Appendix B
Hydraulics

Fargo-Moorhead Metropolitan Area Flood Risk Management

Final Feasibility Report and Environmental Impact Statement

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US Army Corps of Engineers ®

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Appendix B

Hydraulics

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B.1.0 GENERAL

This appendix presents a summary of the Phase 4 hydraulic analyses of existing conditions and proposed diversion alternatives for the Red River of the North (RRN) in the vicinity of Fargo and Moorhead. The previous screenings of flood mitigation alternatives presented in the Phase 2 and 3 versions of this appendix are not repeated. All elevations referred to in this appendix are in NAVD 1988 unless otherwise noted

Most of the pertinent hydraulic information has been developed by a team of consulting engineering firms led by Moore Engineering, Incorporated. This work was performed under an in-kind services contract through the local sponsors and is documented in the report, *Red River Diversion, Fargo-Moorhead Metro Flood Risk Management Project, Feasibility Study, Phase 4, February 28, 2011*. **The consultant's report is provided as Attachment 5 of this Feasibility Study. The General Report and Appendices B, C, and F of Attachment 5 cover most of the hydraulic aspects of this study. Instead of repeating that information here, the reader is referred to Attachment 5 for most of the hydraulic information.** This appendix will simply provide a brief summary regarding the status of hydraulic studies and the status of District Quality Control (DQC) and Agency Technical Review (ATR) efforts.

B.2.0 STATUS OF HYDRAULIC STUDIES

B.2.1. WORK COMPLETED

After the downstream impacts of the project developed in Phase 3 were analyzed, it was determined that they were not fully definable and another approach was needed. Phase 4 focuses on developing a Locally Preferred Plan (LPP) that has all impacts fully defined. Following the consideration of multiple options, the USACE and local project sponsors decided to pursue an option that included raising the water levels, or staging, upstream of the FM Metro area. While this would impact homes and properties on the upstream side of the project, this option allows the impacts to be fully defined and there are fewer structures affected by shifting the impacts upstream and the mitigation measures are also less costly for the upstream staging option. In addition to utilizing the existing topography, much of which is already inundated by flooding, this concept also includes a constructed storage area that provides some control on the storage and subsequent release of the water to help with the timing and the impacts at peak flood stage.

In order to develop a design that incorporates the benefits of the upstream storage and staging, the design for this phase required the use of unsteady state modeling techniques. Up until this point, unsteady flow models had only been used to analyze the impacts of the project designs that had been developed with steady state methods, which are simpler and less time intensive to use. The Phase 4 effort involved extensive upgrades to the existing unsteady state existing conditions models followed by full project design utilizing these models to analyze the benefits of upstream storage and staging. This phase involved only the redesign of the Locally Preferred Plan (LPP), which is the North Dakota East Diversion. The design for the Federally Comparable Plan (FCP) completed in Phase 3 is adequate since the project impacts have been fully defined and upstream staging is not required to fully define these impacts. Therefore, no updates were completed for the FCP as part of Phase 4. The background information on the FCP design and other information on the alternatives on the Minnesota side of the Red River can be found in the reports published for Phase 3 and earlier phases of this study. However, the FCP design was analyzed with the updated models developed for Phase 4 to evaluate the downstream impacts.

The hydraulic analysis spans approximately 325 miles of the Red River of the North from near Abercrombie, North Dakota through Fargo, North Dakota and Moorhead, Minnesota to the downstream end at Drayton, North Dakota. The communities of Fargo and Moorhead are located approximately 453 river miles above the mouth of the Red River of the North at Lake Winnipeg, Manitoba. The river model geometry is highlighted in Figure B-1 (taken from Attachment 5). It includes the Red River of the North main stem and several tributaries.

When it was found that downstream impacts could not be fully defined (zero impact location) within the original study extents, the model was first extended to River Mile 316 near Thompson, North Dakota (Phase 3), and then to River Mile 198 at Drayton (Phase 4). It has also been extended upstream on the Red River of the North to near Abercrombie, North Dakota at approximately River Mile 524. The model was also extended farther upstream on the Sheyenne and Maple Rivers to better define the breakouts and flow distribution on the western side of the project.

B.2.1.1. Existing Condition Modeling

The Existing Conditions HEC-RAS unsteady flow model was developed with sufficient detail to be used as a baseline for project feasibility design as well as benefit and impact analysis. It was calibrated based on the 2009 spring flood and the calibration was verified using the 2006, 1997, and 2010 historic spring flood events. Stage and discharge hydrographs and high water mark data were used in the calibration effort.

The temporary flood protection measures, present during the historic flood events and necessary for calibration, were removed to produce the geometry for the unprotected condition. The impacts of the project were determined by comparing the with-project models to the existing condition models without emergency measures in place. This allows for the determination of the full benefits of the project by comparing it to the damages that would be incurred if nothing was done to protect the communities. All impacts presented reference the "without emergency protection" condition for both existing and with-project conditions.

A more complete summary of the Existing Condition modeling effort is available in Attachment 5, General Report, Section 3.2. The reader is referred to Attachment 5, Appendix B to understand the details of the Existing Condition hydraulic study.

B.2.1.2. Flood Events Modeled

The 10-percent, 2-percent, 1-percent, and 0.2-percent annual chance synthetic flood events were developed as the primary means to evaluate Existing Conditions, to assist with project feasibility design, and to analyze potential impacts from flood mitigation alternatives (LPP and FCP) being considered as part of this project.

In addition to the synthetic flood events, four larger recent historic flood events in Fargo-Moorhead (1997, 2006, 2009 and 2010) were also simulated. These model runs are not intended for project feasibility design or for flood damage reduction evaluation. However, they provide two very tangible benefits. First, they offer the possibility to better communicate the project impacts to all stakeholders and the general public because they can relate to how the project would change the conditions that were experienced during the recent larger flood events. The second benefit of having conducted these model runs is that they allow estimation of project upstream staging/storage and downstream impacts without having to assume that the magnitude and timing of tributary flows affect the magnitude and timing of flooding downstream; this is better captured with looking at four historic events versus the synthetic event analysis.

A more complete summary of the synthetic and historic flood model development is available in Attachment 5, General Report, Sections 3.2 and 3.3.

B.2.1.3. Locally Preferred Plan (LPP)

Most simply, the LPP involves a diversion and the temporary staging of water upstream to both reduce flood damages for the Fargo-Moorhead metro area and reduce downstream stage impacts. Downstream stage impacts are reduced to 0.0 between Oslo and Drayton. The LPP provides flood damage reduction up to the 0.2-percent chance flood event for nearly 200,000 people and 80 square miles of infrastructure. The major features of the LPP are shown in Figures B-2 and B-3 (taken from Attachment 5). Figure B-3 is included since Figure B-2 doesn't show the extent of temporary staging area upstream of the control structures south of Fargo-Moorhead. Seven river systems (Red River of the North, Wild Rice River, Wolverton Creek, Sheyenne River, Maple River, Lower Rush River, Rush River) and a number of drains are directly affected by the project. The diversion is approximately 36 miles long while the channel length of the Red River of the North cutoff by the diversion is 60.7 miles.

Low flows for the Sheyenne River and the Maple River will cross the diversion via aqueducts while the Rush River, Lower Rush River, Drain 14, and a number of other smaller ditches will be captured by the diversion. High flows on the Sheyenne River and Maple River will pass into the diversion via spillway weirs adjacent to the aqueducts. The Maple River Aqueduct and the Sheyenne River Aqueduct are designed for fish passage. Fish passage structures are provided at five locations (Outlet Drop Structure, Rush River Drop Structure, Lower Rush River Drop Structure, Wild Rice River Control Structure, Red River of the North Control Structure) to assist fish passage while the project is in operation.

Three control structures (Red River of the North, Wild Rice River, Wolverton Creek) and the Diversion Inlet Structure control the amount and timing of diversion flow and staged water. The area of water staging includes an area of more controlled storage, named Storage Area 1, which assists with reducing stage impacts downstream of the project. The maximum staged water elevation is 922.9 and occurs for the 1.0-percent chance flood event. The maximum staged water elevation causes the maximum diverted flow of approximately 19,000 cfs. The staged water elevation is slightly lower for the 0.2-percent chance flood event due to fewer challenges in reducing downstream stage impacts. For floods larger than the 0.2-percent chance event that would cause higher staging elevations, flow will be allowed to exit the staging area to the west over a weir at elevation 922.9 along Cass County Highway 17. This means that water will diverted into the Sheyenne River basin for extremely large flood events. Except for the weir at elevation 922.9 along Cass County Highway 17, the staged water, both upstream of the diversion and in Storage Area 1, will be contained by levees with 5 feet of freeboard above the 922.9 elevation. Attachment 5, Appendix F indicates the levees have a top elevation of 927.0, but these levees will actually have to be at elevation 927.9. Cost contingencies have been set to cover the additional cost of this additional levee height.

For the case of high flow conditions on the Sheyenne River while peak flooding is occurring on the Red and Wild Rice Rivers, the capacity of the proposed diversion channel is such that flooding along the Sheyenne River will be no worse than it is under existing conditions. Breakout flows from the Sheyenne River that cross Cass County Highway 17 and enter the Wild Rice River basin under existing conditions will be directed to the diversion channel via a new ditch. The exact location and design and location of this ditch have yet to be determined, but it is known that the ditch can be designed to function without inducing additional flooding. Other smaller ditches may also be required to prevent drainage issues along the proposed tie-back levees.

Summary stage and flow information for the LPP is provided as Tables B-1 and B-2 (taken from Attachment 5). It is important to note that the degree of staging needed to reduce downstream stage impacts for the historic event simulations is in-line with the required staging identified for the synthetic events. This supports the timing used for the synthetic event simulations and the final staging elevations for the synthetic flood events.

The operational scheme developed for the primary flood control structures at this feasibility level has been based on synthetic and historical floods for peak flows on the

Red River of the North with coincidental events on its tributaries as well as on synthetic floods for peak flows on the ND tributaries (directly affected by the LPP diversion channel) with coincidental events on the Red River of the North. These events provide a sample of flood events with a variety of hydrograph shapes and peak flows. However, it is recognized that a more complete evaluation (possibly including stochastic modeling) would be needed to develop the detailed operational plan of the flood control structures during final design. Furthermore, it is important to acknowledge that, following the example of the Manitoba Floodway and other similar large flood control projects, the operational plan developed during final design will be subject to further refinements and modifications after the project has been in place during actual flood events.

The project goal is to not cause any additional flooding along the Sheyenne River, Maple River, Lower Rush River, Rush River, and local drains adjacent to the LPP diversion. For many areas flooding will be reduced since the design water surface profile in the LPP diversion will be lower than it is in the existing Horace to West Fargo and West Fargo diversions, but to preserve existing floodplains the inlets to the diversion will not be designed to reduce flooding for the 1-percent chance and less frequent peak flows from the Sheyenne/Drain 14/Maple/Rush and local drainage areas (for these events the goal is to maintain the existing condition). The design presented does not completely meet the goals stated above. The current design does have areas where flooding is worse than the existing condition. Resolving the issue of increased flood stages will be relatively easy in some areas and more of a challenge in others. Cost contingencies have been raised to address these issues.

A more complete summary of the LPP is available in Attachment 5, General Report, Section 3.3. The reader is referred to Attachment 5, Appendices C and F to understand the details of the LPP hydraulic study.

B.2.1.4. Federally Comparable Plan (FCP)

The FCP diversion alternative for the Phase 4 feasibility study is the same as the one presented in the Phase 3 report. The main features consist of a control structure on the Red River of the North, the diversion channel, and the outlet structure for the diversion channel. The FCP diversion channel starts approximately one mile north of the confluence of the Red River of the North and Wild Rice River, extends north around the Cities of Moorhead and Dilworth and ultimately re-enters the Red River of the North near its confluence with the Sheyenne River. The alignment is approximately 25 miles long. In addition to the main diversion channel, this alignment requires additional channels upstream of the Red River control structure to prevent stage increases upstream of the project along the Red River of the North and Wild Rice River. A supplementary extension channel parallels the Red River of the North upstream of the entrance to the diversion channel to allow for additional capacity to offset blockage of the breakouts to Cass County Drains 27 and 53. This secondary FCP extension channel is approximately 3 miles long and has a 50 foot bottom width. A second, shorter channel, the Wild Rice Breakout Channel, was added near the intersection of I-29 and Cass County Highway 16. This channel, which is less than one mile long and crosses under I-29, will convey water across I-29 that would have naturally broken out to Cass County Drain 27 and has a 50 foot bottom width. Additionally, the FCP includes 20 roadway bridges and 4 railroad bridges that cross the diversion channel.

Similar to the LPP, Phase 4 includes the modeling of Existing Conditions and With-Project (FCP) for the historic 1997, 2006, 2009, and 2010 spring floods to determine the downstream impacts. These impacts are related to the loss of floodplain storage and changes to timing as a result of the flows conveyed through the diversion channel. For the FCP downstream impact analysis, the emergency protection measures that were in place during these historic event calibrations/verifications were not included. The FCP diversion channel from the Phase 3 design was incorporated into the Phase 4 HEC-RAS unsteady flow model. The With-Project water surface profiles were then compared to the Existing Conditions water surface profile to quantify the project impacts.

Phase 4 also includes the modeling of Existing Conditions and With-Project (FCP) for four synthetic events (0.2-, 1-, 2-, and 10-percent chance design floods) to determine the downstream impacts. These impacts are related to the loss of floodplain storage and changes to timing as a result of the diversion channel. The FCP diversion channel from the Phase 3 design was incorporated into the Phase 4 HEC-RAS unsteady flow model. The With-Project water surface profiles were then compared to the Existing Conditions water surface profile to quantify the project impacts.

Summary stage and flow information for the FCP is provided as Tables B-3 and B-4 (taken from Attachment 5). There is no upstream staging and the downstream impacts gradually attenuate downstream.

A more complete summary of the LPP is available in Attachment 5, General Report, Section 3.4. The reader is referred to Attachment 5, Appendices C and F to understand the details of the FCP hydraulic study.

B.2.2. ADDITIONAL STUDY INFORMATION

Ice analyses of existing conditions and the diversion alternatives are being performed by the Ice Engineering Group at the U.S. Army Cold Regions Research and Engineering Laboratory (CRREL). The ice analyses consist of three phases: 1) A review and analysis of historical ice and hydro-met data combined with an ice monitoring program, 2) An assessment of impacts to the ice regime in the vicinity of FM as a result of the diversion project, and 3) Analyses of the ice interaction with the RNN diversion and tributary crossing structures. These three phases involve reviewing historic ice data including ice thickness measurements and photos, simulating ice-affected water surface profiles with HEC-RAS, and using the state-of-the-art two-dimensional ice-hydraulic model DynaRICE to assess overall project performance with the expected ice conditions. Preliminary information from the ice study is included in Attachment 5, Appendix F, Exhibit J and a draft ice study report is provided as Attachment 1 to this appendix. The preliminary information has been used in updating costs associated with ice management (see Appendix L for cost information).

An assessment of project impacts to the Richland County drains is provided as Attachment 2 to this appendix.

Geomorphic studies are being conducted but these efforts have been delayed due to a longer-than-expected testing schedule, early snow conditions, and high water and in the Fargo-Moorhead area. Preliminary geomorphic information is included as Attachment 5, Appendix F, Exhibit I. Since that that Exhibit I was prepared additional sediment transport data has been collected during the spring 2011 high flows by the USGS and made available in a preliminary nature. The sediment data collected in 2011 was compared to the data collected during the spring 2010 high flow period. A comparison of the 2010 to 2011 sediment transport data and the conclusion that the additional data does support the conclusions of the Feasibility Study on the potential impact of the proposed project on sediment transport and geomorphology in the studied rivers. A memorandum discussing the 2011 sediment transport data is provided as Attachment 3 to this appendix.

Questions regarding whether the LPP would increase the duration of flooding downstream of the proposed diversion were raised during the public comment review period. A discussion of existing conditions vs. LPP hydrographs at three locations downstream of the proposed diversion (Hendrum, Halstad, and Grand Forks) is provided as Attachment 4 to this appendix.

The wind setup and wave height analysis for the LPP staging area is provided as Attachment 5. A minimum of 3 feet of freeboard is recommended.

B.3.0 QUALITY ASSURANCE / QUALITY CONTROL

B.3.1. CONTRACTOR QA/QC

The contractor conducted a series of internal QA/QC reviews that involved both reviewers on the project design team and reviewers independent of the design team. The comments and responses developed during the review of the existing conditions models are documented in Attachment 5, Appendix B (Section B6.0 and Exhibit H). The comments and responses for the with-project models are documented in Attachment 5, Appendix C (Section 2.15 and Exhibit 5). The contractor QA/QC reviews were extensive and must be read to fully understand the complete QA/QC effort.

B.3.2. DISTRICT QUALITY CONTROL (DQC) AND AGENCY TECHNICAL REVIEW (ATR)

The draft hydrology report and draft unsteady HEC-RAS models were sent out for District Quality Control (DQC) and Agency Technical Review (ATR) in late August 2010. The reviews were conducted by independent Corps' reviewers. Vicksburg District (MVK) performed the DQC and Omaha District (NWO) performed the ATR. The comments and responses are provided in the following table.

FARGO MOORHEAD METRO FEASIBILITY STUDY

UNSTEADY FLOW MODELING

DISTRICT QUALITY CONTROL (DQC) AND AGENCY TECHNICAL REVIEW (ATR) COMMENTS

Reviewer	District	Review Type	#	Date of Comment	Comments	Initial Response	Final Response
Malcolm Dove	MVK	DQC	1	31-Aug-2010	The downstream water level increases that are related to storage area routing can be refined with a sensitivity analysis and possibly with a modified diversion routing approach. RAS model testing is under way, and Mr. Malcolm Dove at CEMVK-ED-H is POC for this information.	Storage connection and lateral structure weir coefficients were adjusted as part of a Sensitivity Analysis. The impacts on the model results are small as outlined in the attached sensitivity analysis results.	The sensitivity analyses are documented in Attachment 5, Appendix B (Section B3.5.3, Exhibit F, and Exhibit G).
Mike Alexander	MVK	DQC	2	31-Aug-2010	A LIDAR data set is available for the study drainage basin that may allow a quick 2-dimensional flow routing model that uses the overbank bathymetry instead of a storage area concept. Two-dimensional modeling development, time and cost, and solution quality assessments are also being prepared at MVK. POC for this effort is Mr. Mike Alexander at CEMVK-ED-HH.	No Response.	Due to the importance of detailed road and culvert information and considering the extent of the model, it was decided that a 2-D model is not practical at this time.
Aaron Buesing	MVP	DQC	3	31-Aug-2010	Consider going through all HTab parameter settings. By making sure the curves don't go too much higher than the highest expected energy elevation and using a higher number of points (up to 100), the increment can be reduced. This might improve storage accounting. Reducing the increment of hydraulic property curves could help model stability, if that's been an issue, but it also could hurt.	H-tab - cross sections, bridge, culvert, inline structures, Storage Area Connections have been adjusted. Elevations were set based on 0.2-percent chance event +5 feet. Increments were set at the maximum (100).	No further reponse.
Aaron Buesing	MVP	DQC	4	31-Aug-2010	SA Conn:BufSC26: A value of 10000, which I think is meant to be the Max Flow (Recommended), has been entered as the Tail water maximum elevation (optional).	This has been revised.	No further reponse.
Aaron Buesing	MVP	DQC	5	31-Aug-2010	Pan through all of the Hydraulic Property Tables plots (cross-sections, internal boundaries, storage area connections) to see if anything else looks strange. That's how the SA Conn:BufSC26 issue was discovered. Do #3 above first so you limit these plots to the real range of interest - right now you'll find some strange looking plots, but the elevations where the curves are strange are well above the 500-yr maximum water surface elevation.	Response to Comment # 3 was completed, and additional QC was performed.	No further reponse.
Aaron Buesing	MVP	DQC	6	31-Aug-2010	The flooded outlines suggest that additional sections between Georgetown and Shelly should be added as storage areas to the HEC-RAS model. It also may make sense to convert the outer portion of a few cross-sections in this area to storage areas.	The 1-percent chance event ND Diversion and 0.2-percent chance event Existing Conditions flood outlines were mapped to identify the floodplain extent. Storage areas were added, where necessary, to ensure the model encompasses the full floodplain.	No further reponse.
Aaron Buesing	MVP	DQC	7	31-Aug-2010	The possibility of water being moved too fast through connected storage areas was discussed internally and Mark Jensen (HEC) was also consulted. Water moving too fast certainly could be an issue for inundated storage areas where flow can go out as fast as it can come in (level pool routing would govern, which would move water too fast). However, the timing of water going into and out of storage should be good. If the modeler does a good job deciding what should be modeled in the cross-section data and what should be modeled as a storage area, the model will probably will do a good job on timing, but obviously that isn't always easy to do - sensitivity analyses are good. Mark Jensen suggested lowering storage area connection weir coefficients as a means to slow the movement of the flood wave through storage areas, thereby performing a sensitivity analysis. HEC is working on implementing 2d diffusion wave routing for storage areas in HEC-RAS, but this is still a few months away. HEC will be asked to see if there's any possibility of making this model a test case for the 2d diffusion wave routing effort. As seen from MVK's comments, storage routing is an issue being investigated by both MVP and MVK.	Storage Area Connection weir coefficients and Lateral Structure weir coefficients have been analyzed for Existing Conditions and ND Diversion for the 1-percent chance flood with coefficients = 1.5, 2.0, 2.6 and 3.0 depending on the location. The impacts on the model results were minimal as outlined in the attached sensitivity analysis.	The sensitivity analyses are documented in Attachment 5, Appendix B (Section B3.5.3, Exhibit F, and Exhibit G). The HEC-RAS 2-D diffusive wave functionality is not far enough along to be used for this study.
Aaron Buesing	MVP	DQC	8	2-Sep-2010	There are sections along the ND diversion, outside the area of protection, that are modeled as storage areas for existing conditions but are not modeled for the ND diversion alternative.	Storage areas were added west of the ND Diversion alignment. This allows hydrology to be inserted in the same location for existing conditions and with project conditions. A weir will need to be developed to convey water into the diversion and it is anticipated that existing conditions flood elevations will be maintained.	No further reponse.
Aaron Buesing	MVP	DQC	9	2-Sep-2010	There are sections modeled as storage areas along the ND diversion, outside the area of protection, that are not connected to the diversion via a lateral weirs. Allowing water into these section from the diversion may increase available storage and possibly slow down the progression of the hydrograph.	Original assumptions didn't include any water to breakout of the diversion once it was in. As noted in the response to comment 8, the need for additional storage areas for both existing and with-project conditions has been reviewed and additional storage areas have been added to the model for both conditions. Storage areas have been added along the diversion near the Rush River as well as between the Sheyenne River and Maple River.	No further reponse.
Malcolm Dove	MVK	DQC	10	7-Sep-2010	Model, General: If the existing model geometry had been structured such that the flowlines from the existing model could be plotted against the ND-Diversion and the MN-Diversion output, it would have been more obvious where the effects of increased flows as well as hydrograph timing were occurring.	Dummy reaches would need to be incorporated into the existing conditions model as well as each of the diversion models to allow common reach names. This would likely cause additional model instability. It is not anticipated that this would be done. It would however, be advantageous to view all of profiles simultaneously.	No further reponse.

FARGO MOORHEAD METRO FEASIBILITY STUDY

UNSTEADY FLOW MODELING

DISTRICT QUALITY CONTROL (DQC) AND AGENCY TECHNICAL REVIEW (ATR) COMMENTS

Reviewer	District	Review Type	#	Date of Comment	Comments	Initial Response	Final Response
Malcolm Dove	MVK	DQC	11	7-Sep-2010	Model, Diversion Channels: It is noted that the ND and MN Diversion channels have n-values of 0.030 which is relatively smooth for an earth lined channel. Normally the roughness coefficient for a newly constructed channel increases with age and a plan should be designed on what will be expected in the future. It is good to look at other aged cut channels that have deteriorated and use n-values similar to those channels. The Sheyenne Diversion (Existing condition) has assigned n-values of 0.040, which would probably be in the range of the ND-Diversion channel or the MN-Diversion channel after it has aged.	A sensitivity analysis was performed using n-values of 0.035 and 0.040. The impact on the model results was minimal as noted in the attached sensitivity analysis. Based on a review of similar projects, such as the Winnipeg Diversion, an n-value of 0.030 seems appropriate.	The sensitivity analyses are documented in Attachment 5, Appendix B, Exhibit F.
Malcolm Dove	MVK	DQC	12	7-Sep-2010	Hydrology, General: Timing of the flood hydrographs in a plan of this type can result in misleading stage increases. Sensitivity analysis is important in determining the final answer concerning how flood flow attenuates within a river reach.	Sensitivity of hydrograph timing such as the Elm River was completed during calibration and model development. It is an issue and we saw localized diversion impact differences. More information is needed on the timing and pattern of the significantly large drainage area of the Elm River. Gaging data from the USGS during the spring 2010 flood event has provided some guidance on timing. Additional sensitivity of tributary timing/contributions from the Sheyenne/Maple/Buffalo River watersheds was performed as part of a sensitivity analysis. The impact on the model results was small as noted in the attached sensitivity analysis results	The sensitivity analyses are documented in Attachment 5, Appendix B, Exhibit F.
Malcolm Dove	MVK	DQC	13	7-Sep-2010	Hydrology, Maple River: It is noted that the flow file for Maple River at the upper end (Sta 36989) uses a file from the MAPLE RIVER/MOUTH/FLOW /BALANCED HYDROGRAPHS.dss file for the 100year frequency plan. It is not clear if this flow was computed for the total stream above the mouth or above Sta 36989. This should be clarified.	There is a minor drainage area between the mouth and upper end of the model. It was assumed it would be negligible. This would be better defined with detailed hydrology review of Sheyenne/Maple Rivers and the expansion of the model geometry along the Lower Sheyenne and Maple Rivers which is ongoing (see response to Comment 8). See also the response to Comment 12.	The HEC-RAS was updated based on the revised hydrologic information found in Appendix A, which includes local flow for the reach in question.
Malcolm Dove	MVK	DQC	14	7-Sep-2010	Hydrology, Maple River: Under the existing condition 100-year plan, there is an additional flow that was entered at Sta. 7823 that comes from the flow hydrograph of the Sheyenne River above the mouth of the Maple River. It is noted that this hydrograph (Existing condition) is reduced to 0.525 of the Sheyenne River and produces a peak inflow of 7500cfs. Then in the ND-Div plan, the Sheyenne River hydrograph is increased to 1.05 and entered uniform from Sta. (33749 to 29611) and produces a peak inflow of 16080 cfs on 1 April. This results in the following changes (The peak time of the hydrograph is changed from 3April-1200hrs at Sta. 36989 to 2April-at the junction of the ND Diversion). This should be reconsidered due to the increase in inflow and the fact that the peak stage in the reach is over a foot higher for the ND Diversion plan than the Existing Condition plan. This would result in more flooding along the Maple River.	Existing Conditions Upper end of Maple River RS 36989 = 12,500 cfs. This is the same for both plans. Upper end of Sheyenne River - RS 232811 = 4,600 cfs. This is the same for both plans. The Sheyenne River Channel only allows 4,600 cfs to be contained in the channel upstream of Horace (near Sheyenne and Sheyenne Diversion). Everything else breaks out of channel. The breakout flows are approximately 15,000 cfs and they re-enter the system near the Maple/Sheyenne confluence. This comes down to a level of detail in the model. Upstream of the Maple/Sheyenne confluence everything floods out on large events. Instead of placing the entire hydrograph on the Maple River, we placed half of it along the Maple and half of it along the Sheyenne at nearly the same location. When the ND Diversion becomes active, it wouldn't seem appropriate to place the inflow on both the Maple and the Sheyenne. Therefore the entire hydrograph was placed on the Maple River. This creates issues with containing the large hydrograph. For the purposes of the model, the hydrograph was to be inserted, a set amount sent downstream on the Maple, and the remaining hydrograph was to be sent down the diversion. Additional storage areas could be placed west of the diversion, but if this attenuates the peak on existing conditions, we would have to increase the inflow hydrographs to maintain our Red River calibration hydrograph downstream of the Buffalo River. Breakout Flows - Existing Conditions Maple River RS 7823 - Multiplier Ratio = 0.525 Sheyenne River RS 108266 - Multiplier Ratio = 0.525	The entire Sheyenne/Maple drainage system from their confluence upstream to where they enter Glacial Lake Agassiz has been added to the unsteady HEC-RAS model. Therefore all breakout flows out of the Sheyenne and Maple Rivers are routed downstream through storage areas and intervening ditches. Also the coincident flow hydrology has been updated. The hydrology is documented in Appendix A. The updated HEC-RAS model and results are documented in Attachment 5, Appendices B, C, and F.

FARGO MOORHEAD METRO FEASIBILITY STUDY
UNSTEADY FLOW MODELING
DISTRICT QUALITY CONTROL (DQC) AND AGENCY TECHNICAL REVIEW (ATR) COMMENTS

Reviewer	District	Review Type	#	Date of Comment	Comments	Initial Response	Final Response
Malcolm Dove	MVK	DQC	15	8-Sep-2010	I have read and understand the response to comment #14, however after our conference call this morning (Aaron Buesing, Mike Alexander and myself) I feel I need to address Greg's comments in more detail. Normally when flow (15000cfs) as stated breaks out of banks and goes into storage it rarely comes back into the stream as a peak flow of 15000cfs. It has to come back overbank when the channel water elevation has receded to allow it or it re-enters downstream through a side channel when the difference in water surface in the main channel and tributary allow it. It is noted that under the existing condition scenario there is an approximate 50-50 split in the 15000 cfs between Maple River and Sheyenne River, however, under the ND Diversion plan the total peak flow of 15000cfs is entered into the Maple River between station 33749 to 29811. The reality of this much flow entering back into the river when the water surface elevation is already above top bank and spread out over an approximate mile wide cross-section needs to be re-analyzed. Forcing the flow into the channel results in a total flow of about 29000cfs being pushed into the diversion in a relatively narrow opening. It is understood that this project is on a short schedule, however the study should keep this area in mind on how this overbank flow will be handled in the future detailed studies.	Agreed. The hydrology is being refined and more geometry is being developed (PIE Model) to route the hydrographs and breakout flows through the Sheyenne and Maple reaches. As discussed in previous conference calls, if the flows are attenuated through storage/routing, which I also believe they should be, then additional water needs to be added for all scenarios to re-build the hydrograph to match on the Red River downstream of the Buffalo River. Sensitivity analysis was performed as outlined in the response to comment 12 and additional model geometry is being added along the lower Sheyenne and Maple Rivers as outlined in the response to comment 8. Additional refinement will need to be done as part of final design.	The entire Sheyenne/Maple drainage system from their confluence upstream to where they enter Glacial Lake Agassiz has been added to the unsteady HEC-RAS model. Therefore all breakout flows out of the Sheyenne and Maple Rivers are routed downstream through storage areas and intervening ditches. Also the coincident flow hydrology has been updated. The hydrology is documented in Appendix A. The updated HEC-RAS model and results are documented in Attachment 5, Appendices B, C, and F.
Roger Kay	NWO	ATR	16	8-Sep-2010	“2009 Model” Folder RAS: I am curious as to why the model (2009 FP) was calibrated to discharge, rather than stage at measured gage locations (at least it seems to be, as results are presented showing the match in discharge, rather than stage)? The measurements of stage are nearly always more accurate than measurements and estimations of discharge, which is why the HEC-RAS Users Manual suggests calibrating an unsteady model to stage, rather than discharge. A quick comparison of actual discharge to measured discharge at the downstream model boundary seems to show too high of a peak discharge compared to measured (~10,000 cfs), although total volume matches within ~3%. This may indicate that the model is too smooth, leading to flows reaching downstream portions of the model too soon, which may lead to increased stages for the design conditions (although this may be at least partially tied to the use of a rating curve for the downstream boundary condition – see comment below).	The model was calibrated to both stage and discharge. The 2009 discharge at Halstad was 67,500 cfs. The discharge at Thompson was 61,100 cfs. This includes adding the Marsh River, Goose River, Sandhill River, and local ungagged drainage areas. We couldn’t get the model to reflect this discharge reduction. When the model was extended downstream to Drayton, ND, the 2009 calibration includes an "S" curve to reflect real discharge reduction upstream of Thompson. The Thompson gage hydrograph matches Grand Forks with reasonable local inflows.	The calibration effort involved stage and discharge calibration. The calibration effort is described in Attachment 5, Appendix B (Section B5.0 and Exhibits A - D).
Roger Kay	NWO	ATR	17	8-Sep-2010	“2009 Model” Folder RAS: Using a rating curve for the downstream boundary condition on an unsteady HEC-RAS model is probably more appropriate if there is a control structure at the downstream boundary (such as a weir or gated structure). Using the rating curve prevents the “looped” rating curve from being duplicated, and this effect propagates some distance upstream. Please be certain to ascertain how far up from the downstream boundary that results should not be used to determine differences between with- and with-out project conditions profiles. A better solution would be to extend the model far enough downstream of the area of interest, such that the boundary condition assumed has no impact on computed profiles for the area of interest. This may end up an iterative process.	Subsequent to the July 30, 2010 submittal, the model has been extended farther downstream to near Drayton, ND. The impact on the model results at Thompson as a result of this extension was small.	Stage impacts are essentially the same for at least the downstream-most 67 river miles of the HEC-RAS model (stage impacts vary from 0.04 to 0.08 feet along that reach). Based on this, we feel it is safe to assume that the rating curve boundary condition is affecting stage impact results at the boundary by no more than a few hundreths of a foot. This effect is on the order of the HEC-RAS calculation tolerances and is therefore considered negligible. The absolute stage values at the boundary may be off by a greater degree, but the concern are stage impacts, not absolute values.
Roger Kay	NWO	ATR	18	8-Sep-2010	“2009 Model” Folder RAS: Need to keep bank stations at bank stations – e.g. x-sec 1939454 has “n” value of 0.04 from Sta. 19332.06 to 19569.7, but bank stations from Sta. 19306.08 to 19637.6; the “n”-value between Sta. 19306.08 and 19332.06 and between 19569.7 and 19637.6 is 0.1, which seems to be too high for this channel. A cursory review of this location in Google Earth shows a channel that is about 230 feet wide, bank to bank, not 330 feet. A previous comment addressed cross-sections where the bank stations were being placed incorrectly, as it does impact the calculations in RAS. A review of other cross-sections shows this to be an issue at a large number of cross-sections. Please ensure that all cross-sections use an appropriate station for the bank stations and that the “n”-value for the channel itself does not contain areas of high “n”-values.	During calibration and after the first round of comments, the bank stations were evaluated and the n values were verified for the model reach to Halstad, MN. Model extension from Halstad to Thompson and from Thompson to Drayton utilized existing USACE model geometry that was incorporated with little revision to the geometry. See also the response to comment 19. Additional review and detail will be added to the model reach downstream from Halstad as part of the future modeling work that is planned.	Bank stations and Manning's n values were re-evaluated and adjusted where deemed appropriate. The revised models are documented in Attachment 5, Appendices B, C, and F. The study contractors have completed an extensive independent QA/QC effort that is documented in Attachment 5, Appendix C (Sections C2.15 and Exhibit 5).

FARGO MOORHEAD METRO FEASIBILITY STUDY

UNSTEADY FLOW MODELING

DISTRICT QUALITY CONTROL (DQC) AND AGENCY TECHNICAL REVIEW (ATR) COMMENTS

Reviewer	District	Review Type	#	Date of Comment	Comments	Initial Response	Final Response
Roger Kay	NWO	ATR	19	8-Sep-2010	“2009 Model” Folder RAS: Remember that a cross-section is supposed to be representative of the floodplain and channel from a point midway between the upstream and downstream cross-section. Again, going to x-sec 1939454 as an example – the area adjacent to the channel has an “n”-value of 0.1, which appears to be representing an area of trees. These areas are 73 feet wide at the left bank and 83 feet wide at the right bank. These distances seem reasonable for the actual location of the cross-section. However, this seems to be about the minimum width for trees adjacent to the channel within the reach this cross-section should be representing; the maximum width of tree coverage on either bank is several times greater. The same appears true for a majority of the cross-sections, at least in the lower portion of the model, as they seem to be located at areas where trees do not extend very far from the channel. Whether this cross-section arrangement was by design or chance, it will have an impact on the model and how it replicates measured water surfaces, but more importantly, may be making the model too smooth, and thereby not attenuating flows quite as much as they should. I would suggest that you carefully determine an average width of tree area next to the	The original model development was downstream to Halstad. Model geometry, calibration, and synthetic event calibration for 10, 50, 100, and 500 year events from Halstad to Thompson occurred in a very short time frame less than 2 weeks. It was understood that the downstream reaches would have less detail and less review prior to using. The reaches from Thompson to Drayton (not included in this review) were obtained from the USACE from previous studies. Additional review and detail will be added to the model reach downstream from Halstad as part of the future modeling work that is planned.	Bank stations and Manning's n values were re-evaluated and adjusted where deemed appropriate. The revised models are documented in Attachment 5, Appendices B, C, and F. The study contractors have completed an extensive independent QA/QC effort that is documented in Attachment 5, Appendix C (Sections C2.15 and Exhibit 5).
Roger Kay	NWO	ATR	20	8-Sep-2010	“2009 Model” Folder RAS: In the 2009 calibration model, there are a number of storage areas that have water surface elevations rising/falling that do not appear to be connected to the change in stage on riverine reaches; rather, they appear to be due to improperly set initial conditions. It doesn’t appear that any of these would have a significant impact on the final solution, but all initial conditions should be double-checked.	Some rivers or storage areas needed higher flows or starting WSEs to provide stable initial starting conditions. It is anticipated that this goes away prior to the event hydrograph and does not create any impact on the results. They will be revised if time permits.	Starting water surface elevations for the storage areas were reviewed and set to minimize unrealistic water surface elevations at the beginning of the simulation. For model stability some initial water surface elevations are still set somewhat high, but this does not have a significant impact of the model results.
Roger Kay	NWO	ATR	21	8-Sep-2010	“2009 Model” Folder RAS: Some cross-sections appear to be incorrectly described with “n”-values. For instance, from x-sec 2129130 to 2129181, there is only one or two “n”-values for the entire section, with the channel being far rougher than the x-secs upstream or downstream.	This is an error and has been changed.	No further reponse.
Roger Kay	NWO	ATR	22	8-Sep-2010	“2009 Model” Folder RAS: The elevations of some bank stations seem excessively high compared to nearby cross-sections; e.g. x-sec 1986219 has bank elevations more than 20 feet higher than nearby cross-sections. Please verify that the banks are being defined in the proper location at each cross-section.	All cross sections in the primary model (downstream to Halstad) were reviewed at one time. Cross Section 1986219 has been changed as it was missed originally.	Bank stations and Manning's n values were re-evaluated and adjusted where deemed appropriate. The revised models are documented in Attachment 5, Appendices B, C, and F. The study contractors have completed an extensive independent QA/QC effort that is documented in Attachment 5, Appendix C (Sections C2.15 and Exhibit 5).
Roger Kay	NWO	ATR	23	8-Sep-2010	“2009 Model” Folder RAS: The MN Diversion and ND Diversion plans were delivered without up-to-date results. After correcting some missing DSS links in the unsteady flow files, neither plan ran to completion – the ND Diversion plan went unstable after 4½ hours simulation time, while the MN Diversion plan ran to the end of simulation, but hung up on the post-processing. It is unfortunate that the models supplied do not run to completion, as that lowers my confidence in their use in modeling design flows. Please ensure that the most recent model results are sent for review, and that all plans run without bombing out.	We ran into this during model calibration and runs. However, all of the models did have a successful run through completion. HEC was contacted about the post processing error and stated that there was a bug in the programming and the model went into an unrecoverable loop. They also stated that it wouldn't be fixed until the next version which is a long ways out.	No further reponse.

FARGO MOORHEAD METRO FEASIBILITY STUDY							
UNSTEADY FLOW MODELING							
DISTRICT QUALITY CONTROL (DQC) AND AGENCY TECHNICAL REVIEW (ATR) COMMENTS							
Reviewer	District	Review Type	#	Date of Comment	Comments	Initial Response	Final Response
Roger Kay	NWO	ATR	24	8-Sep-2010	"10yr Model" folder (and other "X-yr Model" folders): Be sure to check for model instabilities – view animated profiles for various reaches. For instance, on the "MN DIV SHORT DNS" reach, the water surface inexplicably rises in the lower portion of the model at time step 23Mar2006 2400 hrs, then quickly lowers. Further upstream, in the lower portion of the "MN DIV SHORT EXT" reach, there is a large drop in water surface for much of the duration of the simulation, and a large drop in water surface at near maximum profiles. Also be sure to use the general profile plots and animate them for various variables to troubleshoot problem areas – for instance, on the same reach as above, the top widths show rapid expansion and contraction, which indicates that some encroachments may be necessary along the diversion reach. There are a great number of areas that I would suggest investigating for making model changes, but given the quick turn-around on this review, I was not able to "play around" with the model to see what those changes may do. Please verify that results make sense throughout the course of the simulation, not just the maximum water surface, as this can have an impact on movement of water through the system, and may lead to under- or over-stating the downstream impacts of the project.	This is a result of the short timeline of the project. The diversion geometry was created with the steady state model. Several iterations of model geometries have been received as the design parameters and hydrology have changed. The turn around time on inserting the geometry and re-running the unsteady flows (10, 50, 100, 500 year events) was a week to two weeks on average. Results were required in such a short time, we had enough time to get the model to run and verify that the discharges seemed reasonable. "the water surface inexplicably rises in the lower portion of the model" This issue presents itself and then goes away one interval later. Profiles were reviewed and used for diagnosis. This is very isolated and does not seem to be an issue. Further upstream, in the lower portion of the "MN DIV SHORT EXT" reach, there is a large drop in water surface for much of the duration of the simulation, and a large drop in water surface at near maximum profiles. This Channel Extension comment is not applicable. The channel extension was in the geometry from the beginning of the project, but as shown in the model results discharges 190 cfs. It was identified that the channel extension was not needed anymore for capacity and with the given timeline was easier to leave it in with minimal flows to provide model stability than completely remove it. Final adjustments can be made to the model with final design once a single plan is selected.	The study contractors have completed an extensive independent QA/QC effort that is documented in Attachment 5, Appendix C (Sections C2.15 and Exhibit 5).
Roger Kay	NWO	ATR	25	8-Sep-2010	"10yr Model" folder (and other "X-yr Model" folders): Be sure to check inflow and outflow volumes at all boundary conditions to be sure the same volume is passing out of the model as coming into the model. A quick check of the downstream boundary condition shows 2.4% more volume of flow for the MN diversion, compared to the existing conditions, while the ND diversion only has 0.2% more flow volume (10-yr). For the 100-yr condition, the MN diversion shows 1.3% more flow volume and the ND diversion 0.8% more flow volume than the existing condition. Since balanced hydrographs are being used, I would have expected the flow volumes to be closer, especially for the two diversion alternatives, unless the simulation is leaving considerably more flow in storage areas for the various plans (although a spot check of various storage areas did not reveal any with significant storage at the end of simulation) or there are differing amounts of storage at simulation start.	The volumes at the downstream end have continuously been checked throughout modeling. One thing to consider is that the diversion reaches need water in them in order to remain stable, therefore just placing the diversion geometries in the model require additional volume to be added to the system. With that being said 1-2% additional volume has been assumed to be acceptable. We did also check to verify if any water is remaining in the system.	No further reponse.
Roger Kay	NWO	ATR	26	8-Sep-2010	"10yr Model" folder (and other "X-yr Model" folders): There is no channel ice for any of the design conditions. Please be sure to include ice impacts on assessing with- and with-out project conditions (as discussed previously).	This will be addressed during final design.	Ice studies have not been completed, but a preliminary ice assessment is provided at Attachment 5, Appendix F, Exhibit J. Retaining ice upstream of the structures appears feasible due to the low velocities. The impact of having less flow through the area of protection has yet to be determined.
Roger Kay	NWO	ATR	27	8-Sep-2010	"10yr Model" folder (and other "X-yr Model" folders): Be sure to double-check initial conditions for all storage areas – a spot-check of various storage areas shows multiple storage areas that experience a foot or more of change in water surface in the first 12 hours of simulation. This may have an impact on total volume passed out of the model and water surfaces in the first few days of simulation. As an example, refer to X-Sec 82415 on Wild Rice ND and SA WRSA 300 – note how flows in the river do not correspond with the input hydrograph for this reach for several days (noted in 10- and 100-yr models, did not check other models for this, but likely present as well).	This can be reviewed further. Again, it comes down to a level of detail and time constraint to review all of the initial conditions that don't present issues during the event hydrograph. It does not seem to impact the results and will be addressed with final design.	Starting water surface elevations for the storage areas were reviewed and set to minimize unrealistic water surface elevations at the beginning of the simulation. For model stability some initial water surface elevations are still set somewhat high, but this does not have a significant impact of the model results.
Roger Kay	NWO	ATR	28	8-Sep-2010	"10yr Model" folder (and other "X-yr Model" folders): What differences in the 3 geometries (Existing, MN Diversion, ND Diversion) exist between the 10-, 50-, 100-, and 500-yr models? I did a spot-check of various cross-sections, storage areas and structures, and nothing jumped out at me, but it was not possible to compare everything. If there are no differences in geometry, then the different flow events should be in one project to make comparisons easier between flow events. If there are differences, please list them, and why they are necessary.	Very minimal changes. The 50-year and 100-year are identical. The 10-year required additional tweaking where tributaries are connected to "ease" the transitions, especially on the diversions where the diversion channel is well above the Red River channel. The 500-year geometry required storage areas WRSA348 and WRSA349 to be removed and simplified. The geometries will be combined.	The final feasibility study models have identical geometries for the 10-, 50-, 100-, and 500-yr models.

FARGO MOORHEAD METRO FEASIBILITY STUDY
UNSTEADY FLOW MODELING
DISTRICT QUALITY CONTROL (DQC) AND AGENCY TECHNICAL REVIEW (ATR) COMMENTS

Reviewer	District	Review Type	#	Date of Comment	Comments	Initial Response	Final Response
Roger Kay	NWO	ATR	29	8-Sep-2010	“10yr Model” folder (and other “X-yr Model” folders): Model should be extended further downstream, given the magnitude of differences at the current downstream boundary, to verify that increases in water surface do not negatively impact existing flood protection projects that are currently certified against the 100-yr flood. Additionally, the downstream boundary condition should not use a rating curve, but rather a normal depth condition, and be sufficiently downstream of the area of concern so that profiles are not impacted by the assumption for downstream boundary condition.	Subsequent to the July 30, 2010 submittal, the model has been extended farther downstream to near Drayton, ND. The impact on the model results at Thompson as a result of this extension was small.	No further reponse.
Roger Kay	NWO	ATR	30	8-Sep-2010	General Comment: The model geometry is quite extensive. Due to the extensive nature of the model, it was not possible to review all aspects of the model in the time allotted. There may be substantial issues with the model not presented in these comments.	Agreed. Due to the extensive nature of the model and the given time constraints, it was and is not possible to review every detail of the model. The internal review during calibration and model development has identified issues and trends within the model that have been identified for significance and dealt with in determining how they may affect the downstream impacts. With the given time allowed for the project, it is not anticipated that the review comments will have significant impacts on the results.	The study contractors have completed an extensive independent QA/QC effort that is documented in Attachment 5, Appendix C (Sections C2.15 and Exhibit 5).
Roger Kay	NWO	ATR	31	8-Sep-2010	General Comment: For model review, it would be preferable to maintain directory structure, so that the reviewer can rerun various plans without having to reconnect various DSS files that are in different locations than originally run.	All dss files were available. The next submittal will include all .dss files combined into one.	The directory structure now allows for the model to be transferred and run without loss of DSS file connections.
Roger Kay	NWO	ATR	32	8-Sep-2010	General Comment: The invert profiles for the diversion plans are exceptionally flat – on the magnitude of ½- to 1-foot per mile. While this is not overly flat for a naturally occurring stream, it seems extremely flat for a constructed channel, at least as far as maintaining positive grade throughout the entire reach during construction. This could lead to maintenance issues, as low spots will likely retain local runoff and develop more “wetland”-type vegetation. Additionally, I see no mention of a sediment analysis to assess the long-term diversion channel morphology. While the geometries presented are not overly optimistic for constructed conditions (i.e. assumption seem reasonable for roughness, geometry, etc), I think long-term performance of the project needs a closer look, so as to evaluate project performance over the life of the project. This may show an increased need for O&M over that assumed, which may impact project BCR.	Will likely be evaluated with final design. Not a hydraulic issue	A report on sediment transport and geomorphology is included as Attachment 5, Appendix F, Exhibit I. Based on the study effort to date, significant sedimentation in the diversion channel is not expected. The final geomorphology study is expected to be complete in late 2011.
Roger Kay	NWO	ATR	33	8-Sep-2010	General Comment: Timing of tributary flows has the potential for a major impact on the difference between with- and with-out diversion project water surface profiles downstream of the diversion. How much work was put into assessing the sensitivity of the model to the tributary flow timing? It may be wise to assess project performance assuming a “worst-case” and “best-case” scenario for timing of the balanced hydrographs, based on the historic record, if this has not already been done.	The tributary hydrographs have been reviewed especially locations downstream of the diversion reaches (Elm River). The timing has shown to have an impact on downstream results. Additional hydrology support would be required for this task. Or a sensitivity could be applied by shifting local inflows around. One thing to keep in mind is... If the Elm River is shifted a day earlier or later, something else must be justified to be shifted to fill the remaining portion of the balanced hydrograph just downstream at Halstad. Additional sensitivity analysis was performed as outlined under the response to Comment 12 and the model response to the changes was small.	The sensitivity analyses are documented in Attachment 5, Appendix B, Exhibit F. The entire Sheyenne/Maple drainage system from their confluence upstream to where they enter Glacial Lake Agassiz has been added to the unsteady HEC-RAS model. Therefore all breakout flows out of the Sheyenne and Maple Rivers are routed downstream through storage areas and intervening ditches. Also the coincident flow hydrology has been updated. The hydrology is documented in Appendix A. The updated HEC-RAS model and results are documented in Attachment 5, Appendices B, C, and F.
Roger Kay	NWO	ATR	34	8-Sep-2010	General Comment: Some comments presented here (e.g. #18) are similar to comments made previously, but do not appear to have been completely addressed throughout the model. Also, there are several issues raised in comments above that lead me to think the model may be passing flows downstream a little too quickly, thereby overstating the with-project impacts on computed water surface. However, there are also some issues that may be causing the model to understate impacts – it is impossible to evaluate without making extensive (and careful) changes. Even though this project has been on a fast-track schedule, it is important to address modeling issues (and in a consistent manner), as they may have substantial impact on the final answer. It has been my experience that a good answer that is late is almost always preferable to a bad answer that is on time.	We agree with this comment. Additional sensitivity analysis is being performed, mainly, 1. Weir coefficients for storage connections and lateral structures. 2. Need for additional storage areas downstream from diversion outlet. 3. Need for storage areas/connections along the ND Diversion Channel. 4. Diversion Channel Roughness Coefficients. 5. Timing of hydrographs for tributaries including the Sheyenne/Maple/Buffalo/Elm River and local inflows. The results fo the sensitivity analysis that have been performed to date are attached. Overall, the sensitivity analysis shows the impact of these changes on the downstream impacts is small. The work related to the storage area/connectsions and flow distribution along the lower Sheyenne and Maple Rivers is ongoing.	The sensitivity analyses are documented in Attachment 5, Appendix B (Section B3.5.3, Exhibit F, and Exhibit G).

FARGO MOORHEAD METRO FEASIBILITY STUDY							
UNSTEADY FLOW MODELING							
DISTRICT QUALITY CONTROL (DQC) AND AGENCY TECHNICAL REVIEW (ATR) COMMENTS							
Reviewer	District	Review Type	#	Date of Comment	Comments	Initial Response	Final Response
Roger Kay	NWO	ATR	35	8-Sep-2010	General Comment: In spite of what some of my comments may suggest, I think overall the work done to date with the HEC-RAS modeling has been tremendous given the time constraints, as this is a highly complex model. I will be very interested in seeing what the final model shows for results.	-	No further reponse.

Table B-1

Summary HEC-RAS Unsteady Flow Model Results for Design Floods - Locally Preferred Plan

North Dakota Diversion (LPP) - 0.2% Chance Event (No Protection)							
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	804.12	168,364	804.23	171,002	0.11	2,638
Minimum Impact Location	1410241	812.15	152,872	812.19	156,165	0.04	3,294
Oslo Gage	1416287	813.88	152,851	813.93	156,084	0.05	3,232
Grand Forks Gage	1558518	836.36	146,225	836.58	149,112	0.22	2,887
Maximum Impact Location	1561353	838.53	102,444	838.80	102,054	0.27	-390
Thompson Gage	1667877	850.69	112,422	850.64	111,394	-0.05	-1,027
Halstad Gage	1981580	871.54	101,754	871.32	92,746	-0.22	-9,007
Fargo Gage (13th Ave S, 12th Ave S)	2388223	905.8 (43.06*)	61,717	902.77 (40.03*)	29,865	-3.03	-31,852
US Diversion**	2531315	915.94	28,577	922.44	27,846	6.50	-731
Hickson Gage**	2563754	919.69	35,636	922.54	32,491	2.85	-3,145
Abercrombie**	2764835	940.90	44,308	940.91	44,308	0.01	0

North Dakota Diversion (LPP) - 1% Chance Event (No Protection)							
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	119,255	801.81	120,751	0.08	1,496
Minimum Impact Location	1410241	811.47	113,625	811.51	115,682	0.04	2,057
Oslo Gage	1416287	813.01	113,556	813.07	115,628	0.06	2,071
Grand Forks Gage	1558518	832.97	107,980	833.21	110,497	0.24	2,517
Maximum Impact Location	1573768	835.27	80,735	835.56	80,686	0.29	-49
Thompson Gage	1667877	847.35	82,926	847.39	82,608	0.04	-317
Halstad Gage	1981580	869.09	71,581	869.03	70,992	-0.06	-589
Fargo Gage (13th Ave S, 12th Ave S)	2388223	903.86 (41.12*)	34,875	893.54 (30.8*)	11,718	-10.32	-23,157
US Diversion**	2531315	914.65	21,458	922.88	11,024	8.23	-10,434
Hickson Gage**	2563754	917.52	21,730	922.90	18,655	5.38	-3,075
Abercrombie**	2764835	935.62	23,000	935.73	23,000	0.11	0

North Dakota Diversion (LPP) - 2% Chance Event (No Protection)							
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	800.72	100,869	800.80	102,165	0.08	1,296
Minimum Impact Location	1410241	811.12	97,700	811.15	98,889	0.03	1,189
Oslo Gage	1416287	812.53	97,643	812.57	98,857	0.04	1,215
Grand Forks Gage	1558518	831.13	91,118	831.31	92,619	0.18	1,501
Maximum Impact Location	1602184	836.27	69,861	836.65	70,584	0.38	723
Thompson Gage	1667877	844.83	69,367	845.07	70,104	0.24	737
Halstad Gage	1981580	867.99	59,416	867.99	59,542	0.00	126
Fargo Gage (13th Ave S, 12th Ave S)	2388223	902.6 (39.86*)	29,167	892.72 (29.98*)	10,603	-9.88	-18,565
US Diversion**	2531315	913.76	18,435	920.86	10,477	7.10	-7,959
Hickson Gage**	2563754	916.34	18,898	920.92	18,428	4.58	-470
Abercrombie**	2764835	934.48	20,726	934.62	20,726	0.14	0

North Dakota Diversion (LPP) - 10% Chance Event (No Protection)							
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	62,917	798.54	63,042	0.01	125
Minimum Impact Location	1327581	803.44	57,657	803.45	58,094	0.01	437
Oslo Gage	1416287	810.51	59,092	810.55	59,629	0.04	537
Grand Forks Gage	1558518	825.98	56,662	826.09	57,169	0.11	507
Maximum Impact Location	1561283	826.49	43,551	826.61	43,504	0.12	-47
Thompson Gage	1667877	837.58	42,815	837.62	42,843	0.04	28
Halstad Gage	1981580	864.55	34,653	864.43	34,160	-0.12	-493
Fargo Gage (13th Ave S, 12th Ave S)	2388223	897.33 (34.59*)	17,024	891.86 (29.12*)	10,156	-5.47	-6,868
US Diversion**	2531315	908.06	10,333	916.29	8,861	8.23	-1,472
Hickson Gage**	2563754	910.21	10,428	916.80	10,077	6.59	-351
Abercrombie**	2764835	929.05	11,278	929.16	11,278	0.11	0

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

** Discharge does not include flow conveyed in the floodplain outside the main conveyance channel of the Red River

Table B-2

Summary HEC-RAS Unsteady Flow Model Results for Historic Floods - Locally Preferred Plan

North Dakota Diversion (LPP) - 1997 Event (No Protection)

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	123,404	801.94	123,251	-0.01	-153
Oslo Gage	1416287	813.29	124,661	813.30	124,735	0.01	74
Minimum Impact Location	1555329	833.59	119,246	833.60	119,281	0.01	35
Grand Forks Gage	1558518	834.04	119,103	834.05	119,142	0.01	39
Thompson Gage	1667877	847.29	78,351	847.43	79,439	0.14	1,088
Maximum Impact Location (Nielsville)	1829877	860.86	71,728	861.11	72,925	0.25	1,197
Halstad Gage	1981580	868.65	64,821	868.78	66,780	0.13	1,959
Fargo Gage (13th Ave S, 12th Ave S)	2388223	902.42 (39.68*)	27,574	893.11 (30.37*)	9,968	-9.31	-17,606
US Diversion**	2531315	911.89	13,686	921.60	9,530	9.71	-4,156
Hickson Gage**	2563754	913.85	13,729	921.63	13,235	7.78	-494
Abercrombie**	2764835	931.08	13,995	931.36	13,995	0.28	0

North Dakota Diversion (LPP) - 2006 Event (No Protection)

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	78,252	799.46	78,666	0.02	414
Oslo Gage	1416287	811.58	74,550	811.61	75,093	0.03	543
Minimum Impact Location	1443147	813.86	75,635	813.88	76,312	0.02	677
Grand Forks Gage	1558518	828.63	72,782	828.72	73,387	0.09	605
Thompson Gage	1667877	840.63	52,499	840.84	53,273	0.21	775
Maximum Impact Location	1749702	848.33	52,262	848.59	53,030	0.26	768
Halstad Gage	1981580	866.64	43,060	866.70	43,552	0.06	492
Fargo Gage (13th Ave S, 12th Ave S)	2388223	899.57 (36.83*)	21,028	891.96 (29.22*)	10,109	-7.61	-10,919
US Diversion**	2531315	910.60	14,053	918.72	9,530	8.12	-4,523
Hickson Gage**	2563754	913.11	14,313	918.90	14,362	5.79	49
Abercrombie**	2764835	931.58	15,027	931.74	15,027	0.16	0

North Dakota Diversion (LPP) - 2009 Event (No Protection)

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	85,308	799.84	85,166	-0.01	-143
Minimum Impact Location	1345544	805.87	91,028	805.88	90,929	0.01	-99
Oslo Gage	1416287	812.02	85,672	812.04	84,367	0.02	-1,304
Grand Forks Gage	1558518	829.33	77,165	829.39	77,550	0.06	385
Maximum Impact Location	1561353	830.20	63,468	830.28	63,506	0.08	38
Thompson Gage	1667877	843.05	61,510	843.07	61,577	0.02	67
Halstad Gage	1981580	867.60	55,176	867.56	54,910	-0.04	-266
Fargo Gage (13th Ave S, 12th Ave S)	2388223	902.66 (39.92*)	29,234	893.46 (30.72*)	11,561	-9.20	-17,674
US Diversion**	2531315	914.24	23,639	921.62	10,897	7.38	-12,742
Hickson Gage**	2563754	917.76	24,393	921.64	24,562	3.88	170
Abercrombie**	2764835	937.51	28,176	937.59	28,176	0.08	0

North Dakota Diversion (LPP) - 2010 Event (No Protection)

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	65,928	798.72	66,106	0.01	177
Minimum Impact Location	1327581	803.80	66,011	803.81	65,808	0.01	-203
Oslo Gage	1416287	811.09	67,101	811.07	66,850	-0.02	-251
Grand Forks Gage	1558518	827.23	63,406	827.19	63,172	-0.04	-235
Thompson Gage	1667877	840.28	52,023	840.44	52,694	0.16	672
Halstad Gage	1981580	866.55	42,389	866.70	43,585	0.15	1,196
Maximum Impact Location (Hendrum)	2038409	870.62	38,264	870.86	39,350	0.24	1,085
Fargo Gage (13th Ave S, 12th Ave S)	2388223	899.77 (37.03*)	21,481	892.38 (29.64*)	10,291	-7.39	-11,190
US Diversion**	2531315	910.17	12,352	918.90	8,623	8.73	-3,729
Hickson Gage**	2563754	912.23	12,677	918.98	12,686	6.75	8
Abercrombie**	2764835	930.57	13,236	930.74	13,236	0.17	0

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

** Discharge does not include flow conveyed in the floodplain outside the main conveyance channel of the Red River

Table B-3

Summary HEC-RAS Unsteady Flow Model Results for Design Floods - Federally Comparable Plan

Minnesota Diversion (FCP) - 0.2% Chance Event (No Protection)

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	168,364	804.27	170,409	0.15	2,045
Oslo Gage	1416287	813.88	152,851	813.95	157,374	0.07	4,523
Minimum Impact Location	1416400	814.23	152,852	814.29	157,375	0.06	4,522
Grand Forks Gage	1558518	836.36	146,225	836.72	150,748	0.36	4,523
Maximum Impact Location	1580152	839.75	102,174	840.20	104,725	0.45	2,551
Thompson Gage	1667877	850.69	112,422	850.93	115,330	0.24	2,908
Halstad Gage	1981580	871.54	101,754	871.72	104,334	0.18	2,580
Fargo Gage (13th Ave S, 12th Ave S)	2388223	905.8 (43.06*)	61,717	902.83 (40.09*)	30,044	-2.97	-31,673
US Diversion**	2470898	910.99	32,153	910.81	34,471	-0.18	2,319
Hickson Gage**	2563754	919.69	35,636	919.67	35,565	-0.02	-71
Abercrombie**	2764835	940.90	44,308	940.90	44,308	0.00	0

Minnesota Diversion (FCP) - 1% Chance Event (No Protection)

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	119,255	801.92	122,945	0.19	3,690
Minimum Impact Location	1408098	811.34	113,281	811.39	116,227	0.05	2,946
Oslo Gage	1416287	813.01	113,556	813.09	116,500	0.08	2,944
Grand Forks Gage	1558518	832.97	107,980	833.35	112,047	0.38	4,067
Thompson Gage	1667877	847.35	82,926	848.11	88,519	0.76	5,593
Maximum Impact Location	1813905	860.78	75,611	862.01	81,907	1.23	6,296
Halstad Gage	1981580	869.09	71,581	869.68	80,624	0.59	9,043
Fargo Gage (13th Ave S, 12th Ave S)	2388223	903.86 (41.12*)	34,875	894.91 (32.17*)	11,756	-8.95	-23,119
US Diversion**	2470898	910.13	29,330	910.71	22,794	0.58	-6,536
Hickson Gage**	2563754	917.52	21,730	917.51	21,734	-0.01	3
Abercrombie**	2764835	935.62	23,000	935.62	23,000	0.00	0

Minnesota Diversion (FCP) - 2% Chance Event (No Protection)

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	100,869	800.83	102,845	0.11	1,976
Oslo Gage	1416287	812.53	97,643	812.56	98,491	0.03	848
Minimum Impact Location	1448026	814.89	84,147	814.91	85,013	0.02	867
Grand Forks Gage	1558518	831.13	91,118	831.26	92,141	0.13	1,023
Thompson Gage	1667877	844.83	69,367	845.61	73,330	0.78	3,963
Maximum Impact Location	1829650	858.51	63,541	859.52	67,966	1.01	4,425
Halstad Gage	1981580	867.99	59,416	868.47	65,150	0.48	5,735
Fargo Gage (13th Ave S, 12th Ave S)	2388223	902.6 (39.86*)	29,167	894.02 (31.28*)	10,878	-8.58	-18,289
US Diversion**	2470898	909.54	27,658	909.4	27,987	-0.14	329
Hickson Gage**	2563754	916.34	18,898	916.37	18,925	0.03	27
Abercrombie**	2764835	934.48	20,726	934.49	20,726	0.01	0

Minnesota Diversion (FCP) - 10% Chance Event (No Protection)

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	62,917	798.57	63,651	0.04	734
Minimum Impact Location	1410241	809.75	58,880	809.78	59,596	0.03	717
Oslo Gage	1416287	810.51	59,092	810.56	59,699	0.05	607
Grand Forks Gage	1558518	825.98	56,662	826.10	57,258	0.12	596
Thompson Gage	1667877	837.58	42,815	837.82	43,590	0.24	775
Halstad Gage	1981580	864.55	34,653	864.88	35,715	0.33	1,063
Maximum Impact Location	2236491	883.37	29,991	883.82	32,040	0.45	2,048
Fargo Gage (13th Ave S, 12th Ave S)	2388223	897.33 (34.59*)	17,024	892.66 (29.92*)	9,933	-4.67	-7,091
US Diversion**	2470898	904.54	16,759	904.71	17,329	0.17	570
Hickson Gage**	2563754	910.21	10,428	910.27	10,459	0.06	31
Abercrombie**	2764835	929.05	11,278	929.05	11,278	0.00	0

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

** Discharge does not include flow conveyed in the floodplain outside the main conveyance channel of the Red River

Table B-4

Summary HEC-RAS Unsteady Flow Model Results for Historic Floods - Federally Comparable Plan

Minnesota Diversion (FCP) - 1997 Event (No Protection)

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	123,404	802.05	125,375	0.10	1,971
Oslo Gage	1416287	813.29	124,661	813.34	126,501	0.05	1,840
Minimum Impact Location	1425253	814.37	107,206	814.40	108,227	0.03	1,021
Grand Forks Gage	1558518	834.04	119,103	834.21	120,893	0.17	1,790
Thompson Gage	1667877	847.29	78,351	847.66	81,143	0.37	2,792
Maximum Impact Location	1813905	859.97	71,913	860.6	74,743	0.63	2,830
Halstad Gage	1981580	868.65	64,821	868.92	68,476	0.27	3,655
Fargo Gage (13th Ave S, 12th Ave S)	2388223	902.42 (39.68*)	27,574	894.1 (31.36*)	9,978	-8.32	-17,596
US Diversion**	2470898	908.85	23,779	908.94	25,235	0.09	1456
Hickson Gage**	2563754	913.85	13,729	914.00	13,738	0.15	10
Abercrombie**	2764835	931.08	13,995	931.08	13,995	0.00	0

Minnesota Diversion (FCP) - 2006 Event (No Protection)

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	78,252	799.47	78,770	0.03	518
Oslo Gage	1416287	811.58	74,550	811.60	74,929	0.02	379
Minimum Impact Location	1448026	814.15	67,113	814.16	67,444	0.01	331
Grand Forks Gage	1558518	828.63	72,782	828.69	73,160	0.06	378
Thompson Gage	1667877	840.63	52,499	840.84	53,450	0.21	951
Halstad Gage	1981580	866.64	43,060	866.86	44,955	0.22	1,895
Maximum Impact Location	2058853	871.99	36,500	872.36	38,554	0.37	2,054
Fargo Gage (13th Ave S, 12th Ave S)	2388223	899.57 (36.83*)	21,028	893.15 (30.41*)	10,078	-6.42	-10,950
US Diversion**	2470898	906.81	20,782	906.53	20,782	-0.28	0.00
Hickson Gage**	2563754	913.11	14,313	913.15	14,352	0.04	39
Abercrombie**	2764835	931.58	15,027	931.58	15,027	0.00	0

Minnesota Diversion (FCP) - 2009 Event (No Protection)

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	85,308	799.98	87,702	0.13	2,393
Minimum Impact Location	1410241	810.81	83,759	810.89	87,295	0.08	3,536
Oslo Gage	1416287	812.02	85,672	812.16	87,316	0.14	1,645
Grand Forks Gage	1558518	829.33	77,165	829.83	80,831	0.50	3,666
Thompson Gage	1667877	843.05	61,510	843.97	65,379	0.92	3,869
Maximum Impact Location	1789494	853.76	58,180	854.88	62,266	1.12	4,086
Halstad Gage	1981580	867.6	55,176	868.02	60,798	0.42	5,622
Fargo Gage (13th Ave S, 12th Ave S)	2388223	902.66 (39.92*)	29,234	894.03 (31.29*)	11,964	-8.63	-17,270
US Diversion**	2470898	909.61	28,395	909.47	27,912	-0.14	-483
Hickson Gage**	2563754	917.76	24,393	917.75	24,407	-0.01	14
Abercrombie**	2764835	937.51	28,176	937.51	28,176	0.00	0

Minnesota Diversion (FCP) - 2010 Event (No Protection)

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	65,928	798.76	66,687	0.05	759
Oslo Gage	1416287	811.09	67,101	811.11	67,463	0.02	363
Minimum Impact Location	1467237	815.28	66,433	815.30	66,870	0.02	437
Grand Forks Gage	1558518	827.23	63,406	827.29	63,783	0.06	377
Thompson Gage	1667877	840.28	52,023	840.55	53,139	0.27	1,116
Maximum Impact Location	1829650	853.73	49,914	854.1	51,122	0.37	1,208
Halstad Gage	1981580	866.55	42,389	866.76	43,888	0.21	1,499
Fargo Gage (13th Ave S, 12th Ave S)	2388223	899.77 (37.03*)	21,481	893.37 (30.63*)	10,231	-6.40	-11,250
US Diversion**	2470898	906.89	20,427	906.8	21,469	-0.09	1043
Hickson Gage**	2563754	912.23	12,677	912.42	12,697	0.19	20
Abercrombie**	2764835	930.57	13,236	930.57	13,236	0.00	0

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

**Discharge does not include flow conveyed in the floodplain outside the main conveyance channel of the Red River

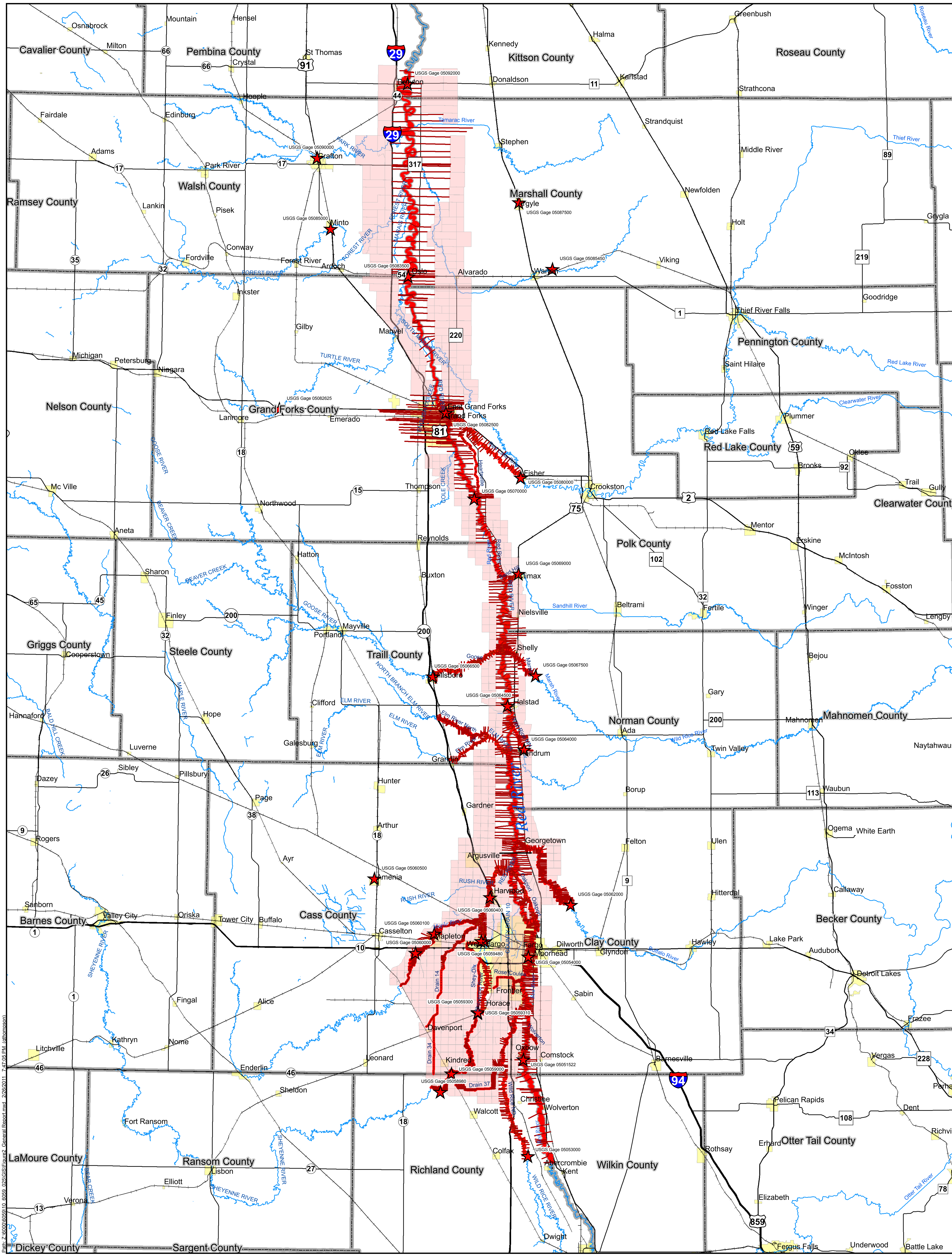

Figure B-1

Unsteady HEC-RAS Modeling
Study Area Map

- USGS Gages
- Existing Conditions Model Cross Sections
- Existing Conditions Model Reaches
- Existing Conditions Model Storage Areas
- Model Cross Sections
- City Boundaries
- Counties

0 3 6 12 18
Miles

USACE-MVP-0000087955

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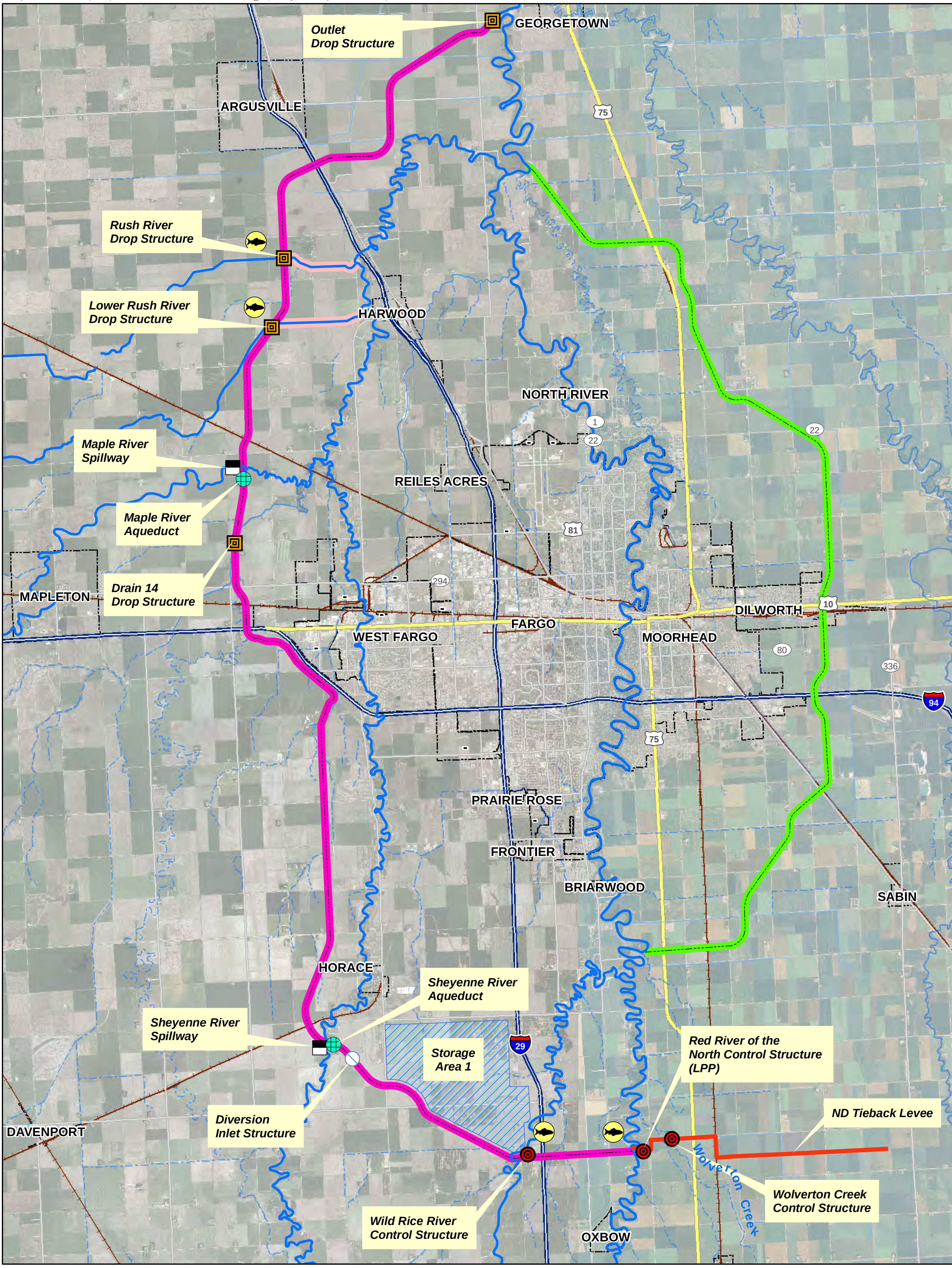

Hydraulic Structures

- 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 B-2

PROJECT OVERVIEW

Fargo - Moorhead Area

N

0 0.5 1 2 3 4 Miles

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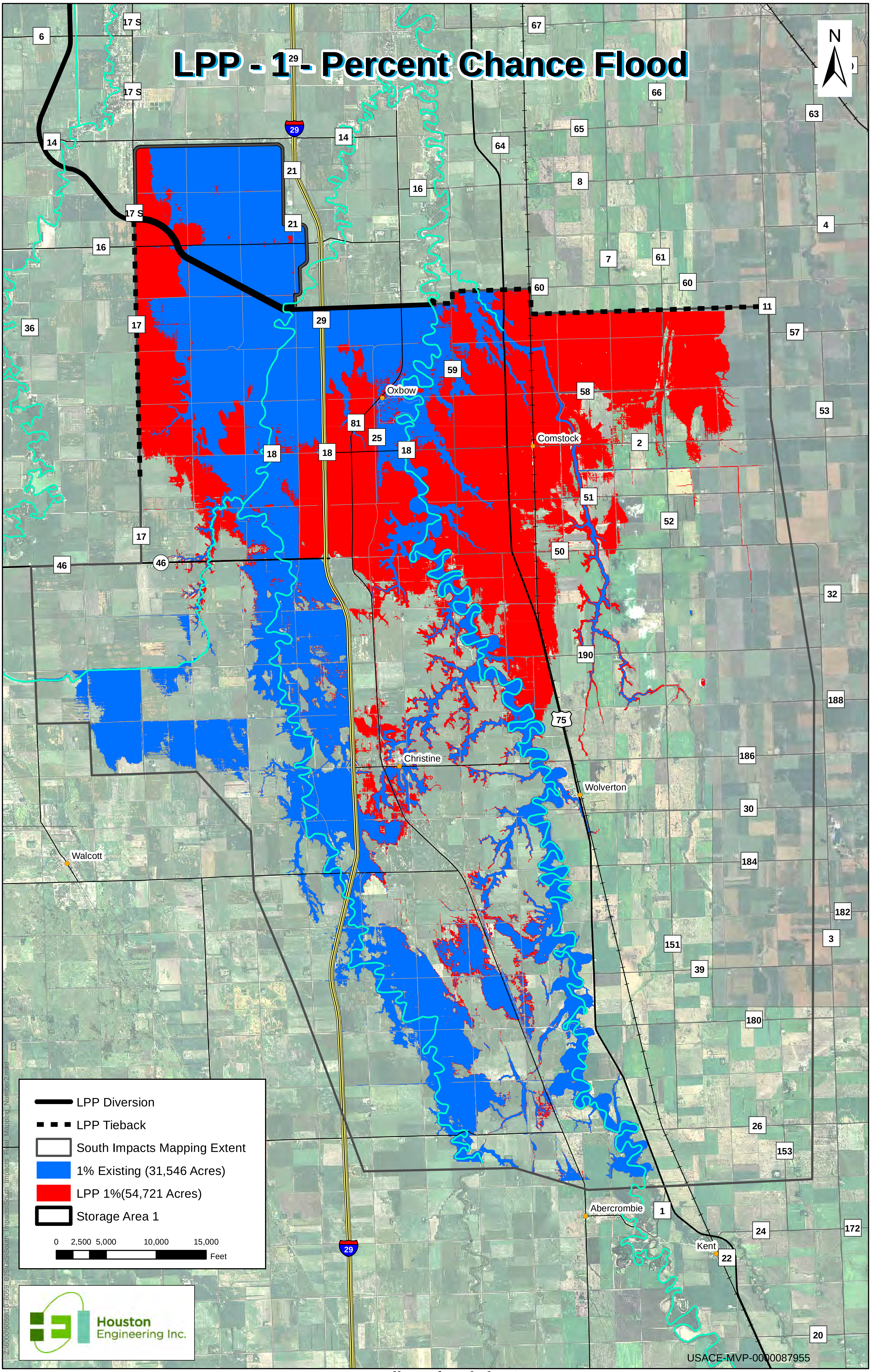

Figure B-3: Locally Preferred Plan Staging Area

Attachment 1

Ice Analysis for
Red River of the North Diversion Project
Fargo ND, Moorhead, MN

DRAFT REPORT

Ice Analysis for
Red River of the North Diversion Project
Fargo ND, Moorhead, MN

by:

Andrew M. Tuthill
Meredith L. Carr

HEC-RAS results added by Aaron Buesing, St. Paul District

Ice Engineering Group
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July 11, 2011

Contents

1. Introduction

The all-time peak stage on the Red River of the North (RRN) at Fargo, ND occurred on March 28, 2009, causing major flooding in the Fargo and Moorhead (F/M) area (Fig. 1). 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 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 on the North Dakota side (Fig. 2). The canal would cross and intercept flow from five tributaries to re-enter the RRN about 34 RM north of Fargo. It is estimated that 10-20 years will be required to build the project at an estimated cost of about $1.4 billion. Construction will take place from downstream to upstream, intercepting the downstream tributaries first and building the upstream crossing and diversion structures later on. This will allow time to monitor ice conditions and refine ice aspects of the design of the more complex upstream structures.

This report describes the first phase of ice analyses and field observations being done in support of project design. A main objective is to identify and design for possible ice problems at the RRN diversion and tributary crossing structures. A second objective is to anticipate possible changes to the ice regime on the RRN in the Fargo-Moorhead (F-M) area. A concern is that the project may change nature of ice breakup on the mainstem RRN. Under existing conditions the spring rise is long and gradual allowing most of the RRN ice to melt by the time of peak stage. With the diversion in operation, the peak at Fargo, though lower, may occur before the ice has melted. This change could lead to breakup ice jamming and possible flooding at lower discharges than before.

1.1. Approach

The ice study consists of three phases: 1.) a review and analysis of historical ice and hydro-met data combined with an ice monitoring program, 2.) an assessment of impacts to the ice regime in the vicinity of F-M as a result of the diversion project and 3.) analyses of ice interaction with the RNN diversion and tributary crossing structures. This interim report describes the results of the first phase.

Historical information and hydro-meteorological data related to spring flooding and ice processes on the RRN in the project area were compiled and reviewed. Performance of similar flood control projects was also reviewed. Satellite imagery and aerial photos were compiled and incorporated in the above analysis of historical and hydro-met information. A model was developed to calculated ice growth and decay for the period of record, and it was validated with observed data. Ice conditions were monitored at key locations on the ground and from the air during the winter of 2011.

Looking beyond this phase of the study, possible changes to the RRN ice regime in the vicinity of Fargo-Moorhead as a result of the diversion will be assessed. The HEC-RAS model will be used to simulate ice-affected water surface profiles in this section of river during the spring breakup period using inputs from the historical and hydro-met analyses and the ice monitoring program. HEC-RAS simulation results will be used to assess the potential for ice jam flooding on this section of the RRN under with-project conditions.

2. Initial Investigations and Ice Monitoring

The initial ice investigations included a review of historical spring flood information analysis of hydro-meteorological data. The performance of similar flood control projects under ice conditions was reviewed and a model developed to calculated RRN ice growth and decay for the last several decades. Ice conditions were monitored from the ground, by aircraft and the use of satellite imagery.

2.1. Historical Spring Floods on the RRN

General Ice Processes on the RRN

The RRN in the vicinity of Fargo and Moorhead is low gradient 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. Ice-out on the RRN is typically gradual without dynamic ice runs. While water levels may rise well above the top of bank elevation, the ice cover typically melts in place, often confined within bends or held within the channel by natural levees or trees along the banks. An important question in this study is the timing of ice-out with respect to the timing of peak discharge and stage. Other important questions relate to how and where the ice moves or accumulates during the breakup period.

Winter period flood problems on the RNN appear to be increasing with time as shown in the time series of annual peak discharges (Fig. 3). Of the 8 highest peak discharges, 6 occurred in the last 14 years and 5 of those 6 occurred between Dec. 1 and April 15 when there was a strong likelihood of ice on the river. Fig. 4 compares hydrographs for the six most recent large spring floods at Fargo showing that the timing of peak stage falls within a 4 week time frame and the rise from base flow to peak typically takes about 1-1/2 weeks. Fig. 5 shows the probability distribution of annual of peak discharges at Fargo showing the majority of the largest floods to have occurred during the ice season. In addition to the Fargo-Moorhead area which experienced a flood or record on March 28, 2009, ice-related flooding has occurred at a number of other locations along the RRN including Oslo, Drayton and Pembina, and Grand Forks. Damages at Grand Forks there have been greatly reduced by flood control works built since the 1997 flood of record. In 1997, flooding was widespread along the RRN at locations such as Wahpeton, Tyler, Breckenridge, Hickson, Fargo, Drayton, Grand Forks and Emerson. Table 1 lists the 8 most severe flood events in terms of peak discharge at Fargo since 1897, 7 of which occurred between Dec. 1 and April 15.

Table 1. Historic RNN Floods at Fargo-Moorhead.

Year	**Day**	**Peak Stage** (ft)	**Peak Discharge** (cfs)	**Recurrence Interval for Discharge** (years)
2009	3/28	40.84	29,500	112
1997	4/17	39.57	28,000	56
2011	4/9	38.74	25,700	37
1969	4/15	37.34	25,300	28
1897	4/7	39.10	25,000	22
2010	3/21	36.99	21,200	19
2001	4/14	36.69	20,300	16
2006	4/5	37.13	19,900	14

Plots of discharge, stage, air temperature and AFDD for the years 1958-2011 can be found in Appendix A (available upon request). A discussion of these hydro-meteorological data appears in Section 2.2.

The CRREL Ice Jam Database (IJDB) reports less severe ice related 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 .

Below is a discussion of historic flood imagery and the timing of ice-out on the RNN in the vicinity of Fargo and Moorhead for the recent floods of 1997, 2001, 2006, 2009, 2010 and 2011. These observations are later compared to the 2011 field observations and also used to validate the ice growth model described in Section 2.4. The 27 photographs referenced in this section appear in Appendix B.

For these six floods, the more confined, faster flowing section of the RRN within the F-M metropolitan area had melted out before the time of peak stage. Upstream and downstream of the cities, remnants of sheet ice remained within tree-lined channels of the RRN and tributaries including the general locations of the inlet and outlet of the proposed ND-side diversion canal. Ice jams were documented upstream of the RRN bridges in the F/M metropolitan area but these accumulations caused little stage rise and melted rapidly in the presence of the high flows. More serious ice jams have been reported at bridges and culverts along the tributaries particularly the Sheyenne R. and the canal crossings within the West Fargo Diversion Project.

1997 Flood

The 1997 flood peaked late in Fargo on 4/17 at 39.57 ft (Figs.4 and 8). Satellite images at this time (Figs B1-B3) show remnants of the ice cover in some of the sharper bends above and below F/M, but the river is mostly ice free through the metropolitan area. The images show residual ice within the original channel near the start and end of the proposed ND-side diversion canal. Fig. B4 shows no remaining ice no ice within the F/M Metropolitan area at the time of the 4/17/1997 peak. The ground was mostly free of snow cover at this time making it difficult to distinguish fields from flooded areas. USGS discharge measurements at Fargo indicate the presence of an ice cover at the gage on 25 March and intermittent shore ice until 4/11. The last ice-affected gage reading at Fargo was on 4/2/1997.

2001 Flood
Flood stages peaked in Fargo on 4/14 at 36.65 ft. Aerial photos at about this time show no evidence of an ice cover or ice floes in the F-M area (Fig. B5 and B6). USGS discharge measurements at Halstad, Fargo and Hickson reported ice-free conditions sometime before 4/10, 4/3 and 4/9, respectively. The last ice affected gage reading at Fargo occurred on 3/10/01. This information suggests that ice had released of melted prior to the peak of the flood.

2006 Flood
The 2006 flood peaked at Fargo on 4/5 at 36.82 ft. Aerial Photos taken near the peak, show no remaining ice cover in the bends of the Red River in the Fargo area (Fig. B7 and B8). Figs.B9-B11 show the progressive melting of an ice accumulation upstream of the 12th Ave. Bridge in Fargo between 4/1 and 4/4/2006. USGS measurements at the Fargo Gage documented the presence of ice on 3/28 and ice-free conditions starting on 4/1/2006.

2009 Flood
The flood of record peaked at the Fargo Gage on 3/28/2009 at 40.65 ft. Figs. B12 and B13 are satellite images near the time of peak. As in the other cases, the river was ice free through most the F-M metropolitan area by the time of peak. Upstream and downstream of the city shine lines of ice are visible within flooded bends at the start and end of the proposed ND diversion canal, at the Wild Rice confluence and north of F-M. An aerial photo of the confluence of the RRN and Wild Rice River shows considerable ice remaining within in the flooded river channels (Fig. B14). Fig. B15 shows the RRN downstream of the Wild Rice into Fargo to be mostly clear of ice by the time of peak. The 2009 flood differs from others in that the ground was snow covered giving much of the flooded areas a gray slushy appearance.

Fig. B16 shows ice accumulated upstream of the 12th Ave. N. Bridge in Fargo on 3/24. Fig. B17 shows open water on the RRN in Moorhead on 3/25 and Figs.B18 and B19 show open river conditions in downtown F/M and upstream at 14th Ave. N in Fargo respectively. By 3/27 no ice remained behind the major area bridges (Fig. B20).

USGS measurements at Fargo indicated an ice cover on 3/21 March, and an ice free condition by 3/24/2009. Both the Halstad and Hickson Gages retained their ice longer, until 3/28 and 4/1 respectively.

2010 Flood
The 2010 flood peaked early at 36.99 ft on 3/21. An image just prior to the peak show ice free channels in parts of Fargo (Fig. B21). A series of image from 13-15 Mar (Figs.B22-24) show the gradual melt-out of ice on a section of the RRN just north of the downtown area , suggesting that much of the RNN in the Fargo area melted out prior to the 3/21 peak. The record of images from the Fargo USGS Gage Web camera show breakup occurring gradually between 3/13 and 3/15 (see FargoGage2010Breakup.pdf). Fig. B25 shows large ice floes still passing through the 1st Ave. Bridge in Moorhead on 3/20. Fig. B26 shows ice in lodged bends or moving as large floes south of F-M in the vicinity of the start of the proposed diversion canal. Farther north and into Fargo, the river appears ice free (Fig. B27). USGS discharge measurements indicate ice on the RRN downstream at the Hickson Gage on 3/20, but this ice was reported to have cleared by 3/23. The RRN was noted to be ice covered at the Fargo Gage on 3/17 March, but clear by 3/19.

2011 Flood
The 2011 flood peaked in Fargo at 38. 74 ft on 4/9. As with the other floods described above, the RRN channel through the F/M metropolitan area was ice free at this time. The USGS reported an ice-free condition at the Fargo Gage in 3/16. The record of images from the Fargo USGS Gage Web camera show breakup in the section of river upstream of the F-M bridges occurring later, between 4/4 and 4/5 (see FargoGage2011Breakup.pdf). Although sections of the channel had opened up through the cities by the end of March, cold weather persisted and much of the river in the project area upstream and downstream retained a nearly complete ice cover until about 4/1 when temperatures moderated. Some ice jams occurred at bridges along the West Fargo Diversion canal and were broken using large excavators. The 2011 ice-out is described in more detail in Section 2.5 which reports on field monitoring efforts.

2.2. Analysis of Hydro-Meteorological Data

Daily average discharge, stage, air temperature, and net accumulated freezing degree days (AFDD) plotted for 1958-2011period of record for the following locations shown in Table 2.

Table 2. Hydro-Meteorological Data; RRN and Tributaries near Fargo-Moorhead.

Parameter	*Location*
Daily Average Discharge and Stage USGS Gauging Stations	RRN Gage @ Kindred, ND
	RRN @ Fargo, ND
	RRN @ Halstad, ND
	Wild Rice R. @ Mantador, ND
	Sheyenne R. at Kindred, ND
	Maple River at Mapleton, ND
Daily Average Air Temperature	Hector Int. Airport, Fargo, ND
Accumulated Freezing Degree Days	

Plots of these data for the December-April period for the years of 1958-2011 appear in Appendix A (available upon request). Plots of the three largest floods; 2009, 2011 and 1997 are shown in Figs.6, 7, and 8 respectively. Hydro-met data were analyzed to identify ice characteristics associated with major spring floods. For each year of record back to 1958, Table 3 lists the day of peak stage, daily average discharge and accumulated freezing degree day for the day of peak ($AFDD_{peak}$). $AFDD_{peak}$ is typically preceded by winter maximum AFDD ($AFDD_{max}$) which is also listed. Maximum ice thickness $t_{i\ max}$ was calculated from $AFDD_{max}$.

$$\overline{} \qquad (1)$$

US Army (2005) *C* is a coefficient typically ranging from 0.3 to 0.6 inches. A *C* value of 0.59 was used based on field-measured ice thickness data. An important parameter in this study is the condition of the ice cover by the time of peak stage. Equation 1 is a good predictor of thermal ice growth ice on slow moving rivers such as the RRN but is poorly suited for estimating ice melting, hence the need for the more sophisticated ice melting model which is presented in Section 2.4. As a rough indicator of ice conditions at the time of peak stage, the time intervals between $AFDD_{max}$ and $AFDD_{peak}$ are listed, as well as the corresponding decrease in AFDD.

The photo analysis in Section 2.1 indicate that ice-out along the RRN though the F-M metropolitan area typically occurs before the time of peak while upstream and downstream ice covers tend to stay in place longer. The last day of reported ice at the Fargo Gage is listed in Table 3 for the 1990-2011 period with the number of days between ice-out to peak stage. Table 4 compares these parameters for the 6 largest floods with average values for the entire 1958-2011 period showing discharge to be the most important factor. For the flood years, the average day of peak stage is 7 days later than the long-term average date and the average discharge 16,400 cfs higher. Average $AFDD_{peak}$, $AFDD_{max}$ are slightly greater for flood years with calculated ice thickness 1.7 inches higher. This suggests that ice is a minor player in the maximum flood levels experienced on the RRN in the F-M area. The decrease in AFDD for the flood years is nearly the same as the average; 76 compared to 74, but the period of warming is on average 6 days longer. Similarly, the time interval between the ice-out at Fargo and peak stage is much greater for the flood years; 14 days compared to the long term average of 1 day.

Table 4. Hydro-Meteorological Data for Major Floods vs. Long Term Averages

Year of Flood	Day of Peak Stage	Peak Stage (ft)	Peak Discharge (cfs)	AFDD Day of Peak Discharge	AFDD max (F)	Day of AFDD max	Max Ice Thickness (in)	AFDD max to Peak Stage (days)	Δ AFDD Max to Peak	Day of Ice Out at Fargo	Ice-Out to Peak Stage (days)
2009	28-Mar	40.65	29,500	2508	2533	14-Mar	30	14	-25	24-Mar	4
1997	17-Apr	39.57	27,630	2968	2988	12-Apr	32	5	-20	2-Apr	15
2011	9-Apr	38.74	25,700	2450	2600	1-Apr	30	8	-150	16-Mar	24
2010	21-Mar	36.99	21,200	1924	1981	5-Mar	26	16	-57	19-Mar	2
2006	5-Apr	36.82	19,400	1444	1536	25-Mar	23	11	-92	1-Apr	4
2001	14-Apr	36.65	20,200	2638	2750	27-Mar	31	18	-112	10-Mar	35
Flood Yrs.	5-Apr	38.24	23,938	2322	2398	24-Mar	29	12	-76	22-Mar	14
All Years	29-Mar	26.25	7,550	2,062	2,137	21-Mar	27	6	-74	26-Mar	1
Difference	7	12.0	16388	260	261	3	1.7	6	-2	-4	13

2.3. Performance of Similar Flood Control Projects Under Ice Conditions

A 29-mile long floodway was built on the Red River at Winnipeg, Manitoba following the devastating flood of May 1950. From its completion in 1969 to 2006, the project has saved the city an estimated $10 billion in damages. Much of the information below is from Andres (2005). The Red River in the vicinity of Winnipeg experiences is about double in size and discharge and experiences more dynamic ice runs and ice jams than the RRN in the vicinity of F-M. Due to a much larger drainage area, the spring flood period is more drawn out at Winnipeg, lasting up to one month. Breakup on the Red River at Winnipeg precedes the flood peak by a longer interval than on the RRN near F-M. The breakup can be a thermal or melt-in-place event typically when breakup discharge is below about 42,000 cfs or, for larger flows, a dynamic downstream progressing event with jams consolidating and increasing is size towards the Red River confluence with Lake Winnipeg.

Typically Red River ice cover upstream of Winnipeg is well deteriorated or gone by the start of the diversion into the floodway. This occurs at an open water discharge of about 42,000 cfs (35,000 with ice on the river). For the flow range potentially affected by ice conditions gates on the Red River below the Floodway inlet are lowered in small increments to maintain "natural condition" river stages upstream of the diversion. This lack of upstream stage fluctuation would tend to maintain an intact ice cover allowing it to melt in place rather than be drawn into the Floodway. At capacity, the Winnipeg Floodway can convey 56,000 cfs about 40% of the upstream river flow of 140,000 cfs.

Efforts are made to prevent ice from entering the Floodway with this year's breakup being an example. On April 7, 2011 several major ice jams had formed on the Red River at Winnipeg and downstream. CBC News reported that on that day the Floodway was "not operational" stating the "officials do not like to activate the gates when there is still ice on the river because of concern that the ice could clog up the floodway and create jams or cause structural damage to the bridge supports". Most of the ice jams on the Red River at Winnipeg had reportedly released by April 8.

The Floodway was activated on April 9. Videos from the Web show some ice pieces and debris passing over the inlet weir into the Floodway but this minor amount of ice appears to not have been a problem. Based on the Winnipeg experience, it appears that the proposed F-M diversion project can be operated to avoid ice problems.

2.4 Ice Growth and Decay Model

2.4.1. Background

Ice growth, decay and breakup were calculated by the unified degree-day method (UDDM) presented by Shen and Yapa (1985). This method improved upon the traditional modified Stefan degree day method which is only strictly valid for ice formation:

$$t_i = C\sqrt{NetAFDD} \qquad (2.2.4.1)$$

where C is a coefficient ranging from 0.3 to 0.6 (US Army, 2005) and AFDD is a summation of daily average degrees of frost. AFDD on day *n* are calculated by

$$AFDD = \sum_{1}^{n}(32 - T_a) \qquad (2.2.4.2)$$

where T_a is the daily average air temperature in ºF. The AFDD curve is then shifted up to a start value of zero on the date where it takes on consistently positive values, yielding the Net AFDD curve.

The unified degree-day method separates the phases of ice growth, deterioration and break-up. For break-up and deterioration, the following equation is used:

$$h = \left(h_o^{\,2} + \alpha S\right)^{1/2} - \beta t^{\theta} \qquad (2.2.4.3)$$

Where h_o is the initial ice cover thickness, α, β and θ are coefficients, and S is the AFDD since the ice cover formed. The final term represent heat flux from the water to the ice cover, which is only substantial when water temperatures are significantly higher than freezing, which is not the case on the Red River, so it will be neglected. The α coefficient differs between the formation and decay phase such that:

$$\alpha = \begin{cases} \alpha_o & for T_a^{\,2} \geq T_B \; growth \\ \alpha_o - m(T_a^{\,2} - T_B) & for T_a^{\,2} < T_B \; decay \end{cases} \qquad (2.2.4.4)$$

T_a^2 is the average freezing degree-day for the two preceding days and T_B is a value which represents the change to the decay condition such that when T_a^2 decreases to T_B decay begins. During the decay period, once h is smaller than $h_{max} - \Delta h_r$ ice thickness is no longer allowed to increase in equation (2.2.4.3). Once α reaches a lower limit of α_1, the breakup period begins. Ice thickness during the breakup period is evaluated as:

$$h_{j+1} = h_j - b \qquad (2.2.4.5)$$

where j is the day and b represents the loss of ice thickness to breakup.

In summary, to use the unified degree-day method for estimating ice thickness on the Red River of the North, the following parameters must be calibrated: α_o, α_1, m, T_B, Δh_r, and b.

2.4.2. Calibration

Hydro-meteorological data was used to calibrate and evaluate the ice thickness according to the unified degree-day method. At the USGS gage in Fargo, flow was recorded since 1901, water temperatures since 1998 and stage since 2000. Air temperature minima and maxima were recorded since 1948.

Ice measurements taken at the Fargo gage by the Corps in 2001-2003 and again in 2011 by Andy Tuthill of CRREL (Table 5).

Table 5. Ice Field Measurements and evaluated growth coefficients

Date	**Measured thickness (in)**	**AFDD (deg F-day)**	***C***	α_o
2/22/2001	27.72	2133	0.57	0.36
2/28/2002	12	693	0.38	0.21
2/27/2003	26.88	1684	0.63	0.43
1/10/2011	19	1001	0.56	0.37
		average*	0.59	0.39

** assuming 2002 data was an outlier*

The coefficient C, from the Stefan equation was evaluated for each of the measurement dates. An average of $C = 0.59$ was determined, assuming the 2002 measurement was an outlier.

To determine the growth coefficient α_o for the unified degree-day method, the freezeup date and initial freezeup ice thickness had to be determined. Using USGS records, the first date during which the flow was estimated due to ice effects at the Fargo gage was assumed to be the freezeup date (Table 6). For years without USGS records, the freezeup date was assumed to occur on the day of the average AFDD = 159 of the known data.

Table 6. Estimated Freezeup dates at the Fargo Gage

Water Year	Date	AFDD (deg F-day)
2010	5-Dec	63
2009	21-Nov	71
2008	26-Nov	22
2007	28-Nov	11
2006	2-Dec	164
2005	30-Nov	57
2004	19-Nov	77
2003	30-Nov	114
2002	29-Dec	321
2001	25-Nov	205
2000	13-Dec	45
1999	11-Dec	85
1998	15-Nov	78
1995	5-Jan	536
1994	25-Nov	104
1993	21-Dec	440
1992	22-Nov	173
1991	19-Dec	175
1990	3-Dec	270
	average	159

The ice thickness at freeze-up was estimated using the method from Pariset et al. (1966):

$$V = \sqrt{2g\frac{\rho - \rho_i}{\rho}h}\,\frac{h}{H} \qquad (2.2.4.6)$$

where ρ and ρ_i are the densities of water and ice, respectively, g is the acceleration due to gravity, the velocity $V=Q/BH$, Q is the flowrate, H is the depth, and B is the river width. The river width was assumed to be about 150 feet wide, as recorded during the ice measurements listed above. The formation thickness, h_o, was evaluated from Equation (2.2.4.6) for all the years during which stage measurements were available. For years when stage measurements were not recorded, the average of the evaluated h_o was assumed. Equation (2.2.4.3) was used to evaluate α_o for the years of the field measurements (Table 7). α_o =0.39 was calibrated from the average of α_o evaluated from the field thickness measurements, assuming the 2002 measurement was an outlier.

To calibrate the remaining parameters, α_1, *m*, T_B, Δh_r, and *b*, ice out date was required. Two methods to indicate ice out date were used. First, the date when the water temperature rose above freezing was proposed. Second, information on ice conditions was gleaned from USGS field reports, which indicated whether the river was ice covered during discharge measurements. These data, however, only describe conditions at the gage and provide a window for ice out because measurements were irregular (NWIS, 2011). The years 2006, 2009 and 2010 were used as calibration years because USGS measurements provided a small window which also matched the rise above freezing temperatures. The following table lists the parameters which were found to provide the best match for the data. Figs.1-3 show the water years used for calibration. It can be seen from these data that ice out occurs quickly after the maximum ice thickness and peak AFDD. This results in a small period of deterioration before the lower limit α_1 is reached and the breakup phase begin.

Table 7: Calibrated parameters

C	0.59	
α_o	0.39	
α_1	0.32	
m	0.04	
T_B	-1.1	ºF
Δh_r	2	in
b	3	in

2.4.3. Conclusions

Table 8 details the peak stages, peak AFDD and ice thickness for each year of record. Results show that the peak AFDD and maximum ice thickness occurred within a period of 15 days before to 40 days after the time of peak stage. On average peak stage occurred 10 days after the time of calculated maximum ice thickness, with a median of 11.5 days. The Δ AFDD ranges from -431 to -15 ºF-day, with an average of Δ AFDD= -88. The small difference in days and Δ AFDD between AFDD peak and stage peak indicates again that the ice deterioration period is very short on the Red River of the North at Fargo.

Table 9 shows the ice out dates by the three criteria: water temperature rise above freezing, first USGS observation of open water and the results of the unified-degree day method. By the unified degree-day method, for period of record, 38 ice-outs occurred before the peak stage, while 28 occurred after. For the known flood years of 2001, 2009 and 2010, the UDDM predicted ice-out at Fargo before the time of peak stage in agreement with observations presented in Section 2.2. For the flood of 1997 the UDDM, predicted ice-out in Fargo to occur after the peak stage. This error could have resulted from the fact that the UDDM models only the affects of thermal growth and decay due to ambient temperature and does not capture the effect of the warm inflow from tributaries. Also, all calibration was based on the ice-out date at the Fargo gage, which does not represent the entire study area. The observations presented in Section 2.2 suggest that residual ice covers can persist until the time of peak flow on sections of the RRN upstream and downstream of metropolitan F-M. The UDDM results do agree with the observations that, for many years, particularly ones with floods, ice-out occurs before or during the peak stage event. With more calibration the UDDM model could be used to model ice growth and decay at key locations such as the start and end of the diversion canal and tributary crossing structures. Appendix C (available upon request) contains plots of the ice growth and decay for each year of the period of record.

2.5. Field Monitoring of Ice Conditions 2011

CRREL made two trips to the F/M area during the 2011 winter to observe ice conditions and gather ice-related information. The first visit took place from January 11-13 under mid-winter conditions and the second on April 6-9 during the ramp up to Fargo's 3rd largest flood in history. The objective of the midwinter visit was to document ice formation processes, visit locations important to the proposed project and measure ice thickness at key locations. The winter visit also provided a low water inspection of RRN and tributary sites for signs of past ice action such as tree scarring and abrasion as well as ice damage to banks and structures. The spring visit took place during the rise to peak stage which occurred on April 9th . The extent and condition of the melting ice cover was documented as well as any evidence of ice movement or ice jamming.

Important locations included the section of RRN through Fargo-Moorhead and the sections of river upstream and downstream of the metropolitan area in the vicinity of the proposed diversion structure and canal outlet. Tributaries were inspected in vicinity of proposed diversions and canal crossings during the midwinter and breakup periods. Mid-winter ice thickness measured at the above-mentioned sites and used to calibrate the ice thickness model. The breakup period field observations relied on aerial reconnaissance as widespread flooding prevented access to many sites by car or foot. In addition to observing breakup period ice extent and condition, ice melting processes were documented along with ice transport and ice jamming locations. A series of captioned photos documenting the mid-winter and breakup period ice observations are included in Appendix D.

2.5.1. Mid-Winter Period

Sites visited during the Jan. 10-11 period are listed in Table 10 and shown on Fig. 8.

Table 10. Sites Visited and Measured ice Thickness, Jan. 10-11, 2011

Sites	*Ice Thickness (in)*
RRN vicinity of proposed diversion	12-19
RRN County Rd. (CR) 16 Bridge	
RRN Metropolitan Fargo-Moorhead	
Wild Rice R. canal crossing site	19
Sheyenne R. CR 16 Bridge (0.5 mi u/s of canal crossing site)	13.5-18
Sheyenne R. CR 10 Bridge	
Maple R. 168th Ave. (0.5 mi. d/s of canal crossing site)	

Sheyenne R. 32nd St. W	
Sheyenne R. CR 32 Bridge	
RRN vicinity of proposed canal outlet	18-21

The winter of 2010-2011 was relatively cold and snowy with over 1000 AFDD by the end of the first week in January and a snow depth of about 2 ft. These conditions produced an ice cover thickness of about 19 on slower moving portions the RRN and tributaries with about 12 in of ice on faster flowing sections. In the steeper section of the RRN through the F/M metropolitan area, open leads persisted (D13and D14).

January 10-11, 2010

In the vicinity of the proposed diversion, the banks were tiered with a low bank height of about 8 ft above base flow stage and high bank height in the15-20 ft range (D1). Abraded bark and minor scarring was seen on some trees near the between freezeup level and the top of high bank height (D2). There was no sign of significant tree scarring such as one sees on river with dynamic ice runs. Abrasion lines were seen on large trees near the proposed diversion location, 5-8 ft above the high bank level, probably the result of ice or debris movement across meanders during major floods (D3). The solid sheet ice and lack of tree scar evidence ice movement was also seen downstream of the diversion area from the CR 16 RRN Bridge crossing (D4).

The Wild Rice River in the vicinity of the proposed canal crossing and diversion was lined by 10-15-ft high banks confined within tree covered natural levees (D5 and D6). The sheet ice cover here was 19 in-thick.

The Sheyenne R. 0.5 mi upstream of the proposed canal crossing CR 16 Bridge had 10-ft high banks lined with dense tree growth (D7). Other than minor abrasion of tree bark, no evidence of dynamic ice action was found (D8). Measured ice thickness was between 14 to 20 inches.

The Sheyenne R. downstream of West Fargo was similar in appearance (D10). Under the deep snow cover, the baseflow channel of the Maple River near the proposed canal crossing location was difficult to distinguish within the treed river corridor (D9).

Approaching its confluence with the RRN, the Sheyenne R. channel was wider than upstream with lower banks and less dense tree cover (D11).

The RRN channel near proposed canal outlet was about 150 ft wide with 8-ft-high lower banks and higher banks about 20 ft above the base flow elevation. Here the sheet ice cover was 18-21-in-thick (D12).

2.5.2. Breakup Period

Significant melting of the ice cover occurred during the course of the April 6-9, 2011 field visit. The last two weeks of March 2011 had been unseasonably cold, putting off the start of the rise until the early of April. The ice in the vicinity of the Fargo Gage had released on April 4-5, leading to nearly ice-free conditions on the RRN in the FM metropolitan area by April 6 (D17 and D18). A series of web camera images from the Fargo Gage for the 2011 ice-out period can be seen in the attached FargoGage2011Breakup.pdf. A similar collection of images for 2010 shows the ice release occurring in phases on March 14-16 (FargoGage2010Breakup.pdf). In both cases, the ice in the vicinity of the gage the upstream sheet ice cover fractured into large floes that drifted downstream uneventfully without jamming or shoving.

April 6, 2011
Upstream of F-M, the RRN ice cover was for the most part intact in the vicinity of the proposed diversion structure on the evening of April 6 (D15). Similarly decayed sheet ice remained in the series of bends on the RRN in the vicinity of the Wild Rice R. confluence (D16).

April 7, 2011
A mid-day recon flight on April 7 found similar conditions on the RRN near the proposed diversion (D21-23) and the Wild Rice confluence (D20) on April 7 as on the previous evening. Some of the bends closer to F-M contained accumulations of ice floes (D19) but had negligible backwater effect.

The Wild Rice R. and surrounding land were completely flooded in the vicinity of the proposed diversion canal with trees providing the only clue as to the location of the river banks. (D24 and D39).

The Sheyenne R. was bankfull and covered in decayed sheet ice in the vicinity of CR 16 and the proposed canal crossing 0.5 mi upstream (D25, 26,40 and 41). Several miles upstream, broken floes had accumulated at the CR 14 Bridge (D42) and currents were fast enough downstream of the bridge to tilt floes on edge (D43).

Downstream, flow being diverted from the ice covered Sheyenne R. into the West Fargo Flood Control Project had caused floes to accumulate at the CR 17 Bridge, with no blockage or backwater effect (D28, D45-46). Along the West Fargo Diversion Canal minor accumulations of floes could be seen upstream of the railroad bridge near I-94 and the 13th Ave Bridge (D29, 30, 31, 47). The previous day, an ice blockage at the railroad bridge had created 3 ft of backwater and large excavators were used to break the jam and help the floes through the narrow openings. Reportedly ice blockages at bridges along the Sheyenne West Fargo Diversion Canal were a greater problem in 2010 when the earlier more rapid rise to peak resulted in the movement of thicker stronger ice floes.

Northward towards Harwood the Sheyenne R. was mostly ice free (D32 and 34). The Maple R. and surrounding fields in the vicinity of the proposed canal crossing was completely flooded and discernable only by traces of remaining sheet ice and trees lining the banks (D33).

The Sheyenne in the vicinity of its confluence with the Rush River held linear remnants of its sheet ice cover while the Rush maintained a decayed sheet ice cover on its channelized downstream portion (D35).

The RRN was mostly clear of ice from F-M upstream beyond Georgetown with some areas of drifting ice floes (D36). In the vicinity of the proposed canal outlet, the RRN was out of bank flooding adjacent fields (D37).

April 8, 2011
By the afternoon of April 8th, much of the residual ice covers seen the previous two days had disappeared. On the RRN in the vicinity of the Wild Rice confluence and the proposed diversion, most of the ice was gone (D50-52).

The Sheyenne from CR 16 to the West Fargo diversion was also clear of ice except for isolated minor jams at bridges and bends (D53-56).

2.6. Field Possible Changes to the Red River of the North Ice Regime in the Vicinity of Fargo Moorhead as a result of the Diversion Project

Possible changes to the RRN ice regime in the vicinity of Fargo-Moorhead as a result of the diversion project were assessed. The HEC-RAS model was then used to simulate with project water surface profiles in this section of river during the spring breakup period with inputs derived from the historical and preceding historical review, ice observations and hydro-met analyses. The potential for ice jamming for the with-diversion case was compared to existing ice-hydraulic conditions during recent spring floods in the F-M metropolitan area.

2.6.1. <u>HEC-RAS Modeling of Ice and Hydraulic Conditions in the Vicinity of Fargo-Moorhead.</u>

Inputs
Inputs to the HEC-RAS modeling of potential ice jams relied on a number of assumptions on breakup period ice conditions, ice sources reaches ice transport and ice jam locations. Also important was the range of river discharge at which the ice cover could be expected to remain intact, melt away, or transport downstream of form jams.

Under current plans it is expected that the mainstem RRN will convey about 10-11 Kcfs through the city during a 100-yr. flood event while the remainder passes through the diversion canal. During a 500 yr flood event, as much as 25 Kcfs may pass down the mainstem RRN through the F-M metropolitan area.

Ice hydraulic characteristics of recent floods were reviewed to determine the range of discharges above which ice is no longer a factor in the F-M area. The USGS daily discharge records indicate the dates that the Fargo gage is no longer ice-affected as far back as 1990. This transition to open water conditions would mean that no ice jams would exist downstream at the Main Ave., NP Railroad and Center Ave. Bridges to increase stage at the gage.

At the time of the USGS reported ice-free date, any remaining sheet ice on the upstream sections of the RRN would be melted or lodged in bends, otherwise this ice would release and accumulate at the downtown bridges, particularly the NP RR Bridge whose piers are closely-spaced. It is safe to assume that once the USGS declares the stage record as no longer ice affected, that the potential for ice jams in the F-M metropolitan area is past for that year.

The photo record that some ice accumulated at the 12 Ave Bridge on 4/1/2006, the USGS ice-out date. In the photos, this 2006 ice accumulation did not appear to be causing a significant stage rise and was mostly melted by the next day. In a discussion with the Fargo City engineers on 1/11/11, Mark Bittner said that the 2010 flood came early, cresting on 3/21. Though there was concern about ice jams in the city, he said that nearly all the ice was gone by 3/18 and the releasing floes passed through the bridges without significant jams. Mr. Bittner said that during the 2010 ice-out, some citizens reported a backup on the RRN at the 12 Ave. Bridge prompting a level survey by the City that found only a small difference in upstream and downstream stages.

For the 1990-2011 period, daily average discharges on the USGS-reported days of ice-out average 10,400 cfs. The highest ice-out day flow of 15,360 cfs, occurred on 3/19/2010, the earliest of the recent flood events. This is not surprising since one would expect thicker and more competent ice early on in the breakup season.

Based on these observations, 10,000 cfs represents a maximum upper limit flow for the HEC-RAS modeling of potential ice jams in the F-M metropolitan area. At flows approaching the 15 Kcfs level, the ice volume remaining in the system would be unlikely to cause significant jams.

The existing conditions 2006 flood hydrograph was used as the future conditions through the cities flow for the 100 yr flood event. In this instance the predicted maximum flow through town will be on the order of 11-12 Kcfs, very similar to 2006 conditions.

Two ice sets of ice conditions were simulated for 10,000 cfs through the protected area using HEC-RAS. The first case assumed an initial 1.0 ft-thick ice cover with a Mannings n of 0.012 at all locations along the mainstem RRN the inlet and outlet of the diversion canal. It was assumed that during the rise to 10,000 cfs the ice cover has thinned from its initial 2.5 ft thickness down to about 1.0 ft. The HEC-RAS model run indicates stages through town would be about 1.0 ft higher at 10,000 cfs with this ice cover. Ice cover would deteriorate more with higher flows through town. With 15,000 cfs allowed through the protected area, ice should be essentially gone.

The second case assumes a minor ice jam between the 12 Avenue Bridge and the upstream railroad bridge for the 10,000 cfs flow-through-town condition. Open water conditions are assumed to exist from there up to the Wild Rice Confluence. Maximum upstream stage impacts are about 1.0 ft. A jam is not expected when the flow through town is 15,000 cfs or greater.

2.6.2. Comparison of Ice Jam Potential in the F-M Metropolitan Area with and without-Diversion

For the 100 year event with the diversion project operating RRN flow through the cities will reach a maximum of about 10 Kcfs. Review of recent flood events indicates that by the time the discharge at the Fargo Gage reaches 10 Kcfs, much of the ice in this section of river has melted and open water conditions prevail. Based on this analysis of historical data, it is expected that the potential for ice jamming with and without project will be quite similar. One would still expect minor ice jams at the 12th Ave. Bridge as they occur under existing conditions. At through-town flows associated with greater than 100 year events (up to 25 Kcfs) one would expect open water conditions to prevail and ice to be no factor.

3. Response to ATR Comments and Questions

3.1. What happens to flows in the diversion channel and protected area if we have a large snowmelt flood event, augmented by rainfall, followed by extreme cold (like in 2009), wherein more ice is produced in the diversion channel and protected area? If the flood peak is already in the system, how much might newly formed ice impact the project stages?

It is unlikely that sufficient ice would be formed on the canal to cause any problems. Taking 2009 as an example, the AFDD curve increased by about 50 during the last week in March. Assuming quiescent and water close to the freezing water temperatures (≤33 °F), this would result in a maximum of about 3-4 inches of ice growth. With the high discharges and water velocities (on the order of 2-3 ft/s) one would not expect much ice growth on the canals or the natural channels during a week-long period with daily average air temperatures on the order of 20°F or above.

3.2. How will an earlier peak downstream of the project affect ice and damages caused by ice? The unsteady RAS modeling shows the peak being 1 to 2 days earlier with the project.

An earlier peak downstream of the project may accelerate the melting and release of any remaining ice in the section of the RRN downstream of the canal outlet. The lower with-diversion flows in the bypassed reach of the RRN plus a possible backwater effect from canal outflow will reduce water velocities in the F-M section of the RRN. This may prolong the ice-out period in the F-M section. It is unlikely that these changes to the flow regime in the vicinity of F-M will increases the tendency for ice jamming since water velocities will be lower.

3.3. Is it valid to use the Manning Equation for flow in the aqueduct with an ice cover; since the hydraulic radius is changed and the ice occupies a portion of the cross sectional flow area?

Yes, as long as flow is uniform (water surface slope = channel bed slope. etc) and the channel is wide enough that the ice cover can be considered floating (usually the case).

The hydraulic radius (flow area / wetted perimeter) needs to be calculated for the ice cover case (about half the under ice depth for a wide rectangular channel).

A composite Manning n_c that factors flow resistance for both the bed and the ice underside should be used in the Manning Equation. n_c can be calculated from Eq. 12 in White (1999).

http://www.crrel.usace.army.mil/library/crrelreports/CR99_11.pdf

The ice routines within HEC-RAS are useful for this type of calculation particularly where flow conditions are gradually-varied rather than uniform.

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Table 3. Hydro-meteorological data for the Red River of the North at Fargo, ND, 1958-2011. Recent flood years are highlighted.

Year	Day of Peak Stage	Peak Stage (ft)	Peak Discharge (cfs)	AFDD Day of Peak	AFDD max (F)	Day of AFDD max	Max Ice Thickness (in)	AFDD max to Peak Stage (days)	Δ AFDD Max to Peak	Day of Ice Out at Fargo	Ice-Out to Peak Stage (days)
2011	9-Apr	38.74	25700	2450	2600	1-Apr	30	8	-150	16-Mar	24
2010	21-Mar	36.99	21200	2027	2103	12-Mar	27	9	-76	19-Mar	2
2009	28-Mar	40.65	29500	1634	1650	5-Mar	24	23	-16	24-Mar	4
2008	23-Mar	16.2	967	2376	2400	14-Mar	29	9	-24	13-Apr	-21
2007	27-Mar	22.09	6484	2387	2387	28-Mar	29	-1	0	26-Mar	1
2006	5-Apr	36.82	19400	1444	1520	17-Mar	23	19	-76	1-Apr	4
2005	1-Apr	18.52	3908	1350	1280	25-Mar	21	7	70	3-Apr	-2
2004	26-Mar	16.18	1379	1900	1995	25-Mar	26	1	-95	1-Apr	-6
2003	27-Mar	15.76	1708	2000	2150	12-Mar	27	15	-150	27-Mar	0
2002	1-Apr	15.88	1793	1373	1450	8-Apr	22	-7	-77	23-Mar	9
2001	14-Apr	36.65	20200	2627	2750	1-Apr	31	13	-123	10-Mar	35
2000	11-Mar		3484	1151	1250	17-Mar	21	-6	-99	11-Mar	0
1999	22-Mar	20.81	4900	1150	1256	27-Feb	21	23	-106	27-Mar	-5
1998	2-Mar		8029	1240	1450	14-Mar	22	-12	-210	16-Mar	-14
1997	17-Apr	39.57	27630	2968	3000	16-Mar	32	32	-32	2-Apr	15
1996	15-Apr	28.75	9,940	2967	2988	12-Apr	32	3	-21		
1995	22-Mar		5484	2769	2987	12-Apr	32	-21	-218	7-Apr	-16
1994	22-Mar		10466	2470	2500	27-Mar	30	-5	-30	1-Apr	-10
1993	5-Apr	28.27	10100	2259	2400	21-Mar	29	15	-141	1-Apr	4
1992	11-Mar		1697	1297	1344	23-Mar	22	-12	-47	11-Mar	0
1991	3-Apr		1092	1672	1750	26-Feb	25	36	-78	3-Apr	0

1990	21-Mar		602	1609	1750	15-Mar	25	6	-141	1-Apr	-11
1989	9-Apr	35.39	18900	2452	2500	27-Mar	30	13	-48		
1988	11-Mar	15.1	981	1833	1950	25-Mar	26	-14	-117		
1987	10-Mar		1930	1155	1220	21-Mar	21	-11	-65		
1986	3-Apr	27.19	18600	2452	2620	13-Mar	30	21	-168		
1985	23-Mar		2380	2062	2200	20-Mar	28	3	-138		
1984	1-Apr	28.27	9450	2261	2151	9-Mar	27	23	110		
1983	8-Mar		765	1279	1450	20-Mar	22	-12	-171		
1982	4-Apr	25.07	5800	2893	2900	23-Mar	32	12	-7		
1981	25-Mar		350	1444	1550	8-Apr	23	-14	-106		
1980	5-Apr	20.74	4730	2139	2250	6-Mar	28	30	-111		
1979	19-Apr	34.93	17200	3357	3500	24-Mar	35	26	-143		
1978	2-Apr	34.41	17000	3021	3150	10-Apr	33	-8	-129		
1977	31-Mar		329	2363	2510	18-Mar	30	13	-147		
1976	30-Mar	18.7	3000	1816	1900	7-Mar	26	23	-84		
1975	21-Apr		8380	1922	1867	21-Mar	25	31	55		
1974	14-Apr	20.25	4040	2460	2510	5-Apr	30	9	-50		
1973	15-Mar	16.14	1830	1962	1950	7-Apr	26	-23	12		
1972	25-Mar	25.3	7080	2442	2510	28-Feb	30	25	-68		
1971	21-Mar		1430	2293	2350	13-Mar	29	8	-57		
1970	12-Apr		1770	2329	2301	16-Mar	28	27	28		
1969	15-Apr	37.34	23900	2457	2418	3-Apr	29	12	39		
1968	19-Mar		510	1790	1800	2-Apr	25	-14	-10		
1967	31-Mar		4230	2280	2450	14-Mar	29	17	-170		
1966	22-Mar	30.16	10600	2383	2402	21-Mar	29	1	-19		
1965	15-Apr	30.5	11300	2029	2180	8-Mar	28	38	-151		
1964	19-Apr	16.22	2330	1575	1800	5-Apr	25	14	-225		
1963	1-Apr		1290	1926	1808	31-Mar	25	1	118		

1962	19-Apr		6400	2402	2500	23-Mar	30	27	-98		
1961	8-Mar		508	1800	1800	15-Apr	25	-38	0		
1960	8-Apr	12.48	3700	2182	2250	27-Feb	28	40	-68		
1959	7-Apr		696	2103	2220	24-Mar	28	14	-117		
1958	11-Mar		654	1349	1450	21-Mar	22	-101			
Averages	29-Mar	26.3	7550	2062	2137	21-Mar	27	6	-74	26-Mar	1

Year of Flood	Day of Peak Stage	Peak Stage (ft)	Peak Discharge (cfs)	AFDD Day of Peak Discharge	AFDD max (F)	Day of AFDD max	Max Ice Thickness (in)	AFDD max to Peak Stage (days)	Δ AFDD Max to Peak	Day of Ice Out at Fargo	Ice-Out to Peak Stage (days)
2009	28-Mar	40.65	29,500	1634	1650	5-Mar	24	23	-16	24-Mar	4
1997	17-Apr	39.57	27,630	2968	3000	16-Mar	32	32	-32	2-Apr	15
2011	9-Apr	38.74	25,700	2450	2600	1-Apr	30	8	-150	16-Mar	24
1969	15-Apr	37.34	23,900	2457	2418	3-Apr	29	12	39		
2010	21-Mar	36.99	21,200	2027	2103	12-Mar	27	9	-76	19-Mar	2
2006	5-Apr	36.82	19,400	1444	1520	17-Mar	23	19	-76	1-Apr	4
2001	14-Apr	36.65	20,200	2627	2750	1-Apr	31	13	-123	10-Mar	35
Averages: Flood Yrs.	6-Apr	38.11	23,933	2230	2292	21-Mar	28	17	-62	22-Mar	14
1958-2011	29-Mar	26.25	7,550	2,062	2,137	21-Mar	27	6	-74	26-Mar	1

Table 8: Peak Stage, Maximum AFDD, Ice Thickness (h), and Differences for Period of Record

	Peak Stage				**Maximum AFDD for Winter**				**Maximum Ice Thickness**	**$AFDD_{max}$ to Peak Stage**	
Water Year	Day	Stage (ft)	Q (cfs)	AFDD °F-day	Day of $AFDD_{max}$ (°F)	$AFDD_{max}$ (°F)	h (AFDD) (in)	h (unified AFDD) (in)	h_{max} (unified AFDD) (in)	(days)	Δ AFDD (°F)
2010	21-Mar	36.95	21100	1924	5-Mar	1981	26	27	27	16	-57
2009	28-Mar	40.65	29100	2508	14-Mar	2533	30	31	31	14	-25
2008	23-Mar	15.26	968	2373	28-Mar	2388	29	30	30	-5	-15
2007	6-Apr	25.63	8770	1459	20-Mar	1570	23	24	24	17	-111
2006	5-Apr	37.04	19800	1444	25-Mar	1536	23	23	23	11	-92
2005	1-Apr	18.61	3990	1692	21-Mar	1782	25	26	26	11	-90
2004	26-Mar	16.18	1320	1910	22-Mar	1940	26	27	27	4	-30
2003	22-Apr	15.86	1780	1720	13-Mar	2151	27	28	28	40	-431
2002	1-Apr	15.93	1940	1375	5-Apr	1417	22	21	21	-4	-41
2001	14-Apr	36.65	20200	2638	27-Mar	2750	31	31	31	18	-112
2000	11-Mar	*	3580	1156	22-Feb	1257	21	22	22	17	-100
1999	22-Mar	20.81	4800	1539	14-Mar	1581	23	24		8	-43
1998	2-Mar	*	8210	1188	16-Mar	1410	22	23	25	-14	-223
1997	17-Apr	39.57	27800	2968	12-Apr	2988	32	33	33	5	-20
1996	17-Apr	39.54	9340	2843	7-Apr	2922	32	33	33	10	-79
1995	22-Mar	*	10500	1664	11-Mar	1773	25	22	22	11	-108
1994	22-Mar	*	3000	2464	17-Mar	2507	30	30	30	5	-43
1993	5-Apr	28.27	9900	2260	23-Mar	2344	29	27	27	13	-84
1992	11-Mar	*	1760	1297	25-Feb	1343	22	21	21	14	-46
1991	3-Apr	*	1250	1662	9-Mar	1807	25	25	25	25	-144
1990	21-Mar	*	610	1608	6-Mar	1660	24	23	23	15	-52

Peak Stage	**Peak Stage**				**Maximum AFDD for Winter**				**Maximum Ice Thickness**	**$AFDD_{max}$ to Peak Stage**	
Water Year	Day	Stage (ft)	Q (cfs)	AFDD °F-day	Day of $AFDD_{max}$ (°F)	$AFDD_{max}$ (°F)	h (AFDD) (in)	h (unified AFDD) (in)	h_{max} (unified AFDD) (in)	(days)	Δ AFDD (°F)
1989	9-Apr	35.39	18600	2453	24-Mar	2489	29	30	30	16	-35
1988	11-Mar	15.1	924	1829	21-Mar	1925	26	26	26	-10	-97
1987	10-Mar	*	1930	1155	13-Mar	1180	20	20	20	-3	-25
1986	3-Apr	27.19	8420	2453	20-Mar	2619	30	31	31	14	-166
1985	23-Mar	*	2380	2069	8-Mar	2153	27	28	28	15	-84
1984	1-Apr	28.27	9450	2261	20-Mar	2320	28	29	29	12	-59
1983	8-Mar	*	765	1279	23-Mar	1381	22	22	22	-15	-102
1982	4-Apr	25.07	5800	2879	8-Apr	2909	32	33	33	-4	-30
1981	25-Mar	*	350	1444	10-Mar	1545	23	23	23	15	-100
1980	5-Apr	20.74	4730	2161	25-Mar	2231	28	28	28	11	-70
1979	19-Apr	34.93	17200	3357	9-Apr	3448	35	36	36	10	-91
1978	2-Apr	34.41	16800	3022	25-Mar	3068	33	33	33	8	-45
1977	31-Mar	*	329	2364	7-Mar	2499	29	30	30	24	-135
1976	30-Mar	18.7	3000	1817	21-Mar	1895	26	20	26	9	-78
1975	21-Apr	*	8380	1923	5-Apr	2019	27	27	27	16	-96
1974	14-Apr	20.25	4040	2460	4-Apr	2526	30	30	30	10	-66
1973	15-Mar	16.14	1830	1889	28-Feb	1948	26	26	26	15	-59
1972	25-Mar	25.3	6420	2442	13-Mar	2501	30	30	30	12	-59
1971	21-Mar	*	1430	2293	24-Mar	2330	28	29	29	-3	-37
1970	12-Apr	*	1770	2330	3-Apr	2418	29	29	29	9	-88

	Peak Stage				Maximum AFDD for Winter				Maximum Ice Thickness	$AFDD_{max}$ to Peak Stage	
Water Year	Day	Stage (ft)	Q (cfs)	AFDD °F-day	Day of $AFDD_{max}$ (°F)	$AFDD_{max}$ (°F)	h (AFDD) (in)	h (unified AFDD) (in)	h_{max} (unified AFDD) (in)	(days)	Δ AFDD (°F)
1969	15-Apr	37.34	23900	2458	2-Apr	2626	30	31	31	13	-169
1968	19-Mar	*	510	1790	3-Mar	1856	25	26	26	16	-66
1967	31-Mar	*	4230	2280	21-Mar	2402	29	29		10	-122
1966	22-Mar	30.16	10600	2383	8-Mar	2450	29	30	30	14	-67
1965	15-Apr	30.5	11300	3112	31-Mar	3210	33	34	34	15	-98
1964	19-Apr	16.22	2280	1564	31-Mar	1809	25	25	25	19	-245
1963	1-Apr	*	1290	1927	21-Mar	2078	27	27	27	11	-151
1962	19-Apr	*	6400	2402	24-Mar	2468	29	30	30	26	-66
1961	8-Mar	*	508	1795	27-Feb	1814	25	25	25	9	-19
1960	8-Apr	12.48	3700	2183	25-Mar	2220	28	28	32	14	-37
1959	7-Apr	*	696	2106	21-Mar	2303	28	29	29	17	-197
1958	11-Mar	*	654	1342	21-Mar	1413	22	22	22	-10	-71
1957	27-Mar	*	1580	2055	19-Mar	2126	27	28	28	8	-72
1956	16-Apr	12.54	3810	2910	31-Mar	2968	32	33	33	16	-58
1955	4-Apr	11.12	2600	2094	28-Mar	2203	28	28	28	7	-109
1954	28-Mar	*	1060	1812	3-Apr	1907	26	26	26	-6	-95
1953	26-Mar	*	1790	1569	16-Mar	1613	24	24	24	10	-44
1952	16-Apr	28.79	16200	2608	28-Mar	2711	31	31	31	19	-103
1951	11-Apr	20.73	7990	2812	24-Mar	2897	32	32	32	18	-85
1950	7-Apr	20.88	7680	2851	13-Apr	2894	32	32	32	-6	-43
1949	4-Apr	*	1780	2559	23-Mar	2605	30	31	33	12	-46

** when peak stage could not be identified from USGS records, the date of peak stage was assumed to occur on the same day as the peak flow*

Table 9: Ice-out Dates by Several Methods: Water Temperature Rise, Fargo Gage Observed as Clear, and Unified AFDD Method

	Water Temp Rises Above Freezing		**Fargo Gage observed as clear of ice**		**Unified AFDD method, ti=0**		**Water Temp Rise to Peak Stage**		**Unified AFDD Method to Peak Stage**	
Water Year	Day	AFDD ºF-day	Day	AFDD ºF-day	Day	ºF-day	Days	Δ AFDD ºF-day	Days	Δ AFDD ºF-day
2010	19-Mar	1925.2	19-Mar	1925	19-Mar	1925	2	55	2	55
2009	24-Mar	2468.8	24-Mar	2484	24-Mar	2469	4	64	4	64
2008	26-Feb	2095.9	23-Apr	2131	8-Apr	2316	25	292	-16	72
2007	27-Mar	1461.8	28-Mar	1447	30-Mar	1426	10	108	7	145
2006	4-Apr	1464.8	2-Apr	1479	4-Apr	1465	1	72	1	72
2005	2-Apr	1680	15-Apr	1395	31-Mar	1702	-1	102	1	80
2004	30-Mar	1875.6	1-Apr	1859	1-Apr	1859	-4	65	-6	81
2003	4-Apr	1969	7-Apr	1969	20-Mar	2080	18	182	33	71
2002	11-Apr	1376.1	27-Mar	1391	14-Apr	1308	-10	40	-13	109
2001	3-Apr	2734.6	3-Apr	2735	10-Apr	2682	11	16	4	68
2000	9-Mar	1131.6	15-Mar	1180	3-Mar	1190	2	125	8	67
1999	25-Mar	1538.7	28-Mar	1504	24-Mar	1539	-3	43	-2	43
1998			25-Mar	1389	31-Mar	1326			-29	85
1997	11-Apr	2985.8	3-Apr	2915	25-Apr	2827	6	2	-8	161
1996					18-Apr	2823			-1	99
1995			2-Feb	1621	17-Mar	1708			5	65
1994					28-Mar	2489			-6	18
1993			5-Apr	2260	3-Apr	2273			2	72
1992			18-Mar	1317	6-Mar	1280			5	63
1991			2-Apr	1687	19-Mar	1770			15	37
1990			28-Mar	1652	17-Mar	1591			4	69

	Water Temp rises above freezing		Fargo Gage observed as clear of ice		Unified AFDD method, ti=0		Water Temp Rise to Peak Stage		Unified AFDD Method to Peak Stage	
Water Year	Day	AFDD ºF-day	Day	AFDD ºF-day	Day	ºF-day	Days	Δ AFDD ºF-day	Days	Δ AFDD ºF-day
1989			5-Apr	2451	5-Apr	2451			4	37
1988			7-Apr	1790	30-Mar	1899			-19	27
1987			31-Mar	1125	24-Mar	1101			-14	80
1986			2-Apr	2467	1-Apr	2478			2	140
1985			3-Apr	2008	21-Mar	2094			2	59
1984			30-Mar	2273	3-Apr	2241			-2	79
1983					8-Apr	1333			-31	48
1982					21-Apr	2769			-17	140
1981					19-Mar	1497			6	47
1980					8-Apr	2114			-3	117
1979					23-Apr	3295			-4	153
1978					5-Apr	3011			-3	57
1977					17-Mar	2429			14	70
1976					28-Mar	1837			2	58
1975					16-Apr	1963			5	56
1974					19-Apr	2396			-5	130
1973					19-Mar	1891			-4	57
1972					23-Mar	2443			2	58
1971					5-Apr	2303			-15	27
1970					15-Apr	2306			-3	113
1969					14-Apr	2481			1	146
1968					14-Mar	1851			5	5

	Water Temp rises above freezing		Fargo Gage observed as clear of ice		Unified AFDD method, ti=0		Water Temp Rise to Peak Stage		Unified AFDD Method to Peak Stage	
Water Year	Day	AFDD ºF-day	Day	AFDD ºF-day	Day	ºF-day	Days	Δ AFDD ºF-day	Days	Δ AFDD ºF-day
1967					1-Apr	2286			-1	116
1966					21-Mar	2387			1	63
1965					19-Apr	3084			-4	126
1964					9-Apr	1756			10	53
1963					30-Mar	1977			2	101
1962					5-Apr	2449			14	20
1961					10-Mar	1799			-2	14
1960					14-Apr	2113			-6	107
1959					31-Mar	2213			7	90
1958					30-Mar	1339			-19	74
1957					27-Mar	2055			0	72
1956					13-Apr	2924			3	44
1955					3-Apr	2109			1	95
1954					13-Apr	1765			-16	141
1953					26-Mar	1569			0	44
1952					11-Apr	2674			5	37
1951					8-Apr	2829			3	69
1950					25-Apr	2767			-18	128
1949					12-Apr	2451			-8	155

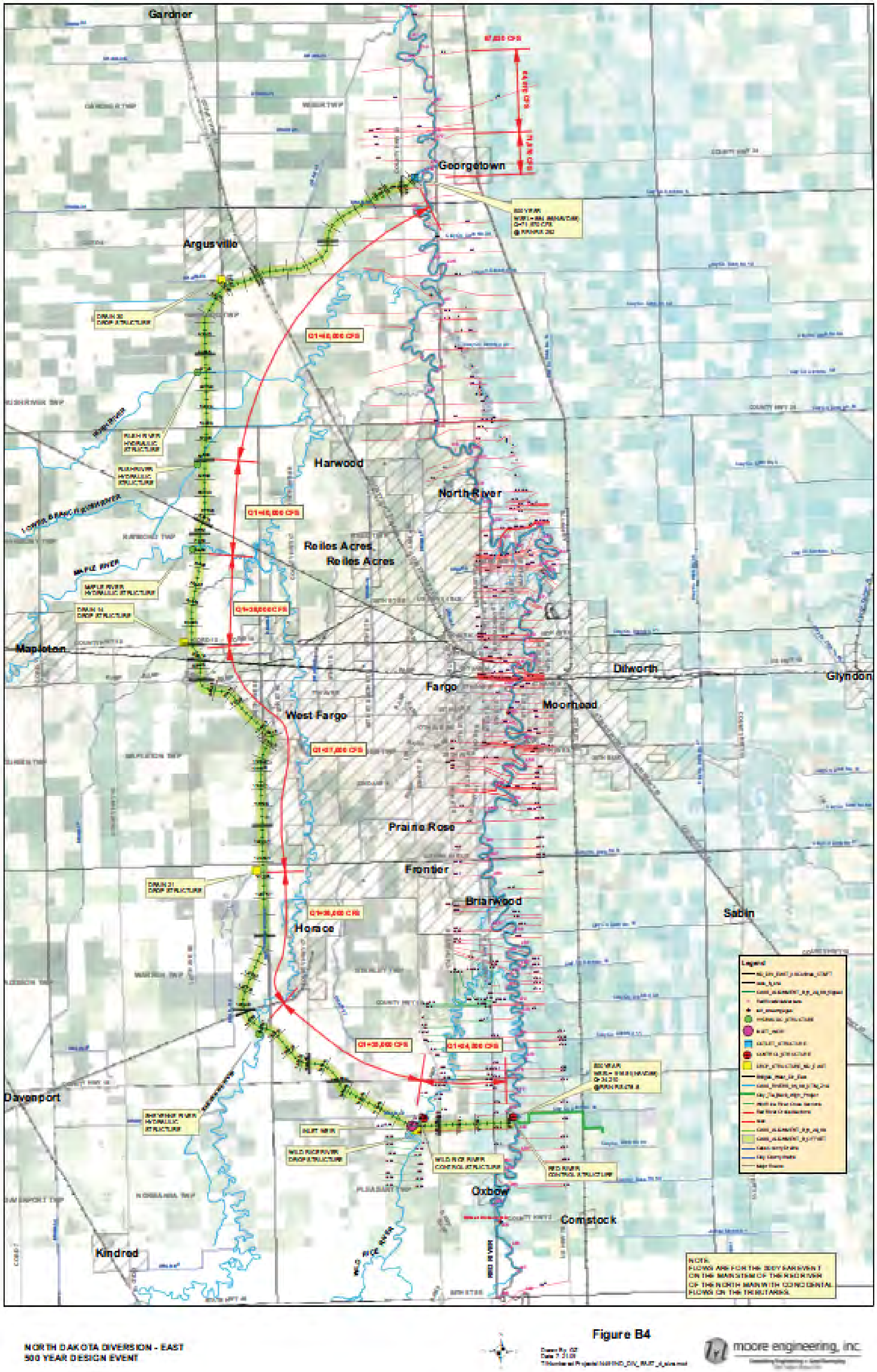

Fig. 1. Map of project area showing the North Dakota side diversion alternative.

Fig. 2. RRN water surface elevation at Hickson, Fargo and Halstad during the 2009 flood.

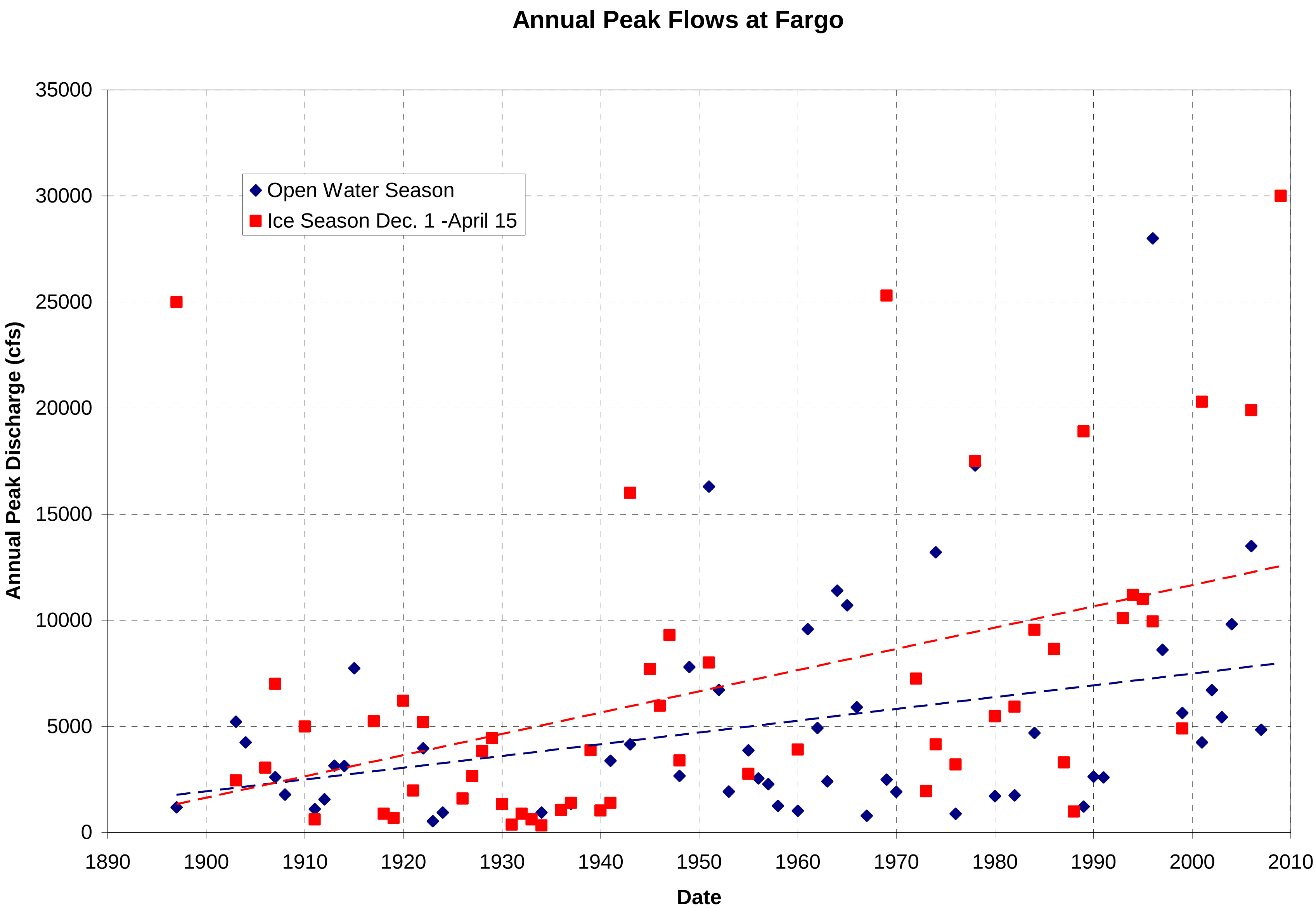

Fig. 3. Annual peak discharges at Fargo.

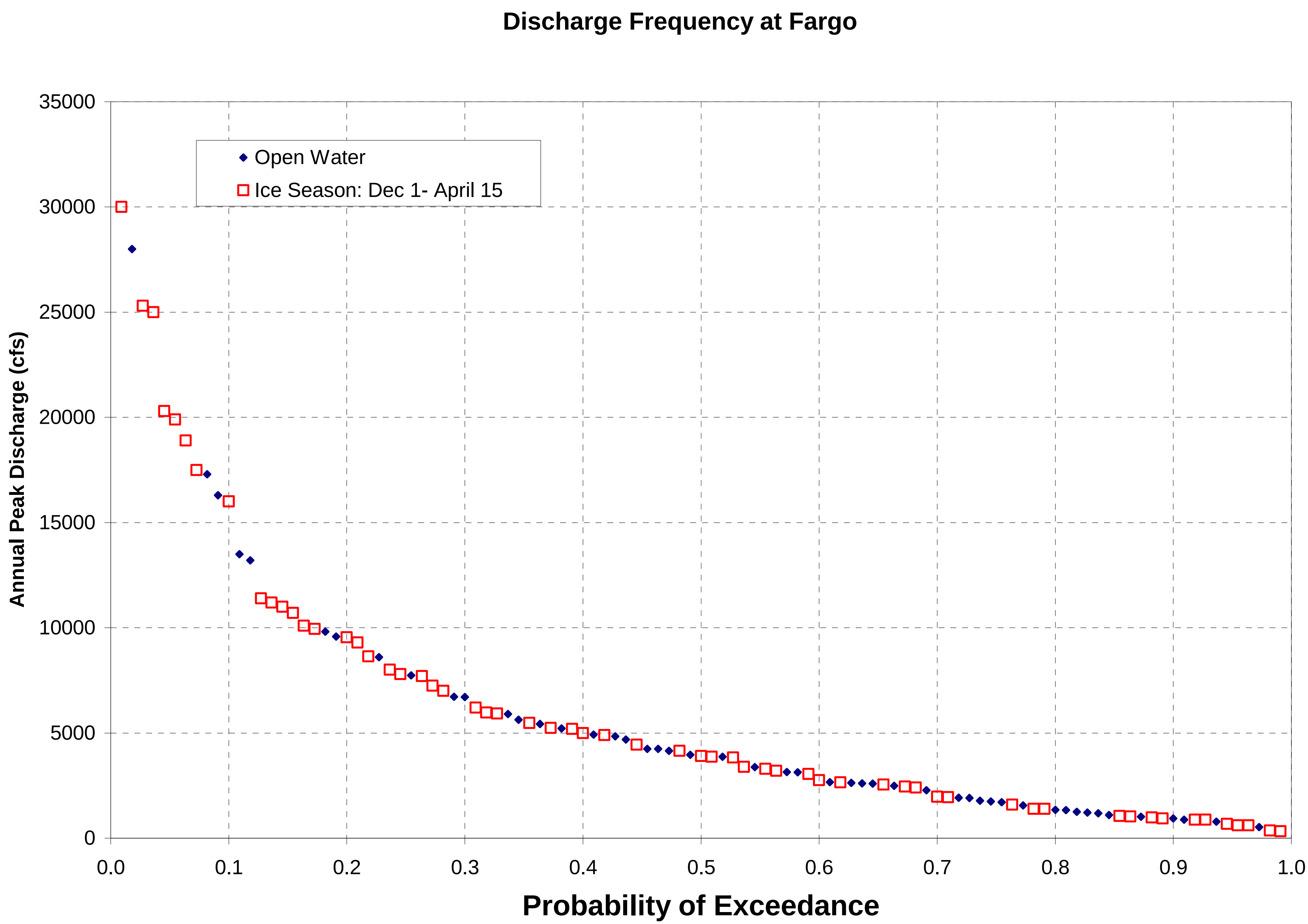

Figure 4. Probability distribution of peak discharges at Fargo.

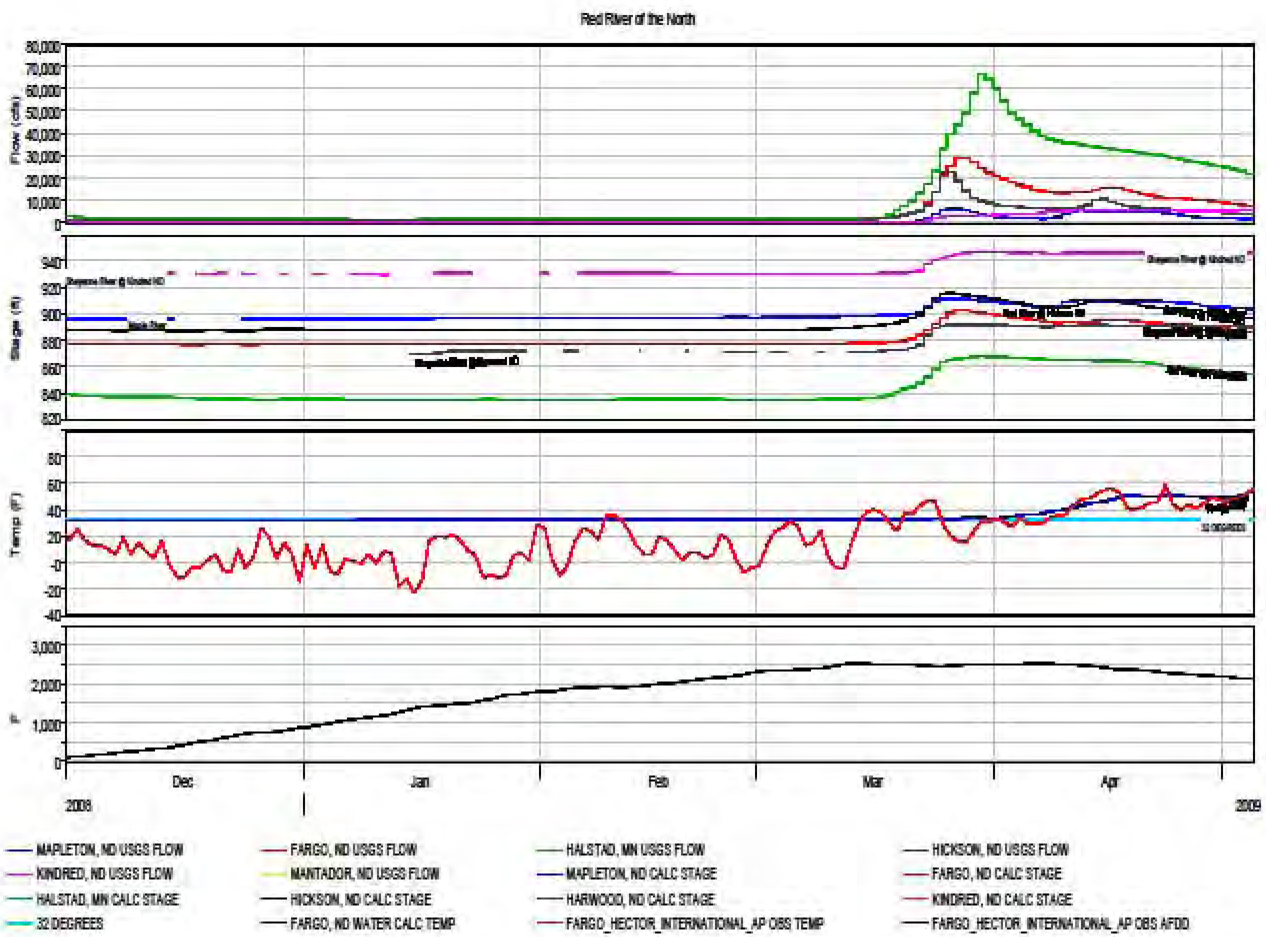

Figure 5. Daily average air temperature and AFDD, and discharge stage and stage for RRN and tributaries in the Fargo-Moorhead area, 2009 Flood

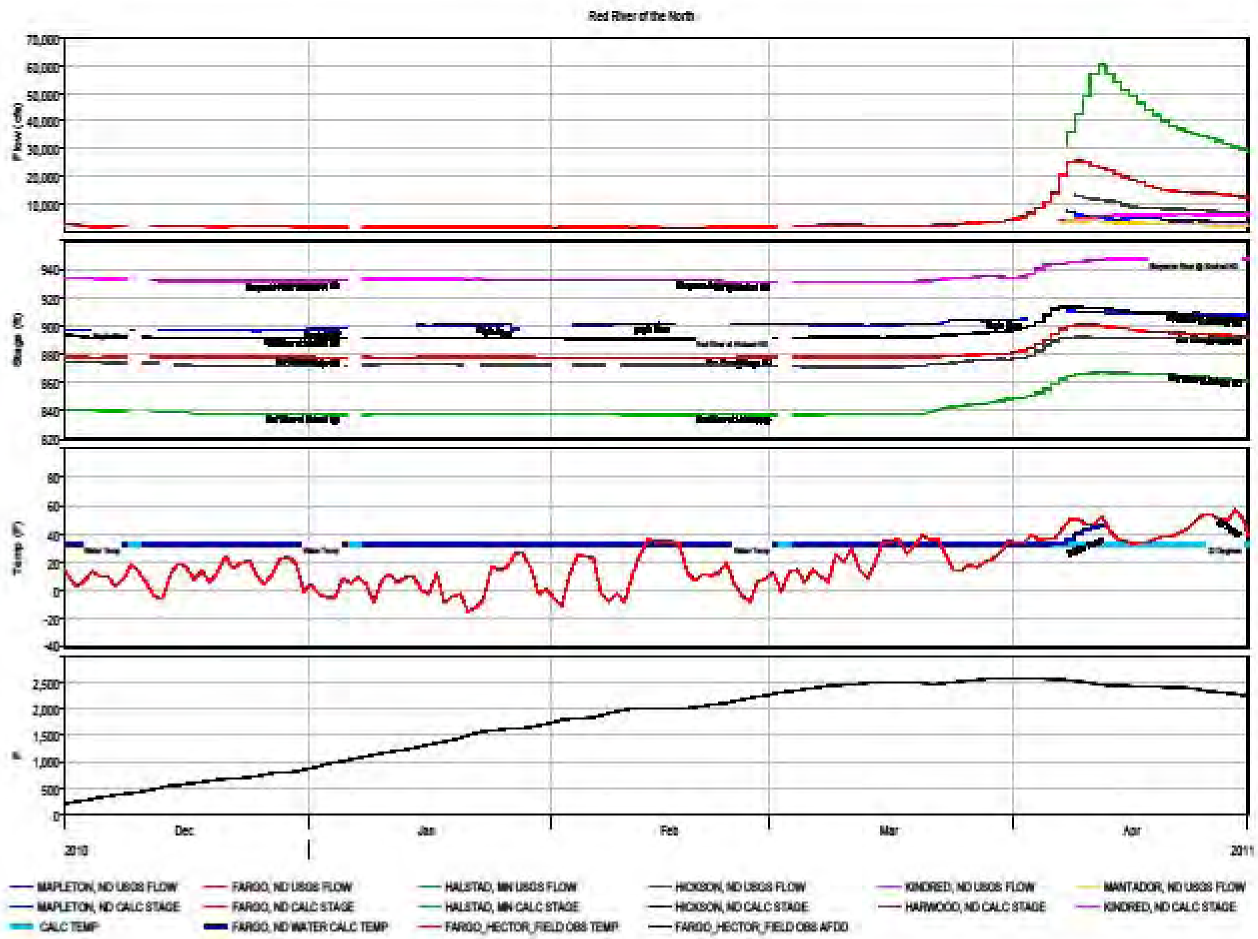

Figure 6. Daily average air temperature and AFDD, and discharge stage and stage for RRN and tributaries in the Fargo-Moorhead area, 2011 flood.

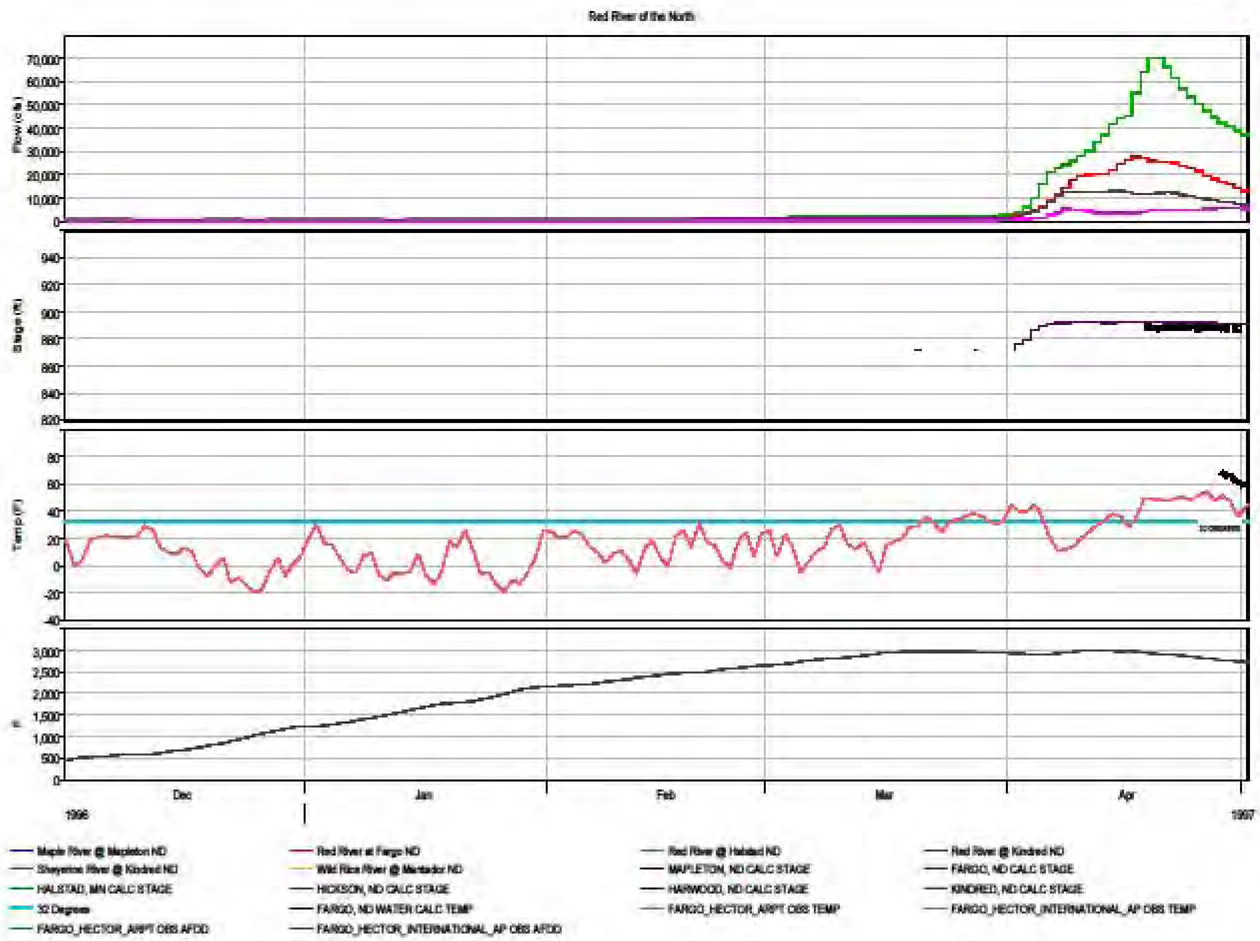

Figure 7. Daily average air temperature and AFDD, and discharge stage and stage for RRN and tributaries in the Fargo-Moorhead area; 1997 flood.

Appendix B

Images 1997, 2001, 2006, 2009, and 2010 Floods
Red River of the North, Fargo-Moorhead Area

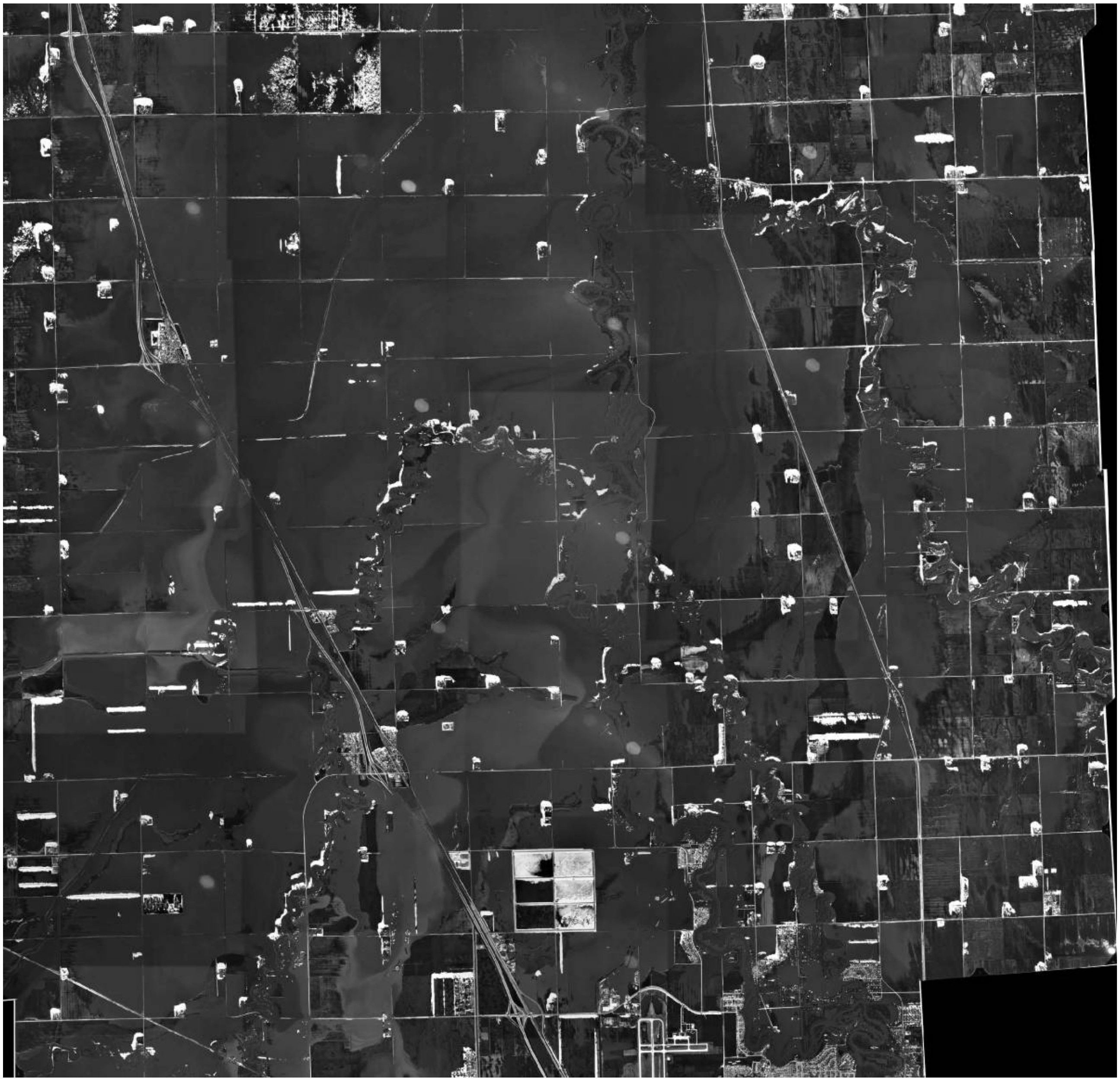

Figure B1 Aerial view north of Fargo during April 1997 flood.

Figure B2. Aerial view of Fargo and surrounds during April 1997 flood.

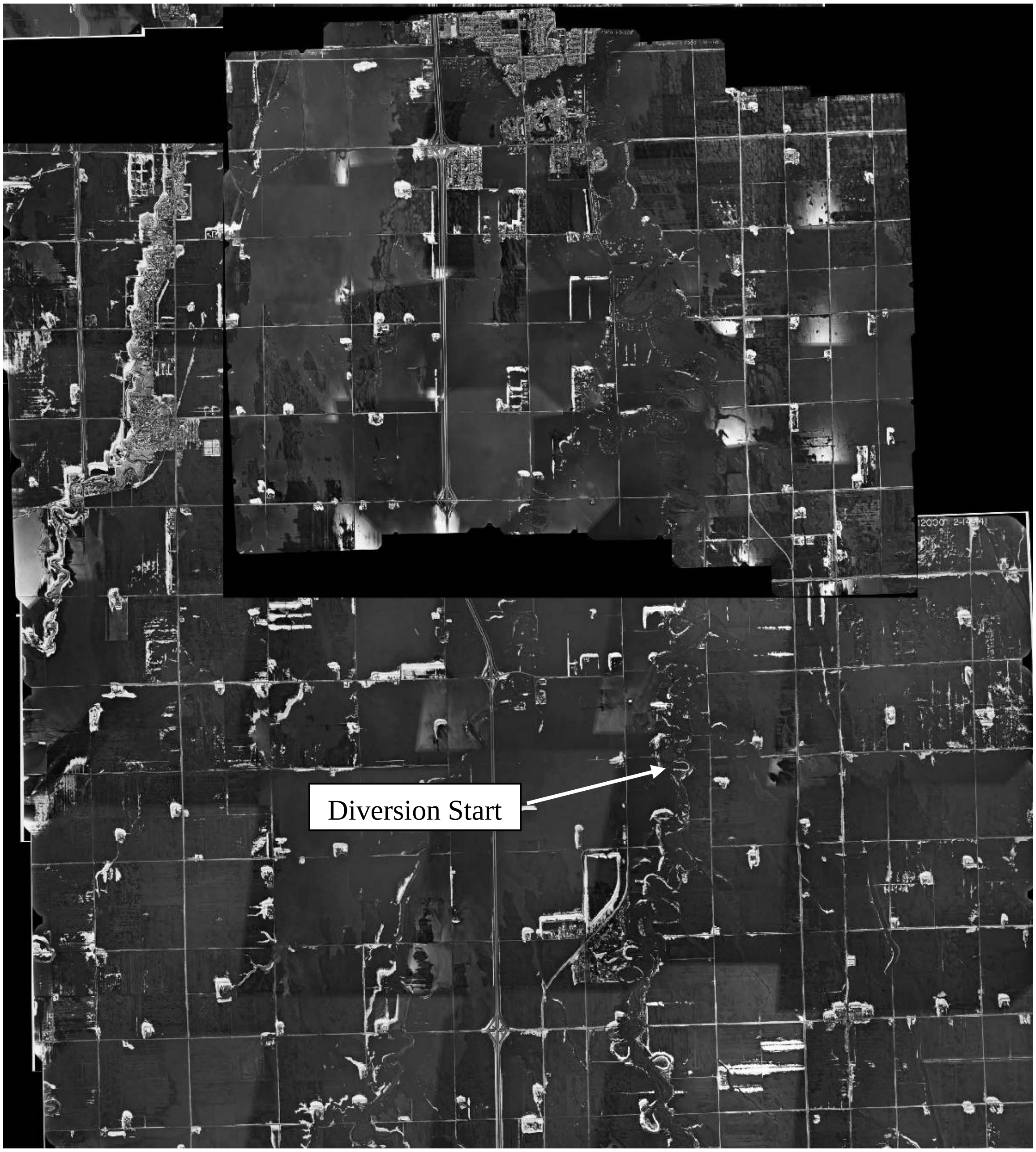

Fig. B3. Aerial view south of Fargo during April 1997 flood.

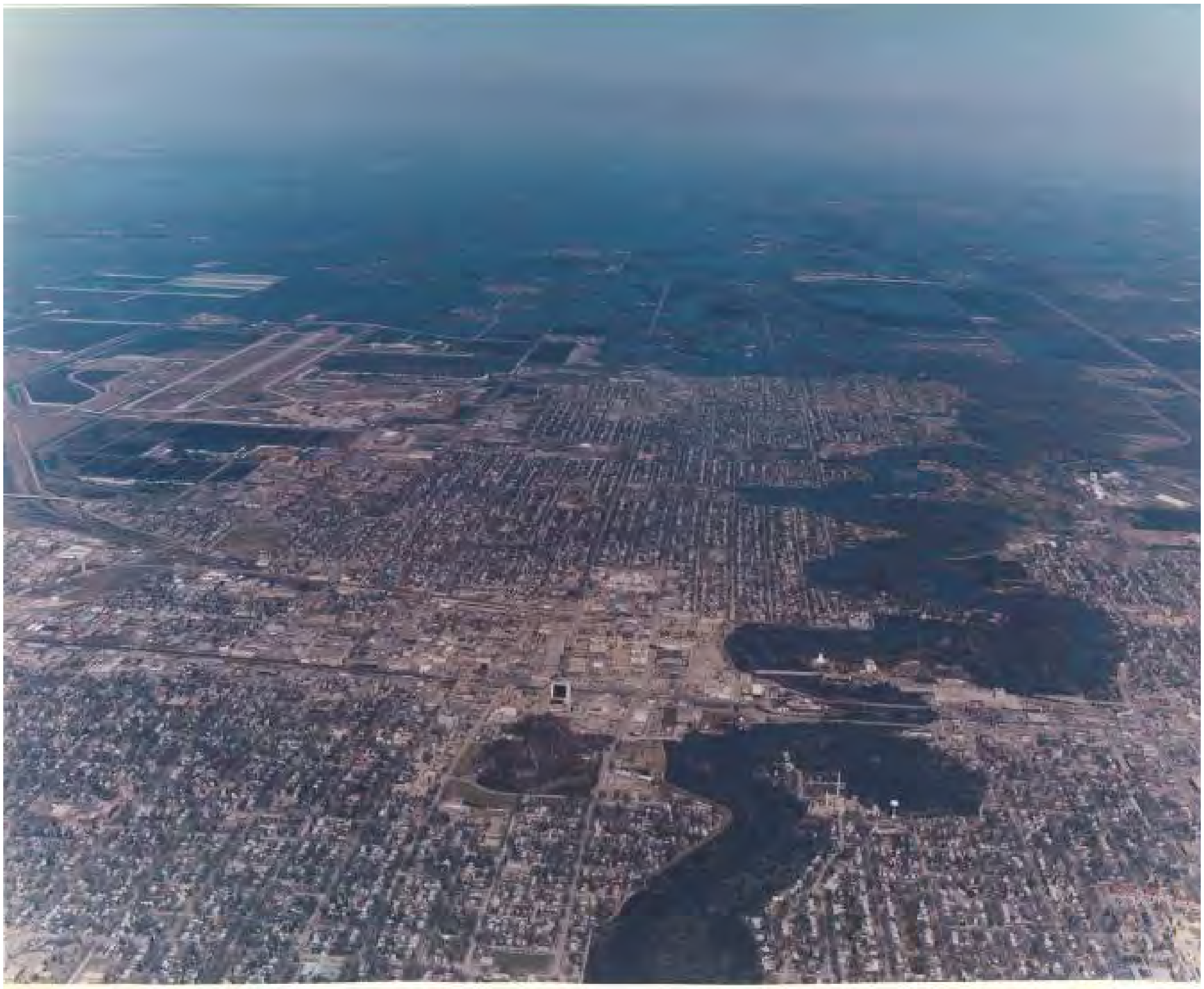

Figure B4. View of Fargo during the 1997 flood looking north. (Photo Courtesy of City of Fargo, ND)

Figure B5. View looking upstream of Fargo and Moorhead during the April 2001 flood (Photos Courtesy of North Dakota State University)

Figure B6. View looking west of Fargo and Moorhead during the April 2001 flood (Photos Courtesy of North Dakota State University)


Figure B7. View of 2006 RRN flood looking north toward F/M.


Figure A8. View of 2006 RRN flood looking north from Fargo.

Figure B9. Ice and debris jam at 12th Avenue N. Bridge in Fargo on 4/1/2006. RRN stage at Fargo was 27.5 ft. (Photo Courtesy of North Dakota State University)

Figure B10. Debris jam at flooded 12th Avenue N. Bridge in Fargo on 4/2/2006. Red River stage at Fargo was 31.9 ft. (Photo Courtesy of North Dakota State University)

Figure B11: View of flooded 12th Avenue N Bridge flooded on 4/4/2006. No ice visible. RRN stage at Fargo was 37.1 ft. (Photo Courtesy of North Dakota State University)

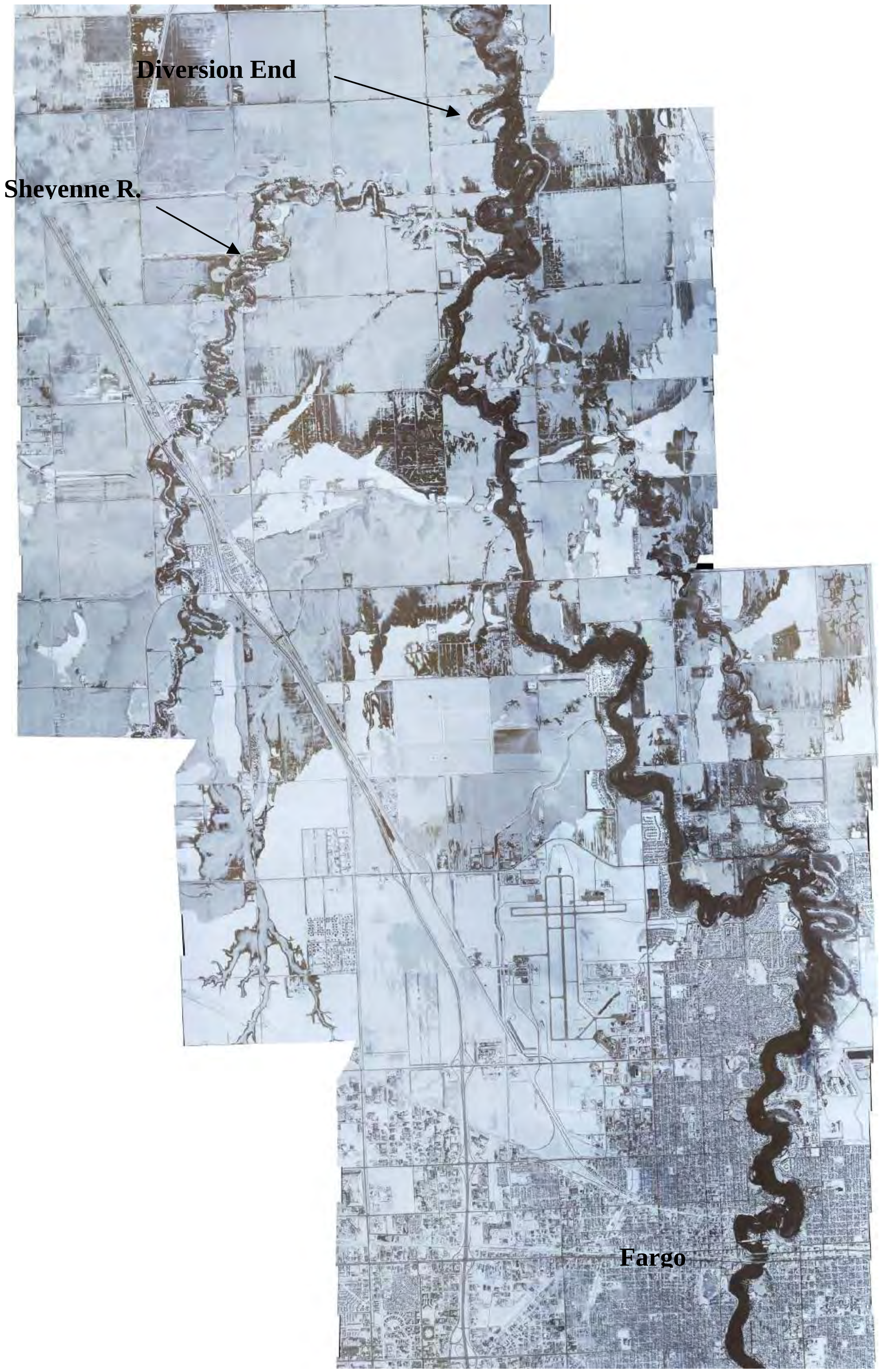

Figure BA12. Aerial view of the March 2009 flood. Fargo and north.

Figure B13. Aerial view of the March 2009 flood. Fargo and south.

Figure B14. March 2009. View looking south at confluence of RRN (left) and Wild Rice River (right). (Photo Courtesy City of Fargo, ND)

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Figure B15. March 2009. View looking of RRN and Fargo-Moorhead. (Photo Courtesy City of Fargo, ND)

Figure B16. Ice and debris jam at 12th Avenue N. Bridge in Fargo on 3/24/2009, RRN stage at Fargo: 32.4 ft. (Photo Courtesy of North Dakota State University).

Figure B17. Return to freezing temperatures with snowfall on 3/25/2009. View downstream from the Main Ave. Bridge in Moorhead. RRN stage at Fargo: 36.6 ft. (Photo Courtesy of North Dakota State University)

Figure B18. View of Center Ave. and BNSF Railroad Bridges across RRN toward Fargo on 3/25/2009. RRN stage at Fargo: 36.6 ft. (Photo Courtesy of North Dakota State University)

Figure B19. Temporary dike along RRN at Elm St. and 14th Ave. N. in Fargo on 3/26/2009. . RRN stage at Fargo): 39.6 ft (Photo Courtesy of North Dakota State University)

Figure B20: View southwest of RRN and BNSF Railroad and Main Ave. Bridges on 3/27/2009. RRN stage at Fargo close to peak at 40.7 ft. (Photo Courtesy of North Dakota State University)

Figure A21: Water rising across Elm St. N at El Zagal Golf Course on 3/13/2010. Red River stage at Fargo: 18.7 ft. (Photo Courtesy of North Dakota State University)

Figure B22. View of RRN looking downstream towards downtown Fargo on 3/13/2010. RRN stage at Fargo: 18.7 ft. (Photo Courtesy of North Dakota State University)

Figure B23. View of RRN looking downstream towards downtown Fargo on 3/14/2010. RRN stage at Fargo: 22.5 ft. (Photo Courtesy of North Dakota State University)

Figure B24. View of RRN looking downstream towards downtown Fargo on 3/15/2010. RRN stage at Fargo: 25.0 ft. (Photo Courtesy of North Dakota State University)

Figure B25. Ice passing the 1st Ave. Bridge in Moorhead on 3/20/2010. RRN stage at Fargo: 36.6 ft. (Photo Courtesy of North Dakota State University)

Figure B26. View of RRN in the vicinity of the start of the diversion looking north during March 2010 flood. (Photo Courtesy City of Fargo, ND)

Figure B27: View of RRN looking north towards F/M during March 2010 flood. (Photo Courtesy City of Fargo, ND)

Appendix D

Photo Log of Ice Conditions During Field Visits to RRN Fargo-Moorhead

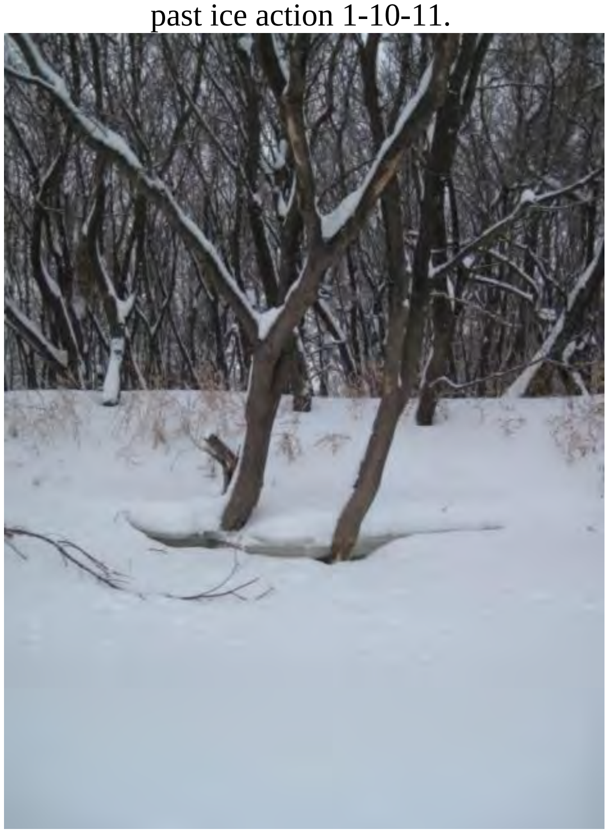
January 10-12, 2011
April 6-9, 1011

Appendix D Photo Index

D1. RRN near diversion. 12-19 in thick sheet ice cover 1-10-11.
D2. RRN near diversion. Minor ice tree scarring 1-10-11.
D3. RRN near diversion. Minor ice tree abrasion at 2009 peak flood levels.
D4. RRN looking downstream from CR 16 Bridge 1-10-11.
D5. Wild Rice River near diversion 1-10-11.
D6. Wild Rice River near diversion 1-10-11. Sheet ice cover 19-in-thick
D7. Sheyenne R. CR 16 upstream of crossing 1-11-11. Sheet ice 14-18-in-thick,
D8. Sheyenne R. CR 16 upstream of crossing 1-11-11.
D9. Maple R. near crossing. 1-1-11.
D10. Sheyenne River south of Harwood at 32nd Ave. 1-1-11.
D11. Sheyenne River near RRN confluence 1-1-11. Tiered banks, wider channel.
D12. RRN near proposed canal outlet 1-11-11. sheet ice18-21-in-thick
D13. RRN downstream of Constitution Br. F/M on 1-12-11. Open leads, faster flow.
D14. RRN from Veterans Br. F/M on 1-12-11, looking upstream towards gage.
D15. RRN near diversion 4-6-11 showing decayed sheet ice on river channel.
D16. RRN looking west at Wild Rice R confluence, showing decayed sheet ice 4-6-11.
D17. Small accumulation of floes on RRN at Gooseberry Mound Park in Moorhead 4-6-11.
D18. RRN Gooseberry Mound Parlkto F/M Bridges river nearly free of ice 4-7-11.
D19. Residual ice on RRN downstream of US Rt. 81 Bridge on 4-7-11.
D20. Remaining ice cover in bends near RRN-Wild Rice Confluence on 4-7-11.
D21. RRN in vicinity of proposed diversion, looking downstream on 4-7-11.
D22. RRN in vicinity of proposed diversion looking upstream on 4-7-11.
D23. Decayed sheet ice on RRN near diversion on 4-7-11
D24. Flooding along Wild Rice R. on 4-7-11 looking NE from proposed diversion to RRN.
D25. Sheyenne R. looking NW at CR 16 and proposed canal crossing 4-7-11.
D26. Residual ice on Sheyenne R. in vicinity of proposed canal 4-7-11.
D27. Sheyenne River diversion into W. Fargo FC system near Horace 4-7-11.
D28. Sheyenne River diversion into W. Fargo canal system at CR 17 near I-94 4-7-11.
D29. Looking west at remaining ice accumulations in W. Fargo diversion canal on 4-7-11.
D30. Minor ice accumulations at RR Bridge in W. Fargo diversion canal on 4-7-11.
D31. Canal branch reentering Sheyenne R in W. Fargo 4-7-11.
D32. Sheyenne River nearly ice-free south of Harwood at 32nd St. 4-7-11.
D33. Residual ice in Maple R. Canal crossing area 4-7-11.
D34. Accumulation of floes on Sheyenne R south of Harwood CR 17. 4-7-11.
D35. Remaining ice in vicinity of Rush R. Sheyenne confluence 4-7-11.
D36. Drifting ice on RRN near Georgetown 4-7-11.
D37. RRN in vicinity of outlet of proposed diversion canal 4-7-11.
D38. RRN ice-free at downtown Fargo-Moorhead on 4-7-11.
D39. Debris and residual ice upstream of CR 16 Br. over Wild Rice R. 4-7-11.
D40. Decayed sheet ice on Sheyenne R. upstream of CR 16 4-7-11.
D41. Remaining ice cover on Sheyenne R. downstream of CR 16 4-7-11.
D42. Ice accumulation on Sheyenne R. upstream of CR 14 4-7-11.
D43. Fast currents and shoved ice floes, Sheyenne R. downstream of CR 14 4-7-11.
D44. Melting sheet ice on Sheyenne R upstream of 52nd Ave. W. 4-7-11.

D45. Decayed sheet ice on Sheyenne R upstream CR 17 Br. 4-7-11.
D46. Sheyenne diversion into W. Fargo FC Project at CR 17 Br. 4-7-11.
D47. Accumulated floes upstream of 13th Ave Bridge W. Fargo Diversion 4-7-11.
D48. Residual sheet ice on Sheyenne R. upstream of 12th Ave. NW 4-7-11.
D49. Ice-free RRN looking upstream from Veterans Br. Moorhead, 4-7-11.
D50. Remnants of sheet ice cover on Wild Rice R. near confluence with RRN 4-8-11.
D51. RRN ice nearly melted on at proposed diversion location 4-8-11.
D52. View from proposed diversion westward towards Wild Rice R. 4-8-11.
D53. Accumulated floes at Sheyenne R.diversion CR 17 Br.4-8-11.
D54. Minor ice jam on Sheyenne R. upstream of upstream 17th Ave. SE 4-8-11.
D55. Decayed floes in bend along CR 8 4-8-11.
D56. Remaining ice at diversion of Sheyenne at CR 17 Br.4-8-11.

D1. RRN near diversion. 12-19 in thick sheet ice cover, tiered banks, no evidence of significant past ice action 1-10-11.



D2. RRN near diversion, minor ice tree scarring at base flow and intermediate flood levels. 1-10-11.

D3. RRN near diversion. Minor ice tree abrasion at 2009 peak flood levels.

D4. RRN looking downstream from CR 16 Bridge 1-10-11.

D5. Wild Rice River near diversion 1-10-11.

D6. Wild Rice River near diversion 1-10-11. Sheet ice cover 19-in-thick, treed natural levees, no evidence of significant ice action in the past.

D7. Sheyenne R. CR 16 upstream of crossing 1-11-11. Sheet ice 14-18-in-thick, treed natural levees. Little sign of significant ice movement in past.

D8. Sheyenne R. CR 16 upstream of crossing 1-11-11. Minor abrasion of tree bark at bankfull height.

D9. Maple R. near crossing. 1-1-11.

D10. Sheyenne River south of Harwood at 32nd Ave. 1-1-11. Confined channel, treed natural levees.


D11. Sheyenne River near RRN confluence 1-1-11. Tiered banks, wider channel.


D12. RRN near proposed canal outlet 1-11-11. sheet ice18-21-in-thick, tiered banks, wide channel.

D13. RRN downstream of Constitution Br. F/M on 1-12-11. Open leads, faster flow.

D14. RRN from Veterans Br. F/M on 1-12-11, looking upstream towards gage.

D15. RRN near diversion 4-6-11 showing decayed sheet ice on river channel.

D16. RRN looking west at Wild Rice R confluence, showing decayed sheet ice on river channel 4-6-11.

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D18. RRN Gooseberry Mound Parlkto F/M Bridges river nearly free of ice 4-7-11.

D19. Residual ice on RRN downstream of US Rt. 81 Bridge on 4-7-11.

D20. Remaining ice cover in bends near RRN-Wild Rice Confluence on 4-7-11.

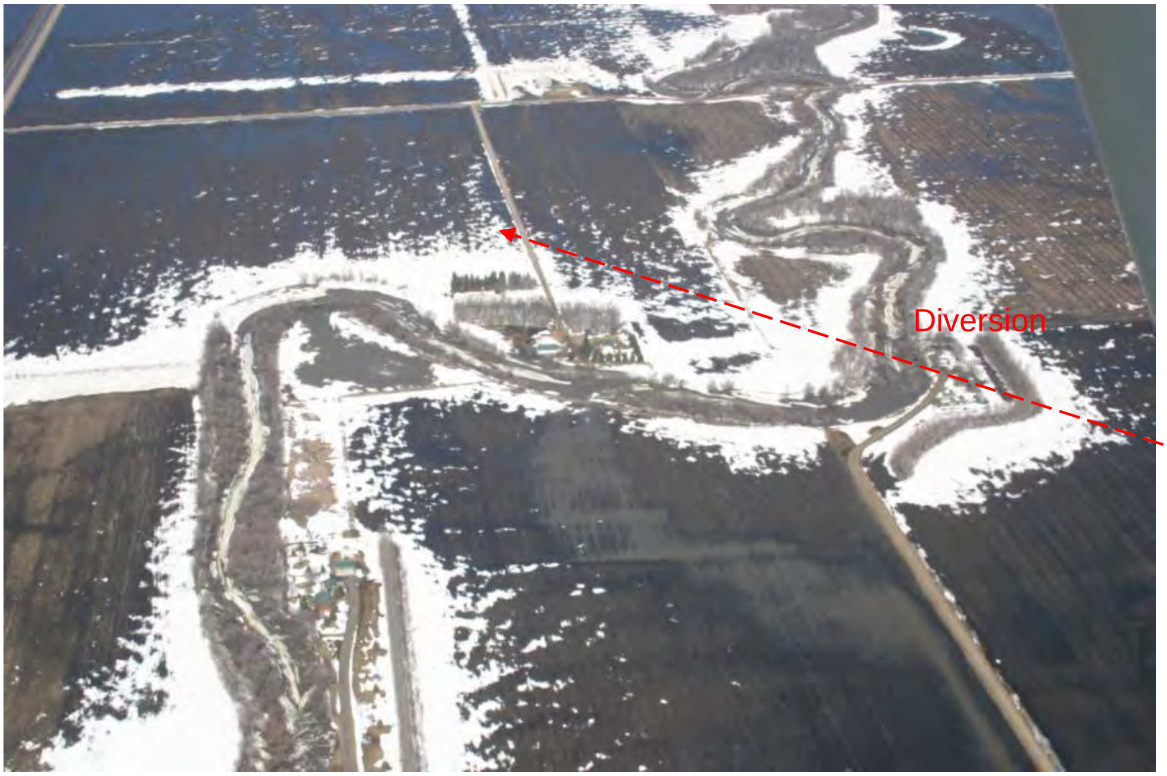
D21. RRN in vicinity of proposed diversion, looking downstream on 4-7-11.

D22. RRN in vicinity of proposed diversion looking upstream on 4-7-11.

D23. Decayed sheet ice on RRN near diversion on 4-7-11 showing trace of Wild Rice R and flooded areas to the west.

D24. Flooding along Wild Rice R. on 4-7-11 looking NE from proposed diversion to RRN.

D25. Sheyenne R. looking NW at CR 16 and proposed canal crossing 4-7-11.



D26. Residual ice on Sheyenne R. in vicinity of proposed canal 4-7-11.

D27. Sheyenne River diversion into W. Fargo FC system near Horace 4-7-11.

D28. Sheyenne River diversion into W. Fargo canal system at CR 17 near I-94 4-7-11.

D29. Looking west at remaining ice accumulations in W. Fargo diversion canal on 4-7-11.


D30. Minor ice accumulations at RR Bridge in W. Fargo diversion canal on 4-7-11.

D31. Minor ice accumulation in canal branch reentering Sheyenne R in W. Fargo 4-7-11.

D32. Sheyenne River nearly ice-free south of Harwood at 32nd St. 4-7-11.

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D35.Remaining ice in vicinity of Rush R. Sheyenne confluence 4-7-11.

D36. Drifting ice on RRN near Georgetown 4-7-11.

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D41. Remaining ice cover on Sheyenne R. downstream of CR 16 4-7-11.

D42. Ice accumulation on Sheyenne R. upstream of CR 14 4-7-11.

D43. Fast currents and shoved ice floes, Sheyenne R. downstream of CR 14 4-7-11.

D44. Melting sheet ice on Sheyenne R upstream of 52nd Ave. W. 4-7-11.

D45. Decayed sheet ice on Sheyenne R upstream CR 17 Br. 4-7-11.

D46. Floes and sheet ice at Sheyenne diversion into W. Fargo FC Project at CR 17 Br. 4-7-11.

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D51. RRN ice nearly melted on at proposed diversion location 4-8-11.

D52. View from proposed diversion westward towards Wild Rice R. 4-8-11.

D53. Accumulated floes at diversion of Sheyenne into W. Fargo FC Project at CR 17 Br. 4-8-11.

D54. Minor ice jam on Sheyenne R. upstream of upstream 17th Ave. SE 4-8-11.

D55. Decayed floes in bend along CR 8 4-8-11.

D56. Remaining ice at diversion of Sheyenne into W. Fargo FC Project at CR 17 Br. 4-8-11.

Attachment 2

Response to
Richland County Drain Comment

MEMO

HoustonEngineering Inc.

To: U.S. Army Corps of Engineers – St. Paul District
City of Fargo, North Dakota
City of Moorhead, Minnesota

Date: July 5, 2011

From: Gregg Thielman, P.E., CFM
Houston Engineering, Inc.

Subject: FM Metro Flood Risk Management Project - Richland County Drain Impact Assessment

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Introduction:

On June 13, 2011, the Richland County Water Resource District (WRD) submitted comments on the proposed Fargo Moorhead Metro Flood Risk Management Project diversion alternatives as part of the comment period for the Supplemental Draft Environmental Impact Study. Specifically, the WRD requested the following information for the 100-year (1-percent chance) and 500-year (0.2-percent chance) flood events compared to existing conditions:

A. Analysis of impacts to the legal drains in the northern end of Richland County.

B. Analysis of impacts, which include stage increases, changes in flow, velocities, and drainage patterns on the Wild Rice, Sheyenne, and Red Rivers within Richland County.

C. Analysis of impacts on all other natural drainage systems in Richland County.

Response:

Our initial response to these questions is as follows:

A. Richland County Legal Drain Impacts:

Figures 1 and 2 highlight existing legal drains in Richland County within the staging area for the Locally Preferred Plan (LPP) diversion alternative (North Dakota Diversion). These include Richland County Drains 2, 5, 17, 19, 27, 37, 48, 57, and 64. The impact to these drains as a result of the LPP diversion alternative will be increased stages when the upstream staging area is being utilized. The staging impacts on the drains will be less than 1 foot for the 1-percent and 0.2-percent chance flood events as shown in Figures 1 and 2. These figures show the depth difference in feet between the LPP diversion alternative and existing conditions for the 1-percent chance and 0.2-percent chance flood event based on the unsteady HEC-RAS model results as presented in the April, 2011 technical appendices. Given the small impact of the staging area, we believe the impacts on the Richland County legal drains will be minor. In addition, runoff from the local drains will typically occur prior to peak flows on the Red River and Wild Rice River which will further reduce impacts.

For the Federally Comparable Plan (FCP) diversion alternative, there will be no impacts to legal drains in Richland County since the diversion will be operated so that existing conditions are matched upstream from the diversion inlet.

B. Wild Rice, Sheyenne, and Red River Impacts:

Similar to the legal drain impacts, the impacts to the Wild Rice and Red River for the LPP diversion alternative will be increased stages when the upstream staging area is being utilized. Peak stage impacts for the LPP are summarized in the following table:

River	Model Location	Location Description	Existing Conditions		LPP		Difference	
			1-percent chance	0.2-percent chance	1-percent chance	0.2-percent chance	1-percent chance	0.2-percent chance
			Elevation	Elevation	Elevation	Elevation	(ft)	(ft)
Red	2582884	Cass/Richland County Line	919.1	922.2	923.1	923.4	4.0	1.1
Red	2655411	Christine, ND (Upstream HWY 30)	925.2	929.9	926.2	930.0	1.0	0.0
Red	2764835	Abercrombie, ND	935.6	940.9	935.7	940.9	0.1	0.0
Wild Rice	109200	Upstream of HWY 46	923.5	925.3	923.8	925.3	0.3	0.0
Wild Rice	169892	Upstream of I-29	931.5	934.0	931.5	933.9	0.0	0.0
Wild Rice	225847	Abercrombie, ND	937.9	940.7	937.9	940.7	0.0	0.0

*Elevations Reference NAVD88

Table 1 – Wild Rice and Red River stage impacts for LPP diversion alternative.

The peak stage increases for the Wild Rice and Red Rivers occur at the Cass/Richland County line and diminish as you proceed upstream. Stages on the Sheyenne River in Richland County will not be impacted for the LPP diversion alternative.

There will be no impact on discharge for the Wild Rice and Red Rivers as a result of the LPP diversion alternative and any impact on velocity will be small. There will likely be some reduction in velocity during the staging of water for those areas that are impacted by higher stage. Breakout flows from the Sheyenne River into the Wild Rice River will still occur similar to existing conditions.

For the Federally Comparable Plan (FCP) diversion alternative, there will be no impacts to the Wild Rice, Sheyenne, or Red River in Richland County since the diversion will be operated so that existing conditions are matched upstream from the diversion inlet.

C. Other Natural Drainage System Impacts:

Impacts to other natural drainage systems in Richland County will also be related to increased stages when the upstream staging area is being utilized for the LPP diversion alternative as highlighted in Figures 1 and 2. No impacts will occur for the FCP diversion alternative since the diversion will be operated so that existing conditions are matched upstream from the diversion inlet.

Figures:

Figure 1 – Richland County Drain Impacts – LPP 1% chance flood event.
Figure 2 – Richland County Drain Impacts – LPP 0.2% chance flood event.

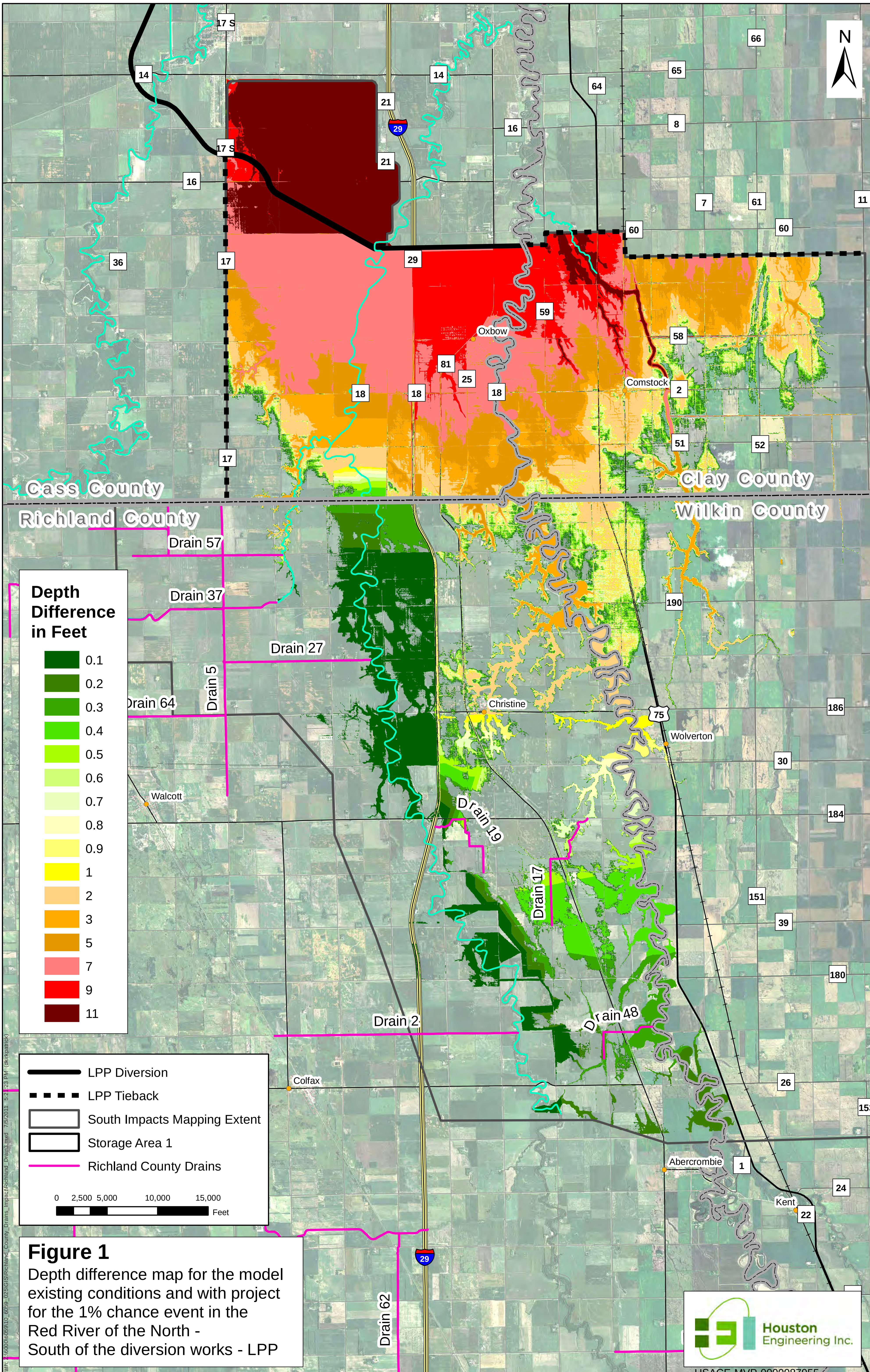


Figure 1
Depth difference map for the model existing conditions and with project for the 1% chance event in the Red River of the North - South of the diversion works - LPP

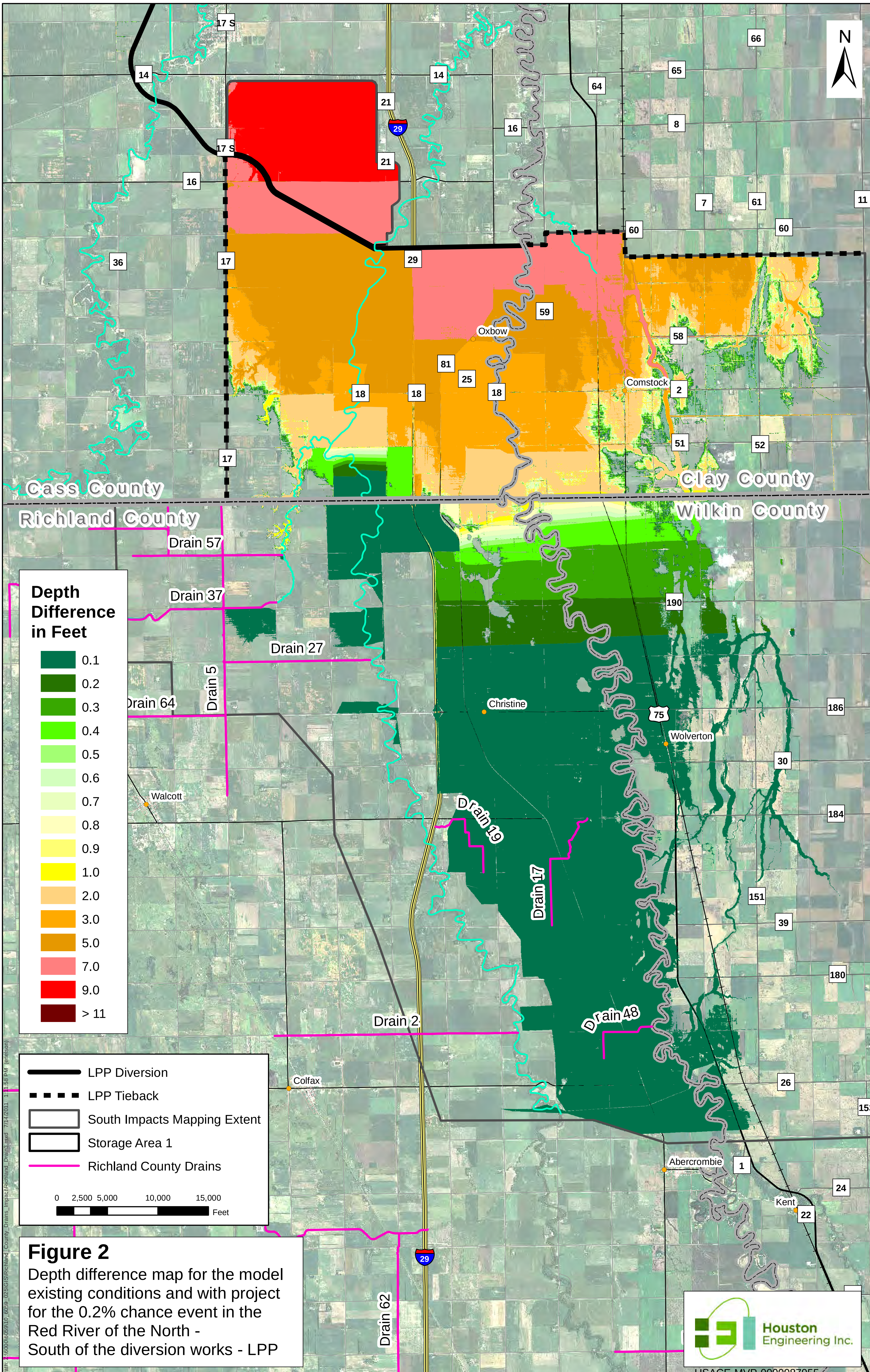


Figure 2
Depth difference map for the model existing conditions and with project for the 0.2% chance event in the Red River of the North - South of the diversion works - LPP

USACE-MVP-0000087955

Attachment 3

2011 Sediment Transport Data from USGS

resourceful. naturally.
engineering and environmental consultants

BARR

Technical Memorandum

To: Michelle Schneider – USACE
From: Peter Hinck and Miguel Wong
Subject: 2011 Sediment Transport Data from USGS
Date: July 8, 2011
Project: 34091004.00 420 400
c: Jon Sobiech, Elliott Stefanik and Aaron Buesing – USACE

The USGS collected sediment data on the Red River of the North (RRN) and its tributaries in and around the cities of Fargo and Moorhead during the spring flood of 2010. This data was summarized in the USGS Scientific Investigations Report 2011-5064 titled "Sediment Concentrations, Load, and Particle Size Distributions in the Red River of the North and Selected Tributaries near Fargo, North Dakota during the 2010 Spring High-Flow Event" (Blanchard et al., 2010). The relevance of this information for the feasibility of the proposed diversion of the RRN and its tributaries was discussed in detail in Exhibit I of Appendix F of the *Fargo-Moorhead Metro Flood Risk Management Project, Feasibility Study, Phase 4* (Version 2 of Exhibit I dated March 9, 2011).

Subsequent to the publication of the Feasibility Study, Phase 4 report, the USGS provided information pertaining to collected sediment data on the RRN and its tributaries during the spring flood of 2011. These new data are currently available in preliminary form from the USGS and will be summarized in a forthcoming USGS report. The purpose of this memorandum is to assess whether these new (preliminary) data alter any of the conclusions presented in the Feasibility Study, Phase 4 report. The sediment sampling in 2011 included additional sites beyond those sampled in 2010, but only sites that were sampled during both flood events are discussed in this memorandum.

Sampling Period Comparison

Sediment data were collected in 2011 between April 4 and May 17 (44 days) at the locations shown in Table 2, with between 16 and 19 sampling events at each site. This represents a significantly longer period of record and more sampling events than in 2010, when measurements were taken 8 to 12 times over a period of 20 days. In both years the majority of the sediment data were collected on the receding limb of the flood hydrograph, and this collection extended over a much longer period in 2011. Figure 1 shows the measured flows at the sampling location on the RRN near Fargo for both the 2010 and 2011 events. Figure 2 shows the measured total sediment transport at the same location for both events. Note from these figures that the peak flow at this location in 2011 was 23,400 cfs compared to 21,100 cfs in 2010. Also note that the shape of the receding limb of the hydrograph in 2011 is similar to that in 2010, but the flows are approximately 3,000-4,000 cfs larger in 2011.

Sediment Loading Comparison

Total sediment loading for the period of concurrent measurements is shown in Table 1 for 2010 and in Table 2 for 2011. The differences in total sediment loading (suspended sediment load, bedload, and total sediment load) for the two events are affected by the differences in the length of the monitoring period discussed above. Because the extended monitoring period in 2011 included more sampling events on the receding limb of the hydrograph, the total load shown in Table 2 is more influenced by sediment transport conditions during the receding limb of the hydrograph, hence the daily average sediment loads over the period of measurement are smaller in 2011.

At the locations studied here, the peak sediment loading shown for 2011 in Table 2 is neither consistently greater nor less than that shown for 2010 in Table 1. The RRN could be considered the exception to this general, somewhat simplified assessment. The peak flow in the RRN was greater in 2011 than 2010 (see Figure 1) and the peak sediment load for both sites in the RRN was greater in 2011 than 2010. However, sediment loads in 2010 are similar or greater than those in 2010 for most of the receding limb of the hydrograph even though flows in 2010 were smaller than in 2011. These measurements would suggest that an improved characterization of sediment transport conditions should be based on an evaluation of river morphodynamics over the entire flood hydrograph that complements the attempt to develop unique relationships between instantaneous flows and sediment transport rates. For the RRN in particular, further analysis is recommended to examine the effect that differences between the two flood events (beyond simply the peak flow value) may have had on the quantity of sediment transport and its channel morphology.

Despite some differences in sediment loading between the two events, it is clear from the data presented in Table 1 and Table 2 that the broad characteristics of the RRN and its tributaries are consistent. The vast majority of the sediment load in this system is transported as suspended sediment, with bedload representing generally less than 1% of the total sediment in transport. Sediment loading in the Sheyenne River is proportionally greater than in the other rivers, driven by the higher observed loading from suspended sediment in the Sheyenne River.

Figure 3 shows the observed suspended sediment concentrations at the locations monitored in 2010 and 2011. The range of observed concentrations is broadly similar between the two flood events for all sites, although the bulk of the data (central 50%) shows lower concentrations at most sites in 2011 compared to 2010. As indicated above, this may be due to the fact that the longer measurement period in 2011 included more observations on the receding limb of the hydrograph. As in 2010, the suspended sediment concentrations in the Sheyenne River, both upstream and downstream of the existing Horace/West Fargo diversion, were significantly greater than for the other rivers.

Sediment Balance Comparison

The data collected in both 2010 and 2011 on the RRN and Wild Rice River (WRR) are 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 simplified analysis is that no significant 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 4 and Figure 5, the flow and sediment balances for both events are reasonably close, given the necessary simplifying assumptions indicated above. For both years, the total flow measured in Fargo was slightly higher than the total flow measured at the upstream RRN and WRR locations (5-9% difference). For both years, the total sediment load measured in Fargo was slightly less than the total load measured at the upstream RRN and WRR locations (1-3% difference).

The sediment balance result for 2011 would demonstrate that, as expected based on the comparable analysis based on the 2010 sediment transport data, the sediment load in the RRN through the cities of Fargo and Moorhead is neither increasing nor decreasing. More importantly, it would imply that 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.

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. The calculated balances for 2010 and 2011 are shown in Figure 6and Figure 7, respectively.

In 2010, both the flow and sediment load in the Sheyenne River downstream of the Horace/West Fargo Diversion were approximately 47-48% of the flow and sediment load in the Sheyenne River upstream of the existing diversion. In other words, the existing diversion did not affect the proportion of sediment with respect to flows (or sediment concentration) in the Sheyenne River. In 2011, a greater portion of the upstream flow (48%) than of the upstream sediment (41%) was measured in the Sheyenne River downstream of the existing diversion, and the suspended sediment concentrations in 2011 were slightly lower downstream than upstream Sheyenne River. This change from 2010 to 2011 is not large enough to indicate a significant difference in the fluvial system, and may be related to the differences in timing between the two flood events (the peak flow conditions persisted much longer on the Sheyenne in 2011 than they did in 2010). More interestingly, the measurements in 2011 would suggest that the existing diversion could be more efficient in diverting (out of the Sheyenne River natural system) sediments than water, and this is possible because of the fine nature of the sediments mobilized by the Sheyenne River primarily in suspension (over 99.5% in both years).

Sediment Gradation Comparison

Figure 8 through Figure 31 present the measured gradations of both the bed sediment and the bedload material as well as the fraction of suspended sediment that is finer than sand at each location in both 2010 and 2011. In general, the preliminary data from 2011 shows similar behavior as that observed in 2010 for all locations. Minor differences include:

- Several measurements of coarser bed material in the WRR in 2011 (Figure 9), consisting of coarse sand to fine gravel;
- Measurements of coarser suspended material in the upstream Sheyenne River in 2011 (see note on Figure 13), with as much as 43% of the suspended material being sand or larger particles;

- A wider range of measured fines content in the suspended material in the downstream Sheyenne River in 2011 (see note on Figure 17), with 10-30% of the suspended material being sand or larger particles;
- A wider range of bedload gradations in the Maple River in 2011 (compare Figure 22 and Figure 23);
- Slightly finer bed material in the RRN near Fargo in 2011 (compare Figure 28 and Figure 29), and a slightly larger amount of measured coarse suspended material (see note on Figure 29).

It should be noted that none of these differences indicates a significant shift in the fluvial system or invalidates the conclusions of the Feasibility Study, Phase 4 report. The differences appear to be the result of natural variability in complex riverine systems.

Conclusions

The preliminary sediment transport data for the 2011 high-flow event support the conclusions of the Feasibility Study, Phase 4 report with regards to the potential impact of the proposed project on sediment transport and geomorphology in the studied rivers. Because the RRN and its tributaries are dominated by the transport of fine suspended material, the diversion of a fraction of the river flow is expected to divert an approximately 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 Horace/West Fargo Diversion of the Sheyenne River provides an example of the potential impacts that can be expected from the proposed diversion. As discussed in the Feasibility Study, Phase 4 report, 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 approximate proportion to the flow. The slight differences in flow and sediment transport observed in the Sheyenne system in 2011 warrant further analysis but do not indicate a significant difference in the behavior of the fluvial system.

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 nor degrading, with sediment transport primarily in suspension. These characteristics are not expected to change significantly following implementation of the proposed diversion works.

Figure 1 – Comparison of 2010 and 2011 Sampling Period Flows for the RRN near Fargo

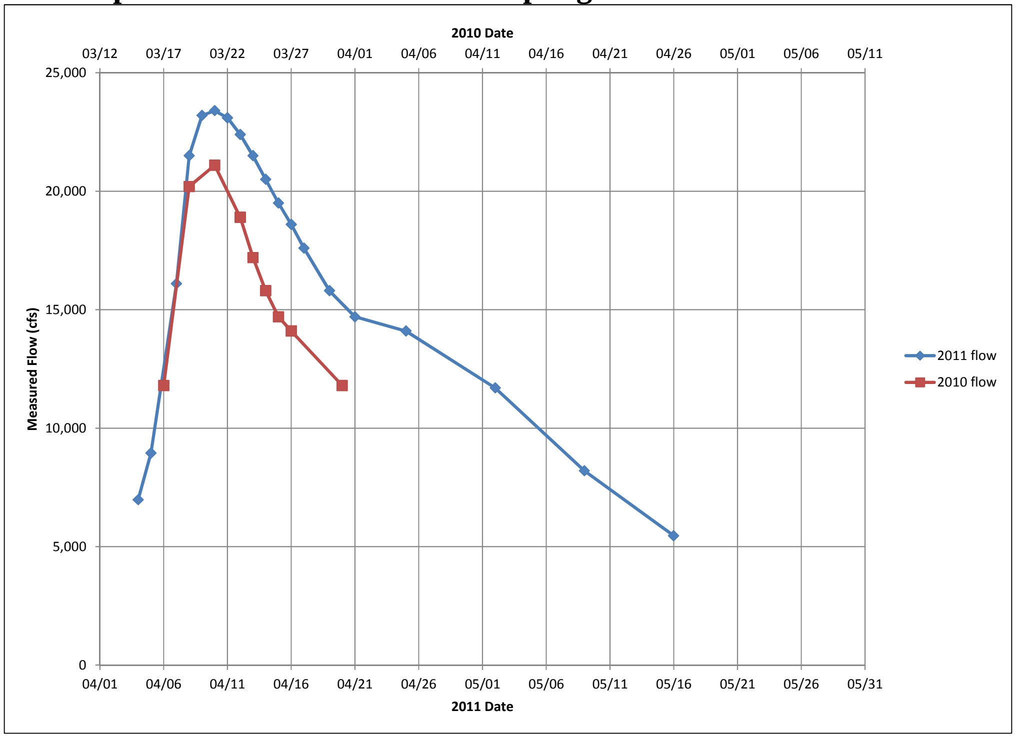


Figure 2 – Comparison of 2010 and 2011 Sampling Period Loads for the RRN near Fargo

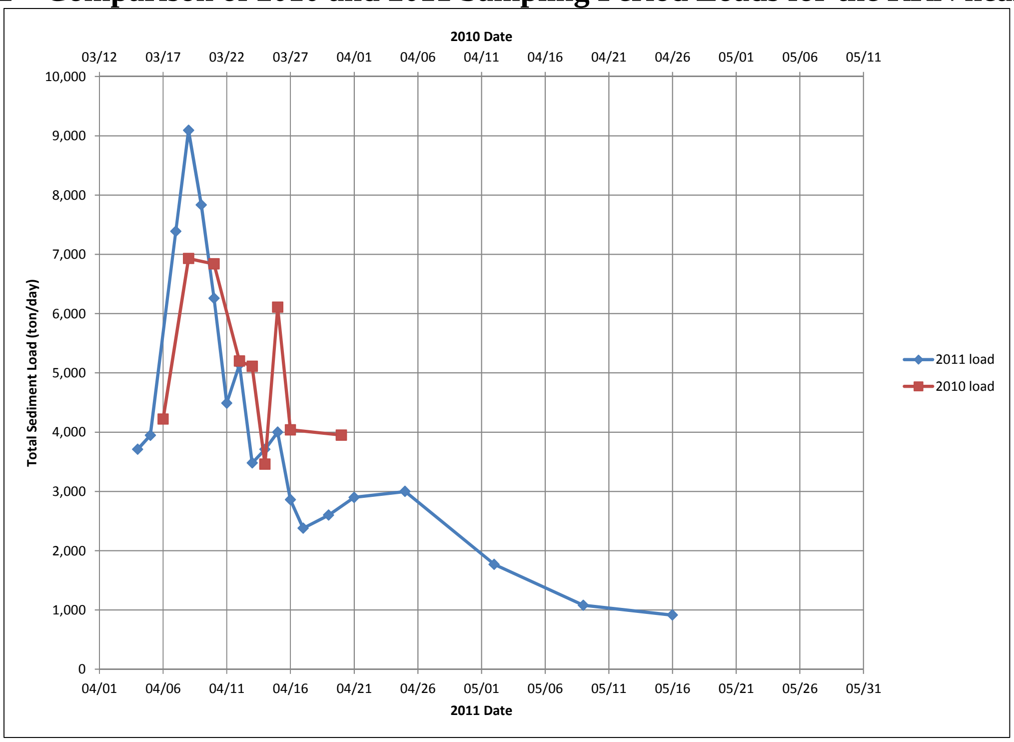


Table 1 – Observed Sediment Loading (*2010*)

Site Name	Time Period[1]	Total Flow (million ft^3)	Total Sus. Sed. Load (tons)	Total Bedload (tons)[2]	Total Sed. Load (tons)	Peak Sed. Load (tons/day)
Wild Rice River near St. Benedict	March 18, 2010 - March 31, 2010	8,780	43,260	31.8 (0.07%)	43,300	4,020
Sheyenne River above Sheyenne River Diversion near Horace	March 24, 2010 - April 7, 2010	5,340	119,590	40.7 (0.03%)	119,630	14,600
Sheyenne River at Horace	March 24, 2010 - April 7, 2010	2,580	56,370	9.3 (0.01%)	56,380	5,250
Maple River below Mapleton	March 19, 2010 - April 6, 2010	4,660	31,520	70.9 (0.2%)	31,600	4,840
Red River of the North near Christine	March 18, 2010 - March 31, 2010	10,030	30,780	171 (0.6%)	30,950	3,950
Red River of the North near Fargo	March 18, 2010 - March 31, 2010	19,810	72,080	27.6 (0.04%)	72,110	6,930

Table 2 – Observed Sediment Loading (*2011*)

Site Name	Time Period[1]	Total Flow (million ft^3)	Total Sus. Sed. Load (tons)	Total Bedload (tons)[2]	Total Sed. Load (tons)	Peak Sed. Load (tons/day)
Wild Rice River near St. Benedict	April 6, 2011 - May 16, 2011	17,960	67,610	195 (0.3%)	67,800	3,570
Sheyenne River above Sheyenne River Diversion near Horace	April 9, 2011 - May 16, 2011	14,730	175,130	84.0 (0.05%)	175,220	13,210
Sheyenne River at Horace	April 9, 2011 - May 16, 2011	7,060	72,450	220 (0.3%)	72,670	6,280
Maple River below Mapleton	April 7, 2011 - May 16, 2011	11,900	47,220	104 (0.2%)	47,320	3,650
Red River of the North near Christine	April 6, 2011 - May 16, 2011	26,710	49,700	756 (1.5%)	50,450	4,370
Red River of the North near Fargo	April 6, 2011 - May 16, 2011	48,650	117,460	91.7 (0.08%)	117,550	9,090

[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 to March 31, 2010 and April 6 to May 16, 2011) and on the Sheyenne River above and below the Sheyenne River Diversion (March 24 to April 7, 2010 and April 9 to May 16, 2011).
[2] Percentage values represent bedload as a fraction of total sediment load for the time periods shown.

Figure 3 – Comparison of 2010 and 2011 Suspended Sediment Concentration Data

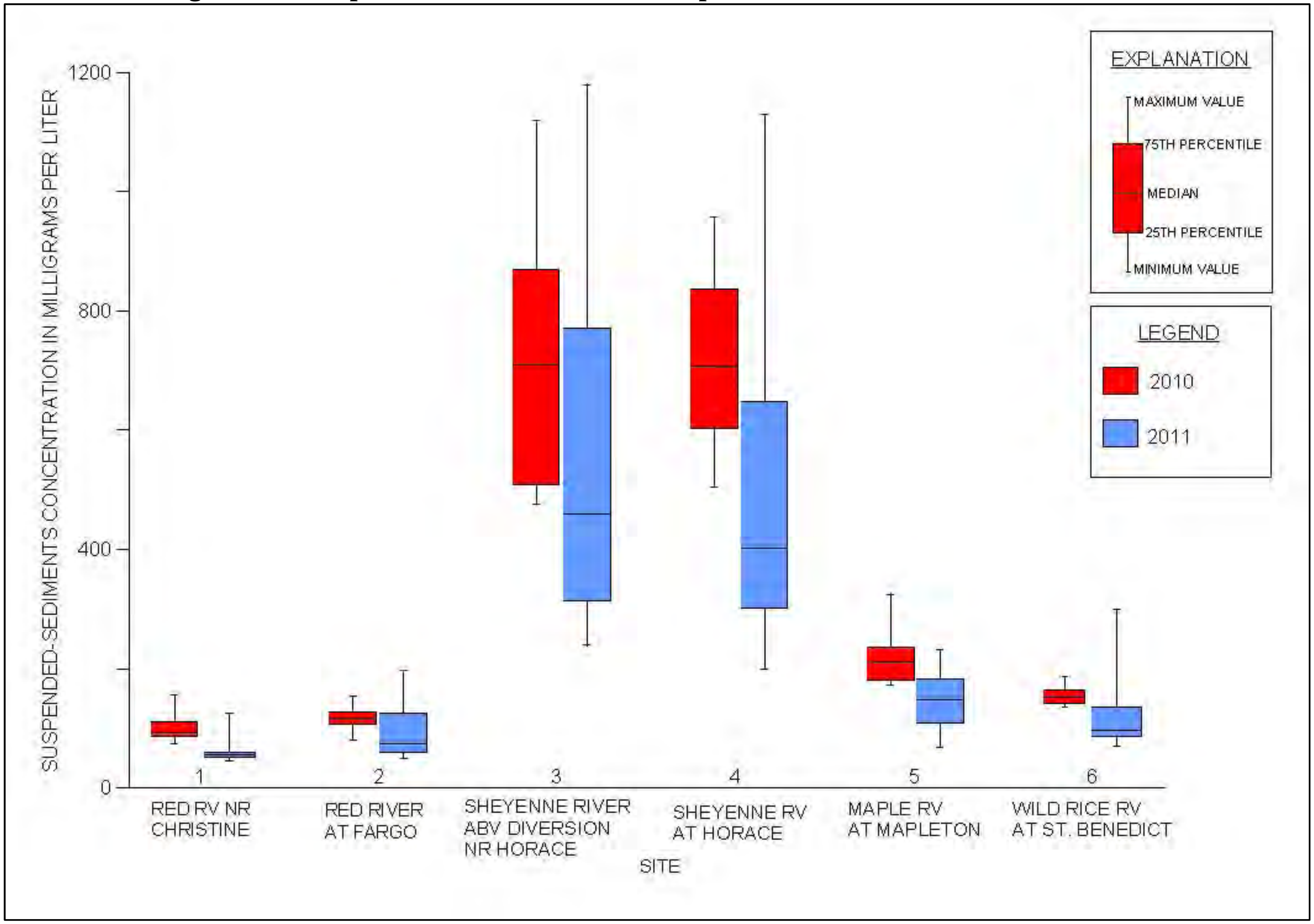


Figure 4 – Flow and Sediment Balance (*2010*) – Red River of the North

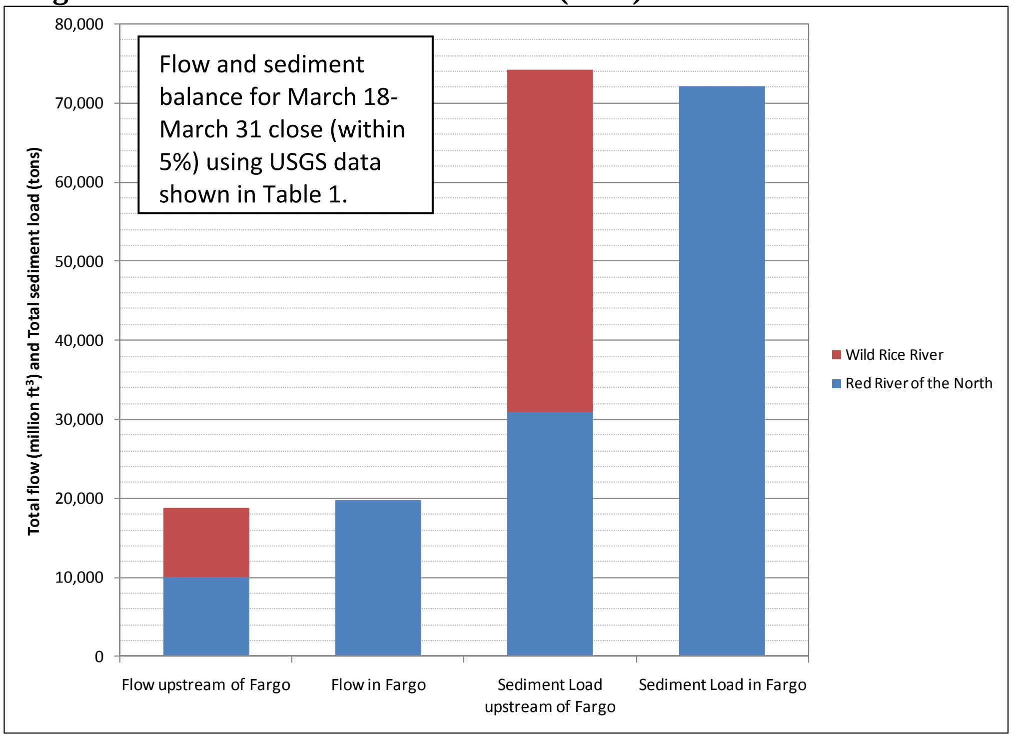


Figure 5 – Flow and Sediment Balance (*2011*) – Red River of the North

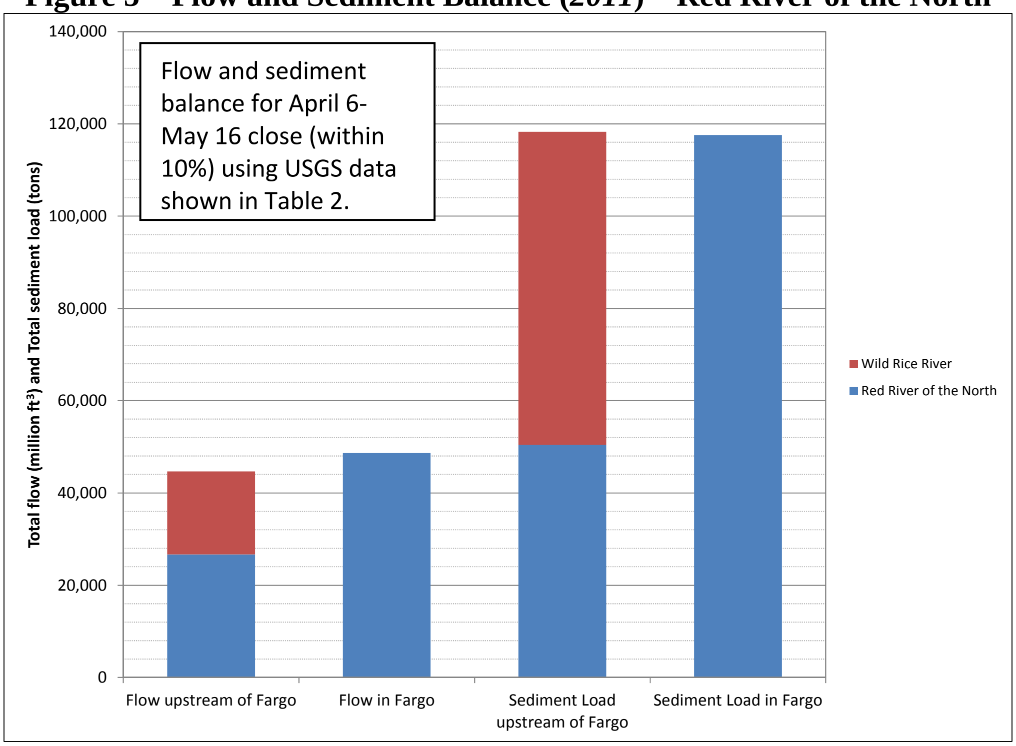


Figure 6 – Flow and Sediment Balance (*2010*) – Sheyenne River

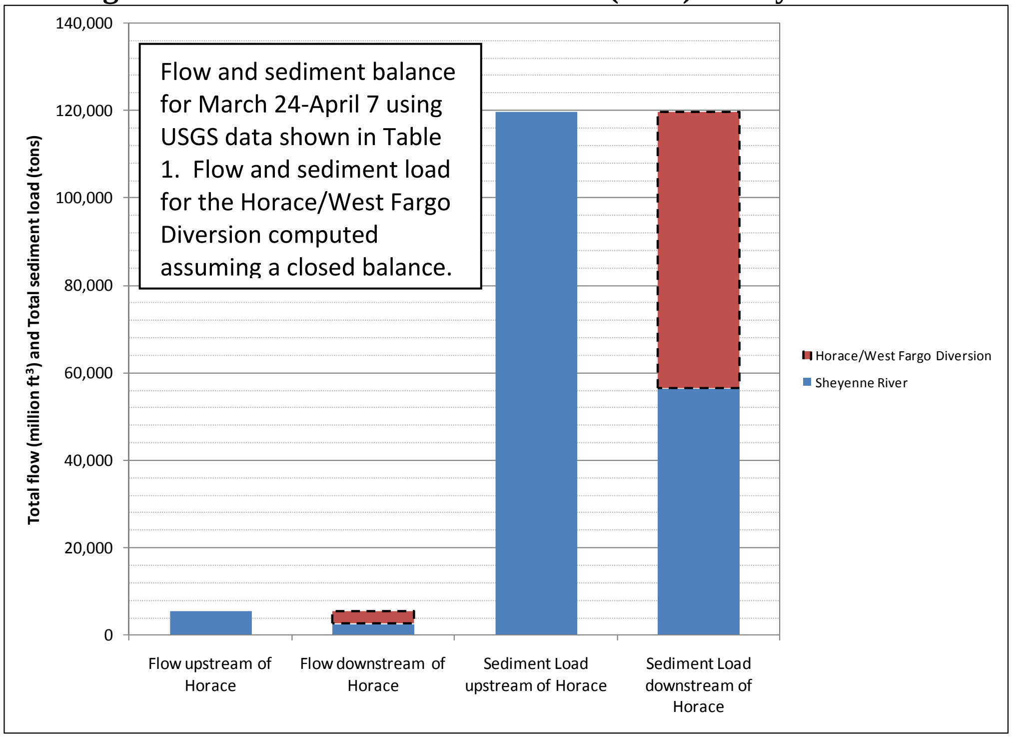


Figure 7 – Flow and Sediment Balance (*2011*) – Sheyenne River

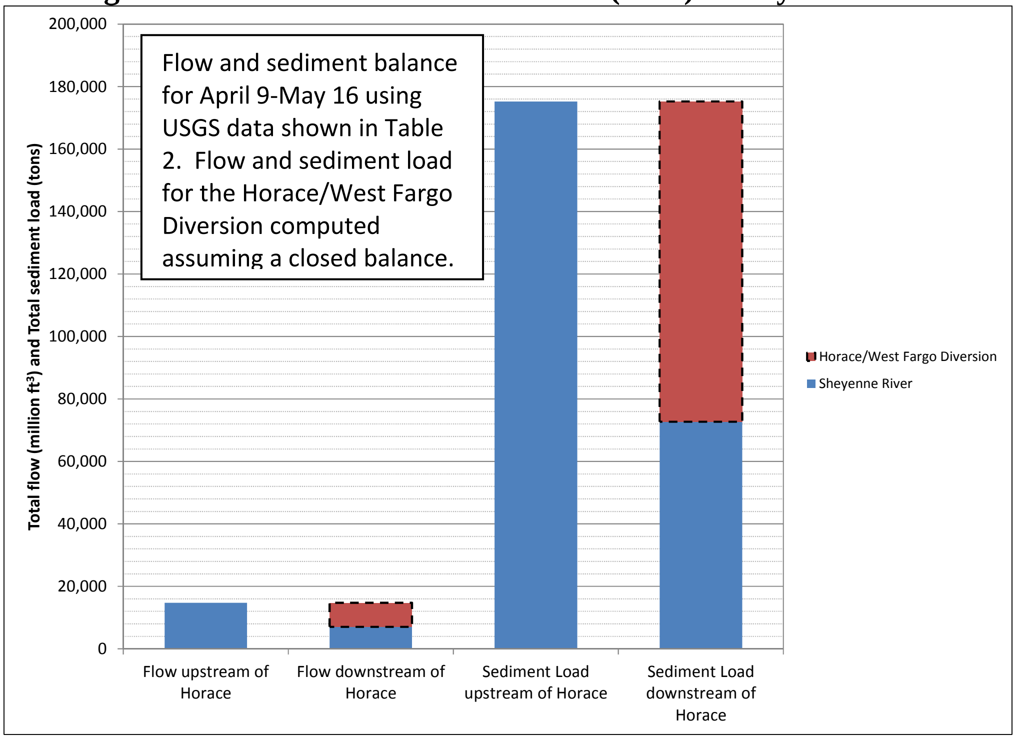


Figure 8 – Bed Sediment Gradation (*2010*) – Wild Rice River near St. Benedict

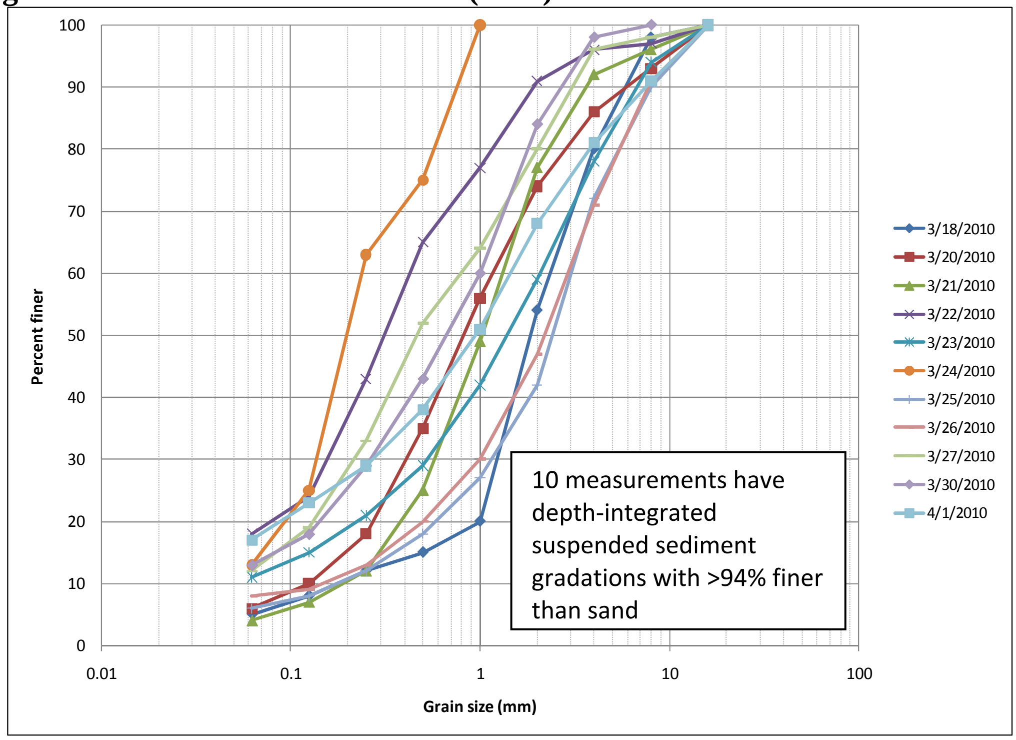


Figure 9 – Bed Sediment Gradation (*2011*) – Wild Rice River near St. Benedict

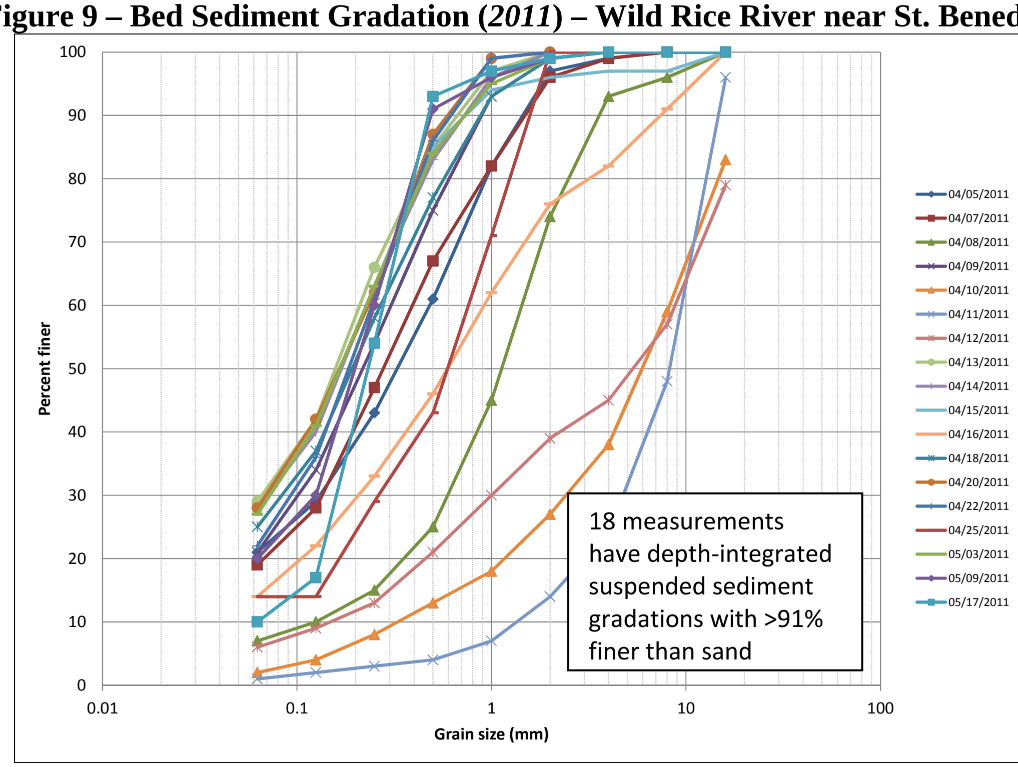


Figure 10 – Bedload Gradation (*2010*) – Wild Rice River near St. Benedict

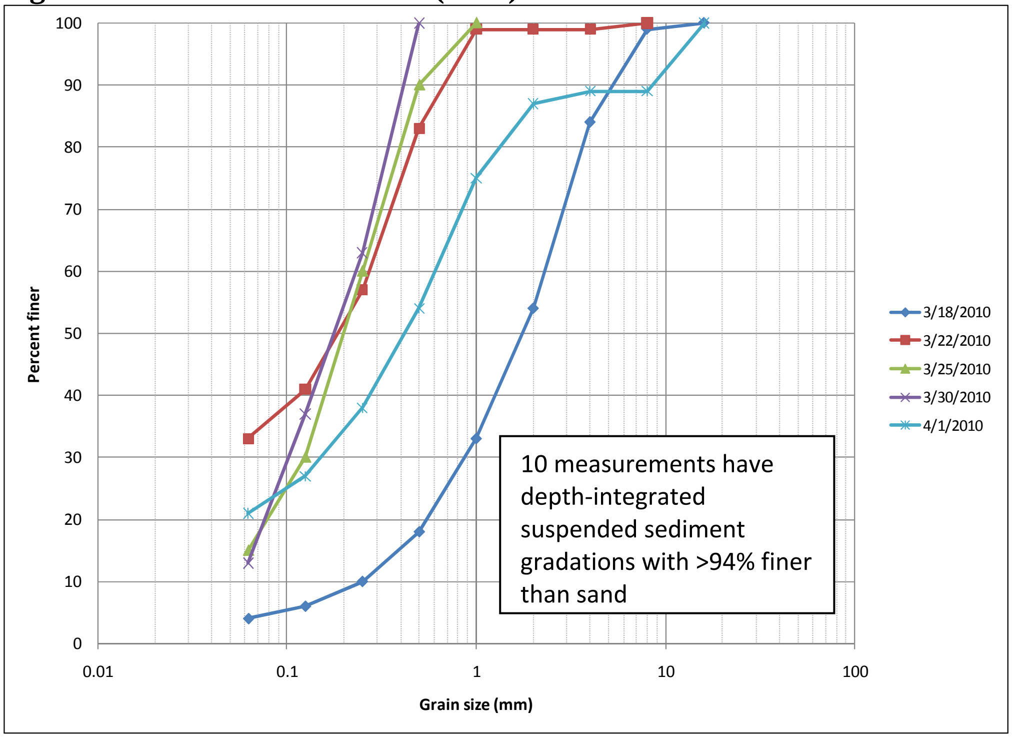


Figure 11 – Bedload Gradation (*2011*) – Wild Rice River near St. Benedict

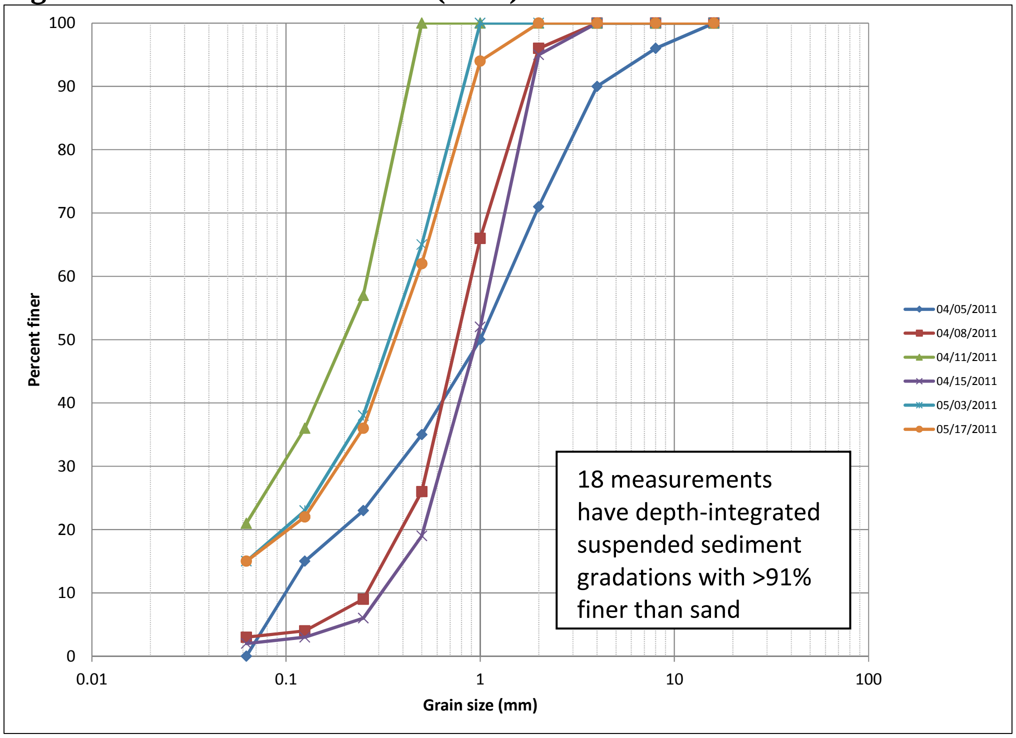


Figure 12 – Bed Sediment Gradation (*2010*) – Sheyenne River abv. Diversion near Horace

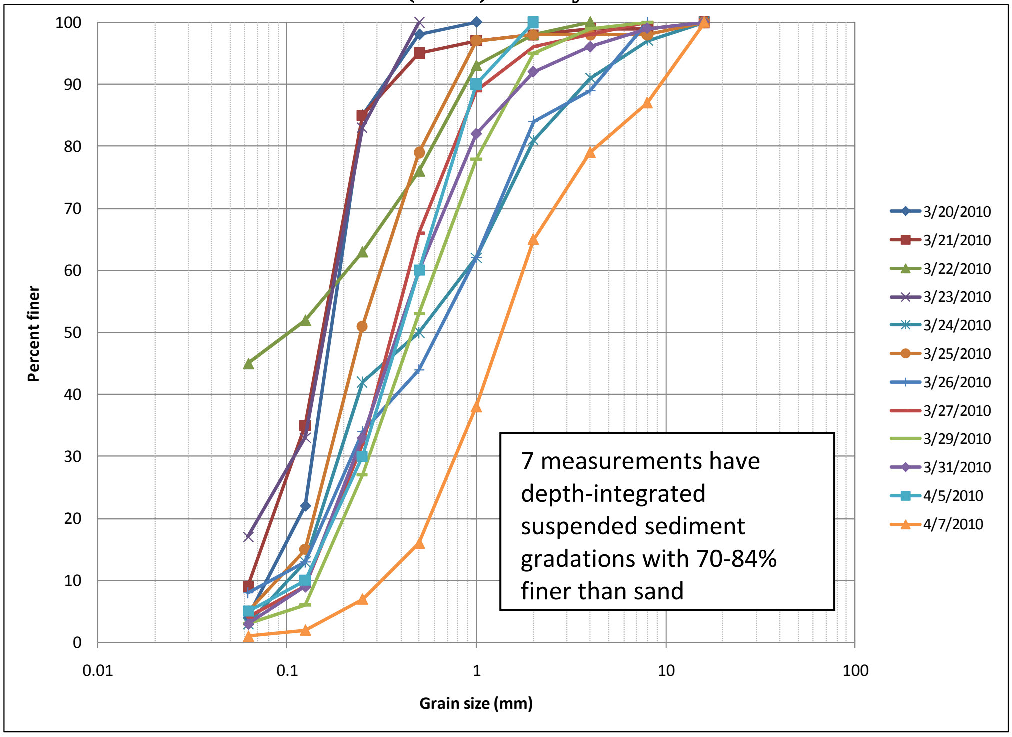


Figure 13 – Bed Sediment Gradation (*2011*) – Sheyenne River abv. Diversion near Horace

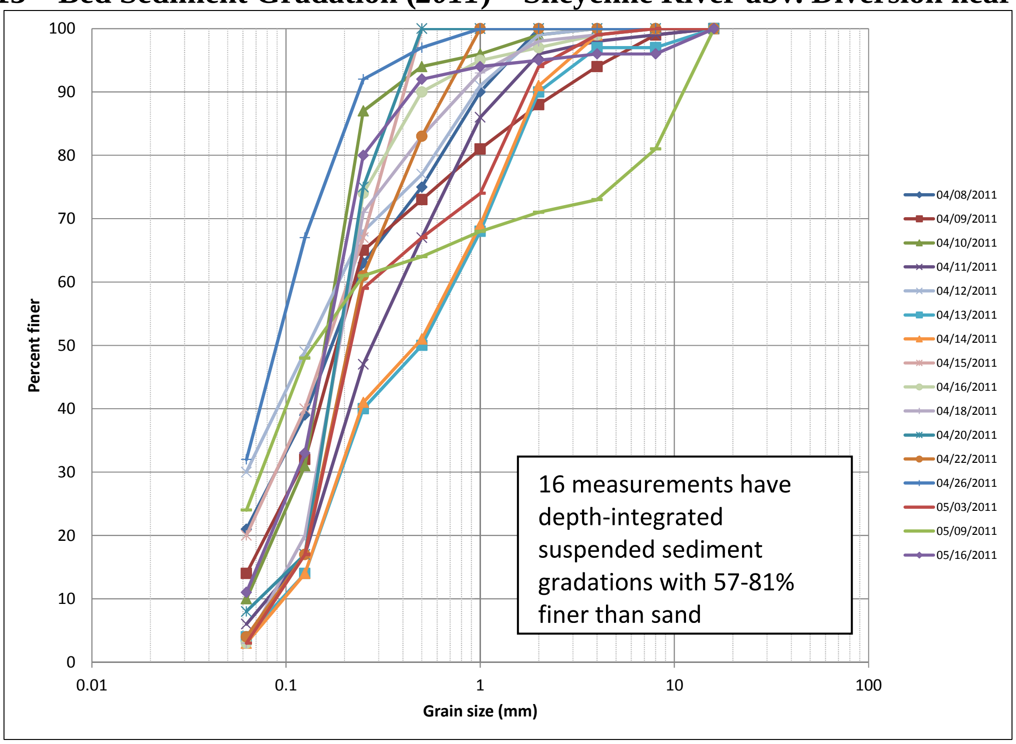


Figure 14 – Bedload Gradation (*2010*) – Sheyenne River abv. Diversion near Horace

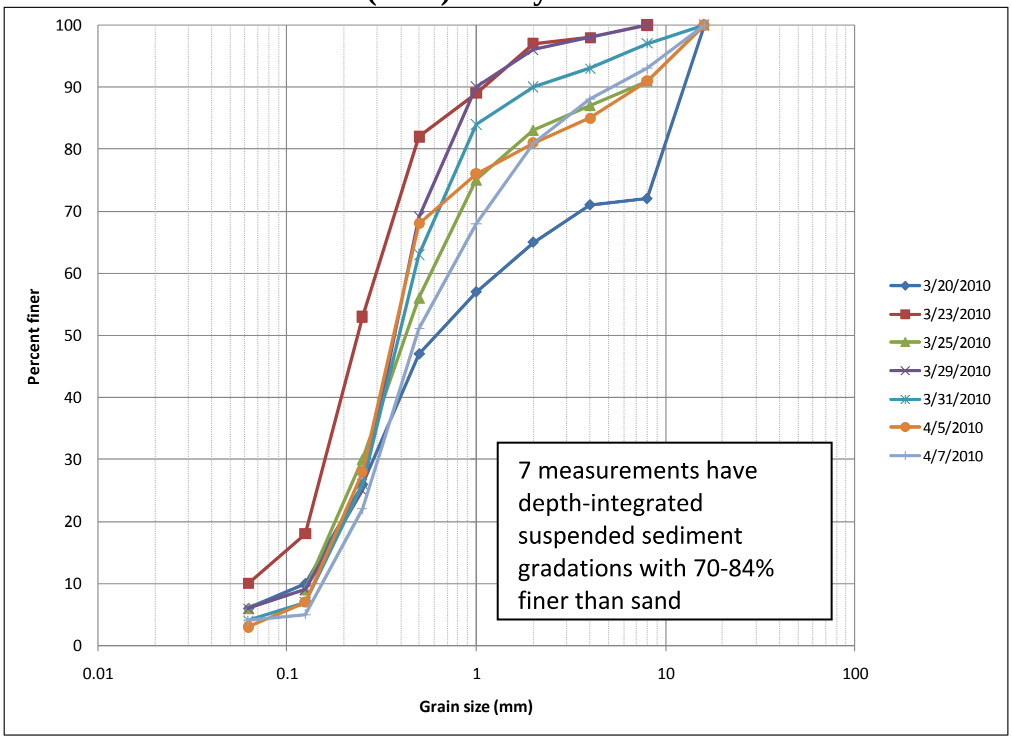


Figure 15 – Bedload Gradation (*2011*) – Sheyenne River abv. Diversion near Horace

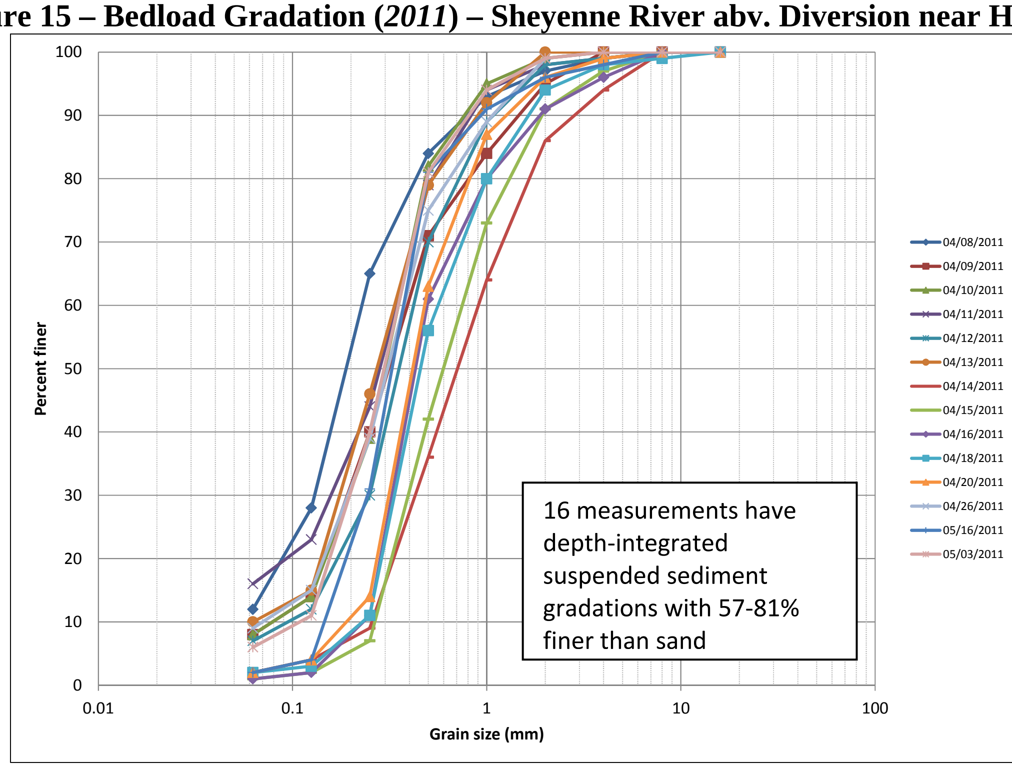


Figure 16 – Bed Sediment Gradation (*2010*) – Sheyenne River at Horace

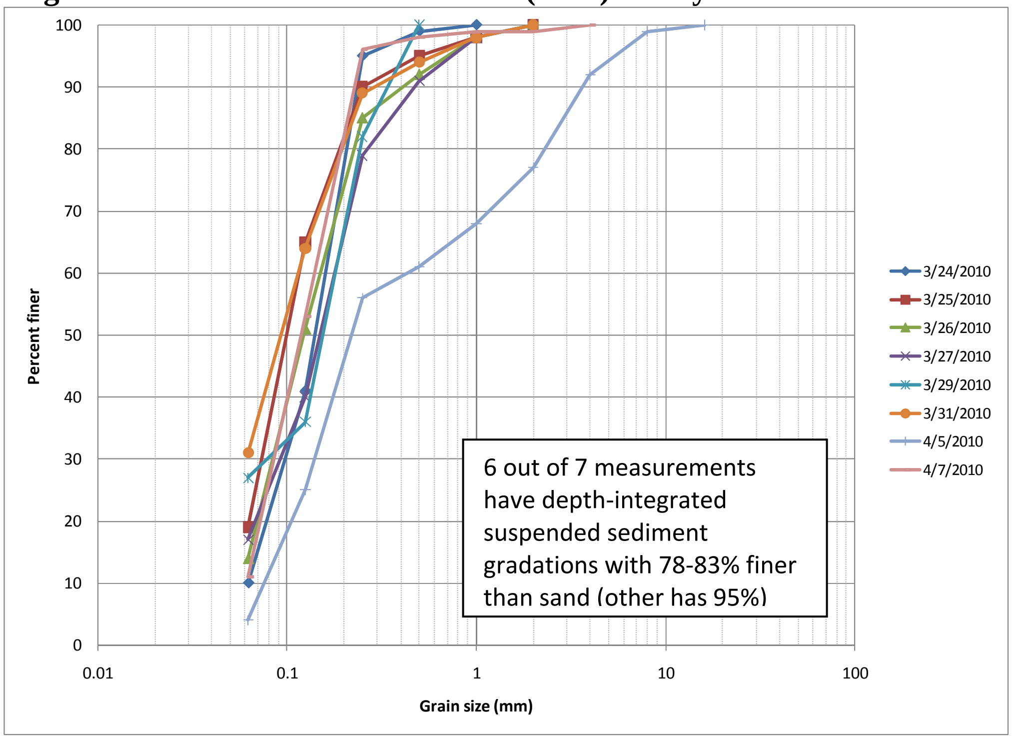


Figure 17 – Bed Sediment Gradation (*2011*) – Sheyenne River at Horace

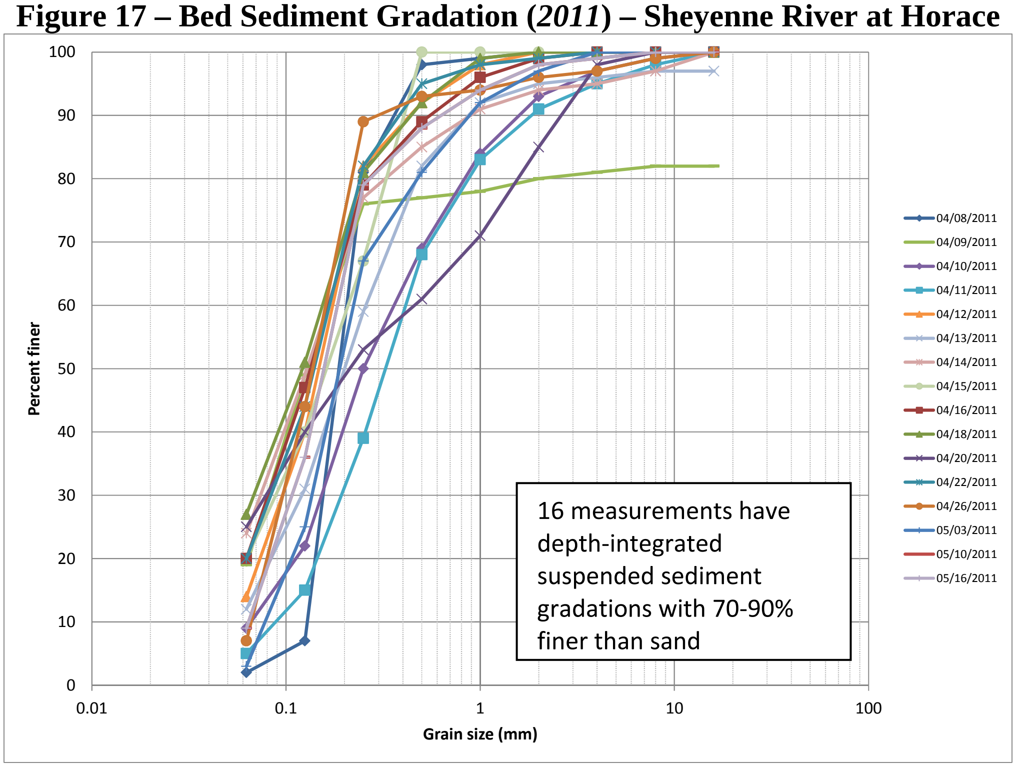


Figure 18 – Bedload Gradation (*2010*) – Sheyenne River at Horace

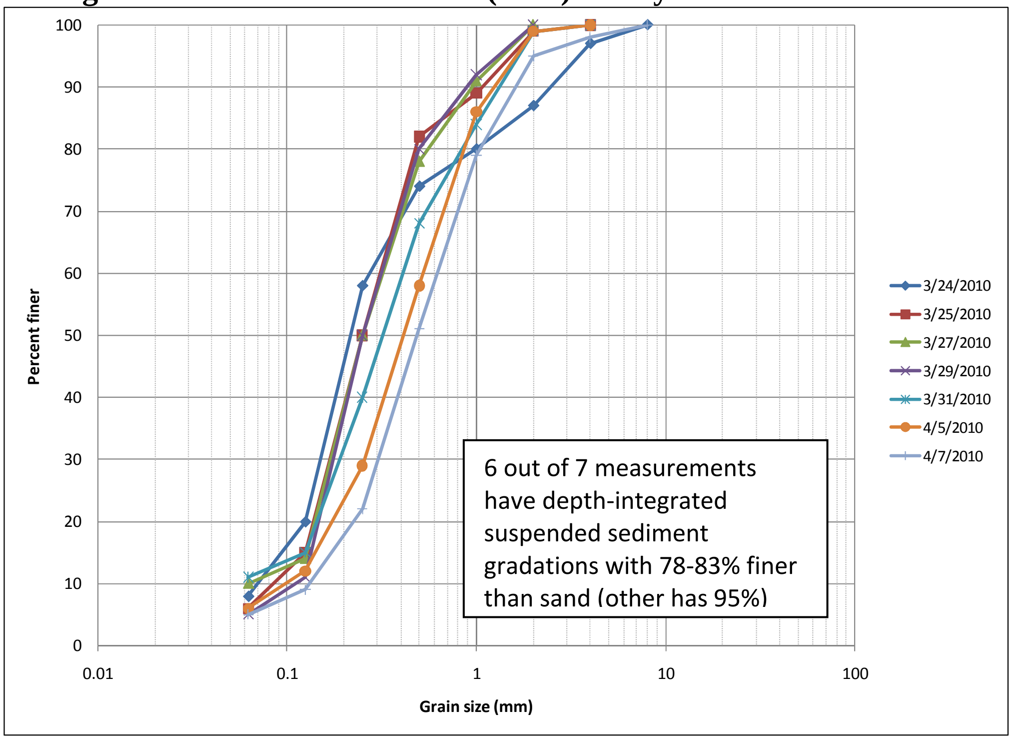


Figure 19 – Bedload Gradation (*2011*) – Sheyenne River at Horace

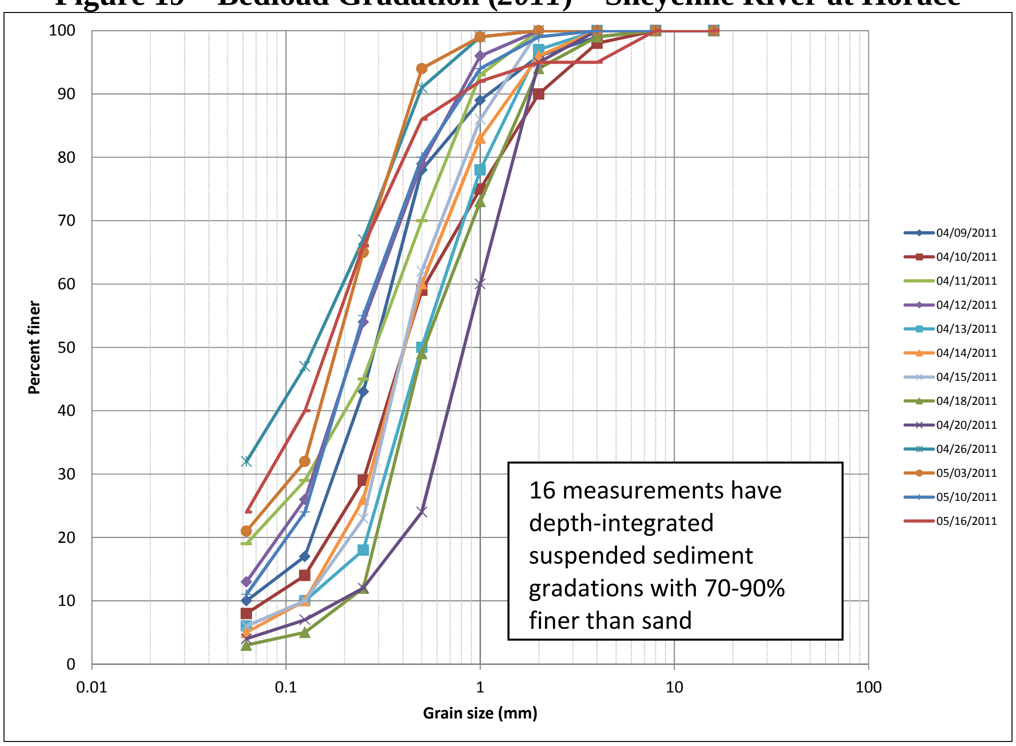


Figure 20 – Bed Sediment Gradation (*2010*) – Maple River below Mapleton

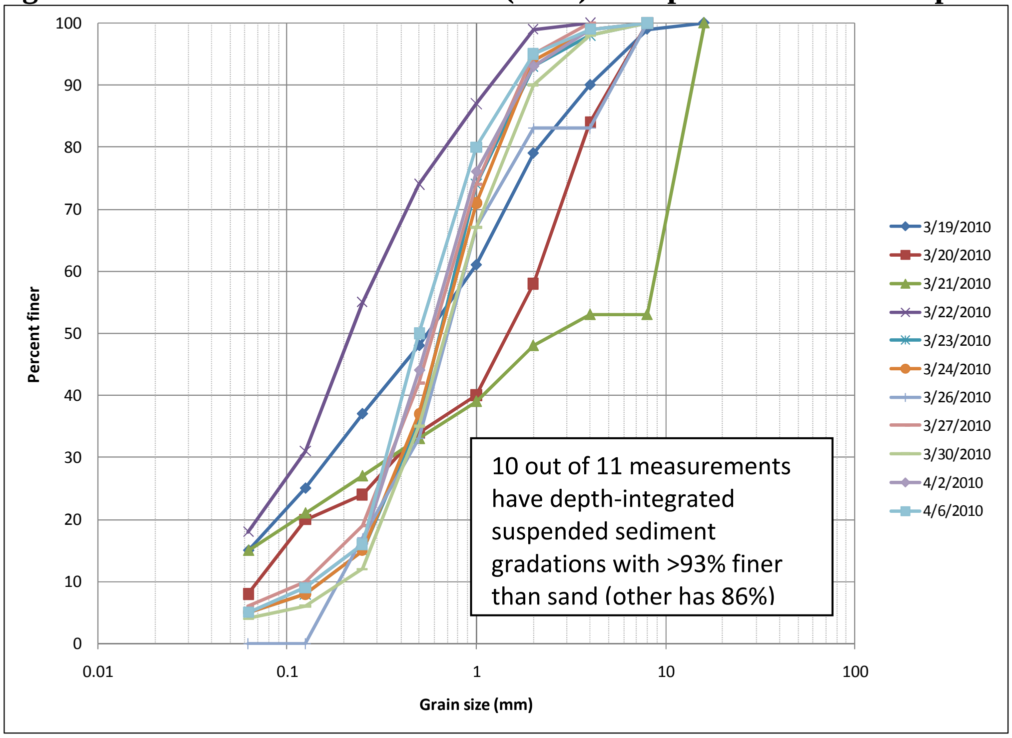


Figure 21 – Bed Sediment Gradation (*2011*) – Maple River below Mapleton

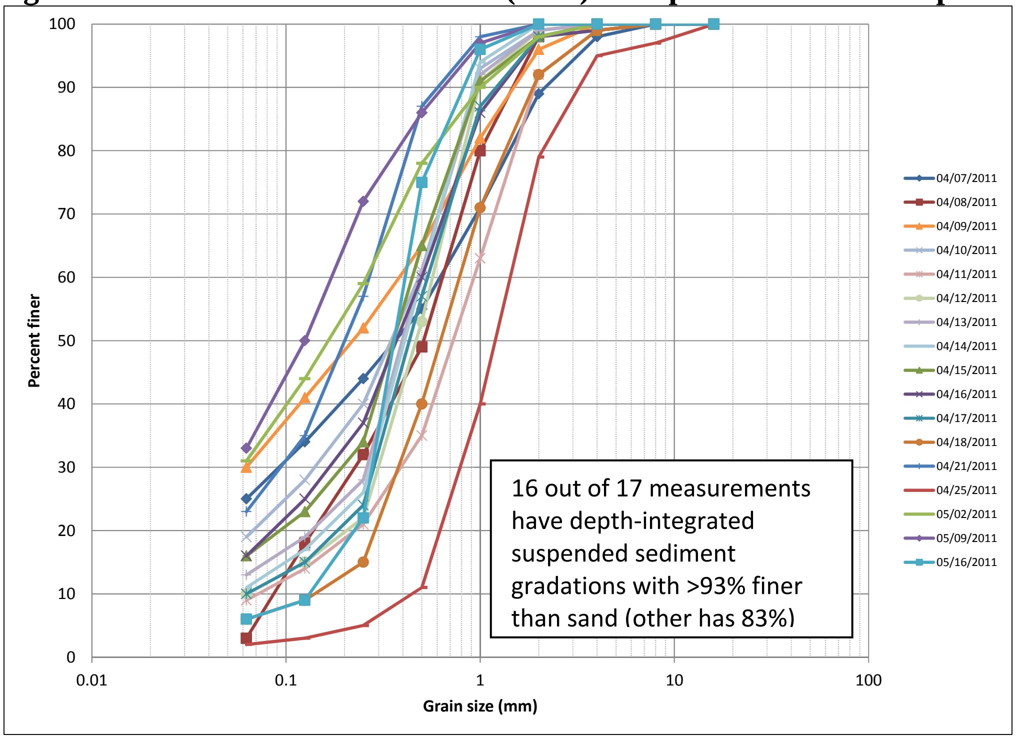


Figure 22 – Bedload Gradation (*2010*) – Maple River below Mapleton

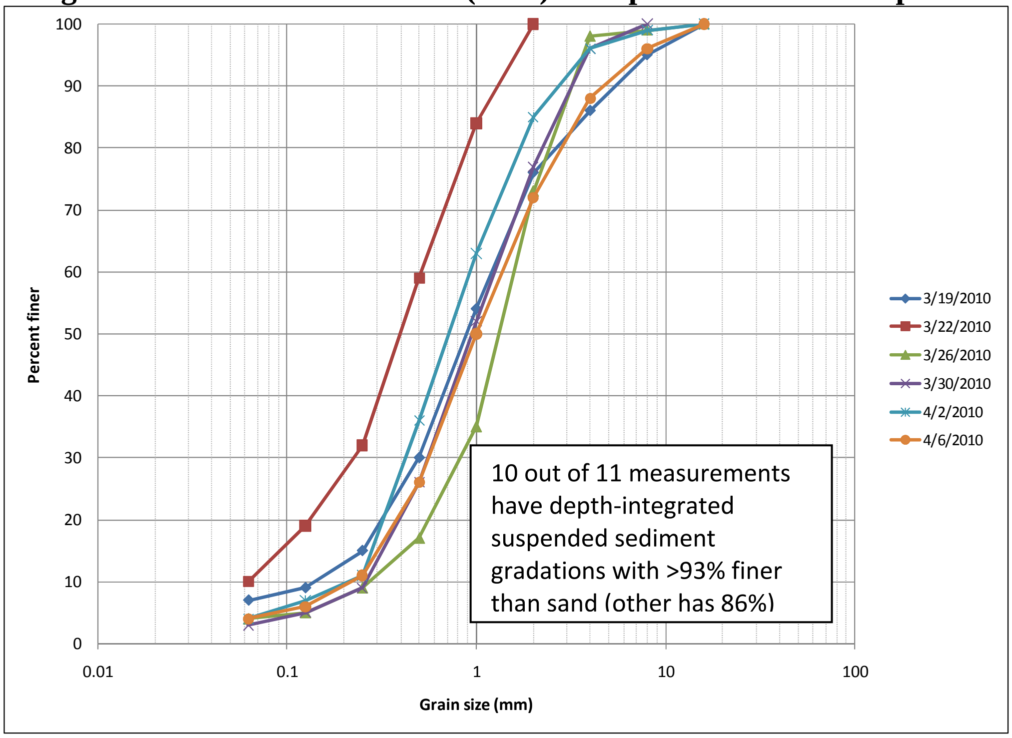


Figure 23 – Bedload Gradation (*2011*) – Maple River below Mapleton

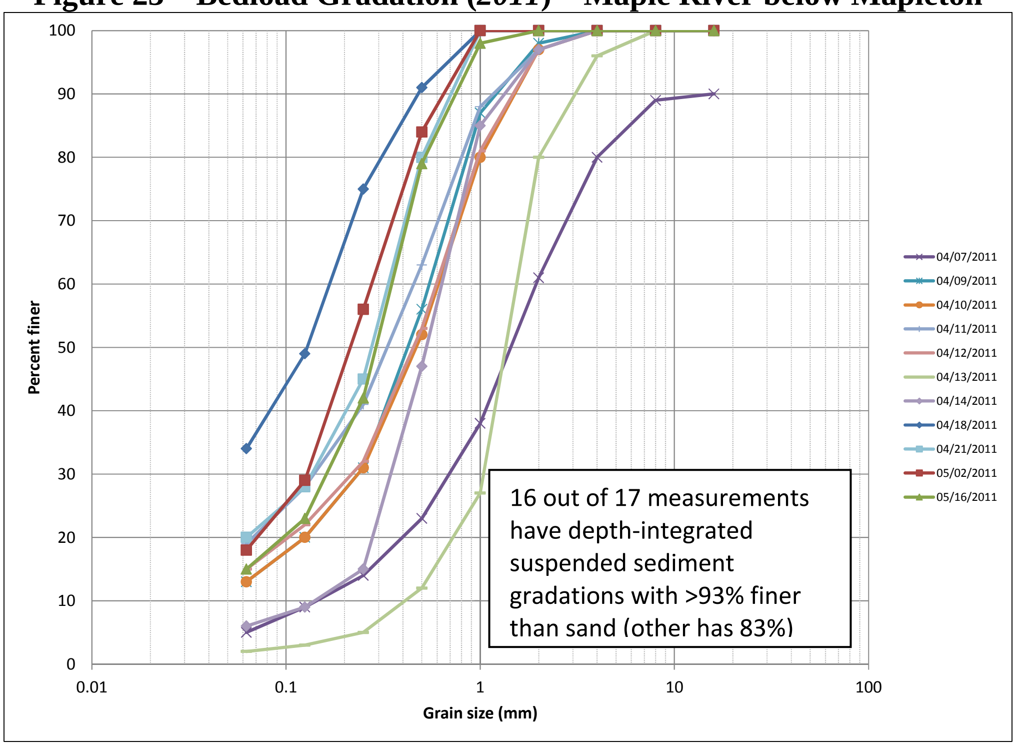


Figure 24 – Bed Sediment Gradation (*2010*) – Red River of the North near Christine

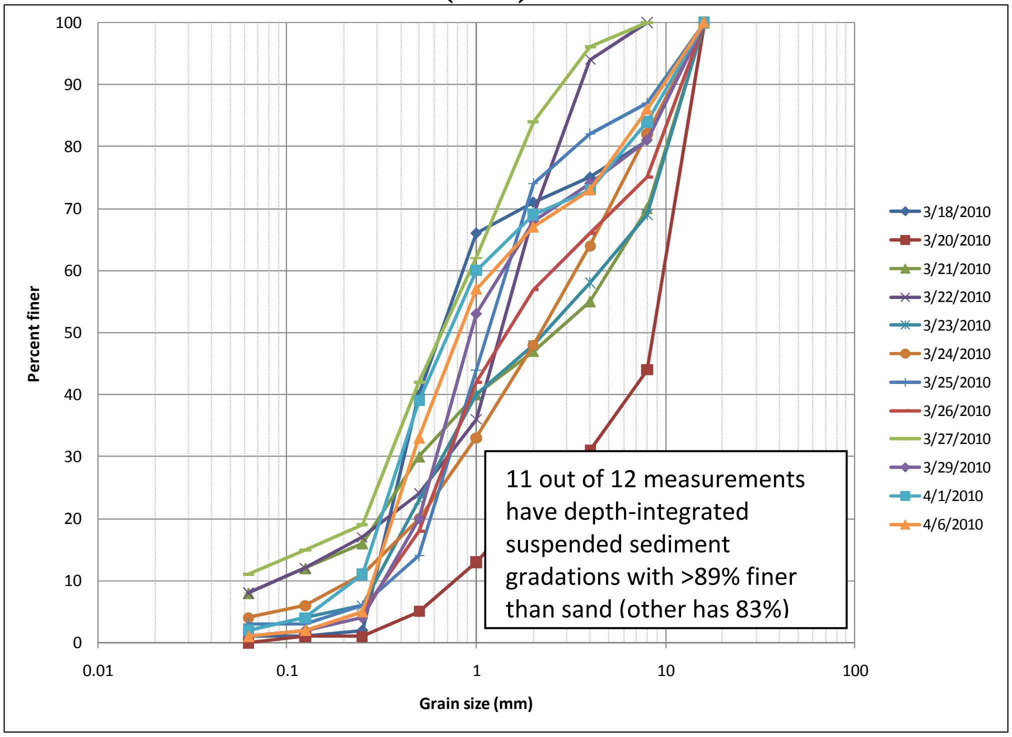


Figure 25 – Bed Sediment Gradation (*2011*) – Red River of the North near Christine

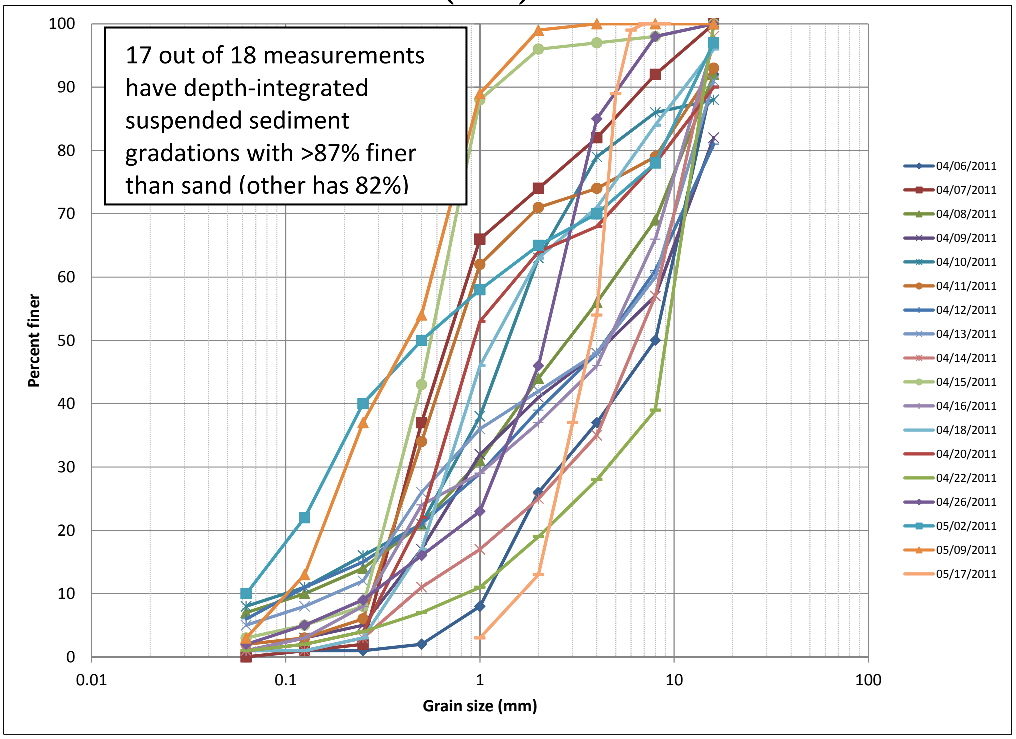


Figure 26 – Bedload Gradation (*2010*) – Red River of the North near Christine

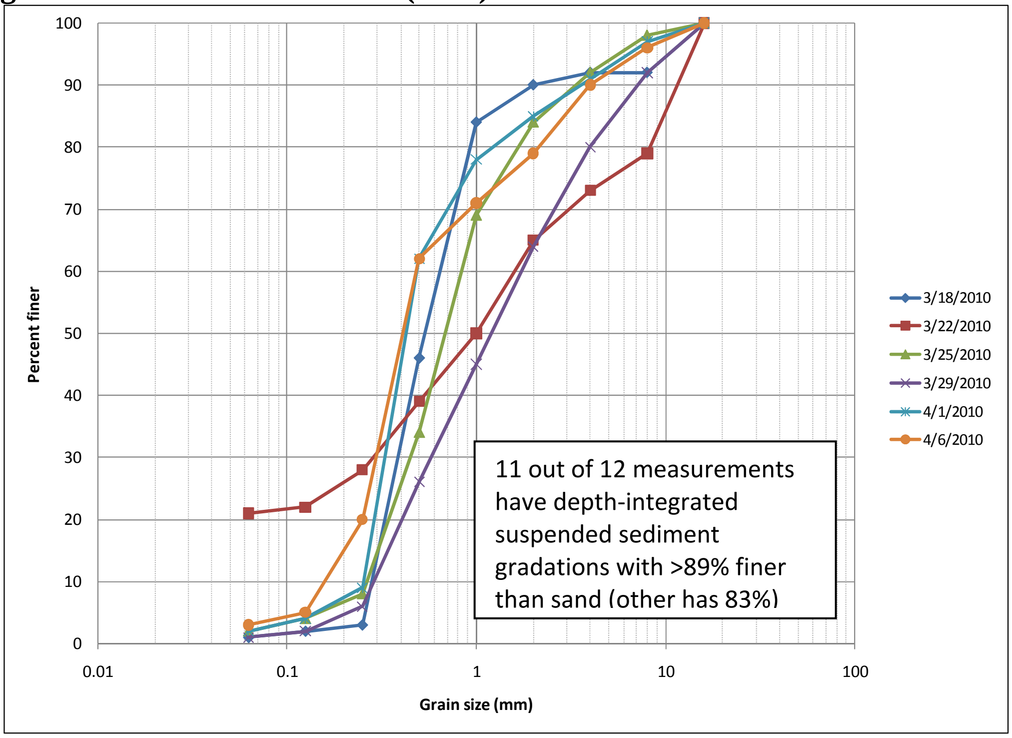


Figure 27 – Bedload Gradation (*2011*) – Red River of the North near Christine

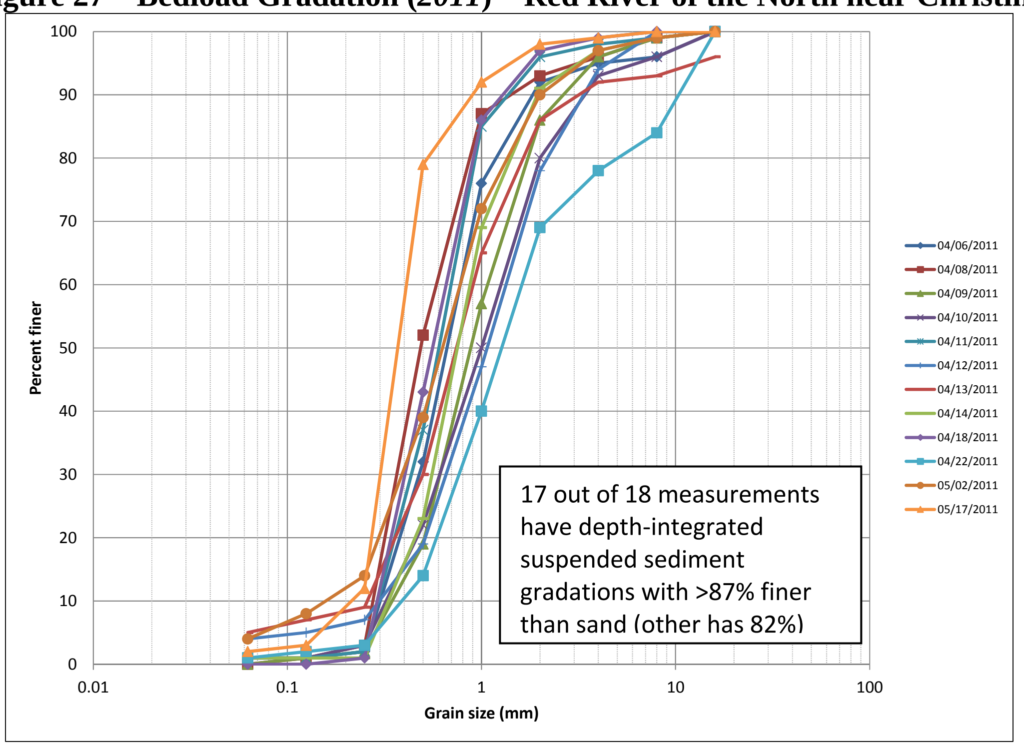


Figure 28 – Bed Sediment Gradation (*2010*) – Red River of the North at Fargo

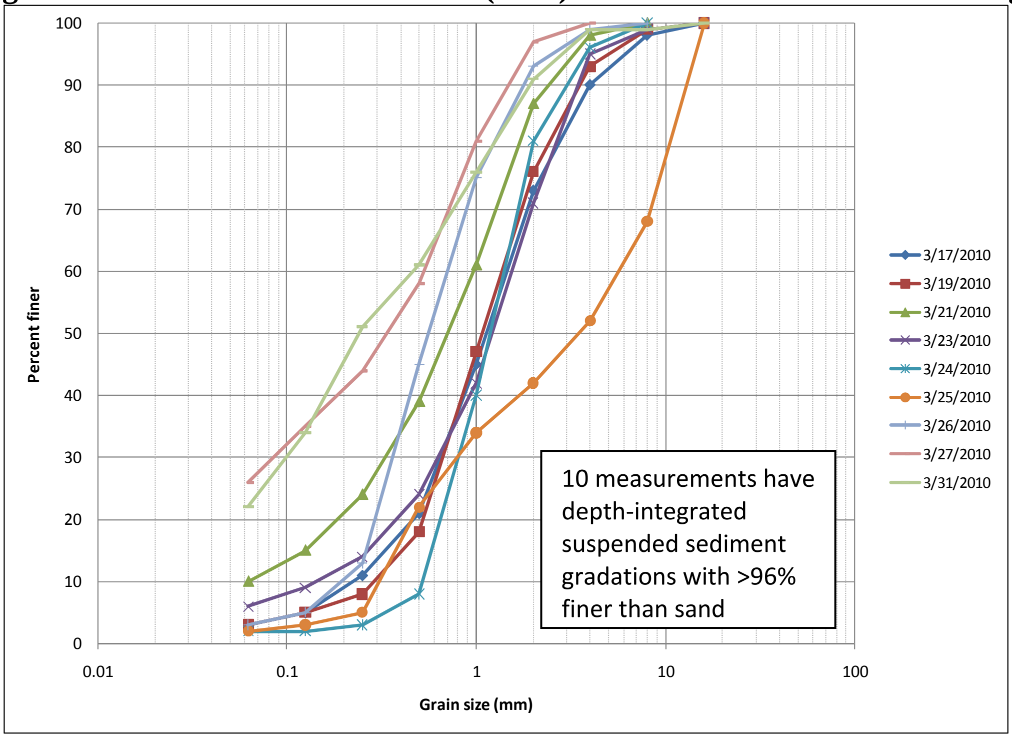


Figure 29 – Bed Sediment Gradation (*2011)* – Red River of the North at Fargo

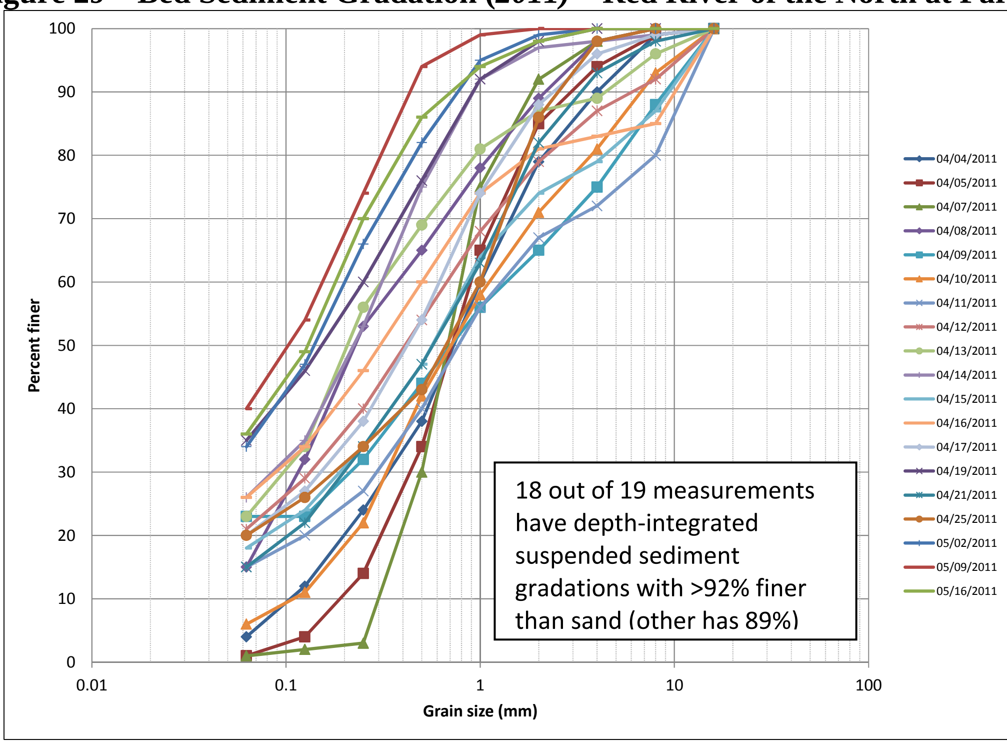


Figure 30 – Bedload Gradation (*2010*) – Red River of the North at Fargo

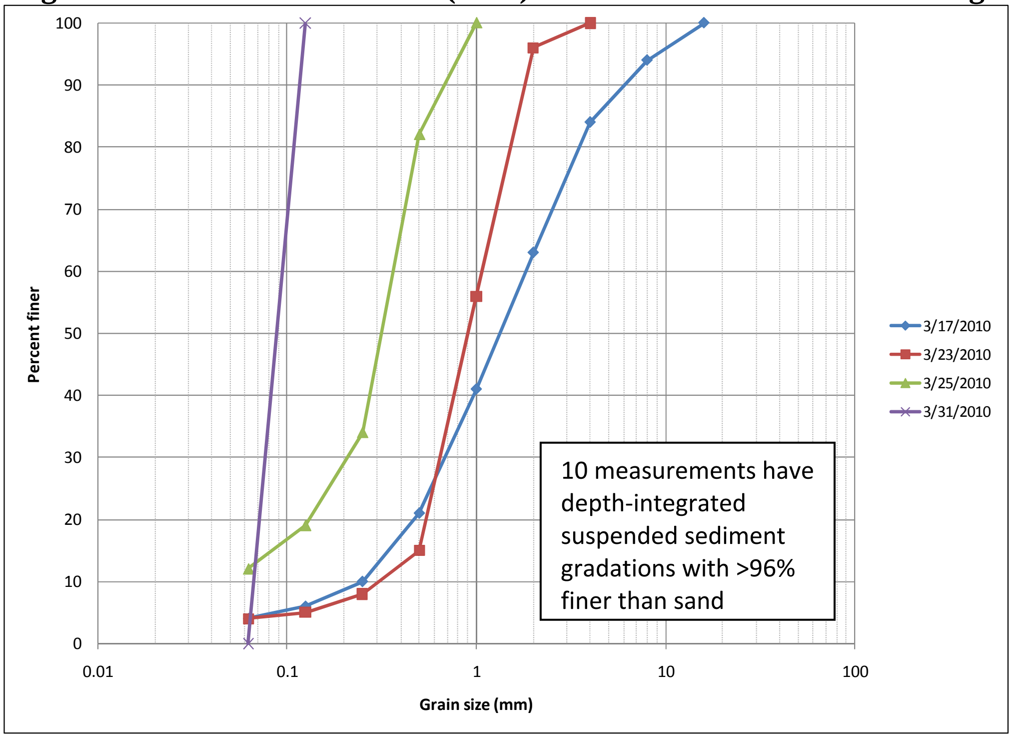
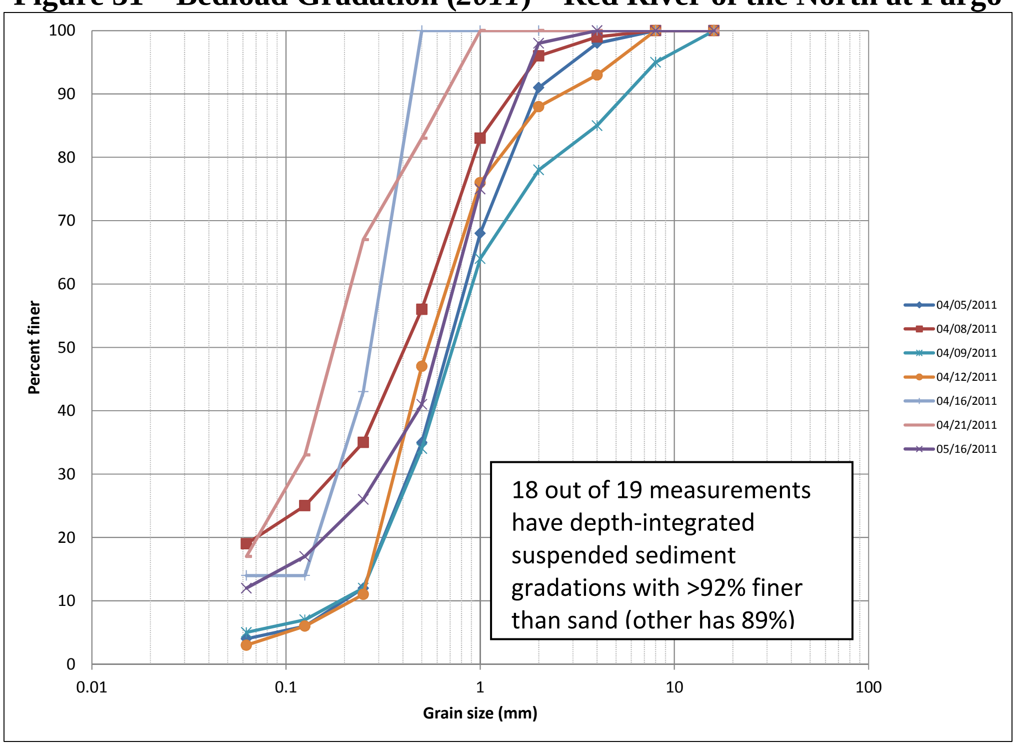


Figure 31 – Bedload Gradation (*2011*) – Red River of the North at Fargo

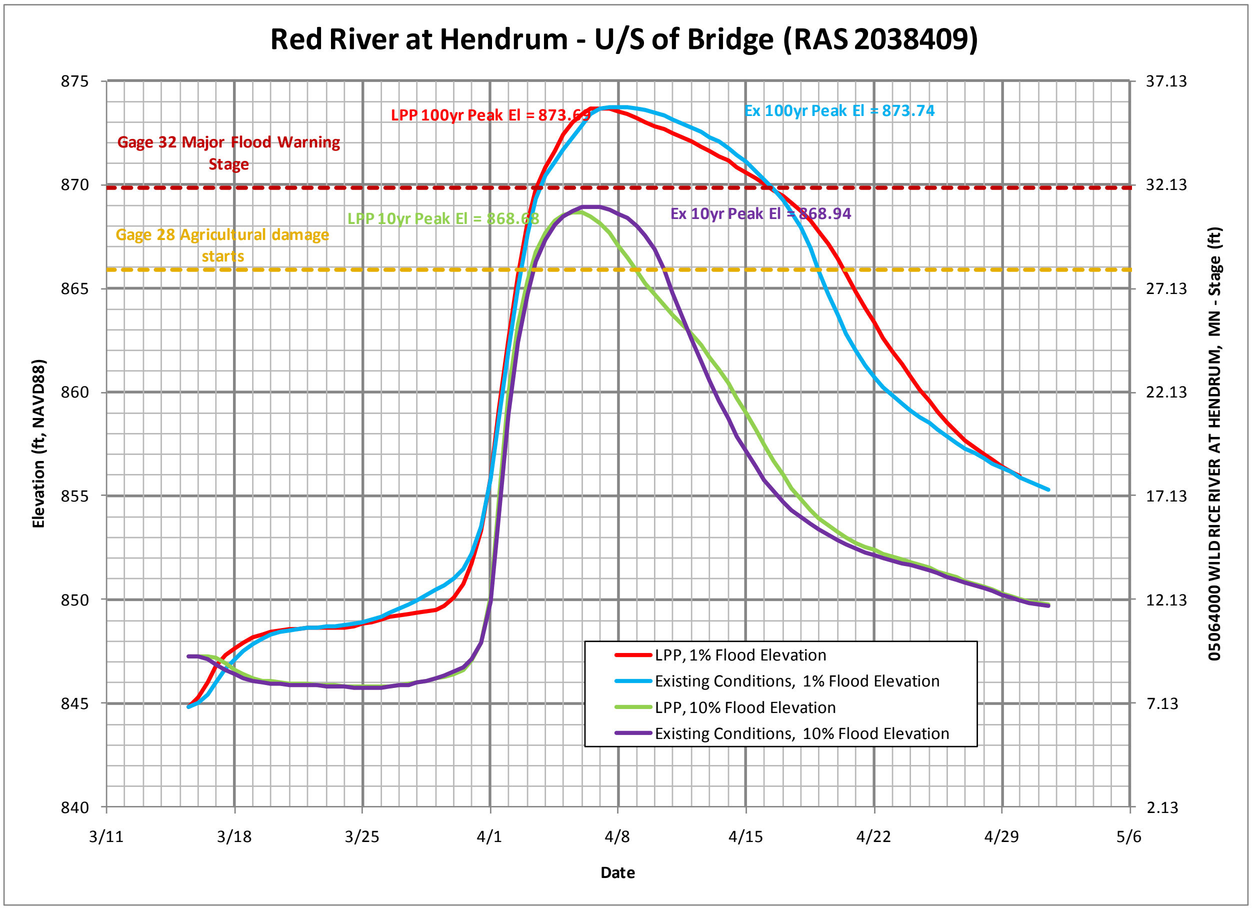


Attachment 4

LPP Hydrographs Downstream of Project

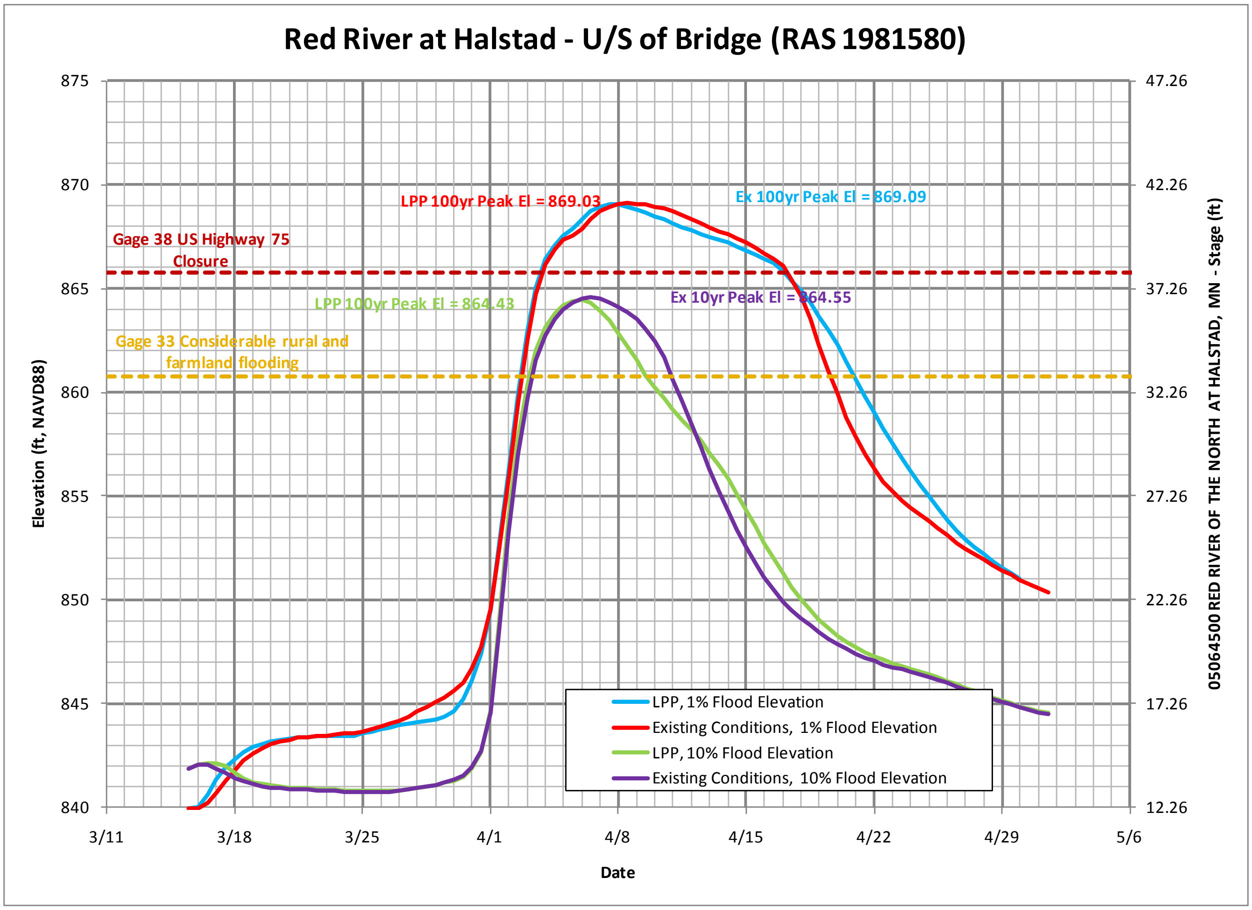
Flood Hydrograph Information Downstream of the LPP Project

1. Hendrum



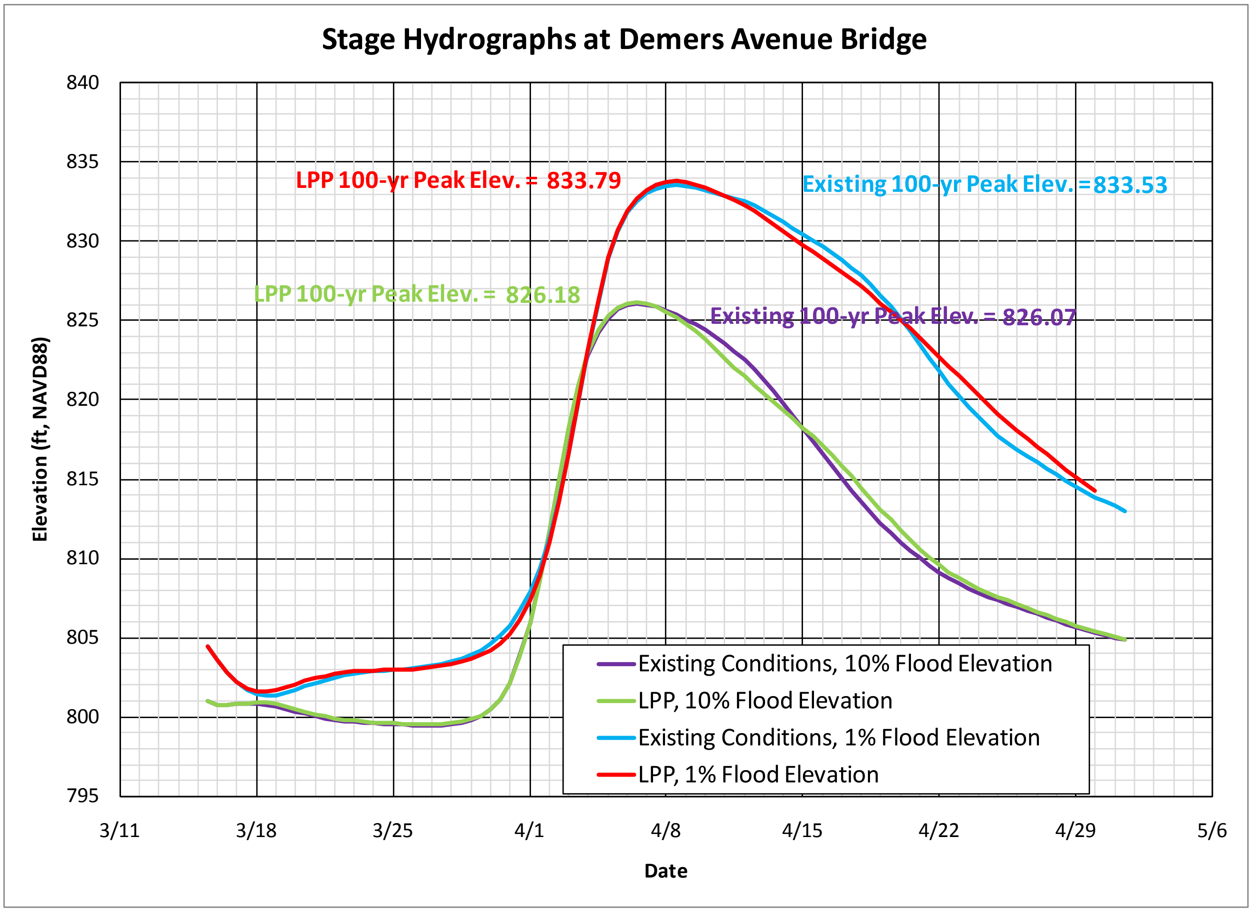
LPP 1% flood and 10% flood peak stages are slightly lower than they are for existing conditions. The duration of major flood stage for the 1% flood is no longer than it is for existing conditions, and the duration of the highest 2 to 3 feet of flooding is slightly less than for existing conditions. There is a slightly longer duration of flooding on lower land near the river for the 1% flood, but the duration of flooding on this lower land is somewhat less for the 10% flood.

2. Halstad



As is the case at Hendrum, the LPP 1% flood and 10% flood peak stages are slightly lower than they are for existing conditions. For the 1% flood, the duration of the highest stages is approximately the same as it is for existing conditions, and the duration is slightly shorter for lower land near the river. The duration of flooding on lower land near the river is shorter for the 10% flood.

3. Grand Forks



Stage hydrographs at the Demers Avenue bridge in Grand Forks for the 10% and 1% chance flood events show that the peak elevation is slightly higher with the LPP, but the duration of stages above elevation 819 for the 10% flood and above 825 for the 1% flood are somewhat less. While the hydrology and hydraulics used for this feasibility study are different from what was used for the Grand Forks' current DFIRM, they are sufficient to conclude that at Grand Forks the LPP will increase peak stages on the order of 0.1 to 0.3 ft for moderate and major floods and but will slightly reduce the duration of the highest stages. The higher stage might be a negative impact for any particular flood, but the shorter duration may be a benefit for any particular flood. The higher stage does not significantly reduce the level of flood risk reduction provided by the Grand Forks / East Grand Forks project and FEMA has stated that Flood Insurance Study updates will not be required for Polk County, MN and Grand Forks County, ND with the LPP.

Attachment 5

LPP Staging Area Wind/Wave Analysis

LPP Staging Area Wind/Wave Analysis

I. Wind Data

Data was collected from the NOAA National Climatic Data Center online resource in the form of Daily Surface Data. The weather station used for this study was the WBAN 14914 - Fargo Hector International Airport in Fargo, Cass County, ND. A wide range of wind data was available including the Fastest 5-second Wind Direction and Speed (F5SC), the Fastest 2-minute Wind Direction and Speed (F2MN), the Resultant Wind Direction and Speed (RDIR, RWND), as well as many others. The preferred method for obtaining a wave generating wind speed, as described in the Shore Protection Manual, is by converting the Fastest Mile Wind Direction and Speed (FSMI) to a longer duration. In the absence of available Fastest Mile Wind Direction and Speed for this location, the Fastest 2-minute Wind Direction and Speed was used instead. This data was collected for the past 10 years (2001-2010) and sorted by month and direction.

The current alignment of the diversion and tieback levees at the upstream staging area, oriented generally East to West, gives a South wind as the perpendicular wind speed. Waves perpendicular to the structure give higher overtopping rates than waves that hit the structure with an angle of incidence. In addition, the predominant wind direction (averaged over this 10 year period) is a SSW wind. For these reasons, wind speeds with directions between 135° and 225° from North were analyzed and summarized in the following table.

Summary of 322859 14914 FARGO HECTOR INTL AP Fastest 2-min Wind Speeds for South Winds (135 - 225 degrees) from 2001 through 2010 (in miles/hour)												
	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Min	9	9	8	10	10	8	9	8	10	9	8	9
Max	35	33	38	47	51	36	32	33	30	32	35	36
Avg	19	20	22	23	25	22	20	21	20	20	20	20

Average Max Monthly	37	mph
Average Max Mar-Aug	40	mph
Highest Max Monthly	51	mph

To account for a duration-averaged wind speed, the maximum 2-minute south wind over the 10 year period (51 mph) was taken to convert to a 1-hour wind speed. Using Figure 3-13 in the SPM, a 51 mph 2-minute wind converts to a 41.8 mph 1-hour wind. This wind speed compares closely with the average maximum Spring-Summer wind speed of 40 mph. A 1-hour design wind speed of 42 mph was chosen for calculations.

II. Fetch Delineation

Fetch delineation is a critical value for both wind setup calculations and for wave height development. In general, the staging area depths will be shallow, in the 5-15 ft depth range. This lower average depth for the staging area will tend to produce higher wind setup, or the tilting of the water surface in an enclosed basin caused by wind shear stress (EM 1110-2-1414 3-5). The simplified estimate for wind setup gives an inversely proportional relationship between setup and average depth.

$$S = U^2F / 1400d$$

where

S = setup relative to the SWL (ft)
U = wind speed (mph)
F = fetch (miles)
d = average water depth over fetch (ft)

This equation yields values for wind setup of 0.75 ft to 1.25 ft for fetches ranging from 3-4 miles and average depths from 5-10 feet

III. Wave Development

Shallow depths have the opposite effect on wave development. Extreme wave heights in coastal engineering applications are often limited by shallow-water depths. Depending on local water depth and wave climate, the distribution of significant waves can be expected to be limited to 0.6 times the water depth (EM 1110-2-1100 II-8-25). Submerged roadways along the fetch profile create this limiting effect on wave development. Wave development profiles were generated for various fetches to see if the roadways caused a significant limiting effect on wave development. The limiting effect was found to be minor, and water depth actually increases closer to the structure, so depth-limiting of waves was ignored.

With depth-limiting ignored, the assumption for simplified wave prediction is that the waves are fetch-limited rather than duration limited. Fetch-limited waves assume the wind blows in essentially a constant direction for sufficient time to achieve steady-state waves. The time required for waves crossing an average fetch length of 3.5 miles with winds of 40-50 mph is greater than one hour (CEM Eq. II-2-35). Conservatively, the 1-hour wind speed of 42 mph will be used for wave development. Using various unique single wind-fetches, a range of wave heights and wave periods were developed using the equations found in the CEM Chapter II, Part 2. The wave heights range from 2.4 ft to 2.9 ft and wave periods range from 2.5 seconds to 2.9 seconds. These values, calculated from equations found in the CEM, are slightly more conservative than those calculated using the Automated Coastal Engineering System (ACES) software.

IV. Required Freeboard from Overtopping and Wind Setup

Using the overtopping formula developed by van der Meer and Janssen (CEM Eq. VI-5-42) with a grass levee slope of 1V:4H and an allowable overtopping value of 0.1 cfs/ft, overtopping freeboard is less than 2 ft for all the selected wind fetches. The average ultimate freeboard (overtopping freeboard plus wind setup) is 2.50 ft with a maximum ultimate freeboard of 2.75 ft. ***It is recommended that a minimum value of 3 feet of freeboard be considered in the design of all levees in the vicinity of the staging area.***