were calculated using a HEC-RAS unsteady flow model that accounts for flow through the floodplain and off channel storage areas, compared to Phase 3 that used a combination of a HEC-RAS steady flow model for the Red River and rating curves for the tributaries. The Phase 4 model includes storage areas and storage area connections to provide an accurate depiction of how flows enter and exit the main channel and accounts for conveyance of flow through the floodplain.

Actual velocities in the Wild Rice River have been measured at the USGS Gage 05053000 at Abercrombie (approximately 10 miles upstream of the Diversion Channel). Figure F-A15 present average channel velocities plotted against flow in the Wild Rice River. Figure F-A16 presents velocity distributions in the Wild Rice River measured on March 24, 2009 and May 27, 2009.

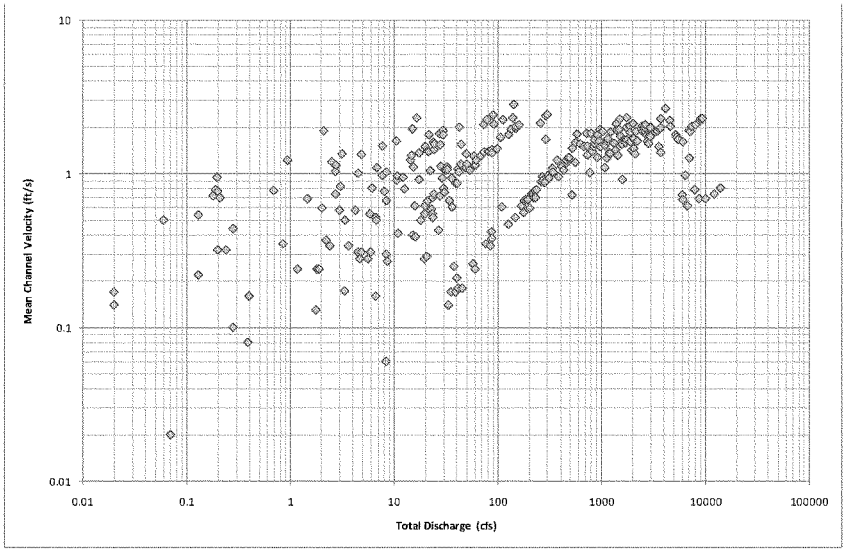

Figure F-A15 Average channel velocities versus total discharge in the Wild Rice River measured at USGS gage 05053000 (Abercrombie, ND)

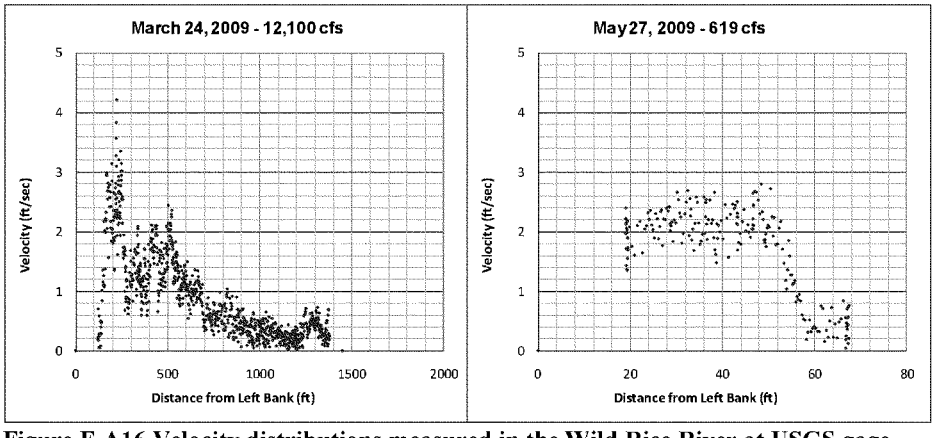

Figure F-A16 Velocity distributions measured in the Wild Rice River at USGS gage 05053000 (Abercrombie, ND)

12,100 cfs corresponds to approximately a 50-year local event (For Phase 4, Table F-A9 only includes conveyed by the main channel of the Wild Rice River and does not include flows in the overbanks.) and 619 cfs corresponds to less than a 2-year local event in the Wild Rice River based on Phase 3 hydrology (hydrology for low flow events was not updated as part of the Phase 4 analysis).

F-A4.0 BACKGROUND HYDROLOGY OF THE SHEYENNE RIVER

Flows, water surface elevations, and average velocities for the Sheyenne River for the 10-, 50-, 100-, and 500-year events are presented for the local tributary hydrology in Table FA-11. Water surface elevations in the Sheyenne River for the flood events listed above, plotted against the cross section data, and are presented for the local hydrology in Figure FA-17. During Phase 3 it was assumed that the Sheyenne River had a maximum capacity of approximately 4,600 cfs and flows beyond this capacity were assumed to breakout to the Drain 14 and 21C systems. For Phase 4 the HEC-RAS unsteady flow model accounts for these breakout flows to the Drain 14 and 21C systems within the model, and as a result the conveyance capacity of the Sheyenne River is approximately 4,000 cfs.

Table F-A11 Background Existing Local Hydrology of the Sheyenne River at the Approximate Location of the LPP Diversion Channel

Event (yr)	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)*	WSEL (ft)	Velocity (ft/s)*
	Phase 3			Phase 4		
2	1,200	912.71	1.85	-	-	-
5	2,400	916.02	2.47	-	-	-
10	3,400	917.69	2.93	3,843	918.78	2.71
50	4,500	919.35	3.07	4,021	919.08	2.77
100	4,600	919.50	3.06	4,040	919.11	2.77
500	4,600	919.50	3.06	4,059	919.14	2.78

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
Note: Phase 3 values based on steady flow analysis
Note: Phase 4 values based on unsteady flow analysis
*Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

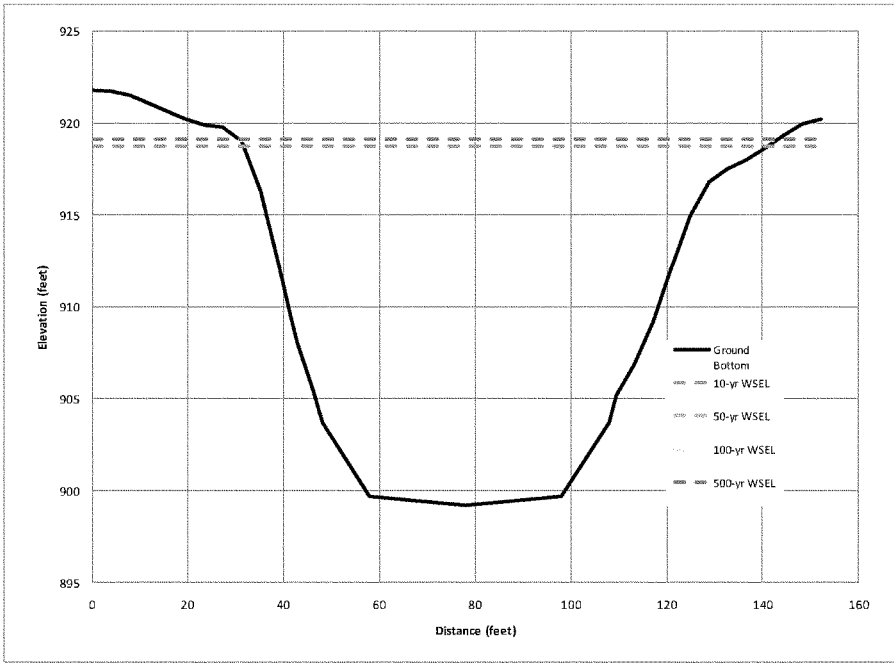

Figure F-A17 Background Existing Local Hydrology Water Surface Elevations in the Sheyenne River at the Approximate Location of the LPP Diversion Channel

Flows, water surface elevations, and average velocities for the Sheyenne River for the 10-, 50-, 100-, and 500-year events are presented for coincidental tributary hydrology in Table FA-12. Water surface elevations in the Sheyenne River for the flood events listed above, plotted against the cross section data, and are presented for coincidental tributary hydrology in Figure FA-18.

Table F-A12 Background Existing Coincidental Hydrology of the Sheyenne River at the Approximate Location of the LPP Diversion Channel

Event (yr)*	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)**	WSEL (ft)	Velocity (ft/s)**
	Phase 3			Phase 4		
2	1,325	913.27	1.89	-	-	-
5	1,935	915.15	2.19	-	-	-
10	2,565	916.31	2.56	1,787	914.50	1.77
50	4,510	919.37	3.07	3,627	918.42	2.62
100	4,600	919.50	3.06	3,921	918.96	2.72
500	4,600	919.50	3.06	4,011	919.10	2.75

* Event recurrence interval refers to the Red River flow record.
Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
Note: Phase 3 values based on steady flow analysis
Note: Phase 4 values based on unsteady flow analysis
**Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

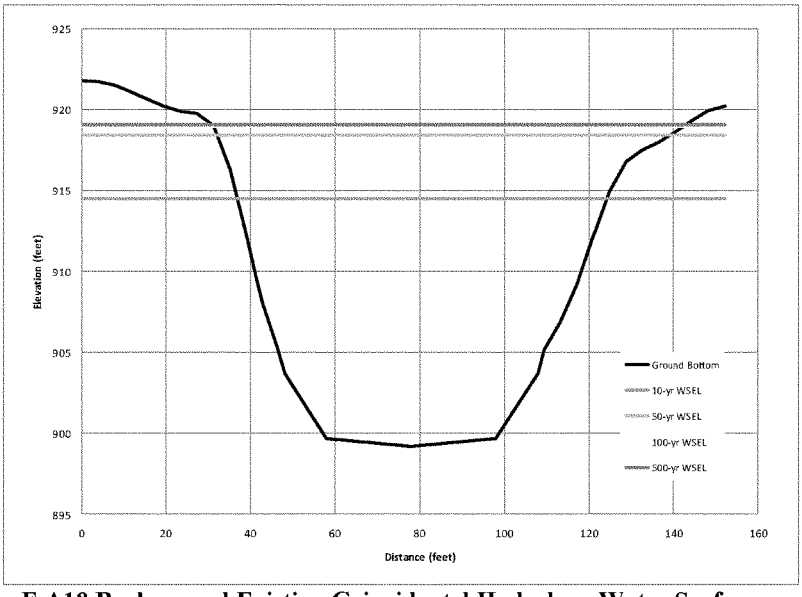

Figure F-A18 Background Existing Coincidental Hydrology Water Surface Elevations in the Sheyenne River at the Approximate Location of the LPP Diversion Channel

Flows and water surface elevations in Phase 3 and Phase 4 can be different. Phase 4 water surface elevations were calculated using a HEC-RAS unsteady flow model that accounts for flow through the floodplain and off channel storage areas and accounts for available storage in the floodplain when computing water surface elevations. The Phase 4 model includes storage areas and storage area connections to provide an accurate depiction of how flows enter and exit the main channel and accounts for conveyance of flow through the floodplain. As compared to Phase 3 that used a rating curve that closely matched the Phase 4 HEC-RAS model for flows contained within the tributary channel, but did not accurately characterize the stage-flow relationship for higher stages when flows leave the main tributary channel and are conveyed through the floodplain.

Actual velocities in the Sheyenne River have been measured at the USGS Gage 05059300, located above the Sheyenne River Diversion near Horace, North Dakota (approximately 1 mile downstream of the Diversion Channel as the crow flies). Figure F-A19 presents average channel velocities plotted against flow in the Sheyenne River near Horace, ND. Figure F-A20 presents velocity distributions in the Sheyenne River measured on April 30, 2009 and June 30, 2009.

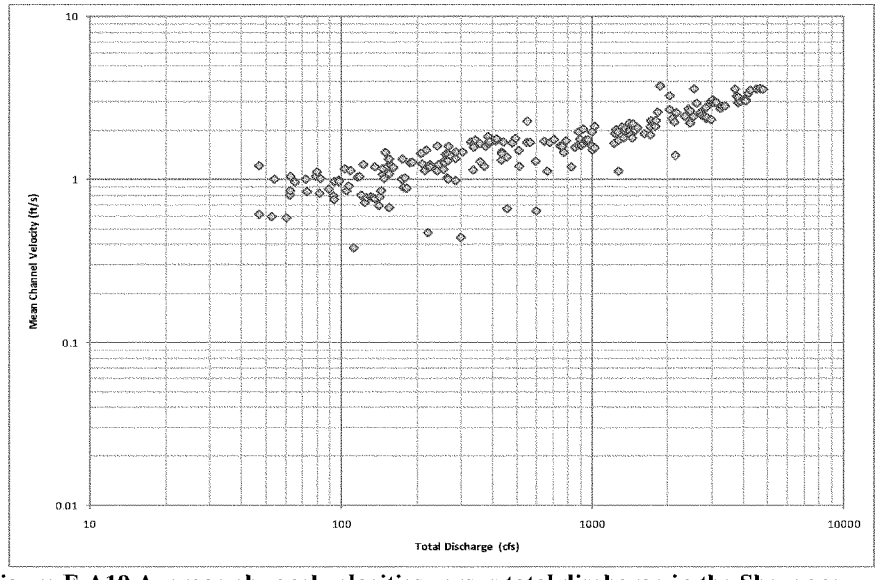

Figure F-A19 Average channel velocities versus total discharge in the Sheyenne River measured at USGS gage 05059300 (Horace, ND)

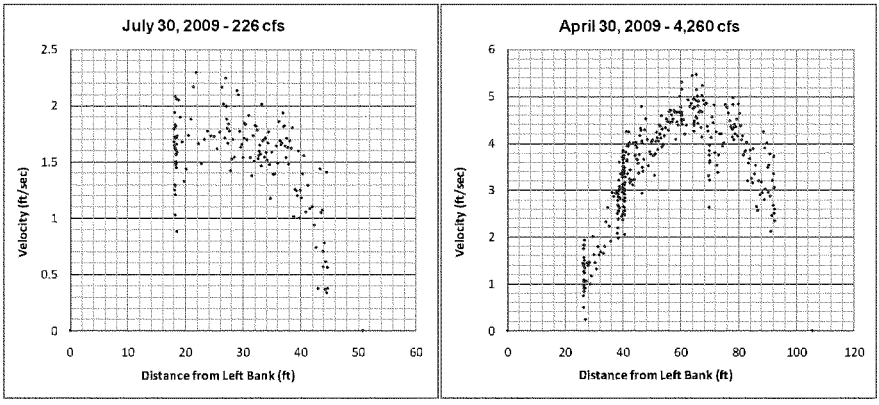

Figure F-A20 Velocity distributions measured in the Sheyenne River at USGS gage 05059300 (Horace, ND)

226 cfs corresponds to the mean annual flow and 4260 cfs corresponds to between a 20-year and 50-year local event in the Sheyenne River based on Phase 3 hydrology. Although approximately 4,000 cfs is the maximum capacity of the Sheyenne River before flows begin to breakout to the Drain 14 and 21C systems to the west.

F-A5.0 BACKGROUND HYDROLOGY OF THE MAPLE RIVER

Flows, water surface elevations, and average velocity for the Maple River for the 10-, 50-, 100-, and 500-year events are presented for the local tributary hydrology in Table FA-13. Water surface elevations in the Maple River for the flood events listed above, plotted against the cross section data, and are presented for the local tributary hydrology in Figure FA-21.

Table F-A13 Background Existing Local Hydrology of the Maple River at the Approximate Location of the LPP Diversion Channel

Event (yr)	Total Flow (cfs) *	River Flow (cfs)	WSEL (ft)	Velocity (ft/s)	River Flow (cfs)**	WSEL (ft)	Velocity (ft/s)**
	Phase 3				Phase 4		
2	970	970	890.73	2.26	-	-	-
5	2,010	2,010	894.18	2.76	-	-	-
10	3,550	3,550	896.82	3.10	4,804	899.07	2.89
50	6,430	5,180	898.77	3.12	5,366	899.94	2.51
100	8,270	5,240	898.83	3.11	5,179	900.11	2.31
500	9,590	5,290	899.14	2.88	4,840	900.36	2.02

* For large flows on the Maple River, the total flow is split between the overland flow, Maple River flow and Drain 14 flows.

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.

Note: Phase 3 values based on steady flow analysis

Note: Phase 4 values based on unsteady flow analysis

**Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

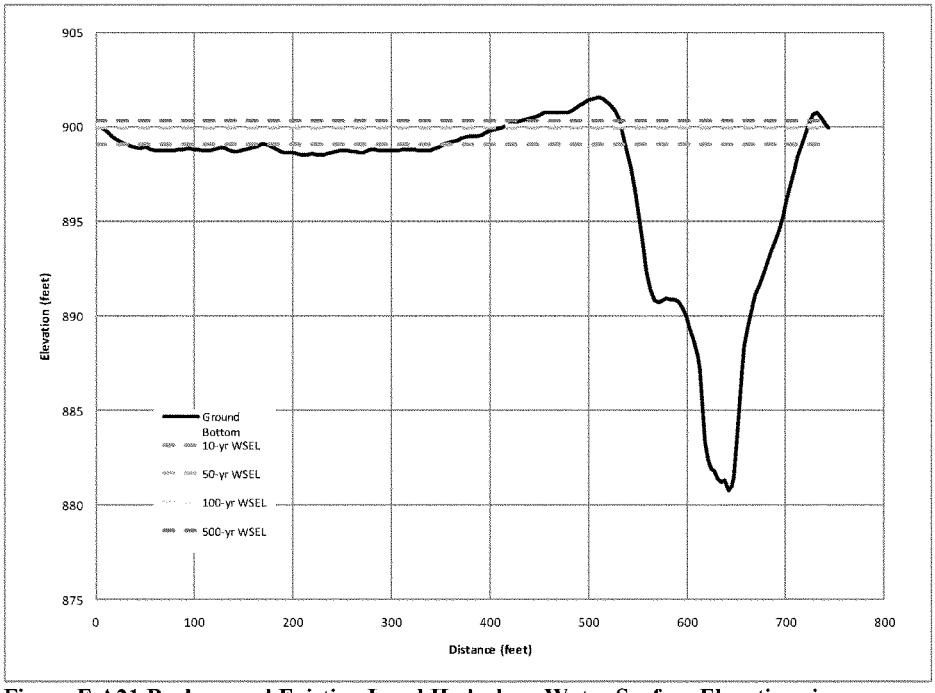

Figure F-A21 Background Existing Local Hydrology Water Surface Elevations in the Maple River at the Approximate Location of the LPP Diversion Chaunel

Flows, water surface elevations, and average velocity for the Maple River for the 10, 50-, 100-, and 500-year events are presented for the coincidental tributary hydrology in Table FA-14. Water surface elevations in the Maple River for the flood events listed above, plotted against the cross section data, and are presented for the coincidental tributary hydrology in Figure FA-22.

Table F-A14 Background Existing Coincidental Hydrology of the Maple River at the Approximate Location of the LPP Diversion Channel

Event (yr)*	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)**	WSEL (ft)	Velocity (ft/s)**
	Phase 3			Phase 4		
2	1,370	891.17	2.99	-	-	-
5	2,000	893.52	3.05	-	-	-
10	2,650	895.36	2.95	5,004	899.08	3.00
50	4,400	898.83	2.61	5,368	899.95	2.50
100	4,925	899.96	2.16	5,228	900.08	2.35
500	5,115	900.34	2.06	5,059	900.21	2.20

*Event recurrence interval refers to the Red River flow record.
Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
Note: Phase 3 values based on steady flow analysis
Note: Phase 4 values based on unsteady flow analysis
**Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

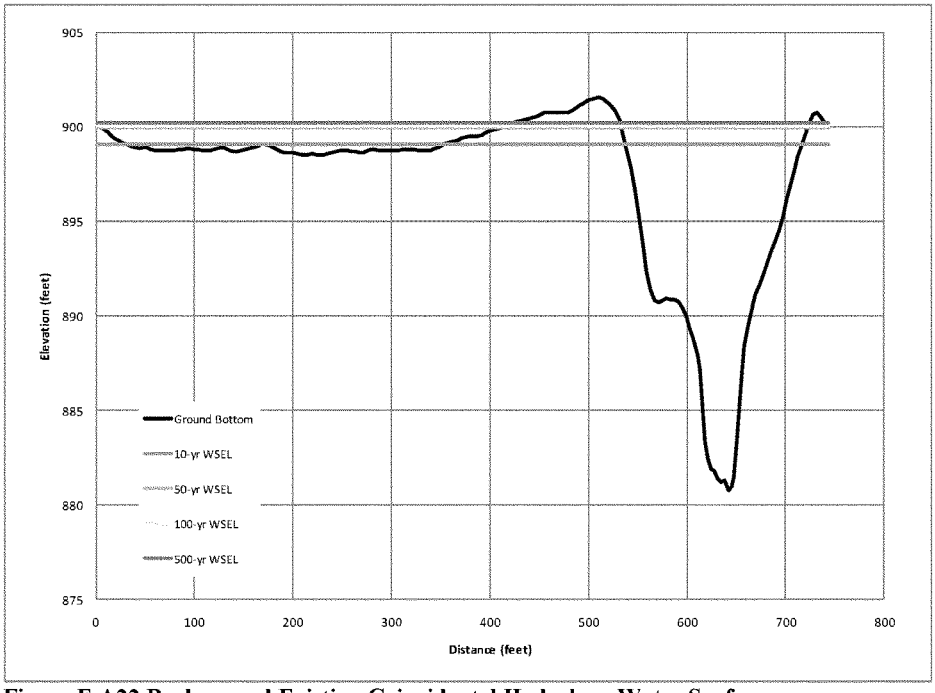

Figure F-A22 Background Existing Coincidental Hydrology Water Surface Elevations in the Maple River at the Approximate Location of the LPP Diversion Channel

Flows and water surface elevations in Phase 3 and Phase 4 can be different. Phase 4 water surface elevations were calculated using a HEC-RAS unsteady flow model that accounts for flow through the floodplain and off channel storage areas and accounts for available storage in the floodplain when computing water surface elevations. The Phase 4 model includes storage areas and storage area connections to provide an accurate depiction of how flows enter and exit the main channel and accounts for conveyance of flow through the floodplain. As compared to Phase 3 that used a rating curve that closely matched the Phase 4 HEC-RAS model for flows contained within the tributary channel, but did not accurately characterize the stage-flow relationship for higher stages when flows leave the main tributary channel and are conveyed through the floodplain.

Actual velocities in the Maple River have been measured at the USGS Gage 05060100 located below Mapleton, North Dakota (approximately five miles upstream of the Diversion Channel as the crow flies). Figure F-A23 presents the average channel velocities plotted against flow in the Maple River. Figure F-A24 presents velocity distributions in the Maple River measured on April 1, 2009 and April 12, 2009.

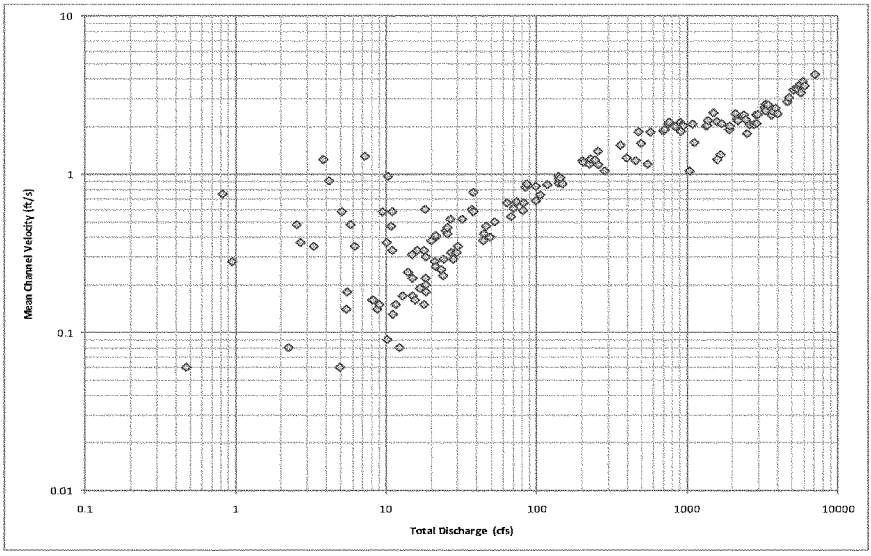

Figure F-A23 Average channel velocities versus total discharge in the Maple River measured at USGS gage 05060100 (Mapleton, ND)

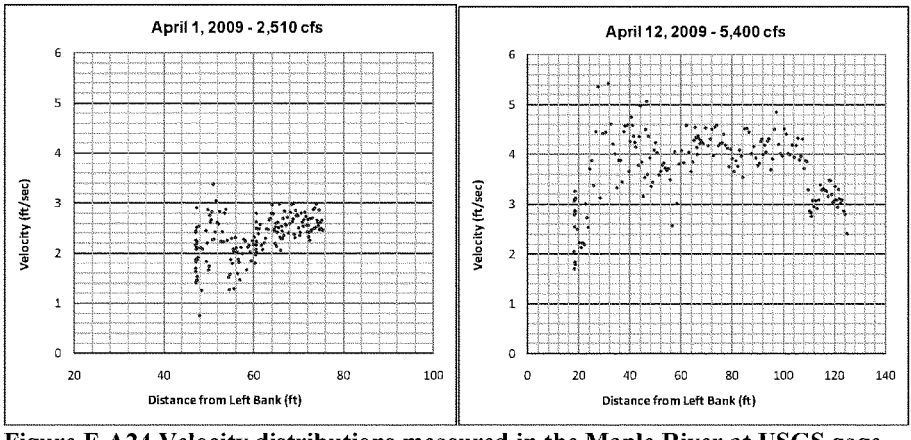

Figure F-A24 Velocity distributions measured in the Maple River at USGS gage 05060100 (Mapleton, ND)

2,510 cfs corresponds to between a 5- and 10-year event and 5,400 cfs corresponds to approximately a 50-year local event in the Maple River.

F-A6.0 BACKGROUND HYDROLOGY OF THE LOWER RUSH RIVER

Flows, water surface elevations, and average velocities for the Lower Rush River for the 10-, 50-, 100-, and 500-year events are presented for the local tributary hydrology in Table FA-15. Water surface elevations in the Lower Rush River for the flood events listed above, plotted against the cross section data, and are presented for the local tributary hydrology in Figure FA-25.

Table F-A15 Background Existing Local Hydrology of the Lower Rush River at the Approximate Location of the LPP Diversion Channel

Event (yr)	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)*	WSEL (ft)	Velocity (ft/s)**
	Phase 3			Phase 4		
2	302	889.07	1.20	-	-	-
5	774	890.97	1.49	-	-	-
10	1,200	891.47	0.98	315	892.93	6.32
50	2,369	892.08	0.72	841	894.67	5.75
100	2,937	892.29	0.70	921	894.85	5.51
500	4,365	892.80	0.68	983	894.98	4.36

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
Note: Phase 3 values based on steady flow analysis
Note: Phase 4 values based on unsteady flow analysis
*Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.
** Phase 4 velocity values are associated with the peak water surface elevation and may not represent the peak velocity. Velocity values taken from the culverts in the Lower Rush River at ShySC131B

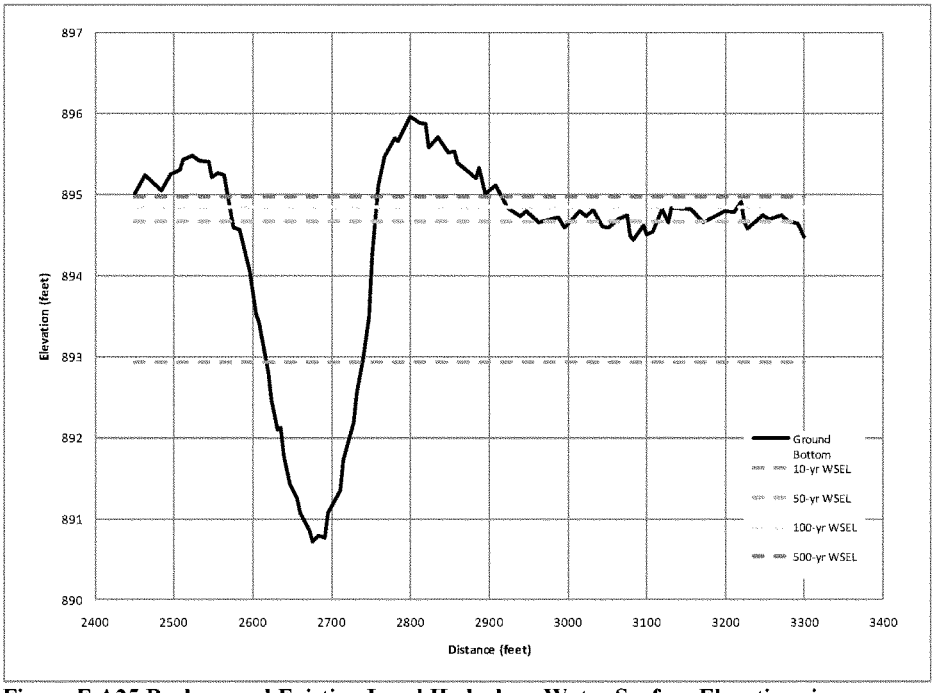

Figure F-A25 Background Existing Local Hydrology Water Surface Elevations in the Lower Rush River at the Approximate Location of the LPP Diversion Channel

Flows and water surface elevations in Phase 3 and Phase 4 can be different. In Phase 3 an assumed flow distribution was used to determine how flows were distributed between the Rush and Lower Rush Rivers. During Phase 4, additional analysis was completed, as discussed in Appendix B, to determine how flow is distributed between the Rush and Lower Rush Rivers.

Flows, water surface elevations, and average velocities for the Lower Rush River for the 10-, 50-, 100-, and 500-year events are presented for the coincidental tributary hydrology in Table FA-16. Water surface elevations in the Lower Rush River for the flood events listed above, plotted against the cross section data, and are presented for the coincidental tributary hydrology in Figure FA-26. The water surface elevations for coincidental hydrology are higher than for the local hydrology, even though the flows for coincidental hydrology are much lower than for the local hydrology. This is due to backwater effects from the Red River of the North.

Table F-A16 Background Existing Coincidental Hydrology of the Lower Rush River at the Approximate Location of the LPP Diversion Channel

Event (yr)*	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)**	WSEL (ft)	Velocity (ft/s)***
	Phase 3			Phase 4		
2	85	886.45	1.19	-	-	-
5	123	887.05	1.13	-	-	-
10	163	889.73	0.53	326	893.02	6.56
50	285	893.89	0.03	797	894.64	5.74
100	355	895.18	0.02	880	894.83	5.24
500	380	895.9	0.02	847	894.74	5.79

*Event recurrence interval refers to the Red River flow record.
Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
Note: Phase 3 values based on steady flow analysis
Note: Phase 4 values based on unsteady flow analysis
**Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.
*** Phase 4 velocity values are associated with the peak water surface elevation and may not represent the peak velocity. Velocity values taken from the culverts in the Lower Rush River at ShySC131B

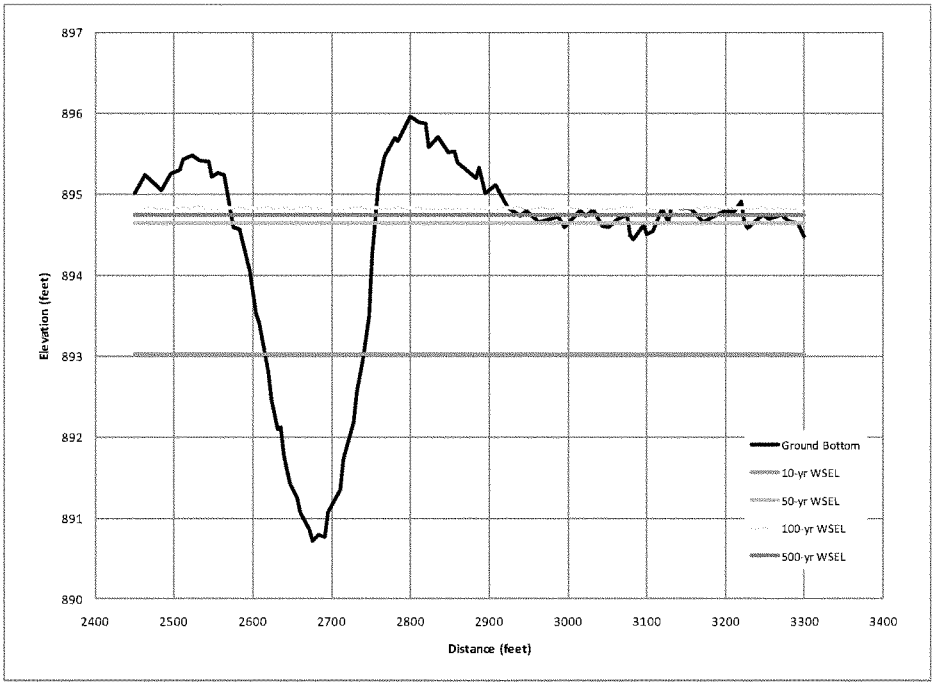

Figure F-A26 Background Existing Coincidental Hydrology Water Surface Elevations in the Lower Rush River at the Approximate Location of the LPP Diversion Channel

The implementation of an improved hydrologic model has decreased the flows and water surface elevations in Phase 4. In Phase 3, the discharges in the Lower Rush River were calculated from a rating curve. The Phase 4 hydrologic model incorporated storage areas and storage area connections outside of the typical river cross sections. The storage areas and their connections provided a more accurate description of the water movement into the Lower Rush River from the surrounding landscape.

F-A7.0 BACKGROUND HYDROLOGY OF THE RUSH RIVER

Flows, water surface elevations, and average velocities for the Rush River for the 10-, 50-, 100-, and 500-year events are presented for the local tributary hydrology in Table FA-17. Water surface elevations in the Rush River for the flood events listed above, plotted against the cross section data, and are presented for the local tributary hydrology in Figure FA-27.

Table F-A17 Background Existing Local Hydrology of the Rush River at the Approximate Location of the LPP Diversion Channel

Event (yr)	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)*	WSEL (ft)	Velocity (ft/s)*
	Phase 3			Phase 4		
2	415	886.50	1.04	-	-	-
5	1,065	889.20	1.13	-	-	-
10	1,650	889.79	0.81	616	899.48	0.28
50	3,258	890.52	0.61	1,222	891.08	0.78
100	4,040	890.76	0.62	1,536	891.36	0.89
500	6,008	891.30	0.65	2,666	891.87	0.99

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
Note: Phase 3 values based on steady flow analysis
Note: Phase 4 values based on unsteady flow analysis
*Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

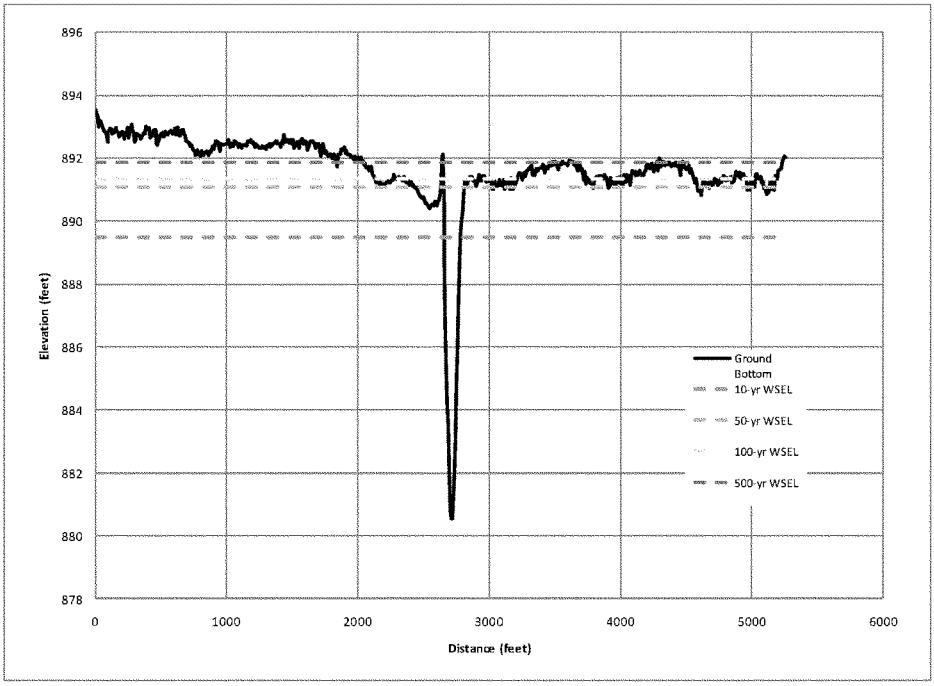

Figure F-A27 Background Existing Local Hydrology Water Surface Elevations in the Rush River at the Approximate Location of the LPP Diversion Channel

Flows, water surface elevations, flow area, and average velocity for the Rush River for the 10-, 50-, 100-, and 500-year events are presented for coincidental tributary hydrology in Table FA-18. Water surface elevations in the Rush River for the flood events listed above, plotted against the cross section data, and are presented for coincidental tributary hydrology in Figure FA-28. The water surface elevations for coincidental hydrology are higher than for the local hydrology, even though the flows for coincidental hydrology are much lower than for the local hydrology. This is due to backwater effects from the Red River of the North.

Table F-A18 Background Existing Coincidental Hydrology of the Rush River at the Approximate Location of the LPP Diversion Channel

Event (yr)*	Flow (cfs)	WSEL (ft)	Velocity (ft/s)	Flow (cfs)**	WSEL (ft)	Velocity (ft/s)**
	Phase 3			Phase 4		
2	170	883.34	1.20	-	-	-
5	246	886.44	0.63	-	-	-
10	326	889.12	0.37	1,433	889.73	1.10
50	570	893.08	0.03	2,887	891.67	1.43
100	710	894.21	0.03	3,137	891.78	1.27
500	760	894.98	0.03	3,892	891.92	1.37

*Note: Event recurrence interval refers to the Red River flow record.
Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
Note: Phase 3 values based on steady flow analysis
Note: Phase 4 values based on unsteady flow analysis
**Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow.

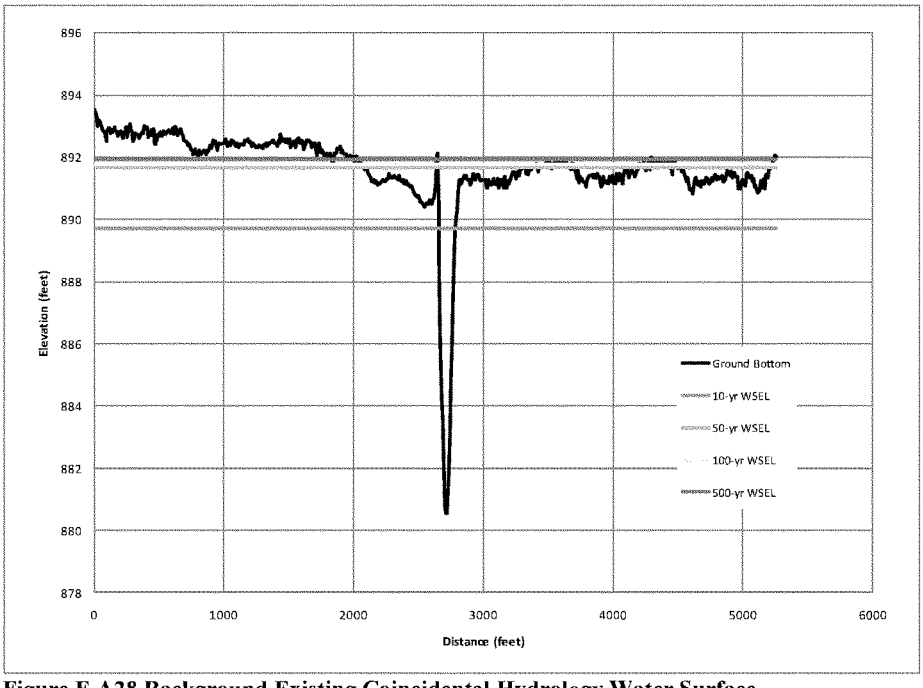

Figure F-A28 Background Existing Coincidental Hydrology Water Surface Elevations in the Rush River at the Approximate Location of the LPP Diversion Channel

The implementation of an improved hydrologic model has decreased the flows and water surface elevations in Phase 4. In Phase 3, the discharges in the Rush River were calculated from a rating curve. The Phase 4 hydrologic model incorporated storage areas and storage area connections outside of the typical river cross sections. The storage areas and their connections provided a more accurate description of the water movement into the Rush River from the surrounding landscape.

F-A8.0 BACKGROUND HYDROLOGY OF WOLVERTON CREEK

Flows, water surface elevations, and average velocities for Wolverton Creek for the 10-, 50-, 100-, and 500-year events are presented for the local tributary hydrology in Table FA-19. Water surface elevations in Wolverton Creek for the flood events listed above, plotted against the cross section data, and are presented for the local tributary hydrology in Figure FA-29.

Table F-A19 Background Existing Local Hydrology of Wolverton Creek at the Approximate Location of Wolverton Creek Control Structure

Event (yr)	Flow (cfs)*	WSEL (ft)	Velocity (ft/s)*
	Phase 4		
10	100	903.77	0.39
50	92	909.87	0.03
100	41	911.77	0.01
500	96	914.12	0.01

Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
Note: Phase 4 values based on unsteady flow analysis
*Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow. During large flood events the maximum water surface elevation is typically controlled by the water surface elevation on the Red River. Therefore the flow and velocity that correspond to the maximum water surface elevation are typically lower than the peak local flow or velocity on Wolverton Creek that occur before the peak water surface elevation.

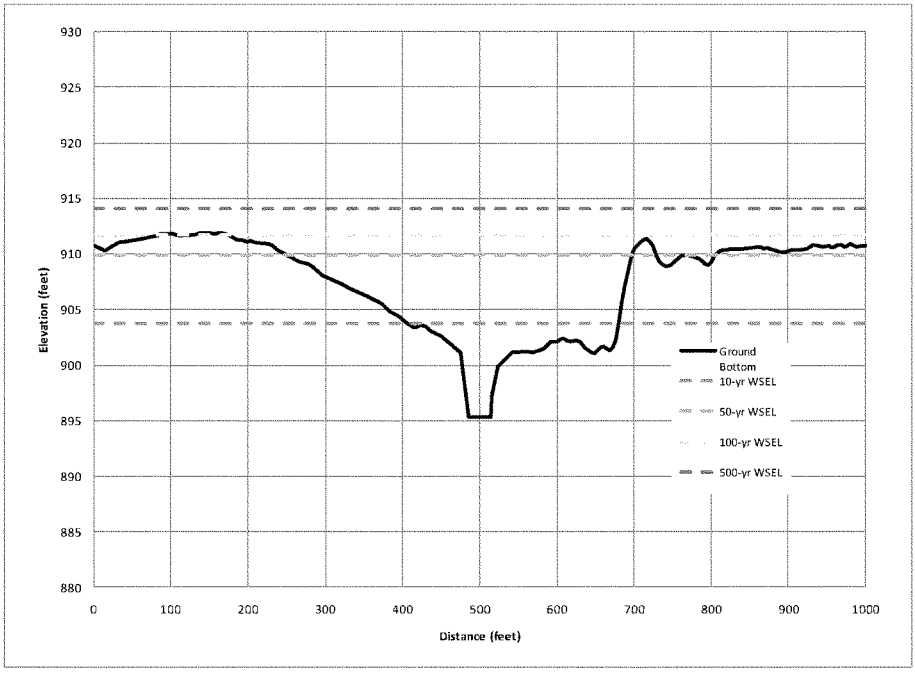

Figure F-A29 Background Existing Local Hydrology Water Surface Elevations in Wolverton Creek at the Approximate Location of the Wolverton Creek control structure (Station 9173)

Flows, water surface elevations, flow area, and average velocity for Wolverton Creek for the 10-, 50-, 100-, and 500-year events are presented for coincidental tributary hydrology in Table FA-20. Water surface elevations in Wolverton Creek for the flood events listed above, plotted against the cross section data, and are presented for coincidental tributary hydrology in Figure FA-30.

Table F-A20 Background Existing Coincidental Hydrology of Wolverton Creek at the Approximate Location of the LPP Diversion Channel

Event (yr)*	Flow (cfs)**	WSEL (ft)	Velocity (ft/s)**
	Phase 4		
5	-	-	-
10	325	907.42	0.89
50	846	912.98	0.14
100	997	913.79	0.14
500	952	914.88	0.11

*Note: Event recurrence interval refers to the Red River flow record.
Note: Phase 4 values are taken from the cross section and do not account for flows through the adjacent storage areas.
**Phase 4 values are associated with the peak water surface elevation and may not represent the peak flow. During large flood events the maximum water surface elevation is typically controlled by the water surface elevation on the Red River. Therefore the flow and velocity that correspond to the maximum water surface elevation are typically lower than the peak local flow or velocity on Wolverton Creek that occur before the peak water surface elevation.

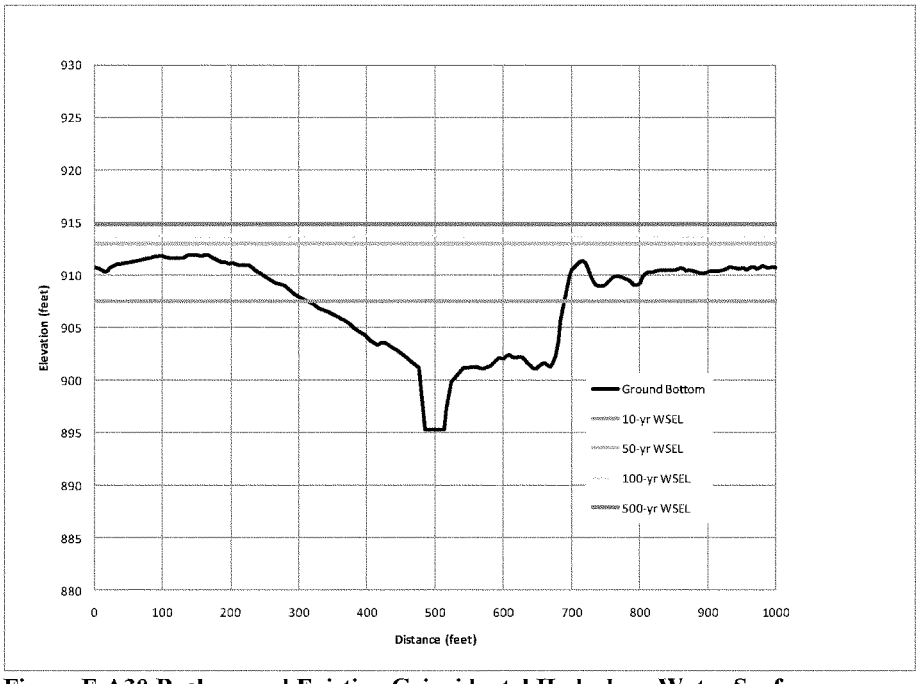

Figure F-A30 Background Existing Coincidental Hydrology Water Surface Elevations in Wolverton Creek at the Approximate Location of the Wolverton Creek control structure (Station 9173)

RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT B – ALTERNATIVES MATRIX FOR CONTROL STRUCTURE ON RED RIVER OF THE NORTH AND INLET WEIR OF DIVERSION CHANNEL

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

By: Barr Engineering Co.

FINAL – Version February 28, 2011

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HYDRAULIC STRUCTURES

EXHIBIT B – ALTERNATIVES MATRIX FOR CONTROL STRUCTURE ON RED RIVER OF THE NORTH AND INLET WEIR OF DIVERSION CHANNEL

F-B1.0 ALTERNATIVES MATRIX FOR CONTROL STRUCTURES AND INLET WEIRS

Several design concepts for the Red River Control Structure and Diversion Inlet Structure were considered in this feasibility analysis. The advantages and disadvantages of eight (8) combinations of Red River Control Structure and Diversion Inlet Structure are presented in the following table, referred to as an "Alternatives Matrix". The advantages and disadvantages described in the alternatives matrix include hydraulic performance, flood flows, low flows, potential environmental impacts, permitting, and operation and maintenance. The combinations of structures included in the alternatives matrix are listed below:

1.) Control structure on the Red River with gated openings and operable gates from above, passive weir inlet to diversion; this is the design recommended as of 02-28-2011

2.) Control structure on the Red River with gated openings and operable gates from above, passive stepped weir inlet to diversion

3.) Control structure on the Red River with ungated low box culverts and operable gates from above, passive weir inlet to diversion

4.) Control structure on the Red River with open area and natural channel bottom, operable gates above, and passive weir inlet to diversion

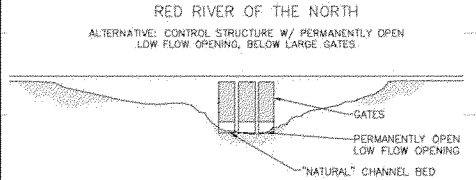
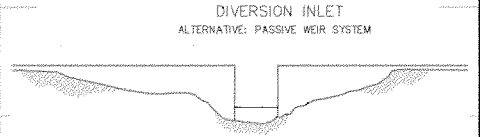
5.) Weir Control Structure on Diversion, with no structure on Red River

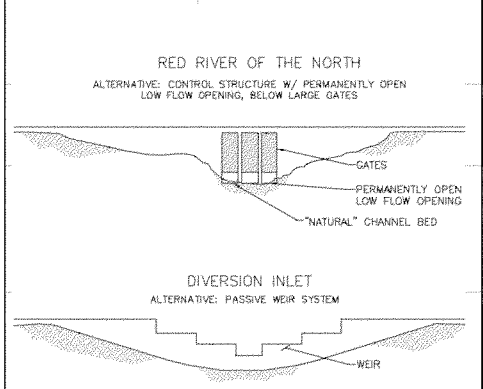
6.) Weir Control Structure on Diversion, with no structure on Red River (longer weir)

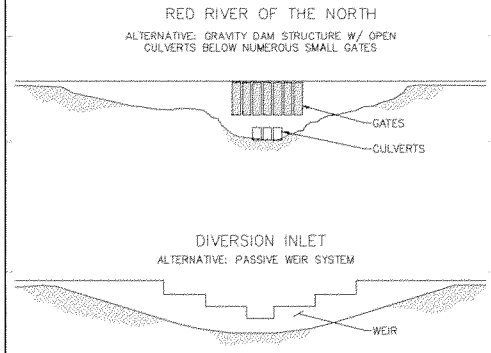
7.) Weir Control Structure on Diversion with gates below 5-year crest elevation, with no structure on Red River

8.) Channel constriction (ungated) on the Red River and operable Diversion Inlet (gated)

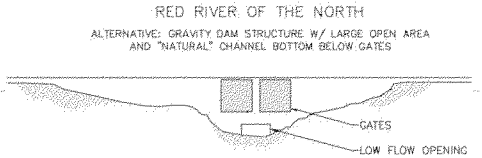
Table F-B1 Red River Control Structure and Diversion Inlet Alternatives Matrix

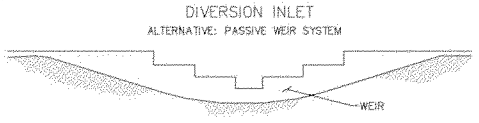
Alt. No.	Hydraulic Structure	Structure Alternative	Design Consideration	Advantages	Disadvantages	Risks and Failure Mode
1	Diversion Inlet	Control structure on the Red RIver with gated openings and operable gates from above, passive weir inlet to diversion. **This is the design recommended as of 02-28-2011.**	**Hydraulic Performance:**	High level of control of upstream Red River water surface profile; ability to stage water upstream of control structures		
sketch below:  		**Flood Flows:**	Provides floodplain access of Red River channel upstream of structure; limits flood flows into protected area		Risk of gate malfunction and flooding of protected area; Failure mode still diverts water around the protected area. Bulkhead slots provide backup
			Low Flows:	Passive transmission of low flows into protected area through low openings		
			Potential Environmental Impacts:		Periodic high velocities could pose environmental impact issues	
			Permitting:		Building a structure on the Red River poses regional and international permitting issues	
			Operation and Maintenance:		Ongoing operation and maintenance costs for superstructure adding to life-cycle cost	Ice/gate interaction, debris fouling, ice fouling

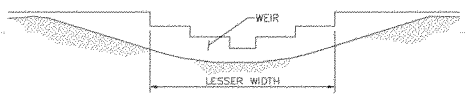
Alt. No.	Hydraulic Structure	Structure Alternative	Design Consideration	Advantages	Disadvantages	Risks and Failure Mode
2	Diversion Inlet	Control structure on the Red River with ungated openings and operable gates from above, stepped passive weir inlet to diversion. **This is the design recommended as of 12-31-2009.**	**Hydraulic Performance:**	High level of control of upstream Red River water surface profile; ability to match existing rating curve on the Red River	Unable to stage sufficient water upstream of control structure	
sketch below: 		**Flood Flows:**	Provides floodplain access of Red River channel upstream of structure; limits flood flows into protected area		Risk of gate malfunction and flooding of protected area; Failure mode still diverts water around the protected area. Bulkhead slots provide backup
			Low Flows:	Passive transmission of low flows into protected area through low openings		
			Potential Environmental Impacts:		Periodic high velocities could pose environmental impact issues	
			Permitting:		Building a structure on the Red River poses regional and international permitting issues	
			Operation and Maintenance:		Ongoing operation and maintenance costs for superstructure adding to life-cycle cost	Ice/gate interaction, debris fouling, ice fouling

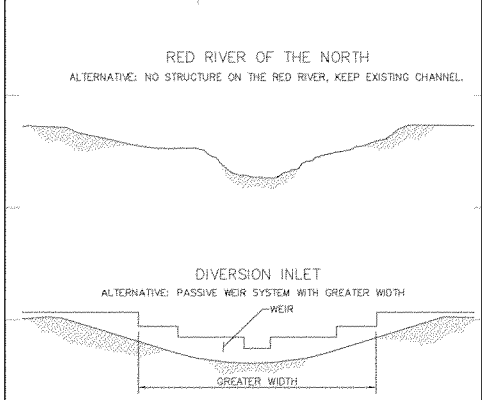
Alt. No.	Hydraulic Structure	Structure Alternative	Design Consideration	Advantages	Disadvantages	Risks and Failure Mode
3	Diversion Inlet	Control structure on the Red River with ungated low box culverts and operable gates above, passive weir inlet to diversion.	**Hydraulic Performance:**	High level of control of upstream Red River water surface profile; ability to match existing rating curve on the Red River	Unable to stage sufficient water upstream of control structure	
sketch below: 		**Flood Flows:**	Provides floodplain access of Red River channel upstream of structure; limits flood flows into protected area		Risk of gate malfunction and flooding of protected area; Failure mode still diverts water around the protected area
			Low Flows:	Passive transmission of low flows into protected area through culverts		
			Potential Environmental Impacts:		Box culverts replacing channel bed could pose environmental impact issues; periodic high velocities	
			Permitting:		Building a structure on the Red River poses regional and international permitting issues	
			Operation and Maintenance:		Ongoing operation and maintenance costs for superstructure adding to life-cycle cost	Ice/gate interaction, debris fouling, ice fouling

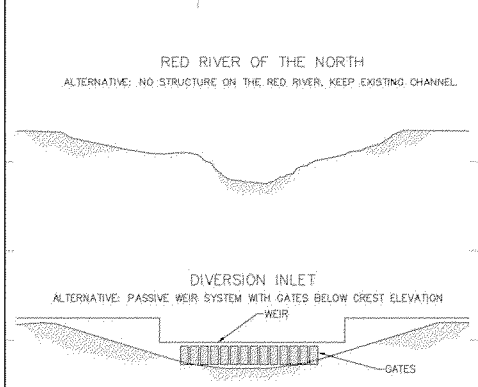
Alt. No.	Hydraulic Structure	Structure Alternative	Design Consideration	Advantages	Disadvantages	Risks and Failure Mode
4	Diversion Inlet	Control structure on the Red River with open area and natural channel bottom, operable gates above, and passive weir inlet to diversion	**Hydraulic Performance:**	High level of control of upstream Red River water surface profile; ability to match existing rating curve on the Red River	High velocities through low opening could result in erosion impacting structural integrity	
sketch below:			**Flood Flows:**	Provides floodplain access of Red River channel upstream of structure; limits flood flows into protected area		Risk of gate malfunction and flooding of protected area; Failure mode still diverts water around the protected area
			Low Flows:	Passive transmission of low flows into protected area through low opening		
			Potential Environmental Impacts:	Potentially less impact to channel than option 1 during lower flow velocities	Periodic high velocities could pose environmental impact issues and may affect channel bed	
			Permitting:		Building a structure on the Red River poses regional and international permitting issues	
			Operation and Maintenance:		Ongoing operation and maintenance costs for superstructure adding to life-cycle cost	Ice/gate interaction, debris fouling, ice fouling



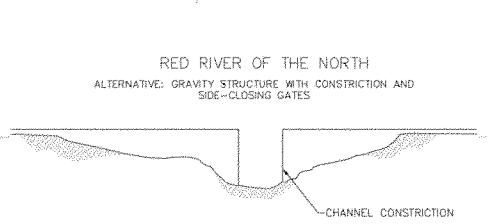


Alt. No.	Hydraulic Structure	Structure Alternative	Design Consideration	Advantages	Disadvantages	Risks and Failure Mode
5	Diversion Inlet	Weir Control Structure on Diversion, with no structure on Red River	**Hydraulic Performance:**		No good control of upstream flows passing into protected side.	
sketch below: RED RIVER OF THE NORTH ALTERNATIVE: NO STRUCTURE ON THE RED RIVER, KEEP EXISTING CHANNEL DIVERSION INLET ALTERNATIVE: PASSIVE WEIR SYSTEM WITH LESSER WIDTH  			**Flood Flows:**		Impact to water surface profiles in the Red River and the channel's access to the floodplain upstream	Benefits significantly reduced; B/C ratio much less than 1 (i.e. no project) due to limitations of flow sent to diversion channel resulting in higher flows in protected area
			Low Flows:	Passive transmission of low flows into protected area through existing channel		
			Potential Environmental Impacts:	Less immediate impact to the existing Red River channel bed	Possible environmental impact to upstream floodplain due to impacts on water surface profile	
			Permitting:	Not building a structure on the Red River poses fewer regional and international permitting issues		
			Operation and Maintenance:	Less costly to operate and maintain than structures that span Red River		Ice fouling, debris fouling at diversion inlet

Alt. No.	Hydraulic Structure	Structure Alternative	Design Consideration	Advantages	Disadvantages	Risks and Failure Mode
6	Diversion Inlet	Weir Control Structure on Diversion, with no structure on Red River (longer weir)	**Hydraulic Performance:**		No good control of upstream flows passing into protected side.	
sketch below: 			**Flood Flows:**		Impact to water surface profiles in the Red River and the channel's access to the floodplain upstream	Benefits significantly reduced; risk of B/C ratio much less than 1 (i.e. no project) due to limitations of flow sent to diversion channel resulting in higher flows in protected area
			Low Flows:	Passive transmission of low flows into protected area through existing channel	Possible environmental impact to upstream floodplain due to impacts on water surface profile	
			Potential Environmental Impacts:	Less immediate impact to the existing Red River channel bed		
			Permitting:	Not building a structure on the Red River poses fewer regional and international permitting issues		
			Operation and Maintenance:	Less costly to operate and maintain than structures that span Red River		Ice fouling, debris fouling at diversion inlet

Alt. No.	Hydraulic Structure	Structure Alternative	Design Consideration	Advantages	Disadvantages	Risks and Failure Mode
7	Diversion Inlet	Weir Control Structure on Diversion with gates below 5-year crest elevation, with no structure on Red River	**Hydraulic Performance:**	More control of upstream water surface profile than options 5 or 6.	Low level of control of upstream Red River water surface profile. No good control of flows passing into protected side.	
sketch below: 			**Flood Flows:**		Potential Impact to water surface profiles in the Red River and the channel's access to the floodplain upstream	Benefits significantly reduced; risk of B/C ratio less than 1 (i.e. no project) due to limitations of flow sent to diversion channel resulting in higher flows in protected area. Failure of diversion structure results in catastrophic flooding
			Low Flows:	Passive transmission of low flows into protected area through existing channel		
			Potential Environmental Impacts:	Less immediate impact to the existing Red River channel bed	Possible environmental impact to upstream floodplain due to impacts on water surface profile	
			Permitting:	Not building a structure on the Red River poses fewer regional and international permitting issues		
			Operation and Maintenance:		More costly than passive weir system on diversion inlet	Ice/gate interaction, debris fouling, ice fouling at diversion inlet

Alt. No.	Hydraulic Structure	Structure Alternative	Design Consideration	Advantages	Disadvantages	Risks and Failure Mode
8	Diversion Inlet	Channel Constriction (ungated) on the Red River and operable Diversion Inlet (gated)	**Hydraulic Performance:**	More control of upstream water surface profile than options 5 or 6.	Low level of control of upstream Red River water surface profile. No good control of flows passing into protected side.	
sketch below:			**Flood Flows:**	Provides some floodplain access of Red River channel upstream of structure		Benefits significantly reduced; risk of B/C ratio less than 1 (i.e. no project). Risk of gate malfunction and flooding of protected area.; failure mode increases risk of flood flows into the protected area
			Low Flows:	Passive transmission of low flows into protected area through open area		
			Potential Environmental Impacts:	Less immediate impact to the existing Red River channel bed	Possible environmental impact to upstream floodplain due to impacts on water surface profile	
			Permitting:	Leaving an unrestricted portion of the Red River may reduce perceptions it is a dam	Building a structure on the Red River poses regional and international permitting issues	
			Operation and Maintenance:	Perhaps less costly to operate and maintain than structures that span the entire Red River	Costly to operate and maintain superstructure adding to life-cycle cost	Ice/gate interaction, debris fouling, ice fouling at diversion inlet



RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT C – ALTERNATIVES MATRIX FOR DIVERSION STRUCTURES AT NORTH DAKOTA TRIBUTARIES

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

By: Barr Engineering Co.

FINAL - February 28, 2011

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT C– ALTERNATIVES MATRIX FOR DIVERSION STRUCTURES AT NORTH DAKOTA TRIBUTARIES

This exhibit presents 1) a parametric study of the Maple and Sheyenne River Diversion Channel Crossing Structures and 2) Alternatives matrix for Diversion Structures at the North Dakota tributaries. The parametric study was completed in May 2010, it is being published for the first time in this Phase 3 report, and it discusses the effect of changing a variety of structural dimensions on the hydraulics of the Sheyenne and Maple River structures. The alternatives matrix was completed in December 2009 and it was printed as Exhibit E of Appendix F of the Phase 2 report submitted on January 6th, 2010.

F-C1.0 PARAMETRIC STUDY OF THE MAPLE AND SHEYENNE RIVER DIVERSION CHANNEL CROSSING STRUCTURES

The following information was previously presented as Exhibit C of Appendix F of the Phase 3 report submitted on August 6th, 2010, and is included here for completeness.

The parametric study presented here was used in May 2010 to determine the effect of changing a variety of structural dimensions of the Maple and Sheyenne aqueduct structures on the hydraulics of the LPP Diversion Channel. The goal of this parametric study was to assist the coordination between the structural design and hydraulic design teams.

F-C1.1 INPUTS

The parametric study was completed using the May 24, 2010 HEC-RAS model of the North Dakota East 35k alignment - Phase 3.0 Year 0 Hydrology model. The 5-, 10-, 20-, 50-, 100-, 200-, and 500-year events in the Red River of the North were modeled.

A variety of structural dimensions were changed in the model:

1. The width of the Diversion Channel at the aqueduct varied between 250 feet, 275 feet, 300 feet and 350 feet.

2. The thickness of the piers in the Diversion Channel section at the aqueduct crossing varied between 1 foot, 2 feet, 3 feet, and 4 feet.

3. The spacing of the piers (centerline to centerline) in the Diversion Channel section at the aqueduct crossing varied between 25 feet, 30 feet, 30.5 feet, 37.5 feet, and 43.75 feet.

4. The length of the Diversion Channel underneath the aqueduct varied between 65 feet and 80 feet.

5. The thickness of the aqueduct (bottom) slab carrying the Maple or Sheyenne River over the Diversion Channel varied between 1.5 feet, 2 feet, 3 feet, 3.5 feet, and 4 feet.

6. The elevation of the top of the Maple River aqueduct structure varied between 892.7, 893.7, 894.7, and 898.7 feet.

The Diversion Channel transitions into and out of the aqueduct crossing (i.e., the radius of the vertical wing wall, distance between cross sections, location of levees, etc.) were changed in accordance to the modification with the structural dimensions listed above.

It should be noted that the May 24, 2010 model is not the final Phase 3 HEC-RAS model used for assessing impacts. The May 24, 2010 model included a variety of parameters that were changed for the Phase 3 model. These outdated parameters included a 125 foot bottom width upstream of the Sheyenne River, and the levees along the Connecting Channel were located at the bank station. The three gates on the Red River Control Structure were only 40 feet wide and were optimized during the work in April 2010. The HEC-RAS model was updated two days later on May 26, 2010 to have a 100 foot bottom width upstream of the Sheyenne River and a 50 foot levee setback from the bank station along the Connecting Channel. The gates were subsequently changed to 50 feet wide on May 27, 2010 and gate operations were optimized on June 1, 2010. The parametric study was completed before the May 26, 2010 model was available. Additional changes to the hydrology and some cross sections were made in the June 9, 2010 model. Therefore, the results described in this parametric study are not final results. However, these results allow to weight the relative changes in hydraulics based on changes in structural dimensions.

F-C1.2 RESULTS

The results of 22 different model runs are shown in Table F-C1 for the Sheyenne River crossing and Table FC-2 for the Maple River crossing. The main conclusions from this parametric study can be summarized as:

1. Widening the Diversion Channel from 250 feet to 350 feet while holding all other variables the same, results in decreasing head loss (by approximately 55 to 60%) and decreasing velocities (by approximately 30 to 35%) in the Diversion Channel at the tributary crossing. (See model runs 1, 4 and 13).

2. Increasing the thickness of the piers in the Diversion Channel from 1 foot to 3 feet while holding all other variables the same, results in increasing head loss (by approximately 15 to 20%) and increasing velocities (by less than 10%) in the Diversion Channel at the tributary crossing. (See model runs 1 and 3).

3. Increasing the spacing of the piers in the Diversion Channel from 25 feet to 30.5 feet while holding all other variables the same, results in decreasing head loss (by less than 10%) and decreasing velocities (by less than 10%) in the Diversion Channel at the tributary crossing. (See model runs 18, 19, 20. and 21).

4. Increasing the length of the Diversion Channel underneath the aqueduct from 65 feet and 80 feet has no effect on head loss and velocities in the Diversion Channel at the tributary crossing. (See model runs 4, 5, 15 and 16).

5. Increasing the thickness of the aqueduct (bottom) slab carrying the Maple or Sheyenne River over the Diversion Channel from 1.5 feet to 3.5 feet while holding all other variables the same, results in increasing head loss (by approximately 25 to 40%) and increasing velocities (by approximately 10 to 15%) in the Diversion Channel at the tributary crossing. (See model runs 1, 2, 4 and 11).

6. Increasing the top of the Maple River aqueduct by 1 foot to an elevation of 893.7 would prevent the 50-year event in the Diversion Channel from overtopping the aqueduct (i.e., the 100-year event would overtop the structure) but would increase the head loss (by approximately 15 to 20%) and increase the velocities (by less than 10%) in the Diversion Channel at the Maple River crossing. (See model runs 6, 7, 15 and 16).

7. Increasing the top of the Maple River aqueduct by 6 feet to an elevation of 898.7 would prevent the 500-year event from overtopping the Maple River aqueduct but would significantly increase the head loss (by approximately 75%) and the velocities (by approximately 25 to 30%) in the Diversion Channel at the Maple River crossing. (See model runs 4 and 9).

8. Increasing the elevation of the top of the Maple River aqueduct has little effect on the velocities and head loss at the Sheyenne River crossing, although the water surface elevations at the Sheyenne River crossing in the Diversion Channel may increase slightly. (See model runs 8, 9, 14, 15, 16 and 17).

Table FC-1 Summary of Parametric Study Results for the Diversion Channel Crossing of the Sheyenne River

Run	Diversion Channel Width (ft)	Pier Width (ft)	Pier Spacing (ft)	Length of Diversion Channel (ft)	Thickness of Slab (ft)	Elevation of Top of Maple Structure (ft)	500-year W5EL Upstream of Structure (ft)	500-year WSEL Down-stream of Structure (ft)	500-year Head Loss Across the Structure (ft)	500-year Velocity at Structure (ft/s)
1	250	1	25	65	1.5	892.7	911.75	909.95	1.80	9.37
2	250	1	25	65	3.5	892.7	912.50	910.12	2.38	10.74
3	250	3	25	65	1.5	892.7	912.12	909.97	2.15	10.11
4	300	1	25	65	1.5	892.7	911.19	909.94	1.25	7.83
5	300	1	25	80	1.5	892.7	911.19	909.94	1.25	7.83
6	300	4	37.5	65	4	893.7	912.20	910.05	2.15	9.90
7	300	2	25	65	2	893.7	911.42	909.95	1.47	8.38
8	300	1	25	65	1.5	894.7	911.22	909.98	1.24	7.83
9	300	1	25	65	1.5	898.7	911.28	910.03	1.25	7.83
10	300	3	25	65	1.5	892.7	911.44	909.94	2.50	8.47
11	300	1	25	65	3.5	892.7	911.69	909.97	1.72	8.99
12	300	3	25	65	3.5	892.7	912.02	909.96	2.06	9.72
13	350	1	25	65	1.5	892.7	910.84	909.93	0.91	6.72
14	350	4	43.75	65	4	892.7	911.50	909.97	1.53	8.39
15	350	4	43.75	65	4	893.7	911.49	909.97	1.52	8.36
16	350	4	43.75	80	4	893.7	911.49	909.97	1.52	8.36
17	350	4	43.75	65	4	894.7	911.51	909.99	1.52	8.36
18	275	2	25	65	2	893.7	911.74	909.99	1.75	9.15
19	275	2	30.5	65	2	893.7	911.68	909.98	1.70	9.01
20	275	3	25	65	3	893.7	912.32	910.02	2.30	10.2
21	275	3	30.5	65	3	893.7	912.15	910.02	2.13	9.96
22	300	3	30	65	3	893.7	911.80	910.00	1.80	9.17

Table FC-2 Summary of Parametric Study Results for the Diversion Channel Crossing of the Maple River

Run	Diversion Channel Width (ft)	Pier Width (ft)	Pier Spacing (ft)	Length of Diversion Channel (ft)	Thickness of Slab (ft)	Elevation of Top of Maple Structure (ft)	500-year WSEL Upstream of Structure (ft)	500-year WSEL Down-stream of Structure (ft)	500-year Head Loss Across the Structure (ft)	500-year Velocity at Structure (ft/s)	Event that Overtops the Maple Structure (year)
1	250	1	25	65	1.5	892.7	897.58	895.83	1.75	9.32	50
2	250	1	25	65	3.5	892.7	898.40	896.17	2.23	10.28	50
3	250	3	25	65	1.5	892.7	897.84	895.82	2.02	9.87	50
4	300	1	25	65	1.5	892.7	897.10	895.89	1.21	7.78	50
5	300	1	25	80	1.5	892.7	897.10	895.89	1.21	7.78	50
6	300	4	37.5	65	4	893.7	898.38	896.22	2.16	9.95	100
7	300	2	25	65	2	893.7	897.50	895.86	1.64	8.80	100
8	300	1	25	65	1.5	894.7	897.49	895.88	1.61	8.76	100
9	300	1	25	65	1.5	898.7	898.00	895.88	2.12	9.85	>500
10	300	3	25	65	1.5	892.7	897.09	895.88	1.21	7.78	50
11	300	1	25	65	3.5	892.7	897.52	895.87	1.65	8.83	50
12	300	3	25	65	3.5	892.7	897.51	895.87	1.64	8.83	50
13	350	1	25	65	1.5	892.7	896.79	895.91	0.88	6.63	100
14	350	4	43.75	65	4	892.7	897.32	895.91	1.41	8.11	50
15	350	4	43.75	65	4	893.7	897.55	895.88	1.67	8.73	100
16	350	4	43.75	80	4	893.7	897.55	895.88	1.67	8.73	100
17	350	4	43.75	65	4	894.7	897.80	895.88	1.92	9.31	100
18	275	2	25	65	2	893.7	897.78	895.85	1.93	9.55	100
19	275	2	30.5	65	2	893.7	897.73	895.86	1.87	9.44	100
20	275	3	25	65	3	893.7	898.20	895.84	2.36	10.45	100
21	275	3	30.5	65	3	893.7	898.12	895.85	2.27	10.27	100
22	300	3	30	65	3	893.7	897.82	895.87	1.95	9.51	100

F-C2.0 ALTERNATIVES MATRIX FOR DIVERSION STRUCTURES AT NORTH DAKOTA TRIBUTARIES

The following information was previously presented as Exhibit C of Appendix F of the Phase 3 report submitted on August 6th, 2010, and is included here for completeness.

Several design concepts for the tributary crossing structures were investigated during Phase II (November and December of 2009 with the submittal on December 31, 2009) to determine if they were feasible. The advantages and disadvantages of eight (8) alternatives and the original Phase I recommended design of the tributary crossing structures for the Sheyenne, Maple, Lower Rush and Rush Rivers are presented in the following text. Alternative deigns analyzed during Phase II were intended to:

- Reduce the construction costs
- Allow fish passage
- Address winter freezing
- Address ice flow in the Diversion Channel during flood events

During Phase I, which was submitted on August 31, 2009, concept level designs of hydraulic structures for tributaries of the Red River of the North that cross the proposed Diversion Channel were developed. Crossings were developed based on the primary design constraint that the head loss across each crossing should not be greater than 0.5-ft. As a result, a closed box culvert aqueduct-type structure was recommended at each tributary crossing. However, during Phase II the primary design constraint has changed such that an increase of the water surface elevation at the inlet to the Diversion Channel in the Red River is not allowed. This shift in design criteria allows for additional alternate crossing designs that were not possible during Phase I of the study.

Additionally, during Phase I it was assumed that the Diversion Channel would be dry (i.e., the Diversion Channel would only convey flow during flood events). However, for Phase II the US Army Corps of Engineers (USACE) and the City of Fargo have concluded that the Diversion Channel will not necessarily be dry, and will almost always have some water in it. The water will most likely be due to inflows from drain tiles of surrounding fields. This allows consideration of passing less flow into the protected area. During Phase I, a minimum flow corresponding to the 5-year flood event in the tributary would be conveyed into the protected area. During the Phase II analysis in November and December of 2009, it was agreed upon with the USACE that a minimum of the 2-year event in the tributary would be conveyed into the protected area. By reducing the tributary flow conveyed into the protected area from the 5-year flow to the 2-year flow, additional flow will be directed into the Diversion Channel. This reduction in flow to the protected area will result in smaller crossing structures and a larger spillway which will reduce the overall cost of the crossing structure. Similar to the Phase I design, it was assumed that no flow from the ND Diversion Channel into the protected area would be allowed.

Following is a description of the design developed during Phase I of the study, and alternatives that were analyzed in Phase II of the study. These alternatives and combinations of these alternatives were modeled in HEC-RAS. The optimum Phase II designs are described in the text of Appendix F.

F-C2.1 PHASE I PROPOSED DESIGN – AQUEDUCT IN TRIBUTARY AND TRANSITION IN DIVERSION CHANNEL – ALL TRIBUTARIES.

The Phase I design consisted of an aqueduct represented by enclosed box culverts to pass the 5-year flow from the tributaries into the protected area over the Diversion Channel, and a weir spillway to divert waters from the tributary into the ND Diversion Channel. The total height of the box culvert is 6.25 feet with an effective flow depth of 3.5 feet. This translates to very high velocities in the tributary box culvert and a higher risk of ice jams in the tributary. This design was proposed for all five major tributary crossings: the Wild Rice River, Sheyenne River, Maple River, Lower Rush River and Rush River. At the crossing, the Diversion Channel will be concrete lined to reduce channel roughness and increase velocities to compensate for the reduction in flow area without resulting in greater water depths. The transition from the excavated Diversion Channel to the concrete lined section would have contraction and expansion zones of 2.5:1 (or 400 feet) and 4:1 (or 600 feet) respectively, with a central rectangular cross section that is wide enough to allow placement of the enclosed box culverts to pass flows from the tributary into the protected area. The central rectangular cross section in the Diversion Channel was 200 feet wide for all tributaries. The length of the central rectangular cross section in the Diversion Channel was 400 feet for the Wild Rice River and Maple River and 200 feet for the remaining three tributaries, which translates to a total length of concrete lined section in the Diversion Channel of 1400 feet or 1600 feet. Additionally, the crossing is completed with a siphon to pass winter flows in the tributary to the protected area when there is the possibility of damage to the box culverts due to freeze-thaw cycles. Figure F-C1 shows a schematic of the design submitted for Phase I of the study.

F-C2.2 ALTERNATIVE 1: PHASE I DESIGN WITH OPEN AQUEDUCT AND GATES

Alternative 1 would be similar to the Phase I design with the exception of removing the top of the box culverts that function as the aqueduct. Removing the top of the box culverts would allow mixing of flow in some of the tributaries with the diversion channel during large flow events. Flow from the aqueduct will be able to overtop into the diversion channel, and similarly flow from the diversion channel will be able to be directed into the protected area. In order to control the amount of flow that is passed into the protected area an additional set of gates will be placed on the downstream side of the tributary crossing. Figure F-C2 illustrates these modifications to the Phase I design.

During low flows this design will perform similar to the Phase I design, flow in the tributary and diversion channel will remain separate. However, during high flow events water will be able to overtop the aqueduct into the Diversion Channel. By removing the top of the aqueduct it reduces the possibility of freeze-thaw damage to the aqueduct during spring and autumn flows when the water is contained within the enclosed box culvert. By reducing this freeze-thaw risk it is possible that a siphon to convey winter flows will not be necessary. Removing the top of the aqueduct also allows for natural light into the crossing which may help encourage fish passage

through the structure. Finally, it is possible that with this design a separate spillway from the tributary into the Diversion Channel will not be necessary. During high flow events on the tributary the aqueduct will overtop directly into the Diversion Channel.

However, by allowing mixing of flow some flow will be diverted from the diversion channel into the protected area. By adding gates at the downstream end of each crossing the manual labor involved during a flood event to control the flow into the protected area increases. Further analysis needs to be completed to understand how this alternative will impact winter freezing in the crossing structure and how the proposed modifications will impact ice flow in both the tributary and Diversion Channel.

F-C2.3 ALTERNATIVE 2: PHASE I DESIGN AND PASS ALL DIVERSION CHANNEL FLOW BELOW AQUEDUCT

The Alternative 2 design would be similar to the Phase I design with the exception that all of the flow in the Diversion Channel will be passed below the aqueduct which can be accomplished in one of the three ways described below.

First, the width of the Diversion Channel at the tributary crossing (L) can be increased. Increasing the width of the Diversion Channel increases the cross sectional area available to convey flow in the Diversion Channel which will theoretically reduce the water surface elevation. These modifications to the Phase I design are shown in Figure F-C3. However, due to tailwater effects from the Red River for large events, this first option is not feasible as it is not possible to significantly lower the water surface elevation in the Diversion Channel by widening it.

Second, inverts of the aqueduct can be increased to an elevation above the water surface elevation in the diversion channel. These modifications to the Phase I design are shown in Figure F-C3A.

Third, the invert of the Diversion Channel can be lowered at the tributary crossings. This could be completed by lowering the entire cross section to match the invert of a low flow channel.

Theoretically, by removing the aqueduct from intersecting the flow in the Diversion Channel, no head losses will occur at the crossing because there will not be any restrictions in the diversion channel. Additionally, this would reduce the stress applied on the aqueduct structure during high flow events by flow in the diversion channel which may reduce the cost to construct the structure. However, widening the diversion channel would require additional excavation which would increase the cost.

Increasing the aqueduct invert elevations will impact upstream flood elevations on the tributaries and possibly create ponded areas. Raising the invert of the aqueduct without a sufficient transition to the regular invert elevation of the tributary could lead to conditions that mimic a low-head dam. Additionally, increasing the inverts of the aqueduct could divert additional water from the tributaries into the Diversion Channel and result in impacting water surface elevations on the tributary upstream of the Diversion Channel. Finally, the impacts on ice flow when

passing all the Diversion Channel flow below the tributary crossing is unclear and requires further analysis.

F-C2.4 ALTERNATIVE 3: ELIMINATE TRANSITION TO CONCRETE LINED SECTION

The Alternative 3 design would eliminate the transition to a concrete lined section all together. Box culverts would be placed along the bottom of the Diversion Channel to convey flow. The bottom width of the diversion channel would be increased so that all of the flow passes below the tributary crossing. Tributary flow would be conveyed over the top of the box culverts in an open natural channel. A schematic of this crossing is shown in Figure F-C4. However, due to tailwater effects from the Red River for large events, this alternative is not feasible as it is not possible to significantly lower the water surface elevation in the Diversion Channel by widening it.

If this option were feasible, the following items would have been taken into account when deciding on a Phase II design: (a) replacing the aqueduct with an open channel reduces the possibility of freeze-thaw damage during spring and autumn flows when the water is contained within the enclosed box culvert, (b) reducing the risk of freeze-thaw allows for the possibility that a siphon to convey winter flows will not be necessary, (c) removing the concrete lined channel and siphon may result in reduced construction costs, (d) providing a tributary crossing with a natural bottom (i.e. earth bottom rather than concrete lined) may encourage fish passage, and (e) providing an open top may also better handle ice flows than the current design.

Replacing the concrete lined channel with an earth berm and box culverts would also likely result in an increase in head loss across the crossing. It is unclear how wide the diversion channel would need to be to avoid impacts to the water surface elevation at the inlet to the Diversion Channel. Finally, similar to Alternative 2 it is unclear how well this crossing will handle ice flows in the Diversion Channel because all of the flow is conveyed below the tributary crossing.

F-C2.5 ALTERNATIVE 4: PHASE I DESIGN & REPLACE LONG CONCRETE TRANSITION WITH VERTICAL WALLS

The Alternative 4 design would be similar to the Phase I design with the exception of removing the concrete lined transitions upstream and downstream of the tributary crossing and replacing them with radial vertical walls or with tapered vertical wing walls. Figure F-C5 illustrates the radial vertical walls on the Phase I design. The primary benefit of removing the concrete lined transitions include a reduction of the length of concrete lined channel at each crossing which should ultimately result in reduced construction costs. However removing the transitions will also reduce the hydraulic efficiency of the crossing.

Overall, if the tributary is kept in a box culvert closed aqueduct, similar to the Phase I design, it is anticipated that it will handle ice flows and winter freezing similar to the Phase I design. Additionally if the aqueduct is not modified from the Phase I design it is anticipated that this type of crossing will have similar benefits and restrictions to fish passage as the Phase I design.

F-C2.6 ALTERNATIVE 5: PHASE I DESIGN WITH TALLER BOX CULVERTS

The Alternative 5 design would be similar to the Phase I design with the exception of increasing the height of the box culverts used for the aqueduct. Increasing the height of the boxes will increase the hydraulic efficiency of the crossing. Figure F-C6 illustrates these modifications to the Phase I design. With a taller box culvert the required width of the tributary crossing (X2) can be reduced. Consequently, reducing the width of the tributary crossing will correspond to an overall reduction to the required length of concrete lined channel at each crossing. Additionally, it is anticipated that the larger opening will encourage better fish passage through the structure when compared to the Phase I design, however the flow is still constrained in a closed box culvert and during high flows high velocities are still anticipated. Finally, it is also anticipated that the larger openings will facilitate ice flow in the tributaries during winter months.

Consequently, the additional box culvert height will reduce the cross sectional area of the diversion channel increasing the head loss at each crossing. However, for Phase II design, the design constraint at each crossing is to not increase the water surface elevation in the Red River at the inlet to the diversion (rather than the Phase I constraint of only allowing 0.5-ft of head loss at each crossing), and it is possible that increasing the height of the aqueduct will result in increasing the head loss across an individual crossing while not impacting the water surface elevation at the inlet to the diversion channel.

F-C2.7 ALTERNATIVE 6: DIVERT RUSH AND LOWER RUSH DIRECTLY INTO DIVERSION CHANNEL

The Rush River and Lower Rush River will be diverted directly into the Diversion Channel. The portions of these Rivers downstream of the Diversion Channel are primarily straight drainage channels and do not display many characteristics typically associated with natural streams. By diverting these two channels directly into the Diversion Channel, two crossing structures are eliminated from the overall design thereby reducing the overall costs. Additionally, removing the crossing structures will likely facilitate the handling of ice flows in both the Diversion Channel and tributaries. Fish passage structures could also be designed for these tributaries. Downstream of these Rivers, habitat enhancements and low flow channel meandering could be implemented, increasing the quality and quantity of habitat in these Rivers, when compared to existing conditions.

F-C2.8 ALTERNATIVE 7: REDUCE FLOWS INTO THE PROTECTED AREA

By reducing the tributary flow conveyed into the protected area from the 5-year flow to the 2-year flow additional flow will be directed into the Diversion Channel. This reduction in flow will result in smaller crossing structures and a larger spillway which will reduce overall cost of the crossing structure.

F-C2.9 ALTERNATIVE 8: PHASE I WITH SIDE CHANNELS AT MAPLE CROSSINGS ("BITTNER ALTERNATIVE")

This alternative was originally proposed by Mark Bittner from the City of Fargo. The "Bittner Alternative" as understood by Barr, is described below and shown in Figures F-C7A and F-C7B.

The invert elevation of the Maple River crossing of the ND Diversion Channel would be identical to the natural channel bottom on either side of the ND Diversion Channel (i.e. the

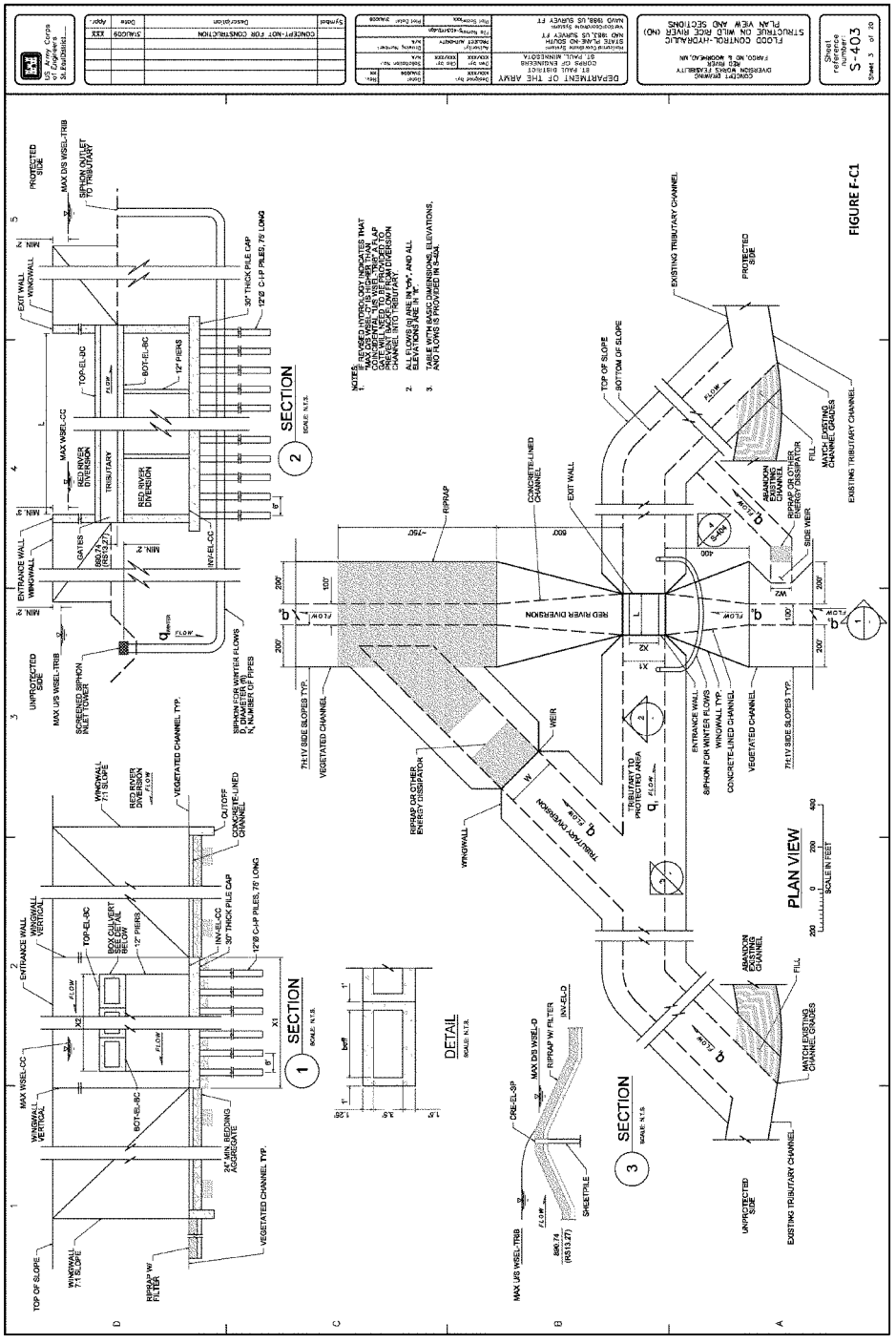
bottom would not be raised as had been done in Alternative 2 in which the tributary crossings were modified to pass all flow below the aqueduct). The Maple River crossing would be a rectangular channel with a bottom width of 105 feet. The sides of the channel would be approximately 20 feet tall to prevent the Maple River 2-year flow overflowing into the ND Diversion Channel and to prevent the Diversion Channel from overtopping the Maple River Crossing during high flow events in the ND Diversion Channel. In order to route the high flows in the ND Diversion Channel, an additional channel would be constructed to the east side (protected area side) of the ND Diversion. The second channel would have the same bed slope as the ND Diversion Channel but its invert elevation would be the same as the Maple River at the Maple River Crossing. Three gates would be installed to control the flow of water in the second channel. One gate would be installed in the Maple River to control the amount of flow passing into the protected area. The other two gates would be installed in the second channel on either side of the Maple River to keep the 2-year flow within the Maple River Channel. The gates could then be raised for larger events (such as the 100-year event or greater) which require additional capacity from the second channel. Therefore, for this Alternative 8, no weir spillway located on the west side (unprotected side) of the ND Diversion to divert water from the tributary to the Diversion Channel is needed.

A summary of the efforts were presented to Mr. Bittner. His response was that he was thinking of a slightly different alignment, shown in Figures F-C8A through F-C8D. This configuration would include low flow culverts under the tributary crossings for the Maple and Sheyenne Rivers. The tributaries would remain an open aqueduct. The Diversion Channel would not narrow as it approaches the tributary to allow for a longer weir length as high flows in the Diversion Channel overtop the aqueduct.

The Bittner alternative as initially understood by Barr would be effective in conveying the Diversion Channel flow pass the Maple River without raising the water surface in the RRN beyond the acceptable amount; however the configuration of three gates was too costly. The reconfigured Bittner alternative raised water surface elevations in the RRN by too significantly to be a feasible alternative.

F-C2.10 CONCLUSIONS

In conclusion, the modified Phase II design constraints allowed for further analysis of the selected crossing in Phase I as well as consideration of additional crossing alternatives. Alternative designs discussed above were intended to reduce project cost, allow fish passage, and address winter freezing and ice flow. However, not every alternative design discussed provided an improvement over the Phase I design for all of the design considerations. During the modeling process, all alternatives were modeled in HEC-RAS. The optimum Phase II designs for the Sheyenne, Maple, Lower Rush and Rush Rivers crossing with the ND Diversion Channel are described in Appendix F.


FLOOD CONTROL -HYDRAULIC
STRUCTURE ON MILD ROCK RIVER (ND)
PLAN VIEW AND SECTIONS
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
ST. PAUL, MINNESOTA
STATE PLANE NO SOUTH
NAD 1983 US SURVEY FT
NAVD 1988, US SURVEY FT
CONCEPT-NOT FOR CONSTRUCTION
SECTION
DETAIL
PLAN VIEW
RED RIVER DIVERSION
TRIBUTARY
TRIBUTARY TO PROTECTED AREA
VEGETATED CHANNEL TYP.
CONCRETE-LINED CHANNEL
ENTRANCE WALL
WINGWALL
SIPHON FOR WINTER FLOWS
EXISTING TRIBUTARY CHANNEL
PROTECTED SIDE
UNPROTECTED SIDE
RIPRAP
SHEETPILE
FIGURE F-C1

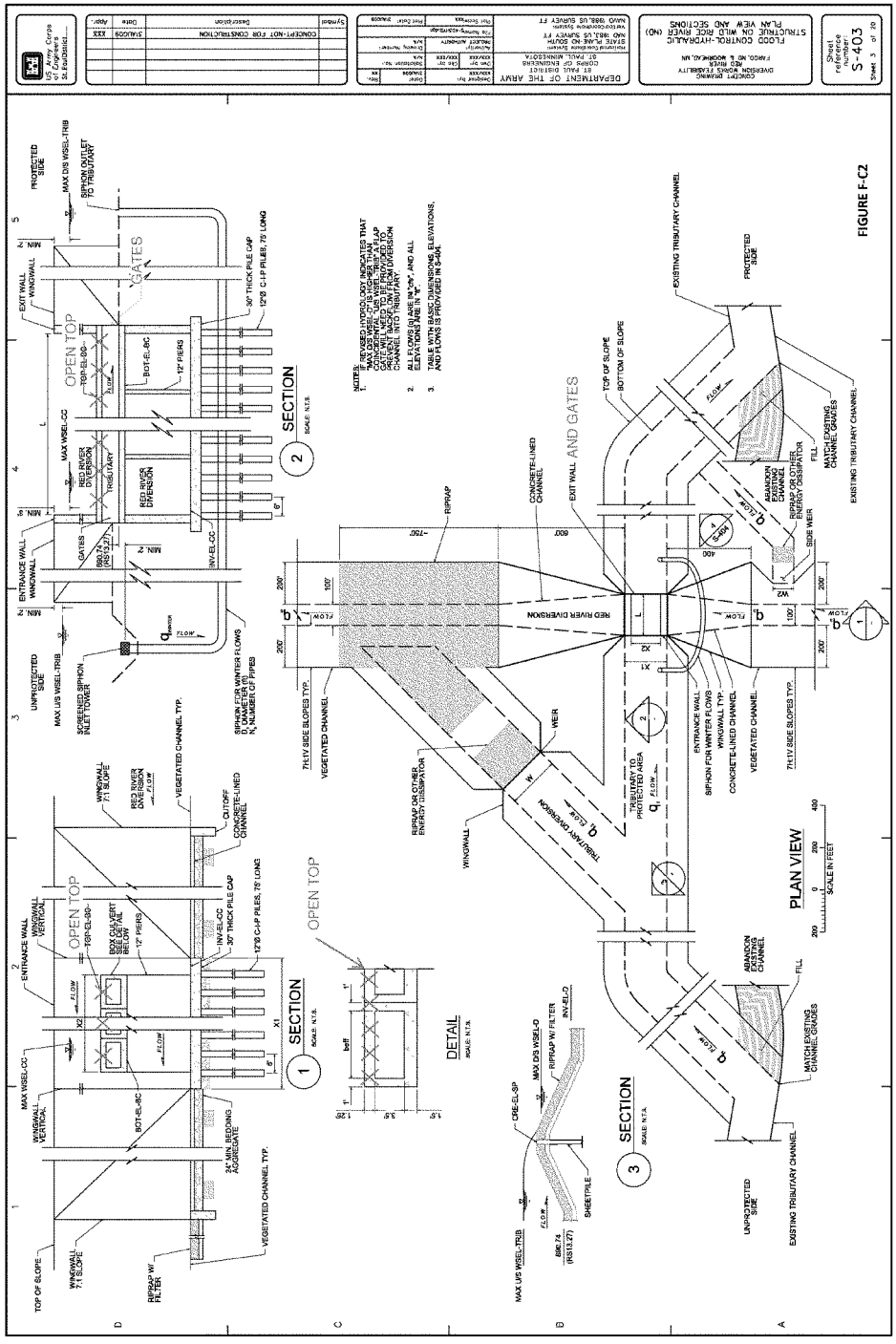
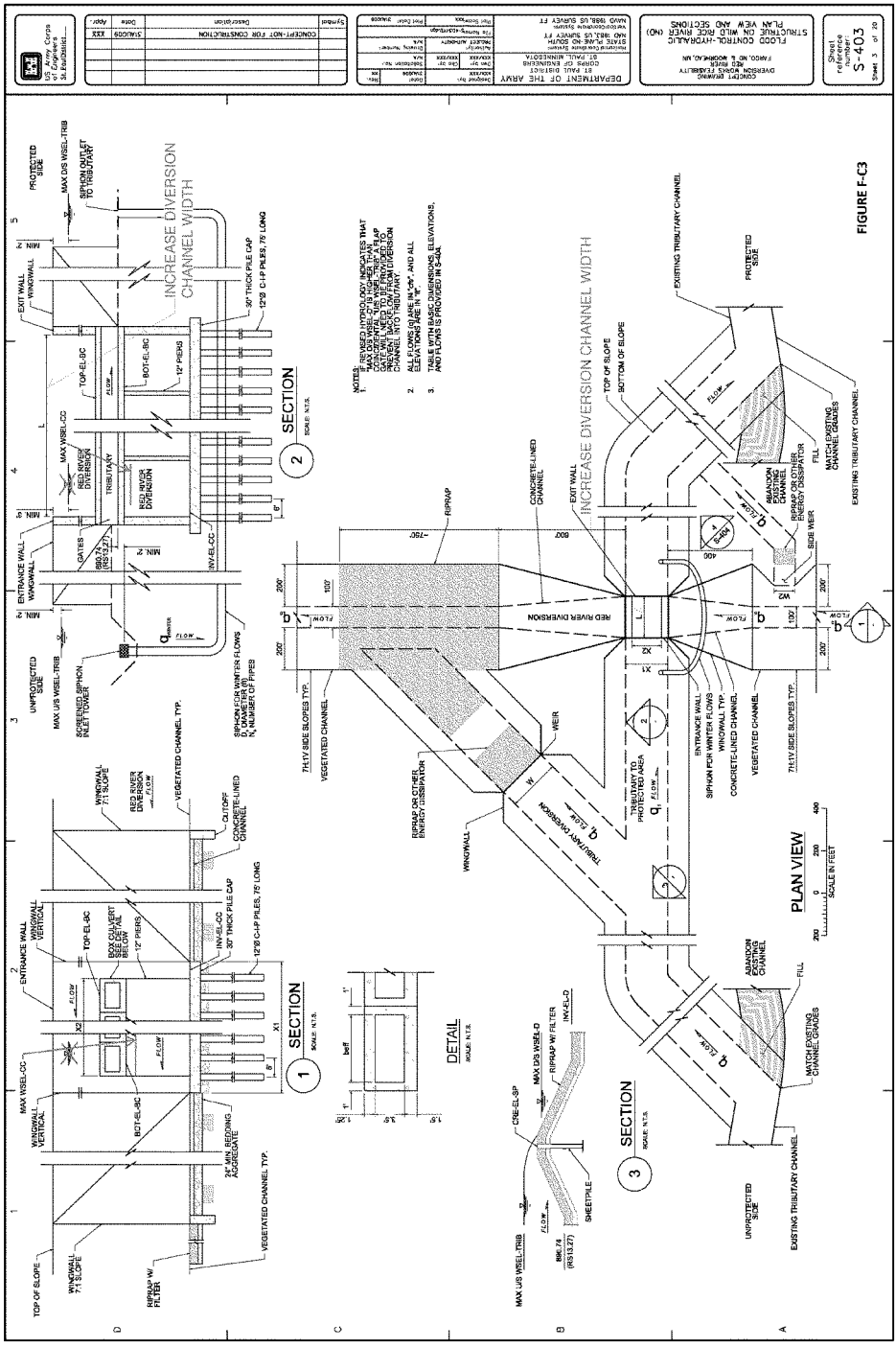


FIGURE F-C2
PLAN VIEW
SECTION
DETAIL
RED RIVER DIVERSION
EXISTING TRIBUTARY CHANNEL
PROTECTED SIDE
UNPROTECTED SIDE
CONCRETE-LINED CHANNEL
VEGETATED CHANNEL
RIPRAP
WEIR
SIDE WEIR
GATES
OPEN TOP
FLOW
SCALE IN FEET


FLOOD CONTROL -HYDRAULIC
STRUCTURE ON MILD RED RIVER (N0)
PLAN VIEW AND SECTIONS
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
ST. PAUL, MINNESOTA
STATE PLANE NO SOUTH
NAD 1983, US SURVEY FT
NAVD 1988, US SURVEY FT
S-403
CONCEPT-NOT FOR CONSTRUCTION
SECTION
DETAIL
PLAN VIEW
INCREASE DIVERSION CHANNEL WIDTH
RED RIVER DIVERSION
TRIBUTARY DIVERSION
SIPHON FOR WINTER FLOWS
EXISTING TRIBUTARY CHANNEL
FIGURE F-C3

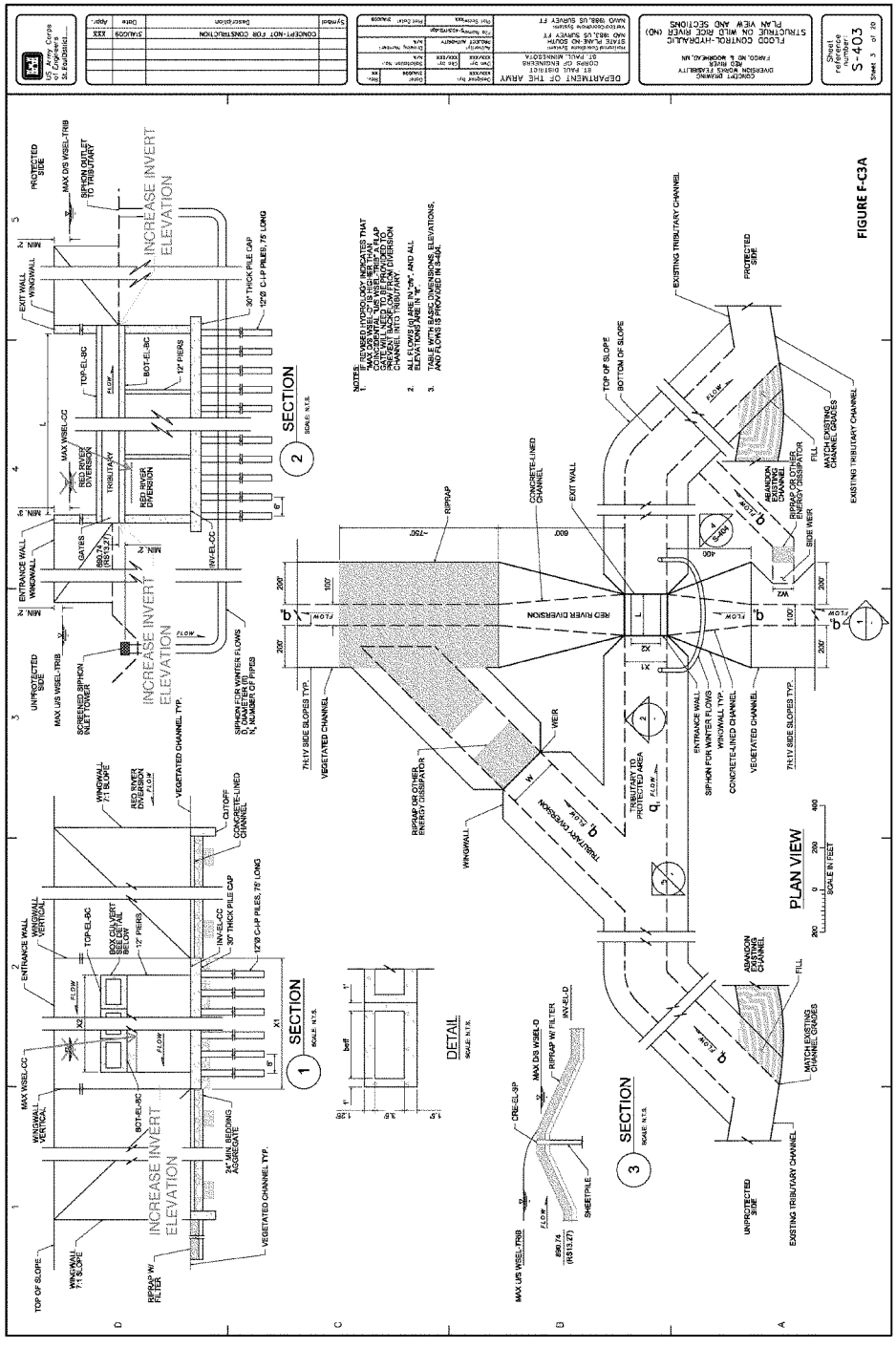
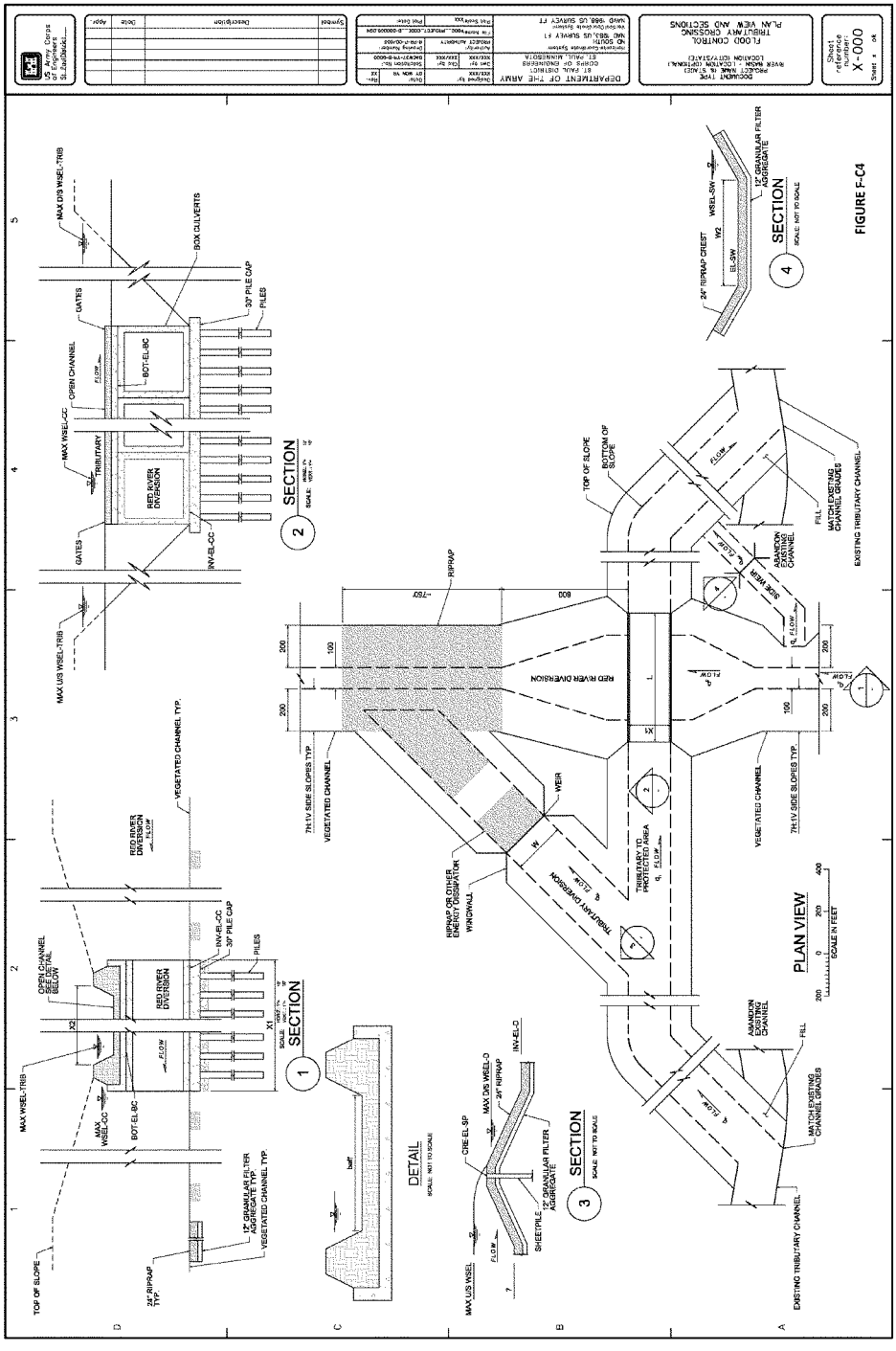
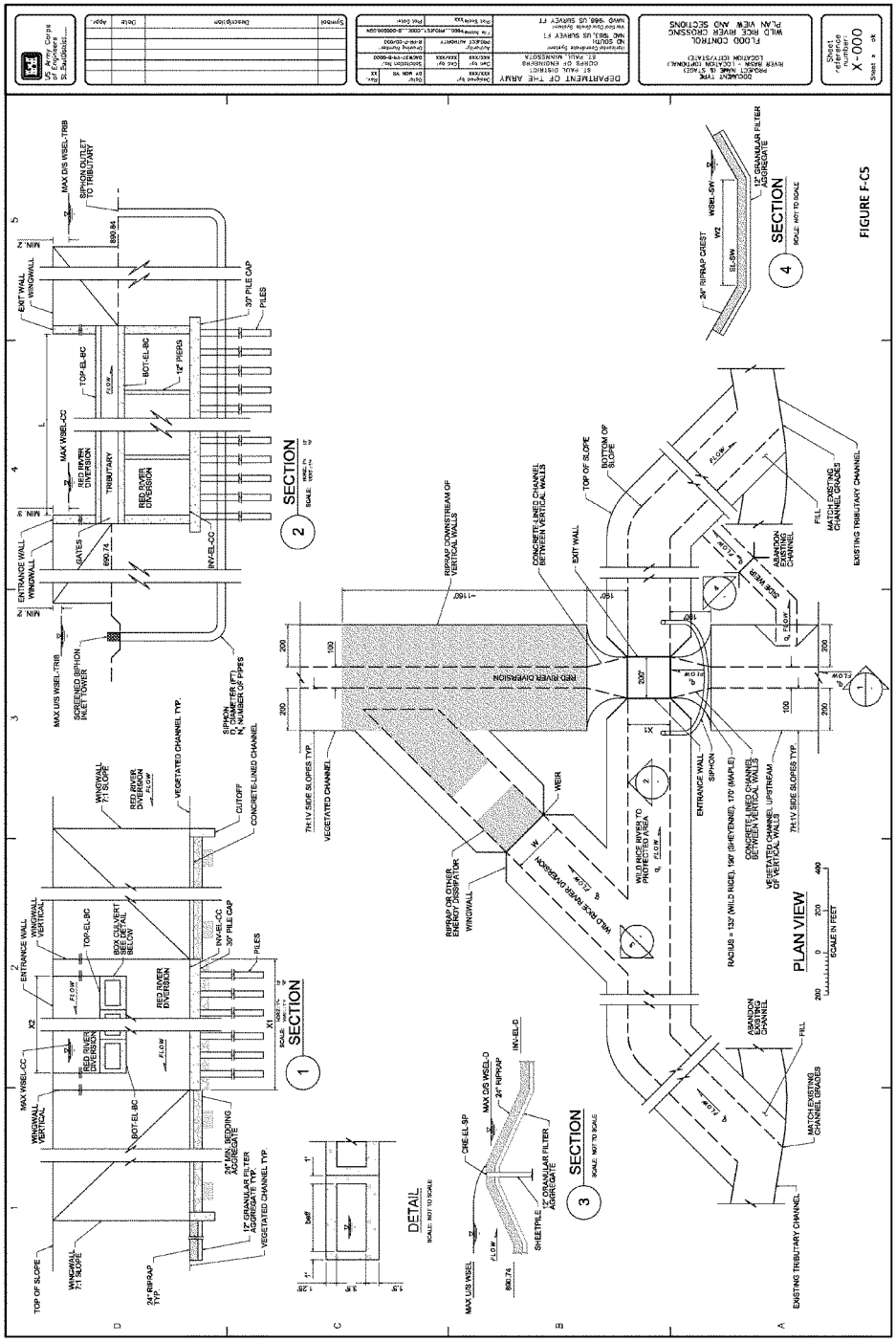


FIGURE F-C3A
PLAN VIEW
SECTION
DETAIL
RED RIVER DIVERSION
FLOOD CONTROL-HYDRAULIC
STRUCTURE ON MILD ROC RIVER (ND)
PLAN VIEW AND SECTIONS
DEPARTMENT OF THE ARMY
ST PAUL DISTRICT
CORPS OF ENGINEERS
ST PAUL, MINNESOTA
DESIGN WORKS FEASIBILITY
RED RIVER
GENERAL DRAWING
S-403
Sheet 3 of 28
CONCEPT-NOT FOR CONSTRUCTION
SCALE IN FEET
TOP OF SLOPE
BOTTOM OF SLOPE
EXISTING TRIBUTARY CHANNEL
PROTECTED SIDE
UNPROTECTED SIDE
CONCRETE-LINED CHANNEL
VEGETATED CHANNEL
RIPRAP
WEIR
SIDE WEIR
ENTRANCE WALL
EXIT WALL
WINGWALL
SIPHON FOR WINTER FLOWS
ABANDON EXISTING CHANNEL
MATCH EXISTING CHANNEL GRADES
FILL
RIPRAP OR OTHER ENERG. DISSIPATOR
SHEETPILE
RIPRAP W/ FILTER


PLAN VIEW
SECTION
DETAIL
FLOOD CONTROL
TRIBUTARY CROSSING
PLAN VIEW AND SECTIONS
RED RIVER DIVERSION
TRIBUTARY
RIPRAP
VEGETATED CHANNEL
7H:1V SIDE SLOPES TYP.
WEIR
SHEETPILE
12" GRANULAR FILTER AGGREGATE
24" RIPRAP TYP.
MATCH EXISTING CHANNEL GRADES
EXISTING TRIBUTARY CHANNEL
ABANDON EXISTING CHANNEL
FILL
TOP OF SLOPE
BOTTOM OF SLOPE
SIDE WEIR
GATES
BOX CULVERTS
30" PILE CAP
PILES
OPEN CHANNEL
FLOW
SCALE IN FEET
FIGURE F-C4


FLOOD CONTROL
WILD RICE RIVER CROSSING
PLAN VIEW AND SECTIONS
DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
ST. PAUL, MINNESOTA
NAD 1983 US SURVEY FT
NAVD 1988 US SURVEY FT
PLAN VIEW
SECTION
DETAIL
FIGURE F-C5

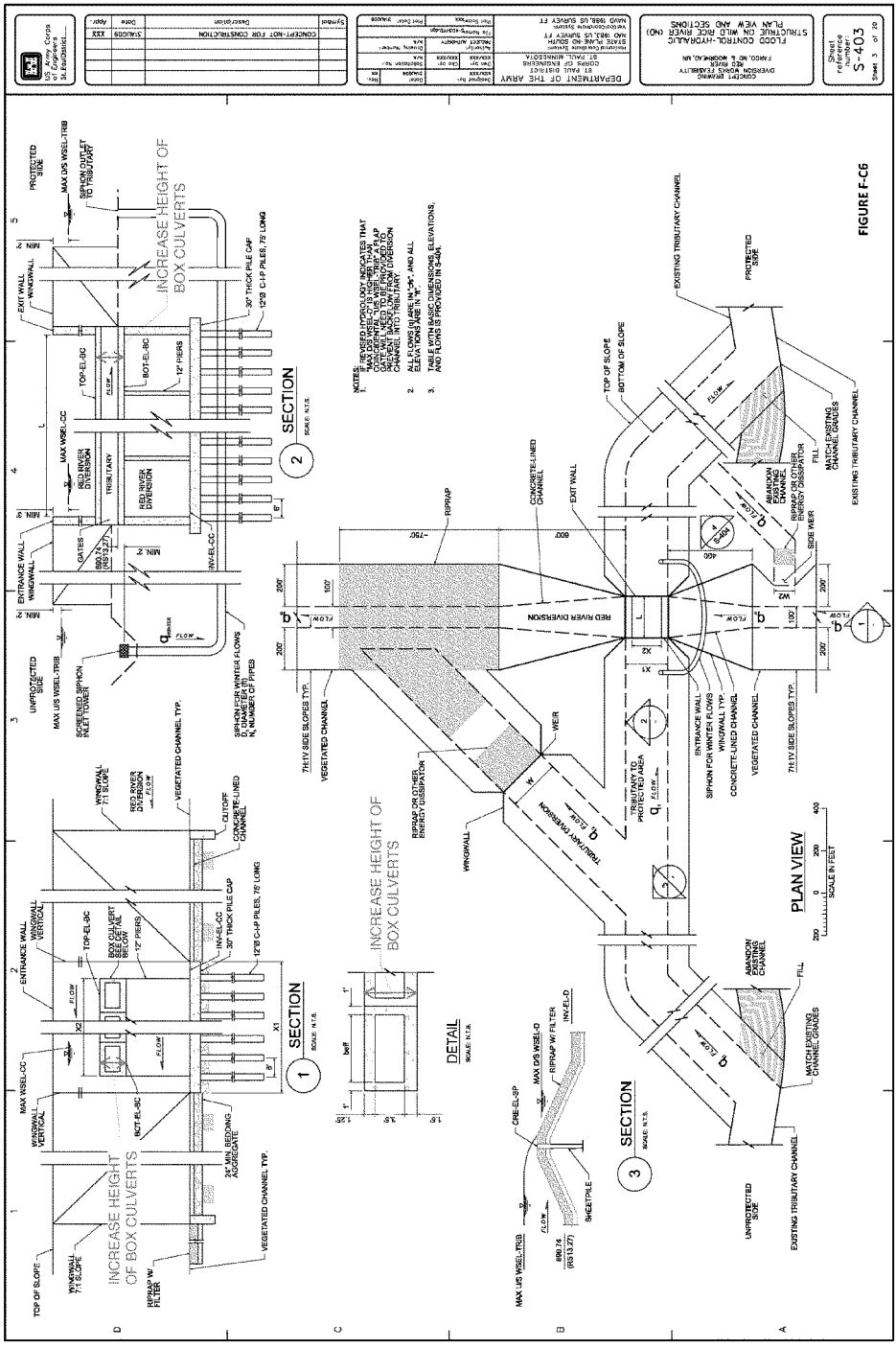
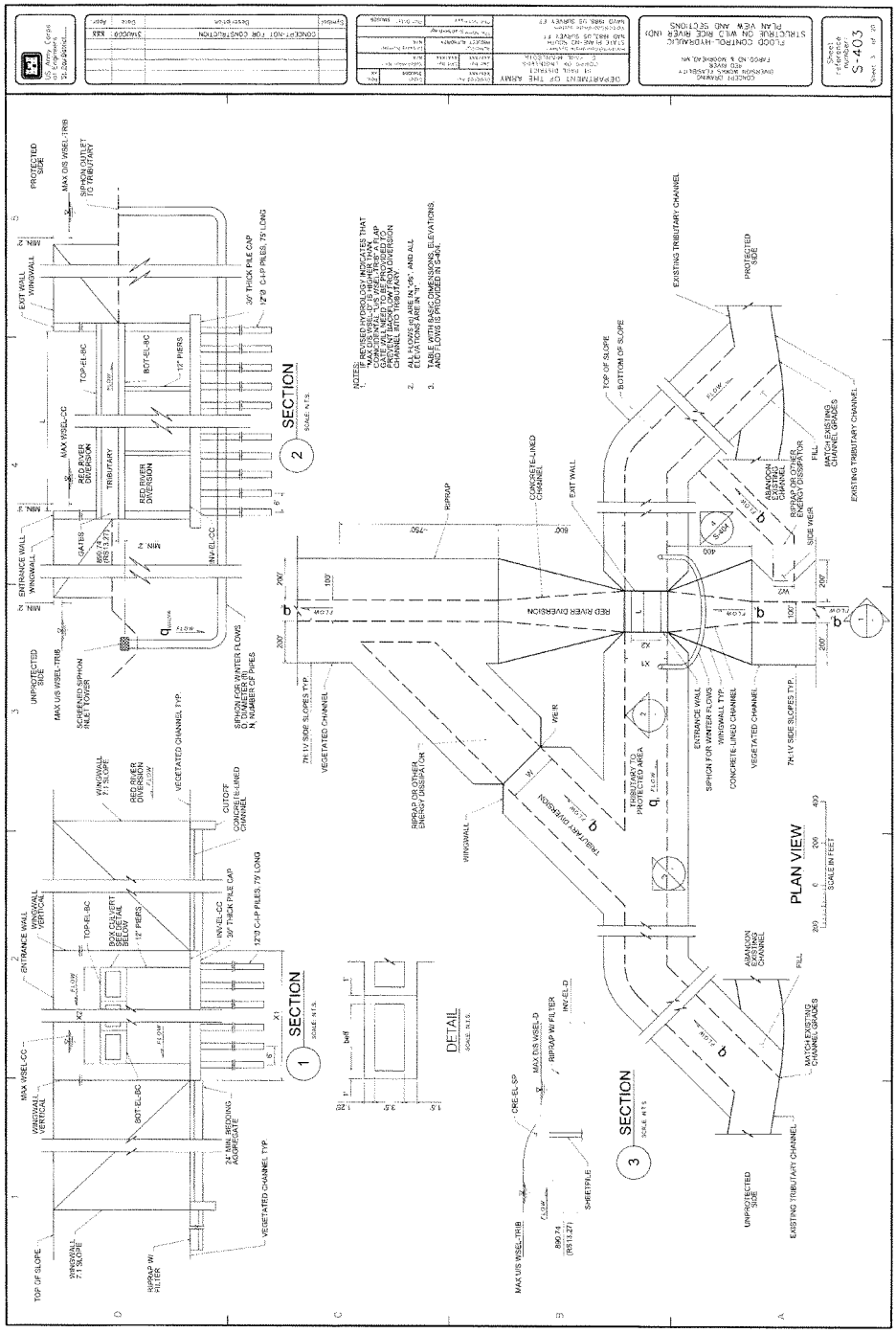


FIGURE F-C6
PLAN VIEW
SECTION
DETAIL
INCREASE HEIGHT OF BOX CULVERTS
RED RIVER DIVERSION
TRIBUTARY
EXISTING TRIBUTARY CHANNEL
PROTECTED SIDE
UNPROTECTED SIDE
RIPRAP
CONCRETE-LINED CHANNEL
VEGETATED CHANNEL TYP.
SIPHON FOR WINTER FLOWS
ENTRANCE WALL
WINGWALL
EXIT WALL
SIDE WEIR
SHEETPILE
FLOW
SCALE IN FEET


PLAN VIEW
SECTION
DETAIL
RED RIVER DIVERSION
TRIBUTARY
SIPHON OUTLET TO TRIBUTARY
SIPHON FOR WINTER FLOWS
SIPHON FOR WINTER FLOWS
SCREENED SIPHON INLET TOWER
EXISTING TRIBUTARY CHANNEL
PROTECTED SIDE
UNPROTECTED SIDE
ABANDON EXISTING CHANNEL
MATCH EXISTING CHANNEL GRADES
RIPRAP OR OTHER ENERGY DISSIPATOR
SIDE WEIR
CONCRETE-LINED CHANNEL
VEGETATED CHANNEL
RIPRAP OR OTHER ENERGY DISSIPATION
TOP OF SLOPE
BOTTOM OF SLOPE
EXIT WALL
WINGWALL
ENTRANCE WALL
CUTOFF
RIPRAP W/ FILTER
SHEETPILE
SCALE IN FEET
FLOOD CONTROL, HYDRAULIC STRUCTURE ON MID RIDGE UNDIVERSION WORKS, FLOODWAY PLAN VIEW AND SECTIONS
S-403

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MINNEAPOLIS, MINNESOTA - HIBBING, MINNESOTA
DULUTH, MINNESOTA
ANN ARBOR, MICHIGAN - JEFFERSON CITY, MISSOURI

DATE 12/17/09 SHEET NO. 1
PROJECT NAME Fargo By-pass
PROJECT NUMBER 34-09-1034
SUBJECT Maple Crossing of Diversion Channel

COMPUTED	CHECKED	SUBMITTED
BY JDW	BY	TO
DATE	DATE	DATE

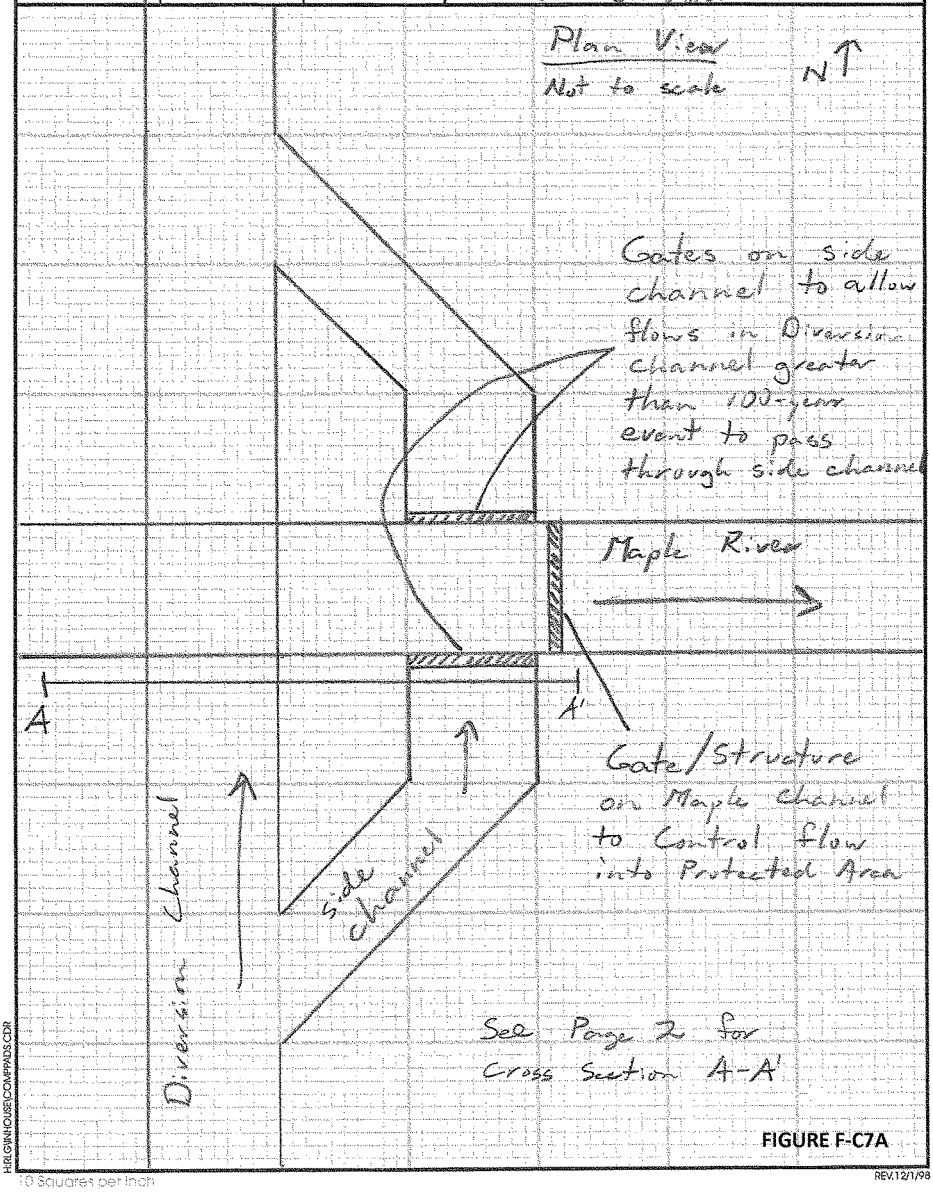

FIGURE F-C7A

HELGWINHOUSE\COMPPADS.CDR
10 Squares per Inch
REV.12/1/98

BARR
MINNEAPOLIS, MINNESOTA - HIBBING, MINNESOTA
DULUTH, MINNESOTA
ANN ARBOR, MICHIGAN - JEFFERSON CITY, MISSOURI

COMPUTED	CHECKED	SUBMITTED
BY JDW	BY	TO
DATE	DATE	DATE

DATE 12/17/09 SHEET NO. 2
PROJECT NAME Fargo By-pass
PROJECT NUMBER 34-09-1004
SUBJECT Maple River Crossing

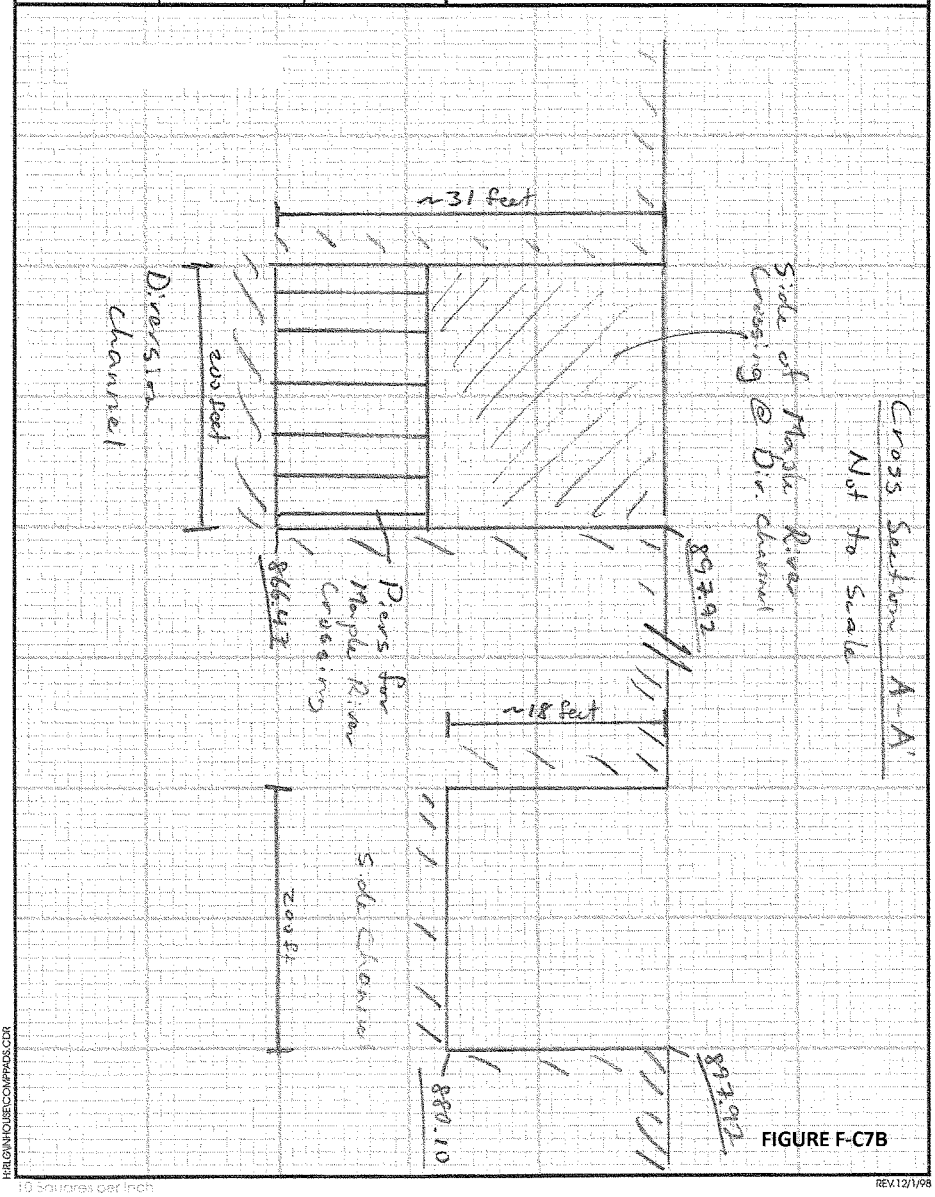

FIGURE F-C7B

H:\BG\NHOUSE\COMPPADS.CDR
10 volumes per iron
REV.12/1/98

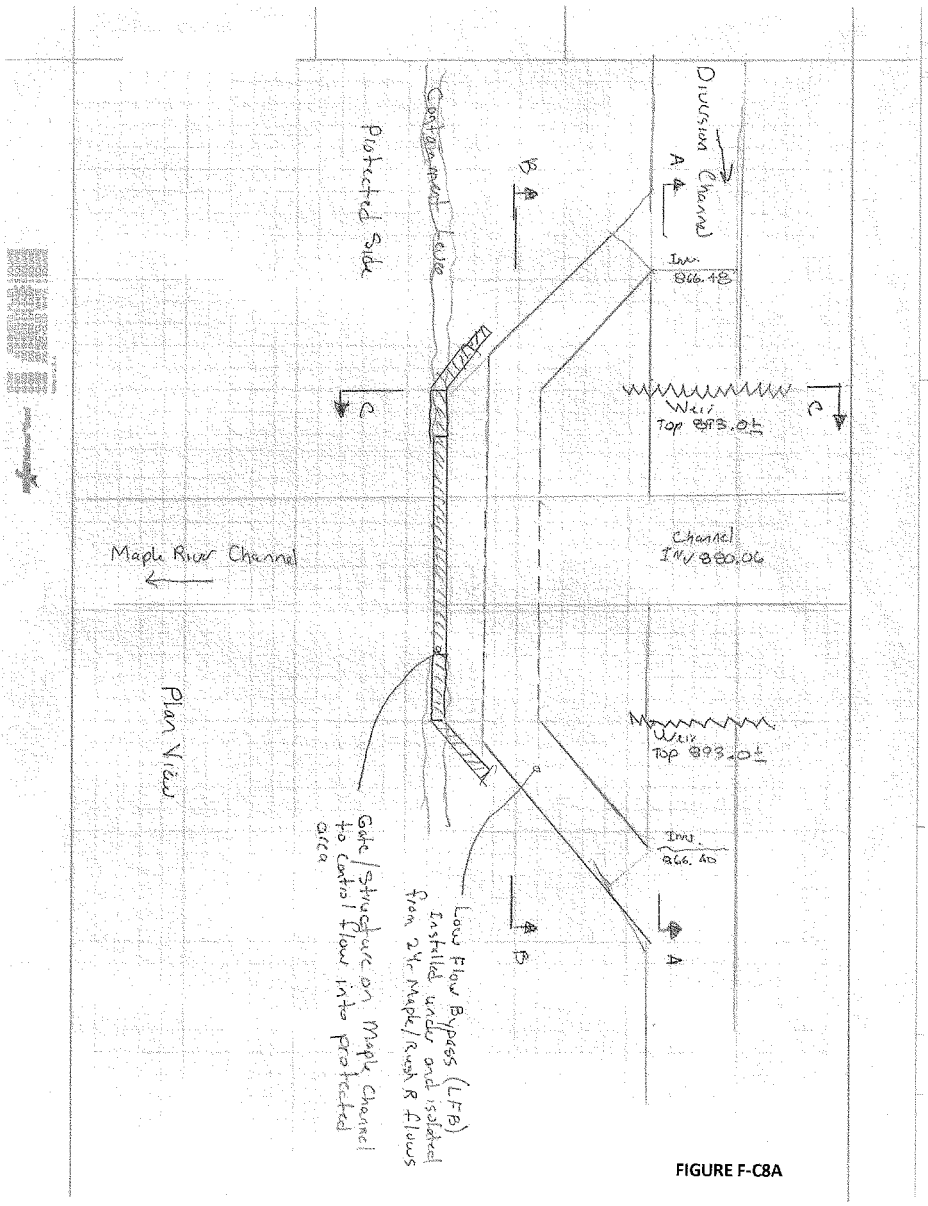
Diversion Channel
A
A
Inv. 866.48
Weir Top 893.0±
C
C
Protected Side
Containment Levee
B
A
Channel Inv. 880.06
Maple River Channel
Weir Top 893.0±
Inv. 866.40
B
A
Low Flow Bypass (LFB) Installed under and isolated from 24" Maple/Rush R flows
Gate/structure on Maple Channel to control flow into protected area
Plan View

FIGURE F-C8A

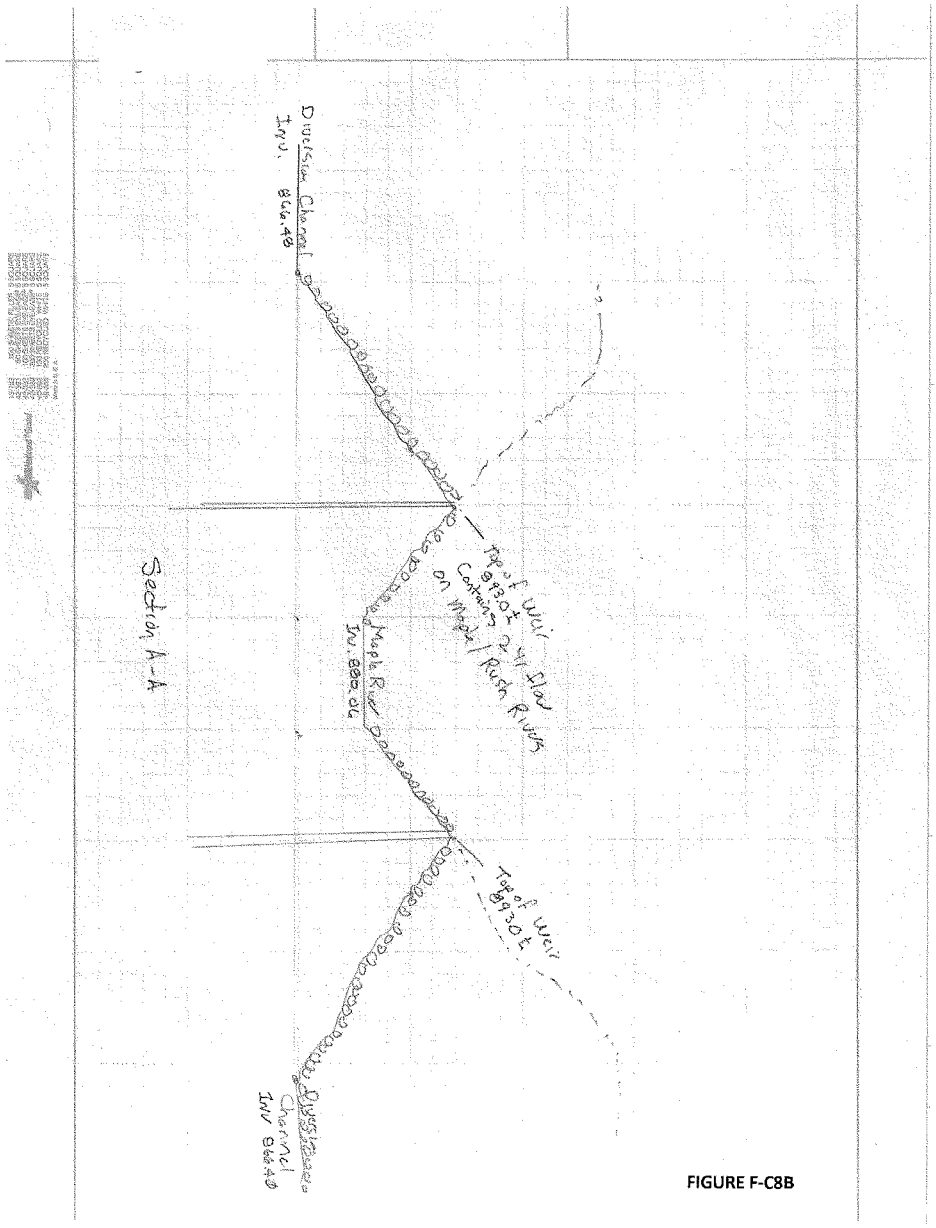
Section A-A
Top of Weir
Elev. 89.30 ft.
Top of Weir
Elev. 89.30 ft.
Inv. 882.04
Inv. 886.49
Inv. 886.49

FIGURE F-C8B

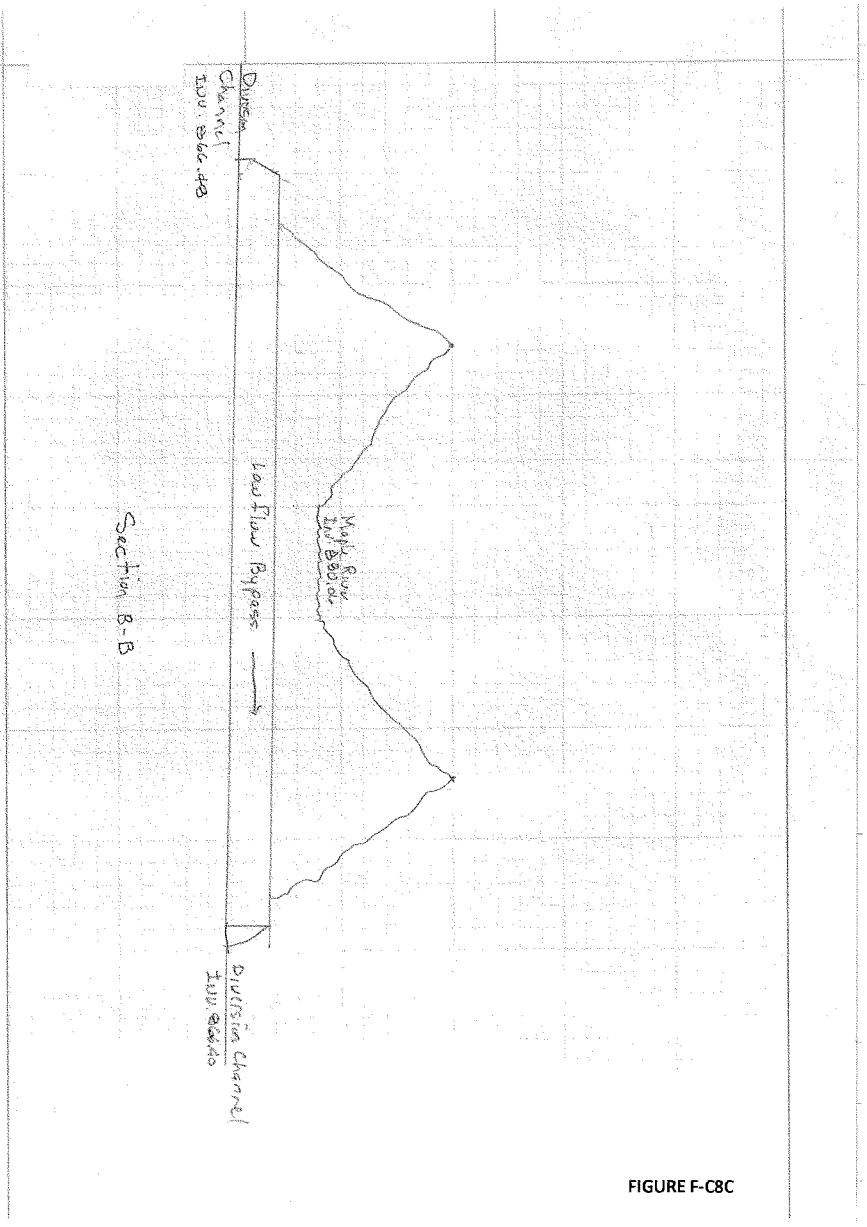

FIGURE F-C8C

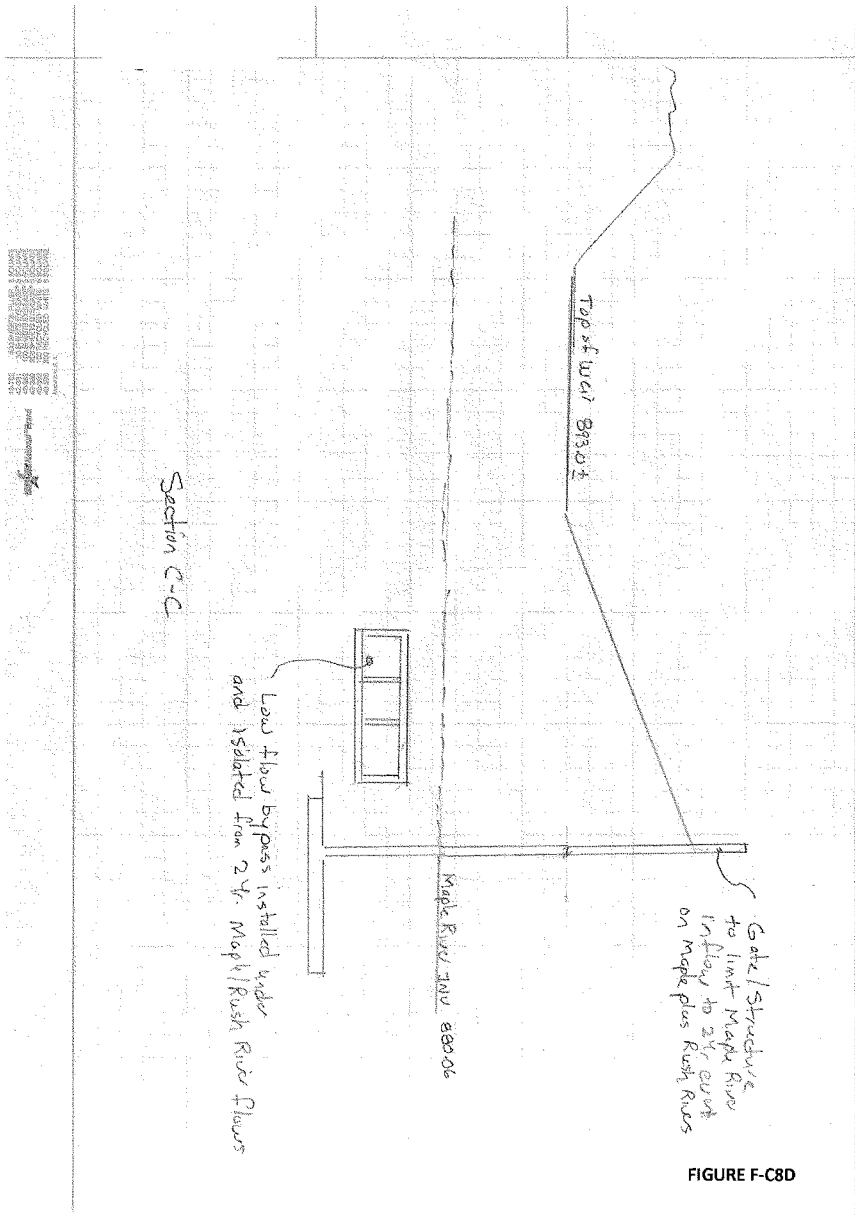
Top of wall 893.0±
Gate/Structure to limit Maple River inflow to 2yr event on Maple plus Rush Rver
Maple River 100 880.06
Low flow bypass installed under and isolated from 2 Yr. Maple/Rush River flows
Section C-C

FIGURE F-C8D

RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT D– HYDRAULIC DESIGN COMPUTATIONS

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

By: Barr Engineering Co.

FINAL –February 28, 2011

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT D – HYDRAULIC DESIGN COMPUTATIONS

F-D1.0 FEDERALLY COMPARABLE PLAN (FCP)

The following information regarding the FCP was previously presented as part of Appendix F Exhibit D in the Phase 3 report submitted on August 6th, 2010, and is included here for completeness.

F-D1.1 CONTROL STRUCTURE ON RED RIVER OF THE NORTH

A description of the FCP control structure on the Red River of the North can be found in Appendix F. Appendix F, Exhibit B discusses the variety of structure types that were investigated for this control structure during the Phase 2 design. This portion of Exhibit D presents design concepts and calculations for the Red River of the North control structure, detailed HEC-RAS modeling outputs at the control structure, and comparisons of water surface elevations at the control structure using Phase 3 hydrology.

F-D1.1.1 Flow Partitioning

The partitioning of the flow between the protected area downstream of the Red River Control Structure and the Diversion Channel was performed by Moore Engineering in HEC-RAS using preliminary designs of the Red River Control Structure and Diversion Inlet Structure. The resulting flows are presented in Table F-D1, Table F-D2, and Table F-D3 for the Year 0, Year 25, and Year 50 hydrology, respectively. A graphical representation of these results, along with a tabular summary, is presented in Appendix F. The flows presented in Appendix F include the 2-year, 5-year, 10-year, 20-year, 50-year, 100-year, 200-year, and 500-year events on the Red River of the North for the Year 0, Year 25, and Year 50 hydrology.

F-D1.1.2 Gate Number and Sizing – Concepts

The number and sizing of the gates in the Red River Control Structure is driven by the ability of the design to achieve the desired flow through the gates as determined by Moore Engineering for each given flood event without increasing or decreasing the upstream water surface elevation. Initially, three concepts which included operable gates above permanently open areas were considered and presented to the U.S. Army Corps of Engineers (USACE) during a meeting on December 2, 2009. Those concepts are shown in Exhibit B of Appendix F and include a design with three identical gates, a design with five identical gates, and a design with two large identical gates combined with two smaller gates.

Based on discussion with the USACE and preliminary modeling of these concepts during Phase 2, the concept with 3 identical gates was selected for further evaluation. This selection was made because it allowed a wider gate width (for ice and boat passage) and redundancy of gates for operation and maintenance issues.

F-D1.1.3 Gate Number and Sizing – Optimization

After the three identical gate concept was selected for further analysis, optimization of the gate number and sizing was performed using the orifice equation during Phase 2 (Exhibit C of Appendix F in the January 6, 2010 submittal). For Phase 3, a HEC-RAS model developed by Moore Engineering was used to further refine the optimization of gate number and sizing. During Phase 3, following the recommendation from USACE – Environmental, a width of 50 feet was selected for each of the three gates. Preference was given for multiple, identically sized gates for operation and maintenance purposes. Also, it is desirable to have as little impact on the smaller, higher frequency, flood events (i.e. no change in WSEL or gate operation required); thus, a minimum open width is necessary to avoid flow constriction during smaller flood events.

The 500-year event (during which the flow through the structure is the greatest) results in the greatest open area and therefore can be used to determine the minimum number of gates required. For all smaller flood events, the gates may be partially or fully closed to achieve the desired flow into the protected area. The gate height is a user-defined variable in this process. In optimizing the design of the Red River Control Structure for the FCP alignment, 49-feet tall openings were used for the 35,000 cfs diversion. The number of gates necessary for the FCP (as determined using the above process) is three 50-feet wide gates. The height of the gates is 47 feet. The height of the ungated, open area below the gates is 2 feet. Thus, the maximum open height is 49 feet when the gates are completely open. A concept drawing for the control structure is presented in Figure F-D1.

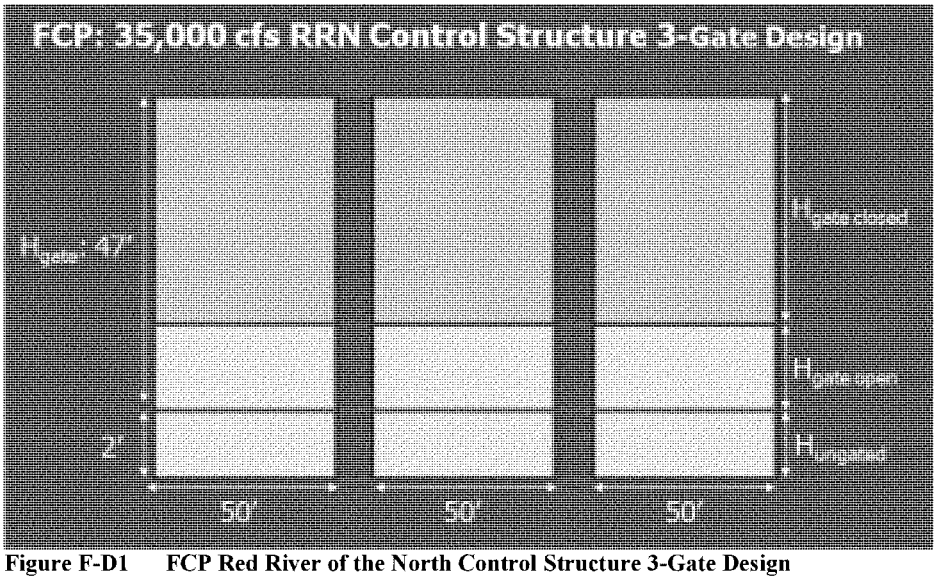

Figure F-D1 FCP Red River of the North Control Structure 3-Gate Design

F-D1.1.4 Gate Operation Optimization and HEC-RAS Results

After the gate openings were estimated as described above, the HEC-RAS model developed by Moore Engineering was used to fine-tune the gate openings for the different flow events. Flows, water surface elevations, head difference, gate opening height and average velocities for varying flood events at the FCP Red River of the North control structure are presented for Year 0, Year 25 and Year 50 hydrology in Tables F-D1, F-D2 and F-D3, respectively. These variables were obtained from the HEC-RAS models with the exception of average velocity through the gates. HEC-RAS does not provide this variable so it was calculated by dividing the flow by the flow area. The location of the control structure in the FCP HEC-RAS model is inline structure station 467. Figures F-D2, F-D3, and F-D4 show the comparison of the headwater and tailwater elevations for various flood events at the FCP Control Structure for the Year 0, Year 25, and Year 50 hydrology, respectively.

Table F-D1 HEC-RAS Outputs for the Proposed FCP Control Structure at the Red River of the North for Year 0 Hydrology

Event (yr)	Existing Flow into Protected Area (cfs)	Proposed Flow through Gates into Protected Area (cfs)	Proposed Headwater Elevation in Red River* (ft)	Proposed Tailwater Elevation in Red River* (ft)	Head Diff-erence (ft)	Gate Open Height (ft)	Average Velocity through Gates (ft/s)
99.99%	50	50	878.94	878.93	0.01	49	0.02
99%	251	251	879.61	879.6	0.01	49	0.11
Winter Mean	255	255	879.62	879.61	0.01	49	0.12
Median	360	360	879.97	879.96	0.01	49	0.16
September Mean	370	370	880.00	879.99	0.01	49	0.16
95%	550	550	880.59	880.58	0.01	49	0.24
90%	829	829	881.51	881.5	0.01	49	0.33
80%	1,360	1,360	883.17	883.16	0.01	49	0.50
April & May Mean	1,656	1,360	884.11	884.09	0.02	49	0.58
2	5600	5,600	892.51	892.48	0.03	49	1.36
5	12,150	9,640	901.18	897.97	3.21	5.55	11.8
10	17,000	9,654	904.53	898.28	6.25	4	16.4
20	22,000	9,759	906.86	898.6	8.26	3.52	18.7
50	29,300	9,803	909.5	898.88	10.62	2.7	24.6
100	34,700	10,929	910.82	899.97	10.85	3	24.5
200	46,200	16,367	912.53	904.01	8.52	5.8	17.6
500	61,700	26,938	913.61	909.5	4.11	13.5	13.1

* Headwater at Station 467.1 and Tailwater at Station 466.9

Table F-D2 HEC-RAS Outputs for the Proposed FCP Control Structure at the Red River of the North for Year 25 Hydrology

Event (yr)	Existing Flow into Protected Area (cfs)	Proposed Flow through Gates into Protected Area (cfs)	Proposed Headwater Elevation in Red River* (ft)	Proposed Tailwater Elevation in Red River* (ft)	Head Diff-erence (ft)	Gate Open Height (ft)	Average Velocity through Gates (ft/s)
2	4,352	4,352	890.37	890.35	0.02	49	1.14
5	10,608	9,640	899.64	897.9	1.74	7.5	8.6
10	15,394	9,647	903.49	898.21	5.28	4.35	14.8
20	20,345	9,741	906.18	898.54	7.64	3.65	17.8
50	27,441	9,845	909.04	898.88	10.16	2.85	23.0
100	32,921	9,918	910.61	899.07	11.54	2.5	26.4
200	42,242	14,209	912.06	902.53	9.53	4.7	20.2
500	57,641	22,686	913.75	907.72	6.03	9.5	15.9

* Headwater at Station 467.1 and Tailwater at Station 466.9

Table F-D3 HEC-RAS Outputs for the Proposed FCP Control Structure at the Red River of the North for Year 50 Hydrology

Event (yr)	Existing Flow into Protected Area (cfs)	Proposed Flow through Gates into Protected Area (cfs)	Proposed Headwater Elevation in Red River* (ft)	Proposed Tailwater Elevation in Red River* (ft)	Head Diff-erence (ft)	Gate Open Height (ft)	Average Velocity through Gates (ft/s)
2	3,506	3,506	888.7	888.68	0.02	49	0.99
5	9,161	9,161	897.39	897.33	0.06	44.1	1.4
10	13,965	9,546	902.66	898.06	4.6	4.6	13.8
20	18,855	9,588	905.56	898.35	7.21	3.7	17.3
50	25,764	9,649	908.31	898.67	9.64	2.95	21.8
100	31,304	10,176	910.03	899.32	10.71	2.8	24.2
200	38,787	12,071	911.66	900.92	10.74	3.4	23.7
500	54,034	19,605	913.32	906.04	7.28	7.5	17.4

* Headwater at Station 467.1 and Tailwater at Station 466.9

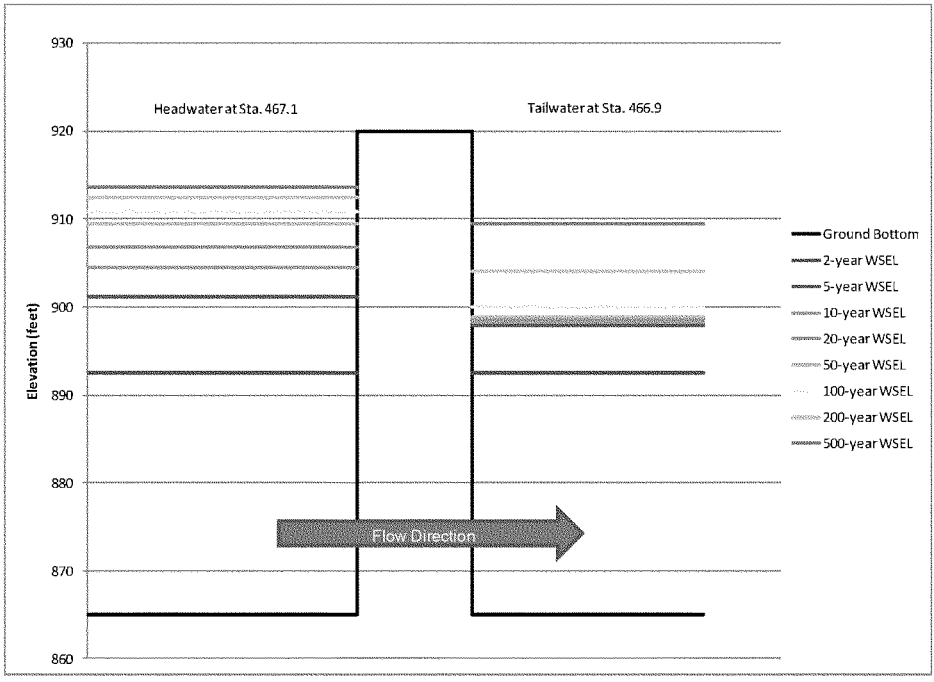

Figure F-D2 Comparison of the Headwater and Tailwater Elevations at the FCP Control Structure for Year 0 Hydrology

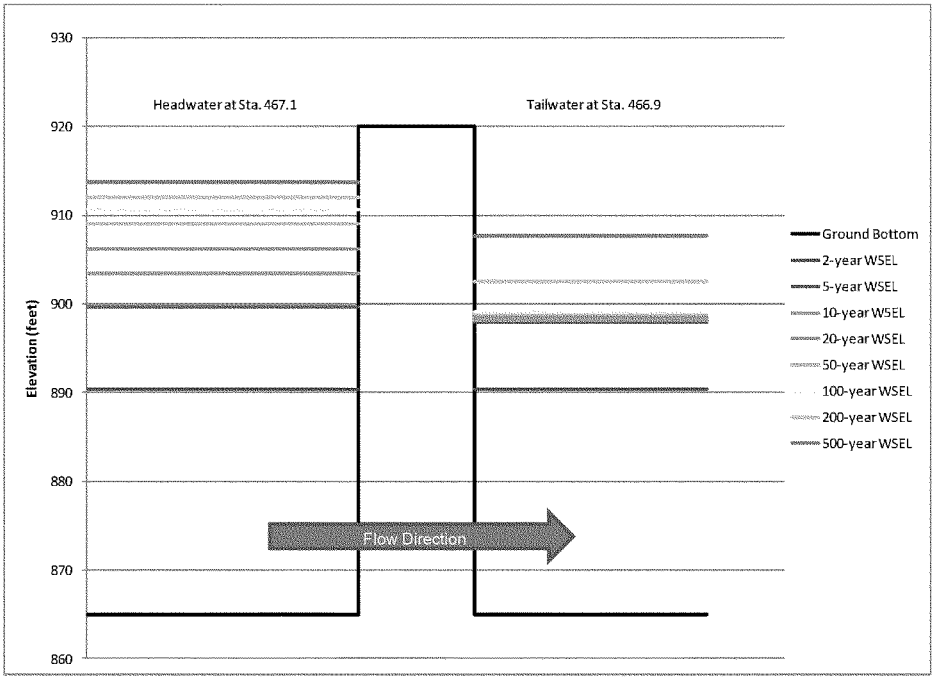

Figure F-D3 Comparison of the Headwater and Tailwater Elevations at the FCP Control Structure for Year 25 Hydrology

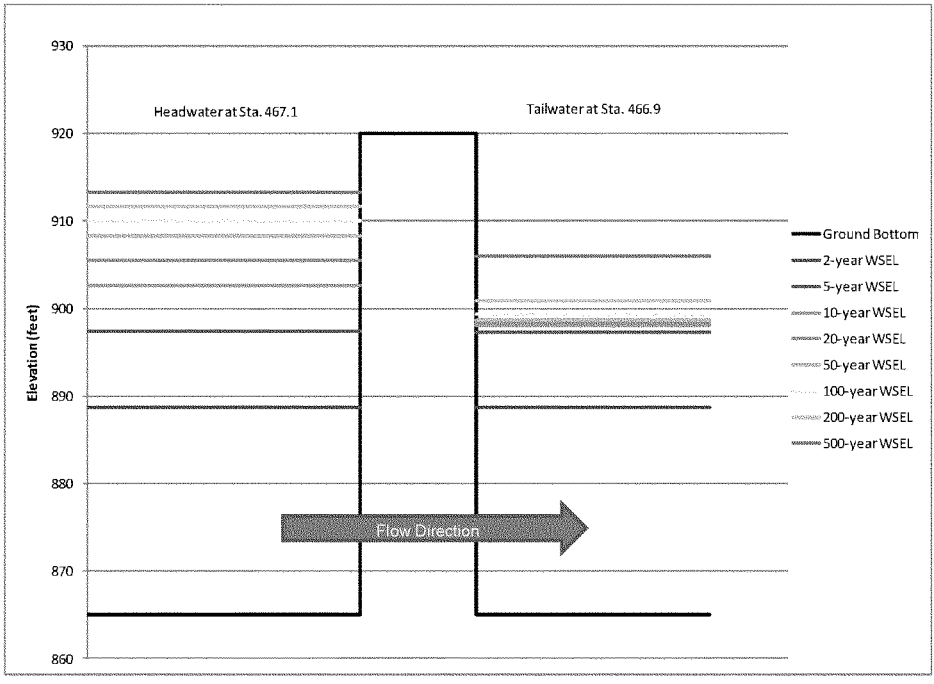

Figure F-D4 Comparison of the Headwater and Tailwater Elevations at the FCP Control Structure for Year 50 Hydrology

F-D1.1.5 Transition from the Natural Channel to the Control Structure

The transition from the natural channel into the control structure is accomplished with 7:1 side slopes, as shown in Figure F-D5.

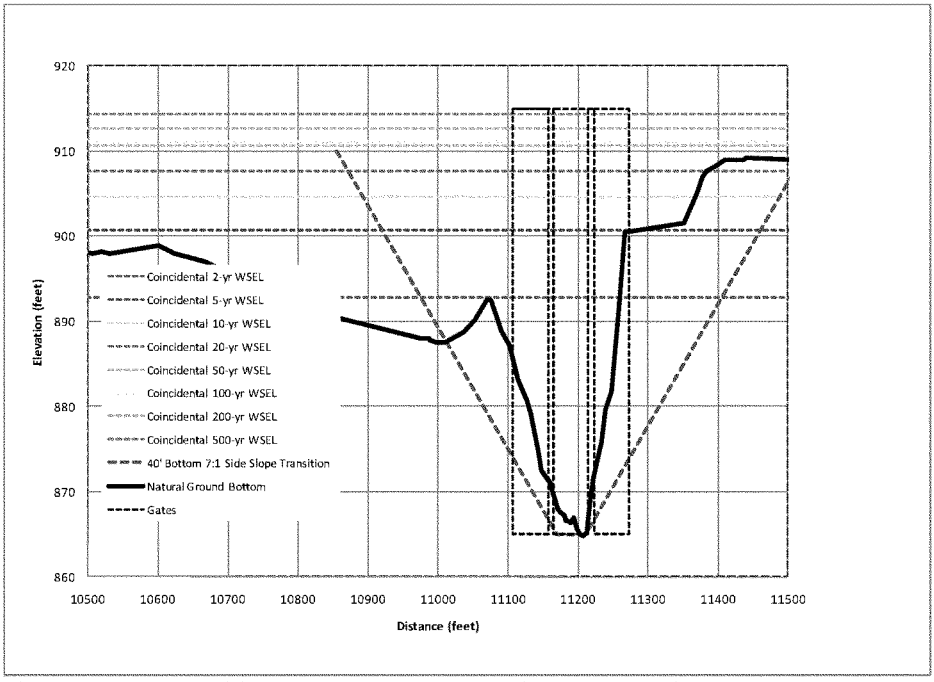

Figure F-D5 Transition from the Natural Red River Cross Section to the FCP Control Structure

F-D1.2 INLET WEIR AT RED RIVER OF THE NORTH

A description of the inlet weir to the FCP diversion can be found in Appendix F. This portion of Exhibit D presents the design dimensions of the weir as well as detailed HEC-RAS modeling results at the inlet weir location using Phase 3 hydrology.

F-D1.2.1 Design of Inlet Weir at Red River of the North

The structure designed to pass water from the Red River to the Diversion Channel for the FCP alignment is a multi-tier weir, as shown in Figure F-D6. The design was created and optimized by Moore Engineering. HEC-RAS modeling results for the inlet weir for Year 0, Year 25, and Year 50 are presented in Tables F-D4, F-D5, and F-D6, respectively.

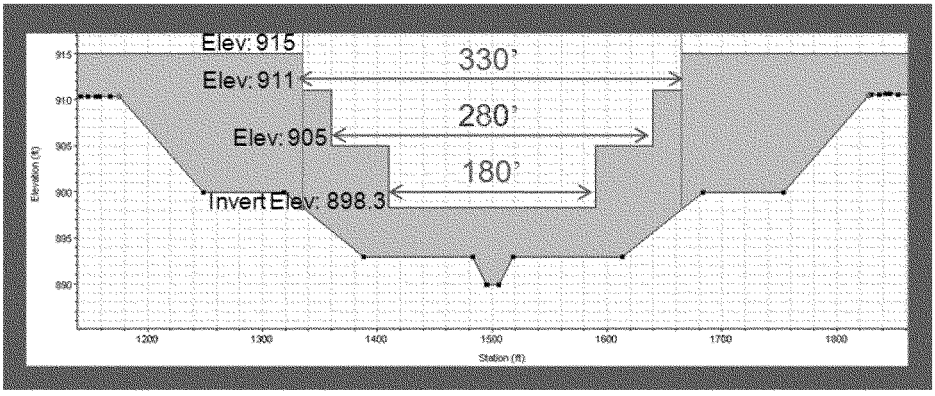

Figure F-D6 FCP Inlet Weir at Diversion Channel

Table F-D4 HEC-RAS Results for the Proposed FCP Inlet Weir at the Diversion Channel for the Red River of the North (Year 0 Hydrology)

Event in Red River (year)	Q Total (cfs)	Q Weir (cfs)	*Headwater (ft)	**Tailwater (ft)	Head Difference (ft)	Flows Area (sq ft)	Velocity over Weir (ft/s)
5	2510	2510	901.35	897.65	3.70	549	4.57
10	7346	7346	904.50	901.90	2.60	1116	6.58
20	12241	12241	906.68	904.18	2.50	1676.4	7.30
50	19497	19497	909.13	906.91	2.22	2362.4	8.25
100	23771	23771	910.41	908.32	2.09	2720.8	8.74
200	29833	29833	912.02	910.13	1.89	3222.6	9.26
500	34762	34762	913.24	911.46	1.78	3625.2	9.59

*HEC-RAS cross section 127253

**HEC-RAS cross section 127233

Table F-D5 HEC-RAS Results for the Proposed FCP Inlet Weir at the Diversion Channel for the Red River of the North (Year 25 Hydrology)

Event in Red River (year)	Q Total (cfs)	Q Weir (cfs)	*Headwater (ft)	**Tailwater (ft)	Head Difference (ft)	Flows Area (sq ft)	Velocity over Weir (ft/s)
5	968	968	899.92	895.52	4.40	291.6	3.32
10	5747	5747	903.58	901.02	2.56	950.4	6.05
20	10604	10604	906.04	903.47	2.57	1497.2	7.08
50	17596	17596	908.52	906.25	2.27	2191.6	8.03
100	23003	23003	910.19	908.07	2.12	2659.2	8.65
200	28033	28033	911.57	909.62	1.95	3074.1	9.12
500	34955	34955	913.28	911.51	1.77	3638.4	9.61

*HEC-RAS cross section 127253

**HEC-RAS cross section 127233

Table F-D6 HEC-RAS Results for the Proposed FCP Inlet Weir at the Diversion Channel for the Red River of the North (Year 50 Hydrology)

Event in Red River (year)	Q Total (cfs)	Q Weir (cfs)	*Headwater (ft)	**Tailwater (ft)	Head Difference (ft)	Flows Area (sq ft)	Velocity over Weir (ft/s)
5	0	0	898.32	889.92	8.40	3.6	0.00
10	4419	4419	902.73	900.20	2.53	797.4	5.54
20	9267	9267	905.47	902.86	2.61	1337.6	6.93
50	16115	16115	908.04	905.71	2.33	2057.2	7.83
100	21128	21128	909.63	907.46	2.17	2502.4	8.44
200	26716	26716	911.23	909.22	2.01	2961.9	9.02
500	34429	34429	913.16	911.38	1.78	3598.8	9.57

*HEC-RAS cross section 127253

**HEC-RAS cross section 127233

F-D1.3 OUTLET TO RED RIVER OF THE NORTH

The Outlet of the Diversion Channel into the Red River of the North consists of riprap over the downstream 300 feet of the Diversion Channel. Details are provided in Appendix F.

F-D2.0 LOCALLY PREFERRED PLAN (LPP)

F-D2.1 CONTROL STRUCTURE ON THE RED RIVER OF THE NORTH

A description of the LPP control structure on the Red River of the North can be found in Appendix F. Appendix F, Exhibit B discusses the variety of structure types that were

investigated for this control structure during the previous design phases. This portion of Exhibit D presents design concepts and calculations for the Red River of the North control structure, detailed HEC-RAS modeling outputs, and comparisons of water surface elevations using Phase 4 hydrology and the HEC-RAS unsteady flow model.

F-D2.1.1 Flow Partitioning

The partitioning of the flow between the protected area downstream of the Red River Control Structure and through the inlet weir to the diversion channel is determined by the gate openings at the control structures located on the Red River, Wild Rice River, and Wolverton Creek. The resulting peak flows through the Red River Control Structure and the diversion channel inlet are presented in Table F-D8. The flows presented include 10-year, 50-year, 100-year, and 500-year events on the Red River of the North from Phase 4, as well as more frequent events from Phase 3 (which were not analyzed during Phase 4).

F-D2.1.2 Gate Number and Sizing – Concepts

The number and sizing of the gates in the Red River Control Structure is controlled by:

1.) Ability of the design to achieve the desired flow through the gates as determined by the target elevation at the USGS gage in Fargo, as shown in Table F-D7, while minimizing the impact to water surface elevations downstream of the Project.

2.) Provide sufficient capacity to convey low flows into the protected area and not restrict use of the river.

Table F-D7 Target Elevations at the USGS Gage in Fargo

Event	Target Elevation (NAVD88 ft)
10-Year	891.99
50-Year	892.74
100-Year	893.40
500-Year	902.66

Initially, three concepts which included operable gates above permanently open areas were considered and presented to the U.S. Army Corps of Engineers (USACE) during a meeting on December 2, 2009. Those concepts are shown in Exhibit B of Appendix F and include a design with three identical gates, a design with five identical gates, and a design with two large identical gates combined with two smaller gates.

Based on discussion with the USACE and preliminary modeling of these concepts during Phase 2, the concept with 3 identical gates was selected. This selection was made because it allowed a wider gate width (for ice and boat passage) and redundancy of gates for operation and maintenance issues.

During Phase 4 the number of gates and gate widths did not change from Phase 3. As a result of staging water on the Red River upstream of the Project the gate height increased to prevent water from overtopping the Red River Control Structure. In addition, in order

to control the amount of flow into the protected area and upstream staging throughout the duration of the event, preliminary gate operations that resulted in variable gate openings over the pass of the hydrograph were developed for each flood event to control the flow through the Red River Control Structure.

F-D2.1.3 Gate Number and Sizing – Optimization

During Phase 3, following the recommendation from USACE – Environmental, a width of 50 feet was selected for each of the three gates, and this was not changed during Phase 4. Preference was given for multiple, identically sized gates for operation and maintenance purposes. Also, it is desirable to have as little impact on the smaller, more frequent, flood events (i.e. no change in WSEL or gate operation required); thus, a minimum open width is necessary to avoid flow constriction during smaller flood events.

After the three identical gate concept was selected for further analysis, preliminary gate operations were developed to control the amount of flow through the control structure. During Phase 4, the HEC-RAS unsteady flow model was used to further refine the optimization of the required gate operations throughout the duration of each flood event.

During previous design phases, the 500-year event (during which the flow through the structure is the greatest) resulted in the greatest open area and therefore could be used to determine the minimum number of gates required. For all smaller flood events, the gates may be partially or fully closed to achieve the desired flow into the protected area. During Phase 4 the gates are partially closed for each of the flood events analyzed (10-year, 50-year, 100-year, and 500-year). As a result the gate height is defined by the elevation that water is staged upstream of the Project. In optimizing the design of the Red River Control Structure for the LPP alignment, 50-feet tall gate openings were used for the LPP diversion. The number of gates necessary for the LPP (as determined during Phase 2) is three 50-feet wide gates. A concept drawing for the control structure is presented in Figure F-D7.

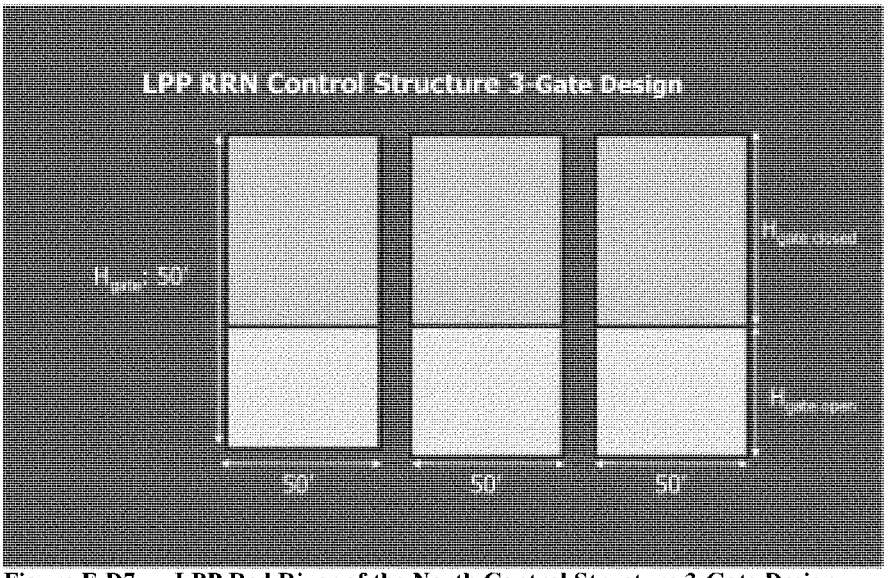

Figure F-D7 LPP Red River of the North Control Structure 3-Gate Design

F-D2.1.4 Gate Operation Optimization and HEC-RAS Results
Preliminary gate operational scheme was developed to achieve

1.) Minimize downstream impacts
2.) Maintain a target flood elevation at the USGS gage in Fargo
3.) Limit the amount of upstream staging.

In general, gate operations included lowering the gate at the beginning of the event, maintaining a constant opening through the peak of the event, and then opening the gates at the end of the event. For each flood event, there are multiple potential gate operational schemes. By simply varying how the gates are operated can result in moving from a decrease to an increase in downstream water surface elevations. The gate operational scheme presented for each of the flood events is one possible way to control how water is conveyed downstream of the Project. During final design, a complete analysis of how gate operations impact the downstream water surface elevations should be completed.

Following discussions with the USACE it was determined to utilize user defined rules to calculate the flow through the control structure, rather than the default HEC-RAS gate routines. This was done primarily because the default gate routines in HEC-RAS for tainter gates assume the gates are elevated over a sill, which does not accurately characterize the proposed gates at the Red River control structure, which are located near the bottom of the river without a sill, therefore weir flow conditions would not apply. Based on discussions with the USACE and review of data provided by the USACE including field measurements from Mississippi River Lock and Dams 6-10 and studies complete by the Bureau of Reclamation, it was found that the orifice equation could be

used to characterize the flow through the gates throughout the duration of the event. The orifice equation could be used throughout the duration of the event due to the high levels of submergence at the control structure gates throughout the duration of the event (which are a result of staging and the large flood events analyzed as part of Phase 4). The orifice discharge coefficient was modified between 0.8, for submergences less than 0.67, to 1.0 when the submergence is greater than 0.8 (and linear interpolation in between), based on the level of submergence for each time step.

User defined rules were also utilized to control the gate opening throughout the duration of the event. This provided the flexibility to control the gate opening based on the water surface elevation at the USGS gage in Fargo, and to have additional control over the amount of water being let into the protected area throughout the duration of a flood event.

The operation scheme of the gates proposed in this feasibility analysis is as follows. At the beginning of each flood event the gates begin to close so that they are in the lowest position at the beginning of the event, or at the beginning of the rising limb of the hydrograph. The gates remain in the closed position during the first half of the flood event, when the majority of the flow is conveyed through the Diversion Channel. Only after the peak flow rate has passed through the downstream end of the Diversion Channel the gates on the Red River begin to open. Allowing the peak flow in the Diversion Channel to reach the Red River before the control structure gates begin to open results in decoupling the peak flow rate in the Diversion Channel from the peak flow rate on the Red River. This allows flood waters to be diverted around the Cities while minimizing downstream impacts to the floodplain.

When the gates start to open, they are opened at a rate to increase water surface elevations at the USGS gage in Fargo to the target elevation. As a result of allowing the peak flow on the Diversion Channel to pass through the system first, the control structure gates may be opened either faster, to reduce the duration that water is stored upstream of the Cities or slower to further reduce the water surface elevation at the USGS gage in Fargo. This allows the Cities to potentially achieve additional protection beyond the Phase 3 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 potential additional benefit warrants further evaluation during future design phases. Based on correspondence with the USACE of February 12, 2011 it was determined that the Phase 4 models should match the target elevations at the USGS gage within 0.10-0.15 ft. Matching the target elevation to within 0.15 ft would match the project benefits calculated by the USACE within 5% from Phase 3.

In the HEC-RAS unsteady flow models, the Phase 4 gate operations only open the gates 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 at Fargo. If the gates are operated continuously 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.

Maximum flows, water surface elevations, head difference, gate opening height and velocities during each flood event at the LPP Red River of the North control structure are presented in Table F-D8. These results were extracted from the HEC-RAS models with the exception of average velocity through the gates. HEC-RAS does not provide this result so it was calculated by dividing the discharge by the flow area. The location of the control structure in the LPP HEC-RAS model is inline structure station 2529023. Figure F-D8 show the comparison of the headwater and tailwater elevations at the LPP Control Structure.

Operating the gates at the Red River control structure restricts the flow that is conveyed into the protected area. As a result water is stored in the Red River and adjacent floodplain upstream of the control structures. Figures F-D9 – F-D12 show how operating the gates extends the duration at which flood waters are stored upstream of the control structures.

Table F-D8 HEC-RAS Outputs for the Proposed LPP Control Structure at the Red River of the North

	Phase 3							Phase 4			
Event (yr)	Existing Flow into Protected Area (cfs)	Proposed Flow through Gates into Protected Area (cfs)	Proposed Headwater Elevation in Red River* (ft)	Proposed Tailwater Elevation in Red River* (ft)	Head Difference (ft)	Gate Open Height (ft)	Average Velocity through Gates (ft/s)	Existing Flow into Protected Area (cfs)	Proposed Max. Flow through Gates into Protected Area[3] (cfs)	Proposed Max. Headwater Elevation in Red River** (ft)	Proposed Max. Tailwater Elevation in Red River** (ft)
2[+]	3,139	3,139	894.53	894.51	0.02	44.00	1.10	-	-	-	-
5[+]	6,160	5,497	905.18	902.21	2.97	3.30	11.1	-	-	-	-
10	10,500	7,051	908.83	902.69	6.14	2.95	15.9	10,271	9,407	916.29	904.40
50	21,000	10,918	913.38	903.77	9.61	3.00	24.3	18,207	10,178	920.85	905.15
100	25,000	12,252	914.87	904.65	10.22	3.20	25.5	21,458	11,569	922.87	906.37
500	32,000	13,077	917.84	914.1	3.74	7.00	12.5	28,623	30,569	922.33[4]	914.84

[+] Phase 3 feasibility design steady flow models

* Headwater at Station 478.77 and Tailwater at Station 478.76

** Headwater at Station 2529066 and Tailwater at Station 2528979

[3]Flow into protected area listed is maximum flow. Due to operation of the gates this does not correspond to the same time as the maximum head differential across the control structure.

[4] Maximum elevation for 500-year event is lower than the 100-year event due to gate operations required to minimize impacts to the floodplain downstream of the Project

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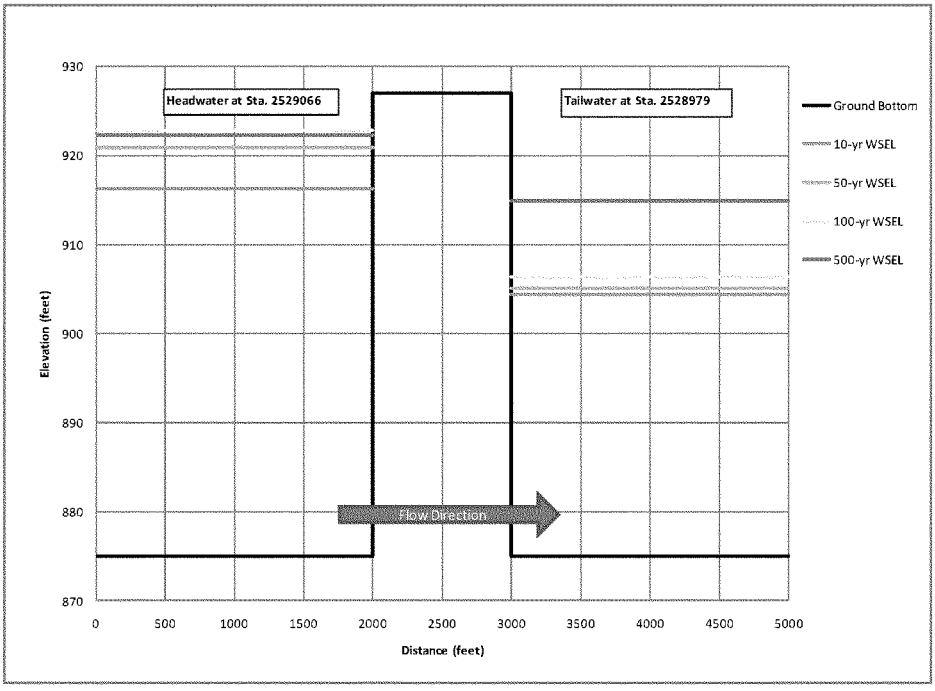

Figure F-D8 Comparison of the Headwater and Tailwater Elevations at the LPP Control Structure

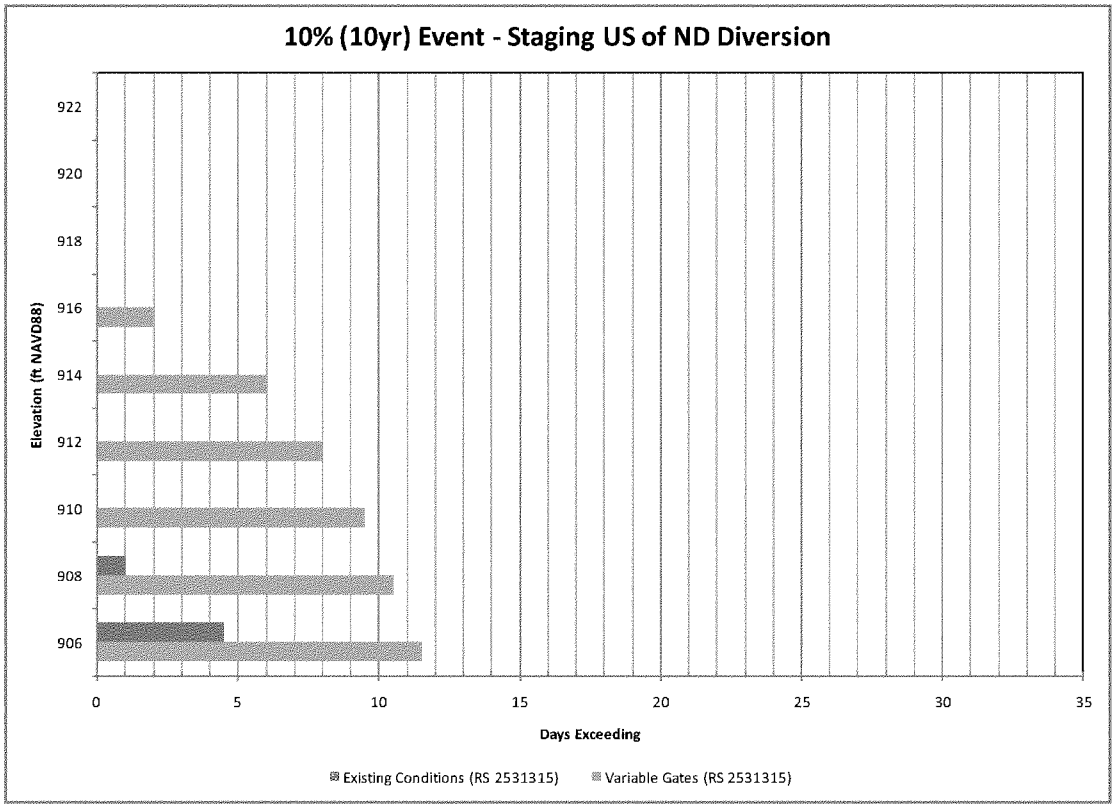

Figure F-D9 Comparison of Increased Water Surface Elevations Upstream of the Red River Control Structure for the 10-Year Event

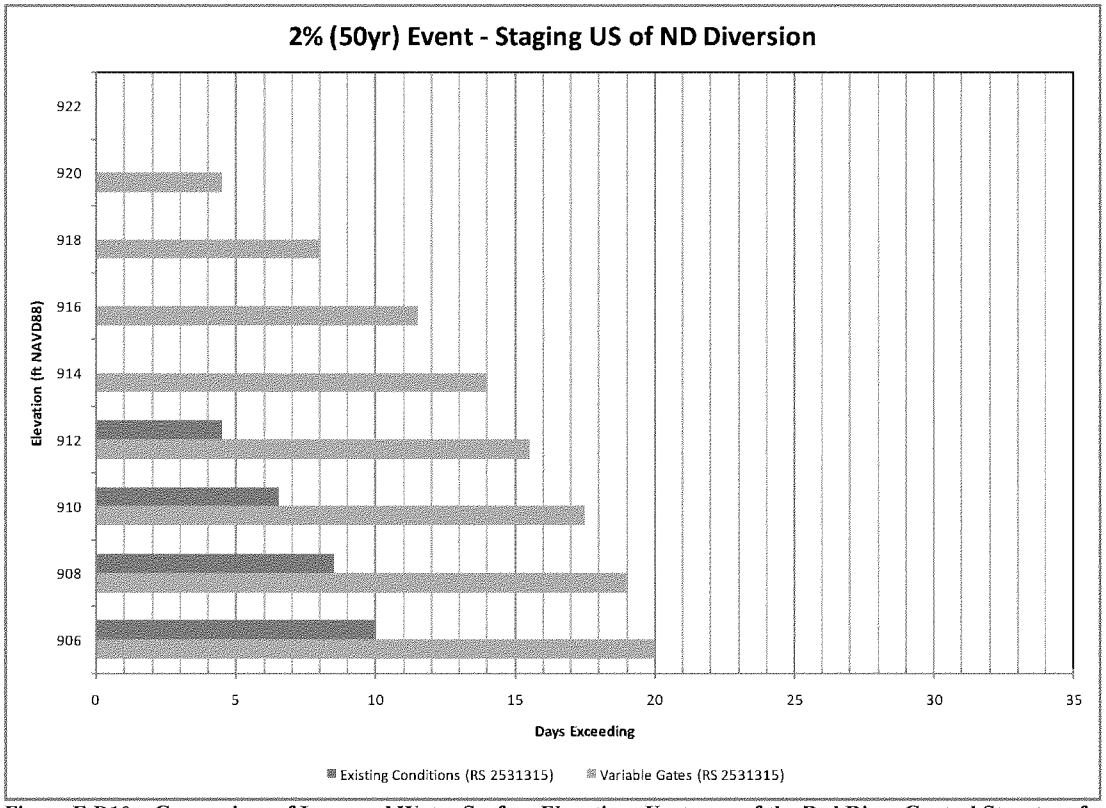

Figure F-D10 Comparison of Increased Water Surface Elevations Upstream of the Red River Control Structure for the 50-Year Event

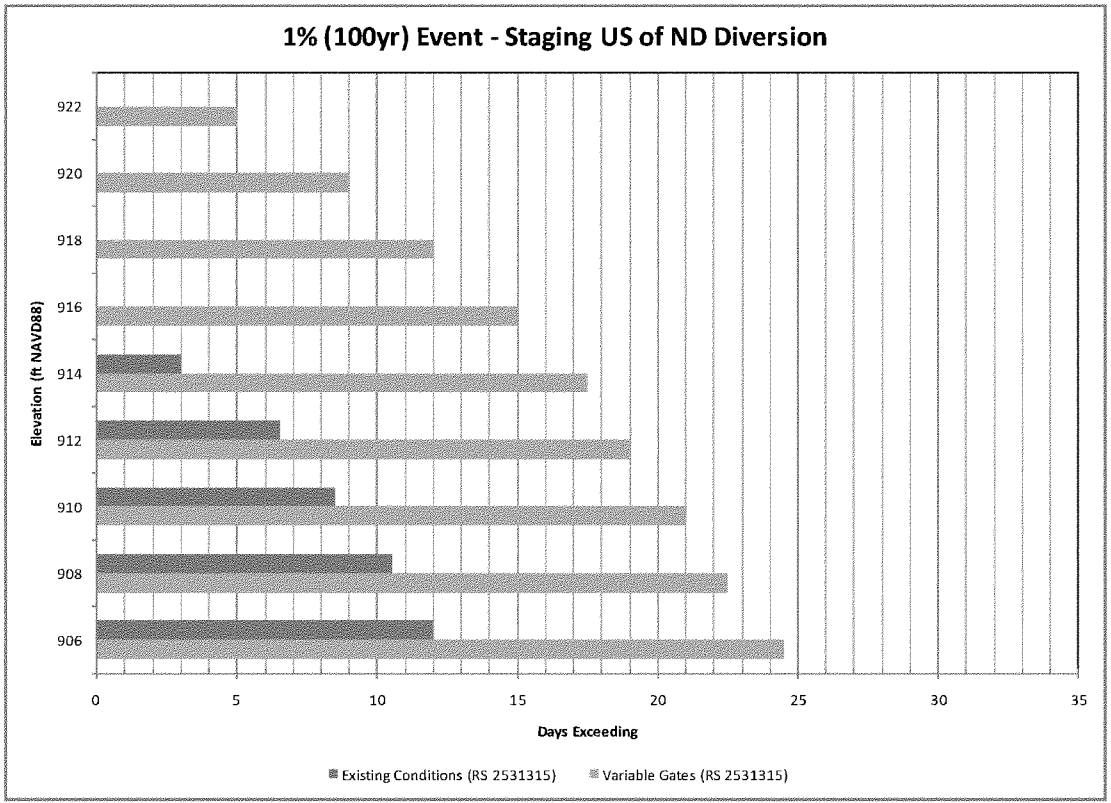

Figure F-D11 Comparison of Increased Water Surface Elevations Upstream of the Red River Control Structure for the 100-Year Event

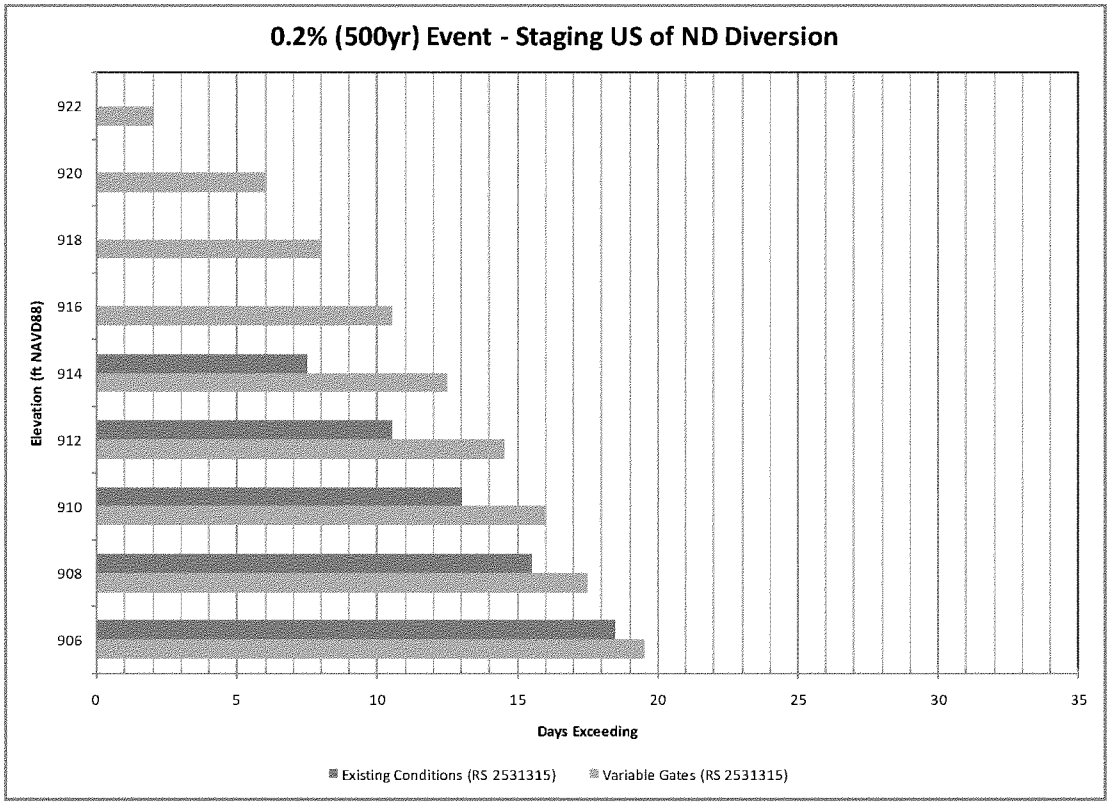

Figure F-D12 Comparison of Increased Water Surface Elevations Upstream of the Red River Control Structure for the 500-Year Event

F-D2.1.5 Initial Gate Operation

During Phase 3 feasibility design with the HEC-RAS steady flow models, it was assumed that flow was not diverted into the Diversion Channel until 9,600 cfs occurred at the USGS gage in Fargo. (However, for evaluation of downstream impacts in Phase 3, the use of HEC-RAS unsteady flow model showed that the start of gate closing needed to begin for flows smaller than 9,600 cfs when the peak of the hydrograph exceeded 9,600 cfs.) Phase 4 analysis using the HEC-RAS unsteady flow model indicates that in order to minimize impacts to the floodplain downstream of the project for events larger than 9,600 cfs, the gates at the control structures will need to be operated during the rising limb of the hydrograph, before 9,600 cfs occurs at the USGS gage in Fargo. With regards to operation of the control structure gates, the Phase 4 analysis found that:

1. The gates will not be operated when the forecasted peak flow 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.

2. In order to achieve the goal of no downstream impacts for events larger than 9,600 cfs, the gates must operate on the rising limb of the hydrograph – and this might start before 9,600 cfs is reached in Fargo.

3. The Cities of Fargo and Moorhead may determine not to operate the system for events larger than 9,600 cfs (for instance during summer floods, when historic peak flows are max 12,000-13,000 cfs or a stage of 30 = the beginning of major flooding).

HEC-RAS unsteady flow models completed as part of the Phase 4 analysis start with the gates beginning in a position that is less than fully open (the gates are sufficiently open such that water is not staged upstream of the control structure at the very beginning of the simulation). For all events, the gates are then gradually closed over several days so that they reach the minimum opening during the rising limb of the hydrograph. This was done to keep the HEC-RAS model computationally stable and 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 resulting in downstream impacts. This further evaluation (optimization of the gates operation to reduce their duration of operation) should be considered in future phases.

Table F-D9 includes a summary of the flows and stages in the Phase 4 HEC-RAS models for each of the 4 design events at which the gates would begin restricting flows passed into the protected area.

Table F-D9 Maximum Flow and Stage at the Fargo Gage before Control Structure Gates Restrict Flow into the Protected Area

Design Event	Maximum Flow in Fargo before Gate begins restricting flow	Maximum Stage in Fargo before Gate begins restricting flow
10-Year	1,800	878.8
50-Year	2,400	880.5
100-Year	2,700	881.2
500-Year	5,300	885.4
USGS Gage Rating Curve	*9,600*	*~891.5*

In general, in order to minimize the impacts to the floodplain downstream of the project the gates begin to restrict flow earlier for more frequent events. This is primarily due to two reasons.

1. Downstream impacts are caused by the volume and timing along the entire rising limb of the hydrograph - not just the peak flow 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.

2. 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.

Although the system goes into operation before 9,600 cfs at the Fargo gage, the system only begins operation approximately 7-10 days before 9,600 cfs would occur at the Fargo gage for the 100-year synthetic design event.

Flows presented in Table F-D9 above are the ones associated to a given shape of the synthetic hydrographs (i.e., the design floods). If the hydrograph shape would be different, while maintaining the hydrograph peak flow for a given return period, the answer could be different regarding when the gates need to be operated. Finally, it is also important to acknowledge that in order to eliminate impacts downstream for relatively frequent events (like the 10-yr event), even when these impacts are so small that they do not significantly increase the risk of flooding in the downstream communities, project design considerations are "obligating" operation of the system in advance of the 9,600 cfs. Figures F-D13 – F-D16 show how the Red River control structure gates operate for each event, as well as how the peak flow rates in the diversion channel and Red River are decoupled to minimize downstream impacts.

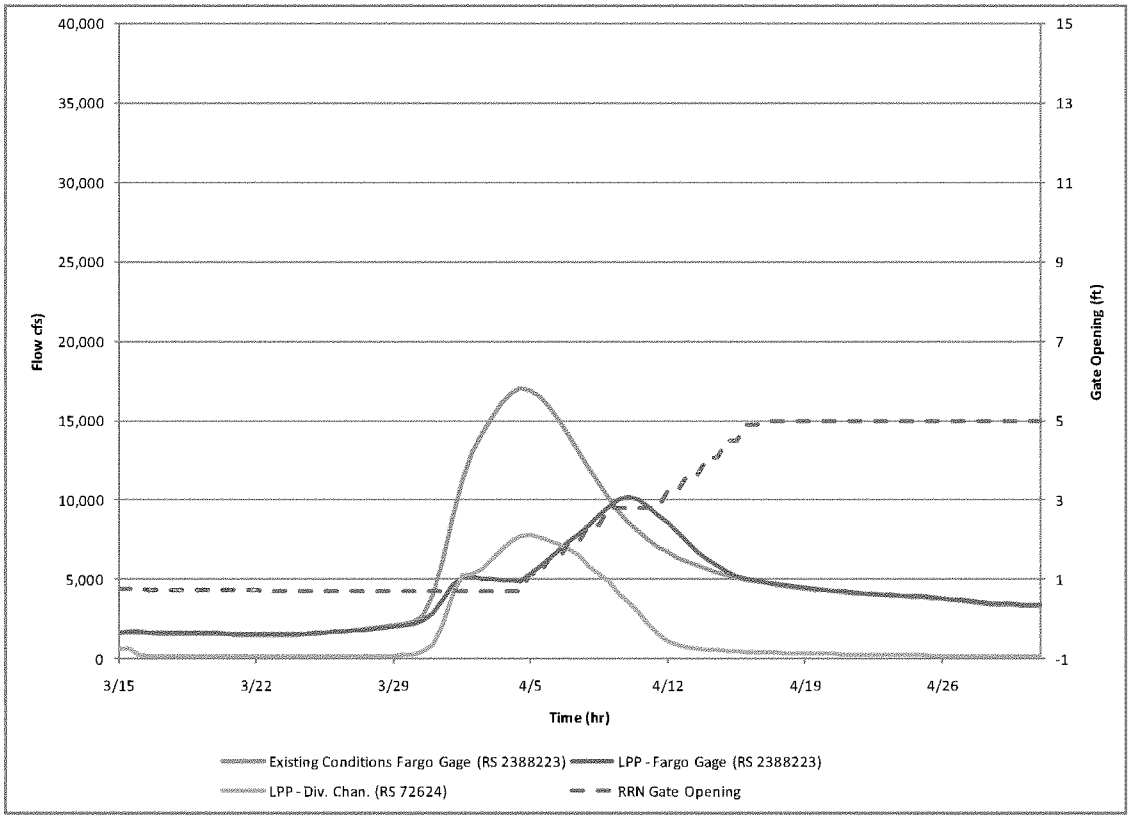

Figure F-D13 Decoupled Peak Flows in the Diversion Channel and Red River for the 10-Year Event

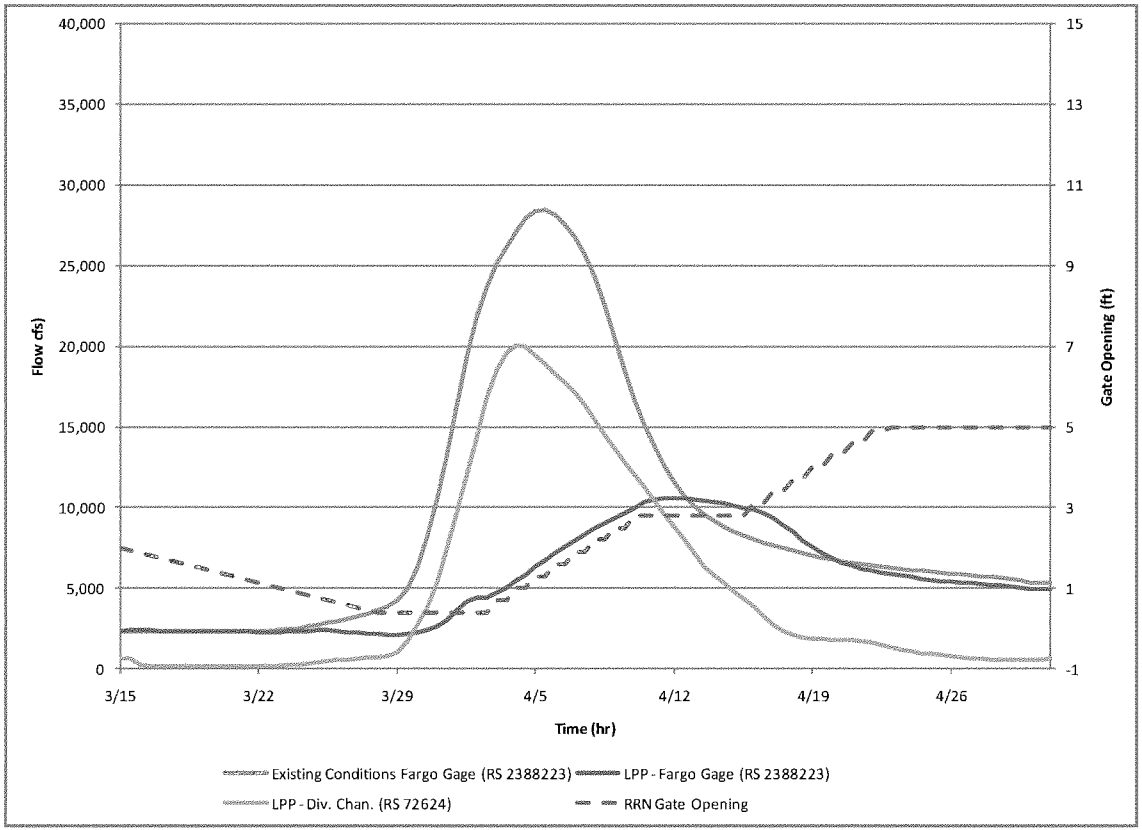

Figure F-D14 Decoupled Peak Flows in the Diversion Channel and Red River for the 50-Year Event

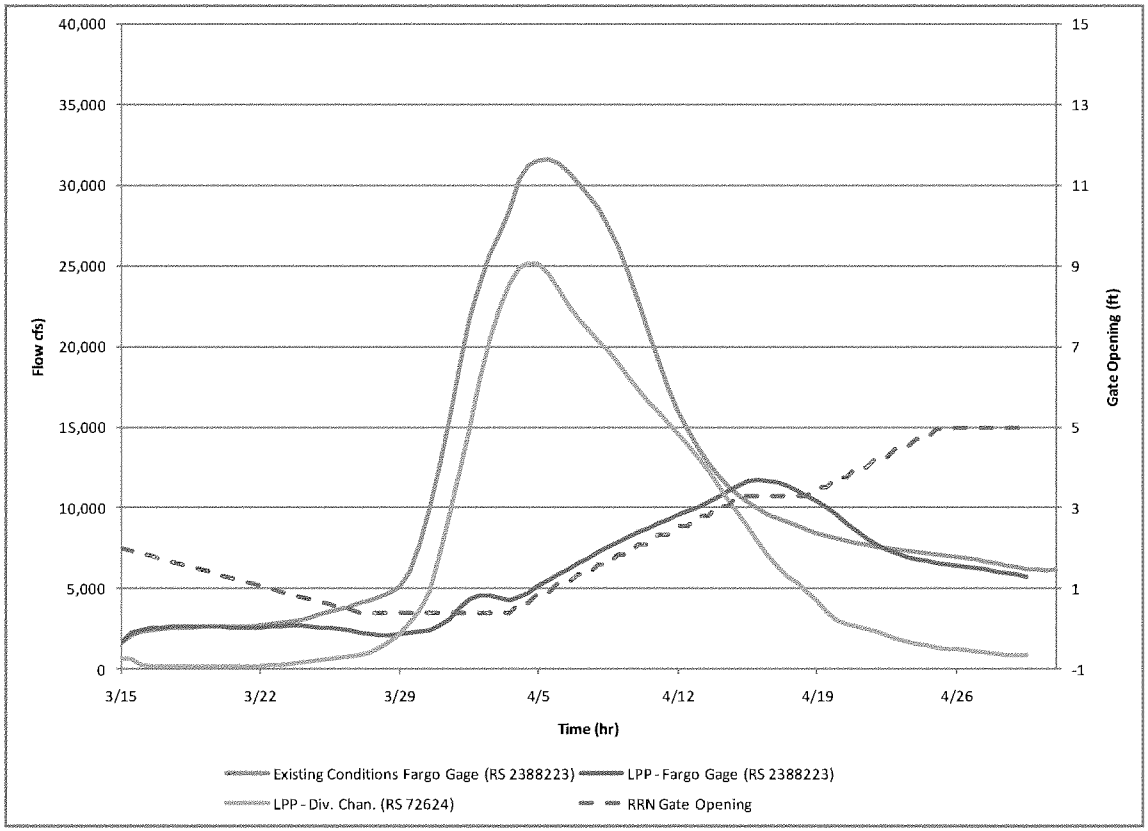

Figure F-D15 Decoupled Peak Flows in the Diversion Channel and Red River for the 100-Year Event

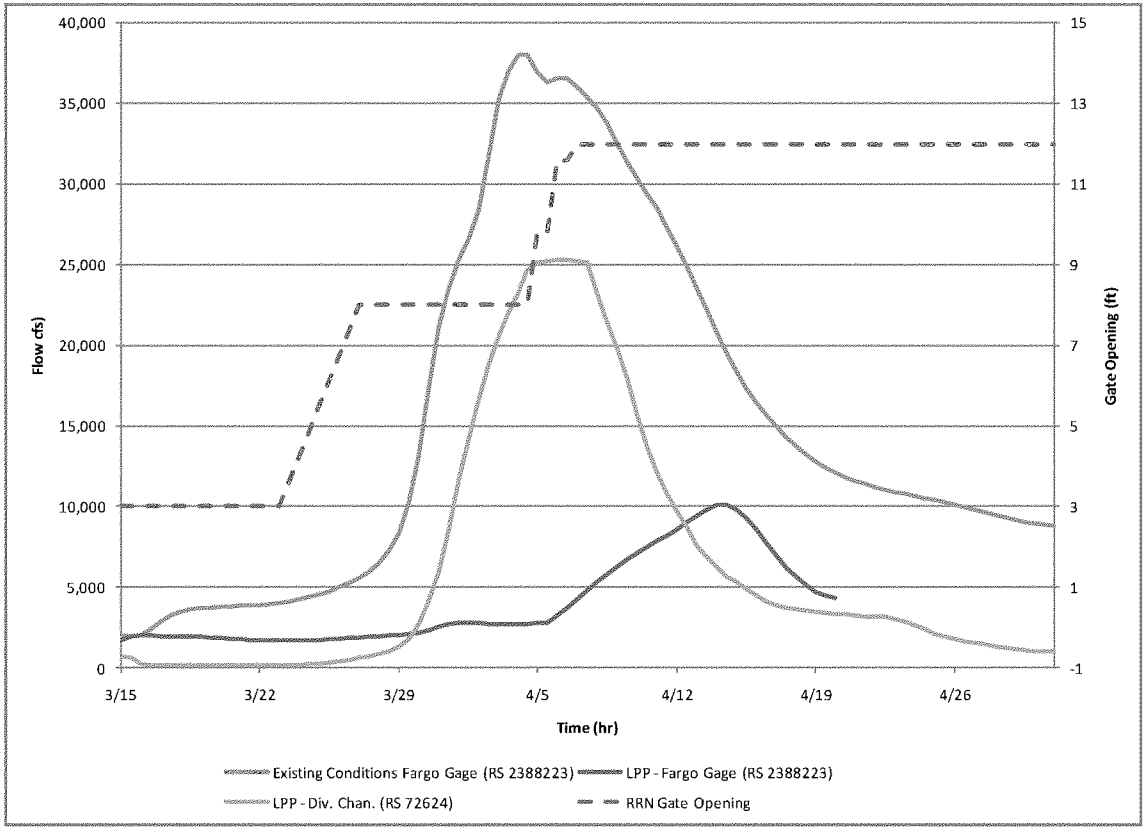

Figure F-D16 Decoupled Peak Flows in the Diversion Channel and Red River for the 500-Year Event

F-D2.1.6 Transition from the Natural Channel to the Control Structure
The transition from the natural channel into the control structure is accomplished with 7:1 side slopes, as shown in Figure F-D17. For more details on the geotechnical slope stability analysis of this reach, see Exhibit N of Appendix F.

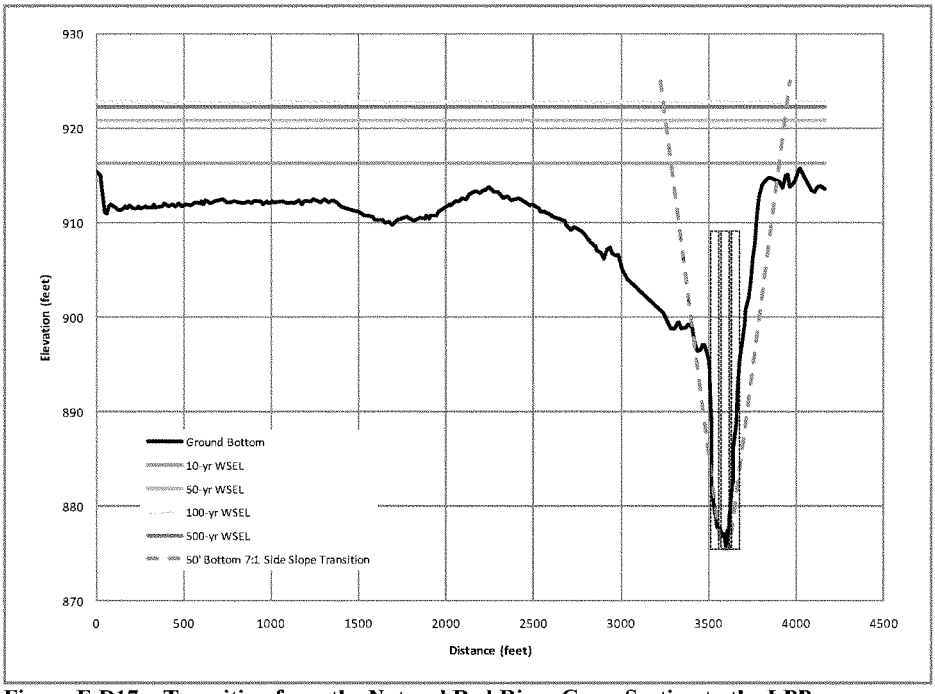

Figure F-D17 Transition from the Natural Red River Cross Section to the LPP Control Structure

F-D2.2 INLET WEIRS AT WILD RICE RIVER

A description of the passive (i.e. without moveable parts like gates) diversion inlet weir that is used to control water entering the LPP Diversion Channel from the Red River of the North and Wild Rice Rivers can be found in the main section of Appendix F. This portion of Exhibit D presents a comparison of water surface elevations at the Connection Channel and the diversion inlet weir (or Inlet Structure) and detailed HEC-RAS modeling outputs using Phase 4 hydrology.

F-D2.2.1 Comparison of LPP Diversion Channel and Wild Rice River Hydrology
Flows, water surface elevations (WSEL), and average velocities for the LPP Diversion Channel just downstream of the Inlet Structure to the west of the Wild Rice River (Station 152517) for the 10-, 50-, 100-, and 500-year events are presented in Table F-

D10. Water surface elevations in the LPP Diversion Channel just downstream of the primary Inlet Structure (Station 152517), in the Wild Rice River for local tributary flood events, and in the Wild Rice River for coincidental flood events, plotted against the cross section data, are presented in Figure F-D18. Tabular results of local and coincidental flood events for existing conditions in the Wild Rice River are presented in Appendix F, Exhibit A.

Table F-D10 LPP Diversion Channel Downstream of the Inlet Structure to the East of the Sheyenne River

	Phase 3[1]			Phase 4[2]		
Event (yr)	Flow (cfs)	WSEL (ft)	Average Velocity (ft/s)	Flow (cfs)	WSEL (ft)	Average Velocity (ft/s)[3]
5	2,534	896.57	2.03	-	-	-
10	7,284	901.88	2.42	7,207	896.57	2.31
50	19,387	908.70	3.09	15,737	902.44	2.76
100	24,178	910.95	3.22	19,046[4]	904.44	2.82
500	34,915	915.35	3.48	18,404	905.46	2.56

[1] Phase 3 data from station 173349 from the Phase 3 HEC-RAS steady flow model. All Phase 3 information based on Year 0 hydrology.
[2] Phase 4 data from station 152517 from the Phase 4 HEC-RAS unsteady flow mode.
[3] Maximum average velocity at time of maximum water surface elevation (which does not always occur at the same time as the peak flow)
[4] Peak flow downstream of the Inlet structure is controlled by the amount of upstream staging, which is larger for the 100-year than the 500-year

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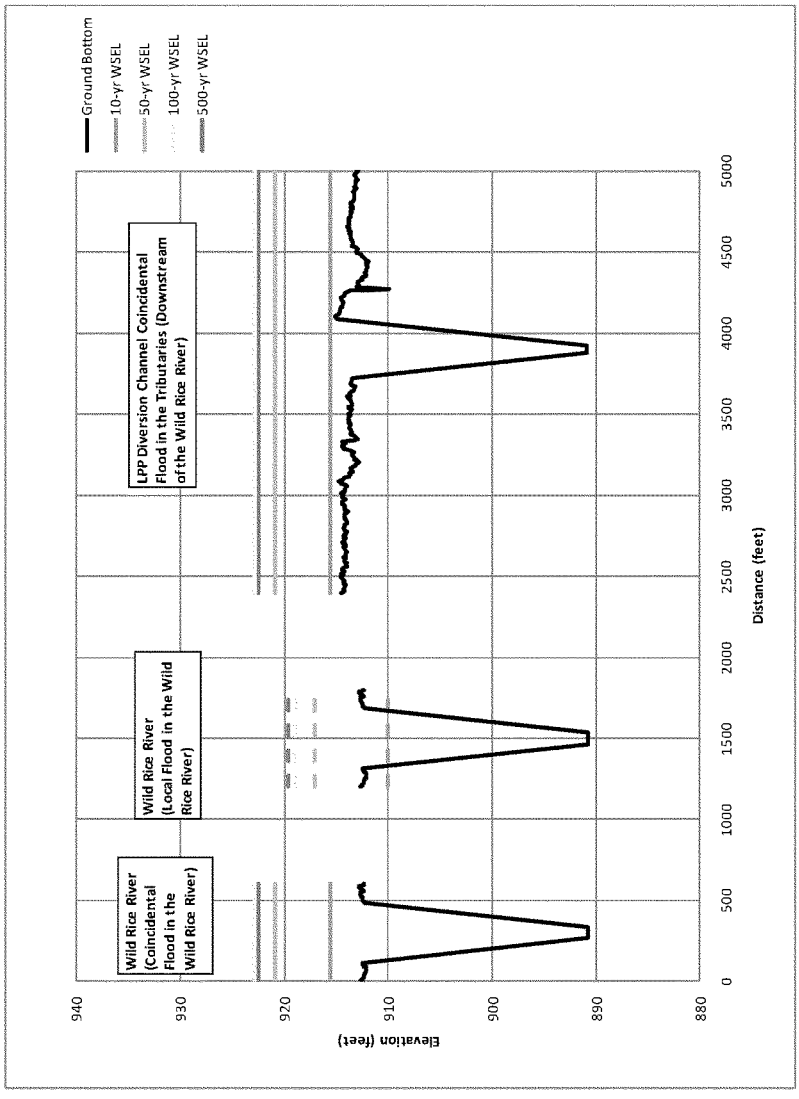


Figure F-D18 Hydrology in the LPP Diversion Channel Downstream of the Primary Inlet Structure and in the Wild Rice River

F-D2.2.2 Connecting Channel Weir (East Weir)
This hydraulic structure was included in the Phase 3 feasibility design, but it has been dropped from the Phase 4 design because it is not needed given the upstream staging.

F-D2.2.3 Primary Inlet Structure (West Weir)
As a result of modifications to the Diversion Channel there is an approximately 19-ft drop from the spill crest elevation of the inlet weir to the bottom of the Diversion Channel. In order to securely convey flow into the Diversion Channel, the broad crested type inlet weir designed for Phase 3 was modified to an ogee type spillway. To prevent ponding at the inlet weir, the Diversion Channel east of the weir is sloped back towards the Wild Rice River. Figure F-D19 and Figure F-D20 show plan and profile schematics of the proposed Inlet Structure. Additional details regarding the proposed dimensions and location of the primary Inlet Structure on the Diversion Channel to the west of the Wild Rice River are shown in Appendix F.

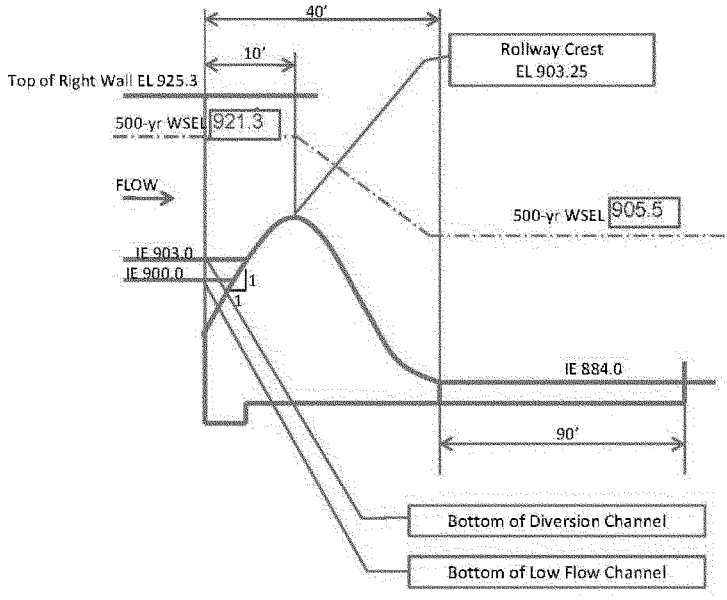


Figure F-D19 Ogee Weir Inlet to Diversion Channel Profile View Schematic

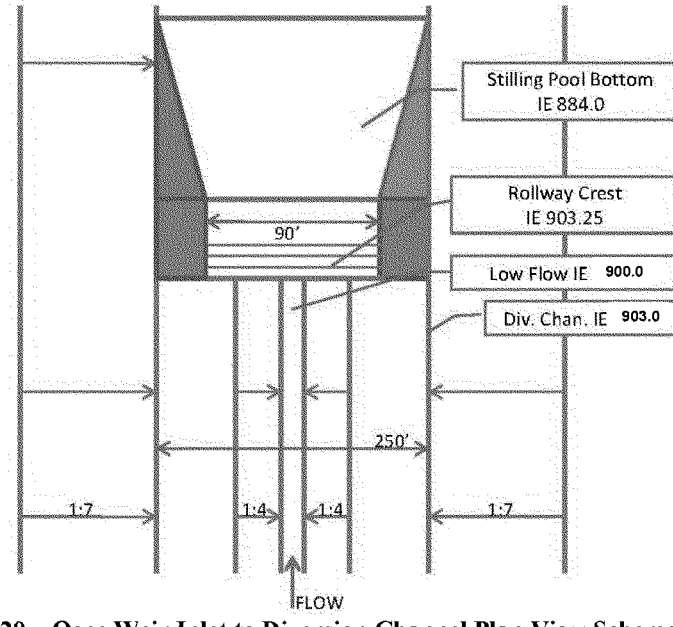


Figure F-D20 Ogee Weir Inlet to Diversion Channel Plan View Schematic

The maximum flow over the weir, headwater elevation, tailwater elevation, head difference, and velocities over the inlet weir for the 10-, 50-, 100-, and 500-year events are presented in Table F-D11. These variables were obtained from the HEC-RAS models with the exception of velocity over the weir. HEC-RAS does not provide this variable so it was calculated by dividing the flow by the flow area over the inlet weir. A comparison of headwater and tailwater elevations across the inlet structure (Sta. 152522) are presented in Figure F-D21 for peak flows on the Red River using Phase 4 hydrology.

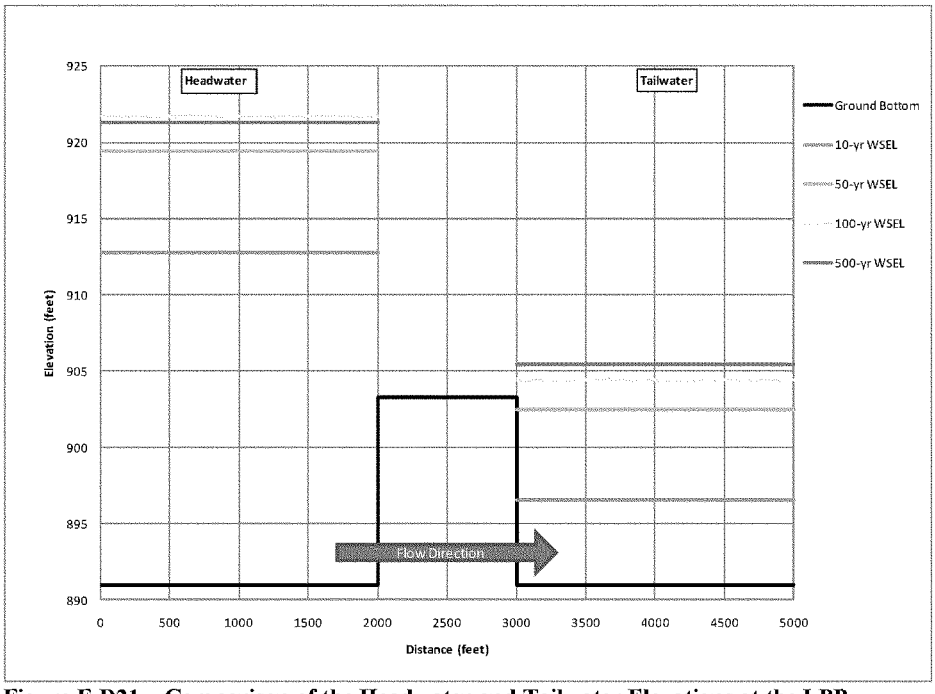


Figure F-D21 Comparison of the Headwater and Tailwater Elevations at the LPP Diversion Channel Inlet Weir for Coincidental Hydrology

Table F-D11 HEC-RAS Maximum Outputs for the LPP Diversion Channel Primary Inlet Weir

	Phase 3[1]					Phase 4				
Event (yr)	Flow over Weir (cfs)	Headwater Elevation (ft)	Tailwater Elevation (ft)	Head Difference (ft)	Velocity (ft/s)	Max. Flow over Weir (cfs)[2]	Max. Headwater Elevation (ft)[2]	Max. Tailwater Elevation (ft)[2]	Head Difference (ft)	Max. Velocity (ft/s)[2]
5	2,534	906.18	896.69	9.49	3.97	-	-	-	-	-
10	7,284	908.51	902.03	6.48	5.64	7,207	912.74	896.57	16.17	8.2
50	19,387	912.75	908.92	3.83	7.82	15,737	919.44	902.44	17.00	10.6
100	24,178	914.16	911.18	2.98	8.42	19,046	921.71	904.44	17.27	11.3
500	34,915	916.93	915.60	1.33	8.35	18,404	921.28	905.46	15.82	11.1

[1] Phase 3 Year 0 hydrology

[2] Maximum headwater elevation and maximum tailwater elevation do not occur at same time due to gate operations.

F-D2.3 CONTROL STRUCTURE ON THE WILD RICE RIVER

For the LPP alignment, a control structure on the Wild Rice River is necessary at the confluence of the Connecting Channel and the Wild Rice River. The combination of the Wild Rice River Control Structure and the Inlet Structure (primary inlet to the diversion channel; see Section F-D2.2) split the upstream flow in the Connecting Channel (during flood events) and the upstream flow in the Wild Rice River between the Diversion Channel and the protected area of the Wild Rice River. Based on Phase 3 hydrology, up to approximately between the 2-year and 5-year flood event in the Wild Rice River, it is expected that the Wild Rice River Control Structure will divert water into the Connecting Channel which will flow back to the Red River (this flow is the reverse of what will occur for larger flood events).

This portion of Exhibit D presents design concepts and calculations for the Wild Rice River Control Structure, detailed HEC-RAS modeling outputs, and comparisons of water surface elevations.

F-D2.3.1 Flow Partitioning

The partitioning of the flow between the protected area downstream of the Wild Rice River Control Structure and the Diversion Channel downstream of the Wild Rice River was performed using the HEC-RAS unsteady flow model. If the gates begin to close, in anticipation of a flood event that will exceed 9,600 cfs, it is possible for the inlet weir to be overtopped when a lower flow rate occurs at the USGS gage in Fargo. During the events analyzed as part of Phase 4, the gates at the Wild Rice River control structure are partially closed which results in water being stored upstream of the control structure.

F-D2.3.2 Gate Number, Sizing, and Optimization Using Orifice Equation and Modeling Results

The same general design concept (identical, wide gates) used for the Red River Control Structure was applied to the Wild Rice River Control Structure. Preference was given for multiple, identically sized gates for maintenance purposes. Also, it is desirable to have minimal impact on smaller, higher frequency, flood events (e.g. up to the 2-year event); thus, a minimum open width is necessary to avoid flow constriction during smaller flood events. The gate height was determined based on the staging elevation during the 500-year event. The gate opening during each design event was determined based on the total allowable flow into the protected area to reach the target elevation at the USGS gage in Fargo. During Phase 4, the gate openings on the Wild Rice River remained constant throughout the duration of the flood event, and flows passed into the protected area were mostly controlled by the control structure on the Red River. During subsequent design phases a detailed operations plan should be generated to control the gates at both the Wild Rice River and Red River control structures simultaneously.

Similar to the Red River control structure flow through the Wild Rice River control structure gates were calculated using the orifice equation. The discharge coefficient varied throughout the event based on the level of submergence at the structure.

Section 2.1.4 contains a summary of how flow through the control structure was calculated.

Water surface elevations downstream of the control structure were estimated using the HEC-RAS unsteady flow model. During previous design phases a gate width of 30-ft was selected based on recommendations from the USACE – Environmental. The 30-ft height of the control structure is based on the upstream WSEL during the 500-year event. A concept drawing for the control structure is presented in Figure F-D22.

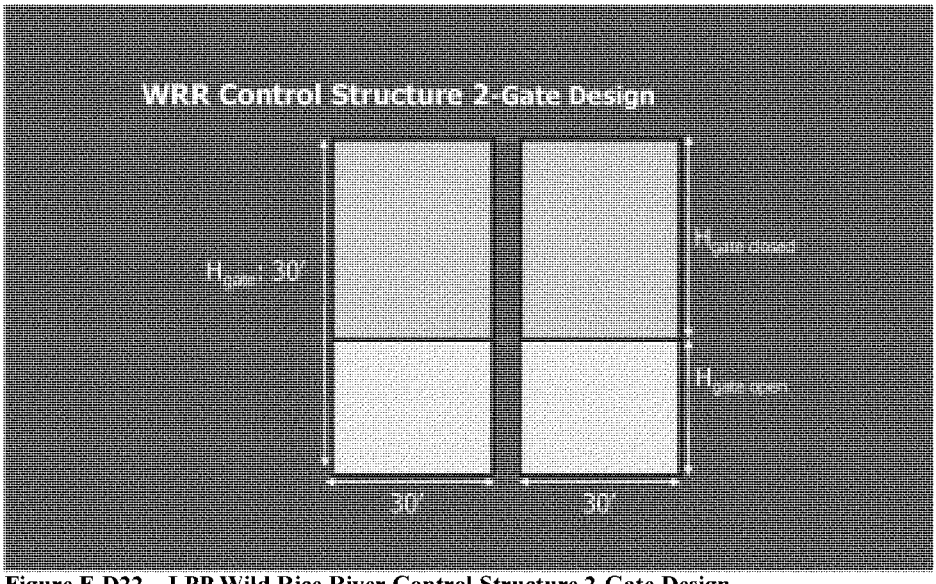


Figure F-D22 LPP Wild Rice River Control Structure 2-Gate Design

Maximum flows, water surface elevations, head difference, gate opening height and velocities for varying flood events at the LPP Wild Rice River Control Structure are presented in Tables F-D12 and F-D13. These variables were obtained from the HEC-RAS unsteady flow models, with the exception of velocity through the gates. HEC-RAS does not provide this variable so it was calculated by dividing the flow by the flow area. Figures F-D23 and F-D24 shows the comparison of the headwater and tailwater elevations for various flood events at the LPP Control Structure.

Table F-D12 Modeling Outputs for the Proposed Wild Rice River Control Structure Phase 4 Hydrology

	Phase 3							Phase 4						
Event (yr)	Existing Flow into Protected Area (cfs)	Proposed Flow through Gates into Protected Area (cfs)	Proposed Headwater Elevation in Wild Rice River* (ft)	Proposed Tailwater Elevation in Wild Rice River** (ft)	Head Difference (ft)	Gate Open Height (ft)	Velocity through Gates (ft/s)	Existing Flow into Protected Area (cfs)	Proposed Flow through Gates into Protected Area (cfs)	Proposed Headwater Elevation in Wild Rice River*** (ft)	Proposed Tailwater Elevation in Wild Rice River*** (ft)	Head Difference (ft)	Gate Open Height (ft)	Velocity through Gates (ft/s)
10	6185	2350	908.65	905.30	3.35	3.33	11.8	6,393	1,114	915.61	901.43	14.18	0.75	23.2
50	11655	2350	912.93	905.36	7.56	2.22	17.7	8,641	459	920.84	900.73	20.11	0.25	30.6
100	13780	2350	914.33	905.48	8.85	2.05	19.1	8,503	387	922.86	901.56	21.3	0.2	38.8
500	18342	2350	917.11	911.12	5.99	2.49	15.7	8,767	1,686	922.45	911.90	10.55	1.6	17.6

Coinc. indicates e.g., the 10-year coincidental event

* Headwater located at HEC-RAS Station 177958.79 of with-project model (between the east and west weirs within the diversion)

**Tailwater located at HEC-RAS Station 12.85 of Wild Rice River downstream of control structure

*** Headwater and Tailwater taken from IS 66102

Table F-D13 Modeling Outputs for the Proposed Wild Rice River Control Structure Phase 4 Local Peak Flows on the Tributaries

Phase 4							
Event (yr)	Existing Flow into Protected Area (cfs)	Proposed Flow through Gates into Protected Area (cfs)	Proposed Headwater Elevation in Wild Rice River* (ft)	Proposed Tailwater Elevation in Wild Rice River* (ft)	Head Difference (ft)	Gate Open Height (ft)**	Velocity through Gates (ft/s)
10	5,444	1,790	910.09	902.64	7.45	2.0	15.0
50	8,824	1,982	917.10	903.32	13.78	2.0	16.5
100	8,688	2,027	918.94	903.43	15.51	2.0	16.9
500	9,565	2,037	919.61	909.93	9.68	2.0	17.0

* Headwater and Tailwater taken from IS 66102

** Gate openings for local event were kept constant with coincidental event. During future design phases this should be reviewed and revised.

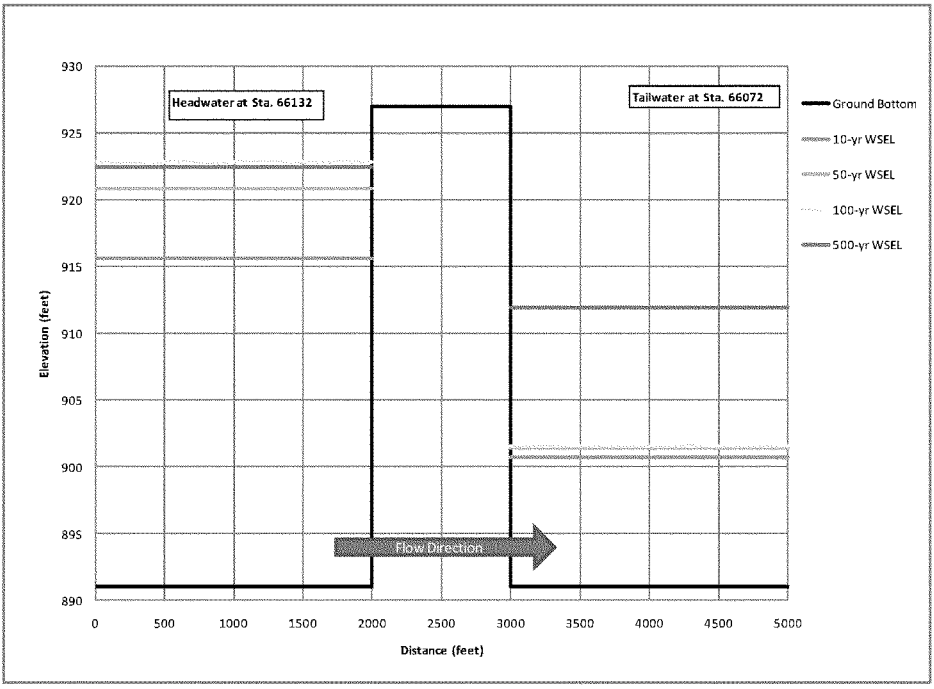


Figure F-D23 Comparison of the Headwater and Tailwater Elevations at the Wild Rice River Control Structure for Coincidental Hydrology

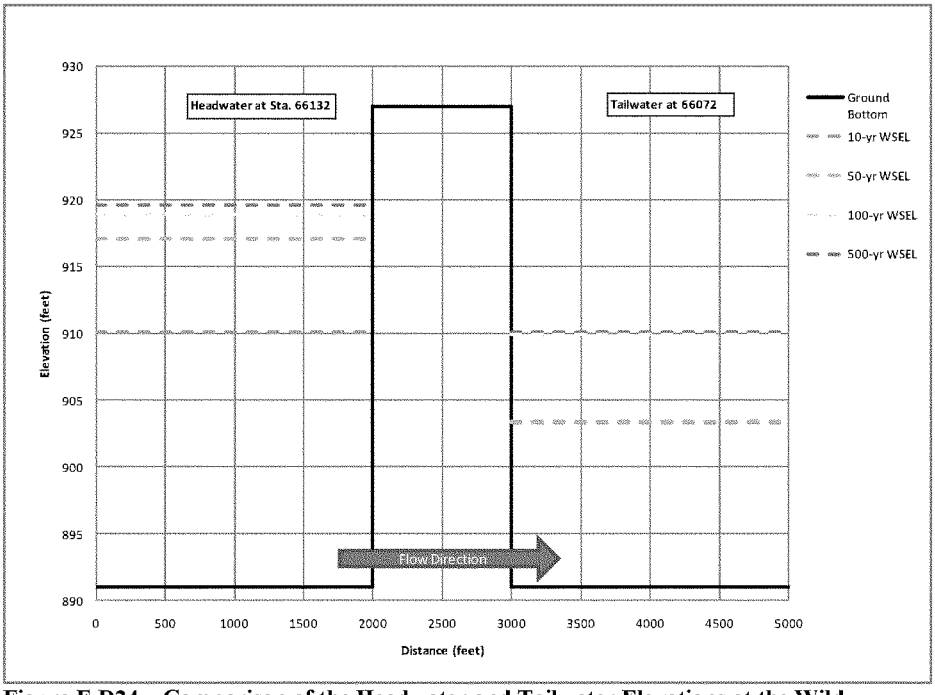


Figure F-D24 Comparison of the Headwater and Tailwater Elevations at the Wild Rice River Control Structure for Local Hydrology

F-D2.3.3 Transition from the Natural Channel to the Control Structure

The transition from the natural channel into the control structure is accomplished with 7:1 side slopes, as shown in Figure F-D25. For more details on the geotechnical slope stability analysis of this reach, see Exhibit N of Appendix F.

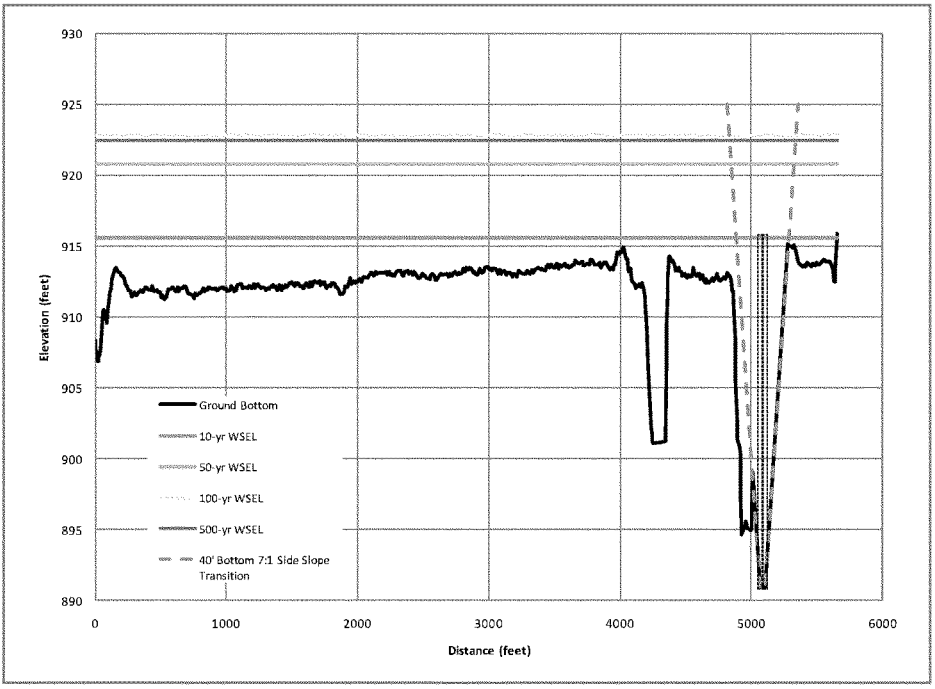


Figure F-D25 Transition from the Natural Wild Rice River Cross Section to the Wild Rice River Control Structure

F-D2.4 DIVERSION CHANNEL TRANSITION AND AQUEDUCT AT SHEYENNE RIVER

A description of the LPP Diversion Channel transition and aqueduct structure at the Sheyenne River can be found in the main section of Appendix F. Appendix F, Exhibit C discusses the variety of structure types that were investigated for this tributary crossing during Phase 2 design. This portion of Exhibit D presents a comparison of water surface elevations at the Sheyenne River structure, design calculations of the aqueduct crossing, and detailed HEC-RAS modeling outputs in the Diversion Channel beneath the aqueduct crossing.

F-D2.4.1 Comparison of LPP Diversion Channel and Sheyenne River Hydrology

Maximum flows, water surface elevations, and velocities for the LPP Diversion Channel downstream of the Sheyenne River crossing for the 10-, 50-, 100-, and 500-year events are presented in Table F-D14. The location shown in Table F-D14 is where flows diverted from the Sheyenne River enter the LPP Diversion Channel via the spillway weir and the Diversion Channel has returned to its typical cross section. Water surface elevations in the LPP Diversion Channel just downstream of the Sheyenne River

structure, in the Sheyenne River for local tributary flood events, and in the Sheyenne River for coincidental flood events, plotted against the cross section data, are presented in Figure F-D26. Tabular results of existing conditions local and coincidental flood events in the tributaries are presented in Appendix F, Exhibit A.

Table F-D14 LPP Diversion Channel Downstream of the Sheyenne River Crossing

	Phase 3*			Phase 4**		
Event (yr)	**Flow (cfs)**	**WSEL (ft)**	**Average Velocity (ft/s)**	**Flow (cfs)**	**WSEL (ft)**	**Velocity (ft/s)*** **
10	8,406	897.67	2.53	7,353	895.8	2.23
50	22,201	904.55	3.32	17,417	901.6	2.80
100	27,071	906.63	3.45	20,832	903.5	2.85
500	37,808	910.32	3.77	21,068	904.7	2.64

* Model results from Station 148766. Phase 3 HEC-RAS steady flow model.

** Model results from Station 146691. Phase 4 HEC-RAS unsteady flow model.

*** Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

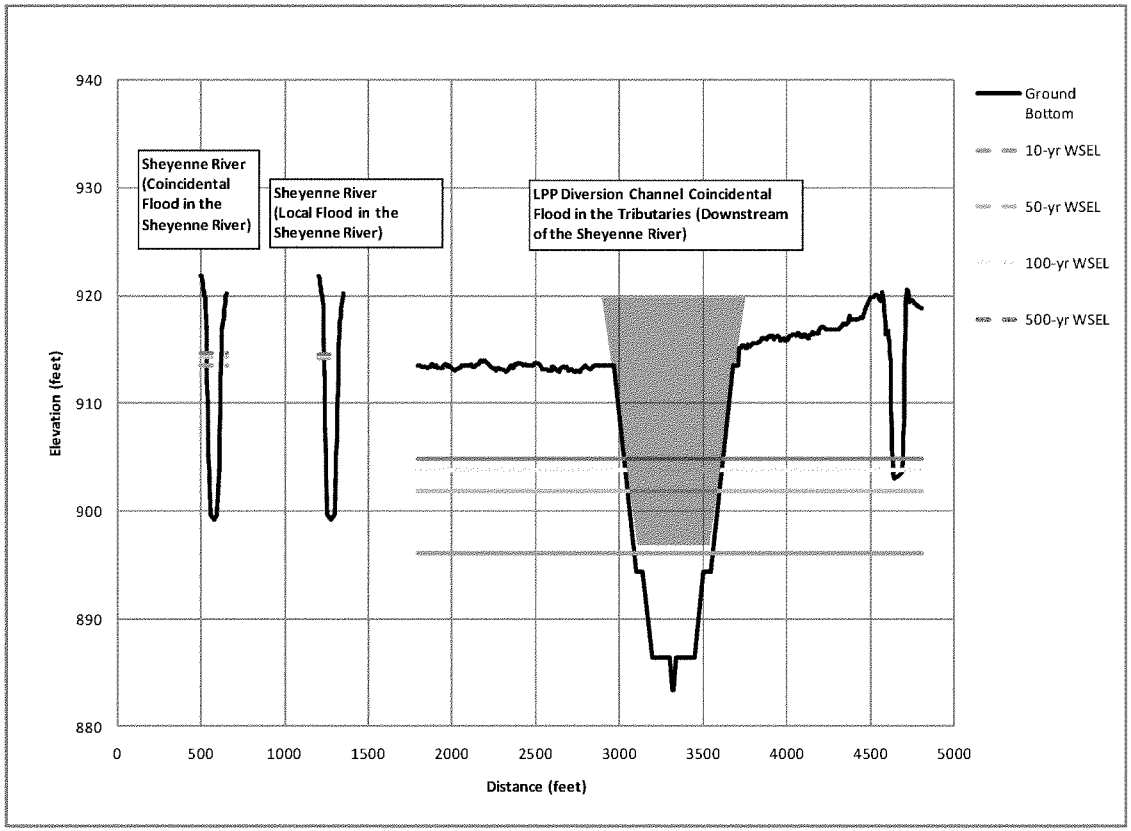
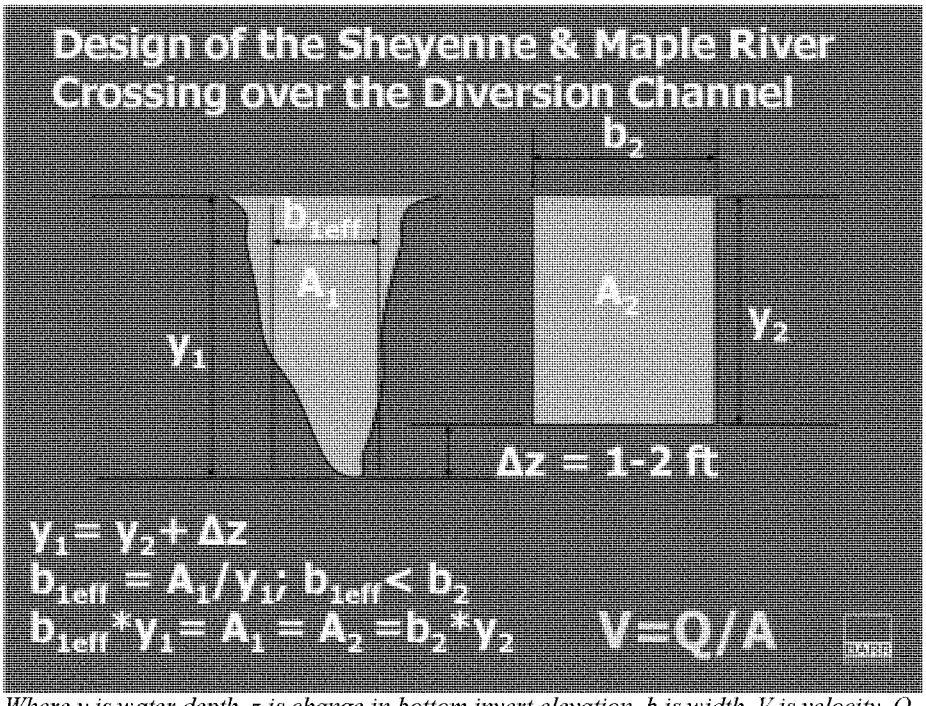


Figure F-D26 LPP Diversion Channel Downstream of the Sheyenne River Structure and in the Sheyenne River

F-D2.4.2 Design of Sheyenne River Open Aqueduct

The Sheyenne River open aqueduct was sized to pass the 2-year local flow in the Sheyenne River, or 1,200 cfs, into the protected area. The aqueduct was sized to maintain the same cross sectional area and flow velocity as the corresponding natural cross section for events smaller than the 2-year event in the tributary, as shown in Figure F-D27. The invert of the aqueduct was lowered approximately one-half foot below the existing invert, from an elevation of 899.2 to 898.7 feet. In comparison to the natural variation in the Sheyenne River's channel bottom, this change should be relatively insignificant since gradual transitions will be used on both the entrance and exit to the aqueduct. Figure F-D28 shows the Sheyenne River thalweg (deepest continuous line along the channel bottom) based on the Phase 4 HEC-RAS unsteady flow model under existing conditions, and with the Sheyenne River aqueduct lowered one-half foot. Table F-D15 present the calculations used in Phase 3 to size the Sheyenne River open aqueduct crossing over the LPP Diversion Channel. The width of the flow area for the Sheyenne River open aqueduct was found to be 46.4 feet. This was rounded to 50 feet for structural design. This design from Phase 3 was used in the Phase 4 analysis. The top of structure was set at 916 feet, one foot above the 500-year local peak event through the aqueduct.



Where y is water depth, z is change in bottom invert elevation, b is width, V is velocity, Q is flow and A is area.

Figure F-D27 Schematic Showing Calculations Used to Maintain Cross Section-Averaged Flow Velocity in the Sheyenne and Maple River Aqueduct Crossings.

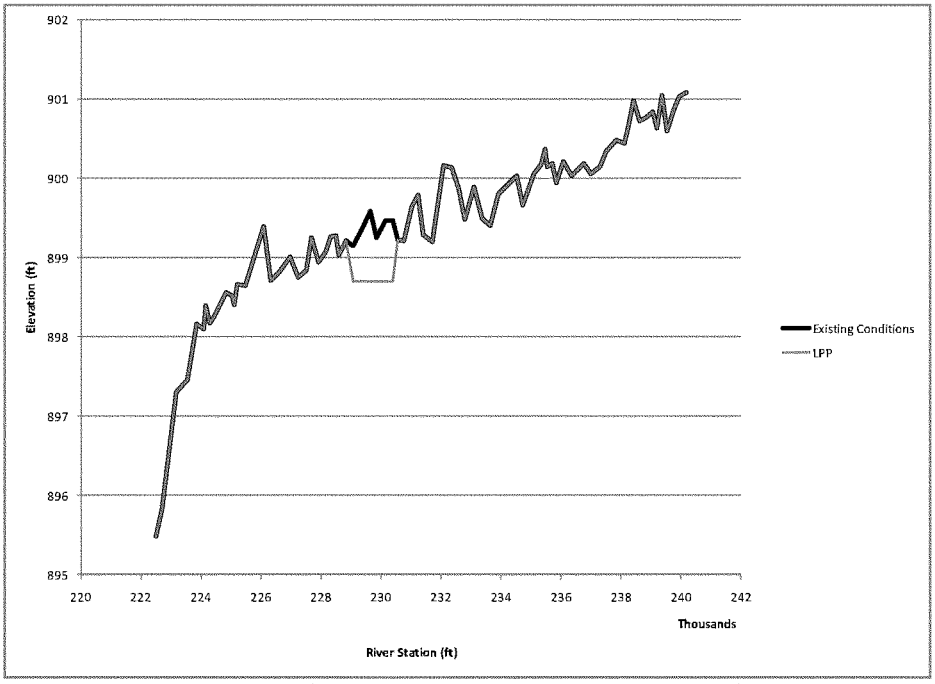


Figure F-D28 Comparison of the Sheyenne River Thalweg under Existing and Proposed Conditions

Table F-D15 Calculations Used to Size the Sheyenne River Open Aqueduct Crossing over the LPP Diversion Channel (from Phase 3)

2-Year Local Event Variables	Existing Natural Cross Section	Open Aqueduct Crossing
Water Surface Elevation (ft)	912.71	912.71
Channel Bottom Invert (ft)	897.7	898.7
Depth of Water (ft)	15.01	14.01
Flow (cfs)	1200	1200
Flow area (ft^2)	650.35	650.35
Average Velocity (ft/s)	1.85	1.85
Acceleration of Gravity (ft/s^2)	32.2	32.2
Average Channel Width (ft)	43.33	46.42
Change in Bottom Invert from Natural Conditions (ft)	0	1
Energy (ft)	15.063	15.063

Depending on the actual head in the Sheyenne River and on how much flow the Sheyenne River spillway weir diverts into the LPP Diversion Channel, somewhat more than the 2-year local event (as much as the 5-year local event) could enter the protected area. For events larger than the 2-year local event when water will be diverted over the spillway weir, the water surface elevation in the Sheyenne River open aqueduct will be less than the water surface elevation under existing conditions (Appendix F, Exhibit A - Background Hydrology). The design of the spillway weir is discussed in this exhibit in Section F-D2.5. The velocities in the Sheyenne River open aqueduct, shown in Tables F-D16 and F-D17, will never be greater than 3 feet per second and will always remain in the range of measured existing velocities as seen in Figure F-A31 Exhibit A.

Table F-D16 Elevations, Velocities and Flows in the Sheyenne River Aqueduct for Coincidental Events

	Phase 3					Phase 4				
Event Return Period (yr)	Flow in River Upstream of Aqueduct (cfs)	WSEL Upstream of Aqueduct (ft)	Flow in River into Protected Area (cfs)	WSEL of River in Aqueduct (ft)	Average Velocity in Aqueduct (ft/s)	Flow in River Upstream of Aqueduct (cfs)*	WSEL Upstream of Aqueduct (ft)	Flow in River into Protected Area (cfs)*	WSEL of River in Aqueduct (ft)	Average Velocity in Aqueduct (ft/s)*
Mean Annual Flow	236	903.24	236	903.27	1.11	-	-	-	-	-
2 – Local	1200	912.71	1200	912.71	1.85	-	-	-	-	-
2	1325	913.27	1200	913.09	1.85	-	-	-	-	-
5	1935	915.15	1350	914.34	1.92	-	-	-	-	-
10	2,565	916.31	1,443	915.12	1.95	1,783	913.5	971	913.5	1.38
50	4,510	919.37	1,696	917.16	2.04	3,629	914.4	1,151	914.5	1.52
100	4,600	919.50	1,707	917.25	2.05	4,176	914.8	1,160	914.8	1.51
500	4,600	919.50	1,707	917.25	2.05	4,368	914.9	1,073	914.9	1.38

* Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

Table F-D17 Elevations, Velocities and Flows in the Sheyenne River Aqueduct for Local Events

Event Return Period (yr)	Flow in River Upstream of Aqueduct (cfs)*	WSEL Upstream of Aqueduct(ft)	Flow in River into Protected Area (cfs)*	WSEL of River in Aqueduct (ft)	Average Velocity in Aqueduct (ft/s)*
Phase 4					
10	3,996	914.4	1,816	914.3	2.44
50	4,479	914.6	1,905	914.5	2.52
100	4,526	914.7	1,918	914.5	2.54
500	4,671	914.8	1,960	914.6	2.57

* Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

Table F-D16 shows a significant difference in the water surface elevations between Phase 3 and Phase 4. For events greater than or equal to the 50-year event, the Phase 4 WSELs are between 4 and 5 feet lower than the Phase 3 WSELs. This is the reason why the spillway width has changed from the previous design of 55 ft wide to the current design of 300 ft wide. The head on the weir is much less in Phase 4, necessitating a significantly wider weir to maintain flows through the aqueduct less than 2,000 cfs, such that the current level of flood protection provided by the Horace to West Fargo diversion is maintained.

The transition from the natural channel into the open aqueduct is accomplished with 5:1 side slopes as shown in Figure F-D29. For more details about the slope stability analysis of this reach, see Exhibit N of Appendix F.

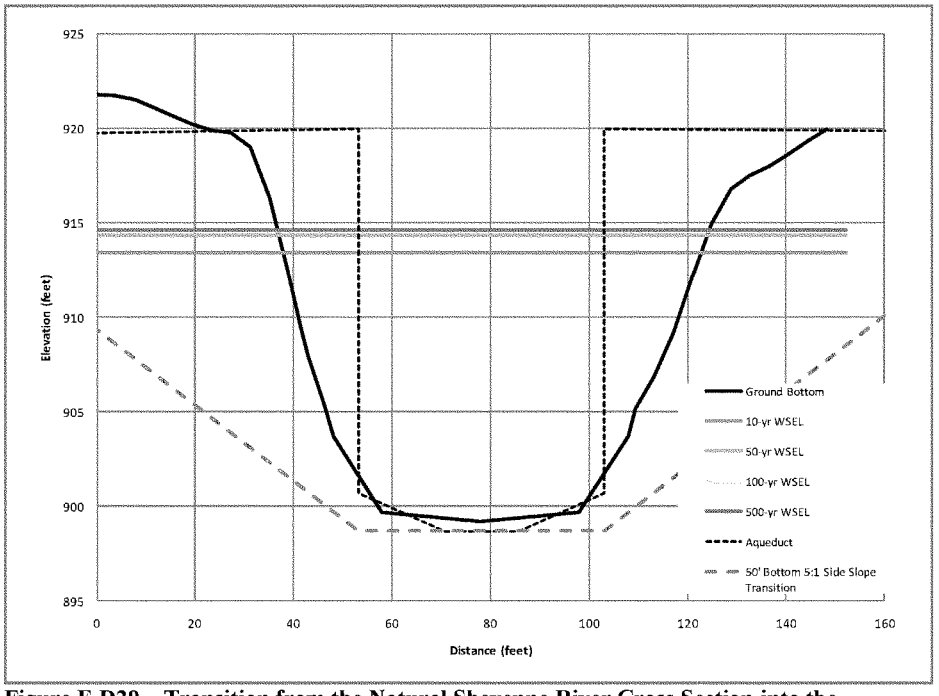


Figure F-D29 Transition from the Natural Sheyenne River Cross Section into the Aqueduct

F-D2.4.3 Design of the Low Flow Channel in the Sheyenne River Aqueduct

The Sheyenne open aqueduct will include a smaller winter channel to ensure that low flows during winter will not freeze. According to records from 1949 to 2008 at the USGS gage at Kindred, the mean annual flow from December to February is 99.5 cfs. Using Manning's equation, a low flow channel with dimensions of 10 feet by 10 feet

would convey this flow across the open aqueduct. The lowest recorded flow in the winter months is 17.5 cfs. In the next phase of design, the design of the low flow channel could be further refined to look at a variety of low flow and freezing conditions that the structure may encounter and to optimize dimensions to encourage fish passage during very low flows.

F-D2.4.4 Design of LPP Diversion Channel Transition at the Sheyenne River Structure
No mixing of Sheyenne River and Diversion Channel water will occur at the open aqueduct. Flow in the Sheyenne River open aqueduct will be able to overtop the top of the wall at an elevation of 916 feet into the LPP Diversion Channel but would necessitate an event larger than the 500-year event. In Phase 4, flow through the aqueduct was completely contained within the aqueduct.

The opening under the aqueduct was maximized to minimize the head difference in the diversion channel across the structure. The bottom width of the diversion channel is 250 feet so the width of the opening was set at 250 feet. The number of piers and the size of each pier were set based on structural stability rather than flow conditions. The design was checked in the HEC-RAS unsteady flow model to ensure that the head drop across the structure was acceptable. Pressurized flow underneath the aqueduct in the Diversion Channel will occur for events in the Red River greater than or equal to the 50-year event. The proposed dimensions and location of the Sheyenne River structure are shown in Appendix F.

The total flow in the LPP Diversion Channel beneath the aqueduct, headwater elevation, tailwater elevation, head difference, and average velocities for the 10-, 50-, 100-, and 500-year events are presented in Tables F-D18 and F-D19. These variables were obtained from the HEC-RAS models with the exception of average velocity beneath the aqueduct. HEC-RAS does not provide this variable so it was calculated by dividing the flow by the flow area.

Table F-D18 Coincidental Event HEC-RAS Outputs for the LPP Diversion Channel at the Sheyenne River Aqueduct

	Phase 3					Phase 4				
Event (yr)	Flow in Diversion Channel (cfs)	Headwater Elevation in Diversion Channel (ft)	Tailwater Elevation in Diversion Channel (ft)	Head Difference (ft)	Average Velocity (ft/s)	Flow in Diversion Channel (cfs)*	Headwater Elevation in Diversion Channel (ft)	Tailwater Elevation in Diversion Channel (ft)	Head Difference (ft)	Average Velocity (ft/s)*
10	7,284	897.87	897.82	0.05	1.94	7,161	896.3	896.3	0.0	2.5
50	19,387	905.19	904.69	0.50	5.17	15,437	902.2	902.0	0.2	5.2
100	24,178	907.58	906.77	0.81	6.45	18,560	904.2	903.9	0.3	6.2
500	34,915	912.26	910.44	1.82	9.32	18,099	905.2	904.9	0.3	6.1

* Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

Table F-D19 Local Event HEC-RAS Outputs for the LPP Diversion Channel at the Sheyenne River Aqueduct

	Phase 4				
Event (yr)	Flow in Diversion Channel (cfs)*	Headwater Elevation in Diversion Channel (ft)	Tailwater Elevation in Diversion Channel (ft)	Head Difference (ft)	Average Velocity (ft/s)*
10	1,930	890.7	890.7	0.0	1.2
50	9,079	898.0	897.9	0.1	3.0
100	11,726	899.8	899.8	0.0	3.9
500	12,674	901.4	901.3	0.1	4.3

* Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

F-D2.5 SPILLWAY AT SHEYENNE RIVER

A weir spillway has been selected to divert water from the tributary into the LPP Diversion Channel. The design elevation of the Sheyenne River spillway in Phase 4 is fixed at 912.56 ft, which is the water surface elevation on the Sheyenne River associated with the 2-year local flood event (1,200 cfs). The water surface elevations in the LPP Diversion Channel are at least 2 feet below the weir crest elevation, so submerged flow conditions do not occur over the weir. Under these conditions, a weir coefficient of 3.1 is appropriate and was used. The width of the spillway is the only variable that was altered. Starting at the design width of 55 feet from Phase 3, the width was increased until the desired flow split was met. The local peak flow of the 500-year event on the Sheyenne River was the limiting event used to size the weir since it had higher flows and lower WSELs than the coincidental values (to peaks in the Red River of the North). The maximum flow allowed to pass through to the protected side is 2,000 cfs due to limits in the existing Horace to West Fargo diversion channel. Based on this constraint, the weir is designed to be 300 feet wide.

Table F-D20 presents the results from Phase 3 using the HEC-RAS steady flow model. Tables F-D21 and F-D22 present the results from Phase 4 using the HEC-RAS unsteady flow models.

Table F-D20 Phase 3 Design Results for the 55-Ft.-Wide Sheyenne River Spillway Using the Coincidental Flow (spillway elevation 912.71 ft)

	Upstream WSEL (ft)	Upstream River Flow (cfs)	Diverted Flow (cfs)	Flow into Protected Area (cfs)
10-year	916.3	2,565	1,122	1,443
50-year	919.4	4,510	2,814	1,696
100-year	919.5	4,600	2,893	1,707
500-year	919.5	4,600	2,893	1,707

Table F-D21 Phase 4 Design Results for the 300-Ft.-Wide Sheyenne River Spillway Using the Coincidental Flow (spillway elevation 912.56 ft)

	Upstream WSEL (ft)	Upstream River Flow (cfs)	Diverted Flow (cfs)	Aqueduct WSEL (ft)	Flow into Protected Area (cfs)
10-year	913.5	1,783	812	913.5	971
50-year	914.5	3,629	2,478	914.5	1,151
100-year	914.8	4,176	3,016	914.8	1,160
500-year	914.9	4,368	3,295	914.9	1,073

Table F-D22 Phase 4 Design Results for the 300-Ft.-Wide Sheyenne River Spillway Using the Peak Flow (spillway elevation 912.56 ft)

	Upstream WSEL (ft)	Upstream River Flow (cfs)	Diverted Flow (cfs)	Aqueduct WSEL (ft)	Flow into Protected Area (cfs)
10-year	914.4	3,996	2,180	914.3	1,816
50-year	914.6	4,479	2,574	914.5	1,905
100-year	914.7	4,526	2,609	914.5	1,918
500-year	914.8	4,671	2,712	914.6	1,960

Water surface elevations were also checked along the entire reach of the Sheyenne River to ensure that the aqueduct structure was not causing significant upstream flooding. Figures F-D30 through F-D37 compare existing conditions and LPP conditions along the entire reach of the Sheyenne River. These figures confirm that the aqueduct structure allowing the Sheyenne River to cross the LPP diversion channel is not adversely affecting the upstream portion of the reach. In fact, it is lowering water levels through and downstream of the aqueduct during the large events shown. As discussed, this structure will not affect flows less than or equal to the 2-year event due to the elevation of the spillway.

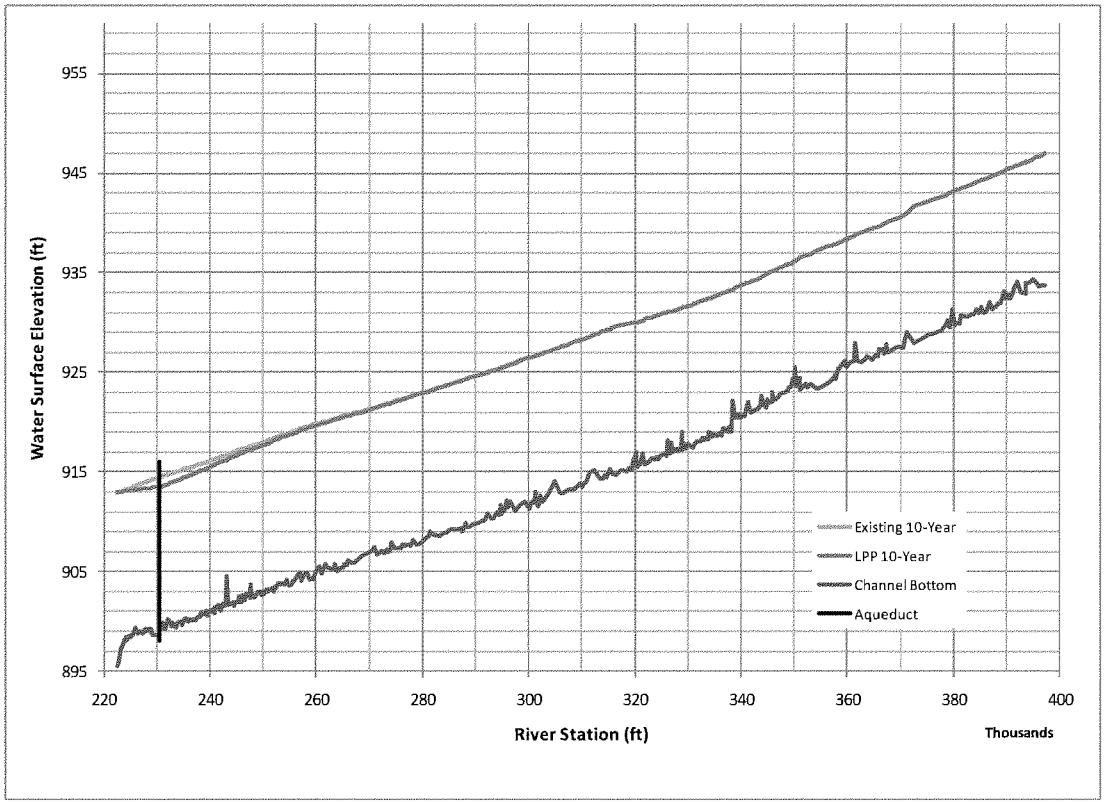


Figure F-D30 Comparing Existing Conditions to LPP Conditions Along the Sheyenne River for the Coincidental 10-Year Event

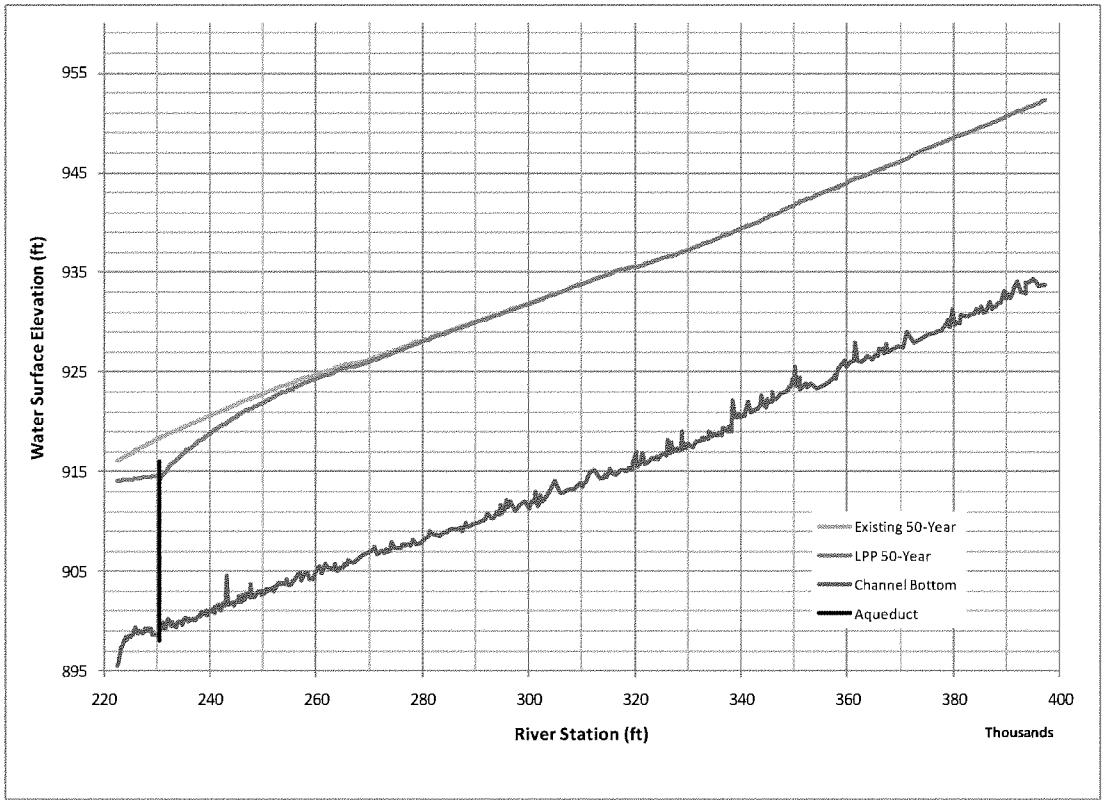


Figure F-D31 Comparing Existing Conditions to LPP Conditions Along the Sheyenne River for the Coincidental 50-Year Event

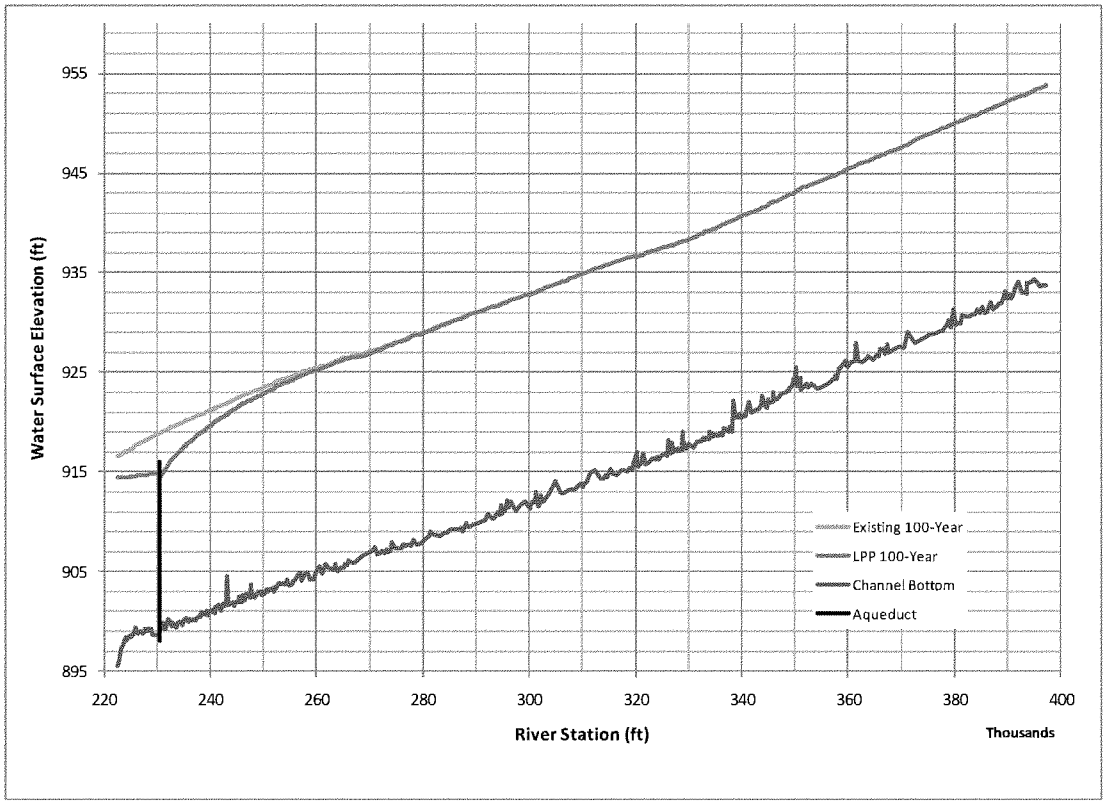


Figure F-D32 Comparing Existing Conditions to LPP Conditions Along the Sheyenne River for the Coincidental 100-Year Event

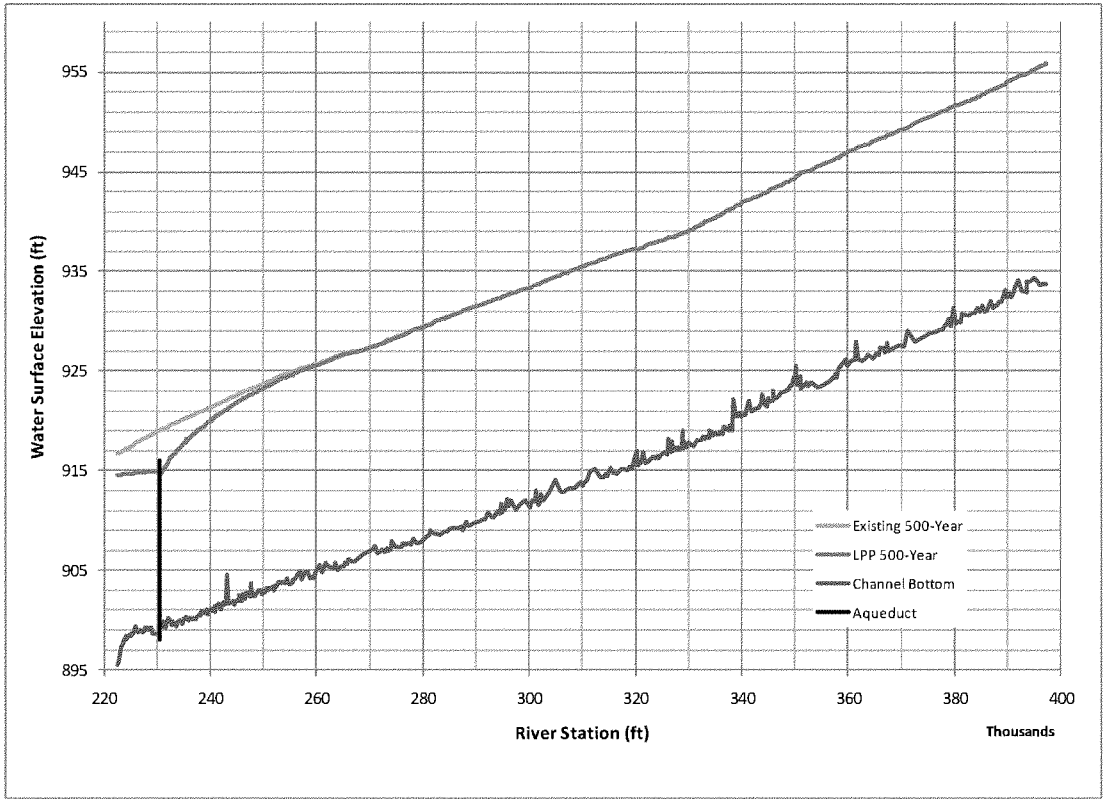


Figure F-D33 Comparing Existing Conditions to LPP Conditions Along the Sheyenne River for the Coincidental 500-Year Event

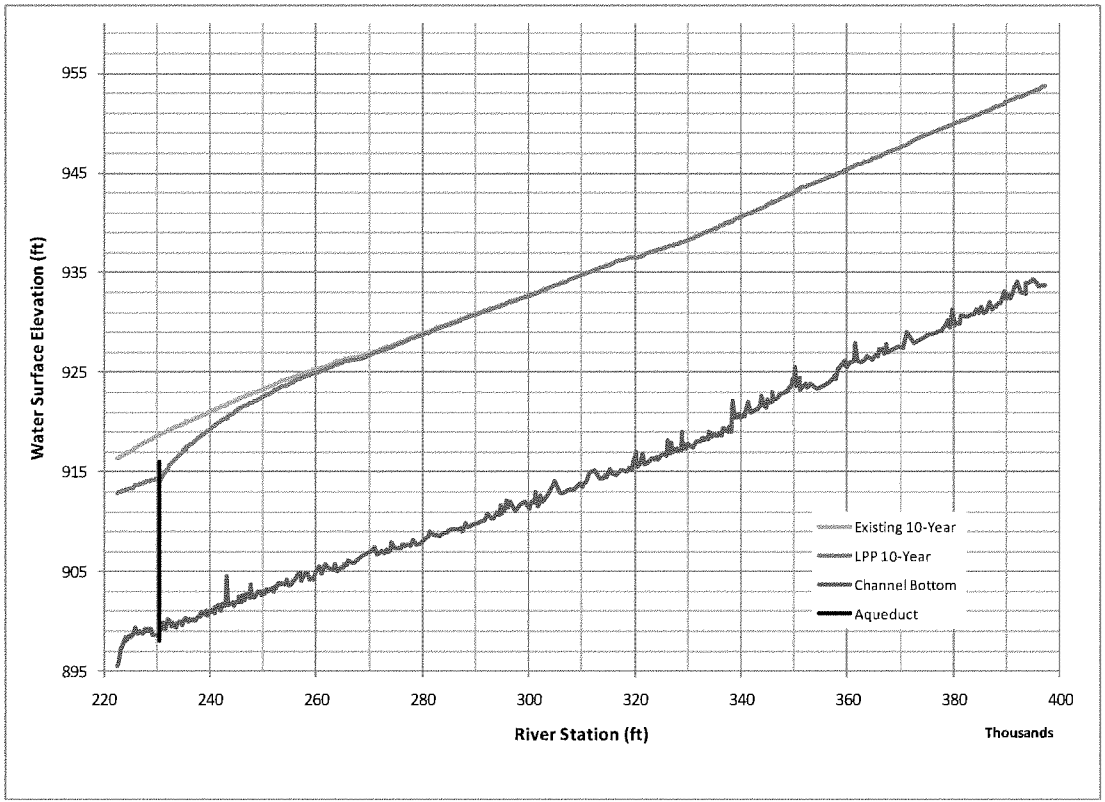


Figure F-D34 Comparing Existing Conditions to LPP Conditions Along the Sheyenne River for the Local 10-Year Event

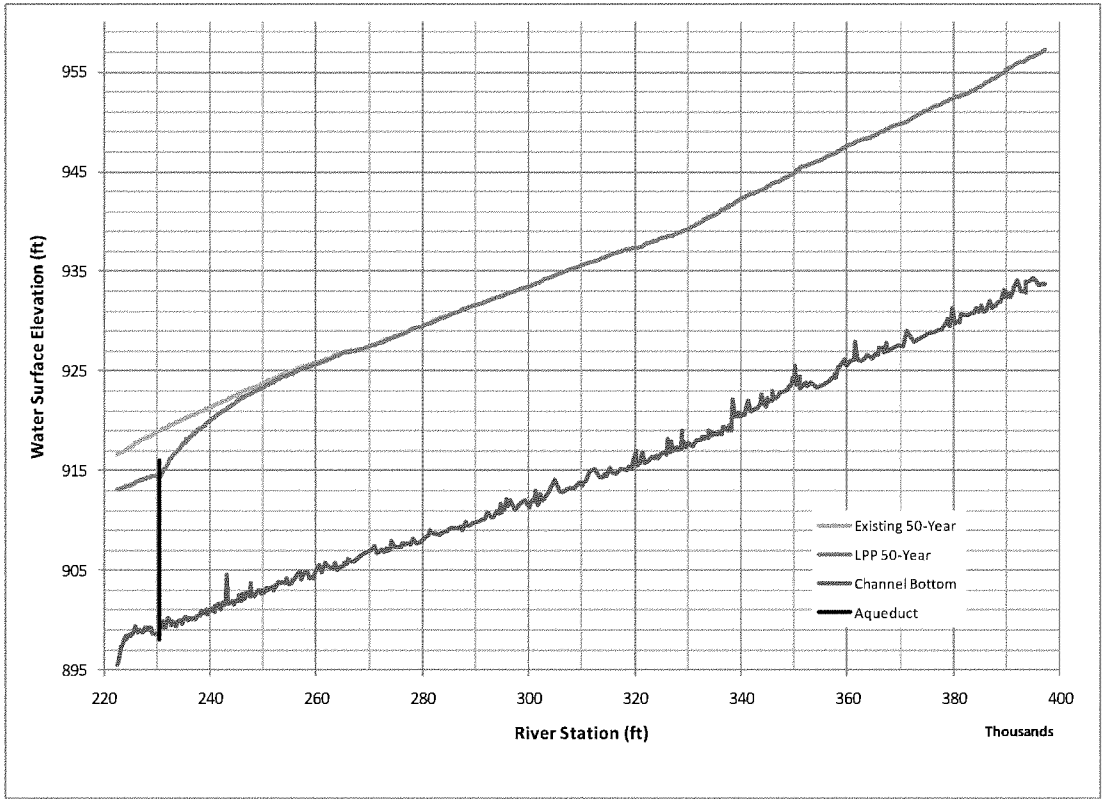


Figure F-D35 Comparing Existing Conditions to LPP Conditions Along the Sheyenne River for the Local 50-Year Event

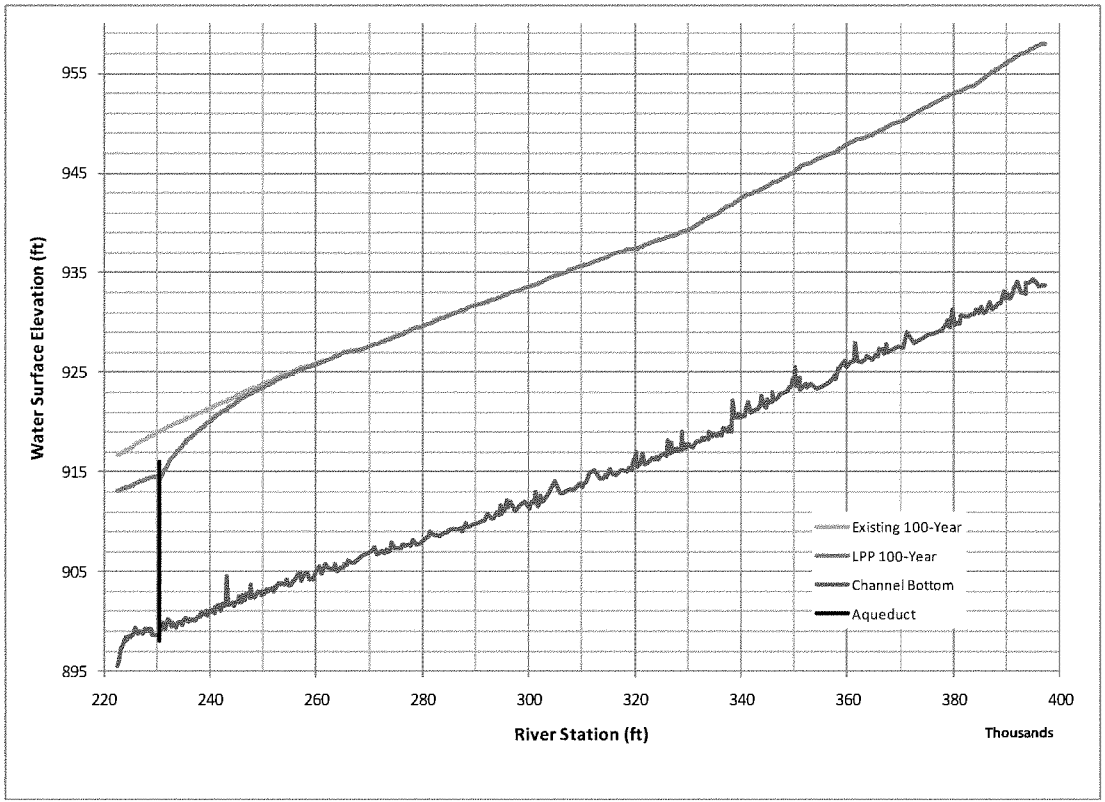


Figure F-D36 Comparing Existing Conditions to LPP Conditions Along the Sheyenne River for the Local 100-Year Event

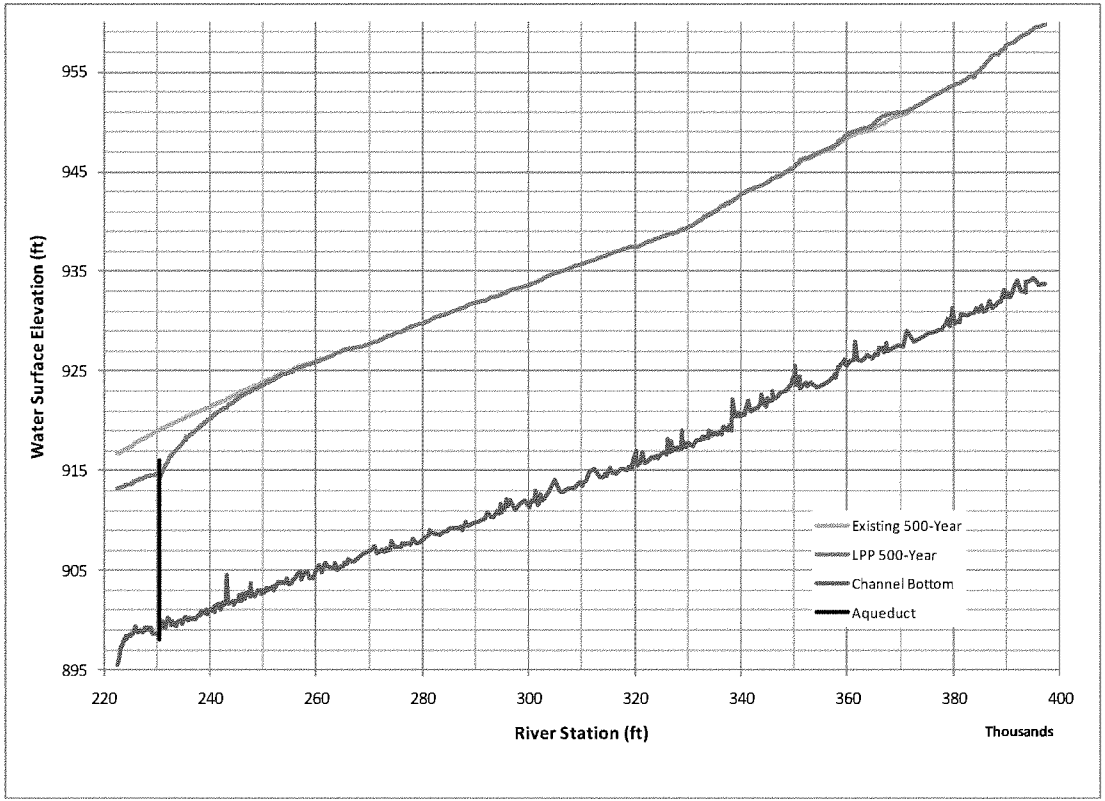


Figure F-D37 Comparing Existing Conditions to LPP Conditions Along the Sheyenne River for the Local 500-Year Event

F-D2.6 DIVERSION CHANNEL TRANSITION AND AQUEDUCT AT MAPLE RIVER

A description of the LPP Diversion Channel transition and aqueduct structure at the Maple River can be found in the main section of Appendix F. Appendix F, Exhibit C discusses the variety of structure types that were investigated for this tributary crossing during the Phase 2 design. This portion of Exhibit D presents a comparison of water surface elevations at the Maple River structure, design calculations of the aqueduct crossing, and detailed HEC-RAS modeling outputs in the Diversion Channel beneath the aqueduct crossing.

F-D2.6.1 Comparison of LPP Diversion Channel and Maple River Hydrology

Maximum flows, water surface elevations, and velocities for the LPP Diversion Channel downstream of the Maple River crossing for the 10-, 50-, 100-, and 500-year events are presented in Table F-D23. The location shown in Table F-D23 is where flows diverted from the Maple River enter the LPP Diversion Channel via the spillway weir and the Diversion Channel has returned to its typical cross section. Water surface elevations in the LPP Diversion Channel just downstream of the Maple River structure, in the Maple River for local tributary flood events, and in the Maple River for coincidental flood events, plotted against the cross section data, are presented in Figure F-D38. Tabular results of existing conditions local and coincidental flood events in the tributaries are presented in Appendix F, Exhibit A.

Table F-D23 LPP Diversion Channel Downstream of the Maple River Crossing

	Phase 3*			Phase 4**		
Event (yr)	Flow (cfs)	WSEL (ft)	Average Velocity (ft/s)	Flow (cfs)***	WSEL (ft)	Average Velocity (ft/s)***
10	9,260	884.82	1.93	9,001	886.1	2.26
50	24,870	891.39	2.88	23,775	892.8	3.21
100	31,823	893.60	3.16	28,991	894.3	3.51
500	42,992	896.66	3.54	30,804	894.7	3.59

* Phase 3 values from Station 71959 in the Phase 3 HEC-RAS steady flow model

** Phase 4 values from Station 70018 in the Phase 4 HEC-RAS unsteady flow model

*** Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

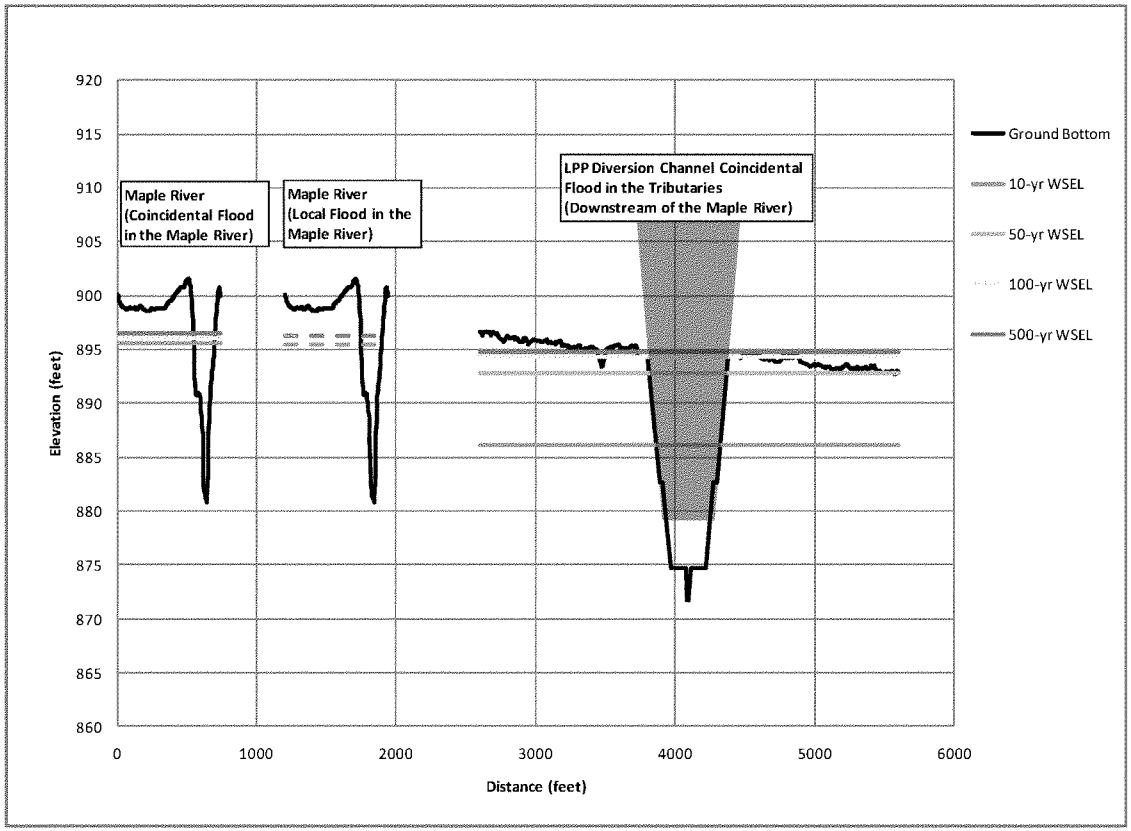


Figure F-D38 LPP Diversion Channel Downstream of the Maple River Structnre and in the Maple River

F-D2.6.2 Design of Maple River Open Aqueduct

The Maple River open aqueduct was sized to pass the 2-year equivalent flow into the protected area. For Phase 3 hydrology, this is equal to the 2-year flow for the Maple River (970 cfs) plus the two year flow for the Lower Rush and Rush Rivers (302 cfs and 415 cfs, respectively), or a total of 1,687 cfs. The Lower Rush River and Rush River 2-year flows were added because these rivers will be completely diverted into the LPP Diversion Channel; this additional flow will aim to maintain bankfull flows in the downstream reach of the Sheyenne River. The aqueduct was sized to maintain the same cross sectional area and flow velocity as the corresponding natural cross section for events smaller than the equivalent 2-year event in the tributary, as shown in Figure F-D39. The invert of the aqueduct was raised approximately 1.5 feet above the existing invert, from an elevation of 879.6 to 881.06 feet, to allow for more flow area and lower velocities in the LPP Diversion Channel under the crossing. In comparison to the natural variation in the Maple River's channel bottom, this change is not significant since gradual transitions will be used on both the entrance and exit to the aqueduct. Figure F-D40 shows the Maple River thalweg (deepest continuous line along the channel bottom) under existing conditions and with the Maple River aqueduct raised 1.5 feet. Table F-D24 presents the calculations used in Phase 3 to size the Maple River open aqueduct crossing over the LPP Diversion Channel. The width of the flow area for the Maple River open aqueduct was found to be 50.3 feet. This was rounded to 50 feet for structural design. The top of structure was set at 902 feet, one foot above the 500-year peak on the Red River of the North event through the LPP diversion channel.

Table F-D24 Calculations Used to Size the Maple River Open Aqueduct Crossing over the LPP Diversion Channel (from Phase 3)

2-Year Local Event Variables	Existing Natural Cross Section	Open Aqueduct Crossing
Water Surface Elevation (ft)	893.32	893.69
Channel Bottom Invert (ft)	880.06	881.06
Depth of Water (ft)	13.26	12.63
Flow (cfs)	1687	1687
Flow Area (ft^2)	635.21	635.21
Average Velocity (ft/s)	2.66	2.66
Acceleration of Gravity (ft/s^2)	32.2	32.2
Average Channel Width (ft)	47.90	50.3
Change in Bottom Invert from Natural Conditions (ft)	0	1
Energy (ft)	13.370	13.738

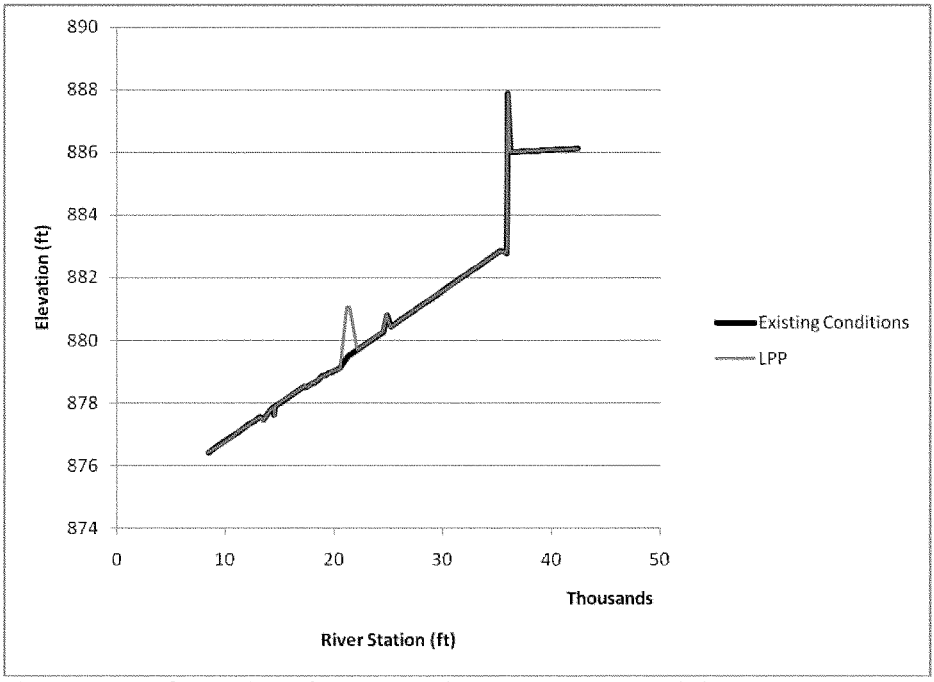


Figure F-D39 Comparison of the Maple River Thalweg under Existing and Proposed Conditions

Depending on the actual head in the Maple River for a certain event and also on how much flow the spillway weir diverts into the LPP Diversion Channel, more than the 2-year local event but less than the 10-year local event could enter the protected area. For events larger than the 2-year equivalent local event when water will be diverted over the spillway weir, the water surface elevation in the Maple River open aqueduct will be less than under existing conditions (Appendix F, Exhibit A).

Table F-D25 Elevations, Velocities aud Flows in the Maple River Aqueduct for Coincidental Events

	Phase 3					Phase 4				
Event Return Period (yr)*	Flow in River Upstream of Aqueduct (cfs)	WSEL Upstream of Aqueduct(ft)	Flow in River into Protected Area (cfs)	WSEL of River in Aqueduct (ft)	Average Velocity in Aqueduct (ft/s)	Flow in River Upstream of Aqueduct (cfs)*	WSEL Upstream of Aqueduct(ft)	Flow in River into Protected Area (cfs)*	WSEL of River in Aqueduct (ft)	Average Velocity in Aqueduct (ft/s)*
10	2,650	895.36	1,796	894.32	2.71	5,478	896.0	2,732	894.9	4.18
50	4,400	898.83	1,992	895.32	2.79	6,994	896.6	3,007	895.6	4.35
100	4,925	899.96	2,043	895.57	2.82	7,079	896.6	2,991	895.7	4.29
500	5,115	900.34	2,416	897.41	2.96	9,119	897.3	3,581	896.4	4.90

* Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

Table F-D26 Elevations, Velocities and Flows in the Maple River Aqueduct for Local Events

	Phase 4				
Event Return Period (yr)*	Flow in River Upstream of Aqueduct (cfs)*	WSEL Upstream of Aqueduct (ft)	Flow in River into Protected Area (cfs)*	WSEL of River in Aqueduct (ft)	Average Velocity in Aqueduct (ft/s)*
10	5,206	895.9	2,561	894.7	3.95
50	7,407	896.7	3,084	895.6	4.45
100	7,595	896.8	3,122	895.7	4.48
500	7,736	896.9	2,840	896.2	3.95

* Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

The transition from the natural channel into the open aqueduct is accomplished with 5:1 side slopes as shown in Figure F-D40. For more details about the geotechnical slope stability analysis of this reach, see Exhibit N of Appendix F.

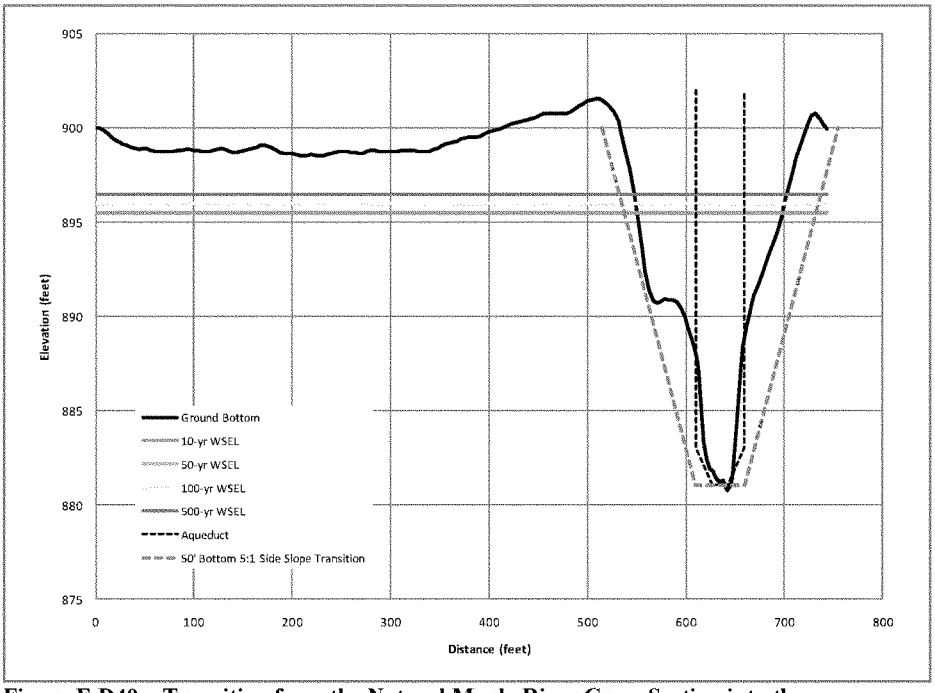


Figure F-D40 Transition from the Natural Maple River Cross Section into the Aqueduct

F-D2.6.3 Design of the Low Flow Channel in the Maple River Aqueduct

The Maple open aqueduct will include a smaller winter channel to ensure that low flows during winter will not freeze. According to records from 1949 to 2008 at the USGS gage at Mapleton, the mean annual flow from December to February is 10.9 cfs. Using Manning's equation, a low flow channel with dimensions of 4 feet by 4 feet would convey this flow across the open aqueduct. The lowest recorded flow in the winter months is 0 cfs. In the next phase of design, the design of the low flow channel could be further refined to look at a variety of low flow and freezing conditions that the structure may encounter and to optimize dimensions to encourage fish passage under a variety of flows.

F-D2.6.4 Design of LPP Diversion Channel Transition at the Maple River Structure

No mixing of Maple River and Diversion Channel water will occur at the open aqueduct. Flow in the Maple River open aqueduct will be able to overtop the top of the wall at an

elevation of 902 feet into the LPP Diversion Channel but would necessitate an event larger than the 500-year event. In Phase 4, flow through the aqueduct was completely contained within the aqueduct.

The opening under the aqueduct was maximized to minimize the head difference in the diversion channel across the structure. The bottom width of the diversion channel is 250 feet so the width of the opening was set at 250 feet. The number of piers and the size of each pier were set based on structural stability rather than flow conditions. The design was checked in the HEC-RAS unsteady flow model to ensure that the head drop across the structure was acceptable. Pressurized flow underneath the aqueduct in the Diversion Channel will occur for events in the Red River greater than or equal to the 10-year event. The proposed dimensions and location of the Maple River structure are shown in Appendix F.

The total flow in the LPP Diversion Channel beneath the aqueduct, headwater elevation, tailwater elevation, head difference, and average velocities for the 10-, 50-, 100-, and 500-year events are presented in Tables F-D27 and F-D28. These variables were obtained from the HEC-RAS models with the exception of average velocity beneath the aqueduct. HEC-RAS does not provide this variable so it was calculated by dividing the flow by the flow area.

Table F-D27 Coincidental Event HEC-RAS Outputs for the LPP Diversion Channel at the Maple River Aqueduct

	Phase 3					Phase 4				
Event (yr)	Flow in Diversion Channel (cfs)	Headwater Elevation in Diversion Channel (ft)	Tailwater Elevation in Diversion Channel (ft)	Head Difference (ft)	Average Velocity (ft/s)	Flow in Diversion Channel (cfs)*	Headwater Elevation in Diversion Channel (ft)	Tailwater Elevation in Diversion Channel (ft)	Head Difference (ft)	Average Velocity (ft/s)*
10	8,406	885.01	884.88	0.13	2.65	6,895	886.5	886.3	0.2	4.3
50	22,462	892.53	891.48	1.05	7.07	20,078	895.1	893.0	2.1	12.5
100	28,941	895.46	893.69	1.77	8.99	25,122	897.9	894.5	3.4	15.7
500	40,293	899.22	896.73	2.49	9.54	25,293	900.8	895.0	5.8	15.8

* Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

Table F-D28 Local Event HEC-RAS Outputs for the LPP Diversion Channel at the Maple River Aqueduct

	Phase 4				
Event (yr)	Flow in Diversion Channel (cfs)*	Headwater Elevation in Diversion Channel (ft)	Tailwater Elevation in Diversion Channel (ft)	Head Difference (ft)	Average Velocity (ft/s)*
10	3,623	883.2	883.2	0.0	2.3
50	14,424	890.9	890.2	0.7	9.0
100	19,738	893.5	892.2	1.3	12.3
500	25,224	896.3	894.1	2.2	15.7

* Flow and velocity values correspond to the same time as the maximum water surface elevation and may not be the maximum flow or velocity over the duration of the flood event.

F-D2.7 SPILLWAY AT MAPLE RIVER

A weir spillway has been selected to divert water from the tributary into the LPP Diversion Channel. The design elevation of the Maple River spillway in Phase 4 is fixed at 893.63 ft, which is the water surface elevation on the Maple River associated with the 2-year equivalent local flood event (1,687 cfs; sum of 2-year events on the Maple River, Lower Rush River, and Rush River). The spillway is not submerged under normal conditions (the tailwater is less than two-thirds the critical depth over the weir crest elevation). However, for events greater than or equal to the 100-year event, the weir becomes submerged. A weir coefficient of 3.1 was used, but in the next stage of design a lower coefficient (2.6) would be recommended.

The width of the spillway is the only variable that was altered. Starting at the design width of 150 feet from Phase 3, the width was increased until the desired flow split was met. The constraint for this particular design was to match closely the results from Phase 3, with 3,000 cfs being a rough maximum for flow through the aqueduct to the protected side. As the weir width increased, the benefits from the increase became marginal. Therefore, the weir is designed to be 300 feet wide because it is a reasonably large size which limits the flow through the aqueduct. Table F-D29 presents the results from Phase 3 using the HEC-RAS steady flow model. Tables F-D30 and F-D31 present the results from Phase 4 using the HEC-RAS unsteady flow models.

Table F-D29 Phase 3 Design Results for the 150-Ft.-Wide Maple River Spillway Using the Coincidental Flow (spillway elevation 893.32 ft)

	Upstream WSEL (ft)	Upstream River Flow (cfs)	Diverted Flow (cfs)	Flow into Protected Area (cfs)
10-year	895.36	2,650	854	1,796
50-year	898.83	4,400	2,408	1,992
100-year	899.96	4,925	2,882	2,043
500-year	900.34	5,115	2,699	2,416

Table F-D30 Phase 4 Design Results for the 300-Ft.-Wide Maple River Spillway Using the Coincidental Flow (spillway elevation 893.63 ft)

	Upstream WSEL (ft)	Upstream River Flow (cfs)	Diverted Flow (cfs)	Aqueduct WSEL (ft)	Flow into Protected Area (cfs)
10-year	896.0	5,478	2,747	894.9	2,732
50-year	896.6	6,994	3,988	895.6	3,007
100-year	896.6	7,079	4,088	895.7	2,991
500-year	897.3	9,119	5,538	896.4	3,581

Table F-D31 Phase 4 Design Results for the 300-Ft.-Wide Maple River Spillway Using the Peak Flow (spillway elevation 893.63 ft)

	Upstream WSEL (ft)	Upstream River Flow (cfs)	Diverted Flow (cfs)	Aqueduct WSEL (ft)	Flow into Protected Area (cfs)
10-year	895.9	5,206	2,645	894.7	2,561
50-year	896.7	7,407	4,323	895.6	3,084
100-year	896.8	7,595	4,473	895.7	3,122
500-year	896.9	7,736	4,896	896.2	2,840

Water surface elevations were also checked along the entire reach of the Maple River to ensure that the aqueduct structure was not causing significant upstream flooding. Figures F-D43 through F-D50 compare existing conditions and LPP conditions along the entire reach of the Maple River. These figures show there are no significant adverse impacts upstream or downstream of the aqueduct structure. There is a small difference in the existing conditions model compared to the LPP model between river stations 27,000 and 40,000. This difference is due to minor geometry differences in the two models shown in Figures F-D41 and F-D42.

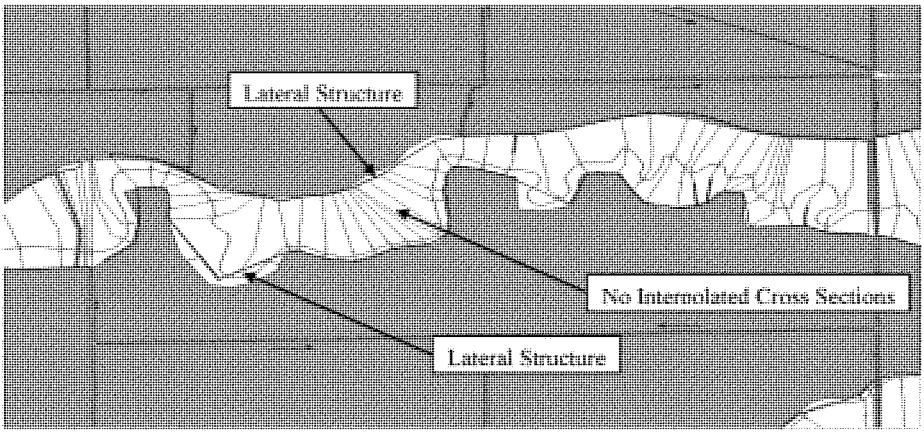


Figure F-D41 Geometry Near the Maple River Structure in the Existing Conditions Model

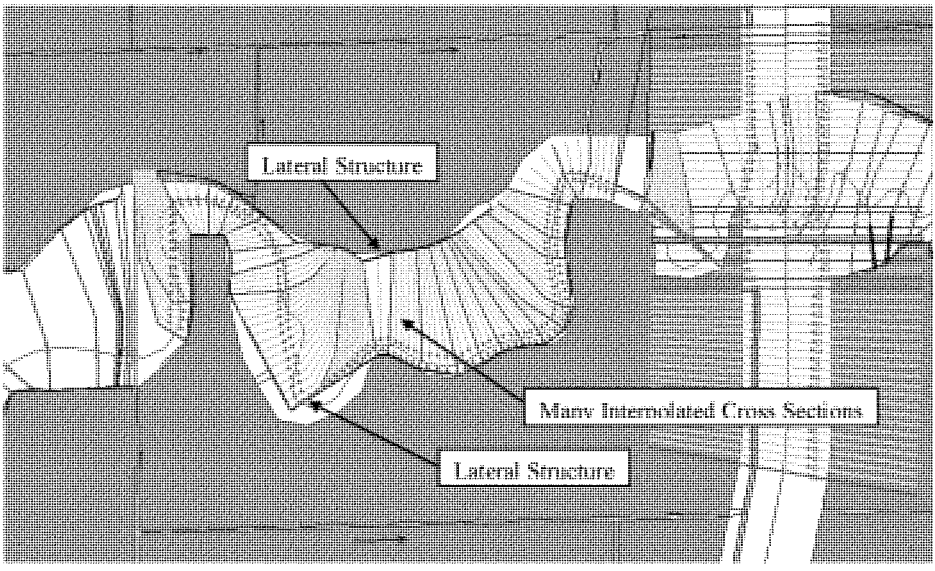


Figure F-D42 Geometry Near the Maple River Structure in the LPP Model

In Figure F-D41, the geometry shows existing lateral structures along the length of the Maple River. These structures allow high flows (greater than or equal to the 50-year event) to spill over into the adjacent storage areas (floodplain). In the LPP model, the lateral structures were initially removed to resolve stability issues. However, removing these structures prevented flow from leaving the main channel. Because of this, flows through the channel in the LPP model were higher than they ought to be, increasing the water levels. Lateral structures were added to the LPP model, but due to the addition of interpolated cross sections, the lateral structures are not exactly the same as in the existing conditions model. This is the reason for the small difference in WSEL in this area.

It is clear that the structure does not raise water levels for the smaller 10-year event (Figure F-D43). This is because in the existing conditions, the smaller event does not have water elevations high enough to spill into the floodplain. In this case, the lateral structures play no part. For higher events, the difference in this particular area of the reach is shown. However, there is no difference in WSEL upstream of this area when comparing existing conditions to the LPP conditions. This reinforces the fact that the small geometry difference in this particular area is the cause of the WSEL difference, not the Maple River structure itself.

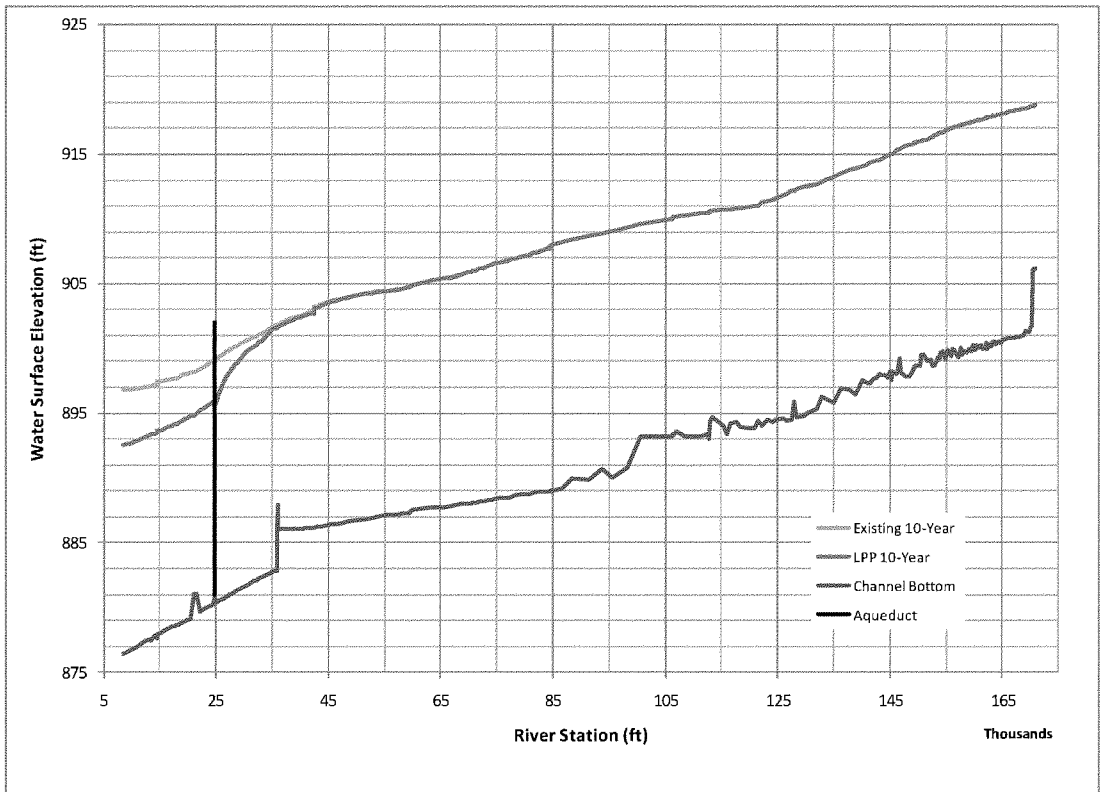


Figure F-D43 Comparing Existing Conditions to LPP Conditions Along the Maple River for the Coincidental 10-Year Event

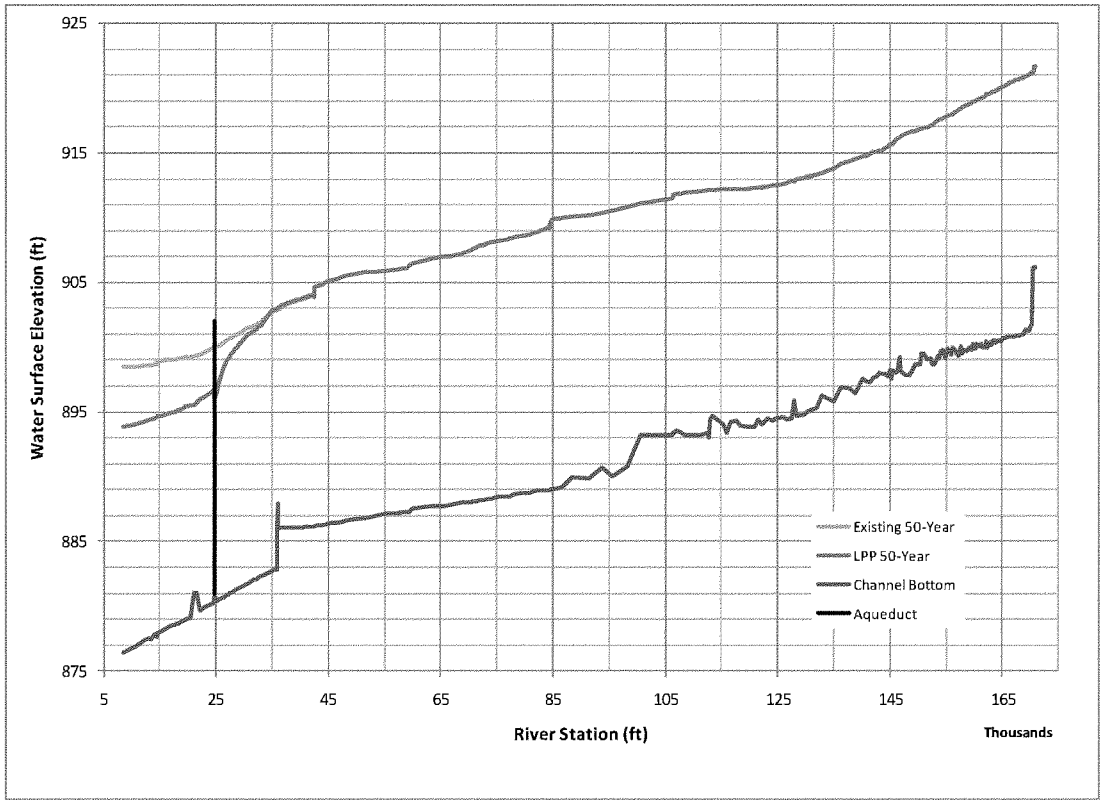


Figure F-D44 Comparing Existing Conditions to LPP Conditions Along the Maple River for the Coincidental 50-Year Event

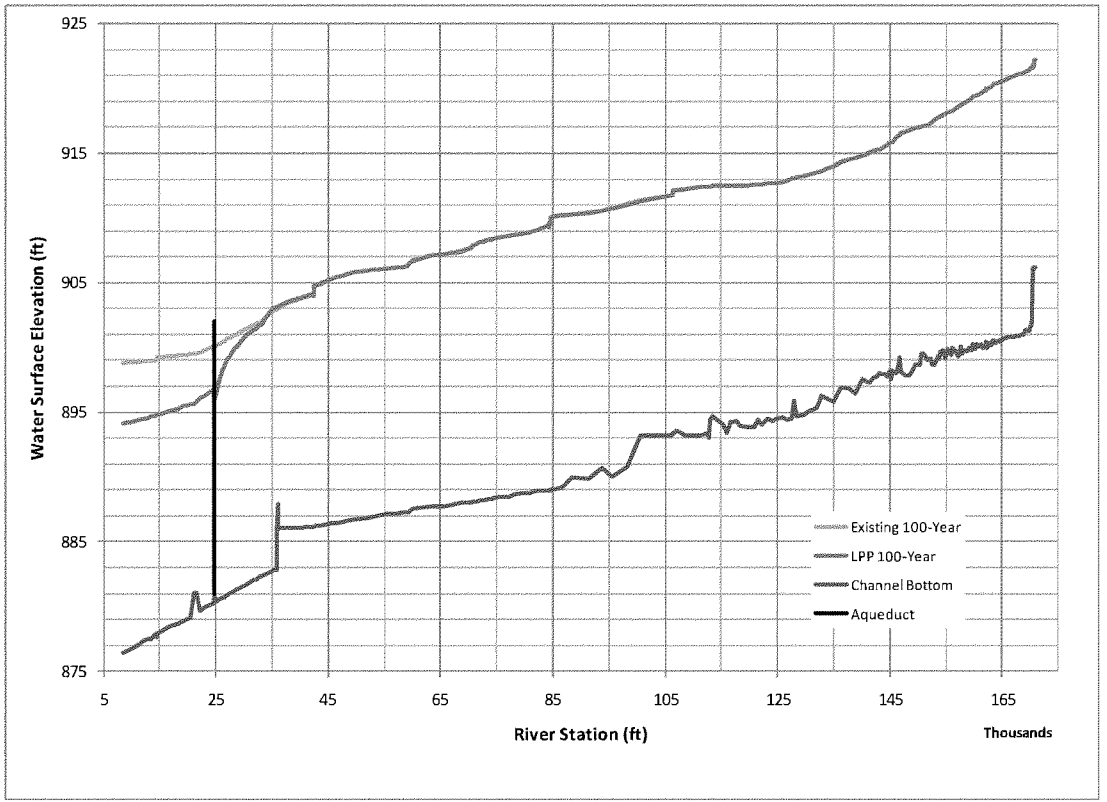


Figure F-D45 Comparing Existing Conditions to LPP Conditions Along the Maple River for the Coincidental 100-Year Event

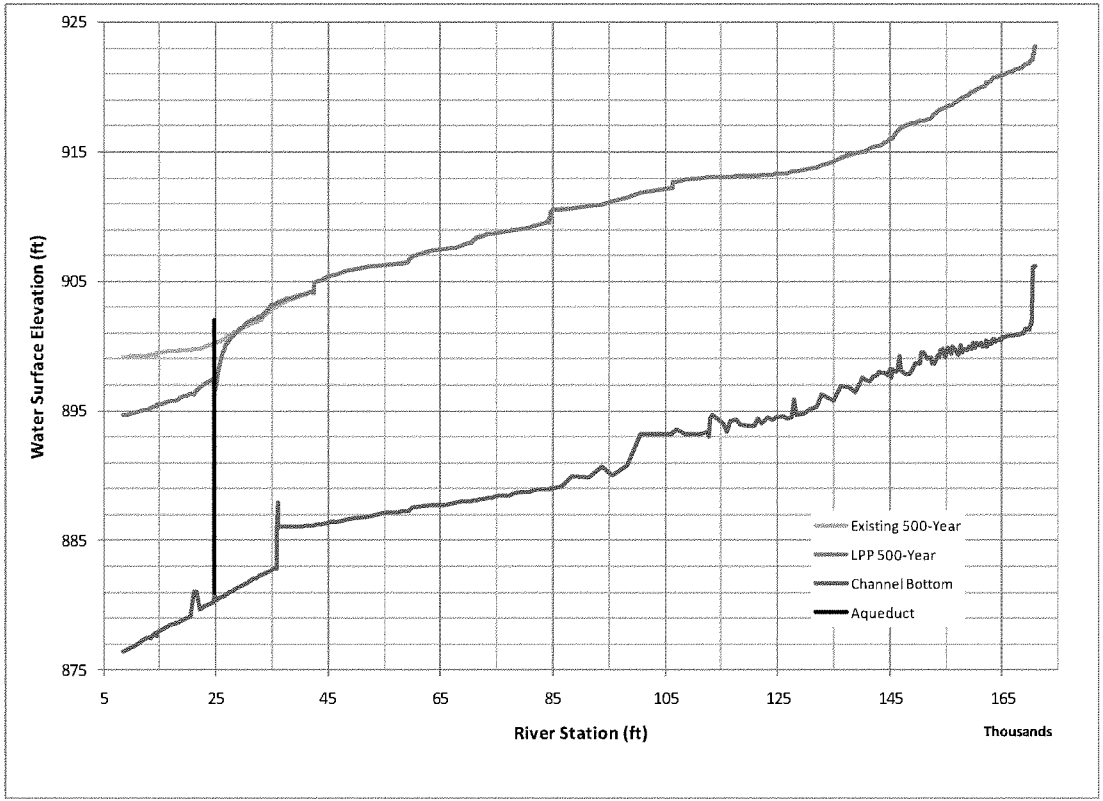


Figure F-D46 Comparing Existing Conditions to LPP Conditions Along the Maple River for the Coincidental 500-Year Event

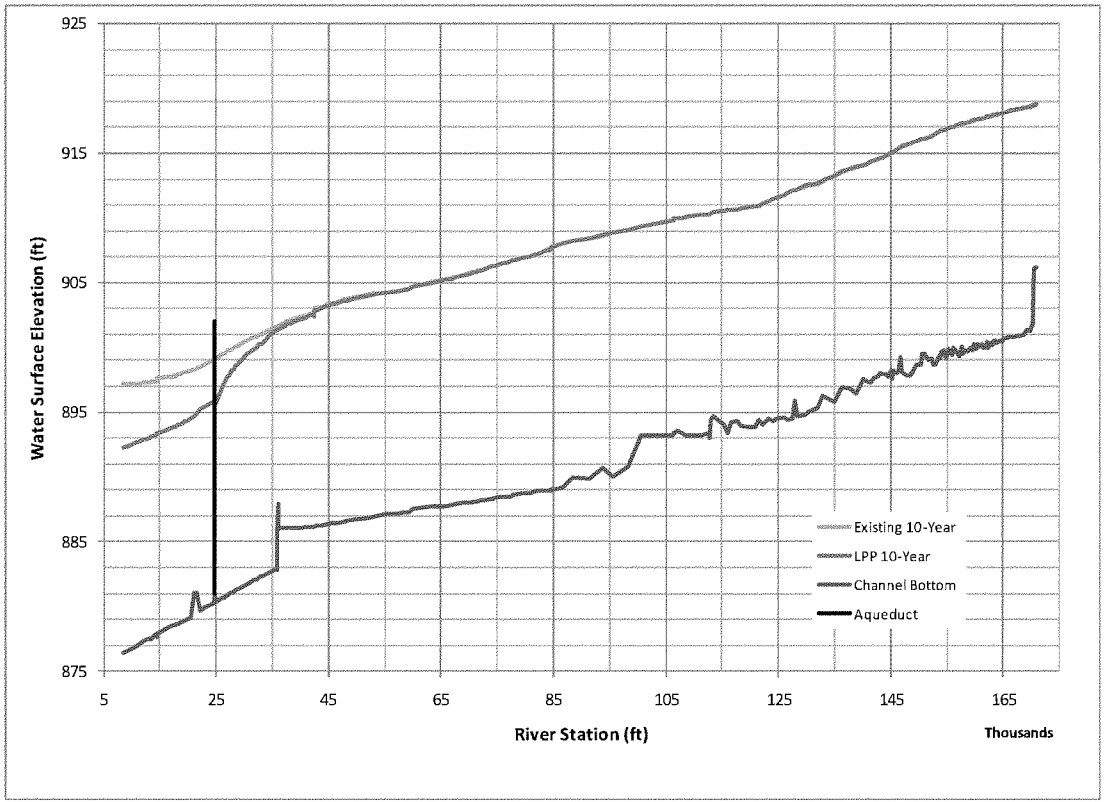


Figure F-D47 Comparing Existing Conditions to LPP Conditions Along the Maple River for the Local 10-Year Event

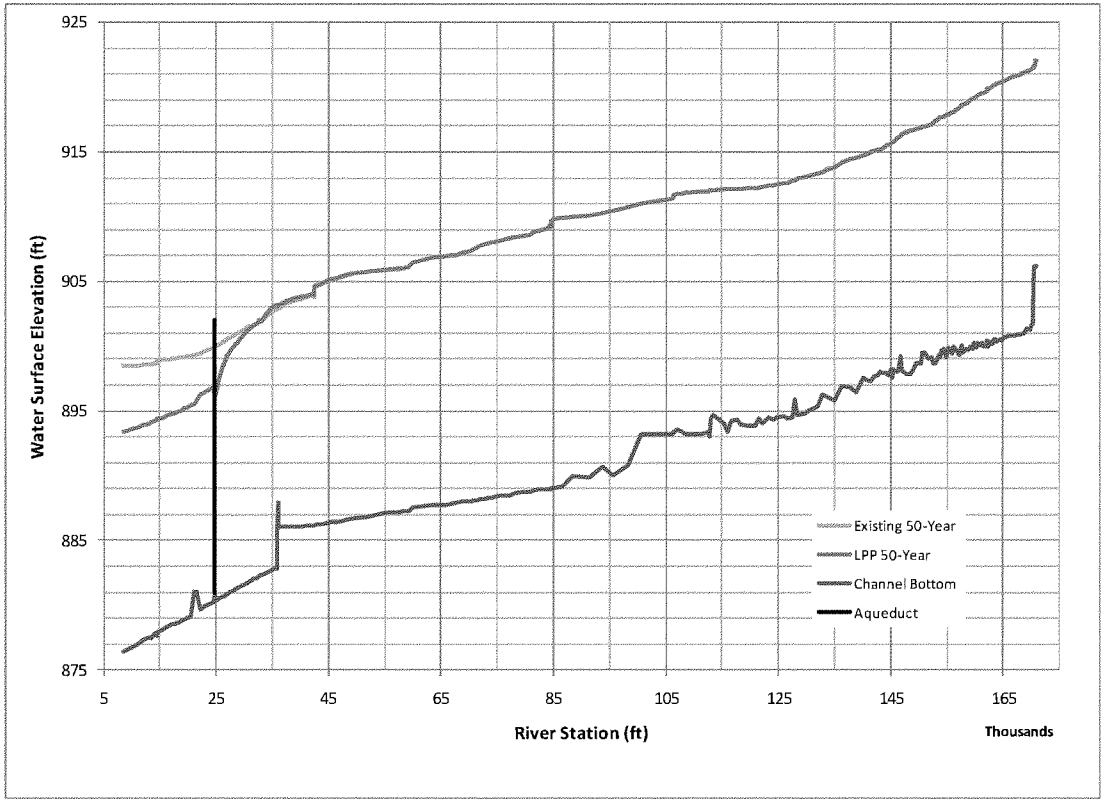


Figure F-D48 Comparing Existing Conditions to LPP Conditions Along the Maple River for the Local 50-Year Event

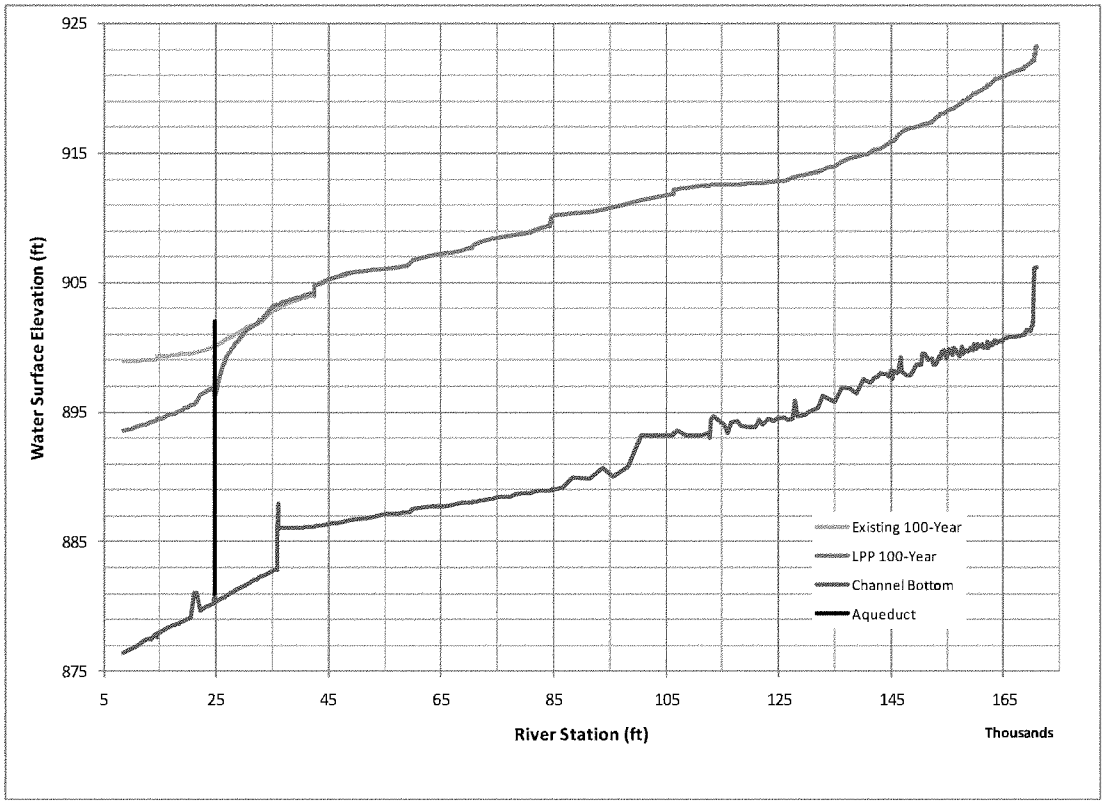


Figure F-D49 Comparing Existing Conditions to LPP Conditions Along the Maple River for the Local 100-Year Event

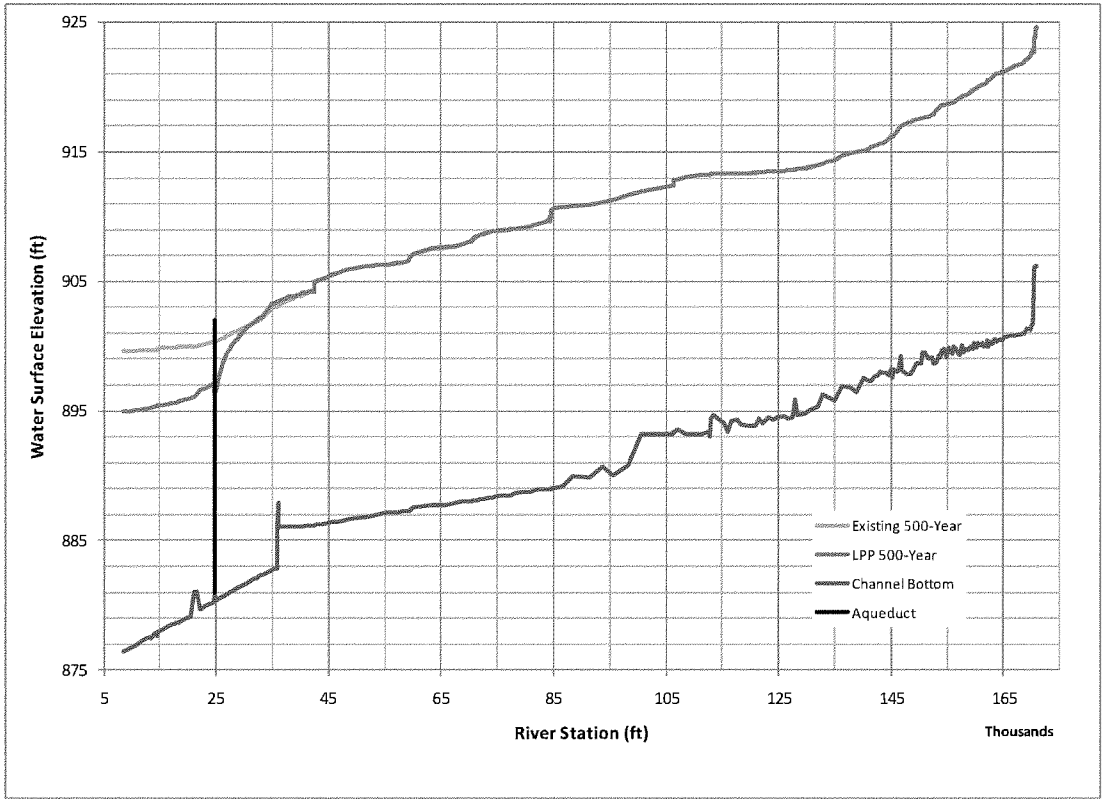


Figure F-D50 Comparing Existing Conditions to LPP Conditions Along the Maple River for the Local 500-Year Event

F-D2.8 DROP STRUCTURE AT LOWER RUSH RIVER

A description of the drop structure at the Lower Rush River can be found in the main section of Appendix F. This portion of Exhibit D presents a comparison of water surface elevations at the Lower Rush River structure and design calculations of the drop structure.

F-D2.8.1 Comparison of LPP Diversion Channel and Lower Rush River Hydrology
Flows, water surface elevations, flow areas, and average velocities for the Diversion Channel at the Lower Rush River drop structure for the 10-, 50-, 100-, and 500-year events are presented in Table F-D32 and F-D33. The locations shown in the tables are Station 57083 for Phase 3 and 59662 for Phase 4 which is where flows diverted from the Lower Rush River enter the Diversion Channel. Water surface elevations in the Diversion Channel at the Lower Rush River structure, in the Lower Rush River for local tributary flood events, and in the Lower Rush River for coincidental flood events, plotted against the cross section data, are presented in Figure F-D51. Tabular results of existing conditions local and coincidental flood events in the tributaries are presented in Appendix F, Exhibit A.

Table F-D32 Phase 3 Hydrology in the Diversion Channel Downstream of the Lower Rush River Structure (Station 57083)

Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
5	3,543	879.38	3,340	1.06
10	9,423	883.54	5,300	1.78
20	15,782	886.13	6,642	2.38
50	25,155	889.02	8,254	3.05
100	32,178	890.89	9,357	3.44
200	37,906	892.29	10,296	3.71
500	43,372	893.67	11,276	3.91

Table F-D33 Phase 4 Hydrology in the Diversion Channel Downstream of the Lower Rush River Structure (Station 59662)

Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
10	7,561	883.93	3,672	2.06
50	20,703	890.28	6,693	3.09
100	24,941	891.63	7,625	3.27
500	28,350	892.50	8,302	3.41

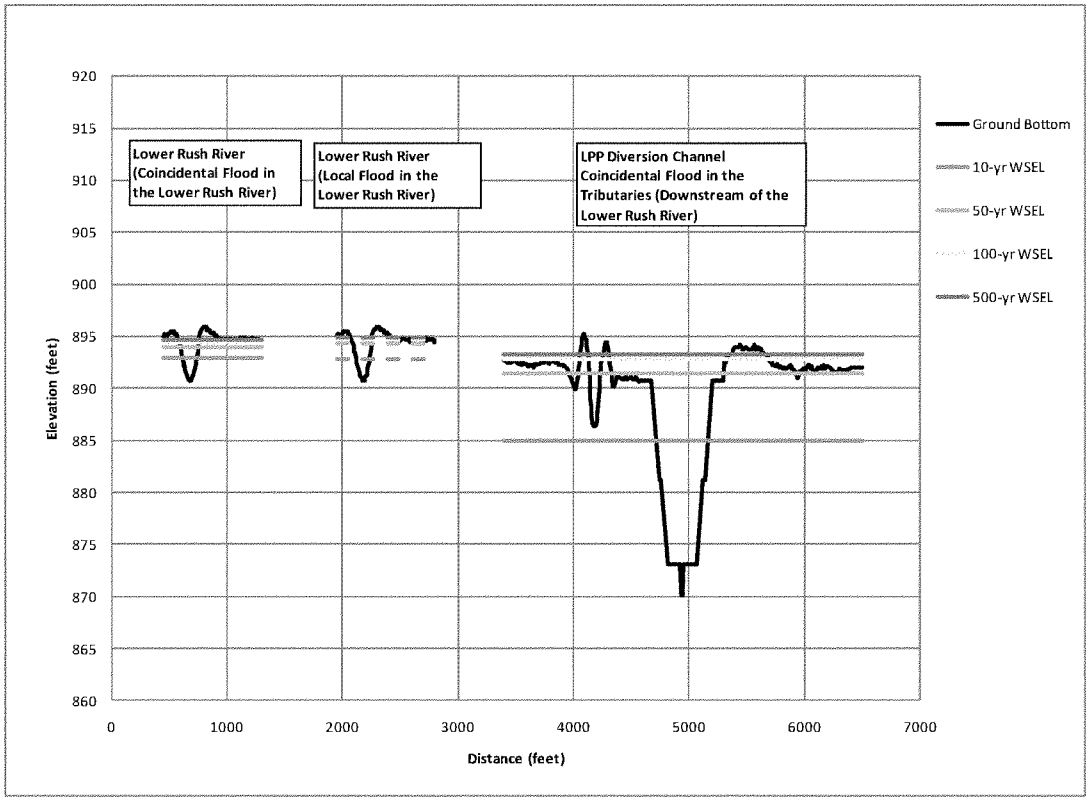


Figure F-D51 Phase 4 Hydrology in the Diversion Channel at the Lower Rush River Structure and in the Lower Rush River

F-D2.8.2 Design of Lower Rush River Drop Structure

The stepped concrete drop spillway was designed to pass the 500-year local event. For average and low flows, the majority of the flow passes into the Diversion Channel via the fish passage. The fish passage channel is discussed in Appendix F, Exhibit G and the low flow channel in the Diversion Channel extending from the Lower Rush River to the Red River of the North is discussed in Appendix F, Exhibit K. The width of the Lower Rush River stepped drop structure is 60 feet. The drop structure is designed to handle the 500-yr local event with a 50 foot stilling basin. The stilling basin length is dependent on the length of the hydraulic jump. Dimensions of the Lower Rush River stepped spillway are outlined in Table F-D34. The calculations used to determine the 60 foot width of the stepped drop structure are presented in Table F-D35.

Table F-D34 Lower Rush River Stepped Spillway Parameters

Tributary bed invert elevation	885.4	ft
Diversion channel bed elevation	872.17	ft
Diversion channel 500-yr flood elevation (local on the Tributaries)	891.97	ft
Diversion channel 10-yr flood elevation	883.87	ft
10-yr flood elevation on the Lower Rush	891.55	ft
WSEL difference between the 10-yr local event on the Lower Rush and the Diversion Channel bed invert	19.4	ft
Crest of steps	886.76	ft
Number of steps	16	
Height of each step	0.9	ft
Length of each step	1.5	ft
Total height of steps	14.6	ft
Total length of steps	24.0	ft
Spillway width	60	ft

Table F-D35 Flow Regime Over the Steps and Stilling Basin Parameters of the Lower Rush River Stepped Spillway with a 60-Foot Width

Local Event (yr)	Flow in Tributary (cfs)	Critical Depth (ft)	Flow Regime	Water Depth Upstream of Hydraulic Jump (ft)*	Water Depth Down-stream of Hydraulic Jump (ft)*	Velocity Down-stream of Hydraulic Jump (ft/s)	Length of Hydraulic Jump (ft)
10	528	1.34	skim	0.5	2.85	3.1	17
50	1601	2.81	skim	0.92	6.49	4.1	39
100	1,887	3.13	skim	1.03	7.24	4.3	43
500	2,178	3.45	skim	1.13	7.94	4.6	48

* Tailwater effects were not incorporated into the sizing of the stilling basin.

F-D2.9 DROP STRUCTURE AT RUSH RIVER

A description of the drop structure at the Rush River can be found in Appendix F. This portion of Exhibit D presents a comparison of water surface elevations at the Rush River structure and design calculations of the drop structure.

F-D2.9.1 Comparison of Diversion Channel and Rush River Hydrology

Flows, water surface elevations, flow areas, and average velocities for the Diversion Channel at the Rush River drop structure for the 5-, 10-, 20-, 50-, 100-, 200-, and 500-year events with the Phase 3 hydrology are presented in Table F-D36. Table F-D37 provides the flows, water surface elevations, flow areas, and average velocities for the Diversion Channel at the Rush River drop structure for the 10-, 50-, 100-, and 500- year events with the Phase 4 hydrology. The location shown in the tables is Station 47124 for Phase 3 and Station 42719 for Phase 4 which is where flows diverted from the Rush River enter the Diversion Channel. Water surface elevations in the Diversion Channel at the Rush River structure, in the Rush River for local tributary flood events, and in the Rush River for coincidental flood events, plotted against the cross section data, are presented in Figure F-D52. Tabular results of existing conditions local and coincidental flood events in the tributaries are presented in Appendix F, Exhibit A.

Table F-D36 Phase 3 Hydrology in the Diversion Channel Downstream of the Rush River Structure (Station 47124)

Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
5	3,789	879.16	4,146	0.91
10	9,749	883.09	6,261	1.60
20	16,202	885.45	7,658	2.20
50	25,725	888.06	9,211	2.93
100	32,888	889.79	10,237	3.38
200	38,626	891.12	11,030	3.70
500	44,132	892.48	11,835	3.95

Table F-D37 Phase 4 Hydrology in the Diversion Channel Downstream of the Rush River Structure (Station 47219)

Event (yr)	Flow (cfs)	WSEL (ft)	Flow Area (sq ft)	Velocity (ft/s)
10	7,932	882.46	4,139	1.92
50	22,441	888.11	7,072	3.17
100	26,886	889.35	8,051	3.34
500	29,437	890.14	8,669	3.40

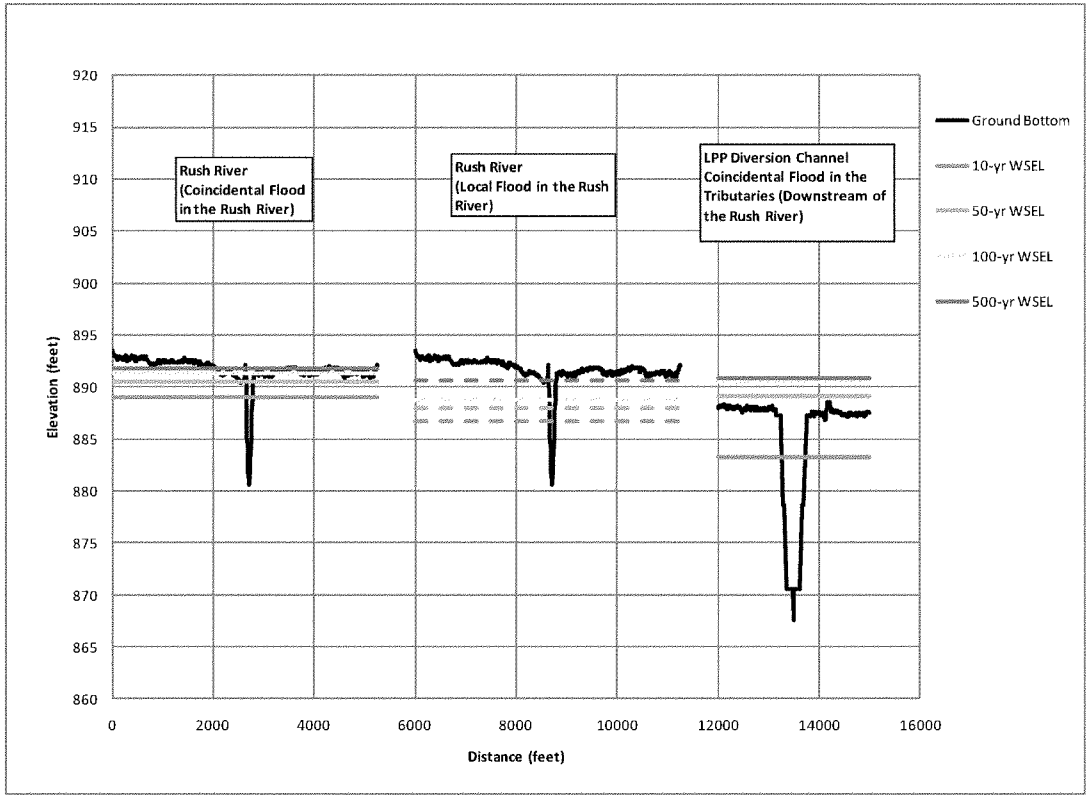


Figure F-D52 Phase 4 Hydrology in the Diversion Channel Downstream of the Rush River Structure and in the Rush River

F-D2.9.2 Design of Rush River Drop Structure

The stepped concrete drop spillway was designed to pass the 500-year local event. For average and low flows, the majority of the flow passes into the Diversion Channel via the fish passage. The fish passage channel is discussed in Appendix F, Exhibit G and the low flow channel in the Diversion Channel extending from the Rush River to the Red River of the North is discussed in Appendix F, Exhibit K. The width of the Rush River stepped drop structure is 100 feet. The drop structure is designed to handle the 500-yr local event with a 50 foot stilling basin. The stilling basin length is dependent on the length of the hydraulic jump. Dimensions of the Rush River stepped spillway are outlined in Table F-D38. The calculations used to determine the 100 foot width of the stepped drop structure are presented in Table F-D39.

Table F-D38 Rush River Stepped Spillway Parameters

Tributary bed invert elevation	879.5	ft
Diversion channel bed elevation	869.62	ft
Diversion channel 500-yr flood elevation (local on the Tributaries)	889.85	ft
Diversion channel 10-yr flood elevation	882.57	ft
10-yr flood elevation on the Rush	889.68	ft
WSEL difference between the 10-yr local event on the Rush and the Diversion Channel bed invert	20.1	ft
Crest of steps	880.9	ft
Number of steps	10	
Height of each step	1.1	ft
Length of each step	1.7	ft
Total height of steps	11.3	ft
Total length of steps	17.0	ft
Spillway width	100	ft

Table F-D39 Flow Regime over the Steps and Stilling Basin Parameters of the Rush River Stepped Spillway with a 100-Foot Width

Local Event (yr)	Flow in Tributary (cfs)	Critical Depth (ft)	Flow Regime	Water Depth Upstream of Hydraulic Jump (ft)*	Water Depth Down-stream of Hydraulic Jump (ft)*	Velocity Down-stream of Hydraulic Jump (ft/s)	Length of Hydraulic Jump (ft)
10	98	0.31	nappe	0.16	0.53	1.9	3
50	588	1.02	trans	0.39	2.18	2.7	13
100	930	1.39	skim	0.48	3.1	3	19
500	4,210	3.8	skim	1.34	8.41	5	*50*

* Tailwater effects were not incorporated into the sizing of the stilling basin.

F-D2.10 OUTLET 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 feet of the Diversion Channel. This outlet configuration was possible because the Diversion Channel outlet invert 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 Channel downstream of the Wild Rice River was reduced from 35,000 cfs 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 of the Diversion Channel increased from approximately 11 ft to 20 ft as shown in Figure F-D53.

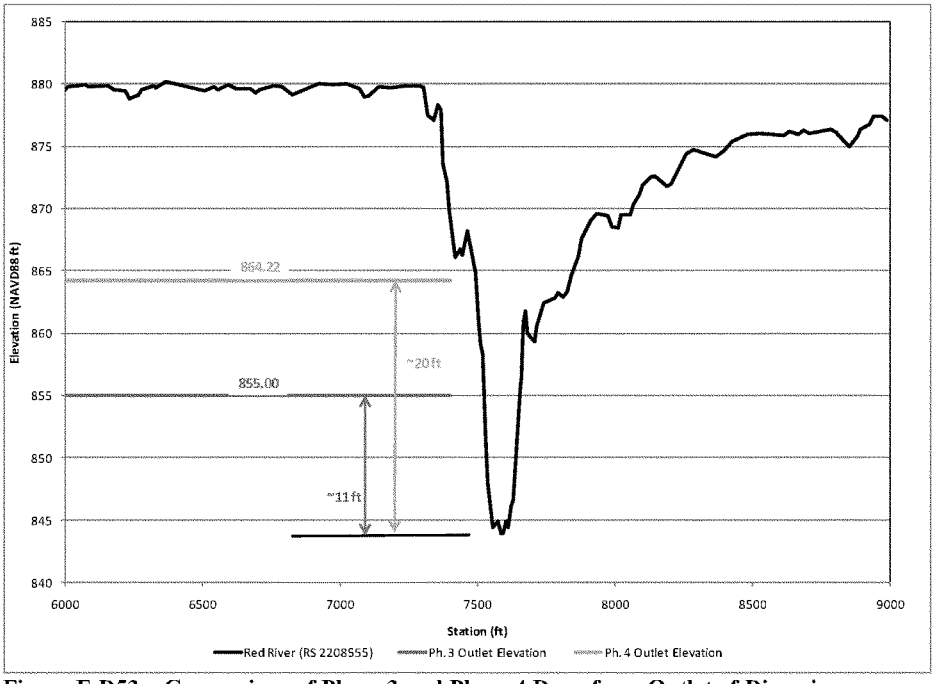


Figure F-D53 Comparison of Phase 3 and Phase 4 Drop from Outlet of Diversion Channel to Red River

In order to securely convey flow over the drop from the Diversion Channel to the Red River, the outlet structure was modified from a riprap channel to a concrete ogee type spillway. To prevent ponding at the outlet from the Diversion Channel the crest of the ogee spillway was set slightly above the main invert of the Diversion Channel. Figure F-D54 and Figure F-D55 show plan and profile schematics of the proposed Diversion Channel Outlet Structure.

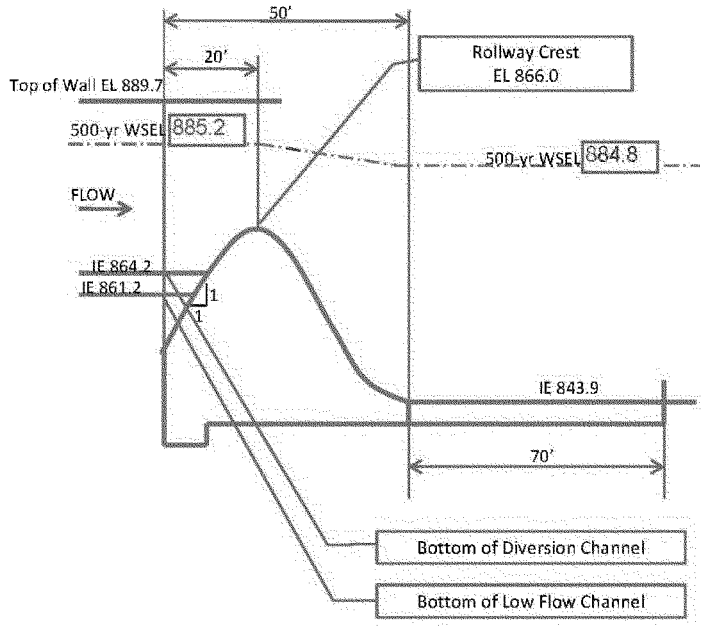


Figure F-D54 Outlet from Diversion Channel Profile View Schematic

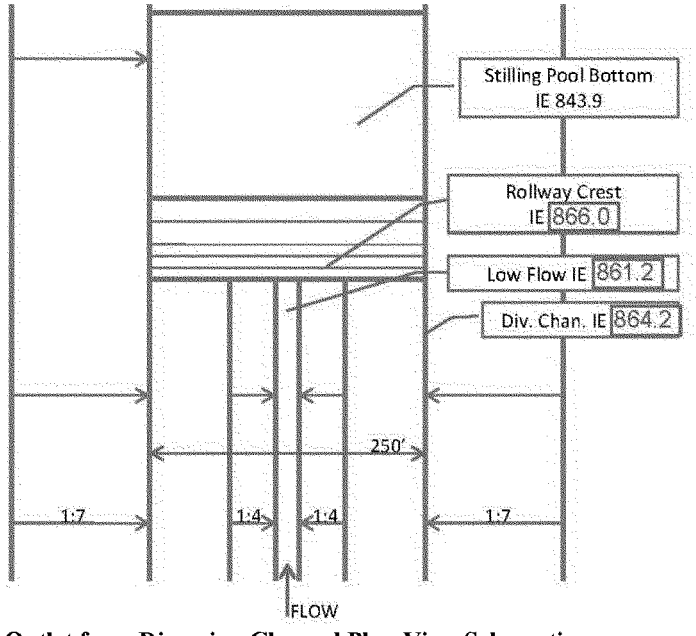


Figure F-D55 Outlet from Diversion Channel Plan View Schematic

F-D2.11 CONTROL STRUCTURE ON WOLVERTON CREEK

A description of the LPP control structure on Wolverton Creek can be found in Appendix F. This portion of Exhibit D presents design concepts and calculations for the Wolverton Creek control structure, detailed HEC-RAS modeling outputs at the control structure, and comparisons of water surface elevations at the control structure using Phase 4 hydrology.

F-D2.11.1 Flow Partitioning

For the events analyzed during Phase 4, the Wolverton Creek control structure functions as an open-close structure. In other words the gates on Wolverton Creek are either completely open or completely closed. During flood events, when the gates are closed, flows on Wolverton Creek, upstream of the control structure discharge into the area of staged water and are conveyed into the protected area either through one of the control structures located on the Red River or Wild Rice River, or through the inlet to the Diversion Channel.

F-D2.11.2 Gate Number and Sizing – Concepts

The number and sizing 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.

F-D2.11.3 Gate Number and Sizing

It is desirable to have as little impact on the smaller, more frequent, flood events (i.e. no change in WSEL or gate operation required). For these events the gates on Wolverton Creek will remain fully open. For larger flood events, 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 were left in the closed position, and flows into the protected area were controlled by the gates located on the Red River and Wild Rice River. In order to maintain computational stability in the HEC-RAS unsteady flow model, however, the Wolverton Creek control structure gates were left partially open. In reality, when the Wolverton Creek control structure gates are operated they will either be fully open or fully closed during a flood event.

The gate height selected for the Wolverton Creek control structure matches the height of the existing culverts, 10-ft tall. However, the top of the control structure was determined based on the water surface elevation upstream of the control structures during a 500-year flood event. A concept drawing for the control structure is presented in Figure F-D56.

Water surface elevations on either side of the Wolverton Creek control structure for varying flood events are presented Table F-D40 and graphically in Figure F-57. The location of the control structure in the LPP HEC-RAS model is inline structure station 9079.5.

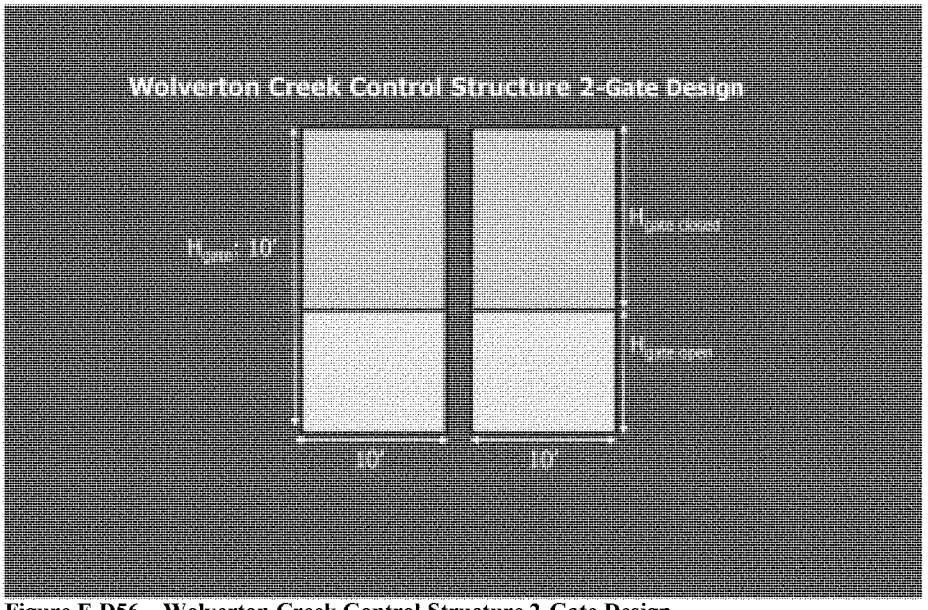


Figure F-D56 Wolverton Creek Control Structure 2-Gate Design

Table F-D40 HEC-RAS Outputs for the Proposed LPP Control Structure on Wolverton Creek

Event (yr)	Existing Water Surface Elevation (ft)[2]	Existing Flow into Protected Area (cfs)[2]	Proposed Headwater Elevation[1] (ft)	Proposed Tailwater Elevation[1] (ft)
10-Year	907.47	1950.39	916.3	903.75
50-Year	912.98	2509.00	920.86	904.44
100-Year	913.80	2731.78	922.88	905.62
500-Year	914.87	4291.07	922.46	913.86

[1]In the Phase 4With Project HEC-RAS model the gates at the Wolverton Creek control structure are only open slightly. In reality it is assumed they will be completely closed
[2]The existing flow and water surface elevations in this portion of Wolverton Creek are controlled by tailwater from the Red River

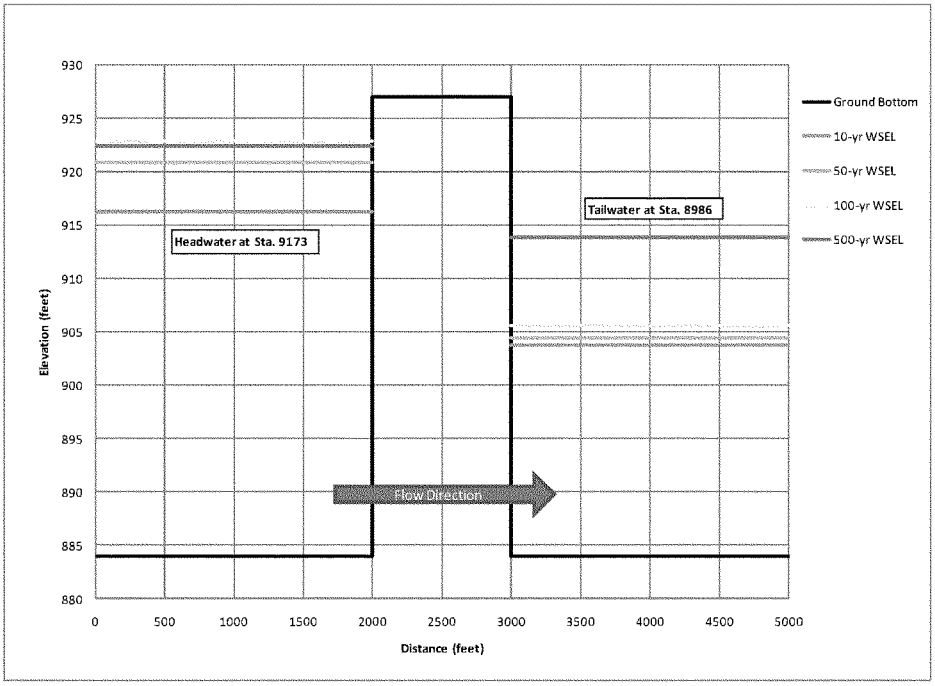


Figure F-D57 Comparison of the Headwater and Tailwater Elevations at the Wolverton Creek Control Structure

F-D2.11.4 Transition from the Natural Channel to the Control Structure

The transition from the natural channel into the control structure is accomplished with 5:1 side slopes, as shown in Figure F-D58.

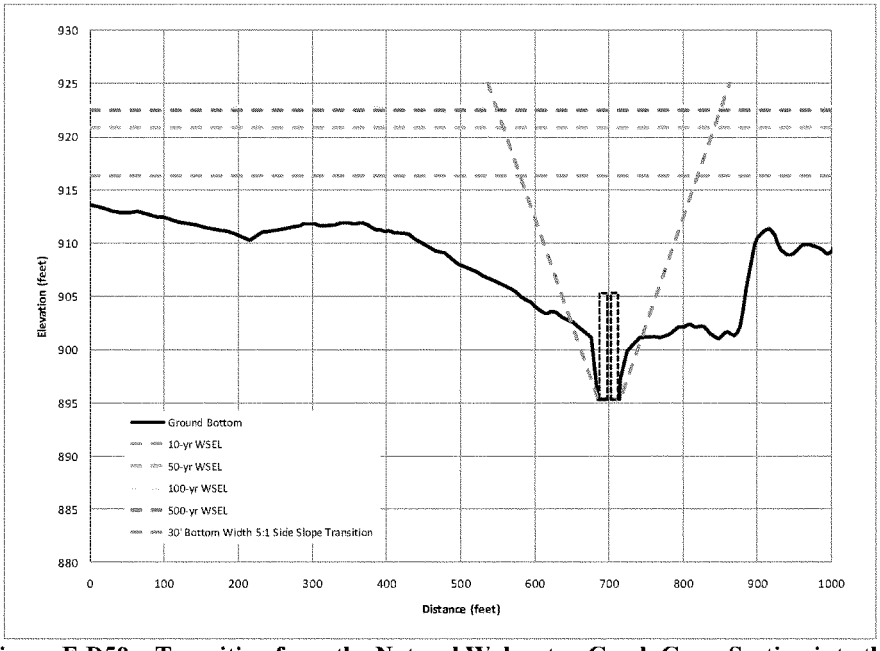


Figure F-D58 Transition from the Natural Wolverton Creek Cross Section into the Control Structure (Station 9173)

F-D3.0 WATER SURFACE ELEVATION PROFILES ALONG THE DIVERSION CHANNEL

F-D3.1 WATER SURFACE ELEVATION PROFILES ALONG THE FCP DIVERSION CHANNEL

The following information regarding the FCP was previously presented as part of Exhibit F in the Phase 3 report submitted on August 6th, 2010, and is included here for completeness.

The water surface elevation profiles along the FCP Diversion Channel for the 5-, 10-, 20-, 50-, 100-, 200-, and 500-year events are presented in Figures F-D59 to F-D66 for Year 0 Hydrology, Figures F-D67 to F-D74 for Year 25 Hydrology, and Figures F-D75 to F-D82 for Year 50 Hydrology. The location of the Inlet Weir is noted on the figures and head difference over this structure can be seen. Tailwater effects from the Red River of the North are apparent for all events in the downstream portion of the FCP Diversion Channel. No water from the Red River is diverted into the FCP Diversion Channel for the 5-year event for Year 50 hydrology; however, backwater effects from the Red River can be seen in the downstream portion of the FCP Diversion Channel (Figure F-D75 and F-D76).

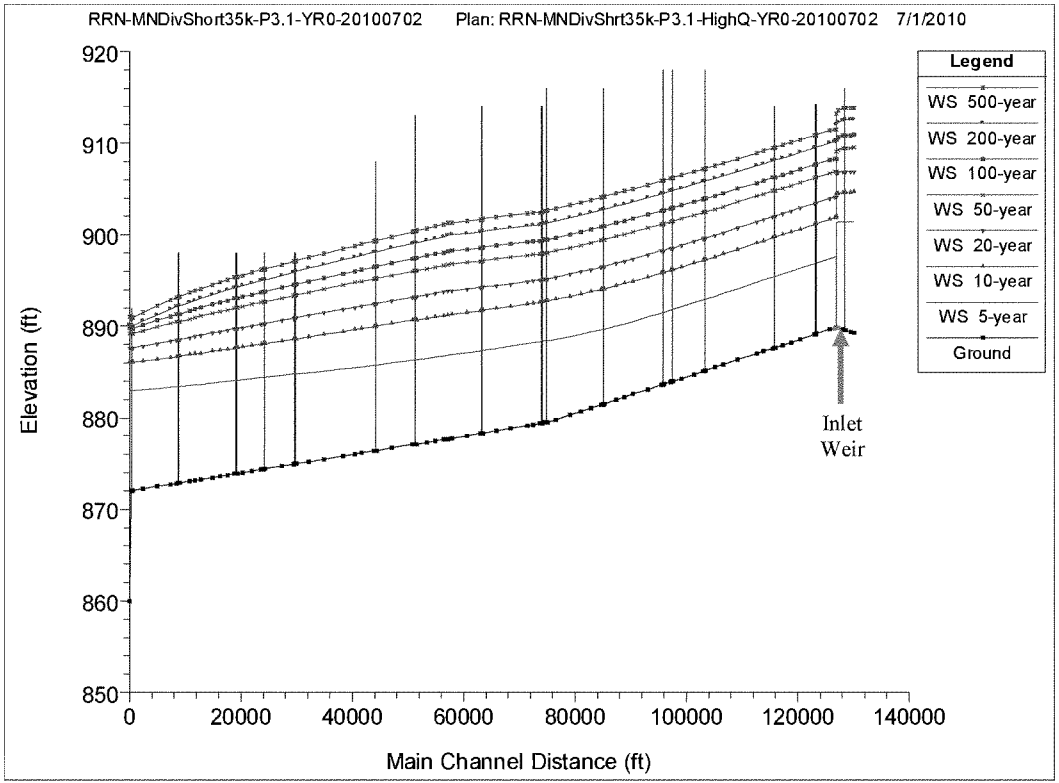


Figure F-D59 FCP Diversion Channel Water Surface Elevation Profile for All Modeled Events for Year 0 Hydrology

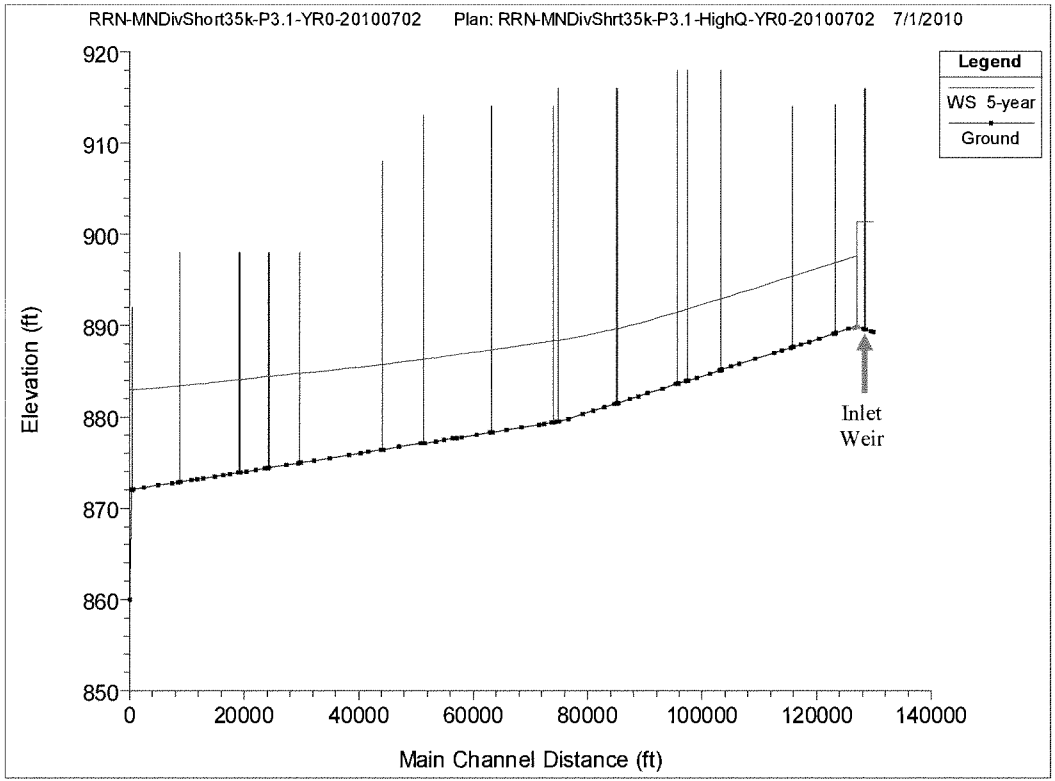


Figure F-D60 FCP Diversion Channel Water Surface Elevation Profile for the 5-Year Event for Year 0 Hydrology

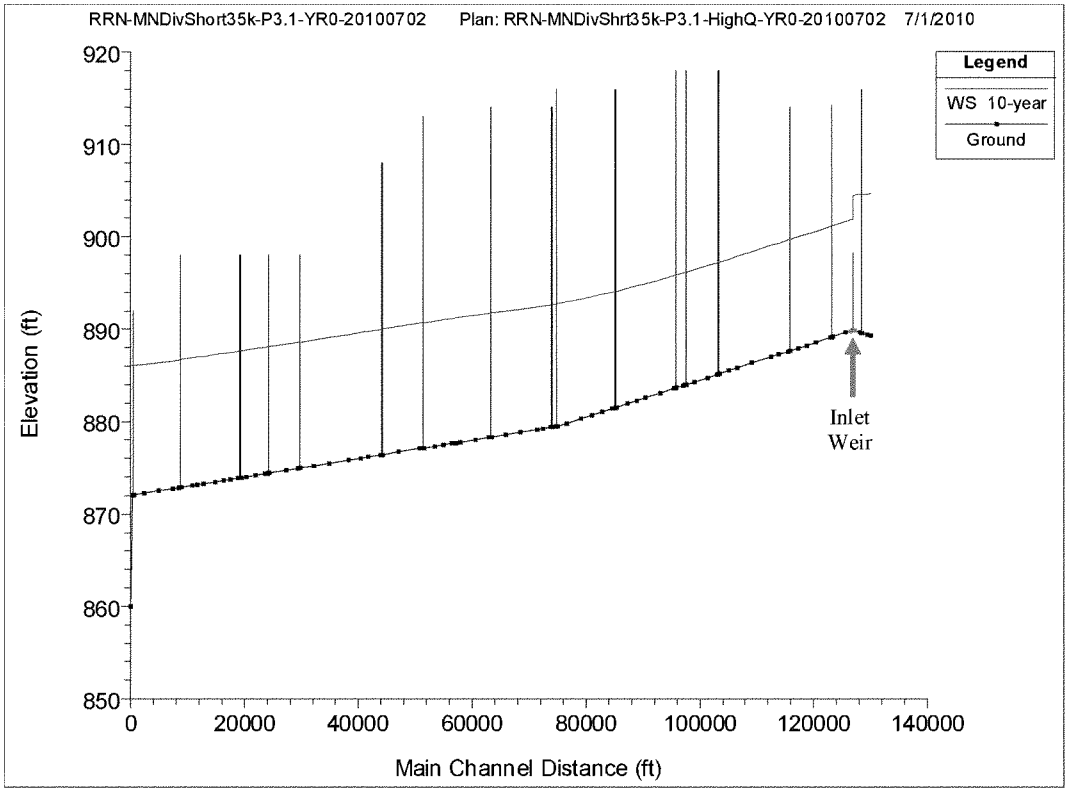


Figure F-D61 FCP Diversion Channel Water Surface Elevation Profile for the 10-Year Event for Year 0 Hydrology

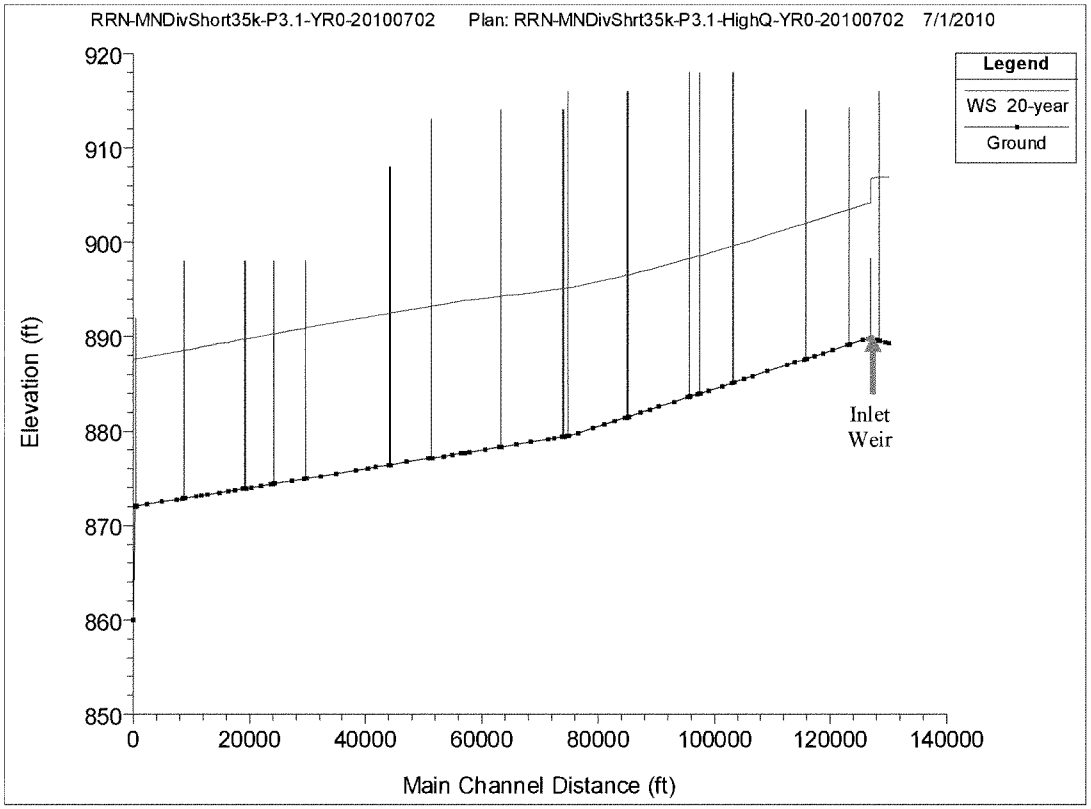


Figure F-D62 FCP Diversion Channel Water Surface Elevation Profile for the 20-Year Event for Year 0 Hydrology

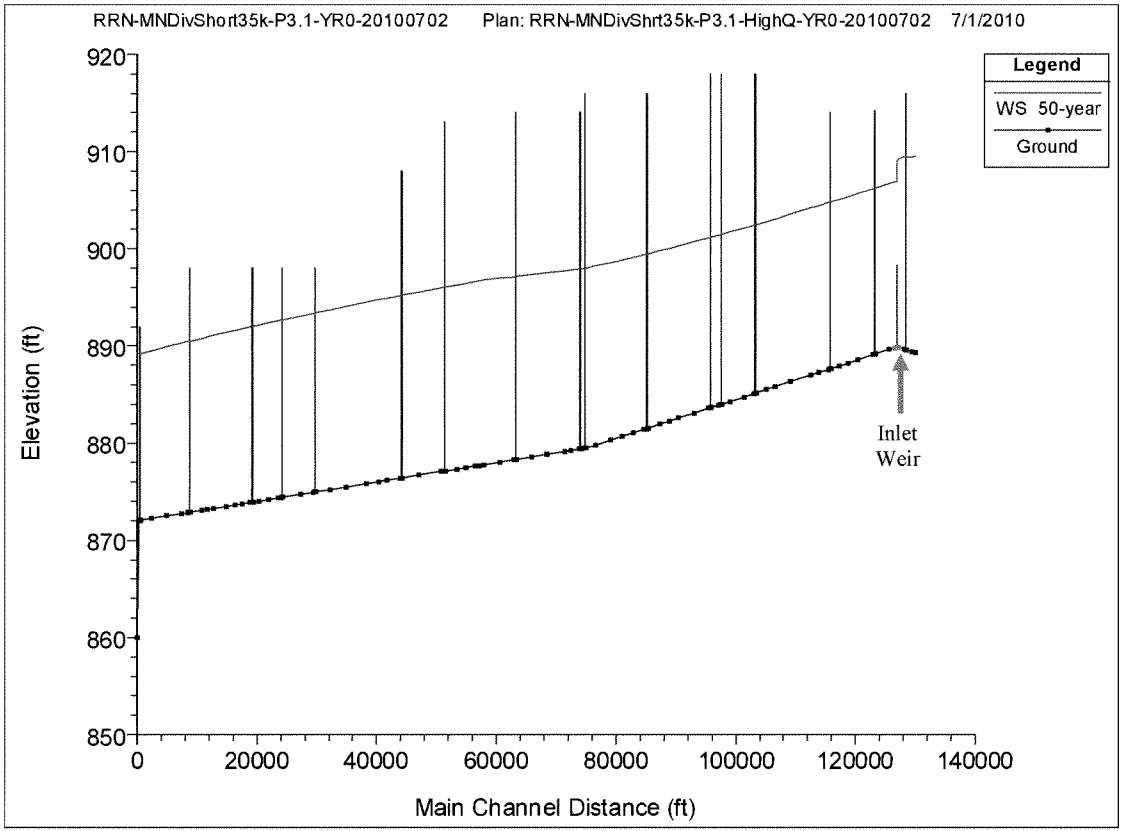


Figure F-D63 FCP Diversion Channel Water Surface Elevation Profile for the 50-Year Event for Year 0 Hydrology

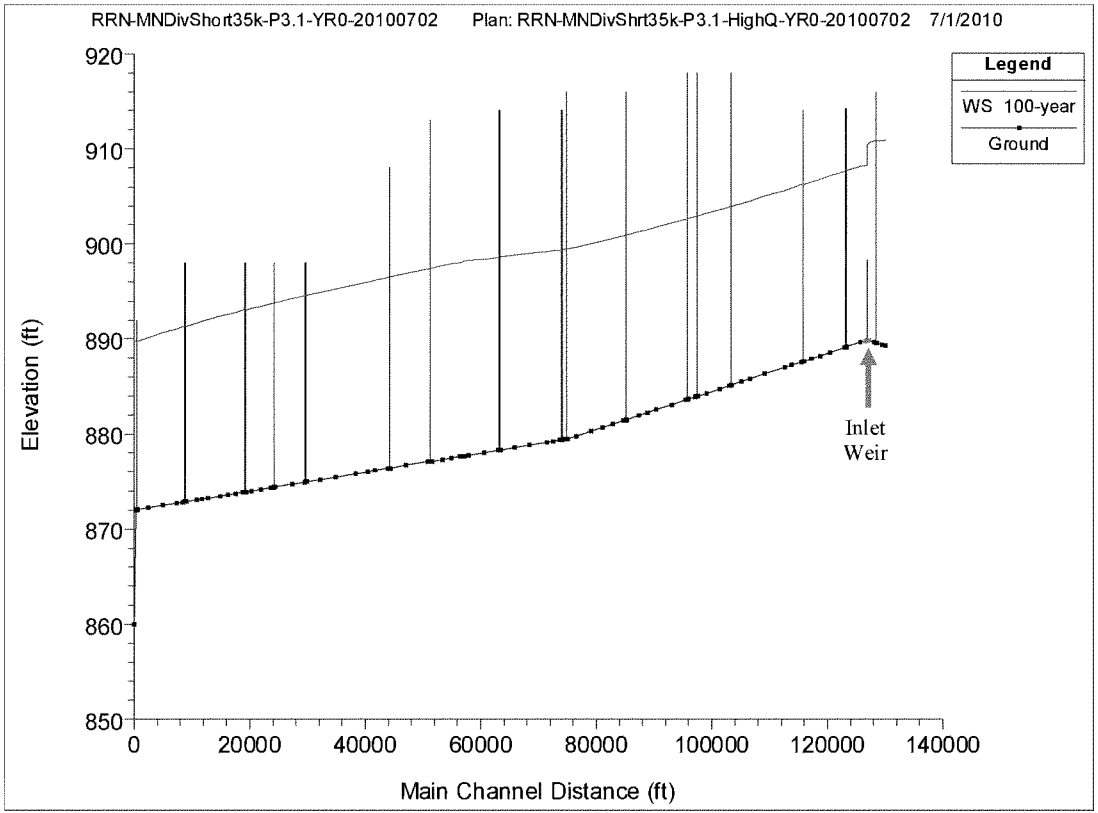


Figure F-D64 FCP Diversion Channel Water Surface Elevation Profile for the 100-Year Event for Year 0 Hydrology

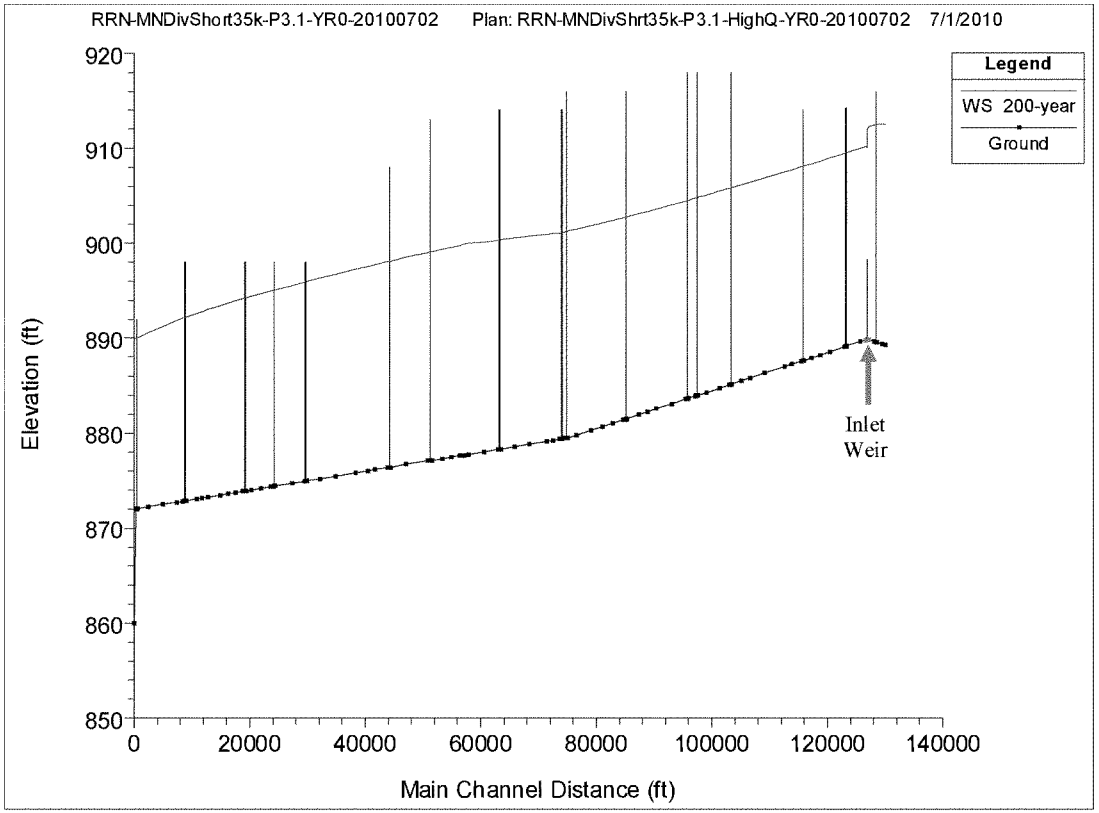


Figure F-D65 FCP Diversion Channel Water Surface Elevation Profile for the 200-Year Event for Year 0 Hydrology

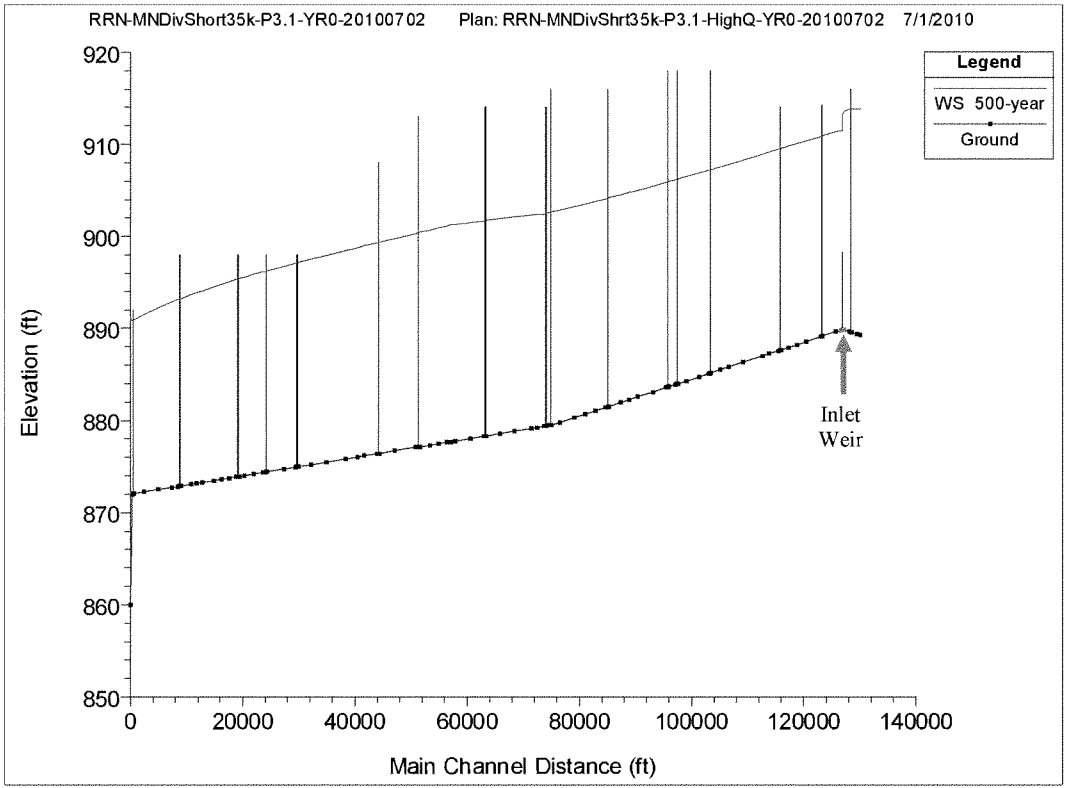


Figure F-D66 FCP Diversion Channel Water Surface Elevation Profile for the 500-Year Event for Year 0 Hydrology

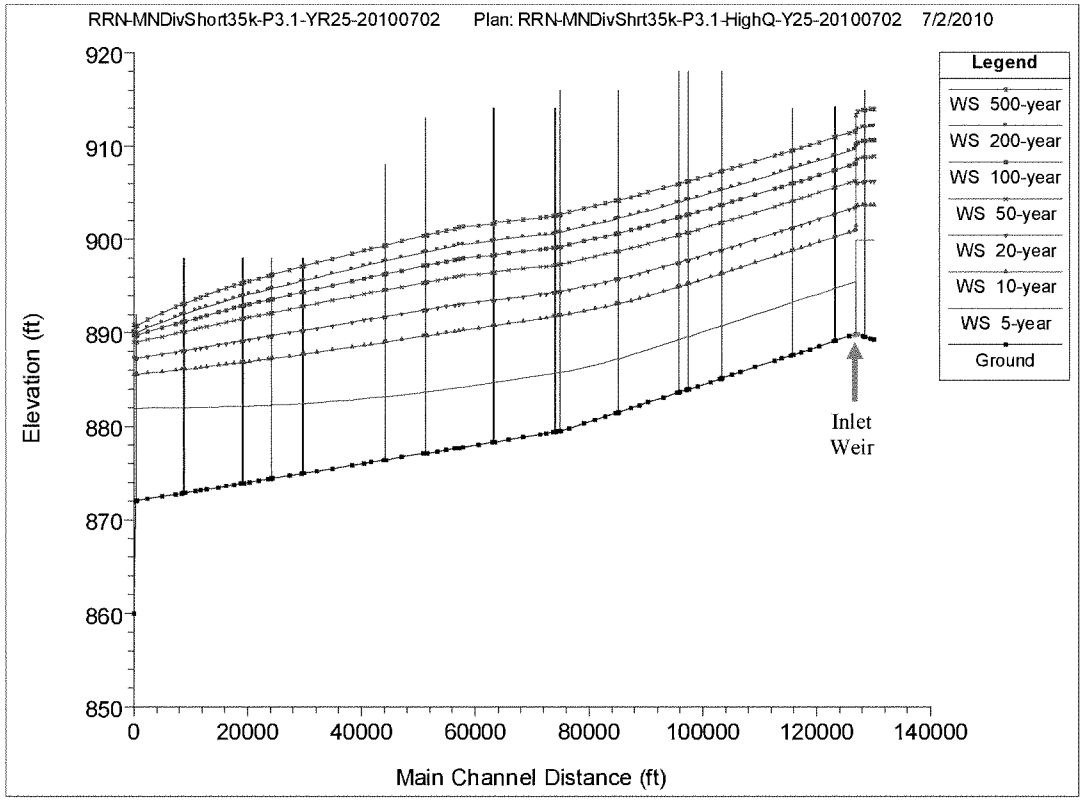


Figure F-D67 FCP Diversion Channel Water Surface Elevation Profile for All Modeled Events for Year 25 Hydrology

887

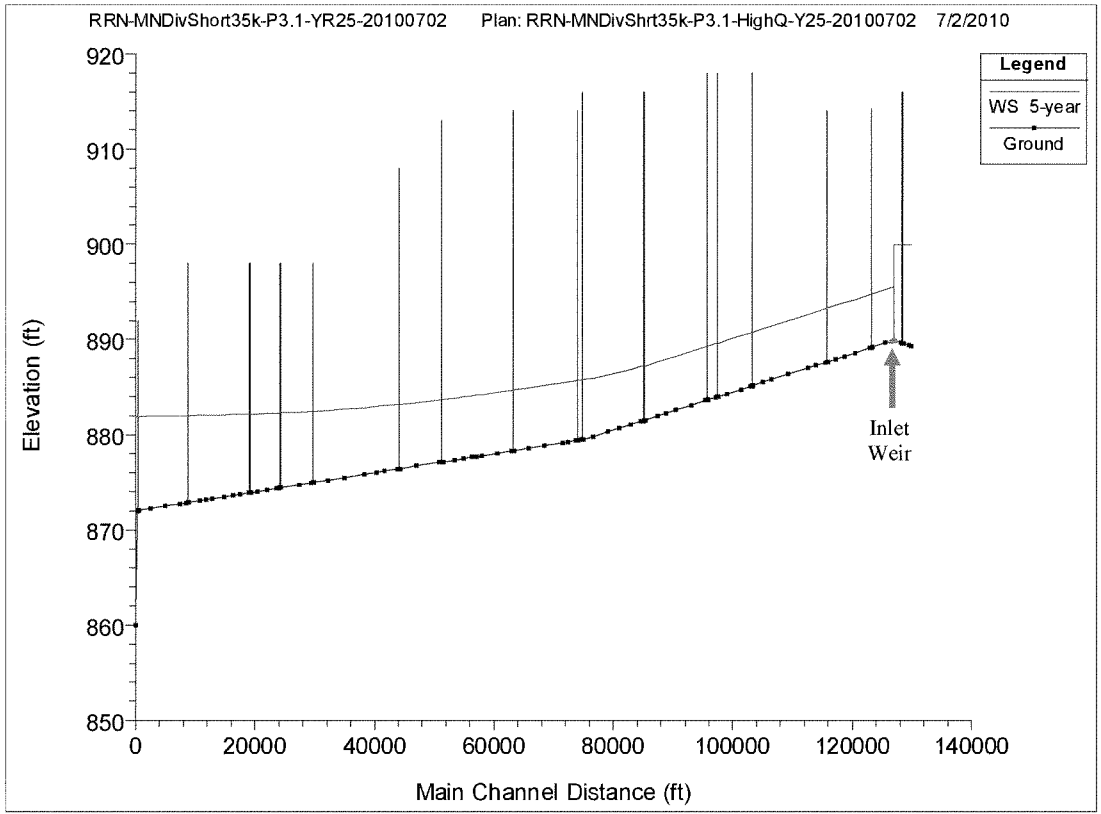


Figure F-D68 FCP Diversion Channel Water Surface Elevation Profile for the 5-Year Event for Year 25 Hydrology

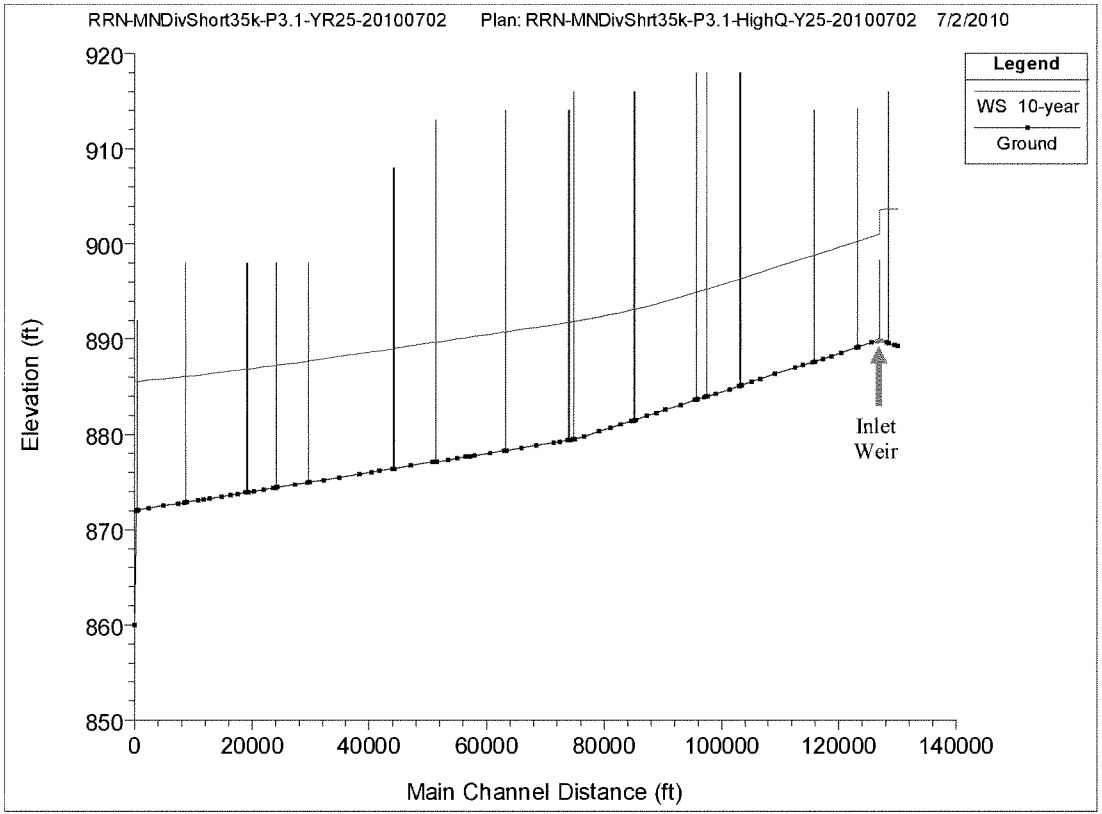


Figure F-D69 FCP Diversion Channel Water Surface Elevation Profile for the 10-Year Event for Year 25 Hydrology

688

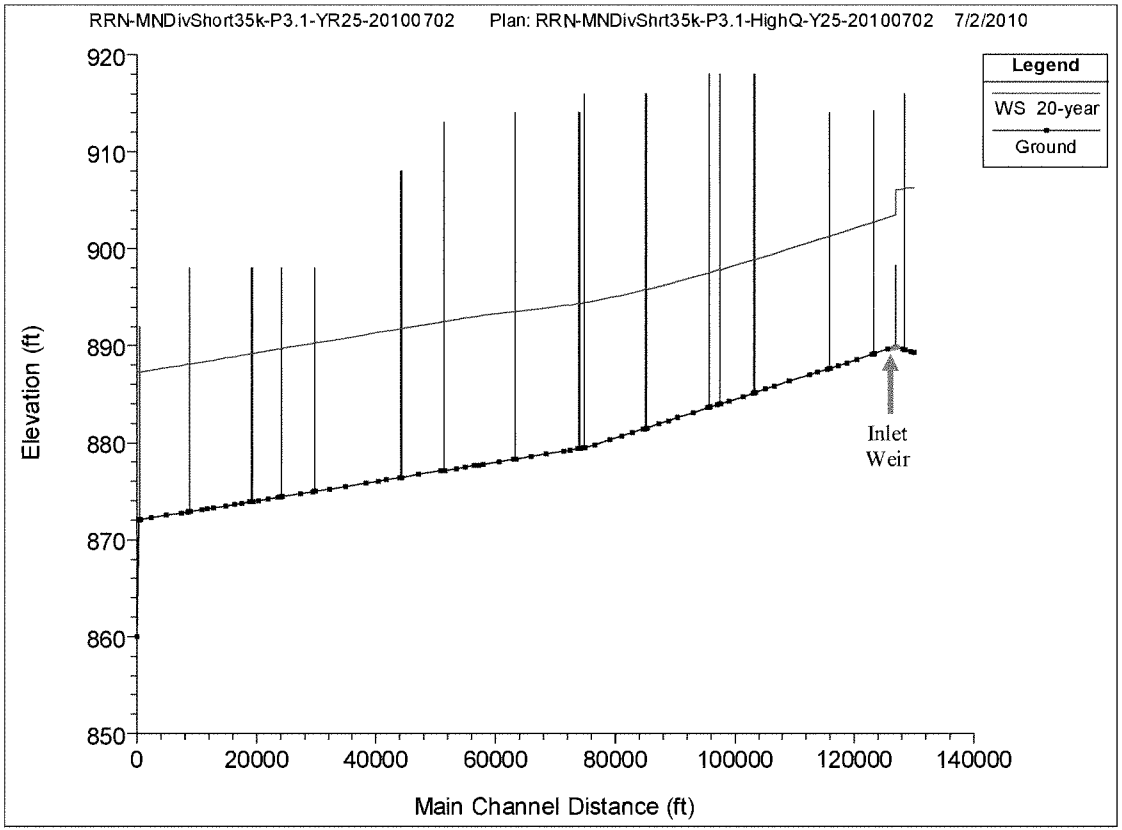


Figure F-D70 FCP Diversion Channel Water Surface Elevation Profile for the 20-Year Event for Year 25 Hydrology

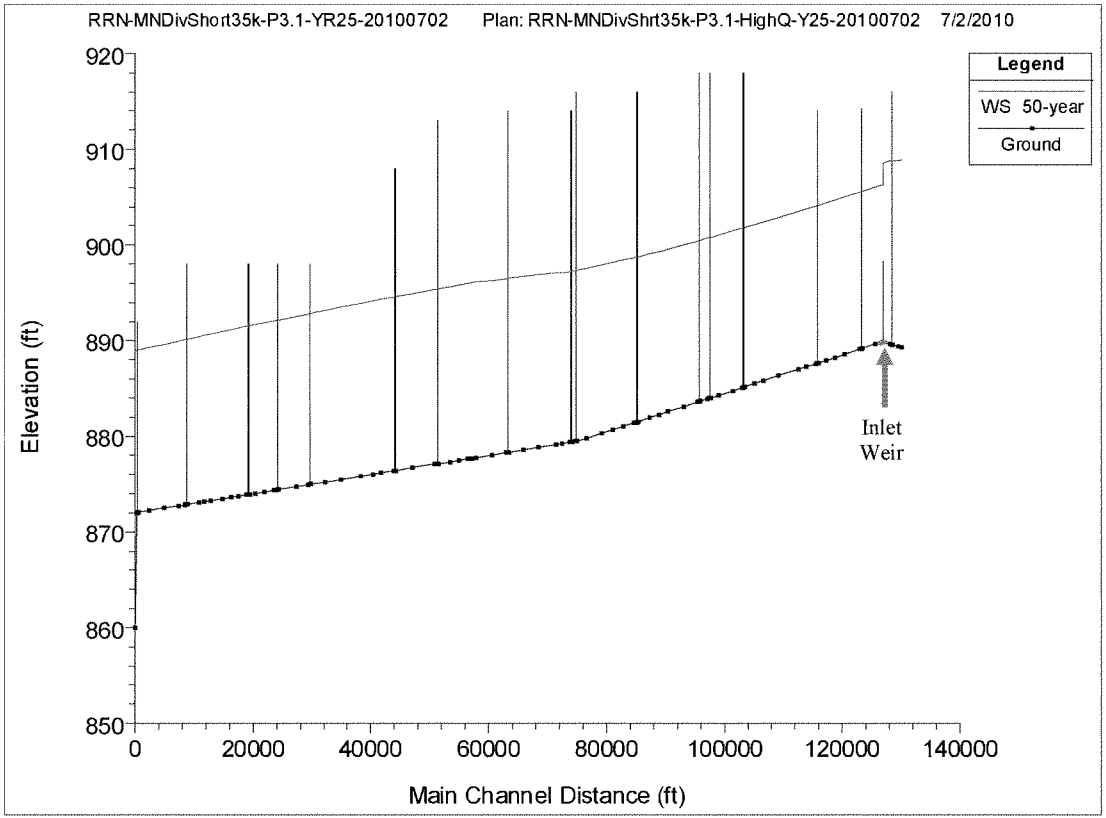


Figure F-D71 FCP Diversion Channel Water Surface Elevation Profile for the 50-Year Event for Year 25 Hydrology

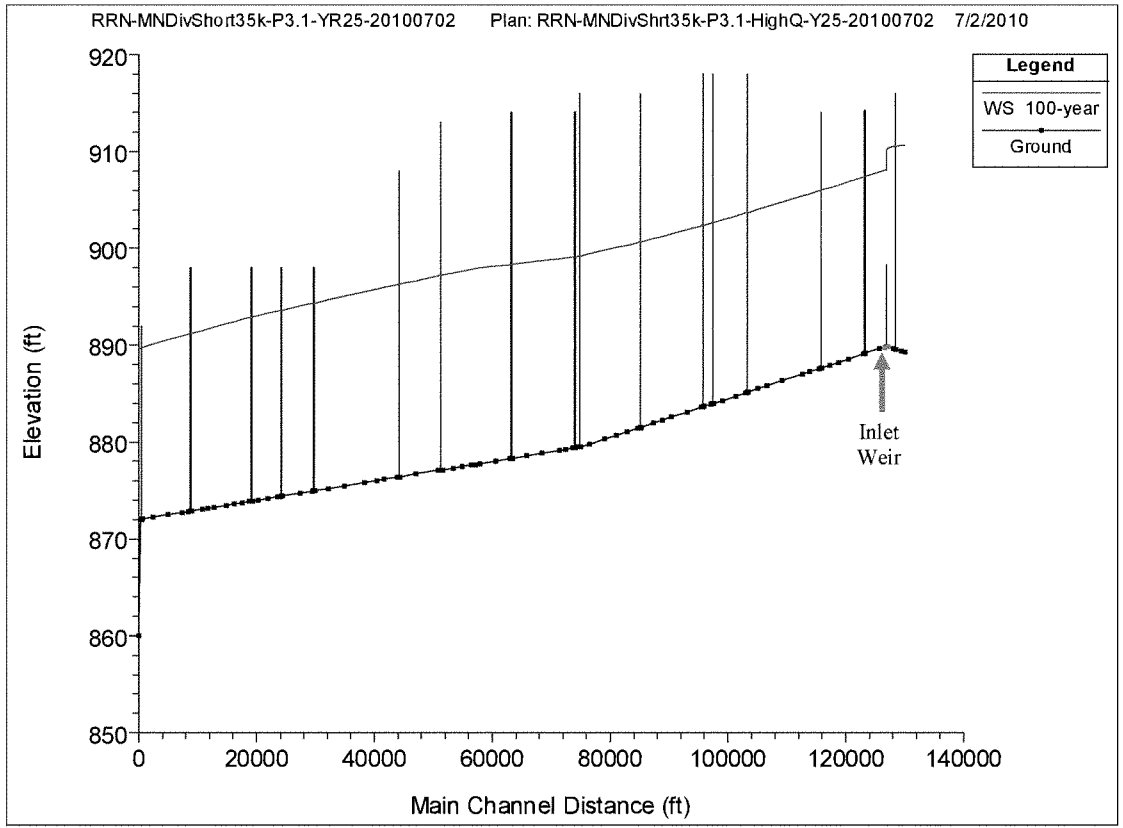


Figure F-D72 FCP Diversion Channel Water Surface Elevation Profile for the 100-Year Event for Year 25 Hydrology

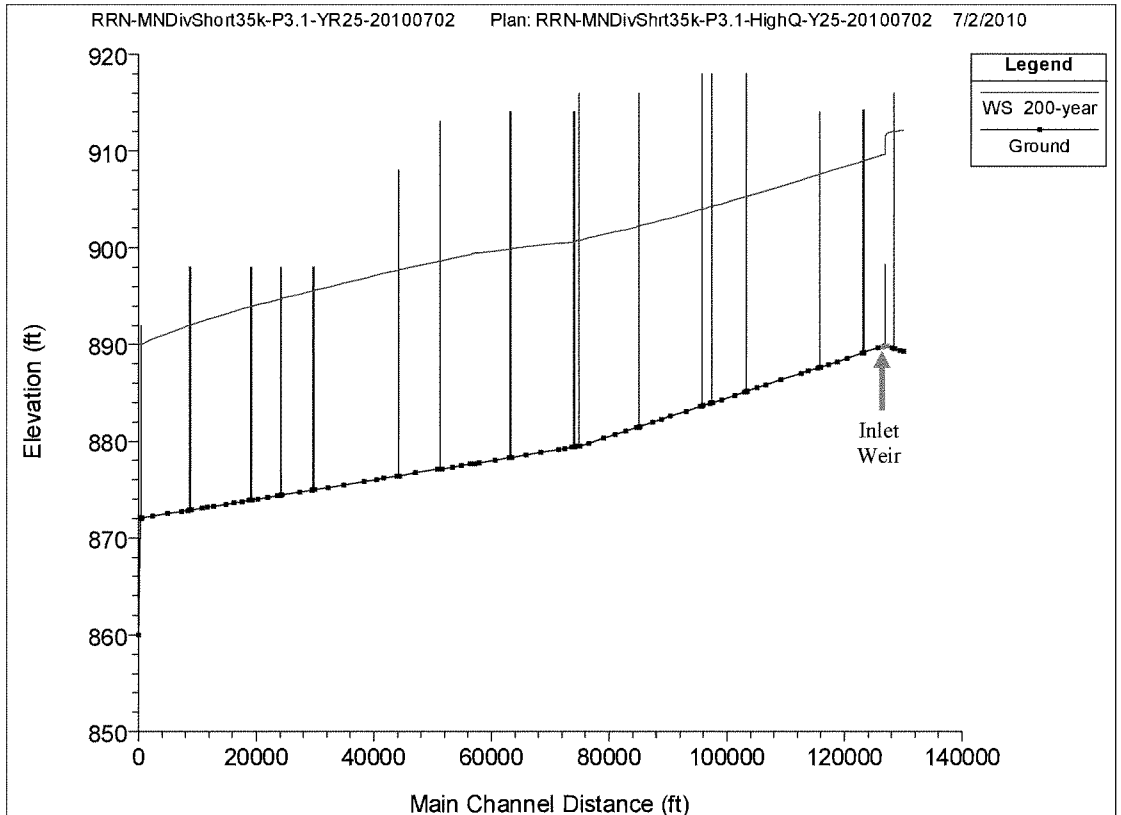


Figure F-D73 FCP Diversion Channel Water Surface Elevation Profile for the 200-Year Event for Year 25 Hydrology

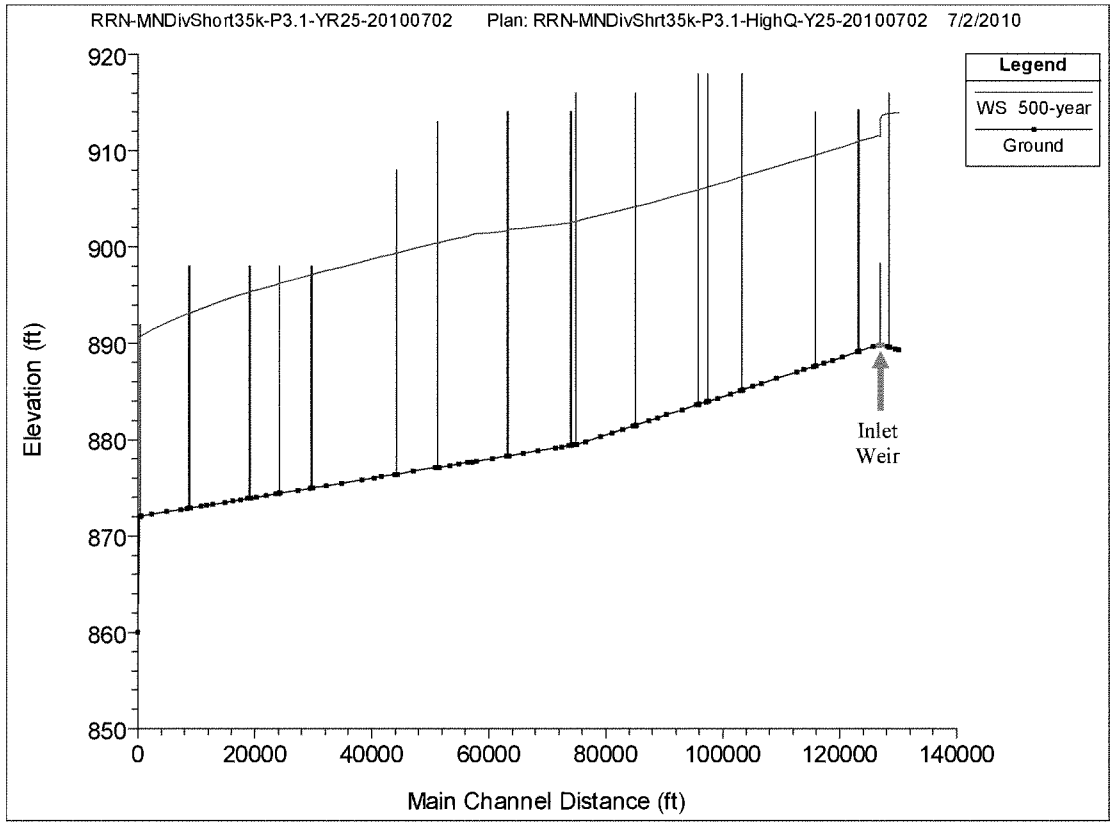


Figure F-D74 FCP Diversion Channel Water Surface Elevation Profile for the 500-Year Event for Year 25 Hydrology

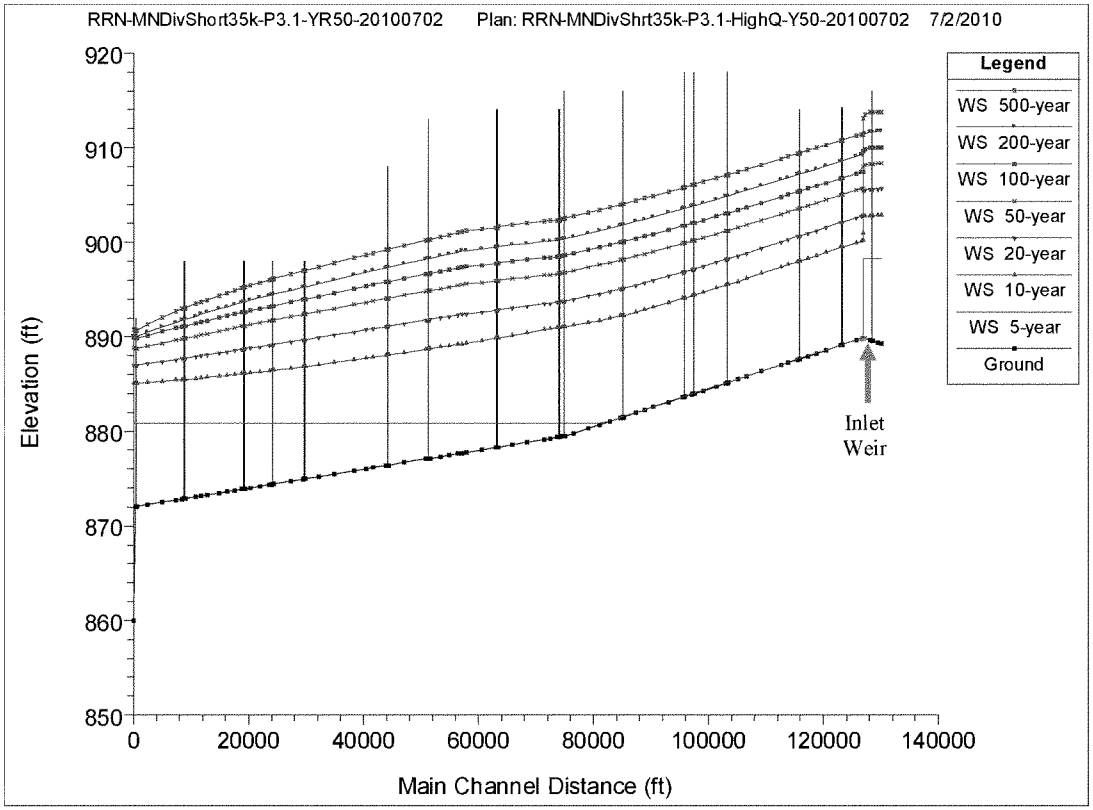


Figure F-D75 FCP Diversion Channel Water Surface Elevation Profile for All Modeled Events for Year 50 Hydrology

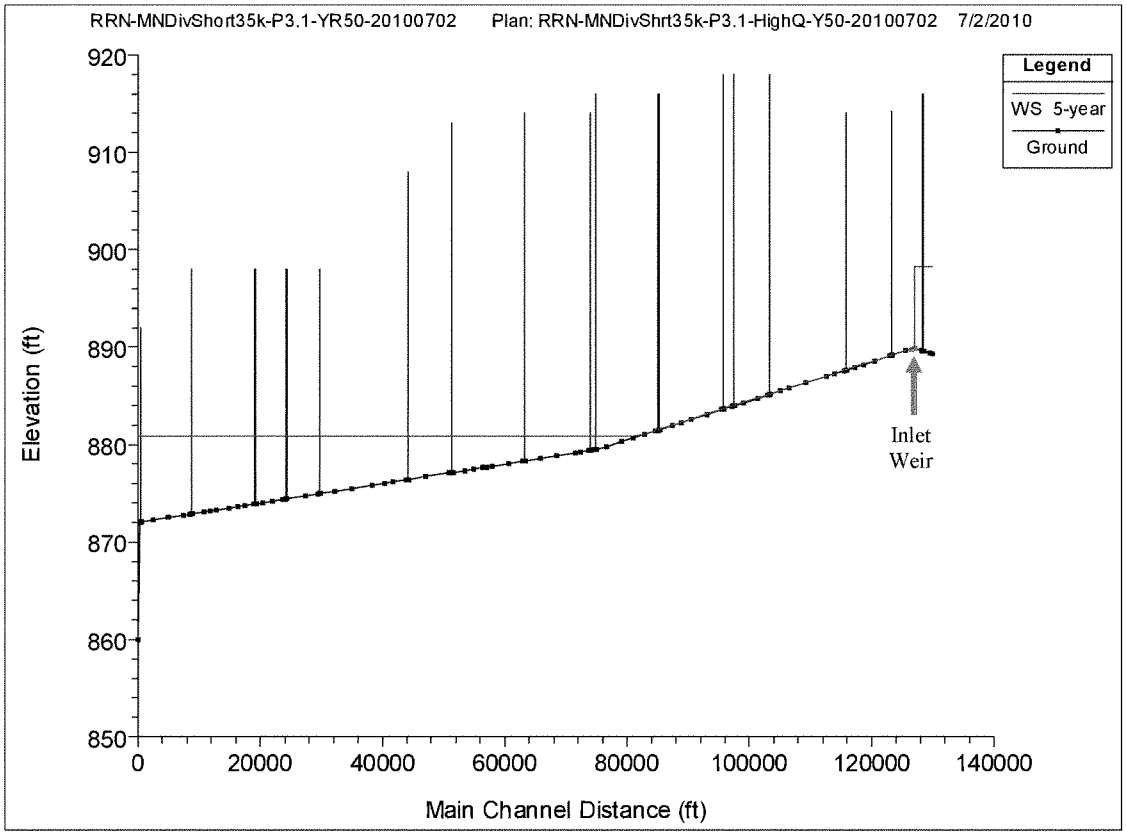


Figure F-D76 FCP Diversion Channel Water Surface Elevation Profile for the 5-Year Event for Year 50 Hydrology

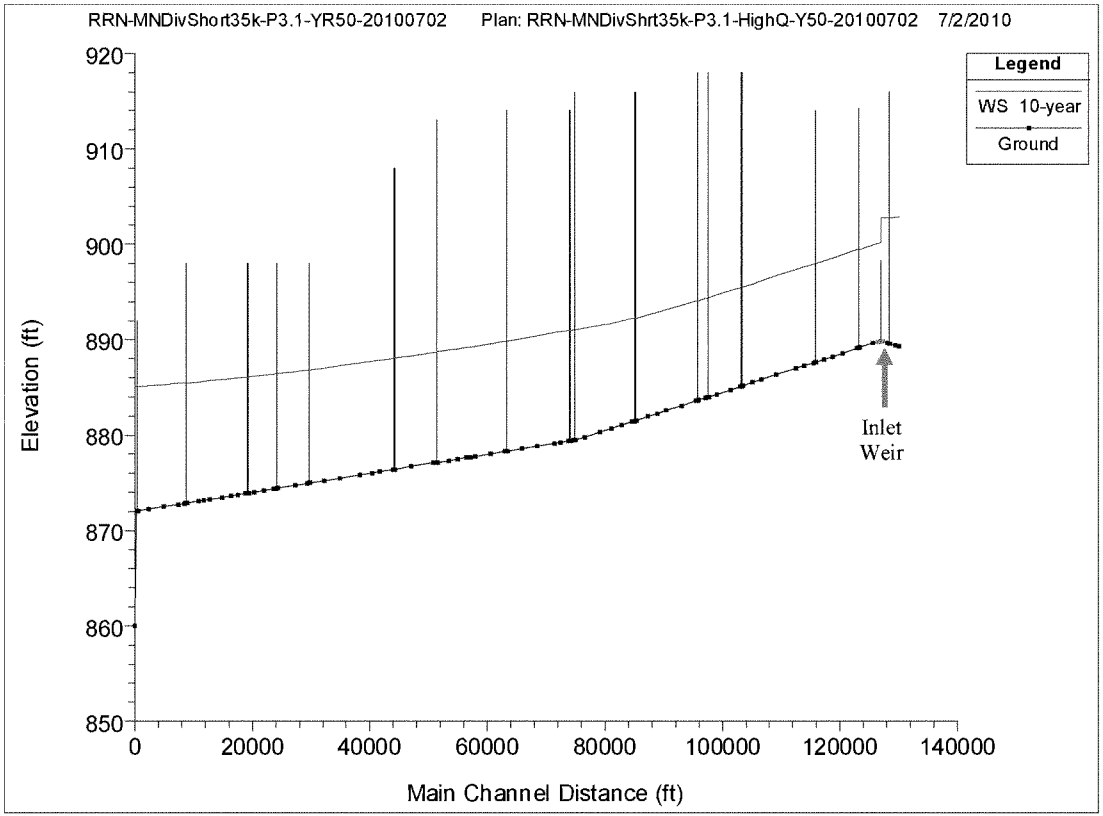


Figure F-D77 FCP Diversion Channel Water Surface Elevation Profile for the 10-Year Event for Year 50 Hydrology

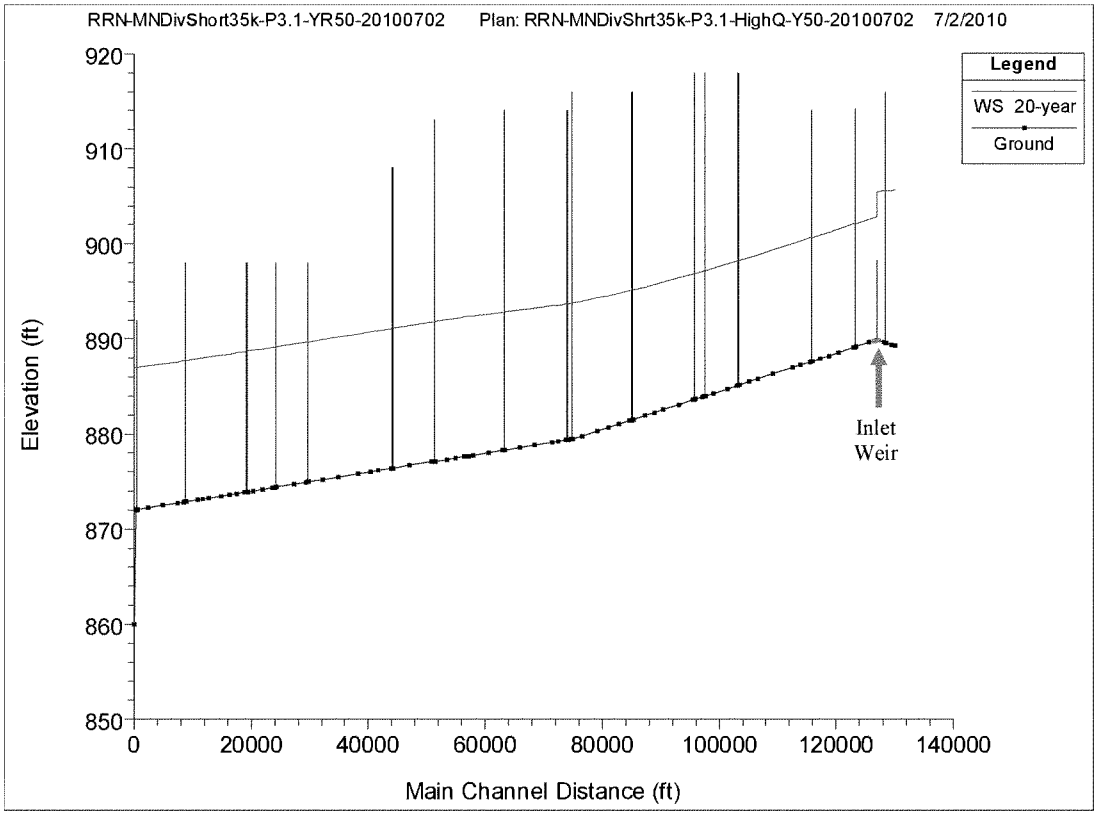


Figure F-D78 FCP Diversion Channel Water Surface Elevation Profile for the 20-Year Event for Year 50 Hydrology

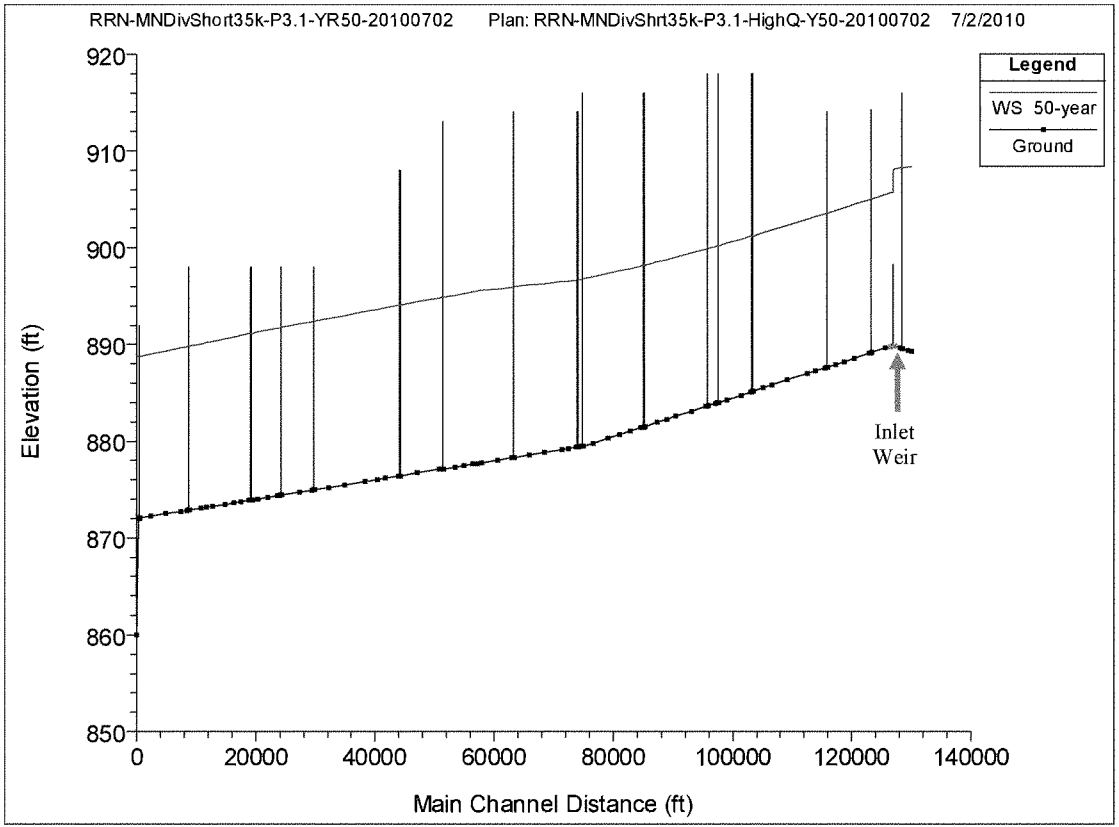


Figure F-D79 FCP Diversion Channel Water Surface Elevation Profile for the 50-Year Event for Year 50 Hydrology

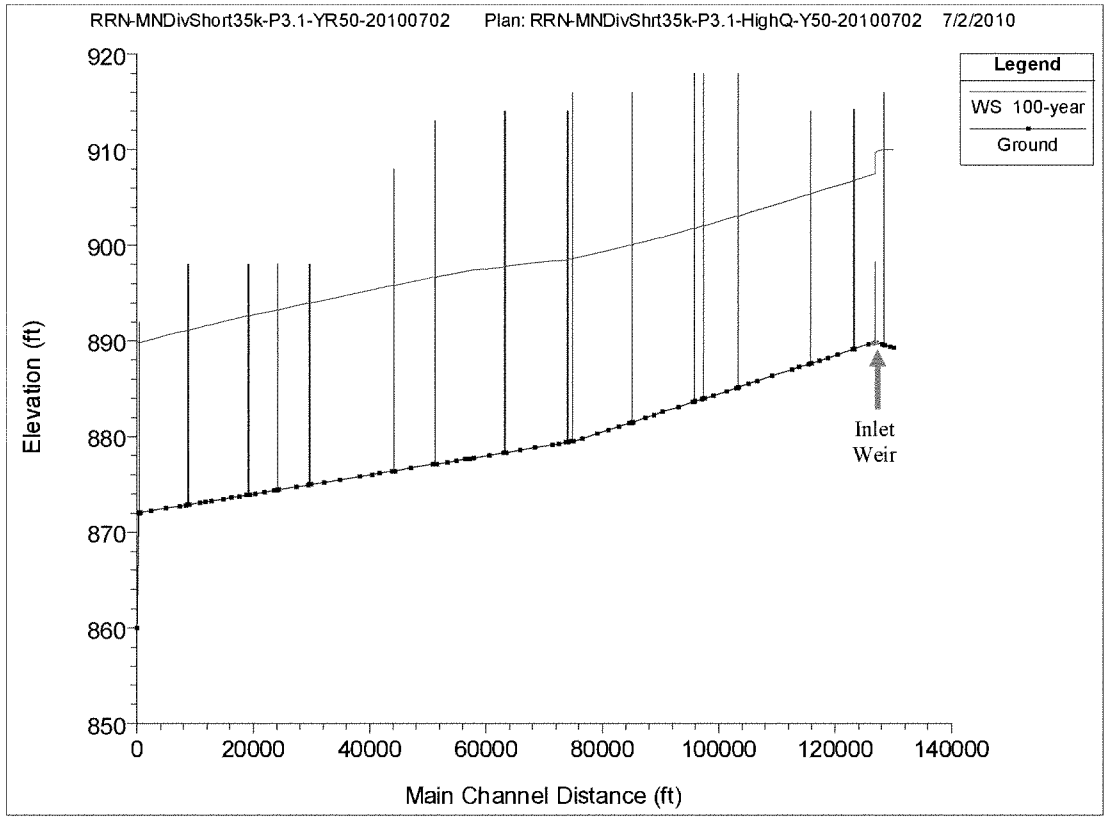


Figure F-D80 FCP Diversion Channel Water Surface Elevation Profile for the 100-Year Event for Year 50 Hydrology

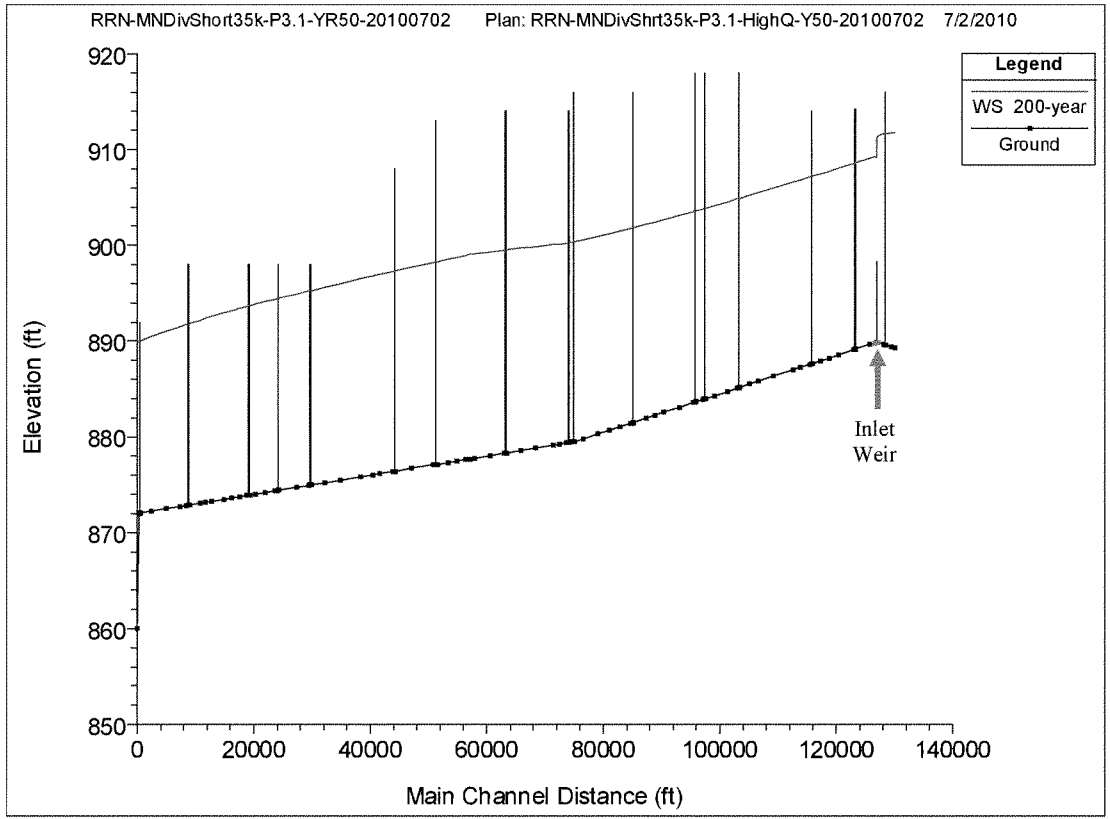


Figure F-D81 FCP Diversion Channel Water Surface Elevation Profile for the 200-Year Event for Year 50 Hydrology

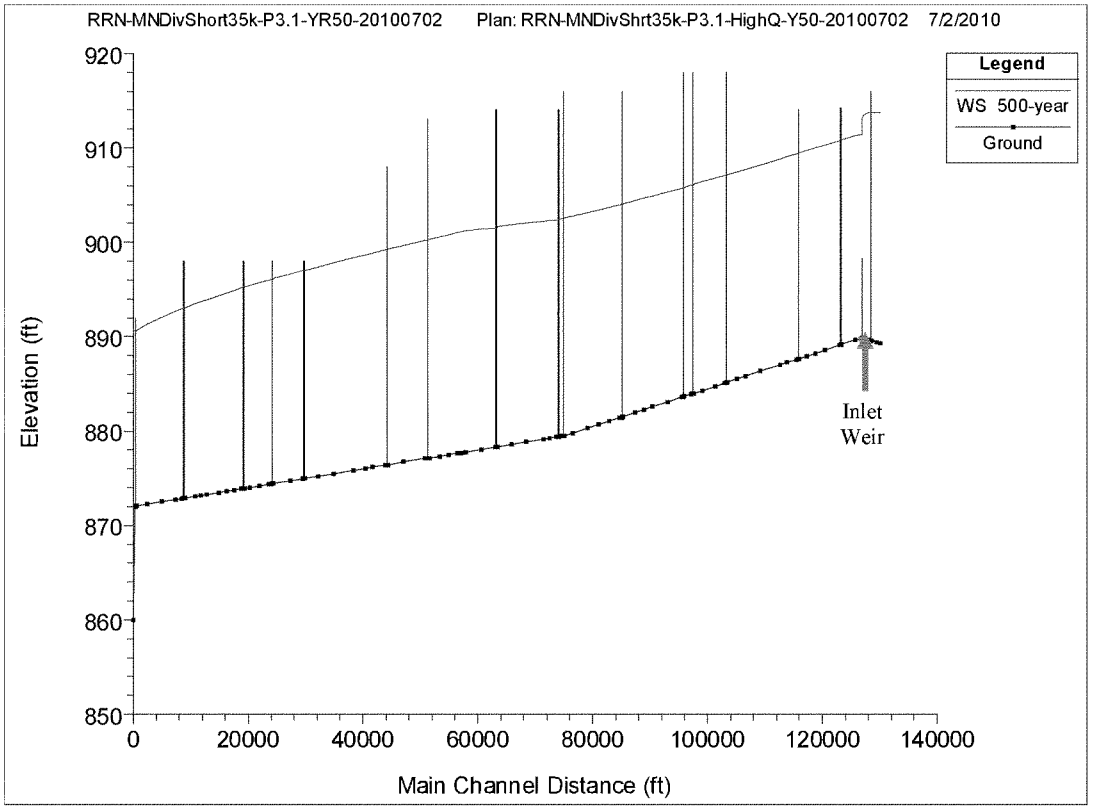


Figure F-D82 FCP Diversion Channel Water Surface Elevation Profile for the 500-Year Event for Year 50 Hydrology

F-D3.2 WATER SURFACE ELEVATION PROFILES ALONG THE LPP DIVERSION CHANNEL

The maximum water surface elevation profiles along the LPP Diversion Channel for the 10-, 50-, 100-, and 500-year events are presented in Figures F-D83, F-D88, F-D93, and F-D98. The location of the Inlet Weir (primary Inlet Structure to the Diversion Channel located to the west of the Wild Rice River), Sheyenne River crossing structure, Maple River crossing structure, and Outlet structure are noted on the figures. Figures F-D84 to F-D87, F-D89 to F-D92, F-D94 to F-D97, and F-D99 to F-D102 include stage and flow hydrographs over the duration of the 10, 50-, 100-, and 500-year events in four locations in the Diversion Channel listed below.

1. Upstream of the Inlet Weir (RS 152527)
2. Between the Sheyenne River and Maple River (RS 83654)
3. Between the Maple River and the Lower Rush River (RS 64696)
4. Between the Rush River and the Diversion Outlet (RS 29253)

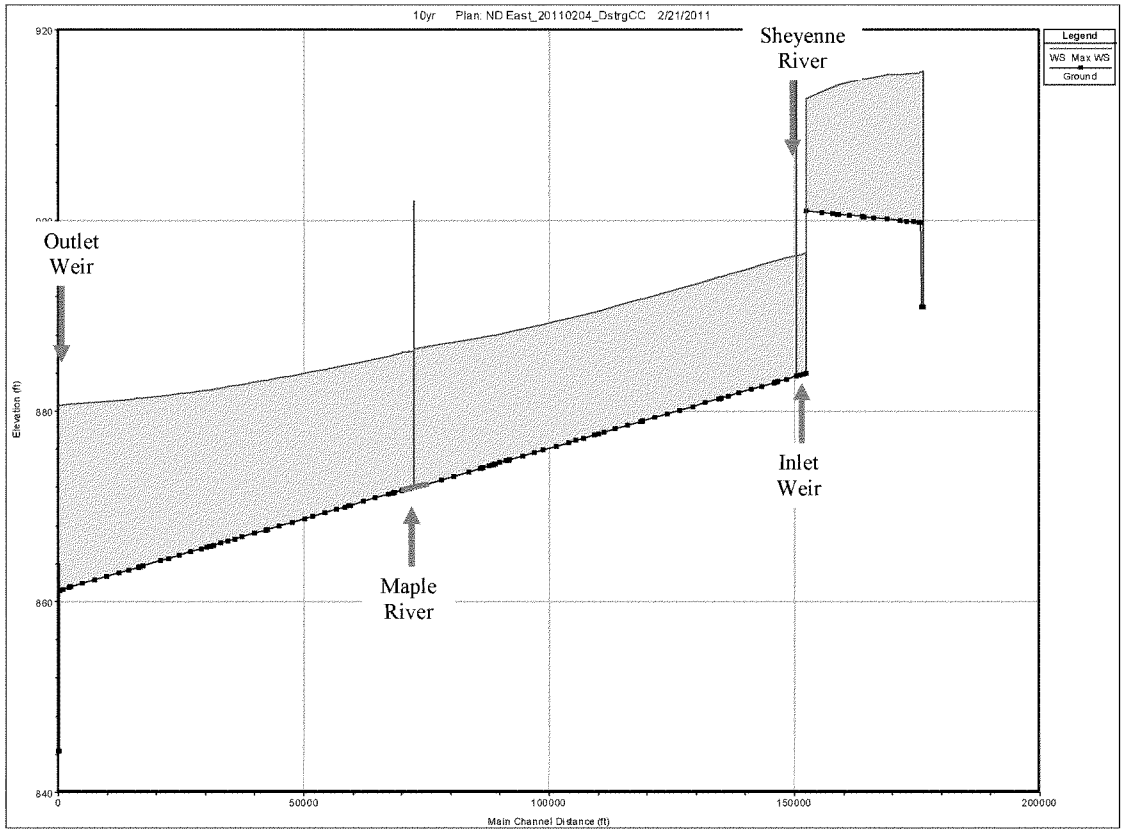


Figure F-D83 LPP Diversion Channel Water Surface Elevation Profile for the 10-Year Event

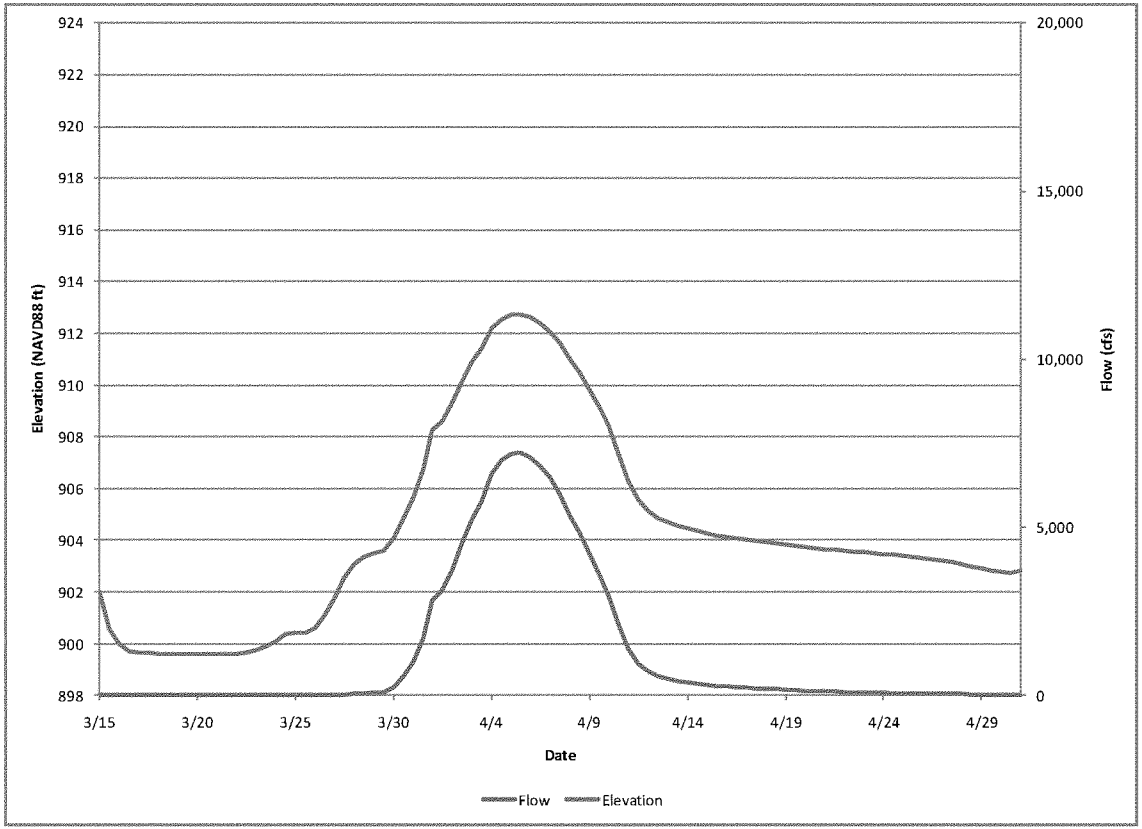


Figure F-D84 Hydrograph Upstream of the Diversion Inlet Structure (RS 152527) for the 10-Year Event

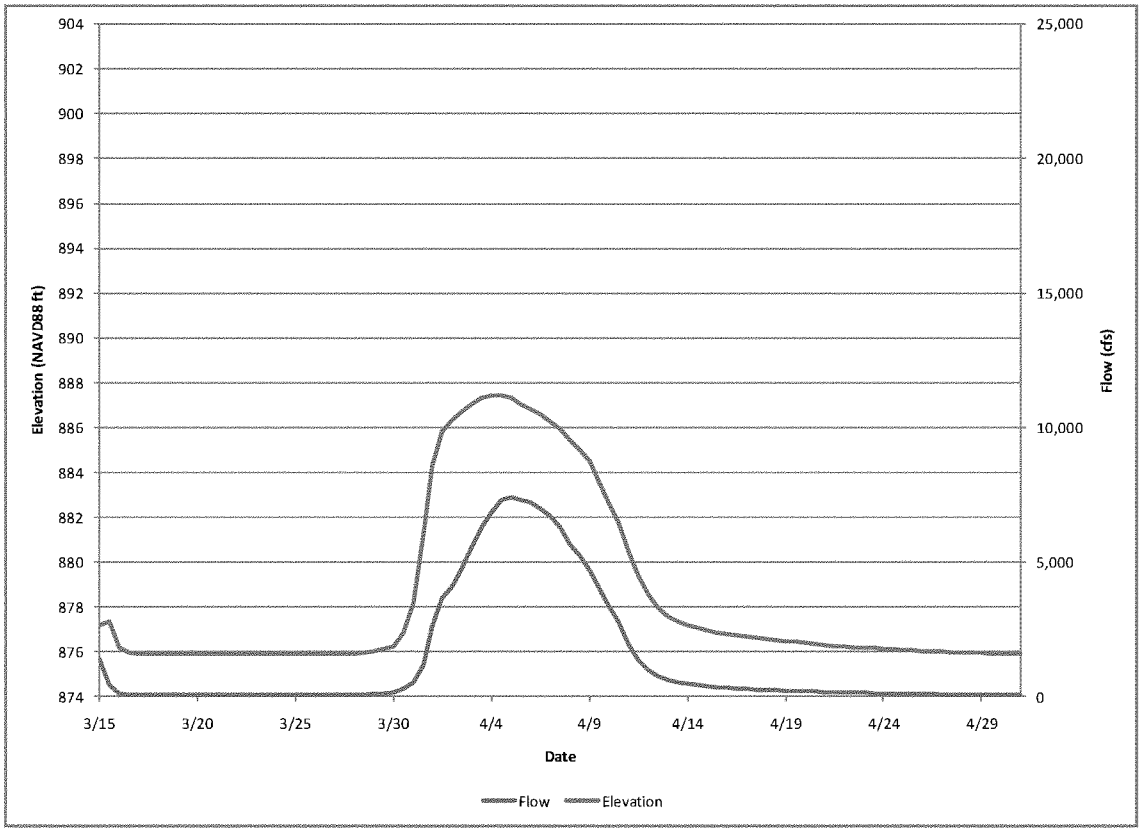


Figure F-D85 Hydrograph between the Sheyenne River and Maple River (RS 83654) for the 10-Year Event

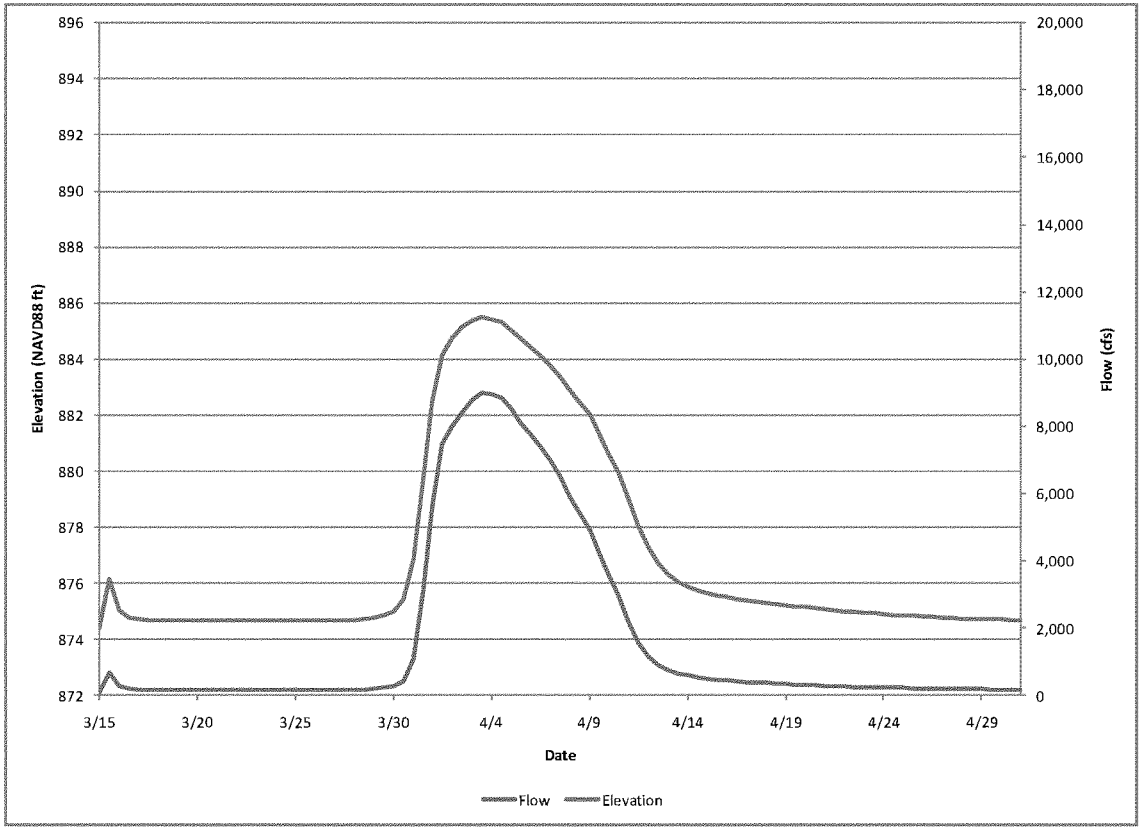


Figure F-D86 Hydrograph between the Maple River and Lower Rush River (RS 64696) for the 10-Year Event

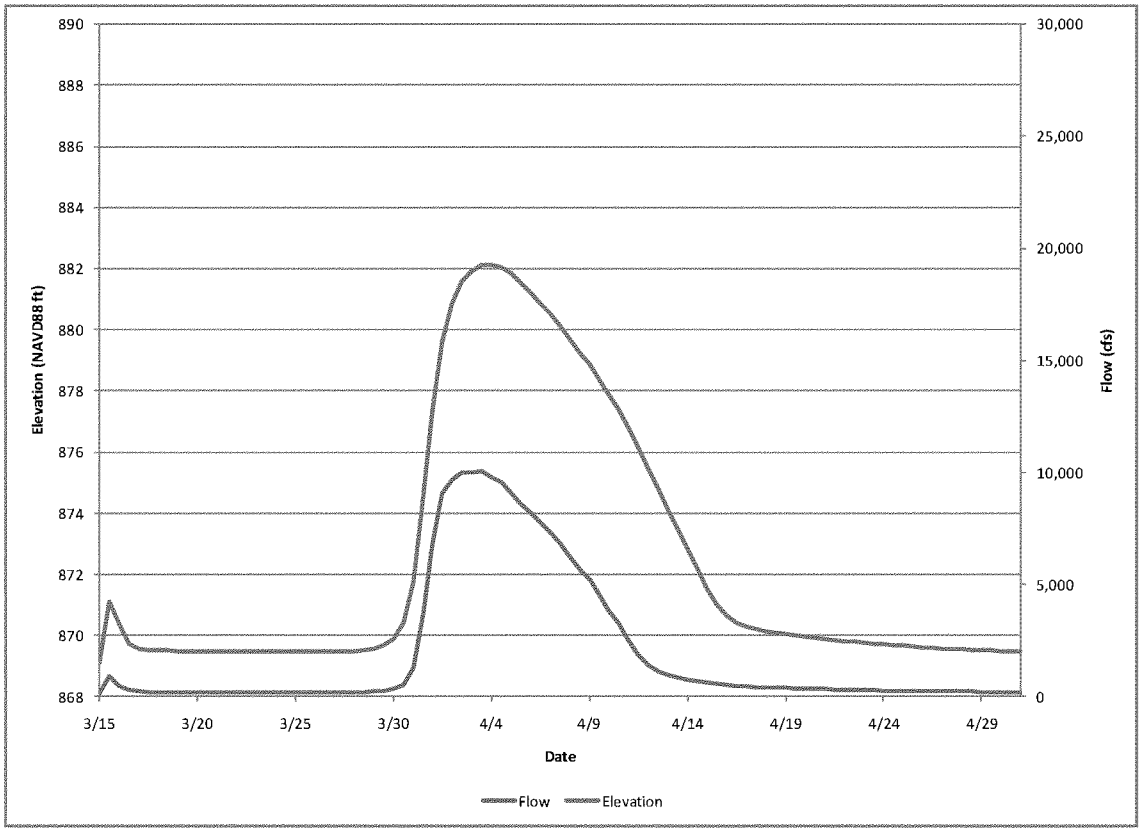


Figure F-D87 Hydrograph between the Rush River and the Outlet Structure (RS 29253) for the 10-Year Event

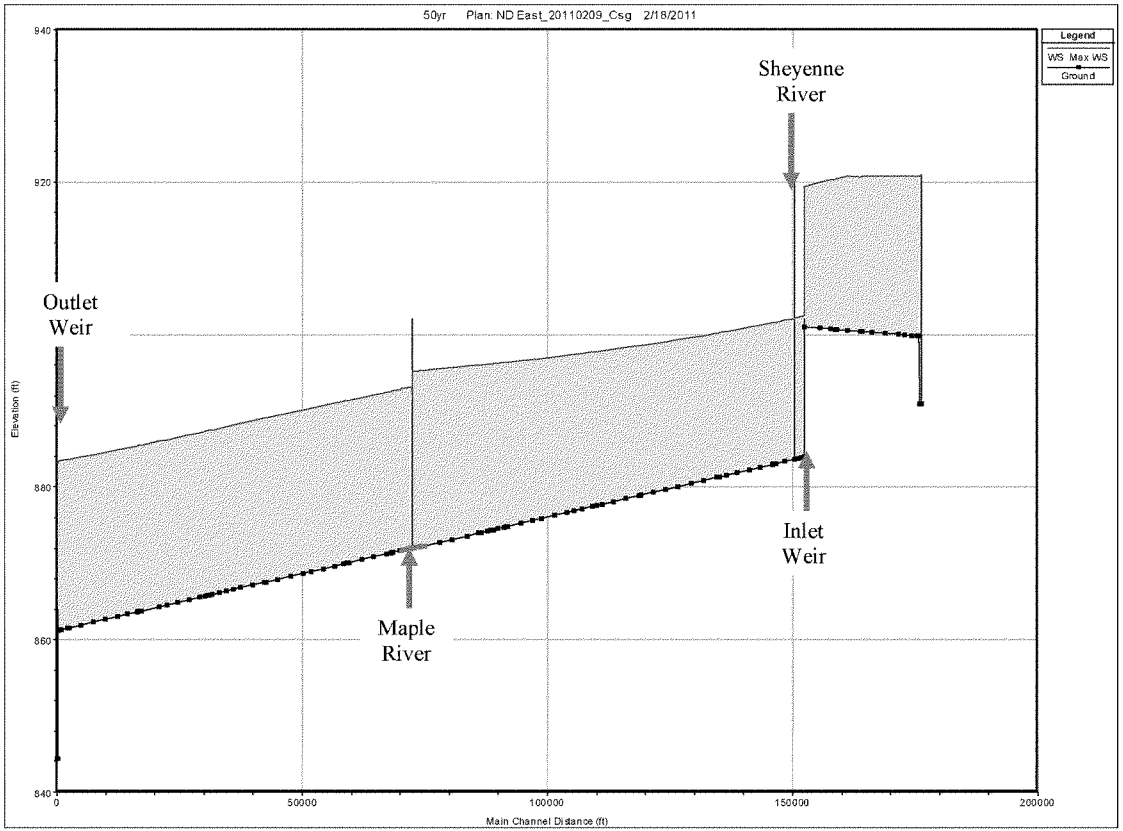


Figure F-D88 LPP Diversion Channel Water Surface Elevation Profile for the 50-Year Event

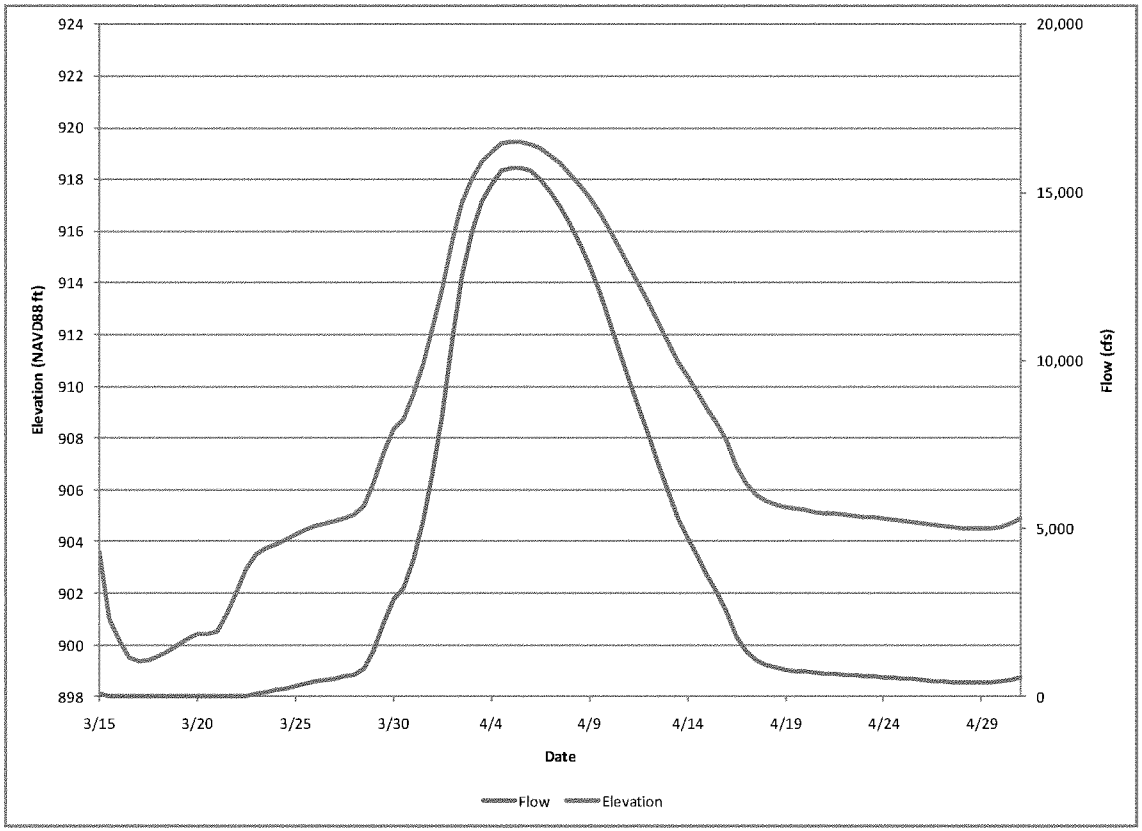


Figure F-D89 Hydrograph Upstream of the Diversion Inlet Structure (RS 152527) for the 50-Year Event

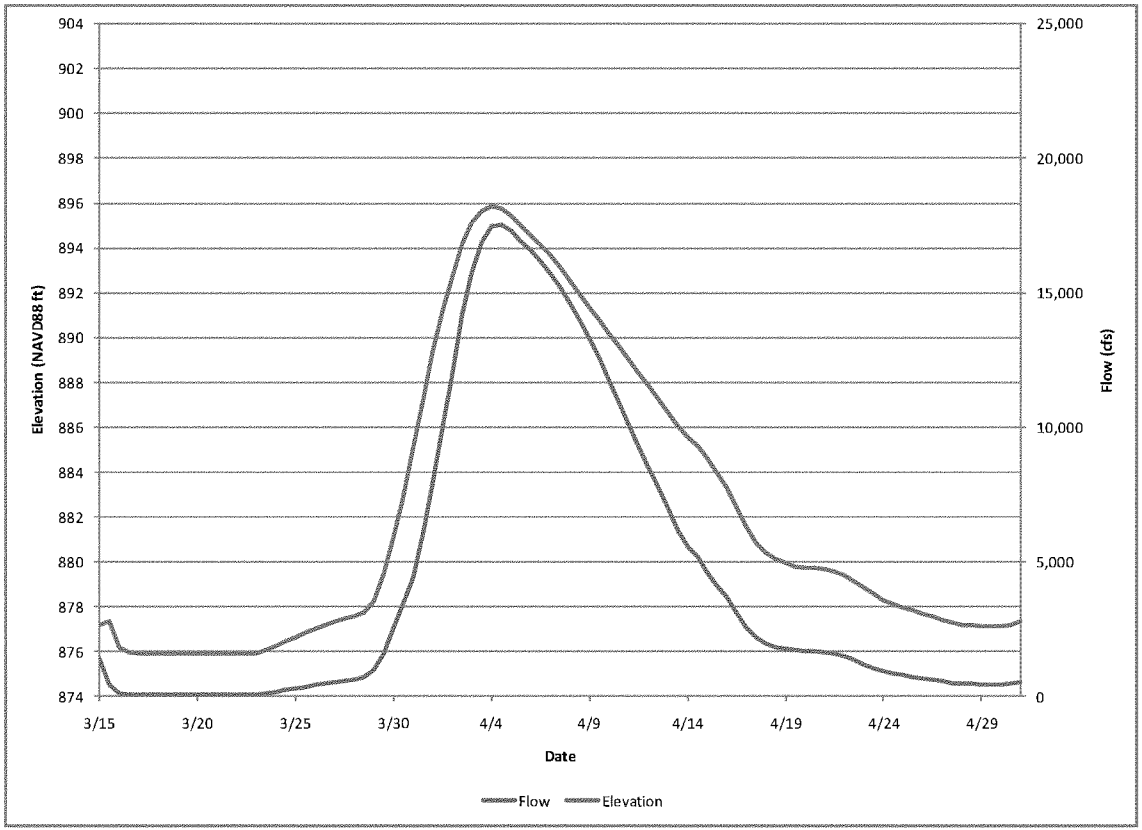


Figure F-D90 Hydrograph between the Sheyenne River and Maple River (RS 83654) for the 50-Year Event

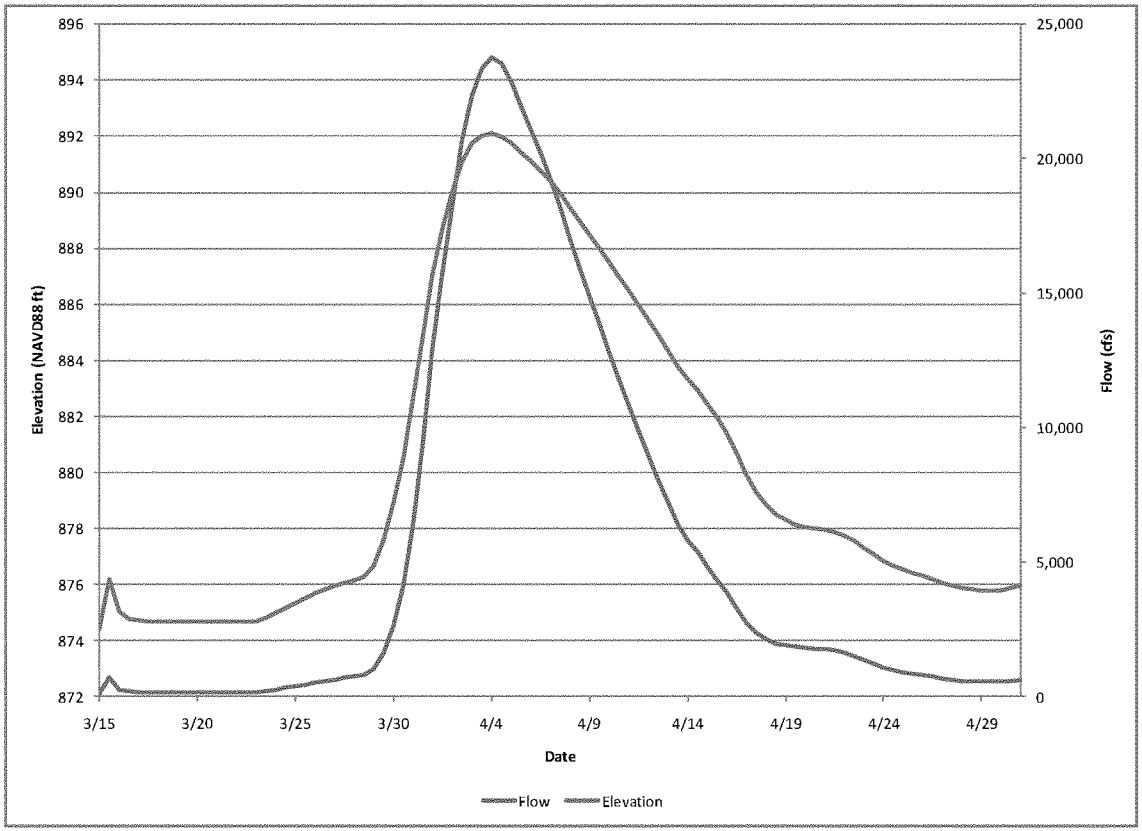


Figure F-D91 Hydrograph between the Maple River and Lower Rush River (RS 64696) for the 50-Year Event

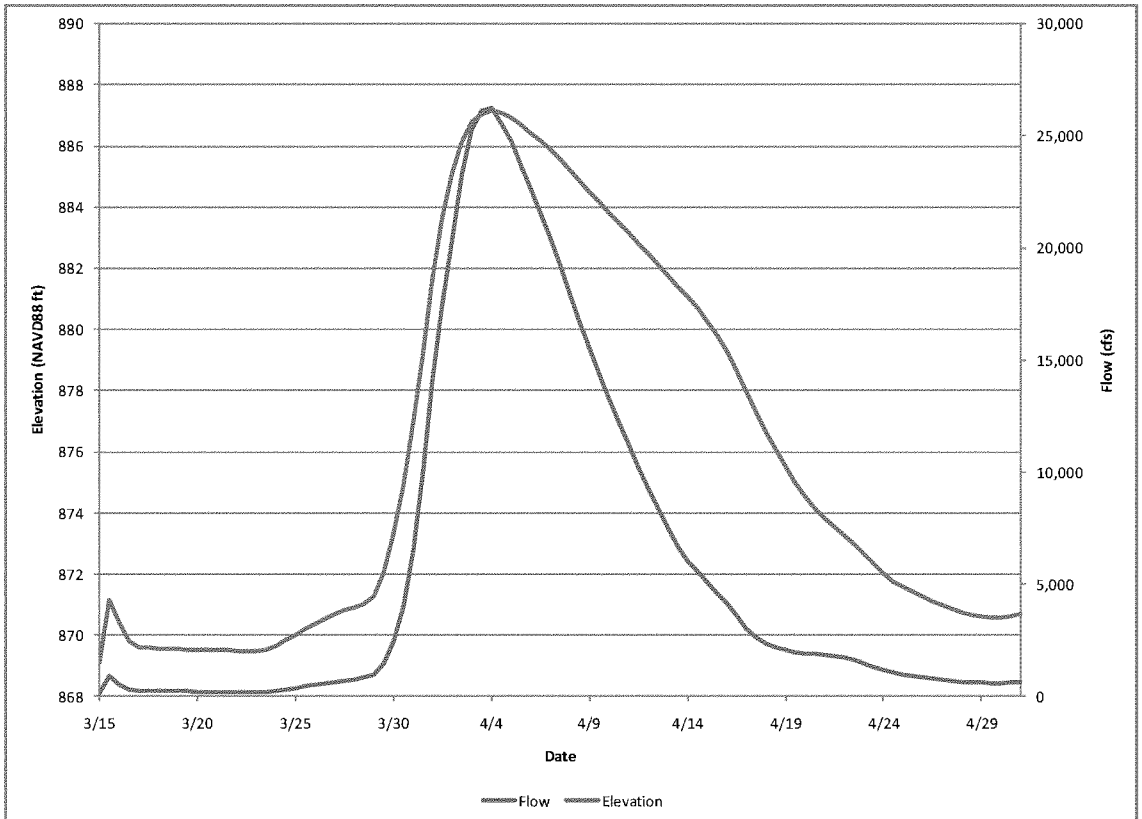


Figure F-D92 Hydrograph between the Rush River and the Outlet Structure (RS 29253) for the 50-Year Event

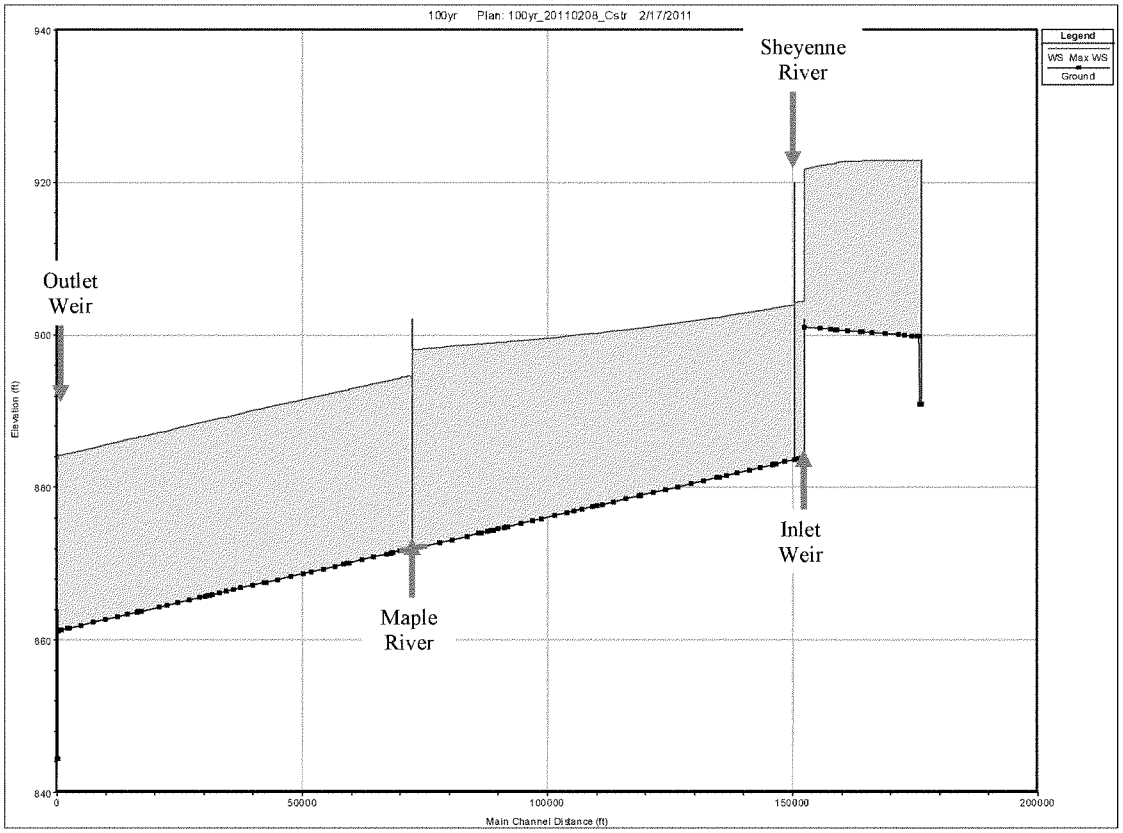


Figure F-D93 LPP Diversion Channel Water Surface Elevation Profile for the 100-Year Event

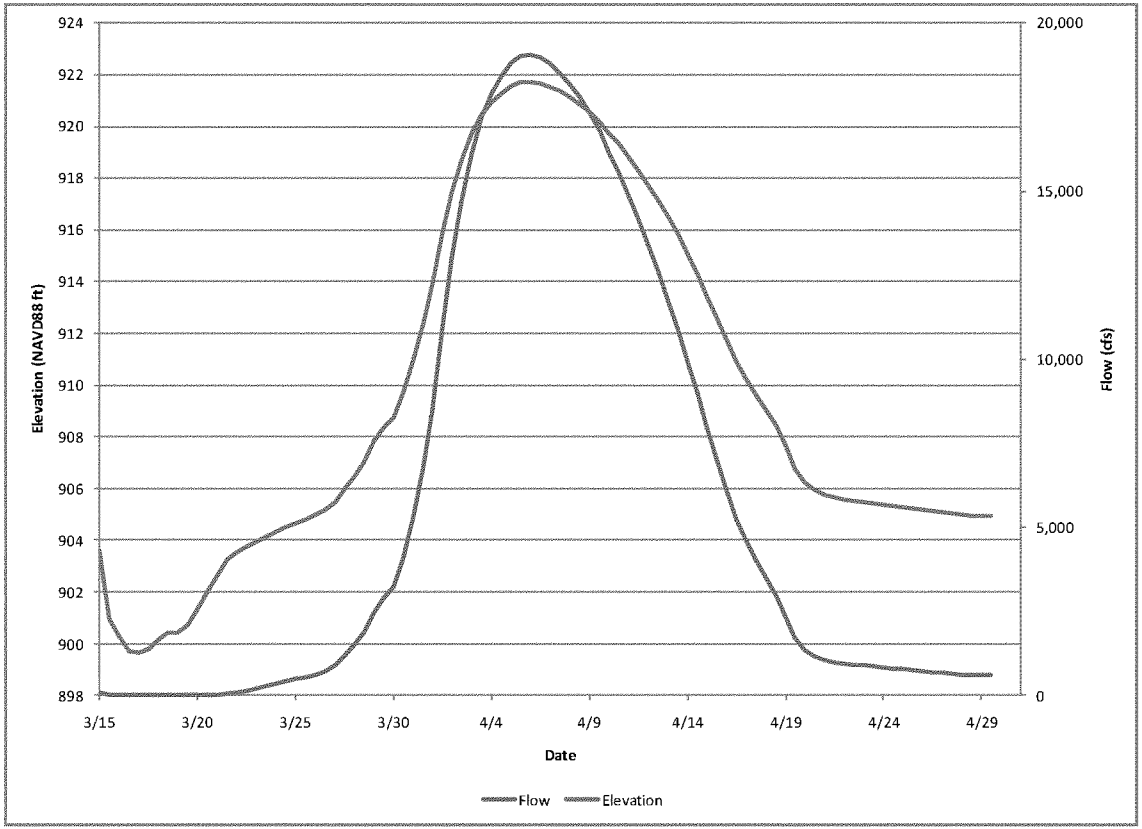


Figure F-D94 Hydrograph Upstream of the Diversion Inlet Structure (RS 152527) for the 100-Year Event

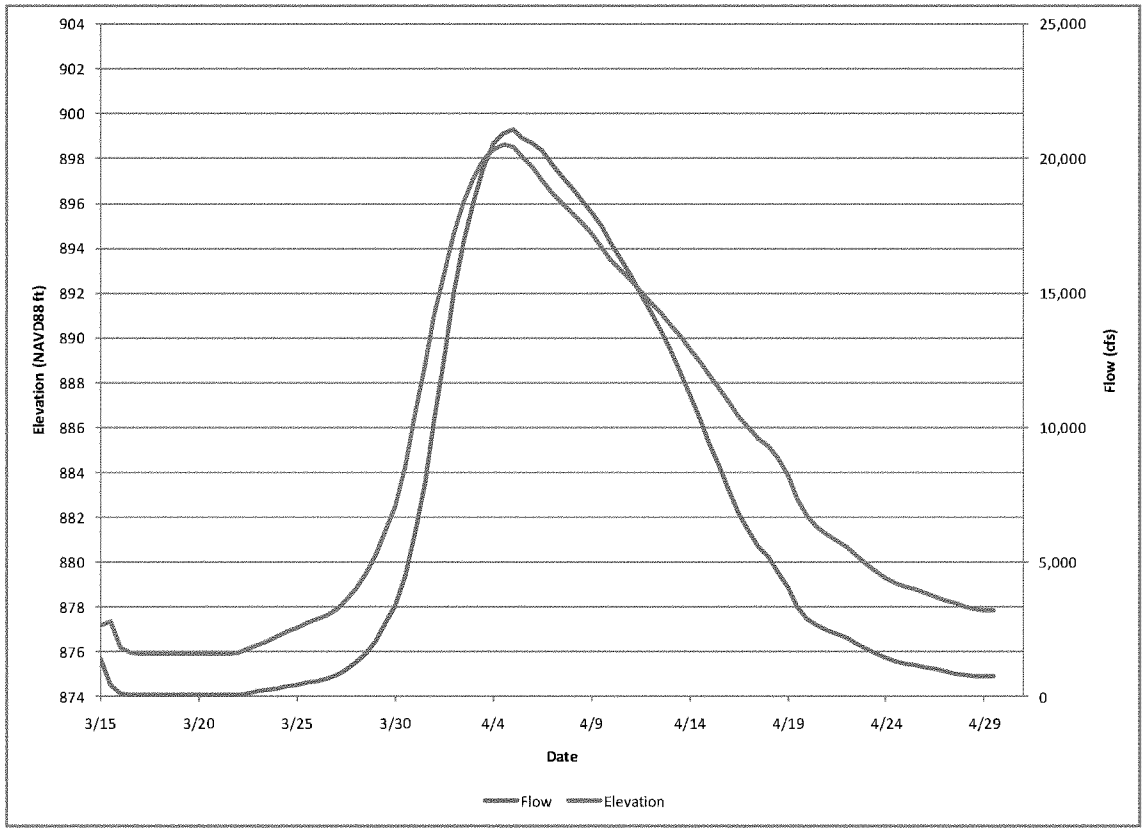


Figure F-D95 Hydrograph between the Sheyenne River and Maple River (RS 83654) for the 100-Year Event

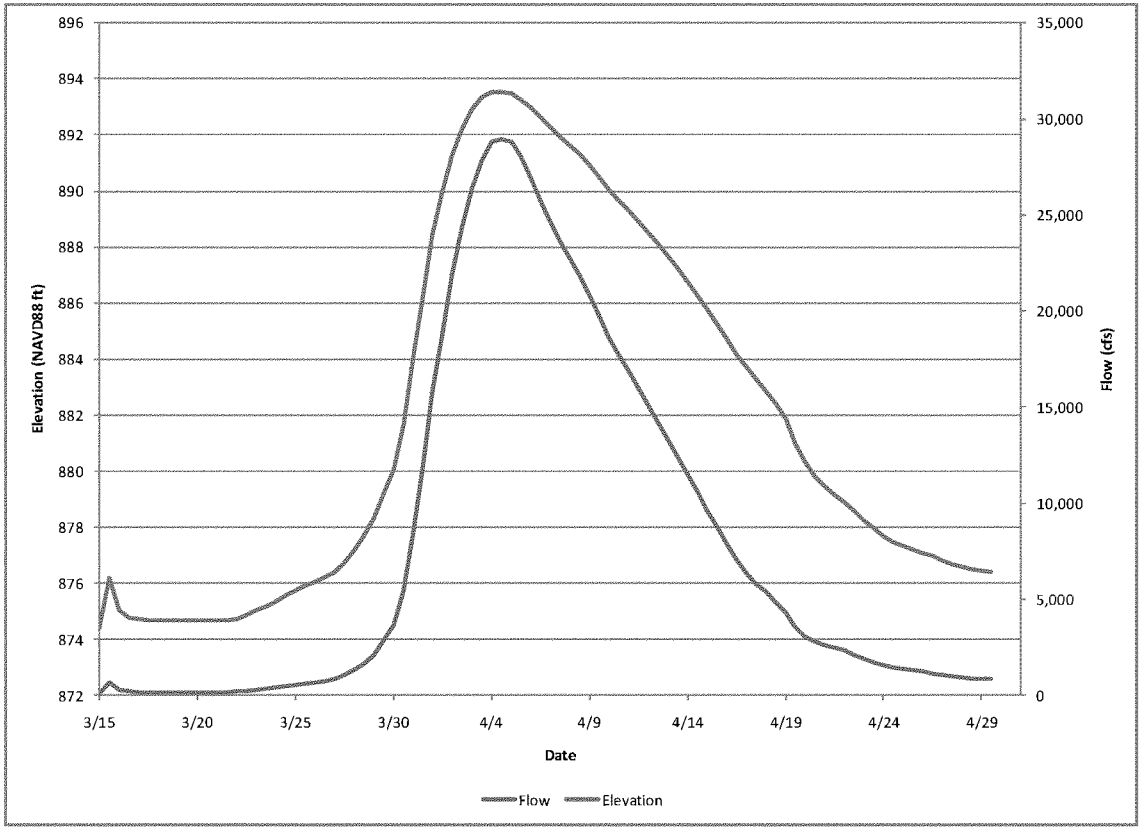


Figure F-D96 Hydrograph between the Maple River and Lower Rush River (RS 64696) for the 100-Year Event

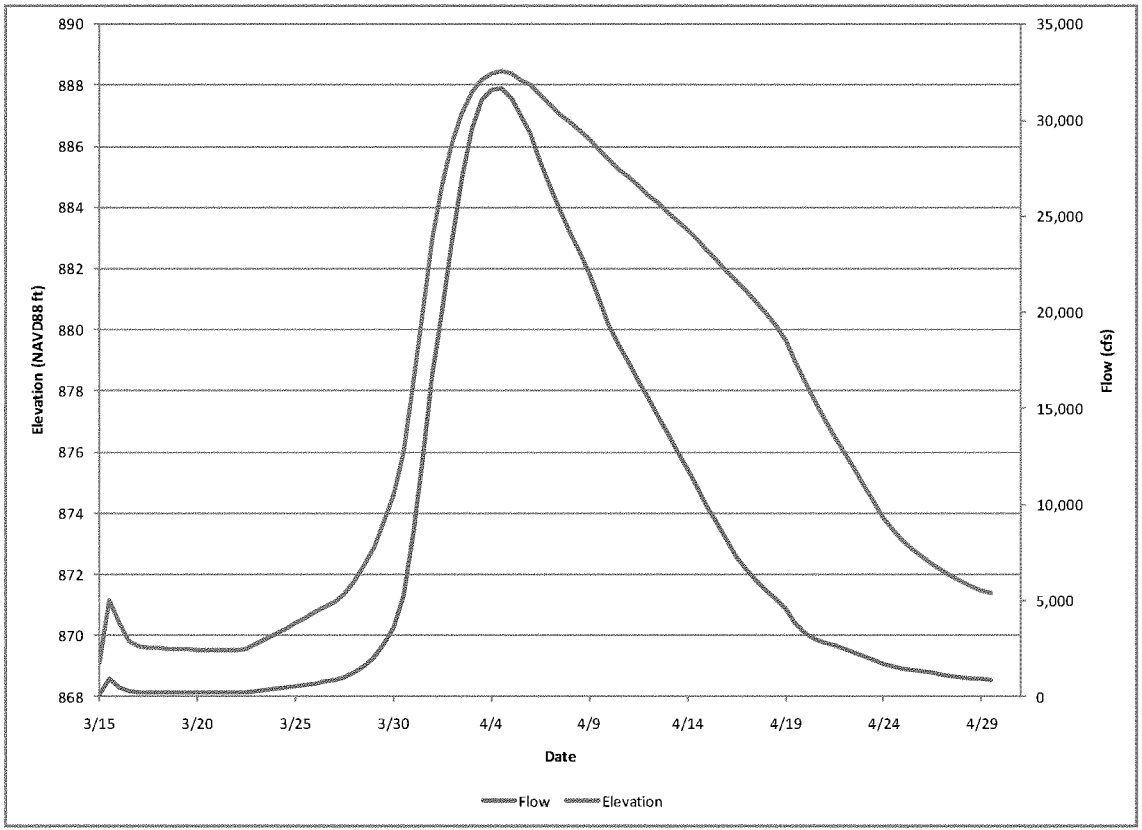


Figure F-D97 Hydrograph between the Rush River and the Outlet Structure (RS 29253) for the 100-Year Event

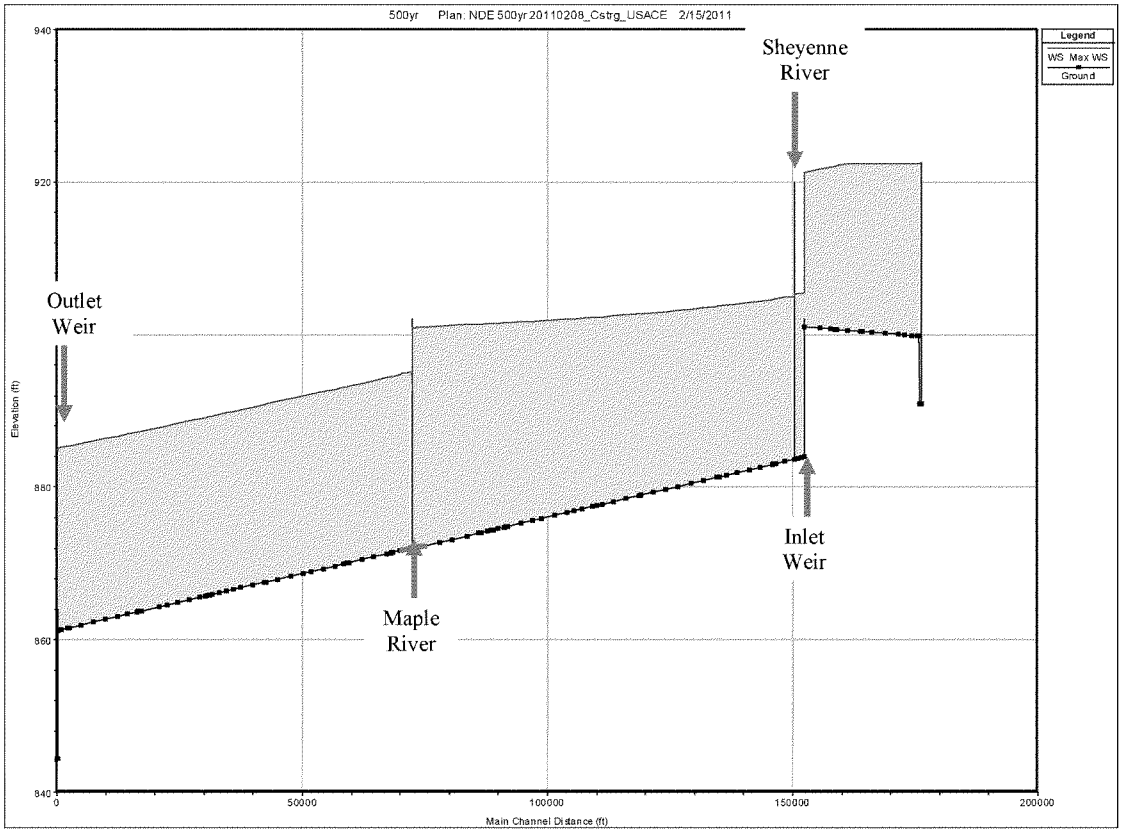


Figure F-D98 LPP Diversion Channel Water Surface Elevation Profile for the 500-Year Event

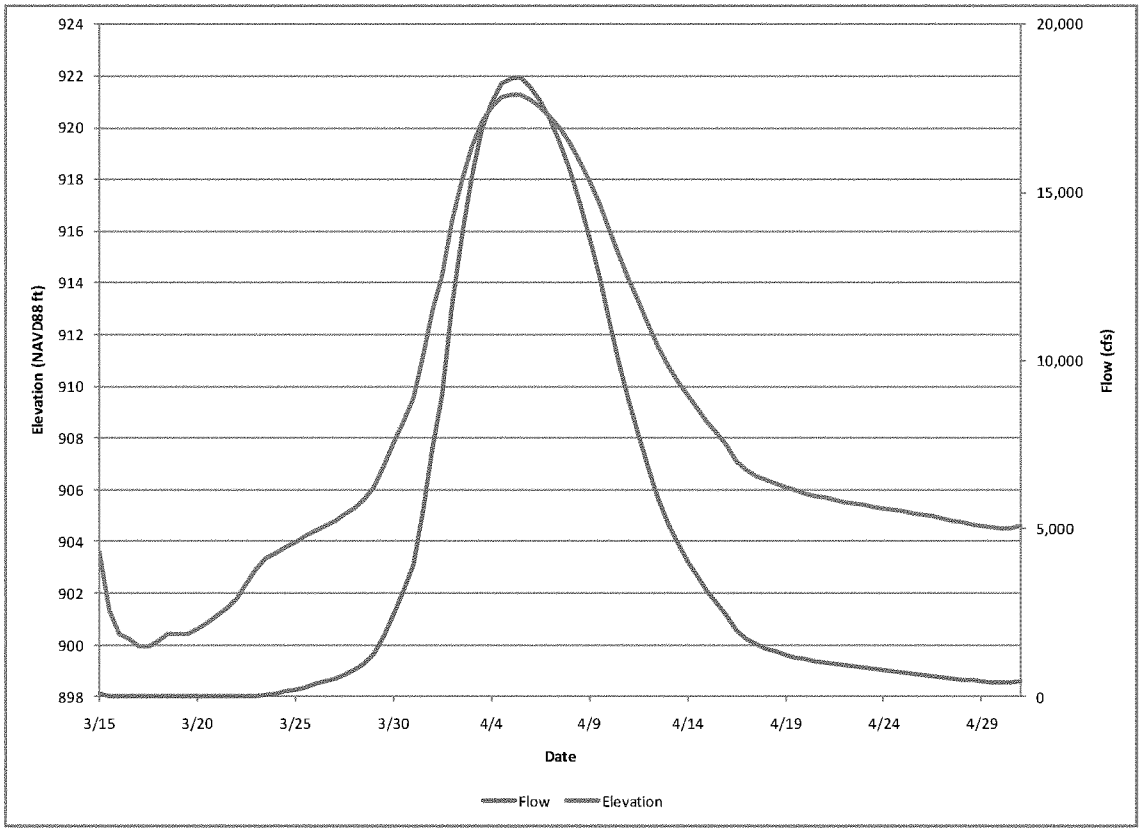


Figure F-D99 Hydrograph Upstream of the Diversion Inlet Structure (RS 152527) for the 500-Year Event

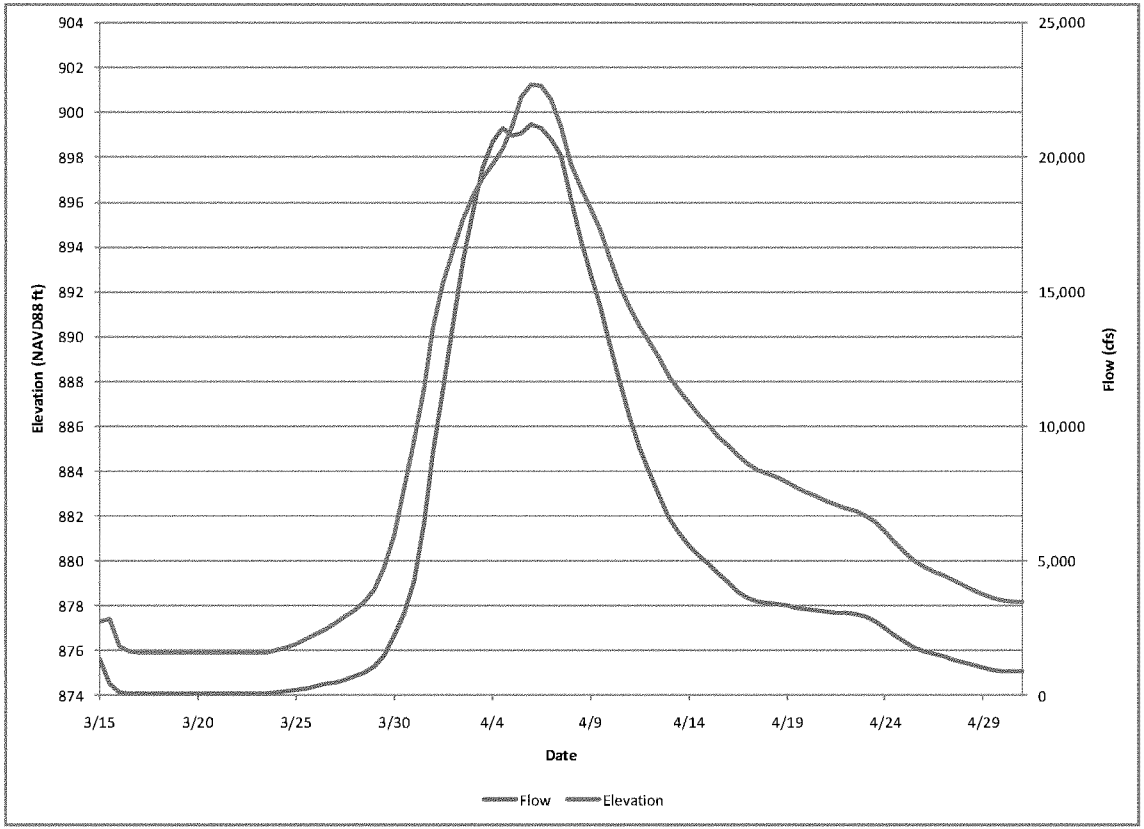


Figure F-D100 Hydrograph between the Sheyenne River and Maple River (RS 83654) for the 500-Year Event

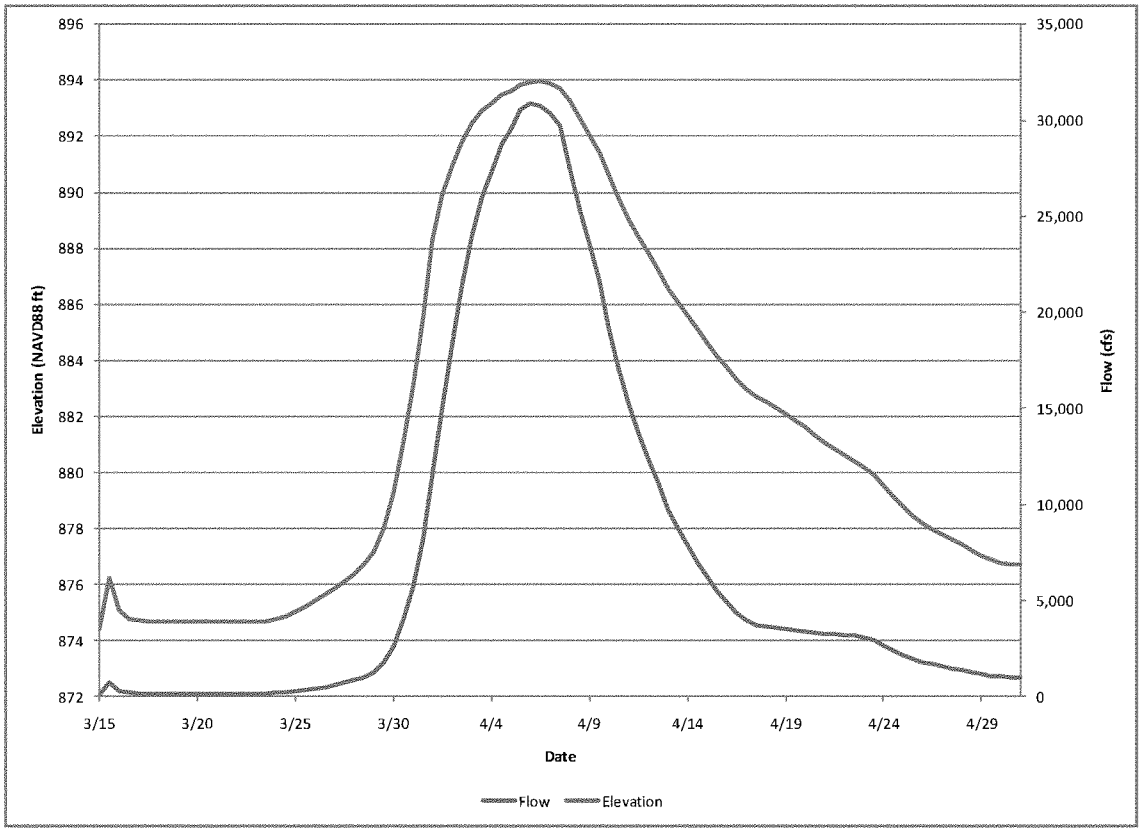


Figure F-D101 Hydrograph between the Maple River and Lower Rush River (RS 64696) for the 500-Year Event

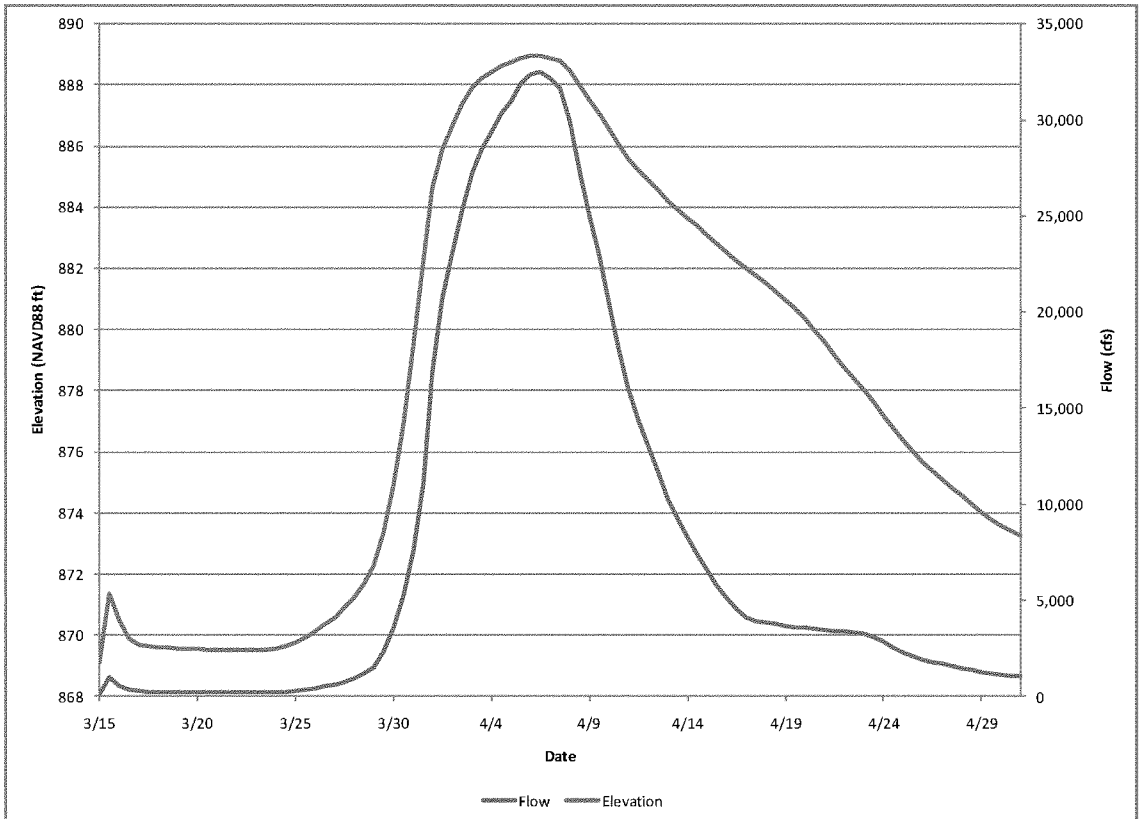


Figure F-D102 Hydrograph between the Rush River and the Outlet Structure (RS 29253) for the 500-Year Event

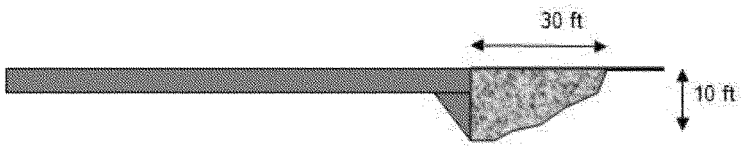
F-D4.0 EROSION PROTECTION CALCULATIONS FOR DOWSTREAM OF THE RED RIVER OF THE NORTH AND WILD RICE RIVER CONTROL STRUCTURES

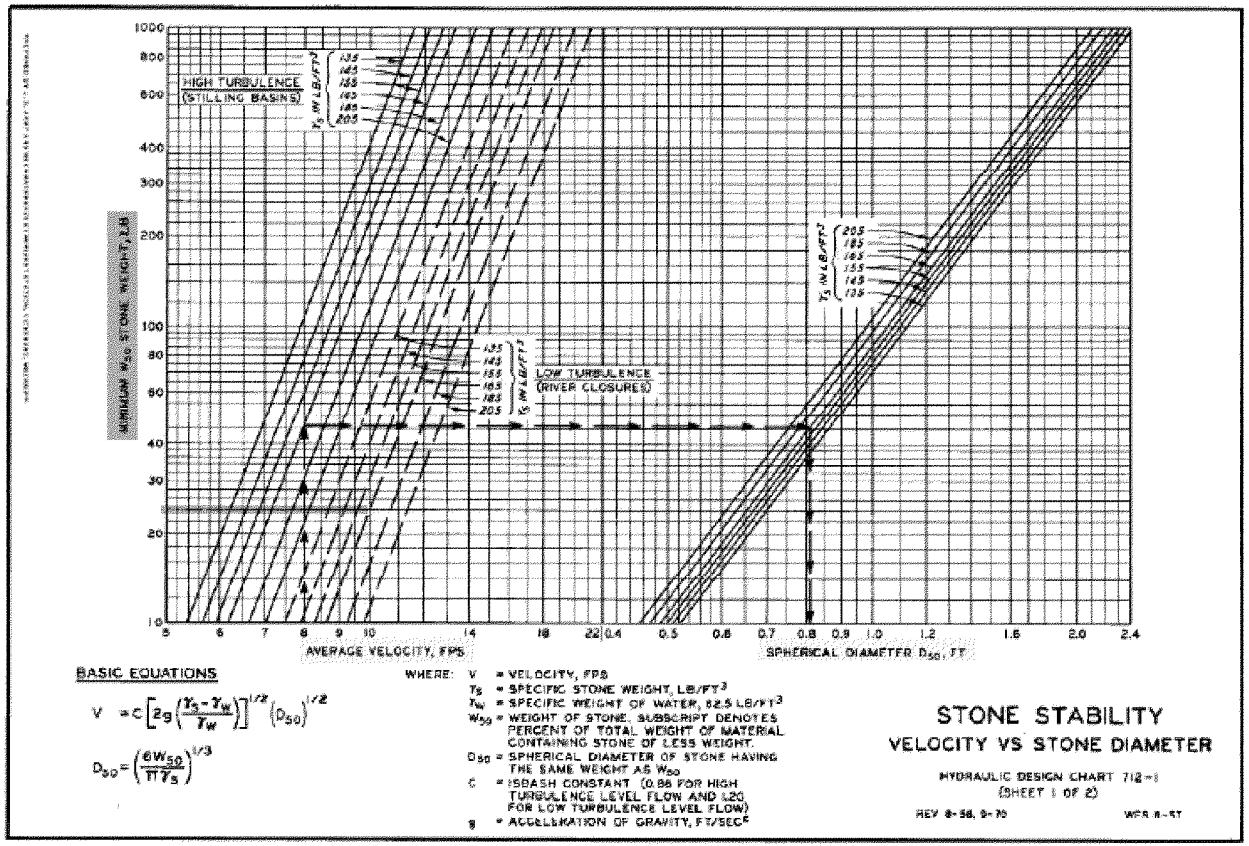
The following information was previously presented as Exhibit D of Appendix F of the Phase 3 report submitted on August 6th, 2010, and is included here for completeness. Flow velocities and return periods have been updated to reflect Phase 4 hydrology unless otherwise noted.

While the range of flow velocities through the gates of the Red River of the North (RRN) and Wild Rice River (WRR) control structures can be quite large, approximately 2-25 ft/sec and approximately 15 – 40 ft/sec, respectively, the velocities significantly decrease once the water is through the gates. For example, at the RRN control structure, HEC-RAS modeling results show that at the bounding cross section on the downstream side of the control structure, velocities range from 1.7 ft/second for the 10-year event to 4.3 ft/second for the 500-year event (downstream flow velocities are less for the WRR structure). In addition, for events greater than the 5-year event (based on Phase 3 hydrology), the gates of the RRN and WRR control structures are submerged; therefore, the jet of water coming through the gates quickly dissipates under so much head. Once the gate configurations for both of the control structures are more well-defined during the next phase of the project, the jet velocities at the end of the concrete slab will be calculated for both structures. For this phase of the project, riprap sizing calculations were performed for erosion protection purposes using the conservative assumption that velocities downstream of the control structures will not exceed 10 ft/sec. The unit weight of stone, which generally varies from 150 to 175 lbs/ft^3, was assumed to be 165 lbs/ft^3. The following page shows the stone stability nomograph from Hydraulic Design Chart 712-1 which shows the relationship between velocity and stone diameter sizing. For a velocity of 10 ft/sec and a unit weight of stone of 165 lbs/ft^3, the spherical diameter of a D50 stone is 0.65 feet.

As discussed in EM 1100-2-1601, stone size should be increased to resist hydrodynamic and a variety of nonhydrodynamic-imposed forces and/or uncontrollable physical conditions. The stone size increase can be accomplished by including a minimum safety factor of 1.1. This safety factor can be increased for a variety of conditions including situations where severe freeze-thaw is anticipated.

For this phase of the project, a minimum median D50 stone size diameter of 1.0 foot has been assumed. A minimum cut-off wall depth of 10 feet has also been assumed, and the length of riprap needed downstream of the cut-off wall has been assumed to be 30 feet (3 times the height of the cut-off wall), as shown conceptually in the drawing below.




HIGH TURBULENCE
(STILLING BASINS)
LOW TURBULENCE
(RIVER CLOSURES)
AVERAGE STONE WEIGHT, W50, LB
AVERAGE VELOCITY, FPS
SPHERICAL DIAMETER D50, FT
BASIC EQUATIONS
WHERE: V = VELOCITY, FPS
Ys = SPECIFIC STONE WEIGHT, LB/FT3
Yw = SPECIFIC WEIGHT OF WATER, 62.5 LB/FT3
W50 = WEIGHT OF STONE, SUBSCRIPT DENOTES PERCENT OF TOTAL WEIGHT OF MATERIAL CONTAINING STONE OF LESS WEIGHT.
D50 = SPHERICAL DIAMETER OF STONE HAVING THE SAME WEIGHT AS W50
C = ISBASH CONSTANT (0.86 FOR HIGH TURBULENCE LEVEL FLOW AND 1.20 FOR LOW TURBULENCE LEVEL FLOW)
g = ACCELERATION OF GRAVITY, FT/SEC2
STONE STABILITY
VELOCITY VS STONE DIAMETER
HYDRAULIC DESIGN CHART 712-1
(SHEET 1 OF 2)
REV 8-58, 9-70
WES 8-57

RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT E – HYDRAULIC DESIGN COMPUTATIONS STORAGE AREA 1

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

By: BARR ENGINEERING

FINAL - February 28, 2011

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT E – HYDRAULIC DESIGN COMPUTATIONS STORAGE AREA 1

F-E1.0 INTRODUCTION

This exhibit documents the analysis used to develop preliminary design recommendations for creating Storage Area 1 (SA1) along the North Dakota East Diversion Channel as part of Phase 4 of the Fargo-Moorhead Metro Flood Risk Management Project.

The design scenarios discussed in this exhibit are preliminary only. The result and conclusions of this analysis are based on the current working hydraulic model for the project. As elements of the hydraulic model are changed, added or removed, portions of this analysis may need to be updated.

F-E2.0 BACKGROUND

F-E2.1 PHASE 4 OBJECTIVES

In Phase 3, this project looked at a design scenario that minimized staging of water upstream of the Protected Area, confining adverse flooding impacts to areas downstream of the Protected Area. In Phase 4, the objective is to develop a design scenario where flood controls keep downstream impacts negligible by staging floodwaters upstream of the Protected Area. Control structures on the major rivers limit the flows passing through and around the control Protected Area. Excess water is stored in designated areas. Most of these areas are to the south of the North Dakota Diversion Channel. The exception being SA1, which is a 4360 acre area on the north side of the diversion channel.

F-E2.2 MODELING APPROACH

F-E2.2.1 Hydrology

The hydrology used to generate flow data for the unsteady HEC-RAS model for the project was developed previously and not modified as part of the SA1 preliminary design task. The four design flood events are the 10-, 50-, 100-, and 500-year. For more information on the project hydrology, see Exhibit A Background Hydrologic Information and Appendix A.

F.E2.2.2 Hydraulics

The hydraulic analysis for Phase 4 is based on results from unsteady HEC-RAS models for each of the design flood events. Models were run using HEC-RAS version 4.1.

F-E2.3 KEY TERMS

The following terms are key components of the flood control system in the vicinity of SA1. A brief definition of these terms is provided below. Each will be discussed in greater detail within the text of this exhibit.

Diversion Channel – Generic term for the channel to be constructed around the City of Fargo. The channel begins and ends on the Red River and intersects the North Dakota tributaries to the Red River. This exhibit focuses on the portion of the Diversion Channel on the south side of SA1 between the Wild Rice Control Structure and the Main Inlet Weir.

Main Inlet Weir – An in-line structure in the Diversion Channel that marks the transition between the reach south of SA1 where major flood events are outside the channel banks, to a deeper channel that contains the design flood events within the channel banks.

Protected Area – Developed area, which includes the cities of Fargo and Moorhead, that will be protected from the design flood events by this project.

Wild Rice Control Structure – Gated structure that controls flow from the Wild Rice River into the Protected Area.

Modified Channel – In order to construct the Wild Rice Control Structure in the dry, a modified channel will be excavated for the Wild Rice River that cuts off a meander near the southeast corner of SA1.

Inlet-Outlet Opening – A large opening in the levee around SA1 that allows water from the Diversion Channel to enter and leave SA1.

East Outlet – A gated structure in the East Levee for SA1 that will allow for natural drainage of SA1 to the Wild Rice River during normal conditions. The structure will be closed during flood events.

North Outlet – A gated structure in the North Levee for SA1 that will allow for natural drainage from SA1 to Rose Coule Drain 27 during normal conditions. The structure will be closed during flood events.

Tie-Back Levee – In addition to SA1 other areas will be inundated during major flood events. To contain this inundation there will be Tie-Back levees on the east and west sides of the staging area. The west Tie-Back levee extends from the Main Inlet Weir to the south at elevation 923.

Wild Rice River – Unless otherwise stated, any mention of the Wild Rice River in this exhibit refers to the North Dakota Wild Rice River, as opposed to the Minnesota Wild Rice River.

F-E3.0 DESIGN OBJECTIVES

The intent of SA1 is to provide contained flood storage upstream of the Protected Area. The design of SA1 must balance the design objectives identified below.

Storage – The goal of providing storage in SA1 is to minimize the depth of staging upstream. The lower the upstream staging the smaller the inundation footprint for major flood events on upstream properties.

Footprint – To achieve a meaningful storage volume in SA1, there will need to be a significant property takes. The layout of SA1 should minimize the number of property owners and structures affected.

Safety – SA1 will be constructed immediately south of the Protected Area. The long term integrity of the levees around SA1 is critical to the safety of the community and the success of the project.

Simplicity – The flood control systems proposed by this project requires some complex elements. The operations and maintenance of SA1 should be kept simple as much as possible, so SA1 can operate effectively with minimal human input.

Flexibility – Design of SA1 should allow for continued use of the area by the community when the area is not flooded. This includes allowing some roads to continue to pass through the Storage Area.

F-E4.0 SITE DESCRIPTION

The area designated for SA1 is predominantly farm fields with a few structures that mostly consist of houses and farm buildings. Topographically SA1 is between the Sheyenne River and the Wild Rice River. Relative to existing infrastructure SA1 is bounded by County Road 17 (Co Rd 17) on the west, Co Rd 14 on the north, and Interstate-29 (I-29) on the east. Figure F-E1 is a map that shows proposed flood control measures and existing roads in the vicinity of SA1.

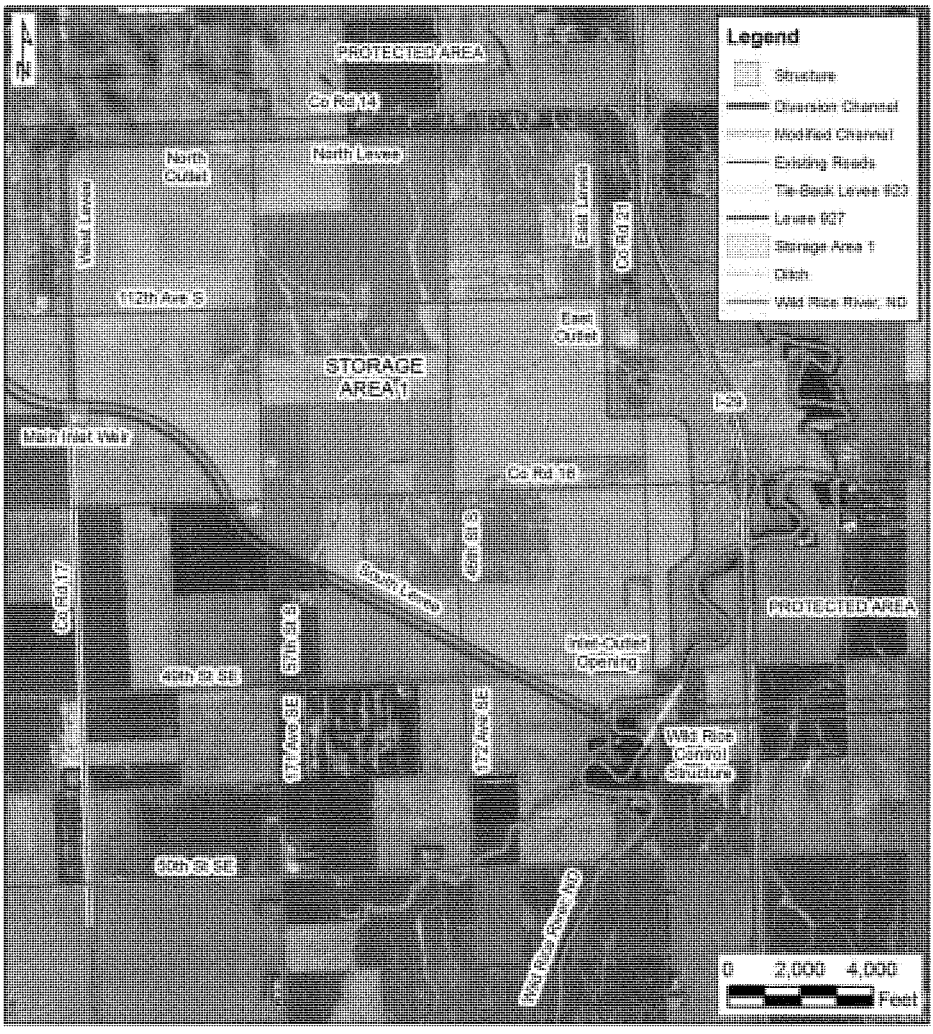

Figure F-E1 Vicinity map for Storage Area 1

Figure F-E2 shows the same area as Figure F-E1, but with a colored elevation gradient ranging from elevation 900 to 923. The topography generally slopes to the north, with some areas draining to the northeast. The low end of this range corresponds to the bottom of the drainage ditch at the North Outlet. The high end of the elevation range corresponds to the crest elevation of the Tie-Back Levee to be constructed along Co Rd 17. The existing ground elevations within SA1 range between 908 and 915.

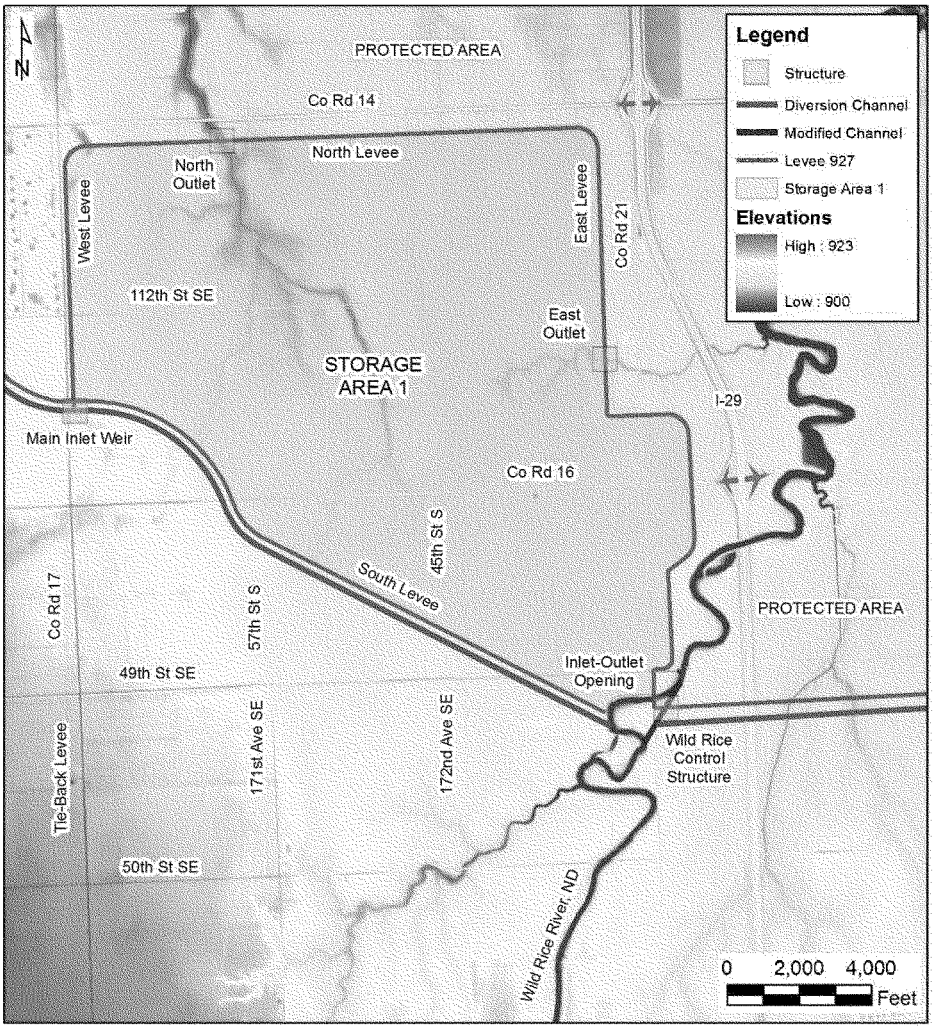

Figure F-E2 Elevation gradient map for Storage Area 1

There are two primary drainage basins within SA1. The larger of the two basins drains north to Rose Coule Drain 27. The smaller basin drains east to the Wild Rice River.

F-E5.0 PRELIMINARY DESIGN

This section describes the recommended preliminary design for SA1 and related infrastructure. A detailed description of how these recommendations were developed is provided in the Design Analysis section below.

The proposed footprint for SA1 is shown in Figure F-E1. This 4360 acre area will be surrounded by a levee at elevation 927. The crest elevation for the levee assumes a minimum of 4-feet of freeboard above the peak stage for the 100-year and 500-year flood events.

SA1 will provide over 55,000 acre-feet of storage during the 100-year and 500-year flood events. Initial modeling results indicated that the inclusion of SA1 results in roughly a 2-foot reduction in the upstream staging elevation during those same flood events.

There will be three openings in the SA1 levees. The Inlet-Outlet Opening will be a 1400-foot gap in the levee at the southeast corner of SA1. This opening hydraulically connects the Wild Rice River and the Diversion Channel with SA1.

The other two openings will be the North Outlet and East Outlet. These will be gated structures that will be left open to allow for natural drainage during non-flood conditions. During major floods the gates will be closed so that flood waters are not released into the Protected Area. After the flood event has passed the gates will be opened to allow low areas within SA1 to drain.

Co Rd 16 and a portion of Co Rd 21 will be maintained through SA1. To accomplish this, the roads will need to be elevated over the levee crest. By maintaining roads within SA1, it will be possible for local property owners to continue to use the land during normal, non-flood, conditions.

F-E6.0 DESIGN ANALYSIS

The preliminary design for SA1 balances an array of factors and design criteria. The major considerations affecting the design are discussed below.

F-E6.1 FOOTPRINT

The area delineated for the proposed storage area is approximately 4360 acres, with a perimeter of about 58,000 feet. It is roughly bounded by the proposed Diversion Channel to the south, Co Rd 17 to the west, Co Rd 14 to the north and I-29 to the East. Geographically, this area is bounded by the Sheyenne River to the west and the Wild Rice River to the east (Figure F-E1).

The footprint for SA1 was selected to minimize impacts to existing infrastructure, maximize storage capacity, and effectively interact with other flood control elements within the project.

The perimeter stays east of Co Rd 17 and south of Co Rd 14, so that the only County Roads affected are 16 and 21. Existing structures within SA1 will need to be removed. The proposed footprint was chosen in part because of the low density of existing structures.

Where practical, the levee alignment was adjusted to avoid unnecessary impacts to structures. The perimeter levees avoid taking the structures along the south side of Co Rd 14 and those along Co Rd 21.

Constructing a basin this large will require significant property takes. Careful consideration was given regarding how to achieve the maximum benefit for the community with minimum adverse impacts to individual property owners. There will no doubt be objections to placement of SA1 by those adversely impacted. The challenge for the project stakeholders is to evaluate the short and long-term impacts of this feature and determine if the benefits are worth the cost.

A separate analysis by Moore Engineering is assessing the hydraulic impact of an even larger SA1 that extends further to the north.

F-E6.2 PROPOSED LEVEES

The levee surrounding SA1 has a consistent template—the 15-foot wide crest is at elevation 927 and the side slopes are 4:1. The levee top width and side slopes were set based on a preliminary geotechnical analysis, discussed in Exhibit N Slope Stability. The crest elevation of the levee assumes a minimum of 4-feet of freeboard above the 100-year and 500-year peak upstream stage.

With the exception of the Inlet-Outlet Opening, the perimeter of SA1 is essentially one continuous levee. For organizational purposes the sections of levee on each of the four sides of SA1 were named South Levee, West Levee, North Levee, and East Levee.

The levees will be constructed from Diversion Channel spoils and from borrow trenches within SA1. A 100-foot buffer will separate the toe of levee from the edge of the adjacent borrow trench. The width and depth of the trenches have been approximated so as to provide sufficient fill material for the levees and to provide internal drainage conveyance for ditches interrupted by the placement of the levees.

Details specific to each section of levee are discussed below.

F-E6.2.1 South Levee

The South Levee parallels the Diversion Channel (Figure F-E1). The south toe of slope for the levee is offset 300-feet from the top of bank on the north side of the channel. Spoils from the excavation of the Diversion Channel will be used to construct the South Levee. The height of the levee above existing grade ranges from 10 feet at the west end to 18 feet near the east end. Figure F-E3 shows the top elevation of the South Levee relative to the Diversion Channel thalweg and bank elevations.

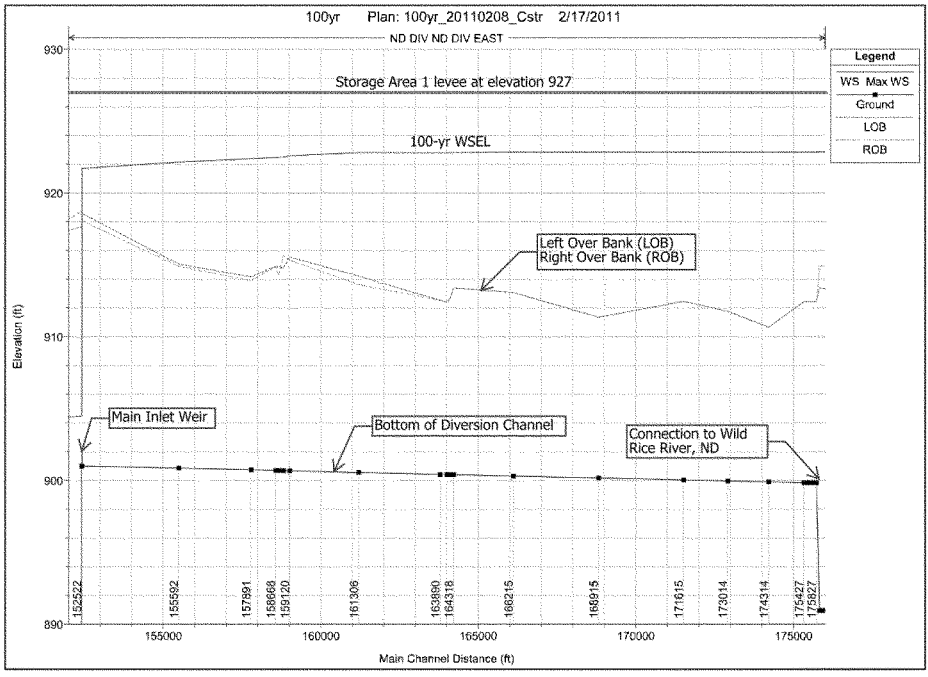

Figure F-E3 Diversion Channel profile along southern edge of Storage Area 1

The profile plot in Figure F-E3 illustrates the average height for the South Levee. The left overbank (LOB) and right overbank (ROB) profiles provide a rough indication of the existing ground elevation along the South Levee alignment. The ground generally slopes from west to east towards the Wild Rice River.

No structures are planned for construction in the South Levee. Co Rd 16 will pass up and over the South Levee. The levee will interrupt 49th St SE, 57th St SE, and 45th St SE. See the Roadway Impacts section in this exhibit for more details.

F-E6.2.2 West Levee

The West Levee runs along the east side of Co Rd 17, with an offset of approximately 150 feet from centerline of road to centerline of levee (Figure F-E1). It is assumed that no structures will be constructed in the West Levee. No roads will be carried over the West Levee. The levee will interrupt 112 St SE.

F-E6.2.3 North Levee

The North Levee is on the south side the Co Rd 14, with an offset of approximately 600-feet from centerline of road to centerline of levee. (Figure F-E1). This offset avoids

the taking of five existing residential structures and their associated out-buildings along the south side of Co Rd 14.

The North Outlet will be constructed in the North Levee and in-line with a major drainage ditch that would otherwise be blocked by the levee. The design of this structure is discussed in a separate section below. There are no roadways over the top of the North Levee. The levee interrupts 57th St SE and 45th St SE.

F-E6.2.4 East Levee

The East Levee runs along the west side of I-29 (Figure F-E1). At the north end the levee alignment is on the west side of Co Rd 21. Once the levee gets south of a developed section along Co Rd 21, the alignment shifts to the east side of Co Rd 21. The southern portion of the levee jogs back to the west to avoid impacts to the Wild Rice River, ND.

The East Outlet will be constructed in the East Levee and in-line with the major drainage ditch that would otherwise be blocked by the levee. The design of the structure is discussed in a separate section below.

Both Co Rd 21 and Co Rd 16 will pass up and over the East Levee embankment. The levee interrupts 112th St SE. These changes are discussed further under Roadway Impacts.

F-E6.3 DIVERSION CHANNEL

There are three distinct reaches of the Diversion Channel near SA1. The reach east of SA1 connects the Wild Rice River with the Red River. The reach west of SA1 is a deep channel design to contain and convey all of the design flood events for the project.

The reach most frequently discussed in this exhibit is the portion of the Diversion Channel immediately south of SA1 that extends from the Wild Rice River to the Main Inlet Weir. As seen in Figure F-E3 above, this reach of the Diversion Channel slopes from west to east, towards the Wild Rice River. There are two primary reasons for this design. First, the existing ground for this reach of the Diversion Channel generally slopes to the Wild Rice River. Second, the hydraulic design requires that the Main Inlet Weir crest elevation be above the thalweg for the Diversion Channel. Sloping the channel towards the Main Inlet weir would require a low flow path through the Main Inlet Weir, which would add complexity and cost to the project. Figure F-E4 shows a typical cross-section for the Diversion Channel.

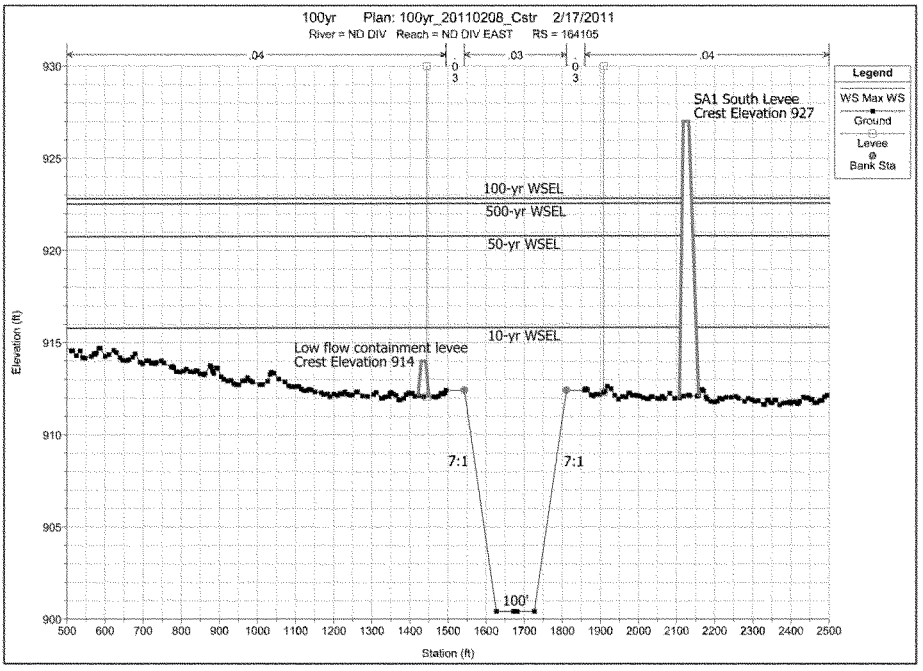

Figure F-E4 Diversion Channel cross-section

Some of the spoils from the excavation of the Diversion Channel will be used to raise the south bank of the diversion channel up to at least elevation 914. As seen in Figure F-E4, fill will be needed for roughly the eastern two-thirds of the Diversion Channel's left overbank. The remainder of the spoils from the Diversion Channel excavation will be used to construct the South Levee and portions of the East and West Levees.

The bottom width for the SA1 section of the Diversion Channel is 100-feet. This width was selected to be slightly wider than the 90 foot crest length for the Main Inlet Weir. The side-slopes for the channel are 7:1 (H:V).

F-E6.3 1 Main Inlet Weir

The purpose of the Main Inlet Weir is to control the flow of water from the staging area into the portion of the Diversion Channel that is designed to contain the four design flood events. The 90-foot Ogee weir has a crest elevation of 903.25. Water surface elevations below the weir are roughly 20 feet lower than those upstream of the weir. The design of the Main Inlet Weir is discussed in detail in Exhibit D – Diversion Structures.

F-E6.4 INTERNAL DRAINAGE

As discussed in the Site Description section above, there are two existing drainage basins within SA1. The larger basin drains north to Rose Coule Drain 27. The smaller basin drains east to the Wild Rice River. Levee construction around SA1 has the potential to cut off these drainage basins from their existing downstream flow paths.

F-E6.4.1 Outlet Options

Two options were considered for dealing with internal drainage within SA1.

Option 1
Construct two outlet control structures through the proposed levees and in-line with the existing drainage ditches.

Advantages
- Maintains existing drainage patterns
- Requires limited grading above and beyond what is already required for excavation of the borrow trenches to construct the levees around SA1

Disadvantages
- Control structures are expensive
- Control structures add complexity to the future operation and maintenance of SA1
- Placing an outlet control structure through the levee creates a potential weak point in the levee embankment

Option 2
Use borrow trenches and modified ditch channels to route all surface drainage south to the diversion channel.

Advantages
- No operation requirements and minimal maintenance
- Leaves levee embankments uninterrupted

Disadvantages
- Alters existing drainage patterns
- May require significant amounts of additional grading to collect and convey water to the south against existing grades
- May require fill in some places to remove low points and make it possible to drain water back to the diversion channel
- Would require the construction of large culverts under existing roads

The preliminary design for SA1 is based on Option 1. It assumes the construction of a North Outlet and East Outlet in-line with the two largest drainage ditches leaving the site. By maintaining existing drainage patterns, the design limits potentially adverse impacts on

downstream property owners who may rely on runoff from the watershed for irrigation. The cost to route surface runoff from over 4360 acres against the natural slope of the land would likely be more expensive than constructing two outlet control structures.

While Option 1 takes advantage of the existing ditch drainage system, not all ditches within the SA1 footprint drain to the North Outlet or East Outlet locations. Several smaller ditches, particularly in the northeast corner of SA1, drain away from the site on their own. These ditches will be intercepted by the borrow trenches excavated to construct the West, North and East Levees. Borrow trenches along the West and North Levees will drain to the North Outlet. Borrow trenches along the East Levee will drain to the East Outlet. A small area adjacent to the Wild Rice River will drain back through the Inlet-Outlet Opening for SA1.

F-E6.4.2 North Outlet

The drainage area for the North Outlet is roughly 2810-acres. Concrete retaining walls will create a 10-foot wide break in the levee embankment. A sluice gate and stop logs will control flow through the opening. The outlet will be constructed in-line with the existing drainage ditch. Figure F-E5 is a vicinity map for the North Outlet.

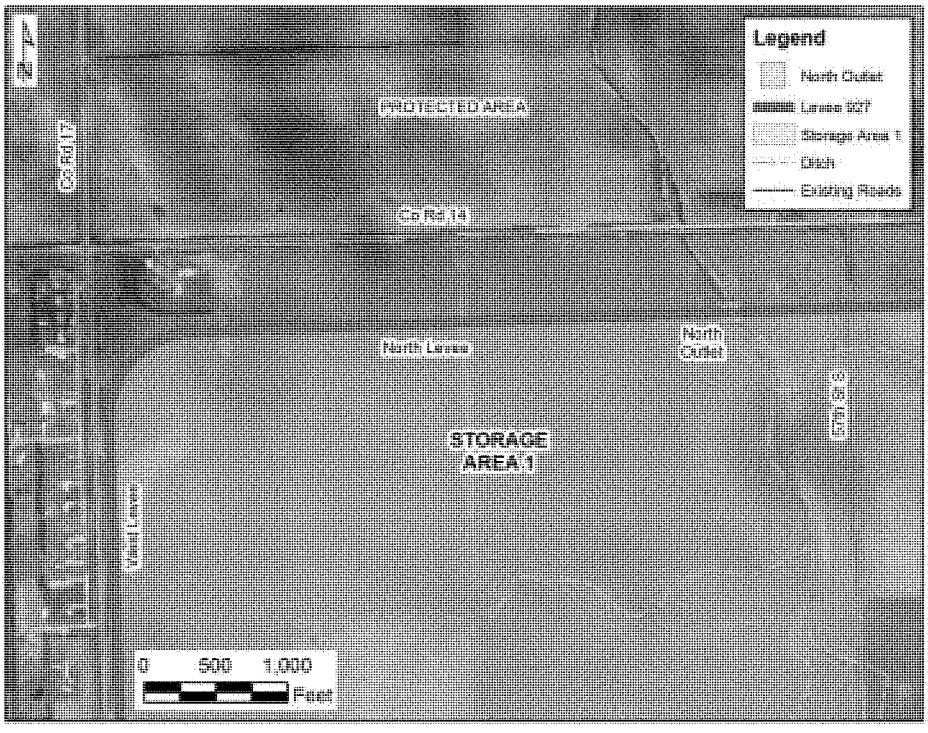

Figure F-E5 Vicinity map for North Outlet from Storage Area 1

Sizing of the North Outlet was based on a hydrologic analysis of the contributing drainage area and an assessment of existing drainage structures along the ditch. From aerial imagery, the culvert crossing beneath County Road 14 (Co Rd 14) appears to be a double 6'x6' or 8'x8' concrete box culvert.

F-E6.4.3 East Outlet

The drainage area for the East Outlet will be roughly 1500 acres. Concrete retaining walls will create a 10-foot wide break in the levee embankment. A sluice gate and stop logs will control flow through the opening. The outlet will be constructed in-line with the existing drainage ditch. Figure F-E6 provides a vicinity map for the East Outlet.

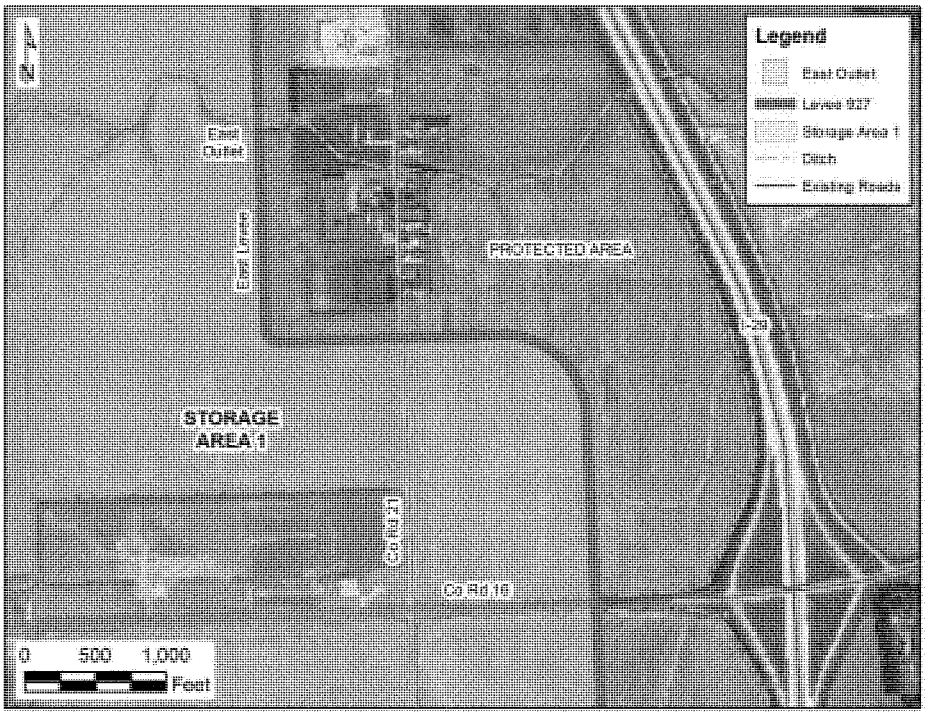

Figure F-E6 Vicinity map for East Outlet from Storage Area 1

Sizing of the East Outlet was based on a hydrologic analysis of the contributing drainage area and an assessment of existing drainage structures along the ditch. From aerial imagery, the culvert under Co Rd 21 appears to be a circular CMP pipe, perhaps 36" in diameter.

F-E6.4.4 Operations and Maintenance

The gates for the East Outlet and North Outlet will generally remain open to allow for the natural drainage of surface runoff from SA1. When major flooding is imminent, the gates to both outlet control structures will be closed for the duration of the flood event. Once the floodwaters have subsided, the gates will be reopened to allow for the drainage of portions of SA1 that are too low to drain out through the Inlet-Outlet Opening.

The final design will need to determine appropriate energy dissipation measures downstream of the outlets. Possible solutions include baffles, stilling basin or sill. The size of the outlet, the gate control rules and the amount of energy dissipation will all depend on the upstream storage depth after a flood and the desired drawdown time.

Access to the North and East Outlet structures will be from inside the Protected Area so that operators can operate the gates as-needed before during and after a flood event. East Outlet access will be from Co Rd 21. North Outlet access will be from Co Rd 14.

F-E6.5 HYDRAULICS

SA1 will be hydraulically connected with the Wild Rice River and the Diversion Channel through the Inlet-Outlet Opening, which will be a 1400-foot gap in the levee perimeter near the Wild Rice Control Structure. Figure F-E7 shows approximate layout for the proposed flood control infrastructure in and around the Inlet-Outlet Opening.

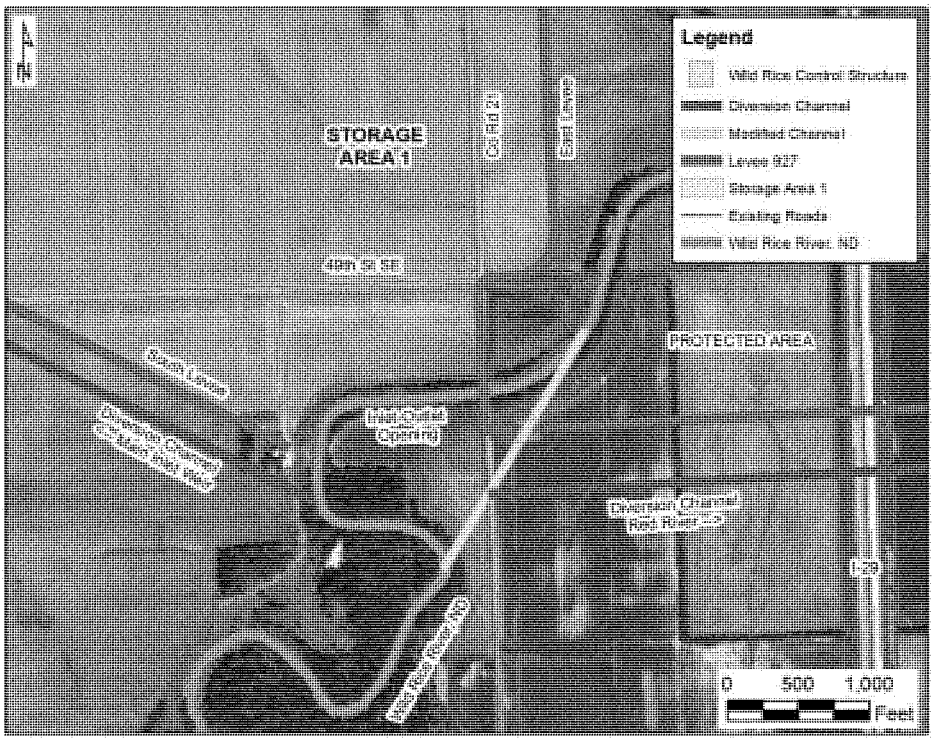

Figure F-E7 Vicinity Map for Inlet-Outlet Opening for Storage Area 1

F-E6.5.1 Wild Rice Control Structure

In order to evaluate the Inlet-Outlet Opening it is necessary to understand how the Wild Rice Control Structure and related flood control measures function. A detailed discussion of the design for the Wild Rice Control Structure is in Exhibit D Hydraulic Design

Computations – Diversion Structures. An abbreviated description follows. In order to construct the Wild Rice Control Structure in the dry, a modified channel for the Wild Rice River will be constructed roughly as shown in Figure F-E7. There are three major areas of disturbance on the Wild Rice River. The two connection points for the Modified Channel and the extension of the levee across the channel on the same alignment as Co Rd 21. There will likely be minor impacts to the channel bank from the portion of the Diversion Channel that extends west along the south side of SA1. The bypassed meander of the Wild Rice River will remain in place as a slough.

With this configuration in place, the Wild Rice River will flow as normal during non-flood events. Surface drainage collected by the Diversion Channel will drain to the Wild Rice River. Surface drainage collected by the east reach of the Diversion Channel will drain to the Red River.

As water levels rise during major flood events (e.g. 10-year or more) the flood levels in the Red River and the Wild Rice River will equalize via the east reach of the Diversion Channel. Water will also begin backing up into the SA1 reach of the diversion channel. The Wild Rice Control Structure and Red River Control Structure limit flows into the Protected Area. When upstream flows exceed the capacity of the gate openings for the Wild Rice and Red River control structures, water will begin backing up into low-lying areas, including SA1.

F-E6.5.2 Inlet-Outlet Options

The two primary options for the flow of floodwaters into and out of SA1 are an uncontrolled opening or a control structure.

Option 1 – Uncontrolled Opening
Leave an opening in the levee that allows water in SA1 to equalize with the water in the diversion channel.

Advantages

- Simple design that does not require operation during flood events
- Low cost alternative since it means less levee construction and little to no engineering required for the Inlet-Outlet
- Minimizes the amount of time that the levee has to withstand elevated flood waters

Disadvantages

- Lack of control over how long water is detained in SA1

Option 2 – Control Structure
A controlled opening that could be opened to allow water into the storage area and temporarily closed until after the peak of the flood has passed and the water can be safely released downstream.

Advantages

- Ability to regulate the rate of inflow and outflow from SA1 in order to maximize flood mitigation benefits

Disadvantages

- Adds complexity to the design
- Creates an additional structure that must be operated during flood events
- Increases initial construction costs and long term maintenance costs
- Retention of water within SA1 extends the time that a large volume of water is being held back by the SA1 levee

Option 2 operates using the following design scenario. The Main Inlet Weir is replaced with a gated structure. During the rising leg of the hydrograph, the Main Inlet Gate is closed and the SA1 control structure gate remains open. Once SA1 is full, the Inlet-Outlet Gate is closed and the Main Inlet Gate is opened. After flood waters recede, the SA1 control structure gates are opened to release the stored water.

Preliminary modeling results indicate that although retaining water in SA1 helps reduce the duration of upstream staging, it has no benefit in relation to downstream impacts. The limited flood mitigation benefit and the other significant disadvantages to using a control structure led to the decision to leave the inlet-outlet for SA1 uncontrolled.

F-E6.5.3 Inlet-Outlet Opening Size and Location

Having opted for an uncontrolled opening, the subsequent design questions for the Inlet-Outlet Opening are how wide should the opening be, where should it occur, and at what elevation should water enter the storage area. For SA1 to be hydraulically connected to the upstream staging area, the opening for SA1 must be along the boundary with the Diversion Channel. The preliminary design analysis looked at various combinations of opening locations, widths, and elevations.

Opening Location

Hydraulically, the location of the opening had minimal effect on upstream and downstream impacts. It was noted that an opening placed further to the west would result in lower peak stage in SA1 because the peak water surface elevations at the Main Inlet Weir are lower than at the Wild Rice River. This drop in the water surface elevation from east to west is shown in Figure F-E3. However, it is somewhat questionable whether this drop will be as pronounced as the model indicates. The RAS model assumes that conveyance occurs in the diversion channel and the adjacent storage areas simply match the channel profile. It is more likely that for most major floods there will be no defined flow path and the water surface elevation will be relatively constant between the Wild Rice River and the Main Inlet Weir. For modeling purposes, the opening location was placed at the southeast corner of SA1 to more accurately model the peak stage within SA1. A more important factor governing the Opening location turns out to be the existing ground elevations and their effect on the opening elevation.

Opening Elevation
A lower opening elevation means that water enters SA1 sooner. It also means that less water will be left in the system once the flooding subsides. Opening elevations as low as 904 and as high as 918 were tested. From a practical standpoint however, the existing ground elevation effectively sets the lower limit for the opening. There is limited storage in SA1 below elevation 908 and the elevations along the Diversion Channel range from 910 to 915 (Figure F-E2). Even if water is allowed to enter the area below the existing grade, there will not be meaningful storage until the water surface gets above 910.

Opening Width
Various opening widths ranging from 1000-feet, to 10,000 feet were tested. The design analysis found that wider openings tend to reduce upstream staging and increase downstream water surface elevations. If the opening is too narrow, the peak elevation in SA1 does not have enough time to match the peak in the Diversion Channel before the peak has passed. However, the differences between the tested scenarios were on the orders of a few hundredths of a foot. While adjusting the opening width could be used to refine the model during final design, the cost of earthwork and site constraints will play the biggest role in determining the opening width.

The opening widths tested all assume that the South Levee exists in some form. However, an alternative scenario would be to leave the south side of SA1—no South Levee at all. The major apparent advantage to this approach would be an 18,000 foot reduction in levee construction. However, there are several compelling reasons to construct the South Levee.

First, spoils from the excavation of the Diversion Channel need to go somewhere. It is assumed that it will be cheaper construct levees from materials immediately adjacent to the levee embankment, rather than hauling the material in from other places. This means using Diversion Channel for embankments adjacent to the channel, and borrow trenches to construct other levees that are not adjacent to a planned excavation.

Second, the staging area will create a vast expanse of water during major flood events. Having the South Levee will help break up wave action created by wind sweeping across the staged water.

Third, it is assumed that in the future local sponsors will try to find a way to reclaim the land used for SA1 through other mitigation measures that eliminate the need for SA1. If this happens it will be much more feasible for them to close a narrow gap between the South Levee and the East Levee than to construct over 3 miles of new levee.

Fourth, if there is ever a problem with one of the other levees for SA1, it would be possible to temporary fill in the Inlet-Outlet Opening to keep floodwaters out of SA1 while the problem is fixed.

Preliminary Design

Given these considerations, the preliminary design for the Inlet-Outlet Opening is a 1400-foot wide gap between at the southeast corner of SA1 near the Wild Rice Control Structure. The length was set primarily based on existing topography near the Wild Rice Control Structure. The meander in the Wild Rice River makes it unnecessary to continue the Diversion Channel excavation further east. With no spoils to place, it becomes a convenient place to leave an opening in the perimeter of the storage area. The existing ground elevations at the opening are between 912 and 914. Figure F-E8 displays a color elevation gradient for the area near the opening.

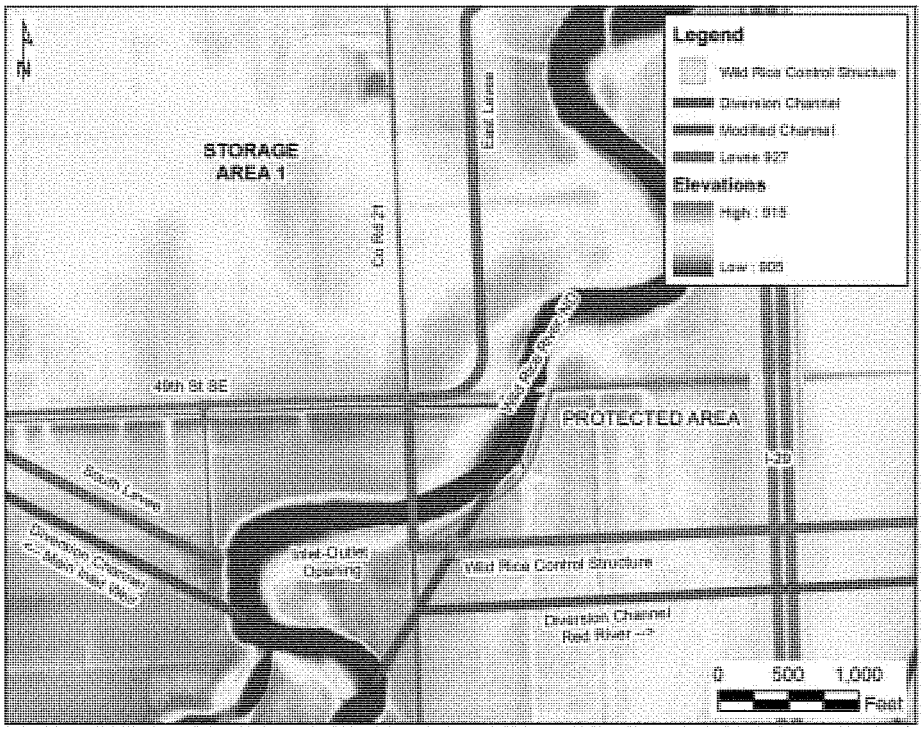

Figure F-E8 Elevation gradient map near Inlet-Outlet Opening for Storage Area 1

The hydraulic model assumes that borrow trenches will bring down the opening elevation to between 910 and 911. it is assumed that some excavation will be required to drain the ditch on the south side of 49th St SE through the Inlet-Outlet Opening to the Wild Rice River. This grading will create the low point where rising flood waters will first enter SA1. Figure F-E9 shows a cross section of the Wild Rice River in the middle of the opening.

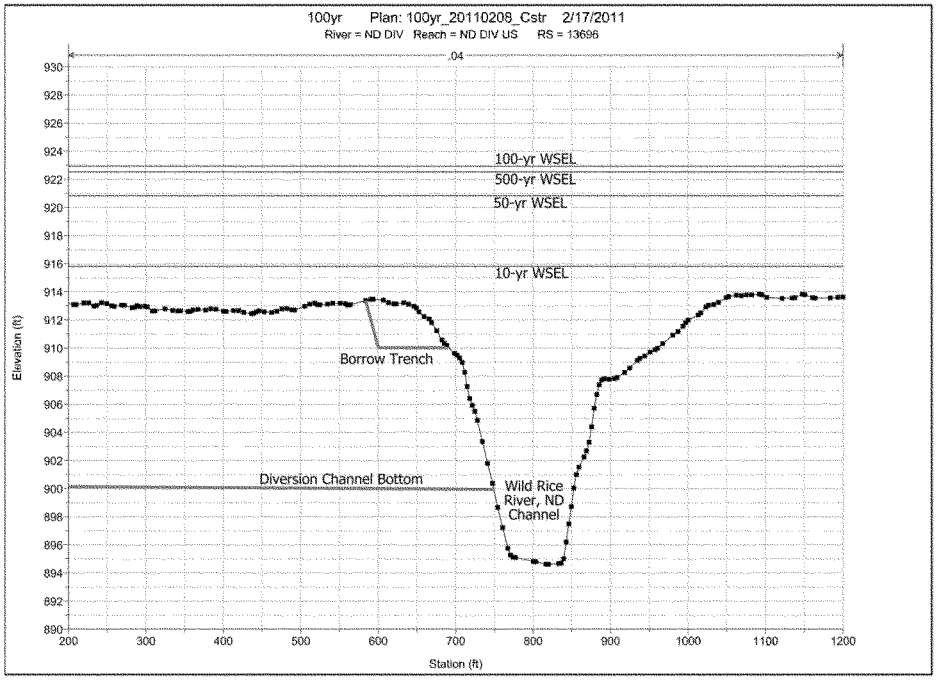

Figure F-E9 Wild Rice River cross-section at Inlet-Outlet Opening

F-E6.6 STORAGE

The storage volume achieved for SA1 is a product of the upstream staging elevation, the existing ground elevation and the footprint of the storage area. Of the available places for storage, the existing ground elevations are lowest on the north side of the Diversion Channel, which corresponds to a larger storage capacity for SA1. As discussed above, the footprint was maximized within the constraints of existing infrastructure.

The depth of staging upstream of the Protected Area is primarily dependent on the operation of the gated control structures on the rivers intersecting the Diversion Channel. Excess water is staged upstream of the gates. Table F-E1 contains the peak stage and storage values for the four design flood events.

Table F-E1 Stage-storage values for design flood events in Storage Area 1

Flood Event (yr)	Peak Stage (ft)	Peak Storage (ac-ft)
10	915.8	24,540
50	920.8	46,370
100	922.9	55,170
500	922.5	53,450

Figure F-E10 presents the data in Table F-E1 graphically in relation to the stage-storage curve for SA1.

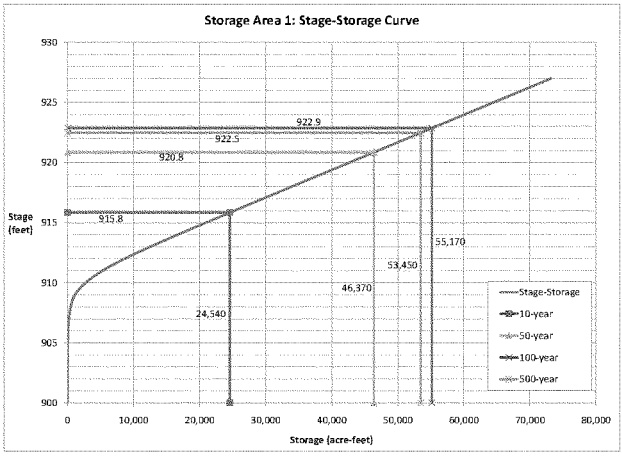

Figure F-E10 Stage-storage provided by Storage Area 1 for the design flood events

Note that the 100-year peak stage is higher than the 500-year peak stage. This phenomenon is caused by the Phase 4 project design objective to have no adverse impact on downstream flood elevations. Since the downstream 500-year floodplain is deeper and wider than the 100-year floodplain, it can receive more water. This corresponds to larger gate openings for the 500-year event than for the 100-year event. As discussed above, the restrictions created by the control structure gates cause water to stage upstream. A smaller opening for a given inflow will result in a higher stage. In this case, the difference in the size of the

gate openings is large enough that even with a larger inflow for the 500-year event, the upstream staging requirement is less than that for the 100-year event. For a more detailed discussion of this topic, see Exhibit D Hydraulic Design Computations – Diversion Structures.

The extension of the stage-storage curve to elevation 927 provides an indication as to amount of excess storage available should there ever be a flood that exceeds the design flood events.

F-E6.7 ROADWAY IMPACTS

The footprint of SA1 and the Diversion Channel will have impacts on existing roadways. Roads that intersect the Diversion Channel will either need to bridge the channel or become discontinuous. It is assumed that County Roads through SA1 would be maintain by bringing them up and over the levee embankment, while lesser roads will be interrupted at intersections with the levee. Impacts to specific roads are discussed below.

F-E6.7.1 County Road 21

Co Rd 21 runs north-south on the west side of I-29 (Figure F-E1). There are two major modifications to Co Rd 21 that will occur because of the proposed flood control measures.

The first modification is near the southeast corner of SA1. Under existing conditions, a bridge carries Co Rd 21 over the Wild Rice River. The construction of the Wild Rice Control Structure and adjacent levees will result in the removal of the Co Rd 21 bridge (Figure F-E7). Co Rd 21 will be discontinued from 50th St SE to Co Rd 16.

The second modification is near St. Benedict's Church, where the East Levee jogs to the west in order to avoid impacts to the church and nearby infrastructure (Figure F-E6). Co Rd 21 will be elevated to pass over the levee crest. This means raising the road roughly 15-feet so that it can pass over the levee crest at elevation 927.

F-E6.7.2 Country Road 16

Co Rd 16 runs east-west through SA1 and forms the interchange with I-29 at freeway exit 54 (Figure F-E1). There are three major modifications to Co Rd 16 that will occur because of the proposed flood control measures.

The first modification is at the intersection with the East Levee for SA1, just west of the freeway interchange (Figure F-E8). To maintain continuity for Co Rd 16, the roadway will need to be raised roughly 13 feet so it can pass over the levee crest at elevation 927. Similarly, the intersection with the South Levee for SA1 will mean raising Co Rd 16 roughly 10 feet. Figure F-E11 shows a closer view of the intersection of Co Rd 16 with the South Levee and the Diversion Channel.

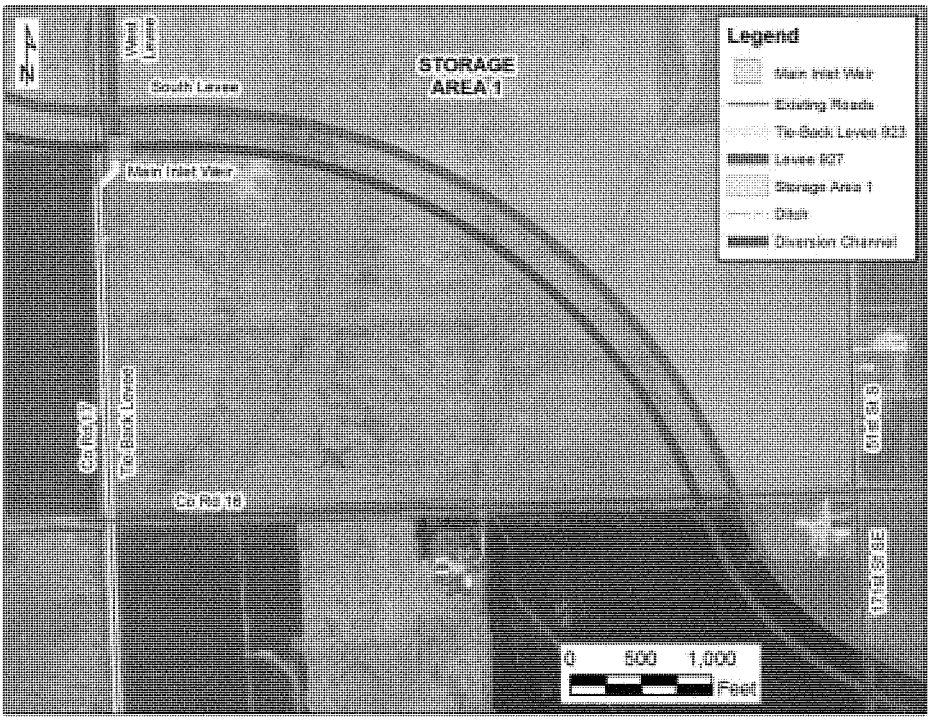

Figure F-E11 Vicinity Map for Main Inlet Weir and Diversion Channel Crossings

The third modification is the intersection of Co Rd 16 with the Diversion Channel (Figure F-E11). To maintain the continuity of Co Rd 16, a bridge will be constructed over the Diversion Channel. Figure F-E4 shows a cross section of the Diversion Channel and adjacent levees near the Co Rd 16 crossing. This reach of the Diversion Channel is not designed to convey or contain the design flood events. Therefore, it is neither necessary or practical to construct a bridge that is above the 100-year or 500-year events. Instead, the bridge deck will be kept near existing grades, and designed to be overtopped during major flood events.

Because peak conveyance of major floods will be primarily in the floodplain, there is no anticipated hydraulic effect on the overall flood control system. As such, the bridge was not included in the hydraulic model. A separate hydraulic analysis for the bridge will be necessary during final design.

F-E6.7.3 County Road 17

Co Rd 17 runs north-south along the west edge of what will be SA1 (Figure F-E1). There are three major modifications of Co Rd 17 that will occur as a result of the proposed flood control measures.

The first modification is the raising of roughly 3 miles of Co Rd 17 to at least elevation 923 to create the west Tie-Back Levee. The purpose of the levee is to limit the extent of the upstream staging area during major flood events. The 923 top elevation is slightly above the modeled peaks staging elevation for the 100-year and 500-year events. At the Diversion Channel the crest of the Tie-Back Levee will be increased 924, so that if the levee is ever overtopped, water will begin spilling over the road several hundred feet south of the Diversion Channel. The elevation of Co Rd 17 will extend approximately 2.5 to 3.0 miles to the south until the existing road is at or above elevation 923. The approximate extent of the Tie-Back Levee can be seen in Figure F-E1.

Figure F-E12 shows a cross section for the Diversion Channel along the alignment for Co Rd 17. The schematic showing the Tie-Back Levee is on the left overbank and shows how the Levee gets higher as it approaches the Diversion Channel.

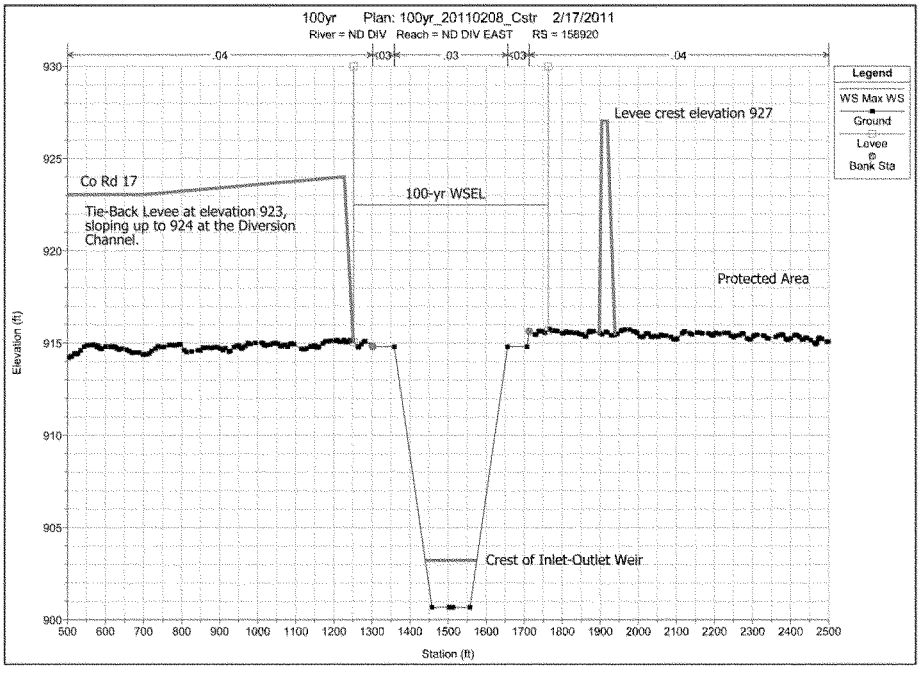

Figure F-E12 Diversion Channel cross-section at Tie-Back Levee

The second modification is a bridge crossing the diversion channel near the southwest corner of what will be SA1 (Figure F-E11). For preliminary design and cost estimating purposes, this bridge was considered an independent structure that would cross the Diversion Channel downstream of the Main Inlet Weir. As discussed above, the Main Inlet Weir is where staged waters upstream of the Protected Area are funneled into the main reach of the Diversion Channel that is designed to contain the design flood events within its banks. The Co Rd 17 Bridge will be constructed such that it is above the 100-year and 500-year water surface elevations and does not otherwise create a constriction for flows in the Diversion Channel. For final design it is anticipated that cost savings could be achieved by combining structural elements of the Main Inlet Weir and the Co Rd 17 Bridge.

The third modification to Co Rd 17 will be raising the roadway so that it clears the levee crest at elevation 927 on the north side of the Diversion Channel (Figure F-E12).

F-E6.7.4 Minor Roads

Construction of SA1 and adjacent flood control infrastructure will interrupt several minor roads. Table F-E3 lists the affected roads. It will be up to the local jurisdictions to determine if these roads will continue to be maintained within SA1. The locations of these roads are shown on in Figure F-E1.

Table F-E2 Effects on minor roads due to proposed flood control measures

Road	Impact
112th Ave S	Interrupted by East Levee Interrupted by West Levee
172nd Ave SE	Interrupted by Diversion Channel and South Levee Interrupted by North Levee
171st St Ave SE / 57th St S	Interrupted by Diversion Channel and South Levee Interrupted by North Levee
49th St SE	Interrupted by Diversion Channel and South Levee

Some of these roads may still need to be maintained in order to provide maintenance access to flood control structures. For example, 57th St. S off of Co Rd 14 would be a convenient access point for the North Outlet (Figure F-E5). It is assumed that fields within SA1 will continue to be farmed, which is an additional reason for maintaining minor roads within the storage area.

F-E7.0 SUMMARY

Storage Area 1 is a 4360 acre area on the north side of the Channel between the Wild Rice River and the Sheyenne River in North Dakota. It will be surrounded by a levee at elevation 927 which provides 4-feet of freeboard above the 100-year and 500-year flood events. It will provide over 55,000 acre-feet of storage during the 100-year and 500-year flood events.

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

County Roads 16 and 21 will be maintained through the storage area, but minor roads will be interrupted by the levee embankments.

RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT F – HYDRAULIC DESIGN COMPUTATIONS – DRAINS (FLOODPLAIN IMPACTS TO THE WEST)

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

By: Barr Engineering Co.

FINAL – February 28, 2011

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT F – HYDRAULIC DESIGN COMPUTATIONS – DRAINS (FLOODPLAIN IMPACTS TO THE WEST)

F-F1.0 DRAIN 14

A description of the drop structure at Drain 14 can be found in Appendix F. This portion of Exhibit F presents a comparison of water surface elevations at Drain 14 structure and design calculations of the drop structure.

F-F1.1 COMPARISON OF LPP DIVERSION CHANNEL AND DRAIN 14 HYDROLOGY

Flows, water surface elevations, and average velocities for the Diversion Channel at the Drain 14 drop structure for the 10-, 50-, 100-, and 500-year events are presented in Table F-F1. The location shown in the tables is Station 78110 which is where flows from Drain 14 discharge to the Diversion Channel. Water surface elevations in the Diversion Channel at the Drain 14 structure, in Drain 14 for local tributary flood events, and in Drain 14 for coincidental flood events, plotted against the cross section data, are presented in Figures F-F1 and F-F2.

Table F-F1 Phase 4 Hydrology in the Diversion Channel Downstream of the Drain 14 Structure (Station 78110)

Event (yr)	Flow (cfs)*	WSEL (ft)	Velocity (ft/s)*
10	7,545	887.00	1.93
50	19,675	895.53	2.36
100	24,785	898.36	2.46
500	25,074	901.10	2.08

* Flow and velocity information correspond to same time that the maximum water surface elevation occurs and may not be the maximum flow or velocity over the duration of the simulation

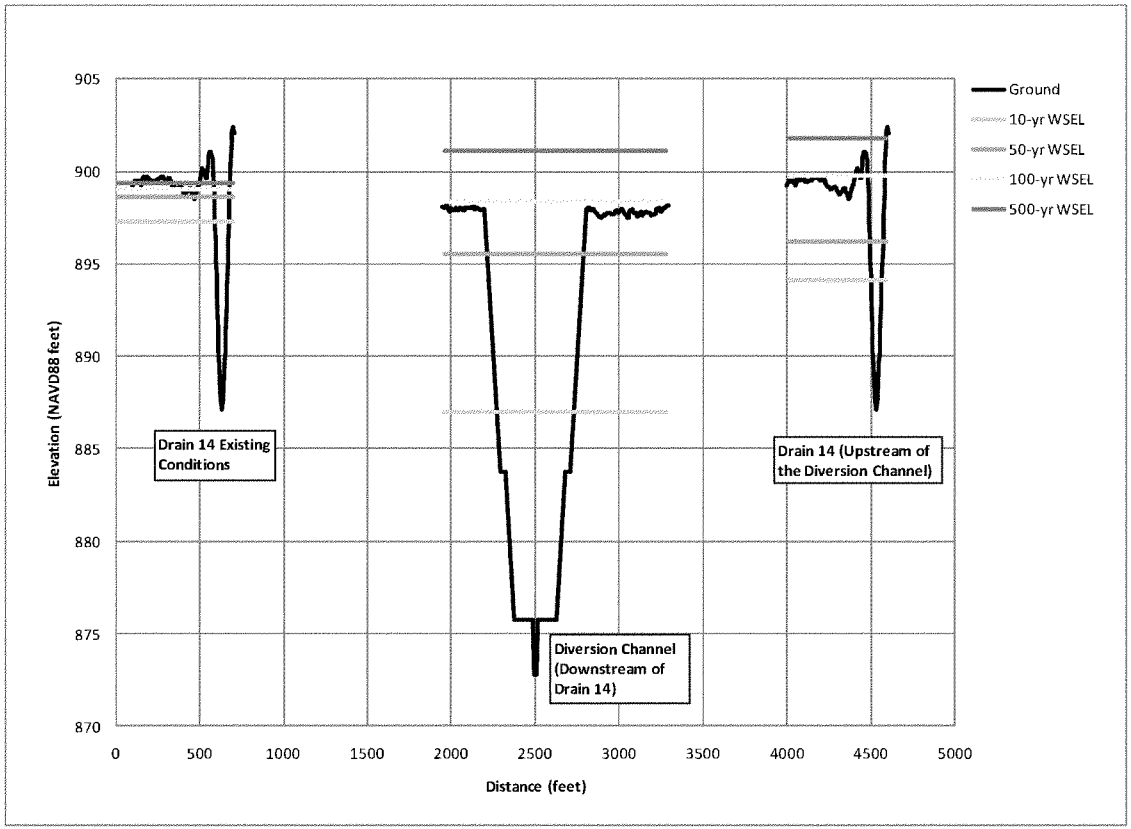


Figure F-F1 Phase IV Hydrology in the Diversion Channel at the Drain 14 Structure and in Drain 14

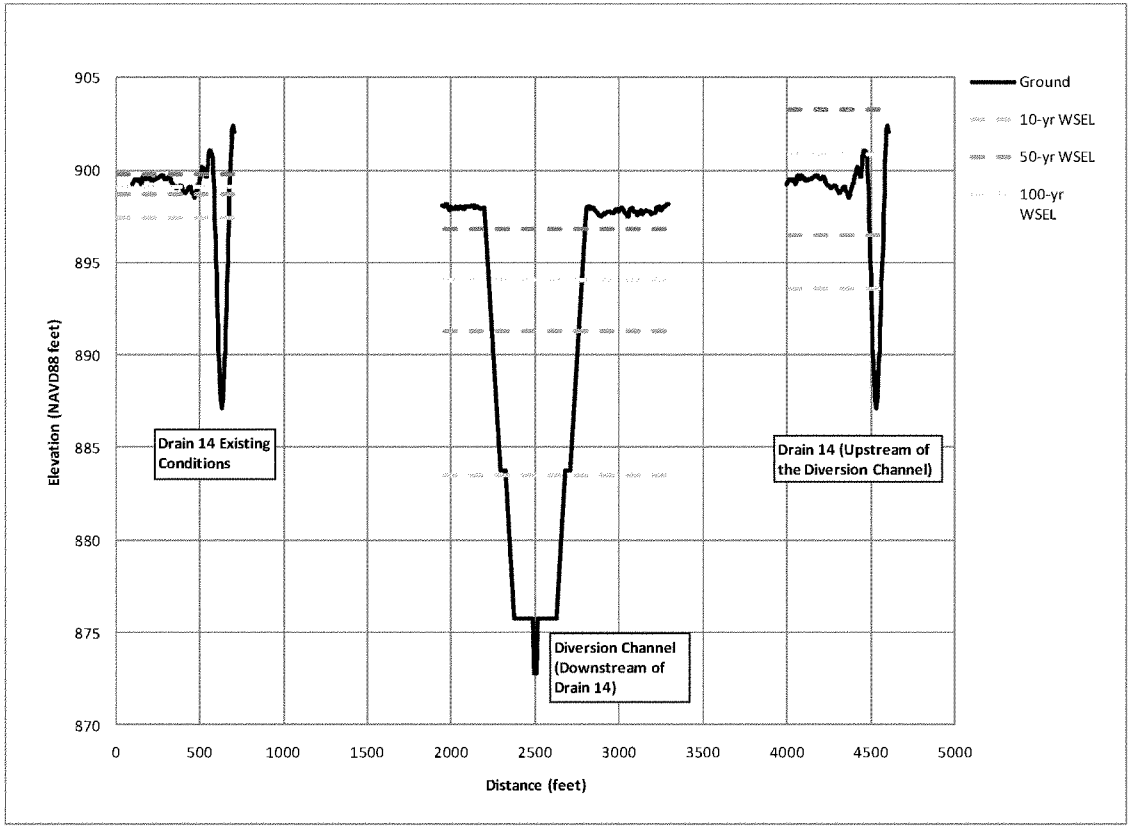


Figure F-F2 Phase IV Local Hydrology in the Diversion Channel at the Drain 14 Structure and in Drain 14

F-F1.2 DESIGN OF THE DRAIN 14 DROP STRUCTURE

The stepped concrete drop spillway was designed to pass the 500-year local event which is approximately 1,200 cfs larger than the 500-year coincidental event. For smaller more frequent events, less than the 50-year event, the water surface elevation in the Diversion Channel remains below the outlet from Drain 14. For larger flow events, the water surface elevation in the Diversion Channel is above the spillcrest of the Drain 14 drop structure, which results in an area of local staging on Drain 14. The water surface profiles upstream of the Drain 14 drop structure for existing conditions and With-Project conditions are shown in Figures F-F3 to F-F6 for the coincidental events and Figures F-F7 to F-F10 for the local events.

The width of the Drain 14 stepped drop structure is 30 feet. The drop structure is designed to handle the 500-yr local event with a 100 foot stilling basin. The stilling basin length is dependent on the length of the hydraulic jump. Dimensions of the Drain 14 stepped spillway are outlined in Table F-F2. The calculations used to determine the 30 foot width of the stepped drop structure are presented in Table F-F3.

Table F-F2 Drain 14 Stepped Spillway Parameters

Tributary bed invert elevation	888.0	Ft
Diversion channel bed elevation	873.26	Ft
Diversion channel 500-yr flood elevation (local on the Tributaries)	903.27	Ft
Diversion channel 10-yr flood elevation (local on the Tributaries)	883.47	Ft
10-yr flood elevation on Drain 14	893.55	Ft
WSEL difference between the 10-yr local event on Drain 14 and the Diversion Channel bed invert	20.3	Ft
Crest of steps	888.0	Ft
Number of steps	21	
Height of each step	0.7	Ft
Length of each step	1.5	Ft
Total height of steps	14.7	Ft
Total length of steps	31.6	Ft
Spillway width	30	Ft

Table F-F3 Flow Regime Over the Steps and Stilling Basin Parameters of the Drain 14 Stepped Spillway with a 30 Foot Width

Local Event (yr)	Flow in Drain 14 (cfs)	Critical Depth (ft)	Flow Regime	Water Depth Upstream of Hydraulic Jump (ft)*	Water Depth Down-stream of Hydraulic Jump (ft)*	Velocity Down-stream of Hydraulic Jump (ft/s)	Length of Hydraulic Jump (ft)
10	1,608	4.47	skim	1.5	10.1	5.3	60
50	2,463	5.94	skim	2.2	12.7	6.5	76
100	4,001	8.21	skim	3.4	16.4	8.2	98
500	4,280	8.58	skim	3.6	16.9	8.4	*102*

* Tailwater effects were not incorporated into the sizing of the stilling basin.

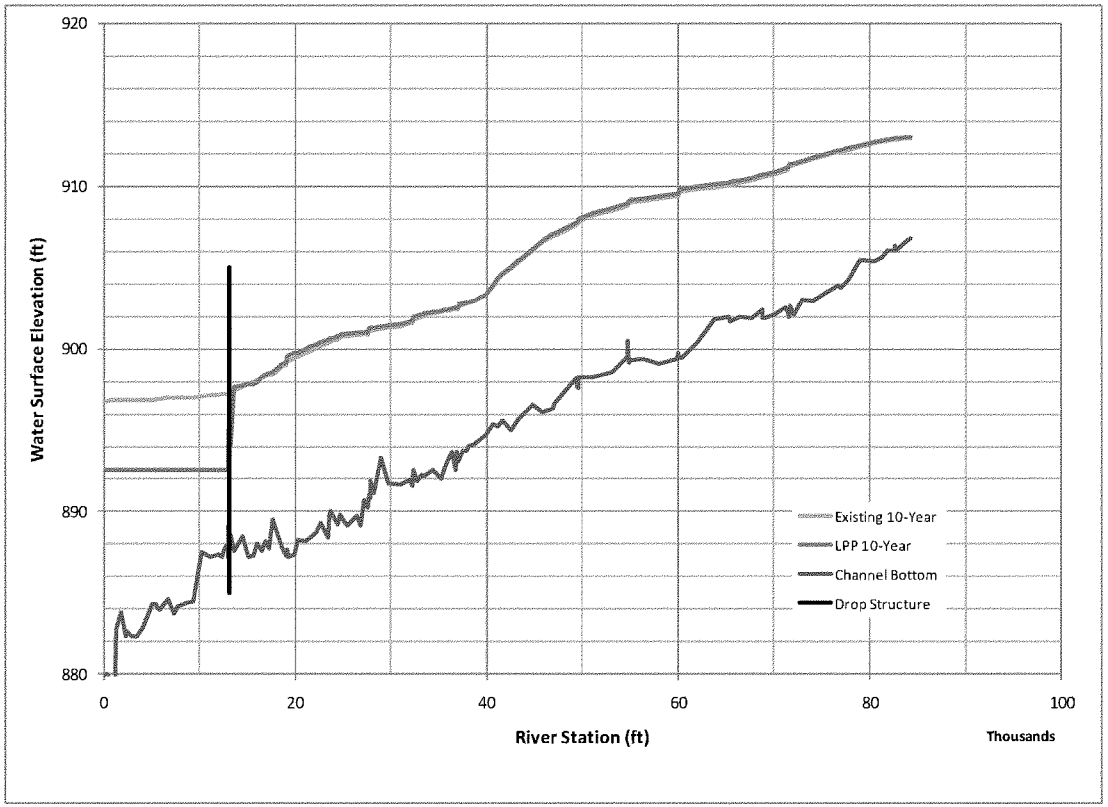
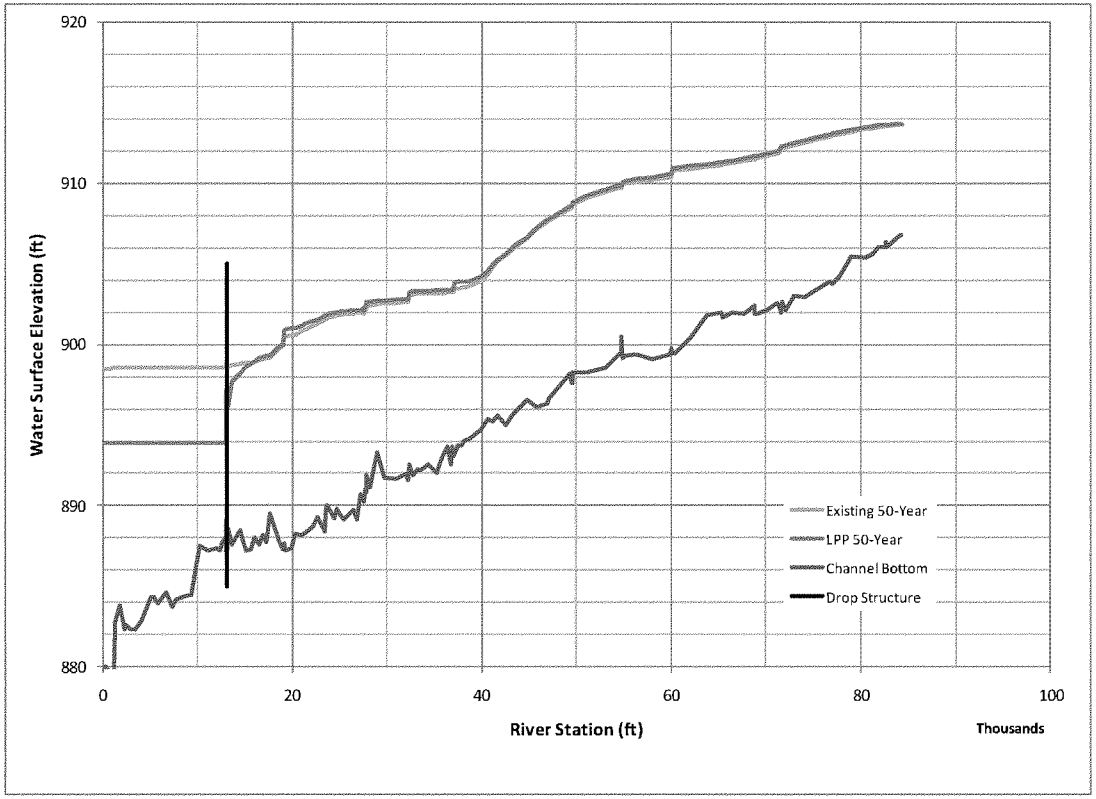


Figure F-F3 Water Surface Profile Comparison of Existing and With-Project Conditions for the 10-Year Coincidental Event



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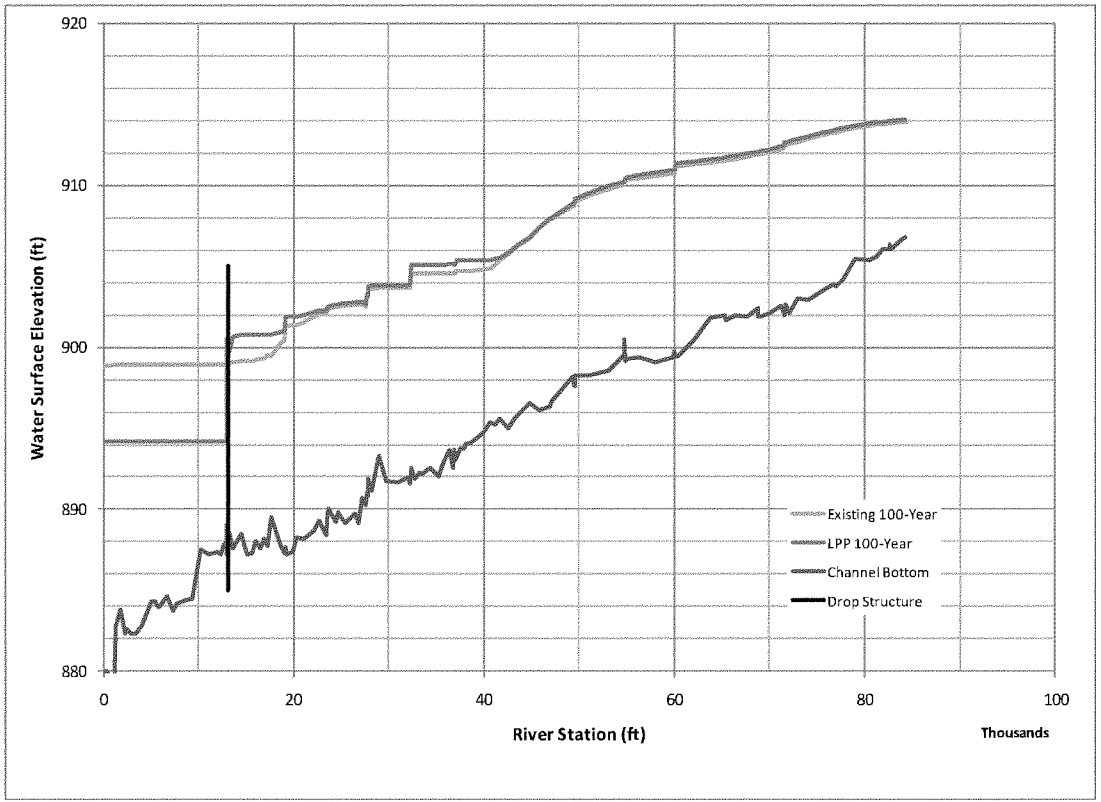
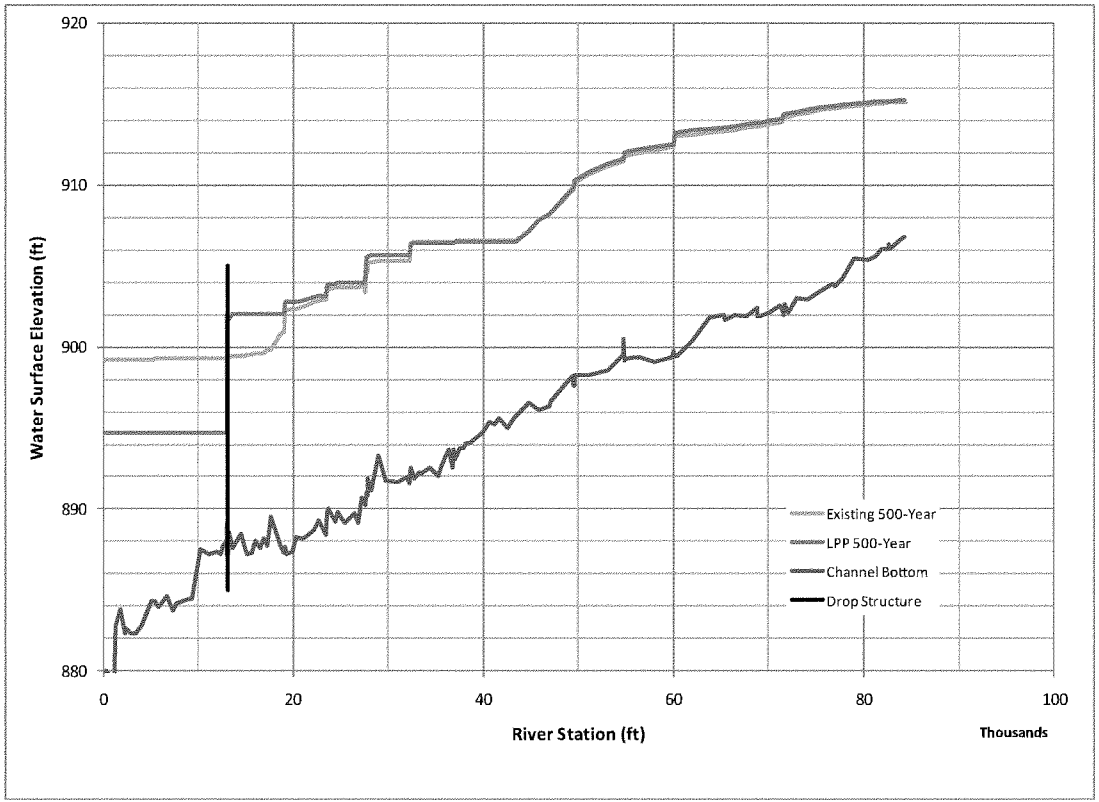


Figure F-F5 Water Surface Profile Comparison of Existing and With-Project Conditions for the 100-Year Coincidental Event



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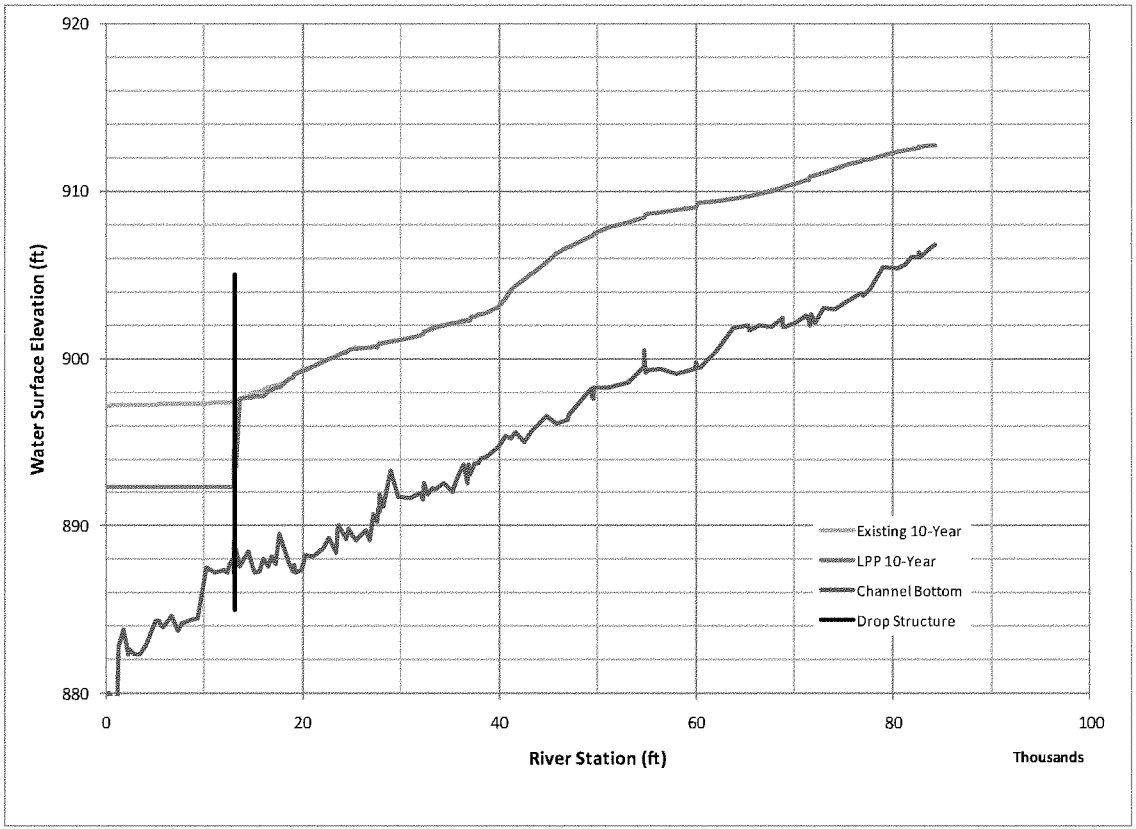

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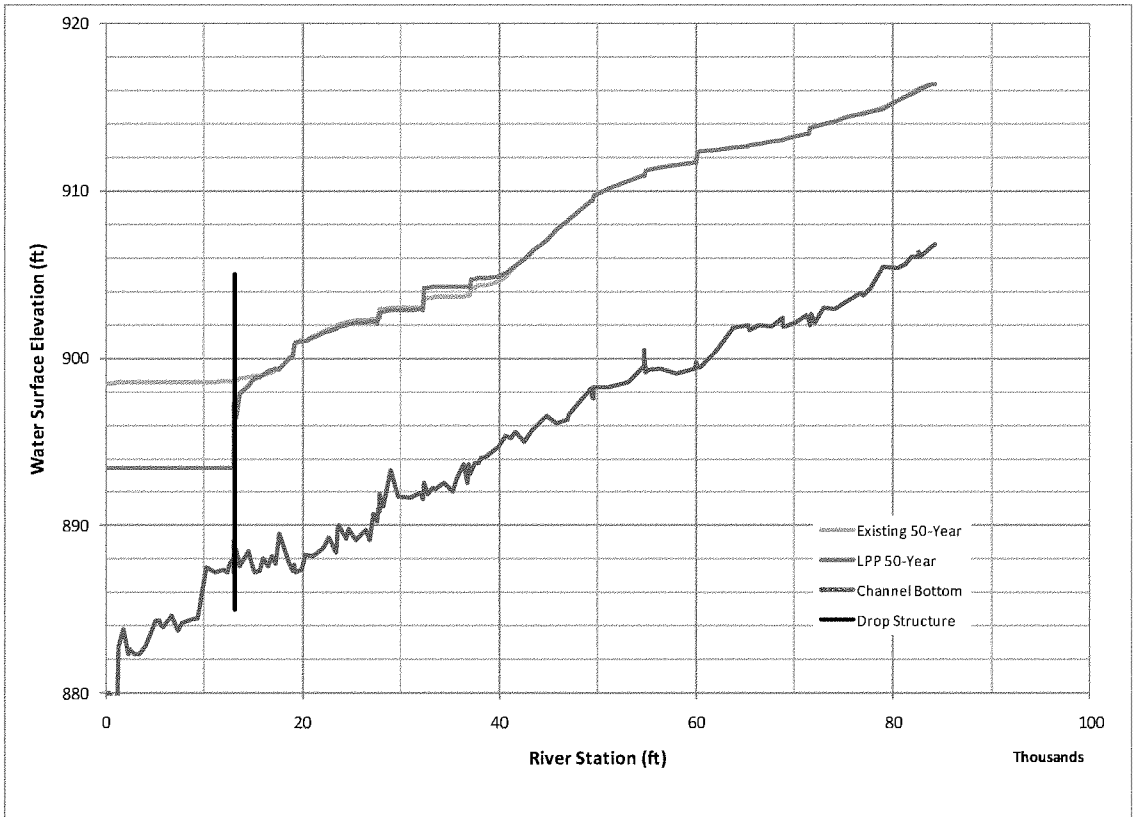

Figure F-F8 Water Surface Profile Comparison of Existing and With-Project Conditions for the 50-Year Local Event

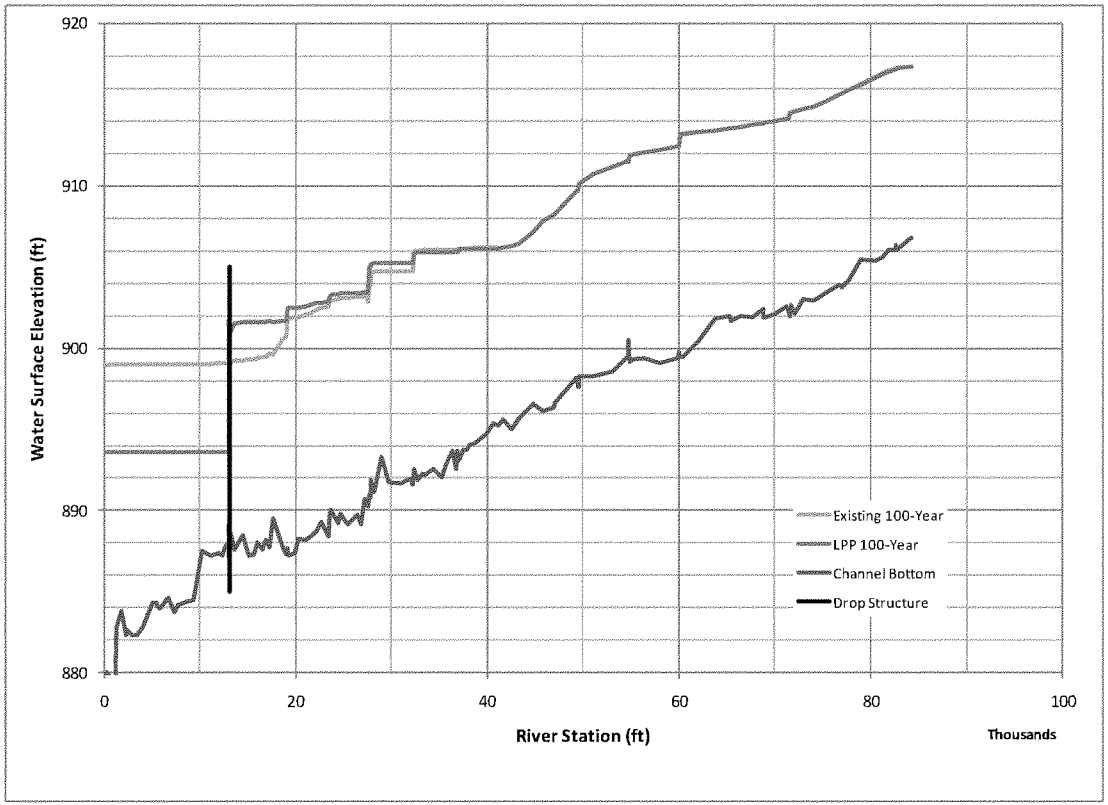

Figure F-F9 Water Surface Profile Comparison of Existing and With-Project Conditions for the 100-Year Local Event

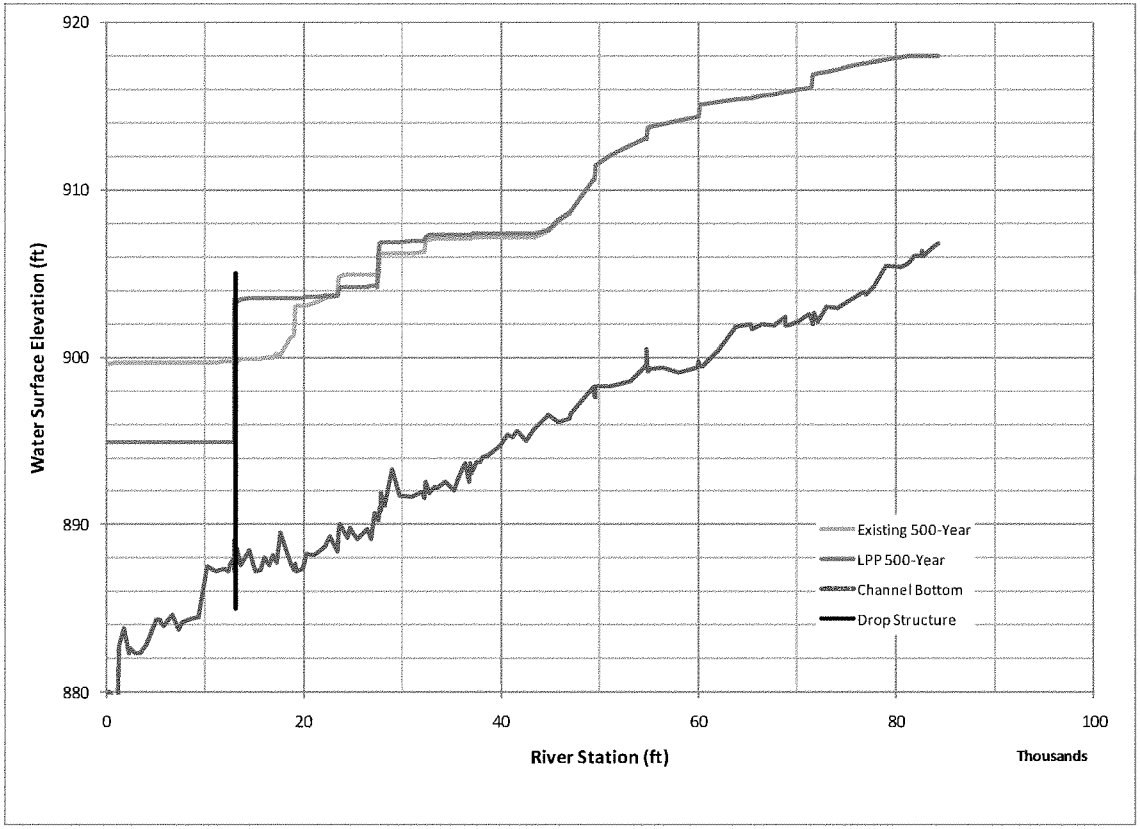

Figure F-F10 Water Surface Profile Comparison of Existing and With-Project Conditions for the 500-YearLocal Event

F-F2.0 LOCAL DRAINAGE

A description of the local drainage inlets to the LPP Diversion Channel can be found in Appendix A of the Phase 3 report submitted July 30th, 2010. This includes other major drainage ditches, in addition to Drain 14, such as Drain 21C and Drains 13 and 30 as well as other minor unnamed local drainage ditches. This portion of Exhibit F presents a summary of the local discharge locations into the Diversion Channel.

Local drainage inlets into the Diversion Channel were not resized as part of the Phase 4 analysis. 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. Impacts to the floodplain upstream of each of these drainage locations were verified and are discussed in Section F-F3.0. Table F-F4 includes a summary of the local discharge locations included in the Phase 4 HEC-RAS unsteady flow model.

Table F-F4 Summary of Local Drains that Discharge into the LPP Diversion Channel

HEC-RAS Station	Bank[1]	Structure Type	Diameter (ft)	Number of Culverts[2]
8322	L	Circular Culvert	5.5	1
50657	L	Circular Culvert	6	1
54245	L	Circular Culvert	4.5	1
64646	L	Circular Culvert	4.5	1
69218	L	Circular Culvert	3.5	1
73260	L	Circular Culvert	3.5	1
77104	L	Circular Culvert	4.5	2
89561	L	Circular Culvert	4	1
92149	L	Circular Culvert	5.5	2
95154	L	Circular Culvert	4	1
99921	L	Circular Culvert	3	1
101491	L	Circular Culvert	6	1
105869	L	Circular Culvert	3.5	1
108024	L	Circular Culvert	3.5	1
110137	L	Circular Culvert	6	3
114743	R	Circular Culvert	4	1
118893	R	Circular Culvert	3.5	1
119402	L	Circular Culvert	4.5	2
119407	R	Circular Culvert	4	1
123907	R	Circular Culvert	4	1
125423	L	Box Culvert 17'x25'	NA	1
130363	L	Circular Culvert	4	1
136016	L	Circular Culvert	3.5	1

[1] Facing downstream
[2] If more than one culvert is required, the station is defined as the center location of the culverts

F-F3.0 FLOODPLAIN IMPACTS TO THE WEST

As part of the Phase 4 analysis, impacts to the floodplain west of the Diversion Structure, between the Sheyenne River and Maple River were characterized to quantify the affect the Diversion Channel has on the existing floodplain. This portion of Exhibit F presents a summary of the impacts to the 100-year floodplain to the west of the Diversion Channel.

Based on discussions with the USACE the 100-year local event on the tributaries was selected to characterize impacts to the floodplain west of the Diversion Channel. The HEC-RAS unsteady flow model developed for Phase 4 was used with local 100-year hydrographs on the tributaries provided by the USACE for this analysis. Figure F-F11 presents how the floodplain west of the Diversion Channel between the Sheyenne River and the Maple River is impacted by the Project.

In general the majority of the floodplain located to the west of the Diversion Channel is not impacted by the Project. There are only 10 locations not immediately adjacent to the Diversion Channel where the floodplain is altered by more that 1ft.

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-year flood elevation in this area. Potential methods to mitigate this increase in flood elevations in this area could include

1. Constructing a backflow prevention device on the Drain 14 outlet
2. Grading an overflow to the Maple River
3. Grading an overflow to the local drains to the south

Methods for addressing impacts in this location were not analyzed during the Phase 4, but should be completed during final design.

Additional impacted areas along the Sheyenne River and Maple River where 100-year flood elevations decrease are due to grade control not being included in the HEC-RAS unsteady flow model. Grade control structures are included in the Project cost estimates, but were not included in the hydraulic modeling completed during Phase 4. Finally, other minor decreases in flood elevation immediately west of the Diversion Channel may be attributed to local drainage discharging immediately into the Diversion Channel. The HEC-RAS unsteady flow model developed for Phase 4 does not include a berm on the west side of the Diversion Channel to control how local drainage enters the Diversion Channel. This is a project feature that should be given additional consideration during final design.

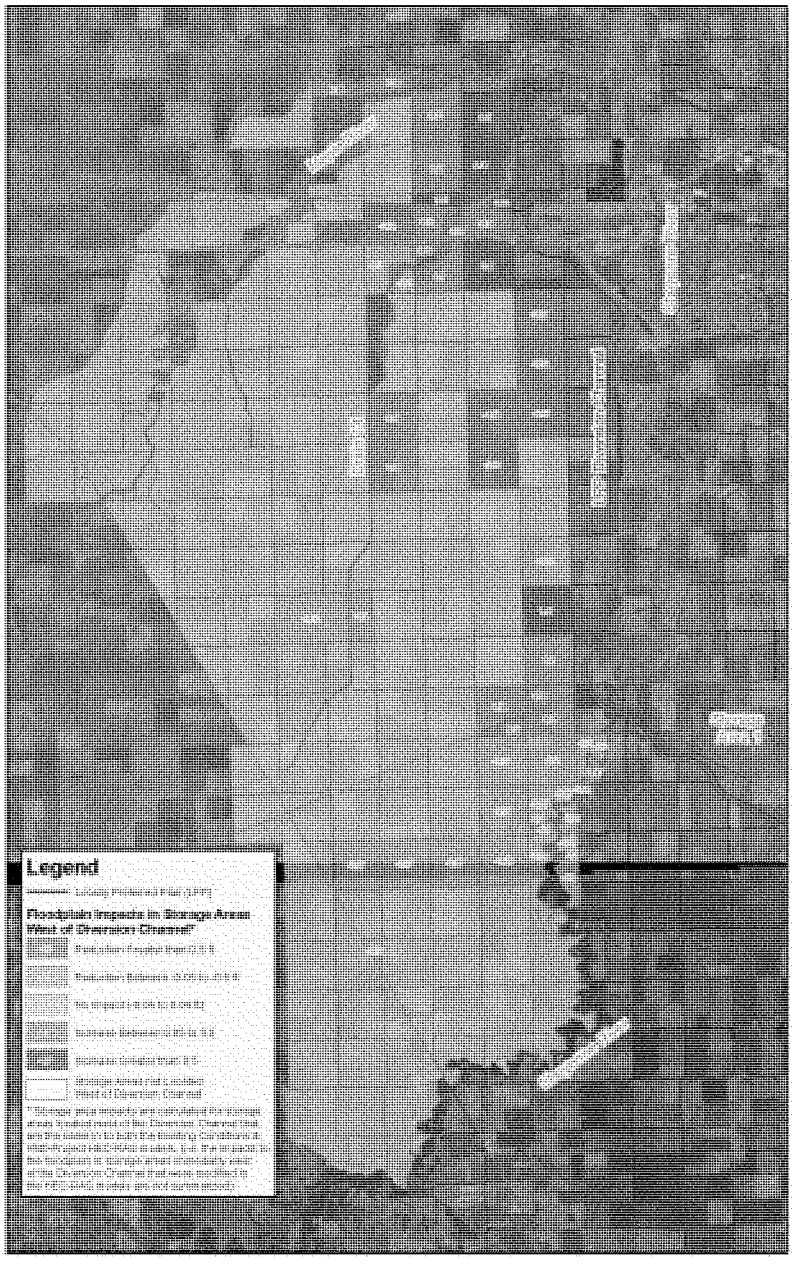

Figure F-F11 Floodplain Impacts West of the Diversion Channel between the Sheyenne River and Maple River for the 100-Year Local Tributary Event

RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT G – HYDRAULIC DESIGN COMPUTATIONS - FISH PASSAGE

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

By: Barr Engineering Co.

FINAL – February 28, 2011

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT G – HYDRAULIC DESIGN COMPUTATIONS – FISH PASSAGE

F-G1.0 FISH PASSAGE DESIGN

F-G1.1 INTRODUCTION

This exhibit describes the design of fish passages at the proposed control structures on the Red River, Wild Rice River, Rush River, and Lower Rush River. At the Wild Rice River and Red River control structures, the fish passages are designed to allow fish to move from the protected areas of the Wild Rice River and Red River to the upstream side of the respective control structures. At the Rush River and Lower Rush River control structures, the fish passages are designed to allow fish within the Diversion to pass into the upstream sides of the Rush and Lower Rush Rivers, as well as to allow low flows in the Rush and Lower Rush Rivers to drain into the Diversion Channel when water surface elevations are below the invert of the drop structures. Fish passages have not been designed for the Sheyenne or Maple Rivers as the aqueduct allows for adequate fish passage. Fish passages over the Main Inlet (Locally Preferred Plan, or LPP), and diversion inlet weir (Federally Comparable Plan, or FCP) do not include additional structures.

At the Red River Control Structure (FCP and LPP) and Wild Rice Control Structure (LPP), fish passages have been designed to function between the 5-year and 50-year events on the Red River. The Red River and Wild Rice River control structures are designed such that fish may pass through the primary control structures during events less than the 5-year event on the Red River. For larger events, fish passage structures next to the control structures are necessary, as the velocities through the control structures are expected to inhibit upstream fish movement. At the Rush River and Lower Rush River drop structures, fish passage will function for flows less than approximately that which is coincidental to the 10-year event on the Red River.

This exhibit describes the design of preliminary fish passages at several locations along the diversion channel. The design is similar for all fish passage locations, although the number, length, and elevations of fish passage structures vary between sites.

F-G1.2 DESIGN CONSIDERATIONS

The fish passage designs presented in this section are guided by several design considerations based on discussions with the USACE and the USACE's Lock and Dam 22 Fish Passage Improvement Project Implementation Report – Appendix H. These design goals include:

- Design incorporating pool-riffle sequences
- Maximum velocity through riffles of 6 feet per second

- Average velocity through pools of ~1.5 feet per second
- Minimum depth of 5 feet in pools
- Average slope of 1 to 3 percent
- Minimum depth of 1 foot at entrance to fish passage
- Desired flow of 1-2 percent of the flow through the control structure
- Downstream invert ~1.5 feet below the 5-year tailwater elevation

The designs presented in this section meet these goals with the exception of the desired flow through the fish passage structure. The extreme variability of the upstream water surface elevation creates a wide range of flows through the fish passage structures, including flows less than 1 percent of the total flow as well as flows greater than 2 percent of the total flow through the control structure.

At the Rush and Lower Rush River, the fish passage was also designed to allow low flows in the Rush and Lower Rush River to pass into the Diversion Channel without cresting the drop structures (preventing the pooling of water at these structures).

The above parameters have remained unchanged from the Phase 3 analysis and design. However, changes in the general design and modeling of the diversion have required an update in the fish passage design. The changes in the fish passage design can be attributed to the following:

- An increase in the diversion bottom elevation due to the bottom width change from 100-ft to 250-ft,
- Implementation of a new hydrologic model (unsteady flow modeling and inclusion of storage areas) has changed the predicted water surface elevations,
- Greater upstream staging and maintained water surface elevations in the protected area have created larger water surface elevation differences across the control structures

F-G1.3 DESIGN CONCEPT – RED RIVER AND WILD RICE RIVER FISH PASSAGE

The fish passage design at each control structure includes one or more parallel sequences of pools and riffles (see Appendix F), referred to in this section as "channels". A one foot drop in elevation occurs in the riffle between pools. The number of pools and riffles is based on the difference in headwater and tailwater elevations at each structure. The number of channels necessary is based on the variability in headwater elevations. The headwater and tailwater elevations used for fish passage design are presented in Table F-G1.

Table F-G1 Headwater and Tailwater Elevations for Fish Passage Design

Plan	Location/Control Structure	10-year Event (on Red River)			50-year Event (on Red River)		
		Headwater (ft)	Tailwater (ft)	HW-TW (ft)	Headwater (ft)	Tailwater (ft)	HW-TW (ft)
FCP	Red River	900.17	898.60	1.57	909.33	899.43	9.90
LPP	Red River	916.29	904.40	11.9	920.86	905.15	15.7
LPP	Wild Rice River	915.61	901.43	14.2	920.84	900.73	20.1

Multiple fish passage channels are necessary at the Red River Control Structure and Wild Rice Control Structure to operate between the 5-year and 50-year events due to headwater elevations that vary by several feet. As the headwater rises, the velocity in the channel increases. At some point, the headwater results in velocities too high for fish passage. At this elevation, flow through the channel is prevented by closing gates at the upstream end of the channel, and flow into another channel with a higher upstream invert elevation is allowed by opening another set of gates. Thus, each channel is designed to operate for a range of headwater elevations. The number of channels may be minimized by maximizing the range of headwater elevations over which each fish passage channel functions. Figure F-G1 shows the water surface elevation and velocity operation ranges for the complete (two fish passages) Phase III design at the Red River of the North control structure.

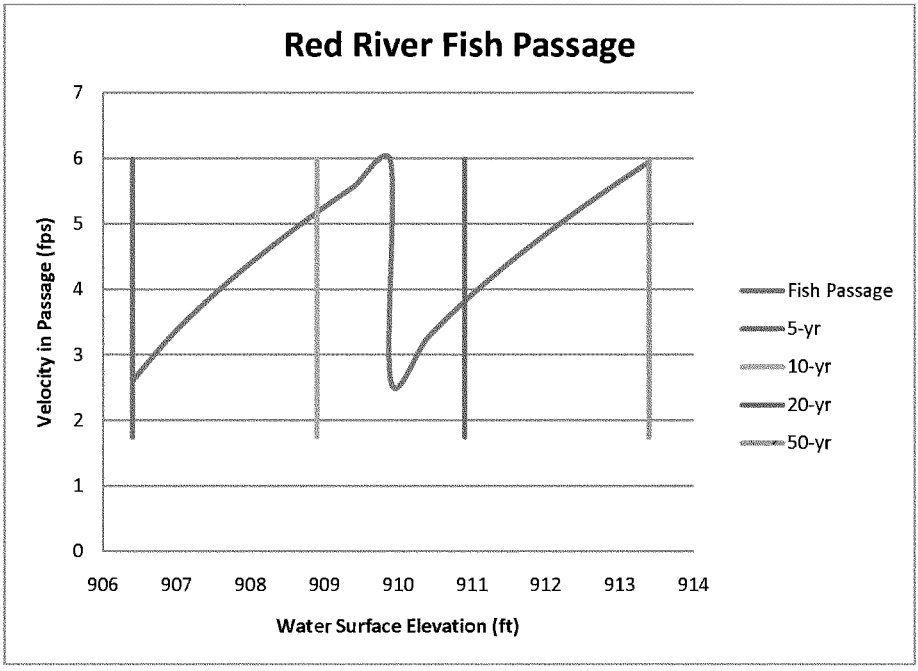

Figure F-G1 Phase III Fish Passage Operation at the Red River of the North LPP Control Structure

This design concept requires the operation of gates at some structures during flood events. At various flood stages, gates will be either open or closed (the system is not designed such that gates remain partially open, as is the case at the Red River Control Structure).

F-G1.3.1 Gate Design

At the upstream end of the Red River and Wild Rice River fish passage channels, a gated area connects the upstream river reach to a pool downstream of the gates (see Appendix F). The preliminary gate design includes three gates 10 feet wide by 5 feet tall, for a total gated area of 150 square feet at the upstream end of each fish passage channel. The gate height of 5 feet is based on the maximum headwater at the upstream end of the fish passage for which velocities in the fish passage channel remain acceptable while the gates are open.

F-G1.3.2 Fish Passage Inlet Invert Design

The invert elevations for each set of fish passage gates are listed in Table F-G2 below:

Table F-G2 Fish Passage Gate Invert Elevations by Structure

Plan	Location/Control Structure	Fish Passage Channel	Gate Invert (ft)	Number of Gates	Gate Width (ft)	Gate Height (ft)
FCP	Red River	1	899.2	3	10	5
FCP	Red River	2	902.7	3	10	5
FCP	Red River	3	906.2	3	10	5
LPP	Red River	1	912.9	3	10	5
LPP	Red River	2	916.4	3	10	5
LPP	Wild Rice River	1	912.9	3	10	5
LPP	Wild Rice River	2	916.4	3	10	5

Determination of the above Phase 4 inlet inverts involved a lengthy optimization process. Three alternatives were evaluated during this optimization process:

- Alternative 1: Inlet invert based on Phase 3 guidelines (1-ft below the 5-year event water surface elevation).
- Alternative 2: Inlet invert based on operating during the peak water surface elevation of the 50-year design storm with two fish passages.
- Alternative 3: Inlet invert set at the approximate water surface elevation for which the gates become fully opened during the receding limb of the hydrograph.

Alternative 1 represents the design approach used in Phase 3, in which it is assumed that fish passage will be maintained through the control structure until flows reach the 5-year event, at which point the control structure gates will be closed, necessitating a dedicated fish passage. Therefore, fish passage is designed to begin at the 5-year event and maximize fish passage through larger events. In Phase 4, it is estimated that larger flood events may require shutting the control structure gates earlier in the hydrograph (before water surface elevations reach the 5-year event). Thus, fish passage may be required at lower elevations. Alternative 2 lowers the invert of the lower fish passage relative to Alternative 1 to minimize this impact, at the cost of providing no fish passage above the 50-year event. As indicated by Phase 4 HEC-RAS modeling, the control structure gates may be opened following the peak water surface elevation of larger flood events but prior to the water surface elevation falling below that of the 5-year event. Thus, there may be periods along the falling limb of the hydrograph were fish passage is inhibited. Alternative 3 above places the invert of the lowest fish passage at an elevation to maximize fish passage along the falling limb of the hydrograph.

F-G1.3.2.1 Alternative 1 Red River of the North Fish Passage Inlet Invert Design

For Alternative 1, the inlet and outlet inverts were set at 1-ft and 1.5-ft, respectively, below the 5-year water surface elevation. Since the modeling conducted in Phase 4 does not include the 5-year event, this event water surface elevation was chosen to be 1-ft

below the 10-year event for this design. Based on this assumption, the invert of the first fish passage on the Red River is at 914.3-ft. The downstream invert is at 901.9-ft. The fish passage becomes operable when the water depth reaches one foot (915.3-ft). The fish passage becomes inoperable and the inflow gate must be shut when the water depth reaches 4.5-ft (918.8-ft). This span allows one fish passage to cover both the 5-year and 10-year design floods.

To reach the 50-year flood event a second passage is needed. The upstream invert of the second passage will be located one foot below the peak elevation for the first passage (917.8-ft). The downstream invert of the second passage is designed conservatively and will be 1.5 feet below the 5-year flood event (901.9-ft). Similarly to the first passage, once the water surface upstream reaches a depth of 4.5-ft (922.3) the velocities in the channel become too high for fish passage. The water surface elevation for the 50-year flood (920.9-ft) lies within the operable range of the second passage. Figures F-G2 and F-G3 present the water surface elevation hydrographs for the 10-year and 50-year storms respectively and the inlet invert elevations for the two fish passage structures at the diversion control structure.

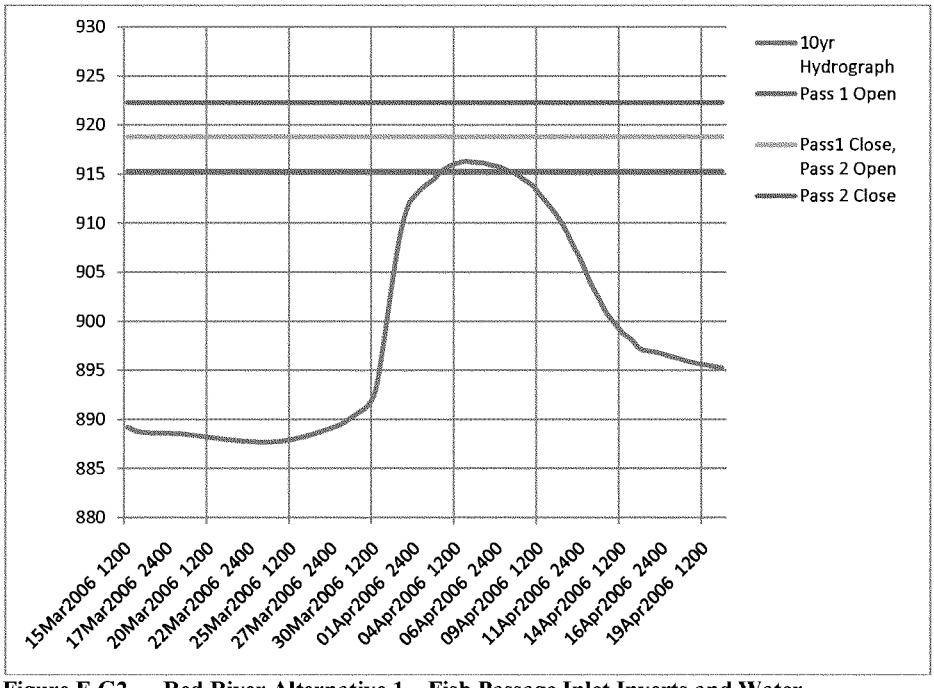

Figure F-G2 Red River Alternative 1 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Red River of the North Control Structure for the 10-year Event

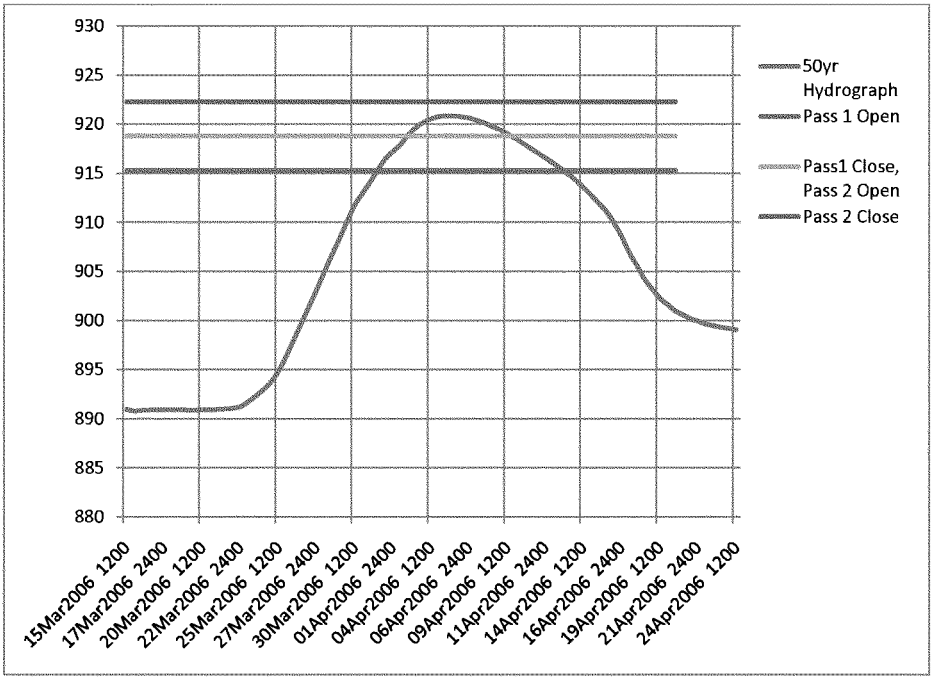

Figure F-G3 Red River Alternative 1 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Red River of the North Control Structure for the 50-year Event

The Phase 3 design approach with the inlet invert at the 5-year storm allows for fish passage for the 5-year, 10-year, and 50-year flood event peaks, as shown in Figures F-G2 and F-G3. However, the performance of the fish passage for events outside of this range, and how they perform for the entire storm hydrograph, should be determined. To evaluate this, three time parameters are defined; rising limb, peak, and falling limb. The rising limb time parameter represents the duration of time along the hydrograph from the control structure gate closing (fish passage is stopped) to the start of fish passage operation (water surface elevation reaches 1-ft above the fish passage invert). The peak parameter represents the duration of time the hydrograph exceeds the maximum water surface elevation for fish passage on the upper passage. The falling limb parameter represents the duration of time from the water surface elevation occurring below the first fish passage invert to the water surface elevation when the control structure gates are fully opened.

Evaluating the above three parameters is further complicated by the gate operation. When a large, event is predicted (greater than 9,600 cfs), the gates must be closed on the rising

limb of the hydrograph (approximately 7-10 days before the 9,600 cfs discharge is reached at the Fargo gauge). In the HEC-RAS models, the gates have been closed slowly over the course of several days to improve stability in the model. In reality, the gates could be closed within a shorter duration. However, the data presented in Table F-G3 for the rising limb closure duration is based on the approximate inflection point for the rising limb of the synthetic, modeled hydrograph. This represents a worst case scenario. It is also important to note that the actual duration of fish passage closure for the rising limb will vary depending on the shape and peak flow of the hydrograph.

The peak and falling limb fish passage closure durations are easier to predict. The peak is simply determined by the maximum fish passage water surface elevation and the hydrograph peak. For the falling limb fish passage closure duration, the models can be used to predict the time of gate opening and the hydrograph will provide the duration of closure. Table F-G3 presents the results of the fish passage closure analysis in terms of number of days and approximate days per year (days closed multiplied by flood event probability).

Table F-G3 Red River Alternative 1 – Duration in Days of Fish Passage Closure Due to Velocities Exceeding 6-fps

	Rising Limb		Peak		Falling Limb		Total	
Flood Event	**Days Out[(a)]**	**Days Per Year Out**	**Days Out**	**Days Per Year Out**	**Days Out**	**Days Per Year Out**	**Days Out**	**Days Per Year Out**
10	9	1	0	0	10	1	19	2.0
50	10	<1	0	0	10	<1	20	<1
100	10	<1	4	<1	10	<1	24	<1
500	7	<1	1	<1	0	0	8	<1

(a) Please see the preceding paragraphs for a discussion about the rising limb closure quantification

Note: The duration of closure is shorter for the 500-year event than compared to the 100-year event due to the narrowing of the hydrograph for the 500-year event, i.e. rising and falling limbs occur closer together. Please see Figure F-G4

From Table F-G3, it is evident that fish passage would not be possible for 10 days on the falling limb of the hydrograph for the 10-year and 50-year events. The 100-year event will have four days when fish are unable to pass during the peak. Due to the narrowing of the hydrograph for the 500-year event, fish pass will only be restricted for 1 day at the peak. Figure F-G4 presents a comparison of the 100-yr water surface elevation hydrograph and the 500-yr water surface elevation hydrograph at the Red River of the North control structure. The plot provides a clear view of the difference in the receding limbs of the hydrographs which produces the decrease in closure days for the 500-yr event in Table F-G3. This plot is characteristic of only the project conditions. Finally, the gates are expected to opened near the peak in the hydrograph for the 500-year storm, so fish passage is not expected to be restricted for the falling limb (peak velocities through the gates should be approximately 5.4 fps).

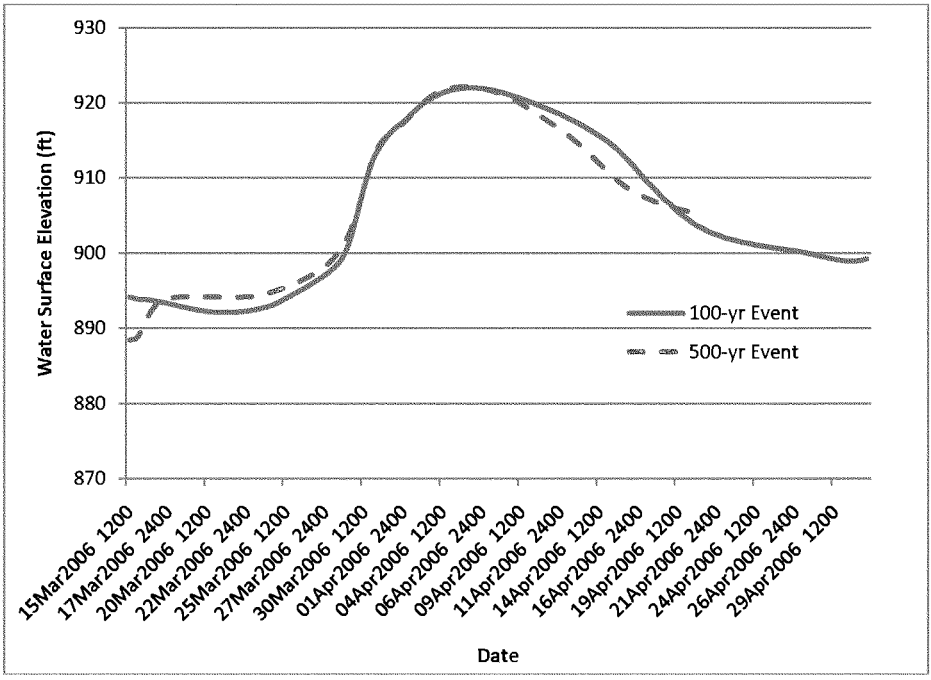

Figure F-G4 Comparison off RRN Control Structure Water Surface Elevation for the 100-year Event and 500-year Event

Alternatively, it is desirable to know how often each fish passage will be open. From this information it is possible to compare the advantage of allowing fish passage for a set duration to the associated cost of this infrastructure. Table F-G4 presents the duration in days the Alternative 1 fish passage (both fish passage 1 and 2) will be open. For instance, a 10-year event will only allow fish passage for a total of 4 days in passage 1. This appears short, however it is also providing passage for events down to the 5-year event. The second passage will provide fish movement for 7 days during the 50-year and 100-year events.

Table F-G4 Red River Alternative 1 – Duration in Days Fish Passage is Operational

Flood Event	Pass 1		Pass 2	
	Days in Operation	Days Per Year in Operation	Days in Operation	Days Per Year in Operation
10	4	<1	0	0
50	6	<1	7	<1
100	5	<1	7	<1
500	4	<1	6	<1

F-G 1.3.2.2 Alternative 1 Red River of the North Fish Passage Frequency Analysis

Detailed records exist of the Red River of the North discharges at Fargo, ND for each year back to the early 20th century. Correlating the maximum operable water surface elevation of each fish passage to a discharge at the Red River of the North Fargo, ND gauge (based on exceedance probability) allows for the quantification of the fish passage's closure frequency using the historic flow record at Fargo. Table F-G5 provides the upstream operational WSEL's for each fish passage and correlates it with a discharge at the Red River of the North Fargo, ND gauge. This was accomplished by noting the peak WSEL's for each Phase 4 flood event (10-year, 50-year, 100-year, 500-year) and pulling the corresponding peak discharge for the Fargo, ND gauge from the HEC-RAS models. An exponential function was fit to this data and used to interpolate the Fargo, ND discharge for each fish passage operational WSEL range. By correlating the fish passage operation ranges with a discharge at Fargo, ND a frequency analysis can be conducted with the historic data. Table F-G5 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) WSEL's for the 1901 to 2009 historic record. Figure F-G5 is a plot of the fitted exponential model for the Red River of the North fish passage WSEL's and the Fargo, ND gauge.

Table F-G5 Red River Alternative 1 – Fish Passage Operational Range WSEL, Corresponding Interpolated Discharges, and Flood Frequency Data for 1901-2009 at the Red River of the North Fargo, ND Gauge

	Fish Passage Open El. (ft)	Fish Passage Closed El. (ft)	Pre-project Q 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 Q 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 yr)	913.7	917.2	12653	10	116	17551	6	46
Pass 2 (~20-50 yr)	917.2	920.7	17551	6	46	24346	3	14
Pass 3 (>50 yr)	920.7	924.2	24346	3	14	33772	0	0

(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.

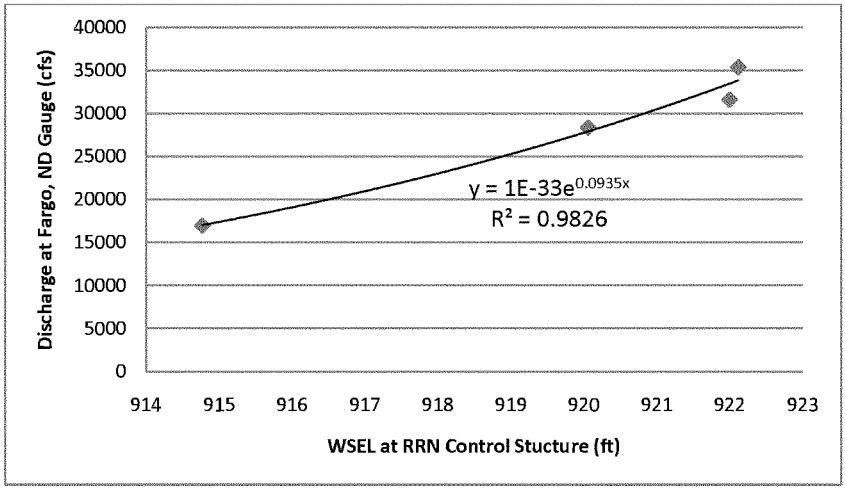

Figure F-G5 RRN Control Structure Water Surface Elevation versus Discharge at the RRN Fargo, ND Gauge

It is important to note for this analysis, there were only four data points available for the curve fitting. In addition, the lowest and highest water surface elevations in Table F-G5 extend outside of the available data range.

F-G1.3.2.3 Alternative 2 Red River of the North Fish Passage Inlet Invert Design

For Alternative 2, the maximum water surface elevation for the second passage was set at the peak of the 50-year event. Based on this assumption, the invert of the first fish passage on the Red River is at 912.9-ft. The downstream invert is at 901.9-ft (1.5-ft below the 5-year tailwater elevation). The fish passage becomes operable when the water depth reaches one foot (913.9-ft). The fish passage becomes inoperable and the inflow gate must be shut when the water depth reaches 4.5-ft (917.4-ft). This span allows one fish passage to cover both the 5-year and 10-year design floods. This design also allows for operation at water surface elevations less than the 5-year event. This is beneficial as larger flood events would require shutting the control structure before the water surface elevation rises to the 5-year event. In addition, the fish passage will be more flexible for use during smaller, more frequent events between 9,600 cfs (approximately the 3.5-year event) and the 5-year event.

The upstream invert of the second passage will be located one foot below the peak elevation for the first passage (917.4-ft). The downstream invert of the second passage is designed conservatively and will be 1.5 feet below the 5-year flood event (901.9-ft). Similarly to the first passage, once the water surface upstream reaches a depth of 4.5-ft (920.9) the velocities in the channel become too high for fish passage. The water surface elevation for the 50-year flood (920.9-ft) lies within the operable range of the second passage. Figures F-G6 and F-G7 present the water surface elevation hydrograph for the

10-year and 50-year storms respectively and the inlet invert elevations for the two fish passage structures at the diversion control structure.

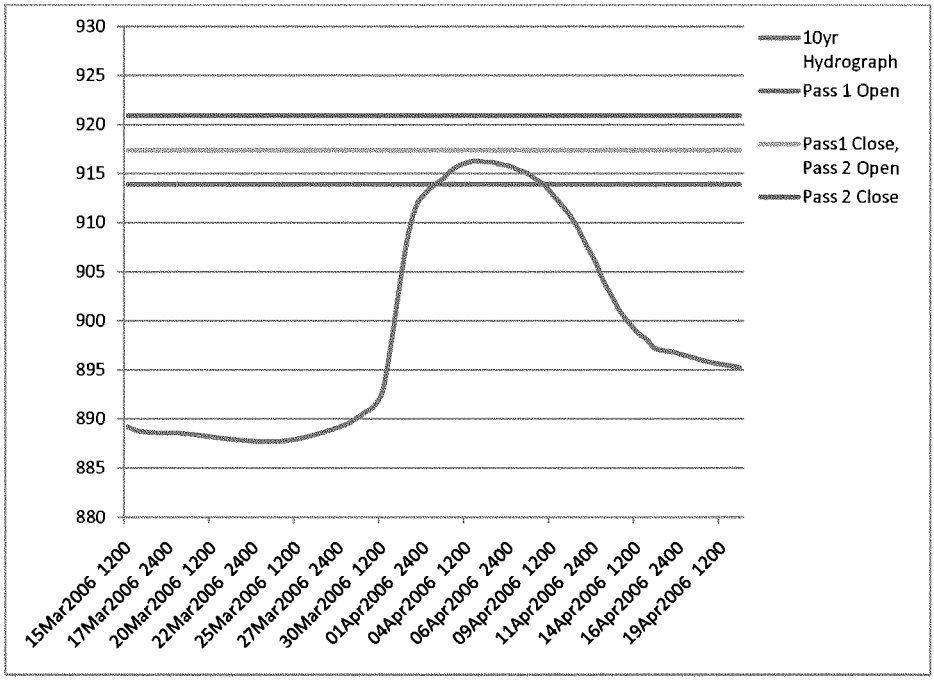

Figure F-G6 Red River Alternative 2 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Red River of the North Control Structure for the 10-year Event

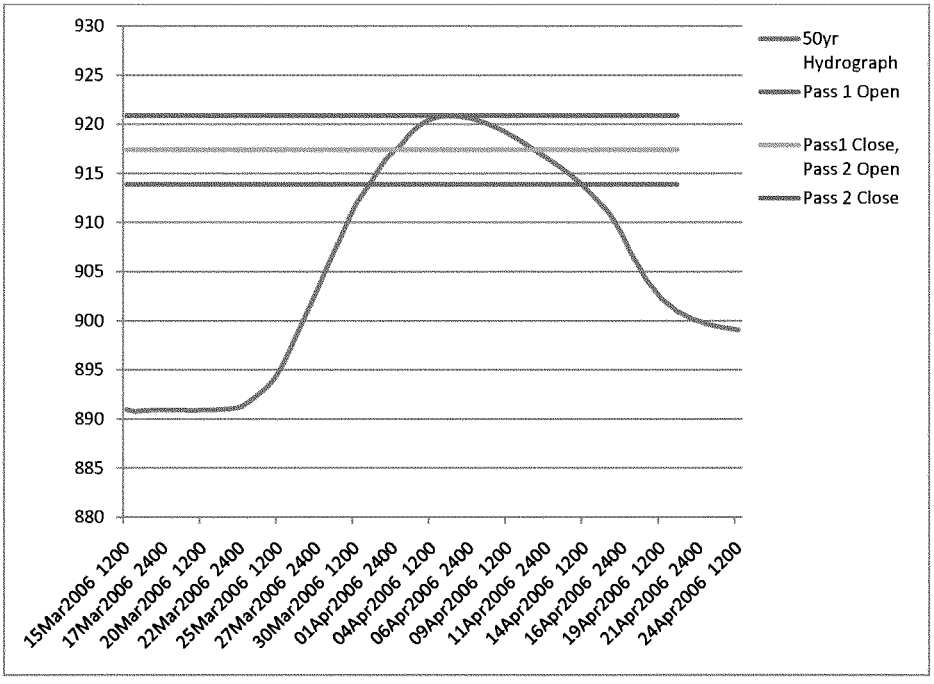

Figure F-G7 Red River Alternative 2 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Red River of the North Control Structure for the 50-year Event

As discussed for Alternative 1, Table F-G6 presents the results of the fish passage closure analysis in terms of number of days and approximate days per year (days closed multiplied by flood event probability).

Table F-G6 Red River Alternative 2 – Duration in Days of Fish Passage Closure Due to Velocities Exceeding 6-fps

Flood Event	Rising Limb		Peak		Falling Limb		Total	
	Days Out[(a)]	Days Per Year Out	Days Out	Days Per Year Out	Days Out	Days Per Year Out	Days Out	Days Per Year Out
10	8	1	0	0	9	1	17	2
50	10	<1	0	0	9	<1	19	<1
100	9	<1	8	<1	9	<1	26	<1
500	6	<1	5	<1	0	0	11	<1

(a) Please see the Red River of the North Alternative 1 design discussion preceding Table F-G3 about the rising limb closure quantification.

Note: The duration of closure is shorter for the 500-year event than compared to the 100-year event due to the narrowing of the hydrograph for the 500-year event, i.e. rising and falling limbs occur closer together. Please see Figure F-G4.

Table F-G6 shows that fish passage will not be possible for nine days on the falling limb of the hydrograph for the 10-year, 50-year, and 100-year events. The 100-year event will have eight days when fish are unable to pass during the peak. Due to the narrowing of the hydrograph for the 500-year event, fish passage will only be restricted for 5 days at the peak (See explanation in Section F-G1.3.2.1). Also, the gates are expected to opened near the peak in the hydrograph for the 500-year event, so fish passage is not expected to be restricted for falling limb (peak velocities through the control structure gates should be approximately 5.4 fps). Since this passage has been lowered as compared to Alternative 1, it has slightly fewer days closed. In addition, this passage will have fewer days closed for the more frequent events smaller than the 10-year flood.

Alternatively, it is desirable to know how often each fish passage will be open. From this information it is possible to compare the advantage of allowing fish passage for a set duration to the cost of the associated infrastructure. Table F-G7 presents the duration in days the Alternative 2 fish passage (both passages 1 and 2) will be open. For instance, a 10-year event will only allow fish passage for a total of 6 days in passage 1. This appears short, however it is also providing passage for events down to the 5-year event. The second passage will only provide fish movement for 9 days during the 50-year and 100-year events.

Table F-G7 Red River Alternative 2 – Duration in Days Fish Passage is Operational

Flood Event	Pass 1		Pass 2	
	Days in Operation	Days Per Year in Operation	Days in Operation	Days Per Year in Operation
10	6	1	0	0
50	5	<1	9	1
100	5	<1	5	<1
500	4	<1	4	<1

F-G 1.3.2.4 Alternative 2 Red River of the North Fish Passage Frequency Analysis

Detailed records exist of the Red River of the North discharges at Fargo, ND for each year back to the early 20th century. Correlating the maximum operable WSEL of each fish passage to a discharge at the Red River of the North Fargo, ND gauge (based on exceedance probability) allows for the quantification of the fish passage's closure frequency using the historic flow record at Fargo. Table F-G8 provides the upstream operational WSEL's for each fish passage and correlates it with a discharge at the Red River of the North Fargo, ND gauge. This was accomplished by noting the peak WSEL's for each Phase 4 flood event (10-year, 50-year, 100-year, 500-year) and pulling the corresponding peak discharge for the Fargo, ND gauge from the HEC-RAS models. An exponential function was fit to this data and used to interpolate the Fargo, ND discharge for each fish passage operational WSEL range. By correlating the fish passage operation ranges with a discharge at Fargo, ND a frequency analysis can be conducted with the historic data. Table F-G8 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) WSEL's for the 1901 to 2009 historic record. Figure F-G8 is a plot of the fitted exponential model for the Red River of the North fish passage WSEL's and the Fargo, ND gauge.

Table F-G8 Red River Alternative 2 – Fish Passage Operational Range WSEL, Corresponding Interpolated Discharges, and Flood Frequency Data for 1901-2009 at the Red River of the North Fargo, ND Gauge

	Fish Passage Open El. (ft)	Fish Passage Closed El. (ft)	Pre-project Q 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 Q 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 yr)	913.9	917.4	12892	10	116	17883	6	44
Pass 2 (~20-50 yr)	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.

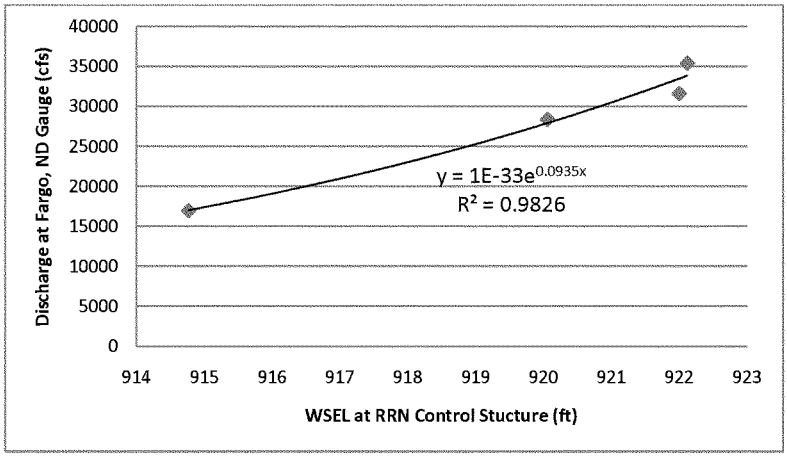

Figure F-G8 RRN Control Structure Water Surface Elevation versus Discharge at the RRN Fargo, ND Gauge

It is important to note for this analysis, there were only four data points available for the curve fitting. In addition, the lowest and highest WSEL's in Table F-G5 extend outside of the available data range.

F-G1.3.2.5 Alternative 3 Red River of the North Fish Passage Inlet Invert Design

For Alternative 3, the minimum water surface elevation for the first passage was set at the approximate water surface elevation that coincides with the full opening of the control structure gates on the receding limb of the 50-year event hydrograph. Based on this assumption, the invert of the first fish passage on the Red River is at 898.3-ft. The downstream invert is at 901.9-ft (1.5-ft below the 5-year tailwater elevation). The fish passage becomes operable when the water depth reaches one foot (899.3-ft). The fish passage becomes inoperable and the inflow gate must be shut when the water depth reaches 4.5-ft (902.8-ft). This span does not allow the fish passage to cover the peak of the 5-year and 10-year design floods.

The upstream invert of the second passage will be located one foot below the peak elevation for the first passage (901.8-ft). The downstream invert of the second passage is designed conservatively and will be 1.5 feet below the 5-year flood event (901.9-ft). Similarly to the first passage, once the water surface upstream reaches a depth of 4.5-ft (906.3) the velocities in the channel become too high for fish passage. The water surface elevation for the 10-year flood (916.29-ft) lies outside the operable range of the second passage, thus the second passage will not handle the peak of the 10-year event. Figures F-G9 and F-G10 present the water surface elevation hydrograph for the 10-year and 50-year events respectively and the inlet invert elevations for the two fish passage structures at the diversion control structure.

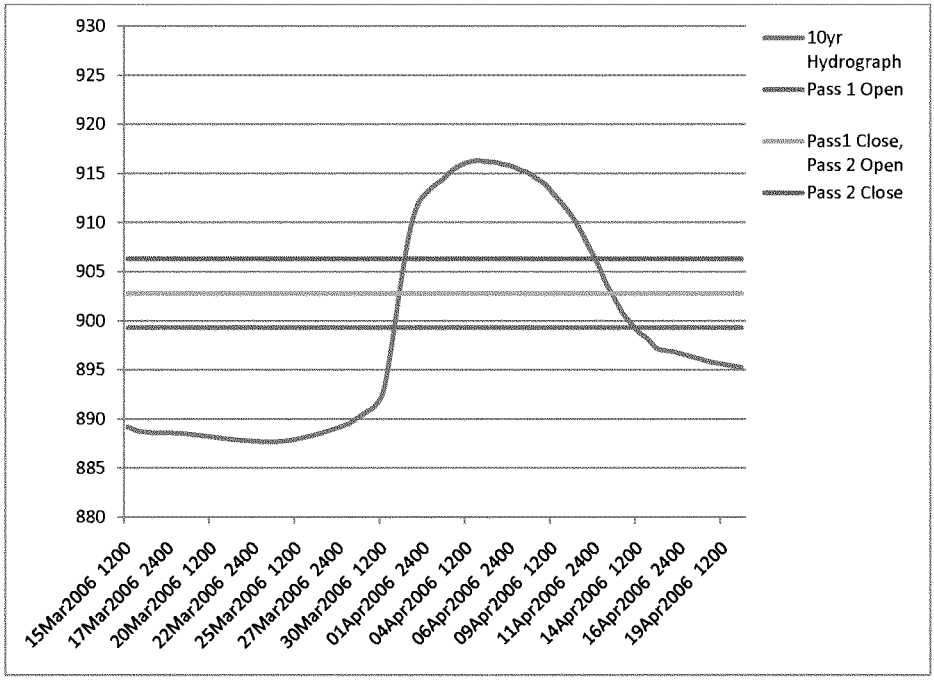

Figure F-G9 **Red River Alternative 3 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Red River of the North Control Structure for the 10-year Event**

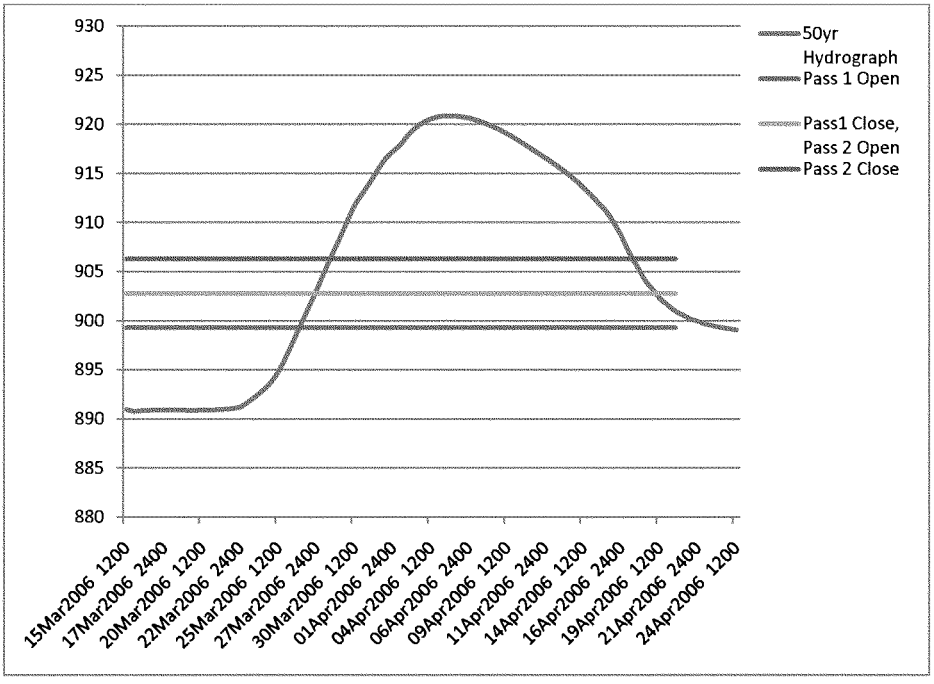

Figure F-G10 Red River Alternative 3 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Red River of the North Control Structure for the 50-year Event

As discussed for Alternative 1, Table F-G9 presents the results of the fish passage closure analysis in terms of number of days and approximate days per year (days closed multiplied by flood event probability).

Table F-G9 Red River Alternative 3 – Duration in Days of Fish Passage Closure Due to Velocities Exceeding 6-fps

	Rising Limb		Peak		Falling Limb		Total	
Flood Event	Days Out[(a)]	Days Per Year Out	Days Out	Days Per Year Out	Days Out	Days Per Year Out	Days Out	Days Per Year Out
10	6	<1	11	1	3	<1	20	2
50	5	<1	20	<1	0	0	25	<1
100	3	<1	24	<1	0	0	27	<1
500	3	<1	19	<1	0	0	22	<1

(a) Please see the Red River of the North Alternative 1 design discussion preceding Table F-G3 about the rising limb closure quantification.

Note: The duration of closure is shorter for the 500-year event than compared to the 100-year event due to the narrowing of the hydrograph for the 500-year event, i.e. rising and falling limbs occur closer together. Please see Figure F-G4.

Table F-G9 shows that fish passage will not be possible for 3 days on the falling limb of the hydrograph for the 10-year event. The 50-, 100-, and 500-year events will not have any days closed for the falling limb. However, there will be 11, 20, 24, and 19 days of closure for the 10-, 50-, 100-, and 500-year events, respectively. This passage is effective at eliminating the days of closure for the falling limb, however, there is a significant increase in the number of closure days at the peak of the hydrograph.

Alternatively, it is desirable to know how often each fish passage will be open. From this information it is possible to compare the advantage of allowing fish passage for a set duration to the cost of the associated infrastructure. Table F-G10 presents the duration in days the Alternative 3 fish passage (both passages 1 and 2) will be open. For instance, a 10-year event will only allow fish passage for a total of 2 days in passage 1. The second passage will only provide fish movement for 3 days during the 50-year and 100-year events.

Table F-G10 Red River Alternative 3 –Duration in Days Fish Passage is Operational

	Pass 1		Pass 2	
Flood Event	Days in Operation	Days Per Year in Operation	Days in Operation	Days Per Year in Operation
10	2	<1	2	<1
50	6	<1	3	<1
100	8	<1	3	<1
500	5	<1	11	<1

F-G 1.3.2.6 Alternative 1 Wild Rice River Fish Passage Inlet Invert Design

The design of the Wild Rice River Fish passage was conducted in the same manner as was done for the Red River of the North Fish Passages. Three alternatives were evaluated with the goal of optimizing the number of days the fish passage is in operation and keep the number of days it is closed for each event to a minimum.

For Alternative 1, the inlet and outlet inverts were set at 1-ft and 1.5-ft, respectively, below the 5-year water surface elevation. Because the unsteady flow modeling conducted in Phase 4 does not include the 5-year event, the 5-year event water surface elevation was chosen to be 1-ft below the 10-year event for this design. Based on this assumption, the invert of the first fish passage on the Wild Rice River is at 913.6-ft. The downstream invert is at 898.9-ft. The fish passage becomes operable when the water depth reaches one foot (914.6-ft). The fish passage becomes inoperable and the inflow gate must be shut when the water depth reaches 4.5-ft (918.1-ft). This span allows one fish passage to cover both the 5-year and 10-year design floods.

To reach the 50-year flood event a second passage is needed. The upstream invert of the second passage will be located one foot below the peak elevation for the first passage (917.1-ft). The downstream invert of the second passage is designed conservatively and will be 1.5 feet below the 5-year flood event (898.9-ft). Similarly to the first passage, once the water surface upstream reaches a depth of 4.5-ft (921.6) the velocities in the channel become too high for fish passage. The water surface elevation for the 50-year flood (920.9-ft) lies within the operable range of the second passage. Figures F-G11 and F-G12 present the water surface elevation hydrograph for the 10-year and 50-year events respectively and the inlet invert elevations for the two fish passage structures at the Wild Rice River control structure.

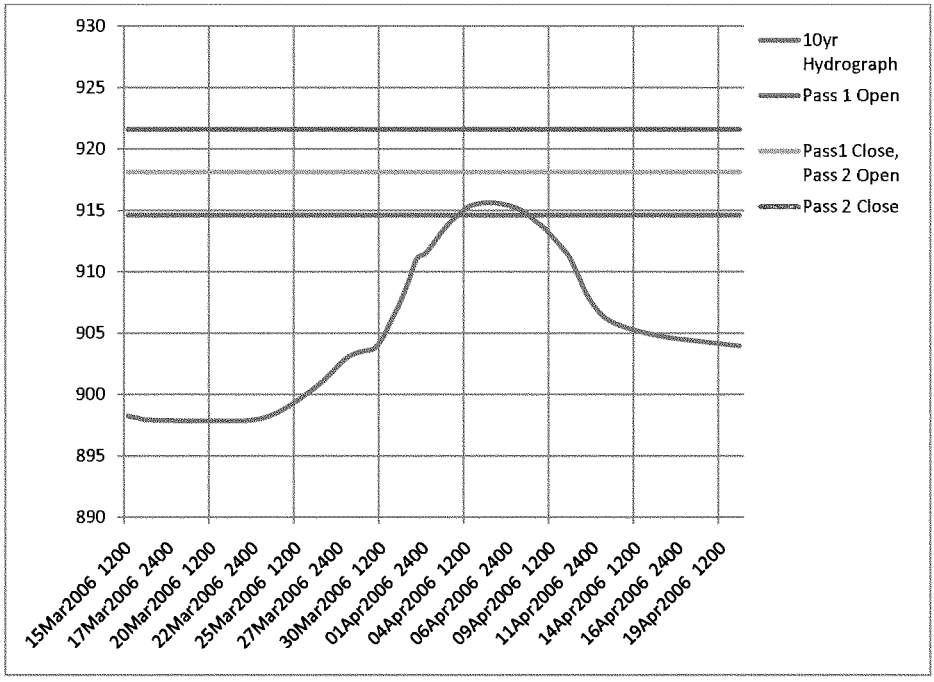

Figure F-G11 Wild Rice River Alternative 1 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Wild Rice River Control Structure for the 10-year Event

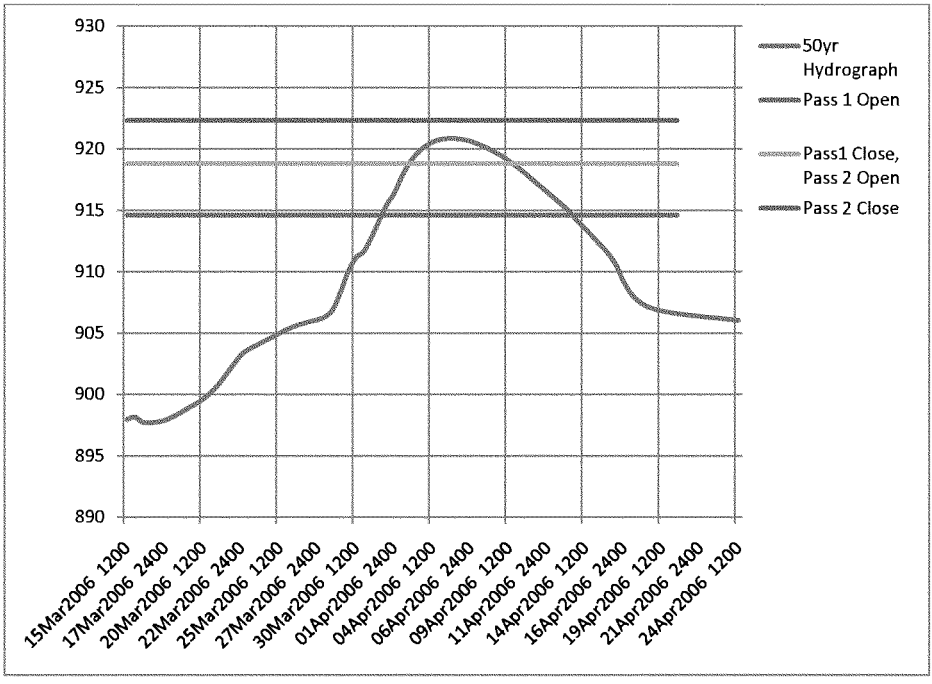

Figure F-G12 Wild Rice River Alternative 1 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Wild Rice River Control Structure for the 50-year Event

As discussed for Red River of the North Alternative 1, Table F-G11 presents the results of the fish passage closure analysis in terms of number of days and approximate days per year (days closed multiplied by flood event probability). The gate operation for the Wild Rice River begins partially closed in the model: similar to the Red River control structure. However, the Wild Rice River control structure does not open back up on the falling limb of the hydrograph, which increases the number of fish passage closure days for the falling limb of the Wild Rice hydrograph when compared to the Red River of the North data.

Table F-G11 Wild Rice River Alternative 1 – Duration in Days of Fish Passage Closure Due to Velocities Exceeding 6-fps

	Rising Limb		Peak		Falling Limb		Total	
Flood Event	Days Out*	Days Per Year Out	Days Out	Days Per Year Out	Days Out	Days Per Year Out	Days Out	Days Per Year Out
10	12	1	0	0	13	1	25	3
50	12	<1	0	0	11	<1	23	<1
100	13	<1	6	<1	14	<1	33	<1
500	13	<1	4	<1	19	<1	36	<1

(a) Please see the Red River of the North Alternative 1 design discussion preceding Table F-G3 about the rising limb closure quantification.

Note: The duration of closure is shorter for the 500-year event than compared to the 100-year event due to the narrowing of the hydrograph for the 500-year event, i.e. rising and falling limbs occur closer together. Please see Figure F-G4.

Table F-G11, shows that fish passage will not be possible for 13, 11, 14, and 19 days on the falling limb of the hydrograph for the 10-year, 50-year, 100-year, and 500-year events. The 100-year event will have 6 days when fish are not able to pass during the peak. Due to the narrowing of the hydrograph for the 500-year event, fish pass will only be restricted for 4 days at the peak (See explanation in Section F-G1.3.2.1). Also, the gates are expected to opened near the peak in the hydrograph for the 500-year storm, so fish passage is not expected to be restricted for the falling limb (peak velocities through the control structure gates should be approximately 5.4 fps).

Alternatively, it is desirable to know how often each fish passage will be open. From this information it is possible to compare the advantage of allowing fish passage for a set duration to the cost of the associated infrastructure. Table F-G12 presents the duration in days the Alternative 1 fish passage (both passages 1 and 2) will be open. For instance, a 10-year event will only allow fish passage for a total of 5 days in passage 1. This appears short, however it is also providing passage for events down to the 5-year event. The second passage will only provide fish movement for 8 days during the 50-year and 100-year events.

Table F-G12 Wild Rice River Alternative 1 – Duration in Days Fish Passage is Operational

Flood Event	Pass 1		Pass 2	
	Days in Operation	Days Per Year in Operation	Days in Operation	Days Per Year in Operation
10	5	<1	0	0
50	5	<1	8	<1
100	5	<1	8	<1
500	4	<1	7	<1

F-G 1.3.2.7 Alternative 2 Wild Rice River Fish Passage Inlet Invert Design

For Alternative 2, the maximum water surface elevation for the second passage was set at the peak of the 50-year event. Based on this assumption, the invert of the first fish passage on the Wild Rice River is at 912.9-ft. The downstream invert is at 901.9-ft (1.5-ft below the 5-year tailwater elevation). The fish passage becomes operable when the water depth reaches one foot (913.9-ft). The fish passage becomes inoperable and the inflow gate must be shut when the water depth reaches 4.5-ft (917.4-ft). This span allows one fish passage to cover both the 5-year and 10-year design floods.

The upstream invert of the second passage will be located one foot below the peak elevation for the first passage (917.4-ft). The downstream invert of the second passage is designed conservatively and will be 1.5 feet below the 5-year flood event (901.9-ft). Similarly to the first passage, once the water surface upstream reaches a depth of 4.5-ft (920.9) the velocities in the channel become too high for fish passage. The water surface elevation for the 50-year flood (920.9-ft) lies within the operable range of the second passage. Figures F-G13 and F-G14 present the water surface elevation hydrograph for the 10-year and 50-year storms respectively and the inlet invert elevations for the two fish passage structures at the Wild Rice River control structure.

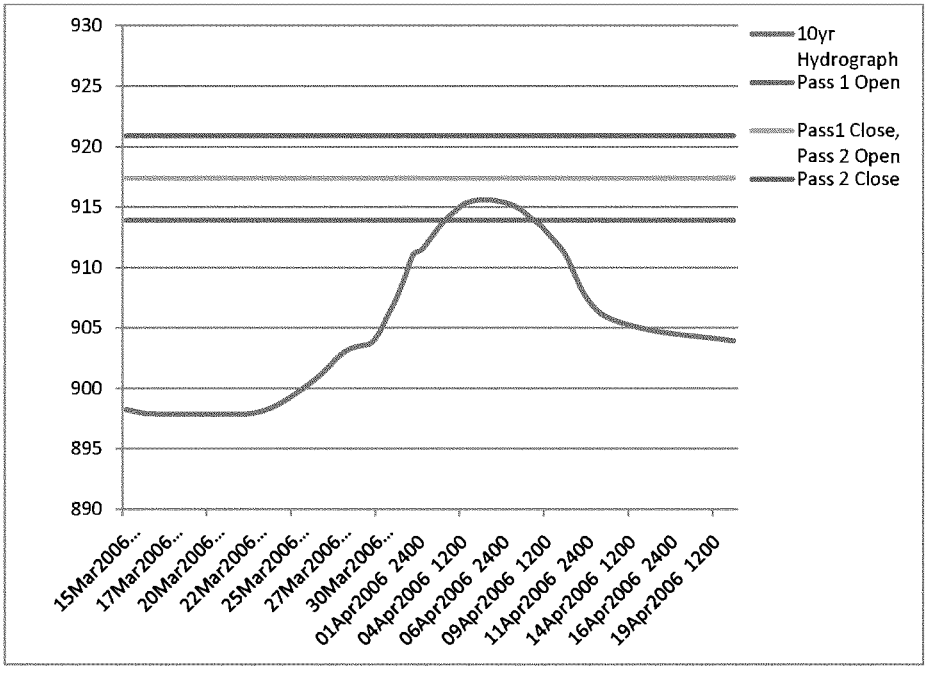

Figure F-G13 Wild Rice River Alternative 2 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Wild Rice River Control Structure for the 10-year Event

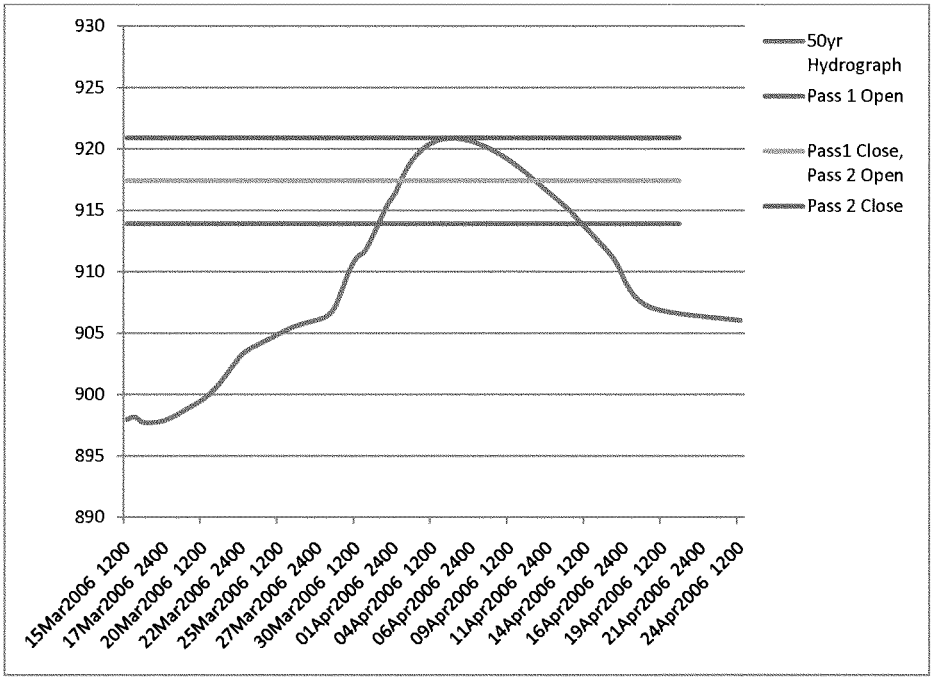

Figure F-G14 Wild Rice River Alternative 2 – Fish Passage Inlet Inverts and Water Surface Elevation Hydrograph at the Wild Rice River Control Structure for the 50-year Event

As discussed for the Red River of the North Alternative 1, Table F-G13 presents the results of the fish passage closure analysis in terms of number of days and approximate days per year (days closed multiplied by flood event probability). The gate operation for the Wild Rice River begins partially closed in the model, similar to the Red River control structure. However, the Wild Rice River control structure does not open back up on the falling limb of the hydrograph, which increases the number of fish passage closure days for the falling limb of the Wild Rice hydrograph when compared to the Red River of the North data.

Table F-G13 Wild Rice River Alternative 2 – Duration in Days of Fish Passage Closure Due to Velocities Exceeding 6-fps

Flood Event	Rising Limb		Peak		Falling Limb		Total	
	Days Out[(a)]	Days Per Year Out	Days Out	Days Per Year Out	Days Out	Days Per Year Out	Days Out	Days Per Year Out
10	11	1	0	0	12	1	23	2
50	11	<1	0	0	11	<1	22	<1
100	12	<1	6	<1	13	<1	31	<1
500	12	<1	4	<1	19	<1	34	<1

(a) Please see the Red River of the North Alternative 1 design discussion preceding Table F-G3 about the rising limb closure quantification.

Note: The duration of closure is shorter for the 500-year event than compared to the 100-year event due to the narrowing of the hydrograph for the 500-year event, i.e. rising and falling limbs occur closer together. Please see Figure F-G4.

From Table F-G13, it is evident fish passage will not be possible for 12, 11, 13, and 19 days on the falling limb of the hydrograph for the 10-year, 50-year, 100-year, and 500-year events, respectively. The 100-year event will have six days when fish are not able to pass during the peak (see Section F-G1.3.2.1 for explanation). Due to the narrowing of the hydrograph for the 500-year event, fish passage will only be restricted for 4 days at the peak. Since this passage has been lowered as compared to Alternative 1, it has slightly fewer days closed. In addition, this passage will have fewer days closed for the more frequent events smaller than the 10-year flood.

Alternatively, it is desirable to know how often each fish passage will be open. From this information it is possible to compare the advantage of allowing fish passage for a set duration to the cost of the associated infrastructure. Table F-G14 presents the duration in days the Alternative 2 fish passage (both passages 1 and 2) will be open. For instance, a 10-year event will only allow fish passage for a total of 6 days in passage 1. This seems short, however it is also providing passage for events down to the 5-year event. The second passage will only provide fish movement for 9 days during the 50-year and 100-year events.

Table F-G14 Wild Rice River Alternative 2 – Duration in Days Fish Passage is Operational

Flood Event	Pass 1		Pass 2	
	Days in Operation	Days Per Year in Operation	Days in Operation	Days Per Year in Operation
10	6	1	0	0
50	5	<1	8	<1
100	5	<1	8	<1
500	4	<1	7	<1

F-G1.3.2.8 Alternative 3 Wild Rice River Fish Passage Inlet Invert Design

For Alternative 3 on the Red River of the North, the minimum water surface elevation for the first passage was set at the approximate water surface elevation that coincides with the full opening of the control structure gates on the receding limb of the 50-year event hydrograph. The control structure on the Wild Rice River however, does not open its gates on the receding limb in the same way the Red River of the North control structure does. Instead, the Wild Rice River control structure leaves its gates only slightly open until the end of the event. Thus, the Alternative 3 assessment is not necessary.

F-G1.3.3 Channel Design

Each set of gates is connected to a separate fish passage channel that descends from the upstream pool to the downstream tailwater. Thus, three fish passage channels are necessary to cover the 5-year to 50-year flows at the Red River control structure for the FCP. Two fish passage channels are necessary to cover the 5-year to 50-year flows at the Red River Control Structure and the Wild Rice River Control Structure.

Although each fish passage channel has a different upstream elevation (based on the gate invert), each fish passage channel at a control structure ties into the same downstream elevation, set at least 1.5 feet below the tailwater at that location coincidental to the 5-year event on the Red River.

Each fish passage channel uses a sequence of pools and riffles to achieve the elevation drop between the upstream gates and downstream tailwater. A one foot drop in elevation occurs in the riffle between pools. The number of pools and riffles included in each fish passage channel is based on the difference in gate invert and tailwater elevation. Thus, the length of each fish passage channel varies. The average slope of each fish passage channel is 2.2 percent (see discussion of pool and riffle design below). The number of pools and riffles, and length of each fish passage channel is summarized in Table F-G15.

Table F-G15 Red River and Wild Rice River Fish Passage Channel Pools and Riffles

Plan	Location/ Control Structure	Fish Passage Channel	Riffle Length (ft)	Number of Riffles	Pool Length Top (ft)	Pool Length Bottom (ft)	Number of Pools	Length (ft)	Slope (%)
FCP	Red River	1	20	3	40	20	3	135	2.2
FCP	Red River	2	20	6	40	20	6	270	2.2
FCP	Red River	3	20	10	40	20	10	450	2.2
LPP	Red River	1	20	11	40	20	11	660	2.2
LPP	Red River	2	20	15	40	20	15	900	2.2
LPP	Wild Rice River	1	20	14	40	20	14	840	2.2
LPP	Wild Rice River	2	20	18	40	20	18	1080	2.2

F-G1.3.4 Riffle Design

The riffles are designed to achieve a 1 foot drop over a 20 foot length (5 percent slope). The invert of the first riffle downstream of the gate is equal to the gate invert. The first riffle downstream of the gates is designed with a 10 foot bottom width and 3:1 horizontal to vertical sideslopes (see Figure F-G15).

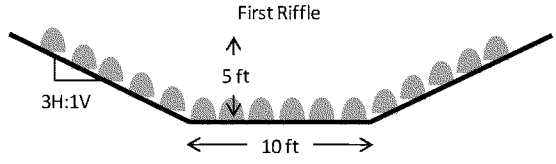

Figure F-G15 Cross Section of First Riffle in Fish Passage

The dimensions of the first riffle are chosen to limit flow over the riffle for larger headwater elevations. When the water depth over the gate (and thus over the riffle) is 4.5 feet (the upper limit of the gate operational range, see section F-G1.5), the flow over the riffle is approximately 600 cfs, or 6 percent of the flow through the Red River control structure. To limit flow velocities to less than 6 feet per second over the first riffle, a high channel roughness (Manning's n of approximately 0.11) must be achieved.

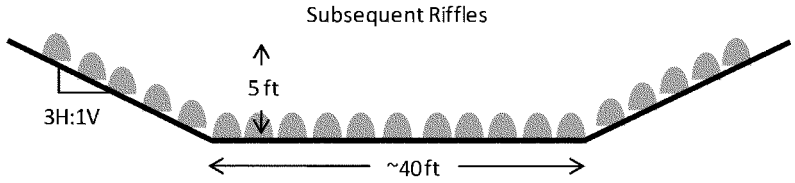

Figure F-G16 Cross Section of Subsequent Riffles in Fish Passage

Subsequent riffles are designed with dimensions similar to the pool dimensions for ease of construction. Thus, the water depths (and velocities) over these riffles are less than the first riffle. Riffles downstream of the first riffle are designed with a bottom width on the order of 40 feet and 3:1 side slopes (see Figure F-G16). The upstream invert of each subsequent riffle is 1 foot below that of the upstream riffle. The last riffle in the pool-riffle sequence of each channel is located below the 5-year tailwater elevation.

F-G1.3.5 Pool Design

Level pools located between riffles provide areas of low velocity for fish to rest between areas of high velocity while moving upstream. The pool design includes a bottom width of 40 feet and 3:1 side slopes (see Figures F-G17 and F-G18). The minimum length of the pools in the direction of flow is calculated using volumetric energy calculation used in the USACE's Lock and Dam 22 Fish Passage Improvement Project Implementation Report – Appendix H.

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where:

K	=	volumetric energy (foot pounds per second per cubic foot)
	=	specific weight of water (1.94 slugs per cubic foot)
g	=	gravitational acceleration (32.2 feet per second per second)
Q	=	discharge (cubic feet per second)
	=	head difference between pools (feet)
Vp	=	volume of pool (cubic feet)

Pools should be sized to achieve a volumetric energy (K) of between 3.1 to 4.2 foot pounds per second per cubic foot. Solving the equation for K = 3.1 and using a discharge of 625 cfs (see discussion of fish passage operation, Section F-G1.5) yields a pond volume of 12,600 cubic feet. Using a pool geometry with a 40 foot bottom width, 3:1 side slopes in the direction perpendicular to the flow, 2:1 side slopes in the flow direction, and a maximum pool depth of 7.6 feet (see discussion of fish passage operation, Section F-G1.5), the required pool length is 26 feet. For design feasibility, the bottom width will be increased to 20 feet in the flow direction, and the top width will be 40 feet.

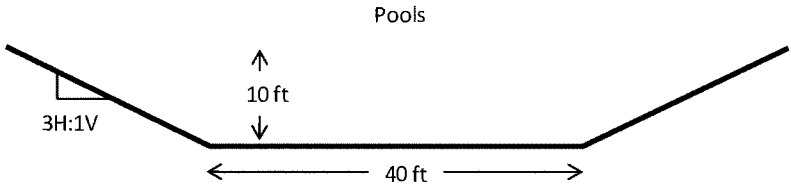


Figure F-G17 Cross Section of Pools in Fish Passage Perpendicular to Flow Direction

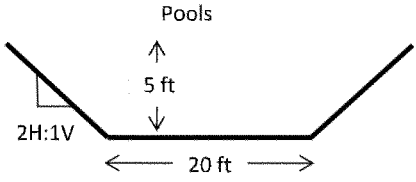


Figure F-G18 Cross Section of Pools in Fish Passage Parallel to Flow Direction

F-G1.4 FISH PASSAGE DESIGN – RUSH RIVER AND LOWER RUSH RIVER FISH PASSAGE

The fish passage design at the Rush River and Lower Rush River also utilizes sequences of pools and riffles (see Appendix F) with one opening at the upstream end of the sequence (or channel). The pool and riffle designs are identical to those discussed for the Red River and Wild Rice River fish passages (see Section F-F1.3.4 and Section F-F1.3.5) with the exception of the initial 10 foot wide riffle. The Rush and Lower Rush Rivers do not need to restrict flow through the fish passages, which allows all of the riffles to be 40-ft wide.

At each tributary, a single fish passage channel is used, thereby minimizing the need for operational control during flood events. At these locations, the fish passage will remain open during all flood events. Thus, for some events, the upstream head on the fish passage will result in velocities through the fish passage channel that prevent fish passage (i.e. head greater than 4.5 feet, see Table F-G22). The headwater and tailwater elevations used for fish passage design at the Rush River and Lower Rush River are summarized in Table F-G16.

Table F-G16 Invert, Headwater and Tailwater Elevations for Fish Passage Design at the Rush River and Lower Rush River

Location/Control Structure	Fish Passage Upstream Invert (ft)	Fish Passage Downstream Invert (ft)	10-year Event (on Red River)		
			Headwater (ft)	Tailwater (ft)	HW-TW (ft)
Rush River[(a)]	880.0[(a)]	866.58	883.52	883.35	0.17
Lower Rush River[(a)]	885.9[(a)]	869.12	887.77	885.0	2.8

(a) Upstream inverts based on channel invert, as opposed to water surface elevations corresponding to an event on the Red River, as is the design concept for the Red River and Wild Rice River fish passages.

At the Rush River, the inlet invert is set at an elevation of 880.0, the approximate channel bottom. The fish passage channel includes a sequence of 14 pools and riffles; the fish passage drops from the tributary channel bottom to an elevation equal to the invert of the low flow channel in the Diversion Channel (see Table F-G17). This allows low flows in the Rush River to pass into the Diversion Channel while allowing fish to pass upstream into the Rush River. This structure will allow fish passage until slightly above the 10-year coincidental event on the Red River.

At the Lower Rush River, the inlet invert is set at an elevation of 885.9, the approximate channel bottom. The fish passage channel includes a sequence of 17 pools and riffles; the fish passage drops from the tributary channel bottom to an elevation equal to the invert of the low flow channel in the Diversion Channel (see Table F-G17). This allows low flows in the Lower Rush River to pass into the Diversion Channel while allowing fish to pass upstream into the Lower Rush River. This structure will allow fish passage up to approximately 1 foot below the 50-year event on the Red River.

Table F-G17 Rush River and Lower Rush River Fish Passage Channel Pools and Riffles

Location/Control Structure	Fish Passage Channel	Riffle Length (ft)	Number of Riffles	Pool Length Top (ft)	Pool Length Bottom (ft)	Number of Pools	Length (ft)	Slope (%)
Rush River	1	20	14	40	20	14	840	2.2
Lower Rush River	1	20	17	40	20	17	1020	2.2

Tables F-G18 and F-G20 provide the duration in days the Rush and Lower Rush fish passage will be closed due to average velocities exceeding 6-fps. The Rush and Lower Rush fish passages will remain fully passable for the 10 year storm, however there will be days the fish passage will be closed for the 50-, 100-, and 500-year design events. Tables F-G19 and F-G21 display the number of days the Rush and Lower Rush fish passages will be open for fish movement upstream for each representative design event.

Table F-G18 Duration in Days of Rush Fish Passage Closure Due to Velocities Exceeding 6-fps

Flood Event	Peak	
	Days Out	Days Per Year Out
10	0	0
50	10	<1
100	14	<1
500	13	<1

Note: The duration of closure is shorter for the 500-year event than compared to the 100-year event due to the narrowing of the hydrograph for the 500-year event, i.e. rising and falling limbs occur closer together. Please see Figure F-G4.

Table F-G19 Duration in Days Rush Fish Passage is Operational

Flood Event	Pass 1	
	Days in Operation	Days Per Year in Operation
10	10	1
50	7	<1
100	7	<1
500	10	<1

Table F-G20 Duration in Days of Lower Rush Fish Passage Closure Due to Velocities Exceeding 6-fps

Flood Event	Peak	
	Days Out	Days Per Year Out
10	0	0
50	4	<1
100	8	<1
500	8	<1

Note: The duration of closure is shorter for the 500-year event than compared to the 100-year event due to the narrowing of the hydrograph for the 500-year event, i.e. rising and falling limbs occur closer together. Please see Figure F-G4.

Table F-G21 Duration in Days Lower Rush Fish Passage is Operational

Flood Event	Pass 1	
	Days in Operation	Days Per Year in Operation
10	4	<1
50	6	<1
100	5	<1
500	6	<1

F-G1.5 HYDRAULICS OF FISH PASSAGE OPERATION

The hydraulics of each fish passage channel varies as the headwater elevation at the entrance to the fish passage channel increases. Thus, the flow through a fish passage channel, velocity through the pools and riffles, and depth through the pools and riffles vary between the 5-, 10-, 20-, and 50-year events. 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 Table F-G22. Calculations were performed using Manning's equation through the first riffle (the limiting channel geometry), and extrapolating those results to other pool and riffle geometries using continuity of flow.

Table F-G22 Hydraulics of Fish Passage Channels for Varying Headwater Conditions

Head on Gate Invert (ft)	Velocity through Gates (ft/sec)	Depth in First Pool (ft)	Velocity in First Pool (ft/sec)	Depth in Later Pools (ft)	Velocity in Later Pools (ft/sec)	Depth in First Riffle (ft)	Velocity in First Riffle (ft/sec)	Depth in Later Riffles (ft)	Velocity in Later Riffles (ft/sec)	Flow (cfs)
0.0[(a)]	--	--	--	--	--	--	--	--	--	--
0.5[(a)]	--	--	--	--	--	--	--	--	--	--
1.0	1.2	6.0	0.1	5.5	0.1	1.0	2.6	0.5	1.8	35
1.5	1.6	6.5	0.2	5.7	0.2	1.5	3.3	0.7	2.4	71
2.0	2.1	7.0	0.3	6.0	0.4	2.0	3.8	1.0	3.0	123
2.5	2.5	7.5	0.4	6.3	0.5	2.5	4.3	1.3	3.5	187
3.0	3.0	8.0	0.5	6.6	0.7	3.0	4.8	1.6	4.0	271
3.5	3.5	8.5	0.7	6.9	0.9	3.5	5.2	2.0	4.5	371
4.0	4.1	9.0	0.8	7.2	1.1	4.0	5.6	2.3	5.0	489
4.5	4.6	9.5	1.0	7.6	1.3	4.5	5.9	2.7	5.5	625

(a) Gate to fish passage remains closed until headwater is 1 foot greater than gate invert

The critical condition (i.e. highest velocities and highest flows) occur when the headwater is greatest. The use of only two sets of gates and fish passage channels at the Red River and Wild Rice River for the LPP result in a wide range of possible hydraulic conditions during operation. Limiting the range of flow and velocity through the fish passage would require more fish passage channels with a smaller gap in invert upstream elevations. Single fish passage channels were selected for the Lower Rush and Rush Rivers so that

the passages do not have to be actively managed, minimizing the operational complexity of the system.

Throughout the range of potential headwaters and flows through each fish passage channel, the maximum velocities in the riffles and gates remain below 6 feet per second. The average flow through the pools is 1.3 feet per second or less. In the case of the Red River and Wild Rice River fish passages, only one set of gates is opened at a time, and gates are closed as the upstream water surface elevation increases. At the Lower Rush River and Rush River, the fish passage gates remain open regardless of upstream headwater conditions (although velocities prohibit fish passage above a given water surface elevation).

F-G1.6 LOCATION OF FISH PASSAGE CHANNELS

Similar design concepts have been used for fish passage at the Red River control structures and tributary control structures. Each location, however, presents unique topography and control structure design, among which fish passage must be situated.

F-G1.6.1 Fish Passage Location – Red River Control Structure (FCP)
The design described in this subsection includes three fish passage channels of varying lengths at the Red River control structure for the FCP. The gates at the upstream ends of the fish passage channels will be placed in the east wingwall of the control structure (see Appendix F), and extend north to the constructed Red River channel downstream of the control structure. The downstream ends of the fish passage channels will merge at the Red River channel, downstream of the hydraulic jump, at an elevation of 896.7 feet. The downstream end of the fish passage will be located as close to the control structure as possible provide an alternative route to fish that reach that point but cannot proceed through the control structure due to high velocities.

F-G1.6.2 Fish Passage Location – Red River Control Structure (LPP)
The design described in this subsection includes two fish passage channels of varying lengths at the Red River control structure. The gates at the upstream ends of the fish passage channels will be placed in the east wingwall of the control structure (see Appendix F), and extend north to the constructed Red River channel downstream of the control structure. Placing the fish passage on the east side of the control structure will reduce the chances of fish emerging from the fish passage channel and being swept into the connecting channel on the west side of the Red River. The downstream ends of the fish passage channels will merge at the Red River channel, downstream of the hydraulic jump. The fish passage includes 11 and 15 sequences of pools and riffles for the lower and upper channels, respectively. The downstream end of the fish passage will be located as close to the control structure as possible to provide an alternative route for fish that reach that point but cannot proceed through the control structure due to high velocities.

F-G1.6.3 Fish Passage Location – Wild Rice River Control Structure (LPP)
The design described in this subsection includes two fish passage channels of varying lengths at the Wild Rice River control structure. The gates at the upstream ends of the fish passage channels will be placed in the west wingwall of the control structure (see

Appendix F). Placing the fish passage on the west side of the control structure will prevent fish already swimming upstream from having to move downstream to reach the unprotected reach of the Wild Rice River. The downstream ends of the fish passage channels will merge in the protected reach of the Wild Rice River, downstream of the hydraulic jump. The fish passage includes 14 and 18 sequences of pools and riffles for the lower and upper channels, respectively. The downstream end of the fish passage will be located as close to the control structure as possible to provide an alternative route for fish that reach that point but cannot proceed through the control structure due to high velocities.

F-G1.6.4 Fish Passage Location – Rush River Control Structure (LPP)
The design described in this subsection includes one fish passage channel including 14 sequences of pools and riffles at the Rush River control structure. The inlet invert at the upstream end of the fish passage channel will be located on the Rush River upstream of the drop structure (see Appendix F) with an invert elevation set at the approximate tributary invert. The fish passage includes 14 sequences of pools and riffles. The downstream end of the fish passage channel will merge with the diversion channel downstream of the drop structure, at an elevation of approximately 866.58 feet.

F-G1.6.5 Fish Passage Location – Lower Rush River Control Structure (LPP)
The design described in this subsection includes one fish passage channel including 17 sequences of pools and riffles at the Lower Rush River control structure. The inlet invert at the upstream end of the fish passage channel will be located on the Lower Rush River upstream of the drop structure (see Appendix F) with an invert elevation set at the approximate tributary invert. The fish passage includes 17 sequences of pools and riffles. The downstream end of the fish passage channel will merge with the diversion channel downstream of the drop structure, at an elevation of approximately 869.12 feet.

F-G1.7 FISH PASSAGE DESIGN DISCUSSION

It was necessary to update the Phase 3 design due to the use of a HEC-RAS unsteady flow model and the need for upstream staging. Due to the increase in the upstream staging in Phase 4, the inlet inverts for the Red River and the Wild Rice River fish passages were necessarily raised to provide proper operation. However, the downstream water surface elevations remained similar to the Phase 3 water surface elevations, which necessitated an increase in the length of each fish passage. The length, and subsequently the cost, has nearly doubled when compared to Phase 3.

Three inlet invert elevation alternatives were evaluated for the Red River of the North:

- Alternative 1: Inlet invert based on Phase 3 guidelines (1-ft below the 5-year event water surface elevation).
- Alternative 2: Inlet invert based on operation during the peak water surface elevation of the 50-year design storm with two fish passages.
- Alternative 3: Inlet invert set at the approximate water surface elevation for which the gates become fully opened during the receding limb of the hydrograph.

Since the gate operations of the Wild Rice River remain relatively closed for the duration of the storm event, Alternative 3 was not evaluated for this control structure fish passage. In addition, the Rush and Lower Rush have set inlet and outlet invert elevations and will remain similar to their Phase 3 design.

Each alternative was evaluated for the number of days the control structure and fish passage combination will not allow fish passage and the number of days it will allow fish passage for the 10-year, 50-year, 100-year, and 500-year hydrographs. Based on this information, the alternative which provides the greatest amount of service can be chosen. Alternative 1 and 2 for both the Red River and Wild Rice River control structures produced nearly identical results for flood events evaluated. Alternative 2 has a slightly lower inlet invert which allowed it to include more days on the falling limb of the hydrograph. Conversely, Alternative 1 had a higher inlet invert, which produced more days of operation for the higher events. However, since the smaller events are more frequent, the lower invert of Alternative 2 will provide more days of operation than Alternative 1 over time. In other words, if Alternative 2 had been evaluated for smaller events, i.e. the 5-year design event, Alternative 2 would have provided more days of available fish passage. Alternative 2 is the feasibility design selected for the Red River of the North and the Wild Rice River fish passages.

During this evaluation, it became clear that the second fish passage at both the Red River of the North and Wild Rice River may not be the most cost effective form of environmental impact mitigation. Tables F-G7 and F-G14 show the Red River of the North lower fish passage will be open for 6 days in the 10-year event and the Wild Rice River lower fish passage will remain open 5 days in the 10-year event. This is a very short duration of operation, however, the lower fish passage will also be effective for smaller events, and these events will be more frequent. On the other hand, the upper fish passage at both the Red River of the North and the Wild Rice River will only be open for 9 days and 8 days, respectively, for the 50-year event. This event will be infrequent, and providing 9 or 8 days of fish passage may not be worth the price tag for each fish passage.

F-G2.0 VELOCITIES IN THE DIVERSION CHANNEL

The following section presents velocity profiles in the LPP and FCP Diversion Channels. The velocity profiles present expected average velocities that fish would encounter if they were to enter the Diversion Channel. The plots are taken from the HEC-RAS models.

In general, positive spikes represent bridges where roads cross the diversion channel. The negative spike for the 10-, 50-, and 500-year events occurs upstream of the diversion inlet structure. The slope of the diversion channel upstream of the inlet structure is opposite the flow direction, which makes this area a large pool for these flood events.

Figures F-G19 to F-G23 present velocities along the LPP Diversion Channel. Figures F-G24 to F-G38 present the velocity profiles for the 10-, 50-, 100-, and 500-year events for four locations along the diversion channel. Maximum velocity in the LPP Diversion Channel occurs downstream of the I-29 bridge crossing for the 50-, 100-, and 500-year

events. Maximum velocities for these events are approximately 3.7 fps. For the 10-year event the maximum velocity occurs at the inlet weir located upstream of the Sheyenne River aqueduct. The maximum velocity for the 10-year event is approximately 3.4 fps. For the 10-year event, the higher velocities in the LPP Diversion Channel occur upstream in the diversion channel. For the 50-year event and greater, the higher velocities in the LPP Diversion Channel occur downstream in the diversion channel, where the flows are higher.

Figures F-G39 to F-G46 present velocities along the FCP Diversion Channel for Year 0 hydrology. Figures F-G47 to F-G54 present velocities along the FCP Diversion Channel for Year 25 hydrology. Figures F-G55 to F-G62 present velocities along the FCP Diversion Channel for Year 50 hydrology. Maximum velocity in the FCP Diversion Channel occurs at the inlet weir for all modeled events. Bench width in the Diversion Channel affects the velocities. For events greater than the 10-year, the lowest velocities in the FCP Diversion Channel occur in the reach with 250' wide benches.

F-G2.1 VELOCITIES IN THE LPP DIVERSION CHANNEL

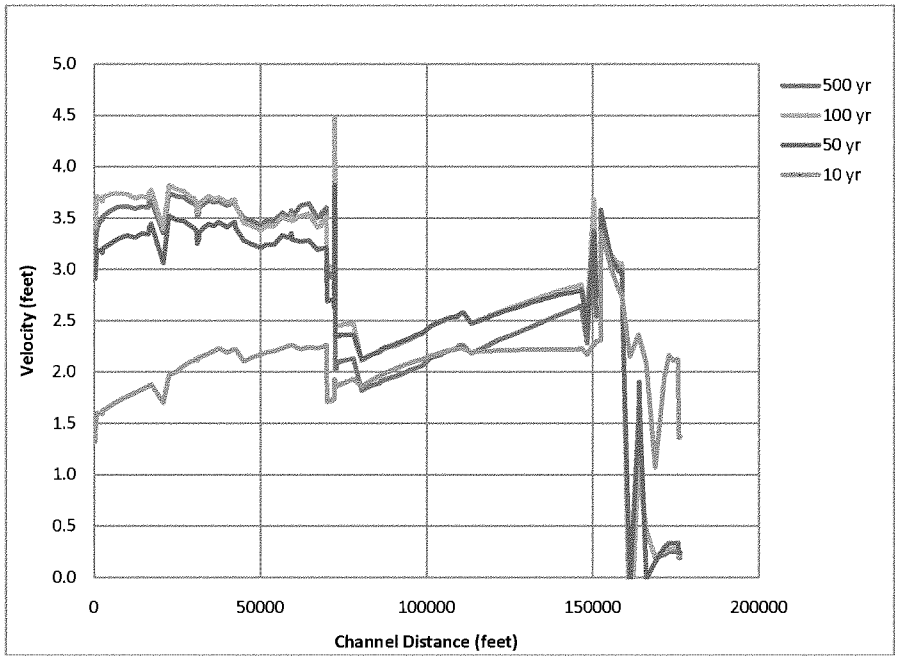


Figure F-G19 LPP Diversion Channel Velocities for All Modeled Events

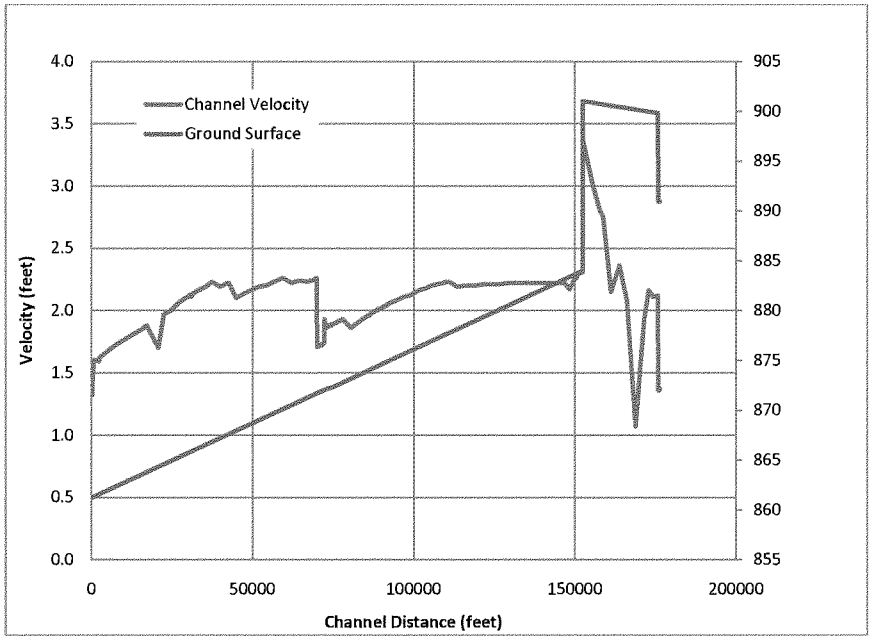


Figure F-G20 LPP Diversion Channel Velocities for the 10-Year Event

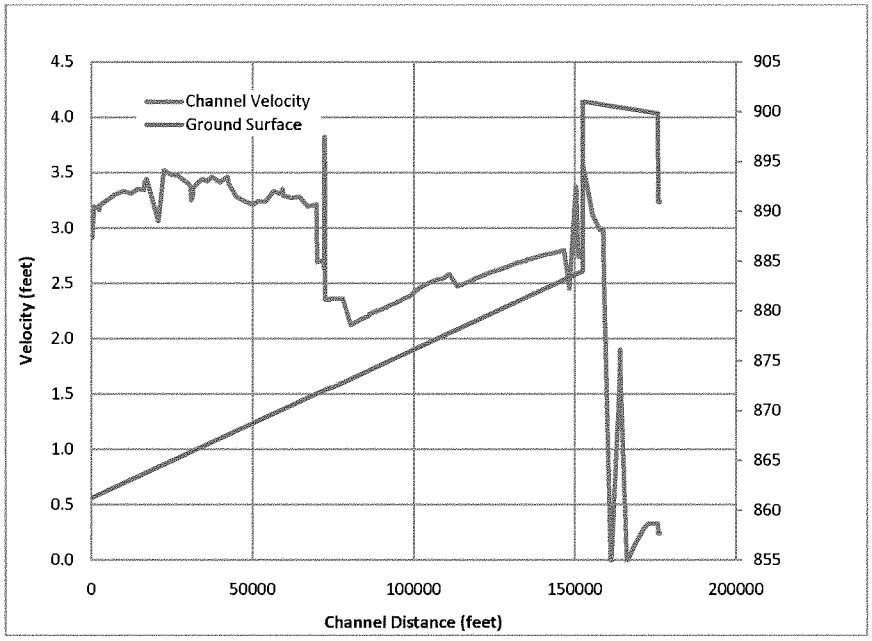


Figure F-G21 LPP Diversion Channel Velocities for the 50-Year Event

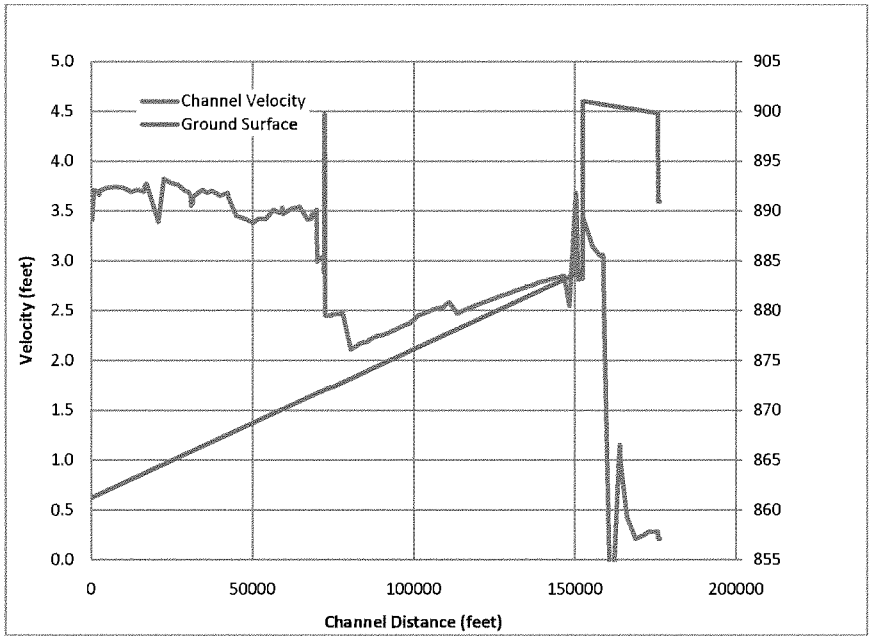


Figure F-G22 LPP Diversion Channel Velocities for the 100-Year Event

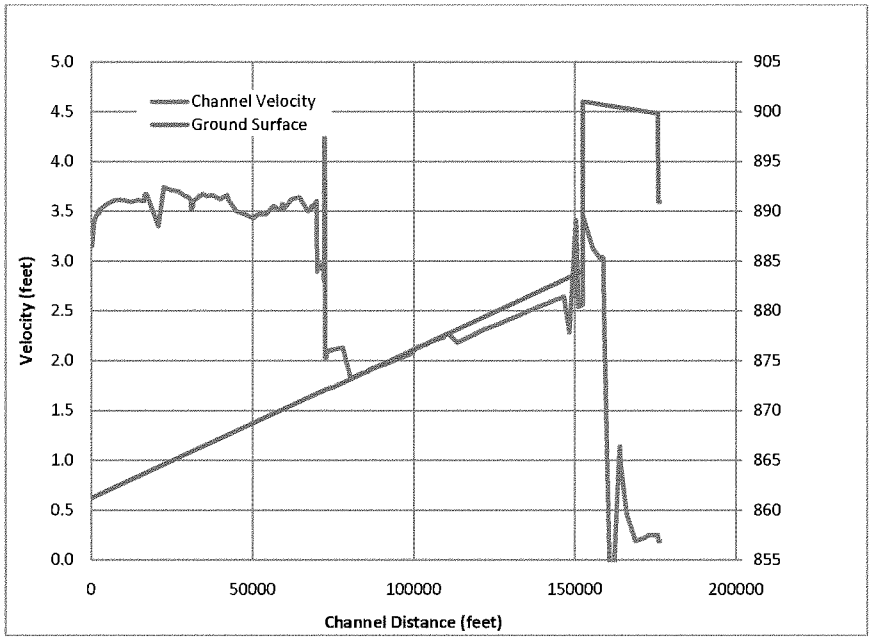


Figure F-G23 LPP Diversion Channel Velocities for the 500-Year Event

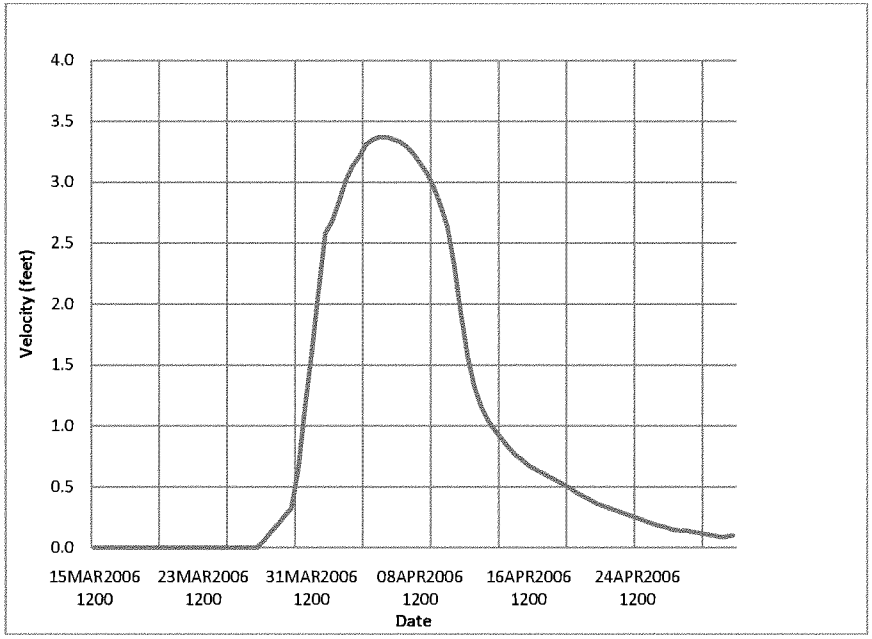


Figure F-G24 LPP Diversion Channel Velocity Profile for the 10-Year Event at Station 152527

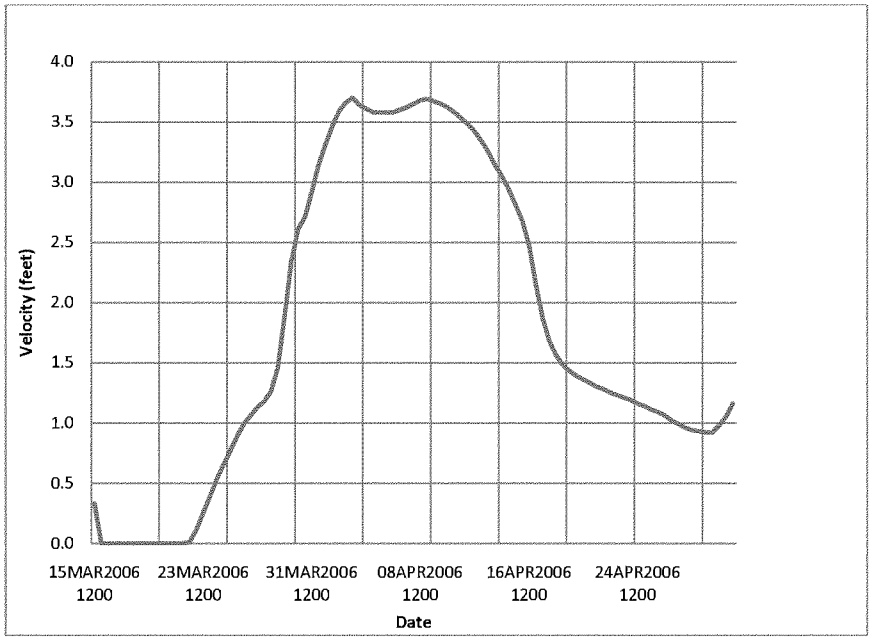


Figure F-G25 LPP Diversion Channel Velocity Profile for the 50-Year Event at Station 152527

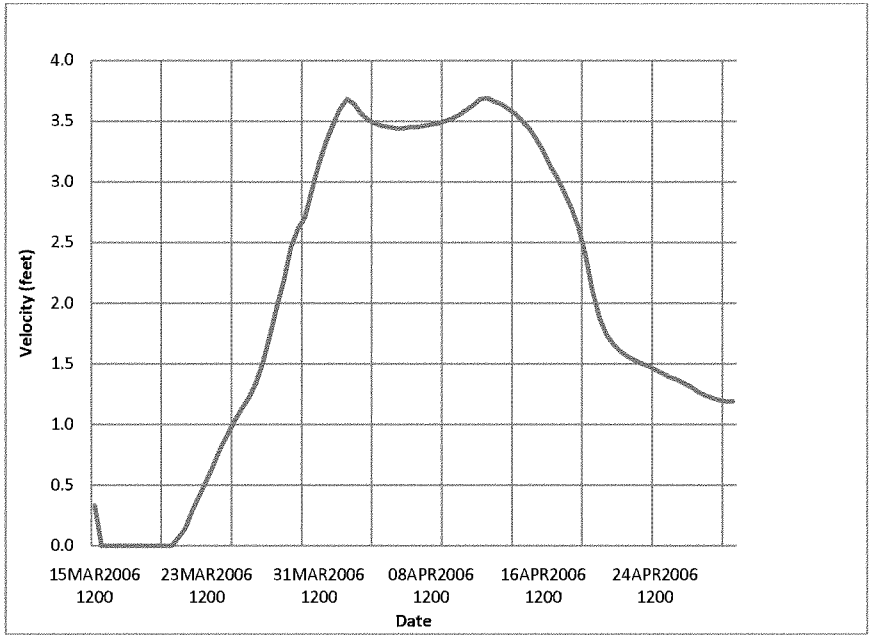


Figure F-G26 LPP Diversion Channel Velocity Profile for the 100-Year Event at Station 152527

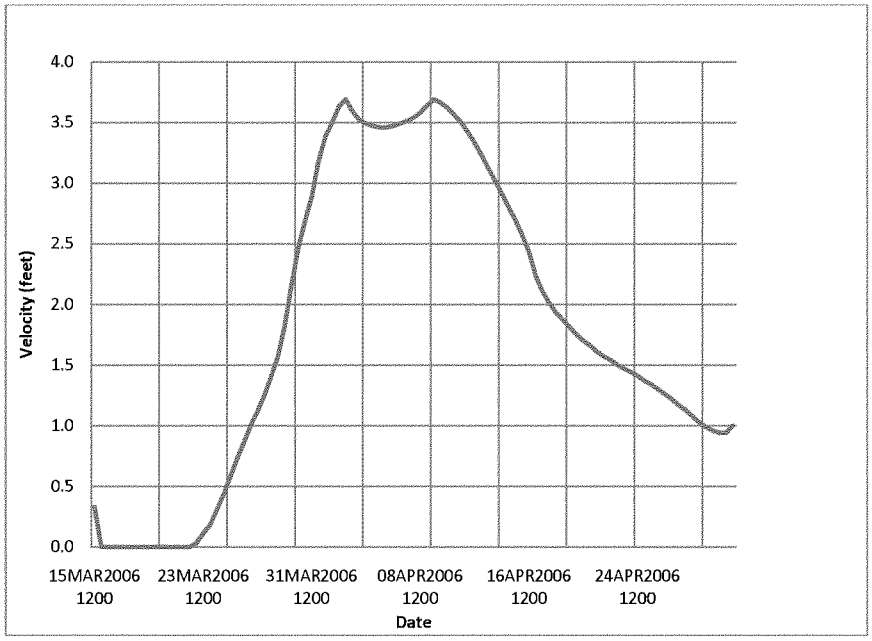


Figure F-G27 LPP Diversion Channel Velocity Profile for the 500-Year Event at Station 152527

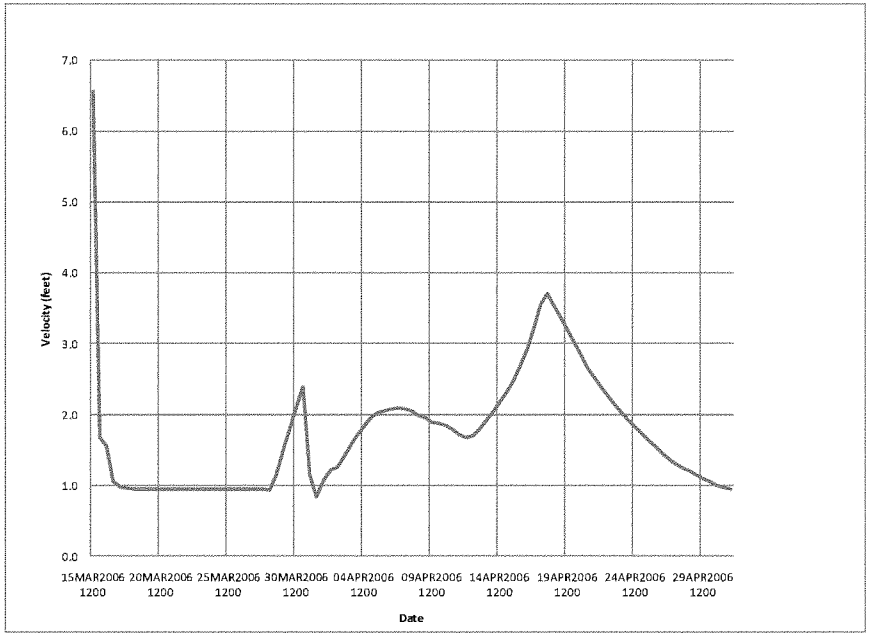


Figure F-G28 LPP Diversion Channel Velocity Profile for the 10-Year Event at Station 83654

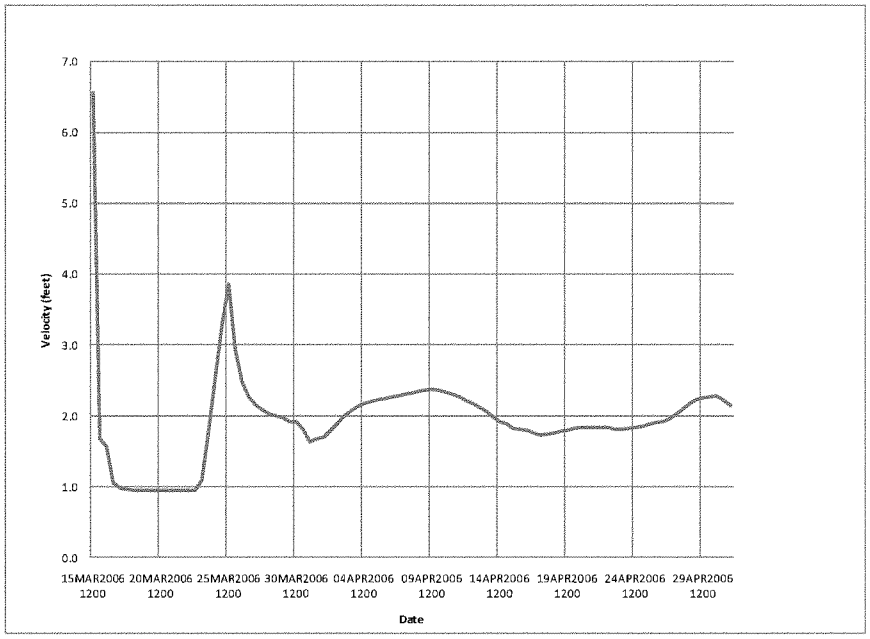


Figure F-G29 LPP Diversion Channel Velocity Profile for the 50-Year Event at Station 83654

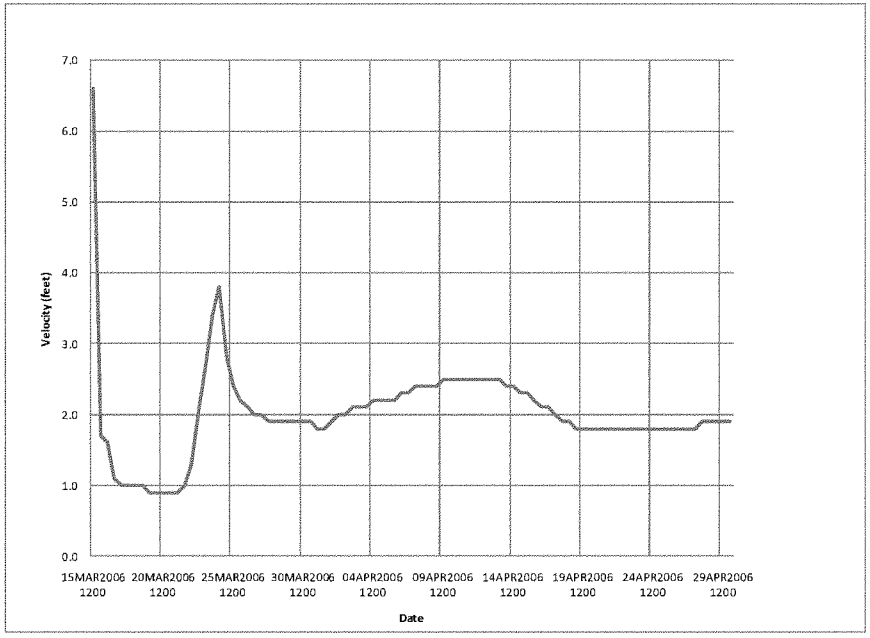


Figure F-G30 LPP Diversion Channel Velocity Profile for the 100-Year Event at Station 83654

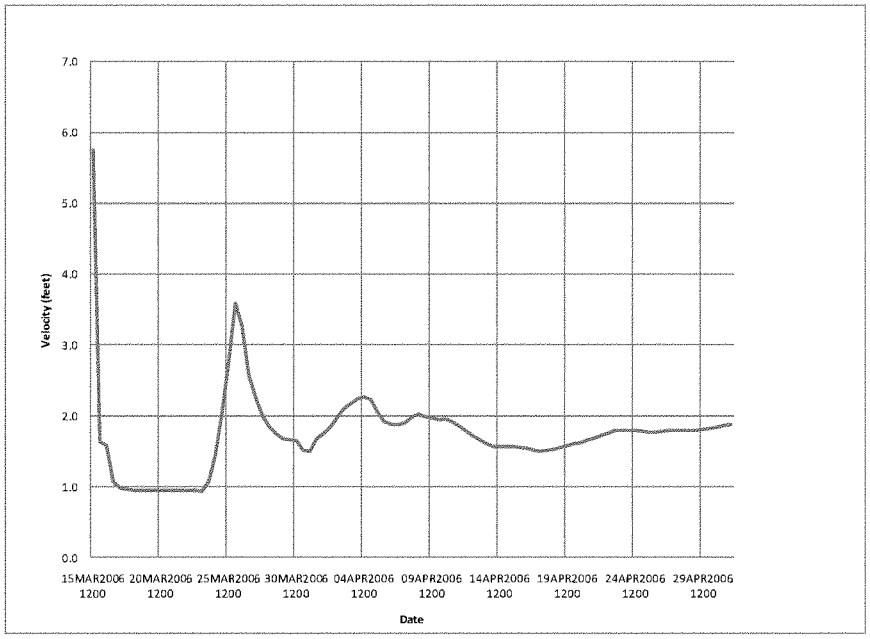


Figure F-G31 LPP Diversion Channel Velocity Profile for the 500-Year Event at Station 83654

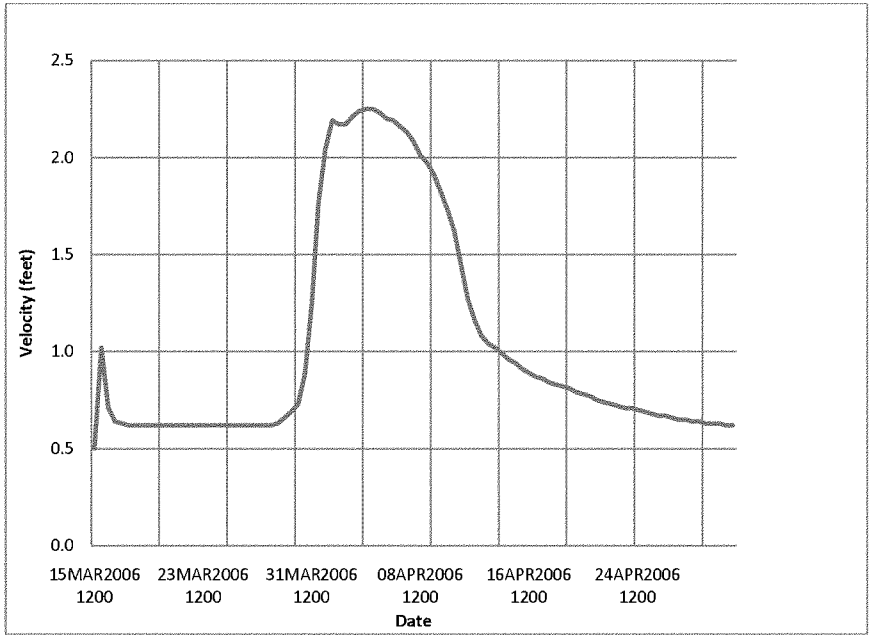


Figure F-G32 LPP Diversion Channel Velocity Profile for the 10-Year Event at Station 64696

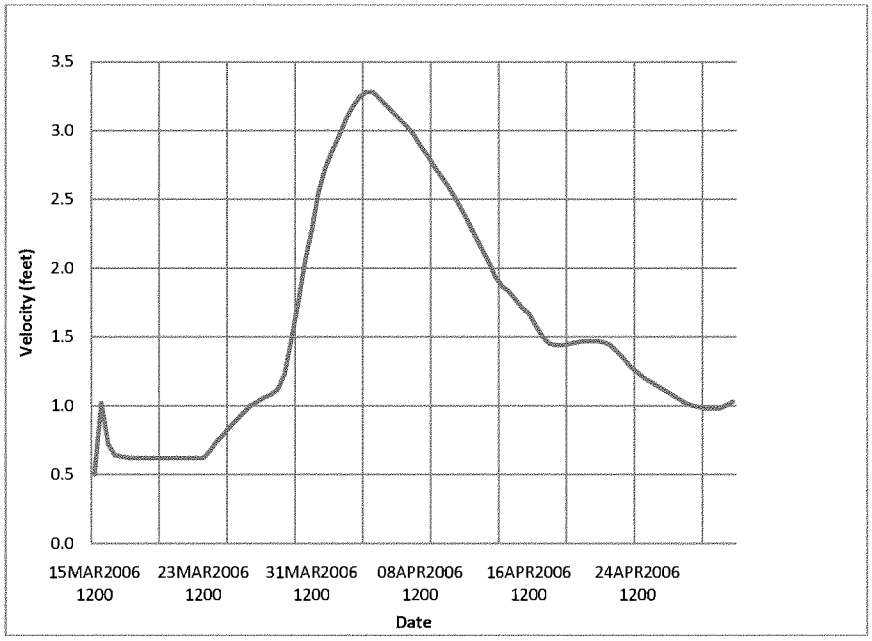


Figure F-G33 LPP Diversion Channel Velocity Profile for the 50-Year Event at Station 64696

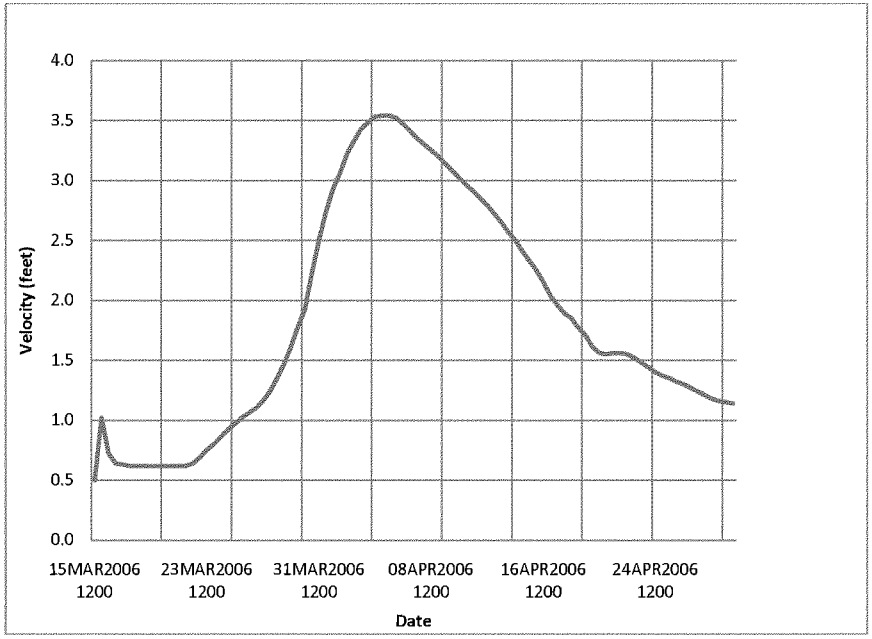


Figure F-G34 LPP Diversion Channel Velocity Profile for the 100-Year Event at Station 64696

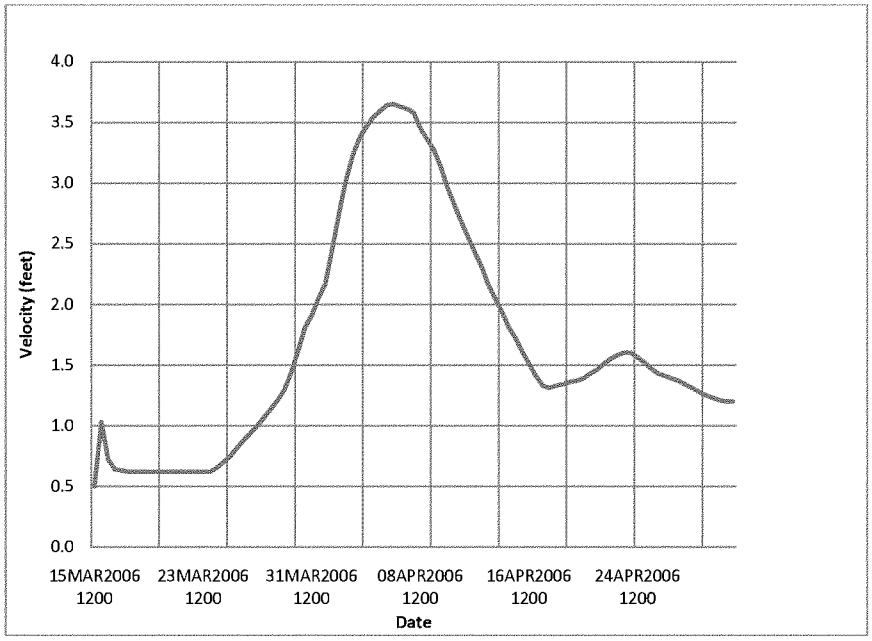


Figure F-G35 LPP Diversion Channel Velocity Profile for the 500-Year Event at Station 64696

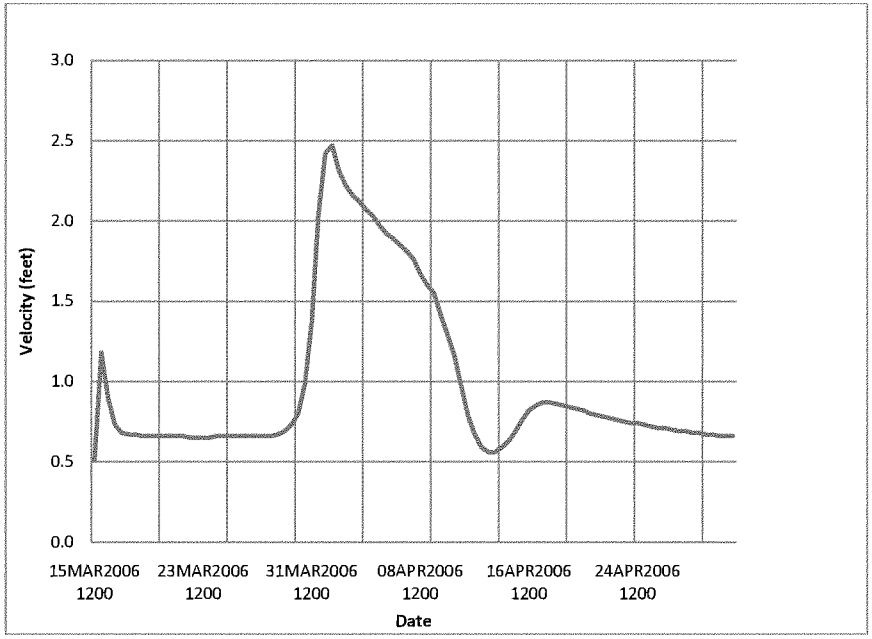


Figure F-G36 LPP Diversion Channel Velocity Profile for the 10-Year Event at Station 29253

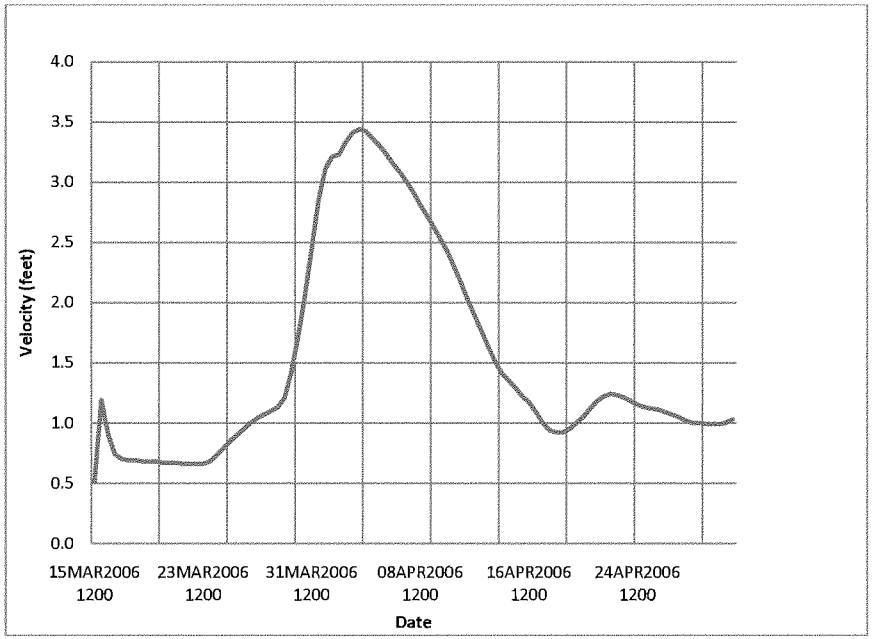


Figure F-G37 LPP Diversion Channel Velocity Profile for the 50-Year Event at Station 29253

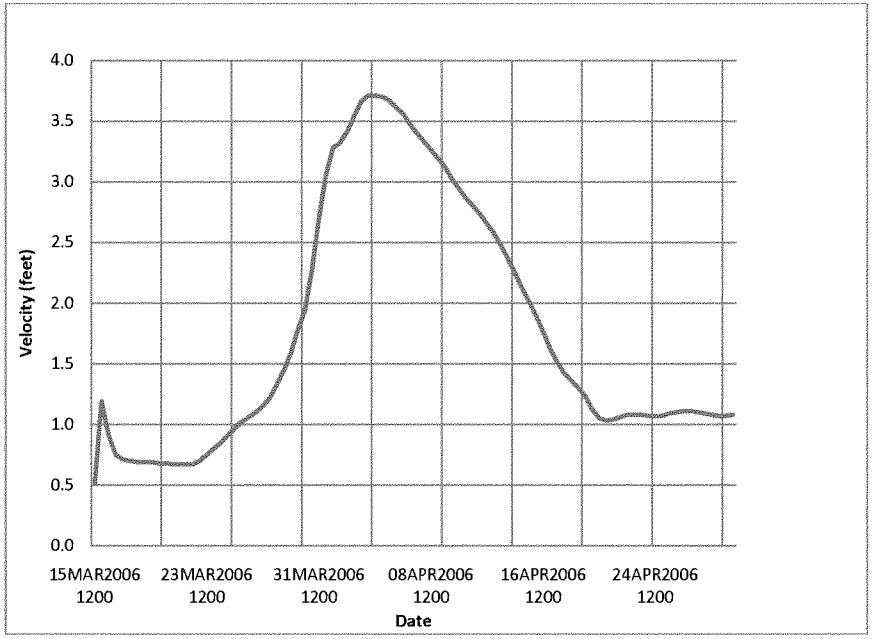


Figure F-G38 LPP Diversion Channel Velocity Profile for the 100-Year Event at Station 29253

F-G2.2 VELOCITIES IN THE FCP DIVERSION CHANNEL – YEAR 0 HYDROLOGY

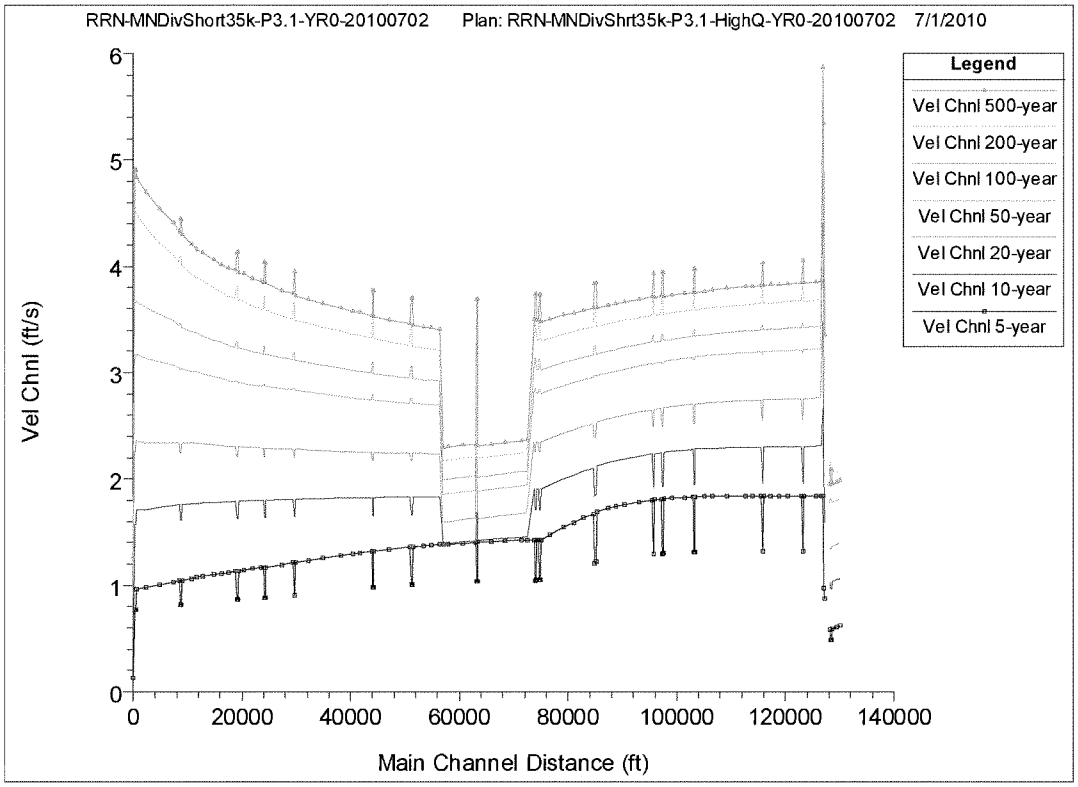


Figure F-G39 FCP Diversion Channel Velocities for All Modeled Events for Year 0 Hydrology

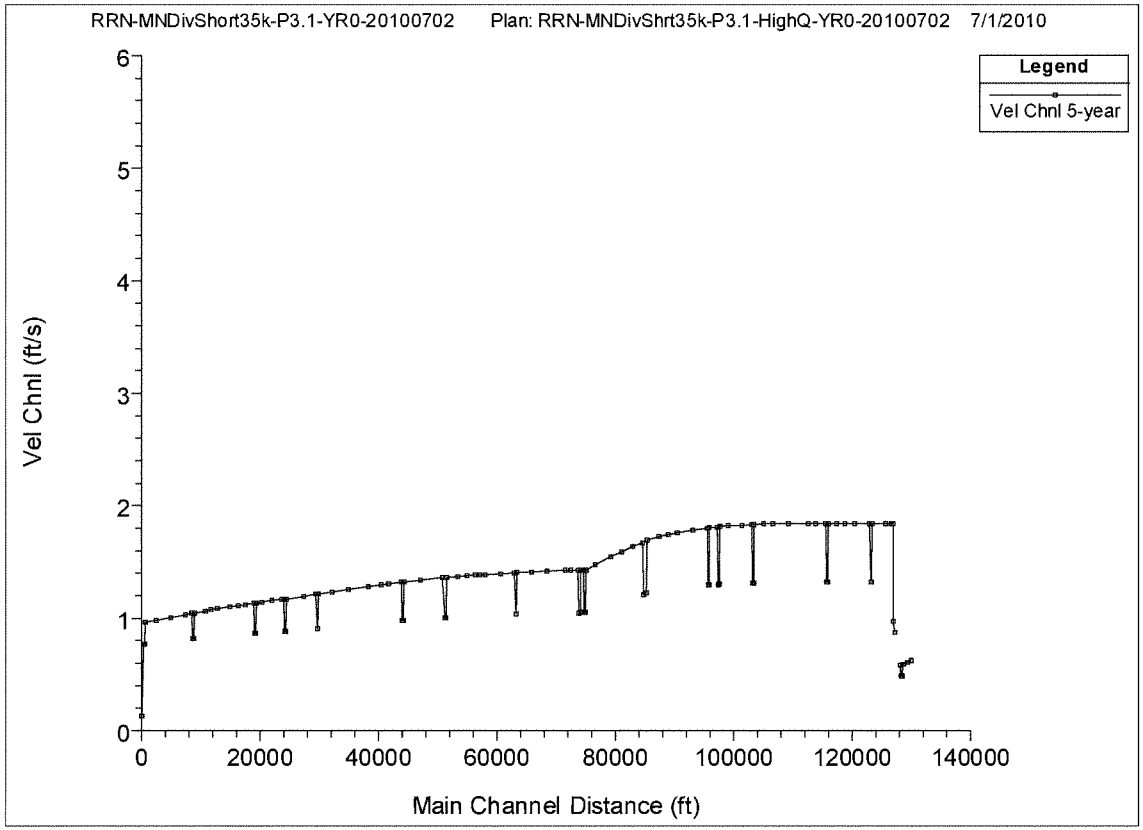


Figure F-G40 FCP Diversion Channel Velocities for the 5-Year Event for Year 0 Hydrology

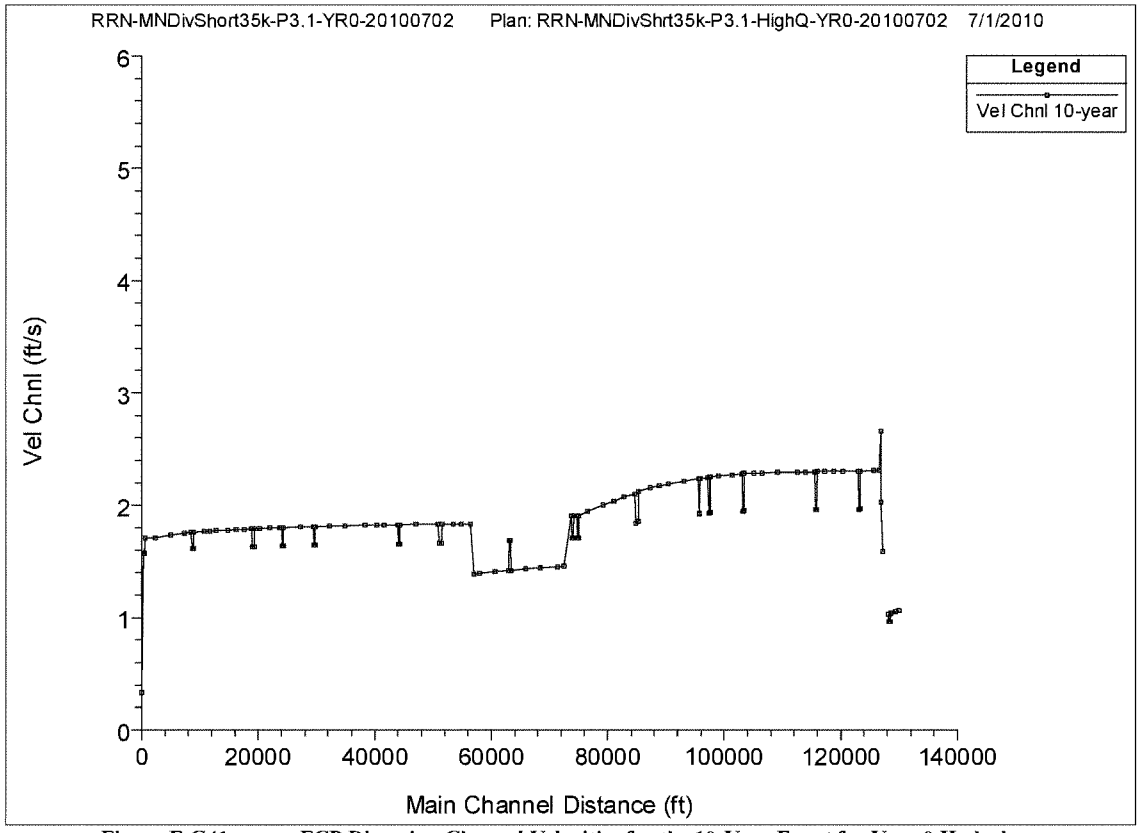


Figure F-G41 FCP Diversion Channel Velocities for the 10-Year Event for Year 0 Hydrology

1040

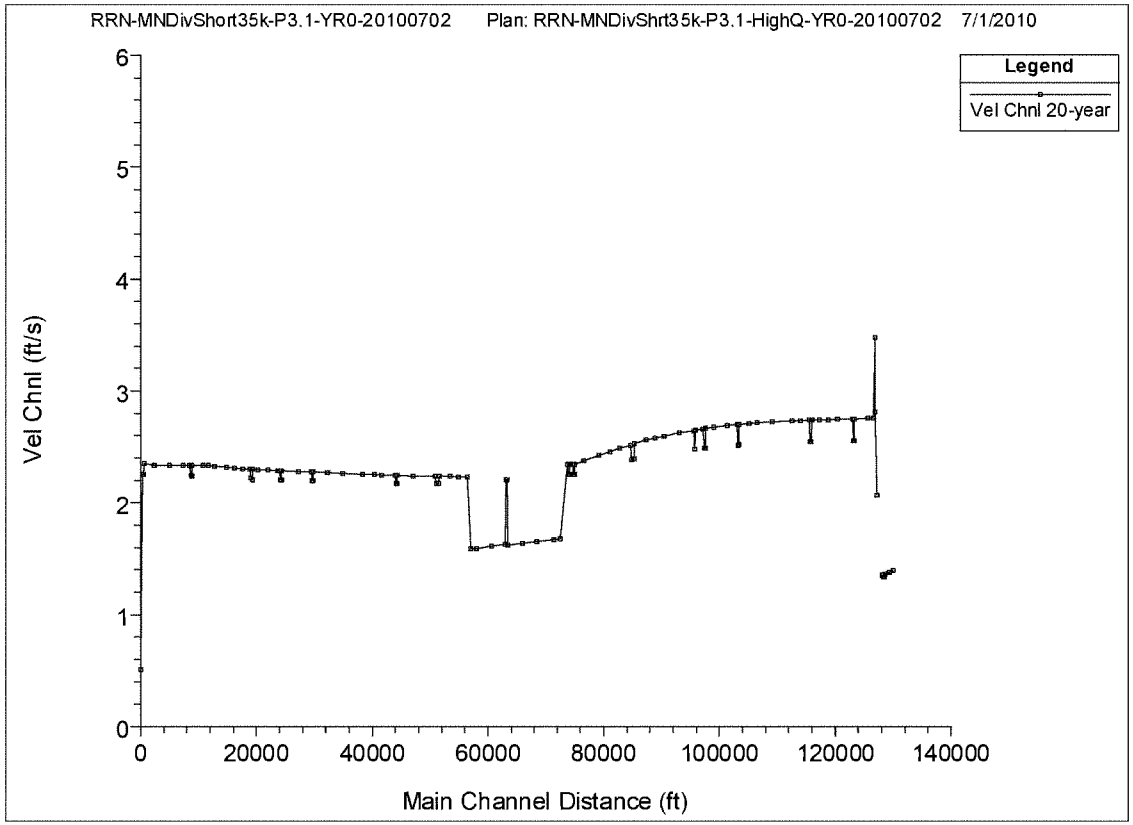


Figure F-G42 FCP Diversion Channel Velocities for the 20-Year Event for Year 0 Hydrology

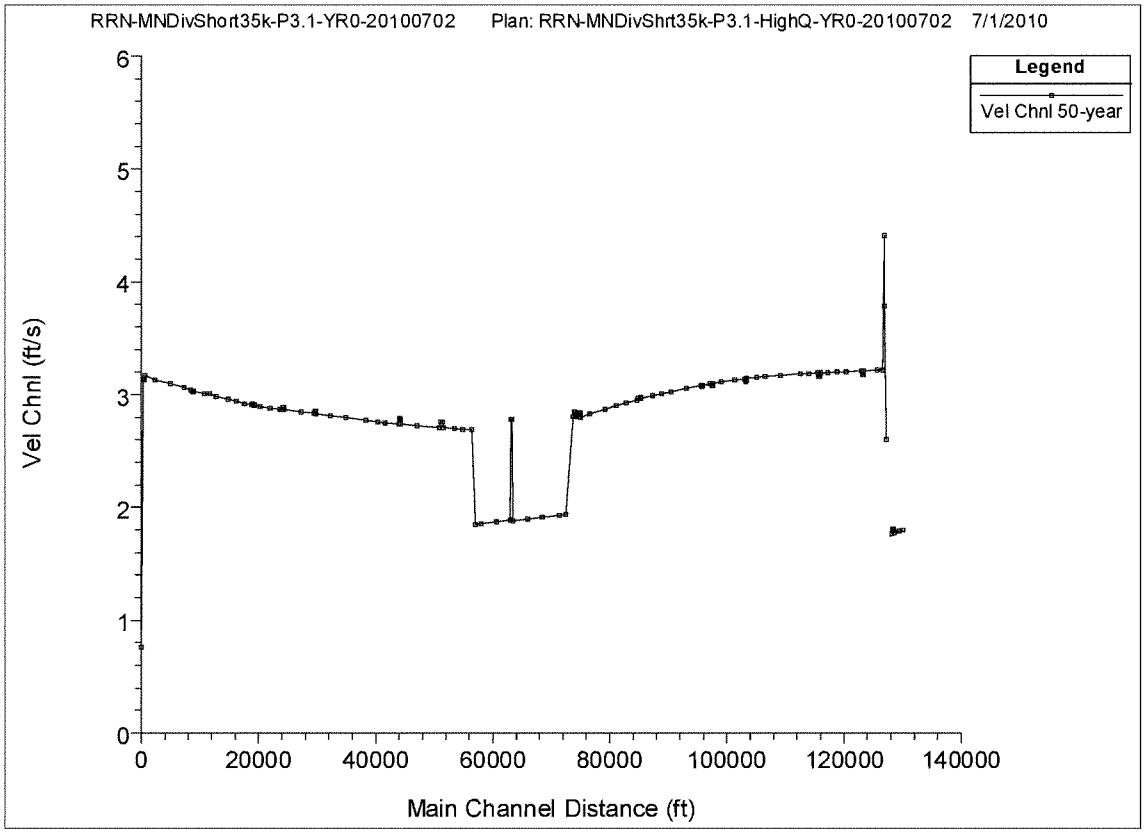


Figure F-G43 FCP Diversion Channel Velocities for the 50-Year Event for Year 0 Hydrology

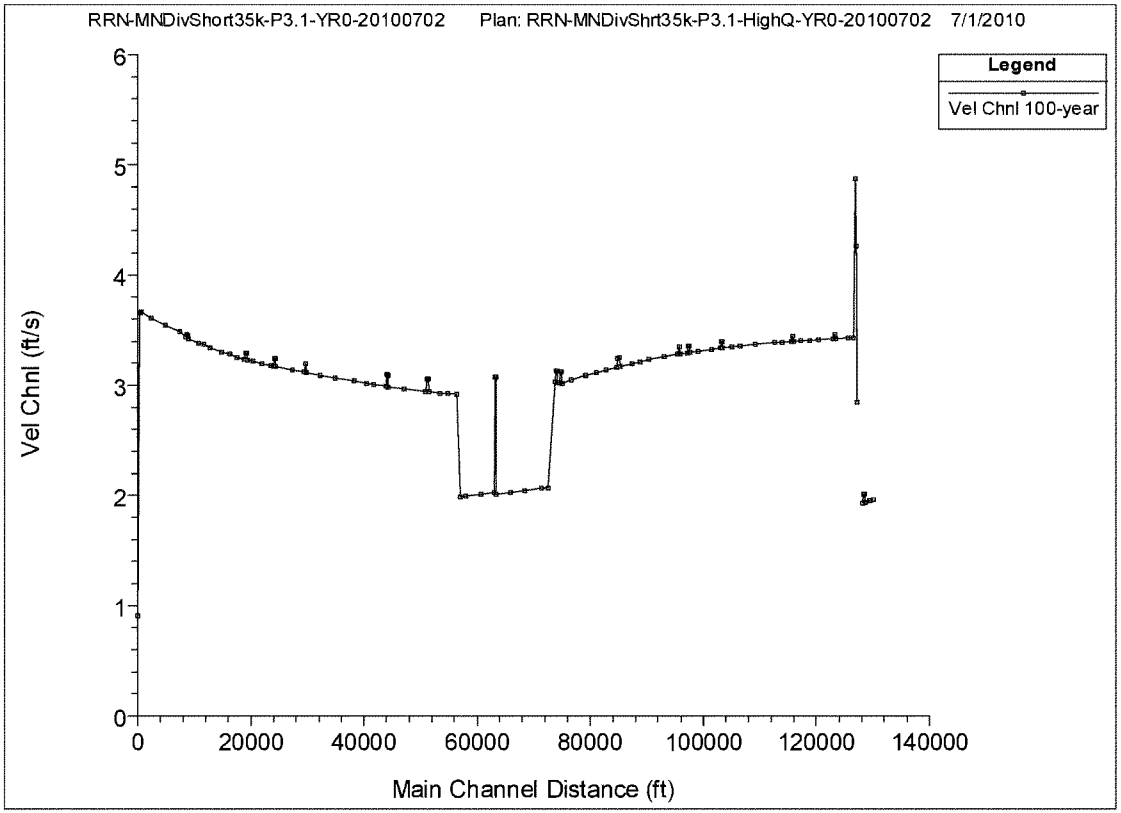


Figure F-G44 FCP Diversion Channel Velocities for the 100-Year Event for Year 0 Hydrology

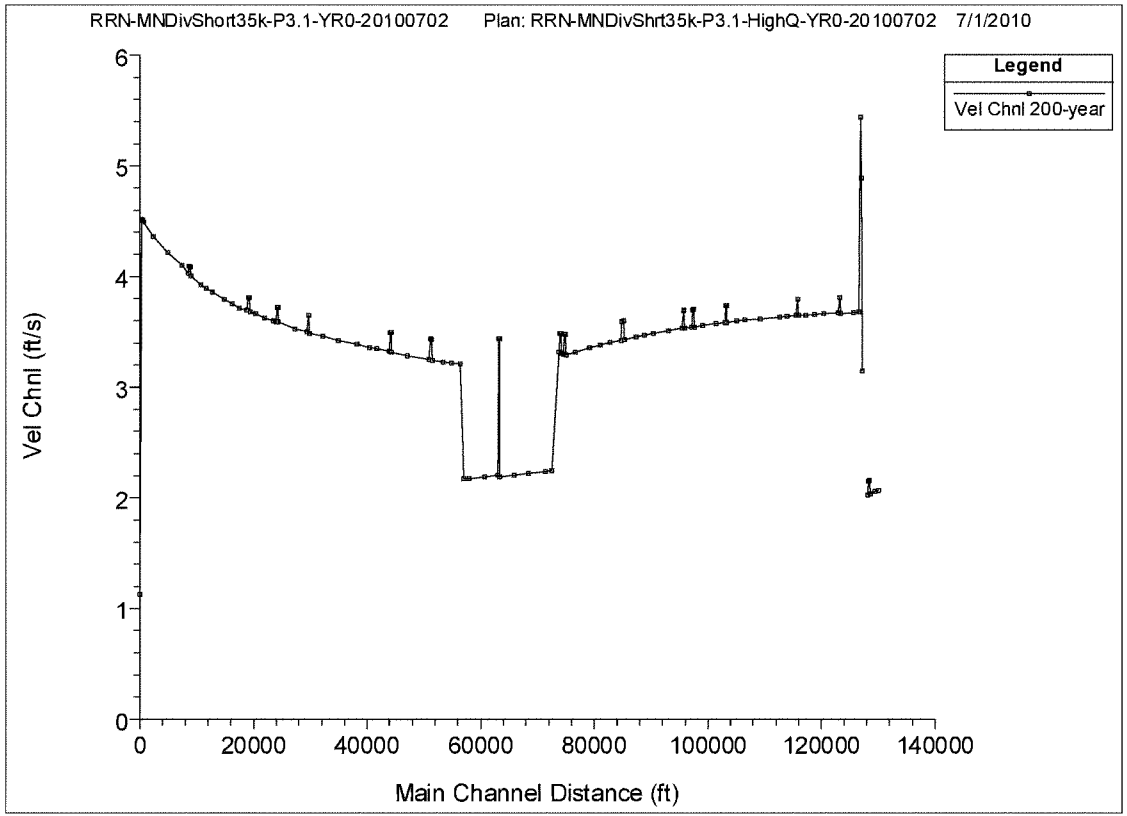


Figure F-G45 FCP Diversion Channel Velocities for the 200-Year Event for Year 0 Hydrology

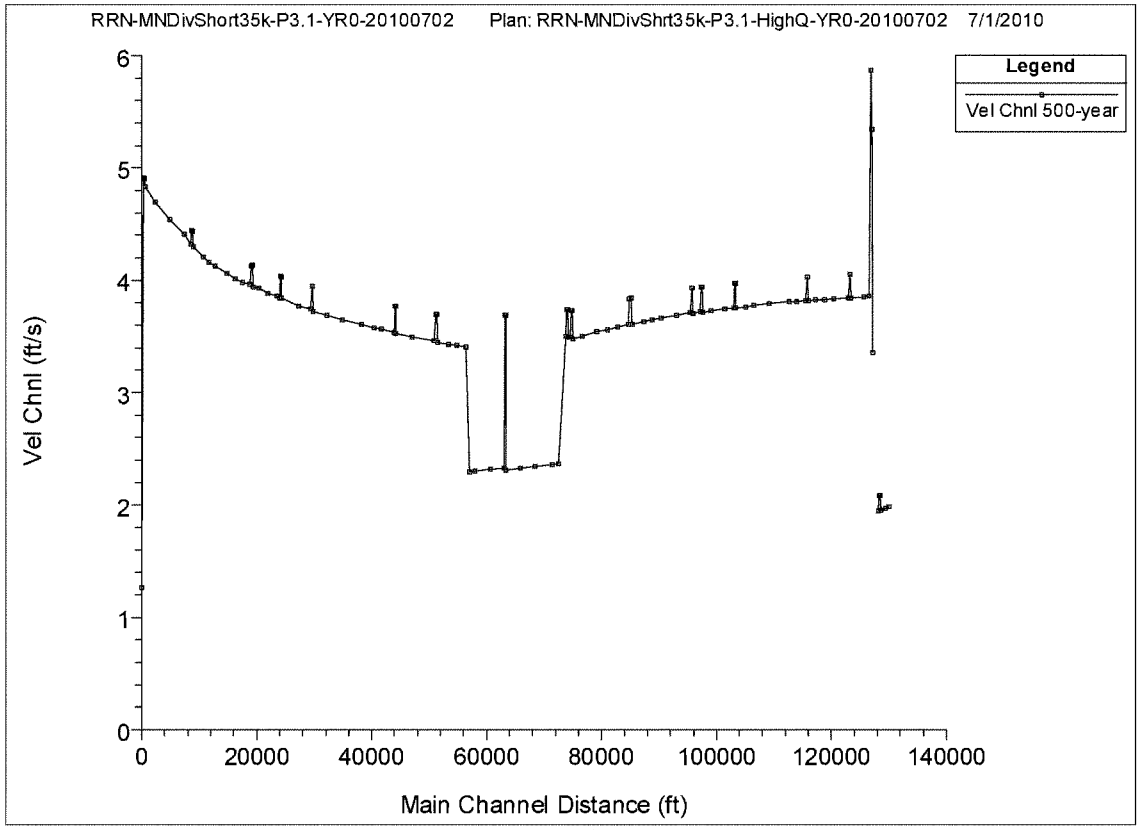


Figure F-G46 FCP Diversion Channel Velocities for the 500-Year Event for Year 0 Hydrology

F-G2.3VELOCITIES IN THE FCP DIVERSION CHANNEL – YEAR 25 HYDROLOGY

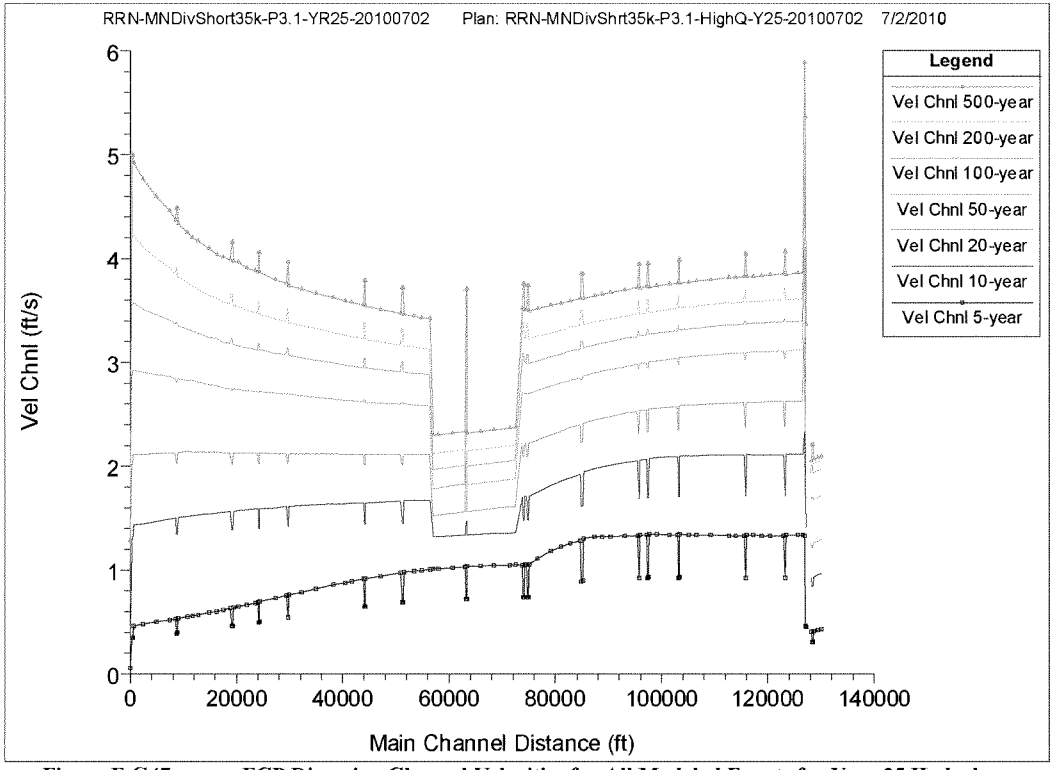


Figure F-G47 FCP Diversion Channel Velocities for All Modeled Events for Year 25 Hydrology

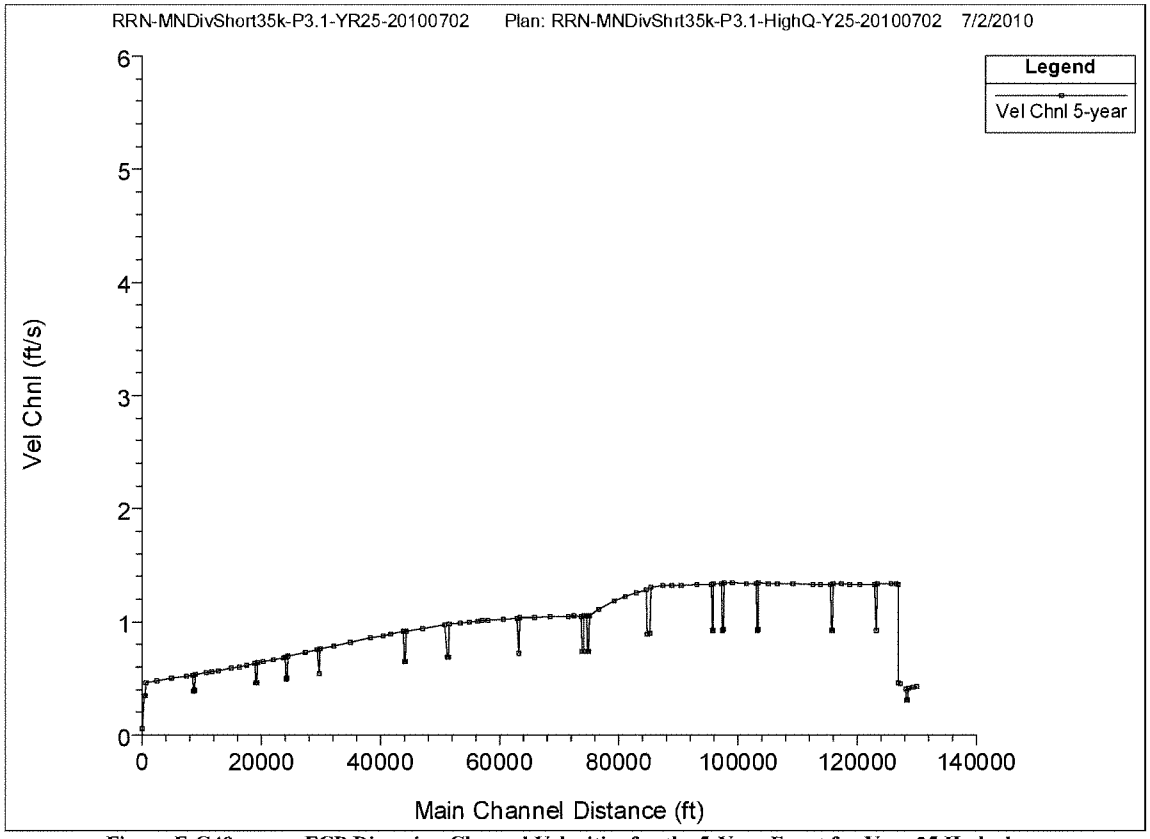


Figure F-G48 FCP Diversion Channel Velocities for the 5-Year Event for Year 25 Hydrology

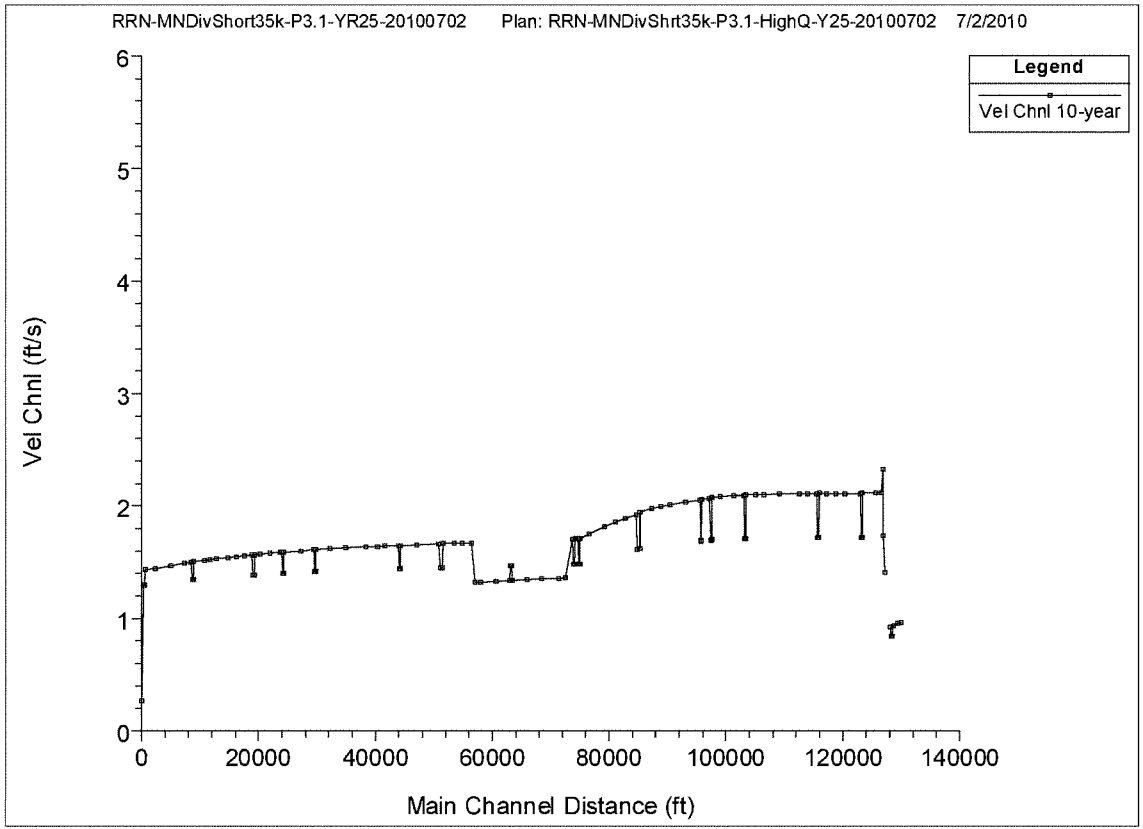


Figure F-G49 FCP Diversion Channel Velocities for the 10-Year Event for Year 25 Hydrology

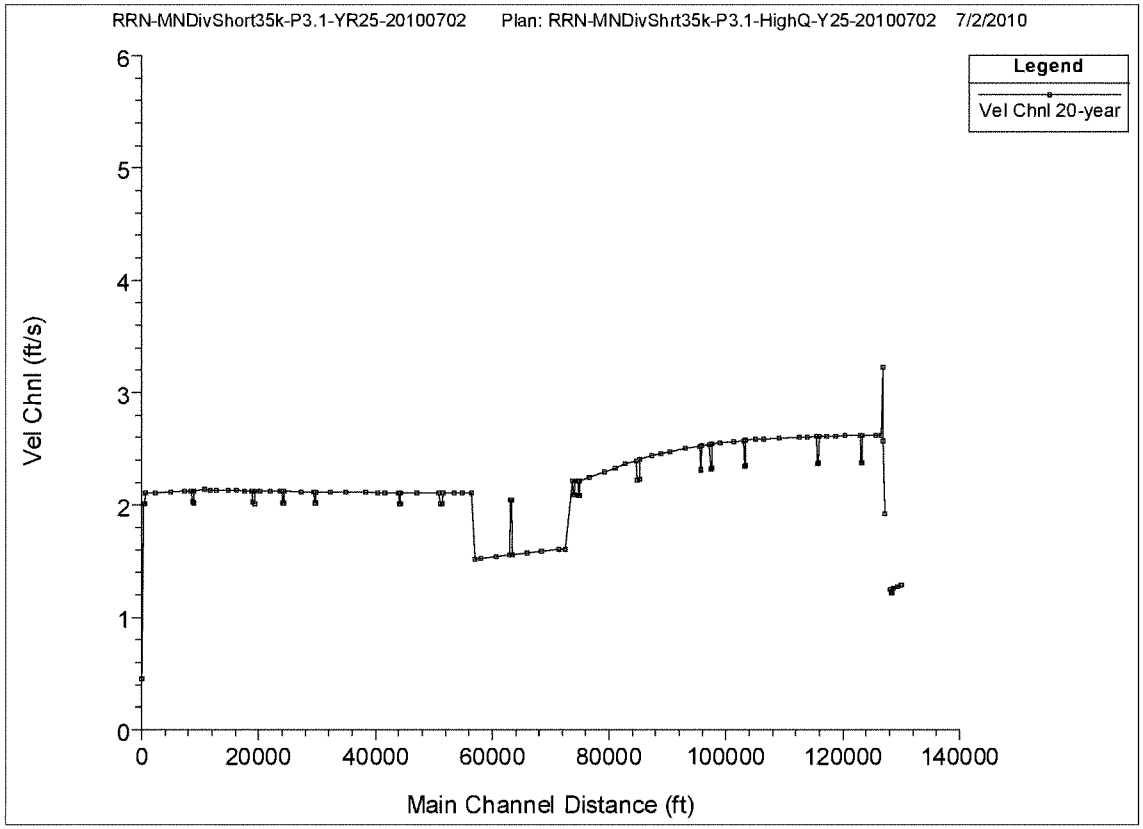


Figure F-G50 FCP Diversion Channel Velocities for the 20-Year Event for Year 25 Hydrology

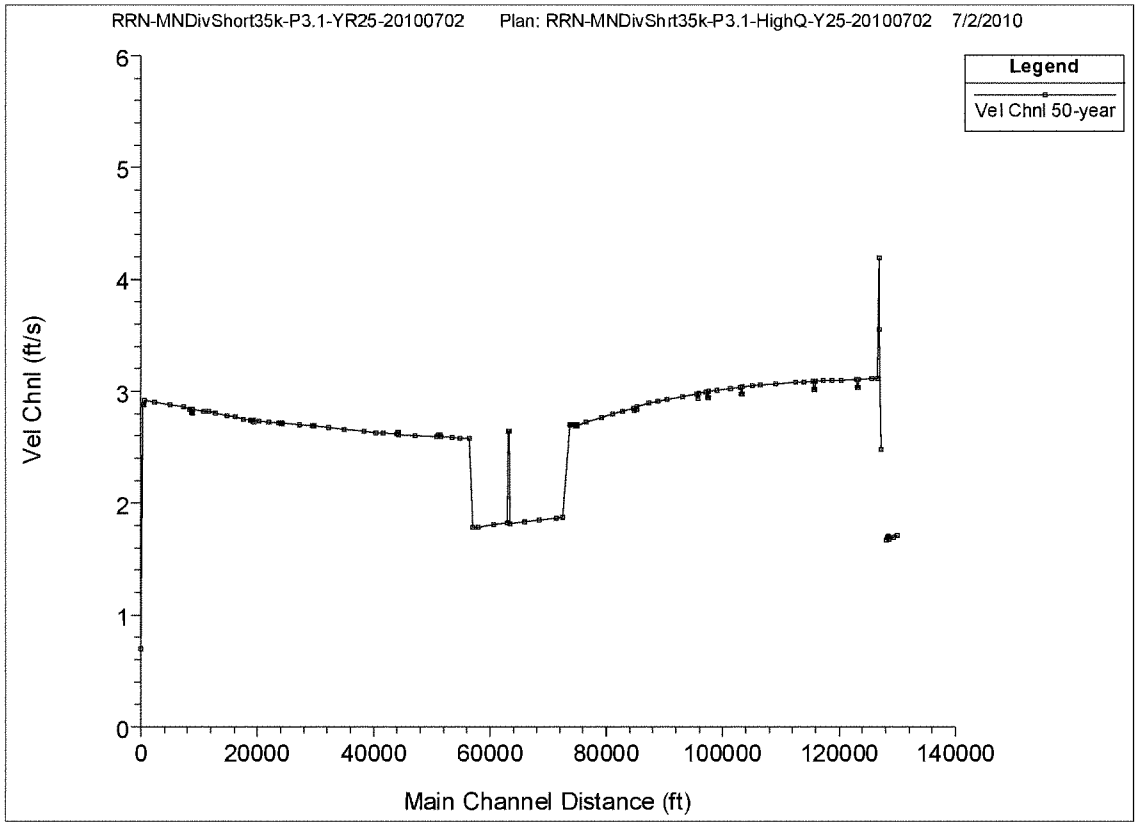


Figure F-G51 FCP Diversion Channel Velocities for the 50-Year Event for Year 25 Hydrology

1050

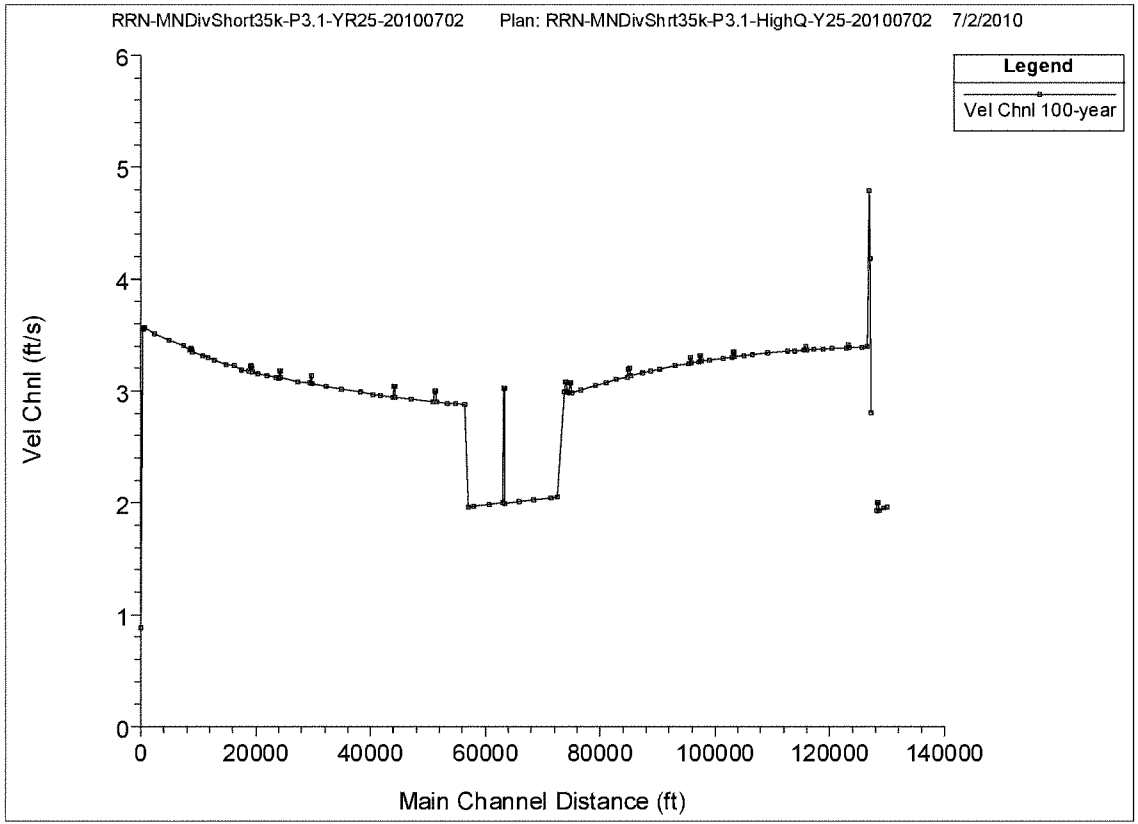


Figure F-G52 FCP Diversion Channel Velocities for the 100-Year Event for Year 25 Hydrology

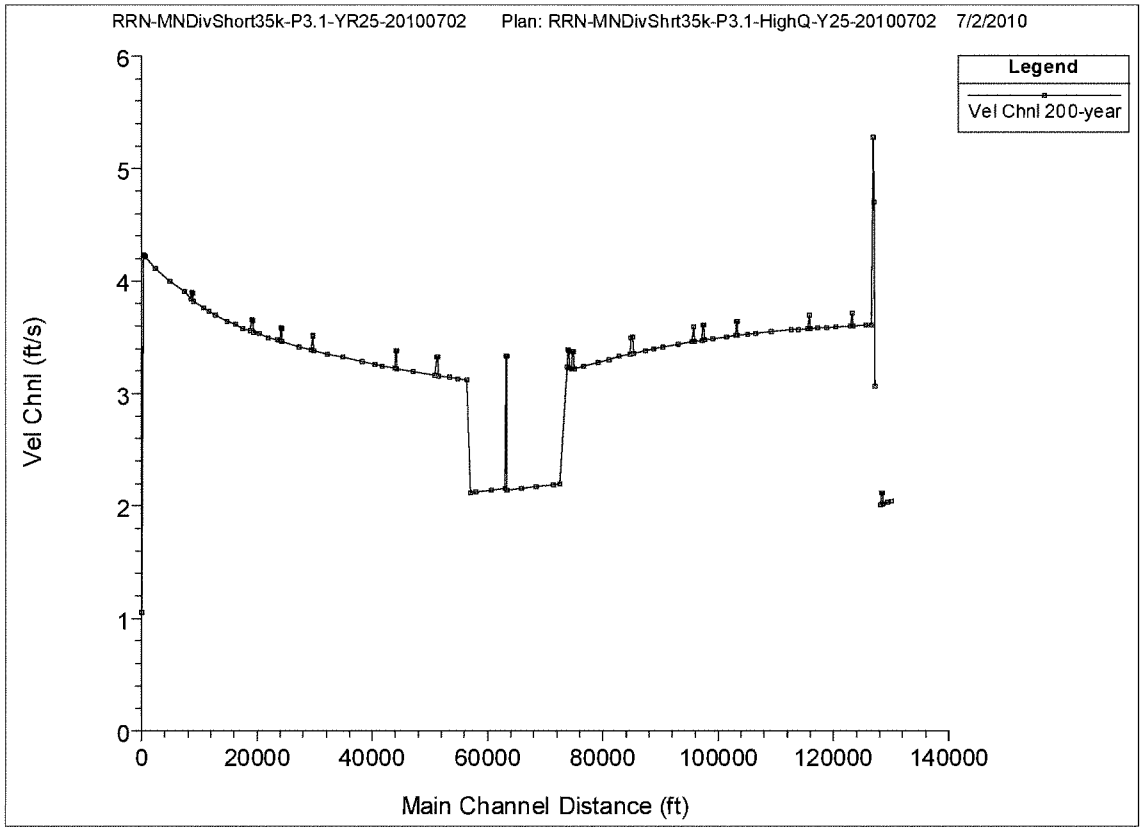


Figure F-G53 FCP Diversion Channel Velocities for the 200-Year Event for Year 25 Hydrology

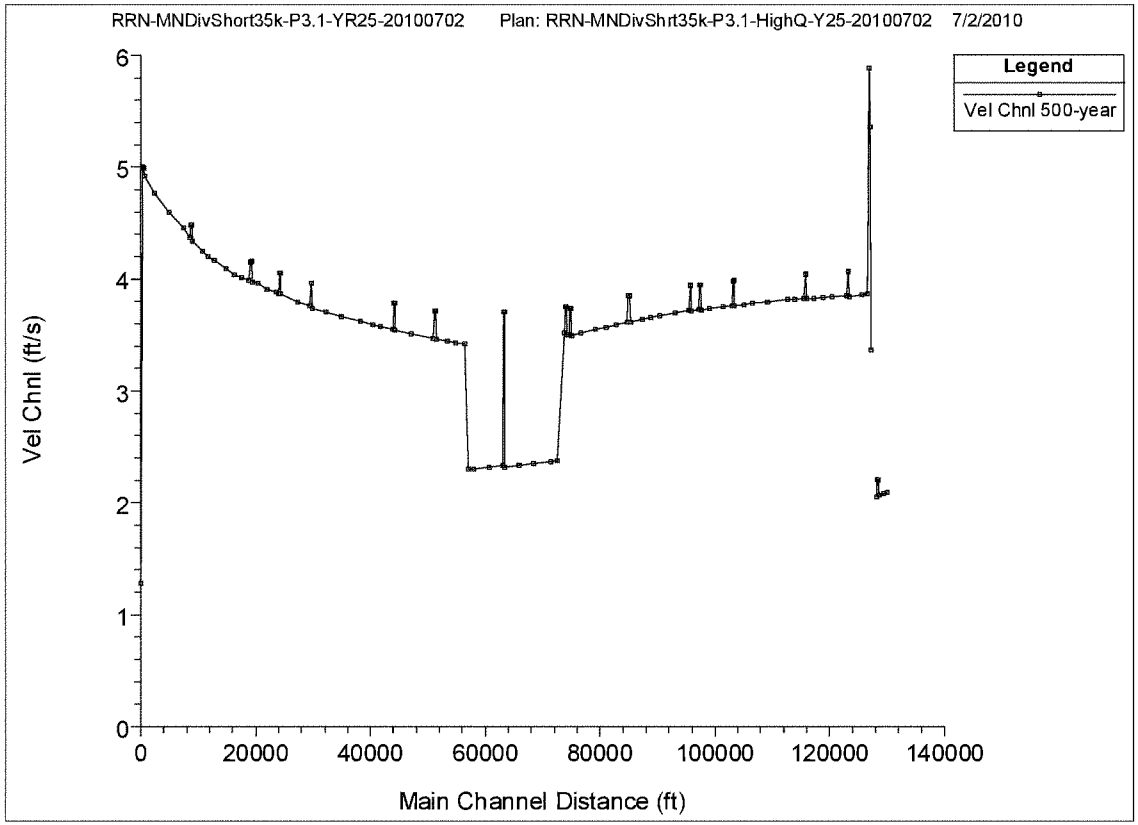


Figure F-G54 FCP Diversion Channel Velocities for the 500-Year Event for Year 25 Hydrology

F-G2.4VELOCITIES IN THE FCP DIVERSION CHANNEL – YEAR 50 HYDROLOGY

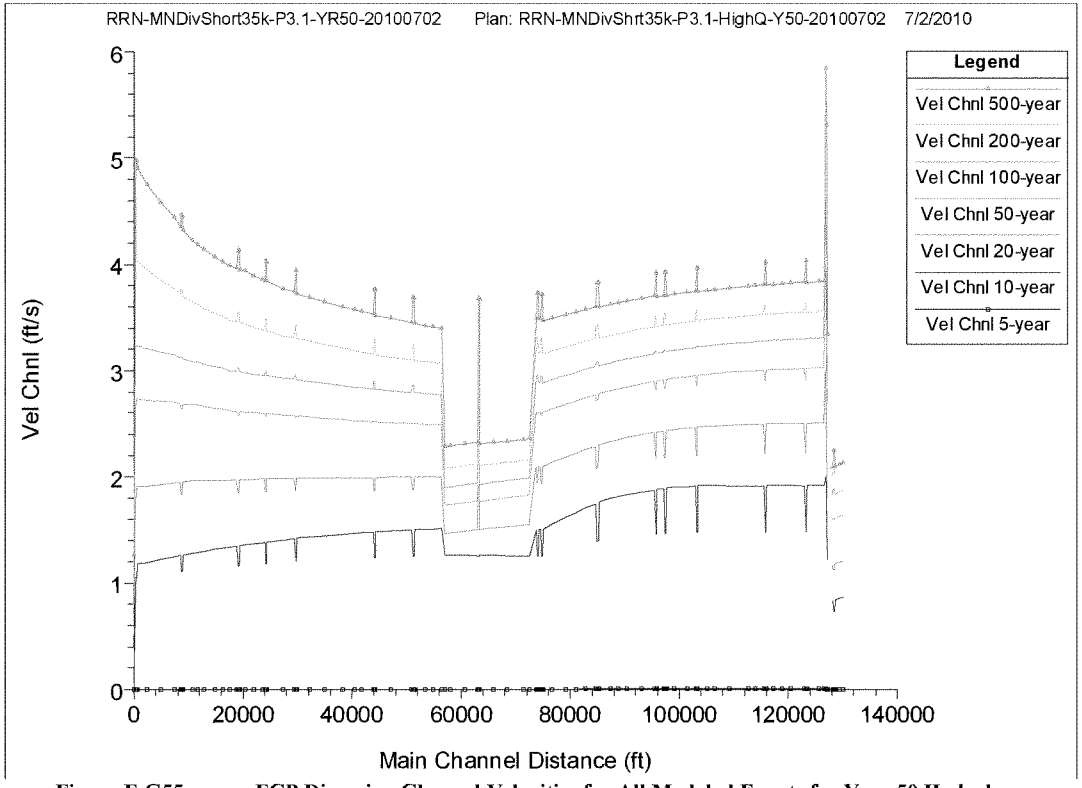


Figure F-G55 **FCP Diversion Channel Velocities for All Modeled Events for Year 50 Hydrology**

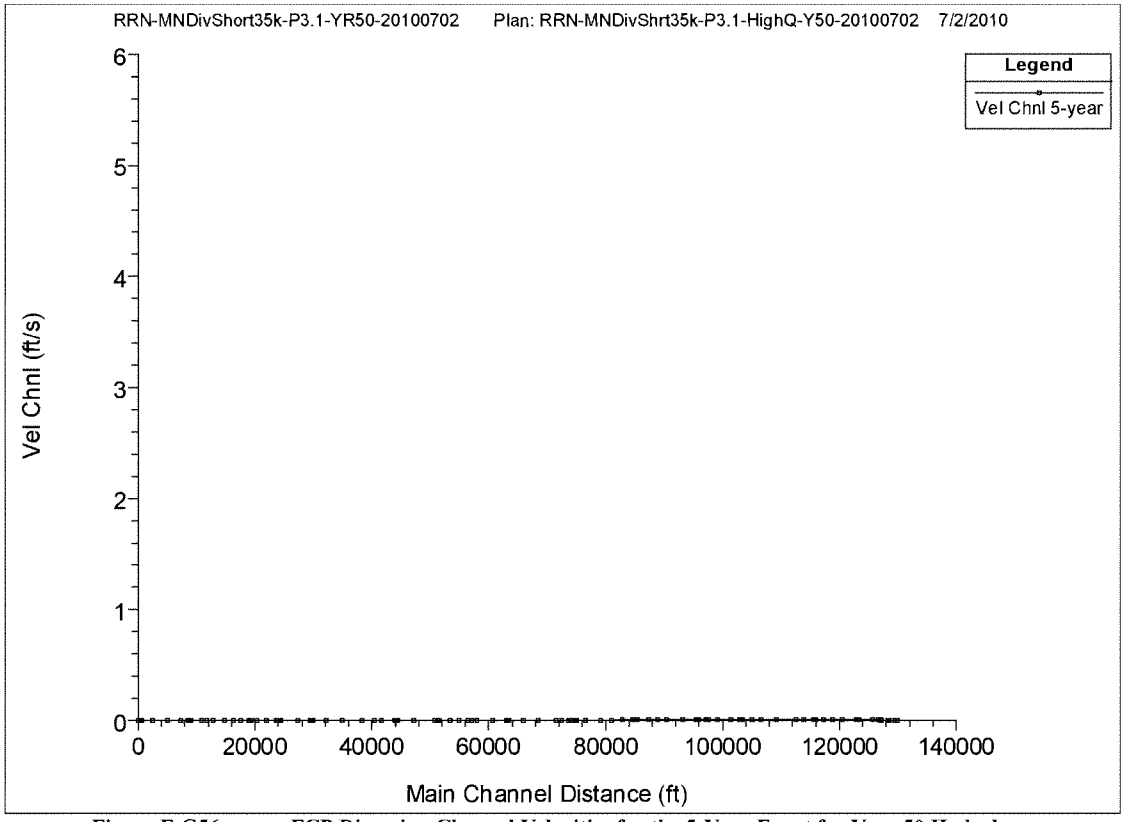


Figure F-G56 FCP Diversion Channel Velocities for the 5-Year Event for Year 50 Hydrology

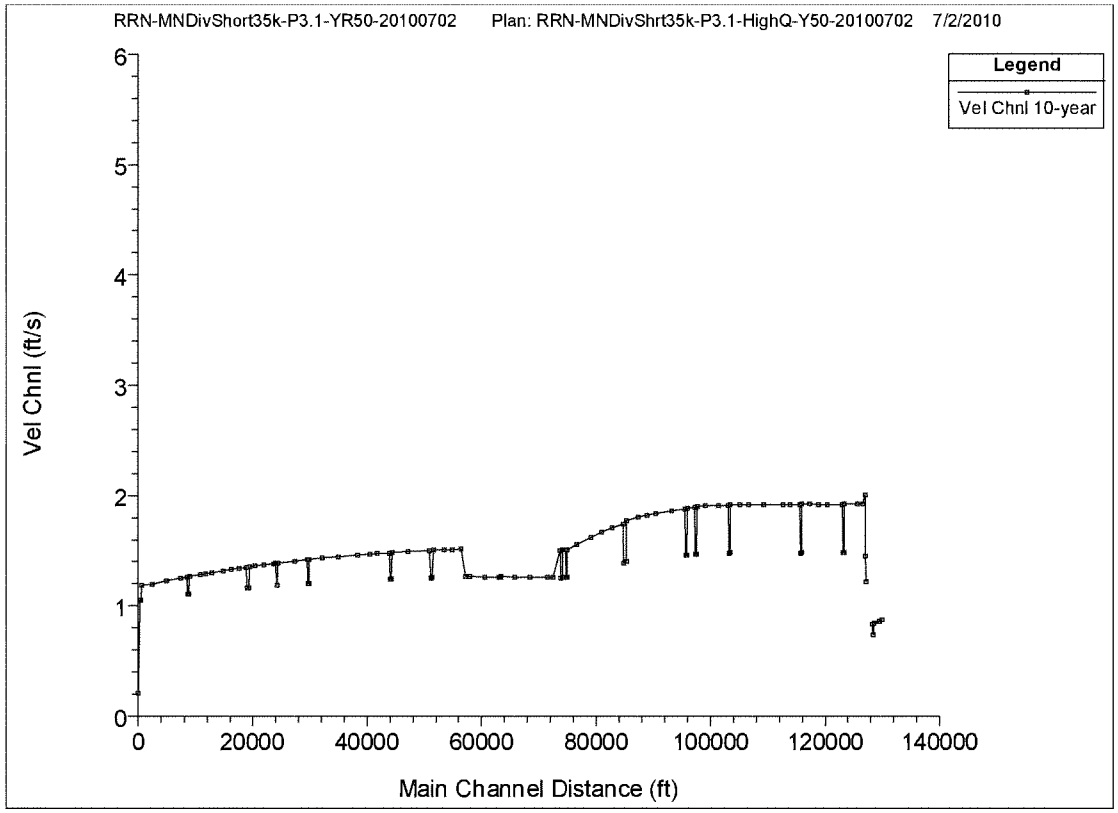


Figure F-G57 FCP Diversion Channel Velocities for the 10-Year Event for Year 50 Hydrology

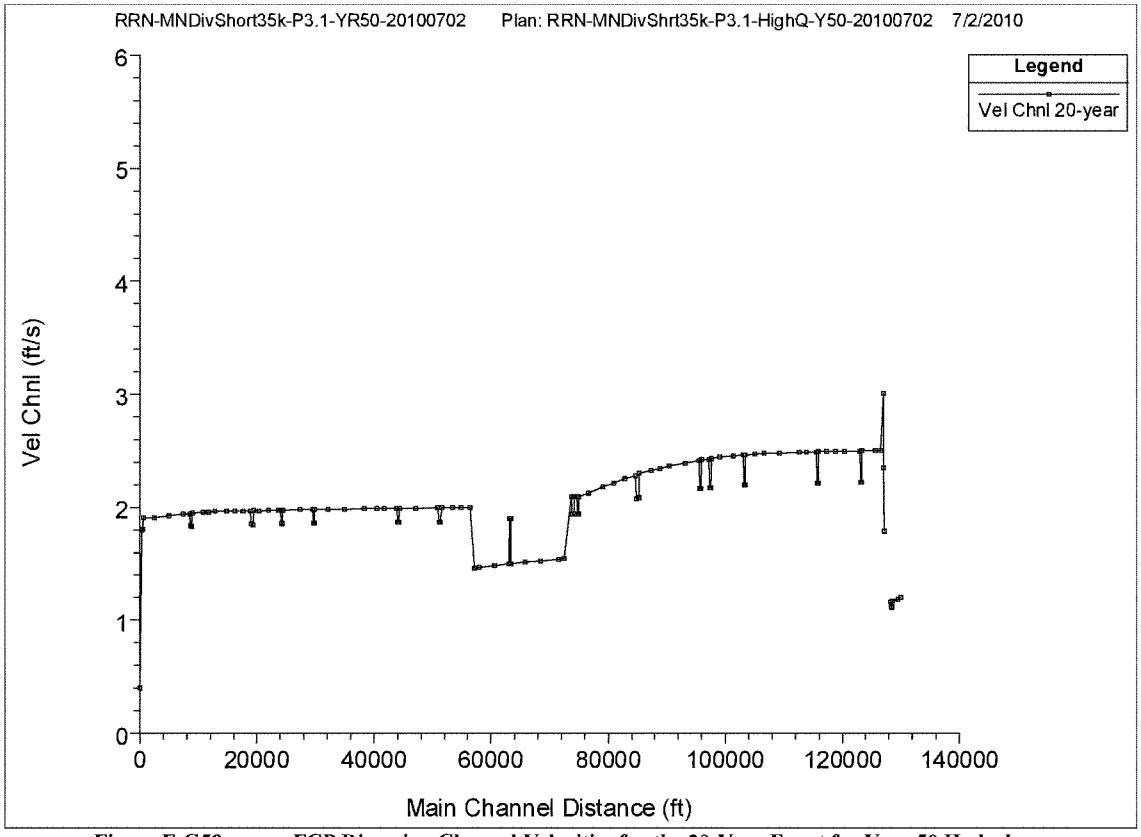
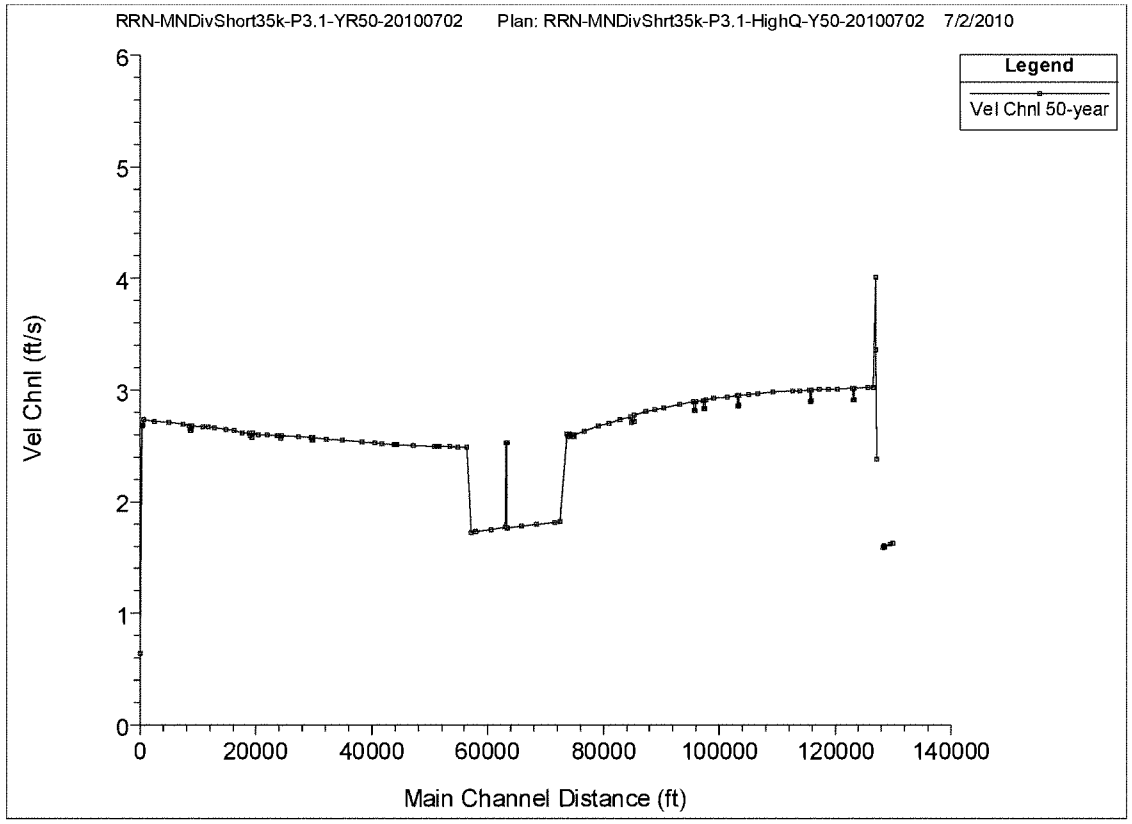


Figure F-G58 FCP Diversion Channel Velocities for the 20-Year Event for Year 50 Hydrology



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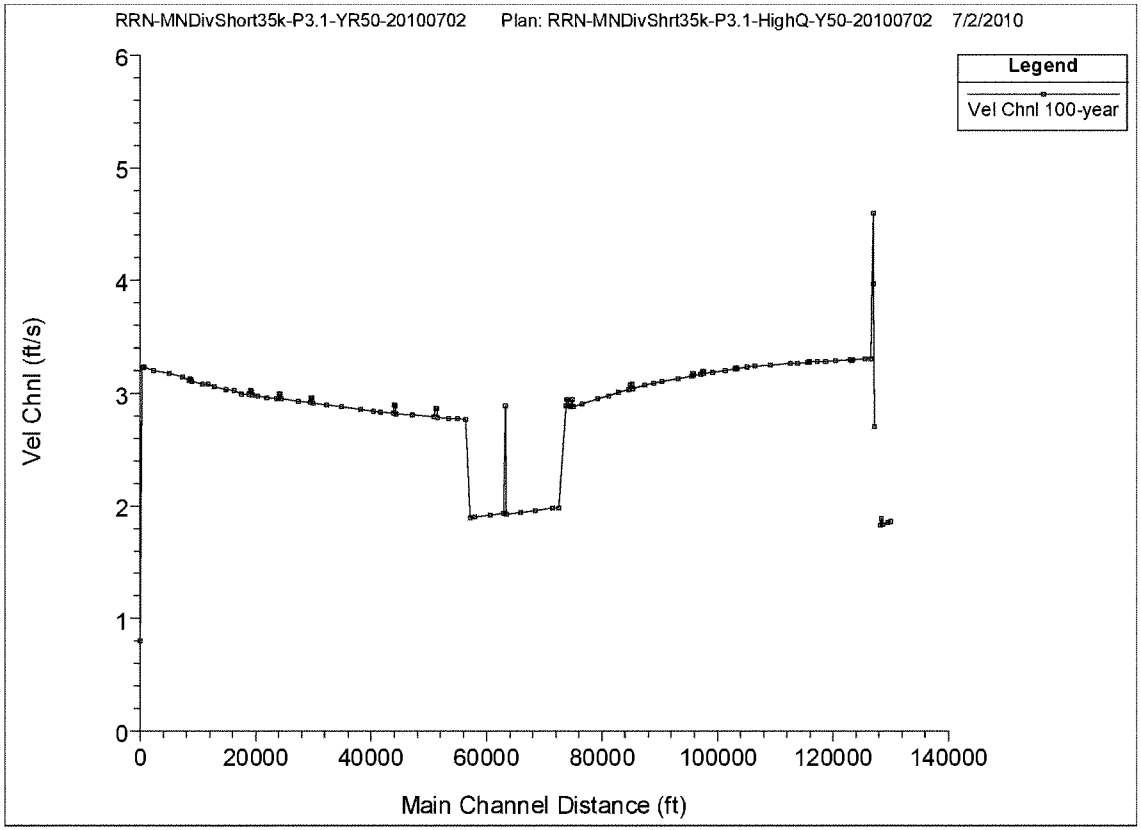
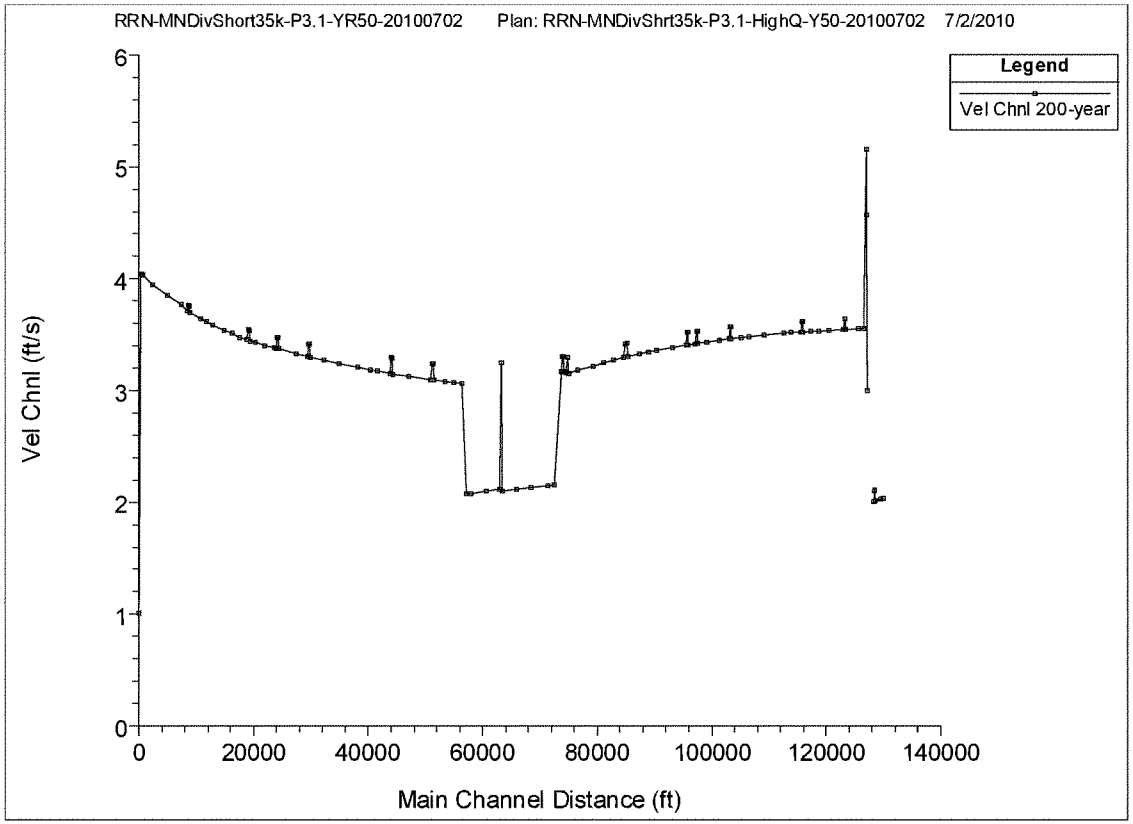


Figure F-G60 FCP Diversion Channel Velocities for the 100-Year Event for Year 50 Hydrology



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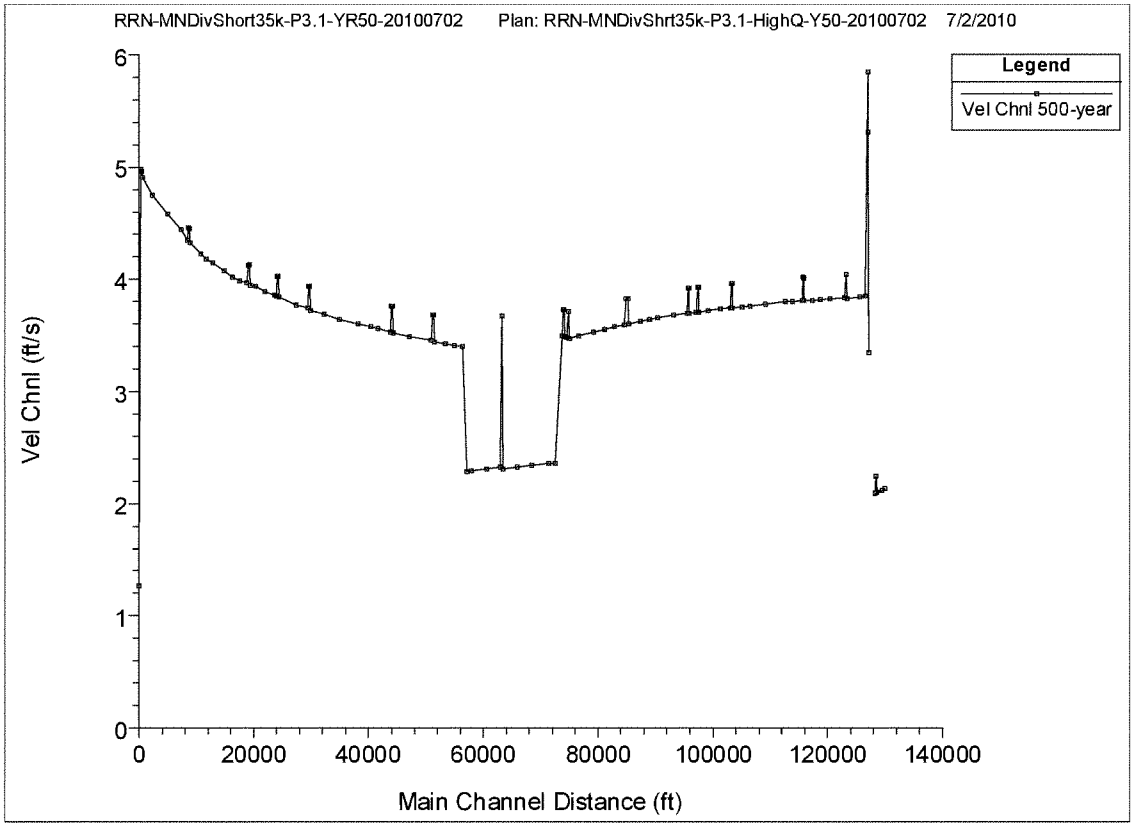
1060



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RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT H – 2D HYDRAULIC MODELING OF DIVERSION STRUCTURES AND FISHWAYS

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

By: Barr Engineering Co.

FINAL – February 28, 2011

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT H – 2D HYDRAULIC MODELING OF DIVERSION STRUCTURES AND FISHWAYS

Notwithstanding the changes in water surface elevations and flows at, or dimensions of, some of the primary project features, the general configuration and concept functioning of these features is maintained in Phase 4. Therefore, the results presented in Exhibit G of the Phase 3 report submitted on August 6, 2010, are repeated in the Phase 4 report because they provide an approximate representation of flow velocity patterns, in particular during the most frequent events.

F-H1.0 INTRODUCTION

Two-dimensional (2D) hydraulic models of the diversion structures and fish passage ways were created to capture the (vertically-averaged) velocity distribution across channels or structures as it relates to fish passage. For each structure, pre-project (existing) conditions were compared to post-construction (with structures in place) conditions. Models of post-construction conditions were based on Phase 3 structure and channel modification plans. This analysis included modeling seven diversion/crossing structures in addition to the two outlets from the diversion channel for a total of nine structures:

- RRN control structure for Minnesota plan
- RRN control structure for North Dakota plan
- Wild Rice control structure
- Sheyenne River crossing of diversion channel
- Maple River crossing of diversion channel
- Lower Rush River connection to diversion channel
- Rush River connection to diversion channel
- Outlet to RRN, Minnesota Plan
- Outlet to RRN, North Dakota Plan

Multiple flow events were analyzed for each structure. For most structures, a low flow event and a high flow event were selected. For some structures, multiple flow events representing various flow regimes were selected. Only Year 0 of the Phase 3 hydrology (e.g., Year 0 scenario, but not Year 25 or Year 50 scenarios) was used in modeling, and only the Locally Preferred Plan (LPP) and Federally Comparable Plan (FCP) plans (Phase 3) were analyzed. Unless otherwise noted, the flow events (e.g., 100-year event) in all the tributaries and the Diversion Channel refer to coincidental events in the RRN.

F-H2.0 METHODOLOGY

F-H2.1 2D FLOW MODEL

The 2D flow model used for this analysis was Adaptive Hydraulics (ADH) revision # 5939. ADH-5939 is a two-dimensional model developed by the coastal and hydraulic laboratory (CHL), engineer research and development center (ERDC) and the USACE. ADH-5939 was selected by the USACE as the numerical model for this analysis because of its ability to adapt numerical meshes to efficiently compute a steady state solution. All models completed as part of this analysis were steady state models that used US customary units.

Preprocessing of model inputs including developing the mesh was completed using AquaVeo's Surface-Water Modeling System Version 10.1 (SMS). ADH-5939 was run independently of SMS from the Microsoft command line. ADH boundary conditions files were created in Microsoft Notepad and manually adjusted for each model. Finally, ADH model results were imported back into SMS for post-processing and viewing of the results.

F-H2.2 MODEL COMPLEXITY AND DOMAIN

The level of detail incorporated into the models was consistent with a feasibility-level analysis. The intent was to capture the effects of the location and the general type/shape of major features of structures. The feasibility-level analysis assumes more detailed 2D flow models (and possibly physical scale models, or 3D flow models) will be completed during future phases of the overall project (e.g., detailed design phase). The models are not intended to be used to analyze other aspects of the project (e.g., ice loading/passage, sediment transport, etc.). This analysis also assumes that the models developed will be used only for this current phase of analysis.

Numerous assumptions were necessary to model the complicated geometry of structures. For example, ADH does not have a routine to model flow under gates. The size and complexity of the models (i.e., mesh size and density, number of nodes, and extent of model domains) was established at a level of detail consummate with Phase 3 analysis. The models were developed and refined only to a level required to achieve convergence, and at which the resulting velocity distribution appeared to be sensible.

Modeling of structures (or parts of structures) was done only where necessary in order to achieve the objectives of this phase, namely at flow junctions and at flow transitions. Model domains were set to include approximately 1,000-2,000 ft of channel upstream/downstream of the structure location.

For low flow conditions, model domains were established to capture the extents of the inundated area. However, for larger flow conditions that result in larger inundated areas, this was not always possible due to the flat terrain. In these locations model domains were clipped along roads, topographic ridges (that may be inundated), or at distance sufficiently away from the area of interest in locations where it was assumed that flow

outside the model domain could largely be considered ineffective. Ineffective area was defined by reviewing existing RRN existing FIS HEC-RAS models.

F-H2.3 MESH/ELEMENT SIZE

ADH required the use of triangular elements. Element size was varied at different areas of mesh in order to capture detail of the topography and structures. At gate openings or through structures, elements were set as small as 10 ft; out in middle of flood plain, where bathymetry is very flat and flow is slow, elements were set at roughly 50.

F-H2.4 AERIAL IMAGES AND LAND USE

Aerial images were used to assign material types in the 2D models. Areas of similar land use were identified based on the 2009 Minnesota aerial photography collected by the U.S. Department of Agriculture, Farm Service Agency, as part of the Farm Service Agency's National Agriculture Imagery Program (NAIP). Based on the aerial photographs, existing conditions land use was divided into three categories, riverbed, grass, and trees, which were then assigned material properties in the 2D models. Land use for post-construction conditions was assigned as either constructed channel or concrete, based the structure's grading plan.

F-H2.5 MANNING'S VALUES

Manning's values were assigned based on material type. Three material types were used for modeling pre-construction conditions (riverbed, trees, and grass). Additional materials (concrete, constructed channel, and water surface) were added for post-constriction conditions. Initial runs used Manning's values that were consistent with available HEC-RAS models. Manning's values used in the HEC-RAS models ranged between 0.04-0.055 for the riverbed, 0.06-0.16 for grass and tree land use types in the overbanks, and 0.030 for constructed channels. Manning's values greater than 0.080 caused ADH to have a difficult time converging on a solution. Additionally, the higher Manning's values resulted in upstream water surface elevations that were higher than the HEC-RAS models. Consequently, Manning's values used in the 2D models were significantly lower than those used in the HEC-RAS models.

Final values of Manning's n were determined after calibrating the models both to upstream water surface from available HEC-RAS models and to available measured velocity distributions. See Section F-H3.3 below for a discussion of model calibration. Final values of Manning's n used in the models are shown in Table F-H1.

Table F-H1 Final Manning's Values

Material	Manning's Value
Riverbed	0.030
Grass	0.040
Trees	0.060
Concrete	0.014
Constructed Channel	0.030
Water Surface*	0.010

*Water surface material was only used for the Maple River Case 4 model where only the upper portion of the diversion channel was modeled.

F-H2.6 EDDY VISCOSITY

The Estimated Eddy Viscosity (EEV) card was used to calculate eddy viscosity in the models. ADH uses the estimated eddy viscosity coefficient to weight each eddy viscosity term calculated by the model. The EEV may range between 0.1-1.0. The EEV card used for this modeling analysis was set to 0.5 for all models.

F-H2.7 TOPOGRAPHY/BATHYMETRY

Meshes for the 2D models were created by combining available topographic and bathymetric data. Models of existing conditions were based on available LiDAR data for areas above the normal waterline and on interpolated HEC-RAS sections for areas below the normal water line. The LiDAR data gathered in 2008 was used to create the ground surface profile in the overbank portions of the models. The LiDAR data was gathered as part of the Red River Basin Mapping Initiative. Bathymetric data from HEC-RAS cross sections were merged with the LiDAR data for areas of the channel where LiDAR data did not survey the channel below the water level. All elevation data used for this analysis is presented in North American Vertical Datum 1988 (NAVD88).

For post-construction conditions sections of river that were impacted by construction were replaced by the appropriate grading plan. Grading plans at the structures dated July 10, 2010 were used to develop topography for all post-construction conditions.

F-H2.8 MESH REFINEMENT

In areas where initial element sizes large, or areas with complex flow patterns ADH has the ability to automatically refine the mesh during the model simulation. The number of times ADH can refine a mesh element, or the number of times an original mesh element may be subdivided by ADH, is specified in the boundary conditions file with the MP ML card. The maximum number of times ADH may subdivide a mesh element is determined by $2^{MP\ ML}$. For example, for a MP ML card value of 3 one element may be split into 8 elements during the model run.

For this analysis the MP ML card ranged between 1 and 3. Increasing the MP ML card past 3 significantly increased model run time. Therefore in areas with complex flow patterns, or in areas of interest, smaller elements were used. This allowed the MP ML card to stay below 3 and allowed the model to converge on a solution.

F-H2.9 ERROR SETTINGS

During model runs ADH determines if refinement of the mesh (i.e. subdividing elements) is required by calculating the error for each element and comparing the calculated error to the MP SRT card in the boundary conditions file.

For this analysis the MP SRT card was set between 10 and 100. Decreasing the MP SRT card below 10 allowed ADH to subdivide more elements which significantly increased model run time. Therefore, in areas with complex flow patterns or areas of interest, smaller elements were used rather than further reducing the MP SRT card. This allowed the MP SRT card to stay within the specified range for this analysis and allowed the model to converge on a solution.

F-H2.10 WETTING/DRYING

During model runs some elements become wet or dry as the water surface rises and falls as the model searches for a solution. ADH accounts for this wetting and drying of elements by gradually turning elements on and off as the model works to converge on solution. The MP DTL card controls when elements are turned on and off within ADH. For this analysis the MP DTL card drying depth was set at 0.3, and the wetting depth was set at 0.5-0.6.

F-H2.11 CONVERGENCE AND RUN TIME

ADH will continue to run until the simulation either reaches the final time step or the spatial residual is less than the tolerance specified in the boundary conditions file. For this analysis the tolerance was specified by the IP NTL card as 0.001, and the model run time was set at 100,000 seconds. If the models reached the final run time of 100,000 seconds before converging, the error, velocity, and depth output files were compared. If the maximum error was less than 100 and the velocity and depth results remained constant between 90,000 -100,000 seconds, then the models were considered to have reached a solution. If these criteria were not achieved, then adjustments to model parameters were completed and the model was rerun.

The hot start files for all the models assumed that the starting water-surface elevation was level. This resulted in ADH having a difficult time starting the model (at time 0 seconds) with the desired steady flow rate. This was because introducing a high flow rate to a level pool resulted in a quick change to water-surface elevations (and elements quickly wetting or drying) which in some cases resulted in the model crashing. Therefore, to ramp the model up one of two methodologies were used:

1. The downstream water-surface elevation boundary condition was set and remained constant throughout the model run. The upstream flow rate boundary

condition was initially set low, and was gradually increased until it reached its final value.

2. The upstream flow boundary condition was set and remained constant throughout the model run. The downstream water-surface elevation boundary condition was initially set high, and was gradually decreased until it reached its final value.

F-H2.12 SMOOTH INFLOW AREAS

ADH had trouble running if there were dry elements along the inflow boundary. ADH also had trouble appropriately distributing flow across the inflow boundary over varying topography. Therefore, for many of the models an inflow ramp was added at the upstream end of the model mesh such that the upstream boundary would remain wet throughout the duration of the model run. This also allowed the inflow to spread out across the mesh prior to reaching the study area, rather than being artificially distributed by the boundary condition. The Manning's values at the ramps were set between 0.030-0.035

F-H2.13 LID ELEVATIONS

The post-construction conditions at the Maple River crossing and Sheyenne River crossing were each simulated with pressure flow below the aqueduct crossing. Pressure flow below the aqueduct was modeled using the Lid function in ADH. The Lid elevation was set at an elevation that resulted in the steady state water depth below the aqueduct crossing that corresponds to the available area below the crossing. In both cases it was necessary to set the Lid elevation lower than the low chord of the aqueduct crossing in order to achieve the proper area of flow under the aqueduct since the current algorithms in ADH solve for the pressure head and do not directly solve for the depth of water.

F-H3.0 MODEL DEVELOPMENT

F-H3.1 STRUCTURES MODELED

This analysis included modeling 7 diversion/crossing structures in addition to the two outlets from the diversion channel for a total of 9 structures:

- RRN control structure for Minnesota plan
- RRN control structure for North Dakota plan
- Wild Rice control structure
- Sheyenne River crossing of diversion channel
- Maple River crossing of diversion channel
- Lower Rush River connection to diversion channel
- Rush River connection to diversion channel
- Outlet to RRN, Minnesota Plan
- Outlet to RRN, North Dakota Plan

Areas of each structure modeled and flows conditions used in this analysis are described in the following sections.

F-H3.1.1 RRN Control Structure for Minnesota Plan
Approximately 17,000 linear feet of RRN channel and floodplain were modeled for existing conditions at the location of the RRN control structure for the MN plan. The model domain for this location includes four major bends in the RRN. A low and a high flow condition were modeled for existing conditions. The low flow event for this location was 9,600 cfs in the RRN. This flow event corresponds to the flow rate in the RRN just prior to flow from the RRN being diverted into the Diversion Channel for post-construction conditions. The high flow event selected for this location was the 20-year event.

Approximately the same section of RRN was modeled for post-construction conditions, and the same flow conditions were modeled. For the 9,600 cfs flow event, the control structure gates will be completely open, there will be open channel flow through control structure, and there will be no water flowing over the diversion channel inlet weir or into the fish pass entrance on the RRN. For the 20-year flow, the control structure gates will be partially closed, restricting flow through the structure and creating a submerged hydraulic jump downstream of the control structure. Flow will also enter the fish pass, and there will be flow over the inlet weir into the diversion channel. Since ADH does not have a gate-flow routine, flow through the control structure gates was not included in this analysis. Consequently, for the 20-year flow event the model domain was divided into area upstream of the control structure and area downstream of the control structure. The fish pass will consist of a series of riffles and pools, which will be designed following the guidance obtained from the Natural Resource Agencies and the main findings of work done by the Corps of Engineers for Lock and Dam 22. Only the connection of the fish pass to the main channel of the RRN was modeled for this analysis.

F-H3.1.2 RRN Control Structure for ND Plan
Approximately 7,000 linear feet of RRN channel and floodplain were modeled for existing conditions at the location of the RRN control structure for the ND plan. The model domain for this location on the RRN includes two major bends in the RRN. A low and a high flow condition were modeled for existing conditions. The low flow event for this location was 6,100 cfs in the RRN. This flow event corresponds to the flow rate in the RRN just prior to flow in the WRR overtopping the east weir into the connecting channel for post-construction conditions. The high flow event selected for this location was the 20-year event.

Approximately the same section of RRN was modeled for post-construction conditions; the same flow conditions were modeled. For the 6,100 cfs flow event, the control structure gates will be completely open and there will be open channel flow through control structure. For this low flow event there will be no water flowing over the connecting channel inlet weir east of the Wild Rice River (however approximately 2,000 ft of backwater in the connecting channel was included in the 2D model domain) or into the fish pass entrance on the RRN. For the 20-year flow the control structure gates will be partially closed, restricting flow through the structure and creating a submerged hydraulic jump downstream of the control structure. Flow will also enter the fish pass and flow over the inlet weir into the connecting channel. Since ADH does not have a gate-

flow routine, flow through the control structure gates was not included in this analysis. Consequently for the 20-year flow event the model domain was divided into area upstream of the control structure and an area downstream of the control structure. The fish pass will consist of a series of riffles and pools, which will be designed following the guidance obtained from the Natural Resource Agencies and the main findings of work done by the Corps of Engineers for Lock and Dam 22. Only the connection of the fish pass to the main channel of the RRN was modeled for this analysis.

F-H3.1.3 Wild Rice Control Structure

Approximately 10,800 linear feet of the Wild Rice River were modeled for existing conditions. Two flow conditions were modeled for the existing conditions. . The flow rates were 3,021 cfs and 8,648 cfs, which are the 5-year and 20-year coincidental flows, respectively (i.e. the flow rate on the Wild Rice River that coincides with the respective flow event in the RRN).

The post-construction conditions analysis included a 2,352 cfs flow event and the same 20-year flow as the existing conditions analysis. The 2,352 cfs flow is the approximate flow before water begins to be diverted into the Connecting Channel between the Wild Rice River and the Red River. This flow event covered the same upstream and downstream domains as existing conditions, and included the short diversion through the control structure. The model also included the entire channel between the weirs connecting the Wild Rice River to the Connecting Channel and the Diversion Channel.

Since ADH does not have a gate-flow routine, and since there were concerns about model stability for flow over weirs, the 20-year flow event for the post-construction conditions was divided into four models sections: 1) approximately 6,750 feet upstream of the control structure and 3,250 feet between the two weirs; 2) 2,700 feet downstream of the control structure; 3) 1,350 feet upstream of the weir between the Connecting Channel and the Wild Rice River; and 4) 1,500 feet downstream of the weir between the Wild Rice River and the Diversion Channel. The analysis used flows of 8,648 cfs in the Wild Rice River; 5,985 cfs over the weir from the Connecting Channel; 12,283 cfs to the Diversion Channel; 24 cfs to the fish pass downstream; and a water surface elevation of 910.67 at the control structure. A flow of 2.350 cfs was modeled to pass through the gates, and a water surface elevation of 904.9 was used as the downstream boundary condition.

F-H3.1.4 Sheyenne River Crossing of Diversion Channel

This study reach consisted of approximately 7,800 feet of the Sheyenne River at the crossing of the Diversion Channel. The analysis for the existing conditions used the 2-year local and 50-year coincidental flows of 1,200 cfs and 4,510 cfs, respectively.

The post-construction conditions considered four flow cases, two through the aqueduct and two in the diversion channel. The two cases in the aqueduct included the 2-year local flow (1,200 cfs) and the 10-year flow (2,565 cfs). The two cases in the Diversion Channel included Case 2 (5-year event) for the Diversion Channel (which is 2,554 cfs in the Diversion Channel and 735 cfs over the weir from the Sheyenne River,) and Case 3

(50-year event) on the Diversion channel which is 19,547 cfs in the Diversion Channel and 2,814 cfs over the weir from the Sheyenne River.

F-H3.1.5 Maple River Crossing of Diversion Channel

Approximately 8,500 linear feet of Maple River channel and floodplain were modeled for existing conditions at the location the Maple River crosses the diversion channel. The model domain for this location on the Maple River includes nine major bends in the river. A low and a high flow condition were modeled for existing conditions. The low flow event for this location was 1,685 cfs. This flow event corresponds to the minimum flow that will be allowed to pass through to the protected area during a flood event. This flow rate is the combined 2-year flows in the Maple River, Lower Rush River, and Rush River. The high flow event selected for this location was the 50-year coincidental event (i.e. the flow rate on the Maple that coincides with the 50-year flow in the diversion channel).

For post-construction conditions two flow events were modeled for the aqueduct crossing the diversion channel. The two flow rates for the Maple River and aqueduct were Case 1 (1685 cfs) and Case 2 (the 5-year coincidental event). For Case 1, all flow is allowed to pass through the Maple River aqueduct into the protected area and no flow is diverted over the spillway into the diversion channel. For Case 2, flow is allowed to pass over the spillway into the diversion channel as well as be conveyed through the aqueduct into the protected area.

In addition, three flow conditions were modeled in the diversion channel at the Maple River. The three flow rates modeled for the diversion channel were Case 2 (2,000 cfs), Case 3 (50-year event), and Case 4 (100-year event). Case 2 corresponds approximately to the 4-year event in the diversion channel. For this flow rate there is open channel flow in the diversion channel below the aqueduct and no inflow from the Maple River spillway. (Note: Case 2 for the diversion channel does not correspond to Case 2 for the aqueduct. This is because the 5-year flow in the diversion channel results in pressurized flow below the Maple River aqueduct, which is analyzed as part of Case 3). Case 3 is the 50-year flow in the diversion channel. For this event there is pressurized flow below the aqueduct and inflow from the Maple River spillway. Finally, for Case 4 there is pressurized flow below the aqueduct and inflow from the Maple River spillway, and overtopping of the diversion channel into the aqueduct. The model for Case 4 includes the upper layers the diversion channel that overtops the Maple River aqueduct.

F-H3.1.6 Lower Rush River Connection to Diversion Channel

Approximately 600ft of Lower Rush River was modeled upstream of the spillway to the diversion channel, and approximately 1,000 ft for the diversion channel upstream and downstream of the spillway. Two different flow conditions were modeled, the mean annual flow in the Lower Rush and the coincidental 20-year flow event. Both flow conditions were modeled for the area upstream of the spillway and in the diversion channel. For the model of the Lower Rush River upstream of the spillway, the inflow and water surface elevation boundary conditions were set based on the Phase 3 hydrology rating curve. These boundary conditions are the same for existing conditions as for post-construction conditions. Since the existing channel geometry is also the same as the post-

construction geometry, the velocity distributions in the Lower Rush channel will be identical for existing versus post-construction conditions. Consequently, only the post-construction conditions were modeled.

The 20-year coincidental flow on the Lower Rush River (210 cfs) is less than the 2-year local flow on the Lower Rush River (302 cfs). Therefore for this analysis it has been assumed that all flow will be directed over the spillway to the diversion channel and no flow will be directed over the drop structure to the diversion channel.

F-H3.1.7 Rush River Connection to Diversion Channel

Approximately 600ft of Rush River was modeled upstream of the spillway to the diversion channel, and approximately 1,000 ft for the diversion channel upstream and downstream of the spillway. Two different flow conditions were modeled, the mean annual flow in the Rush River and the coincidental 20-year flow event. Both flow conditions were modeled for the area upstream of the spillway and in the diversion channel. For the model of the Rush River upstream of the spillway, the inflow and water surface elevation boundary conditions were set based on the Phase 3 hydrology rating curve. These boundary conditions are the same for existing conditions as for post-construction conditions. Since the existing channel geometry is also the same as the post-construction geometry, the velocity distributions in the Rush channel will be identical for existing versus post-construction conditions. Consequently, only the post-construction conditions were modeled.

The 20-year coincidental flow on the Rush River (420 cfs) is approximately equal to the 2-year local flow on the Rush River (415 cfs). Therefore for this analysis it has been assumed that all flow will be directed over the spillway to the diversion channel and no flow will be directed over the drop structure.

F-H3.1.8 Outlet to RRN, Minnesota Plan

Approximately 6,000 feet of the RRN was modeled at the location where the diversion channel outlets to the RRN for the MN plan. Two flow conditions were modeled for the MN outlet for both existing and post-construction conditions, the 2-year and 20-year events. During the 2-year event there is no inflow from the diversion channel to the RRN. However, during the 2-year event the RRN backs up into the downstream portion of the diversion channel. Approximately 1,600 linear feet of diversion channel was included to model the velocity distribution in this backwater area. For the 20-year event there are inflows from both the RRN and diversion channel.

F-H3.1.9 Outlet to RRN, ND Plan

Approximately 6,000 feet of the RRN was modeled at the location where diversion channel outlets to the RRN for the ND plan. Two flow conditions were modeled for the ND outlet for both existing and post-construction conditions, the 2-year and 20-year flow events. There is no flow from the diversion channel to the RRN during the 5-year event. During this low flow event the RRN will back up into the downstream portion of the diversion channel. Approximately 1,600 linear feet of diversion channel was included to

model the velocity distribution in this backwater area. For the 20-year event there are inflows from both the RRN and diversion channel.

F-H3.2 BOUNDARY CONDITIONS

Boundary conditions for the 2D models were assigned based on the available data for each flow condition. Boundary conditions were determined using Phase 3 hydrology, tributary rating curves, Phase 3 HEC-RAS models, and from the hydraulic structure designs.

The existing conditions boundary conditions for the inlet and outlet models along the RRN were determined based on the existing conditions HEC-RAS models. Existing conditions boundary conditions on the tributaries were determined based on Phase 3 hydrology and the rating curve at each tributary.

The post-construction boundary conditions at the inlet and outlet structures were determined based on the Phase 3 – Year 0 HEC-RAS models. The post-construction boundary conditions in the diversion channel at each of the tributaries were determined based on the post-construction Phase 3 Year 0 HEC-RAS models. The post-construction boundary conditions along each tributary were determined based on calculations completed during the design of the hydraulic structures. Boundary conditions for all models are shown in Table F-H2.

Table F-H2 Boundary Conditions

Structure Location	Development Condition	Structure Sub-Part	Flow Condition	Upstream Boundary Condition - 1	Upstream Boundary Condition - 2	Downstream Boundary Condition - 1	Downstream Boundary Condition - 2	Downstream Boundary Condition - 3
MN RRN Control Structure	Existing Conditions	-	9,600 cfs	Q RRN (cfs) 9,600		WSEL RRN (ft) 898.09		
MN RRN Control Structure	Existing Conditions	-	20-year	Q RRN (cfs) 22,000		WSEL RRN (ft) 907.13		
MN RRN Control Structure	Post-Construction	Structure	9,600 cfs	Q RRN (cfs) 9,429	Q Connecting Channel (cfs) 171	WSEL RRN (ft) 898.09		
MN RRN Control Structure	Post-Construction	Structure - Upstream	20-year	Q RRN (cfs) 17,758	Q Connecting Channel (cfs) 4,242	WSEL Weir to Diversion(ft) 906.64	Q Control Structure (cfs) 9,815	Q Fish Pass (cfs) 98
MN RRN Control Structure	Post-Construction	Structure - Downstream	20-year	Q Control Structure (cfs) 9,815	Q Fish Pass(cfs) 98	WSEL RRN (ft) 899.08		
ND RRN Control Structure	Existing Conditions	-	6,100 cfs	Q RRN (cfs) 6,100		WSEL RRN (ft) 901.98		
ND RRN Control Structure	Existing Conditions	-	20-year	Q RRN (cfs) 14,800		WSEL RRN (ft) 910.93		
ND RRN Control Structure	Post-Construction	Structure	6,100 cfs	Q RRN (cfs) 6,100		WSEL RRN (ft) 901.98		
ND RRN Control Structure	Post-Construction	Structure - Upstream	20-year	Q RRN (cfs) 14,800		WSEL Connecting Channel (cfs) 910.91	Q Control Structure (cfs) 8,815	Q Fish Pass(cfs) 88
ND RRN Control Structure	Post-Construction	Structure - Downstream	20-year	Q Control Structure (cfs) 8,815	Q Fish Pass(cfs) 88	WSEL RRN (ft) 905.7		
Wild Rice River	Existing Conditions	-	5-year	Q WRR (cfs) 3,021		WSEL WRR (ft) 906.76		
Wild Rice River	Existing Conditions	-	20-year	Q WRR (cfs) 8,648		WSEL WRR (ft) 914.59		
Wild Rice River	Post-Construction	Structure	2,352 cfs	Q WRR (cfs) 2,352		WSEL WRR (ft) 904.9		
Wild Rice River	Post-Construction	Structure - Upstream	20-year	Q WRR (cfs) 8,648	Q Connecting Channel (cfs) 5,985	WSEL Control Structure (ft) 910.67	Q Diversion Channel (cfs) 12,283	Q Fish Pass(cfs) 24
Wild Rice River	Post-Construction	Structure - Connecting	20-year	Q Connecting Channel (cfs) 5,985		WSEL Connecting Channel(ft) 910.67		
Wild Rice River	Post-Construction	Structure - Diversion	20-year	Q Diversion Channel (cfs) 12,283		WSEL Diversion Channel(ft) 904.6		
Wild Rice River	Post-Construction	Structure - Downstream	20-year	Q Control Structure (cfs) 2,350	Q Fish Pass(cfs) 24	WSEL RRN (ft) 904.9		
Sheyenne River	Existing Conditions	-	2-year (Local)	Q Sheyenne (cfs) 1,200		WSEL Sheyenne (ft) 912.71		
Sheyenne River	Existing Conditions	-	50-year	Q Sheyenne (cfs) 4,510		WSEL Sheyenne (ft) 919.37		
Sheyenne River	Post-Construction	Aqueduct	Case1 (2-year - Local)	Q Sheyenne (cfs) 1,200		WSEL Sheyenne (ft) 912.71		
Sheyenne River	Post-Construction	Aqueduct	Case2 (10-year)	Q Sheyenne (cfs) 2,565		WSEL Sheyenne (ft) 915.12	Q Spillway (cfs) 1,122	
Sheyenne River	Post-Construction	Diversion Channel	Case2 (5-year)	Q Diversion Channel (cfs) 2,554	Q Spillway (cfs) 735	WSEL Diversion Channel (ft) 892.83		
Sheyenne River	Post-Construction	Diversion Channel	Case3 (50-year)	Q Diversion Channel (cfs) 19,547	Q Spillway (cfs) 2,814	WSEL Diversion Channel (ft) 904.82		
Maple River	Existing Conditions	-	2-year (Min. pass through Q)	Q Maple (cfs) 1,685		WSEL Maple (ft) 893.32		
Maple River	Existing Conditions	-	50-year	Q Maple (cfs)		WSEL Maple (ft)		

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Table F-H2 Boundary Conditions

Structure Location	Development Condition	Structure Sub-Part	Flow Condition	Upstream Boundary Condition - 1	Upstream Boundary Condition - 2	Downstream Boundary Condition - 1	Downstream Boundary Condition - 2	Downstream Boundary Condition - 3
				4,400		898.83		
Maple River	Post-Construction	Aquaduct	Case1 (2-year - Min pass through Q)	Q Maple (cfs) 1,685		WSEL Maple (ft) 893.32		
Maple River	Post-Construction	Aquaduct	Case2 (5-year)	Q Maple (cfs) 2,000		WSEL Maple (ft) 892.45	Q Spillway (cfs) 301	
Maple River	Post-Construction	Diversion Channel	Case2 (2,000 cfs)	Q Diversion Channel (cfs) 2,000		WSEL Diversion Channel (ft) 877.2		
Maple River	Post-Construction	Diversion Channel	Case3 (50-year)	Q Diversion Channel (cfs) 23,118	Q Spillway (cfs) 2,408	WSEL Diversion Channel (ft) 891.68		
Maple River	Post-Construction	Diversion Overtopping	Case4 (100-year)	Q Upper Layer Div. Channel (cfs) 3,513	Q Maple (cfs) 2,043	WSEL Diversion Channel (ft) 895.56	Q Maple Gates (cfs) 2,043	
Lower Rush River	Post-Construction	Lower Rush Fish Pass	MAF	Q Lower Rush (cfs) 10		WSEL Fish Pass (ft) 886.7		
Lower Rush River	Post-Construction	Lower Rush Fish Pass	20-year	Q Lower Rush (cfs) 210		WSEL Fish Pass (ft) 891.8		
Lower Rush River	Post-Construction	Diversion Channel	MAF	Q Fish Pass (cfs) 10		WSEL Diversion Channel (ft) 863.46		
Lower Rush River	Post-Construction	Diversion Channel	20-year	Q Diversion Channel (cfs) 16,249	Q Fish Pass (cfs) 210	WSEL Diversion Channel (ft) 886.3		
Rush River	Post-Construction	Rush Fish Pass	MAF	Q Rush (cfs) 17		WSEL Fish Pass (ft) 880.1		
Rush River	Post-Construction	Rush Fish Pass	20-year	Q Rush in (cfs) 420		WSEL Fish Pass (ft) 891.09		
Rush River	Post-Construction	Diversion Channel	MAF	Q Fish Pass (cfs) 17		WSEL Diversion Channel (ft) 861.26		
Rush River	Post-Construction	Diversion Channel	20-year	Q Diversion Channel (cfs) 16,879	Q Fish Pass (cfs) 420	WSEL Diversion Channel (ft) 885.13		
MN RRN Outlet	Existing Conditions	-	2-year	Q RRN (cfs) 8,328		WSEL RRN (ft) 874.29		
MN RRN Outlet	Existing Conditions	-	20-year	Q RRN (cfs) 28,491		WSEL RRN (ft) 887.17		
MN RRN Outlet	Post-Construction	Outlet	2-year	Q RRN (cfs) 8,328		WSEL RRN (ft) 874.29		
MN RRN Outlet	Post-Construction	Outlet	20-year	Q RRN (cfs) 16,375	Q Diversion Channel (cfs) 12,116	WSEL RRN (ft) 887.17		
ND RRN Outlet	Existing Conditions	-	2-year	Q RRN (cfs) 8328		WSEL RRN (ft) 870.05		
ND RRN Outlet	Existing Conditions	-	20-year	Q RRN (cfs) 28,491		WSEL RRN (ft) 883.21		
ND RRN Outlet	Post-Construction	Outlet	2-year	Q RRN (cfs) 8,688		WSEL RRN (ft) 870.26		
ND RRN Outlet	Post-Construction	Outlet	20-year	Q RRN (cfs) 13,362	Q Diversion Channel (cfs) 16,879	WSEL RRN (ft) 883.37		

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F-H3.3 MODEL CALIBRATION

Models were calibrated by visually comparing the resulting velocity distributions from models of existing conditions to available measured velocity distributions as well as existing HEC-RAS results. Typically, in calibrating 2D models, the Manning's n values and the EEV card values can be adjusted to best match velocity distributions and water surface elevations. In this analysis, it was decided to hold the EEV card constant at a value of 0.5. Only the Manning's n values for the various materials were adjusted during the calibration process.

F-H3.3.1 Compare upstream water surface elevations to HEC-RAS

Due to the relatively short stream-wise length of the models and low slope of the energy grade line (EGL), generally there was little variation in upstream water surface elevation as a function of user-supplied model coefficients. For 2D models of the RRN and Diversion Channel, the upstream water-surface elevation in the 2D model was compared to the water-surface elevation calculated in the Phase 3-Year 0 or existing conditions HEC-RAS models. The length of the model reach and the land use materials in the model affected the match of the 2D model elevations to the HEC-RAS elevations. For example the water surface elevation on the RRN for the 20-year event at the Minnesota inlet at the upstream end of the 2D model was only 0.1-ft higher than the HEC-RAS model developed for this phase of the study. For reaches where trees lined the channel, the 2D model results deviated further from the HEC-RAS model results. One example is the 20-year event on the RRN at the location of the ND outlet. In this area the water surface elevation at the upstream end of the 2D model is 0.3-ft higher than the HEC-RAS model.

F-H3.3.2 Compare velocity distribution to measured data

Data available from USGS stations on the RRN and the modeled tributaries were used to compare the velocity distributions at locations along the portion of modeled channels to measured velocity distributions for similar flow rates for existing conditions. This was done to verify that the model was calculating velocity distributions within the channel approximated past measured velocity distributions. It was not expected that the modeled velocity distributions would match the measured distributions exactly, due to differences in geometry (e.g. the stream may have been straighter where the velocity was measured, and have more bends in the modeled portion), roughness of the stream bed, or bathymetry. However, this did provide validation that the existing conditions models approximated the measured data. For example, the velocities on the Maple for the existing conditions 2-year event (1,685 cfs) were 2.0-2.2 ft/s near the thalweg and 0.5 ft/s near the banks. In comparison the measured velocities at USGS station 05060100 on April 1, 2009 at 2,510 cfs show channel velocities ranging from 2.0-3.0 ft/s. In a few locations modeled velocities did not match the measured velocities as well. For example, for the 20-year event (22,000 cfs) at the MN inlet location for existing conditions the modeled velocity was 1.7-2.0 ft/s. Measured velocity data from USGS gage 05051522 at Hickson, from March 25, 2009, (21,500 cfs) ranged from approximately 2.0-2.5 ft/sec.

F-H4.0 RESULTS

The following section discusses the results of the two-dimensional flow models. The section is organized by structure. The results are presented in two formats: screen captures from computer simulations, and graphs. The screen captures from computer simulations show the resulting velocity fields for the various structures and flow conditions. The results are superimposed over aerial images. Color is used indicate the magnitude of the velocity, blue corresponds to slow velocities and red corresponds to high velocities. Vectors are plotted on the screen captures to show the direction of flow throughout the velocity field. For the most part the areas covered by color and vectors are the wetted areas under the given flow condition; the areas not covered by color and vectors are dry. Exceptions include some areas of standing water with velocities very close to zero that, for some reason, SMS does not color. Graphs are shown to highlight and compare flow velocities and/or depths for one flow condition to another (e.g., existing conditions versus post-construction conditions), or for one location to another on a given structure (e.g., through-structure location to downstream location). Graphs show either the velocity or depth plotted as a function of distance across a river/channel section. The location of the river channel sections from which data in the graphs is taken is shown on the screen captures as solid black lines.

F-H4.1 RRN CONTROL STRUCTURE FOR MINNESOTA PLAN

F-H4.1.1 Existing Conditions

Results for the Minnesota inlet control structure are shown in Figures F-H1 to F-H14. Two flow events were analyzed for the Minnesota inlet control structure, 9,600 cfs and the 20-year event. During the 9,600 cfs event flows are primarily contained within the river banks and follow the general river alignment. Along the main channel velocities remain consistent throughout the section of river analyzed varying from 2.0-2.3 ft/s along the thalweg and 0.0-0.5 at the banks. During the 20-year event flow overtop the river banks and flows in the floodplain before reconnecting back to the Red River. Because there is more flow in the overbanks during the 20-year event velocities along the thalweg vary throughout the section of river analyzed and range between 0.7-2.5 ft/s and between 0.3-1.5 ft/s along the banks. In floodplain velocities range between 0-0.5 ft/s, and at large river bends, flow begins to flow overland short circuiting the bend rather than following the channel alignment.

F-H4.1.2 Post-Construction Conditions

Velocity distributions post-construction were analyzed for the same two flow conditions, a 9,600 cfs event and the 20-year event. During the 9,600 cfs event all flow is allowed to pass through the structure on the Red River into the protected area; no flow is directed over the weir in the diversion channel. Therefore water in the connecting channel flows west towards the Red River during low flow events. Velocities in the connecting channel and immediately upstream of the weir to the diversion channel are approximately 0.0-0.3 ft/s. Velocities along the Red River remain similar to existing conditions, 2.0-2.3 ft/s along the thalweg of the channel, and approximately 2.0 ft/s through the control structure. Immediately upstream and downstream from the Red River structure in the constructed channels velocities are slightly lower, 1.3-1.5 ft/s, than existing conditions because the

size of the constructed channel is slightly larger than the adjacent river section. During the 9,600 cfs event there are storage areas of inundation in the diversion channel and in the portion of Red River channel that was disconnected to construct the control structure that have very low velocities 0.0-0.2 ft/s.

During the 20-year event flow is directed into the diversion channel. Velocities upstream and downstream of the control structure on the Red River along the channel thalweg are approximately 1.0-1.5 ft/s. The velocities in the Red River are lower than existing conditions primarily because a significant amount of the flow is conveyed in the connecting channel which has velocities between 1.0-1.5 ft/s. During the 20-year event some flow is directed into the diversion channel. The velocity at the entrance to the diversion channel is 2.0-2.5 ft/s. Similar to the 9,600 cfs event there is an area of inundation upstream and downstream of the control channel in the portion of Red River that was disconnected during construction that act as storage areas and have velocities of 0.0-0.2 ft/s. Finally, the velocity in the downstream portion of the fish pass, where it connects to the constructed channel is approximately 0.7 ft/s. The area of higher velocity in the fish pass is a result of model boundary conditions and does not reflect the velocity in the fish pass. Because the entrance to the fish pass is submerged during the 20-year event, varying the geometry of the fish pass had no impact on the modeled velocities.

F-H4.2 RRN CONTROL STRUCTURE FOR NORTH DAKOTA PLAN

F-H4.2.1 Existing Conditions

Results for the North Dakota inlet control structure are shown in Figures F-H15 to F-H26. Two flow events were analyzed for the North Dakota Inlet structure, 6,100 cfs and the 20-year event. During the 6,100 cfs event flows just begin to overtop the river banks, which create inundated areas in the overbanks that have very low velocities. Along the main channel velocities remain consistent throughout the section of river analyzed varying from 1.5-2.0 ft/s along the thalweg and 0.5-1.0 at the banks. During the 20-year event flow overtop the river banks and flows in the floodplain before reconnecting back to the Red River and the downstream end of the modeled area. Velocities along the thalweg range between 1.7-2.2 ft/s and between 0.6-1.1 ft/s along the banks. In floodplain velocities range between 0-0.5 ft/s, and at large river bends, flow begins to flow overland short circuiting the bend rather than following the channel.

F-H4.2.2 Post-Construction Conditions

Velocity distributions post-construction were analyzed for the same two flow conditions, a 6,100 cfs event and the 20-year event. During the 6,100 cfs event all flow is allowed to pass through the structure on the Red River into the protected area; no flow is directed over the weir in the connecting channel located east of the Wild Rice River. Velocities along the Red River remain similar to existing conditions, 1.5-2.0 ft/s along the thalweg of the channel, and approximately 1.5 ft/s through the control structure. Immediately upstream and downstream from the Red River structure in the constructed channels velocities are slightly lower, 1.0-1.5 ft/s, than existing conditions because the size of the constructed channel is slightly larger than the adjacent river section. During the 6,100 cfs event there are storage areas of inundation in the connecting channel and in the portion of

Red River channel that was disconnected to construct the control structure that have very low velocities 0.0-0.2 ft/s.

During the 20-year event some flow is directed into the connecting channel. Velocities upstream of the control structure along the channel thalweg are approximately 2.0-2.5 ft/s and decrease to 1.5-2.0 ft/s in the channel downstream of the structure in the protected area. The velocity in the entrance to the connecting channel is 1.0-1.5 ft/s. Similar to the 6,100 cfs event there is an area of inundation downstream of the control channel in the portion of Red River that was disconnected during construction. Finally, the velocity in the downstream portion of the fish pass, where it connects to the constructed channel is approximately 0.1-0.3 ft/s. Because the entrance to the fish pass is submerged during the 20-year event varying the geometry of the fish pass had no impact on the modeled velocities.

F-H4.3 WILD RICE CONTROL STRUCTURE

F-H4.3.1 Existing Conditions

Results for the Wild Rice River structure are shown in Figures F-H27 to F-H42. Two existing conditions flow events, the 5-year and 20-year coincidental events, were simulated for the Wild Rice River. During the 5-year event, flows are starting to spill over the river banks. There is a location just downstream from the location of the proposed diversion channel crossing where flow leaves the main channel and then returns the river channel at the next river bend. Velocities along the channel thalweg are approximately 2.0 - 2.2 ft/s and 0.5 ft/s near the banks. During the 20-year event the river banks are overtopped and there are large inundated areas where velocities are very small. During the 20-year event velocities along the channel thalweg are approximately 1.5-2.0 ft/s and less than 0.5 ft/s near the banks and in the floodplain. Overall, the velocity distribution across the channel remains relatively constant for both the 5-year and 20-year existing conditions models throughout the area of river analyzed.

F-H4.3.2 Post-Construction Conditions

Velocity distributions for the post-construction aqueduct crossing were analyzed for two flow conditions, the 2,352 cfs and 20-year events. During the 2,352 cfs event, all flow is allowed to pass between the two weirs in the diversion channel and through the control structure on the Wild Rice River into the protected area. For larger events the weir in the diversion channel east of the Wild Rice River is over topped. Velocities upstream and downstream from the diversion channel are approximately 2.0 ft/s along the thalweg. In the constructed connecting channel velocities are slightly lower, 1.2-1.5 ft/s due to the increase in channel cross sectional area. Because there is no flow over the east or west weirs the section of diversion channel between the two weirs becomes a large backwater area, similar to the two sections of Wild Rice River that were abandoned to construct the crossing, where the velocities are less than 0.1 ft/s. Velocity on the Wild Rice River increase to 2.5-2.7 ft/s as flow is restricted and passes through the control structure.

During the 20-year event, water from the RRN connecting channel flows over the east weir and mixes with inflows from the Wild Rice River. Some flow is allowed to pass through the Wild Rice River control structure into the protected area, and the rest of the

flow is directed over the west weir into the diversion channel. The velocities in the connecting channel vary between 1.5 ft/s at the center of the channel to less than 0.5 ft/s near the sides. During the 20-year event, all of the flow is contained within the connecting channel, and given the uniform channel geometry the velocity distribution remains constant throughout the section of channel velocity analyzed.

Velocities upstream of the diversion channel area vary between 2.5-4.0 ft/s along the thalweg, and 0.0-0.5 ft/s near the banks and in the floodplain. Velocities are higher compared to existing conditions because the downstream water surface elevation has been lowered for the same flow rate. Flow from the Wild Rice River mixes with inflows from the connecting channel. In the diversion channel velocities east of the Wild Rice River are approximately 1.5 ft/s and increase to 3.5-4.0 ft/s in the western portion of the diversion channel. Velocities near the crest of the west weir reach 6.0-7.0 ft/s. Velocities through the control structure on the Wild Rice River are 2.0-2.5 ft/s.

Velocities downstream of the Wild Rice River control structure in the constructed channel vary between 1.5 ft/s near the middle of the channel to 0.5 ft/s closer to the banks. Flow in the constructed channel mixes with the small inflow from the fish pass downstream of the control structure. As a result of the relatively small amount of flow in the fish pass the velocity vector field in the connecting channel is not greatly disturbed by flows from the fish pass. Velocities near the fish pass entrance are approximately 0.3 ft/s. After the constructed channel rejoins the undisturbed section of the Wild Rice River velocities increase to 1.5-2.0 ft/s which is similar to existing conditions. Finally, there is a large area of backwater area in the section of Wild Rice River that was abandoned to construct the control structure. In these area velocities are less than 0.1 ft/s.

F-H4.4 SHEYENNE RIVER CROSSING OF DIVERSION CHANNEL

F-H4.4.1 Existing Conditions

Results for the Sheyenne River structure are shown in Figures F-H43 to F-H62. Two existing conditions flow events, the 2-year and 50-year events, were analyzed for the Sheyenne River. During the 2-year event (which was defined as the 2-year local flow event for the Sheyenne River) flow stays primarily within the river banks. Velocities along the channel thalweg are approximately 1.5-2.0 ft/s and less than 0.5 ft/s near the banks. During the 50-year event, the river banks are overtopped and there are large inundated areas where velocities are very small. For this analysis only the effective flow area defined by the FIS HEC-RAS models adjacent to the River was analyzed. During the 50-year event velocities along the channel thalweg are approximately 2.5-3.0 ft/s and 0.5 ft/s near the banks. Overall the velocity distribution across the channel remains relatively constant for both existing conditions events throughout the area of river analyzed.

F-H4.4.2 Post-Construction Conditions – Aqueduct Crossing

Velocity distributions for the post-construction aqueduct crossing was analyzed for two flow conditions, Case 1 (2-year local event) and Case 2 (10-year coincidental event). During the 2-year event all flow is allowed to pass over the aqueduct into the protected area. The area upstream of the spillway acts as a backwater storage area and velocities are less than 0.1 ft/s. Velocities upstream and downstream from the aqueduct are similar to

the existing conditions event; approximately 1.5-1.8 ft/s along the thalweg. Near the aqueduct velocities increase slightly to 1.8 ft/s as flow is restricted and passes through the constructed aqueduct channel.

During the 10-year event, some flow is directed over the spillway into the diversion channel. Velocities upstream of the spillway along the channel thalweg are approximately 2.5-3.0 ft/s and decrease to 1.5-2.0 ft/s downstream of the spillway as some flow is redirected to the spillway. Velocities over the aqueduct are approximately 2.0 ft/s and decrease to 1.5-2.0 ft/s in the channel downstream of the aqueduct in the protected area. The velocity at the entrance to the spillway is 1.5 ft/s and increases to approximately 5.0 ft/s at the near the crest of the spillway as the water depth decreases.

F-H4.4.3 Post-Construction Conditions – Diversion Channel

Two flow conditions in the diversion channel were analyzed for post-construction conditions, the 5- and 50-year events. During the 5-year event the water surface elevation in the diversion channel remains below the bottom of the aqueduct crossing. During this event the velocities along the center of the diversion channel are 1.5-2.0 ft/s, and remain constant throughout the section of diversion channel modeled. Model results indicate that flow from the Sheyenne spillway enters the diversion channel at velocities near 5.0-7.0 ft/s. This high velocity is due to the assumed slope of the spillway as it connects to the Diversion Channel.

During the 50-year event the water surface elevation in the diversion channel is higher than the bottom of the aqueduct. This results in pressurized flow below the Sheyenne River aqueduct crossing. Velocities in the diversion channel upstream of the aqueduct are approximately 2.5-3.3 ft/s and increase to 6.0-7.0 ft/s below the aqueduct crossing. Downstream of the aqueduct crossing the velocities return to 2.5-3.0 ft/s as the channel expands. The spillway velocity during the 50-year event is 2.5 ft/s which is similar to the velocities in the diversion channel.

F-H4.5 MAPLE RIVER CROSSING OF DIVERSION CHANNEL

F-H4.5.1 Existing Conditions

Results for the Maple River structure are shown in Figures F-H63 to F-H83. Two existing conditions flow events, the 2-year and 50-year events, were analyzed for the Maple River. During the 2-year event (which is defined as the minimum pass through flow of the 2-year events on the Maple, Rush, and Lower Rush Rivers) flow stays primarily within the river banks. Velocities along the channel thalweg are approximately 2.0 - 2.2 ft/s and 0.5 ft/s near the banks. During the 50-year event the river banks are overtopped and there are large inundated areas where velocities are very small. During the 50-year event velocities along the channel thalweg are approximately 2.0-3.0 ft/s and 0.5 ft/s near the banks. Overall the velocity distribution across the channel remains relatively constant for both the 2-year and 50-year existing conditions models throughout the area of river analyzed.

F-H4.5.2 Post-Construction Conditions – Aqueduct Crossing
Velocity distributions for the post-construction aqueduct crossing were analyzed for two flow conditions, the 2-year (Case 1) and 5-year (Case 2) events. During the 2-year event all flow is allowed to pass over the aqueduct into the protected area. Velocities upstream and downstream from the aqueduct are similar to the existing conditions event; approximately 2.0 ft/s along the thalweg. Near the aqueduct velocities increase slightly to 2.5 ft/s as flow is restricted and passes through the constructed aqueduct.

During the 5-year event some flow is directed over the spillway into the diversion channel. Velocities upstream of the aqueduct along the channel thalweg are approximately 2.0-2.5 ft/s and increase to 2.8 ft/s over the aqueduct before returning to 2.0-2.5 ft/s in the channel downstream of the aqueduct in the protected area. The velocity at the entrance to the spillway is 0.5 ft/s and increases to 2.5 ft/s at the near the crest of the spillway as the water depth decreases. For both the 2- and 5-year events there are areas of inundation to the east and west of the diversion channel in the portion of Maple River channel that was disconnected to construct the aqueduct crossing.

F-H4.5.3 Post-Construction Conditions – Diversion Channel
Three flow conditions in the diversion channel were analyzed for post-construction conditions, a 2000 cfs event (Case 2), 50-year (Case 3) and 100-year (Case 4) events. During the 2000 cfs event the water surface elevation in the diversion channel is below the bottom of the aqueduct crossing. During this event the velocities along the center of the diversion channel are 1.5-2.0 ft/s, and remain constant below the aqueduct.

During the 50-year event the water surface elevation in the diversion channel is higher than the bottom of the aqueduct. This results in pressurized flow below the Maple River crossing. Velocities in the diversion channel upstream of the aqueduct are approximately 2.5-3.0 ft/s and increase to 7.0-8.0 ft/s below the aqueduct crossing. Downstream of the aqueduct crossing the velocities return to 2.5-3.0 ft/s as the channel expands. The spillway velocity during the 50-year event is 1.5-2.0 ft/s which is similar to the velocities in the diversion channel.

Finally, during the 100-year event water in the diversion channel overtops the aqueduct and flow from the diversion channel mixes with flow in the aqueduct. For this case only the upper layers of the diversion channel were included in this analysis. During this event water flows into the diversion channel aqueduct at velocities of 7.0-10.0 ft/s and mixes with flow in the aqueduct which is traveling at approximately 1.5-2.0 ft/s. The flows mix and some flow is pass through gates into the protected area with velocities between 2.5-3.0 ft/s and the remaining flow is conveyed downstream in the diversion channel at velocities of 6.0-7.0 ft/s. Downstream of the aqueduct velocities return to 2.5-3.0 ft/s in the full diversion channel section.

F-H4.6 LOWER RUSH RIVER CONNECTION TO DIVERSION CHANNEL

F-H4.6.1 Post-Construction Conditions – Spillway

Results for the Lower Rush River are shown in Figures F-H84 to F-H87. Two flow conditions were analyzed for the Lower Rush River upstream of the spillway; the mean average flow (MAF) and the 20-year coincidental event. During the MAF event all flow is contained within the banks and is directed over the spillway. Velocities in the channel are approximately 0.1 ft/s and approach 0.3 ft/s near the spillway. During the 20-year event flow just beings to overtop the river banks (i.e. in the Lower Rush River the 20-year coincidental flow rate 210 cfs is less than the 2-year local event 302 cfs). During the 20-year event velocities along the channel thalweg are approximately 0.4 ft/s and approach 0.8 ft/s near the spillway. Overall the velocity distribution across the channel remains relatively constant for both the 2-year and 20-year existing conditions models throughout the section of river analyzed.

F-H4.6.2 Post-Construction Conditions – Diversion Channel

Two flow conditions were analyzed for the diversion channel adjacent to the Lower Rush River spillway; the mean average flow (MAF) in the Lower Rush and the 20-year event in the diversion channel. During the MAF event there is no flow in the diversion channel upstream of the Lower Rush connecting structure. Velocities in the channel downstream of the spillway are approximately 0.3 ft/s and 0.2 ft/s upstream of the spillway. During the 20-year event there is a relatively large amount of flow in the diversion channel compared to the contributing flow from the Lower Rush River spillway, and the addition of the flows from the spillway have no visible effect on the velocities in the diversion channel. During the 20-year event velocities in the diversion channel are approximately 2.5 ft/s and approach 0.2 ft/s in the spillway.

F-H4.7 RUSH RIVER CONNECTION TO DIVERSION CHANNEL

F-H4.7.1 Post-Construction Conditions – Spillway

Results for the Rush River are shown in Figures F-H88 to F-H91 Two flow conditions were analyzed for the Rush River upstream of the spillway; the mean average flow (MAF) and the 20-year coincidental event. During the MAF event all flow is contained within the banks and is directed over the spillway. Velocities in the channel are approximately 0.3 ft/s and approach 1.0 ft/s near the spillway. During the 20-year event flow just beings to overtop the river banks (i.e. in the Rush River the 20-year coincidental flow rate 410 cfs is approximately equal to the 2-year local event 415 cfs). During the 20-year event velocities along the channel thalweg are approximately 0.2 ft/s and approach 0.6 ft/s near the spillway. Overall the velocity distribution across the channel remains relatively constant for both the 2-year and 20-year existing conditions models throughout the section of river analyzed.

F-H4.7.2 Post-Construction Conditions – Diversion Channel

Two flow conditions were analyzed for the diversion channel adjacent to the Rush River spillway; the mean average flow (MAF) in the Lower Rush and the 20-year event in the diversion channel. During the MAF event there is no flow in the diversion channel and all of the flow is contained in the bottom of the diversion channel. Velocities in the channel downstream of the spillway are approximately 0.9 ft/s and less than 0.1ft/s in the

diversion channel upstream of the spillway. During the 20-year event there is a relatively large amount of flow in the diversion channel compared to the contributing flow from the Rush River spillway, and the addition of the flows from the spillway have no visible effect on the velocities in the diversion channel. During the 20-year event velocities in the diversion channel are approximately 2.7 ft/s and approach 0.3 ft/s in the spillway.

F-H4.8 OUTLET TO RRN, MINNESOTA PLAN

F-H4.8.1 Existing Conditions

Results for the Minnesota alignment outlet to the RRN are shown in Figures F-H92 to F-H99. Two flow conditions were analyzed for the outlet of the MN Plan, the 2- and 20-year events. During the 2-year event the flow is primarily contained within the river banks and follows the river alignment. Velocities along the channel thalweg are approximately 2.0 - 2.5 ft/s and 1.5-2.0 ft/s near the banks. During the 20-year event the river banks are overtopped, and water flows through the overbanks rather than following the river alignment. During the 20-year event velocities along the channel thalweg are approximately 2.5-3.0 ft/s, 1.0-1.5 ft/s near the banks, and 0-0.5 ft/s in the floodplain. Overall the velocity distribution across the channel remains relatively constant for both the 2-year and 20-year existing conditions models throughout the section of river analyzed.

F-H4.8.2 Post-Construction Conditions

During the 2-year event there is no flow in the diversion channel. During this event water from the RRN backs up into the lower portion of the diversion channel. Velocities in the RRN upstream and downstream from the diversion channel outlet remain similar to existing conditions velocities along the thalweg, 2.0-2.5 ft/s, and near the banks1.5-2.0 ft/s. During this flow condition the downstream portion of the diversion channel is a backwater area and velocities are near zero.

During the 20-year event there is flow in the diversion channel. Velocities in the RRN upstream of the diversion channel outlet are between 1.0-1.5 ft/s along the thalweg and 0.5-1.0 near the banks, which is slightly lower than existing conditions due to the lower flow rate in the RRN. Velocities in the RRN increase downstream of the outlet of the diversion channel to 2.5-3.0 ft/s along the thalweg and 1.5-2.0 near the banks, which is similar to the existing conditions velocity distribution. Velocities at the center of the diversion channel are between 2.0-2.5 ft/s.

F-H4.9 OUTLET TO RRN, NORTH DAKOTA PLAN

F-H4.9.1 Existing Conditions

Results for the North Dakota alignment outlet to the RRN are shown in Figures F-H100 to F-H107. Two flow conditions were analyzed for the outlet of the ND Plan, the 2- and 20-year events. During the 2-year event the flow is primarily contained within the river banks and follows the river alignment. At locations of river meanders flow overtops the banks and flows back into the RRN. Velocities along the channel thalweg are approximately 2.0 - 2.5 ft/s and 1.5-2.0 ft/s near the banks. Overall the velocity distribution across the channel remains relatively constant for both the 2-year existing conditions model throughout the section of river analyzed. During the 20-year event the

river banks are overtopped, and water flows through the overbanks rather than following the river alignment. During the 20-year event velocities near the channel thalweg vary between 0.5-1.5 ft/s, and are generally less than 0.5 ft/s in the floodplain. In general, during the 20-year event velocities are remain relatively uniform across the throughout the floodplain. This is due to the large amount of flow that occurs in the overbanks, which does not follow the alignment of the channel due to the large meanders in this section of river.

F-H4.9.2 Post-Construction Conditions

During the 2-year event there is no flow in the diversion channel. During this event water from the RRN backs up into the lower portion of the diversion channel. Velocities in the RRN upstream and downstream from the diversion channel outlet remain similar to existing conditions velocities along the thalweg, 2.0-2.5 ft/s, and near the banks1.5-2.0 ft/s. During this flow condition the downstream portion of the diversion channel is a backwater area and velocities are near zero.

During the 20-year event there is flow in the diversion channel. Velocities in the RRN upstream and downstream of the diversion channel outlet are between 0.5-1.0 ft/s, which is similar to existing conditions due to the large amount of flow through the floodplain. Velocities at the center of the diversion channel are between 1.5-2.0 ft/s.

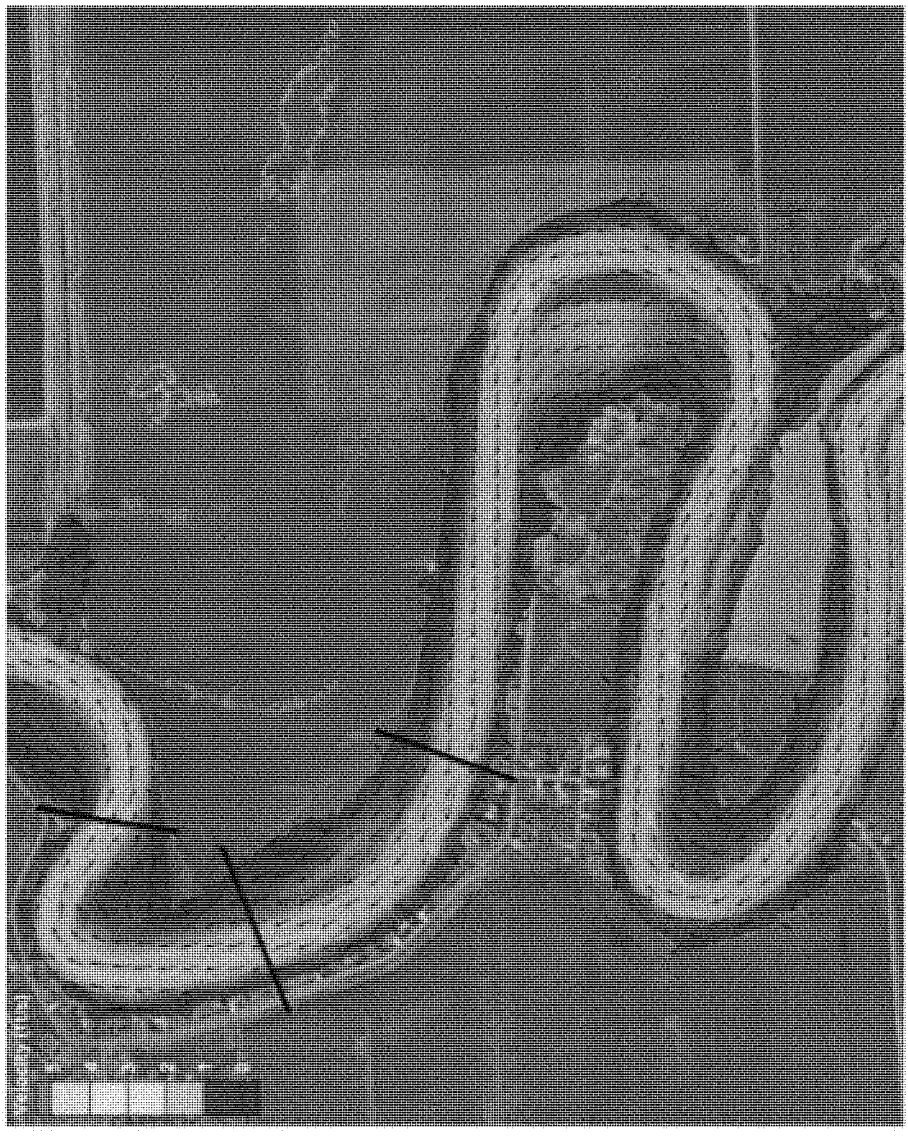

Figure F-H1 Velocity Field for Minnesota Control Structure Existing Conditions 9,600 cfs Event Flow

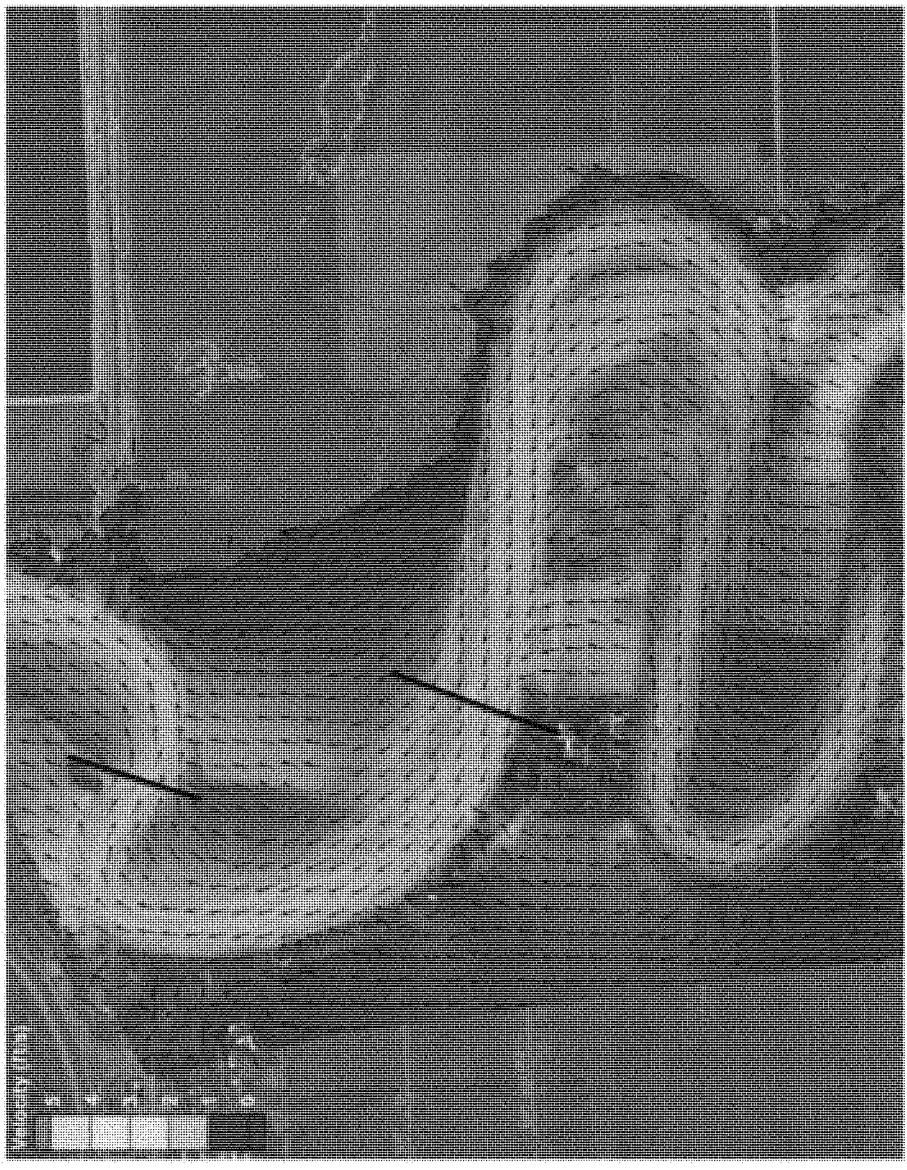

Figure F-H2 Velocity Field for Minnesota Control Structure Existing Conditions 20-Year Event Flow

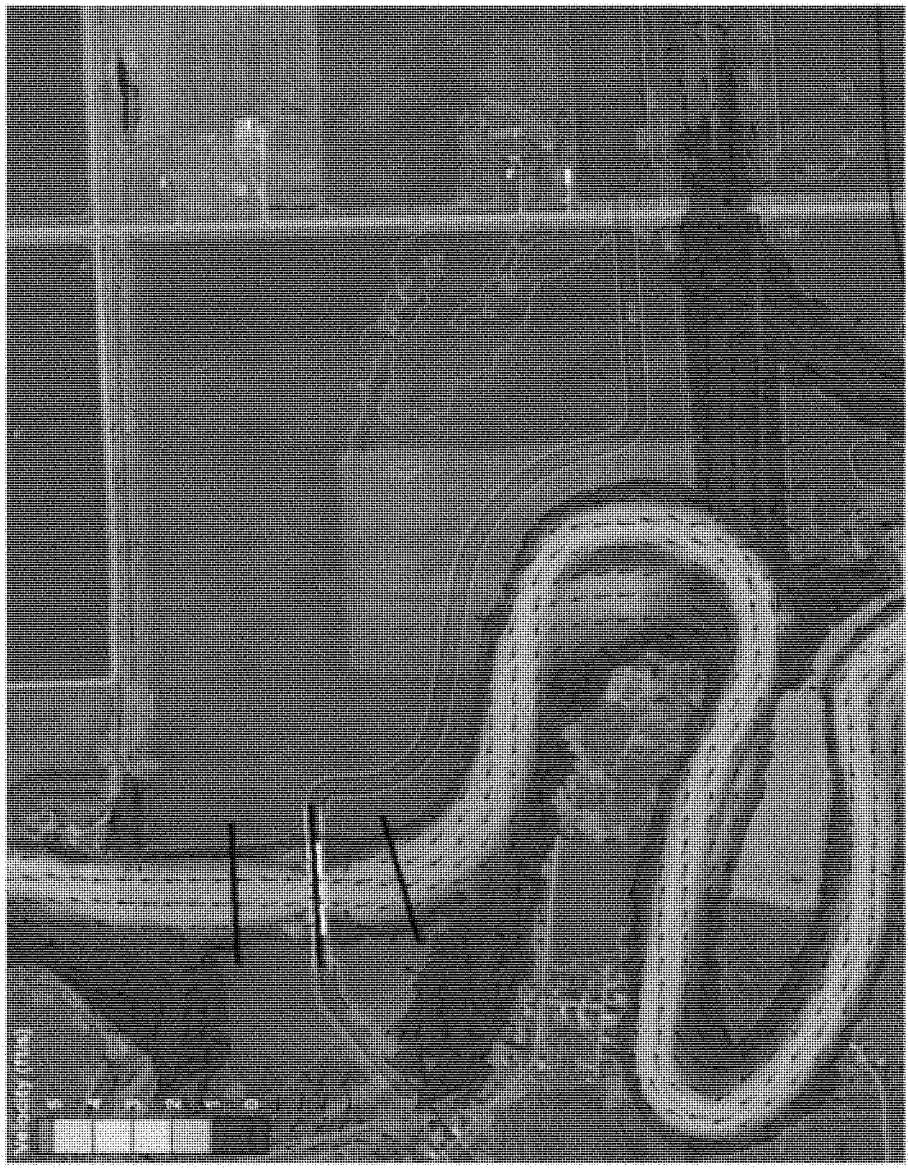

Figure F-H3 Velocity Field for Minnesota Control Structure Post-construction Conditions 9,600 cfs Event Flow

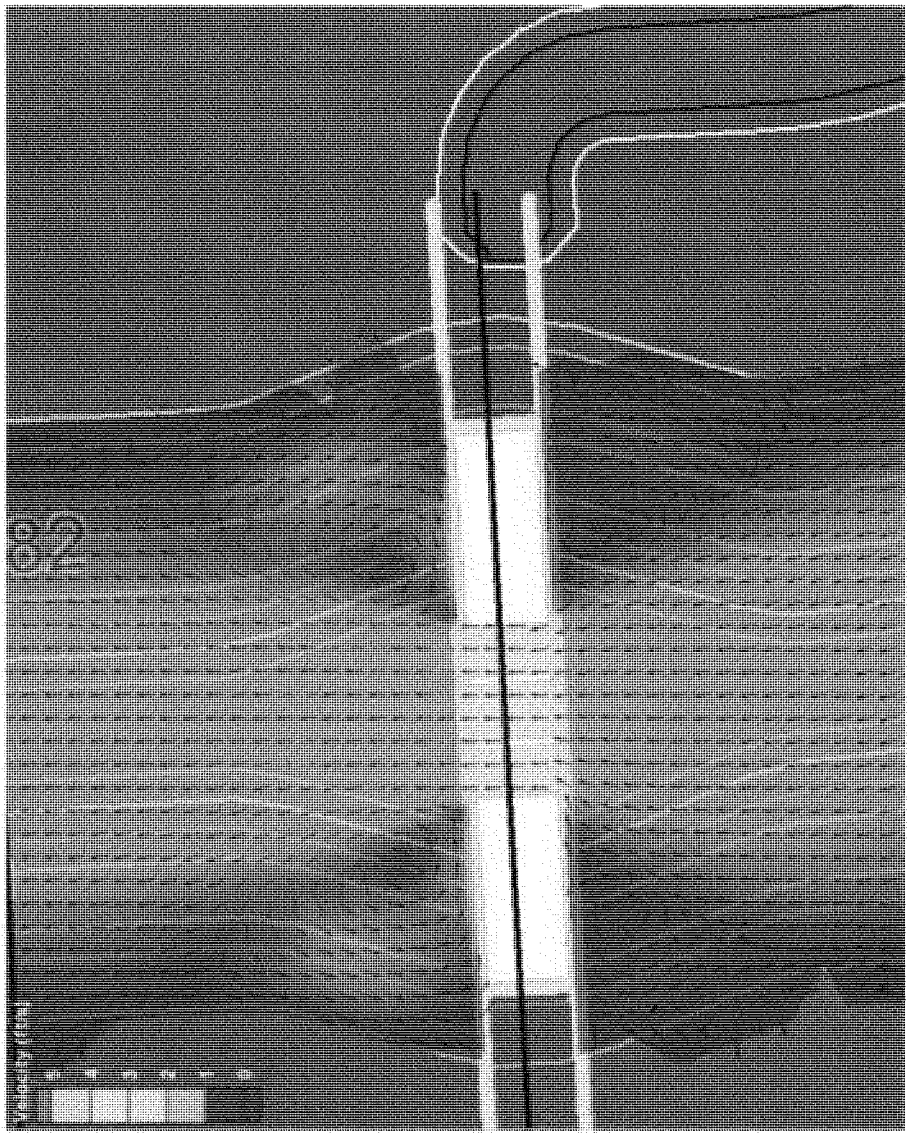

Figure F-H4 Velocity Field for Minnesota Control Structure Post-construction Conditions Control Structure Close Up 9,600 cfs Event Flow

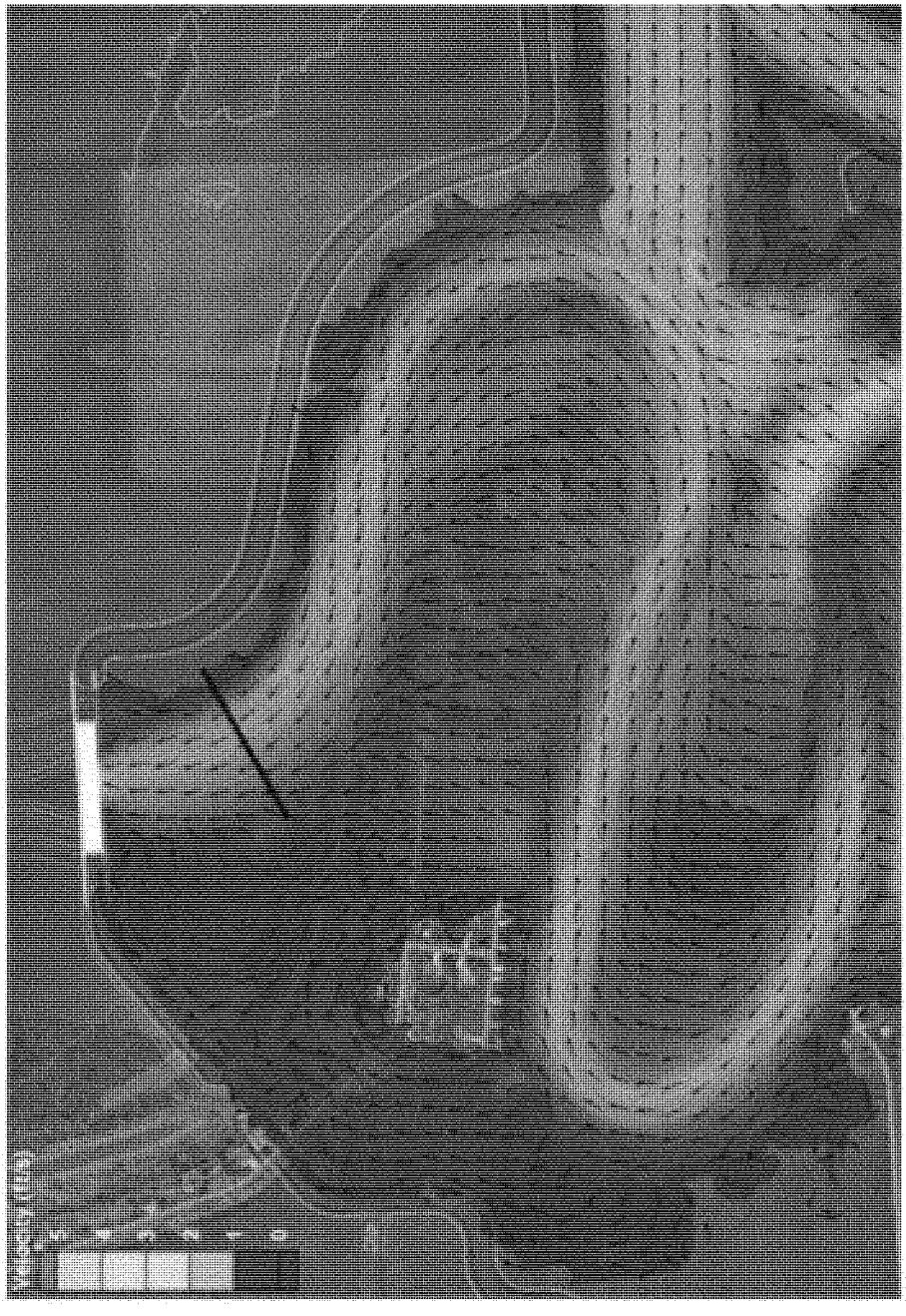

Figure F-H5 Velocity Field for Minnesota Control Structure Post-construction Conditions Upstream 20-Year Event Flow

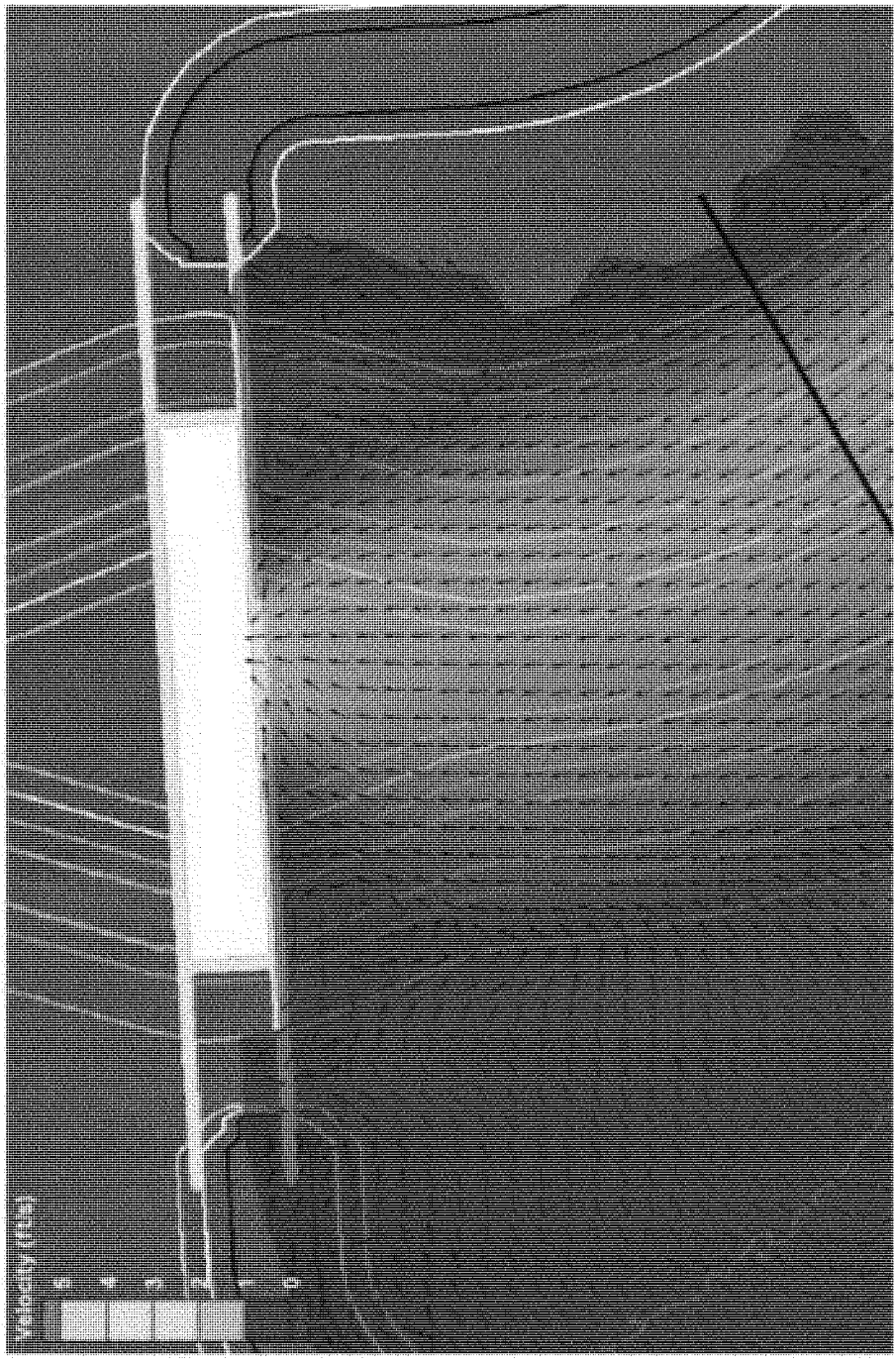

Figure F-H6 Velocity Field for Minnesota Control Structure Post-construction Conditions Upstream Control Structure Close Up 20-Year Event Flow

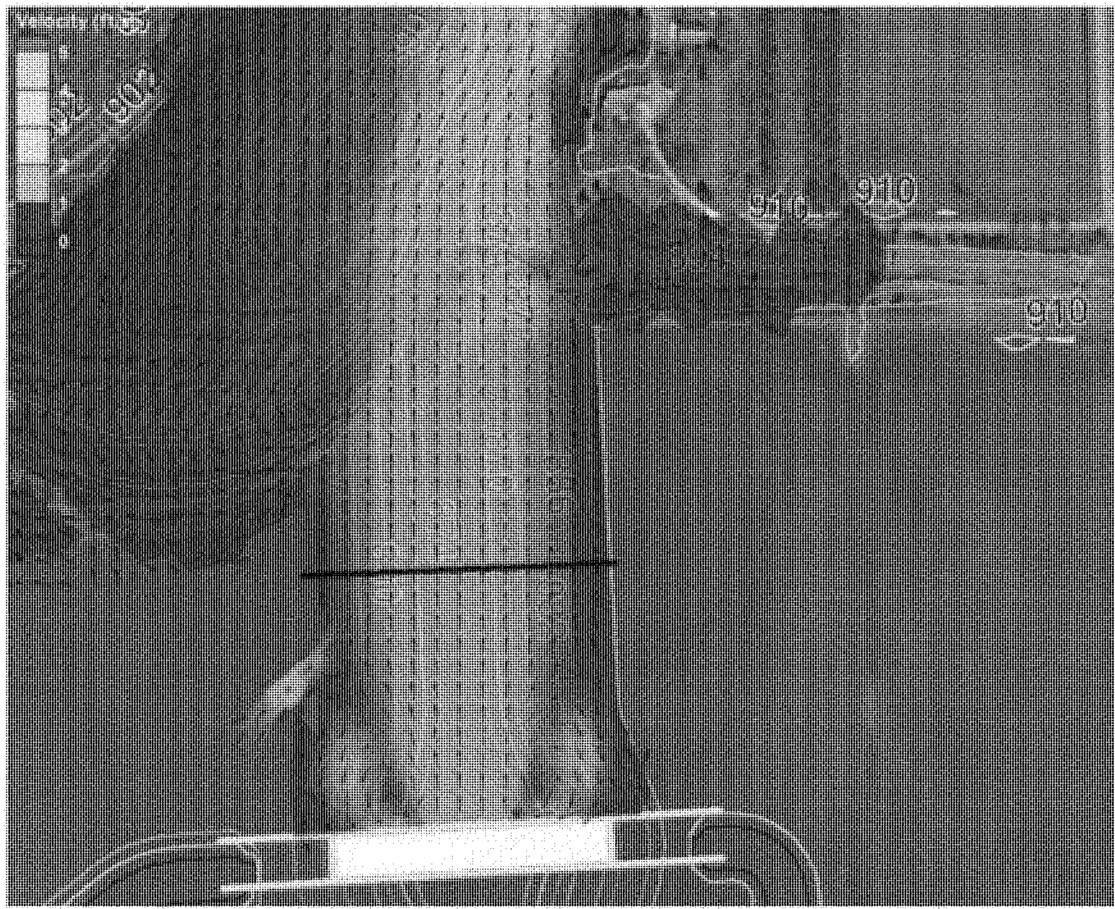

Figure F-H7 Velocity Field for Minnesota Control Structure Post-construction Conditions Downstream 20-Year Event Flow

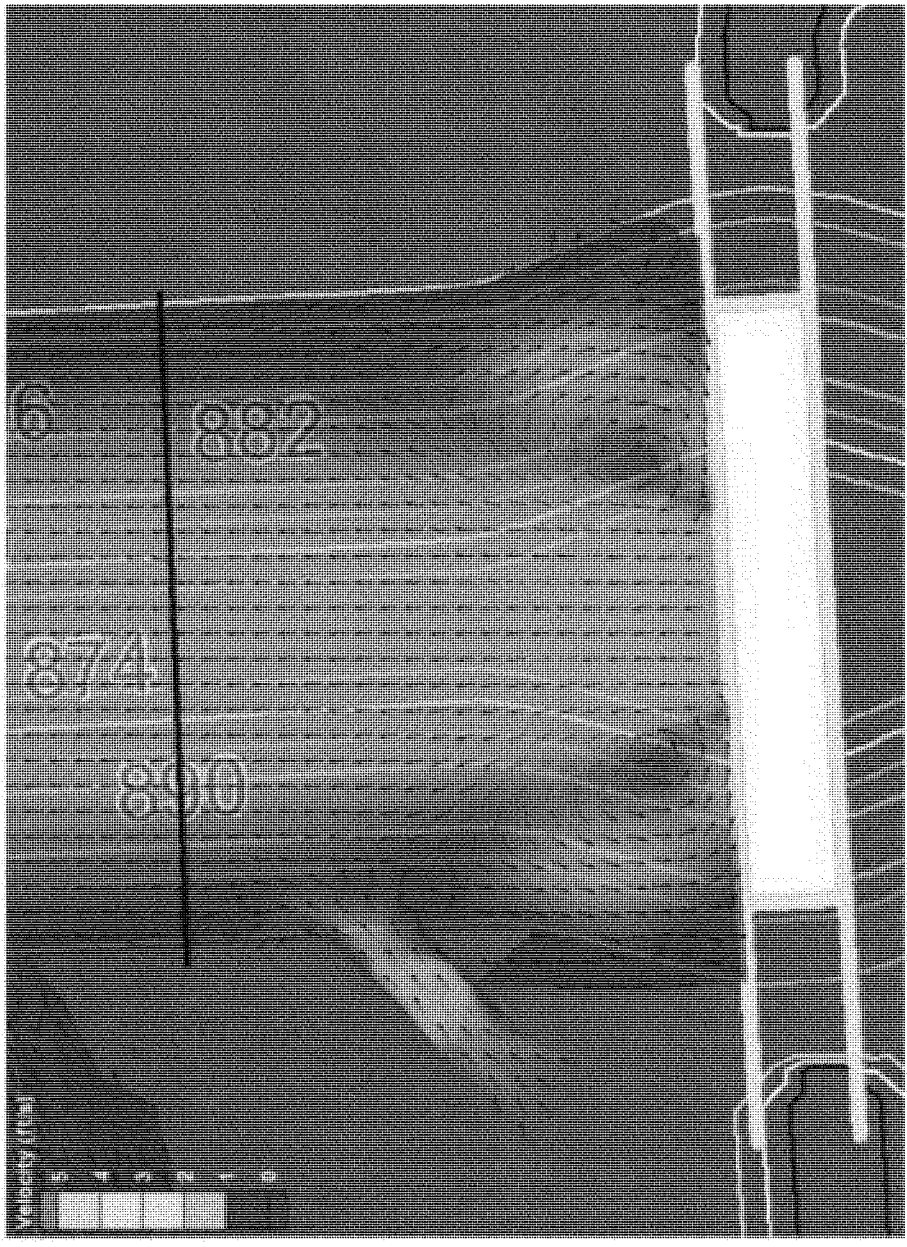

Figure F-H8 Velocity Field for Minnesota Control Structure Post-construction Conditions Downstream Control Structure Close Up 20-Year Event Flow

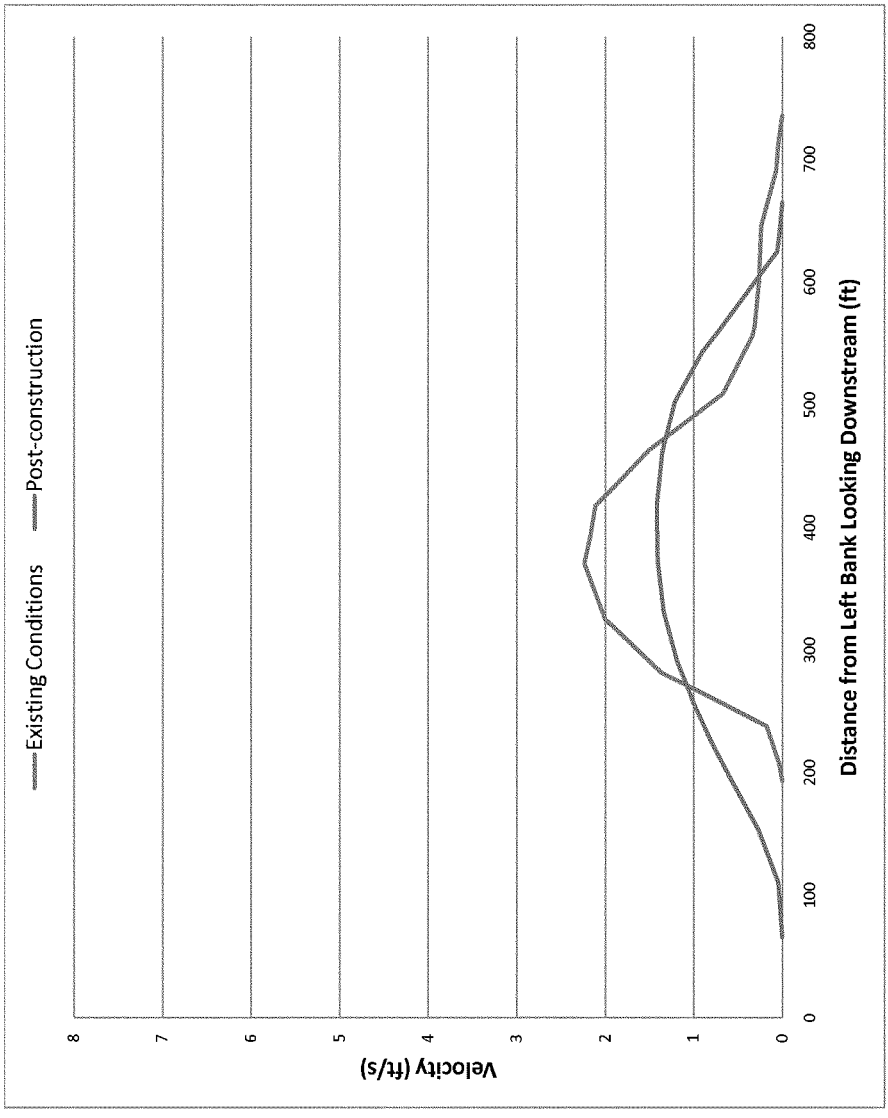

Figure F-H9 Cross Sectional Velocity Data for Minnesota Control Structure Upstream During 9,600 cfs Event

Cross Sectional Velocity Data for Minnesota Control Structure (9,600 cfs). Compares velocity at existing conditions to post-construction upstream of structure.

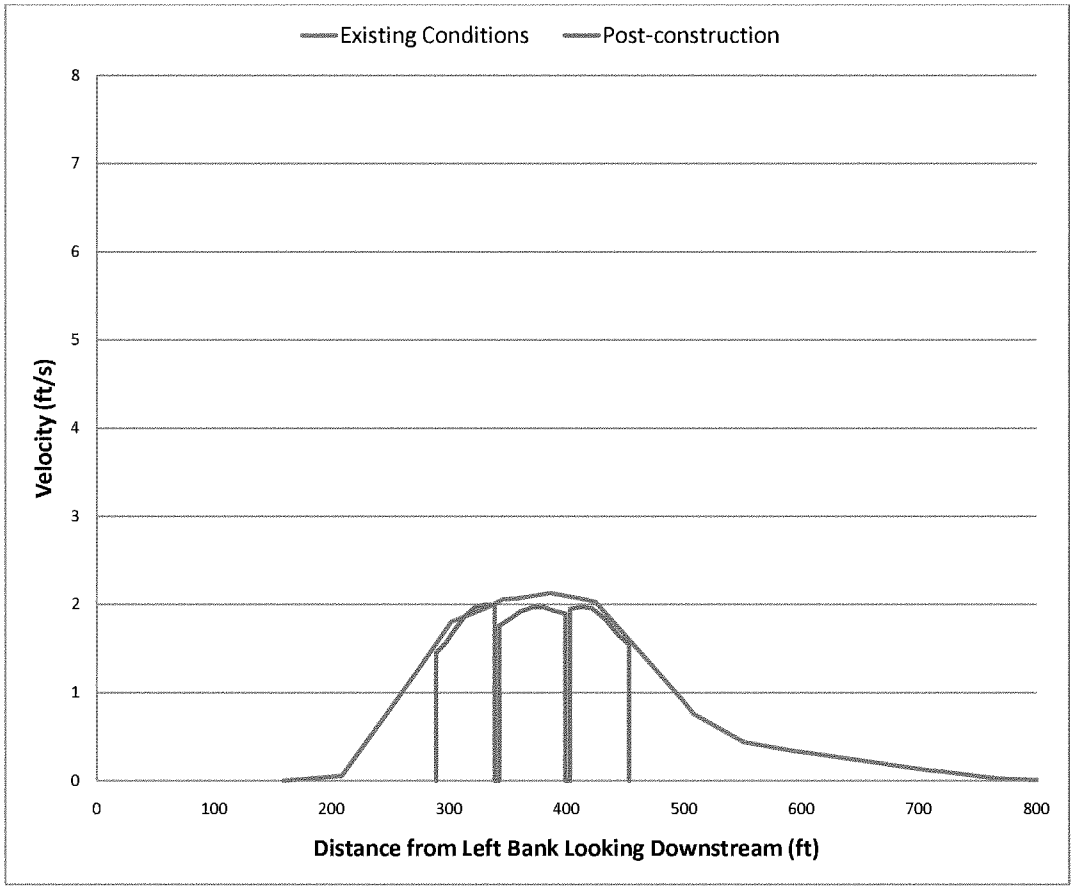

Figure F-H10 Cross Sectional Velocity Data for Minnesota Control Structure Through Structure During 9,600 cfs Event

Cross Sectional Velocity Data for Minnesota Control Structure (9,600 cfs). Compares velocity at existing conditions to post-construction through structure.

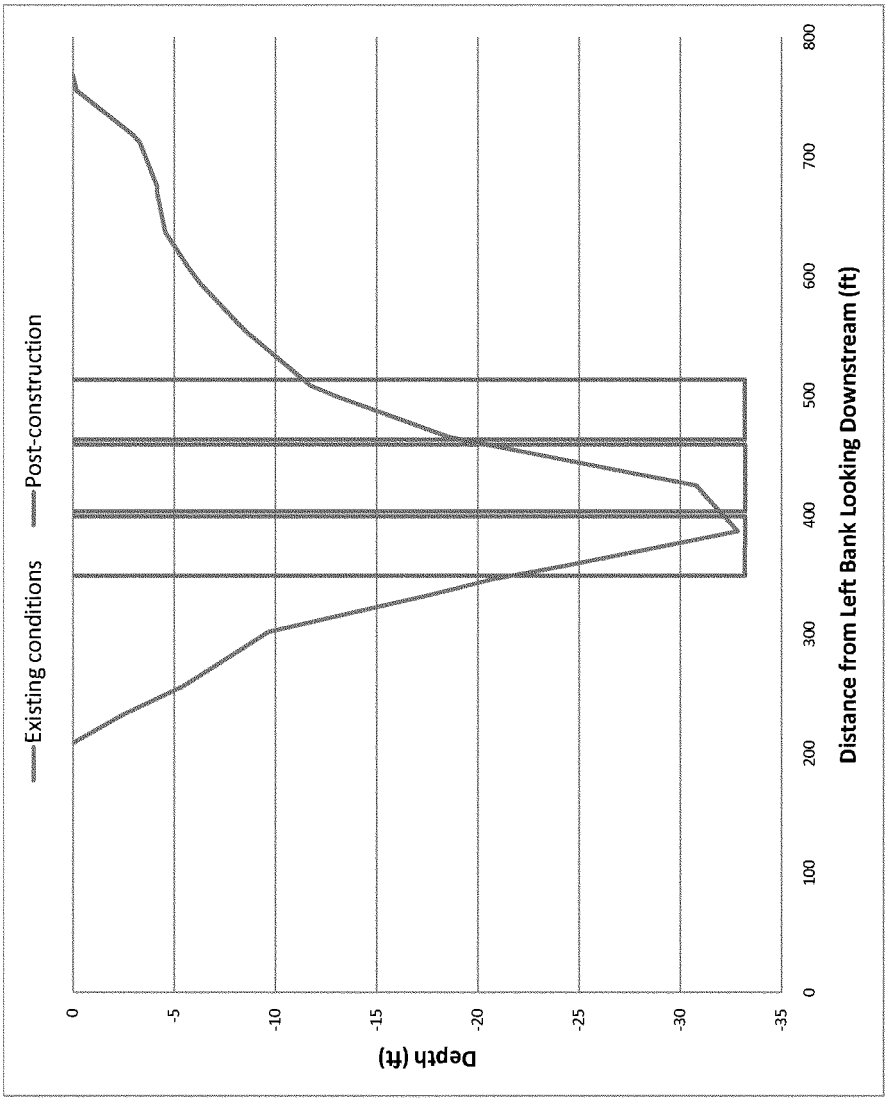

Figure F-H11 Cross Sectional Depth Data for Minnesota Control Structure Through Structure During 9,600 cfs Event

Cross Sectional Depth Data for Minnesota Control Structure (9,600 cfs). Compares depth at existing conditions to post-construction through structure.

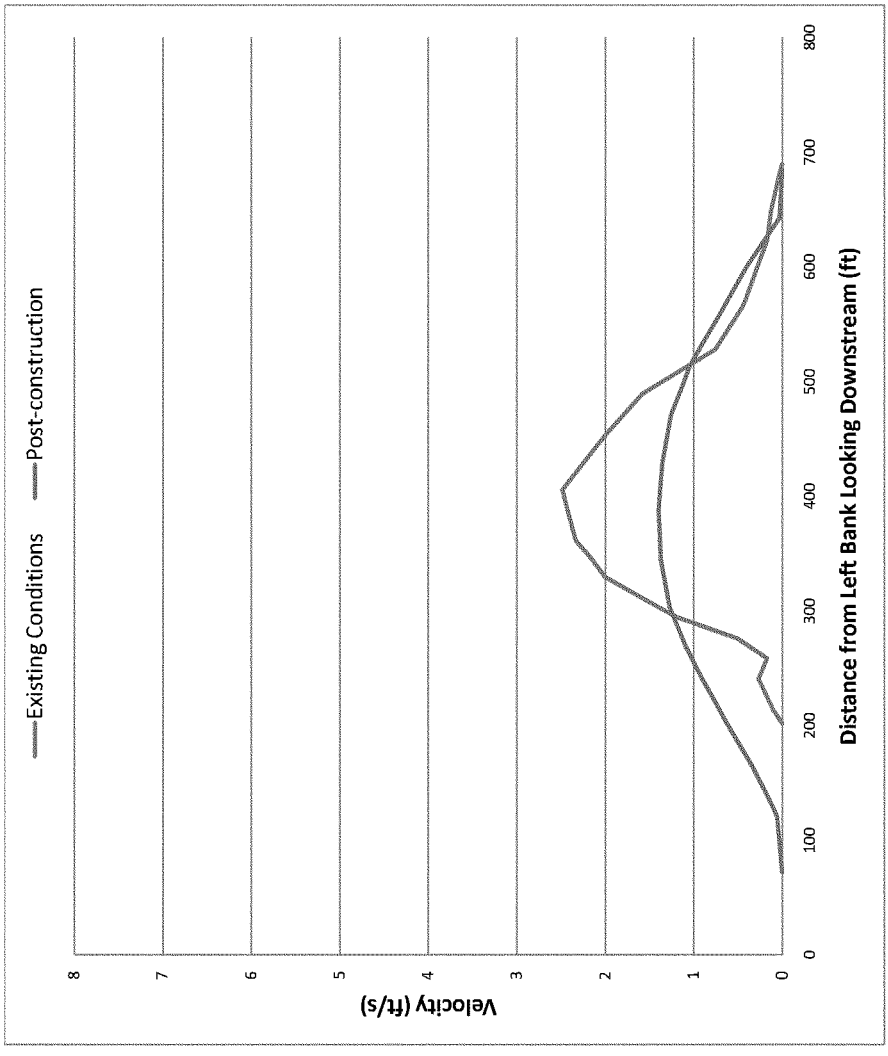

Figure F-H12 Cross Sectional Velocity Data for Minnesota Control Structure Downstream During 9,600 cfs Event

Cross Sectional Velocity Data for Minnesota Control Structure (9,600 cfs). Compares velocity at existing conditions to post-construction downstream of structure.

Figure F-H13 Cross Sectional Velocity Data for Minnesota Control Structure Upstream During 20-Year Event

Cross Sectional Velocity Data for Minnesota Control Structure (20-Year). Compares velocity at existing conditions to post-construction upstream of structure.

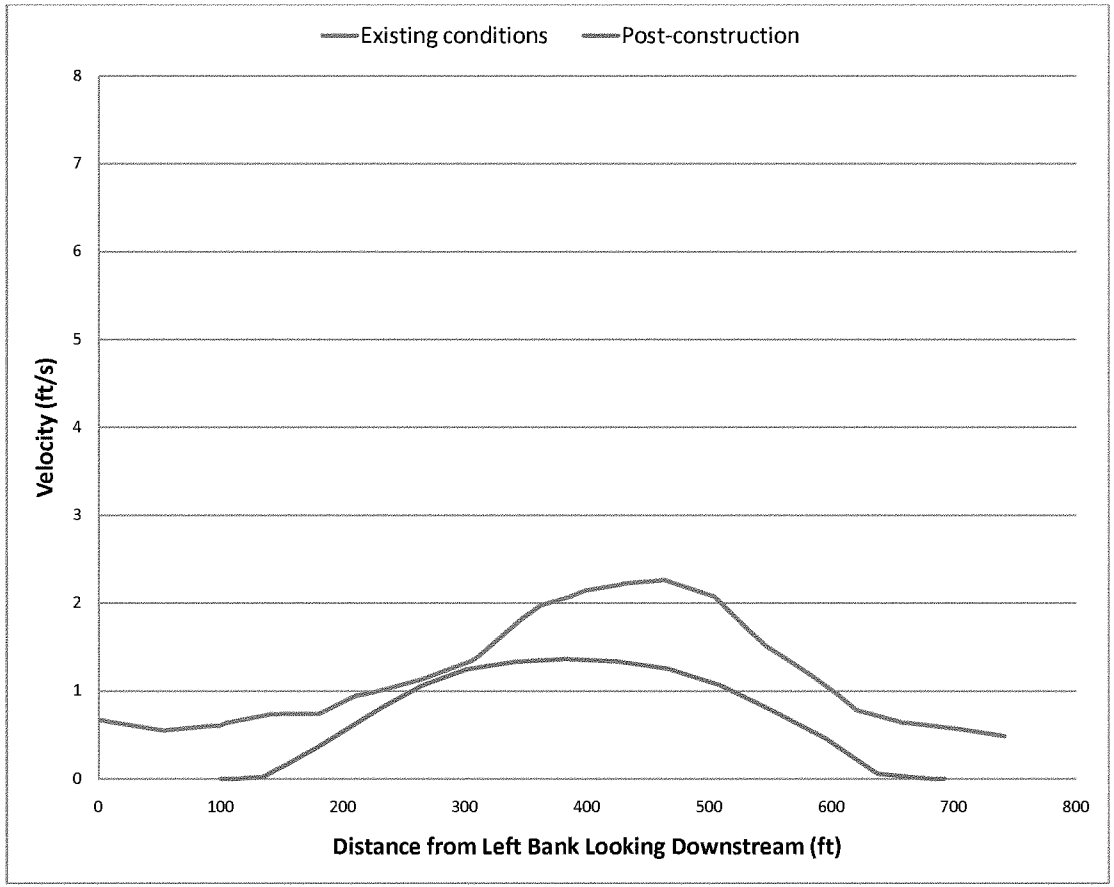

Figure F-H14 Cross Sectional Velocity Data for Minnesota Control Structure Downstream During 20-Year Event

Cross Sectional Velocity Data for Minnesota Control Structure (20-Year). Compares velocity at existing conditions to post-construction downstream of structure.

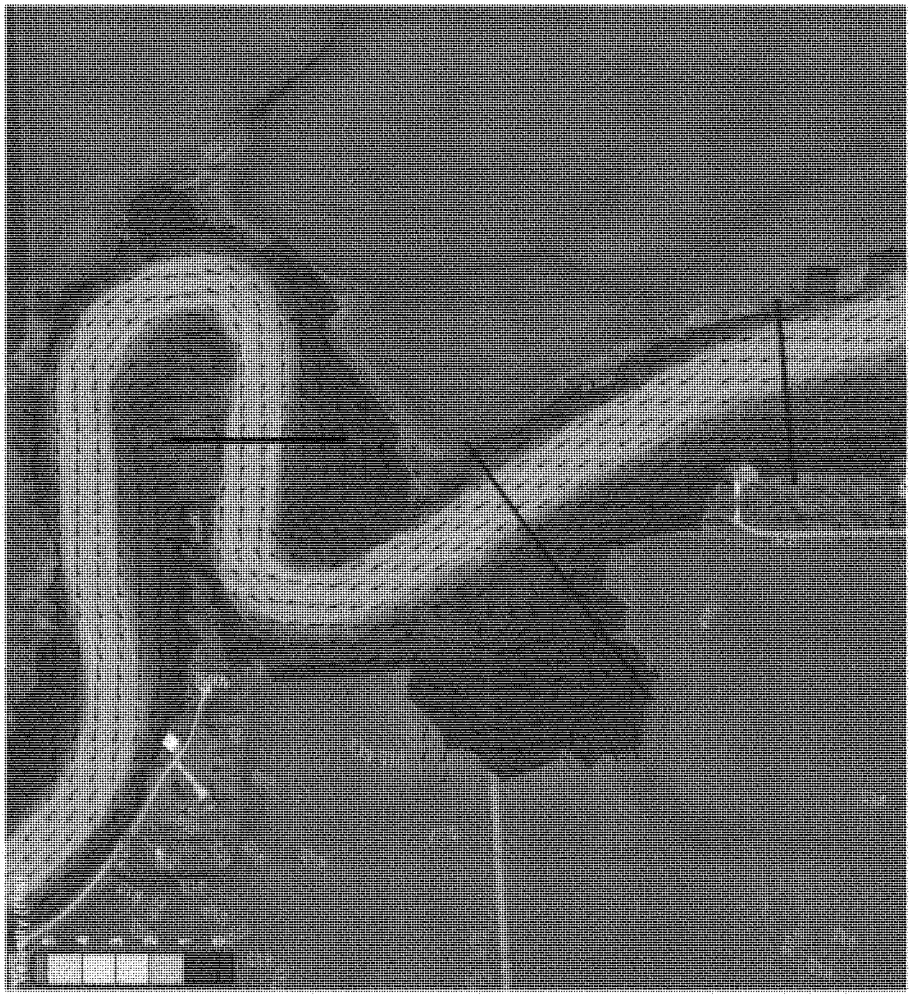

Figure F-H15 Velocity Field for North Dakota Control Structure Existing Conditions 6,100 cfs Event Flow

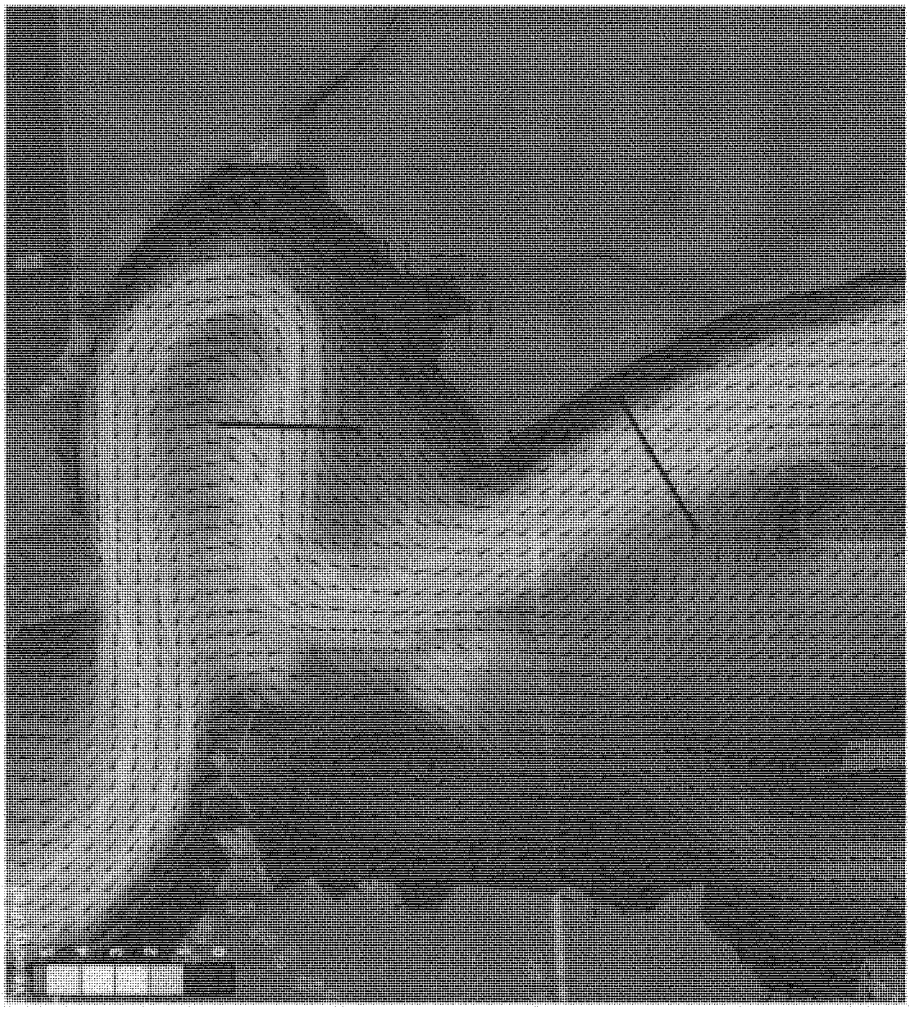

Figure F-H16 Velocity Field for North Dakota Control Structure Existing Conditions 20-Year Event Flow

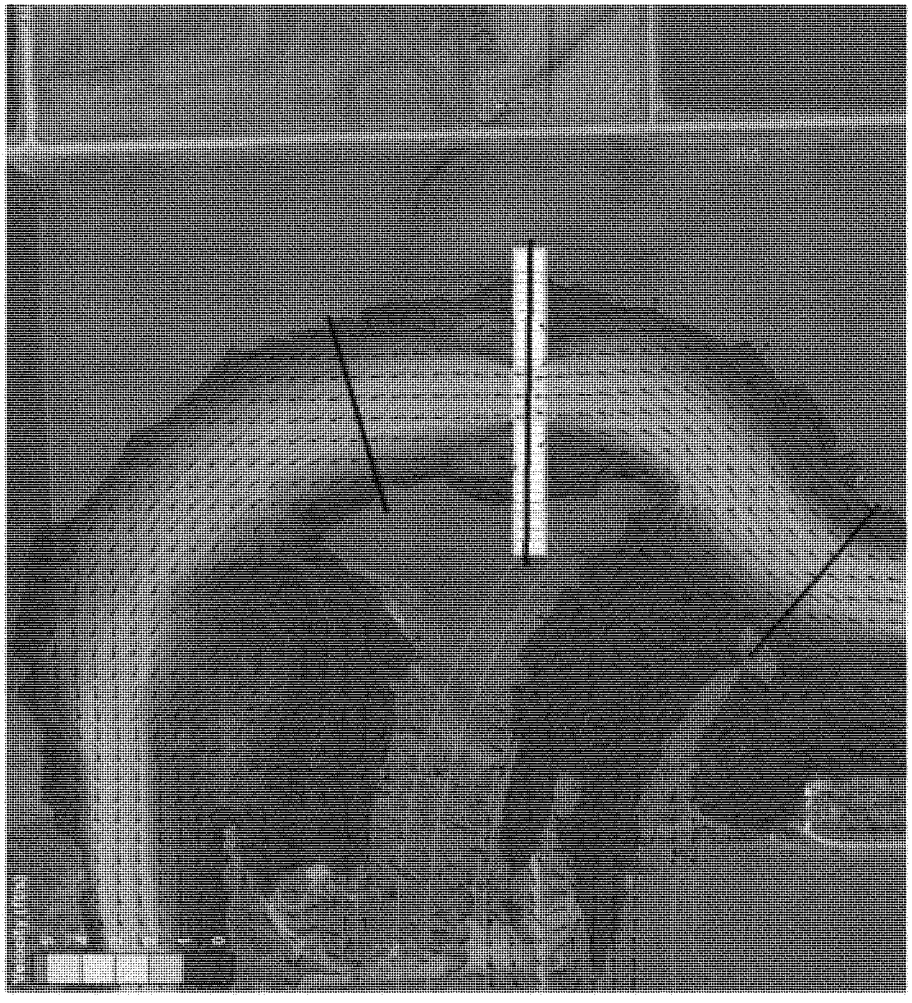

Figure F-H17 Velocity Field for North Dakota Control Structure Post-construction Conditions 6,100 cfs Event Flow

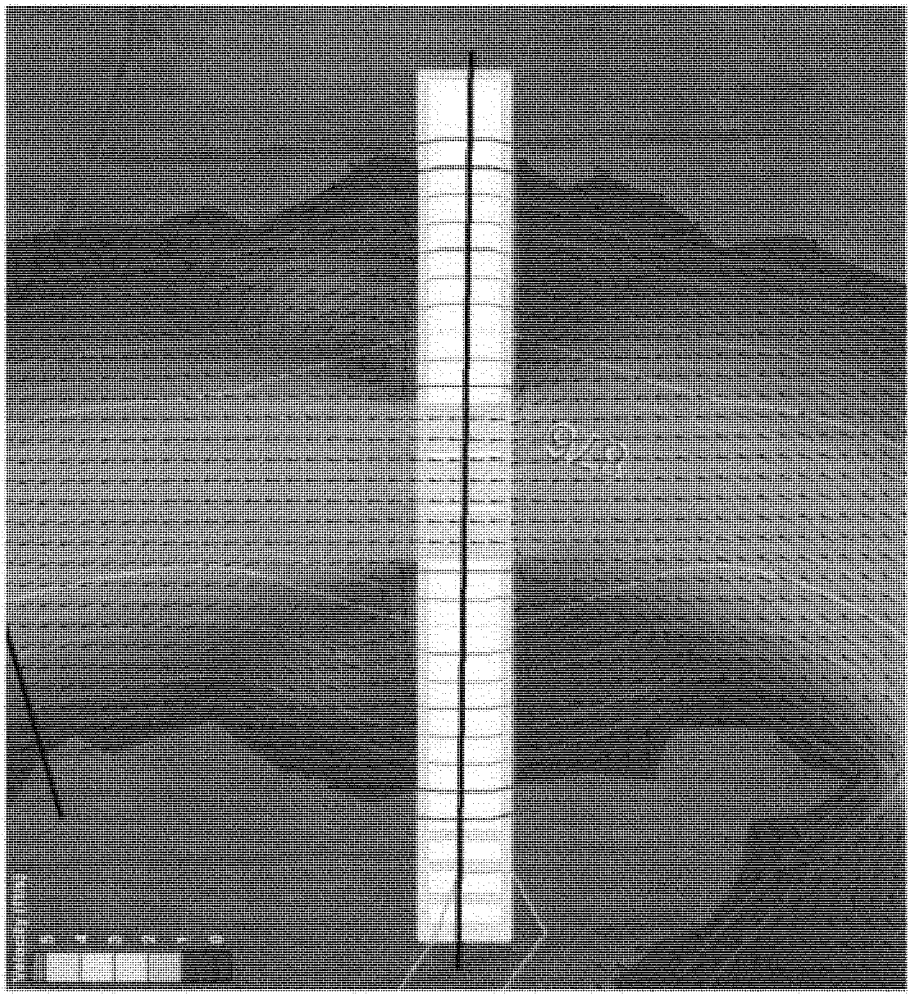

Figure F-H18 Velocity Field for North Dakota Control Structure Post-construction Conditions Close Up 6,100 cfs Event Flow

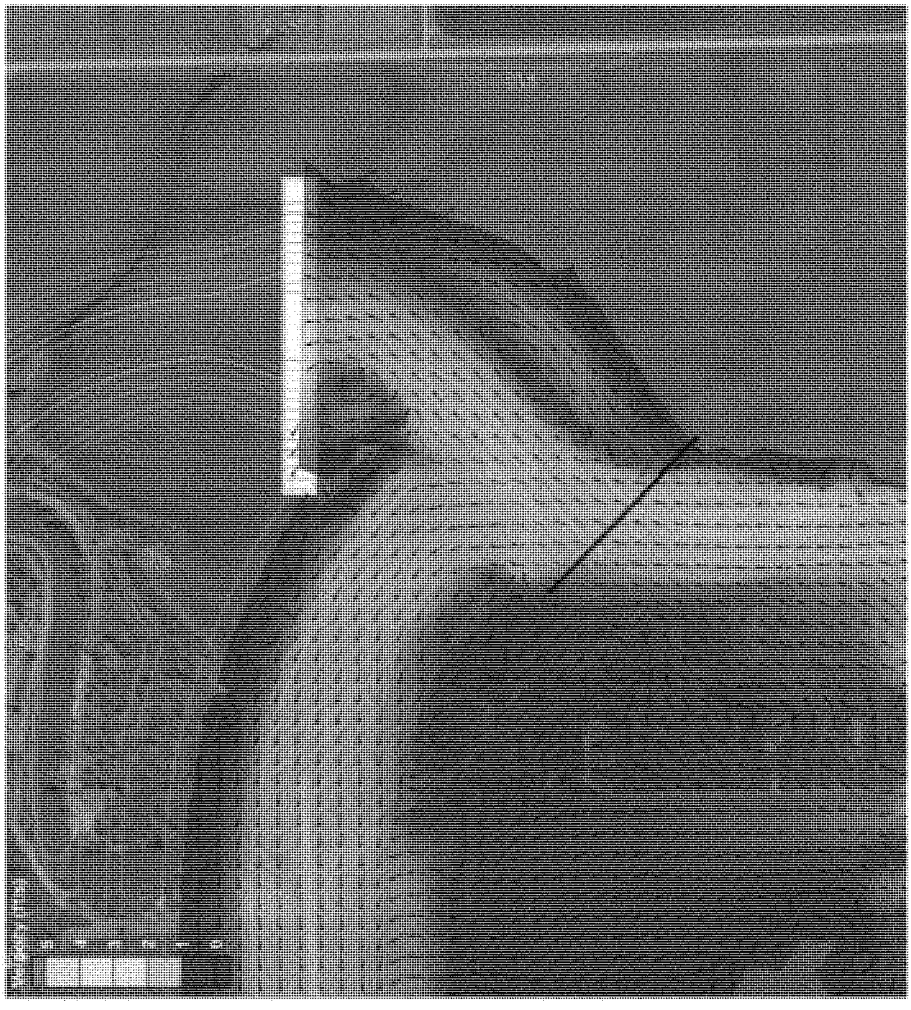

Figure F-H19 Velocity Field for North Dakota Control Structure Post-construction Conditions Upstream 20-Year Event Flow

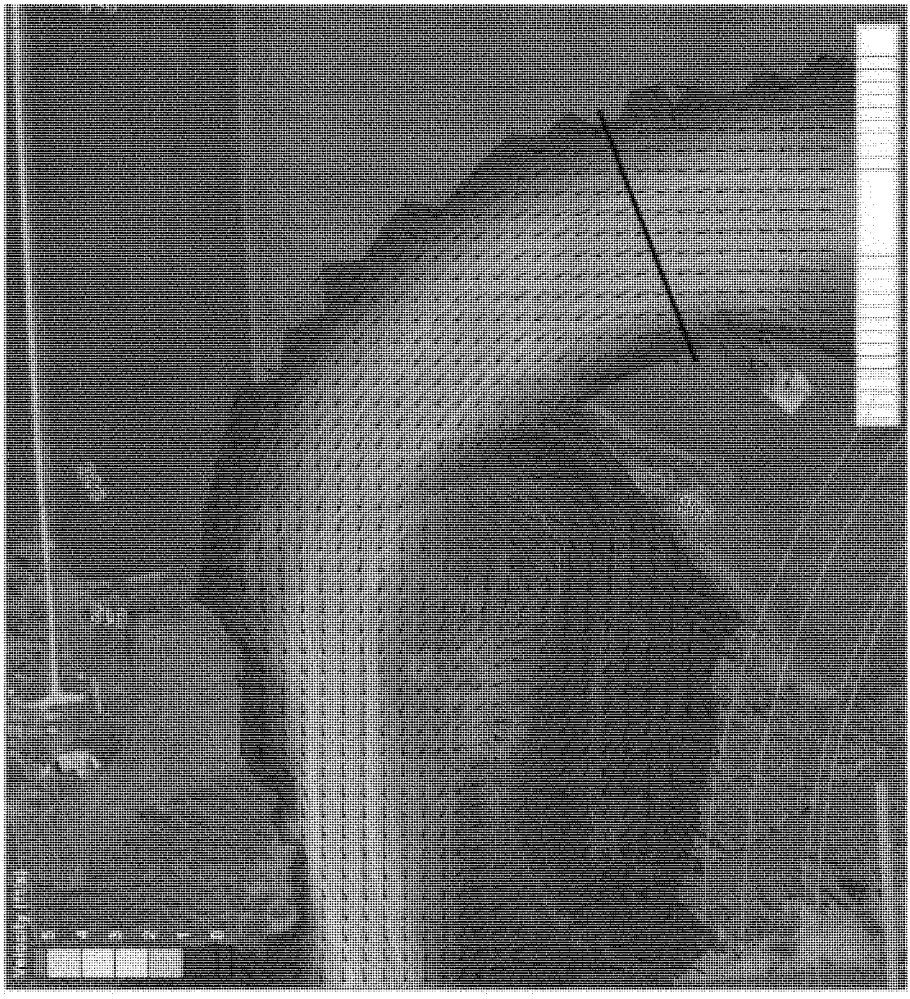

Figure F-H20 Velocity Field for North Dakota Control Structure Post-construction Conditions Downstream 20-Year Event Flow

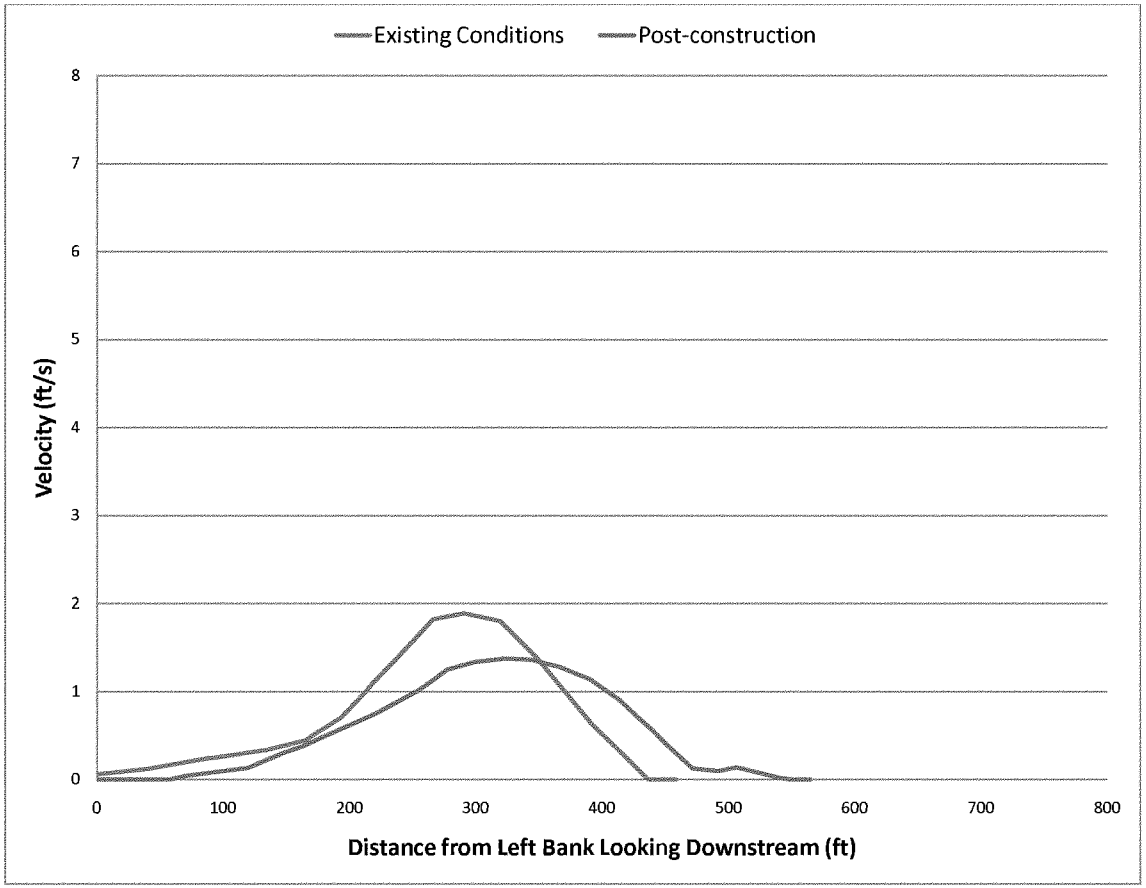

Figure F-H21 Cross Sectional Velocity Data for North Dakota Control Structure Upstream During 6,100 cfs Event

Cross Sectional Velocity Data for North Dakota Control Structure (6,100 cfs). Compares velocity at existing conditions to post-construction upstream of structure.

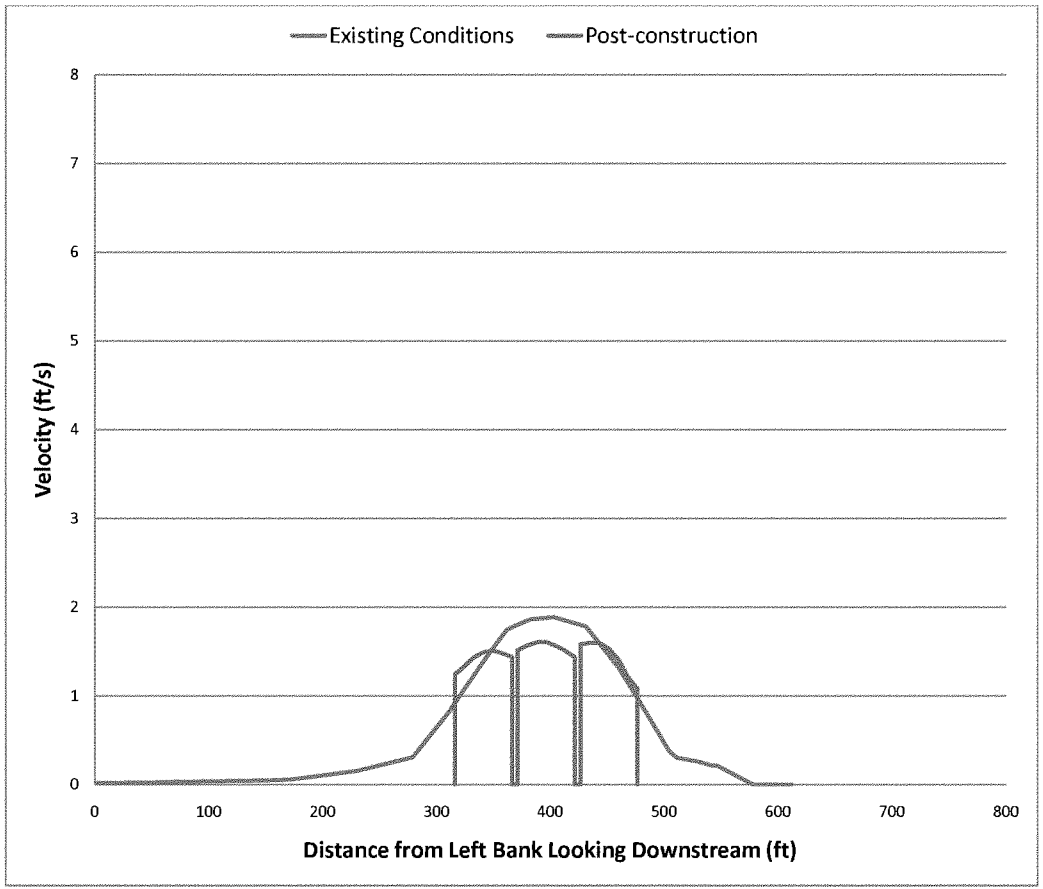

Figure F-H22 Cross Sectional Velocity Data for North Dakota Control Structure Through Structure During 6,100 cfs Event

Cross Sectional Velocity Data for North Dakota Control Structure (6,100 cfs). Compares velocity at existing conditions to post-construction through structure.

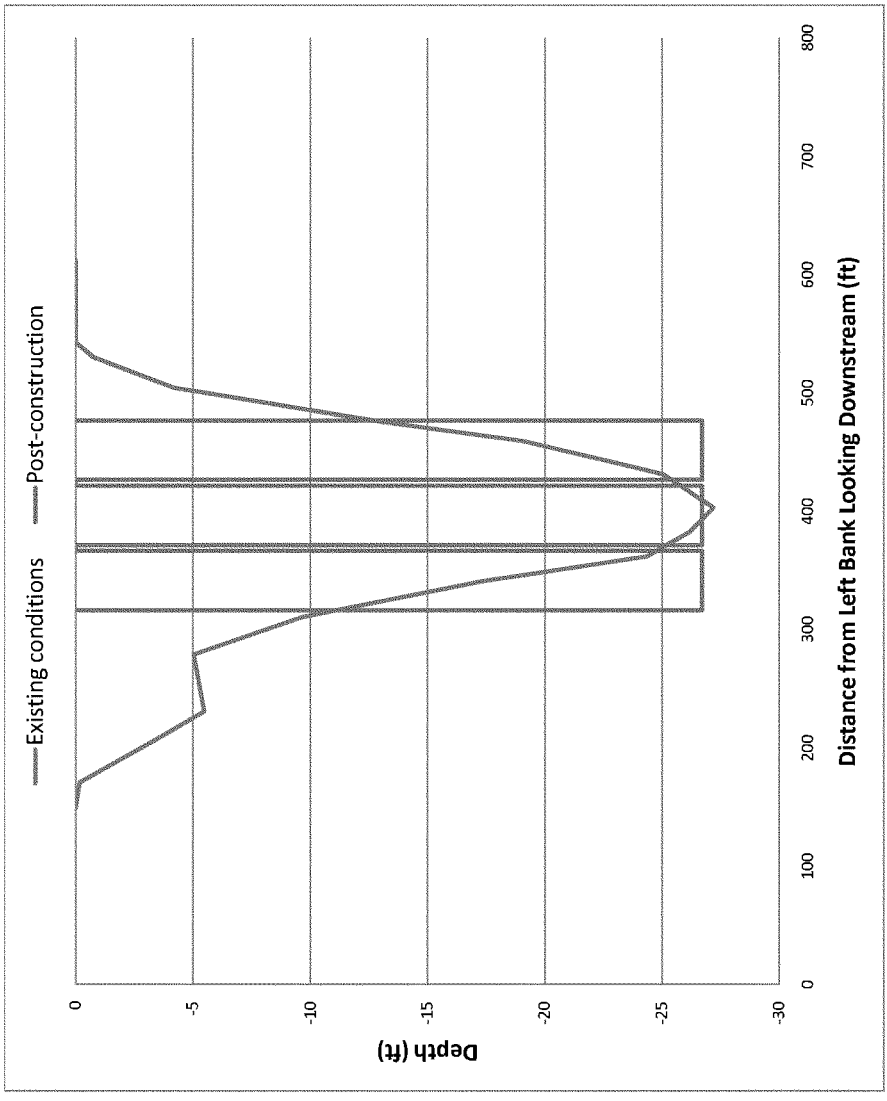

Figure F-H23 Cross Sectional Depth Data for North Dakota Control Structure Through Structure During 6,100 cfs Event

Cross Sectional Depth Data for North Dakota Control Structure (6,100 cfs). Compares depth at existing conditions to post-construction through structure.

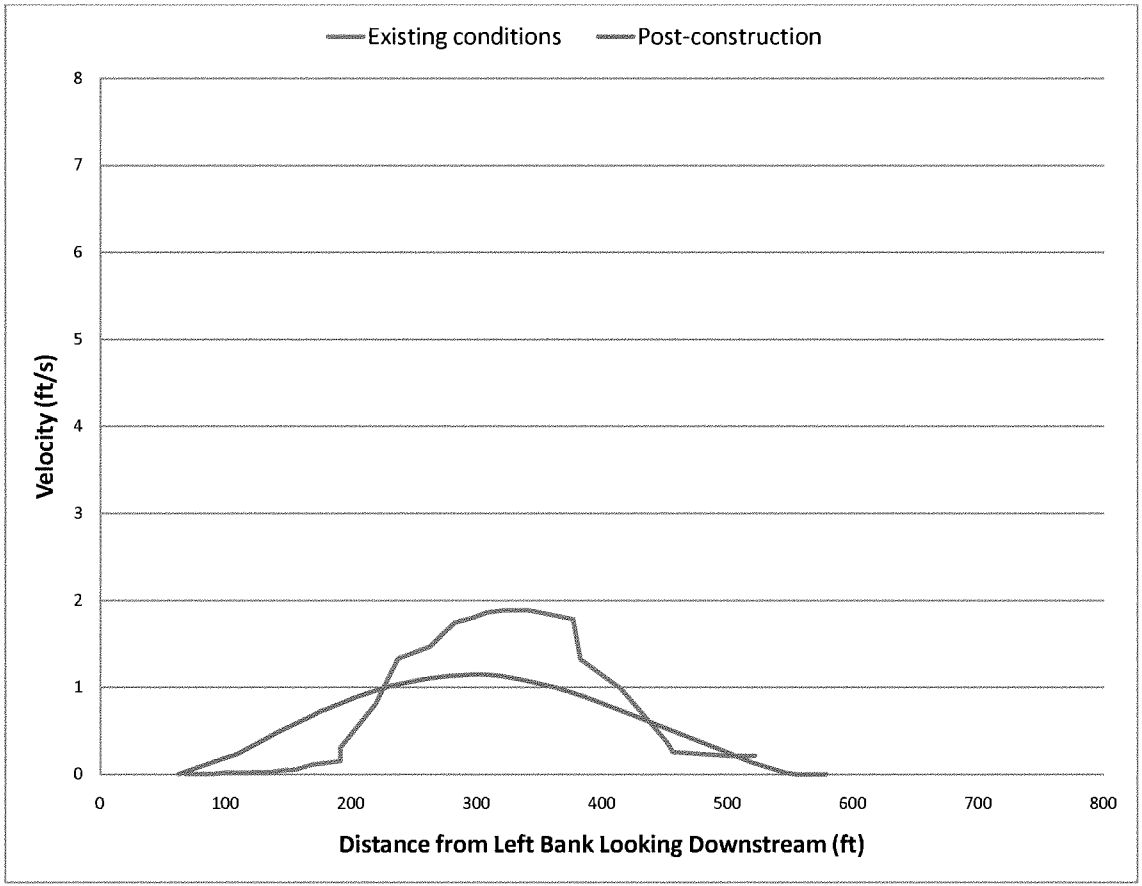

Figure F-H24 Cross Sectional Velocity Data for North Dakota Control Structure Downstream During 6,100 cfs Event

Cross Sectional Velocity for North Dakota Control Structure (6,100 cfs) Compares velocity at existing conditions to post-construction downstream of structure.

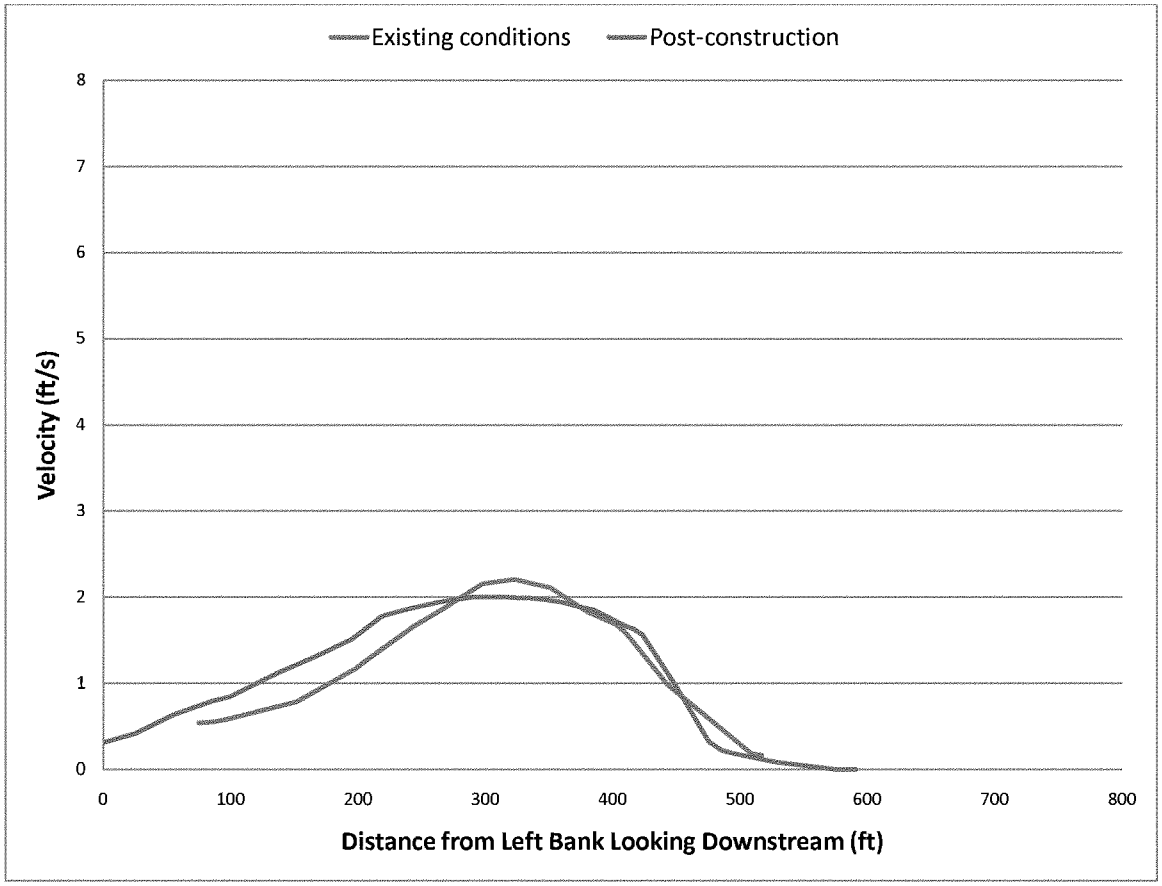

Figure F-H25 Cross Sectional Velocity Data for North Dakota Control Structure Upstream During 20-Year Event

Cross Sectional Velocity for North Dakota Control Structure (20-Year cfs) Compares velocity at existing conditions to post-construction upstream of structure.

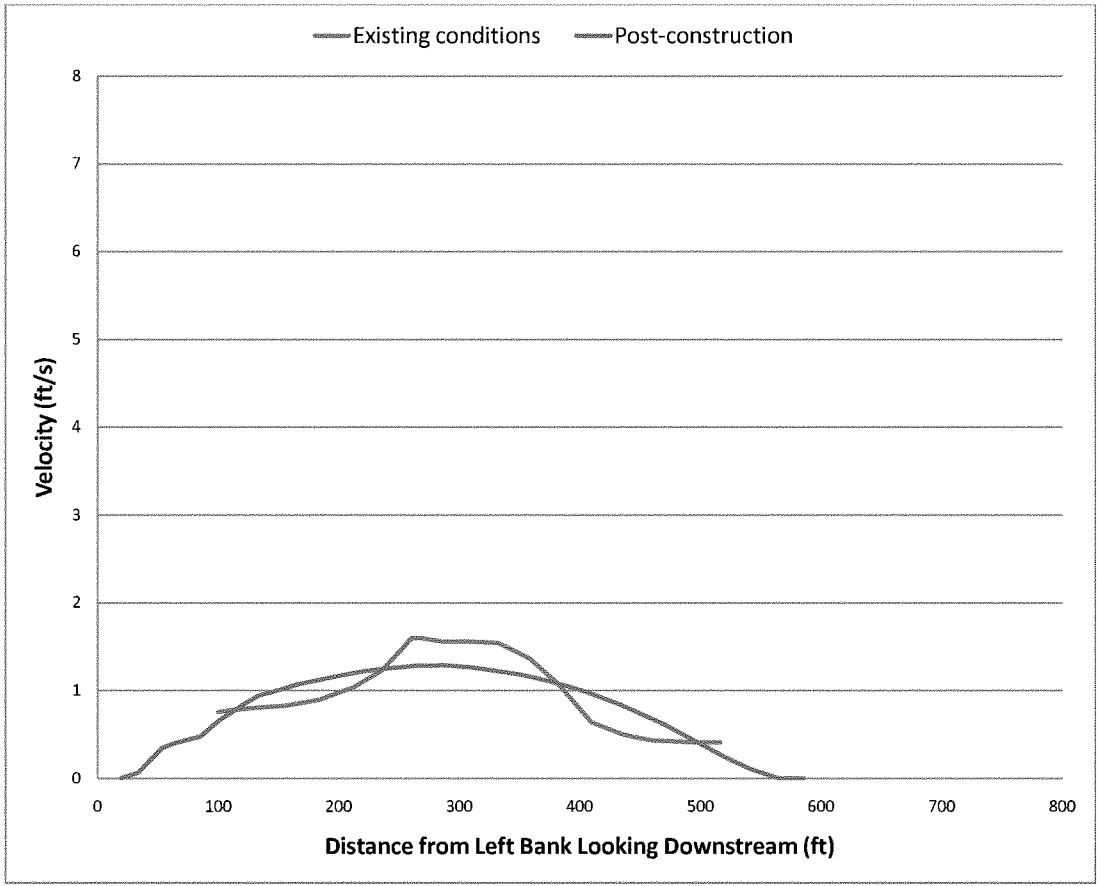

Figure F-H26 Cross Sectional Velocity Data for North Dakota Control Structure Downstream During 20-Year Event

Cross Sectional Velocity for North Dakota Control Structure (20-Year) Compares velocity at existing conditions to post-construction downstream of structure.

Figure F-H27 Velocity Field for Wild Rice River Control Structure Existing Conditions 5-Year Event Flow

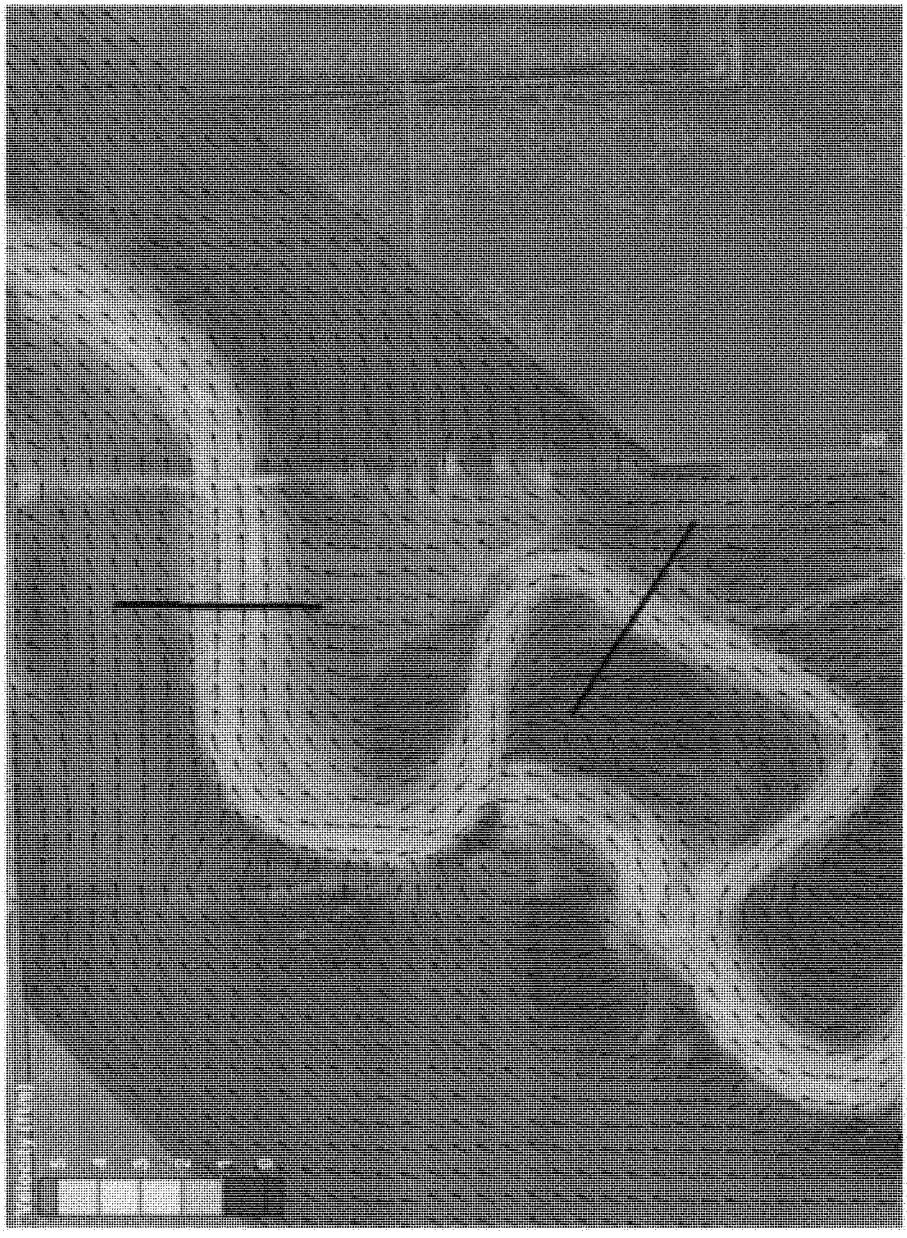

Figure F-H28 Velocity Field for Wild Rice River Existing Conditions 20-Year Event Flow

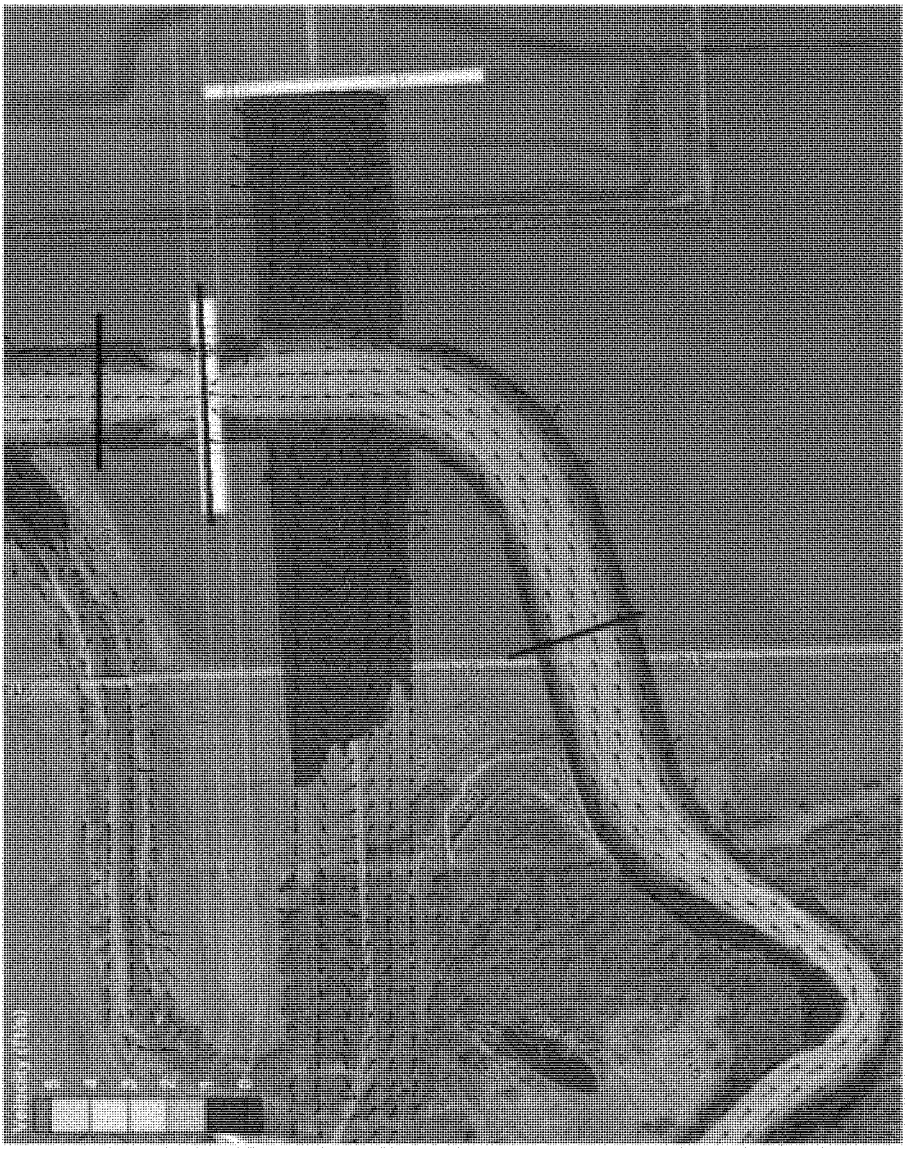

Figure F-H29 Velocity Field for Wild Rice River Post-construction Conditions 5-Year Event Flow

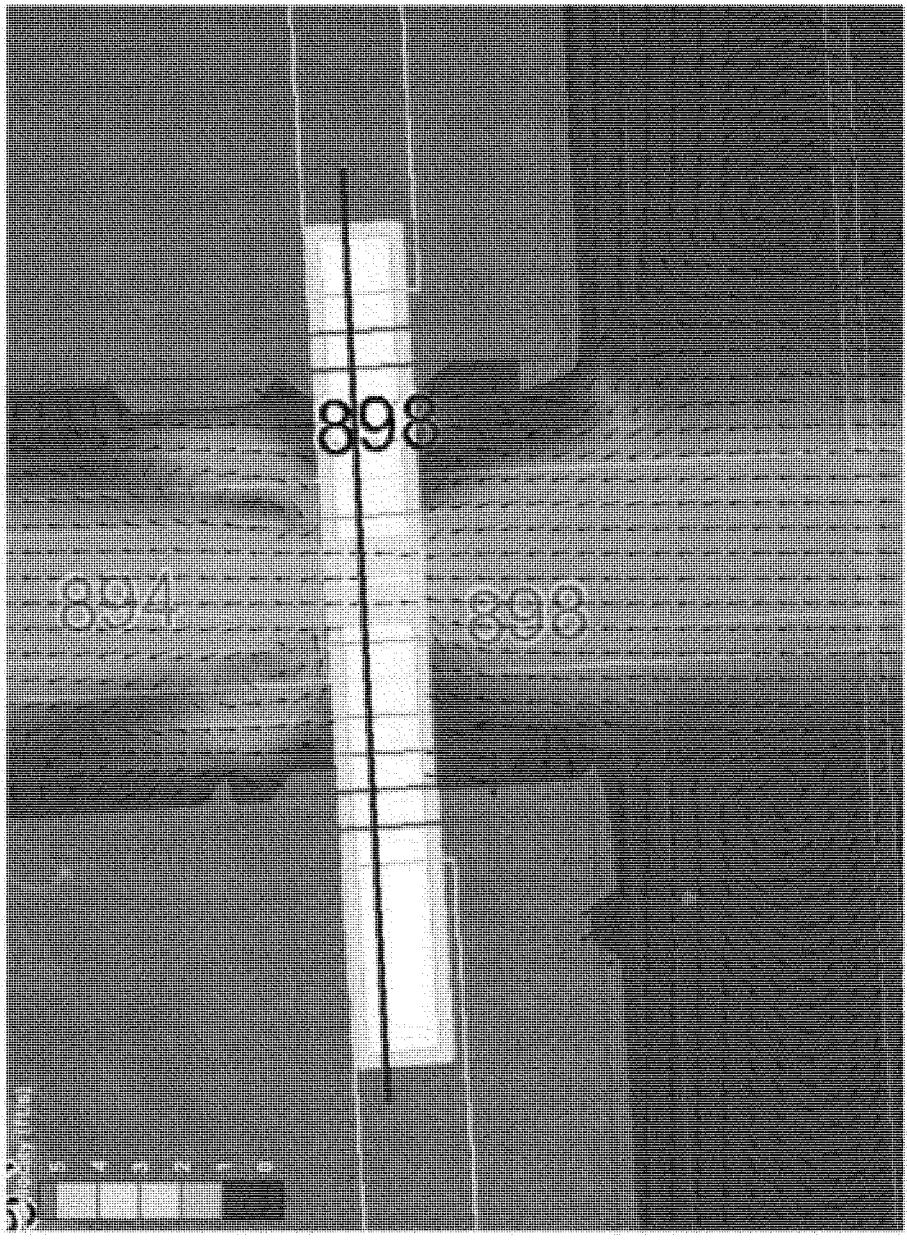

Figure F-H30 Velocity Field for Wild Rice River Post-construction Conditions Close Up 5-Year Event Flow

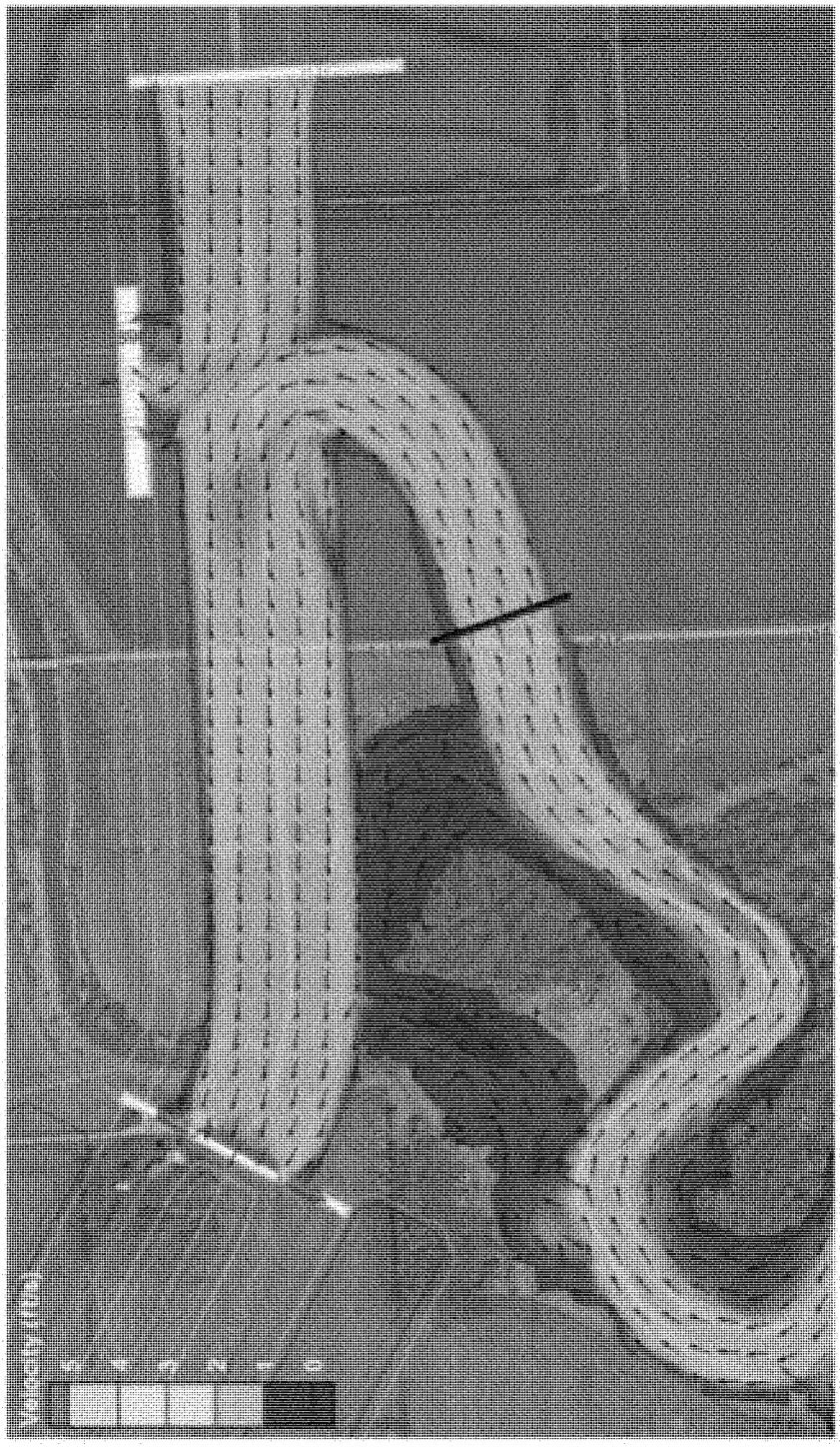

Figure F-H31 Velocity Field for Wild Rice River Post-construction Conditions Upstream 20-Year Event Flow

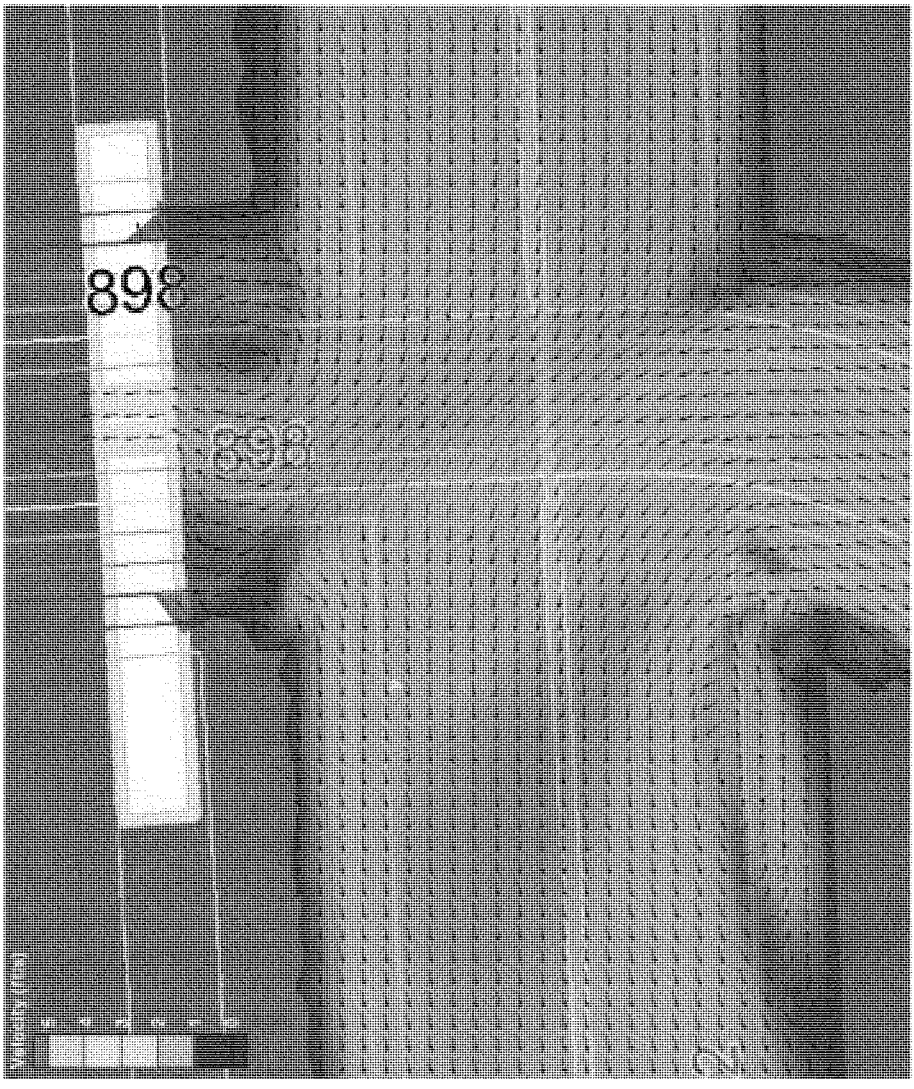

Figure F-H32 Velocity Field for Wild Rice River Post-construction Conditions Upstream Close Up 20-Year Event Flow

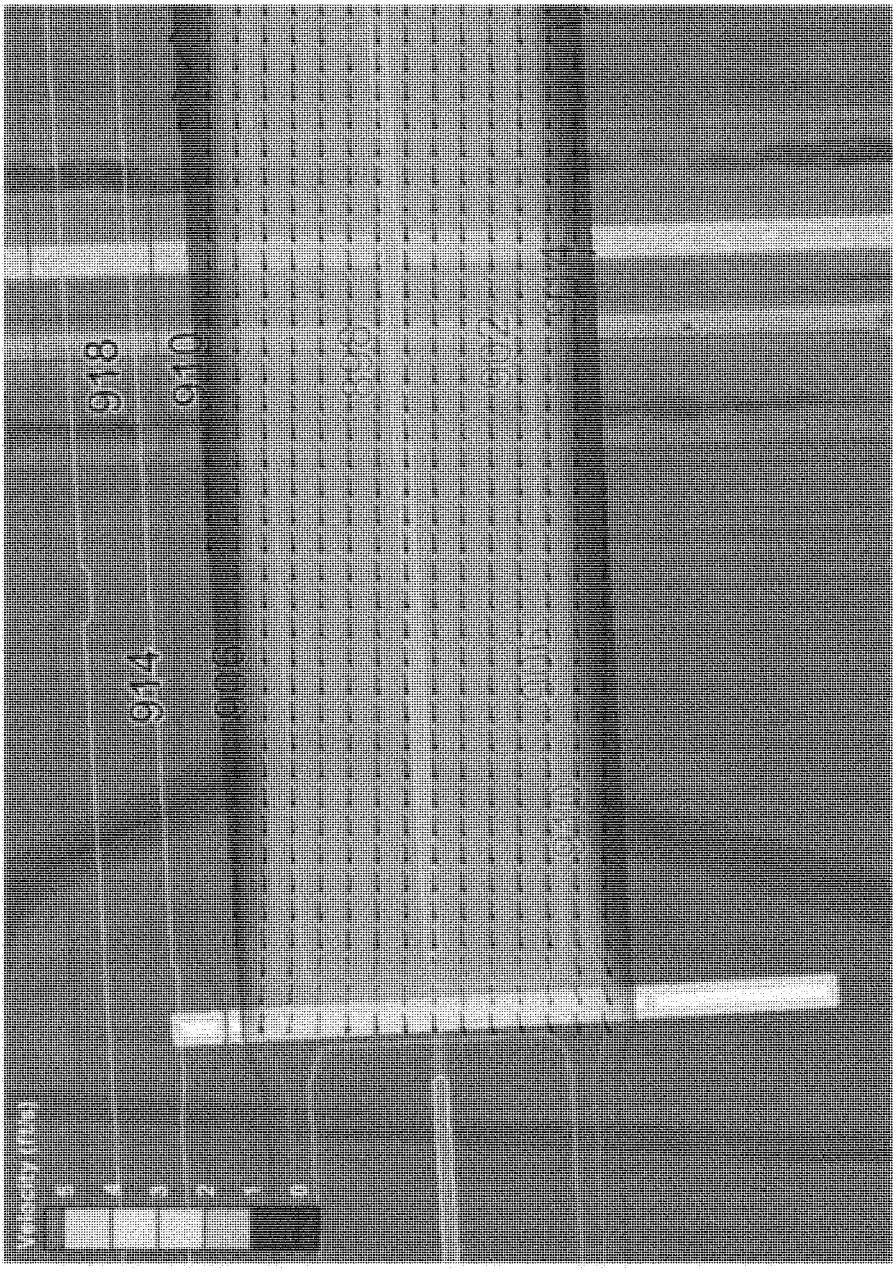

Figure F-H33 Velocity Field for Wild Rice River Post-construction Connecting Channel 20-Year Event Flow

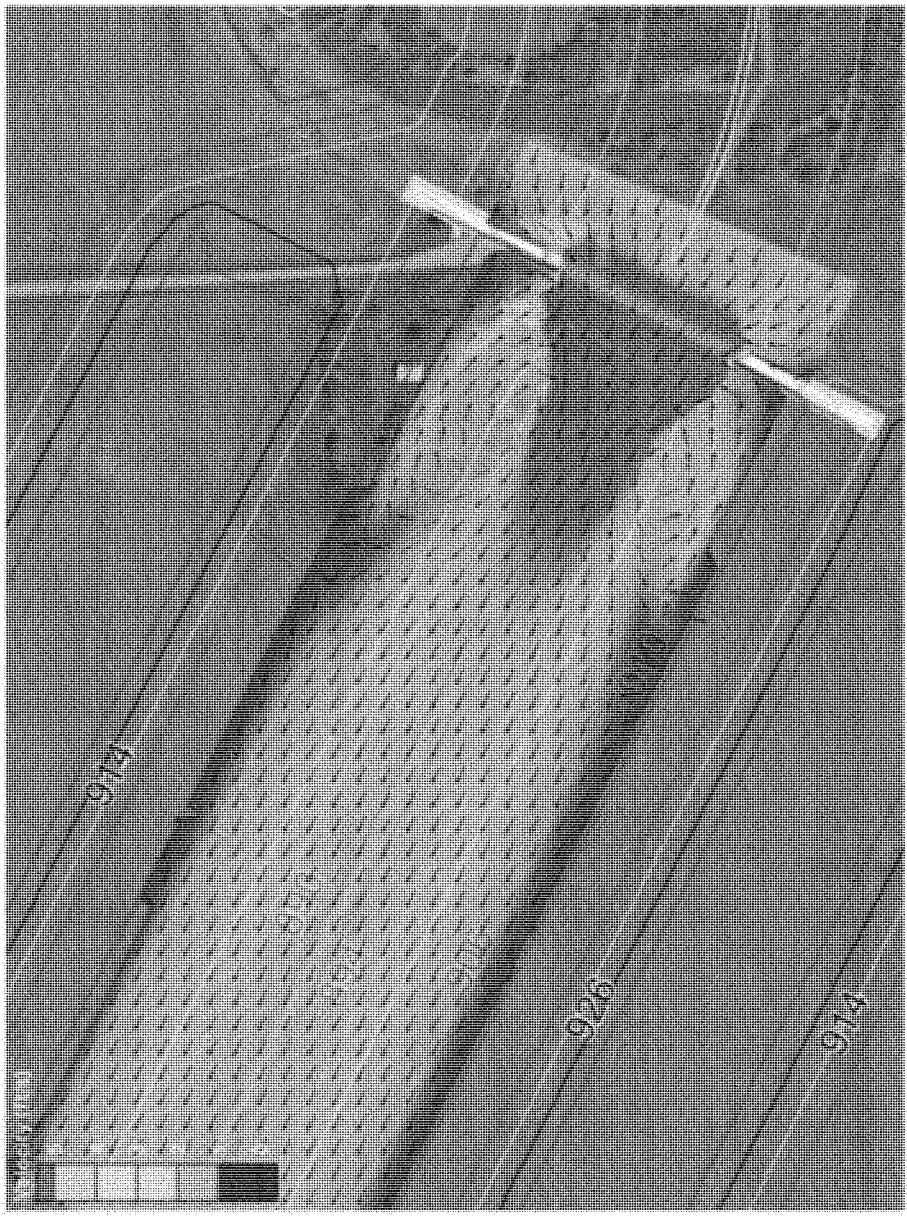

Figure F-H34 Velocity Field for Wild Rice River Post-construction Diversion Channel 20-Year Event Flow

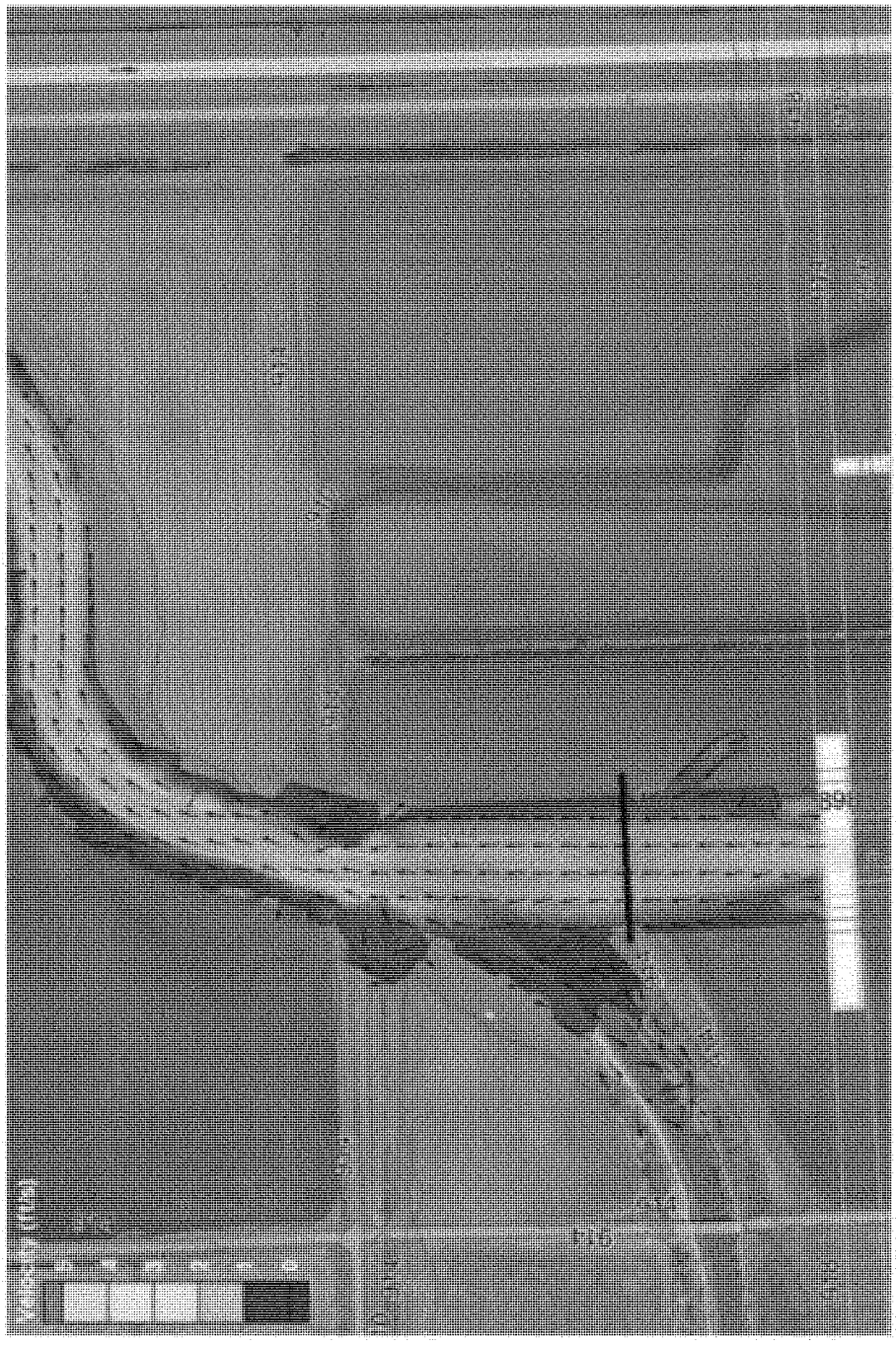

Figure F-H35 Velocity Field for Wild Rice River Post-construction Conditions Downstream 20-Year Event Flow

Figure F-H36 Velocity Field for Wild Rice River Post-construction Conditions Downstream Close Up 20-Year Event Flow

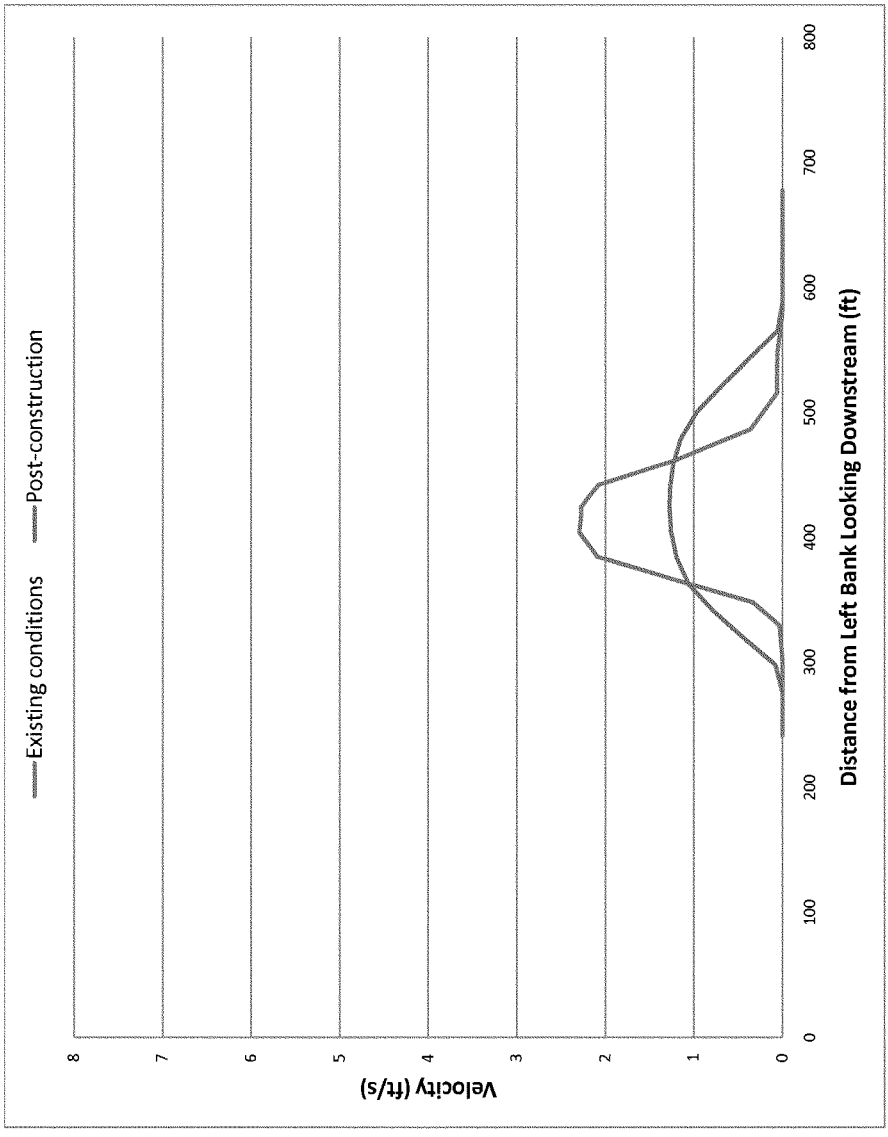

Figure F-H37 Cross Sectional Velocity Data for Wild Rice River Control Structure Upstream During 5-Year Event

Cross Sectional Velocity Data for Wild Rice Control Structure (5-Year) Compares velocity at existing conditions to post-construction upstream of structure.

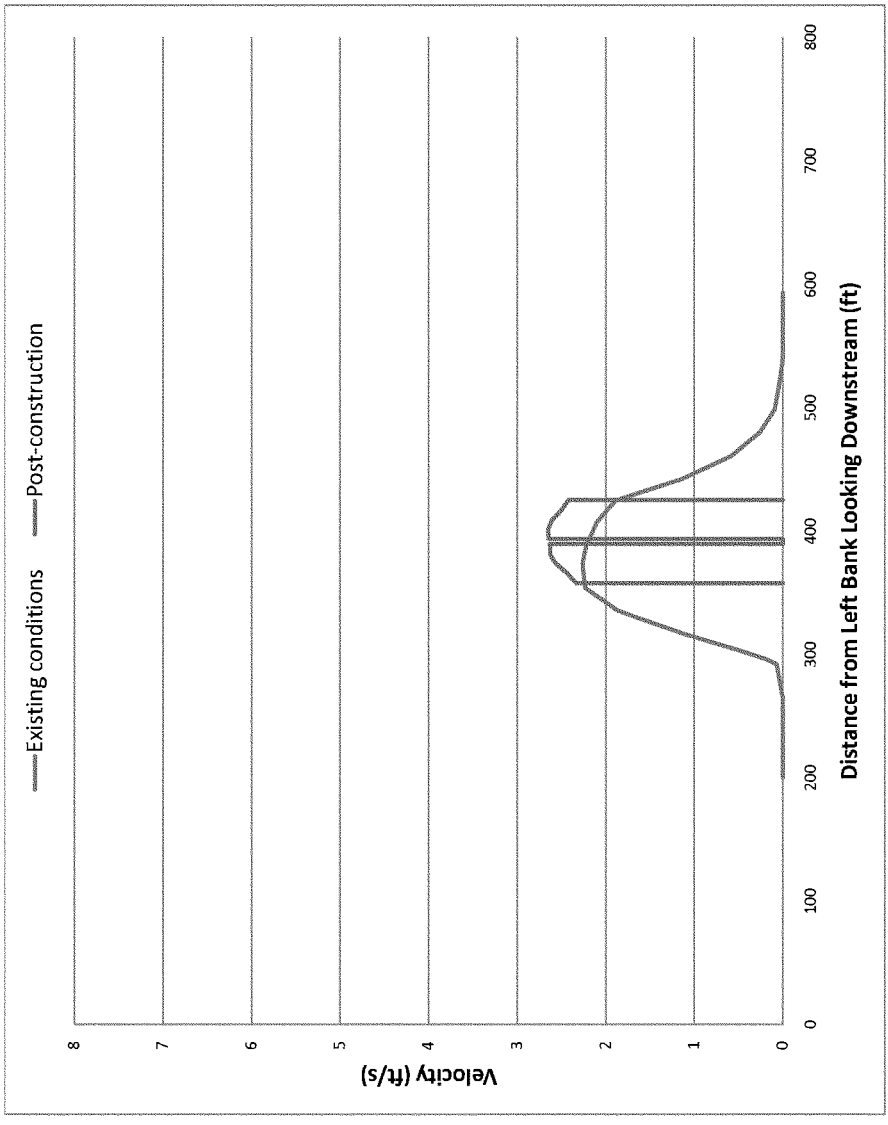

Figure F-H38 Cross Sectional Velocity Data for Wild Rice River Control Structure Through Structure During 5-Year Event

Cross Sectional Velocity Data for Wild Rice Control Structure (5-Year). Compares velocity at existing conditions to post-construction through structure.

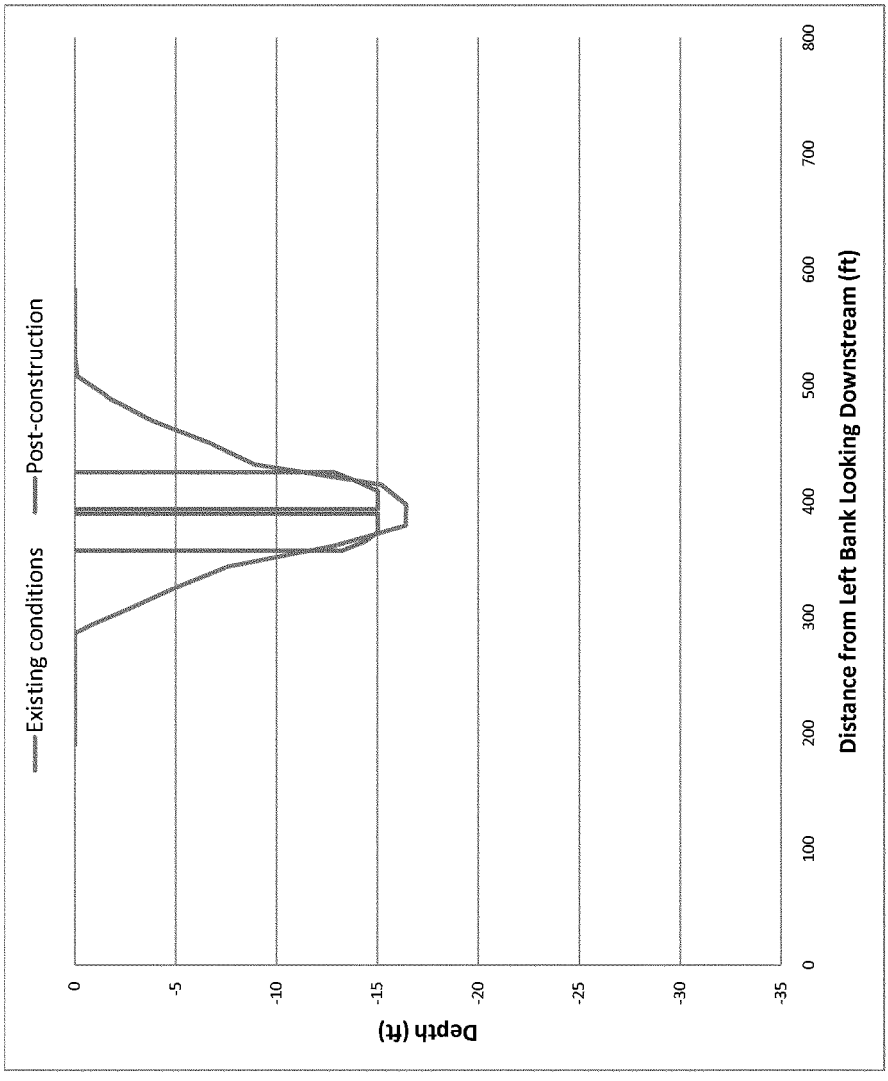

Figure F-H39 Cross Sectional Depth Data for Wild Rice River Control Structure Downstream During 5-Year Event

Cross Sectional Depth Data for Wild Rice Control Structure (5-Year). Compares depth at existing conditions to post-construction downstream of structure.

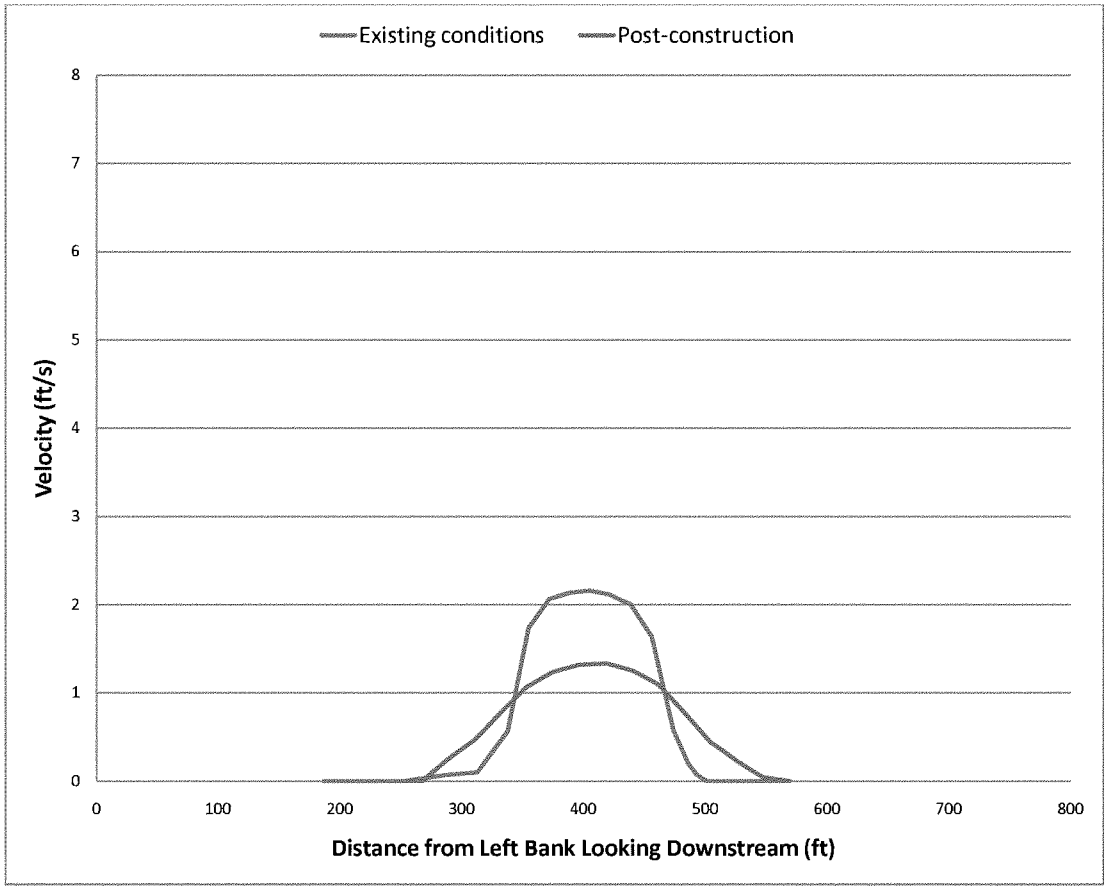

Figure F-H40 Cross Sectional Velocity Data for Wild Rice River Control Structure Downstream During 5-Year Event

Cross Sectional Velocity Data for Wild Rice Control Structure (5-Year). Compares velocity at existing conditions to post-construction downstream of structure.

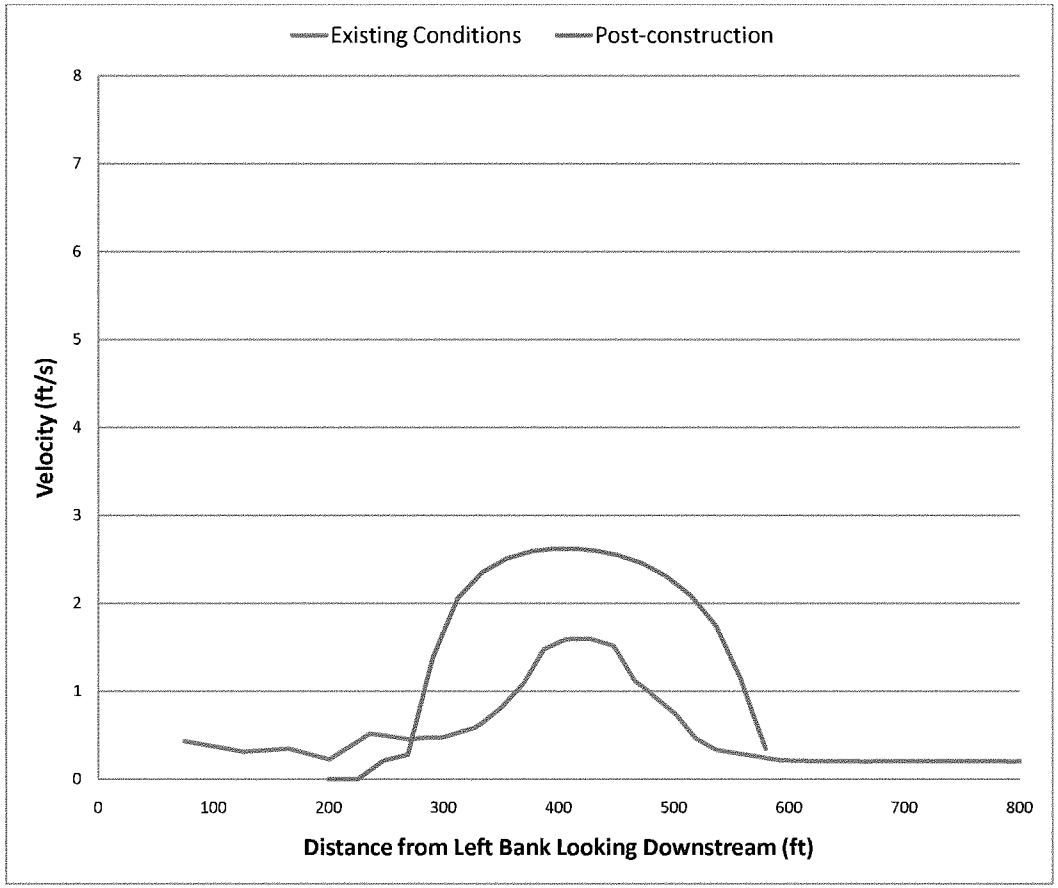

Figure F-H41 Cross Sectional Velocity Data for Wild Rice River Control Structure Upstream During 20-Year Event

Cross Sectional Velocity Data for Wild Rice Control Structure (20-Year). Compares velocity at existing conditions to post-construction upstream of structure.

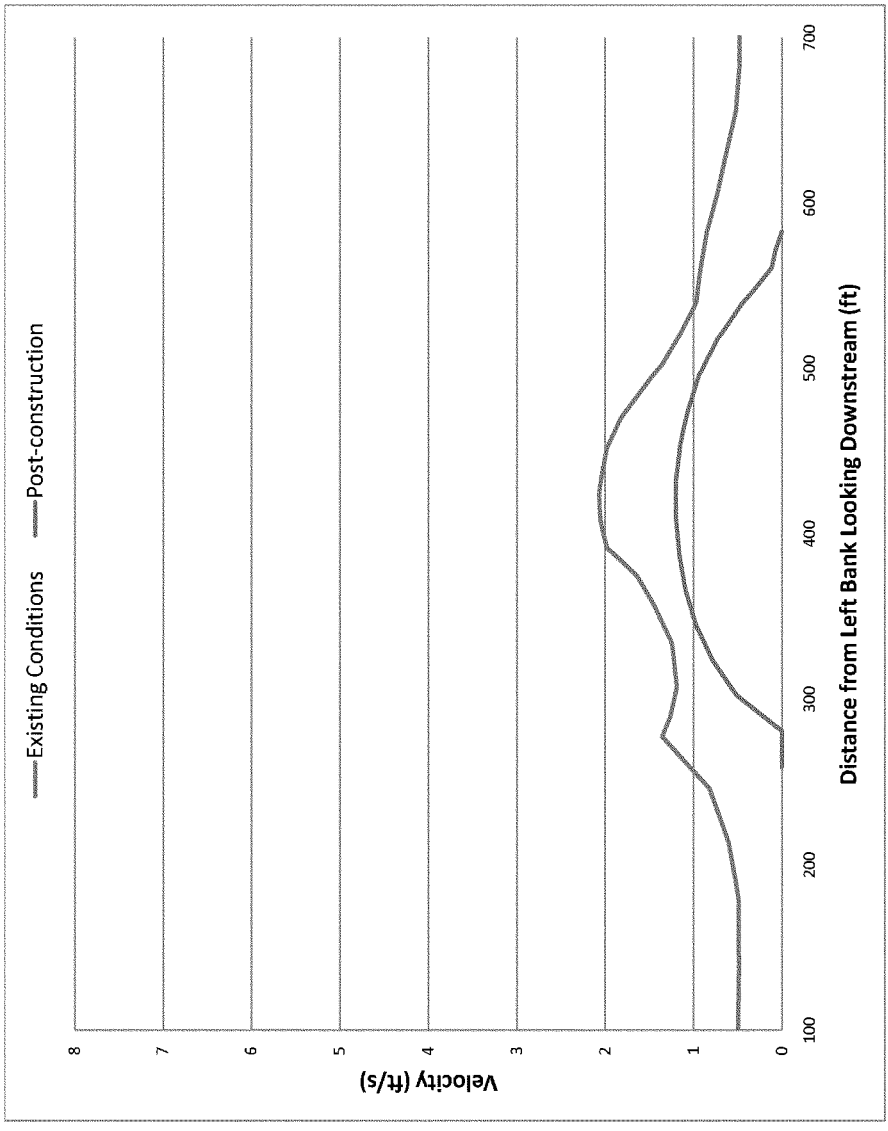

Figure F-H42 Cross Sectional Velocity Data for Wild Rice River Control Structure Downstream During 20-Year Event

Cross Sectional Velocity Data for Wild Rice Control Structure (20-Year). Compares velocity at existing conditions to post-construction downstream of structure.

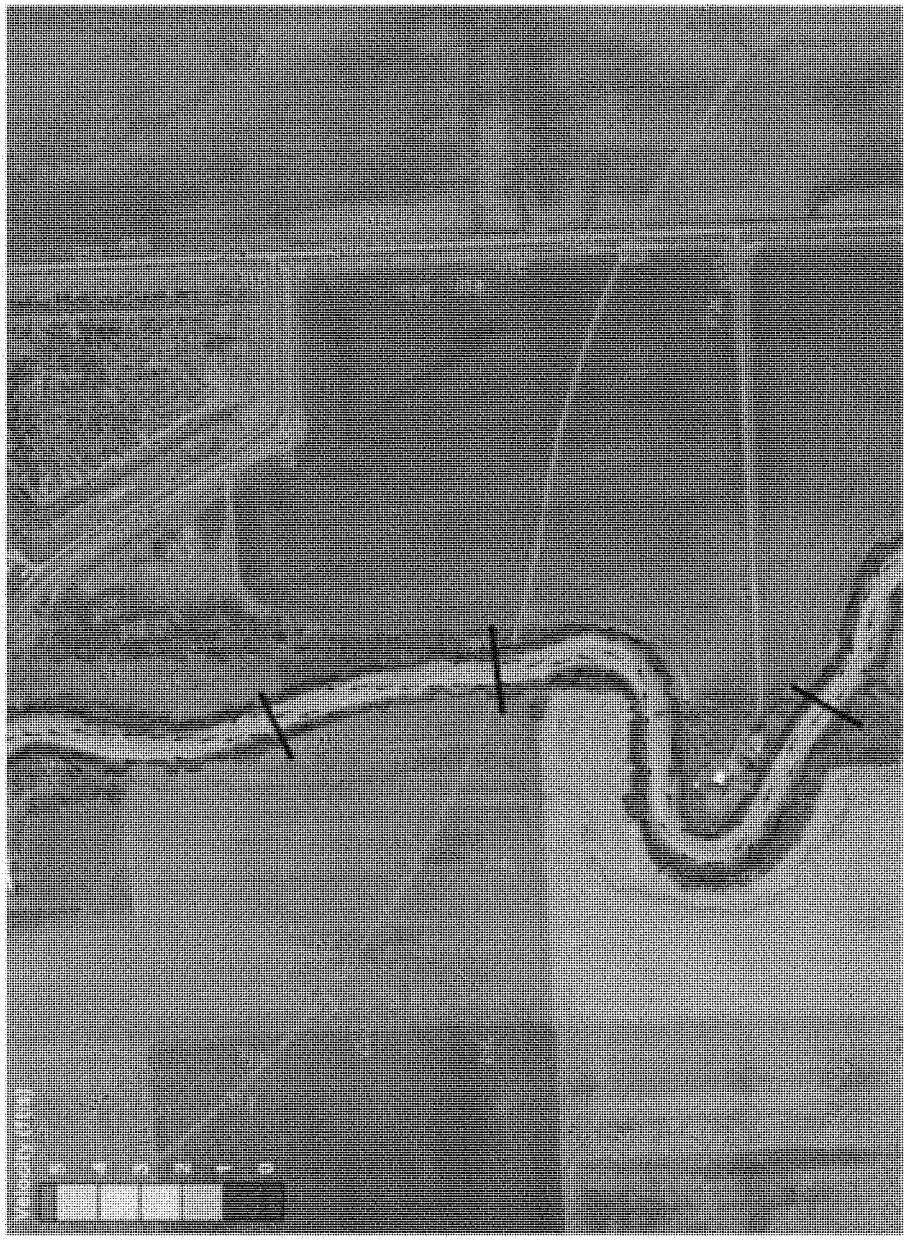

Figure F-H43 Velocity Field for Sheyenne River Existing Conditions 2-Year Event Flow

Figure F-H44 Velocity Field for Sheyenne River Existing Conditions 50-Year Event Flow

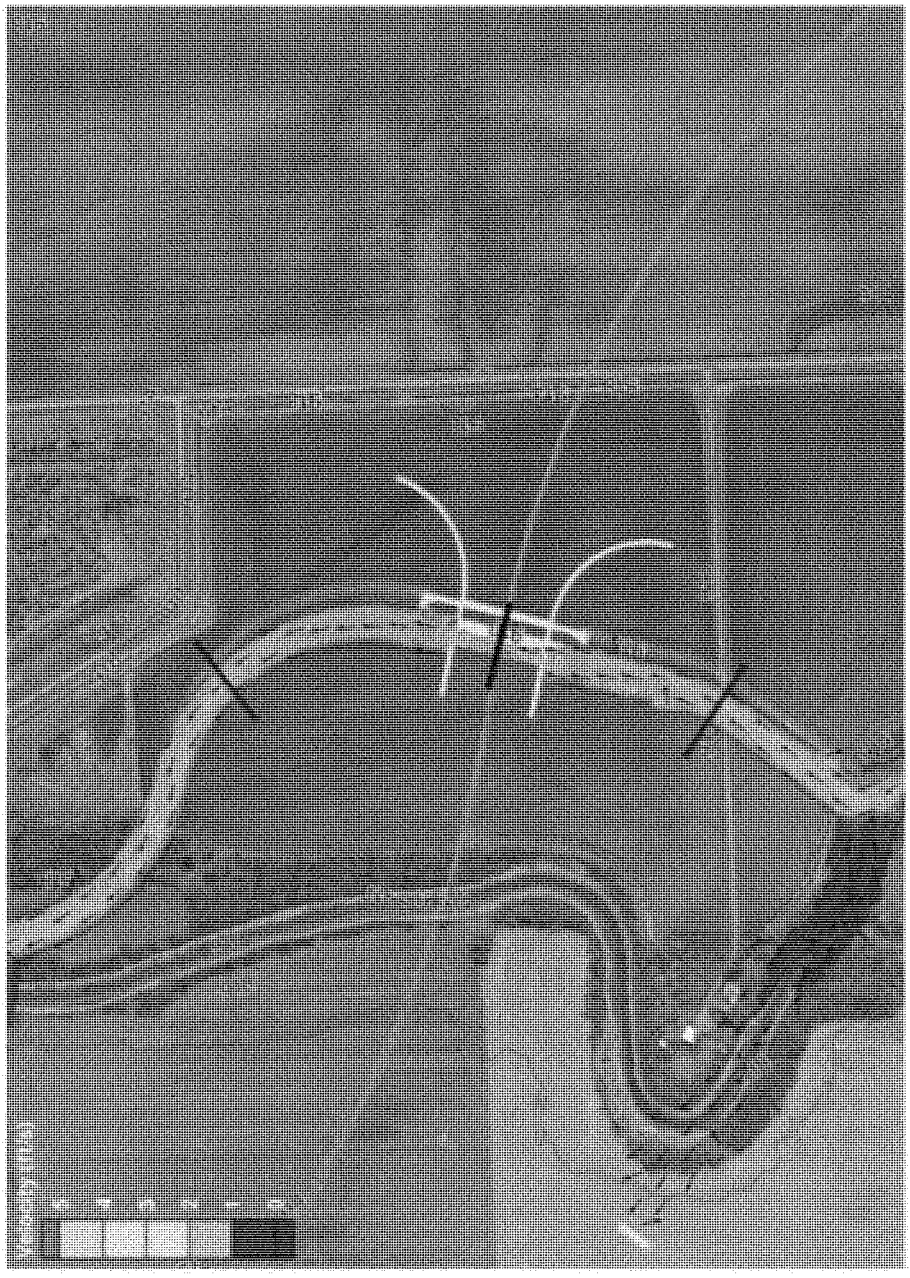

Figure F-H45 Velocity Field for Sheyenne River Aqueduct Post-construction Conditions 2-Year Event Flow

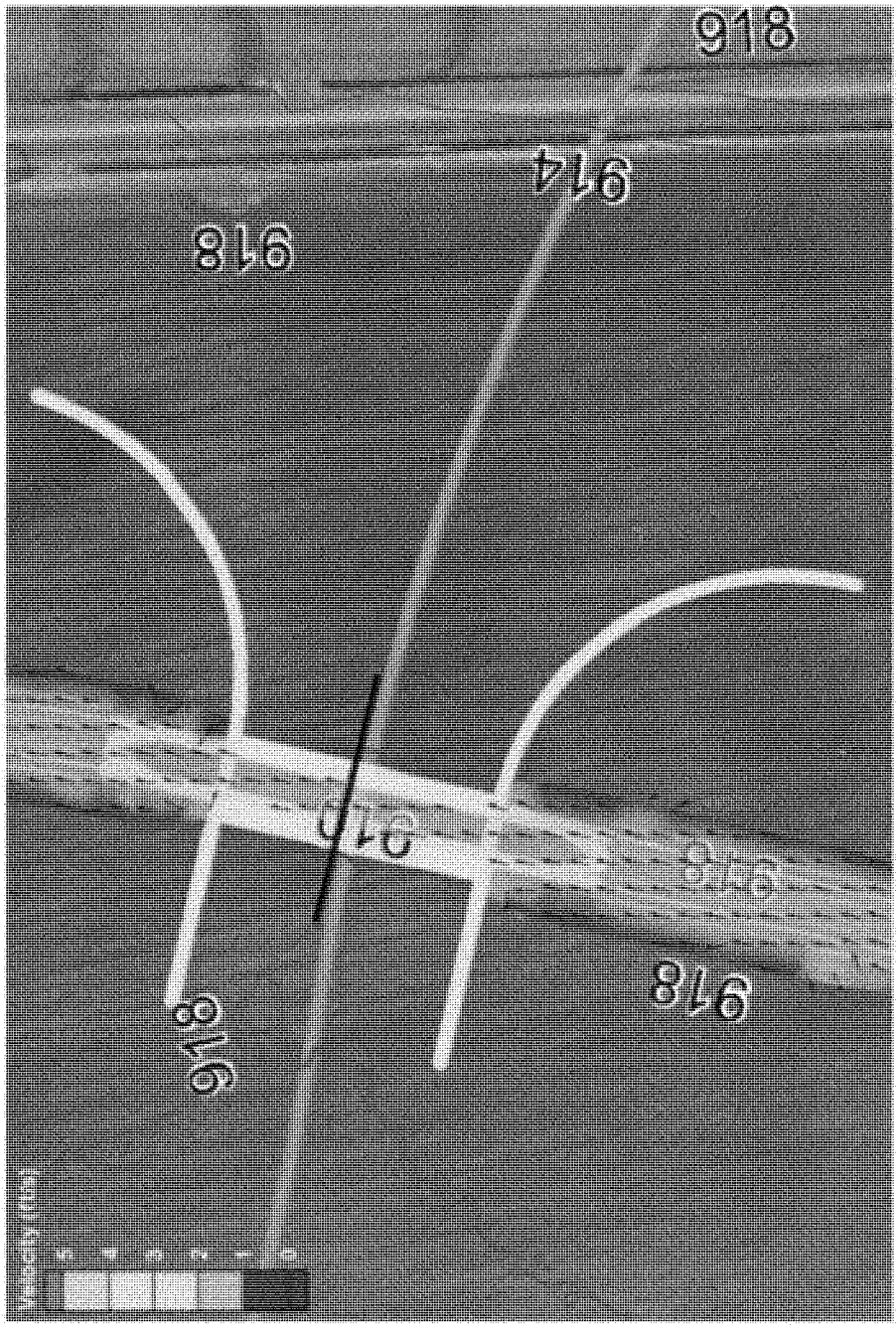

Figure F-H46 Velocity Field for Sheyenne River Aqueduct Post-construction Conditions Close Up 2-Year Event Flow

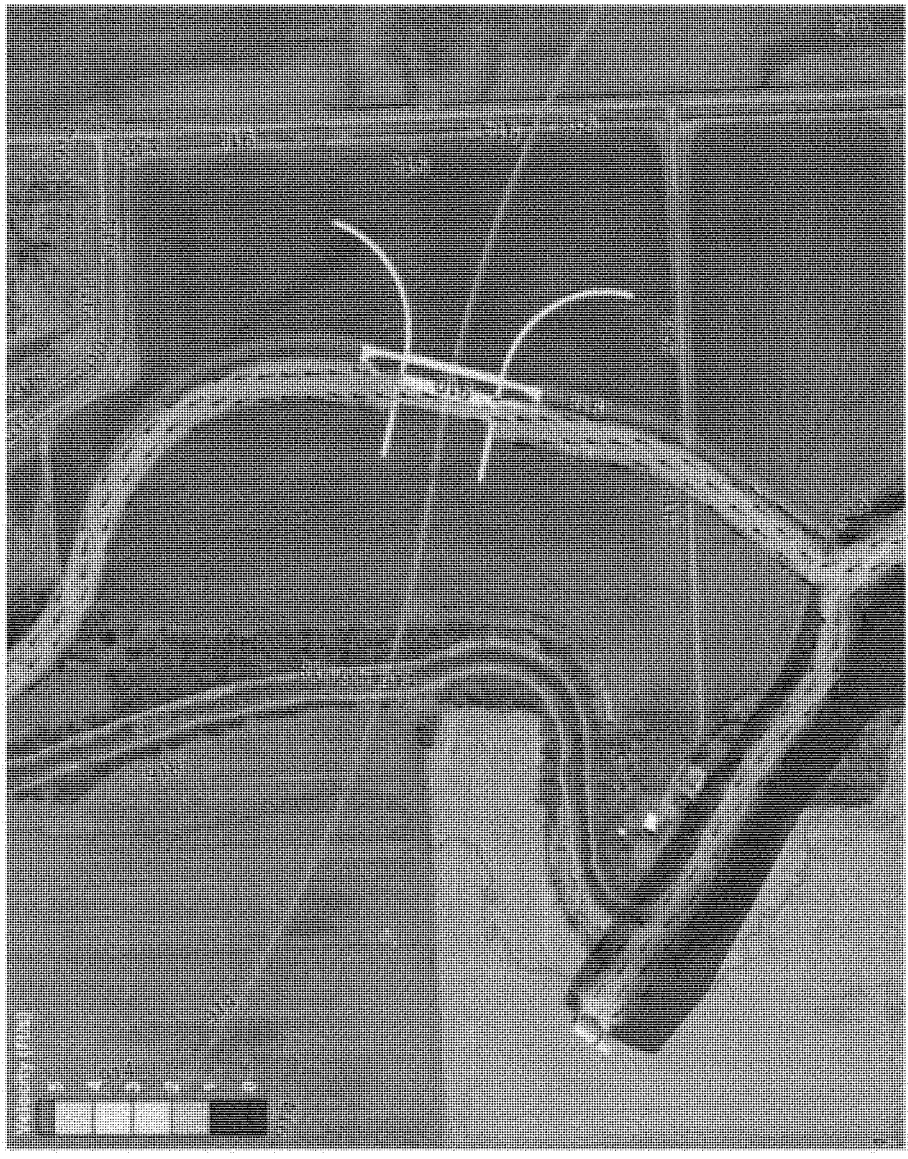

Figure F-H47 Velocity Field for Sheyenne River Aqueduct Post-construction Conditions 10-Year Event Flow

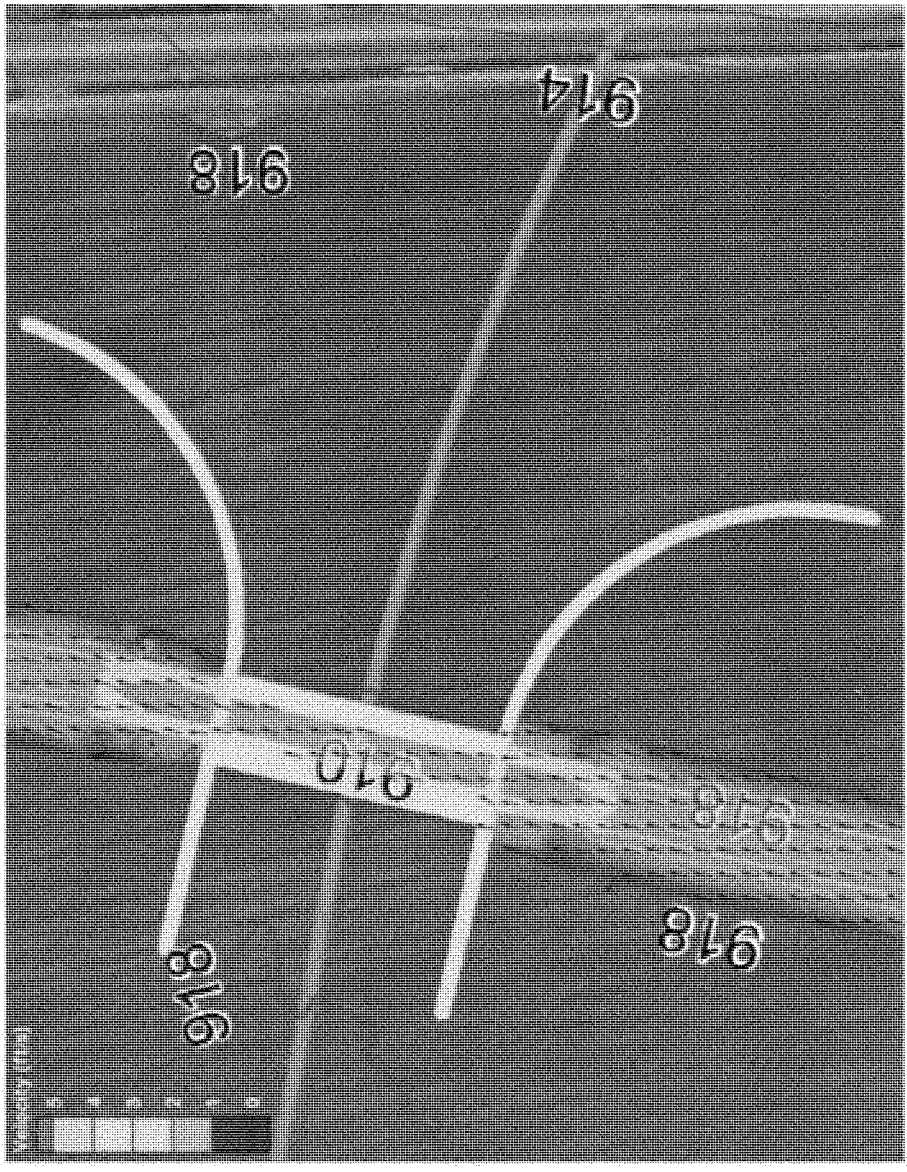

Figure F-H48 Velocity Field for Sheyenne River Aqueduct Post-construction Conditions Close Up 10-Year Event Flow

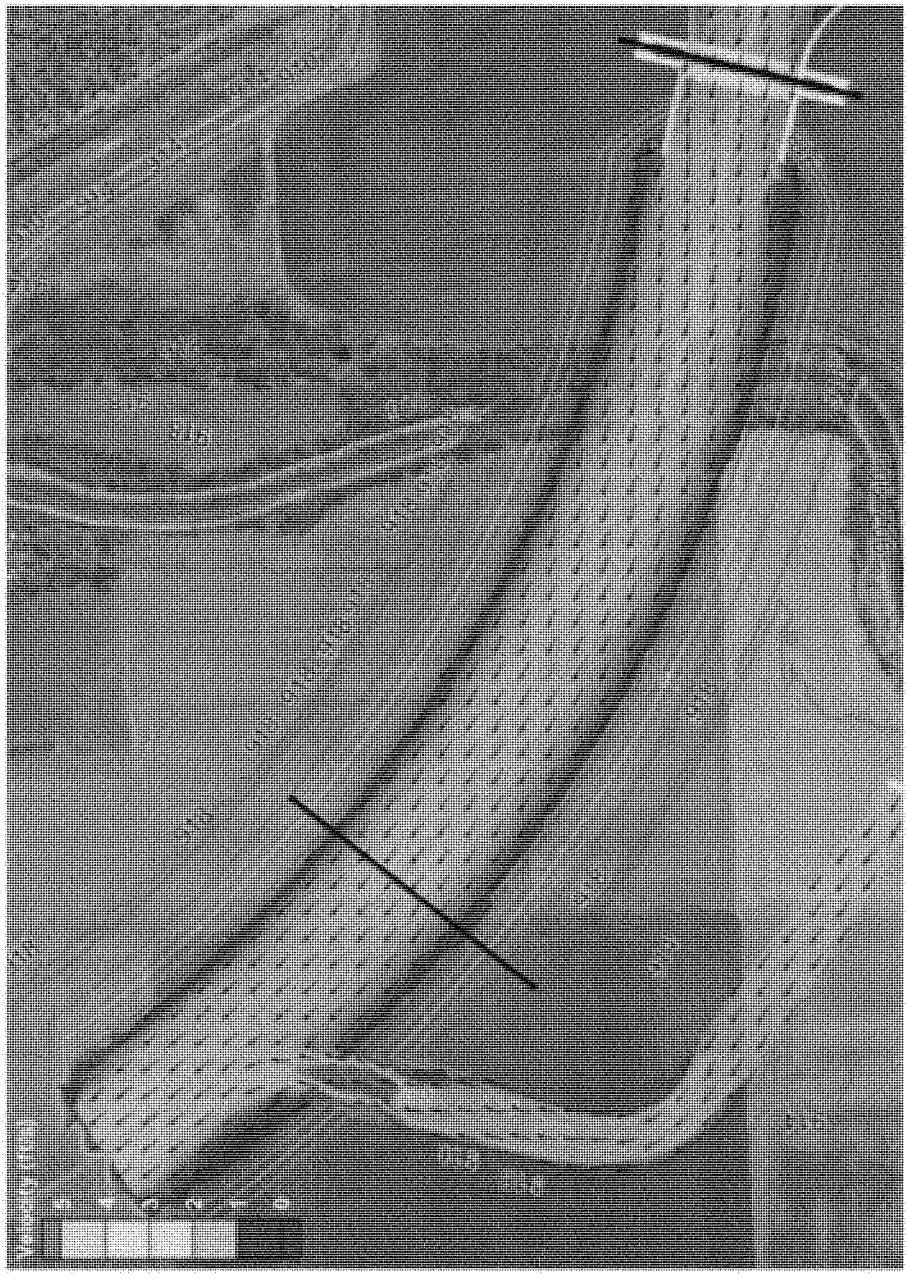

Figure F-H49 Velocity Field for Sheyenne River Diversion Channel Post-construction Conditions 5-Year Event Flow

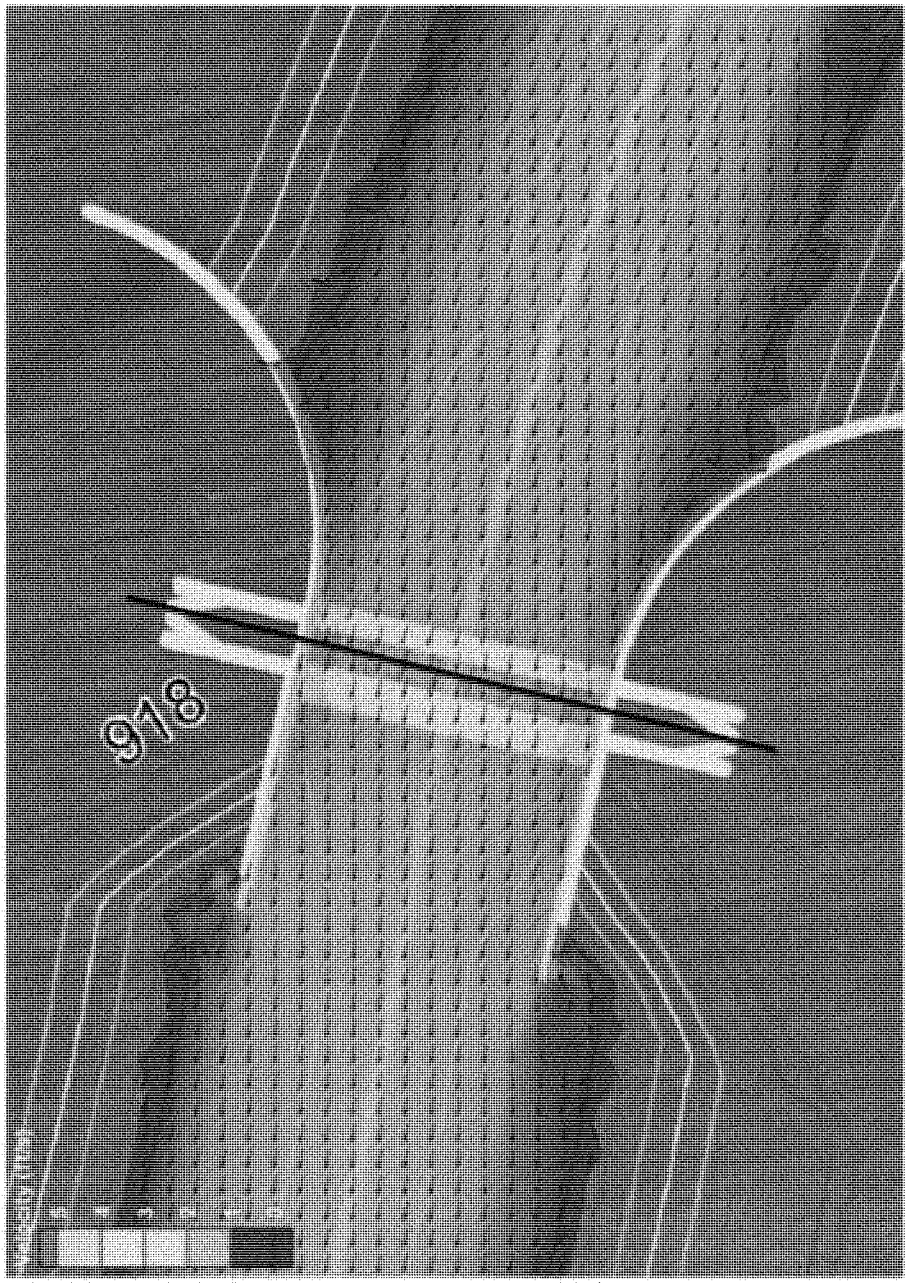

Figure F-H50 Velocity Field for Sheyenne River Diversion Channel and Aqueduct Crossing Post-construction Conditions Close Up 5-Year Event Flow

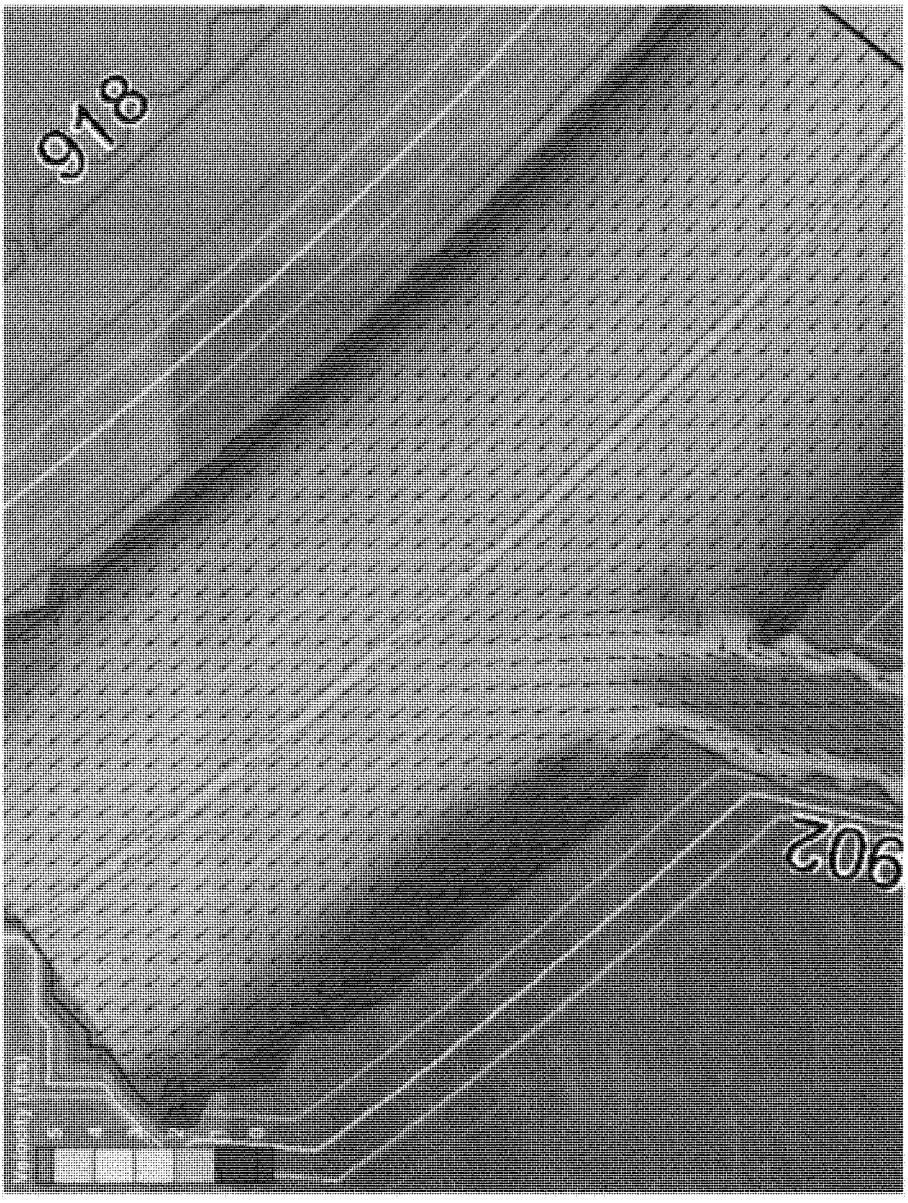

Figure F-H51 Velocity Field for Sheyenne River Diversion Channel and Spillway Post-construction Conditions Close Up 5-Year Event Flow

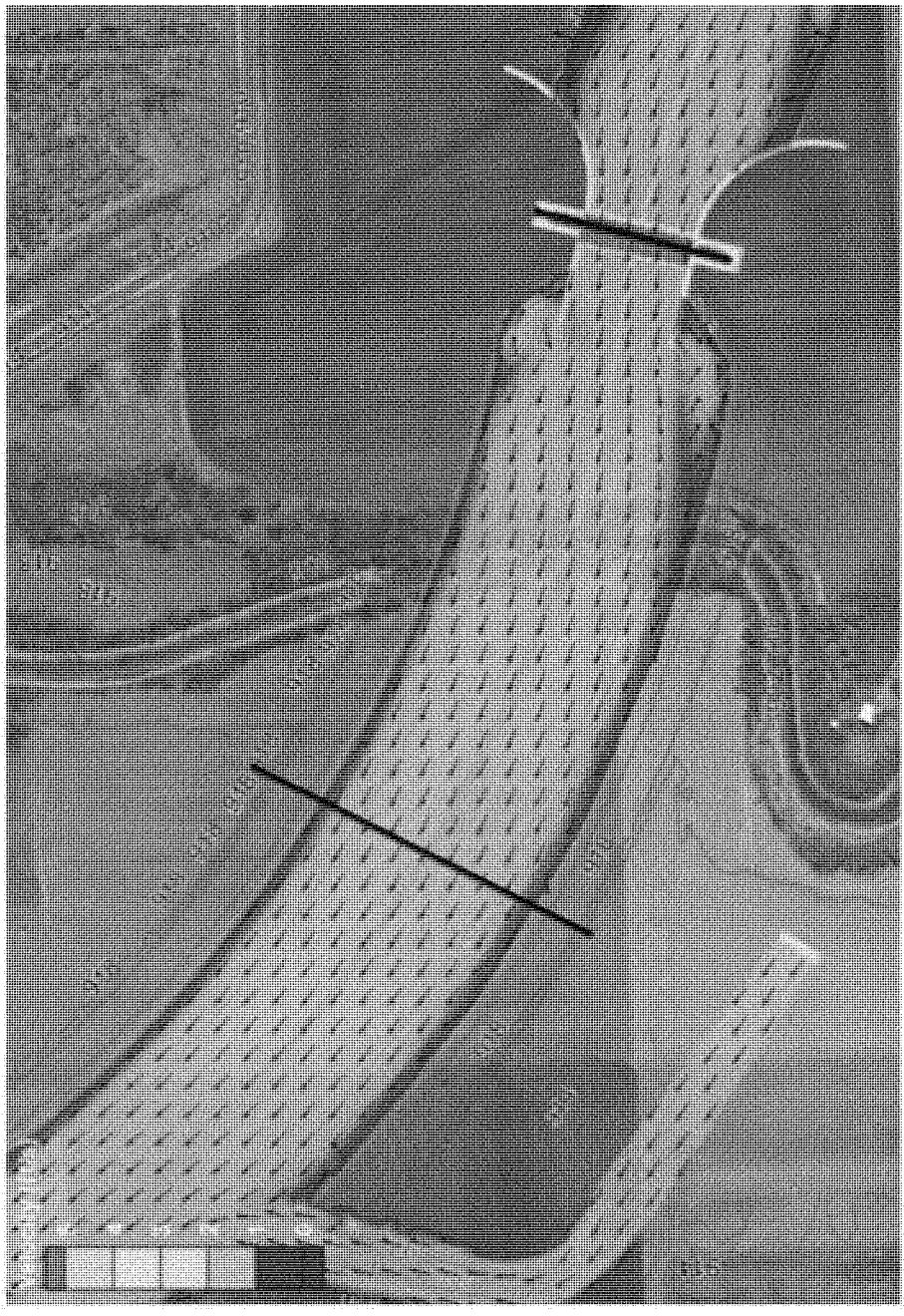

Figure F-H52 Velocity Field for Sheyenne River Diversion Channel Post-construction Conditions 50-Year Event Flow

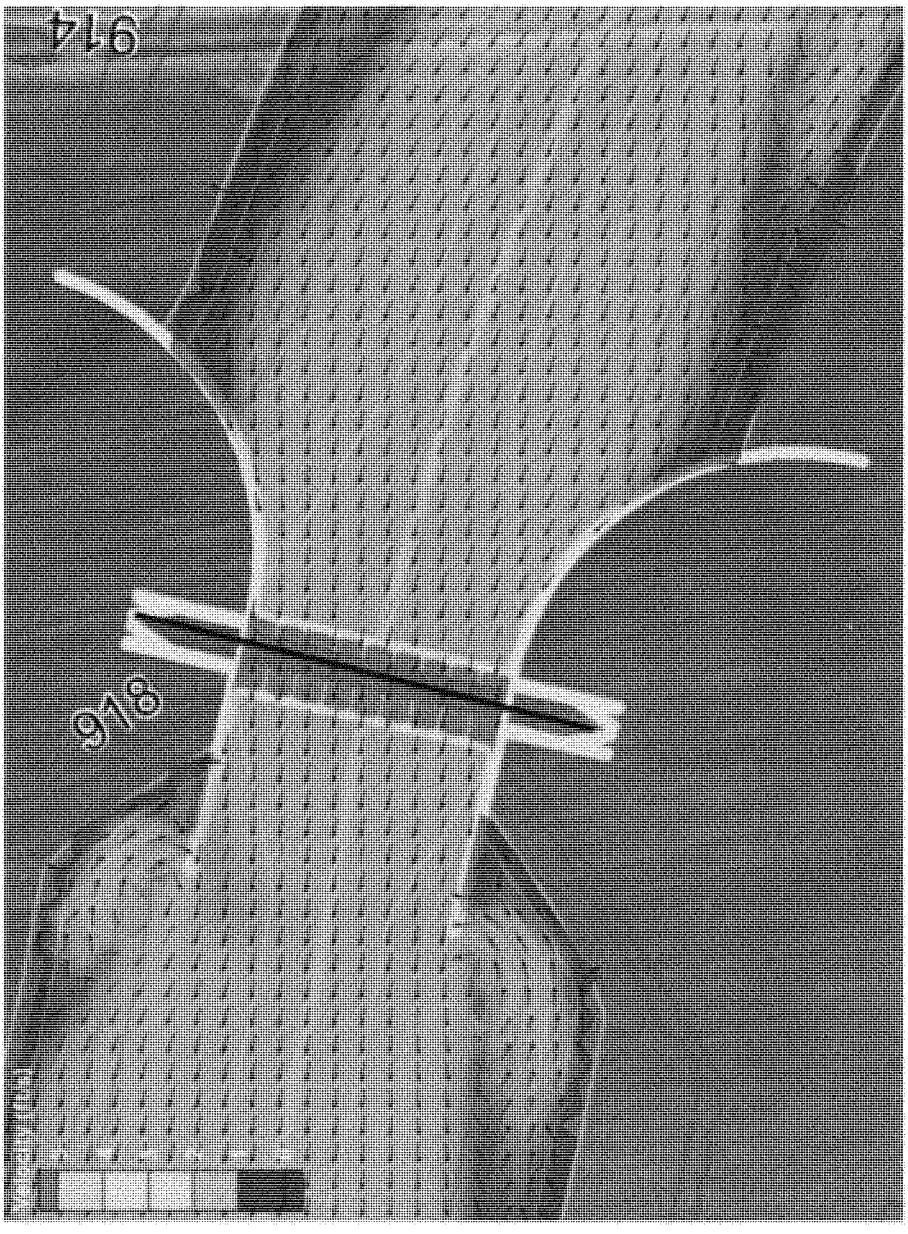

Figure F-H53 Velocity Field for Sheyenne River Diversion Channel and Aqueduct Crossing Post-construction Conditions Close Up 50-Year Event Flow

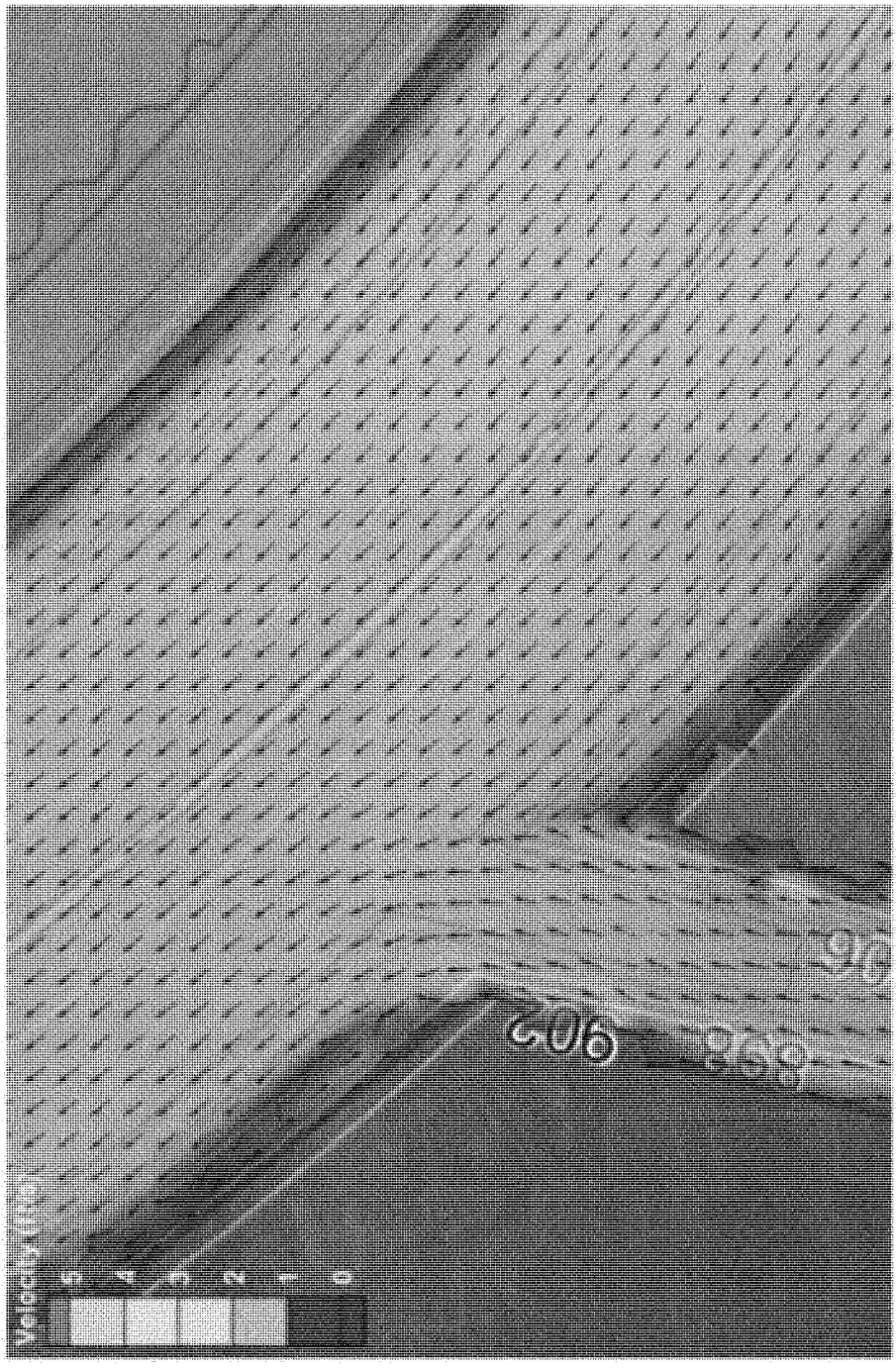

Figure F-H54 Velocity Field for Sheyenne River Diversion Channel and Spillway Post-construction Conditions Close Up 50-Year Event Flow

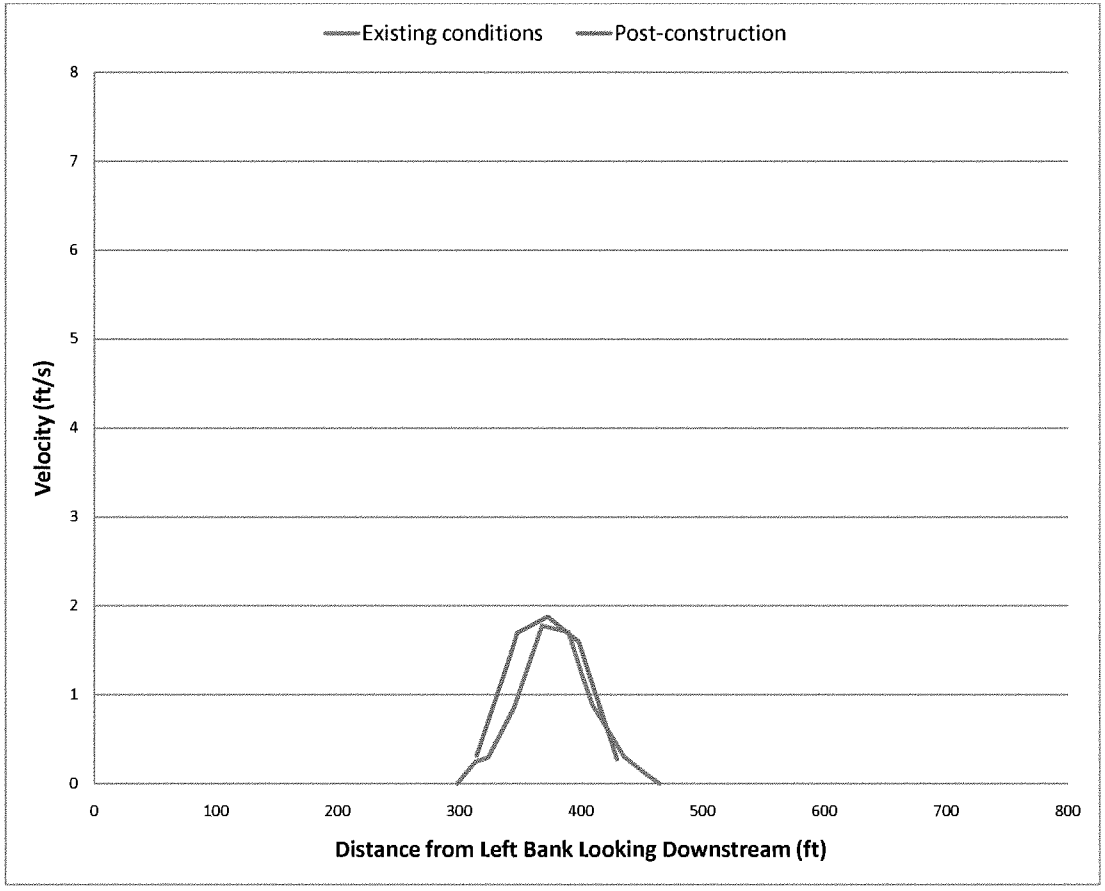

Figure F-H55 Cross Sectional Velocity Data for Sheyenne River Aqueduct Upstream During 2-Year Event

Cross Sectional Velocity Data for Sheyenne River Aqueduct (2-Year). Compares velocity at existing conditions to post-construction upstream of structure.

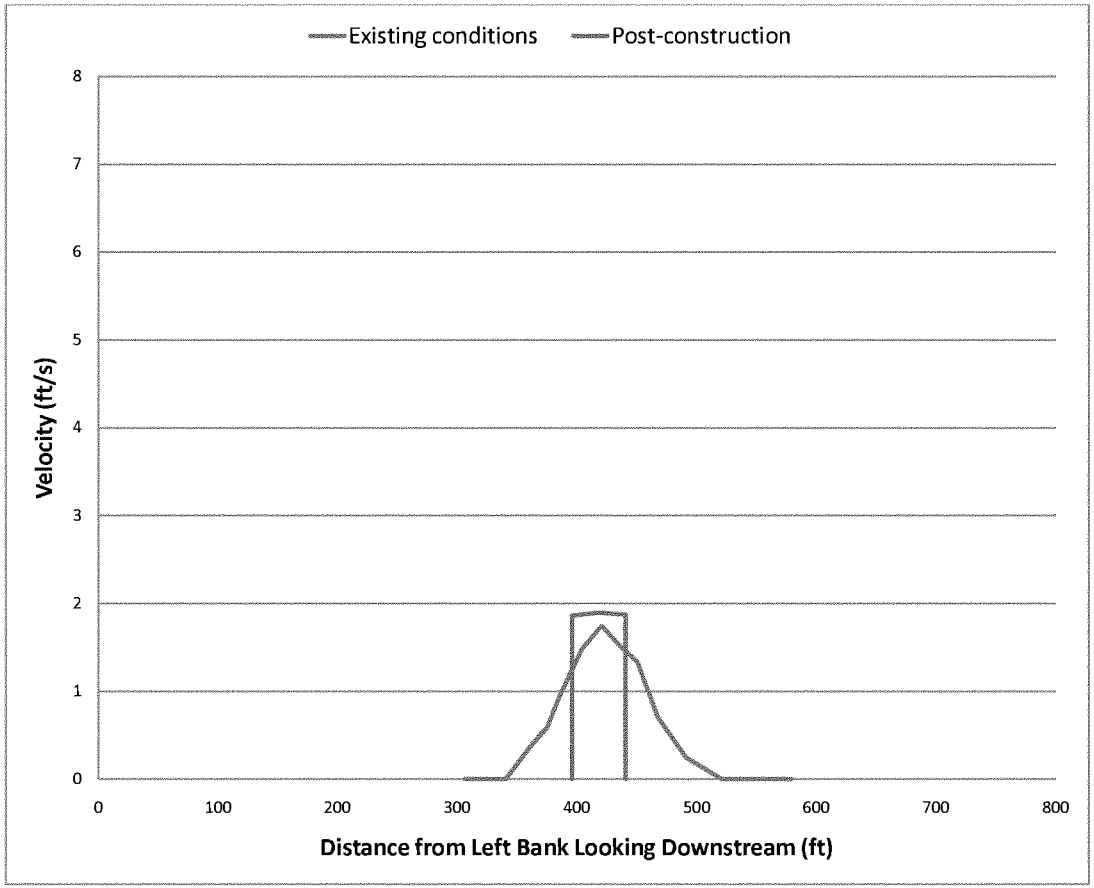

Figure F-H56 Cross Sectional Velocity Data for Sheyenne River Aqueduct Through Structure During 2-Year Event

Cross Sectional Velocity Data for Sheyenne River Aqueduct (2-Year). Compares velocity at existing conditions to post-construction through structure.

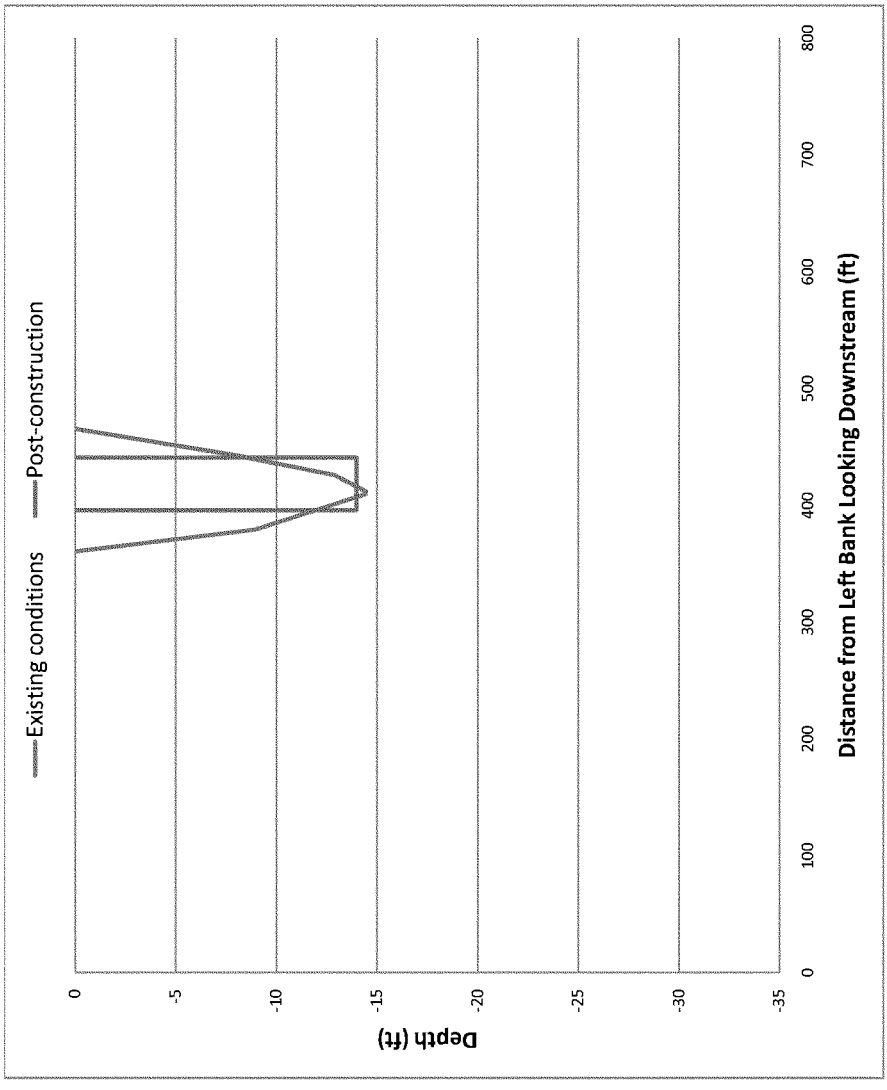

Figure F-H57 Cross Sectional Depth Data for Sheyenne River Aqueduct Through Structure During 2-Year Event

Cross Sectional Depth Data for Sheyenne River Aqueduct (2-Year). Compares depth at existing conditions to post-construction through structure.

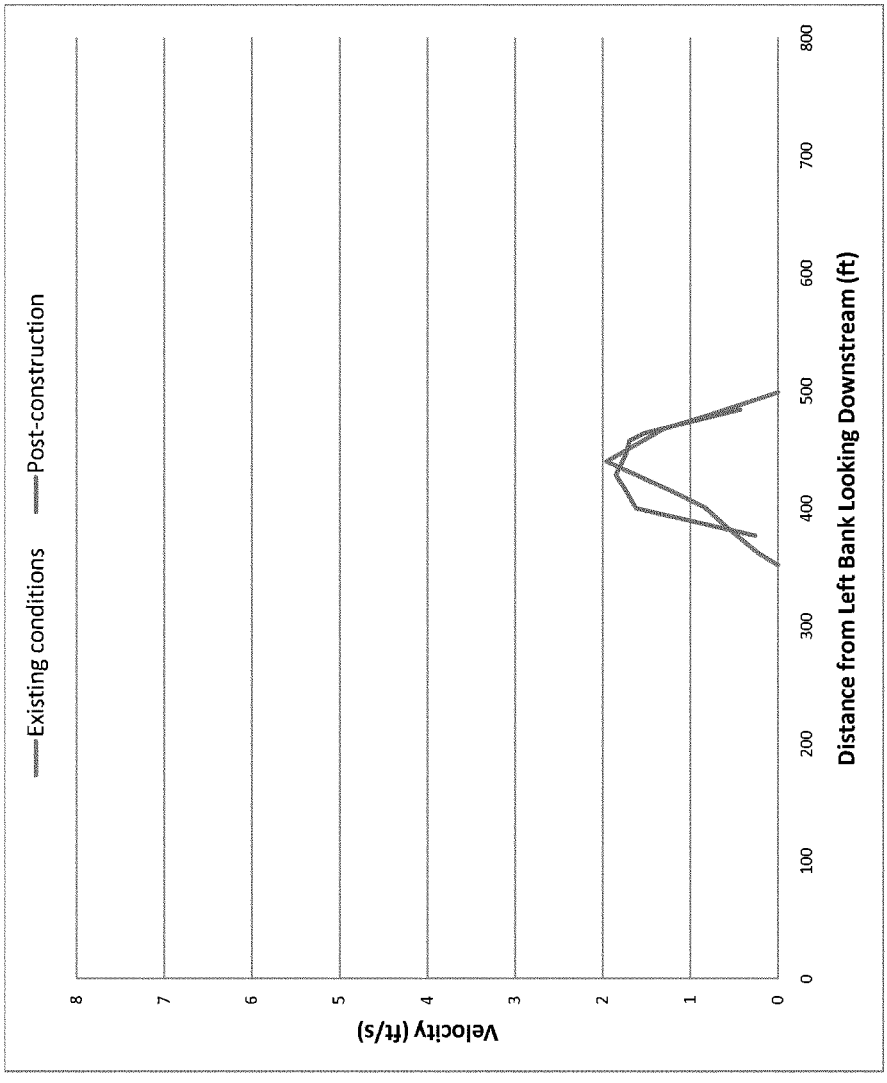

Figure F-H58 Cross Sectional Velocity Data for Sheyenne River Aqueduct Downstream During 2-Year Event

Cross Sectional Velocity Data for Sheyenne River Aqueduct (2-Year). Compares velocity at existing conditions to post-construction downstream of structure.

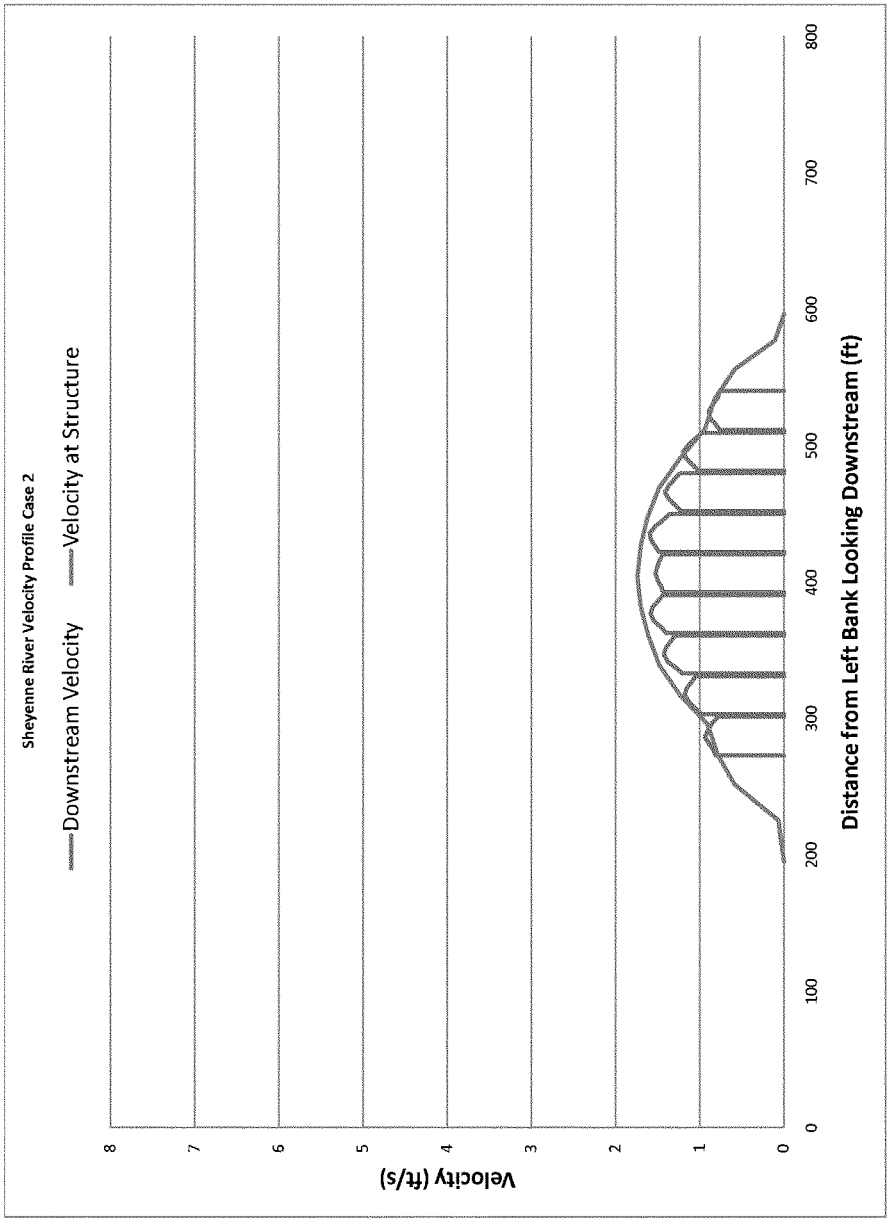

Figure F-H59 Cross Sectional Velocity Data for Sheyenne River Diversion Channel Through Structure During 5-Year Event
Cross Sectional Velocity Data for Sheyenne River Diversion Channel (5-Year). Compares velocity at structure to velocity downstream.

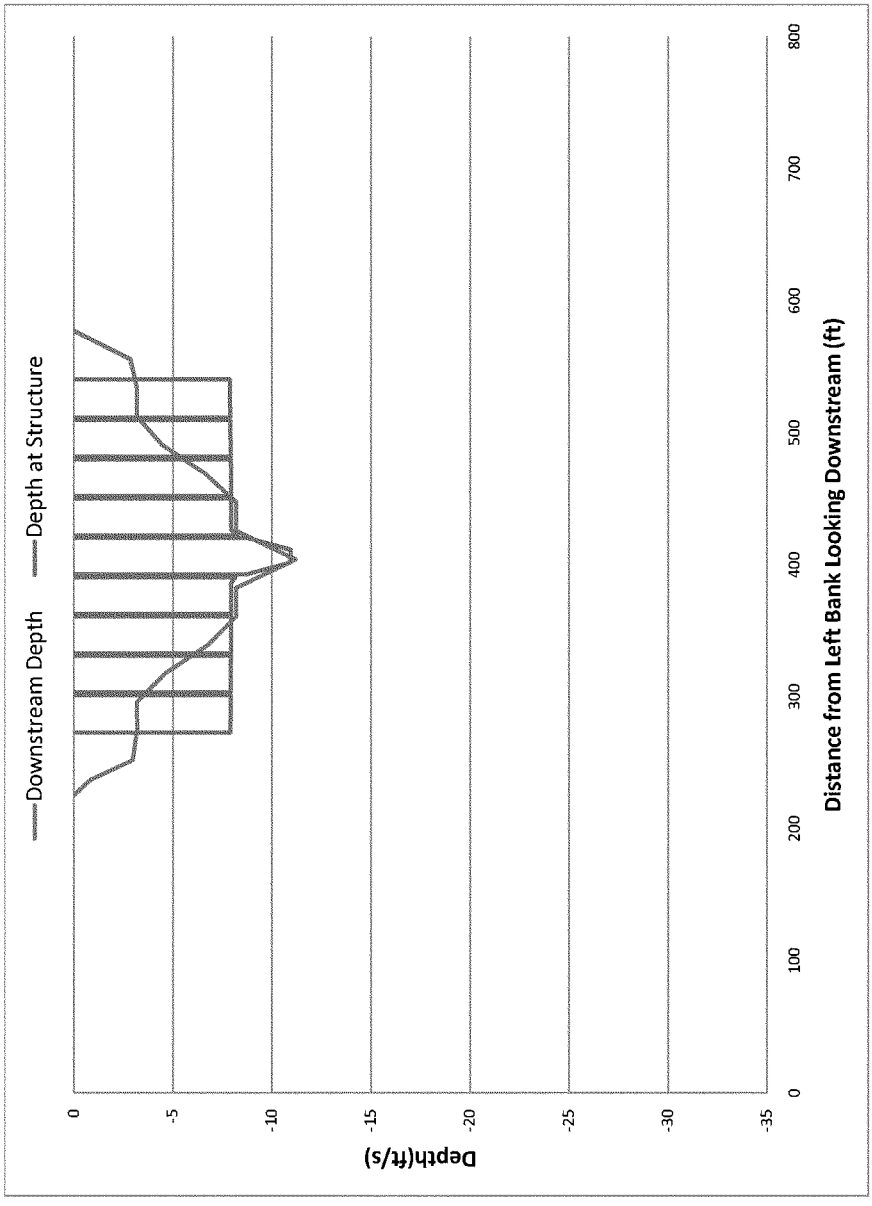

Figure F-H60 Cross Sectional Depth Data for Sheyenne River Diversion Channel Through Structure During 5-Year Event
Cross Sectional Depth Data for Sheyenne River Diversion Channel (5-Year). Compares depth at structure to depth downstream.

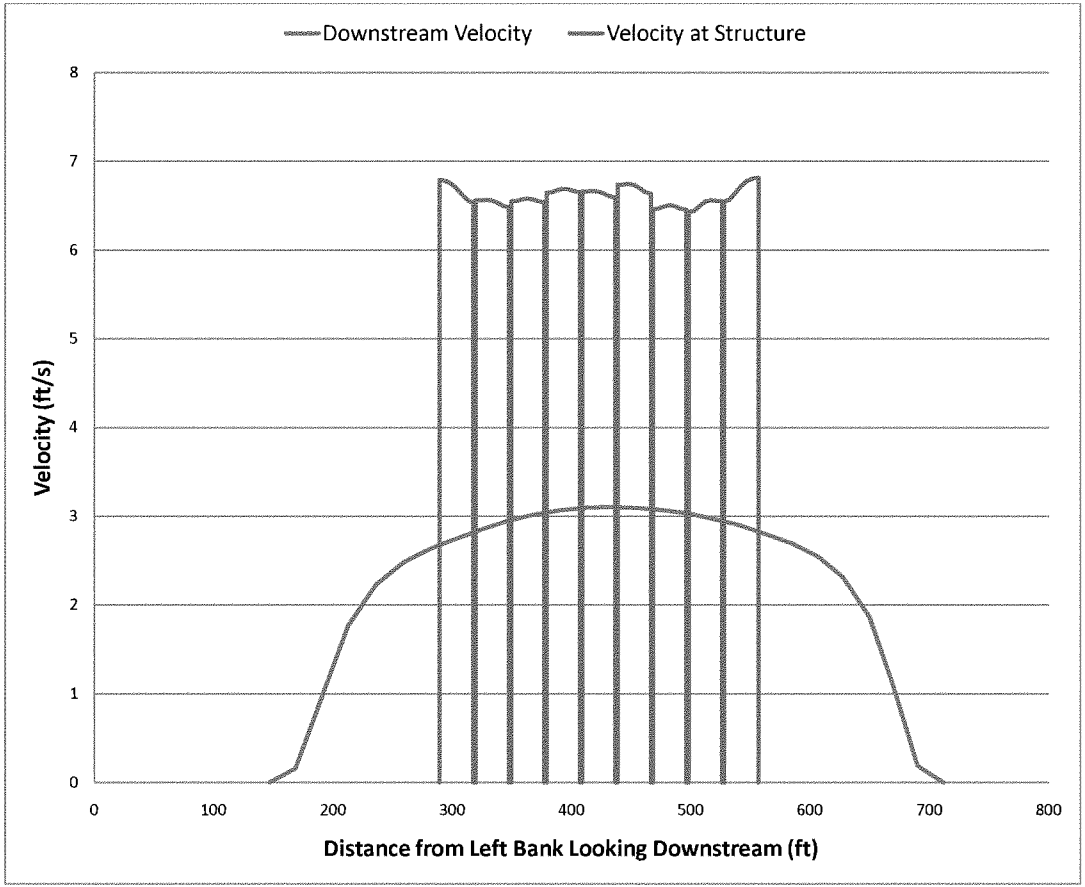

Figure F-H61 Cross Sectional Velocity Data for Sheyenne River Diversion Channel Through Structure During 50-Year Event

Cross Sectional Velocity Data for Sheyenne River Diversion Channel (50-Year). Compares velocity at structure to velocity downstream.

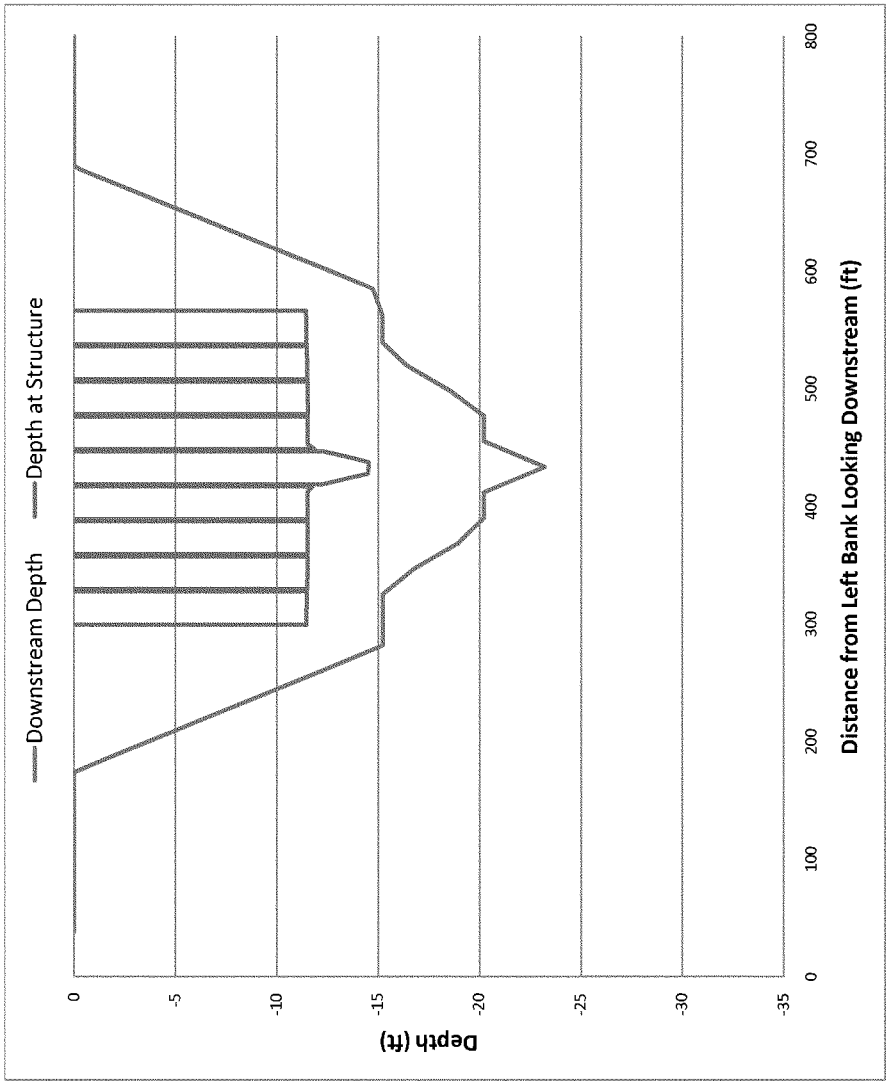

Figure F-H62 Cross Sectional Depth Data for Sheyenne River Diversion Channel Through Structure During 50-Year Event
Cross Sectional Depth Data for Sheyenne River Diversion Channel (50-Year). Compares depth at structure to depth downstream.

Figure F-H63 Velocity Field for Maple River Existing Conditions 2-Year Event Flow

Figure F-H64 Velocity Field for Maple River Existing Conditions 50-Year Event Flow

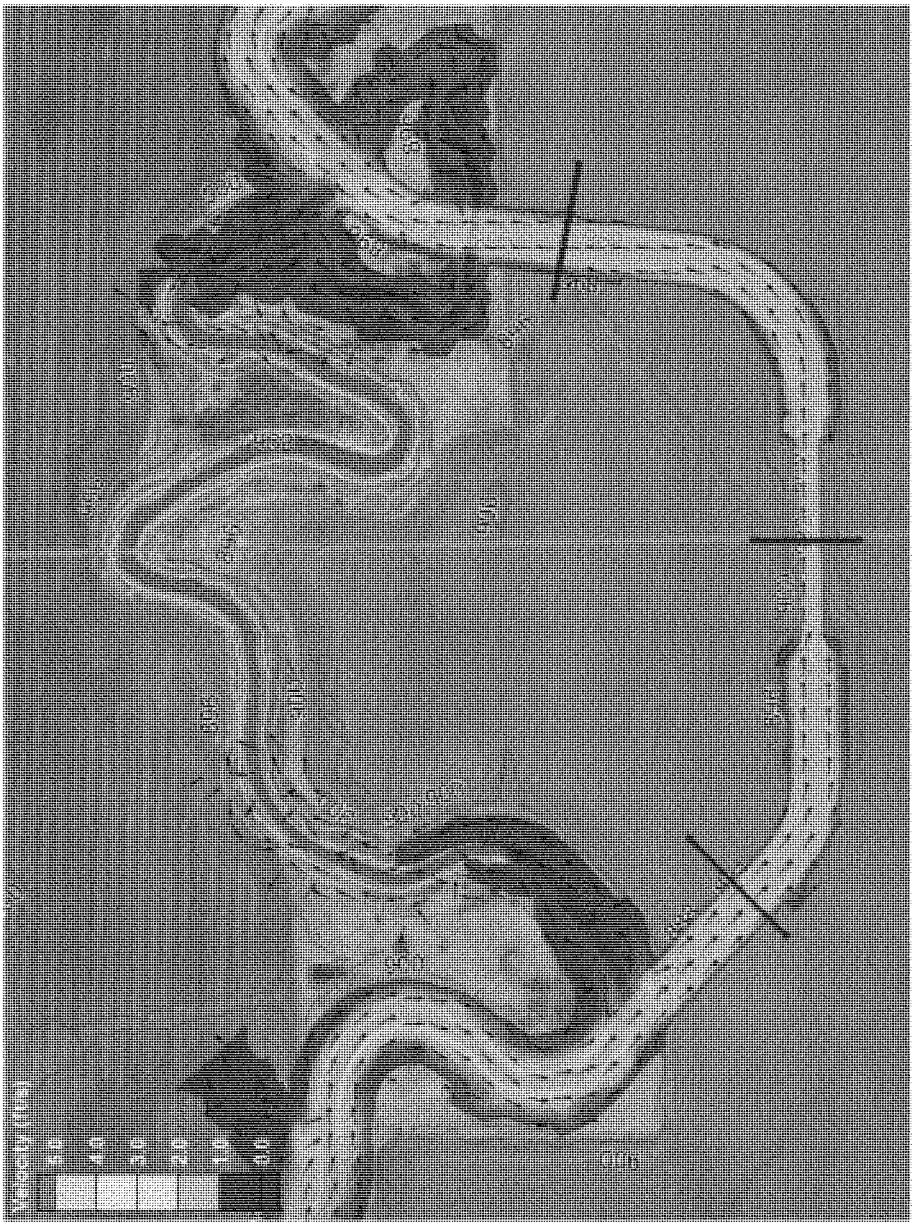

Figure F-H65 Velocity Field for Maple River Aqueduct Post-construction Conditions 2-Year Event Flow

Figure F-H66 Velocity Field for Maple River Aqueduct Post-construction Conditions Close Up 2-Year Event Flow

Figure F-H67 Velocity Field for Maple River Aqueduct Post-construction Conditions 5-Year Event Flow

Figure F-H68 Velocity Field for Maple River Aqueduct Post-construction Conditions Close Up 5-Year Event Flow

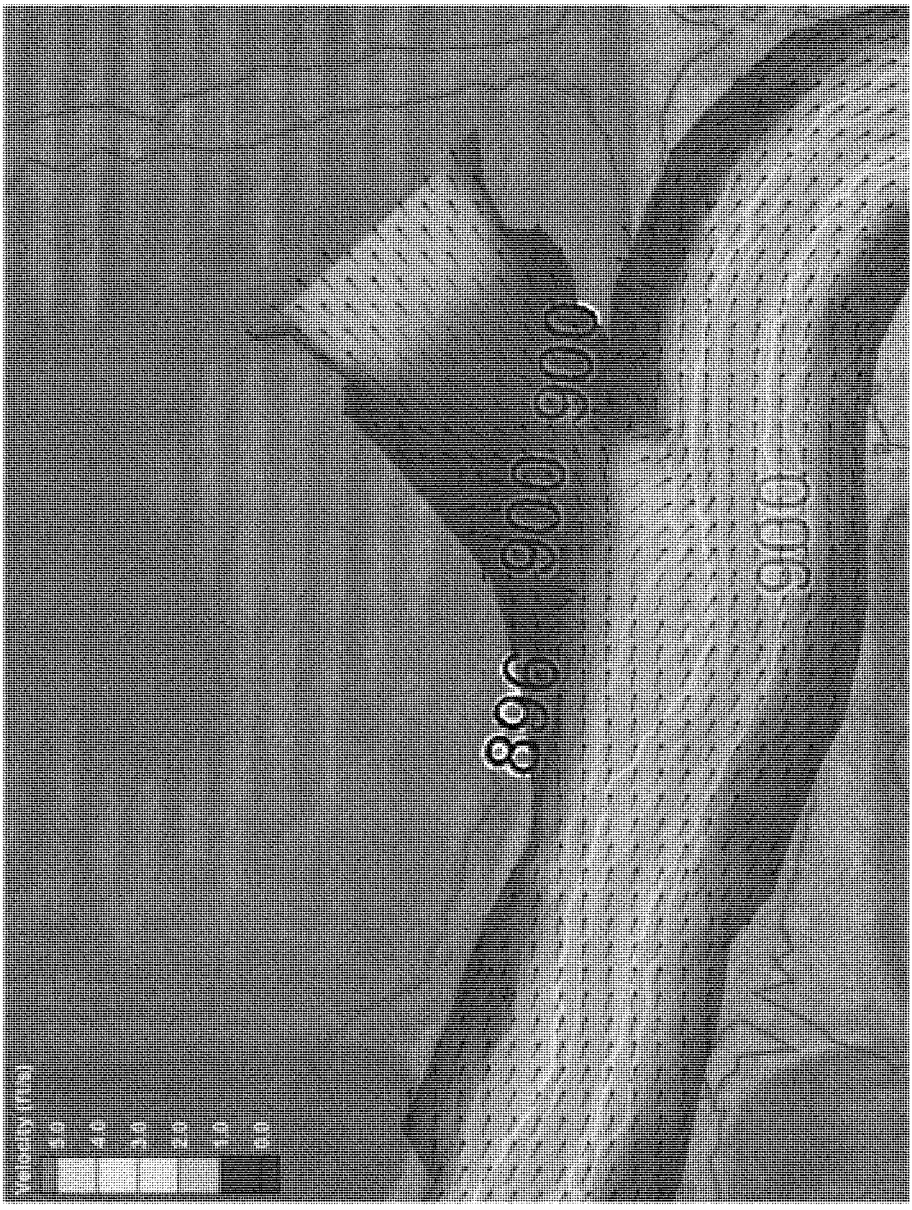

Figure F-H69 Velocity Field for Maple River Spillway Post-construction Conditions Close Up 5-Year Event Flow

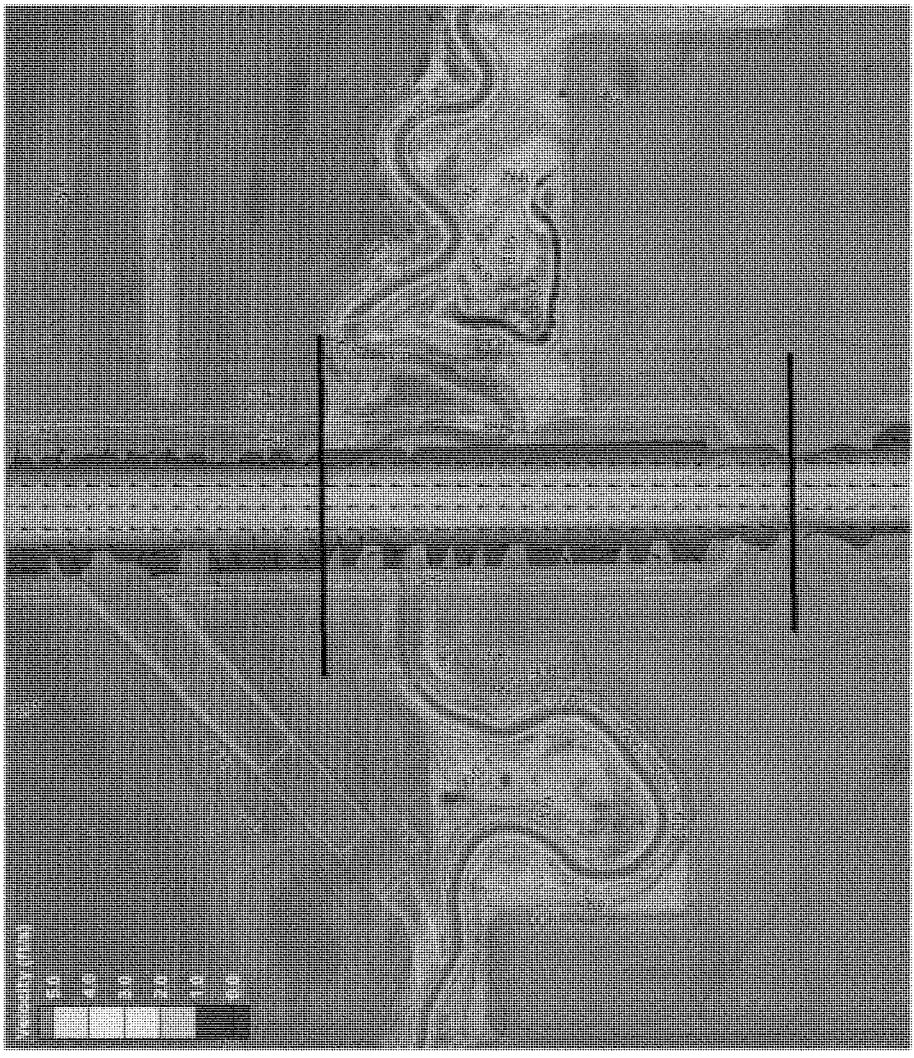

Figure F-H70 Velocity Field for Maple River Diversion Channel Post-construction Conditions 2,000 cfs Event Flow

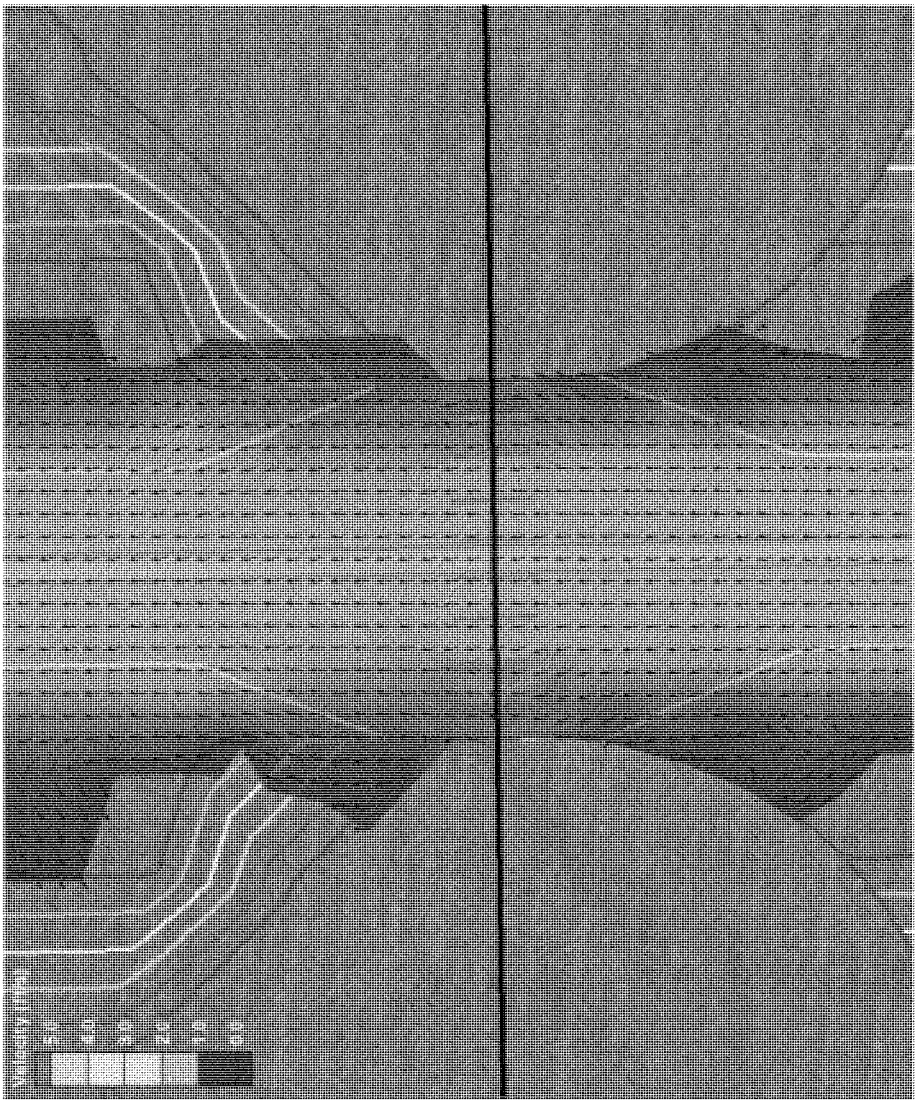

Figure F-HY1 Velocity Field for Maple River Diversion Channel Post-construction Conditions Close Up 2,000 cfs Event Flow

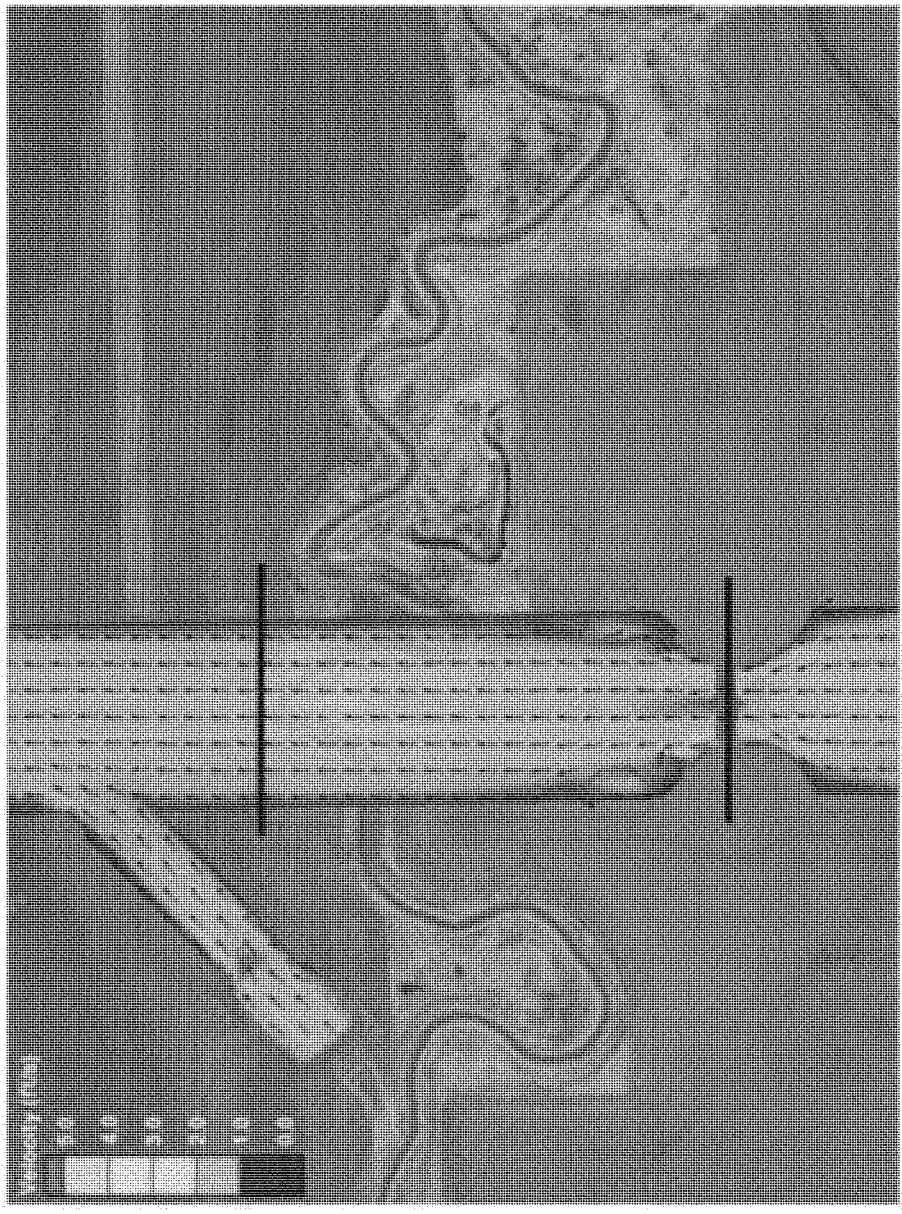

Figure F-H72 Velocity Field for Maple River Diversion Channel Post-construction Conditions 50-Year Event Flow

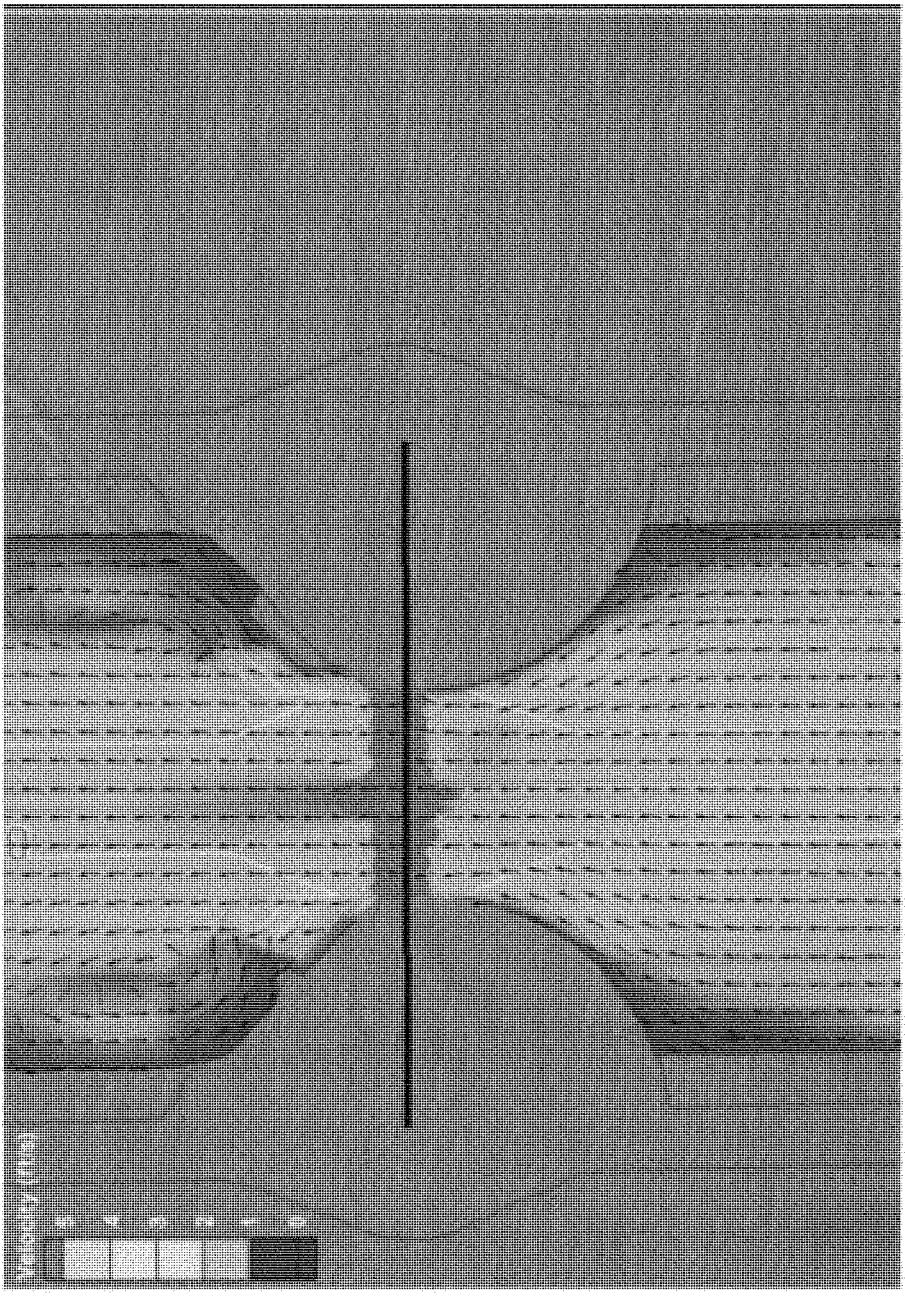

Figure F-H73 Velocity Field for Maple River Diversion Channel Crossing Post-construction Conditions Close Up 50-Year Event Flow

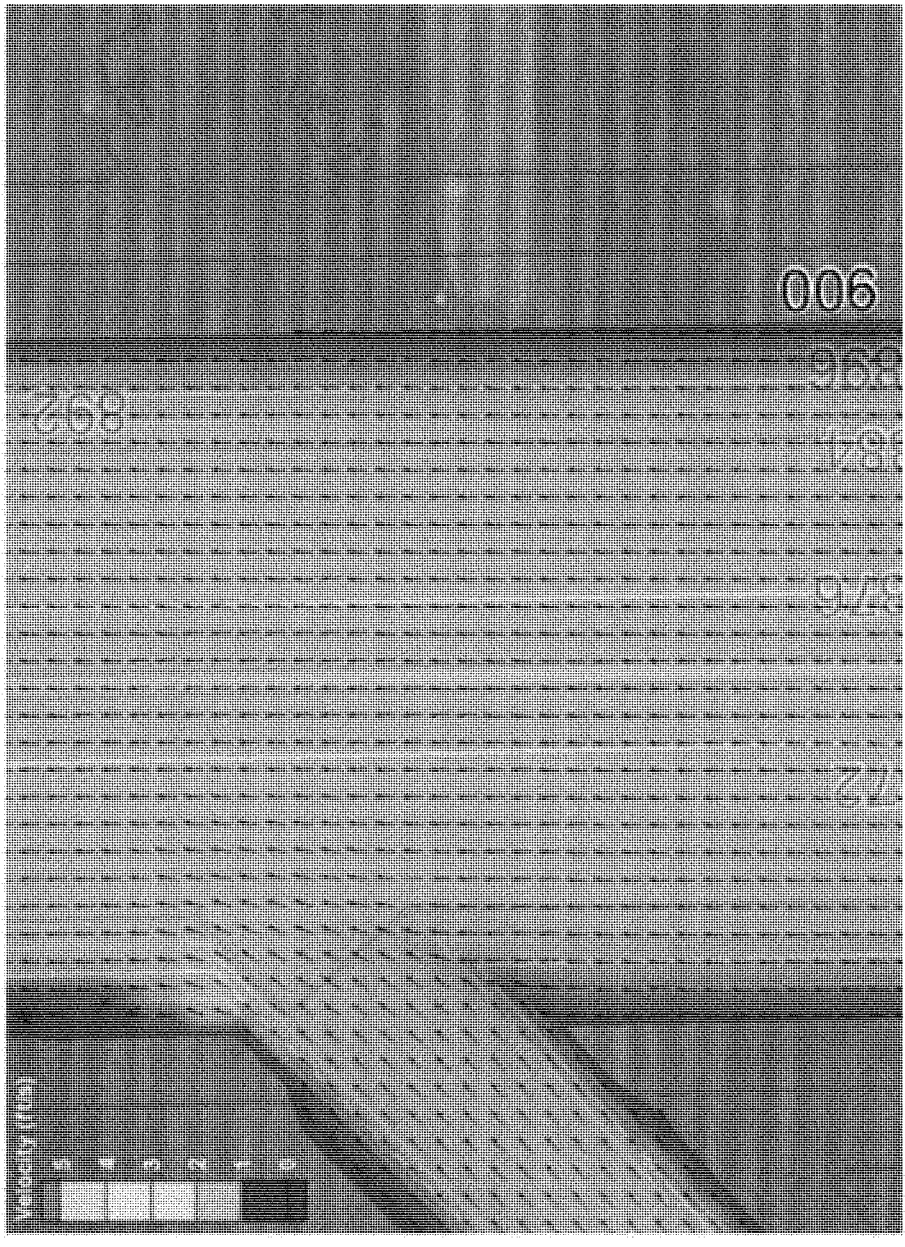

Figure F-H74 Velocity Field for Maple River Spillway Post-construction Conditions Close Up 50-Year Event Flow

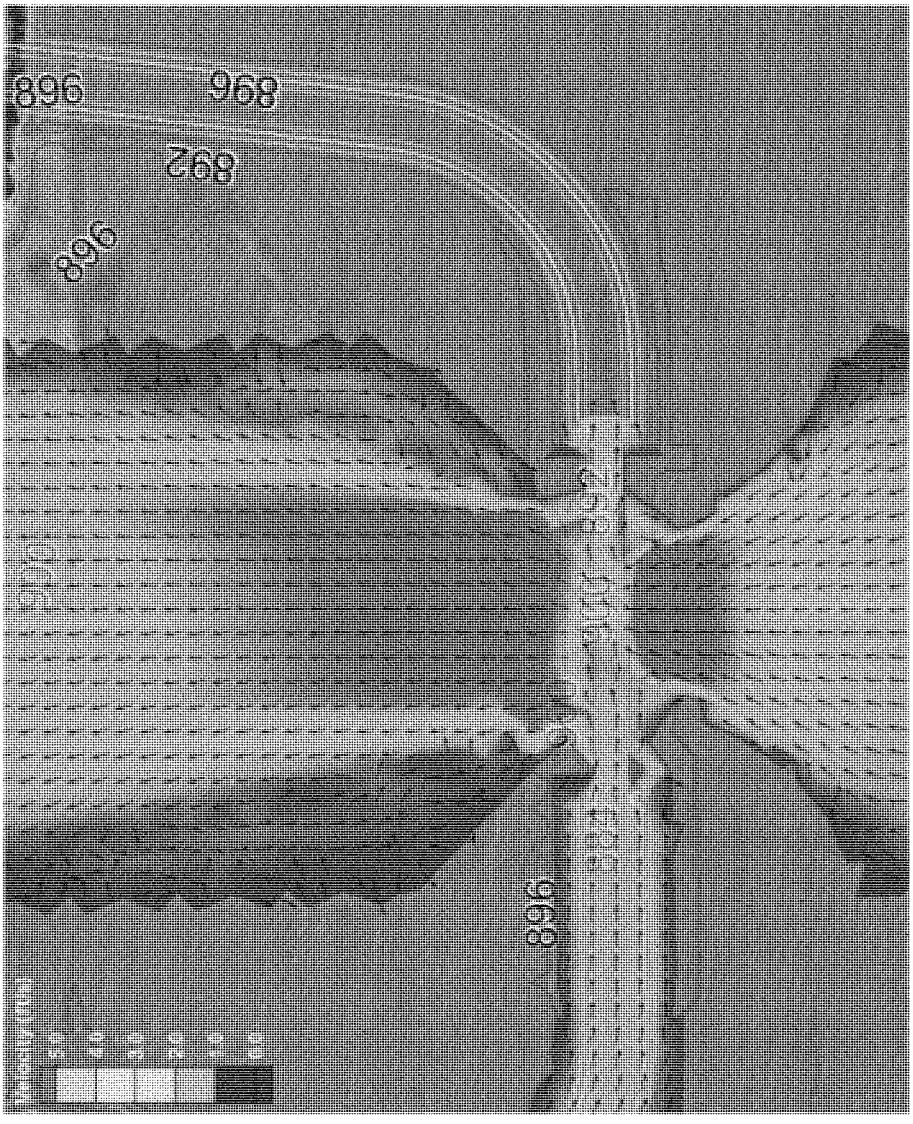

Figure F-H75 Velocity Field for Maple River Diversion Channel Post-construction Conditions 100-Year Event Flow

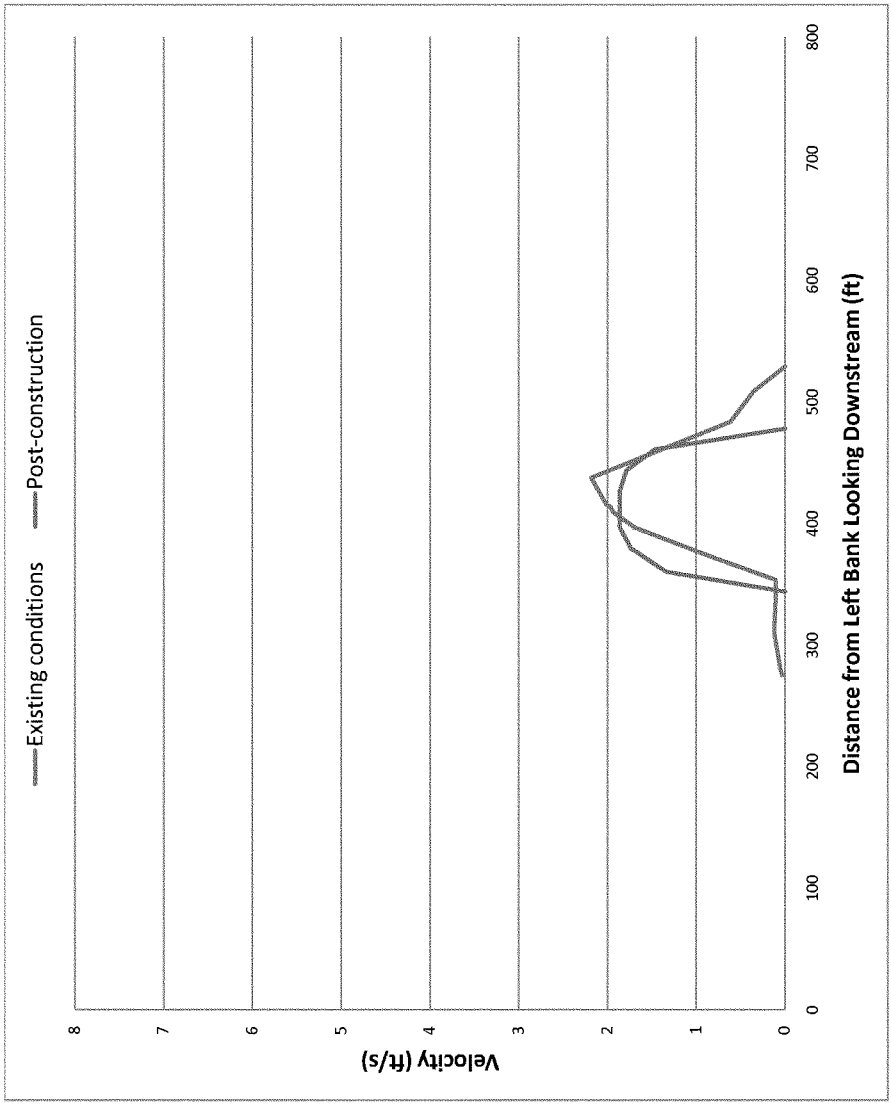

Figure F-H76 Cross Sectional Velocity Data for Maple River Aqueduct Upstream During 2-Year Event

Cross Sectional Velocity Data for Maple River Aqueduct (2-Year). Compares velocity at existing conditions to post-construction upstream of structure.

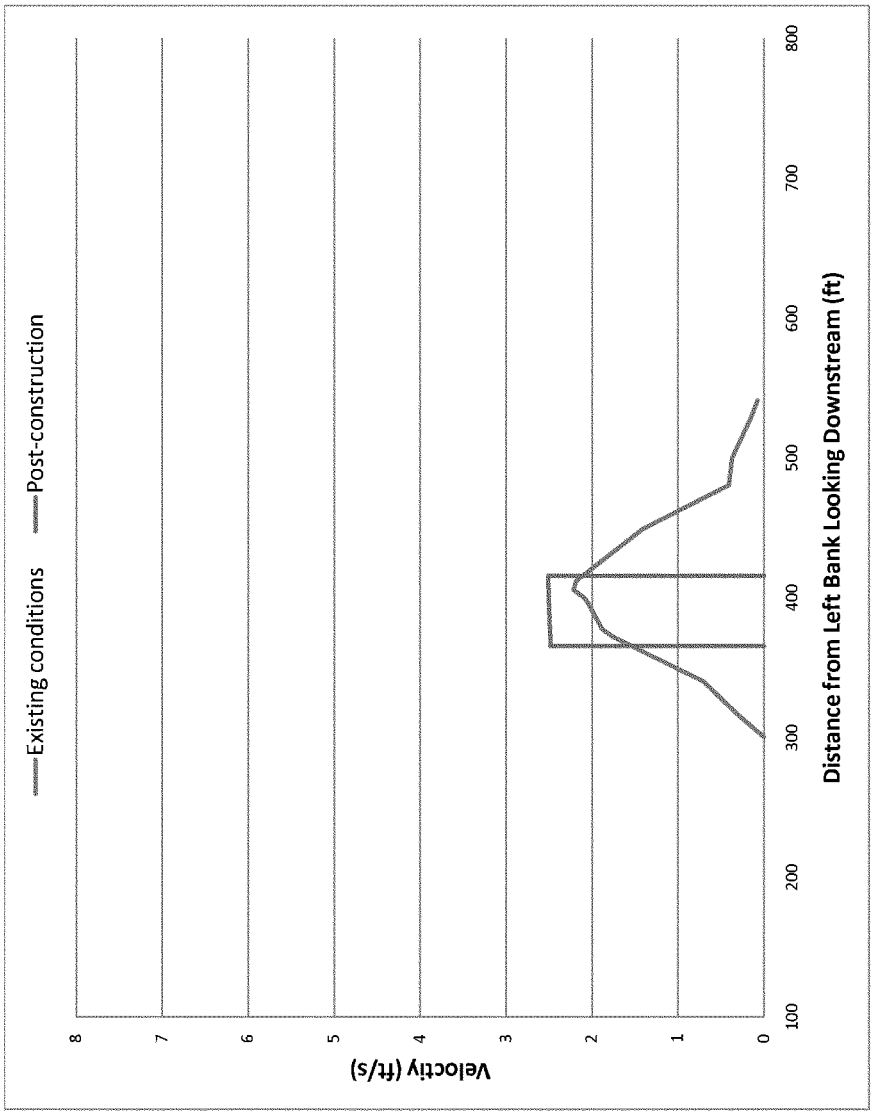

Figure F-H77 Cross Sectional Velocity Data for Maple River Aqueduct Through Structure 2-Year Event

Cross Sectional Velocity data for Maple River Aqueduct (2-Year). Compares velocity at existing conditions to post-construction through structure.

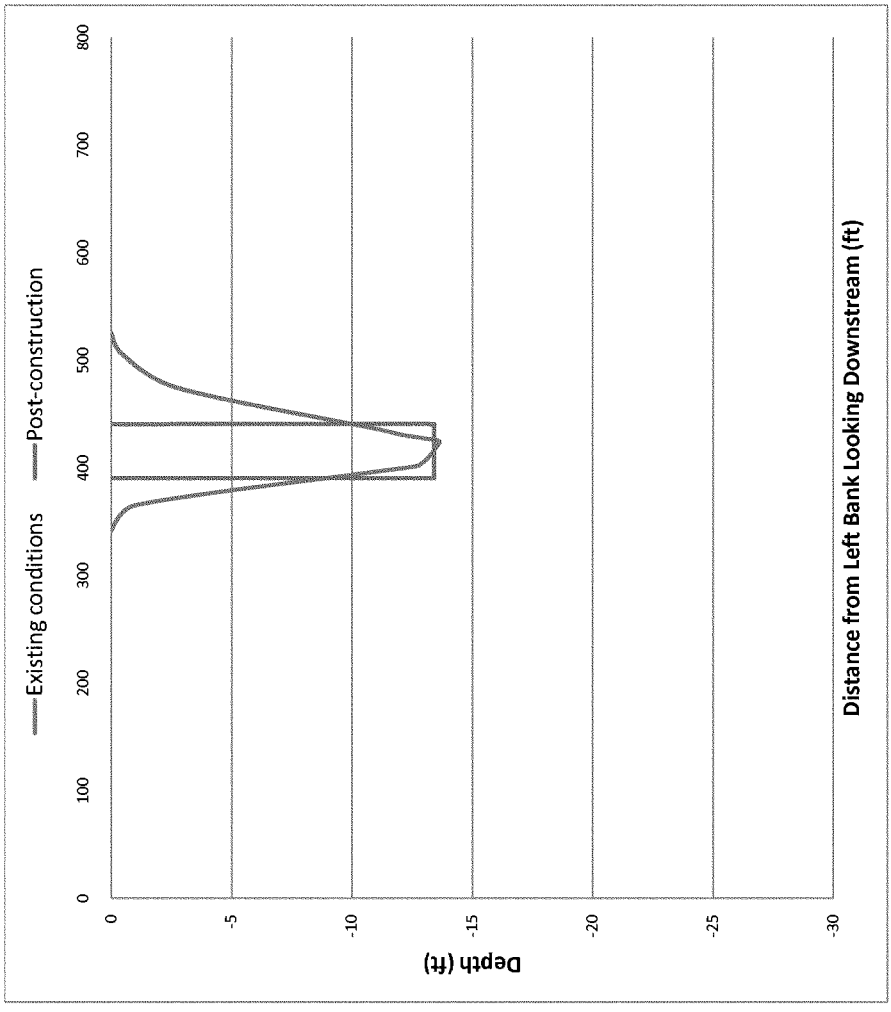

Figure F-H78 Cross Sectional Depth Data for Maple River Aqueduct Through Structure 2-Year Event
Cross Sectional Depth Data for Maple River Aqueduct (2-Year). Compares depth at existing conditions to post-construction through structure.

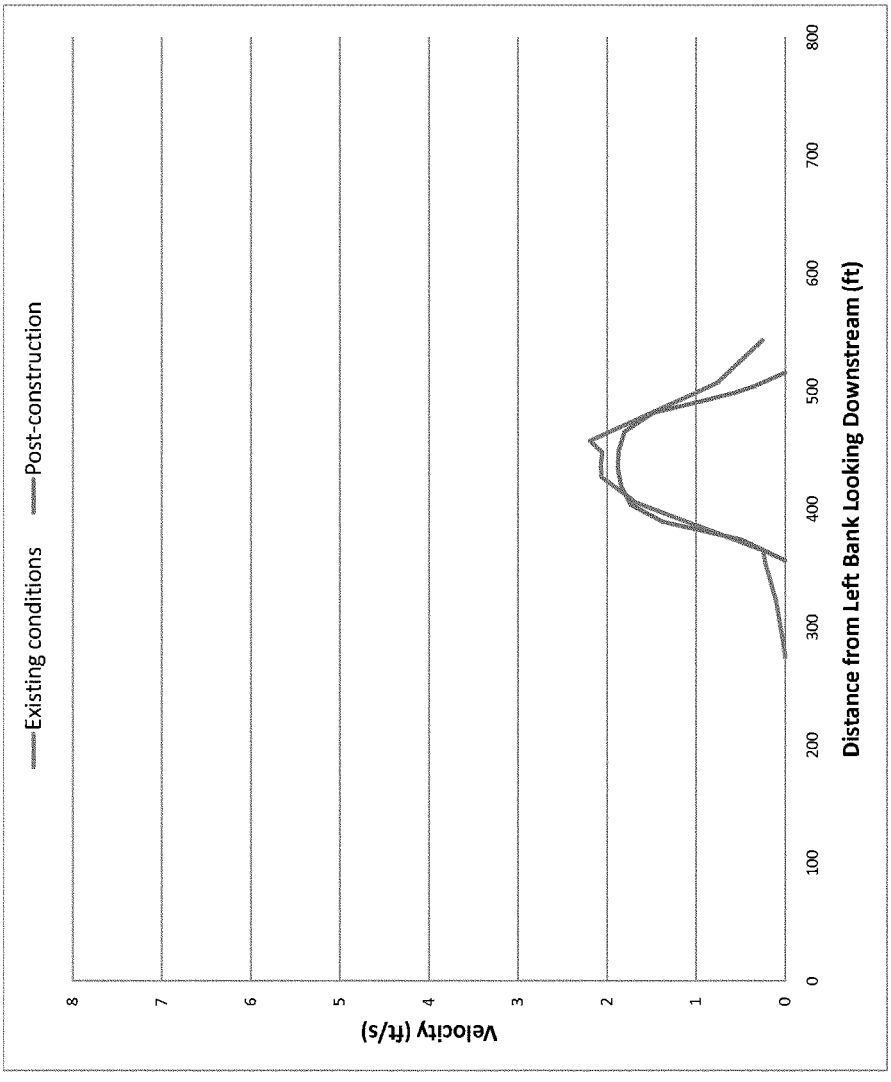

Figure F-H79 Cross Sectional Velocity Data for Maple River Aqueduct Downstream During 2-Year Event

Cross Sectional Velocity Data for Maple River Aqueduct (2-Year). Compares velocity at existing conditions to post-construction through structure.

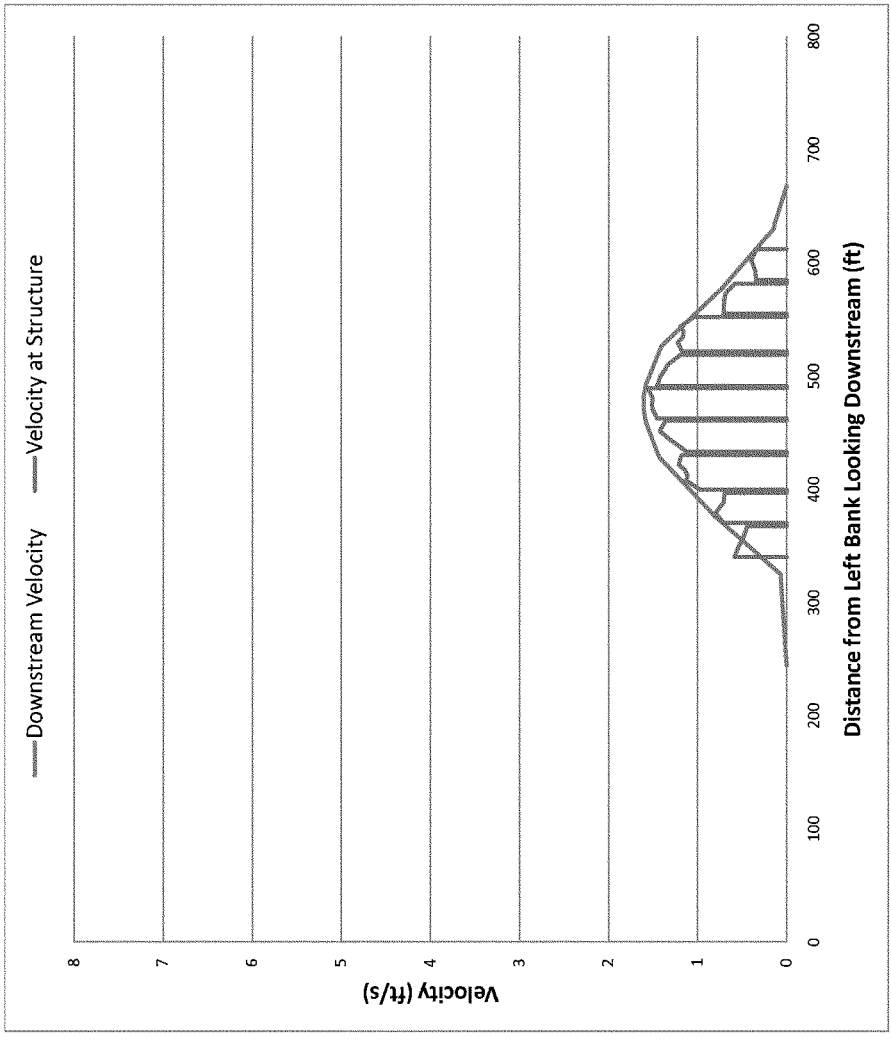

Figure F-H80 Cross Sectional Velocity Data for Maple River Diversion Channel and Downstream During 2,000 cfs Event
Cross Sectional Velocity Data for Maple River Aqueduct (2,000 cfs). Compares velocity at structure to downstream.

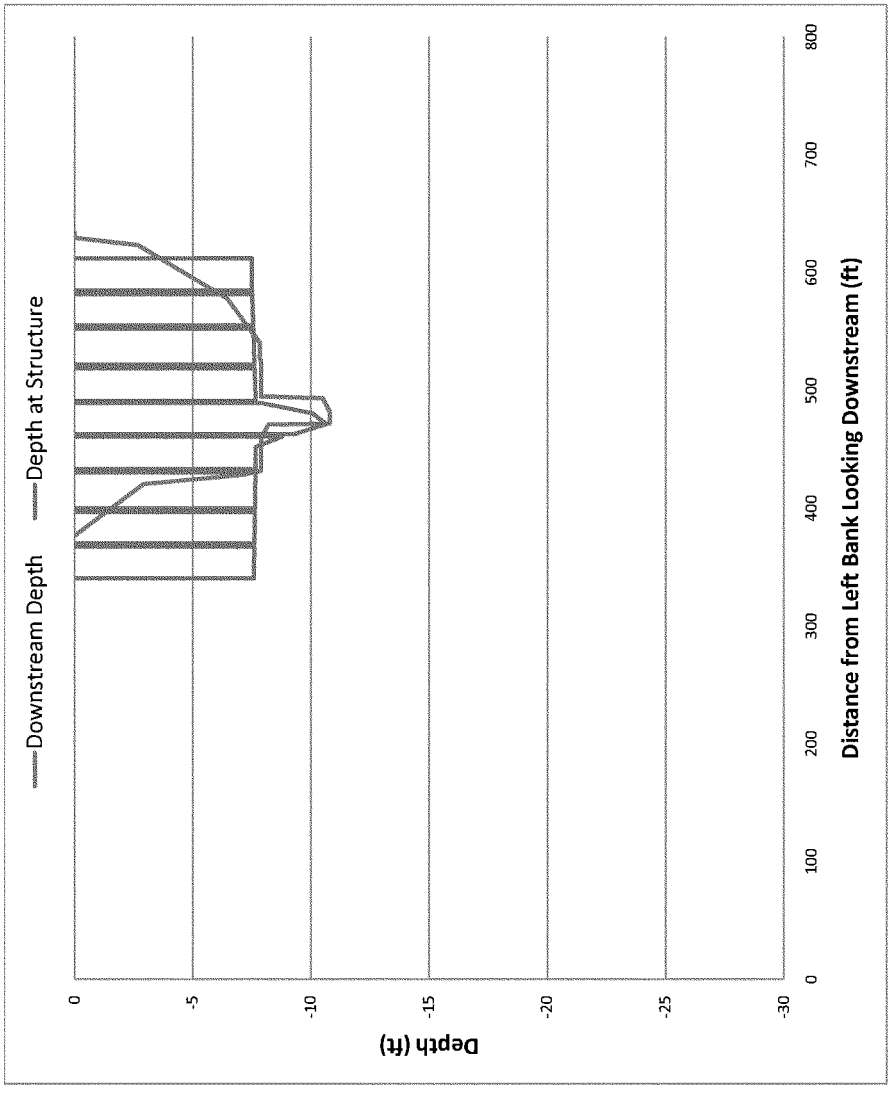

Figure F-H81 Cross Sectional Depth Data for Maple River Diversion Channel and Downstream During 2,000 cfs Event
Cross Sectional Depth Data for Maple River Aqueduct (2,000 cfs). Compares depth at structure to downstream.

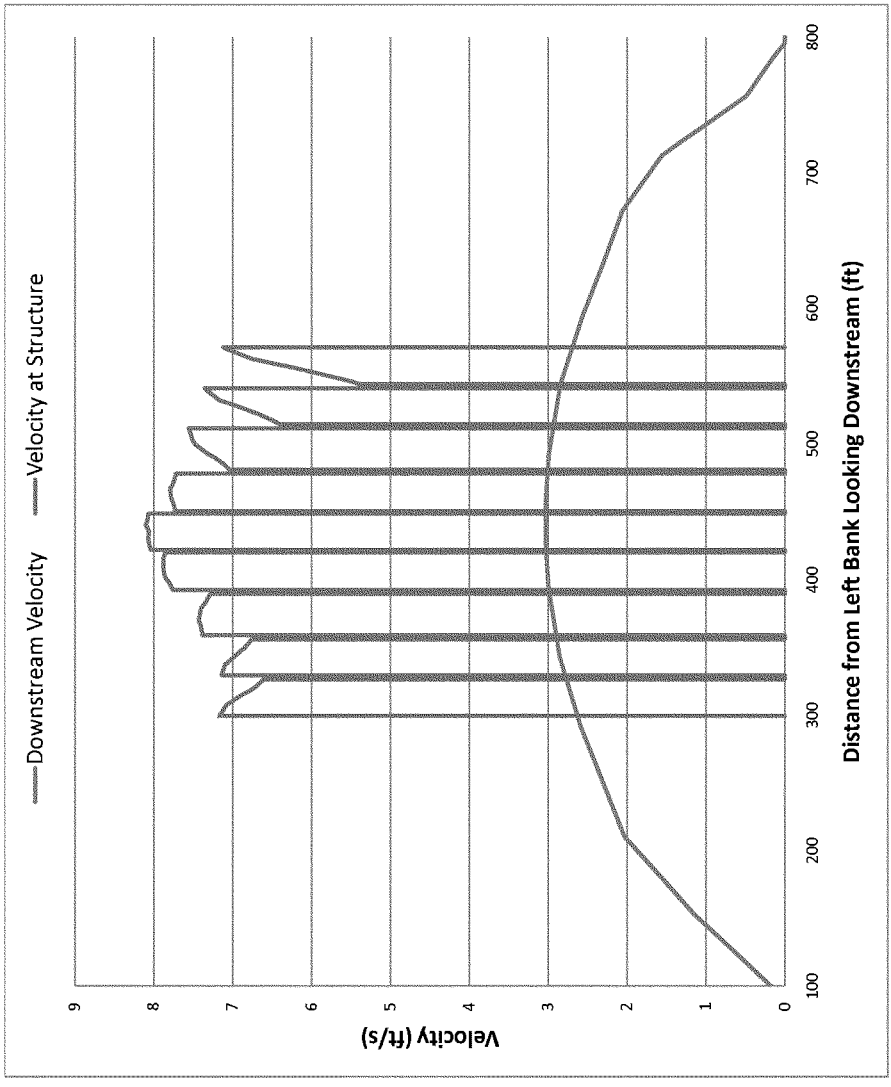

Figure F-H82 Cross Sectional Velocity Data for Maple River Diversion Channel and Downstream During 50-Year Event
Cross Sectional Velocity Data for Maple River Aqueduct (50-Year). Compares velocity at structure to downstream.

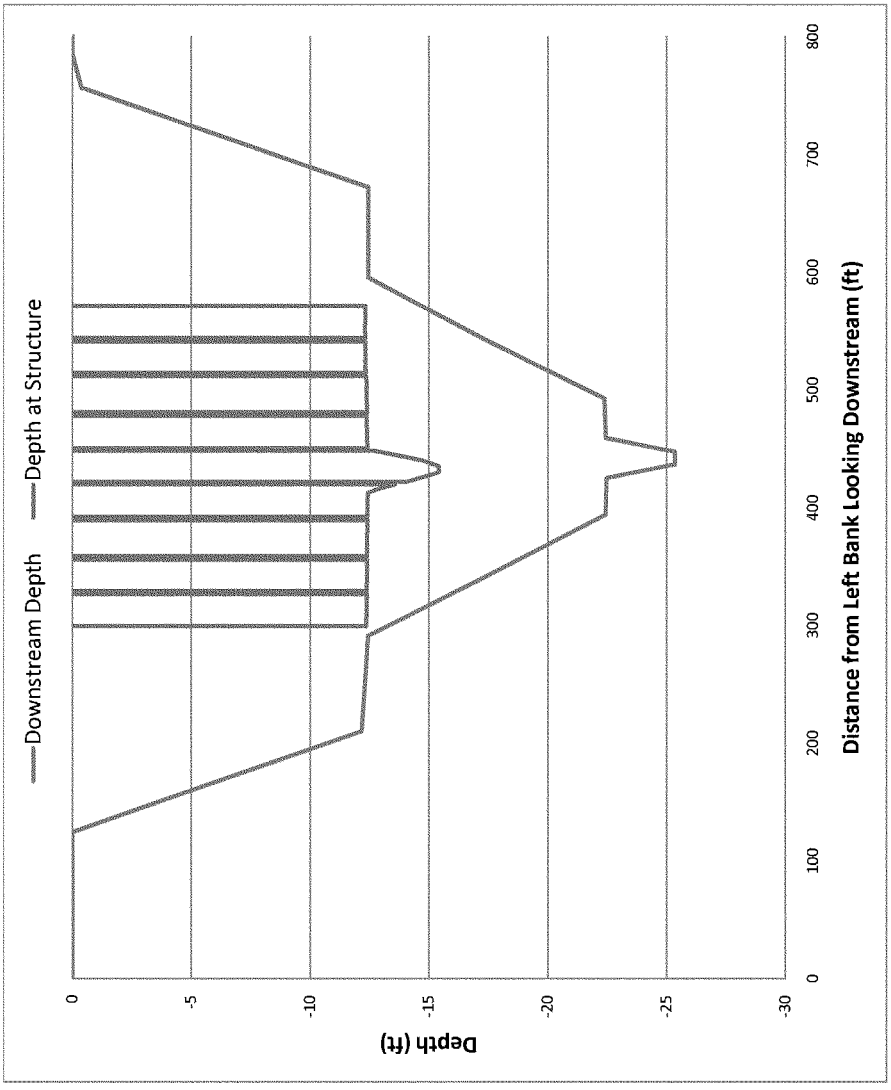

Figure F-H83 Cross Sectional Depth Data for Maple River Diversion Channel and Downstream During 50-Year Event
Cross Sectional Depth Data for Maple River Aqueduct (50-Year). Compares depth at structure to downstream.

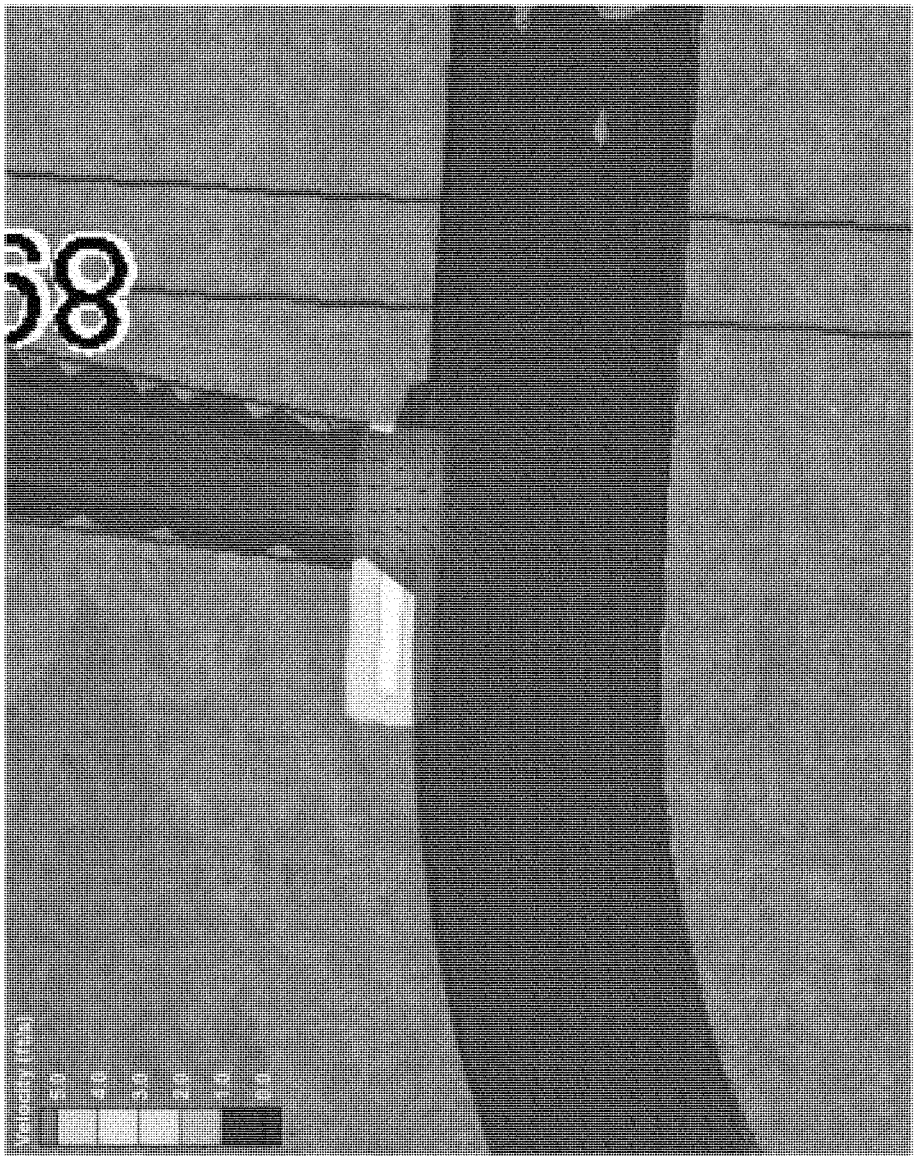

Figure F-H84 Velocity Field for Lower Rush River Structure Post-construction Conditions Mean Annual Flow Event

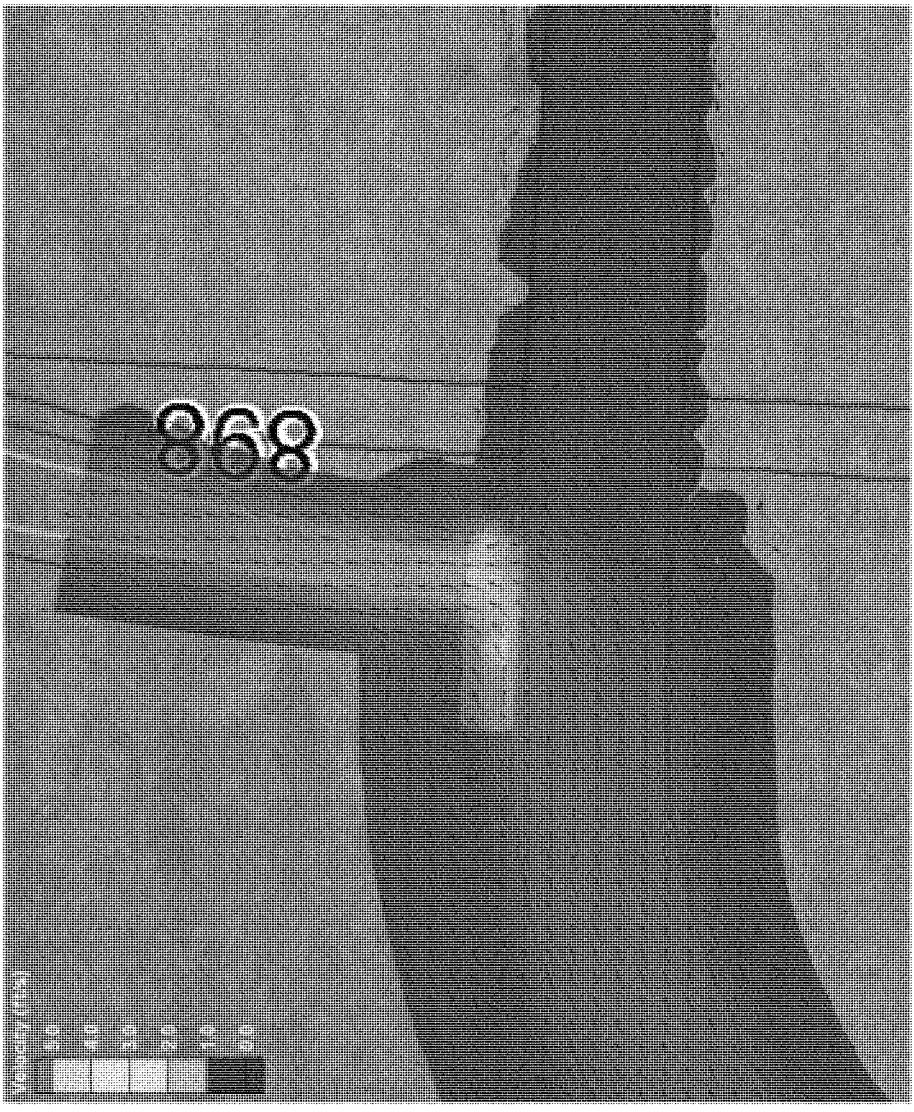

Figure F-H85 Velocity Field for Lower Rush River Structure Post-construction Conditions 20-Year Event Flow

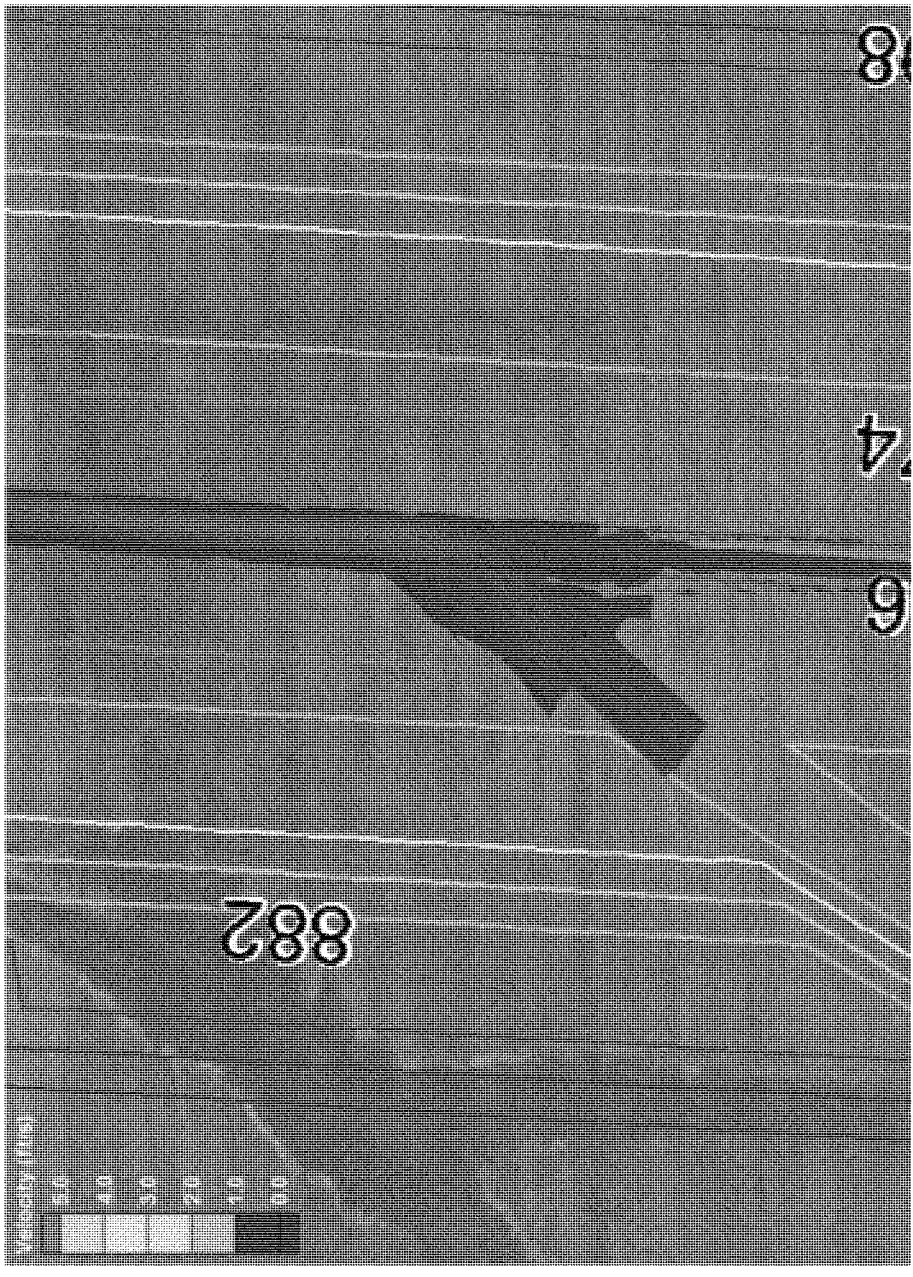

Figure F-H86 Velocity Field for Lower Rush River Diversion Channel Post-construction Conditions Mean Annual Flow Event

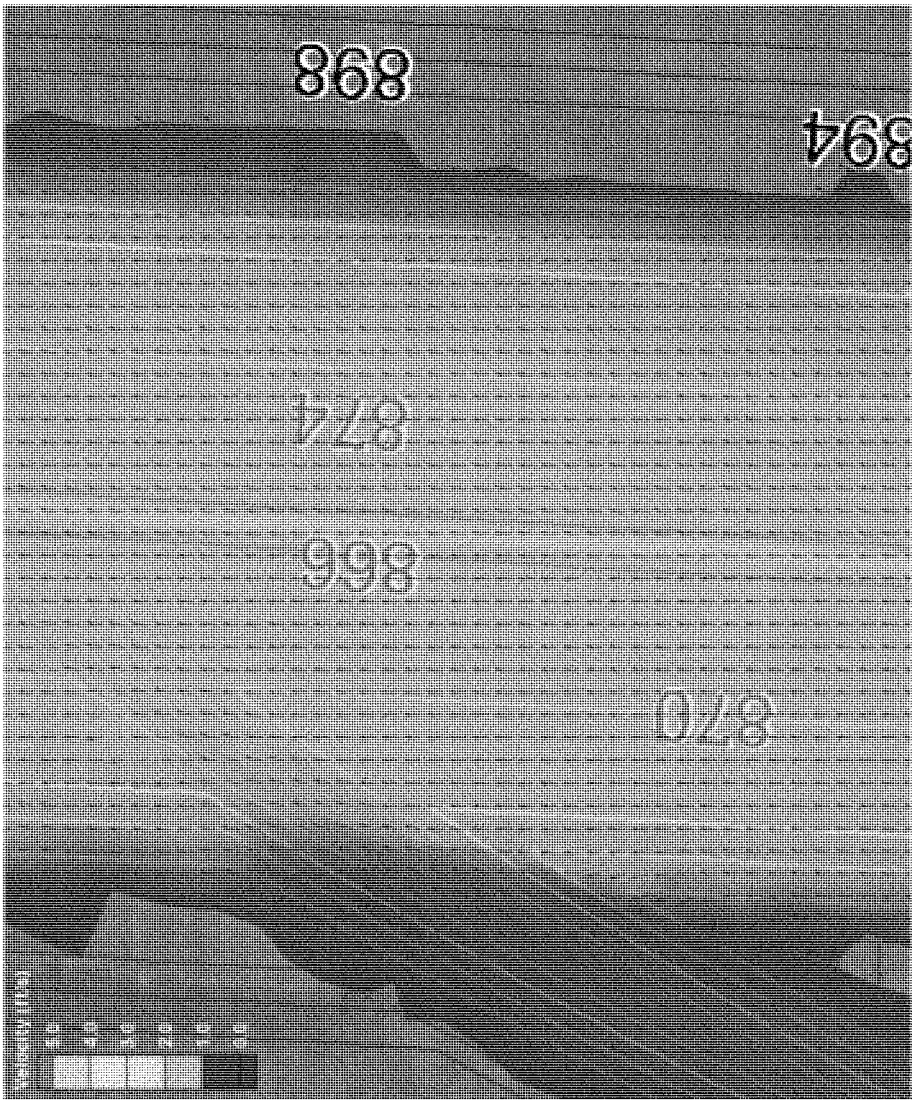

Figure F-H87 Velocity Field for Lower Rush River Diversion Channel Post-construction Conditions 20-Year Event Flow

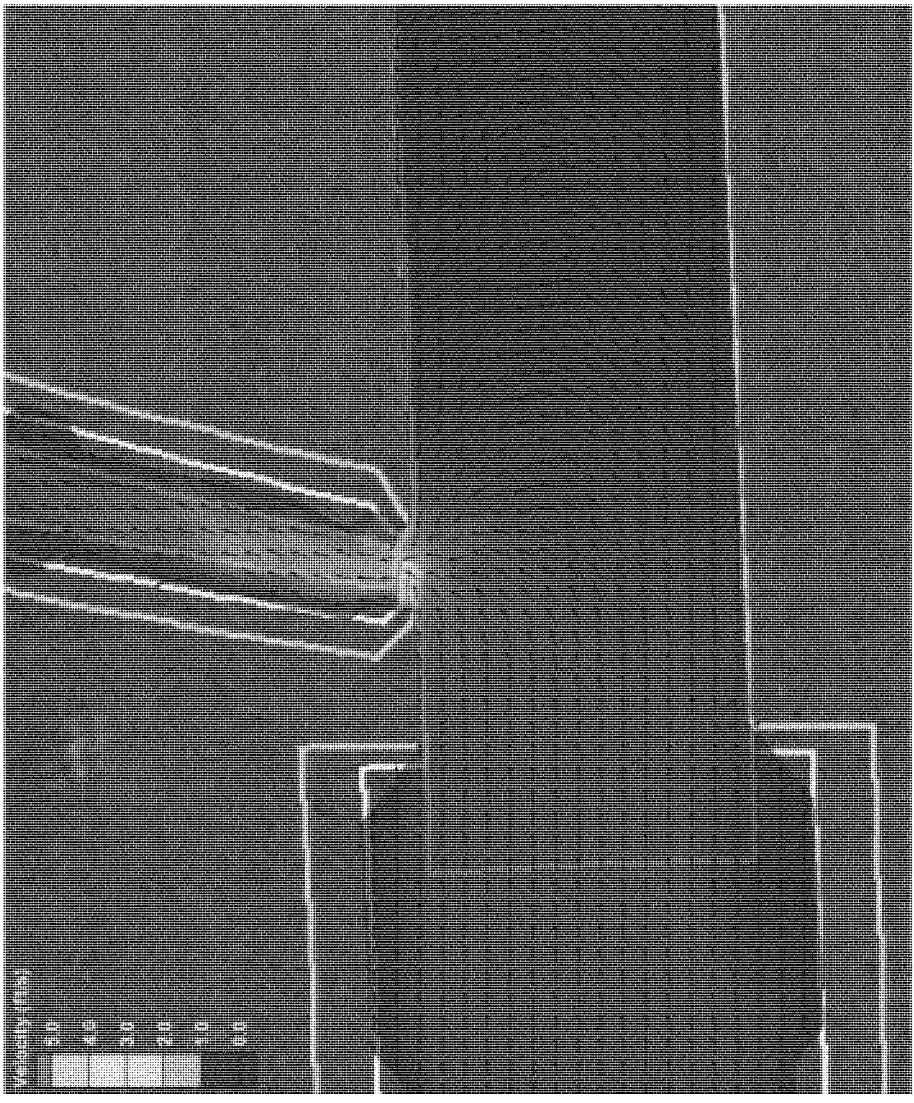

Figure F-H88 Velocity Field for Rush River Structure Post-construction Conditions Mean Annual Flow Event

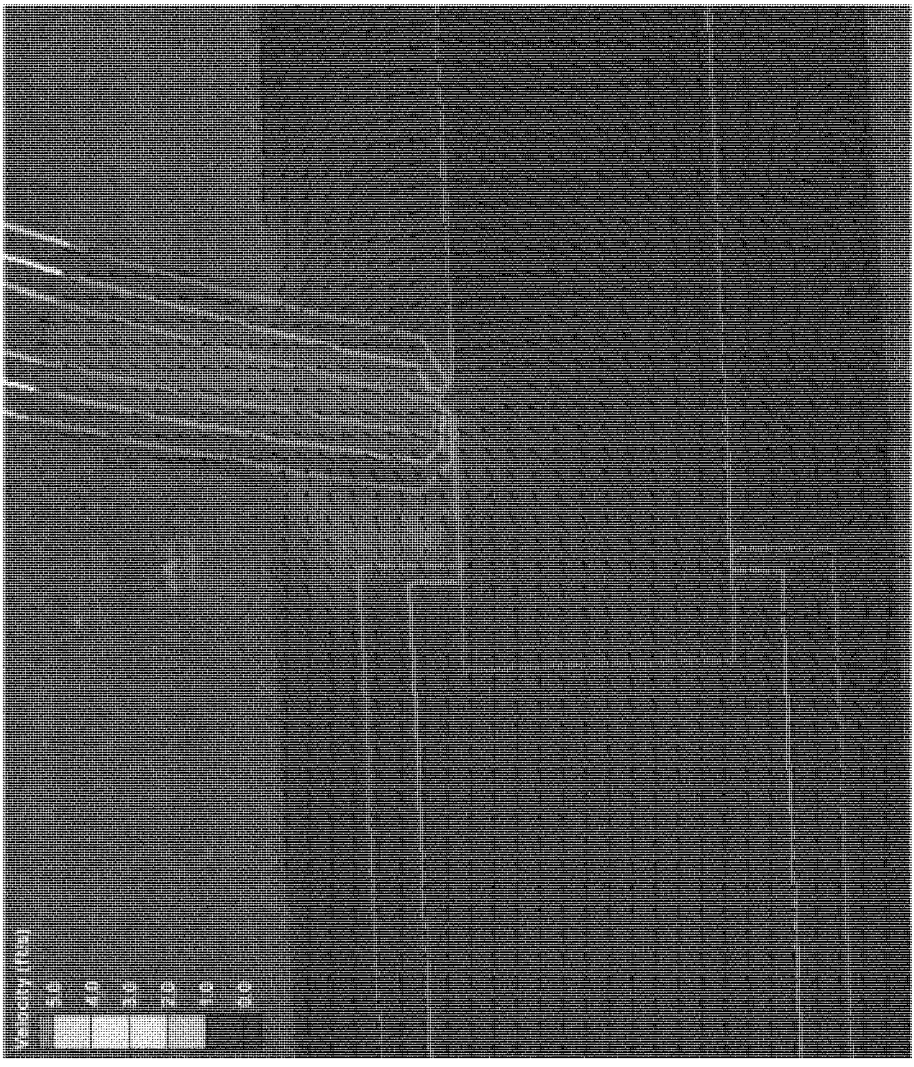

Figure F-H89 Velocity Field for Rush River Structure Post-construction Conditions 20-Year Event Flow

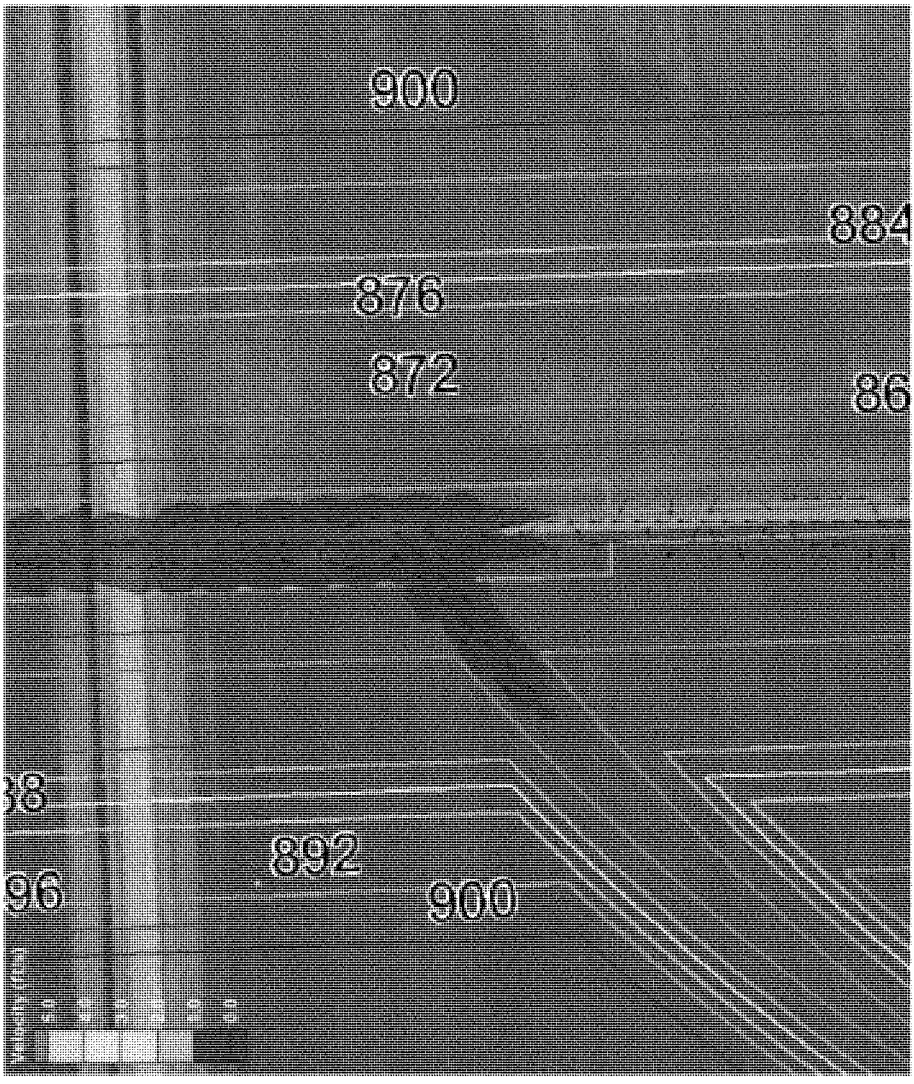

Figure F-H90 Velocity Field for Rush River Diversion Channel Post-construction Conditions Mean Annual Flow Event

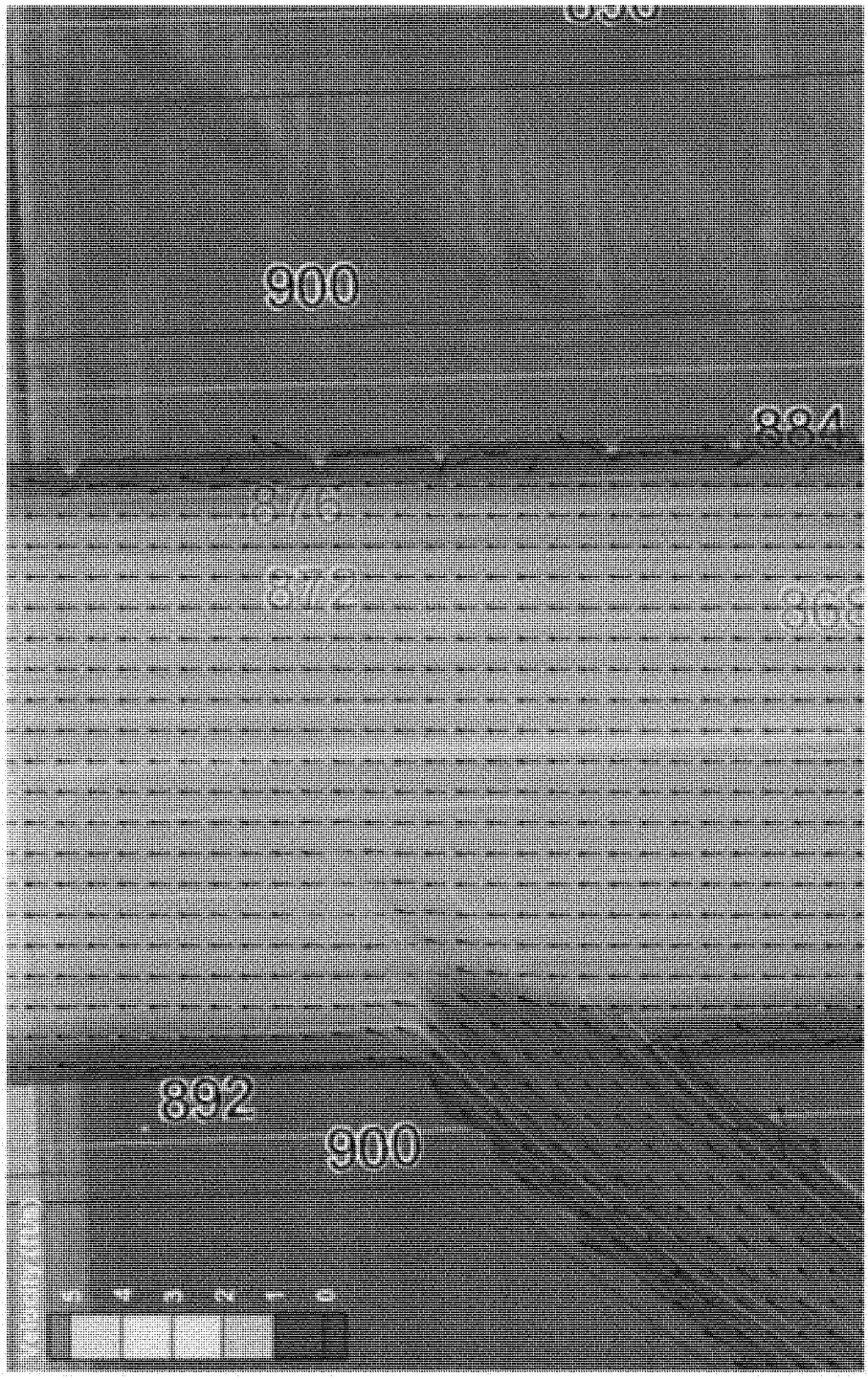

Figure F-H91 Velocity Field for Rush River Diversion Channel Post-construction Conditions 20-Year Event Flow

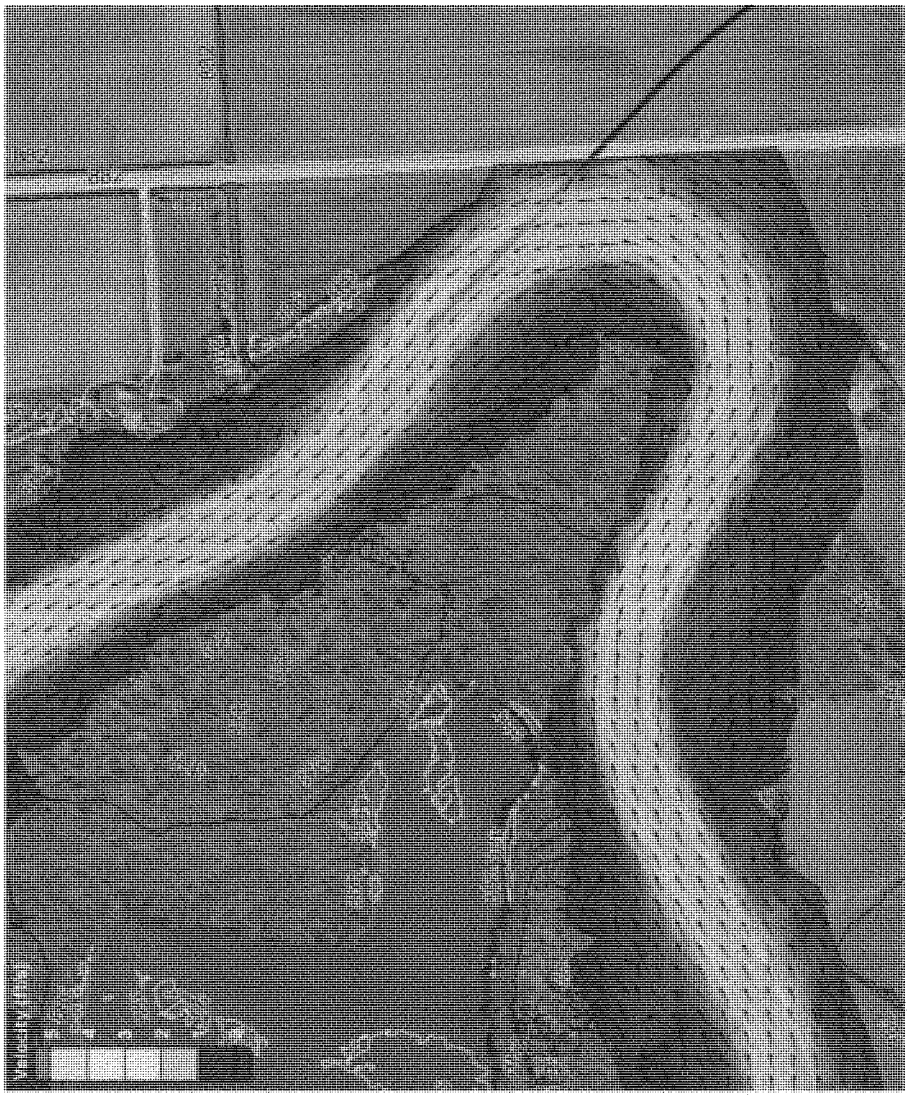

Figure F-H92 Velocity Field for Minnesota Outlet to Red River of the North Existing Conditions 2-Year Event Flow

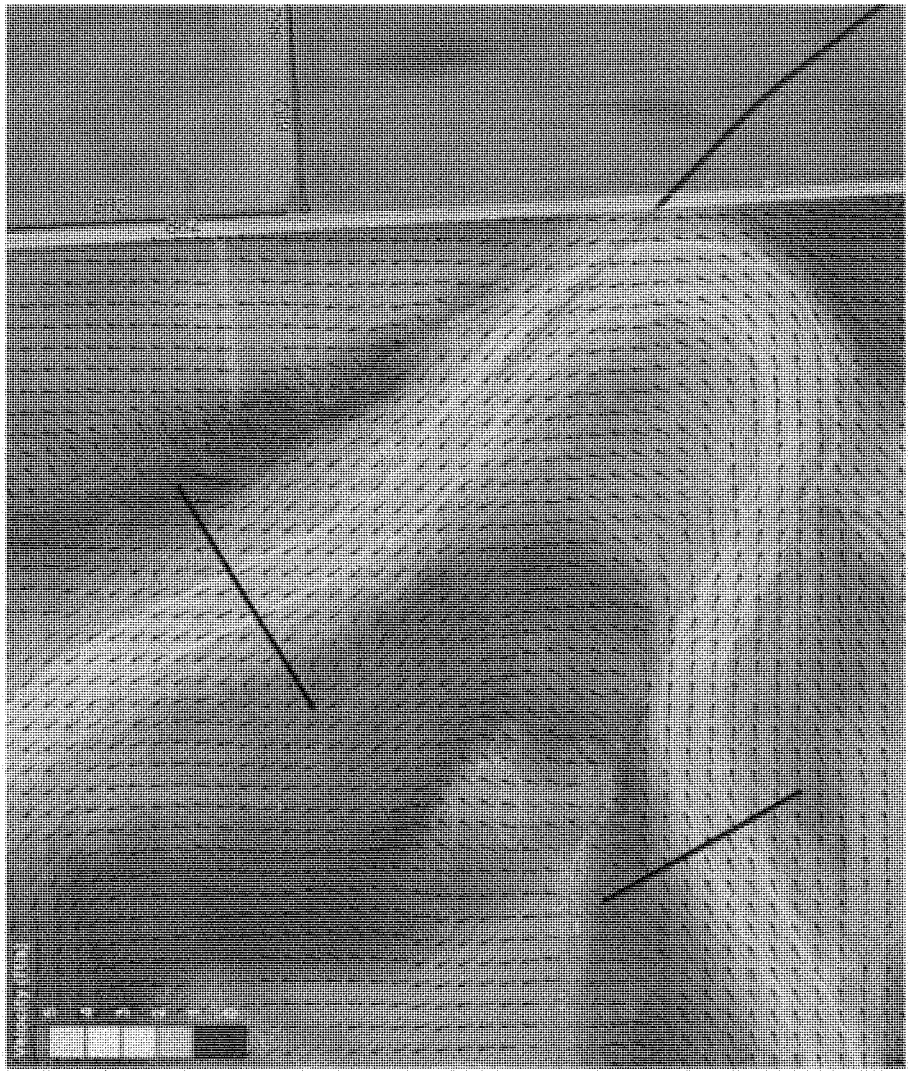

Figure F-H93 Velocity Field for Minnesota Outlet to Red River of the North Existing Conditions 20-Year Event Flow

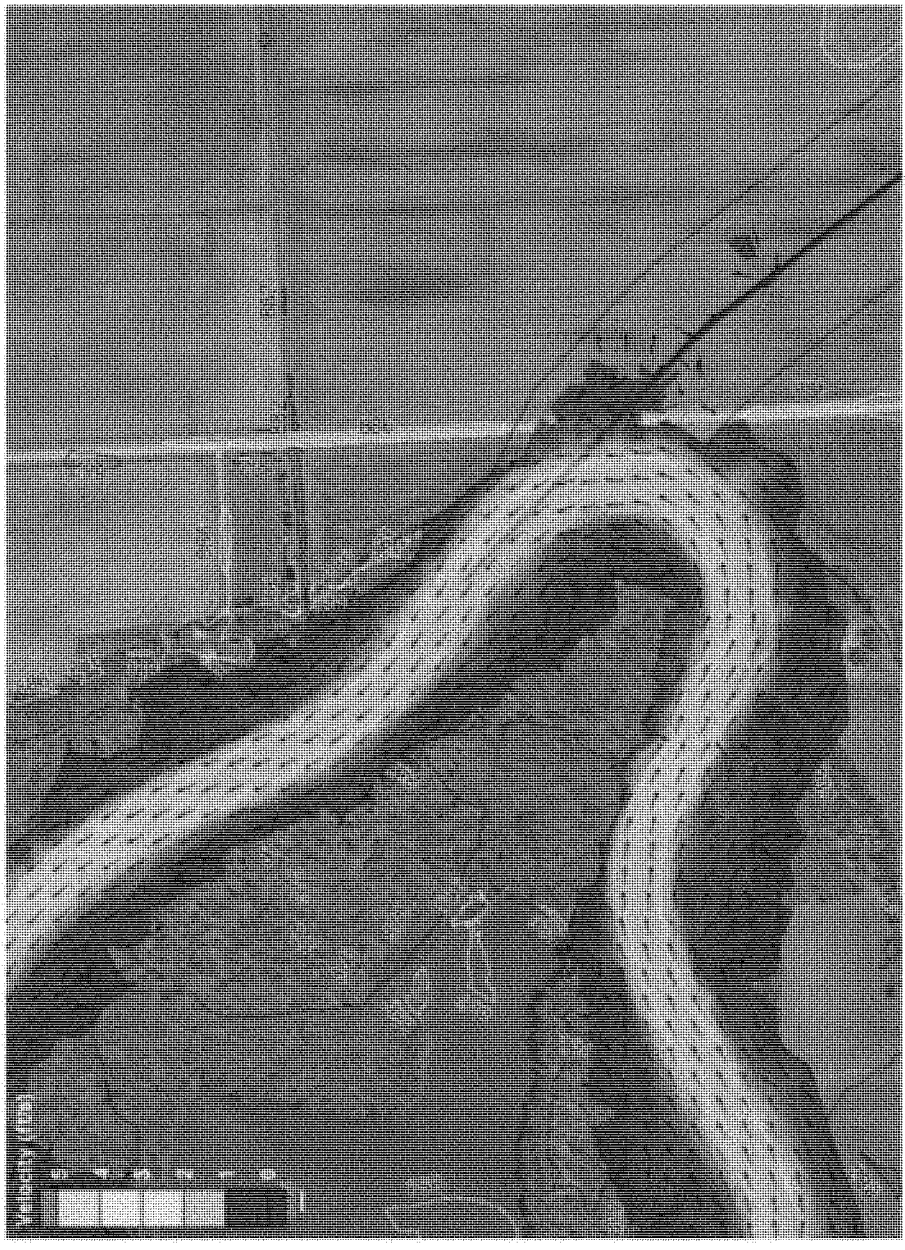

Figure F-H94 Velocity Field for Minnesota Outlet to Red River of the North Post-construction Conditions 2-Year Event Flow

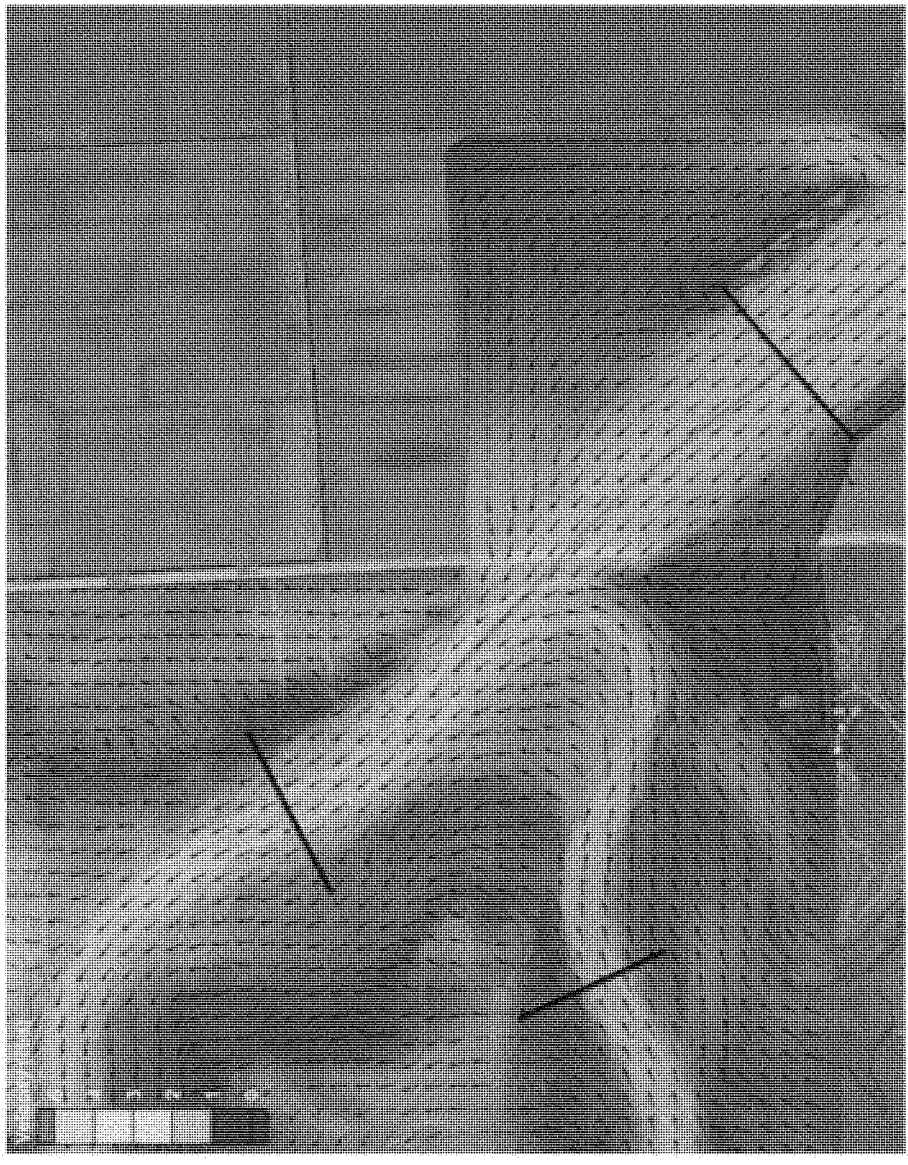

Figure F-H95 Velocity Field for Minnesota Outlet to Red River of the North Post-construction Conditions 20-Year Event Flow

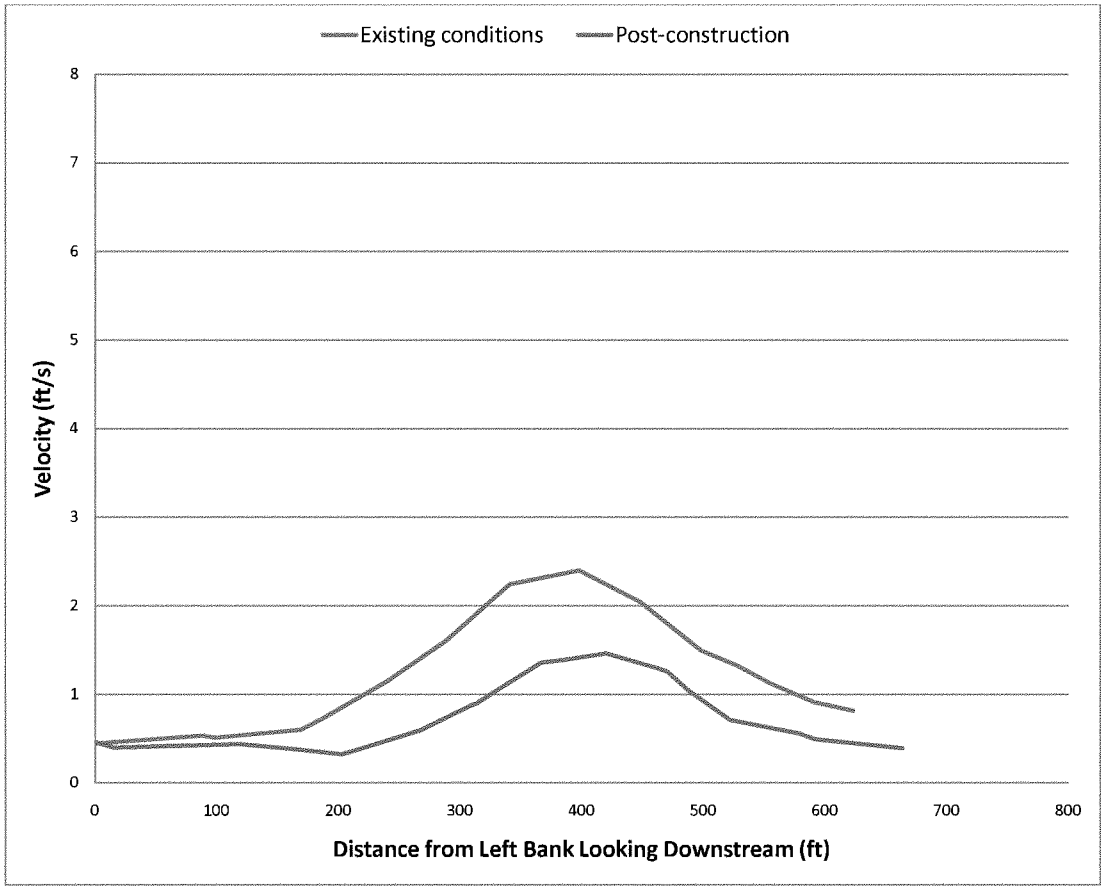

Figure F-H96 Cross Sectional Velocity Data for Minnesota Outlet to Red River of the North Upstream During 20-Year Event

Cross Sectional Velocity Data for Minnesota Outlet (20-Year). Compares velocity at existing conditions to post-construction upstream of structure.

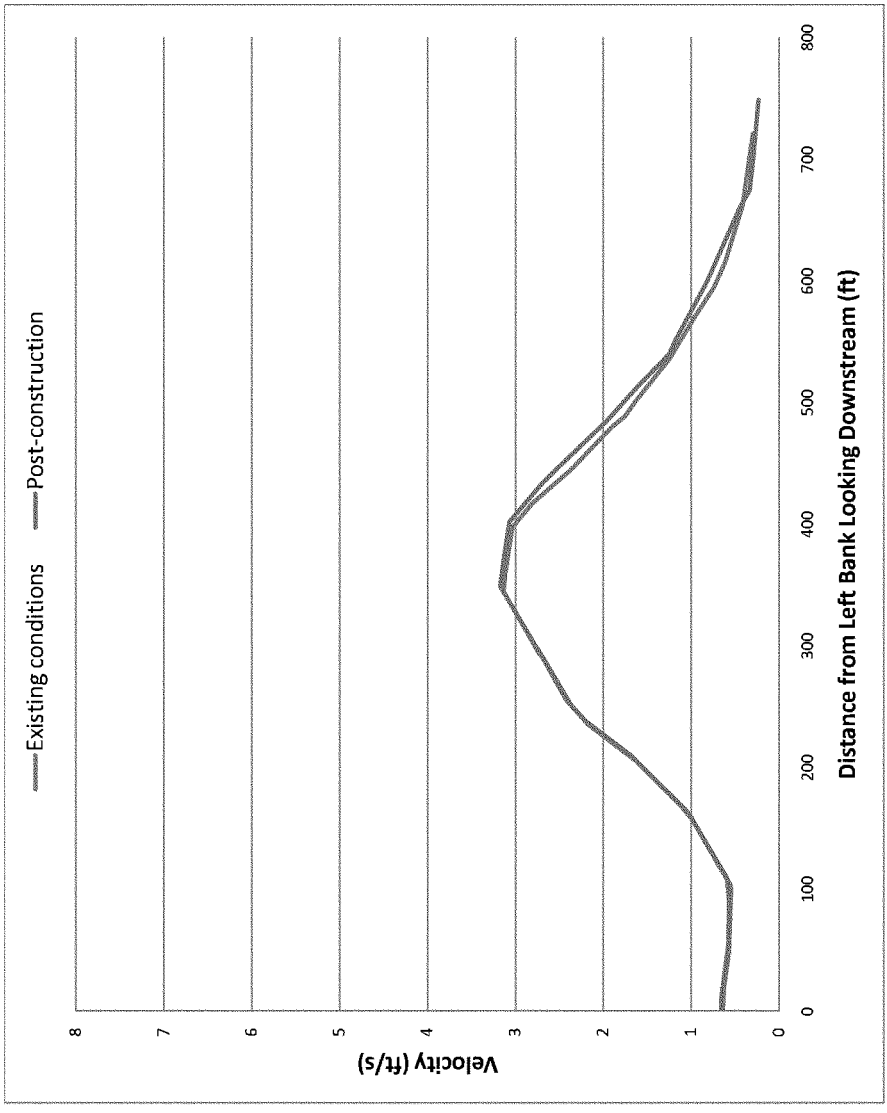

Figure F-H97 Cross Sectional Velocity Data for Minnesota Outlet to Red River of the North Downstream During 20-Year Event

Cross Sectional Velocity Data for Minnesota Outlet (20-Year). Compares velocity at existing conditions to post-construction downstream of structure.

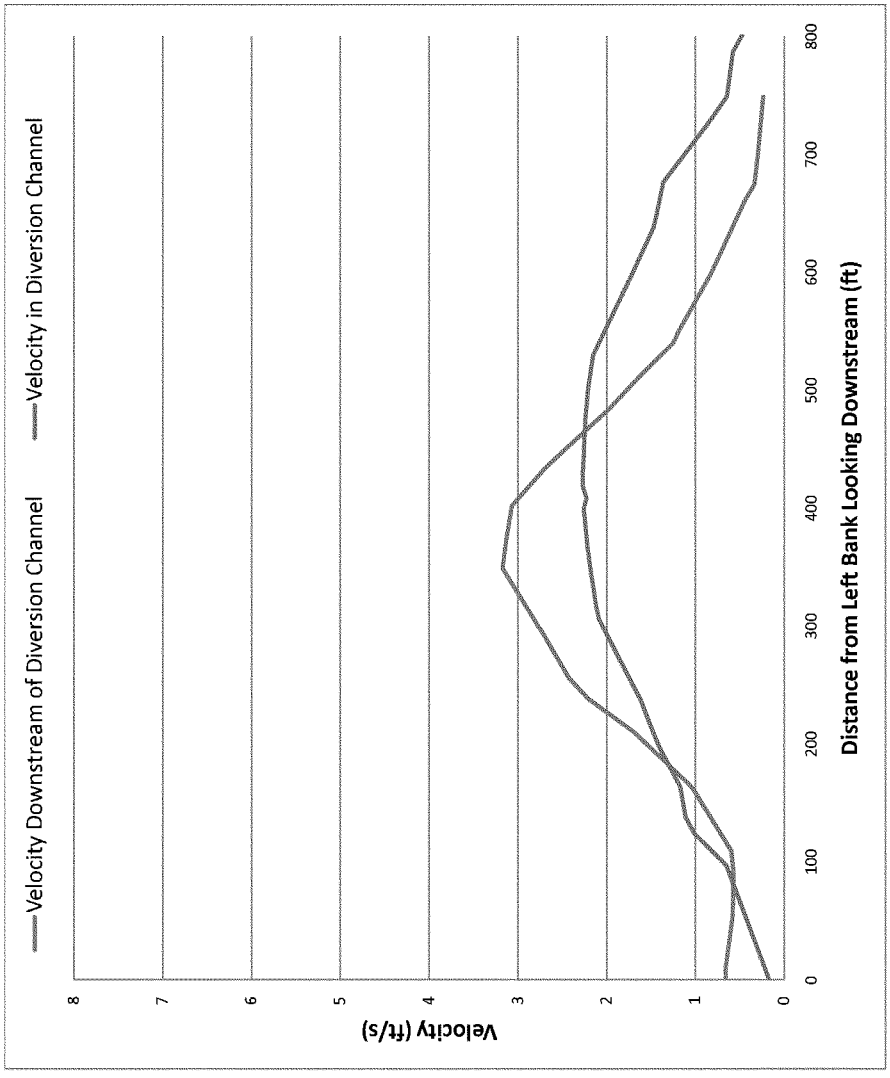

Figure F-H98 Cross Sectional Velocity Data for Minnesota Outlet to Red River of the North Diversion Channel During 20-Year Event
Cross Sectional Velocity Data for Minnesota Outlet (20-Year). Compares velocity in diversion channel to downstream.

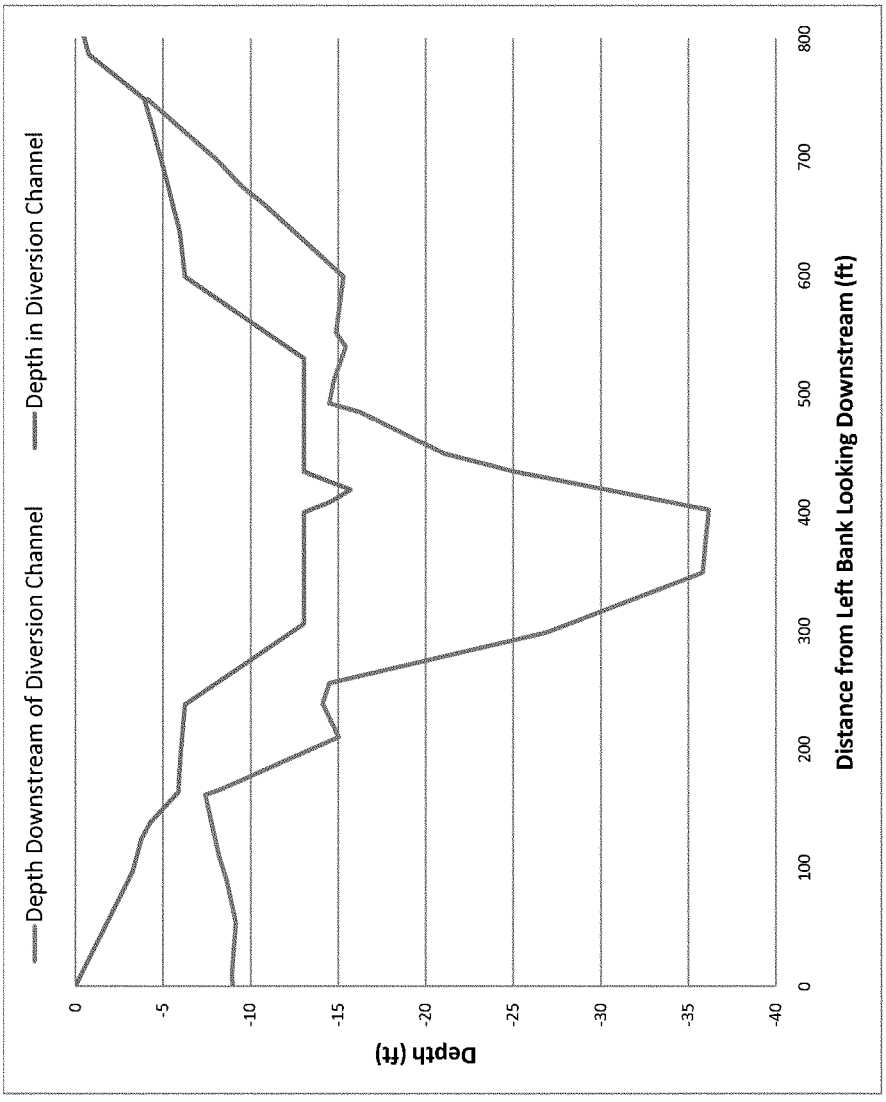

Figure F-H99 Cross Sectional Depth Data for Minnesota Outlet to Red River of the North Diversion Channel During 20-Year Event
Cross Sectional Depth Data for Minnesota Outlet (20-Year). Compares depth in diversion channel to downstream.

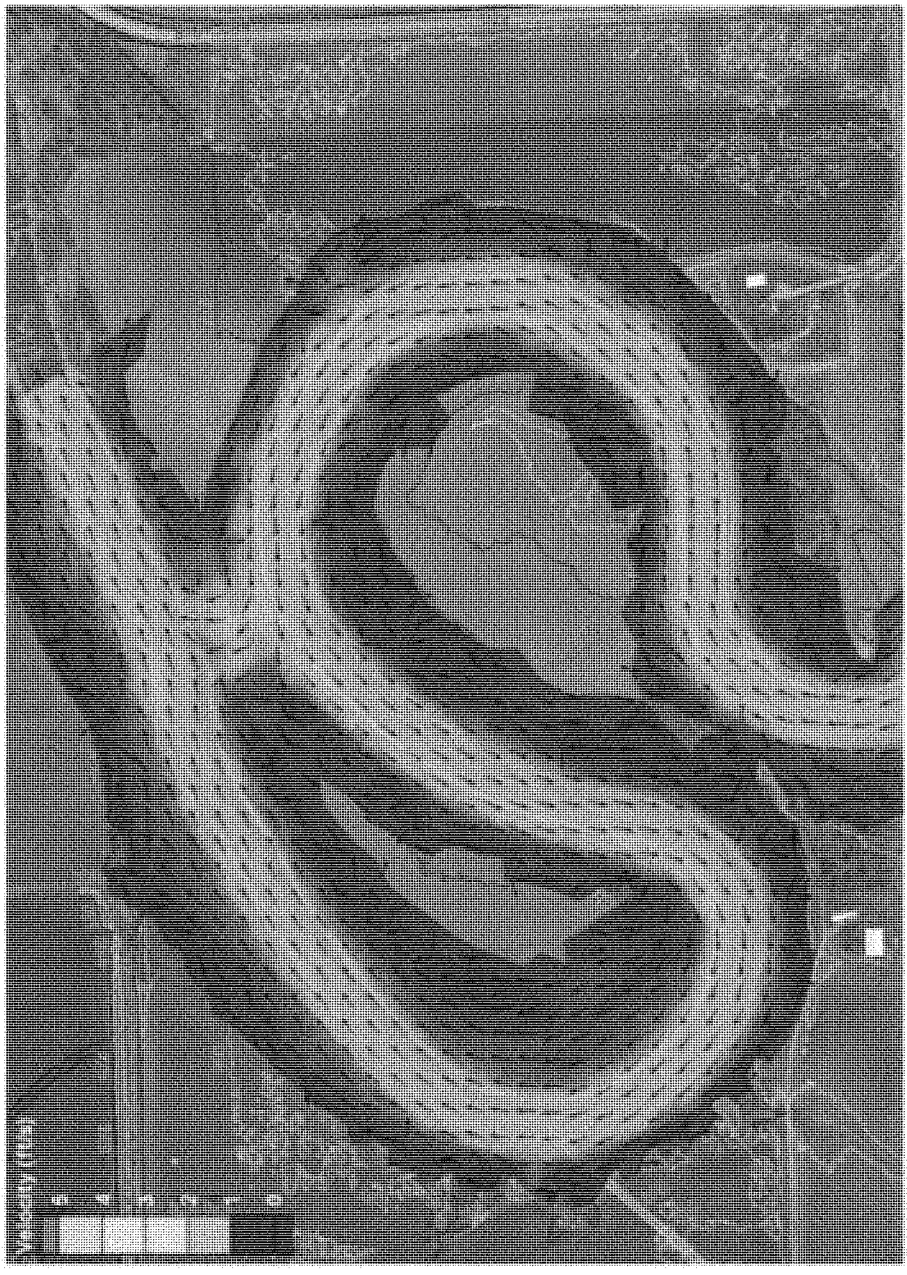

Figure F-H100 Velocity Field for North Dakota Outlet to Red River of the North Existing Conditions 2-Year Event Flow

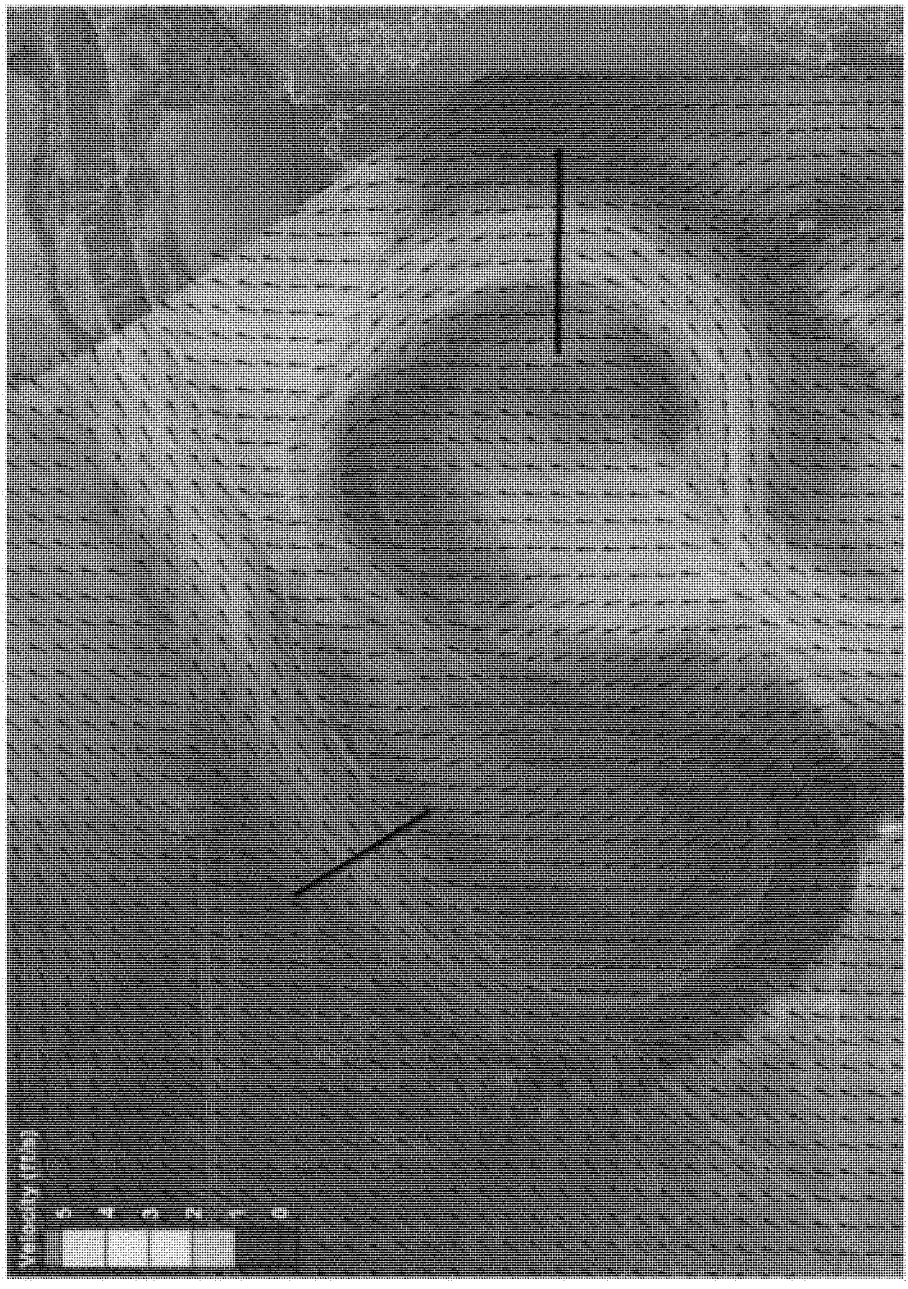

Figure F-H101 Velocity Field for North Dakota Outlet to Red River of the North Existing Conditions 20-Year Event Flow

Figure F-H102 Velocity Field for North Dakota Outlet to Red River of the North Post-construction Conditions 2-Year Event Flow

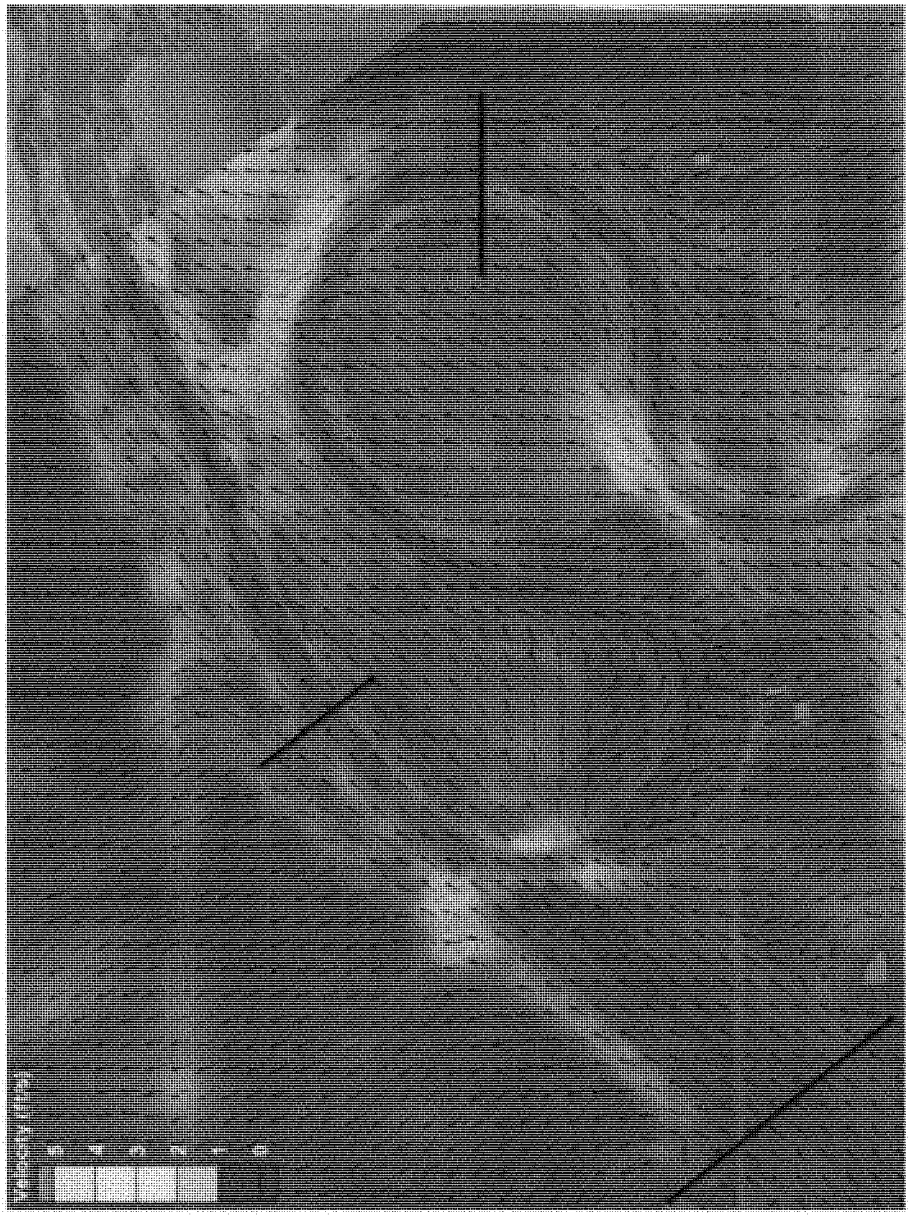

Figure F-H103 Velocity Field for North Dakota Outlet to Red River of the North Post-construction Conditions 20-Year Event Flow

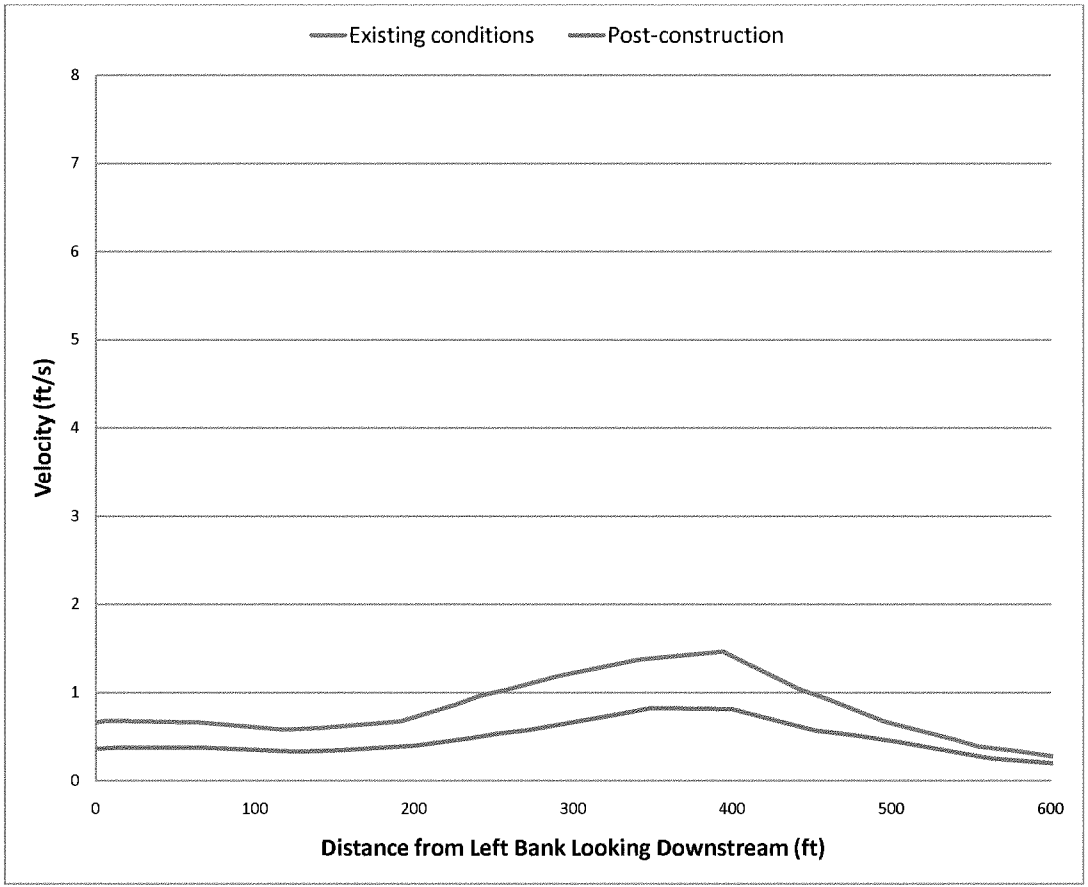

Figure F-H104 Cross Sectional Velocity Data for North Dakota Outlet to Red River of the North Upstream During 20-Year Event

Cross Sectional Velocity Data for North Dakota Outlet (20-Year). Compares velocity at existing conditions to post-construction upstream of structure.

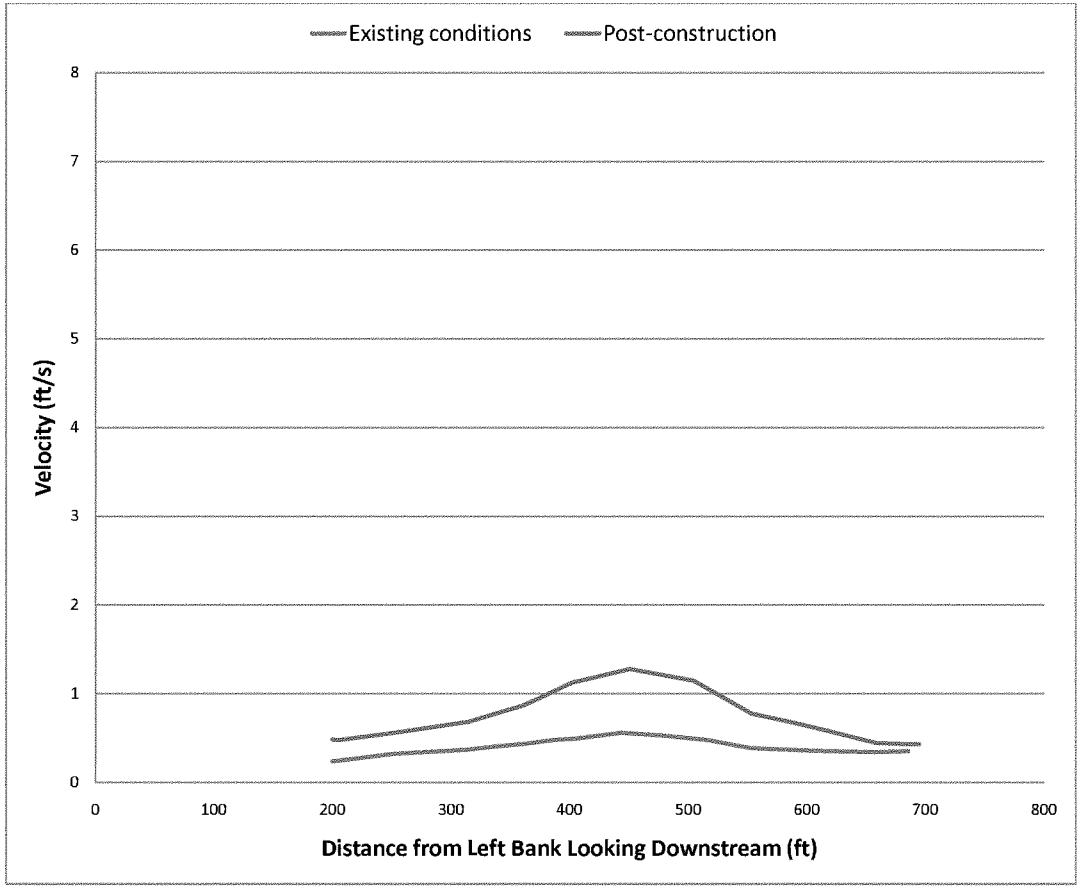

Figure F-H105 Cross Sectional Velocity Data for North Dakota Outlet to Red River of the North Downstream During 20-Year Event

Cross Sectional Velocity Data for North Dakota Outlet (20-Year). Compares velocity at existing conditions to post-construction downstream of structure.

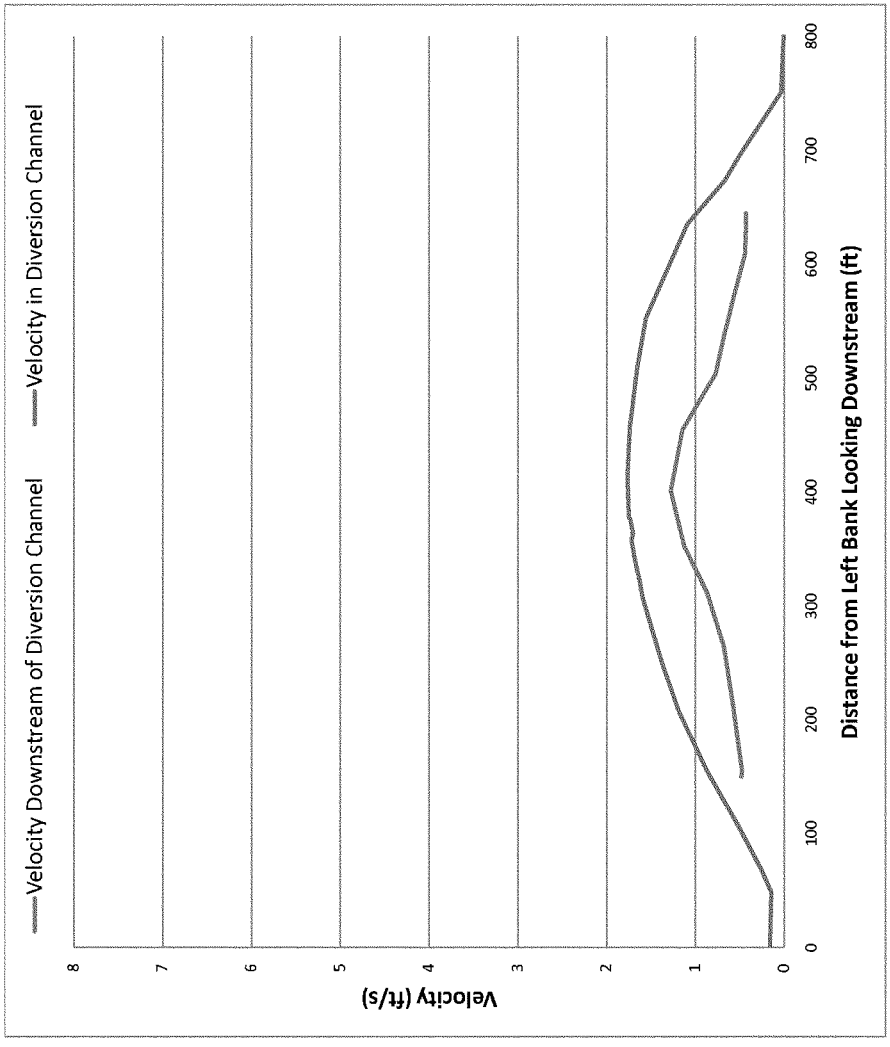

Figure F-H106 Cross Sectional Velocity Data for North Dakota Outlet to Red River of the North Diversion Channel During 20-Year Event
Cross Sectional Velocity Data for North Dakota Outlet (20-Year). Compares velocity in diversion channel to downstream.

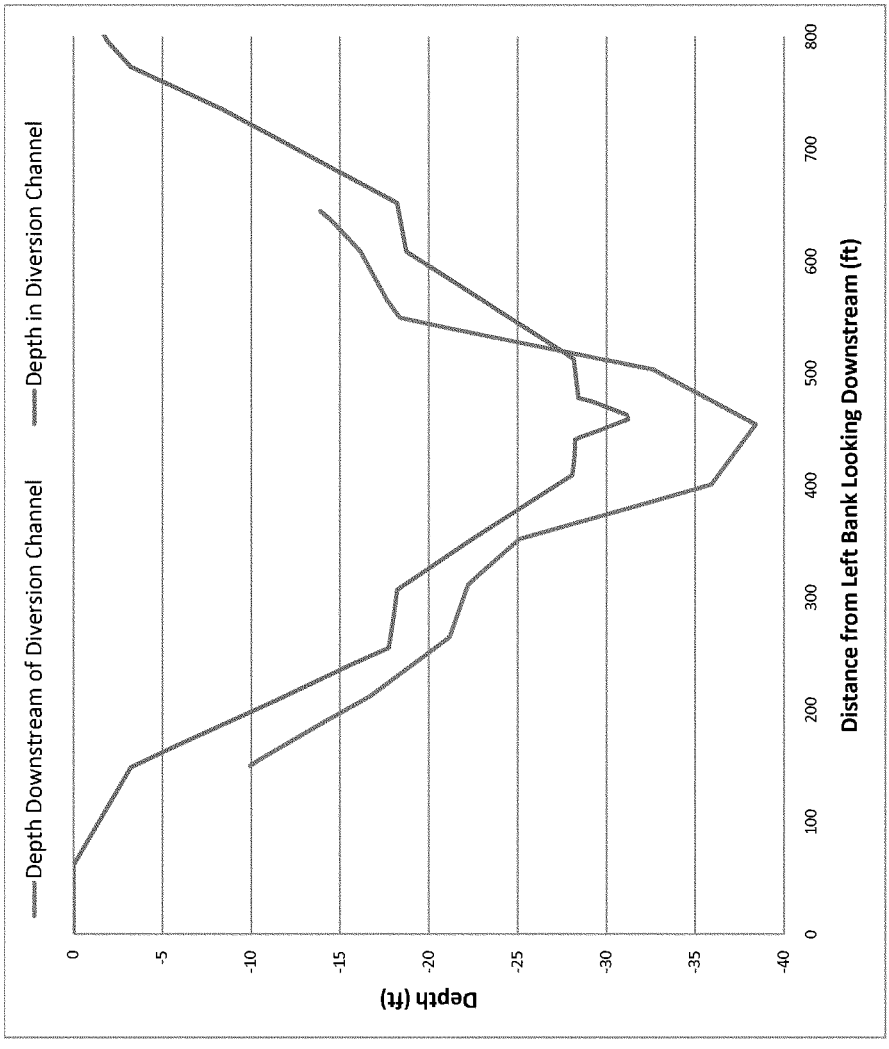

Figure F-H107 Cross Sectional Depth Data for North Dakota Outlet to Red River of the North Diversion Channel During 20-Year Event
Cross Sectional Depth Data for North Dakota Outlet (20-Year). Compares depth in diversion channel to downstream.

RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT I – SEDIMENT TRANSPORT

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

By: Barr Engineering Co.

VERSION 2 – March 9, 2011

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT I – SEDIMENT TRANSPORT

F-I1.0 INTRODUCTION AND GEOLOGIC SETTING

In order to characterize the sediment transport and geomorphology of the rivers in the vicinity of the cities of Fargo and Moorhead, it is important to first understand the geologic setting of the greater Red River Basin. The geology, topography, and soils of the area are the fundamental influences (along with hydrology) that govern river shape and morphology.

F-I1.1 STUDY LOCATION IN REGIONAL GEOLOGY

The central feature of the Red River Basin is the Red River Valley, the flat plain that once was the bed of Glacial Lake Agassiz. The lake formed at the southern edge of the Laurentide Ice Sheet and remained in existence from approximately 11,500 to 7,500 years before present (Teller and Clayton, 1983). Within the study area and over much of the old lake bed, the lake left behind a 150 to 300 foot layer of primarily silts and clays (Klausing, 1968; Fenton et al., 1983; Tornes and Brigham, 1994) over a 50 to 60 mile wide area stretching from south of Breckenridge, Minnesota to Winnipeg, Manitoba. This area is known as the "lake plain" in descriptions of the Red River Basin (see Figure F-I1). Within the lake plain, topographic relief is minimal (Figure F-I2) and the typical slope is less than 5 feet per mile (0.1%, Figure F-I3).

The lake plain is bordered by steeper beach ridges, which formed the shoreline of Glacial Lake Agassiz. Glacial rivers flowing into the lake deposited coarser sediment (sands and gravels) in these areas (Christensen, 2007), creating deltas that are mostly buried beneath later lake-deposited fine sediment. The surficial geology of the study area is shown in Figure F-I4. Regional soil survey information (Figure F-I5) shows that the sandiest soils in the Red River Basin are concentrated along the shoreline areas, approximately 20 miles from the proposed LPP Diversion Channel.

The Cities of Fargo and Moorhead sit at the center of the Red River Valley and the lake plain, although the beach ridges with higher slopes and sandier soils are closer to the Red River of the North on the eastern (Minnesota) side.

F-I1.2 GENERAL STREAM CHARACTERISTICS

F-I1.2.1 Wild Rice River (North Dakota)

The Wild Rice River (hereafter WRR) enters the lake plain near Wahpeton, ND and flows northward for more than 60 miles before joining the Red River of the North (hereafter RRN) approximately 10 miles south of the Cities of Fargo and Moorhead. Like the RRN, which it parallels, the WRR is highly meandering and has a very low gradient of approximately 0.7 feet per mile (0.01%). The banks of the WRR are typically

gently sloped from 4-5H:1V. There is one low-head dam on the WRR approximately 3.5 miles upstream of the confluence with the RRN.

F-I1.2.2 Sheyenne River

The Sheyenne River enters the lake plain near Kindred, ND and flows northward for approximately 75 miles before joining the RRN near Harwood. Note from Figures F-I1 to F-I5 that the Sheyenne River enters the lake plain relatively closer to the cities of Fargo and Moorhead than the other rivers in this study. According to Klausing (1968), the glacial Lake Agassiz deposits that form the Sheyenne River delta is located approximately 4 miles upstream from where the Sheyenne River crosses Kindred, and approximately 12 miles southwest from the upstream reach of the proposed LPP Diversion Channel. The Sheyenne River delta includes well-sorted deposits of thinly laminated silt and fine to medium sand, and it ranges in thickness from 0 to about 37 meters (120 feet). The reach of the Sheyenne River downstream of Kindred appears to be fluvial (not glacial) in origin, with few or no sand and gravel deposits.

Midway along the portion of the Sheyenne River within the lake plain (42 miles upstream of the RRN) the existing Horace/West Fargo Diversion channel routes a portion of the Sheyenne River water around Horace and areas of West Fargo during high flow conditions. Farther downstream, near Interstate 94 (29.5 miles upstream of the RRN) the West Fargo/Riverside Diversion channel routes additional flow around West Fargo and Riverside. Under the highest flow conditions all of the flow in the Sheyenne River is transferred to this diversion channel and direct flow down the main stem of the Sheyenne River is stopped entirely. The combined diversion channel rejoins the main stem of the Sheyenne River near the confluence with the Maple River, approximately 19 river miles upstream of the RRN.

The Sheyenne River is highly meandering, with a gradient of approximately 0.8 to 1.1 feet per mile (0.01% to 0.02%) upstream of the confluence with the Maple River. The river gradient steepens somewhat downstream of the diversion channels near the confluence with the Maple (to about 2.8 feet per mile or 0.05%), then returns to its previous range for the rest of the distance to the RRN. The banks of the Sheyenne River are steeper than those of the WRR, with the lower part of the banks typically sloped from 1-2H:1V. In addition to the two diversion structures that control flow into the Horace/West Fargo and West Fargo/Riverside Diversions, there is a low-head dam downstream of the Main Avenue bridge on the portion of the Sheyenne River bypassed by the West Fargo/Riverside Diversion.

F-I1.2.3 Maple River

The Maple River enters the lake plain near Leonard, ND and flows northwest for approximately 68 miles before joining the Sheyenne River near Riverside, ND. The Maple River delta is located at the edge of the lake plain, approximately 60 miles upstream from the confluence with the Sheyenne River (see Figure F-I4). This delta includes deposits of silt, sand, gravel and a few boulders (probably ice-rafted erratic), and it ranges in thickness from 0 to about 15 meters (50 feet). Klausing (1968) suggests that

the lower reach of the Maple River, near the proposed Diversion Channel, is of post-glacial origin and is characterized by the absence of sand and gravel deposits.

Similar to the WRR, the gradient of the Maple River is approximately 0.7 feet per mile (0.01%), and the banks are gently sloped from 3-5H:1V. There are two existing low-head dams on the Maple River, one upstream of the Interstate 94 crossing near Mapleton and one immediately upstream of the confluence with the Sheyenne River.

F-I1.2.4 Red River of the North

The RRN originates in the cities of Wahpeton and Breckenridge at the confluence of the Otter Tail and Bois de Sioux Rivers, approximately 187 miles upstream of Fargo and Moorhead. Similar to the Wild Rice and Maple Rivers, the gradient of this portion of the RRN is approximately 0.6 feet per mile (0.01%), and the banks are sloped near 3H:1V.

Brooks (2003a, 2005) indicates that the suspended sediment load of the RRN is composed primarily of silt with some clay. Paakh et al. (2006) state that the fine clay and silt sediments in the Red River Valley Lake Plain are easily suspended and tend to stay in suspension even during relatively low-flow conditions. Lauer et al. (2006) hypothesize that although some of the RRN sediment moves as bedload in the form of aggregated pellets of fine sand size, most of the bed sediment is transported in disaggregated form as silt and clay in suspension. Thus over engineering time scales, unless there is a significant change in the sediment supply from the watershed, potential changes of the RRN channel geometry would be associated with channel migration rather than with bed aggradation or degradation. However, Brooks (2003b) reports a very slow net expansion of RRN (meander) bends with channel migration rates in the order of 4 centimeters per year (1.6 inches per year) over the past 1,000 years. Therefore, the RRN can be considered a stable riverine system, an opinion that is shared by Prof. Gary Parker (personal communication), one of the world leading experts in river morphodynamics.

There are several existing low-head dams on the RRN in the area of interest, two upstream of the confluence with the WRR (near Christine and Hickson) and three in the vicinity of Fargo and Moorhead (Fargo South, Midtown, and North dams). The three dams in the cities of Fargo and Moorhead have been retrofitted with rock spillways to increase public safety and to improve fish passage up the RRN during low flow conditions. Similar retrofitting is proposed for the dams at Christine and Hickson, but has not been constructed.

F-I2.0 PREVIOUS STUDIES

The sections below provide a brief summary of several previous studies consulted for the current analysis from the literature and project data available for the RRN and its tributaries. This is not intended to be an exhaustive list of the available literature on the geomorphology of the RRN.

F-I2.1 SHEYENNE RIVER GEOMORPHOLOGY STUDY

A study of the geomorphology of the entire length of the Sheyenne River was prepared for the USACE in 2001 by West Consultants (West 2001). The study evaluated existing

conditions at various locations along the Sheyenne River and provided predictions of the behavior of the river under several hydrologic and Devils Lake discharge future scenarios.

Of relevance to the current study are the findings at two of the monitored sub-reaches near the proposed diversion channel for the RRN (see Figure F-I6). At both of these locations the stream bed was found to consist of primarily fine sand, with finer material (silt and clay) in the banks at the more downstream location. At both locations the surveyed river cross sections showed relatively steep stream banks with slopes ranging from 1-3H:1V. Analysis of the sediment transport processes at both sub-reaches implied the river channel has widened slightly over the last 50 years and will likely continue to widen to achieve equilibrium conditions (West 2001).

F-I2.2 SOUTH BRANCH OF THE BUFFALO RIVER SEDIMENT MODEL

In 2006 the St. Anthony Falls Laboratory at the University of Minnesota performed sediment production modeling on the watershed of the South Branch of the Buffalo River for Houston Engineering and the Buffalo-Red River Watershed District (Lauer et al. 2006). The study used a sediment production model developed by the US Department of Agriculture to quantify surface and bank erosion rates for various segments of the Buffalo River watershed.

Although the Buffalo River is outside of the project area for the current study (see Figure F-I6), it occupies a similar position in the watershed to the Sheyenne River system. The stream bed sediment collected on the Buffalo River and its tributaries shows generally fine-grained bed material (silt and clay) at sites located within the glacial lake plain. At sites located along the glacial beach line and in the uplands, the stream bed material is typically fine to medium sand. The sediment budget for the watershed indicates that the majority of the sediment load to the river system is generated in the steeper portions of the watershed, outside of the glacial lake plain. The majority of the sandy material is deposited quickly once the streams enter the lake plain and is not transported to the confluence with the RRN.

F-I2.3 SOUTHSIDE FLOOD CONTROL STUDY

As part of a larger study performed by Moore Engineering of potential flood control measures on the RRN in south Fargo, Barr Engineering assessed the geomorphology of the river in 2008 (Barr 2008).

A general review of literature on the RRN and analysis of the available suspended sediment data indicated that the river is in morphodynamic equilibrium, with no appreciable net aggradation or erosion. Eight bed sediment samples were collected for this project downstream of the confluence of the RRN and the WRR (see Figure F-I6). With the exception of the sampling location nearest to the mouth of the WRR, all samples indicate that the bed of the RRN consists of silt and clay with very little sand (less than 10% by weight). The slightly greater quantity of sand in the remaining sample is interpreted as being delivered by the WRR but deposited in a short stretch of the RRN.

These observations confirm the interpretation of the RRN as being a stable riverine system.

F-I3.0 SEDIMENT TRANSPORT DATA

The sediment transport data considered in this analysis was collected by the USGS at various locations in the study area, as described below.

F-I3.1 USGS SEDIMENT SAMPLING (PRE-2010)

Historical sediment data is available from the USGS for the RRN and several of its tributaries. The data was generally collected at established gage locations, and includes observations throughout the calendar year and for a range of flow conditions. See Figure F-I6 for the monitoring locations considered in this analysis, and Table F-I1 for a summary of the available data.

The historical sediment data used in this study includes numerous measurements of depth-integrated suspended sediment concentration (SSC) and the fraction of the suspended material that is finer than sand. Locations with only total suspended solids (TSS) data or with only a single measurement were not included in this analysis. Complete gradations of suspended sediment and bed material are available for a limited set of monitoring locations and dates, primarily in the late 1970s.

F-I3.2 USGS SEDIMENT SAMPLING (SPRING 2010)

The USACE contracted with the USGS to determine sediment concentrations, loads, and particle size distributions at six sites in the RRN and its tributaries during the spring high flow of 2010 (Blanchard et al. 2010). See Figure F-I6 for the monitoring locations. Sampling began on March 24, 2010 and the last measurement was taken on April 7, 2010; data was collected only when the rivers were not covered by ice, resulting in different numbers of samples for the different locations.

The sediment data collected in 2010 included the following measurements during the peak and recession of spring flood waters: depth-integrated suspended sediment concentration (SSC), suspended sediment concentration at discrete points in the water column, suspended sediment gradation, bedload magnitude and gradation, and bed material gradation. Figures and tables presented in the 2010 USGS report will not be reproduced here but should be used for reference.

A second season of sediment data collection at the same sites is planned by the USGS for the spring high flow of 2011. This additional data will be used in further analysis of the proposed diversion project and in the concurrent study discussed below.

F-I3.3 DATA CONSIDERED BY CONCURRENT STUDIES

Concurrent with this feasibility study the USACE has contracted with West Consultants for a detailed study of the geomorphology of the RRN and its tributaries within the project area. This study will involve field data collection including channel cross-sectional surveys and bank and bed material sampling (some of which is referenced in

Section 4.1) as well as review of historic aerial photos, surveys, construction documents, hydrologic models, and sediment borings. This data will be used to develop a detailed morphological classification of each stream reach, assess the stability of the channel and the current sediment transport conditions, and evaluate the effects of future conditions with the proposed diversion project in place.

F-I4.0 CHARACTERIZATION OF EXISTING CONDITIONS

F-I4.1 SEDIMENT DATA ANALYSIS

F-I4.1.1 Wild Rice River (North Dakota)

As discussed in Section F-I1.2, the WRR is a meandering, low-gradient river that flows across the lake plain for many miles before approaching the monitoring locations near the cities of Fargo and Moorhead. The fine soils in the lake plain and along the river banks contribute sediment to the river that is transported primarily in suspension because of its low settling velocity. In this regard conditions on the WRR at the monitoring locations shown in Figure F-I6 are typical of many of the other streams within the glacial lake plain.

Historic monitoring data from the USGS shows that the SSC in the WRR is typically below 100 mg/L (the median SSC is 42 mg/L), although concentrations have been observed as high as 540 mg/L (Figure F-I7). Although there are limited historic measurements of the suspended sediment particle size in the WRR, the available data consistently show that over 90% of the suspended sediment is finer than sand (i.e. silts and clays). The 2010 USGS sampling data for SSC shows results similar to the historic data (over 90% finer than sand), albeit with a slightly higher range of observed SSC (136 to 186 mg/L). Measurements of SSC at different points in the water column show no significant vertical differences in SSC or in the fraction of the suspended material that is finer than sand.

Both the historic and the 2010 USGS data show that the bed material in the WRR is medium sand, with a typical median particle size of approximately 1 mm (Figures F-I9 and F-I10). There are two observations in the historic record of more finely grained bed material, both under lower-flow conditions in the summer (as opposed to during spring floods). The material transported as bedload is fine to medium sand, similarly sized to the majority of the samples of bed material (Figure F-I11).

The SSC and bedload data collected by the USGS in the spring of 2010 show that the vast majority of the sediment load in the WRR, over 99.9% by weight, is transported in suspension (Table F-I2). The total sediment load in the WWR in the spring of 2010 peaked at or possibly before the peak flow, driven by the highest measured SSC at and before the peak of the hydrograph.

In summary, these data indicate that the sediment load carried by the WRR in the project area is overwhelmingly fine suspended material, which is likely transported long distances from its origin in overland and bank erosion. Some of this material may settle to the bed of the river during periods of lower river velocity, but typical high flows are likely sufficient to re-suspend any settled material, leading to minimal net change in channel dimensions over time.

F-I4.1.2 Sheyenne River

Like the WRR, the Sheyenne River is a meandering, low-gradient river that transports sediment primarily in suspension. However, as noted in Section F-I1.2, the Sheyenne has several characteristics that make it distinct from the other rivers considered in this analysis. The banks of the Sheyenne River are generally steeper than those of the WRR, and the gradient of the Sheyenne River is slightly greater. Also significant to this study is the fact that flows in the Sheyenne River are managed by two existing diversion projects, which significantly alter the flow regime through portions of the river channel.

The highest SSC levels observed by the USGS in the 2010 study were on the Sheyenne River just upstream of the Horace/ West Fargo Diversion, ranging from 476 to 1120 mg/L with average values near 700 mg/L. The suspended sediment was observed to contain more coarse material than at the WRR, with fine sand representing between 15 and 30% of the material in suspension. The historic USGS data on the Sheyenne River at Kindred (i.e. approximately 30 miles upstream of the 2010 sampling location) show generally lower SSC (median 73 mg/L and maximum 880 mg/L), although the fraction of fine material was observed to vary even more widely (Figures F-I7 and F-I8). As for the WRR, the 2010 measurements of SSC at different points in the water column show no significant vertical differences in SSC or in the fraction of the suspended material that is finer than sand.

The 2010 USGS data indicates that the bed material in the Sheyenne River at Horace is fine to medium sand, with a typical median particle size of approximately 0.4 mm (Figure F-I13). The material transported as bedload is similarly sized (Figure F-I14). These characterizations of bed material are somewhat coarser than indicated by the historic data at Kindred (typically fine sand as shown in Figure F-I12).

As observed for the WRR, the 2010 measurements show that the vast majority of the sediment load in the Sheyenne River, over 99.9% by weight, is transported in suspension (Table F-I2). The total sediment load in the spring of 2010 peaked at the peak flow, driven by the highest measured SSC at the peak of the hydrograph.

The 2010 USGS data collected on the Sheyenne River at Horace (downstream of the Horace/West Fargo Diversion) show SSC and bed material characteristics that are similar to the upstream Sheyenne River. The observed SSC downstream of the diversion is nearly identical to that upstream (average values near 700 mg/L), and the suspended material is slightly finer than in the upstream Sheyenne River (78 to 83% finer than sand, compared to 70 to 84% above the diversion). The bed and bedload material in the

Sheyenne River downstream of the diversion are slightly finer than in the Sheyenne River upstream of the diversion (Figures F-I15 and F-I6).

The sediment load data for the Sheyenne River downstream of the Horace/West Fargo Diversion indicate that the suspended material is diverted from the main Sheyenne River in proportion to the flow diversion (Table F-I2), and with similar timing. The small quantity of bedload, however, was not diverted proportionally: the measured bedload downstream of the diversion was approximately one-quarter of that measured upstream, while the total flow passing into the protected areas was approximately half of that in the upstream Sheyenne River. This observation may indicate that a portion of the bedload material is deposited in the Sheyenne River upstream of the diversion structure, which could contribute to the slightly coarser bed material upstream of the diversion (compare Figure F-I13 with Figure F-I15). The bed material samples collected in 2010 by West Consultants along the Horace/West Fargo diversion channel (see Figure F-I6 for locations) show gradations generally similar to the bed material sampled by the USGS in the Sheyenne River downstream of the diversion, suggesting that coarser bedload is not transported into the diversion channel. Any deposition of coarser bedload upstream of the diversion structure is likely minor.

Like the WRR, the data indicate that the sediment load carried by the Sheyenne River is primarily fine material. The higher SSC levels and slightly coarser material suspended in the Sheyenne River suggest that there is more active bank erosion in the Sheyenne River system than in the WRR, providing a source of more sediment close to the monitoring locations. This finding is consistent with the results of the West 2001 geomorphology study, which predicted that the lower portions of the Sheyenne River would slowly widen by eroding the steep banks.

The available sediment data does not suggest that the geomorphology of the Sheyenne River has been significantly impacted since the construction of the Horace/West Fargo and West Fargo/Riverside diversions. Bed and bank samples collected by the USGS and West Consultants in 2010 show similar gradations on the Sheyenne River upstream and downstream of the diversion channels: bed material of fine sand and banks of primarily silt and clay. The West 2010 data for the Horace/West Fargo Diversion channel shows some evidence of deposition of fine material at the upstream end of the diversion channel, likely left behind as floodwaters recede and flow ceases in the diversion channel. However, there is no indication of significant net deposition of fine material in the main Sheyenne River as a result of the diversion projects.

F-I4.1.3 Maple River

The available cross-sectional and sediment transport data indicates that the Maple River is very similar to the WRR. Both rivers have similar gradients and bank slopes, and both rivers traverse the lake plain for significant distances before approaching the study area. There is no historic data available for the Maple River (see Table F-I1), so the discussion here is limited to analysis of the 2010 USGS sediment data.

The 2010 USGS monitoring data shows that SSC in the Maple River is somewhat higher than in the WRR, with a range of 172 to 325 mg/L (median 211 mg/L). Similar to the WRR, the 2010 data show that over 90% of the suspended sediment in the Maple River is finer than sand. Measurements of SSC at different points in the water column show no significant vertical differences in SSC or in the fraction of the suspended material that is finer than sand, except for one mid-depth sample that may represent equipment malfunction.

Similar to the WRR, the 2010 USGS data shows that the bed material in the Maple River is medium sand, with a typical median particle size of approximately 0.7 mm (Figure F-I17). The material transported as bedload is medium sand, slightly coarser than the majority of the samples of bed material with a typical median particle size of approximately 1 mm (Figure F-I18).

Like the WRR and the Sheyenne River, the majority of the sediment load in the Maple River is transported in suspension. Bedload was a slightly higher portion of the total load in the Maple River, but suspended material still accounted for 99.8% by weight of the sediment transport (Table F-I2). The total sediment load in the Maple River in the spring of 2010 peaked at or possibly before the peak flow, although there was a secondary peak of high SSC observed as the flood waters receded.

In summary, these data indicate that the sediment load carried by the Maple River in the project area is overwhelmingly fine suspended material, which is likely transported long distances from its origin in overland and bank erosion. Some of this material may settle to the bed of the river during periods of lower river velocity, but typical high flows are likely sufficient to re-suspend any settled material, leading to minimal net change in channel dimensions over time.

F-I4.1.4 Red River of the North

Like its tributaries, the RRN transports the large majority of its sediment load as suspended fine material (silts and clays). The historic USGS data indicates that SSC levels and their variability both increase moving downstream from Hickson (median SSC 72 mg/L) to Fargo (median SSC 112 mg/L), as shown in Figure F-I7. The majority of samples at all three historic USGS monitoring locations on the RRN show that greater than 90% of the suspended material is finer than sand, although there are occasional (11 out of a combined 160 samples) outliers with higher sand fractions (Figure F-I8). In 2010, the monitoring locations on the RRN had the lowest SSC observed, with concentrations ranging from 74 to 156 mg/L and median values from 93 to 116 mg/L. As in the other rivers in this study, there are no significant or consistent vertical differences in SSC or in the fraction of the suspended material that is finer than sand.

The bed material sampled by the USGS in 2010 at both locations on the RRN was medium sand, with typical median particle size of approximately 1 mm (Figures F-I20 and F-I22). The material transported as bedload is similarly sized, although slightly finer and with a very sporadic distribution for the samples collected on the RRN in Fargo

(Figures F-I21 and F-I23). These observations compare well with the historic bed sediment gradations measured at the USGS gage on the RRN at Hickson (Figure F-I19).

The sandy bed material collected by the USGS from the RRN is somewhat inconsistent with the results of other sampling efforts in the sampling area. As described in Section 2.3, samples collected for the Southside Flood Control Study (Barr 2008) show primarily fine-grained bed material with less than 10% sand. In addition, recent sampling by West Consultants as a part of the study described in Section F-I3.4 show predominantly fine bed material at locations both upstream and downstream of the confluence of the RRN with the WRR (see Figure F-I6 for locations). Additional bed samples by West Consultants at locations in Fargo show mostly fine material, although one sample contains 47% sand.

The observed differences in bed material gradation may simply be due to spatial heterogeneity along the river bed or they may be a reflection of the bias in bed sediment sampling locations. The USGS samples (both in 2010 and in the historic data set) are collected from bridges, and the bed material may be affected by road runoff, channel armoring, and constricted flows and may not represent conditions in the remainder of the river channel. In addition, the monitoring locations near Hickson (historic) and Christine (2010) are both upstream of low-head dams on the RRN, which may cause atypical deposition in these areas. Further study, including borings of river bed sediment profiles, is warranted to better characterize the conditions in the study area.

Regardless of the exact nature of the RRN bed material, the data collected by the USGS in 2010 demonstrates that bedload is a small fraction of the total sediment transport in the RRN (Table F-I2). At the upstream site near Christine, 99.5% of the measured total sediment load was from suspended material, dominated by a single high measurement of bedload at the peak of the hydrograph. At the downstream site in Fargo, the bedload was of similar magnitude to the other sampling locations and the suspended sediment accounted for over 99.9% of the total sediment transport. At both locations the total sediment load peaked at or just before the peak flow, and the peak SSC was observed as the flood waters receded.

Like the tributaries considered in this analysis, sediment transport in the RRN is dominated by the movement of suspended fine material. This suspended material is well-distributed throughout the vertical water column and is transported through the study area with minimal interaction with the stream bed.

F-I4.2 SEDIMENT BALANCE ANALYSIS

Total sediment loading for the period of monitoring is reported in the summary of the USGS 2010 sediment data collection (Blanchard et al. 2010). Total flows and sediment loads for concurrent periods are shown in Table F-I2 to facilitate interpretation of the sediment balance for several of the study rivers.

F-I4.2.1 Red River of the North and Wild Rice River

The data collected in 2010 on the RRN and WRR is sufficient to perform a simple flow and sediment balance for the portion of the RRN upstream of the confluence with the Sheyenne River. In the absence of other data, the assumption for this analysis is that no additional flow or sediment enters the WRR or RRN between the upstream monitoring locations (WRR near St. Benedict and RRN near Christine) and the downstream location (RRN near Fargo).

As shown in Figure F-I24, both the flow and sediment balance for this system are reasonably close, given the simplifying assumptions made above. The total flow measured at the upstream gaging locations from March 18, 2010 to March 31, 2010 was 18,810 million cubic feet (MCF). The total flow measured in Fargo during the same period was 19,810 MCF, a difference of approximately 5%. The total sediment load (suspended and bedload) measured at the upstream gaging locations was 74,250 tons; the total sediment load measured in Fargo was 72,110, a difference of approximately 3%.

This simple sediment balance indicates that, as expected based on the discussion above, the sediment load in the RRN through the cities of Fargo and Moorhead is neither increasing or decreasing. The RRN does not appear to be gaining sediment (via erosion) or losing sediment (via aggradation) over this reach. This corroborates the description of the RRN as a stable riverine system, with sediment loading from fine suspended material that is primarily washed through the system.

F-I4.2.2 Sheyenne River and Horace/West Fargo Diversion

A similar sediment balance can be performed for the Sheyenne River system at the Horace/West Fargo Diversion. Because there was not a monitoring location on the diversion channel itself, the flow and sediment balance must be assumed to close. The flow and sediment load on the Horace/West Fargo Diversion can then be calculated.

As shown in Figure F-I25, the Horace/West Fargo Diversion is calculated to transport just over half (52-53%) of the total flow and sediment load from the upstream Sheyenne River, with the remainder passing into the protected area in the Sheyenne River. The total flow measured on the Sheyenne River upstream of the diversion from March 24, 2010 to April 7, 2010 was 5,340 MCF, while the total flow measured downstream of the diversion channel during the same period was 2,580 MCF. The total sediment load (suspended and bedload) measured on the Sheyenne River upstream of the diversion was 119,630 tons; the total sediment load measured downstream of the diversion channel was 56,380.

The sediment load passed from the main Sheyenne River to the protected area (through the diversion structure) is almost exactly proportional to the transmitted flow. This is a result of the nearly-identical measured SSC at the monitoring locations upstream and downstream of the diversion structure, and the minimal contribution of bedload to the total sediment load. Note in Table F-I2 that although approximately half of the flow and total sediment load are passed to the diversion channel, the measured bedload in the Sheyenne River downstream of the Horace/West Fargo Diversion is only 23% of that

measured upstream of the diversion. This result may indicate that some of the small quantity of bedload material is preferentially retained in the Sheyenne River upstream of the diversion structure.

F-I5.0 POTENTIAL PROJECT IMPACTS

The proposed project to divert flood waters from the RRN and its tributaries around the cities of Fargo and Moorhead involves the construction of a diversion channel and a number of hydraulic structures. Water from the RRN and selected tributaries (Wild Rice, Sheyenne, and Maple Rivers) will be passed into the protected area, with the portion of the flow diverted depending on the magnitude of the flood. This diversion will effectively reduce the flood flows through the protected area for events greater than the 2-yr to the 10-yr return period event, potentially impacting the sediment transport and geomorphology of the streams in the protected area.

F-I5.1 IMPACTS TO SEDIMENT TRANSPORT

The proposed changes to the flow regime in the RRN and its tributaries within the protected area will change the capacity of the rivers to transport sediment. However, because all of the affected rivers are dominated by the transport of fine suspended material (as shown in this analysis), the diversion of a fraction of the river flow is expected to divert a proportional fraction of the total sediment load transported as suspended sediment. This suspended sediment, being fine-grained with very slow settling velocities, can be expected to move through the diversion system and return to the RRN downstream of Fargo and Moorhead. The changes to river flows within the protected area are not expected to be sufficient for the remaining fraction of the suspended sediment to settle. Therefore, the total sediment budget of the system will be essentially unchanged.

The Horace/West Fargo Diversion of the Sheyenne River provides an example of the potential maximum impacts that can be expected from the proposed diversion. As discussed in this analysis, the Sheyenne River system has coarser bed material and more coarse suspended sediment than the other affected rivers, meaning that the impacts of diversion on sediment transport would be expected to be the most significant. However, even the somewhat coarser suspended sediment in the Sheyenne River is passed into the protected area and to the diversion channel in proportion to the flow, validating the description of expected future conditions proposed above. Although the bed material of the Sheyenne River (and most of the other rivers in this study) appears to consist of fine or medium sand, this more coarse material is not transported in significant quantities through the system. What little bedload sediment is in the Sheyenne River appears to move primarily past the diversion structure and into the protected area, although some sediment may be retained upstream of the diversion structure.

F-I5.2 IMPACTS TO GEOMORPHOLOGY

The potential project impacts to the geomorphology of the RRN and its tributaries from the discussed changes in the sediment transport regime are expected to be negligible. The RRN is a stable riverine system, neither aggrading or degrading, with sediment

transport primarily in suspension. These characteristics are not expected to change significantly following implementation of the proposed diversion works.

Again, the existing Horace/West Fargo Diversion of the Sheyenne River provides an example of the potential maximum impacts to geomorphology expected from the proposed diversion. The presence of the diversion channel and diversion control structures does not appear to have altered the geomorphic behavior of the river within the protected area. Recent modeling (West 2001) has shown that monitored locations on the lower Sheyenne River experienced only slight adjustments to channel shape over more than 50 years, including periods after the construction of the two Sheyenne River diversion projects, which have been in place for nearly 20 years. The slight widening of the river in these areas was expected based on the hydrology of the larger Sheyenne River, and was not interpreted as a response to local changes in the flow regime caused by the diversion. It is expected that the diversion of additional flood water proposed by the current project will have a similarly small impact on the geomorphology of the Sheyenne River (and by extension the other rivers in the study). This initial conclusion will be examined more thoroughly in the ongoing geomorphic study of the area rivers being performed by West Consultants.

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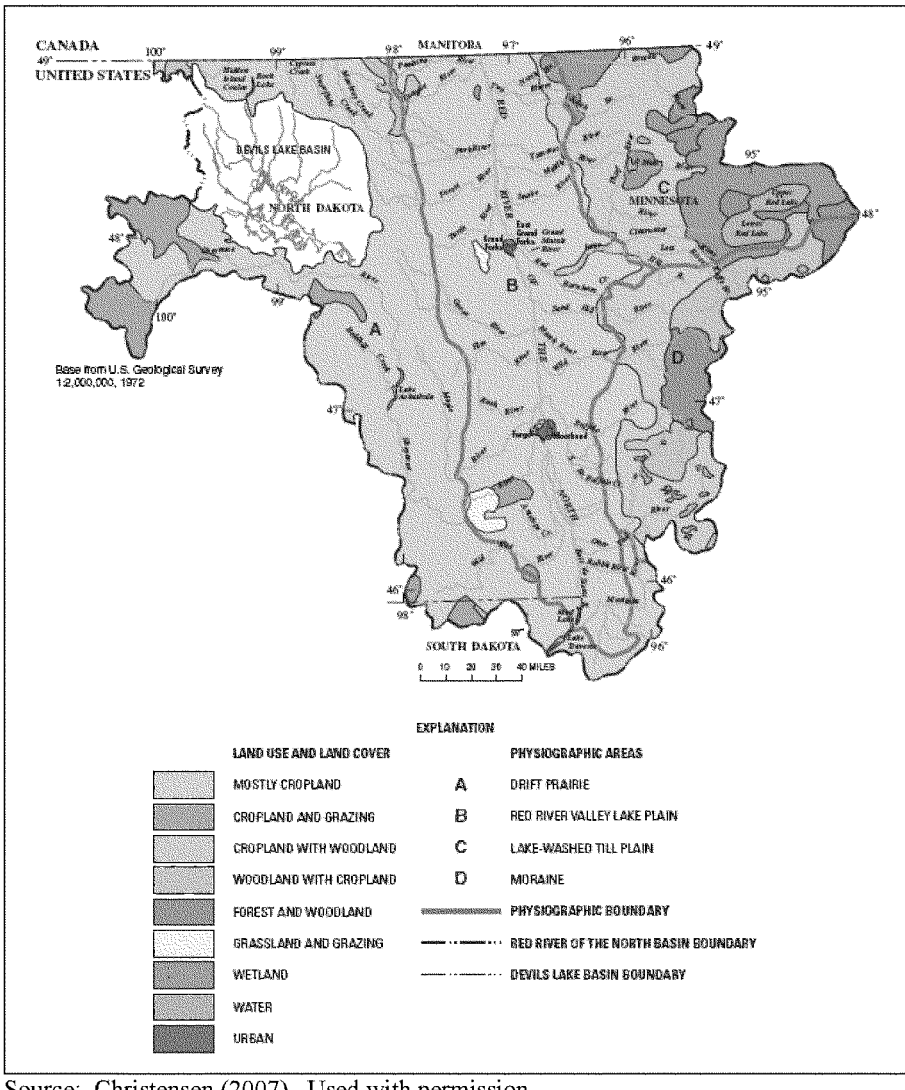
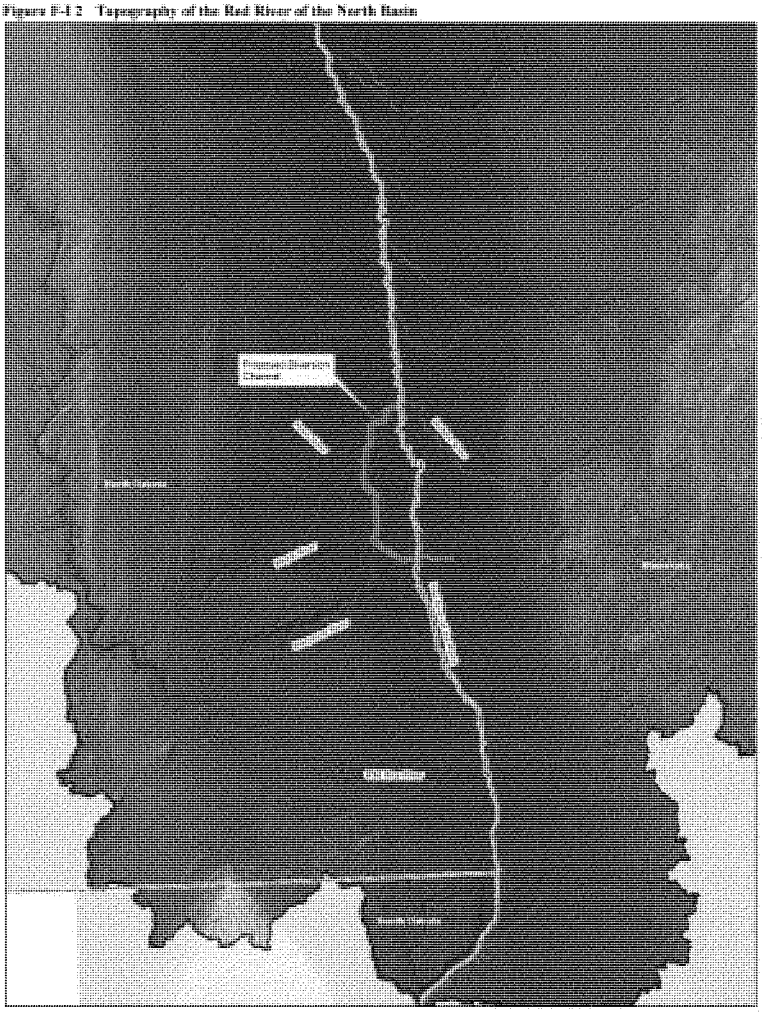
Figure F-I1 Land Use and Land Cover and Physiographic Areas in the Red River of the North Basin



Source: Christensen (2007). Used with permission.

Figure F-12 Topography of the Red River of the North Basin

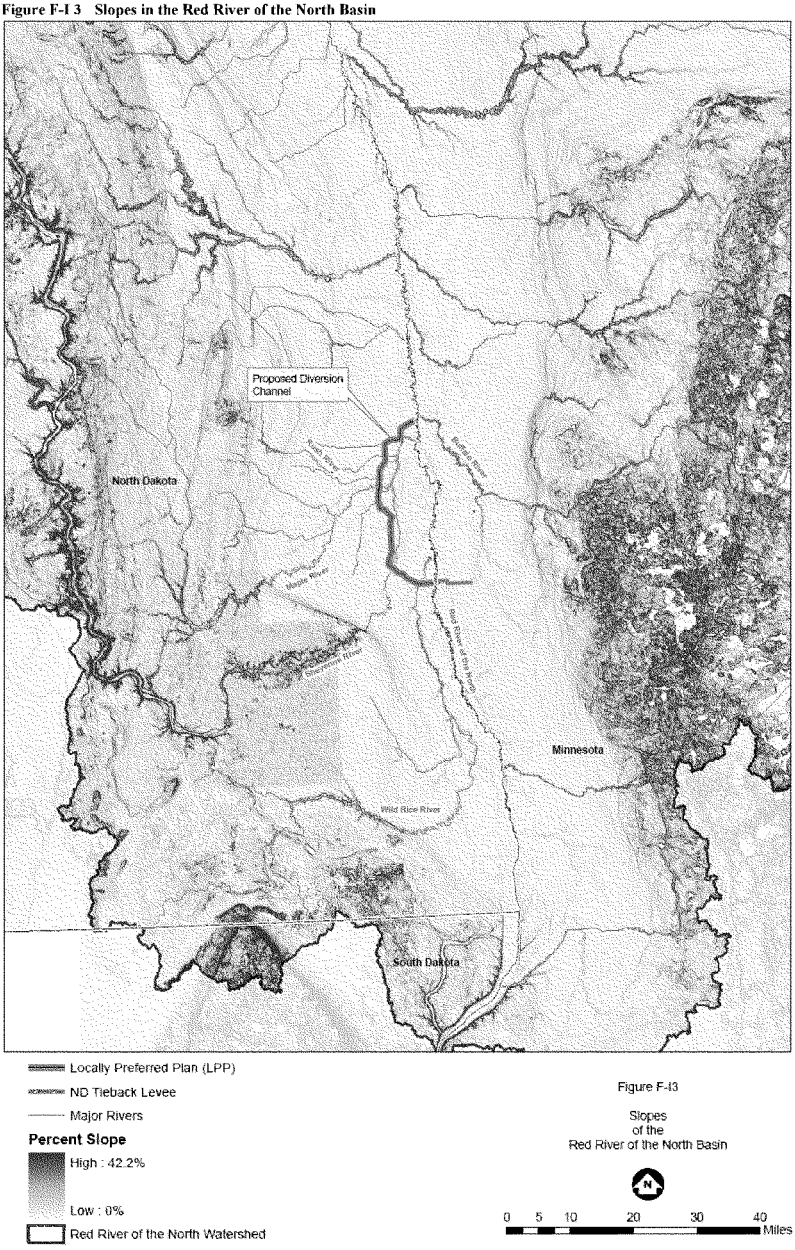


Locally Preferred Plan (LPP)
ND Tieback Levee
Major Rivers
Red River of the North Watershed

Figure F-12
Topography
of the
Red River of the North Basin

0 5 10 20 30 40 Miles

Figure F-I 3 Slopes in the Red River of the North Basin



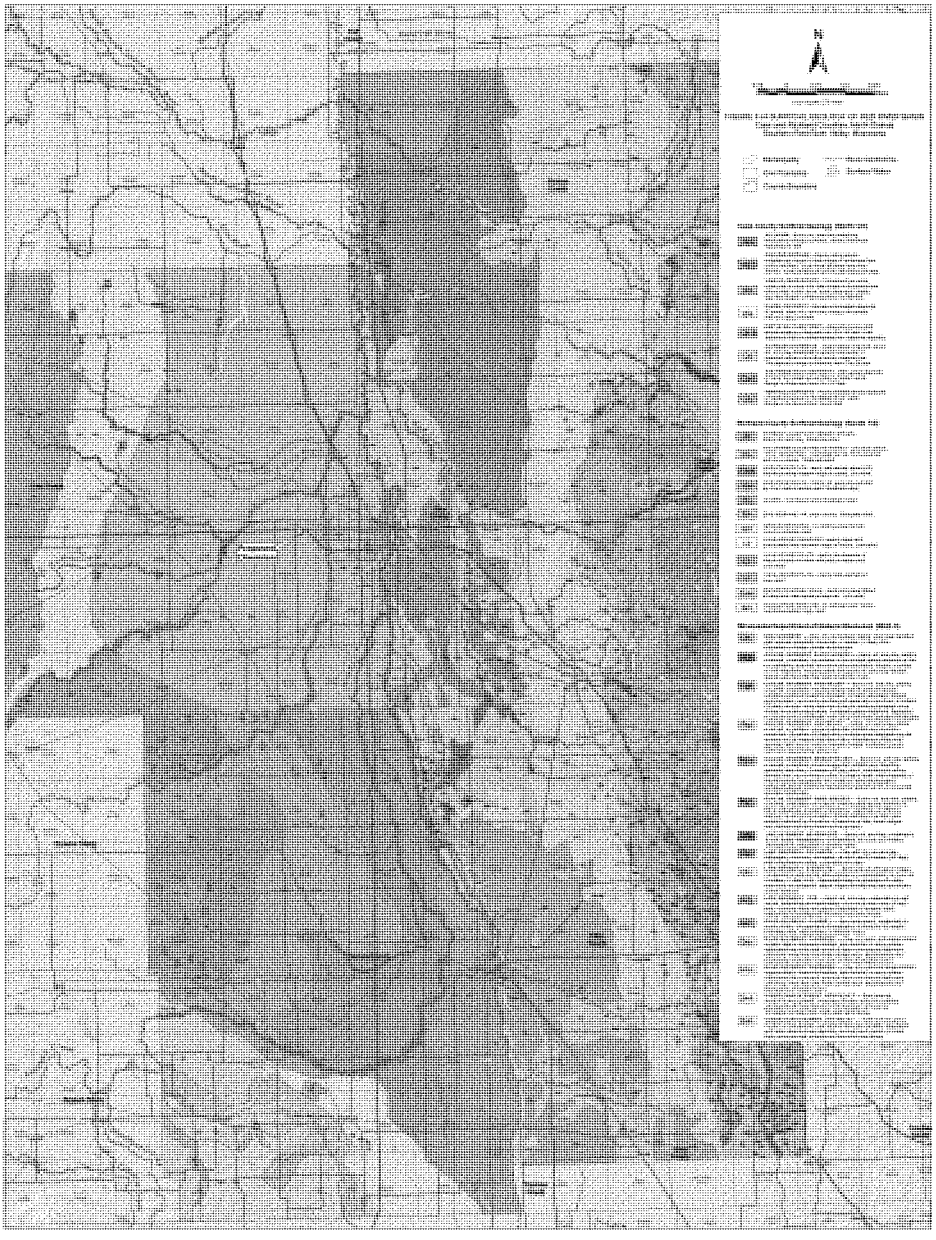


Figure F-I 5 Percent Sand in Surface Soils

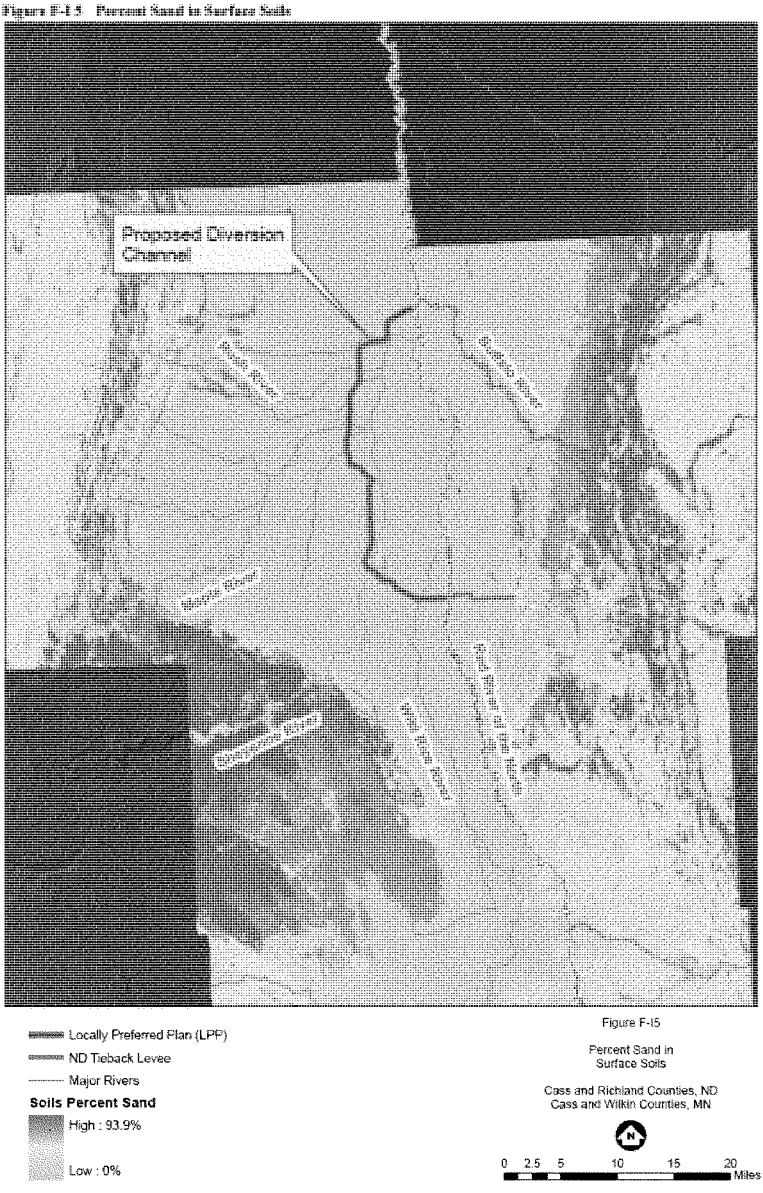


Figure F-16 Locations of Available Sediment Data

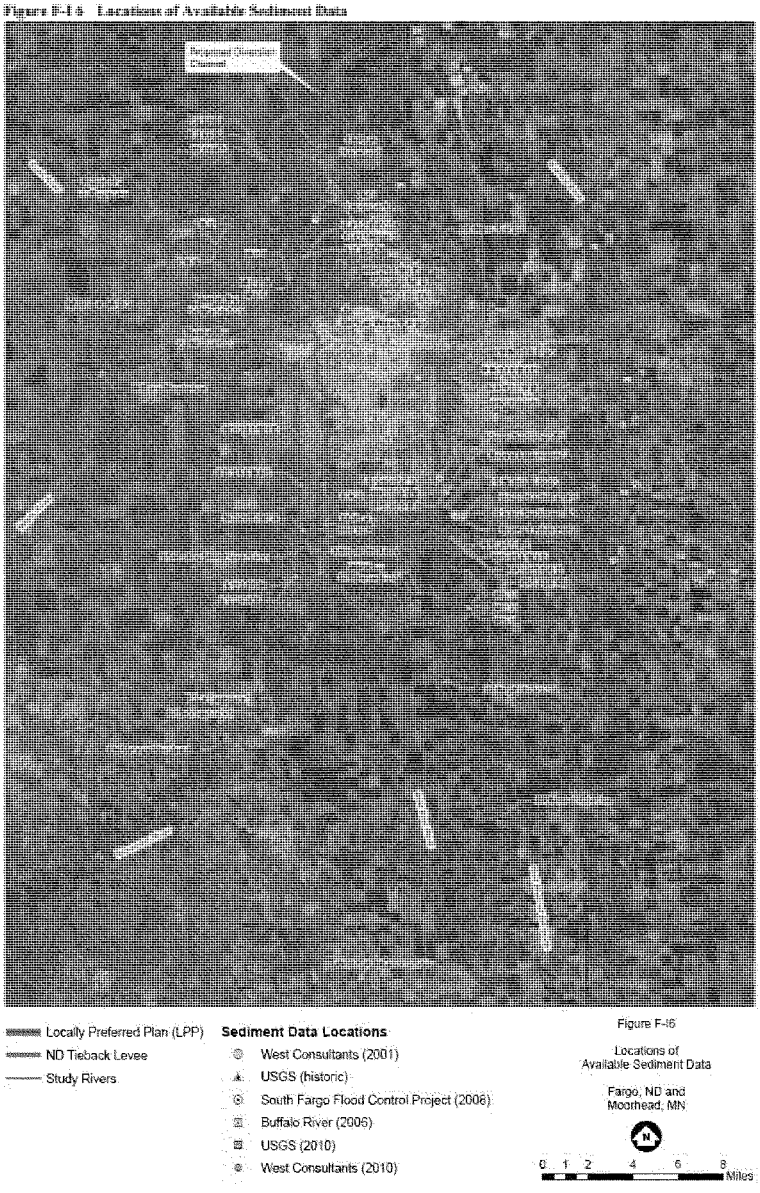


Table F-I1 Available Historic Sediment Data

USGS Site ID	Site Name	Suspended Sediment Concentration	Suspended Sediment % Finer than Sand	Suspended Sediment Gradation	Bed Sediment Gradation	Used in this Analysis
05053000	Wild Rice River near Abercrombie, ND	1975-1981, 1994 (n = 62)	N/A	1975-1979, 1994 (n = 9)	N/A	YES
05059000	Sheyenne River near Kindred, ND	1975-1995, 2001 (n = 112)	1975-1995, 2001 (n = 106)	1977-1978, 1980, 2001 (n = 11)	1976-1978 (n = 12)	YES
05060400	Sheyenne River at Harwood, ND	1997-1999 (n = 21)	1997-1999 (n = 20)	1997-1999 (n = 7)	N/A	YES
05060000	Maple River near Mapleton	1975 (n = 1)	N/A	N/A	N/A	NO
05051522	Red River of the North at Hickson	1975-1981, 1997-1999, 2003 (n = 102)	1975-1980, 1997-1999, 2003 (n = 45)	1978-1980, 1998-1999 (n = 10)	1976, 1978-1980 (n = 7)	YES
05053800	Red River of the North above Fargo	1994-1999 (n = 38)	1994-1999 (n = 38)	1998-1999 (n = 4)	N/A	YES
05054000	Red River of the North at Fargo	1975, 2001, 2003-2010 (n = 78)	1975, 2001, 2003-2010 (n = 77)	N/A	N/A	YES

Table F-I2 2010 Observed Sediment Loading

Site Name	Time Period[1]	Total Flow (million ft^3)	Total Suspended Sediment Load (tons)	Total Bedload (tons)[2]	Total Sediment Load (tons)
Wild Rice River near St. Benedict	March 18, 2010 - March 31, 2010	8,780	43,260	31.8 (0.07%)	43,300
Sheyenne River above Sheyenne River Diversion near Horace	March 24, 2010 - April 7, 2010	5,340	119,590	40.7 (0.03%)	119,630
Sheyenne River at Horace	March 24, 2010 - April 7, 2010	2,580	56,370	9.3 (0.01%)	56,380
Maple River below Mapleton	March 19, 2010 - April 6, 2010	4,660	31,520	70.9 (0.2%)	31,600
Red River of the North near Christine	March 18, 2010 - March 31, 2010	10,030	30,780	171 (0.6%)	30,950
Red River of the North near Fargo	March 18, 2010 - March 31, 2010	19,810	72,080	27.6 (0.04%)	72,110

[1] Time period shown does not represent the complete monitoring period for all sites. The periods shown represent concurrent data on the WRR and RRN (March 18, 2010 to March 31, 2010) and on the Sheyenne River above and below the Sheyenne River Diversion (March 24, 2010 to April 7, 2010).

[2] Percentage values represent bedload as a fraction of total sediment load for the time periods shown.

Figure F-17 Suspended Sediment Concentrations (historic)

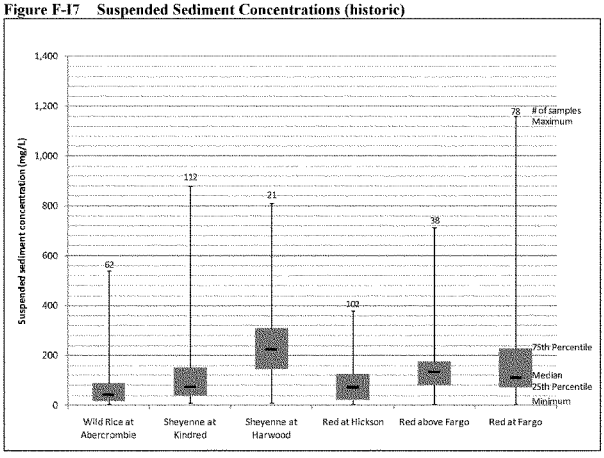


Figure F-18 Suspended Sediment Percent Finer than Sand (historic)

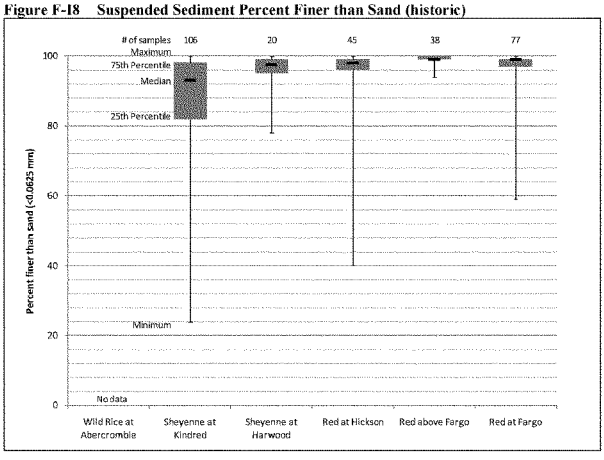


Figure F-I9 Bed Sediment Gradation (historic) – Wild Rice River near Abercrombie

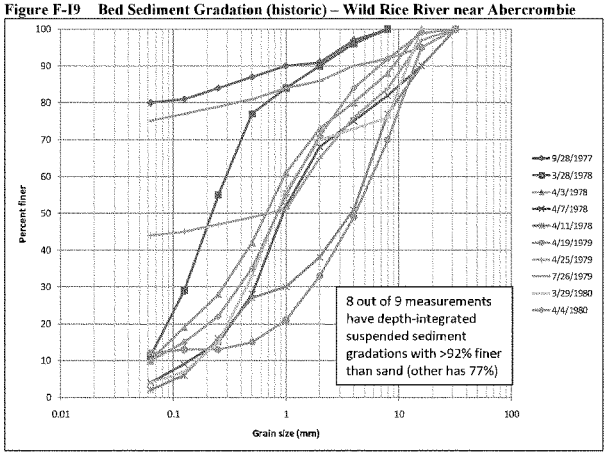


Figure F-I10 Bed Sediment Gradation (2010) – Wild Rice River near St. Benedict

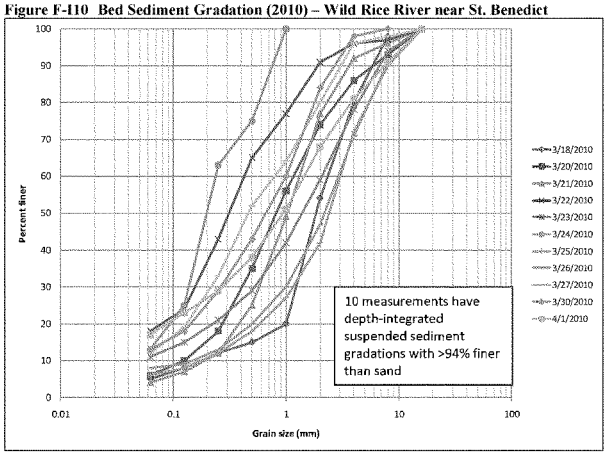


Figure F-I11 Bedload Gradation (2010) – Wild Rice River near St. Benedict

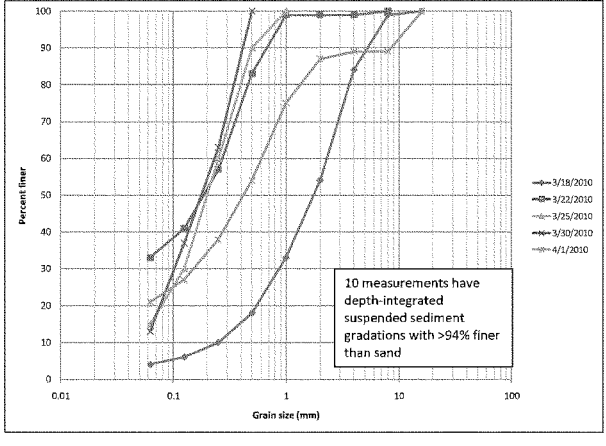


Figure F-I12 Bed Sediment Gradation (historic) – Sheyenne River near Kindred

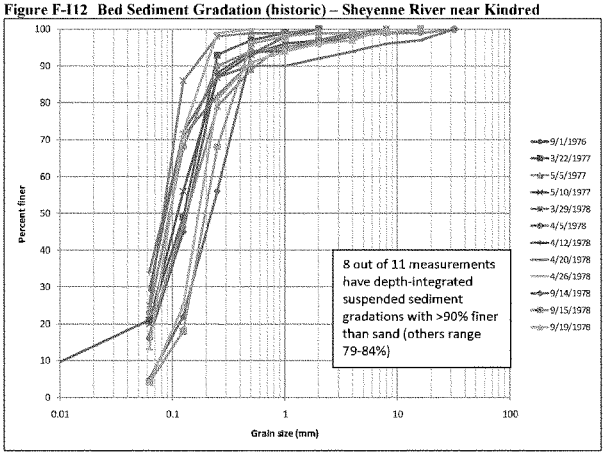


Figure F-I13 Bed Sediment Gradation (2010) – Sheyenne River above Sheyenne River Diversion near Horace

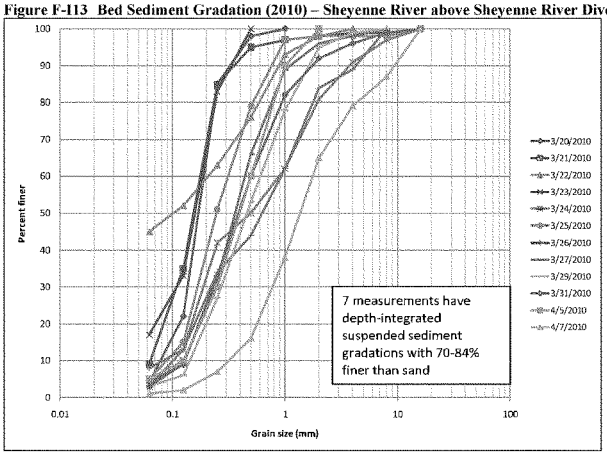


Figure F-I14 Bedload Gradation (2010) – Sheyenne River above Sheyenne River Diversion near Horace

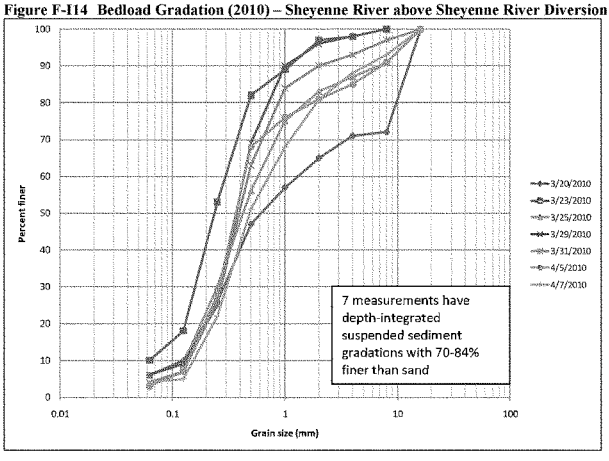


Figure F-I15 Bed Sediment Gradation (2010) – Sheyenne River at Horace

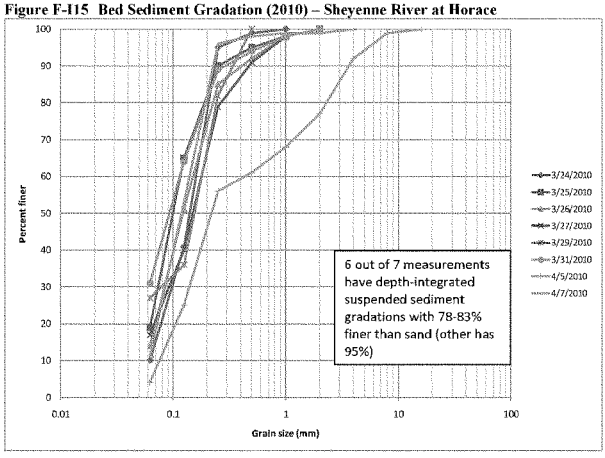


Figure F-I16 Bedload Gradation (2010) – Sheyenne River at Horace

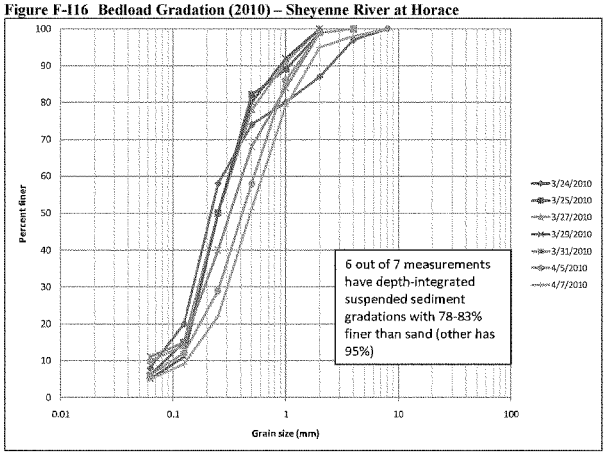


Figure F-I17 Bed Sediment Gradation (2010) – Maple River below Mapleton

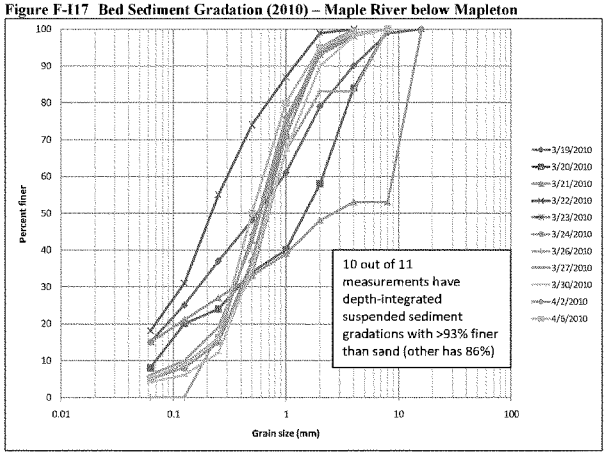


Figure F-I18 Bedload Gradation (2010) – Maple River below Mapleton

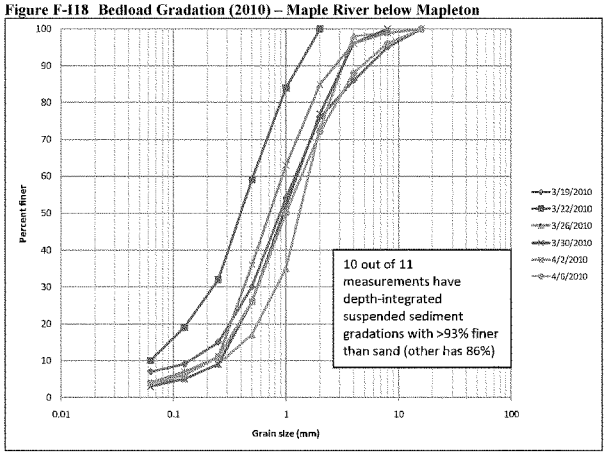


Figure F-I19 Bed Sediment Gradation (historic) – Red River of the North near Hickson

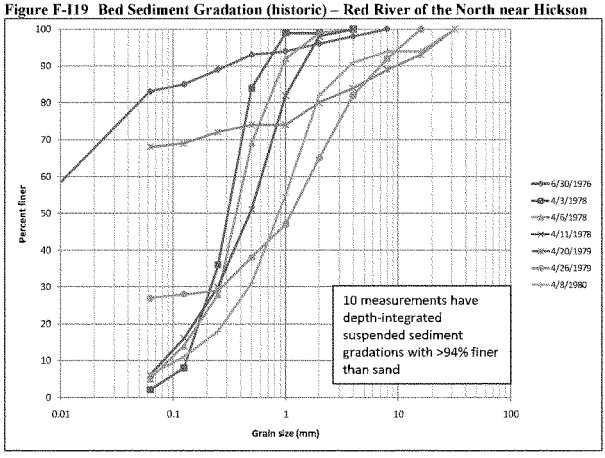


Figure F-I20 Bed Sediment Gradation (2010) – Red River of the North near Christine

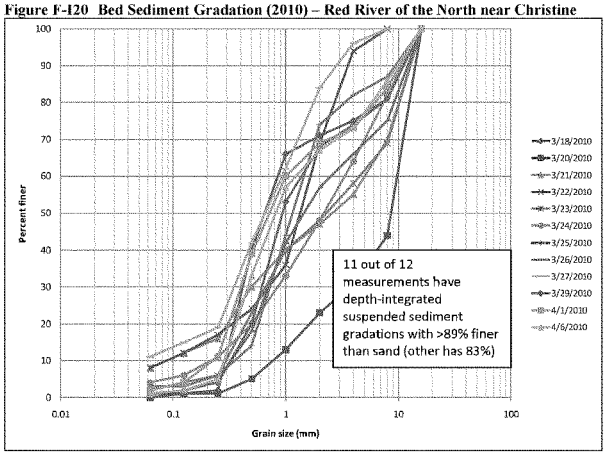


Figure F-I21 Bedload Gradation (2010) – Red River of the North near Christine

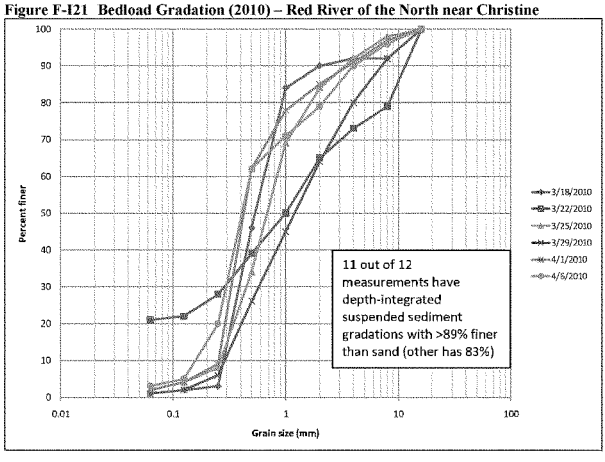


Figure F-I22 Bed Sediment Gradation (2010) – Red River of the North at Fargo

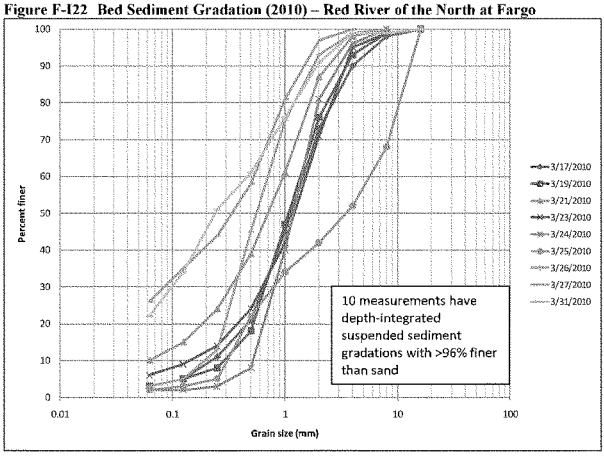


Figure F-I23 Bedload Gradation (2010) – Red River of the North at Fargo

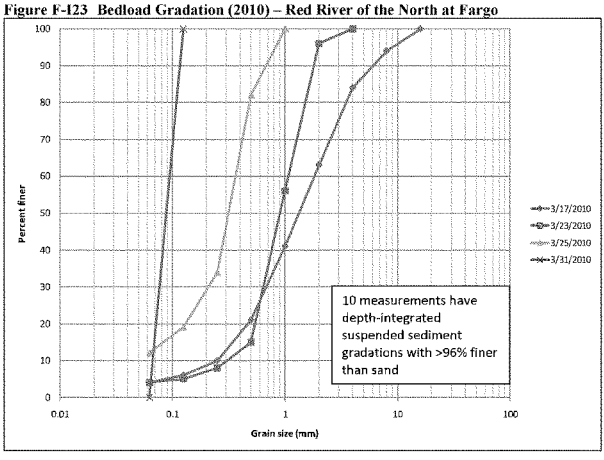


Figure F-I24 Flow and Sediment Balance (2010) – Red River of the North

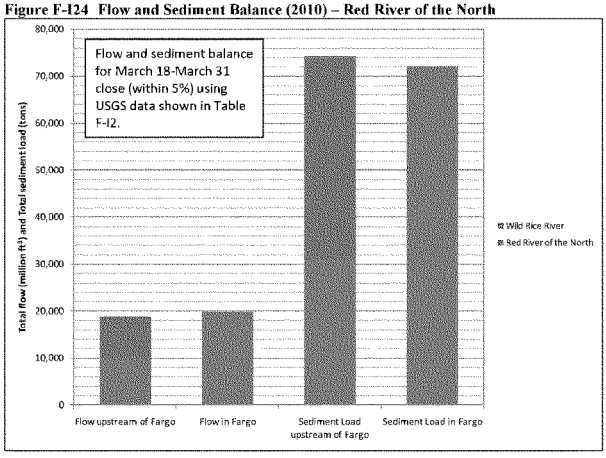
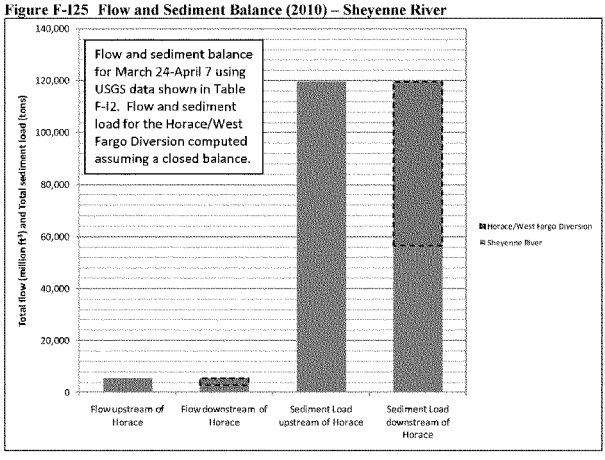


Figure F-I25 Flow and Sediment Balance (2010) – Sheyenne River



RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT J – ICE ASPECTS OF PRELIMINARY DESIGNS

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

By: Barr Engineering Co.

FINAL – Version February 28, 2010

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT J – ICE ASPECTS OF PRELIMINARY DESIGNS

F-J1.0 MEMORANDUM FROM ANDREW TUTHILL, USACE-CRREL, FEBRUARY 16, 2011

MEMO FOR RECORD:
RE: Preliminary assessment of the ice and debris control included in the Phase 3 report. based on details in the Phase 3-Appendix F drawings.
DATE: Feb. 16, 2011.

Review of additional information in Appendix F of the Phase 3 report, support the preliminary findings of the December 31, 2009 "Ice Aspects of Preliminary Designs" (Exhibit E). The biggest concern is still that the diversion and crossing structures will retain large ice floes and possibly debris and, under certain conditions, ice and debris will be drawn into the diversion canal. The preventative measures proposed in the 12/31/09 report included retention booms where surface water velocities are expected to be low ($\leq$ 2-1/4 ft/s) and possibly piers in areas where velocities are expected to be higher.

Measures to control drifting ice using booms are similar to those used to control debris, but there are important differences. The main one is that the retained ice (and the ice that may get past the boom into the canal) melts, particularly under conditions of high flow and a rising hydrograph. This is a mitigating factor should the ice control measures not perform as designed. This is not true with debris which may remain in place to cause problems throughout the event. Following the event, the debris becomes an O&M issue as it will need to be removed. It is not certain what the debris load is on the RRN and tributaries in the project area. More needs to be learned about this. DPW's charged with debris removal after flood events are potentially good information sources.

The ongoing ice analysis will help narrow the discharge range for which the ice control measures will be needed. A mitigating factor is that, above some return interval discharge (Tr $\geq$ 10-20?), very little ice remains in the system and ice control becomes increasingly less important. We are currently trying to determine the dates for each year when ice essentially disappeared. This date, or time window, is estimated from USGS discharge measurement and water temperature data, aerial and satellite imagery and an ice formation and decay model that is being developed. Our aim is to relate discharge and stage to a probability of maximum ice thickness.

For estimating ice-structure interaction, useful new information in Phase 3 Appendix F includes the graphics showing how the structures look under various discharge and diversion scenarios. Also useful were the water velocity distributions from the ADH modeling. From analysis of historic ice breakups completed so far, it appears that much

of the RRN ice remains as large floes during the breakup period. This may be less true for the tributary ice. Based on these information sources, ice-structure interactions and possible ice control measures are discussed. It should be noted that this assessment is preliminary and section 2.3 of the ongoing ice study will treat this subject in greater detail.

RRN Diversion Structure

The greatest ice concerns at the RRN diversion structure will probably be for discharges in the Tr = 5-10 year range where about 2/3 of the flow goes through the structure and 1/3 diverts into the canal (Fig. F06). Here large floes may accumulate against the structure and some may be drawn into the canal entrance. Fig. F-G17 shows ADH calculated velocities in the 1-2 ft/s range upstream of the structure at a ‹ 5-year Q, with no flow entering the canal. This could cause a single layer accumulation of floes upstream of the structure and the canal entrance area. As the velocity increases to the Q=20 yr level, provided the ice had not melted, it would be drawn into the canal. Here the ADH predicted velocities are on the order of 2-3 ft/s (Fig. F-G19). At the 20 yr Q, the gates would be open about 5 ft which is not sufficient to draw floes beneath. If the floes were large as field evidence suggests, a floating boom would probably keep most of this ice out of the canal.

WRR Diversion Structure

The WRR structure will probably accumulate a single layer of large floes and not pass much ice at the 5-year discharge. Figure F-G29 shows average water velocities upstream of the structure to be in the 1-2 ft/s range for this condition. At the 20-year flow level, any of the accumulated ice that remains would be drawn over the downstream weir and into the canal as a result of water velocities as high as 4-5 ft/s. If our analysis indicates the likelihood of significant ice remaining in the system for Q › 20 yr flow levels, then the WRR ice will need to be retained upstream of the canal junction. ADH calculated velocities upstream of the structure are on the order of 2-3 ft/s, indicating that a boom might work provided the ice floes were large (Fig. F-G31).

Sheyenne and Maple River Crossing Structures

Under Flow Scenario 2 (Q ~ 10 yr?) (Fig F26), ice floes would accumulate upstream of the aqueduct and ice would at some point be drawn over the grade control structure and spillway into the canal. The same would be true for Flow Scenario 3 (Q~20 yr ?) if ice still remained in the system. Due to the width reduction, it is unlikely that the aqueduct would not convey any ice downstream; the ice would probably jam at the aqueduct entrance which would be undesirable as it would increase upstream stage and the diversion flow into the canal. To prevent ice from jamming in the aqueduct entrance and entering the canal, the ice would need to be retained somewhere upstream of the rock grade control structure. The existing conditions ADH simulations at 2 and 50 year flows show average velocities upstream of the structures in the 2-3 ft/s range (Figs. F-G43, F-G44, F-G63 and F-G64). This indicates that floating booms would likely to retain the bulk of the ice upstream of these crossing structures under the estimated range of hydraulic conditions of the Phase 3 design. The booms would be at their upper limit of their performance range though.

Respectfully Submitted,

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F-J2.0 BARR ENGINEERING JULY 8, 2010, MEMORANDUM FROM DISCUSSION WITH ANDREW TUTHILL, USACE-CRREL

The following information was previously presented as Exhibit E of Appendix F of the Phase 3 report submitted on August 6th, 2010, and is included here for completeness.

Brian LeMon and Sarah Stratton had a conference call with ice expert Andy Tuthill to discuss general ice control strategies for the FMM diversion project. Notes from that conversation follow:

- Approximate Costs:
  - Floating ice boom (e.g. Lake Erie) – roughly $1 million and under[1]
  - Ice control piers – roughly $1 million and over[2]
  - Allegheny River ice boom - $1000 to $1500/foot (note that this number represents the cost of the boom itself, i.e. does not include anchors). Matt Metzger has a cost estimate for the Allegheny Boom which is roughly $3,000/ft[3].

- We explained that our focus has been to keep ice out of the diversion, and Andy agreed that this should be the approach.

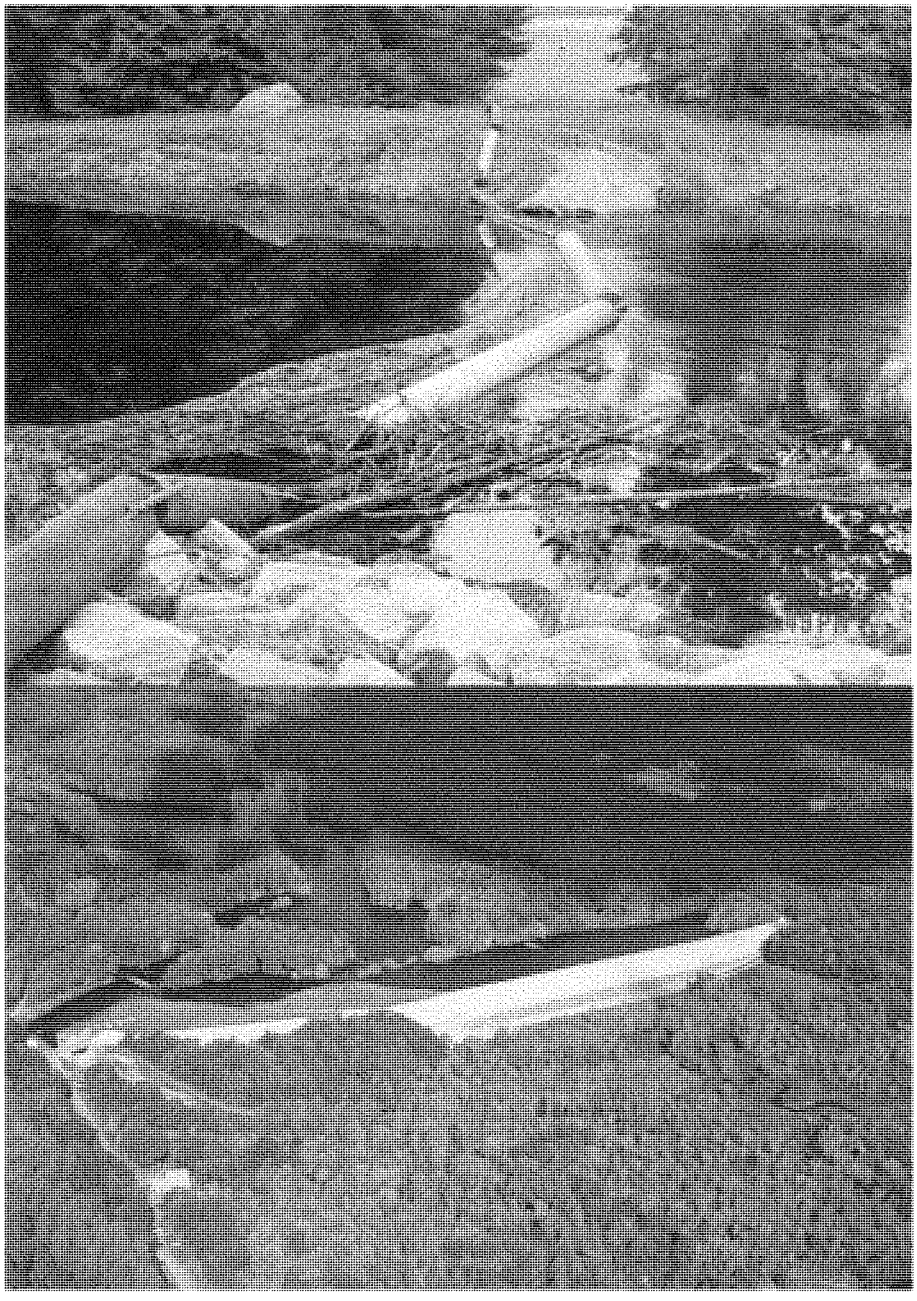
- Andy stated that the most important single variable with ice control is the velocity of the water conveying the ice. Also important are water surface slope, ice thickness and ice strength.
  - These factors determine whether the sheet ice cover will bring into small or large pieces
  - Another variable is the water surface elevation (WSEL) during break-up events and the increase in WSEL from winter base flow levels to break-up levels.
  - General rule of thumb is that the higher the velocity, the more difficult it is to control ice
    - Higher velocities make it harder to retain ice and make the ice more prone to move

[1] Based on a recent estimate to replace the Oil City, PA ice boom, a ballpark cost for an ice or debris boom consisting of steel pipe pontoons ~2' in diameter is about $2500/ft (including the anchors).
[2] Based on the Cazenovia Creek ice control structure completed in 2005, the cost of a pier-type ice control structure is about $ 10,000/ft of river width. These concrete piers are about 10' high and 5' in diameter with 12' gaps between. A large part of the expense is the foundation and post tensioned anchors to the underlying bedrock. This type of structure also requires considerable bank armoring where flow exits and reenters the main channel. Debris accumulation and bank erosion problems are currently being addressed.
[3] This was the approximate cost of the original boom which was over-designed and overly complex. Modern designs are simpler and more efficient and constructed of off-the shelf components such as steel pipe.

- We asked Andy to provide some general guidelines for how to choose ice control measures and here are his responses:
    - An ice expert needs to study the river and ice floes (historic data, talking to people who know about the river, hydrology, etc.)
    - A time-tested rule of thumb is that ice floes will accumulate edge-to-edge (juxtapose) when the water surface velocity is $\leq$ 2.25 ft/second (for deeper water):
        - Generally speaking, ice booms will retain ice when velocities are equal to or less than 2.25 ft/sec
        - For velocities above 2.25 ft/sec, piers are an ice retention option – but piers are more complicated (and expensive) and often get plugged with debris and ice. Piers (or timber cribs) have been used successfully for stabilizing sheet ice in relatively quiescent hydraulic settings.

    - The second time-tested criteria for ice retention is a Froude number of 0.08 or less (for shallow water) – a boom will typically work if the Froude number is 0.08 or less.
    - If ice floes are greater than ¼ of the channel width, they are more likely to create "arching" (<2.5 ft/sec) (*Calkins, D.J., and G.D. Ashton (1975) Arching of Fragmented Ice Covers. US Army Cold Regions Research and Engineering Laboratory, Hanover, New Hampshire, Special Report 222).*

- We asked Andy about how the ice booms function when the water surface elevation and width of the water surface vary a lot. Andy mentioned that this is done a lot in flood control reservoirs (although the booms are usually to keep debris away from the outlet), and cited the West Hill Dam as an example – Andy sent photos of that:



- In this situation, the boom is adjustable and the excess length of boom is just strung along the ground.
- Andy did state that ice booms and trash booms are pretty much the same thing.

- As mentioned previously, piers or timber cribs can be used for stabilizing sheet ice. In this application they are relatively widely spaced (50 ft or greater). This may not apply to the FMM situation where water velocity could be sufficiently high to fragment the sheet ice and push the floes between widely spaced piers or, in the case of narrower spacing, back up ice rubble and flow until sufficient head is developed to push the ice between the piers.
  - When piers are "dolphined" (w/ pier spacing closer to 50' apart), the piers are used the keep big sheets of ice from getting into navigation channels. Andy cited the St. Lawrence River at Lake St. Peter as an example of this.

- Brian asked about the possibility of using heat at some structures to help with ice control.
  - Andy said that heat can be used where a power plant is present – he cited the Dresden Island nuclear power plant on the Kankakee River in IL as an example.

- Brian asked if Andy could share any horror stories and/or tell us about additional things we should be concerned about:
  - Cazennova Creek (written-up in many of the papers and presentations we have regarding ice control)
    - Concrete piers 12 ft apart.
    - Concept is that when ice gets caught up behind piers, the water spreads onto the floodplain then back into the channel.
      - A physical model at CRREL showed relatively uniform lateral outflow and inflow of the water that bypassed the ice jam via the floodplain.
      - The main problem with these piers is that debris quantities were underestimated. Following high flow events "the ice control structure looked like a beaver dam". More flow than anticipated bypassed the structure via floodplain and that flow tended to concentrate at low points rather than distribute uniformly.
      - As a result, the riprap protection along the adjacent bank was severely eroded. Measures are currently being developed to correct the problems and avoid the development of a new channel flanking the structure.
  - Flow beneath an ice jam caries the potential for "under ice hydraulic scour"
    - The scour occurs since the ice jam decreases the flow area increasing the near-bed water velocities. The roughness of the ice

cover also increases the turbulent kinetic energy of the flow, increasing the bed shear.

- Recommendations from Andy:
  - Put effort into trying to determine where ice control is actually needed (i.e. don't over-design for ice control).
    - "Probably don't get a big ice run on the RRN."
    - Further study needs to be done, specifically for ice potential at/near the RRN and WRR control structures.
    - Andy also commented that he is not currently comfortable with his lack of knowledge of the ice situation for the FMM project. He mentioned in a follow-up email that he thinks this is an extremely interesting project and he is glad to be involved.
  - With regards to having permanent ice removal (e.g. cranes) on site, Andy said this seems like an "emergency" measure and does not recommend relying heavily on this approach.
    - Too expensive, too much maintenance.
    - Use an emergency response protocol instead.
  - With regards to trying to contain the ice somewhere then remove it with equipment, Andy does not recommend this either – "it's probably futile to pull ice out of the river during an ice run and it's never been done."
    - However, Andy also mentioned that equipment is often used to help move ice through structures such as bridge openings during an ice event.
  - At the Sheyenne and Maple structures, a sheer boom could be used at the entrance to the spillway channel (perhaps even if velocities along the tributary are higher than 2.25 ft/sec since the boom would be parallel to the tributary.
  - Keep under ice hydraulic scour in mind.
    - Ice/sediment/channel bottom interface.
    - We'll want to control where the ice jams (i.e. we want to make sure that if a jam occurs, it does not occur where scour may impact a structure – in this case, most likely at the RRN and WRR).

F-J3.0 ICE ASPECTS OF PRELIMINARY DESIGNS, RED RIVER OF THE NORTH DIVERSION PROJECT, BY ANDREW TUTHILL, USACE-CRREL

The following information was previously presented as Exhibit E of Appendix F of the Phase 3 report submitted on August 6th, 2010, and is included here for completeness. The tables, figures, and appendices referred to in the text are included within this section F-J3.0.

Ice Aspects of Preliminary Designs Red River of the North Diversion Project Fargo ND, Moorhead, MN

to

Barr Engineering
and
St. Paul District USACE

by:

Ice Engineering Group,
Remote Sensing/ GIS Branch
US Army Engineer Research and Development Center
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Dec. 31, 2009

1. Introduction

The all-time peak stage on the Red River of the North at Fargo, ND occurred on March 28, 2009, causing major flooding in the Fargo and Moorhead area. In terms of river discharge, this event had a return interval of about 100 years. The RRN also experienced major flooding in the early spring of 1997 with the greatest damages occurring farther downstream at Grand Forks. Though discharge was the predominant factor, ice and ice jams played a role in both these events. To mitigate future flooding, plans are being developed for diversion channels to bypass a large portion of RRN flood flows around Fargo and Moorhead. The diversion alternatives being evaluated call for a gated structure located on the RRN about 27 river miles (RM) south of Fargo that would divert as much as two-thirds of the 100-year flow into bypass canals around the cities. The diversion flow would re-enter the RRN about 34 RM north of Fargo, about near the near Georgetown, MN. Fig. 1 shows the project area with one of the preliminary design alternatives. The objective of this review is to identify important ice issues associated with the preliminary project plans.

This report describes ice processes on the RRN in the vicinity of Fargo and how ice may impact operation of the proposed project. Much of the preliminary ice analysis is based on observations and data from the 2009 and 1997 floods as well as review of flow and stage records. Preliminary plans of diversion structures by Barr Engineering are also assessed in terms of their expected ice-passage performance.

2. General Ice Processes on the RRN

The RRN is a flat and meandering river with low water velocities during the ice formation period favoring the growth of sheet ice that often exceeds 2 ft in thickness by

late winter. Although mainstem flood flow velocities can be as high as 5-6 ft/s, spring breakup on the RRN is typically gradual without dynamic ice runs. While water levels may rise well above the top of bank elevation, the thick sheet ice typically melts in place, often lodged in tight bends or contained within the channel by trees that line the banks. Fig. 2. shows a satellite image of the RRN at the time of the peak stages on March 28, 2009. In the vicinity of the proposed diversion, the flooded width is about 1/3 mile, increasing to over 1 mile farther the north. One can see snake-like remnants of the sheet ice cover indicating the location of the river channel before the flow went out of bank. Through the settled part of Fargo and Moorhead and northward, the floodway narrows to about ¼ mile or less with little ice visible. Downstream of the Sheyenne confluence, the flooded width increases to about ½ mile up to the tight S-bend where the proposed channel would re-enter the RRN. In this bend, one can see the remaining trace of the sheet ice cover.

This interpretation of RRN ice processes based on the 3/28/09 satellite image agrees with general observations by Kate White of CRREL from helicopter flights on April 1 and 3, 2009. Appendix A contains summary of Dr. White's observations.

The daily stage record for the 2009 flood event shows no evidence of significant ice movement on the RRN (Fig. 3.). Daily discharge is somewhat more peaked than the stage hydrographs, but still not characteristic of a breakup involving ice jamming or ice releases (Fig. 4). It may be that the extremely high discharges in 2009 overwhelmed the ice effects. In the more moderate breakup of April 2, 2005 where stage at Fargo reached 19.2 ft, the stage hydrograph saw rapid changes as great as 2 ft attributed to ice jams and releases. From the 2009 discharge data, the river became ice free at Hickson on 3/24 and at Halstad on 3/25.

Winter period flood problems on the RNN appear to be increasing with time. In 2009, in addition to Fargo and Moorhead, ice-related flooding occurred at Oslo, Grand Forks, Drayton and Pembina, though the potential impacts at Grand Forks were greatly reduced by flood control works built since 1997. In 1997, flooding was widespread along the RRN at locations such as Wahpeton, Tyler, Breckenridge, Hickson, Fargo, Drayton, Grand Forks and Emerson. At Fargo, the 1997 flood crested on April 10 at 37.6 ft, only 3.4 ft lower than the 2009 peak. The CRREL Ice Jam Database (IJDB) reports less severe ice jam flooding along the RRN in 1989, 1969, 1961, 1960, 1959, 1957, 1955, 1954, 1952, 1948, 1946, and 1945, though many of these are ice-affected gage reports with no mention of damages. One IJDB report of interest is a "large ice obstruction" at Hickson, ND on April 9, 1997 which is about 2 RM upstream of the proposed diversion shown in Fig.1. The timing of the peaks is March 26, 28 and 30 for Hickson, Fargo and Halstad respectively. Hickson is about 14 air miles upstream of Fargo while Halstad is 35 air miles downstream. The Wild Rice River which enters the RRN about 7 air miles above Fargo and the Sheyenne and Buffalo River which enter the RRN 11 and 14 miles downstream respectively have a large influence on the stage and discharge data shown in Figs. 3 and 4.

If the ice period on the RRN is defined as December 1 – April 15, then 61 of 109 peak stages at Fargo occurred with some ice present on the river. Fig. 5 plots open-water and ice season peak discharges at Fargo with time, showing increasing trendlines, particularly for the winter-season events. Fig. 6 shows a discharge frequency curve with the data sorted into ice-season and open water season. Of the 21 events above the 5-year discharge, two-thirds occurred during the ice season which emphasizes the importance of winter hydrology and ice processes in the project design.

3. Ice Issues for the Proposed RRN Diversion Structure

A gated structure is proposed to divert flow from the RRN into bypass canal(s) around Fargo and Moorhead. This structure would have three openings, 40-ft-wide × 44.1-ft-high Fig. 7. Underflow radial gates would close off all but the lowest 4.1 ft. of each opening. Table 1, based on data from Barr Engineering for one of the diversion alternatives being considered, lists flood discharges and the flow split between the upstream diversion(s) and the RRN Diversion Structure. For the 5-year flow and lower, the gates would fully open and all flow would pass the through the structure. For the 10 to 100-year discharge range, all but about 10,000 cfs would be diverted around Fargo and Moorhead. For the 200 and 500 return interval flows, about two-thirds of the total river flow would be diverted into the bypass canal(s).

Table 1. Diversion Structure Data and Calculations

								Average	Approach	Average	
				Upstream	WL Change	Gate	Total	Velocity	Flow Area	Approach	Opening/
Event	Q tot	Q thru	Qdiv	Depth	Thru Gate	Opening	Opening	at Gates		Velocity	u/s Depth
(Years)	(cfs)	(cfs)	(cfs)	(ft)	(ft)	(ft)	(ft)	(ft/s)	(ft^2)	(ft/s)	
2	3500	3500	0	30.2	0.7	40.0	44.1	1.0	5,294	0.66	1
5	9600	9600	0	39.8	1.1	40.0	44.1	2.0	7,550	1.27	1
10	14500	10274	4226	43.5	3.9	2.6	6.7	12.8	8,493	1.21	0.15
20	19000	9917	9083	45.7	6.5	0.9	5	16.5	9,074	1.09	0.11
50	25500	9528	15972	48.1	9.1	0.0	4.1	19.4	9,723	0.98	0.09
100	30000	9792	20208	49	9.6	0.0	4.1	19.9	9,972	0.98	0.08
200	39000	12806	26194	49.9	8.1	1.7	5.8	18.4	10,222	1.25	0.12
500	53000	17940	35060	51.5	6.3	5.2	9.3	16.1	10,673	1.68	0.18

In terms of ice, it is good that the gates start in the open position and be progressively closed as flow increases up to the 100-year level. This mode of operation would avoid gate freezing problems since, by the time the 10-year flow occurred, any ice on the gate abutments would probably have melted. In the opposite scenario of opening the gates as the flow increases, freezing of gates and gate seals would likely be a problem.

Water velocity provides a good indicator of how the ice floes will accumulate upstream of the structure. The average approach velocities in Table 1 were calculated from the 1-D continuity. The upstream flow areas were calculated based on the listed depths from Barr Engineering and an assumed a trapezoidal channel cross section with a 120-ft base width and 1V:2H side slopes. A long-held rule of thumb predicts that ice floes will accumulated edge-to edge (juxtapose) and not under-turn and submerge where water velocity is less than about 2-1/4 ft/s. In all the above cases, the juxtaposition criterion is easily met, so, if the upstream ice cover were fractured and free from the banks, the floes would accumulate edge to edge upstream of the gate rather than form a multi-layer (shoved) ice accumulation.

In the case of the 5 year and lower discharges, these floes would either pass the gate or the ice fragments would arch and stop upstream. Calkins and Ashton 1975 predict that concentrated ice floes will arch across an opening where the average floe diameter is less than about one quarter of the opening width, so if the floe were smaller than about 10 ft, the ice pieces would probably go through.

In terms of passing ice or debris, a larger opening of the central gate is preferable to lesser but equal openings of all three gates. A central gate opening is also better than having the middle gate closed and the side ones open. This may not be an important factor since ice will likely only pass the structure for flows $\leq$ the 5-year discharge (explained below). A minimum width of 40 ft for the central gate would ensure optimal ice passage. Assuming that ice floes will be passed through the central gate, decreasing the width of the side gates would not have a negative effect on ice passage.

For these lower flows all three gates will be fully open. Ice and debris booms would be beneficial in terms of channeling the ice and debris through the central gate, as suggested by Barrr in Fig. 8. Many off-the-shelf debris boom designs cannot withstand serious ice action. Recent steel boom designs used on Canadian Rivers do however, and this technology might be useful in the current project (Appendix B).

For all the other discharges and gate opening conditions listed, the ice will most likely not pass the structure. This is because the gate opening heights are relatively small fraction the upstream depth (Table 1). This prediction of non-passage of ice is based on experience with ice passage on large river dams were the gate opening needs to be from one-half to one-third the upstream depth in order to pass ice. An example is the Mel Price Dam on the Mississippi River where the normal pool depth is about 40 ft and an under-flow tainter gate opening needs to be at least 15 ft to draw broken ice pieces beneath.

The design intent is to preserve existing ice passage conditions on the RRN main channel while preventing ice from entering and jamming in the diversion canals. For this reason, the sheet ice cover on the RRN may need to be stabilized and retained at the canal entrances. This could be accomplished with rows of piers across the canal entrances but detailed analyses of expected ice type, floe diameters and approach velocities, etc. would be needed for the designs under consideration.

4. Ice Issues at Diversion Canal-Tributary Crossings

The diversion canal will cross three tributaries, the Wild Rice (WR), the Sheyenne and the Maple. In the case of the Wild Rice, a diversion structure similar to the RRN gated one is proposed. This would have two-30-ft-wide by 22-ft-high gates that would divert most of the WR flood flow over a weir into the canal. In the proposed design, the Sheyenne and Maple Rivers would be conveyed over the diversion canal by concrete aqueducts. Diagonal canals would divert a portion of the flood flows from these two tributaries over weirs into the canal. Rock arch drop structures would be built across the tributaries upstream of the diversions. The Maple river aqueduct would have gates to

regulate diversion flow while the Sheyenne outflow would be governed by water levels at the weir. In a non-flood situation, the diversion canal will be dry with the exception of a low flow channel to convey local runoff.

The Wild Rice, Sheyenne and Maple Rivers, like the RRN are low gradient tightly meandering rivers, not known for dynamic breakup ice runs. Still some breakup ice jamming occurs, and, combined with elevated water levels due to high discharge, small jams can exacerbate the flooding. The CRREL Ice Jam Database reports a number of ice jams at locations on the Wild Rice River such as Mantador, Great Bend, Rutland and Cayuga, all too far upstream to influence ice processes in the vicinity of the diversion canal crossing. On the Sheyenne River, the IJDB reported major ice jam flooding at Harwood in 2009 and a jam in West Fargo. Backwater due to ice was reported at Kindred in 1969 and West Fargo and Kindred in 1961. Kindred is about 5 RM upstream of the proposed canal crossing structure.

At the Wild Rice crossing, one would expect ice issues similar to those at the RRN diversion structure. For all but the 2- and 5-year discharges, the gates will be closed, leaving only the 2-ft bedload opening. Assuming a trapezoidal approach channel with 2H:1 V side slopes, Calculated approach velocities at the depths provided by Barr Engineering would be at or below about 1.5 ft /s, favoring accumulation of ice floes by juxtaposition and no underturning. Depending on surface concentration and floe diameter, the ice pieces would either pass the 30-ft-wide gate opening or arch and stop upstream, as discussed in the previous section. For the higher flood flows with upstream depths in the 17-25 ft range, the 2-ft opening would be too small draw ice floes downs and through the gates. Similar to the RRN diversion structure, provisions would be needed to prevent ice floes on the Wild Rice River from entering the diversion canal. This could likely be accomplished by row of piers upstream of the canal entrance weir.

At the Sheyenne and Maple crossings, assuming the ice cover fractures, the ice pieces may accumulate at the constriction at the aqueduct entrances. The flow diverted from the river upstream of the aqueduct will also decrease the water velocity and ice conveyance capacity downstream favoring jamming. Provided the water velocity in this area does not exceed about 2-1/4 ft/s, the ice floes will accumulate edge to edge (juxtapose) and not shove-thicken[1] to block off much flow area. Again, some type of ice retention piers will likely be needed to keep ice pieces from passing into the diversion canal upstream.

At the Maple crossing, gates and operator deck abutments will tend to retain large ice floes in addition to the ice retaining effects of the channel constriction and the discharge reduction.

Both crossing structures are supported by concrete walls spaced about 20-ft apart. Diversion channel flow passes beneath the tributary through these conduits, in either an open channel flow or pressurized flow condition. It will be critical that these conduits do not become ice or debris blocked. That the canal is dry (except for low flow drainage) in

[1] Shove-thickening is a process of ice jam formation where floes under-turn to form a multi-layer ice accumulation.

non-flood conditions is fortunate in that it will be ice free at the start of the breakup period. One concern is that the spring thaw might occur in several phases. Under this scenario, the drainage canal might fill. A return to colder air temperatures might cause an ice cover to form on the canal as discharge and water levels subside. A second thaw and rise in canal discharge could then break up the ice cover on the canal and jam the ice in the conduit openings.

5. Additional Ice Concerns Related to the RRN Diversion Project

One concern is that, for large flow events, the bypassed flow may move the hydrograph downstream more quickly and change how the ice cover releases where it re-enters the RRN. Much of the water volume that now fills the overbank areas in a large flood event would be transposed downstream under post-project conditions. The canal outflow would amount to the addition of a large tributary. This large discharge influx might break up the local ice cover and cause ice jams where none occurred before. For example, the satellite photo in Fig. 2 shows an S-shaped section of sheet ice at the proposed re-entry point that might break up and re-jam downstream.

6. Preliminary Conclusions

1. Although peak stages on the RRN are often ice-affected, the ice breakup on the RRN is typically gradual with little dynamic movement of ice floes. Ice jams do occur however, and when the river is already at or above flood stage, these jams can greatly exacerbate the situation. During the peak floods 1997 and 2009, much flow was out of bank with short sections of sheet ice remaining in the channel bends.

2. The long-term RRN gage data suggest an increasing trend in annual peak discharge magnitude and variability, with the bulk of the peak flow events occurring during the ice season. If it is decided that these trends are real, some conservatism may need to be added to design to accommodate increased future peak discharges.

3. Under flood conditions, in the area of the proposed diversion structure, the 40-50 ft depths and low approach velocities will cause sheet ice and large floes to accumulate upstream of the structure without passing the gates. Provided the floes accumulate edge-to-edge and do not thicken into a multi-layer ice accumulation, this does not pose a problem in terms of ice passing into the diversion channel, provided the ice can be retained in the RRN (see Item 4 below).

Ice may pass the gates at the 2 and 5 year discharges provided the ice floes do not arch and stop upstream of the gate opening(s).

4. The project design must include provisions to prevent RRN ice floes from entering the diversion canal(s). This might consist of rows of piers or piles to retain the floes in the main channel of the RRN. Depending on hydraulic conditions it may be possible to retain the ice floes in the main river channel using the less expensive alternative of ice

booms. The necessity and design of ice retention schemes will depend on further analysis of expected ice conditions and ice processes in the vicinity of the canal entrances need to be examined in detail.

5. The crossing structure at the Wild Rice River will have similar ice issues as the RRN diversion structure. It is predicted that the gated structure will retain Wild Rice river ice under all but the lowest flood discharges. At the Sheyenne and Maple crossing structures, ice accumulations are possible at the transitions from the natural channels to the aqueducts. This will be acceptable as long as the accumulated ice does not shove-thicken into a multi-layer ice jam. As with the RRN diversion structure, provisions will be needed retain ice in tributary channels and prevent it from passing into the diversion canal.

6. Changes in the breakup period hydrograph as a result of the project, and how the bypassed flow re-enters the RRN need to be examined, particularly in terms of causing ice jams where none occurred before.

Respectfully Submitted,

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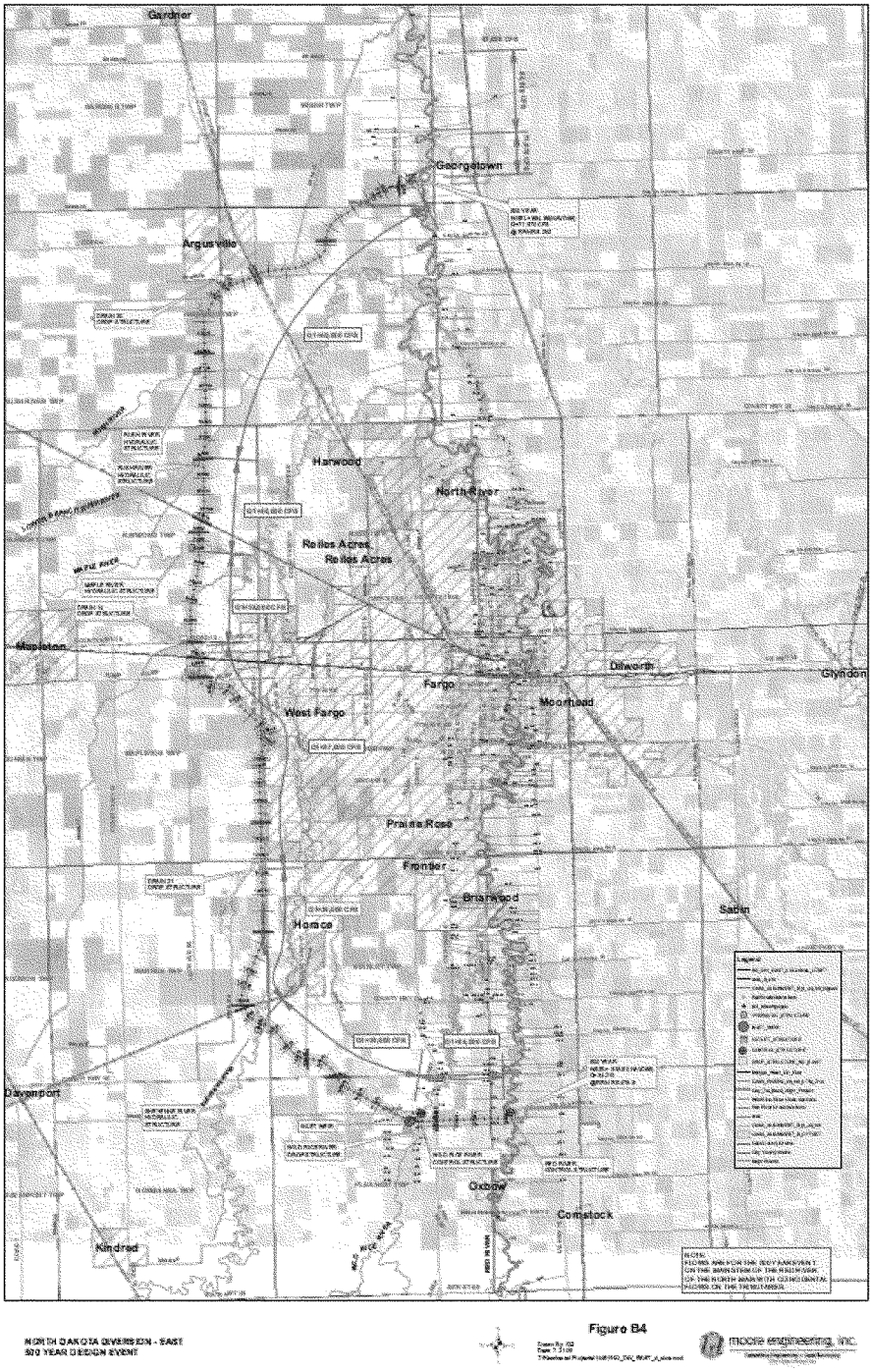


Fig. 1. Map of project area showing one of the diversion alternatives.

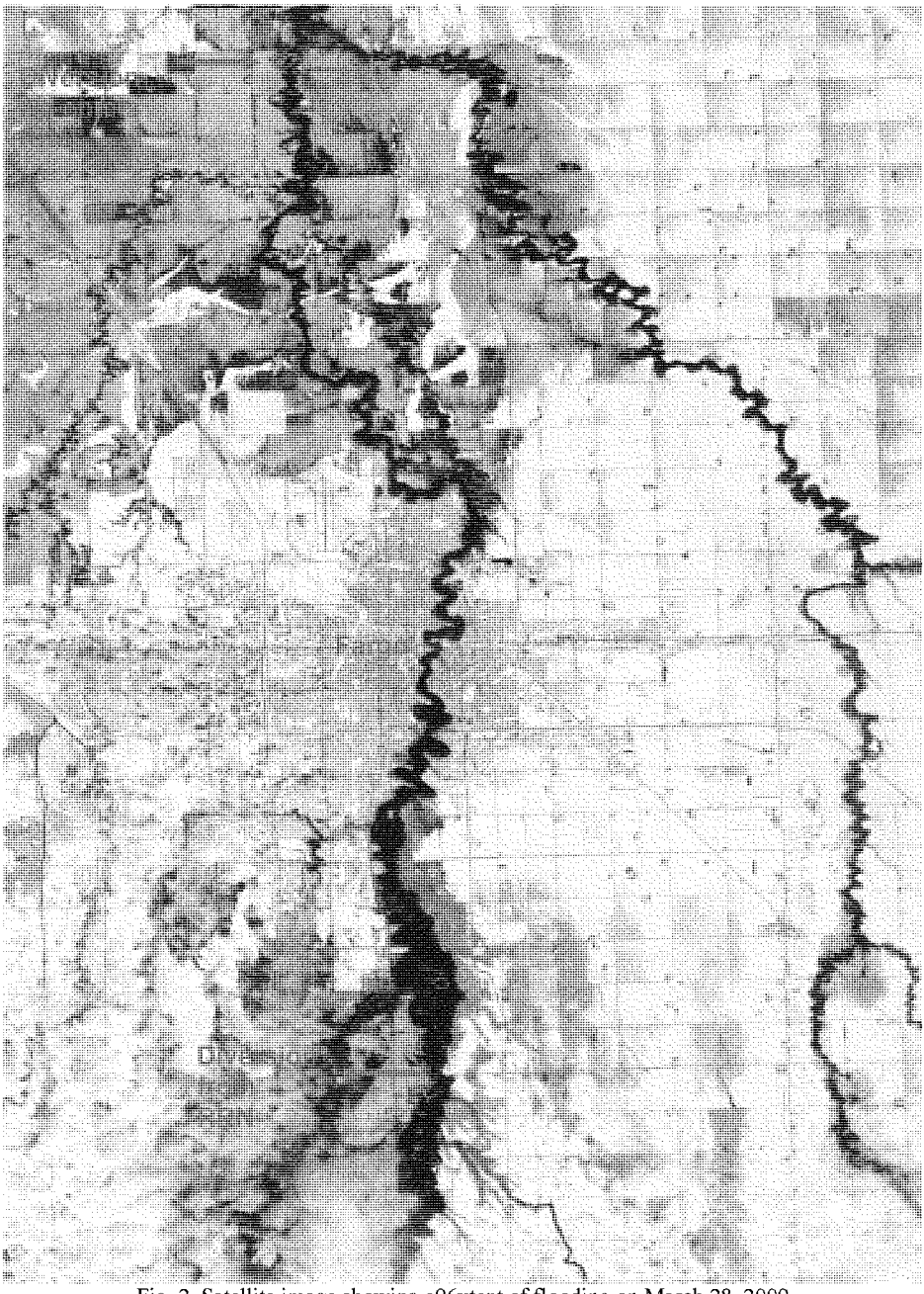


Fig. 2. Satellite image showing e96xtent of flooding on March 28, 2009

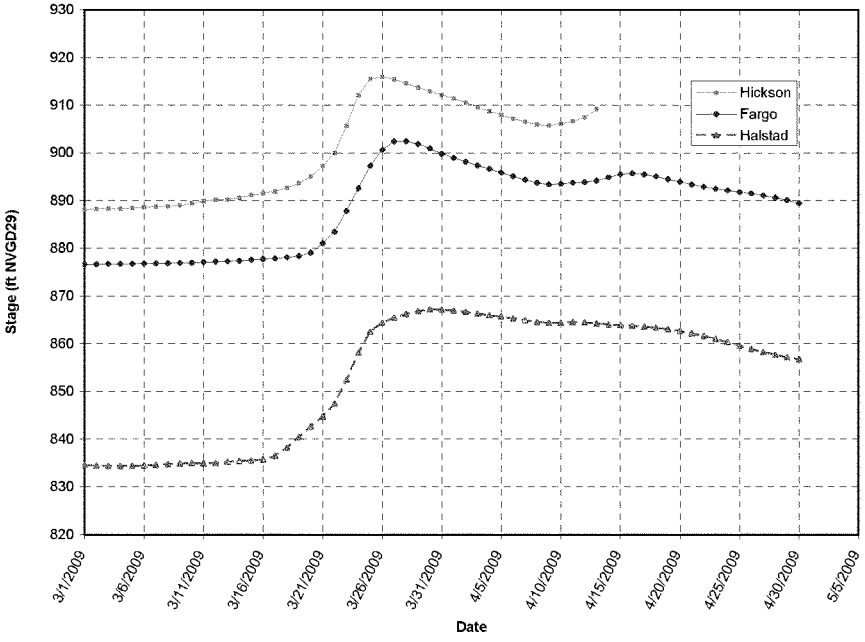


Fig. 3. RRN water surface elevation at Hickson, Fargo and Halstad during the 2009 flood.

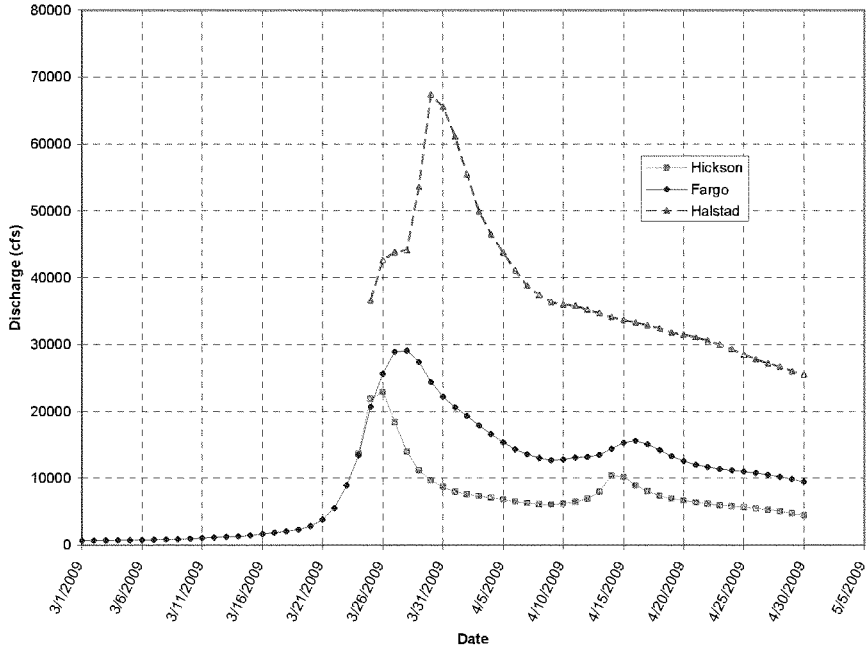


Fig. 4. RRN discharge at Hickson, Fargo and Halstad during the 2009 flood.

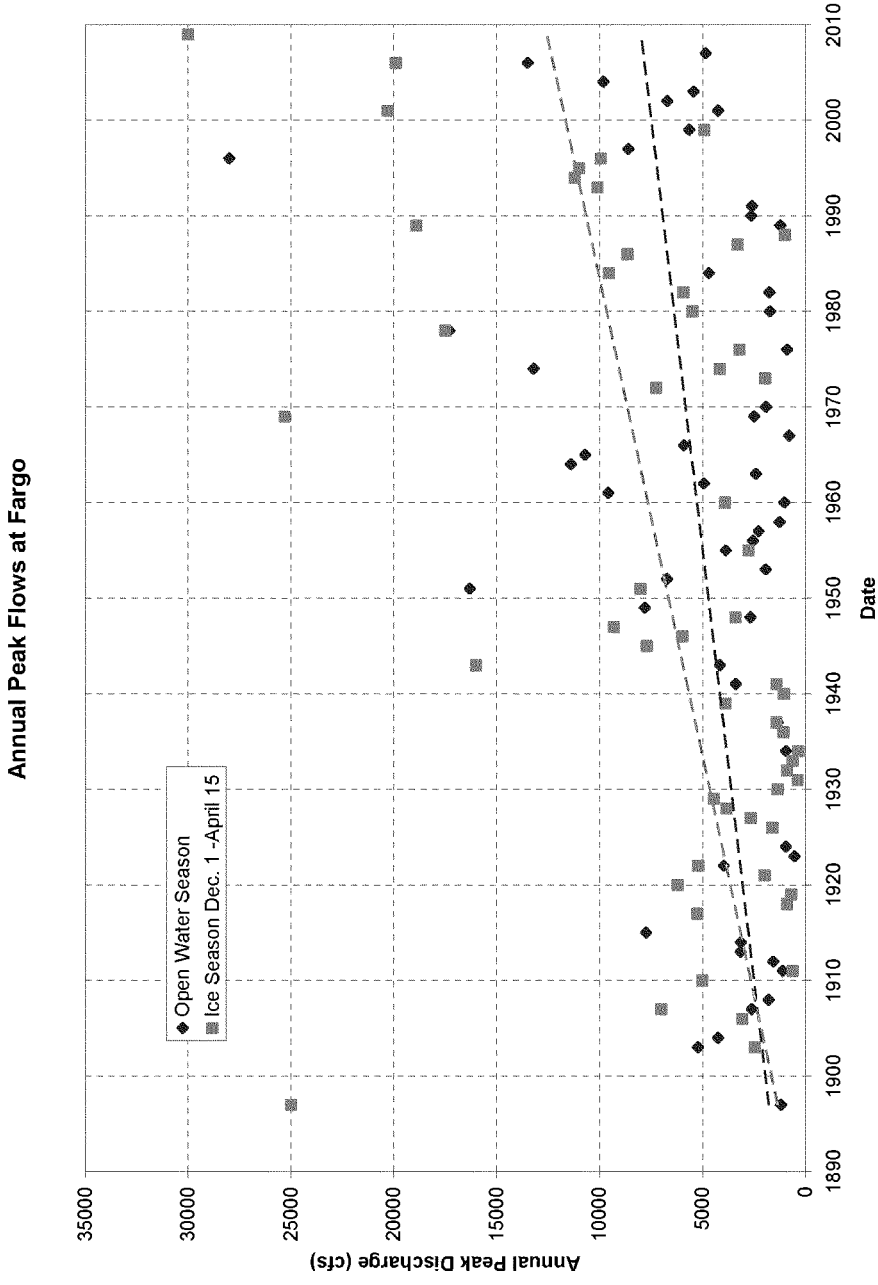


Fig. 5. Annual peak discharges at Fargo.

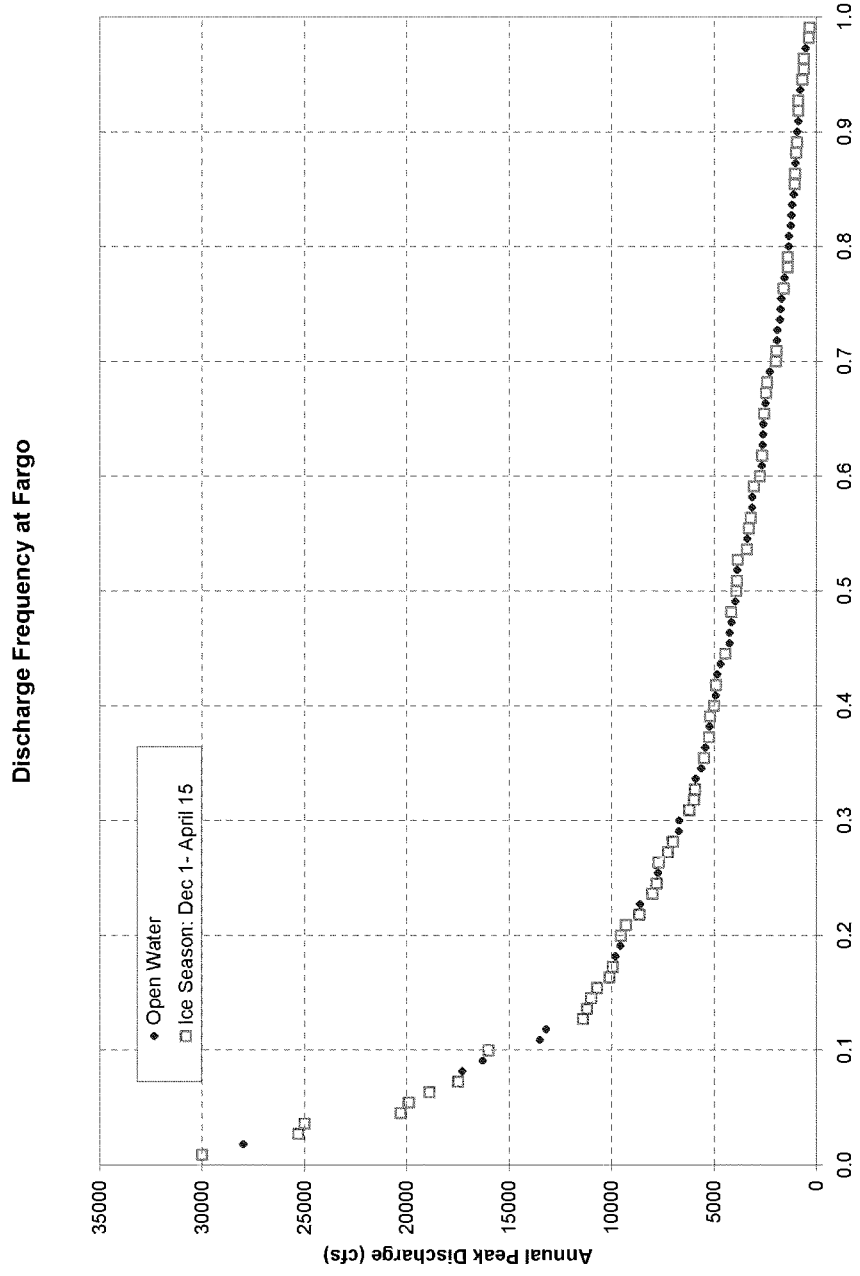


Fig. 6. Probability distribution of peak discharges at Fargo.

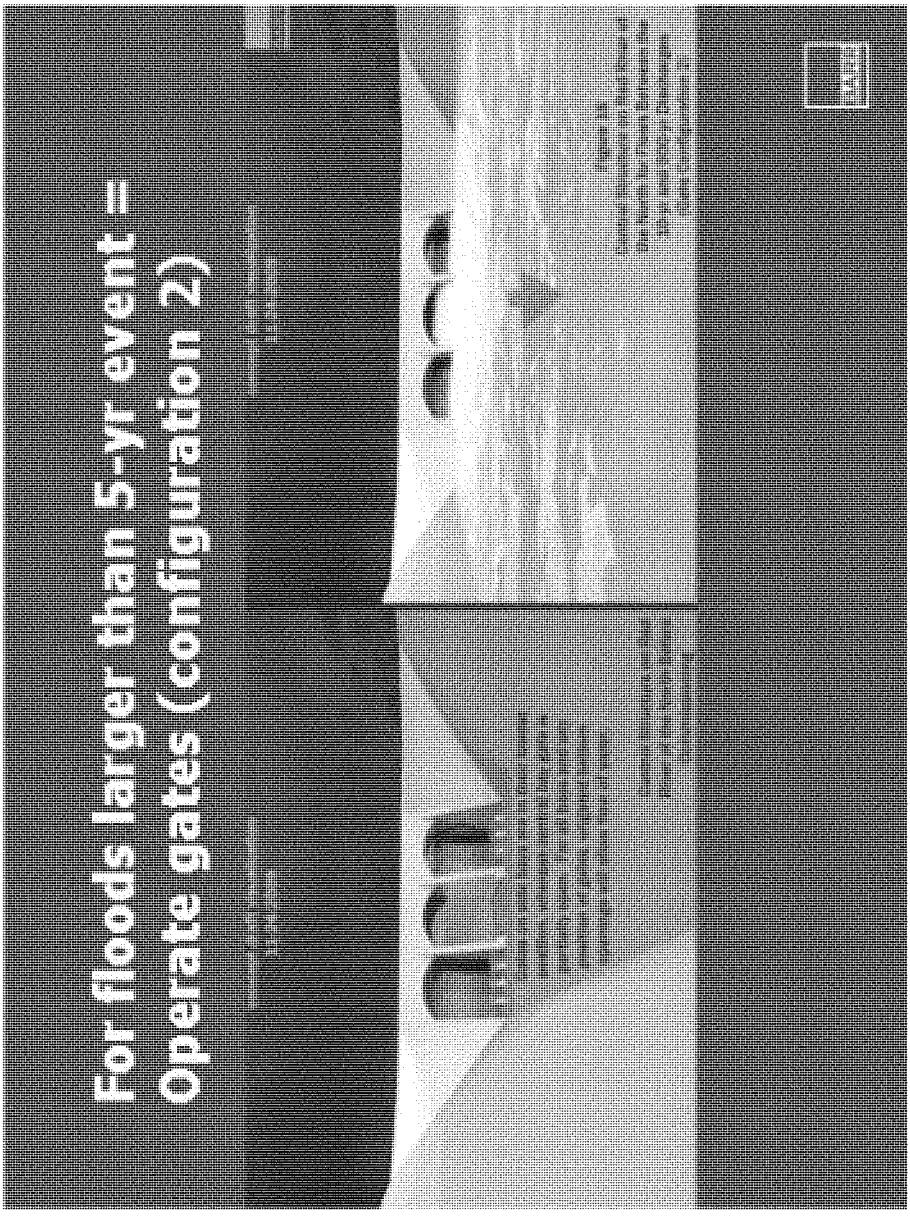
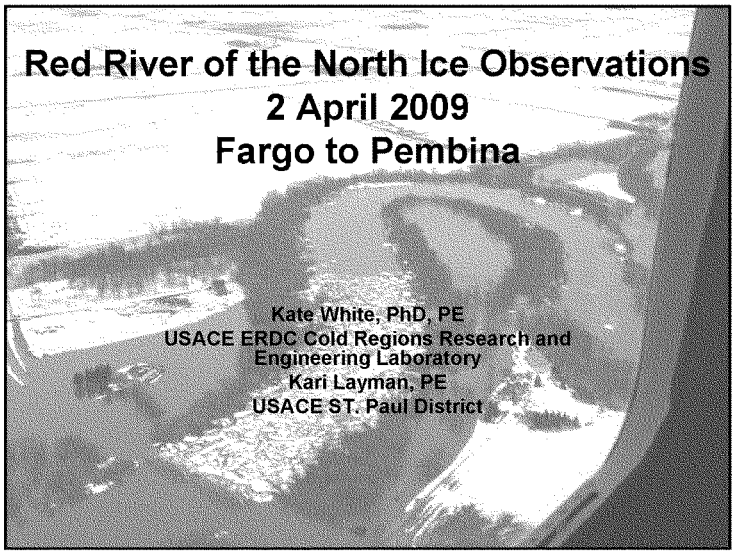


Fig. 7. Proposed RRN Diversion Structure.

Fig. 8. Possible ice and debris boom configuration.

APPENDIX A

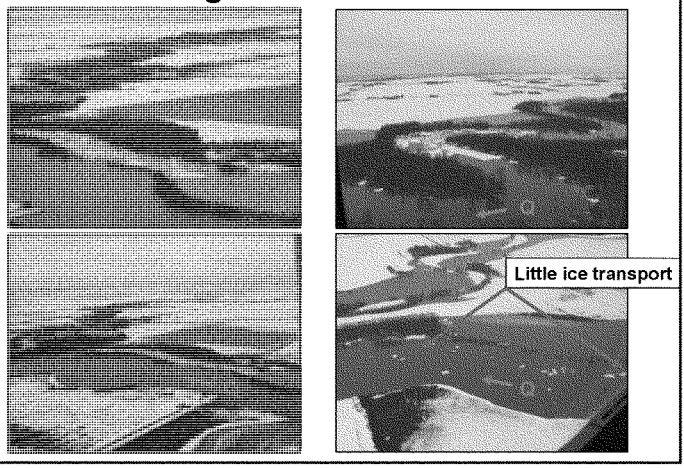


Ice Observations 2 April 2009 Fargo to Pembina

- **BLUF: No Ice Jams**
  - Fargo to Grand Forks:
    - little ice, some big sheets but mostly juxtaposed, single-layer ice pieces, decay evident, some ice transport
  - Grand Forks to Oslo:
    - more large ice sheets but open water as well, ice decay evident
  - Oslo to Drayton:
    - mostly large ice sheets, less decay
  - Drayton to Pembina:
    - large ice sheets interspersed with open water, little decay

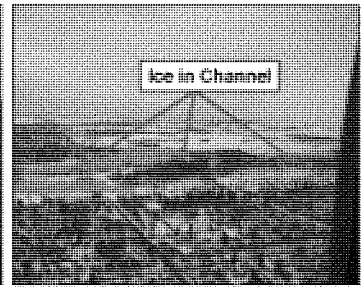
APPENDIX A

Typical ice accumulations Fargo to Grand Forks



Grand Forks

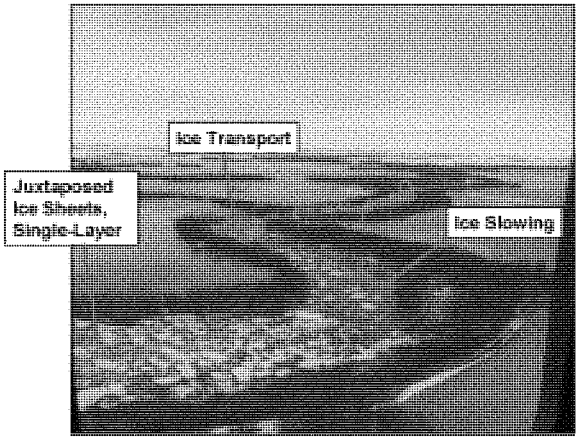
Grand Forks/East Grand Forks



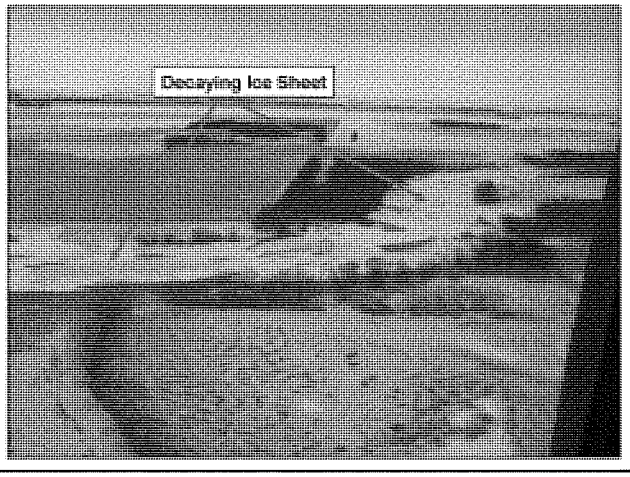
Upstream portion of Grand Forks

APPENDIX A

Ice transport between Grand Forks and Oslo

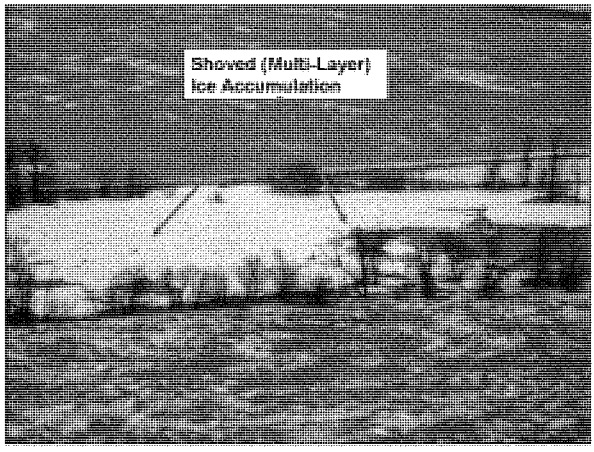


Typical Ice Conditions Oslo to Drayton



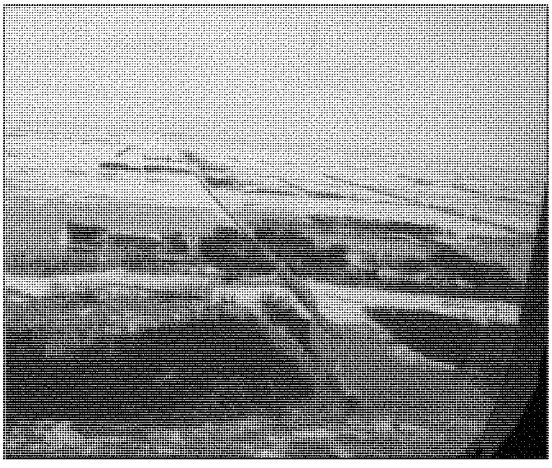
APPENDIX A

Typical Ice Conditions Oslo to Drayton



Drayton

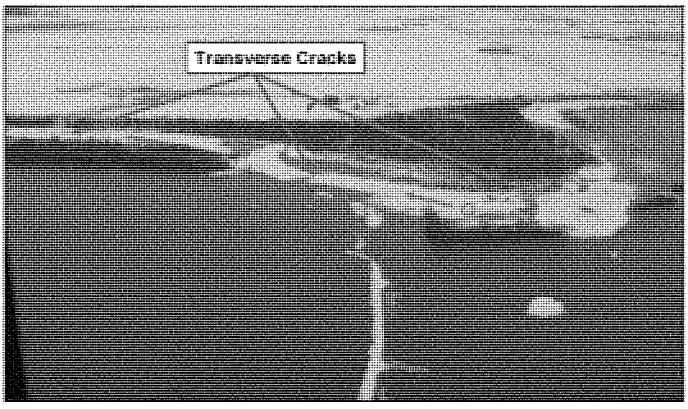
Bridge North of Drayton



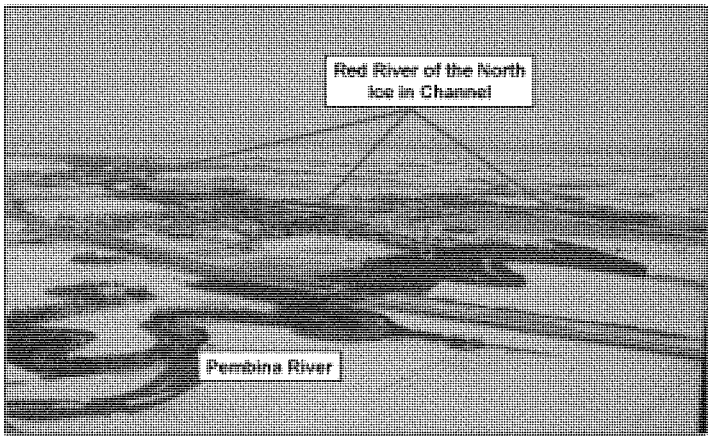
Second Bridge North of Drayton

APPENDIX A

Typical Ice Conditions Drayton to Pembina



Pembina



APPENDIX B

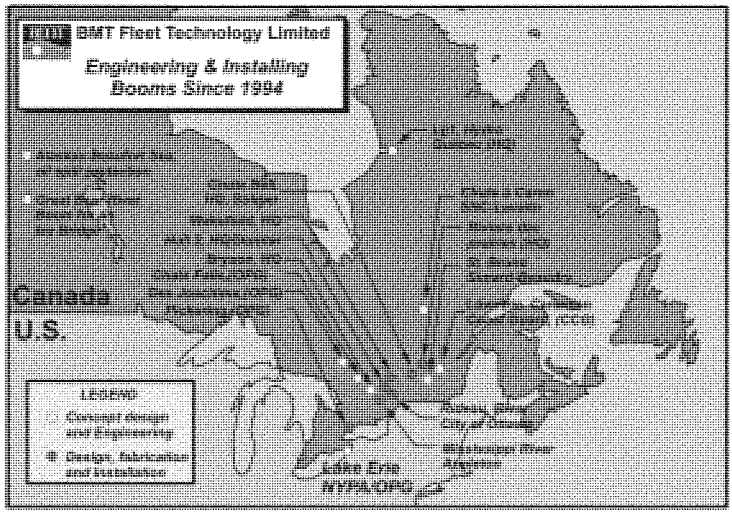
Boom Projects

BMT Fleet Technology Limited

Over the past ten years, BMT Fleet Technology Limited has gained significant experience in the re-engineering of some of the longest ice control booms in North America and in the construction of several small ones.

Without exception, steel booms, using cylindrical pipes with capped ends, replaced the timber pontoons for several reasons:

- Lower Capital Costs
- Lower Maintenance Costs
- Constant density/buoyancy, for optimization to achieve the best results for each particular site
- Better visibility for improved safety of boaters
- Environmentally friendly



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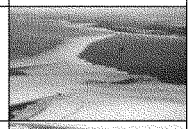
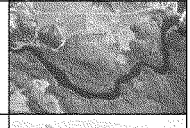
APPENDIX B

Boom Projects

BMT Fleet Technology Limited

Client	Contact	Description	Location *	Duration	A view
New York Power Authority, Niagara Power Project	Mr. Randy Crissman	Engineering services for the Lake Erie – Upper Niagara River Ice Boom. Assessment, monitoring, concept design.	North-east end of Lake Erie *	5 years, 1993 to 1997.	
Canadian Coast Guard, Laurentian Region	Mr. Stéphane Dumont	Engineering services for the development of concept design for the Lavaltrie Ice Boom.	St. Lawrence River at Lavaltrie *	12 weeks, from Aug 1993 to Oct 1993.	
SNC-LAVALIN	Mr. Bertrand Massé	Design of a replacement ice boom upstream of Chute à Caron hydroelectric dam.	Chicoutimi, Quebec *	4 weeks, 1997	
Merol Power	Mr. Ed Olshesky	Ice observations and design of ice booms for the protection of Appleton Power Plant.	Appleton, Ontario	Jan to April 1998	
Hydro Quebec, Maisonneuve Region	Mr. Alain Cyr	Design, fabrication and deployment of an ice boom to protect the Hull 2 and E.B. Eddy hydroelectric power plants' intakes from ice.	Ottawa River near Hull, Quebec	10 weeks, Sep 1997 to Nov 1997.	
City of Ottawa	Mr. Tony Garnett	Design, fabrication and deployment of an ice boom to reduce the amount of ice generated in the Rideau River.	Rideau River in Ottawa	8 weeks, Sep to Oct 1998.	
IRAP	Dr. M. Bishop	A study to evaluate a system to sink and re-float ice boom pontoons.	The Canadian Coast Guard Base in Prescott	January to July 1999.	
Soliger, Hydro Québec, Région Maisonneuve	Mr. Marc Richard	Design, fabrication and delivery of an ice boom to protect the Bell Falls hydroelectric power plant intake from ice.	Rivière Rouge at Bell Falls. To be installed in June 1999 *	8 weeks, Sep 1998 to Oct 1998.	
Ontario Hydro, Pickering Nuclear GS	Mr. Mark Arnone	Design, fabrication and deployment of an ice boom to protect the Pickering Nuclear Power Plant intake.	Lake Ontario, at Pickering *	3 weeks, Jan 27 to Feb 20 1999.	
Hydro Quebec, Wakefield	Mr. André Chouinard	Concept development, design, fabrication and deployment of an ice boom to accelerate the formation of a smooth ice cover leading to higher power production.	Gatineau River, Downstream of Paugan Dam *	April 1999 to November 1999.	

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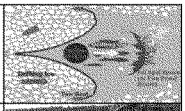
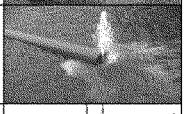
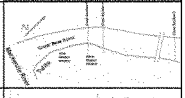
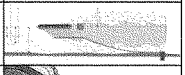
APPENDIX B

Boom Projects

BMT Fleet Technology Limited

Minerals Management Services, Arctic Region	Mr. Joseph Mullin	Develop a method for the recovery of oil spilled in ice covered water. A modified ice boom design will be used.	Prudhoe Bay	September 1999	
Ontario Hydro	M. Tony Bennett	A study to define the factors that caused the failure of two safety booms.	Chats Falls, Des Joachims*	November 1999 (Project in progress).	
Hydro-Québec	M. Gaétan Lesage	Design of a debris boom to retain floating debris masses.	Complex Lagrande, LG1	December 1999.	
Ontario Hydro, Pickering Nuclear GS	Mr. Bob Ross	Design, fabrication and deployment of an ice and safety boom to protect the Pickering Nuclear Power Plant intake from ice and from people entering the intake.	Lake Ontario, Pickering*	August 2000.	
Canadian Hydro Developers	Mr. Mike Stockton	Design, fabrication and deployment of an ice boom to minimize power losses due to ice blockages.	Appleton, Ontario	November 2000 In progress	
Simard Beaudry	Mr. Denis Robitaille	Concept design of an ice boom that was installed to facilitate the construction of a bridge on the Richelieu River.	South of Montreal, St Bruno	September 2000	
Canadian Coast Guard, Laurentian Region	Mr. Stéphane Dumont	A study to evaluate the feasibility of applying a new technique to sink and re-float ice booms for the Yamachiche boom.	The Port of Prescott and in Lac St. Pierre at Yamachiche	June 2000 – May 2001	
Ministry of Transport, NWT	Mr. Les Shaw	Site visit and concept design development for an ice boom to stabilize an ice bridge, along the Mackenzie Highway.	Tulita, Great Bear River	September 2000	
Bue Offshore	M. C Grolaston	Design of ice boom systems to protect an offshore living facility to be deployed in the Caspian Sea.	Caspian Sea	September 2001 to Nov 2001	
Hydro-Québec	M. Gaétan Lesage	Phase II: Detailed design of a debris boom to retain floating debris masses.	Complex Lagrande, LG1	January to March 2002.	
Hydro Quebec, Maisonneuve Region	Mr. Alain Cyr/ Andre Chouinard	Ice observations and assessment of Hull 2 hydroelectric power plant's winter performance.	Ottawa River near Hull, Quebec.	September 2001 to March 2002.	

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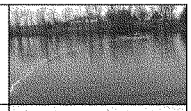
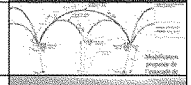
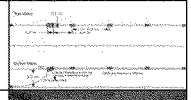
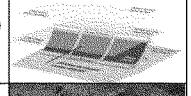
APPENDIX B

Boom Projects

BMT Fleet Technology Limited

Canadian Hydro Developers and IRAP	Mr. Mike Stockton / Ms. Liza Medec	Development of a boom using a net to deploy in fast moving currents	Appleton, Ontario.	November 2001 to February 2002	
Hydro Quebec, Maisonneuve Region	Mr. Alain Cyr/ Andre Chouinard	Design of an ice and debris boom for the Bryson Power Plant.	Ottawa River near Bryson.	September 2001 to March 2002.	
Hydro-Québec	M. Raymond-Marie Tremblay	Construction of an ice boom with a net to reduce the amount of frazil drifting toward the Rivière Des Prairies Power Plant.	Rivière des Prairies	September 2001 to April 2002	
Hydro-Québec	M. Raymond-Marie Tremblay	Improve the efficiency of the boom without a net.	Rivière des Prairies	April to September 2002	
Hydro Quebec, Maisonneuve Region	Mr. Alain Cyr/ Andre Chouinard	Construction of an ice boom for the Bryson Power Plant.	Ottawa River near Bryson.	October to December 2002.	
Domtar Inc., Eddy Specialty Papers.	Mr. Jim Collings	The development of a safety boom design criteria and the development of a design for implementation in 2003.	Ottawa River, Ottawa-Gatineau	September to November 2002	
Hydro Quebec, Maisonneuve Region	Mr. Andre Chouinard	Installation of a web camera via satellite communication system to observe Bryson Ice Boom.	Ottawa River at Bryson.	November to December 2002.	
Domtar Inc., Eddy Specialty Papers.	Mr. Jim Collings	The installation of an 800m long safety boom. The boom consisted of eight spans and nine anchors.	Ottawa River, Ottawa-Gatineau	September to November 2003	
Ministry of Environment of Québec	Mr. Jean-Francois Cyr	Feasibility study to assess the effectiveness of a headloss boom to control the outflow of water from a lake.	Lac Des Deux Montagnes, Montreal	January to April 2003 (in progress)	
Ontario Power Generation	Andre Bourbonnais	Approximately twenty-six booms were designed and installed by others on a number of watercourses for Ontario Power Generation during summer 2003. Several of the booms were damaged or broke and some were completely removed from the sites. BMT analysed the failure mechanisms and recommended corrective actions.	Renfrew, Ontario	June 2004	
Hydro Quebec, Maisonneuve Region	André Chouinard	Design, fabrication and installation of two booms, one for ice retention and one for security purposes. This project included the installation of a web camera to facilitate observation of the boom site.	Gatineau River at Farmers Rapids	October to December 2004	

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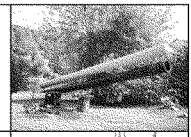
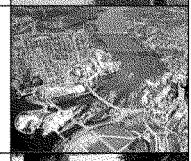
APPENDIX B

Boom Projects

BMT Fleet Technology Limited

Hydro Québec, Région Beauharnois	Raymond-Marie Tremblay	This project consisted of the design and installation of a boom to control ice and debris. The boom included a section of pontoons with neutrally-buoyant pipes below the surface pontoon in order to retain submerged debris up to one metre underwater.	Red River at Chute Bell	July to October 2004	
Hydro Québec, Région Beauharnois	Yves Leduc	Concept design of three safety booms; two to be placed upstream and downstream of the Carillon hydroelectric station, and; one to be placed downstream of the Chute Bell station.	Red River and Ottawa River	November to December 2004	
Domtar Inc., Eddy Specialty Papers.	Brian Overton	The study developed design criteria for the construction of three safety booms downstream of the Chaudière Dam and surrounding hydroelectric stations. The booms keep the users of the Ottawa River from coming close to the dangerous flows that can originate from the powerhouses and spillways.	Ottawa River, Ottawa-Gatineau.	September 2004 to May 2005	
Hydro-Québec, Région Montréal	Raymond-Marie Tremblay	This study measured the noise levels generated by the boom deployed on the Des Prairies River during the 2004/055 winter. The noise level was found to be directly proportional to the flow in the river.	Rivière des Prairies, Laval QC	September 2004 to April 2005	
Ontario Power Generation	Steve Chu	This study looked at options to improve the effectiveness of an oil-skimming boom system currently deployed downstream of an electrical generating station on the north shore of Lake Erie. The boom captures oil that is accidentally spilled or leaked into the cooling water system for the plant.	Lake Érie	September 2004 to April 2005	
Ontario Power Generation	Barrie Askew	This project consisted of the design, fabrication and installation of a boom to control ice and debris.	Seymour	April to December 2005	
Hydro Quebec, Maisonneuve Region	Henri Bertrand	This project consisted of the design, fabrication and installation of a boom to control divert debris to a desired location.	Bryson	September to November 2005	

** These projects were existing booms that BMT Fleet Technology Limited re-engineered to improve their performance in retaining ice.*

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RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT K – EROSION CONTROL METHODS FOR LOW FLOW CHANNEL

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

By: Barr Engineering Co.

FINAL – February 28, 2011

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT K – EROSION CONTROL METHODS FOR LOW FLOW CHANNEL

The following information was previously presented as Exhibit H of Appendix F of the Phase 3 report submitted on August 6th, 2010, and is included here for completeness.

F-K1.0 BACKGROUND INFORMATION

The proposed low-flow channel would consist of a meandering channel within the larger flow diversion, at least for the reach from the confluence of the diversion channel with the Lower Rush River downstream to the confluence of the diversion channel with the Red River of the North (RRN). This reach would receive all flow from the Lower Rush River and the Rush River, and the meandering low-flow channel would provide some measure of mitigation for the loss of the tributaries east of the diversion. The low-flow channel will also extend upstream of the Lower Rush River, since this reach would receive direct precipitation runoff as well as runoff from tile inlets and other miscellaneous inflows. The focus here is on the portion that is downstream of the Lower Rush River, since it will convey the most flow and will be subject to backwater effects from the RRN.

F-K2.0 HYDROLOGY OF THE LOW FLOW CHANNEL

It is assumed that the low-flow channel would convey flows up to the bankfull flow in either the Rush or Lower Rush, plus the coincidental flow in the other channel and the direct runoff from the upstream diversion channel. The bankfull flow rate is often cited as having a 1.5 year return frequency, but may typically range from an approximate 1-year to 2-year return frequency. The bankfull flow rate for either stream has not been measured or calculated, however individual 2-year flows are estimated to be 415 cfs and 302 cfs for the Rush and Lower Rush, respectively. The bankfull flow in either stream is likely to be somewhat less; the coincidental flow from the two channels and watershed inputs would need to be considered together. For now, we will assume that the minimum design flow for the low-flow channel is 415 cfs.

F-K3.0 HYDRAULICS OF THE LOW FLOW CHANNEL

The conceptual design for the low-flow channel consists of a 10-foot bottom width, 4:1 side slopes and 3-foot depth. The low-flow channel would have a top width of approximately 34 feet. The average slope of the diversion channel is 0.0002. However, because the low-flow channel will be meandering it will have a lower slope – perhaps as low as 0.00013 for a sinuosity of 1.5. This slope is very mild, and would result in a bankfull capacity of 40 to 60 cfs for the range of likely channel slopes, assuming a Manning's "n" value of 0.035. In order to convey the assumed design flow, the channel would need to be significantly larger in cross-section. The current channel geometry of the Rush River would provide a useful starting point of reference. Preliminary review indicates that it is indeed a significantly larger channel than the current low-flow channel.

F-K4.0 ALTERNATIVE LOW FLOW CHANNEL DESIGN

If the low-flow channel size is indeed increased significantly, the top width will rapidly approach that of the primary channel, which will have a bottom width of 100-feet. An alternative design for the low-flow channel would be to simply create a "V" shaped bottom to the primary channel. The low point of the "V" could follow a sinuous pattern within the primary channel. Such a channel would accommodate the anticipated large range of inflows from the Rush and Lower Rush in a controlled fashion.

F-K5.0 BACKWATER FROM THE RED RIVER OF THE NORTH

As the diversion channel approaches the RRN on the downstream side, it will become influenced to a greater degree by periodic inundation from sub 5-year flows in the RRN. Those flows are large enough to submerge the downstream end of the diversion channel, but not large enough to flow into the diversion from the upstream side. Inundation here is defined as overtopping the banks of the low-flow channel. As a point of reference, the 90-percent exceedance flow (1,633 cfs) will inundate approximately 2,000 feet or 3 percent of the diversion channel (as measured from the RRN to the Lower Rush). The 80-percent exceedance flow (2,515 cfs) will inundate approximately 12,000 feet or 20 percent of the diversion channel. The duration of flooding will need to be considered in the selection of appropriate vegetation for these areas, or hard armoring may be required if it is determined that the vegetation cannot survive the inundation.

F-K6.0 STABILIZATION ALTERNATIVES

Riprap lining of the low-flow channel has been suggested as a stabilization measure. The cost of armoring the 10-foot wide low-flow channel has been estimated at $20 million for the reach from the Lower Rush to the RRN, and $60 million for a low flow channel throughout the entire diversion channel (not accounting for sinuosity of the low flow channel).

With the alternative channel design recommended above, it is suggested instead that only the lower banks of the primary channel be armored with riprap to a depth below that of anticipated scour. The armoring should extend to a height that would protect against very frequent flows. The proposed armoring method would extend 3 feet below the primary channel bottom, 4 feet wide at the primary channel bottom, with a 12-inch layer extending 10-feet up the primary channel bank from the bottom (protecting 1-foot vertically above the primary channel bottom). It is estimated that this will require approximately 35% less riprap compared to armoring a narrower low-flow channel. The riprap should be covered with topsoil and vegetated in order to provide a natural appearance.

A number of measures could be introduced to stabilize the bottom of the primary channel/low-flow channel while preserving ecological function. Some portion of the bottom would not be amenable to vegetation establishment since it would have flowing water for much of the time. The most important stabilization measure would be native

grasses that are appropriate for the soil and climate conditions. Suitable grasses may include cord grass, switch grass, bluestem and Indian grass. These deep-rooted grasses would help to maintain the integrity of the low-flow channel banks.

Grade control may be necessary depending on the final geometry of the low-flow channel and detailed channel hydraulics. For cost-estimating purposes it is recommended that grade-control measures be installed across the entire width of the primary channel every 5,000 feet of channel length (1-foot of primary channel drop). The grade-control measures would be largely buried unless erosion exposes them. It is assumed they would be constructed of rock material but sheet pile could also be used. The rock grade control is assumed to be 100-feet wide (primary channel width) by 20 feet wide by 5 feet deep (370 cubic yards per grade control structure). Approximately 12 structures would be required in the reach downstream of the Lower Rush River. As with the bank protection, the rock should be covered with topsoil and vegetated in order to provide a natural appearance.

F-K7.0 CONCLUSION

The intent of the low-flow channel is to mitigate the loss of habitat from diverting the Rush and Lower Rush Rivers into the diversion channel. It appears that the low-flow channel as currently designed is significantly undersized and would overtop its banks very frequently. The hydrology of the Rush and Lower Rush will have to be considered carefully, and other contributing watershed area will need to be considered to develop an appropriate channel size to more closely mimic natural conditions. An alternative low-flow channel design has been proposed consisting of a shallow "V" shaped bottom to the primary channel. The bottom would be stabilized with native grasses and supplemented by periodic grade-control measures for additional protection. The duration of inundation of the diversion channel by the RRN will need to be considered, but should impact a relatively small portion of the channel.

RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT L – REVIEW OF GEOTECHNICAL DATA

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

By: Barr Engineering Co.

FINAL – February 28, 2010

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT L - REVIEW OF GEOTECHNICAL DATA

F-L1.0 INTRODUCTION

This exhibit presents a review and interpretation of the existing geotechnical data to generate the parameters used in the preliminary design. The data described herein were collected during The Fargo-Moorhead Metro Feasibility Study (FMMFS) initiated by the U.S. Army Corps of Engineers (USACE) in 2008. Most of the data collected for use during Phase 3 of the FMMFS are associated with what was known as the In-Town Levee Alternative and the Minnesota Diversion Alternative. Additional data were collected in 2010 for use during Phase 4 of the FMMFS. The locations for data collection focused on the proposed structure locations where minimal information was available.

Currently, there are two alternatives being evaluated and they are the North Dakota Diversion (LPP) Alternative and the Minnesota Diversion (FCP) Alternative. The existing data were utilized for this Phase 4 report and it is considered representative of the overall ground conditions for the concept feasibility designs presented in Appendix F.

F-L2.0 GEOTECHNICAL EXPLORATION

The USACE performed a geotechnical exploration as part of the FMMFS. The results of the exploration are summarized in Appendix I of the report USACE (2010). The regional design issues associated with geology, topography, hydrogeology, and seismic risk are discussed in detail in that report and thus are not addressed herein. This exhibit focuses on the site specific data utilized for feasibility design purposes. As a result, the following review discusses the field exploration and laboratory testing as they relate to the design parameters.

F-L2.1 FIELD EXPLORATION

The field exploration conducted by the USACE included machine borings for disturbed sample collection and offset boreholes for undisturbed sample collection. Additionally, Cone Penetration Test (CPT) soundings were conducted to obtain in-situ soils information. The boring logs of the machine borings and CPT logs of the CPT soundings are included in Appendix I of the USACE (2010) report.

F-L2.1.1 Soil Borings

The initial investigation program during the Spring/Summer 2009 included 51 exploratory borings (machine borings) for the In-Town Levee Alternative. There were a total of 10 offset holes for undisturbed sample collection during this initial investigation. Subsequently, there were 85 machine holes completed also for the In-Town Levee Alternative distributed as 45 holes on the Fargo side and 40 on the Moorhead side.

The investigation for the Minnesota Diversion Alternative included 40 exploratory borings. A subsequent subsurface investigation was completed in December 2009 along the North Dakota diversion alternative. This investigation also included 2 offset holes for collection of undisturbed samples.

The exploratory borings were conducted using an all-terrain vehicle (ATV) rig. The borings were conducted using continuous sampling alternated with split spoon sampling. The undisturbed samples were classified in the field by a geologist and stored in jars for later testing in the laboratory.

The offset holes were performed near the machine borings with the purpose of obtaining 5-inch undisturbed samples. One undisturbed soil sample was obtained for each formation in each offset hole. A total of 12 undisturbed borings were completed, obtaining 46 undisturbed samples. These undisturbed borings were distributed throughout the project, with four being completed for the in-town levee alignment in Fargo, three for the in-town levee alignment in Moorhead, three along the Minnesota Diversion alternative, and two along the North Dakota Diversion alternative.

The subsequent Phase 4 program carried out during the Spring/Summer 2010 included a total of 47 borings, with 17 on the Fargo side and 30 on the Moorhead side.

F-L2.1.2 CPT Soundings

A total of 64 CPT soundings were performed during the Phase 3 field investigation. A total of 45 soundings were performed on the Moorhead side and 19 soundings on the Fargo side. All the CPT soundings were performed as part of the In-Levee Alternative. The CPT soundings were conducted using a 20-ton CPT truck mounted rig from the USACE Savannah District.

In the CPT a cylindrical cone is pushed vertically into the ground at a constant rate of penetration of 20 mm/s. During penetration, measurements are made of cone tip resistance, side friction of the cylindrical shaft just above the tip, and pore-water pressure generated by cone penetration.

F-L2.2 LABORATORY TESTING

Disturbed samples collected during the field exploration were taken to the laboratory for index property testing. Undisturbed samples collected during the field exploration were taken to the laboratory for undisturbed laboratory testing including shear strength and compressibility testing. The following briefly describes in more detail the laboratory testing performed on disturbed and undisturbed samples.

F-L2.2.1 Index Properties

Index property testing was mainly performed on disturbed samples and included water content, unit weight, specific gravity, liquid limit, plastic limit, and grain size. The tests were performed following the appropriate ASTM standard. The results of this testing helped identify characteristics of the soils and define the stratigraphy. This characterization allowed identification of the different soil formations at the site

including: Alluvium, Sherack, Poplar River, Brenna, Oxidized Brenna, Argusville, and Till. Section F-L3.0 discusses the findings regarding index properties.

F-L2.2.2 Shear Strength Tests
The majority of the laboratory testing had the objective of determining the shear strength of the different soils involved. The shear strength testing involved the following tests: 1) isotropically consolidated undrained triaxial compression test with pore-water pressure measurements (R-Bar tests); 2) direct shear test (DS); and 3) unconsolidated undrained tests (Q tests). Some limited index property testing was also performed on undisturbed samples. Section F-L3.0 discusses the findings regarding strength tests.

F-L2.2.3 Compressibility
A limited number of consolidation tests were performed on undisturbed samples. Section F-L3.0 discusses the findings regarding compressibility.

F-L3.0 INTERPRETATION OF GEOTECHNICAL DATA

F-L3.1 INDEX PROPERTIES

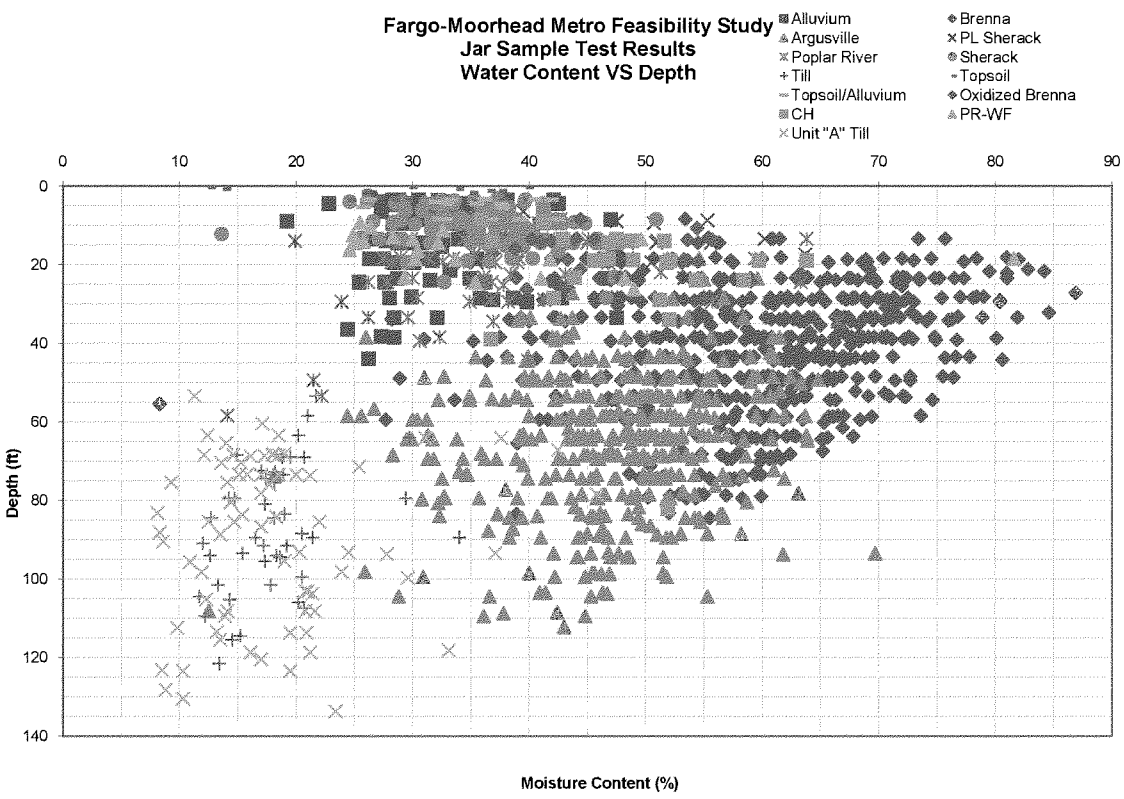
This section discusses the results of the index properties of the different formations encountered at the site. Some of the data include test results of the same formations in samples taken in other USACE projects involving Lake Agassiz clays.

F-L3.1.1 Water Content
Figure F-L1a shows the results of the moisture content versus depth for the different formations in the study area. It can be seen from Figure F-L1a that water content ranges between 8 and 86 percent depending on the formation. Figure F-L1b shows the results of the moisture content versus elevation for the different formations.

Table F-L1 summarizes the average water content and the standard deviation for each formation. It can be seen from Table F-L1 that the Brenna Formation exhibits the highest average water content with a value of 59.2 percent whereas the Till exhibits the lowest average water content with a value of 18.1 percent.

Table F-L1 Average Water Content Per Formation

Formation	Average Water Content (%)	Standard Deviation (%)
Alluvium	31.78	5.13
Argusville	47.13	8.16
Brenna	59.24	10.65
Oxidized Brenna	49.00	7.43
PL Sherack	50.75	7.01
Poplar River	36.98	10.14
Sherack	36.48	5.45
Till	18.14	4.41
Unit "A" Till	18.26	7.60
Topsoil/Alluvium	35.19	4.72



Fignre F-L1a Water Content vs. Depth

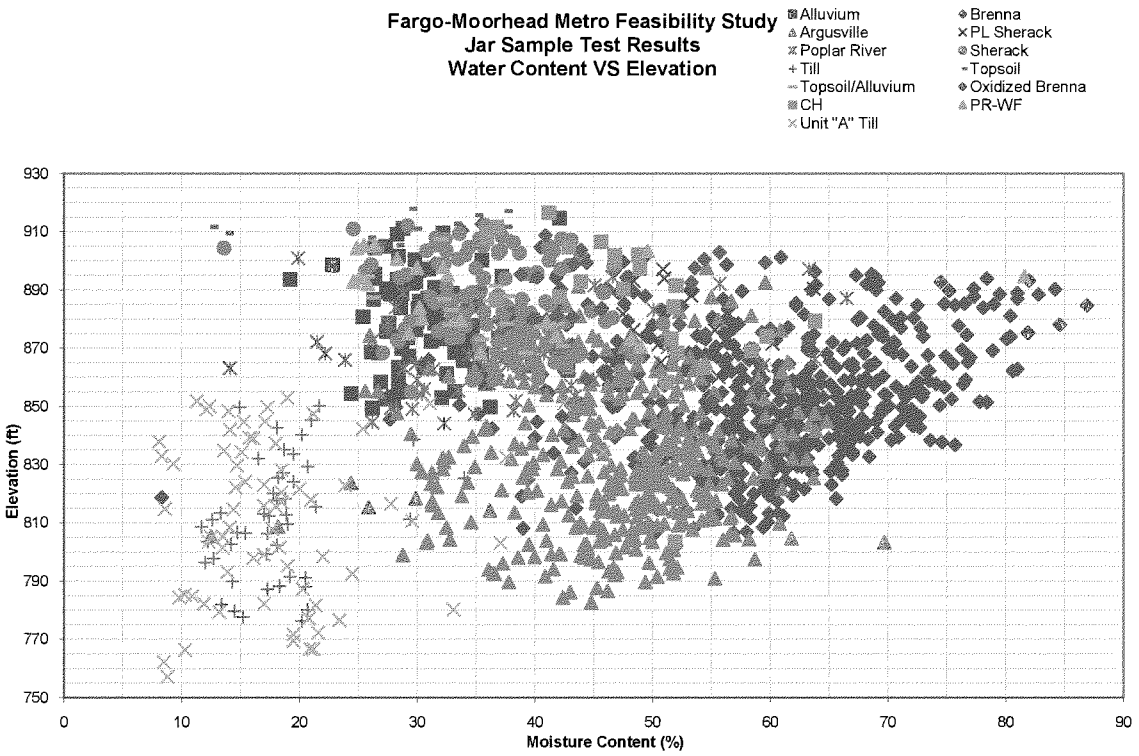


Figure F-L1b Water Content vs. Elevation

F-L3.1.2 Unit Weight

Table F-L2 summarizes the average saturated unit weight and the standard deviation for each formation. It can be seen from Table F-L2 that the Brenna Formation exhibits the lowest average saturated unit weight with a value of 104.1 pcf whereas the Poplar River-West Fargo exhibits the highest average saturated unit weight with a value of 123.2 pcf.

Table F-L2 Average Unit Weight Per Formation

Formation	Average Unit Weight (pcf)	Standard Deviation (pcf)
Alluvium	119.9	4.8
Sherack	117.6	4.0
PL Sherack	113.9	0.7
Poplar River - West Fargo	123.2	1.3
Poplar River - Harwood	115.6	0.5
Poplar River	119.4	3.9
Oxidized Brenna	111.3	6.2
Brenna	104.1	9.0
Brenna/Argusville Transition	107.8	3.8
Argusville	106.5	3.2

F-L3.1.3 Atterberg Limits

Figure F-L2a shows the results of the liquid limit versus depth for the different formations in the study area. It can be seen from Figure F-L2a that water content ranges between 1 and 134 percent depending on the formation. Figure F-L2b shows the results of the liquid limit versus elevation for the different formations.

Table F-L3 summarizes the liquid limit and the standard deviation for each formation. It can be seen from Table F-L3 that the PL Sherack exhibits the highest average liquid limit with a value of 100.3 percent and the Brenna Formation exhibits the second highest average liquid limit with a value of 95.4 percent whereas the Till exhibits the lowest average liquid limit with a value of 29.6 percent.

Table F-L3 Average Liquid Limit Per Formation

Formation	Average Liquid Limit (%)	Standard Deviation (%)
Alluvium	57.9	11.2
Argusville	72.0	14.2
Brenna	95.4	15.0
Oxidized Brenna	93.5	17.6
PL Sherack	100.3	8.6
Poplar River	50.6	18.1
Sherack	64.9	14.3
Till	29.6	6.2
Unit "A" Till	27.8	7.1
Topsoil/Alluvium	58.9	7.8

The liquid limit and water content data were used to compute the liquidity index. Figures F-L3a and F-L3b show the liquidity index versus depth and elevation for the different formations in the study area, respectively. It can be seen from the figures that most of the data are between a liquidity index of 0.0 and 0.7.

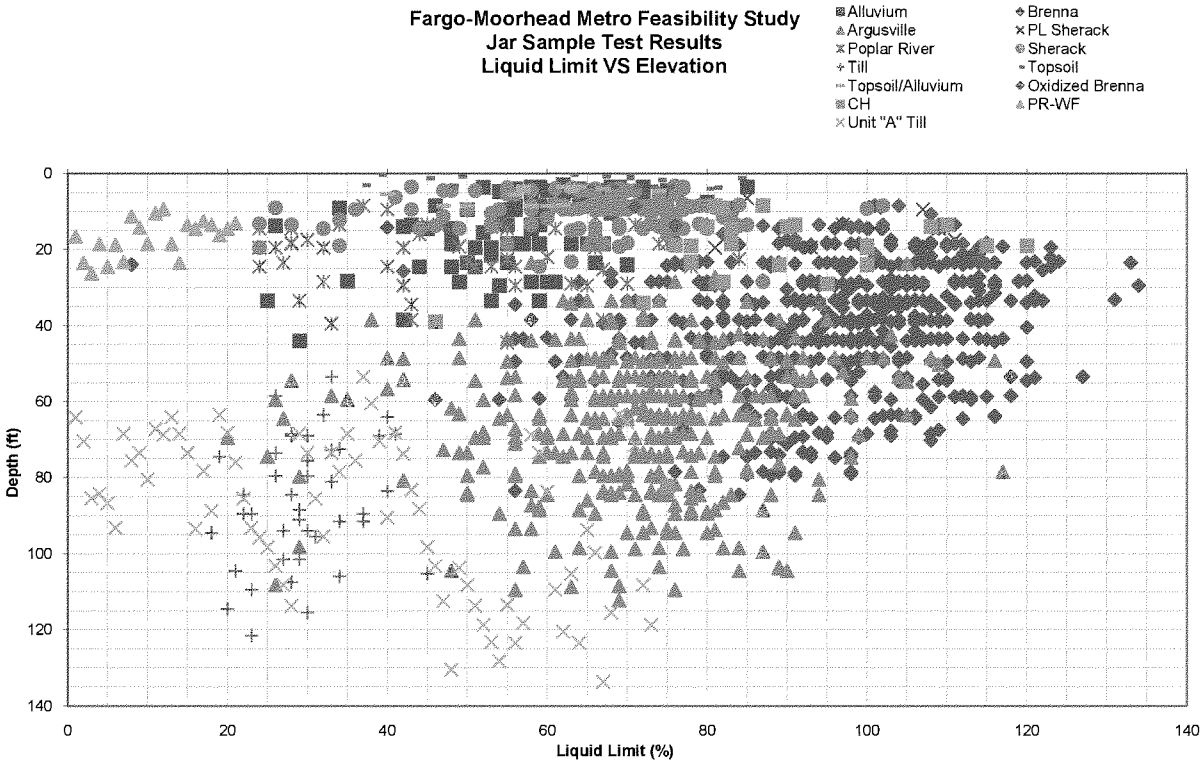


Figure F-L2a Liquid Limit vs. Depth

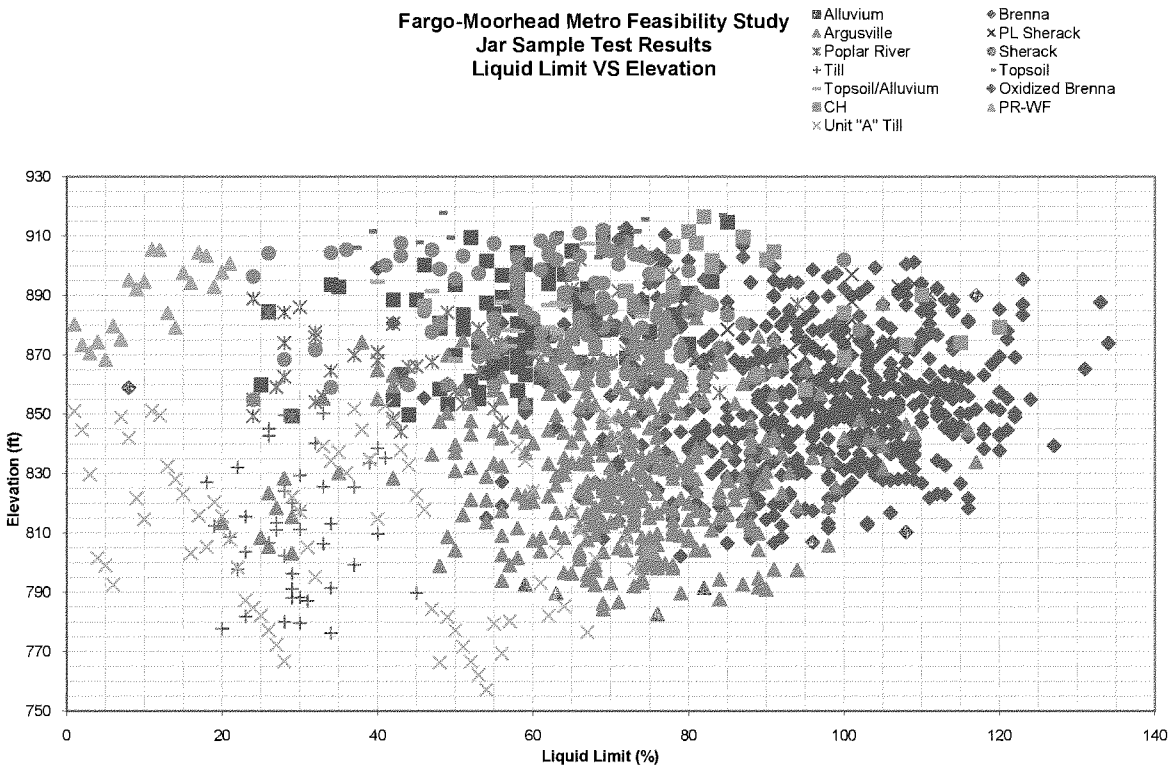


Figure F-L2b Liquid Limit vs. Elevation

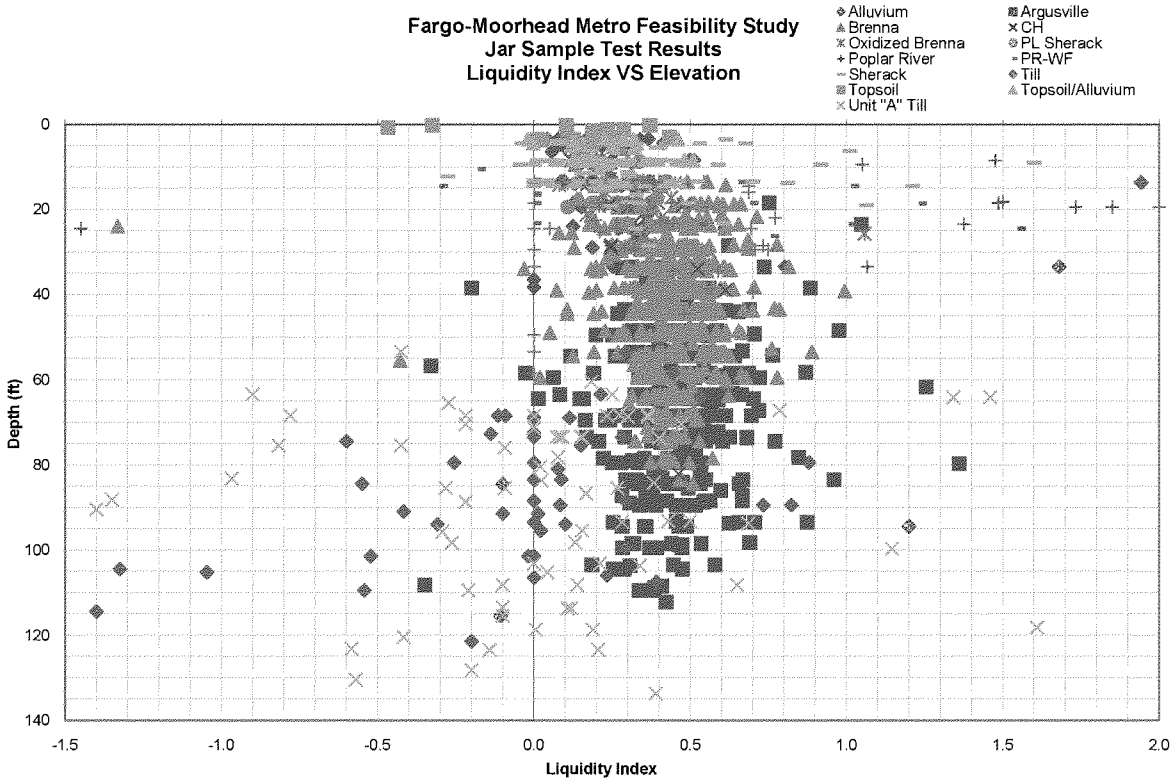


Figure F-L3a Liquidity Index vs. Depth

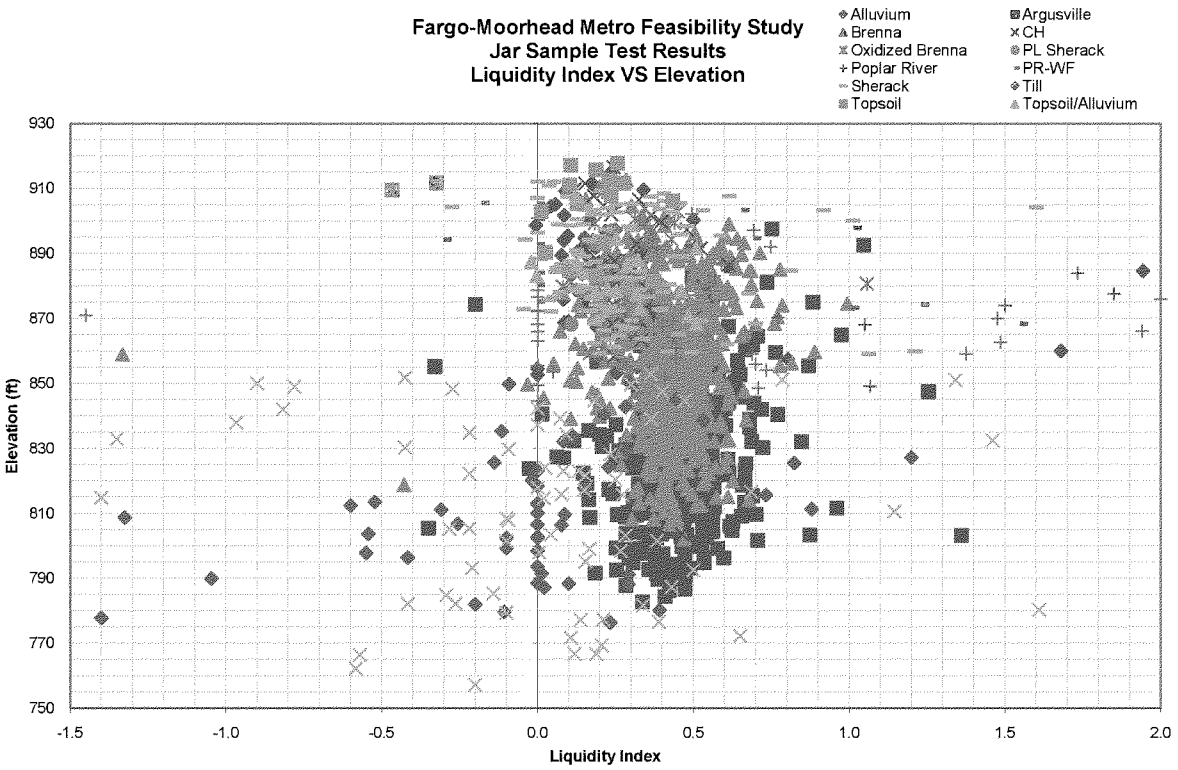


Figure F-L3b Liquidity Index vs. Elevation

F-L3.2 SHEAR STRENGTH

This section of the report discusses the shear strength of the soils involved in the project. The shear strength discussion utilizes the laboratory tests and CPT results. The interpretation of the shear strength evaluates the undrained and drained shear strength.

F-L3.2.1 Undrained Shear Strength

The undrained shear strength was initially interpreted based on the laboratory test results. The FMMFS included a significant amount of laboratory undrained shear strength data. The data were collected from Unconsolidated Undrained (UU) tests on undisturbed samples from different formations. The standard procedure used in the FMMFS consisted of testing three specimens at a given depth.

Figure F-L4 shows the results of UU tests versus depth of the different formations encountered at the study area. All of the tested samples included in the FMMFS are within the upper 85 ft. Each point in Figure F-L4 represents the average of the three specimens tested at that depth. It can be seen in Figure F-L4 that the shear strength plot shows the typical results of a clay deposit with a desiccated crust in the upper 15 to 20 ft and then the undrained shear strength tests increases linearly with depth. Figure F-L4 also includes two lines that represent the upper and lower bound of the data. In the upper portion of Figure F-L4, generally above 15 ft to 20 ft, data correspond to the Sherack, Poplar, and Oxidized Brenna. On the other hand, below 20 ft, most of the data are associated with the Brenna and Argusville formations.

The undrained shear strength was also interpreted using the CPT sounding results. An extensive amount of CPT soundings were available from the FMMFS and were utilized to estimate the undrained shear strength using the following equation:

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where

q_t = is the corrected cone tip resistance
σ_{vo} = total vertical stress
N_{kt} = cone factor

The laboratory test data included in Figure F-L4 were used to estimate the cone factor N_{kt} as 19. Figure F-L5 shows the undrained shear strength vs. depth obtained from the CPT soundings in North Dakota (using an N_{kt} of 19). Figure F-L5 also includes the upper and lower bound from the CPT soundings included in Figure F-L4. It can be seen from Figure F-L5 that the undrained shear strength from the CPT shows the general tendency of a typical clay deposit with a desiccated crust at the top and then increasing with depth until it encounters the till at depths varying between 70 ft to 110 ft.

Similarly, Figures F-L6 and F-L7 show the undrained shear strength vs. depth obtained from laboratory testing and the CPT soundings in Minnesota (using an N_{kt} of 19), respectively. Figure F-L7 also includes the upper and lower bound from the CPT soundings included in Figure F-L6.

Figures F-L4 through Figure F-L7 validate the selected cone factor N_{kt} of 19. Furthermore, these figures are used to estimate undrained shear strength for design.

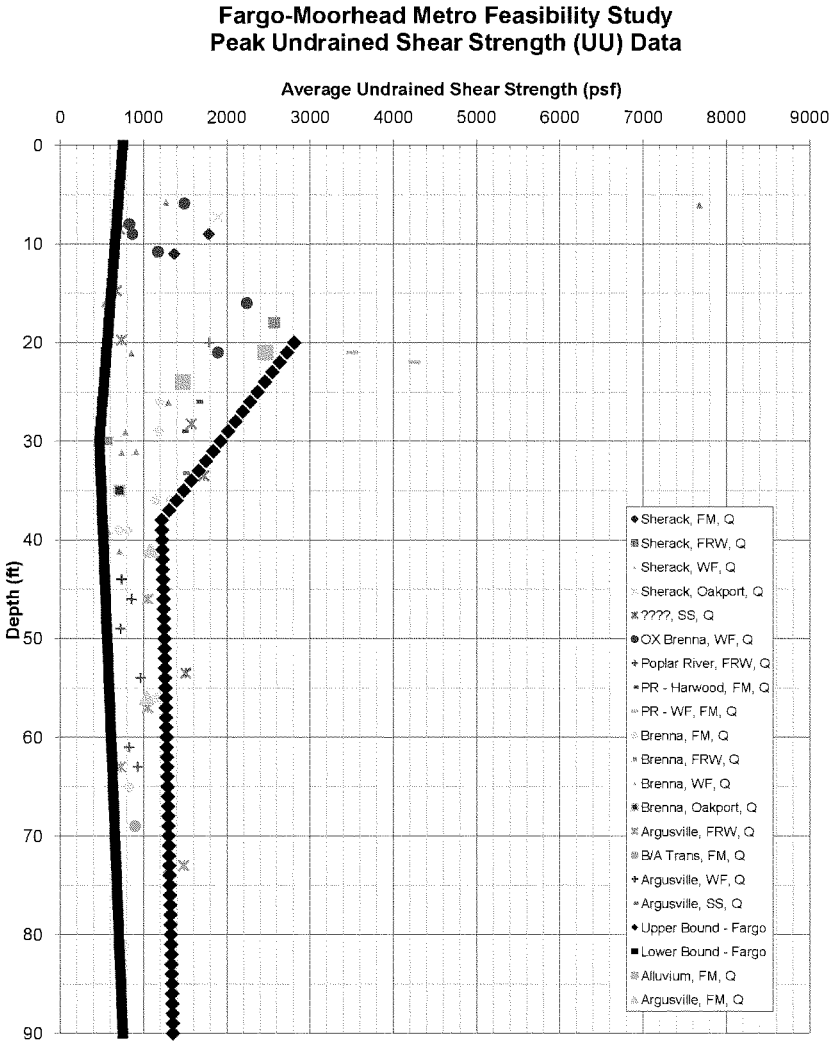

Figure F-L4 Peak Undrained Strength vs. Depth with Upper and Lower Bounds Taken From North Dakota CPT Soundings

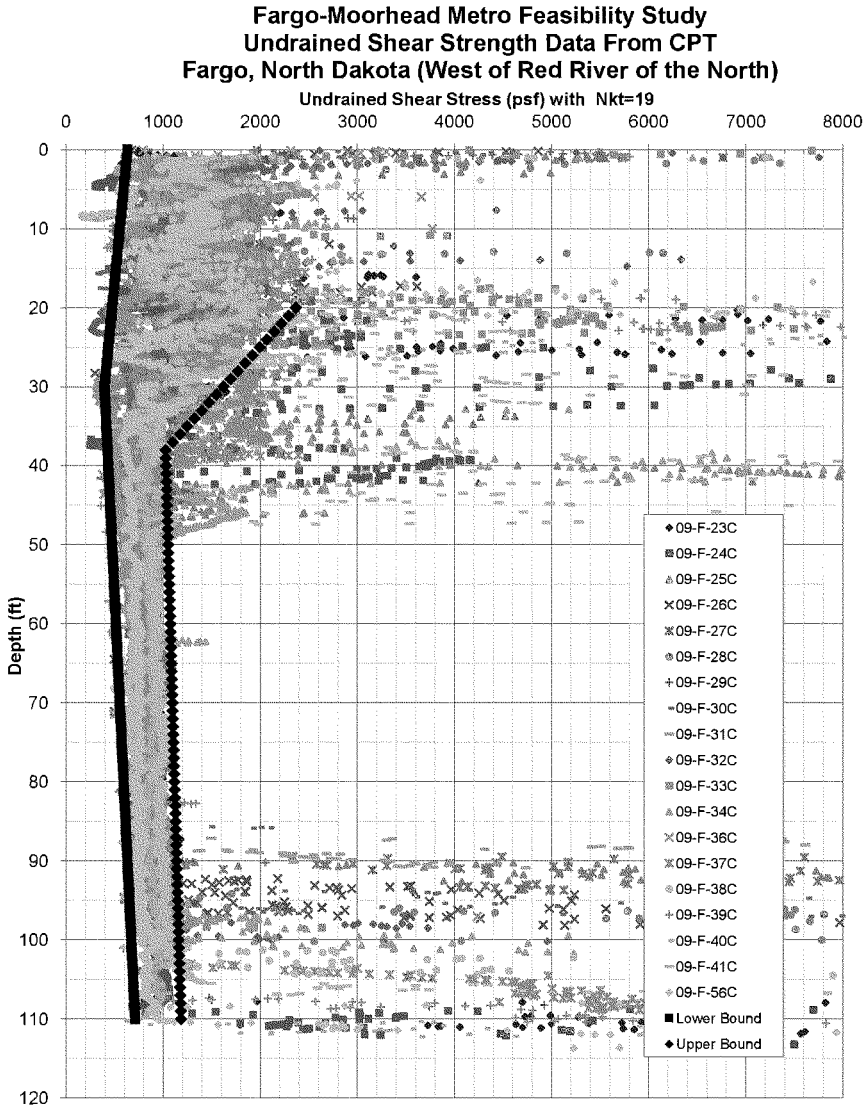

Figure F-L5 Undrained Shear Strength vs. Depth Obtained from CPT Soundings in North Dakota

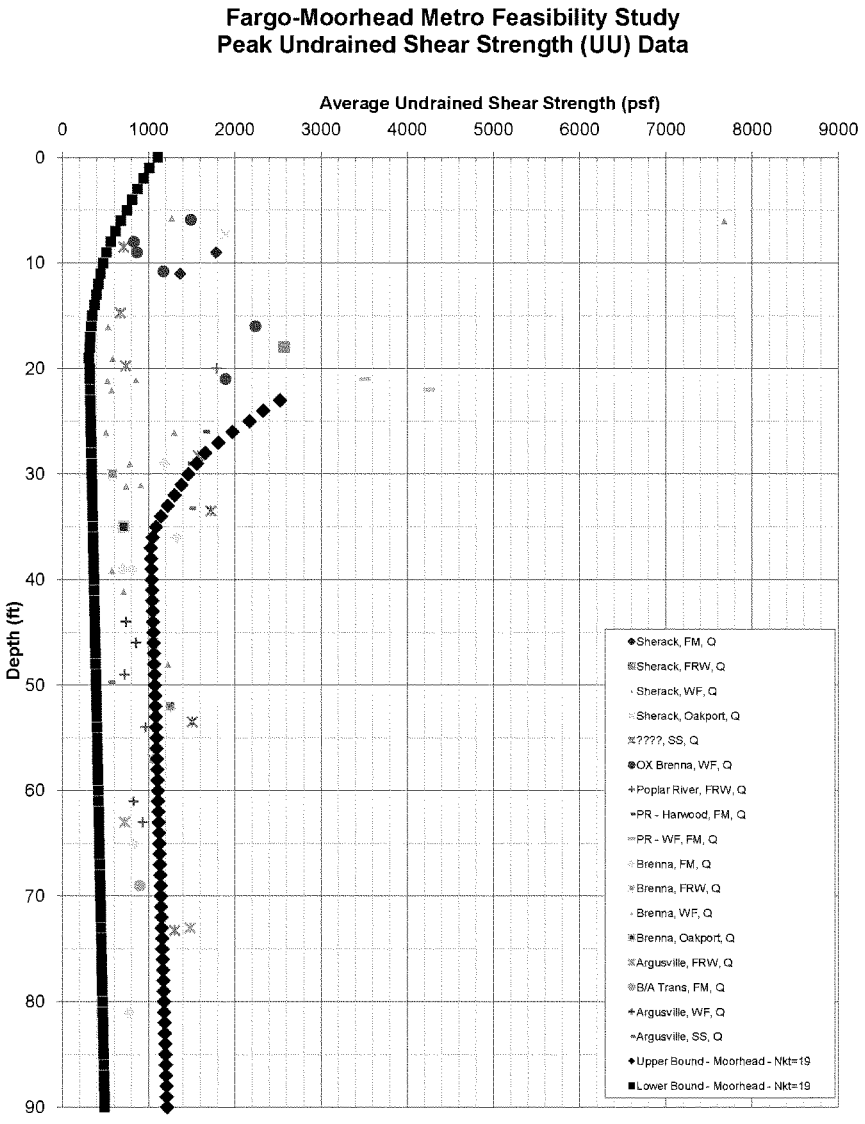

Figure F-L6 Peak Undrained Strength vs. Depth with Upper and Lower Bounds Taken From Minnesota CPT Soundings

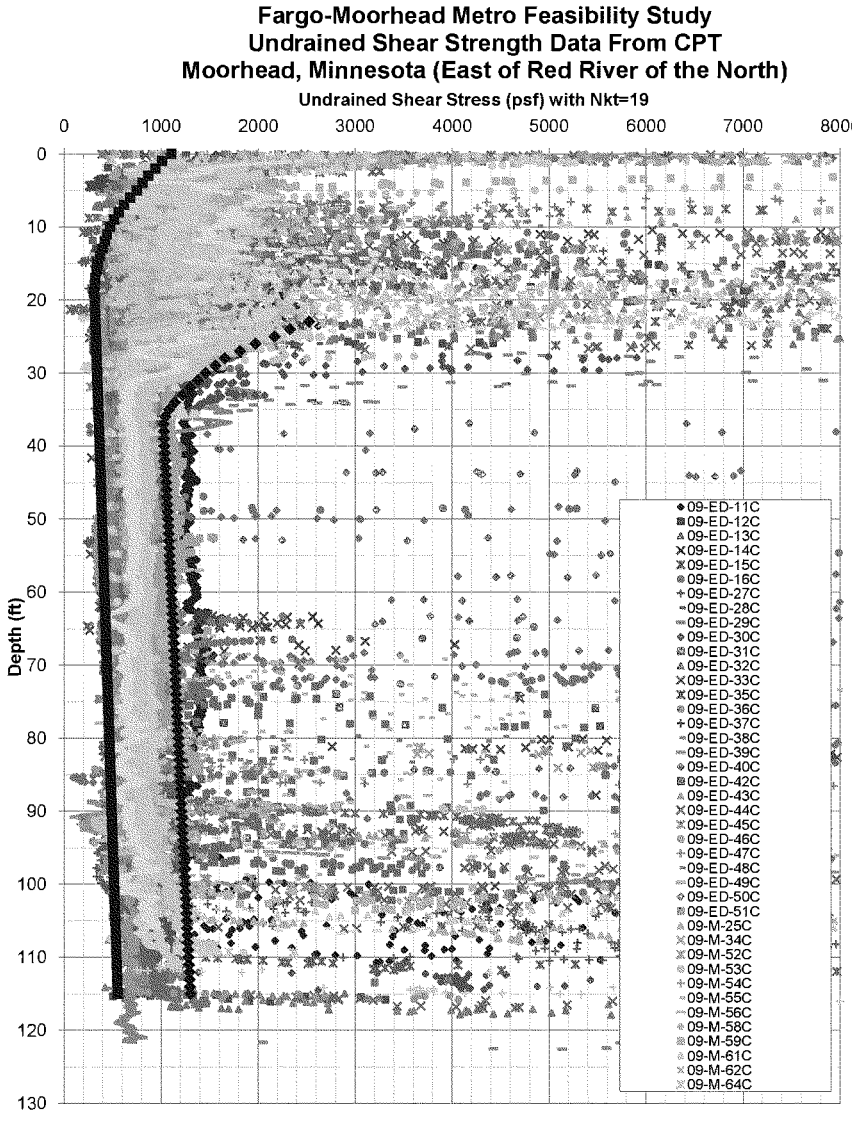

Figure F-L7 Undrained Shear Strength vs. Depth Obtained from CPT Soundings in Minnesota

F-L3.2.2 Drained Shear Strength

The drained shear strength was interpreted based on the laboratory test results on undisturbed samples. The laboratory testing included consolidated undrained triaxial shear tests with pore-water pressure measurements (R-bar) and direct shear tests (DS). All the laboratory data used to interpret the drained shear strength were included in the FMMFS and are part of a large database which contains information on Lake Agassiz soils from this and other projects.

Actual laboratory testing includes samples from the Sherack, Poplar River, Brenna, and Argusville formations. Data included in the FMMFS is summarized in Table F-L4.

Table F-L4 Summary of Drained Shear Strength

	Effective Stress [1]	
Formation	**ϕ′**	**c′ (psf)**
Alluvium	31	0
Sherack	28	0
Poplar River-West Fargo	34	0
Poplar River-Harwood	26	0
Oxidized Brenna [2]	19	0
Brenna [2]	13	0
Argusville [2]	15	0
Till	34	0

(1) The effective stress parameters for the formations are based on the R-Bar triaxial and direct shear tests. The failure criterion is defined as ultimate deviator stress which equates to the deviator stress at 15% and 20% axial strain. It is assumed that there is no cohesion intercept for the Mohr-Coulomb shear strength envelope.

(2) A curvilinear shear strength envelope was developed for the effective stress analysis of the diversion channel excavated slope. The curvilinear envelope is defined as the line one standard deviation less than the most likely value (MLV).

In the interpretation of the triaxial test data in terms of effective stress in Table F-L4, the failure criteria utilized the stress at 15% strain. This is known as the large strain failure criterion. The reason for utilization of this failure criterion is based on the USACE experience with the Red River Valley clays which exhibit the strain softening phenomena. The large strain failure criterion is applicable when analyzing channels, slopes, and levees which are subject to strain softening. In the case of analysis and design of shallow or deep foundations under drained conditions, the existing design methodology utilizes the drained strength defined by the peak deviator stress criterion. As a result, the triaxial laboratory data were revisited and new failure envelopes were developed using the peak deviator stress as failure criterion. More discussion on this topic is included in Exhibit O.

Figure F-L8 shows the results of the reinterpretation of the triaxial test results of the Brenna formation using the peak failure criterion. It can be seen from Figure F-L8 that utilizing the peak stress as failure criterion results in a higher effective friction angle than using the 15% strain criterion.

A similar reinterpretation of the triaxial test results using the peak failure criterion was performed for the Oxidized Brenna and Argusville formations. The use of the peak stress as

failure criterion resulted in higher friction angles than using the 15% strain criterion. These reinterpreted results were used in the analysis and design of deep foundations under drained conditions. This aspect is further discussed in Exhibit O.

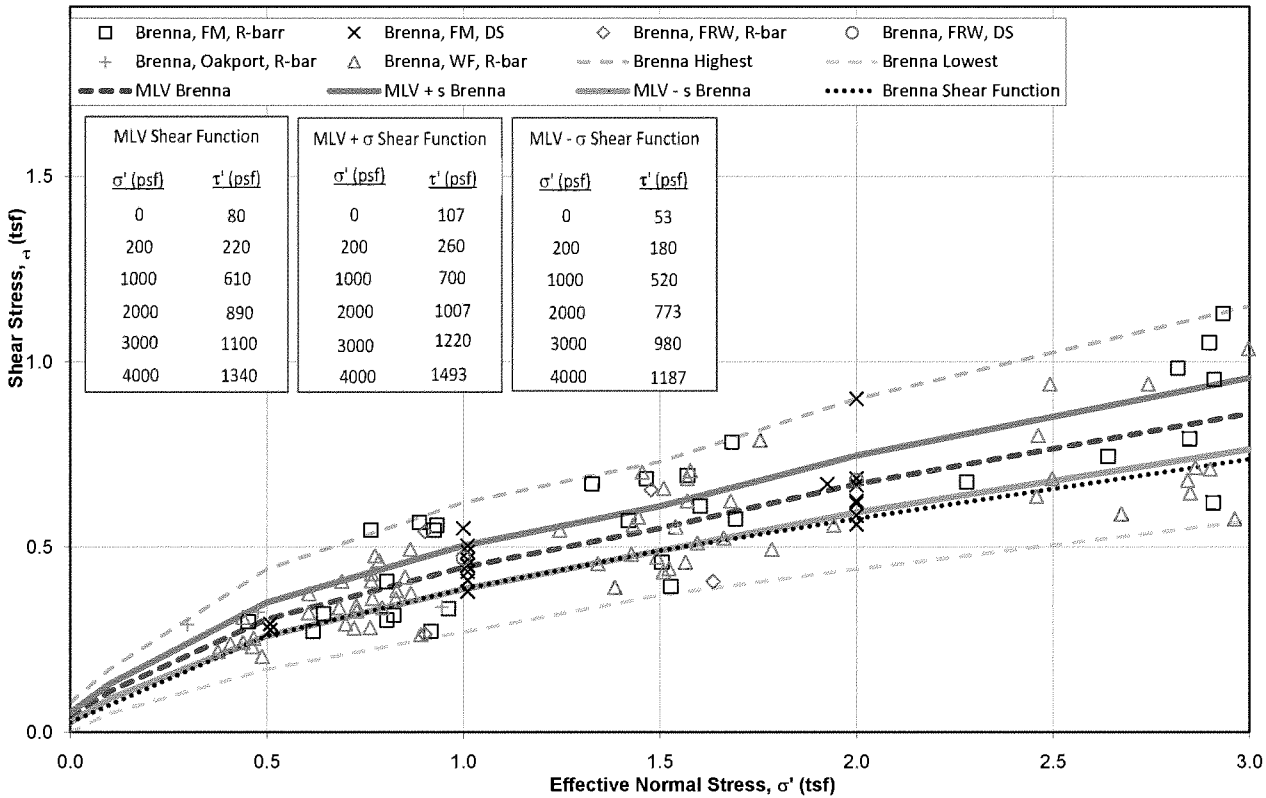

Figure F-L8 Shear Strength of Brenna Formation using Peak Stress as Failure Criterion

F-L3.3 COMPRESSIBILITY

The compressibility characteristics of the different soils involved in the study area were tested using the incremental loading test procedure. The results of the tests are summarized in Table F-L5.

Table F-L5 Summary of Consolidation Tests

Project, Formation, Boring, Sample No., Specimen No.	p_c' (tsf)	p_o' (tsf)	OCR	Cc	Cr	USCS Soil Type	Formation	Project
FRW PL Sherack 01-5MU, 1	1.95	0.56	3.48	0.22	0.07		PL Sherack	FRW
FRW PL Sherack 01-5MU, 1	1.95	0.84	2.32	0.22	0.07		PL Sherack	FRW
FRW Sherack 01-12MU, 1	3.20	0.72	4.44	0.17	0.04		Sherack	FRW
FRW Brenna 01-5MU, 2	2.95	1.60	1.84	1.16	0.19		Brenna	FRW
FRW Brenna 01-5MU, 2	2.95	1.80	1.64	1.16	0.19		Brenna	FRW
FRW Argusville 01-5MU, 4	2.60	2.20	1.18	1.05	0.18		Argusville	FRW
FRW Argusville 01-5MU, 4	2.60	2.30	1.13	1.05	0.18		Argusville	FRW
SS ST-10,	2.52	0.63	3.97	0.30	0.10	CH		SS
SS ST-10,	4.38	0.78	5.63	0.80	0.23	CH		SS
SS ST-10,	2.48	0.96	2.60	0.84	0.16	CH		SS
SS SB-11,	4.30	1.19	3.62	0.60	0.10	CH		SS
SS SB-17,	3.70	1.12	3.30	0.57	0.11	CH		SS
SS-WRL ST-7,	1.04	0.70	1.48	0.26	0.10	CH		SS-WRL
SS-WRL ST-7,	3.1	0.86	3.60	0.67	0.19	CH		SS-WRL
SS-WRL ST-9,	2.3	0.56	4.08	0.32	0.12	CH		SS-WRL
SA-52ndAVE ST-1,	4.0	1.09	3.65	0.76	0.15	CH		SA-52ndAVE
SA-52ndAVE ST-2,	4.5	0.72	6.22	0.33	0.04	CH-ML		SA-52ndAVE
SA-52ndAVE SL-3,	1.8	1.44	1.25	0.24	0.04	CH		SA-52ndAVE
SS-32ND ST-13,	2.0	0.82	2.43	0.17	0.03	CH		SS-32ND
SS-32ND ST-13,	2.7	0.97	2.80	0.83	0.15	CH		SS-32ND
FM PR - WF 09-25MU-M, 2	1.6	0.93	1.72	0.05	0.01	ML	PR - WF	FM
FM PR - Harwood 09-25MU-M, 3	3.4	1.04	3.27	0.29	0.07	CH	PR - Harwood	FM
FM Brenna 09-25MU-M, 4	2.9	1.34	2.15	1.28	0.24	CH	Brenna	FM
FM Argusville 09-25MU-M, 5	2.8	1.95	1.44	0.80	0.13	CH	Argusville	FM
FM Brenna 09-25MU-F, 4	2.8	1.70	1.62	1.10	0.23	CH	Brenna	FM
FM Sherack 09-26MU-F, 1	1.4	0.51	2.80	0.20	0.07	CH	Sherack	FM
FM Brenna 09-26MU-F, 3	3.7	1.04	3.56	0.77	0.19	CH	Brenna	FM
FM Alluvium 09-27MU-F, 1	1.6	0.42	3.81	0.23	0.08	CH	Alluvium	FM
FM Argusville 09-27MU-F, 4	2.8	1.82	1.51	0.87	0.16	CH	Argusville	FM
FM PL Sherack 09-34MU-M, 2	3.8	0.80	4.75	0.80	0.22	CH	PL Sherack	FM
FM OX Brenna 09-59MU-F, 2	3.0	0.95	3.17	0.65	0.17	CH	Brenna	FM
FM Brenna/Argusville 09-59MU, 3	2.8	1.30	2.16	0.41	0.08	CH	Brenna	FM
FM OX Brenna 09-60MU, 2	5.6	0.90	6.20	0.65	0.12	CH	Brenna	FM
FM Brenna 09-60MU, 3	4.7	1.10	4.25	1.06	0.17	CH	Brenna	FM
FM Alluvium 10-78MU, 1	2.9	0.61	4.75	0.28	0.03	CH	Brenna	FM
FM Brenna 10-78MU, 2	3.6	0.90	4.00	0.54	0.04	CH	Brenna	FM
FM Argusville 10-78MU, 3	4.5	1.60	2.81	0.82	0.05	CH	Brenna	FM
FM Alluvium 10-79MU, 1	4.0	0.90	4.44	0.31	0.05	CH	Brenna	FM

Project, Formation, Boring, Sample No., Specimen No.	p_c' (tsf)	p_o' (tsf)	OCR	Cc	Cr	USCS Soil Type	Formation	Project
FM Argusville 10-79MU, 2	3.9	1.44	2.71	0.69	0.05	CH	Brenna	FM
FM Alluvium 10-80MU, 1	2.0	1.00	2.00	0.14	0.02	CL-SM	Alluvium	FM
FM Brenna 10-80MU, 2	5.0	1.30	3.85	0.63	0.04	CH	Brenna	FM
FM Argusville 101-80MU, 3	4.4	1.70	2.59	0.73	0.09	CH	Brenna	FM

F-L4.0 SUMMARY

The existing geotechnical information from the FMMFS was reviewed. This included the field and laboratory data including index properties, shear strength, and compressibility. The information available was fairly complete for this concept feasibility design phase (additional data will be required for future phases and final design) and only two additions or revisions of the existing data were made to use in this Phase 4. The first addition was the interpretation of the CPT to estimate the undrained shear strength. In this regard, the existing information was further expanded with additional values of undrained shear strength from the CPT. The undrained shear strength is a very important parameter for deep foundation design. All the structures will be supported by deep foundations for this project. The other aspect reviewed was the drained shear strength of the Brenna, Oxidized Brenna, and Argusville formations to be used in the analysis and design of deep foundations under drained conditions. The data presented in the FMMFS utilized the large strain (15% strain) failure criterion, which accounts for the softening phenomena mainly applicable in design of slope stability problems. The data were revisited and a new failure envelopes were developed using the peak stress failure criterion. This design failure criterion is applicable in the design of deep foundations under drained conditions.

RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT M – SEEPAGE ANALYSIS

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

By: Barr Engineering Co.

FINAL –February 28, 2010

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT M - SEEPAGE ANALYSIS

F-M1.0 SEEPAGE ANALYSIS

The following information was previously presented as Exhibit J of Appendix F of the Phase 3 report submitted on August 6th, 2010, and is included here for completeness.

A transient seepage analysis was initially performed as part of Phase 3 to compute the anticipated uplift pressures along the bottom of the control structures on the Red River of the North and the Wild Rice River. 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-year 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 1st, 2010, between Barr Engineering, the US Army Corps of Engineers, 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-foot sheet piles are still anticipated to be installed on the upstream and downstream edges of the structures for piping and scour protection.

RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT N – SLOPE STABILITY

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

By: Barr Engineering

FINAL –February 28, 2010

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT N: SLOPE STABILITY

F-N1.0 INTRODUCTION

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 consistency and exhibit high plasticity, which leads to low shear strengths. The shear strengths can be especially low under drained conditions (long-term) because of the mineralogical composition of the material, and thus 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 US Army Corps of Engineers (USACE). The structures for which these additional analyses were performed were:

- 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 this exhibit provides methodology, input parameters, results, and recommendations.

F-N2.0 GEOLOGY

The Fargo-Moorhead area is covered by 200 to 300 feet of clays, tills, and granular sediments from past glaciation associated with the formation of glacial Lake Agassiz. The Sherack Formation, Brenna Formation, Argusville Formation, and glacial till are the primary soil units in the area, though other more localized soil units have been identified such as the Poplar River Formation, Plastic Laminated Sherack Formation, and Oxidized Brenna Formation. The material types of most interest for geotechnical engineers in the Red River Valley are the Brenna and Argusville Formations, which exhibit high plasticity . The clay was deposited after the most recent glacial event, and it is characterized as a lacustrine clay but it was altered at some locations during lower lake levels where erosion channels were filled with sediments. Additionally, exposure to the atmosphere may have caused overconsolidation of the formation. The Sherack Formation was deposited during

higher lake levels and may contain clays and silts as well as organics. The Sherack Formation is continuously present except in areas where relatively recent alluvial or fluvial processes have eroded it away.

F-N3.0 SLOPE STABILITY MODELING

A slope stability analysis was carried out to ensure that slopes near the hydraulic structures and the tie-back levees associated with Storage Area 1 exhibit adequate factors of safety.

F-N3.1 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 instability 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.

Slope stability factors of safety are computed as the summation of forces (or moments) that resist movement divided by the summation of forces (or moments) that cause movement. Conditions that may decrease stability in a slope include increasing water levels or pore-water pressure, loading on the crest of a slope, or removal of material or water from the slope toe.

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 porewater pressures do not have time to dissipate during shearing. This scenario typically applies to rapid loading from, for example, embankment construction where the loading takes place quickly relative to the permeability of the soils. It is often referred to as the "end-of-construction" case.

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

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 using 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 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 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 for the transient analyses.

F-N3.2 SOFTWARE

The slope stability analysis was conducted using GeoStudio 2007, a computer-modeling program developed by GEO-SLOPE International, Ltd. Within GeoStudio 2007, two modules were used: SEEP/W and SLOPE/W.

SEEP/W uses the finite-element analysis technique to model groundwater flow and porewater pressure distribution within porous materials such as soil. This method was chosen because its comprehensive formulation makes possible to analyze both simple and highly complex seepage problems. It can model saturated and unsaturated flow, steady-state and transient conditions, and a variety of boundary conditions. SEEP/W generates an output file containing the heads at the nodes of the finite element mesh. Integration of the modules within the software package allows the use of a SEEP/W head file in the slope stability program to compute effective stresses. In this manner, the impact of seepage on stability can be evaluated.

The slope stability analysis was conducted using SLOPE/W. This model uses limit equilibrium theory to compute a factor of safety for earth and rock slopes. It is capable of using a variety of methods to compute the factor of safety of a slope while analyzing complex geometry, stratigraphy, and loading conditions. SLOPE/W allows the user to import the groundwater head file from the seepage analysis to compute effective stresses.

Spencer's method was used as the search technique to determine the factor of safety of the embankment in this stability analysis. This method is considered the most adequate because it satisfies all conditions of static equilibrium and provides a factor of safety based on both force and moment equilibrium.

F-N3.3 PARAMETERS

The input parameters for modeling were developed by St. Paul District of the USACE. They are summarized below.

F-N3.3.1 Seepage Analysis: Mesh and Boundary Conditions

The mesh for the seepage analysis consisted of triangular and quadrilateral, unstructured finite elements. The mesh was made fine enough to capture the effects of any areas of high hydraulic gradient and to ensure that the boundary conditions were applied effectively to the model.

Boundary conditions applied in the seepage model include:

- Total heads applied along the upstream model boundaries for the approach channel and hydraulic structure modeling. The total heads were fixed at various values as discussed later in this exhibit. Model boundaries were placed far from the area of interest (250 feet from channel centerline for tributary approach channels, 500 feet from the channel centerline for the Red River Control Structure approach channel, and 250 feet from the radial wall centerline for hydraulic structures) such that boundary effects could be minimized.

- Potential seepage face review nodes placed along the slopes. During model cycling, these nodes check for any pressure heads or total flux values above zero and, if found, resets them to zero iteratively. This makes it possible for a realistic phreatic surface to develop.

- Default no-flow boundary conditions on the bottom of the model.

- Default no-flow boundary conditions with infinite elements were placed on the left and right boundaries for the Storage Area 1 modeling.

The steady-state analysis included the following assumptions:

- Storage Area 1: Two water levels on the interior of the model were assumed: (a) at the ground surface of 908 and (2) at Elevation 922 to simulate flood conditions.

- Tributary Approach Channels at Maple, Sheyenne, and Wild Rice Rivers: There is always water in the approach channel corresponding to the normal water elevation. This elevation corresponds to the mean annual flow. The phreatic surface was assumed to be hydrostatic.

- Red River Control Structure Approach Channel: There is always water in the approach channel corresponding to the normal water elevation. This elevation corresponds to the mean annual flow. The phreatic surface was assumed to slope toward the channel because this channel is significantly deeper than the tributary approach channels. The total head boundary condition was fixed at Elevation 893.

- Maple River and Sheyenne Hydraulic Structures: There is no water in the diversion channel. This represents worst-case conditions because having the weight of the water acting against the slope only increases the factor of safety.

- There is no infiltration on the ground surface.

F-N3.3.2 Seepage Analysis: Material Properties

The USACE performed an extensive seepage analysis for the main channel slopes. The parameters used in modeling presented herein were obtained from testing and interpretation performed by the USACE.

Three additional material types were used in seepage modeling in addition to the material types used by the USACE. They were Levee Fill, Granular Backfill, and Concrete.

The seepage properties used in modeling are shown in Table F-N1.

Table F-N1 Seepage Parameters Used in Modeling

Material	Soil Class	Saturated Hydraulic Conductivity, k		Porosity, n	Coefficient of Volume Compressibility, m_v	Residual Volumetric Water Content, $(\theta_{vw})_r$
		cm/sec	ft/day	-	1/psf	-
Sherack	CH	1.0E-06	2.8E-03	0.50	9.0E-06	0.050
Oxidized Brenna	CH	5.0E-07	1.4E-03	0.55	1.0E-05	0.055
Brenna	CH	1.0E-07	2.8E-04	0.63	3.0E-05	0.063
Argusville	CH	1.0E-07	2.8E-04	0.60	3.0E-05	0.060
Till	CL	5.0E-06	1.4E-02	0.45	3.0E-05	0.045
Levee Fill	CH	2.0E-07(1)	5.7E-04(1)	0.50	9.0E-06	0.050
Granular Backfill	SP	1.0E-02	2.8E+01	0.40	3.0E-05	0.040

(1)- Assumed to be half order of magnitude lower than Sherack permeability

The Concrete was assumed to have very low permeability, and the same permeability characteristics as the Sherack Formation were used for this material.

F-N3.3.3 Stability Analysis: Material Properties

As for the stability analysis, the USACE also performed an extensive slope stability analysis for the main channel slopes. The material properties determined by the USACE for their analysis were used in the stability modeling described herein. The strength parameters used by the USACE were defined using the large strain failure criterion which is appropriate in the Red River Valley clays to simulate the softening phenomenon.

The unit weight and strength properties used in the models are shown in Table F-N2.

Table F-N2 Unit Weight and Shear Strength Parameters Used in Modeling

Material	Soil Class	Saturated Unit Weight	ESSA (Drained)		USSA (Undrained)	
			Cohesion	Friction Angle	Cohesion	Friction Angle
		pcf	psf	deg	psf	deg
Sherack	CH	117	0	28	1400	0
Oxidized Brenna [1]	CH	111	0	19	1000	0
Brenna [1]	CH	103	0	13	650	0
Argusville [1]	CH	107	0	15	825	0
Till [2]	CL	122	0	34	1900	0
Levee Fill [3]	CH	117	200	28	1400	0
Granular Backfill	SP	125	0	30	0	30

(1)- Non-linear failure envelope used for these materials (drained)
(2)- Assumed to be impenetrable for wedge (composite) failure
(3)- Assumed to consist of compacted Sherack Formation; a nominal cohesion was added to account for compaction (drained)

As noted, the glacial till material was selected as an impenetrable material so that wedge failures could be modeled accurately. When weaker soils overlie much stronger soils, such as the Argusville Formation overlying the glacial till, there is a tendency for the lower portion of the potential failure surface to be truncated and much of the failure surface is coincident with the contact between the weak and strong soils. Modeling the stronger soil as impenetrable allows the model to find these potential failure surfaces, which in many instances are the controlling cases. Impenetrable materials do not require unit weight input values.

F-N4.0 STORAGE AREA 1 LEVEES

A slope stability analysis was carried out to demonstrate that proposed levees associated with Storage Area 1 are stable.

F-N4.1 GEOMETRY

The levee geometry was assumed to have a 15-foot crest width, a crest elevation of 927, and 4H:1V slopes. The ground surface was assumed 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. Figure F-N1 shows typical levee geometry.

Boring logs in the vicinity of the proposed Storage Area 1 indicate the general stratigraphy found elsewhere along the diversion channel alignment, but some variability

existed. To account for this variability in the modeling and its impact on levee stability, four stratigraphic cases were analyzed. Table F-N3 shows the four stratigraphy types and the elevations for the top contact of each material.

Table F-N3 Stratigraphy Types for Storage Area 1 Levees

	Top of Unit Elevation (feet)			
Material	Type 1	Type 2	Type 3	Type 4
Sherack/Alluvium	908	908	908	908
Oxidized Brenna	None	900	900	None
Brenna	900	895	None	900
Argusville	880	880	880	890
Till	850	850	850	850

F-N4.2 RESULTS

Using the stratigraphy types shown in Table F-N3, 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 and USSA factor of safety results are shown in Table F-N4.

Table F-N4 Factor of Safety Results for Storage Area 1 Levees (Steady-State)

	Factor of Safety			
Analysis Type	Type 1	Type 2	Type 3	Type 4
ESSA	1.29	1.39	1.67	1.33
USSA	2.21	2.28	2.37	2.26

Because the Type 1 and Type 4 stratigraphy resulted in inadequate factors of safety, a transient seepage analysis was performed. This procedure (established in a meeting with the USACE on December 13th, 2010) involved instantly placing a total head boundary condition at 922 on the upstream face of the levee and then determining the permeability values required for the flow to reach close to steady-state at an arbitrary time of 30 days. Steady-state seepage was defined by tracking pore pressures at randomly placed nodes throughout the levee. All permeability values were adjusted up and down together using the same multiplication factor such that the relative permeabilities remained unchanged. These permeability values were then used with the 100-year hydrograph and a full transient analysis resulting in computed factors of safety with time. The minimum factor of safety was then reported for the upstream and downstream levee faces (ESSA and USSA). The Type 1 and Type 4 stratigraphy factors of safety are shown in Table F-N5. Type 2 and Type 3 were not analyzed because their steady-state factors of safety were considered adequate.

Table F-N5 Factor of Safety Results for Storage Area 1 Levees (Transient)

Analysis Type	Factor of Safety	
	Type 1	Type 4
ESSA Downstream	1.45	1.43
ESSA Upstream	1.53	1.59
USSA Downstream	2.24	2.31
USSA Upstream	2.37	2.44

It can be seen in Table F-N5 that the Type 1 and Type 4 factors of safety are adequate because they all exceed 1.40, which is the most stringent factor of safety requirement for levees. All factor of safety and seepage model outputs are attached.

F-N5.0 APPROACH CHANNELS

A slope stability analysis was performed on several proposed approach channels 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- and 500-year flood conditions and using the Phase 4 hydrographs that provided water levels for flood conditions over a period of 36 days. These analyses did not represent the governing case for slope stability, so the results are not presented in this exhibit.

F-N5.1 MAPLE RIVER

The Maple River stratigraphy from the ground surface consists of Sherack, Oxidized Brenna, Brenna, Argusville, and glacial till. The normal river elevation is at 881.5 feet above sea level. For steady-state conditions at a slope of 3H:1V, the left and right bank were found to have a factor of safety of 1.25 for the ESSA. This value does not meet USACE requirements of a factor of safety greater than or equal to 1.40. Figure F-N2 shows the initial model created with slopes of 3H:1V. Because the factor of safety was not met, two alternatives were developed in order to obtain a factor of safety of at least 1.40.

The first alternative was designed to decrease the slope steepness by increasing the horizontal to vertical displacement. Slopes were increased until a steady state factor of safety of 1.40 or greater was reached. The Maple River slopes were found to have a stable factor of safety when slopes were at 4.25H:1V.

The second alternative is to install steel piles into the 3H:1V river bank slopes. Two options were evaluated within this alternative. The first option was to use two rows of

steel pipe piles with smaller diameter and lower shear capacity spaced every 12 feet on both sides of the river bank. The second option was to use one row of piles on each side of the river bank having a larger diameter and higher shear capacity. Both alternatives were evaluated and the results are discussed below. A cost comparison for each option was created in order to determine the most economical solution.

F-N5.1.1 Alternative 1: Change in Slope

The first alternative flattens the slope while maintaining a river channel bottom width of 20 feet and a depth of 19.3 feet. The required factor of safety was met when the channel slopes were at 4.25H:1V. Because the other tributary approach channels required flatter slopes and for consistency, a slope of 5H:1V was be used for all tributary approach channels. Figure F-N3 shows the Maple River steady-state analysis geometry with river bank slopes of 5H:1V. The required excavation to create a 3H:1V channel (including the channel and both banks) is 70.0 CY per linear foot of channel. At an estimated $3.49 per CY, the total excavation cost would be around $244.35 per linear foot of channel. The required excavation to create a 5H:1V channel (including channel and both banks) is 97.6 CY per linear foot of channel. At an estimated $3.49 per CY, the total excavation cost would be around $340.78 per foot along the alignment. A summary of all the cost estimates is included in this Exhibit.

F-N5.1.2 Alternative 2-Option 1: Two Rows of Piles

The option consists of placing steel piles in the river bank slope of 3H:1V. This option consists of two rows of 24-inch hollow steel pipe piles with a wall thickness of ½ inch spaced at 12 feet along the length of the channel. The estimated channel length is about 3,000 feet. The piles are 57 feet long and 61 feet long, allowing them to extend 5 feet into the glacial till. Figure F-N4 shows the river cross-section with a single row of piles in place. The cost of excavation for 3H:1V slopes would be 70 CY per foot along the alignment at $3.49 per CY for a total of $244.35 per foot along the alignment. The piles would result in 19.7 VLF (vertical linear feet) per foot along the alignment at $117.77 with a total pile cost of $2,316.18 per foot along the alignment. The total cost of this alternative is $2,560.54 per foot along the alignment.

F-N5.1.3 Alternative 2-Option 2: One Row of Piles

The option consists of one row of piles on both sides of the river bank spaced every 12 feet along the channel length. Figure F-N5 shows the river cross-section with a single row of piles in place. The steel pipe design consists of 30-inch hollow steel pipe piles with a wall thickness of ½ inch. The piles are 59 feet long allowing it to extend 5 feet into the glacial till. The cost of excavation for 3H:1V slopes would be 70 CY per foot along the alignment at $3.49 per CY for a total of $244.35 per foot along the alignment. The piles would result in 9.8 VLF per foot along the alignment at $157.80 with a total pile cost of $1,551.73 per foot along the alignment. The total cost of this alternative is $1796.08 per foot along the alignment.

F-N5.1.4 Cost Comparison

A cost comparison for the three slope stability options confirms that the most economical alternative would be to excavate the tributary river channel slopes to 5H:1V. Once all the river tributaries were modeled, the minimum slope of 5H:1V was required for the Red River and Wild Rice River.

F-N5.2 SHEYENNE RIVER

The Sheyenne River stratigraphy from the ground surface consists of Sherack, Oxidized Brenna, Brenna, Argusville, and glacial till. The normal river elevation is at 903.24 feet above sea level. The initial model consisted of 3H:1V slopes. The left and right bank had a factor of safety of 1.16, not meeting the USACE requirement of 1.40. Figure F-N6 shows a river cross-section of the initial model. The slopes were then flattened while maintaining a river channel bottom width of 20 feet and a depth of 21.9 feet. The required factor of safety was met when the channel slopes were 5H:1V. Figure F-N7 shows the new cross-section for a river channel with slopes 5H:1V.

F-N5.3 WILD RICE RIVER

The Wild Rice River stratigraphy from the ground surface consists of Sherack, Oxidized Brenna, Argusville, and glacial till. The normal river elevation is at 894.08 feet above sea level. The initial model evaluated at a slope of 3H:1V resulted in a right and left bank factor of safety of 1.27. This does not meet the USACE requirement of 1.40. Figure F-N8 shows the initial model river cross-section. The river bank slopes on the Wild Rice River were then flattened while maintaining a river channel bottom width of 40 feet and depth of 21.7 feet. The required factor of safety was met when the channel slopes were at 3.5H:1V. Figure F-N9 shows the new cross-section for a river channel with slopes 5H:1V.

F-N5.4 RED RIVER CONTROL STRUCTURE

The Red River stratigraphy from the ground surface consists of Sherack, Oxidized Brenna, Argusville Formation, and glacial till. The normal river elevation is at 882.96 feet above sea level. The initial model evaluated at a slope of 3H:1V resulted in a right and left bank factor of safety of 0.99. This does not meet stability nor the minimum USACE required factor of safety of 1.40. Figure F-N10 shows the initial model river cross-section. The slopes were then flattened while maintaining a river channel bottom width of 50 feet and a depth of 38.4 feet. The required factor of safety was met when the channel slopes were 7H:1V. Figure F-N11 shows the new cross-section for a river channel with slopes 7H:1V.

F-N6.0 HYDRAULIC STRUCTURES AT MAPLE AND SHEYENNE 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.

F-N6.1 GEOMETRY

Radial walls are required at these structures to minimize head loss as the flow in the diversion channel passes underneath the aqueducts at their respective river crossings. After some discussion regarding the geometry of these walls and based on the input from the USACE for the Red River Control Structure, it was decided that the walls should extend all the way to the top of the aqueduct channel. This created a very high wall, and the global stability of this wall needed to be checked.

The structural analysis of the wall resulted in a 4-foot-thick reinforced concrete stem and footing, with the toe and heel extending 12 feet from the stem. This resulted in an overall footing width of 28 feet. Five rows of piles are spaced at 6 feet nominally parallel to the wall (longitudinally) and 6.25 feet perpendicular to the wall (transversely). The H-piles were sized as HP 14x73 piles for structural purposes with HP 14x89 piles used in the cost estimate to allow for corrosion. Granular backfill was assumed to be placed behind the wall and above a 1H:1V cut slope extending upward from the heel of the footing. Drainage was provided via weep holes in the wall.

At the Maple River, the stem extends from Elevation 872.06 (channel invert) upward to Elevation 903.5 (top of wall). At the Sheyenne River, the stem extends from Elevation 883.68 (channel invert) upward to Elevation 917.5 (top of wall). See Figures F-N12 and F-N13 for representations of the geometry used in modeling for the Maple and Sheyenne hydraulic structures, respectively.

F-N6.2 PILES

The piles described above were accounted for in the limit equilibrium slope stability model using the parameters shown in Table F-N6 below.

Table F-N6 Pile Properties

Parameter	Maple	Sheyenne
Pile Length [ft]	40.6*	37.0*
Pile Spacing (longitudinal) [ft]	6.0	6.0
Pile Spacing (lateral) [ft]	6.25	6.25
Shear Capacity in Yield, Unfactored [kips]	106	118
Factor of Safety	1.4	1.4
Shear Capacity in Yield, Allowable [kips]	74.3	90.0

*- Piles extend about 7.5 feet into glacial till

The shear force for the piles was applied parallel to the slip surface rather than perpendicular to the reinforcement. The unfactored shear capacity in yield was changed in the model until an adequate stability factor of safety was achieved. A factor of safety of 1.40 was used for the pile shear capacity because it represents the highest required factor of safety for global slope/wall stability and it can be considered conservative for any other stability cases in which lower factors of safety are required.

F-N6.3 RESULTS

Total heads for the steady-state seepage model for the Maple River hydraulic structure are shown in this Exhibit. The plot shows that the phreatic surface gradually decreases from its far-field value (250 feet back from wall; total head of 881.50; normal flow for the Maple River) along the left to the granular backfill where it slopes steeply toward the weep hole at the bottom of the wall. Total heads for the steady-state seepage model for the Sheyenne River hydraulic structure are also shown in this Exhibit. The plot shows that the phreatic surface gradually decreases from its far-field value (250 feet back from wall; total head of 903.24; normal flow for the Sheyenne River) along the left to the granular backfill where it slopes steeply toward the weep hole at the bottom of the wall.

For the Maple River hydraulic structure, stability modeling results are shown in this Exhibit for the cases without and with piles, respectively. It should be noted that the entry and exit ranges and the strength of the concrete was established such that the method found a potential failure surface corresponding to global stability. Output reports are attached for the modeling runs.

The ESSA factors of safety are shown in Table F-N7 below. USSA factors of safety were much higher and are not presented here.

Table F-N7 Hydraulic Structure Factors of Safety

Scenario	Modeled Factor of Safety		Required Factor of Safety
	Maple	Sheyenne	
No Piles	0.50	0.55	1.40
With Piles	1.41	1.40	1.40

As discussed previously, the unfactored shear capacities presented in Table F-N6 were obtained by changing the capacities until the required global factor of safety was achieved. The actual shear capacity of the piles is about is 206 kips based on 50 ksi steel using only the area of the web for shear capacity. The web area for HP 14x73 piles is 6.87 in^2.

It can be seen that piles are required to ensure adequate global stability. It should be noted, however, that using the shear capacity for the global stability analysis takes only the structural capacity of the pile into account and not the soil-structure interaction. In other words, the effect of the piles being forced into the soil within the sliding mass is not considered.

Using a simplified limit equilibrium approach is considered adequate for this feasibility study. In final design, more sophisticated analysis methods will be warranted and the soil-structure interaction will be addressed. If this analysis indicates that the factor of safety against global instability is inadequate, additional piles and/or tie-backs may be required.

F-N7.0 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. This approach is considered somewhat conservative, and improvement techniques could be the less expensive option for final design.

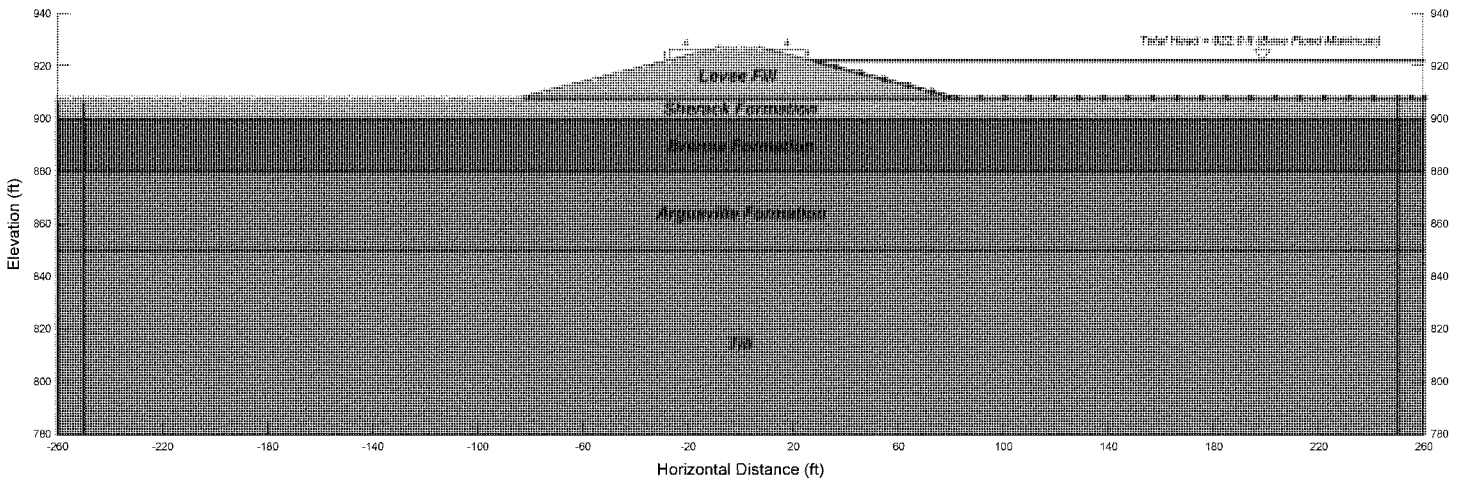
Possible improvement techniques to consider in final design are:

- Piles
- Deep soil mixing
- Lightweight fill

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

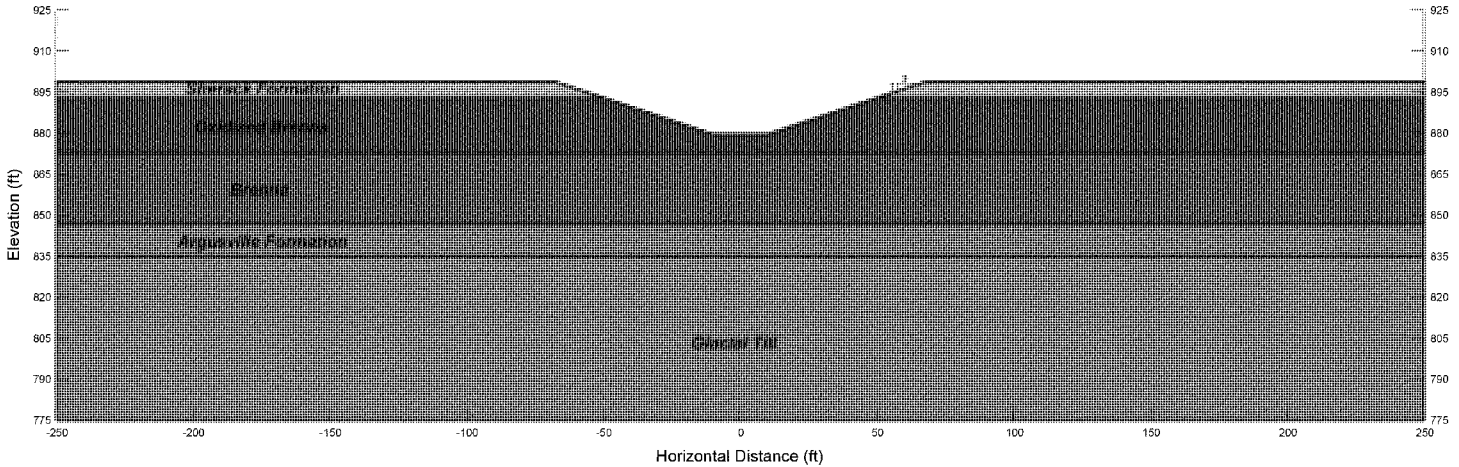
Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_1_FIGURE_F-N1.gsz
Last Saved Date: 2/24/2011

Figure F-N1. Typical Storage Area 1 Levee Geometry



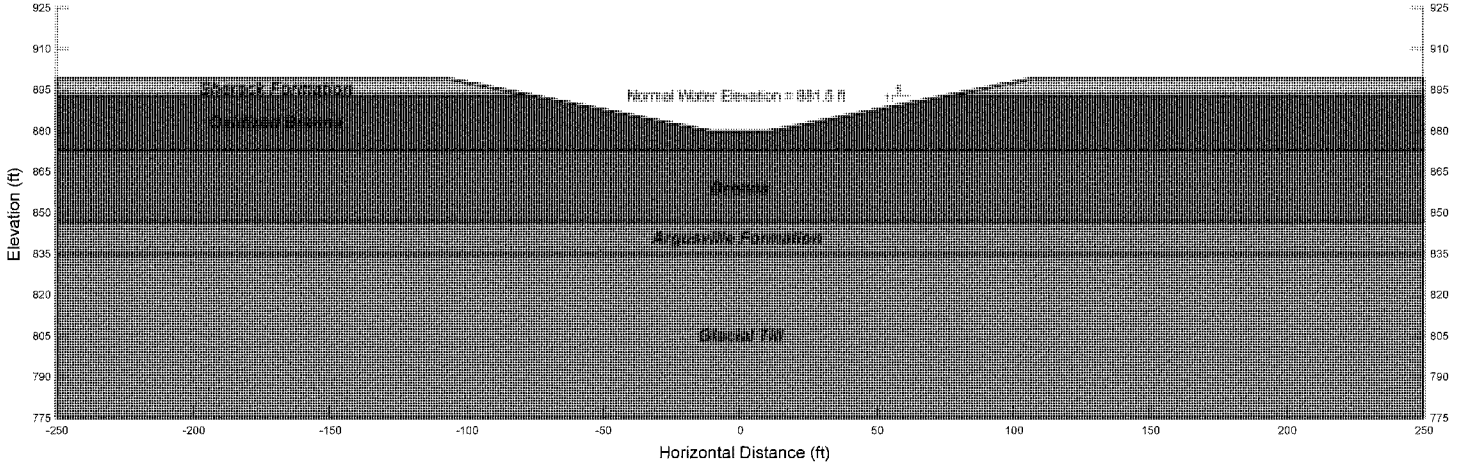
Fargo-Moorhead, Phase 4, Maple River Upstream Channel Stability Analysis
File Name: Maple River Channel_3to1slope_FIGURES.gsz

Figure F-N2. Maple River Approach Channel Geometry (3H:1V)



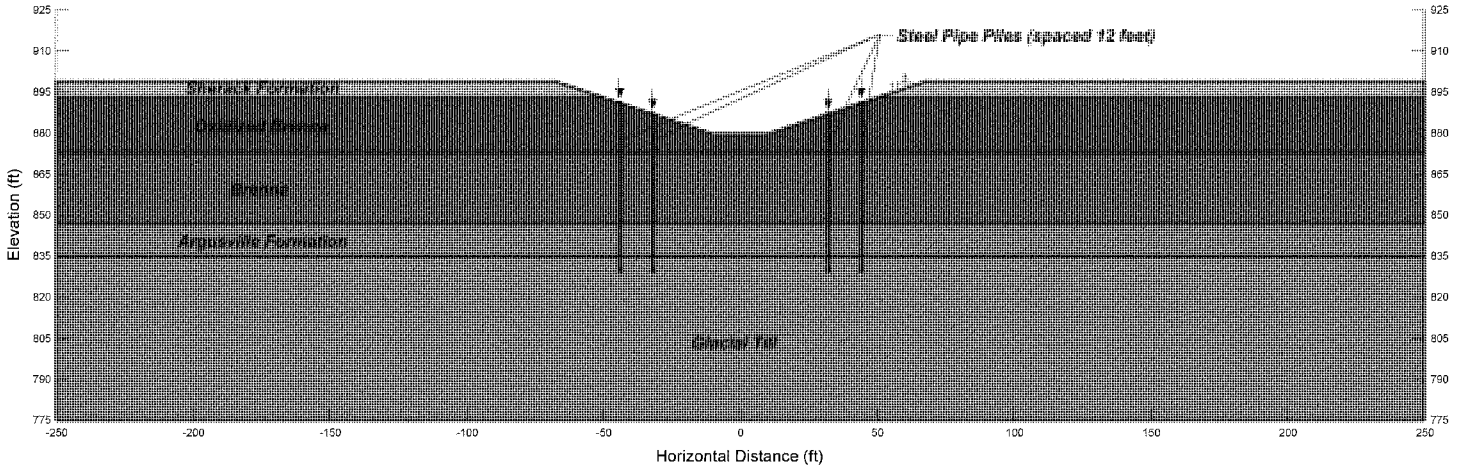
Fargo-Moorhead, Phase 4, Maple River Upstream Channel Stability Analysis
File Name: Maple River Channel_5to1slope_FIGURES.gsz

Figure F-N3. Maple River Approach Channel Geometry (5H:1V) Option 1: Flatten Slopes



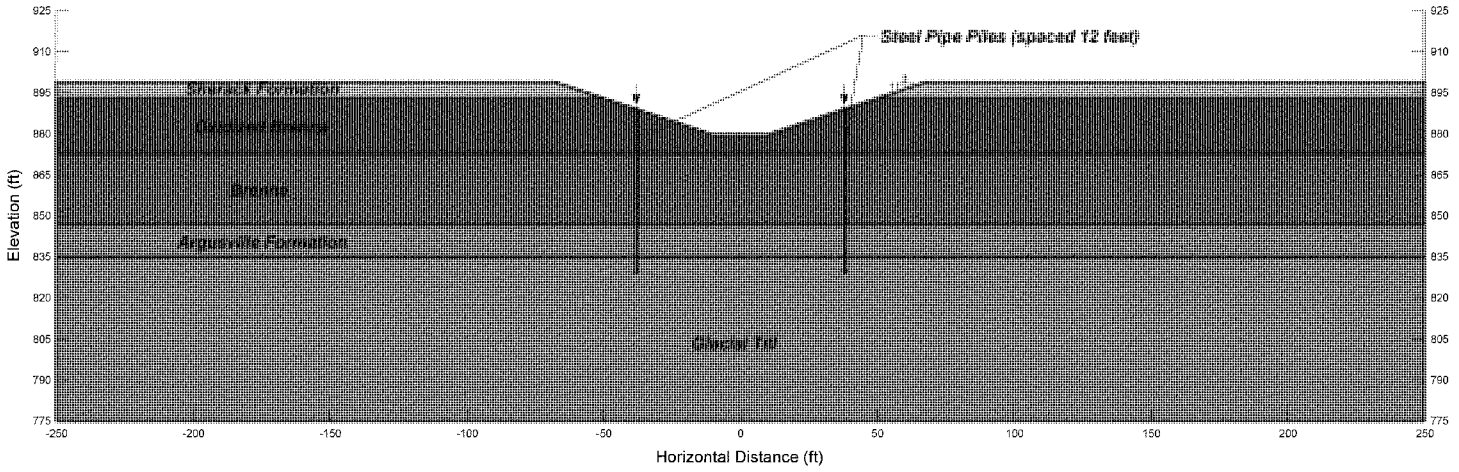
Fargo-Moorhead, Phase 4, Maple River Upstream Channel Stability Analysis
File Name: Maple River Channel_3to1slope_FIGURES.gsz

Figure F-N4. Maple River Approach Channel Geometry (3H:1V)
Option 2: Two Rows of Piles



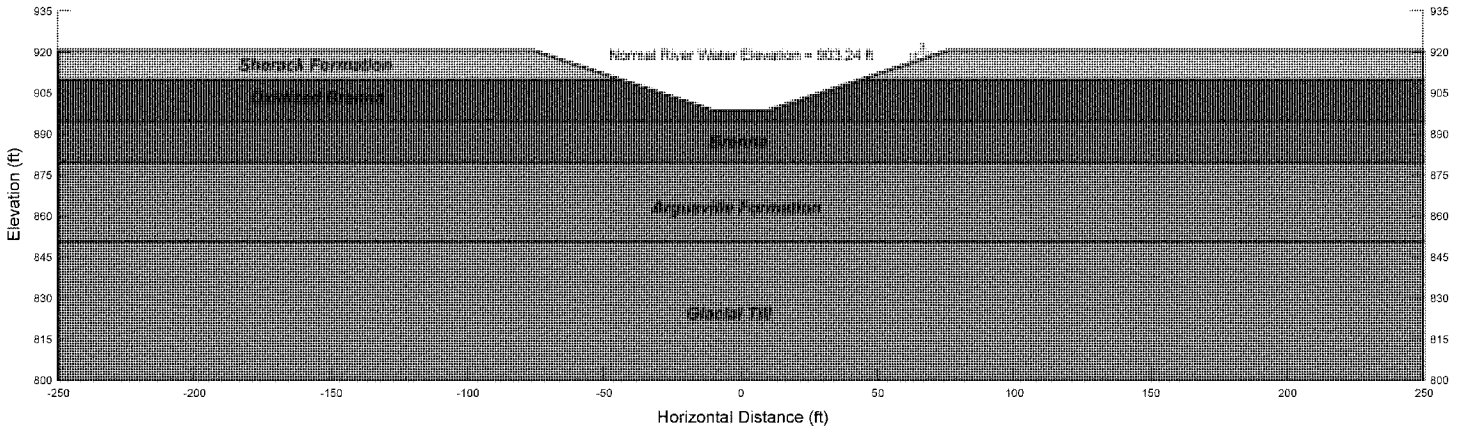
Fargo-Moorhead, Phase 4, Maple River Upstream Channel Stability Analysis
File Name: Maple River Channel_3to1slope_FIGURES.gsz

Figure F-N5. Maple River Approach Channel Geometry (3H:1V) Option 3: One Row of Piles



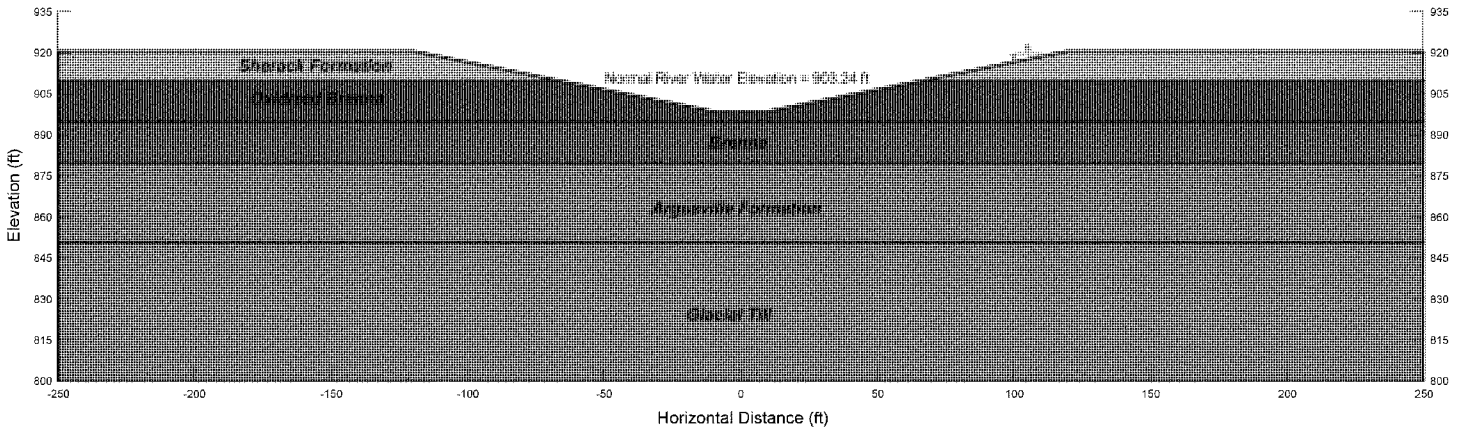
Fargo-Moorhead, Phase 4, Sheyenne River Upstream Channel Stability Analysis
File Name: Sheyenne River Channel_3to1_FIGURES.gsz

Figure F-N6. Sheyenne River Approach Channel Geometry (3H:1V)



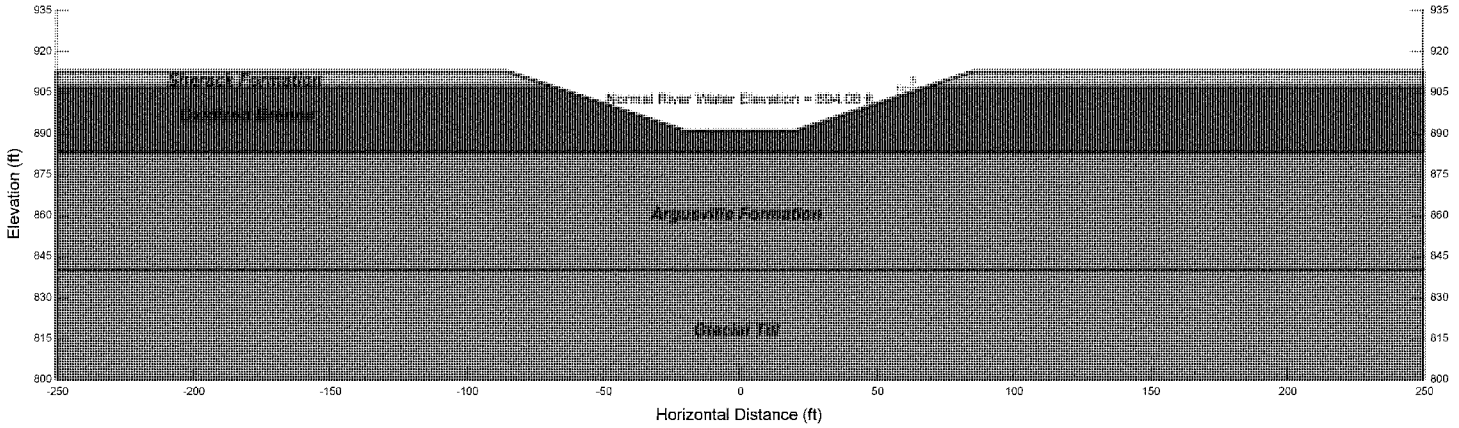
Fargo-Moorhead, Phase 4, Sheyenne River Upstream Channel Stability Analysis
File Name: Sheyenne River Channel_5to1_FIGURES.gsz

Figure F-N7. Sheyenne River Approach Channel Geometry (5H:1V)



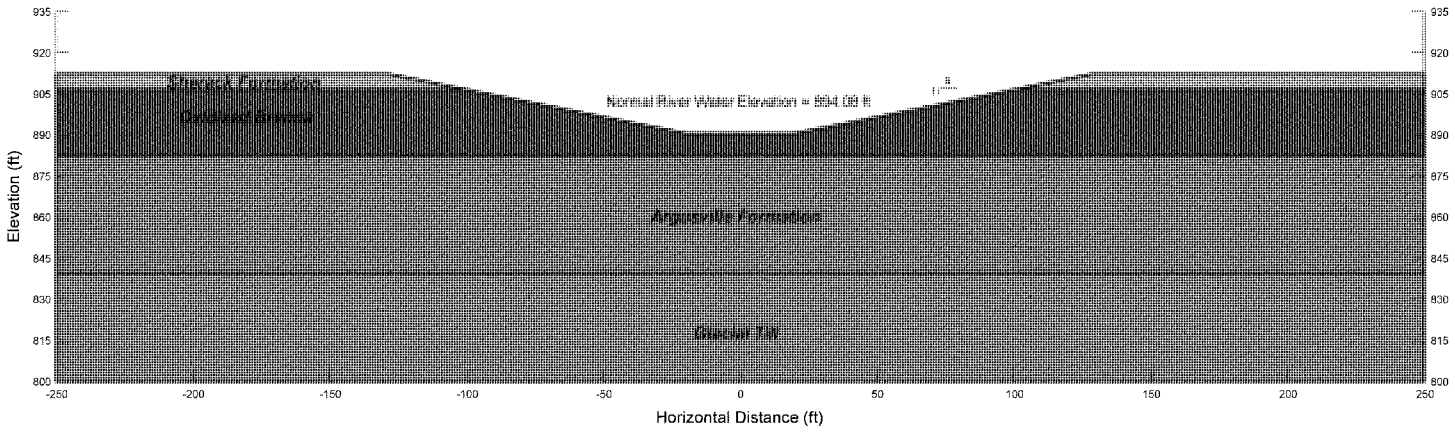
Fargo-Moorhead, Phase 4, Wild Rice River Upstream Channel Stability Analysis
File Name: Wild Rice River Channel_3to1_FIGURES.gsz

Figure F-N8. Wild Rice River Approach Channel Geometry (3H:1V)



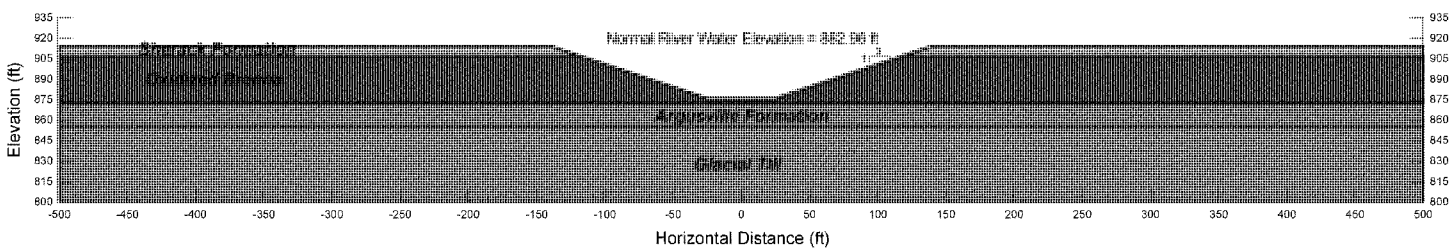
Fargo-Moorhead, Phase 4, Wild Rice River Upstream Channel Stability Analysis
File Name: Wild Rice River Channel_5to1_FIGURES.gsz

Figure F-N9. Wild Rice River Approach Channel Geometry (5H:1V)



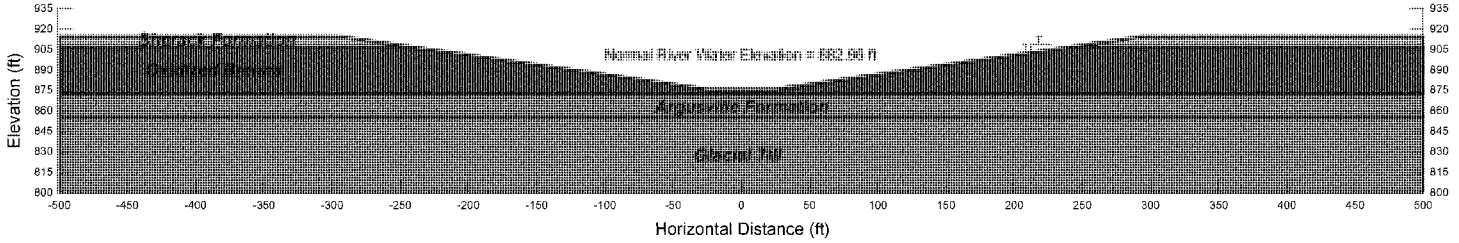
Fargo-Moorhead, Phase 4, Red River Upstream Channel Stability Analysis
File Name: Red River Channel_3to1_FIGURES.gsz

Figure F-N10. Red River Approach Channel Geometry (3H:1V)



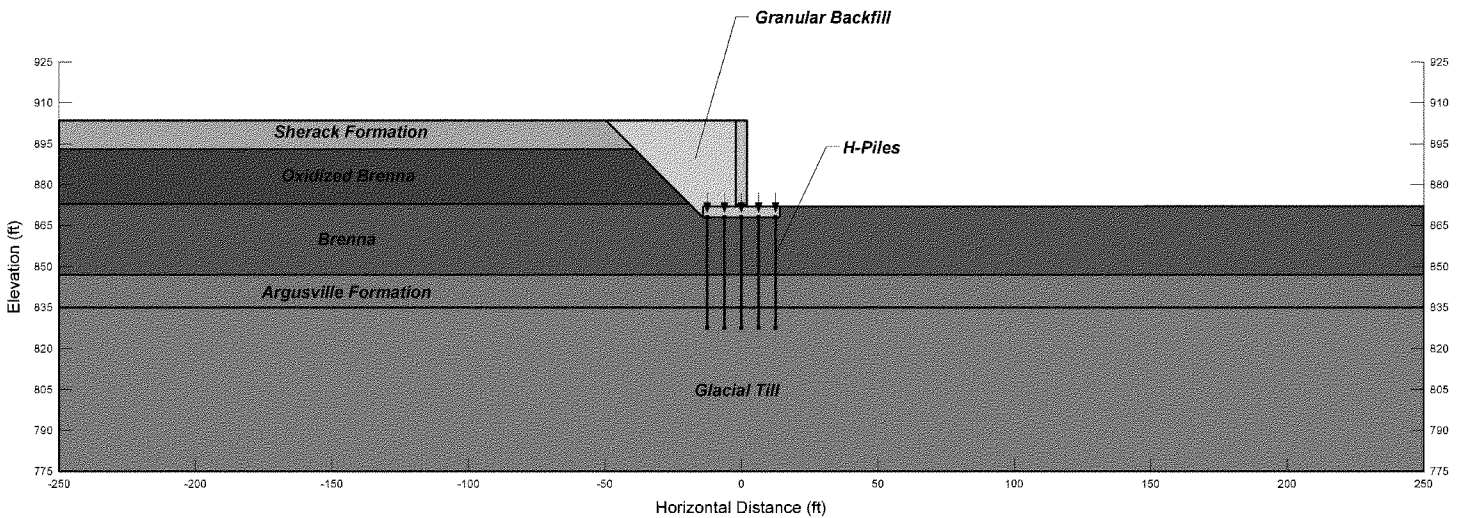
Fargo-Moorhead, Phase 4, Red River Upstream Channel Stability Analysis
File Name: Red River Channel_7to1_FIGURES.gsz

Figure F-N11. Red River Approach Channel Geometry (7H:1V)



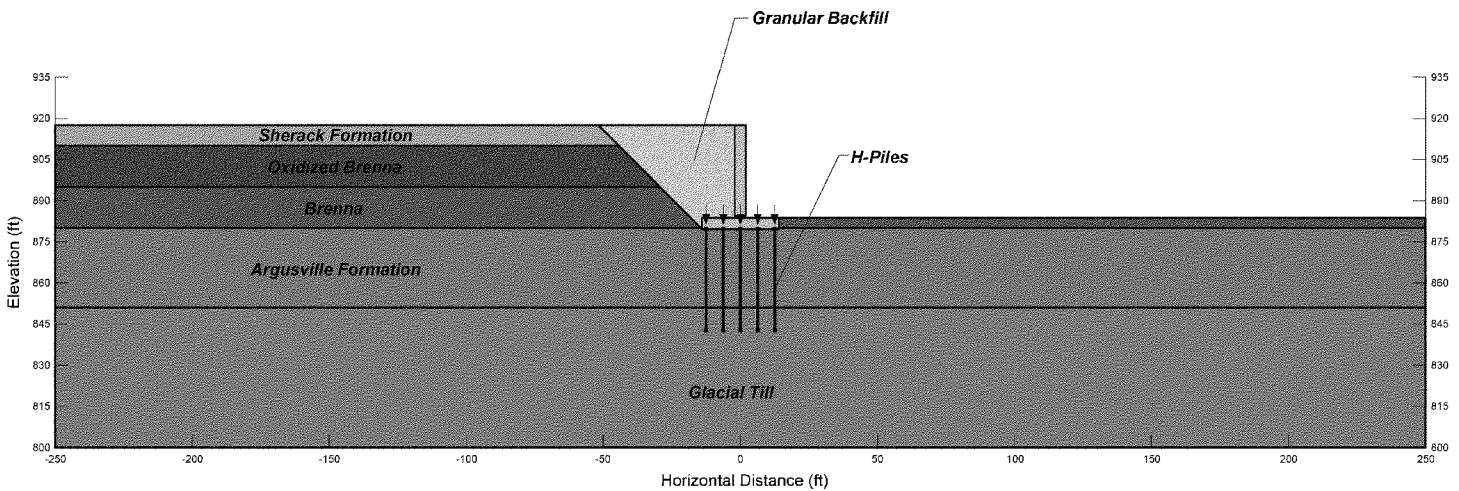
Fargo-Moorhead, Phase 4, Maple River Wall Stability Analysis
File Name: Maple River Wall_FIGURES.gsz

Figure F-N12. Maple River Hydraulic Structure Global Stability Geometry



Fargo-Moorhead, Phase 4, Sheyenne River Wall Stability Analysis
File Name: Sheyenne River Wall.gsz

Figure F-N13. Sheyenne River Hydraulic Structure Global Stability Geometry



Levee Storage Area 1

Type 1

2/2/2011

Levee Fill

8 ft Sherack

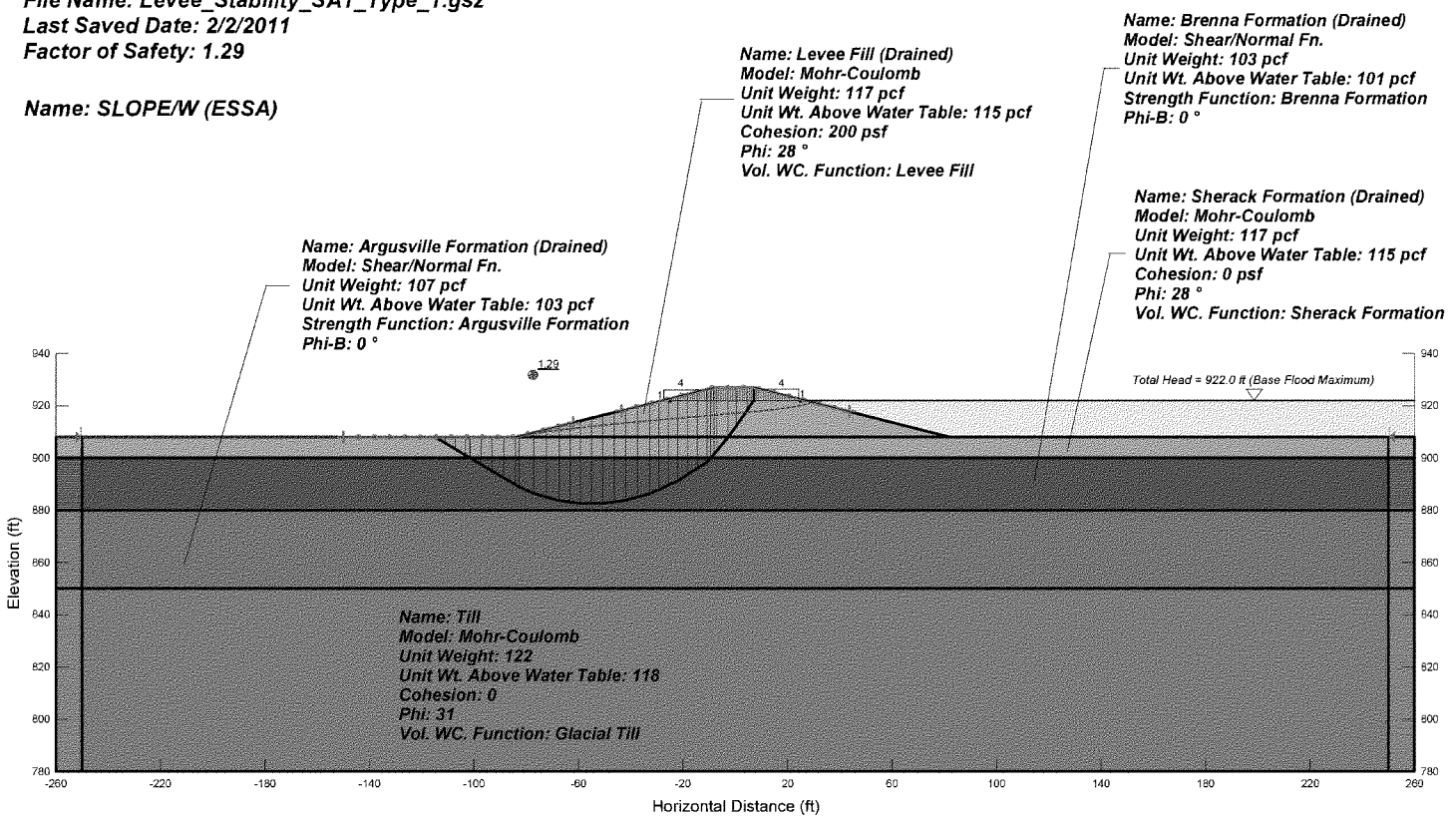
20 ft Brenna

30 ft Argusville

Till

Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_1.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 1.29

Name: SLOPE/W (ESSA)



Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
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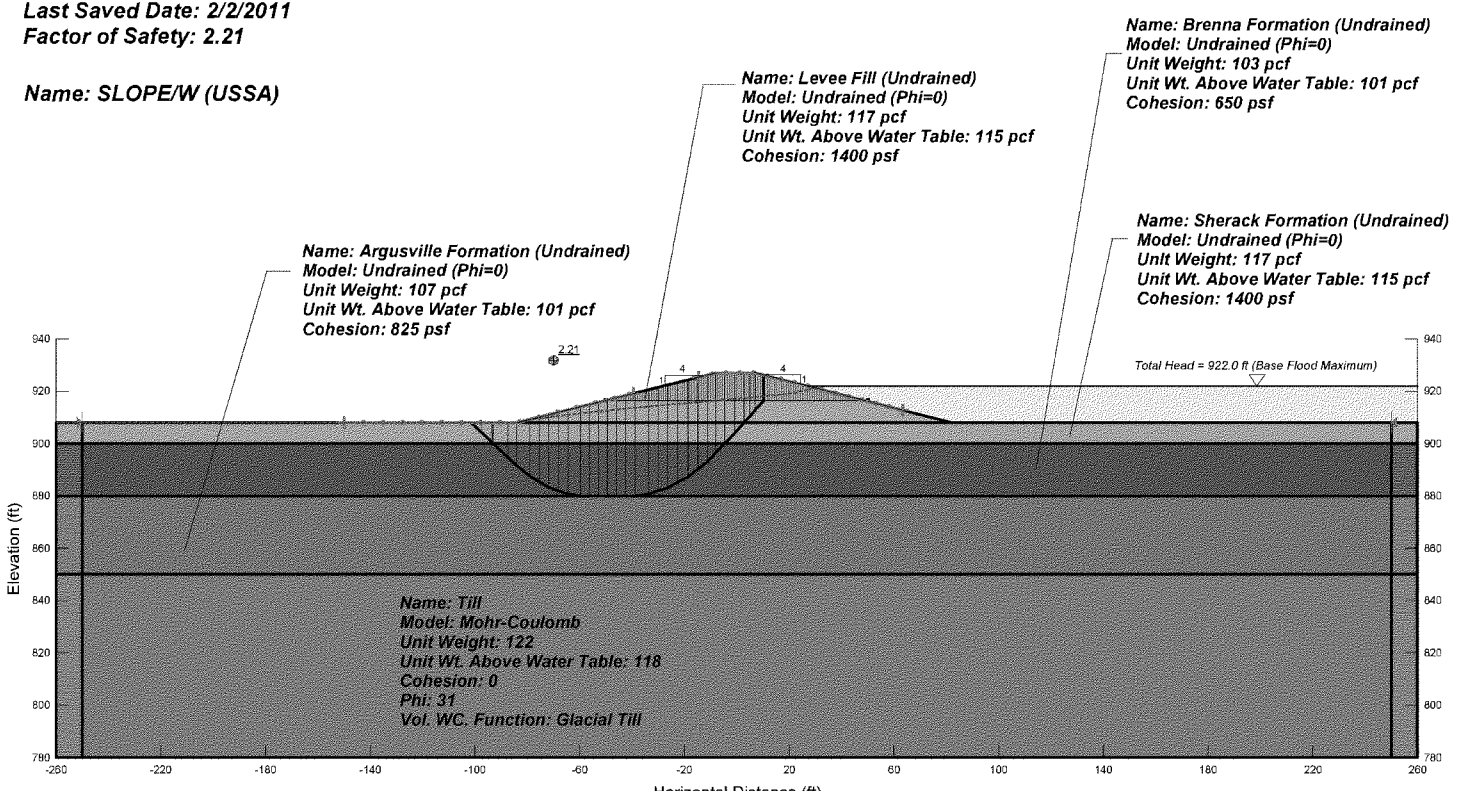
Factor of Safety: 2.21

Name: SLOPE/W (USSA)



Levee Storage Area 1

Type 2

2/2/2011

Levee Fill

8 ft Sherack

5 ft Oxidized Brenna

15 ft Brenna

30 ft Argusville

Till

Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
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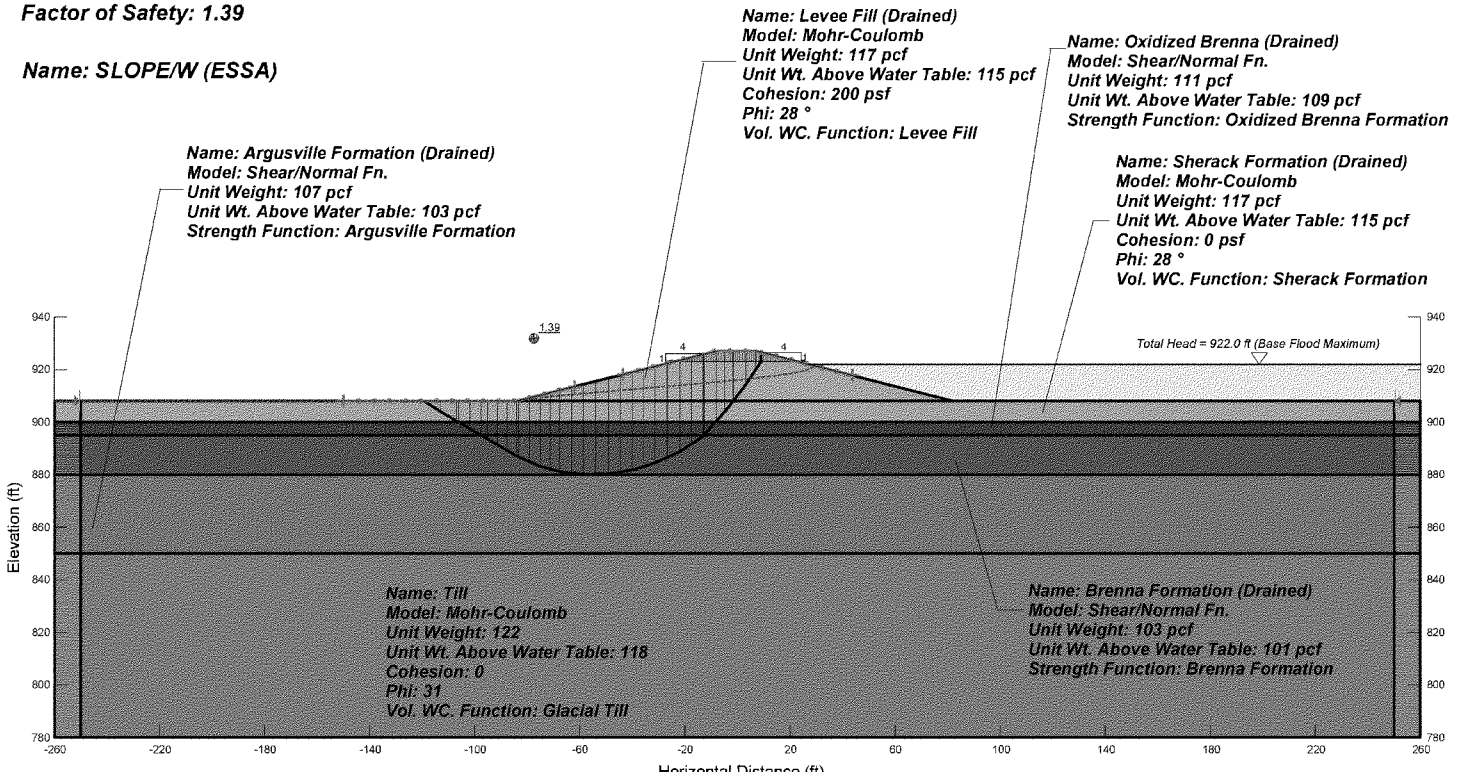
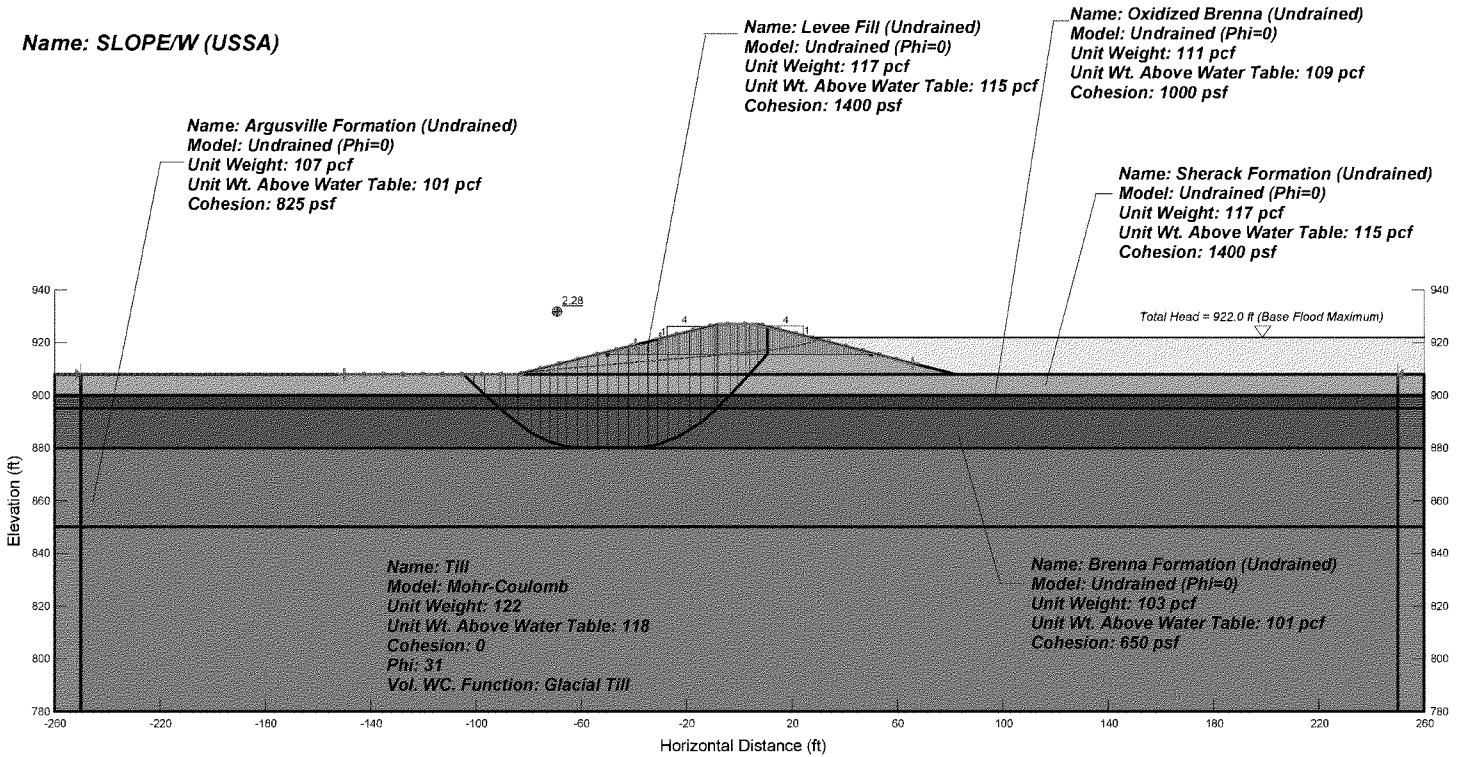
Last Saved Date: 2/2/2011
Factor of Safety: 1.39

Name: SLOPE/W (ESSA)



Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_2.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 2.28



Levee Storage Area 1

Type 3

2/2/2011

Levee Fill

8 ft Sherack

20 ft Oxidized Brenna

30 ft Argusville

Till

Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
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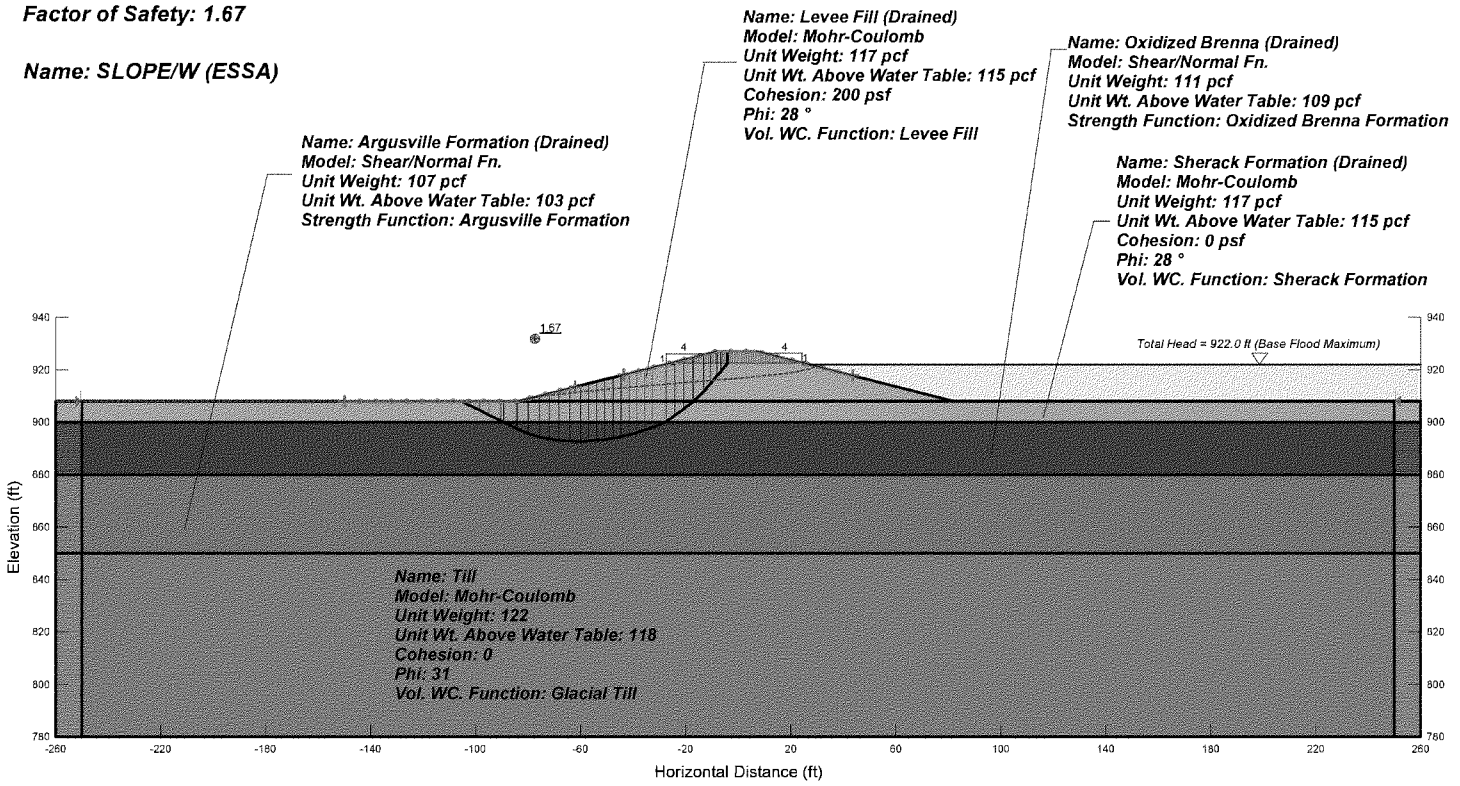
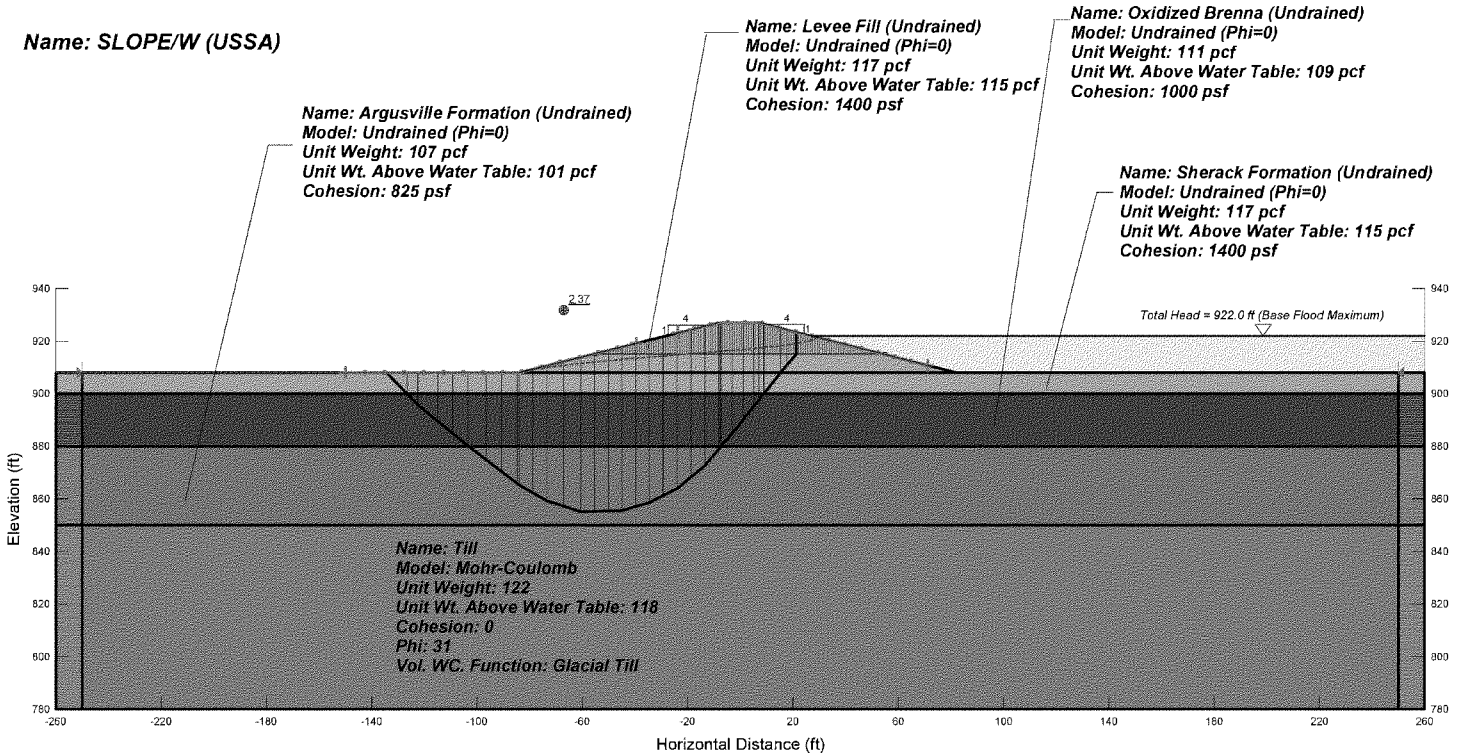
Last Saved Date: 2/2/2011
Factor of Safety: 1.67

Name: SLOPE/W (ESSA)



Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_3.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 2.37



Levee Storage Area 1

Type 4

2/2/2011

Levee Fill

8 ft Sherack

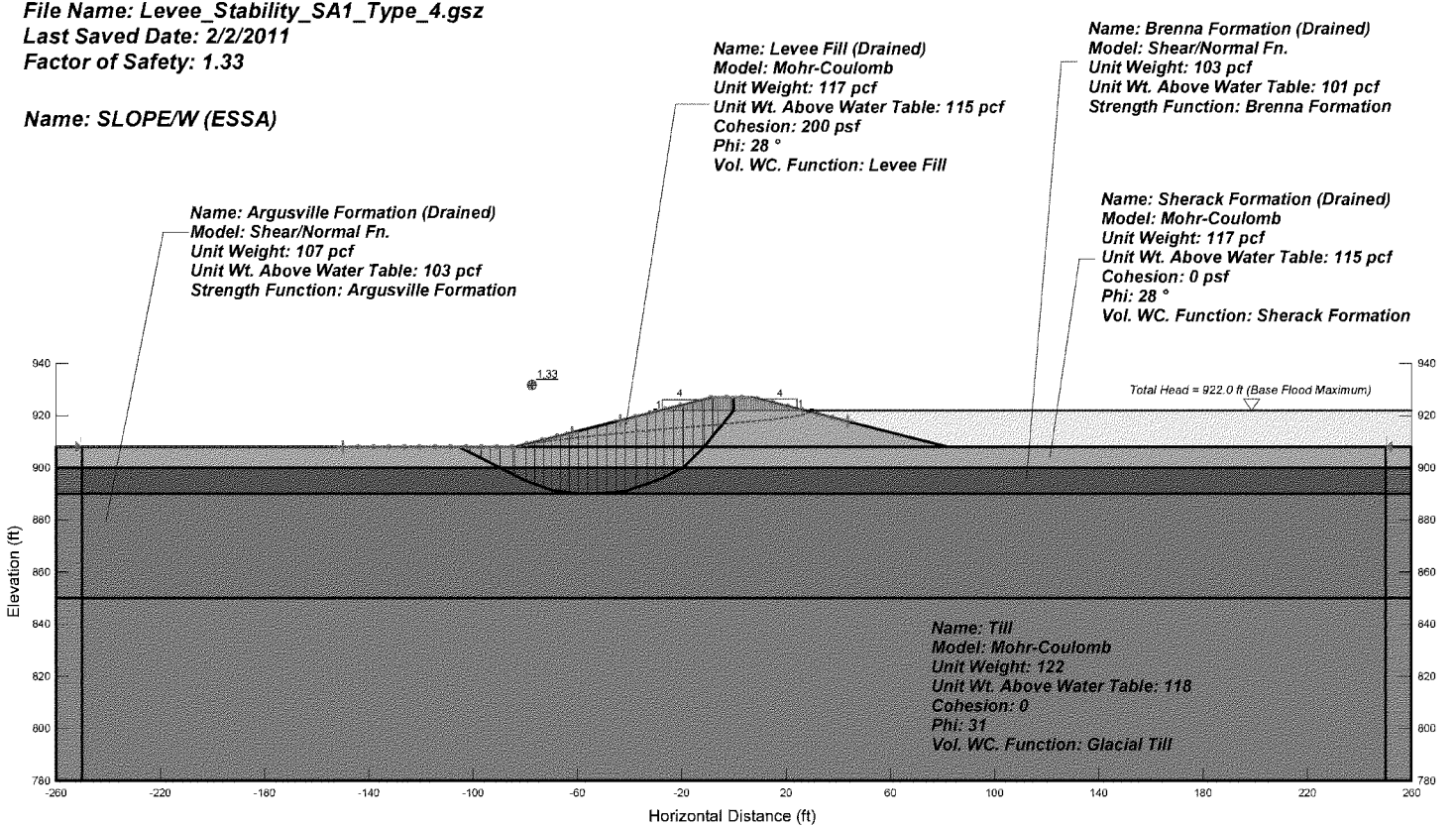
10 ft Brenna

40 ft Argusville

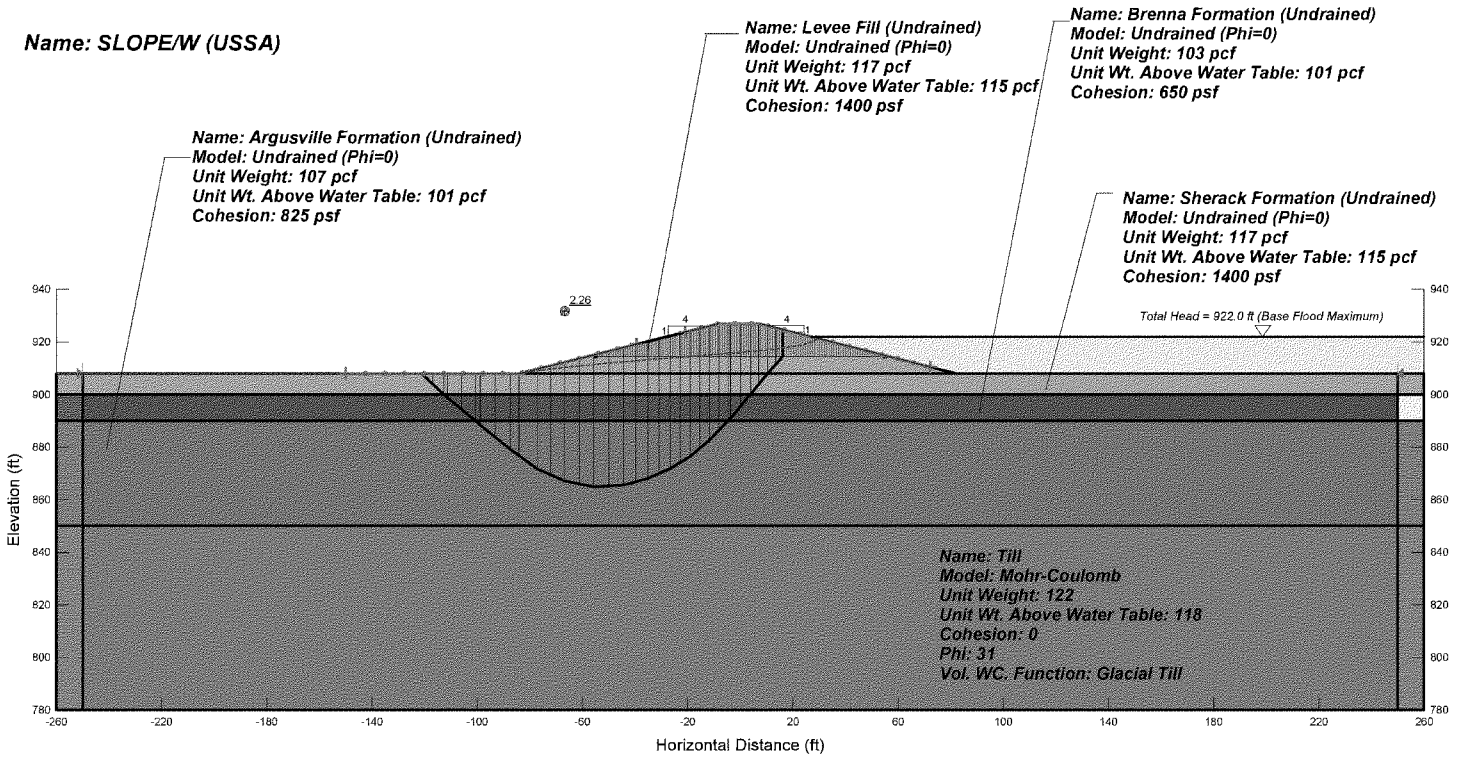
Till

Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_4.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 1.33

Name: SLOPE/W (ESSA)



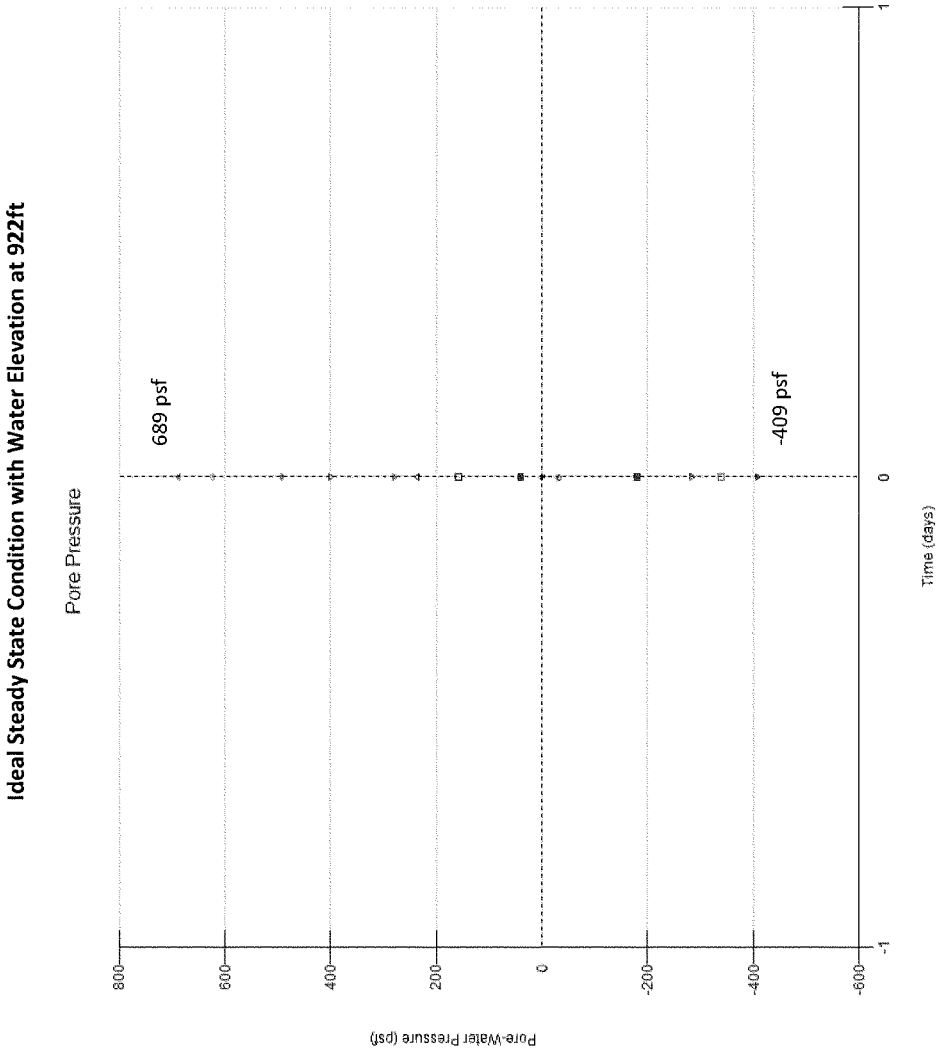
Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_4.gsz
Last Saved Date: 1/25/2011
Factor of Safety: 2.26

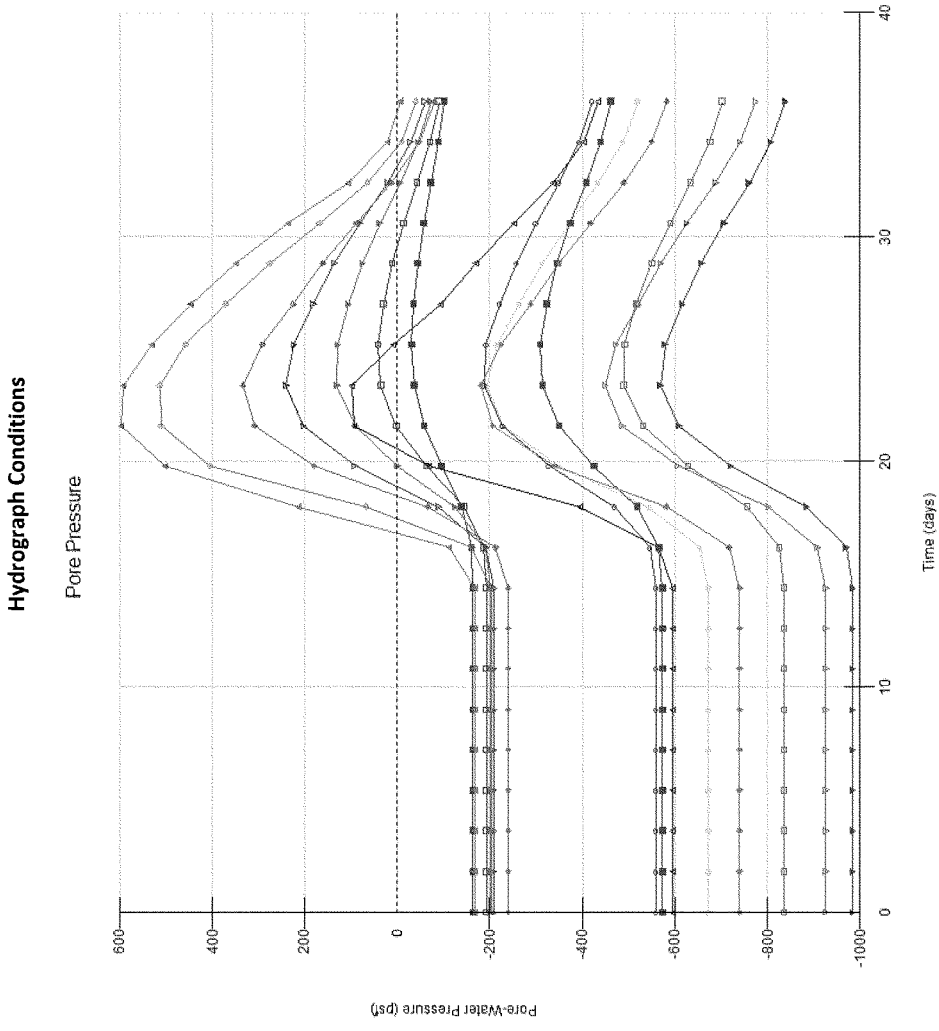


Type 1 (FS = 1.29)

and

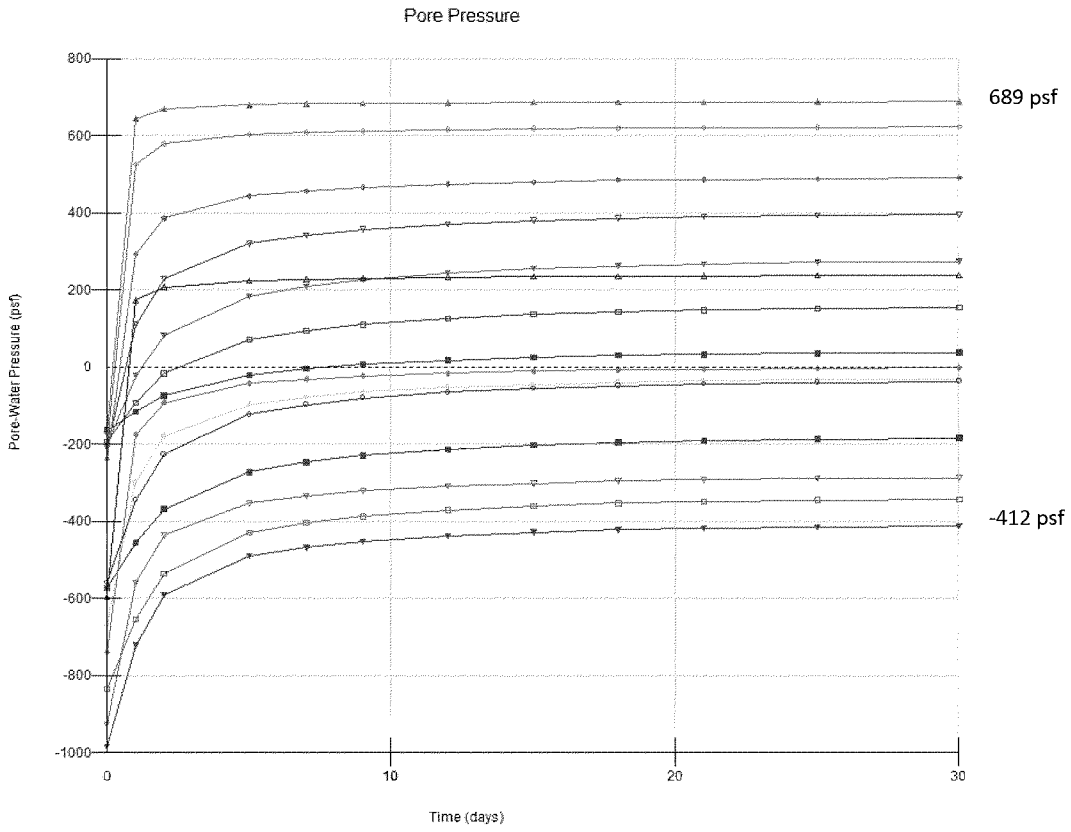
Type 4 (FS = 1.33)


Ideal Steady State Condition with Water Elevation at 922ft
Pore Pressure
689 psf
-409 psf
Pore-Water Pressure (psf)
800
600
400
200
0
-200
-400
-600
-1
0
1
Time (days)

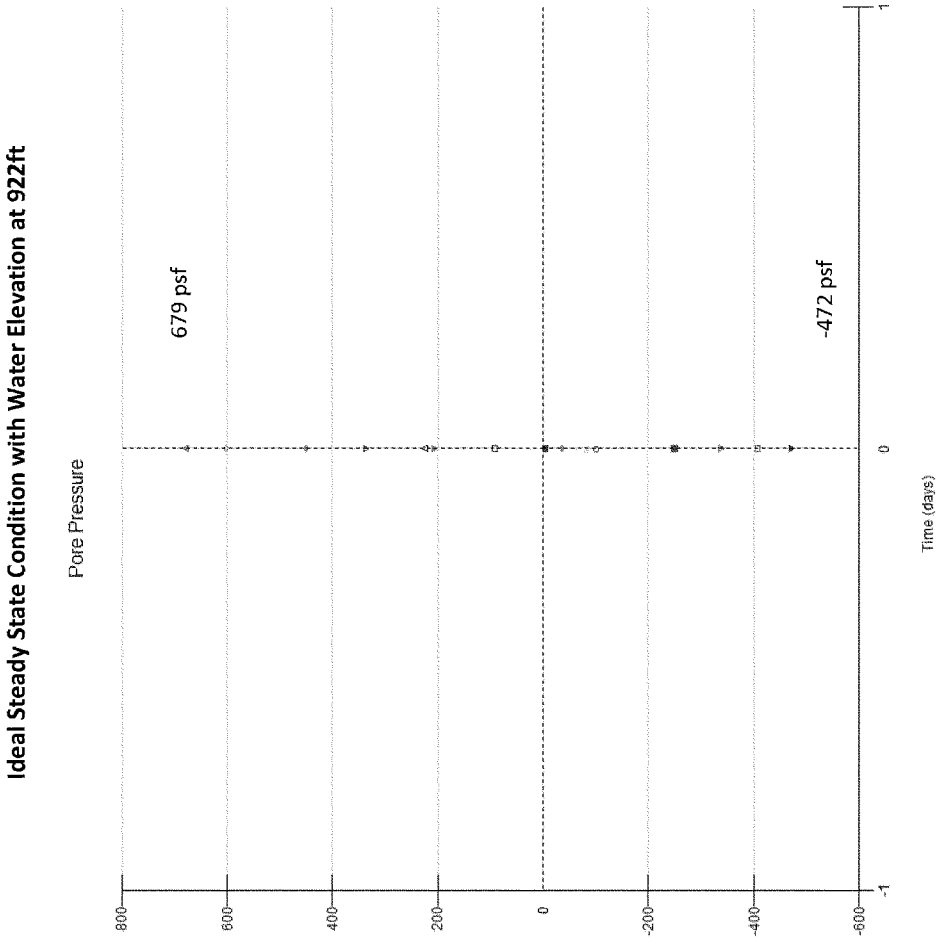

Hydrograph Conditions
Pore Pressure
Pore-Water Pressure (psf)
600
400
200
0
-200
-400
-600
-800
-1000
0
10
20
30
40
Time (days)

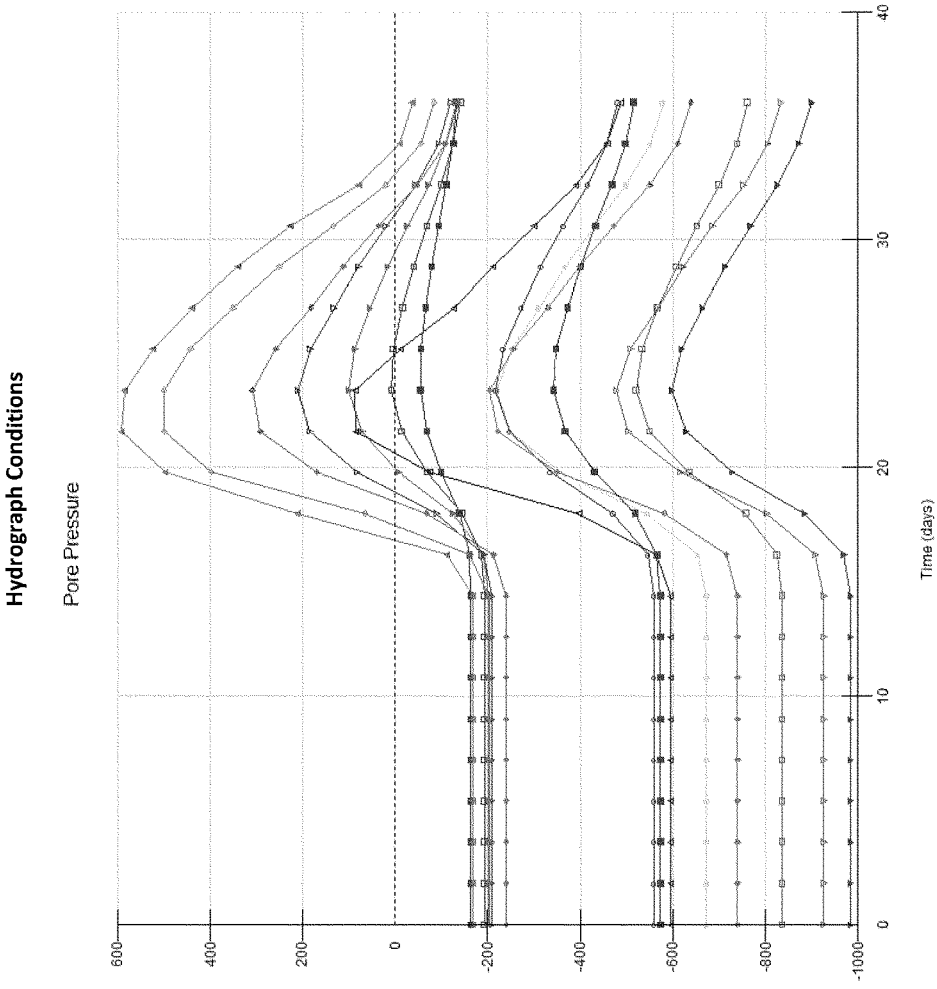
Steady State Condition in 30 Days

Embankment Fill = 18.6 ft/day **Sherack =** 93 ft/day **Brenna =** 9.3 ft/day **Argusville =** 9.3 ft/day **Till =** 1860 ft/day



Type 2 (FS = 1.39)

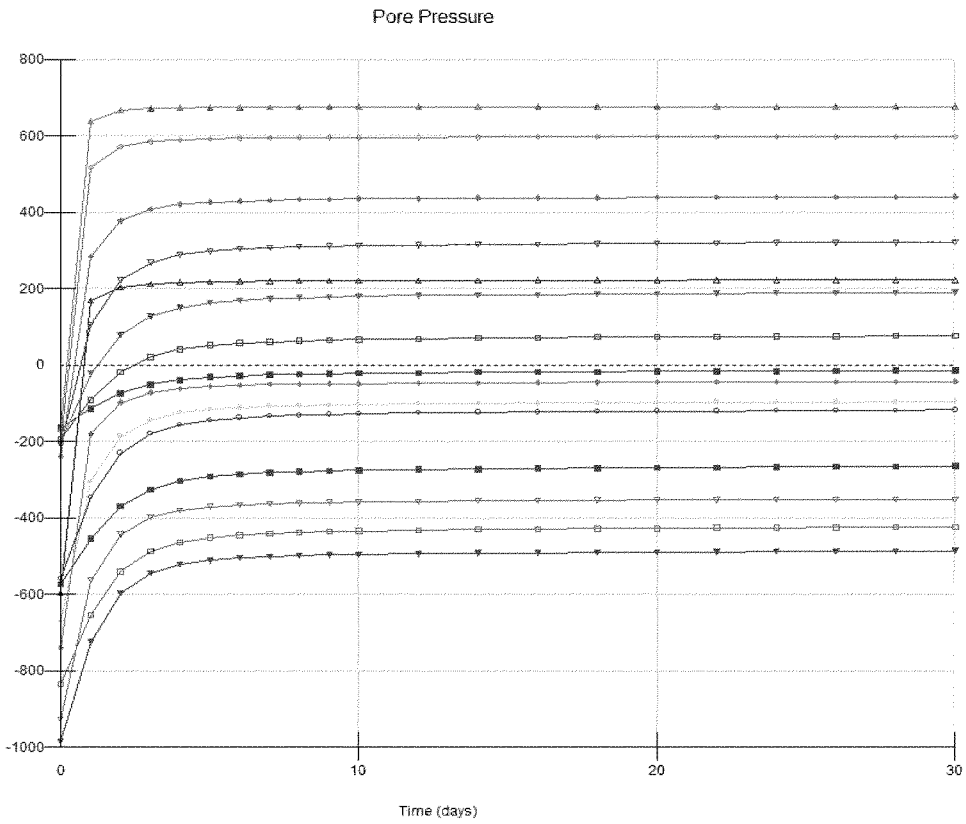

Ideal Steady State Condition with Water Elevation at 922ft
Pore Pressure
679 psf
-472 psf
800
600
400
200
0
-200
-400
-600
-1
0
1
Time (days)


Hydrograph Conditions
Pore Pressure
600
400
200
0
-200
-400
-600
-800
-1000
0
10
20
30
40
Time (days)

Steady State Condition in 30 Days

Embankment Fill = 18.6 ft/day **Sherack =** 93 ft/day **Oxidized Brenna =** 46.5 ft/day **Brenna =** 9.3 ft/day

Argusville = 9.3 ft/day **Till =** 1860 ft/day



677 psf

-486 psf

Levee Storage Area 1

Type 1

2/2/2011

Levee Fill

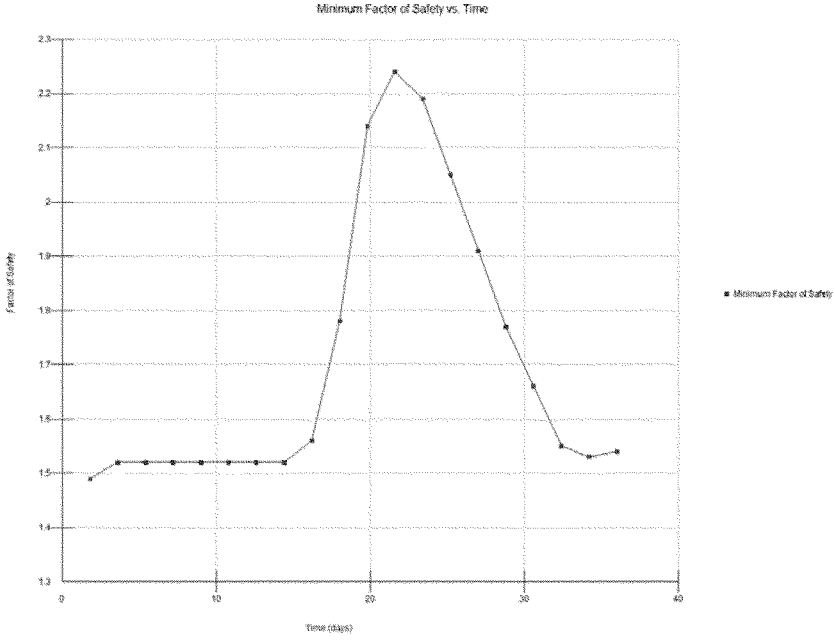
8 ft Sherack

20 ft Brenna

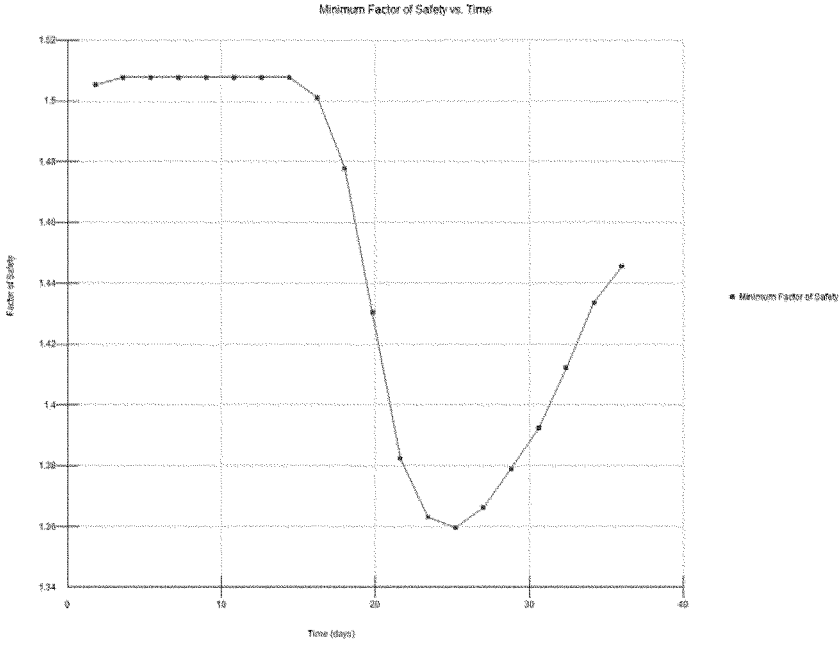
30 ft Argusville

Till

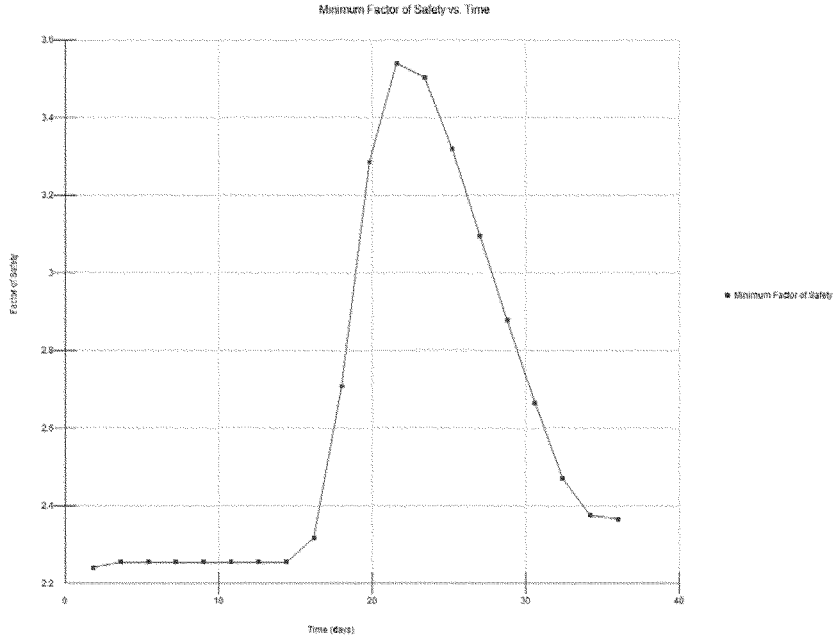
Factor of Safety for ESSA Transient Hydrograph (36 days) Left to Right



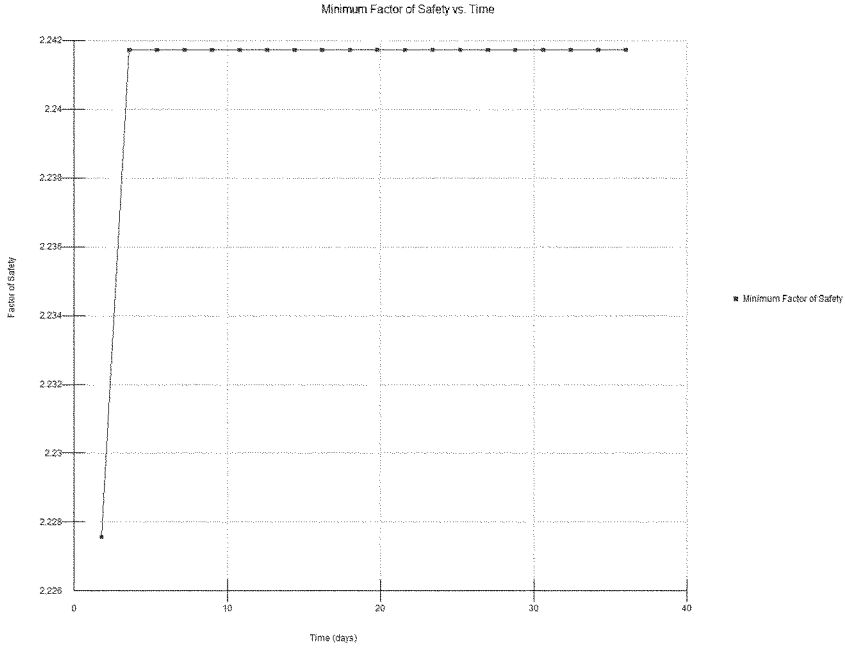
Factor of Safety for ESSA Transient Hydrograph (36 days) Right to Left



Factor of Safety for USSA Transient Hydrograph (36 days) Left to Right



Factor of Safety for USSA Transient Hydrograph (36 days) Right to Left



Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
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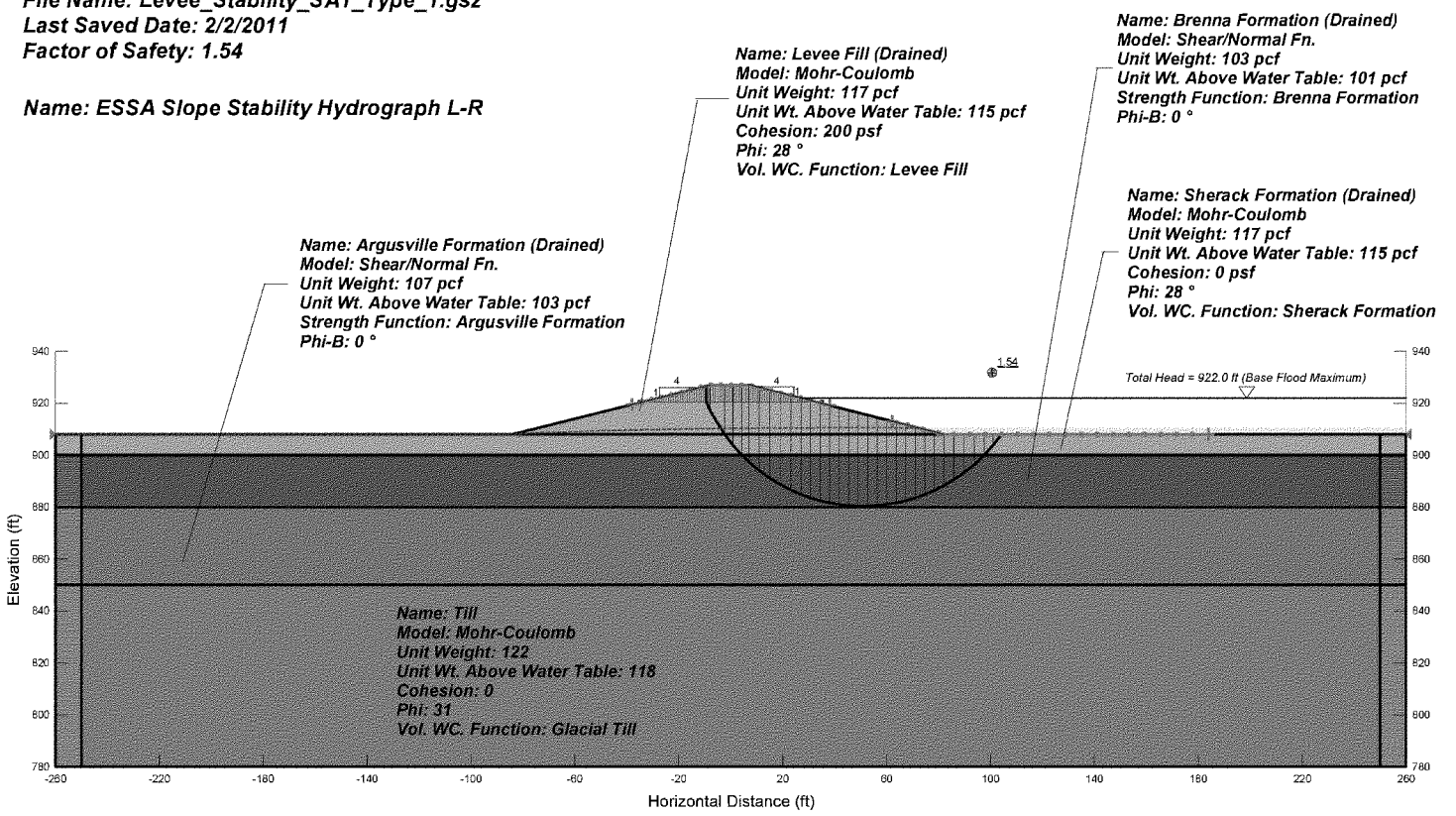
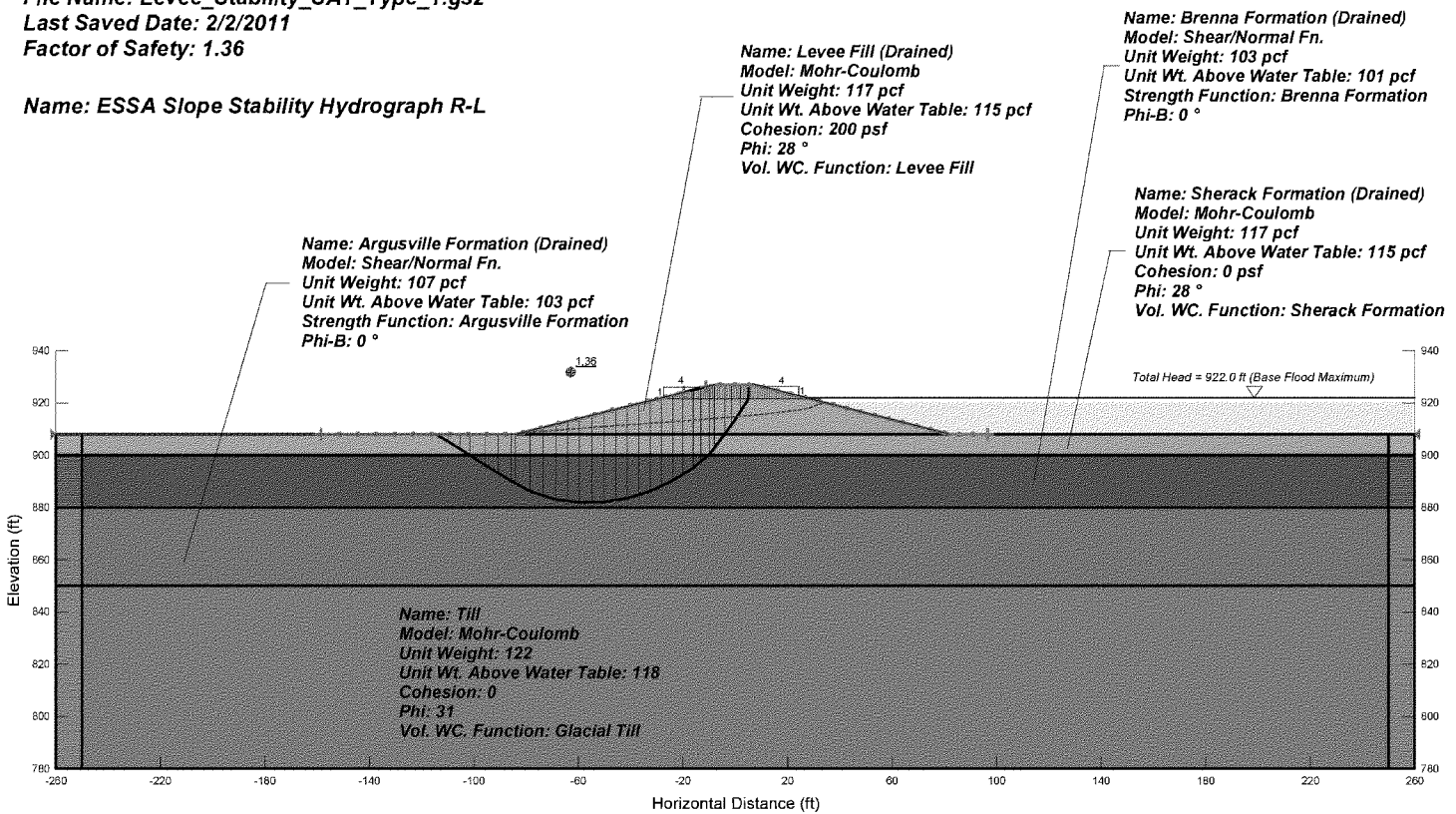
Last Saved Date: 2/2/2011
Factor of Safety: 1.54

Name: ESSA Slope Stability Hydrograph L-R



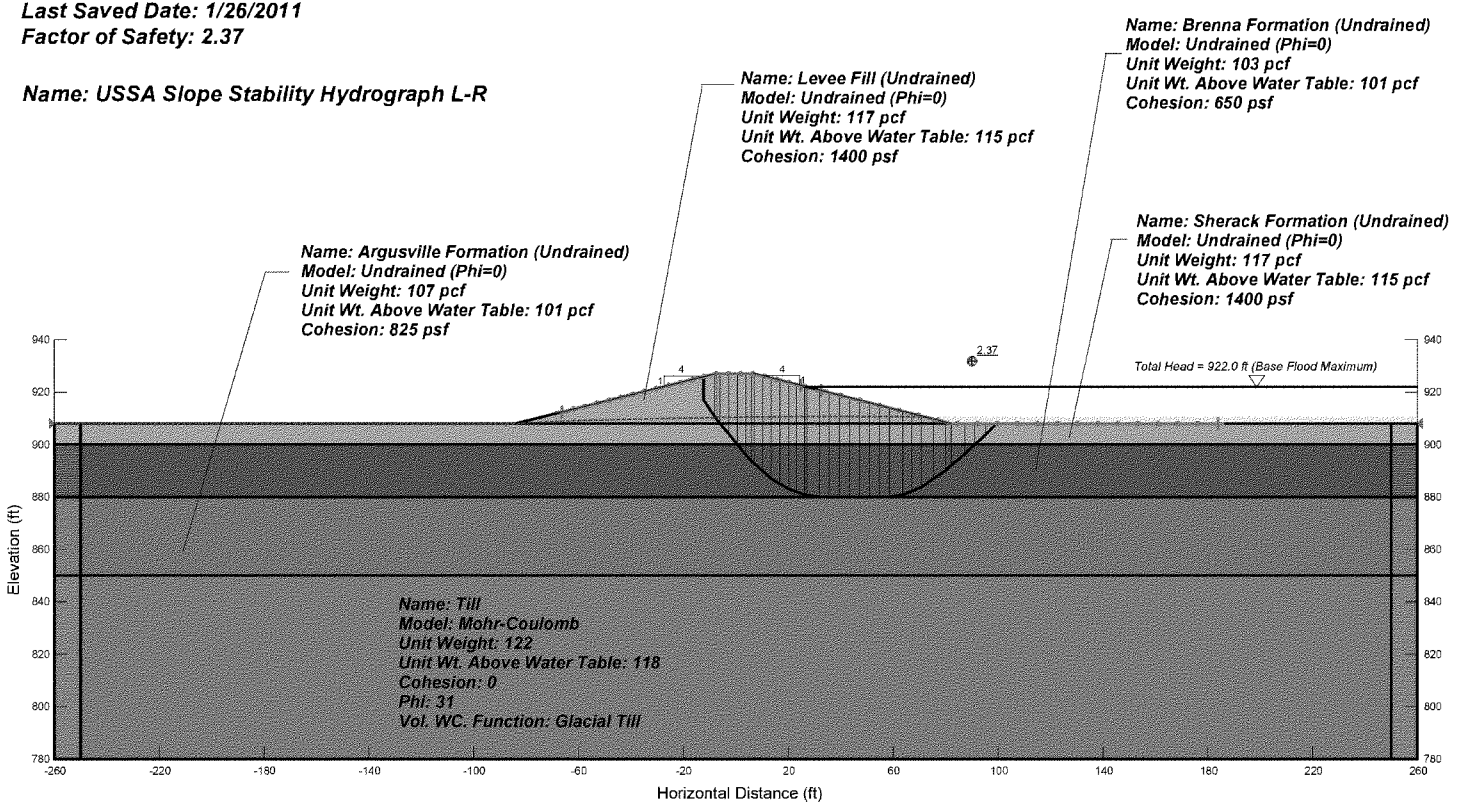
Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_1.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 1.36

Name: ESSA Slope Stability Hydrograph R-L



Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_1.gsz
Last Saved Date: 1/26/2011
Factor of Safety: 2.37

Name: USSA Slope Stability Hydrograph L-R



Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_1.gsz
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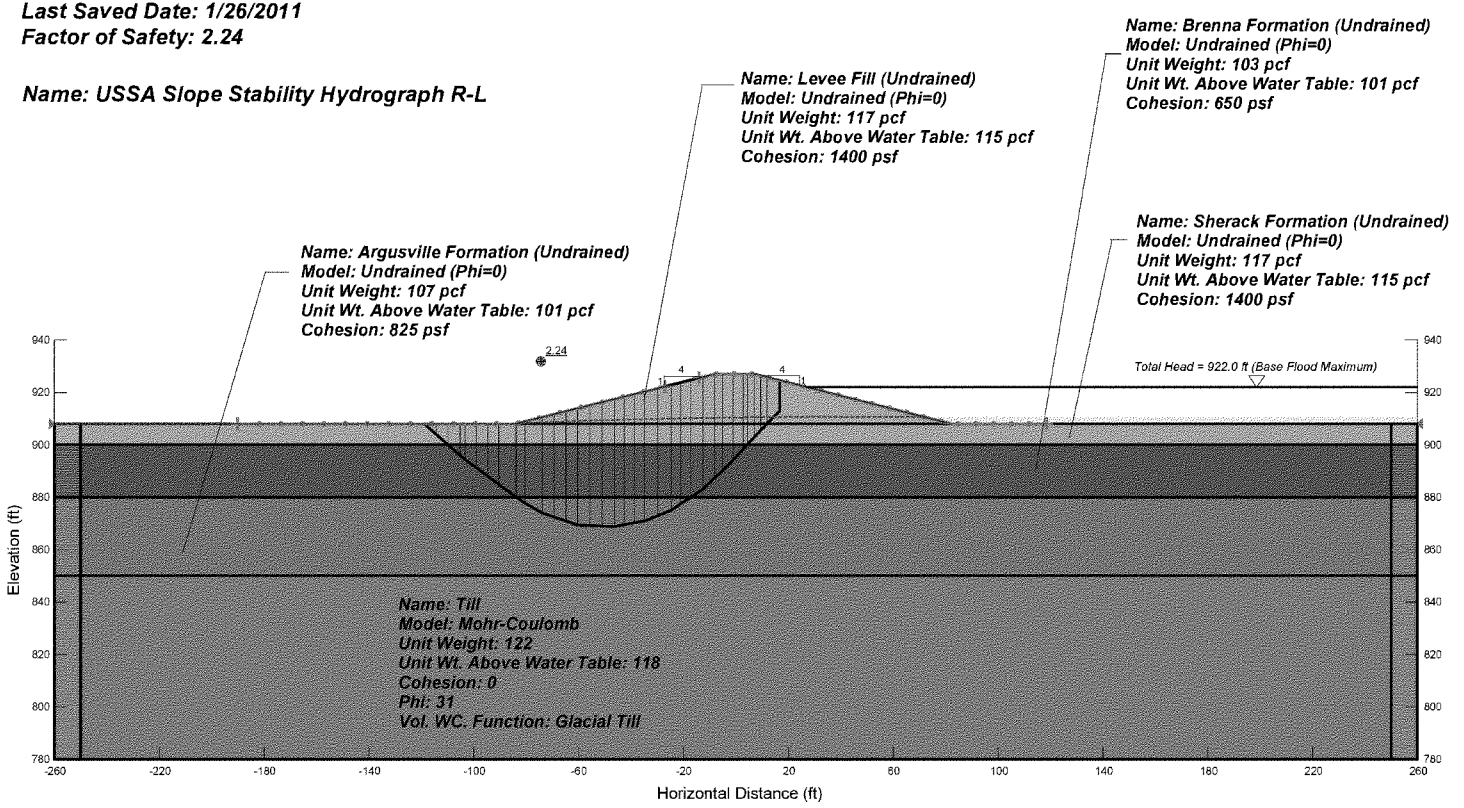
Factor of Safety: 2.24



Levee Storage Area 1

Type 2

2/2/2011

Levee Fill

8 ft Sherack

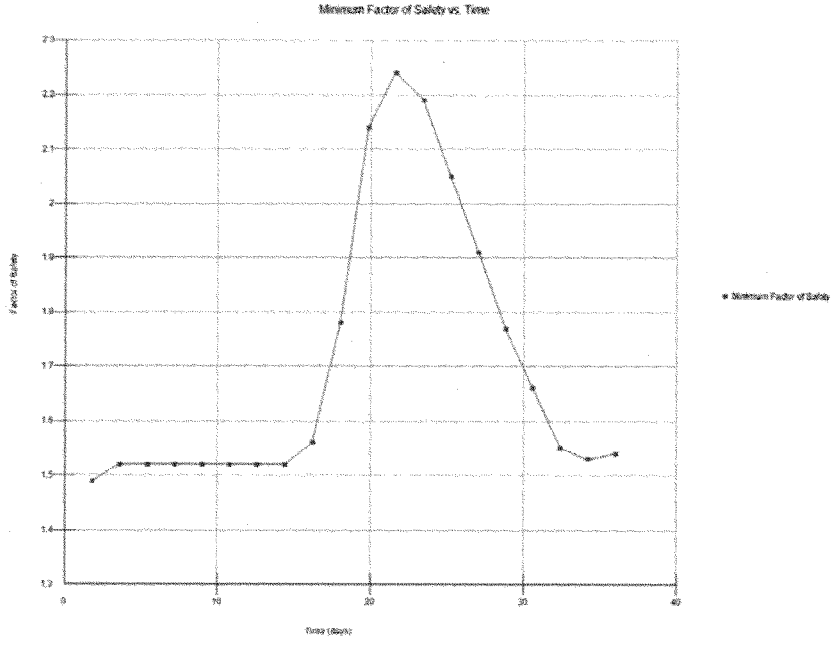
5 ft Oxidized Brenna

15 ft Brenna

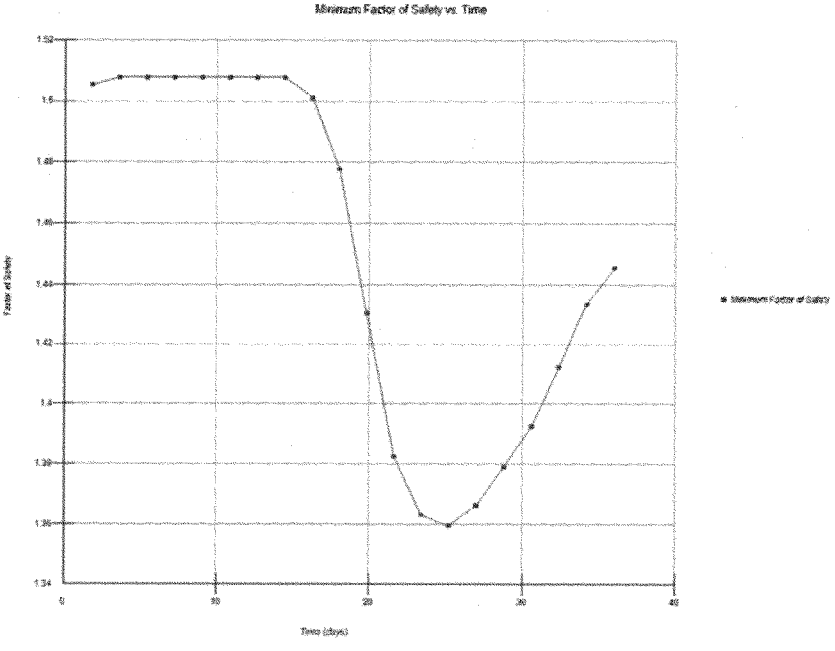
30 ft Argusville

Till

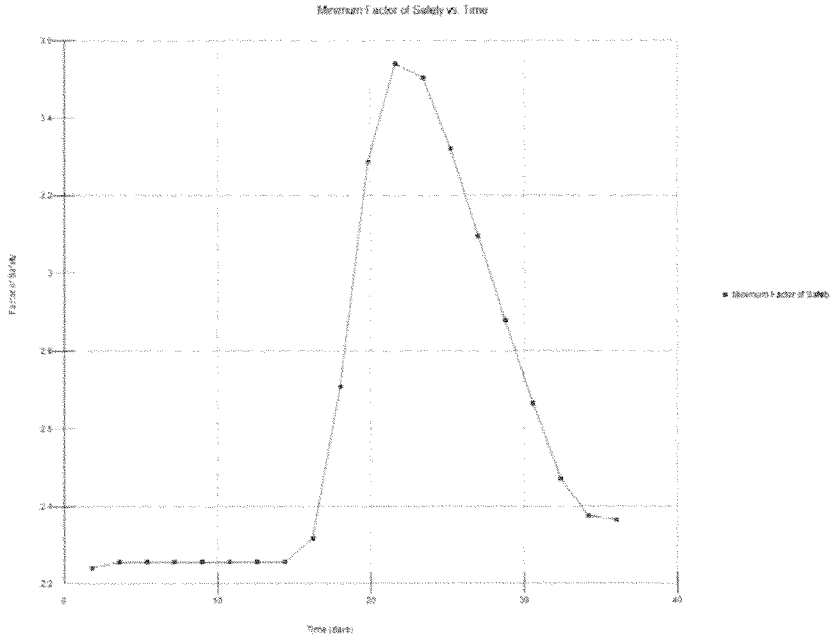
Factor of Safety for ESSA Transient Hydrograph (36 days) Left to Right



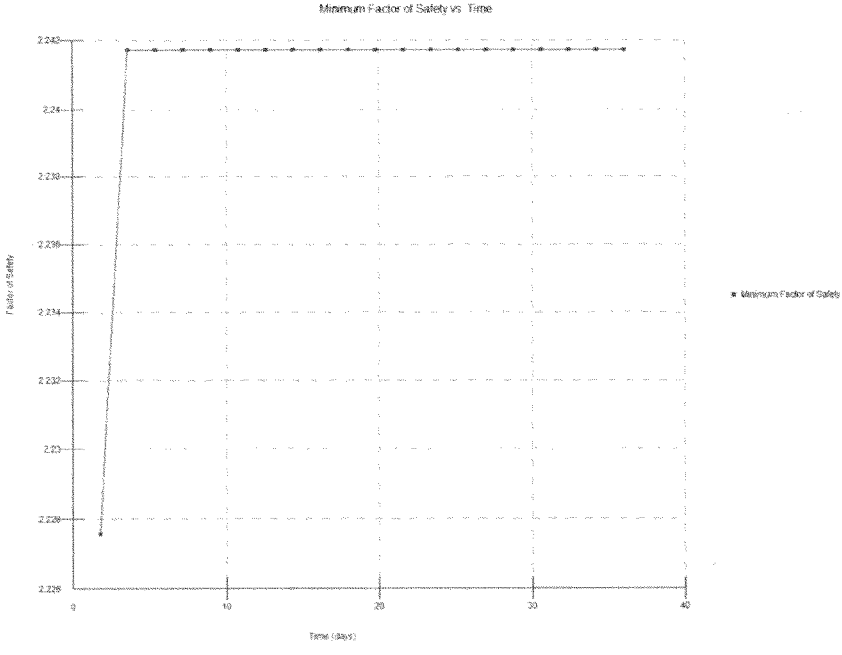
Factor of Safety for ESSA Transient Hydrograph (36 days) Right to Left



Factor of Safety for USSA Transient Hydrograph (36 days) Left to Right

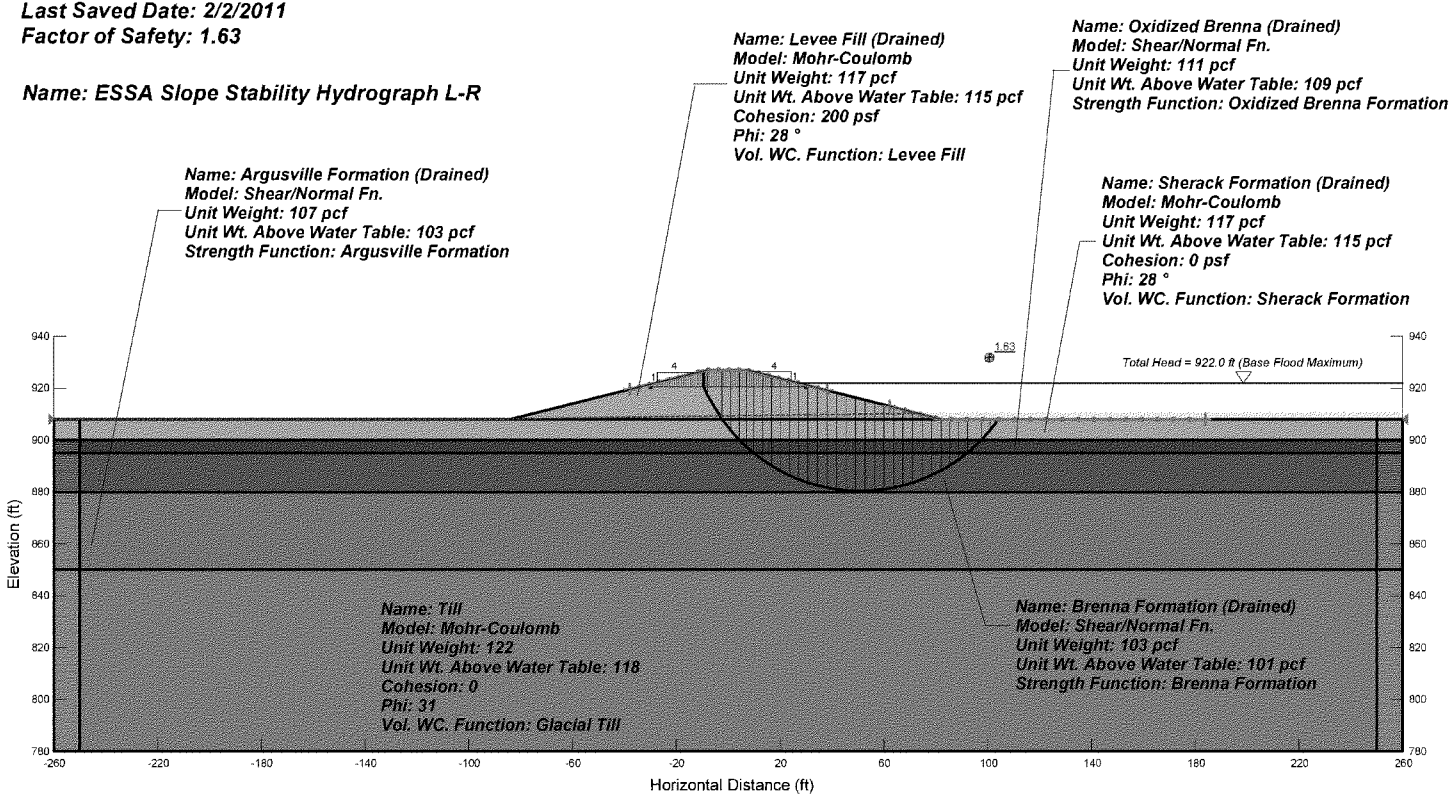


Factor of Safety for USSA Transient Hydrograph (36 days) Right to Left



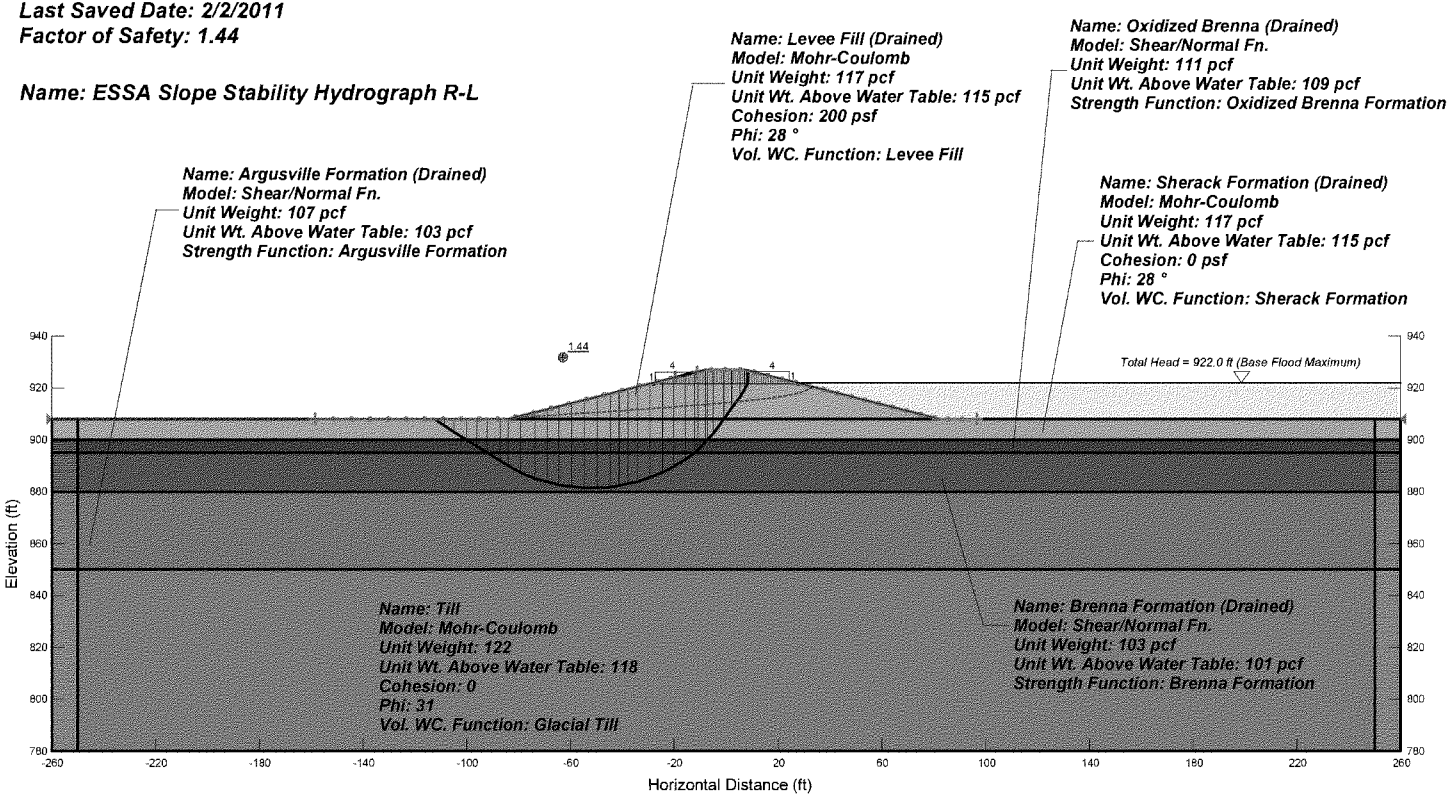
Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_2.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 1.63

Name: ESSA Slope Stability Hydrograph L-R

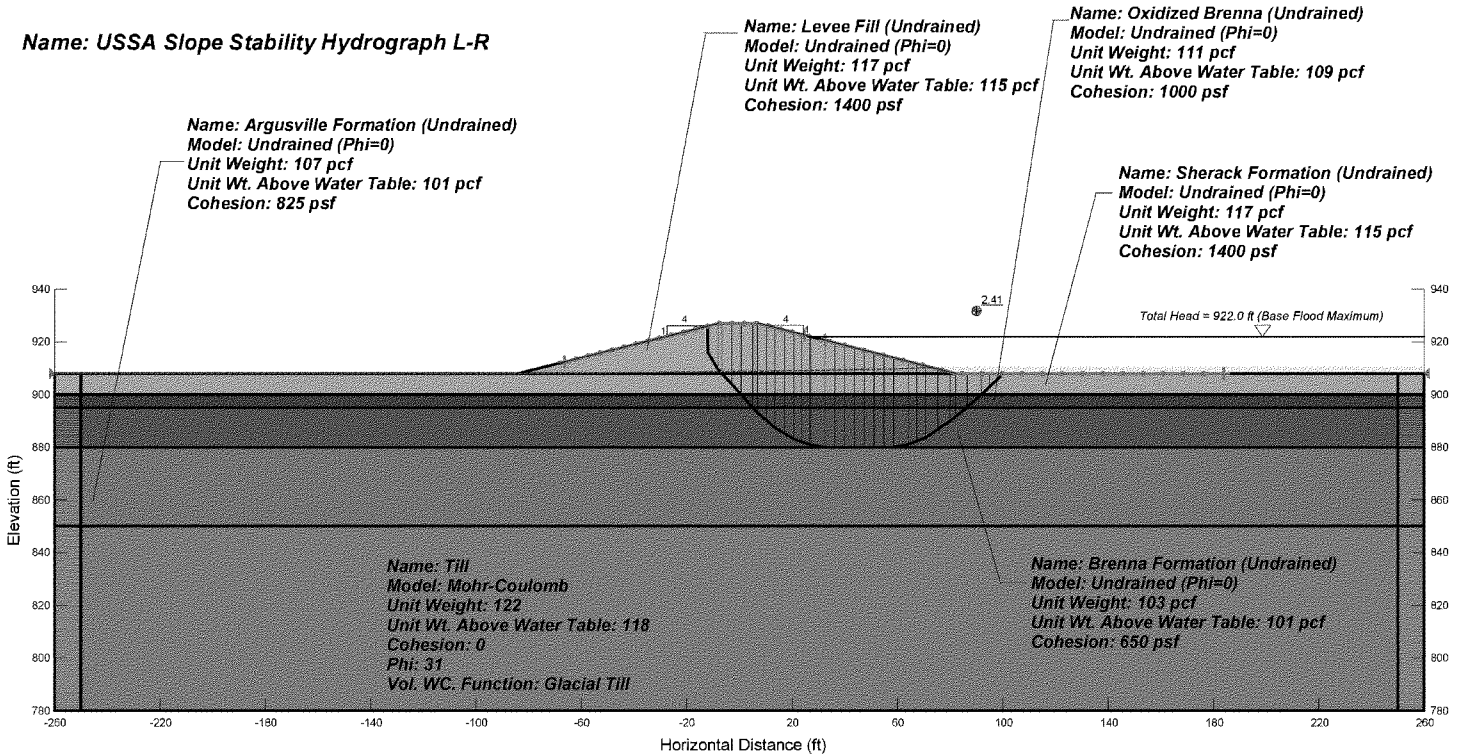


Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_2.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 1.44

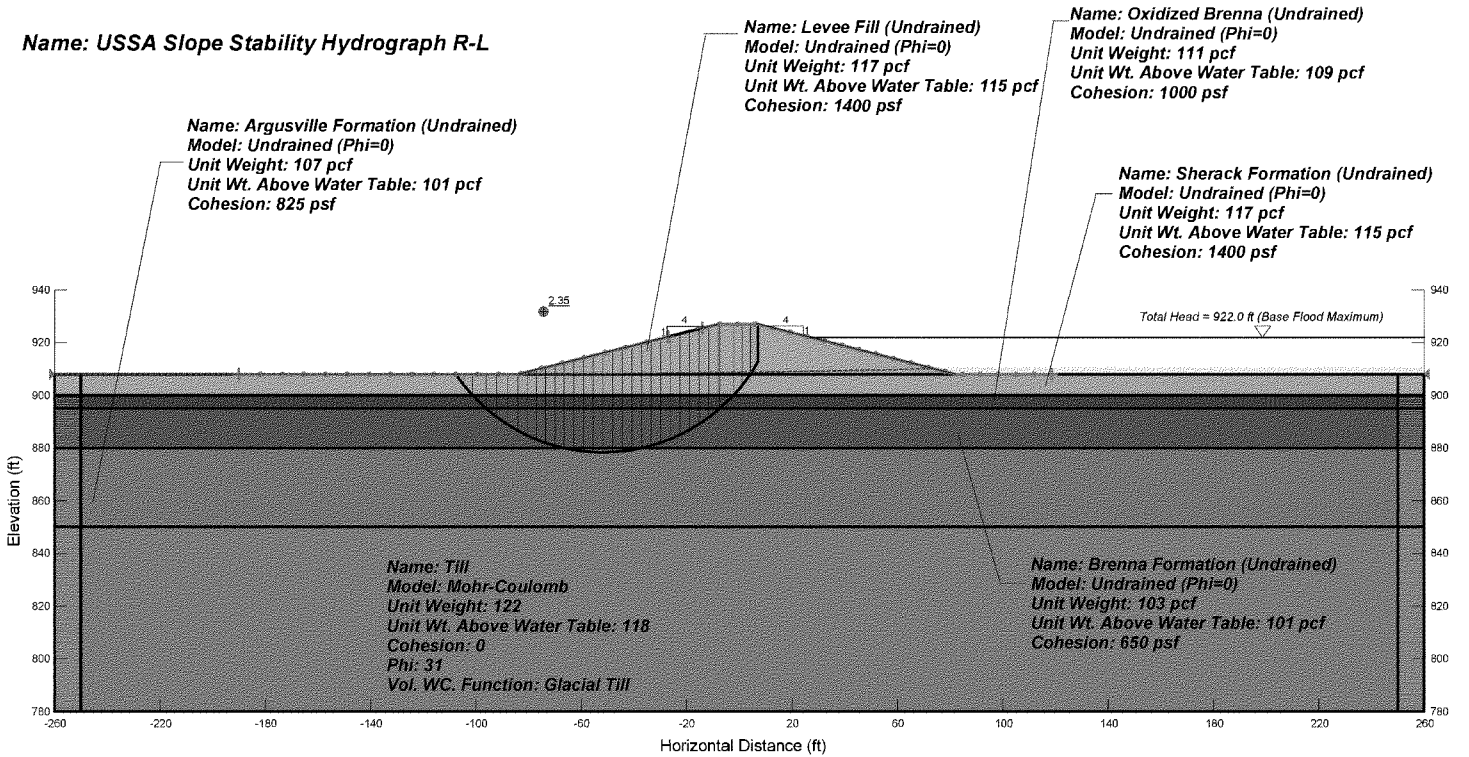
Name: ESSA Slope Stability Hydrograph R-L



Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_2.gsz
Last Saved Date: 2/1/2011
Factor of Safety: 2.41



Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_2.gsz
Last Saved Date: 2/1/2011
Factor of Safety: 2.35



Levee Storage Area 1

Type 4

2/2/2011

Levee Fill

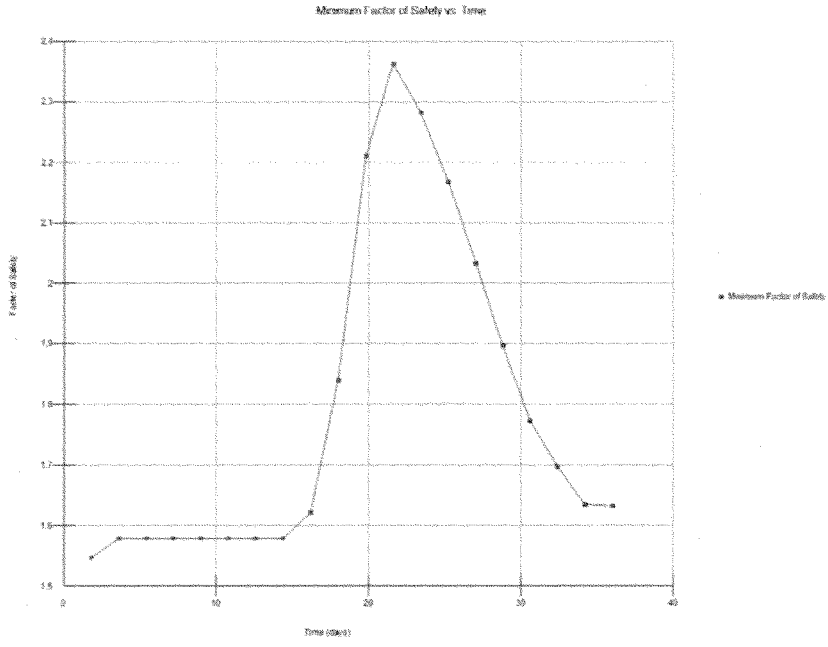
8 ft Sherack

10 ft Brenna

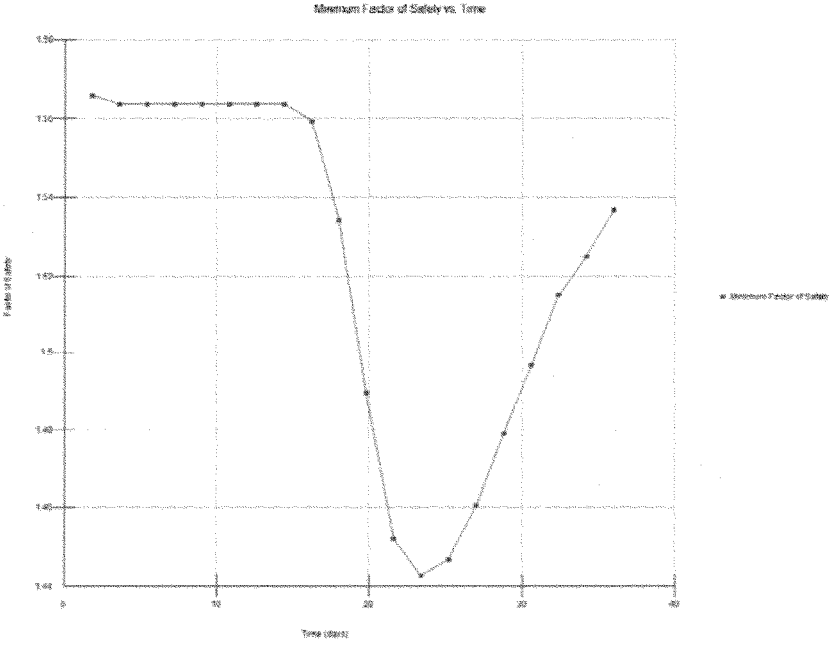
40 ft Argusville

Till

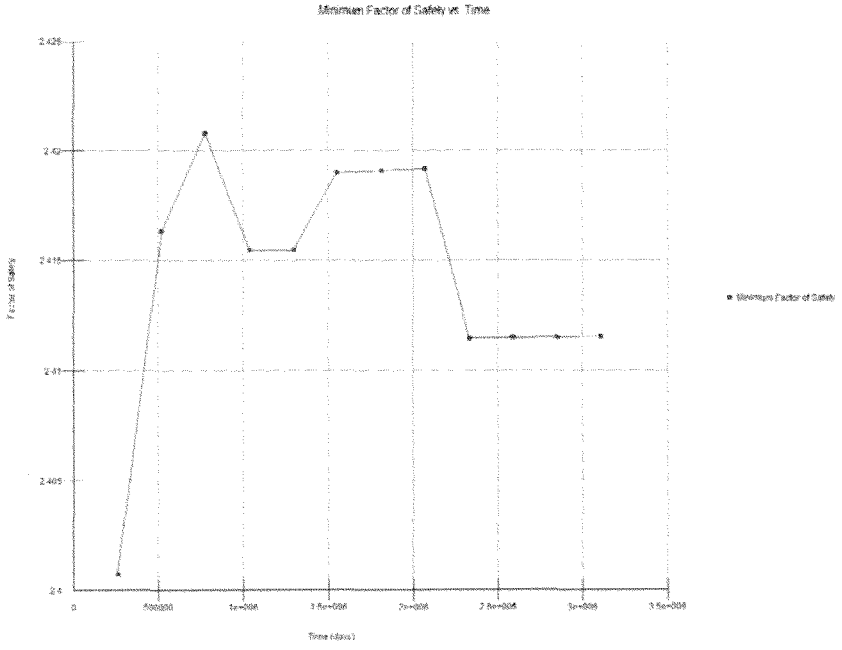
Factor of Safety for ESSA Transient Hydrograph (36 days) Left to Right



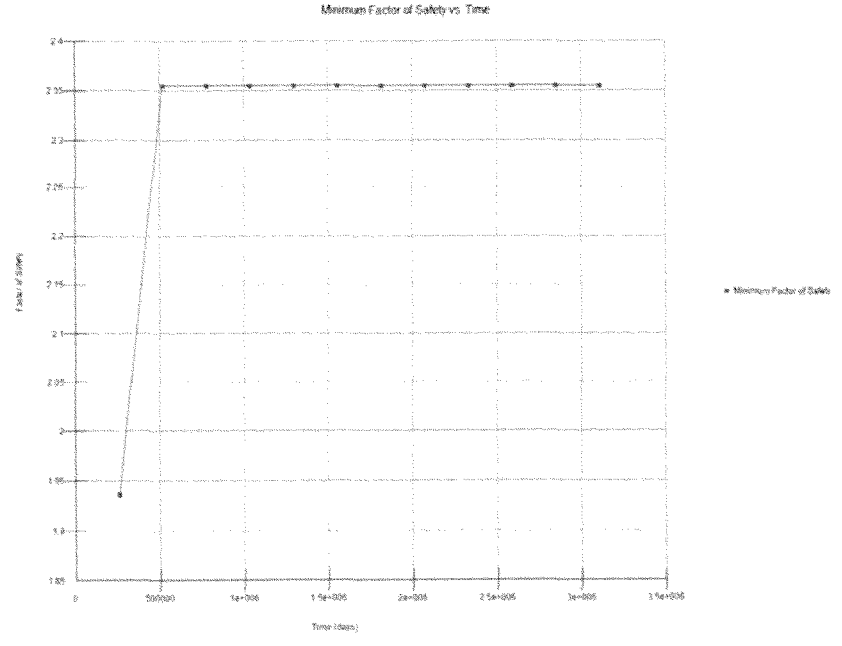
Factor of Safety for ESSA Transient Hydrograph (36 days) Right to Left



Factor of Safety for USSA Transient Hydrograph (36 days) Left to Right

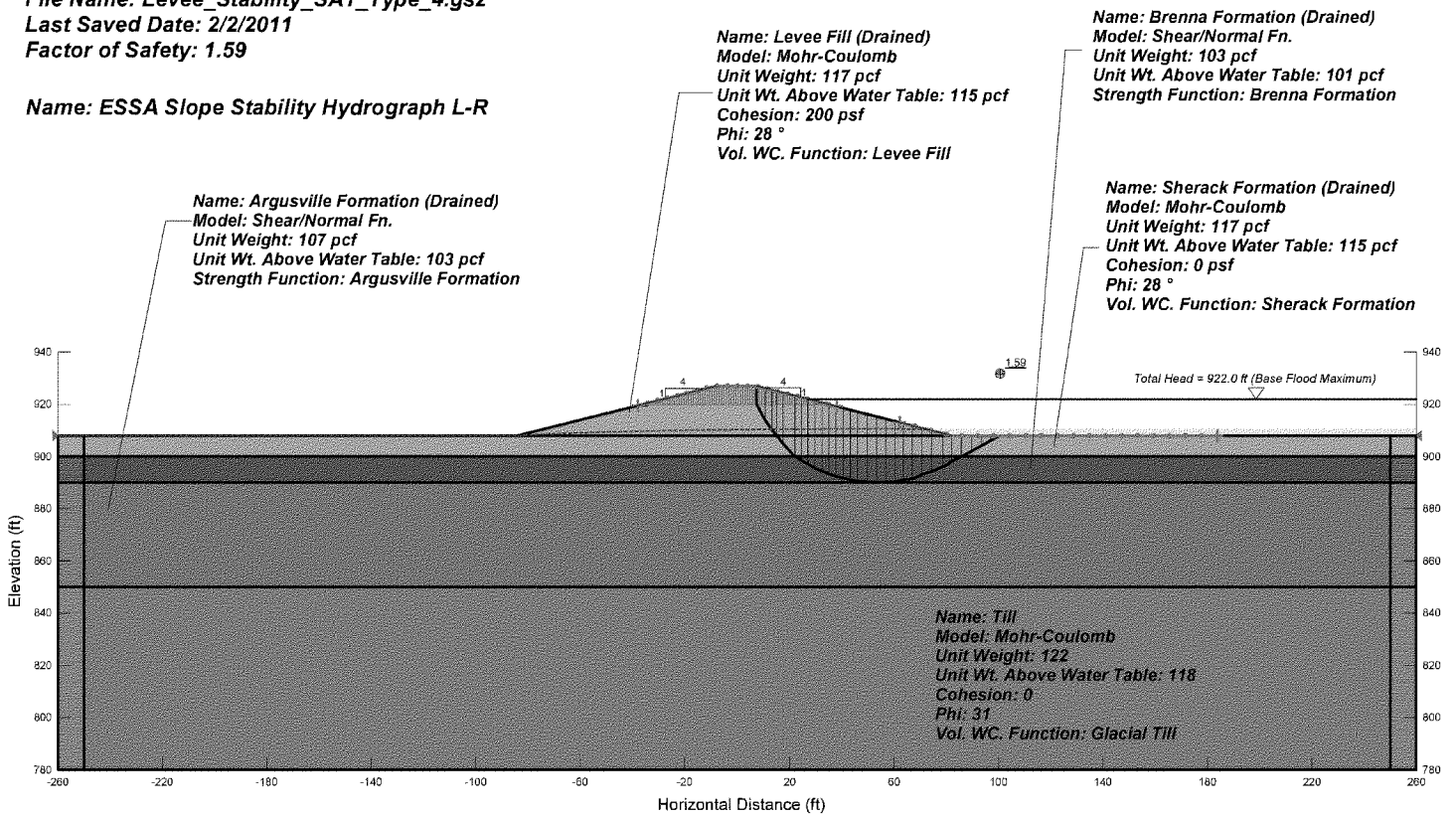


Factor of Safety for USSA Transient Hydrograph (36 days) Right to Left



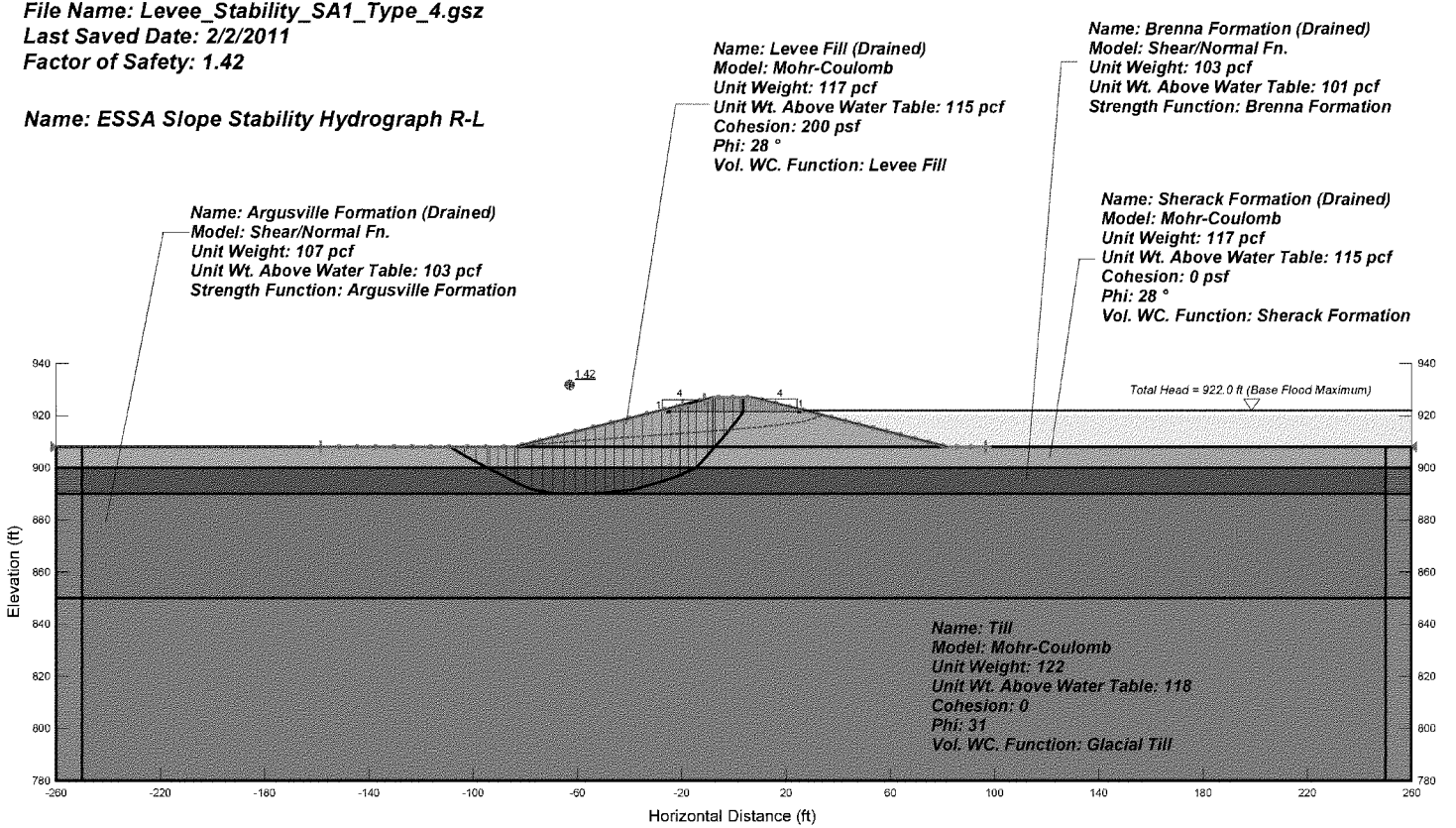
Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_4.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 1.59

Name: ESSA Slope Stability Hydrograph L-R



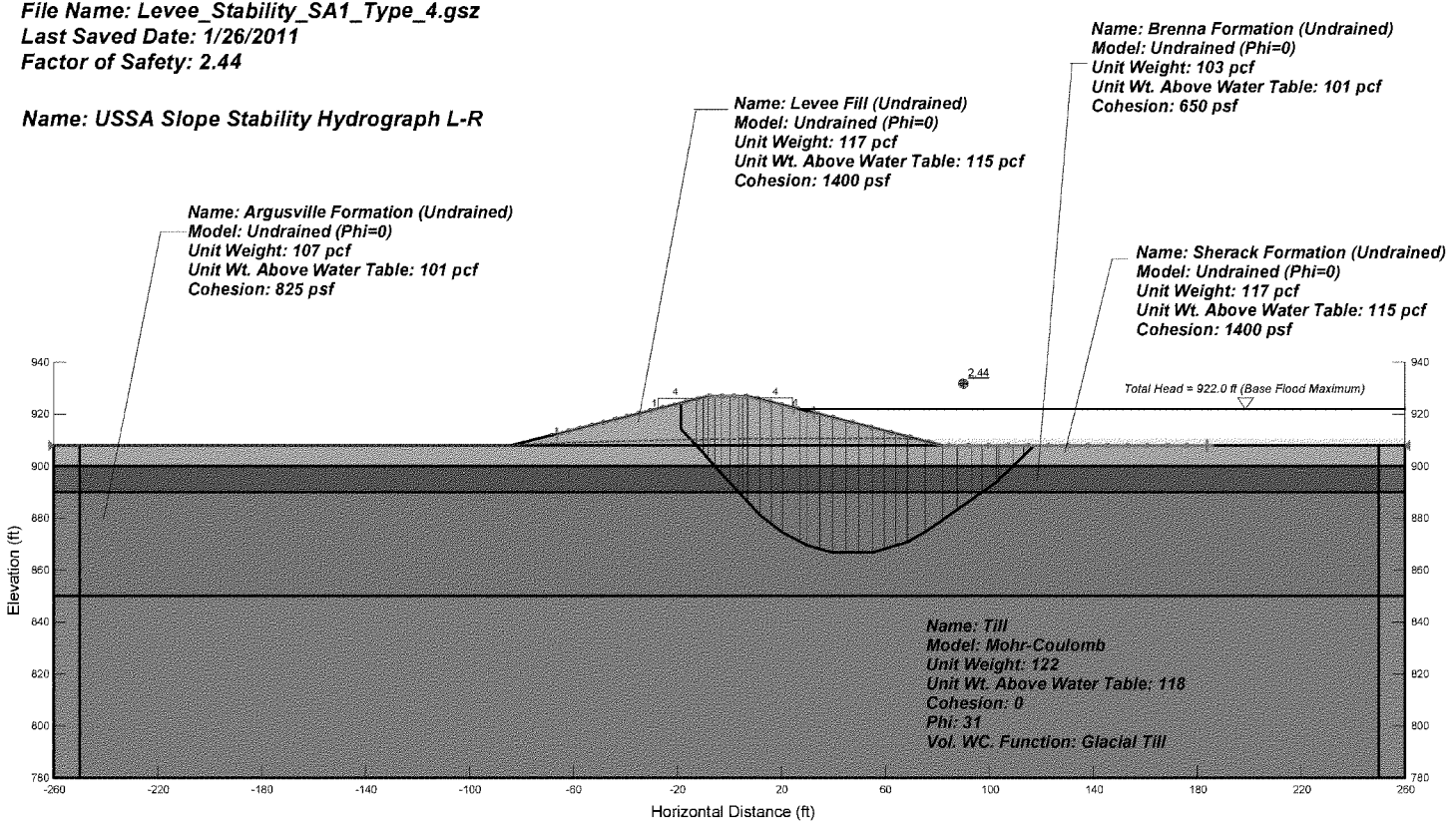
Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_4.gsz
Last Saved Date: 2/2/2011
Factor of Safety: 1.42

Name: ESSA Slope Stability Hydrograph R-L

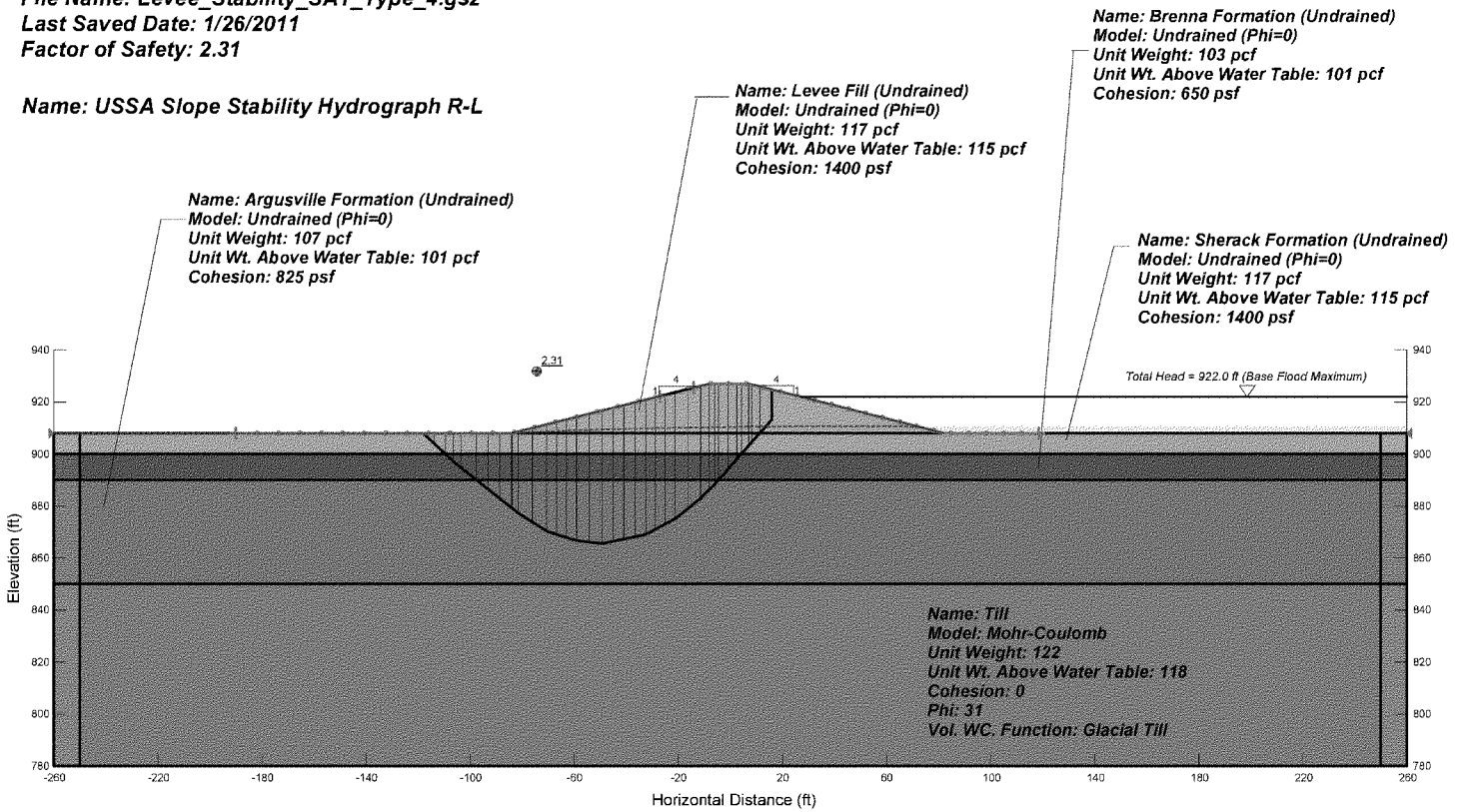


Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_4.gsz
Last Saved Date: 1/26/2011
Factor of Safety: 2.44

Name: USSA Slope Stability Hydrograph L-R

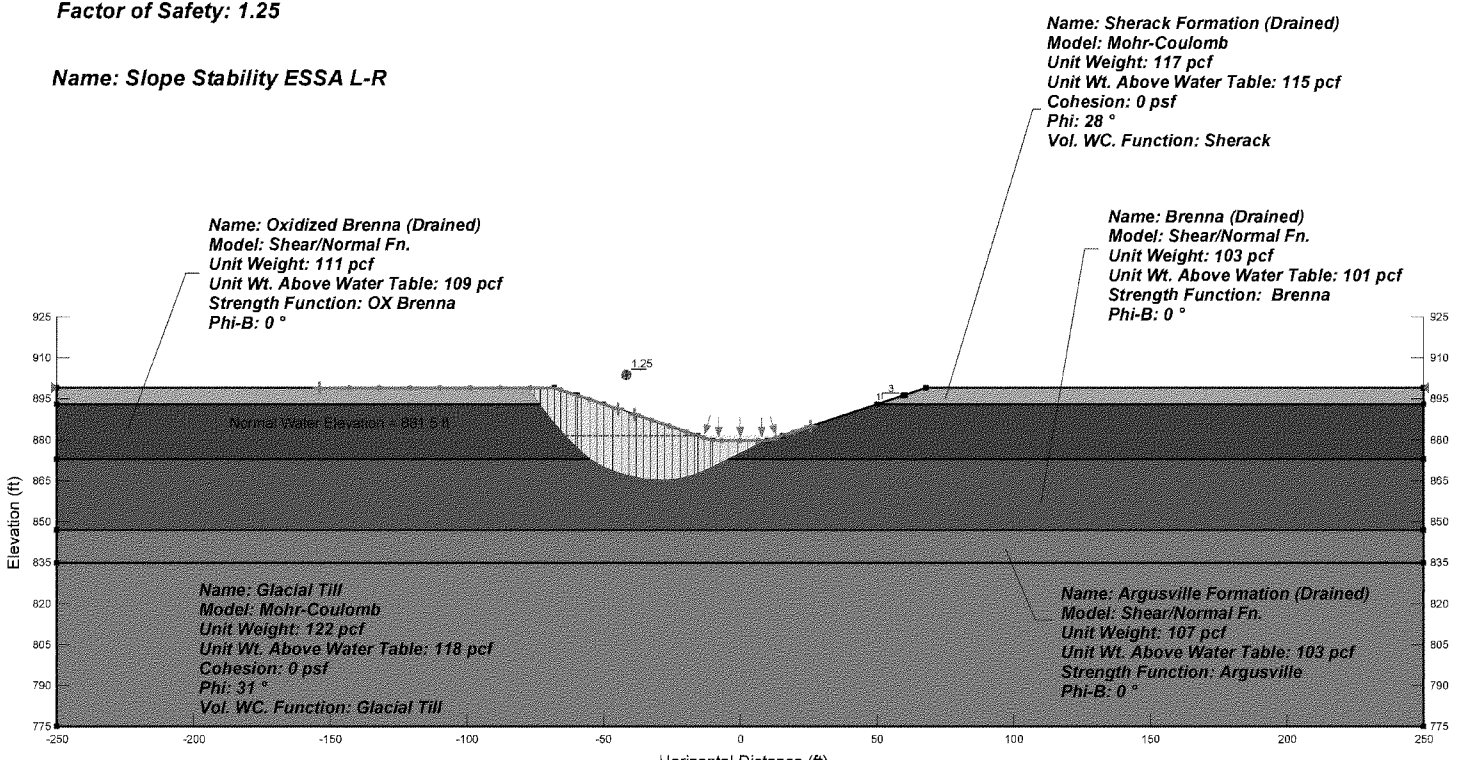


Fargo-Moorhead, Phase 4, Storage Cell Levees, Seepage and Stability Analysis
Stability Analysis, Steady State Seepage, Base Flood Case (Max Head)
File Name: Levee_Stability_SA1_Type_4.gsz
Last Saved Date: 1/26/2011
Factor of Safety: 2.31



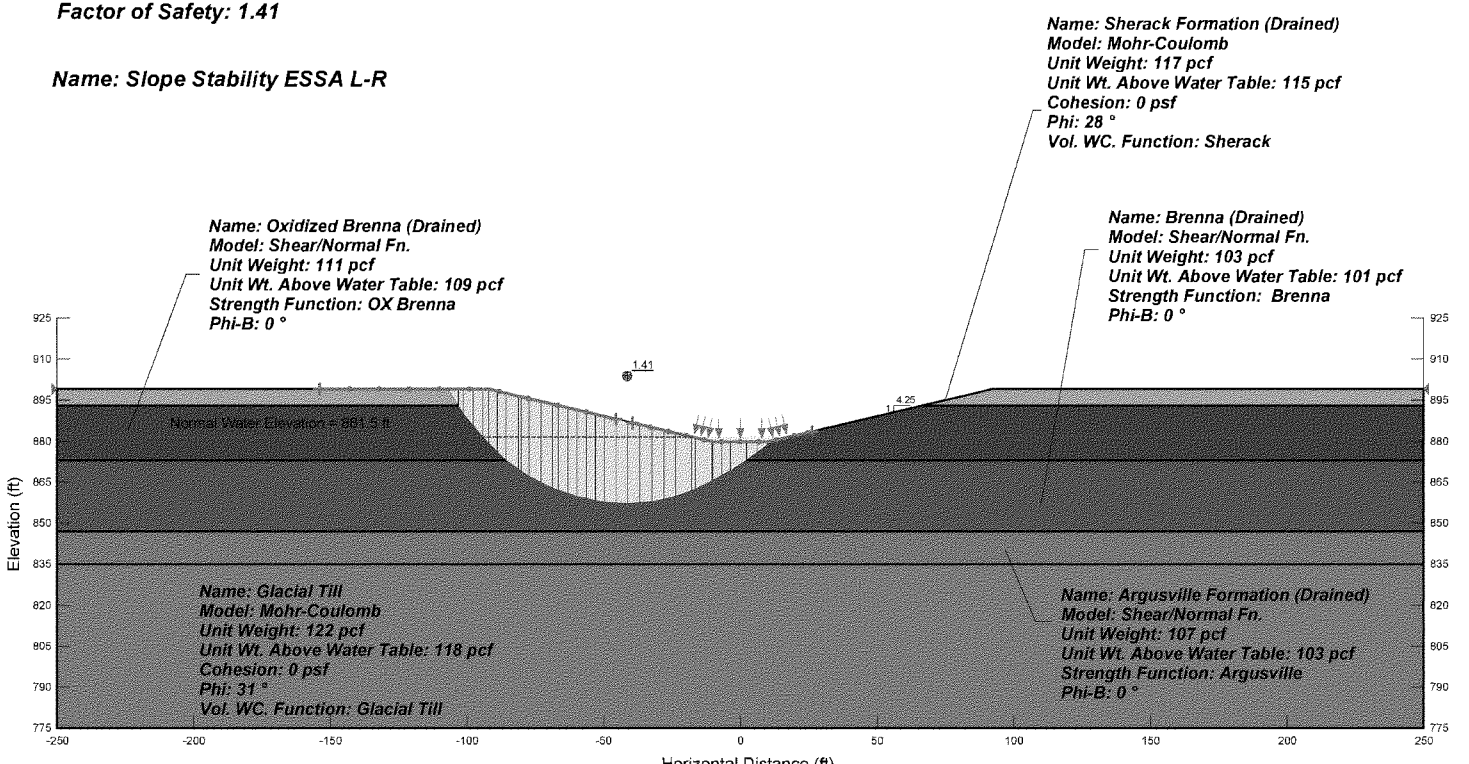
Fargo-Moorhead, Phase 4, Maple River Upstream Channel
Stability Analysis
File Name: Maple River Channel_3to1slope.gsz
Last Saved Date: 2/4/2011
Factor of Safety: 1.25

Name: Slope Stability ESSA L-R



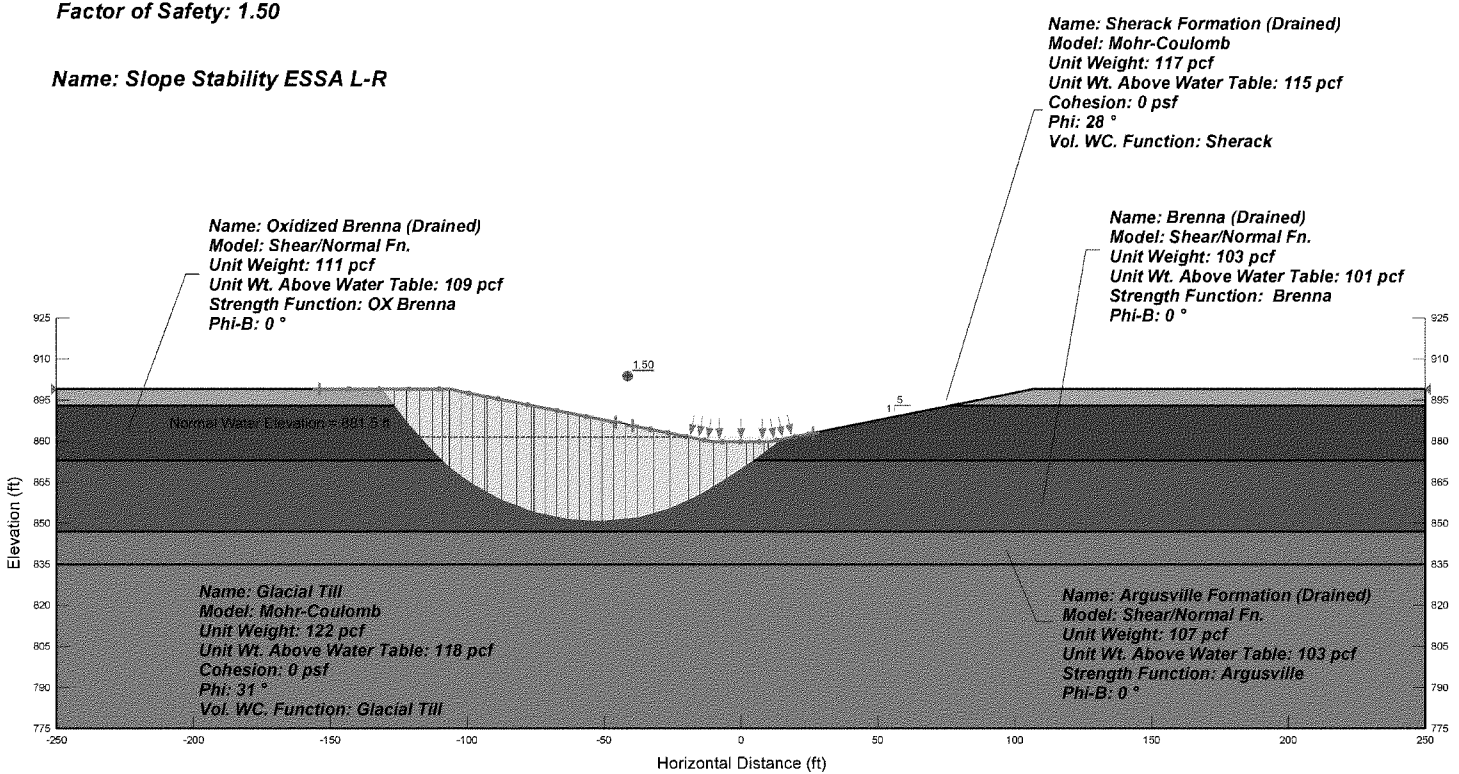
Fargo-Moorhead, Phase 4, Maple River Upstream Channel
Stability Analysis
File Name: Maple River Channel_4.25to1slope.gsz
Last Saved Date: 2/6/2011
Factor of Safety: 1.41

Name: Slope Stability ESSA L-R



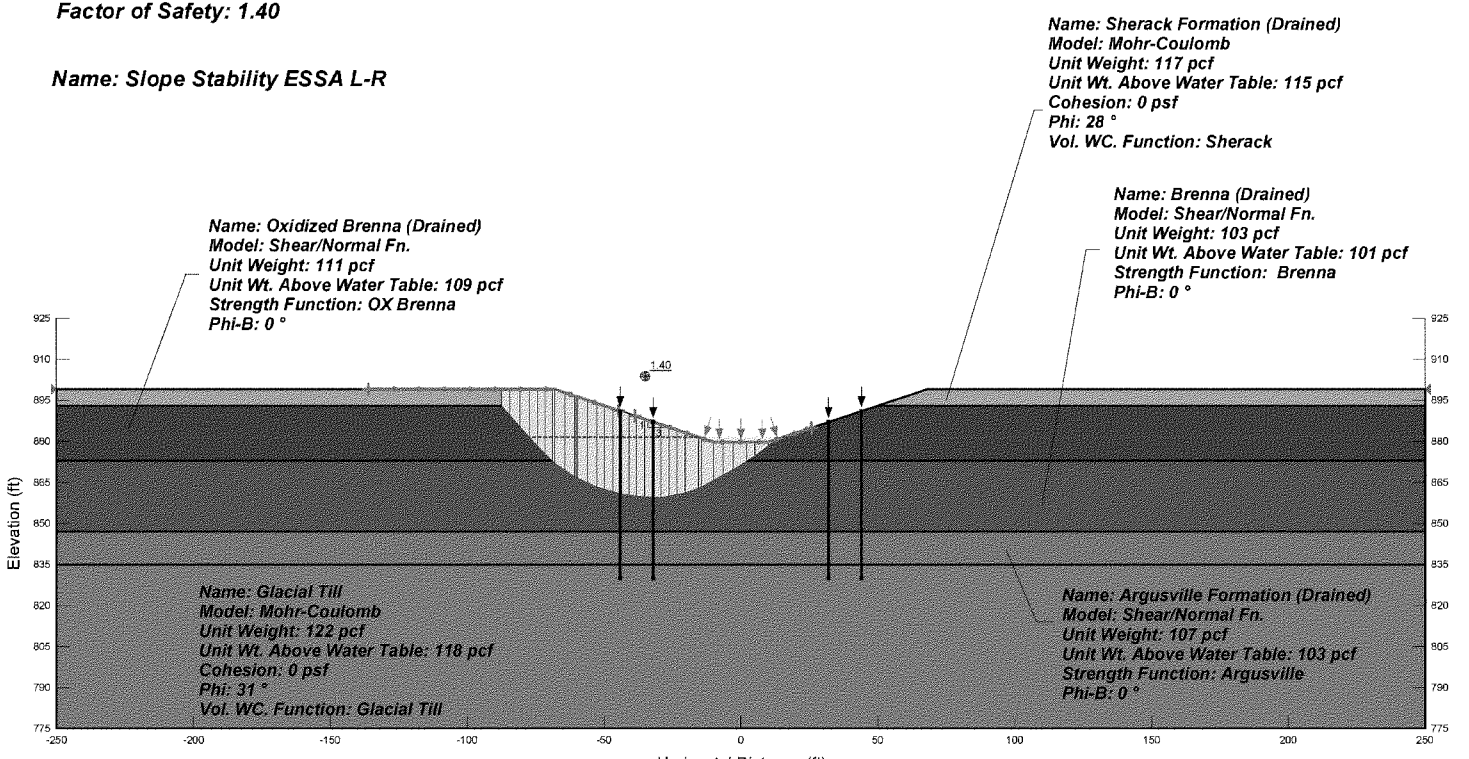
Fargo-Moorhead, Phase 4, Maple River Upstream Channel
Stability Analysis
File Name: Maple River Channel_5to1slope.gsz
Last Saved Date: 2/6/2011
Factor of Safety: 1.50

Name: Slope Stability ESSA L-R



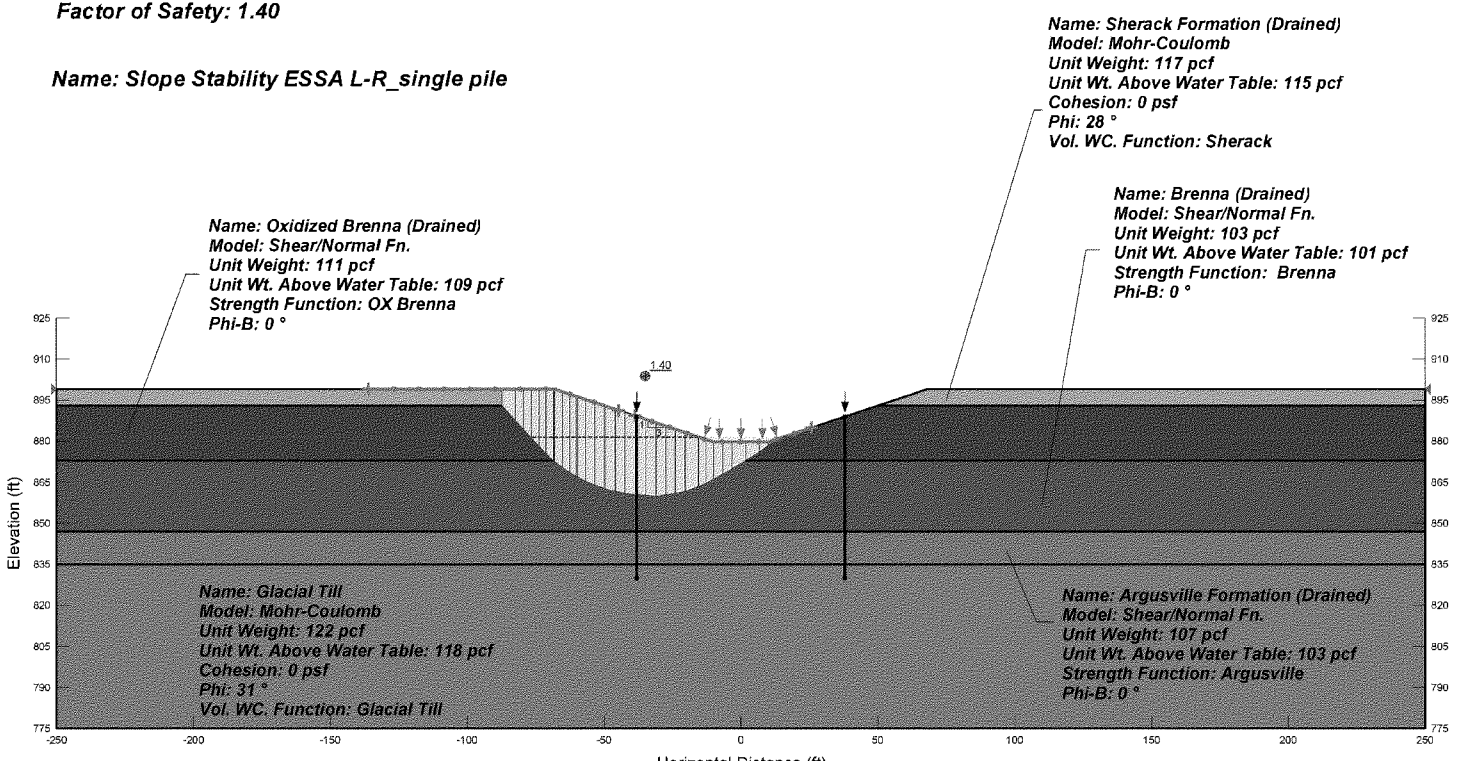
Fargo-Moorhead, Phase 4, Maple River Upstream Channel
Piles: 12ft spacing, 38,000 Shear Force
File Name: Maple River Channel_3to1_piles.gsz
Last Saved Date: 2/4/2011
Factor of Safety: 1.40

Name: Slope Stability ESSA L-R



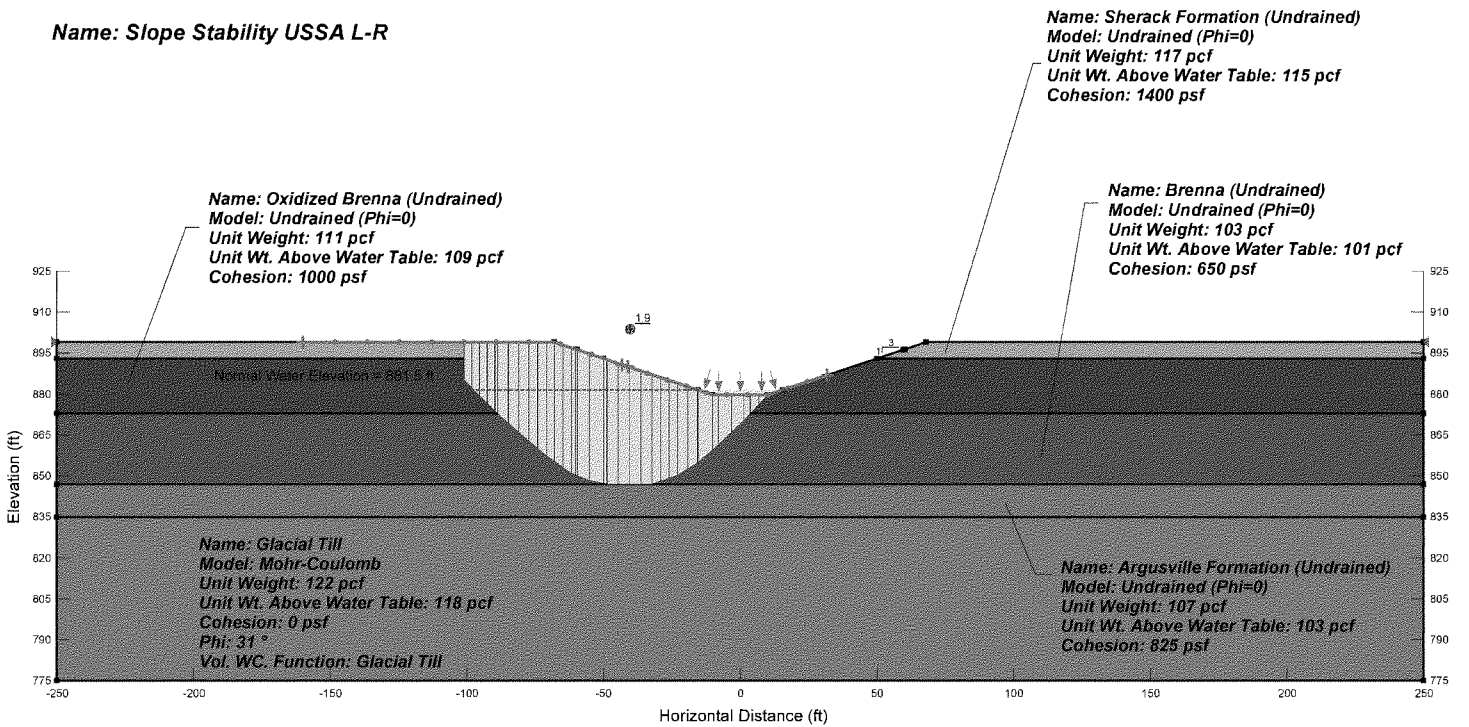
Fargo-Moorhead, Phase 4, Maple River Upstream Channel
Piles: 12ft spacing, 38,000 Shear Force
File Name: Maple River Channel_3to1_piles.gsz
Last Saved Date: 2/7/2011
Factor of Safety: 1.40

Name: Slope Stability ESSA L-R_single pile



Fargo-Moorhead, Phase 4, Maple River Upstream Channel
Stability Analysis
File Name: Maple River Channel_3to1slope.gsz
Last Saved Date: 2/4/2011
Factor of Safety: 1.9

Name: Slope Stability USSA L-R



Fargo-Moorhead Metro Flood Risk Management Project
Phase 4

Last Updated MRM 09Feb2011 **DRAFT**

Tributary Channels Slope Stability Cost Alternatives on a per-linear-foot-basis
Quantities and concept design by JDG, KNA, DMH2
Note: costs are Phase 3 contract costs and do not include contingency, costs are for comparitive purposes only

Engineer's Opinion of Cost

Option 1 - Grade Excavated Side Slopes 5H:1V

Item (per LF channel)	Qty	Unit	Unit Cost	Extended Cost	
Excavation and Spoil (full channel xs)	97.6	BCY	$ 3.49	$ 340.78	5H:1V side slopes, 20' bottom width, 19.3' depth
Total (per LF channel)				$ 340.78	

Option 2 - Grade Excavated Side Slopes 3H:1V, 1); Install two 24" piles each bank, spacing 12 ft apart. The Shear Force per pile is 38,000 lbs

Item (per LF channel)	Qty	Unit	Unit Cost	Extended Cost	
Excavation and Spoil (full channel xs)	70.0	BCY	$ 3.49	$ 244.35	3H:1V side slopes, 20' bottom width, 19.3' depth
Drilled Shaft Piles (38,000 lbs shear force per pile)	19.7	VLF	$ 117.77	$ 2,316.18	2 rows 57 LF and 61 LF of 24" diameter piles (each bank), 1/2" wall thickness, no concrete; spacing 12' o.c.
Total (per LF channel)				$ 2,560.54	

Option 3 - Grade Excavated Side Slopes 3H:1V; Install one 30" pile per bank, 12 ft spacing with a Shear Force per pile of 75,000 lbs

Item (per LF channel)	Qty	Unit	Unit Cost	Extended Cost	
Excavation and Spoil (full channel xs)	70.0	BCY	$ 3.49	$ 244.35	3H:1V side slopes, 20' bottom width, 19.3' depth
Drilled Shaft Piles (75,000 lbs shear force per pile)	9.8	VLF	$ 157.80	$ 1,551.73	1 row 59 LF of 30" diameter piles (each bank), 1/2" wall thickness, no concrete; spacing 12' o.c.
Total (per LF channel)				$ 1,796.08	

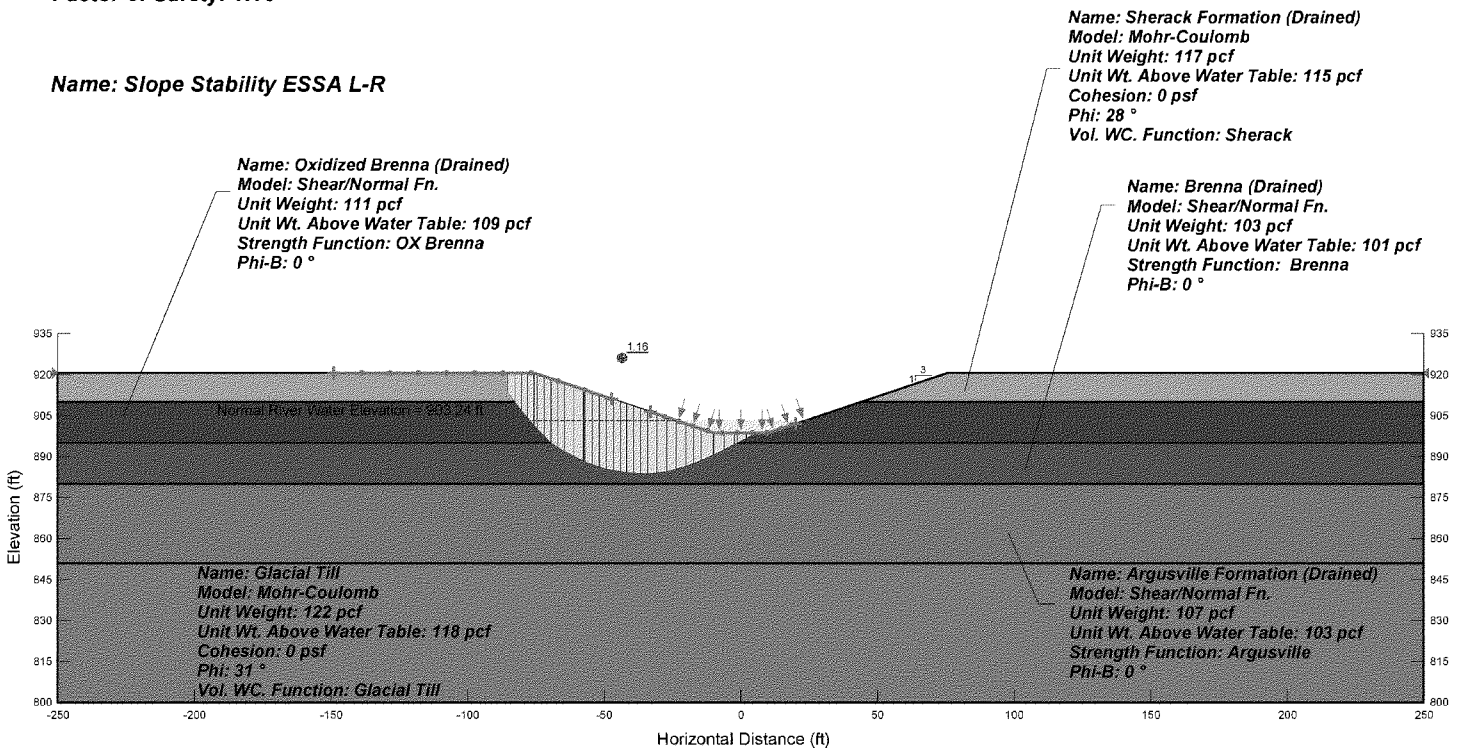
Estimated Unit Cost Calculation

2006		*2010 (x1.13)*		size		
$ 0.55	per vlf	$ 0.62	per vlf	Mobilization	2006 Barr #23-60-0024	
$ 123.00	per vlf	$ 138.99	per vlf	Drilling 48"	2006 Barr #23-60-0024	
$ 143.00	per vlf	$ 161.59	per vlf	Steel pipe piles 48"	2006 Barr #23-60-0024	253.89 lbs/ft
$ 84.00	CY	$ 94.92	CY	Concrete ($84/CY)	2006 Barr #23-60-0024	
$ 4.83	CY	$ 5.46	CY	Load and haul excess ($5/CY)	2006 Barr #23-60-0024	
$ 0.56	lb	$ 0.63	lb	pile cost	2006 Barr #23-60-0024	
		2010	*2010*			
		75k shear (30")	*38k shear 24"*			
		$ 0.62	$ 0.62	Mobilization		
		$ 54.29	$ 34.75	Drilling	xs area ratio	
		$ 101.89	$ 81.50	Steel pipe piles 1/2" wall thickness	161.02 lbs/vlf for 30" and 128.8 lbs/vlf for 24"	
		$ -	$ -	Concrete ($84/CY)		
		$ 0.99	$ 0.90	Load and haul excess ($5/CY)		
		$ 157.80	$ 117.77	Estimated cost per VLF per pile		

Notes:
Barr Engineering #23-60-0024 (Croakston), Cost Quote: 2006 Skyline Steel $142/VLF for 48" diameter, 1/2" wall thickness, delivered (Skyline Steel, $1,120/ton, 253.89 lb/ft)
Cost estimate 2006 from P:\Mpls\23 MN\60\2360024_MovedFromMpls_P\ $2.1 Million for 6510 LF, or $322/LF for 48" dia. Concrete filled shaft piles
Time Factor 2010/2006 = 1.13 RS Means

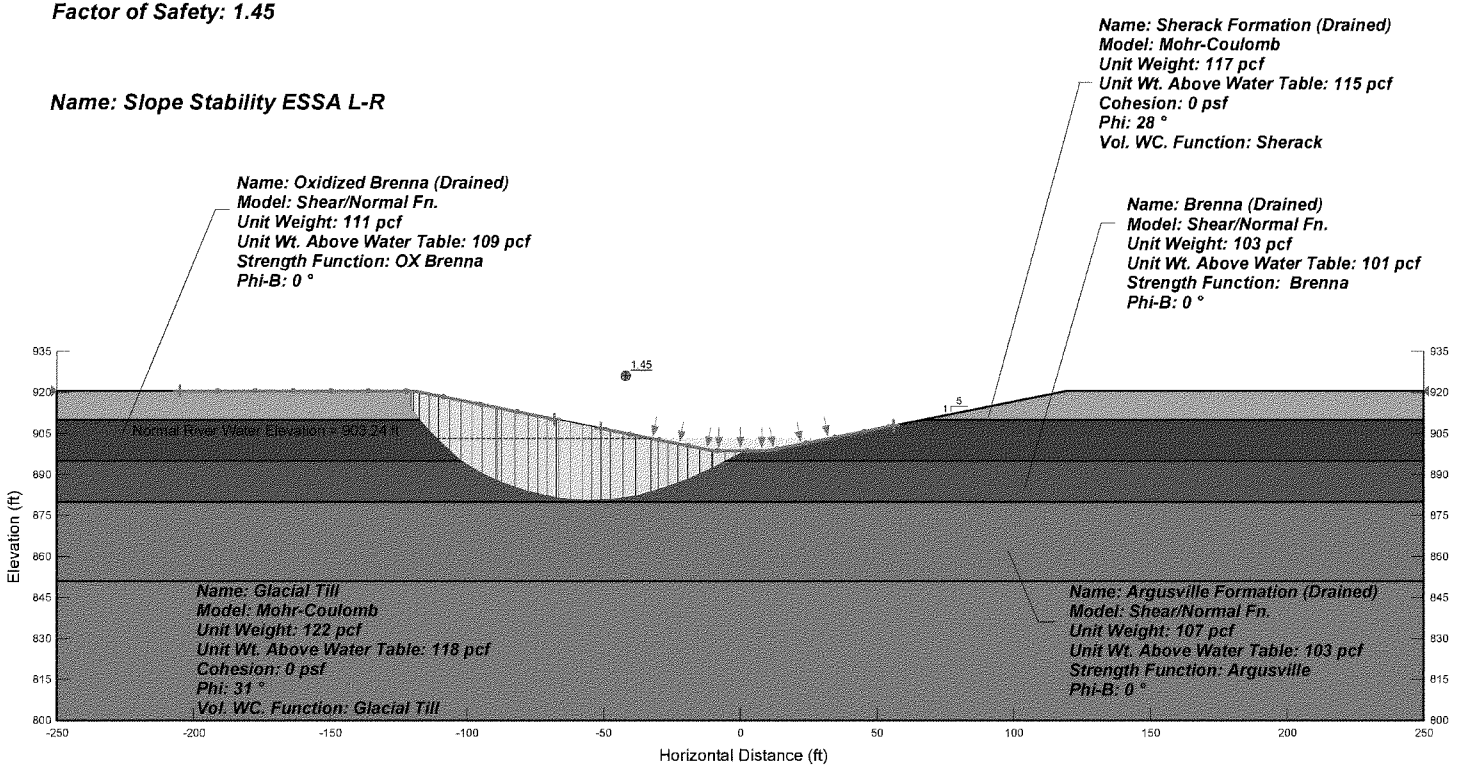
Fargo-Moorhead, Phase 4, Sheyenne River Upstream Channel
Stability Analysis
File Name: Sheyenne River Channel_3to1.gsz
Last Saved Date: 2/6/2011
Factor of Safety: 1.16

Name: Slope Stability ESSA L-R



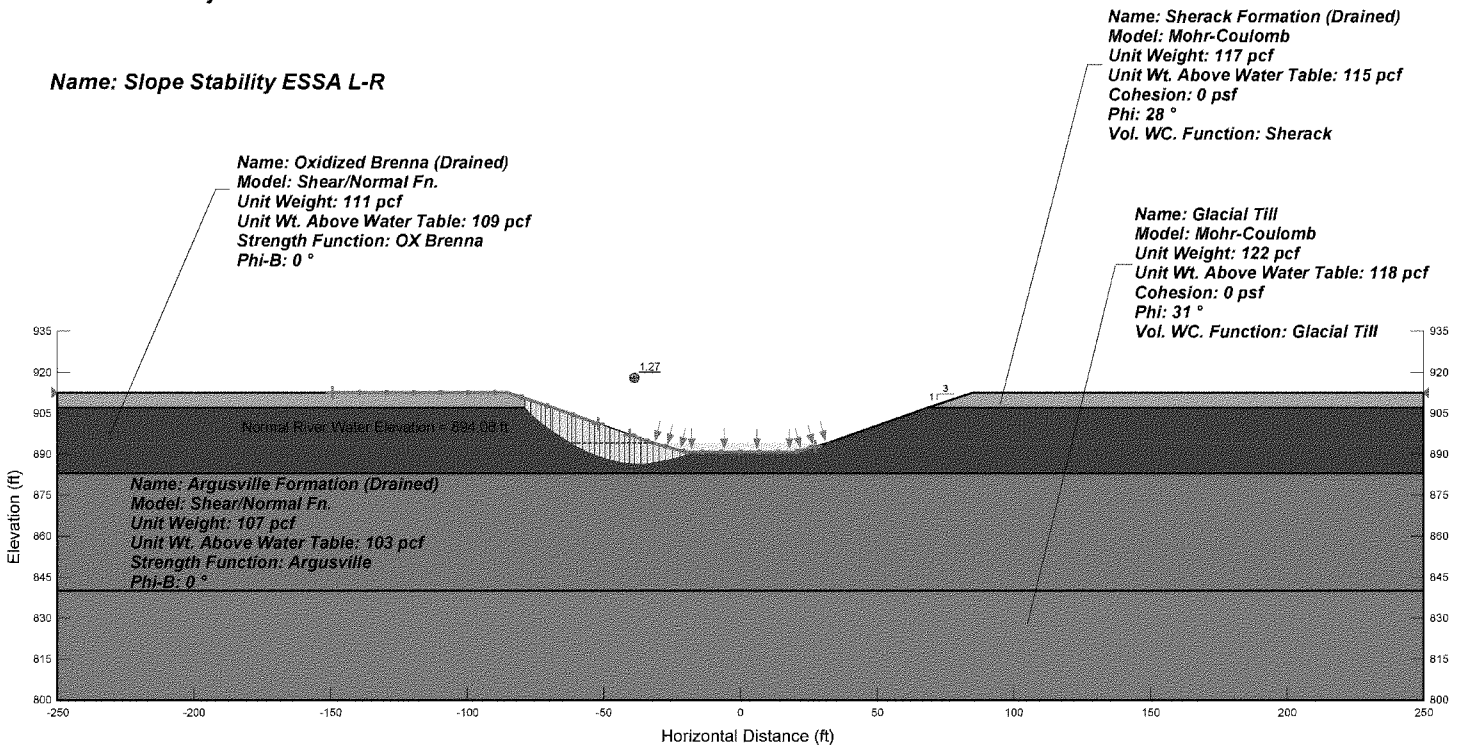
Fargo-Moorhead, Phase 4, Sheyenne River Upstream Channel
Stability Analysis
File Name: Sheyenne River Channel_5to1.gsz
Last Saved Date: 2/6/2011
Factor of Safety: 1.45

Name: Slope Stability ESSA L-R



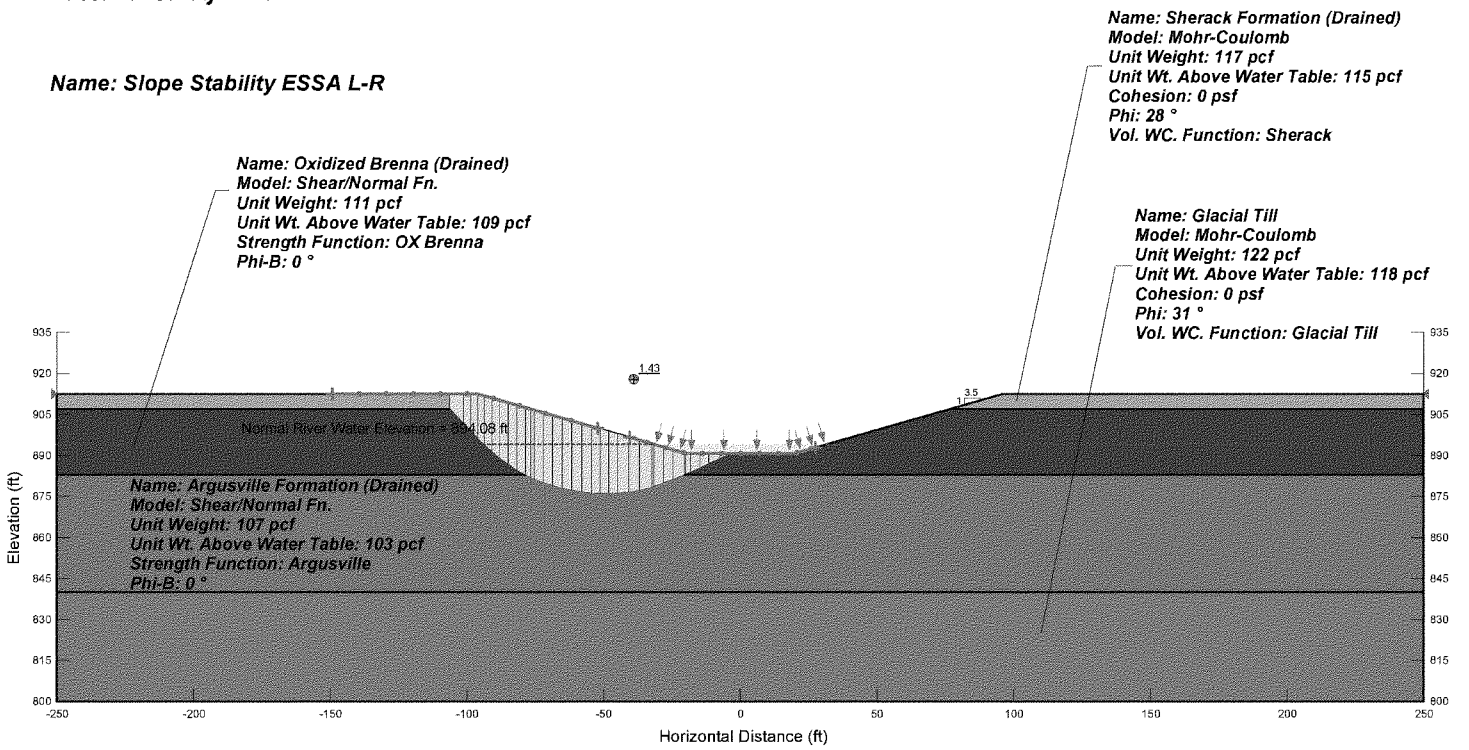
Fargo-Moorhead, Phase 4, Wild Rice River Upstream Channel
Stability Analysis
File Name: Wild Rice River Channel_3to1.gsz
Last Saved Date: 2/6/2011
Factor of Safety: 1.27

Name: Slope Stability ESSA L-R



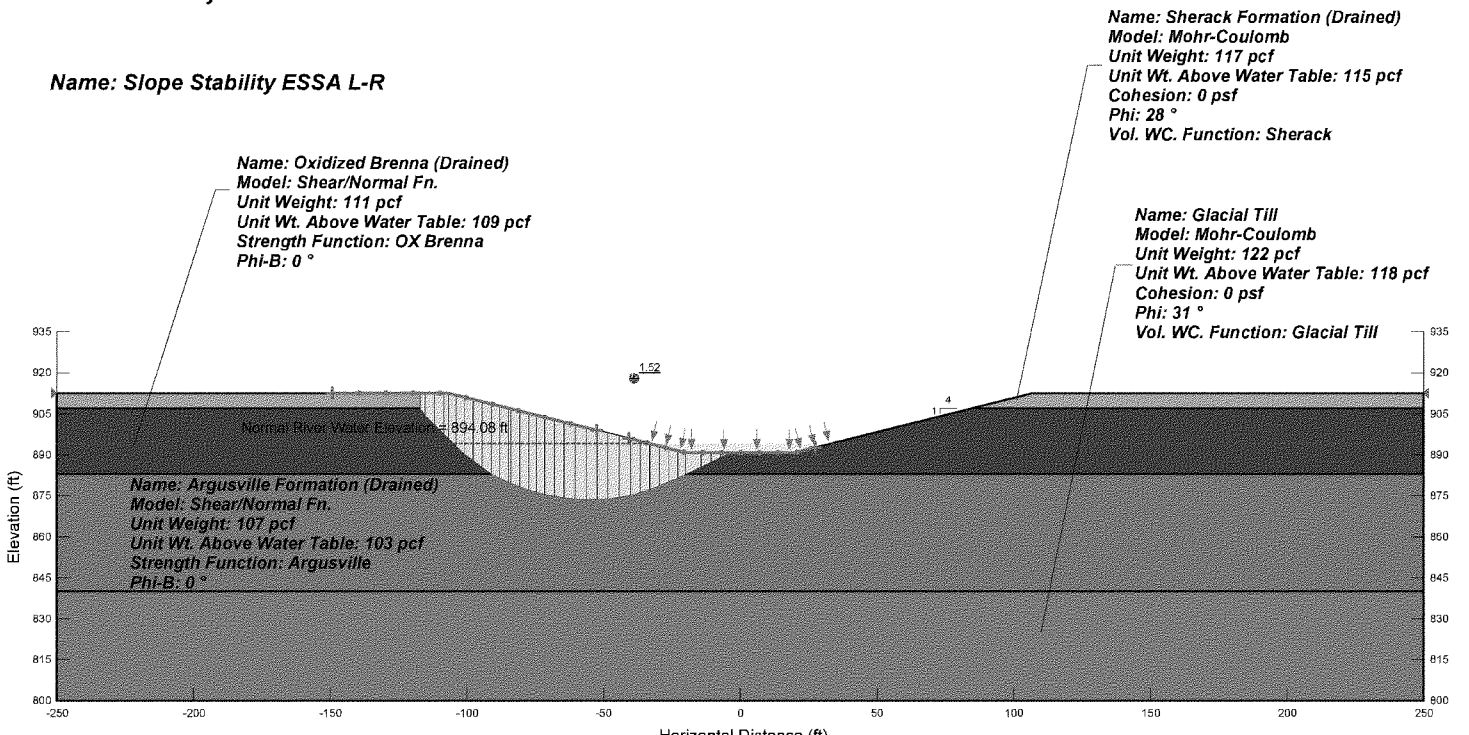
Fargo-Moorhead, Phase 4, Wild Rice River Upstream Channel
Stability Analysis
File Name: Wild Rice River Channel_3.5to1.gsz
Last Saved Date: 2/6/2011
Factor of Safety: 1.43

Name: Slope Stability ESSA L-R



Fargo-Moorhead, Phase 4, Wild Rice River Upstream Channel
Stability Analysis
File Name: Wild Rice River Channel_4to1.gsz
Last Saved Date: 2/6/2011
Factor of Safety: 1.52

Name: Slope Stability ESSA L-R



Fargo-Moorhead, Phase 4, Red River Upstream Channel Stability Analysis
File Name: Red River Channel_3to1_UPDATED.gsz
Last Saved Date: 2/27/2011
Factor of Safety: 0.99

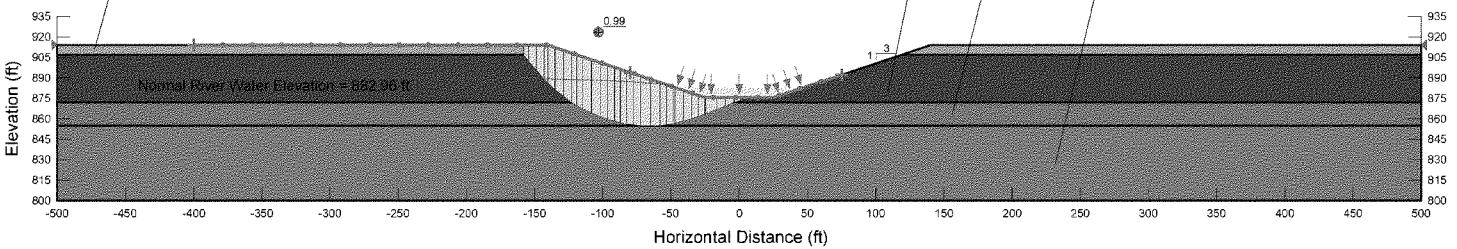
Name: Slope Stability ESSA L-R

Name: Oxidized Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Strength Function: OX Brenna
Phi-B: 0 °

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville
Phi-B: 0 °

Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack_Formation

Name: Glacial Till
Model: Bedrock (Impenetrable)



Fargo-Moorhead, Phase 4, Red River Upstream Channel Stability Analysis
File Name: Red River Channel_7to1_UPDATED.gsz
Last Saved Date: 2/26/2011
Factor of Safety: 1.62

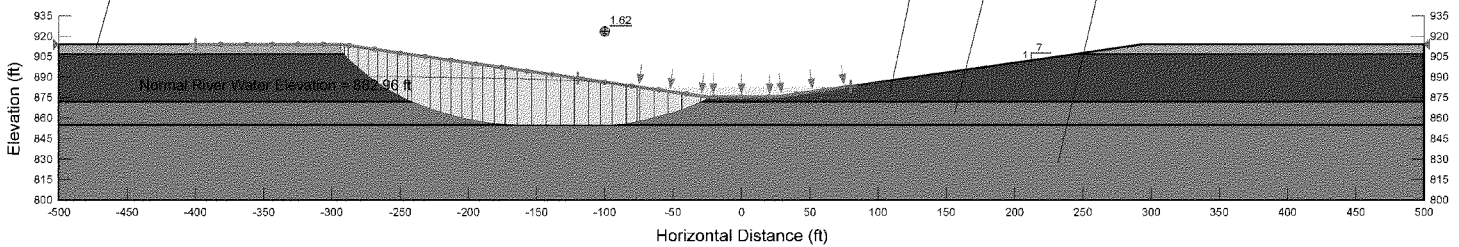
Name: Slope Stability ESSA L-R

Name: Oxidized Brenna (Drained)
Model: Shear/Normal Fn.
Unit Weight: 111 pcf
Unit Wt. Above Water Table: 109 pcf
Strength Function: OX Brenna
Phi-B: 0 °

Name: Argusville Formation (Drained)
Model: Shear/Normal Fn.
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 103 pcf
Strength Function: Argusville
Phi-B: 0 °

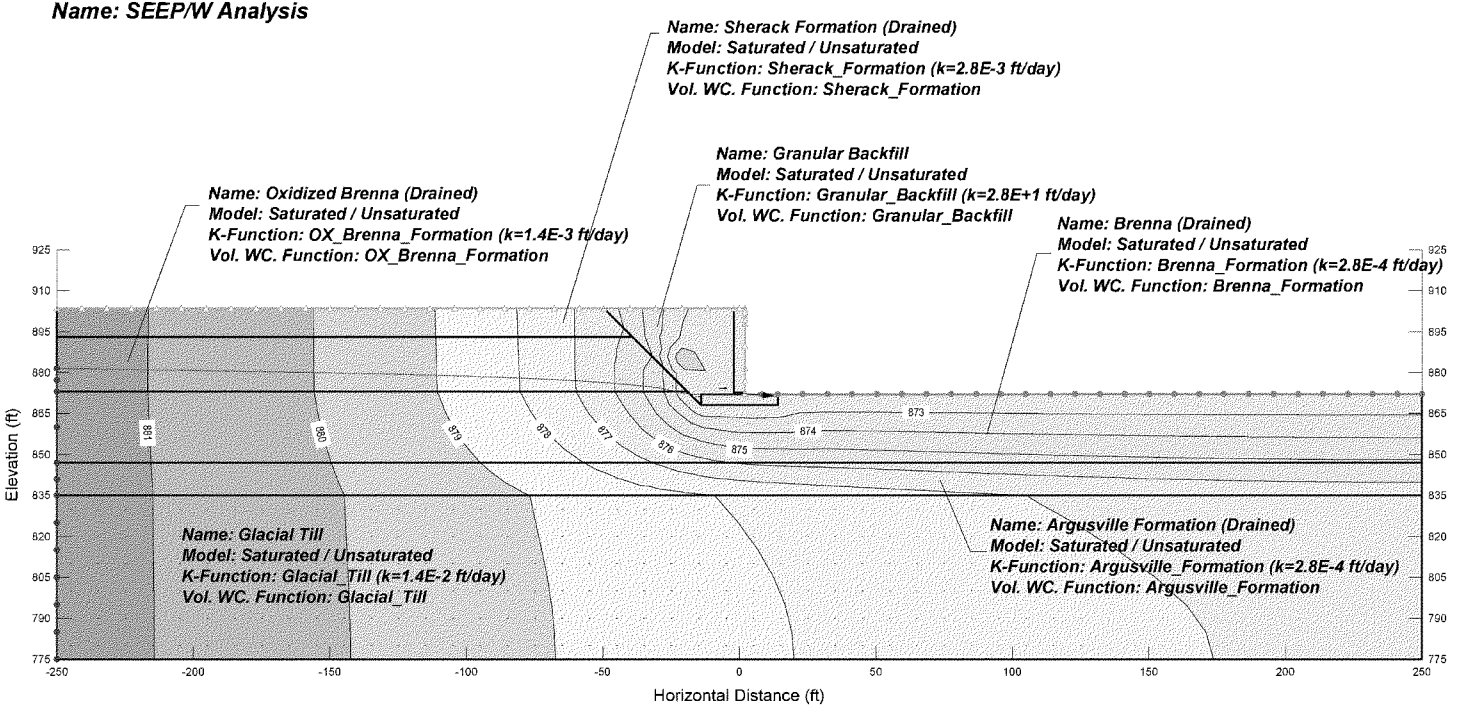
Name: Sherack Formation (Drained)
Model: Mohr-Coulomb
Unit Weight: 117 pcf
Unit Wt. Above Water Table: 115 pcf
Cohesion: 0 psf
Phi: 28 °
Vol. WC. Function: Sherack_Formation

Name: Glacial Till
Model: Bedrock (Impenetrable)



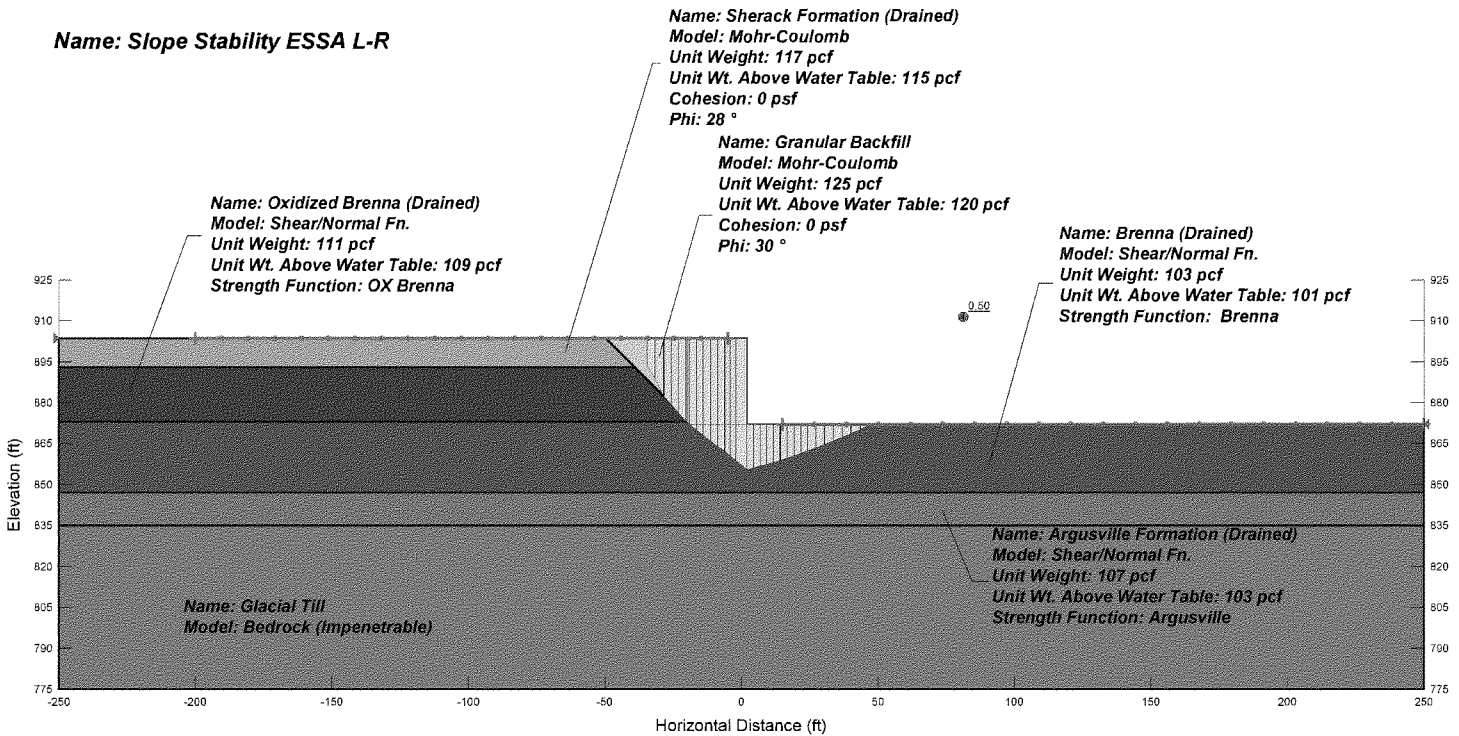
Fargo-Moorhead, Phase 4, Maple River Wall
Stability Analysis
File Name: Maple River Wall_UPDATED.gsz
Last Saved Date: 2/26/2011

Name: SEEP/W Analysis



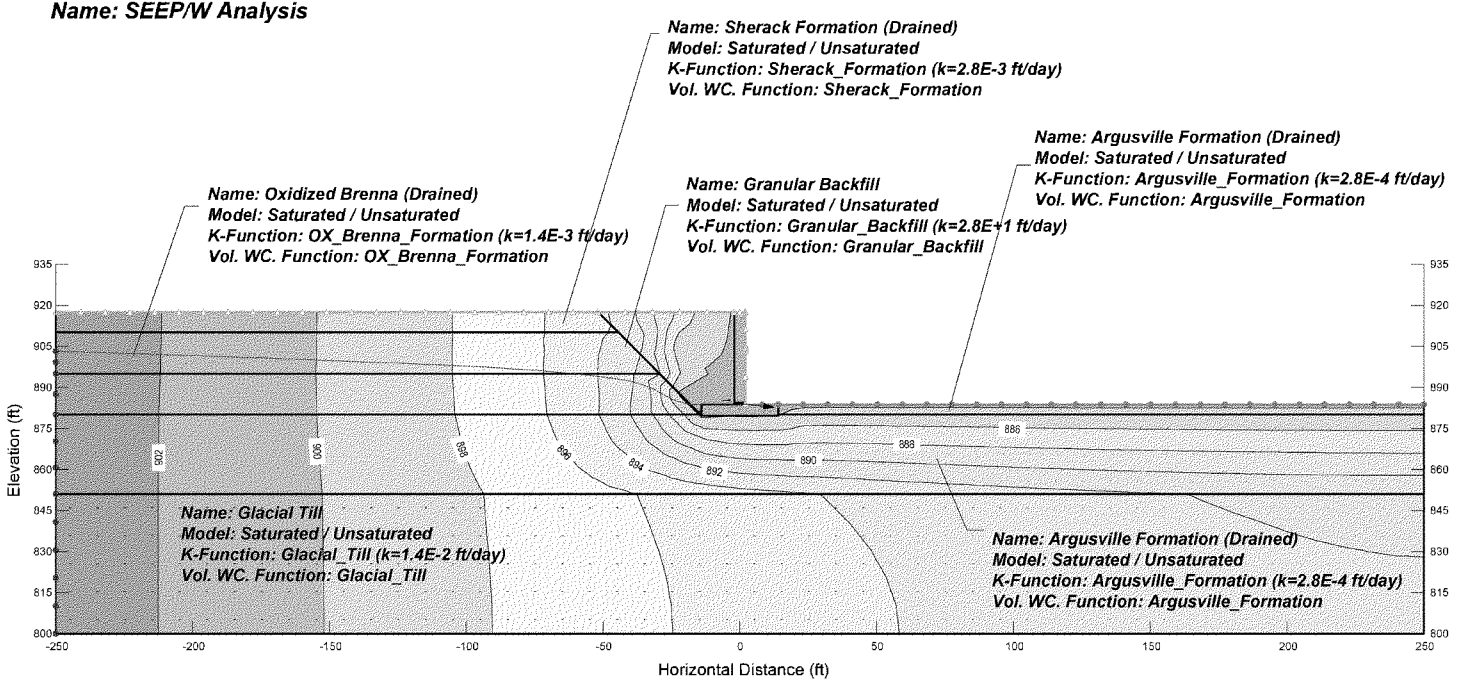
Fargo-Moorhead, Phase 4, Maple River Wall Stability Analysis
File Name: Maple River Wall_UPDATED.gsz
Last Saved Date: 2/26/2011
Factor of Safety: 0.50

Name: Slope Stability ESSA L-R



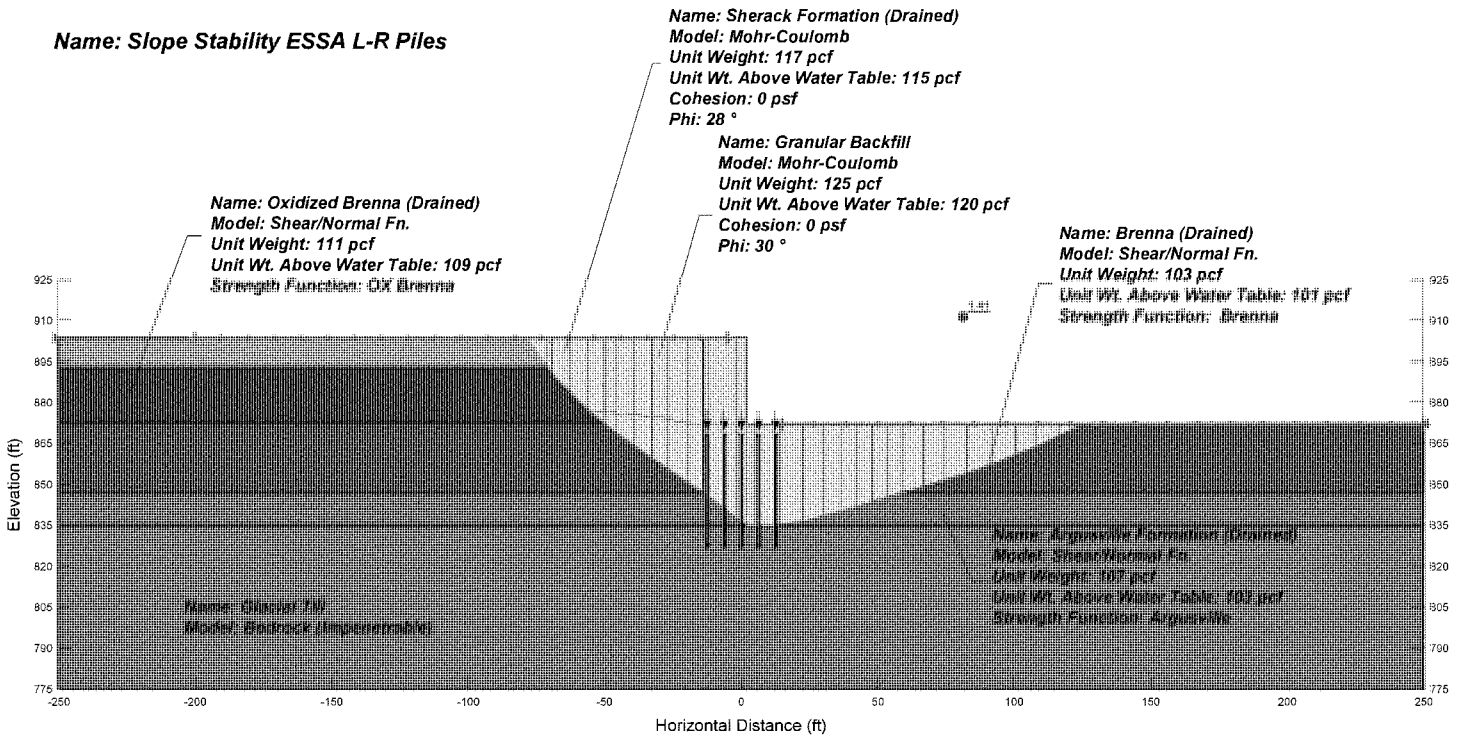
Fargo-Moorhead, Phase 4, Sheyenne River Wall
Stability Analysis
File Name: Sheyenne River Wall_UPDATED.gsz
Last Saved Date: 2/26/2011

Name: SEEP/W Analysis



Fargo-Moorhead, Phase 4, Maple River Wall Stability Analysis
File Name: Maple River Wall_UPDATED.gsz
Last Saved Date: 2/26/2011
Factor of Safety: 1.41

Name: Slope Stability ESSA L-R Piles



Fargo-Moorhead, Phase 4, Sheyenne River Wall
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Factor of Safety: 0.55

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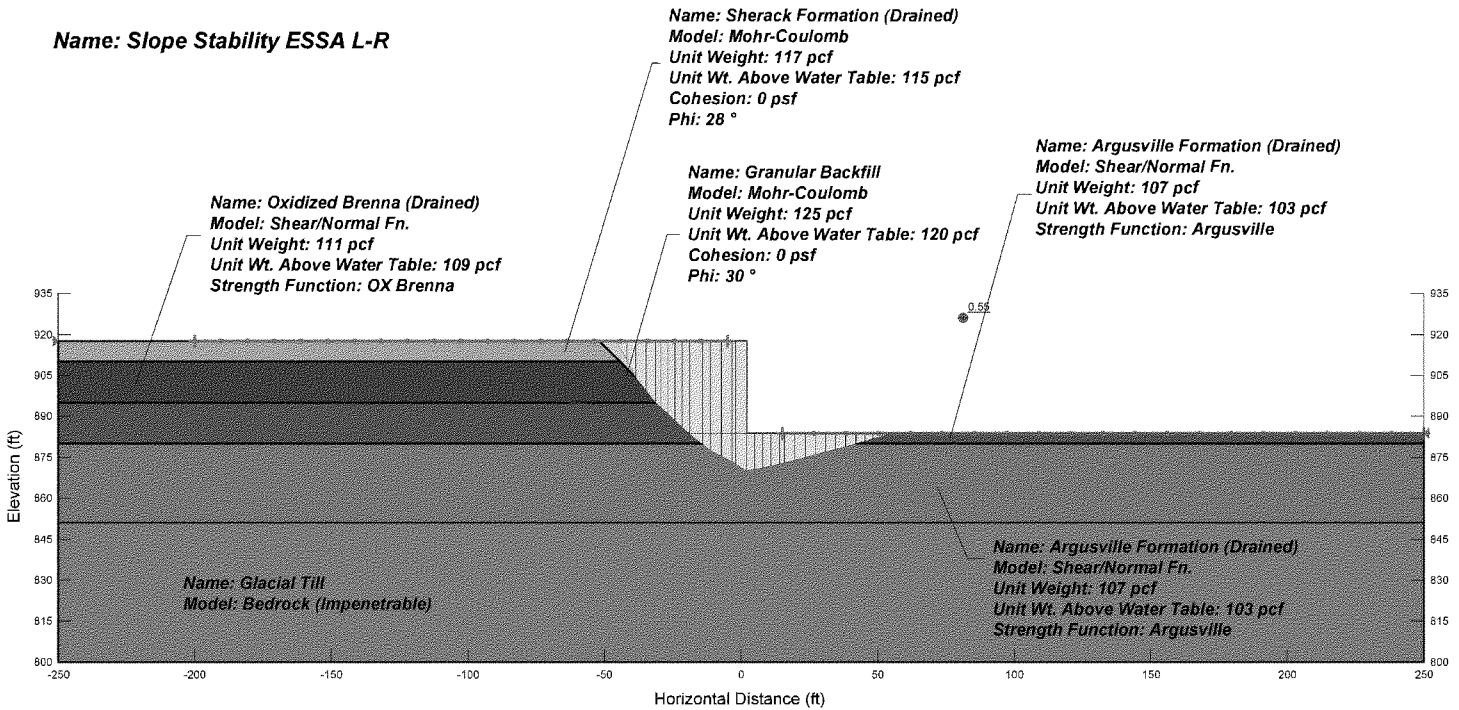

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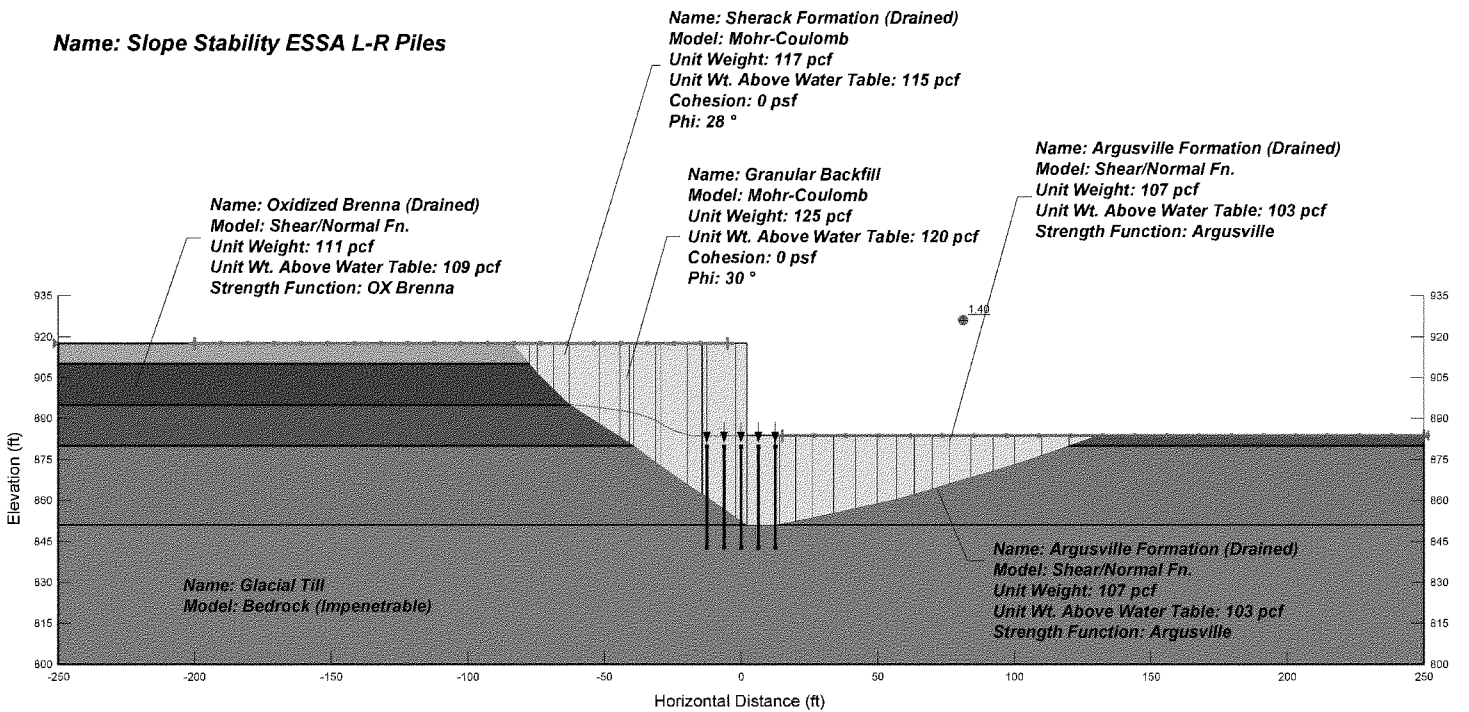

RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT O – PILE FOUNDATIONS

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

By: Barr Engineering Co.

FINAL – February 28, 2010

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT O – PILE FOUNDATIONS

F-O1.0 INTRODUCTION TO PILE CAPACITY ANALYSES

Deep foundations are required for support of the water control and aqueduct structures along the proposed project alignments. The structures and location along the project alignments are shown in Table F-O1. Each of the proposed structures is situated with a base foundation elevation that is at or very near the weak, compressible Brenna or Argusville formations which are incapable of supporting the heavy concrete structures. The intent of the deep foundations is to eliminate the foundation bearing loads on the weaker clays and transfer the load through the clays into a glacial till bearing layer. It is worthwhile acknowledging, however, that the pile will still develop side-resistance capacity in the clay layers. For this project, HP14X73 H-piles were used and the structural evaluation will be discussed in Appendix F.

Table F-O1 Structures Requiring Pile Foundations

Structure	Station Location
Red River Control Structure (FCP) MN Foundations and Wing walls	MN 128703
Red River Control Structure (LPP) ND Foundations and Wing walls	RRN 2529023
Wild Rice River Control Structure	WRR 66102
Sheyenne River Aqueduct Crossing Foundations	150467
Maple River Aqueduct Crossing Foundations	72543
Lower Rush (vertical drop)	60755
Rush (vertical drop)	45110
Inlet Ogee	152522
Outlet Ogee	363
Wolverton	WC 9079.5
Storage Area Outlet	WRR 75000

F-O2.0 REGIONAL FOUNDATION EXPERIENCE

Deep foundations are routinely used in this region of the country to support heavily-loaded structures due to the deeper deposits of medium to stiff yet compressible, Brenna and Argusville clay formations. Historically, heavily-loaded structures founded on shallow foundations bearing on or near these layers have performed poorly due to shear failures and long-term settlement. Because each of the proposed structures is situated with a base elevation that is at or near the weak, compressible layers formation, poor performance would result.

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 axial allowable pile capacities, used by governmental transportation agencies in the area and other organizations, generally range from 60 to 100 tons per pile depending on length and size of pile. All piles used in this region are supported by the glacial till layer which exists below the Brenna and Argusville formations at depths ranging from 80 to 100 feet or more below the existing ground surface. The glacial till layer is very dense or hard and it has been found during investigations conducted by the US Army Corps of Engineers and through other information compiled from the Minnesota and North Dakota Departments of Transportation, that the Standard Penetration (SPT) N-values are greater than 50 blows per foot and even higher values of 100 blows per foot have been reported. Historically, sampling in the layer for laboratory testing has not been performed other than SPT samples so there is no documented strength data on the 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. Pile penetrations in the glacial till are reportedly a maximum of 5 to 7 feet before meeting refusal. Deeper penetrations are a result of slightly weathered glacial till layers that are found above the very dense or hard layer. This indicates that the glacial till has a very high undocumented strength and the use of the undrained pile capacity based on the undrained strength provided by the US Army Corps of Engineers is conservative. Typical axial pile capacities range from 60 tons to greater than 100 tons for 9-5/8-inch to 12-3/4-inch pipe-piles or HP12X60 to HP14X89 H-piles. Often, pile designers use the structural capacity of the pile element itself when founded on the till.

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 alignments. For structural reasons, an HP14X73 was used in the analysis. This decision to use the HP14X73 is discussed in Appendix F and it is consistent with many foundations constructed in the region.

F-O3.0 GEOLOGY

The Fargo-Moorhead area is covered by 200 to 300 feet of clays, tills, and granular sediments from past glaciation associated with the formation of glacial Lake Agassiz.

The Sherack Formation, Brenna Formation, Argusville Formation, and glacial till are the primary soil units in the area, though other more localized soil units have been identified such as the Poplar River Formation, Plastic Laminated Sherack Formation, and Oxidized Brenna Formation. The material types of most interest for the design of the deep foundations are the Brenna and Argusville Formations, which are relatively medium to stiff and highly plastic materials. The clay was deposited after an additional glacial event, and it is characterized as a lacustrine clay but it was altered at some locations during lower lake levels where erosion channels were filled with sediments. Due to the depth of the foundations, the Sherack and other surficial layers are of little importance. The deeper glacial till layer encountered below the Brenna and Argusville is hard to dense, depending on the constituents, and it will provide the bearing resistance for the piles. It should be noted that although the Brenna and Argusville are relatively weak compared to the glacial till layer, they are overconsolidated meaning that each layer has experienced a pressure greater than the existing overburden pressure. Overconsolidation can be caused by changes in groundwater levels and erosion of surficial soils among other mechanisms. Overconsolidation tends to affect the strength and behavior of soils in regard to foundation design and settlement.

F-O4.0 PILE CAPACITY ANALYSIS

An analysis of driven pile foundations was conducted at each structure to determine the appropriate pile length and capacity for structural design.

F-O4.1 METHODOLOGY

The main objective of the pile capacity analysis was to evaluate the axial and lateral capacity and settlement of pile foundations at each of the structures. The engineering manual, EM 1110-2-2906 "Design of Pile Foundations" (15 Jan 1991), was used as guidance for estimation of axial pile capacities. Although the manual is outdated (with regard to certain current practices) and other design methods and manuals are available, the methodology presented in this manual provided enough relevant guidance to complete the analysis. The manual requires that for the calculation of axial and lateral pile capacity both the drained and undrained strengths are used. The governing condition is then used for each case to determine the pile spacing and layout during structural design.

The undrained or short-term analysis 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 load application. This scenario typically applies to loading from transient conditions such as floods where the load is applied for a short period of time relative to the time that takes excess pore-water pressure dissipation which is related to the permeability of the soils.

The drained, effective stress, or long-term analysis 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 shear induced pore-water pressures are not generated. The dead load of the structure is considered a long-term case.

Similar to the axial analysis, the drained and undrained lateral capacity of the piles were also computed for use in resisting the lateral design loads. The capacity was determined for the piles at three acceptable displacements which were established as the lateral movement criteria. The criteria are 0.5 inches, 0.67, and 0.875 inches of lateral movement. The criteria were developed based on structural requirements discussed in other sections of Appendix F.

The pile settlement under working loads was also estimated at each of the structures for a single pile. For the detailed design, it is recommended that the capacity of the piles be adjusted to account for group effects.

F-O4.2 SOFTWARE

Three different design software packages were used to facilitate the calculation of pile capacities and settlement. Each software and calculation methods are discussed in the following sections.

AllPile

The program, AllPile version 7 analyzes pile load capacity efficiently and accurately. AllPile can handle all types of piles: drilled shaft, driven pile, augercast pile, steel pipe pile, H-pile, timber pile, tapered pile, bell pile, shallow foundation, etc. The program can define new pile types and use customized input parameters based on local practices and experience. The program is capable of performing calculations for lateral capacity and deflection, axial capacity and settlement. The lateral capacity calculation uses COM624S, which is the same method as FHWA's COM624P and is comparable with Ensoft's Lpile. The settlement is calculated using the Vesic methodology.

APILE

The APILE software, developed by Ensoft Inc., is used to compute the axial capacity, as a function of depth, of a driven pile in clay, sand, or mixed-soil profiles. The main computational methods used by *APILE* are those established by the American Petroleum Institute (API) in their manual on recommended practice, API-RP2A. The procedures from API have been adopted by a number of organizations.

In addition to computations based on API-RP2A criteria, APILE offers alternative methods for computing the axial-capacity of driven piles, including: (i) U.S. Army Corps of Engineers (USACE) method, (ii) U.S. Federal Highway Administration (FHWA) method, and (iii) the Lambda-Method.

The APILE program also provides some flexibility in allowing users to specify any set of values for load transfer in side resistance and end bearing as a function of depth. This feature is useful for cases when site measurements were made from instrumented axial load tests.

LPILE

The LPILE Version 5 software, developed by Ensoft Inc., is used to compute the lateral capacity of pile foundations in varying stratigraphy. LPILE uses the p-y method to

calculate the shear, moment, and deformation of the pile under various load conditions. LPILE is an updated version of lateral pile analysis compared to AllPile which uses COM624.

F-O4.3 DESIGN STRENGTH PARAMETERS

Geotechnical data and a summary of strength parameters for each soil layer were provided by the St. Paul District of the US Army Corps of Engineers. The Corps has completed a preliminary geotechnical investigation program along the project alignments to evaluate the strength of the soils for slope stability modeling. These strength parameters vary significantly from the parameters required for pile capacity analyses. The soil shear strength parameters (failure envelopes) used in slope stability analyses were defined using the large strain failure criterion to incorporate effect of softening phenomenon applicable to slope stability problems. The pile capacity analyses methods are based on the use of peak strengths for the drained and undrained strength parameters and therefore a reanalysis of the geotechnical data was required. However, the unit weight of the soils provided by the Corps for the stability analyses were considered adequate for the purposes of the pile analyses. The revised design parameters are summarized in Exhibit L of Appendix F.

F-O4.3.1 Peak Undrained Strength Parameters

The peak undrained strength of the Brenna and Argusville plays an important role in the pile capacity calculations because these formations occur along the entire side of the pile and provide significant side resistance. Therefore, it was necessary to develop the peak parameters from the soil test results performed by the USACE. The process to evaluate the strength has been discussed in Exhibit L of Appendix F and the results presented indicated that at the depths of the pile foundation, the undrained shear strength of the Brenna formation is about 900 psf. The strength of the Argusville formation was slightly higher at 950 psf. Other soil layers encountered less frequently were the Sherack and the West Fargo. These parameters were also estimated based on the strength results by the USACE and are shown in Table F-O2.

The peak undrained shear strength of the glacial till layer was more difficult to define due to the lack of strength testing of this material. Standard penetration tests indicate the material is generally medium dense to very dense or hard with N-values ranging from 30 to over 100 blows per foot. The N-values relate to an undrained shear strength of about 7500 psf. Slightly lower strength values were observed in the upper, weather glacial till layers and are accounted for by a relationship with N-values. Table F-O2 presents the design undrained strength parameters for the preliminary pile capacity analyses.

F-O4.3.2 Peak Drained Strength Parameters

The analysis of long-term or drained pile capacity requires the friction angle of the soil material and the use of the "sand" design methodology. Therefore, the friction angles of the Brenna and Argusville formations are important to the evaluation of pile capacity for the long-term condition. The analysis of these two layers was discussed previously in Exhibit L of Appendix F. The glacial till layer is represented by friction angles from actual laboratory tests. The strength of the till is expected to vary based on the degree of

weathering and was modeled using two till layers. The first is the top layer that simulates the weathered till (about 5 ft thick) and the second is the non-weathered till underneath it. The results of the analysis and the design parameters in provided in Table F-O2.

Table F-O2 Design Soil Strengths

Fargo Moorhead Flood Control Preliminary Pile Design Parameters			
Formation	**Unit Weight**	**Selected Shear Strength Parameters**	
	γ_m (pcf)	**Effective Friction Angle** ϕ'	**Undrained Shear Strength Su (psf)**
Sherack	117.6	30	1000
West Fargo Poplar River	123.2	30	3000
Oxidized Brenna	111.3	25	1000
Brenna	104.1	25	800~900
Argusville	106.5	26	950
Weathered Till	130	31	6000[(1)]
Till	130	34	7500

(1) Weathered till surficial layer generally 5 feet thick.

Table F-O3 Lateral Pile Design Parameters

Fargo Moorhead Flood Control Lateral Pile Design Parameters			
Formation	**Undrained**		**Drained**
	c (psi)	ε_{50} **(%)**	**k (pci)**
Sherack	6.9	0.01	60
West Fargo Poplar River	6.9	0.01	60
Oxidized Brenna and Brenna	5.55-6.9	0.01	20
Argusville	6.6	0.01	20
Till	41.7-52	0.004	125

F-O4.3.3 Properties for Lateral Loading Analyses

Both the drained and undrained soil properties were estimated for the lateral pile analyses using documentation provided in the AllPile 7 software manual or LPile Version 5. The

correlations for the undrained lateral load parameters were based on undrained shear strength of the soils. For the drained strength parameters, correlations between N-values and friction angle were used to determine the moduli.

F-O5.0 SITE CONDITIONS

As discussed in section F-O3.0, there are about three to four soil layers at each site. For the purpose of the deep foundations analyses each site was reviewed along the project profile to determine the individual soil layers that exist below the foundation level. Generally, the foundation levels of the structures are at or very near the top of the Brenna layer except in one case where the Argusville was encountered at the foundation level. Where found at each of the locations, the Brenna is about 10 to 30 feet thick. Where the Argusville exists, it is about 18 feet thick.

F-O6.0 RESULTS

The results of the axial pile capacity calculations are summarized in Table F-O4. Table F-O4 includes the ultimate axial pile capacity and skin friction under drained and undrained conditions for individual piles. These pile capacities were computed using the design methodology previously described. Pile capacity calculations under undrained conditions used the clay-type soil modeling in the software. Pile capacity calculations under drained conditions used the sand-type soil modeling in the computational software.

The results of the axial pile capacity analyses are shown in Table F-O4. The table shows the range in capacity for each of the pile capacity analysis methods. Considerable variation is observed in the results as expected due to the differences in calculation methodology utilized in each analysis method. At each structure the axial pile capacity was computed and reported. The design pile capacity was calculated by computing the average between the maximum and minimum capacity values computed from the four calculation methods. This reporting procedure for the pile capacity was requested by the USACE and was used to reduce the sometimes large range difference in estimated/computed pile capacities for this preliminary evaluation.

Table F-O4 Summary of Individual Pile Capacities by Method

		Ultimate Drained Capacity[1]	Skin Friction Drained	Ultimate Undrained Capacity[1]	Skin Friction Undrained
	Analysis Method	**(kips)**	**(kips)**	**(kips)**	**(kips)**
Phase 3 Analysis RRN ND	FHWA (Apile)	137.6	36.1	268.7	175.5
	USACE (Apile)	95.5	31	267.7	174.5
2' into hard till	Lambda 2 (Apile)	157.5	40.8	213.7	120.5
	API RP-2A (Apile)	158.9	42.2	209.5	127.7
	Navfac (Allpile)	120.2		237.4	
	Average (2)	**127.2**	**36.6**	**239.1**	**148**
Phase 4 Analysis RRN ND	FHWA (Apile)	125.8	35.9	267.8	174.6
	USACE (Apile)	86.7	30.7	274.2	181.0
	Lambda 2 (Apile)	163.0	41	212.8	119.6
	API RP-2A (Apile)	164.3	42.4	221.3	128.1
	Average (2)	**125.5**	**36.6**	**243.5**	**150.3**
Phase 3 Analysis Wild Rice	FHWA (Apile)	206.5	66.4	339.6	220.5
	USACE (Apile)	107.5	43.1	321.3	196.7
1' into hard till	Lambda 2 (Apile)	227	66	252.7	128.9
	API RP-2A (Apile)	242.2	81.3	265.6	172.4
	Navfac (Allpile)	153.4		283.6	
	Average (2)	**174.85**	**62.2**	**296.15**	**174.7**
Phase 4 Analysis Wild Rice	FHWA (Apile)	173.8	71.8	349.9	256.7
	USACE (Apile)	101.1	45.1	349.4	256.2
	Lambda 2 (Apile)	237.6	67.2	269.0	175.8
	API RP-2A (Apile)	255.9	85.5	290.0	196.8
	Average (2)	**178.5**	**65.3**	**309.5**	**216.3**
Phase 3 Analysis Sheyenne	FHWA (Apile)	165.5	49.3	296.2	203.1
	USACE (Apile)	101.5	37	260.8	167.6
3' into hard till	Lambda 2 (Apile)	183.4	49.9	199.3	106.1
	API RP-2A (Apile)	190.7	57.2	209.7	116.6
	Navfac (Allpile)	131.4		234.9	

	Analysis Method	Ultimate Drained Capacity[1] (kips)	Skin Friction Drained (kips)	Ultimate Undrained Capacity[1] (kips)	Skin Friction Undrained (kips)
	Average (2)	**146.1**	**47.1**	**247.75**	**154.6**
Phase 4 Analysis Sheyenne	FHWA (Apile)	145	45.5	287.1	193.9
	USACE (Apile)	90.6	34.6	291.3	198.1
	Lambda 2 (Apile)	185.5	49.4	220.0	126.8
	API RP-2A (Apile)	191.1	54.8	234.1	140.9
	Average (2)	**140.9**	**44.7**	**255.7**	**169.5**
Phase 3 Analysis Maple River	FHWA (Apile)	133.3	35	261.3	168.1
	USACE (Apile)	97.8	33.3	252	158.8
5' into hard till	Lambda 2 (Apile)	153.1	40.1	196.1	102.9
	API RP-2A (Apile)	153.6	40.7	202.5	109.3
	Navfac (Allpile)	120.2		228	
	Average (2)	**125.7**	**37**	**228.7**	**135.5**
Phase 4 Analysis Maple River	FHWA (Apile)	136.6	41.5	278.5	185.3
	USACE (Apile)	92.6	36.6	261.1	167.9
	Lambda 2 (Apile)	177.1	48	198.4	105.3
	API RP-2A (Apile)	179.6	50.5	207.3	114.1
	Average (2)	**136.1**	**43.5**	**238.5**	**145.3**
Phase 3 Analysis Lower Rush	FHWA (Apile)	229.3	82.7	373.1	279.9
	USACE (Apile)	113.3	48.9	322	229
1' into hard till	Lambda 2 (Apile)	238.7	69	254.8	161.6
	API RP-2A (Apile)	262.8	93.1	269.3	176.1
	Navfac (Allpile)	153.4		283.8	
	Average (2)	**188.05**	**71**	**313.95**	**220.75**
Phase 4 Analysis Lower Rush	FHWA (Apile)	184.2	82.7	374.6	281.4
	USACE (Apile)	104.2	48.2	384.7	291.6
	Lambda 2 (Apile)	251.9	71.4	303.2	210
	API RP-2A (Apile)	275.8	95.3	324.9	231.7
	Average (2)	**190**	**71.8**	**343.9**	**250.8**

		Ultimate Drained Capacity[1]	Skin Friction Drained	Ultimate Undrained Capacity[1]	Skin Friction Undrained
	Analysis Method	**(kips)**	**(kips)**	**(kips)**	**(kips)**
Phase 3 Analysis Rush River	FHWA (Apile)	232.3	84.1	212.7	287.7
	USACE (Apile)	109.9	45.4	205.9	212.7
3' into hard till	Lambda 2 (Apile)	245.2	70.6	238.5	145.4
	API RP-2A (Apile)	270.6	96	256.6	163.5
	Navfac (Allpile)	153.4		269.9	
	Average (2)	**190.25**	**70.7**	**237.9**	**216.55**
Phase 4 Analysis Rush River	FHWA (Apile)	184.6	83.1	379.1	285.9
	USACE (Apile)	100	44	413.2	320.1
	Lambda 2 (Apile)	256.2	71.5	323.1	229.9
	API RP-2A (Apile)	281.1	96.3	352.0	258.8
	Average (2)	**190.6**	**70.1**	**368.2**	**275.0**
Phase 4 Analysis Wolverton Structure	FHWA (Apile)	480.4	379.0	618.8	525.8
	USACE (Apile)	255.2	178.7	549.4	456.4
5' into hard till	Lambda 2 (Apile)	725.0	380.4	499.5	406.5
	API RP-2A (Apile)	817.2	472.6	543.4	450.4
	Average (2)	**536.2**	**325.7**	**559.2**	**566.2**
Phase 4 Analysis Inlet Structure	FHWA (Apile)	119.3	32.1	255.4	162.4
	USACE (Apile)	83.6	28.1	256.3	163.3
1' into hard till	Lambda 2 (Apile)	158.1	39.8	193.9	100.8
	API RP-2A (Apile)	157.7	39.5	203	110
	Average (2)	**120.9**	**33.9**	**225.1**	**132.1**
Phase 4 Analysis Outlet Structure	FHWA (Apile)	214.3	113	417.3	324.3
	USACE (Apile)	113.3	57.8	396.4	303.4
5' into hard till	Lambda 2 (Apile)	313.1	102	310.8	217.8
	API RP-2A (Apile)	344.7	133.5	339.3	246.2
	Average (2)	**229.0**	**95.7**	**363.8**	**211.0**

		Ultimate Drained Capacity[1]	Skin Friction Drained	Ultimate Undrained Capacity[1]	Skin Friction Undrained
	Analysis Method	**(kips)**	**(kips)**	**(kips)**	**(kips)**
Phase 4 Analysis Storage Area 1 Structure	FHWA (Apile)	181.6	80.3	360.6	267.6
	USACE (Apile)	101.4	44.9	328.4	235.3
0' into hard till	Lambda 2 (Apile)	262.5	78.9	246.0	152.9
	API RP-2A (Apile)	283.5	99.8	272.8	179.7
	Average (2)	**192.5**	**72.4**	**303.3**	**210.3**

(1) Pile capacity shall be the minimum of the soil capacity as reported in the table or the structural capacity of the pile with loads applied.
(2) The reported average capacity represent the average between the maximum and minimum capacity computed from the four calculation methods.

A pile tip penetration was estimated and is reported for each structure and ranges from about 1 to 5 feet of embedment into the hard or very dense glacial till bearing layer. Additional penetration during driving is unlikely due to hard driving conditions which could damage the pile or driving equipment.

A detailed summary of the pile capacities at each structure is presented in Exhibit O Table F-O6 and should be used for the preliminary design and layout. The pile tip elevation is presented in the table and the total length ranges from 25 to 116 feet depending on the structure foundation depth and depth to the glacial till layer. The undrained 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 most of the piles for this project are much shorter, by 40 to 50 feet, than most bridge foundation piles, the skin friction component is much less than for bridge foundations but the end bearing is similar for the same size pile. The ultimate drained or long-term axial pile capacities ranged from 125.7 to greater than 500 kips and are significantly less than the capacities determined for the undrained analysis. It should be noted that these analyses are based on the ultimate soil capacity and the structural capacity of the pile section may govern in some instances depending on the axial load and moment applied computed.

A preliminary sensitivity analysis was performed at the Wild Rice River structure to evaluate the effect of upward or downward groundwater flow on the axial pile capacity. The upward and downward flow primarily affects the drained pile capacity. The analysis using the Allpile software used an assumed value of 10 feet head above or below the estimated static level at the structure location. In order to evaluate this condition using the software, the unit weights were adjusted to account for increased effective unit weights where the groundwater flow was downward. In the case where upward flow was

simulated, a reduced effective stress was applied in each soil layer. The ultimate static drained capacity of the pile foundation is 153.4 kips. Using this methodology described the effect of upward flow reduces the ultimate capacity to 111.6 kips and the effect of downward flow increases the ultimate capacity to about 195.3 kips. This results in an increase or decrease from the static capacity in the drained condition as shown in Table F-O5.

Table F-O5 Sensitivity Analysis of Assumed Groundwater Conditions at Wild Rice River Control Structure (Drained Analysis)

Flow Condition	Head Conditions	Ultimate Axial Pile Capacity (kips)	% Difference from Static Level
Upward Flow	+10 feet of head	111.6	73%
Existing Conditions	-	153.4	0
Downward Flow	-10 feet of head	195.3	127%

The lateral pile capacities were computed at allowable pile top movement for a fixed head condition. The fixed head condition is appropriate due to the thickness and size of the pile caps and mats. The capacities were evaluated at the design pile length and for lateral deflections of 0.5 inches, 0.67 inches, and 0.875 inches which represent potential different structural load case scenarios. The results of the analyses show that lateral pile capacities range from 35 to 48 kips for the undrained loading condition. For the drained loading condition, the lateral pile capacities range from 21 to 36 kips.

Negative skin friction could occur due to the existence of the softer, compressible Brenna and Argusville formations under foundations if fill is placed adjacent to the structures in excess of the existing ground surface. The amount and extent of the negative skin friction varies depending on the individual site conditions. The effect of negative skin friction should be evaluated on a case by case basis during final design.

F-O7.0 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 toe resistance 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 otherwise called pile setup. 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 set up and strength gain could contribute additional capacity that reduces the number of piles and increases their spacing.

Table F-O6 Design Pile Capacities

Fargo-Moorhead Food Control Structures Preliminary Pile Foundation Analyses HP 14X73 A_{tip} = 198.5 in^2, A_{steel} = 21.4 in^2, perimeter = 56.4 in, width (b) = 14.6 in, I = 729 in^4													
Structure	**Diversion Channel Station Location**	**Approximate Ground (Bank) Surface Elevation (ft)**	**Invert Elevation (ft)**	**Estimated Fonndation Elevation (ft)**	**Estimated Ground Water Elevation (ft)**	**Design Condition/Tip Elevations**			**Ultimate Axial Capacity (kips)**	**Allowable Lateral Capacity (kips) (fixed head - single pile)**			**Estimated Settlement at allowable load**
										0.5"	**0.67"**	**0.875"**	
Red River Control Structure (LPP) ND Foundations and Wing walls	RRN 2529023	906	875.51	869.51	876.7	Phase 3	Undrained Analysis 835.8'	Total	239.1	35	41	48	<0.5"
								Uplift Resistance	148				
							Drained Analysis	Total	127.2	22	28	35	
								Uplift Resistance	36.6				
						Phase 4	Undrained Analysis 835.8'	Total	243.5	-			
								Uplift Resistance	150.3				
							Drained Analysis	Total	125.5				
								Uplift Resistance	36.6				
Red River Control Structure (FCP) MN Foundations and Wing walls PHASE 3	MN 128703	Right 906 Left 893	861	857	880.17	Undrained Analysis 815'		Total	269.4	35	41	47	<0.5"
								Uplift Resistance	176.2				
						Drained Analysis		Total	141.6	21	29	33	
								Uplift Resistance	26.9				
Wild Rice River Control Structure	WRR 66102	913	890.8	884.80	908	Phase 3	Undrained Analysis 836.8'	Total	296.2	35	41	48	<0.5"
								Uplift Resistance	174.7				
							Drained Analysis	Total	174.9	23	29	34	
								Uplift Resistance	62.2				
						Phase 4	Undrained Analysis 836.8'	Total	309.5	-			
								Uplift Resistance	216.3				
							Drained Analysis	Total	178.5				
								Uplift Resistance	65.3				

Fargo-Moorhead Food Control Structures
Preliminary Pile Foundation Analyses
HP 14X73
A_{tip} = 198.5 in^2, A_{steel} = 21.4 in^2, perimeter = 56.4 in, width (b) = 14.6 in, I = 729 in^4

Structure	Diversion Channel Station Location	Approximate Ground (Bank) Surface Elevation (ft)	Invert Elevation (ft)	Estimated Foundation Elevation (ft)	Estimated Ground Water Elevation (ft)	Design Condition/Tip Elevations			Ultimate Axial Capacity (kips)	Allowable Lateral Capacity (kips) (fixed head - single pile) 0.5"	0.67"	0.875"	Estimated Settlement at allowable load
Sheyenne River Aqueduct Crossing Fonndations	150467	916	883.68	879.68	908	Phase 3	Undrained Analysis 842.8'	Total	247.8	36	41	48	<0.5"
								Uplift Resistance	154.6				
							Drained Analysis	Total	146.1	23	29	36	
								Uplift Resistance	47.1				
						Phase 4	Undrained Analysis 842.8'	Total	255.7	-			
								Uplift Resistance	169.5				
							Drained Analysis	Total	140.9				
								Uplift Resistance	44.7				
Maple River Aqueduct Crossing Foundations	72543	898	872.06	868.06	893	Phase 3	Undrained Analysis 827.5'	Total	228.7	36	42	48	<0.5"
								Uplift Resistance	135.5				
							Drained Analysis	Total	125.7	23	29	33	
								Uplift Resistance	37				
						Phase 4	Undrained Analysis 827.5'	Total	238.5	-			
								Uplift Resistance	145.3				
							Drained Analysis	Total	136.1				
								Uplift Resistance	43.5				

1405

Fargo-Moorhead Food Control Structures
Preliminary Pile Foundatiou Analyses
HP 14X73

A_{tip} = 198.5 in^2, A_{steel} = 21.4 in^2, perimeter = 56.4 in, width (b) = 14.6 in, I = 729 in^4

Structure	Diversion Channel Station Location	Approximate Ground (Bank) Surface Elevation (ft)	Invert Elevation (ft)	Estimated Foundation Elevation (ft)	Estimated Ground Water Elevation (ft)	Design Condition/Tip Elevations			Ultimate Axial Capacity (kips)	Allowable Lateral Capacity (kips) (fixed head - single pile)			Estimated Settlement at allowable load
										0.5"	0.67"	0.875"	
Lower Rush River Drop Structure	60755	891.5	872.2	860.7	885	Phase 3	Undrained Analysis 809.4'	Total	314	35	42	48	<0.5"
								Uplift Resistance	220.8				
							Drained Analysis	Total	188	22	26	33	
								Uplift Resistance	71.0				
						Phase 4	Undrained Analysis 809.4'	Total	343.9	-			
								Uplift Resistance	250.8				
							Drained Analysis	Total	190				
								Uplift Resistance	71.8				
Rush River Drop Structure	45110	889	869.6	858.1	885.3	Phase 3	Undrained Analysis 806.9'	Total	237.9	35	42	48	<0.5"
								Uplift Resistance	216.6				
							Drained Analysis	Total	190.3	23	28	34	
								Uplift Resistance	70.7				
						Phase 4	Undrained Analysis 806.9'	Total	368.2	-			
								Uplift Resistance	275.0				
							Drained Analysis	Total	190.6				
								Uplift Resistance	70.1				
Storage Area 1 Outlet PHASE 4	WRR 75000			900+/-	908+/-	Undrained Analysis 845'		Total	303.3	-	-	-	-
								Uplift Resistance	210.3				
						Drained Analysis		Total	192.5	-	-	-	
								Uplift Resistance	72.4				

1406

Fargo-Moorhead Food Control Structures Preliminary Pile Foundation Analyses HP 14X73 A_{tip} = 198.5 in^2, A_{steel} = 21.4 in^2, perimeter = 56.4 in, width (b) = 14.6 in, I = 729 in^4												
Structure	**Diversion Channel Station Location**	**Approximate Ground (Bank) Surface Elevation (ft)**	**Invert Elevation (ft)**	**Estimated Foundation Elevation (ft)**	**Estimated Ground Water Elevation (ft)**	**Design Condition/Tip Elevations**		**Ultimate Axial Capacity (kips)**	**Allowable Lateral Capacity (kips) (fixed head - single pile)**			**Estimated Settlement at allowable load**
									0.5"	**0.67"**	**0.875"**	
Diversion Inlet Structure PHASE 4	152522	918.3	884	880	910.3	Undrained Analysis 845'	Total	225.1	30	35	41	<0.5"
							Uplift Resistance	132.1				
						Drained Analysis	Total	120.6	22	27	35	
							Uplift Resistance	33.9				
Diversion Outlet Ogee Structure PHASE 4	363	881	843.9	840	878.7	Undrained Analysis 778'	Total	363.8	35	41	48	<0.5"
							Uplift Resistance	211.0				
						Drained Analysis	Total	229.0	22	28	35	
							Uplift Resistance	95.7				
Wolverton Structure PHASE 4	WC 9079.5		895.3	891	898.4	Undrained Analysis 775'	Total	559.2	37	43	50	<0.5"
							Uplift Resistance	566.2				
						Drained Analysis	Total	536.2	28	35	42	
							Uplift Resistance	325.7				

RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT P – STRUCTURAL DESIGN CRITERIA

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

By: BARR ENGINEERING

FINAL – February 28, 2011

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F-P4 Emails between Barr Engineering and Corps of Engineers MVP discussing structural design criteria

APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT P – STRUCTURAL DESIGN CRITERIA

F-P1.0 STRUCTURAL DESIGN CRITERIA

The attachments to this Exhibit P include:

- Emails between Barr Engineering and Corps of Engineers MVP discussing structural design criteria – February 2011
- Emails between Barr Engineering and Corps of Engineers MVP discussing structural design criteria – June and July 2010
- June 7, 2010 Memorandum from Paul Nielsen and Miguel Wong, Barr Engineering to Tony Fares, Corps of Engineers MVP entitled Structural Design Criteria
- Section 10.7.2.4 – Horizontal Pile Foundation Movement – from the AASHTO LRFD Bridge Design Specifications Manual

Paul Nielsen

From: Miguel Wong
Sent: Tuesday, February 01, 2011 9:54 AM
To: 'Fares, Tony S MVP'; 'Hokens, Kent D MVP'; Buesing, Aaron W MVP; Brian K. LeMon; Tor S. Hansen; Paul Nielsen; Mustafa B. Igdelioglu
Cc: Aaron Grosser; Jed Greenwood; Brandon J. Barnes
Subject: FW: Ice forces on structures (UNCLASSIFIED)

fyi

Miguel Wong, PE, PhD

Senior Water Resources Engineer
Minneapolis office: 952.832.2632 | cell: 612.703.6562 mailto:mwong@barr.com www.barr.com

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resourceful. naturally.

-----Original Message-----
From: Tuthill, Andrew M ERDC-CRREL-NH [mailto:Andrew.M.Tuthill@usace.army.mil]
Sent: Tuesday, February 01, 2011 9:52 AM
To: Miguel Wong
Subject: Ice forces on structures (UNCLASSIFIED)

Classification: UNCLASSIFIED
Caveats: NONE

Miguel:

Here is a preliminary answer to the ice force question. The 5000 psf (34.7 psi) ice pressure with a 2-ft-thick ice cover is equivalent to 10 kips per lineal ft, a commonly used design number for static ice loads on rigid structures due to thermal expansion of an ice cover. It is usually applied uniformly to long structural elements such as the crest of a dam or a lock wall. I assume that the 100 and 500 year flood discharges would be used to estimate the elevations that these ice loads would be expected to act on the structures.

In terms static ice forces, the approach may be overly conservative because, as the stage rises, the ice cover progressively thins and weakens. The 2009 flood, which is approximately the 100-year event, is an example. The photos taken a the time of peak show very little remaining ice along the RRN or tribs. Since the near-peak channel is so much wider than the freezeup channel, the remaining ice is unconfined by the banks or structures, so static ice forces would not apply. The 5000 psf x 2-ft-thick ice would be an appropriate structural design number at freezeup discharges and water levels that are well within banks.

At higher water levels, dynamic forces due to crushing or bending failure of ice floes would govern. These ice pressures are usually higher than the static ones but applied to smaller areas such as the pier noses. Calculation methods are described in the AASHTO (1998) "LRFD Bridge Design Specifications" which are excerpted below along with typical ice strength values. This loading scenario assumes an environmental driving force such as current and/or wind sufficient to push a large ice floe against the structure and cause failure either in bending or crushing. In case of the RRN spring floods, t and p both likely decrease as the water level rises. For now we could use the following:

-For late winter conditions at the start of the rise, one could use t = 2 ft and p = 167 psi ("where breakup or major ice movement occurs at melting temperatures but the ice moves in large pieces and is internally sound")

-By the time the stage reaches top of bank (2-year discharge ?), one could assume that the ice cover had thinned to 1 ft and the ice had weakened to 111 psi (" where breakup occurs at melting temperatures and the ice structure is somewhat disintegrated:)

-Once the river is well out of bank, dynamic ice forces would likely be negligible since the ice "structure would be substantially disintegrated". We could assume p <= 56 psi and t<= 1 ft. Here the ice forces on the structures would probably result not from ice failure but from ice accumulating upstream due to driving forces of water current and wind drag. Forces exerted by accumulated ice can be calculated from drag equations and would be much less
than static or dynamic ice forces.

This is just a start. We can develop better relationships between ice thickness, ice strength and river stage by comparing the results of the ice thickness model (based on historic air temperature) with historic stage hydrographs. This will improve our estimate of ice forces as water levels rise under melting conditions.

Please call if you have questions or would like to discuss.

Thanks,

Andy

Andrew M. Tuthill, P. E.
U S Army Cold Regions Research and Engineering Laboratory
72 Lyme Rd.
Hanover, NH 03755
603-646-4225 office
603-643-3354 home
603-306-6699 cell

Classification: UNCLASSIFIED
Caveats: NONE

Classification: UNCLASSIFIED
Caveats: NONE

Paul Nielsen

From: Tor S. Hansen
Sent: Tuesday, February 01, 2011 2:39 PM
To: 'tony.s.fares@mvp02.usace.army.mil'; 'kent.d.hokens@usace.army.mil'; 'Aaron.W.Buesing@usace.army.mil'
Cc: Mustafa B. Igdelioglu; Miguel Wong; Paul Nielsen; Brian K. LeMon; Aaron Grosser
Subject: Summary of Conclusions from February 1 structural meeting for Fargo- Moorehead Metro Flood Risk Management Project Phase 4

All:

Below is Barr's summary of today's meeting regarding load cases, ice loads, and structure design elevations for the structures associated with the Fargo-Moorehead Flood Risk Management Project for Phase 4. If you disagree with any of the following conclusions please let me know. Also, please let me know if there is anything I have forgotten or not included from the meeting today. The following conclusions will govern the design of the structures for Phase 4 of the project.

Table 1 – Gated Structures Summary of Load Cases

Load Case	Event Category	Allowable Pile Deflection (inches)	Factor of Safety for Piles	Soil Condition
1 – 100 yr flood	Usual	0.67 (note 1)	2.00	Undrained
1.1 – 100 yr + ice (note 4 and 5)	Unusual	0.875 (note 2)	1.50	Undrained
2 – 500 yr flood	Unusual	0.875 (note 2)	1.50	Undrained
2.1 – 500 yr + 5ft	Extreme	1.000 (note 3)	1.15	Undrained
3 – construction	Unusual	0.67	1.50	Undrained
4 – Normal low flow	Usual	0.50	2.00	Drained

Note 1: 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.
Note 2: 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.
Note 3: It was agreed that for this extreme event an allowable deflection of 1.0-inch is acceptable.
Note 4: 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.
Note 5: An ice/debris load of 500 PLF along the structure will be used for the wing and retaining wall structures.
Note 6: A 10-foot long SSP cut-off will be provided on upstream and downstream edges of gated structures. (i.e. 2 – rows of SSP)
Note 7: 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)
Note 8: The structure freeboard will extend 2 –additional feet for a total height of 7-feet above the 500 year flood elevation.
Note 9: 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 2 – Aqueduct Structures

Load Case	Event Category	Factor of Safety for Piles	Soil Condition
1 – 100 yr flood	Usual	2.00	Undrained
2 – 100 yr + ice	Unusual	1.50	Undrained

(note 1,2,4)			
3 – 500 yr flood	Unusual	1.50	Undrained
4 – 500 yr (+match levee elevation) (note 7)	Extreme	1.15	Undrained
5 – construction	Unusual	1.50	Undrained
6 – Normal low flow (note 3)	Usual	2.00	Drained

Note 1: 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.
Note 2: 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.
Note 3: 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.
Note 4: An ice/debris load of 500 PLF along the structure will be used for the wing and retaining wall structures.
Note 5: 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)
Note 6: 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)
Note 7: The top of structure design elevation will match the elevation of the highest levee adjacent to the structure.

Thank you all again for attending today's meeting. It was very productive.

Tor S. Hansen, PE

Vice President
Senior Civil Engineer
Minneapolis office: 952.832.2758
thansen@barr.com
www.barr.com

Paul Nielsen

From: Miguel Wong
Sent: Monday, June 07, 2010 4:31 PM
To: 'tony.s.fares@usace.army.mil'
Cc: Paul Nielsen; Brian K. LeMon; Evans, Craig O MVP
Subject: Structural Design Criteria
Attachments: Phase 3_Structural Design Criteria_07June2010.pdf

Dear Tony,

Find attached a memorandum summarizing the primary structural design considerations that we propose to follow for the Phase 3 feasibility design of the main hydraulic structures. This memorandum is in addition to the discussion with Ken Hoakens on the specs for piles (that you confirmed to agree with), and the ongoing work on Geotechnical Engineering that is being completed with input from Kurt Heckendorf.

In general, we are following the framework used by the Corps of Engineers for this type of work, but we would still appreciate very much to have your buy-in on the methodology before the calcs are completed.

We have tentatively assumed that you will be able to provide feedback by this Friday, June 11th (we are adding a task to the Critical Path Items Checklist). Let us know if you would be able to meet this tight schedule, otherwise please let us know an alternative due date.

Thanks a lot,

Miguel

Miguel Wong, PE, PhD

Senior Water Resources Engineer
Minneapolis office: 952.832.2632
cell: 612.703.6562
mwong@barr.com
www.barr.com

Paul Nielsen

From: Miguel Wong
Sent: Monday, June 21, 2010 11:31 AM
To: Paul Nielsen; Mustafa B. Igdelioglu
Cc: Tor S. Hansen
Subject: FW: FMM Critical Path Checklist Item

fyi

Miguel Wong, PE, PhD

Senior Water Resources Engineer
Minneapolis office: 952.832.2632
cell: 612.703.6562
mwong@barr.com
www.barr.com

From: Sarah Stratton
Sent: Monday, June 21, 2010 11:13 AM
To: Craig.O.Evans@usace.army.mil; tony.s.fares@usace.army.mil; Brian K. LeMon; Miguel Wong
Cc: Sarah Stratton; Miguel Wong
Subject: FMM Critical Path Checklist Item

A task has been added or updated:

Follow-up on structural design criteria
Requestor: Brian LeMon(952-832-2774; blemon@barr.com). USACE to provide feedback on email from Paul Nielsen, Barr to Tony Fares, USACE on June 18th 4.24 pm. The email summarized conference call on Tuesday, June 15th at 3.30 pm - including:

Red River Control Structure

Case = 1; Flood Event = 100 yr; Allow. Stress Increase = 1.0; Lateral Pile deflection = 0.67 inches; comments = w/ 2 ft ice pressure

Case = 2; Flood Event = Top of Structure; Allow. Stress Increase = 1.33; Lateral Pile deflection = 0.875 inches; comments = HW = 500 yr + 2 ft free board, TW = 500yr, w/2 ft ice pressure

Case = 3; Flood Event = Construction; Allow. Stress Increase = 1.33; Lateral Pile deflection = 0.67 inches; comments = no HW, TW, ice

Please verify that ice pressure is required for Case 2. We assume that the Flood Events listed for each case are maximum levels and that lower flood events may control.

Aqueducts, Wild Rice River Control structure, & Drop structures

Controlling cases will need to be evaluated differently from Red River Control Structure to determine stability and structural components since the diversion channel and tributary rivers could be at different flood events. Allowable pile loading and deflections will be based on original structural criteria sent by Barr to USACE.

Sheet pile weirs

Will be designed base on undrained & drained soil parameters determined under Barr's geotechnical work in Phase 3.

Reply by Tony Fares on 6/21/10.

I confirm the 3 loading cases as described for the Red River Control Structure. According to Hydraulics, the 100 year event will produce the highest head differential on the structure.
Ice pressure is required for case 2 if HW ice level is at or below the top of the structure.
The Wild Rice River Control Structure should be designed for the same loading cases as the Red River Control Structure: 100 year, 500 year and construction (structure is totally built before letting water into it). For allowable pile loading and deflections, use the same as for the Red River Control Structure.
For the rest of the structures allowable pile loading and deflections will be based on original structural criteria sent by Barr to USACE.
The assumption for Sheet Pile design is acceptable. Take into consideration expected scour and erosion.
For straight line sheet pile cantilevered wall, they better be short due to the soft clay. Since the weir walls are like U shape walls you can consider anchoring the opposite walls together if that produce more stable U walls.

Category: Geotechnical and Structural

Due Date: 6/23/2010

Assigned To:
Craig Evans
Tony Fares
Brian LeMon
Miguel Wong

Login to Barr Project Management websites at: http://www.barr.com/clientre/Login.asp to update the completed date.

Paul Nielsen

From: Fares, Tony S MVP [tony.s.fares@mvp02.usace.army.mil]
Sent: Friday, July 02, 2010 12:59 PM
To: Paul Nielsen
Cc: Hokens, Kent D MVP; Heckendorf, Kurt A MVP
Subject: FW: Load Cases

Paul.
Please update the load cases per Kent Hoken's email as included below.
Please relay the info to Mustafa so he can utilize them in his design of the aqua duct structures.
Let us know if you need further assistance.

Tony Fares, P.E.
Structural Engineer
USACE St. Paul District
email: tony.s.fares@usace.army.mil
Tel: (651) 290 5568

-----Original Message-----
From: Hokens, Kent D MVP
Sent: Friday, July 02, 2010 7:09 AM
To: Fares, Tony S MVP
Cc: Wittine, Eric A MVP
Subject: Load Cases

Tony,

Thinking about this last night, I would suggest the following load cases for the Fargo feasibility design.

100 yr flood - Usual
100 yr flood + ice - Unusual
500 yr flood - Unusual
500 yr flood + ice - Extreme
Normal low flow - Usual

This would be mostly consistent with the latest direction and provide a reasonable but conservative cost estimate.

Kent

From:	Heckendorf, Kurt A MVP
To:	Miguel Wong; Brian K. LeMon
Cc:	Sarah Stratton; Matthew R. Metzger; Jed Greenwood; Aaron Grosser; Ivan Contreras; Paul Nielsen; Mustafa B. Igdelioglu; mbittner@cityoffargo.com; awalker@cityoffargo.com; bob.zimmerman@ci.moorhead.mn.us; Lesher, Michael D MVP; Pang, Edith P MVP; Hansen, Jeffrey L MVP; Fares, Tony S MVP; Stefanik, Elliott L MVP; Snyder, Aaron M MVP; Evans, Craig O MVP; Lee Beauvais; Stuart Dobberpuhl; C. Gregg Thielman; Hokens, Kent D MVP; Wittine, Eric A MVP; Dahlquist, Michael S MVP; Behling, Christopher W MVP; Rydeen, David W MVP; Morey, Darrell W MVP
Subject:	FMMFS: Allowable Capacity for Piles
Date:	Friday, July 02, 2010 2:22:45 PM

Everyone,

The St. Paul PDT members had a meeting this morning to discuss the pile capacities for the Fargo-Moorhead project. We feel that for the feasibility study, we need to follow EM 1110-2-2906 and be computing capacities using both undrained and drained shear strengths. We would have to obtain a waiver if we were to neglect the drained capacity and this would take some time. For the feasibility study, we want to make sure we capture the foundation costs and do not want to under estimate the costs.

Given the time frame, the Corps was hoping that Barr could look into what pile spacings are required for the different load cases (i.e. end-of-construction, normal river elevation & structural dead load, and the flood events). The appropriate uplift pressures should be taken into account with each load case. If the pile spacing is physically possible (i.e. spacing is 4 feet or more) we feel we could live with this. If not, then the alternative would be drilled shafts / caissons.

Along a parallel track, the Corps was also hoping that Barr could look into drilled shaft / caisson foundation for the structures and what the associated costs would be. This would help the entire team decide which direction to go for the final.

The biggest thing is that whatever direction we take, we need to have a cost estimate completed towards the end of July to ensure that we can meet the schedule.

If any of the calculations for the pile spacing can be sent to Kurt, Tony, Eric, and Kent for our review, it would be greatly appreciated. (Aaron Grosser proved the pile capacity calcs for the Maple River Structure in email today at 13:00 and we'll be looking at this.)

Please contact myself or Tony Fares if you have any questions or concerns.

We can discuss this in further details at the Weekly PDT meeting on Tuesday 6 JUL at 10:00 AM.

Respectfully,
Kurt
Kurt A. Heckendorf, P.E.
U.S. Army Corps of Engineers, St. Paul District
Geotechnical and Geology Section
Engineering and Construction Division, Design Branch
Phone: 651-290-5411
Fax: 651-290-5805

We have moved, new mailing address, effective June 14, 2010
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St. Paul, MN 55101-1678

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Minneapolis, MN • Hibbing, MN • Duluth, MN • Ann Arbor, MI • Jefferson City, MO • Bismarck, ND

Memorandum

To: Tony Fares – Corps of Engineers MVP

From: Paul Nielsen, Miguel Wong – Barr Engineering

Subject: Structural Design Criteria

Date: June 7, 2010

Project: Fargo-Moorhead Metro Flood Risk Management Project, Feasibility Study, Phase 3

c: Craig Evans – Corps of Engineers MVP; Brian LeMon – Barr Engineering

Draft for discussion

Introduction

The purpose of this memorandum is to introduce the methodology and design criteria that will be used by the A-E team to complete the feasibility structural design of the main hydraulic structures that are part of the Red River diversion project (Fargo Moorhead Metro Feasibility Study – Phase 3). It is the expectation of the A-E team that the U.S. Army Corps of Engineers (USACE) will provide feedback on the contents of this memorandum before the actual Phase 3 feasibility structural design calculations are completed.

The first section of this memo identifies the USACE documents that were reviewed to set up the constraints and criteria that will be used to guide the structural calculations for this project. Following this will be a number of sections that summarize the main constraints and criteria as they pertain to each specific element of design. The elements of design considered in this memo are:

- Stability for Concrete Structures including those at:
  - Red River of the North
  - Wild Rice River
  - Sheyenne River
  - Maple River
  - Lower Rush River
  - Rush River
- Loads and Load Combinations
- Hydraulic Loads

- Geotechnical
- Sheet Pile Weirs
- Reinforced Concrete Design

General Structural Guidance

The USACE Engineer Regulation ER 1110-2-1150 "Engineering and Design for Civil Work Projects", Section 13.5 (Engineering during Feasibility Phase) indicates that the first engineering objective is to "Provide engineering data and analyses sufficient to develop the complete project schedule and cost estimate." The USACE Engineer Regulation ER 1110-2-1150, Appendix C-7 formulates the structural requirements that need to be included in the feasibility engineering appendix.

The USACE is governed by Engineer Regulations (ER's), Engineer Manuals (EM's), Engineer Technical Letters (ETL's) and Engineering Circulars (EC's). These documents are available online at the following web site: http://www.usace.army.mil/inet/usace-docs. It is our understanding that industry standards shall apply when USACE criteria are not applicable.

The USACE documents to be used in Phase 3 designs include:

- EM 1110-2-1612 "Ice Engineering"
- EM 1110-2-2100 "Stability Analysis of Concrete Structures"
- EM 1110-2-2104 "Structural Design for Reinforced-Concrete Hydraulic Structures"
- EM 1110-2-2200 "Gravity Dam Design"
- EM 1110-2-2502 "Retaining and Flood Walls"
- EM 1110-2-2504 "Design of Sheet Pile Walls"
- EM 1110-2-2702 "Design of Spillway Tainter Gates"
- EM 1110-2-2906 "Design of Pile Foundations
- ER 1110-2-1806 "Earthquake Design and Analysis for Corps of Engineers Projects"

Stability for Concrete Structures

The USACE Engineer Manual EM 1110-2-2100 provides guidance for stability of concrete gravity structures. Where there are conflicting stability criteria with other documents, this manual will govern. Appendix B of this manual describes load classifications (Usual, Unusual, Extreme) and all loading-

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condition requirements for each structure as specified in various manuals. This manual will be followed for Phase 3 feasibility designs except as noted below, for pile analysis and design.

The soil conditions along the entire project (see Geotechnical), are underlain by "soft" clay, susceptible to large settlements. Therefore, piles will be used at all concrete structures. Stability requirements outlined in USACE Engineer Manual EM 1110-2-2100 do not cover pile supported structures. Piles will be designed per USACE Engineer Manual EM 1110-2-2906, and this design will include the following considerations:

- The purpose of a pile foundation is to transfer and distribute load through a material or stratum with inadequate bearing, sliding or uplift capacity to a firmer stratum capable of supporting the load without detrimental displacement. The conventional stability check of sliding, rotation, floatation and bearing per USACE Engineer Manual EM 1110-2-2100 is no longer applicable.
- A rigid cap analysis will determine the axial load to each pile. Horizontal loads will be assumed to be evenly resisted by each vertical pile. For battered piles, the horizontal component of the batter provides an additional lateral resistance to horizontal loads.
- Vertical and lateral loads on each pile will be checked to make sure the pile capacity (with Factor of Safety) and stresses are not exceeded in accordance with USACE Engineer Manual EM 1110-2-2906, Section 4-2 (Loading Conditions). The number of piles and spacing are adjusted to meet the required pile capacity and stresses.
- In general, stresses for the usual and extreme case may be increased up to 33% and 75% respectively. However, checks must be made to ensure that the structure will not catastrophically fail during or after extreme loading conditions.
- The Factor of Safety (F.S.) associated with each load case varies with the frequency (probability) of each flood event. Assuming that piles will be test loaded, the Usual case is based on a 10 yr event with a F.S. =2.0, the Unusual case is based on 300 yr event with a F.S. =1.5, and the Extreme case is based on the maximum design flood (MDF) with a F.S. = 1.15. The MDF is the designation used to represent the maximum loading condition (as judged by the minimum factor of safety) and must be determined for each structure. The MDF may be represented by any large flood event up to the probable maximum flood (PMF). For this Phase 3 we will assume the MDF occurs when the headwater is at the top of the structure prior to overtopping. This will represent a flood greater than the 500 yr event due to freeboard and will be classified as an Extreme event.

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Loads and Load Combinations

Load classifications and combinations described in USACE Engineer Manual EM 1110-2-2100 Appendix B will be used to determine the loads and applicable load combinations, including:

- The load conditions that a structure may encounter during its service life are grouped into the load condition categories of Usual, Unusual and Extreme (per USACE Engineer Manual EM 1110-2-2100, Section 3-2).
- USACE Engineer Manual EM 1110-2-2100 Table B-1 lists 7 basic Loading Conditions for a gravity dam (Structure Type: Gravity Dams, EM 1110-2-2200; Navigation Dams, EM 1110-2-2607):

Load Case	Loading Description	Classification
1	Construction Condition	UN
2	Normal Operating	U
3	Infrequent Flood	UN
4	Construction with Operational Basis Earthquake (OBE)	E
5	Coincident Pool with OBE	UN
6	Coincident Pool with Maximum Design Earthquake (MDE)	E
7	Maximum Design Flood (MDF)	U/UN/E

* U = usual, UN = unusual, E = extreme

- Load Conditions 2 (Usual), 3 (Unusual), and 7 (Extreme) will be checked during the stability analysis of the piles. Conditions 1 (construction), 4, 5, and 6 will not be checked. USACE Engineer Regulation ER 1110-2-1806 indicates that the project is located in Seismic Zone 0. Conditions 4, 5, and 6 all include earthquake loading, which typically does not control in Seismic Zone 0. For final design all load conditions should be checked.
- Basic Loads used for stability include:
    - Vertical dead load of concrete.
    - Vertical dead load of water on foundation, openings, and downstream back face of structure.
    - Uplift loads on foundation (see Hydraulic Loads & Geotechnical).

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- Horizontal water pressures (static) based on headwater and tailwater elevations (see Hydraulic Loads).
- USACE Engineer Manual EM 1110-2-1612: Ice loads assume 2 ft thick at 5,000 psf acting at headwater elevation. No ice pressure is assumed on downstream face. For final design impact loads from floating ice and debris should be investigated.

Hydraulic Loads

Hydraulic information will be provided based on revised models for the Phase 3 work. Both horizontal and vertical loads due to water pressures will be considered during feasibility designs in Phase 3. Hydrostatic loads from HEC-RAS modeling using Phase 3 hydrology during10-yr, 200-yr, and 500-yr events will be provided for each structure. Uplift loads will be evaluated for structure based on the hydraulic gradient, permeability of soil and cut-off wall recommendations provided during Geotechnical evaluation.

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Geotechnical

Soil properties for the soil profile at a given structure will be modeled using current industry standard software, such as DRIVEN, ALLPILE, A-PILE and/or L-PILE to determine the vertical and lateral capacity of the piles in accordance with USACE Engineer Manual EM 1110-2-2906. Allowable displacement at the pile cap will depend on the load classification as follows:

Load Classification	Vertical Displacement (inches)	Horizontal Displacement (inches)
Usual	0.25	0.50
Unusual	0.33	0.67
Extreme	0.44	0.875

The larger displacements for the Unusual and Extreme load classifications have been assumed by Barr Engineering. They correspond to an increase of 133% and 175% of the deformation allowed for the Usual case based on Barr's interpretation of the allowable stress increase of 33% and 75% allowed in USACE Engineer Manual EM 1110-2-2906. The deflections above represent deflection limits for gated structures like dams; larger deflections may be allowed for less sensitive structures if the stresses in the structure and piles are not excessive. For final design, allowable deflection limits should be established for each structure.

A seepage analysis will be made to determine the requirements of cut-off walls and uplift pressures on the Red River Control Structure and Maple Hydraulic Structure. No site specific seepage analysis will be performed at the other concrete structures. Results from the two analyses above will be used as a reference to determine approximate cut-off wall requirements at the other structures. For final design, a seepage analysis should be made for each structure to determine uplift mitigation requirements like drains and cut-off walls.

Sheet Pile Weirs

Sheet pile structures will be designed in accordance with the requirements of USACE Engineer Manual EM 1110-2-2504. It is anticipated that all test results will not be completed prior to the submission of the Phase 3 report. Soil parameters provide by the USACE in Table I-2 of the January 2010 "Geotechnical Design and Geology" will be used to determine the lateral earth pressures for both the undrained and drained parameters sheeting driven in clay.

Reinforced Concrete Design

Preliminary sizing of reinforced concrete components will be made in accordance with USACE Engineer Manual EM 1110-2-2104, including the following design considerations:

- A hydraulic load factor, Hf = 1.3 (accept 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 between $0.25\rho_b$ and $0.375\rho_b$.

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I = influence factor of the effective group embedment (dim.)

D' = effective depth taken as $2D_b/3$ (ft.)

D_b = depth of embedment of piles in layer that provides support, as specified in Figure 10.7.2.3.1-1 (ft.)

$N1_{60}$ = *SPT* blow count corrected for both overburden and hammer efficiency effects (blows/ft.) as specified in Article 10.4.6.2.4.

q_c = static cone tip resistance (ksf)

Alternatively, other methods for computing settlement in cohesionless soil, such as the Hough method as specified in Article 10.6.2.4.2 may also be used in connection with the equivalent footing approach.

The corrected *SPT* blow count or the static cone tip resistance should be averaged over a depth equal to the pile group width B below the equivalent footing. The *SPT* and *CPT* methods (Eqs. 1 and 2) shall only be considered applicable to the distributions shown in Figure 10.7.2.3.1-1b and Figure 10.7.2.3.1-2.

10.7.2.4 Horizontal Pile Foundation Movement

Horizontal movement induced by lateral loads shall be evaluated. The provisions of Article 10.5.2.1 shall apply regarding horizontal movement criteria.

The horizontal movement of pile foundations shall be estimated using procedures that consider soil-structure interaction. Tolerable lateral movements of piles shall be established on the basis of confirming compatible movements of structural components, e.g., pile to column connections, for the loading condition under consideration.

The effects of the lateral resistance provided by an embedded cap may be considered in the evaluation of horizontal movement.

The orientation of nonsymmetrical pile cross-sections shall be considered when computing the pile lateral stiffness.

Lateral resistance of single piles may be determined by static load test. If a static lateral load test is to be performed, it shall follow the procedures specified in ASTM D 3966.

The effects of group interaction shall be taken into account when evaluating pile group horizontal movement. When the *P-y* method of analysis is used, the values of P shall be multiplied by P-multiplier values, P_m, to account for group effects. The values of P_m provided in Table 1 should be used.

C10.7.2.4

Pile foundations are subjected to horizontal loads due to wind, traffic loads, bridge curvature, vessel or traffic impact and earthquake. Batter piles are sometimes used but they are somewhat more expensive than vertical piles, and vertical piles are more effective against dynamic loads.

Methods of analysis that use manual computation were developed by Broms (*1964a and 1964b*). They are discussed in detail by Hannigan et al. (*2005*). Reese developed analysis methods that model the horizontal soil resistance using *P-y* curves. This analysis has been well developed and software is available for analyzing single piles and pile groups (*Reese, 1986*; *Williams et al., 2003*; and *Hannigan et al., 2005*).

Deep foundation horizontal movement at the foundation design stage may be analyzed using computer applications that consider soil-structure interaction. Application formulations are available that consider the total structure including pile cap, pier and superstructure (*Williams et al., 2003*).

If a static load test is used to assess the site specific lateral resistance of a pile, information on the methods of analysis and interpretation of lateral load tests presented in the *Handbook on Design of Piles and Drilled Shafts Under Lateral Load* (*Reese, 1984*) and *Static Testing of Deep Foundations* (*Kyfor et al., 1992*) should be used.

Table 10.7.2.4-1 Pile P-Multipliers, P_m, for Multiple Row Shading (averaged from *Hannigan et al., 2005*).

Pile *CTC* spacing (in the direction of loading)	*P*-Multipliers, P_m		
	Row 1	Row 2	Row 3 and higher
3*B*	0.7	0.5	0.35
5*B*	1.0	0.85	0.7

Loading direction and spacing shall be taken as defined in Figure 1. If the loading direction for a single row of piles is perpendicular to the row (bottom detail in the figure), a group reduction factor of less than 1.0 should only be used if the pile spacing is 5*B* or less, i.e., a P_m of 0.7 for a spacing of 3*B*, as shown in Figure 1.

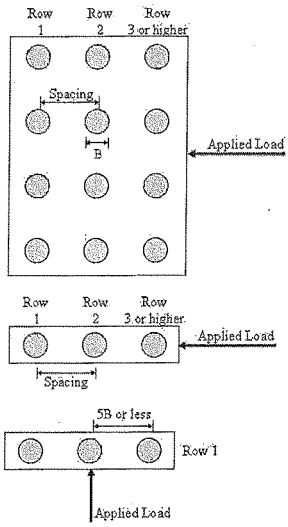

Figure 10.7.2.4-1 Definition of loading direction and spacing for group effects.

Since many piles are installed in groups, the horizontal resistance of the group has been studied and it has been found that multiple rows of piles will have less resistance than the sum of the single pile resistance. The front piles "shade" rows that are further back.

The P-multipliers, P_m, in Table 1 are a function of the center-to-center (*CTC*) spacing of piles in the group in the direction of loading expressed in multiples of the pile diameter, *B*. The values of P_m in Table 1 were developed for vertical piles only.

Horizontal load tests have been performed on pile groups, and multipliers have been determined that can be used in the analysis for the various rows. Those multipliers have been found to depend on the pile spacing and the row number in the direction of loading. To establish values of P_m for other pile spacing values, interpolation between values should be conducted.

The multipliers on the pile rows are a topic of current research and may change in the future. Values from recent research have been tabulated by Hannigan et al. (*2005*). Averaged values are provided in Table 1.

Note that these *P-y* methods generally apply to foundation elements that have some ability to bend and deflect. For large diameter, relatively short foundation elements, e.g., drilled shafts or relatively short stiff piles, the foundation element rotates rather than bends, in which case strain wedge theory (*Norris, 1986*; *Ashour et al., 1998*) may be more applicable. When strain wedge theory is used to assess the lateral load response of groups of short, large diameter piles or shaft groups, group effects should be addressed through evaluation of the overlap between shear zones formed due to the passive wedge that develops in front of each shaft in the group as lateral deflection increases. Note that P_m in Table 1 is not applicable if strain wedge theory is used.

Batter piles provide a much stiffer lateral response than vertical piles when loaded in the direction of the batter.

RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT Q – STRUCTURAL DESIGN COMPUTATIONS— CONTROL STRUCTURES

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

By: BARR ENGINEERING

FINAL – Version February 28, 2011

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ELECTRONIC ATTACHMENTS ONLY

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APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT Q – STRUCTURAL DESIGN COMPUTATIONS—CONTROL STRUCTURES

F-Q1.0 GROUP AND RIGID CAP ANALYSIS RESULTS

Attachments F-Q1.1 through F-Q1.5 of this Exhibit Q present the results from the group and rigid cap structural design analyses for the Red River of the North (LPP) control structure wing walls A and F.

F-Q2.0 STRUCTURAL PILE CAPACITY RESULTS

Attachment F-Q2.1 of this Exhibit Q presents the structural pile capacities for the following structural components: HP14x73 (usual case), HP14x89 (usual case), Red River (LPP) Control Structure Wing Wall A HP14x89, and Red River (LPP) Control Structure Wing Wall F HP14x89.

F-Q3.0 ICE LOADS ON PIERS

Attachment F-Q3.1 of this Exhibit Q presents the ice loads on piers for the following structural components: Red River of the North (LPP) and Wild Rice River Gated Control Structures.

F-Q4.0 COMPLETE SET OF STRUCTURAL COMPUTATIONS

The complete set of structural computations for all load conditions for the Red River of the North (LPP) control structure and Wild Rice River control structure is included electronically on the attached DVD.

ATTACHMENT

F-Q1.1 Summary of Group and Rigid Cap Analyses Results

BARR ENGINEERING			DATE	2/21/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER		
PKN 2/21/11		PKN	SUBJECT	GOUP vs RIGID Cap Analysis	

Control Structure: ND RRN Wing Wall SECTION A Summary Pile Reactions

103% of ALLOWABLE O.K.

Load Case	Pile Row	1	2	3	4	5	6	7	8	Allowable Pile Loads
	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)	51.59	53.28	44.97	34.93	20.13	5.52	-8.91	-23.57	59.775
	Horiz. Displace (in.)	-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	
	Fy (k)	-33.57	-28.87	-28.91	-30.80	-31.32	-31.40	-31.47	-31.54	Horiz Loads/pile
	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	
	Horiz. Displace (in.)	-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	
	Fy (k)	-33.922	-29.177	-29.206	-31.107	-31.619	-31.674	-31.73	-31.783	Horiz Loads/pile

Control Structure: ND RRN Wing Wall SECTION F Summary Pile Reactions

Load Case	Pile Row	1	2	3	4	5	Allowable Pile Loads
	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 (in.)	-0.4354	-0.4354	-0.4354	-0.4354	-0.4354	
	Mz max (k-ft)	152.50	140.00	139.17	140.83	139.17	
	Fy (k)	-28.61	-25.32	-25.23	-25.85	-25.55	Horiz Loads/pile

Control Structure: Wolverton Creek Wing Wall SECTION A Summary Pile Reactions

Load Case	Pile Row	1	2	3	4		Allowable Pile Loads
	Rigid Cap Analysis						
1	Axial Load (tons/pile)	53.29	36.74	12.07	-12.59		100
	Horiz Resist Capacity(k)	61.70	31.00	31.00	31.00	Row 1 includes 3": 12" batter	
	GROUP						
	Axial Load (tons/pile)	89.88	38.17	12.25	-16.37	Local Axis	100
	Horiz. Displace (in.)	-0.1033	-0.1033	-0.1033	-0.1033	Global	
	Mz max (k-ft)	41.17	63.92	64.00	63.92		
	Fy (k)	-80.78	-22.18	-22.24	-22.28		Horiz Loads/pile

BATTERED Row

ATTACHMENT

F-Q1.2 Rigid Cap Analyses Results for the Red River (LPP) Control Structure Wing Wall A

BARR ENGINEERING			DATE 2/4/2011	SHEET NO.
			PROJECT NAME Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN 2/4/11		PKN	SUBJECT Control Structure: ND RRN 35k s35k Load Cases: 1 SECTION A	

Case 1
Event 100 years

H.W. EL. = 922.01 ft
T.W. EL. = 902.41 ft

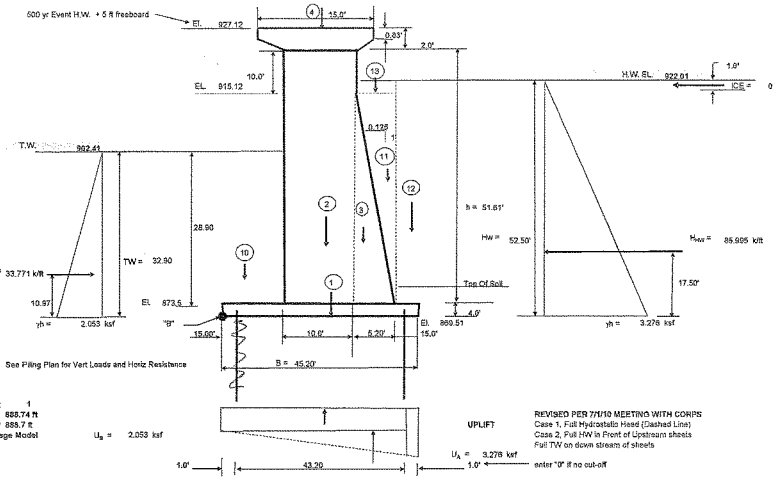
500 yr H.W. EL = 922.12 ft

Top of Fig EL. = 873.51 ft

File:

State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 891.2 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Case 1 or 2: 1
Normal Water Level, El. 885.74 ft
Δh normal = 885.7 ft
See Geotechnical seepage Model U_B = 2.053 ksf

UPLIFT
U_A = 3.278 ksf

REVISED PER 7/1/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Fig concrete	1	30	45.20	4.00	0.15	rec	813.6	22.60	18,388.4
Bridge Slab	4	30	15.60	0.83	0.15	rec	58.0	20.00	1,126.5
Bridge Slab	4	30	12.50	1.17	0.15	rec	65.8	20.00	1,316.3
Monolith	2	30	10.50	51.61	0.15	rec	2322.5	20.00	46,449.0
Monolith	3	30	5.20	41.61	0.15	tri	487.0	26.73	13,018.1
					D.L. Concrete	ΣVc =	3744.9	ΣMv =	80,292.2

CONSTANT FOR ALL LOAD CASES

T.W on ftg Monolith	10	30	15.00	28.90	0.0624	rec	811.5	7.50	6,086.3
H.W. on Monolith Slope	11	30	5.20	41.61	0.0624	tri	202.6	26.73	5,415.5
H.W. Above Slope	13	30	5.20	5.00	0.0624	rec	97.1	27.60	1,651.8
H.W. on Mono. ftg	12	30	15.00	48.50	0.0624	rec	1361.9	37.70	51,344.8
					D.L. Water	ΣVw =	2443.1	ΣMv =	64,498.1

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	30	45.20	2.053	rec	-2783.9	22.60	-62,918
	U_A	30	45.20	1.223	tri	-829.2	36.13	-24,969
					ΣU =	-3613.1	ΣMu =	-87,886

Horizontal Loads		L ft	H ft	Pressure ksf		ICE K	arm ft	Mu ft-k
	ICE	30	1.00	0.00	rec	0.0	52	0.0

REVISED PER 2/1/11 MEETING WITH CORPS
Ice Loading is for Ice Floe on single Pier Only on Gated Structure
use 600 plf along Wing Walls

		L ft	Force k/ft		H K	arm ft	Mw ft-k
Water Loads	H_{hw}	30	33.771	tri	1013.14	10.97	11110.72
	H_{ww}	30	-85.995	tri	-2579.85	17.50	-45147.38
				ΣW =	-1566.71	ΣMw =	-34036.7

Overturning Moments	ΣMOT = M_A +M_W +M_{ICE} =	-121943 kip-ft
Resisting Moments	ΣMR = M_V =	144980 kip-ft

Sum of Moments	ΣMnet = M_R + M_{OT} =	23,047 kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	2,575 kips
Sum of Horizontal Forces	H = Ice + water =	-1,567 kips

Location of Resultant Xr = ΣM / P = 8.95 ft from Toe
e = B/2 - Xr = 13.65 ft
B/6 = 7.534 ft

BARR ENGINEERING			DATE 2/4/2011	SHEET NO.
			PROJECT NAME Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN 2/4/11		PKN	SUBJECT Control Structure: ND RRN 35k s25k Load Cases: 1,1	SECTION A

Case 1.1
Event 100 years

H.W. EL. = 922.01 ft
T.W. EL. = 902.41 ft

500 yr H.W. EL. = 922.12 ft

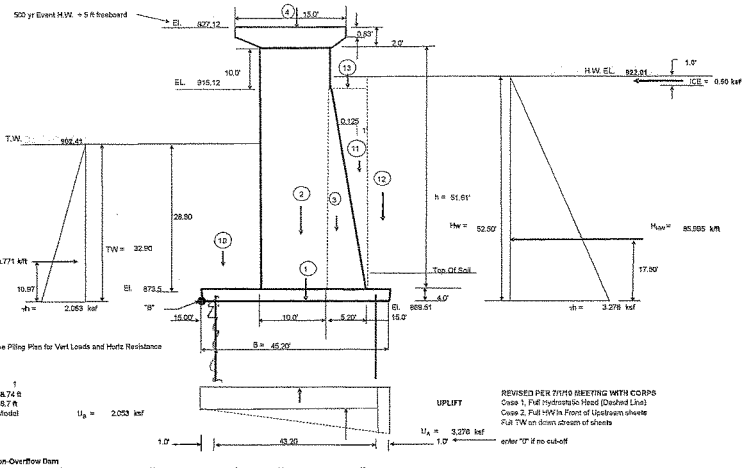
Top of Fig EL. = 873.51 ft

File:

State Building Codes
Frost Depth = 5.0 ft — provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.2 ft — DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section

Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft — overlap distance at stepped ftg



Case 1 or 2: 1
Normal Water Level, El. 888.74 ft
Δh normal = 888.7 ft
See Geotechnical seepage Model

U_B = 2.053 ksf

UPLIFT

U_A = 3.278 ksf

1.0' — 43.20 — 1.0' — enter "0" if no cut-off

REVISED PER 2/1/11 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	30	45.20	4.00	0.15	rec	813.6	22.60	18,388.4
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	20.00	1,120.5
Bridge Slab	4	30	12.50	1.17	0.15	rec	65.9	20.00	1,316.3
Monolith	2	30	10.00	51.61	0.15	rec	2322.5	20.00	46,449.0
Monolith	3	30	5.20	41.61	0.15	tri	487.0	26.73	13,018.1
						D.L. Concrete	ΣVc = 3744.9	ΣM_V =	80,292.2

CONSTANT FOR ALL LOAD CASES

T.W on ftg Monolith	10	30	15.00	28.90	0.0624	rec	811.5	7.50	6,086.3
H.W. on Monolith Slope	11	30	5.20	41.61	0.0624	tri	202.5	26.73	5,415.5
H.W. Above Slope	12	30	5.20	8.39	0.0624	rec	67.1	27.60	1,851.6
H.W. on Mono. ftg	13	30	15.00	46.50	0.0624	rec	1261.9	37.70	51,344.8
						D.L. Water	ΣVw = 2343.1	ΣM_V =	64,698.1

Uplift Loads		L ft	W ft	Pressure ksf			U K	arm ft	Mu ft-k
	U_B	30	45.20	2.053		rec	-2783.9	22.60	-62,918
	U_A	30	45.20	1.223		tri	-829.2	30.13	-24,989
						ΣU =	-3613.1	ΣM_U =	-87,906

Horizontal Loads		L ft	H ft	Pressure ksf			ICE K	arm ft	Mu ft-k
	ICE	30	1.00	-0.50		rec	-15.0	52	-780.0

REVISED PER 2/1/11 MEETING WITH CORPS
Ice Loading is for Ice Flow on single Pier Only on Gated Structure
use 500 plf along Wing Walls

Water Loads	L ft	Force k/ft			H K	arm ft	Mw ft-k
H_{TW}	30	33.771		tri	1013.14	10.97	11110.72
H_{NW}	30	-85.995		tri	-2579.85	17.50	-45147.38
					ΣW = -1566.71	ΣM_W =	-34036.7

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-122723 kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	144960 kip-ft

Sum of Moments	ΣMnet = $M_R + M_{OT}$ =	22,267 kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	2,575 kips
Sum of Horizontal Forces	H = Ice + water =	-1,582 kips

Location of Resultant
$X_r = \Sigma M / P$ = 8.65 ft from Toe
e = B/2 - X_r = 13.95 ft
B/6 = 7.534 ft

BARR ENGINEERING			DATE 2/4/2011					SHEET NO.
			PROJECT NAME Flood Control ND Diversion Inlet - Corps Of Engineers					
COMPUTED PKN 2/4/11	CHECKED	SUBMITTED PKN	PROJECT NUMBER					
			SUBJECT	Control Structure:	ND RRN	35k	s35k	SECTION A
				Load Cases: 2				

Case 2
Event 500 years

H.W. EL. = 922.12 ft
T.W. EL. = 914.41 ft

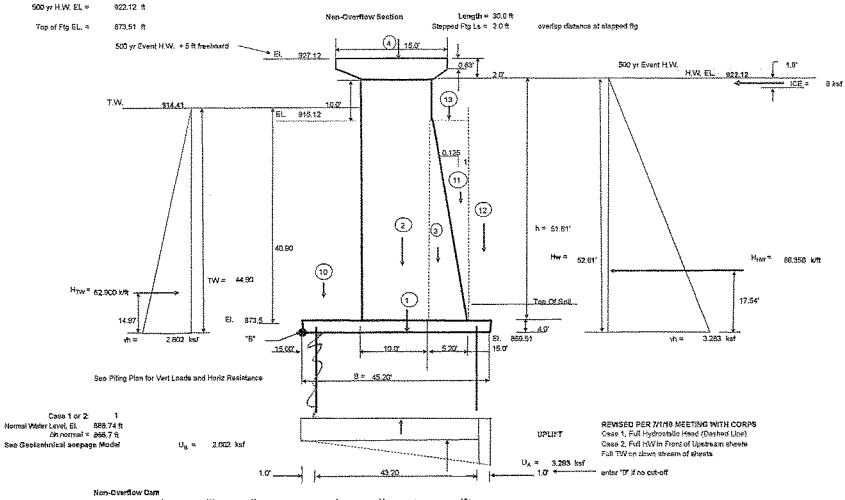
500 yr H.W. EL. = 922.12 ft

Top of Ftg EL. = 873.51 ft

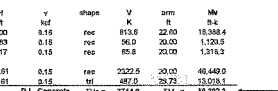
File:

State Building Codes
Frost Depth = 5.0 ft — provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 851.2 ft — DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section — Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft — overlap distance at stepped ftg



Case 1 or 2: 1
Normal Water Level, El. 888.74 ft
Gh normal = 866.7 ft
See Geotechnical seepage Model — U_B = 2.002 ksf



REVISED PER 7/1/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	30	45.20	4.00	0.15	rec	813.6	22.60	18,388.4
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	29.00	1,120.5
Bridge Slab	4	30	12.50	1.17	0.15	rec	65.8	29.00	1,318.3
Monolith	2	30	10.00	51.61	0.15	rec	2322.5	20.00	46,449.0
Monolith	3	30	5.20	41.61	0.15	tri	487.0	39.73	13,019.1
					D.L. Concrete		ΣVc = 3744.9	ΣMv =	80,295.2

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Monolith	19	30	15.00	40.90	0.0624	rec	1148.5	7.50	8,613.5
H.W. on Monolith Slope	11	30	5.20	41.61	0.0624	tri	202.6	26.73	5,415.6
H.W. Above. Slope	13	30	5.20	7.00	0.0624	rec	68.2	27.60	1,881.2
H.W. on Mono. ftg	12	30	15.00	46.61	0.0624	rec	1305.0	37.70	51,461.0
						D.L. Water	ΣVw = 2784.2	ΣMw =	67,371.3

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	30	45.20	2.802	rec	-3799.3	22.60	-85,866
	U_A	30	45.20	0.461	tri	-326.2	30.13	-9,830
					ΣU =	-4125.5	ΣMu =	-95,696

Horizontal Loads		L ft	H ft	Pressure ksf		ICE K	arm ft	Mu ft-k
	ICE	30	1.00	0.00	rec	0.0	52.11	0.0

REVISED PER 2/1/11 MEETING WITH CORPS
Ice Loading is for Ice Flow on single Pier Only on Gated Structure
use 500 plf along Wing Walls

Water Loads	L ft	Force k/ft		H K	arm ft	Mw ft-k
H_{TW}	30	52.900	tri	1586.99	14.97	23741.58
H_{HW}	30	-88.358	tri	-2650.67	17.54	-46431.75
			ΣW =	-783.69	ΣMw =	-17189.9

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-112868 kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	147684 kip-ft

Sum of Moments	ΣMnet = $M_R + M_{OT}$ =	34,778 kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	2,484 kips
Sum of Horizontal Forces	H = Ice + water =	-784 kips

Location of Resultant	Xr = ΣM / P =	14.47 ft from Toe
	e = B/2 - Xr =	8.13 ft
	B/6 =	7.534 ft

BARR ENGINEERING			DATE 3/4/2011	SHEET NO.
			PROJECT NAME Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN 2/4/11		PKN	SUBJECT Control Structure: ND RRN 35k s30k	
			Load Cases: 2.1	SECTION A

Case 2.1 — Per COE 2/1/11: HW = 500yr + 5' w/ No Ice — File:

Event 500 years

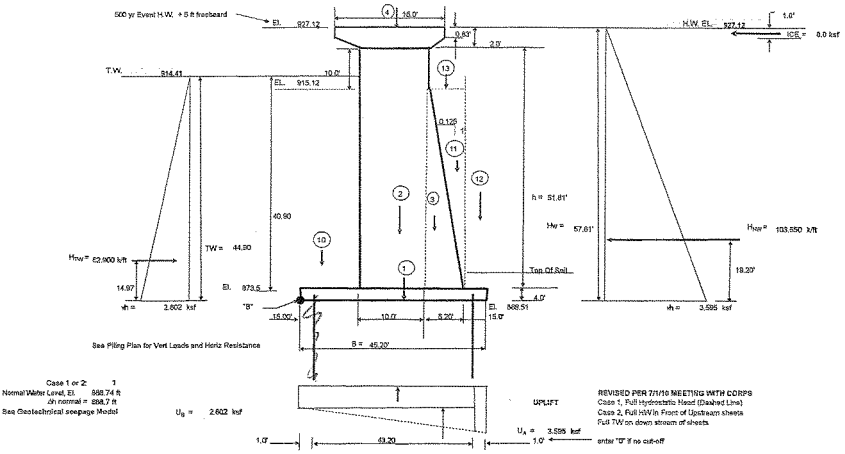
State Building Codes
Frost Depth = 5.0 ft — provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.2 ft — DEC, JAN, FEB Mean Water Elevation

H.W. EL. = 927.12 ft
T.W. EL. = 914.41 ft

500 yr H.W. EL = 922.12 ft

Top of Ftg EL. = 873.51 ft

Non-Overflow Section
Length = 30.0 ft
Stepped Ftg Le = 2.0 ft — overlap distance at stepped ftg



See Piling Plan for Vert Loads and Horiz Resistance

Case 1 or 2: 1
Normal Water Level, El. 888.74 ft
Δh normal = 894.7 ft
See Geotechnical seepage Model — U_B = 2.602 ksf

UPLIFT

REVISED PER 2/1/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets.

U_A = 3.595 ksf
1.0' — 49.20 — 1.0'
enter "0" if no cut-off

Non-Overflow Dam

	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Vertical Loads									
Ftg concrete	1	30	45.20	4.00	0.15	rec	813.6	22.60	18,384.4
Bridge Slab	4	30	16.00	0.83	0.15	rec	99.0	20.00	1,320.5
Bridge Slab	4	30	12.50	1.17	0.15	rec	85.8	20.00	1,716.3
Monolith	2	30	10.00	51.61	0.15	rec	2322.5	30.03	49,449.0
Monolith	3	30	5.20	41.61	0.15	tri	467.0	29.73	13,916.1
						D.L. Concrete	ΣVc = 3744.9	ΣMv =	80,292.2

CONSTANT FOR ALL LOAD CASES

T.W on ftg Monolith	10	30	15.00	40.90	0.0624	rec	1148.5	7.50	8,613.5
H.W. on Monolith Slope	11	30	5.20	41.61	0.0624	tri	202.8	26.73	5,415.5
H.W. Above Slope	13	30	5.20	12.00	0.0624	rec	116.8	27.30	3,224.9
H.W. on Mono. ftg	12	30	15.00	53.61	0.0624	rec	1503.4	37.70	56,754.9
						D.L. Water	ΣVw = 2973.3	ΣMv =	74,008.2

		L ft	W ft	Pressure ksf			U K	arm ft	Mu ft-k
Uplift Loads									
	U_B	30	45.20	2.602		rec	-3798.3	22.60	-85,886
	U_A	30	45.20	0.793		tri	-537.7	30.13	-16,204
						SU =	-4337.8	ΣMu =	-102,071

		L ft	H ft	Pressure ksf			ICE K	arm ft	Mu ft-k
Horizontal Loads									
	ICE	30	1.00	0.00		rec	0.0	57.11	0.0

REVISED PER 2/1/11 MEETING WITH CORPS
Ice Loading is for Ice Floe on single Pier Only on Gated Structure
use 690 plf along Wing Walls

		L ft	Force k/ft		H K	arm ft	Mw ft-k
Water Loads							
	H_{nw}	30	62.900	tri	1886.99	14.97	28241.88
	H_{sw}	30	-103.550	tri	-3105.50	19.33	-59955.19
					ΣW = -1218.52	ΣMyr =	-31413.3

Overturning Moments	ΣMOT = M_D +M_W +M_{ICE} =	-133454 kip-ft
Resisting Moments	ΣMR = M_V =	154300 kip-ft

Sum of Moments	ΣMnet = M_R + M_{OT} =	20,846 kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	2,381 kips
Sum of Horizontal Forces	H = Ice + water =	-1,220 kips

Location of Resultant	Xr = ΣM / P =	8.74 ft from Toe
	e = B/2 - Xr =	13.60 ft
	B/6 =	7.534 ft

BARR ENGINEERING			DATE	2/4/2011				SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers				
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER					
PKN		PKN	SUBJECT	Control Structure;	ND RRN	35k	s35k	
2/4/11				Load Cases: 3				SECTION A

Case 3
Event 2 years

H.W. EL. = ft
T.W. EL. = ft

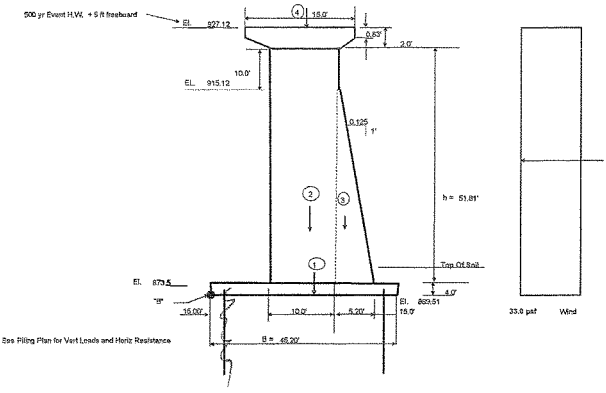
500 yr H.W. EL. = 922.12 ft

Top of Flg EL. = 873.51 ft

File:

State Building Codes
Frost Depth = 5.0 ft — provide min frost flg protection during Dec, Jan, Feb, March
Water El. = 961.2 ft — DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section
Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft — overlap distance at stepped ftg



Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	30	45.20	4.00	0.15	rec	813.6	22.60	18,388.4
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	20.00	1,120.5
Bridge Slab	4	30	12.50	1.17	0.15	rec	65.8	20.00	1,316.3
Monolith	2	30	10.00	51.81	0.15	rec	2322.5	20.00	46,449.0
Monolith	3	30	5.20	41.61	0.15	tri	487.0	26.73	13,018.1
					D.L. Concrete	ΣVc =	3744.9	ΣMy =	80,292.2

← CONSTANT FOR ALL LOAD CASES

Horizontal Loads		L ft	H ft	Pressure ksf		H K	arm ft	Mw ft-k
	Wind	30	57.81	0.03	rec	47.0	28.905	-1,642.9

Overturning Moments	$\Sigma M_{OT} = M_W =$	-1643 kip-ft
Resisting Moments	$\Sigma M_R = M_V =$	80292 kip-ft

Sum of Moments	$\Sigma Mnet = M_R + M_{OT} =$	78,649 kip-ft
Sum of Vertical Forces	P = Conc =	3,745 kips
Sum of Horizontal Forces	H = Wind =	-57 kips

Location of Resultant
$Xr = \Sigma M / P =$ 21.00 ft from Toe
e = B/2 - Xr = 1.80 ft
B/6 = 7.554 ft

BARR ENGINEERING			DATE 2/4/2011				SHEET NO.
			PROJECT NAME Fleed Control ND Diversion Inlet - Corps Of Engineers				
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER				
PKN 2/4/11		PKN	SUBJECT	Control Structure: Load Cases: 4	ND RRN 35k	s35k	SECTION A

Case 4
Event years

File:

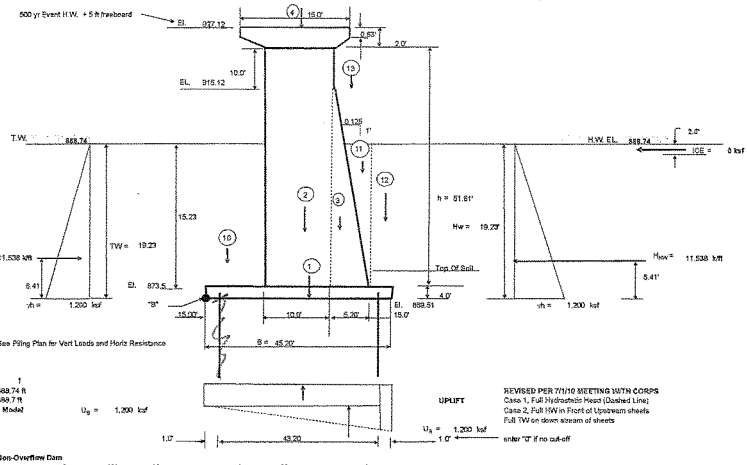
State Building Codes
Frost Depth = 5.0 ft — provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.2 ft — DEC, JAN, FEB Mean Water Elevation

H.W. EL. = 888.74 ft — Normal Water Level
T.W. EL. = 888.74 ft

500 yr H.W. EL. = 922.12 ft

Top of Ftg EL. = 873.51 ft

Non-Overflow Section
Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft — overlap distance at stepped ftg



Case 1 or 2: 1
Normal Water Level, El. 888.74 ft
Δh normal = 888.7 ft
See Geotechnical seepage Model

U_B = 1.200 ksf

UPLIFT

REVISED PER 7/1/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

U_A = 1.200 ksf
1.0' — 43.20 — 1.0'
enter "0" if no cut-off

Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	30	45.20	4.00	0.15	rec	813.6	22.60	18,388.4
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	20.00	1,120.5
Bridge Slab	4	30	12.50	1.17	0.15	rec	65.8	20.00	1,316.3
Monolith	2	30	10.00	51.61	0.15	rec	2322.5	20.00	46,449.6
Monolith	3	30	5.20	51.61	0.15	tri	487.0	26.73	13,018.1
					D.L. Concrete		ΣVc = 3744.9	ΣMy =	80,292.2

CONSTANT FOR ALL LOAD CASES

T.W on ftg Monolith	10	30	15.00	15.23	0.0624	rec	427.7	7.50	3,207.4
H.W. on Monolith Slope	11	30	1.90	15.23	0.0624	tri	27.1	25.63	695.7
H.W. Above Slope	12	30	0.00	28.38	0.0624	rec	0.0	26.00	0.0
H.W. on Mono. ftg	13	30	15.00	15.23	0.0624	rec	427.7	37.70	16,123.3
					D.L. Water		ΣVw = 882.5	ΣMy =	20,026.4

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	30	45.20	1.200	rec	-1627.2	22.60	-36,775
	U_A	30	45.20	0.000	tri	0.0	30.13	0
					ΣU =	-1627.2	ΣMu =	-36,775

Horizontal Loads		L ft	H ft	Pressure ksf		ICE K	arm ft	Mu ft-k
	ICE	30	2.00	0.00	rec	0.0	18.23	0.0

REVISED PER 2/1/11 MEETING WITH CORPS
Ice Loading is for Ice Floe on single Pier Only on Gated Structure
use 500 plf along Wing Walls

Water Loads	L ft	Force k/ft		H K	arm ft	Mw ft-k
H_{hw}	30	11.538	tri	346.13	6.41	2218.67
H_{nw}	30	-11.538	tri	-346.13	6.41	-2218.67
			ΣW =	0.00	ΣMw =	0.0

Overturning Moments	$\Sigma M_{OT} = M_U + M_N + M_{OE} =$	-36775 kip-ft
Resisting Moments	$\Sigma M_R = M_V =$	100019 kip-ft

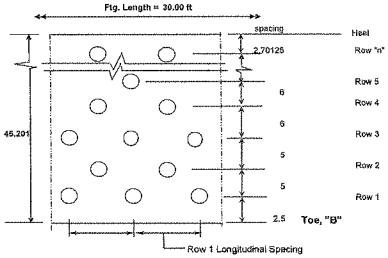
Sum of Moments	ΣMnet = $M_R + M_{OT} =$	63,243 kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	3,000 kips
Sum of Horizontal Forces	R = Ice + water =	0 kips

Location of Resultant
Xr = ΣM / P = 21.18 ft from Toe
e = B/2 - Xr = 1.42 ft
B/6 = 7.534 ft

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED PKN 2/4/11	CHECKED	SUBMITTED PKN	PROJECT NUMBER		
			SUBJECT	Pile Capacity Control Structure: 35k	SECTION A

PILE FOUNDATION DESIGN

FLOW



PILE PATTERN GEOMETRY

L = 30.00 ft

Transverse Spacing		Distance to Toe, d_{toe}	Longitudinal Spacing	Batter	Piles per Row (N)	Edge Dist (ft)		Trial N	
Row 1 to Toe	2.50 ft	2.5 ft	4.25 ft	0 °1/12"	7	2.25	1	8	
Row 1 to Row 2	6.00 ft	7.5 ft	4.25 ft	0 °1/12"	7	2.25	2	8	14
Row 2 to Row 3	6.00 ft	12.5 ft	4.25 ft	0 °1/12"	7	2.25	3	8	21
Row 3 to Row 4	6.00 ft	18.5 ft	5.00 ft	0 °1/12"	6	2.50	4	7	27
Row 4 to Row 5	6.00 ft	24.5 ft	5.00 ft	0 °1/12"	6	2.50	5	7	33
Row 5 to Row 6	6.00 ft	30.5 ft	5.00 ft	0 °1/12"	6	2.50	6	7	39
Row 6 to Row 7	6.00 ft	36.5 ft	5.00 ft	0 °1/12"	6	2.50	7	7	45
Row 7 to Row 8	6.00 ft	42.5 ft	5.00 ft	0 °1/12"	6	2.50	8	7	51
Row 8 to Row 9	0.00 ft	0.0 ft	0.00 ft	0 °1/12"	0	15.00	0	0	
Row 9 to Row 10	0.00 ft	0.0 ft	0.00 ft	0 °1/12"	0	15.00	0	0	
Row 10 to Row 11	0.00 ft	0.0 ft	0.00 ft	0 °1/12"	0	15.00	0	0	
Row 11 to Row 12	0.00 ft	0.0 ft	0.00 ft	0 °1/12"	0	15.00	0	0	
Row 12 to Row 13	0.00 ft	0.0 ft	0.00 ft	0 °1/12"	0	15.00	0	0	
Row 13 to Row 14	0.00 ft	0.0 ft	0.00 ft	0 °1/12"	0	15.00	0	0	
Row 14 to Row 15	0.00 ft	0.0 ft	0.00 ft	0 °1/12"	0	15.00	0	0	
Last Row to Heel	2.70 ft								
	45.20 ft		Note: Enter 0 for Longitudinal Spacing for Rows Not Used)		ΣN = 51			59	

Pile Properties: Pile Type: HP (C.I.P or HP) Pile Length = **34.7 ft** Pile Tip El. 835.8 Pile Cap Embed = 1.00 ft

HP Nominal Depth, h = 14.0 in Wt. per ft, plf 74 **Total pile Length = 1,770 LF**

Pile Group Properties

N.A. of Pile Group to Toe

$X_{NA} = (\Sigma N * d_{toe}) / \Sigma N =$ **21.03 ft**

	Dist. From N.A. to Pile Row	d	N	$I = N * d^2$
1	Dist. To Row 1	18.53 ft	7	2403.4
2	Dist. To Row 2	13.53 ft	7	1281.3
3	Dist. Row 3	8.53 ft	7	509.3
4	Dist. Row 4	2.53 ft	6	38.4
5	Dist. Row 5	-3.47 ft	6	72.3
6	Dist. Row 6	-9.47 ft	6	538.2
7	Dist. Row 7	-15.47 ft	6	1436.0
8	Dist. Row 8	-21.47 ft	6	2765.9
0	Row 9 (not used)	0.00 ft	0	0.0
0	Row 10 (not used)	0.00 ft	0	0.0
0	Row 11 (not used)	0.00 ft	0	0.0
0	Row 12 (not used)	0.00 ft	0	0.0
0	Row 13 (not used)	0.00 ft	0	0.0
0	Row 14 (not used)	0.00 ft	0	0.0
0	Row 15 (not used)	0.00 ft	0	0.0
			51	$\Sigma I =$ 9044.7

ALLOWABLE LOADS (from Geotechnical)

Service	Allowable Pile Loads						
Case	1 Usual	1.1 Unsual	2 Unusual	2.1 Extreme	3 Unusual	4 Usual	
Axial	59.8 tons	79.7 tons	79.7 tons	104.0 tons	79.7 tons	31.8 tons	
Tension	40.8 tons	54.1 tons	54.1 tons	70.8 tons	54.1 tons	7.5 tons	
Lateral	31.0 kips/pile	35.0 kips/pile	35.0 kips/pile	35.0 kips/pile	30.0 kips/pile	22.0 kips/pile	w/o Group effects

Summary Pile Reactions

Load Combinations	Pile Loads (tons/pile)															Max. Vertical Load (Tons)	Horiz Pile Group Capacity (k)	
Allowable Pile Loads	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15			
1 59.8 tons	57.1	48.5	39.9	29.6	19.3	9.0	-1.4	-11.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	57.1	1,581	OK
1.1 79.7 tons	57.9	49.1	40.3	29.7	19.1	8.6	-2.0	-12.6	0.0	0.0	0.0	0.0	0.0	0.0	0.0	57.9	1,785	OK
2 79.7 tons	39.7	35.4	31.0	25.8	20.5	15.3	10.1	4.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	39.7	1,785	OK
2.1 104.0 tons	53.3	45.2	37.1	27.4	17.7	8.0	-1.7	-11.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	53.3	1,785	OK
3 79.7 tons	39.8	36.8	36.8	36.7	36.7	36.7	36.8	36.8	0.0	0.0	0.0	0.0	0.0	0.0	0.0	36.8	1,785	OK
4 31.8 tons	28.9	29.1	29.2	29.3	29.5	29.6	29.8	29.9	0.0	0.0	0.0	0.0	0.0	0.0	0.0	29.9	1,122	OK
															Max Service : P =	57.9		

Using solid mechanics equations adapted for discrete elements, the forces in the pile rows for different load combinations are determined.
The force in each pile row is found using:

$$\text{Pile Load} = P / N + M_{NA} / I$$

First, the moment about the toe must be translated to get the moment about the neutral axis of the pile group.

$$e_{toe} = M_{toe} / P$$

Then the eccentricity about the neutral axis of the pile group is

$$e_{NA} = X_{NA} - e_{toe}$$

The moment about the neutral axis of the pile group becomes

$$M_{NA} = P * e_{NA}$$

For battered pile, the Vertical pile load needs to be transformed to the axial load along the pile axis

Paxial = 1.000 Pvert

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			SUBJECT Pile Capacity Control Structure: 35k	SECTION A

FORCE RESULTANT (see Stability Analysis)

CASE	Event	Vertical Load P (kips)	Horizontal Load H (kips)	ΣM_{toe} (kip-ft)	$e_{toe} = M_{toe} / P$ (ft)	$e_{NA} = X_{NA} - e_{toe}$ (ft)	$M_{NA} = P * e_{NA}$ (kip-ft)	
1	100	2,575	1,567	23,047	8.95	12.08	31099	Usual
1.1	100	2,575	1,582	22,267	8.65	12.38	31879	Unusual
2	500	2,404	704	34,778	14.47	6.56	15768	Unusual
2.1	500	2,381	1,220	20,816	8.74	12.29	29256	Extreme
3	Dry	3,745	57	78,649	21.00	0.03	103	Unusual
4	Daily	3,600	0	63,543	21.18	-0.15	-452	Usual

SERVICE

Case 1
Flood Event 100
Usual

Vertical Load, P = 2575 kips
Horizontal Load, H = 1567 kips
M_{NA} = 31099 kip-ft

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Vertical Pile Loading	P / N	+	$M_{NA} * d / \Sigma I$	= Pile Loads		Axial Pile Load
1 Row 1	50.5		63.7	114.2 kips/pile	57.1 tons/pile	57.1 tons/pile
2 Row 2	50.5		46.5	97.0 kips/pile	48.5 tons/pile	48.5 tons/pile
3 Row 3	50.5		29.3	79.8 kips/pile	39.9 tons/pile	39.9 tons/pile
4 Row 4	50.5		8.7	59.2 kips/pile	29.6 tons/pile	29.6 tons/pile
5 Row 5	50.5		-11.9	38.6 kips/pile	19.3 tons/pile	19.3 tons/pile
6 Row 6	50.5		-32.6	17.9 kips/pile	9.0 tons/pile	9.0 tons/pile
7 Row 7	50.5		-53.2	-2.7 kips/pile	-1.4 tons/pile	-1.4 tons/pile
8 Row 8	50.5		-73.8	-23.3 kips/pile	-11.7 tons/pile	-11.7 tons/pile
9 Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10 Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11 Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12 Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13 Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14 Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15 Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
				max:	57.1 tons/pile	max: 57.1 tons/pile

Assumed lateral Capacity: 31.0 kips/pile

Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resistance due to Bending, kips	Group Efficiency	Lateral Resitance	
1 Row 1	0	7	0.0	217	1.000	217 kips	
2 Row 2	0	7	0.0	217	1.000	217 kips	
3 Row 3	0	7	0.0	217	1.000	217 kips	
4 Row 4	0	6	0.0	186	1.000	186 kips	
5 Row 5	0	6	0.0	186	1.000	186 kips	
6 Row 6	0	6	0.0	186	1.000	186 kips	
7 Row 7	0	6	0.0	186	1.000	186 kips	
8 Row 8	0	6	0.0	186	1.000	186 kips	
9 Row 9	0	0	0.0	0	1.000	0 kips	
10 Row 10	0	0	0.0	0	1.000	0 kips	
11 Row 11	0	0	0.0	0	1.000	0 kips	
12 Row 12	0	0	0.0	0	1.000	0 kips	
13 Row 13	0	0	0.0	0	1.000	0 kips	
14 Row 14	0	0	0.0	0	1.000	0 kips	
15 Row 15	0	0	0.0	0	1.000	0 kips	
		51		1581		1581 kips	OK

Case 1.1
Flood Event 100
Unusual

Vertical Load, P = 2575 kips
Horizontal Load, H = 1582 kips
M_{NA} = 31879 kip-ft

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Vertical Pile Loading	P / N	+	$M_{NA} * d / \Sigma I$	= Pile Loads		Axial Pile Load
1 Row 1	50.5		65.3	115.8 kips/pile	57.9 tons/pile	57.9 tons/pile
2 Row 2	50.5		47.7	98.2 kips/pile	49.1 tons/pile	49.1 tons/pile
3 Row 3	50.5		30.1	80.6 kips/pile	40.3 tons/pile	40.3 tons/pile
4 Row 4	50.5		8.9	59.4 kips/pile	29.7 tons/pile	29.7 tons/pile
5 Row 5	50.5		-12.2	38.3 kips/pile	19.1 tons/pile	19.1 tons/pile
6 Row 6	50.5		-33.4	17.1 kips/pile	8.6 tons/pile	8.6 tons/pile
7 Row 7	50.5		-54.5	-4.0 kips/pile	-2.0 tons/pile	-2.0 tons/pile
8 Row 8	50.5		-75.7	-25.2 kips/pile	-12.6 tons/pile	-12.6 tons/pile
9 Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10 Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11 Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile

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12	Row 12	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
					max:	57.9 tons/pile	max:	57.8 tons/pile

Assumed lateral Capacity: 35.0 kips/pile

	Horizontal Pile Capacity	Batter °/ft	N	Resistance due to Batter, kips	Resistance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	7	0.0	245	1.000	245 kips	
2	Row 2	0	7	0.0	245	1.000	245 kips	
3	Row 3	0	7	0.0	245	1.000	245 kips	
4	Row 4	0	6	0.0	210	1.000	210 kips	
5	Row 5	0	6	0.0	210	1.000	210 kips	
6	Row 6	0	6	0.0	210	1.000	210 kips	
7	Row 7	0	6	0.0	210	1.000	210 kips	
8	Row 8	0	6	0.0	210	1.000	210 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			51		1785		1785 kips	OK

Case 2
Flood Event 500
Unusual

Vertical Load, P = 2404 kips
Horizontal Load, H = 704 kips
M_{bh} = 15768 kip-ft

	Vertical Pile Loading	P / N	+	M_{bh} * d / Σ I	= Pile Loads		Axial Pile Load
1	Row 1	47.1		32.3	79.4 kips/pile	39.7 tons/pile	39.7 tons/pile
2	Row 2	47.1		23.6	70.7 kips/pile	35.4 tons/pile	35.4 tons/pile
3	Row 3	47.1		14.9	62.0 kips/pile	31.0 tons/pile	31.0 tons/pile
4	Row 4	47.1		4.4	51.5 kips/pile	25.8 tons/pile	25.8 tons/pile
5	Row 5	47.1		-6.1	41.1 kips/pile	20.5 tons/pile	20.5 tons/pile
6	Row 6	47.1		-16.5	30.6 kips/pile	15.3 tons/pile	15.3 tons/pile
7	Row 7	47.1		-27.0	20.2 kips/pile	10.1 tons/pile	10.1 tons/pile
8	Row 8	47.1		-37.4	9.7 kips/pile	4.8 tons/pile	4.8 tons/pile
9	Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12	Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
					max:	39.7 tons/pile	max: 39.7 tons/pile

Assumed lateral Capacity: 35.0 kips/pile

	Horizontal Pile Capacity	Batter °/ft	N	Resistance due to Batter, kips	Resistance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	7	0.0	245	1.000	245 kips	
2	Row 2	0	7	0.0	245	1.000	245 kips	
3	Row 3	0	7	0.0	245	1.000	245 kips	
4	Row 4	0	6	0.0	210	1.000	210 kips	
5	Row 5	0	6	0.0	210	1.000	210 kips	
6	Row 6	0	6	0.0	210	1.000	210 kips	
7	Row 7	0	6	0.0	210	1.000	210 kips	
8	Row 8	0	6	0.0	210	1.000	210 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			51		1785		1785 kips	OK

Case 2.1
Flood Event 500
Extreme

Vertical Load, P = 2381 kips
Horizontal Load, H = 1220 kips
M_{bh} = 29296 kip-ft

	Vertical Pile Loading	P / N	+	M_{bh} * d / Σ I	= Pile Loads	Axial Pile Load

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1	Row 1	46.7	59.9	106.6 kips/pile		53.3 tons/pile		53.3 tons/pile
2	Row 2	46.7	43.8	90.5 kips/pile		45.2 tons/pile		45.2 tons/pile
3	Row 3	46.7	27.6	74.3 kips/pile		37.1 tons/pile		37.1 tons/pile
4	Row 4	46.7	8.2	54.9 kips/pile		27.4 tons/pile		27.4 tons/pile
5	Row 5	46.7	-11.2	35.5 kips/pile		17.7 tons/pile		17.7 tons/pile
6	Row 6	46.7	-30.6	16.1 kips/pile		8.0 tons/pile		8.0 tons/pile
7	Row 7	46.7	-50.0	-3.4 kips/pile		-1.7 tons/pile		-1.7 tons/pile
8	Row 8	46.7	-69.4	-22.8 kips/pile		-11.4 tons/pile		-11.4 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
					max:	53.3 tons/pile	max:	53.3 tons/pile

Assumed lateral Capacity: 35.0 kips/pile

	Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	7	0.0	245	1.000	245 kips	
2	Row 2	0	7	0.0	245	1.000	245 kips	
3	Row 3	0	7	0.0	245	1.000	245 kips	
4	Row 4	0	6	0.0	210	1.000	210 kips	
5	Row 5	0	6	0.0	210	1.000	210 kips	
6	Row 6	0	6	0.0	210	1.000	210 kips	
7	Row 7	0	6	0.0	210	1.000	210 kips	
8	Row 8	0	6	0.0	210	1.000	210 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			51		1785		1785 kips	OK

Case 3
Flood Event Dry
Unusual

Vertical Load, P = 3745 kips
Horizontal Load, H = 57 kips
M_{ML} = 103 kip-ft

	Vertical Pile Loading	P / N	+	M_{ML} * d / Σ I	= Pile Loads		Axial Pile Load
1	Row 1	73.4		0.2	73.6 kips/pile	36.8 tons/pile	36.8 tons/pile
2	Row 2	73.4		0.2	73.6 kips/pile	36.8 tons/pile	36.8 tons/pile
3	Row 3	73.4		0.1	73.5 kips/pile	36.8 tons/pile	36.8 tons/pile
4	Row 4	73.4		0.0	73.5 kips/pile	36.7 tons/pile	36.7 tons/pile
5	Row 5	73.4		0.0	73.4 kips/pile	36.7 tons/pile	36.7 tons/pile
6	Row 6	73.4		-0.1	73.3 kips/pile	36.7 tons/pile	36.7 tons/pile
7	Row 7	73.4		-0.2	73.3 kips/pile	36.6 tons/pile	36.6 tons/pile
8	Row 8	73.4		-0.2	73.2 kips/pile	36.6 tons/pile	36.6 tons/pile
9	Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12	Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
						max: 36.8 tons/pile	max: 36.8 tons/pile

Assumed lateral Capacity: 35.0 kips/pile

	Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance
1	Row 1	0	7	0.0	245	1.000	245 kips
2	Row 2	0	7	0.0	245	1.000	245 kips
3	Row 3	0	7	0.0	245	1.000	245 kips
4	Row 4	0	6	0.0	210	1.000	210 kips
5	Row 5	0	6	0.0	210	1.000	210 kips
6	Row 6	0	6	0.0	210	1.000	210 kips
7	Row 7	0	6	0.0	210	1.000	210 kips
8	Row 8	0	6	0.0	210	1.000	210 kips
9	Row 9	0	0	0.0	0	1.000	0 kips
10	Row 10	0	0	0.0	0	1.000	0 kips
11	Row 11	0	0	0.0	0	1.000	0 kips
12	Row 12	0	0	0.0	0	1.000	0 kips
13	Row 13	0	0	0.0	0	1.000	0 kips
14	Row 14	0	0	0.0	0	1.000	0 kips
15	Row 15	0	0	0.0	0	1.000	0 kips

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51	1785	1785 kips	OK	

Case 4
Flood Event Daily
Usual

Vertical Load, P = 3000 kips
Horizontal Load, H = 0 kips
M_{res} = -452 kip-ft

Vertical Pile Loading	P / N +	M_{res} * d / Σ I	= Pile Loads		Axial Pile Load
1 Row 1	58.8	-0.9	57.9 kips/pile	28.9 tons/pile	28.9 tons/pile
2 Row 2	58.8	-0.7	58.1 kips/pile	29.1 tons/pile	29.1 tons/pile
3 Row 3	58.8	-0.4	58.4 kips/pile	29.2 tons/pile	29.2 tons/pile
4 Row 4	58.8	-0.1	58.7 kips/pile	29.3 tons/pile	29.3 tons/pile
5 Row 5	58.8	0.2	59.0 kips/pile	29.5 tons/pile	29.5 tons/pile
6 Row 6	58.8	0.5	59.3 kips/pile	29.6 tons/pile	29.6 tons/pile
7 Row 7	58.8	0.8	59.6 kips/pile	29.8 tons/pile	29.8 tons/pile
8 Row 8	58.8	1.1	59.9 kips/pile	29.9 tons/pile	29.9 tons/pile
9 Row 9	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10 Row 10	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11 Row 11	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12 Row 12	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13 Row 13	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14 Row 14	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15 Row 15	0.0	0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
			max:	29.9 tons/pile	max: 29.9 tons/pile

Assumed lateral Capacity: 22.0 kips/pile

Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resistance due to Bending, kips	Group Efficiency	Lateral Resitance
1 Row 1	0	7	0.0	154	1.000	154 kips
2 Row 2	0	7	0.0	154	1.000	154 kips
3 Row 3	0	7	0.0	154	1.000	154 kips
4 Row 4	0	6	0.0	132	1.000	132 kips
5 Row 5	0	6	0.0	132	1.000	132 kips
6 Row 6	0	6	0.0	132	1.000	132 kips
7 Row 7	0	6	0.0	132	1.000	132 kips
8 Row 8	0	6	0.0	132	1.000	132 kips
9 Row 9	0	0	0.0	0	1.000	0 kips
10 Row 10	0	0	0.0	0	1.000	0 kips
11 Row 11	0	0	0.0	0	1.000	0 kips
12 Row 12	0	0	0.0	0	1.000	0 kips
13 Row 13	0	0	0.0	0	1.000	0 kips
14 Row 14	0	0	0.0	0	1.000	0 kips
15 Row 15	0	0	0.0	0	1.000	0 kips
		51		1122		1122 kips

OK

BARR ENGINEERING		DATE	2/4/2011	SHEET NO.
		PROJECT NAME	RRN Control Structure	
COMPUTED PKN 2/4/11	CHECKED	SUBMITTED PKN	PROJECT NUMBER SUBJECT ACOE Stability Requirements	

SECTION A

EM 1110-2-2100	Stability Analysis of Concrete Structures	12/1/2005	* Pile-founded structures are not included
EM 1110-2-2200	Gravity Dam Design	6/30/1995	
EM 1110-2-2104	Strength Design for Reinforced-Concrete Hydraulic Structures	6/30/1992	8/20/2003
EM 1110-2-2906	Design of Pile Foundations	1/15/1991	

Load Condition Pobabilities

See 8/21/10, response from COE to critical path check list item:
Follow-up on structural design criteria

Case:	1	2	3
	Usual	Unusual	Unusual
Return Period, yrs:	100	500 + free Bd	
allowable stress increase:	1	33%	33%
Lateral pile deflection, in.:	0.67	0.875	0.67
Ice thickness, ft:	2	2	0

Ice

Piles

Minimum Factor of Safety, Ultimate axial capacity EM 1110-2-2906

	Usual	Unusual	Extreme		Ratios Usual/ Unusual	Usual/ Extreme
compression	2.00	1.50	1.15	verified by load tests	1.333	1.739
tension	2.00	1.50	1.15		1.333	1.739
compression	2.50	1.90	1.40	verified by pile driver analyzer	1.316	1.786
tension	3.00	2.25	1.70		1.333	1.765
compression	3.00	2.25	1.70	not verified by load test	1.333	1.765
tension	3.00	2.25	1.70		1.333	1.765

	Usual	Unusual	Extreme	
EM 1110-2-2906, P. 4-9		1.33	1.75	← increase per allowable stress increase
Vertical displacemet	0.25 in	0.33 in	0.44 in	
Horizonatl Displacement	0.50 in	0.67 in	0.88 in	

Pile Capacity: HP14x73

Case	Usual 1	Unusual 1.1	Unusual 2	Extreme 2.1	Unusual 3	Usual 4	Note
Pile Tip El.	835.8	835.8	835.8	835.8	835.8	835.8	Case 1, 2, 3: Flood & Construction loading using Undrained Analysis
Ultimate Axial capacity:	239.1 k	239.1 k	239.1 k	239.1 k	239.1 k	127.2 k	Case 4: Long-term loading using Drained Analysis
Ultimate Uplift capacity:	162.4 k	162.4 k	162.4 k	162.4 k	162.4 k	30.2 k	Cases 1.1 & 2.1 include ice, use next higher load classification
Horizonatl Displacement:	0.67 in	0.875 in	0.875 in	1.000 in	0.67 in	0.50 in	← Per Corp meeting 2/1/11
Allow. Lateral Capacity:	31 k	35 k	35 k	35 k	30 k	22 k	← FOR PHASE 4 : Use GROUP FOR Lateral capacity
FS:	2.00	1.50	1.50	1.15	1.50	2.00	← Per Corp meeting 2/1/11
Allow. Axial Compression:	119.6 k	159.4 k	159.4 k	207.9 k	159.4 k	63.6 k	
Allow. Axial Tension:	81.2 k	108.3 k	108.3 k	141.2 k	108.3 k	15.1 k	

Pile P-Multipliers, Pm for multiple Row Shading (ave from Hannigan....2005) See AASHTO P. 10-88

Pile Spacing	Row 1	Row 2	Row 3	Row n
3 B	0.7	0.5	0.35	0.35
5 B	1	0.85	0.7	0.7
η Long	0.893	1.000	1.000	1.000
η Trans	0.796	0.613	0.463	0.575
η Total	1.000	1.000	1.000	1.000

FOR PHASE 4 : NO Pm applied for Lateral capacity

B = 1.1666667

Pile Spacing	Row 1	Row 2	Row 3	Row n	
					parallel
Ratio s/B	4.286	4.286	5.143	5.143	
					perp
Ratio s/B	3.643	3.643	3.643	4.286	

		s/B	Formula
Enloft			
trailing Pile		5.37	$0.5791\ (s/B)^{-0.251}$
leading Pile		3.37	$0.7309\ (s/B)^{-0.0707}$
side side		3.28	$0.5292\ (s/B)^{-0.2629}$

Hydraulic Data - RRN Mainstream - Optimized w/Control Structures

1/20/2011

	Structure: Event Year	ND East Red River 35k Headwater ft	Tailwater ft	Case
1/20/2011	2	896.55	896.82	
	5	906.37	902.2	
1/20/2011	10	914.77	903.05	
	20	919.87	903.23	
1/20/2011	50	920.02	902.11	
1/20/2011	100	922.01	902.41	1
	200	916.79	908.37	
	** 300	918.567	910.383	
1/20/2011	500	922.12	914.41	2

**

**

**300 yr event is linearly interpolated between 200 & 500 yr event

ATTACHMENT

F-Q1.3 Group Analyses Results for the Red River (LPP) Control Structure Wing Wall A

RRN_WWA90.gp8d

RRN
WING WALL
SEC A

```
RRN Wingwall A
GROUP8.0.4
* Analysis Parameters
1 3 0 0 0 0 0 1 0 1 0 0
* Pile Cap Dimensions
0.0000000E+00 0.0000000E+00 0.0000000E+00
* Load Cases
2 0 0 1
0 1 0 1 1 100 100
0.0001 0.0001 1 1
Load Case  1
0 0 0 0 1 0 0
2 1 1 0
1   Fx        Fy                  Xc  Yc  Zc
2575000 -1567000 0 0 0 0 0 107.4 180
0
0
0 0 0 1 1 100 100
0.0001 0.0001 1 1
Load Case 1.1 w/ice
0 0 0 0 1 0 0
0 1 1 0
1   Fx        Fy                  Xc  Yc  Zc
2575000 -1582000 0 0 0 0 0 148.56 180
0
0
* Load Combinations
0 0 0 1
* Distributed Load Sets
0
* Pile Group Configuration
51
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 30 27 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 30 78 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 30 129 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 30 180 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 30 231 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 30 282 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 30 333 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 90 27 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 90 78 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 90 129 0 90 -36
2 2 1 0 0 0
```

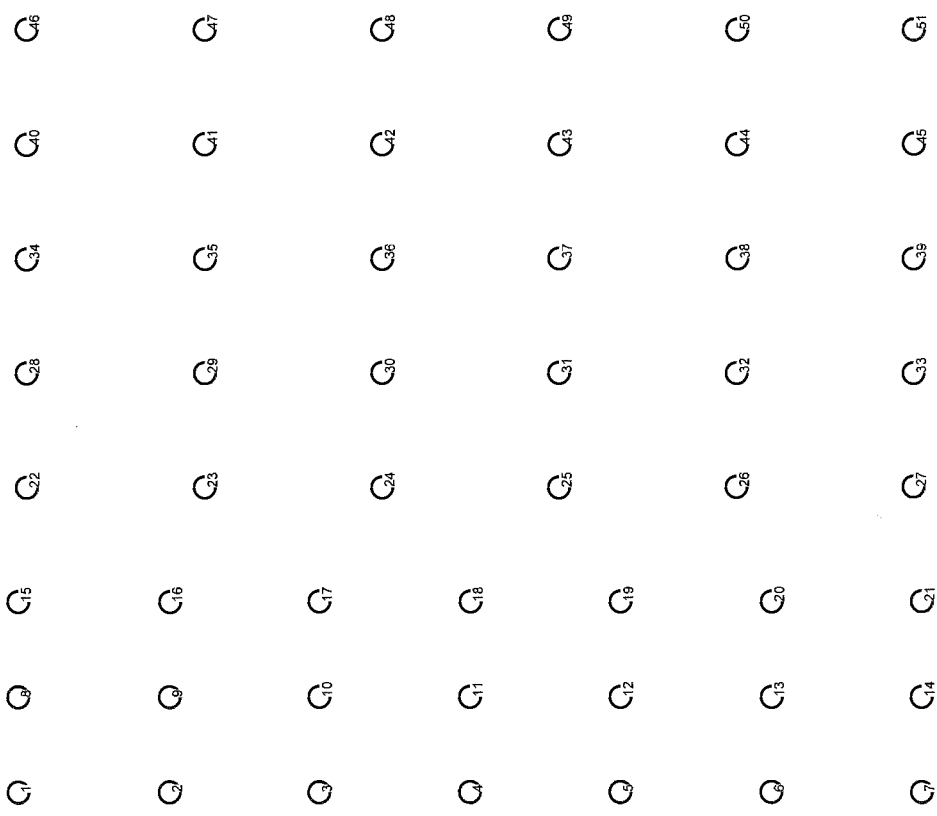
LOADING LOCATION

51 PILES

```
0 0 1 1 1 1 1
-12 90 180 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 90 231 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 90 282 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 90 333 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 27 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 78 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 129 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 180 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 231 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 282 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 333 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 222 30 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 222 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 222 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 222 210 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 222 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 222 330 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 294 30 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 294 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 294 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 294 210 0 90 -36
2 2 1 0 0 0
```

```
0 0 1 1 1 1 1
-12 294 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 294 330 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 366 30 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 366 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 366 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 366 210 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 366 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 366 330 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 438 30 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 438 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 438 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 438 210 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 438 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 438 330 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 510 30 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 510 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 510 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 510 210 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 510 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 510 330 0 90 -36
* Pile Properties
1
0 1
100 900 29000000 1
```

```
0 900 1 0 0
* Pile Cross Sections
1
0 0 4
HP14x89
13.83 0 14.585 13.83 0 0 0.615 0.615
1 26.1 904 326 1808 2.09728E10 0 0 0 0
* Soil Layers
3
0 1
0 324
2 10
0.0235 4.514 0 0 0 0.01 0 0 0 0
0.0235 4.514 0 0 0 0.01 0 0 0 0
1 3
324 384
2 10
0.03449 13.194 0 0 0 0.007 0 0 0 0
0.03449 13.194 0 0 0 0.007 0 0 0 0
2 3
384 1200
2 10
0.03449 13.194 0 0 0 0.007 0 0 0 0
0.03449 13.194 0 0 0 0.007 0 0 0 0
* End of file
```


C46 C47 C48 C49 C50 C51
C40 C41 C42 C43 C44 C45
C34 C35 C36 C37 C38 C39
C28 C29 C30 C31 C32 C33
C22 C23 C24 C25 C26 C27
C15 C16 C17 C18 C19 C20 C21
C8 C9 C10 C11 C12 C13 C14
C1 C2 C3 C4 C5 C6 C7

RRN_WWA90.gp8t

Time and Date of Analysis

Date: February 03, 2011 Time: 13:05:52

***** COMPUTATION RESULTS *****

RRN Wingwall A

***** LOAD CASES RESULTS *****

LOAD CASE : 1
CASE NAME : Load Case 1
LOAD TYPE : Dead, DL

CASE 1 100 YR FLOOD

REDUCTION FACTORS FOR CLOSELY-SPACED PILE GROUPS, COMBINED Y AND Z DIRECTIONS
ESTIMATED USING MOVEMENT IN THE DIRECTION OF PILE CAP DISPLACEMENTS

GROUP NO	P-FACTOR	Y-FACTOR
1	0.9974	1.0000
2	0.9948	1.0000
3	0.9948	1.0000
4	0.9948	1.0000
5	0.9948	1.0000
6	0.9948	1.0000
7	0.9974	1.0000
8	0.8025	1.0000
9	0.7682	1.0000
10	0.7682	1.0000
11	0.7682	1.0000
12	0.7682	1.0000
13	0.7682	1.0000
14	0.8025	1.0000
15	0.8025	1.0000
16	0.7682	1.0000
17	0.7682	1.0000
18	0.7682	1.0000
19	0.7682	1.0000
20	0.7682	1.0000
21	0.8025	1.0000
22	0.8764	1.0000
23	0.8666	1.0000
24	0.8622	1.0000
25	0.8622	1.0000
26	0.8666	1.0000
27	0.8764	1.0000
28	0.8949	1.0000
29	0.8913	1.0000
30	0.8913	1.0000
31	0.8913	1.0000
32	0.8913	1.0000
33	0.8949	1.0000
34	0.8949	1.0000
35	0.8913	1.0000
36	0.8913	1.0000
37	0.8913	1.0000
38	0.8913	1.0000
39	0.8949	1.0000
40	0.8949	1.0000
41	0.8913	1.0000
42	0.8913	1.0000
43	0.8913	1.0000
44	0.8913	1.0000
45	0.8949	1.0000
46	0.8949	1.0000
47	0.8913	1.0000
48	0.8913	1.0000

CASE 2 INFO STARTS ON P. 11

49	0.8913	1.0000
50	0.8913	1.0000
51	0.8949	1.0000

* TABLE L * COMPUTATION ON PILE CAP

* EQUIVALENT CONC. LOAD AT ORIGIN *

VERT. LOAD, LBS	HOR. LOAD Y, LBS	HOR. LOAD Z, LBS
2.57500E+06	-1.56700E+06	0.00000

MOMENT X ,IN-LBS	MOMENT Y,IN-LBS	MOMENT Z,IN-LBS
2.82060E+08	4.63500E+08	-2.76555E+08

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

VERTICAL ,IN	HORIZONTAL Y,IN	HORIZONTAL Z,IN
0.12245	-0.54503	2.00672E-08

ANGLE ROT. X,RAD	ANGLE ROT. Y,RAD	ANGLE ROT. Z,RAD
-7.87989E-11	2.29369E-14	3.11077E-04

* TABLE M * COMPUTATION ON INDIVIDUAL PILE

THE GLOBAL STRUCTURAL COORDINATE SYSTEM

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. X,IN	DISP. Y,IN	DISP. Z,IN	ROT. X,RAD	ROT. Y,RAD	ROT. Z,RAD
1	0.1131	-0.5488	1.7704E-08	-7.8799E-11	2.2937E-14	3.1108E-04
2	0.1131	-0.5488	1.7704E-08	-7.8799E-11	2.2937E-14	3.1108E-04
3	0.1131	-0.5488	1.7704E-08	-7.8799E-11	2.2937E-14	3.1108E-04
4	0.1131	-0.5488	1.7704E-08	-7.8799E-11	2.2937E-14	3.1108E-04
5	0.1131	-0.5488	1.7704E-08	-7.8799E-11	2.2937E-14	3.1108E-04
6	0.1131	-0.5488	1.7704E-08	-7.8799E-11	2.2937E-14	3.1108E-04
7	0.1131	-0.5488	1.7704E-08	-7.8799E-11	2.2937E-14	3.1108E-04
8	9.4449E-02	-0.5488	1.2976E-08	-7.8799E-11	2.2937E-14	3.1108E-04
9	9.4449E-02	-0.5488	1.2976E-08	-7.8799E-11	2.2937E-14	3.1108E-04
10	9.4449E-02	-0.5488	1.2976E-08	-7.8799E-11	2.2937E-14	3.1108E-04
11	9.4449E-02	-0.5488	1.2976E-08	-7.8799E-11	2.2937E-14	3.1108E-04
12	9.4449E-02	-0.5488	1.2976E-08	-7.8799E-11	2.2937E-14	3.1108E-04
13	9.4449E-02	-0.5488	1.2976E-08	-7.8799E-11	2.2937E-14	3.1108E-04
14	9.4449E-02	-0.5488	1.2976E-08	-7.8799E-11	2.2937E-14	3.1108E-04
15	7.5785E-02	-0.5488	8.2477E-09	-7.8799E-11	2.2937E-14	3.1108E-04
16	7.5785E-02	-0.5488	8.2477E-09	-7.8799E-11	2.2937E-14	3.1108E-04
17	7.5785E-02	-0.5488	8.2477E-09	-7.8799E-11	2.2937E-14	3.1108E-04
18	7.5785E-02	-0.5488	8.2477E-09	-7.8799E-11	2.2937E-14	3.1108E-04
19	7.5785E-02	-0.5488	8.2477E-09	-7.8799E-11	2.2937E-14	3.1108E-04
20	7.5785E-02	-0.5488	8.2477E-09	-7.8799E-11	2.2937E-14	3.1108E-04
21	7.5785E-02	-0.5488	8.2477E-09	-7.8799E-11	2.2937E-14	3.1108E-04
22	5.3387E-02	-0.5488	2.5741E-09	-7.8799E-11	2.2937E-14	3.1108E-04
23	5.3387E-02	-0.5488	2.5741E-09	-7.8799E-11	2.2937E-14	3.1108E-04
24	5.3387E-02	-0.5488	2.5741E-09	-7.8799E-11	2.2937E-14	3.1108E-04
25	5.3387E-02	-0.5488	2.5741E-09	-7.8799E-11	2.2937E-14	3.1108E-04
26	5.3387E-02	-0.5488	2.5741E-09	-7.8799E-11	2.2937E-14	3.1108E-04
27	5.3387E-02	-0.5488	2.5741E-09	-7.8799E-11	2.2937E-14	3.1108E-04
28	3.0989E-02	-0.5488	-3.0994E-09	-7.8799E-11	2.2937E-14	3.1108E-04
29	3.0989E-02	-0.5488	-3.0994E-09	-7.8799E-11	2.2937E-14	3.1108E-04
30	3.0989E-02	-0.5488	-3.0994E-09	-7.8799E-11	2.2937E-14	3.1108E-04
31	3.0989E-02	-0.5488	-3.0994E-09	-7.8799E-11	2.2937E-14	3.1108E-04
32	3.0989E-02	-0.5488	-3.0994E-09	-7.8799E-11	2.2937E-14	3.1108E-04
33	3.0989E-02	-0.5488	-3.0994E-09	-7.8799E-11	2.2937E-14	3.1108E-04
34	8.5919E-03	-0.5488	-8.7729E-09	-7.8799E-11	2.2937E-14	3.1108E-04
35	8.5919E-03	-0.5488	-8.7729E-09	-7.8799E-11	2.2937E-14	3.1108E-04
36	8.5919E-03	-0.5488	-8.7729E-09	-7.8799E-11	2.2937E-14	3.1108E-04
37	8.5919E-03	-0.5488	-8.7729E-09	-7.8799E-11	2.2937E-14	3.1108E-04
38	8.5919E-03	-0.5488	-8.7729E-09	-7.8799E-11	2.2937E-14	3.1108E-04
39	8.5919E-03	-0.5488	-8.7729E-09	-7.8799E-11	2.2937E-14	3.1108E-04

40	-1.3806E-02	-0.5488	-1.4446E-08	-7.8799E-11	2.2937E-14	3.1108E-04
41	-1.3806E-02	-0.5488	-1.4446E-08	-7.8799E-11	2.2937E-14	3.1108E-04
42	-1.3806E-02	-0.5488	-1.4446E-08	-7.8799E-11	2.2937E-14	3.1108E-04
43	-1.3806E-02	-0.5488	-1.4446E-08	-7.8799E-11	2.2937E-14	3.1108E-04
44	-1.3806E-02	-0.5488	-1.4446E-08	-7.8799E-11	2.2937E-14	3.1108E-04
45	-1.3806E-02	-0.5488	-1.4446E-08	-7.8799E-11	2.2937E-14	3.1108E-04
46	-3.6203E-02	-0.5488	-2.0120E-08	-7.8799E-11	2.2937E-14	3.1108E-04
47	-3.6203E-02	-0.5488	-2.0120E-08	-7.8799E-11	2.2937E-14	3.1108E-04
48	-3.6203E-02	-0.5488	-2.0120E-08	-7.8799E-11	2.2937E-14	3.1108E-04
49	-3.6203E-02	-0.5488	-2.0120E-08	-7.8799E-11	2.2937E-14	3.1108E-04
50	-3.6203E-02	-0.5488	-2.0120E-08	-7.8799E-11	2.2937E-14	3.1108E-04
51	-3.6203E-02	-0.5488	-2.0120E-08	-7.8799E-11	2.2937E-14	3.1108E-04
MINIMUM	-3.6203E-02	-0.5488	-2.0120E-08	-7.8799E-11	2.2937E-14	3.1108E-04
Pile N.	46	1	46	1	1	1
MAXIMUM	0.1131	-0.5488	1.7704E-08	-7.8799E-11	2.2937E-14	3.1108E-04
Pile N.	1	1	1	1	1	1

* PILE TOP REACTIONS * Axial — Horiz

PILE GROUP	FOR. X,LBS	FOR. Y,LBS	FOR. Z,LBS	MOM X,LBS-IN	MOM Y,LBS-IN	MOM Z,LBS-IN	STRESS,LBS/IN**2
1	1.2317E+05	-3.3566E+04	7.4621E-04	-7.1459E-10	-4.2804E-02	-2.3170E+06	2.2443E+04
2	1.2317E+05	-3.3506E+04	7.4485E-04	-7.1459E-10	-4.2752E-02	-2.3141E+06	2.2421E+04
3	1.2317E+05	-3.3506E+04	7.4485E-04	-7.1459E-10	-4.2752E-02	-2.3141E+06	2.2421E+04
4	1.2317E+05	-3.3506E+04	7.4485E-04	-7.1459E-10	-4.2752E-02	-2.3141E+06	2.2421E+04
5	1.2317E+05	-3.3506E+04	7.4485E-04	-7.1459E-10	-4.2752E-02	-2.3141E+06	2.2421E+04
6	1.2317E+05	-3.3506E+04	7.4485E-04	-7.1459E-10	-4.2752E-02	-2.3141E+06	2.2421E+04
7	1.2317E+05	-3.3566E+04	7.4621E-04	-7.1459E-10	-4.2804E-02	-2.3170E+06	2.2443E+04
8	1.0656E+05	-2.8865E+04	4.7253E-04	-7.1459E-10	-2.8383E-02	-2.0848E+06	2.0030E+04
9	1.0656E+05	-2.8003E+04	4.5886E-04	-7.1459E-10	-2.7822E-02	-2.0421E+06	1.9703E+04
10	1.0656E+05	-2.8003E+04	4.5886E-04	-7.1459E-10	-2.7822E-02	-2.0421E+06	1.9703E+04
11	1.0656E+05	-2.8003E+04	4.5886E-04	-7.1459E-10	-2.7822E-02	-2.0421E+06	1.9703E+04
12	1.0656E+05	-2.8003E+04	4.5886E-04	-7.1459E-10	-2.7822E-02	-2.0421E+06	1.9703E+04
13	1.0656E+05	-2.8003E+04	4.5886E-04	-7.1459E-10	-2.7822E-02	-2.0421E+06	1.9703E+04
14	1.0656E+05	-2.8865E+04	4.7253E-04	-7.1459E-10	-2.8383E-02	-2.0848E+06	2.0030E+04
15	8.9943E+04	-2.8906E+04	3.0112E-04	-7.1459E-10	-1.8042E-02	-2.0848E+06	1.9393E+04
16	8.9943E+04	-2.8043E+04	2.9238E-04	-7.1459E-10	-1.7684E-02	-2.0420E+06	1.9066E+04
17	8.9943E+04	-2.8043E+04	2.9238E-04	-7.1459E-10	-1.7684E-02	-2.0420E+06	1.9066E+04
18	8.9943E+04	-2.8043E+04	2.9238E-04	-7.1459E-10	-1.7684E-02	-2.0420E+06	1.9066E+04
19	8.9943E+04	-2.8043E+04	2.9238E-04	-7.1459E-10	-1.7684E-02	-2.0420E+06	1.9066E+04
20	8.9943E+04	-2.8043E+04	2.9238E-04	-7.1459E-10	-1.7684E-02	-2.0420E+06	1.9066E+04
21	8.9943E+04	-2.8906E+04	3.0112E-04	-7.1459E-10	-1.8042E-02	-2.0848E+06	1.9393E+04
22	6.9869E+04	-3.0797E+04	1.0008E-04	-7.1459E-10	-5.8619E-03	-2.1763E+06	1.9324E+04
23	6.9869E+04	-3.0557E+04	9.9297E-05	-7.1459E-10	-5.8314E-03	-2.1645E+06	1.9234E+04
24	6.9869E+04	-3.0449E+04	9.8948E-05	-7.1459E-10	-5.8176E-03	-2.1592E+06	1.9193E+04
25	6.9869E+04	-3.0449E+04	9.8948E-05	-7.1459E-10	-5.8176E-03	-2.1592E+06	1.9193E+04
26	6.9869E+04	-3.0557E+04	9.9297E-05	-7.1459E-10	-5.8314E-03	-2.1645E+06	1.9234E+04
27	6.9869E+04	-3.0797E+04	1.0008E-04	-7.1459E-10	-5.8619E-03	-2.1763E+06	1.9324E+04
28	4.0257E+04	-3.1321E+04	-1.2294E-04	-7.1459E-10	7.1398E-03	-2.1979E+06	1.8355E+04
29	4.0257E+04	-3.1234E+04	-1.2257E-04	-7.1459E-10	7.1261E-03	-2.1938E+06	1.8323E+04
30	4.0257E+04	-3.1234E+04	-1.2257E-04	-7.1459E-10	7.1261E-03	-2.1938E+06	1.8323E+04
31	4.0257E+04	-3.1234E+04	-1.2257E-04	-7.1459E-10	7.1261E-03	-2.1938E+06	1.8323E+04
32	4.0257E+04	-3.1234E+04	-1.2257E-04	-7.1459E-10	7.1261E-03	-2.1938E+06	1.8323E+04
33	4.0257E+04	-3.1321E+04	-1.2294E-04	-7.1459E-10	7.1398E-03	-2.1979E+06	1.8355E+04
34	1.1048E+04	-3.1395E+04	-3.4925E-04	-7.1459E-10	2.0203E-02	-2.1979E+06	1.7235E+04
35	1.1048E+04	-3.1308E+04	-3.4825E-04	-7.1459E-10	2.0164E-02	-2.1936E+06	1.7203E+04
36	1.1048E+04	-3.1308E+04	-3.4825E-04	-7.1459E-10	2.0164E-02	-2.1936E+06	1.7203E+04
37	1.1048E+04	-3.1308E+04	-3.4825E-04	-7.1459E-10	2.0164E-02	-2.1936E+06	1.7203E+04
38	1.1048E+04	-3.1308E+04	-3.4825E-04	-7.1459E-10	2.0164E-02	-2.1936E+06	1.7203E+04
39	1.1048E+04	-3.1395E+04	-3.4925E-04	-7.1459E-10	2.0203E-02	-2.1979E+06	1.7235E+04
40	-1.7814E+04	-3.1467E+04	-5.7734E-04	-7.1459E-10	3.3269E-02	-2.1977E+06	1.7494E+04
41	-1.7814E+04	-3.1382E+04	-5.7580E-04	-7.1459E-10	3.3208E-02	-2.1937E+06	1.7463E+04
42	-1.7814E+04	-3.1382E+04	-5.7580E-04	-7.1459E-10	3.3208E-02	-2.1937E+06	1.7463E+04
43	-1.7814E+04	-3.1382E+04	-5.7580E-04	-7.1459E-10	3.3208E-02	-2.1937E+06	1.7463E+04
44	-1.7814E+04	-3.1382E+04	-5.7580E-04	-7.1459E-10	3.3208E-02	-2.1937E+06	1.7463E+04
45	-1.7814E+04	-3.1467E+04	-5.7734E-04	-7.1459E-10	3.3269E-02	-2.1977E+06	1.7494E+04
46	-4.7139E+04	-3.1542E+04	-8.0744E-04	-7.1459E-10	4.6342E-02	-2.1977E+06	1.8617E+04
47	-4.7139E+04	-3.1455E+04	-8.0521E-04	-7.1459E-10	4.6256E-02	-2.1935E+06	1.8585E+04
48	-4.7139E+04	-3.1455E+04	-8.0521E-04	-7.1459E-10	4.6256E-02	-2.1935E+06	1.8585E+04
49	-4.7139E+04	-3.1455E+04	-8.0521E-04	-7.1459E-10	4.6256E-02	-2.1935E+06	1.8585E+04
50	-4.7139E+04	-3.1455E+04	-8.0521E-04	-7.1459E-10	4.6256E-02	-2.1935E+06	1.8585E+04
51	-4.7139E+04	-3.1542E+04	-8.0744E-04	-7.1459E-10	4.6342E-02	-2.1977E+06	1.8617E+04
MINIMUM	-4.7139E+04	-3.3566E+04	-8.0744E-04	-7.1459E-10	-4.2804E-02	-2.3170E+06	1.7203E+04
Pile N.	46	1	46	1	1	1	35
MAXIMUM	1.2317E+05	-2.8003E+04	7.4621E-04	-7.1459E-10	4.6342E-02	-2.0420E+06	2.2443E+04
Pile N.	1	9	1	1	46	16	1

THE PILE COORDINATE SYSTEM (LOCAL AXES)

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. x,IN	DISP. y,IN	DISP. z,IN	ROT. x,RAD	ROT. y,RAD	ROT. z,RAD
1	0.1131	-0.5488	1.7704E-08	-7.8799E-11	2.2937E-14	3.1108E-04
2	0.1131	-0.5488	1.7704E-08	-7.8799E-11	2.2937E-14	3.1108E-04
3	0.1131	-0.5488	1.7704E-08	-7.8799E-11	2.2937E-14	3.1108E-04
4	0.1131	-0.5488	1.7704E-08	-7.8799E-11	2.2937E-14	3.1108E-04
5	0.1131	-0.5488	1.7704E-08	-7.8799E-11	2.2937E-14	3.1108E-04
6	0.1131	-0.5488	1.7704E-08	-7.8799E-11	2.2937E-14	3.1108E-04
7	0.1131	-0.5488	1.7704E-08	-7.8799E-11	2.2937E-14	3.1108E-04
8	9.4449E-02	-0.5488	1.2976E-08	-7.8799E-11	2.2937E-14	3.1108E-04
9	9.4449E-02	-0.5488	1.2976E-08	-7.8799E-11	2.2937E-14	3.1108E-04
10	9.4449E-02	-0.5488	1.2976E-08	-7.8799E-11	2.2937E-14	3.1108E-04
11	9.4449E-02	-0.5488	1.2976E-08	-7.8799E-11	2.2937E-14	3.1108E-04
12	9.4449E-02	-0.5488	1.2976E-08	-7.8799E-11	2.2937E-14	3.1108E-04
13	9.4449E-02	-0.5488	1.2976E-08	-7.8799E-11	2.2937E-14	3.1108E-04
14	9.4449E-02	-0.5488	1.2976E-08	-7.8799E-11	2.2937E-14	3.1108E-04
15	7.5785E-02	-0.5488	8.2477E-09	-7.8799E-11	2.2937E-14	3.1108E-04
16	7.5785E-02	-0.5488	8.2477E-09	-7.8799E-11	2.2937E-14	3.1108E-04
17	7.5785E-02	-0.5488	8.2477E-09	-7.8799E-11	2.2937E-14	3.1108E-04
18	7.5785E-02	-0.5488	8.2477E-09	-7.8799E-11	2.2937E-14	3.1108E-04
19	7.5785E-02	-0.5488	8.2477E-09	-7.8799E-11	2.2937E-14	3.1108E-04
20	7.5785E-02	-0.5488	8.2477E-09	-7.8799E-11	2.2937E-14	3.1108E-04
21	7.5785E-02	-0.5488	8.2477E-09	-7.8799E-11	2.2937E-14	3.1108E-04
22	5.3387E-02	-0.5488	2.5741E-09	-7.8799E-11	2.2937E-14	3.1108E-04
23	5.3387E-02	-0.5488	2.5741E-09	-7.8799E-11	2.2937E-14	3.1108E-04
24	5.3387E-02	-0.5488	2.5741E-09	-7.8799E-11	2.2937E-14	3.1108E-04
25	5.3387E-02	-0.5488	2.5741E-09	-7.8799E-11	2.2937E-14	3.1108E-04
26	5.3387E-02	-0.5488	2.5741E-09	-7.8799E-11	2.2937E-14	3.1108E-04
27	5.3387E-02	-0.5488	2.5741E-09	-7.8799E-11	2.2937E-14	3.1108E-04
28	3.0989E-02	-0.5488	-3.0994E-09	-7.8799E-11	2.2937E-14	3.1108E-04
29	3.0989E-02	-0.5488	-3.0994E-09	-7.8799E-11	2.2937E-14	3.1108E-04
30	3.0989E-02	-0.5488	-3.0994E-09	-7.8799E-11	2.2937E-14	3.1108E-04
31	3.0989E-02	-0.5488	-3.0994E-09	-7.8799E-11	2.2937E-14	3.1108E-04
32	3.0989E-02	-0.5488	-3.0994E-09	-7.8799E-11	2.2937E-14	3.1108E-04
33	3.0989E-02	-0.5488	-3.0994E-09	-7.8799E-11	2.2937E-14	3.1108E-04
34	8.5919E-03	-0.5488	-8.7729E-09	-7.8799E-11	2.2937E-14	3.1108E-04
35	8.5919E-03	-0.5488	-8.7729E-09	-7.8799E-11	2.2937E-14	3.1108E-04
36	8.5919E-03	-0.5488	-8.7729E-09	-7.8799E-11	2.2937E-14	3.1108E-04
37	8.5919E-03	-0.5488	-8.7729E-09	-7.8799E-11	2.2937E-14	3.1108E-04
38	8.5919E-03	-0.5488	-8.7729E-09	-7.8799E-11	2.2937E-14	3.1108E-04
39	8.5919E-03	-0.5488	-8.7729E-09	-7.8799E-11	2.2937E-14	3.1108E-04
40	-1.3806E-02	-0.5488	-1.4446E-08	-7.8799E-11	2.2937E-14	3.1108E-04
41	-1.3806E-02	-0.5488	-1.4446E-08	-7.8799E-11	2.2937E-14	3.1108E-04
42	-1.3806E-02	-0.5488	-1.4446E-08	-7.8799E-11	2.2937E-14	3.1108E-04
43	-1.3806E-02	-0.5488	-1.4446E-08	-7.8799E-11	2.2937E-14	3.1108E-04
44	-1.3806E-02	-0.5488	-1.4446E-08	-7.8799E-11	2.2937E-14	3.1108E-04
45	-1.3806E-02	-0.5488	-1.4446E-08	-7.8799E-11	2.2937E-14	3.1108E-04
46	-3.6203E-02	-0.5488	-2.0120E-08	-7.8799E-11	2.2937E-14	3.1108E-04
47	-3.6203E-02	-0.5488	-2.0120E-08	-7.8799E-11	2.2937E-14	3.1108E-04
48	-3.6203E-02	-0.5488	-2.0120E-08	-7.8799E-11	2.2937E-14	3.1108E-04
49	-3.6203E-02	-0.5488	-2.0120E-08	-7.8799E-11	2.2937E-14	3.1108E-04
50	-3.6203E-02	-0.5488	-2.0120E-08	-7.8799E-11	2.2937E-14	3.1108E-04
51	-3.6203E-02	-0.5488	-2.0120E-08	-7.8799E-11	2.2937E-14	3.1108E-04
MINIMUM	-3.6203E-02	-0.5488	-2.0120E-08	-7.8799E-11	2.2937E-14	3.1108E-04
Pile N.	46	1	46	1	1	1
MAXIMUM	0.1131	-0.5488	1.7704E-08	-7.8799E-11	2.2937E-14	3.1108E-04
Pile N.	1	1	1	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	AXIAL,LBS	LAT. y,LBS	LAT. z,LBS	MOM x,LBS-IN	MOM y,LBS-IN	MOM z,LBS-IN	STRESS,LBS/IN**2
1	1.2317E+05	-3.3566E+04	7.4621E-04	-7.1459E-10	-4.2804E-02	-2.3170E+06	2.2443E+04
2	1.2317E+05	-3.3506E+04	7.4485E-04	-7.1459E-10	-4.2752E-02	-2.3141E+06	2.2421E+04
3	1.2317E+05	-3.3506E+04	7.4485E-04	-7.1459E-10	-4.2752E-02	-2.3141E+06	2.2421E+04
4	1.2317E+05	-3.3506E+04	7.4485E-04	-7.1459E-10	-4.2752E-02	-2.3141E+06	2.2421E+04
5	1.2317E+05	-3.3506E+04	7.4485E-04	-7.1459E-10	-4.2752E-02	-2.3141E+06	2.2421E+04
6	1.2317E+05	-3.3506E+04	7.4485E-04	-7.1459E-10	-4.2752E-02	-2.3141E+06	2.2421E+04
7	1.2317E+05	-3.3566E+04	7.4621E-04	-7.1459E-10	-4.2804E-02	-2.3170E+06	2.2443E+04
8	1.0656E+05	-2.8865E+04	4.7253E-04	-7.1459E-10	-2.8383E-02	-2.0848E+06	2.0030E+04
9	1.0656E+05	-2.8003E+04	4.5886E-04	-7.1459E-10	-2.7822E-02	-2.0421E+06	1.9703E+04
10	1.0656E+05	-2.8003E+04	4.5886E-04	-7.1459E-10	-2.7822E-02	-2.0421E+06	1.9703E+04

11	1.0656E+05	-2.8003E+04	4.5886E-04	-7.1459E-10	-2.7822E-02	-2.0421E+06	1.9703E+04
12	1.0656E+05	-2.8003E+04	4.5886E-04	-7.1459E-10	-2.7822E-02	-2.0421E+06	1.9703E+04
13	1.0656E+05	-2.8003E+04	4.5886E-04	-7.1459E-10	-2.7822E-02	-2.0421E+06	1.9703E+04
14	1.0656E+05	-2.8865E+04	4.7253E-04	-7.1459E-10	-2.8383E-02	-2.0848E+06	2.0030E+04
15	8.9943E+04	-2.8906E+04	3.0112E-04	-7.1459E-10	-1.8042E-02	-2.0848E+06	1.9393E+04
16	8.9943E+04	-2.8043E+04	2.9238E-04	-7.1459E-10	-1.7684E-02	-2.0420E+06	1.9066E+04
17	8.9943E+04	-2.8043E+04	2.9238E-04	-7.1459E-10	-1.7684E-02	-2.0420E+06	1.9066E+04
18	8.9943E+04	-2.8043E+04	2.9238E-04	-7.1459E-10	-1.7684E-02	-2.0420E+06	1.9066E+04
19	8.9943E+04	-2.8043E+04	2.9238E-04	-7.1459E-10	-1.7684E-02	-2.0420E+06	1.9066E+04
20	8.9943E+04	-2.8043E+04	2.9238E-04	-7.1459E-10	-1.7684E-02	-2.0420E+06	1.9066E+04
21	8.9943E+04	-2.8906E+04	3.0112E-04	-7.1459E-10	-1.8042E-02	-2.0848E+06	1.9393E+04
22	6.9869E+04	-3.0797E+04	1.0008E-04	-7.1459E-10	-5.8619E-03	-2.1763E+06	1.9324E+04
23	6.9869E+04	-3.0557E+04	9.9297E-05	-7.1459E-10	-5.8314E-03	-2.1645E+06	1.9234E+04
24	6.9869E+04	-3.0449E+04	9.8948E-05	-7.1459E-10	-5.8176E-03	-2.1592E+06	1.9193E+04
25	6.9869E+04	-3.0449E+04	9.8948E-05	-7.1459E-10	-5.8176E-03	-2.1592E+06	1.9193E+04
26	6.9869E+04	-3.0557E+04	9.9297E-05	-7.1459E-10	-5.8314E-03	-2.1645E+06	1.9234E+04
27	6.9869E+04	-3.0797E+04	1.0008E-04	-7.1459E-10	-5.8619E-03	-2.1763E+06	1.9324E+04
28	4.0257E+04	-3.1321E+04	-1.2294E-04	-7.1459E-10	7.1398E-03	-2.1979E+06	1.8355E+04
29	4.0257E+04	-3.1234E+04	-1.2257E-04	-7.1459E-10	7.1261E-03	-2.1938E+06	1.8323E+04
30	4.0257E+04	-3.1234E+04	-1.2257E-04	-7.1459E-10	7.1261E-03	-2.1938E+06	1.8323E+04
31	4.0257E+04	-3.1234E+04	-1.2257E-04	-7.1459E-10	7.1261E-03	-2.1938E+06	1.8323E+04
32	4.0257E+04	-3.1234E+04	-1.2257E-04	-7.1459E-10	7.1261E-03	-2.1938E+06	1.8323E+04
33	4.0257E+04	-3.1321E+04	-1.2294E-04	-7.1459E-10	7.1398E-03	-2.1979E+06	1.8355E+04
34	1.1048E+04	-3.1395E+04	-3.4925E-04	-7.1459E-10	2.0203E-02	-2.1979E+06	1.7235E+04
35	1.1048E+04	-3.1308E+04	-3.4825E-04	-7.1459E-10	2.0164E-02	-2.1936E+06	1.7203E+04
36	1.1048E+04	-3.1308E+04	-3.4825E-04	-7.1459E-10	2.0164E-02	-2.1936E+06	1.7203E+04
37	1.1048E+04	-3.1308E+04	-3.4825E-04	-7.1459E-10	2.0164E-02	-2.1936E+06	1.7203E+04
38	1.1048E+04	-3.1308E+04	-3.4825E-04	-7.1459E-10	2.0164E-02	-2.1936E+06	1.7203E+04
39	1.1048E+04	-3.1395E+04	-3.4925E-04	-7.1459E-10	2.0203E-02	-2.1979E+06	1.7235E+04
40	-1.7814E+04	-3.1467E+04	-5.7734E-04	-7.1459E-10	3.3269E-02	-2.1977E+06	1.7494E+04
41	-1.7814E+04	-3.1382E+04	-5.7580E-04	-7.1459E-10	3.3208E-02	-2.1937E+06	1.7463E+04
42	-1.7814E+04	-3.1382E+04	-5.7580E-04	-7.1459E-10	3.3208E-02	-2.1937E+06	1.7463E+04
43	-1.7814E+04	-3.1382E+04	-5.7580E-04	-7.1459E-10	3.3208E-02	-2.1937E+06	1.7463E+04
44	-1.7814E+04	-3.1382E+04	-5.7580E-04	-7.1459E-10	3.3208E-02	-2.1937E+06	1.7463E+04
45	-1.7814E+04	-3.1467E+04	-5.7734E-04	-7.1459E-10	3.3269E-02	-2.1977E+06	1.7494E+04
46	-4.7139E+04	-3.1542E+04	-8.0744E-04	-7.1459E-10	4.6342E-02	-2.1977E+06	1.8617E+04
47	-4.7139E+04	-3.1455E+04	-8.0521E-04	-7.1459E-10	4.6256E-02	-2.1935E+06	1.8585E+04
48	-4.7139E+04	-3.1455E+04	-8.0521E-04	-7.1459E-10	4.6256E-02	-2.1935E+06	1.8585E+04
49	-4.7139E+04	-3.1455E+04	-8.0521E-04	-7.1459E-10	4.6256E-02	-2.1935E+06	1.8585E+04
50	-4.7139E+04	-3.1455E+04	-8.0521E-04	-7.1459E-10	4.6256E-02	-2.1935E+06	1.8585E+04
51	-4.7139E+04	-3.1542E+04	-8.0744E-04	-7.1459E-10	4.6342E-02	-2.1977E+06	1.8617E+04
MINIMUM	-4.7139E+04	-3.3566E+04	-8.0744E-04	-7.1459E-10	-4.2804E-02	-2.3170E+06	1.7203E+04
Pile N.	46	1	46	1	1	1	35
MAXIMUM	1.2317E+05	-2.8003E+04	7.4621E-04	-7.1459E-10	4.6342E-02	-2.0420E+06	2.2443E+04
Pile N.	1	9	1	1	46	16	1

* EFFECTS FOR LATERALLY LOADED PILE *

* MINIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION		BENDING MOMENT		SHEAR FORCE		SOIL REACTION	
TOTAL	FLEXURAL RIGIDITY							
	y-DIR	z-DIR	z-DIR	y-DIR	y-DIR	z-DIR	y-DIR	z-DIR
STRESS	z-DIR	y-DIR						
	IN	IN	LBS-IN	LBS-IN	LBS	LBS	LBS/IN	LBS/IN
LBS/IN**2	LBS-IN**2	LBS-IN**2						
*****	**********	**********	**********	**********	**********	**********	**********	**********
**********	**********	**********						
1	-0.5490	-1.1000E-10	-7.3800E+05	-4.2800E-02	-3.3600E+04	-1.4900E-04	-248.00	-7.4900E-06
4720.0	2.6200E+10	9.4500E+09						
x(IN)	0.0000	234.00	171.00	135.00	0.0000	207.00	108.00	234.00
459.00	0.0000	0.0000						
2	-0.5490	-1.1500E-10	-7.3800E+05	-4.2800E-02	-3.3600E+04	-1.4800E-04	-247.00	-7.0700E-06
4720.0	2.6200E+10	9.4500E+09						
x(IN)	0.0000	234.00	171.00	135.00	0.0000	207.00	108.00	234.00
459.00	0.0000	0.0000						
3	-0.5490	-1.1500E-10	-7.3800E+05	-4.2800E-02	-3.3600E+04	-1.4800E-04	-247.00	-7.0700E-06
4720.0	2.6200E+10	9.4500E+09						
x(IN)	0.0000	234.00	171.00	135.00	0.0000	207.00	108.00	234.00
459.00	0.0000	0.0000						
4	-0.5490	-1.1500E-10	-7.3800E+05	-4.2800E-02	-3.3600E+04	-1.4800E-04	-247.00	-7.0700E-06
4720.0	2.6200E+10	9.4500E+09						
x(IN)	0.0000	234.00	171.00	135.00	0.0000	207.00	108.00	234.00
459.00	0.0000	0.0000						
5	-0.5490	-1.1500E-10	-7.3800E+05	-4.2800E-02	-3.3600E+04	-1.4800E-04	-247.00	-7.0700E-06
4720.0	2.6200E+10	9.4500E+09						
x(IN)	0.0000	234.00	171.00	135.00	0.0000	207.00	108.00	234.00
459.00	0.0000	0.0000						

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      6      -0.5490 -1.1500E-10 -7.3800E+05 -4.2800E-02 -3.3600E+04 -1.4800E-04     -247.00 -7.0700E-06
4720.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      234.00      171.00      135.00      0.0000      207.00      108.00      234.00
459.00      0.0000      0.0000
      7      -0.5490 -1.1000E-10 -7.3800E+05 -4.2800E-02 -3.3600E+04 -1.4900E-04     -248.00 -7.4900E-06
4720.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      234.00      171.00      135.00      0.0000      207.00      108.00      234.00
459.00      0.0000      0.0000
      8      -0.5490 -8.3100E-11 -6.6400E+05 -2.8400E-02 -2.8900E+04 -9.1700E-05     -205.00 -4.1700E-06
4080.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      243.00      180.00      144.00      0.0000      216.00      108.00      252.00
468.00      0.0000      0.0000
      9      -0.5490 -5.9400E-11 -6.5200E+05 -2.7800E-02 -2.8000E+04 -9.1700E-05     -197.00 -5.9800E-06
4080.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
477.00      0.0000      0.0000
     10      -0.5490 -5.9400E-11 -6.5200E+05 -2.7800E-02 -2.8000E+04 -9.1700E-05     -197.00 -5.9800E-06
4080.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
477.00      0.0000      0.0000
     11      -0.5490 -5.9400E-11 -6.5200E+05 -2.7800E-02 -2.8000E+04 -9.1700E-05     -197.00 -5.9800E-06
4080.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
477.00      0.0000      0.0000
     12      -0.5490 -5.9400E-11 -6.5200E+05 -2.7800E-02 -2.8000E+04 -9.1700E-05     -197.00 -5.9800E-06
4080.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
477.00      0.0000      0.0000
     13      -0.5490 -5.9400E-11 -6.5200E+05 -2.7800E-02 -2.8000E+04 -9.1700E-05     -197.00 -5.9800E-06
4080.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
477.00      0.0000      0.0000
     14      -0.5490 -8.3100E-11 -6.6400E+05 -2.8400E-02 -2.8900E+04 -9.1700E-05     -205.00 -4.1700E-06
4080.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      243.00      180.00      144.00      0.0000      216.00      108.00      252.00
468.00      0.0000      0.0000
     15      -0.5490 -5.0700E-11 -6.6400E+05 -1.8000E-02 -2.8900E+04 -5.8100E-05     -205.00 -2.7800E-06
3450.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      243.00      180.00      144.00      0.0000      216.00      108.00      252.00
477.00      0.0000      0.0000
     16      -0.5490 -4.0500E-11 -6.5100E+05 -1.7700E-02 -2.8100E+04 -5.7700E-05     -197.00 -3.5200E-06
3450.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
468.00      0.0000      0.0000
     17      -0.5490 -4.0500E-11 -6.5100E+05 -1.7700E-02 -2.8100E+04 -5.7700E-05     -197.00 -3.5200E-06
3450.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
477.00      0.0000      0.0000
     18      -0.5490 -4.0500E-11 -6.5100E+05 -1.7700E-02 -2.8100E+04 -5.7700E-05     -197.00 -3.5200E-06
3450.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
468.00      0.0000      0.0000
     19      -0.5490 -4.0500E-11 -6.5100E+05 -1.7700E-02 -2.8100E+04 -5.7700E-05     -197.00 -3.5200E-06
3450.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
468.00      0.0000      0.0000
     20      -0.5490 -4.0500E-11 -6.5100E+05 -1.7700E-02 -2.8100E+04 -5.7700E-05     -197.00 -3.5200E-06
3450.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      252.00      180.00      144.00      0.0000      225.00      108.00      252.00
477.00      0.0000      0.0000
     21      -0.5490 -5.0700E-11 -6.6400E+05 -1.8000E-02 -2.8900E+04 -5.8100E-05     -205.00 -2.7800E-06
3450.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      243.00      180.00      144.00      0.0000      216.00      108.00      252.00
477.00      0.0000      0.0000
     22      -0.5490 -1.5100E-11 -6.9300E+05 -5.8600E-03 -3.0800E+04 -1.9500E-05     -221.00 -1.0000E-06
2680.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      243.00      171.00      144.00      0.0000      216.00      108.00      243.00
459.00      0.0000      0.0000
     23      -0.5490 -1.7300E-11 -6.8900E+05 -5.8300E-03 -3.0600E+04 -1.9000E-05     -219.00 -7.8200E-07
2680.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      243.00      171.00      144.00      0.0000      216.00      108.00      243.00
459.00      0.0000      0.0000
     24      -0.5490 -1.7900E-11 -6.8700E+05 -5.8200E-03 -3.0500E+04 -1.8900E-05     -218.00 -7.1000E-07
2680.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      243.00      171.00      144.00      0.0000      216.00      108.00      243.00
459.00      0.0000      0.0000
     25      -0.5490 -1.7900E-11 -6.8700E+05 -5.8200E-03 -3.0500E+04 -1.8900E-05     -218.00 -7.1000E-07
2680.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      243.00      171.00      144.00      0.0000      216.00      108.00      243.00
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468.00      0.0000      0.0000
      26      -0.5490 -1.7300E-11 -6.8900E+05 -5.8300E-03 -3.0600E+04 -1.9000E-05     -219.00 -7.8200E-07
2680.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      171.00      144.00      0.0000      216.00      108.00      243.00
459.00      0.0000      0.0000
      27      -0.5490 -1.5100E-11 -6.9300E+05 -5.8600E-03 -3.0800E+04 -1.9500E-05     -221.00 -1.0000E-06
2680.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      243.00      171.00      144.00      0.0000      216.00      108.00      243.00
459.00      0.0000      0.0000
      28      -0.5490 -3.1000E-09 -6.9900E+05 -2.0300E-03 -3.1300E+04 -1.2300E-04     -225.00 -1.0800E-06
1540.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
      29      -0.5490 -3.1000E-09 -6.9700E+05 -2.0200E-03 -3.1200E+04 -1.2300E-04     -225.00 -1.0800E-06
1540.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
      30      -0.5490 -3.1000E-09 -6.9700E+05 -2.0200E-03 -3.1200E+04 -1.2300E-04     -225.00 -1.0800E-06
1540.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
      31      -0.5490 -3.1000E-09 -6.9700E+05 -2.0200E-03 -3.1200E+04 -1.2300E-04     -225.00 -1.0800E-06
1540.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
      32      -0.5490 -3.1000E-09 -6.9700E+05 -2.0200E-03 -3.1200E+04 -1.2300E-04     -225.00 -1.0800E-06
1540.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
      33      -0.5490 -3.1000E-09 -6.9900E+05 -2.0300E-03 -3.1300E+04 -1.2300E-04     -225.00 -1.0800E-06
1540.0  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
      34      -0.5490 -8.7700E-09 -6.9700E+05 -5.7100E-03 -3.1400E+04 -3.4900E-04     -226.00 -3.0600E-06
423.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
      35      -0.5490 -8.7700E-09 -6.9600E+05 -5.6900E-03 -3.1300E+04 -3.4800E-04     -225.00 -3.0500E-06
423.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
      36      -0.5490 -8.7700E-09 -6.9600E+05 -5.6900E-03 -3.1300E+04 -3.4800E-04     -225.00 -3.0500E-06
423.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
      37      -0.5490 -8.7700E-09 -6.9600E+05 -5.6900E-03 -3.1300E+04 -3.4800E-04     -225.00 -3.0500E-06
423.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
      38      -0.5490 -8.7700E-09 -6.9600E+05 -5.6900E-03 -3.1300E+04 -3.4800E-04     -225.00 -3.0500E-06
423.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
      39      -0.5490 -8.7700E-09 -6.9700E+05 -5.7100E-03 -3.1400E+04 -3.4900E-04     -226.00 -3.0600E-06
423.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
      40      -0.5490 -1.4400E-08 -6.9600E+05 -9.3500E-03 -3.1500E+04 -5.7700E-04     -226.00 -5.0300E-06
683.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
      41      -0.5490 -1.4400E-08 -6.9500E+05 -9.3400E-03 -3.1400E+04 -5.7600E-04     -225.00 -5.0200E-06
683.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
      42      -0.5490 -1.4400E-08 -6.9500E+05 -9.3400E-03 -3.1400E+04 -5.7600E-04     -225.00 -5.0200E-06
683.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
      43      -0.5490 -1.4400E-08 -6.9500E+05 -9.3400E-03 -3.1400E+04 -5.7600E-04     -225.00 -5.0200E-06
683.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
      44      -0.5490 -1.4400E-08 -6.9500E+05 -9.3400E-03 -3.1400E+04 -5.7600E-04     -225.00 -5.0200E-06
683.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
      45      -0.5490 -1.4400E-08 -6.9600E+05 -9.3500E-03 -3.1500E+04 -5.7700E-04     -226.00 -5.0300E-06
683.00  2.6200E+10  9.4500E+09
```

x(IN)	0.0000	0.0000	171.00	144.00	9.0000	9.0000	108.00	63.000
468.00	0.0000	0.0000						
46	-0.5490	-2.0100E-08	-6.9500E+05	-1.3000E-02	-3.1500E+04	-8.0700E-04	-226.00	-7.0100E-06
1810.0	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	171.00	144.00	9.0000	9.0000	108.00	63.000
459.00	0.0000	0.0000						
47	-0.5490	-2.0100E-08	-6.9300E+05	-1.3000E-02	-3.1500E+04	-8.0500E-04	-225.00	-6.9900E-06
1810.0	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	171.00	144.00	9.0000	9.0000	108.00	63.000
459.00	0.0000	0.0000						
48	-0.5490	-2.0100E-08	-6.9300E+05	-1.3000E-02	-3.1500E+04	-8.0500E-04	-225.00	-6.9900E-06
1810.0	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	171.00	144.00	9.0000	9.0000	108.00	63.000
459.00	0.0000	0.0000						
49	-0.5490	-2.0100E-08	-6.9300E+05	-1.3000E-02	-3.1500E+04	-8.0500E-04	-225.00	-6.9900E-06
1810.0	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	171.00	144.00	9.0000	9.0000	108.00	63.000
459.00	0.0000	0.0000						
50	-0.5490	-2.0100E-08	-6.9300E+05	-1.3000E-02	-3.1500E+04	-8.0500E-04	-225.00	-6.9900E-06
1810.0	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	171.00	144.00	9.0000	9.0000	108.00	63.000
459.00	0.0000	0.0000						
51	-0.5490	-2.0100E-08	-6.9500E+05	-1.3000E-02	-3.1500E+04	-8.0700E-04	-226.00	-7.0100E-06
1810.0	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	171.00	144.00	9.0000	9.0000	108.00	63.000
459.00	0.0000	0.0000						
Min.	-0.5490	-2.0100E-08	-7.3800E+05	-4.2800E-02	-3.3600E+04	-8.0700E-04	-248.00	-7.4900E-06
423.00	2.6200E+10	9.4500E+09						
Pile N.	1	46	1	1	1	46	1	1
34	1	1						

* MAXIMUM VALUES AND LOCATIONS *

193,33 K-ft

PILE	DEFLECTION		BENDING MOMENT		SHEAR FORCE		SOIL REACTION	
TOTAL FLEXURAL RIGIDITY	y-DIR	z-DIR	z-DIR	y-DIR	y-DIR	z-DIR	y-DIR	z-DIR
STRESS z-DIR	IN	IN	LBS-IN	LBS-IN	LBS	LBS	LBS/IN	LBS/IN
LBS/IN**2 LBS-IN**2	LBS-IN**2							
***** **********	**********	**********	**********	**********	**********	**********	**********	**********
********** **********	**********							
1	-0.5490	1.7700E-08	2.3200E+06	1.2300E-02	7800.0	7.4900E-04	196.00	6.8000E-06
2.2400E+04 2.6200E+10	9.4500E+09							
x(IN)	270.00	0.0000	0.0000	0.0000	234.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
2	-0.5490	1.7700E-08	2.3100E+06	1.2300E-02	7780.0	7.4700E-04	195.00	6.7800E-06
2.2400E+04 2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
3	-0.5490	1.7700E-08	2.3100E+06	1.2300E-02	7780.0	7.4700E-04	195.00	6.7800E-06
2.2400E+04 2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
4	-0.5490	1.7700E-08	2.3100E+06	1.2300E-02	7780.0	7.4700E-04	195.00	6.7800E-06
2.2400E+04 2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
5	-0.5490	1.7700E-08	2.3100E+06	1.2300E-02	7780.0	7.4700E-04	195.00	6.7800E-06
2.2400E+04 2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
6	-0.5490	1.7700E-08	2.3100E+06	1.2300E-02	7780.0	7.4700E-04	195.00	6.7800E-06
2.2400E+04 2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
7	-0.5490	1.7700E-08	2.3200E+06	1.2300E-02	7800.0	7.4900E-04	196.00	6.8000E-06
2.2400E+04 2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
8	-0.5490	1.3000E-08	2.0800E+06	8.1000E-03	6690.0	4.7400E-04	176.00	4.0900E-06
2.0000E+04 2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
9	-0.5490	1.3000E-08	2.0400E+06	7.9500E-03	6520.0	4.6000E-04	174.00	3.9300E-06
1.9700E+04 2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
10	-0.5490	1.3000E-08	2.0400E+06	7.9500E-03	6520.0	4.6000E-04	174.00	3.9300E-06

Row 1

Row 2

1.9700E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
11	-0.5490	1.3000E-08	2.0400E+06	7.9500E-03	6520.0	4.6000E-04	174.00	3.9300E-06	
1.9700E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
12	-0.5490	1.3000E-08	2.0400E+06	7.9500E-03	6520.0	4.6000E-04	174.00	3.9300E-06	
1.9700E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
13	-0.5490	1.3000E-08	2.0400E+06	7.9500E-03	6520.0	4.6000E-04	174.00	3.9300E-06	
1.9700E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
14	-0.5490	1.3000E-08	2.0800E+06	8.1000E-03	6690.0	4.7400E-04	176.00	4.0900E-06	
2.0000E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
15	-0.5490	8.2500E-09	2.0800E+06	5.1400E-03	6670.0	3.0200E-04	176.00	2.6000E-06	
1.9400E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
16	-0.5490	8.2500E-09	2.0400E+06	5.0400E-03	6500.0	2.9300E-04	173.00	2.5000E-06	
1.9100E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
17	-0.5490	8.2500E-09	2.0400E+06	5.0400E-03	6500.0	2.9300E-04	173.00	2.5000E-06	
1.9100E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
18	-0.5490	8.2500E-09	2.0400E+06	5.0400E-03	6500.0	2.9300E-04	173.00	2.5000E-06	
1.9100E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
19	-0.5490	8.2500E-09	2.0400E+06	5.0400E-03	6500.0	2.9300E-04	173.00	2.5000E-06	
1.9100E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
20	-0.5490	8.2500E-09	2.0400E+06	5.0400E-03	6500.0	2.9300E-04	173.00	2.5000E-06	
1.9100E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
21	-0.5490	8.2500E-09	2.0800E+06	5.1400E-03	6670.0	3.0200E-04	176.00	2.6000E-06	
1.9400E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
22	-0.5490	2.5700E-09	2.1800E+06	1.6700E-03	7080.0	1.0000E-04	186.00	8.7900E-07	
1.9300E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	243.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
23	-0.5490	2.5700E-09	2.1600E+06	1.6600E-03	7030.0	9.9500E-05	184.00	8.7000E-07	
1.9200E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	243.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
24	-0.5490	2.5700E-09	2.1600E+06	1.6500E-03	7000.0	9.9100E-05	184.00	8.6600E-07	
1.9200E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	243.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
25	-0.5490	2.5700E-09	2.1600E+06	1.6500E-03	7000.0	9.9100E-05	184.00	8.6600E-07	
1.9200E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	243.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
26	-0.5490	2.5700E-09	2.1600E+06	1.6600E-03	7030.0	9.9500E-05	184.00	8.7000E-07	
1.9200E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	243.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
27	-0.5490	2.5700E-09	2.1800E+06	1.6700E-03	7080.0	1.0000E-04	186.00	8.7900E-07	
1.9300E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	243.00	0.0000	342.00	63.000	
0.0000	0.0000	0.0000							
28	-0.5490	8.6400E-12	2.2000E+06	7.1400E-03	7150.0	2.4800E-05	187.00	2.0700E-06	
1.8400E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	252.00	0.0000	0.0000	243.00	225.00	342.00	243.00	
0.0000	0.0000	0.0000							
29	-0.5490	1.2200E-11	2.1900E+06	7.1300E-03	7130.0	2.4200E-05	187.00	1.7200E-06	
1.8300E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00	
0.0000	0.0000	0.0000							

30	-0.5490	1.2200E-11	2.1900E+06	7.1300E-03	7130.0	2.4200E-05	187.00	1.7200E-06
1.8300E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
31	-0.5490	1.2200E-11	2.1900E+06	7.1300E-03	7130.0	2.4200E-05	187.00	1.7200E-06
1.8300E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
32	-0.5490	1.2200E-11	2.1900E+06	7.1300E-03	7130.0	2.4200E-05	187.00	1.7200E-06
1.8300E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
33	-0.5490	8.6400E-12	2.2000E+06	7.1400E-03	7150.0	2.4800E-05	187.00	2.0700E-06
1.8400E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	252.00	0.0000	0.0000	243.00	225.00	342.00	243.00
0.0000	0.0000	0.0000						
34	-0.5490	3.6500E-11	2.2000E+06	2.0200E-02	7130.0	6.7900E-05	187.00	4.6300E-06
1.7200E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
35	-0.5490	4.2900E-11	2.1900E+06	2.0200E-02	7110.0	6.6900E-05	187.00	4.0400E-06
1.7200E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
36	-0.5490	4.2900E-11	2.1900E+06	2.0200E-02	7110.0	6.6900E-05	187.00	4.0400E-06
1.7200E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
37	-0.5490	4.2900E-11	2.1900E+06	2.0200E-02	7110.0	6.6900E-05	187.00	4.0400E-06
1.7200E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
38	-0.5490	4.2900E-11	2.1900E+06	2.0200E-02	7110.0	6.6900E-05	187.00	4.0400E-06
1.7200E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
39	-0.5490	3.6500E-11	2.2000E+06	2.0200E-02	7130.0	6.7900E-05	187.00	4.6300E-06
1.7200E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
40	-0.5490	7.5000E-11	2.2000E+06	3.3300E-02	7100.0	1.0900E-04	188.00	6.1100E-06
1.7500E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
41	-0.5490	7.5200E-11	2.1900E+06	3.3200E-02	7080.0	1.0900E-04	187.00	6.0800E-06
1.7500E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
42	-0.5490	7.5200E-11	2.1900E+06	3.3200E-02	7080.0	1.0900E-04	187.00	6.0800E-06
1.7500E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
43	-0.5490	7.5200E-11	2.1900E+06	3.3200E-02	7080.0	1.0900E-04	187.00	6.0800E-06
1.7500E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
44	-0.5490	7.5200E-11	2.1900E+06	3.3200E-02	7080.0	1.0900E-04	187.00	6.0800E-06
1.7500E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
45	-0.5490	7.5000E-11	2.2000E+06	3.3300E-02	7100.0	1.0900E-04	188.00	6.1100E-06
1.7500E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
46	-0.5490	1.0700E-10	2.2000E+06	4.6300E-02	7070.0	1.5000E-04	187.00	8.0900E-06
1.8600E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
47	-0.5490	1.1300E-10	2.1900E+06	4.6300E-02	7050.0	1.4900E-04	186.00	7.4700E-06
1.8600E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
48	-0.5490	1.1300E-10	2.1900E+06	4.6300E-02	7050.0	1.4900E-04	186.00	7.4700E-06
1.8600E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
49	-0.5490	1.1300E-10	2.1900E+06	4.6300E-02	7050.0	1.4900E-04	186.00	7.4700E-06
1.8600E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00

Row 6

Row 7

Row 8

RRN_WWA90.gp8t

0.0000	0.0000	0.0000						
50	-0.5490	1.1300E-10	2.1900E+06	4.6300E-02	7050.0	1.4900E-04	186.00	7.4700E-06
1.8600E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
51	-0.5490	1.0700E-10	2.2000E+06	4.6300E-02	7070.0	1.5000E-04	187.00	8.0900E-06
1.8600E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
Max.	-0.5490	1.7700E-08	2.3200E+06	4.6300E-02	7800.0	7.4900E-04	196.00	8.0900E-06
2.2400E+04	2.6200E+10	9.4500E+09						
Pile N.	1	1	1	46	1	1	1	46
1	1	1						

LOAD CASE : 2
CASE NAME : Load Case 1.1 w/ice
LOAD TYPE : Dead, DL

REDUCTION FACTORS FOR CLOSELY-SPACED PILE GROUPS, COMBINED Y AND Z DIRECTIONS
ESTIMATED USING MOVEMENT IN THE DIRECTION OF PILE CAP DISPLACEMENTS

GROUP NO	P-FACTOR	Y-FACTOR
1	0.9974	1.0000
2	0.9948	1.0000
3	0.9948	1.0000
4	0.9948	1.0000
5	0.9948	1.0000
6	0.9948	1.0000
7	0.9974	1.0000
8	0.8025	1.0000
9	0.7682	1.0000
10	0.7682	1.0000
11	0.7682	1.0000
12	0.7682	1.0000
13	0.7682	1.0000
14	0.8025	1.0000
15	0.8025	1.0000
16	0.7682	1.0000
17	0.7682	1.0000
18	0.7682	1.0000
19	0.7682	1.0000
20	0.7682	1.0000
21	0.8025	1.0000
22	0.8764	1.0000
23	0.8666	1.0000
24	0.8622	1.0000
25	0.8622	1.0000
26	0.8666	1.0000
27	0.8764	1.0000
28	0.8949	1.0000
29	0.8913	1.0000
30	0.8913	1.0000
31	0.8913	1.0000
32	0.8913	1.0000
33	0.8949	1.0000
34	0.8949	1.0000
35	0.8913	1.0000
36	0.8913	1.0000
37	0.8913	1.0000
38	0.8913	1.0000
39	0.8949	1.0000
40	0.8949	1.0000
41	0.8913	1.0000
42	0.8913	1.0000
43	0.8913	1.0000
44	0.8913	1.0000
45	0.8949	1.0000
46	0.8949	1.0000
47	0.8913	1.0000
48	0.8913	1.0000
49	0.8913	1.0000
50	0.8913	1.0000
51	0.8949	1.0000

RRN_WWA90.gp8t

* TABLE L * COMPUTATION ON PILE CAP

* EQUIVALENT CONC. LOAD AT ORIGIN *

VERT. LOAD, LBS	HOR. LOAD Y, LBS	HOR. LOAD Z, LBS
2.57500E+06	-1.58200E+06	0.00000

MOMENT X ,IN-LBS	MOMENT Y,IN-LBS	MOMENT Z,IN-LBS
2.84760E+08	4.63500E+08	-3.82542E+08

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

VERTICAL ,IN	HORIZONTAL Y,IN	HORIZONTAL Z,IN
0.10208	-0.54842	5.38292E-13

ANGLE ROT. X,RAD	ANGLE ROT. Y,RAD	ANGLE ROT. Z,RAD
-2.11583E-15	1.00455E-18	2.37446E-04

THE GLOBAL STRUCTURAL COORDINATE SYSTEM

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. X,IN	DISP. Y,IN	DISP. Z,IN	ROT. X,RAD	ROT. Y,RAD	ROT. Z,RAD
1	9.4952E-02	-0.5513	4.7483E-13	-2.1158E-15	1.0045E-18	2.3745E-04
2	9.4952E-02	-0.5513	4.7483E-13	-2.1158E-15	1.0045E-18	2.3745E-04
3	9.4952E-02	-0.5513	4.7483E-13	-2.1158E-15	1.0045E-18	2.3745E-04
4	9.4952E-02	-0.5513	4.7483E-13	-2.1158E-15	1.0045E-18	2.3745E-04
5	9.4952E-02	-0.5513	4.7483E-13	-2.1158E-15	1.0045E-18	2.3745E-04
6	9.4952E-02	-0.5513	4.7483E-13	-2.1158E-15	1.0045E-18	2.3745E-04
7	9.4952E-02	-0.5513	4.7483E-13	-2.1158E-15	1.0045E-18	2.3745E-04
8	8.0705E-02	-0.5513	3.4788E-13	-2.1158E-15	1.0045E-18	2.3745E-04
9	8.0705E-02	-0.5513	3.4788E-13	-2.1158E-15	1.0045E-18	2.3745E-04
10	8.0705E-02	-0.5513	3.4788E-13	-2.1158E-15	1.0045E-18	2.3745E-04
11	8.0705E-02	-0.5513	3.4788E-13	-2.1158E-15	1.0045E-18	2.3745E-04
12	8.0705E-02	-0.5513	3.4788E-13	-2.1158E-15	1.0045E-18	2.3745E-04
13	8.0705E-02	-0.5513	3.4788E-13	-2.1158E-15	1.0045E-18	2.3745E-04
14	8.0705E-02	-0.5513	3.4788E-13	-2.1158E-15	1.0045E-18	2.3745E-04
15	6.6458E-02	-0.5513	2.2093E-13	-2.1158E-15	1.0045E-18	2.3745E-04
16	6.6458E-02	-0.5513	2.2093E-13	-2.1158E-15	1.0045E-18	2.3745E-04
17	6.6458E-02	-0.5513	2.2093E-13	-2.1158E-15	1.0045E-18	2.3745E-04
18	6.6458E-02	-0.5513	2.2093E-13	-2.1158E-15	1.0045E-18	2.3745E-04
19	6.6458E-02	-0.5513	2.2093E-13	-2.1158E-15	1.0045E-18	2.3745E-04
20	6.6458E-02	-0.5513	2.2093E-13	-2.1158E-15	1.0045E-18	2.3745E-04
21	6.6458E-02	-0.5513	2.2093E-13	-2.1158E-15	1.0045E-18	2.3745E-04
22	4.9362E-02	-0.5513	6.8590E-14	-2.1158E-15	1.0045E-18	2.3745E-04
23	4.9362E-02	-0.5513	6.8590E-14	-2.1158E-15	1.0045E-18	2.3745E-04
24	4.9362E-02	-0.5513	6.8590E-14	-2.1158E-15	1.0045E-18	2.3745E-04
25	4.9362E-02	-0.5513	6.8590E-14	-2.1158E-15	1.0045E-18	2.3745E-04
26	4.9362E-02	-0.5513	6.8590E-14	-2.1158E-15	1.0045E-18	2.3745E-04
27	4.9362E-02	-0.5513	6.8590E-14	-2.1158E-15	1.0045E-18	2.3745E-04
28	3.2266E-02	-0.5513	-8.3750E-14	-2.1158E-15	1.0045E-18	2.3745E-04
29	3.2266E-02	-0.5513	-8.3750E-14	-2.1158E-15	1.0045E-18	2.3745E-04
30	3.2266E-02	-0.5513	-8.3750E-14	-2.1158E-15	1.0045E-18	2.3745E-04
31	3.2266E-02	-0.5513	-8.3750E-14	-2.1158E-15	1.0045E-18	2.3745E-04
32	3.2266E-02	-0.5513	-8.3750E-14	-2.1158E-15	1.0045E-18	2.3745E-04
33	3.2266E-02	-0.5513	-8.3750E-14	-2.1158E-15	1.0045E-18	2.3745E-04
34	1.5170E-02	-0.5513	-2.3609E-13	-2.1158E-15	1.0045E-18	2.3745E-04
35	1.5170E-02	-0.5513	-2.3609E-13	-2.1158E-15	1.0045E-18	2.3745E-04
36	1.5170E-02	-0.5513	-2.3609E-13	-2.1158E-15	1.0045E-18	2.3745E-04
37	1.5170E-02	-0.5513	-2.3609E-13	-2.1158E-15	1.0045E-18	2.3745E-04
38	1.5170E-02	-0.5513	-2.3609E-13	-2.1158E-15	1.0045E-18	2.3745E-04
39	1.5170E-02	-0.5513	-2.3609E-13	-2.1158E-15	1.0045E-18	2.3745E-04
40	-1.9264E-03	-0.5513	-3.8843E-13	-2.1158E-15	1.0045E-18	2.3745E-04
41	-1.9264E-03	-0.5513	-3.8843E-13	-2.1158E-15	1.0045E-18	2.3745E-04
42	-1.9264E-03	-0.5513	-3.8843E-13	-2.1158E-15	1.0045E-18	2.3745E-04
43	-1.9264E-03	-0.5513	-3.8843E-13	-2.1158E-15	1.0045E-18	2.3745E-04
44	-1.9264E-03	-0.5513	-3.8843E-13	-2.1158E-15	1.0045E-18	2.3745E-04
45	-1.9264E-03	-0.5513	-3.8843E-13	-2.1158E-15	1.0045E-18	2.3745E-04
46	-1.9023E-02	-0.5513	-5.4077E-13	-2.1158E-15	1.0045E-18	2.3745E-04
47	-1.9023E-02	-0.5513	-5.4077E-13	-2.1158E-15	1.0045E-18	2.3745E-04
48	-1.9023E-02	-0.5513	-5.4077E-13	-2.1158E-15	1.0045E-18	2.3745E-04

49	-1.9023E-02	-0.5513	-5.4077E-13	-2.1158E-15	1.0045E-18	2.3745E-04
50	-1.9023E-02	-0.5513	-5.4077E-13	-2.1158E-15	1.0045E-18	2.3745E-04
51	-1.9023E-02	-0.5513	-5.4077E-13	-2.1158E-15	1.0045E-18	2.3745E-04
MINIMUM	-1.9023E-02	-0.5513	-5.4077E-13	-2.1158E-15	1.0045E-18	2.3745E-04
Pile N.	46	1	46	1	1	1
MAXIMUM	9.4952E-02	-0.5513	4.7483E-13	-2.1158E-15	1.0045E-18	2.3745E-04
Pile N.	1	1	1	1	1	1

* PILE TOP REACTIONS * AXIAL HORIZ

PILE GROUP	FOR. X,LBS	FOR. Y,LBS	FOR. Z,LBS	MOM X,LBS-IN	MOM Y,LBS-IN	MOM Z,LBS-IN	STRESS,LBS/IN**2
1	1.0700E+05	-3.3922E+04	1.9960E-08	-1.9187E-14	-1.1441E-06	-2.3568E+06	2.2127E+04
2	1.0700E+05	-3.3859E+04	1.9924E-08	-1.9187E-14	-1.1427E-06	-2.3537E+06	2.2104E+04
3	1.0700E+05	-3.3859E+04	1.9924E-08	-1.9187E-14	-1.1427E-06	-2.3537E+06	2.2104E+04
4	1.0700E+05	-3.3859E+04	1.9924E-08	-1.9187E-14	-1.1427E-06	-2.3537E+06	2.2104E+04
5	1.0700E+05	-3.3859E+04	1.9924E-08	-1.9187E-14	-1.1427E-06	-2.3537E+06	2.2104E+04
6	1.0700E+05	-3.3859E+04	1.9924E-08	-1.9187E-14	-1.1427E-06	-2.3537E+06	2.2104E+04
7	1.0700E+05	-3.3922E+04	1.9960E-08	-1.9187E-14	-1.1441E-06	-2.3568E+06	2.2127E+04
8	9.4323E+04	-2.9177E+04	1.2638E-08	-1.9187E-14	-7.5837E-07	-2.1224E+06	1.9849E+04
9	9.4323E+04	-2.8306E+04	1.2261E-08	-1.9187E-14	-7.4314E-07	-2.0792E+06	1.9519E+04
10	9.4323E+04	-2.8306E+04	1.2261E-08	-1.9187E-14	-7.4314E-07	-2.0792E+06	1.9519E+04
11	9.4323E+04	-2.8306E+04	1.2261E-08	-1.9187E-14	-7.4314E-07	-2.0792E+06	1.9519E+04
12	9.4323E+04	-2.8306E+04	1.2261E-08	-1.9187E-14	-7.4314E-07	-2.0792E+06	1.9519E+04
13	9.4323E+04	-2.8306E+04	1.2261E-08	-1.9187E-14	-7.4314E-07	-2.0792E+06	1.9519E+04
14	9.4323E+04	-2.9177E+04	1.2638E-08	-1.9187E-14	-7.5837E-07	-2.1224E+06	1.9849E+04
15	8.1641E+04	-2.9206E+04	8.0420E-09	-1.9187E-14	-4.8160E-07	-2.1223E+06	1.9362E+04
16	8.1641E+04	-2.8338E+04	7.8013E-09	-1.9187E-14	-4.7192E-07	-2.0793E+06	1.9033E+04
17	8.1641E+04	-2.8338E+04	7.8013E-09	-1.9187E-14	-4.7192E-07	-2.0793E+06	1.9033E+04
18	8.1641E+04	-2.8338E+04	7.8013E-09	-1.9187E-14	-4.7192E-07	-2.0793E+06	1.9033E+04
19	8.1641E+04	-2.8338E+04	7.8013E-09	-1.9187E-14	-4.7192E-07	-2.0793E+06	1.9033E+04
20	8.1641E+04	-2.8338E+04	7.8013E-09	-1.9187E-14	-4.7192E-07	-2.0793E+06	1.9033E+04
21	8.1641E+04	-2.9206E+04	8.0420E-09	-1.9187E-14	-4.8160E-07	-2.1223E+06	1.9362E+04
22	6.4547E+04	-3.1107E+04	2.6548E-09	-1.9187E-14	-1.5556E-07	-2.2151E+06	1.9417E+04
23	6.4547E+04	-3.0864E+04	2.6343E-09	-1.9187E-14	-1.5476E-07	-2.2031E+06	1.9325E+04
24	6.4547E+04	-3.0755E+04	2.6252E-09	-1.9187E-14	-1.5439E-07	-2.1977E+06	1.9284E+04
25	6.4547E+04	-3.0755E+04	2.6252E-09	-1.9187E-14	-1.5439E-07	-2.1977E+06	1.9284E+04
26	6.4547E+04	-3.0864E+04	2.6343E-09	-1.9187E-14	-1.5476E-07	-2.2031E+06	1.9325E+04
27	6.4547E+04	-3.1107E+04	2.6548E-09	-1.9187E-14	-1.5556E-07	-2.2151E+06	1.9417E+04
28	4.1944E+04	-3.1619E+04	-3.3048E-09	-1.9187E-14	1.9232E-07	-2.2373E+06	1.8721E+04
29	4.1944E+04	-3.1529E+04	-3.2953E-09	-1.9187E-14	1.9195E-07	-2.2329E+06	1.8687E+04
30	4.1944E+04	-3.1529E+04	-3.2953E-09	-1.9187E-14	1.9195E-07	-2.2329E+06	1.8687E+04
31	4.1944E+04	-3.1529E+04	-3.2953E-09	-1.9187E-14	1.9195E-07	-2.2329E+06	1.8687E+04
32	4.1944E+04	-3.1529E+04	-3.2953E-09	-1.9187E-14	1.9195E-07	-2.2329E+06	1.8687E+04
33	4.1944E+04	-3.1619E+04	-3.3048E-09	-1.9187E-14	1.9232E-07	-2.2373E+06	1.8721E+04
34	1.9593E+04	-3.1674E+04	-9.3414E-09	-1.9187E-14	5.4180E-07	-2.2372E+06	1.7864E+04
35	1.9593E+04	-3.1586E+04	-9.3155E-09	-1.9187E-14	5.4079E-07	-2.2329E+06	1.7831E+04
36	1.9593E+04	-3.1586E+04	-9.3155E-09	-1.9187E-14	5.4079E-07	-2.2329E+06	1.7831E+04
37	1.9593E+04	-3.1586E+04	-9.3155E-09	-1.9187E-14	5.4079E-07	-2.2329E+06	1.7831E+04
38	1.9593E+04	-3.1586E+04	-9.3155E-09	-1.9187E-14	5.4079E-07	-2.2329E+06	1.7831E+04
39	1.9593E+04	-3.1674E+04	-9.3414E-09	-1.9187E-14	5.4180E-07	-2.2372E+06	1.7864E+04
40	-2459.5	-3.1730E+04	-1.5416E-08	-1.9187E-14	8.9139E-07	-2.2371E+06	1.7207E+04
41	-2459.5	-3.1642E+04	-1.5373E-08	-1.9187E-14	8.8973E-07	-2.2328E+06	1.7174E+04
42	-2459.5	-3.1642E+04	-1.5373E-08	-1.9187E-14	8.8973E-07	-2.2328E+06	1.7174E+04
43	-2459.5	-3.1642E+04	-1.5373E-08	-1.9187E-14	8.8973E-07	-2.2328E+06	1.7174E+04
44	-2459.5	-3.1642E+04	-1.5373E-08	-1.9187E-14	8.8973E-07	-2.2328E+06	1.7174E+04
45	-2459.5	-3.1730E+04	-1.5416E-08	-1.9187E-14	8.9139E-07	-2.2371E+06	1.7207E+04
46	-2.4587E+04	-3.1783E+04	-2.1527E-08	-1.9187E-14	1.2410E-06	-2.2368E+06	1.8052E+04
47	-2.4587E+04	-3.1694E+04	-2.1468E-08	-1.9187E-14	1.2387E-06	-2.2324E+06	1.8019E+04
48	-2.4587E+04	-3.1694E+04	-2.1468E-08	-1.9187E-14	1.2387E-06	-2.2324E+06	1.8019E+04
49	-2.4587E+04	-3.1694E+04	-2.1468E-08	-1.9187E-14	1.2387E-06	-2.2324E+06	1.8019E+04
50	-2.4587E+04	-3.1694E+04	-2.1468E-08	-1.9187E-14	1.2387E-06	-2.2324E+06	1.8019E+04
51	-2.4587E+04	-3.1783E+04	-2.1527E-08	-1.9187E-14	1.2410E-06	-2.2368E+06	1.8052E+04
MINIMUM	-2.4587E+04	-3.3922E+04	-2.1527E-08	-1.9187E-14	-1.1441E-06	-2.3568E+06	1.7174E+04
Pile N.	46	1	46	1	1	1	41
MAXIMUM	1.0700E+05	-2.8306E+04	1.9960E-08	-1.9187E-14	1.2410E-06	-2.0792E+06	2.2127E+04
Pile N.	1	9	1	1	46	9	1

THE PILE COORDINATE SYSTEM (LOCAL AXES)

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. x,IN	DISP. y,IN	DISP. z,IN	ROT. x,RAD	ROT. y,RAD	ROT. z,RAD
1	9.4952E-02	-0.5513	4.7483E-13	-2.1158E-15	1.0045E-18	2.3745E-04

2	9.4952E-02	-0.5513	4.7483E-13	-2.1158E-15	1.0045E-18	2.3745E-04
3	9.4952E-02	-0.5513	4.7483E-13	-2.1158E-15	1.0045E-18	2.3745E-04
4	9.4952E-02	-0.5513	4.7483E-13	-2.1158E-15	1.0045E-18	2.3745E-04
5	9.4952E-02	-0.5513	4.7483E-13	-2.1158E-15	1.0045E-18	2.3745E-04
6	9.4952E-02	-0.5513	4.7483E-13	-2.1158E-15	1.0045E-18	2.3745E-04
7	9.4952E-02	-0.5513	4.7483E-13	-2.1158E-15	1.0045E-18	2.3745E-04
8	8.0705E-02	-0.5513	3.4788E-13	-2.1158E-15	1.0045E-18	2.3745E-04
9	8.0705E-02	-0.5513	3.4788E-13	-2.1158E-15	1.0045E-18	2.3745E-04
10	8.0705E-02	-0.5513	3.4788E-13	-2.1158E-15	1.0045E-18	2.3745E-04
11	8.0705E-02	-0.5513	3.4788E-13	-2.1158E-15	1.0045E-18	2.3745E-04
12	8.0705E-02	-0.5513	3.4788E-13	-2.1158E-15	1.0045E-18	2.3745E-04
13	8.0705E-02	-0.5513	3.4788E-13	-2.1158E-15	1.0045E-18	2.3745E-04
14	8.0705E-02	-0.5513	3.4788E-13	-2.1158E-15	1.0045E-18	2.3745E-04
15	6.6458E-02	-0.5513	2.2093E-13	-2.1158E-15	1.0045E-18	2.3745E-04
16	6.6458E-02	-0.5513	2.2093E-13	-2.1158E-15	1.0045E-18	2.3745E-04
17	6.6458E-02	-0.5513	2.2093E-13	-2.1158E-15	1.0045E-18	2.3745E-04
18	6.6458E-02	-0.5513	2.2093E-13	-2.1158E-15	1.0045E-18	2.3745E-04
19	6.6458E-02	-0.5513	2.2093E-13	-2.1158E-15	1.0045E-18	2.3745E-04
20	6.6458E-02	-0.5513	2.2093E-13	-2.1158E-15	1.0045E-18	2.3745E-04
21	6.6458E-02	-0.5513	2.2093E-13	-2.1158E-15	1.0045E-18	2.3745E-04
22	4.9362E-02	-0.5513	6.8590E-14	-2.1158E-15	1.0045E-18	2.3745E-04
23	4.9362E-02	-0.5513	6.8590E-14	-2.1158E-15	1.0045E-18	2.3745E-04
24	4.9362E-02	-0.5513	6.8590E-14	-2.1158E-15	1.0045E-18	2.3745E-04
25	4.9362E-02	-0.5513	6.8590E-14	-2.1158E-15	1.0045E-18	2.3745E-04
26	4.9362E-02	-0.5513	6.8590E-14	-2.1158E-15	1.0045E-18	2.3745E-04
27	4.9362E-02	-0.5513	6.8590E-14	-2.1158E-15	1.0045E-18	2.3745E-04
28	3.2266E-02	-0.5513	-8.3750E-14	-2.1158E-15	1.0045E-18	2.3745E-04
29	3.2266E-02	-0.5513	-8.3750E-14	-2.1158E-15	1.0045E-18	2.3745E-04
30	3.2266E-02	-0.5513	-8.3750E-14	-2.1158E-15	1.0045E-18	2.3745E-04
31	3.2266E-02	-0.5513	-8.3750E-14	-2.1158E-15	1.0045E-18	2.3745E-04
32	3.2266E-02	-0.5513	-8.3750E-14	-2.1158E-15	1.0045E-18	2.3745E-04
33	3.2266E-02	-0.5513	-8.3750E-14	-2.1158E-15	1.0045E-18	2.3745E-04
34	1.5170E-02	-0.5513	-2.3609E-13	-2.1158E-15	1.0045E-18	2.3745E-04
35	1.5170E-02	-0.5513	-2.3609E-13	-2.1158E-15	1.0045E-18	2.3745E-04
36	1.5170E-02	-0.5513	-2.3609E-13	-2.1158E-15	1.0045E-18	2.3745E-04
37	1.5170E-02	-0.5513	-2.3609E-13	-2.1158E-15	1.0045E-18	2.3745E-04
38	1.5170E-02	-0.5513	-2.3609E-13	-2.1158E-15	1.0045E-18	2.3745E-04
39	1.5170E-02	-0.5513	-2.3609E-13	-2.1158E-15	1.0045E-18	2.3745E-04
40	-1.9264E-03	-0.5513	-3.8843E-13	-2.1158E-15	1.0045E-18	2.3745E-04
41	-1.9264E-03	-0.5513	-3.8843E-13	-2.1158E-15	1.0045E-18	2.3745E-04
42	-1.9264E-03	-0.5513	-3.8843E-13	-2.1158E-15	1.0045E-18	2.3745E-04
43	-1.9264E-03	-0.5513	-3.8843E-13	-2.1158E-15	1.0045E-18	2.3745E-04
44	-1.9264E-03	-0.5513	-3.8843E-13	-2.1158E-15	1.0045E-18	2.3745E-04
45	-1.9264E-03	-0.5513	-3.8843E-13	-2.1158E-15	1.0045E-18	2.3745E-04
46	-1.9023E-02	-0.5513	-5.4077E-13	-2.1158E-15	1.0045E-18	2.3745E-04
47	-1.9023E-02	-0.5513	-5.4077E-13	-2.1158E-15	1.0045E-18	2.3745E-04
48	-1.9023E-02	-0.5513	-5.4077E-13	-2.1158E-15	1.0045E-18	2.3745E-04
49	-1.9023E-02	-0.5513	-5.4077E-13	-2.1158E-15	1.0045E-18	2.3745E-04
50	-1.9023E-02	-0.5513	-5.4077E-13	-2.1158E-15	1.0045E-18	2.3745E-04
51	-1.9023E-02	-0.5513	-5.4077E-13	-2.1158E-15	1.0045E-18	2.3745E-04
MINIMUM	-1.9023E-02	-0.5513	-5.4077E-13	-2.1158E-15	1.0045E-18	2.3745E-04
Pile N.	46	1	46	1	1	1
MAXIMUM	9.4952E-02	-0.5513	4.7483E-13	-2.1158E-15	1.0045E-18	2.3745E-04
Pile N.	1	1	1	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	AXIAL,LBS	LAT. y,LBS	LAT. z,LBS	MOM x,LBS-IN	MOM y,LBS-IN	MOM z,LBS-IN	STRESS,LBS/IN**2
1	1.0700E+05	-3.3922E+04	1.9960E-08	-1.9187E-14	-1.1441E-06	-2.3568E+06	2.2127E+04
2	1.0700E+05	-3.3859E+04	1.9924E-08	-1.9187E-14	-1.1427E-06	-2.3537E+06	2.2104E+04
3	1.0700E+05	-3.3859E+04	1.9924E-08	-1.9187E-14	-1.1427E-06	-2.3537E+06	2.2104E+04
4	1.0700E+05	-3.3859E+04	1.9924E-08	-1.9187E-14	-1.1427E-06	-2.3537E+06	2.2104E+04
5	1.0700E+05	-3.3859E+04	1.9924E-08	-1.9187E-14	-1.1427E-06	-2.3537E+06	2.2104E+04
6	1.0700E+05	-3.3859E+04	1.9924E-08	-1.9187E-14	-1.1427E-06	-2.3537E+06	2.2104E+04
7	1.0700E+05	-3.3922E+04	1.9960E-08	-1.9187E-14	-1.1441E-06	-2.3568E+06	2.2127E+04
8	9.4323E+04	-2.9177E+04	1.2638E-08	-1.9187E-14	-7.5837E-07	-2.1224E+06	1.9849E+04
9	9.4323E+04	-2.8306E+04	1.2261E-08	-1.9187E-14	-7.4314E-07	-2.0792E+06	1.9519E+04
10	9.4323E+04	-2.8306E+04	1.2261E-08	-1.9187E-14	-7.4314E-07	-2.0792E+06	1.9519E+04
11	9.4323E+04	-2.8306E+04	1.2261E-08	-1.9187E-14	-7.4314E-07	-2.0792E+06	1.9519E+04
12	9.4323E+04	-2.8306E+04	1.2261E-08	-1.9187E-14	-7.4314E-07	-2.0792E+06	1.9519E+04
13	9.4323E+04	-2.8306E+04	1.2261E-08	-1.9187E-14	-7.4314E-07	-2.0792E+06	1.9519E+04
14	9.4323E+04	-2.9177E+04	1.2638E-08	-1.9187E-14	-7.5837E-07	-2.1224E+06	1.9849E+04
15	8.1641E+04	-2.9206E+04	8.0420E-09	-1.9187E-14	-4.8160E-07	-2.1223E+06	1.9362E+04
16	8.1641E+04	-2.8338E+04	7.8013E-09	-1.9187E-14	-4.7192E-07	-2.0793E+06	1.9033E+04
17	8.1641E+04	-2.8338E+04	7.8013E-09	-1.9187E-14	-4.7192E-07	-2.0793E+06	1.9033E+04
18	8.1641E+04	-2.8338E+04	7.8013E-09	-1.9187E-14	-4.7192E-07	-2.0793E+06	1.9033E+04
19	8.1641E+04	-2.8338E+04	7.8013E-09	-1.9187E-14	-4.7192E-07	-2.0793E+06	1.9033E+04

20	8.1641E+04	-2.8338E+04	7.8013E-09	-1.9187E-14	-4.7192E-07	-2.0793E+06	1.9033E+04
21	8.1641E+04	-2.9206E+04	8.0420E-09	-1.9187E-14	-4.8160E-07	-2.1223E+06	1.9362E+04
22	6.4547E+04	-3.1107E+04	2.6548E-09	-1.9187E-14	-1.5556E-07	-2.2151E+06	1.9417E+04
23	6.4547E+04	-3.0864E+04	2.6343E-09	-1.9187E-14	-1.5476E-07	-2.2031E+06	1.9325E+04
24	6.4547E+04	-3.0755E+04	2.6252E-09	-1.9187E-14	-1.5439E-07	-2.1977E+06	1.9284E+04
25	6.4547E+04	-3.0755E+04	2.6252E-09	-1.9187E-14	-1.5439E-07	-2.1977E+06	1.9284E+04
26	6.4547E+04	-3.0864E+04	2.6343E-09	-1.9187E-14	-1.5476E-07	-2.2031E+06	1.9325E+04
27	6.4547E+04	-3.1107E+04	2.6548E-09	-1.9187E-14	-1.5556E-07	-2.2151E+06	1.9417E+04
28	4.1944E+04	-3.1619E+04	-3.3048E-09	-1.9187E-14	1.9232E-07	-2.2373E+06	1.8721E+04
29	4.1944E+04	-3.1529E+04	-3.2953E-09	-1.9187E-14	1.9195E-07	-2.2329E+06	1.8687E+04
30	4.1944E+04	-3.1529E+04	-3.2953E-09	-1.9187E-14	1.9195E-07	-2.2329E+06	1.8687E+04
31	4.1944E+04	-3.1529E+04	-3.2953E-09	-1.9187E-14	1.9195E-07	-2.2329E+06	1.8687E+04
32	4.1944E+04	-3.1529E+04	-3.2953E-09	-1.9187E-14	1.9195E-07	-2.2329E+06	1.8687E+04
33	4.1944E+04	-3.1619E+04	-3.3048E-09	-1.9187E-14	1.9232E-07	-2.2373E+06	1.8721E+04
34	1.9593E+04	-3.1674E+04	-9.3414E-09	-1.9187E-14	5.4180E-07	-2.2372E+06	1.7864E+04
35	1.9593E+04	-3.1586E+04	-9.3155E-09	-1.9187E-14	5.4079E-07	-2.2329E+06	1.7831E+04
36	1.9593E+04	-3.1586E+04	-9.3155E-09	-1.9187E-14	5.4079E-07	-2.2329E+06	1.7831E+04
37	1.9593E+04	-3.1586E+04	-9.3155E-09	-1.9187E-14	5.4079E-07	-2.2329E+06	1.7831E+04
38	1.9593E+04	-3.1586E+04	-9.3155E-09	-1.9187E-14	5.4079E-07	-2.2329E+06	1.7831E+04
39	1.9593E+04	-3.1674E+04	-9.3414E-09	-1.9187E-14	5.4180E-07	-2.2372E+06	1.7864E+04
40	-2459.5	-3.1730E+04	-1.5416E-08	-1.9187E-14	8.9139E-07	-2.2371E+06	1.7207E+04
41	-2459.5	-3.1642E+04	-1.5373E-08	-1.9187E-14	8.8973E-07	-2.2328E+06	1.7174E+04
42	-2459.5	-3.1642E+04	-1.5373E-08	-1.9187E-14	8.8973E-07	-2.2328E+06	1.7174E+04
43	-2459.5	-3.1642E+04	-1.5373E-08	-1.9187E-14	8.8973E-07	-2.2328E+06	1.7174E+04
44	-2459.5	-3.1642E+04	-1.5373E-08	-1.9187E-14	8.8973E-07	-2.2328E+06	1.7174E+04
45	-2459.5	-3.1730E+04	-1.5416E-08	-1.9187E-14	8.9139E-07	-2.2371E+06	1.7207E+04
46	-2.4587E+04	-3.1783E+04	-2.1527E-08	-1.9187E-14	1.2410E-06	-2.2368E+06	1.8052E+04
47	-2.4587E+04	-3.1694E+04	-2.1468E-08	-1.9187E-14	1.2387E-06	-2.2324E+06	1.8019E+04
48	-2.4587E+04	-3.1694E+04	-2.1468E-08	-1.9187E-14	1.2387E-06	-2.2324E+06	1.8019E+04
49	-2.4587E+04	-3.1694E+04	-2.1468E-08	-1.9187E-14	1.2387E-06	-2.2324E+06	1.8019E+04
50	-2.4587E+04	-3.1694E+04	-2.1468E-08	-1.9187E-14	1.2387E-06	-2.2324E+06	1.8019E+04
51	-2.4587E+04	-3.1783E+04	-2.1527E-08	-1.9187E-14	1.2410E-06	-2.2368E+06	1.8052E+04
MINIMUM	-2.4587E+04	-3.3922E+04	-2.1527E-08	-1.9187E-14	-1.1441E-06	-2.3568E+06	1.7174E+04
Pile N.	46	1	46	1	1	1	41
MAXIMUM	1.0700E+05	-2.8306E+04	1.9960E-08	-1.9187E-14	1.2410E-06	-2.0792E+06	2.2127E+04
Pile N.	1	9	1	1	46	9	1

* EFFECTS FOR LATERALLY LOADED PILE *

* MINIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION		BENDING MOMENT		SHEAR FORCE		SOIL REACTION	
TOTAL FLEXURAL RIGIDITY	y-DIR	z-DIR	z-DIR	y-DIR	y-DIR	z-DIR	y-DIR	z-DIR
STRESS z-DIR y-DIR	IN	IN	LBS-IN	LBS-IN	LBS	LBS	LBS/IN	LBS/IN
LBS/IN**2 LBS-IN**2 LBS-IN**2								
***** ********** **********	**********	**********	**********	**********	**********	**********	**********	**********
********** ********** **********								
1	-0.5510	-3.5000E-15	-7.4100E+05	-1.1400E-06	-3.4000E+04	-3.8800E-09	-249.00	-1.3800E-10
4100.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	234.00	171.00	135.00	0.0000	207.00	108.00	234.00
459.00	0.0000	0.0000						
2	-0.5510	-3.5600E-15	-7.4000E+05	-1.1400E-06	-3.3900E+04	-3.8600E-09	-249.00	-1.2900E-10
4100.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	234.00	171.00	135.00	0.0000	207.00	108.00	234.00
459.00	0.0000	0.0000						
3	-0.5510	-3.5600E-15	-7.4000E+05	-1.1400E-06	-3.3900E+04	-3.8600E-09	-249.00	-1.2900E-10
4100.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	234.00	171.00	135.00	0.0000	207.00	108.00	234.00
459.00	0.0000	0.0000						
4	-0.5510	-3.5600E-15	-7.4000E+05	-1.1400E-06	-3.3900E+04	-3.8600E-09	-249.00	-1.2900E-10
4100.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	234.00	171.00	135.00	0.0000	207.00	108.00	234.00
459.00	0.0000	0.0000						
5	-0.5510	-3.5600E-15	-7.4000E+05	-1.1400E-06	-3.3900E+04	-3.8600E-09	-249.00	-1.2900E-10
4100.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	234.00	171.00	135.00	0.0000	207.00	108.00	234.00
459.00	0.0000	0.0000						
6	-0.5510	-3.5600E-15	-7.4000E+05	-1.1400E-06	-3.3900E+04	-3.8600E-09	-249.00	-1.2900E-10
4100.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	234.00	171.00	135.00	0.0000	207.00	108.00	234.00
459.00	0.0000	0.0000						
7	-0.5510	-3.5000E-15	-7.4100E+05	-1.1400E-06	-3.4000E+04	-3.8800E-09	-249.00	-1.3800E-10
4100.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	234.00	171.00	135.00	0.0000	207.00	108.00	234.00
459.00	0.0000	0.0000						
8	-0.5510	-1.3700E-15	-6.6700E+05	-7.5800E-07	-2.9200E+04	-2.5100E-09	-206.00	-1.8200E-10

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3610.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
468.00       0.0000       0.0000
       9      -0.5510 -2.2800E-15 -6.5400E+05 -7.4300E-07 -2.8300E+04 -2.3500E-09     -198.00 -9.8900E-11
3610.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       252.00       180.00       144.00       0.0000       216.00       108.00       252.00
468.00       0.0000       0.0000
      10      -0.5510 -2.2800E-15 -6.5400E+05 -7.4300E-07 -2.8300E+04 -2.3500E-09     -198.00 -9.8900E-11
3610.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       252.00       180.00       144.00       0.0000       216.00       108.00       252.00
477.00       0.0000       0.0000
      11      -0.5510 -2.2800E-15 -6.5400E+05 -7.4300E-07 -2.8300E+04 -2.3500E-09     -198.00 -9.8900E-11
3610.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       252.00       180.00       144.00       0.0000       216.00       108.00       252.00
477.00       0.0000       0.0000
      12      -0.5510 -2.2800E-15 -6.5400E+05 -7.4300E-07 -2.8300E+04 -2.3500E-09     -198.00 -9.8900E-11
3610.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       252.00       180.00       144.00       0.0000       216.00       108.00       252.00
468.00       0.0000       0.0000
      13      -0.5510 -2.2800E-15 -6.5400E+05 -7.4300E-07 -2.8300E+04 -2.3500E-09     -198.00 -9.8900E-11
3610.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       252.00       180.00       144.00       0.0000       216.00       108.00       252.00
468.00       0.0000       0.0000
      14      -0.5510 -1.3700E-15 -6.6700E+05 -7.5800E-07 -2.9200E+04 -2.5100E-09     -206.00 -1.8200E-10
3610.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
477.00       0.0000       0.0000
      15      -0.5510 -7.0000E-16 -6.6600E+05 -4.8200E-07 -2.9200E+04 -1.6100E-09     -206.00 -1.2900E-10
3130.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
468.00       0.0000       0.0000
      16      -0.5510 -1.4500E-15 -6.5400E+05 -4.7200E-07 -2.8400E+04 -1.4900E-09     -198.00 -6.1400E-11
3130.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
477.00       0.0000       0.0000
      17      -0.5510 -1.4500E-15 -6.5400E+05 -4.7200E-07 -2.8400E+04 -1.4900E-09     -198.00 -6.1400E-11
3130.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
468.00       0.0000       0.0000
      18      -0.5510 -1.4500E-15 -6.5400E+05 -4.7200E-07 -2.8400E+04 -1.4900E-09     -198.00 -6.1400E-11
3130.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
468.00       0.0000       0.0000
      19      -0.5510 -1.4500E-15 -6.5400E+05 -4.7200E-07 -2.8400E+04 -1.4900E-09     -198.00 -6.1400E-11
3130.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
468.00       0.0000       0.0000
      20      -0.5510 -1.4500E-15 -6.5400E+05 -4.7200E-07 -2.8400E+04 -1.4900E-09     -198.00 -6.1400E-11
3130.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
468.00       0.0000       0.0000
      21      -0.5510 -7.0000E-16 -6.6600E+05 -4.8200E-07 -2.9200E+04 -1.6100E-09     -206.00 -1.2900E-10
3130.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       252.00       180.00       144.00       0.0000       225.00       108.00       252.00
468.00       0.0000       0.0000
      22      -0.5510 -4.7000E-16 -6.9500E+05 -1.5600E-07 -3.1100E+04 -5.0500E-10     -222.00 -1.9700E-11
2470.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       243.00       171.00       144.00       0.0000       216.00       108.00       243.00
459.00       0.0000       0.0000
      23      -0.5510 -4.9800E-16 -6.9100E+05 -1.5500E-07 -3.0900E+04 -4.9800E-10     -220.00 -1.6300E-11
2470.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       243.00       171.00       144.00       0.0000       216.00       108.00       243.00
459.00       0.0000       0.0000
      24      -0.5510 -5.0500E-16 -6.8900E+05 -1.5400E-07 -3.0800E+04 -4.9500E-10     -219.00 -1.5000E-11
2470.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       243.00       171.00       144.00       0.0000       216.00       108.00       243.00
459.00       0.0000       0.0000
      25      -0.5510 -5.0500E-16 -6.8900E+05 -1.5400E-07 -3.0800E+04 -4.9500E-10     -219.00 -1.5000E-11
2470.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       243.00       171.00       144.00       0.0000       216.00       108.00       243.00
459.00       0.0000       0.0000
      26      -0.5510 -4.9800E-16 -6.9100E+05 -1.5500E-07 -3.0900E+04 -4.9800E-10     -220.00 -1.6300E-11
2470.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       243.00       171.00       144.00       0.0000       216.00       108.00       243.00
459.00       0.0000       0.0000
      27      -0.5510 -4.7000E-16 -6.9500E+05 -1.5600E-07 -3.1100E+04 -5.0500E-10     -222.00 -1.9700E-11
2470.0  2.6200E+10  9.4500E+09
     x(IN)       0.0000       243.00       171.00       144.00       0.0000       216.00       108.00       243.00
459.00       0.0000       0.0000
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      28      -0.5510 -8.3700E-14 -7.0100E+05 -5.4400E-08 -3.1600E+04 -3.3100E-09     -227.00 -2.9000E-11
1610.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
      29      -0.5510 -8.3700E-14 -7.0000E+05 -5.4200E-08 -3.1500E+04 -3.3000E-09     -226.00 -2.8900E-11
1610.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
      30      -0.5510 -8.3700E-14 -7.0000E+05 -5.4200E-08 -3.1500E+04 -3.3000E-09     -226.00 -2.8900E-11
1610.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
      31      -0.5510 -8.3700E-14 -7.0000E+05 -5.4200E-08 -3.1500E+04 -3.3000E-09     -226.00 -2.8900E-11
1610.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
      32      -0.5510 -8.3700E-14 -7.0000E+05 -5.4200E-08 -3.1500E+04 -3.3000E-09     -226.00 -2.8900E-11
1610.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
      33      -0.5510 -8.3700E-14 -7.0100E+05 -5.4400E-08 -3.1600E+04 -3.3100E-09     -227.00 -2.9000E-11
1610.0  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
468.00      0.0000      0.0000
      34      -0.5510 -2.3600E-13 -7.0000E+05 -1.5300E-07 -3.1700E+04 -9.3500E-09     -227.00 -8.1700E-11
751.00  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
      35      -0.5510 -2.3600E-13 -6.9900E+05 -1.5200E-07 -3.1600E+04 -9.3200E-09     -226.00 -8.1400E-11
751.00  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
      36      -0.5510 -2.3600E-13 -6.9900E+05 -1.5200E-07 -3.1600E+04 -9.3200E-09     -226.00 -8.1400E-11
751.00  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
      37      -0.5510 -2.3600E-13 -6.9900E+05 -1.5200E-07 -3.1600E+04 -9.3200E-09     -226.00 -8.1400E-11
751.00  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
      38      -0.5510 -2.3600E-13 -6.9900E+05 -1.5200E-07 -3.1600E+04 -9.3200E-09     -226.00 -8.1400E-11
751.00  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
      39      -0.5510 -2.3600E-13 -7.0000E+05 -1.5300E-07 -3.1700E+04 -9.3500E-09     -227.00 -8.1700E-11
751.00  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      0.0000      0.0000      108.00      63.000
459.00      0.0000      0.0000
      40      -0.5510 -3.8800E-13 -6.9900E+05 -2.5100E-07 -3.1700E+04 -1.5400E-08     -227.00 -1.3500E-10
94.200  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
      41      -0.5510 -3.8800E-13 -6.9800E+05 -2.5000E-07 -3.1600E+04 -1.5400E-08     -226.00 -1.3400E-10
94.200  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
      42      -0.5510 -3.8800E-13 -6.9800E+05 -2.5000E-07 -3.1600E+04 -1.5400E-08     -226.00 -1.3400E-10
94.200  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
      43      -0.5510 -3.8800E-13 -6.9800E+05 -2.5000E-07 -3.1600E+04 -1.5400E-08     -226.00 -1.3400E-10
94.200  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
      44      -0.5510 -3.8800E-13 -6.9800E+05 -2.5000E-07 -3.1600E+04 -1.5400E-08     -226.00 -1.3400E-10
94.200  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
      45      -0.5510 -3.8800E-13 -6.9900E+05 -2.5100E-07 -3.1700E+04 -1.5400E-08     -227.00 -1.3500E-10
94.200  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
      46      -0.5510 -5.4100E-13 -6.9800E+05 -3.4800E-07 -3.1800E+04 -2.1500E-08     -227.00 -1.8700E-10
942.00  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
      47      -0.5510 -5.4100E-13 -6.9600E+05 -3.4700E-07 -3.1700E+04 -2.1500E-08     -226.00 -1.8700E-10
942.00  2.6200E+10  9.4500E+09
    x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
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468.00      0.0000      0.0000
      48     -0.5510 -5.4100E-13 -6.9600E+05 -3.4700E-07 -3.1700E+04 -2.1500E-08     -226.00 -1.8700E-10
942.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
      49     -0.5510 -5.4100E-13 -6.9600E+05 -3.4700E-07 -3.1700E+04 -2.1500E-08     -226.00 -1.8700E-10
942.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
      50     -0.5510 -5.4100E-13 -6.9600E+05 -3.4700E-07 -3.1700E+04 -2.1500E-08     -226.00 -1.8700E-10
942.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000
      51     -0.5510 -5.4100E-13 -6.9800E+05 -3.4800E-07 -3.1800E+04 -2.1500E-08     -227.00 -1.8700E-10
942.00  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      171.00      144.00      9.0000      9.0000      108.00      63.000
468.00      0.0000      0.0000

     Min.     -0.5510 -5.4100E-13 -7.4100E+05 -1.1400E-06 -3.4000E+04 -2.1500E-08     -249.00 -1.8700E-10
94.200  2.6200E+10  9.4500E+09
   Pile N.         1          46           1           1           1          46           1          46
40          1           1

       * MAXIMUM VALUES AND LOCATIONS *

     PILE          DEFLECTION                BENDING MOMENT          SHEAR FORCE          SOIL REACTION
TOTAL       FLEXURAL RIGIDITY
                 y-DIR       z-DIR       z-DIR       y-DIR       y-DIR       z-DIR       y-DIR       z-DIR
STRESS        z-DIR       y-DIR
                 IN          IN        LBS-IN      LBS-IN        LBS         LBS       LBS/IN      LBS/IN
LBS/IN**2   LBS-IN**2   LBS-IN**2
     *****  **********  **********  **********  **********  **********  **********  **********  **********
**********  **********  **********
       1      -0.5510  4.7500E-13  2.3600E+06  3.2700E-07      7790.0  2.0000E-08      197.00  1.8100E-10
2.2100E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      342.00      63.000
0.0000      0.0000      0.0000
       2      -0.5510  4.7500E-13  2.3500E+06  3.2600E-07      7770.0  2.0000E-08      197.00  1.8100E-10
2.2100E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      342.00      63.000
0.0000      0.0000      0.0000
       3      -0.5510  4.7500E-13  2.3500E+06  3.2600E-07      7770.0  2.0000E-08      197.00  1.8100E-10
2.2100E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      342.00      63.000
0.0000      0.0000      0.0000
       4      -0.5510  4.7500E-13  2.3500E+06  3.2600E-07      7770.0  2.0000E-08      197.00  1.8100E-10
2.2100E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      342.00      63.000
0.0000      0.0000      0.0000
       5      -0.5510  4.7500E-13  2.3500E+06  3.2600E-07      7770.0  2.0000E-08      197.00  1.8100E-10
2.2100E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      342.00      63.000
0.0000      0.0000      0.0000
       6      -0.5510  4.7500E-13  2.3500E+06  3.2600E-07      7770.0  2.0000E-08      197.00  1.8100E-10
2.2100E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      342.00      63.000
0.0000      0.0000      0.0000
       7      -0.5510  4.7500E-13  2.3600E+06  3.2700E-07      7790.0  2.0000E-08      197.00  1.8100E-10
2.2100E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      234.00      0.0000      342.00      63.000
0.0000      0.0000      0.0000
       8      -0.5510  3.4800E-13  2.1200E+06  2.1600E-07      6700.0  1.2700E-08      177.00  1.0900E-10
1.9800E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
0.0000      0.0000      0.0000
       9      -0.5510  3.4800E-13  2.0800E+06  2.1100E-07      6530.0  1.2300E-08      175.00  1.0500E-10
1.9500E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
0.0000      0.0000      0.0000
      10      -0.5510  3.4800E-13  2.0800E+06  2.1100E-07      6530.0  1.2300E-08      175.00  1.0500E-10
1.9500E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
0.0000      0.0000      0.0000
      11      -0.5510  3.4800E-13  2.0800E+06  2.1100E-07      6530.0  1.2300E-08      175.00  1.0500E-10
1.9500E+04  2.6200E+10  9.4500E+09
     x(IN)      0.0000      0.0000      0.0000      0.0000      252.00      0.0000      342.00      63.000
0.0000      0.0000      0.0000
      12      -0.5510  3.4800E-13  2.0800E+06  2.1100E-07      6530.0  1.2300E-08      175.00  1.0500E-10
1.9500E+04  2.6200E+10  9.4500E+09
```

Row 1

Row 2

x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
13	-0.5510	3.4800E-13	2.0800E+06	2.1100E-07	6530.0	1.2300E-08	175.00	1.0500E-10
1.9500E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
14	-0.5510	3.4800E-13	2.1200E+06	2.1600E-07	6700.0	1.2700E-08	177.00	1.0900E-10
1.9800E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
15	-0.5510	2.2100E-13	2.1200E+06	1.3700E-07	6690.0	8.0600E-09	177.00	6.9200E-11
1.9400E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
16	-0.5510	2.2100E-13	2.0800E+06	1.3400E-07	6520.0	7.8200E-09	175.00	6.6500E-11
1.9000E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
17	-0.5510	2.2100E-13	2.0800E+06	1.3400E-07	6520.0	7.8200E-09	175.00	6.6500E-11
1.9000E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
18	-0.5510	2.2100E-13	2.0800E+06	1.3400E-07	6520.0	7.8200E-09	175.00	6.6500E-11
1.9000E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
19	-0.5510	2.2100E-13	2.0800E+06	1.3400E-07	6520.0	7.8200E-09	175.00	6.6500E-11
1.9000E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
20	-0.5510	2.2100E-13	2.0800E+06	1.3400E-07	6520.0	7.8200E-09	175.00	6.6500E-11
1.9000E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
21	-0.5510	2.2100E-13	2.1200E+06	1.3700E-07	6690.0	8.0600E-09	177.00	6.9200E-11
1.9400E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	252.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
22	-0.5510	6.8600E-14	2.2200E+06	4.4100E-08	7090.0	2.6600E-09	187.00	2.3300E-11
1.9400E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	243.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
23	-0.5510	6.8600E-14	2.2000E+06	4.3900E-08	7030.0	2.6400E-09	185.00	2.3000E-11
1.9300E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	243.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
24	-0.5510	6.8600E-14	2.2000E+06	4.3800E-08	7000.0	2.6300E-09	185.00	2.2900E-11
1.9300E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	243.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
25	-0.5510	6.8600E-14	2.2000E+06	4.3800E-08	7000.0	2.6300E-09	185.00	2.2900E-11
1.9300E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	243.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
26	-0.5510	6.8600E-14	2.2000E+06	4.3900E-08	7030.0	2.6400E-09	185.00	2.3000E-11
1.9300E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	243.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
27	-0.5510	6.8600E-14	2.2200E+06	4.4100E-08	7090.0	2.6600E-09	187.00	2.3300E-11
1.9400E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	243.00	0.0000	342.00	63.000
0.0000	0.0000	0.0000						
28	-0.5510	4.5900E-16	2.2400E+06	1.9200E-07	7180.0	6.3600E-10	189.00	3.4900E-11
1.8700E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
29	-0.5510	5.0800E-16	2.2300E+06	1.9200E-07	7160.0	6.2800E-10	189.00	3.0200E-11
1.8700E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
30	-0.5510	5.0800E-16	2.2300E+06	1.9200E-07	7160.0	6.2800E-10	189.00	3.0200E-11
1.8700E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
31	-0.5510	5.0800E-16	2.2300E+06	1.9200E-07	7160.0	6.2800E-10	189.00	3.0200E-11
1.8700E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
32	-0.5510	5.0800E-16	2.2300E+06	1.9200E-07	7160.0	6.2800E-10	189.00	3.0200E-11

M3

RRN_WWA90.gp8t

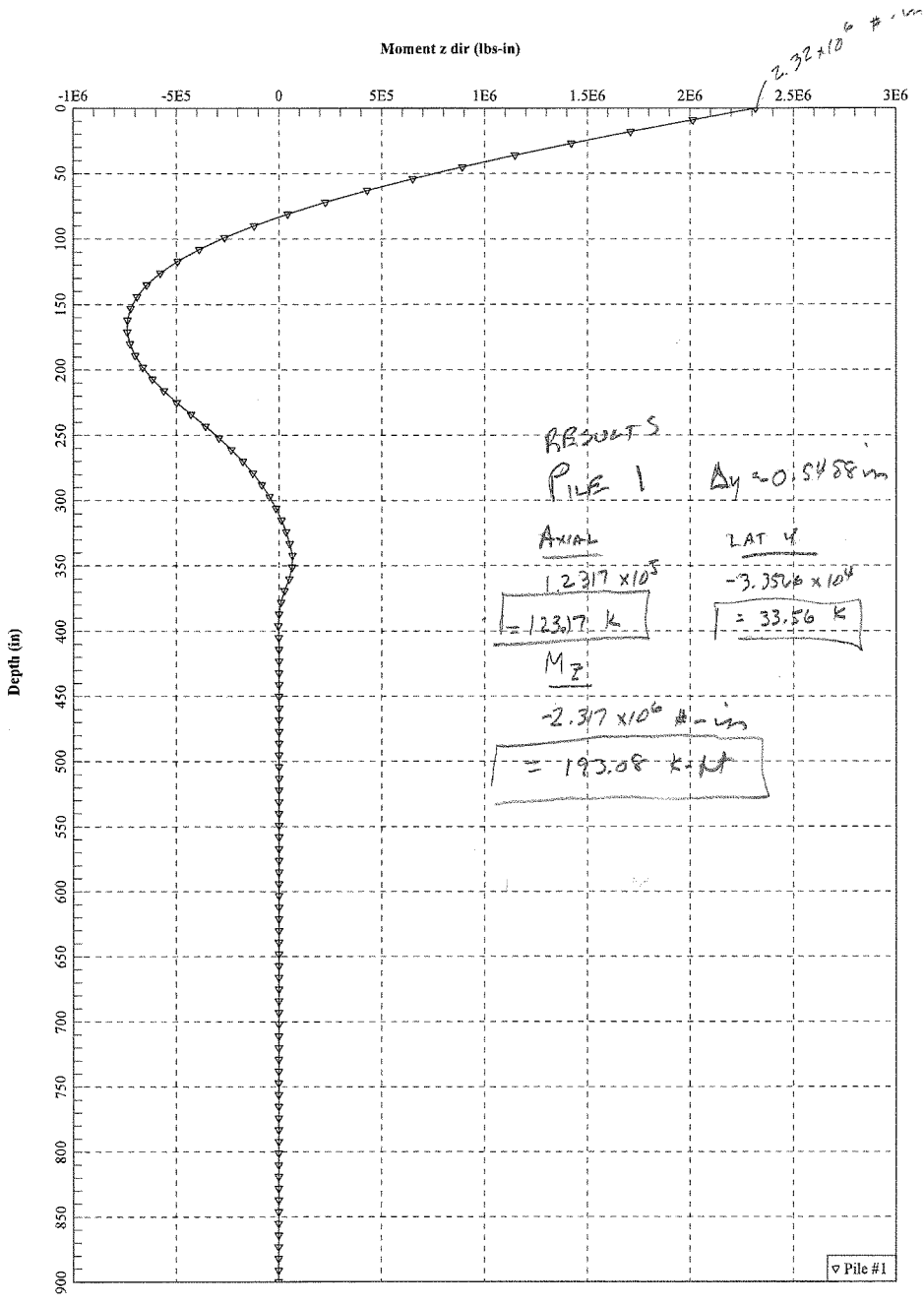
1.8700E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
33	-0.5510	4.5900E-16	2.2400E+06	1.9200E-07	7180.0	6.3600E-10	189.00	3.4900E-11
1.8700E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
34	-0.5510	1.3800E-15	2.2400E+06	5.4200E-07	7160.0	1.7700E-09	189.00	8.7900E-11
1.7900E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
35	-0.5510	1.4500E-15	2.2300E+06	5.4100E-07	7140.0	1.7500E-09	188.00	8.1000E-11
1.7800E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
36	-0.5510	1.4500E-15	2.2300E+06	5.4100E-07	7140.0	1.7500E-09	188.00	8.1000E-11
1.7800E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
37	-0.5510	1.4500E-15	2.2300E+06	5.4100E-07	7140.0	1.7500E-09	188.00	8.1000E-11
1.7800E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
38	-0.5510	1.4500E-15	2.2300E+06	5.4100E-07	7140.0	1.7500E-09	188.00	8.1000E-11
1.7800E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
39	-0.5510	1.3800E-15	2.2400E+06	5.4200E-07	7160.0	1.7700E-09	189.00	8.7900E-11
1.7900E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
40	-0.5510	2.3600E-15	2.2400E+06	8.9100E-07	7130.0	2.8800E-09	188.00	1.3400E-10
1.7200E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
41	-0.5510	2.4700E-15	2.2300E+06	8.9000E-07	7110.0	2.8600E-09	188.00	1.2300E-10
1.7200E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
42	-0.5510	2.4700E-15	2.2300E+06	8.9000E-07	7110.0	2.8600E-09	188.00	1.2300E-10
1.7200E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
43	-0.5510	2.4700E-15	2.2300E+06	8.9000E-07	7110.0	2.8600E-09	188.00	1.2300E-10
1.7200E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
44	-0.5510	2.4700E-15	2.2300E+06	8.9000E-07	7110.0	2.8600E-09	188.00	1.2300E-10
1.7200E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
45	-0.5510	2.3600E-15	2.2400E+06	8.9100E-07	7130.0	2.8800E-09	188.00	1.3400E-10
1.7200E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
46	-0.5510	3.4900E-15	2.2400E+06	1.2400E-06	7110.0	3.9500E-09	188.00	1.6000E-10
1.8100E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
47	-0.5510	3.5900E-15	2.2300E+06	1.2400E-06	7090.0	3.9200E-09	187.00	1.4800E-10
1.8000E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
48	-0.5510	3.5900E-15	2.2300E+06	1.2400E-06	7090.0	3.9200E-09	187.00	1.4800E-10
1.8000E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
49	-0.5510	3.5900E-15	2.2300E+06	1.2400E-06	7090.0	3.9200E-09	187.00	1.4800E-10
1.8000E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
50	-0.5510	3.5900E-15	2.2300E+06	1.2400E-06	7090.0	3.9200E-09	187.00	1.4800E-10
1.8000E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						
51	-0.5510	3.4900E-15	2.2400E+06	1.2400E-06	7110.0	3.9500E-09	188.00	1.6000E-10
1.8100E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	243.00	216.00	342.00	243.00
0.0000	0.0000	0.0000						

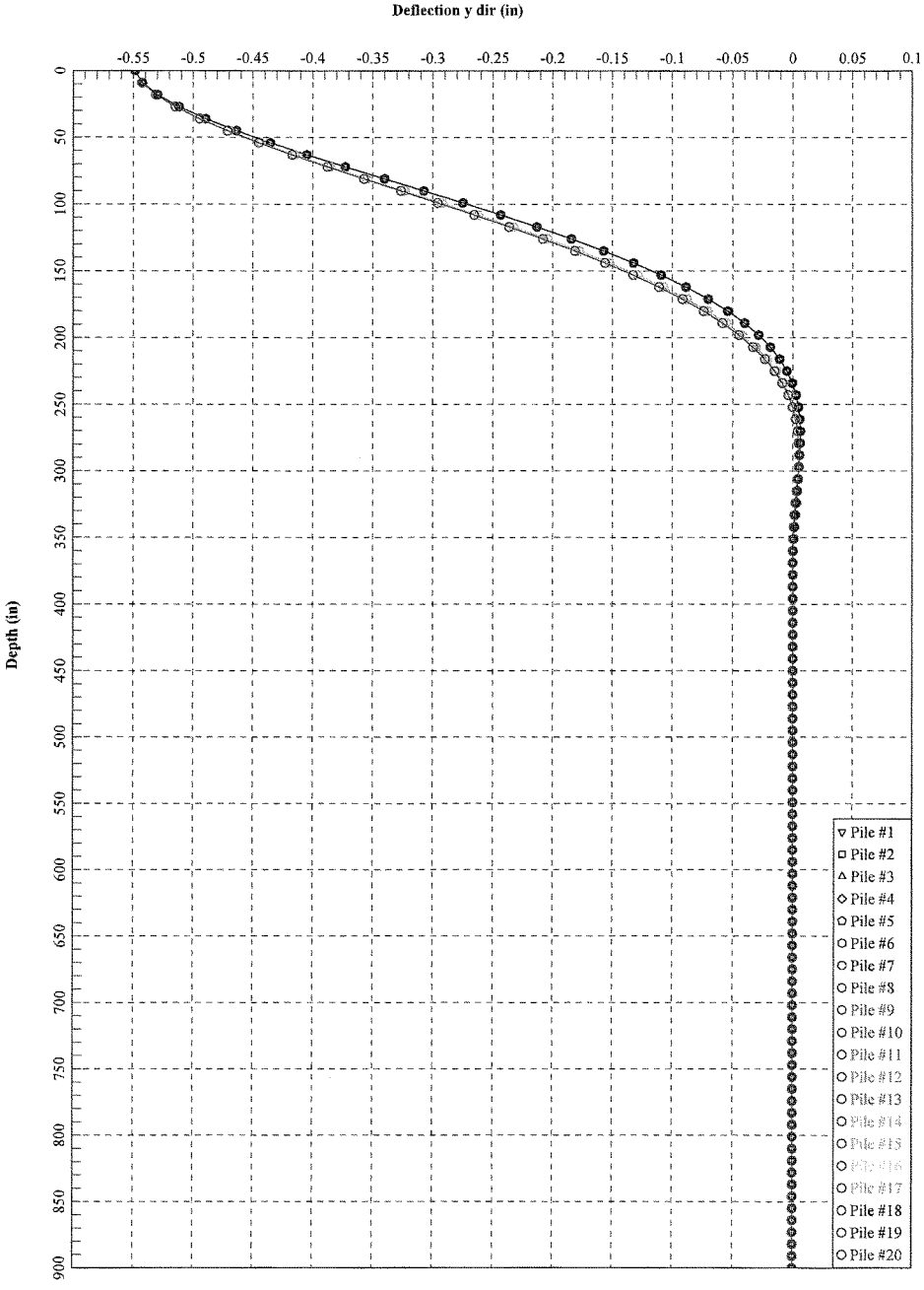
Row 6

Row 7

Row 9

Max.	-0.5510	4.7500E-13	2.3600E+06	1.2400E-06	7790.0	2.0000E-08	197.00	1.8100E-10
2.2100E+04	2.6200E+10	9.4500E+09						
Pile N.	1	1	1	46	1	1	1	1
1	1	1						


Moment z dir (lbs-in)
-1E6
-5E5
0
5E5
1E6
1.5E6
2E6
2.5E6
3E6
Depth (in)
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800
850
900
2.32 x 10^6 #-in
RESULTS
PILE 1
Δy = 0.5458 in
AXIAL
1.2317 x 10^5
= 123.17 k
LAT V
-3.3566 x 10^4
= 33.56 k
Mz
-2.317 x 10^6 #-in
= 193.08 k-ft
▽ Pile #1


Deflection y dir (in)
-0.55
-0.5
-0.45
-0.4
-0.35
-0.3
-0.25
-0.2
-0.15
-0.1
-0.05
0
0.05
0.1
Depth (in)
0
50
100
150
200
250
300
350
400
450
500
550
600
650
700
750
800
850
900
Pile #1
Pile #2
Pile #3
Pile #4
Pile #5
Pile #6
Pile #7
Pile #8
Pile #9
Pile #10
Pile #11
Pile #12
Pile #13
Pile #14
Pile #15
Pile #16
Pile #17
Pile #18
Pile #19
Pile #20

ATTACHMENT

F-Q1.4 Rigid Cap Analyses Results for the Red River (LPP) Control Structure Wing Wall F

BARR ENGINEERING			DATE 2/4/2011	SHEET NO.
			PROJECT NAME Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN 2/4/11		PKN	SUBJECT Control Structure: ND RRN 35k ≤35k Load Cases: 1	SECTION F

Case 1
Event 100 years

File:

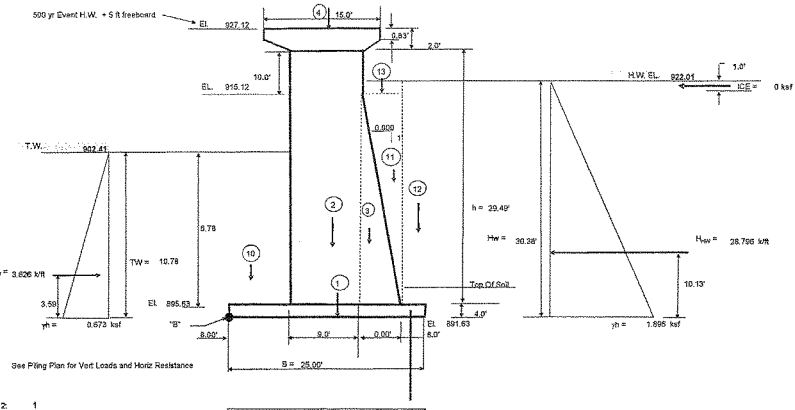
State Building Codes
Frost Depth = 5.0 ft — provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 891.2 ft — DEC, JAN, FEB Mean Water Elevation

H.W. EL. = 922.01 ft
T.W. EL. = 902.41 ft

500 yr H.W. EL. = 922.12 ft

Top of Ftg EL. = 895.63 ft

Non-Overflow Section — Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft — overlap distance at stepped ftg



Case 1 or 2: 1
Normal Water Level, El. 888.74 ft
Δh normal = 888.7 ft
See Geotechnical seepage model

UPLIFT

REVISED PER 7/1/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets
enter "0" if no cut-off

Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	30	25.00	4.00	0.15	rec	450.0	12.50	5,625.0
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	12.50	700.3
Bridge Slab	4	30	12.00	1.17	0.15	rec	63.2	12.50	789.8
Monolith	2	30	9.00	29.49	0.15	rec	1194.3	12.50	14,929.9
Monolith	3	30	0.00	19.49	0.15	tri	0.0	17.00	0.0
					D.L. Concrete		ΣVc = 1763.6		ΣMv = 22,044.4

← CONSTANT FOR ALL LOAD CASES

		L ft	W ft	Pressure ksf			U K	arm ft	Mu ft-k
T.W on ftg Monolith	10	30	8.00	8.78	0.0624	rec	101.5	4.00	406.1
H.W. on Monolith Slope	11	30	0.00	19.49	0.0624	tri	0.0	17.00	0.0
H.W. Above Slope	13	30	0.00	8.89	0.0624	rec	0.0	17.00	0.0
H.W. on Mono. ftg	12	30	8.00	29.38	0.0624	rec	395.1	21.00	8,296.4
					D.L. Water		ΣVw = 496.6		ΣMv = 8,702.6

Uplift Loads		L ft	W ft	Pressure ksf			U K	arm ft	Mu ft-k
	U_B	30	25.00	0.673		rec	-504.5	12.50	-6,306
	U_A	30	25.00	1.223		tri	-458.6	16.67	-7,644
						ΣU =	-963.1	ΣMu =	-13,950

Horizontal Loads		L ft	H ft	Pressure ksf			ICE K	arm ft	Mu ft-k
	ICE	30	1.00	0.00		rec	0.0	29.88	0.0
		L		Force			H	arm	Mw
Water Loads		ft		k/ft			K	ft	ft-k
	H_{hw}	30		3.626		tri	108.77	3.59	390.65
	H_{hw}	30		-28.796		tri	-863.88	10.13	-8748.18
						ΣH =	-755.10	ΣMw =	-8357.3

REVISED PER 2/1/11 MEETING WITH CORPS
Ice Loading is for Ice Floe on single Pier Only on Gated Structure
use 500 plf along Wing Walls

Overturning Moments	$\Sigma M_{OT} = M_V + M_W + M_{ICE}$ =	-22308 kip-ft
Resisting Moments	$\Sigma M_R = M_U$ =	30747 kip-ft

Sum of Moments	ΣMnet = $M_R + M_{OT}$ =	8,439 kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	1,297 kips
Sum of Horizontal Forces	H = Ice + water =	-755 kips

Location of Resultant	Xr = ΣM / P =	6.51 ft from Toe
	e = B/2 - Xr =	5.99 ft
	B/6 =	4.167 ft

BARR ENGINEERING			DATE 2/4/2011			SHEET NO. 1
			PROJECT NAME Flood Control ND Diversion Inlet - Corps Of Engineers			
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER			
PKN		PKN	SUBJECT Control Structure:	NO RRN 35k	s35k	
2/4/11			Load Cases: 1,1			SECTION F

Case 1,1
Event 100 years

H.W. EL. = 922.01 ft
T.W. EL. = 902.41 ft

500 yr H.W. EL. = 922.12 ft

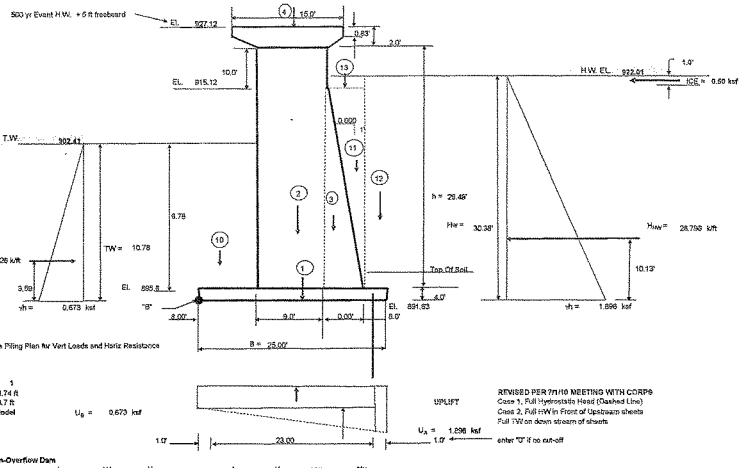
Top of Pig EL. = 893.63 ft

File:

State Building Codes
Frost Depth = 5.0 ft — provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.2 ft — DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section

Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft — overlap distance at stepped ftg



See Piling Plan for Vert Loads and Horiz Resistance

Case 1 or 2: 1
Normal Water Level, El. 888.74 ft
Δh normal = 888.7 ft
See Geotechnical seepage Model

U_B = 0.673 ksf

UPLIFT

REVISED PER 7/1/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

U_A = 1.896 ksf

1.0' — 23.00 — 1.0'

enter "0" if no cut-off

Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	30	25.00	4.00	0.15	rec	450.0	12.50	5,625.0
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	12.50	700.3
Bridge Slab	4	30	12.00	1.17	0.15	rec	63.2	12.50	790.8
Monolith	2	30	9.00	29.49	0.15	rec	1194.3	12.50	14,929.9
Monolith	3	30	0.00	19.49	0.15	tri	0.0	17.00	0.0
						D.L. Concrete	ΣVc =	1783.6	ΣMv =	22,044.4

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Monolith	10	30	8.00	6.78	0.0624	rec	101.8	4.00	406.1
H.W. on Monolith Slope	11	30	0.00	19.49	0.0624	tri	0.0	17.00	0.0
H.W. Above Slope	13	30	0.00	8.80	0.0624	rec	0.0	17.00	0.0
H.W. on Mono. ftg	12	30	8.00	36.36	0.0624	rec	363.1	21.00	8,256.4
						D.L. Water	ΣVw =	464.9	ΣMw =	8,662.6

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	30	25.00	0.673	rec	-504.5	12.50	-6,306
	U_A	30	25.00	1.223	tri	-458.6	16.67	-7,644
					ΣU =	-963.1	ΣMu =	-13,950

Horizontal Loads		L ft	H ft	Pressure ksf		ICE K	arm ft	Mu ft-k
	ICE	30	1.00	-0.50	rec	-15.0	29.88	-448.2

REVISED PER 9/1/11 MEETING WITH CORPS
Ice Loading is for Ice Floe on single Pier Only on Gated Structure
use 500 plf along Wing Walls

	L ft	Force k/ft		H K	arm ft	Mw ft-k
Water Loads						
H_{TW}	30	3.626	tri	108.77	3.63	393.85
H_{HW}	30	-28.756	tri	-903.88	10.13	-9148.18
			ΣW =	-758.19	ΣMw =	-8357.3

Overturning Moments	ΣMot = M_U + M_W + M_{ICE} =	-22795 kip-ft
Resisting Moments	$ΣM_R$ = M_V =	30747 kip-ft
Sum of Moments	ΣMnet = M_R + Mot =	7,951 kip-ft
Sum of Vertical Forces	P = Conc + Water + UpliR =	1,297 kips
Sum of Horizontal Forces	H = Ice + water =	-778 kips

Location of Resultant
Xr = ΣM / P = 6.16 ft from Toe
e = B/2 - Xr = 6.34 ft
B/3 = 4.167 ft

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER		
PKN 2/4/11		PKN	SUBJECT	Control Structure; ND RRN 35k x35k Load Cases: 2	SECTION F

Case 2
Event 500 years

H.W. EL. = 922.12 ft
T.W. EL. = 914.41 ft

500 yr H.W. EL. = 922.12 ft

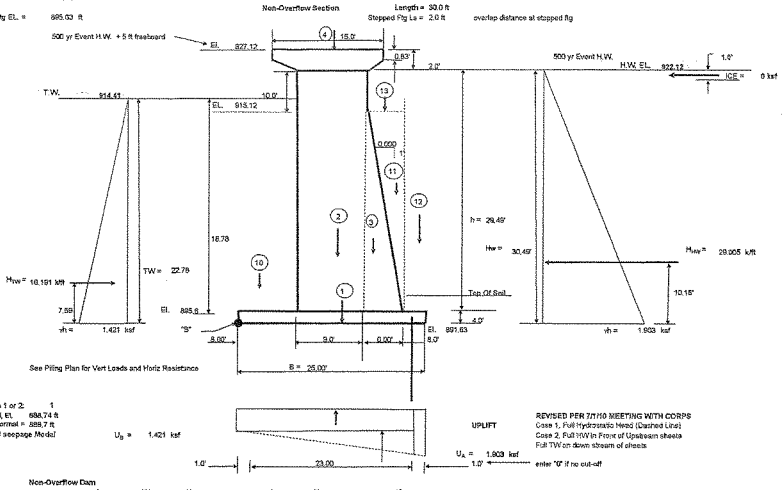
Top of Pfg EL. = 895.63 ft

File:

State Building Code
Frost Depth = 5.0 ft — provide min frost flg protection during Dec, Jan, Feb, March
Water El. = 891.2 ft — DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section

Length = 30.0 ft
Stepped Flg Le = 2.0 ft — overlap distance at stepped flg



See Piling Plan for Vert Loads and Horiz Resistance

Case 1 or 2: 1
Normal Water Level, El. 888.74 ft
Δh normal = 888.7 ft
See Geotechnical seepage Model — U_B = 1.421 ksf

UPLIFT — U_A = 1.903 ksf

REVISED PER 7/7/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Frove of Upstream sheets
Full TW on down stream of sheets
enter "0" if no cut-off

Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Flg concrete	1	30	25.00	4.00	0.15	rec	450.0	12.50	5,625.0
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	12.50	700.3
Bridge Slab	4	30	12.00	1.17	0.15	rec	63.2	12.50	789.8
Monolith	2	30	9.00	28.49	0.15	rec	1154.3	12.50	14,429.3
Monolith	3	30	6.00	19.49	0.15	tri	52.6	17.00	0.0
					D.L. Concrete		ΣVc = 1783.6	ΣMv =	22,044.4

CONSTANT FOR ALL LOAD CASES

		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
T.W on flg Monolith	10	30	6.00	18.78	0.0624 rec	281.2	4.00	1,125.0
H.W. on Monolith Slope	11	30	0.00	19.49	0.0624 tri	0.0	17.00	0.0
H.W. Above Slope	13	30	0.00	7.00	0.0624 rec	0.0	17.00	0.0
H.W. on Mono. flg	12	30	6.00	28.49	0.0624 rec	392.7	21.00	8,321.0
					D.L. Water	ΣW = 673.9		ΣMv = 9,446.0

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	30	25.00	1.421	rec	-1065.1	12.50	-13,309
	U_A	30	25.00	0.481	tri	-180.4	18.67	-3,007
						ΣU = -1246.6		ΣMu = -16,333

Horizontal Loads		L ft	H ft	Pressure ksf		ICE K	arm ft	Mo ft-k
	ICE	30	1.00	0.00	rec	0.0	29.95	0.0

REVISED PER 7/7/11 MEETING WITH CORPS
Ice Loading is for Ice Floe on single Pier Only on Gated Structure
use 600 plf along Wing Walls

Water Loads	L ft	Force k/ft		H K	arm ft	Mw ft-k
H_{hw}	30	16.191	tri	485.72	7.59	3688.21
H_{ww}	30	-29.905	tri	-470.14	10.16	-5843.55
				ΣW = -384.43	ΣMw =	-5155.3

Overturning Moments	$\Sigma M_{OT} = M_o + M_W + M_{ICE}$ =	-21485 kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	31500 kip-ft

Sum of Moments	ΣMnet = $M_R + M_{OT}$ =	10,012 kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	1,196 kips
Sum of Horizontal Forces	H = Ice + water =	-384 kips

Location of Resultant	Xr = ΣM / P =	8.38	ft from Toe
	e = B/2 - Xr =	4.12	ft
	B/6 =	4.167	ft

BARR ENGINEERING			DATE 2/4/2011	SHEET NO. 1
			PROJECT NAME Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN 2/4/11		PKN	SUBJECT Control Structure: ND RRN 35k ≤35k Load Cases: 2.1	SECTION F

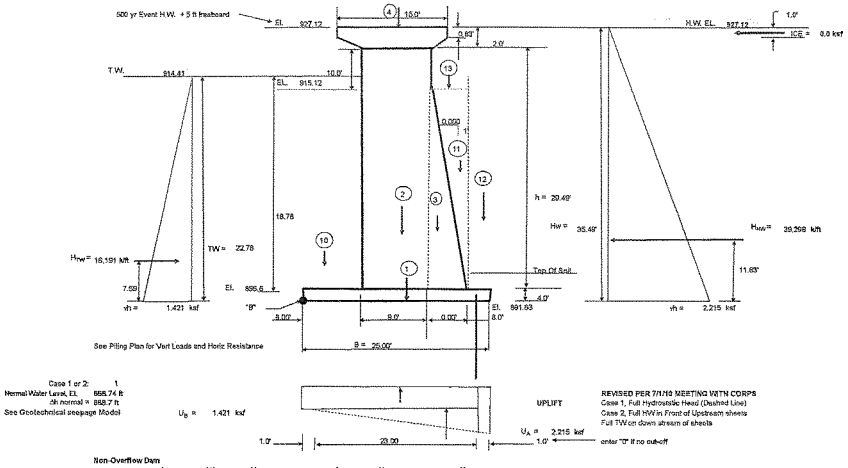
Case 2.1 — Per COE 2/1/11: HW = 500yr + 3' w/ No Ice — Pile:

Event 500 years

State Building Codes
Frost Depth = 5.0 ft — provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.2 ft — DEC, JAN, FEB Mean Water Elevation

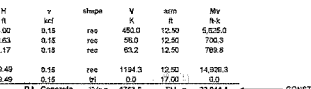
H.W. EL. = 927.12 ft
T.W. EL. = 914.41 ft
500 yr H.W. EL = 922.12 ft
Top of Ftg EL. = 895.53 ft

Non-Overflow Section
Length = 30.0 ft
Stepped Ftg Ls = 2.0 ft — overlap distance at stepped ftg



See Piling Plan for Vert Loads and Horiz Resistance

Case 1 or 2: 1
Normal Water Level, El. 885.74 ft
dh normal = 888.7 ft
See Geotechnical seepage Model — U_B = 1.421 ksf



UPLIFT — U_A = 2.215 ksf — 1.0' — enter "0" if no cut-off

REVISED PER 7/1/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	30	25.00	4.00	0.15	rec	450.0	12.50	5,625.0
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	12.50	700.3
Bridge Slab	4	30	12.00	1.17	0.15	rec	63.2	12.50	789.8
Monolith	2	30	8.00	29.49	0.15	rec	1184.3	12.50	14,800.3
Monolith	3	30	0.00	18.49	0.15	tri	0.0	17.00	0.0
						D.L. Concrete	ΣVc = 1763.6	ΣMv =	22,044.4

CONSTANT FOR ALL LOAD CASES

T.W on ftg Monolith	10	30	8.00	18.78	0.0624	rec	281.2	4.00	1,125.0
H.W. on Monolith Slope	11	30	0.00	18.49	0.0624	tri	0.0	17.00	0.0
H.W. Above Slope	13	30	0.00	12.00	0.0624	rec	0.0	17.00	0.0
H.W. on Mono. ftg	12	30	8.00	31.49	0.0624	rec	471.6	21.00	9,903.5
						D.L. Water	ΣVw = 752.8	ΣMv =	11,028.5

Uplift Loads		L ft	W ft	Pressure ksf			U K	arm ft	Mu ft-k
	U_B	30	25.00	1.421		rec	-1066.1	12.50	-13,326
	U_A	30	25.00	0.783		tri	-287.4	16.67	-4,957
							ΣU = -1343.5	ΣMu =	-18,283

Horizontal Loads		L ft	H ft	Pressure ksf			ICE K	arm ft	Mu ft-k
	ICE	30	1.00	0.00		rec	0.0	34.99	0.0
		L ft		Force k/ft			H K	arm ft	Mw ft-k
Water Loads	H_{TW}	30		16.191		tri	485.72	7.59	3688.21
	H_{HW}	30		-39.298		tri	-1178.93	11.63	-13946.74
							ΣW = -693.21	ΣMw =	-10258.5

REVISED PER 2/1/11 MEETING WITH CORPS
Ice Loading is for Ice Floe on single Pier Only on Gated Structure
use 500 plf along Wing Walls

Overturning Moments	$\Sigma M_{OT} = M_S + M_W + M_{ICE}$ =	-26542 kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	33073 kip-ft

Sum of Moments	ΣMnet = $M_R + M_{OT}$ =	6,531 kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	1,153 kips
Sum of Horizontal Forces	H = Ice + water =	-693 kips

Location of Resultant	Xr = ΣM / P =	3.93 ft from Toe
	e = B/2 - Xr =	8.57 ft
	B/6 =	4.167 ft

BARR ENGINEERING			DATE 2/4/2011					SHEET NO.
			PROJECT NAME Flood Control ND Diversion Inlet - Corps Of Engineers					
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER					
PKN		PKN	SUBJECT	Control Structure:	ND RRN	35k	s35k	
2/4/11				Load Cases: 3				SECTION F

Case 3
Event 2 years

H.W. EL = ft
T.W. EL = ft

500 yr H.W. EL = 922.12 ft

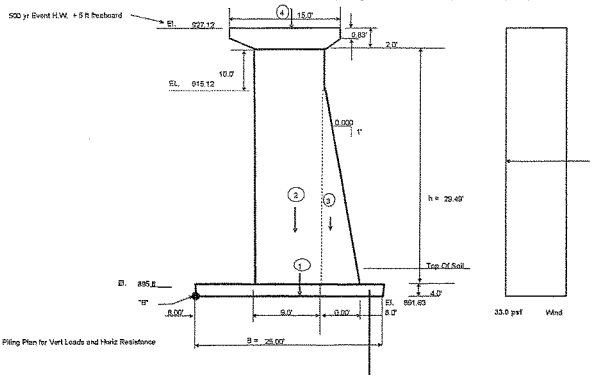
Top of Pig EL. = 895.63 ft

Pile:

State Building Codes
Frost Depth = 5.0 ft — provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.2 ft — DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section

Length = 50.0 ft
Stepped Ftg Ls = 2.0 ft — overlap distance at stepped ftg



Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	30	25.00	4.00	0.15	rec	450.0	12.50	5,625.0
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	12.50	700.3
Bridge Slab	4	30	12.00	1.17	0.15	rec	63.2	12.50	789.8
Monolith	2	30	8.00	29.49	0.15	rec	1184.3	12.50	14,803.3
Monolith	3	30	0.00	19.49	0.15	tri	0.0	17.00	0.0
					D.L. Concrete		ΣVc = 1761.6		ΣMy = 22,044.4

← CONSTANT FOR ALL LOAD CASES

Horizontal Loads	L ft	H ft	Pressure ksf		H K	arm ft	Mv ft-k
Wind	30	35.49	-0.03	rec	-35.1	17.745	-623.5

Overturning Moments	$\Sigma M_{OT} = M_W =$	-623 kip-ft
Resisting Moments	$\Sigma M_R = M_V =$	22044 kip-ft

Sum of Moments	$\Sigma M_{net} = M_R + M_{OT} =$	21,421 kip-ft
Sum of Vertical Forces	P = Conc. =	1,764 kips
Sum of Horizontal Forces	H = Wind =	-35 kips

Location of Resultant
$X_r = \Sigma M / P =$ 12.15 ft from Toe
$e = B/2 - X_r =$ 0.35 ft
$B/6 =$ 4.167 ft

BARR ENGINEERING			DATE 3/4/2011				SHEET NO. 1
			PROJECT NAME Flood Control ND Diversion Inlet - Corps Of Engineers				
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER				
PKN		PKN	SUBJECT Control Structure:	ND RRN 35k	s36k		
3/4/11			Load Cases: 4			SECTION F	

Case 4
Event years

File:

H.W. EL. = 888.74 ft
T.W. EL. = 888.74 ft
Normal Water Level

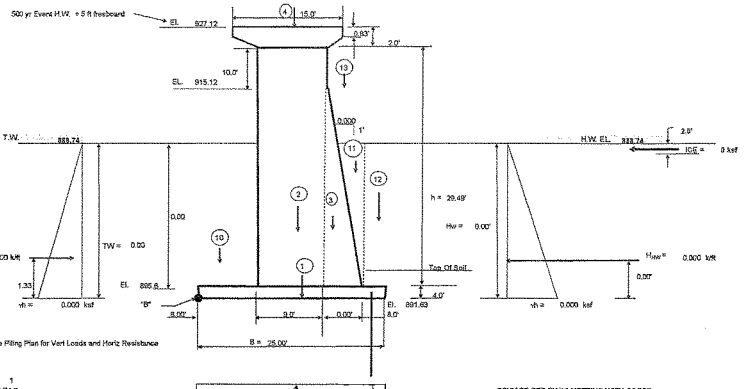
500 yr H.W. EL. = 922.12 ft

Top of Fig EL. = 896.63 ft

State Building Codes
Frost Depth = 5.0 ft — provide min frost fig protection during Dec, Jan, Feb, March
Water El. = 881.2 ft — DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section

Length = 30.0 ft
Stepped Fig Le = 2.0 ft — overlap distance at stepped ftg



Case 1 or 2: 1
Normal Water Level, El. 888.74 ft
At normal = 888.7 ft
See Geotechnical seepage Model

U_B = 0.000 ksf

U_A = 0.000 ksf
enter "0" if no cut-off

UPLIFT

REVISED PER 7/1/10 MEETING WITH CORPS
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

Non-Overflow Dam

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Fig concrete	1	30	25.00	4.00	0.15	rec	450.0	12.50	5,625.0
Bridge Slab	4	30	15.00	0.83	0.15	rec	56.0	12.50	700.3
Bridge Slab	4	30	12.00	1.17	0.15	rec	63.2	12.50	789.8
Monolith	2	30	9.00	29.49	0.15	rec	1194.3	12.50	14,929.3
Monolith	3	30	0.00	18.49	0.15	tri	0.0	17.00	0.0
						D.L. Concrete	ΣVc = 1763.6	ΣMv =	22,044.4

CONSTANT FOR ALL LOAD CASES

		L	W	H	γ	shape	V	arm	Mv
T.W on ftg Monolith	10	30	8.00	0.00	0.0624	rec	0.0	4.00	0.0
H.W. on Monolith Slope	11	30	0.00	-8.83	0.0624	tri	0.0	17.00	0.0
H.W. Above Slope	13	30	0.00	-26.38	0.0624	rec	0.0	17.00	0.0
H.W. on Mono. ftg	12	30	8.00	0.00	0.0624	rec	0.0	21.00	0.0
						D.L. Water	ΣVw = 0.0		ΣMw = 0.0

Uplift Loads		L ft	W ft	Pressure ksf			U K	arm ft	Mu ft-k
	U_B	30	25.00	0.000		rec	0.0	12.50	0
	U_A	30	25.00	0.000		tri	0.0	16.67	0
							ΣU = 0.0	ΣMu =	0

Horizontal Loads		L ft	H ft	Pressure ksf			ICE K	arm ft	Mu ft-k
	ICE	30	2.00	0.00		rec	0.0	-1	0.0
		L		Force			H	arm	Mw
Water Loads		ft		k/ft			K	ft	ft-k
	H_{hw}	30		0.000		tri	0.00	1.33	0.00
	H_{hw}	30		0.000		tri	0.00	0.00	0.00
							ΣW = 0.00	ΣMH =	0.0

REVISED PER 3/1/11 MEETING WITH CORPS
Ice Loading is for Ice Plug on single Pier Only on Gated Structure
use 600 plf along Wing Walls

Overturning Moments	ΣMot = M_U + M_W + M_{ICE} =	0 kip-ft
Resisting Moments	ΣMr = M_V =	22044 kip-ft

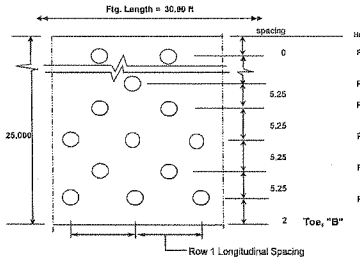
Sum of Moments	ΣMnet = M_R + M_{OT} =	22,044 kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	1,764 kips
Sum of Horizontal Forces	H = Ice + water =	0 kips

Location of Resultant: Xr = ΣM / P = 12.50 ft from Toe
e = B/2 - Xr = - ft
B/6 = 4.167 ft

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED PKN 2/4/11	CHECKED	SUBMITTED PKN	PROJECT NUMBER		
			SUBJECT	Pile Capacity Control Structure: 35k	SECTION F

PILE FOUNDATION DESIGN

FLOW



L = 30.00 ft

PILE PATTERN GEOMETRY

Transverse Spacing		Distance to Toe, d_{toe}	Longitudinal Spacing	Batter	Piles per Row (N)	Edge Dist (ft)		Trial N	
Row 1 to Toe	2.00 ft	2.0 ft	4.25 ft	0 °/12"	6	4.38	1	6	
Row 1 to Row 2	5.25 ft	7.3 ft	4.25 ft	0 °/12"	6	4.38	2	8	12
Row 2 to Row 3	5.25 ft	12.5 ft	5.00 ft	0 °/12"	5	5.00	0	7	17
Row 3 to Row 4	5.25 ft	17.8 ft	5.00 ft	0 °/12"	6	2.50	1	7	23
Row 4 to Row 5	5.25 ft	23.0 ft	4.25 ft	0 °/12"	6	4.38	2	8	29
Row 5 to Row 6	0.00 ft	0.0 ft	0.00 ft	0 °/12"	0	15.00	0	0	35
Row 6 to Row 7	0.00 ft	0.0 ft	0.00 ft	0 °/12"	0	15.00	0	0	41
Row 7 to Row 8	0.00 ft	0.0 ft	0.00 ft	0 °/12"	0	15.00	0	0	47
Row 8 to Row 9	0.00 ft	0.0 ft	0.00 ft	0 °/12"	0	15.00	0	0	
Row 9 to Row 10	0.00 ft	0.0 ft	0.00 ft	0 °/12"	0	15.00	0	0	
Row 10 to Row 11	0.00 ft	0.0 ft	0.00 ft	0 °/12"	0	15.00	0	0	
Row 11 to Row 12	0.00 ft	0.0 ft	0.00 ft	0 °/12"	0	15.00	0	0	
Row 12 to Row 13	0.00 ft	0.0 ft	0.00 ft	0 °/12"	0	15.00	0	0	
Row 13 to Row 14	0.00 ft	0.0 ft	0.00 ft	0 °/12"	0	15.00	0	0	
Row 14 to Row 15	0.00 ft	0.0 ft	0.00 ft	0 °/12"	0	15.00	0	0	
Last Row to Heel	2.00 ft								
	25.00 ft								
			Note: Enter 0 for Longitudinal Spacing for Rows Not Used		ΣN =	29		38	

Pile Properties:	Pile Type: HP	(C.I.P or HP)	Pile Length = 56.8 ft	Ftg El.	891.63
				Pile Tip El.	835.8
	HP Nominal Depth, h = 14.0 in			Pile Cap Embed =	1.00 ft
	Wt. per ft, plf 74		Total pile Length =	1,648 LF	

Pile Group Properties

N.A. of Pile Group to Toe

$X_{NA} = (\Sigma N * d_{toe}) / \Sigma N =$ 12.50 ft

	Dist. From N.A. to Pile Row	d	N	$I = N * d^2$
1	Dist. To Row 1	10.50 ft	6	661.5
2	Dist. To Row 2	5.25 ft	6	165.4
0	Dist. Row 3	0.00 ft	5	0.0
1	Dist. Row 4	-5.25 ft	6	165.4
2	Dist. Row 5	-10.50 ft	6	661.5
0	Row 6 (not used)	0.00 ft	0	0.0
0	Row 7 (not used)	0.00 ft	0	0.0
0	Row 8 (not used)	0.00 ft	0	0.0
0	Row 9 (not used)	0.00 ft	0	0.0
0	Row 10 (not used)	0.00 ft	0	0.0
0	Row 11 (not used)	0.00 ft	0	0.0
0	Row 12 (not used)	0.00 ft	0	0.0
0	Row 13 (not used)	0.00 ft	0	0.0
0	Row 14 (not used)	0.00 ft	0	0.0
0	Row 15 (not used)	0.00 ft	0	0.0
			29	$\Sigma I =$ 1653.8

ALLOWABLE LOADS (from Geotechnical)

Service	Allowable Pile Loads						
Case	1 Usual	1.1 Unsual	2 Unusual	2.1 Extreme	3 Unusual	4 Usual	
Axial	59.8 tons	79.7 tons	79.7 tons	104.0 tons	79.7 tons	31.8 tons	
Tension	40.6 tons	54.1 tons	54.1 tons	70.6 tons	54.1 tons	7.6 tons	
Lateral	31.0 kips/pile	35.0 kips/pile	35.0 kips/pile	35.0 kips/pile	31.0 kips/pile	22.0 kips/pile	w/o Group effects

Summary Pile Reactions

Load Combinations	Pile Loads (tons/pile)															Max. Vertical Load (Tons)	Horiz Pile Group Capacity (k)	
Allowable Pile Loads	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15			
1 59.8 tons	47.0	34.7	22.4	10.0	-2.3	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	47.0	899	OK
1.1 79.7 tons	48.5	35.4	22.4	9.3	-3.7	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	48.5	1,015	OK
2 79.7 tons	36.2	28.4	20.6	12.8	5.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	36.2	1,015	OK
2.1 104.0 tons	51.2	35.6	19.9	4.2	-11.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	51.2	1,015	OK
3 79.7 tons	32.4	31.4	30.4	29.4	28.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	32.4	1,015	OK
4 31.8 tons	30.4	30.4	30.4	30.4	30.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	30.4	638	OK
															Max Service : P =	51.2		

Using solid mechanics equations adapted for discrete elements, the forces in the pile rows for different load combinations are determined.
The force in each pile row is found using:

$$\text{Pile Load} = P / N + M_{toe} / I$$

First, the moment about the toe must be translated to get the moment about the neutral axis of the pile group.

$$e_{toe} = M_{toe} / P$$

Then the eccentricity about the neutral axis of the pile group is

$$e_{NA} = X_{NA} - e_{toe}$$

The moment about the neutral axis of the pile group becomes

$$M_{NA} = P * e_{NA}$$

For battered pile, the Vertical pile load needs to be transformed to the axial load along the pile axis

Paxial = 1.000 Pvert

BARR ENGINEERING			DATE 2/4/2011	SHEET NO.
			PROJECT NAME Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED PKN 2/4/11	CHECKED	SUBMITTED PKN	PROJECT NUMBER SUBJECT Pile Capacity Control Structure: 35k	SECTION F

FORCE RESULTANT (see Stability Analysis)

CASE	Event	Vertical Load P (kips)	Horizontal Load H (kips)	ΣM_{toe} (kip-ft)	$e_{toe} = M_{toe} / P$ (ft)	$e_{HA} = X_{wh} - e_{toe}$ (ft)	$M_{HA} = P * e_{HA}$ (kip-ft)	
1	100	1,297	755	5,439	6.51	5.99	7773	Usual
1.1	100	1,297	770	7,991	5.16	6.34	8222	Unusual
2	500	1,199	384	10,012	6.38	4.12	4926	Unusual
2.1	500	1,153	693	4,531	3.93	8.57	9880	Extreme
3	Dry	1,764	35	21,421	12.15	0.35	623	Unusual
4	Daily	1,764	0	22,044	12.50	0.00	0	Usual

SERVICE

Case 1
Flood Event **100**
Usual

Vertical Load, P = 1297 kips
Horizontal Load, H = 755 kips
M_{HA} = 7773 kip-ft

29

Vertical Pile Loading	P / N	+	$M_{HA} * d / \Sigma I$	= Pile Loads		Axial Pile Load
1 Row 1	44.7		49.4	94.1 kips/pile	47.0 tons/pile	47.0 tons/pile
2 Row 2	44.7		24.7	69.4 kips/pile	34.7 tons/pile	34.7 tons/pile
3 Row 3	44.7		0.0	44.7 kips/pile	22.4 tons/pile	22.4 tons/pile
4 Row 4	44.7		-24.7	20.0 kips/pile	10.0 tons/pile	10.0 tons/pile
5 Row 5	44.7		-49.4	-4.6 kips/pile	-2.3 tons/pile	-2.3 tons/pile
6 Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
7 Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
8 Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
9 Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10 Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11 Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12 Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13 Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14 Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15 Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
				max:	47.0 tons/pile	max: 47.0 tons/pile

Assumed lateral Capacity: 31.0 kips/pile

Horizontal Pile Capacity	Batter °/ft	N	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1 Row 1	0	6	0.0	186	1.000	186 kips	
2 Row 2	0	6	0.0	186	1.000	186 kips	
3 Row 3	0	5	0.0	155	1.000	155 kips	
4 Row 4	0	6	0.0	186	1.000	186 kips	
5 Row 5	0	6	0.0	186	1.000	186 kips	
6 Row 6	0	0	0.0	0	1.000	0 kips	
7 Row 7	0	0	0.0	0	1.000	0 kips	
8 Row 8	0	0	0.0	0	1.000	0 kips	
9 Row 9	0	0	0.0	0	1.000	0 kips	
10 Row 10	0	0	0.0	0	1.000	0 kips	
11 Row 11	0	0	0.0	0	1.000	0 kips	
12 Row 12	0	0	0.0	0	1.000	0 kips	
13 Row 13	0	0	0.0	0	1.000	0 kips	
14 Row 14	0	0	0.0	0	1.000	0 kips	
15 Row 15	0	0	0.0	0	1.000	0 kips	
		29		699		899 kips	OK

Case 1.1
Flood Event **100**
Unusual

Vertical Load, P = 1297 kips
Horizontal Load, H = 770 kips
M_{HA} = 8222 kip-ft

29

Vertical Pile Loading	P / N	+	$M_{HA} * d / \Sigma I$	= Pile Loads		Axial Pile Load
1 Row 1	44.7		52.2	96.9 kips/pile	48.5 tons/pile	48.5 tons/pile
2 Row 2	44.7		26.1	70.8 kips/pile	35.4 tons/pile	35.4 tons/pile
3 Row 3	44.7		0.0	44.7 kips/pile	22.4 tons/pile	22.4 tons/pile
4 Row 4	44.7		-26.1	18.6 kips/pile	9.3 tons/pile	9.3 tons/pile
5 Row 5	44.7		-52.2	-7.5 kips/pile	-3.7 tons/pile	-3.7 tons/pile
6 Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
7 Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
8 Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
9 Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10 Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11 Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED PKN 2/4/11	CHECKED	SUBMITTED PKN	PROJECT NUMBER		
			SUBJECT	Pile Capacity Control Structure: 35k	SECTION F

12	Row 12	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
					max:	48.6 tons/pile	max:	48.6 tons/pile

Assumed lateral Capacity: 35.0 kips/pile

	Horizontal Pile Capacity	Batter °/ft	N	Resistance due to Batter, kips	Resistance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	6	0.0	210	1.000	210 kips	
2	Row 2	0	6	0.0	210	1.000	210 kips	
3	Row 3	0	5	0.0	175	1.000	175 kips	
4	Row 4	0	6	0.0	210	1.000	210 kips	
5	Row 5	0	6	0.0	210	1.000	210 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			29		1015		1015 kips	OK

Case 2
Flood Event 500
Unusual

Vertical Load, P = 1195 kips
Horizontal Load, H = 384 kips
M_{HA} = 4926 kip-ft

	Vertical Pile Loading	P / N	+	M_{HA} * d / Σ I	= Pile Loads		Axial Pile Load
1	Row 1	41.2		31.3	72.5 kips/pile	36.2 tons/pile	36.2 tons/pile
2	Row 2	41.2		15.6	56.8 kips/pile	28.4 tons/pile	28.4 tons/pile
3	Row 3	41.2		0.0	41.2 kips/pile	20.6 tons/pile	20.6 tons/pile
4	Row 4	41.2		-15.6	25.6 kips/pile	12.8 tons/pile	12.8 tons/pile
5	Row 5	41.2		-31.3	9.9 kips/pile	5.0 tons/pile	5.0 tons/pile
6	Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
7	Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
8	Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
9	Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12	Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
					max:	36.2 tons/pile	max: 36.2 tons/pile

Assumed lateral Capacity: 35.0 kips/pile

	Horizontal Pile Capacity	Batter °/ft	N	Resistance due to Batter, kips	Resistance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	6	0.0	210	1.000	210 kips	
2	Row 2	0	6	0.0	210	1.000	210 kips	
3	Row 3	0	5	0.0	175	1.000	175 kips	
4	Row 4	0	6	0.0	210	1.000	210 kips	
5	Row 5	0	6	0.0	210	1.000	210 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			29		1015		1015 kips	OK

Case 2.1
Flood Event 500
Extreme

Vertical Load, P = 1153 kips
Horizontal Load, H = 693 kips
M_{HA} = 9680 kip-ft

	Vertical Pile Loading	P / N	+	M_{HA} * d / Σ I	= Pile Loads	Axial Pile Load

1484

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED PKN 2/4/11	CHECKED	SUBMITTED PKN	PROJECT NUMBER		
			SUBJECT	Pile Capacity Control Structure: 35k	SECTION F

1	Row 1	39.8	62.7	102.5 kips/pile		51.2 tons/pile		51.2 tons/pile
2	Row 2	39.8	31.4	71.1 kips/pile		35.6 tons/pile		35.6 tons/pile
3	Row 3	39.8	0.0	39.8 kips/pile		19.9 tons/pile		19.9 tons/pile
4	Row 4	39.8	-31.4	8.4 kips/pile		4.2 tons/pile		4.2 tons/pile
5	Row 5	39.8	-62.7	-23.0 kips/pile		-11.5 tons/pile		-11.5 tons/pile
6	Row 6	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
7	Row 7	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
8	Row 8	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
					max:	51.2 tons/pile	max:	51.2 tons/pile

Assumed lateral Capacity: 35.0 kips/pile

	Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resistance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	6	0.0	210	1.000	210 kips	
2	Row 2	0	6	0.0	210	1.000	210 kips	
3	Row 3	0	5	0.0	175	1.000	175 kips	
4	Row 4	0	6	0.0	210	1.000	210 kips	
5	Row 5	0	6	0.0	210	1.000	210 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			29		1015		1015 kips	OK

Case 3
Flood Event Dry
Unusual

Vertical Load, P = 1764 kips
Horizontal Load, H = 35 kips
M_{mt} = 623 kip-ft

	Vertical Pile Loading	P / N	+	M_{mt}* d / Σ I	= Pile Loads		Axial Pile Load
1	Row 1	60.8		4.0	64.8 kips/pile	32.4 tons/pile	32.4 tons/pile
2	Row 2	60.8		2.0	62.8 kips/pile	31.4 tons/pile	31.4 tons/pile
3	Row 3	60.8		0.0	60.8 kips/pile	30.4 tons/pile	30.4 tons/pile
4	Row 4	60.8		-2.0	58.8 kips/pile	29.4 tons/pile	29.4 tons/pile
5	Row 5	60.8		-4.0	56.9 kips/pile	28.4 tons/pile	28.4 tons/pile
6	Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
7	Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
8	Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
9	Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12	Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
					max:	32.4 tons/pile	max: 32.4 tons/pile

Assumed lateral Capacity: 35.0 kips/pile

	Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resistance due to Bending, kips	Group Efficiency	Lateral Resitance
1	Row 1	0	6	0.0	210	1.000	210 kips
2	Row 2	0	6	0.0	210	1.000	210 kips
3	Row 3	0	5	0.0	175	1.000	175 kips
4	Row 4	0	6	0.0	210	1.000	210 kips
5	Row 5	0	6	0.0	210	1.000	210 kips
6	Row 6	0	0	0.0	0	1.000	0 kips
7	Row 7	0	0	0.0	0	1.000	0 kips
8	Row 8	0	0	0.0	0	1.000	0 kips
9	Row 9	0	0	0.0	0	1.000	0 kips
10	Row 10	0	0	0.0	0	1.000	0 kips
11	Row 11	0	0	0.0	0	1.000	0 kips
12	Row 12	0	0	0.0	0	1.000	0 kips
13	Row 13	0	0	0.0	0	1.000	0 kips
14	Row 14	0	0	0.0	0	1.000	0 kips
15	Row 15	0	0	0.0	0	1.000	0 kips

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER		
PKN		PKN	SUBJECT	Pile Capacity	
2/4/11				Control Structure: 38k	SECTION F

	29	1015	1015 kips	OK	

Case 4
Flood Event Daily
Usual

Vertical Load, P = 1764 kips
Horizontal Load, H = 0 kips
M_{xx} = 0 kip-ft

Vertical Pile Loading	P / N	+	M_{xx}* d / Σ I	= Pile Loads		Axial Pile Load
1 Row 1	60.8		0.0	60.8 kips/pile	30.4 tons/pile	30.4 tons/pile
2 Row 2	60.8		0.0	60.8 kips/pile	30.4 tons/pile	30.4 tons/pile
3 Row 3	60.8		0.0	60.8 kips/pile	30.4 tons/pile	30.4 tons/pile
4 Row 4	60.8		0.0	60.8 kips/pile	30.4 tons/pile	30.4 tons/pile
5 Row 5	60.8		0.0	60.8 kips/pile	30.4 tons/pile	30.4 tons/pile
6 Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
7 Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
8 Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
9 Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10 Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11 Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12 Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13 Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14 Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15 Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
					max: 30.4 tons/pile	max: 30.4 tons/pile

Assumed lateral Capacity; 22.0 kips/pile

Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance
1 Row 1	0	6	0.0	132	1.000	132 kips
2 Row 2	0	6	0.0	132	1.000	132 kips
3 Row 3	0	5	0.0	110	1.000	110 kips
4 Row 4	0	6	0.0	132	1.000	132 kips
5 Row 5	0	6	0.0	132	1.000	132 kips
6 Row 6	0	0	0.0	0	1.000	0 kips
7 Row 7	0	0	0.0	0	1.000	0 kips
8 Row 8	0	0	0.0	0	1.000	0 kips
9 Row 9	0	0	0.0	0	1.000	0 kips
10 Row 10	0	0	0.0	0	1.000	0 kips
11 Row 11	0	0	0.0	0	1.000	0 kips
12 Row 12	0	0	0.0	0	1.000	0 kips
13 Row 13	0	0	0.0	0	1.000	0 kips
14 Row 14	0	0	0.0	0	1.000	0 kips
15 Row 15	0	0	0.0	0	1.000	0 kips
		29		838		838 kips

OK

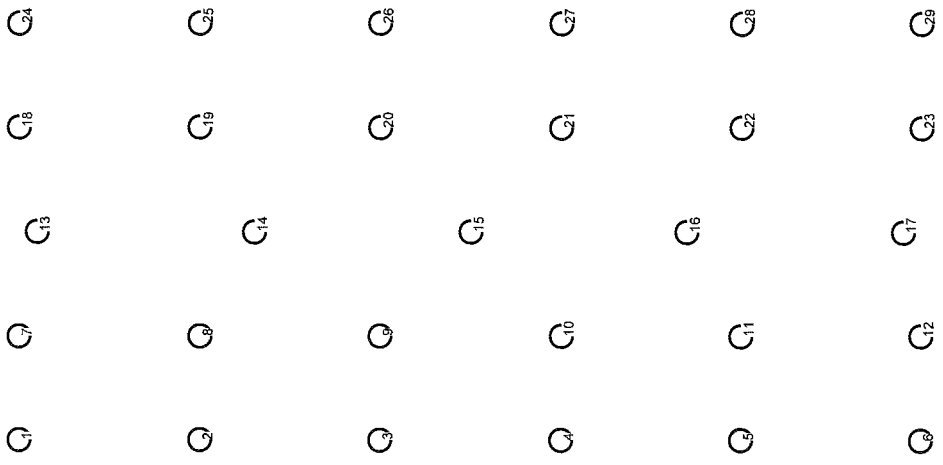
ATTACHMENT

F-Q1.5 Group Analyses Results for the Red River (LPP) Control Structure Wing Wall F

BARR ENGINEERING			DATE	2/4/2011	SHEET NO.
			PROJECT NAME	Flood Control ND Diversion Inlet - Corps Of Engineers	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER		
PKN		PKN	SUBJECT	Pile Coordinates	
2/4/11				Control Structure: 35k	SECTION F

Pile Coordinates

	x (in)	z (in)	Batter
1	24	30.00	
2	24	90.00	
3	24	150.00	
4	24	210.00	
5	24	270.00	
6	24	330.00	
7	87	30.00	
8	87	90.00	
9	87	150.00	
10	87	210.00	
11	87	270.00	
12	87	330.00	
13	150	36.00	
14	150	108.00	
15	150	180.00	
16	150	252.00	
17	150	324.00	
18	213	30.00	
19	213	90.00	
20	213	150.00	
21	213	210.00	
22	213	270.00	
23	213	330.00	
24	276	30.00	
25	276	90.00	
26	276	150.00	
27	276	210.00	
28	276	270.00	
29	276	330.00	
0	0	0.00	
0	0	0.00	
0	0	0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RRN_WWF90.gp8t

Time and Date of Analysis

Date: February 16, 2011 Time: 11:52:01

***** COMPUTATION RESULTS *****

RRN Wingwall F

***** LOAD CASES RESULTS *****

LOAD CASE : 1
CASE NAME : Load Case 1
LOAD TYPE : Dead, DL

REDUCTION FACTORS FOR CLOSELY-SPACED PILE GROUPS, COMBINED Y AND Z DIRECTIONS
ESTIMATED USING MOVEMENT IN THE DIRECTION OF PILE CAP DISPLACEMENTS

GROUP NO	P-FACTOR	Y-FACTOR
1	1.0000	1.0000
2	1.0000	1.0000
3	1.0000	1.0000
4	1.0000	1.0000
5	1.0000	1.0000
6	1.0000	1.0000
7	0.8387	1.0000
8	0.8237	1.0000
9	0.8237	1.0000
10	0.8237	1.0000
11	0.8237	1.0000
12	0.8387	1.0000
13	0.8316	1.0000
14	0.8272	1.0000
15	0.8274	1.0000
16	0.8272	1.0000
17	0.8316	1.0000
18	0.8569	1.0000
19	0.8517	1.0000
20	0.8574	1.0000
21	0.8574	1.0000
22	0.8517	1.0000
23	0.8569	1.0000
24	0.8387	1.0000
25	0.8237	1.0000
26	0.8237	1.0000
27	0.8237	1.0000
28	0.8237	1.0000
29	0.8387	1.0000

* TABLE L * COMPUTATION ON PILE CAP

* EQUIVALENT CONC. LOAD AT ORIGIN *

1297 K 755 K LC 1 100 YR

VERT. LOAD, LBS	HOR. LOAD Y, LBS	HOR. LOAD Z, LBS
1.29701E+06 ✓	-7.55105E+05	0.00000

MOMENT X ,IN-LBS	MOMENT Y,IN-LBS	MOMENT Z,IN-LBS
1.35919E+08	2.33462E+08 ✓	-1.01271E+08 ✓

* DISPLACEMENT OF GROUPED PILE FOUNDATION AT ORIGIN *

```
RRN Wingwall F
GROUP8.0.4
* Analysis Parameters
1 3 0 0 0 0 0 1 0 1 0 0
* Pile Cap Dimensions
0.0000000E+00 0.0000000E+00 0.0000000E+00
* Load Cases
1 0 0 1
0 1 0 1 1 100 100
0.0001 0.0001 1 1
Load Case  1
0 0 0 0 1 0 0
2 1 1 0
1
1297010 -755105 0 0 0 0 0 78.08 180
0
0
* Load Combinations
0 0 0 1
* Distributed Load Sets
0
* Pile Group Configuration
29
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 24 30 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 24 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 24 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 24 210 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 24 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 24 330 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 87 30 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 87 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 87 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 87 210 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 87 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 87 330 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 36 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 108 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 180 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 252 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 150 324 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 213 30 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
```

```
-12 213 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 213 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 213 210 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 213 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 213 330 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 276 30 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 276 90 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 276 150 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 276 210 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 276 270 0 90 -36
2 2 1 0 0 0
0 0 1 1 1 1 1
-12 276 330 0 90 -36
* Pile Properties
1
0 1
100 900 29000000 1
0 900 1 0 0
* Pile Cross Sections
1
0 0 4
HP14x89
13.83 0 14.585 13.83 0 0 0.615 0.615
1 26.1 904 326 1808 2.09728E10 0 0 0 0
* Soil Layers
3
0 1
0 590
2 10
0.0235 4.514 0 0 0 0.01 0 0 0 0
0.0235 4.514 0 0 0 0.01 0 0 0 0
1 3
590 650
2 10
0.03449 13.194 0 0 0 0.007 0 0 0 0
0.03449 13.194 0 0 0 0.007 0 0 0 0
2 3
650 1200
2 10
0.03449 13.194 0 0 0 0.007 0 0 0 0
0.03449 13.194 0 0 0 0.007 0 0 0 0
* End of file
```

RRN_WWF90.gp8t

VERTICAL ,IN	HORIZONTAL Y,IN	HORIZONTAL Z,IN
0.13173	-0.42851	-3.21107E-11

ANGLE ROT. X,RAD	ANGLE ROT. Y,RAD	ANGLE ROT. Z,RAD
2.17671E-13	2.19705E-17	5.73412E-04

LOCAL AXIS FOR VERT. PILES

EQUAL .4354" ∠ 0.67"

* TABLE M * COMPUTATION ON INOIVIDUAL PILE

THE GLOBAL STRUCTURAL COORDINATE SYSTEM

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. X,IN	DISP. Y,IN	DISP. Z,IN	ROT. X,RAD	ROT. Y,RAD	ROT. Z,RAD
1	0.1180	-0.4354	-2.6886E-11	2.1767E-13	2.1970E-17	5.7341E-04
2	0.1180	-0.4354	-2.6886E-11	2.1767E-13	2.1970E-17	5.7341E-04
3	0.1180	-0.4354	-2.6886E-11	2.1767E-13	2.1970E-17	5.7341E-04
4	0.1180	-0.4354	-2.6886E-11	2.1767E-13	2.1970E-17	5.7341E-04
5	0.1180	-0.4354	-2.6886E-11	2.1767E-13	2.1970E-17	5.7341E-04
6	0.1180	-0.4354	-2.6886E-11	2.1767E-13	2.1970E-17	5.7341E-04
7	8.1843E-02	-0.4354	-1.3173E-11	2.1767E-13	2.1970E-17	5.7341E-04
8	8.1843E-02	-0.4354	-1.3173E-11	2.1767E-13	2.1970E-17	5.7341E-04
9	8.1843E-02	-0.4354	-1.3173E-11	2.1767E-13	2.1970E-17	5.7341E-04
10	8.1843E-02	-0.4354	-1.3173E-11	2.1767E-13	2.1970E-17	5.7341E-04
11	8.1843E-02	-0.4354	-1.3173E-11	2.1767E-13	2.1970E-17	5.7341E-04
12	8.1843E-02	-0.4354	-1.3173E-11	2.1767E-13	2.1970E-17	5.7341E-04
13	4.5718E-02	-0.4354	5.4017E-13	2.1767E-13	2.1970E-17	5.7341E-04
14	4.5718E-02	-0.4354	5.4017E-13	2.1767E-13	2.1970E-17	5.7341E-04
15	4.5718E-02	-0.4354	5.4017E-13	2.1767E-13	2.1970E-17	5.7341E-04
16	4.5718E-02	-0.4354	5.4017E-13	2.1767E-13	2.1970E-17	5.7341E-04
17	4.5718E-02	-0.4354	5.4017E-13	2.1767E-13	2.1970E-17	5.7341E-04
18	9.5931E-03	-0.4354	1.4253E-11	2.1767E-13	2.1970E-17	5.7341E-04
19	9.5931E-03	-0.4354	1.4253E-11	2.1767E-13	2.1970E-17	5.7341E-04
20	9.5931E-03	-0.4354	1.4253E-11	2.1767E-13	2.1970E-17	5.7341E-04
21	9.5931E-03	-0.4354	1.4253E-11	2.1767E-13	2.1970E-17	5.7341E-04
22	9.5931E-03	-0.4354	1.4253E-11	2.1767E-13	2.1970E-17	5.7341E-04
23	9.5931E-03	-0.4354	1.4253E-11	2.1767E-13	2.1970E-17	5.7341E-04
24	-2.6532E-02	-0.4354	2.7967E-11	2.1767E-13	2.1970E-17	5.7341E-04
25	-2.6532E-02	-0.4354	2.7967E-11	2.1767E-13	2.1970E-17	5.7341E-04
26	-2.6532E-02	-0.4354	2.7967E-11	2.1767E-13	2.1970E-17	5.7341E-04
27	-2.6532E-02	-0.4354	2.7967E-11	2.1767E-13	2.1970E-17	5.7341E-04
28	-2.6532E-02	-0.4354	2.7967E-11	2.1767E-13	2.1970E-17	5.7341E-04
29	-2.6532E-02	-0.4354	2.7967E-11	2.1767E-13	2.1970E-17	5.7341E-04
MINIMUM	-2.6532E-02	-0.4354	-2.6886E-11	2.1767E-13	2.1970E-17	5.7341E-04
Pile N.	24	1	1	1	1	1
MAXIMUM	0.1180	-0.4354	2.7967E-11	2.1767E-13	2.1970E-17	5.7341E-04
Pile N.	1	1	24	1	1	1

* PILE TOP REACTIONS *

AXIAL HORIZ. 28.61 K 152.60 K-IN

PILE GROUP	FOR. X,LBS	FOR. Y,LBS	FOR. Z,LBS	MOM X,LBS-IN	MOM Y,LBS-IN	MOM Z,LBS-IN	STRESS,LBS/IN**2
1	1.1124E+05	-2.8611E+04	-1.2801E-06	3.5978E-12	7.0633E-05	-1.8312E+06	1.8270E+04
2	1.1124E+05	-2.8611E+04	-1.2801E-06	3.5978E-12	7.0633E-05	-1.8312E+06	1.8270E+04
3	1.1124E+05	-2.8611E+04	-1.2801E-06	3.5978E-12	7.0633E-05	-1.8312E+06	1.8270E+04
4	1.1124E+05	-2.8611E+04	-1.2801E-06	3.5978E-12	7.0633E-05	-1.8312E+06	1.8270E+04
5	1.1124E+05	-2.8611E+04	-1.2801E-06	3.5978E-12	7.0633E-05	-1.8312E+06	1.8270E+04
6	1.1124E+05	-2.8611E+04	-1.2801E-06	3.5978E-12	7.0633E-05	-1.8312E+06	1.8270E+04
7	8.1402E+04	-2.5323E+04	-5.5921E-07	3.5978E-12	3.1944E-05	-1.6757E+06	1.5937E+04
8	8.1402E+04	-2.4997E+04	-5.5278E-07	3.5978E-12	3.1683E-05	-1.6602E+06	1.5818E+04
9	8.1402E+04	-2.4997E+04	-5.5278E-07	3.5978E-12	3.1683E-05	-1.6602E+06	1.5818E+04
10	8.1402E+04	-2.4997E+04	-5.5278E-07	3.5978E-12	3.1683E-05	-1.6602E+06	1.5818E+04
11	8.1402E+04	-2.4997E+04	-5.5278E-07	3.5978E-12	3.1683E-05	-1.6602E+06	1.5818E+04
12	8.1402E+04	-2.5323E+04	-5.5921E-07	3.5978E-12	3.1944E-05	-1.6757E+06	1.5937E+04
13	5.1568E+04	-2.5228E+04	2.2853E-08	3.5978E-12	-1.3000E-06	-1.6677E+06	1.4732E+04
14	5.1568E+04	-2.5134E+04	2.2768E-08	3.5978E-12	-1.2968E-06	-1.6634E+06	1.4700E+04
15	5.1568E+04	-2.5139E+04	2.2773E-08	3.5978E-12	-1.2970E-06	-1.6637E+06	1.4702E+04
16	5.1568E+04	-2.5134E+04	2.2768E-08	3.5978E-12	-1.2968E-06	-1.6634E+06	1.4700E+04
17	5.1568E+04	-2.5228E+04	2.2853E-08	3.5978E-12	-1.3000E-06	-1.6677E+06	1.4732E+04
18	1.0840E+04	-2.5854E+04	6.1971E-07	3.5978E-12	-3.4909E-05	-1.6930E+06	1.3366E+04
19	1.0840E+04	-2.5745E+04	6.1724E-07	3.5978E-12	-3.4813E-05	-1.6879E+06	1.3327E+04
20	1.0840E+04	-2.5863E+04	6.1992E-07	3.5978E-12	-3.4917E-05	-1.6934E+06	1.3369E+04
21	1.0840E+04	-2.5863E+04	6.1992E-07	3.5978E-12	-3.4917E-05	-1.6934E+06	1.3369E+04
22	1.0840E+04	-2.5745E+04	6.1724E-07	3.5978E-12	-3.4813E-05	-1.6879E+06	1.3327E+04

TONS 55.62 40.7 25.78 5.42

	AXIAL Row 1	LOADS TONS Row 2	Row 3	Row 4	Row 5
RIGID CAP	47.0	34.7	22.4	10.0	-2.3
GROUP	55.62	40.7	25.78	5.42	-15.14

CASE 1.1

23	1.0840E+04	-2.5854E+04	6.1971E-07	3.5978E-12	-3.4909E-05	-1.6930E+06	1.3366E+04
24	-3.0283E+04	-2.5550E+04	1.2060E-06	3.5978E-12	-6.7856E-05	-1.6743E+06	1.3968E+04
25	-3.0283E+04	-2.5230E+04	1.1922E-06	3.5978E-12	-6.7306E-05	-1.6595E+06	1.3855E+04
26	-3.0283E+04	-2.5230E+04	1.1922E-06	3.5978E-12	-6.7306E-05	-1.6595E+06	1.3855E+04
27	-3.0283E+04	-2.5230E+04	1.1922E-06	3.5978E-12	-6.7306E-05	-1.6595E+06	1.3855E+04
28	-3.0283E+04	-2.5230E+04	1.1922E-06	3.5978E-12	-6.7306E-05	-1.6595E+06	1.3855E+04
29	-3.0283E+04	-2.5550E+04	1.2060E-06	3.5978E-12	-6.7856E-05	-1.6743E+06	1.3968E+04
MINIMUM	-3.0283E+04	-2.8611E+04	-1.2801E-06	3.5978E-12	-6.7856E-05	-1.8312E+06	1.3327E+04
Pile N.	24	1	1	1	24	1	19
MAXIMUM	1.1124E+05	-2.4997E+04	1.2060E-06	3.5978E-12	7.0633E-05	-1.6595E+06	1.8270E+04
Pile N.	1	8	24	1	1	25	1

THE PILE COORDINATE SYSTEM (LOCAL AXES)

* PILE TOP DISPLACEMENTS *

PILE GROUP	DISP. x,IN	DISP. y,IN	DISP. z,IN	ROT. x,RAD	ROT. y,RAD	ROT. z,RAD
1	0.1180	-0.4354	-2.6886E-11	2.1767E-13	2.1970E-17	5.7341E-04
2	0.1180	-0.4354	-2.6886E-11	2.1767E-13	2.1970E-17	5.7341E-04
3	0.1180	-0.4354	-2.6886E-11	2.1767E-13	2.1970E-17	5.7341E-04
4	0.1180	-0.4354	-2.6886E-11	2.1767E-13	2.1970E-17	5.7341E-04
5	0.1180	-0.4354	-2.6886E-11	2.1767E-13	2.1970E-17	5.7341E-04
6	0.1180	-0.4354	-2.6886E-11	2.1767E-13	2.1970E-17	5.7341E-04
7	8.1843E-02	-0.4354	-1.3173E-11	2.1767E-13	2.1970E-17	5.7341E-04
8	8.1843E-02	-0.4354	-1.3173E-11	2.1767E-13	2.1970E-17	5.7341E-04
9	8.1843E-02	-0.4354	-1.3173E-11	2.1767E-13	2.1970E-17	5.7341E-04
10	8.1843E-02	-0.4354	-1.3173E-11	2.1767E-13	2.1970E-17	5.7341E-04
11	8.1843E-02	-0.4354	-1.3173E-11	2.1767E-13	2.1970E-17	5.7341E-04
12	8.1843E-02	-0.4354	-1.3173E-11	2.1767E-13	2.1970E-17	5.7341E-04
13	4.5718E-02	-0.4354	5.4017E-13	2.1767E-13	2.1970E-17	5.7341E-04
14	4.5718E-02	-0.4354	5.4017E-13	2.1767E-13	2.1970E-17	5.7341E-04
15	4.5718E-02	-0.4354	5.4017E-13	2.1767E-13	2.1970E-17	5.7341E-04
16	4.5718E-02	-0.4354	5.4017E-13	2.1767E-13	2.1970E-17	5.7341E-04
17	4.5718E-02	-0.4354	5.4017E-13	2.1767E-13	2.1970E-17	5.7341E-04
18	9.5931E-03	-0.4354	1.4253E-11	2.1767E-13	2.1970E-17	5.7341E-04
19	9.5931E-03	-0.4354	1.4253E-11	2.1767E-13	2.1970E-17	5.7341E-04
20	9.5931E-03	-0.4354	1.4253E-11	2.1767E-13	2.1970E-17	5.7341E-04
21	9.5931E-03	-0.4354	1.4253E-11	2.1767E-13	2.1970E-17	5.7341E-04
22	9.5931E-03	-0.4354	1.4253E-11	2.1767E-13	2.1970E-17	5.7341E-04
23	9.5931E-03	-0.4354	1.4253E-11	2.1767E-13	2.1970E-17	5.7341E-04
24	-2.6532E-02	-0.4354	2.7967E-11	2.1767E-13	2.1970E-17	5.7341E-04
25	-2.6532E-02	-0.4354	2.7967E-11	2.1767E-13	2.1970E-17	5.7341E-04
26	-2.6532E-02	-0.4354	2.7967E-11	2.1767E-13	2.1970E-17	5.7341E-04
27	-2.6532E-02	-0.4354	2.7967E-11	2.1767E-13	2.1970E-17	5.7341E-04
28	-2.6532E-02	-0.4354	2.7967E-11	2.1767E-13	2.1970E-17	5.7341E-04
29	-2.6532E-02	-0.4354	2.7967E-11	2.1767E-13	2.1970E-17	5.7341E-04
MINIMUM	-2.6532E-02	-0.4354	-2.6886E-11	2.1767E-13	2.1970E-17	5.7341E-04
Pile N.	24	1	1	1	1	1
MAXIMUM	0.1180	-0.4354	2.7967E-11	2.1767E-13	2.1970E-17	5.7341E-04
Pile N.	1	1	24	1	1	1

* PILE TOP REACTIONS *

PILE GROUP	AXIAL,LBS	LAT. y,LBS	LAT. z,LBS	MOM x,LBS-IN	MOM y,LBS-IN	MOM z,LBS-IN	STRESS,LBS/IN**2
1	1.1124E+05	-2.8611E+04	-1.2801E-06	3.5978E-12	7.0633E-05	-1.8312E+06	1.8270E+04
2	1.1124E+05	-2.8611E+04	-1.2801E-06	3.5978E-12	7.0633E-05	-1.8312E+06	1.8270E+04
3	1.1124E+05	-2.8611E+04	-1.2801E-06	3.5978E-12	7.0633E-05	-1.8312E+06	1.8270E+04
4	1.1124E+05	-2.8611E+04	-1.2801E-06	3.5978E-12	7.0633E-05	-1.8312E+06	1.8270E+04
5	1.1124E+05	-2.8611E+04	-1.2801E-06	3.5978E-12	7.0633E-05	-1.8312E+06	1.8270E+04
6	1.1124E+05	-2.8611E+04	-1.2801E-06	3.5978E-12	7.0633E-05	-1.8312E+06	1.8270E+04
7	8.1402E+04	-2.5323E+04	-5.5921E-07	3.5978E-12	3.1944E-05	-1.6757E+06	1.5937E+04
8	8.1402E+04	-2.4997E+04	-5.5278E-07	3.5978E-12	3.1683E-05	-1.6602E+06	1.5818E+04
9	8.1402E+04	-2.4997E+04	-5.5278E-07	3.5978E-12	3.1683E-05	-1.6602E+06	1.5818E+04
10	8.1402E+04	-2.4997E+04	-5.5278E-07	3.5978E-12	3.1683E-05	-1.6602E+06	1.5818E+04
11	8.1402E+04	-2.4997E+04	-5.5278E-07	3.5978E-12	3.1683E-05	-1.6602E+06	1.5818E+04
12	8.1402E+04	-2.5323E+04	-5.5921E-07	3.5978E-12	3.1944E-05	-1.6757E+06	1.5937E+04
13	5.1568E+04	-2.5228E+04	2.2853E-08	3.5978E-12	-1.3000E-06	-1.6677E+06	1.4732E+04
14	5.1568E+04	-2.5134E+04	2.2768E-08	3.5978E-12	-1.2968E-06	-1.6634E+06	1.4700E+04
15	5.1568E+04	-2.5139E+04	2.2773E-08	3.5978E-12	-1.2970E-06	-1.6637E+06	1.4702E+04
16	5.1568E+04	-2.5134E+04	2.2768E-08	3.5978E-12	-1.2968E-06	-1.6634E+06	1.4700E+04
17	5.1568E+04	-2.5228E+04	2.2853E-08	3.5978E-12	-1.3000E-06	-1.6677E+06	1.4732E+04
18	1.0840E+04	-2.5854E+04	6.1971E-07	3.5978E-12	-3.4909E-05	-1.6930E+06	1.3366E+04
19	1.0840E+04	-2.5745E+04	6.1724E-07	3.5978E-12	-3.4813E-05	-1.6879E+06	1.3327E+04

20	1.0840E+04	-2.5863E+04	6.1992E-07	3.5978E-12	-3.4917E-05	-1.6934E+06	1.3369E+04
21	1.0840E+04	-2.5863E+04	6.1992E-07	3.5978E-12	-3.4917E-05	-1.6934E+06	1.3369E+04
22	1.0840E+04	-2.5745E+04	6.1724E-07	3.5978E-12	-3.4813E-05	-1.6879E+06	1.3327E+04
23	1.0840E+04	-2.5854E+04	6.1971E-07	3.5978E-12	-3.4909E-05	-1.6930E+06	1.3366E+04
24	-3.0283E+04	-2.5550E+04	1.2060E-06	3.5978E-12	-6.7856E-05	-1.6743E+06	1.3968E+04
25	-3.0283E+04	-2.5230E+04	1.1922E-06	3.5978E-12	-6.7306E-05	-1.6595E+06	1.3855E+04
26	-3.0283E+04	-2.5230E+04	1.1922E-06	3.5978E-12	-6.7306E-05	-1.6595E+06	1.3855E+04
27	-3.0283E+04	-2.5230E+04	1.1922E-06	3.5978E-12	-6.7306E-05	-1.6595E+06	1.3855E+04
28	-3.0283E+04	-2.5230E+04	1.1922E-06	3.5978E-12	-6.7306E-05	-1.6595E+06	1.3855E+04
29	-3.0283E+04	-2.5550E+04	1.2060E-06	3.5978E-12	-6.7856E-05	-1.6743E+06	1.3968E+04
MINIMUM	-3.0283E+04	-2.8611E+04	-1.2801E-06	3.5978E-12	-6.7856E-05	-1.8312E+06	1.3327E+04
Pile N.	24	1	1	1	24	1	19
MAXIMUM	1.1124E+05	-2.4997E+04	1.2060E-06	3.5978E-12	7.0633E-05	-1.6595E+06	1.8270E+04
Pile N.	1	8	24	1	1	25	1

* EFFECTS FOR LATERALLY LOADED PILE *

* MINIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION		BENDING MOMENT		SHEAR FORCE		SOIL REACTION	
TOTAL FLEXURAL RIGIDITY	y-DIR	z-DIR	z-DIR	y-DIR	y-DIR	z-DIR	y-DIR	z-DIR
STRESS z-DIR y-DIR	IN	IN	LBS-IN	LBS-IN	LBS	LBS	LBS/IN	LBS/IN
LBS/IN**2 LBS-IN**2 LBS-IN**2	*****	**********	**********	**********	**********	**********	**********	**********
********** ********** **********								
1	-0.4350	-2.6900E-11	-6.3000E+05	-2.0300E-05	-2.8600E+04	-1.2800E-06	-222.00	-1.2100E-08
4260.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	0.0000	153.00	135.00	0.0000	0.0000	99.000	54.000
531.00 0.0000 0.0000								
2	-0.4350	-2.6900E-11	-6.3000E+05	-2.0300E-05	-2.8600E+04	-1.2800E-06	-222.00	-1.2100E-08
4260.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	0.0000	153.00	135.00	0.0000	0.0000	99.000	54.000
522.00 0.0000 0.0000								
3	-0.4350	-2.6900E-11	-6.3000E+05	-2.0300E-05	-2.8600E+04	-1.2800E-06	-222.00	-1.2100E-08
4260.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	0.0000	153.00	135.00	0.0000	0.0000	99.000	54.000
522.00 0.0000 0.0000								
4	-0.4350	-2.6900E-11	-6.3000E+05	-2.0300E-05	-2.8600E+04	-1.2800E-06	-222.00	-1.2100E-08
4260.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	0.0000	153.00	135.00	0.0000	0.0000	99.000	54.000
531.00 0.0000 0.0000								
5	-0.4350	-2.6900E-11	-6.3000E+05	-2.0300E-05	-2.8600E+04	-1.2800E-06	-222.00	-1.2100E-08
4260.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	0.0000	153.00	135.00	0.0000	0.0000	99.000	54.000
522.00 0.0000 0.0000								
6	-0.4350	-2.6900E-11	-6.3000E+05	-2.0300E-05	-2.8600E+04	-1.2800E-06	-222.00	-1.2100E-08
4260.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	0.0000	153.00	135.00	0.0000	0.0000	99.000	54.000
531.00 0.0000 0.0000								
7	-0.4350	-1.3200E-11	-5.7800E+05	-9.1600E-06	-2.5300E+04	-5.6000E-07	-190.00	-5.0700E-09
3120.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	0.0000	162.00	135.00	0.0000	0.0000	108.00	63.000
549.00 0.0000 0.0000								
8	-0.4350	-1.3200E-11	-5.7300E+05	-9.1200E-06	-2.5000E+04	-5.5400E-07	-187.00	-4.9900E-09
3120.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	0.0000	162.00	135.00	0.0000	0.0000	108.00	63.000
558.00 0.0000 0.0000								
9	-0.4350	-1.3200E-11	-5.7300E+05	-9.1200E-06	-2.5000E+04	-5.5400E-07	-187.00	-4.9900E-09
3120.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	0.0000	162.00	135.00	0.0000	0.0000	108.00	63.000
558.00 0.0000 0.0000								
10	-0.4350	-1.3200E-11	-5.7300E+05	-9.1200E-06	-2.5000E+04	-5.5400E-07	-187.00	-4.9900E-09
3120.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	0.0000	162.00	135.00	0.0000	0.0000	108.00	63.000
558.00 0.0000 0.0000								
11	-0.4350	-1.3200E-11	-5.7300E+05	-9.1200E-06	-2.5000E+04	-5.5400E-07	-187.00	-4.9900E-09
3120.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	0.0000	162.00	135.00	0.0000	0.0000	108.00	63.000
558.00 0.0000 0.0000								
12	-0.4350	-1.3200E-11	-5.7800E+05	-9.1600E-06	-2.5300E+04	-5.6000E-07	-190.00	-5.0700E-09
3120.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	0.0000	162.00	135.00	0.0000	0.0000	108.00	63.000
549.00 0.0000 0.0000								
13	-0.4350	1.0000E-14	-5.7400E+05	-1.3000E-06	-2.5200E+04	-4.7400E-09	-189.00	-3.7200E-10
1980.0 2.6200E+10 9.4500E+09								
x(IN)	0.0000	198.00	162.00	135.00	0.0000	216.00	108.00	234.00

549.00 0.0000 0.0000
14 -0.4350 1.0000E-14 -5.7300E+05 -1.3000E-06 -2.5100E+04 -4.6700E-09 -188.00 -3.4000E-10
1980.0 2.6200E+10 9.4500E+09
x(IN) 0.0000 198.00 162.00 135.00 0.0000 216.00 108.00 234.00
558.00 0.0000 0.0000
15 -0.4350 1.0000E-14 -5.7300E+05 -1.3000E-06 -2.5200E+04 -4.6700E-09 -188.00 -3.3900E-10
1980.0 2.6200E+10 9.4500E+09
x(IN) 0.0000 198.00 162.00 135.00 0.0000 216.00 108.00 234.00
558.00 0.0000 0.0000
16 -0.4350 1.0000E-14 -5.7300E+05 -1.3000E-06 -2.5100E+04 -4.6700E-09 -188.00 -3.4000E-10
1980.0 2.6200E+10 9.4500E+09
x(IN) 0.0000 198.00 162.00 135.00 0.0000 216.00 108.00 234.00
558.00 0.0000 0.0000
17 -0.4350 1.0000E-14 -5.7400E+05 -1.3000E-06 -2.5200E+04 -4.7400E-09 -189.00 -3.7200E-10
1980.0 2.6200E+10 9.4500E+09
x(IN) 0.0000 198.00 162.00 135.00 0.0000 216.00 108.00 234.00
549.00 0.0000 0.0000
18 -0.4350 -9.8700E-14 -5.8200E+05 -3.4900E-05 -2.5900E+04 -1.1800E-07 -194.00 -5.0600E-09
415.00 2.6200E+10 9.4500E+09
x(IN) 0.0000 234.00 162.00 135.00 0.0000 207.00 108.00 234.00
558.00 0.0000 0.0000
19 -0.4350 -9.4600E-14 -5.8000E+05 -3.4800E-05 -2.5700E+04 -1.1900E-07 -193.00 -5.4200E-09
415.00 2.6200E+10 9.4500E+09
x(IN) 0.0000 234.00 162.00 135.00 0.0000 207.00 108.00 234.00
549.00 0.0000 0.0000
20 -0.4350 -9.9100E-14 -5.8200E+05 -3.4900E-05 -2.5900E+04 -1.1800E-07 -194.00 -5.0200E-09
415.00 2.6200E+10 9.4500E+09
x(IN) 0.0000 234.00 162.00 135.00 0.0000 207.00 108.00 234.00
558.00 0.0000 0.0000
21 -0.4350 -9.9100E-14 -5.8200E+05 -3.4900E-05 -2.5900E+04 -1.1800E-07 -194.00 -5.0200E-09
415.00 2.6200E+10 9.4500E+09
x(IN) 0.0000 234.00 162.00 135.00 0.0000 207.00 108.00 234.00
558.00 0.0000 0.0000
22 -0.4350 -9.4600E-14 -5.8000E+05 -3.4800E-05 -2.5700E+04 -1.1900E-07 -193.00 -5.4200E-09
415.00 2.6200E+10 9.4500E+09
x(IN) 0.0000 234.00 162.00 135.00 0.0000 207.00 108.00 234.00
549.00 0.0000 0.0000
23 -0.4350 -9.8700E-14 -5.8200E+05 -3.4900E-05 -2.5900E+04 -1.1800E-07 -194.00 -5.0600E-09
415.00 2.6200E+10 9.4500E+09
x(IN) 0.0000 234.00 162.00 135.00 0.0000 207.00 108.00 234.00
558.00 0.0000 0.0000
24 -0.4350 -1.2300E-13 -5.7400E+05 -6.7900E-05 -2.5600E+04 -2.3700E-07 -190.00 -1.6200E-08
1160.0 2.6200E+10 9.4500E+09
x(IN) 0.0000 243.00 162.00 135.00 9.0000 216.00 108.00 234.00
558.00 0.0000 0.0000
25 -0.4350 -5.7000E-14 -5.6900E+05 -6.7300E-05 -2.5200E+04 -2.5200E-07 -187.00 -2.3200E-08
1160.0 2.6200E+10 9.4500E+09
x(IN) 0.0000 261.00 162.00 135.00 9.0000 225.00 108.00 234.00
558.00 0.0000 0.0000
26 -0.4350 -5.7000E-14 -5.6900E+05 -6.7300E-05 -2.5200E+04 -2.5200E-07 -187.00 -2.3200E-08
1160.0 2.6200E+10 9.4500E+09
x(IN) 0.0000 261.00 162.00 135.00 9.0000 225.00 108.00 234.00
558.00 0.0000 0.0000
27 -0.4350 -5.7000E-14 -5.6900E+05 -6.7300E-05 -2.5200E+04 -2.5200E-07 -187.00 -2.3200E-08
1160.0 2.6200E+10 9.4500E+09
x(IN) 0.0000 261.00 162.00 135.00 9.0000 225.00 108.00 234.00
558.00 0.0000 0.0000
28 -0.4350 -5.7000E-14 -5.6900E+05 -6.7300E-05 -2.5200E+04 -2.5200E-07 -187.00 -2.3200E-08
1160.0 2.6200E+10 9.4500E+09
x(IN) 0.0000 261.00 162.00 135.00 9.0000 225.00 108.00 234.00
558.00 0.0000 0.0000
29 -0.4350 -1.2300E-13 -5.7400E+05 -6.7900E-05 -2.5600E+04 -2.3700E-07 -190.00 -1.6200E-08
1160.0 2.6200E+10 9.4500E+09
x(IN) 0.0000 243.00 162.00 135.00 9.0000 216.00 108.00 234.00
558.00 0.0000 0.0000

Min. -0.4350 -2.6900E-11 -6.3000E+05 -6.7900E-05 -2.8600E+04 -1.2800E-06 -222.00 -2.3200E-08
415.00 2.6200E+10 9.4500E+09
Pile N. 1 1 1 24 1 1 1 25
18 1 1

M3

* MAXIMUM VALUES AND LOCATIONS *

PILE	DEFLECTION		BENDING MOMENT		SHEAR FORCE		SOIL REACTION	
TOTAL FLEXURAL RIGIDITY	y-DIR	z-DIR	z-DIR	y-DIR	y-DIR	z-DIR	y-DIR	z-DIR
STRESS z-DIR	y-DIR							
	IN	IN	LBS-IN	LBS-IN	LBS	LBS	LBS/IN	LBS/IN
LBS/IN**2 LBS-IN**2	LBS-IN**2							

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*****	**********	**********	**********	**********	**********	**********	**********	**********
**********	**********	**********						
1	-0.4350	2.0300E-13	1.8300E+06	7.0600E-05	6860.0	2.5500E-07	71.300	1.0200E-08
1.8300E+04	2.6200E+10	9.4500E+09						
x(IN)	261.00	225.00	0.0000	0.0000	225.00	198.00	261.00	225.00
0.0000	0.0000	0.0000						
2	-0.4350	2.0300E-13	1.8300E+06	7.0600E-05	6860.0	2.5500E-07	71.300	1.0200E-08
1.8300E+04	2.6200E+10	9.4500E+09						
x(IN)	261.00	225.00	0.0000	0.0000	225.00	198.00	261.00	225.00
0.0000	0.0000	0.0000						
3	-0.4350	2.0300E-13	1.8300E+06	7.0600E-05	6860.0	2.5500E-07	71.300	1.0200E-08
1.8300E+04	2.6200E+10	9.4500E+09						
x(IN)	261.00	225.00	0.0000	0.0000	225.00	198.00	261.00	225.00
0.0000	0.0000	0.0000						
4	-0.4350	2.0300E-13	1.8300E+06	7.0600E-05	6860.0	2.5500E-07	71.300	1.0200E-08
1.8300E+04	2.6200E+10	9.4500E+09						
x(IN)	261.00	225.00	0.0000	0.0000	225.00	198.00	261.00	225.00
0.0000	0.0000	0.0000						
5	-0.4350	2.0300E-13	1.8300E+06	7.0600E-05	6860.0	2.5500E-07	71.300	1.0200E-08
1.8300E+04	2.6200E+10	9.4500E+09						
x(IN)	261.00	225.00	0.0000	0.0000	225.00	198.00	261.00	225.00
0.0000	0.0000	0.0000						
6	-0.4350	2.0300E-13	1.8300E+06	7.0600E-05	6860.0	2.5500E-07	71.300	1.0200E-08
1.8300E+04	2.6200E+10	9.4500E+09						
x(IN)	261.00	225.00	0.0000	0.0000	225.00	198.00	261.00	225.00
0.0000	0.0000	0.0000						
7	-0.4350	8.7400E-14	1.6800E+06	3.1900E-05	6020.0	1.1100E-07	59.700	5.3000E-09
1.5900E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	234.00	0.0000	0.0000	234.00	207.00	270.00	234.00
0.0000	0.0000	0.0000						
8	-0.4350	4.3500E-14	1.6600E+06	3.1700E-05	5940.0	1.1700E-07	58.400	9.4100E-09
1.5800E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	234.00	216.00	279.00	234.00
0.0000	0.0000	0.0000						
9	-0.4350	4.3500E-14	1.6600E+06	3.1700E-05	5940.0	1.1700E-07	58.400	9.4100E-09
1.5800E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	234.00	216.00	279.00	234.00
0.0000	0.0000	0.0000						
10	-0.4350	4.3500E-14	1.6600E+06	3.1700E-05	5940.0	1.1700E-07	58.400	9.4100E-09
1.5800E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	234.00	216.00	279.00	234.00
0.0000	0.0000	0.0000						
11	-0.4350	4.3500E-14	1.6600E+06	3.1700E-05	5940.0	1.1700E-07	58.400	9.4100E-09
1.5800E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	243.00	0.0000	0.0000	234.00	216.00	279.00	234.00
0.0000	0.0000	0.0000						
12	-0.4350	8.7400E-14	1.6800E+06	3.1900E-05	6020.0	1.1100E-07	59.700	5.3000E-09
1.5900E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	234.00	0.0000	0.0000	234.00	207.00	270.00	234.00
0.0000	0.0000	0.0000						
13	-0.4350	5.4000E-13	1.6700E+06	3.7400E-07	5950.0	2.2900E-08	58.600	2.0600E-10
1.4700E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	0.0000	270.00	63.000
0.0000	0.0000	0.0000						
14	-0.4350	5.4000E-13	1.6600E+06	3.7300E-07	5940.0	2.2800E-08	58.500	2.0500E-10
1.4700E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	0.0000	270.00	63.000
0.0000	0.0000	0.0000						
15	-0.4350	5.4000E-13	1.6600E+06	3.7300E-07	5940.0	2.2800E-08	58.500	2.0500E-10
1.4700E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	0.0000	270.00	63.000
0.0000	0.0000	0.0000						
16	-0.4350	5.4000E-13	1.6600E+06	3.7300E-07	5940.0	2.2800E-08	58.500	2.0500E-10
1.4700E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	0.0000	270.00	63.000
0.0000	0.0000	0.0000						
17	-0.4350	5.4000E-13	1.6700E+06	3.7400E-07	5950.0	2.2900E-08	58.600	2.0600E-10
1.4700E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	0.0000	270.00	63.000
0.0000	0.0000	0.0000						
18	-0.4350	1.4300E-11	1.6900E+06	9.9300E-06	6050.0	6.2000E-07	60.600	5.5900E-09
1.3400E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	0.0000	270.00	63.000
0.0000	0.0000	0.0000						
19	-0.4350	1.4300E-11	1.6900E+06	9.9000E-06	6030.0	6.1700E-07	60.200	5.5600E-09
1.3300E+04	2.6200E+10	9.4500E+09						
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	0.0000	270.00	63.000
0.0000	0.0000	0.0000						
20	-0.4350	1.4300E-11	1.6900E+06	9.9300E-06	6060.0	6.2000E-07	60.700	5.6000E-09

M2

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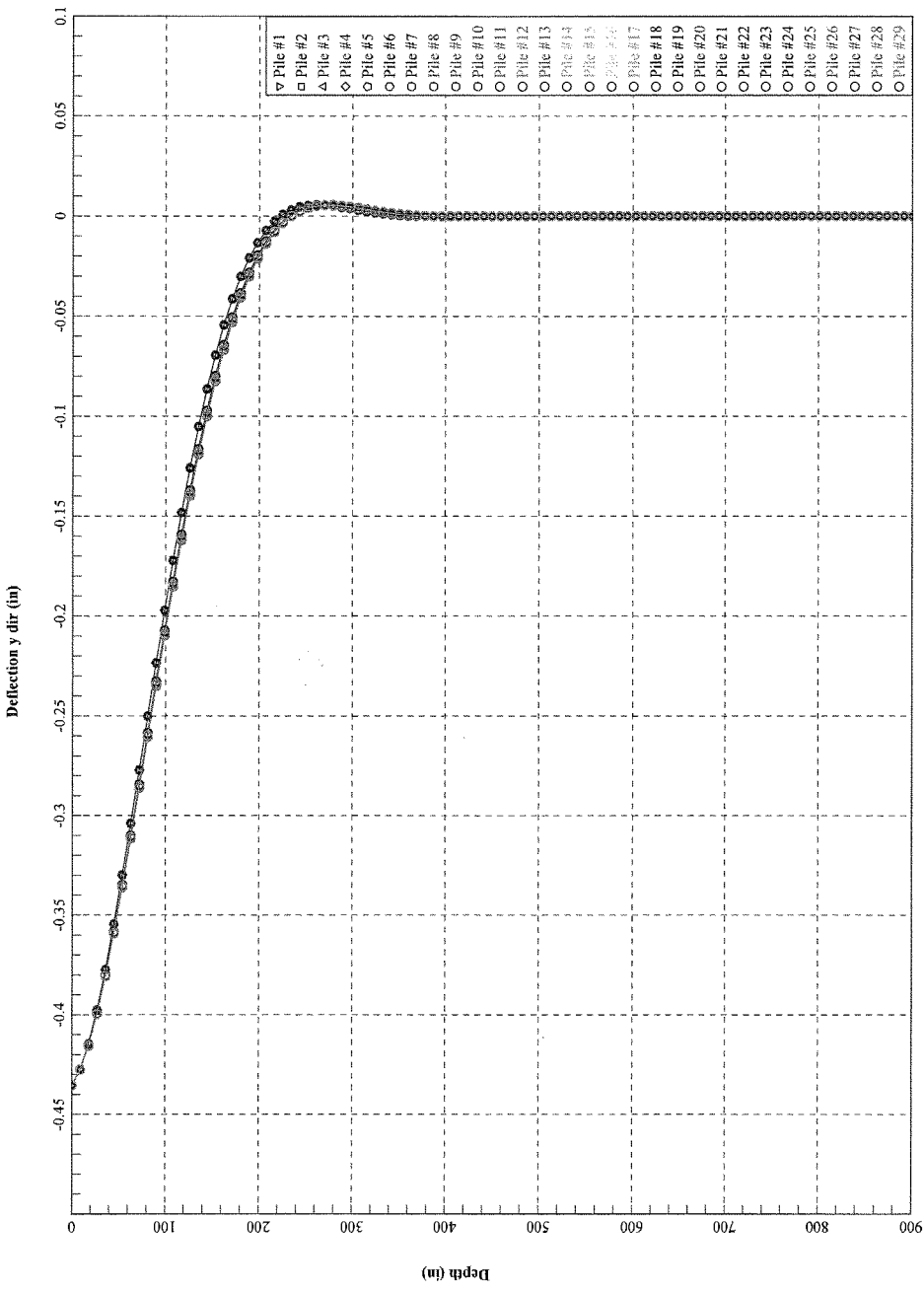
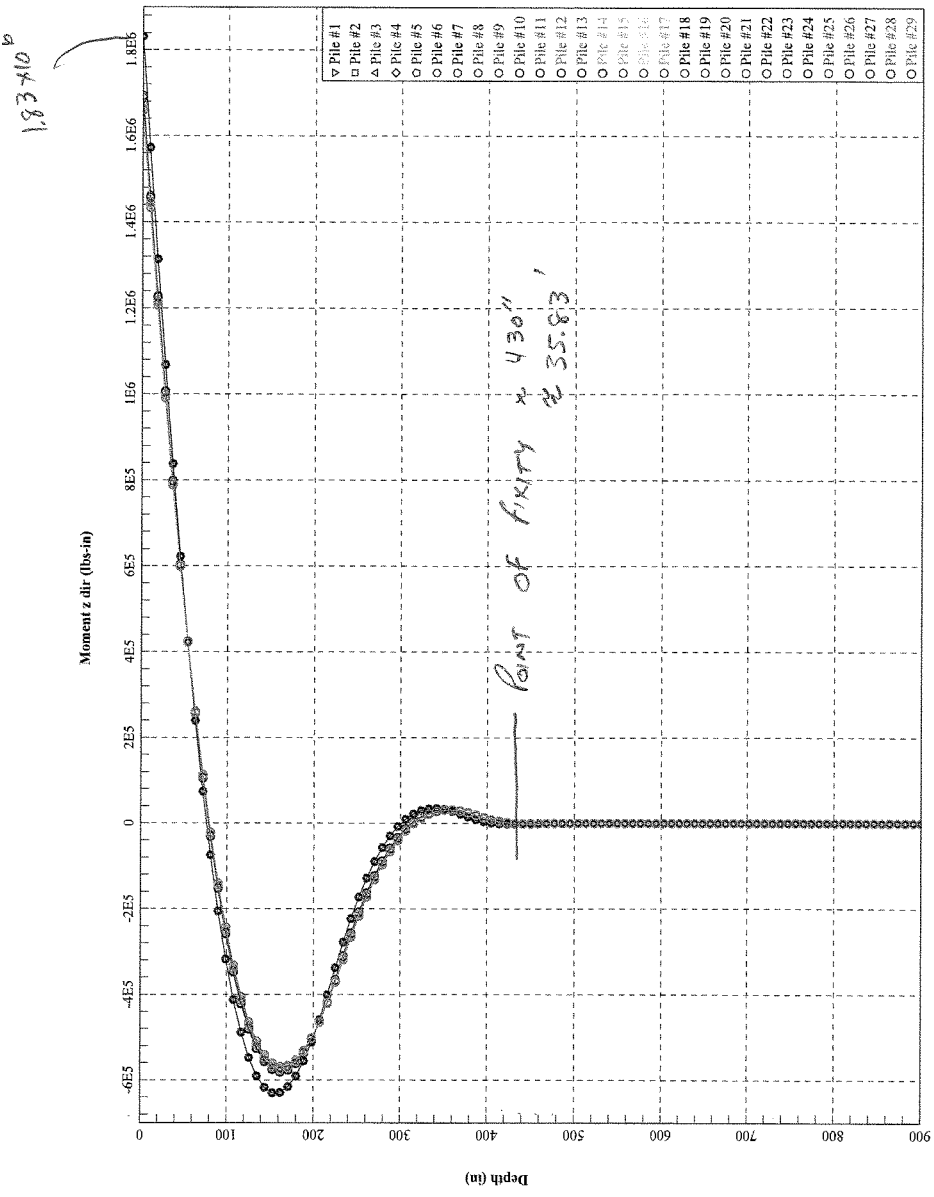
1.3400E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	0.0000	234.00	0.0000	270.00	63.000
0.0000	0.0000	0.0000							
21	-0.4350	1.4300E-11	1.6900E+06	9.9300E-06	6060.0	6.2000E-07	60.700	5.6000E-09	
1.3400E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	0.0000	270.00	63.000	
0.0000	0.0000	0.0000							
22	-0.4350	1.4300E-11	1.6900E+06	9.9000E-06	6030.0	6.1700E-07	60.200	5.5600E-09	
1.3300E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	0.0000	270.00	63.000	
0.0000	0.0000	0.0000							
23	-0.4350	1.4300E-11	1.6900E+06	9.9300E-06	6050.0	6.2000E-07	60.600	5.5900E-09	
1.3400E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	0.0000	270.00	63.000	
0.0000	0.0000	0.0000							
24	-0.4350	2.8000E-11	1.6700E+06	1.9300E-05	5930.0	1.2100E-06	58.800	1.0800E-08	
1.4000E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	9.0000	270.00	63.000	
0.0000	0.0000	0.0000							
25	-0.4350	2.8000E-11	1.6600E+06	1.9200E-05	5860.0	1.1900E-06	58.200	1.0600E-08	
1.3900E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	9.0000	279.00	63.000	
0.0000	0.0000	0.0000							
26	-0.4350	2.8000E-11	1.6600E+06	1.9200E-05	5860.0	1.1900E-06	58.200	1.0600E-08	
1.3900E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	9.0000	279.00	63.000	
0.0000	0.0000	0.0000							
27	-0.4350	2.8000E-11	1.6600E+06	1.9200E-05	5860.0	1.1900E-06	58.200	1.0600E-08	
1.3900E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	9.0000	279.00	63.000	
0.0000	0.0000	0.0000							
28	-0.4350	2.8000E-11	1.6600E+06	1.9200E-05	5860.0	1.1900E-06	58.200	1.0600E-08	
1.3900E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	9.0000	279.00	63.000	
0.0000	0.0000	0.0000							
29	-0.4350	2.8000E-11	1.6700E+06	1.9300E-05	5930.0	1.2100E-06	58.800	1.0800E-08	
1.4000E+04	2.6200E+10	9.4500E+09							
x(IN)	0.0000	0.0000	0.0000	0.0000	234.00	9.0000	270.00	63.000	
0.0000	0.0000	0.0000							
Max.	-0.4350	2.8000E-11	1.8300E+06	7.0600E-05	6860.0	1.2100E-06	71.300	1.0800E-08	
1.8300E+04	2.6200E+10	9.4500E+09							
Pile N.	1	24	1	1	1	24	1	24	
1	1	1							

Row 5


Deflection y dir (in)
Depth (in)
Pile #1
Pile #2
Pile #3
Pile #4
Pile #5
Pile #6
Pile #7
Pile #8
Pile #9
Pile #10
Pile #11
Pile #12
Pile #13
Pile #18
Pile #19
Pile #20
Pile #21
Pile #22
Pile #23
Pile #24
Pile #25
Pile #26
Pile #27
Pile #28
Pile #29


Moment z dir (lbs-in)
Depth (in)
Pile #1
Pile #2
Pile #3
Pile #4
Pile #5
Pile #6
Pile #7
Pile #8
Pile #9
Pile #10
Pile #11
Pile #12
Pile #13
Pile #14
Pile #15
Pile #16
Pile #17
Pile #18
Pile #19
Pile #20
Pile #21
Pile #22
Pile #23
Pile #24
Pile #25
Pile #26
Pile #27
Pile #28
Pile #29
Point of fixity = 430"
= 35.83'

ATTACHMENT

F-Q2.1 Structural Pile Capacity Results

BARR ENGINEERING			DATE	SHEET NO.
			PROJECT NAME	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN 2/9/11		PKN	SUBJECT EM 1110-2-2906 HP 14X73 Axial Capacity compression	

EM 1110-2-2906 Design of Pile Foundations 1/15/1991

(1) Steel Piles. Allowable tension and compression stresses are given for both the lower and upper regions of the pile. Since the lower region of the pile is subject to damage during driving, the basic allowa stress should reflect a high factor of safety. The distribution of allowable tension or compression stress along the length of the pile is shown in Figure 4-1. This factor of safety may be decreased if more is known about the actual driving conditions. Pile shoes should be used when driving in dense sand strata, gravel strata, cobble-boulder zones, and when driving piles to refusal on a hard layer of bedrock. Bending effects are usually minimal in the lower region of the pile. The upper region of the pile may be subject to the effects of bending and buckling as well as axial load. Since damage in the upper region usually apparent during driving, a higher allowable stress is permitted. The upper region of the pile is actualy designed as a beam-column, with due consideration to lateral support conditions. The allowable stresses for fully supported piles are as follows:

STEEL PROPERTIES:

Fy=	50	ksi
E=	29000	ksi

BEAM PROPERTIES:

Beam Type:	HP	W or HP			
depth=	14	nominal depth	code:	14073	
wt/ft =	73				
member	HP	14X73			
Ag=	21.4	in^2	T =	11.22	in
d=	13.6	in	Ix=	729	in^4
tw=	0.505	in	Sx=	107	in^3
bf=	14.6	in	rx=	5.84	in
tf=	0.505	in	Iy=	261	in^4
rt=	3.9		Sy=	35.8	in^3
d/Af=	1.85		ry=	3.49	in

HP 14x73 A

120 TON

Mx allow = 1224 K-ft

Load Type :	Usual	enter Usual, Unusual or Extreme	Usual	Unusual	Extreme	
allow. stress increase	1		1	1.33	1.75	allowable stress increase EM 1110-2-2906, 4-1

Loading:

Mx=	122.92	k-ft
My=	0.00	k-ft
V =	0.00	k
Pa=	240.00	k
Lb=	1.00	ft
Cb =	1.00	
Kx=	1.00	
Ky=	1.00	
Lx=	1.00	ft
Ly=	1.00	ft

Cb = 1.75 + 1.05 (M1/M2) + 0.3 (M1/M2) <= 2.3
M1/M2 is ratio of smaller to larger moments at ends of unbraced member.
M1/M2 is positive when bent in reverse curvature.
If Bending moment within an unbraced length is larger than at both ends than Cb = 1.

M1= 0.00
M2= 1.00
Cb = 1.75

Tension or Compression in Lower region

Fa = 1/3 * Fy * 5/6 =	13.89	ksi	Concentric axial tension or compression
Fa = 1/3 * Fy =	16.67	ksi	Concentric axial tension or compression only with driving shoe
Fa = 1 / 2.5 * Fy =	20.00	ksi	Concentric axial tension or compression only with driving shoe, at least one axial load test and use of a pile driving analyzer to verify the pile capacity and integrity
Select Fa =	13.89	ksi	Select from above values depending on driving shoe & testing
stress increase * Fa =	13.89	ksi	Select from above values depending on driving shoe & testing
fa = Pa / Ag =	11.21	ksi	**OK.....Fa > fa**
fa / Fa =	**0.81**		**Lower Region OK**

Pmax = Fa*A = 297.2 k

Combined bending and axial compression in Upper Region

fa/Fa + fbx / Fb + fby / Fb <= 1.0

allow. stress increase 1

allowable axial stress				
Fa = 5/6 * 3/5 Fy =	25.00	ksi		25 ksi
allowable bending stress			w/o Stress Increase	
Fb = 5/6 * 3/5 Fy =	25.00	ksi	for noncompact sections	25 ksi
Fb = 5/6 * 2/3 Fy =	27.78	ksi	for compact sections	27.777778 ksi

Bending Calculations: AISC Chapter F

AISC TABLE B5.1			Compact	Noncompact	
bf/2*tf=	14.46	noncompact	9.19	13.44	
d/tw=	26.93	ok	90.51	-	webs in flexural compression

Slender - Check AISC Appendix B5

Fbx =	25.00	ksi includes allowable stress increase	Fby =	25.00	ksi includes allowable stress increase
fbx= Mx/Sx =	13.79	ksi OK	fby= My/Sy =	0.00	ksi OK

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			PROJECT NAME	
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fbx / Fbx = 0.551 fby / Fby = 0.000

Axial Load Calculation: Upper Pile Region

Fa= 25.000 ksi includes allowable stress increase

fa = Pa/A = 11.21 ksi Pmax = Fa*A = 535.0 k

fa/Fa = 0.45 ksi

Combined Bending and Axial Compression in Upper Pile Region

fa/Fa + fbx / Fb + fby / Fb ≤ 1.0

fa / Fa =	0.45
fbx / Fbx =	0.551
fby / Fby =	0.000
	1.00 OK

BARR ENGINEERING			DATE	SHEET NO.
			PROJECT NAME	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN 2/9/11		PKN	SUBJECT EM 1110-2-2906 HP 14X89 Axial Capacity compression	

EM 1110-2-2906 Design of Pile Foundations 1/15/1991

(1) Steel Piles. Allowable tension and compression stresses are given for both the lower and upper regions of the pile. Since the lower region of the pile is subject to damage during driving, the basic allowa stress should reflect a high factor of safety. The distribution of allowable tension or compression stress along the length of the pile is shown in Figure 4-1. This factor of safety may be decreased if more is known about the actual driving conditions. Pile shoes should be used when driving in dense sand strata, gravel strata, cobble-boulder zones, and when driving piles to refusal on a hard layer of bedrock. Bending effects are usually minimal in the lower region of the pile. The upper region of the pile may be subject to the effects of bending and buckling as well as axial load. Since damage in the upper region usually apparent during driving, a higher allowable stress is permitted. The upper region of the pile is actually designed as a beam-column, with due consideration to lateral support conditions. The allowable stresses for fully supported piles are as follows:

STEEL PROPERTIES:

Fy= 50 ksi
E= 29000 ksi

BEAM PROPERTIES:

Beam Type: HP W or HP
depth= 14 nominal depth code: 14089
wt/ft = 89
member HP 14X89
Ag= 26.1 in^2 T = 11.18 in
d= 13.8 in Ix= 904 in^4
tw= 0.615 in Sx= 131 in^3
bf= 14.7 in rx= 5.88 in
tf= 0.615 in Iy= 326 in^4
rt= 3.94 Sy= 44.3 in^3
d/Af= 1.53 ry= 3.53 in

Load Type : **Usual** enter Usual, Unusual or Extreme

	Usual	Unusual	Extreme	
allow. stress increase	1	1.33	1.75	allowable stress increase EM 1110-2-2906, 4-1

Loading:

Mx= 172.53 k-ft
My= 0.00 k-ft
V = 0.00 k
Pa= 240.00 k

Cb = 1.75 + 1.05 (M1/M2) + 0.3 (M1/M2) <= 2.3
M1/M2 is ratio of smaller to larger moments at ends of unbraced member.
M1/M2 is positive when bent in reverse curvature.
If Bending moment within an unbraced length is larger than at both ends than Cb = 1.

Lb= 1.00 ft
Cb = 1.00
Kx= 1.00
Ky= 1.00
Lx= 1.00 ft
Ly= 1.00 ft
M1= 0.00
M2= 1.00
Cb = 1.75

HP 14x89 X
P_y = 120 ton
M_x = 172.5 k-ft

Tension or Compression in Lower region

Fa = 1/3 * Fy * 5/6 =	13.89 ksi		Concentric axial tension or compression
Fa = 1/3 * Fy =	16.67 ksi		Concentric axial tension or compression only with driving shoe
Fa = 1 / 2.5 * Fy =	20.00 ksi		Concentric axial tension or compression only with driving shoe, at least one axial load test and use of a pile driving analyzer to verify the pile capacity and integrity
Select Fa =	**13.89** ksi		Select from above values depending on driving shoe & testing
stress increase * Fa =	**13.89** ksi		Select from above values depending on driving shoe & testing
fa = Pa / Ag =	9.20 ksi	**OK.....Fa > fa**	
fa / Fa =	**0.66**	**Lower Region OK**	**Pmax = Fa*A = 362.5 k**

Combined bending and axial compression in Upper Region

fa/Fa + fbx / Fb + fby / Fb <= 1.0 allow. stress increase 1

allowable axial stress				
Fa = 5/6 * 3/5 Fy =	25.00 ksi			25 ksi
allowable bending stress			w/o Stress Increase	
Fb = 5/6 * 3/5 Fy =	25.00 ksi	for noncompact sections		25 ksi
Fb = 5/6 * 2/3 Fy =	27.78 ksi	for compact sections		27.777778 ksi

Bending Calculations: AISC Chapter F

AISC TABLE B5.1			Compact	Noncompact	
bf/2*tf=	11.95	noncompact	9.19	13.44	
d/tw=	22.44	ok	90.51	-	webs in flexural compression

Noncompact Section

Fbx = 25.00 ksi includes allowable stress increase **Fby = 25.00 ksi** includes allowable stress increase

fbx= Mx/Sx = 15.80 ksi OK **fby= My/Sy = 0.00 ksi** OK

BARR ENGINEERING			DATE	SHEET NO.
			PROJECT NAME	
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fbx / Fbx = 0.632 fby / Fby = 0.000

Axial Load Calculation: Upper Pile Region

Fa= 25.000 ksi includes allowable stress increase

fa = Pa/A = 9.20 ksi Pmax = Fa*A = 652.5 k

fa/Fa = 0.37 ksi

Combined Bending and Axial Compression in Upper Pile Region

fa/Fa ± fbx / Fb ± fby / Fb ≤ 1.0

fa / Fa =	0.37
fbx / Fbx =	0.632
fby / Fby =	0.000
	1.00 OK

BARR ENGINEERING			DATE	SHEET NO.
			PROJECT NAME	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN 2/9/11		PKN	SUBJECT EM 1110-2-2906 HP 14X89 Axial Capacity compression	

EM 1110-2-2906 Design of Pile Foundations 1/15/1991

(1) Steel Piles. Allowable tension and compression stresses are given for both the lower and upper regions of the pile. Since the lower region of the pile is subject to damage during driving, the basic allowa stress should reflect a high factor of safety. The distribution of allowable tension or compression stress along the length of the pile is shown in Figure 4-1. This factor of safety may be decreased if more is known about the actual driving conditions. Pile shoes should be used when driving in dense sand strata, gravel strata, cobble-boulder zones, and when driving piles to refusal on a hard layer of bedrock. Bending effects are usually minimal in the lower region of the pile. The upper region of the pile may be subject to the effects of bending and buckling as well as axial load. Since damage in the upper region usually apparent during driving, a higher allowable stress is permitted. The upper region of the pile is actually designed as a beam-column, with due consideration to lateral support conditions. The allowable stresses for fully supported piles are as follows:

ARN WINGWALL A

STEEL PROPERTIES:

Fy= 50 ksi
E= 29000 ksi

BEAM PROPERTIES:

Beam Type: HP W or HP
depth= 14 nominal depth code: 14089
wt/ft = 89
member HP 14X89

Ag=	26.1 in^2	T =	11.18 in
d=	13.8 in	Ix=	904 in^4
tw=	0.615 in	Sx=	131 in^3
bf=	14.7 in	rx=	5.88 in
tf=	0.615 in	Iy=	326 in^4
n=	3.94	Sy=	44.3 in^3
d/Af=	1.53	ry=	3.53 in

Load Type : Usual enter Usual, Unusual or Extreme

	Usual	Unusual	Extreme	
allow. stress increase	1	1.33	1.75	allowable stress increase EM 1110-2-2906, 4-1

Loading:

PILE 1 LOADING FROM GROUP

Mx=	193.08	k-ft
My=	0.00	k-ft
V =	0.00	k
Pa=	123.20	k

6 1.56 Tons

Cb = 1.75 + 1.05 (M1/M2) + 0.3 (M1/M2) <= 2.3
M1/M2 is ratio of smaller to larger moments at ends of unbraced member.
M1/M2 is positive when bent in reverse curvature.
If Bending moment within an unbraced length is larger than at both ends than Cb = 1.

Lb=	1.00	ft	M1= 0.00
Cb =	1.00		M2= 1.00
Kx=	1.00		Cb = 1.75
Ky=	1.00		
Lx=	1.00	ft	
Ly=	1.00	ft	

Tension or Compression in Lower region

Fa = 1/3 * Fy * 5/6 =	13.89	ksi	Concentric axial tension or compression
Fa = 1/3 * Fy =	16.67	ksi	Concentric axial tension or compression only with driving shoe
Fa = 1 / 2.5 * Fy =	20.00	ksi	Concentric axial tension or compression only with driving shoe, at least one axial load test and use of a pile driving analyzer to verify the pile capacity and integrity
Select Fa =	13.89	ksi	Select from above values depending on driving shoe & testing
stress increase * Fa =	13.89	ksi	Select from above values depending on driving shoe & testing
fa = Pa / Ag =	4.72	ksi	**OK.....Fa > fa**
fa / Fa =	**0.34**		**Lower Region OK** **Pmax = Fa*A =** **362.5 k**

Combined bending and axial compression in Upper Region

fa/Fa + fbx / Fb + fby / Fb <= 1.0 allow. stress increase 1

allowable axial stress
Fa = 5/6 * 3/5 Fy = 25.00 ksi 25 ksi

allowable bending stress
Fb = 5/6 * 3/5 Fy = 25.00 ksi for noncompact sections w/o Stress Increase 25 ksi
Fb = 5/6 * 2/3 Fy = 27.78 ksi for compact sections 27.777778 ksi

Bending Calculations: AISC Chapter F

AISC TABLE B5.1			Compact	Noncompact	
bf/2*tf=	11.95	noncompact	9.19	13.44	
d/tw=	22.44	ok	90.51	-	webs in flexural compression

Noncompact Section

Fbx = 25.00 ksi includes allowable stress increase **Fby = 25.00 ksi** includes allowable stress increase

fbx= Mx/Sx = **17.69 ksi** OK **fby= My/Sy =** **0.00 ksi** OK

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			PROJECT NAME	
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PKN 2/9/11		PKN	SUBJECT EM 1110-2-2906 HP 14X89 Axial Capacity compression	

fbx / Fbx = 0.707 fby / Fby = 0.000

Axial Load Calculation: Upper Pile Region

Fa= 25.000 ksi includes allowable stress increase

fa = Pa/A = 4.72 ksi Pmax = Fa*A = 652.5 k

fa/Fa = 0.19 ksi

Combined Bending and Axial Compression in Upper Pile Region

fa/Fa ± fbx / Fb ± fby / Fb ≤ 1.0

fa / Fa = 0.19
fbx / Fbx = 0.707
fby / Fby = 0.000
0.90 OK

BARR ENGINEERING			DATE	SHEET NO.
			PROJECT NAME	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN 2/16/11		PKN	SUBJECT EM 1110-2-2906 HP 14X89 Axial Capacity compression	RRN Wing Wall F

EM 1110-2-2906 Design of Pile Foundations 1/15/1991

(1) Steel Piles. Allowable tension and compression stresses are given for both the lower and upper regions of the pile. Since the lower region of the pile is subject to damage during driving, the basic allowa stress should reflect a high factor of safety. The distribution of allowable tension or compression stress along the length of the pile is shown in Figure 4-1. This factor of safety may be decreased if more is known about the actual driving conditions. Pile shoes should be used when driving in dense sand strata, gravel strata, cobble-boulder zones, and when driving piles to refusal on a hard layer of bedrock. Bending effects are usually minimal in the lower region of the pile. The upper region of the pile may be subject to the effects of bending and buckling as well as axial load. Since damage in the upper region usually apparent during driving, a higher allowable stress is permitted. The upper region of the pile is actually designed as a beam-column, with due consideration to lateral support conditions. The allowable stresses for fully supported pilas are as follows:

STEEL PROPERTIES:

Fy= 50 ksi
E= 29000 ksi

BEAM PROPERTIES:

Beam Type: HP W or HP
depth= 14 nominal depth code: 14089
wt/ft = 89
member HP 14X89

Ag=	26.1 in^2	T =	11.18 in
d=	13.8 in	Ix=	904 in^4
tw=	0.615 in	Sx=	131 in^3
bf=	14.7 in	rx=	5.88 in
tf=	0.615 in	Iy=	326 in^4
rt=	3.94	Sy=	44.3 in^3
d/Af=	1.53	ry=	3.53 in

GROUP OUT PUT FOR ROW 1

P = 55.62 TON

M_X = 152.6 K-FT

Load Type :	Usual	enter Usual, Unusual or Extreme	Usual	Unusual	Extreme	
allow. stress increase	1		1	1.33	1.75	allowable stress increase EM 1110-2-2906, 4-1

Loading:

Mx= 189.26 k-ft
My= 0.00 k-ft
V = 0.00 k
Pa= 200.00 k = 100 TON

FOR 100 TON PILE $M_{X\,MAX}$ = 189.26 K-FT

Cb = 1.75 + 1.05 (M1/M2) + 0.3 (M1/M2)^2 <= 2.3
M1/M2 is ratio of smaller to larger moments at ends of unbraced member.
M1/M2 is positive when bent in reverse curvature.
If Bending moment within an unbraced length is larger than at both ends than Cb = 1.

Lb= 1.00 ft
Cb = 1.00
Kx= 1.00
Ky= 1.00
Lx= 1.00 ft
Ly= 1.00 ft
M1= 0.00
M2= 1.00
Cb = 1.75

Tension or Compression in Lower region

Fa = 1/3 * Fy * 5/6 =	13.89 ksi		Concentric axial tension or compression
Fa = 1/3 * Fy =	16.67 ksi		Concentric axial tension or compression only with driving shoe
Fa = 1 / 2.5 * Fy =	20.00 ksi		Concentric axial tension or compression only with driving shoe, at least one axial load test and use of a pile driving analyzer to verify the pile capacity and integrity
Select Fa =	**13.89 ksi**		Select from above values depending on driving shoe & testing
stress increase * Fa =	**13.89 ksi**		Select from above values depending on driving shoe & testing
fa = Pa / Ag =	7.66 ksi	**OK.....Fa > fa**	
fa / Fa =	**0.55**	**Lower Region OK**	**Pmax = Fa*A = 362.5 k**

Combined bending and axial compression in Upper Region

fa/Fa ± fbx / Fb ± fby / Fb ≤ 1.0 allow. stress increase 1

allowable axial stress
Fa = 5/6 * 3/5 Fy = 25.00 ksi — 25 ksi

allowable bending stress
Fb = 5/6 * 3/5 Fy = 25.00 ksi for noncompact sections — w/o Stress Increase 25 ksi
Fb = 5/6 * 2/3 Fy = 27.78 ksi for compact sections — 27.777778 ksi

Bending Calculations: AISC Chapter F

AISC TABLE B5.1		Compact	Noncompact	
bf/2*tf=	11.95 noncompact	9.19	13.44	
d/tw=	22.44 ok	90.51	-	webs in flexural compression

Noncompact Section

Fbx = 25.00 ksi includes allowable stress increase **Fby = 25.00 ksi** includes allowable stress increase

fbx= Mx/Sx = 17.34 ksi OK **fby= My/Sy =** 0.00 ksi OK

BARR ENGINEERING			DATE	SHEET NO.
			PROJECT NAME	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN 2/16/11		PKN	SUBJECT EM 1110-2-2906 HP 14X89 Axial Capacity compression RRN Wing Wall F	

fbx / Fbx = 0.693 fby / Fby = 0.000

Axial Load Calculation: Upper Pile Region

Fa= 25.000 ksi includes allowable stress increase

fa = Pa/A = 7.66 ksi

Pmax = Fa*A = 652.5 k

fa/Fa = 0.31 ksi

Combined Bending and Axial Compression in Upper Pile Region

fa/Fa + fbx / Fb + fby / Fb ≤ 1.0

fa / Fa = 0.31
fbx / Fbx = 0.693
fby / Fby = 0.000
1.00 OK

ATTACHMENT

F-Q3.1 Ice Loads on Piers

BARR ENGINEERING			DATE	SHEET NO.
			PROJECT NAME	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	
PKN 2/2/11		PKN	SUBJECT ICE LOADS on Piers	

AASHTO 3.9 Ice Loads

RRN & WILD RICE

3.9.2 Dynamic Ice Forces on Piers

3.9.2.2 Crushing and Flexing

if $w/t \leq 6.0$, then

F = lesser of either Fc or, when ice failure by flexure is considered applicable as described herein, Fb, and

if $w/t > 6.0$

F = Fc

$Fc = Ca\ p\ t\ w$

$Fb = Cn\ p\ t^2$

$Ca = (5\ t/w + 1)^{0.5}$

$Cn = 0.5 / [\tan(\alpha - 15)]$

where:

t = thickness of ice (ft)
α = inclination of the nose to the vertical (deg)
p = effective ice crushing stength as specified in Article 3.9.2.1 (ksf)
w = pier width at level of ice action (ft)
Fc = horizontal ice force caused by ice floes that fail by crushing over the full width of the pier (kip)
Fb = horizontal ice force caused by ice floes that fail by flexure as they ride up the inclined pier nose (kip)
Ca = coefficient accounting for the effect of the pier width/ice thickness ratio where the floe fails by crushing
Cn = coefficient accounting for the inclination of the pier nose with respect to a vertical

where $\alpha \leq 15$ deg, ice failure by flexure shall not be considered to be a possible ice failure mode for the purpose of caculating the horizontal force, F, in which case F shall be taken as Fc

PIER WIDTH

t =	1	ft			
α =	90	deg			
p =	56	psi	=	8.064 ksf	
w =	8	ft			
t/w =	0.125			w/t = 8	> 6 Use Fc
Ca =	1.274755				
Cn =	0.133975				
Fc =	82.24	k			
Fb =	1.08	k		F = 82.24 k	

3.9.2.4 Combination of Longitudinal and Transverse Forces (Piers Parallel to Flow)

$Ft = F / [2 \tan(\beta / 2 + \theta_f)]$

β = 100 deg, nose angle in horizontal plane for a round nose taken as 100 deg
θ_f = 10 deg, friction angle between ice and pier nose, deg

Ft = 23.74 k

A) A longitudinal force equal to F shall be combined with the transverse force of 0.15F, or

B) A longitudinal force equal to 0.5F shall be combined with the transverse force of Ft

F (k)	Ft (k)
82.24	12.34
41.12	23.74

Both the longitudinal and transverse forces shall be assumed to act at the pier nose

3.9.4 Hanging Dams and Ice Jams

The frazil accumulation in a hanging dam may be taken to exert a pressure of 0.2 to 2.0 ksf as it moves by the pier. An ice jam may be taken to exert a pressure of 0.02 to 0.20 ksf.

RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT R – STRUCTURAL DESIGN COMPUTATIONS – AQUEDUCTS

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

By: Barr Engineering Co.

FINAL – February 28, 2011

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F-R4.1 Sheyenne River Aqueduct Structural Computations
F-R4.2 Sheyenne River Aqueduct Structure Pile Computations
F-R4.3 Sheyenne River Aqueduct Structure Wall Panel E Computations
F-R4.4 Sheyenne River Aqueduct Structure Drawings

F-R5.1 Sheyenne River Aqueduct Structure Pile Computations
F-R5.2 Sheyenne River Aqueduct Structure Wall Panel E Computations
F-R5.3 Maple River Aqueduct Structure Drawings

ELECTRONIC ATTACHMENTS ONLY

The complete set of structural computations for all load conditions for the Sheyenne River aqueduct structure and Maple River aqueduct structure is included electronically on the attached DVD.

F-R6.1 Sheyenne River Aqueduct Structure – Low Flow Channel Walls Computations
F-R6.2 Sheyenne River Aqueduct Structure – Mat Foundation Computations
F-R6.3 Sheyenne River Aqueduct Structure – Pile Computations
F-R6.4 Sheyenne River Aqueduct Structure – Retaining Walls Panel A Computations
F-R6.5 Sheyenne River Aqueduct Structure – Retaining Walls Panel B Computations
F-R6.6 Sheyenne River Aqueduct Structure – Retaining Walls Panel C Computations
F-R6.7 Sheyenne River Aqueduct Structure – Retaining Walls Panel D Computations
F-R6.8 Sheyenne River Aqueduct Structure – Retaining Walls Panel E Computations
F-R5.9 Sheyenne River Aqueduct Structure – Retaining Walls Panel F Computations
F-R6.10 Sheyenne River Aqueduct Structure – Retaining Walls Panel G Computations
F-R6.11 Sheyenne River Aqueduct Structure – Retaining Walls Panel H Computations
F-R6.12 Sheyenne River Aqueduct Structure – Retaining Walls Panel I Computations
F-R6.13 Sheyenne River Aqueduct Structure – Tributary Channel Deck Computations
F-R6.14 Sheyenne River Aqueduct Structure – Wall and Piers Computations
F-R6.15 Sheyenne River Aqueduct Structure – R2 STAAD Input
F-R6.16 Sheyenne River Aqueduct Structure – R2 STAAD Output
F-R6.17 Sheyenne River Aqueduct Structure – R2 STAAD Detailed Output
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F-R7.1 Maple River Aqueduct Structure – Pile Computations
F-R7.2 Maple River Aqueduct Structure – Retaining Walls Panel A Computations
F-R7.3 Maple River Aqueduct Structure – Retaining Walls Panel B Computations
F-R7.4 Maple River Aqueduct Structure – Retaining Walls Panel C Computations
F-R7.5 Maple River Aqueduct Structure – Retaining Walls Panel D Computations
F-R7.6 Maple River Aqueduct Structure – Retaining Walls Panel E Computations
F-R7.7 Maple River Aqueduct Structure – Retaining Walls Panel F Computations
F-R7.8 Maple River Aqueduct Structure – Retaining Walls Panel G Computations
F-R7.9 Maple River Aqueduct Structure – Retaining Walls Panel H Computations
F-R7.10 Maple River Aqueduct Structure – Retaining Walls Panel I Computations

APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT R – STRUCTURAL DESIGN COMPUTATIONS - AQUEDUCTS

F-R1.0 BASIS OF DESIGN

Structural design of the aqueducts at, and the diversion channel crossings of the Sheyenne and Maple Rivers, will be based on the following assumptions.

F-R1.1 APPLICABLE CODES, STANDARDS AND GUIDELINES

United State Army Corps of Engineers (USACE) Engineer Manuals:

- EM 1110-2-1612 "Ice Engineering"
- EM 1110-2-2100 "Stability Analysis of Concrete Structures"
- EM 1110-2-2104 "Structural Design for Reinforced-Concrete Hydraulic Structures"
- EM 1110-2-2502 "Retaining and Flood Walls"
- EM 1110-2-2504 "Design of Sheet Pile Walls"
- EM 1110-2-2906 "Design of Pile Foundations"

American Concrete Institute (ACI)

- ACI 301 Standard Specification for Structural Concrete
- ACI 302.1R Guide for Concrete Floor & Slab Construction
- ACI 318 Building Code Requirements for Structural Concrete
- ACI 350R, Environmental Engineering Concrete Structures
- ACI 350.3/350.3R, Seismic Design of Liquid-Containing Concrete Structures
- ACI 360R, Design of Slabs on Grade

American Society of Civil Engineers (ASCE)

- ASCE 7-05 Minimum Design Loads for Buildings and Other Structures

F-R1.2 DESIGN LOADS

F-R1.2.1 Loading Conditions

According to the discussions with USACE, the following seven loading cases were evaluated to generate the design conditions for the different structures and their components:

- Case 1 – Usual Loading Condition – 100-Year Flood
- Case 2 – Unusual Loading Condition – 100-Year Flood + Ice Loading
- Case 3 – Unusual Loading Condition – 500-Year Flood
- Case 4 – Extreme Loading Condition – Water Level is at the top of the structures
- Case 5 – Usual Loading Condition – Normal Flow + Ice
- Case 6 – Unusual Loading Condition – End of Construction condition

F-R1.2.2 Load Combinations
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 increased 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 year 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)

F-R1.2.3 Loads
Dead Loads: Selfweight of the concrete structure is assumed as dead loads by assuming normal weight concrete density of 150 lbs/ft^3.

Live Loads: HS-20 loading is assumed on the access bridge. HS-20 load is assumed to be 640 psf for a 12-ft wide strip. HS-20 loading governs over any other live loads that can be subjected on the access bridge slab.

Wind Loads: No wind loads are assumed since the hydrostatic load on the walls are significantly larger than the wind loads.

Seismic Loads: The peak ground acceleration with 2% probability of exceedance in 50 years for the project site is between 0.02g and 0.08g according to the 2003 National Seismic Hazard Maps provided by the United States Geological Survey. Any anticipated earthquake damage in this zone is considered to be minor and is not expected to have an influence on the future stability of the project structures. In accordance with Chapter III of the FERC *Engineering Guidelines for Evaluation of Hydropower Projects* seismic loading is not considered when the peak ground accelerations for the site are less than 0.1g at the site. The mapped spectral accelerations based on USGS Seismic Maps for Latitude = 46° 43' 59.78" (46.733271°) and Longitude = -96° 48' 22.95" (-96.806374°) are as follows:

- Short Period, $S_s = 0.078g$
- 1-second Period, $S_1 = 0.022g$

Hydrostatic Loads: Internal and external fluid loading is determined by the specific gravity of the water of 62.4 pcf. The tributary channel walls are designed for maximum water levels at the top of the walls assuming no fluid pressure on the opposite side of the wall. Different uplift pressures are assumed underneath the mat foundation depending on the load conditions as shown on Table F-R1 and F-R2.

Hydrostatic fluid load behind the approach and wing wall is neglected during the design. Weep holes and drainage system are assumed to be installed behind these retaining walls to avoid any accumulation of water.

Ice Loading: Ice loading of 10,000 pounds per linear foot (plf) for an ice thickness of 2-feet applied to the contact surface of the structure is assumed in the tributary channel for normal flow case. Ice load is applied at El. 903.24. As a secondary protection, de-icers will be installed inside the tributary walls to avoid ice building. Impact structures will be installed in the diversion channel to avoid any big ice particles hitting the structure.

Recent review of the ice conditions during the 2009 flood in the Fargo/Moorhead area work by ice engineering expert Andrew Tuthill of the US Army Cold Regions Research and Engineering Laboratory (CRREL), revealed that the static ice loads assumed during 100 and 500 year events were unrealistic and conservative. Mr. Tuthill provided recommended ice thicknesses and effective ice crushing strengths to be used for ice floes acting on piers. Calculations for the dynamic ice force follow the methods outlined in AASHTO LRFD Bridge Design Specifications.

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 9:52 a.m. These loads will be applied to the piers.

Earth Loading: The below grade walls and retaining walls around the aqueducts is designed using the maximum exterior soil loads and the minimum internal loads. No surcharge assumed.

Lateral Soil Pressures:

- Friction Angle, $\phi = 30$
- Active Pressure Coefficient, Ka = 0.33 (Rankine)
- Moist Soil Unit Weight, $\vartheta s = 120$ pcf
- Saturated Soil Unit Weight, $\vartheta s = 125$ pcf
- Moist Soil Active Pressure, Pa = Ka ϑs Z = 0.27 x 125 = 34 pcf
- Saturated Soil =(ϑs Z – u) Ka + u = (125–62.4) 0.33 = 19 pcf

F-R1.3 DESIGN APPROACH AND 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-feet 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 350 ft beyond the downstream face of the Diversion channel.
h) Stem of the approach and wing walls varies from 14 to 33 feet with 4 ft thick.

2) Reinforcing Steel
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.

5) The soil conditions along the entire project are underlain by "soft" clay, susceptible to large settlements. Therefore, aqueducts structures, approach and wing walls are supported by H-piles.

F-R2.0 SHEYENNE RIVER AQUEDUCT STRUCTURE

Table F-R1 presents the key elevations and loading conditions for the Sheyenne River Aqueduct Structure. Table F-R2 presents the pile capacity of the Sheyenne River Aqueduct Structure. Figure F-R1 presents a typical cross section of the Sheyenne River Aqueduct Structure.

Table F-R1 Sheyenne River Aqueduct – Key Elevations and Loading Conditions

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tributary - Water El. (ft)	914.56	914.56	914.67	917.5	903.24	**NA**
Diversion - Head Water El. (ft)	902.12	902.12	903.32	917.5	**NA**	**NA**
Diversion - Tail Water El. (ft)	901.91	901.91	903.06	917.5	**NA**	**NA**
Tributary - T.O. Wall El. (ft)	917.5					
Tributary - T.O. Deck L.P. El.(ft)	898.7					
Tributary - T.O. Deck H.P. El.(ft)	900.7					
Diversion - T.O. Mat El. (ft)	901.68					
Tributary - Deck Slab thickness @ L.P. (ft)	2					
Tributary - Deck Slab thickness @ H.P. (ft)	4					
Diversion - Mat Slab thickness (ft)	4					
Tributary - Water height (ft)	**15.86**	**15.86**	**15.97**	**18.8**	**4.54**	**NA**
Diversion - Head Water height (ft)	**0.44**	**0.44**	**1.64**	**15.82**	**NA**	**NA**
Ice	**NA**	2ft Ice	**NA**	**NA**	2ft Ice	**NA**
Ice Load	**NA**	10 kips/ft	**NA**	**NA**	10 kips/ft	**NA**
Ice Load El. (ft)	**NA**	**914.56**	**NA**	**NA**	**903.24**	**NA**
Uplift @ HW (ft)	**4.44**	**4.44**	**5.64**	**19.82**	**NA**	**NA**
Uplift @ TW (ft)	**4.23**	**4.23**	**5.38**	**19.82**	**NA**	**NA**
Pile Condition	**Undrained**	**Undrained**	**Undrained**	**Undrained**	**Drained**	**Undrained**
Load Category	**Usual**	**Unusual**	**Unusual**	**Extreme**	**Usual**	**Unusual**
Safety Factors	2	1.5	1.5	1.15	2	1.5
Allowable Lateral Capacity (tons)	**18**	**20.5**	**20.5**	**24**	**11.5**	**20.5**
Allowable Pile Capacity (tons) - Axial	**61.95**	**82.60**	**82.60**	**107.74**	**36.525**	**82.60**
Allowable Pile Capacity (tons) - Uplift	**38.65**	**51.53**	**51.53**	**67.22**	**5.9**	**51.53**

Table F-R2 Sheyenne River Aqueduct – Pile Capacity

Pile Capacity	Ultimate Axial Capacity	Allowable Lateral Capacity (kips)		
		0.5" (Usual)	0.67" (Unusual)	0.875" (Extreme)
Undrained - Axial	247.8	36	41	48
Undrained - Uplift	154.6			
Drained - Axial	146.1	23	29	36
Drained - Uplift	23.6			

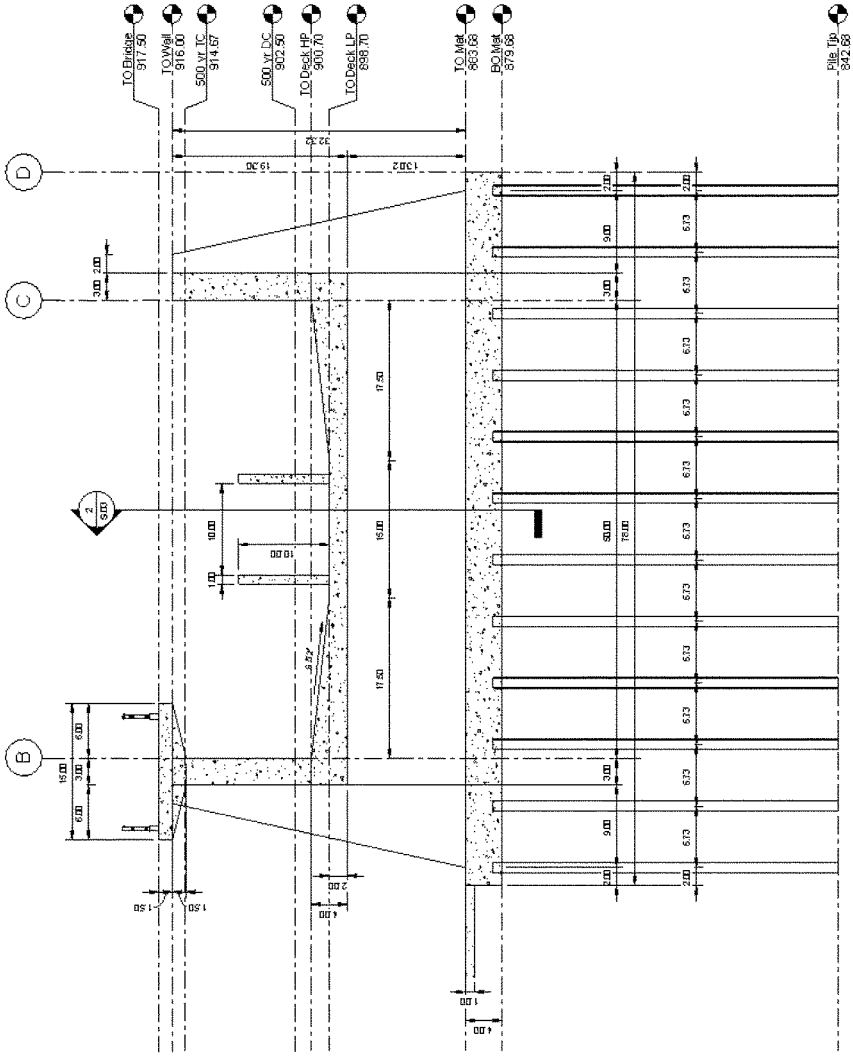

Figure F-R1 Sheyenne River Aqueduct Structure Typical Section (Looking North)

F-R3.0 MAPLE RIVER AQUEDUCT STRUCTURE

Table F-R3 presents the key elevations and loading conditions for the Maple River Aqueduct Structure. Table F-R4 presents the pile capacity of the Maple River Aqueduct Structure. Figure F-R2 presents a typical cross section of the Maple River Aqueduct Structure.

Table F-R3 Maple River Aqueduct – Key Elevations and Loading Conditions

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Constructio n
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tributary - Water El. (ft)	914.56	914.56	914.67	917.5	903.24	**NA**
Diversion - Head Water El. (ft)	902.12	902.12	903.32	917.5	**NA**	**NA**
Diversion - Tail Water El. (ft)	901.91	901.91	903.06	917.5	**NA**	**NA**
Tributary - T.O. Wall El. (ft)	917.5					
Tributary - T.O. Deck L.P. El.(ft)	898.7					
Tributary - T.O. Deck H.P. El.(ft)	900.7					
Diversion - T.O. Mat El. (ft)	901.68					
Tributary - Deck Slab thickness @ L.P. (ft)	2					
Tributary - Deck Slab thickness @ H.P. (ft)	4					
Diversion - Mat Slab thickness (ft)	4					
Tibutary - Water height (ft)	**15.86**	**15.86**	**15.97**	**18.8**	**4.54**	**NA**
Diversion - Head Water height (ft)	**0.44**	**0.44**	**1.64**	**15.82**	**NA**	**NA**
Ice	**NA**	2ft Ice	**NA**	**NA**	2ft Ice	**NA**
Ice Load	**NA**	10 kips/ft	**NA**	**NA**	10 kips/ft	**NA**
Ice Load El. (ft)	**NA**	**914.56**	**NA**	**NA**	**903.24**	**NA**
Uplift @ HW (ft)	**4.44**	**4.44**	**5.64**	**19.82**	**NA**	**NA**
Uplift @ TW (ft)	**4.23**	**4.23**	**5.38**	**19.82**	**NA**	**NA**

Table F-R3 Maple River Aqueduct – Key Elevations and Loading Conditions (continued)

Pile Condition	Undrained	Undrained	Undrained	Undrained	Drained	Undrained
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Safety Factors	2	1.5	1.5	1.15	2	1.5
Allowable Lateral Capacity (tons)	18	20.5	20.5	24	11.5	20.5
Allowable Pile Capacity (tons) - Axial	61.95	82.60	82.60	107.74	36.525	82.60
Allowable Pile Capacity (tons) - Uplift	38.65	51.53	51.53	67.22	5.9	51.53

Table F-R4 Maple River Aqueduct – Pile Capacity

Pile Capacity	Ultimate Axial Capacity	Allowable Lateral Capacity (kips)		
		0.5" (Usual)	0.67" (Unusual)	0.875" (Extreme)
Undrained - Axial	228.7	36	42	48
Undrained - Uplift	135.5			
Drained - Axial	125.7	23	29	33
Drained - Uplift	18.5			

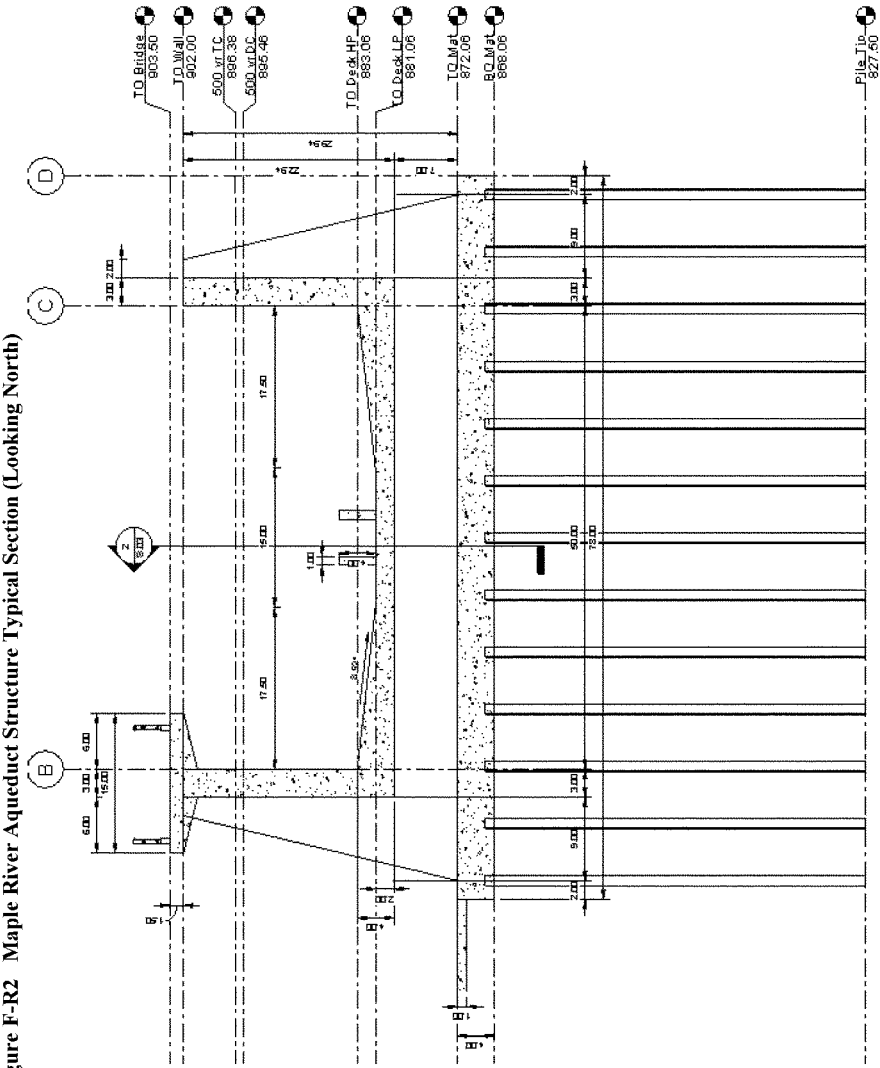

Figure F-R2 Maple River Aqueduct Structure Typical Section (Looking North)

ATTACHMENT

F-R4.1 Sheyenne River Aqueduct Structural Computations

BARR — MINNEAPOLIS, MINNESOTA - HIBBING, MINNESOTA
DULUTH, MINNESOTA
ANN ARBOR, MICHIGAN - JEFFERSON CITY, MISSOURI

			DATE	SHEET NO. 1
			PROJECT NAME	Fuy. marshall Div. Chan.
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	32491004
BY mkd	BY	TO	SUBJECT	Aqueduct Preliminary Reinforcement Design.
DATE 2/21/11	DATE	DATE		

Aqueduct MAT FOUNDATION

STAAD Results.

Moment

M_{xmax} = 102.34 k-ft/ft LC# 50 P2# 14951 M_{xy} = 1.375 k-ft/ft

M_{xmin} = −94.78 k-ft/ft LC# 32 P# 9534 M_{xy} = −1.322 k-ft/ft

M_{ymax} = 106.209 k-ft/ft LC# 50 P# 13278 M_{xy} = 10.59 k-ft/ft

M_{ymin} = −96.415 k-ft/ft LC# 47 P# 9279 M_{xy} = −2.619 k-ft/ft

Same

Note orientation + → Bottom
− → Top

M_y = 106.209 + 10.59 = 116.78 k-ft/ft

b = 12"
h = 48"

cover = 1.5"
for Bottom Bars due to Moment

$A_{s req}$ = 1.021 in²/ft.

A_{smin} T&S > #5 @ 12" A_s = 1.0 in²/ft

Asphalt slacked #906"

A_s = 2 in²/ft

H:\RGN\HOUSE\CONVPADS.CDR

REV.12/1/98

WALL / SLAB REINFORCEMENT DESIGN TEMPLATE

REF. 1. ACI 318 Building Code and Commentary
2. ACI 350 Code Requirements for Environmental Structures

Password: obgmbi
Please Fill in the gray boxes.

Client Name:	U.S. ARMY CORPS OF ENGINEERS	**Design By:**	MBI
Project Name:	FARGO – MOORHEAD , FEASIBILITY STUDY, PHASE 4	**Review By:**	
Work Description:	Sheyenne Aquaduct Structure - Mat Foundation	**Date:**	2/14/2011
		Job #:	34091004

Cross Section Properties			
Height	h (in)	48	in
Width	b (in)	12	in
Section as		Slab Section	
Section Modulus	S (in3)	4608	in^3
Design Forces			
Shear	Vu (kips)	35	kip
Moment	Mu (k-ft/ft)	146.78	kip-ft / ft

Material Properties				
Concrete	f_c' (ksi)	4	ksi	
	E (ksi)		ksi	
Steel	f_y (ksi)	60	ksi	
	E (ksi)	29000	ksi	
	n	9		

Factors	
ϕFlex	0.9
ϕShear	0.75
Governing Code	
ACI 350- 01	

Flexural Reinforcement Design

Cover	1.5	in	*Minimum Required Steel*			
Length Bet. Mov. Jt.	50	ft	$3f_c'^{1/2}/f_y$ =	0.00316	200/fy =	0.00333
Bar index	# 5		$\rho_{min\ Flex}$ =	0.0033333	$A_{s\ min\ Flex}$ =	1.308 in^2
d (Tension Reinf.)	32.6875	in	$\rho_{min\ T\&S}$ =	0.005	$A_{s\ min\ T\&S}$ =	0.720 in^2
Design Mom. (Fact.)	146.78	kip-ft / ft	$\rho_{Balance}$ =	0.0285068	$A_{s\ max}$ =	11.1818 in^2
			Rn	0.153	Constant	0.090
					OK	
a	1.502	in	$A_{s\ Required}$ =	1.021 in^2	ρ Required =	0.0028
k	0.046		$A_{s\ Provide}$ =	1.308 in^2		
j	0.98					
Moment Calculated	146.78	kip-ft	$A_{s\ Selected}$ =	# 5	$A_{s\ Selected}$ =	2.000 in^2
Difference Δ	0.000		Spacing @	6 in	< Amax	
a'	2.941		Spacing is OK per ACI 318			
β_1	0.85					
c	3.460		Tension controlled	ϕ	0.9	
ε_t	0.0253	> 0.005	Same as f assumed initially, OK.			
ϕM_n	280.95	kip-ft / ft > Design Moment, Section OK				

Cracking moment Capacity

$f_{r\ cr}$	474.34	psi	
$M_{r\ Cracking}$ =	182.15	k-ft	

Flexural Moment Capacity

$A_{s\ Selected}$ =	2.000	in^2
$M_{r\ Flex}$ =	280.95	k-ft

Shear Reinforcement Design

Vc	49.62	kip			
ϕShear	0.75				
ϕVc	37.21	kip	>	Vu =	35.00 kip
Vs	9.45	kip			

Minimum Shear Reinforcement

11.4.6 — Minimum shear reinforcement

11.4.6.1 — A minimum area of shear reinforcement, $A_{v\ min}$, shall be provided in all reinforced concrete flexural members (prestressed and nonprestressed) where V_u exceeds $0.5\phi V_c$, except in members satisfying one or more of (a) through (f):

(a) Footings and solid slabs.

(b) Hollow-core units with total untopped depth not greater than 12.5 in. and hollow-core units where V_u is not greater than $0.5\phi V_{cw}$.

(c) Concrete joist construction defined by 8.13;

(d) Beams with h not greater than 10 in.;

(e) Beam integral with slabs with h not greater than 24 in. and not greater than the larger of 2.5 times thickness of flange, and 0.5 times width of web;

(f) Beams constructed of steel fiber-reinforced, normalweight concrete with f_c' not exceeding 6000 psi, h not greater than 24 in., and V_u not greater than $\phi 2\sqrt{f_c'}\, b_w d$.

Is any of the requirements (a thru f) satisfied?				YES	
Av Min / s =	0.0095		OR	50bw/fyt=	0.010
Av Min/s =	0.01				
No Shear reinforcement required.					

Spacing Requirement

s = d/2	16.34375 in		OR	24 in
4fc^0.5*bw*d	99.23 kip		>	Vs = 9.45 kip
smax =	17.00 in			
$A_{v\ Min}$ =	0.17 in^2			
Av/s = Vs/fyt*d	0.0048			
$A_{v\ Required}$ =	0.08 in^2	$A_{v\ Provide}$ =	0.17 in^2	
Stirrup Size	# 3	$A_{s\ Selected}$ =	0.220 in^2	
# Legs	2	> Av Provide, OK		

Calculation of Moment Capacity with Min. Reinforcement

Analize as a: 8

1	$\rho_{min\ Flex.}$	0.00333	Flexural Members
2	$\rho_{min\ Wall\ Ver.}$	0.0015	Wall Vertical (ACI 318 Section 14.3.2)
3	$\rho_{min\ Wall\ Hor.}$	0.0025	Wall Horizontal (ACI 318 Section 14.3.2)
4	$\rho_{min\ Slab\ T\&S}$	0.0018	Temp. & Shrink for Slabs (ACI 318 Section 7.12.2.1)
5	$\rho_{min\ Wall\ Ver.}$	0.003	Wall Vertical (ACI 350 Section 14.3.2)
6	$\rho_{min\ Wall\ Hor.}$	0.005	Wall Horizontal (ACI 350 Section 14.3.3)
7	$\rho_{min\ Slab\ T\&S}$	0.0018	Temp. & Shrink (ACI 318 Section 7.12.2.1)
8	Select Reinforcement Below		

$A_{s\ Selected}$ # 9 @ 12 in $A_{s\ Selected}$ 1 in^2

Than ρ_{min} 0.00000

$A_s = \rho_{min}$*bd or bh	1.0000 in^2	***One Face Only*** As=	0.5 in^2
T = A_s*fy	60.000 kip	T = A_s*fy	30.000 kip
a = T / (0.85 b fc')	1.471 in	a = T / (0.85 b fc')	0.735
M = T*(d-a/2)	159.761 kip-ft	M = T*(d-a/2)	80.800 kip-ft
M_u = ϕ*M	143.785 kip-ft	M_u = ϕ*M	72.720 kip-ft

BARR ENGINEERING
MINNEAPOLIS, MINNESOTA - HIBBING, MINNESOTA
DULUTH, MINNESOTA
ANN ARBOR, MICHIGAN - JEFFERSON CITY, MISSOURI

COMPUTED	CHECKED	SUBMITTED
BY mld	BY	TO
DATE 2/21/11	DATE	DATE

DATE | SHEET NO. 2
PROJECT NAME Twp marker 1 Div. Ch.
PROJECT NUMBER 32/09/1001
SUBJECT

TRIBUTARY CHANNEL FURNACE SAND

SJAMS Results

remarks

$M_{x\,max}$ = 33.81 k-ft/ft LC# 32 R# 1133 M_{xy} = -0.434 k-ft/ft

$M_{x\,min}$ = -106.32 k-ft/ft LC# 15 R# 637 M_{xy} = -2.121 k-ft/ft

$M_{y\,max}$ = 42.958 k-ft/ft LC# 55 R# 716 M_{xy} = -2.517 k-ft/ft

$M_{y\,min}$ = -77.671 k-ft/ft LC# 37 R# 711 M_{xy} = 0.713 k-ft/ft

Governs

Plate coordinates

y = Longitudinal
x = Transverse
+ = Top
- = Bottom

M = 42.958 + 2.517 = 45.46 k-ft/ft

b = 12"
h = 24"

$A_{s\,required}$ = .1666 in²/ft

$A_{s\,min}$ = .1 in²/ft.

Asphalt selected #9@6" A_s = 2.0 in²/ft ✓ ok.

H:\PROJECTS\BRR\003\CAD\PRADS.CDR

REV.12/1/98

WALL / SLAB REINFORCEMENT DESIGN TEMPLATE

REF. 1. ACI 318 Building Code and Commentary
2. ACI 350 Code Requirements for Environmental Structures

Password: obgmbi
Please Fill in the gray boxes.

Client Name:	U.S. ARMY CORPS OF ENGINEERS	**Design By:**	MBI
Project Name:	FARGO – MOORHEAD , FEASIBILITY STUDY, PHASE 4	**Review By:**	
Work Description:	Sheyenne Aquaduct Structure - Tributary Channel Deck	**Date:**	2/14/2011
		Job #:	34091004

Cross Section Properties			
Height	h (in)	24	in
Width	b (in)	12	in
Section as		Slab Section	
Section Modulus	S (in3)	1152	in^3
Design Forces			
Shear	Vu (kips)	35	kip
Moment	Mu (k-ft/ft)	145.4	kip-ft / ft

Material Properties				
Concrete	f_c' (ksi)	4	ksi	
	E (ksi)		ksi	
Steel	f_y (ksi)	60	ksi	
	E (ksi)	29000	ksi	
	n	9		

Factors	
φFlex	0.9
φShear	0.75
Governing Code	
ACI 350- 01	

Flexural Reinforcement Design

Cover	3	in	*Minimum Required Steel*		
Length Bet. Mov. Jt.	50	ft	$3\sqrt{f_c'}/f_y$ =	0.00316	200/fy = 0.00333
Bar index	# 5		$\rho_{min\ Flex}$ =	0.0033333	$A_{s\ min\ Flex}$ = 0.828 in^2
d (Tension Reinf.)	20.6875	in	$\rho_{min\ T\&S}$ =	0.005	$A_{s\ min\ T\&S}$ = 0.720 in^2
Design Mom. (Fact.)	145.4	kip-ft / ft	$\rho_{Balance}$ =	0.0285068	$A_{s\ max}$ = 7.07681 in^2
			Rn	0.377	Constant 0.222
					OK
a	2.441	in	$A_{s\ Required}$ =	1.860 in^2	ρ Required = 0.00669
k	0.118		$A_{s\ Provide}$ =	1.860 in^2	
j	0.94				
Moment Calculated	145.40	kip-ft	$A_{s\ Selected}$ =	#5	$A_{s\ Selected}$ = 2.000 in^2
Difference Δ	0.000		Spacing @	6 in	< Amax
a'	2.941		Spacing is OK per ACI 318		
β_1	0.85				
c	3.460		Tension controlled	φ	0.9
ε_t	0.0149	> 0.005	Same as f assumed initially, OK.		
ϕM_n	172.95	kip-ft / ft > Design Moment, Section OK			

Cracking moment Capacity

$f_{r\ cr}$	474.34	psi
$M_{l\ Cracking}$ =	45.54	k-ft

Flexural Moment Capacity

$A_{s\ Selected}$ =	2.000	in^2
$M_{u\ Flex}$ =	172.95	k-ft

Shear Reinforcement Design

Vc	31.40	kip		
φShear	0.75			
φVc	23.55	kip	< Vu =	35.00 kip
Vs	23.12	kip		

Minimum Shear Reinforcement

11.4.6 — Minimum shear reinforcement

11.4.6.1 — A minimum area of shear reinforcement, $A_{v\ min}$, shall be provided in all reinforced concrete flexural members (prestressed and nonprestressed) where V_u exceeds $0.5\phi V_c$, except in members satisfying one or more of (a) through (f):

(a) Footings and solid slabs;

(b) Hollow-core units with total untopped depth not greater than 12.5 in. and hollow-core units where V_u is not greater than $0.5\phi V_{cw}$;

(c) Concrete joist construction defined by 8.13;

(d) Beams with h not greater than 10 in.;

(e) Beam integral with slabs with h not greater than 24 in. and not greater than the larger of 2.5 times thickness of flange, and 0.5 times width of web;

(f) Beams constructed of steel fiber-reinforced, normalweight concrete with f_c' not exceeding 6000 psi, h not greater than 24 in., and V_u not greater than $\phi 2\sqrt{f_c'}\, b_w d$.

Is any of the requirements (a thru f) satisfied? YES

Av Min / s =	0.0095	OR	50bw/fyt=	0.010
Av Min/s =	0.01			

Shear Reinforcement Required

Spacing Requirement

s = d/2	10.34375 in	OR	24 in	
4fc^0.5*bw*d	62.80 kip	>	Vs =	23.12 kip
smax =	11.00 in			
$A_{v\ Min}$ =	0.11 in^2			
Av/s = Vs/fyt*d	0.0186			
$A_{v\ Required}$ =	0.20 in^2	$A_{v\ Provide}$ =	0.20 in^2	
Stirrup Size	#3	$A_{s\ Selected}$ =	0.220 in^2	
# Legs	2	> Av Provide, OK		

Calculation of Moment Capacity with Min. Reinforcement

Analize as a: 6

1	$\rho_{min\ Flex.}$	0.00333	Flexural Members
2	$\rho_{min\ Wall\ Ver.}$	0.0015	Wall Vertical (ACI 318 Section 14.3.2)
3	$\rho_{min\ Wall\ Hor.}$	0.0025	Wall Horizontal (ACI 318 Section 14.3.2)
4	$\rho_{min\ Slab\ T\&S}$	0.0018	Temp. & Shrink for Slabs (ACI 318 Section 7.12.2.1)
5	$\rho_{min\ Wall\ Ver.}$	0.003	Wall Vertical (ACI 350 Section 14.3.2)
6	$\rho_{min\ Wall\ Hor.}$	0.005	Wall Horizontal (ACI 350 Section 14.3.3)
7	$\rho_{min\ Slab\ T\&S}$	0.0018	Temp. & Shrink (ACI 318 Section 7.12.2.1)
8	Select Reinforcement Below		

$A_{s\ Selected}$ #6 @ 12 in $A_{s\ Selected}$ 1 in^2

Than ρ_{min} 0.00000

$A_s = \rho_{min}$*bd or bh	1.0000 in^2	*One Face Only* As=	0.5 in^2
T = A_s*fy	60.000 kip	T = A_s*fy	30.000 kip
a = T / (0.85 b fc')	1.471 in	a = T / (0.85 b fc')	0.735
M = T*(d-a/2)	99.761 kip-ft	M = T*(d-a/2)	50.800 kip-ft
M_u = φ*M	89.785 kip-ft	M_u = φ*M	45.720 kip-ft

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DATE	SHEET NO. 3
PROJECT NAME Bay marked Div. Ch.	
PROJECT NUMBER 32.0162	
SUBJECT	

COMPUTED	CHECKED	SUBMITTED
BY mbl	BY	TO
DATE 2/21/11	DATE	DATE

PIERS / WALLS

moments

$M_{x\,max} = 92.17$ k-ft/ft LC # 37 R # 6519 $M_{y} = 25.92$ k-ft/ft

$M_{x\,min} = -111.725$ k-ft/ft LC # 55 R # 9241 $M_{y} = -25.16$ k-ft/ft

$M_{y\,max} = 105.502$ k-ft/ft LC # 37 R # 6517 $M_{y} = 39.74$ k-ft/ft

$M_{y\,min} = -130.312$ k-ft/ft LC # 27 R # 1176 $M_{y} = 2.755$ k-ft/ft

Combine

$M = 111.725 + 45.16 = 156.88$ k-ft/ft

b = 12"
h = 36"
cover = 3"

$A_{s\,required} = 1.093$ in²/ft

$A_{s\,min\ flex} = 1.368$ in²/ft

A_s provided #9@6" $A_s = 2.0$ in²/ft

H:\PLG\INHOUSE\COMPPADS.CDR
15 Squares per Inch
REV.12/1/98

WALL / SLAB REINFORCEMENT DESIGN TEMPLATE

REF.
1. ACI 318 Building Code and Commentary
2. ACI 350 Code Requirements for Environmental Structures

Password: obgmbi
Please Fill in the gray boxes.

Client Name:	U.S. ARMY CORPS OF ENGINEERS	**Design By:**	MBI
Project Name:	FARGO – MOORHEAD , FEASIBILITY STUDY, PHASE 4	**Review By:**	
Work Describtion:	Sheyenne Aquaduct Structure - Walls and Piers	**Date:**	2/14/2011
		Job #:	34091004

Cross Section Properties			
Height	h (in)	36	in
Width	b (in)	12	in
Section as		Wall Section	
Section Modulus	S (in3)	2592	in^2
Design Forces			
Shear	Vu (kips)	35	kip
Moment	Mu (k-ft/ft)	156.88	kip-ft / ft

Material Properties			
Concrete	f_c' (ksi)	4	ksi
	E (ksi)		ksi
Steel	f_y (ksi)	60	ksi
	E (ksi)	29000	ksi
	n	9	

Factors	
ϕFlex	0.9
ϕShear	0.75
Governing Code	
ACI 350- 01	

Flexural Reinforcement Design

Cover	3	in			
Length Bet. Mov. Jt.	50	ft			
Bar index	# 5				
d (Tension Reinf.)	32.6875	in			
Design Mom. (Fact.)	156.88	kip-ft / ft			
a	1.608	in			
k	0.049				
j	0.98				
Moment Calculated	156.88	kip-ft			
Difference Δ	0.000				
a'	2.941				
β_1	0.85				
c	3.460				
ρ_t	0.0253	>	0.005		
ϕM_n	280.95	kip-ft / ft	>	Design Moment, Section OK	

Minimum Required Steel

$3\sqrt{f'_c}/f_y$ =	0.00316	200/fy =	0.00333
$\rho_{min\ Flex}$ =	0.0033333	$A_{s\ min\ Flex}$ =	1.308 in^2
$\rho_{min\ T\&S}$ =	0.005	$A_{s\ min\ T\&S}$ =	0.720 in^2
$\rho_{balance}$ =	0.0285068	$A_{s\ max}$ =	11.1818 in^2
Rn	0.163	Constant	0.096
		OK	
$A_{s\ Required}$ =	1.093 in^2	ρ Required =	0.00279
$A_{s\ Provide}$ =	1.308 in^2		
$A_{s\ Selected}$ =	# 8	$A_{s\ Selected}$ =	2.000 in^2
Spacing @	6 in	< Amax	

Spacing is OK per ACI 318

Tension controlled ϕ 0.9
Same as f assumed initially, OK.

Cracking moment Capacity

$f_{r\ =}$	474.34 psi		
$M_{r\ Cracking}$ =	102.46 k-ft		

Flexural Moment Capacity

$A_{s\ Selected}$ =	2.000 in^2		
$M_{n\ Flex}$ =	280.95 k-ft		

Shear Reinforcement Design

Vc	49.62 kip				
ϕShear	0.75				
ϕVc	37.21 kip	>	Vu =	35.00 kip	
Vs	9.45 kip				

Minimum Shear Reinforcement
11.4.6 — Minimum shear reinforcement

11.4.6.1 — A minimum area of shear reinforcement, $A_{v\ min}$, shall be provided in all reinforced concrete flexural members (prestressed and nonprestressed) where V_u exceeds $0.5\phi V_c$, except in members satisfying one or more of (a) through (f):

(a) Footings and solid slabs;

(b) Hollow-core units with total untopped depth not greater than 12.5 in. and hollow-core units where V_u is not greater than $0.5\phi V_{cw}$;

(c) Concrete joist construction defined by 8.13;

(d) Beams with h not greater than 10 in.;

(e) Beam integral with slabs with h not greater than 24 in. and not greater than the larger of 2.5 times thickness of flange, and 0.5 times width of web;

(f) Beams constructed of steel fiber-reinforced, normalweight concrete with f'_c not exceeding 6000 psi, h not greater than 24 in., and V_u not greater than $\phi 0.17\sqrt{f'_c}\ b_w d$.

Is any of the requirements (a thru f) satisfied? YES

Av Min / s =	0.0095	OR	50bw/fyt=	0.010
Av Min/s =	0.01			

No Shear reinforcement required.

Spacing Requirement

s = d/2	16.34375 in	OR	24 in	
4fc^0.5*bw*d	99.23 kip	>	Vs =	9.45 kip
smax =	17.00 in			
$A_{v\ Min}$ =	0.17 in^2			
Av/s = Vs/fyt*d	0.0048			
$A_{v\ Required}$ =	0.08 in^2	$A_{v\ Provide}$ =	0.17 in^2	
Stirrup Size	# 3	$A_{v\ Selected}$ =	0.220 in^2	
# Legs	2	> Av Provide, OK		

Calculation of Moment Capacity with Min. Reinforcement

Analize as a: 8

1	$\rho_{min\ Flex.}$	0.00333	Flexural Members
2	$\rho_{min\ Wall\ Ver.}$	0.0015	Wall Vertical (ACI 318 Section 14.3.2)
3	$\rho_{min\ Wall\ Hor.}$	0.0025	Wall Horizontal (ACI 318 Section 14.3.2)
4	$\rho_{min\ Slab\ T\&S}$	0.0018	Temp. & Shrink for Slabs (ACI 318 Section 7.12.2.1)
5	$\rho_{min\ Wall\ Ver.}$	0.003	Wall Vertical (ACI 350 Section 14.3.2)
6	$\rho_{min\ Wall\ Hor.}$	0.005	Wall Horizontal (ACI 350 Section 14.3.3)
7	$\rho_{min\ Slab\ T\&S}$	0.0018	Temp. & Shrink (ACI 318 Section 7.12.2.1)
8	Select Reinforcement Below		

$A_{s\ Selected}$ # 9 @ 12 in $A_{s\ Selected}$ 1 in^2

Than ρ_{min} 0.00000

$A_s = \rho_{min}$*bd or bh	1.0000 in^2	***One Face Only*** As=	0.5 in^2
T = A_s*fy	60.000 kip	T = A_s*fy	30.000 kip
a = T / (0.85 b fc')	1.471 in	a = T / (0.85 b fc')	0.735
M = T*(d-a/2)	159.761 kip-ft	M = T*(d-a/2)	80.800 kip-ft
M_u = ϕ*M	143.785 kip-ft	M_u = ϕ*M	72.720 kip-ft

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DULUTH, MINNESOTA
ANN ARBOR, MICHIGAN - JEFFERSON CITY, MISSOURI

COMPUTED	CHECKED	SUBMITTED
BY mlj	BY	TO
DATE 2/21/11	DATE	DATE

DATE — SHEET NO. 4
PROJECT NAME Twp marked Dr. ch.
PROJECT NUMBER 3240911
SUBJECT

Low flow CHANNEL WALLS

moments

M_{xmax} = 4.011 k-ft/ft LC# 53 P2# 8693 M_{xy} = 0.652 k-ft/ft

M_{xmin} = −0.232 k-ft/ft LC# 55 P2# 8739 M_{xy} = 0.032 k-ft/ft

M_{ymax} = 85.159 k-ft/ft LC# 53 P2# 8665 M_{xy} = 1.157 k-ft/ft

M_{ymin} = −0.551 k-ft/ft LC# 55 P2# 8673 M_{xy} = 0.037 k-ft/ft

Covers M_{ymax} + M_{xymax}

y: vertical
x: horizontal
+ → inside
− → outside

M = 85.159 + 1.157 = 86.316 k-ft/ft

b = 12"
h = 15"
cover = 3"

$A_{s req'd}$ = 1.958 in²/ft

A_{smin} = #9@6" = 1 in²/ft

As Provide sked #9@6" As = 2.00 in²/ft

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WALL / SLAB REINFORCEMENT DESIGN TEMPLATE

REF.
1. ACI 318 Building Code and Commentary
2. ACI 350 Code Requirements for Environmental Structures

Password: obgmbi
Please Fill in the gray boxes.

Client Name:	U.S. ARMY CORPS OF ENGINEERS	**Design By:**	MBI
Project Name:	FARGO – MOORHEAD , FEASIBILITY STUDY, PHASE 4	**Review By:**	
Work Description:	Sheyenne Aquaduct Structure - Low flow Channel Walls	**Date:**	2/14/2011
		Job #:	34091004

Cross Section Properties			
Height	h (in)	15	in
Width	b (in)	12	in
Section as		Wall Section	
Section Modulus	S (in3)	450	in^3
Design Forces			
Shear	Vu (kips)	35	kip
Moment	Mu (k-ft/ft)	86.316	kip-ft / ft

Material Properties				
Concrete	f'_c (ksi)	4	ksi	
	E (ksi)		ksi	
Steel	f_y (ksi)	60	ksi	
	E (ksi)	29000	ksi	
	n	9		

Factors	
ϕFlex	0.9
ϕShear	0.75
Governing Code	
ACI 350- 01	

Flexural Reinforcement Design

Cover	3	in			
Length Bet. Mov. Jt.	50	ft			
Bar index	# 5				
d (Tension Reinf.)	11.6875	in			
Design Mom. (Fact.)	86.316	kip-ft / ft			
a	2.733	in			
k	0.234				
j	0.88				
Moment Calculated	86.32	kip-ft			
Difference Δ	0.000				
a'	2.941				
β_1	0.85				
c	3.460				
ϵ_t	0.0071	>	0.005		
ϕM_n	91.95	kip-ft / ft	>	Design Moment, Section OK	

Minimum Required Steel

$3\sqrt{f'_c}/f_y$ =	0.00316	200/fy =	0.00333
$\rho_{min\ Flex}$ =	0.0033333	$A_{s\ min\ Flex}$ =	0.468 in^2
$\rho_{min\ T\&S}$ =	0.005	$A_{s\ min\ T\&S}$ =	0.450 in^2
$\rho_{balance}$ =	0.0285068	$A_{s\ max}$ =	3.99808 in^2
Rn	0.702	Constant	0.413
		OK	
$A_{s\ Required}$ =	1.858 in^2	P Required =	0.01325
$A_{s\ Provide}$ =	1.858 in^2		
$A_{s\ Selected}$ =	# 5	$A_{s\ Selected}$ =	2.000 in^2
Spacing @	6 in	< Amax	
Spacing is OK per ACI 318			
Tension controlled	ϕ	0.9	
Same as f assumed initially, OK.			

Cracking moment Capacity

$f_{r\ s}$	474.34	psi
$M_{r\ Cracking}$ =	17.79	k-ft

Flexural Moment Capacity

$A_{s\ Selected}$ =	2.000	in^2
$M_{u\ Flex}$ =	91.95	k-ft

Shear Reinforcement Design

Vc	17.74	kip		
ϕShear	0.75			
ϕVc	13.31	kip	<	Vu = 35.00 kip
Vs	33.36	kip		

Minimum Shear Reinforcement
11.4.6 — Minimum shear reinforcement

11.4.6.1 — A minimum area of shear reinforcement, $A_{v\ min}$, shall be provided in all reinforced concrete flexural members (prestressed and nonprestressed) where V_u exceeds $0.5\phi V_c$, except in members satisfying one or more of (a) through (f):

(a) Footings and solid slabs;

(b) Hollow-core units with total untopped depth not greater than 12.5 in. and hollow-core units where V_u is not greater than $0.5\phi V_{cw}$;

(c) Concrete joist construction defined by 8.13;

(d) Beams with h not greater than 10 in.;

(e) Beam integral with slabs with h not greater than 24 in. and not greater than the larger of 2.5 times thickness of flange, and 0.5 times width of web;

(f) Beams constructed of steel fiber-reinforced, normalweight concrete with f'_c not exceeding 6000 psi, h not greater than 24 in., and V_u not greater than $\phi 2\sqrt{f'_c}\ b_w d$.

Is any of the requirements (a thru f) satisfied? YES

Av Min / s =	0.0095	OR	50bw/fyt=	0.010	
Av Min/s =	0.01				
Shear Reinforcement Required					
Spacing Requirement					
s = d/2	5.84375	in	OR	24	in
4fc^0.5*bw*d	35.48	kip	>	Vs =	33.36 kip
smax =	6.00	in			
$A_{v\ Min}$ =	0.06	in^2			
Av/s = Vs/fyt*d	0.0476				
$A_{v\ Required}$ =	0.29	in^2	$A_{v\ Provide}$ =	0.29	in^2
Stirrup Size	# 3		$A_{s\ Selected}$ =	0.220	in^2
# Legs	2		> Av Provide, OK		

Calculation of Moment Capacity with Min. Reinforcement

Analize as a: 8

1	$\rho_{min\ Flex.}$	0.00333	Flexural Members
2	$\rho_{min\ Wall\ Ver.}$	0.0015	Wall Vertical (ACI 318 Section 14.3.2)
3	$\rho_{min\ Wall\ Hor.}$	0.0025	Wall Horizontal (ACI 318 Section 14.3.2)
4	$\rho_{min\ Slab\ T\&S}$	0.0018	Temp. & Shrink for Slabs (ACI 318 Section 7.12.2.1)
5	$\rho_{min\ Wall\ Ver.}$	0.003	Wall Vertical (ACI 350 Section 14.3.2)
6	$\rho_{min\ Wall\ Hor.}$	0.005	Wall Horizontal (ACI 350 Section 14.3.3)
7	$\rho_{min\ Slab\ T\&S}$	0.0018	Temp. & Shrink (ACI 318 Section 7.12.2.1)
8	Select Reinforcement Below		

$A_{s\ Selected}$: # 9 @ 12 in — $A_{s\ Selected}$ = 1 in^2

Than ρ_{min} 0.00000

$A_s = \rho_{min}$*bd or bh	1.0000 in^2	***One Face Only*** As=	0.5 in^2
T = A_s*fy	60.000 kip	T = A_s*fy	30.000 kip
a = T / (0.85 b fc')	1.471 in	a = T / (0.85 b fc')	0.735
M = T*(d-a/2)	54.761 kip-ft	M = T*(d-a/2)	28.300 kip-ft
M_u = ϕ*M	49.285 kip-ft	M_u = ϕ*M	25.470 kip-ft

Barr Engineering		Job No 34091009	Sheet No 1	Rev 2
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Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct F	Date/Time 24-Feb-2011 15:13	

Job Information

	Engineer	Checked	Approved
Name:	mbi		
Date:	1-24-11		

Comments

Only (3) 30-ft bays is modeled for simplicity.
Wall thickness = 36-in
Tributary Slab thickness = 32" (ave)
Access Bridge slab thickness = 27" (avarage)
Matt foundation thickness = 48"
Low flow channel walls = 12"
REV1. Geometry Update
REV2. Wall Thickness ad pile loc update

Structure Type	SPACE FRAME

Number of Nodes	11893	Highest Node	17741
Number of Plates	11666	Highest Plate	17054

Number of Basic Load Cases	19
Number of Combination Load Cases	0

Included in this printout are data for:

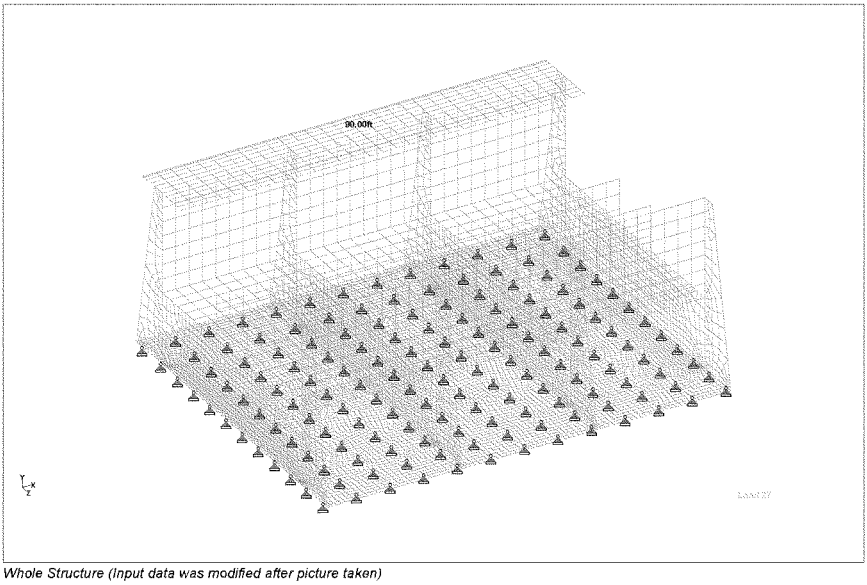
All	The Whole Structure

Included in this printout are results for load cases:

Type	L/C	Name
Primary	3	DEAD LOAD D
Primary	4	LIVE LOAD L
Primary	5	FLUID LOAD F1 (OVERTOPPING DC FUL
Primary	6	FLUID LOAD F2 (OVERTOPPING TC FUL
Primary	7	FLUID LOAD F3 (500YR+ICE)
Primary	8	SOIL LOAD H
Primary	9	FLUID LOAD F4 (LOWFLOW CHANNEL F
Primary	24	1.3(1.4D+1.7L)
Primary	27	1.3(1.4D+1.7L+1.7F1)
Primary	32	1.3(1.4D+1.7L+1.7F2)
Primary	33	1.3(1.4D+1.7L+1.7F3)
Primary	34	1.3(1.4D+1.7L+1.7F4)
Primary	37	1.3(1.4D+1.7L+1.7F1&F2)
Primary	42	1.3(1.4D+1.7L+1.7H)
Primary	45	1.3(1.4D+1.7L+1.7F1+1.7H)
Primary	47	1.3(1.4D+1.7L+1.7F2+1.7H)
Primary	50	1.3(1.4D+1.7L+1.7F3+1.7H)
Primary	53	1.3(1.4D+1.7L+1.7F4+1.7H)
Primary	55	1.3(1.4D+1.7L+1.7 F1&F2+1.7H)

Materials

Mat	Name	E (kip/in^2)	ν	Density (kip/in^3)	α (/°F)
1	STEEL	29E+3	0.300	0.000	6E -6
2	STAINLESSSTEEL	28E+3	0.300	0.000	10E -6
3	ALUMINUM	10E+3	0.330	0.000	13E -6
4	CONCRETE	3.15E+3	0.170	0.000	5E -6



Whole Structure (Input data was modified after picture taken)

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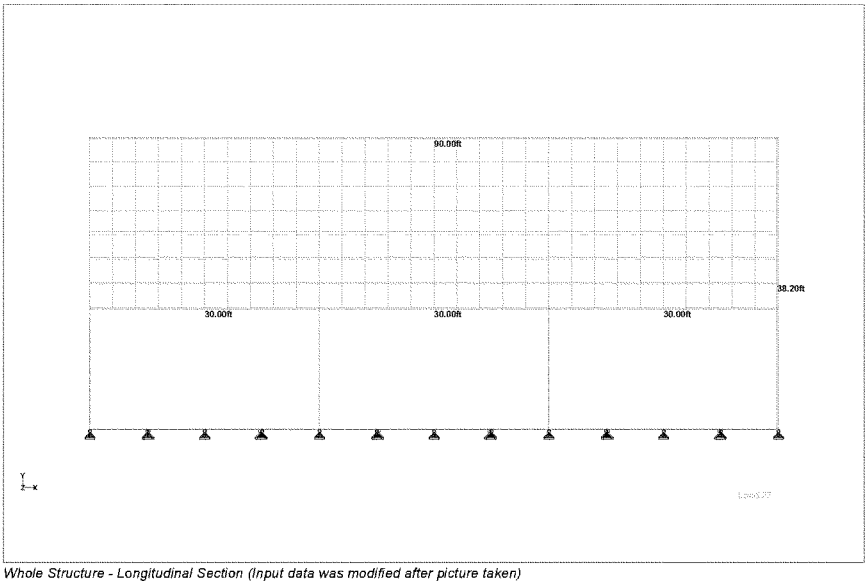
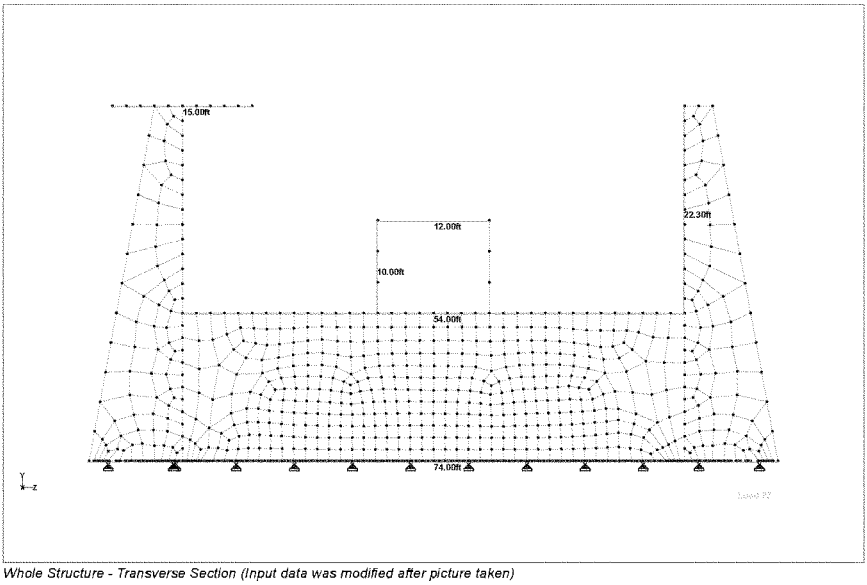
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct F	Date/Time	24-Feb-2011 15:13



Whole Structure - Longitudinal Section (Input data was modified after picture taken)



Whole Structure - Transverse Section (Input data was modified after picture taken)

Supports

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip-ft/deg)	rY (kip-ft/deg)	rZ (kip-ft/deg)
186	Fixed	Fixed	Fixed	-	-	-
190	Fixed	Fixed	Fixed	-	-	-
242	Fixed	Fixed	Fixed	-	-	-
246	Fixed	Fixed	Fixed	-	-	-
298	Fixed	Fixed	Fixed	-	-	-
302	Fixed	Fixed	Fixed	-	-	-
354	Fixed	Fixed	Fixed	-	-	-
358	Fixed	Fixed	Fixed	-	-	-
678	Fixed	Fixed	Fixed	-	-	-
726	Fixed	Fixed	Fixed	-	-	-
774	Fixed	Fixed	Fixed	-	-	-
822	Fixed	Fixed	Fixed	-	-	-
2307	Fixed	Fixed	Fixed	-	-	-
2312	Fixed	Fixed	Fixed	-	-	-
2314	Fixed	Fixed	Fixed	-	-	-
2364	Fixed	Fixed	Fixed	-	-	-

Supports Cont...

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip-ft/deg)	rY (kip-ft/deg)	rZ (kip-ft/deg)
2366	Fixed	Fixed	Fixed	-	-	-
2371	Fixed	Fixed	Fixed	-	-	-
6973	Fixed	Fixed	Fixed	-	-	-
6978	Fixed	Fixed	Fixed	-	-	-
6983	Fixed	Fixed	Fixed	-	-	-
6988	Fixed	Fixed	Fixed	-	-	-
7488	Fixed	Fixed	Fixed	-	-	-
7587	Fixed	Fixed	Fixed	-	-	-
7661	Fixed	Fixed	Fixed	-	-	-
7772	Fixed	Fixed	Fixed	-	-	-
7894	Fixed	Fixed	Fixed	-	-	-
7923	Fixed	Fixed	Fixed	-	-	-
7956	Fixed	Fixed	Fixed	-	-	-
8018	Fixed	Fixed	Fixed	-	-	-
8117	Fixed	Fixed	Fixed	-	-	-
8191	Fixed	Fixed	Fixed	-	-	-
8302	Fixed	Fixed	Fixed	-	-	-
8424	Fixed	Fixed	Fixed	-	-	-
8453	Fixed	Fixed	Fixed	-	-	-
8486	Fixed	Fixed	Fixed	-	-	-
8645	Fixed	Fixed	Fixed	-	-	-
8717	Fixed	Fixed	Fixed	-	-	-
8826	Fixed	Fixed	Fixed	-	-	-
8972	Fixed	Fixed	Fixed	-	-	-
9556	Fixed	Fixed	Fixed	-	-	-
9557	Fixed	Fixed	Fixed	-	-	-
9558	Fixed	Fixed	Fixed	-	-	-
9559	Fixed	Fixed	Fixed	-	-	-
9574	Fixed	Fixed	Fixed	-	-	-
9683	Fixed	Fixed	Fixed	-	-	-
9767	Fixed	Fixed	Fixed	-	-	-
9849	Fixed	Fixed	Fixed	-	-	-
9904	Fixed	Fixed	Fixed	-	-	-
9905	Fixed	Fixed	Fixed	-	-	-
9906	Fixed	Fixed	Fixed	-	-	-
9907	Fixed	Fixed	Fixed	-	-	-
9908	Fixed	Fixed	Fixed	-	-	-
9909	Fixed	Fixed	Fixed	-	-	-
9910	Fixed	Fixed	Fixed	-	-	-
9911	Fixed	Fixed	Fixed	-	-	-
9912	Fixed	Fixed	Fixed	-	-	-
9913	Fixed	Fixed	Fixed	-	-	-
9914	Fixed	Fixed	Fixed	-	-	-
9916	Fixed	Fixed	Fixed	-	-	-
9917	Fixed	Fixed	Fixed	-	-	-

Supports Cont...

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip˚ft/deg)	rY (kip˚ft/deg)	rZ (kip˚ft/deg)
9918	Fixed	Fixed	Fixed	-	-	-
9919	Fixed	Fixed	Fixed	-	-	-
9920	Fixed	Fixed	Fixed	-	-	-
9921	Fixed	Fixed	Fixed	-	-	-
9922	Fixed	Fixed	Fixed	-	-	-
9923	Fixed	Fixed	Fixed	-	-	-
9924	Fixed	Fixed	Fixed	-	-	-
9925	Fixed	Fixed	Fixed	-	-	-
9926	Fixed	Fixed	Fixed	-	-	-
9928	Fixed	Fixed	Fixed	-	-	-
9929	Fixed	Fixed	Fixed	-	-	-
9930	Fixed	Fixed	Fixed	-	-	-
9931	Fixed	Fixed	Fixed	-	-	-
9932	Fixed	Fixed	Fixed	-	-	-
9933	Fixed	Fixed	Fixed	-	-	-
9934	Fixed	Fixed	Fixed	-	-	-
9935	Fixed	Fixed	Fixed	-	-	-
9936	Fixed	Fixed	Fixed	-	-	-
9937	Fixed	Fixed	Fixed	-	-	-
9938	Fixed	Fixed	Fixed	-	-	-
9940	Fixed	Fixed	Fixed	-	-	-
9941	Fixed	Fixed	Fixed	-	-	-
9942	Fixed	Fixed	Fixed	-	-	-
9943	Fixed	Fixed	Fixed	-	-	-
9944	Fixed	Fixed	Fixed	-	-	-
9945	Fixed	Fixed	Fixed	-	-	-
9946	Fixed	Fixed	Fixed	-	-	-
9947	Fixed	Fixed	Fixed	-	-	-
9948	Fixed	Fixed	Fixed	-	-	-
9949	Fixed	Fixed	Fixed	-	-	-
9950	Fixed	Fixed	Fixed	-	-	-
9952	Fixed	Fixed	Fixed	-	-	-
9953	Fixed	Fixed	Fixed	-	-	-
9954	Fixed	Fixed	Fixed	-	-	-
9955	Fixed	Fixed	Fixed	-	-	-
9956	Fixed	Fixed	Fixed	-	-	-
9957	Fixed	Fixed	Fixed	-	-	-
9958	Fixed	Fixed	Fixed	-	-	-
9959	Fixed	Fixed	Fixed	-	-	-
9960	Fixed	Fixed	Fixed	-	-	-
9961	Fixed	Fixed	Fixed	-	-	-
9962	Fixed	Fixed	Fixed	-	-	-
9964	Fixed	Fixed	Fixed	-	-	-
9965	Fixed	Fixed	Fixed	-	-	-
9966	Fixed	Fixed	Fixed	-	-	-

Supports Cont...

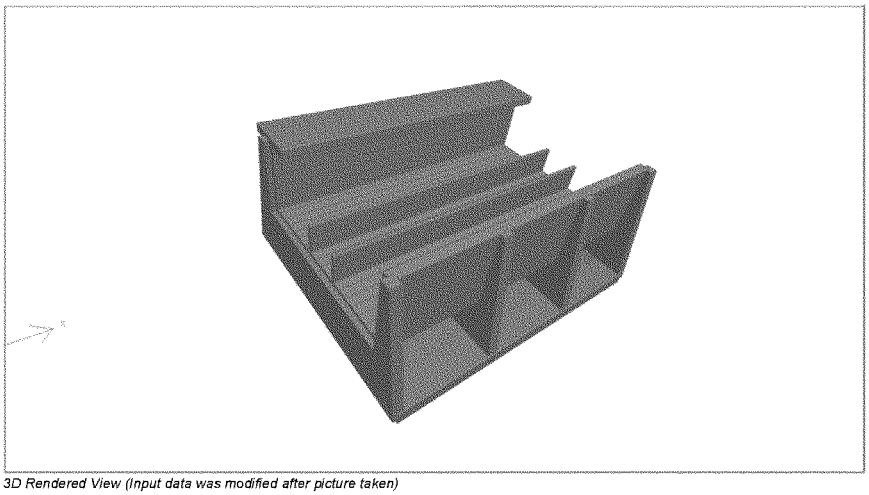
Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip˚ft/deg)	rY (kip˚ft/deg)	rZ (kip˚ft/deg)
9967	Fixed	Fixed	Fixed	-	-	-
9968	Fixed	Fixed	Fixed	-	-	-
9969	Fixed	Fixed	Fixed	-	-	-
9970	Fixed	Fixed	Fixed	-	-	-
9971	Fixed	Fixed	Fixed	-	-	-
9972	Fixed	Fixed	Fixed	-	-	-
9973	Fixed	Fixed	Fixed	-	-	-
9974	Fixed	Fixed	Fixed	-	-	-
9976	Fixed	Fixed	Fixed	-	-	-
9977	Fixed	Fixed	Fixed	-	-	-
9978	Fixed	Fixed	Fixed	-	-	-
9979	Fixed	Fixed	Fixed	-	-	-
9980	Fixed	Fixed	Fixed	-	-	-
9981	Fixed	Fixed	Fixed	-	-	-
9982	Fixed	Fixed	Fixed	-	-	-
9983	Fixed	Fixed	Fixed	-	-	-
9984	Fixed	Fixed	Fixed	-	-	-
9985	Fixed	Fixed	Fixed	-	-	-
9986	Fixed	Fixed	Fixed	-	-	-
9988	Fixed	Fixed	Fixed	-	-	-
9989	Fixed	Fixed	Fixed	-	-	-
9990	Fixed	Fixed	Fixed	-	-	-
9991	Fixed	Fixed	Fixed	-	-	-
9992	Fixed	Fixed	Fixed	-	-	-
9993	Fixed	Fixed	Fixed	-	-	-
9994	Fixed	Fixed	Fixed	-	-	-
9995	Fixed	Fixed	Fixed	-	-	-
9996	Fixed	Fixed	Fixed	-	-	-
9997	Fixed	Fixed	Fixed	-	-	-
9998	Fixed	Fixed	Fixed	-	-	-
10000	Fixed	Fixed	Fixed	-	-	-
10001	Fixed	Fixed	Fixed	-	-	-
10002	Fixed	Fixed	Fixed	-	-	-
10003	Fixed	Fixed	Fixed	-	-	-
10004	Fixed	Fixed	Fixed	-	-	-
10005	Fixed	Fixed	Fixed	-	-	-
10006	Fixed	Fixed	Fixed	-	-	-
10007	Fixed	Fixed	Fixed	-	-	-
10008	Fixed	Fixed	Fixed	-	-	-
10009	Fixed	Fixed	Fixed	-	-	-
10010	Fixed	Fixed	Fixed	-	-	-
11533	Fixed	Fixed	Fixed	-	-	-
12640	Fixed	Fixed	Fixed	-	-	-
13521	Fixed	Fixed	Fixed	-	-	-
13562	Fixed	Fixed	Fixed	-	-	-

Supports Cont...

Node	X (kip/in)	Y (kip/in)	Z (kip/in)	rX (kip*ft/deg)	rY (kip*ft/deg)	rZ (kip*ft/deg)
13915	Fixed	Fixed	Fixed	-	-	-
14085	Fixed	Fixed	Fixed	-	-	-
15692	Fixed	Fixed	Fixed	-	-	-
16287	Fixed	Fixed	Fixed	-	-	-
16500	Fixed	Fixed	Fixed	-	-	-

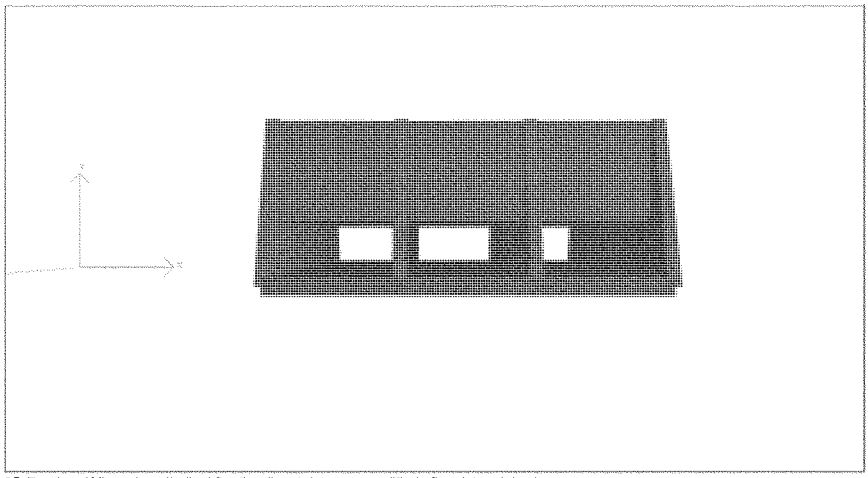
Plate Thickness

Prop	Node A (in)	Node B (in)	Node C (in)	Node D (in)	Material
1	27.000	27.000	27.000	27.000	CONCRETE
2	36.000	36.000	36.000	36.000	CONCRETE
3	12.000	12.000	12.000	12.000	CONCRETE
4	32.000	32.000	32.000	32.000	CONCRETE
5	48.000	48.000	48.000	48.000	CONCRETE

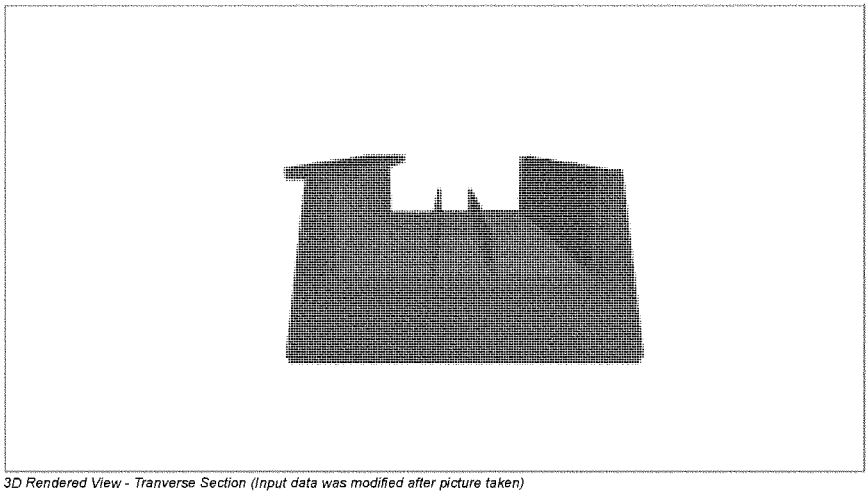


3D Rendered View (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 9	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct F	Date/Time 24-Feb-2011 15:13	



3D Rendered View - Longitudinal Section (Input data was modified after picture taken)



3D Rendered View - Tranverse Section (Input data was modified after picture taken)

Basic Load Cases

Number	Name
3	DEAD LOAD D
4	LIVE LOAD L
5	FLUID LOAD F1 (OVERTOPPING DC FUL
6	FLUID LOAD F2 (OVERTOPPING TC FUL
7	FLUID LOAD F3 (500YR+ICE)
8	SOIL LOAD H
9	FLUID LOAD F4 (LOWFLOW CHANNEL F
24	1.3(1.4D+1.7L)
27	1.3(1.4D+1.7L+1.7F1)
32	1.3(1.4D+1.7L+1.7F2)
33	1.3(1.4D+1.7L+1.7F3)
34	1.3(1.4D+1.7L+1.7F4)
37	1.3(1.4D+1.7L+1.7F1&F2)
42	1.3(1.4D+1.7L+1.7H)
45	1.3(1.4D+1.7L+1.7F1+1.7H)
47	1.3(1.4D+1.7L+1.7F2+1.7H)
50	1.3(1.4D+1.7L+1.7F3+1.7H)
53	1.3(1.4D+1.7L+1.7F4+1.7H)
55	1.3(1.4D+1.7L+1.7 F1&F2+1.7H)

Combination Load Cases

There is no data of this type.

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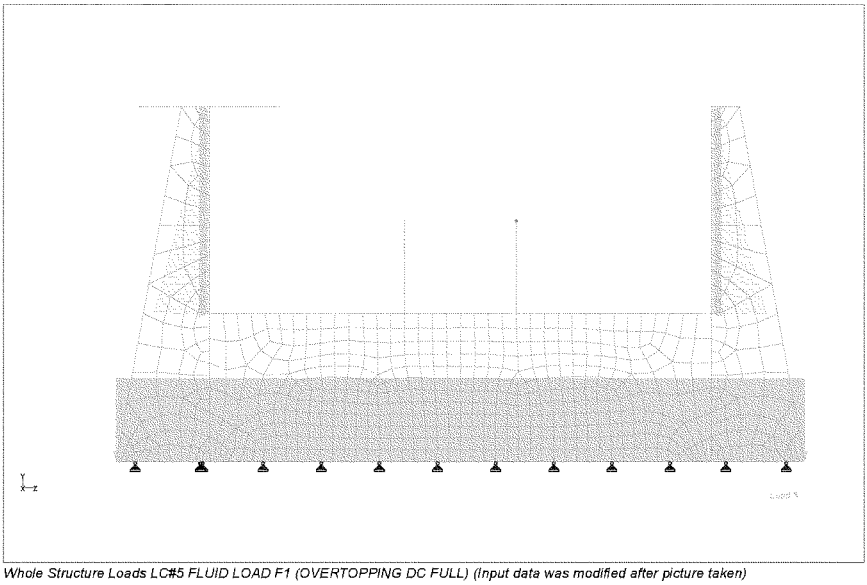
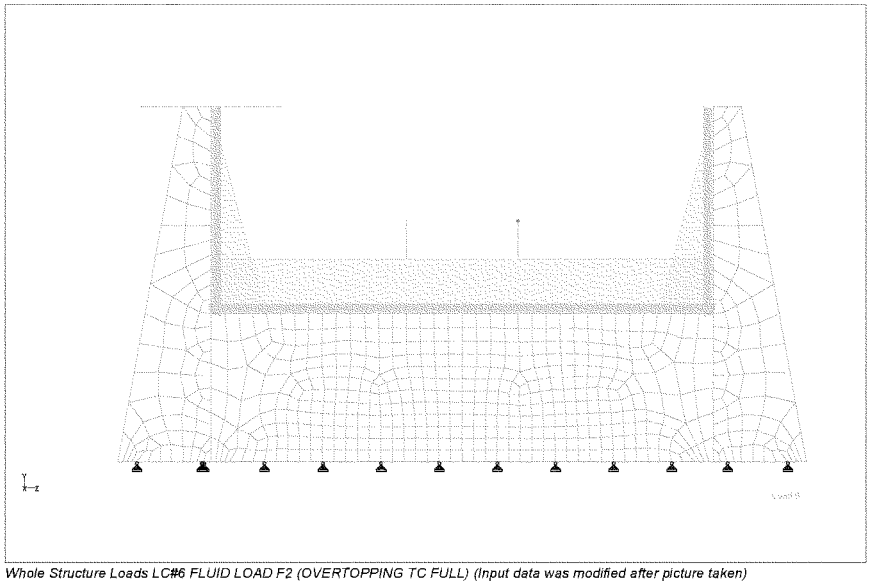
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct F	Date/Time 24-Feb-2011 15:13	



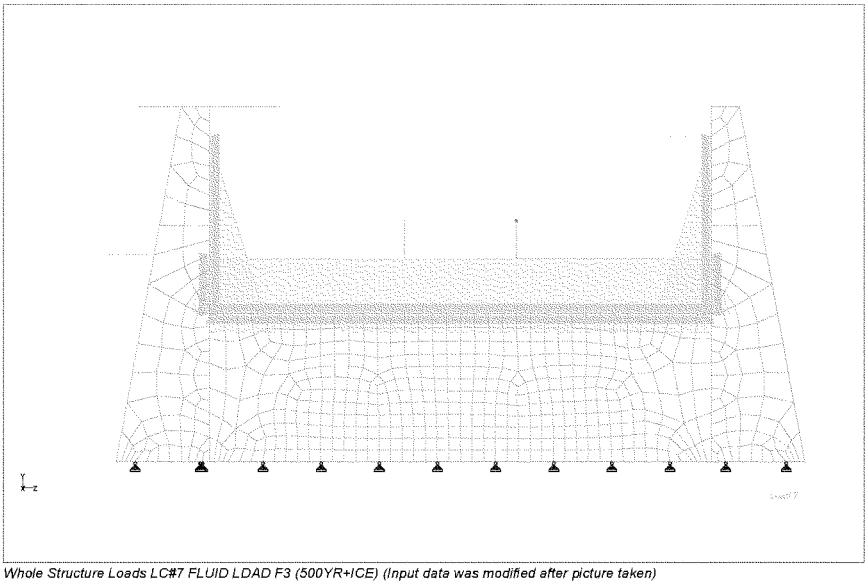
Whole Structure Loads LC#5 FLUID LOAD F1 (OVERTOPPING DC FULL) (Input data was modified after picture taken)

Barr Engineering		Job No 34091009	Sheet No 12	Rev 2
Software licensed to Microsoft	BARR	Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct R	Date/Time 24-Feb-2011 15:13	



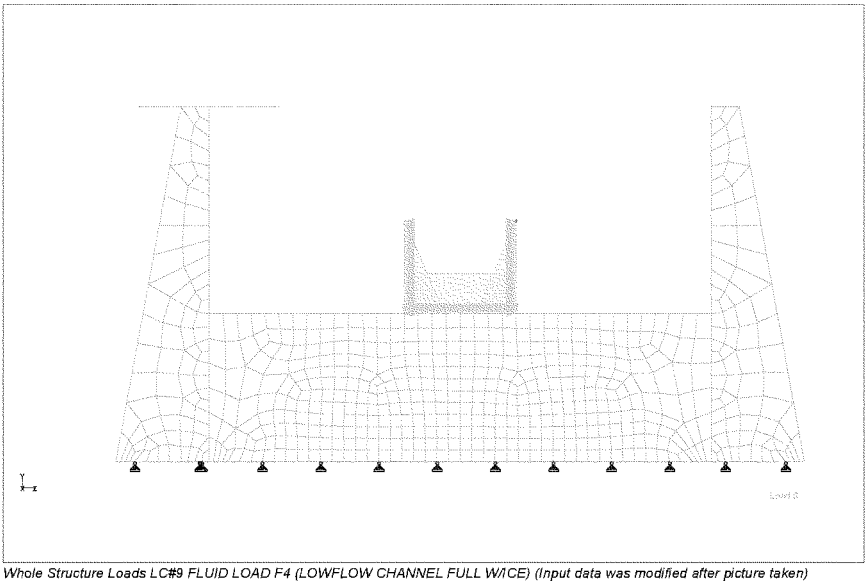
Whole Structure Loads LC#6 FLUID LOAD F2 (OVERTOPPING TC FULL) (Input data was modified after picture taken)

Barr Engineering		Job No 34091009	Sheet No 13	Rev 2
Software licensed to Microsoft	BARR	Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct F	Date/Time 24-Feb-2011 15:13	



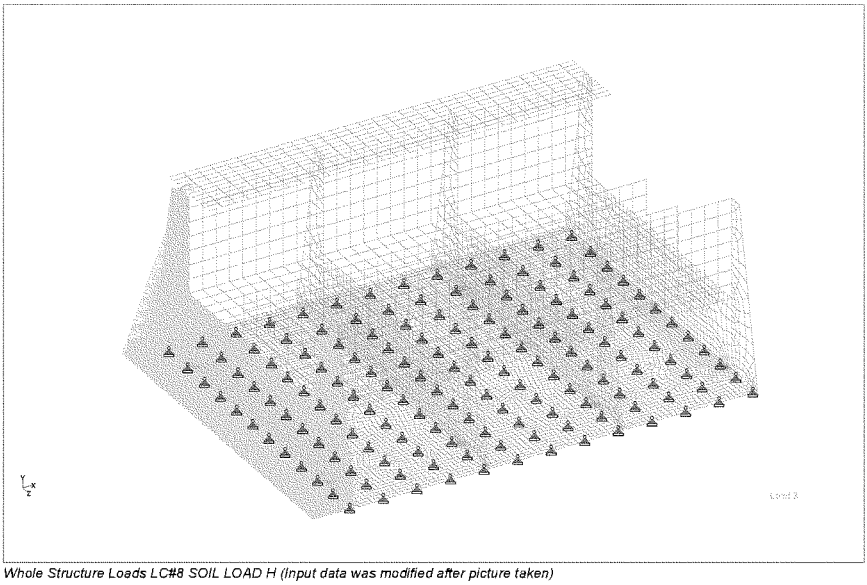
Whole Structure Loads LC#7 FLUID LDAD F3 (500YR+ICE) (Input data was modified after picture taken)

Barr Engineering	Job No 34091009	Sheet No 14	Rev 2
Software licensed to Microsoft	Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4	Ref Phase4		
	By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers	File PH4 Sheyene Aquaduct R	Date/Time 24-Feb-2011 15:13	



Whole Structure Loads LC#9 FLUID LOAD F4 (LOWFLOW CHANNEL FULL W/ICE) (Input data was modified after picture taken)

Barr Engineering		Job No 34091009	Sheet No 15	Rev 2
Software licensed to Microsoft	BARR	Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct F	Date/Time 24-Feb-2011 15:13	



Whole Structure Loads LC#8 SOIL LOAD H (Input data was modified after picture taken)

Node Displacement Summary

	Node	L/C	X (in)	Y (in)	Z (in)	Resultant (in)	rX (rad)	rY (rad)	rZ (rad)
Max X	9370	55:1.3(1.4D+1.	**0.067**	-0.032	-0.008	0.075	-0.000	0.000	-0.000
Min X	9373	37:1.3(1.4D+1.	**-0.031**	-0.032	-0.008	0.045	-0.000	-0.000	0.000
Max Y	9091	27:1.3(1.4D+1.	-0.010	**0.037**	0.225	0.228	0.001	0.000	0.000
Min Y	9074	27:1.3(1.4D+1.	-0.002	**-0.154**	0.253	0.296	0.001	-0.000	0.000
Max Z	9371	53:1.3(1.4D+1.	0.046	-0.011	**1.071**	1.072	0.013	0.001	-0.000
Min Z	9370	53:1.3(1.4D+1.	0.046	-0.012	**-1.074**	1.075	-0.013	-0.001	-0.000
Max rX	9371	53:1.3(1.4D+1.	0.046	-0.011	1.071	1.072	**0.013**	0.001	-0.000
Min rX	9370	53:1.3(1.4D+1.	0.046	-0.012	-1.074	1.075	**-0.013**	-0.001	-0.000
Max rY	9371	53:1.3(1.4D+1.	0.046	-0.011	1.071	1.072	0.013	**0.001**	-0.000
Min rY	9372	34:1.3(1.4D+1.	-0.009	-0.011	1.071	1.071	0.013	**-0.001**	0.000
Max rZ	1498	37:1.3(1.4D+1.	0.001	-0.040	-0.000	0.040	-0.000	-0.000	**0.001**
Min rZ	1470	55:1.3(1.4D+1.	0.037	-0.040	-0.001	0.055	-0.000	0.000	**-0.001**
Max Rst	9370	53:1.3(1.4D+1.	0.046	-0.012	-1.074	**1.075**	-0.013	-0.001	-0.000

Barr Engineering		Job No 34091009	Sheet No 16	Rev 2
Software licensed to Microsoft		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct R	Date/Time 24-Feb-2011 15:13	

Plate Centre Stress Summary

			Shear		Membrane			Bending		
	Plate	L/C	Qx (psi)	Qy (psi)	Sx (psi)	Sy (psi)	Sxy (psi)	Mx (Kip˚ft/ft)	My (Kip˚ft/ft)	Mxy (Kip˚ft/ft)
Max Qx	13085	55:1.3(1.4D+1.	**304.139**	278.927	-45.353	-17.709	-5.714	-60.833	-53.322	-20.557
Min Qx	16314	37:1.3(1.4D+1.	**-345.812**	226.390	-90.256	-13.919	-1.314	-56.291	-27.658	-25.704
Max Qy	14875	37:1.3(1.4D+1.	201.221	**468.462**	-5.563	-34.894	-19.051	-65.071	-77.953	-15.974
Min Qy	13355	55:1.3(1.4D+1.	168.919	**-421.178**	-16.666	-64.264	-16.616	-75.182	-78.306	-7.321
Max Sx	7302	45:1.3(1.4D+1.	-5.043	-1.222	**430.095**	-3.116	-57.900	-16.099	0.011	1.466
Min Sx	9021	37:1.3(1.4D+1.	-2.195	-2.682	**-827.458**	-262.897	-242.399	3.133	0.788	0.630
Max Sy	7301	45:1.3(1.4D+1.	-0.985	0.976	58.983	**443.315**	181.106	-1.542	-18.779	-6.499
Min Sy	8936	32:1.3(1.4D+1.	-0.672	-1.162	44.753	**-939.464**	150.521	0.332	2.034	-0.125
Max Sxy	8933	32:1.3(1.4D+1.	-0.827	-0.049	-654.309	-187.818	**436.748**	1.501	0.504	-0.630
Min Sxy	8686	37:1.3(1.4D+1.	-0.472	-0.399	-55.953	-174.622	**-424.720**	0.132	0.377	0.173
Max Mx	14951	50:1.3(1.4D+1.	85.131	148.824	17.621	-62.184	28.029	**104.339**	-2.101	1.375
Min Mx	8241	55:1.3(1.4D+1.	26.371	21.812	-116.699	-36.040	-58.880	**-111.728**	-13.127	-45.160
Max My	718	55:1.3(1.4D+1.	2.182	116.785	-6.971	-13.460	6.312	29.416	**142.858**	-2.547
Min My	1178	27:1.3(1.4D+1.	32.598	87.277	-63.202	-387.845	128.829	-17.675	**-130.012**	2.753
Max Mxy	15175	55:1.3(1.4D+1.	-25.779	-19.878	1.021	8.958	23.471	43.864	38.920	**56.657**
Min Mxy	7978	55:1.3(1.4D+1.	5.315	30.499	-25.811	-101.556	-27.764	-13.327	-119.389	**-54.827**

Plate Centre Principal Stress Summary

			Principal		Von Mis		Tresca	
	Plate	L/C	Top (psi)	Bottom (psi)	Top (psi)	Bottom (psi)	Top (psi)	Bottom (psi)
Max (t)	8666	53:1.3(1.4D+1.	**3.48E+3**	531.860	3.25E+3	3.24E+3	3.48E+3	3.51E+3
Max (b)	9085	55:1.3(1.4D+1.	1.14E+3	**-1.14E+3**	1.14E+3	529.589	1.15E+3	594.623
Max VM (t)	8666	53:1.3(1.4D+1.	3.48E+3	531.860	**3.25E+3**	3.24E+3	3.48E+3	3.51E+3
Max VM (b)	8665	53:1.3(1.4D+1.	3.33E+3	364.839	3.16E+3	**3.56E+3**	3.33E+3	3.78E+3
Tresca (t)	8666	53:1.3(1.4D+1.	3.48E+3	531.860	3.25E+3	3.24E+3	**3.48E+3**	3.51E+3
Tresca (b)	8665	53:1.3(1.4D+1.	3.33E+3	364.839	3.16E+3	3.56E+3	3.33E+3	**3.78E+3**

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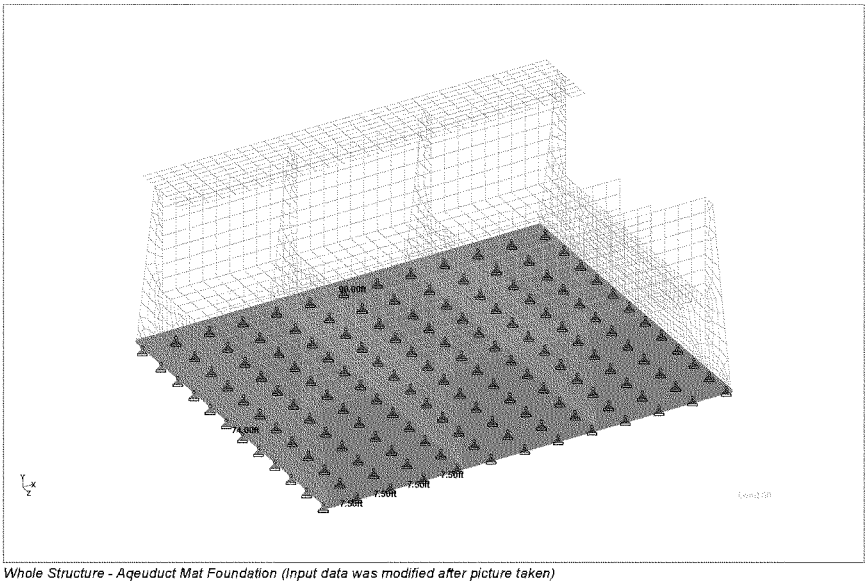
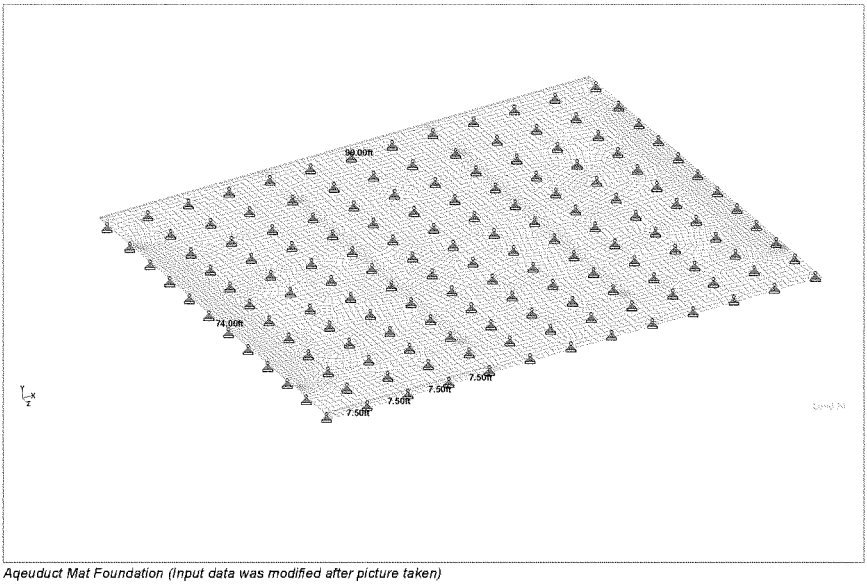
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct F	Date/Time 24-Feb-2011 15:13	



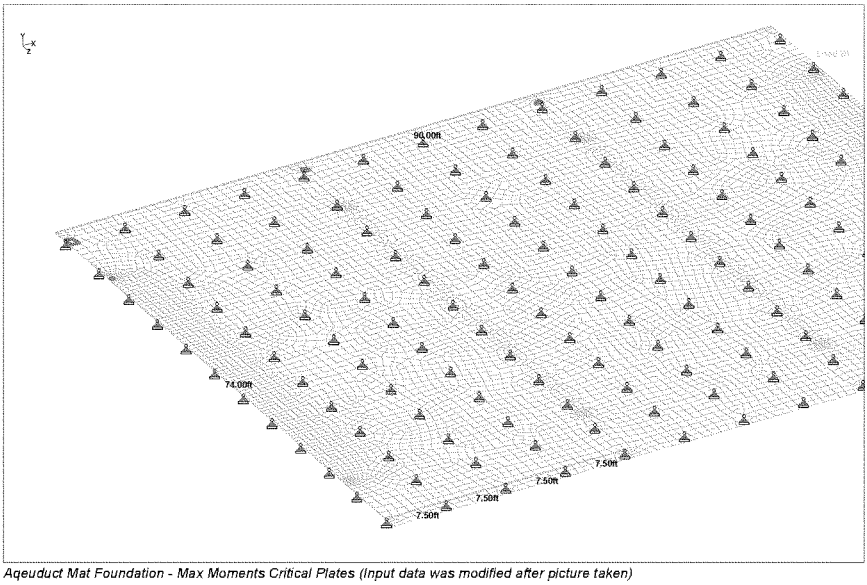
Whole Structure - Aqeuduct Mat Foundation (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 18	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct R	Date/Time	24-Feb-2011 15:13



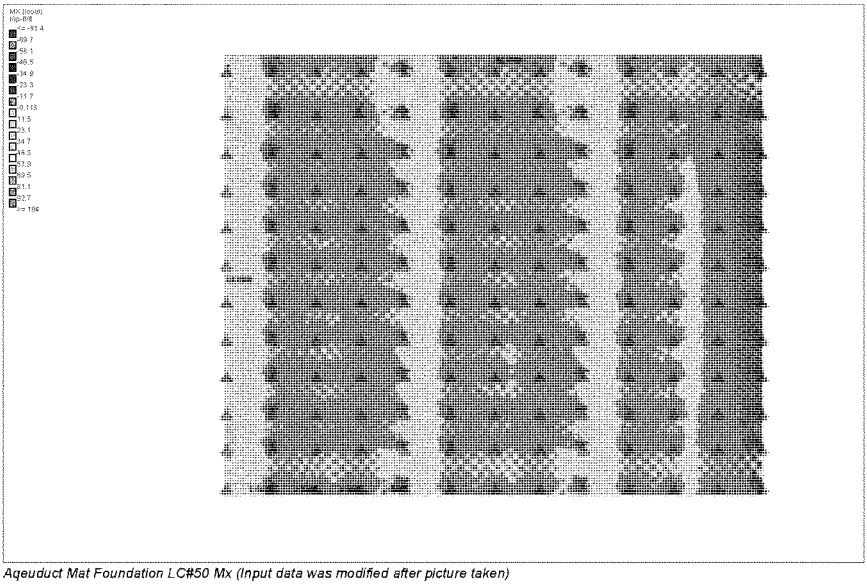
Aqeuduct Mat Foundation (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft	BARR	Job No 34091009	Sheet No 19	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct F	Date/Time 24-Feb-2011 15:13	



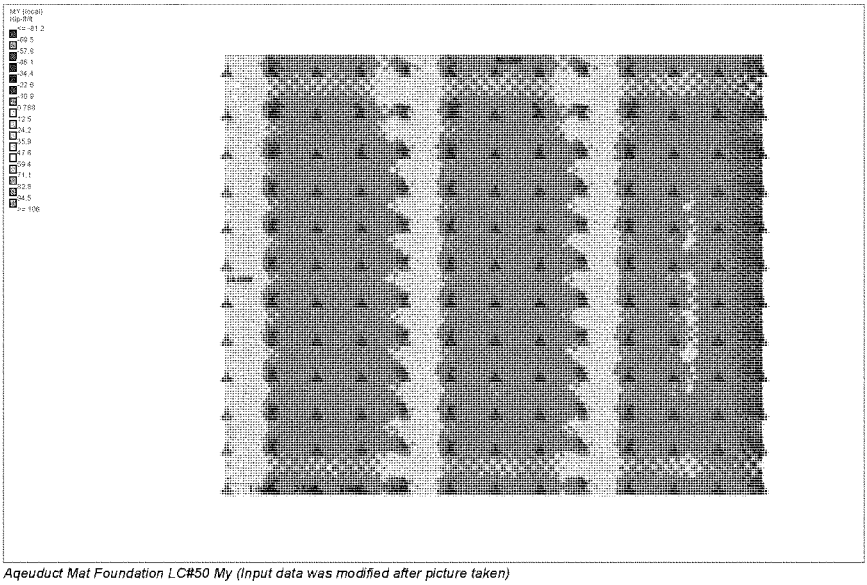
Aqeuduct Mat Foundation - Max Moments Critical Plates (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 20	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct R	Date/Time 24-Feb-2011 15:13	



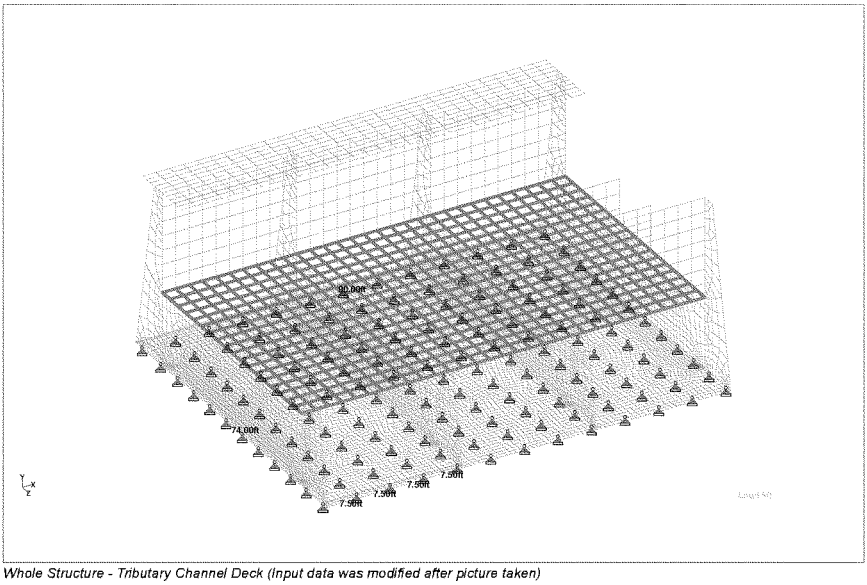
Aqeuduct Mat Foundation LC#50 Mx (Input data was modified after picture taken)

Barr Engineering		Job No 34091009	Sheet No 21	Rev 2
Software licensed to Microsoft	BARR	Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct F	Date/Time 24-Feb-2011 15:13	



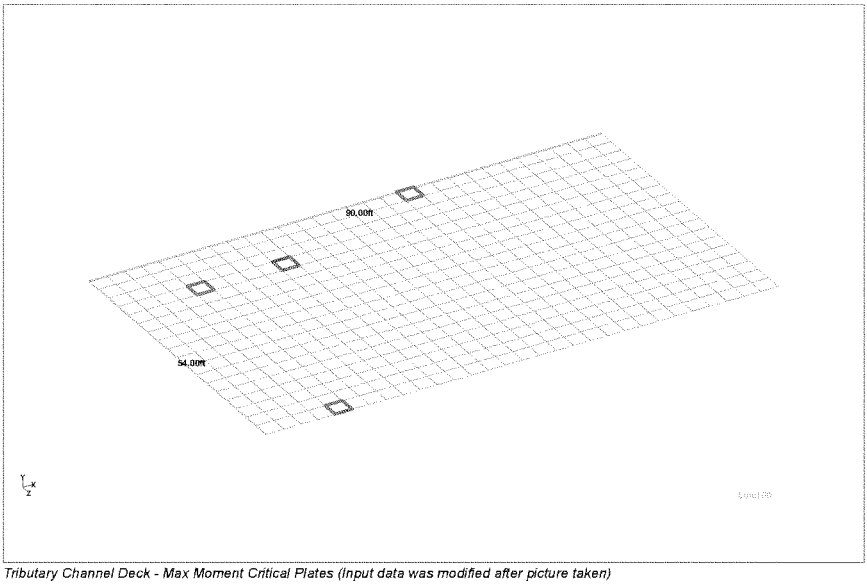
Aqeuduct Mat Foundation LC#50 My (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 22	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct R	Date/Time 24-Feb-2011 15:13	



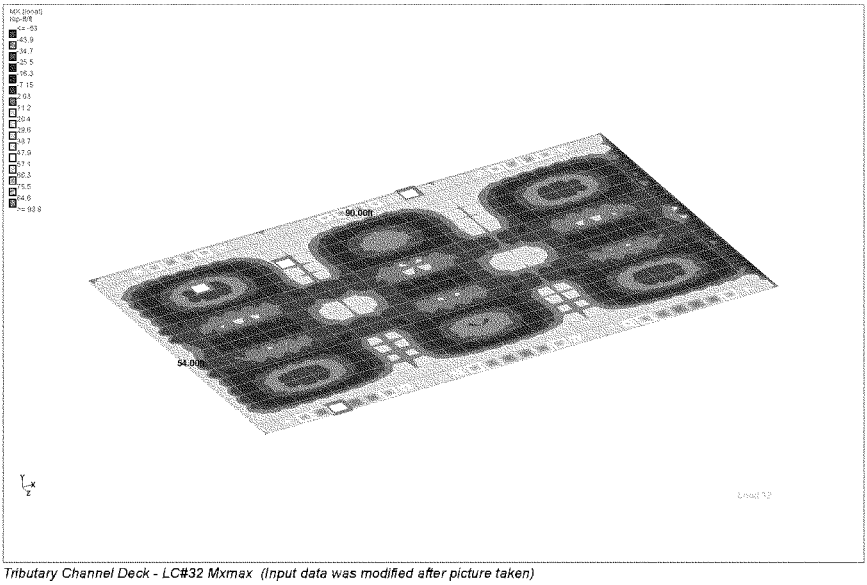
Whole Structure - Tributary Channel Deck (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 23	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct F	Date/Time	24-Feb-2011 15:13



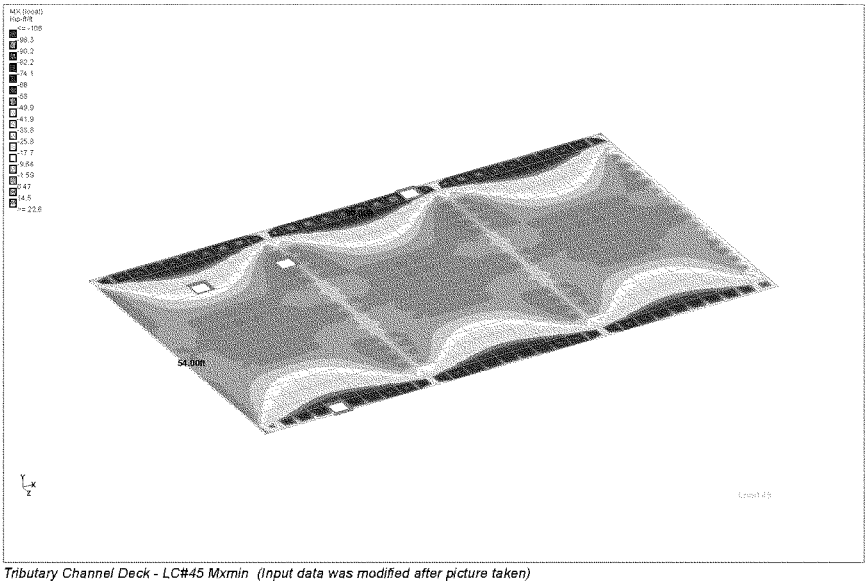
Tributary Channel Deck - Max Moment Critical Plates (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 24	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct R	Date/Time 24-Feb-2011 15:13	



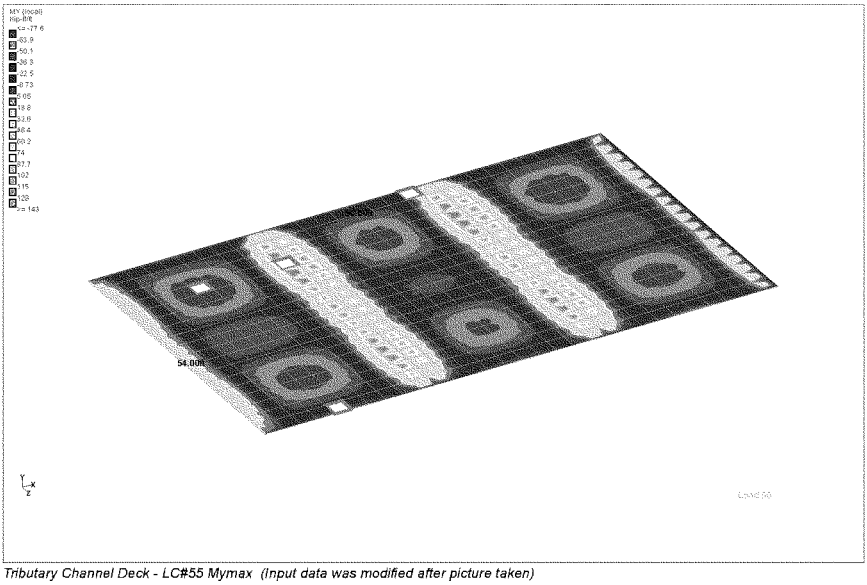
Tributary Channel Deck - LC#32 Mxmax (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 25	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct F	Date/Time	24-Feb-2011 15:13



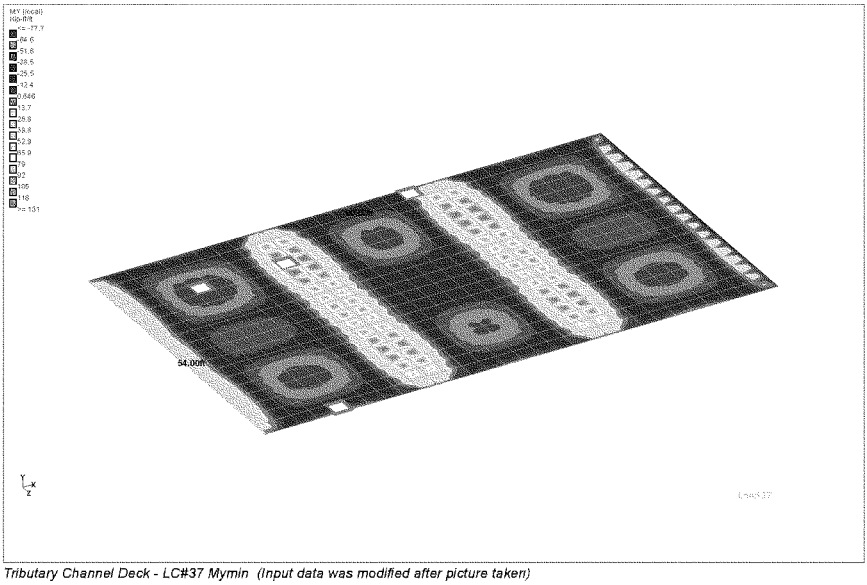
Tributary Channel Deck - LC#45 Mxmin (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 26	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct R	Date/Time	24-Feb-2011 15:13



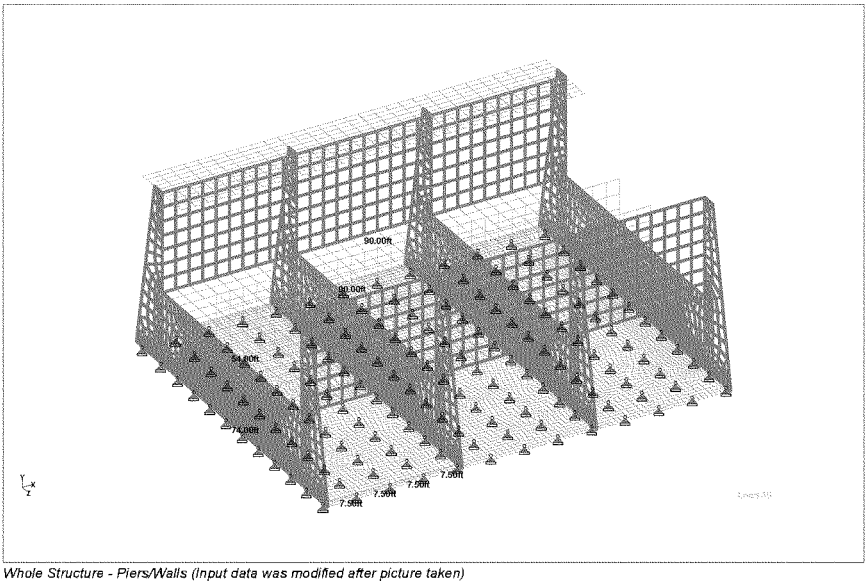
Tributary Channel Deck - LC#55 Mymax (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 27	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct F	Date/Time	24-Feb-2011 15:13



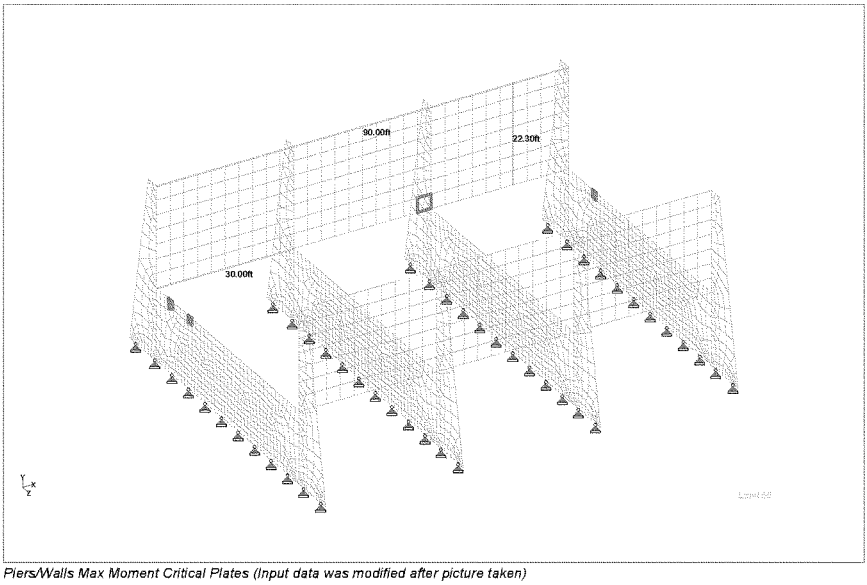
Tributary Channel Deck - LC#37 Mymin (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 28	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct R	Date/Time 24-Feb-2011 15:13	



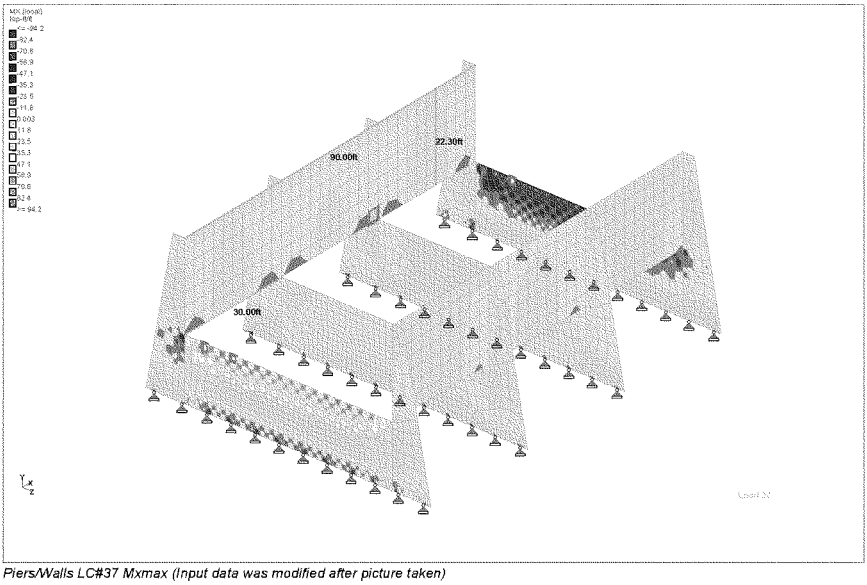
Whole Structure - Piers/Walls (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 29	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct F	Date/Time 24-Feb-2011 15:13	



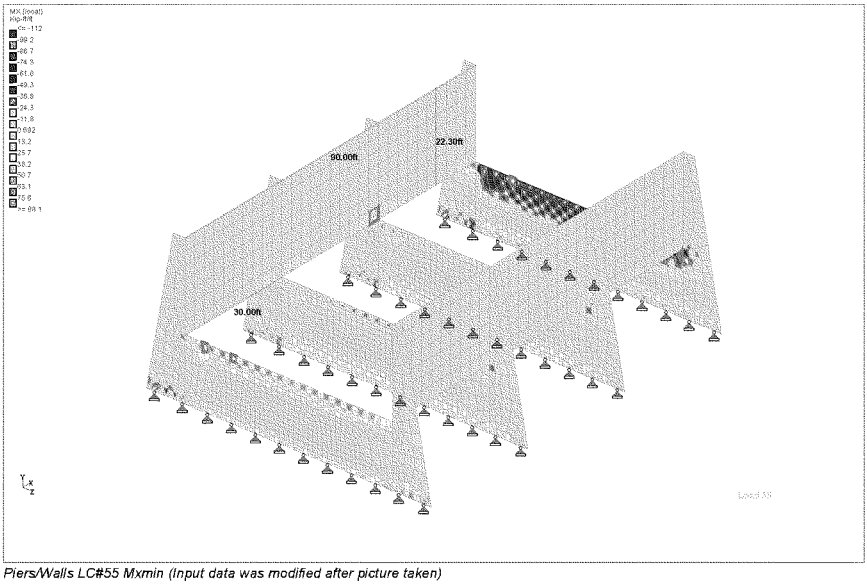
Piers/Walls Max Moment Critical Plates (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 30	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct R	Date/Time 24-Feb-2011 15:13	



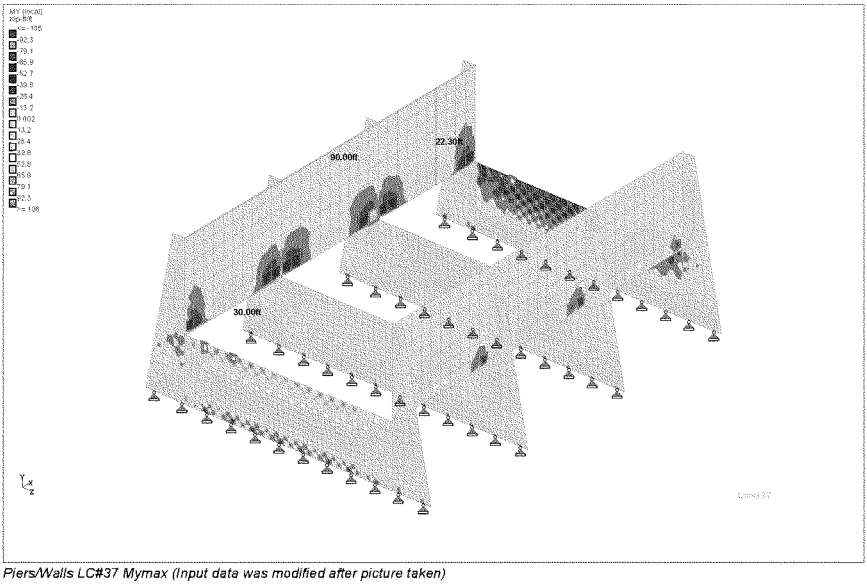
Piers/Walls LC#37 Mxmax (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 31	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct F	Date/Time 24-Feb-2011 15:13	



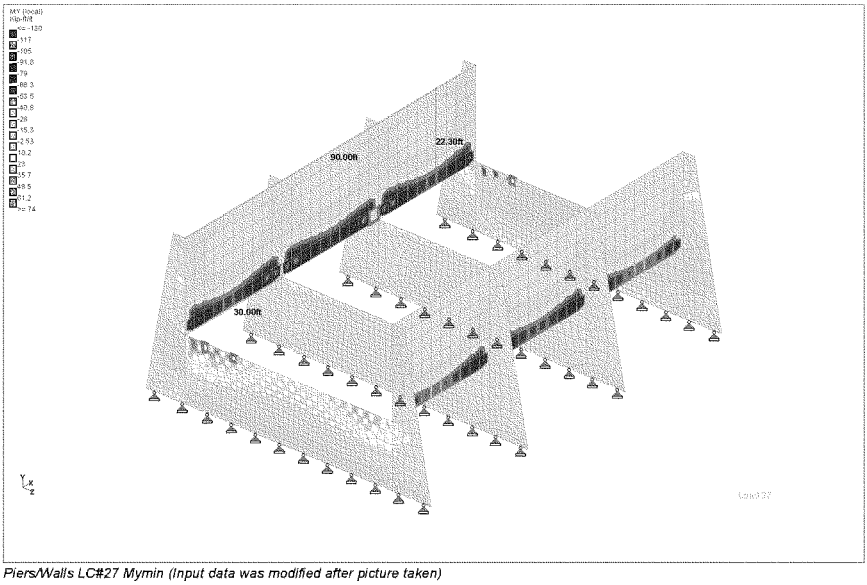
Piers/Walls LC#55 Mxmin (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 32	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct R	Date/Time 24-Feb-2011 15:13	



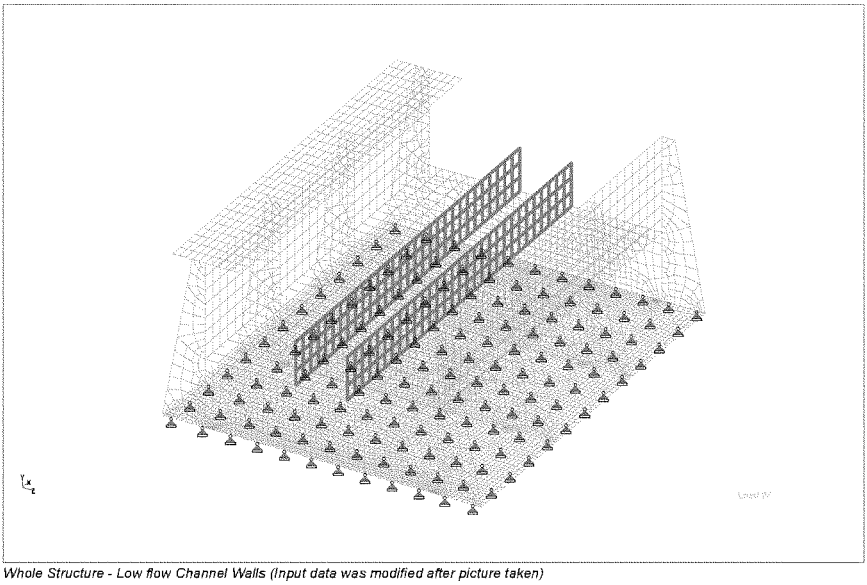
Piers/Walls LC#37 Mymax (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 33	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct F	Date/Time	24-Feb-2011 15:13



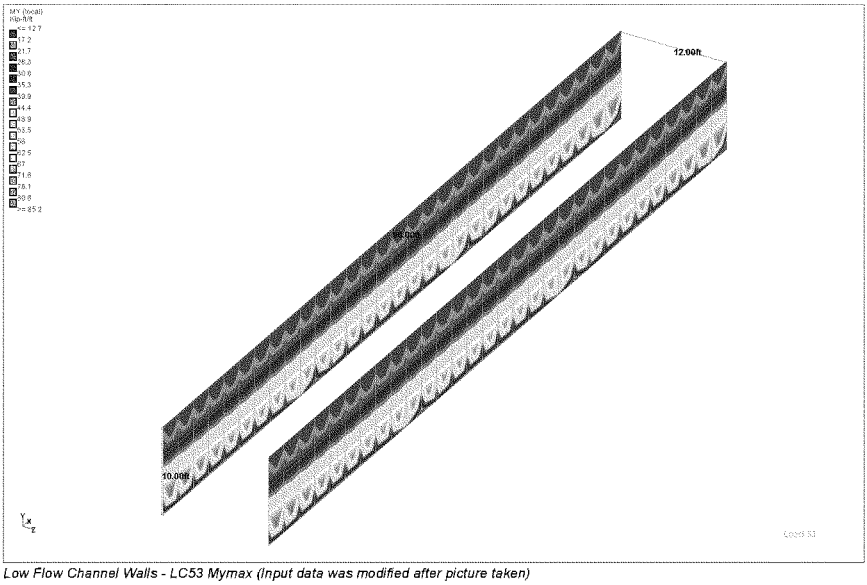
Piers/Walls LC#27 Mymin (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 34	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct R	Date/Time 24-Feb-2011 15:13	



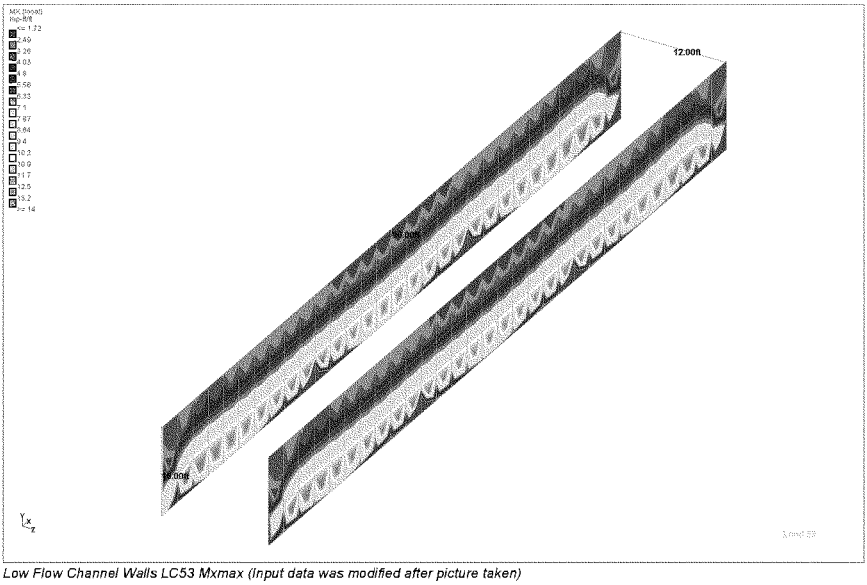
Whole Structure - Low flow Channel Walls (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 35	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct F	Date/Time 24-Feb-2011 15:13	



Low Flow Channel Walls - LC53 Mymax (Input data was modified after picture taken)

Barr Engineering Software licensed to Microsoft		Job No 34091009	Sheet No 36	Rev 2
		Part Sheyene Aquaduct - Partial		
Job Title Fargo- Moorhead Feasibility Study - Phase4		Ref Phase4		
		By mbi	Date 1-24-11	Chd
Client US Army Corps of Engineers		File PH4 Sheyene Aquaduct R	Date/Time 24-Feb-2011 15:13	



Low Flow Channel Walls LC53 Mxmax (Input data was modified after picture taken)

ATTACHMENT

F-R4.2 Sheyenne River Aqueduct Structure Pile Computations

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name:	U.S. ARMY CORPS OF ENGINEERS	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
Work Describtion:	LOAD CASES - SHEYENNE AQUEDUCT - Phase 4	1/24/2011
		34091004
File Path:	P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Sheyene\[34091004 PH4 Sheyene Pile Calcs.xlsx]Load Cases	

REF. 1
2

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tributary - Water El. (ft)	914.56	914.56	914.67	916	903.24	NA
Diversion - Head Water El. (ft)	902.12	902.12	903.32	916	NA	NA
Diversion - Tail Water El. (ft)	901.91	901.91	903.06	916	NA	NA
Tributary - T.O. Wall El. (ft)	916					
Tributary - T.O. Deck L.P. El.(ft)	898.7					
Tributary - T.O. Deck H.P. El.(ft)	900.7					
Diversion - T.O. Mat El. (ft)	883.68					
Tributary - Deck Slab thickness @ L.P. (ft)	2					
Tributary - Deck Slab thickness @ H.P. (ft)	4					
Diversion - Mat Slab thickness (ft)	4					
Tibutary - Water height (ft)	15.86	15.86	15.97	17.3	4.54	NA
Diversion - Head Water height (ft)	18.44	18.44	19.64	32.32	NA	NA
Ice	NA	2ft Ice	NA	NA	2ft Ice	NA
Ice Load	NA	10 kips/ft	NA	NA	10 kips/ft	NA
Ice Laod El. (ft)	NA	914.56	NA	NA	903.24	NA
Uplift @ HW (ft)	22.44	22.44	23.64	36.32	NA	NA
Uplift @ TW (ft)	22.23	22.23	23.38	36.32	NA	NA
Pile Condition	Undrained	Undrained	Undrained	Undrained	Drained	Undrained
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Safety Factors	2	1.5	1.5	1.15	2	1.5
Allwable Lateral Capacity (tons)	18	20.5	20.5	24	11.5	20.5
Allowable Pile Capacity (tons) - Axial	61.95	82.60	82.60	107.74	36.525	82.60
Allowable Pile Capacity (tons) - Uplift	38.65	51.53	51.53	67.22	5.9	51.53

Pile Capacity	Ultimate Axial Capacity (kips)	Allowable Lateral Capacity (kips)		
		0.5" (Usual)	0.67" (Unusual)	0.875" (Extreme)
Undrained - Axial	247.8	36	41	48
Undrained - Uplift	154.6			
Drained - Axial	146.1	23	29	36
Drained - Uplift	23.6			

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name:	U.S. ARMY CORPS OF ENGINEERS	Design By:	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	Review By:	
		Date:	1/24/2011
Work Describtion:	LOAD CASES - SHEYENNE AQUEDUCT - Phase 4	Job #:	34091004'
File Path:	P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Sheyene\[34091004 PH4 Sheyene Pile Calcs.xlsx]Load Cases		

REF. 1
2

Hydrolic Profile

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tributary - Water El. (ft)	914.56	914.56	914.67	916	903.24	NA
Diversion - Head Water El. (ft)	902.12	902.12	903.32	916	NA	NA
Diversion - Tail Water El. (ft)	901.91	901.91	903.06	916	NA	NA
Tributary - T.O. Wall El. (ft)	916					
Tributary - T.O. Deck L.P. El.(ft)	898.7					
Tributary - T.O. Deck H.P. El.(ft)	900.7					
Diversion - T.O. Mat El. (ft)	883.68					
Tributary - Deck Slab thickness @ L.P. (ft)	2					
Tributary - Deck Slab thickness @ H.P. (ft)	4					
Diversion - Mat Slab thickness (ft)	4					
Tibutary - Water height (ft)	15.86	15.86	15.97	17.3	4.54	NA
Diversion - Head Water height (ft)	18.44	18.44	19.64	32.32	NA	NA
Ice	NA	2ft Ice	NA	NA	2ft Ice	NA
Ice Load	NA	10 kips/ft	NA	NA	10 kips/ft	NA
Ice Laod El. (ft)	NA	914.56	NA	NA	903.24	NA
Uplift @ HW (ft)	22.44	22.44	23.64	36.32	NA	NA
Uplift @ TW (ft)	22.23	22.23	23.38	36.32	NA	NA

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name:	U.S. ARMY CORPS OF ENGINEERS	Design By:	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	Review By:	
		Date:	1/24/2011
Work Description:	LOAD CASES - SHEYENNE AQUEDUCT - Phase 4	Job #:	34091004
File Path:	P:\MPLS\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\270 Structural\Aqueducts\Sheyenne\[34091004 PH4 Sheyenne Pile Calcs.xlsx]Load Cases		

Quantity Take Off (Revit)				Material Properties		
Volume of Walls (ft3)	Vw (ft3)	51100	ft3	*Concrete*	γ Concrete (pcf)	150
Volume Tributary Deck Slab (ft3)	Vs (ft3)	41022	ft3	*Steel*	γ Steel (pcf)	495
Volume of Bridge Deck (ft3)	Vs (ft3)	8127	ft3	*Soil Dry*	γs Dry (pcf)	120
Volume Diversion Mat Slab (ft3)	Vs (ft3)	81744	ft3	*Soil Saturated*	γs Sat. (pcf)	130
Total		**181993**	ft3	*Water*	γ Water (pcf)	62.4

Geometry

Tributary Channel			
Tributary - T.O. Wall El. (ft)		9[illegible]	ft
Tributary - T.O. Deck L.P. El.(ft)		[illegible]7	ft
Tributary - T.O. Deck H.P. El.(ft)		[illegible]7	ft
Tributary - Clear Width (ft)	w TC	[illegible]	ft
Tributary - Wall Thickness (ft)	twall TC	[illegible]	ft
Tributary - Deck Slope Width (ft)	lslab slope TC	[illegible]	ft
Tributary - Deck Slab thickness @ L.P. (ft)	tslab TC	1	ft
Tributary - Deck Slab thickness @ H.P. (ft)	tslab TC	4	ft
Tributary - Low Flow Channel height (ft)	hlowflow TC	[illegible]	ft
Tributary - Low Flow Channel width (ft)	wlowflow TC	[illegible]	ft
Tributary - Low Flow Channel thickness (ft)	tlowwall TC	[illegible]	ft
Tributary - Wall Height (ft)	hwall TC	13.3	ft
Diversion Channel			
Diversion - T.O. Wall El. (ft)		[illegible]7	ft
Diversion - T.O. Mat El. (ft)		[illegible]	ft
Diversion - Clear Opening Width	wopen DC	[illegible]	ft
Diversion - # of Openings	#open DC	[illegible]	
Diversion - Wall Thickness (ft)	twall DC	[illegible]	ft
Diversion - Mat Slab thickness (ft)	tslab DC	4	ft
Diversion - Butress height (ft)	hbutress	[illegible]	ft
Diversion - Butress Top width (ft)	wbutress Top	[illegible]	ft
Diversion - Butress Top width (ft)	wbutress Bot	[illegible]	ft
Diversion - Wall Height (ft)	hwall DC	[illegible]	ft
Mat Foundation			
Overall Width (ft)	wmat	**78**	ft
Overall Length (ft)	lmat	**262**	ft
Tributary - Channel Length (ft)	lslab TC	**258**	ft
Tributary - Channel Width (ft)	wslab TC	**56**	ft
Access Bridge			
Overall Width (ft)	wbridge	15	ft
Overall Length (ft)	lbridge	**258**	ft
Minimum Deck Thickness	tbridge	1.5	ft
Maximum Deck Thickness	tbridge	3	ft

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name:	U.S. ARMY CORPS OF ENGINEERS	Design By:	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	Review By:	
		Date:	1/24/2011
Work Description:	LOAD CASES - SHEYENNE AQUEDUCT - Phase 4	Job #:	34091004
File Path:	P:\Mpls\34 NT\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Sheyenne\[34091004 PH4 Sheyenne File Cases.xlsx]Load Cases		

Weight of Structure

Tibutary	Volume (ft3)	Weight (tons)
Walls	23684	1776
Deck	41022	3077
Low Flow Chanel	5160	387
Bridge	8127	860
Sub Total	77993	6100

Diversion	Volume (ft3)	Weight (tons)
Walls	16770	1258
Mat	81744	6131
Butress Walls	5333	400
Sub Total	103847	7788
Total	181840	13638

Whole Structure	Volume (ft3)	Weight (tons)
Walls	50947	3821
Deck	41022	3077
Bridge	8127	860
Mat	81744	6131
Total	181840	13638

Take off (Revit)	Volume (ft3)	Weight (tons)
Walls	51100	3833
Deck	41022	3077
Bridge	8127	610
Mat	81744	6131
Total	181993	13649

Ratio 0.999159089

Forces

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tibutary - Water height (ft)	15.86	15.86	15.97	17.30	4.54	NA
Triburtary - Channel Length (ft)	258.00					
Tributary - Clear Width (ft)	50.00					
Tributary - Water force (psf)	989.66	989.66	996.53	1079.52	283.30	NA
Tributary - Water Volume (ft3)	204594.00	204594.00	206013.00	223170.00	11713.20	NA
Tributary - Water Weight (tons)	6383.33	6383.33	6427.61	6962.90	365.45	NA
Diversion - Head Water height (ft)	18.44	18.44	19.64	32.32	NA	NA
Mat Foundation - Overall Width (ft)	78.00					
Mat Foundation - Clear Length (ft)	162.00					
Diversion - Water force (psf)	1150.66	1150.66	1225.54	2016.77	NA	NA
Diversion - Water Volume (ft3)	233007.84	233007.84	248171.04	408395.52	NA	NA
Diversion - Water Weight (tons)	7269.84	7269.84	7742.94	12741.94	NA	NA
Total Water Weight on the Structure (tons)	13653.18	13653.18	14170.54	19704.84	365.45	NA
Tributary - Uplift on the Deck (ft)	5.42	5.42	6.62	19.30	NA	NA
Tributary - Uplift force (psf)	338.21	338.21	413.09	1204.32	NA	NA
Tributary - Uplift force (tons)	-2443.21	-2443.21	-2984.15	-8700.01	NA	NA
Uplift @ HW (ft)	22.44	22.44	23.64	36.32	NA	NA
Uplift @ TW (ft)	22.23	22.23	23.38	36.32	NA	NA
Diversion - Uplift force on the Mat (psf)	1393.70	1393.70	1467.02	2266.37	NA	NA
Diversion - Uplift force on the Mat (tons)	-14240.87	-14240.87	-14990.05	-23157.75	NA	NA
Total Uplif Force on the Structure (tons)	-16684.08	-16684.08	-17974.20	-31857.76	NA	NA
Weight of Structure (tons)	13649.5					

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name:	U.S. ARMY CORPS OF ENGINEERS	Design By:	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	Review By:	
		Date:	1/24/2011
Work Describtion:	LOAD CASES - SHEYENNE AQUEDUCT - Phase 4	Job #:	34091004
File Path:	P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Sheyene\[34091004 PH4 Sheyene Pile Calcs.xlsx]Load Cases		

Flotation						
ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Nome	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Downward force on the Structure (tons)	10618.57	10618.57	9845.82	1496.56	14014.93	13649.48
Uplift Ratio	1.64	1.64	1.55	1.05	NA	NA
Uplift Ratio (No water in the Tributary)	1.25	1.25	1.19	0.83	NA	NA
Condition	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Sofety Factors - Flotation	1.30	1.20	1.20	1.10	1.30	1.20
Check	OK	OK	OK	NG!!!	OK	OK

Pile Computation						
ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Pile Condition	Undrained	Undrained	Undrained	Undrained	Drained	Undrained
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Sofety Factors	2	1.5	1.5	1.15	2	1.5
Allwable Lateral Capacity (tons)	18	20.5	20.5	24	11.5	20.5
Allowable Pile Capacity (tons) - Axial	61.95	82.60	82.60	107.74	36.53	82.60
# of Piles Required	171.41	128.55	119.20	13.89	383.71	165.25
Uniform Spocing	10.92	12.61	13.09	38.36	7.30	11.12
# of Columns (along length)	35.00	35.00	35.00	35.00	35.00	35.00
Pile Spacing (along length)	7.53	7.53	7.53	7.53	7.53	7.53
# of Rows (olong width)	4.90	3.67	3.41	0.40	10.96	4.72
# of Raws (along widht) provided	12.00	12.00	12.00	12.00	12.00	12.00
Actual Pile Spocing (along width)	6.55	6.55	6.55	6.55	6.55	6.55
Totol number of pile provided	420	420	420	420	420	420
Pile Load	25.28	25.28	23.44	3.56	33.37	32.50
Utilization Ratio	0.41	0.31	0.28	0.03	0.91	0.39
Check	OK	OK	OK	OK	OK	OK

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Final # af Piles	420					
Pile Load	25.28	25.28	23.44	3.56	33.37	32.50
Utilizatian Ratia	0.41	0.31	0.28	0.03	0.91	0.39
Check	OK	OK	OK	OK	OK	OK

Fargo-Moorhead Food Control Structures Preliminary Pile Foundation Analyses HP 14X73											
A_{cap} = 198.5 in², A_{steel} = 21.4 in², perimeter = 56.4 in, width (b) = 14.6 in, I = 729 in⁴											
Structure	Diversion Channel Station Location	Approximate Ground (Bank) Surface Elevation (ft)	Invert Elevation (ft)	Estimated Foundation Elevation (ft)	Estimated Ground Water Elevation (ft)	Design Condition/Tip Elevations		Ultimate Axial Capacity (kips)	Allowable Lateral Capacity (kips) (fixed head - single pile)			Estimated Settlement at allowable load
									0.5"	0.67"	0.875"	
Sheyenne River Aqueduct Crossing Foundations	1493+98	916	~~881.8~~ 883.68	~~877.83~~ 879.68	908	Undrained Analysis 842.8'	Total	247.8	36	41	48	<0.5"
							Uplift Resistance	154.6				
						Drained Analysis 842.8'	Total	146.1	23	29	36	
							Uplift Resistance	23.5				

See following link for details

P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\060 Geotech\Deep Foundations

ATTACHMENT

F-R4.3 Sheyenne River Aqueduct Structure Retaining Wall Panel E Computations

SHEYENE AQUADUCT STRUCTURE

Client Name:	U.S. ARMY CORPS OF ENGINEERS	Design By:	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	Review By:	
Work Describtion:	Sheyenne Aquaduct Structure - Retaining Walls	Date:	2/10/2011
	Panel E	Job #:	34091004
File Path:	P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Sheyene\[34091004 PH4 Sheyene Retaining Walls Panel E.xlsx]Load Cases		
REF.	1		
	2		

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tributary - Water El. (ft)	914.56	914.56	914.67	917.5	903.24	NA
Diversion - Head Water El. (ft)	902.12	902.12	903.32	917.5	NA	NA
Diversion - Tail Water El. (ft)	901.91	901.91	903.06	917.5	NA	NA
Tributary - T.O. Wall El. (ft)	917.5					
Tributary - T.O. Deck L.P. El.(ft)	898.7					
Tributary - T.O. Deck H.P. El.(ft)	900.7					
Diversion - T.O. Mat El. (ft)	883.68					
Tributary - Deck Slab thickness @ L.P. (ft)	2					
Tributary - Deck Slab thickness @ H.P. (ft)	4					
Diversion - Mat Slab thickness (ft)	4					
Tibutary - Water height (ft)	15.86	15.86	15.97	18.8	4.54	NA
Diversion - Head Water height (ft)	18.44	18.44	19.64	33.82	NA	NA
Ice	NA	2ft Ice	NA	NA	2ft Ice	NA
Ice Load	NA	10 kips/ft	NA	NA	10 kips/ft	NA
Ice Laod El. (ft)	NA	914.56	NA	NA	903.24	NA
Uplift @ HW (ft)	22.44	22.44	23.64	37.82	NA	NA
Uplift @ TW (ft)	22.23	22.23	23.38	37.82	NA	NA
Pile Condition	Undrained	Undrained	Undrained	Undrained	Drained	Undrained
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Safety Factors	2	1.5	1.5	1.15	2	1.5
Allwable Lateral Capacity (tons)	18	20.5	20.5	24	11.5	20.5
Allowable Pile Capacity (tons) - Axial	61.95	82.60	82.60	107.74	36.525	82.60
Allowable Pile Capacity (tons) - Uplift	38.65	51.53	51.53	67.22	5.9	51.53

Pile Capacity	Ultimate Axial Capacity (kips)	Allowable Lateral Capacity (kips)		
		0.5" (Usual)	0.67" (Unusual)	0.875" (Extreme)
Undrained - Axial	247.8	36	41	48
Undroined - Uplift	154.6			
Drained - Axial	146.1	23	29	36
Drained - Uplift	23.6			

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.	
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4		
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004		
MBI		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls		
2/11/11				Panel E		

Monolith Structure

ITEM		UNIT	QUANITY	UNIT COST	TOTAL Cost
FURNISH HP14x73 WALL PILING		LF	4,370	0	$0
INSTALL HP14x73 WALL PILING		LF	4,370	0	$0
PILE TEST, 48.0 ft	Long	EA	6	0	$0
FOOTING CONCRETE		CY	618	0	$0
	Forming	SF	1,428		
STEM CONCRETE		CY	701	0	$0
	Forming	SF	9,812		
STEEL REINFORCEMENT		LB	253,760	0	$0
WALL RAILING		LF	140	0	$0
SHEET PILE CUT-OFF WALL		SF	2,800	0	$0
					$0

Structure Length = 140 ft

No. piles = 115 Each

Length = 38 ft

Note: HP14x73 pile used for design, use HP14x73 to allow for corrosion

	LENGTH
(FRONT & Back FACE)	10 FT

Native Soil has low permeability assume cut-off minimal to prevent scour

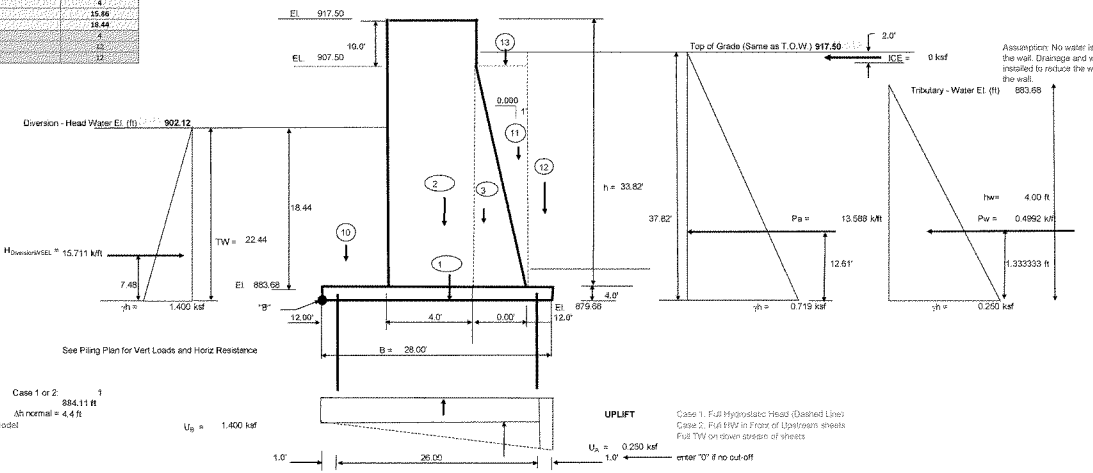
BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls	
				Load Cases: **Case 1 100 yr. flood** **Panel E**	

ID#	Case 1
Name	100 yr. flood
Load Category	Usual
Tributary - Water El. (ft)	914.56
Diversion - Head Water El. (ft)	902.12
Diversion - Tail Water El. (ft)	901.91
Tributary - T.O. Wall El. (ft)	917.5
Tributary - T.O. Deck L.P. El. (ft)	898.7
Tributary - T.O. Deck H.P. El.(ft)	900.7
Diversion - T.O. Mat El. (ft)	883.68
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tributary - Water height (ft)	15.86
Diversion - Head Water height (ft)	18.44
Wall Thickness (ft)	4
Toe (ft)	12
Heel (ft)	12

File:

MN State Building Codes
Frost Depth = 5.0 ft — provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 903.24 ft — DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section — **Length** = 140.0 ft
Stepped Ftg Ls = 2.0 ft — overlap distance at stepped ftg



Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	33.82	0.15	rec	2840.9	14.00	39,772.3
Batter	3	140	0.00	23.82	0.15	tri	0.0	16.00	0.0
					D.L. Concrete	ΣVc =	**5192.9**	ΣM_V =	**72,700.3**

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Stem	10	140	12.00	18.44	0.0624	rec	1933.1	6.00	11,598.6
H.W. on Stem Slope	11	140	0.00	23.82	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	33.82	0.0626	rec	3556.8	22.00	78,249.2
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
					D.L. Water	ΣVw =	**5489.9**	ΣM_V =	**89,847.8**

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	140	28.00	1.400	rec	-5489.0	14.00	-76,846
	U_A	140	28.00	-1.151	tri	2255.3	18.67	42,099
					ΣU =	**-3233.7**	ΣM_U =	**-34,747**

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO - MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34991004	
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls	
				Load Cases: **Case 1 100 yr. flood**	**Panel E**

Horizontal Loads

		L	H	Pressure		ICE	arm	Mu
		ft	ft	ksf		K	ft	ft-k
	ICE	140	2.00	0.00	rec	0.0	36.82	0.0
		L		**Force**		**H**	**arm**	**Mw**
		ft		k/ft		K	ft	ft-k
	SOIL	140		-13.588		-1902.37	12.61	-23982.53
	Water Loads							
H_{tw}		140		15.711	tri	2199.52	7.48	16452.43
H_{tw}		140		-0.499	tri	-69.89	1.33	-93.18
					ΣWater =	**2129.63**	ΣM_W =	**-7623.3**

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{OE}$ =	-42371	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	162548	kip-ft

Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**120,177**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**7,449**	kips
Sum of Horizontal Forces	H = ΣHorizontal	**227**	kips

Location of Resultant: $Xr = \Sigma M / P$ = 16.13 ft from Toe; $e = B/2 - Xr$ = (2.13) ft; B/6 = 4.667 ft

CONCRETE QUANTITIES

			forming	
Ftg conc:	589	cy (includes stepped)	1428	sf
Stem Conc:	701	cy	9812	sf
Total =	1,290			

STEEL REINFORCEMENT: (assumed)

		Bar #	Spacing in	LB /ft	Length ft	# of bars ea	Total wt lb		
	a) Footing								
Top mat	Transverse:	9	6	3.40	27.5	264	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
Bot mat	Transverse:	9	6	3.40	27.5	264	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
							cy		**LB/cy**
							106,991	589	181.6374748
	b) Skin Reinf. On Monolith								
Vert Face	Vertical	9	6	3.40	33.32	280	31,721	63,441.28	
	Longitudinal:	9	6	3.40	139.5	67	31,778	63,556.20	
Top Face	Transverse:	9	6	3.40	3.5	280	3,332		
	Longitudinal:	9	6	3.40	139.5	8	3,794		
Dowels	Vertical I.F.	9	6	3.40	33.3	280	31,721		
	Vertical O.F.	9	6	3.40	33.3	280	31,721		
							cy		**LB/cy**
							134,066	701	191.1270455
						Σ =	**241,058**		
	Lap Splices (long. Bars)	9	3.40	8		467	12,702		
						Σ Bar Wt=	253,760 lb		

FORCES AT THE BOTTOM OF THE STEM

Diversion Face	H ft	γ kcf	Pbase	V K	arm ft	Mv ft-k
Diversion WSEL	18.44	0.0624	1.150656	10.609	6.147	65.21028
Tributary SEL =	33.82	0.019	0.64258	10.866	11.273	122.4964
Tributary WSEL =	0.00	0.0624	0	0.000	0.000	0
Sum				10.866		122.4964
Net Forces				0.257		57.28607

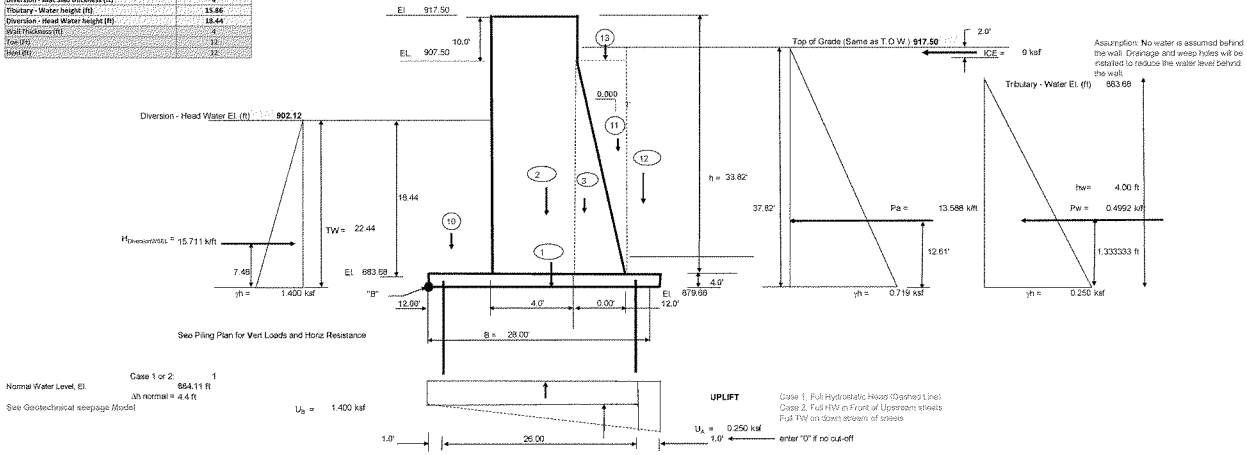
BARR ENGINEERING			DATE	2/11/2011	SHEET NO
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MDI 2/11/11		MDI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls	
			Load Cases: **Case 2 100 yr. flood + ice**		**Panel E**

ID#	Case 2
Name	100 yr. flood + ice
Load Category	Unusual
Tributary - Water El. (ft)	914.56
Diversion - Head Water El. (ft)	902.12
Diversion - Tail Water El. (ft)	901.91
Tributary - T.O. Wall El. (ft)	917.5
Tributary - T.O. Deck L.P. El.(ft)	898.7
Tributary - T.O. Deck H.P. El.(ft)	900.7
Diversion - T.O. Mat El. (ft)	883.68
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	15.86
Diversion - Head Water height (ft)	18.44
Wall Thickness (ft)	4
Toe (ft)	12
Heel (ft)	12

File:

MN State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 903.24 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section **Length** = 140.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



See Piling Plan for Vert Loads and Horiz Resistance

Case 1 or 2: 1
Normal Water Level, El. 894.11 ft
Δh normal = 4.4 ft
See Geotechnical seepage Model

U_B = 1.400 ksf

UPLIFT
Case 1: Full Hydrostatic Head (Dashed Line)
Case 2: Full HW in Front of Upstream sheets
Full TW on down stream of sheets

		Section	L ft	W ft	H ft	γ ksf	shape	V K	arm ft	Mv ft-k
Vertical Loads	Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
	Stem	2	140	4.00	33.82	0.15	rec	2840.9	14.00	39,772.3
	Batter	3	140	0.00	23.82	0.15	tri	0.0	16.00	0.0
							D.L. Concrete	ΣVc = **5192.9**	ΣM_V =	**72,700.3**

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Stem	10	140	12.00	18.44	0.0624	rec	1930.1	6.00	11,580.6
H.W. on Stem Slope	11	140	0.00	23.82	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	33.82	0.0626	rec	3556.8	22.00	78,249.2
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
						D.L. Water	ΣVw = **5489.9**	ΣM_V =	**89,847.8**

		L ft	W ft	Pressure ksf		**U** K	arm ft	Mu ft-k
Uplift Loads	U_B	140	28.00	1.400	rec	-5489.9	14.00	-76,848
	U_A	140	28.00	-1.151	tri	2255.3	18.67	42,099
					ΣU =	**-3233.7**	ΣM_U =	**-34,747**

		L ft	H ft	Pressure ksf	**ICE** K	arm ft	Mu ft-k
Horizontal Loads							

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO - MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls	
				Load Cases: **Case 2** **100 yr. flood + ice**	**Panel E**

ICE	140	2.00	0.00	rec	0.0	36.82	0.0	
	L		Force		**H**	arm	**Mw**	
	ft		k/ft		K	ft	ft-k	
SOIL	140		-13.589		-1902.37	12.61	-23982.53	
Water Loads								
H_{hw}	140		15.711	tri	2199.52	7.48	16452.43	
H_{hw}	140		-0.499	tri	-69.89	1.33	-93.18	
				ΣWater =	**2129.63**	ΣM_W =	**-7623.3**	

Overturning Moments	$\Sigma M_{OT} = M_U + M_H + M_{ICE}$ =	-42371	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	162548	kip-ft
Sum of Moments	$\Sigma M_{net} = M_R + M_{OT}$ =	**120,177**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**7,449**	kips
Sum of Horizontal Forces	H = Σhorizontal	**227**	kips
Location of Resultant	Xr = ΣM / P =	16.13 ft from Toe	
	e = B/2 - Xr =	(2.13) ft	
	B/6 =	4.667 ft	

CONCRETE QUANTITIES

				forming	
Ftg conc:	589	cy (includes stepped)	1428	sf	
Stem Conc.	701	cy	9812	sf	
Total =	1,290				

STEEL REINFORCEMENT: (assumed)

		Bar #	Spacing in	LB /ft	Length ft	# of bars ea	Total wt lb		
a) Footing									
Top mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
Bot mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
								cy	**LB/cy**
							106,991	589	181.6374748
b) Skin Reinf. On Monolith									
Vert Face	Vertical	9	6	3.40	33.32	280	31,721	63441.26	
	Longitudinal:	9	6	3.40	139.5	67	31,778	63556.2	
Top Face	Transversa:	9	6	3.40	3.5	280	3,332		
	Longitudinal:	9	6	3.40	139.5	8	3,794		
Dowels	Vertical I.F.	9	6	3.40	33.3	280	31,721		
	Vertical O.F.	9	6	3.40	33.3	280	31,721		
								cy	**LB/cy**
							134,066	701	191.1270455
						Σ =	**241,058**		
	Lap Splices (long. Bars)	9	3.40		8	467	12,702		
						Σ Bar Wt=	253,760 lb		

FORCES AT THE BOTTOM OF THE STEM

	H ft	γ kcf	Pbase	**V** K	arm ft	**Mv** ft-k
Diversion Face						
Diversion WSEL	18.44	0.0624	1.150656	10.609	6.147	65.21028
Tributary SEL =	33.82	0.018	0.64258	10.866	11.273	122.4964
Tributary WSEL =	0.00	0.0624	0	0.000	0.000	0
Sum				10.866		122.4964
Net Forces				0.257		57.28607

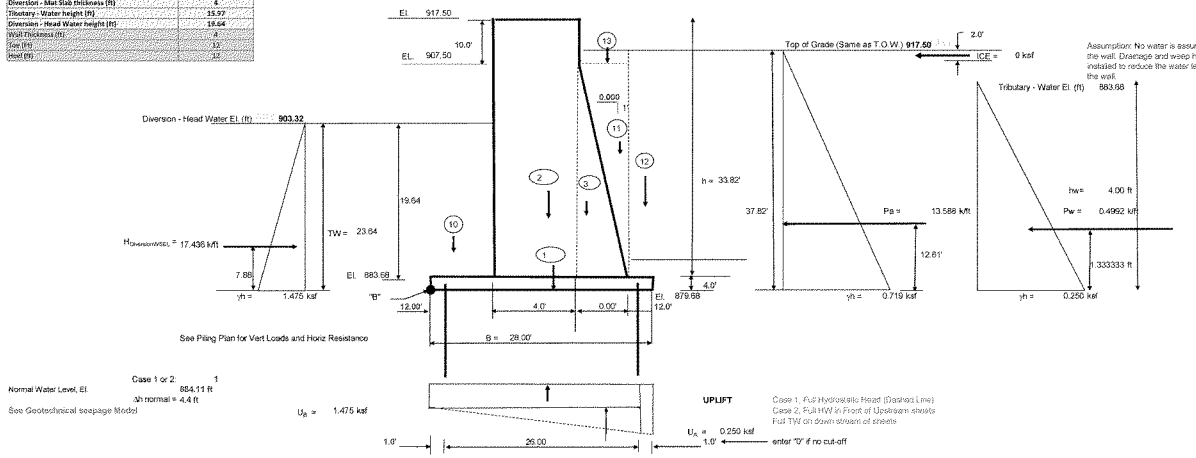
BARR ENGINEERING			DATE	2/11/2011	SHEET NO
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls	
				Load Cases: **Case 3 500 yr. flood** **Panel E**	

ID#	Case 3
Name	500 yr. flood
Load Category	Unusual
Tributary - Water El. (ft)	914.67
Diversion - Head Water El. (ft)	903.32
Diversion - Tail Water El. (ft)	903.05
Tributary - T.O. Wall El. (ft)	917.5
Tributary - T.O. Deck L.P. El.(ft)	898.7
Tributary - T.O. Deck H.P. El.(ft)	900.7
Diversion - T.O. Mat El. (ft)	883.68
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	15.97
Diversion - Head Water height (ft)	19.64
Wall thickness (ft)	4
Toe (ft)	12
Heel (ft)	12

File:

MN State Building Codes
Frost Depth = 5.0 ft provide min frost flg protection during Dec, Jan, Feb, March
Water El. = 903.24 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section **Length** = 140.0 ft
Stepped Flg Ls = 2.0 ft overlap distance at stepped flg



Vertical Loads		Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
	Flg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
	Stem	2	140	4.00	33.82	0.15	rec	2840.9	14.00	39,772.3
	Batter	3	140	0.00	23.82	0.15	tri	0.0	16.00	0.0
							D.L. Concrete	ΣVc = **5192.9**	ΣM_V =	**72,700.3**

← CONSTANT FOR ALL LOAD CASES

T.W on flg Stem	16	140	12.00	19.64	0.0624	rec	2059.9	6.00	12,353.4
H.W. on Stem Slope	11	140	0.00	23.82	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	33.82	0.0626	rec	3556.8	22.00	78,249.2
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
						D.L. Water	ΣVw = **5615.7**	ΣM_V =	**90,602.6**

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	140	28.00	1.475	rec	-5782.5	14.00	-80,955
	U_A	140	28.00	-1.226	tri	2402.1	18.67	44,838
					ΣU =	**-3380.5**	ΣM_U =	**-36,117**

Horizontal Loads	L ft	H ft	Pressure ksf	ICE K	arm ft	Mu ft-k

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	23691004	
MBI		MBI	SUBJECT	Sheyenne Aqueduct Structure - Retaining Walls	
2/11/11			Load Cases:	**Case 3 500 yr. flood**	**Panel E**

ICE	140	2.00	0.00		rec	0.0	36.82	0.0
	L		**Force**			**H**	**arm**	**Mw**
	ft		k/ft			K	ft	ft-k
SOIL	140		-13.588			-1902.37	12.61'	-23982.53
Water Loads								
H_{HW}	140		17.436		tri	2441.06	7.88	19235.51
H_{HW}	140		-0.499		tri	-69.89	1.33	-93.18
					ΣWater =	**2371.17**	ΣM_W =	**-4840.2**

Overturning Moments	$\Sigma M_{OT} = M_O + M_H + M_{ICE}$ =	-40957 kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	163303 kip-ft

Sum of Moments	$\Sigma M_{net} = M_R + M_{OT}$ =	**122,346**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**7,428**	kips
Sum of Horizontal Forces	H = Σhorizontal	**469**	kips

Location of Resultant	Xr = ΣM / P =	16.47	ft from Toe
	e = B/2 - Xr =	(2.47)	ft
	B/6 =	4.667	ft

CONCRETE QUANTITIES

			forming	
Ftg conc:	589	cy (includes stepped)	1428	sf
Stem Conc:	701	cy	9812	sf
Total =	1,290			

STEEL REINFORCEMENT: (assumed)

		Bar #	Spacing in	LB /ft	Length ft	# of bars ea	Total wt lb		
a) Footing									
Top mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
Bot mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
								cy	**LB/cy**
							106,991	589	181.6374748
b) Skin Reinf. On Monolith									
Vert Face	Vertical	9	6	3.40	33.32	280	31,721	63441.28	
	Longitudinal:	9	6	3.40	139.5	67	31,778	63556.2	
Top Face	Transverse:	9	6	3.40	3.5	280	3,332		
	Longitudinal:	9	6	3.40	139.5	8	3,794		
Dowels	Vertical I.F.	9	6	3.40	33.3	280	31,721		
	Vertical O.F.	9	6	3.40	33.3	280	31,721		
								cy	**LB/cy**
							134,066	701	191.1270455
						Σ =	**241,058**		
	Lap Splices (long. Bars)	9	3.40	8		467	12,702		
						Σ Bar Wt=	253,760 lb		

FORCES AT THE BOTTOM OF THE STEM

	H ft	γ kcf	Pbase	V K	arm ft	Mv ft-k
Diversion Face						
Diversion WSEL	19.64	0.0624	1.225536	12.035	6.547	78.78759
Tributary SEL =	33.82	0.019	0.64258	10.866	11.273	122.4964
Tributary WSEL =	0.00	0.0624	0	0.000	0.000	0
Sum				10.866		122.4964
Net Forces				-1.169		43.70877

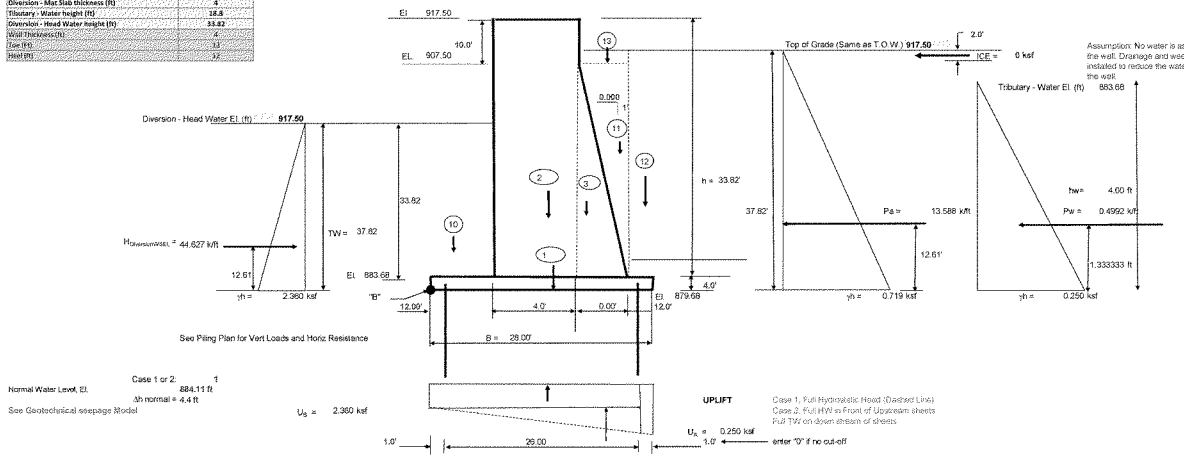
BARR ENGINEERING			DATE	2/11/2011	SHEET NO
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4:	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aqueduct Structure - Retaining Walls	
				Load Cases: **Case 4 T.O. Levee** **Panel E**	

ID#	Case 4
Name	T.O. Levee
Load Category	Extreme
Tributary - Water El. (ft)	NA
Diversion - Head Water El. (ft)	917.5
Diversion - Tail Water El. (ft)	917.5
Tributary - T.O. Wall El. (ft)	917.5
Tributary - T.O. Deck L.P. El.(ft)	898.7
Tributary - T.O. Deck H.P. El.(ft)	900.7
Diversion - T.O. Mat El. (ft)	883.68
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	18.8
Diversion - Head Water height (ft)	33.82
Wall Thickness (ft)	4
Toe (ft)	13
Heel (ft)	12

File:

MN State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 903.24 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section **Length** = 140.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



See Piling Plan for Vert Loads and Horiz Resistance

Case 1 or 2: 1
Normal Water Level, El. 884.11 ft
Δh normal = 4.4 ft
See Geotechnical steepage Model

UPLIFT
Case 1, Full Hydrostatic Head (Dashed Line)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	**V** K	arm ft	**Mv** ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	33.82	0.15	rec	2840.9	14.00	39,772.3
Batter	3	140	0.00	23.82	0.15	tri	0.0	16.00	0.0
						D.L. Concrete	ΣVc = **5192.9**	ΣMv =	**72,700.3**

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Stem	10	140	12.00	33.82	0.0624	rec	3545.4	6.00	21,272.5
H.W on Stem Slope	11	140	0.00	23.82	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	33.82	0.0625	rec	3556.8	22.00	78,249.2
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
						D.L. Water	ΣVw = **7102.2**	ΣMv =	**99,521.7**

Uplift Loads		L ft	W ft	Pressure ksf		**U** K	arm ft	**Mu** ft-k
	U_B	140	28.00	2.360	rec	-9251.1	14.00	-129,515
	U_A	140	28.00	-2.110	tri	4136.3	18.67	77,211
					ΣU =	**-5114.8**	ΣMu =	**-52,304**

Horizontal Loads	L ft	H ft	Pressure ksf	**ICE** K	arm ft	**Mu** ft-k

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34691004	
MBI		MBI	SUBJECT	Sheyenne Aqueduct Structure - Retaining Walls	
2/11/11			Load Cases:	**Case 4 T.O. Levee**	**Panel E**

ICE	140	2.00	0.00		rec	0.0	36.82	0.0
	L		Force			**H**	arm	**Mw**
	ft		k/ft			K	ft	ft-k
SOIL	140		-13.588			-1902.37	12.61'	-23982.53
Water Loads								
H_{hw}	140		44.627		tri	6247.78	12.61	78763.67
H_{lw}	140		-0.499		tri	-69.89	1.33	-93.18
					ΣWater =	**6177.89**	ΣM_W =	**54688.0**

Overturning Moments	$\Sigma M_{OT} = M_O + M_W + M_{ICE}$ =	2384	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	172222	kip-ft

Sum of Moments	$\Sigma M_{net} = M_R + M_{OT}$ =	**174,606**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**7,180**	kips
Sum of Horizontal Forces	H = Σhorizontal	**4,276**	kips

Location of Resultant	Xr = ΣM / P =	24.32	ft from Toe
	e = B/2 - Xr =	(10.32)	ft
	B/6 =	4.667	ft

CONCRETE QUANTITIES

			forming	
Ftg conc:	589	cy (includes stepped)	1428	sf
Stem Conc:	701	cy	9812	sf
Total =	1,290			

STEEL REINFORCEMENT: (assumed)

		Bar #	Spacing in	LB /ft	Length ft	# of bars ea	Total wt lb		
a) Footing									
Top mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
Bot mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
								cy	**LB/cy**
							106,991	589	181.6374748
b) Skin Reinf. On Monolith									
Vert Face	Vertical	9	6	3.40	33.32	280	31,721	63441.28	
	Longitudinal:	9	6	3.40	139.5	67	31,778	63556.2	
Top Face	Transverse:	9	6	3.40	3.5	280	3,332		
	Longitudinal:	9	6	3.40	139.5	8	3,794		
Dowels	Vertical I.F.	9	6	3.40	33.3	280	31,721		
	Vertical O.F.	9	6	3.40	33.3	280	31,721		
								cy	**LB/cy**
							134,066	701	191.1270455
						Σ =	**241,058**		
	Lap Splices (long. Bars)	9	3.40	8		467	12,702		
						Σ Bar Wt=	253,760 lb		

FORCES AT THE BOTTOM OF THE STEM

	H ft	γ kcf	Pbase	V K	arm ft	Mv ft-k
Diversion Face						
Diversion WSEL	33.82	0.0624	2.110368	35.686	11.273	402.3038
Tributary SEL =	33.82	0.019	0.64258	10.866	11.273	122.4964
Tributary WSEL =	0.00	0.0624	0	0.000	0.000	0
Sum				10.866		122.4964
Net Forces				-24.820		-279.807

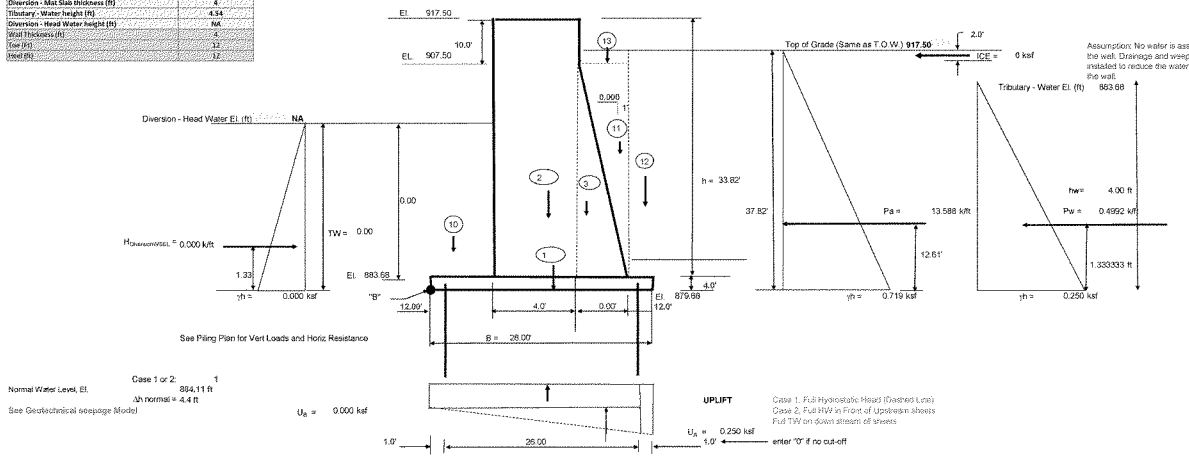
BARR ENGINEERING			DATE	2/11/2011	SHEET NO
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34991004	
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aqueduct Structure - Retaining Walls	
				Load Cases: **Case 5 Normal flow + ice**	**Panel E**

ID#	Case 5
Name	Normal flow + ice
Load Category	Usual
Tributary - Water El. (ft)	903.24
Diversion - Head Water El. (ft)	NA
Diversion - Tail Water El. (ft)	NA
Tributary - T.O. Wall El. (ft)	917.5
Tributary - T.O. Deck L.P. El.(ft)	898.7
Tributary - T.O. Deck H.P. El.(ft)	900.7
Diversion - T.O. Mat El. (ft)	883.68
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	4.54
Diversion - Head Water height (ft)	NA
Wall thickness (ft)	4
Toe (ft)	12
Heel (ft)	12

File:

MN State Building Codes
Frost Depth = 5.0 ft — provide min frost flg protection during Dec, Jan, Feb, March
Water El. = 903.24 ft — DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section — **Length =** 140.0 ft
Stepped Flg Ls = 2.0 ft — overlap distance at stepped flg



Case 1 or 2: 1
Normal Water Level, El. 894.11 ft
Δh normal = 4.4 ft
See Geotechnical seepage Model

U_B = 0.000 ksf

UPLIFT
Case 1, Full Hydrostatic Head (Desired Low)
Case 2, Full HW in Front of Upstream sheets
Full TW on down stream of sheets

1.0' — 26.00 — 1.0'
U_A = 0.250 ksf
enter "0" if no cut-off

Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	33.82	0.15	rec	2840.9	14.00	39,772.3
Batter	3	140	0.00	23.82	0.15	tri	0.0	16.00	0.0
					D.L. Concrete	ΣVc =	**5192.9**	ΣMv =	**72,700.3**

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Stem	10	140	12.00	0.00	0.0624	rec	0.0	6.00	0.0
H.W. on Stem Slope	11	140	0.00	22.82	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	33.82	0.0626	rec	3556.8	22.00	78,249.2
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
					D.L. Water	ΣVw =	**3556.8**	ΣMv =	**78,249.2**

Uplift Loads		L ft	W ft	Pressure ksf		**U** K	arm ft	**Mu** ft-k
	U_B	140	28.00	0.000	rec	0.0	14.00	0
	U_A	140	28.00	0.250	tri	-489.2	18.67	-9,132
					ΣU =	**-489.2**	ΣMu =	**-9,132**

Horizontal Loads	L ft	H ft	Pressure ksf	**ICE** K	arm ft	**Mu** ft-k

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34691004	
MEI		MEI	SUBJECT	Sheyenne Aqueduct Structure - Retaining Walls	
2/11/11				Load Cases: **Case 5 Normal flow + ice**	**Panel E**

ICE	140	2.00	0.00		rec	0.0	36.82	0.0
	L		Force			**H**	arm	**Mw**
	ft		k/ft			K	ft	ft-k
SOIL	140		-13.588			-1902.37	12.61'	-23982.53
Water Loads								
H_{hw}	140		0.000		tri	0.00	1.33	0.00
H_{HW}	140		-0.499		tri	-69.89	0.00	0.00
					ΣWater =	**-69.89**	ΣM_W =	**-23982.5**

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-33115	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	150950	kip-ft

Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**117,835**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**8,260**	kips
Sum of Horizontal Forces	H = Σhorizontal	**-1,972**	kips

Location of Resultant	Xr = ΣM / P =	14.26	ft from Toe
	e = B/2 - Xr =	(0.26)	ft
	B/6 =	4.667	ft

CONCRETE QUANTITIES

			forming	
Ftg conc:	589	cy (includes stepped)	1428	sf
Stem Conc:	701	cy	9812	sf
Total =	1,290			

STEEL REINFORCEMENT: (assumed)

		Bar #	Spacing in	LB /ft	Length ft	# of bars ea	Total wt lb		
a) Footing									
Top mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
Bot mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
								cy	**LB/cy**
							106,991	589	181.6374748
b) Skin Reinf. On Monolith									
Vert Face	Vertical	9	6	3.40	33.32	280	31,721	63441.28	
	Longitudinal:	9	6	3.40	139.5	67	31,778	63556.2	
Top Face	Transverse:	9	6	3.40	3.5	280	3,332		
	Longitudinal:	9	6	3.40	139.5	8	3,794		
Dowels	Vertical I.F.	9	6	3.40	33.3	280	31,721		
	Vertical O.F.	9	6	3.40	33.3	280	31,721		
								cy	**LB/cy**
							134,066	701	191.1270455
						Σ =	**241,058**		
	Lap Splices (long. Bars)	9	3.40	8		467	12,702		
						Σ Bar Wt=	253,760 lb		

FORCES AT THE BOTTOM OF THE STEM

	H ft	γ kcf	Pbase	V K	arm ft	Mv ft-k
Diversion Face						
Diversion WSEL	0.00	0.0624	0	0.000	0.000	0
Tributary SEL =	33.82	0.019	0.64258	10.866	11.273	122.4964
Tributary WSEL =	0.00	0.0624	0	0.000	0.000	0
Sum				10.866		122.4964
Net Forces				10.866		122.4964

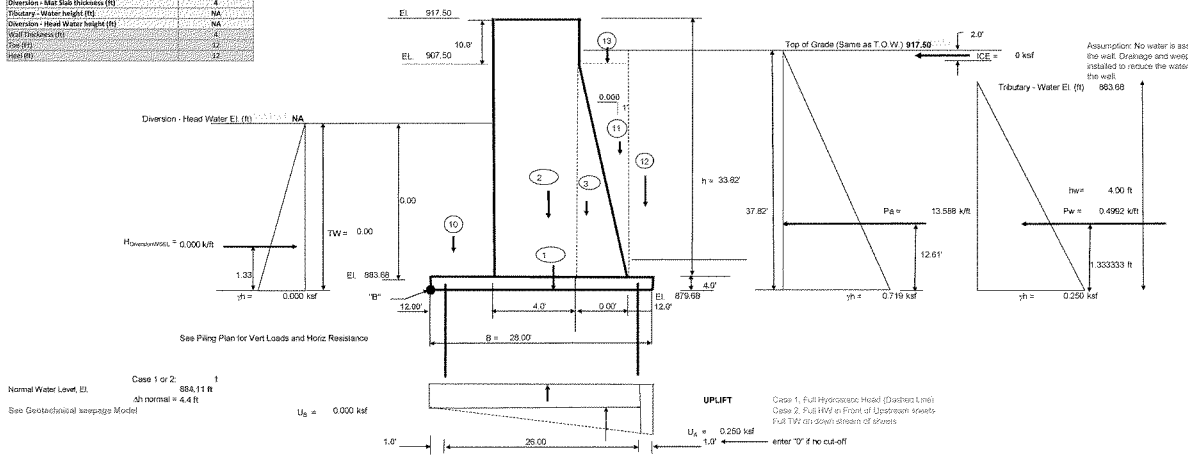
BARR ENGINEERING			DATE	2/11/2011	SHEET NO
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aqueduct Structure - Retaining Walls	
			Load Cases:	**Case 6 Construction**	**Panel E**

ID#	Case 6
Name	Construction
Load Category	Unusual
Tributary - Water El. (ft)	NA
Diversion - Head Water El. (ft)	NA
Diversion - Tail Water El. (ft)	NA
Tributary - T.O. Wall El. (ft)	917.5
Tributary - T.O. Deck L.P. El.(ft)	896.7
Tributary - T.O. Deck H.P. El.(ft)	900.7
Diversion - T.O. Mat El. (ft)	883.68
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	NA
Diversion - Head Water height (ft)	NA
Wall thickness (ft)	4
Toe (ft)	12
Heel (ft)	12

File:

MN State Building Codes
Frost Depth = 5.0 ft provide min frost flg protection during Dec, Jan, Feb, March
Water El. = 903.24 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section **Length** = 140.0 ft
Stepped Flg Ls = 2.0 ft overlap distance at stepped flg



Vertical Loads		Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
	Flg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
	Stem	2	140	4.00	33.82	0.15	rec	2840.9	14.00	39,772.3
	Batter	3	140	0.00	23.82	0.15	tri	0.0	16.00	0.0
							D.L. Concrete	ΣVc = **5192.9**	ΣMv =	**72,700.3**

← CONSTANT FOR ALL LOAD CASES

T.W on flg Stem	10	140	12.00	0.00	0.0624	rec	0.0	6.00	0.0
H.W. on Stem Slope	11	140	0.00	23.82	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	33.82	0.0628	rec	3556.8	22.00	78,249.2
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
						D.L. Water	ΣVw = **3556.8**	ΣMv =	**78,249.2**

Uplift Loads		L ft	W ft	Pressure ksf		**U** K	arm ft	**Mu** ft-k
	U_B	140	28.00	0.000	rec	0.0	14.00	0
	U_A	140	28.00	0.250	tri	-489.2	18.67	-9,132
					ΣU =	**-489.2**	ΣMu =	**-9,132**

Horizontal Loads	L ft	H ft	Pressure ksf	**ICE** K	arm ft	**Mu** ft-k

1590

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091064	
MBI		MBI	SUBJECT	Sheyenne Aqueduct Structure - Retaining Walls	
2/11/11			Load Cases:	**Case 6 Construction**	**Panel E**

ICE	140	2.00	0.00		rec	0.0	36.82	0.0
	L		**Force**			**H**	**arm**	**Mw**
	ft		k/ft			K	ft	ft-k
SOIL	140		-13.588			-1902.37	12.61'	-23982.53
Water Loads								
H_{FW}	140		0.000		tri	0.00	1.33	0.00
H_{HW}	140		-0.499		tri	-69.89	1.33	-93.18
					ΣWater =	**-69.89**	ΣM_W =	**-24075.7**

Overturning Moments	$\Sigma M_{OT} = M_O + M_H + M_{OE}$ =	-33208	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	150950	kip-ft

Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**117,742**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**8,260**	kips
Sum of Horizontal Forces	H = Σhorizontal	**-1,972**	kips

Location of Resultant	Xr = ΣM / P =	14.25	ft from Toe
	e = B/2 - Xr =	(0.25)	ft
	B/6 =	4.667	ft

CONCRETE QUANTITIES

			forming	
Ftg conc:	589	cy (includes stepped)	1428	sf
Stem Conc:	701	cy	9812	sf
Total =	1,290			

STEEL REINFORCEMENT: (assumed)

		Bar #	Spacing in	LB /ft	Length ft	# of bars ea	Total wt lb		
a) Footing								cy	LB/cy
Top mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
Bot mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
							106,991	589	181.6374748
b) Skin Reinf. On Monolith									
Vert Face	Vertical	9	6	3.40	33.32	280	31,721	63441.28	
	Longitudinal:	9	6	3.40	139.5	67	31,778	63556.2	
Top Face	Transverse:	9	6	3.40	3.5	280	3,332		
	Longitudinal:	9	6	3.40	139.5	8	3,794		
Dowels	Vertical I.F.	9	6	3.40	33.3	280	31,721		
	Vertical O.F.	9	6	3.40	33.3	280	31,721		
								cy	LB/cy
							134,066	701	191.1270455
						Σ =	**241,058**		
Lap Splices (long. Bars)		9	3.40	8		467	12,702		
						Σ Bar Wt=	253,760 lb		

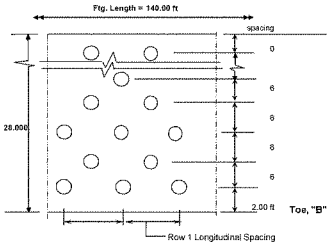
FORCES AT THE BOTTOM OF THE STEM

	H ft	γ kcf	Pbase	V K	arm ft	Mv ft-k
Diversion Face						
Diversion WSEL	0.00	0.0624	0	0.000	0.000	0
Tributary SEL =	33.82	0.019	0.64258	10.866	11.273	122.4964
Tributary WSEL =	0.00	0.0624	0	0.000	0.000	0
Sum				10.866		122.4964
Net Forces				10.866		122.4964

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Sheyenne Aqueduct Structure - Retaining Walls	
				Panel E	

PILE FOUNDATION DESIGN

FLOW



L = 140.00 ft

PILE PATTERN GEOMETRY

Transverse Spacing		Distance to Toe, d_{toe}	Longitudinal Spacing	Batter	Piles per Row (N)	Edge Dist (ft)		Trial N
Row 1 to Toe	2.00 ft	2.0 ft	2.50 ft	0 "/12"	23	42.50	1	57
Row 1 to Row 2	6.00 ft	8.0 ft	5.00 ft	0 "/12"	22	17.50	2	29
Row 2 to Row 3	6.00 ft	14.0 ft	5.00 ft	0 "/12"	22	17.50	3	29
Row 3 to Row 4	6.00 ft	20.0 ft	5.00 ft	0 "/12"	24	12.50	4	29
Row 4 to Row 5	6.00 ft	26.0 ft	5.00 ft	0 "/12"	24	12.50	5	29
Row 5 to Row 6	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	6	0
Row 6 to Row 7	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	7	0
Row 7 to Row 8	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	8	0
Row 8 to Row 9	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	9	0
Row 9 to Row 10	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	10	0
Row 10 to Row 11	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	11	0
Row 11 to Row 12	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	12	0
Row 12 to Row 13	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	13	0
Row 13 to Row 14	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	14	0
Row 14 to Row 15	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	15	0
Last Row to Heel	2.00 ft							
	28.00 ft							

Note: Enter 0 for Longitudinal Spacing for Rows Not Used

ΣN = 115 — Trial N = 173

Pile Properties: Pile Type: HP (C,P or HP) Pile Length = 38.0 ft

HP Nominal Depth, h = 14.0 in

Wt. per ft, plf 73

Ftg EL. 975.68

Pile Tip El. 842.68

Pile Cap Embed = 1.00 ft

Total pile Length = 4,370 LF

Pile Group Properties

N.A. of Pile Group to Toe

$X_{NA} = (\Sigma N * d_{toe}) / \Sigma N =$ **14.21 ft**

	Dist. From N.A. to Pile Row	d	N	$I = N * d^2$
1	Dist. To Row 1	12.21 ft	23	3428.2
2	Dist. To Row 2	6.21 ft	22	848.1
3	Dist. Row 3	0.21 ft	22	1.0
4	Dist. Row 4	-5.79 ft	24	804.9
5	Dist. Row 5	-11.79 ft	24	3336.8
6	Row 6 (not used)	0.00 ft	0	0.0
7	Row 7 (not used)	0.00 ft	0	0.0
8	Row 8 (not used)	0.00 ft	0	0.0
9	Row 9 (not used)	0.00 ft	0	0.0
10	Row 10 (not used)	0.00 ft	0	0.0
11	Row 11 (not used)	0.00 ft	0	0.0
12	Row 12 (not used)	0.00 ft	0	0.0
13	Row 13 (not used)	0.00 ft	0	0.0
14	Row 14 (not used)	0.00 ft	0	0.0
15	Row 15 (not used)	0.00 ft	0	0.0
			115	$\Sigma I =$ **8419.0**

ALLOWABLE LOADS (from Geotechnical)

	Service	Allowable Pile Loads					
	ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
	Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
	Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Allowable Lateral Capacity (tons)		18.0 tons	20.5 tons	20.5 tons	24.0 tons	11.5 tons	20.5 tons
Allowable Pile Capacity (tons) - Axial		82.0 tons	82.0 tons	82.0 tons	107.7 tons	35.5 tons	82.0 tons
Safety Factors		2.00	1.50	1.50	1.15	2.00	1.50

w/o Group effects

Summary Pile Reactions

Load Combinations	Allowable Pile Capacity (tons) - Axial	Pile Loads (tons/pile)												Max. Vertical Load (Tons)	Horiz Pile Group Capacity (lk)	Check
		1	2	3	4	5	6	7	8	9	10	11	12			
Case 1	**82.0 tons**	22.0	27.1	32.2	37.3	42.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	42.4	4,140	**OK**
Case 2	**82.0 tons**	22.0	27.1	32.2	37.3	42.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	42.4	4,715	**OK**
Case 3	**82.0 tons**	20.1	26.1	32.1	38.1	44.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	44.1	4,715	**OK**
Case 4	**107.7 tons**	-21.4	4.5	30.3	56.2	82.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	82.0	5,520	**OK**
Case 5	**35.5 tons**	35.6	35.7	35.9	36.1	36.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	36.2	2,845	**OK**
Case 6	**82.0 tons**	35.6	35.8	35.9	36.0	36.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	36.2	2,845	**OK**

Max Service : P = **82.0**

Using solid mechanics equations adapted for discrete elements, the forces in the pile rows for different load combinations are determined.
The force in each pile row is found using:

$$\text{Pile Load} = P / N + M_{NA} / I$$

First, the moment about the toe must be translated to get the moment about the neutral axis of the pile group.

$$e_{toe} = M_{toe} / P$$

Then the eccentricity about the neutral axis of the pile group is

$$e_{NA} = X_{toe} - e_{toe}$$

The moment about the neutral axis of the pile group becomes

$$M_{NA} = P * e_{NA}$$

For battered pile, the Vertical pile load needs to be transformed to the axial load along the pile axis

Paxial = 1.000 Pvert

FORCE RESULTANT (see Stability Analysis)

CASE	Event		Vertical Load P (kips)	Horizontal	ΣM_{toe} (kip-	$e_{toe} = M_{toe} / P$	$e_{NA} = X_{toe} - e_{toe}$	$M_{NA} = P * e_{NA}$

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BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4 :	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091064	
MBI		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls	
2/11/11				Panel E	

Case 1	**100 yr. flood**	**Usual**	7,449	-227	120,177	16.13	-1.92	-14336
Case 2	**100 yr. flood + ice**	**Unusual**	7,449	-227	120,177	16.13	-1.92	-14336
Case 3	**500 yr. flood**	**Unusual**	7,425	-489	122,346	16.47	-2.26	-16902
Case 4	**T.O. Levee**	**Extreme**	7,180	-4,276	174,908	24.32	-10.11	-72583
Case 5	**Normal flow + ice**	**Usual**	8,260	1,972	117,835	14.26	-0.06	-465
Case 6	**Construction**	**Unusual**	8,260	1,972	117,742	14.25	-0.04	-372

SERVICE

Case **Case 1**
Flood Event **100 yr. flood**
Usual

Vertical Load, P = 7449 kips
Horizontal Load, H = -227 kips
M_{LR} = -14336 kip-ft

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Vertical Pile Loading	P / N	+	M_{LR}* d / Σ I	= Pile Loads			**Axial Pile Load**
1 Row 1	64.8		-20.8	44.0 kips/pile		22.0 tons/pile	22.0 tons/pile
2 Row 2	64.8		-10.6	54.2 kips/pile		27.1 tons/pile	27.1 tons/pile
3 Row 3	64.8		-0.4	64.4 kips/pile		32.2 tons/pile	32.2 tons/pile
4 Row 4	64.8		9.9	74.6 kips/pile		37.3 tons/pile	37.3 tons/pile
5 Row 5	64.8		20.1	84.9 kips/pile		42.4 tons/pile	42.4 tons/pile
6 Row 6	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
7 Row 7	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
8 Row 8	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
9 Row 9	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
10 Row 10	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
11 Row 11	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
12 Row 12	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
13 Row 13	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
14 Row 14	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
15 Row 15	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
					max:	**42.4 tons/pile**	**max: 42.4 tons/pile**

Assumed lateral Capacity: 38.0 kips/pile

Horizontal Pile Capacity	**Batter °/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1 Row 1	0	23	0.0	825	1.000	828 kips	
2 Row 2	0	22	0.0	792	1.000	792 kips	
3 Row 3	0	22	0.0	792	1.000	792 kips	
4 Row 4	0	24	0.0	864	1.000	864 kips	
5 Row 5	0	24	0.0	864	1.000	864 kips	
6 Row 6	0	0	0.0	0	1.000	0 kips	
7 Row 7	0	0	0.0	0	1.000	0 kips	
8 Row 8	0	0	0.0	0	1.000	0 kips	
9 Row 9	0	0	0.0	0	1.000	0 kips	
10 Row 10	0	0	0.0	0	1.000	0 kips	
11 Row 11	0	0	0.0	0	1.000	0 kips	
12 Row 12	0	0	0.0	0	1.000	0 kips	
13 Row 13	0	0	0.0	0	1.000	0 kips	
14 Row 14	0	0	0.0	0	1.000	0 kips	
15 Row 15	0	0	0.0	0	1.000	0 kips	
		115		4140		**4140 kips**	**OK**

Case **Case 2**
Flood Event **100 yr. flood + ice**
Unusual

Vertical Load, P = 7449 kips
Horizontal Load, H = -227 kips
M_{LR} = -14336 kip-ft

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Vertical Pile Loading	P / N	+	M_{LR}* d / Σ I	= Pile Loads			**Axial Pile Load**
1 Row 1	64.8		-20.8	44.0 kips/pile		22.0 tons/pile	22.0 tons/pile
2 Row 2	64.8		-10.6	54.2 kips/pile		27.1 tons/pile	27.1 tons/pile
3 Row 3	64.8		-0.4	64.4 kips/pile		32.2 tons/pile	32.2 tons/pile
4 Row 4	64.8		9.9	74.6 kips/pile		37.3 tons/pile	37.3 tons/pile
5 Row 5	64.8		20.1	84.9 kips/pile		42.4 tons/pile	42.4 tons/pile
6 Row 6	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
7 Row 7	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
8 Row 8	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
9 Row 9	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
10 Row 10	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
11 Row 11	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
12 Row 12	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
13 Row 13	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
14 Row 14	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
15 Row 15	0.0		0.0	0.0 kips/pile		0.0 tons/pile	0.0 tons/pile
					max:	**42.4 tons/pile**	**max: 42.4 tons/pile**

Assumed lateral Capacity: 41.0 kips/pile

Horizontal Pile Capacity	**Batter °/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance
1 Row 1	0	23	0.0	943	1.000	943 kips
2 Row 2	0	22	0.0	902	1.000	902 kips
3 Row 3	0	22	0.0	902	1.000	902 kips

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BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI		MBI	SUBJECT	Sheyenne Aqueduct Structure - Retaining Walls	
2/11/11				Panel E	

4 Row 4	0	24	0.0	984	1.000	984 kips	
5 Row 5	0	24	0.0	984	1.000	984 kips	
6 Row 6	0	0	0.0	0	1.000	0 kips	
7 Row 7	0	0	0.0	0	1.000	0 kips	
8 Row 8	0	0	0.0	0	1.000	0 kips	
9 Row 9	0	0	0.0	0	1.000	0 kips	
10 Row 10	0	0	0.0	0	1.000	0 kips	
11 Row 11	0	0	0.0	0	1.000	0 kips	
12 Row 12	0	0	0.0	0	1.000	0 kips	
13 Row 13	0	0	0.0	0	1.000	0 kips	
14 Row 14	0	0	0.0	0	1.000	0 kips	
15 Row 15	0	0	0.0	0	1.000	0 kips	
		115		4715		**4715 kips**	**OK**

Case **Case 3**
Flood Event **500 yr. flood**
Unusual

Vertical Load, P = 7428 kips
Horizontal Load, H = -469 kips
M_{LL} = -16852 kip-ft

Vertical Pile Loading	P / N	+	M_{LL}* d / Σ I	= Pile Loads			**Axial Pile Load**
1 Row 1	64.6		-24.4	40.2 kips/pile	26.1 tons/pile		26.1 tons/pile
2 Row 2	64.6		-12.4	52.2 kips/pile	26.1 tons/pile		26.1 tons/pile
3 Row 3	64.6		-0.4	64.2 kips/pile	32.1 tons/pile		32.1 tons/pile
4 Row 4	64.6		11.6	76.2 kips/pile	38.1 tons/pile		38.1 tons/pile
5 Row 5	64.6		23.5	88.1 kips/pile	44.1 tons/pile		44.1 tons/pile
6 Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
7 Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
8 Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
9 Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
10 Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
11 Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
12 Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
13 Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
14 Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
15 Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
					max: 44.1 tons/pile		**max: 44.1 tons/pile**

Assumed lateral Capacity: 41.0 kips/pile

Horizontal Pile Capacity	**Batter "/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1 Row 1	0	23	0.0	943	1.000	943 kips	
2 Row 2	0	22	0.0	902	1.000	902 kips	
3 Row 3	0	22	0.0	902	1.000	902 kips	
4 Row 4	0	24	0.0	984	1.000	984 kips	
5 Row 5	0	24	0.0	984	1.000	984 kips	
6 Row 6	0	0	0.0	0	1.000	0 kips	
7 Row 7	0	0	0.0	0	1.000	0 kips	
8 Row 8	0	0	0.0	0	1.000	0 kips	
9 Row 9	0	0	0.0	0	1.000	0 kips	
10 Row 10	0	0	0.0	0	1.000	0 kips	
11 Row 11	0	0	0.0	0	1.000	0 kips	
12 Row 12	0	0	0.0	0	1.000	0 kips	
13 Row 13	0	0	0.0	0	1.000	0 kips	
14 Row 14	0	0	0.0	0	1.000	0 kips	
15 Row 15	0	0	0.0	0	1.000	0 kips	
		115		4715		**4715 kips**	**OK**

Case **Case 4**
Flood Event **T.O. Levee**
Extreme

Vertical Load, P = 7180 kips
Horizontal Load, H = -4276 kips
M_{LL} = -72553 kip-ft

Vertical Pile Loading	P / N	+	M_{LL}* d / Σ I	= Pile Loads			**Axial Pile Load**
1 Row 1	62.4		-105.3	-42.8 kips/pile	-21.4 tons/pile		-21.4 tons/pile
2 Row 2	62.4		-53.5	8.9 kips/pile	4.5 tons/pile		4.5 tons/pile
3 Row 3	62.4		-1.8	60.6 kips/pile	30.3 tons/pile		30.3 tons/pile
4 Row 4	62.4		49.9	112.4 kips/pile	56.2 tons/pile		56.2 tons/pile
5 Row 5	62.4		101.7	164.1 kips/pile	82.0 tons/pile		82.0 tons/pile
6 Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
7 Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
8 Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
9 Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
10 Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
11 Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
12 Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
13 Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
14 Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
15 Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
					max: 82.0 tons/pile		**max: 82.0 tons/pile**

Assumed lateral Capacity: 48.0 kips/pile

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BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4 :	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34991064	
MBI		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls	
2/11/11				Panel E	

Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1 Row 1	0	23	0.0	1104	1.000	1104 kips	
2 Row 2	0	22	0.0	1056	1.000	1056 kips	
3 Row 3	0	22	0.0	1056	1.000	1056 kips	
4 Row 4	0	24	0.0	1152	1.000	1152 kips	
5 Row 5	0	24	0.0	1152	1.000	1152 kips	
6 Row 6	0	0	0.0	0	1.000	0 kips	
7 Row 7	0	0	0.0	0	1.000	0 kips	
8 Row 8	0	0	0.0	0	1.000	0 kips	
9 Row 9	0	0	0.0	0	1.000	0 kips	
10 Row 10	0	0	0.0	0	1.000	0 kips	
11 Row 11	0	0	0.0	0	1.000	0 kips	
12 Row 12	0	0	0.0	0	1.000	0 kips	
13 Row 13	0	0	0.0	0	1.000	0 kips	
14 Row 14	0	0	0.0	0	1.000	0 kips	
15 Row 15	0	0	0.0	0	1.000	0 kips	
		115		5520		5520 kips	OK

Case **Case 5**
Flood Event **Normal flow + ice**
Usual

Vertical Load, P = 8260 kips
Horizontal Load, H = 1972 kips
M_{pile} = -465 kip-ft

Vertical Pile Loading	P / N	+	M_{pile} * d / Σ I	= Pile Loads			Axial Pile Load
1 Row 1	71.8		-0.7	71.2 kips/pile	35.6 tons/pile		35.6 tons/pile
2 Row 2	71.8		-0.3	71.5 kips/pile	35.7 tons/pile		35.7 tons/pile
3 Row 3	71.8		0.0	71.8 kips/pile	35.9 tons/pile		35.9 tons/pile
4 Row 4	71.8		0.3	72.1 kips/pile	36.1 tons/pile		36.1 tons/pile
5 Row 5	71.8		0.7	72.5 kips/pile	36.2 tons/pile		36.2 tons/pile
6 Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
7 Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
8 Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
9 Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
10 Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
11 Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
12 Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
13 Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
14 Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
15 Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
				max:	**36.2 tons/pile**	**max:**	**36.2 tons/pile**

Assumed lateral Capacity: 23.0 kips/pile

Horizontal Pile Capacity	Batter "/ft	N	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1 Row 1	0	23	0.0	529	1.000	529 kips	
2 Row 2	0	22	0.0	506	1.000	506 kips	
3 Row 3	0	22	0.0	506	1.000	506 kips	
4 Row 4	0	24	0.0	552	1.000	552 kips	
5 Row 5	0	24	0.0	552	1.000	552 kips	
6 Row 6	0	0	0.0	0	1.000	0 kips	
7 Row 7	0	0	0.0	0	1.000	0 kips	
8 Row 8	0	0	0.0	0	1.000	0 kips	
9 Row 9	0	0	0.0	0	1.000	0 kips	
10 Row 10	0	0	0.0	0	1.000	0 kips	
11 Row 11	0	0	0.0	0	1.000	0 kips	
12 Row 12	0	0	0.0	0	1.000	0 kips	
13 Row 13	0	0	0.0	0	1.000	0 kips	
14 Row 14	0	0	0.0	0	1.000	0 kips	
15 Row 15	0	0	0.0	0	1.000	0 kips	
		115		2645		2645 kips	OK

Case **Case 6**
Flood Event **Construction**
Unusual

Vertical Load, P = 8260 kips
Horizontal Load, H = 1972 kips
M_{pile} = -372 kip-ft

Vertical Pile Loading	P / N	+	M_{pile} * d / Σ I	= Pile Loads			Axial Pile Load
1 Row 1	71.8		-0.5	71.3 kips/pile	35.6 tons/pile		35.6 tons/pile
2 Row 2	71.8		-0.3	71.5 kips/pile	35.8 tons/pile		35.8 tons/pile
3 Row 3	71.8		0.0	71.8 kips/pile	35.9 tons/pile		35.9 tons/pile
4 Row 4	71.8		0.3	72.1 kips/pile	36.0 tons/pile		36.0 tons/pile
5 Row 5	71.8		0.5	72.4 kips/pile	36.2 tons/pile		36.2 tons/pile
6 Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
7 Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
8 Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
9 Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
10 Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls	
2/11/11				Panel E	

11	Row 11	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
					max:	**36.2 tons/pile**	**max:**	**36.2 tons/pile**

Assumed lateral Capacity: 41.0 kips/pile

	Horizontal Pile Capacity	Batter °/ft	N	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1	Row 1	0	23	0.0	529	1.000	529 kips	
2	Row 2	0	22	0.0	506	1.000	506 kips	
3	Row 3	0	22	0.0	506	1.000	506 kips	
4	Row 4	0	24	0.0	552	1.000	552 kips	
5	Row 5	0	24	0.0	552	1.000	552 kips	
6	Row 6	0	0	0.0	0	1.000	0 kips	
7	Row 7	0	0	0.0	0	1.000	0 kips	
8	Row 8	0	0	0.0	0	1.000	0 kips	
9	Row 9	0	0	0.0	0	1.000	0 kips	
10	Row 10	0	0	0.0	0	1.000	0 kips	
11	Row 11	0	0	0.0	0	1.000	0 kips	
12	Row 12	0	0	0.0	0	1.000	0 kips	
13	Row 13	0	0	0.0	0	1.000	0 kips	
14	Row 14	0	0	0.0	0	1.000	0 kips	
15	Row 15	0	0	0.0	0	1.000	0 kips	
			115		2645		**2645 kips**	**OK**

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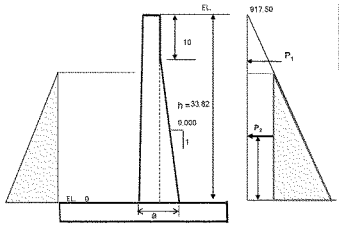
BARR ENGINEERING			DATE	2/11/2011	SHEET NO
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls	
2/11/11				Panel E 0	



CASE	Event		HW	TW	Dh	TW - Rg
Case 1	100 yr. flood	Usual	902.12	901.91	0.21	901.91
Case 2	100 yr. flood + ice	Unusual	902.12	901.91	0.21	901.91
Case 3	500 yr. flood	Unusual	914.670	903.06	11.61	903.06
Case 4	T.O. Levee	Extreme	917.500	917.50	0.00	917.50
Case 5	Normal flow + ice	Usual	0.000	0.000	0.00	0.00
Case 6	Construction	Unusual	0.000	0.000	0.00	0.00

LOAD FACTORS

Hf =	1.30	hydraulic Factor
LF =	1.70	
Unusual & Extreme =	0.75	
TOP THICK =	4.0 ft	48.0 in
Batter at Base =	0.00 ft	0.0 in
a =	4.00 ft	48.0 in

Load Factors - Hydraulic Structures	
live load, LL =	1.7
dead load, DL =	1.4
flood level , FL =	1
Fluid, F =	1.7
hydraulic, Hf =	1.3
direct tension hydraulic, Hf =	1.65
ICE =	1.7

WALL DESIGN:

Horizontal Load Components and Moments about Bottom of Stem (Service)

CASE	Event	Condition	Load Factor	H (kips/ft)	Moment (kip-ft/ ft)	Vu (kips/ft)	Mu (kip-ft/ ft)
Case 1	100 yr. flood	Usual	1	0.26	57.286	0.57	126.60
Case 2	100 yr. flood + ice	Unusual	0.75	0.26	57.286	0.43	94.95
Case 3	500 yr. flood	Unusual	0.75	-1.17	43.709	-1.94	72.45
Case 4	T.O. Levee	Extreme	0.75	-24.83	-279.807	41.14	463.78
Case 5	Normal flow + ice	Usual	1	10.87	122.496	24.01	270.72
Case 6	Construction	Unusual	0.75	10.87	122.496	18.01	203.04

STEM DESIGN VALUES

Mu, k-ft/ft	463.78	k-ft/ft
Vu, k/ft	41.14	k/ft

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI		MBI	SUBJECT	Sheyenne Aquaduct Structure - Retaining Walls	
2/11/11				Panel E 0	

ACI 318-05 w/ Modifications per EM 1110-2-2104 ref: EM 110-2-2104

9.3 - Design Strength

ϕ =	**0.9**	9.3.2.1 - Tension Controlled sections
	0.75	9.3.2.3 - Shear and torsion

FLEXURAL STEEL FOR RECTANGULAR CONCRETE SECTIONS

fy=	60 ksi	
F'c=	4 ksi	
B1=	0.85	
Muh =	464 k-ft /ft	includes: hf = 1.3
Vuh=	41.14 k / ft	
bw=	12 in.	
h=	48 in	
cover=	4 in (include correct stirrup bar dia.)	
d=	43.50 in	
pb=	0.0285	pb=0.85*B1*Fc'/ fy*(87 / (87+fy))
.75*pb=	0.0214	
rho=fy / 0.85*Fc'=	17.647	

TRIAL

Ru=Mu/bd^2 =	272.328		**ACI 10.5.1**		**ACI 10.5.3**
REQ'D p=	0.0047	**O.K.**	p(min)= 3*SQRT(Fc')/fy	200" / fy	4/3*p
p=	FALSE	N.G.		0.00516 0.00333	0.0063
			EM 110-2-2104 2-8 c. (not less than Temp & Shrinkage, half in each face)		
As (REQ'D)=	0.81	in^2	p(min)= 0.0028 /2 = ► As =0.5*p$_{min}$bh =		0.8064 in^2
			As = #9 @ 12 =		1.00 in^2

SELECT STEEL

bar #=	9		
spacing, s=	6	in	
# OF BAR=	1	(ENTER 1 IF PER FT, b=12")	a
As=	1.999	in^2	
d=	43.4375	in	
p = As/bd =	0.0038	**O.K. < 0.375pb**	**EM 110-2-2104**
p =	0.7*5 pb		**MAXIMUM TENSILE REINFORCEMENT**

a) For singly reinforced flexural members

1) p =	**0.75** pb	Recommended limit
2) p =	**0.375** pb	Max. permitted upper limit not requiring special study
3) p =	**0.5** pb	Max. permitted upper limit when excessive deflections are not predicted in ACI 318
4) p =	**>0.5** pb	but ≤ 0.375 pb permitted only if detailed serviceability analysis incl. deflct. Calc.

b) Use of compression reinf. shall be per ACI 318

T= As*fy =	119.9 k	
C= 0.85*Fc' *b*a =	1576.9 a	
a= T/C =	0.076 in	
Mn= T(d - a/2)/12 =	433.7 ft-k	
ϕ Mn=	390.3 ft-k	< Mu N.G.

CHECK SHEAR REINFORCEMENT (ACI 11.3 & EM 1110-2-2104 3-3a)

Vuh =	41.1 k		NO SHEAR REINF. REQUIRED
Vn = Vuh / ϕ =	54.9 k		
Vc = 2*sqrt(Fc') bw * d =	65.9 k		**11.3.1.1**
Vs =Vuh / f - 1.3Vc = No Shear Reinf. Req.	k	NG	Vs(max) ≤ 8*sqrt(fc')bd = **263.7** k

Trial Stirrup Sizes:

# of stirrup legs =	2	(single stirrup = 2, Dbl stirrup = 4)
Stirrup bar size #	4	
Av=	0.393 in^2	
s=	0.000 in	s = Av * fy * d / (Vu / f - Vc)

11.5.5 - Spacing limits for shear reinforcement

s = d/2 =	21.719 in	OR	24	in
s(max)=	10.859 in			
4*sqrt(Fc')*bw*d=	131.9 k	< Vs Reduce Spacing		
USE s=	**0.00** in			
Vs = (Av * Fy * d) / s =	#DIV/0!	k		

11.5.6 - MINIMUM SHEAR REINFORCEMENT

A minimum area of shear reinforcement, Av,min shall be provided in all reinforced concrete flexural members where Vu exceeds **0.5 f Vc**

NOT REQUIRED IF:

a) SLAB OR FOOTING, vc>vn	**O.K.**
b) CONCRETE JOIST ACI 8.11	
c) BEAMS W/ h <= 10"	
h <= 2.5*Bf	
h <= 0.5*tw	
d) WALLS (SEE ACI 11.10.1); vc>vn	**O.K.**

11.5.6.3

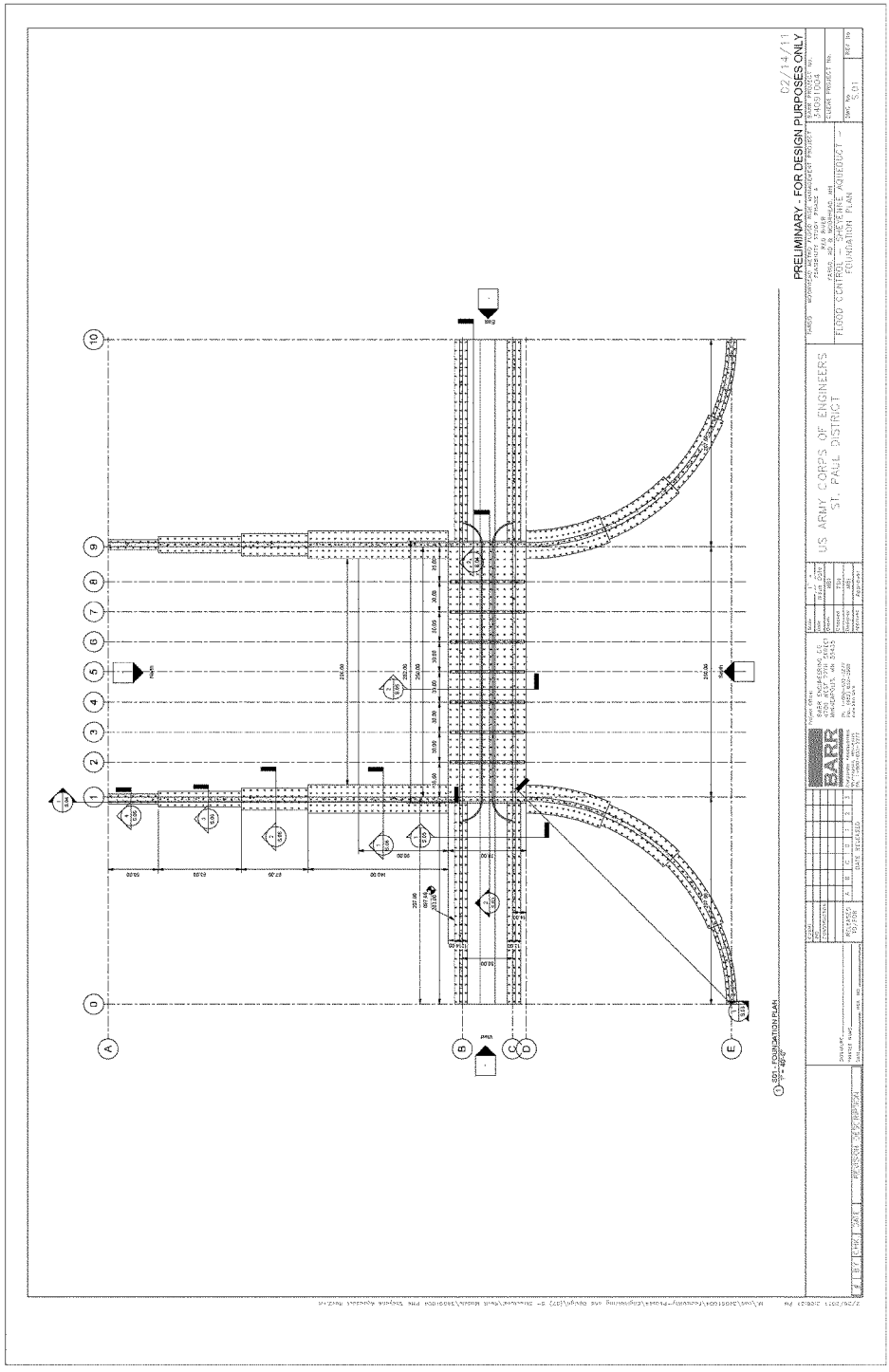
Av,min = 0.75 sqrt(fc) bw*s/fy =	0.70 * s
but not less than 50bw*s/ fy =	29.33333333 * s
s max = Av fy / 0.75 sqrt(fc') bw =	0.00 in
s max = Av fy /50 bw =	0.00 in

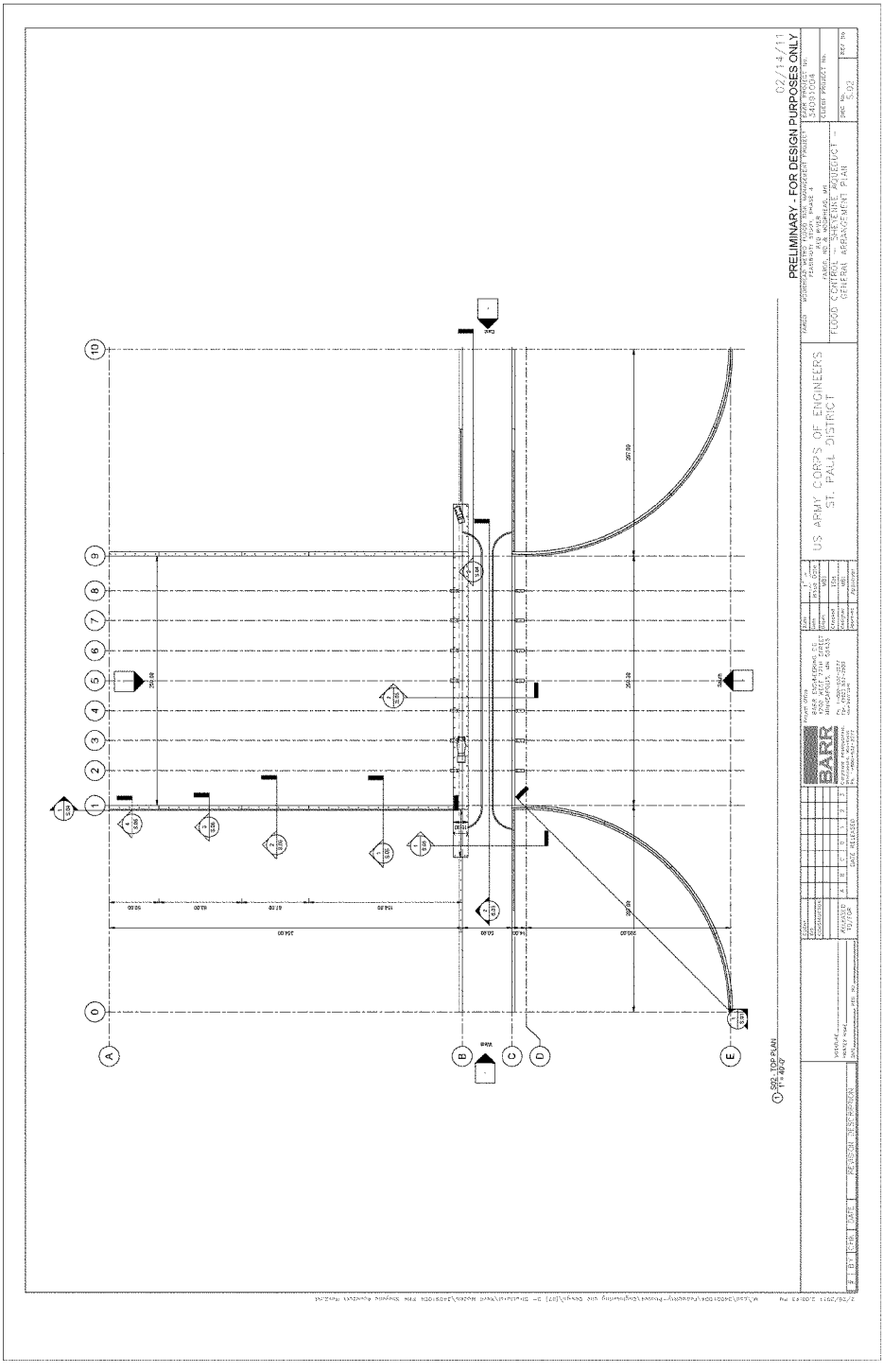
11.5.5.3

Where Vs exceeds 4*sqrt(Fc')*bw*d maximum spacings shall be reduced by one-half

ATTACHMENT

F-R4.4 Sheyenne River Aqueduct Structure Drawings


PRELIMINARY - FOR DESIGN PURPOSES ONLY
02/14/11
US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
FLOOD CONTROL - SHEYENNE AQUEDUCT -
FOUNDATION PLAN
BARR
S101 - FOUNDATION PLAN
1" = 40'-0"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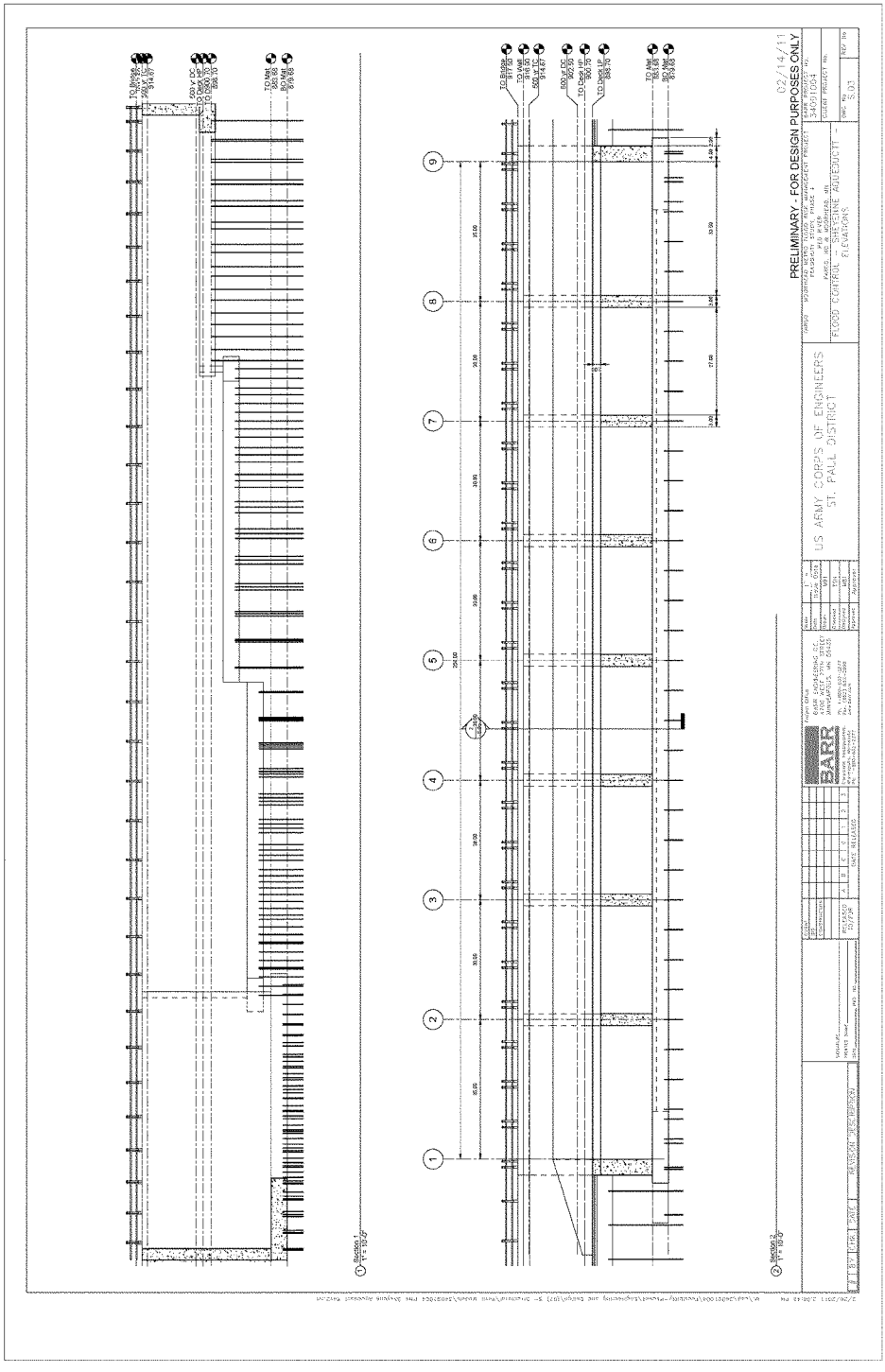
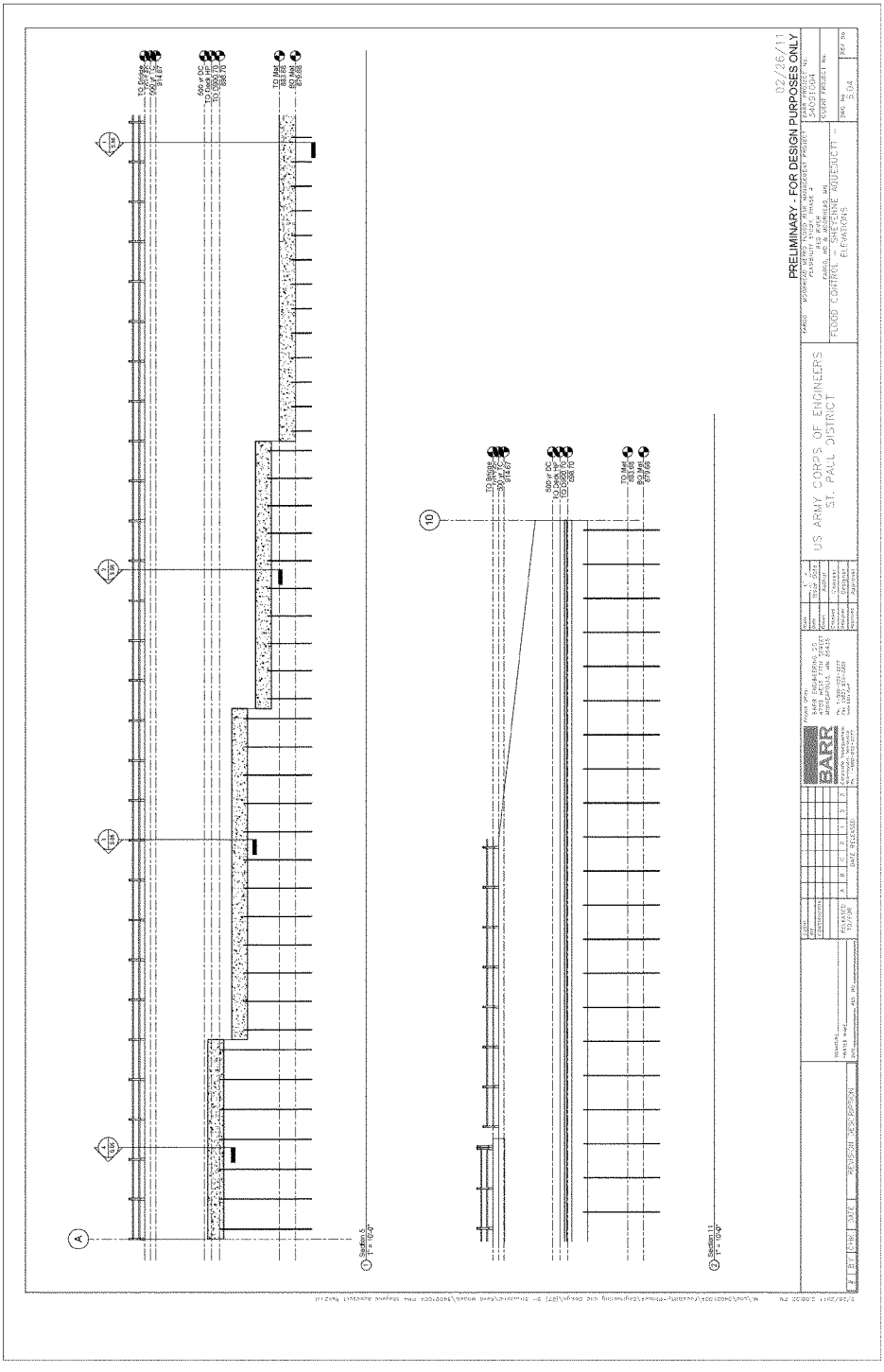
ST. PAUL DISTRICT
FLOOD CONTROL -- SHERBURNE AQUEDUCT
GENERAL ARRANGEMENT PLAN
S.02
BARR
1 SQ2 TOP PLAN
1" = 40'-0"


PRELIMINARY - FOR DESIGN PURPOSES ONLY
02/14/11
U.S. ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
FLOOD CONTROL - SHEYENNE AQUEDUCT - ELEVATIONS
BARR
Section 1
1" = 10'-0"
Section 2
1" = 10'-0"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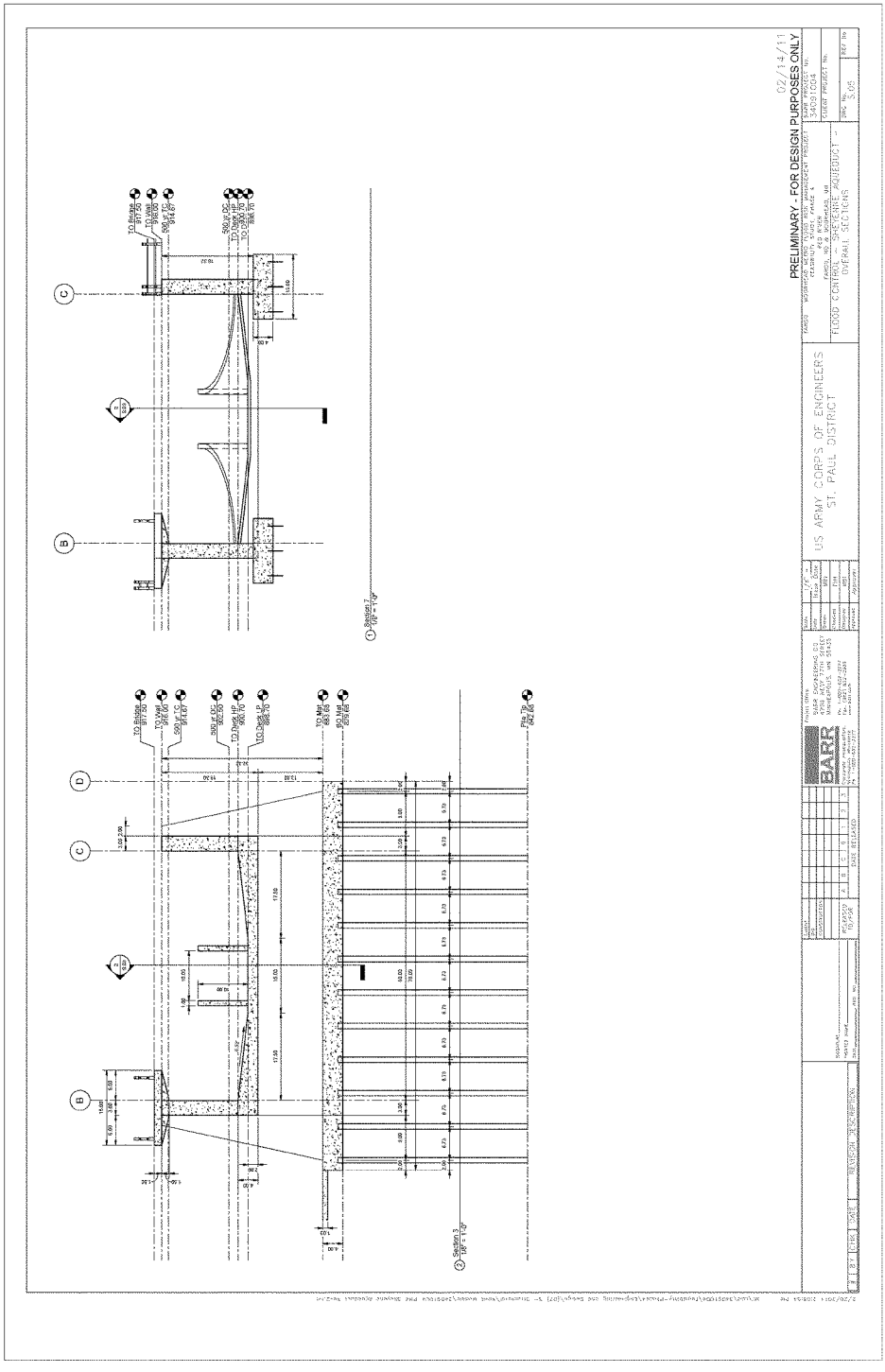
US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
FLOOD CONTROL -- SHEYENNE AQUEDUCT
ELEVATIONS
BARR
TO Bridge
TO Deck
TO Wall
TO Mat
TO Deck W
TO Deck E
Station 5
Station 11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02/14/11
US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
FLOOD CONTROL - SHEYENNE AQUEDUCT
OUTFALL SECTIONS
BARR
Section 7
Section 3

Structural Foundation Schedule

Type	Structural Usage	Count	Width	Length	Foundation Thickness	Area	Forming	Volume	Volume CY	Reinf. (lbs/ft)	Total Reinf. (lbs)
Retaining Footing - 36" x 36" x 48"	Retaining	1	10.00	50.00	4.00	500 SF	510 SF	2000.00 CF	74.07 CY	27.2	13600
Retaining Footing - 36" x 36" x 48"	Retaining	1	10.00	80.50	4.00	805 SF	769 SF	3220.13 CF	119.26 CY	27.2	21897
Retaining Footing - 36" x 36" x 48"	Retaining	1	10.00	50.00	4.00	500 SF	510 SF	2000.00 CF	74.07 CY	27.2	13600
Retaining Footing - 36" x 36" x 48"	Retaining	1	10.00	80.50	4.00	805 SF	769 SF	3220.13 CF	119.26 CY	27.2	21897
Retaining Footing - 36" x 36" x 48": 4						2610 SF	2559 SF	10440.25 CF	386.68 CY		70904
Retaining Footing - 60" x 60" x 48"	Retaining	1	13.00	123.00	4.00	1699 SF	1156 SF	6396.00 CF	236.89 CY	27.2	43493
Retaining Footing - 60" x 60" x 48"	Retaining	1	13.00	123.00	4.00	1599 SF	1156 SF	6396.00 CF	236.89 CY	27.2	43493
Retaining Footing - 60" x 60" x 48"	Retaining	1	13.00	123.00	4.00	1599 SF	1156 SF	6396.00 CF	236.89 CY	27.2	43493
Retaining Footing - 60" x 60" x 48"	Retaining	1	13.00	123.00	4.00	1699 SF	1156 SF	6396.00 CF	236.89 CY	27.2	43493
Retaining Footing - 60" x 60" x 48"	Retaining	1	13.00	80.00	4.00	1040 SF	791 SF	4160.00 CF	154.07 CY	27.2	28285
Retaining Footing - 60" x 60" x 48"	Retaining	1	13.00	80.00	4.00	1040 SF	791 SF	4160.00 CF	154.07 CY	27.2	28285
Retaining Footing - 60" x 60" x 48"	Retaining	1	13.00	80.00	4.00	1040 SF	791 SF	4160.00 CF	154.07 CY	27.2	28285
Retaining Footing - 60" x 60" x 48"	Retaining	1	13.00	80.00	4.00	1040 SF	791 SF	4160.00 CF	154.07 CY	27.2	28285
Retaining Footing - 60" x 60" x 48": 8						10560 SF	7756 SF	42224.00 CF	1563.85 CY		287123
Retaining Footing - 72" x 72" x 48"	Retaining	1	16.00	83.00	4.00	1328 SF	842 SF	5312.00 CF	196.74 CY	27.2	36122
Retaining Footing - 72" x 72" x 48"	Retaining	1	16.00	80.50	4.00	1288 SF	820 SF	5152.22 CF	190.82 CY	27.2	35035
Retaining Footing - 72" x 72" x 48"	Retaining	1	16.00	83.00	4.00	1328 SF	842 SF	5312.00 CF	196.74 CY	27.2	36122
Retaining Footing - 72" x 72" x 48"	Retaining	1	16.00	80.50	4.00	1288 SF	820 SF	5152.22 CF	190.82 CY	27.2	35035
Retaining Footing - 72" x 72" x 48": 4						5232 SF	3324 SF	20908.43 CF	775.13 CY		142313
Retaining Footing - 108" x 108" x 48"	Retaining	1	22.00	67.00	4.00	1474 SF	767 SF	5896.00 CF	218.37 CY	27.2	40093
Retaining Footing - 108" x 108" x 48"	Retaining	1	22.00	80.50	4.00	1771 SF	871 SF	7084.30 CF	262.38 CY	27.2	48173
Retaining Footing - 108" x 108" x 48"	Retaining	1	22.00	67.00	4.00	1474 SF	767 SF	5896.00 CF	218.37 CY	27.2	40093
Retaining Footing - 108" x 108" x 48"	Retaining	1	22.00	80.50	4.00	1771 SF	871 SF	7084.30 CF	262.38 CY	27.2	48173
Retaining Footing - 108" x 108" x 48": 4						6490 SF	3256 SF	25960.59 CF	961.50 CY		176532
Retaining Footing - 144" x 144" x 48"	Retaining	1	28.00	140.00	4.00	3920 SF	1428 SF	15680.00 CF	580.74 CY	27.2	106624
Retaining Footing - 144" x 144" x 48"	Retaining	1	28.00	80.50	4.00	2254 SF	922 SF	9016.35 CF	333.94 CY	27.2	61311
Retaining Footing - 144" x 144" x 48"	Retaining	1	28.00	140.00	4.00	3920 SF	1428 SF	15680.00 CF	580.74 CY	27.2	106624
Retaining Footing - 144" x 144" x 48"	Retaining	1	28.00	80.50	4.00	2254 SF	922 SF	9016.35 CF	333.94 CY	27.2	61311
Retaining Footing - 144" x 144" x 48": 4						12348 SF	4701 SF	49392.76 CF	1829.36 CY		335871
Grand total: 24						37237 SF	21624 SF	148946.04 CF	5516.52 CY		1012533

Floor Schedule

Type	Comments	Elevated	Type Comments	Structural Usage	Count	Thickness	Perimeter	Area	Forming	Volume	Volume (cy)	Est. Reinf.	Total Reinf (lbs)	Takeoff
12" Concrete	SOG	No	Apron Slab	Slab	1	1.00	632.00	20340 SF	632	20340.00 CF	753.33 CY	13.6	276624	Yes
Apron Slab: 1							632.00	20340 SF	632	20340.00 CF	753.33 CY		276624	
18" Concrete	Deck	Yes	Bridge	Slab	1	1.50	546.00	3976 SF	9699	5805.00 CF	215.00 CY	27.2	105264	Yes
18" Concrete	Deck	Yes	Bridge	Slab	1	1.50	126.00	720 SF	909	1080.00 CF	40.00 CY	27.2	19584	Yes
Bridge: 2							756.00	6310 SF	6607	7965.00 CF	255.00 CY		144432	
24" Concrete	Deck	Yes	Deck Slab	Slab	1	2.00	628.00	14507 SF	15763	41022.00 CF	1518.33 CY	27.2	394584	Yes
Deck Slab: 1							628.00	14507 SF	15763	41022.00 CF	1518.33 CY		394584	
6" Concrete	RipRap	No	Grading	Slab	1	0.50	506.00	10198 SF	253	5098.12 CF	0.00 CY	0	0	No
6" Concrete	RipRap	No	Grading	Slab	1	0.50	765.40	21978 SF	583	10988.81 CF	0.00 CY	0	0	No
6" Concrete	RipRap	No	Grading	Slab	1	0.50	506.00	10198 SF	253	5098.12 CF	0.00 CY	0	0	No
Grading: 3							1777.40	42370 SF	889	21185.15 CF	0.00 CY		0	
48" Concrete	SOG	No	Mat Foundation	Slab	1	4.00	680.00	20436 SF	2720	81744.00 CF	3027.56 CY	27.2	555859	Yes
Mat Foundation: 1							680.00	20436 SF	2720	81744.00 CF	3027.56 CY		555859	
Grand total: 9							4545.40	102963 SF	26510	172256.15 CF	5595.22 CY		1371900	

Structural Column Schedule

Type	Count	Length	Top Level	Base Level	Base Offset	Top Offset	Volume	Height
HP14X89	7790	330220.00	BO Mat	Pile Tip	0.00	11.00	6.81 CF	38
38.00: 7790	7790	330220.00						
HP14X89	178	7744.00	BO Mat	Pile Tip	0.00	27.00	7.89 CF	44
44.00: 178	178	7744.00						
HP14X89	144	17200.00	BO Mat	Pile Tip	0.00	13.00	8.96 CF	50
50.00: 144	144	7200.00						
HP14X89	283	14976.00	BO Mat	Pile Tip	0.00	15.00	9.32 CF	52
52.00: 288	288	14976.00						
HP14X89	76	4256.00		Pile Tip	0.00		10.04 CF	56
56.00: 76	76	4256.00						
Grand total: 1674	1474	64196.00						

02/14/11

PRELIMINARY - FOR DESIGN PURPOSES ONLY

#	BY	CHK	DATE	REVISION DESCRIPTION

DRAWING FW161E Roof DATE FILE NO.

CLIENT USE / CONSTRUCTION

RELEASED TO PCSE

A B C D 1 2 3

BARR

BARR ENGINEERING CO. 4700 WEST 77TH STREET MINNEAPOLIS, MN 55435 Ph: 1-(800)-632-2277 Fax: (952) 832-2601 www.barr.com

DESIGN: ISSUE DATE / PROJ. MGR. / DRAWN / CHECKED / DESIGNED / APPROVED

US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT

FARGO MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
PEARSBUTY STUDY, PHASE 6
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL -- "HEYENNE AQUEDUCT --
SCHEDULES

BARR PROJECT No. 34201004
CLIENT PROJECT No.
DWG No. S.07
REV No.

1605

K:\Jobs\34201004\Models\FM6Str\Sheets\Heyenne Aqueduct Schedules.dgn 2/14/2011 2:08:08 PM

Wall Schedule

Type	Type Comments	Comments	Length	Width	Height	Area	Forming	Volume	Volume CY	Reinf. (lbs/sf)	Total Reinf. (lbs)	Takeoff
Exterior - 72" Concrete	Aqueduct	Deck	259.00	6.00	1.50	387 SF	792 SF	1161.00 CF	43.00 CY	27.2	10526	
Exterior - 72" Concrete	Aqueduct	Deck	254.00	6.00	1.46	375 SF	768 SF	375.00 CF	13.89 CY	27.2	10200	
Exterior - 72" Concrete	Aqueduct	Deck	49.00	6.00	1.50	72 SF	162 SF	216.00 CF	8.00 CY	27.2	1958	
Exterior - 72" Concrete	Aqueduct	Deck	48.00	6.00	1.50	72 SF	162 SF	216.00 CF	8.00 CY	27.2	1958	
Exterior - 72" Concrete	Aqueduct	Deck	50.00	6.00	1.44	72 SF	161 SF	72.00 CF	2.67 CY	27.2	1958	
Exterior - 72" Concrete	Aqueduct	Deck	50.00	6.00	1.44	72 SF	161 SF	72.00 CF	2.67 CY	27.2	1958	
Deck: 8			708.00			1050 SF	2206 SF	2112.00 CF	78.22 CY		28560	
Exterior - 12" Concrete	Aqueduct	Low Flow Walls	250.05	1.00	10.05	2501 SF	5022 SF	2500.49 CF	92.61 CY	27.2	68026	
Exterior - 12" Concrete	Aqueduct	Low Flow Walls	250.05	1.00	10.00	2501 SF	5022 SF	2500.49 CF	92.61 CY	27.2	68026	
Exterior - 12" Concrete	Aqueduct	Low Flow Walls	35.46	1.00	5.29	187 SF	385 SF	186.32 CF	6.90 CY	27.2	5098	
Exterior - 12" Concrete	Aqueduct	Low Flow Walls	35.46	1.00	5.29	187 SF	385 SF	186.32 CF	6.90 CY	27.2	5098	
Exterior - 12" Concrete	Aqueduct	Low Flow Walls	35.46	1.00	9.80	346 SF	715 SF	339.55 CF	12.58 CY	27.2	9456	
Exterior - 12" Concrete	Aqueduct	Low Flow Walls	35.46	1.00	9.80	346 SF	715 SF	339.55 CF	12.58 CY	27.2	9456	
Low Flow Walls: 6			541.02			8072 SF	17245 SF	8052.70 CF	224.17 CY		185160	
Exterior - 36" Concrete	Aqueduct	Pier	56.00	3.00	13.02	729 SF	1536 SF	2187.36 CF	81.01 CY	27.2	19832	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aqueduct	Pier	56.00	3.00	13.02	729 SF	1536 SF	2187.36 CF	81.01 CY	27.2	19832	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aqueduct	Pier	56.00	3.00	13.02	729 SF	1536 SF	2187.36 CF	81.01 CY	27.2	19832	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aqueduct	Pier	56.00	3.00	13.02	729 SF	1536 SF	2187.36 CF	81.01 CY	27.2	19832	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aqueduct	Pier	56.00	3.00	13.02	729 SF	1536 SF	2187.36 CF	81.01 CY	27.2	19832	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aqueduct	Pier	56.00	3.00	13.02	729 SF	1536 SF	2187.36 CF	81.01 CY	27.2	19832	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	16.16	178 SF	452 SF	533.28 CF	19.75 CY	27.2	4835	
Pier: 21			546.00			7562 SF	17099 SF	20777.44 CF	843.61 CY		208515	

Wall Schedule

Type	Type Comments	Comments	Length	Width	Height	Area	Forming	Volume	Volume CY	Reinf. (lbs/sf)	Total Reinf. (lbs)	Takeoff
Exterior - 36" Concrete	Aqueduct	Wall	254.00	3.00	15.06	3825 SF	7740 SF	11470.00 CF	425.00 CY	27.2	104040	
Exterior - 36" Concrete	Aqueduct	Wall	254.00	3.00	15.06	3825 SF	7740 SF	11470.00 CF	425.00 CY	27.2	104040	
Wall: 2			508.00			7650 SF	15481 SF	22960.00 CF	850.00 CY		208080	
Aqueduct: 35			2403.92			22365 SF	47021 SF	93662.14 CF	1906.01 CY		608318	
Retaining - 36" Concrete	Retaining	Wall	125.00	3.00	18.03	2253 SF	4615 SF	6760.98 CF	250.37 CY	27.2	61291	
Retaining - 36" Concrete	Retaining	Wall	125.00	3.00	18.03	2253 SF	4615 SF	6760.98 CF	250.37 CY	27.2	61291	
Exterior - 48" Concrete	Retaining	Wall	12.50	4.00	34.52	431 SF	1139 SF	1725.92 CF	63.92 CY	27.2	11736	
Exterior - 48" Concrete	Retaining	Wall	140.00	4.00	33.82	4735 SF	9740 SF	18936.29 CF	701.45 CY	27.2	128787	
Exterior - 48" Concrete	Retaining	Wall	12.50	4.00	35.84	448 SF	1183 SF	1791.82 CF	66.37 CY	27.2	12185	
Exterior - 48" Concrete	Retaining	Wall	67.00	4.00	27.82	1864 SF	3950 SF	7455.76 CF	276.14 CY	27.2	50699	
Exterior - 48" Concrete	Retaining	Wall	83.00	4.00	21.82	1811 SF	3797 SF	7244.24 CF	268.31 CY	27.2	49261	
Exterior - 48" Concrete	Retaining	Wall	50.00	4.00	15.82	791 SF	1709 SF	3164.00 CF	117.19 CY	27.2	21515	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	32.64	2627 SF	5518 SF	10407.49 CF	385.46 CY	27.2	71461	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	26.58	2140 SF	4492 SF	8475.40 CF	313.90 CY	27.2	58195	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	20.62	1652 SF	3468 SF	6543.31 CF	242.34 CY	27.2	44929	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	14.46	1164 SF	2444 SF	4611.22 CF	170.79 CY	27.2	31662	
Exterior - 48" Concrete	Retaining	Wall	83.00	4.00	14.96	793 SF	1708 SF	3172.00 CF	117.48 CY	27.2	21570	
Retaining - 36" Concrete	Retaining	Wall	80.00	3.00	13.74	1099 SF	2281 SF	3564.80 CF	94.99 CY	27.2	29898	
Retaining - 36" Concrete	Retaining	Wall	80.00	3.00	13.74	1099 SF	2281 SF	3564.80 CF	94.99 CY	27.2	29898	
Retaining - 36" Concrete	Retaining	Wall	80.00	3.00	13.74	1099 SF	2281 SF	3297.60 CF	122.13 CY	27.2	29898	
Retaining - 36" Concrete	Retaining	Wall	80.00	3.00	13.74	1099 SF	2281 SF	3297.60 CF	122.13 CY	27.2	29898	
Wall: 30			2300.03			50326 SF	105877 SF	165844.05 CF	6683.11 CY		1368850	
Retaining: 30			2300.03			50326 SF	105877 SF	165844.05 CF	6683.11 CY		1368850	
Grand total: 60			4703.95			72691 SF	152898 SF	239736.18 CF	6879.12 CY		1977196	

02/18/11

PRELIMINARY - FOR DESIGN PURPOSES ONLY

NO.	BY	CHK.	DATE	REVISION DESCRIPTION

SIGNATURE

PRINTED NAME

DATE REG. NO.

ISSUED FOR: Issue 2nd Submission

MILESTONE 10/2/09

DATE RELEASED

BARR

Project Office

BARR ENGINEERING CO
4700 WEST 77TH STREET
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Initial	Issue 2nd
Drawn	Author
Checked	Checker
Designed	Designer
Approved	Approver

US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT

TARGET METROPOLITAN FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 4
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL - SHEYENNE AQUEDUCT - SCHEDULES - II

BARR PROJECT No. 34091004

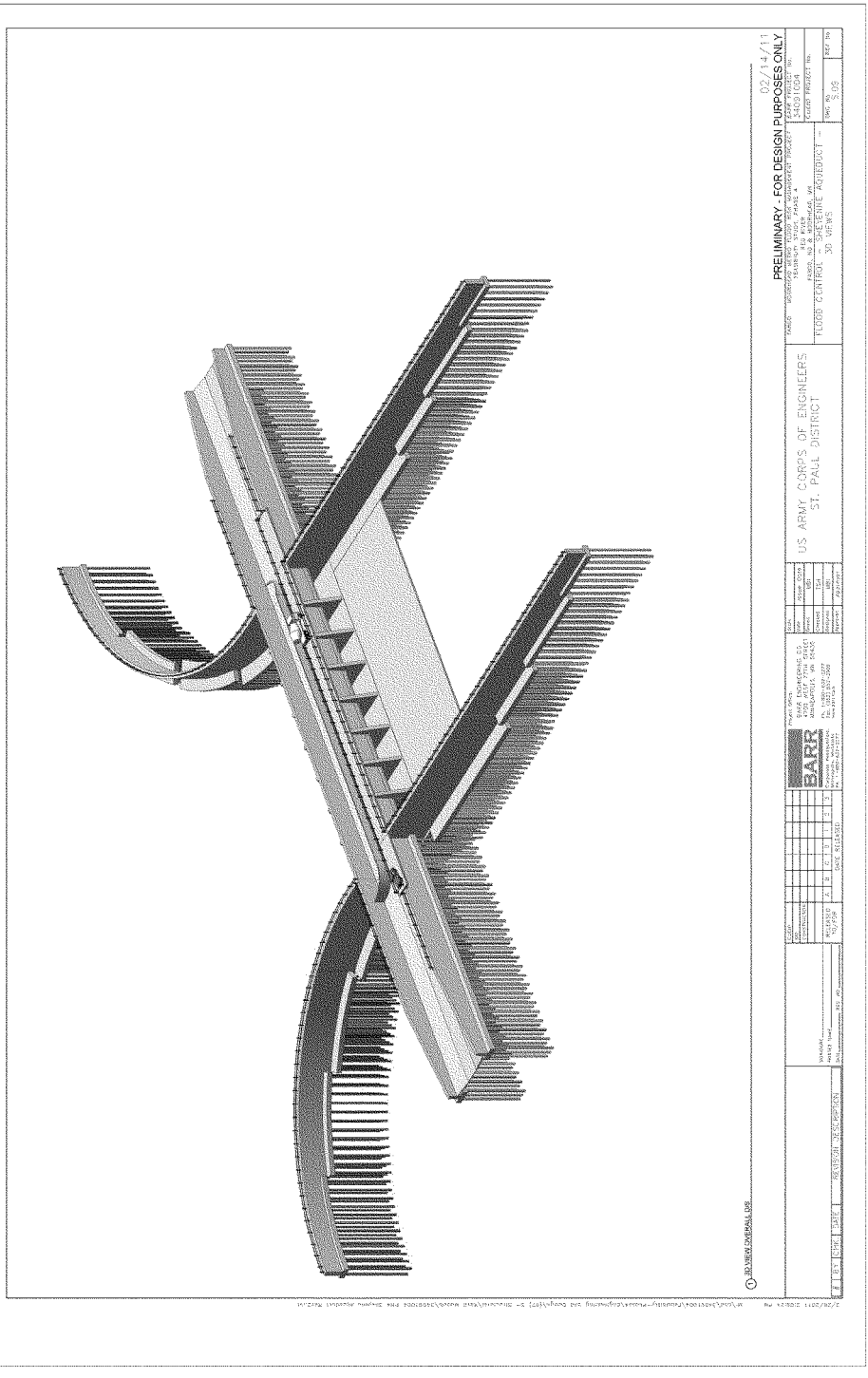
USACE PROJECT No.

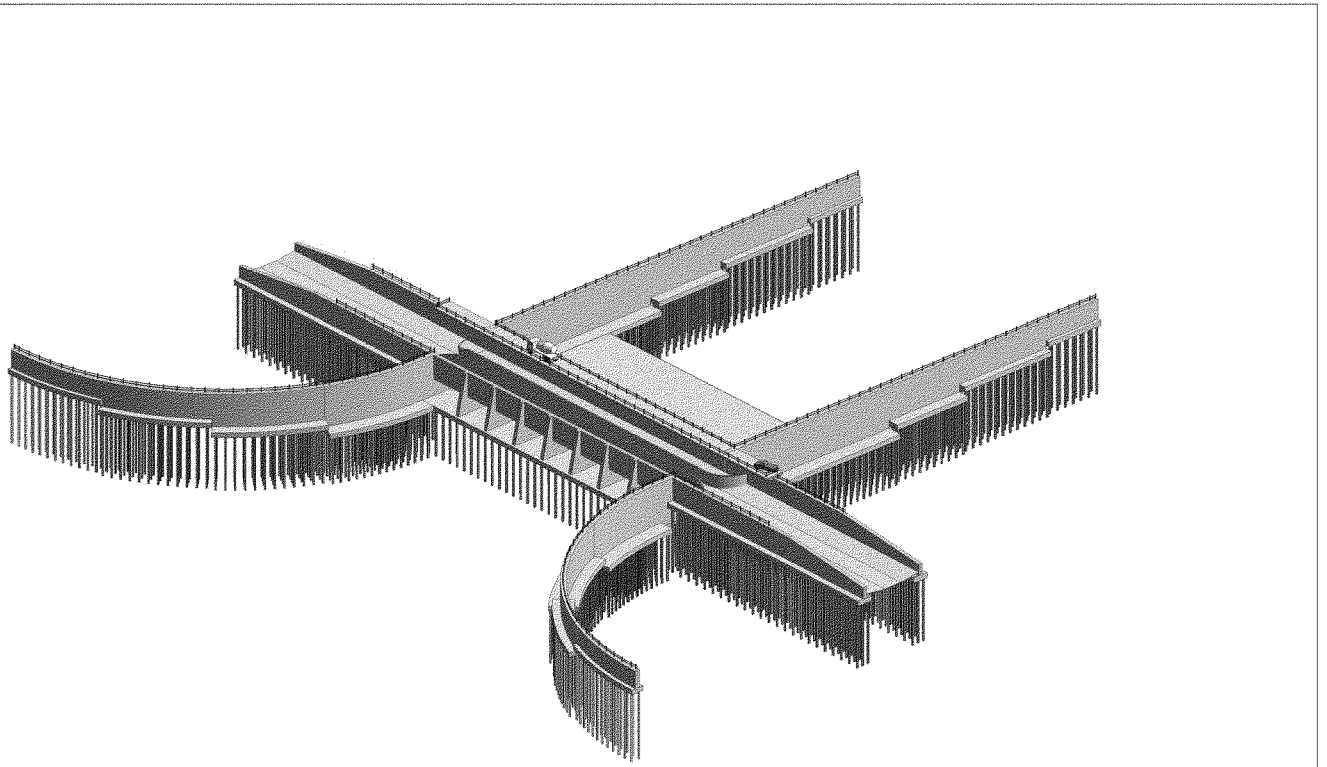
DWG No. S.02

REF No.

1606

Z:\56\34091004\Needs\Phase4\Engineering Lab Design\07) 2- Structural\Sheet Models\34091004 Phase Schedule Aqueduct Wall.rvt


PRELIMINARY - FOR DESIGN PURPOSES ONLY
02/14/11
US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
FLOOD CONTROL - SHERBURNE AQUEDUCT -
3D VIEWS
B.A.R.R.
3D VIEW OVERALL 08
REVISION DESCRIPTION



1 3D VIEW OVERALL U/S

02/26/11

PRELIMINARY - FOR DESIGN PURPOSES ONLY

NO	BY	CHK	DATE	REVISION DESCRIPTION

SIGNATURE

PRINTED NAME

DATE REG. NO.

ACCEPTED
03/08

DATE RELEASED

BARR

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Date	26 Feb 2010
Drawn	Author
Checked	Checker
Designed	Designer
Approved	Approver

US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT

FARGO MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 4
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL – SHEYENNE AQUEDUCT ~
3D VIEWS

BARR PROJECT NO.
34031004

CLIENT PROJECT NO.

DWG No.
3.1C

REV No.

ATTACHMENT

F-R5.1 Maple River Aqueduct Structure Pile Computations

LOAD CASES - MAPLE AQUEDUCT - Phase 4

Client Name:	U.S. ARMY CORPS OF ENGINEERS	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
Work Describtion:	LOAD CASES - MAPLE AQUEDUCT - Phase 4	1/24/2011
		34091004
File Path:	P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Maple\[34091004 PH4 Maple Pile Calcs.xlsx]Load Cases	

REF. 1
2

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tributary - Water El. (ft)	895.99	895.99	896.38	902	881.5	NA
Diversion - Head Water El. (ft)	893.89	893.89	895.46	902	NA	NA
Diversion - Tail Water El. (ft)	892.57	892.57	893.66	902	NA	NA
Tributary - T.O. Wall El. (ft)	902					
Tributary - T.O. Deck L.P. El.(ft)	881.06					
Tributary - T.O. Deck H.P. El.(ft)	883.06					
Diversion - T.O. Mat El. (ft)	872.06					
Tributary - Deck Slab thickness @ L.P. (ft)	2					
Tributary - Deck Slab thickness @ H.P. (ft)	4					
Diversion - Mat Slab thickness (ft)	4					
Tibutary - Water height (ft)	14.93	14.93	15.32	20.94	0.44	NA
Diversion - Head Water height (ft)	21.83	21.83	23.4	29.94	NA	NA
Ice	NA	2ft Ice	NA	NA	2ft Ice	NA
Ice Load	NA	10 kips/ft	NA	NA	10 kips/ft	NA
Ice Laod El. (ft)	NA	895.99	NA	NA	881.5	NA
Uplift @ HW (ft)	25.83	25.83	27.4	33.94	NA	NA
Uplift @ TW (ft)	24.51	24.51	25.6	33.94	NA	NA
Pile Condition	Undrained	Undrained	Undrained	Undrained	Drained	Undrained
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Safety Factors	2	1.5	1.5	1.15	2	1.5
Allwable Lateral Capacity (tons)	18	21	21	24	11.5	21
Allowable Pile Capacity (tons) - Axial	57.18	76.23	76.23	99.43	31.425	76.23
Allowable Pile Capacity (tons) - Uplift	33.88	45.17	45.17	58.91	4.625	45.17

Pile Capacity	Ultimate Axial Capacity (kips)	Allowable Lateral Capacity (kips)		
		0.5" (Usual)	0.67" (Unusual)	0.875" (Extreme)
Undrained - Axial	228.7	36	42	48
Undrained - Uplift	135.5			
Drained - Axial	125.7	23	29	33
Drained - Uplift	18.5			

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name:	U.S. ARMY CORPS OF ENGINEERS	Design By:	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	Review By:	
		Date:	1/24/2011
Work Description:	LOAD CASES - SHEYENNE AQUEDUCT - Phase 4	Job #:	34091004'
File Path:	P:\Mpls\34 NI\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Maple\[34091004 PH4 Maple File Calcs.xlsx]Load Cases		

REF.	
	1
	2

Hydrolic Profile

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tributary - Water El. (ft)	895.99	895.99	896.38	902	881.5	NA
Diversion - Head Water El. (ft)	893.89	893.89	895.46	902	NA	NA
Diversion - Tail Water El. (ft)	892.57	892.57	893.66	902	NA	NA
Tributary - T.O. Wall El. (ft)	902					
Tributary - T.O. Deck L.P. El.(ft)	881.06					
Tributary - T.O. Deck H.P. El.(ft)	883.06					
Diversion - T.O. Mat El. (ft)	872.06					
Tributary - Deck Slab thickness @ L.P. (ft)	2					
Tributary - Deck Slab thickness @ H.P. (ft)	4					
Diversion - Mat Slab thickness (ft)	4					
Tibutary - Water height (ft)	14.93	14.93	15.32	20.94	0.44	NA
Diversion - Head Water height (ft)	21.83	21.83	23.4	29.94	NA	NA
Ice	NA	2ft Ice	NA	NA	2ft Ice	NA
Ice Load	NA	10 kips/ft	NA	NA	10 kips/ft	NA
Ice Laod El. (ft)	NA	895.99	NA	NA	881.5	NA
Uplift @ HW (ft)	25.83	25.83	27.4	33.94	NA	NA
Uplift @ TW (ft)	24.51	24.51	25.6	33.94	NA	NA

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name:	U.S. ARMY CORPS OF ENGINEERS	Design By:	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	Review By:	
		Date:	1/24/2011
Work Description:	LOAD CASES - SHEYENNE AQUEDUCT - Phase 4	Job #:	34091004
File Path:	P:\Mpls\34 NDP\93400\004 Fargo Moorhead Metropolitan Feas. Study\Workfiles_Phase4\070 Structural\Acueducts\Maple\[34091004 PH4 Maple File Calcs v3 Load Cases		

Quantity Take Off (Revit)				Material Properties		
Volume of Walls (ft3)	Vw (ft3)	45558.53	ft3	*Concrete*	γ Concrete (pcf)	150
Volume Tributary Deck Slab (ft3)	Vs (ft3)	41022	ft3	*Steel*	γ Steel (pcf)	495
Volume of Bridge Deck (ft3)	Vs (ft3)	8127	ft3	*Soil Dry*	γs Dry (pcf)	120
Volume Diversion Mat Slab (ft3)	Vs (ft3)	81744	ft3	*Soil Saturated*	γs Sat. (pcf)	130
Total		**176451.53**	ft3	*Water*	γ Water (pcf)	62.4

Geometry

Tributary Channel			
Tributary - T.O. Wall El. (ft)		[illegible]	ft
Tributary - T.O. Deck L.P. El.(ft)		[illegible]	ft
Tributary - T.O. Deck H.P. El.(ft)		[illegible]	ft
Tributary - Clear Width (ft)	w TC	[illegible]	ft
Tributary - Wall Thickness (ft)	twall TC	[illegible]	ft
Tributary - Deck Slope Width (ft)	lslab slope TC	[illegible]	ft
Tributary - Deck Slab thickness @ L.P. (ft)	tslab TC	[illegible]	ft
Tributary - Deck Slab thickness @ H.P. (ft)	tslab TC	[illegible]	ft
Tributary - Low Flow Channel height (ft)	hlowflow TC	[illegible]	ft
Tributary - Low Flow Channel width (ft)	wlowflow TC	[illegible]	ft
Tributary - Low Flow Channel thickness (ft)	tlowwall TC	[illegible]	ft
Tributary - Wall Height (ft)	hwall TC	[illegible]	ft
Diversion Channel			
Diversion - T.O. Wall El. (ft)		[illegible]	ft
Diversion - T.O. Mat El. (ft)		[illegible]	ft
Diversion - Clear Opening Width	wopen DC	[illegible]	ft
Diversion - # of Openings	#open DC	[illegible]	
Diversion - Wall Thickness (ft)	twall DC	[illegible]	ft
Diversion - Mat Slab thickness (ft)	tslab DC	[illegible]	ft
Diversion - Butress height (ft)	hbutress	[illegible]	ft
Diversion - Butress Top width (ft)	wbutress Top	[illegible]	ft
Diversion - Butress Top width (ft)	wbutress Bot	[illegible]	ft
Diversion - Wall Height (ft)	hwall DC	[illegible]	ft
Mat Foundation			
Overall Width (ft)	wmat	**78**	ft
Overall Length (ft)	lmat	**262**	ft
Triburtary - Channel Length (ft)	lslab TC	**258**	ft
Triburtary - Channel Width (ft)	wslab TC	**56**	ft
Access Bridge			
Overall Width (ft)	wbridge	15	ft
Overoll Length (ft)	lbridge	**258**	ft
Minimum Deck Thickness	tbridge	1.5	ft
Maximum Deck Thickness	tbridge	3	ft

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name:	U.S. ARMY CORPS OF ENGINEERS	Design By:	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	Review By:	
		Date:	1/24/2011
Work Describtion:	LOAD CASES - SHEYENNE AQUEDUCT - Phase 4	Job #:	34091004
File Path:	P:\Mpls\34_NDS\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFile_Phase4\004 Structural\Aqueducts\Maple\[34091004 PH4 Maple File Calcs.xls]Load Cases		

Weight of Structure

Tibutary	Volume (ft3)	Weight (tons)
Walls	29319	2199
Deck	41022	3077
Low Flow Chanel	2064	155
Bridge	8127	860
Sub Total	80532	6290

Diversion	Volume (ft3)	Weight (tons)
Walls	9016	676
Mat	81744	6131
Butress Walls	4940	371
Sub Total	95700	7178
Total	176232	13217

Whole Structure	Volume (ft3)	Weight (tons)
Walls	45339	3400
Deck	41022	3077
Bridge	8127	860
Mat	81744	6131
Total	176232	13217

Take off (Revit)	Volume (ft3)	Weight (tons)
Walls	45559	3417
Deck	41022	3077
Bridge	8127	610
Mat	81744	6131
Total	176452	13234

Ratio 0.998757109

Forces

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Categary	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tibutary - Water height (ft)	14.93	14.93	15.32	20.94	0.44	NA
Triburtary - Channel Length (ft)	258.00					
Tributary - Clear Width (ft)	50.00					
Tributary - Water force (psf)	931.63	931.63	955.97	1306.66	27.46	NA
Tributary - Water Volume (ft3)	192597.00	192597.00	197628.00	270126.00	454.08	NA
Tributary - Water Weight (tons)	6009.03	6009.03	6165.99	8427.93	14.17	NA
Diversion - Head Water height (ft)	21.83	21.83	23.40	29.94	NA	NA
Mat Foundation - Overall Width (ft)	78.00					
Mat Foundation - Cleor Length (ft)	162.00					
Diversion - Water force (psf)	1362.19	1362.19	1460.16	1868.26	NA	NA
Diversion - Water Volume (ft3)	275843.88	275843.88	295682.40	378321.84	NA	NA
Diversion - Water Weight (tons)	8606.33	8606.33	9225.29	11803.64	NA	NA
Total Water Weight on the Structure (tons)	14615.36	14615.36	15391.28	20231.57	14.17	NA
Tributary - Uplift on the Deck (ft)	14.83	14.83	16.40	22.94	NA	NA
Tributary - Uplift force (psf)	925.39	925.39	1023.36	1431.46	NA	NA
Tributary - Uplift force (tons)	-6685.03	-6685.03	-7392.75	-10340.84	NA	NA
Uplift @ HW (ft)	25.83	25.83	27.40	33.94	NA	NA
Uplift @ TW (ft)	24.51	24.51	25.60	33.94	NA	NA
Diversion - Uplift force on the Mat (psf)	1570.61	1570.61	1653.60	2117.86	NA	NA
Diversion - Uplift force on the Mat (tons)	-16048.47	-16048.47	-16896.48	-21640.25	NA	NA
Total Uplif Force on the Structure (tons)	-22733.50	-22733.50	-24289.24	-31981.09	NA	NA
Weight of Structure (tons)	13233.9					

LOAD CASES - SHEYENNE AQUEDUCT - Phase 4

Client Name:	U.S. ARMY CORPS OF ENGINEERS	Design By:	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	Review By:	
		Date:	1/24/2011
Work Describtion:	LOAD CASES - SHEYENNE AQUEDUCT - Phase 4	Job #:	34091004'
File Path:	P:\Mpls\34.ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Maple\[34091004 PH4 Maple Pile Calcs.xlsx]Load Cases		

Flotation						
ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Nome	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Downword force on the Structure (tons)	5115.72	5115.72	4335.91	1484.35	13248.03	13233.86
Uplift Ratio	1.23	1.23	1.18	1.05	NA	NA
Uplift Ratio (No woter in the Tributory)	0.96	0.96	0.92	0.78	NA	NA
Condition	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Safety Factors - Flotation	1.30	1.20	1.20	1.10	1.30	1.20
Check	NG!!!	OK	NG!!!	NG!!!	OK	OK

Pile Computation						
ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Pile Condition	Undrained	Undrained	Undrained	Undrained	Drained	Undrained
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Safety Factors	2	1.5	1.5	1.15	2	1.5
Allwable Lateral Capacity (tons)	18	21	21	24	11.5	21
Allowable Pile Capacity (tons) - Axial	57.18	76.23	76.23	99.43	31.43	76.23
# of Piles Required	89.47	67.11	56.88	14.93	421.58	173.60
Uniform Spacing	15.11	17.45	18.96	37.00	6.96	10.85
# of Columns (along length)	35.00	35.00	35.00	35.00	35.00	35.00
Pile Spacing (along length)	7.53	7.53	7.53	7.53	7.53	7.53
# of Rows (along width)	2.56	1.92	1.63	0.43	12.05	4.96
# of Rows (along widht) provided	13.00	13.00	13.00	13.00	13.00	13.00
Actuol Pile Spacing (along width)	6.00	6.00	6.00	6.00	6.00	6.00
Totol number of pile provided	455	455	455	455	455	455
Pile Lood	11.24	11.24	9.53	3.26	29.12	29.09
Utilization Ratio	0.20	0.15	0.13	0.03	0.93	0.38
Check	OK	OK	OK	OK	OK	OK

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Final # of Piles	455					
Pile Load	11.24	11.24	9.53	3.26	29.12	29.09
Utilization Ratio	0.20	0.15	0.13	0.03	0.93	0.38
Check	OK	OK	OK	OK	OK	OK

Fargo-Moorhead Food Control Structures Preliminary Pile Foundation Analyses HP 14X73											
A_{Gross} = 198.5 in², A_{Steel} = 21.4 in², perimeter = 56.4 in, width (b) = 14.6 in, I = 729 in⁴											
Structure	Diversion Channel Station Location	Approximate Ground (Bank) Surface Elevation (ft)	Invert Elevation (ft)	Estimated Foundation Elevation (ft)	Estimated Ground Water Elevation (ft)	Design Condition/Tip Elevations		Ultimate Axial Capacity (kips)	Allowable Lateral Capacity (kips) (fixed head - single pile)			Estimated Settlement at allowable load
									0.5"	0.67"	0.875"	
Maple River Aqueduct Crossing Foundations	725+92	898	~~866.5~~ 872.06	~~862.45~~ 868.06	893	Undrained Analysis 827.5'	Total	228.7	36	42	48	<0.5"
							Uplift Resistance	135.5				
						Drained Analysis	Total	125.7	23	29	33	
							Uplift Resistance	18.5				

See following link for details
P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles\ Phase4\060 Geotech\Deep Foundations

ATTACHMENT

F-R5.2 Maple River Aqueduct Structure Retaining Wall Panel E Computations

MAPLE AQUADUCT STRUCTURE

Client Name:	U.S. ARMY CORPS OF ENGINEERS	Design By:	MBI
Project Name:	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	Review By:	
Work Describtion:	Maple Aquaduct Structure - Retaining Walls	Date:	2/10/2011
	Panel E	Job #:	34091004
File Path:	P:\Mpls\34 ND\09\34091004 Fargo Moorhead Metropolitan Feas. Study\WorkFiles_Phase4\070 Structural\Aqueducts\Maple\[34091004 PH4 Maple Retaining Walls Panel E.xlsx]Piling		
REF.	1		
	2		

ID#	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Tributary - Water El. (ft)	895.99	895.99	896.38	903.5	881.5	NA
Diversion - Head Water El. (ft)	893.89	893.89	895.46	903.5	NA	NA
Diversion - Tail Water El. (ft)	892.57	892.57	893.66	903.5	NA	NA
Tributary - T.O. Wall El. (ft)	903.5					
Tributary - T.O. Deck L.P. El.(ft)	881.06					
Tributary - T.O. Deck H.P. El.(ft)	883.06					
Diversion - T.O. Mat El. (ft)	872.06					
Tributary - Deck Slab thickness @ L.P. (ft)	2					
Tributary - Deck Slab thickness @ H.P. (ft)	4					
Diversion - Mat Slab thickness (ft)	4					
Tibutary - Water height (ft)	14.93	14.93	15.32	22.44	0.44	NA
Diversion - Head Water height (ft)	21.83	21.83	23.4	31.44	NA	NA
Ice	NA	2ft Ice	NA	NA	2ft Ice	NA
Ice Load	NA	10 kips/ft	NA	NA	10 kips/ft	NA
Ice Laod El. (ft)	NA	895.99	NA	NA	881.5	NA
Uplift @ HW (ft)	25.83	25.83	27.4	35.44	NA	NA
Uplift @ TW (ft)	24.51	24.51	25.6	35.44	NA	NA
Pile Condition	Undrained	Undrained	Undrained	Undrained	Drained	Undrained
Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Safety Factors	2	1.5	1.5	1.15	2	1.5
Allwable Lateral Capacity (tons)	18	21	21	24	11.5	21
Allowable Pile Capacity (tons) - Axial	57.18	76.23	76.23	99.43	31.425	76.23
Allowable Pile Capacity (tons) - Uplift	33.88	45.17	45.17	58.91	4.625	45.17

Pile Capacity	Ultimate Axial Capacity (kips)	Allowable Lateral Capacity (kips)		
		0.5" (Usual)	0.67" (Unusual)	0.875" (Extreme)
Undrained - Axial	228.7	36	42	48
Undrained - Uplift	135.5			
Drained - Axial	125.7	23	29	33
Drained - Uplift	18.5			

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls Panel E	

Monolith Structure

ITEM		UNIT	QUANITY	UNIT COST	TOTAL Cost
FURNISH HP14x73 WALL PILING		LF	3,297	0	$0
INSTALL HP14x73 WALL PILING		LF	3,297	0	$0
PILE TEST, 36.4 ft	Long	EA	6	0	$0
FOOTING CONCRETE		CY	618	0	$0
	Forming	SF	1,428		
STEM CONCRETE		CY	652	0	$0
	Forming	SF	9,127		
STEEL REINFORCEMENT		LB	244,455	0	$0
WALL RAILING		LF	140	0	$0
SHEET PILE CUT-OFF WALL		SF	1,400	0	$0
					$0

Structure Length = 140 ft

No. piles = 125 Each
Length = 26.38 ft

Note: HP14x73 pile used for design, use HP14x73 to allow for corrosion

LENGTH
(FRONT FACE) 10 FT
Native Soil has low permeability assume cut-off minimal to prevent scour

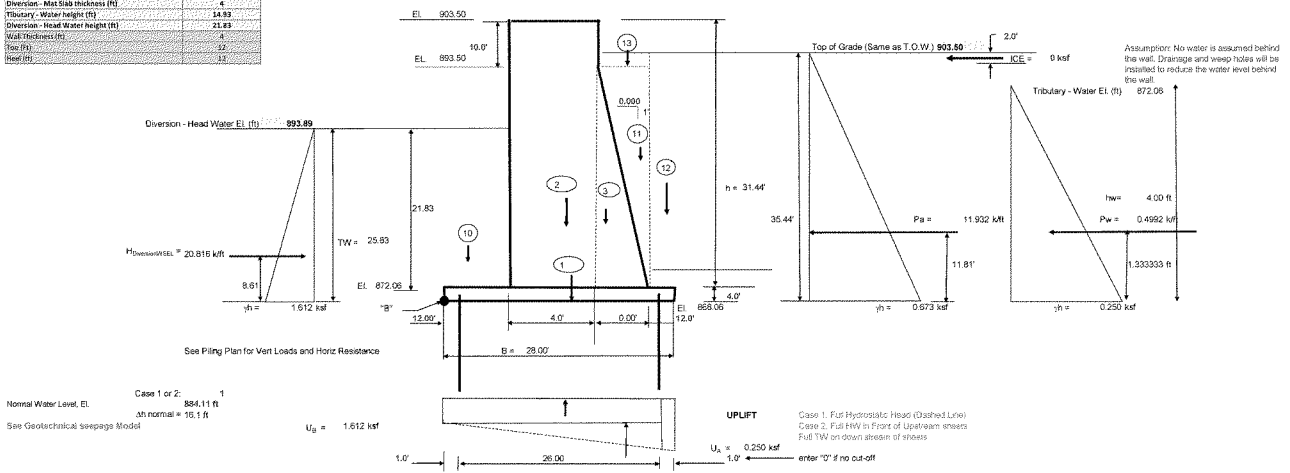
BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Maple Aqueduct Structure - Retaining Walls	
			Load Cases:	**Case 1 100 yr. flood**	**Panel E**

ID#	Case 1
Name	100 yr. flood
Load Category	Usual
Tributary - Water El. (ft)	895.99
Diversion - Head Water El. (ft)	893.89
Diversion - Tail Water El. (ft)	892.57
Tributary - T.O. Wall El. (ft)	903.5
Tributary - T.O. Deck L.P. El.(ft)	881.06
Tributary - T.O. Deck H.P. El.(ft)	883.06
Diversion - T.O. Mat El. (ft)	872.06
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	14.93
Diversion - Head Water height (ft)	21.83
Wall Thickness (ft)	4
Toe (ft)	12
Heel (ft)	12

File:

MN State Building Codes
Frost Depth = 5.0 ft — provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.60 ft — DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section — **Length =** 140.0 ft
Stepped Ftg Ls = 2.0 ft — overlap distance at stepped ftg



Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	31.44	0.15	rec	2641.0	14.00	36,973.4
Batter	3	140	0.00	21.44	0.15	tri	0.0	16.00	0.0
						D.L. Concrete	ΣVc = **4993.0**	ΣM_V =	**69,901.4** ← CONSTANT FOR ALL LOAD CASES
T.W on ftg Stem	10	140	12.00	21.83	0.0624	rec	2285.5	6.00	13,730.9
H.W. on Stem Slope	11	140	0.00	21.44	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	31.44	0.0626	rec	3306.5	22.00	72,742.6
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
						D.L. Water	ΣVw = **5595.0**	ΣM_V =	**86,473.5**

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	140	28.00	1.612	rec	-8318.2	14.00	-88,455
	U_A	140	28.00	-1.362	tri	2669.9	18.67	49,838
					ΣU =	**-3648.3**	ΣM_U =	**-38,617**

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls	
2/11/11				Load Cases: **Case 1 100 yr. flood**	**Panel E**

Horizontal Loads

		L	H	Pressure		**ICE**	arm	**Mu**
		ft	ft	ksf		K	ft	ft-k
	ICE	140	2.00	0.00	rec	0.0	34.44	0.0
		L		**Force**		**H**	arm	**Mw**
		ft		k/ft		K	ft	ft-k
	SOIL	140		-11.932		-1670.47	11.81	-19733.84
	Water Loads							
H_{TW}		140		20.816	tri	2914.28	8.61	25091.96
H_{HW}		140		-0.499	tri	-69.89	1.33	-93.18
					ΣWater =	**2844.39**	ΣM_W =	**5264.9**

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-33352	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	156375	kip-ft

Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**123,023**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**6,940**	kips
Sum of Horizontal Forces	H = Σhorizontal	**1,174**	kips

Location of Resultant	Xr = ΣM / P =	17.73	ft from Toe
	e = B/2 - Xr =	(3.73)	ft
	B/6 =	4.667	ft

CONCRETE QUANTITIES

				forming	
Ftg conc:	589	cy (includes stepped)		1428	sf
Stem Conc:	652	cy		9127	sf
Total =	1,241				

STEEL REINFORCEMENT: (assumed)

		Bar #	Spacing		Length	# of bars	Total wt		
a) Footing			in	LB /ft	ft	ea	lb		
Top mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
Bot mat	Transverse:	9	6	3.40	27.5	284	26,554		
	Longitudinal:	9	6	3.40	141.5	56	26,942		
							cy		**LB/cy**
							106,991	589	181.6374748
b) Skin Reinf. On Monolith									
Vert Face	Vertical	9	6	3.40	30.94	280	29,455	58,909.76	
	Longitudinal:	9	6	3.40	139.5	62	29,407	58,813.20	
Top Face	Transverse:	9	6	3.40	3.5	280	3,332		
	Longitudinal:	9	6	3.40	139.5	8	3,794		
Dowels	Vertical I.F.	9	6	3.40	30.9	280	29,455		
	Vertical O.F.	9	6	3.40	30.9	280	29,455		
							cy		**LB/cy**
							124,898	652	191.5346851
						Σ =	**231,889**		
	Lap Splices (long. Bars)	9	3.40	6		462	12,566		
						Σ Bar Wt=	244,455	lb	

FORCES AT THE BOTTOM OF THE STEM

Diversion Face	H	γ	Pbase	V	arm	**Mv**
	ft	kcf		K	ft	ft-k
Diversion WSEL	21.83	0.0624	1.362192	14.868	7.277	108.1918
Tributary SEL =	31.44	0.019	0.59736	9.390	10.490	98.41243
Tributary WSEL =	0.00	0.0624	0	0.000	0.000	0
Sum				9.390		98.41243
Net Forces				-5.478		-9.77942

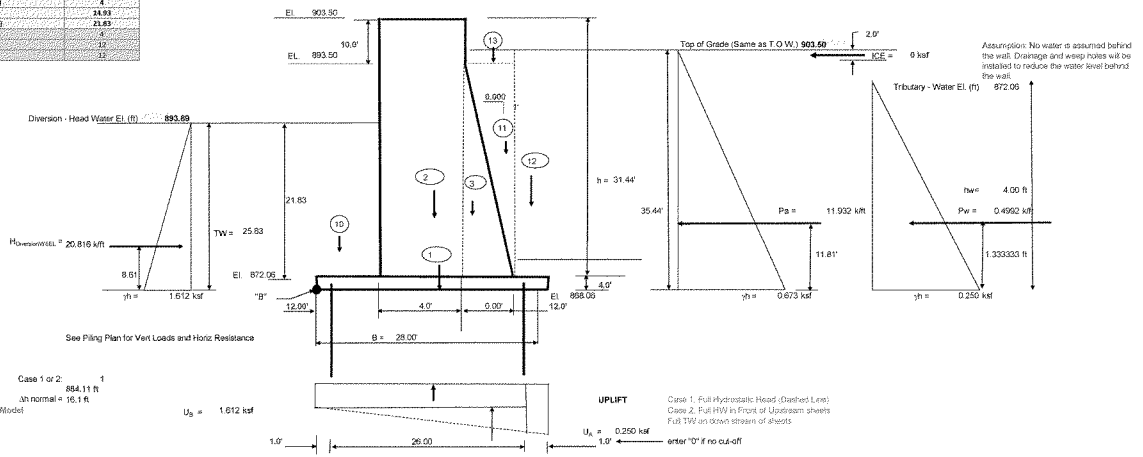
BARR ENGINEERING			DATE	2/11/2011	SHEET NO
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Maple Aqueduct Structure - Retaining Walls	
			Load Cases:	**Case 2 100 yr. flood + ice**	**Panel E**

ID#	Case 2
Name	100 yr. flood + ice
Load Category	Unusual
Tributary - Water El. (ft)	895.99
Diversion - Head Water El. (ft)	893.89
Diversion - Tail Water El. (ft)	892.57
Tributary - T.O. Wall El. (ft)	903.5
Tributary - T.O. Deck L.P. El.(ft)	881.06
Tributary - T.O. Deck H.P. El.(ft)	883.06
Diversion - T.O. Mat El. (ft)	872.06
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tributary - Water height (ft)	24.93
Diversion - Head Water height (ft)	21.83
Wall Thickness (ft)	4
Toe (ft)	12
Heel (ft)	12

File:

MN State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.50 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section **Length** = 140.0 ft
Stepped Ftg Lg = 2.0 ft overlap distance at stepped ftg



Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	31.44	0.15	rec	2641.0	14.00	36,973.4
Batter	3	140	0.00	21.44	0.15	tri	0.0	16.00	0.0
					D.L. Concrete	ΣVc =	**4993.0**	ΣM_V =	**69,901.4**
T.W on ftg Stem	10	140	12.00	21.83	0.0624	rec	2288.5	6.00	13,730.9
H.W. on Stem Slope	11	140	0.00	21.44	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	31.44	0.0626	rec	3306.5	22.00	72,742.6
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
					D.L. Water	ΣVw =	**5595.0**	ΣM_V =	**86,473.5**

CONSTANT FOR ALL LOAD CASES

Uplift Loads	L ft	W ft	Pressure ksf		**U** K	arm ft	Mu ft-k
U_B	140	28.00	1.612	rec	-6318.2	14.00	-88,455
U_A	140	28.00	-1.362	tri	2669.9	18.67	49,836
				ΣU =	**-3648.3**	ΣM_U =	**-38,617**

Horizontal Loads	L ft	H ft	Pressure ksf	**ICE** K	arm ft	Mu ft-k

BARR ENGINEERING			DATE	2/11/2011	SHEET NO
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34051004	
MBI		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls	
2/11/11				Load Cases: **Case 2 100 yr. flood + ice**	**Panel E**

ICE	140	2.00	0.00	rec	0.0	34.44	0.0	
	L		Force		**H**	arm	**Mw**	
	ft		k/ft		K	ft	ft-k	
SOIL	140		-11.932		-1670.47	11.81	-19733.84	
Water Loads								
H_{TW}	140		20.816	tri	2914.28	8.61	25091.96	
H_{HW}	140		-0.499	tri	-69.89	1.33	-93.18	
				ΣWater =	**2844.39**	ΣM_W =	**5264.9**	

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-33352	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	156375	kip-ft
Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**123,023**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**6,940**	kips
Sum of Horizontal Forces	H = ΣHorizontal	**1,174**	kips

Location of Resultant	Xr = ΣM / P =	17.73 ft from Toe
	e = B/2 - Xr =	(3.73) ft
	B/6 =	4.667 ft

FORCES AT THE BOTTOM OF THE STEM

	H	γ	Pbase	V	arm	Mv
Diversion Face	ft	kcf		K	ft	ft-k
Diversion WSEL	21.83	0.0624	1.362192	14.868	7.277	108.1918
Tributary SEL =	31.44	0.019	0.59736	9.390	10.480	98.41243
Tributary WSEL =	0.00	0.0624	0	0.000	0.000	0
Sum				9.390		98.41243
Net Forces				-5.478		-9.77942

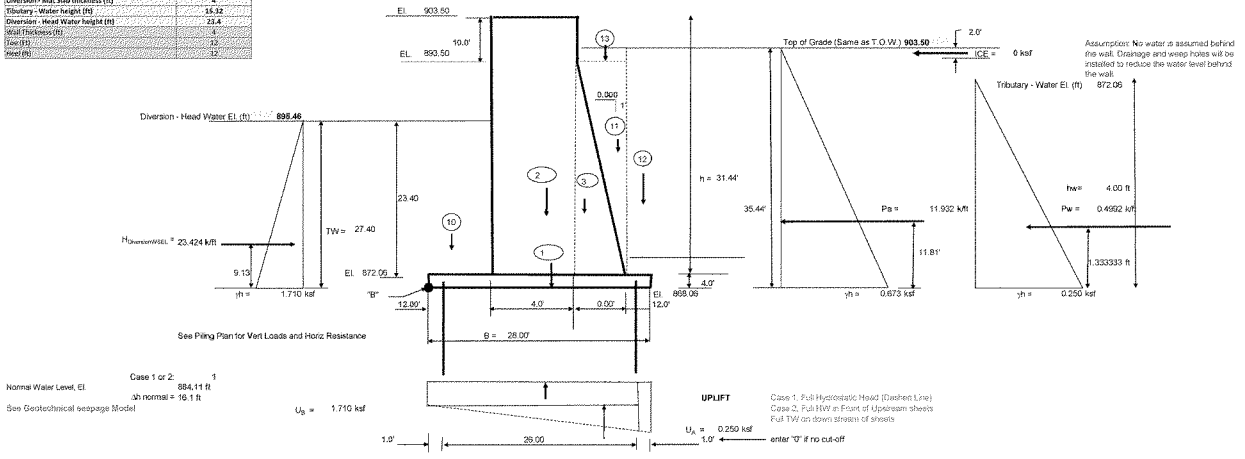
BARR ENGINEERING			DATE	2/11/2011	SHEET NO
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED MBI 2/11/11	CHECKED	SUBMITTED MBI	PROJECT NUMBER	34091004	
			SUBJECT	Maple Aquaduct Structure - Retaining Walls Load Cases: **Case 3 500 yr. flood** **Panel E**	

ID#	Case 3
Name	500 yr. flood
Load Category	Unusual
Tributary - Water El. (ft)	896.18
Diversion - Head Water El. (ft)	898.46
Diversion - Tail Water El. (ft)	893.66
Tributary - T.O. Wall El. (ft)	903.5
Tributary - T.O. Deck L.P. El.(ft)	881.06
Tributary - T.O. Deck H.P. El.(ft)	883.06
Diversion - T.O. Mat El. (ft)	872.06
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	15.32
Diversion - Head Water height (ft)	23.4
Wall Thickness (ft)	4
Toe (ft)	12
Heel (ft)	32

File:

MN State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.50 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section **Length** = 140.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	31.44	0.15	rec	2641.0	14.00	36,973.4
Batter	3	140	0.00	21.44	0.15	tri	0.0	16.00	0.0
						D.L. Concrete	ΣVc = **4993.0**	ΣMv =	**69,901.4**
T.W on ftg Stem	10	140	12.00	23.40	0.0624	rec	2493.1	6.00	14,718.4
H.W. on Stem Slope	11	140	0.00	21.44	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	31.44	0.0626	rec	3306.5	22.00	72,742.6
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
						D.L. Water	ΣVw = **5799.6**	ΣMv =	**87,461.0**

← CONSTANT FOR ALL LOAD CASES

Uplift Loads		L ft	W ft	Pressure ksf		**U** K	arm ft	Mu ft-k
	U_B	140	28.00	1.710	rec	-6702.3	14.00	-93,832
	U_A	140	28.00	-1.460	tri	2861.9	18.67	53,422
					SU =	**-3840.3**	ΣMu =	**-40,409**

Horizontal Loads	L ft	H ft	Pressure ksf	ICE K	arm ft	Mu ft-k

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls	
2/11/11				Load Cases: **Case 3 500 yr. flood**	**Panel E**

ICE	140	2.00	0.00	rec	0.0	34.44	0.0	
	L		Force		**H**	arm	**Mw**	
	ft		k/ft		K	ft	ft-k	
SOIL	140		-11.932		-1670.47	11.81'	-19733.84	
Water Loads								
H_{hw}	140		23.424	tri	3279.32	9.13	29951.12	
H_{hw}	140		-0.499	tri	-69.89	1.33	-93.18	
				ΣWater =	**3209.43**	ΣM_W =	**10124.1**	

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-30285	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	157362	kip-ft

Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**127,077**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**6,912**	kips
Sum of Horizontal Forces	H = Σhorizontal	**1,539**	kips

Location of Resultant	Xr = ΣM / P =	18.38 ft from Toe
	e = B/2 - Xr =	(4.38) ft
	B/6 =	4.667 ft

FORCES AT THE BOTTOM OF THE STEM

	H		γ	Pbase	**V**	arm	**Mv**
	ft		kcf		K	ft	ft-k
Diversion Face							
Diversion WSEL		23.40	0.0624	1.46016	17.084	7.800	133.2542
Tributary SEL =		31.44	0.019	0.59736	9.390	10.480	98.41243
Tributary WSEL =		0.00	0.0624	0	0.000	0.000	0
Sum					9.390		98.41243
Net Forces					-7.693		-34.8418

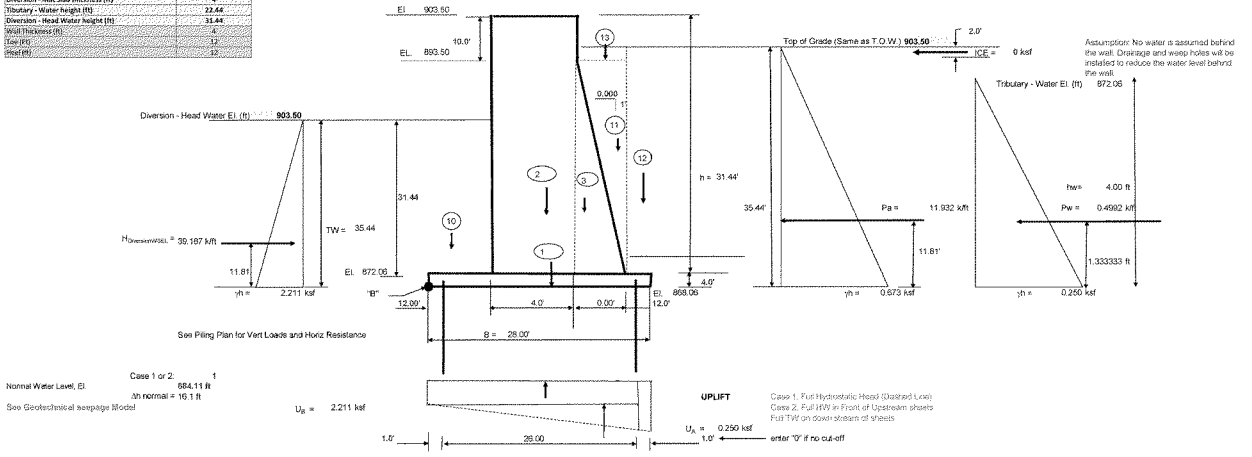
BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED MBJ 2/11/11	CHECKED	SUBMITTED MBJ	PROJECT NUMBER	34091004	
			SUBJECT	Maple Aqueduct Structure - Retaining Walls Load Cases: **Case 4 T.O. Levee** **Panel E**	

ID#	Case 4
Name	T.O. Levee
Load Category	Extreme
Tributary - Water El. (ft)	NA
Diversion - Head Water El. (ft)	903.5
Diversion - Tail Water El. (ft)	903.5
Tributary - T.O. Wall El. (ft)	903.5
Tributary - T.O. Deck L.P. El.(ft)	881.06
Tributary - T.O. Deck H.P. El.(ft)	883.06
Diversion - T.O. Mat El. (ft)	872.06
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	22.44
Diversion - Head Water height (ft)	31.44
Wall Thickness (ft)	4
Toe (ft)	12
Heel (ft)	12

File:

MN State Building Codes
Frost Depth = 5.0 ft — provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.50 ft — DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section **Length** = 140.0 ft
Stepped Ftg Ls = 2.0 ft — overlap distance at stepped ftg



Vertical Loads		Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
	Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
	Stem	2	140	4.00	31.44	0.15	rec	2641.0	14.00	36,973.4
	Batter	3	140	0.00	21.44	0.15	tri	0.0	16.00	0.0
							D.L. Concrete	ΣVc = **4993.0**	ΣMv =	**69,901.4** ← CONSTANT FOR ALL LOAD CASES
	T.W on ftg Stem	10	140	12.00	31.44	0.0624	rec	3295.9	6.00	19,775.5
	H.W. on Stem Slope	11	140	0.00	21.44	0.12	tri	0.0	16.00	0.0
	H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
	Soil on Footing	12s	140	12.00	31.44	0.0826	rec	3306.5	22.00	72,742.6
	H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
							D.L. Water	ΣVw = **6602.4**	ΣMv =	**92,518.1**

Uplift Loads		L ft	W ft	Pressure ksf		LJ K	arm ft	Mu ft-k
	U_B	140	28.00	2.211	rec	-8668.9	14.00	-121,365
	U_A	140	28.00	-1.962	tri	3846.2	18.67	71,778
					ΣU =	**-4823.7**	ΣMu =	**-49,587**

Horizontal Loads	L ft	H ft	Pressure ksf	ICE K	arm ft	Mu ft-k

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls Load Cases: **Case 4 T.O. Levee**	**Panel E**

ICE	140	2.00	0.00		rec	0.0	34.44	0.0
	L		**Force**			**H**	arm	**Mw**
	ft		k/ft			K	ft	ft-k
SOIL	140		-11.932			-1670.47	11.81'	-19733.84
Water Loads								
H_{TW}	140		39.187		tri	5486.18	11.81	64810.07
H_{HW}	140		-0.499		tri	-69.89	1.33	-93.18
					ΣWater =	**5416.29**	ΣM_W =	**44983.1**

Overturning Moments	$\Sigma M_{OT} = M_O + M_W + M_{ICE}$ =	-4604	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	162420	kip-ft
Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**157,816**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**6,772**	kips
Sum of Horizontal Forces	H = Σhorizontal	**3,746**	kips
Location of Resultant	Xr = ΣM / P =	23.31 ft from Toe	
	e = B/2 - Xr =	(9.31) ft	
	B/6 =	4.667 ft	

FORCES AT THE BOTTOM OF THE STEM

	H		γ	Pbase	V	erm	Mv
	ft		kcf		K	ft	ft-k
Diversion Face							
Diversion WSEL		31.44	0.0624	1.961856	30.840	10.480	323.2071
Tributary SEL =		31.44	0.019	0.59736	9.390	10.480	98.41243
Tributary WSEL =		0.00	0.0624	0	0.000	0.000	0
Sum					9.390		98.41243
Net Forces					-21.450		-224.795

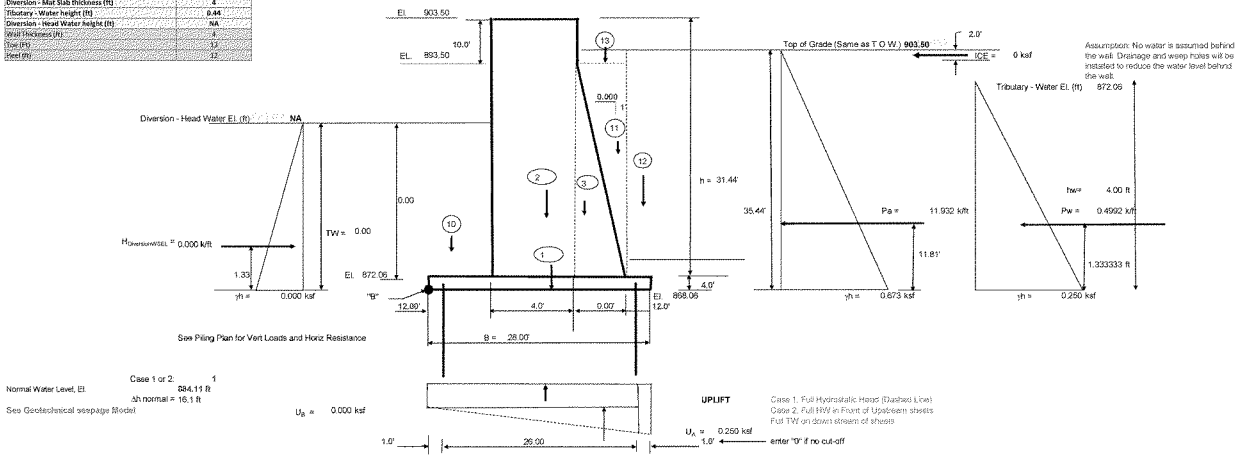
BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED MBI 2/11/11	CHECKED	SUBMITTED MBI	PROJECT NUMBER	34091004	
			SUBJECT	Maple Aqueduct Structure - Retaining Walls Load Cases: **Case 5 Normal flow + ice** **Panel E**	

ID#	Case 5
Name	Normal flow + ice
Load Category	Usual
Tributary - Water El. (ft)	881.5
Diversion - Head Water El. (ft)	NA
Diversion - Tail Water El. (ft)	NA
Tributary - T.O. Wall El. (ft)	903.5
Tributary - T.O. Deck L.P. El.(ft)	881.06
Tributary - T.O. Deck H.P. El.(ft)	893.06
Diversion - T.O. Mat El. (ft)	872.06
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	8.44
Diversion - Head Water height (ft)	NA
Wall Thickness (ft)	4
Toe (ft)	12
Heel (ft)	12

File:

MN State Building Codes
Frost Depth = 5.0 ft provide min frost flg protection during Dec, Jan, Feb, March
Water El. = 881.50 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section **Length** = 140.0 ft
Stepped Flg Ls = 2.0 ft overlap distance at stepped flg



Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Flg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	31.44	0.15	rec	2641.0	14.00	36,973.4
Batter	3	140	0.00	21.44	0.15	tri	0.0	16.00	0.0
					D.L. Concrete	ΣVc =	**4953.0**	ΣMv =	**69,901.4**

← CONSTANT FOR ALL LOAD CASES

T.W on flg Stem	10	140	12.00	0.00	0.0624	rec	0.0	6.00	0.0
H.W. on Stem Slope	11	140	0.00	21.44	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	31.44	0.0626	rec	3306.5	22.00	72,742.6
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
					D.L. Water	ΣVw =	**3306.5**	ΣMv =	**72,742.6**

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	140	28.00	0.000	rec	0.0	14.00	0
	U_A	140	28.00	0.250	tri	-489.2	18.67	-9,132
					ΣU =	**-489.2**	ΣMu =	**-9,132**

Horizontal Loads	L ft	H ft	Pressure ksf	ICE K	arm ft	Mu ft-k

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls	
2/11/11				Load Cases: **Case 5 Normal flow + ice** **Panel E**	

ICE	140	2.00	0.00	rec	0.0	34.44	0.0	
	L		**Force**		**H**	arm	**Mw**	
	ft		k/ft		K	ft	ft-k	
SOIL	140		-11.932		-1670.47	11.81'	-19733.84	
Water Loads								
H_{TW}	140		0.000	tri	0.00	1.33	0.00	
H_{HW}	140		-0.499	tri	-69.89	0.00	0.00	
				ΣWater =	**-69.89**	ΣM_W =	**-19733.8**	

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-28866	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	142644	kip-ft
Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**113,778**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**7,810**	kips
Sum of Horizontal Forces	H = ΣHorizontal	**-1,740**	kips
Location of Resultant	Xr = ΣM / P =	14.57 ft from Toe	
	e = B/2 - Xr =	(0.57) ft	
	B/6 =	4.667 ft	

FORCES AT THE BOTTOM OF THE STEM

	H		γ	Pbase	V	arm	Mv
	ft		kcf		K	ft	ft-k
Diversion Face							
Diversion WSEL		0.00	0.0624	0	0.000	0.000	0
Tributary SEL =		31.44	0.019	0.59736	9.390	10.480	98.41243
Tributary WSEL =		0.00	0.0624	0	0.000	0.000	0
Sum					9.390		98.41243
Net Forces					9.390		98.41243

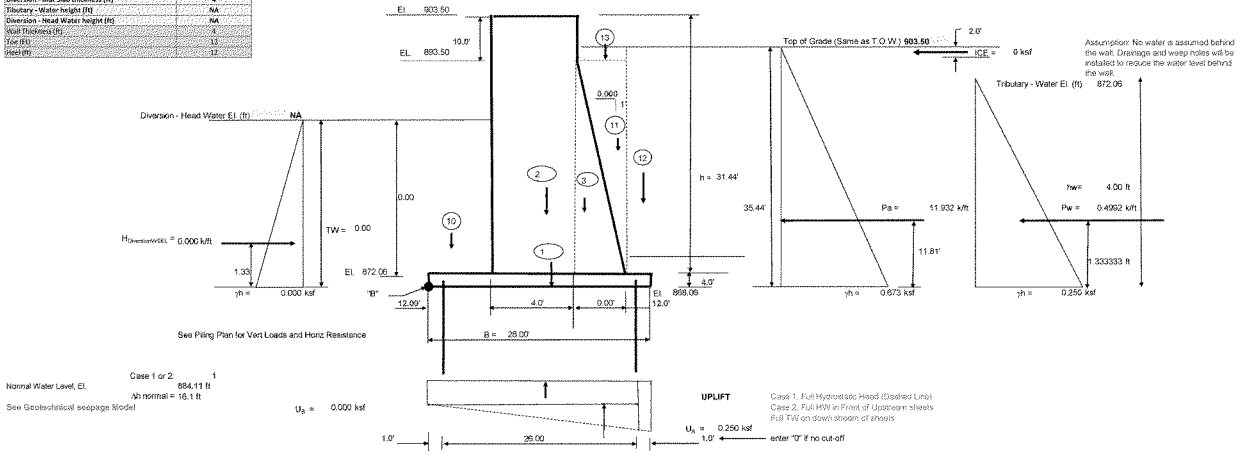
BARR ENGINEERING			DATE	2/11/2011	SHEET NO
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34951004	
MBI 2/11/11		MBI	SUBJECT	Maple Aqueduct Structure - Retaining Walls Load Cases: **Case 6 Construction** **Panel E**	

ID#	Case 6
Name	Construction
Load Category	Unusual
Tributary - Water El. (ft)	NA
Diversion - Head Water El. (ft)	NA
Diversion - Tail Water El. (ft)	NA
Tributary - T.O. Wall El. (ft)	903.5
Tributary - T.O. Deck L.P. El.(ft)	881.06
Tributary - T.O. Deck H.P. El.(ft)	881.06
Diversion - T.O. Mat El. (ft)	872.06
Tributary - Deck Slab thickness @ L.P. (ft)	2
Tributary - Deck Slab thickness @ H.P. (ft)	4
Diversion - Mat Slab thickness (ft)	4
Tibutary - Water height (ft)	NA
Diversion - Head Water height (ft)	NA
Wall Thickness (ft)	4
Toe (ft)	13
Heel (ft)	17

File:

MN State Building Codes
Frost Depth = 5.0 ft provide min frost ftg protection during Dec, Jan, Feb, March
Water El. = 881.50 ft DEC, JAN, FEB Mean Water Elevation

Non-Overflow Section **Length** = 140.0 ft
Stepped Ftg Ls = 2.0 ft overlap distance at stepped ftg



Vertical Loads	Section	L ft	W ft	H ft	γ kcf	shape	V K	arm ft	Mv ft-k
Ftg concrete	1	140	28.00	4.00	0.15	rec	2352.0	14.00	32,928.0
Stem	2	140	4.00	31.44	0.15	rec	2641.0	14.00	36,973.4
Batter	3	140	0.00	21.44	0.15	tri	0.0	16.00	0.0
						D.L. Concrete	ΣVc = **4983.0**	ΣMv =	**69,901.4**

← CONSTANT FOR ALL LOAD CASES

T.W on ftg Stem	10	140	12.00	0.00	0.0624	rec	0.0	6.00	0.0
H.W. on Stem Slope	11	140	0.00	21.44	0.12	tri	0.0	16.00	0.0
H.W. Above Slope	13	140	0.00	10.00	0.12	rec	0.0	16.00	0.0
Soil on Footing	12s	140	12.00	31.44	0.0626	rec	3306.5	22.00	72,742.6
H.W. on Footing	12w	140	12.00	0.00	0.0624	rec	0.0	22.00	0.0
						D.L. Water	ΣVw = **3306.5**	ΣMv =	**72,742.6**

Uplift Loads		L ft	W ft	Pressure ksf		U K	arm ft	Mu ft-k
	U_B	140	28.00	0.000	rec	0.0	14.00	0
	U_A	140	28.00	0.250	tri	-489.2	18.67	-9,132
					ΣU =	**-489.2**	ΣMu =	**-9,132**

Horizontal Loads	L ft	H ft	Pressure ksf	ICE K	arm ft	Mu ft-k

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls Load Cases: **Case 6 Construction**	**Panel E**

ICE	140	2.00	0.00	rec	0.0	34.44	0.0
	L		**Force**		**H**	arm	**Mw**
	ft		k/ft		K	ft	ft-k
SOIL	140		-11.932		-1670.47	11.81'	-19733.84
Water Loads							
H_{TW}	140		0.000	tri	0.00	1.33	0.00
H_{HW}	140		-0.499	tri	-69.89	1.33	-93.18
				ΣWater =	**-69.89**	ΣM_W =	**-19827.0**

Overturning Moments	$\Sigma M_{OT} = M_U + M_W + M_{ICE}$ =	-28959	kip-ft
Resisting Moments	$\Sigma M_R = M_V$ =	142644	kip-ft

Sum of Moments	$\Sigma Mnet = M_R + M_{OT}$ =	**113,685**	kip-ft
Sum of Vertical Forces	P = Conc + Water + Uplift =	**7,810**	kips
Sum of Horizontal Forces	H = Σhorizontal	**-1,740**	kips

Location of Resultant	Xr = ΣM / P =	14.56 ft from Toe
	e = B/2 - Xr =	(0.56) ft
	B/6 =	4.667 ft

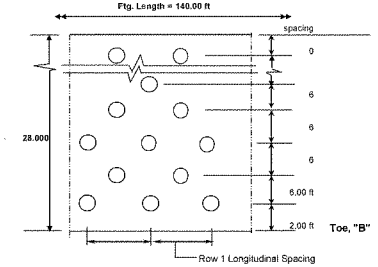
FORCES AT THE BOTTOM OF THE STEM

Diversion Face	H		γ	Pbase	**V**	arm	**Mv**
	ft		kcf		K	ft	ft-k
Diversion WSEL		0.00	0.0624	0	0.000	0.000	0
Tributary SEL =		31.44	0.019	0.59736	9.390	10.480	98.41243
Tributary WSEL =		0.00	0.0624	0	0.000	0.000	0
Sum					9.390		98.41243
Net Forces					9.390		98.41243

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED MBI 2/11/11	CHECKED	SUBMITTED MBI	PROJECT NUMBER	34091004	
			SUBJECT	Maple Aquaduct Structure - Retaining Walls Panel E	

PILE FOUNDATION DESIGN

FLOW



L = 140.00 ft

PILE PATTERN GEOMETRY

	Transverse Spacing	Distance to Toe, d_{toe}	Longitudinal Spacing	Batter	Piles per Row (N)	Edge Dist (ft)		Trial N
Row 1 to Toe	2.00 ft	2.0 ft	2.50 ft	0 "/12"	23	42.50	1	57
Row 1 to Row 2	6.00 ft	8.0 ft	5.00 ft	0 "/12"	24	12.50	2	29
Row 2 to Row 3	6.00 ft	14.0 ft	5.00 ft	0 "/12"	24	12.50	3	29
Row 3 to Row 4	6.00 ft	20.0 ft	5.00 ft	0 "/12"	26	7.50	4	29
Row 4 to Row 5	6.00 ft	26.0 ft	0.00 ft	0 "/12"	28	70.00	5	#DIV/0!
Row 5 to Row 6	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 6 to Row 7	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 7 to Row 8	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 8 to Row 9	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 9 to Row 10	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 10 to Row 11	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 11 to Row 12	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 12 to Row 13	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 13 to Row 14	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Row 14 to Row 15	0.00 ft	0.0 ft	0.00 ft	0 "/12"	0	70.00	0	0
Last Row to Heel	2.00 ft							
	28.00 ft		Note: Enter 0 for Longitudinal Spacing for Rows Not Used)		ΣN = 125			#DIV/0!

Pile Properties: Pile Type: HP (C.I.P or HP) Pile Length = 26.4 ft

HP Nominal Depth, h = 14.0 in

Wt. per ft, plf 73

Total pile Length = 3,298 LF

Ftg El. 868.06

Pile Tip El. 842.68

Pile Cap Embed = 1.00 ft

Pile Group Properties

N.A. of Pile Group to Toe

$X_{NA} = (\Sigma N * d_{toe}) / \Sigma N =$ **14.58 ft**

Dist. From N.A. to Pile Row	d	N	$I = N * d^2$
1 Dist. To Row 1	12.58 ft	23	3637.6
2 Dist. To Row 2	6.58 ft	24	1037.9
3 Dist. Row 3	0.58 ft	24	8.0
4 Dist. Row 4	-5.42 ft	26	764.9
5 Dist. Row 5	-11.42 ft	28	3654.2
6 Row 6 (not used)	0.00 ft	0	0.0
7 Row 7 (not used)	0.00 ft	0	0.0
8 Row 8 (not used)	0.00 ft	0	0.0
9 Row 9 (not used)	0.00 ft	0	0.0
10 Row 10 (not used)	0.00 ft	0	0.0
11 Row 11 (not used)	0.00 ft	0	0.0
12 Row 12 (not used)	0.00 ft	0	0.0
13 Row 13 (not used)	0.00 ft	0	0.0
14 Row 14 (not used)	0.00 ft	0	0.0
15 Row 15 (not used)	0.00 ft	0	0.0
		125	$\Sigma I =$ 9192.5

ALLOWABLE LOADS (from Geotechnical)

	Service	Allowable Pile Loads					
	DW	Case 1	Case 2	Case 3	Case 4	Case 5	Case 6
	Name	100 yr. flood	100 yr. flood + ice	500 yr. flood	T.O. Levee	Normal flow + ice	Construction
	Load Category	Usual	Unusual	Unusual	Extreme	Usual	Unusual
Allowable Lateral Capacity (tons)		18.0 tons	21.0 tons	21.0 tons	24.0 tons	11.5 tons	21.0 tons
Allowable Pile Capacity (tons) - Axial		57.2 tons	76.2 tons	76.2 tons	99.4 tons	31.4 tons	76.2 tons
Safety Factors		2.00	1.50	1.50	1.15	2.00	1.50

w/o Group effects

Summary Pile Reactions

Load Combinations	Pile Loads (tons/pile)												Max. Vertical Load (Tons)	Horiz Pile Group Capacity (k)	Check
Allowable Pile Capacity (tons) - Axial	1	2	3	4	5	6	7	8	9	10	11	12			
Case 1 **57.2 tons**	12.6	19.9	27.1	34.3	41.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	41.5	4,500	**OK**
Case 2 **76.2 tons**	12.5	19.9	27.1	34.3	41.5	0.0	0.0	0.0	0.0	0.0	0.0	0.0	41.5	5,250	**OK**
Case 3 **76.2 tons**	9.5	18.1	26.8	35.5	44.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	44.2	5,250	**OK**
Case 4 **99.4 tons**	-13.7	5.7	25.2	44.7	64.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	64.2	6,000	**OK**
Case 5 **31.4 tons**	31.3	31.3	31.2	31.2	31.2	0.0	0.0	0.0	0.0	0.0	0.0	0.0	31.3	2,875	**OK**
Case 6 **76.2 tons**	31.3	31.3	31.2	31.2	31.1	0.0	0.0	0.0	0.0	0.0	0.0	0.0	31.3	2,875	**OK**

Max Service : P = **64.2**

Using solid mechanics equations adapted for discrete elements, the forces in the pile rows for different load combinations are determined.
The force in each pile row is found using:

$$\text{Pile Load} = P / N + M_{toe} / I$$

First, the moment about the toe must be translated to get the moment about the neutral axis of the pile group.

$$e_{toe} = M_{toe} / P$$

Then the eccentricity about the neutral axis of the pile group is

$$e_{NA} = X_{NA} - e_{toe}$$

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BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Maple Aquaduct Structure – Retaining Walls Panel E	

The moment about the neutral axis of the pile group becomes

$M_{NA} = P * e_{NA}$

For battered pile, the Vertical pile load needs to be transformed to the axial load along the pile axis

Paxial = 1.000 Pvert

FORCE RESULTANT (see Stability Analysis)

CASE	Event		Vertical Load P (kips)	Horizontal	ΣM_{OB} (kips	$e_{NA} = M_{NA} / P$	e_{NA} * X_{NA} - e_{NA}	$M_{NA} = P * e_{NA}$
Case 1	**100 yr. flood**	**Usual**	6,940	-1,174	123,023	17.73	-3.15	-21871
Case 2	**100 yr. flood + ice**	**Unusual**	6,940	-1,174	123,023	17.73	-3.15	-21871
Case 3	**500 yr. flood**	**Unusual**	6,912	-1,539	127,077	18.38	-3.81	-26326
Case 4	**T.O. Levee**	**Extreme**	6,772	-3,746	157,816	23.31	-8.73	-59112
Case 5	**Normal flow + ice**	**Usual**	7,810	1,740	113,778	14.57	0.01	64
Case 6	**Construction**	**Unusual**	7,810	1,740	113,685	14.56	0.02	157

SERVICE

Case **Case 1**

Flood Event **100 yr. flood**

Usual

Vertical Load, P = 6940 kips

Horizontal Load, H = -1174 kips

M_{NA} = -21871 kip-ft 125

Vertical Pile Loading	P / N	+	M_{NA}* d / Σ I	= Pile Loads		**Axial Pile Load**
1 Row 1	55.5		-30.2	25.3 kips/pile	12.6 tons/pile	12.6 tons/pile
2 Row 2	55.5		-15.8	39.7 kips/pile	19.9 tons/pile	19.9 tons/pile
3 Row 3	55.5		-1.4	54.1 kips/pile	27.1 tons/pile	27.1 tons/pile
4 Row 4	55.5		13.0	68.5 kips/pile	34.3 tons/pile	34.3 tons/pile
5 Row 5	55.5		27.4	83.0 kips/pile	41.5 tons/pile	41.5 tons/pile
6 Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
7 Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
8 Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
9 Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10 Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11 Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12 Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13 Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14 Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15 Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
				max:	**41.5 tons/pile**	**max: 41.5 tons/pile**

Assumed lateral Capacity: 36.0 kips/pile

Horizontal Pile Capacity	**Batter °/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1 Row 1	0	23	0.0	828	1.000	828 kips	
2 Row 2	0	24	0.0	864	1.000	864 kips	
3 Row 3	0	24	0.0	864	1.000	864 kips	
4 Row 4	0	26	0.0	936	1.000	936 kips	
5 Row 5	0	28	0.0	1008	1.000	1008 kips	
6 Row 6	0	0	0.0	0	1.000	0 kips	
7 Row 7	0	0	0.0	0	1.000	0 kips	
8 Row 8	0	0	0.0	0	1.000	0 kips	
9 Row 9	0	0	0.0	0	1.000	0 kips	
10 Row 10	0	0	0.0	0	1.000	0 kips	
11 Row 11	0	0	0.0	0	1.000	0 kips	
12 Row 12	0	0	0.0	0	1.000	0 kips	
13 Row 13	0	0	0.0	0	1.000	0 kips	
14 Row 14	0	0	0.0	0	1.000	0 kips	
15 Row 15	0	0	0.0	0	1.000	0 kips	
		125		4500		**4500 kips**	**OK**

Case **Case 2**

Flood Event **100 yr. flood + ice**

Unusual

Vertical Load, P = 6940 kips

Horizontal Load, H = -1174 kips

M_{NA} = -21871 kip-ft 125

Vertical Pile Loading	P / N	+	M_{NA}* d / Σ I	= Pile Loads		**Axial Pile Load**
1 Row 1	55.5		-30.2	25.3 kips/pile	12.6 tons/pile	12.6 tons/pile
2 Row 2	55.5		-15.8	39.7 kips/pile	19.9 tons/pile	19.9 tons/pile
3 Row 3	55.5		-1.4	54.1 kips/pile	27.1 tons/pile	27.1 tons/pile
4 Row 4	55.5		13.0	68.5 kips/pile	34.3 tons/pile	34.3 tons/pile

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BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091604	
MBI		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls	
2/11/11				Panel E	

5	Row 5	55.5	27.4	83.0 kips/pile		41.5 tons/pile		41.5 tons/pile
6	Row 6	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
7	Row 7	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
8	Row 8	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
9	Row 9	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
10	Row 10	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
11	Row 11	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
12	Row 12	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0	0.0	0.0 kips/pile		0.0 tons/pile		0.0 tons/pile
					max:	**41.5 tons/pile**	**max:**	**41.5 tons/pile**

Assumed lateral Capacity: 42.0 kips/pile

	Horizontal Pile Capacity	**Batter °/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance
1	Row 1	0	23	0.0	966	1.000	966 kips
2	Row 2	0	24	0.0	1008	1.000	1008 kips
3	Row 3	0	24	0.0	1008	1.000	1008 kips
4	Row 4	0	26	0.0	1092	1.000	1092 kips
5	Row 5	0	28	0.0	1176	1.000	1176 kips
6	Row 6	0	0	0.0	0	1.000	0 kips
7	Row 7	0	0	0.0	0	1.000	0 kips
8	Row 8	0	0	0.0	0	1.000	0 kips
9	Row 9	0	0	0.0	0	1.000	0 kips
10	Row 10	0	0	0.0	0	1.000	0 kips
11	Row 11	0	0	0.0	0	1.000	0 kips
12	Row 12	0	0	0.0	0	1.000	0 kips
13	Row 13	0	0	0.0	0	1.000	0 kips
14	Row 14	0	0	0.0	0	1.000	0 kips
15	Row 15	0	0	0.0	0	1.000	0 kips
			125		5250		**5250 kips**

OK

Case **Case 3**
Flood Event **500 yr. flood**
Unusual

Vertical Load, P = 6912 kips
Horizontal Load, H = -1539 kips
$M_{o/t}$ = -28326 kip-ft

	Vertical Pile Loading	P / N	+	$M_{o/t}$ * d / Σ I	= Pile Loads		**Axial Pile Load**
1	Row 1	55.3		-36.4	18.9 kips/pile	9.5 tons/pile	9.5 tons/pile
2	Row 2	55.3		-19.0	36.3 kips/pile	18.1 tons/pile	18.1 tons/pile
3	Row 3	55.3		-1.7	53.6 kips/pile	26.8 tons/pile	26.8 tons/pile
4	Row 4	55.3		15.7	71.0 kips/pile	35.5 tons/pile	35.5 tons/pile
5	Row 5	55.3		33.0	88.3 kips/pile	44.2 tons/pile	44.2 tons/pile
6	Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
7	Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
8	Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
9	Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
10	Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
11	Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
12	Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
13	Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
14	Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
15	Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile	0.0 tons/pile
						max: **44.2 tons/pile**	**max:** **44.2 tons/pile**

Assumed lateral Capacity: 42.0 kips/pile

	Horizontal Pile Capacity	**Batter °/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance
1	Row 1	0	23	0.0	966	1.000	966 kips
2	Row 2	0	24	0.0	1008	1.000	1008 kips
3	Row 3	0	24	0.0	1008	1.000	1008 kips
4	Row 4	0	26	0.0	1092	1.000	1092 kips
5	Row 5	0	28	0.0	1176	1.000	1176 kips
6	Row 6	0	0	0.0	0	1.000	0 kips
7	Row 7	0	0	0.0	0	1.000	0 kips
8	Row 8	0	0	0.0	0	1.000	0 kips
9	Row 9	0	0	0.0	0	1.000	0 kips
10	Row 10	0	0	0.0	0	1.000	0 kips
11	Row 11	0	0	0.0	0	1.000	0 kips
12	Row 12	0	0	0.0	0	1.000	0 kips
13	Row 13	0	0	0.0	0	1.000	0 kips
14	Row 14	0	0	0.0	0	1.000	0 kips
15	Row 15	0	0	0.0	0	1.000	0 kips
			125		5250		**5250 kips**

OK

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BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls	
2/11/11				Panel **E**	

Case **Case 4**
Flood Event **T.O. Levee**
Extreme

Vertical Load, P = 8772 kips
Horizontal Load, H = -3746 kips
$M_{t/A}$ = -59112 kip-ft

Vertical Pile Loading	P / N	+	$M_{t/A}$ * d / Σ I	= Pile Loads			**Axial Pile Load**
1 Row 1	54.2		-81.7	-27.5 kips/pile	-13.7 tons/pile		-13.7 tons/pile
2 Row 2	54.2		-42.7	11.5 kips/pile	5.7 tons/pile		5.7 tons/pile
3 Row 3	54.2		-3.7	50.4 kips/pile	25.2 tons/pile		25.2 tons/pile
4 Row 4	54.2		35.2	89.4 kips/pile	44.7 tons/pile		44.7 tons/pile
5 Row 5	54.2		74.2	128.4 kips/pile	64.2 tons/pile		64.2 tons/pile
6 Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
7 Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
8 Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
9 Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
10 Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
11 Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
12 Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
13 Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
14 Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
15 Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
					max: 64.2 tons/pile		**max: 64.2 tons/pile**

Assumed lateral Capacity: 48.0 kips/pile

Horizontal Pile Capacity	**Batter °/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance	
1 Row 1	0	23	0.0	1104	1.000	1104 kips	
2 Row 2	0	24	0.0	1152	1.000	1152 kips	
3 Row 3	0	24	0.0	1152	1.000	1152 kips	
4 Row 4	0	26	0.0	1248	1.000	1248 kips	
5 Row 5	0	28	0.0	1344	1.000	1344 kips	
6 Row 8	0	0	0.0	0	1.000	0 kips	
7 Row 7	0	0	0.0	0	1.000	0 kips	
8 Row 8	0	0	0.0	0	1.000	0 kips	
9 Row 9	0	0	0.0	0	1.000	0 kips	
10 Row 10	0	0	0.0	0	1.000	0 kips	
11 Row 11	0	0	0.0	0	1.000	0 kips	
12 Row 12	0	0	0.0	0	1.000	0 kips	
13 Row 13	0	0	0.0	0	1.000	0 kips	
14 Row 14	0	0	0.0	0	1.000	0 kips	
15 Row 15	0	0	0.0	0	1.000	0 kips	
		125		6000		**6000 kips**	**OK**

Case **Case 5**
Flood Event **Normal flow + ice**
Usual

Vertical Load, P = 7810 kips
Horizontal Load, H = 1740 kips
$M_{t/A}$ = 64 kip-ft

Vertical Pile Loading	P / N	+	$M_{t/A}$ * d / Σ I	= Pile Loads			**Axial Pile Load**
1 Row 1	62.5		0.1	62.6 kips/pile	31.3 tons/pile		31.3 tons/pile
2 Row 2	62.5		0.0	62.5 kips/pile	31.3 tons/pile		31.3 tons/pile
3 Row 3	62.5		0.0	62.5 kips/pile	31.2 tons/pile		31.2 tons/pile
4 Row 4	62.5		0.0	62.4 kips/pile	31.2 tons/pile		31.2 tons/pile
5 Row 5	62.5		-0.1	62.4 kips/pile	31.2 tons/pile		31.2 tons/pile
6 Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
7 Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
8 Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
9 Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
10 Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
11 Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
12 Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
13 Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
14 Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
15 Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
					max: 31.3 tons/pile		**max: 31.3 tons/pile**

Assumed lateral Capacity: 23.0 kips/pile

Horizontal Pile Capacity	**Batter °/ft**	**N**	Resistance due to Batter, kips	Resitance due to Bending, kips	Group Efficiency	Lateral Resitance
1 Row 1	0	23	0.0	529	1.000	529 kips
2 Row 2	0	24	0.0	552	1.000	552 kips

BARR ENGINEERING			DATE	2/11/2011	SHEET NO.
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34031604	
MBI		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls	
2/11/11				Panel E	

3	Row 3	0	24	0.0	552	1.000	552 kips
4	Row 4	0	26	0.0	598	1.000	598 kips
5	Row 5	0	28	0.0	644	1.000	644 kips
6	Row 6	0	0	0.0	0	1.000	0 kips
7	Row 7	0	0	0.0	0	1.000	0 kips
8	Row 8	0	0	0.0	0	1.000	0 kips
9	Row 9	0	0	0.0	0	1.000	0 kips
10	Row 10	0	0	0.0	0	1.000	0 kips
11	Row 11	0	0	0.0	0	1.000	0 kips
12	Row 12	0	0	0.0	0	1.000	0 kips
13	Row 13	0	0	0.0	0	1.000	0 kips
14	Row 14	0	0	0.0	0	1.000	0 kips
15	Row 15	0	0	0.0	0	1.000	0 kips
			125		2875		**2875 kips**

OK

Case **Case 6**
Flood Event **Construction**
Unusual

Vertical Load, P = 7810 kips
Horizontal Load, H = 1740 kips
M_{xx} = 157 kip-ft

	Vertical Pile Loading	P / N	+	M_{xx} * d / Σ I	= Pile Loads			**Axial Pile Load**
1	Row 1	62.5		0.2	62.7 kips/pile	31.3 tons/pile		31.3 tons/pile
2	Row 2	62.5		0.1	62.6 kips/pile	31.3 tons/pile		31.3 tons/pile
3	Row 3	62.5		0.0	62.5 kips/pile	31.2 tons/pile		31.2 tons/pile
4	Row 4	62.5		-0.1	62.4 kips/pile	31.2 tons/pile		31.2 tons/pile
5	Row 5	62.5		-0.2	62.3 kips/pile	31.1 tons/pile		31.1 tons/pile
6	Row 6	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
7	Row 7	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
8	Row 8	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
9	Row 9	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
10	Row 10	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
11	Row 11	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
12	Row 12	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
13	Row 13	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
14	Row 14	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
15	Row 15	0.0		0.0	0.0 kips/pile	0.0 tons/pile		0.0 tons/pile
						max:	**31.3 tons/pile**	**max: 31.3 tons/pile**

Assumed lateral Capacity: 42.0 kips/pile

	Horizontal Pile Capacity	**Batter "/ft**	**N**	Resistance due to Batter, kips	Resistance due to Bending, kips	Group Efficiency	Lateral Resitance
1	Row 1	0	23	0.0	529	1.000	529 kips
2	Row 2	0	24	0.0	552	1.000	552 kips
3	Row 3	0	24	0.0	552	1.000	552 kips
4	Row 4	0	26	0.0	598	1.000	598 kips
5	Row 5	0	28	0.0	644	1.000	644 kips
6	Row 6	0	0	0.0	0	1.000	0 kips
7	Row 7	0	0	0.0	0	1.000	0 kips
8	Row 8	0	0	0.0	0	1.000	0 kips
9	Row 9	0	0	0.0	0	1.000	0 kips
10	Row 10	0	0	0.0	0	1.000	0 kips
11	Row 11	0	0	0.0	0	1.000	0 kips
12	Row 12	0	0	0.0	0	1.000	0 kips
13	Row 13	0	0	0.0	0	1.000	0 kips
14	Row 14	0	0	0.0	0	1.000	0 kips
15	Row 15	0	0	0.0	0	1.000	0 kips
			125		2875		**2875 kips**

OK

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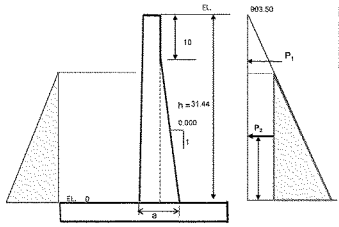
BARR ENGINEERING			DATE	2/11/2011	SHEET NO
			PROJECT NAME	FARGO – MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT, FEASIBILITY STUDY, PHASE 4	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER	34091004	
MBI 2/11/11		MBI	SUBJECT	Maple Aquaduct Structure - Retaining Walls Panel E	0



CASE	Event		HW	TW	Dh	TW -frg
Case 1	100 yr. flood	Usual	893.89	892.57	1.32	892.57
Case 2	100 yr. flood + ice	Unusual	893.89	892.57	1.32	892.57
Case 3	500 yr. flood	Unusual	896.380	893.66	2.72	893.66
Case 4	T.O. Levee	Extreme	903.500	903.50	0.00	903.50
Case 5	Normal flow + ice	Usual	0.000	0.000	0.00	0.00
Case 6	Construction	Unusual	0.000	0.000	0.00	0.00

LOAD FACTORS

Hf =	1.30	hydraulic Factor
LF =	1.70	
Unusual & Extreme =	0.75	
TOP THICK =	4.0 ft	48.0 in
Batter at Base =	0.00 ft	0.0 in
a =	4.00 ft	48.0 in

Load Factors - Hydraulic Structures	
live load, LL =	1.7
dead load, DL =	1.4
flood level , FL =	1
Fluid, F =	1.7
hydraulic, Hf =	1.3
direct tension hydraulic, Hf =	1.65
ICE =	1.7

WALL DESIGN:

Horizontal Load Components and Moments about Bottom of Stem (Service)

CASE	Event	Condition	Load Factor	H (kips/ft)	Moment (kip-ft/ ft)	Vu (kips/ft)	Mu (kip-ft/ ft)
Case 1	100 yr. flood	Usual	1	-5.48	-9.779	-12.11	-21.61
Case 2	100 yr. flood + ice	Unusual	0.75	-5.48	-9.779	-9.08	-16.21
Case 3	500 yr. flood	Unusual	0.75	-7.69	-34.842	-12.75	-57.75
Case 4	T.O. Levee	Extreme	0.75	-21.45	-224.795	35.55	372.60
Case 5	Normal flow + ice	Usual	1	9.39	98.412	20.75	217.49
Case 6	Construction	Unusual	0.75	9.39	98.412	15.56	163.12

STEM DESIGN VALUES

MU, k-ft/ft	372.60	k-ft/ft
VU, k/ft	35.55	k/ft

ACI 318-05 w/ Modifications per EM 1110-2-2104 ref. EM 1110-2-2104

9.3 - Design Strength

ϕ = **0.9** 9.3.2.1 - Tension Controlled sections

0.75 9.3.2.3 - Shear and torsion

FLEXURAL STEEL FOR RECTANGULAR CONCRETE SECTIONS

fy= 60 ksi
f'c= 4 ksi
B1= 0.85

Muh = 373 k-ft /ft Includes: hf = 1.3
Vuh= 35.55 k / ft

bw= 12 in.
h= 48 in
cover= 4 in (include correct stirrup bar dia.)
d= 43.50 in

pb= 0.0285 pb=0.85*B1*f'c/ fy*(87 / (87+fy))
.75*pb= 0.0214

m=fy / 0.85*f'c= 17.647

TRIAL

Ru=Mv/bd² = 218.786 **ACI 10.5.1** **ACI 10.5.3**
REQ'D ρ= 0.0038 **O.K.** ρ(min)= 3*SQRT(f'c)/fy 200* / fy 4/3*ρ
ρ= FALSE N.G. 0.00516 0.00333 0.0050

EM 110-2-2104 2-8 c. (not less than Temp & S'rinkage, half in each face)

As (REQ'D)= 0.81 in² ρ(min)= 0.0028 /2 = ▶ As =0.5*ρ(min)*b = 0.6084 in²
As = #9 @ 12 = 1.00 in².

SELECT STEEL

bar #= 9
spacing, s= 6 in
# OF BARS= 1 (ENTER 1 IF PER FT, b=12") a
As= 1.999 in²
d= 43.4375 in
ρ = As/bd = 0.0038 **O.K.** < 0.375pb **EM 110-2-2104**
ρ = 0.139 pb **MAXIMUM TENSILE REINFORCEMENT**
a) For singly reinforced flexural members
1) ρ = 0.25 pb Recommended limit
2) ρ = 0.375 pb Max. permitted upper limit not requiring special study
3) ρ = 0.5 pb Max. permitted upper limit when excessive deflections are not predicted in ACI 318
4) ρ = >0.5 pb but ≤ 0.375 pb permitted only if detailed serviceability analysis incl. deflect. Calc.
T= As*fy = 119.9 k b) Use of compression reinf. shall be per ACI 318
C= B1*Fc' *b*a = 1268.8 a
a= T/C = 0.095 in
Mn= T(d - a/2)/12 = 433.6 ft-k
ϕ Mn= 390.2 ft-k > Mu O.K.

CHECK SHEAR REINFORCEMENT (ACI 11.3 & EM 110-2-2104 3-2a)

Vuh = 35.6 k NO SHEAR REINF. REQUIRED
Vn = Vuh / ϕ = 47.4 k
Vc = 2*sqrt(f'c) bw * d = 65.9 k **11.3.1.1**
Vs =Vuh / f - 1.3Vc = No Shear Reinf. Req. k NG Vs(max) ≤ 8*sqrt(f'c)bd = 263.7 k

Trial Stirrup Sizes:

# of stirrup legs = 2 (Single stirrup = 2, Dbl stirrup = 4.....)
Stirrup bar size = 4
Av= 0.393 in²
s= 0.000 in s = Av * fy * d / (Vu / f - Vc)

11.5.5 - Spacing limits for shear reinforcement

s = d/2 = 21.719 in OR 24 in
s(max)= 10.859 in
4*sqrt(F'c)*bw*d= 131.9 k < Vs Reduce Spacing

USE s= **0.00** in

Vs = (Av * Fy * d) / s = #DIV/0! k

11.5.6 - MINIMUM SHEAR REINFORCEMENT

A minimum area of shear reinforcement, Av,min shall be provided in all reinforced concrete flexural members where Vu exceeds **0.5 f** Vc
NOT REQUIRED IF:
a) SLAB OR FOOTING. vc>vn **O.K.**
b) CONCRETE JOIST ACI 8.11
c) BEAMS W/ h <= 10"
h <= 2.5*Bf
h <= 0.5*bw
d) WALLS (SEE ACI 11.10.1): vc>vn **O.K.**

11.5.6.3

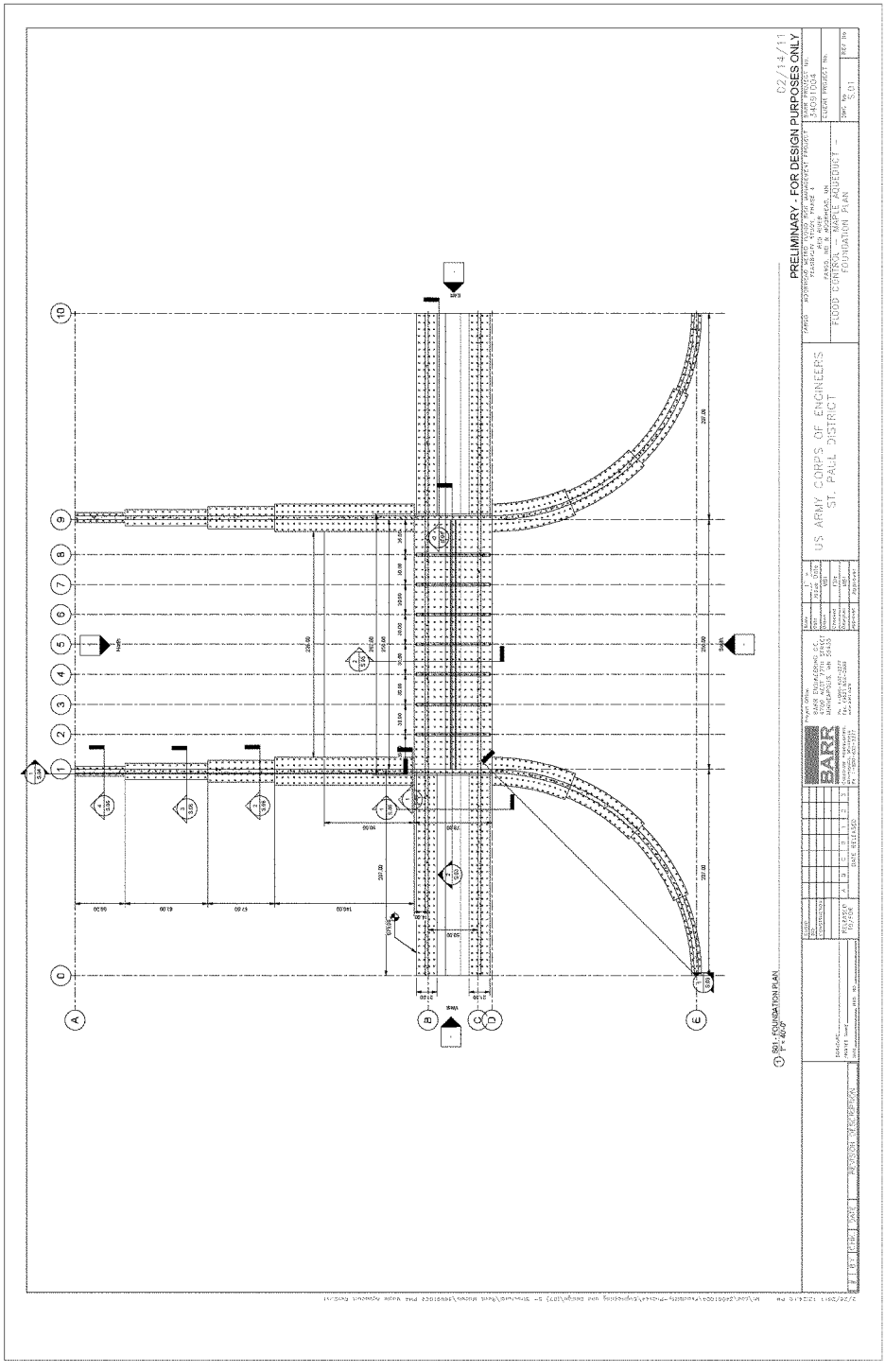
Av,min = 0.75 sqrt(f'c) bw*s/fy = 0.70 * s
but not less than 50bw*s/ fy = 23.33333333 * s
s max = Av fy / 0.75 sqrt(f'c) bw = 0.00 in
s max = Av fy /50 bw = 0.00 in

11.5.5.3

Where Vs exceeds 4*sqrt(F'c)*bw*d maximum spacings shall be reduced by one-half

ATTACHMENT

F-R5.3 Maple River Aqueduct Structure Drawings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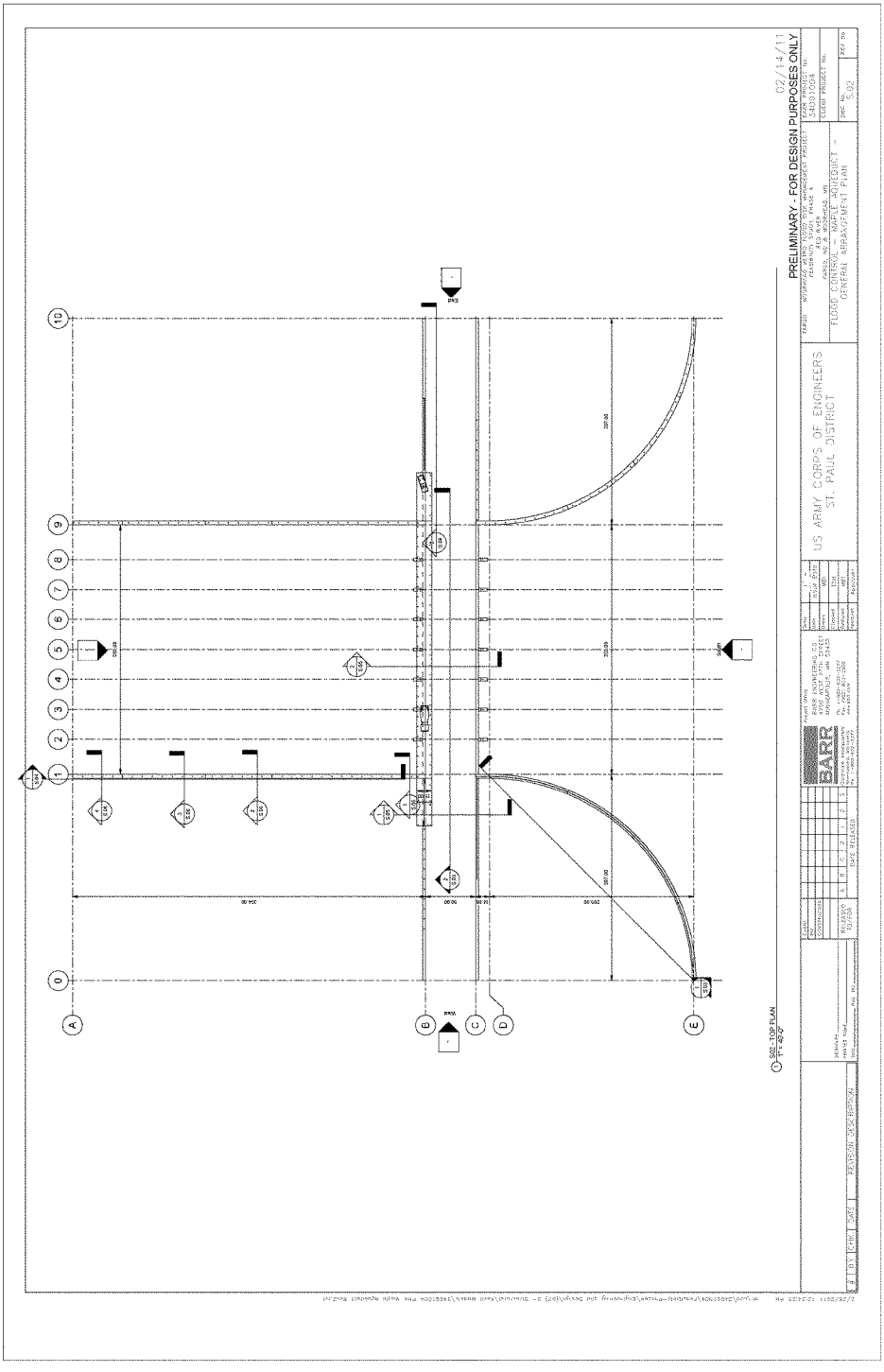
US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
FLOOD CONTROL - MAPLE AQUEDUCT -
FOUNDATION PLAN
BARR
S01 - FOUNDATION PLAN
1" = 40'-0"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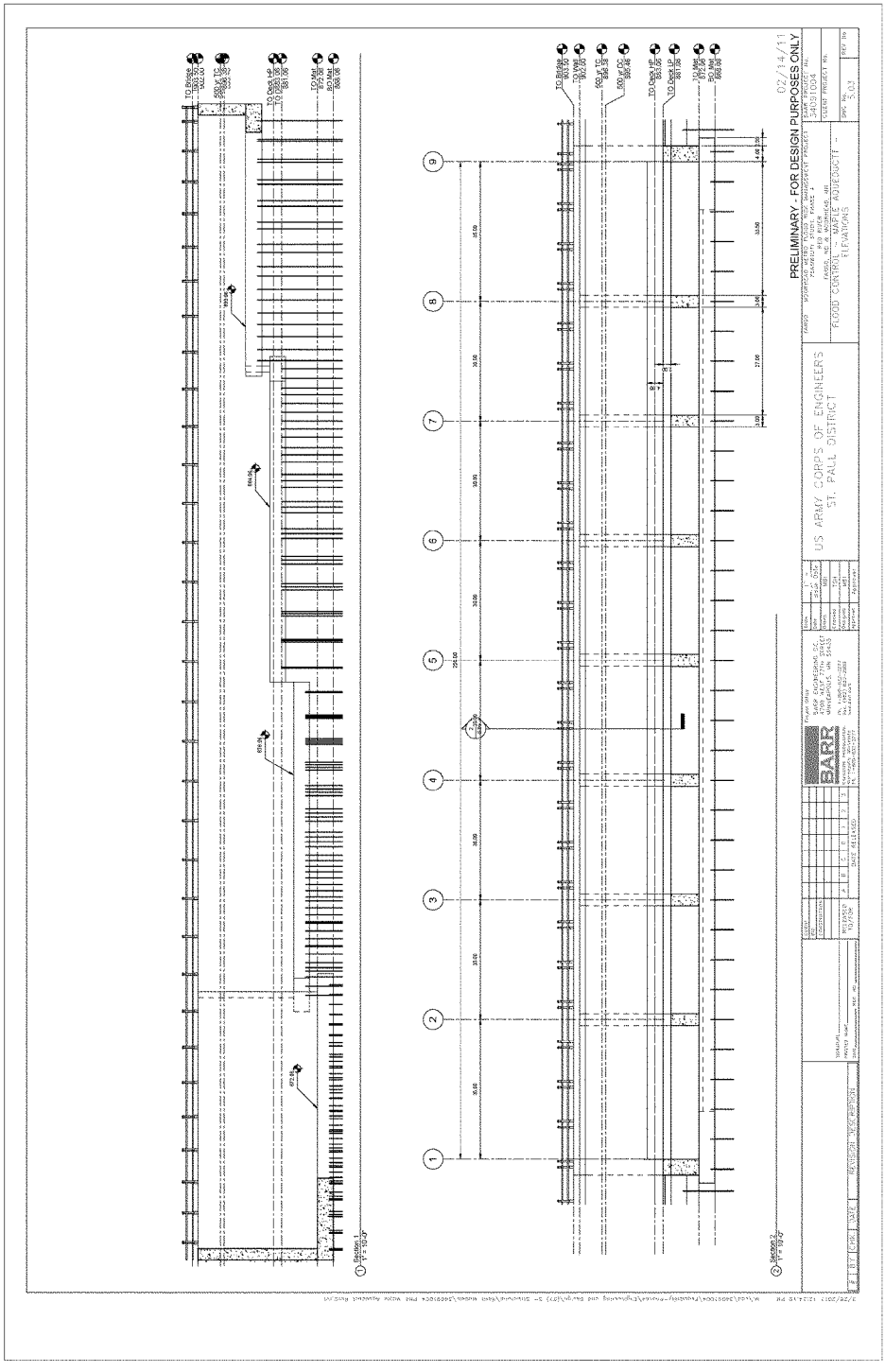
ST. PAUL DISTRICT
FLOOD CONTROL - MAPLE AQUEDUCT
GENERAL ARRANGEMENT PLAN
BARR
SIZE - TOP PLAN
1" = 40'-0"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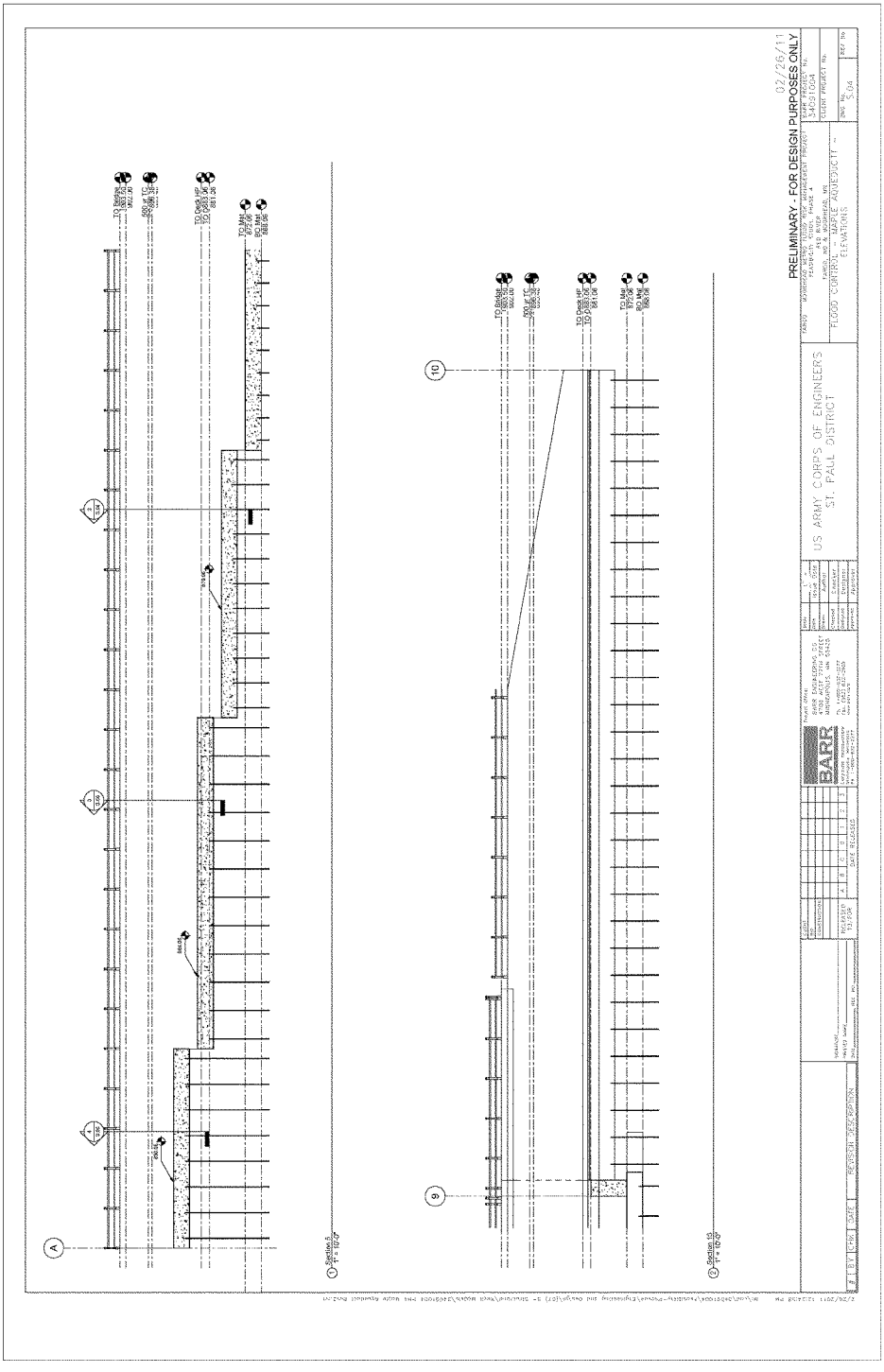
ST. PAUL DISTRICT
FLOOD CONTROL - MAPLE AQUEDUCTF -
ELEVATIONS
BARR
Section 1
Section 2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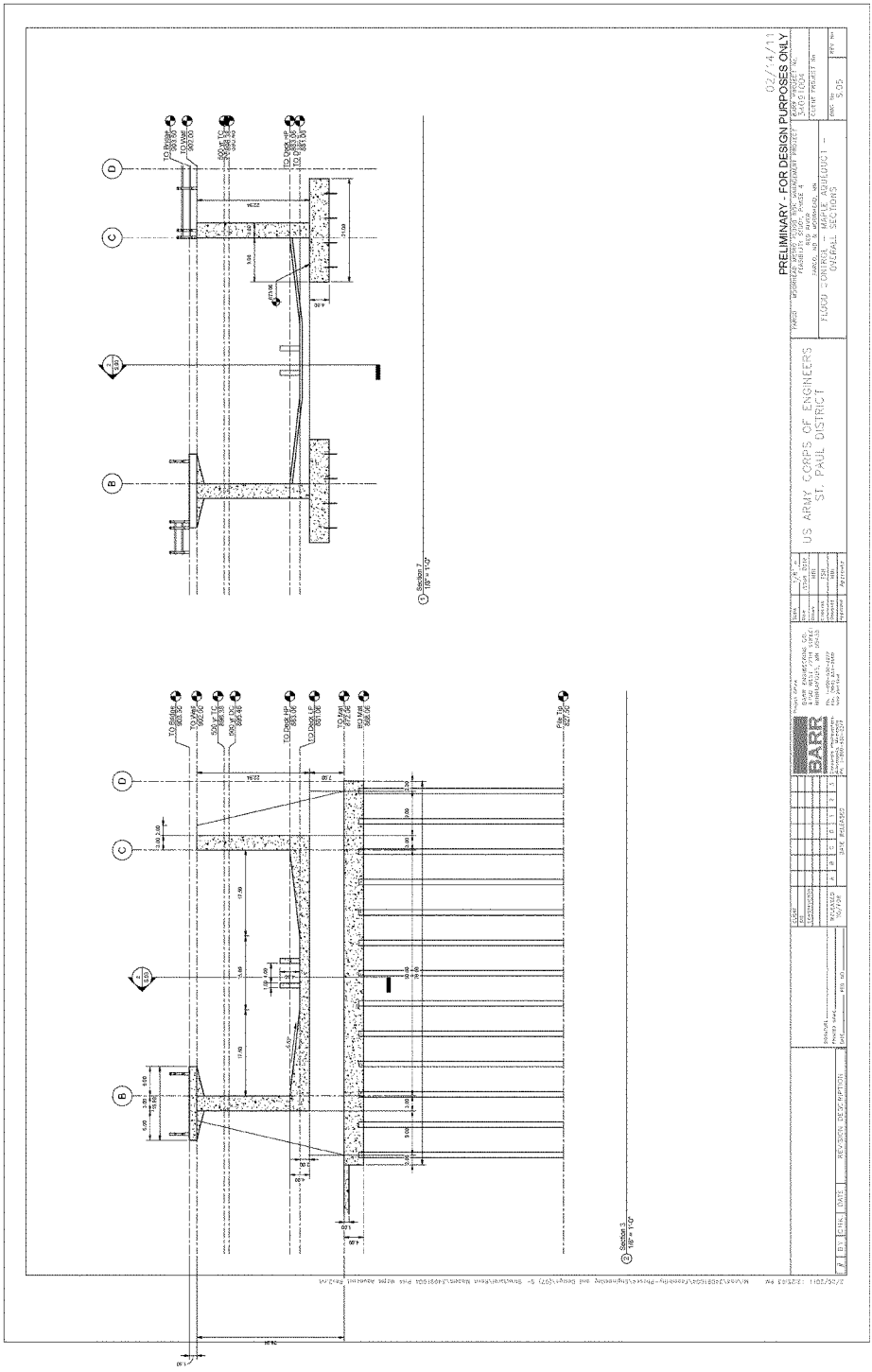
FLOOD CONTROL -- MAPLE ACRES/CITY --
ELEVATIONS
BARR
Section 9
Section 10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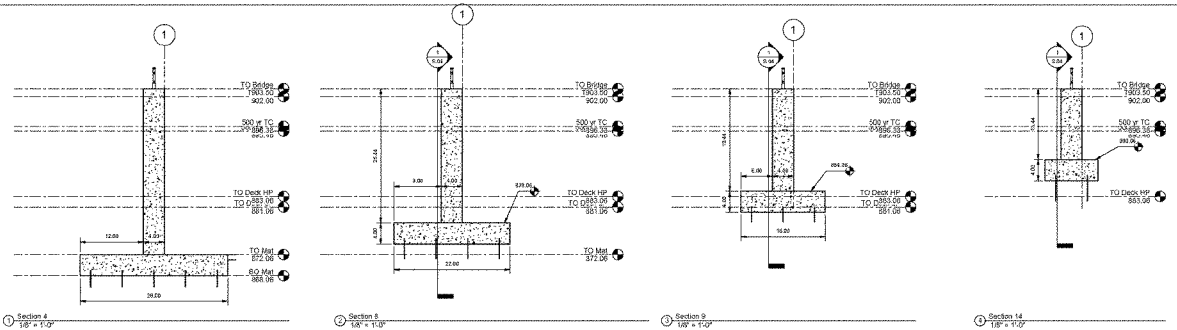
02/4/13
US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
FLOOD CONTROL - MAPLE RIVER
OVERALL SECTIONS
BAR
Section 1
1/8" = 1'-0"
Section 3
1/8" = 1'-0"



02/14/11

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#	BY	CHK	DATE	REVISION DESCRIPTION

ORIGINAL

INITIALS HERE

DATE REQ. NO

Initial						
Init						
COORNUMBER						
RELEASED TO/FOR	A	B	C	D	1	2
DATE RELEASED						

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Scale	1/8" = 1'-0"
Date	15JAN 2008
Drawn	MBJ
Checked	TGH
Quantities	MBJ
Approved	02/13/2007

US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT

FARGO MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 2
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL - MAPLE AQUEDUCT - WALL SECTIONS

CORPS PROJECT No.	
146051004	
LOCAL PROJECT No.	
DWG No. 5.06	REV No

Structural Foundation Schedule

Type	Structural Usage	Count	Width	Length	Foundation Thickness	Area	Forming	Volume	Volume CY	Reinf. (lbs/sf)	Total Reinf. (lbs)
Retaining Footing - 36" x 36" x 48"	Retaining	1	10.00	50.00	4.00	500 SF	516 SF	2000.00 CF	74.07 CY	27.2	13600
Retaining Footing - 36" x 36" x 48"	Retaining	1	10.00	80.50	4.00	805 SF	789 SF	3220.13 CF	119.26 CY	27.2	21897
Retaining Footing - 36" x 36" x 48"	Retaining	1	10.00	50.00	4.00	500 SF	516 SF	2000.00 CF	74.07 CY	27.2	13600
Retaining Footing - 36" x 36" x 48"	Retaining	1	10.00	80.50	4.00	805 SF	789 SF	3220.13 CF	119.26 CY	27.2	21897
Retaining Footing - 36" x 36" x 48": 4						2610 SF	2599 SF	10440.25 CF	386.68 CY		70994
Retaining Footing - 72" x 72" x 48"	Retaining	1	16.00	83.00	4.00	1328 SF	842 SF	5312.00 CF	196.74 CY	27.2	36122
Retaining Footing - 72" x 72" x 48"	Retaining	1	16.00	80.50	4.00	1288 SF	820 SF	5152.22 CF	190.82 CY	27.2	35035
Retaining Footing - 72" x 72" x 48"	Retaining	1	16.00	83.00	4.00	1328 SF	842 SF	5312.00 CF	196.74 CY	27.2	36122
Retaining Footing - 72" x 72" x 48"	Retaining	1	16.00	80.50	4.00	1288 SF	820 SF	5152.22 CF	190.82 CY	27.2	35035
Retaining Footing - 72" x 72" x 48": 4						5232 SF	3324 SF	20928.43 CF	775.13 CY		142313
Retaining Footing - 108" x 108" x 48"	Retaining	1	21.00	203.00	4.00	4263 SF	1904 SF	17052.00 CF	631.56 CY	27.2	115954
Retaining Footing - 108" x 108" x 48"	Retaining	1	21.00	203.00	4.00	4263 SF	1904 SF	17052.00 CF	631.56 CY	27.2	115954
Retaining Footing - 108" x 108" x 48"	Retaining	1	22.00	67.00	4.00	1474 SF	757 SF	5896.00 CF	218.37 CY	27.2	40093
Retaining Footing - 108" x 108" x 48"	Retaining	1	22.00	80.50	4.00	1771 SF	871 SF	7084.30 CF	262.38 CY	27.2	48173
Retaining Footing - 108" x 108" x 48"	Retaining	1	21.00	203.00	4.00	4263 SF	1904 SF	17052.00 CF	631.56 CY	27.2	115954
Retaining Footing - 108" x 108" x 48"	Retaining	1	21.00	123.00	4.00	2583 SF	1224 SF	10332.00 CF	382.67 CY	27.2	70268
Retaining Footing - 108" x 108" x 48"	Retaining	1	22.00	67.00	4.00	1474 SF	757 SF	5896.00 CF	218.37 CY	27.2	40093
Retaining Footing - 108" x 108" x 48"	Retaining	1	22.00	80.50	4.00	1771 SF	871 SF	7084.30 CF	262.38 CY	27.2	48173
Retaining Footing - 196" x 108" x 48"	Retaining	1	21.00	80.00	4.00	1680 SF	859 SF	6720.00 CF	248.89 CY	27.2	45696
Retaining Footing - 108" x 108" x 48": 9						23542 SF	11056 SF	94168.59 CF	3487.73 CY		640546
Retaining Footing - 144" x 144" x 48"	Retaining	1	28.00	140.00	4.00	3920 SF	1426 SF	15680.00 CF	580.74 CY	27.2	106624
Retaining Footing - 144" x 144" x 48"	Retaining	1	28.00	80.50	4.00	2254 SF	922 SF	9016.38 CF	333.94 CY	27.2	61311
Retaining Footing - 144" x 144" x 48"	Retaining	1	28.00	140.00	4.00	3920 SF	1426 SF	15680.00 CF	580.74 CY	27.2	106624
Retaining Footing - 144" x 144" x 48"	Retaining	1	28.00	80.50	4.00	2254 SF	922 SF	9016.38 CF	333.94 CY	27.2	61311
Retaining Footing - 144" x 144" x 48": 4						12348 SF	4701 SF	49392.76 CF	1829.36 CY		335871
Grand total: 21						43733 SF	21633 SF	174930.04 CF	6478.89 CY		1189524

Floor Schedule

Type	Comments	Elevated	Type Comments	Structural Usage	Count	Thickness	Perimeter	Area	Forming	Volume	Volume (cy)	Est. Reinf.	Total Reinf. (lbs)	Takeoff
12" Concrete	SOG	No	Apron Slab	Slab	1	1.00	632.00	20340 SF	632	20340.00 CF	753.33 CY	13.6	276624	Yes
Apron Slab: 1							632.00	20340 SF	632	20340.00 CF	753.33 CY		276624	
18" Concrete	Deck	Yes	Bridge	Slab	1	1.50	546.00	3579 SF	4689	5805.00 CF	215.00 CY	27.2	105264	Yes
18" Concrete	Deck	Yes	Bridge	Slab	1	1.50	126.00	720 SF	909	1080.00 CF	40.00 CY	27.2	19584	Yes
18" Concrete	Deck	Yes	Bridge	Slab	1	1.50	126.00	720 SF	909	1080.00 CF	40.00 CY	27.2	19584	Yes
Bridge: 3							798.00	5110 SF	6507	7965.00 CF	295.00 CY		144432	
24" Concrete	Deck	Yes	Deck Slab	Slab	1	2.00	626.00	14507 SF	15763	41022.00 CF	1519.33 CY	27.2	394584	Yes
Deck Slab: 1							626.00	14507 SF	15763	41022.00 CF	1519.33 CY		394584	
6" Concrete	RipRap	No	Grading	Slab	1	0.50	506.00	10196 SF	253	5098.12 CF	0.00 CY	0	0	No
6" Concrete	RipRap	No	Grading	Slab	1	0.50	506.00	10196 SF	253	5098.12 CF	0.00 CY	0	0	No
Grading: 2							1012.00	20392 SF	506	10196.25 CF	0.00 CY		0	
48" Concrete	SOG	No	Mat Foundation	Slab	1	4.00	506.00	20436 SF	2720	81744.00 CF	3027.56 CY	27.2	555859	Yes
Mat Foundation: 1							506.00	20436 SF	2720	81744.00 CF	3027.56 CY		555859	
Grand total: 9							3750.00	80985 SF	26128	161267.25 CF	5595.22 CY		1371500	

Structural Column Schedule

Type	Count	Length	Top Level	Base Level	Base Offset	Top Offset	Volume	Height
HP14X89	845	35118.20	BO Mat	Pile Tip	0.00	1.00	7.45 CF	41.54
41.59: 845		35118.20						
HP14X89	192	9131.52	BO Mat	Pile Tip	0.00	17.00	8.52 CF	47.54
47.59: 192		9131.52						
HP14X89	482	23308.90	BO Mat	Pile Tip	0.00	8.00	8.70 CF	48.54
48.56: 480		23308.90						
HP14X89	144	7712.64	BO Mat	Pile Tip	0.00	13.00	9.60 CF	53.54
53.59: 144		7712.64						
HP14X89	89	4764.80		Pile Tip	0.00		10.68 CF	59.54
59.56: 80		4764.80						
Grand total: 1741		89035.96						

02/14/11

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#	BY	CHK	DATE	REVISION DESCRIPTION

SIGNATURE

PRINTED NAME

DATE

RED RD

CLIENT

BID

CONSTRUCTION

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A | B | C | D | E | 1 | 2 | 3

DATE RELEASED

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Role	Name
Dsgn	TSW, CWS
Drwn	MSH
Chk'd/Apd	TSH
Designer	MSH
Approver	Approver

US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT

FARGO MOORHEAD FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 4
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL - MAPLE AQUEDUCT - SCHEDULES

BARR PROJECT No.
34091004

CLIENT PROJECT No.

DWG No. S.07

REV No.

1644

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Wall Schedule											
Type	Type Comments	Comments	Length	Width	Height	Area	Forming	Volume	Volume CY	Reinf. (lbs/sf)	Total Reinf. (lbs)	Takeoff
Exterior - 72" Concrete	Aqueduct	Deck	258.00	6.00	1.50	387 SF	792 SF	1161.00 CF	43.00 CY	27.2	10526	
Exterior - 72" Concrete	Aqueduct	Deck	254.00	6.00	1.48	376 SF	769 SF	373.00 CF	13.89 CY	27.2	10200	
Exterior - 72" Concrete	Aqueduct	Deck	48.00	6.00	1.00	72 SF	162 SF	432.00 CF	16.00 CY	27.2	1958	
Exterior - 72" Concrete	Aqueduct	Deck	48.00	6.00	1.50	72 SF	162 SF	216.00 CF	8.00 CY	27.2	1958	
Exterior - 72" Concrete	Aqueduct	Deck	30.00	6.00	1.44	72 SF	161 SF	72.00 CF	2.67 CY	27.2	1958	
Exterior - 72" Concrete	Aqueduct	Deck	50.00	6.00	1.44	72 SF	161 SF	432.00 CF	16.00 CY	27.2	1958	
Deck: 8			708.00			1000 SF	2006 SF	2689.00 CF	99.56 CY		28560	
Exterior - 12" Concrete	Aqueduct	Low Flow Walls	258.05	1.00	4.00	1000 SF	2008 SF	1030.19 CF	37.64 CY	27.2	27206	
Exterior - 12" Concrete	Aqueduct	Low Flow Walls	259.05	1.00	4.00	1000 SF	2008 SF	1030.19 CF	37.64 CY	27.2	27206	
Low Flow Walls: 2			500.10			2000 SF	4017 SF	2060.38 CF	74.09 CY		54410	
Exterior - 36" Concrete	Aqueduct	Pier	56.00	3.00	7.00	392 SF	826 SF	1176.00 CF	43.56 CY	27.2	10662	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aqueduct	Pier	56.00	3.00	7.00	392 SF	826 SF	1176.00 CF	43.56 CY	27.2	10662	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aqueduct	Pier	56.00	3.00	7.00	392 SF	826 SF	1176.00 CF	43.56 CY	27.2	10662	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aqueduct	Pier	56.00	3.00	7.00	392 SF	826 SF	1176.00 CF	43.56 CY	27.2	10662	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aqueduct	Pier	56.00	3.00	7.00	392 SF	826 SF	1176.00 CF	43.56 CY	27.2	10662	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aqueduct	Pier	56.00	3.00	7.00	392 SF	826 SF	1176.00 CF	43.56 CY	27.2	10662	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aqueduct	Pier	56.00	3.00	7.00	392 SF	826 SF	1176.00 CF	43.56 CY	27.2	10662	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Exterior - 36" Concrete	Aqueduct	Pier	11.00	3.00	14.97	165 SF	419 SF	494.01 CF	18.30 CY	27.2	4479	
Pier: 21			546.00			5049 SF	11650 SF	15148.14 CF	561.04 CY		137345	
Exterior - 36" Concrete	Aqueduct	Wall	254.00	3.00	18.64	4735 SF	9582 SF	14205.00 CF	526.11 CY	27.2	128752	
Exterior - 36" Concrete	Aqueduct	Wall	254.00	3.00	18.64	4735 SF	9582 SF	14205.00 CF	526.11 CY	27.2	128752	
Wall: 2			508.00			9470 SF	18164 SF	28410.00 CF	1052.22 CY		257504	
Aqueduct: 31			2262.10			17570 SF	37037 SF	48246.53 CF	1788.01 CY		477598	

Wall Schedule											
Type	Type Comments	Comments	Length	Width	Height	Area	Forming	Volume	Volume CY	Reinf. (lbs/sf)	Total Reinf. (lbs)	Takeoff
Retaining - 36" Concrete	Retaining	Wall	205.00	3.00	22.72	4657 SF	9450 SF	13970.46 CF	517.42 CY	27.2	126665	
Retaining - 36" Concrete	Retaining	Wall	205.00	3.00	22.72	4657 SF	9450 SF	13970.46 CF	517.42 CY	27.2	126668	
Exterior - 48" Concrete	Retaining	Wall	12.50	4.00	32.69	409 SF	1079 SF	1634.64 CF	60.54 CY	27.2	11116	
Exterior - 48" Concrete	Retaining	Wall	140.00	4.00	31.44	4402 SF	9055 SF	17606.40 CF	652.09 CY	27.2	119724	
Exterior - 48" Concrete	Retaining	Wall	12.50	4.00	34.37	430 SF	1134 SF	1718.64 CF	63.65 CY	27.2	11687	
Exterior - 48" Concrete	Retaining	Wall	67.00	4.00	25.44	1704 SF	3612 SF	6817.92 CF	252.52 CY	27.2	46362	
Exterior - 48" Concrete	Retaining	Wall	83.00	4.00	19.44	1614 SF	3383 SF	6454.08 CF	239.04 CY	27.2	43888	
Exterior - 48" Concrete	Retaining	Wall	50.00	4.00	13.44	672 SF	1452 SF	2688.00 CF	99.56 CY	27.2	18278	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	30.23	2434 SF	5109 SF	9641.10 CF	357.08 CY	27.2	66198	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	24.17	1946 SF	4085 SF	7709.01 CF	285.52 CY	27.2	52953	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	15.12	1458 SF	3062 SF	5776.92 CF	213.96 CY	27.2	39666	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	12.06	971 SF	2038 SF	3844.93 CF	142.40 CY	27.2	26400	
Exterior - 48" Concrete	Retaining	Wall	53.00	4.00	9.00	477 SF	1020 SF	1908.00 CF	70.67 CY	27.2	12974	
Retaining - 36" Concrete	Retaining	Wall	125.00	3.00	22.57	2822 SF	5779 SF	8464.86 CF	313.51 CY	27.2	76748	
Retaining - 36" Concrete	Retaining	Wall	205.00	3.00	22.72	4657 SF	9450 SF	13970.46 CF	517.42 CY	27.2	126668	
Exterior - 48" Concrete	Retaining	Wall	12.50	4.00	32.45	406 SF	1071 SF	1622.64 CF	60.10 CY	27.2	11034	
Exterior - 48" Concrete	Retaining	Wall	140.00	4.00	31.44	4402 SF	9055 SF	17606.40 CF	652.09 CY	27.2	119724	
Exterior - 48" Concrete	Retaining	Wall	12.50	4.00	34.13	427 SF	1126 SF	1708.64 CF	63.21 CY	27.2	11605	
Exterior - 48" Concrete	Retaining	Wall	67.00	4.00	25.44	1704 SF	3612 SF	6817.92 CF	252.52 CY	27.2	46362	
Exterior - 48" Concrete	Retaining	Wall	83.00	4.00	19.44	1614 SF	3383 SF	6454.08 CF	239.04 CY	27.2	43888	
Exterior - 48" Concrete	Retaining	Wall	50.00	4.00	13.44	672 SF	1452 SF	2688.00 CF	99.56 CY	27.2	18278	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	30.23	2434 SF	5109 SF	9641.10 CF	357.08 CY	27.2	66198	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	24.17	1946 SF	4085 SF	7709.01 CF	285.52 CY	27.2	52953	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	15.12	1458 SF	3062 SF	5776.92 CF	213.96 CY	27.2	39666	
Exterior - 48" Concrete	Retaining	Wall	80.50	4.00	12.06	971 SF	2038 SF	3844.93 CF	142.40 CY	27.2	26400	
Exterior - 48" Concrete	Retaining	Wall	53.00	4.00	8.89	471 SF	1013 SF	1684.00 CF	68.78 CY	27.2	12811	
Retaining - 36" Concrete	Retaining	Wall	80.00	3.00	15.69	1271 SF	2637 SF	3812.86 CF	141.22 CY	27.2	34570	
Wall: 27			2360.03			51082 SF	106806 SF	165740.17 CF	6879.27 CY		1389440	
Retaining: 27			2360.03			51082 SF	106806 SF	165740.17 CF	6879.27 CY		1389440	
Grand total: 56			4562.12			68652 SF	143843 SF	237866.70 CF	8666.17 CY		1967338	

02/18/11

PRELIMINARY - FOR DESIGN PURPOSES ONLY

#	BY	CHK	DATE	REVISION DESCRIPTION

Drawing Name: _____

Detail No. _____ Sheet No. _____

Submit									
60									
95									
Construction									
RELEASED TO FILE/FOR	A	B	C	D	1	2	3		
DATE RELEASED									

BARR

BARR ENGINEERING CO.
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
Ph. (952) 832-2600
Fax. (952) 832-2601
www.barr.com

Project Office
Corporate Headquarters
Ph. (800) 632-2277

Scale	None
Date	Nov. 2008
Drawn	MSI
Checked	TCH
Reviewed	MSI
Approved	Approved

US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT

FARGO MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE II
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL - MAPLE AQUEDUCT - SCHEDULES - II

BARR PROJECT No.: 34091004
CLIENT PROJECT No.:
DWG No.: S.08
REV No.:

1645

K:\Jobs\34091004\Feasibility\Project\Engineering and Design\S01 - Structural\Revit Models\34091004 Fmw Work Aqueduct Rev7.rvt
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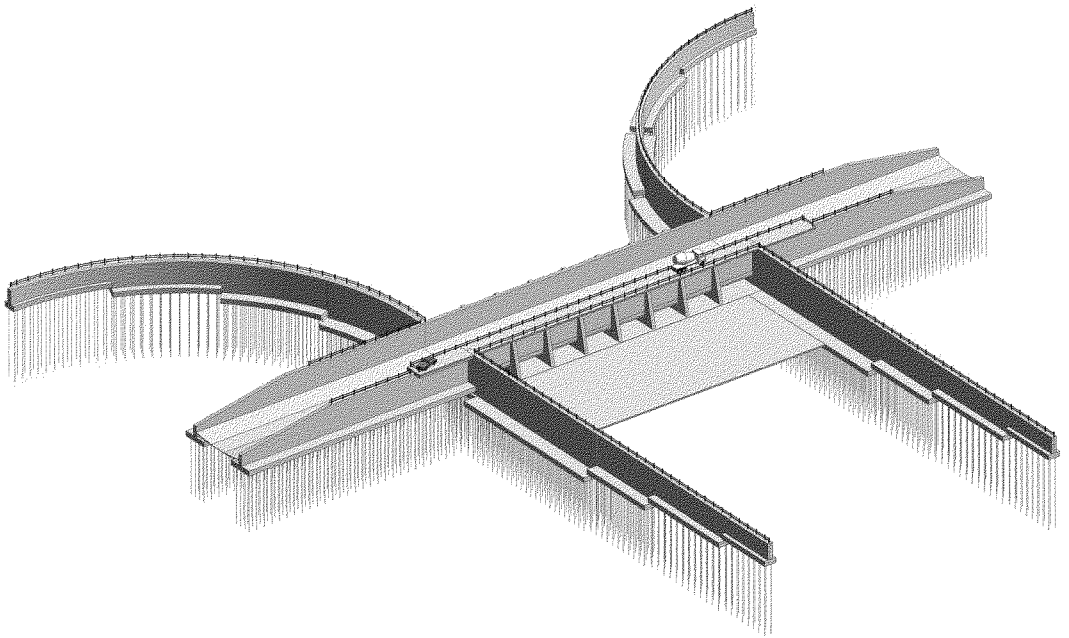

3D VIEW OVERALL D/S

02/14/11

PRELIMINARY - FOR DESIGN PURPOSES ONLY

#	BY	CHK	DATE	REVISION DESCRIPTION

SIGNATURE

PRINTED NAME

DATE

RELEASED TO PSB

DATE RELEASED

BARR ENGINEERING CO
4700 WEST 77TH STREET
MINNEAPOLIS, MN 55435
Ph. 1-952-832-2727
Fax. (952) 832-2601
www.barr.com

Project Office
Corporate Headquarters
Minneapolis, Minnesota
Ph. 1-952-832-2727

Scale	
Date	Issue Date
Drawn	MRH
Checked	TDH
Designed	MRH
Reviewed	MRH
Approved	Applicant

US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT

FARGO — MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
FEASIBILITY STUDY, PHASE 4
RED RIVER
FARGO, ND & MOORHEAD, MN

FLOOD CONTROL – MAPLE AQUEDUCT – 3D
VIEWS

BARR PROJECT No.
34001004

CLIENT PROJECT No.

DWG No
5.09

REV No

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PRELIMINARY - FOR DESIGN PURPOSES ONLY
02/24/11
US ARMY CORPS OF ENGINEERS
ST. PAUL DISTRICT
FARGO
FLOOD CONTROL -- MAPLE AQUEDUCT -- 30'
USACE PROJECT No.
SHEET No.
S-16
REF No
BARR
1) VIEW OVERALL_U8

RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT S – STRUCTURAL DESIGN COMPUTATIONS— SHEETPILE/ROCKFILL SPILLWAYS

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

By: Barr Engineering Co.

FINAL – February 28, 2011

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F-S1.2 Design Computations for Sheetpile Weirs

APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT S – STRUCTURAL DESIGN COMPUTATIONS—
SHEETPILE/ROCKFILL SPILLWAYS

F-S1.0 SHEETPILE/ROCKFILL WEIR SPILLWAYS

A description of the structural design concepts and criteria for the sheetpile weirs can be found in Appendix F. *Attachments F-S1.1 through F-S1.2 of this Exhibit S were previously presented as part of Exhibit P of Appendix F of the Phase 3 report submitted on August 6, 2010 and present the structural design computations for the sheetpile/rockfill weir spillway structures.* This section is applicable for the following structures:

1) FCP Primary Inlet Weir
2) Sheyenne River Spillway Weir
3) Maple River Spillway Weir

ATTACHMENT

F-S1.1 Mohr-Coulomb Envelope for Soils Backfill Behind Weirs

Paul Nielsen

From: Jed Greenwood
Sent: Tuesday, June 15, 2010 2:04 PM
To: Aaron Grosser; Paul Nielsen
Cc: Miguel Wong; Ivan Contreras
Subject: Brenna Peak Strength Envelope

All,

I came up with a drained envelope for use in the Brenna Formation clays. It is based on the peak strength of the material from CIU triax and direct shear testing. It is intended to be used only in design of piles where a sustained load will be present, e.g. soil backfill behind a sheet pile wall, etc. Between 0.5 and 1.0 tsf (1,000 to 2,000 psf), the Mohr-Coulomb envelope is:

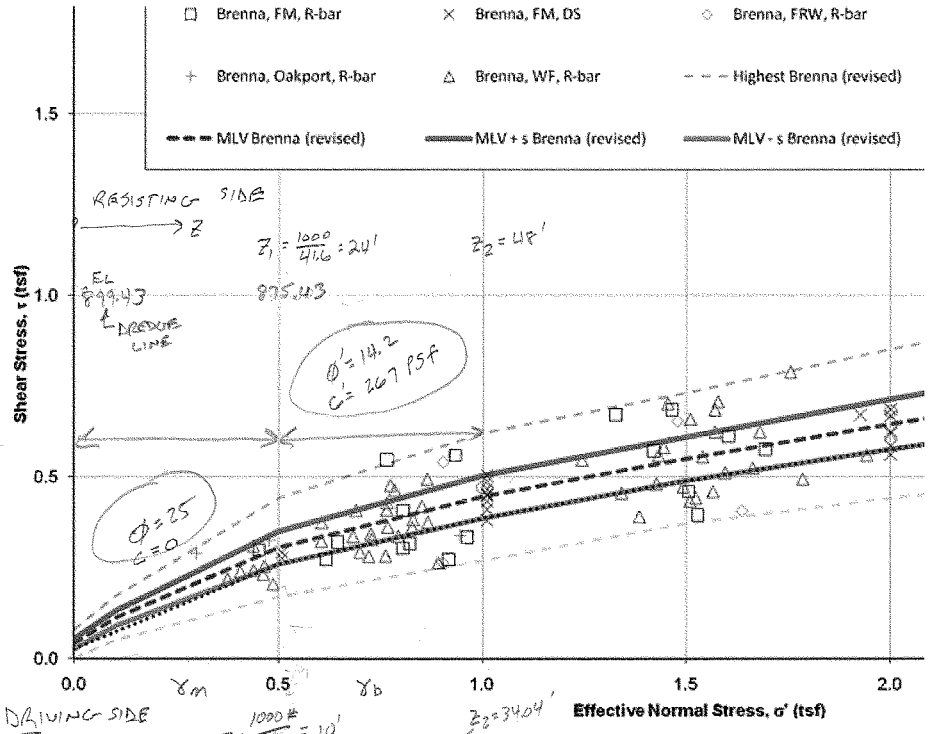
ϕ'=14.2 degrees, c'=267 psf

This envelope was determined based on the methodology the Corps used, which is the most likely value (MLV) minus 1 standard deviation. The plot is below.

Jed

- USE EM 1110-2-2504
- P. 6-1 FOR ALLOW. STRESS
- P. 9-2 CORROSION
- P. 5-3 FOR F.S. (PASSIVE SIDE) IN STABILITY COMP

USED 1.3 INCREASE ON DEPTH FOR F.S. COE METHOD NOT USED IN FEASIBILITY



Jed Greenwood, PE, PG

Geotechnical Engineer
Minneapolis office: 952.832.2773
cell: 952.769.7488
jgreenwood@barr.com
www.barr.com

resourceful. naturally. BARR

ATTACHMENT

F-S1.2 Design Computations for Sheetpile Weirs

X-Sec DEPTH FOR F.S.

$d_1 = 1.3 * 19.44 = 25.3$ ft

USE LARGER SECTION FOR CORROSION IN FUTURE

USE PZ27
$S_X = 30.2$ in³/ft
$I_X = 194.2$ in⁴/ft

FM10DR.OUT

$S_{X\,REQ'D} = \frac{M}{F_S} = \frac{24.349 \times 12}{0.5 * 50\ ksi} = 11.69$ in³/ft

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 16-JUN-2010 TIME: 11.41.00

INPUT DATA

I.--HEADING:
'FLOOD CONTROL METRO FEASIBILITY STUDY, DRAINED PHI=25, C=0
'DIVERSION INLET WEIR, H=10.07 FT

DEFLECTIONS SHOULD USE SOIL-INTERACTION PROGRAM.

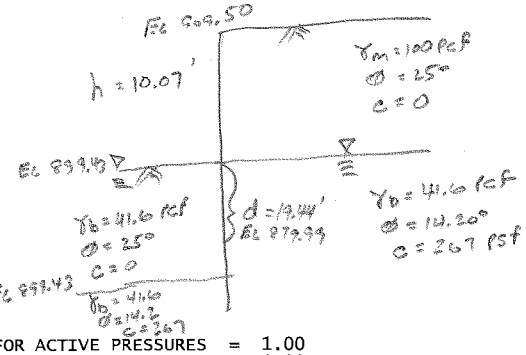
$M = 24.349$ K-ft/ft
$d = 19.44'$
$\Delta = \frac{1.0664 \times 10^{10}}{EI}$
$\Delta = \frac{765.66}{I_x}$

II.--CONTROL
CANTILEVER WALL DESIGN

LEVEL 1 FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.00
LEVEL 1 FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.00

III.--WALL DATA
ELEVATION AT TOP OF WALL = 909.50 (FT)

IV.--SURFACE POINT DATA



IV.A--RIGHTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	909.50

IV.B-- LEFTSIDE

DIST. FROM WALL (FT)	ELEVATION (FT)
.00	899.43

V.--SOIL LAYER DATA

V.A.--RIGHTSIDE LAYER DATA
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.00
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.00

SAT. WGHT. (PCF)	MOIST WGHT. (PCF)	ANGLE OF INTERNAL FRICTION (DEG)	COH-ESION (PSF)	ANGLE OF WALL FRICTION (DEG)	ADH-ESION (PSF)	<--BOTTOM--> ELEV. (FT)	SLOPE (FT/FT)	<-SAFETY-> <-FACTOR-> ACT.	PASS.
104.00	100.00	25.00	.0	.00	.0	899.43	.00	1.00	1.00
104.00	104.00	14.20	267.0	.00	.0			DEF	DEF

V.B.-- LEFTSIDE LAYER DATA
LEVEL 2 FACTOR OF SAFETY FOR ACTIVE PRESSURES = 1.00
LEVEL 2 FACTOR OF SAFETY FOR PASSIVE PRESSURES = 1.00

SAT.	MOIST	ANGLE OF INTERNAL	COH-	ANGLE OF WALL	ADH-	<--BOTTOM-->	<-SAFETY-> <-FACTOR->

WGHT. (PCF)	WGHT. (PCF)	FRICTION (DEG)	ESION (PSF)	FRICTION (DEG)	ESION (PSF)	ELEV. (FT)	SLOPE (FT/FT)	ACT.	PASS.
104.00	104.00	25.00	.0	.00	.0	875.43	.00	1.00	1.00
104.00	104.00	14.20	267.0	.00	.0			DEF	DEF

VI.--WATER DATA

UNIT WEIGHT = 62.40 (PCF)
RIGHTSIDE ELEVATION = 899.43 (FT)
LEFTSIDE ELEVATION = 899.43 (FT)
NO SEEPAGE

VII.--SURFACE LOADS
NONE

VIII.--HORIZONTAL LOADS
NONE

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 16-JUN-2010 TIME: 11.41.12

SOIL PRESSURES FOR
CANTILEVER WALL DESIGN

I.--HEADING

'FLOOD CONTROL METRO FEASIBILITY STUDY, DRAINED PHI=25, C=0
'DIVERSION INLET WEIR, H=10.07 FT

II.--SOIL PRESSURES

RIGHTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS
AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

LEFTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS
AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

	<-LEFTSIDE PRESSURES->		<---NET PRESSURES----> (SOIL PLUS WATER)		<RIGHTSIDE PRESSURES->	
ELEV. (FT)	PASSIVE (PSF)	ACTIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)	ACTIVE (PSF)	PASSIVE (PSF)
909.50	.00	.00	.000	.000	.00	.00
908.50	.00	.00	40.586	246.391	40.59	246.39
907.50	.00	.00	81.172	492.783	81.17	492.78
906.50	.00	.00	121.758	739.174	121.76	739.17
905.50	.00	.00	162.343	985.565	162.34	985.57

904.50	.00	.00	202.929	1231.956	202.93	1231.96
903.50	.00	.00	243.515	1478.348	243.52	1478.35
902.50	.00	.00	284.101	1724.739	284.10	1724.74
901.50	.00	.00	324.687	1971.130	324.69	1971.13
900.50	.00	.00	365.273	2217.522	365.27	2217.52
899.50	.00	.00	405.859	2463.913	405.86	2463.91
899.43+	.00	.00	408.700	2481.160	408.70	2481.16
899.43-	.00	.00	194.564	2347.589	194.56	2347.59
898.50	95.32	15.70	122.686	2395.726	218.01	2411.43
898.43	102.50	16.88	117.276	2399.349	219.77	2416.23
897.50	197.82	32.59	45.398	2447.486	243.22	2480.07
896.91	258.03	42.50	.000	2477.889	258.03	2520.39
896.71	279.18	45.99	-15.945	2488.567	263.23	2534.55
896.50	300.32	49.47	-31.890	2499.246	268.43	2548.71
895.50	402.82	66.35	-109.178	2551.006	293.64	2617.36
894.50	505.32	83.24	-186.466	2602.765	318.85	2686.00
893.50	607.82	100.12	-263.754	2654.525	344.06	2754.65
892.50	710.32	117.00	-341.042	2706.285	369.27	2823.29
891.50	812.82	133.89	-418.330	2758.045	394.49	2891.93
890.50	915.31	150.77	-495.618	2809.805	419.70	2960.58
889.50	1017.81	167.66	-572.906	2861.565	444.91	3029.22
888.50	1120.31	184.54	-650.194	2913.324	470.12	3097.86
887.50	1222.81	201.42	-727.482	2965.084	495.33	3166.51
886.50	1325.31	218.31	-804.770	3016.844	520.54	3235.15
885.50	1427.81	235.19	-882.058	3068.604	545.75	3303.79
884.50	1530.31	252.07	-959.346	3120.364	570.96	3372.44
883.50	1632.81	268.96	-1036.633	3172.124	596.17	3441.08
882.50	1735.30	285.84	-1113.921	3223.884	621.38	3509.72
881.50	1837.80	302.72	-1191.209	3275.643	646.59	3578.37
880.50	1940.30	319.61	-1268.497	3327.403	671.80	3647.01
879.50	2042.80	336.49	-1345.785	3379.163	697.02	3715.66
878.50	2145.30	353.38	-1423.073	3430.923	722.23	3784.30
877.50	2247.80	370.26	-1500.361	3482.683	747.44	3852.94
876.50	2350.30	387.14	-1577.649	3534.443	772.65	3921.59
875.50	2452.80	404.03	-1654.937	3586.202	797.86	3990.23
875.43+	2459.97	405.21	-1597.061	3697.754	799.62	3995.03
875.43-	2333.40	189.35	-1597.061	3697.754	799.62	3995.03
874.50	2397.24	212.80	-1574.168	3846.075	823.07	4058.87
873.50	2465.88	238.01	-1617.601	3889.508	848.28	4127.52
872.50	2534.52	263.22	-1661.033	3932.941	873.49	4196.16
871.50	2603.17	288.43	-1704.466	3976.374	898.70	4264.80
870.50	2671.81	313.64	-1747.899	4019.806	923.91	4333.45
869.50	2740.45	338.85	-1791.332	4063.239	949.12	4402.09
868.50	2809.10	364.06	-1834.764	4106.672	974.33	4470.73

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 16-JUN-2010 TIME: 11.41.31

ÉÍÍÍ

FM10DR.OUT

I.--HEADING

'FLOOD CONTROL METRO FEASIBILITY STUDY, DRAINED PHI=25, C=0
'DIVERSION INLET WEIR, H=10.07 FT

II.--SUMMARY

RIGHTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS
AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

LEFTSIDE SOIL PRESSURES DETERMINED BY COULOMB COEFFICIENTS
AND THEORY OF ELASTICITY EQUATIONS FOR SURCHARGE LOADS.

WALL BOTTOM ELEV. (FT)	:	879.99
PENETRATION (FT)	:	19.44
MAX. BEND. MOMENT (LB-FT)	:	24349.
AT ELEVATION (FT)	:	889.19
MAX. SCALED DEFL. (LB-IN3):		1.0604E+10
AT ELEVATION (FT)	:	909.50

(NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF
ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA
IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)

PROGRAM CWALSHT-DESIGN/ANALYSIS OF ANCHORED OR CANTILEVER SHEET PILE WALLS
BY CLASSICAL METHODS

DATE: 16-JUN-2010 TIME: 11.41.31

COMPLETE RESULTS FOR
CANTILEVER WALL DESIGN

I.--HEADING

'FLOOD CONTROL METRO FEASIBILITY STUDY, DRAINED PHI=25, C=0
'DIVERSION INLET WEIR, H=10.07 FT

II.--RESULTS

ELEVATION (FT)	BENDING MOMENT (LB-FT)	SHEAR (LB)	SCALED DEFLECTION (LB-IN3)	NET PRESSURE (PSF)
909.50	0.	0.	1.0604E+10	.00
908.50	7.	20.	1.0035E+10	40.59
907.50	54.	81.	9.4668E+09	81.17
906.50	183.	183.	8.8982E+09	121.76
905.50	433.	325.	8.3300E+09	162.34
904.50	846.	507.	7.7625E+09	202.93

903.50	1461.	731.	7.1965E+09	243.52
902.50	2320.	994.	6.6331E+09	284.10
901.50	3463.	1299.	6.0738E+09	324.69
900.50	4931.	1644.	5.5204E+09	365.27
899.50	6764.	2029.	4.9757E+09	405.86
899.43	6907.	2058.	4.9379E+09	408.70
899.43	6907.	2058.	4.9379E+09	194.56
898.50	8895.	2205.	4.4426E+09	122.69
898.43	9050.	2214.	4.4058E+09	117.28
897.50	11149.	2289.	3.9250E+09	45.40
896.91	12499.	2303.	3.6297E+09	.00
896.71	12974.	2301.	3.5277E+09	-15.94
896.50	13448.	2296.	3.4266E+09	-31.89
895.50	15715.	2226.	2.9515E+09	-109.18
894.50	17873.	2078.	2.5035E+09	-186.47
893.50	19845.	1853.	2.0863E+09	-263.75
892.50	21553.	1550.	1.7035E+09	-341.04
891.50	22920.	1171.	1.3578E+09	-418.33
890.50	23868.	714.	1.0516E+09	-495.62
889.50	24321.	179.	7.8665E+08	-572.91
888.50	24201.	-432.	5.6362E+08	-650.19
887.50	23431.	-1121.	3.8231E+08	-727.48
886.50	21933.	-1887.	2.4140E+08	-804.77
885.50	19631.	-2731.	1.3826E+08	-882.06
884.50	16446.	-3651.	6.8922E+07	-959.35
883.75	13422.	-4395.	3.5758E+07	-1017.49
883.50	12306.	-4611.	2.7864E+07	-729.71
882.50	7523.	-4760.	7.9692E+06	432.33
881.50	3173.	-3747.	1.1365E+06	1594.37
880.50	417.	-1571.	1.6290E+04	2756.41
879.99	0.	0.	0.0000E+00	3354.02

(NOTE: DIVIDE SCALED DEFLECTION BY MODULUS OF ELASTICITY IN PSI TIMES PILE MOMENT OF INERTIA IN IN**4 TO OBTAIN DEFLECTION IN INCHES.)

III.--SOIL PRESSURES

ELEVATION	< LEFTSIDE PRESSURE (PSF)>		<RIGHTSIDE PRESSURE (PSF)>	
(FT)	PASSIVE	ACTIVE	ACTIVE	PASSIVE
909.50	0.	0.	0.	0.
908.50	0.	0.	41.	246.
907.50	0.	0.	81.	493.
906.50	0.	0.	122.	739.
905.50	0.	0.	162.	986.
904.50	0.	0.	203.	1232.
903.50	0.	0.	244.	1478.
902.50	0.	0.	284.	1725.
901.50	0.	0.	325.	1971.
900.50	0.	0.	365.	2218.
899.50	0.	0.	406.	2464.
899.43+	0.	0.	409.	2481.
899.43-	0.	0.	195.	2348.
898.50	95.	16.	218.	2411.
898.43	102.	17.	220.	2416.
897.50	198.	33.	243.	2480.
896.91	258.	43.	258.	2520.
896.71	279.	46.	263.	2535.
896.50	300.	49.	268.	2549.
895.50	403.	66.	294.	2617.
894.50	505.	83.	319.	2686.
893.50	608.	100.	344.	2755.
892.50	710.	117.	369.	2823.

891.50	813.	134.	394.	2892.
890.50	915.	151.	420.	2961.
889.50	1018.	168.	445.	3029.
888.50	1120.	185.	470.	3098.
887.50	1223.	201.	495.	3167.
886.50	1325.	218.	521.	3235.
885.50	1428.	235.	546.	3304.
884.50	1530.	252.	571.	3372.
883.75	1607.	265.	590.	3424.
883.50	1633.	269.	596.	3441.
882.50	1735.	286.	621.	3510.
881.50	1838.	303.	647.	3578.
880.50	1940.	320.	672.	3647.
879.99	2043.	336.	697.	3716.
878.50	2145.	353.	722.	3784.

RED RIVER DIVERSION

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

APPENDIX F – HYDRAULIC STRUCTURES EXHIBIT T – STRUCTURAL DESIGN COMPUTATIONS— STEPPED SPILLWAYS

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

By: BARR ENGINEERING

FINAL – Version February 28, 2011

TABLE OF CONTENTS

F-T1.0 STEPPED SPILLWAYS 2

ATTACHMENTS

F-T1.1 Design Computations for Stepped Spillway Structures at the Lower Rush and Rush Rivers from Phase 3 submittal

F-T1.2 Design Computations for Stepped Spillway Structures at the Lower Rush and Rush Rivers for Phase 4 submittal

APPENDIX F
HYDRAULIC STRUCTURES

EXHIBIT T – STRUCTURAL DESIGN COMPUTATIONS—STEPPED SPILLWAYS

F-T1.0 STEPPED SPILLWAYS

A description of the structural design concepts and criteria for the stepped spillways at the Lower Rush River and Rush River can be found in Appendix F Section 4.0. *Attachment F-T1.1 of this Exhibit T presents the structural design computations for the Lower Rush River and Rush River stepped spillway structures previously presented as Exhibit P of Appendix F of the Phase 3 report submitted on August 6th, 2010.* Attachment F-T1.2 of this Exhibit T presents the structural design and quantity computations for the Lower Rush River and Rush River stepped spillway structures for Phase 4.

BARR — MINNEAPOLIS, MINNESOTA - HIBBING, MINNESOTA
DULUTH, MINNESOTA
ANN ARBOR, MICHIGAN - JEFFERSON CITY, MISSOURI

COMPUTED	CHECKED	SUBMITTED
BY	BY	TO
DATE	DATE	DATE

DATE | SHEET NO. Ø

PROJECT NAME RED RIVER DIVERSION

PROJECT NUMBER 34/09-1004

SUBJECT

RUSH & LOWER RUSH DIVERSION WALL

STRUCTURES - QUANTITY CALCULATION

INDEX

SUBJECT	PAGE
- LOAD CASES & BASIS	1
- WALL LOADING LC1	2-4
- SPREADSHEET FOR LC1,2,3 FOR ANALYSIS OF OVERTURNING MOMENT & BEARING PRESSURES	5
- WALL LOADING LC 2	6
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- STEPS PILING & WALL ETC, THICKNESS	11
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PROJECT NAME RED RIVER DIVERSION

PROJECT NUMBER 34/09-1004

SUBJECT RUSH & LOWER RUSH RIVER DIV.

PILING DESIGN

LC 1 - ACTIVE SOIL PRESSURE RETAINING WALL W/ WATER TABLE @ 5' DOWN - NO FLOW IN RIVER

USE UNDRAINED PILE VALUES - USUAL LC FS = 2

LC 2 - WATER PRESSURE ONLY AT LOCATION OF NO SOIL - NOT A REAL L.C., BUT SOMETHING ALONG THESE LINES MAY OCCUR NEAR FISH PASSAGE -

USE UNDRAINED PILE VALUES - USUAL LC FS = 2

LC 3 - ACTIVE SOIL PRESSURE W/ WATER TABLE LOW - NO FLOW IN RIVER - USE DRAINED PILE VALUES - ASSUME UNUSUAL LC IS APPROPRIATE FOR THIS F.S. =

BASIS

RETAINING WALLS FOR THE RUSH & LOWER RUSH RIVERS HAVE SIMILAR GEOMETRIES. THIS CALCULATION CONSERVATIVELY TAKES THE WORST CASE WALL & COMPUTES QUANTITIES FOR BOTH RIVER DIVERSIONS BASED ON THIS CRITICAL WALL GEOMETRY (FOOTING WIDTH & THICKNESS + WALL THICKNESS - WALL HEIGHT IS MODIFIED TO MATCH CURRENT LAYOUT). THESE CALCULATIONS & DESIGN WORK ASSOCIATED WITH THESE CALCULATIONS IS ONLY REQ'D IN ORDER TO SUPPORT THE CALCULATION OF CONCRETE & REBAR QUANTITIES. DESIGN SHOULD NOT BE CONSIDERED "FOR CONSTRUCTION"

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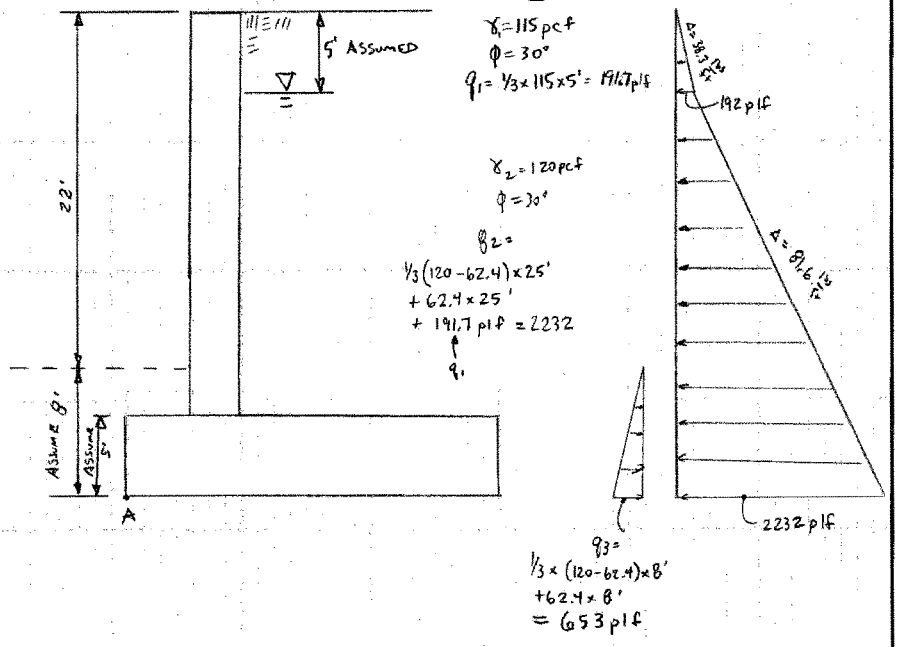
			PROJECT NAME RED RIVER DIVERSION	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER 34/09-1004	
BY JGT	BY	TO	SUBJECT RUSH & LOWER RUSH RIVER WALLS	
DATE 7-22-10	DATE	DATE		

LC 1
LARGEST RETAINED SOIL HT = 22'

ACTIVE PRESSURE



TOTAL LATERAL SHEAR

$= 1/2 \times 192\text{plf} \times 5' + 192\text{plf} \times 25' + 1/2 \times 2040\text{plf} \times 25' - 1/2 \times 653\text{plf} \times 8'$

$480^{\text{lbs}} + 4800^{\text{lbs}} + 25,500^{\text{lbs}} - 2612^{\text{lbs}} = \underline{28,168^{\text{lbs}}}$

TOTAL MOMENT ABOUT A

$480^{\text{lbs}} \times 26.67' + 4800^{\text{lbs}} \times \frac{25'}{2} + 25,500^{\text{lbs}} \times \frac{25'}{3} - 2612^{\text{lbs}} \times \frac{8'}{3} = \underline{278.3^{\text{ft-k}}}$

MOMENT AT BOTTOM OF WALL

$M_u = 1.6 \times \left[480^{\text{lbs}} \times 21.67' + 3840^{\text{lbs}} \times \frac{20'}{2} + 1632\text{plf} \times 1/2 \times 20' \times \frac{20'}{3}\right] = \underline{252.2^{\text{ft-k}}}$

↑ LOAD FACTOR

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			DATE	SHEET NO. 3
			PROJECT NAME RED RIVER DIVERSION	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER 34/09-1004	
BY JGT	BY	TO	SUBJECT	
DATE 7-22-10	DATE	DATE	RUSH & LOWER RUSH RIVER DIVERSION	

ASSUME: 3' THICK WALL, $d = 32"$ $\Rightarrow A_s$ REQD $= 1.83 in^2$ SEE #11 @ 9" $= 2.08 in^2$, ATTACHED SHEET

3'-6" THICK WALL, $d = 38"$ $\Rightarrow A_s$ REQD $= 1.52 in^2$ #11 @ 12" $= 1.56 in^2$

$$\Delta_g \approx \frac{(480 + 3840 + 16{,}320) \times (20')^3 \times (12"/ft)^3}{15 \times (57000 \times \sqrt{4000}) \times (1/12 \times 12" \times 42^3)} = 0.07"$$

Δ_g (3'-6") OK SMALL - ASSUME OK FOR PARTIALLY CRACKED SECTION ALSO

Δ_g (3'-0") $= 0.113"$ ALSO OK - SMALL

* USE 3' THICK WALL

CHECK OVERTURNING RESISTANCE MOMENT

SEE SPREADSHEET

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SHEET #4 34/09-1004

Flexural Concrete Design (As unknown)

Mu	252.2 [ft-k/ft]
d	32 [in]

If f'c=3000psi, fy=60000psi	
Asreq'd	1.855798 [in^2/ft]
As (min)	1.28 [in^2/ft]

If f'c=3000psi, fy=40000psi	
Asreq'd	2.784408 [in^2/ft]
As (min)	1.28 [in^2/ft]

If f'c=4000psi, fy=60000psi	
Asreq'd	1.828861 [in^2/ft] *
As (min)	1.28 [in^2/ft]

If f'c=4000psi, fy=40000psi	
Asreq'd	2.770602 [in^2/ft]
As (min)	1.28 [in^2/ft]

If f'c=2500psi, fy=60000psi	
Asreq'd	1.892169 [in^2/ft]
As (min)	2.522892 [in^2/ft]

If f'c=2500psi, fy=40000psi	
Asreq'd	2.832228 [in^2/ft]
As (min)	1.28 [in^2/ft]

Flexural Concrete Design (Mu cap)

As	1.56 [in^2/ft]
d	42 [in]

If f'c=3000psi, fy=60000psi		
Mu	215.2273 [in^2/ft]	(uses 3/4 A
	284.2836	(uses full A

If f'c=3000psi, fy=40000psi		
Mu	144.7917 [in^2/ft]	(uses 3/4 A
	191.8623 [in^2/ft]	(uses full A

If f'c=4000psi, fy=60000psi		
Mu	216.5257 [in^2/ft]	(uses 3/4 A
	286.6879 [in^2/ft]	(uses full A

If f'c=4000psi, fy=40000psi		
Mu	143.9438 [in^2/ft]	(uses 3/4 A
	191.0479 [in^2/ft]	(uses full A

If f'c=2500psi, fy=60000psi		
Mu	212.7602 [in^2/ft]	(uses 3/4 A
	280.4804 [in^2/ft]	(uses full A

If f'c=2500psi, fy=40000psi		
Mu	143.7385 [in^2/ft]	(uses 3/4 A
	190.2231 [in^2/ft]	(uses full A

IGNORE

* THIS CALC JUSTIFIES #11 @ 9" FOR 252.2 ft-k & 36" WIDE WALL AT WORST CASE WALL MOMENT.

NOTE: REBAR QUANTITY TOTALS ASSUME #11 @ 12" OVER ENTIRE WALL AS THIS CRITICAL MOMENT LIKELY ONLY OCCURS IN SMALL REGION

JGT 7-22-10

RED RIVER DIVERSION
34/09-1004
RUSH & LOWER RUSH DIVERSION
SHEET #5

WALL RESISTANCE AGAINST OVERTURNING

	LOAD CASE 1 RETAINING SOIL	LOAD CASE 2 RETAINING WATER	LC3 Drained
width ftg	22 ft	22 ft	22 ft
thickness ftg	5 ft	5 ft	5 ft
wall ht	25 ft	25 ft	25 ft
wall thickness	3 ft	3 ft	3 ft
location of wall from pt A	8 ft	14 ft	8 ft
Mr	837.5156 ft-kips	689.3906	837 ft-kips
Moverturn	278.3 ft-kips	231.5	165.5347 ft-kips
safety factor against OT	3.009399	2.977929	5.056343

BEARING PRESSURE

Net Moment About center	LC1	LC2
of ftg	141.3469 ft-kips	119.165 ft-kips
e	2.219382 ft	3.145025 ft
B/6	3.666667 ft	3.666667 ft
Q	63.6875 kips	37.89 ft
q max	4.64712 ksf	3.199525 ksf
q min	1.142652 ksf	0.245021 ksf
q norm	2.894886 ksf	1.722273 ksf

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			PROJECT NAME RED RIVER DIVERSION
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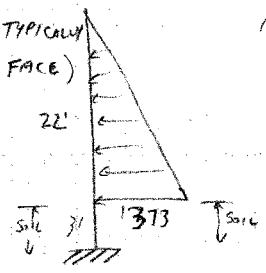
BY JGT	BY	TO	SUBJECT RUSH & LOWER RUSH DIVERSION
DATE 7/22/10	DATE	DATE	

LC 2

CHECK MOMENT UNDER WATER

(NOT REAL REALISTIC CASE AS SOIL IS TYPICALLY ON OPPOSITE FACE)



LOAD FACTOR

$M_u = 1.6 \times 10.33' \times 15.1^K = 249.5^{ft\text{-}k}$

$< 252.2^{ft\text{-}k}$ ok — Doesn't Control Wall Design

$M_A = 15.1^K \times 15.33' = 231.5^{ft\text{-}k}$

$< 278.3^{ft\text{-}k}$

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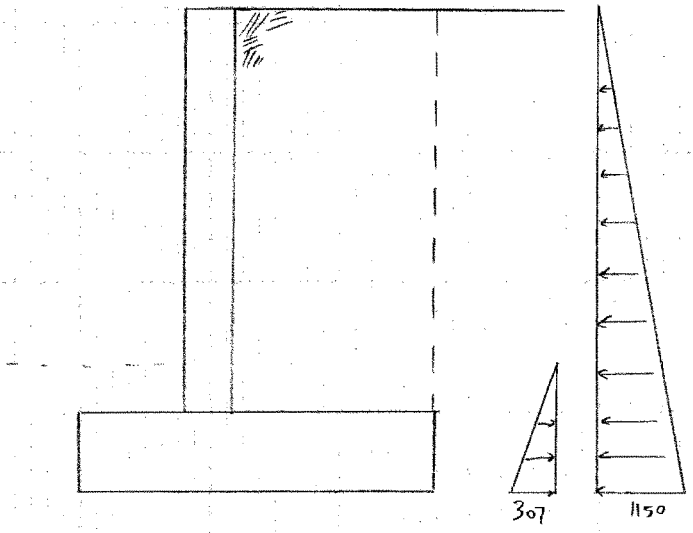
			DATE	SHEET NO. 7
			PROJECT NAME RED RIVER DIVERSION	
COMPUTED	CHECKED	SUBMITTED	PROJECT NUMBER 34/09-1004	
BY JGT	BY	TO	SUBJECT RUSH & LOWER RUSH RIVER DIVERSIONS	
DATE 7/22/10	DATE	DATE		

LC3 - DRAINED - NO H_2O TABLE



$M_{OT} = \frac{1}{2} \times 1150 \times 30 \times \frac{30}{3} - \frac{1}{2} \times 307 \times 8 \times \frac{8}{3} = 169.2^{ft\text{-}k}$

$M_r = .15 \times 5' \times 22' \times 11' + .15 \times 3' \times 25' \times 8' + .115 \times 12.5' \times 25' \times 15.75' = 837^{ft\text{-}k}$

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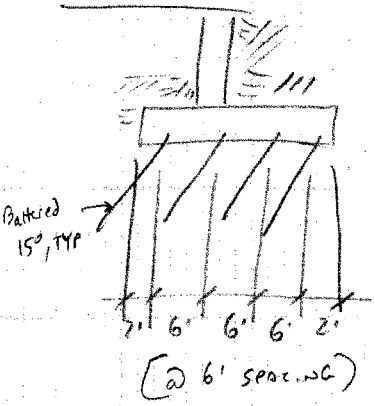
DATE — SHEET NO. 8
PROJECT NAME RED RIVER DIVERSION
PROJECT NUMBER 34/09-1004
SUBJECT RUSH & LOWER RUSH RIVER DIVERSIONS

LC 1 PILING



(@ 6' SPACING)

USE SPACING TO NOT REQUIRE PILING STRENGTH REDUCTIONS [6' > 5x1'/12 = 5.83']
@ 6' LONGITUDINAL SPACING

HORIZ. $4 \times [(237.9 \times \sin 15) + 35]$

$= 386.3^{k}$

REQ'D = $28.2^{k} \times 6' = 169.2^{k}$

F.S. > 2 ok

VERT $4 \times 237.9 \times \cos 15 = 919^{k}$

$2.89^{ksf} \times 6' \times 22' = 382^{k}$

↑ Ave bearing

F.S. = 2.4

MAX ⇒ 4.65^{ksf}

$6' \times 5' \times \left(4.647 - \left(\frac{2.5}{22'} \times (4.647 - 1.143)\right)\right) = 127.5^{k}$

$1 \times 237.9 \times \cos 15 = 230$

F.S. = 1.8

FOR ESTIMATING PURPOSES, ASSUME OK — THIS IS WORST CASE LOAD CONDITION, PILE ARRANGEMENT COULD POSSIBLY BE ADJUSTED LOCALLY IF FOUND UNACCEPTABLE IN FINAL CALCS.

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			PROJECT NAME RED RIVER DIVERSION	
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BY JGT	BY	TO	SUBJECT	
DATE 7-22-10	DATE	DATE	RUSH & LOWER RUSH RIVER DIVERSIONS	

LC 2

UPLIFT ↓

HORIZ = $4 \times (216.6 \times \sin 15 + 35) = 364.2^{k}$

FORCE = $\frac{1}{2} \times 1373 \times 22 = 15.1^{k} \times 6' = 90.6^{k}$

ok

VERT = $4 \times 216.6 \times \cos 15 = 837^{k} > 22' \times 1.72^{KSF} = 37.8^{k} \times 6 = 227^{k}$

ok

VERT MAX — SAY OK AT THIS POINT OF ESTIMATE

SEE LC 1

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BARR	MINNEAPOLIS, MINNESOTA - HIBBING, MINNESOTA DULUTH, MINNESOTA ANN ARBOR, MICHIGAN - JEFFERSON CITY, MISSOURI		DATE	SHEET NO. 10
			PROJECT NAME RED RIVER DIVERSION	
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BY JGT	BY	TO	SUBJECT	
DATE 7-22-10	DATE	DATE	RUSH & LOWER RUSH RIVER DIVERSIONS	

LC 3

Horiz Force = $\frac{1}{2}\left(\frac{1}{3} \times 115\ pcf \times 30'\right) \times 30' = 17.25^{k}$

$17.25^{k} \times 6'$ PILE SPACING $= 103.5^{k}$

PILE RESISTANCE $= 4 \times 188^{k} \times \sin 15 + 4 \times 22 = 282.6^{k}$

F.S. > 2 ok

VERT

$63.69^{k} \times 6' = 382.1^{k}$

4 PILES $\times$ $188^{k} \times \cos 15 = 726.4^{k}$

F.S. = 1.90 > 1.5 ok

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DATE — SHEET NO. 11
PROJECT NAME RED RIVER DIVERSION
PROJECT NUMBER 34/09-1004
SUBJECT RUSH & LOWER RUSH RIVER DIVERSIONS

STEPS (LOWER RUSH)

341.9 ft² × 150pcf = 51.3 K/ft of width

20 ft of WATER = 43.1 K/ft of width

94.4 K/ft of width

Assume 6' SPACING

94.4 × 6 = 566.4K PER ROW

314 × 6 = 1884

237.9 × 6 = 1427 > 566.4 × 2 = 1132 (↑ F. of Safety)

USE 6 PILES PER STEPS
@ 6' O.C. SPACING

WALL FTG

LEAVE AT 5' THICK FROM PREVIOUS ESTIMATE TO ACT AS PILE CAP. MOMENT AT THIS THICKNESS SHOULD BE OK.

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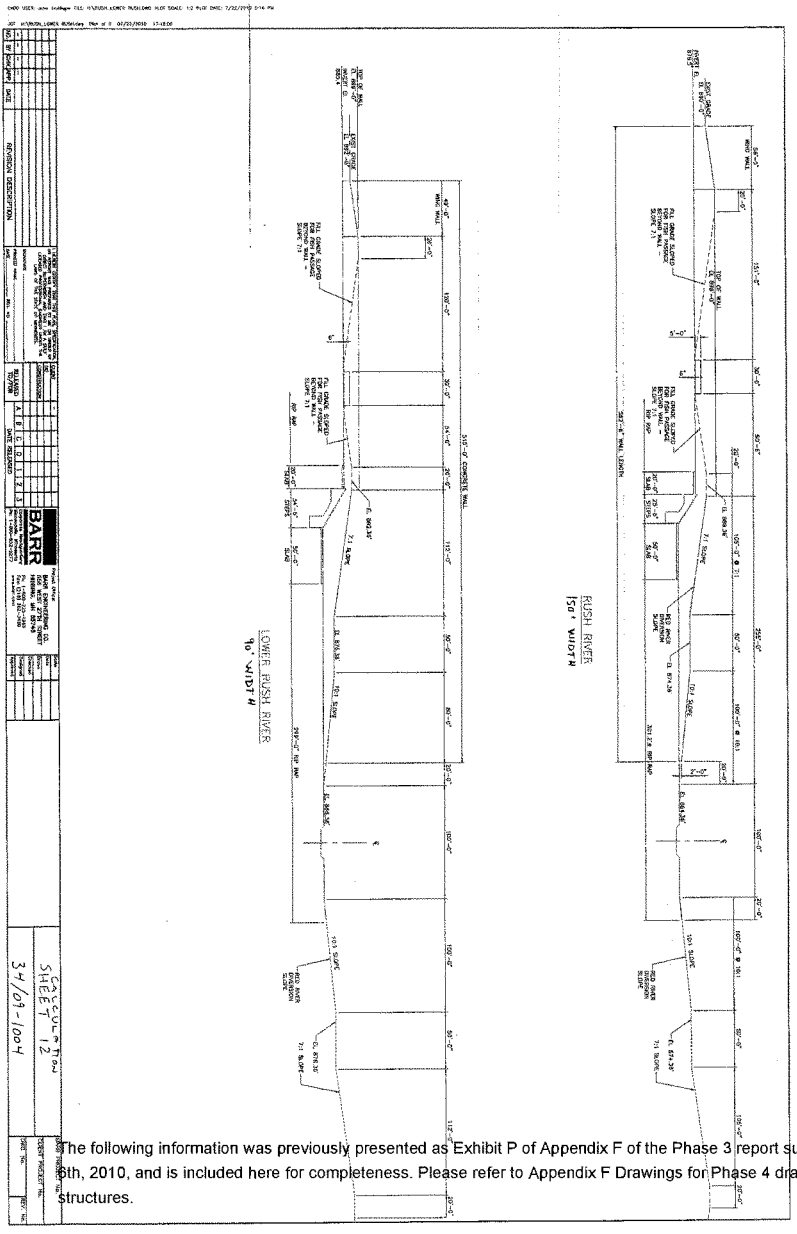

The following information was previously presented as Exhibit P of Appendix F of the Phase 3 report submitted on August 6th, 2010, and is included here for completeness. Please refer to Appendix F Drawings for Phase 4 drawings of these structures.

Sheet 13

Background

This section is applicable for the following structures as part of the Red River Diversion:

1) ND Concrete Structure for Diversion and Termination of Rush River
2) ND Concrete Structure for Diversion and Termination of Lower Rush River

The Rush and Lower Rush Rivers will not continue to the actual Red River under the plan being analyzed in this computation. They will terminate into the Red River Diversion channel, and the Red River Diversion channel will then terminate in the Red River downstream of the Fargo / Moorehead area.

This calculation is used in order to compute quantities to be used as part of a budgetary estimate for the project. This computation is not for final design.

These two structures are very similar, so are analyzed in tandem. Both structures consist of retaining walls on each side of the river. These retaining walls will confine the flow of the river with a top of wall elevation 3 feet above the elevation of a 500 year flood for each river. Once these walls hit the sloping sides of the Red River Diversion, the top of wall is dropped to follow the slope of the diversion sides. At the sloping sides, river flow is required to pool up and pass over and down a set of steps. Prior to running into the Red River Diversion, the north wall has a 30 feet wide x 5 feet high "fish passage" leading to a man-made stream to travel down to the bottom of the Red River Diversion on a much more gradual path.# The "fish passage" has a lower invert elevation than the top step, so will be the usual flow path in the system with the steps providing flood relief.

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

Load Cases used for the Rush & Lower Rush River Structures

1. 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 feet 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.
2. Since the fish passage has no retained soil behind it, this load case looked at 22 feet 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.
3. 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 feet above final grade. Wall thicknesses, footing size, and piling design were computed to satisfy 22 feet retained height. Conservatively, this same concrete thicknesses and pile requirements were continued throughout the design. Footings for walls were set at 5 feet 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

#The design changed from Phase 3 to Phase 4. For Phase 4 this sentence should read: Prior to running into the Red River Diversion, the north wall has a 40 foot wide "fish passage" leading to a series of pools and riffles to travel down to the bottom of the Red River Diversion on a more gradual path

sliding force of the wall. Bottom of wall footings were placed 8 feet below grade for frost protection.

The step weight was also supported by piles. Slabs were provided for 20 feet leading into the steps and 50 feet 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 foot of bar length in tall wall sections, 4 pounds per foot in medium height wall sections, and 3 pounds per foot in short wall sections. Horizontal bars were estimated at 3 pounds per foot. Footing bars were estimated at 4 pounds per foot each direction. The slabs on grade were estimated at 3 pounds per foot in each direction. The steps were computed at 4 pounds per foot 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.

Barr Engineering Company Job Number: 34-091-004.300.700

Project: Fargo-Moorhead Metro Flood Risk Management Project | Computed: JGT | Date: 7/23/2010

Subject: Phase 3 Cost Estimate | Checked: | Date:

Task: Cost Estimate - Quantities - ND East 35K Erosion Control and Diversion-Wide | Sheet: | of

Status: *draft*

Hydraulic Structure - Lower Rush - Structure						
	Earthwork - Structural	Earthwork - Structural Excavation	0	BCY		
		Earthwork - Structural Haul	0	BCY		
		Earthwork - Structural Levee Embankment	0	BCY		
	Drop Structure and Walls		0	LF	number of piles: 0	
	Steel Piles	Steel Piles - Furnish Material	53,701	LF		
		Steel Piles - Set up Rig	0	EA		
		Steel Piles - Unload Piles	99	EA	pilesx73 / 40000	
		Steel Piles - Weld Splices	837	EA	cutoffsx.25	
		Steel Piles - Drive Piling	53,701	LF		
		Steel Piles - Cut Offs	837	EA	no. of piles	
		Steel Piles - Pile Test	1	EA		
	Concrete - Footing	Concrete - Footing - Form Strip and Cure	10,300	SF		
		Concrete - Footing - Pour	4,196	CY		
		Concrete - Footing - 5% Waste	210	CY		
	Concrete - Walls	Concrete - Walls - Build Forms	32,329	SF		
		Concrete - Walls - Assemble and Modify Forms	4	EA		
		Concrete - Walls - Form, Strip and Cure	32,329	SF		
		Concrete - Walls - Pour	1,779	CY		
		Concrete - Walls, Piers and Elevated Deck - 5% Waste	89	CY		
	Concrete - Steps	Concrete - Steps - Build Forms	5,140	SF		
		Concrete - Steps - Assemble and Modify Forms	25	EA		
		Concrete - Steps - Form, Strip and Cure	5,140	SF		
		Concrete - Steps - Pour	1,161	CY		
		Concrete - Steps - 5% Waste	58	CY		
	Concrete - Slabs on Grade	Concrete - Slab - Build Forms	270	SF		
		Concrete - Slab - Assemble and Modify Forms	2	EA		
		Concrete - Slab - Form, Strip and Cure	270	SF		
		Concrete - Slab - Pour	508	CY		
		Concrete - Slab - 5% Waste	30	CY		
	Concrete - Elevated Deck	Concrete - Elevated Deck - Form Strip and Cure	0	SF		
		Concrete - Elevated Deck - Pour	0	CY		
		Concrete - Walls, Piers and Elevated Deck - 5% Waste	0	CY		
	Steel Reinforcement Bars	Steel Reinforcement Bars - Furnish and Install	565,000	LB		
		Steel Reinforcement Bars - Hoist	565,000	LB		
	Hydraulic Gates - Steel	Hydraulic Gates - Steel - Furnish	0	SF		
		Hydraulic Gates - Steel - Install	0	EA		
		Hydraulic Gate Hoist - Furnish and Install	0	EA		
	Bulkheads - Steel	Bulkheads - Steel - Furnish	0	LS		
	Bridge Railing	Bridge Railing - Furnish and Install	0	LF		
	Sheet Piling	Sheet Piling - Furnish	0	SF		
		Sheet Piling - Unload Sheeting	0	EA		
		Sheet Piling - Set up Rig	0	EA		
		Sheet Piling - Drive Sheets	0	EA		
		Sheet Piling - Cut Sheets	0	EA		
	Structural Aggregate Backfill	Aggregate Base - Class V - Furnish and Install	#REF!	CCY		
Hydraulic Structure - Rush - Structure						
	Earthwork - Structural	Earthwork - Structural Excavation	0	BCY		
		Earthwork - Structural Haul	0	BCY		
		Earthwork - Structural Levee Embankment	0	BCY		
	Drop Structure and Walls		0	LF	number of piles: 0	
	Steel Piles	Steel Piles - Furnish Material	56,599	LF		
		Steel Piles - Set up Rig	0	EA		
		Steel Piles - Unload Piles	104	EA	pilesx73 / 40000	
		Steel Piles - Weld Splices	850	EA	cutoffsx.25	
		Steel Piles - Drive Piling	56,599	LF		
		Steel Piles - Cut Offs	900	EA	no. of piles	
		Steel Piles - Pile Test	1	EA		

1677

Sheet 15

Sheet 16

| | | |

WALLS (Both Sides) **PHASE 4**
Lower Rush River

Panel	Stem			Ftg			Concrete Volume		Steel	Piles (Both sides)			
	W	Ht	L	B	Toe	L	Ftg	Stem	Reinf	No.	No. rows	Length	Total
		ft			ft		cy	cy	lbs			ft	ft
A	3	13.1	144	22	6	144	587	210	104,903	96	24	74	7104
B	3	16.5	792	22	6	832	3390	1452	639,613	560	140	74	41440
C	3	16	170	22	6	170	693	302	132,521	112	28	55	6160
D	2.5	11	30	22	6	30	122	31	20,746	24	6	55	1320
E	2.5	8	84	22	6	84	342	62	53,654	28	7	55	1540
						Totals:	5133	2057	951,437	820			57,564

WALLS (Both Sides) **PHASE 4**
Rush River

Panel	Stem			Ftg			Concrete Volume		Steel	Piles (Both Sides)			
	W	Ht	L	B	T	L	Ftg	Stem	Reinf	No.	No. rows	Length	Total
		ft			ft		cy	cy	lbs			ft	ft
A	3	17.5	164	22	6	164	668	319	132,173	112	28	71	7952
B	3	21.5	778	22	6	818	3333	1859	697,041	544	136	71	38624
C	3	16.7	166	22	6	166	676	308	131,448	112	28	55.5	6216
D	2.5	11	30	22	6	30	122	31	20,746	24	6	55.5	1332
E	2.5	8	84	22	6	84	342	62	53,654	28	7	55.5	1554
						Totals:	5141	2578	1,035,061	820			55,678

Note: For Stem Panel B, The Length deducts a 40 ft wide fish bypass on one wall only.
Footing is 5 ft thick.
Vertical stem bars dowels included by adding 11' to vertical bars. (4 feet into footing + 2 ft hook and 5 ft lap)
Horizontal stem bars assume a 5 ft lap every 30 ft.
Footing longitudinal barrs assume 5 ft laps every 30 ft.
Vertical Stem rebar assumed to weigh 3, 4 or 5.3 plf at 12" oc
Horizontal Stem rebar assumed to weigh 3 plf at 12" oc.
Footing rebar is assumed to weigh 4 plf each way at 12" oc (Top and bottom bar layers).

DROP STRUCTURE **PHASE 4**

Lower Rush River

Concrete Volumes			Steel	Piles			
Approach	Stilling	Steps	Reinf	No	No	Length	Total
cy	cy	cy	lbs		Rows	ft	ft
70	170	517	78523	20	10	52	1040
				20	10	60	1200
			Total:	40			2240

DROP STRUCTURE **PHASE 4**

Rush River

Concrete Volumes			Steel	Piles			
Approach	Stilling	Steps	Reinf	No	No	Length	Total
cy	cy	cy	lbs		Rows	ft	ft
117	283	794	118720	34	17	45.5	1547
				17	17	56	952
			Total:	51			2499

Notes: Horizontal rebars assume a 4 ft lap every 30 ft.
Total Slab rebar assumed to weigh 3 plf at 12" oc.
Step rebar assumed to weigh 4 plf at 12" oc based on the total perimeter.
10% added to Step rebar to account for laps.

FOR CONTINUATION OF HOUSE DOCUMENT 112-130

FEASIBILITY REPORT AND ENVIRONMENTAL IMPACT STATEMENT: FARGO-MOORHEAD METROPOLITAN AREA FLOOD RISK MANAGEMENT, JULY 2011

SEE PART 3