RED RIVER DIVERSION

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

APPENDIX B –HYDRAULICS EXISTING CONDITION

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

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B1.0 BACKGROUND AND OVERVIEW

The purpose of this study is to evaluate potential impacts from flood mitigation alternatives being considered as part of the U.S. Army Corps of Engineers (USACE) Fargo-Moorhead Metro Feasibility Study, Phase 4. This includes the Minnesota Diversion alternative (Federally Comparable Plan – FCP) and the North Dakota Diversion alternative (Locally Preferred Plan - LPP). To complete this analysis, an unsteady HEC-RAS model (v. 4.1) was created of the Red River of the North (RRN) and tributaries in the vicinity of Fargo, North Dakota and Moorhead, Minnesota.

The model was developed with sufficient detail to be used as a baseline for project design as well as benefit and impact analysis. The original model, completed after Phase 2 of the feasibility study, extended downstream to near Halstad, MN on the Red River. To further evaluate project impacts, the model was extended farther downstream to near Thompson, ND as part of Phase 3 of the feasibility study and to near Drayton, ND as part of Phase 4 of the feasibility study. The model was calibrated based on the 2009 spring flood. The 2006, 1997, and 2010 spring flood events were also created to verify the calibrated model. The 2006 flood event served as the pattern hydrograph for the synthetic flood events. The 10-, 2-, 1-, and 0.2-percent annual chance synthetic flood events were developed as the primary means to evaluate existing conditions, assist with project design, and to analyze potential impacts from flood mitigation alternatives being considered as part of the Fargo-Moorhead Metro Feasibility Study, Phase 4, including the FCP and LPP alternatives. This report updates previous versions of Appendix B unsteady Hydraulic Modeling dated as follows:

- August 31, 2009 – Draft
- October 1, 2009 – Revised Draft
- January 27, 2010
- February 12, 2010
- April 6, 2010 (Phase 2)
- July 30, 2010 (Phase 3)
- January 31, 2011 (Phase 4)
- February 28, 2011 (Phase 4)

B2.0 STUDY AREA

The hydraulic analysis completed for the Fargo-Moorhead Metro Feasibility Study spans approximately 325 miles of the Red River from Abercrombie, North Dakota through Fargo, North Dakota and Moorhead, Minnesota to the downstream end at Drayton, North Dakota. The City of Fargo is located in Cass County, North Dakota. The City of Moorhead is located in Clay County, Minnesota. Both communities are adjacent to the Red River of the North which forms the border between Minnesota and North Dakota. The communities are located approximately 453 river miles above the mouth of the Red River of the North at Lake Winnipeg, Manitoba.

The study area is highlighted in Figure B1. It includes the Red River of the North main stem and several tributaries. The Phase 2 study area originally extended north to River Mile 375 at Halstad, Minnesota. However, after failing to fully define downstream impacts (zero impact location) within the original study extents, the model was extended to River Mile 316 near Thompson, North Dakota (Phase 3). For Phase 4 of the feasibility study, the model was extended further downstream on the Red River to near Drayton, ND at approximately River Mile 198 and upstream on the Red River to near Abercrombie 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 for these rivers. The downstream boundary locations in each of the phases (Halstad, Thompson, and Drayton) are at USGS stream gages as referenced in Appendix A – Hydrology.

Modeled tributaries to the Red River are the Wild Rice, Sheyenne, Maple, Elm, and Goose Rivers; Rose Coulee; and Cass County Drains 14, 27, 34, 53, and Richland County Drain 37 on the North Dakota side, and the Buffalo, Wild Rice, Marsh, Sandhill, and Red Lake Rivers; Heartsville Coulee; and Wolverton Creek on the Minnesota side as outlined below:

- The Wild Rice River, ND from its junction with the RRN upstream to USGS Gage 05053000 near Abercrombie, ND (River Mile 42.8).

- The Sheyenne River from its junction with the RRN upstream past the City of Kindred to the Gol Bridge (River Mile 75.2), including the West Fargo and Horace to West Fargo Diversion Channels.

- The Maple River from the confluence with the Sheyenne River to near Durbin, ND (River Mile 32.4).

- The Elm River from its confluence with the Red River to near Grandin, ND (River Mile 14.2) and the North Branch of the Elm River from its confluence with the Elm River to near Kelso, ND (River Mile 19.3). This model reach is for routing purposes only and does not contain channel bathymetry or hydraulic structures.

- The Goose River from its confluence with the Red River to USGS Gage 05066500 near Hillsboro, ND (River Mile 28.3). This model reach is for routing purposes only and does not contain channel bathymetry or hydraulic structures.

- Rose Coulee, Cass County Drain 27 (6.0 river miles), and Cass County Drain 53 (1.75 river miles) in south Fargo.

- Cass County Drain 14 (17.7 river miles), Cass County Drain 34 (7.9 river miles) and Richland County Drain 37 (12.1 river miles).

- The Buffalo River from its junction with the RRN upstream to USGS Gage 05062000 near Dilworth, MN (River Mile 34.7).

- The Wild Rice River, MN from its junction with the RRN upstream to USGS Gage 05064000 at Hendrum, MN (River Mile 7.8).

- The Marsh River from its junction with the RRN upstream to USGS Gage 05067500 near Shelly, MN (River Mile 14.4). This model reach is for routing purposes only and does not contain channel bathymetry or hydraulic structures.

- The Sandhill River from its junction with the RRN upstream to USGS Gage 05066500 near Climax, MN (River Mile 3.6). This model reach is for routing purposes only and does not contain channel bathymetry or hydraulic structures.

- The Red Lake River from its junction with the RRN upstream to USGS Gage 05080000 near Fischer, MN (River Mile 28.0).

- Heartsville Coulee from its junction with the Red Lake River to its junction with the Red River (River Mile 10.8)

- Wolverton Creek from its junction with the Red River to U.S. Highway 75 (River Mile 2.7).

B3.0 MODEL GEOMETRY DEVELOPMENT

B3.1 DATUMS

The model has been geo-referenced with the horizontal datum of NAD 1983 UTM Zone 14N, Foot_US. The vertical datum of the model is the North American Vertical Datum of 1988 (NAVD 1988).

B3.2 MODEL GEOMETRY CRITERIA

The unsteady HEC-RAS model geometry was developed by combining geometry from existing unsteady and steady state models with new geometry developed for the project. The following specific criteria were included in the Scope of Work (SOW) for this project:

- Overbanks for channel cross sections were based on the Red River Basin LiDAR collect (Reference 8), or other LiDAR collects as noted in Table B1.

- Channel bathymetry for the reach from River Mile 440.0 to 470.2 was based on RRN soundings that were obtained for Phase 1 of this study. For areas outside the reach defined above, the channel bathymetry was based on the cross sections from existing HEC-RAS and HEC-2 models.

- The model geometry defines the effective flow limits between the river overbanks and the storage areas.

- In the rural areas, storage areas were generally defined as one square mile sections with the storage volume relationship based on the LiDAR data. However, larger storage areas were used where it was deemed appropriate using engineering judgment.

- In the urban areas, the storage areas were based on controlling road profiles and other topographic features with the storage volume relationship based on the most recent LiDAR data.

- Lateral structures and storage area connections have been defined as weirs following section line roads or other controlling roads or topographic features. Road profiles were obtained from the LiDAR data where more detailed information is not available. Detailed surveys were used where available. The use of detailed surveys was limited and generally occurred where existing HEC-RAS model geometry was incorporated into the unsteady HEC-RAS model geometry.

- Culverts three feet and larger in diameter have been included in the lateral structures and storage area connections, where appropriate. For most areas, the culvert locations and size were determined using field reconnaissance for this project. Culvert inverts were approximated using LiDAR data and road elevations and estimates of the cover on the culverts. When surveyed culvert data or data from construction drawings was available, this information was used. Table B1 identifies the level of detail that was utilized for the storage area connections for the various model reaches.

- Time constraints and the large scale of the model expansion did not allow field evaluation of culverts throughout the entire system. Culverts between Perley and Drayton were estimated and separated into one of two categories, small and large. To identify the assumed culverts in the model, the culverts were modeled with unique sizes. Small culverts were modeled as 2.9’ x 2.9’ box culverts and large culverts were set as 5.9’ x 5.9’ box culverts. The small culverts were used to represent typical ditch and field swale conveyance and the large culverts were used to represent larger, more concentrated ditch and drain conveyance.

B3.3 SOURCES OF HEC-RAS GEOMETRY DATA

Where available, geometry from existing models was used to develop the HEC-RAS model geometry for this study. Table B1 summarizes the model extent for the various streams along with the sources for the model geometry. The primary source for cross section and structure data was a combination of existing hydraulic models, bridge plans, and field survey data from previous projects. The primary source for overbank cross section and storage area geometry was the Red River Basin LIDAR data (Reference 8).

Figure B2 shows the combined HEC-RAS model geometry. Figures B3-B18 show the model geometry in greater detail.

Table B1 - Sources of HEC-RAS Geometry Data

Stream	Extent	River Miles	Cross Section Reach	Cross Section and Hydraulic Structure Data Source	Overbank Cross Section and Storage Area Data Source	Level of Detail/Quality of Source Data (High, Medium, Low)
Red River of the North	Abercrombie, ND to Hickson, ND	485.6 to 523.6	2563754 to 2764835	USACE Regional Red River Flood Assessment (Reference 1)	IWI LIDAR (Reference 8)	Low - Utilized channel/structure data from existing USACE hydraulic model supplemented with LIDAR. No structure verification and limited storage areas/connections.
Red River of the North	Hickson, ND to South Fargo, ND	459.2 to 485.6	2424705 to 2563754	Southern Cass County, ND/Clay County, MN FIS (Reference 14)	IWI, City of Fargo, and Southern Cass FIS LIDAR (References 8, 9, and 10)	High - Utilized existing channel/structure data from recent FIS supplemented with LIDAR. Hydraulic structures are current. Culvert inventory utilized for storage connection culverts.
Red River of the North	South Fargo, ND to Perley, MN	403.3 to 459.2	2129283 to 2424705	City of Fargo FIS (Reference 2)	IWI and City of Fargo LIDAR (References 8 and 9)	High - Utilized existing channel/structure data from recent FIS supplemented with LIDAR. Hydraulic structures are current. Culvert inventory utilized for storage connection culverts.
Red River of the North	Perley, MN to Thompson, ND	315.8 to 403.3	1667665 to 2129283	USACE Regional Red River Flood Assessment (Reference 1)	IWI LIDAR (Reference 8)	High/Medium - Utilized existing channel/structure data supplemented with LIDAR. Hydraulic structures are current. Culvert inventory utilized for storage connection culverts for model reach through Halstad, MN. Connection culverts were estimated in Halstad to Thompson reach.
Red River of the North	Thompson, ND to Drayton, ND	197.8 to 315.8	1044619 to 1667665	USACE Regional Red River Flood Assessment (Reference 1)	IWI LIDAR (Reference 8)	Medium - Utilized existing model geometry supplemented with LIDAR. No verification of hydraulic structures. Storage connection culverts were estimated and larger storage cells were used.
ND Wild Rice River	Confluence w/ Red River to USGS Gage 05053000 near Abercrombie, ND	0 to 42.8	460 to 225847	NRCS Floodplain Management Study (Reference 13)	IWI LIDAR (Reference 8)	High - Channel/structure data from existing FIS model. Hydraulic structures have been updated. Most storage connection culverts were inventoried.
Sheyenne River	Confluence w/ Red River to West Fargo Diversion Outlet	0 to 23.6	544 to 124560	Reed Township and City of Harwood FIS (References 15 and 16)	IWI LIDAR (Reference 8)	Medium - Channel/structure data is from older FIS models. No updates were made to hydraulic structures. Most storage connection culverts were inventoried.
West Fargo Diversion/Sheyenne River	Diversion outlet to Horace	23.6 to 42.1	124560 to 222137.2	USACE HEC-2 models	IWI LIDAR (Reference 8)	Medium - Cross Sections cut from LIDAR, hydraulic structure data taken from design plans.
Horace to West Fargo Diversion	Confluence w/ West Fargo Diversion to Horace	0 to 7.2	530.8 to 37976.6	USACE HEC-2 models	IWI LIDAR (Reference 8)	Medium - Cross Sections cut from LIDAR, hydraulic structure data taken from design plans.
Sheyenne River	Horace to Gol Bridge	42.1 to 75.2	222137.2 to 397223	Pacific International Engineering Sheyenne/Maple River FIS (Reference 3)	IWI LIDAR (Reference 8)	Medium - PIE Model geometry was updated by the USACE - St. Paul District. Cross Sections cut from LIDAR, Channel bathymetry and hydraulic structure data was utilized from PIE Model. Storage area culvert connections are from previous surveys.
Richland County Drain 37	Confluence w/ Wild Rice River to near Kindred, ND	0 to 12.1	167 to 53969	Pacific International Engineering Sheyenne/Maple River FIS (Reference 3)	IWI LIDAR (Reference 8)	Medium - PIE Model geometry was updated by the USACE - St. Paul District. Cross Sections cut from LIDAR, Channel bathymetry and hydraulic structure data was utilized from PIE Model. Storage area culvert connections are from previous surveys.
Cass County Drain 14	Confluence w/ Maple River to Davenport, ND	0 to 17.7	84.1 to 93627	Pacific International Engineering Sheyenne/Maple River FIS (Reference 3)	IWI LIDAR (Reference 8)	Medium - PIE Model geometry was updated by the USACE - St. Paul District. Cross Sections cut from LIDAR, Channel bathymetry and hydraulic structure data was utilized from PIE Model. Storage area culvert connections are from previous surveys.
Cass County Drain 34	Confluence w/ Drain 14 to ND Highway 46 near Kindred, ND	0 to 7.9	79.6 to 41500	Pacific International Engineering Sheyenne/Maple River FIS (Reference 3)	IWI LIDAR (Reference 8)	Medium - PIE Model geometry was updated by the USACE - St. Paul District. Cross Sections cut from LIDAR, Channel bathymetry and hydraulic structure data was utilized from PIE Model. Storage area culvert connections are from previous surveys.
Maple River	Confluence w/ Sheyenne River to Durbin, ND	0 to 32.4	384.8 to 170982	Pacific International Engineering Sheyenne/Maple River FIS (Reference 3)	IWI LIDAR (Reference 8)	Medium - PIE Model geometry was updated by the USACE - St. Paul District. Cross Sections cut from LIDAR, Channel bathymetry and hydraulic structure data was utilized from PIE Model. Storage area culvert connections are from previous surveys.

Table B1 - Sources of HEC-RAS Geometry Data

Stream	Extent	River Miles	Cross Section Reach	Cross Section and Hydraulic Structure Data Source	Overbank Cross Section and Storage Area Data Source	Level of Detail/Quality of Source Data (High, Medium, Low)
Elm River	Confluence w/ Red River to Grandin, ND	0 to 14.2	1068 to 74815	No Cross Section or Structure Data included - Model developed for hydrograph routing	IWI LIDAR (Reference 8)	Low - Utilized LIDAR only. No channel bathymetry of hydraulic structures were included.
North Branch of the Elm River	Confluence w/ Elm River to Kelso, ND	0 to 19.3	912 to 102090	No Cross Section or Structure Data included - Model developed for hydrograph routing	IWI LIDAR (Reference 8)	Low - Utilized LIDAR only. No channel bathymetry of hydraulic structures were included.
Goose River	Confluence w/ Red River to USGS Gage 05066500 near Hillsboro, ND	0 to 28.3	623 to 149399.8	No Cross Section or Structure Data included - Model developed for hydrograph routing	IWI LIDAR (Reference 8)	Low - Utilized LIDAR only. No channel bathymetry of hydraulic structures were included.
Rose Coulee/Cass County Drain 27	Confluence w/ Red River to near 52nd Avenue South	0 to 6.0	1217 to 31783	City of Fargo Southside Flood Control Interior Drainage Study (Reference 5)	IWI LIDAR (Reference 8)	High - Utilized unsteady model geometry from the Fargo Southside Flood Control. Cross section and hydraulic structure data is current. Storage area culvert connections were inventoried.
Cass County Drain 53	Confluence w/ Rose Coulee to 64th Avenue South	0 to 1.8	525 to 9250	City of Fargo Southside Flood Control Interior Drainage Study (Reference 5)	IWI LIDAR (Reference 8)	High - Utilized unsteady model geometry from the Fargo Southside Flood Control. Cross section and hydraulic structure data is current. Storage area culvert connections were inventoried.
Buffalo River	Confluence w/ Red River to USGS Gage 05062000 near Dilworth, MN	0 to 34.7	0 to 182958	Buffalo River Flood Insurance Study (Reference 7)	IWI LIDAR (Reference 8)	High - Utilized geometry from existing steady-state HEC-RAS model. Channel cross sections and hydraulic structures are current. Storage connection culverts have been inventoried.
Wolverton Creek	Confluence w/ Red River to U.S. Highway 75	0 to 2.7	1357 to 14321	Southern Cass and Clay County FIS (Reference 14)	IWI LIDAR (Reference 8)	High - Utilized geometry from existing steady-state HEC-RAS model. Channel cross sections and hydraulic structures were surveyed as part fo FIS
MN Wild Rice River	Confluence w/ Red River to USGS Gage 05064000 near Hendrum, MN	0 to 7.8	460 to 41276	Wild Rice River Watershed District Model (Reference 6)	IWI LIDAR (Reference 8)	High - Utilized geometry from existing unsteady HEC-RAS model. Channel cross sections and hydraulic structures are current.
Marsh River	Confluence w/ Red River to USGS Gage 05067500 near Shelly, MN	0 to 14.4	519 to 75831	No Cross Section or Structure Data included - Model developed for hydrograph routing	IWI LIDAR (Reference 8)	Low - Utilized LIDAR only. No channel bathymetry of hydraulic structures were included.
Sandhill River	Confluence w/ Red River to USGS Gage 05069000 near Climax, MN	0 to 3.6	500 to 18847.9	No Cross Section or Structure Data included - Model developed for hydrograph routing	IWI LIDAR (Reference 8)	Low - Utilized LIDAR only. No channel bathymetry of hydraulic structures were included.
Heartsville Coulee	Red River to Red Lake River	0 to 10.8	416 to 57121	USACE Flood Reduction Study for Grand Forks/East Grand Forks (Reference 18)	IWI LIDAR (Reference 8)	Medium - Utilized existing model geometry supplemented with LIDAR. No verification of hydraulic structures. Storage connection culverts were estimated and larger storage cells were used.
Red Lake River	Confluence w/ Red River to USGS Gage 05080000 near Fisher, MN	0 to 28.0	1395 to 147781	USACE Flood Reduction Study for Grand Forks/East Grand Forks (Reference 18)	IWI LIDAR (Reference 8)	Medium - Utilized existing model geometry supplemented with LIDAR. No verification of hydraulic structures. Storage connection culverts were estimated and larger storage cells were used.

B3.4 MODEL NOMENCLATURE

The cross section naming convention used for the HEC-RAS model is based on the distance in feet above the mouth of the stream. Specifically, cross sections on the Red River are referenced above its outlet into Lake Winnipeg. Tributary cross sections are referenced in distance above their confluence with the Red River. Model reaches are based on abbreviations of the stream name and common landmarks or tributaries. Storage areas (SA) and storage connections (SC) are generally referenced to the nearest stream (WRR = Wild Rice River) or town (Fgo = Fargo). There are some exceptions to this naming convention, such as near portions of the Sheyenne and Maple Rivers where no naming convention was used.

B3.5 MODEL GEOMETRY PARAMETERS AND CALIBRATION

The HEC-RAS model was calibrated to the 2009 spring flood event using high water mark and gage data obtained from city, county, and federal agencies. The 2009 spring flood event was chosen for the calibration event because it was the flood of record and was well documented by high water marks and stream gage data. The original model (Phase 2) extended downstream to near Halstad, MN on the Red River. To further evaluate project impacts, the model was extended farther downstream to near Thompson, ND (Phase 3) and Drayton, ND (Phase 4). Emergency flood protection measures, as highlighted in Figure B19 were added to the model geometry as levees for calibration. Calibration was accomplished by adjusting Manning's "n" values, bank stations, overbank reach lengths, and ineffective flow limits based on aerial photographs taken during the 2009 and 1997 floods. Figure B20 shows the calibrated water surface profile for the Red River during the 2009 flood. It includes the peak stages along with high water marks. The model was verified using the spring floods of 2006, 1997, and 2010. Figures B20-B23 show the verification model water surface profiles for the Red River during the 2006, 1997 and 2010 floods. Figures B37-B40 show the locations of high water marks from the 2009, 2006, 1997, and 2010 floods that were utilized for model calibration and verification. High water marks and stream gage data is generally limited to the Red River main stem and major tributaries and is not available for storage connections and breakout reaches. This limited our ability to calibrate and verify the flow through these areas, however, the sensitivity analysis that was performed (see Exhibit F) shows the model results and impacts are not overly sensitive to shifts in discharge and timing between tributaries.

3.5.1 <u>Boundary Conditions</u>

Starting water surface elevations were obtained from rating curves based on high water data; USGS rating curves at Drayton, ND; and the results of the USACE Regional Red River Flood Assessment Report (Reference 1).

3.5.2 <u>Manning's "n" Values</u>

Manning's "n" values were primarily taken from the source models used for the unsteady model development. They were verified using aerial photos and land use GIS data. Some adjustment to the Manning's "n" values was done during model calibration. Table B2 summarizes the Manning's "n" values used for the various stream reaches:

Table B2 – Manning's "n" values

Watercourse	Channel	Overbank
Red River of the North	0.045	0.045 – 0.16
Wild Rice River (ND)	0.035-0.045	0.05-0.13
Sheyenne River	0.035-0.052	0.04-0.10
Richland County Drain 37	0.04	0.04
Cass County Drains 14 and 34	0.035	0.04
Maple River	0.04	0.04-0.08
Elm River and North Branch Elm River *	0.045	0.09
Goose River *	0.045	0.07-0.12
Rose Coulee/Cass County Drains 27 and 53	0.035-0.045	0.04-0.06
Buffalo River	0.04	0.065-0.135
Wolverton Creek	0.045	0.08
Wild Rice River (MN)	0.045	0.09-0.11
Marsh River *	0.045	0.07-0.12
Sandhill River *	0.045	0.12
Red Lake River	0.045	0.055-0.12
Heartsville Coulee	0.012-0.07	0.045-0.12

*Model reaches are for routing purposes and do not contain channel bathymetry.

3.5.3 Weir Coefficients

After a detailed sensitivity analysis (Exhibit F) and discussions with the Hydrologic Engineering Center (HEC) and the project team, weir coefficients were set to 1.0 for storage connections and lateral structures to better represent the conveyance of flow through the storage areas. A weir coefficient memorandum prepared as part of the sensitivity is included in Exhibit G. Some connections and lateral structures had frequently overtopping weirs with very low elevations at or near stream crossings. The low weirs caused instabilities in the model due to excessive conveyance. Therefore, a weir coefficient of 0.5 was used to provide a more reasonable and stable discharge across the weir. Inline structure weir coefficients maintained the model default value of 2.6.

3.5.4 Calibration Tolerances

The models were generally calibrated to a tolerance of within one-half foot of the 2009 spring flood high water marks which matches FEMA's criteria for hydraulic model calibration (Reference 19). Exceptions included the Wild Rice River, ND area and the Oakport, MN area where this tolerance could not be met. This is attributed to the unique partial ice cover conditions and breakout flows that occurred in these areas during the 2009 spring flood event. Exhibits A, B, C, and D contain discharge hydrograph comparisons for select locations from the 2009 flood calibration, and the 2006, 1997, and 2010 spring flood verifications, respectively. Similar to the 2009 flood calibration event, the 2006, 1997, and 2010 flood verification events generally met the calibration tolerance of matching within one-half foot of measured high water marks.

3.4.5 Unsteady Flow Analysis Settings and Parameters

The simulation time of the synthetic models start 15MAR2006 and end 01MAY2006. 2006 was used since the balanced hydrographs were patterned after the 2006 flood event. The historic event start date varied because of the variability in crest date depending on the event. Typically the historic models began 10 days before the respective crest and ran for 1.5 months. The Hydrograph Output Interval and Detailed Output Intervals were both set at 12 Hours. The computation interval began with a warm up time step of 30 seconds and had an actual model run time computation of 5 minutes. A sensitivity analysis was conducted to evaluate model results with a shorter computation interval. A 1 minute interval was used for this. The model took significantly longer to run, however the output results were within 0.01 feet of the results with a 5 minute interval. Due to the complexity of the model, the Water Surface Calculation Tolerance was changed from the default of 0.02 feet to 0.03 feet. Also, the Storage Area Elevation Tolerance was changed from the default of 0.05 feet to 0.1 feet. A sensitivity analysis was conducted to determine the effects of the Storage Area Elevation Tolerance change. It was set back to the default and re-computed. The model produced many “failure to converge” warnings while using the smaller tolerance (0.05’). After a long computation run time, the resulting water surface elevation error averaged 0.07 feet (with the smaller tolerance). This was very similar to the error with the setting at 0.1 feet. Therefore, the Storage Area Elevation Tolerance used in the model project seemed reasonable. The Lateral Structure flow stability factor was changed from the default of 2 to 3. The Gate flow submergence decay exponent was changed from the default of 1 to 3. Table 3 shows the varying parameters in tabular form.

Table B3 – HEC-RAS Unsteady Computation and Option Tolerances

Unsteady Flow Options	Default	Model
Theta (implicit weighting factor) (0.6-1.0)	1	1
Theta for warm up (implicit weighting factor) (0.6-1.0)	1	1
Water surface calculation tolerance (ft)	0.02	0.03
Storage Area elevation tolerance (ft)	0.05	0.1
Flow calculation tolerance (optional) (cfs)	n/a	n/a
Maximum number of iterations (0-40)	20	20
Number of warm up time steps (0-200)	0	50
Time step during warm up period (hrs)	0	0.00833
Minimum time step for time slicing (hrs)	0	0
Maximum number of time slices	20	20
Lateral Structure flow stability factor (1.0-3.0)	2	3
Inline Structure flow stability factor (1.0-3.0)	1	1
Weir flow submergence decay exponent (1.0-3.0)	1	1
Gate flow submergence decay exponent (1.0-3.0)	1	3
DSS Messaging Level (1 to 10, Default = 4)	4	4
Maximum error in water surface solution (Abort Tolerance)	100	100

B4.0 MODEL HYDROGRAPH DEVELOPMENT

The initial phase of this feasibility study was completed for the U.S. Army Corps of Engineers and cities of Fargo, ND and Moorhead, MN in August, 2009. The study utilized Phase 1 hydrology developed by the U.S. Army Corps of Engineers and a steady-state HEC-RAS model for the design of project alternatives including diversion channels and hydraulic structures. Similarly, the Phase 2 and Phase 3 studies utilized the steady-state HEC-RAS model for the design of project features. The Phase 2 and Phase 3 studies also utilized the unsteady HEC-RAS model to evaluate project impacts associated with the proposed diversion alternatives related to the loss of floodplain storage and changes to timing as a result of the proposed diversion channels. Phase 4 of the feasibility study utilizes the unsteady HEC-RAS model for project design and impact evaluation due to the need to consider the staging and storing of water to mitigate project impacts associated with the Locally Preferred Plan (LPP). This model hydrograph development narrative will reference examples from the 2009 historic flood event since it was used for primary calibration. Descriptions of the unique characteristics of the 2009 event and the additional 1997, 2006, and 2010 historic flood verification events will follow in subsequent sections. The methodology used for matching balanced hydrographs for the synthetic events is described in Section B4.4 of this report.

B4.1 UNSTEADY MODEL INFLOWS

Model inflows for the unsteady HEC-RAS model consist of nearly 80 inflow hydrographs including upstream boundary condition inflow hydrographs, lateral hydrographs, and uniform lateral hydrographs. Some originate at USGS gage locations, others are ungaged local inflows and a few are base flows required to maintain model stability. The hydrograph development procedures used for historic events and synthetic events are similar. An inflow hydrograph was inserted at the upper boundary condition of each river reach and intermediate hydrographs were added to help match the target hydrographs. USGS observed hydrographs along the Red River were matched for the historic events and balanced hydrographs were the targets for synthetic events. Additional balanced hydrograph explanation is provided in Section 6 of Appendix A-Hydrology. The observed USGS gage locations and balanced hydrograph locations along the Red River are shown in Figure B24.

B4.2 UPSTREAM FLOW HYDROGRAPHS

The unsteady HEC-RAS model geometry was extended upstream on the Red River and upstream on most of the tributaries to locations with input data from USGS stream gages. This provides the model with sufficient upstream boundary condition inflow data with the exception of those areas where breakouts occur near the upstream gages (discussed in detail in subsequent sections of this report). The following are upstream boundary condition inflow hydrograph locations along with their respective modeling methodologies:

4.2.1 Red River

The original model (Phase 2 and 3) extended upstream to USGS gage 05051522 at Hickson, ND (XS 2563754). The drainage area here is approximately 4,300 square miles. Hickson was a sufficient upstream boundary until the upstream staging feature was added to the LPP in Phase 4. It was then necessary to extend the model an additional 38 river miles upstream to provide geometry for the staging volume and upstream boundary condition. The upstream boundary of the Red River portion of the model is now at cross section 2764835, adjacent to USGS Gage 05053000 on the Wild Rice River at Abercrombie, ND. The next upstream gage on the Red River is at Wahpeton, ND, approximately 25 miles further upstream.

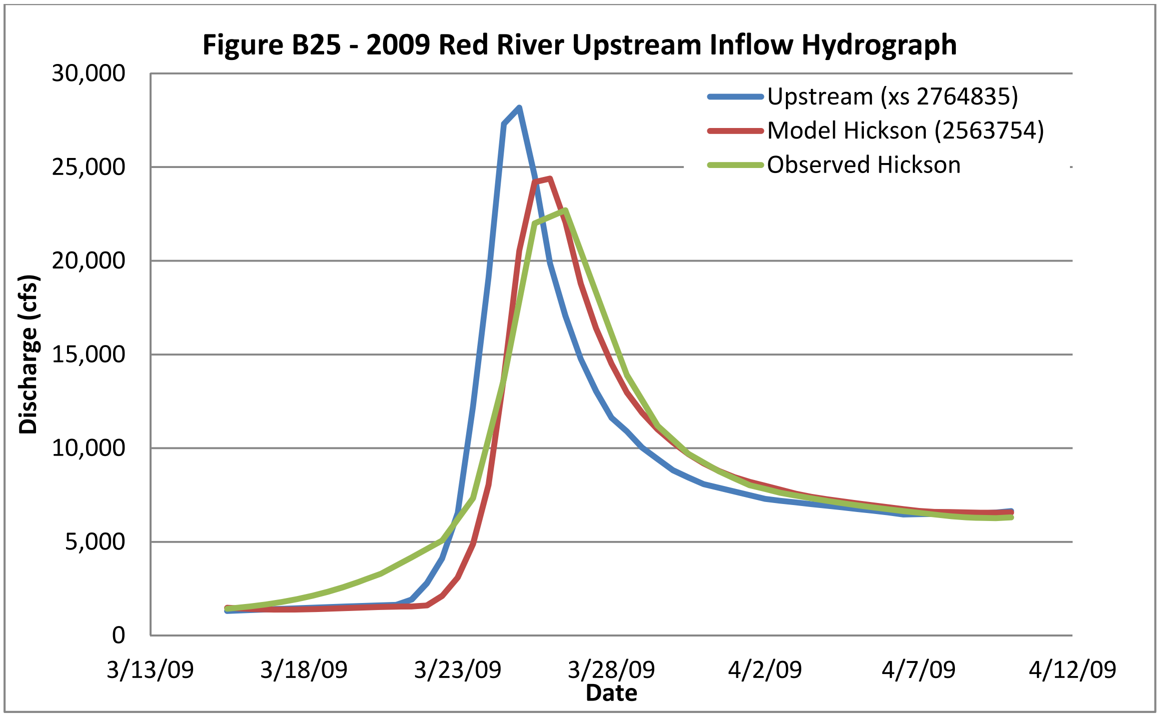
For the extended model, the Red River inflow hydrograph was adjusted so that the routed hydrograph to Hickson would match the observed hydrograph at the Hickson gage. Figure B25 below shows the actual upstream hydrograph in comparison to the Hickson gage for the 2009 event. Notice the modeled hydrograph is actually higher than the observed hydrograph. The modeled hydrograph was increased based on the calibration fit at the Fargo gage. It was justified by accounting for a level of uncertainty within the gaged data. The USGS records actual stage measurements and then they calculate the discharge based on a given rating curve. The USGS rates the quality of the calculated discharge with a quality code as shown in Table B3.

Table B4 – USGS Discharge Measurement Quality Codes

Discharge Measurement Quality Code

Code	Description	
E	Excellent	The data is within 2% (percent) of the actual flow
G	Good	The data is within 5% (percent) of the actual flow
F	Fair	The data is within 8% (percent) of the actual flow
P	Poor	The data are >8% (percent) of the actual flow

The first two-field measurements during the 2009 flood at the Hickson gage were rated "poor" (3/25 and 3/28). The third measurement, after the crest, was rated "fair". This supported the assumption that the actual discharge in the Red River during the crest could be greater than 8% different than the discharge reported by the USGS. The same discharge measurement quality codes were observed and accounted for at each of the gaged locations along the Red River and the contributing tributaries. Exhibits A, B, C and D display Red River modeled hydrographs in comparison to the USGS observed hydrographs and gage quality for the 2009, 2006, 1997 and 2010 events respectively. Additional detail related to the specific gages is noted in Section B5.0 of this report.



4.2.2 Wild Rice River, ND

The hydrograph at the upstream end of the Wild Rice River, ND reach was supplied directly from USGS Gage 05053000 at Abercrombie, ND. The Wild Rice River Watershed consists of approximately 2,080 square miles which is nearly 30 percent of the total drainage area upstream of the Fargo/Moorhead project area.

4.2.3 Sheyenne River

The Sheyenne River contributes a significant amount of runoff to the Red River downstream of the Fargo/Moorhead project area. The upper end of the Sheyenne basin is regulated by Baldhill Dam. The lower end flows uncontrolled through a perched and meandering system. Because the channel is perched, discharges in excess of the channel's capacity are conveyed out of the channel and into the floodplain. These breakout flows either re-enter the system further downstream or become conveyance in a completely different system. Because of the perched channel, the 2%, 1% and 0.2% chance flood discharges are nearly all the same at approximately 4,600 cfs upstream of Horace, ND.

The upper end of the model on the Sheyenne River is upstream of Gol Road, approximately three miles south of Kindred, ND. In 2009, the USGS placed a gage at this location to collect additional information on breakout flows from the Sheyenne River between Gol Road and Kindred. Breakouts from the left bank become part of Cass County Drain 34 and Cass County Drain 14 which later flow into the Maple River. The Maple River joins with the Sheyenne River downstream of West Fargo. Breakouts from the right bank of the Sheyenne River between Gol Road and Kindred exit the system through Richland County Drain 37. These breakout flows become part of the Wild Rice River system and contribute upstream of the project area.

To account for breakout flows in the Sheyenne River system, a hydrograph was inserted into the model upstream of Gol Road. Through an iterative process of adding water and allowing the model geometry to quantify the breakout flows, the hydrograph in the system was reasonably matched at USGS Gage 05059000 at Kindred. The Kindred gage has a much longer history than the USGS Gage 05058980 at Gol Road. It has provided discharge measurements since 1949 and stages since 2000. Breakouts also occur in the system between Kindred and Horace. These breakouts typically bypass Horace and enter the Sheyenne River system either through Cass County Drain 21c or just upstream of Interstate 94. The stage and discharge measurements through the Sheyenne River system from Kindred through West Fargo are very susceptible to ice conditions which induce stage increases and significant breakout flows. Therefore, an average match of discharge at the various gages was determined to be acceptable. Figure B26 is a schematic of the Sheyenne River corridor displaying potential breakout locations along the Sheyenne River as well as inflow hydrograph locations.

4.2.4 Maple River

The Maple River system, once completely unregulated, now receives some flood reduction benefit from storage provided by the Maple River Dam (2009-present). The Maple River has a perched channel similar to that of the Sheyenne River. The Maple River reach of this model extends to Durbin, ND, upstream of a known breakout location. Similar to the Sheyenne River reach between Gol Road and Kindred, the hydrographs at the upper end of the Maple River were inserted into the model near Durbin. These flows were increased, allowed to break out, and the remaining modeled hydrograph was compared to USGS Gage 0506000 on the Maple River near Mapleton. To make this area more complex, two large drainage areas contribute to the system between Durbin and the Mapleton gage. If all inflows were added to the system at Durbin, excessive flows would be diverted at the breakout location. To compensate for this, the inflow hydrograph was divided proportionally between Swan Creek (129 square miles), Buffalo Creek (192 square miles), and the local drainage area between the Maple River Dam and Durbin (208 square miles). Since the 2009 flood event was used for calibration, the hydrograph from USGS Gage 05059715 on the Maple River (outlet of dam) was routed to the upper end of the model. This provided an approximate 1,000 cfs base flow discharge during the first peak of the 2009 flood. The dam hydrograph routed to Mapleton (through the model) was then subtracted from the USGS observed hydrograph near Mapleton. The missing hydrograph was distributed between the three above referenced drainage areas until the hydrograph matched at the Maple River gage near Mapleton. Figure B27 shows the drainage areas, inflow locations, and breakout paths for the Maple River.

The other three historic events occurred prior to the construction of the Maple River Dam. Since the dam outlet hydrograph was unavailable, the observed Maple River discharge hydrograph at USGS Gage 05059700 near Enderlin was used as the inflow hydrograph at the upstream end of the Maple River at Durbin. An iterative process was used to determine an appropriate routing from Enderlin to Durbin, then the model routed the hydrograph from Durbin to Mapleton. This hydrograph was subtracted from the observed hydrograph at the Mapleton gage. As with the 2009 calibration, the missing local hydrograph was distributed between the ungaged areas including: Swan Creek, Buffalo Creek, and the local drainage area between Enderlin and Mapleton. The Enderlin hydrograph and local ungaged hydrographs combine to form the observed hydrograph at the Mapleton gage and breakout flows. The breakout flows are conveyed overland (through storage areas) to Cass County Drain 14 which contributes to the Sheyenne River.

4.2.5 Rush River

The Rush River model geometry was not developed as a detailed river reach. The lower end of it was modeled using storage areas and storage connections simulating the river and drain channels. In Phase 4, the storage areas were extended three to four miles west of the proposed LPP alignment to provide a more consistent geometry upstream of the project for existing conditions and with- project conditions. For hydrograph development purposes, the Rush River area was sub-divided into four basins. Each of the four sub-basins has approximately the same shape and a similar size. Therefore, all of the hydrographs reference USGS Gage 05060500 on the Rush River at Amenia with varying ratios based on size. The first basin, composed of approximately 116 square miles, is upstream of the USGS gage at Amenia, ND. This basin used the Amenia gage hydrograph with an assumed one day route to the storage area input location. The Rush River area between the Amenia gage and the Sheyenne River has a drainage area of approximately 45 square miles. The Lower Rush River contributes approximately 50 square miles. Cass County Drain 13 was also included in the Rush River calculations contributing 58 square miles. Each of the four hydrographs have been inserted as a ratio of the Amenia gage in the appropriate storage area on the west side of the model. Figure B28 shows the Rush River area inflow locations and drainage areas.

4.2.6 Buffalo River

The hydrograph at the upstream end of the Buffalo River reach was supplied directly from USGS Gage 05062000 at Dilworth, MN. The Buffalo River Watershed at Dilworth consists of approximately 975 square miles, and contributes to the system directly downstream of the Fargo/Moorhead project area.

4.2.7 Elm River

The Elm River Watershed consists of approximately 515 square miles. The National Weather Service (NWS) provided estimated hydrographs from their flood prediction model for the 2006 and 2009 flood events for the North Branch of the Elm River and the Elm River at Highway 81 (just downstream of Interstate 29). The estimated North Branch hydrograph accounts for approximately 124 square miles at a location near Kelso, ND. The Elm River (south branch) near Grandin, ND, contributes runoff from approximately 338 square miles. Another 53 square miles between the measured locations and the Red River was accounted for in the local runoff calculations.

To assist with modeling efforts, stream gages were placed at the two Elm River locations (described above) for the 2010 flood event. Data was obtained and utilized in the 2010 verification model. The estimated NWS hydrographs were used for the 2006 and 2009 events. Since estimated data was unavailable for the 1997 event, USGS Gage 05061500 on the South Branch of the Buffalo River at Sabin, MN was used with ratios based on respective drainage areas. The drainage area at this gage is 454 square miles.

4.2.8 Wild Rice River, MN

The hydrograph at the upstream end of the Wild Rice River, MN reach was supplied directly from USGS Gage 05064000 at Hendrum, MN. The Wild Rice River Watershed at Hendrum consists of approximately 1,560 square miles. It is approximately seven river miles upstream of the Red River.

4.2.9 Goose River

The Goose River reach was developed as a simple routing reach. As described above in the geometry section, the model was extended upstream to USGS Gage 05066500 at Hillsboro, ND. The Goose River at Hillsboro contributes approximately 1,093 square miles to the system.

4.2.10 Marsh River

The Marsh River reach was developed as a routing reach. The model was extended upstream to USGS Gage 05067500 near Shelly, MN. The Marsh River near Shelly contributes approximately 220 square miles to the system in addition to breakout flows from the Wild Rice River near Ada, Minnesota.

4.2.11 Sand Hill River

The Sand Hill River reach was developed as a routing reach. The model was extended upstream to USGS Gage 05069000 at Climax, MN. The Sand Hill River at Climax contributes approximately 420 square miles to the system.

4.2.12 Red Lake River

The Red Lake River was extended upstream as a routing reach to USGS Gage 05080000 at Fisher, MN. The Red Lake River at Fisher contributes approximately 5,680 square miles to the system. The Fisher gage is relatively new. It has been recording discharge since 1999. USGS Gage 05079000 on the Red Lake River at Crookston (5,270 SM), upstream of the Fisher gage, has a longer measurement record tracing back to the early 1900's. Records from the Crookston gage were also compared when obtaining historic flow data.

4.2.13 Tributaries Downstream of Grand Forks

The model reach downstream of Grand Forks contains less detail than the upstream reach since existing model geometry was utilized that does not extend up the tributaries. Therefore, the inflow hydrographs between Grand Forks and Drayton were simplified. For hydrograph development during calibration, the Grand Forks hydrograph was subtracted from the Drayton target hydrograph. The resulting hydrograph was proportionally distributed amongst the intermediate drainage areas to create a hydrograph that corresponds to each of the tributaries. The inflow hydrographs were inserted at the tributary locations on the Red River. Tributary reaches were not created and the gages on the tributary rivers were not used. This methodology was originally used for the historic events as well as the synthetic events in Phase 4 for the January 31, 2011 submittal.

Subsequent to the January 31, 2011 submittal, the historic event calibration and verification hydrographs between Grand Forks and Drayton were improved to include gaged tributary data. The Phase 4 synthetic design events still use the proportionally distributed, ungaged hydrographs. Documentation on the watersheds between Grand Forks and Drayton for the historic events is as follows:

- Cole Creek – Cole Creek conveys runoff from approximately 307 square miles south and west of Thompson, ND to a location near USGS Gage 05082500 at Grand Forks. This area remains part of the ungaged drainage area computations.

- Grand Marais River – The Grand Marais River system includes approximately 530 square miles of drainage between Grand Forks and Oslo on the Minnesota side of the Red River. This area also remains part of the ungaged drainage area computations.

- Turtle River – A small portion of the Turtle River (311 SM) is documented with USGS Gage 05082625 data near Arvilla, ND. The remaining 624 square miles is part of the ungaged calculations.

- Forest River – 620 square miles of the Forest River is measured at USGS Gage 05085000 at Minto, ND. The remaining 315 square miles is accounted for in the ungaged hydrographs.

- Snake River – The upper portion of the Snake River is measured at USGS Gage 05085450 near Warren, MN. Here, 176 square miles of runoff is reported. The remaining 612 square miles of the Snake River is part of the ungaged contributions between Oslo, MN and Drayton, ND.

- Middle River – The Middle River is measured at USGS Gage 05087500 at Argyle, MN with 255 square miles of drainage area. The Middle River is a tributary of the Snake River.

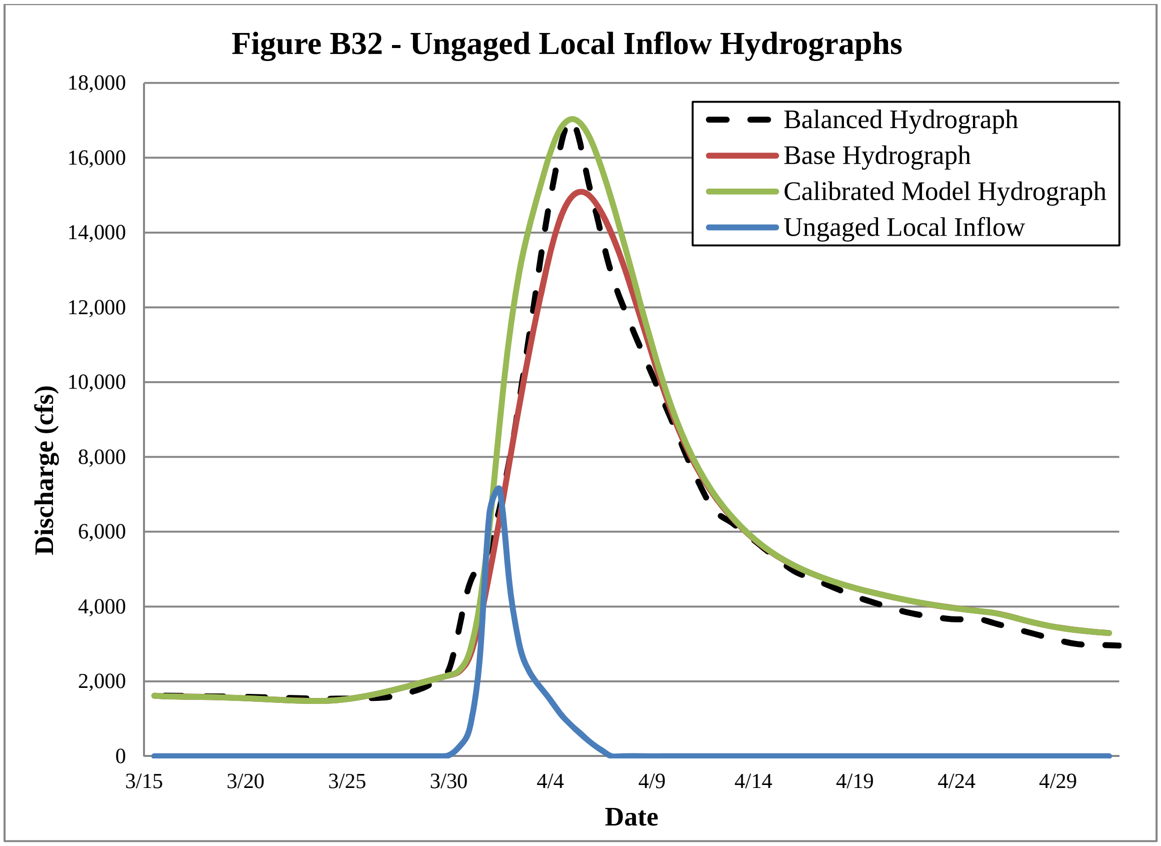
- Park River –USGS Gage 05090000 on the Park River at Grafton measures approximately 695 square miles of drainage area. The remaining 292 square miles is included in the ungaged drainage calculations.

- Tamarac River – The Tamarac River, in Minnesota, includes ungaged drainage from approximately 423 square miles. This contribution enters the Red River just upstream of Drayton, ND.

B4.3 UNGAGED INFLOW HYDROGRAPHS

As discussed above, observed inflow hydrographs were inserted into the model at the upstream ends of tributaries and other large contributing areas. Local inflow hydrographs were estimated to account for inflow downstream of gage locations. They were used to supplement the observed inflows between calibration points. For example, upstream of the Fargo gage, the Hickson and Abercrombie gages provide observed inflows at the upstream ends of the Red River and Wild Rice River respectively. Between these two gages and the Fargo gage there is approximately 384 square miles that is divided into twelve sub-basins. Figure B29 displays a table and series of hydrographs representing typical local inflows upstream of the Fargo gage. The table shows the name given to each sub-basin, the location it was placed in the model and the drainage area. As displayed in the hydrograph example, it was assumed that the duration of the local inflow hydrographs are the same regardless of the size of contributing area. Figure B30 shows a detailed spatial extent of the sub-basins upstream of Fargo. For ease in converting flow files from existing conditions to LPP and FCP conditions, sub-basin delineation was adjusted so the same areas could be to be utilized for all conditions without modifications. The LPP model hydrographs can be used without making changes. “WLVSA1” (Wolverton storage area 1) was split along the FCP alignment. The portion west of the alignment was assumed to drain to the Red River, the east half was inserted into the FCP diversion. A larger scale local inflow drainage area map for the entire model is presented in Figure B31

The “base hydrograph” run includes routing the Hickson and Abercrombie hydrographs to the Fargo gage. This provided a starting point for determining the local inflows. The base hydrograph was then subtracted from the observed target hydrograph at the Fargo gage to obtain a local inflow hydrograph to be distributed throughout the upstream drainage area. An iterative process was used to account for the hydrograph routing time to the Fargo gage. The hydrograph in Figure B32 below displays the results of this procedure for the 10-percent chance flood event aiming to match the provided balanced hydrograph at Fargo. The final calibrated model hydrograph is shown in green.



A similar calibration methodology was used to account for local inflows between the gaged hydrographs at Fargo and Halstad. Here, local drainage areas also extend up the Sheyenne River, Maple River, Rush River, Buffalo River, Wild Rice River, MN, and Elm River totaling approximately 1000 square miles. The third calibration location between Halstad and Thompson included 280 square miles of local inflows up the Marsh River to Shelly, Goose River to Hillsboro, and the Sandhill River to Climax. The Thompson to Grand Forks reach primarily consisted of local Red River drainage and inflows downstream of the Red Lake River gage at Fisher, MN (530 total square miles). Eight river systems contribute to the Red River between Grand Forks and Drayton. As described in Section B4.2 of this report, they include Cole Creek, the Turtle, Forest and Park Rivers on the North Dakota side of the Red River and the Grand Marais, Middle, Snake and Tamarac Rivers on the Minnesota side of the Red River.

B4.4 SYNTHETIC DESIGN EVENT HYDROGRAPHS

Four synthetic event evaluations were used for design and alternative analysis. The 10-, 2-, 1- and 0.2-percent chance flood events were analyzed with the unsteady HEC-RAS model. The modeling methodology for the synthetic events is identical to that of the historic events except that instead of matching an observed stage and discharge at a gage, the synthetic hydrographs are modeled to match balanced hydrographs. The balanced hydrographs were created by the Corps of Engineers at the USGS gage locations on the Red River including Hickson, ND, Fargo, ND, Halstad, MN, Thompson, ND, Grand

Forks, ND, and at the downstream end of the model near Drayton, ND. See Appendix A - Hydrology for additional balanced hydrograph background.

Balanced hydrographs were used as inflow hydrographs throughout the entire Red River model in the Phase 3 study. The balanced hydrograph shape and coincident hydrograph determination introduced an increased level of uncertainty as the model became larger and more detail was added. Between Fargo and Halstad, three major river systems required coincident hydrograph determinations; the Sheyenne River, the Maple River, and the Buffalo River. The balanced hydrographs at Fargo and Halstad were developed and the intermediate locations had to be determined based on drainage area. The Sheyenne River has a very large drainage area with the upper portion of it regulated by Baldhill Dam. Many factors added complexity to the Sheyenne and Maple River systems leaving little basis for determining accurate coincident balanced hydrographs. A sensitivity analysis was conducted prior to the Phase 4 study that provided insight on the effects of modifying inflow hydrographs from the three rivers in question. The sensitivity analysis showed a moderate level of response to hydrograph adjustments and the resulting impacts with the project. The details of the sensitivity analysis are presented in Exhibit F.

After additional evaluation of the most appropriate hydrologic methods and consultation with the Hydrologic Engineering Center, it was agreed that the Phase 4 study hydrology would involve using a pattern hydrograph from a historic event as a basis for simulating tributary inflows. Inflows at the upper end of the model on the Red River originated from a balanced hydrograph at Hickson, ND. The format used for the historic event hydrograph development at Hickson was also used for the balanced hydrographs. The iterative procedure included inserting a modified version of the Hickson balanced hydrograph at the upper end of the Red River at cross section 2764835. The modified hydrograph routed to Hickson (XS 2563754) would match the actual balanced hydrograph as provided in Appendix A – Hydrology.

All tributary inflow hydrographs were developed using the 2006 event hydrograph as a baseline pattern. A ratio was applied to the 2006 hydrographs so the model matched the targeted balanced hydrographs as provided in Appendix A – Hydrology and as shown in Exhibit E of this report. The ratios were typically determined so the runoff volume between the gage locations and the local inflow hydrographs were relatively the same. Table B4 shows the typical tributary multipliers. The ratios varied slightly depending on the volume of water required to match the next downstream balanced hydrograph. The actual ratios used at each gage location for the four design events are shown on Figures B33, B34, B35, and B36 for the 10-, 2-, 1- and 0.2-percent chance events respectively. The local inflow hydrographs for the synthetic events were created in the same manner as they were for the historic events as outlined in Section B4.3 on ungaged inflow hydrographs.

Table B5 – Typical 2006 Gage Multiplier for Synthetic Events

Event	Multiplier*
10-Percent Chance	0.65
2-Percent Chance	1.4
1-Percent Chance	1.8
0.2-Percent Chance	2.3

*values varied slightly by tributary.

As calibration and inflow hydrograph development progressed, it became apparent that the tributary peak timing relative to the Red River played an important role in determining the final size and shape of the Red River hydrographs. The timing of the eight largest historic floods was analyzed to determine the date at which each tributary peaks with respect to the peak on the Red River at adjacent stream gages. This provided support for determining the appropriate timing of the synthetic event tributary hydrographs.

Balanced Hydrographs provided by USACE include:

- Red River of the North - USGS Gage 05051522 at Hickson, ND
- Red River of the North - USGS Gage 05054000 at Fargo, ND
- Red River of the North - USGS Gage 05064500 at Halstad, MN
- Red River of the North - USGS Gage 05070000 at Thompson, ND
- Red River of the North - USGS Gage 05082500 at Grand Forks, ND
- Red River of the North - USGS Gage 05092000 at Drayton, ND

B5.0 MODEL CALIBRATION AND VERIFICATION

The 2009 spring flood event was chosen for model calibration because it is the current flood of record and was well documented by high water marks and stream gage data. The 2006 spring flood was the fifth highest flood on record through the Fargo/Moorhead area and was chosen as the pattern event for the balanced hydrographs. Notice the order of which the verification events have been presented, 2006, 1997, and 2010. This is related to the validity of the verification. The 2006 event had more documented gage records than the 1997 event. The 2010 event hadn't occurred during the Phase 2 study and the observed gage records were not available until late in the Phase 3 study. Much of the 2010 data was considered preliminary at that time so a lesser emphasis was placed on the data. The 2006 flood event was conveyed through the model as the primary verification event. Prior to the 2009 flood, the 1997 spring flood event was the highest flood event on the Red River. The 1997 spring flood was used as the second verification event. The third verification event came from the most recent flood in 2010. The flood of 2010 was recently documented as the sixth highest flood through Fargo/Moorhead and the first time the area has experienced back-to-back years with floodwaters above major flood stage. Figures B37-B40 show the locations of high water marks from the 2009, 2006, 1997, and 2010 floods that were utilized for model calibration and verification.

B5.1 HISTORIC EVENT MODEL GEOMETRY CHANGES

Flood protection measures are typically constructed by communities up and down the Red River. The Cities of Fargo and Moorhead, along with rural Cass and Clay Counties utilize both permanent and emergency flood protection measures. Permanent flood protection includes earthen levees and floodwalls, while emergency measures consist primarily of clay and sandbag levees with a few locations protected with various manufactured products. Regardless of the protection method, the riverine characteristics are relatively the same for modeling purposes. For model calibration and verification, levees were added to the model geometry to simulate flood protection measures. Figure B19 highlights the typical lines of permanent and emergency flood protection measures used during the 2009 flood.

Substantial development has expanded Fargo and Moorhead to the south since the 1997 flood. With the additional development, the Cities have continued to make efforts to provide protection from the Red River and the Wild Rice River. In 1997, the Fargo line of protection paralleled Rose Coulee and Drain 53 east of Interstate 29and south of Interstate 94, and was made up of localized protection west of Interstate 29. Nine years later, in 2006, and then again in 2009, the line of protection was extended approximately 7 miles south to Cass County Road 16. There were two other minor differences between 2006 and 2009 protection limits. As described above, the 2009 event was larger than 2006 requiring a dike to be created along 25th Street between 88th Avenue S. and 100th Avenue S. In addition, overtopping of a half mile length of County Road 16 was prevented in 2009 (just west of Interstate 29). 2010 had similar protection limits to 2009. The geometry for both the historic and synthetic event models included the typical temporary and permanent flood protection measures of cities up and down the Red River including the cities of Grand Forks and East Grand Forks.

B5.2 2009 FLOOD CALIBRATION

The 2009 spring flood was anticipated early in the fall of 2008 when the basin experienced one of the wettest falls on record saturating the landscape leading into freeze up. This was followed by above average snowfall through the winter. The melt began with extreme rainfall throughout the southern part of the Red River Basin that led to a rapid snow melt. The magnitude of the flood intensified when more than 2 inches of rain fell just a few days prior to the crest followed by temperatures falling into the single digits. The National Weather Service crest projections increased several times in the days leading up to the crest. The communities of Fargo and Moorhead narrowly escaped devastation with a successful flood fight. The 2009 flood carried along many unique characteristics of which only some were able to be modeled.

During the calibration, the hydrographs at Abercrombie and Hickson were routed downstream to the Fargo gage. As described in Section B4.2 of this report, local inflows were proportionally added to the system. The required local inflow hydrograph was compared to day-to-day snow water equivalent charts throughout the basin. Early in the calibration, an attempt was made to account for uneven rainfall distribution across the ungaged drainage areas. The result of this was very limited and for calibration purposes, a uniformly distributed hydrograph was used.

Two additional stream gages (HOBO) were placed on the Wild Rice River for the 2009 event. The stream gages provided stage data for comparison. The field flow measurements for the stream gages were only obtained on the receding limb of the hydrograph resulting in a skewed calculated discharge hydrograph that was of limited value.

Eight additional stage gages (HOBO) were placed along the Red River between the Wild Rice River, ND and Georgetown, MN. Although these gages did not provide discharge records, the stage measurements were very beneficial in calibrating the model to match stage throughout the flood.

As noted in the geometry section, geometry parameters such as ineffective flows, overbank reach lengths and manning's n values were adjusted to calibrate not only the peak, but the entire stage and flow hydrograph throughout the Red River. Several sensitivity modifications were made to the geometry to identify the resulting impact and response of the model. At no point did the discharge hydrograph ever match perfectly at the Fargo gage. This is attributed to the rainfall and extreme temperatures leading up to the crest. It was very difficult to obtain the maximum stages corresponding to the observed discharge. Manning's "n" values were set at reasonable values, yet were on the upper end of the accepted range of values. After looking at the calibration in detail, it is anticipated that a vertical variation in "n" value may be beneficial since much of the overbank conveyance is through heavy tree cover. As the stages increase, a larger percentage of the conveyance is through the tree canopy with more restriction. This would be modeled with lower "n" values at lower discharges and increasing "n" values with stage. This level of detail would likely not be feasible in a project as large and complex as this one.

Other uncertainties identified while calibrating the model to the Fargo gage as well as downstream gages involves the quality of the available observed USGS stream gage data. If the discharge data is off by 5% (good), 8% (fair), or greater than 8% (poor), it could easily affect the comparison to the known stage data and surveyed high water marks. It would then show that the model is not accurately calibrated when the real issue is related to the amount of water added to the system. The Hickson stream gage data for 2009 was rated "poor" at the crest and "fair" half way down the receding limb. The Abercrombie stream gage measurements for 2009 were rated "fair" near the crest. This means that each of the upstream gage discharges could be eight percent or greater from the measured values. The Fargo stream gage measurements through the crest were rated "good" meaning the actual discharges could be up to 5 percent from the measured discharges. With discharge measurements of approximately 29,400 cfs near the crest at Fargo, the actual discharge could range from 27,900 cfs to 30,900 cfs. Based on the rating curve at the Fargo gage, this could affect stage by approximately 0.4 feet. The modeled hydrograph compared to the observed hydrograph and quality measurements is shown in Exhibit A for 2009.

The Halstad gage had many issues with discharge measurements during the 2009 flood. It had several field measurements with poor ratings. Discussions with local USGS representatives concurred with the ratings and suggested that the 2009 discharge data at Halstad not be used and more emphasis be placed on the discharge data at the Thompson gage instead. Exhibit A for the Halstad gage reflects the discrepancies between modeled and measured discharges for 2009. Stage data at Halstad did not have issues.

Another difficult characteristic to model is that the cold temperatures during the flood caused lower velocity surface waters to freeze. Although the ice layer wasn't very thick and did not cause ice jams, it may have however created additional surface roughness resulting in stage increases.

Peak stage calibration was relatively easy to achieve in areas other than directly upstream of Fargo/Moorhead near the Wild Rice River, ND confluence, downstream of Fargo/Moorhead through the Oakport Township area and through the encroached floodplain of the Fargo/Moorhead area. The known discrepancies can be attributed to any of the above noted issues.

B5.3 2006 FLOOD VERIFICATION

The 2006 flood crest at the Fargo gage occurred on March 28th. Temperatures were very mild resulting in a uniform runoff. No rainfall or blizzards caused adverse impacts to consider during verification. Much less observed data was available for the 2006 event than the 2009 event. The USGS gages as well as high water elevation marks throughout the Red River with a higher density through the Fargo/Moorhead area, was all that was available

Gage quality data was extremely critical at Hickson, Abercrombie, and at the Fargo gage during the 2006 event. Since the temperatures were relatively warm, local runoff contributed to the river system well before the main crest. Routing the Hickson and Abercrombie hydrographs to the Fargo gage created a hydrograph that was already larger than the gaged data. The discharge measurements were rated "fair" near the peak (19,200 cfs 4/4/2006), so the actual discharges could be up to eight percent from actual (17,700 cfs to 20,700 cfs). With minor local inflow contributions, the modeled discharge hydrograph and resulting stage data provided a reasonable match to high water marks. Here it was assumed that the measured USGS discharge at the Fargo gage was underestimated because the other historic events matched stage and discharge and the discharge in the 2006 model provided a reasonable stage calibration. See Exhibit B for 2006 modeled hydrograph and USGS measured hydrograph comparisons along with the field measurement quality ratings.

B5.4 1997 FLOOD VERIFICATION

Prior to the 2009 flood, the 1997 flood event was the highest on record at the Fargo gage. The 1997 flood was very unique with a freeze thaw cycle that caused bi-modal (dual peak) discharge hydrographs at many locations. The specific timing of the flood peaks often created very wide hydrographs with significant volume. One challenge when developing the 1997 verification was the limited gage data and high water elevation records. The Thompson gage between Halstad, MN and Grand Forks, ND was not yet established in 1997. Hickson, Fargo and Halstad did not have stage hydrographs available. Another notable issue was that the Grand Forks levees overtopped and breached leading up to the crest. Rough estimates of conveyance through the city were computed during post flood evaluations. However, for this project, the permanent protection through Grand Forks, ND and East Grand Forks, MN were assumed to be in place for the 1997 flood verification. Comparison hydrographs, where available for 1997, are shown in Exhibit C.

B5.5 2010 VERIFICATION

The 2010 flood event was originally projected to be as extreme as the flood in the previous year. As the flood crest drew nearer, the prediction narrowed in on elevations slightly less than that of 2009. The 2010 flood came very fast and the crest occurred on March 21st. The largest challenge for the model verification using the 2010 flood event was that at the time of the model development, data was not available yet. Provisional data was supplied by the USGS; however the data has not been completely replaced with approved data. Additional information has recently become available and will be incorporated into the verification in the future. Hydrograph comparisons between the model results and observed data is shown in Exhibit D for 2010.

B6.0 MODEL PEER REVIEW AND QA/QC MEASURES

Due to the complexity of the unsteady HEC-RAS model and the greater role it is playing in the project design for the Phase 4 study, a formalized peer review process was initiated by the Project Team. This effort was led by Stu Dobberpuhl at Moore Engineering and included internal peer review of both the existing conditions and with-project models. Peer review for the existing conditions unsteady model was performed by experts from Moore Engineering, Inc., Barr Engineering, and HDR. The results of the peer review along with responses to the review comments are contained in Exhibit H of this report. Additionally, the peer review resulted in the identification of future modeling needs that area summarized in Appendix C Hydraulics with project conditions.

B7.0 EXISTING CONDITION MODEL RESULTS

B7.1 HISTORIC EVENT RESULTS

In summary, four historic flooding events were evaluated with the unsteady state HEC-RAS model. The 2009 event was used for calibration. The 2006, 1997 and 2010 events were used to verify the calibrated model. Although the historic events were not intended for design, they provided a good sense of assurance that the model was reasonably calibrated and not uniquely connected to one event with abnormal conditions. Existing Conditions Model results along with inundation maps pertaining to specific events are presented with the impact tables and impact maps in Appendix C – Hydraulics Design.

B7.2 SYNTHETIC EVENT RESULTS

After the model was calibrated and verified with the historic events, four synthetic events were developed for use in design and impact analysis. As with the historic events, model results and flood inundation maps are displayed in Appendix C Hydraulics with project conditions.

B8.0 REFERENCES

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15. Federal Emergency Management Agency, Flood Insurance Study, City of Harwood, North Dakota, Cass County, September 4, 2002.

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Appendix B – Hydraulics
Existing Condition

Figures

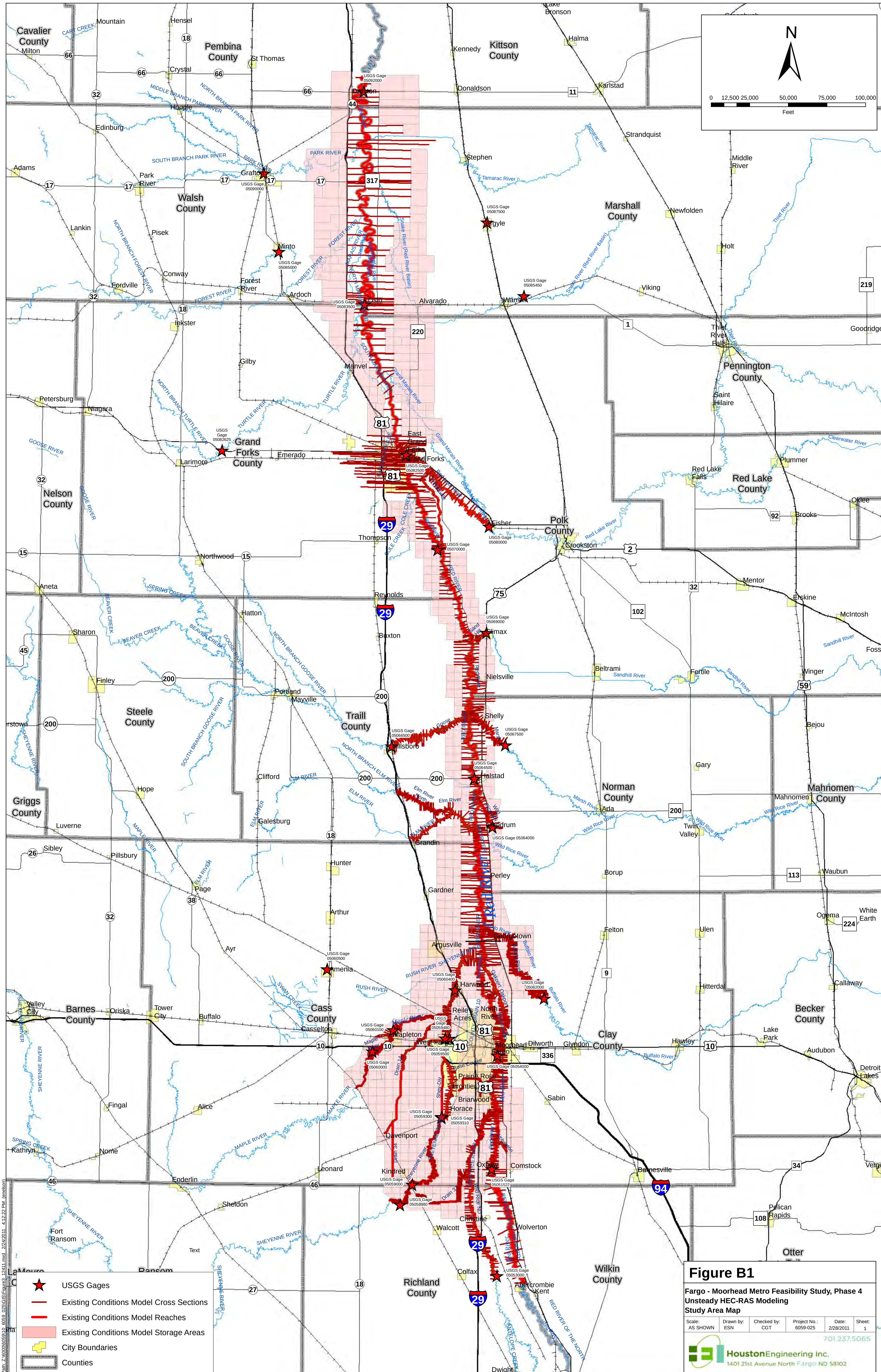

Figure B1

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
Study Area Map

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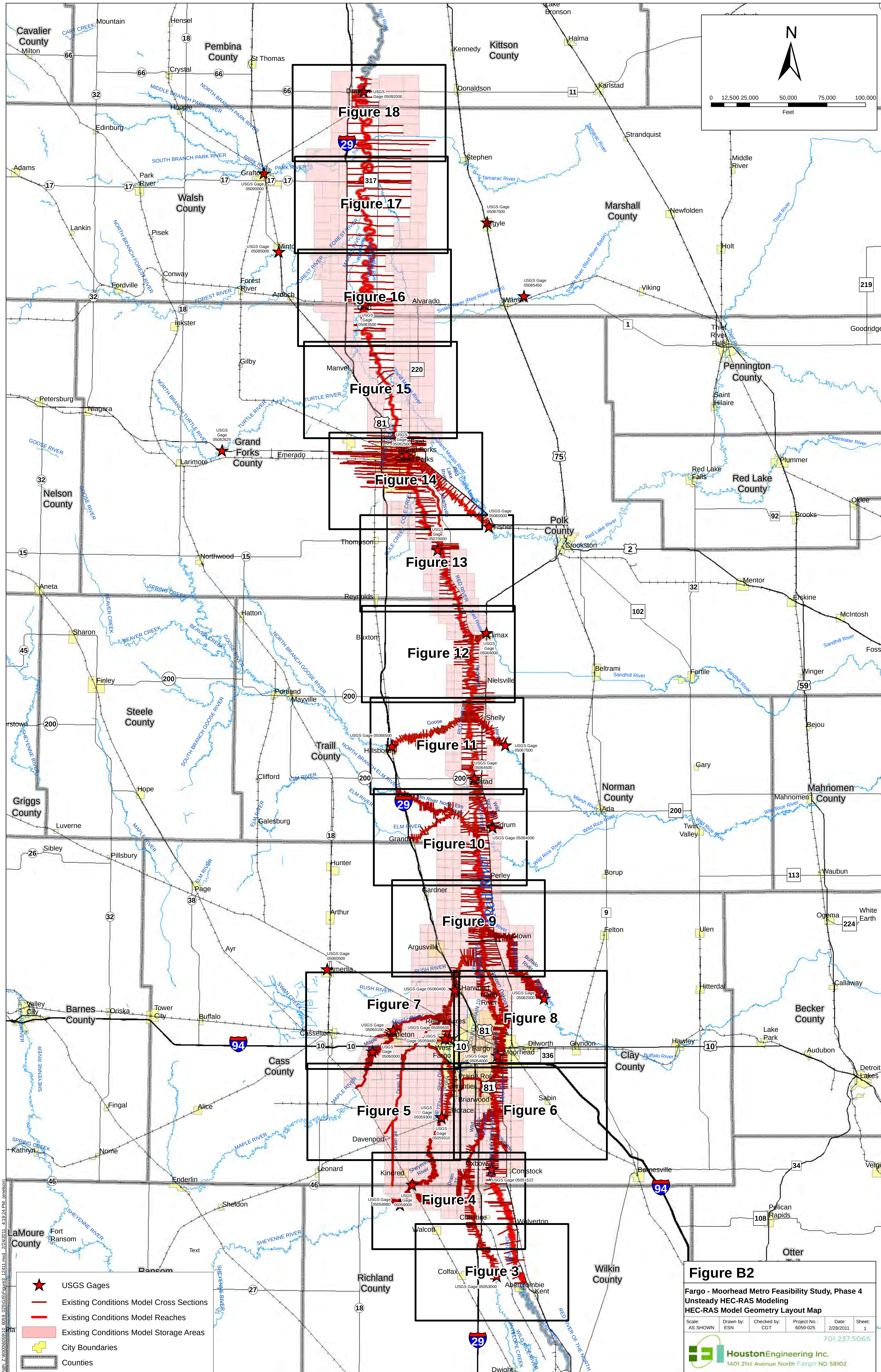

Figure B2

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
HEC-RAS Model Geometry Layout Map

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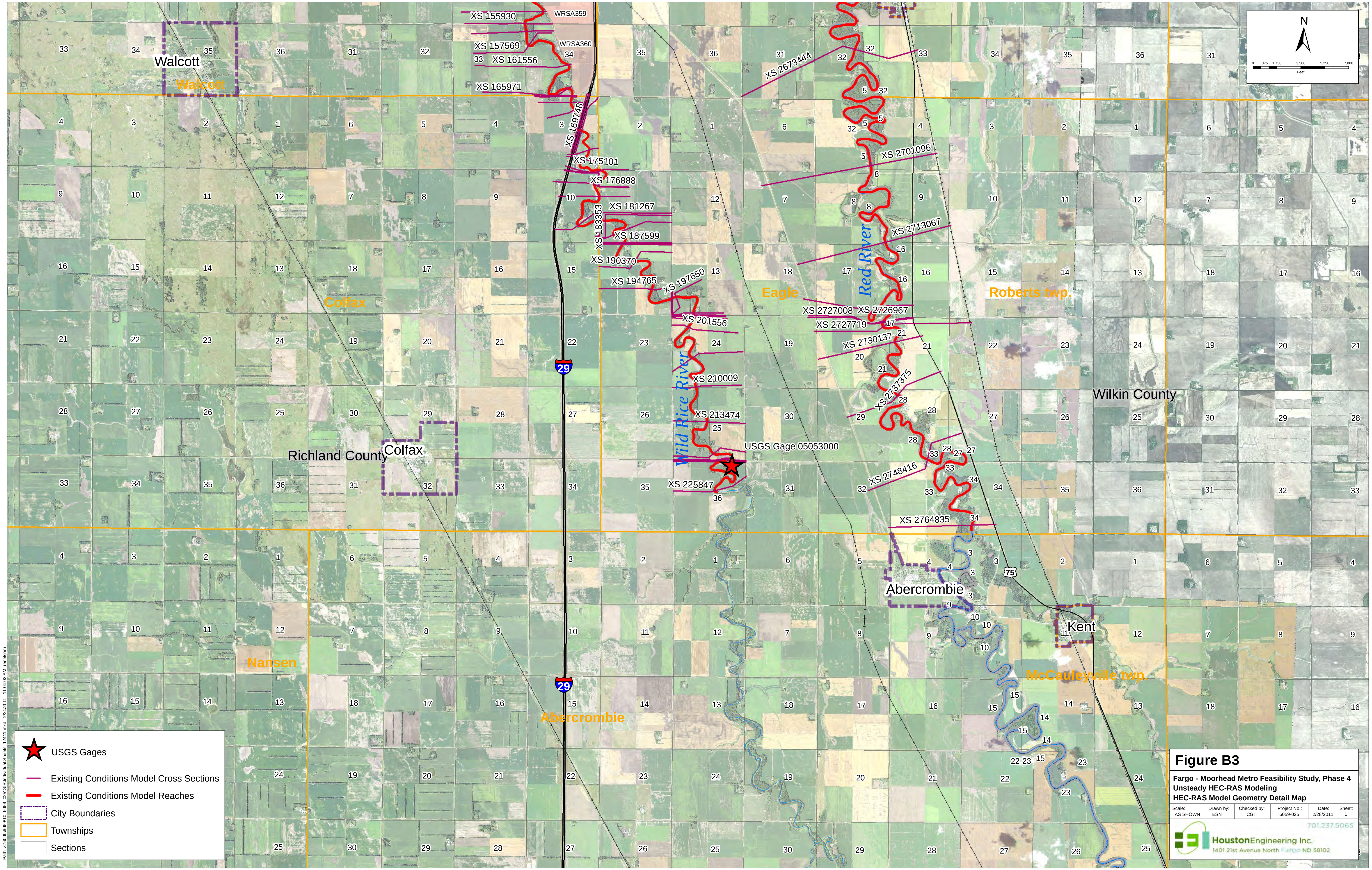

Figure B3

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
HEC-RAS Model Geometry Detail Map

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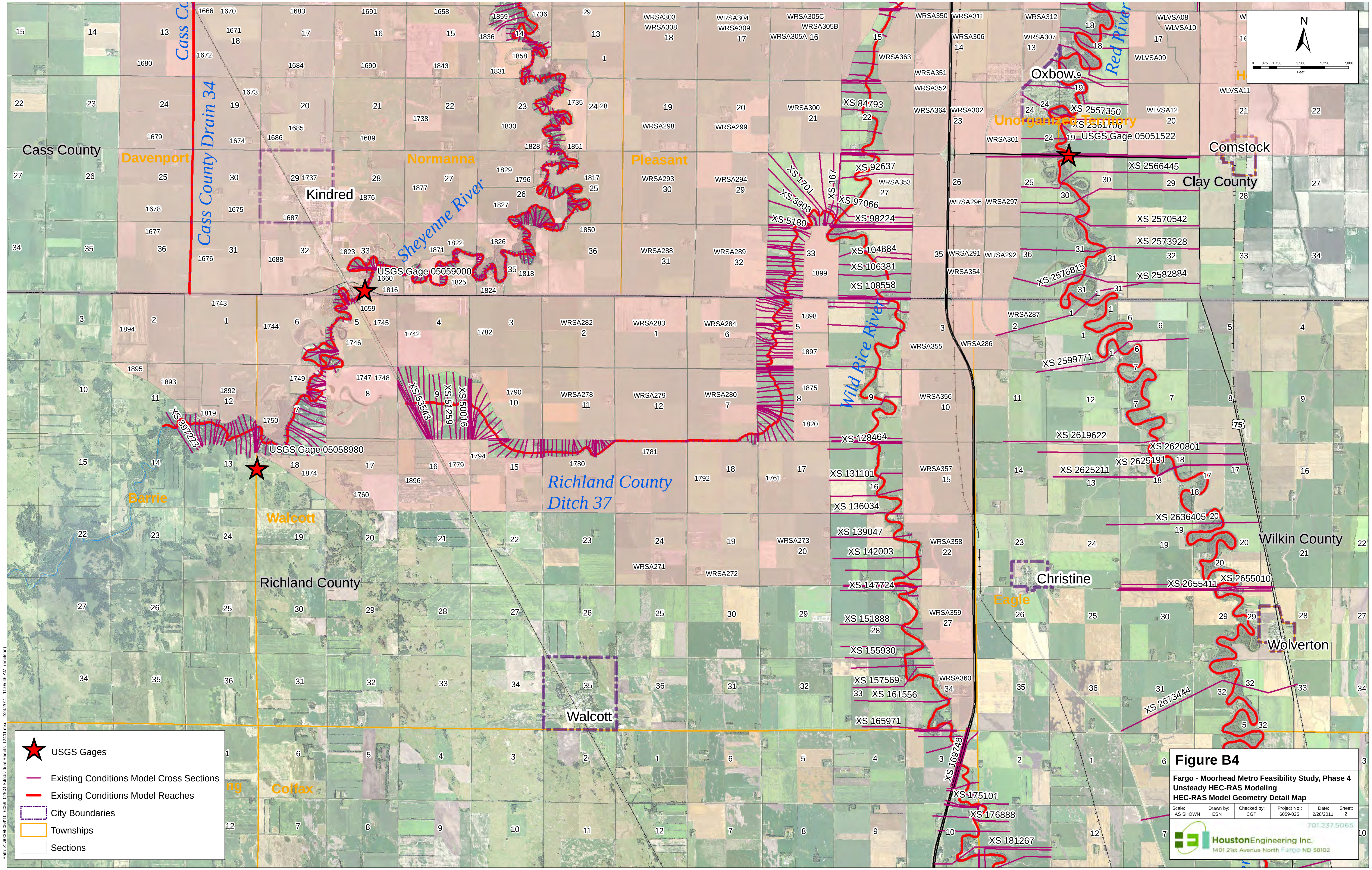

Figure B4

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
HEC-RAS Model Geometry Detail Map

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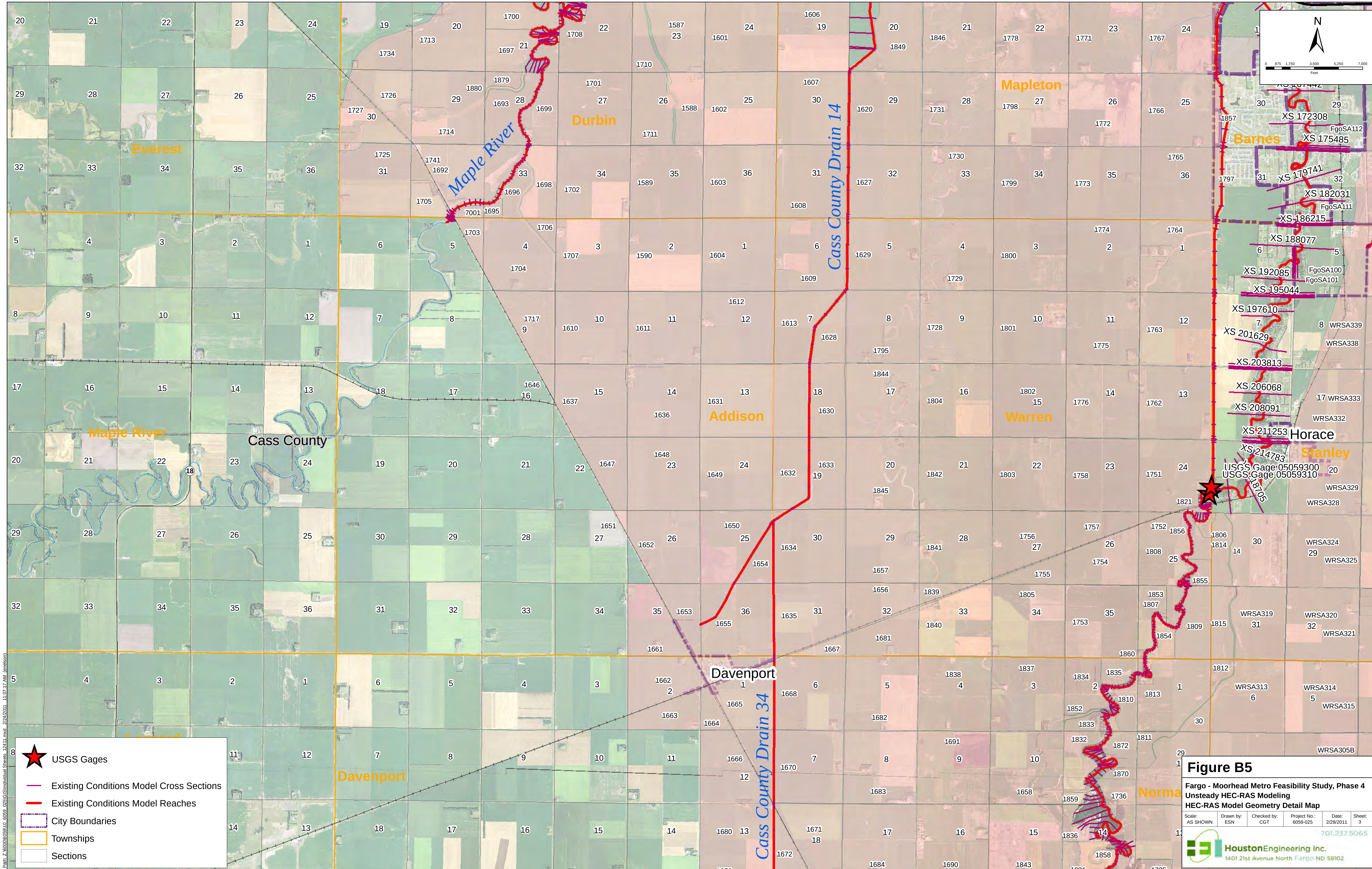

Figure B5

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
HEC-RAS Model Geometry Detail Map

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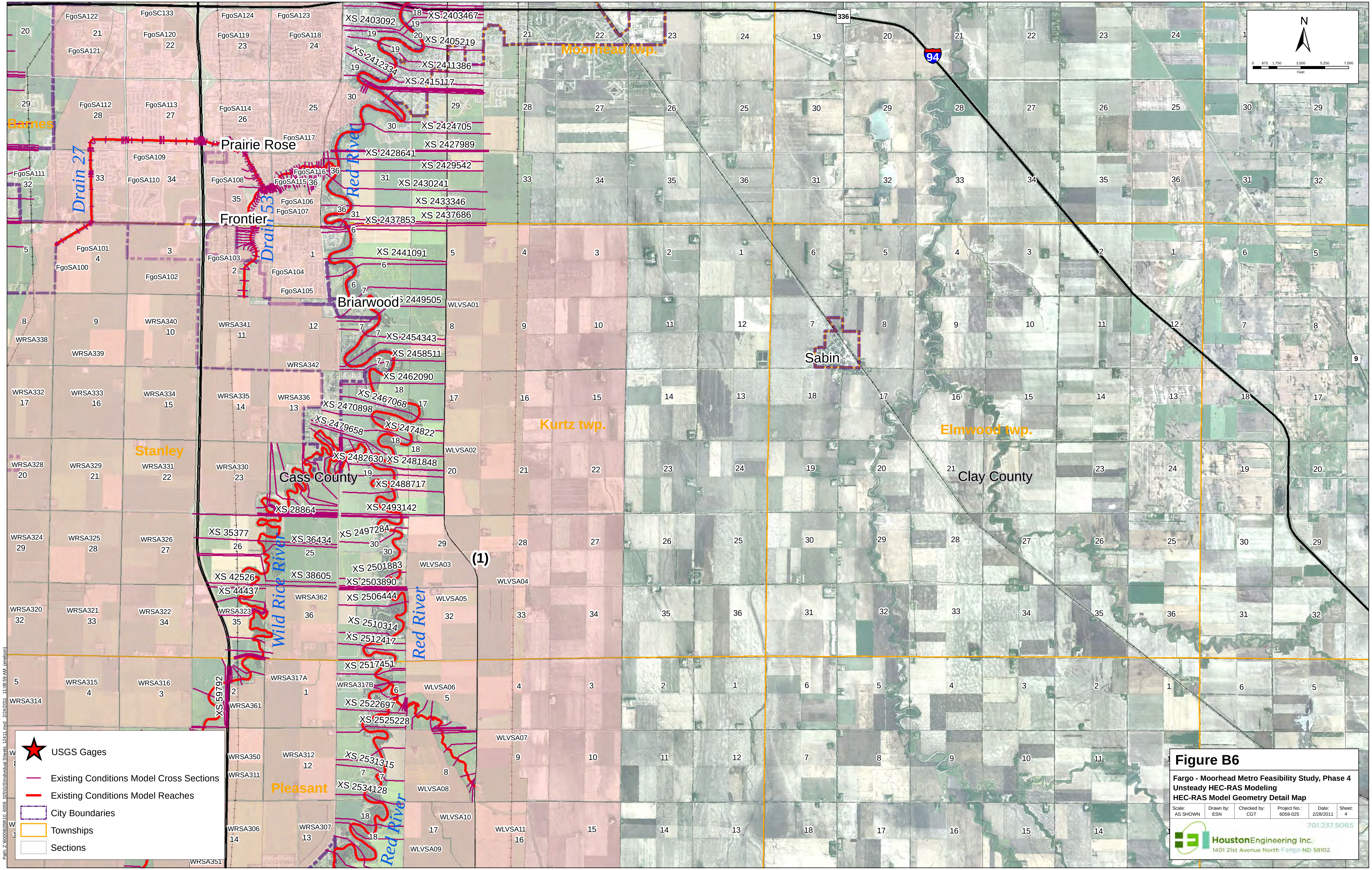

Figure B6

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
HEC-RAS Model Geometry Detail Map

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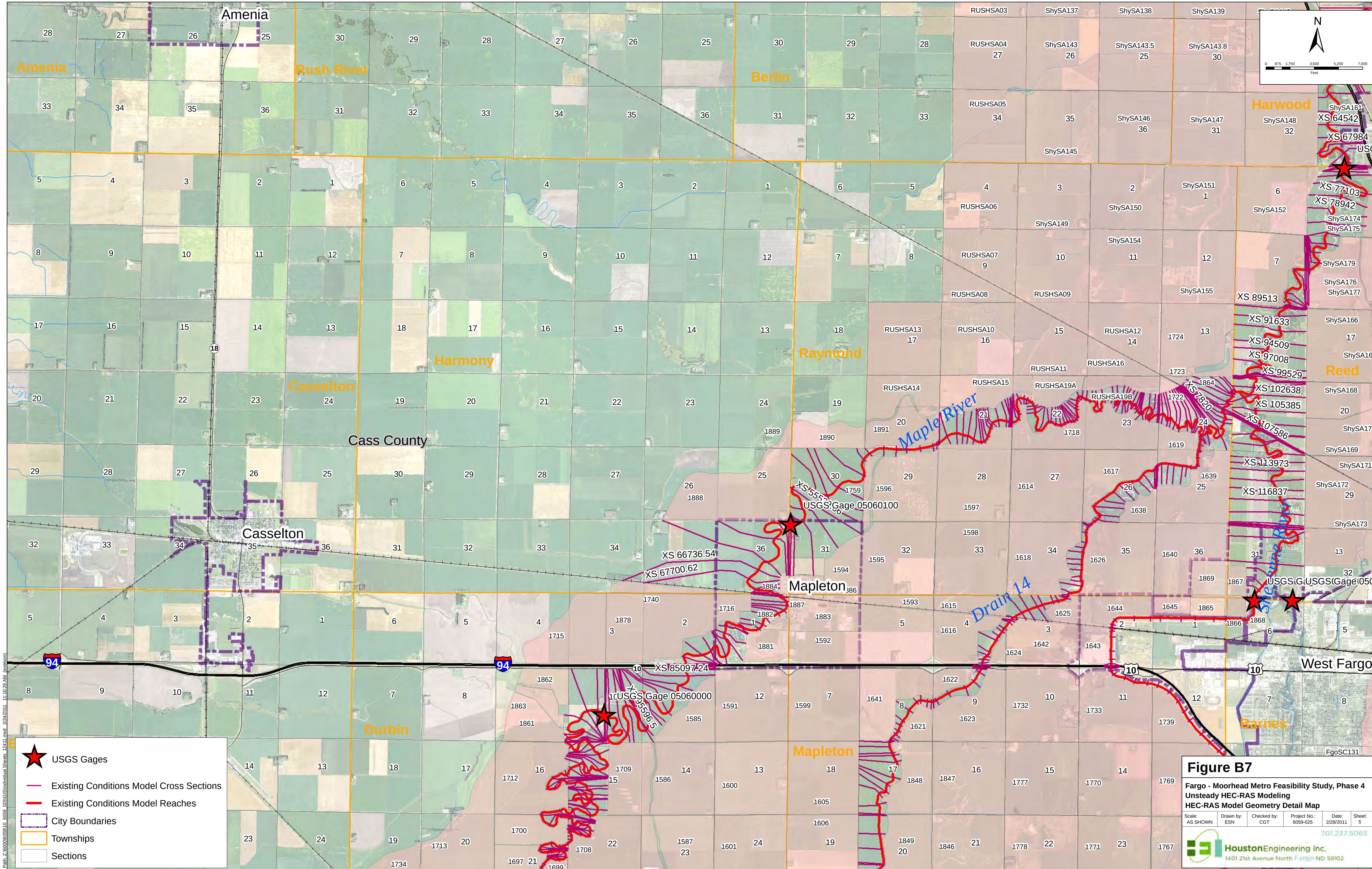

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Harwood
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Raymond
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Maple River
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West Fargo
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Barnes
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USGS Gages
Existing Conditions Model Cross Sections
Existing Conditions Model Reaches
City Boundaries
Townships
Sections
Figure B7
Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
HEC-RAS Model Geometry Detail Map
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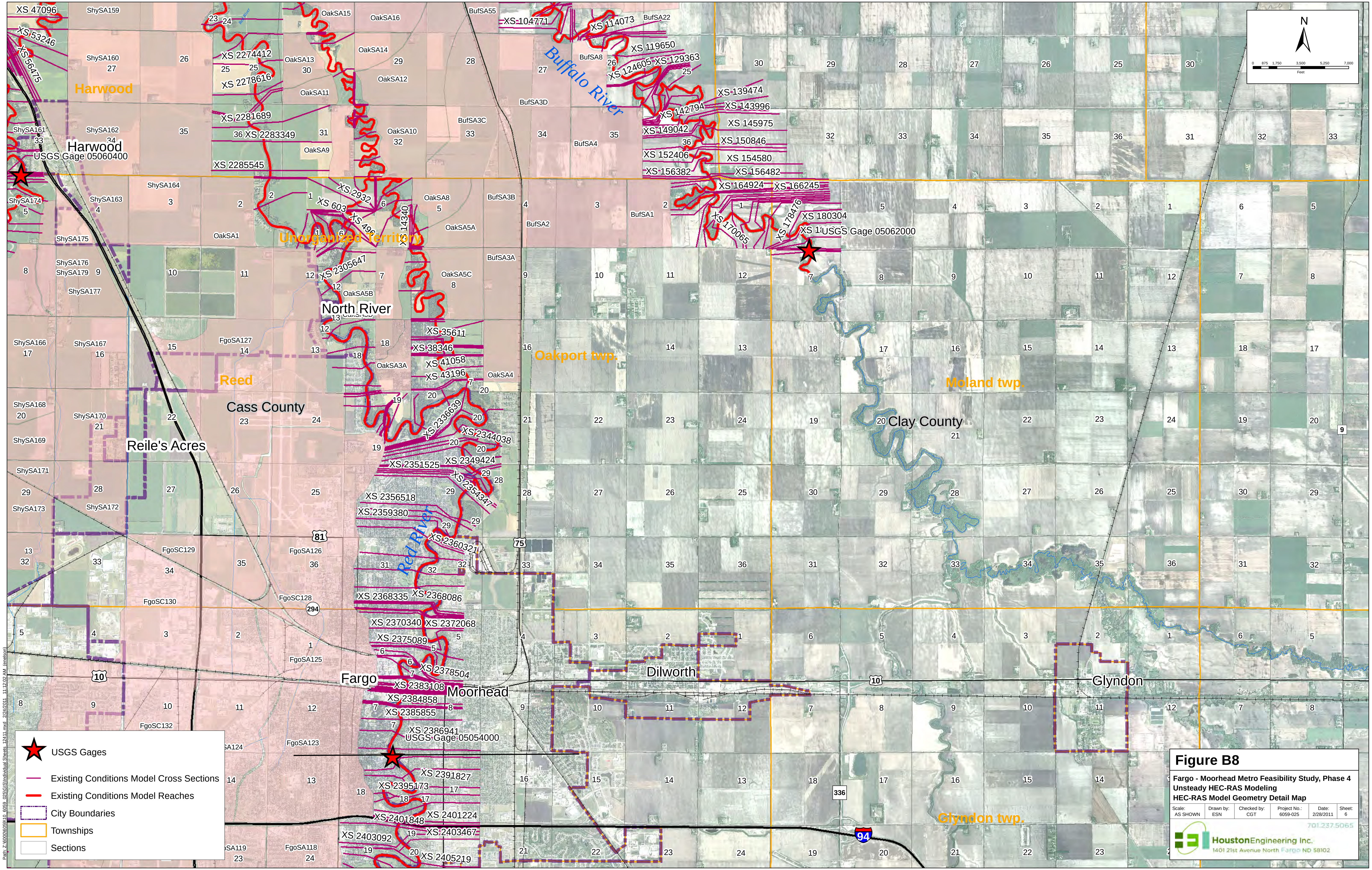

Figure B8

Fargo - Moorhead Metro Feasibility Study, Phase 4
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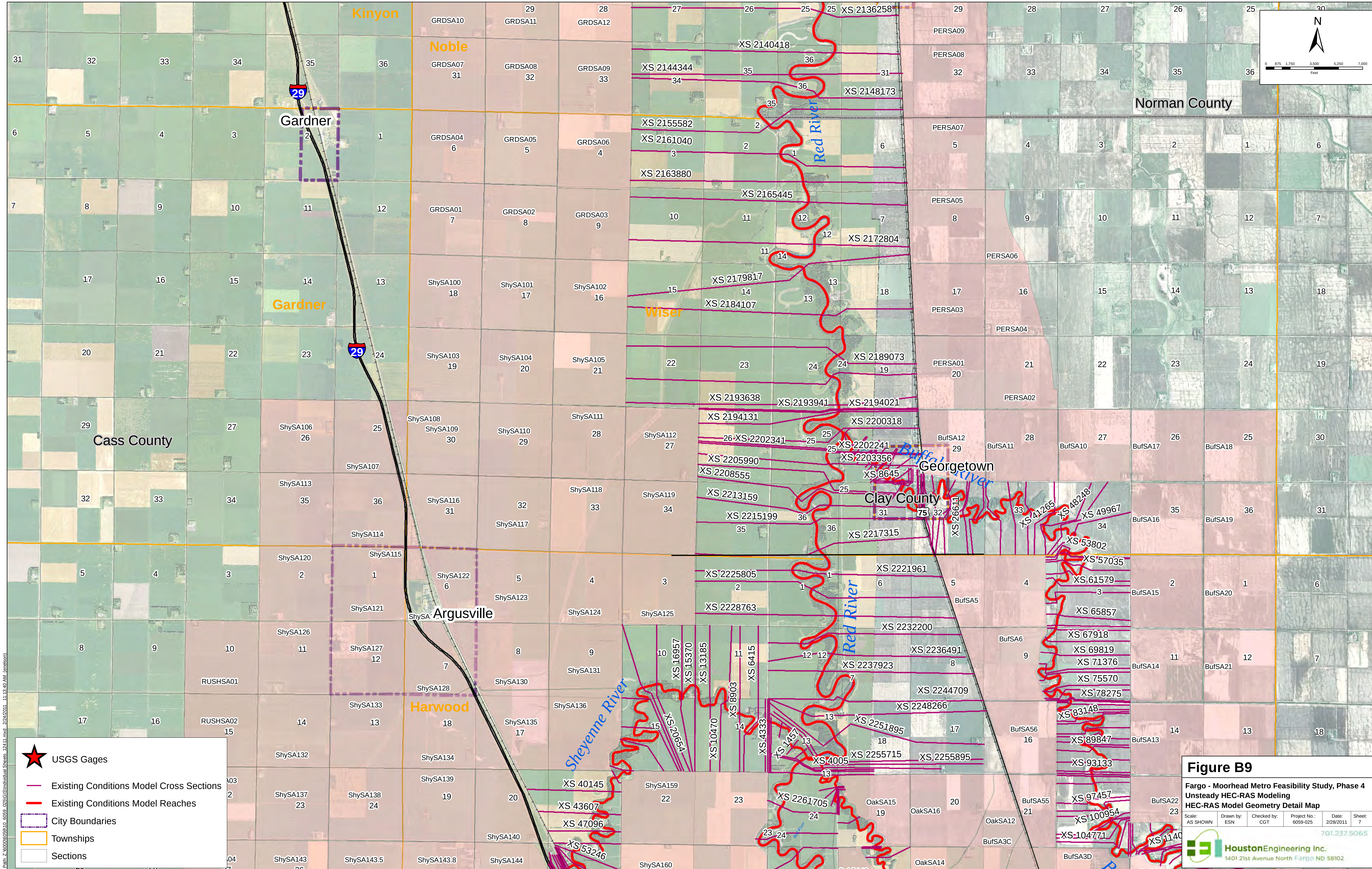

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Fargo - Moorhead Metro Feasibility Study, Phase 4
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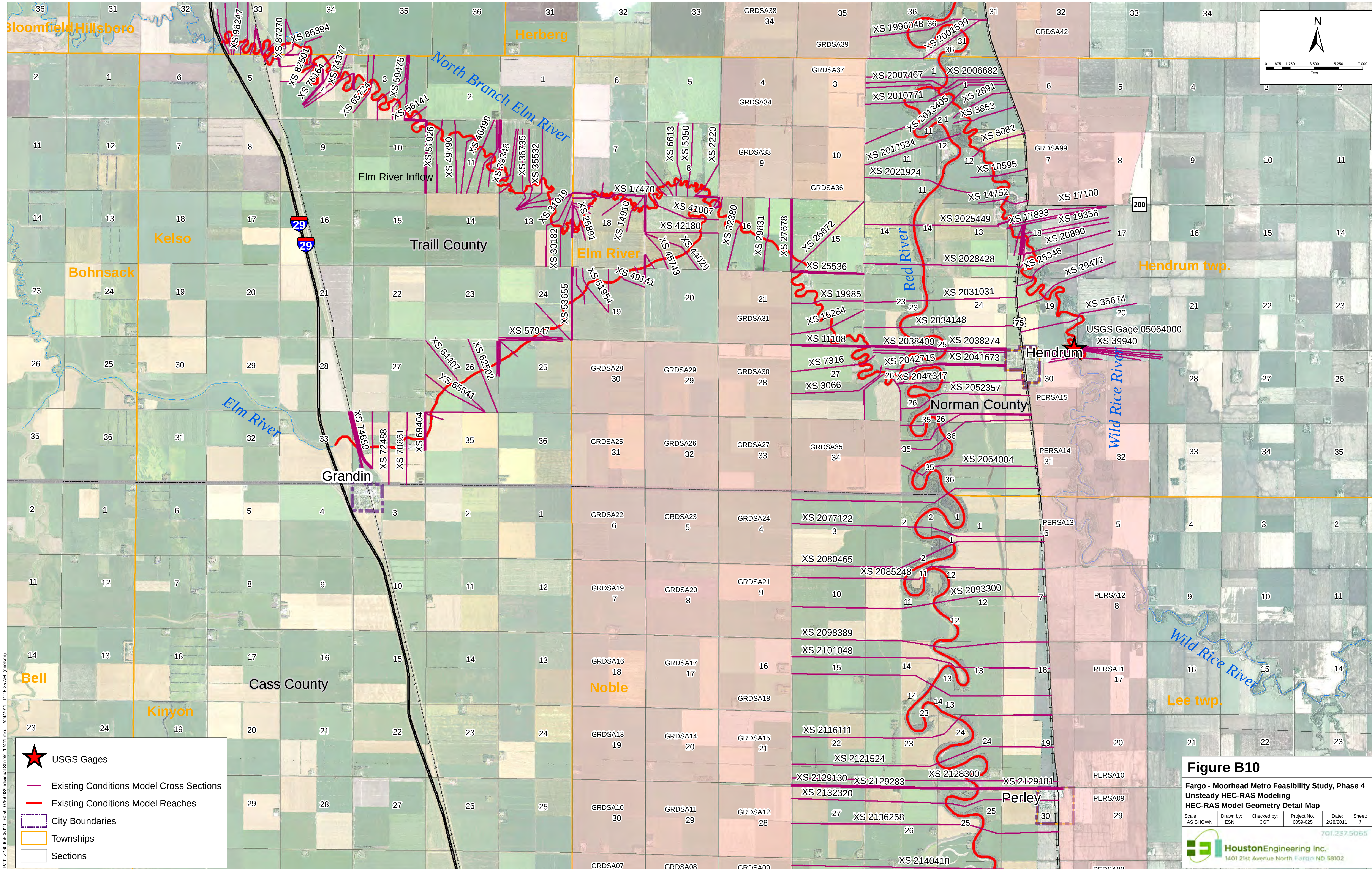

Figure B10

Fargo - Moorhead Metro Feasibility Study, Phase 4
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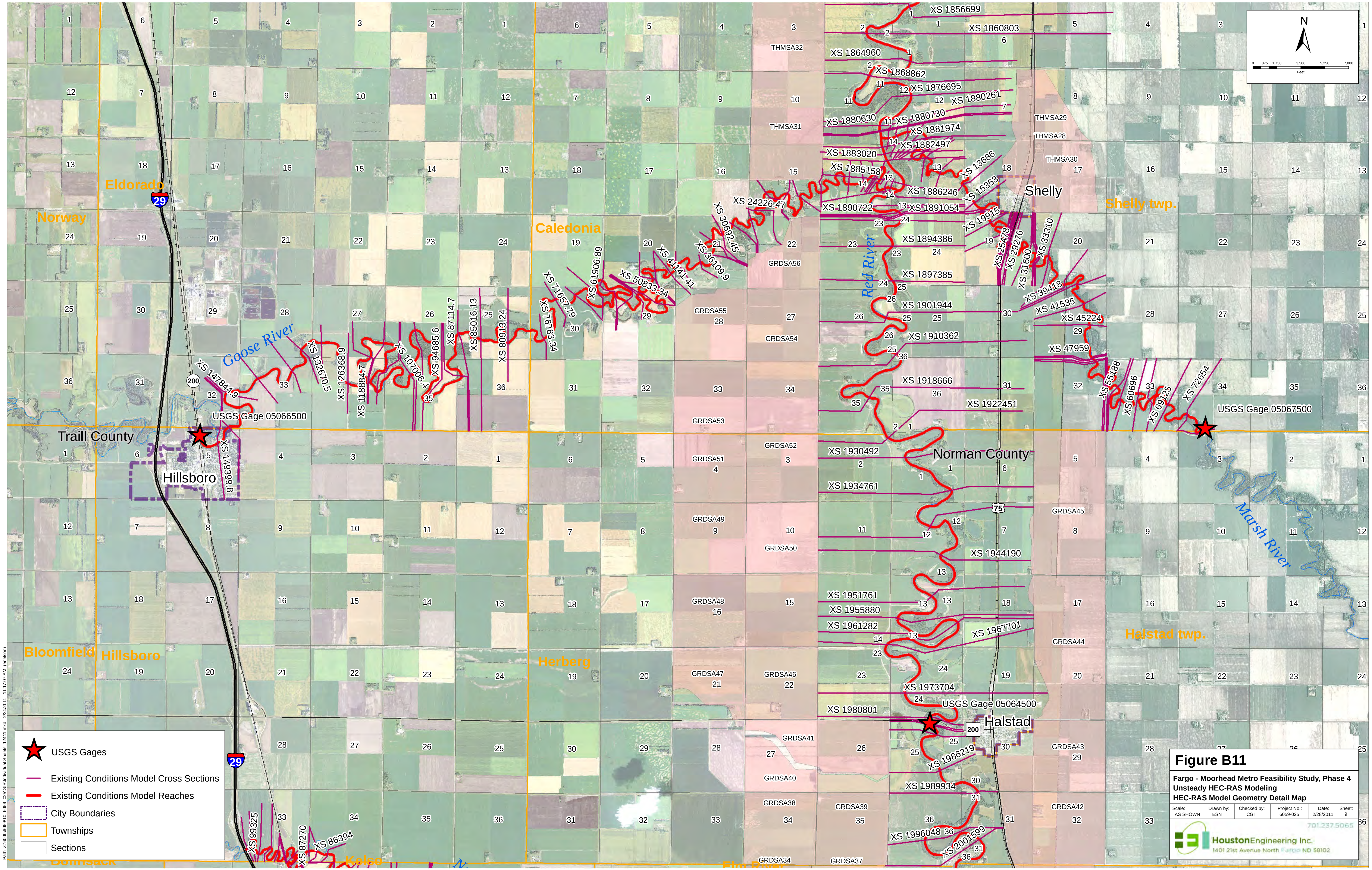
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Bloomfield
Hillsboro
Herberg
Halstad twp.
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Existing Conditions Model Cross Sections
Existing Conditions Model Reaches
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Sections
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HEC-RAS Model Geometry Detail Map
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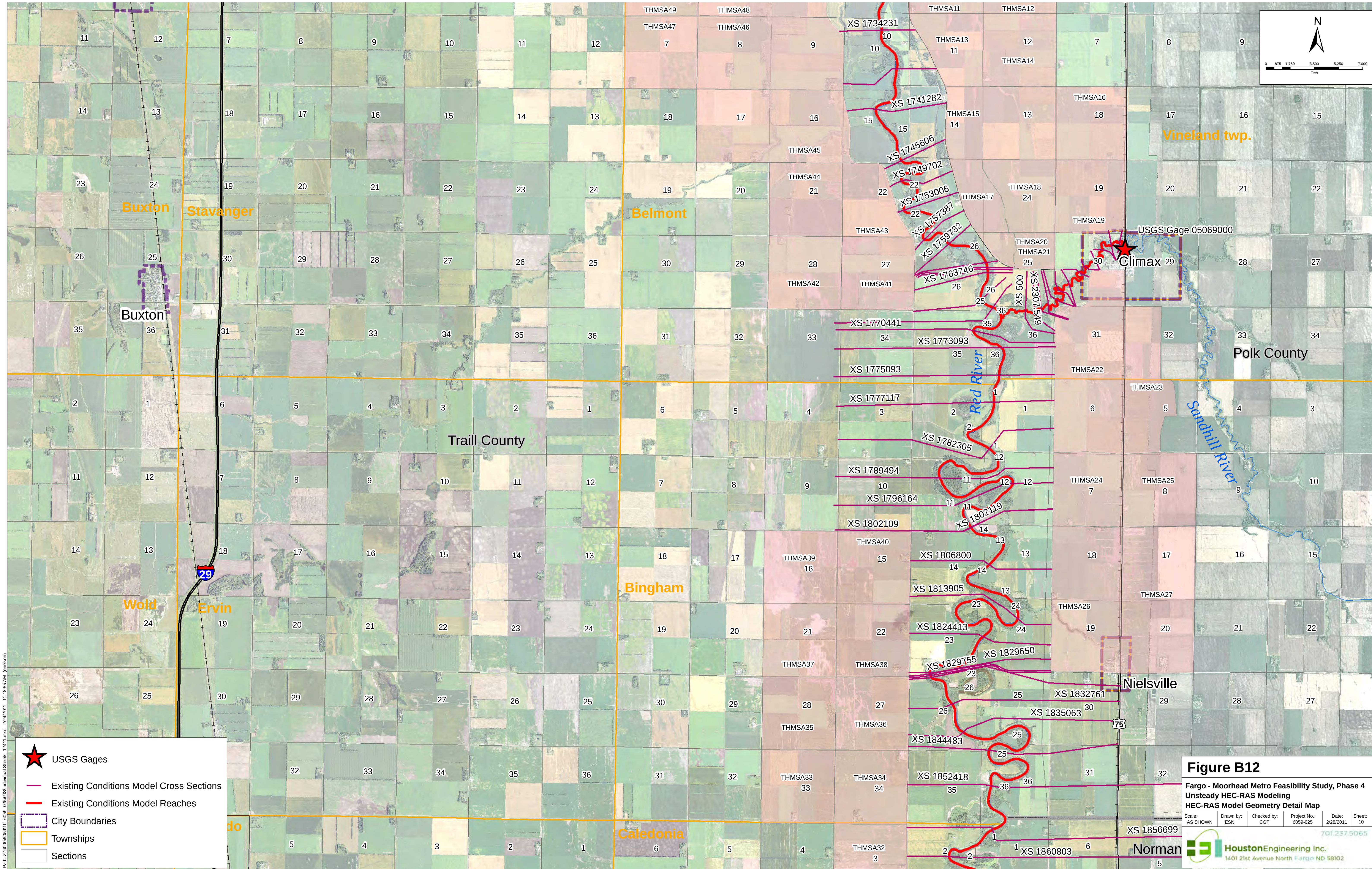

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Fargo - Moorhead Metro Feasibility Study, Phase 4
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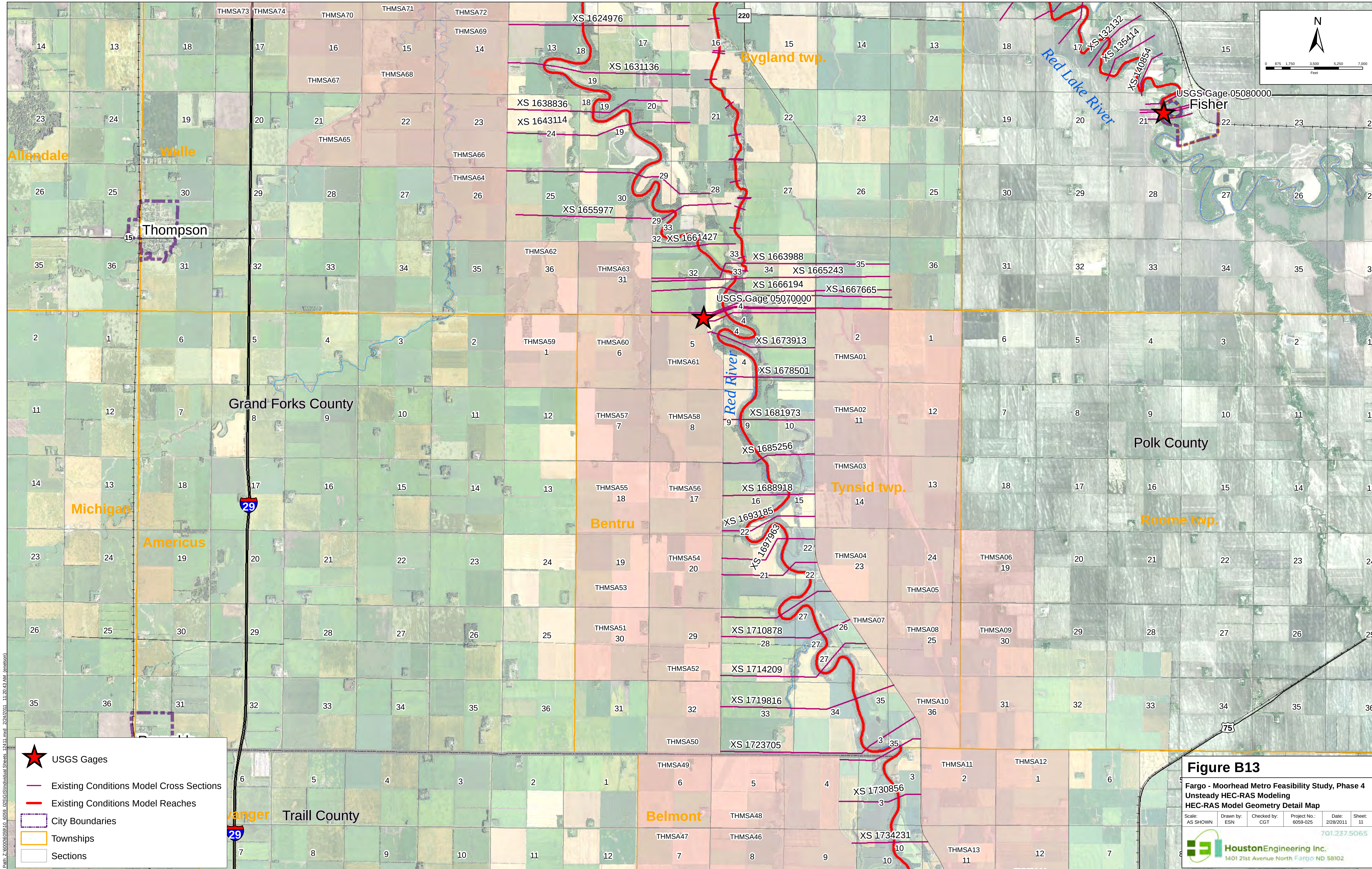

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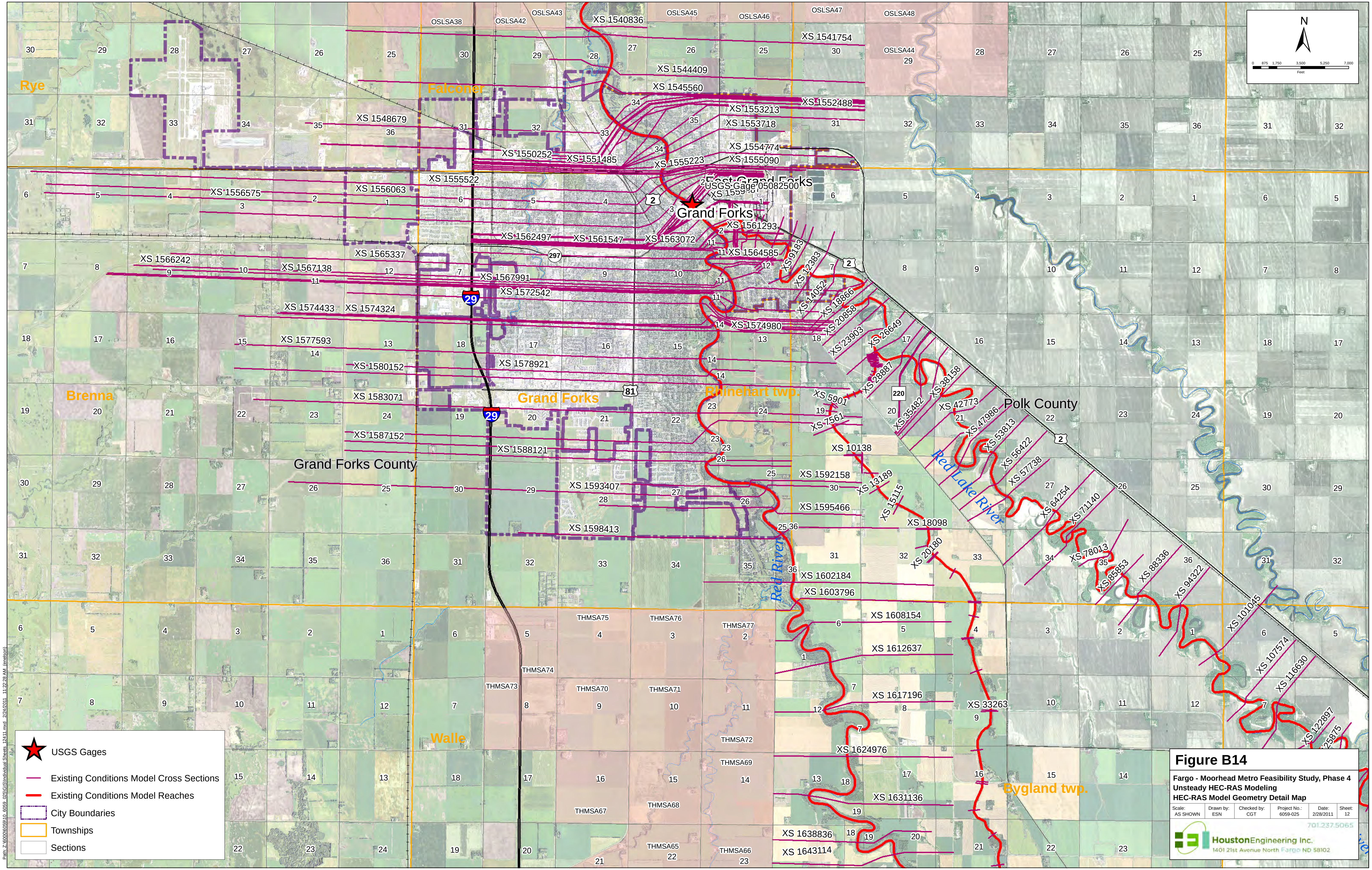

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Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
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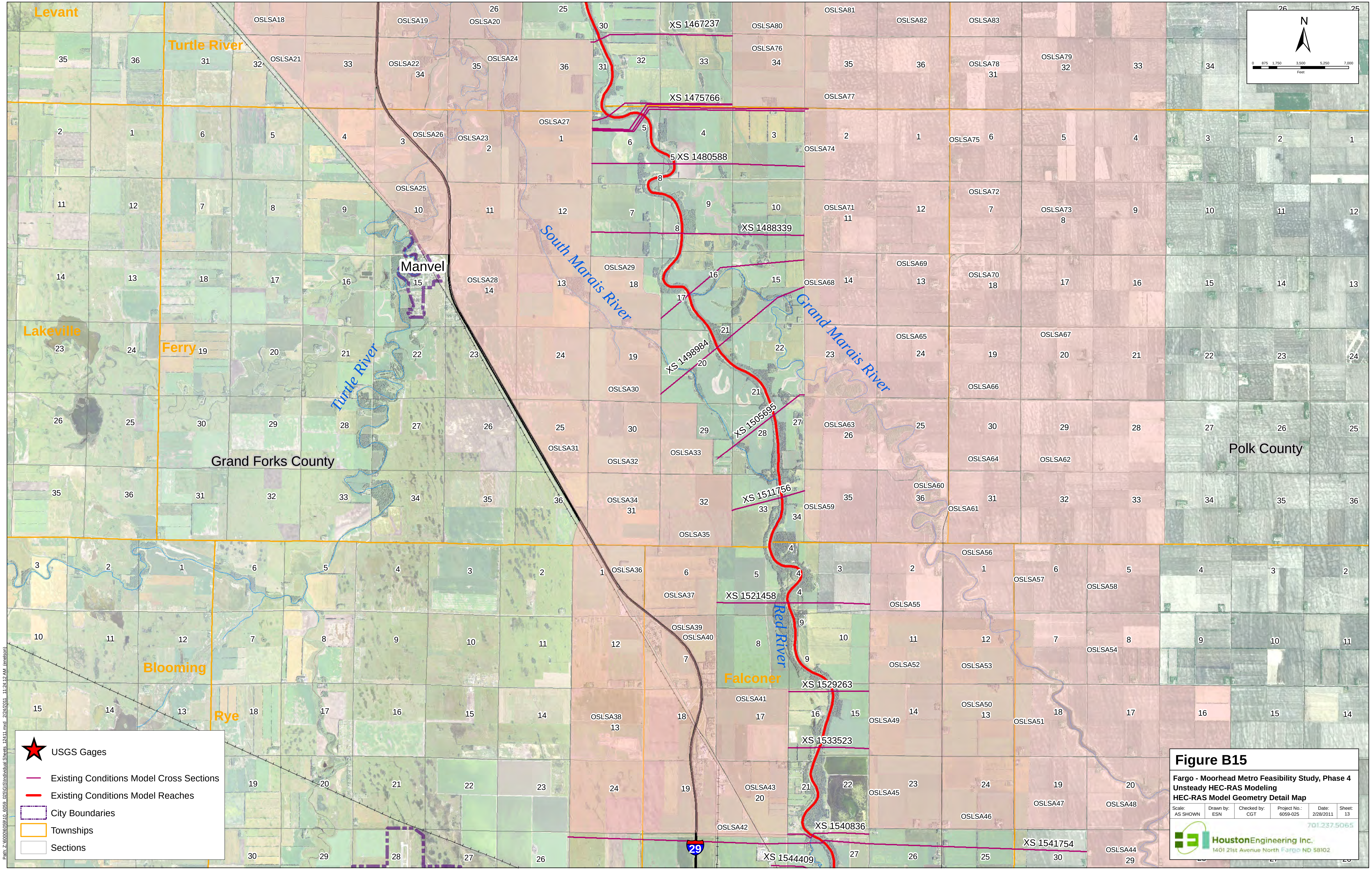
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USGS Gages
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Ferry
Blooming
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Manvel
Grand Forks County
Polk County
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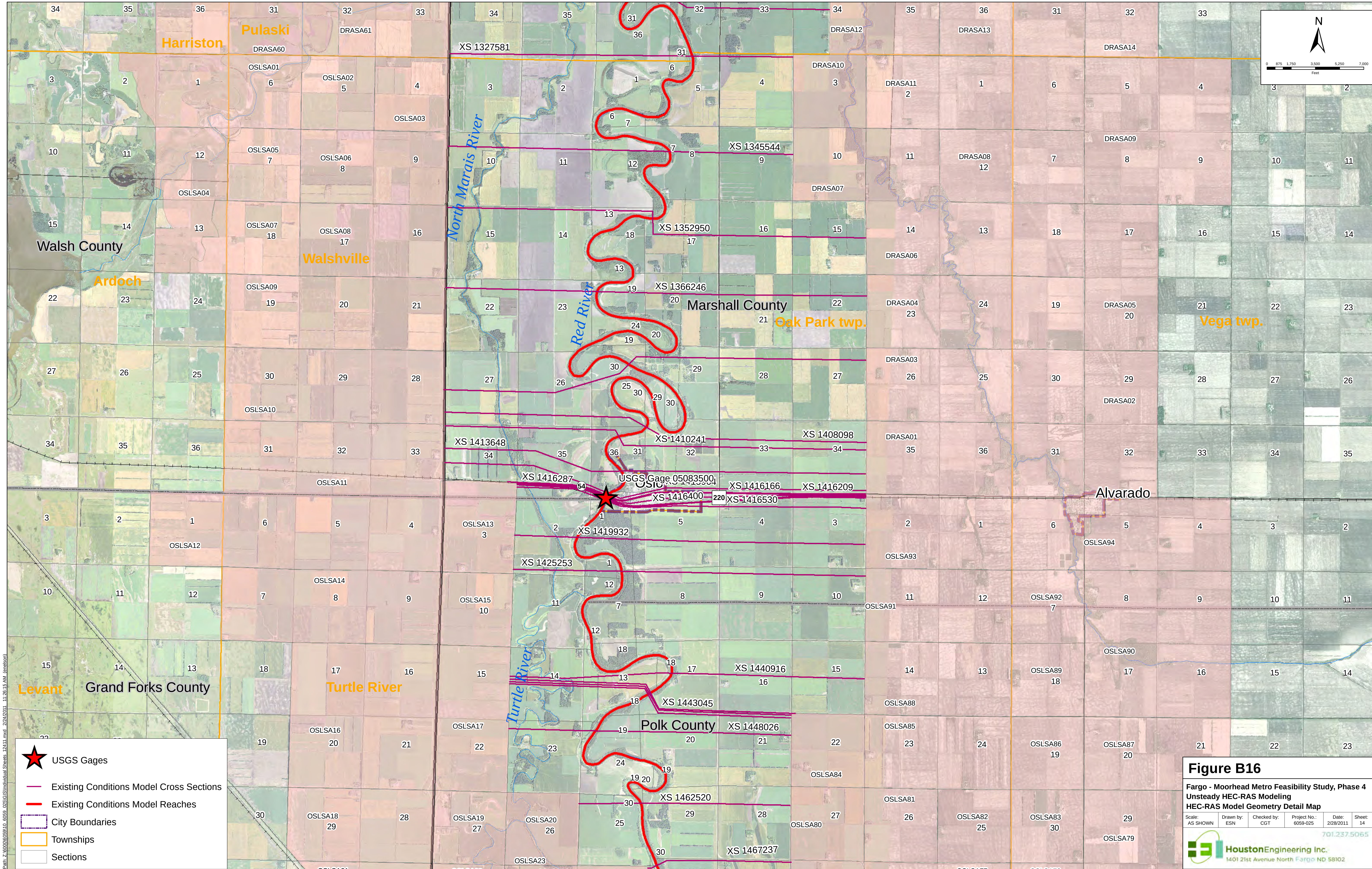

Figure B16

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
HEC-RAS Model Geometry Detail Map

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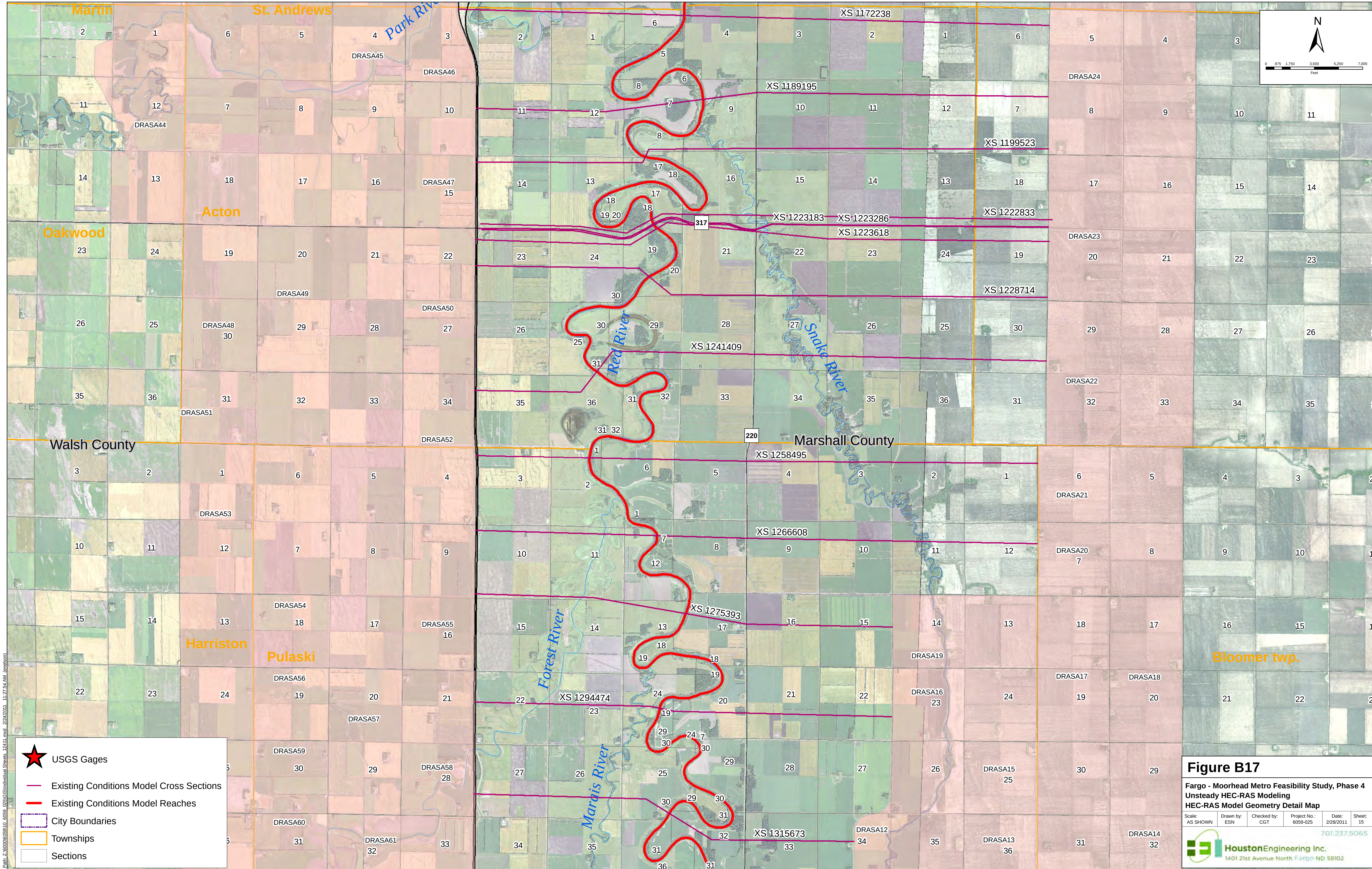

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Forest River
Marais River
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Existing Conditions Model Reaches
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Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
HEC-RAS Model Geometry Detail Map
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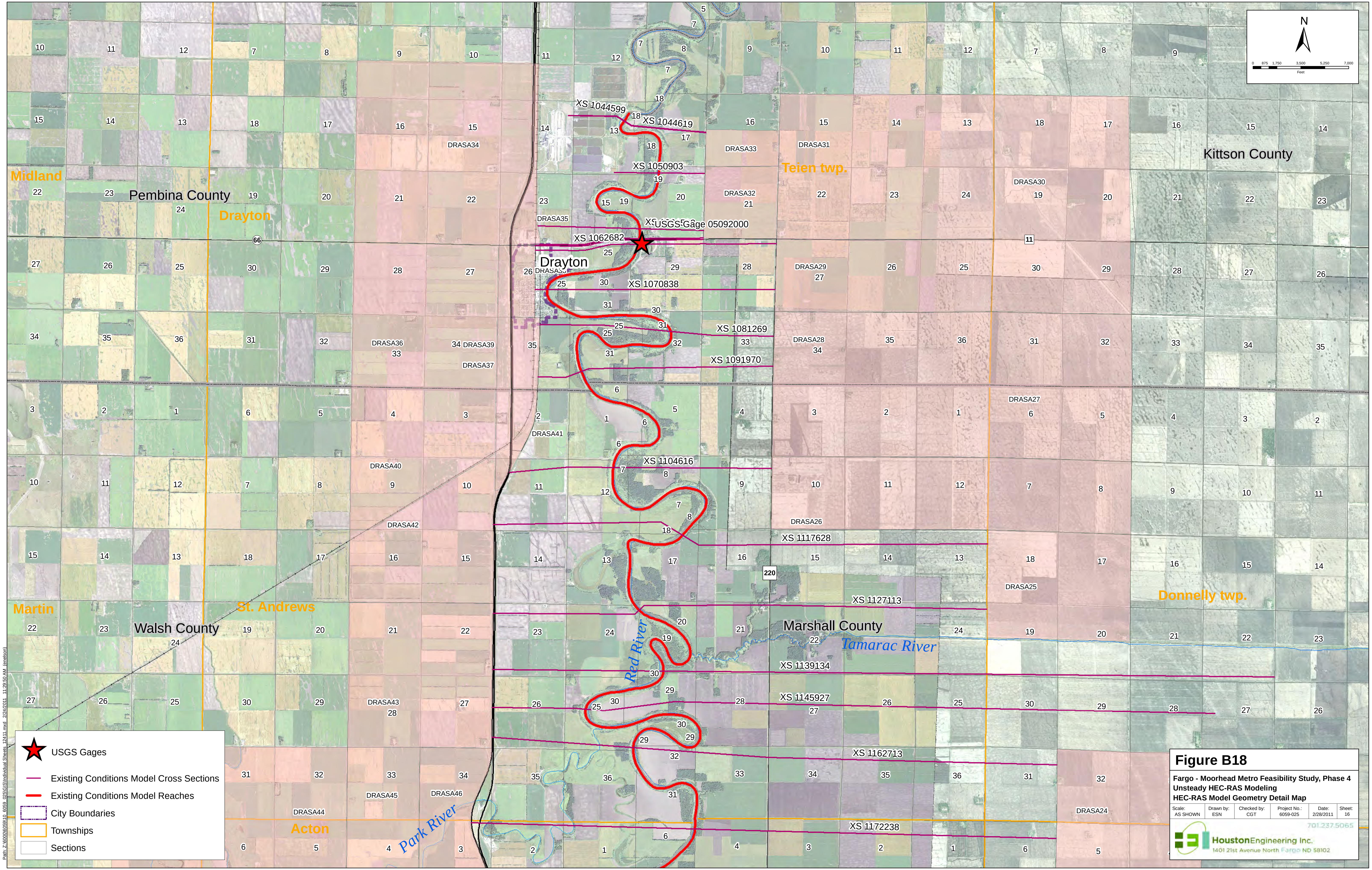
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St. Andrews
Acton
Donnelly twp.
Drayton
Red River
Tamarac River
Park River
USGS Gage 05092000
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Existing Conditions Model Reaches
City Boundaries
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Figure B18
Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
HEC-RAS Model Geometry Detail Map
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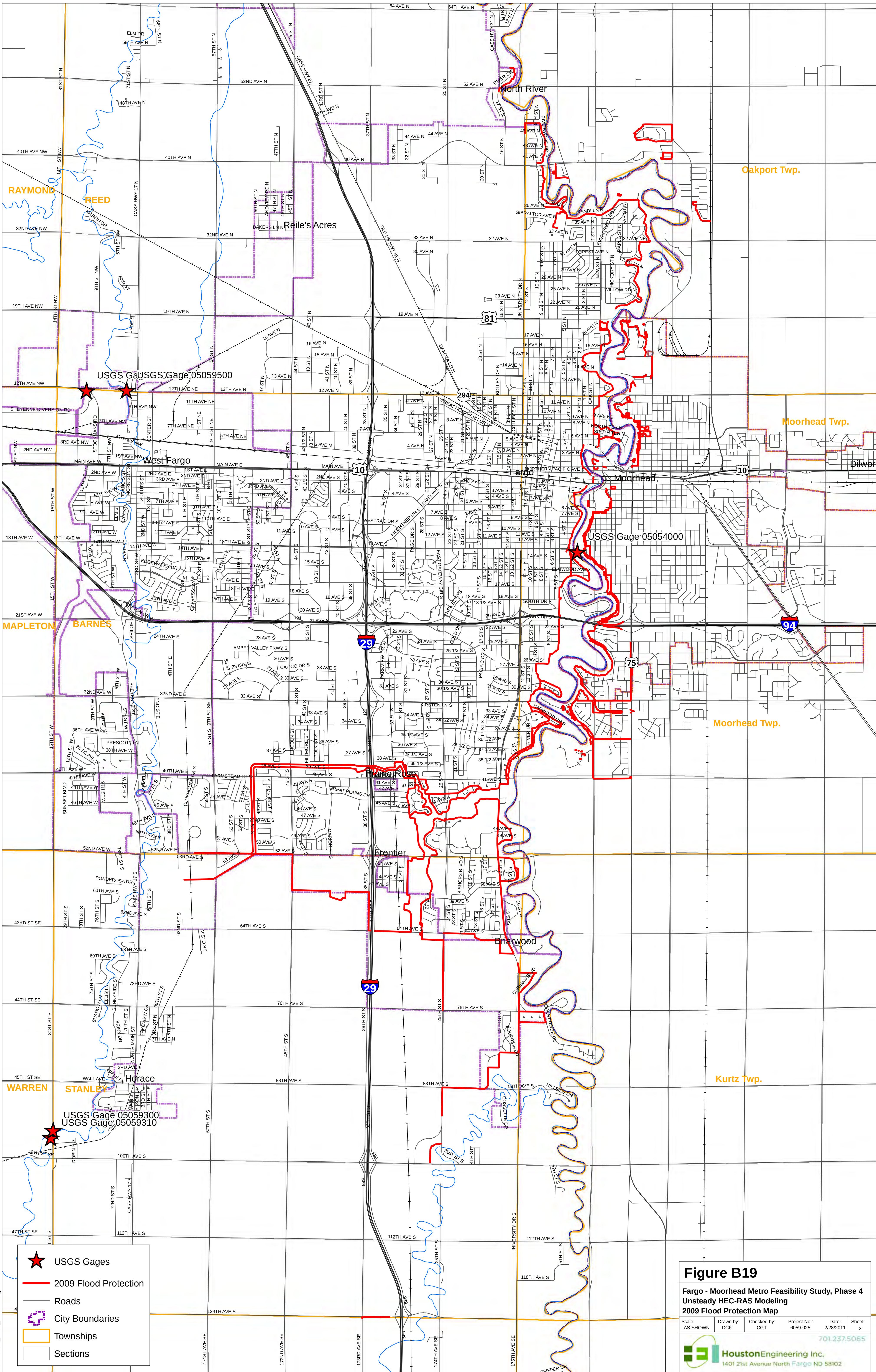

Figure B19

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
2009 Flood Protection Map

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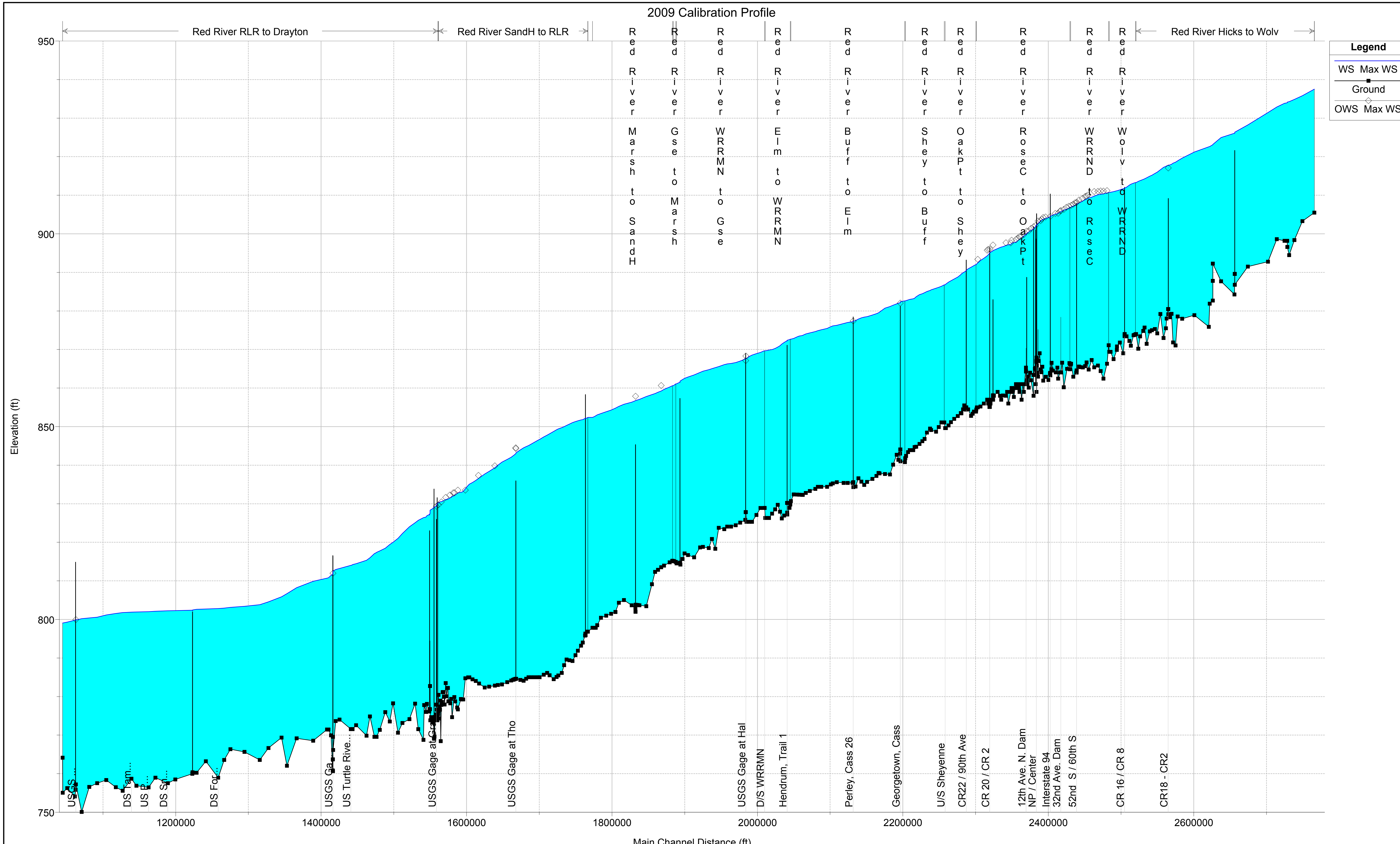

Figure B20 - 2009 Flood Calibration Profile for Red River of the North

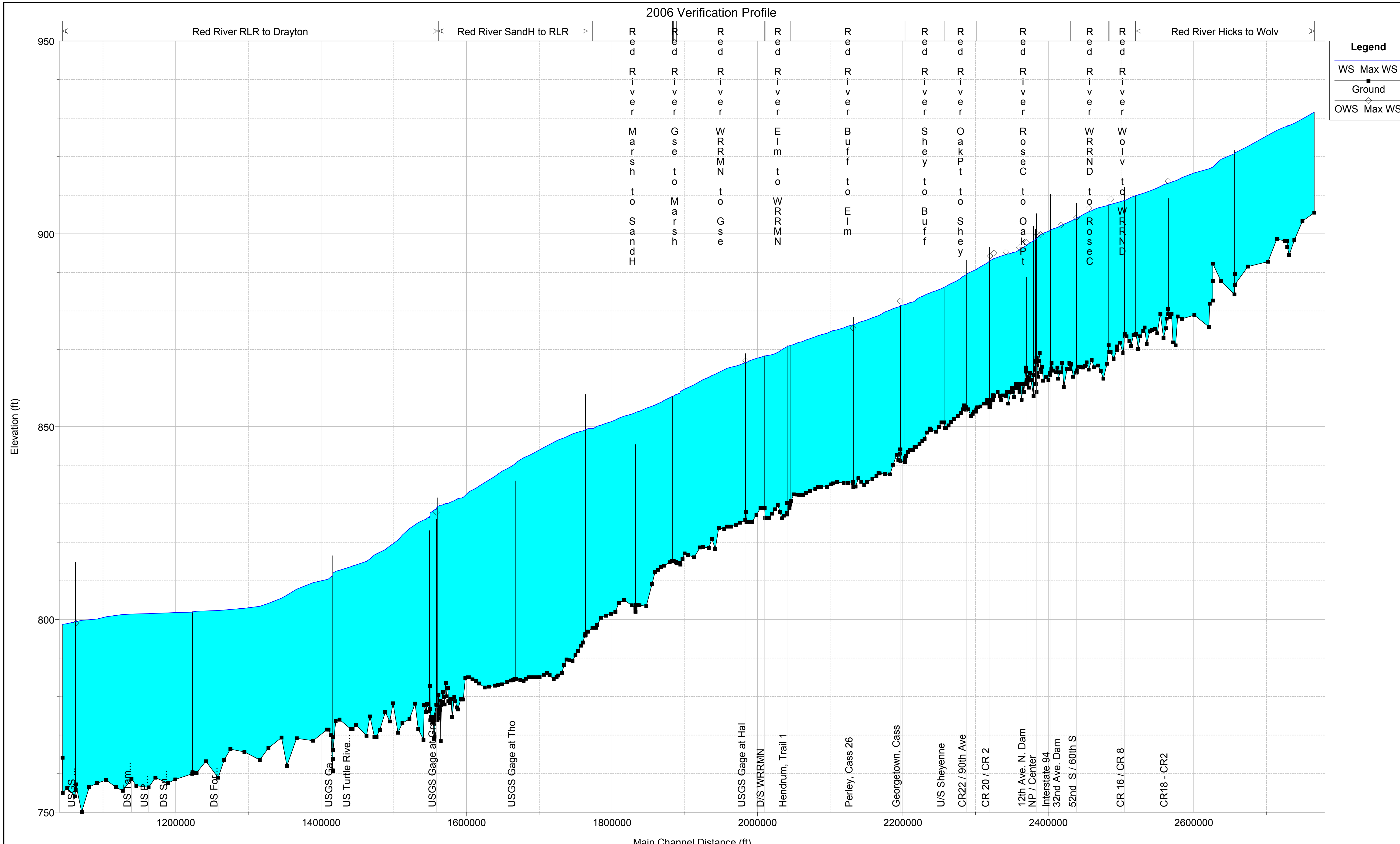

Figure B21 - 2006 Flood Verification Profile for Red River of the North

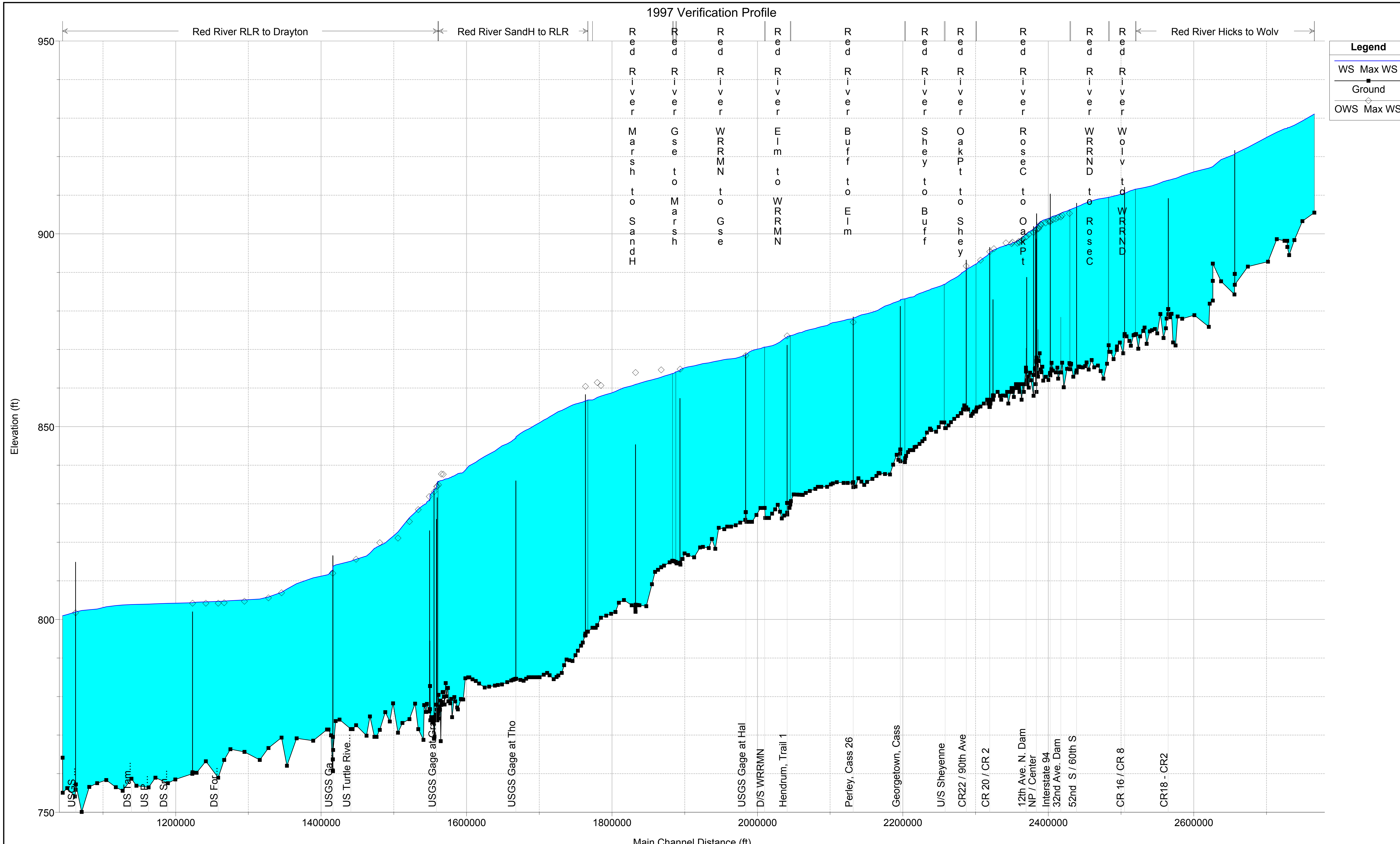

Figure B22 - 1997 Flood Verification Profile for Red River of the North

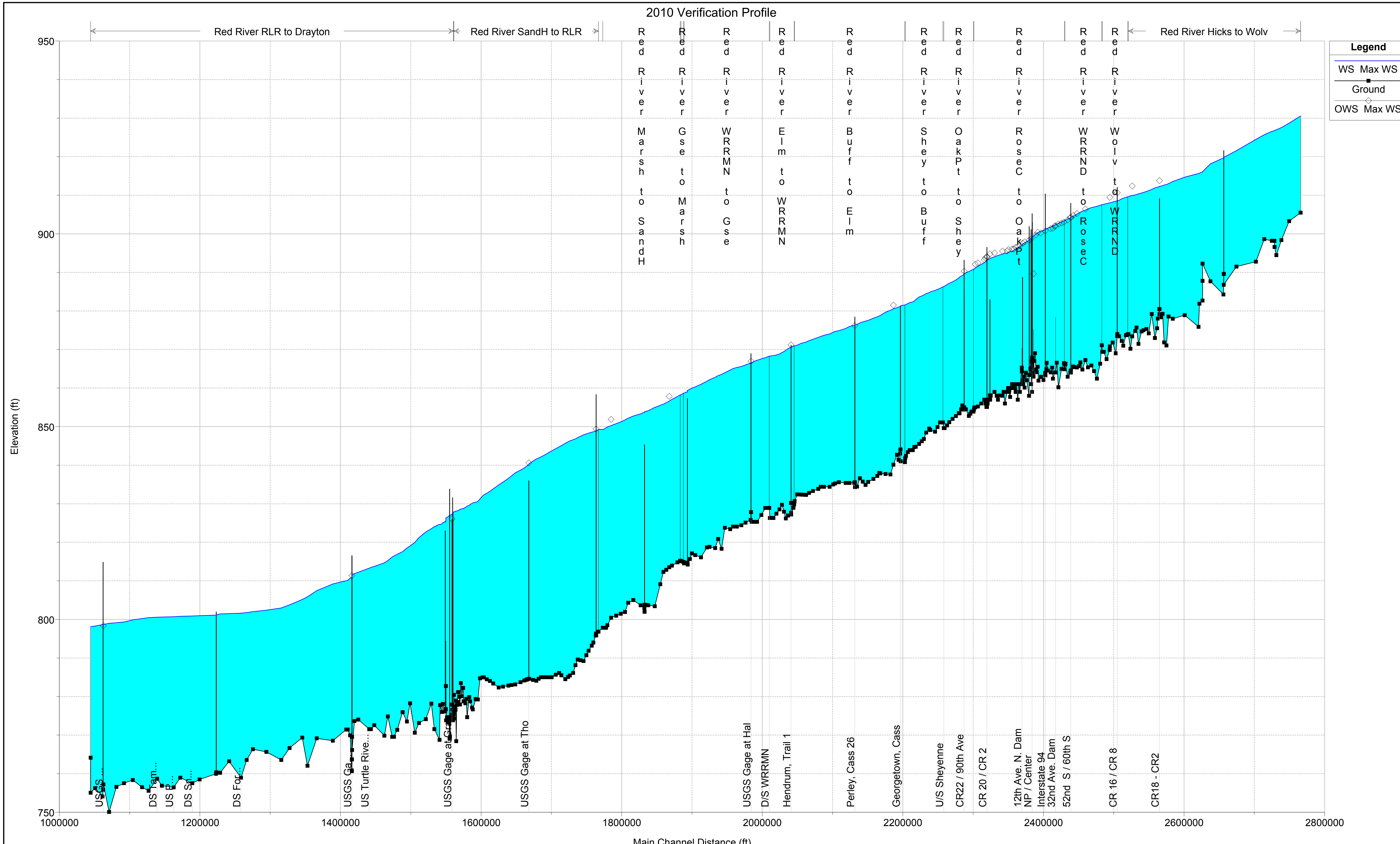

Figure B23 - 2010 Flood Verification Profile for Red River of the North

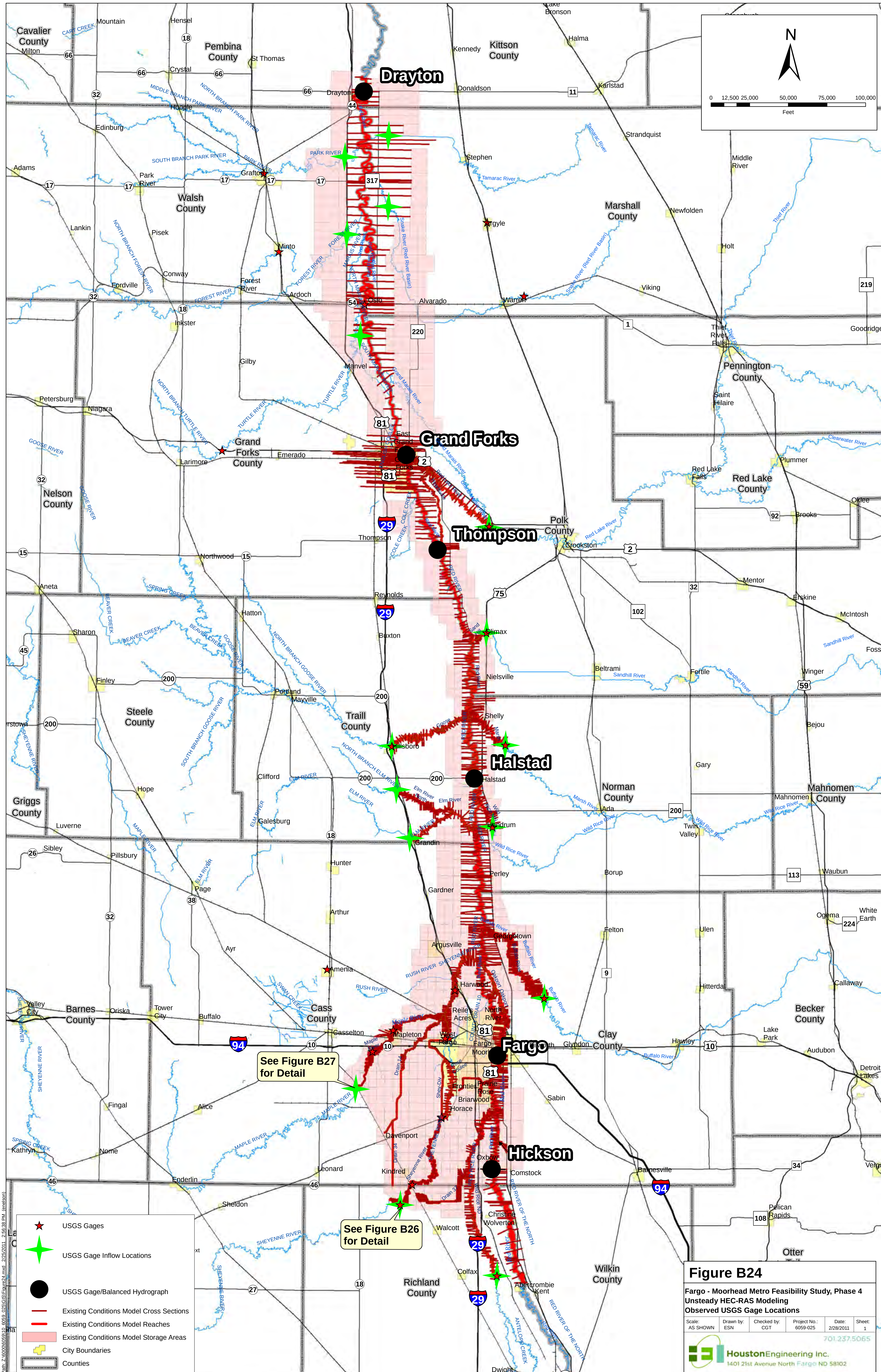

Figure B24

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
Observed USGS Gage Locations

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Figure B25

2009 Red River Upstream Inflow Hydrograph

See page B-16 within Appendix B

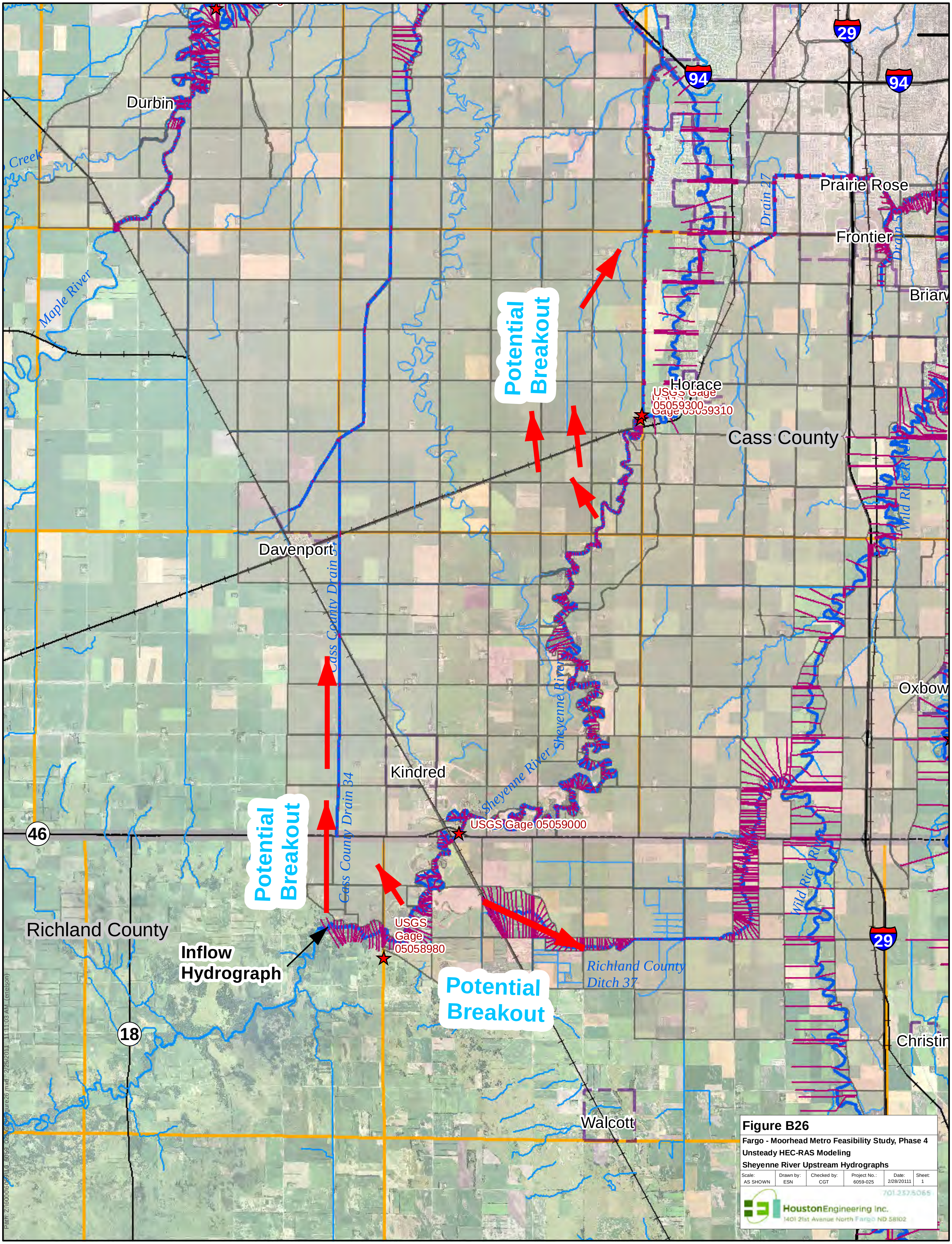

Figure B26

Fargo - Moorhead Metro Feasibility Study, Phase 4

Unsteady HEC-RAS Modeling

Sheyenne River Upstream Hydrographs

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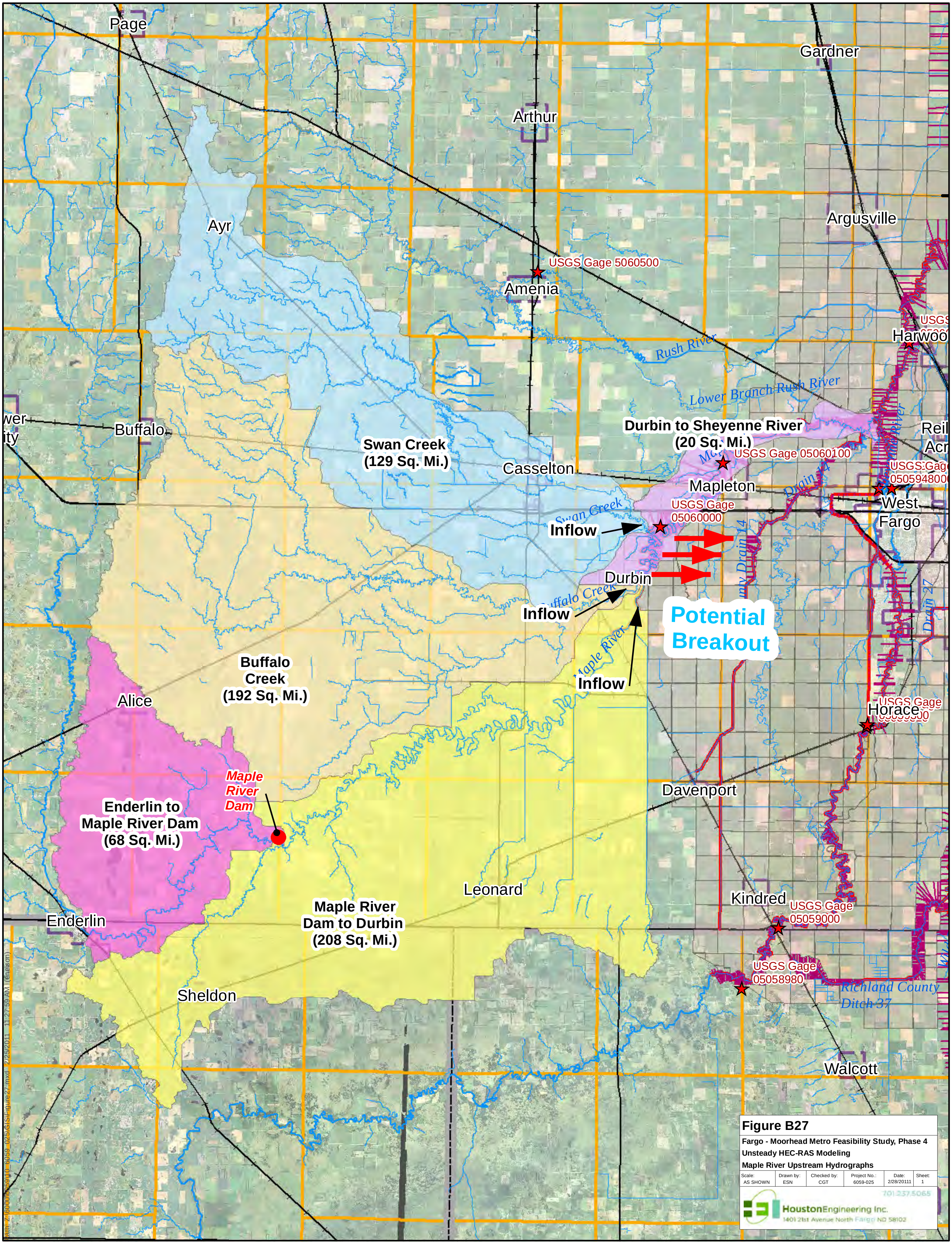
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Rush River
Lower Branch Rush River
Buffalo
Durbin to Sheyenne River
(20 Sq. Mi.)
Swan Creek
(129 Sq. Mi.)
USGS Gage 05060100
Casselton
Mapleton
West
Fargo
USGS Gage
05060000
Swan Creek
Inflow
Durbin
Inflow
Potential
Breakout
Maple River
Inflow
Buffalo
Creek
(192 Sq. Mi.)
Alice
Horace
Maple
River
Dam
Davenport
Enderlin to
Maple River Dam
(68 Sq. Mi.)
Leonard
Kindred
USGS Gage
05059000
Enderlin
Maple River
Dam to Durbin
(208 Sq. Mi.)
USGS Gage
05058980
Richland County
Ditch 37
Sheldon
Walcott
Figure B27
Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
Maple River Upstream Hydrographs
Scale: AS SHOWN
Drawn by: ESN
Checked by: CGT
Project No.: 6059-025
Date: 2/28/20111
Sheet: 1
701.237.5065
HoustonEngineering Inc.
1401 21st Avenue North Fargo ND 58102

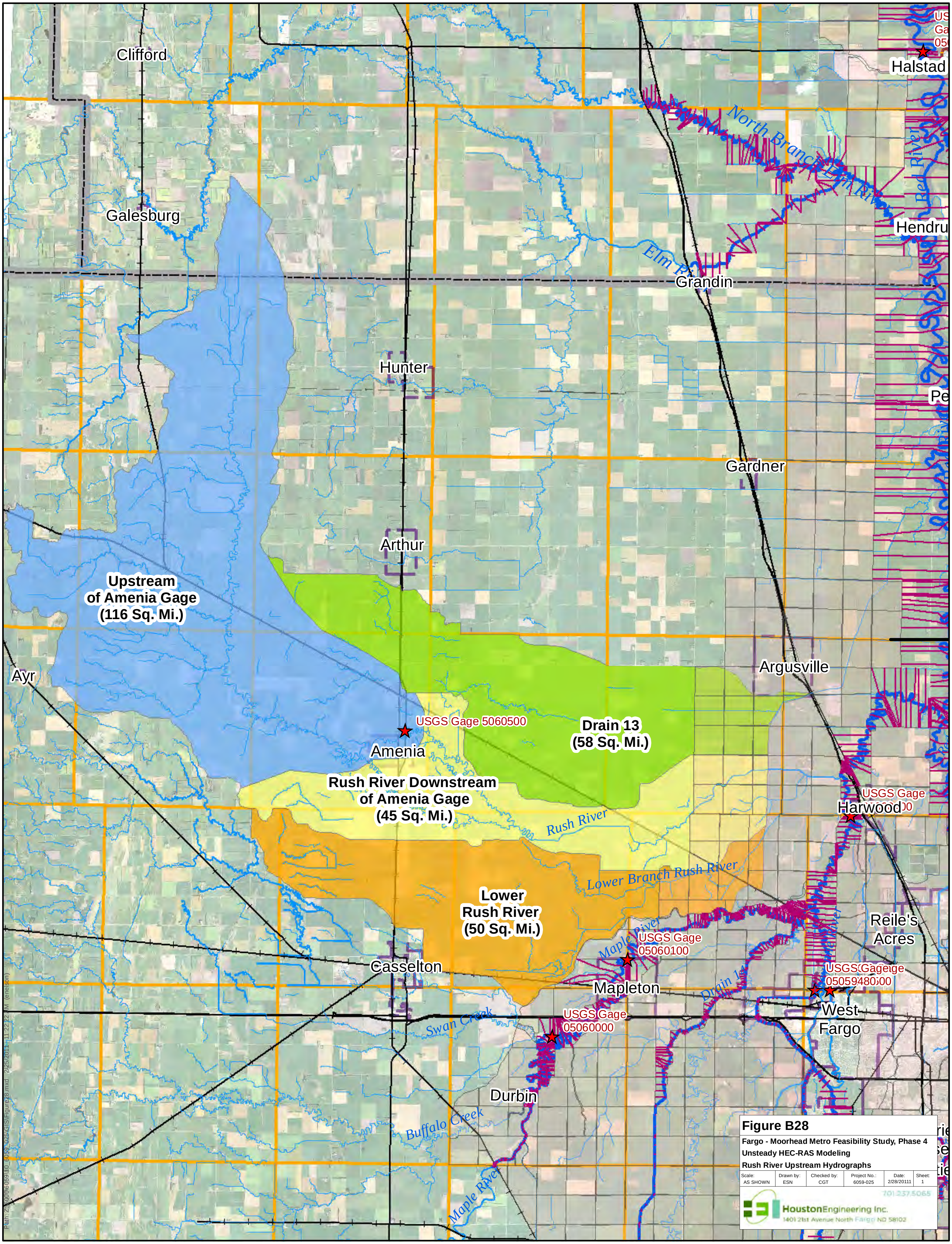

Figure B28					
Fargo - Moorhead Metro Feasibility Study, Phase 4 Unsteady HEC-RAS Modeling Rush River Upstream Hydrographs					
Scale: AS SHOWN	Drawn by: ESN	Checked by: CGT	Project No.: 6059-025	Date: 2/28/20111	Sheet: 1

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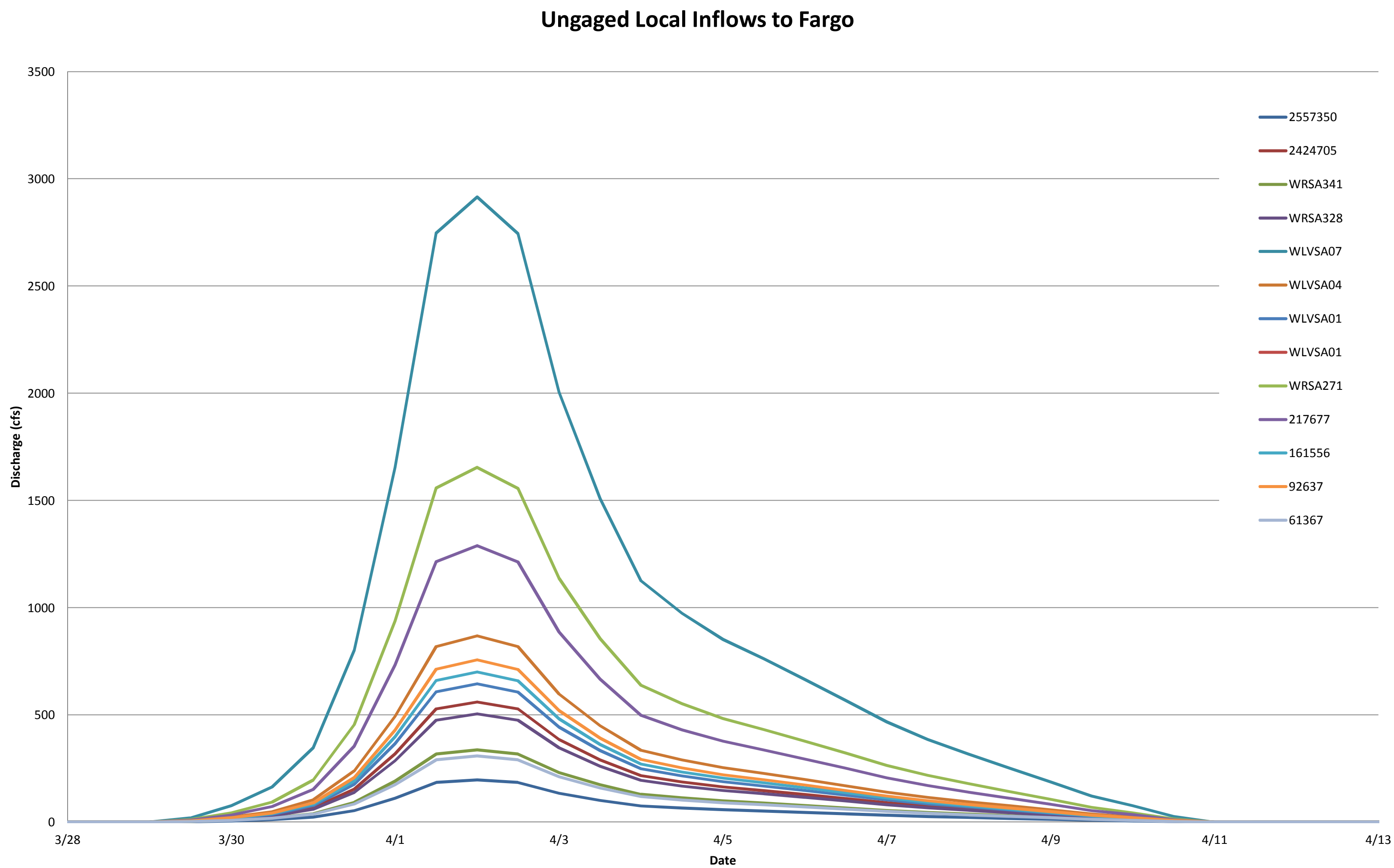
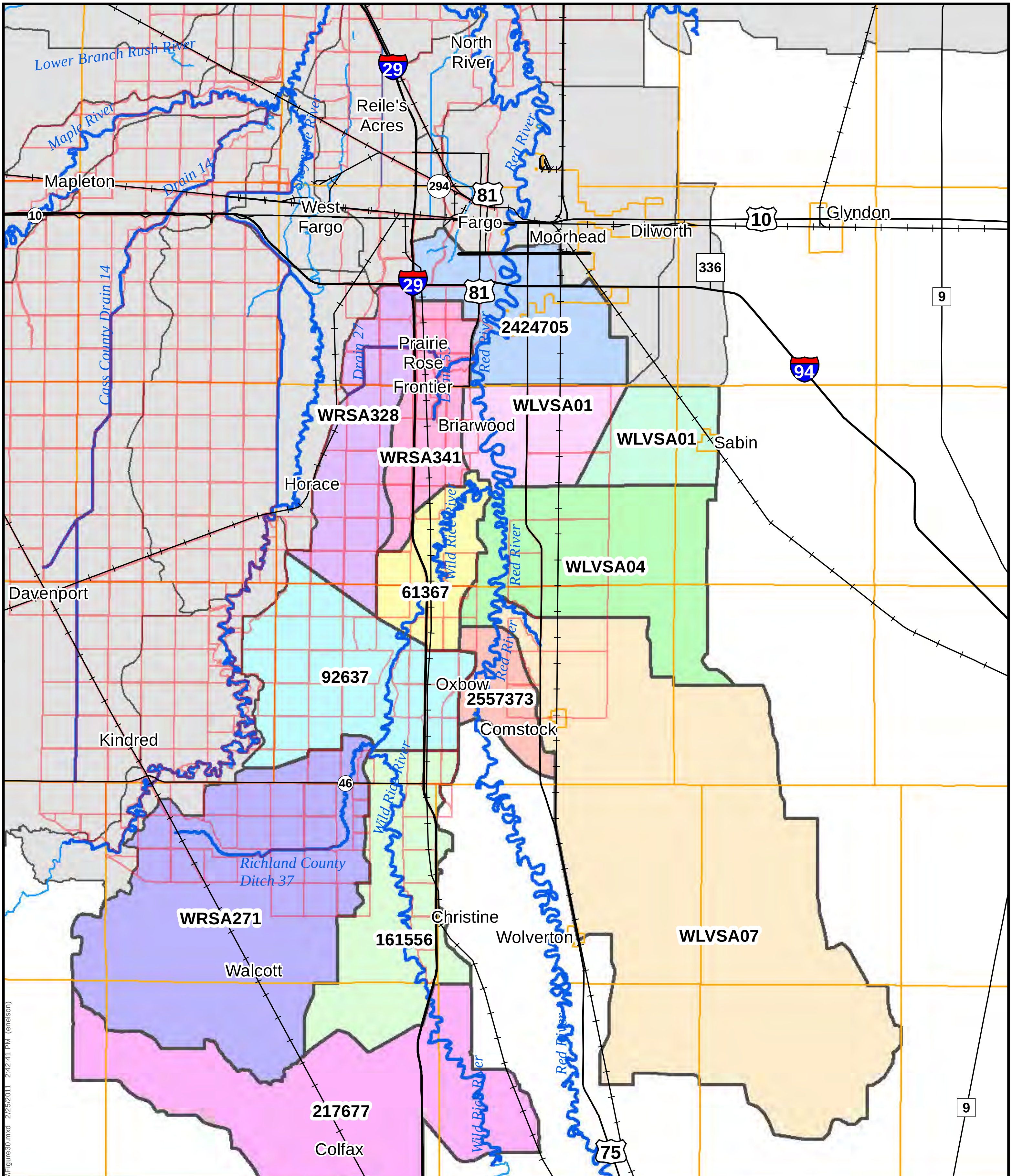
Figure B29 - Upstream of Fargo Gage Local Inflow Table and Hydrographs

River	Red River	Red River	Red River	Red River	Red River	Red River	Red River	Red River	WRRND	WRRND	WRRND	WRRND	WRRND
Name	Hickson to NDDIV	Rose Coulee to Fargo Gage	Drain 53	Drain 27	Wolverton Creek	MN Local NDDIV to WRRND	MN Local WRRND to Rose Coulee - A	MN Local WRRND to Rose Coulee - B	Richland Co. West	Abercrombie to I-29	I-29 to 104884	92637 to ND DIV	ND DIV to RRN
Sta_From	**2557350**	**2424705**	**WRSA341**	**WRSA328**	**WLVSA07**	**WLVSA04**	**WLVSA01**	**WLVSA01**	**WRSA271**	**217677**	**161556**	**92637**	**61367**
Sta_To	2531315	2388223	Storage Area	Storage Area	Storage Area	Storage Area	Storage Area	Storage Area	Storage Area	171972	103632	74680	10468
Area_SM	7	20	12	18	104	31	12	11	59	46	25	27	11





Path: Z:\6000\6059110_6059_025\GIS\Figure30.mxd 2/25/2011 2:42:41 PM (enelson)

Figure B30

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
Upstream of Fargo Sub-Basin Delineation Map

Scale:	Drawn by:	Checked by:	Project No.:	Date:	Sheet:
AS SHOWN	ESN	CGT	6059-025	2/28/20111	1

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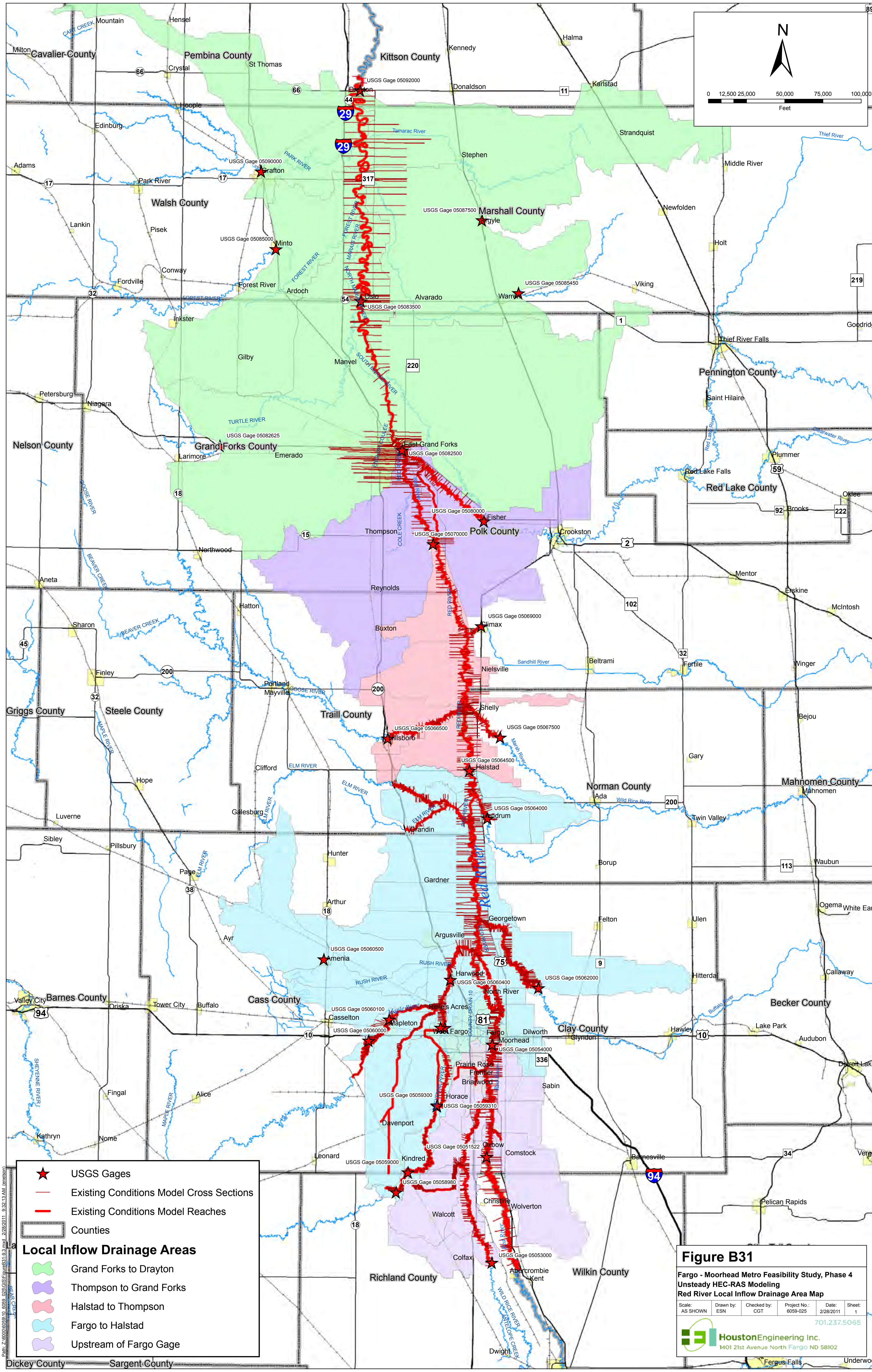

Figure B31

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
Red River Local Inflow Drainage Area Map

Appendix B – Hydraulics
Existing Condition

Figure B32

Ungaged Local Inflow Hydrograph Example

See page B-22 within Appendix B

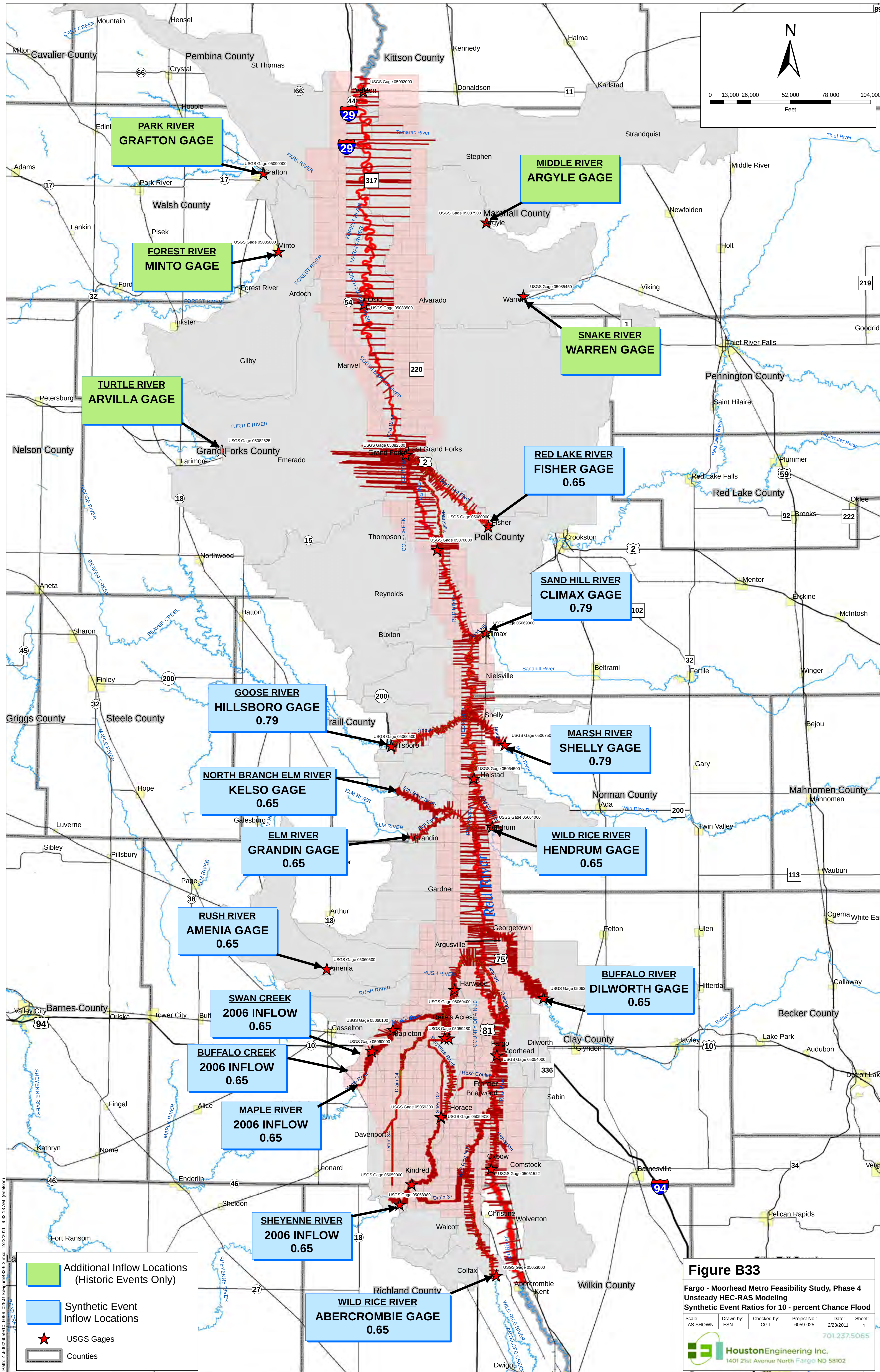

Figure B33

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
Synthetic Event Ratios for 10 - percent Chance Flood

Scale:	Drawn by:	Checked by:	Project No.:	Date:	Sheet:
AS SHOWN	ESN	CGT	6059-025	2/23/2011	1

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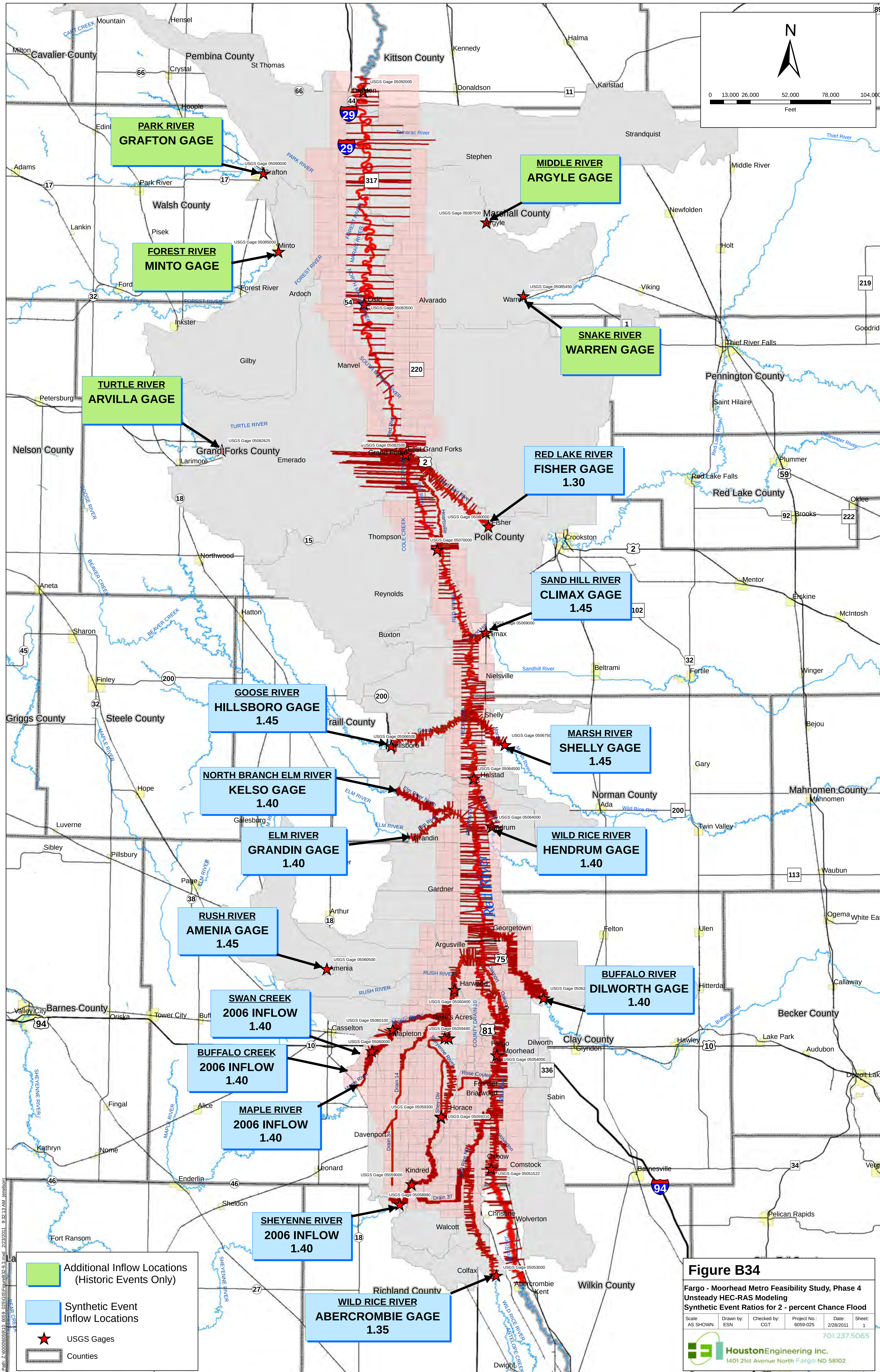

Figure B34
Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
Synthetic Event Ratios for 2 - percent Chance Flood

Scale:	Drawn by:	Checked by:	Project No.:	Date:	Sheet:
AS SHOWN	ESN	CGT	6059-025	2/28/2011	1

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701.237.5065

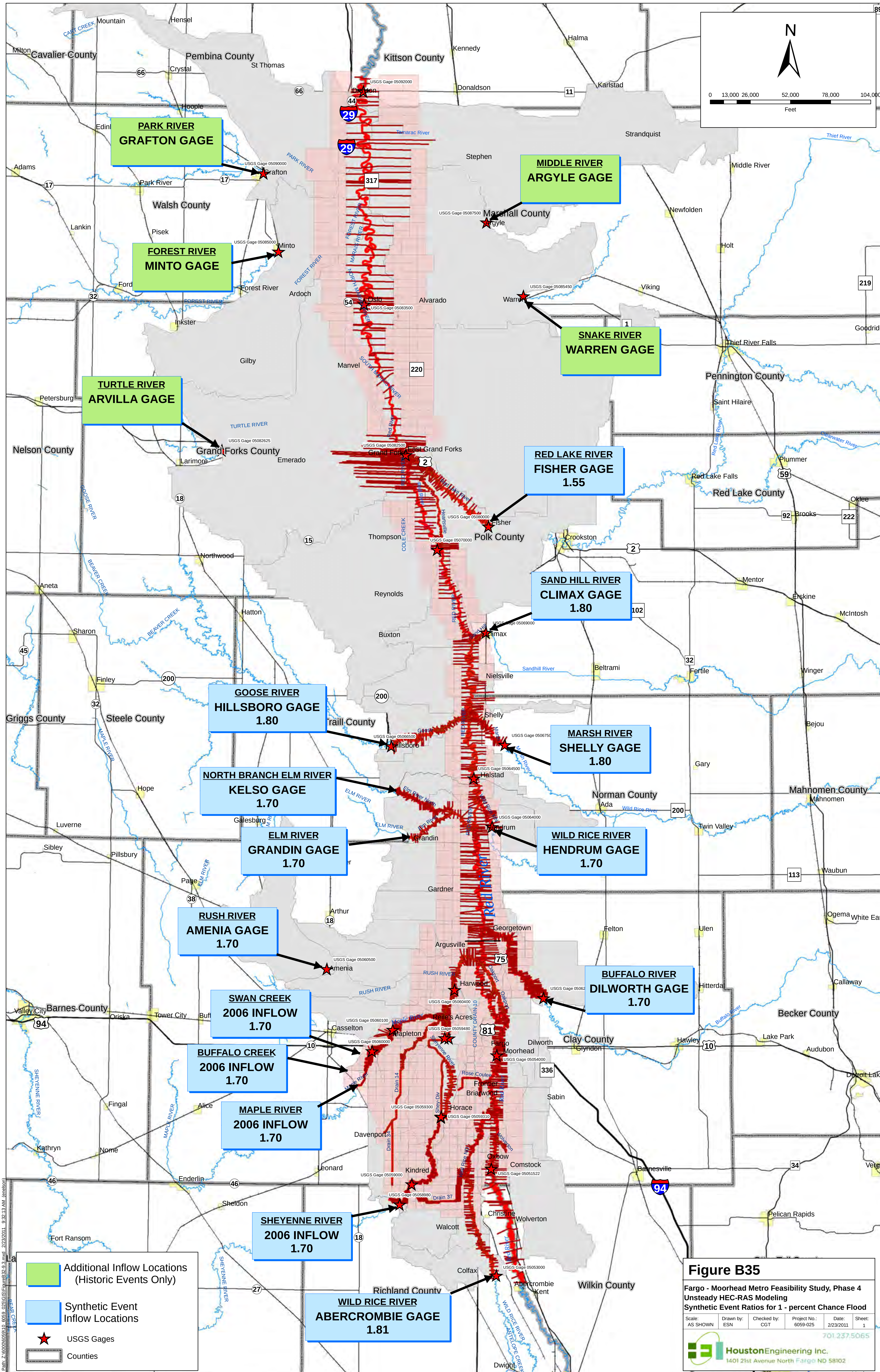

Figure B35

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
Synthetic Event Ratios for 1 - percent Chance Flood

Scale:	Drawn by:	Checked by:	Project No.:	Date:	Sheet:
AS SHOWN	ESN	CGT	6059-025	2/23/2011	1

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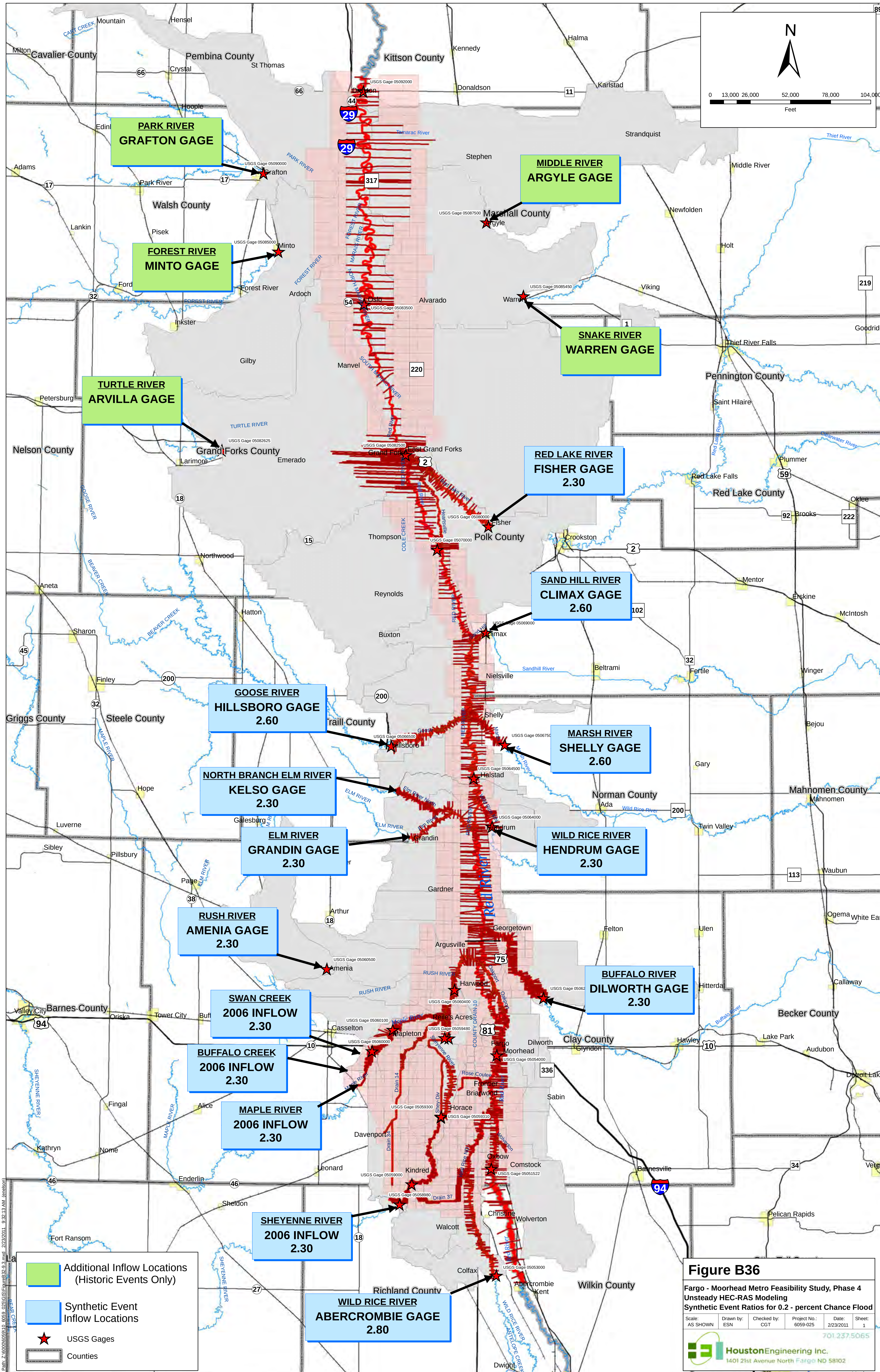

Figure B36

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
Synthetic Event Ratios for 0.2 - percent Chance Flood

Scale:	Drawn by:	Checked by:	Project No.:	Date:	Sheet:
AS SHOWN	ESN	CGT	6059-025	2/23/2011	1

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Houston Engineering Inc.
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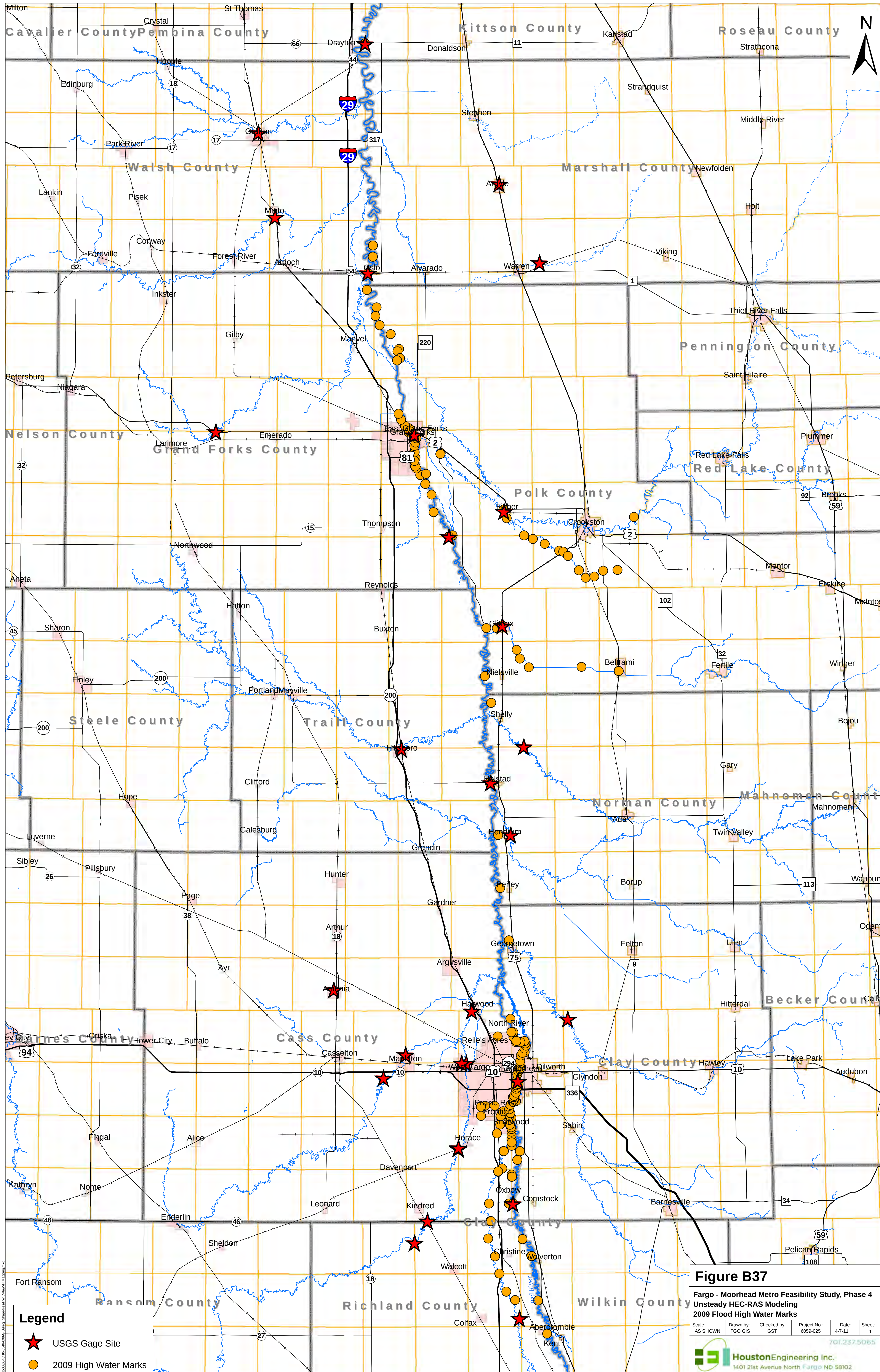

Figure B37

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
2009 Flood High Water Marks

Scale:	Drawn by:	Checked by:	Project No.:	Date:	Sheet:
AS SHOWN	FGO GIS	GST	6059-025	4-7-11	1

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HoustonEngineering Inc.
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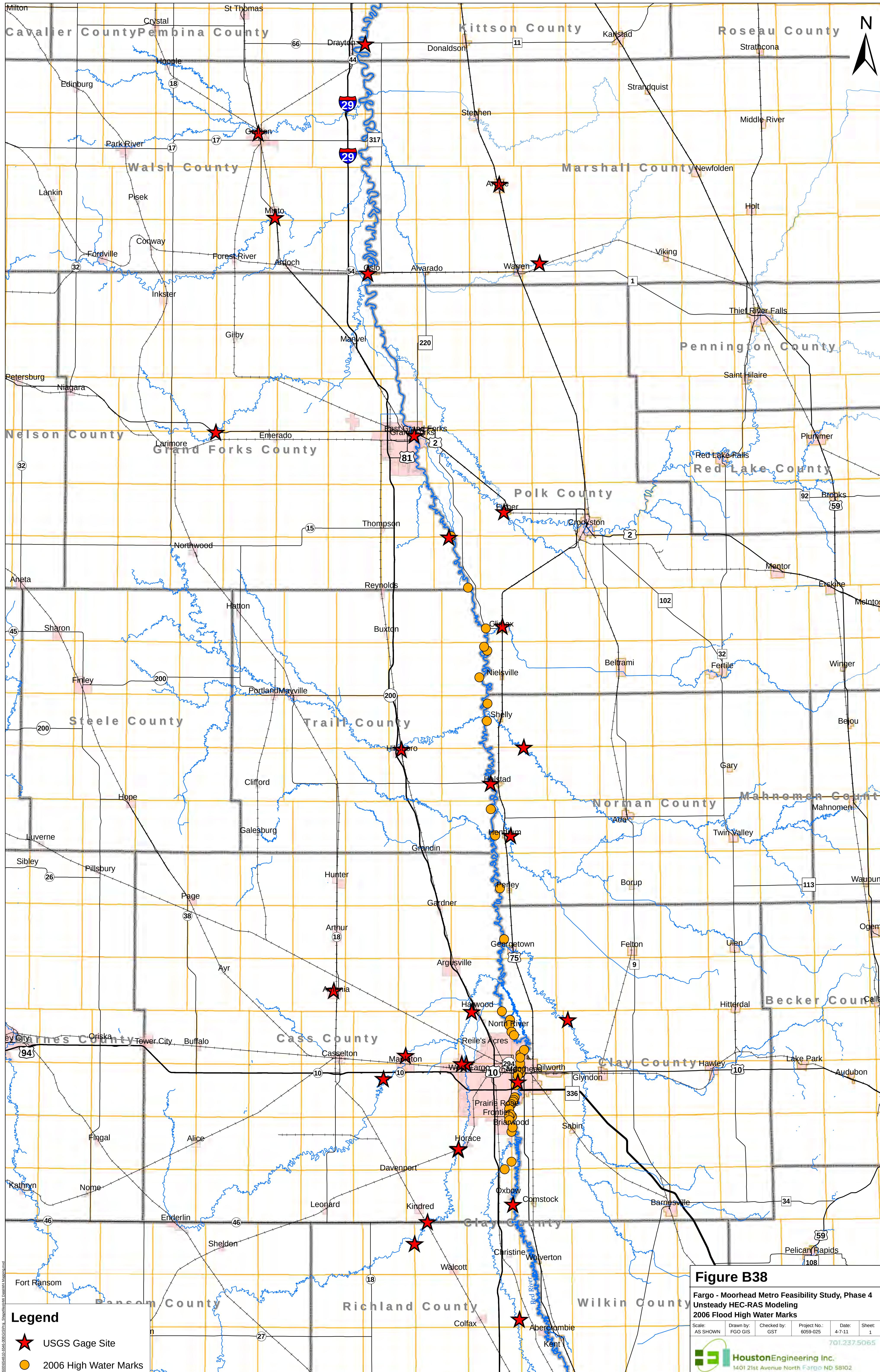


Figure B38

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
2006 Flood High Water Marks

Scale:	Drawn by:	Checked by:	Project No.:	Date:	Sheet:
AS SHOWN	FGO GIS	GST	6059-025	4-7-11	1

701.237.5065
Houston Engineering Inc.
1401 21st Avenue North Fargo ND 58102

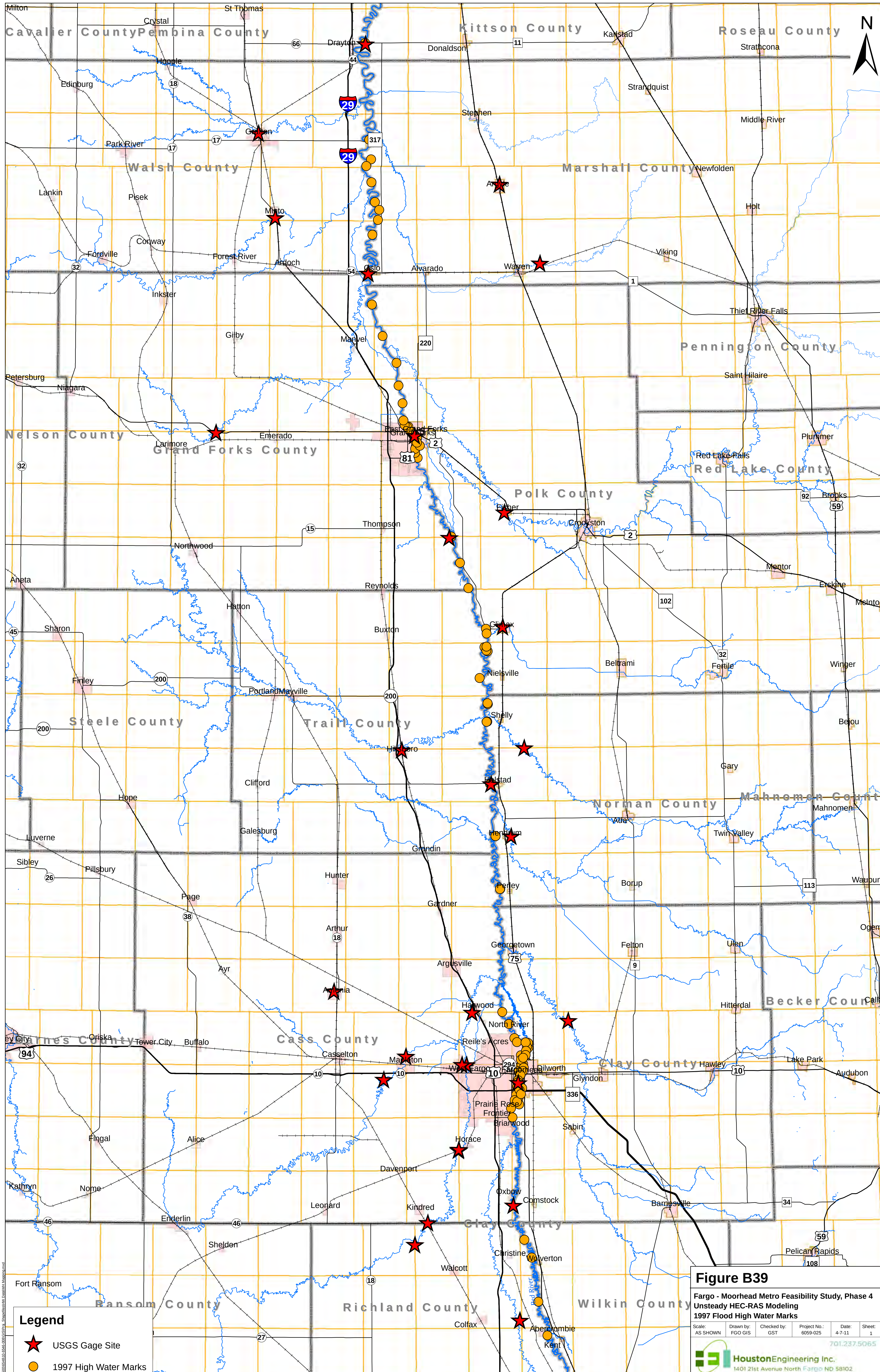


Figure B39

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
1997 Flood High Water Marks

Scale:	Drawn by:	Checked by:	Project No.:	Date:	Sheet:
AS SHOWN	FGO GIS	GST	6059-025	4-7-11	1

701.237.5065
HoustonEngineering Inc.
1401 21st Avenue North Fargo ND 58102

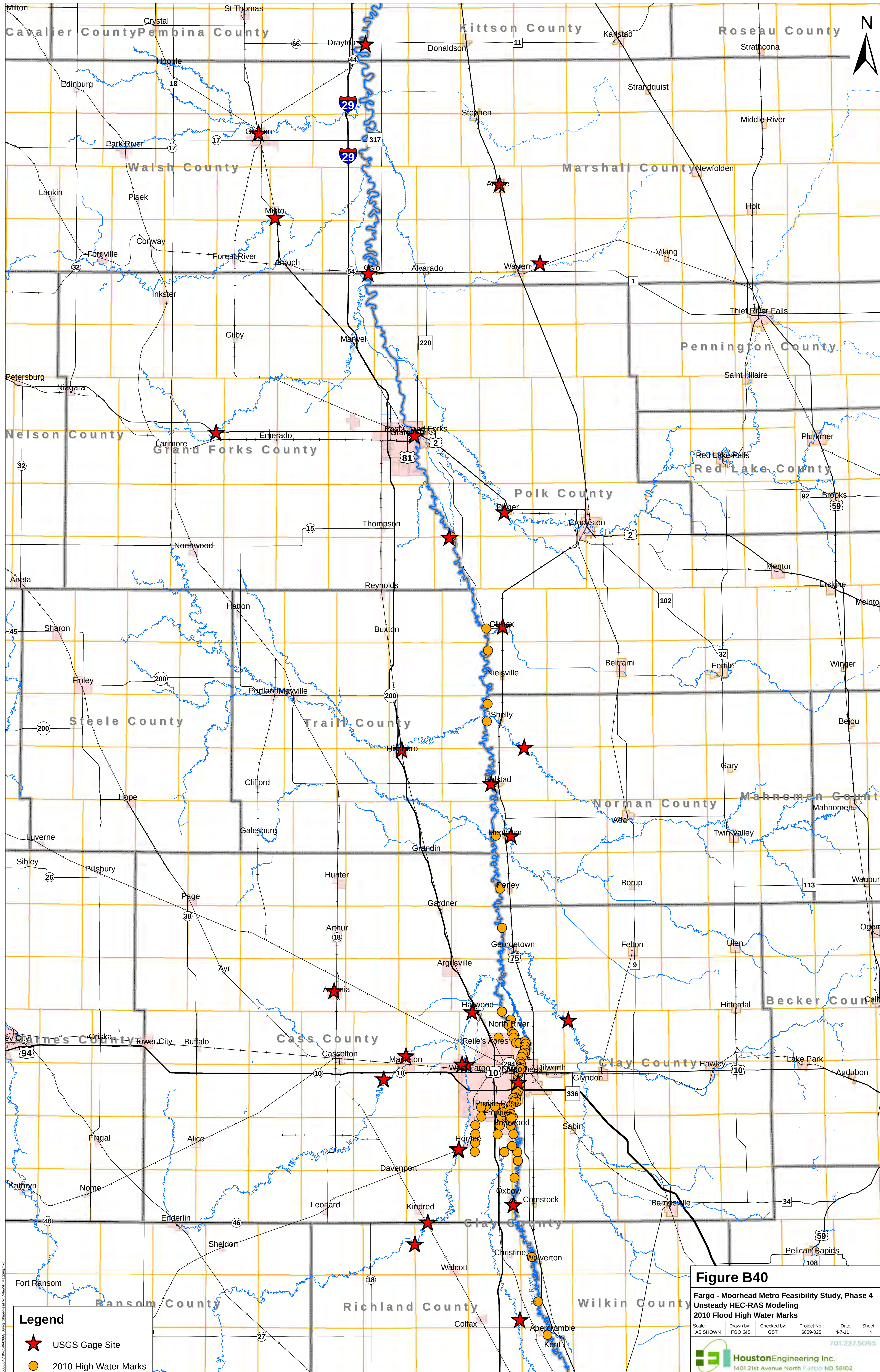


Figure B40

Fargo - Moorhead Metro Feasibility Study, Phase 4
Unsteady HEC-RAS Modeling
2010 Flood High Water Marks

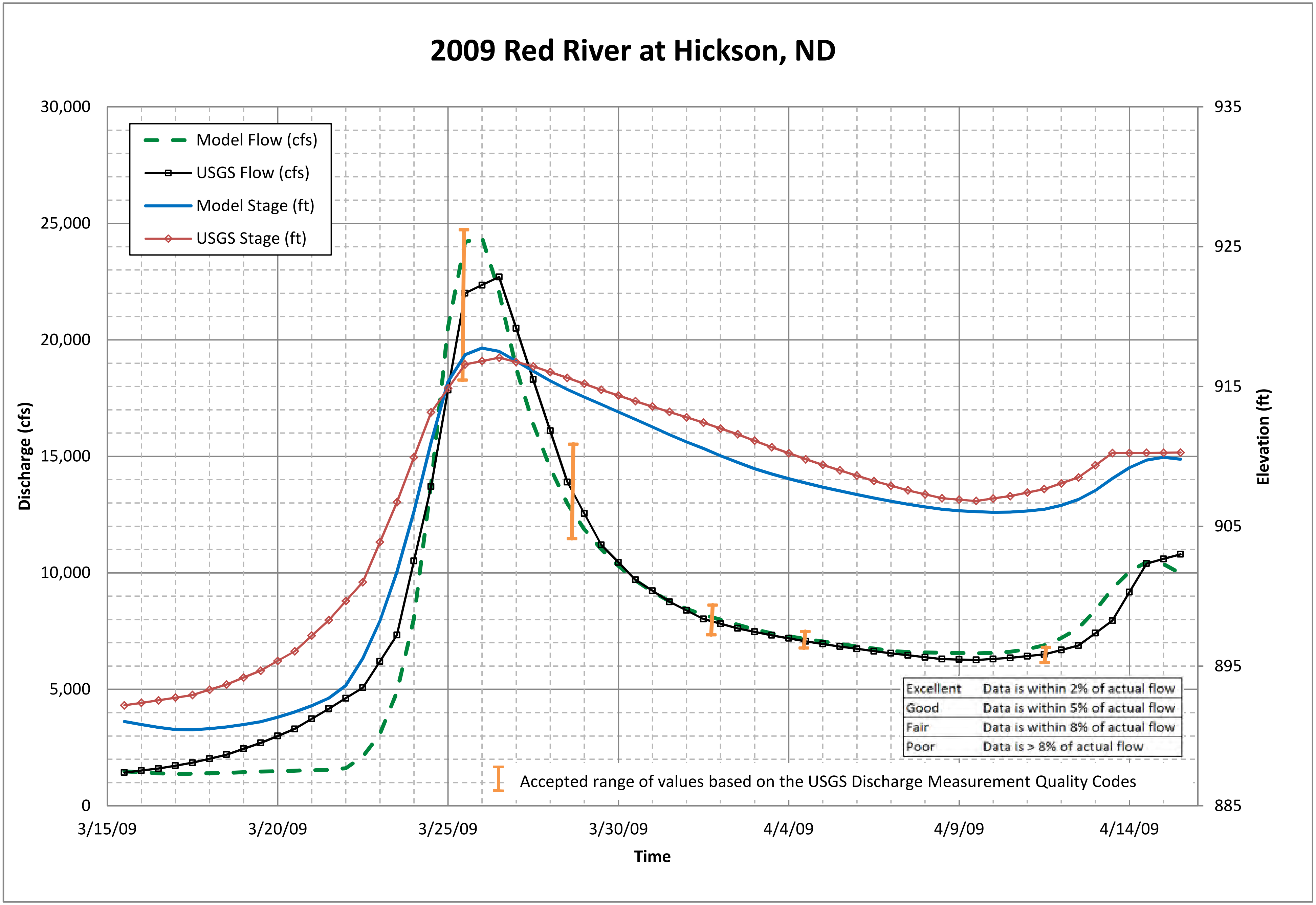
Scale:	Drawn by:	Checked by:	Project No.:	Date:	Sheet:
AS SHOWN	FGO GIS	GST	6059-025	4-7-11	1

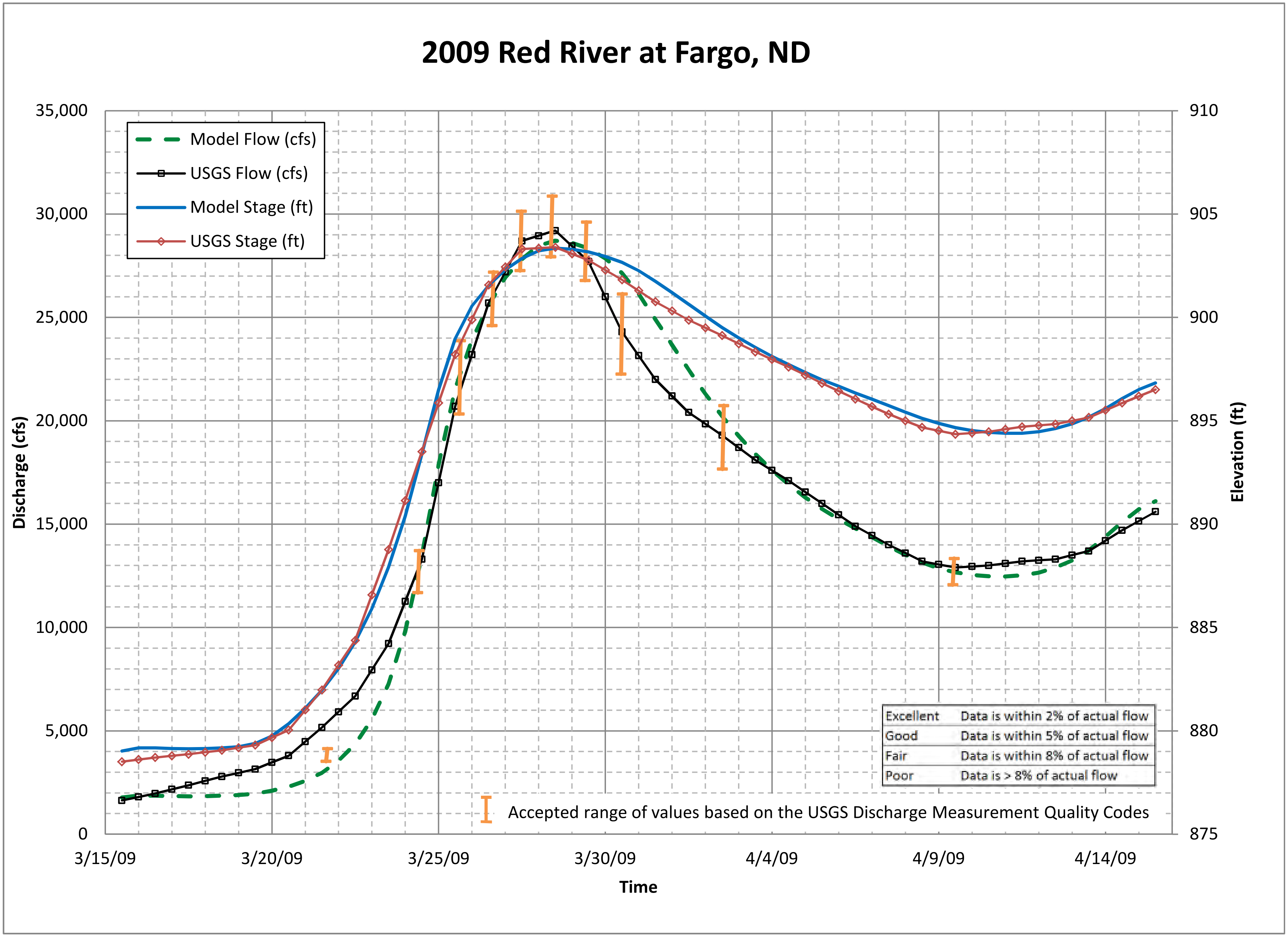
701.237.5065
HoustonEngineering Inc.
1401 21st Avenue North Fargo ND 58102

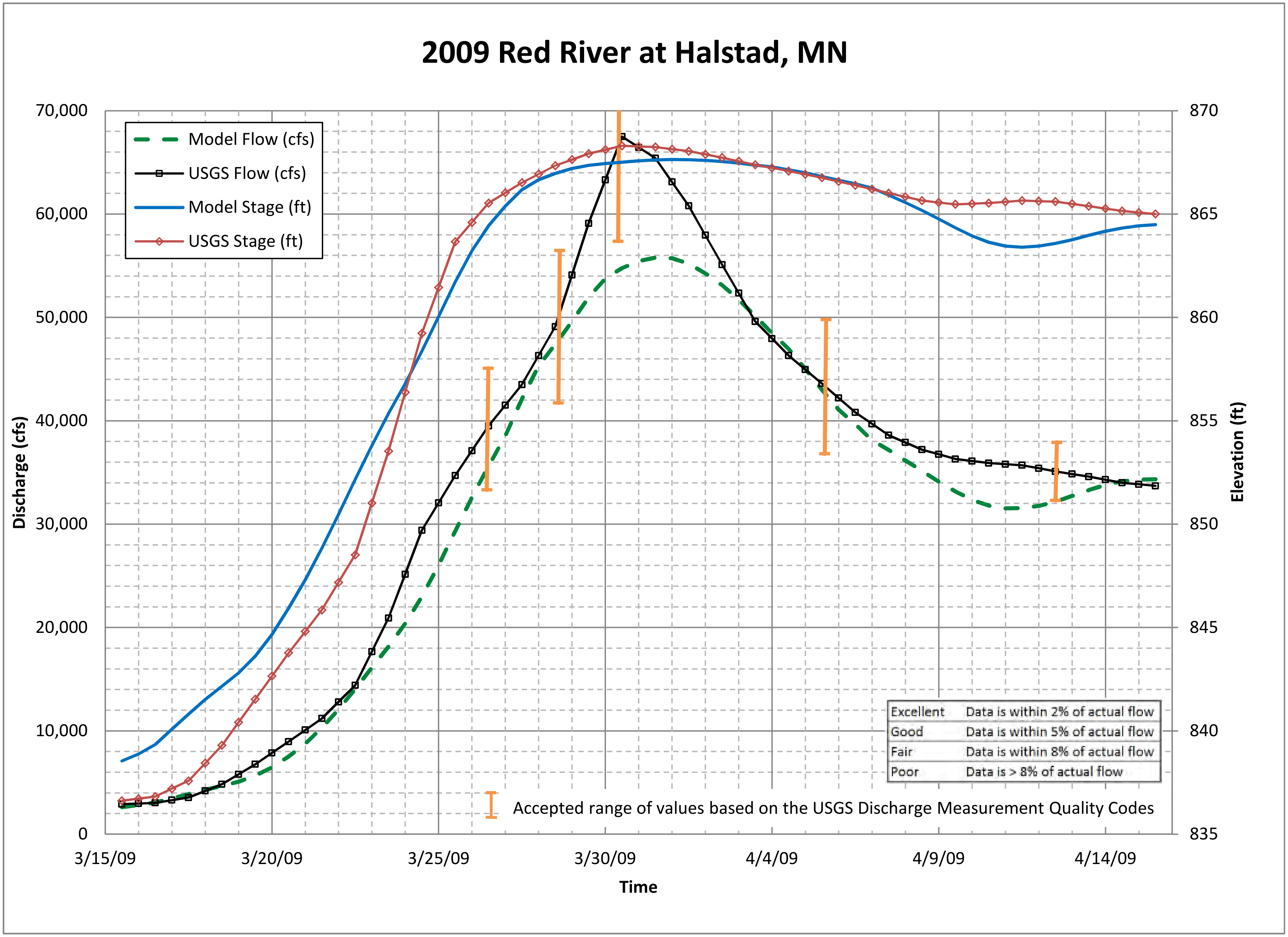
Appendix B – Hydraulics
Existing Condition

Exhibit A

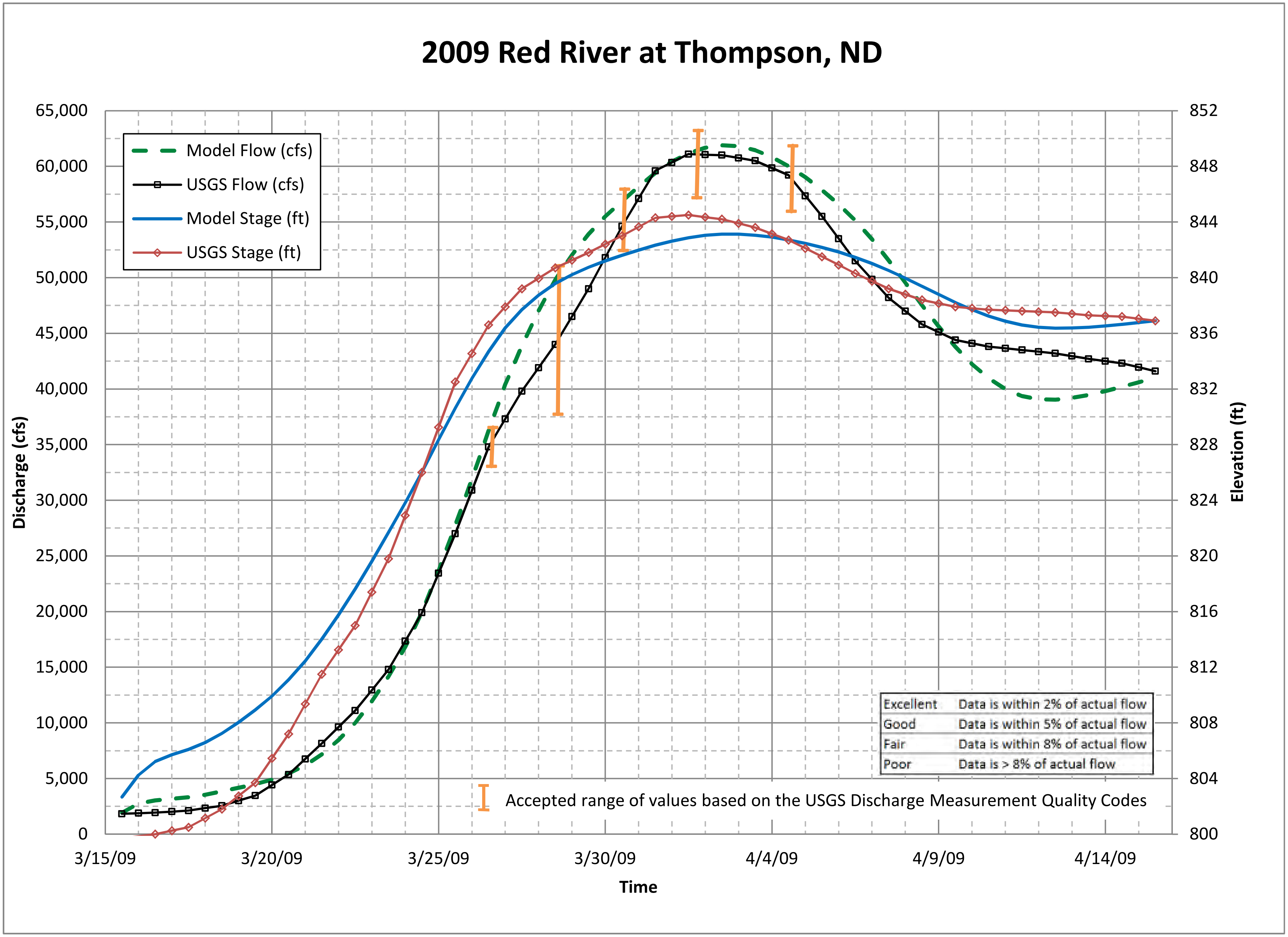
2009 Flood Calibration Discharge Hydrographs

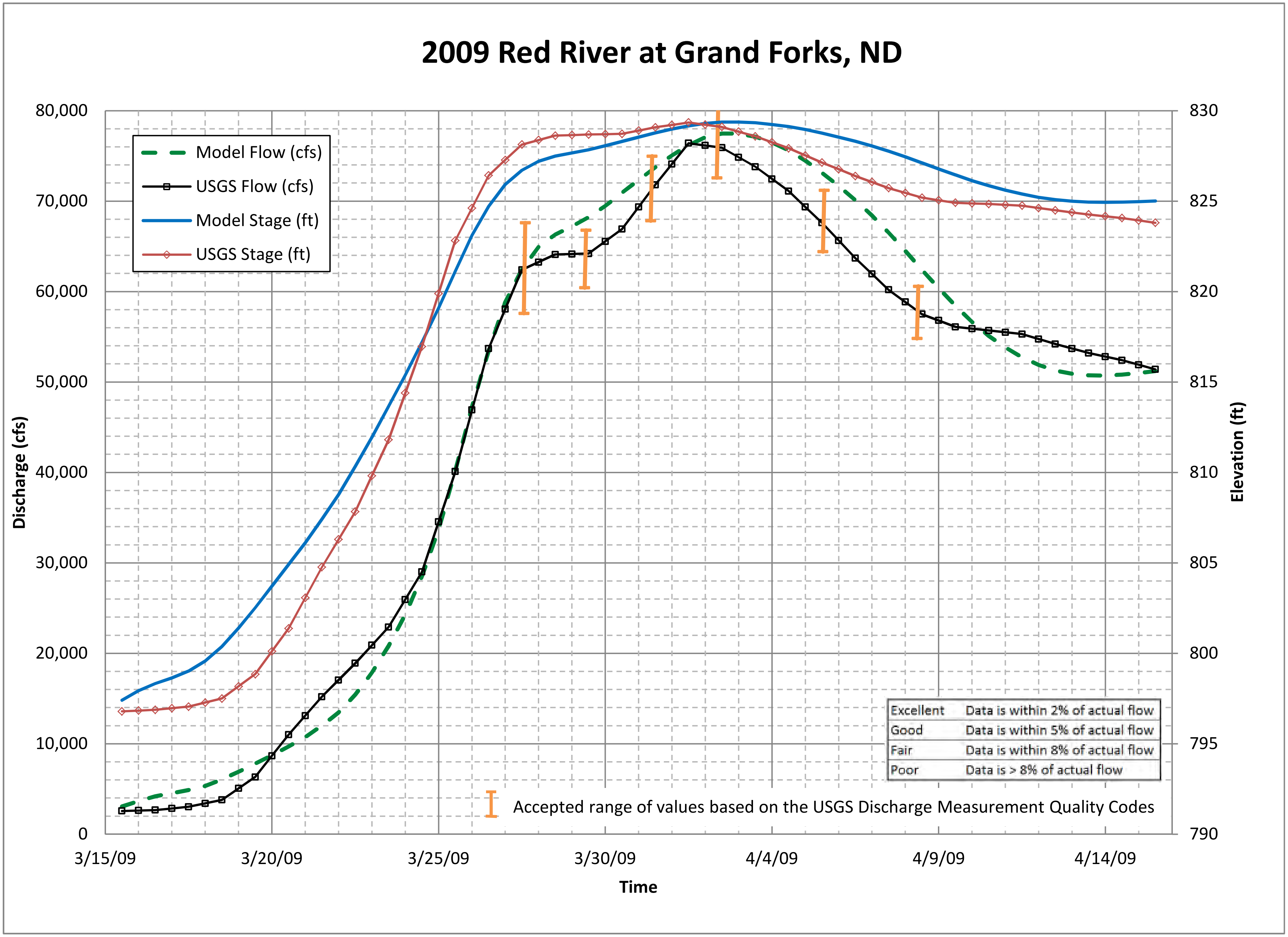

2009 Red River at Hickson, ND
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Discharge (cfs)
Elevation (ft)
Time
30,000
25,000
20,000
15,000
10,000
5,000
0
935
925
915
905
895
885
3/15/09
3/20/09
3/25/09
3/30/09
4/4/09
4/9/09
4/14/09
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Accepted range of values based on the USGS Discharge Measurement Quality Codes

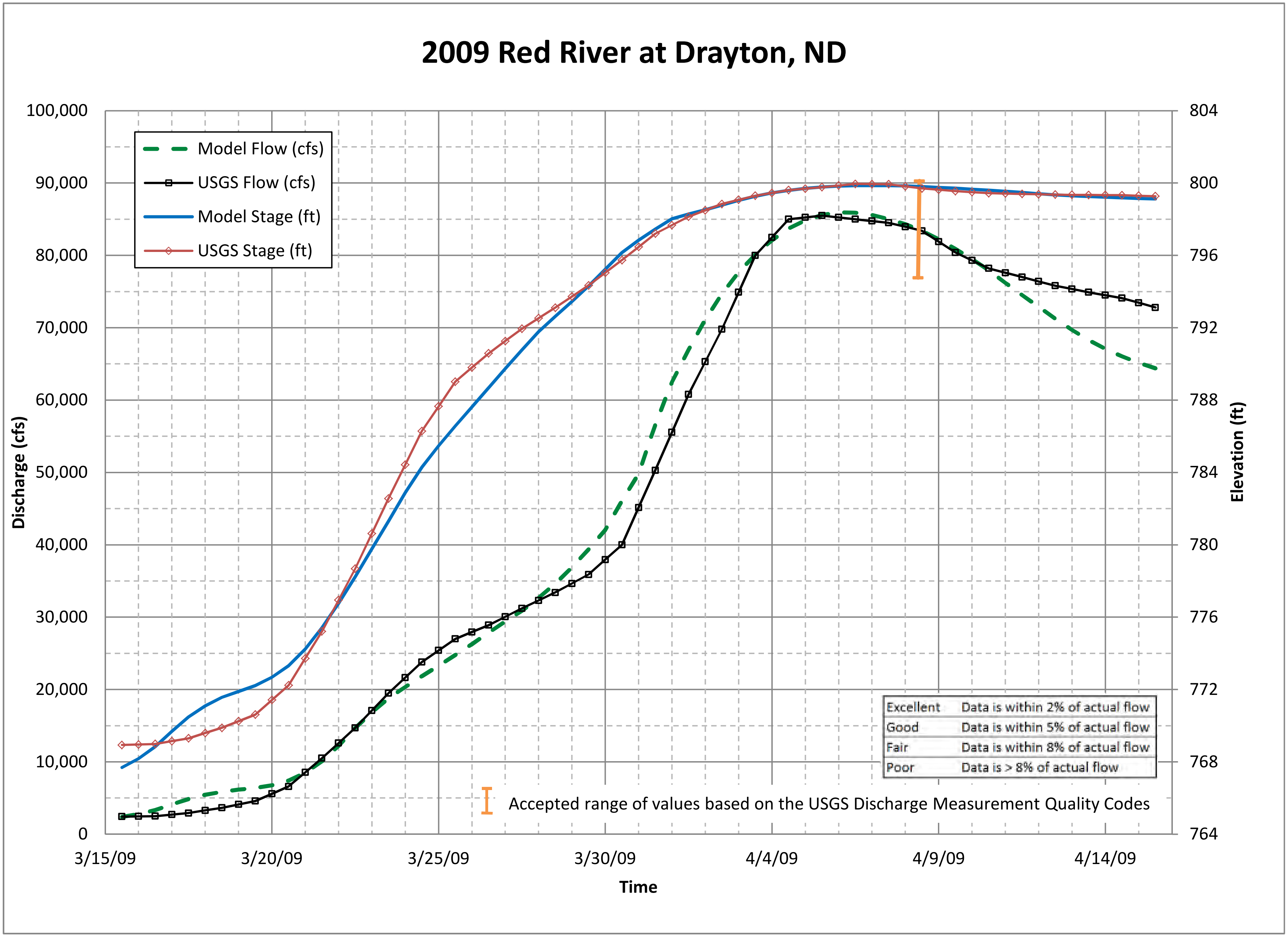

2009 Red River at Fargo, ND
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Discharge (cfs)
Elevation (ft)
Time
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
910
905
900
895
890
885
880
875
3/15/09
3/20/09
3/25/09
3/30/09
4/4/09
4/9/09
4/14/09
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Accepted range of values based on the USGS Discharge Measurement Quality Codes



2009 Red River at Halstad, MN
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Discharge (cfs)
Elevation (ft)
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
870
865
860
855
850
845
840
835
3/15/09
3/20/09
3/25/09
3/30/09
4/4/09
4/9/09
4/14/09
Time
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Accepted range of values based on the USGS Discharge Measurement Quality Codes


2009 Red River at Thompson, ND
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Discharge (cfs)
Elevation (ft)
Time
65,000
60,000
55,000
50,000
45,000
40,000
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
852
848
844
840
836
832
828
824
820
816
812
808
804
800
3/15/09
3/20/09
3/25/09
3/30/09
4/4/09
4/9/09
4/14/09
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Accepted range of values based on the USGS Discharge Measurement Quality Codes

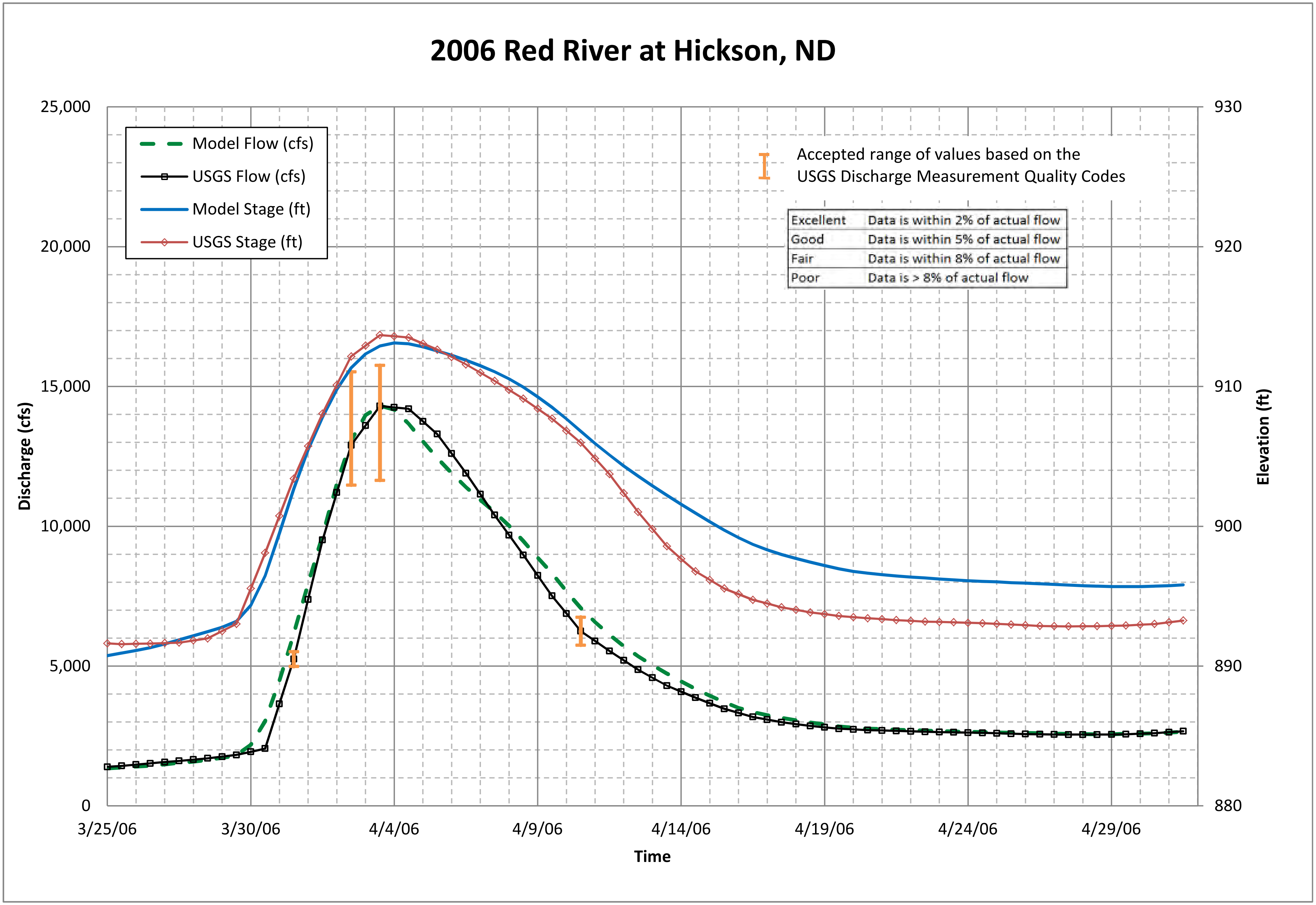

2009 Red River at Grand Forks, ND
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Discharge (cfs)
Elevation (ft)
Time
0
10,000
20,000
30,000
40,000
50,000
60,000
70,000
80,000
790
795
800
805
810
815
820
825
830
3/15/09
3/20/09
3/25/09
3/30/09
4/4/09
4/9/09
4/14/09
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Accepted range of values based on the USGS Discharge Measurement Quality Codes

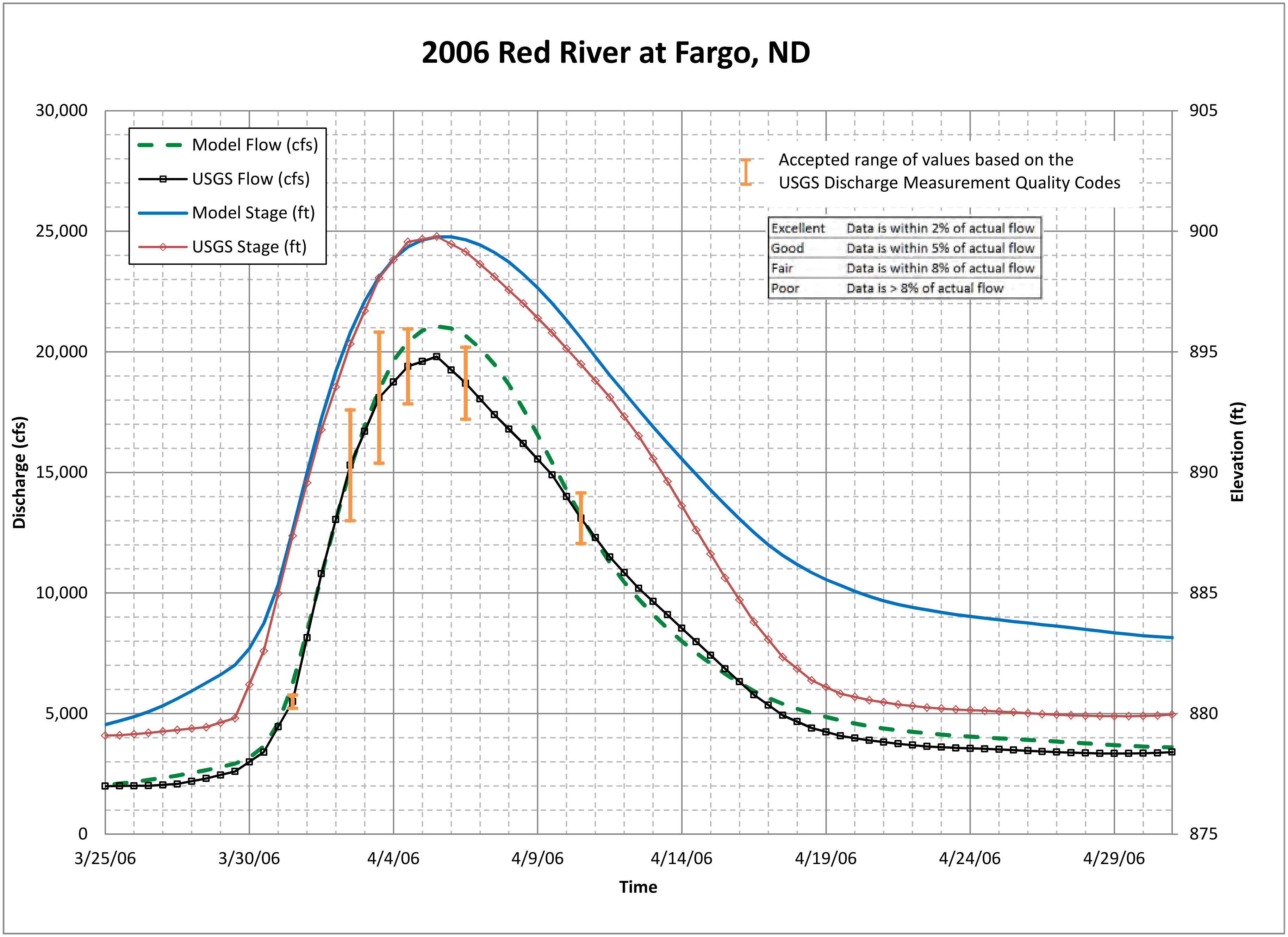

2009 Red River at Drayton, ND
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Discharge (cfs)
Elevation (ft)
Time
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
804
800
796
792
788
784
780
776
772
768
764
3/15/09
3/20/09
3/25/09
3/30/09
4/4/09
4/9/09
4/14/09
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Accepted range of values based on the USGS Discharge Measurement Quality Codes

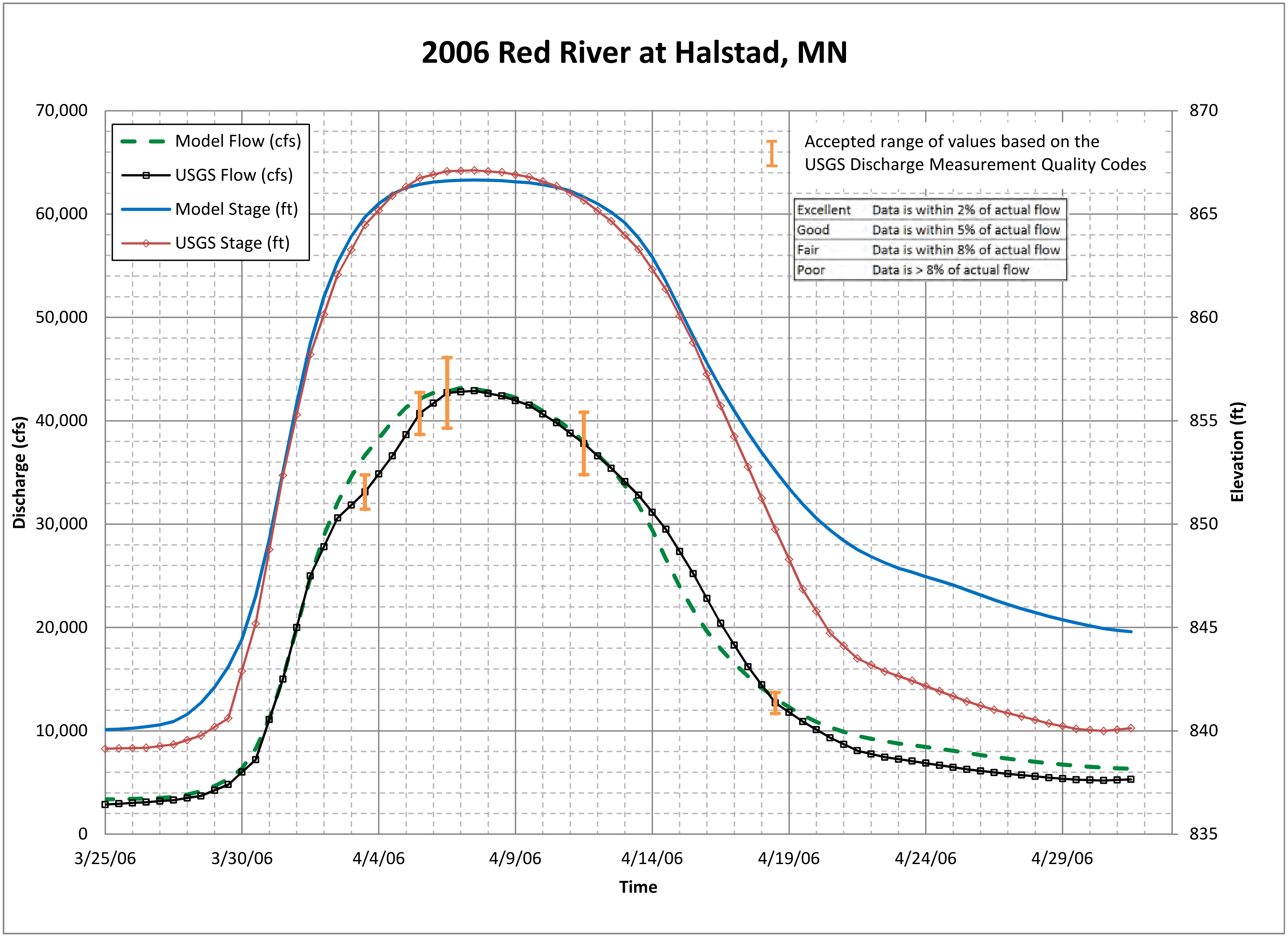
Appendix B – Hydraulics
Existing Condition

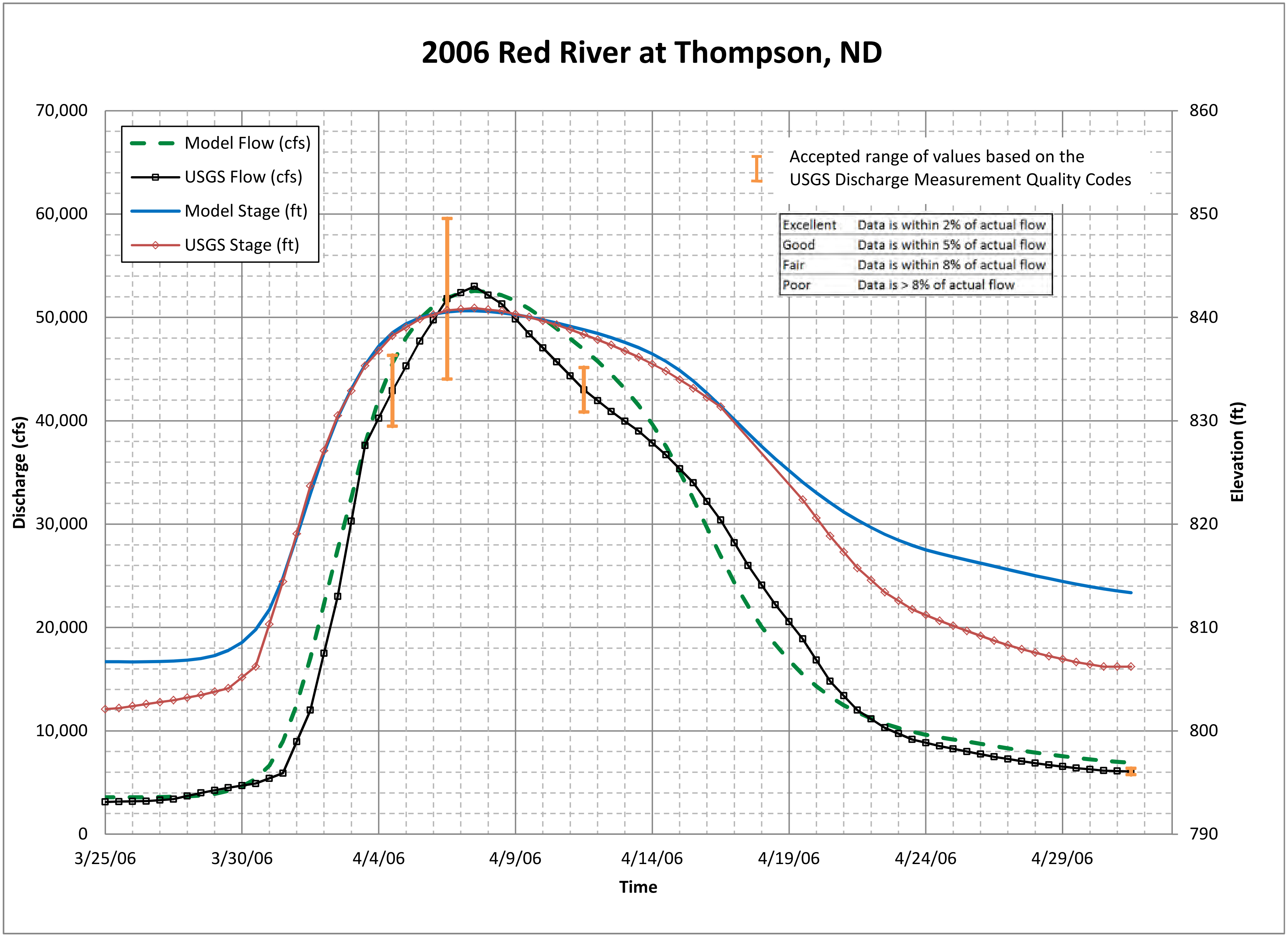
Exhibit B

2006 Flood Verification Discharge Hydrographs


2006 Red River at Hickson, ND
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Accepted range of values based on the
USGS Discharge Measurement Quality Codes
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Discharge (cfs)
Elevation (ft)
Time
25,000
20,000
15,000
10,000
5,000
0
930
920
910
900
890
880
3/25/06
3/30/06
4/4/06
4/9/06
4/14/06
4/19/06
4/24/06
4/29/06


2006 Red River at Fargo, ND
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Accepted range of values based on the USGS Discharge Measurement Quality Codes
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Discharge (cfs)
Elevation (ft)
Time
30,000
25,000
20,000
15,000
10,000
5,000
0
905
900
895
890
885
880
875
3/25/06
3/30/06
4/4/06
4/9/06
4/14/06
4/19/06
4/24/06
4/29/06


2006 Red River at Halstad, MN
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Accepted range of values based on the
USGS Discharge Measurement Quality Codes
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Discharge (cfs)
Elevation (ft)
Time
0
10,000
20,000
30,000
40,000
50,000
60,000
70,000
835
840
845
850
855
860
865
870
3/25/06
3/30/06
4/4/06
4/9/06
4/14/06
4/19/06
4/24/06
4/29/06


2006 Red River at Thompson, ND
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Accepted range of values based on the
USGS Discharge Measurement Quality Codes
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Discharge (cfs)
Elevation (ft)
Time
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
860
850
840
830
820
810
800
790
3/25/06
3/30/06
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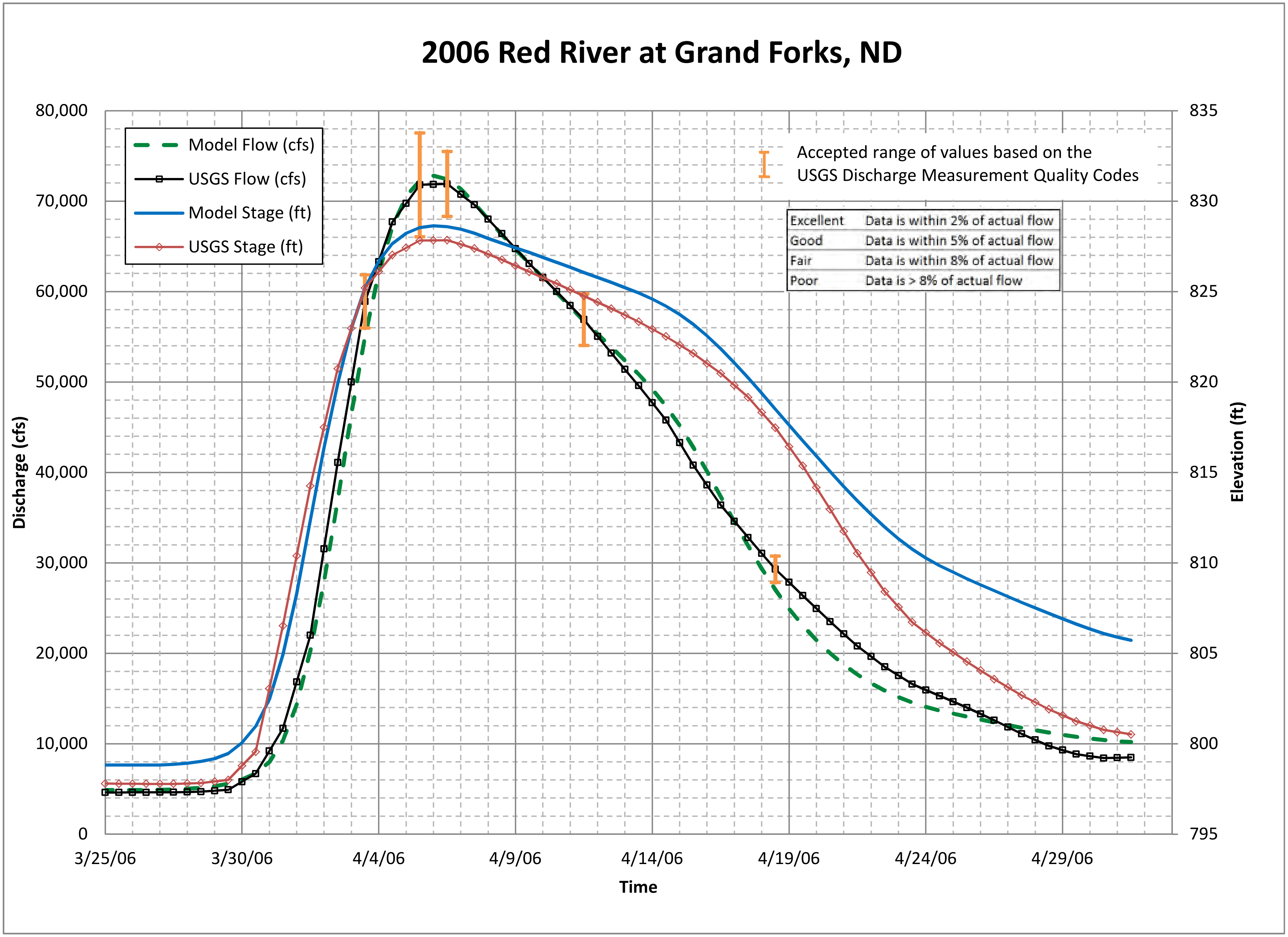
4/9/06
4/14/06
4/19/06
4/24/06
4/29/06


2006 Red River at Grand Forks, ND
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Accepted range of values based on the
USGS Discharge Measurement Quality Codes
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Discharge (cfs)
Elevation (ft)
Time
0
10,000
20,000
30,000
40,000
50,000
60,000
70,000
80,000
795
800
805
810
815
820
825
830
835
3/25/06
3/30/06
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4/9/06
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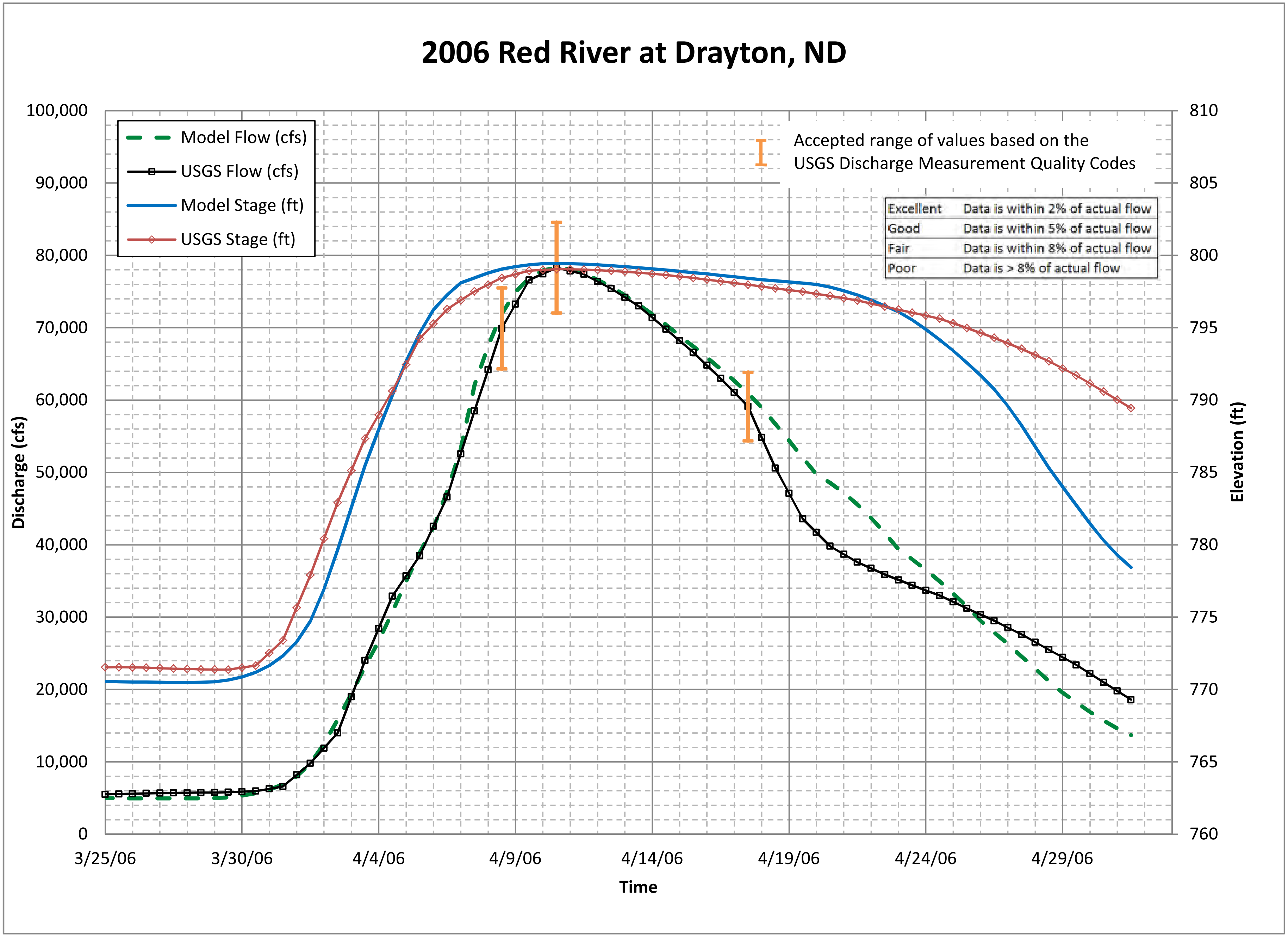
4/19/06
4/24/06
4/29/06

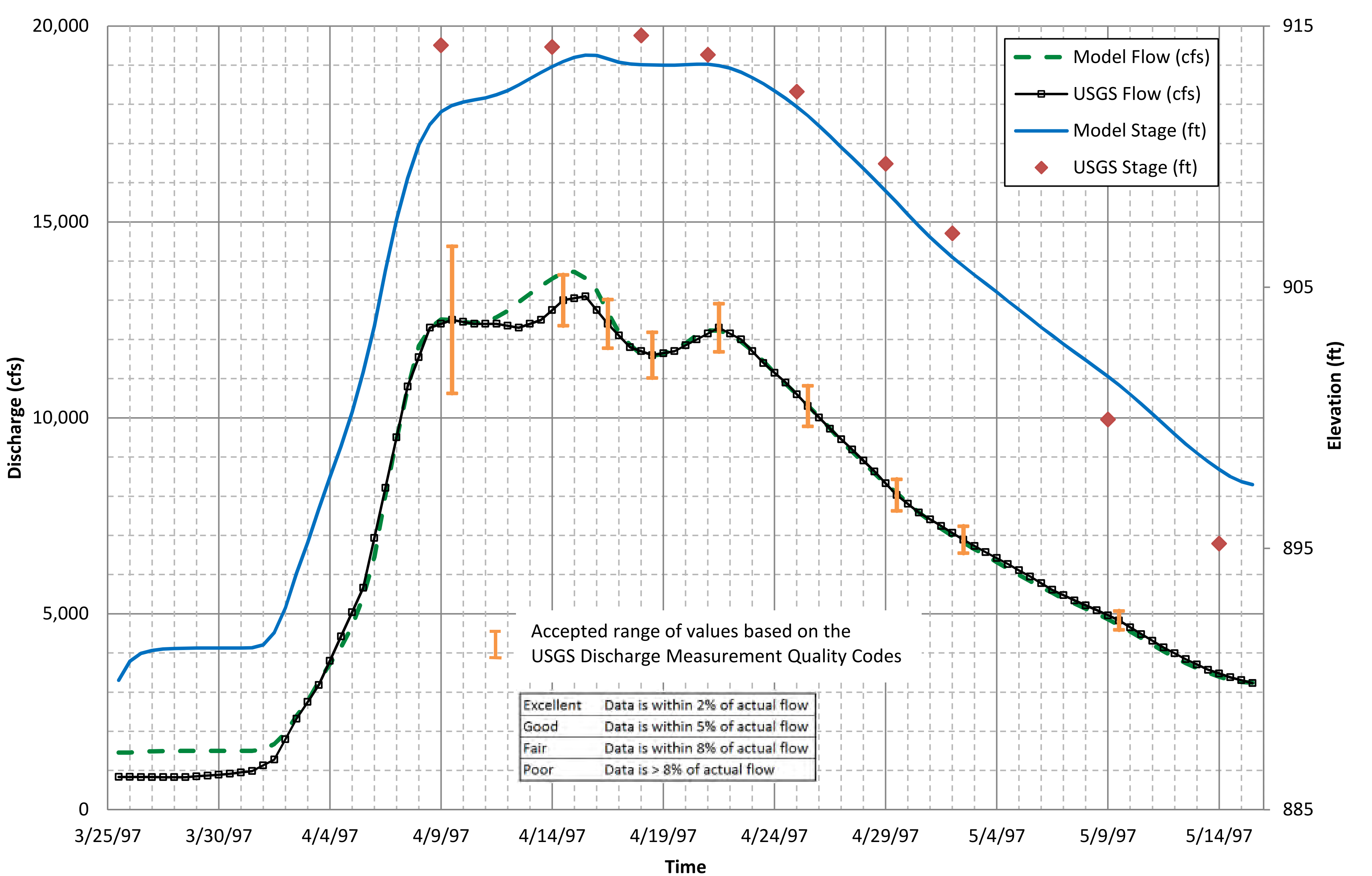

2006 Red River at Drayton, ND
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Accepted range of values based on the
USGS Discharge Measurement Quality Codes
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Discharge (cfs)
Elevation (ft)
Time
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
810
805
800
795
790
785
780
775
770
765
760
3/25/06
3/30/06
4/4/06
4/9/06
4/14/06
4/19/06
4/24/06
4/29/06

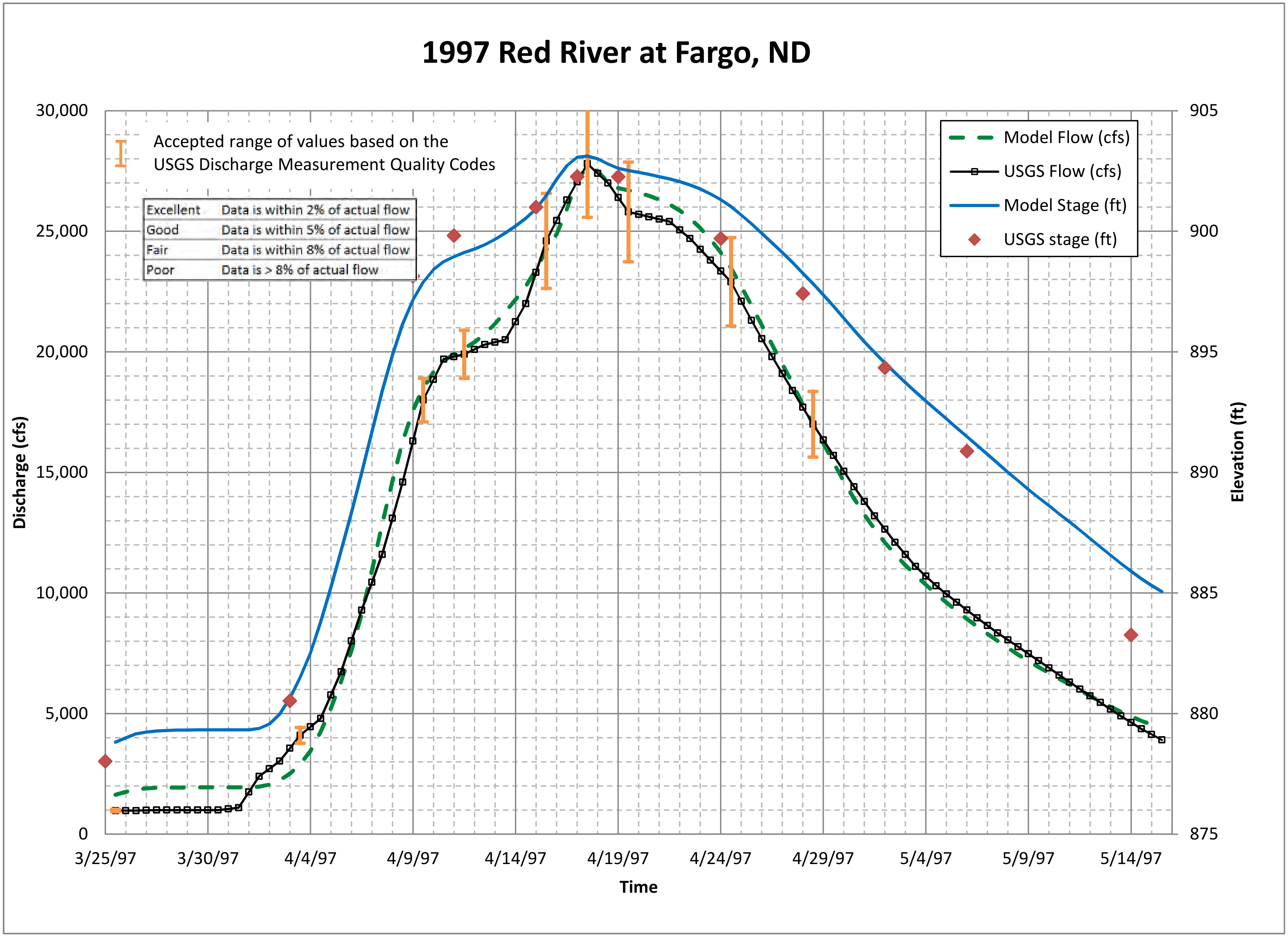
Appendix B – Hydraulics
Existing Condition

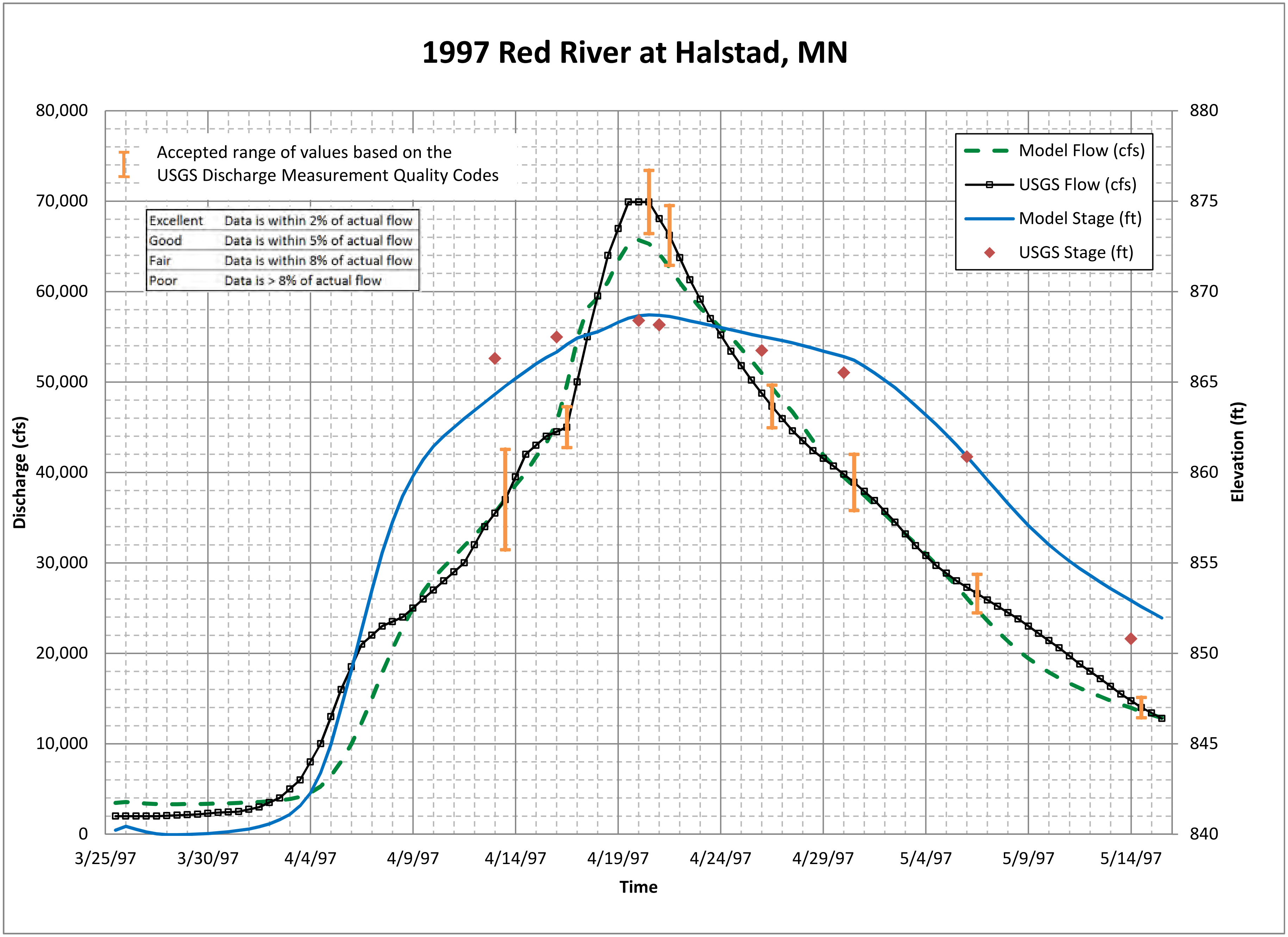
Exhibit C

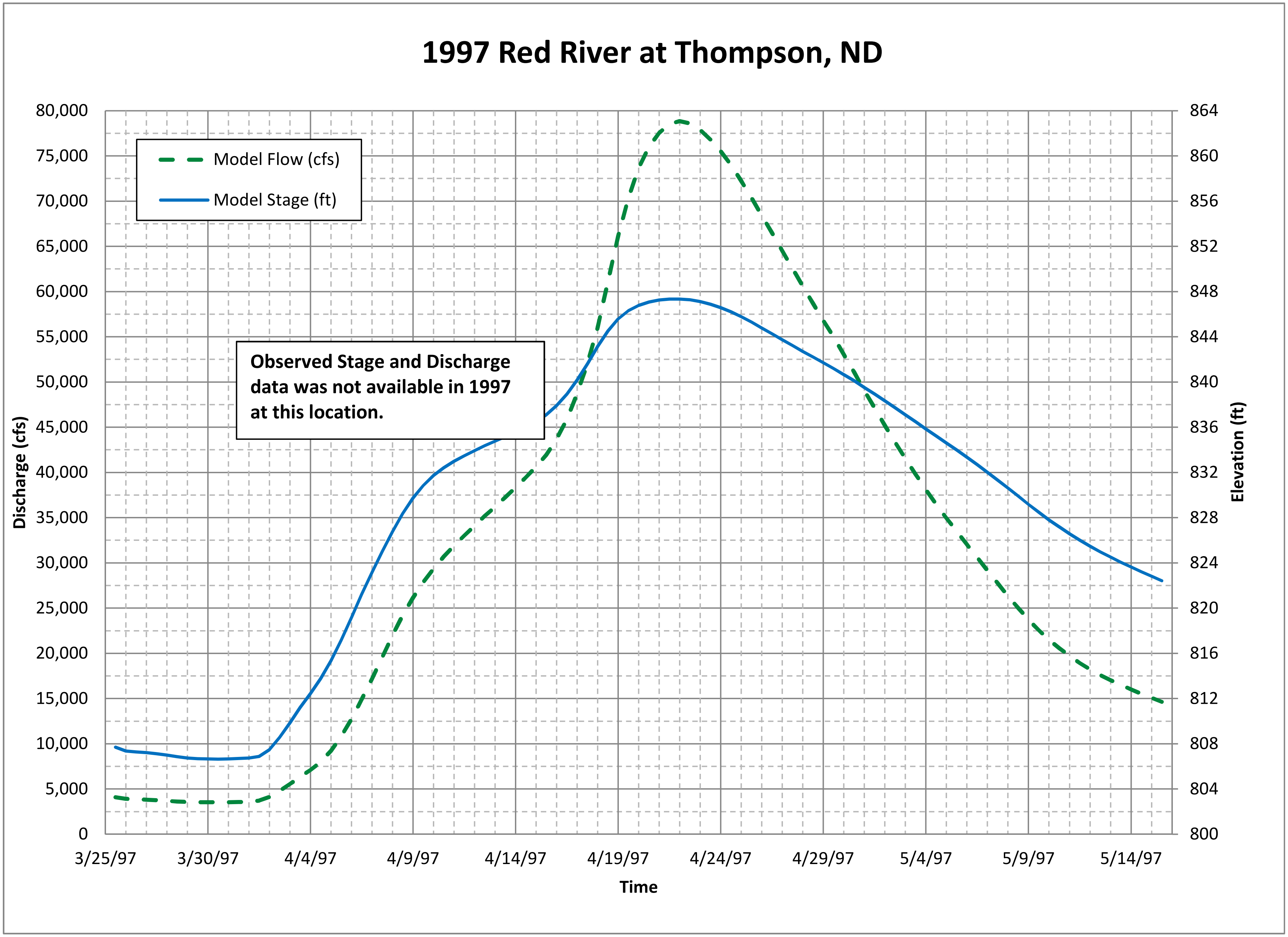
1997 Flood Verification Discharge Hydrographs

1997 Red River at Hickson, ND

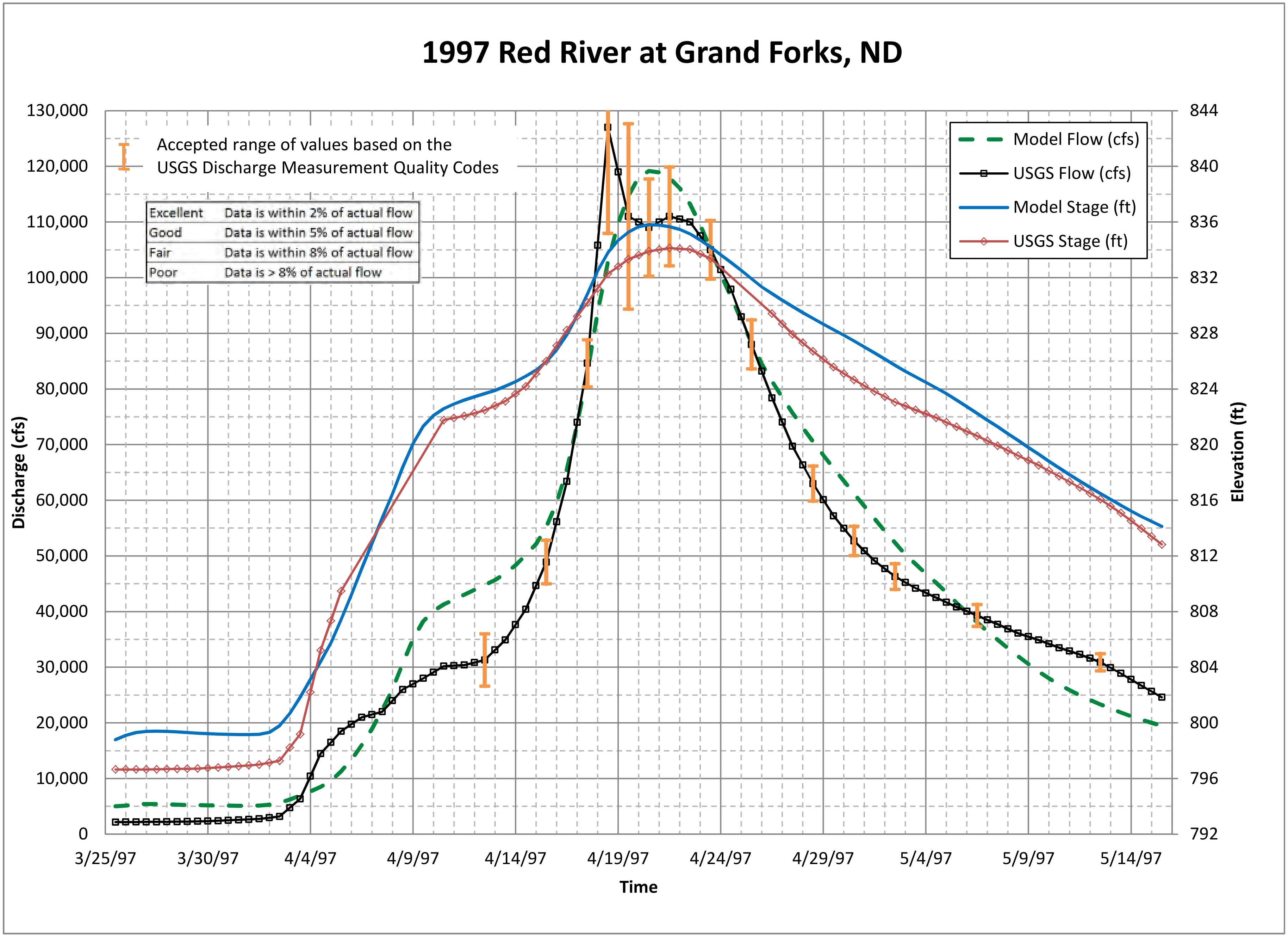



1997 Red River at Fargo, ND
Accepted range of values based on the USGS Discharge Measurement Quality Codes
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS stage (ft)
Discharge (cfs)
Elevation (ft)
Time
30,000
25,000
20,000
15,000
10,000
5,000
0
905
900
895
890
885
880
875
3/25/97
3/30/97
4/4/97
4/9/97
4/14/97
4/19/97
4/24/97
4/29/97
5/4/97
5/9/97
5/14/97

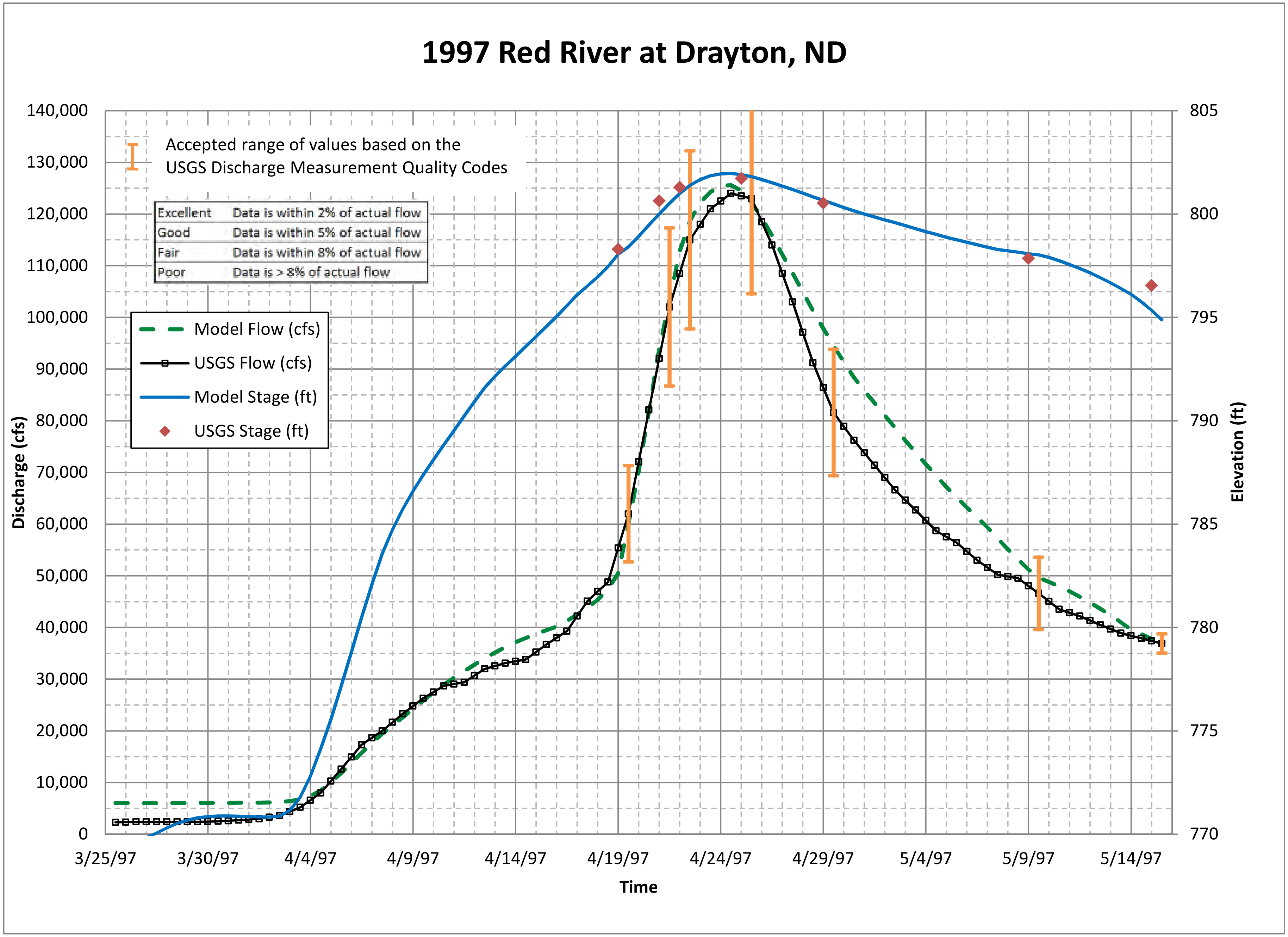

1997 Red River at Halstad, MN
Accepted range of values based on the USGS Discharge Measurement Quality Codes
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Discharge (cfs)
Elevation (ft)
Time
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
880
875
870
865
860
855
850
845
840
3/25/97
3/30/97
4/4/97
4/9/97
4/14/97
4/19/97
4/24/97
4/29/97
5/4/97
5/9/97
5/14/97



1997 Red River at Thompson, ND
Model Flow (cfs)
Model Stage (ft)
Observed Stage and Discharge data was not available in 1997 at this location.
Discharge (cfs)
Elevation (ft)
Time
80,000
75,000
70,000
65,000
60,000
55,000
50,000
45,000
40,000
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
864
860
856
852
848
844
840
836
832
828
824
820
816
812
808
804
800
3/25/97
3/30/97
4/4/97
4/9/97
4/14/97
4/19/97
4/24/97
4/29/97
5/4/97
5/9/97
5/14/97



1997 Red River at Grand Forks, ND
Accepted range of values based on the
USGS Discharge Measurement Quality Codes
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Discharge (cfs)
Elevation (ft)
Time
130,000
120,000
110,000
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
844
840
836
832
828
824
820
816
812
808
804
800
796
792
3/25/97
3/30/97
4/4/97
4/9/97
4/14/97
4/19/97
4/24/97
4/29/97
5/4/97
5/9/97
5/14/97

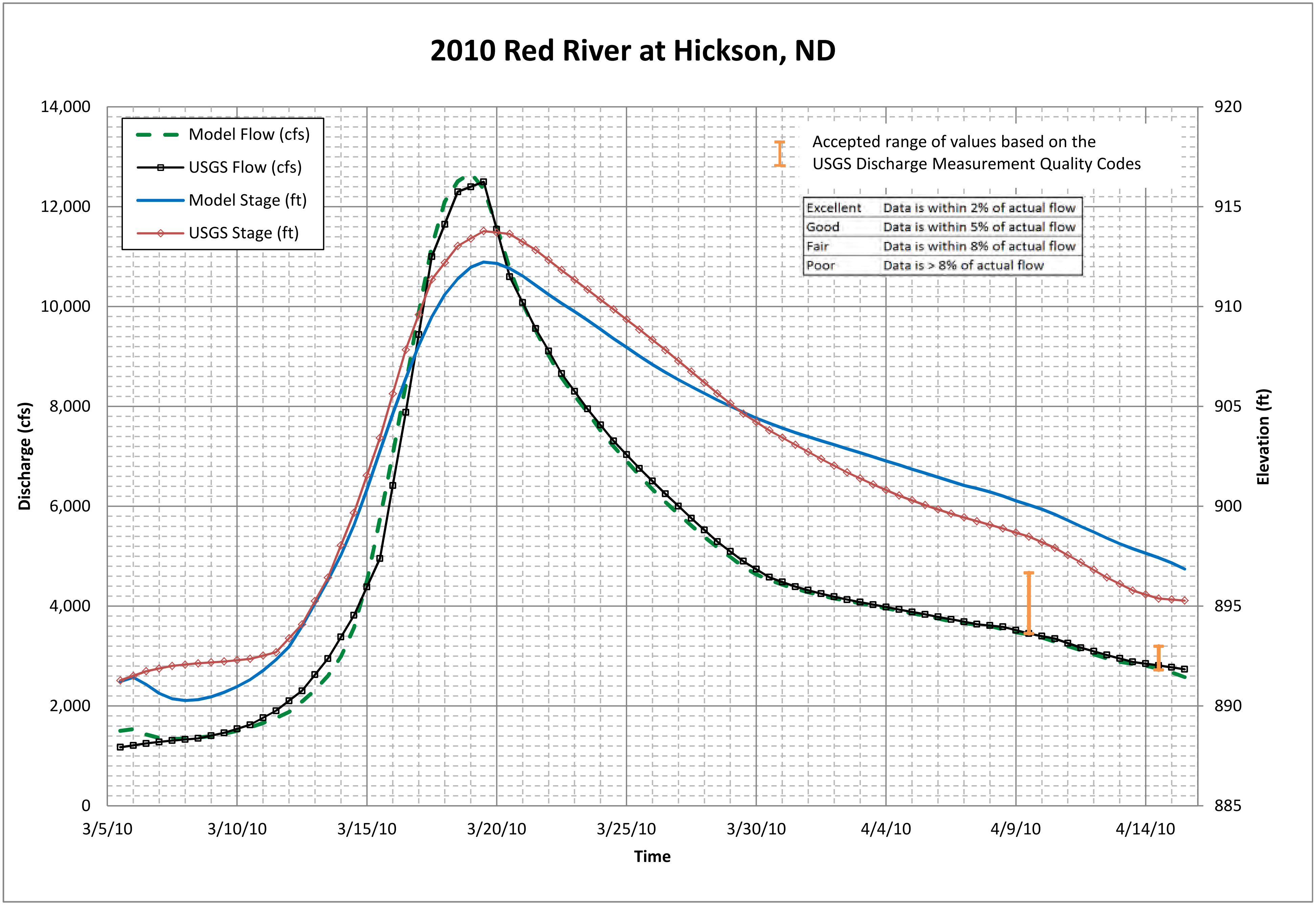


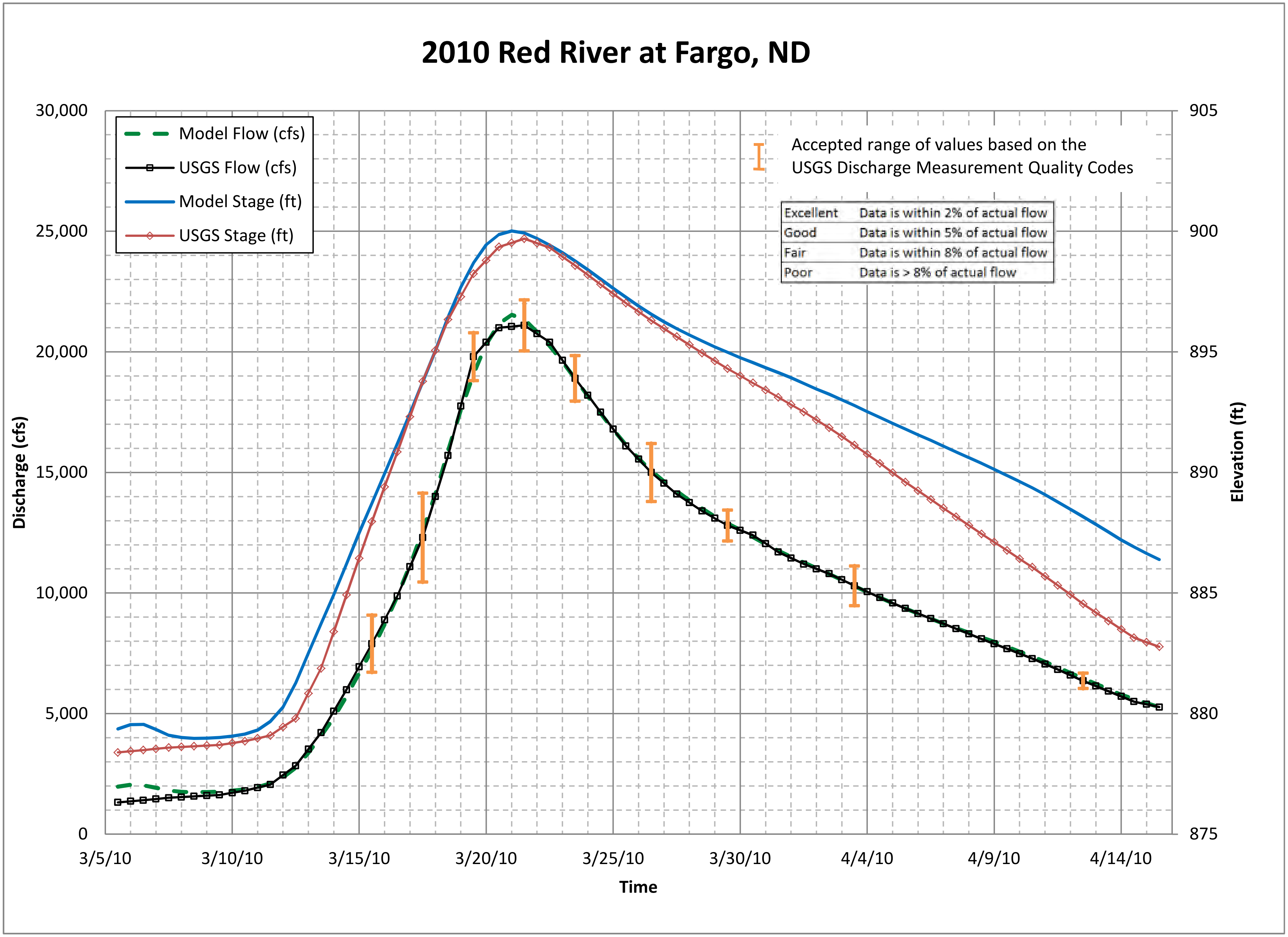
1997 Red River at Drayton, ND
Accepted range of values based on the
USGS Discharge Measurement Quality Codes
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Discharge (cfs)
Elevation (ft)
Time
140,000
130,000
120,000
110,000
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
805
800
795
790
785
780
775
770
3/25/97
3/30/97
4/4/97
4/9/97
4/14/97
4/19/97
4/24/97
4/29/97
5/4/97
5/9/97
5/14/97

Appendix B – Hydraulics
Existing Condition

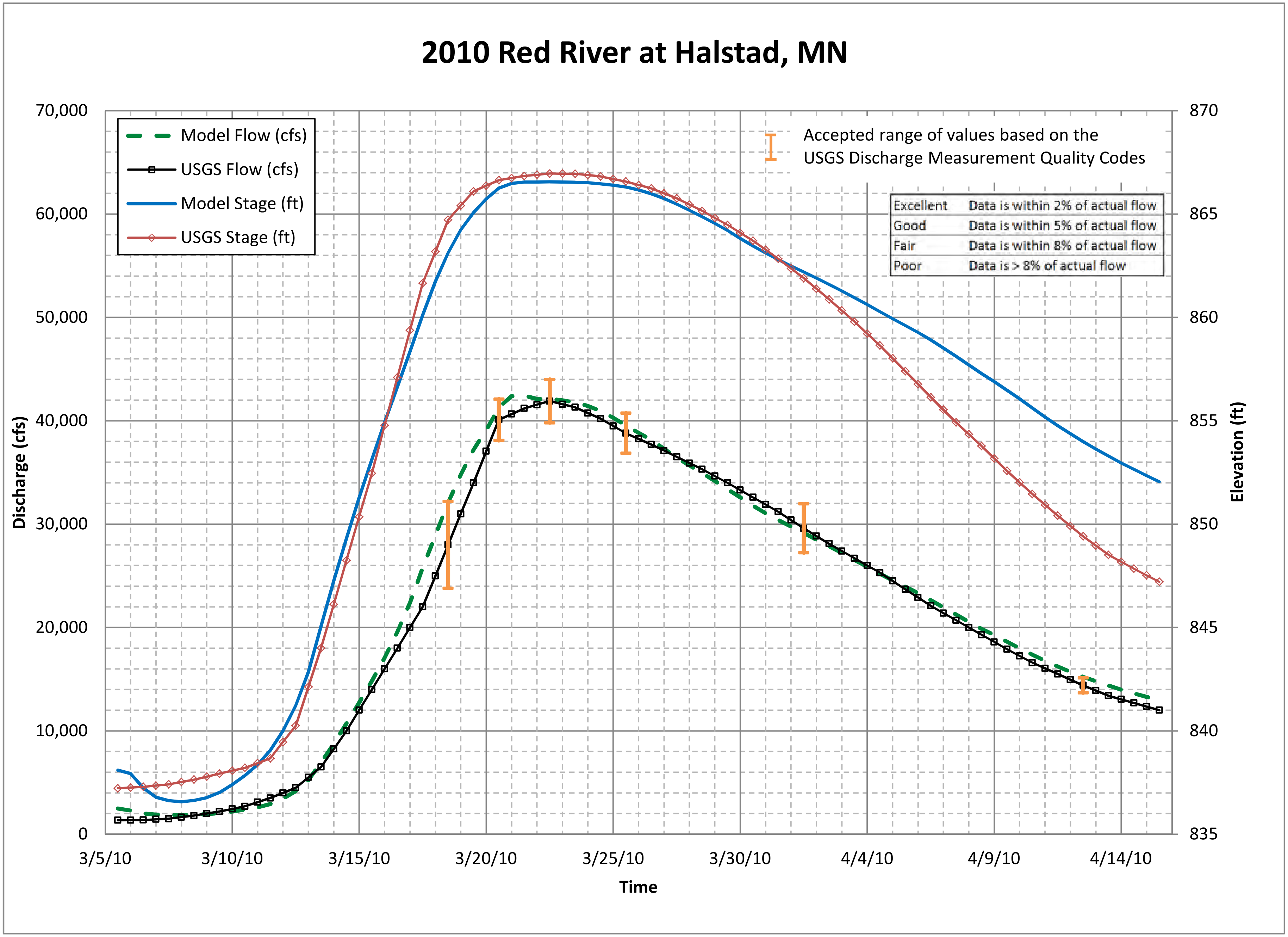
Exhibit D

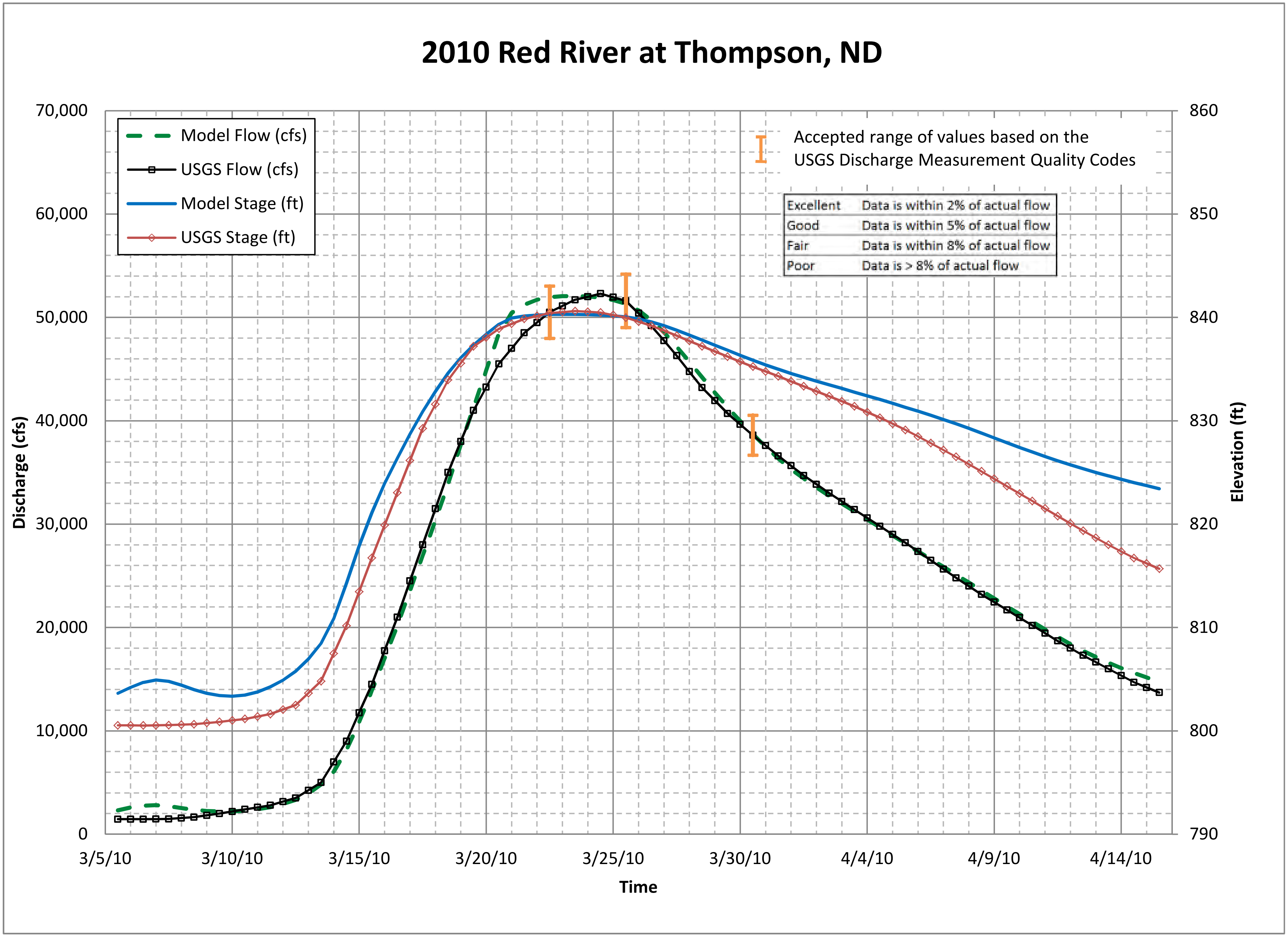
2010 Flood Verification Discharge Hydrographs

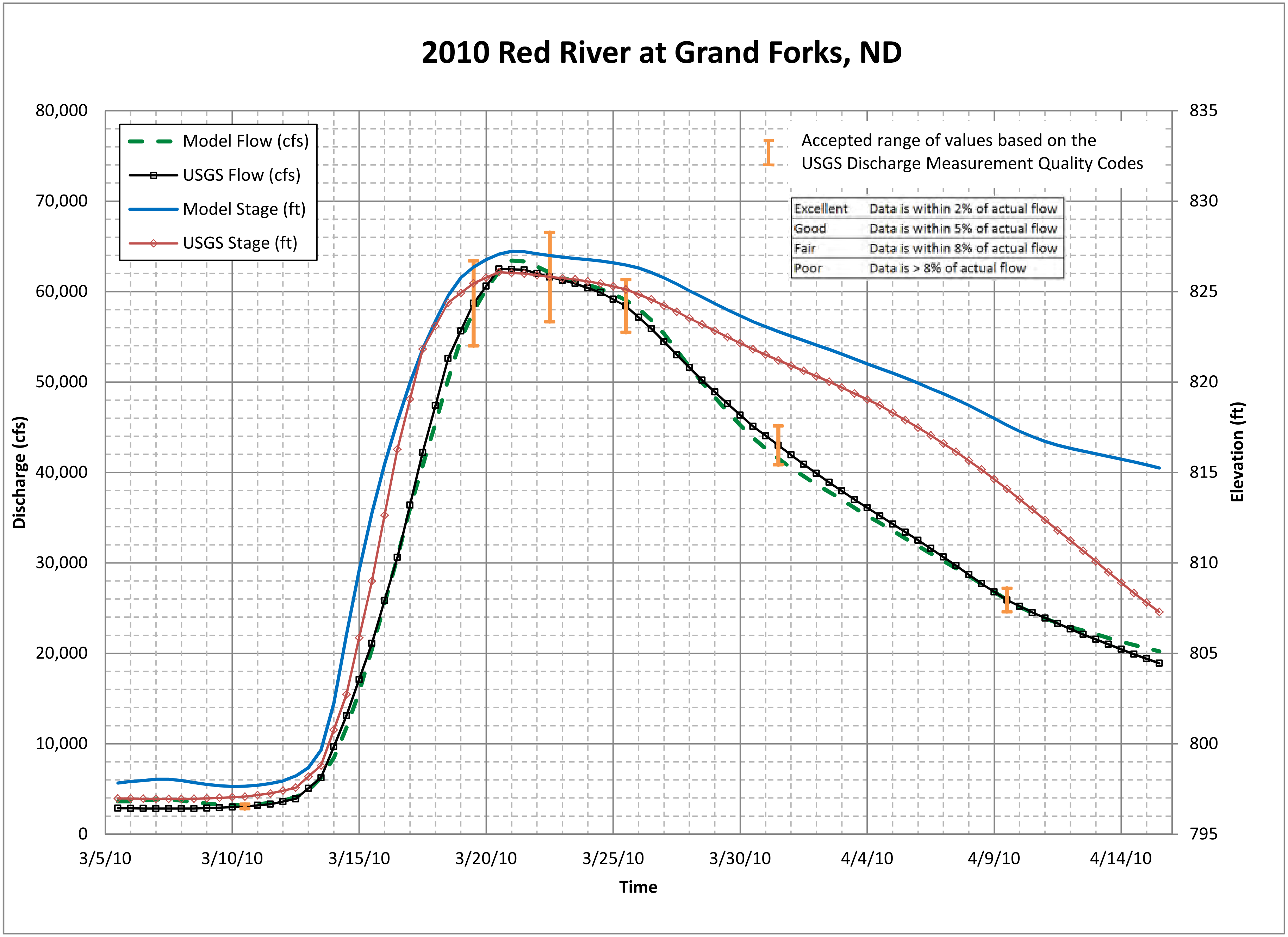

2010 Red River at Hickson, ND
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Accepted range of values based on the USGS Discharge Measurement Quality Codes
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Discharge (cfs)
Elevation (ft)
Time
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
920
915
910
905
900
895
890
885
3/5/10
3/10/10
3/15/10
3/20/10
3/25/10
3/30/10
4/4/10
4/9/10
4/14/10

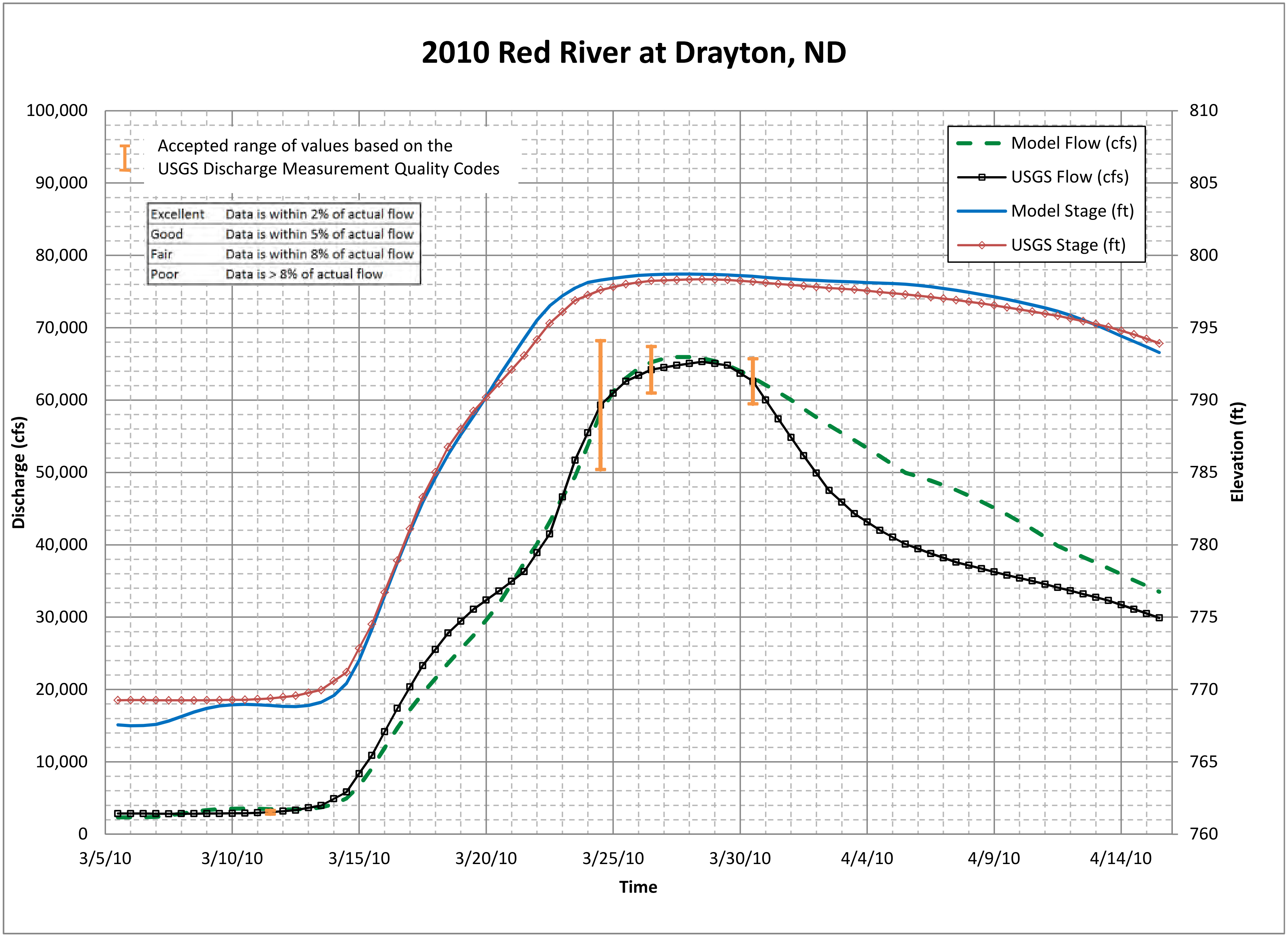


2010 Red River at Fargo, ND
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Accepted range of values based on the
USGS Discharge Measurement Quality Codes
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Discharge (cfs)
Elevation (ft)
Time
30,000
25,000
20,000
15,000
10,000
5,000
0
905
900
895
890
885
880
875
3/5/10
3/10/10
3/15/10
3/20/10
3/25/10
3/30/10
4/4/10
4/9/10
4/14/10


2010 Red River at Halstad, MN
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Accepted range of values based on the
USGS Discharge Measurement Quality Codes
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Discharge (cfs)
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
Elevation (ft)
870
865
860
855
850
845
840
835
3/5/10
3/10/10
3/15/10
3/20/10
3/25/10
3/30/10
4/4/10
4/9/10
4/14/10
Time


2010 Red River at Thompson, ND
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Accepted range of values based on the
USGS Discharge Measurement Quality Codes
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Discharge (cfs)
Elevation (ft)
Time
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
860
850
840
830
820
810
800
790
3/5/10
3/10/10
3/15/10
3/20/10
3/25/10
3/30/10
4/4/10
4/9/10
4/14/10

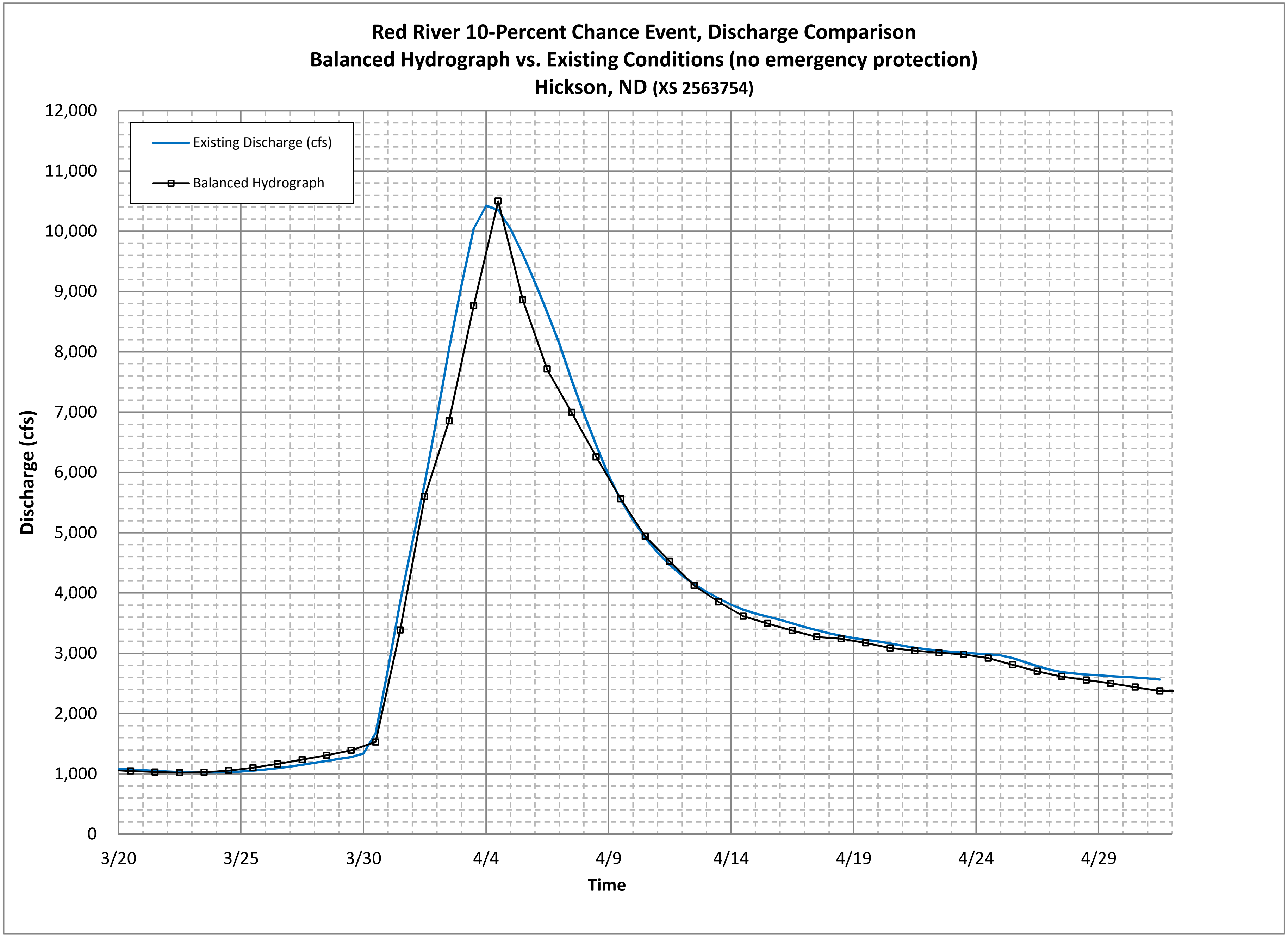

2010 Red River at Grand Forks, ND
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Accepted range of values based on the USGS Discharge Measurement Quality Codes
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Discharge (cfs)
Elevation (ft)
Time
0
10,000
20,000
30,000
40,000
50,000
60,000
70,000
80,000
795
800
805
810
815
820
825
830
835
3/5/10
3/10/10
3/15/10
3/20/10
3/25/10
3/30/10
4/4/10
4/9/10
4/14/10



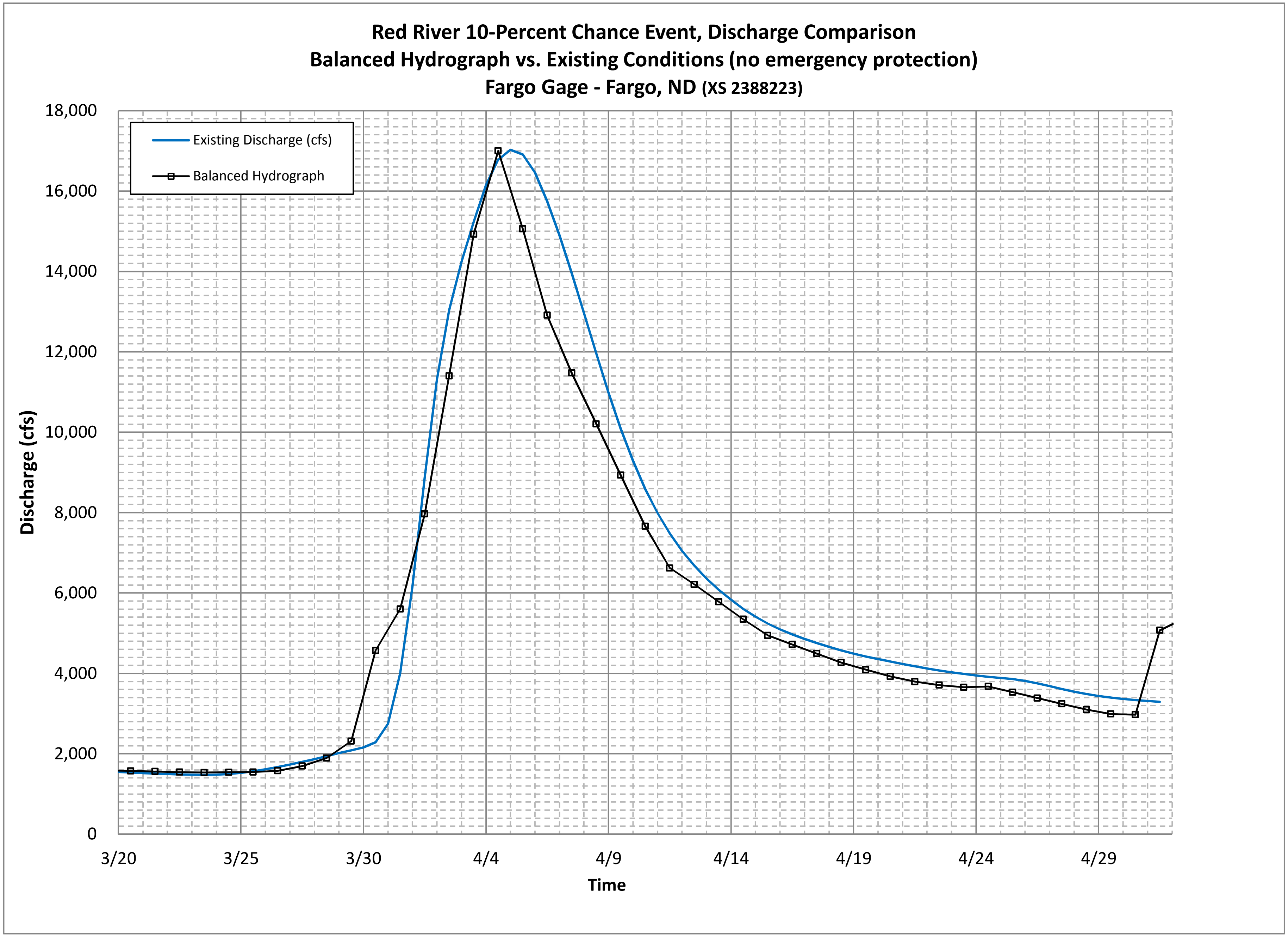
2010 Red River at Drayton, ND
Accepted range of values based on the
USGS Discharge Measurement Quality Codes
Excellent Data is within 2% of actual flow
Good Data is within 5% of actual flow
Fair Data is within 8% of actual flow
Poor Data is > 8% of actual flow
Model Flow (cfs)
USGS Flow (cfs)
Model Stage (ft)
USGS Stage (ft)
Discharge (cfs)
Elevation (ft)
Time
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
810
805
800
795
790
785
780
775
770
765
760
3/5/10
3/10/10
3/15/10
3/20/10
3/25/10
3/30/10
4/4/10
4/9/10
4/14/10

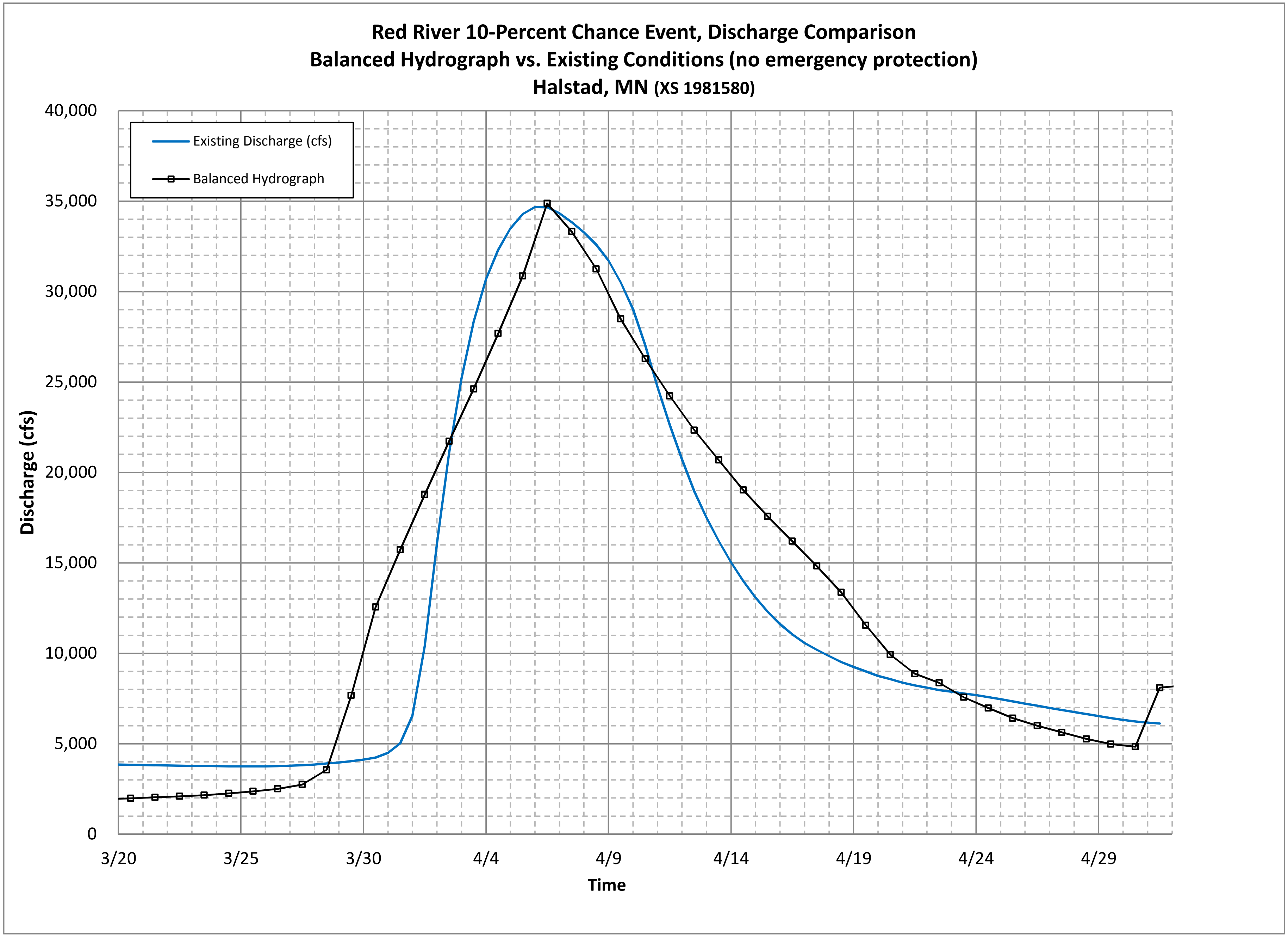
Exhibit E

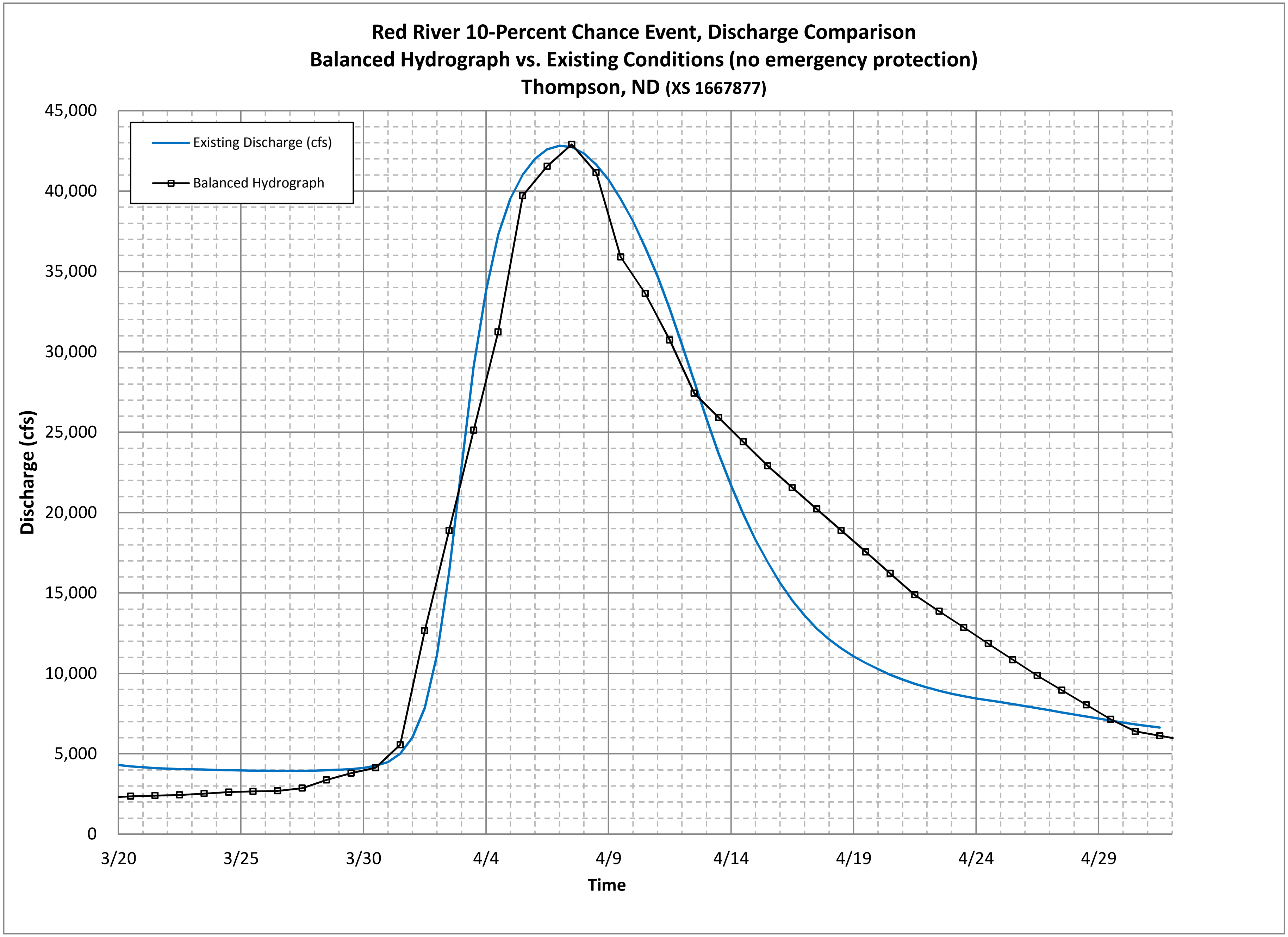
Existing Conditions 10-, 2-, 1-, and 0.2-Percent Chance Hydrographs



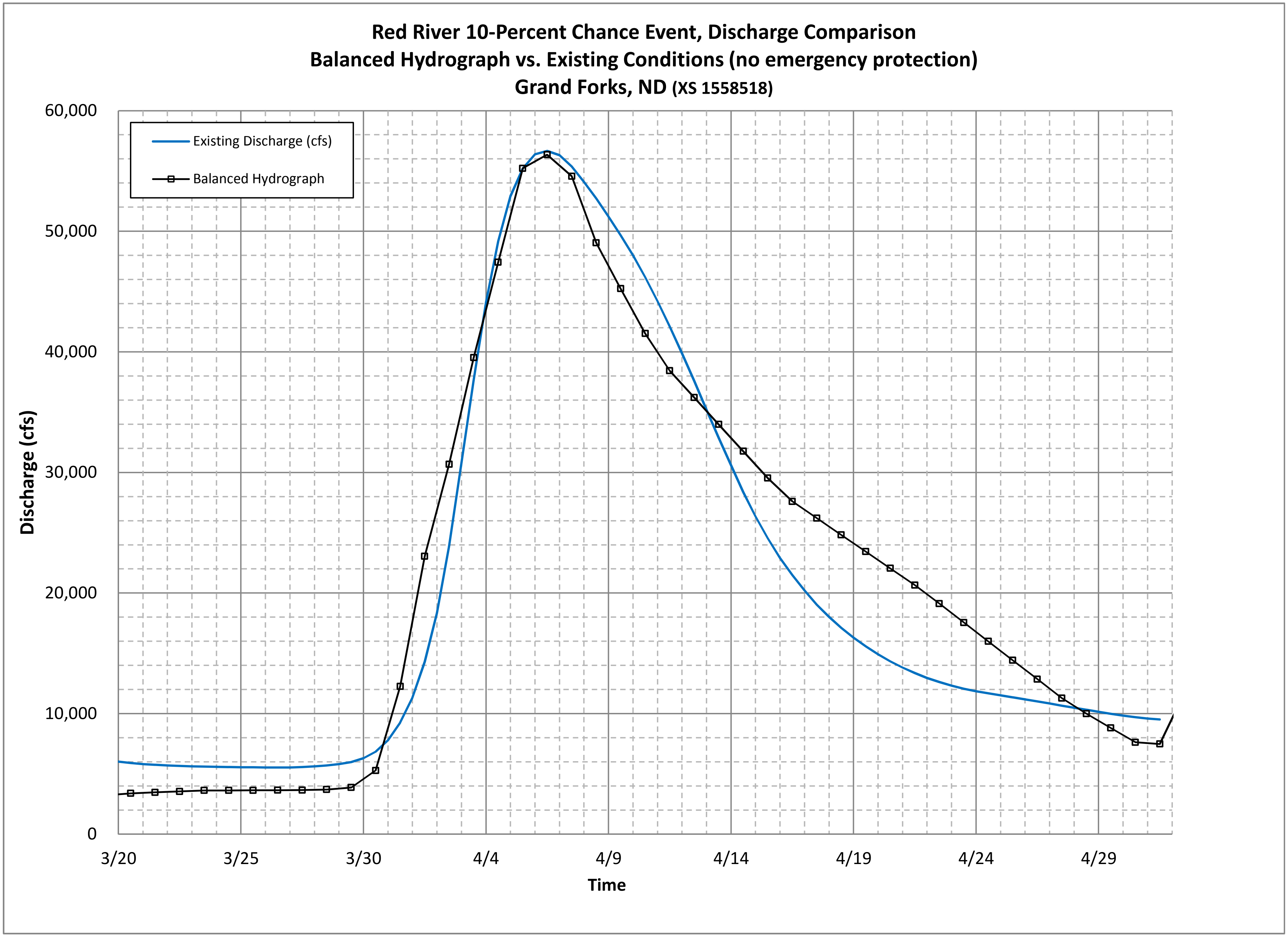
Red River 10-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Hickson, ND (XS 2563754)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
12,000
11,000
10,000
9,000
8,000
7,000
6,000
5,000
4,000
3,000
2,000
1,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time

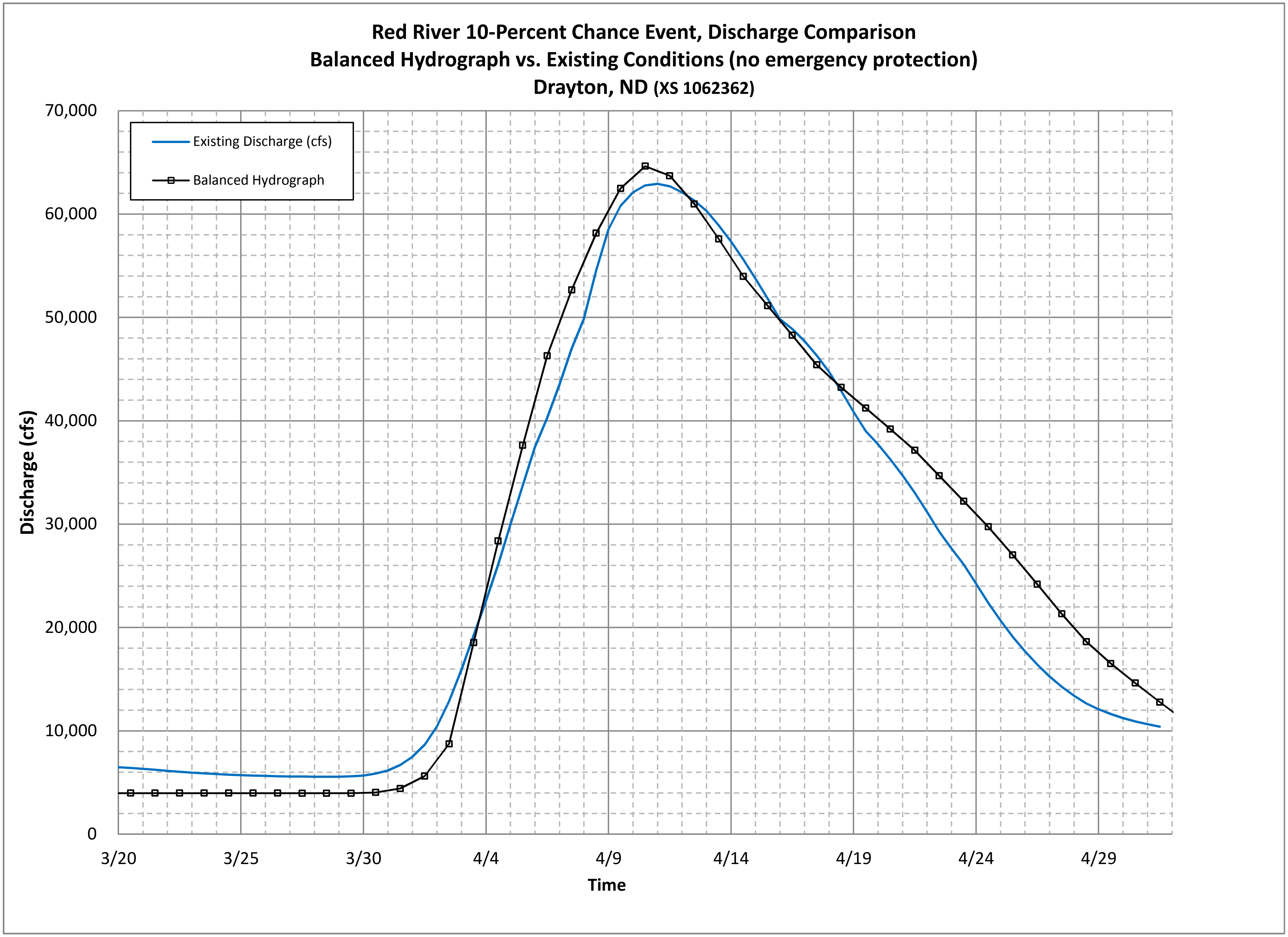

Red River 10-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Fargo Gage - Fargo, ND (XS 2388223)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
18,000
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time

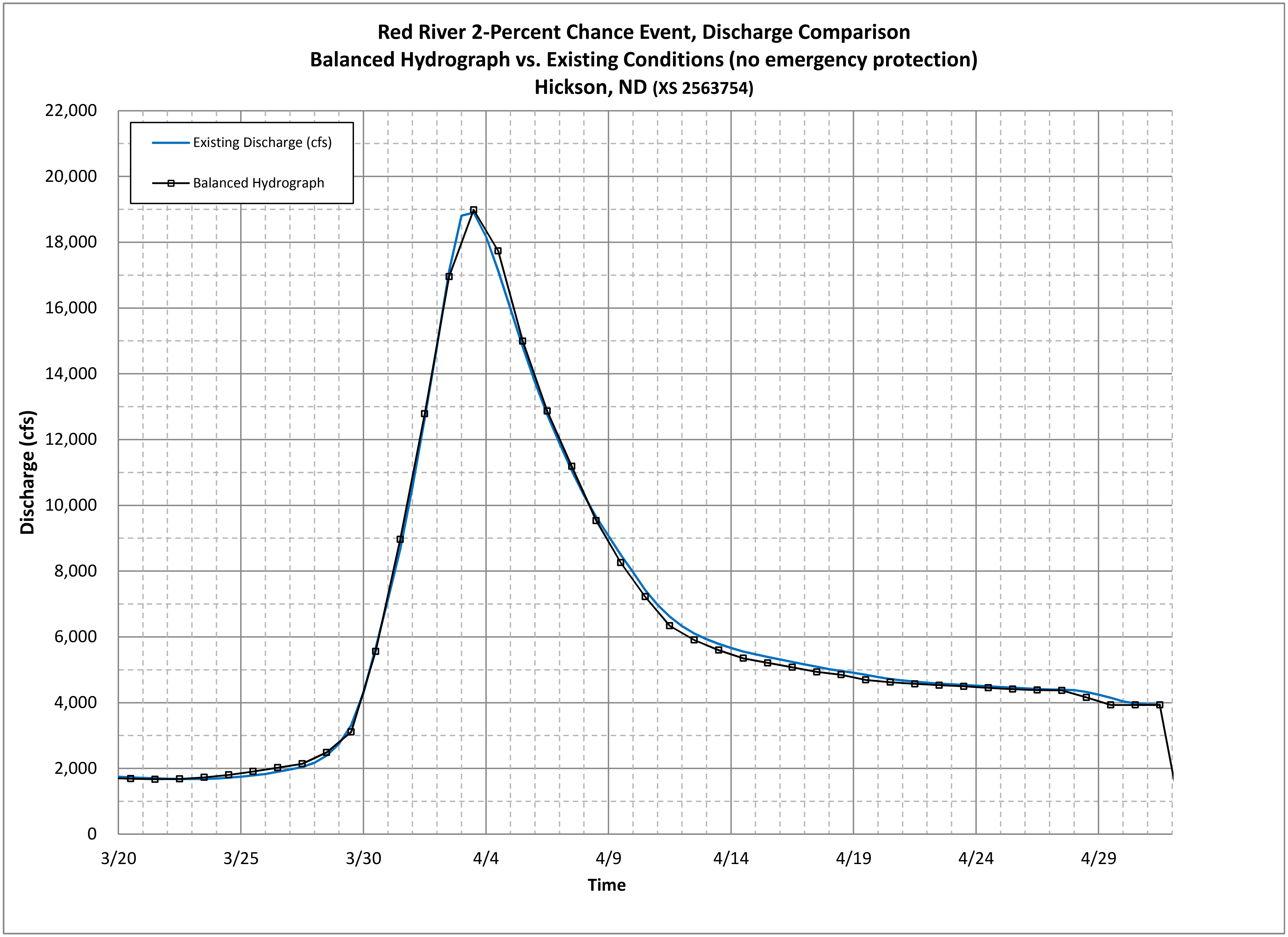

Red River 10-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Halstad, MN (XS 1981580)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
40,000
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time



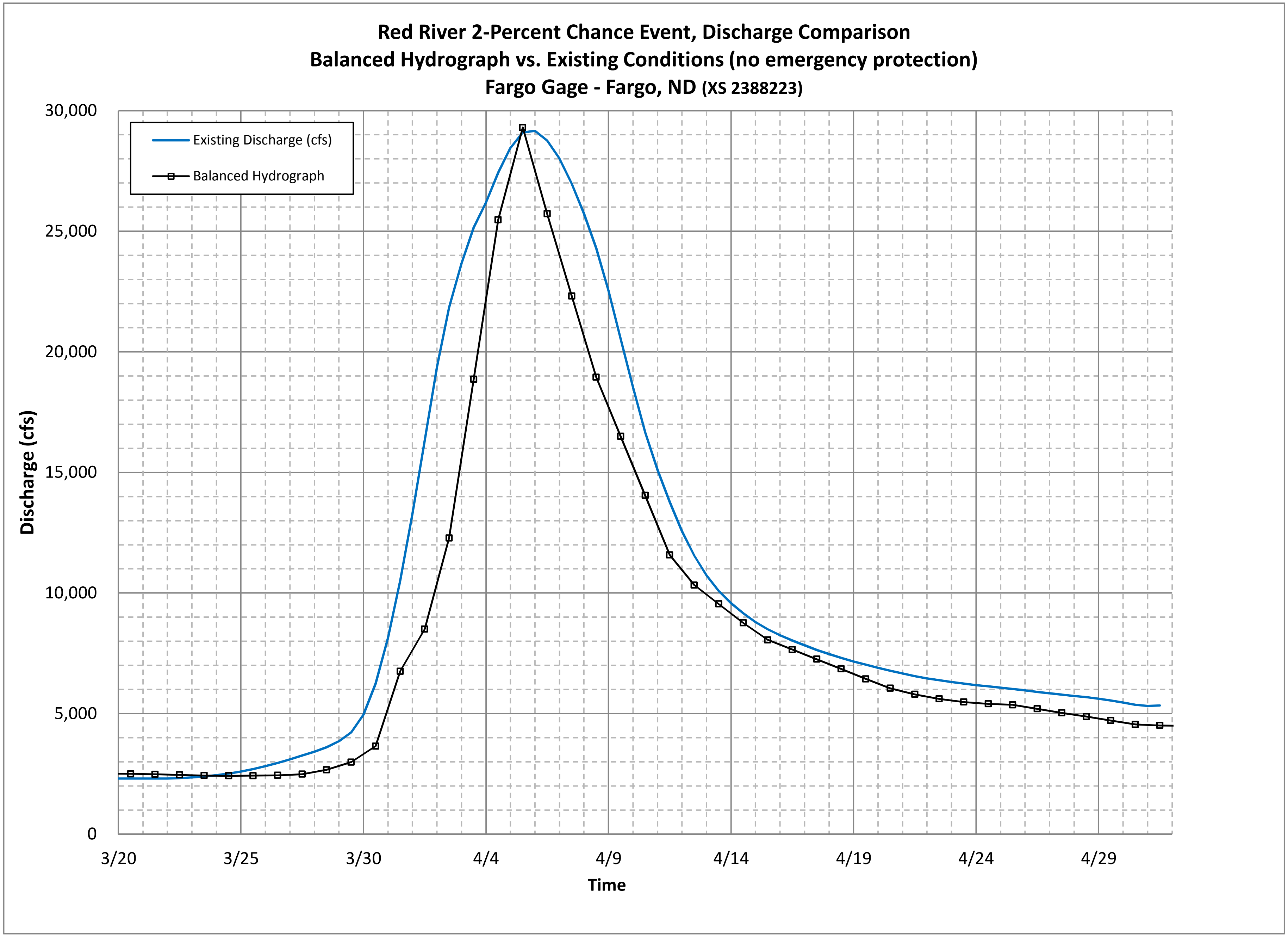
Red River 10-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Thompson, ND (XS 1667877)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
45,000
40,000
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time

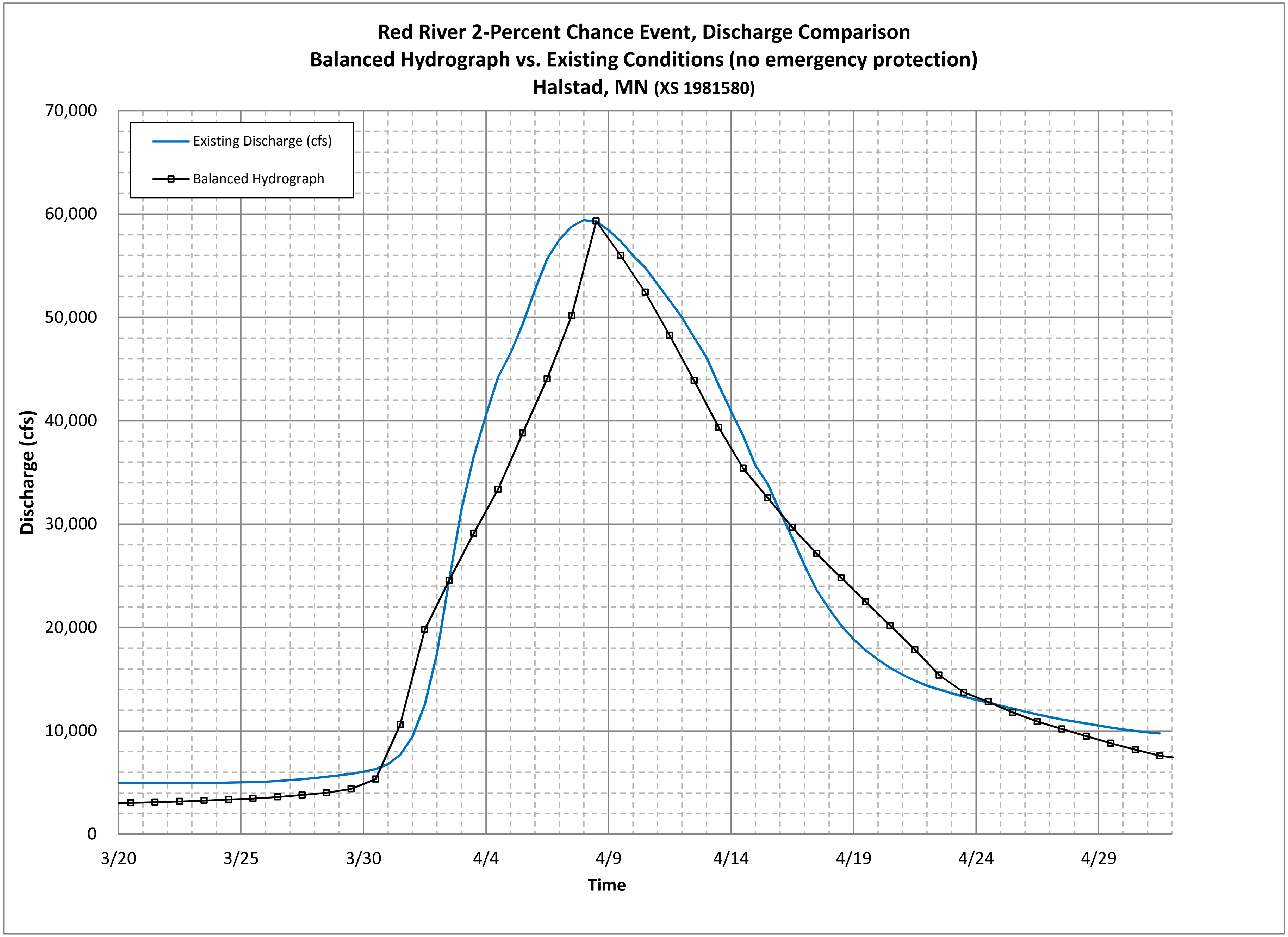

Red River 10-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Grand Forks, ND (XS 1558518)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
60,000
50,000
40,000
30,000
20,000
10,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time


Red River 10-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Drayton, ND (XS 1062362)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time

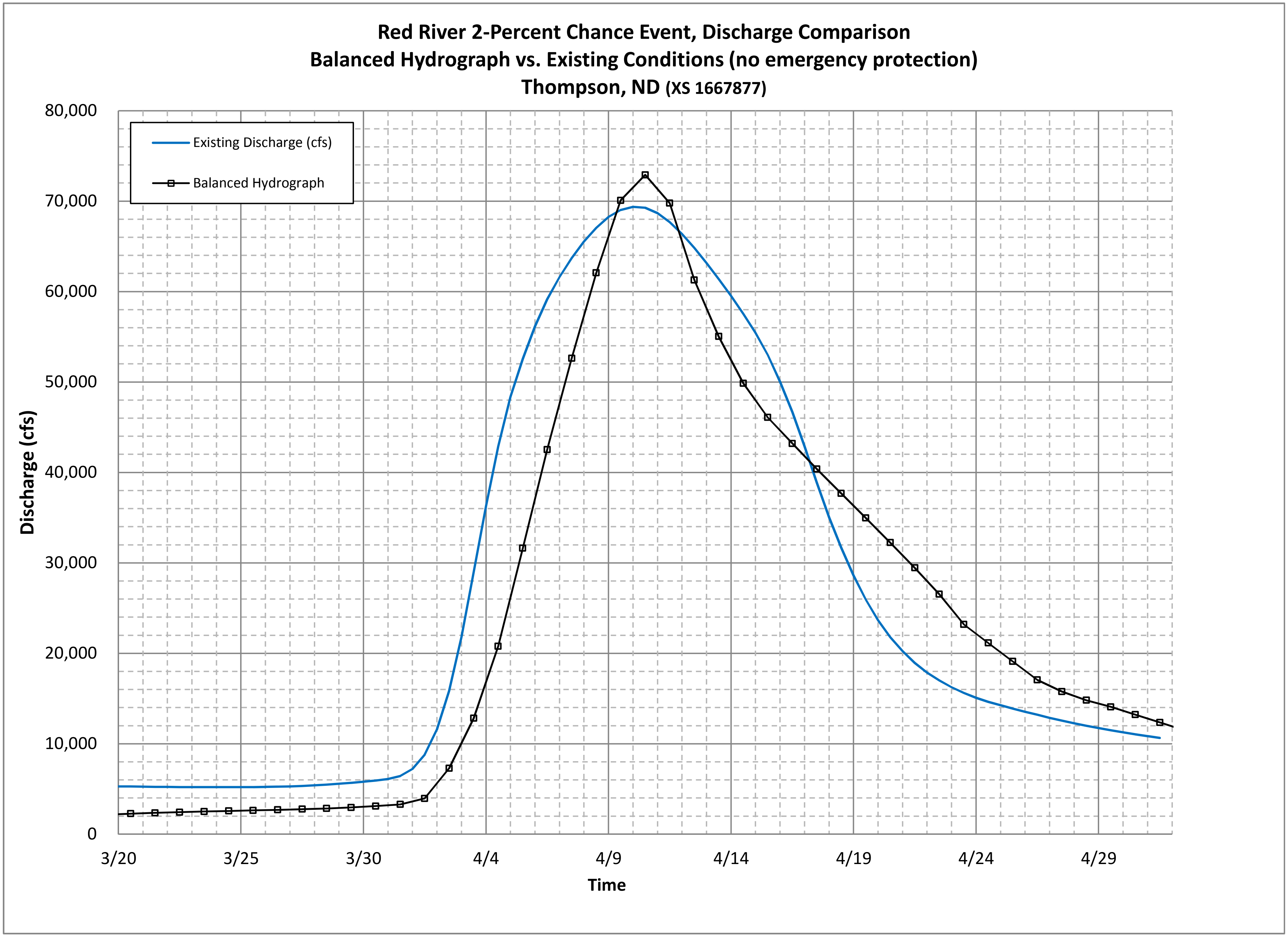


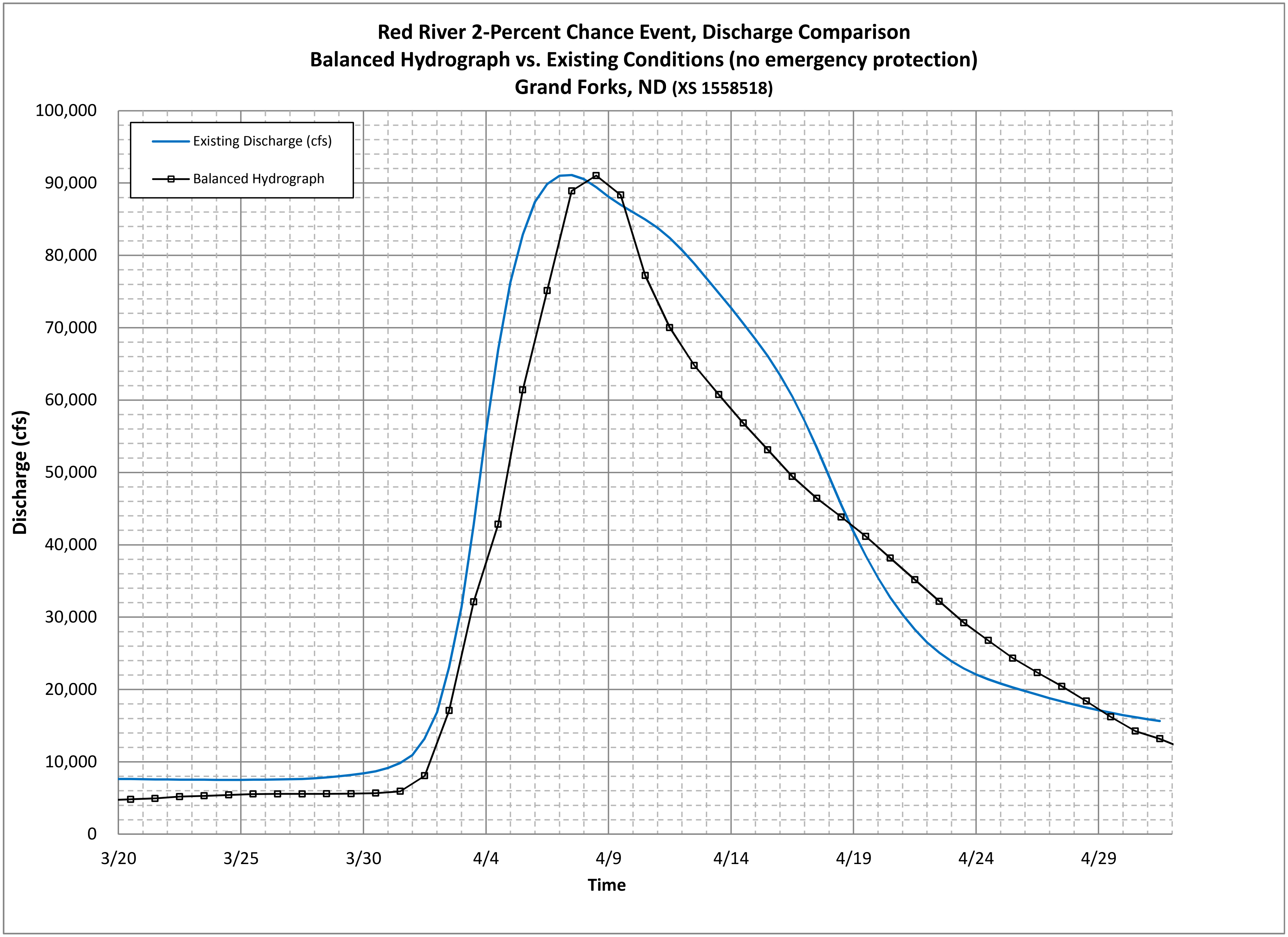
Red River 2-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Hickson, ND (XS 2563754)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
22,000
20,000
18,000
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time


Red River 2-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Fargo Gage - Fargo, ND (XS 2388223)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
30,000
25,000
20,000
15,000
10,000
5,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time

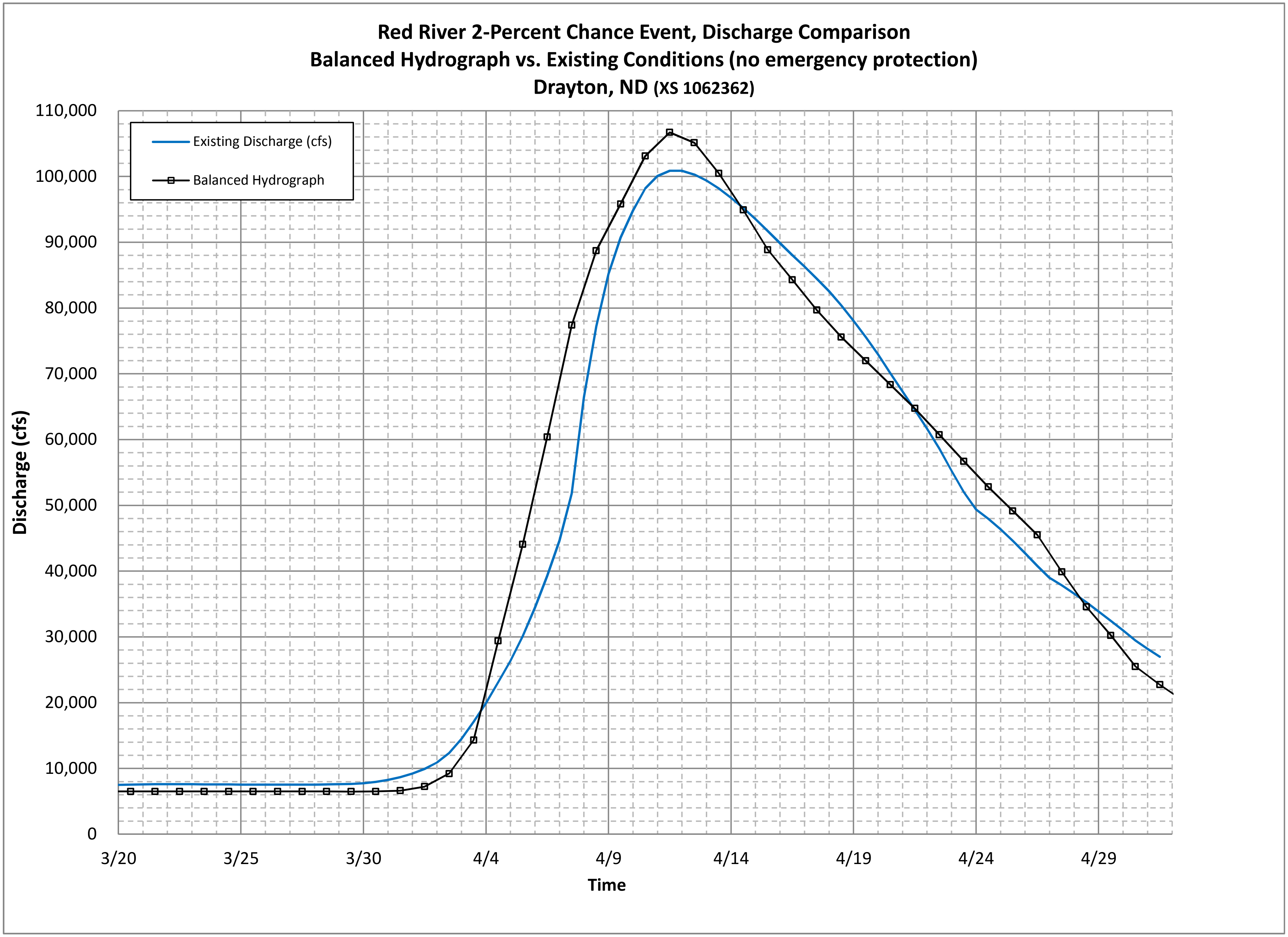


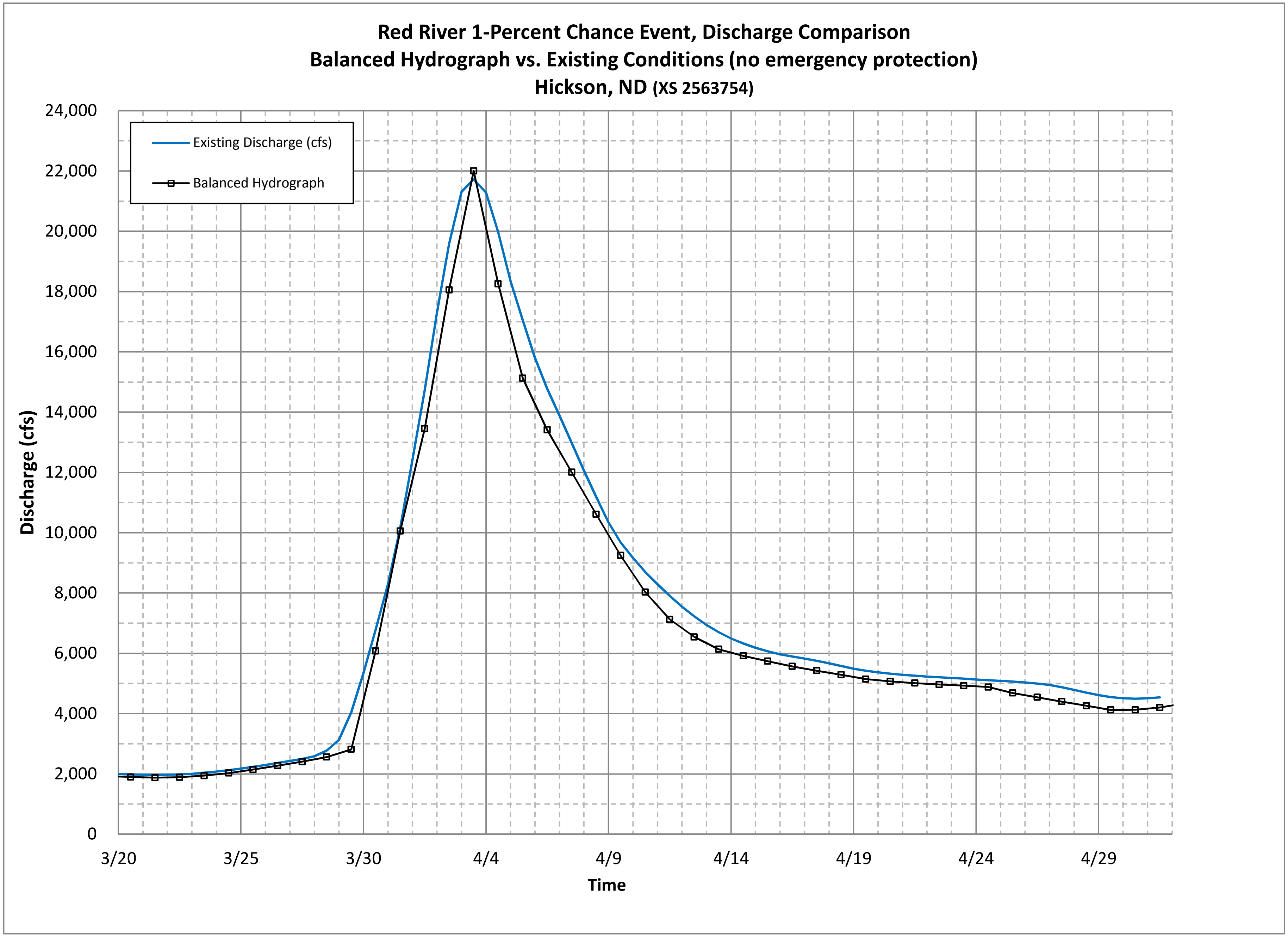
Red River 2-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Halstad, MN (XS 1981580)
Existing Discharge (cfs)
Balanced Hydrograph
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
Discharge (cfs)
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time


Red River 2-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Thompson, ND (XS 1667877)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time

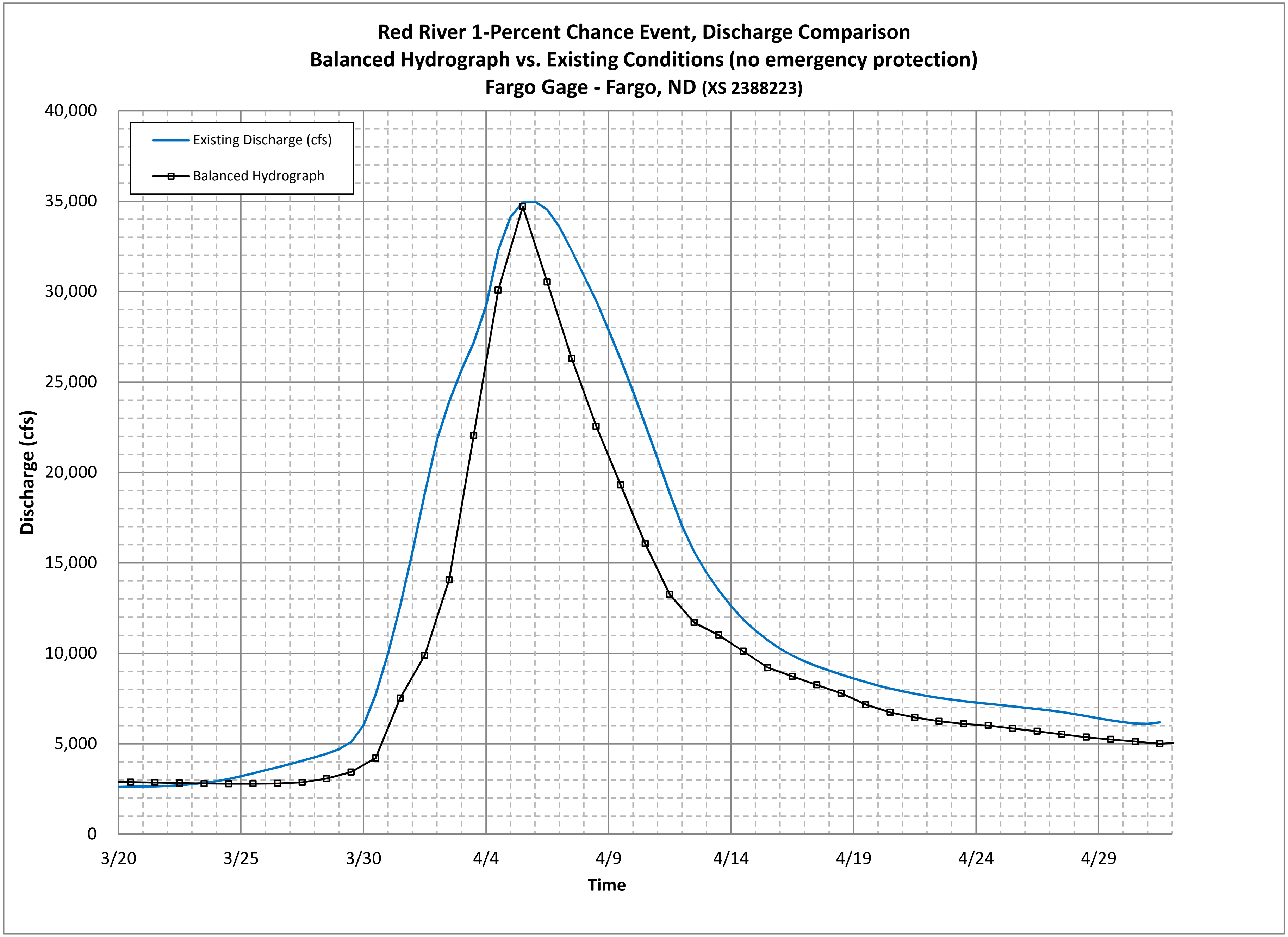


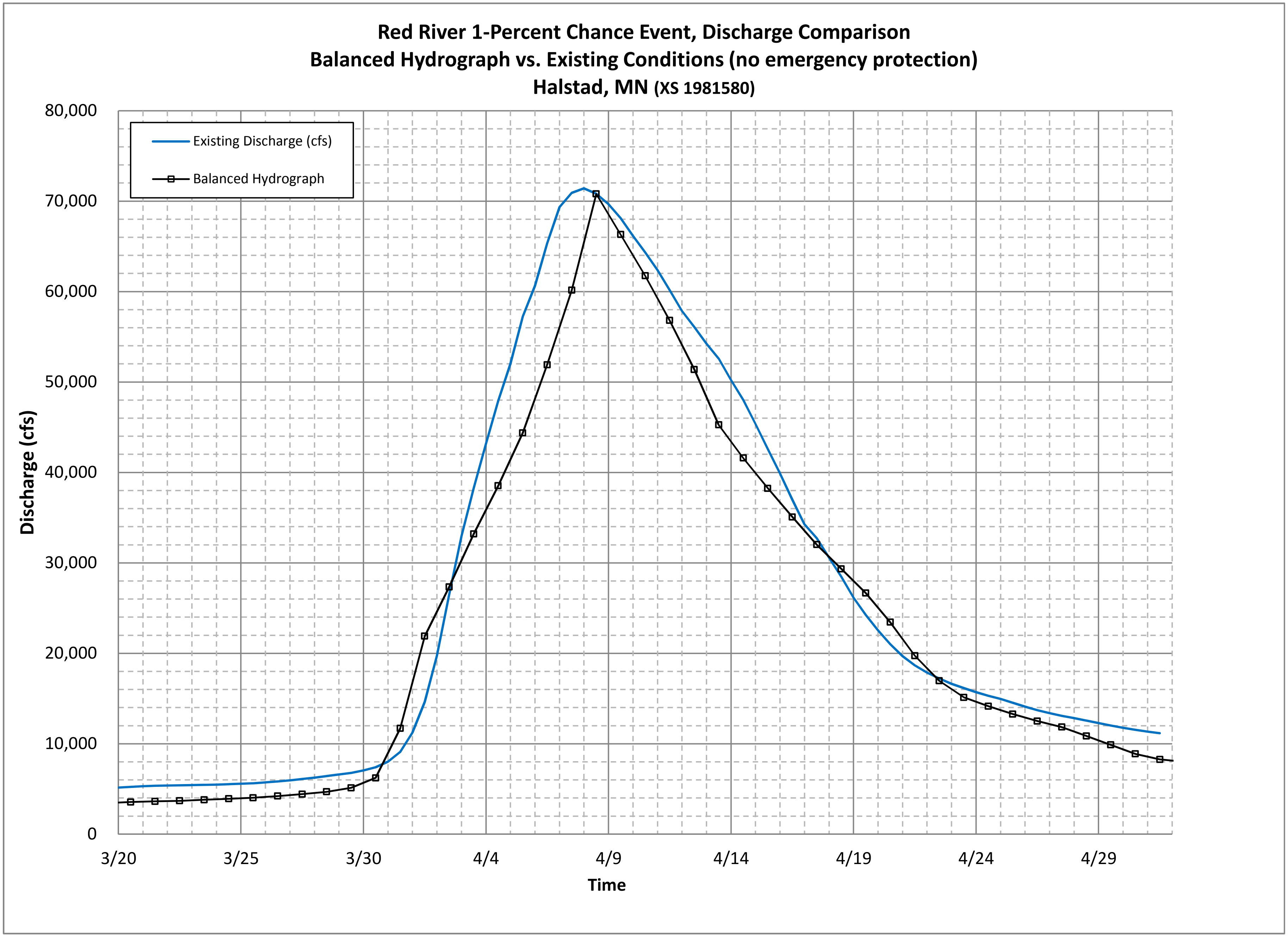
Red River 2-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Grand Forks, ND (XS 1558518)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time


Red River 2-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Drayton, ND (XS 1062362)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
110,000
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time

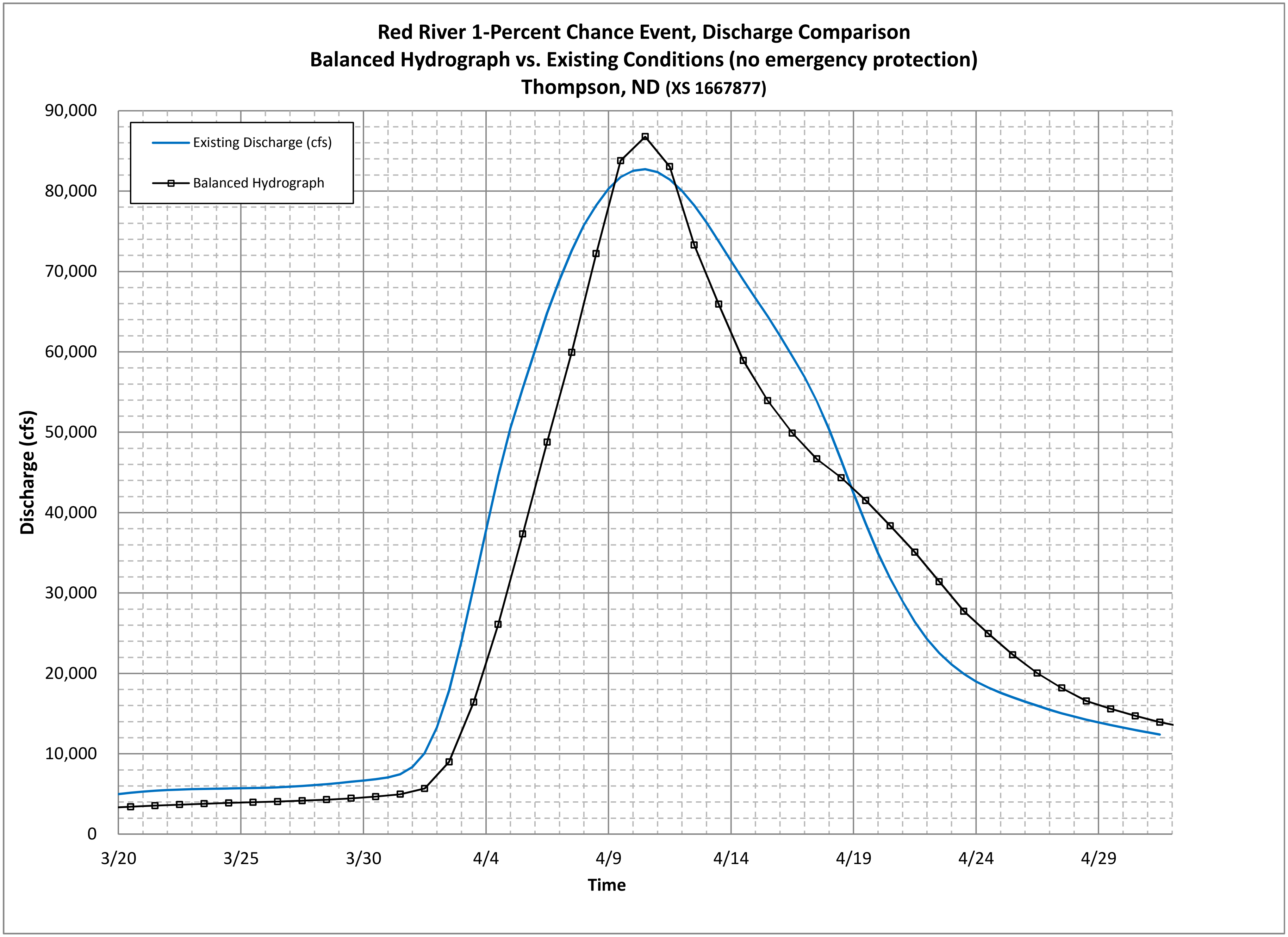


Red River 1-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Hickson, ND (XS 2563754)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
24,000
22,000
20,000
18,000
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time

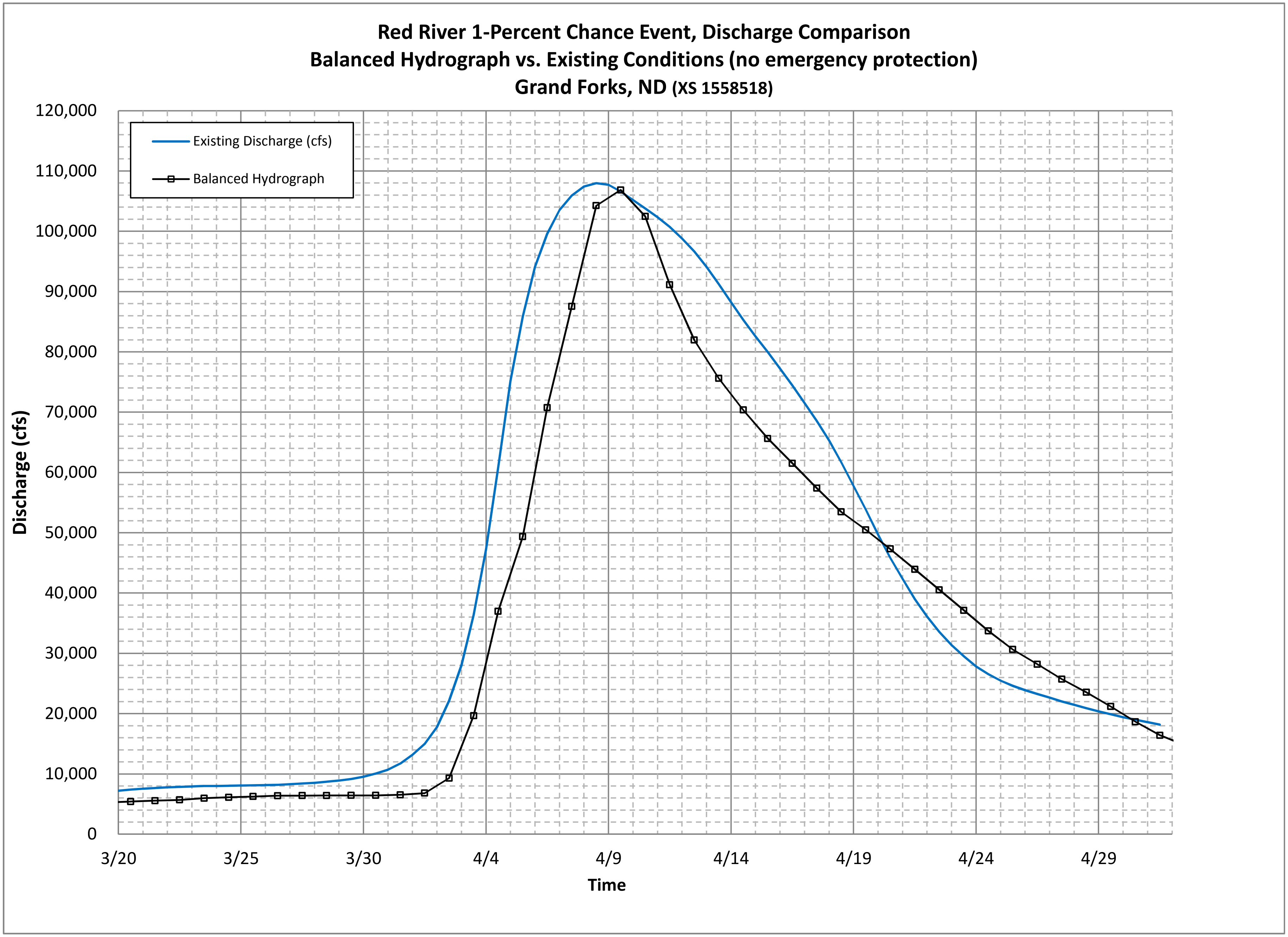

Red River 1-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Fargo Gage - Fargo, ND (XS 2388223)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
40,000
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time

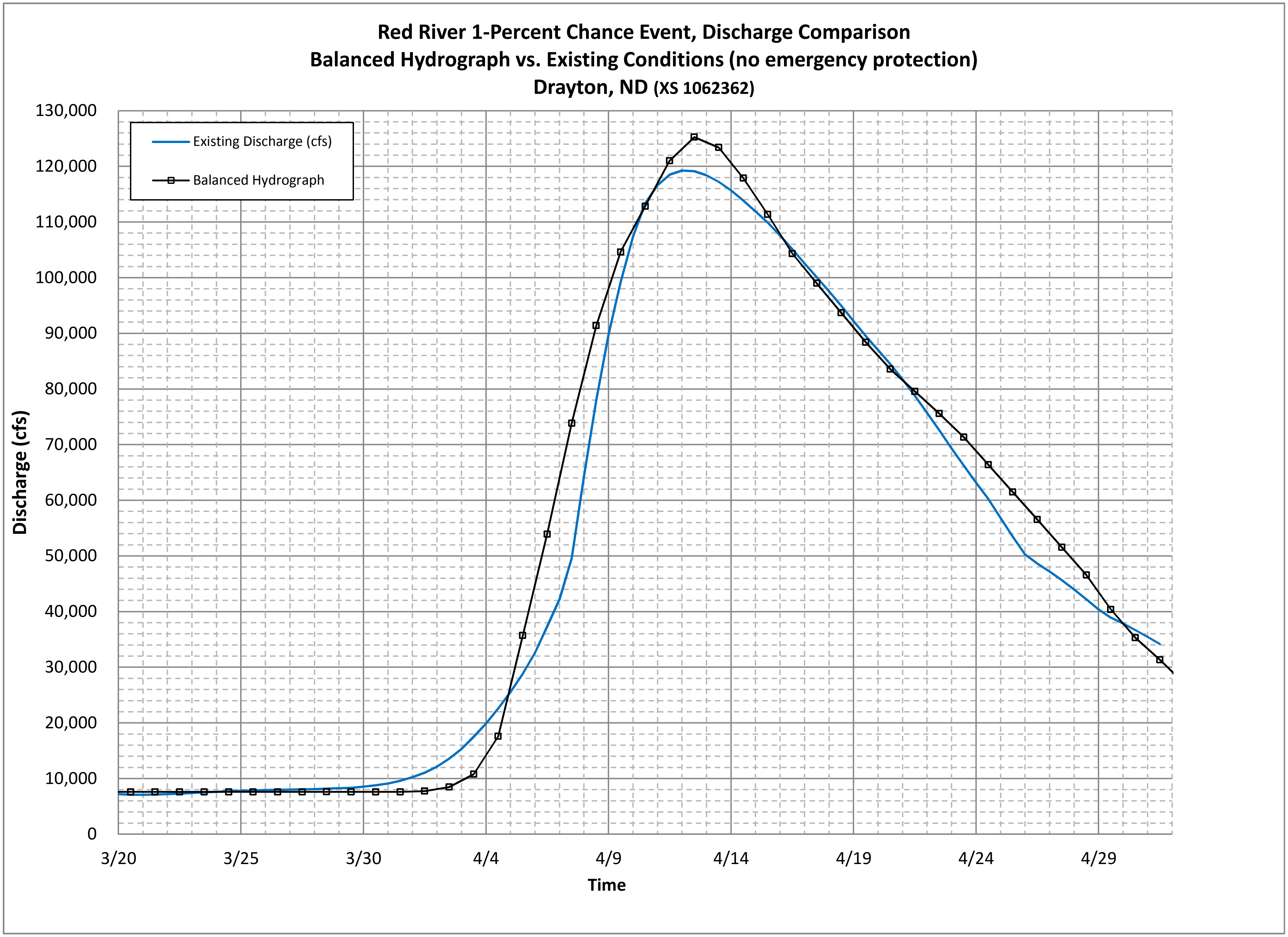


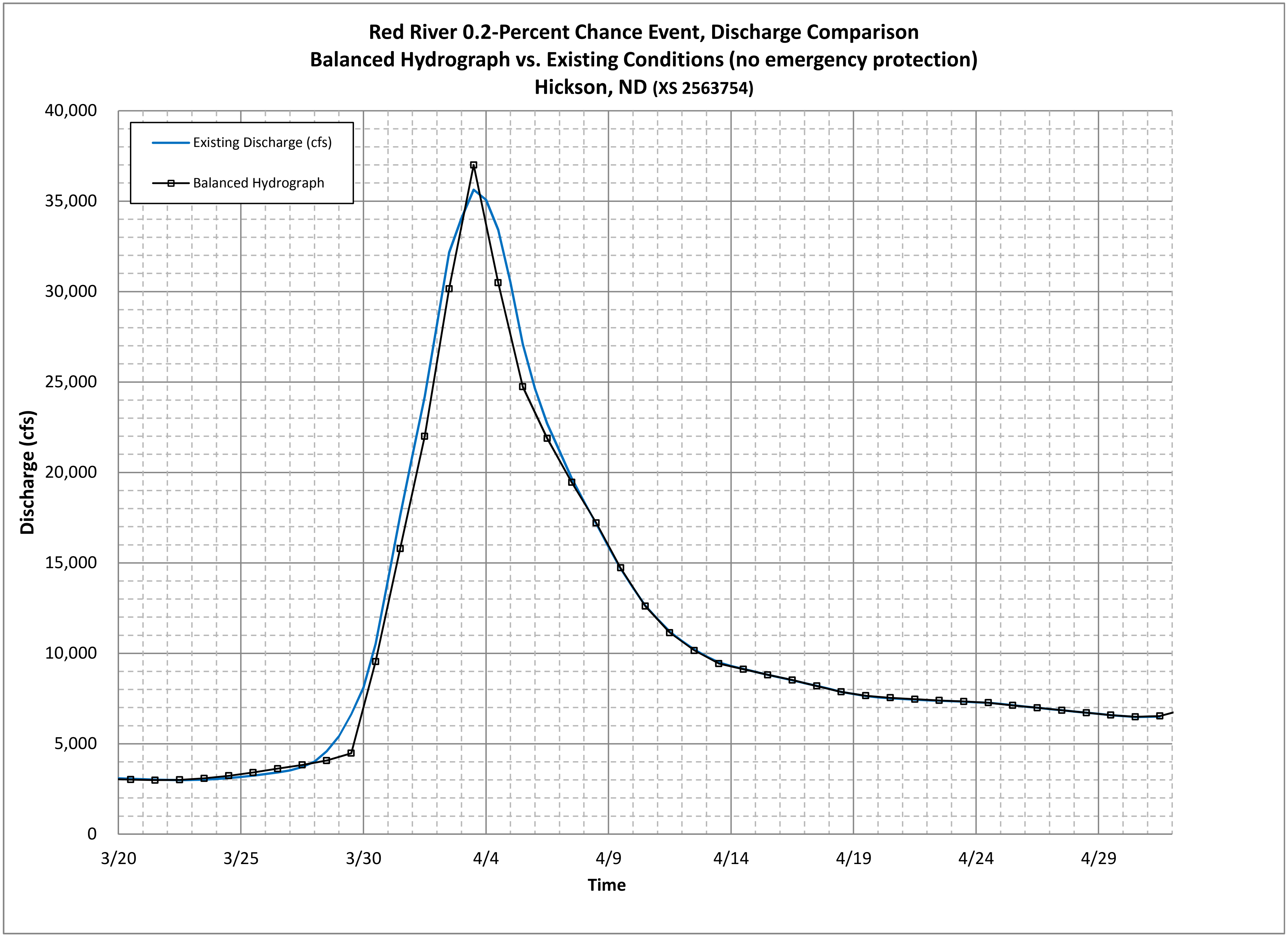
Red River 1-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Halstad, MN (XS 1981580)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time

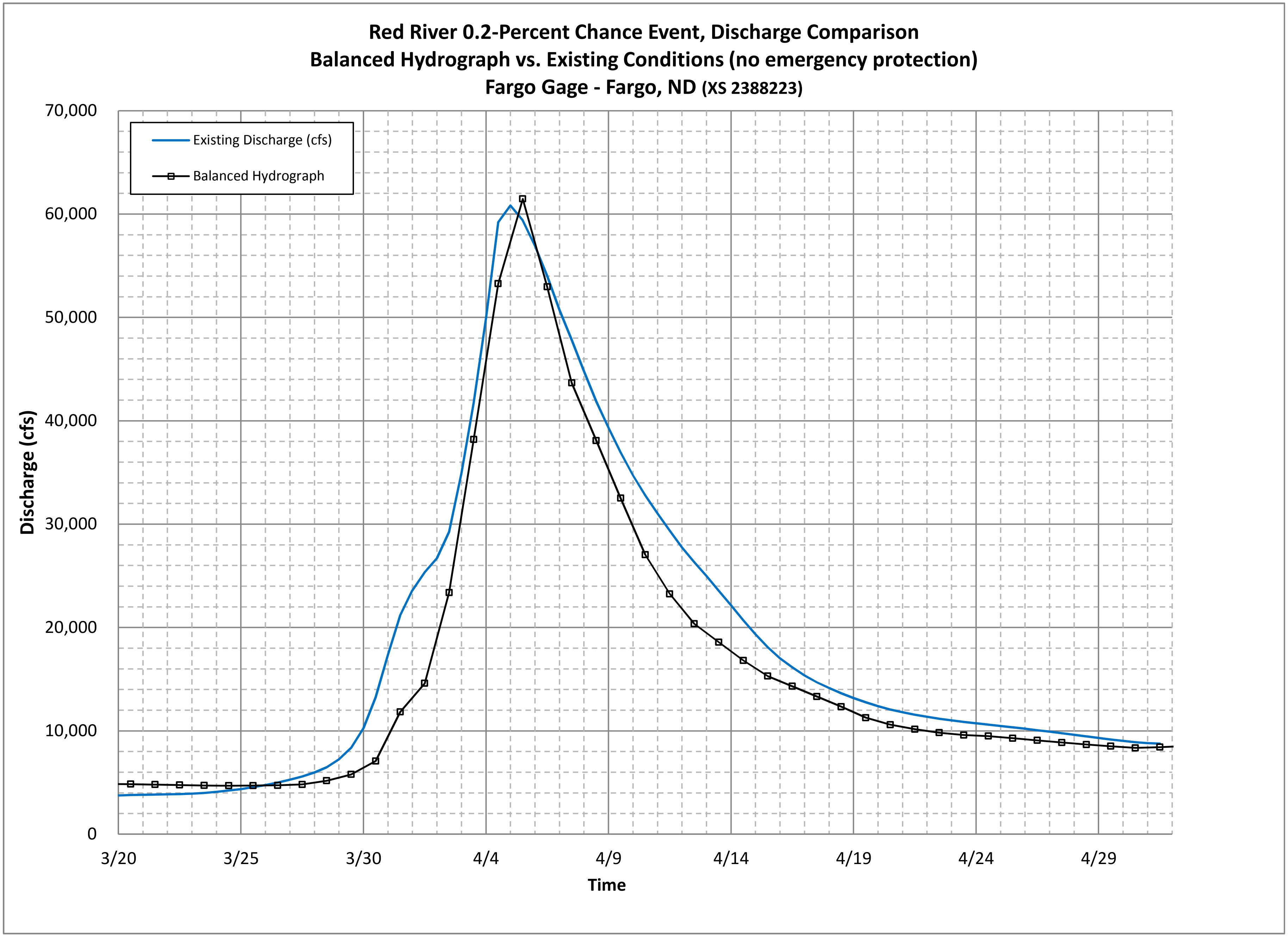


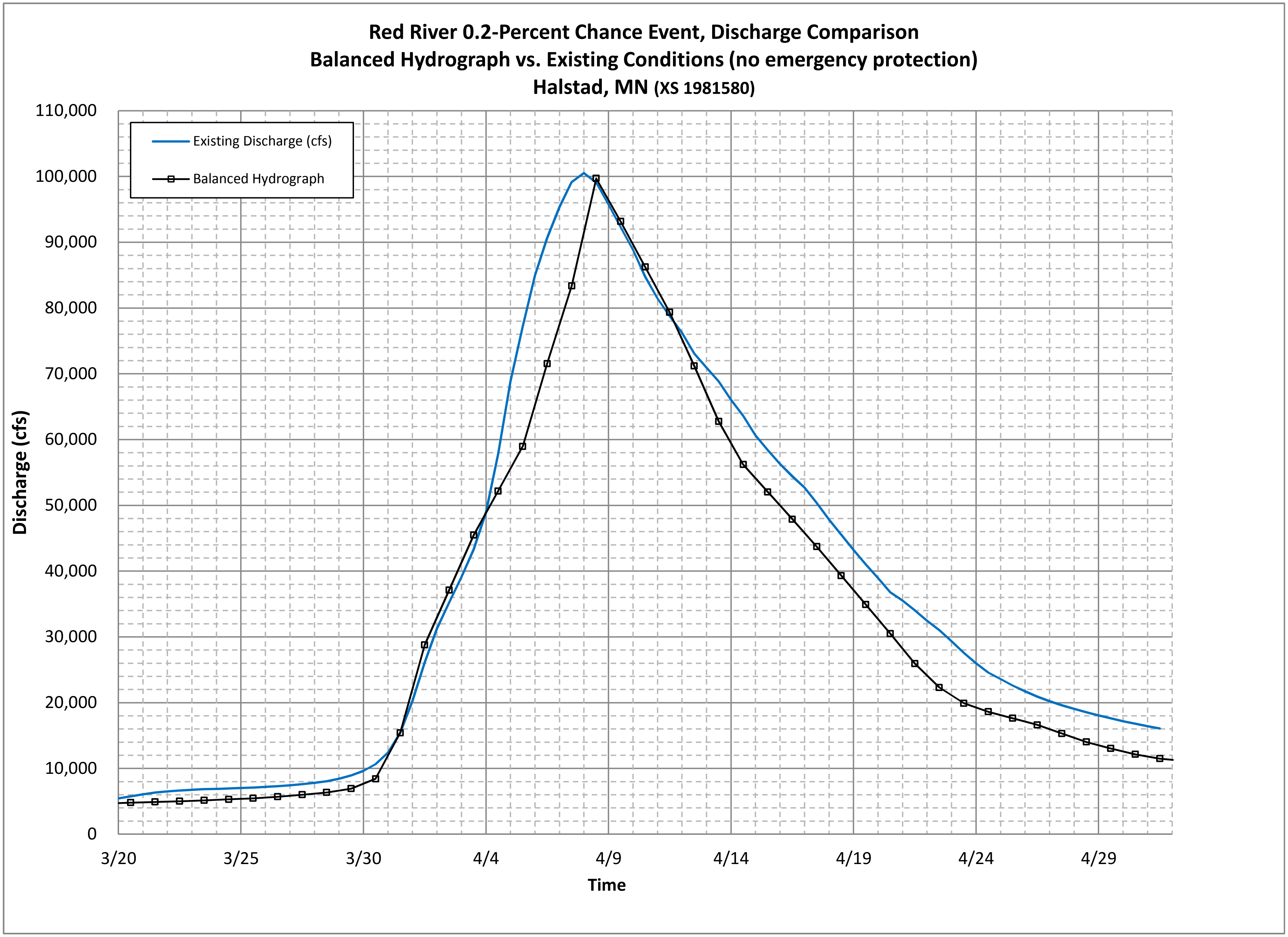
Red River 1-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Thompson, ND (XS 1667877)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time

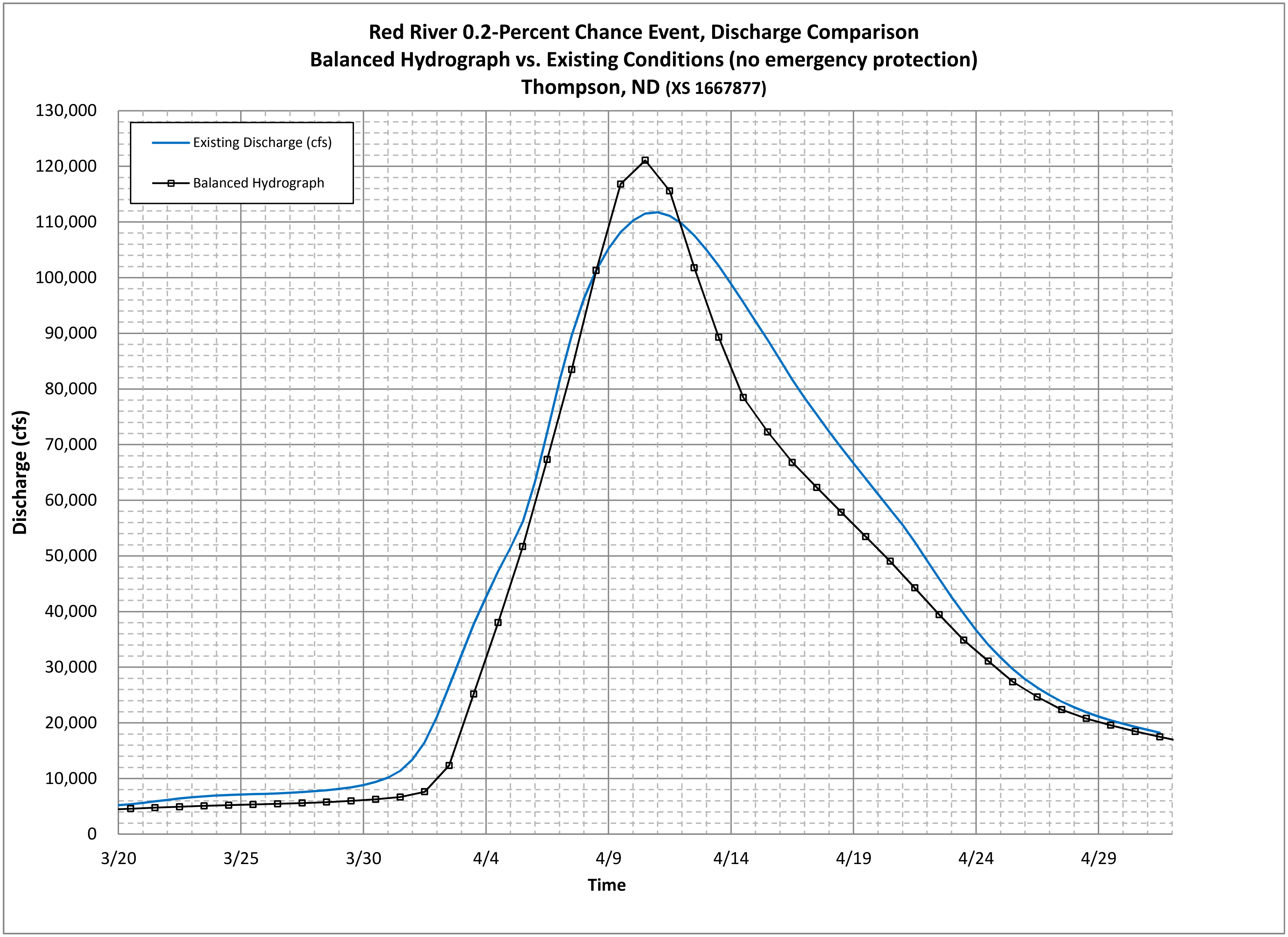

Red River 1-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Grand Forks, ND (XS 1558518)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
120,000
110,000
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time

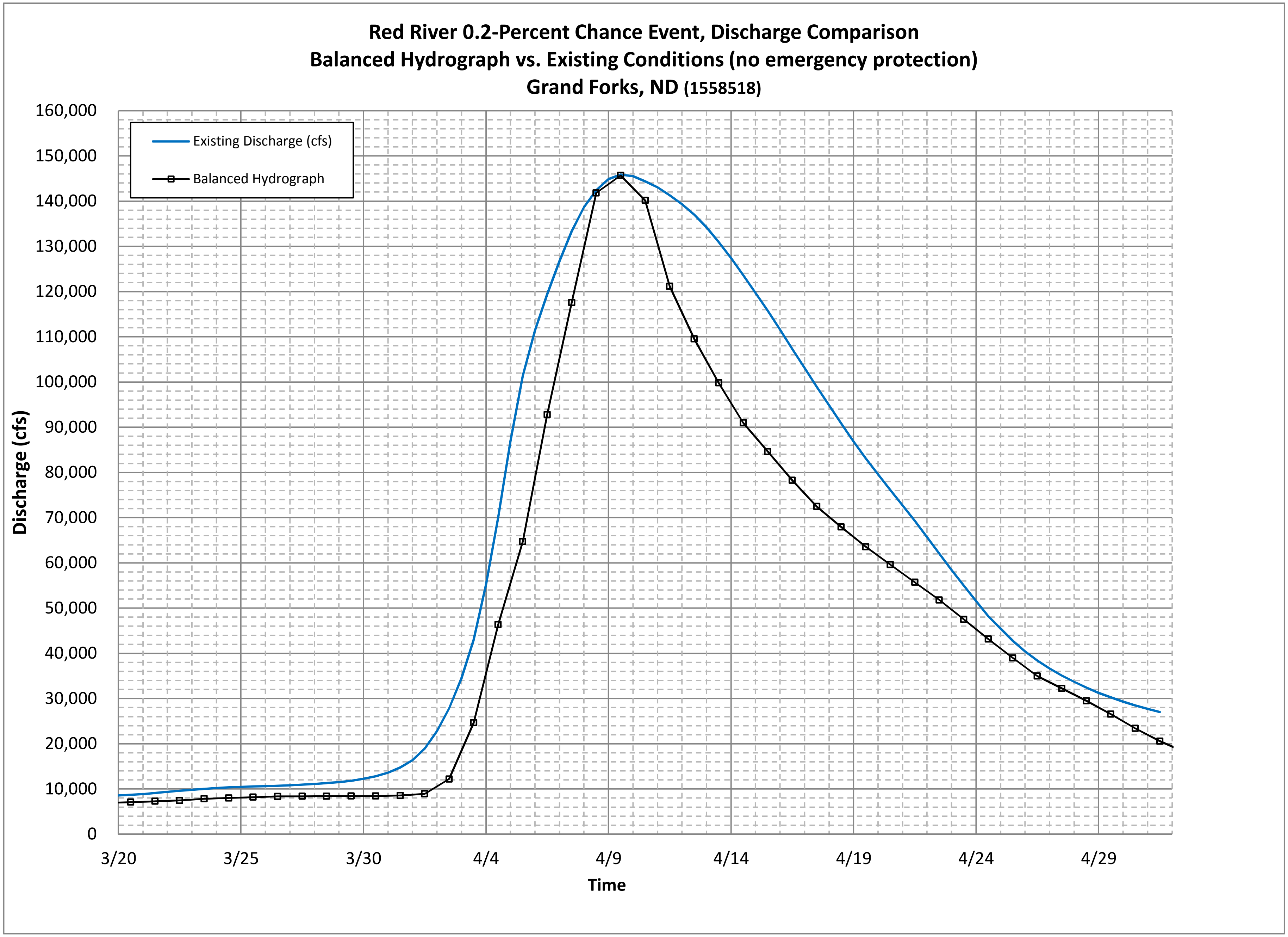

Red River 1-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Drayton, ND (XS 1062362)
Existing Discharge (cfs)
Balanced Hydrograph
130,000
120,000
110,000
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
Discharge (cfs)
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time

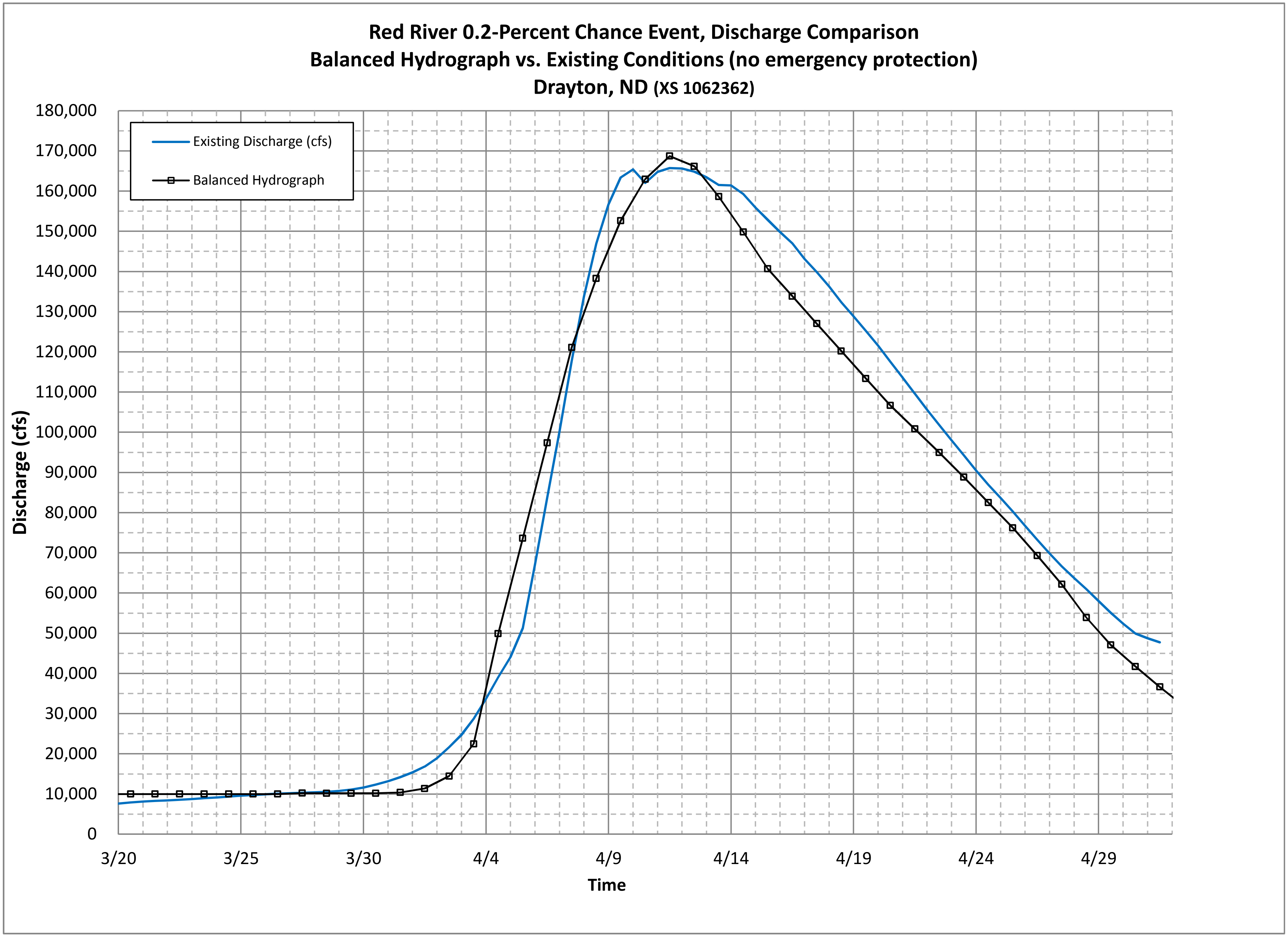

Red River 0.2-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Hickson, ND (XS 2563754)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
40,000
35,000
30,000
25,000
20,000
15,000
10,000
5,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time


Red River 0.2-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Fargo Gage - Fargo, ND (XS 2388223)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time


Red River 0.2-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Halstad, MN (XS 1981580)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
110,000
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time


Red River 0.2-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Thompson, ND (XS 1667877)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
130,000
120,000
110,000
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time


Red River 0.2-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Grand Forks, ND (1558518)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
160,000
150,000
140,000
130,000
120,000
110,000
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time


Red River 0.2-Percent Chance Event, Discharge Comparison
Balanced Hydrograph vs. Existing Conditions (no emergency protection)
Drayton, ND (XS 1062362)
Existing Discharge (cfs)
Balanced Hydrograph
Discharge (cfs)
180,000
170,000
160,000
150,000
140,000
130,000
120,000
110,000
100,000
90,000
80,000
70,000
60,000
50,000
40,000
30,000
20,000
10,000
0
3/20
3/25
3/30
4/4
4/9
4/14
4/19
4/24
4/29
Time

Appendix B – Hydraulics
Existing Condition

Exhibit F

Sensitivity Analysis – (Houston Engineering, Inc.)

MEMO

Technical Memorandum

HoustonEngineering Inc.

To: File: FM Metro Feasibility Study **From:** Greg Thompson

Date: October 5, 2010 **Subject:** Sensitivity Analysis

As part of the ATR comment response, a sensitivity analysis was completed. Specific requests were made as well as other issues identified by engineers familiar with the model to better identify the sensitivity that the assumptions have on the results. All of the sensitivity analyses were conducted with the existing conditions model and the "with project" ND Diversion model for the 1-percent chance event. The following is a list of topics visited in the sensitivity analysis as well as a summary and the results of each:

1. **Storage Area Weir Coefficients**
2. **Lateral Structure Weir Coefficients**
3. **Storage Area and Lateral Structure Weir Coefficients - Combined**
4. **Diversion Channel "n" value**
5. **Hydrology**

1. Storage Area Weir Coefficients

Analysis

Emergency protection through the Fargo Moorhead communities is not accounted for in any of the analyzed alternatives or in the existing conditions base model. This allows significant flood events to convey flood water through the normally protected parts of the Cities as well as other areas modeled as storage areas. Modelers and reviewers have expressed concern with the magnitude of the storage area conveyance and with the sensitivity this may have on the results of the modeling (downstream impacts). The amount of water conveyed through the storage areas is in direct correlation with the weir coefficients in the storage area connections. This analysis compared weir coefficients of 2.0, 2.6 (default) and 3.0.

Results

As expected, the weir coefficients affect the timing of the downstream hydrographs. As the weir coefficients are reduced, less water is conveyed through the storage areas and more is required to pass through the channel, lagging the hydrograph and increasing stage on the Red River. Reducing the weir coefficients by 25% (Default 2.6 to 2.0), causes an average downstream impact increase of 0.05 feet. Increasing the weir coefficients by 15% (Default 2.6 to 3.0) causes an average downstream impact reduction of 0.02 feet. Therefore, changing the storage area connection weir coefficients has little effect on the downstream impacts. See **Table 1.0** for downstream impact comparisons. **Figure 1.0** shows the general trend of downstream impacts. The magnitude of the impacts is directly related to the floodplain width. The slight increases and decreases in impacts related to the sensitivity analysis are also relative to the magnitude of the original downstream impacts.

2. Lateral Structure Weir Coefficients

Analysis

Lateral structures convey water from the primary channel to the overbank storage areas. The original lateral structure weir coefficient was set at 2.0. It was set lower than the storage area connections to reflect a lower efficiency discharging perpendicular to the conveyance of the channel. The sensitivity comparison consisted of a change from 2.0 to 2.6 and 2.0 to 1.5.

Results

Changing the lateral structure weir coefficients did not have a significant impact on the model. The general trend of increasing or decreasing the weir coefficients was as expected. A lower weir coefficient reduces the conveyance to the storage areas and increases the conveyance through the main channel. The results show an increase of 260 cfs (1%) in the channel during a 1-percent chance event and a similar reduction through the storage areas. Raising the weir coefficients had similar effects, however reversed.

3. Storage Area and Lateral Structure Weir Coefficients - Combined

Analysis

It was anticipated that changing the storage area connection and lateral structure weir coefficients would behave differently than analyzing them individually. The lateral structures may restrict water from leaving the main channel, however if the storage area connections are not changed, the storage areas will not pass as much water as they would if the storage area connections and lateral structures were changed together. To analyze this sensitivity, the lateral structure weir coefficients were set to 1.5 and the storage area connection weir coefficients were set to 2.0 (both lowered). This would be the most restrictive overbank scenario of the sensitivity analysis. Another scenario was completed that would reflect a higher overbank storage area conveyance. Here, the storage area connection weir coefficients were set at 3.0, and the lateral structure weir coefficients were set at 3.0.

Results

High weir coefficients conveyed more water outside of the channel resulting in a lower water surface profile. Lower weir coefficients retained more of the water in the channel at a higher water surface profile. Either extreme provided an approximate 500 cfs shift in water between the channel and the storage areas. The discharge differences carried throughout the model beyond Halstad, MN. The resulting sensitivity was greater than analyzing the coefficients individually. However, the impacts were still relatively low with extreme impacts of +0.16' to -0.12' near Climax where the 1-percent chance event impact was originally over 2 feet. See **Table 3.0** for combined downstream impact comparisons with the varied weir coefficients.

4. **Diversion Channel "n" value**

Analysis
The default channel roughness for the diversion channels were 0.03. The North Dakota Diversion was used in this sensitivity analysis. The channel "n" was increased to 0.035 and 0.04. Following the 2009 flood, the Winnipeg Diversion was analyzed and the resulting documentation supported a manning's roughness "n" value of 0.03.

Results
Increasing the "n" value to 0.035 resulted in a downstream reduction of less than 0.1 foot and an upstream stage increase of up to 0.15 feet. As anticipated, water shifted from the diversion channel to the Red River causing stage impacts at the diversion inlet.

Increasing the "n" value to 0.04 resulted in a downstream reduction of approximately 0.1 foot and an upstream stage increase of up to 0.4 feet. As with the 0.035 analysis, water shifted from the diversion channel to the Red River causing stage impacts at the diversion inlet.

5. **Hydrology**

Analysis
Select sites along the Red River have USGS gages with stage and discharge information. Locations along the Red River between USGS gaging sites have been estimated. The hydrology sensitivity analysis was conducted to see how a range of assumptions would impact the model and results. Fargo and Halstad have known USGS gages. An intermediate evaluation point is near Georgetown, MN downstream of the Sheyenne and Buffalo Rivers' confluences with the Red River.

a) A north-south shift was conducted to determine the sensitivity of hydrograph magnitude near Georgetown. This analysis consisted of a reduction in peak discharge near Georgetown of 25%. Additional water was introduced into the system between Georgetown and Halstad to make up the remaining portion of the hydrograph at Halstad.

b) A reversed north-south shift was also conducted. This increases the peak Red River discharge near Georgetown by 25%, and a reduction in contributions between Georgetown and Halstad.

c) Engineers familiar with the hydrology contributed from the North Dakota side of the project expressed concern with the significantly high flows along the Sheyenne and Maple Rivers. Assuming that the original Red River hydrograph near Georgetown is accurate, an east-west shift was conducted to identify the sensitivity of the distribution of inflows between Fargo and Georgetown. The comparison reduced the given Maple River flows and Sheyenne River breakout flows by 25%. To maintain volume near Georgetown, this resulted in a 60%

increase to the Buffalo River hydrograph. The original balanced hydrograph for the Buffalo River near Dilworth was near the low end of the accepted values. Increasing the Buffalo River by 60% raised it near the known 1-percent chance event discharge on the Buffalo River.

d) To magnify the east-west shift and also incorporate a north-south shift, the Maple River and Sheyenne River breakout flows were reduced by 40%. The Buffalo River remained at the 60% increase. This resulted in a smaller hydrograph at Georgetown and an increase in water added between Georgetown and Halstad.

Results
All of the hydrology sensitivities had equal hydrology and hydraulic conditions upstream of Fargo/Moorhead and the diversion alternatives had similar diverted discharges. Therefore, the only changes in the comparison are to hydrographs between Fargo and Halstad.

a) North South Shift - Reducing the hydrograph at Georgetown also included increasing contributions between Georgetown and Halstad. Downstream discharges contributing to the system have their own timing in the existing condition hydrograph. The diversion project reduced the time the flood wave requires to travel to a downstream location, therefore stacking the diverted flow on top of the locally contributing hydrographs. This is shown by the increase in impacts at downstream locations with the north-south shift (See **Table 5a**).

b) North South Shift (reversed) – Less water is introduced between Georgetown and Halstad resulting in reduced downstream impacts. See **Table 5b**.

c) East-West Shift (25%) – The impact of this shift was very minimal (< 0.05'). This is attributed to the volume of water upstream of the Fargo/Moorhead area remaining the same, the amount of diverted water remaining the same, and the hydrographs downstream of the project remaining the same. This also shows that the impacts of the project are not sensitive to the Maple River and Sheyenne River hydrographs, but directly related to the amount of water being diverted from upstream along the Red River and Wild Rice River. It appears as though the timing of the hydrographs between the Maple/Sheyenne and the Red River is similar between existing conditions and with diversion conditions and their flowpaths are relatively the same length. See **Table 5c**.

d) East-West Shift (40%) - The impact of this shift was also very minimal. There were minor reductions in the impacts directly downstream of the diversion outlet followed by minor increases (+0.02') near the highest impact location (Climax). See **Table 5d**.

Table 1.0 - Varying Storage Area Connection Weir Coefficients

North Dakota East 35K Diversion Downstream Impacts 1-Percent Chance Event				
Location	**Station**	ND Diversion with Weir = 2.0	ND Diversion with Weir = 2.6 (default)	ND Diversion with Weir = 3.0
		Benefit/Impact (ft)	Benefit/Impact (ft)	Benefit/Impact (ft)
Drayton Gage	1062682	0.45	0.44	0.44
ND SH#17/ MN SH317	1223183	0.40	0.39	0.39
Co. Hwy 15	1315673	0.34	0.33	0.32
Oslo Gage	1416287	0.40	0.39	0.38
DS Turtle River	1419932	0.36	0.35	0.35
US Turtle River	1440916	0.33	0.32	0.32
DS Grand Forks Levees	1533523	0.45	0.43	0.42
DS Red Lake River	1560870	1.01	0.98	0.96
32nd Ave, Grand Forks	1580152	1.22	1.17	1.15
Thompson Gage	1667665	1.19	1.13	1.09
Co. Hwy 25/ Co. Rd 221	1719816	1.95	1.86	1.81
DS Sandhill River/ Climax	1763746	2.12	2.02	1.97
Maximum Impact Location	1782305	2.14	2.02	1.98
Nielsville	1829650	1.92	1.82	1.78
DS Marsh River	1864960	1.66	1.57	1.55
US Goose River/ Shelly	1890722	1.26	1.20	1.17
Co. Rd 139	1951761	0.84	0.80	0.78
Halstad Gage	1981580	0.90	0.86	0.84
Hendrum	2038359	0.95	0.90	0.88
Perley	2129181	0.61	0.58	0.58
Georgetown	2193941	0.67	0.65	0.63
North River/ Clay Co. Hwy 93	2305647	-5.59	-5.56	-5.52
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	-7.51	-7.44	-7.37
Fargo Gage (13th Ave S, 12th Ave S)	2388223	-9.61	-9.47	-9.37
US Rose Coulee/ US 50th Ave S Moorhead	2430241	-9.61	-9.52	-9.48
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	-9.79	-9.73	-9.69
US ND Wild Rice River	2484618	-8.77	-8.77	-8.76
US Diversion	2531338	0.02	0.02	0.02
Hickson Gage	2563878	0.00	0.01	0.01

Figure 1.0 - Typical Downstream Impact Variation (Related to floodplain width)

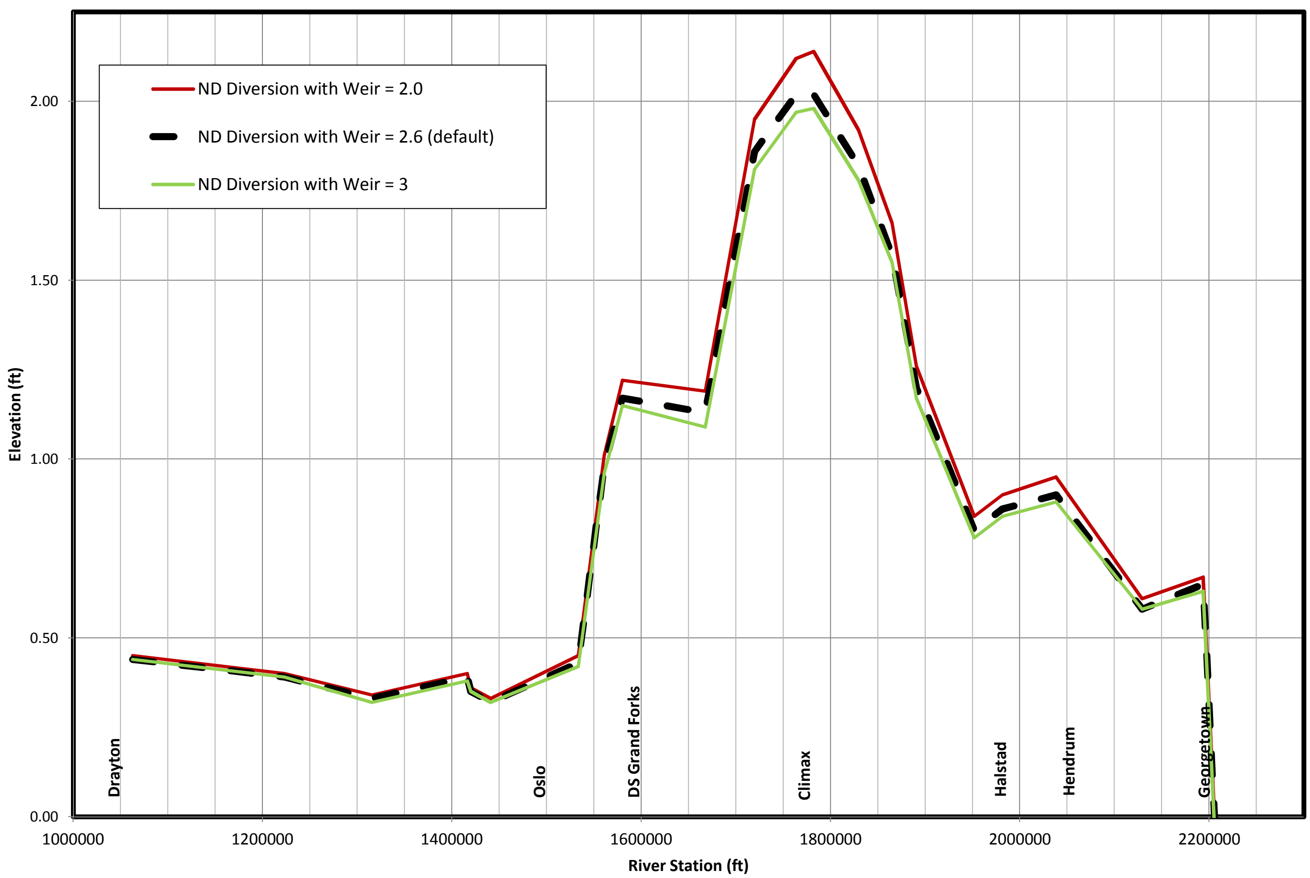


Table 3.0 - Varying Storage Area Connection and Lateral Structure Weir Coefficients

North Dakota East 35K Diversion Downstream Impacts

1-Percent Chance Event

Location	Station	ND Diversion with SAC weir = 2.0 LS weir = 1.5		ND Diversion Original	ND Diversion with SAC weir = 3.0 LS weir = 3.0	
		Benefit/Impact (ft)		Benefit/Impact (ft)	Benefit/Impact (ft)	
Drayton Gage	1062682	0.47	7%	0.44	0.43	-2%
ND SH#17/ MN SH317	1223183	0.40	3%	0.39	0.38	-3%
Co. Hwy 15	1315673	0.35	6%	0.33	0.31	-6%
Oslo Gage	1416287	0.42	8%	0.39	0.37	-5%
DS Turtle River	1419932	0.38	9%	0.35	0.34	-3%
US Turtle River	1440916	0.35	9%	0.32	0.31	-3%
DS Grand Forks Levees	1533523	0.47	9%	0.43	0.41	-5%
DS Red Lake River	1560870	1.08	10%	0.98	0.94	-4%
32nd Ave, Grand Forks	1580152	1.22	4%	1.17	1.11	-5%
Thompson Gage	1667665	1.16	3%	1.13	1.05	-7%
Co. Hwy 25/ Co. Rd 221	1719816	1.97	6%	1.86	1.73	-7%
DS Sandhill River/ Climax	1763746	2.16	7%	2.02	1.88	-7%
Maximum Impact Location	1782305	2.18	8%	2.02	1.90	-6%
Nielsville	1829650	1.96	8%	1.82	1.70	-7%
DS Marsh River	1864960	1.70	8%	1.57	1.48	-6%
US Goose River/ Shelly	1890722	1.29	8%	1.20	1.11	-8%
Co. Rd 139	1951761	0.87	9%	0.80	0.74	-8%
Halstad Gage	1981580	0.93	8%	0.86	0.80	-7%
Hendrum	2038359	0.99	10%	0.90	0.82	-9%
Perley	2129181	0.64	10%	0.58	0.54	-7%
Georgetown	2193941	0.70	8%	0.65	0.59	-9%
North River/ Clay Co. Hwy 93	2305647	-5.52	-1%	-5.56	-5.56	0%
19th Ave N Fargo/ 28th Ave N Moorhead	2360321	-7.51	1%	-7.44	-7.37	-1%
Fargo Gage (13th Ave S, 12th Ave S)	2388223	-9.51	0%	-9.47	-9.51	0%
US Rose Coulee/ US 50th Ave S Moorhead	2430241	-9.67	2%	-9.52	-9.37	-2%
52nd Ave S Fargo/ 60th Ave S Moorhead	2438085	-9.88	2%	-9.73	-9.57	-2%
US ND Wild Rice River	2484618	-8.90	1%	-8.77	-8.58	-2%
US Diversion	2531338	-0.02	0%	0.02	0.07	0%
Hickson Gage	2563878	-0.02	0%	0.01	0.05	0%

SAC = Storage Area Connection
LS = Lateral Structure

Table 5a Sensitivity Analysis - Hydrology Reduced 25% between Fargo and Buffalo								
North Dakota East 35K Diversion - 1-Percent Chance Event								
Location	Station	Existing No Protection Elevation		ND East 35K Diversion Elevation		Difference Project vs. Existing No Protection		Original Submittal
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	
Thompson Gage	1,667,665	847.74	86413	849.00	98251	1.26	11838	1.14
Co. Hwy 25/ Co. Rd 221	1,719,816	853.93	86102	856.01	98092	2.08	11990	1.88
DS Sandhill River/ Climax	1,763,746	857.88	85859	860.15	97900	2.27	12041	2.06
Maximum Impact Location	1,782,305	859.24	81675	861.54	93534	2.30	11859	2.06
Nielsville	1,829,650	861.81	80622	863.87	92482	2.06	11859	1.87
DS Marsh River	1,864,960	863.14	80074	864.93	92115	1.79	12040	1.63
US Goose River/ Shelly	1,890,722	865.21	64673	866.59	72132	1.38	7459	1.24
Co. Rd 139	1,951,761	867.39	70116	868.33	82698	0.94	12582	0.82
Halstad Gage	1,981,580	868.67	69836	869.73	84114	1.06	14278	0.88
Hendrum	2,038,359	873.28	60661	874.34	69838	1.06	9177	0.92
Perley	2,129,181	877.80	50883	878.43	57341	0.63	6458	0.59
Georgetown	2,193,941	881.79		882.38		0.59	9687	0.67
North River/ Clay Co. Hwy 93	2,305,647	893.53		887.62		-5.91		-5.58
19th Ave N Fargo/ 28th Ave N Moorhead	2,360,321	898.98		891.34		-7.64		-7.43
Fargo Gage (13th Ave S, 12th Ave S)	2,388,223	903.00 (*40.26)		893.48 (*30.74)		-9.60	-13542	-9.46
US Rose Coulee/ US 50th Ave S Moorhead	2,430,241	906.15		896.55		-9.60		-9.52
52nd Ave S Fargo/ 60th Ave S Moorhead	2,438,085	907.01		897.21		-9.80		-9.72
US ND Wild Rice River	2,484,618	910.43		901.65		-8.78	-14165	-8.77
US Diversion	2,531,338	914.89		914.90		0.01		0.02
Hickson Gage	2,563,878	917.11	24601	917.11	24542	0.00	-59	0.01

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

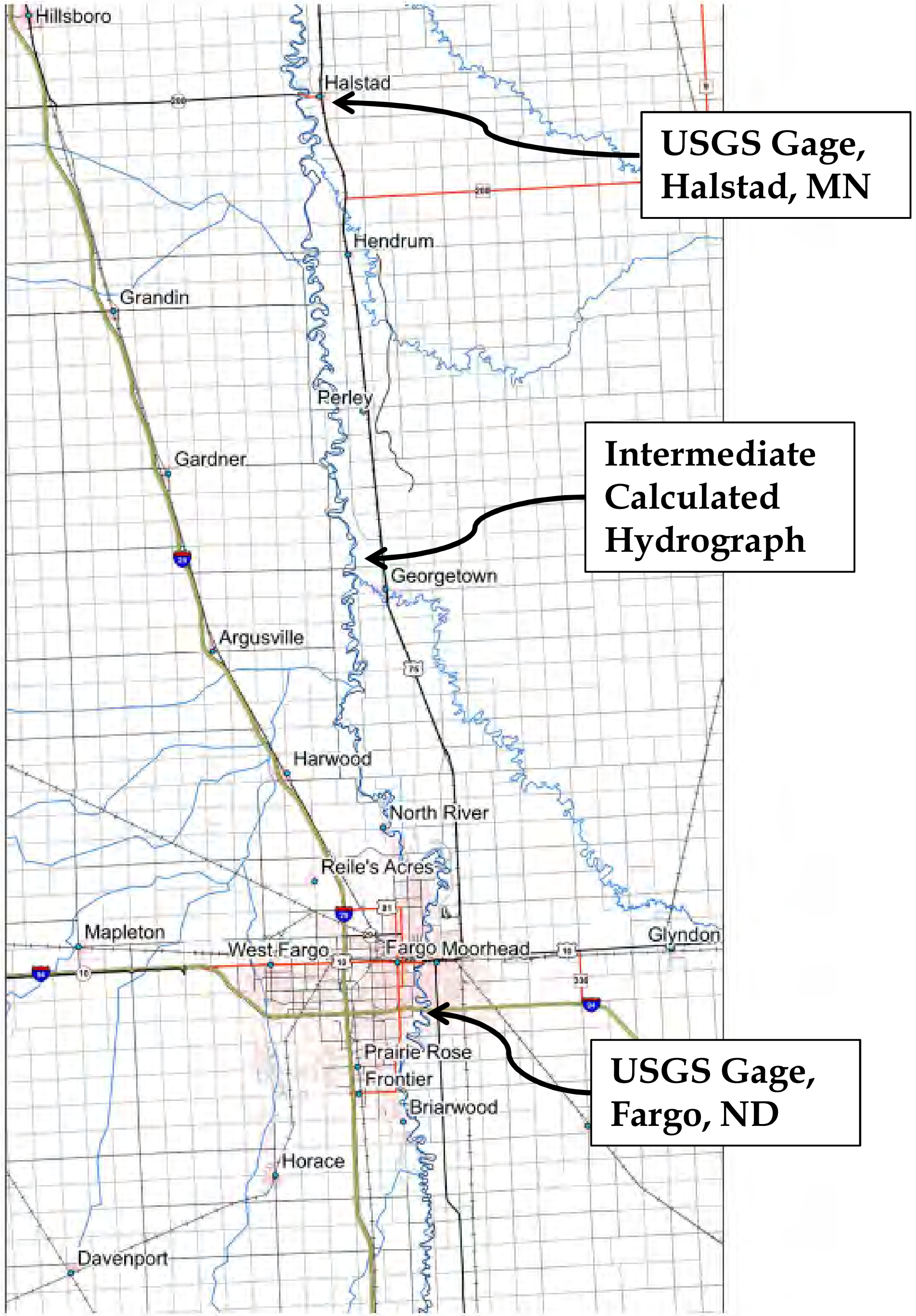
Table 5b Sensitivity Analysis - Hydrology Increased 25% between Fargo and Buffalo								
North Dakota East 35K Diversion - 1-Percent Chance Event								
Location	Station	Existing No Protection Elevation		ND East 35K Diversion Elevation		Difference Project vs. Existing No Protection		Original Submittal
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	
Thompson Gage	1,667,665	847.75	86584	848.84	96538	1.09	9954	1.14
Co. Hwy 25/ Co. Rd 221	1,719,816	853.96	86282	855.74	96303	1.78	10021	1.88
DS Sandhill River/ Climax	1,763,746	857.91	86038	859.84	96093	1.93	10055	2.06
Maximum Impact Location	1,782,305	859.28	81857	861.21	91681	1.93	9823	2.06
Nielsville	1,829,650	861.84	80811	863.58	90639	1.74	9828	1.87
DS Marsh River	1,864,960	863.16	80272	864.66	90183	1.50	9912	1.63
US Goose River/ Shelly	1,890,722	865.23	64673	866.37	72132	1.14	7459	1.24
Co. Rd 139	1,951,761	867.41	70440	868.16	79486	0.75	9046	0.82
Halstad Gage	1,981,580	868.70	70255	869.49	79788	0.79	9533	0.88
Hendrum	2,038,359	873.64	66054	874.48	74294	0.84	8240	0.92
Perley	2,129,181	878.51	61635	879.10	71002	0.59	9367	0.59
Georgetown	2,193,941	882.65		883.24		0.59	6004	0.67
North River/ Clay Co. Hwy 93	2,305,647	893.56		888.22		-5.34		-5.58
19th Ave N Fargo/ 28th Ave N Moorhead	2,360,321	898.99		891.70		-7.29		-7.43
Fargo Gage (13th Ave S, 12th Ave S)	2,388,223	903.00 (*40.26)		893.48 (*30.74)		-9.36	-13522	-9.46
US Rose Coulee/ US 50th Ave S Moorhead	2,430,241	906.15		896.68		-9.47		-9.52
52nd Ave S Fargo/ 60th Ave S Moorhead	2,438,085	907.01		897.33		-9.68		-9.72
US ND Wild Rice River	2,484,618	910.44		901.69		-8.75	-14181	-8.77
US Diversion	2,531,338	914.89		914.91		0.02		0.02
Hickson Gage	2,563,878	917.11	24586	917.12	24527	0.01	-59	0.01

Table 5c Sensitivity Analysis - West Hydrology Reduced 25%								
North Dakota East 35K Diversion - 1-Percent Chance Event								
Location	Station	Existing No Protection Elevation		ND East 35K Diversion Elevation		Difference Project vs. Existing No Protection		Original Submittal
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	
Thompson Gage	1,667,665	847.66	85868	848.82	96345	1.16	10477	1.14
Co. Hwy 25/ Co. Rd 221	1,719,816	853.83	85560	855.70	96108	1.87	10548	1.88
DS Sandhill River/ Climax	1,763,746	857.77	85312	859.80	95897	2.03	10584	2.06
Maximum Impact Location	1,782,305	859.13	81188	861.16	91540	2.03	10352	2.06
Nielsville	1,829,650	861.71	80147	863.54	90491	1.83	10344	1.87
DS Marsh River	1,864,960	863.06	79563	864.63	90048	1.57	10485	1.63
US Goose River/ Shelly	1,890,722	865.15	64673	866.35	72132	1.20	7459	1.24
Co. Rd 139	1,951,761	867.36	69856	868.15	79594	0.79	9738	0.82
Halstad Gage	1,981,580	868.64	69487	869.49	79769	0.85	10283	0.88
Hendrum	2,038,359	873.47	63762	874.36	72164	0.89	8403	0.92
Perley	2,129,181	878.19	56592	878.74	63250	0.55	6659	0.59
Georgetown	2,193,941	882.18		882.78		0.60	7627	0.67
North River/ Clay Co. Hwy 93	2,305,647	893.53		888.09		-5.44		-5.58
19th Ave N Fargo/ 28th Ave N Moorhead	2,360,321	898.98		891.58		-7.40		-7.43
Fargo Gage (13th Ave S, 12th Ave S)	2,388,223	903.00 (*40.26)		893.55 (*30.81)		-9.45	-13545	-9.46
US Rose Coulee/ US 50th Ave S Moorhead	2,430,241	906.15		896.63		-9.52		-9.52
52nd Ave S Fargo/ 60th Ave S Moorhead	2,438,085	907.01		897.28		-9.73		-9.72
US ND Wild Rice River	2,484,618	910.44		901.67		-8.77	-14178	-8.77
US Diversion	2,531,338	914.89		914.90		0.01		0.02
Hickson Gage	2,563,878	917.11	24601	917.11	24542	0.00	-59	0.01

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

Table 5d Sensitivity Analysis - West Hydrology Reduced 40%								
North Dakota East 35K Diversion - 1-Percent Chance Event								
Location	Station	Existing No Protection Elevation		ND East 35K Diversion Elevation		Difference Project vs. Existing No Protection		Original Submittal
		Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	Elevation (ft)	Discharge (cfs)	
Thompson Gage	1,667,665	847.78	86782	848.93	97453	1.15	10671	1.14
Co. Hwy 25/ Co. Rd 221	1,719,816	854.00	86474	855.89	97231	1.89	10757	1.88
DS Sandhill River/ Climax	1,763,746	857.95	86221	860.01	97035	2.06	10814	2.06
Maximum Impact Location	1,782,305	859.31	82094	861.39	92696	2.08	10602	2.06
Nielsville	1,829,650	861.88	81055	863.74	91644	1.86	10588	1.87
DS Marsh River	1,864,960	863.19	80485	864.81	91190	1.62	10706	1.63
US Goose River/ Shelly	1,890,722	865.26	64673	866.48	72132	1.22	7459	1.24
Co. Rd 139	1,951,761	867.43	70727	868.25	80978	0.82	10251	0.82
Halstad Gage	1,981,580	868.71	70266	869.60	81086	0.89	10820	0.88
Hendrum	2,038,359	873.49	63579	874.41	72208	0.92	8629	0.92
Perley	2,129,181	878.15	55576	878.69	61297	0.54	5720	0.59
Georgetown	2,193,941	882.05		882.61		0.56	8101	0.67
North River/ Clay Co. Hwy 93	2,305,647	893.53		888.02		-5.51		-5.58
19th Ave N Fargo/ 28th Ave N Moorhead	2,360,321	898.98		891.53		-7.45		-7.43
Fargo Gage (13th Ave S, 12th Ave S)	2,388,223	903.00 (*40.26)		893.55 (*30.81)		-9.48	-13547	-9.46
US Rose Coulee/ US 50th Ave S Moorhead	2,430,241	906.15		896.61		-9.54		-9.52
52nd Ave S Fargo/ 60th Ave S Moorhead	2,438,085	907.01		897.27		-9.74		-9.72
US ND Wild Rice River	2,484,618	910.44		901.67		-8.77	-14176	-8.77
US Diversion	2,531,338	914.89		914.90		0.01		0.02
Hickson Gage	2,563,878	917.11	24586	917.11	24527	0.00	-59	0.01

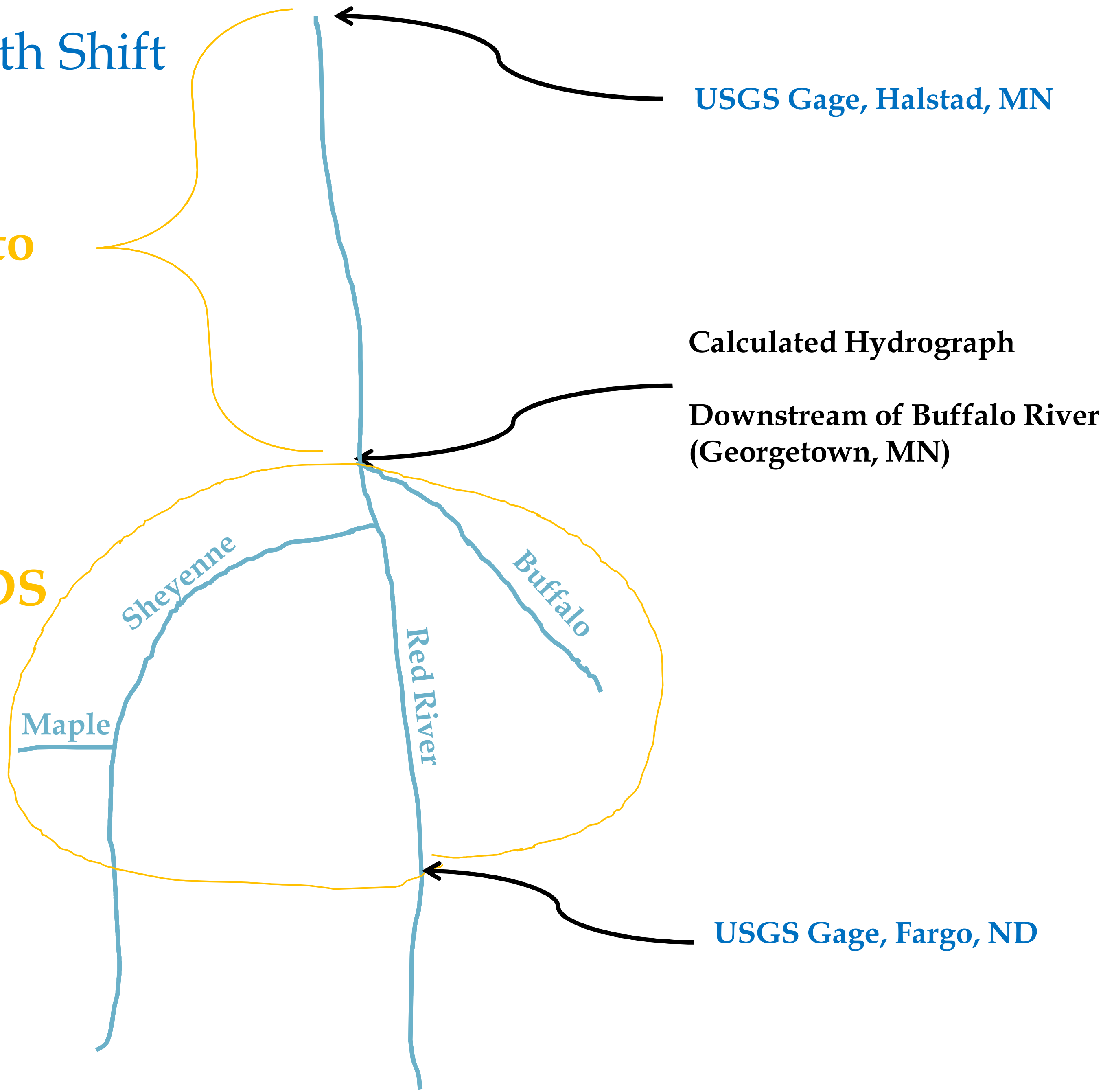
5.0 Hydrology Sensitivity



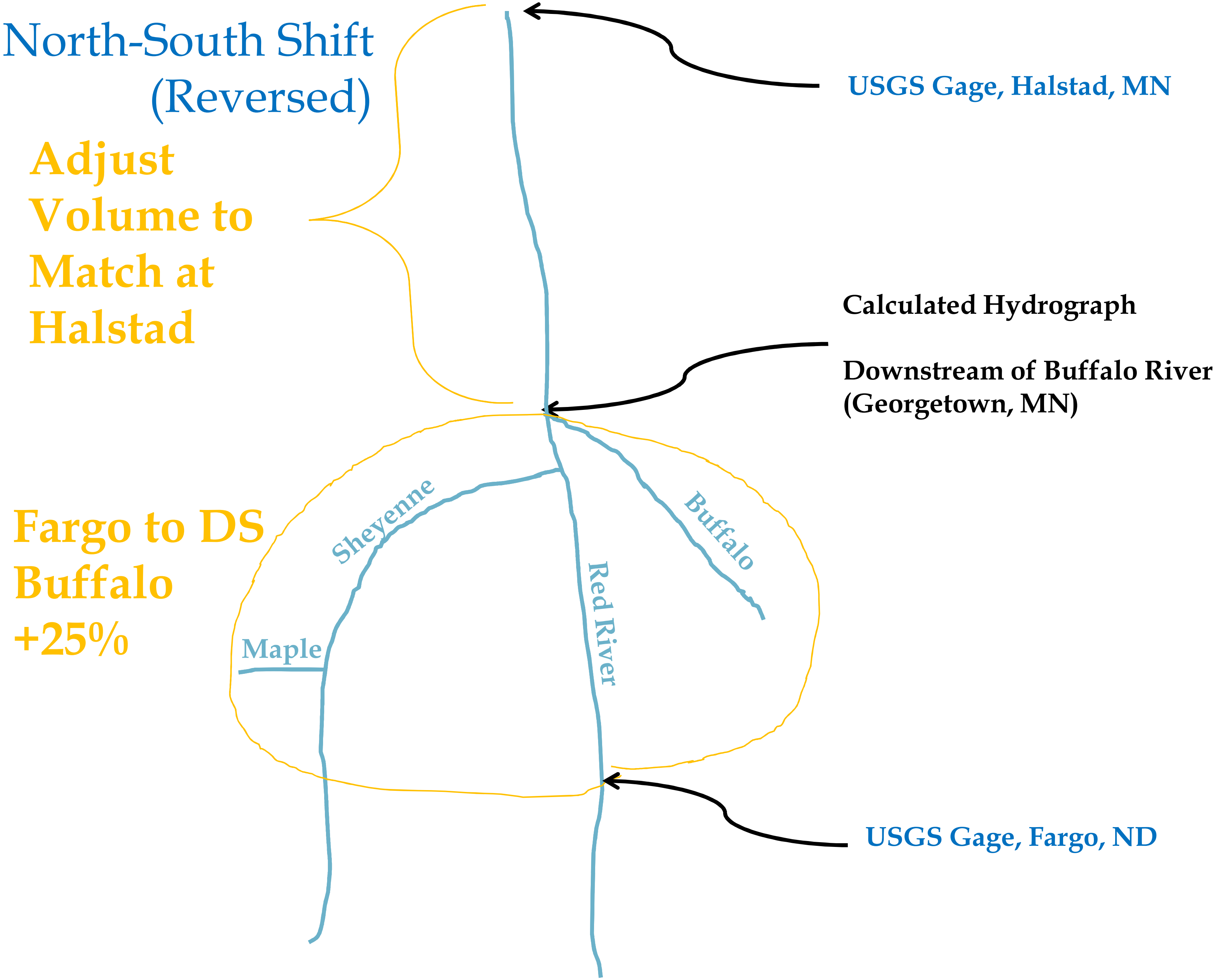
5a) North-South Shift

Adjust Volume to Match at Halstad

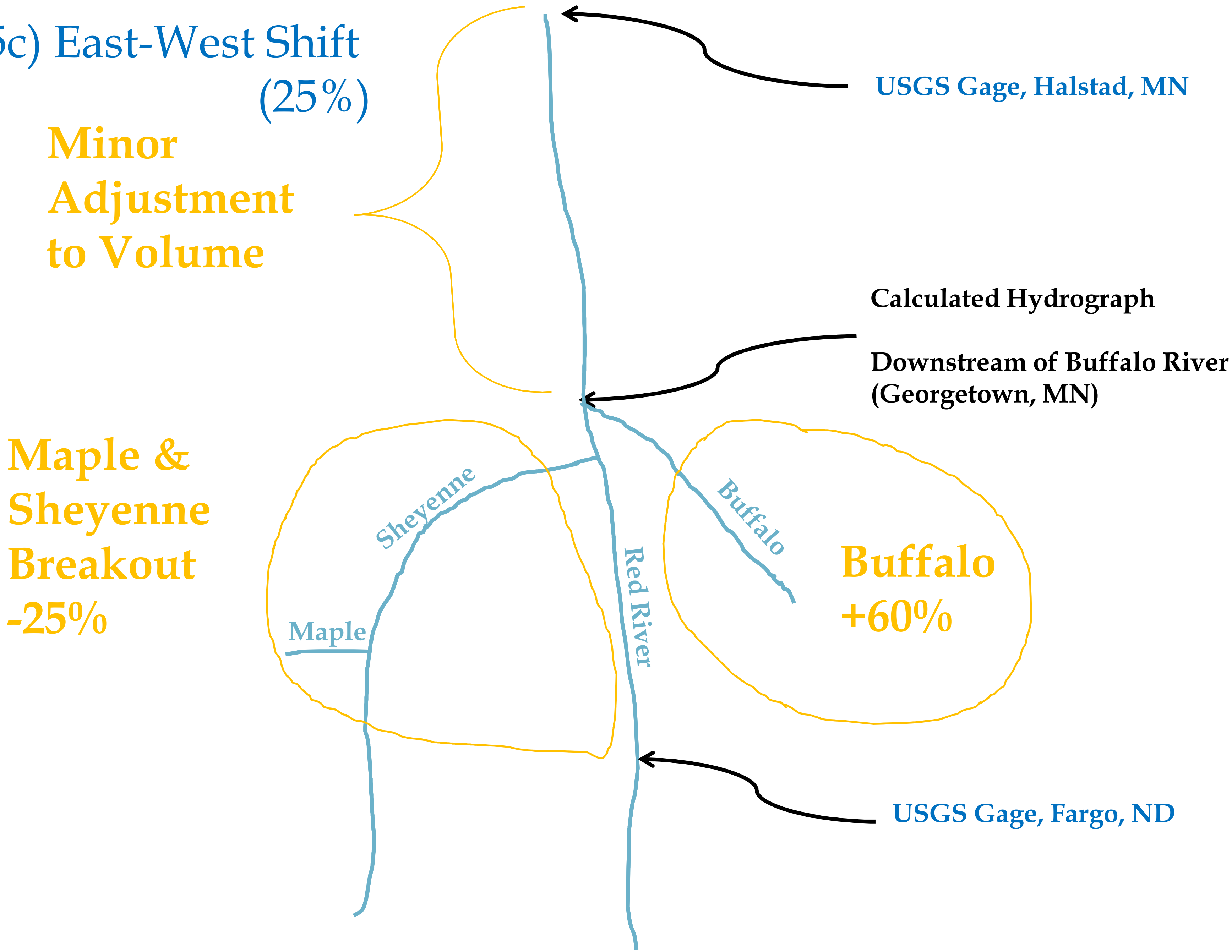
Fargo to DS Buffalo -25%



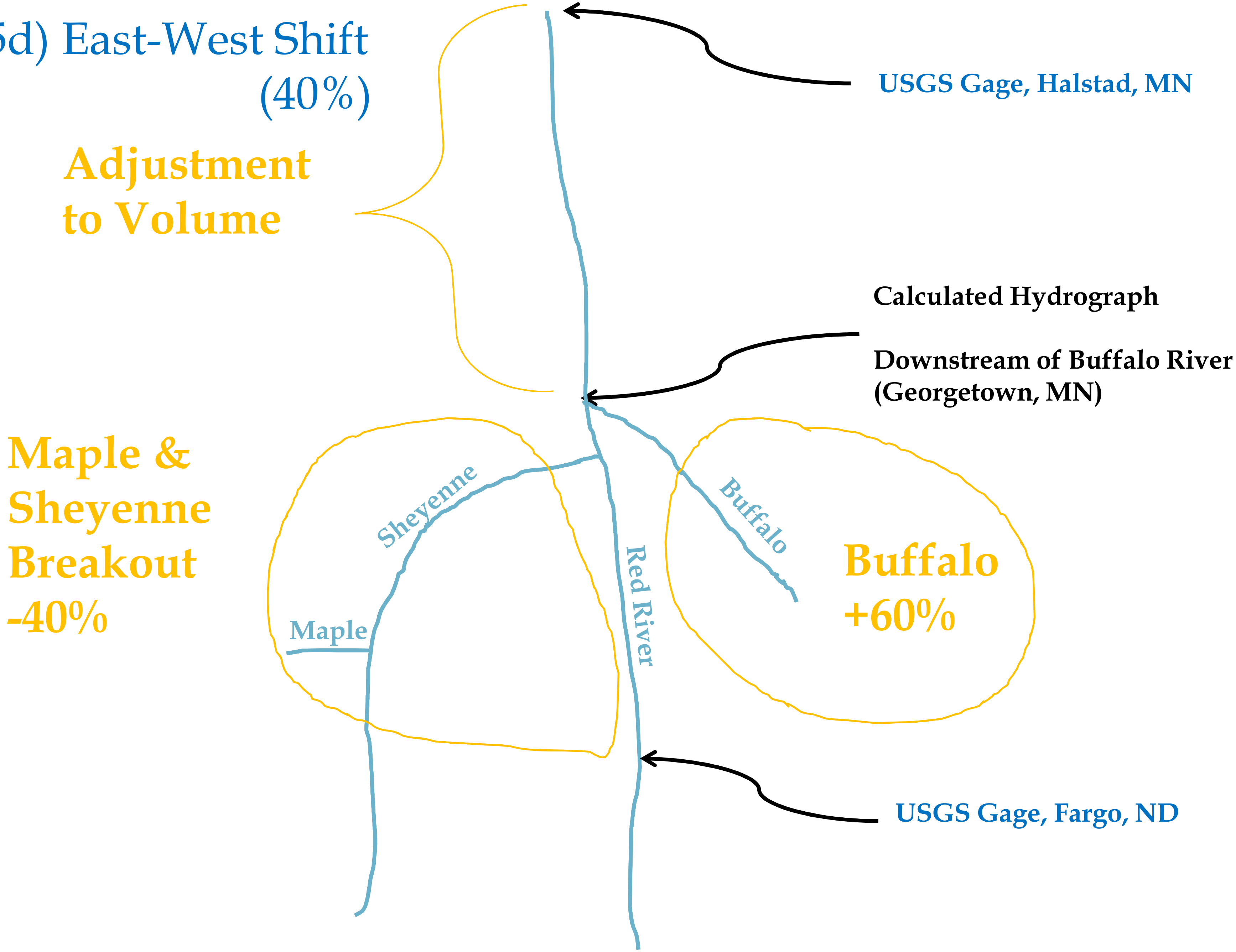
5b) North-South Shift (Reversed)



5c) East-West Shift (25%)



5d) East-West Shift (40%)



Appendix B – Hydraulics
Existing Condition

Exhibit G

Weir Coefficient Sensitivity Analysis – (Barr Engineering)

Memorandum

To: Stu Dobberpuhl, Moore Engineering; Gregg Thielman, Houston Engineering
From: Brandon Barnes
Subject: Test runs to evaluate different model representations of flow conveyance through floodplain
Date: December 17, 2010
Project: 34/09-1004.00 400 002
c: Miguel Wong, Barr Engineering; Mark Forest, HDR Engineering; Aaron Buesing, USACE

This memorandum summarizes Barr Engineering Co. (Barr) test runs to evaluate how flow is conveyed through the Red River of the North floodplain in the HEC-RAS unsteady flow model currently being developed for the U.S. Army Corps of Engineers (USACE) Fargo-Moorhead Metro Flood Risk Management Project (henceforth referred to as the Project), Feasibility Study, Phase 4. The test runs have been conducted in support of the Consulting Team's internal peer review of the HEC-RAS unsteady flow model.

Background

HEC-RAS routes flow through storage cells using the Modified Puls Method (Storage or Level-Pool), in which the continuity equation is used but not the momentum or energy equations. Essentially the inflow, outflow, and volume stored within each storage cell are balanced. The Modified Puls Method requires an empirical relationship between outflow and storage within each storage cell to calculate the resulting water surface elevation, and such relationship accounts for free and submerged flow conditions. Application of the Modified Puls Method results in (1) assuming there is a flat water surface within each storage cell, and (2) not explicitly solving for velocity within a storage cell. This method is typically used for modeling areas of dead storage in floodplains where lateral flows are minimal and the water surface has a flat (or nearly flat) profile. However, there are locations of the HEC-RAS unsteady flow model developed for the Project study area where significant (over 5,000 cfs for the 2009 flood event) flows are conveyed through storage cells. One alternative approach (to the use of storage cells only) would be to work with cross sections that extend into the floodplain, with inline and lateral structures that may attenuate or slow passage of flows through the floodplain. The test runs presented in this memorandum allow a comparison of these two and other intermediate approaches, and they are intended to provide a preliminary evaluation of the potential influence of the floodplain routing model representation on Project design, Project benefits, and ultimately the impacts on flood levels downstream of the Project.

Storage Cell Review

2.5 mile reach

An approximately 2.5 mile long reach of the Wild Rice River – North Dakota was clipped out of the existing HEC-RAS unsteady flow model. This reach of river was selected because there were no lateral inflows or breakout flows to account for during the 2009 flood event, and the majority of the floodplain is contained within the model cross sections. This reach of river has been modeled here in four different ways to evaluate the flow routing through the floodplain when using storage cells only versus other possible model representations of the floodplain:

Model Representation 1. As cross sections only – All ineffective flow areas and bridge crossings were removed, as shown in Figure 1.

Model Representation 2. As cross sections only – Ineffective flow areas and blocked obstructions were added back to the model in approximately the same locations as the October 25th version of the HEC-RAS unsteady flow model of the Project study area.

Model Representation 3. As a combination of cross sections and storage areas – The channel between the bank stations was modeled as a cross section and the overbanks were modeled as storage areas, as shown in Figure 2.

Model Representation 4. As a combination of cross sections and storage areas – The channel between the bank stations was modeled as a cross section and the overbanks were modeled as storage areas. In two intermediate locations, cross sections were extended across the entire floodplain to control flow and water surface elevation in the storage areas, as shown in Figure 3.

Each of the four models was run with three different peak flows: (1) a low flow condition where the flow is primarily contained within the banks of the channel (5,600 cfs), (2) a medium flow condition where flow starts to overtop the banks (9,800 cfs), and (3) the peak flow of the 2009 flood hydrograph (14,000 cfs).

The upstream and downstream boundary conditions were the same for all four model representations. The upstream boundary condition for each model was the inflow hydrograph. For each model representation the inflow hydrograph was placed at a cross section that extended across the entire floodplain at the upstream end of the reach. The downstream boundary condition for each model was normal depth, where HEC-RAS uses Manning's equation to calculate a stage for each computed flow. The friction slope entered as the downstream boundary condition was the approximate slope of the channel near the downstream end of the reach. (This downstream boundary condition could be changed to one based on the downstream rating curve, but the assumption made here does not invalidate the comparison of results for the four model representations.)

The Manning's n values used for all four model representations were selected based on the Manning's n values along the Wild Rice River used in the October 25, 2010 version of the HEC-RAS model. Manning's n values for the channel and overbanks were set at 0.045 and 0.08 respectively. During calibration of the HEC-RAS model to observed events the overbank Manning's values were increased to 0.09-0.13. The Manning's values were not revised for this analysis, which is intended to provide a relative comparison of different model representations of flow routing through the floodplain.

Figures 4-9 show that the change in model representation of routing through the floodplain does not have an appreciable impact on the shape of the flood hydrograph at the downstream end of this 2.5 mile reach. However, modeling the entire reach with storage cells (i.e., model representation 3 and 4) accelerates the peak flow by a couple of hours compared to the cross section models (i.e., model representation 1). This shift of a couple of hours over a 2.5 mile reach might prove to be significant over a longer reach, as changes on both magnitude and timing are important in determining impacts downstream of the Project diversion channel outlet into the Red River of the North.

Water surface profiles along the entire reach of river included in this analysis have been also compared. Water surface profiles for the models with cross sections (i.e., model representations 1 and 2) have a distinct slope, with an overall drop in water surface elevation of 1.4-1.8 ft. In comparison, the model that utilize storage areas (i.e., model representation 3) appear to convey flows more efficiently, and only has a drop in water surface elevation of 0.1-0.3 ft over the reach of river modeled. Water in the storage cell model is allowed to equalize through all of the storage cells, which translates into a relatively flat water surface throughout the entire model. Finally, cross sections were extended across the floodplain to control the flow through the overbank storage cells (i.e., model representation 4). The water surface profile resulting from this model representation has a slope in the sections modeled with cross sections, and level pools in the areas modeled with storage areas (see Figures 10-12).

16 mile reach

To check the impact of a longer reach, an approximately 16 mile reach of the Wild Rice River – North Dakota was clipped out of the existing HEC-RAS unsteady flow model. This reach of river has been modeled here in six different ways to evaluate the flow routing through the floodplain when using storage cells only versus other possible model representations of the floodplain:

Model Representation 1.	As cross sections only – All ineffective flow areas and bridge crossings were removed, as shown in Figure 13. The overbank Manning's n values were set at 0.08
Model Representation 1a.	As cross sections only – All ineffective flow areas and bridge crossings were removed, as shown in Figure 13. The overbank Manning's n values were set at 0.05.

Model Representation 2. As a combination of cross sections and storage areas – The channel between the bank stations was modeled as a cross section and the overbanks were modeled as storage areas, as shown in Figure 14. All discharge coefficients for storage areas and lateral connections were kept at the default values of 3.0 for storage connections and 2.0 for lateral structures.

Model Representation 3. As a combination of cross sections and storage areas – The channel between the bank stations was modeled as a cross section and the overbanks were modeled as storage areas – except the discharge coefficients were all reduced to 1.0 for storage area connections as well as lateral structures.

Model Representation 4. As a combination of cross sections and storage areas - The channel between the bank stations was modeled as a cross section and the overbanks were modeled as storage areas. In four intermediate locations a cross section was extended across the entire floodplain to control flow and water surface elevation in the storage areas, as shown in Figure 15. All discharge coefficients for storage area connections and lateral connections were kept at the default values of 3.0 for storage areas and 2.0 for lateral structures.

Model Representation 5. As a combination of cross sections and storage areas - The channel between the bank stations was modeled as a cross section and the overbanks were modeled as storage areas. In four intermediate locations a cross section was extended across the entire floodplain to control flow and water surface elevation in the storage areas, as shown in Figure 15. All discharge coefficients for storage area connections and lateral connections were reduced to 1.0.

Each of the six models was run with three different peak flows: (1) a low flow condition where the flow is primarily contained within the banks of the channel (5,600 cfs), (2) a medium flow condition where flow starts to overtop the banks (9,800 cfs), and (3) the peak flow of the 2009 flood hydrograph (14,000 cfs). For areas where the floodplain extended beyond the extents of the cross section or storage cells, vertical walls were put in the model to prevent breakout flows. In addition, no tributary inflows were accounted for in this analysis to simplify the modeling effort, which is intended to provide a relative comparison of different model representations of flow routing through the floodplain.

The upstream and downstream boundary conditions were the same for all six model representations. The upstream boundary condition for each model was the inflow hydrograph. For each model representation the inflow hydrograph was placed at a cross section that extended across the entire floodplain at the upstream end of the reach. The downstream boundary condition for each model was normal depth, where HEC-RAS uses Manning's equation to calculate a stage for each computed flow. The friction slope entered as the downstream boundary condition was the approximate slope of the channel near the downstream end of the reach. (This downstream boundary condition could be changed to one based on the downstream rating curve, but the assumption made here does not invalidate the comparison of results for the six model representations.)

The Manning's n values used for all six model representations were selected based on the Manning's n values along the Wild Rice River used in the October 25, 2010 version of the HEC-RAS model. Manning's n values for the channel and overbanks were set at 0.045 and 0.08 respectively. During calibration of the HEC-RAS model to observed events the overbank Manning's values were increased to 0.09-0.13. To evaluate the impact of lowering the overbank Manning's value Model Representation 1a was run with lower Manning's values. The Manning's values were not revised for this analysis to match the current calibrated HEC-RAS model, rather this analysis is intended to provide a relative comparison of different model representations of flow routing through the floodplain.

Figures 16-21 show that the change in model representation of the floodplain does have an impact on the shape of the flood hydrograph at the downstream end of this approximately 16 mile reach. Modeling the overbanks with storage cells and the HEC-RAS default discharge coefficients for the storage area connections and lateral structures (i.e., model representation 2) accelerates the peak flow by 13-19 hours compared to the cross section model (i.e., model representation 1). When the discharge coefficients are lowered for the lateral structures (from 2.0 to 1.0) and storage area connections (from 3.0 to 1.0) (i.e., model representation 3), the shift in timing is reduced to 8-11 hours. If the discharge coefficients are left at their default values, but cross sections are periodically extended across the floodplain (i.e., model representation 4), the shift in timing is 8-12 hours. Finally, if the discharge coefficients are reduced to 1.0 in combination with extending cross sections across the floodplain (i.e., model representation 5), the shift in timing is reduced to 1-6 hours. Table 1 includes a summary of how the hydrograph peak is accelerated as the methodology used to model flow through the floodplain changes.

Table 1. Acceleration in Hydrograph Peak Compared to the Cross Section Only Model

Model	14,000 cfs Peak Flow Rate	9,800 cfs Peak Flow Rate	5,600 cfs Peak Flow Rate
Model Representation 1. Cross Sections Only (Overbank Manning's values of 0.08)	-	-	-
Model Representation 1a. Cross Sections Only (Overbank Manning's values of 0.05)	5 hours	4 hours	2 hours
Model Representation 2. Storage Areas in Overbanks (default discharge coefficients for lateral structures and storage connections)	19 hours	17 hours	13 hours
Model Representation 3. Storage Areas in Overbanks (discharge coefficients for lateral structures and storage area connections of 1.0)	11 hours	11 hours	8 hours
Model Representation 4. Storage Areas in Overbanks with Cross Section Extending Across the Floodplain Approximately every 4 miles (default discharge coefficients for lateral structures and storage connections)	12 hours	11 hours	8 hours
Model Representation 5. Storage Areas in Overbanks with Cross Section Extending Across the Floodplain Approximately every 4 miles (discharge coefficients for lateral structures and storage area connections of 1.0)	1 hour	4 hours	6 hours

In general, as shown in Figures 16, 18, and 20 the change in model representation of flow routing through the floodplain results in a shift in timing for the overall hydrograph at the downstream end of the model of this 16 mile reach.

Water surface profiles along the entire reach of river included in this analysis were also compared. Water surface profiles for the cross section model (i.e., model representation 1) has a distinct slope, with an overall drop in water surface elevation of 19.4-18.5 ft. In comparison, the models that utilize storage areas appear to convey flows more efficiently, and have less of a drop in water surface elevation over the reach modeled (approximately 1.0-2.5 ft less). Table 2 summarizes these results.

Table 2. Drop in Water Surface Elevation through 16 Mile Reach

Model	14,000 cfs Peak Flow Rate	9,800 cfs Peak Flow Rate	5,600 cfs Peak Flow Rate
Model Representation 1. Cross Sections Only (Overbank Manning's values of 0.08)	18.5 ft	19.1 ft	19.4 ft
Model Representation 1a. Cross Sections Only (Overbank Manning's values of 0.05)	18.7 ft	19.3 ft	19.5
Model Representation 2. Storage Areas in Overbanks (default discharge coefficients for lateral structures and storage connections)	16.3 ft	17.4 ft	18.3 ft
Model Representation 3. Storage Areas in Overbanks (discharge coefficients for lateral structures and storage connections of 1.0)	18.1 ft	18.7 ft	19.0 ft
Model Representation 4. Storage Areas in Overbanks with Cross Section Extending Across the Floodplain Approximately every 4 miles	16.7 ft	17.7 ft	18.6 ft
Model Representation 5. Storage Areas in Overbanks with Cross Section Extending Across the Floodplain Approximately every 4 miles (discharge coefficients for lateral structures and storage area connections of 1.0)	18.2 ft	18.9 ft	19.1 ft

Water in the storage cell models is equalized through adjacent storage cells resulting in a flatter water surface throughout the entire model, as shown in Figures 17, 19, and 21. This result could explain why during the calibration effort of the HEC-RAS unsteady flow model for the Project study area, modeled elevations are lower than observed elevations for some locations.

Summary

With respect to the HEC-RAS unsteady flow model for the Project study area, the results of the test runs presented here could suggest that in locations where large amounts of flow are being conveyed through storage cells (e.g., south of the Maple River, Drain 40 east of the Sheyenne River, in the overbanks of the Red River of the North of the confluence with the Buffalo) the modeled water surface profiles might be too flat. More importantly, the model representation of the floodplain in terms of storage cells only, with

HEC-RAS default values used for the discharge coefficients of the storage area connections and lateral structures, could be resulting in flood flows being routed downstream more efficiently (hydraulically speaking) than warranted. The resulting shift in timing could have an impact on evaluation of downstream impacts, even though both Existing Conditions and With-Project unsteady HEC-RAS models use similar methodologies to represent how flow is conveyed through the floodplain.

The test runs presented are set up to demonstrate sensitivity to how the floodplain is modeled. Model sensitivity for the Project unsteady model may vary as the size and location of storage areas vary compared both to the test runs completed and throughout the unsteady model developed for the Project. None of the test runs presented includes obstructions in the floodplain such as road crossings which may alter the sensitivity of the conveyance capacity in the floodplain when comparing different modeling methodologies.

One potential way to model how quickly water is conveyed through the storage cells could be to periodically extend a cross section across the entire floodplain. However, this may not be practical throughout the domain of the HEC-RAS unsteady flow model developed for the Project study area. Another way to potentially slow down flow through the floodplain is to use lower (than the HEC-RAS default values) discharge coefficients for the storage area connections and lateral structures. For the 16 mile reach, a discharge coefficient of less than 1.0 could be required for storage area connections and lateral structures if the goal is to slow down the hydrograph so that model results are comparable to the same reach modeled with only cross sections.

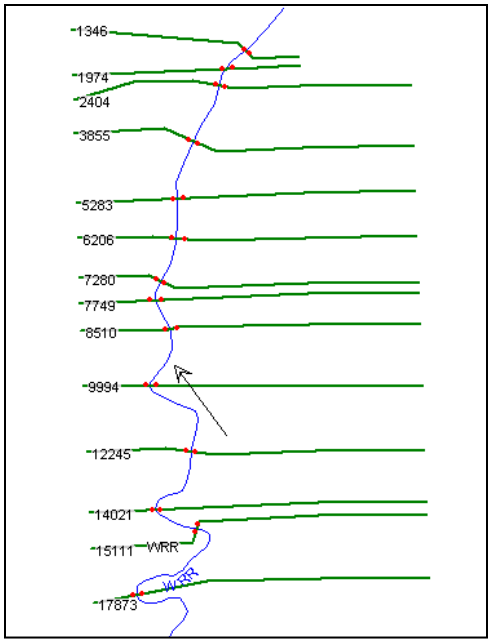


Figure 1. Section of Wild Rice River (2.5 mile) – North Dakota modeled with cross sections only. Geometry for Model Representation 1 and 2

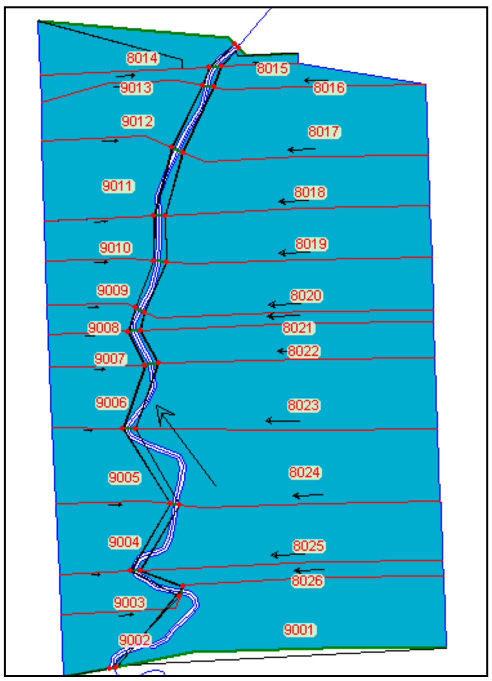


Figure 2. Section of Wild Rice River (2.5 mile) – North Dakota modeled with cross sections between the bank stations and storage areas for the overbanks. The geometry for each storage area connection is taken from the cross section geometry used in the first model. Geometry for Model Representation 3.

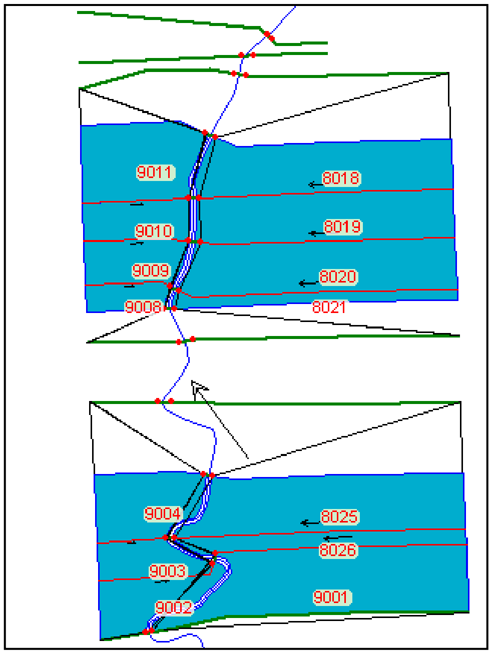


Figure 3. Section of Wild Rice River (2.5 mile) – North Dakota modeled with cross sections between the bank stations, storage areas for the overbanks, and two locations where cross sections extend across the floodplain. The geometry for each storage area connection is taken from the cross section geometry used in the first model. Geometry for Model Representation 4.

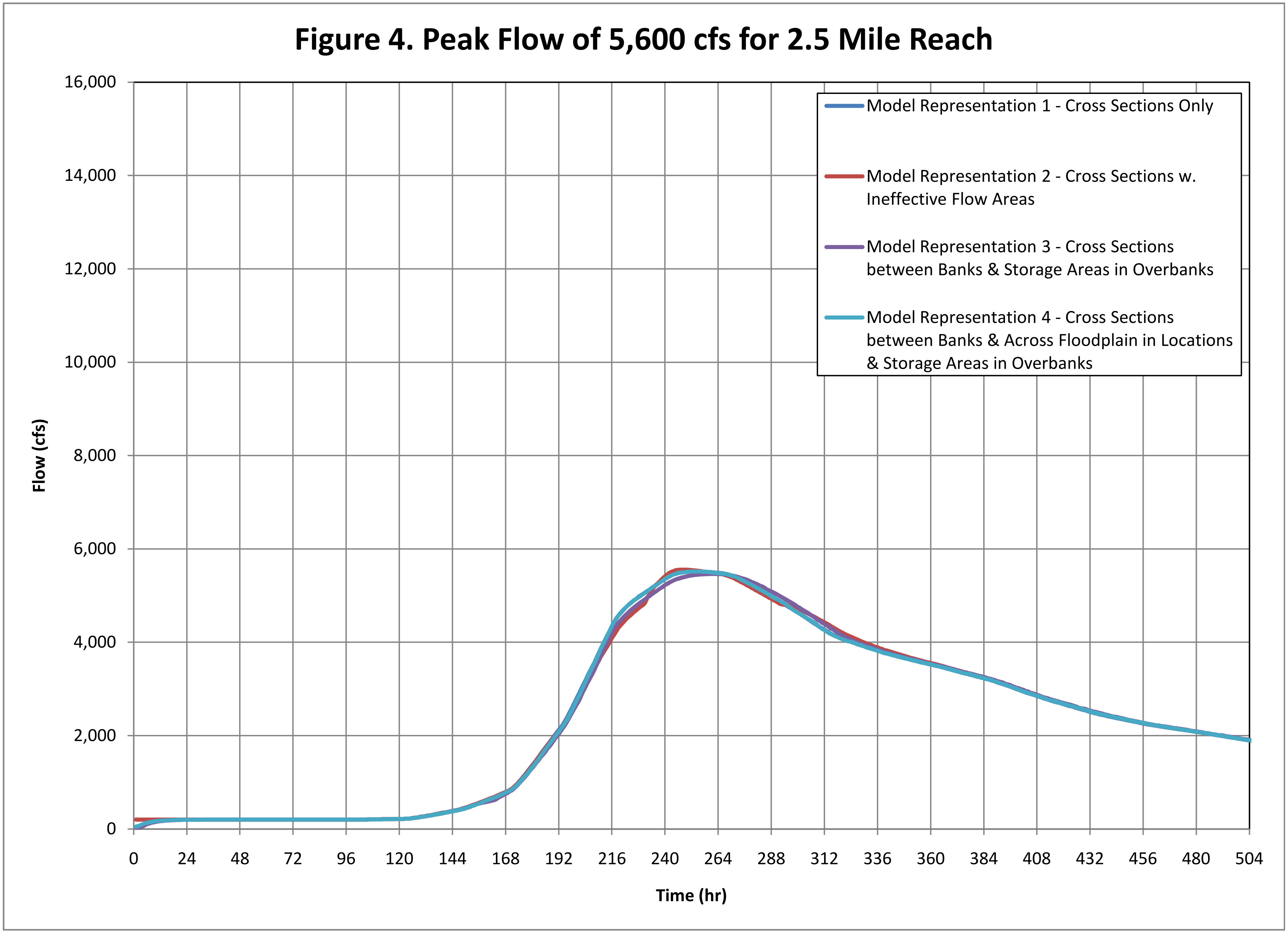


Figure 4. Peak Flow of 5,600 cfs for 2.5 Mile Reach
Flow (cfs)
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360
384
408
432
456
480
504
Time (hr)
Model Representation 1 - Cross Sections Only
Model Representation 2 - Cross Sections w. Ineffective Flow Areas
Model Representation 3 - Cross Sections between Banks & Storage Areas in Overbanks
Model Representation 4 - Cross Sections between Banks & Across Floodplain in Locations & Storage Areas in Overbanks

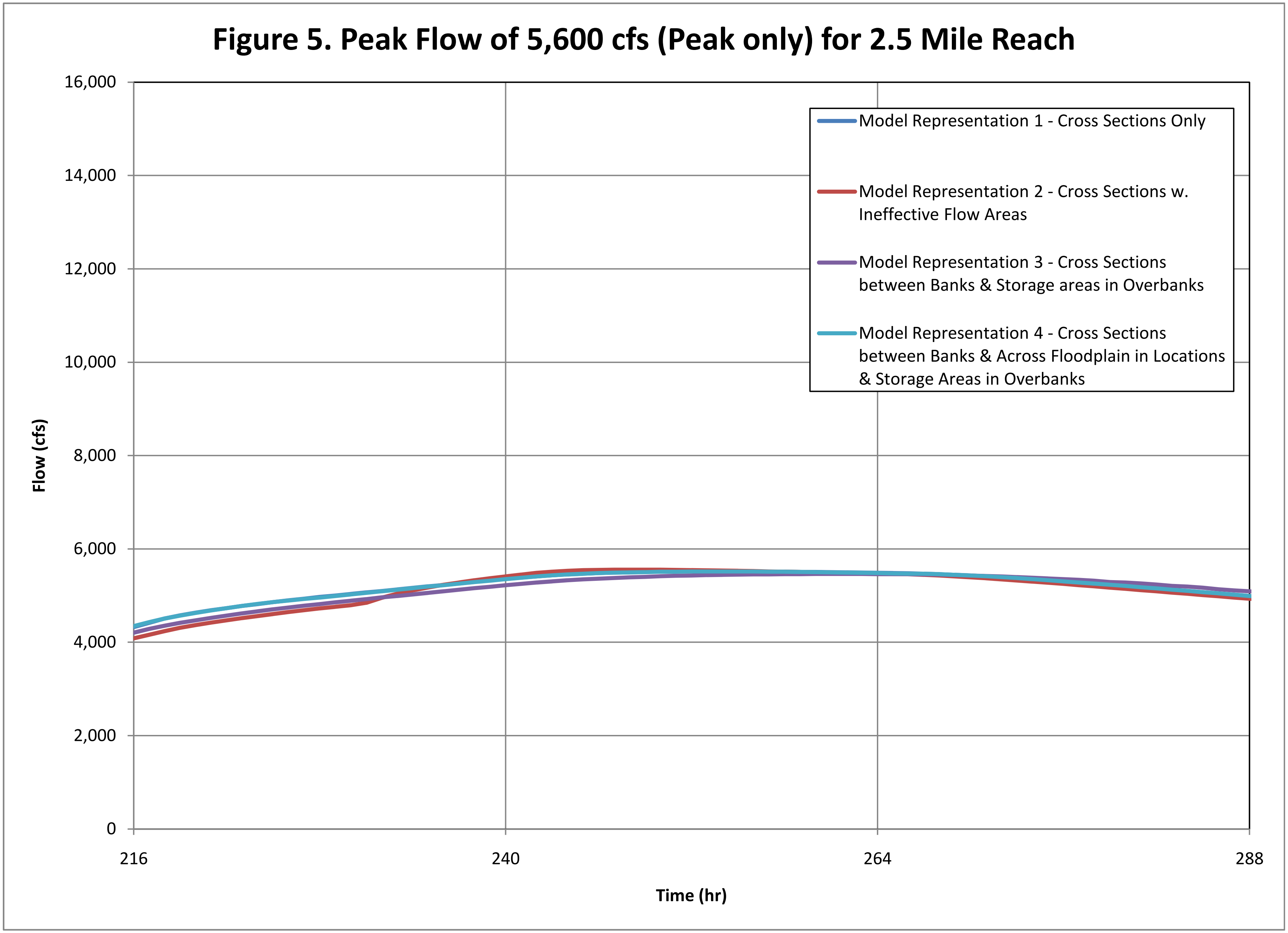


Figure 5. Peak Flow of 5,600 cfs (Peak only) for 2.5 Mile Reach
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
Flow (cfs)
216
240
264
288
Time (hr)
Model Representation 1 - Cross Sections Only
Model Representation 2 - Cross Sections w. Ineffective Flow Areas
Model Representation 3 - Cross Sections between Banks & Storage areas in Overbanks
Model Representation 4 - Cross Sections between Banks & Across Floodplain in Locations & Storage Areas in Overbanks

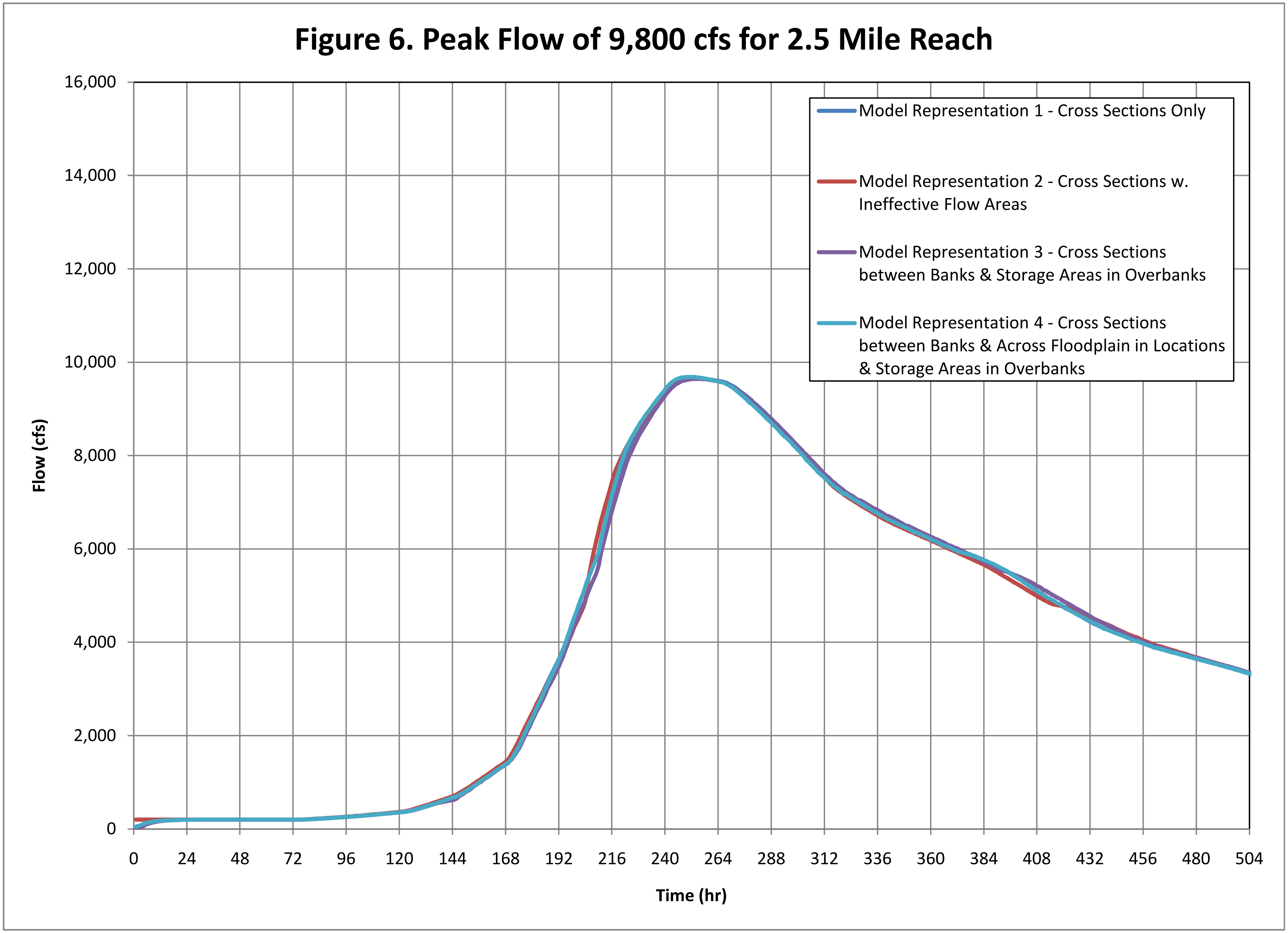


Figure 6. Peak Flow of 9,800 cfs for 2.5 Mile Reach
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
Flow (cfs)
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360
384
408
432
456
480
504
Time (hr)
Model Representation 1 - Cross Sections Only
Model Representation 2 - Cross Sections w. Ineffective Flow Areas
Model Representation 3 - Cross Sections between Banks & Storage Areas in Overbanks
Model Representation 4 - Cross Sections between Banks & Across Floodplain in Locations & Storage Areas in Overbanks

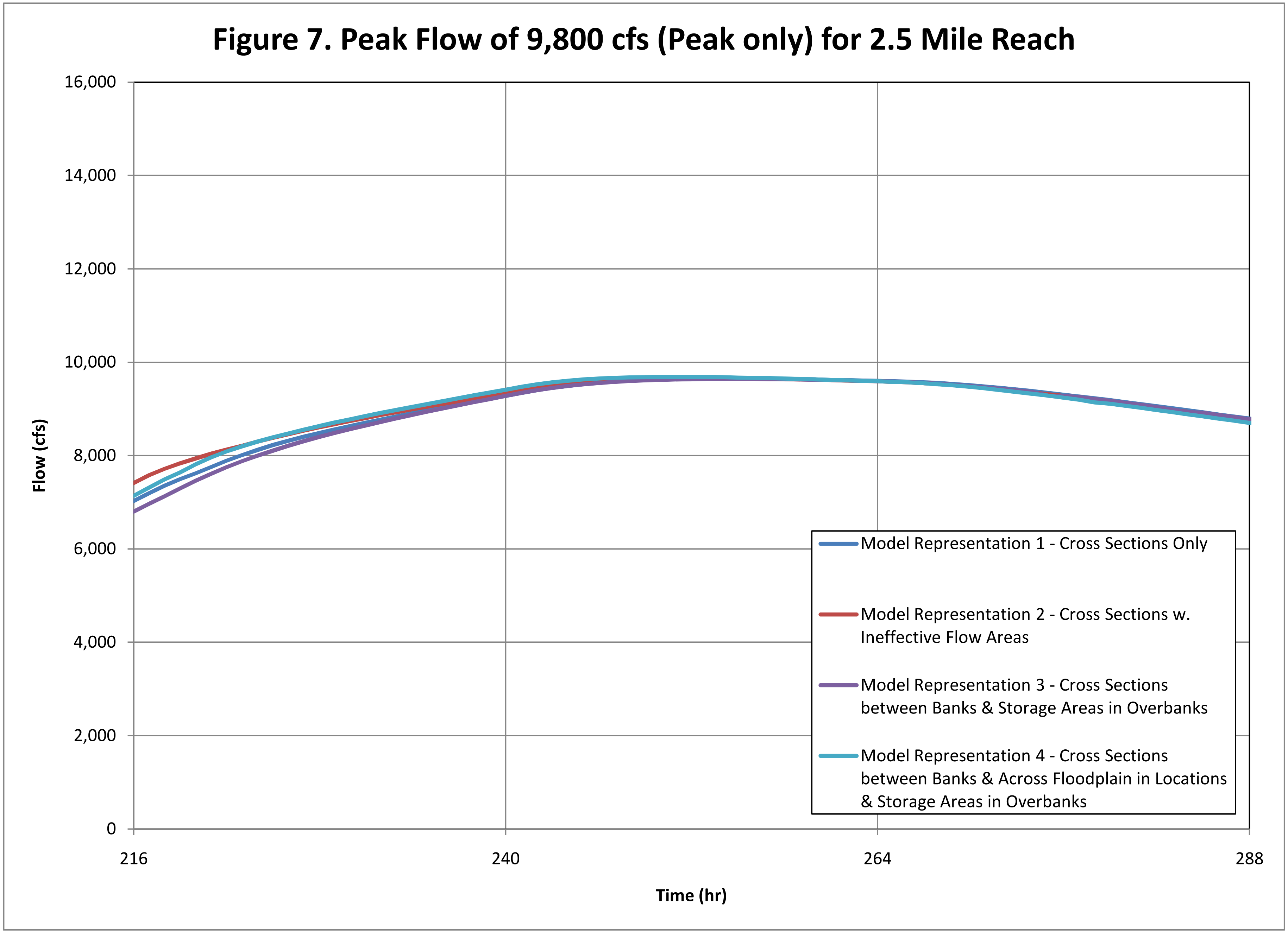


Figure 7. Peak Flow of 9,800 cfs (Peak only) for 2.5 Mile Reach
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
Flow (cfs)
216
240
264
288
Time (hr)
Model Representation 1 - Cross Sections Only
Model Representation 2 - Cross Sections w. Ineffective Flow Areas
Model Representation 3 - Cross Sections between Banks & Storage Areas in Overbanks
Model Representation 4 - Cross Sections between Banks & Across Floodplain in Locations & Storage Areas in Overbanks

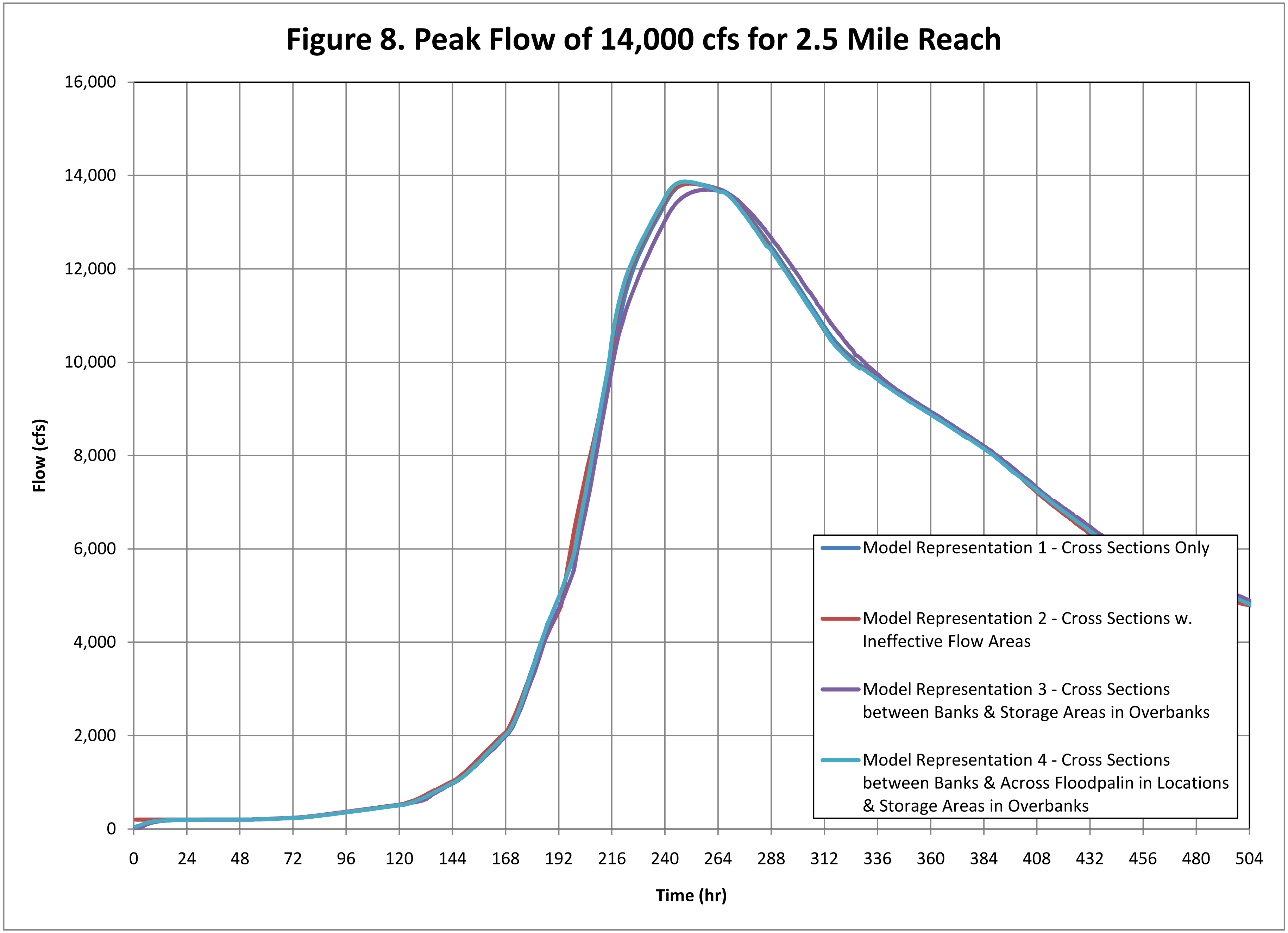


Figure 8. Peak Flow of 14,000 cfs for 2.5 Mile Reach
Flow (cfs)
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360
384
408
432
456
480
504
Time (hr)
Model Representation 1 - Cross Sections Only
Model Representation 2 - Cross Sections w. Ineffective Flow Areas
Model Representation 3 - Cross Sections between Banks & Storage Areas in Overbanks
Model Representation 4 - Cross Sections between Banks & Across Floodpalin in Locations & Storage Areas in Overbanks

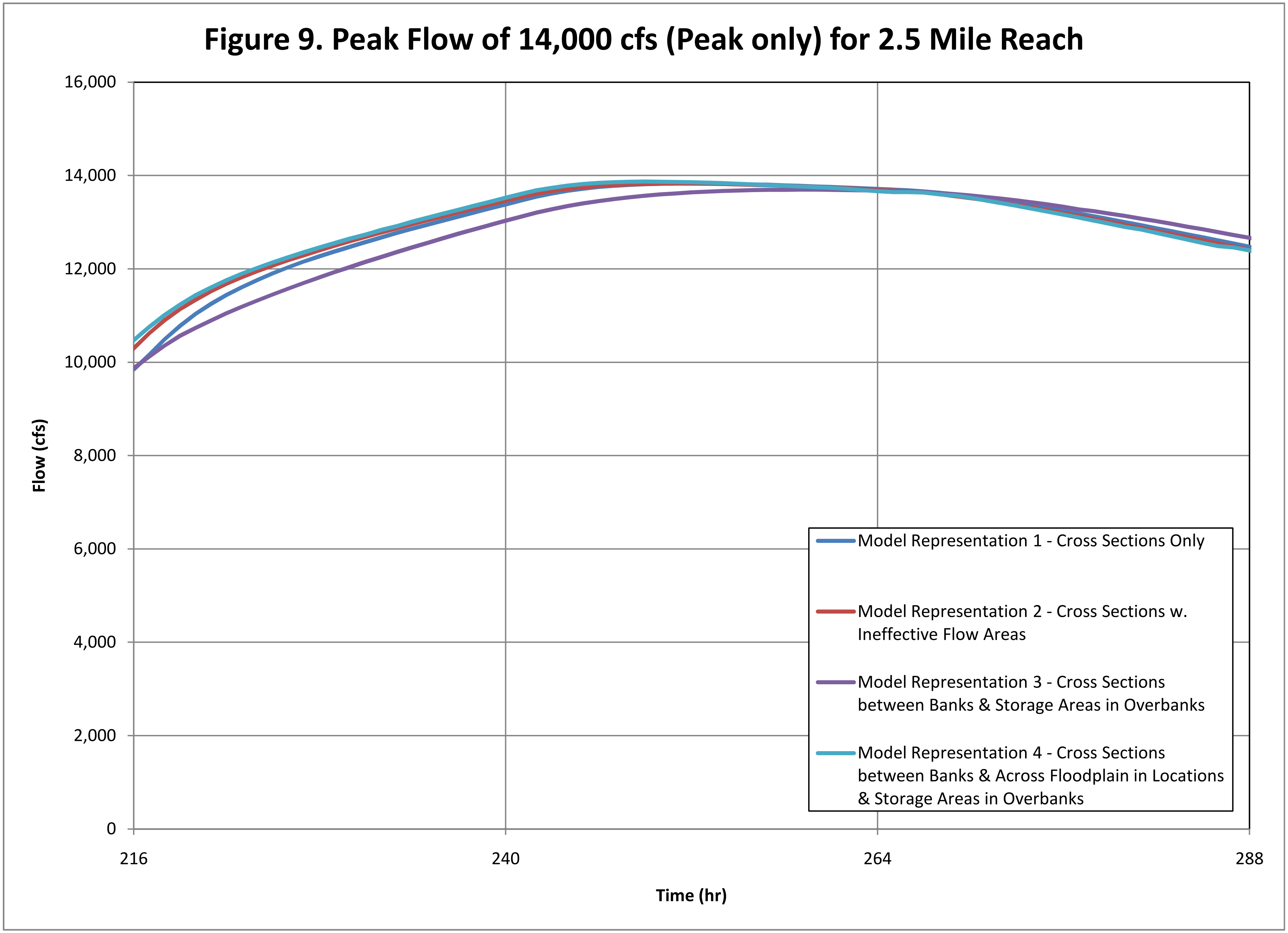


Figure 9. Peak Flow of 14,000 cfs (Peak only) for 2.5 Mile Reach
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
Flow (cfs)
216
240
264
288
Time (hr)
Model Representation 1 - Cross Sections Only
Model Representation 2 - Cross Sections w. Ineffective Flow Areas
Model Representation 3 - Cross Sections between Banks & Storage Areas in Overbanks
Model Representation 4 - Cross Sections between Banks & Across Floodplain in Locations & Storage Areas in Overbanks

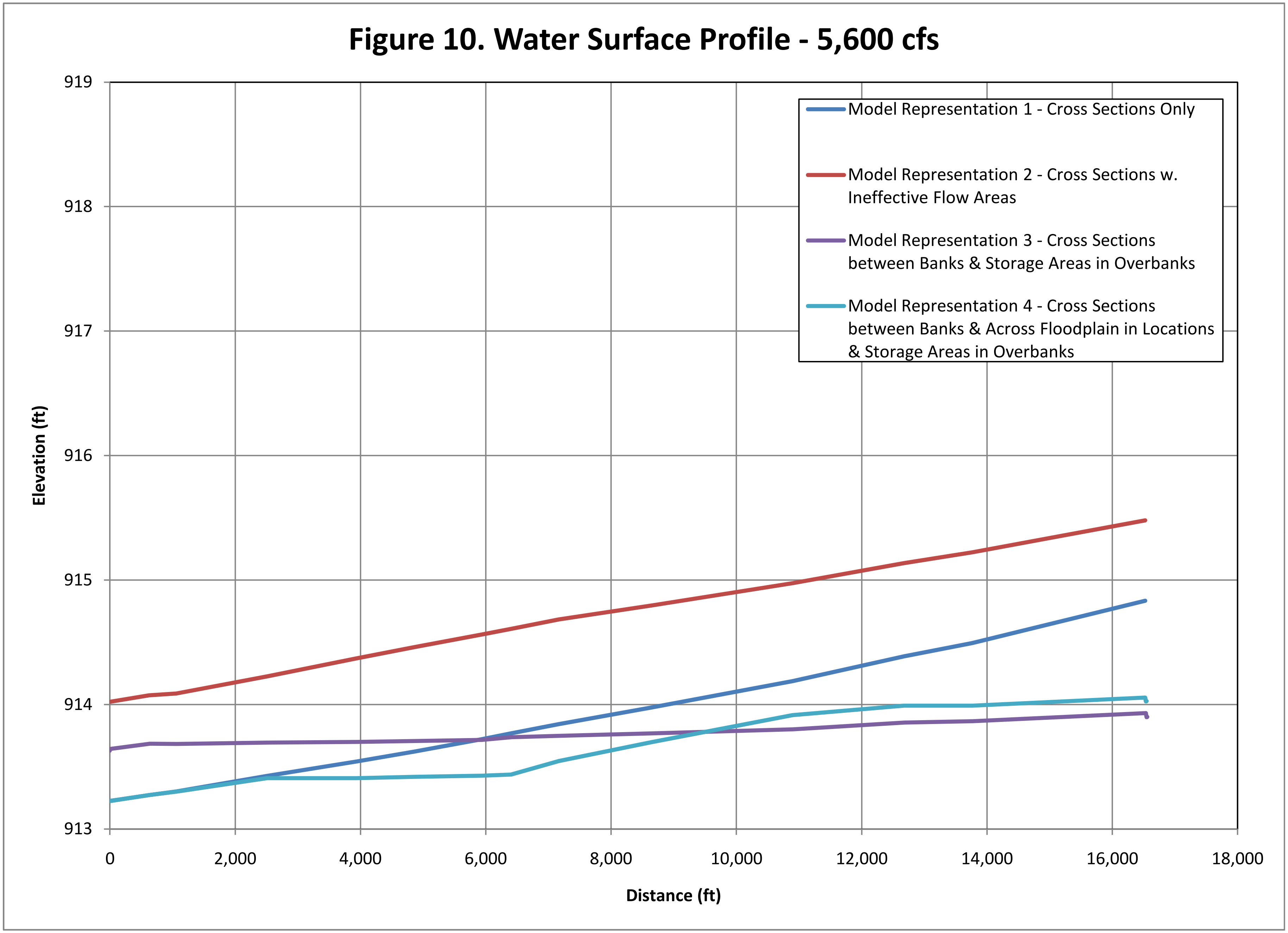


Figure 10. Water Surface Profile - 5,600 cfs
919
918
917
916
915
914
913
Elevation (ft)
0
2,000
4,000
6,000
8,000
10,000
12,000
14,000
16,000
18,000
Distance (ft)
Model Representation 1 - Cross Sections Only
Model Representation 2 - Cross Sections w. Ineffective Flow Areas
Model Representation 3 - Cross Sections between Banks & Storage Areas in Overbanks
Model Representation 4 - Cross Sections between Banks & Across Floodplain in Locations & Storage Areas in Overbanks

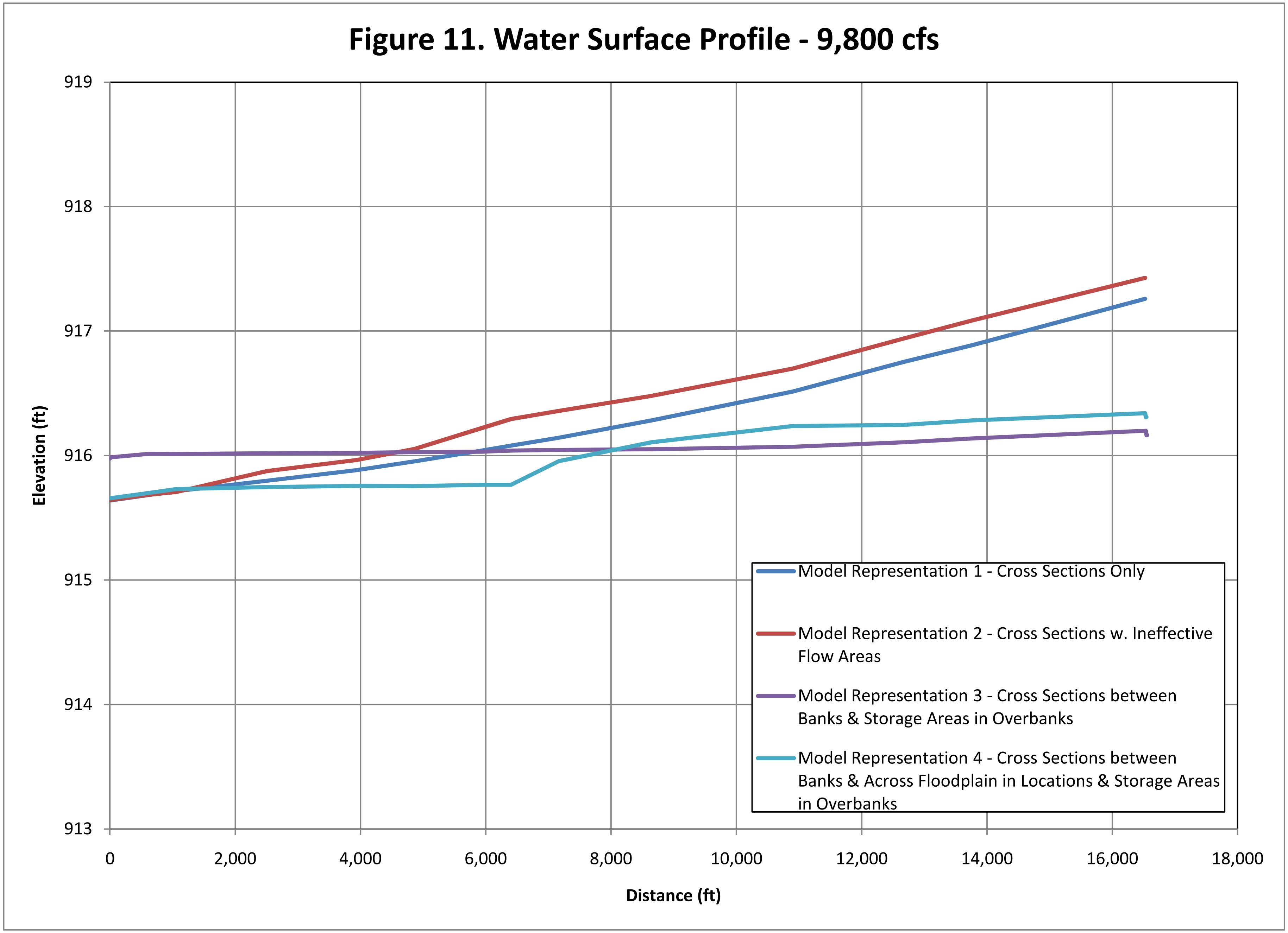
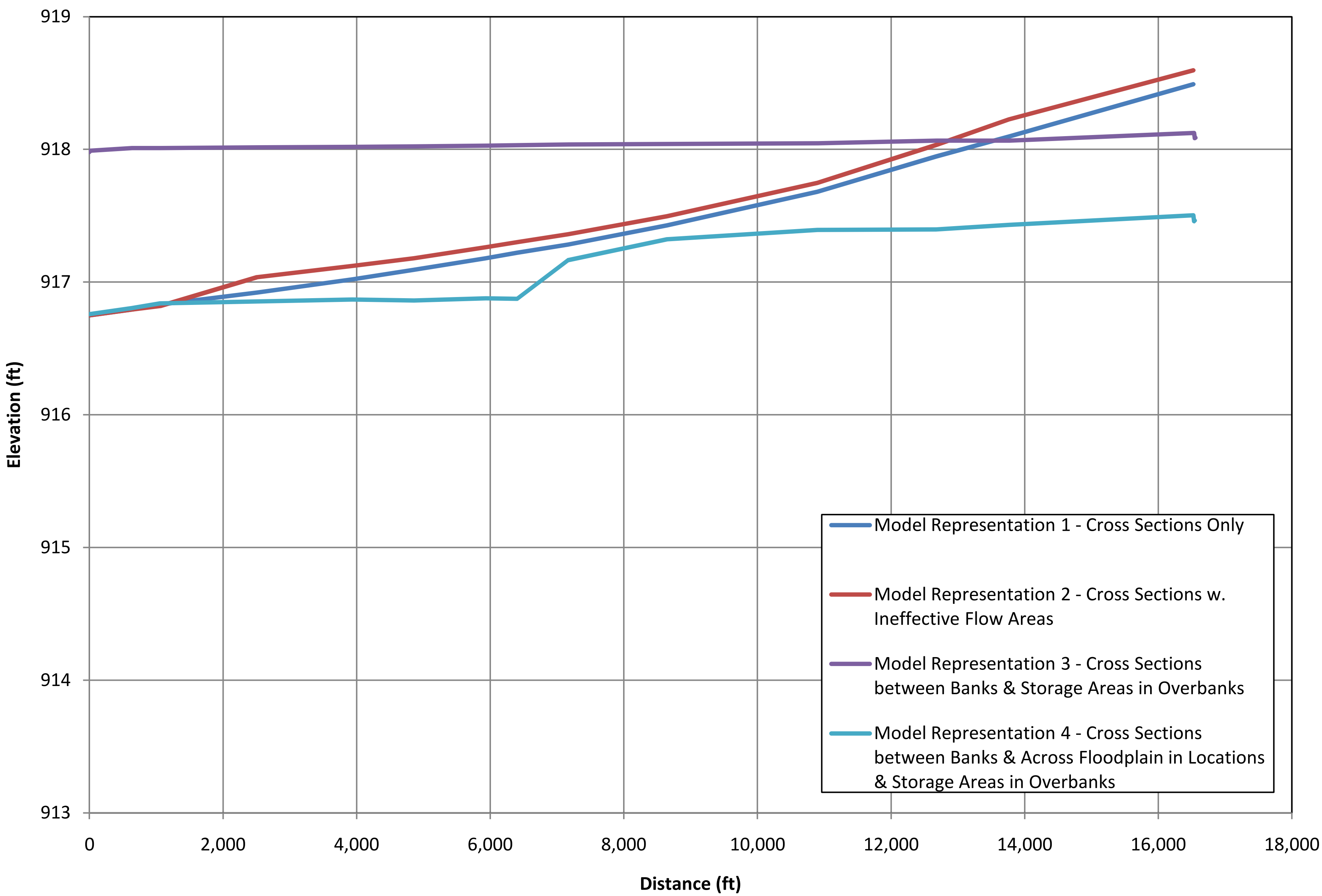


Figure 11. Water Surface Profile - 9,800 cfs
Elevation (ft)
919
918
917
916
915
914
913
0
2,000
4,000
6,000
8,000
10,000
12,000
14,000
16,000
18,000
Distance (ft)
Model Representation 1 - Cross Sections Only
Model Representation 2 - Cross Sections w. Ineffective Flow Areas
Model Representation 3 - Cross Sections between Banks & Storage Areas in Overbanks
Model Representation 4 - Cross Sections between Banks & Across Floodplain in Locations & Storage Areas in Overbanks

Figure 12. Water Surface Profile - 14,000 cfs



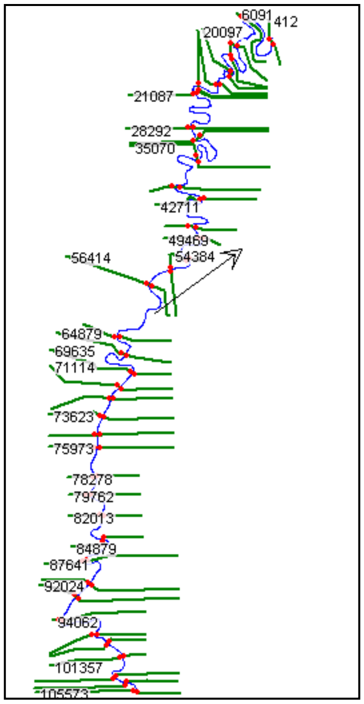


Figure 13. Section of Wild Rice River (16 mile) – North Dakota modeled with cross sections only. Geometry for Model Representation 1 and 1a.

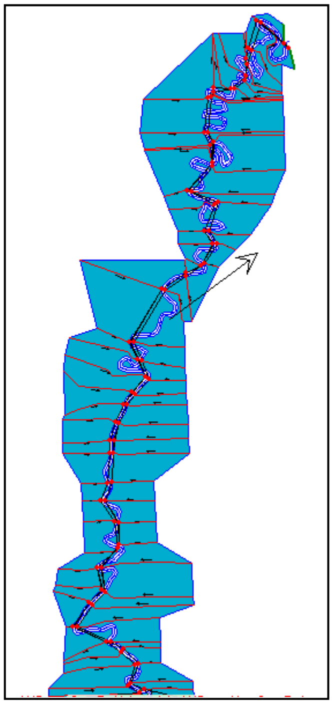


Figure 14. Section of Wild Rice River (16 mile) – North Dakota modeled with cross sections for the bank stations and storage areas in the overbanks. Geometry for Model Representation 2 and 3.

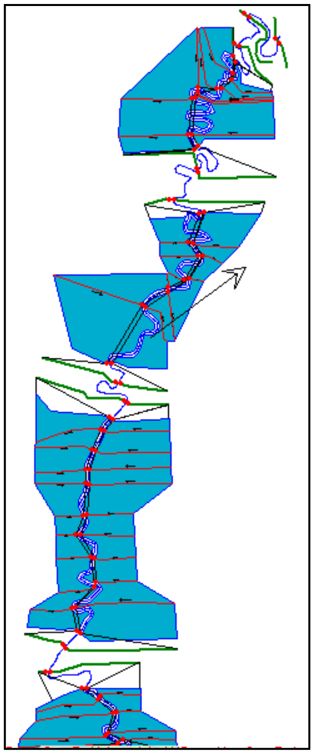


Figure 15. Section of Wild Rice River (16 mile) – North Dakota modeled with cross sections between the bank stations, storage areas for the overbanks, and four locations where cross sections extend across the floodplain. The geometry for each storage area connection is taken from the cross section geometry used in the first model. Geometry for Model Representation 4 and 5.

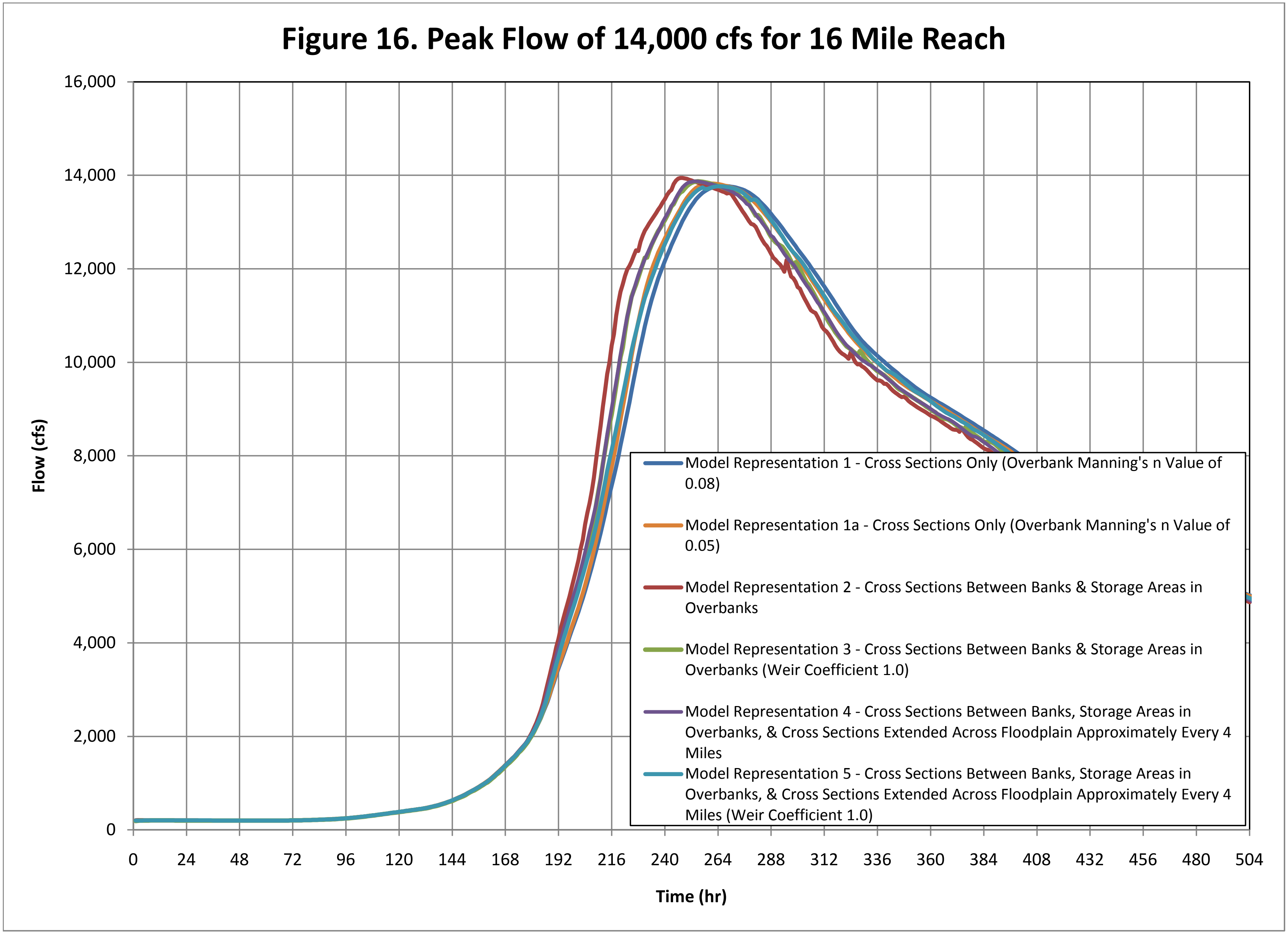
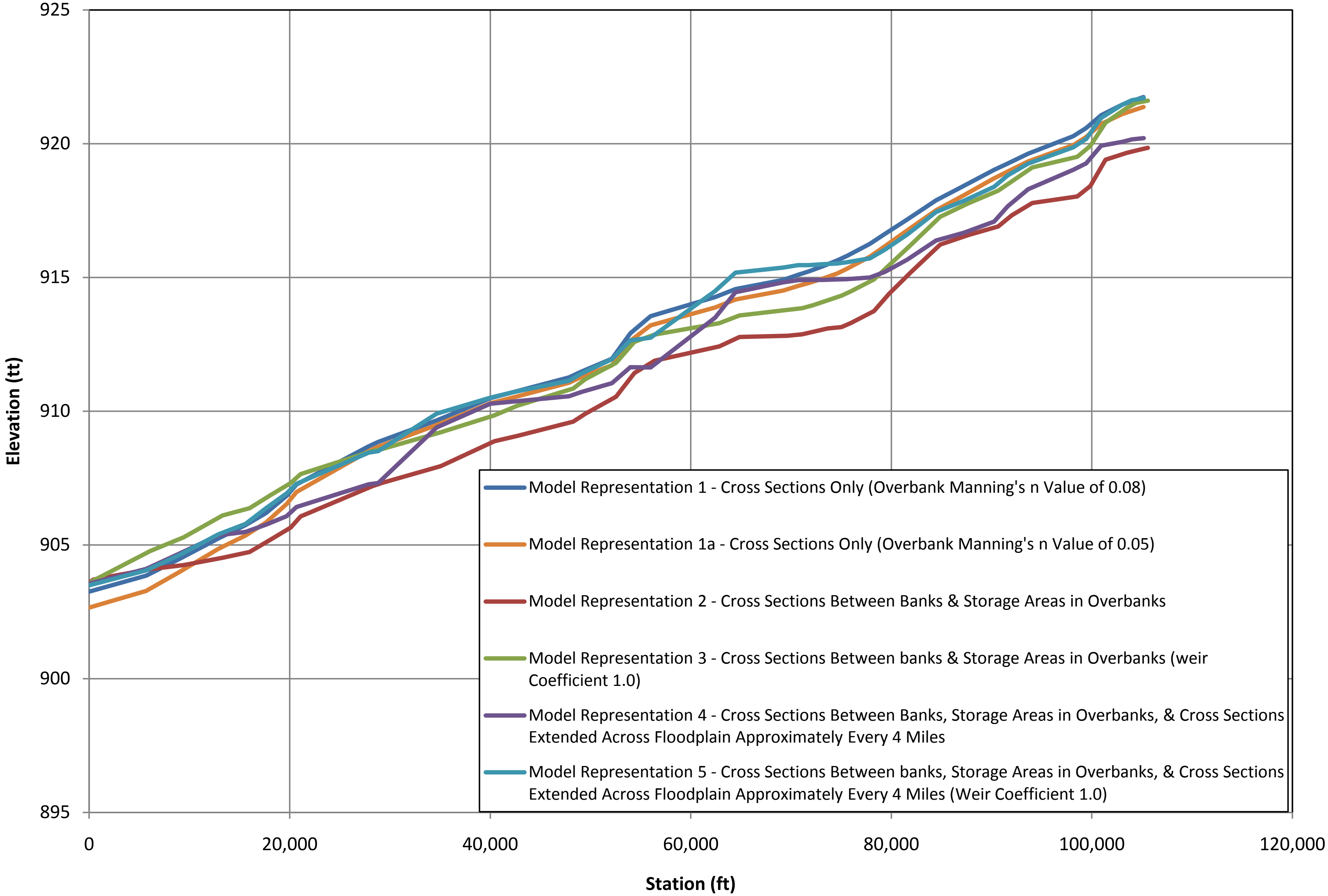


Figure 16. Peak Flow of 14,000 cfs for 16 Mile Reach
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
Flow (cfs)
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360
384
408
432
456
480
504
Time (hr)
Model Representation 1 - Cross Sections Only (Overbank Manning's n Value of 0.08)
Model Representation 1a - Cross Sections Only (Overbank Manning's n Value of 0.05)
Model Representation 2 - Cross Sections Between Banks & Storage Areas in Overbanks
Model Representation 3 - Cross Sections Between Banks & Storage Areas in Overbanks (Weir Coefficient 1.0)
Model Representation 4 - Cross Sections Between Banks, Storage Areas in Overbanks, & Cross Sections Extended Across Floodplain Approximately Every 4 Miles
Model Representation 5 - Cross Sections Between Banks, Storage Areas in Overbanks, & Cross Sections Extended Across Floodplain Approximately Every 4 Miles (Weir Coefficient 1.0)

Figure 17. Water Surface Profile for 16 mile section of River- 14,000 cfs



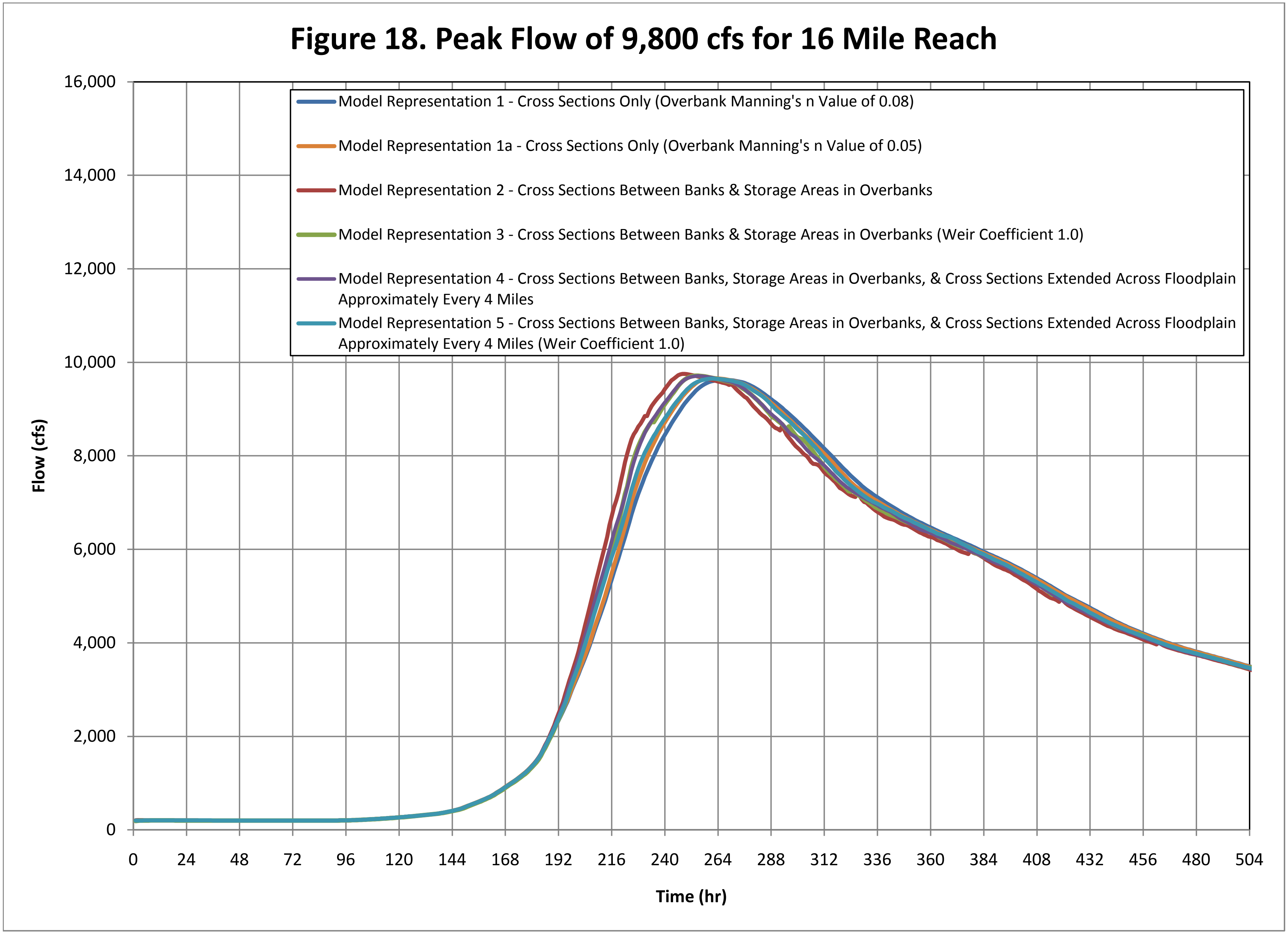


Figure 18. Peak Flow of 9,800 cfs for 16 Mile Reach
Model Representation 1 - Cross Sections Only (Overbank Manning's n Value of 0.08)
Model Representation 1a - Cross Sections Only (Overbank Manning's n Value of 0.05)
Model Representation 2 - Cross Sections Between Banks & Storage Areas in Overbanks
Model Representation 3 - Cross Sections Between Banks & Storage Areas in Overbanks (Weir Coefficient 1.0)
Model Representation 4 - Cross Sections Between Banks, Storage Areas in Overbanks, & Cross Sections Extended Across Floodplain Approximately Every 4 Miles
Model Representation 5 - Cross Sections Between Banks, Storage Areas in Overbanks, & Cross Sections Extended Across Floodplain Approximately Every 4 Miles (Weir Coefficient 1.0)
Flow (cfs)
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360
384
408
432
456
480
504
Time (hr)

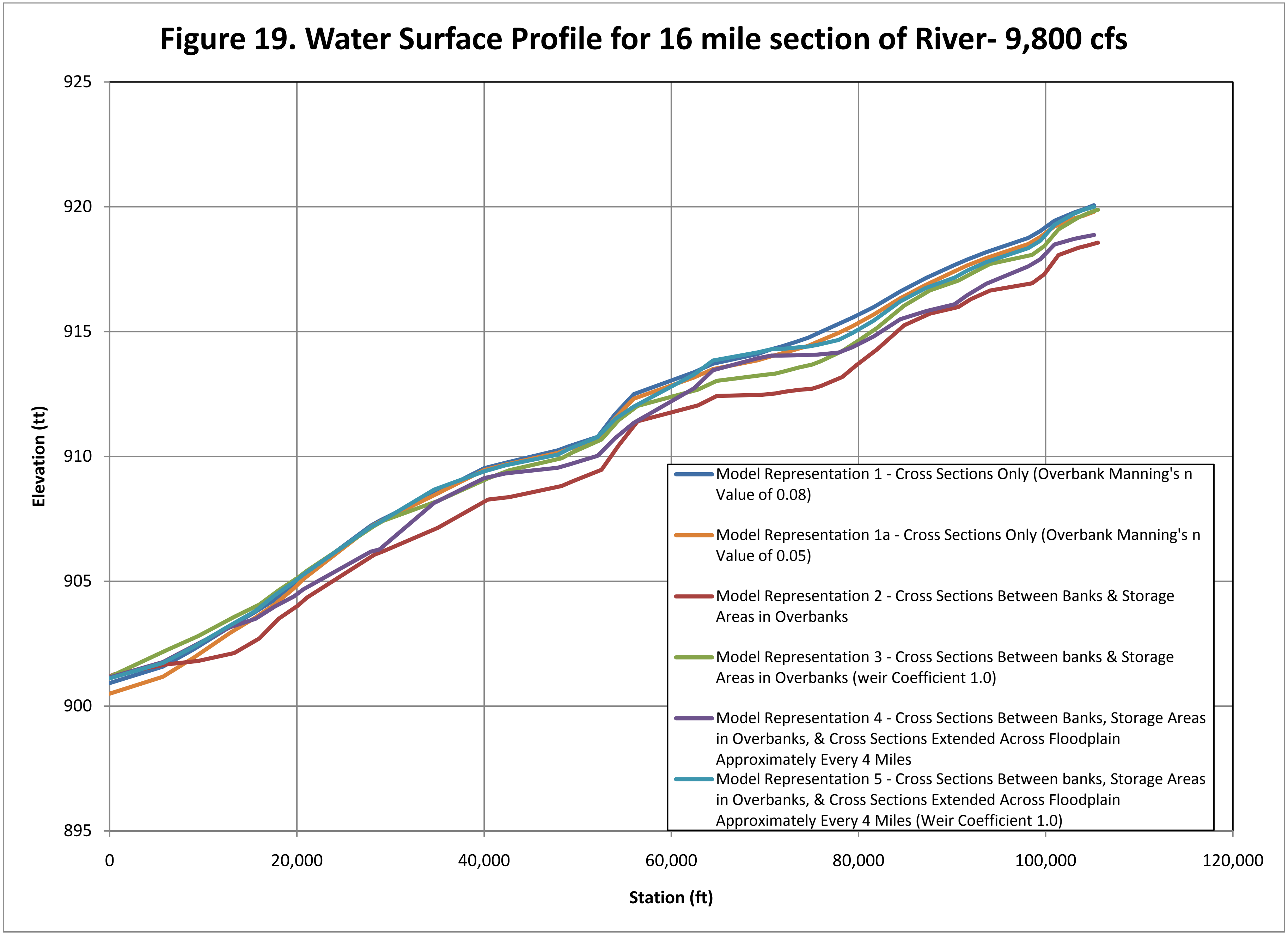


Figure 19. Water Surface Profile for 16 mile section of River- 9,800 cfs
925
920
915
910
905
900
895
Elevation (tt)
0
20,000
40,000
60,000
80,000
100,000
120,000
Station (ft)
Model Representation 1 - Cross Sections Only (Overbank Manning's n Value of 0.08)
Model Representation 1a - Cross Sections Only (Overbank Manning's n Value of 0.05)
Model Representation 2 - Cross Sections Between Banks & Storage Areas in Overbanks
Model Representation 3 - Cross Sections Between banks & Storage Areas in Overbanks (weir Coefficient 1.0)
Model Representation 4 - Cross Sections Between Banks, Storage Areas in Overbanks, & Cross Sections Extended Across Floodplain Approximately Every 4 Miles
Model Representation 5 - Cross Sections Between banks, Storage Areas in Overbanks, & Cross Sections Extended Across Floodplain Approximately Every 4 Miles (Weir Coefficient 1.0)

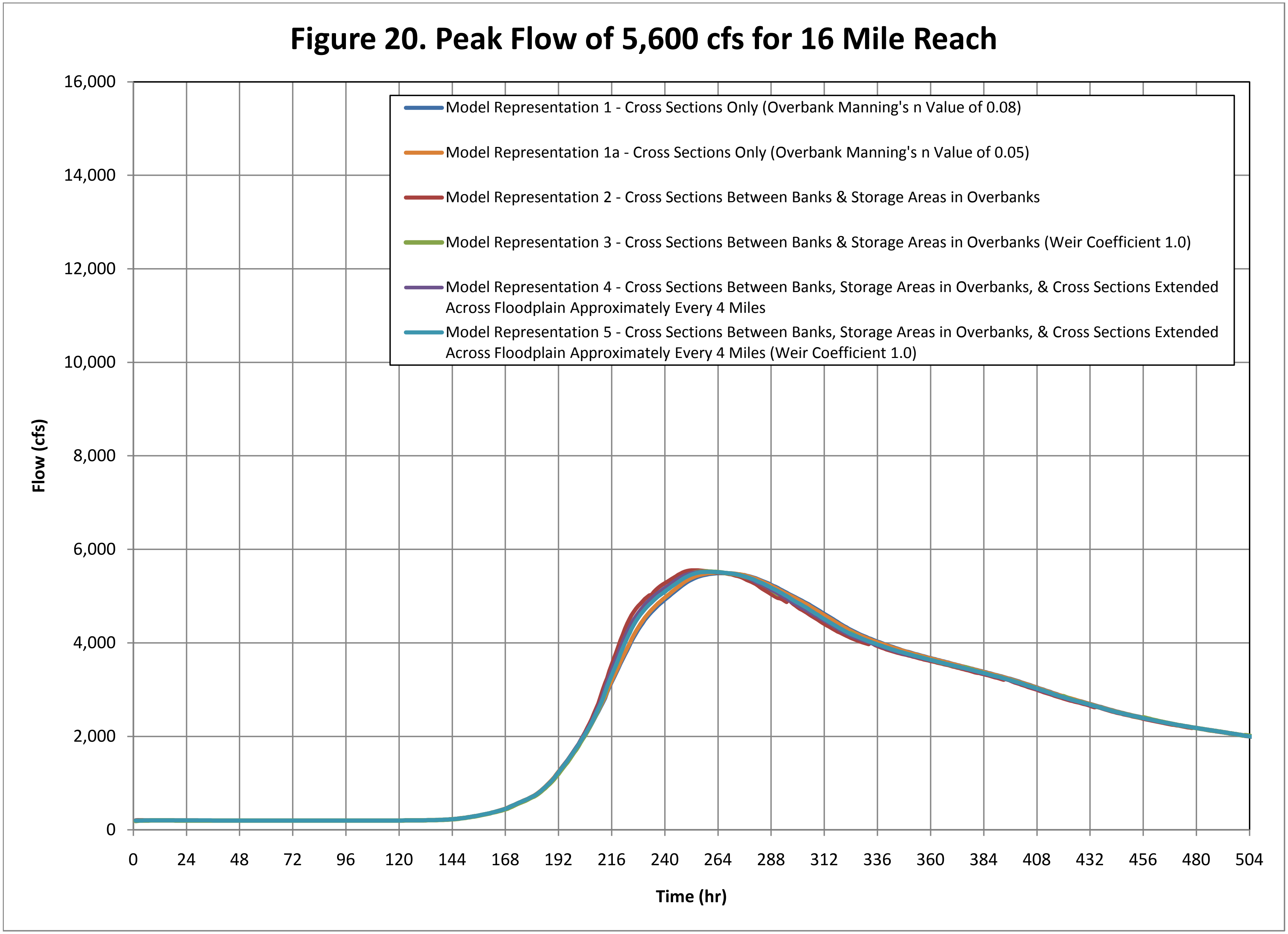


Figure 20. Peak Flow of 5,600 cfs for 16 Mile Reach
Model Representation 1 - Cross Sections Only (Overbank Manning's n Value of 0.08)
Model Representation 1a - Cross Sections Only (Overbank Manning's n Value of 0.05)
Model Representation 2 - Cross Sections Between Banks & Storage Areas in Overbanks
Model Representation 3 - Cross Sections Between Banks & Storage Areas in Overbanks (Weir Coefficient 1.0)
Model Representation 4 - Cross Sections Between Banks, Storage Areas in Overbanks, & Cross Sections Extended Across Floodplain Approximately Every 4 Miles
Model Representation 5 - Cross Sections Between Banks, Storage Areas in Overbanks, & Cross Sections Extended Across Floodplain Approximately Every 4 Miles (Weir Coefficient 1.0)
Flow (cfs)
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
0
24
48
72
96
120
144
168
192
216
240
264
288
312
336
360
384
408
432
456
480
504
Time (hr)

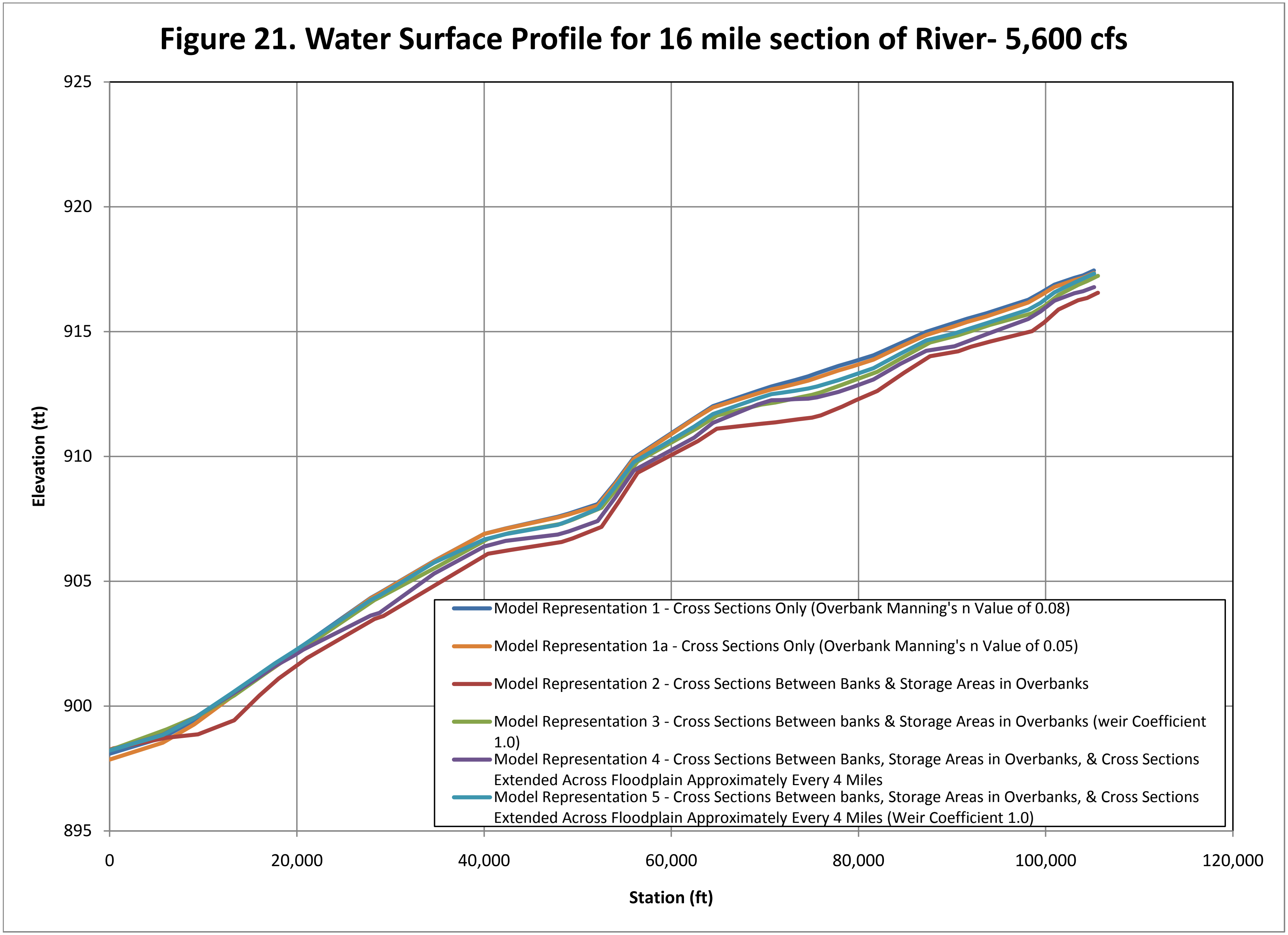


Figure 21. Water Surface Profile for 16 mile section of River- 5,600 cfs
925
920
915
910
905
900
895
Elevation (tt)
0
20,000
40,000
60,000
80,000
100,000
120,000
Station (ft)
Model Representation 1 - Cross Sections Only (Overbank Manning's n Value of 0.08)
Model Representation 1a - Cross Sections Only (Overbank Manning's n Value of 0.05)
Model Representation 2 - Cross Sections Between Banks & Storage Areas in Overbanks
Model Representation 3 - Cross Sections Between banks & Storage Areas in Overbanks (weir Coefficient 1.0)
Model Representation 4 - Cross Sections Between Banks, Storage Areas in Overbanks, & Cross Sections Extended Across Floodplain Approximately Every 4 Miles
Model Representation 5 - Cross Sections Between banks, Storage Areas in Overbanks, & Cross Sections Extended Across Floodplain Approximately Every 4 Miles (Weir Coefficient 1.0)

Appendix B – Hydraulics
Existing Condition

Exhibit H

Model Peer Review and Model QA/QC Measures

RED RIVER DIVERSION

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

APPENDIX B – HYDRAULICS EXISTING CONDITIONS EXHIBIT H – MODEL PEER REVIEW AND MODEL QA/QC MEASURES

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

By: HOUSTON ENGINEERING, INC.

FINAL: February 28, 2011

TABLE OF CONTENTS

1. GENERAL

Developing a model as large and complex as the one created for this project it is of the best interest of the project team, sponsors and US Army Corps of Engineers to have reviews completed to help build the best model and end product. Several internal reviews as well as external an agency reviews were conducted. Many of the comments have been addressed with responses during closure of previous phases of this project. Three reviews were conducted as part of Phase 4. One preliminary internal to the project team review was completed by Moore Engineering, Inc. Another detailed technical review was conducted by Barr Engineering. The third review was a broad, general review completed by HDR. The following sections outline our response to the review comments. Copies of the actual comments are also included with this Exhibit.

2. REVIEWS

2.1 Review 1. Preliminary Internal Review by Moore Engineering, Inc.

The comments provided by Moore Engineering, Inc. were provided while the 2009 Calibration model was being developed. Since the comments were provided early in the model development process, many of them have been incorporated into the final models submitted in Phase 4.

Responses

1. **Cross Section Spacing** – The model geometry was obtained from previous models. Additional cross sections had not yet been added. In response to this comment, additional cross sections were added to the Hickson to Thompson reach of the Red River. Cross sections were added to the extent possible while maintaining perpendicular flow paths through a large, meandering channel and broad floodplain. Additional cross sections should still be added to the Thompson through Drayton reach of the Red River.
2. **2009 Calibration for Next Phase of Modeling** – Matching stage at the Fargo gage and through the protected area was rather challenging. The modeled water surface elevations were generally lower than observed data. Many parameters were adjusted through internal sensitivity analyses in attempt of increasing stage. The final outcome was to maintain a channel "n" value of 0.045. One possible contribution to the higher stages at the lower discharges is the uncertainty of the discharge measurements. Often times field measurements are not available prior to the flood wave progressing downstream. Discharges were lowered in upstream hydrographs to help this situation. The stage and calibration through the Fargo/Moorhead area will continually be addressed in future models.
3. **Weighted Manning's "n" Values** – Generally, the unsteady state model has manning's "n" values on the upper end of accepted values. The particular cross sections referred to in this comment had horizontally varying "n" value errors. They have since been fixed.
4. **XS Delta WS Values** – See response to comment 1. Cross sections have been added.

5. **Manning's "n" Values** – See response to comment 3. Horizontally varied "n" values were fixed.
6. **Channel Bank Locations** – Considerable review and sensitivity iterations were conducted to best simplify the complex flow paths of the Red River into a 1-D model. The bank stations define where overbank flow is compared to channel flow. This defines conveyance areas as well as flow path reach lengths. Aerial photos were used for the 2009 flood to best estimate conveyance area and overbank vs. channel reach lengths.
7. **Correction Hydrograph** – In that version of the calibration, the model matched the hydrograph at Halstad, MN. Routing this hydrograph downstream provided excessive discharges at Thompson without accounting for any local inflows. A conversation with the USGS supplied additional information on what actually occurred during the flood. The field measurements were rated poor and we were advised by the USGS to ignore the values at Halstad and use Thompson instead. The correction hydrograph was removed and the model results at Halstad were not considered for comparison.
8. **Junctions** – The downstream reach lengths were set at zero in case this may double count volumes.
9. **Lateral Weir Profiles** – Moore Engineering provided new storage area connections based on the interstate ramps and Jersey barriers.
10. **Lateral Structure (LS 1658177)** – Additional cross sections were added as part of comment 1 that also addresses this comment.
11. **Flow Roughness Factors** – The flow roughness factors were applied in an attempt to lower the stages at lower flows. This seemed to be addressed as the base flow discharges were lowered. It still may be a manning's "n" value issue to be addressed with comment 2 in the future.
12. **Sheyenne River Breakout** – The reviewed version of the model had very limited detail in this area. Since then, additional detail has been added, however, details as specific as this are difficult to address with a model as complex and as large as this. Stages in the Sheyenne River Area are difficult to adequately match, especially with the quality of the observed data.
13. See Response to 12.
14. The control structure was later added with plan data.
15. The observed hydrograph was moved to the correct location.
16. This hydrograph calibration at this location has since been discontinued. Instead calibration was maintained at the USGS gages, Fargo and Halstad.

2.2 Review 2. Internal Review by Barr Engineering

The comments provided by Barr Engineering were completed after the preliminary 2009 Calibration model was developed. This was intended to be a detailed review of the model geometry, flow files and results. Many of the comments addressed details in non-detailed reaches in the model which do not impact to the objective of the model.

Responses

1. Version 4.1

2. NAVD 88
3. It is in the Phase 4 submittal
4. NAD 83, UTM 14N FOOT_US
5. Feet from stream mouth
6. Agreed.
7. Agreed.
8. No.
9. Cross sections overlap where tributaries meet with the Red River. The Red River cross sections have been created such that their conveyance is consistant through the junction. The tributary cross sections are in place to route the lower tributary flows to the Red River. Additional volume storage caused by overlapping cross sections was assumed to be minimal.
10. Agreed.
11. Agreed.
12. Agreed.
13. Yes. However, a shorter run time produces output with less iteration.
14. Agreed.
15. Model iterates significantly with tighter tolerances.
16. Agreed.
17. Agreed.
18. Agreed.
19. Agreed.
20. Rating Curve
21. Yes, See Exhibit E. Cross section hydrograph output cannot be compared to balanced hydrograph in model. The cross section only represents a portion of the flow. Depending on the location, additional conveyance is added in storage area connections.
22. Agreed.
23. No.
24. In the model that was reviewed, the Wolverton local inflow hydrograph was placed at the upper boundary condition of the stream. When the hydrology was applied, the hydrograph would flow downstream in the channel and it would also flow backwards into the empty storage area. Since then, the hydrograph was applied to the storage area which took care of this issue. Negative flows through Oakport are expected. They are breakout flows from the Red River and will be negative.
25. This is difficult to tell with 12 hour hydrograph output ordinates.
26. Agreed.
27. Yes. Breakouts, Reverse flow due to flood wave on Red River backing into stream reaches. Ex. Sheyenne River, Rose Coulee.
28. Agreed.
29. Yes. See Appendices A, B, C, D for Historic comparisons to USGS gages, and Appendix E for synthetic comparisons to balanced hydrographs.
30. Agreed.
31. Agreed.

32. Reviewed to some extent. It is a large and complex model. Not everything could be addressed with the given schedule.
33. No.
34. n/a
35. Rating Curve based on field measurements on the Red River from USGS Gage 05092000 at Drayton, ND.
36. Much of the examples given were situations where geometry was created with LiDAR only. See Table B1 of Appendix B.
37. The Sheyenne River is a perched channel. Therefore, there are significant breakouts that often times transfer flow out of the localized system. The effective flood insurance study model discharges for the 2-, 1-, and 0.2-percent chance events are all nearly the same on the Sheyenne River. Similar discharges will provide similar water surface profiles and increase the likelyhood of the profiles crossing. Drain 34 was a reach with limited detail. Water surface profiles should be used evaluated with caution.
38. No.
39. No. However, significant detail in floodplain mapping has taken place.
40. Not Checked.
41. Not Checked.
42. HEC-RAS Manual
43. Yes. Table B2 in Appendix B provides a list of the Manning's "n" values used.
44. Agreed.
45. Unsteady flow model parameters are all set to zero. Okay.
46. Additional cross sections were added to the Red River reach between Hickson and Thompson. The Thompson to Drayton reach could be improved with additional cross sections in the future.
47. Yes. Primarily for stability in some locations.
48. 1000 feet is relatively a short distance on the Red River given the dimensions of the channel meander.
49. No. However, most seem to be reasonable.
50. Agreed.
51. Agreed.
52. Agreed.
53. Drains 27 and 53 are small tributary reaches. They have spoil banks on each side. The cross sections are extended beyond the spoil banks, but conveyance is contained in the channel.
54. Consider the majority of the noted river reaches are routing reaches. Also, the Sheyenne River is a perched channel where the water surface will likely be close to the bank stations at full channel capacity.
55. Some locations.
56. Not in routing reaches. See Table B1 of the Appendix B report
57. Not all bridges were checked due to the size and complexity of the model
58. Not checked at all locations.
59. Dependent on source data as explained in Table B1 of the Appendix B report.

60. Most Lateral structure and storage connection weir coefficients are 1. Some were reduced to 0.5 to help reduce iterations where low crossings provided too much flow. Inline structures were set at 2.6.
61. Levees are typically roads or temporary or permanent flood protection. The WSEL on WRRND RS 169582 does go above the cross section plot on the left end, however that portion of the cross section does not convey water (ineffective flow)
62. To the extent possible. Complex model.
63. To the extent possible. Complex model.
64. Reasonable widths.
65. To the extent possible. Complex model.
66. To the extent possible. Complex model.
67. To the extent possible. Complex model.
68. Agreed.
69. Agreed.
70. Agreed.
71. Cross sections width will vary between cross sections in meanders vs. straight reaches. Also, some reaches are for routing purposes and have been created with less detail. The majority of the specifically noted distances are in the RLR to Drayton reach (Red Lake River to Drayton). This reach was primarily used as received with minor modifications for calibration. Additional cross sectios were not created.
72. Not checked.
73. Not revised. Minor issue. Drain 37 is routing reach.
74. Not revised. Minor issue. Drain 37 is routing reach.
75. Cross Section was a sensitivity check and was supposed to be removed. It is now removed.
76. Not revised. Complex model. Could be reviewed in the future. Storage area drains out from initial condition.
77. Not revised. Complex model. Could be reviewed in the future. Initial condition attenuates before flood wave begins.
78. As expected. Similar to the Red River and Rose Coulee.
79. This may be related to the connection with the Sheyenne Diversion Reach and the Inlet structure to the Sheyenne Diversion. It does carry downstream, but is still 7-8 days before the primary flood wave passes through.

2.3 Review 3. Internal Review by HDR

HDR provided comments on the Existing Conditions models prior to the January 31, 2011 submittal. HDR has expanded the previous comments to address the With-Project LPP comments. These were provided on February 25, 2011. The review by HDR addresses larger scale model structure, stability and data fit to observed data.

Responses

1. Table B1 was supplied with the January 31, 2011 submittal that presents the sources of the HEC-RAS geometry data and level of detail and quality of such data.
2. General comment, no response required.
3. General comment, no response required.
4. General comment, no response required.
5. General comment, no response required.
6. It is not likely that the bridge modeling approach will changed for specific event frequencies with separate geometries. It is a complex model many bridges. Additional effort would be required to evaluate the modeling approach of each bridge.
7. The model was developed using model geometry from previous studies. The cross section layout was complete. Additional cross sections were added through the Hickson to Thompson reach on the Red River. During calibration, aerial photos were used to assist in setting ineffective flow locations and elevations.
8. Considerable review and sensitivity iterations were conducted to best simplify the complex flow paths of the Red River in a 1-D model. The bank stations define where overbank flow is compared to channel flow. This defines conveyance areas as well as flow path reach lengths. Aerial photos were used for the 2009 flood to best estimate conveyance area and overbank vs. channel reach lengths. Additionally, the location and number of channel points limit the options for where the bank stations are placed because a bank station has to be placed on a specific channel point. Without adding an arbitrary point to the cross section, the bank station may be slightly higher or lower on one cross section or another. Bank stations through the detailed portion of the Red River (upstream of Halstad) seem fairly consistent. Additional detail should be added to the Halstad to Drayton reach as part of future modeling efforts.
9. Increasing the maximum iterations to 40 may improve the model. However a sensitivity analysis was conducted to test this. Actually, a similar number of maximum iteration locations appear during unsteady flow computations. As expected, the model run time is significantly longer. Model stability was found to improve with a decreased computation time step.
10. The cross section was inserted as a sensitivity analysis and it has now been removed. Matching stage at the Fargo gage and through the protected area was rather challenging. The modeled water surface elevations were generally lower than observed data. Many parameters were adjusted through internal sensitivity analyses in attempt of increasing stage. The final outcome was to maintain a channel "n" value of 0.045.
11. Cross sections overlap where tributaries join with the Red River. The tributaries often approach the Red River at an angle. The Red River cross sections have been created such that their conveyance is maintained through the junction. The tributary cross sections are in place to route the lower tributary flows to the Red River. The tributaries are relatively small in comparison to the Red River. Additional volume storage caused by overlapping cross sections was assumed to be minimal. This appears to be an issue. However, no solution or suggestion was provided as an alternative by the review or project team.

12. In response to this comment, the model was revised to have weir coefficients of 1 for storage area connections and lateral structures as discussed in Section 3.5.3 of Appendix B.
13. Modifying the inflow hydrographs would impact the modeled stages recorded and likely provide a different calibration fit to observed data. Some gage analysis was completed and it appears as though the rating curve is adjusted based on the loop. Additional investigation into this would be beneficial.
14. Aerial photos were used in calibration of the 2009 ad 1997 flood events. The flood inundation extents match the aerial photos reasonably well.
15. The high water marks were obtained through field survey. The data is considered to be reliable. The river conditions such as wave action, local turbulence, gradients caused by velocity were not known when the survey was conducted. Additional detail could be placed on the specific bridges with regard to contraction/expansion.
16. The 2009 flood event had an initial crest and then in some locations experienced a second one. The calibration focused on the first crest and less emphasis on matching the second crest.
17. General comment, no response required.
18. Will consult with USGS again for further information on looped rating curve at Fargo.
19. Will consult with USGS again for further information on looped rating curve at Hickson.
20. The observed hydrograph was an estimate provided by USACE. It is anticipated that the process to development the estimated hydrograph did not have a method of accounting for model geometry storage and routing.
21. Observed stream gage data was obtained from the USGS website. A gage at this location was not available. If there is a stream gage here, it is anticipated that backwater from the Red River would cause issues due to its close proximity to the Red River.
22. Stream gage stage records on the Sheyenne River have heavy influence from ice conditions and breakout discharges. This can also be identified by the rating curve irregularity from the stream gage.
23. The reach can be added again as part of future modeling efforts.
24. The Harwood stream gage has a water impact from the Red River. This is the reason discharges are not calculated here.
25. Cass County Drain 14 conveys local inflow hydrograph water as well as possible breakout water. Inflows entered into the system represent a 104 square mile drainage area that extends upstream to the Sheyenne River and downstream to the confluence of Drain 14 with the Maple River. This inflow should be uniformly distributed.
26. The sources of inflow hydrographs are provided in the February 28, 2011 Appedix B.
27. The HTAB parameter has been fixed.
28. The lateral structure should be fixed as part of future modeling efforts.
29. The geometry was obtained from another project. It will be reviewed during future modeling efforts.

30. The channel bottom on Drain 34 was estimated.
31. Cross sections could be deleted. This is a routing reach only.
32. Elm River is a routing reach.
33. Could add cross sections as part of future modeling efforts.
34. General comment, no response required.
35. Drain 37 is a routing reach with little detail.
36. Inundation limits were compared to aerial photos. They match well.
37. General comment, no response required.
38. General comment, no response required.
39. The observed balanced hydrographs were compared in previous phases. However, now in Phase 4, storage areas were added and some cross sections were shortened to convey the same flow. Reviewing results at specific cross sections only represents part of the transect flow since additional water is conveyed in the storage areas and connections. A direct comparison would not be accurate.
40. Hydrograph comparisons are provided in Exhibit E of Appendix B.
41. The levees match current permanent flood protection through Grand Forks. Distances in the model on the referenced cross section on Figure 23 were verified with distances shown on the aerial images from the 2009 flood event. They are the same. The referenced model cross section in Figure 24 is not the cross section pointed at in the aerial image of Figure 24. The levee shown in the center of the image is the levee protecting Grand Forks, however the reviewer is pointing at the levee on the east side of East Grand Forks that protects the City from the Red Lake River.

REVIEW 1. PRELIMINARY INTERNAL REVIEW BY MOORE ENGINEERING, INC.

FARGO MOORHEAD METRO FEASIBILITY STUDY

UNSTEADY FLOW MODELING - Initial Review Prior to Calibration of the 2009 Event

COMMENTS/RESPONSE 10-28-2010

Reviewer	Org	Review Type	#	Date of Comment	Comments
MEI	MEI	Internal	1	28-Oct-2010	**Cross-section spacing.** In 4 instances, the cross-section spacing is greater than 4 miles and in 108 instances the sections are spaced over 1 mile apart. Consideration should be given to adding additional cross-sections or at least to determining how sensitive the spacing is for the downstream impact analysis. See **Table 1** for a list of sections where the spacing is greater than 1 mile. A spacing of more than 1/3 mile can be pushing the limit for cross-section spacing. In particular, the cross-section spacing upstream and downstream of a lateral structure could be even more sensitive. In the case of the lateral structure that transfer flow to Heartsville Coulee, the spacing is 18,976 feet (see comment 10 on this lateral structure).
MEI	MEI	Internal	2	28-Oct-2010	**2009 Calibration for Next Phase of Modeling.** At RS 2,388,223, a plot showing a comparison of computed versus observed elevations shows a significant difference for flows less than about 14,000 cfs. See **Figure 1**. It is realized that at this stage the model calibration is not calibrated. However, the differences in stage are quite large. This indicates that the "n" values are high for the low flow channel. When the calibration of the model is accomplished, the goal should be to obtain as good a calibration as possible for even low flows.
MEI	MEI	Internal	3	28-Oct-2010	**Weighted Manning's "n" Values.** Plots of weighted Manning's "n" values for the channel as shown in **Figure 2** show that the channel n values are extremely high. It is typical for reviewers to look at this plot to judge the reasonableness of "n" values. It appears only *some* of these "n" values are just merely shifted (i.e., Red River RS 2,316,206 and 2,415,915 and 2,416,271) Note that Table 2 does not pick up all of the sections with n values issues as illustrated by RS 2,416,271.
MEI	MEI	Internal	4	28-Oct-2010	**XS Delta WS Values.** A plot (**Figure 3**) of the change in water surface elevations indicates that some of the changes in water surface elevations are quite large. Consideration should be given to adding cross-sections where the difference is large.
MEI	MEI	Internal	5	28-Oct-2010	**Manning's "n" Values.** The range of "n" values for just the channel portion of the RRN Sections was reviewed. Some of the sections have n values which are outside the normal range. **See Table 2.**
MEI	MEI	Internal	6	28-Oct-2010	**Channel Bank Locations.** The location of the bank stations has been identified previously in the ATR comments dated 10-5-1010 for the unsteady flow model (comment 18). In additional to this the steady flow model had received ATR comments stating that the channel bank stations needed to be brought down in elevation to the primary bank stations. These comments were followed and a good calibration for the entire rating curve at the Fargo Gage was achieved (for both low and high flows). This provided for a reasonable channel "n" value, which may help to address comments 3 and 5. Consideration should be given to address this issue, and not just for the reach below the Thompson gage (but also from Hickson through Fargo to the Thompson gage).
MEI	MEI	Internal	7	28-Oct-2010	**Correction Hydrograph.** A large negative correction hydrograph in the Red River SH to RLR reach indicates a problem with either the calibration discharge hydrograph on the main stem or else the input hydrographs for the tributaries. See Figure 4.
MEI	MEI	Internal	8	28-Oct-2010	**Junctions.** The distance across a junction is being applied twice, which will result in the energy losses calculated by the model twice. This is due to having the distance across the junction in (a) the junction editor and (b) the last downstream cross section of the river upstream of the junction. To correct this issue the distance entered into the last downstream cross section of the river upstream of the junction should be zero.

FARGO MOORHEAD METRO FEASIBILITY STUDY

UNSTEADY FLOW MODELING - Initial Review Prior to Calibration of the 2009 Event

COMMENTS/RESPONSE 10-28-2010

Reviewer	Org	Review Type	#	Date of Comment	Comments
MEI	MEI	Internal	9	28-Oct-2010	**Lateral Weir Profiles.** The lateral weir profiles for storage area connections "FgoSC131" and "FgoSC128" are not representative of the actual highest ground elevation available. These storage area connections represent Interstate-94 west of the Red River in Fargo. This is critical since nearly 9,400 cfs is currently being modeled over I-94 for the 100-year event (peak discharge), with a WSEL of 904.12 at the I-94 bridge. Two issues are identified at this location. The first is that "FgoSC131" should be drawn over the north ramp (west bound) at the 25th Street South crossing, *not* the south ramp (east bound). This will result in a 'lowest road profile' from a current elevation of approximately 902 to approximately 906. "FgoSC128" between 25th Street South and University Drive does not seem to be drawn over the highest available ground elevation over I-94. Identifying the highest ground profile over I-94 would result in a 'lowest road profile' from an elevation of approximately 903 to approximately 904. The second and maybe most crucial feature would be the New Jersey Style road barriers along I-94. These are approximately 3 feet above the road profile. This may control the amount of flow that breaks out of the Red River and flows over I-94. The controlling profile over I-94 needs to be identified and incorporated into the geometry of the model.
MEI	MEI	Internal	10	28-Oct-2010	**Lateral Structure (LS 1,658,177).** There is an extremely long distance between "Red River - WRRMN to RLR" RS's 1,667,665 and 1,648,688, which is 3.6 miles long (18,946.38 feet). This results in 2.54 feet of energy losses for the maximum water surface profile. To compound the matter, there is a lateral structure (LS 1,658,177) between these two cross sections representing breakout flow into Heartsville Coulee. Additional cross sections should be added within this area to properly model the energy equation and the breakout flow into Heartsville Coulee.
MEI	MEI	Internal	11	28-Oct-2010	**Flow Roughness Factors (100-Year).** The "Flow Roughness Factors" are not consistent from reach to reach with the set of flow range values.
MEI	MEI	Internal	12	28-Oct-2010	**Sheyenne River Breakout.** The 2009 calibration currently has breakout flow west of the Sheyenne River near Cass Hwy 14 near Horace. The model currently shows 1,790 cfs flowing through MSHSC338 as lateral wier flow, which represents the railroad profile. This railroad profile does not contain the wooden bridge over Drain 21C. Consideration should be given to modeling this storage area connection as a 'weir and culvert' to ensure sufficient backwater from the railroad embankment is provided. This may help to resolve the issue identified in comment 13.
MEI	MEI	Internal	13	28-Oct-2010	**Sheyenne River Breakout.** The 2009 calibration currently has breakout flow west of the Sheyenne River near Cass Hwy 14 near Horace. The model currently shows 1,780 cfs flowing over Cass Hwy 14 on MSHSC402 as lateral weir flow. According to County Hwy personel this road was never overtopped in 2009, although it did get come up to the road shoulder.
MEI	MEI	Internal	14	28-Oct-2010	**Sheyenne River.** The 2009 calibration geometry does not contain the control structure at the Horace to West Fargo Diversion inlet. Rather, only a rating curve is applied to pull water into the Horace to West Fargo Diversion. Consideration should be given to modeling the box culverts at the control structure, which will impact the amount of water into the protected area of Horace and West Fargo.
MEI	MEI	Internal	15	28-Oct-2010	**Wild Rice River Hydrograph.** An observed hydrograph is currently at RS 169,892, which is upstream of I-29. The correct location of this HOBO data should be at RS 166,766, which is just downstream of the current location, at Richland Hwy 2.
MEI	MEI	Internal	16	28-Oct-2010	**Intermediate Calculated Flow Hydrograph for RRN nr Georgetown.** In the responses to the ATR comments, the discussions address the flow calibration for the RRN at Georgetown. Since this location does not have observed flow data, the overall flow calibration should match the locations where there is observed USGS flow data. The HEC-RAS model should not be **forced** to fit the intermediate calculated flow hydrograph if it is developed by hydrologic techniques.

NOTE: THE COMMENTS ABOVE ARE PRELIMINARY DRAFT COMMENTS AS OF 10-28-2010.

REVIEW 2. INTERNAL REVIEW BY BARR ENGINEERING

Project Number: 34091004.00	
Project Name/Description: Fargo Moorhead Metropolitan Feasibility Study	
Model Name: 500Dec & 2009	
Model Developed By: Houston Engineering	Model Checked By: Barr Engineering
Model Date: 12/22/2010	Model Check Date: 12/29/2010

No.	**General Information/Checks**	**Comments**
1	What version of HEC-RAS was used?	4.1
2	What datum is the survey data and the HEC-RAS model in (NGVD29 or NAVD88)?	Not documented in the model
3	Is the datum noted in the "description" box of the main project screen?	No
4	Is the model georefenced and if so what is the horizontal projection?	Yes it is geo-referenced. Projection is not indicated within the model.
5	Is the stationing reference noted in the "description" box of the main project screen?	No
6	Does the stationing in the HEC-RAS model match the map	Not Checked
7	Is the model in the same datum as the mapping? (mapping should be NAVD88)	Not Checked - No Mapping
8	Are photos attached to cross-sections and structures (i.e. photos facing downstream)?	No
9	Are there any modeling or mapping anomalies that should be pointed out to a reviewer?	Yes. There are several locations where cross sections overlap each other. See Overlapping XS tab of this Excel workbook.
10	Are the "Plan" files clearly labeled/named?	Yes.
11	Was GeoRAS used to create your geometry schematic?	Yes
12	Were cross-sections created using both survey data and digital topography (i.e. LiDAR, 2ft topo, etc.)?	Yes. Some cross sections cite survey data by Houston and others cite LiDAR data in the comment boxes.
13	Is the unsteady model computation time appropriate?	Yes, time step is 5 minutes for a 1-month model duration. Inflow hydrographs have data every 12 hours.
14	If there are options checked on the plan file do these make sense?	Yes.
15	If the tolerances have been changed are these appropriate?	Tolerances are 0.03' for XS and 0.1' for storage areas. Presumably tolerances were modified to achieve stability of the unsteady model.

Hydrology Checks		**Comments**
16	Has the hydrology been reviewed internally?	Not Checked
17	Has the hydrology been reviewed and approved?	Not Checked
18	Is a write-up of the hydrology included?	No
19	Have the reach boundary conditions been reviewed internally?	Not Checked
20	Is the boundary condition a "Known W.S."?	No
21	Do the 10-, 50-, 100-, and 500-yr flows entered in RAS match the flow hydrographs calculated in your model/hydrologic calculations? Are they labeled in HEC-RAS?	Not Checked
22	Are the 10-, 50-, 100-, and 500-yr flow hydrographs labeled as such in the HEC-RAS flow editor?	NA. Due to the size of the model the flow profiles are broken out into separate models.
23	Are there any negative flow hydrographs that may indicate correction hydrographs within HEC-RAS?	Not Checked
24	Do the output hydrographs from HEC-RAS have any abrupt changes?	Wolverton has oscillating flow directions at RS 3017 & 3123, Oakport Upper has significant negative flows just prior to the peak.
25	Are storage areas adjacent to river reach cascading flow ahead of the river?	Not Checked
26	Do the profiles through the entire hydrograph make sense?	Spot check of hydrographs found no glaring problems.
27	Do any river reaches have negative flow?	Yes. See Negative Flows tab of this Excel workbook and comment #9 below.

REVIEW 2. INTERNAL REVIEW BY BARR ENGINEERING

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Hydraulics Checks		**Comments**
28	Is a write-up of the hydraulics included?	No - not applicable at this time
29	Do discharge values match hydrologic analysis results?	Not Checked
30	Do the locations where discharges change in the model agree with locations on drainage area map?	Not Checked
31	Do the discharges change along the stream at the appropriate location?	Not Checked
32	Have the "Summary of Errors, Warnings, and Notes" been reviewed and addressed for each profile and floodway?	It was reviewed. Many "Divided Flow", "Conveyance Ratio", "Energy Loss > 1.0 ft" warnings.
33	Has Check-RAS been run, and each issue addressed?	Not Checked
34	For any remaining Check-RAS issues, has the Check-RAS output been annotated?	Not Checked
35	What is the boundary conditions for the starting water surface elevation?	Boundary Conditions vary depending on the reach. There are four types of boundary conditions used. Flow Hydrograph, Uniform Lateral Inflow, Rating Curve, Lateral Inflow Hydrograph
36	Have you reviewed the HEC-RAS profiles?	Many channel bottoms are higher than the inverts of culverts. For example: Drain 37, Drain 37, RS 53721, RS 41967, RS 37537, RS 35831, RS 33145, RS 25187, RS 19050, RS 4853; Possible difference in datums that wasn't adjusted?
37	Do profiles cross for different return periods?	There are several instances of crossing profiles. Sheyenne River - Horace to WF 186215, 186040, 185962, 185750 (500 & 100 yr cross) Drain 34 - 20561.04, 20501.35 (10 & 50 yr cross). See Profiles tab in this Excel workbook.
38	Have you checked the water surface elevations to make sure there is no negative water surface slope?	No negative slopes found for the 500-yr profile or the 2009 event
39	Have you used the 3D viewer (X-Y-Z plot) to review the 100- and 500-yr floodplains and floodway?	Not Checked
40	Are the WSELs on tributaries lower than the WSELs on the main channel at or near the confluence?	Most are ok. Some come in lower. The Sheyenne River - Maple to Red has cross sections that overlap with the Red River conveyance area. The cross sections outside of the red river conveyance area are have a peak profile that is lower than the red river peak.
41	Are there drawdowns on the profiles?	Not Checked
42	What is the source for your Manning's N values?	Not documented in model
43	Has a summary of the range of Manning's N values been created?	Not documented in model
44	Do the Manning's N values seem reasonable?	Yes. However in some locations they are not consistant. Mannings n values (channel and overbank) in Drain 37 are all 0.04. Mannings n values in the Maple River are 0.04 in the channel and 0.08 in the overbanks, with similar terrain.
45	Do the expansion and contraction coefficients seem reasonable?	The default value for contraction and expansion losses is 0.0 for an unsteady flow model (see clip from HEC Users Manual in the C&E Coef tab). However, because this model was developed from previous (assuming) steady state models there are expansion and contraction coefficients entered in the model. Presumably during model development and calibration the contraction and expansion coefficients were reviewed and adjusted as needed. Most seem reasonable for a steady flow model. However there are instances where the upstream and downstream coefficients are 0.1 nd 0.3 for bridges, culverts and in-line structures, which would be low for cross sections located adjacent to a structure. See highlighted rows in C&E Coef tab.
46	Are the cross sections lengths appropriate for the model reach?	some of the reach lengths are still quite long.
47	Are there interpolated cross sections?	Yes.

REVIEW 2. INTERNAL REVIEW BY BARR ENGINEERING

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48	Have Cross-Sections been placed at least every 1000 feet?	No. About 1/3 of the cross sections (1100+) are spaced more than 1000 along the channel. Given the meandering nature of the channel this is reasonable. However, roughly 150 cross sections are spaced more than 5000 feet apart (channel length). Twenty-six of those are over 10,000 feet apart. The vast majority of these are on the Red River near the upstream and downstream ends of the reach. The longest Channel reach length is over 4 miles at 23,310 feet. See Reach Lengths tab - all Channel Reach Lengths greater than 5000 are highlighted.
49	Are the WSELs for every cross section lower than the cross section end points?	No. The flat topography makes this difficult if not impossible to achieve. Most cross sections seem to be extended to a reasonable degree. Storage areas are used to model areas beyond the extent of the cross sections.
50	Are the cross sections perpendicular to flow?	Yes
51	Do the cross sections locations model constrictions and expansions in the floodplain?	Yes
52	Do the cross section locations model structures in floodplains	Yes
53	Do cross sections overlap storage areas?	Yes, some do. Drain 53 Wtsd, Drain 53, RS 9250 through 8779 AND 3975 through 3621; See additional screen grabs in the Overlapping XS tab.
54	Do the locations of bank stations make sense?	Most do. But nearly 300 have bank stations higher than the 500-year WSEL. Reaches include Drain 14, Drain 34, Drain 37, Elm River South Branch, Elm River North Branch, Red River RLR to Drayton, Rose Coul Drain 27, Shey-Div SheyDiv, Sheyenne WF to Maple, Sheyenne Gol to Horace. See highlighted rows in OB Elev tab of this Excel workbook.
55	At road crossings, are there actual road names in the "description" box of your structure data?	In some locations - but not consistantly throughout the model.
56	Are all road crossings being modeled?	Not Checked
57	Are you using the correct "bridge modeling approach" for each bridge?	Not Checked
58	Is the top of bridge roadway being properly defined?	Not Checked
59	Is the correct top-of-road elevation, low-chord elevation, and deck width being modeled?	Not Checked
60	Are the proper coefficients being used for weirs, pressure, and/or culvert flows?	Most Weir coefficients are 1. The following structures have a coefficient of 0.5: Drain 14, Lower, 37000 LS; Drain 14, Lower, 2900 LS; Maple River, Durbin to D14, 16450 LS; The following structures have a coefficient of 2.6 (default): Shey-Div, SheyDiv, 37912.24 IS; Red River, RLR to Drayton, 1549773 IS; Red River, RoseC to OakPt, 2416121 IS, 2384792 IS, 2368258 IS; Maple River, D14 to Mouth, 845 IS; Maple River, D14 to Mouth, 36000 IS; Heartsville, Main, 1311.5 IS;
61	Are levees used, and if so, is a description included?	Levees are used. I haven't seen a description. Most seem to be roads. I think either a levee needs to be put on the left side of Wild Rice ND, BL Aber, RS 169582, or the cross section needs to be extended to include the road on the left side. WSEL goes above the cross section data on the left end.
62	Are ineffective flow areas modeled correctly?	Not Checked
63	Have ineffective flow area been placed at correct stations/elevations?	Not Checked
64	Do the ineffective flow areas contain any drastic changes in width?	Not Checked
65	Are the ineffective flow areas being modeled correctly at bridges that are not overtopped?	Not Checked
66	Is there ineffective flow areas where high ground prevents flow?	Not Checked
67	Is there appropriate control for split flow?	Not Checked
68	Are split flows at lateral overflow weirs set up correctly?	Not Checked
69	Are all lateral flow structures connected to another reach or storage area?	Yes
70	Are all storage areas connected to another storage area or reach via lateral structure or storage area connection?	Yes

REVIEW 2. INTERNAL REVIEW BY BARR ENGINEERING

Project Number: 34091004.00	
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Additional Notes:

71	Does the channel width vary from one cross section to the next?	Some cross sections have large changes in channel width between cross sections. See Bank Sta tab in this Excel workbook.
72	Maple River, Durbin to D14, 162161.2BR	Should bridge have a 3rd pier? Simply looks funny.
73	Drain 37, Drain 37, RS 45478	Roadway deck upstream distance and width are 10' and 10'. The two culverts through the road have 35' and 41' as the upstream distance and culvert length. Culverts are not within the road.
74	Drain 37, Drain 37, RS 19050	Distance to upstream XS from deck and culvert are not equal (20' and 1')
75	Red River, RoseC to OakPt, RS 2422105	XS looks like it was inadvertantly moved. Looks like a copy of RS 2424705.
76	WRSA340	Check the initial condition. Resulting hydrogragh starts high with high outflow, approaches "empty", nothing ever comes in.
77	WRSA300	Initial condition seems high. Connection to WRSA305C has 2 box culverts at 904 and 907.4. Initial condition is 913.2. Sends 700 cfs to WRSA305C at time 0.
78	Drain 14, Lower, RS 84.09075	Negative Flow, likely due to passing flood wave at the downstream connection. Some of the "upstream reaches" (Drains, small tributaries) have a negative flow at the downstream end of the reach towards the end of the simulation.
79	Interface of Sheyenne River, Horace to WF and Gol to Horace	Initial condition WS profile seems really funny. Causes a sharp change in the profile at the initial steps. The effects of this are definitely seen downstream to the end of the Sheyenne.

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Memo

To:	Stuart Dopperpuhl, Moore Engineering Gregg Theilman, Houston Engineering Lee Beauvais, Moore Engineering Miguel Wong, Barr Engineering		
From:	Mark Forest, HDR	Project:	FARGO-MOORHEAD METRO FLOOD RISK MANAGEMENT PROJECT
CC:	Michael Johnston, HDR Nate Dalagar, HDR		
Date:	February 25, 2011	Job No:	145025

RE: Technical Review – Unsteady Flood Models

Purpose

The purpose of this memo is to present the findings from a review of the 2009 calibration unsteady flow model (2009.prj), the synthetic event models and the LPP alternative models as one set of combined comments. Previous comments had been provided by HDR on November 3, 2010 and January 21, 2011. These previous comments have undergone some revisions and additions in this set of comments. This memorandum supersedes those previous two memorandums.

Comment Response and Comment Closure

Since this memorandum supersedes those previous two memorandums, comment closure on the previous two sets of comments may not be needed. Comment responses to this set of comments would be more appropriate since this set of comments has been revised to reflect additional review based on additional information and updated models. Closure of these comments is recommended for project QA/QC documentation purposes.

Data Provided For Review

Data provided included:

Additional data was transmitted by FTP site that included inundation mapping for the 2009 event. A revised version of the 10-, 50-, 100- and 500-year event existing conditions models dated February 18, 2011 (ExCond.prj) was also provided to replace the December 2010 and January 2011 versions of these models.

The models included in this final review include:

<u>Historic Events – Existing Conditions (Calibration and Verification)</u>

- 1997 (1/28/11)
- 2006 (1/27/11)
- 2009 (1/27/11)
- 2010 (1/28/11)

<u>Historic Events - LPP:</u>

- 1997 (Version 5, 2/18/11)
- 2006 (Version 2, 2/16/11)
- 2009 (Version 4, 2/18/11)
- 2010 (Version 4, 2/18/11)

Synthetic Events – LPP:

- 10-Year (Version 6a, 2/18/11)
- 50-Year (Version 7, 2/16/11)
- 100-Year (Version 14, 2/17/11)
- 500-Year (Version 13, 2/16/11)

Additional Data Provided as Background or Supporting Information:

- Phase 3 Report
- LPP_minus_ExCon_100YR_Feb17_North_detailed_scale.pdf
- LPP_minus_ExCon_100YR_Feb17_South_detailed_scale.pdf
- LPP_minus_ExCon_500YR_Feb16_North_detailed_scale.pdf
- LPP_minus_ExCon_500YR_Feb16_South_detailed_scale.pdf
- ExistingConditions_DepthGrids.pdf
- FCP_Downstream_Book.pdf
- LPP_Downstream_Book.pdf
- LPP_Upstream_Book.pdf
- MN_DepthGrids.pdf
- ND_DepthGrids.pdf

This review focused on model structure, stability and data fit to observed data. The hydrologic data used to represent the calibration and synthetic events were not reviewed. It is my understanding that QA/QC review of these project elements was performed by others. The review was initiated with the 2009 event calibration since it provides the basis for most of the model parameters used in simulation of the synthetic events. Comments pertaining to the synthetic events follows the comments pertaining to the 2009 calibration.

Model Suitability for Feasibility Analysis

The stated purpose of this modeling effort was for a feasibility level study. At the feasibility level, it is anticipated that future refinements may be necessary to refine the selected alternative(s) and ultimately to accurately reflect the selected design condition and for quantification of project benefits. The comments represented in this memo are not atypical of the types of refinements needed as the project progresses into the next phases of concept refinement and design. In general the model appears to provide a very reasonable simulation of the observed events in recent years. The model also reasonably simulates the complex behaviors that are observed in the field measurements made by the USGS at a number of locations throughout the project area. Therefore, while the comments demonstrate the need for some model refinements during the ongoing planning and design process, these comments do not suggest that the model is not adequate for feasibility level planning evaluations.

It is my observation that refinements to the hydraulic models would provide some improvements to the model results, particularly on the tributaries to the Red River. The level of refinement needed will vary depending on the needs for identification of project impacts and quantification of project benefits. In other words, some of these tributaries may not need to be refined if those reaches are used primarily for routing of hydrographs within the study area but the results are not used for evaluation of impacts, benefits or interior drainage.

The greatest level of uncertainty in the model is the hydrology. The limited gage data on many of the tributaries creates a higher degree of uncertainty with respect to hydrograph shape, volume and timing of tributary hydrographs entering the Red River from the tributaries. There may also be uncertainty with regard to the variability between events that has occurred historically from these tributaries. The uncertainty in the hydrology is particularly important in this case due to the very shallow gradients of the watercourses in the study area. These very shallow gradients result in more pronounced backwater impacts from flow and volume variability that extends for significant distances upstream of a confluence. This condition is noted in the data presented below that shows that at many gage locations the stage can vary by 2 to 8 feet for a given discharge because of these downstream influences. These differences can be event variability or differences noted between the observed stages during the rising and receding limbs of an event hydrograph.

In addition, modeling tools are undergoing very rapid improvements in their capability to simulate the complex channel and overbank behaviors that are exhibited in this study area. For example, future phases of this project will be able to take advantage of the two dimensional tools that are in the process of being incorporated into HEC-RAS by the team at the Hydrologic Engineering Center. These enhancements to HEC-RAS should allow more accurate simulations of the project features that are currently proposed in this feasibility study. This project may also present a unique opportunity to test those new HEC-RAS two dimensional modeling features during the beta testing phase of the software.

Comments – General Comments

1. This model is extremely complex and includes data from other modeling efforts on many of the tributaries. It is recommended that the source of data for each model element be clearly documented relative to the source of the data, level of detail it represents, verification of model inputs as well as any known limitations with the data sets. For example, the data set for the Elm River does not include some of the bridges. Therefore, the water surface profiles for the Elm River would not be accurately represented and should not be used for any purpose. If these were used to simply route inflow hydrographs from gage or model hydrograph input locations, this should be documented in the hydraulic report with a warning regarding the use of the resulting water surface elevations and flood limits. It was also be important to verify that these inaccuracies would not impact the flood damage and benefit calculations.

2. Due to the shallow gradient combined with the large tributary volume that joins the Red River in the study reach, the system exhibits significant backwater influences that vary from event to event depending upon the timing and volume of tributary inflows. As will be exhibited from the data presented below, the system can have a wide range in stages for the same discharge.

3. Since not all of the tributaries are gauged, development of statistics for coincident flows is not possible for each watercourse. Therefore, simplifying assumptions are required and have been applied to both the calibration models and synthetic events. These conditions demonstrate that one of the greatest uncertainties in this modeling effort is the hydrologic inputs. Determination of downstream impacts may need to consider this uncertainty with a sensitivity analysis during later stages of design refinements.

4. During the evaluation of these models, HEC-RAS version 4.1 was found to have a bug that prevented complete post-processing of the output data. This was resolved by HEC with the provision of a beta version of 4.2 that could be used to post-process the computations performed by version 4.1.

5. HEC is in the process of developing 2D modeling options for use within HEC-RAS that will integrate with the 1D channel elements. While they have made significant process on the development of the 2D model code, HEC has reported that the beta version that integrates this code into HEC-RAS will not be available until the first half of 2012. The timing of this release could coincide with future refinements of the models for this project as the design progresses and could provide a very useful tool for modeling the complex overbank interactions that occur in the study area.

6. Many of the bridges in the model have piers and profiles that are overtopped during higher flow rates. The methods selected typically include only the energy method for both low flow and overtopping conditions. Consideration of momentum for low flow and pressure/weir flow for overtopping conditions should be considered. It is recognized that multiple recurrence intervals being analyzed with the same data set. In cases where there is submergence of the bridge (such as in the 500-year profile), the energy equation would be more appropriate. While that same bridge may have weir flow in the 100-year event without complete or highly submerged conditions. In that instance, the bridges modeling methods may vary between recurrence intervals to reflect these ranges in conditions

7. A key consideration in setting up a one dimensional model is appropriate alignment of each cross section to maintain a cross section that is consistently perpendicular to the flow patterns in the floodplain. This is important for two primary reasons; incorrect alignment can bias flows especially with wide overbank flow, and incorrect alignment misrepresents where the water surface elevations are represented in the floodplain for determining extents and depth or misrepresent the water surface at a lateral structure that is located at the end of the cross section. This often requires more than two angle points in the cross section alignment to accomplish this. Typically (as described in the RAS manual) this requires starting with a map that shows the primary flow vectors in all parts of the floodplain. Sometimes this is a reasonable approximation based on the topographic mapping that might have to

be re-evaluated after a preliminary model is established. Flooding event photos and video often provide valuable insights. Due to the very shallow gradients that are experienced in this project areas, this may not be a significant factor, but should be reviewed for reasonableness.

8. Choice of bank stations is important and should be consistent. Typically the bank stations are either at the edge of the active channel (near ordinary high water) or at the break in slope at the top of bank. These decisions should carefully consider how conveyance is being represented as well as velocity distributions in the channel. If the model will be used for computing scour or doing a floodway analysis, choice of bank stations is also a critical consideration. The plot below (Figure 1) shows variability in bank station elevation that appears to suggest some inconsistency in the selection of bank station location.

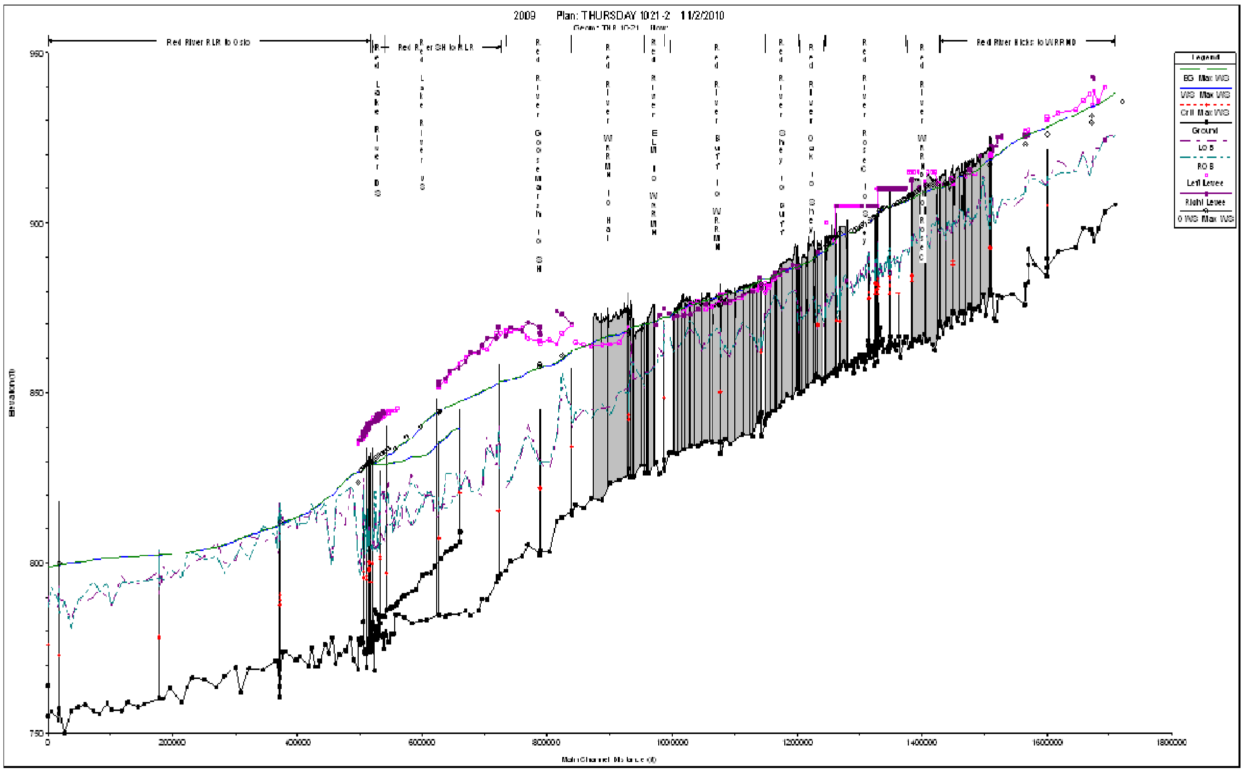

Figure 1 - Plot showing inconsistent bank stations

9. The maximum iterations is set to 20 rather than the maximum value of 40. At some cross sections, the model reaches the maximum number of iterations. Minor improvements could be obtained by changing this value to 40 to allow those locations to come closer to closure on the solution.

10. Cross section 2422105 appears to have an error in the georeferencing (see Figure 2). The n values for the main channel on the Red River are set to 0.045. Photos suggest that the main channel roughness should not be that high, as seen in the photo below (Figure 5). However, the velocities in the main channel range from 1.3 to 3.5 feet per second for the Red River in most parts of the model. So, the model is likely to be relatively insensitive to this parameter. More frequent events may be more impacted by a high estimate of n value than the extreme events.

Figure 2 - Example Photo of Main Channel

11. Some of the cross sections overlap one another in the overbank areas (see Figure 3 and 4). This condition should never exist for a number of reasons. From a hydraulic perspective, cross sections should not overlap or cross once another. This suggests that two computed water surface elevations occupy the same space and conveyance may be using the same area. From a post-processing perspective, the output will also represent two different elevations at the same location (overlapping breaklines).

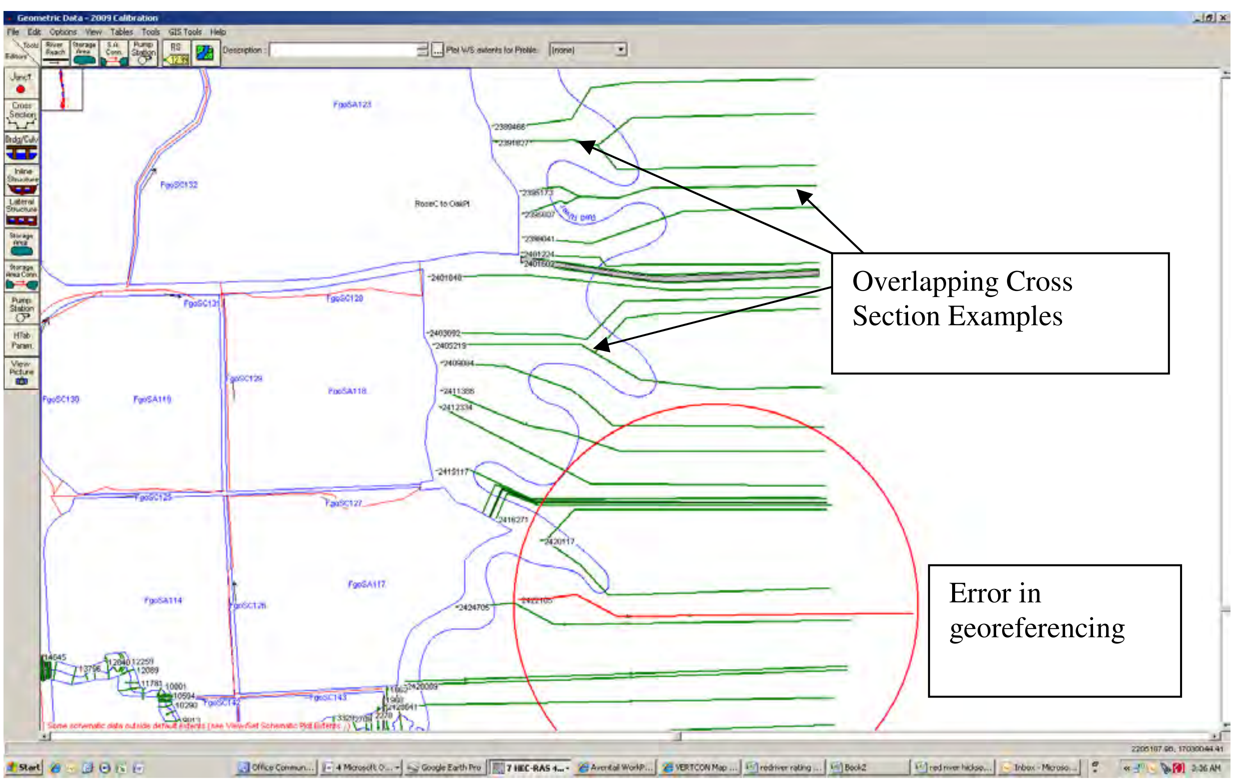

Figure 3 - Examples of Cross Section Cut Line Problems

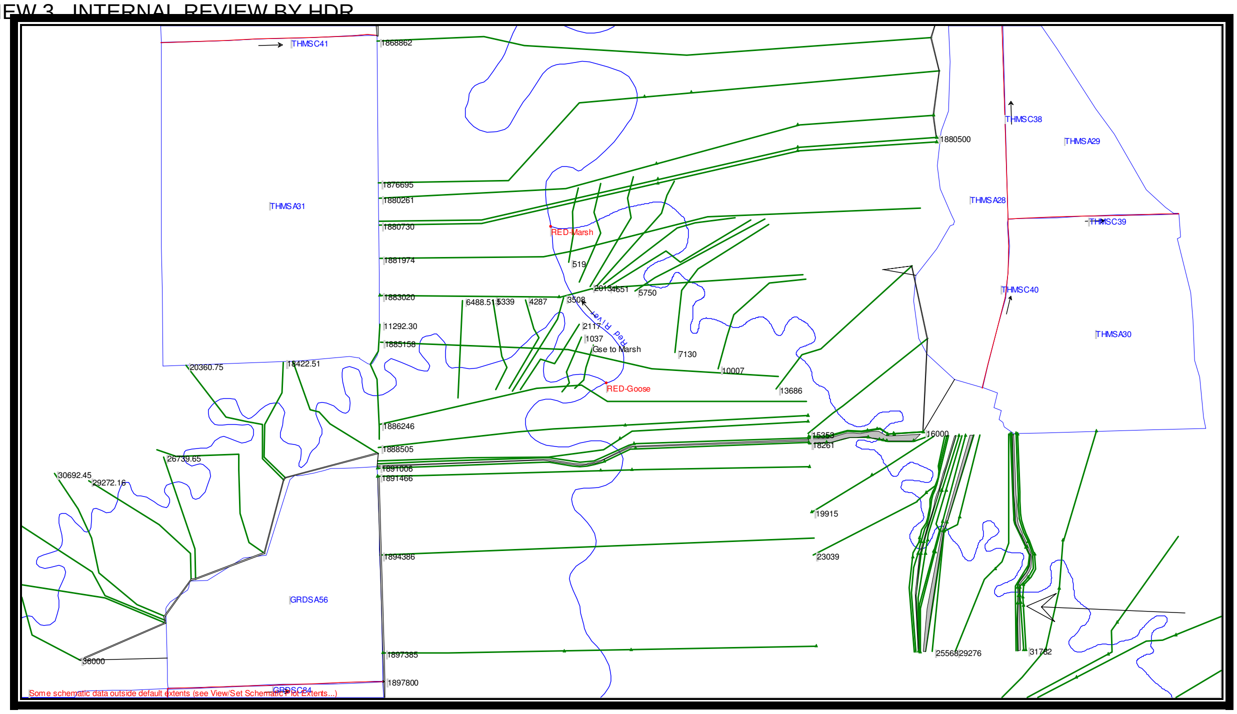

Figure 4 - Overlapping Cross Sections

12. Weir flow coefficients for storage area connections and lateral structures can range from values less than 1 for flow overtopping a natural ground topographic divide to values in the range of what was used (2.6 for storage area connections and 2.0 for lateral structures). I spoke to Gary Brunner (HEC) again about this topic in November. He and I also explored a range of values for use in a similar unsteady flow model for the Truckee River Project in Nevada. HEC-RAS uses the defined values for a starting point and then checks tailwater and headwater at each time step to check for submergence. The values are adjusted for submerged conditions and it will also calculate flow in both directions depending upon tailwater and headwater conditions for that time step. Often use of values above 2 can generate flows across lateral structures that are too rapid. A number of factors should be considered when selecting these values:

- Is the overtopping section is a paved section with little or no vegetation?
- What does the overtopping feature look like with regard to roughness, width, irregularity, etc?
- Is there shallow sheet flow that occurs between the overtopping section and the next downstream section or ponding area (if it is connected to another storage area). RAS does not explicitly consider the overland flow component. The calculations assume a series of cascading reservoirs when storage areas are linked in series. The flow from the storage area outflow reports directly to the connecting element. Therefore, your only means for slowing down this flow connection, is to use a lower weir flow coefficient (when using a broad crested weir).
- If the high point is natural ground with an uneven surface and vegetative coverage, this value could be less than 1. There is no specific guidance for the selection of these values. Calibration or validation runs using observed data is the only way to verify these decisions. We had very good validation data for the Truckee River model. It required values that ranged from 1.0 to 1.5 in many instances that are similar to this example, in order to get reasonable results.

Comments – Calibration Events (Existing Condition)

13. The “observed” hydrographs within the model domain are based on a best fit rating curve using direct and indirect measurements at this location. The model shows that the rating curves at many of the gage locations are looped, which suggest that the observed hydrograph at these location could be suspect. The USGS typically converts the recorded stage hydrograph into a flow hydrograph using a rating curve that is based on the best fit to the field measurements. The looped rating curves noted at many locations within the data set suggest that a single rating curve would provide an imprecise

method for estimation of the flow hydrograph from the recorded stage hydrograph data. If the rating curve during the rising limb differs from the rating curve during the receding limb, a single rating curve could be inaccurate. As a result, the hydrograph shape and volume could be misrepresented. The observed hydrographs may need to be adjusted based on an improved rating curve considering this behavior during the rising and receding limb of the hydrograph. This may significantly change the hydrograph volume for the observed hydrograph which will change your inflow hydrographs as well. Adjustment to the observed hydrographs and input hydrographs to account for the different rating curve that exists during the rising and falling limb of the hydrograph, could improve the volume inputs to the model and the results comparison to observed data. I would recommend that this be discussed with the USGS to determine if this behavior has been documented or considered, and the results of that discussion be documented in the report. I would also suggest that the potential range of error be tested using one of the observed hydrographs to see if the volume estimate changes significantly.

14. Do we have flood photos or videos that would be helpful to confirm flooding extends and flow behaviors? If so, how well does the model match those observations? If this information exists, it would be useful to show comparisons on the reports of model results compared to observed behaviors (inundation limits, overtopping of lateral structures and storage area connections, depth at buildings, etc.. This is useful validation information.

15. In some reaches we are matching high water and in some we are high or low. It is assumed that some of the high water data are from gaged locations which can usually be considered to be reliable. If some of the data is not from gage locations, how confident are we in quality of those data? Did we verify that these data were collected in areas that were reasonably protected from wave action or local turbulence? With regard to areas where we are low upstream of bridges, version 4.1 does allow you to use higher contraction and expansion losses where more severe contractions and expansions occur. The unsteady equations account for most of these pressure terms without needing to include additional contraction and expansion losses. But, there are times when additional losses do need to be considered with contractions or expansions that are more severe. These values are used differently in the unsteady solution than how they are used in the steady flow solution. Typically the values you would use are slightly lower than the values you would choose for steady flow. In areas where we are too high, we may need to consider our potential sources of error.

16. The match to the observed hydrograph near the downstream end of the model, appears to be a reasonable match for the 2009 calibration event for peak flow and timing but appears to have insufficient volume. The observed hydrograph could be somewhat misrepresented if the USGS uses a single rating curve to convert the recorded stage hydrograph to a discharge hydrograph due to the looped nature of the rating data that can be seen in Figure 6. The volume during the receding limb appears to be underestimated. The USGS data at this location suggest some looping behavior in the rating curve that is more pronounced for flows less than 40,000 cfs (Figure 6). This behavior is controlled or muted in the model by the fixed rating curve that is used as the downstream boundary condition. The model would need to be extended downstream to capture that behavior in the unsteady flow model at this location.

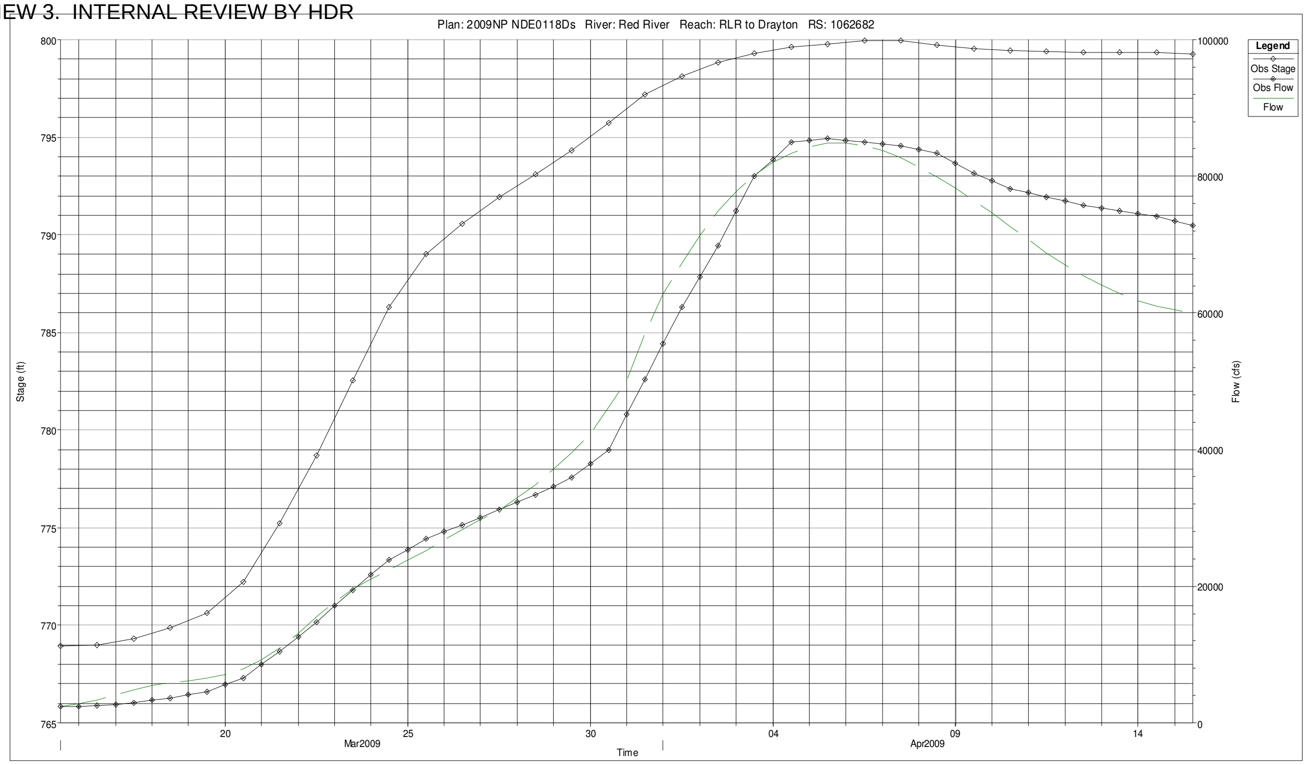

Figure 5 - Red River at Drayton Comparison of Computed and Observed Hydrographs at USGS Gage 05092000, Cross Section 1062682

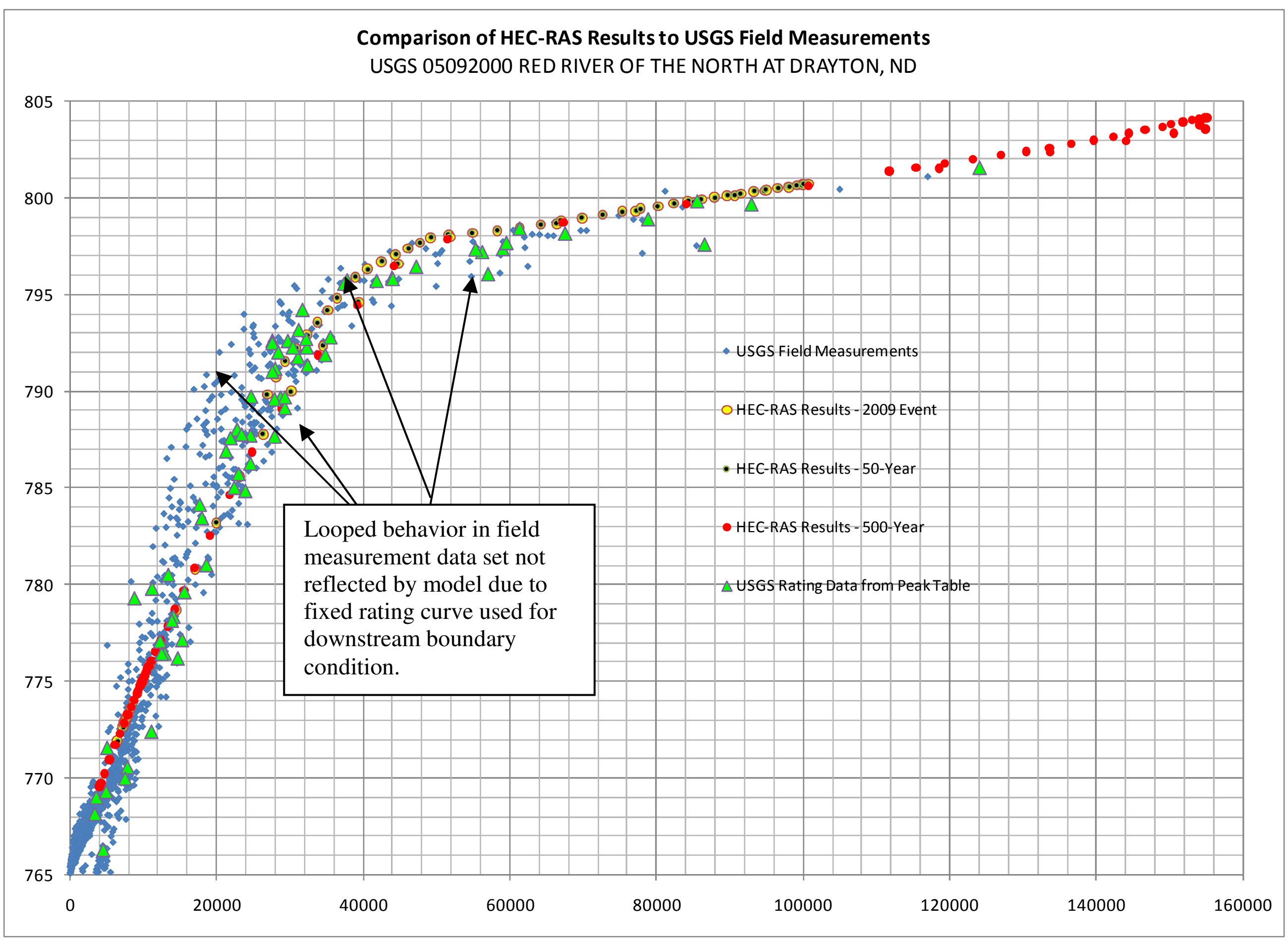

Figure 6 - Comparison of USGS Field Measurements to Model Results, Red River at Drayton, CS 1062682

17. Figure 7 compares the model results to the USGS field measurements at the Grand Forks Gage. The data suggests a fairly reasonable match at higher flow rates but more conservative estimates of stage for flows between 10,000 and 60,000 cfs. There is also a minor looping effect observed in the field measurements that is not represented in the model results.

18. The match to the rating curve and hydrograph at 2388223 is a reasonable match to observed data and shows a very slight over-prediction of stage during the rising limb and a slight under-prediction of stage and flow during the receding limb compared to the USGS data (Red River of the North at Fargo, ND). However, as discussed in Comment #14, the direct measurement data suggests a looped rating curve that is more pronounced than the model suggests (see Figures 9 and 10 below). Figure 10 superimposes the 2009, 50-year and 100-year model HEC-RAS results on the USGS measurement data and the published peak and stage values from the peak flow data table. The USGS data is on the 1929 datum. An approximate correction of 1.0 feet was used to adjust the data sets to match. While the data set appears to be reasonable match at the upper end, the model suggests only a limited loop for the 2009 event, but a more pronounced loop for the 50-year and 100-year events. The measurement data shows a wider variability in stages for a given discharge at the lower end of the curve. I would suggest that we verify the rating data used by the USGS for conversion of the stage hydrograph to a flow hydrograph. If they are using a different rating curve for the rising and falling limbs, these ratings should be compared to the model output data to determine differences between observation and model results. Since the degree of variability can be influenced by the volume, magnitude and duration of the observed event, it may not be possible to make any strong conclusions from these data without further evaluation of other historic events

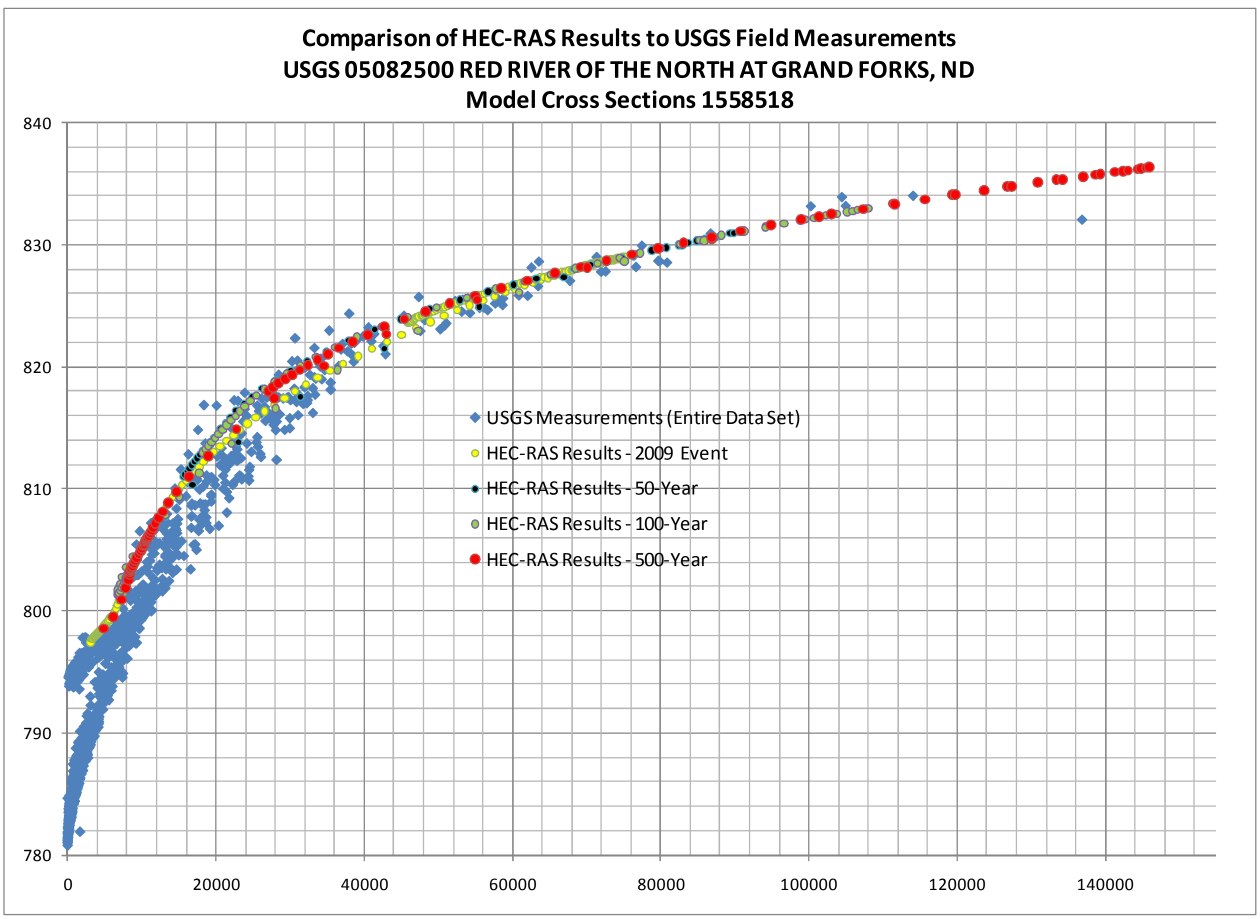

Figure 7 - Comparisong of Model Results to Field Measurements at Cross Sections 1558518

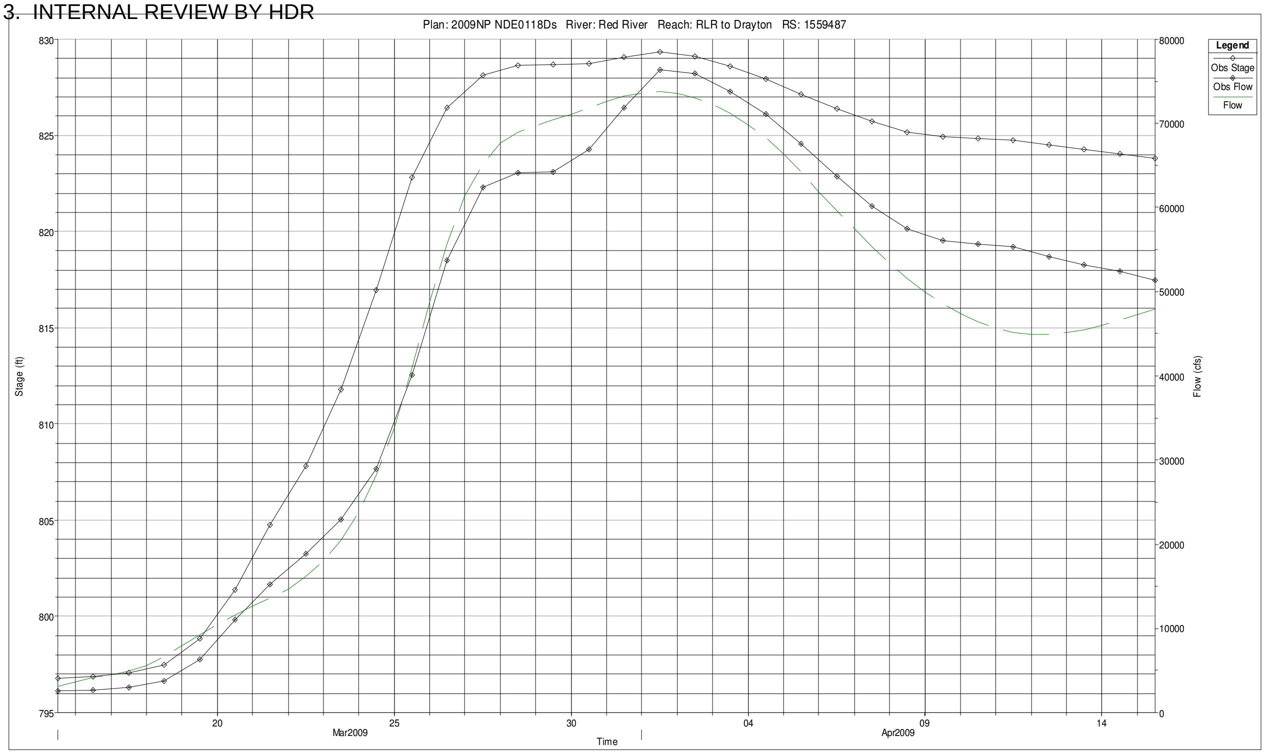

Figure 8 – Comparison 2009 Event Observed and Computed Hydrographs at Cross Section 1558518

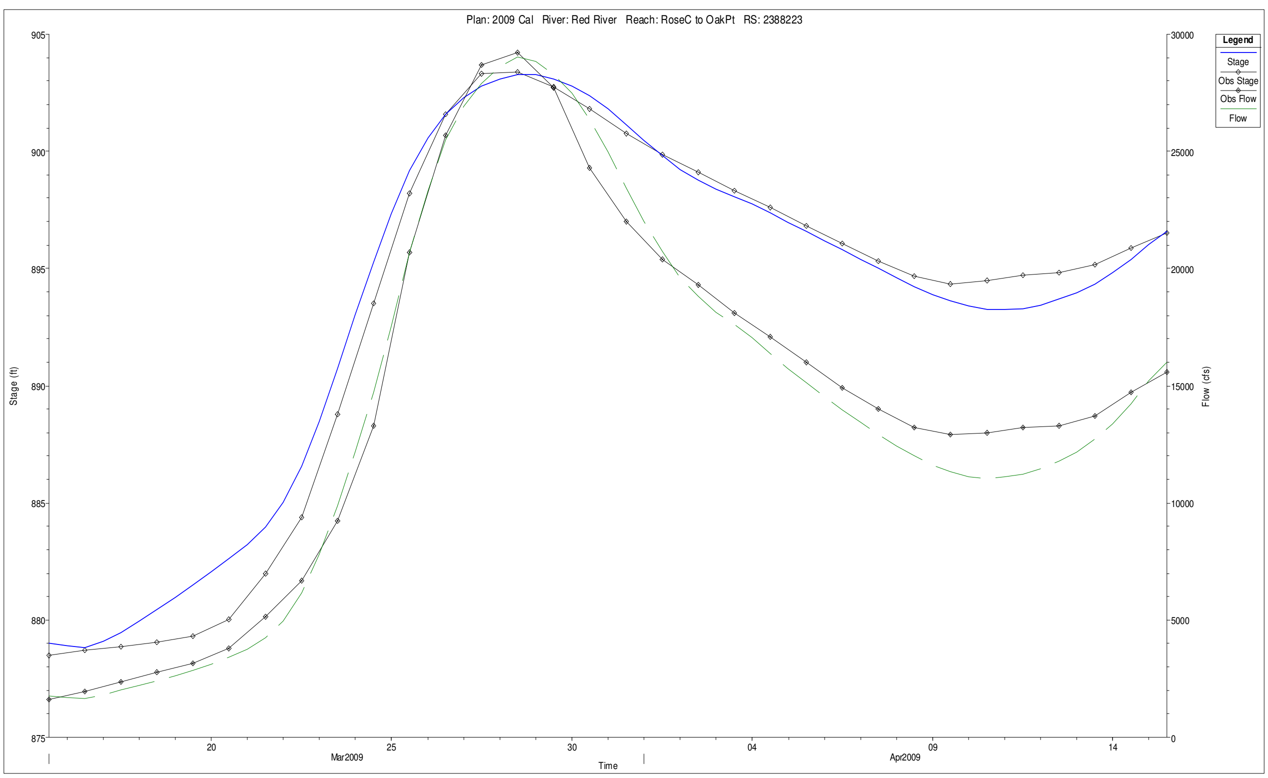

Figure 9 – 2009 Event Calibration Results at Red River Section 2388223

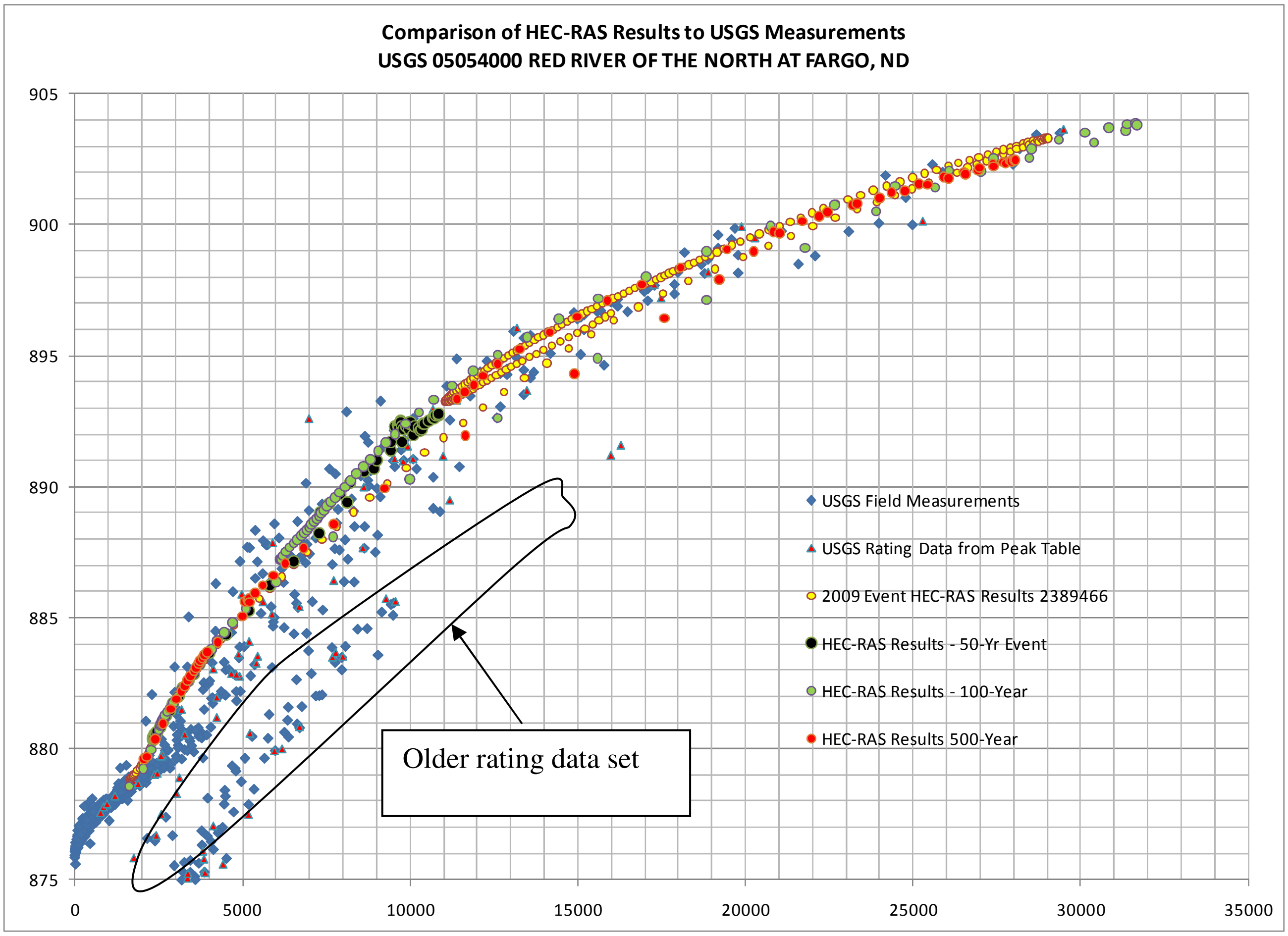

Figure 10 - Comparison of RAS Results (Yellow Dots) at 2388223 to USGS Field Measurements (Blue)

19. A comparison of the USGS field measurements was made at cross section 2563754 (Figure 11). The results appear to be a relatively reasonable match and suggest a pronounced looped rating curve as does the field measured data set. However, the model results show a much more pronounced looped effect than the USGS rating data seems to suggest. The data also suggests that magnitude of the looped behavior varies significantly between events. This would suggest that the rating data used for conversion of the stage to flow hydrographs may not be a good fit to every event. As can be seen from the published peak flow data set, that the published estimates for the recorded peak stages are based upon an assumption that there is a lesser looped effect. I would suggest that we obtain the rating data from the USGS and compare it to the model results.

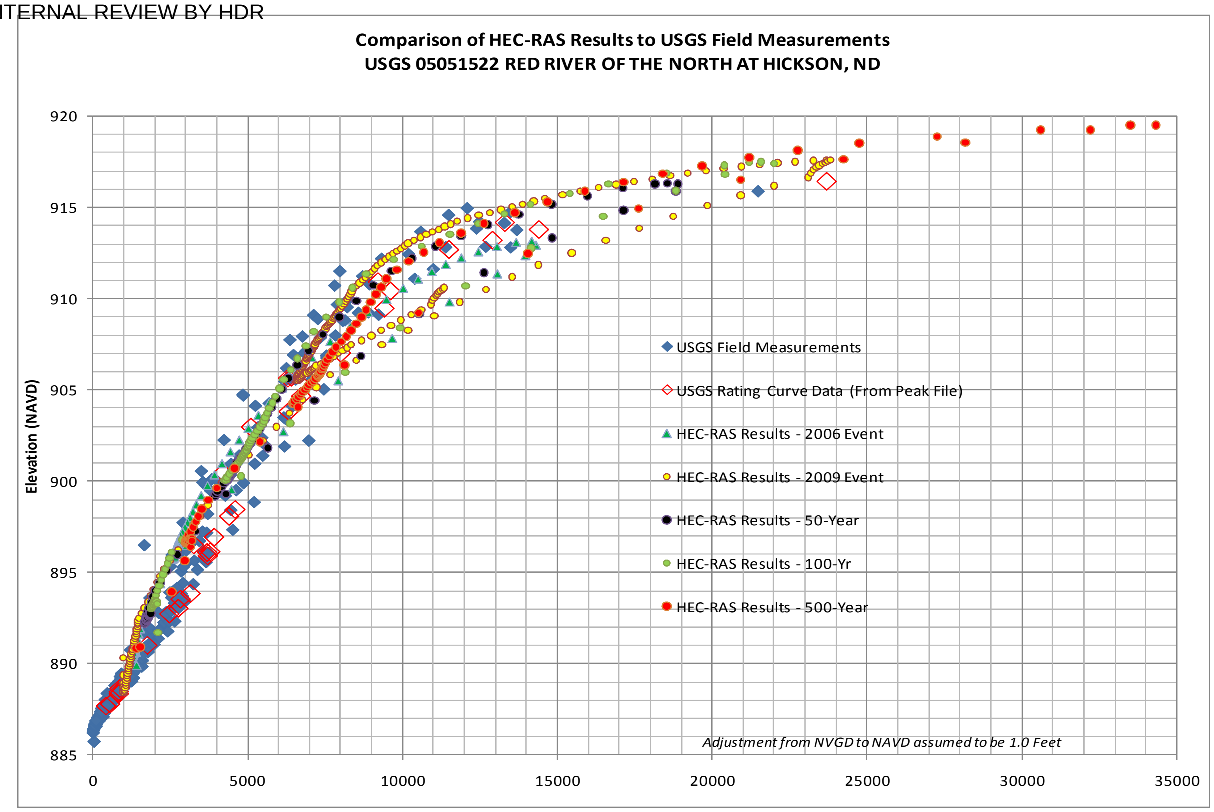

Figure 11 - Comparison of Measured and Computed Stage Data at Cross Section 2563754

20. The match to the observed hydrograph at cross section 10470 of the Buffalo River is not a good match (Figure 12) for a location that is near the upstream end of the model. This suggests that the inflow hydrograph should be adjusted or additional inflow is occurring that is not accounted for. The volume at this location does not match the observed hydrograph.

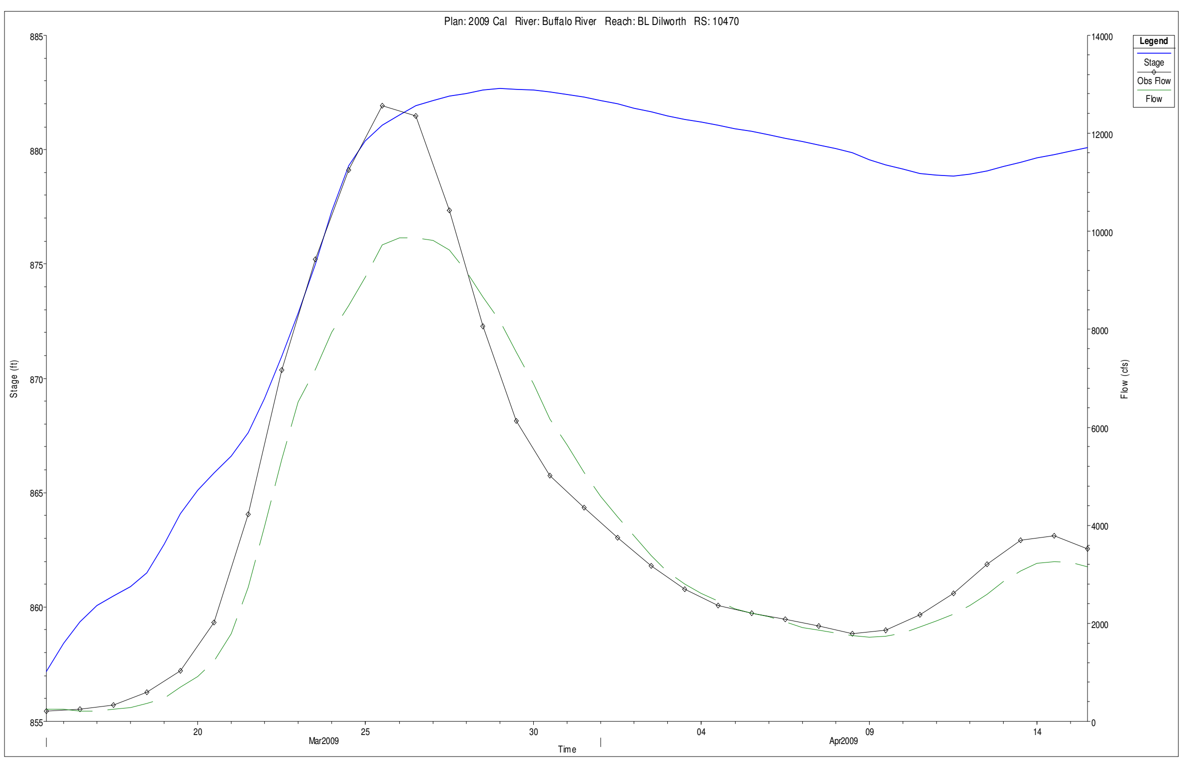

Figure 12 - Hydrograph Comparison at Buffalo River at Station 10470

REVIEW 3. INTERNAL REVIEW BY HDR

21. There is a gage at the downstream end of the Buffalo River Reach (USGS 05062095 BUFFALO RIVER AT U.S. HWY. 75 IN GEORGETOWN, MN). The USGS site says that the state maintains the data base. Did you verify if data was available? A gage at this location would provide useful calibration data.

22. The behavior of the Sheyenne River at 12th Avenue is very erratic due to downstream backwater influences (Figure 13). The comparison of the model results with the USGS field measurements shown in Figure 13 seems to suggest that the predicted stages are generally underestimated by as much as two feet. The datum associated with the USGS gage may need to be verified to determine if this is a model inaccuracy or datum shift. In this case, the datum is reported to the nearest hundredth of a foot which suggests that it has been surveyed.

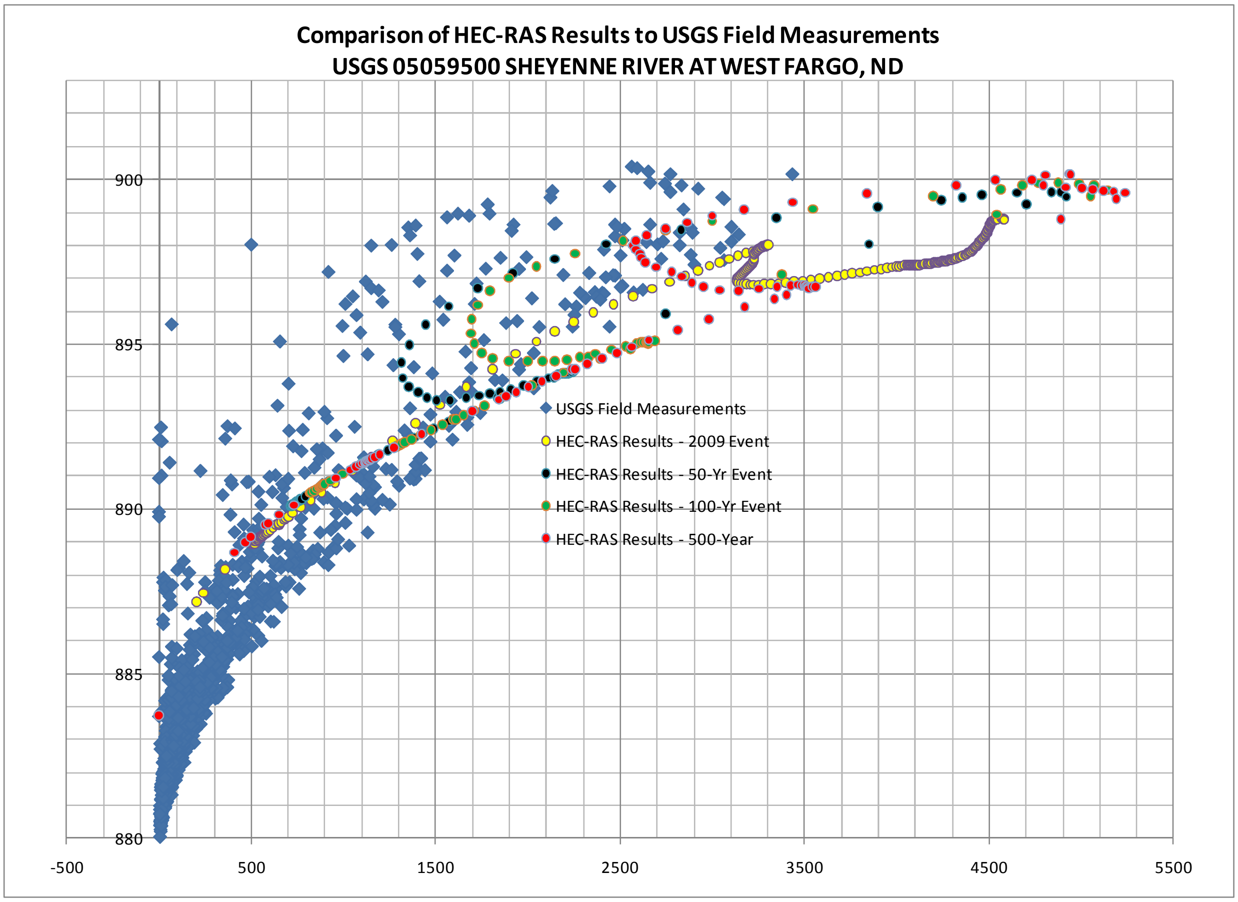

Figure 13 - Comparison of Measured Values and Results at 12th Avenue

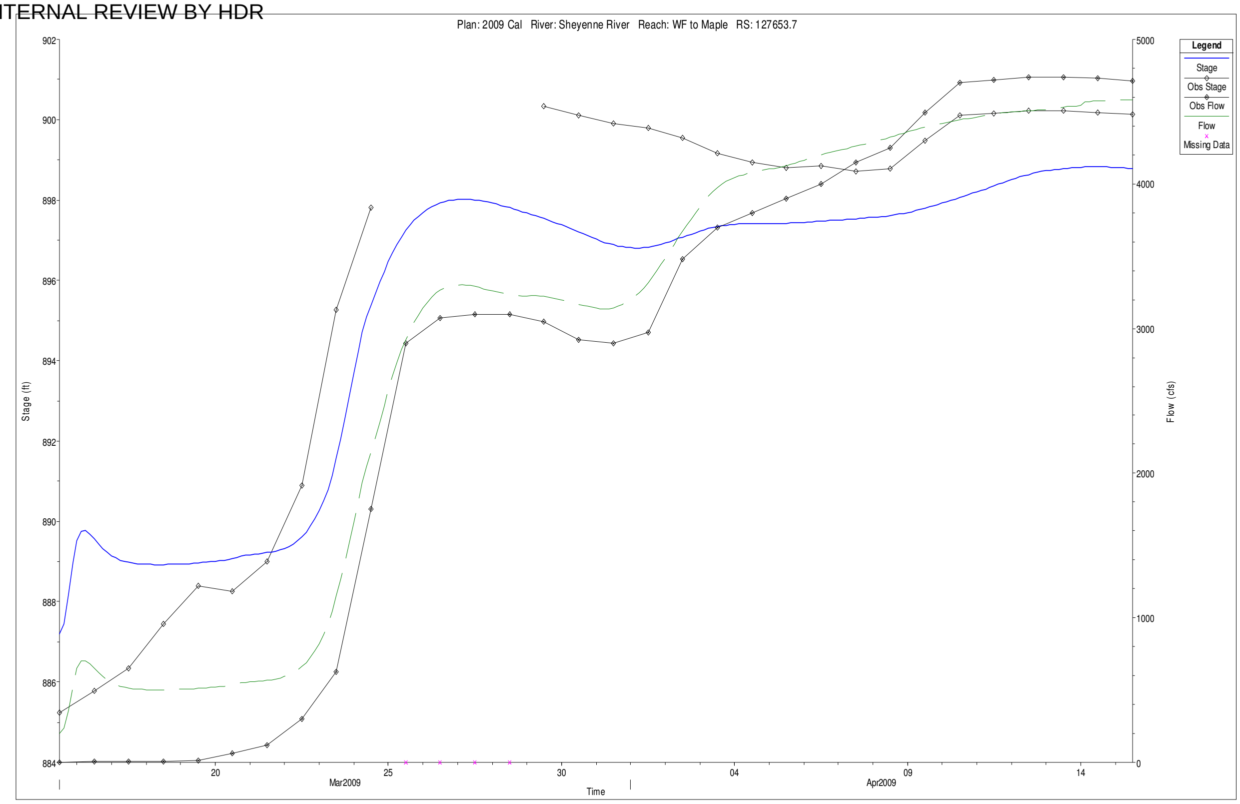

Figure 14 - Comparison of Observed and Computed Hydrographs at 12th Avenue

23. The Sheyenne River downstream of the above reach (Figure 15) should be inspected for potential causes for the underpredicted stages at this location. One potential cause is the fact that there is a bypass channel downstream of this location that is being modeled as a single cross section, rather than as two interconnected reaches of the same stream. This approach also requires that the bridges downstream being modeled as a multiple opening analysis with the same water surface elevation at each opening. The bypass channel also has a shorter reach length and is at a different elevation profile compared to the natural channel. If it is important to accurately predict water surface elevations in this area, better results could be obtained by modeling these reaches separately.

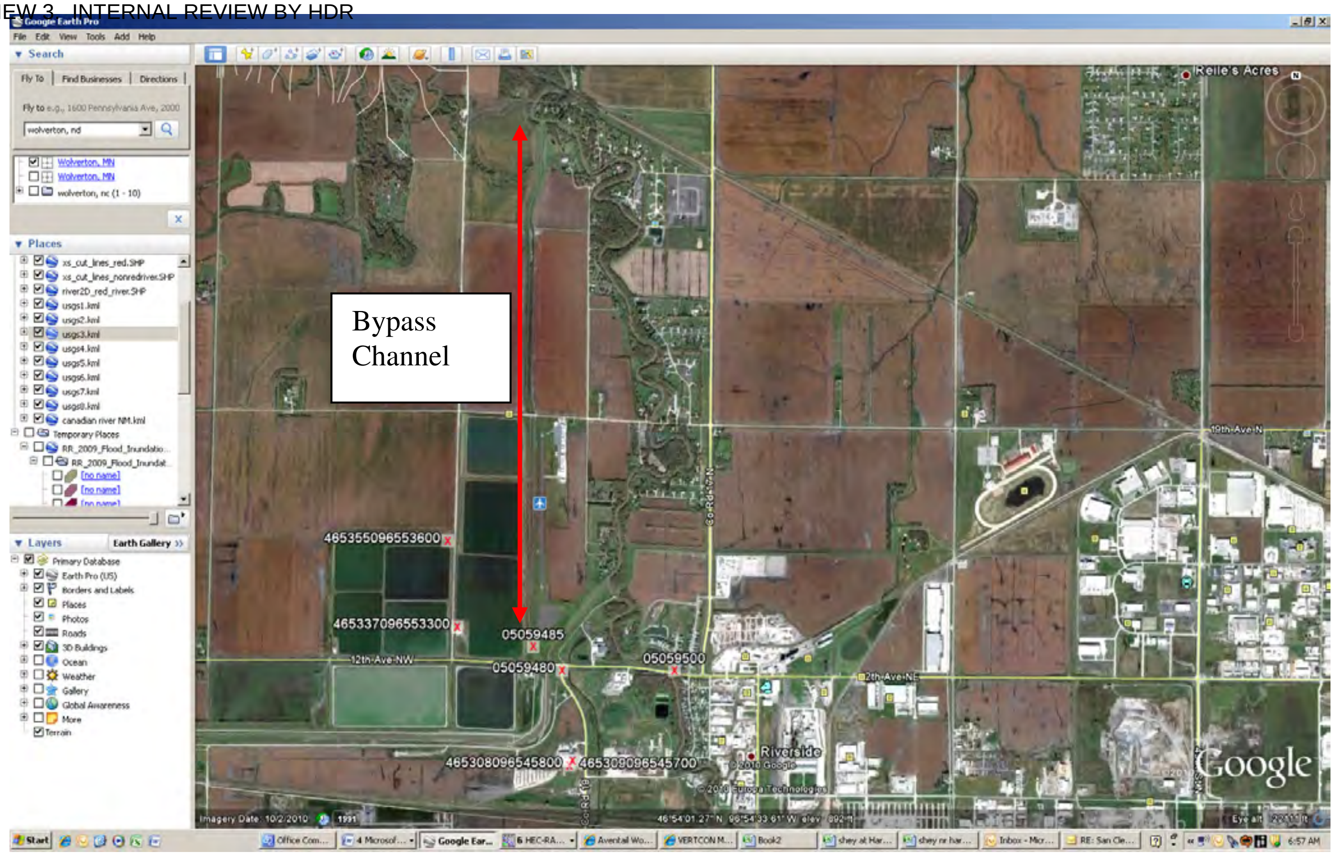

Figure 15 - Sheyenne Reach Below 12th Avenue

24. The Sheyenne River at Cross Section 71250 was compared to USGS field measurements (Figure 16). Comparison at this location may be difficult since it appears that that gage datum is approximate. However, two things are noted in the comparison. First, the range of stages due to downstream influences varies between the more frequent and less frequent events. Secondly, it appears that the model is over-predicting stage for the lower flow rates compared to observed events. The match is better as higher stages.

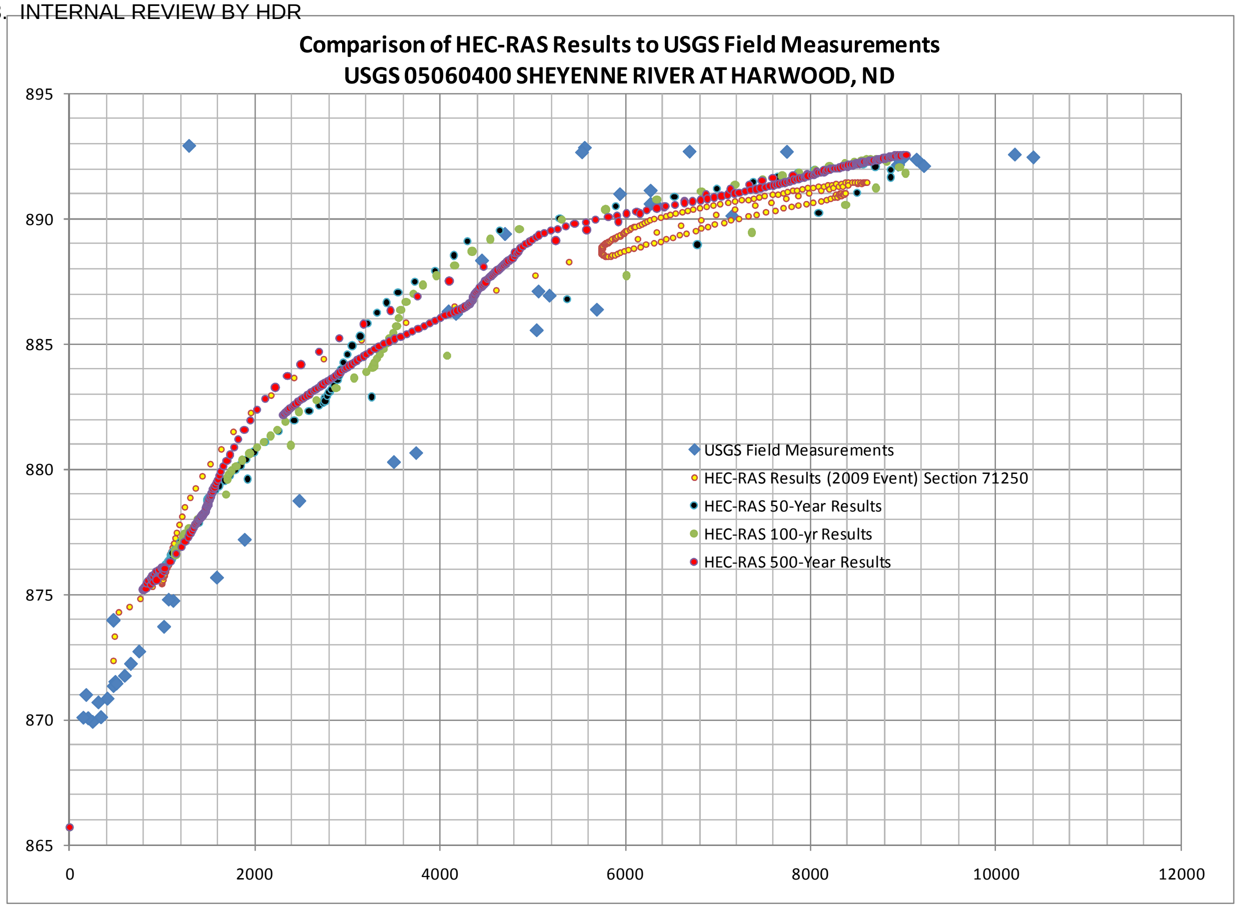

Figure 16 - Comparison of USGS Field Measurements to Results at Cross Section 71250

25. The inflow hydrograph for Drain 14 is entered as a lateral inflow hydrograph over a large reach of Drain 14. The aerial photo (Figure 17) suggest that this inflow enters as at a concentrated location:

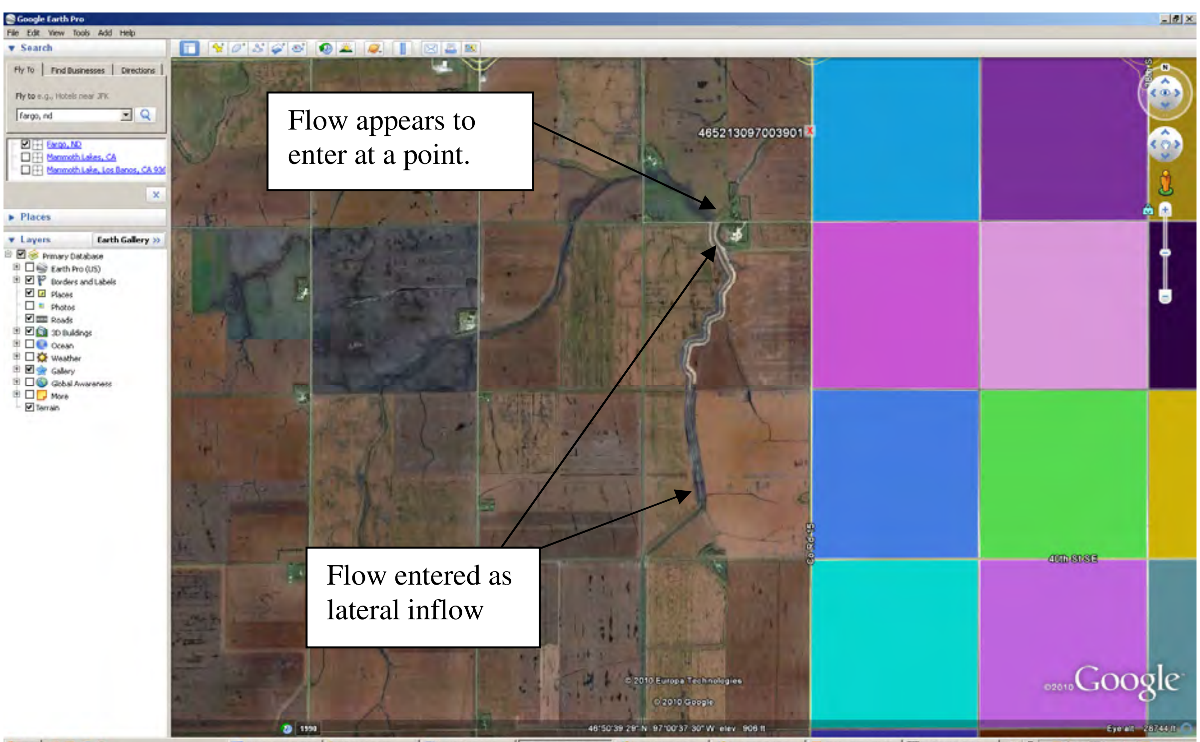

Figure 17- Drain 14 Inflow Hydrograph Comment

26. The sources for the inflow hydrographs for ungaged locations should be explained in the report.

27. When executing the model, a warning message is generated at Goose Creek cross section 6488. The hydraulic properties table includes data to elevation 860.8, but the maximum water surface reached 860.1, resulting in this message. This is caused by user specified HTAB parameters. At this cross section, an interval of 0.33 feet is specified which causes the values to only be computed to an elevation of 860.8.

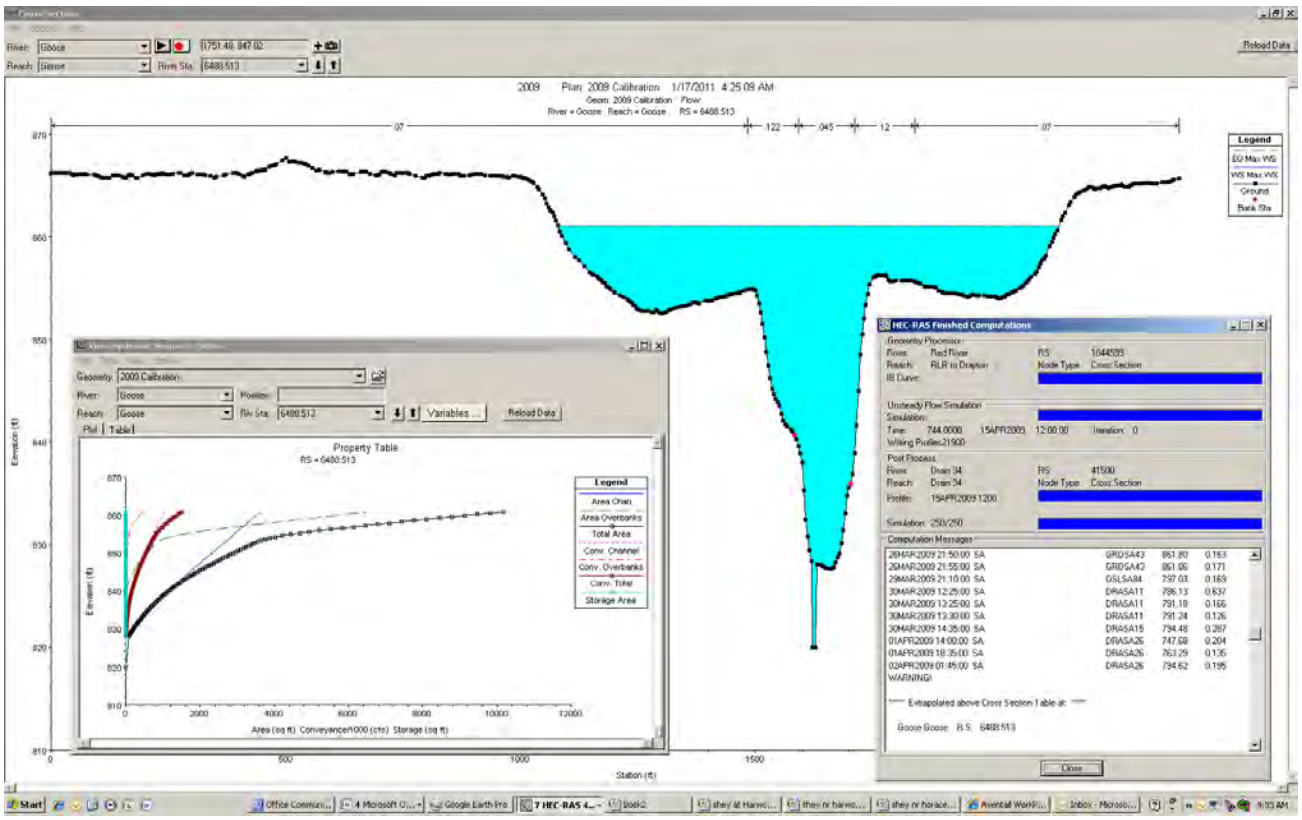

Figure 18 - Warning Message Associated with Goose Creek Section 6488

28. Lateral weir 135900 on Wild Rice is not properly linked and is showing up as overlapping the downstream lateral weir.

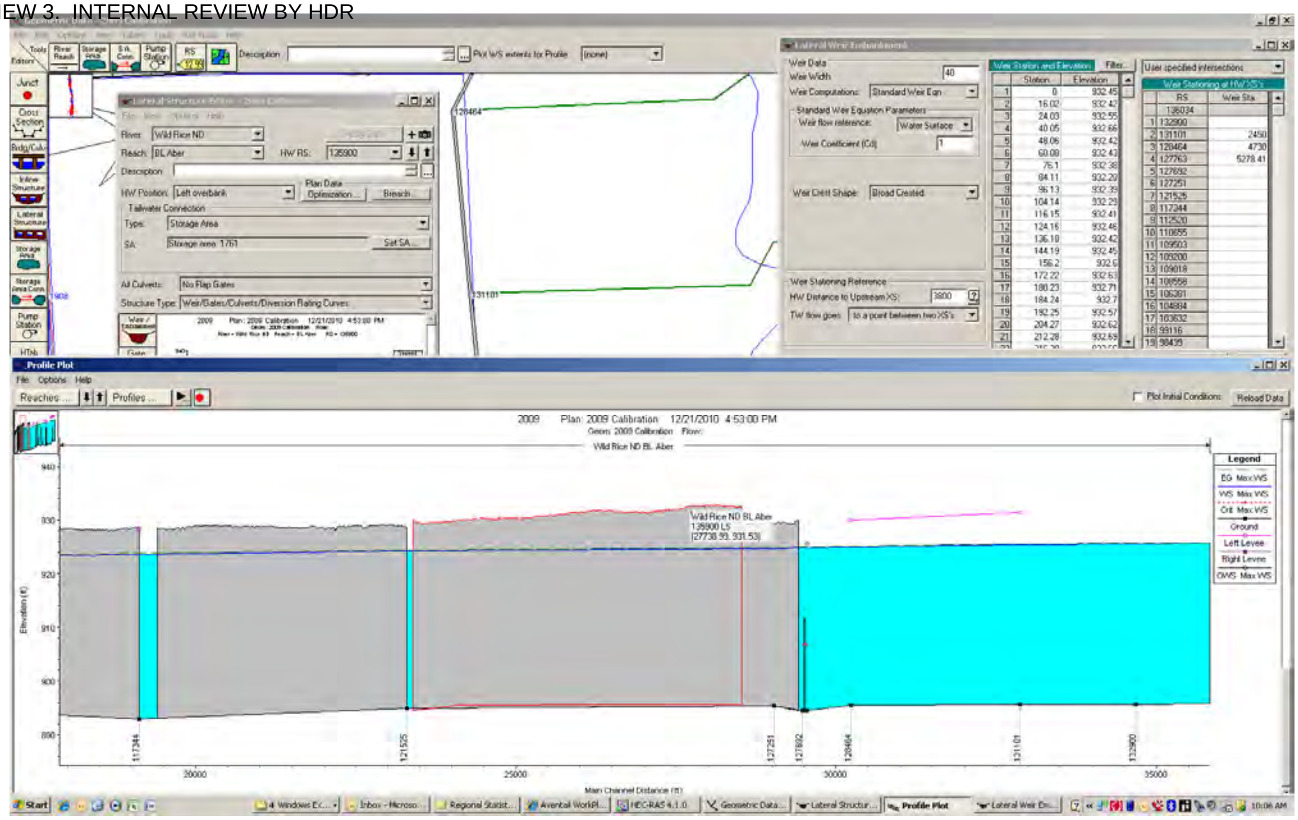

Figure 19 - Wild Rice LS 135900

29. The top of roadway deck for the bridge at cross section 1558704 (Red River) does not appear to be complete on the right side.

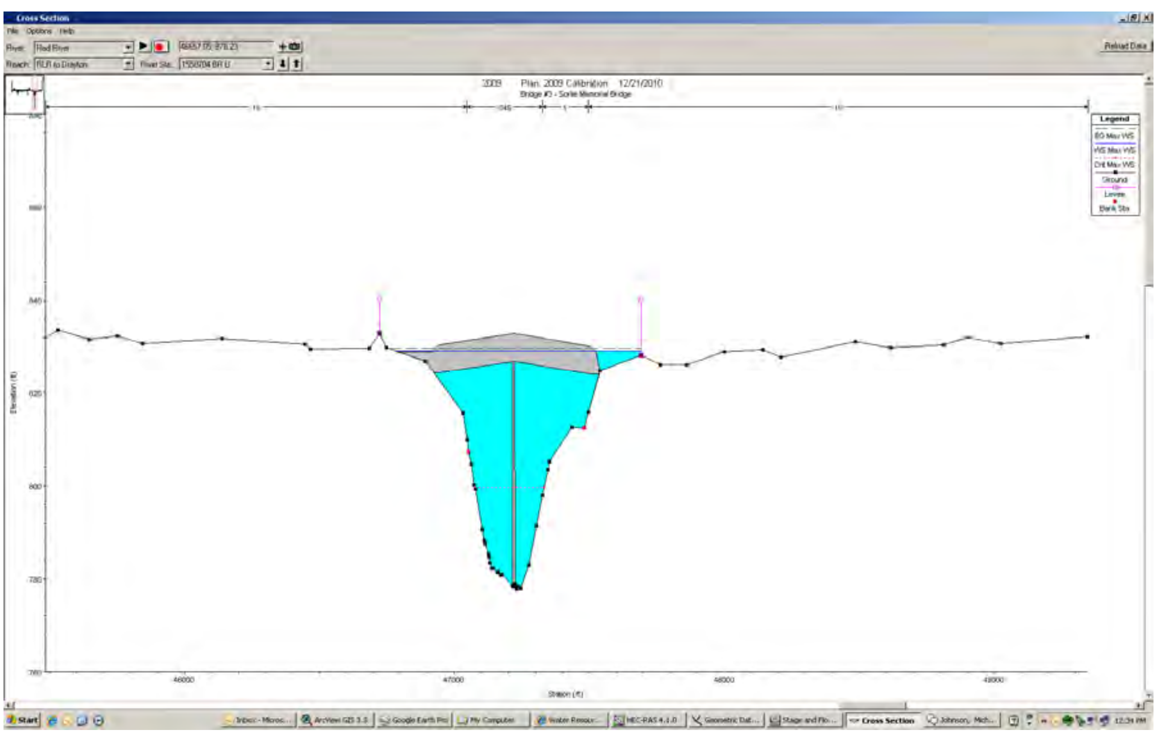

Figure 20 - Red River Bridge #3

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30. The culverts on Drain 34 (i.e., 25749, Figure 18) are shown as being perched above the channel bottom. Is that correct?

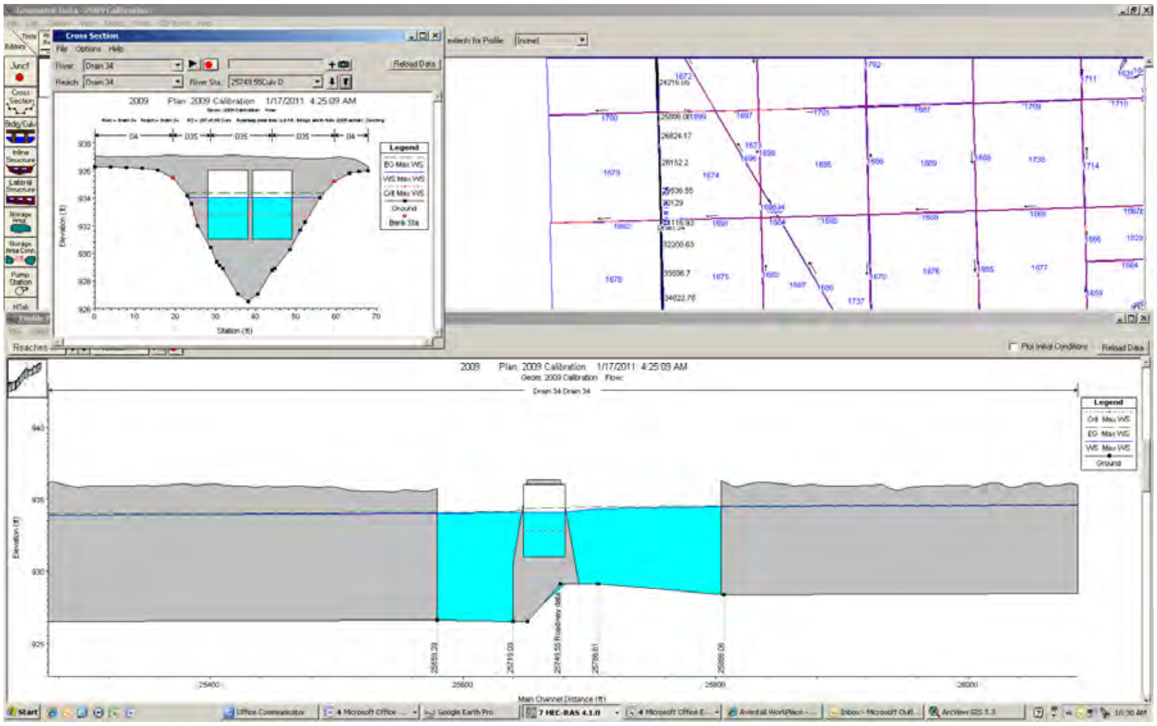

Figure 21 - Drain 34 Perched Culvert

31. There are two bridges in the upstream reach of the South Branch of Elm River that are not included in the model. If the bridge is not going to be modeled, I would suggest the deletion of the cross sections on the upstream side of the first bridge.

32. Many of the bridges are missing from the North Branch of Elm River also. The cross section orientation and inclusion/exclusion of flow in portions of the cross section appear to be inconsistent with flow dynamics.

33. Warning messages suggest that there are several locations within the model where the conveyance ratio or energy losses in excess of one foot suggests that additional cross sections may be needed. Adding additional cross sections at some of these locations could improve model stability.

34. The 2009 event had a peak discharge and volume that is substantially less than the 100-year and 500-year volume. There is a lack of sufficient data to perform a validation or calibration for events in this range of magnitude.

35. The culverts at Drain 37, 37537 are shown to be below grade (Figure 20) which suggest that they should either be partially filled with sediment or the channel data is not capturing the actual invert. This condition exists on some of the other culverts in this reach, suggesting that the channel invert data might not be accurately represented.

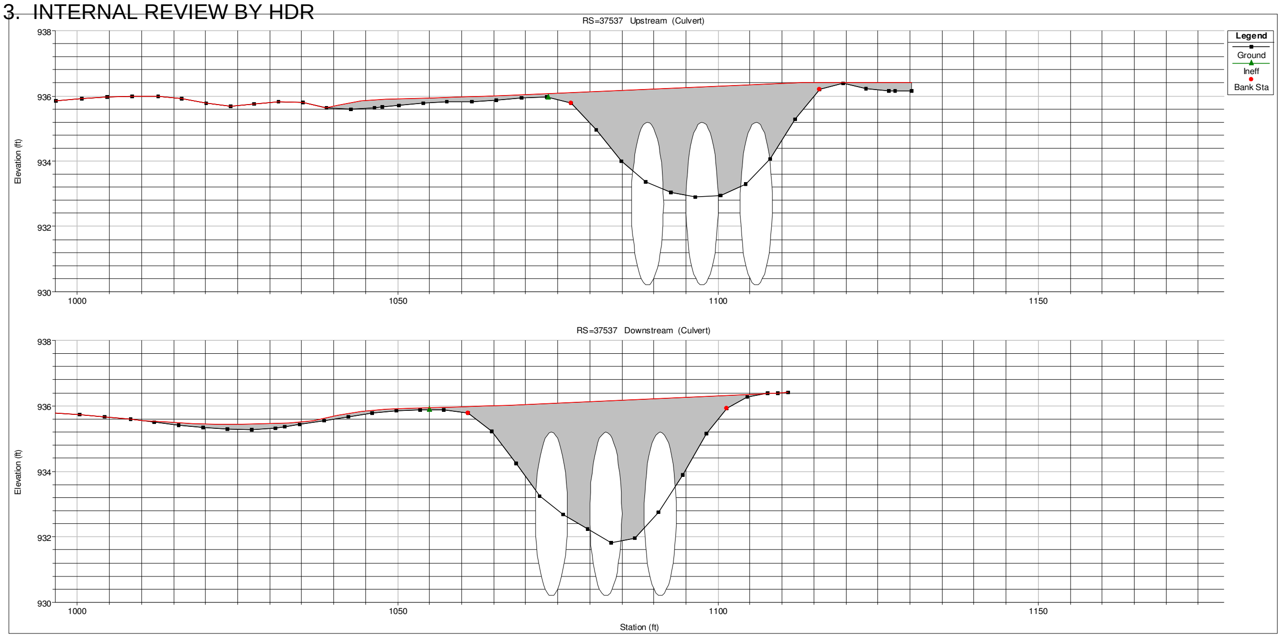

Figure 22 - Drain 37, Culverts at 37537

36. A comparison between the post-processed inundation limits should be made to event photos to validate the flooding extents predicted by the model. This information was not available for my review.

Comments – Synthetic Event Existing Condition

37. The comments above that pertain to the 2009 model are also relevant to the synthetic event models.

38. The hydrologic inputs to the synthetic event models were not reviewed. I understand that an independent review of those data has been performed by others.

39. At some locations, the balanced hydrographs have been entered for comparison. However, in many locations the balanced hydrographs are missing which does not allow a direct comparison between the target discharge and hydrograph volume.

40. A comparison needs to be made between the model results and the peak discharge estimates by the Corps. A quick comparison of the 100-year model results show some discrepancies between the two sources. The hydrology report prepared by the Corps provides a 100-year peak flow estimate of 112,000 cfs at Grand Forks and 67000 at Halstad. The model shows a peak flow of 108,000 and 71,400 cfs at these locations, respectively. How will these variations be reconciled?

41. It is not clear why the flows are being constrained by artificial levees in the Grand Forks Reach (Figure 23 and 24). The aerial photos do not reveal any levees at the existing channel banks. This may be the cause for the higher predicted stages that can be observed in Figure 7.

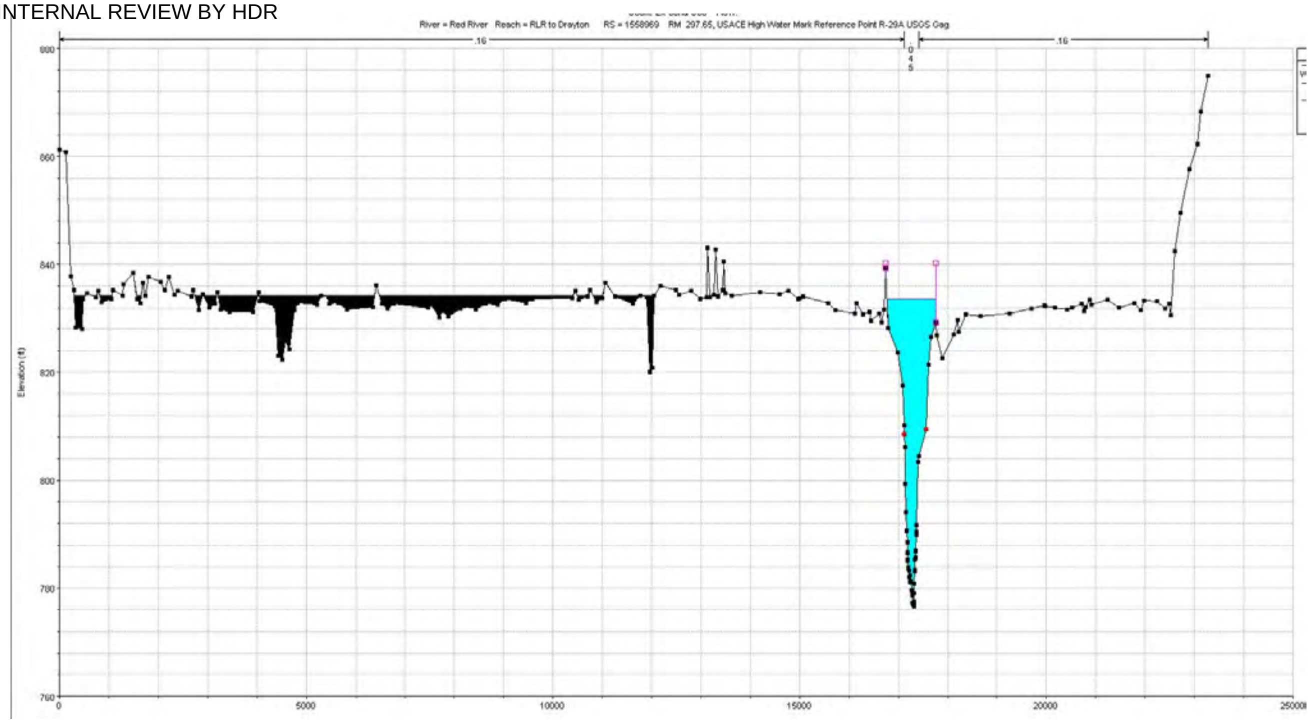

Figure 23 - Artificial Levees in Grand Forks

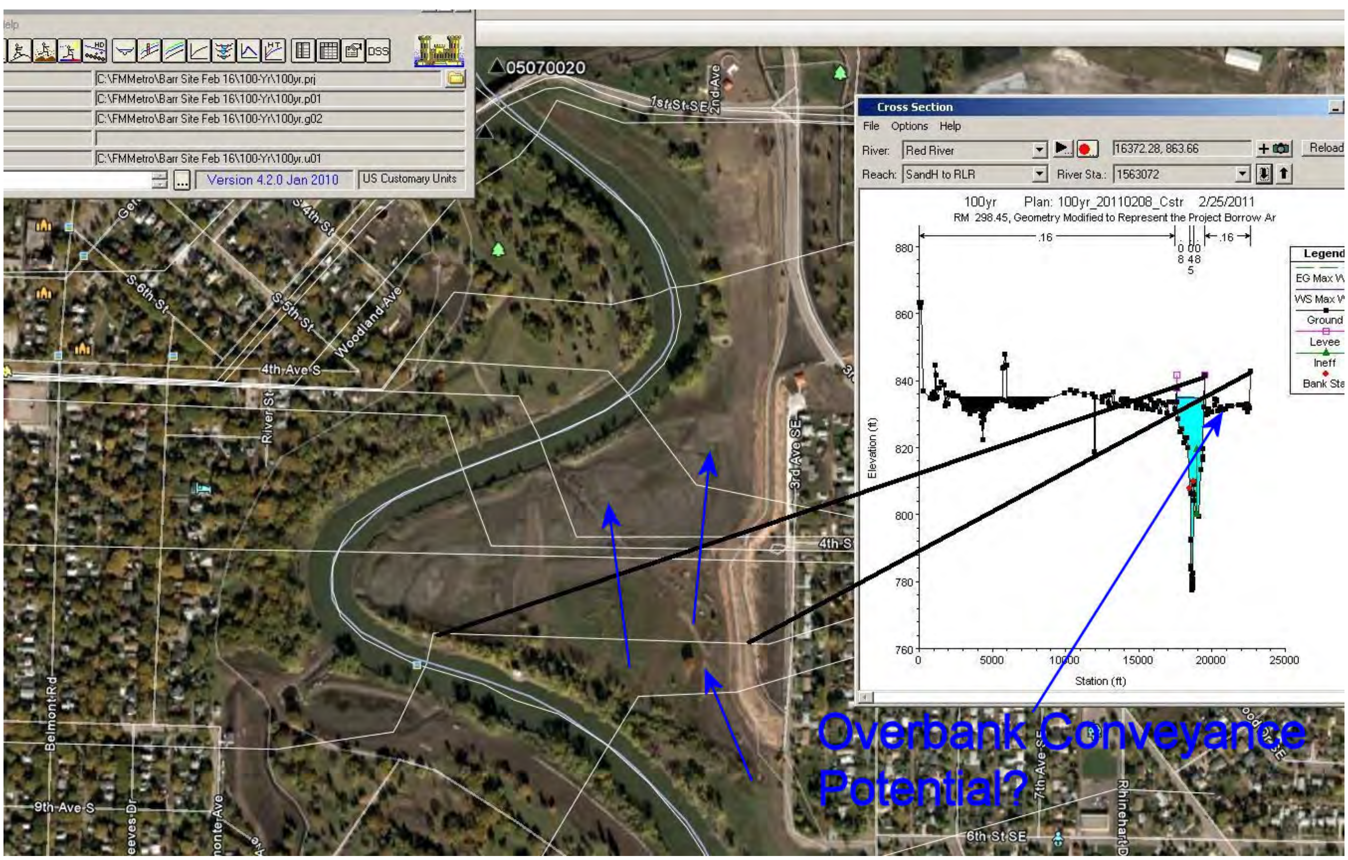

Figure 24 - Grand Forks Reach - Question Pertaining to Artificial Levees

Comments – Proposed Condition for LPP

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

43. The 100-year model goes unstable near the very end of the simulation.

44. I am assuming that independent verification has been performed of the data inputs to assure that the data represented in the model geometry matches the preliminary plans. I did not perform these checks.

45. The design concept requires human intervention based on operational criteria for downstream flows and predicted inflows. Have alternatives been explored that do not require gates? Assuming that other outlet configurations have been explored and found to be deficient, how much risk is associated with the range of potential system flow contributions and variability in how the operating criteria might be employed?

46. Area 1 has a bottom elevation that is 25 to 30 feet higher than the diversion channel. Would additional storage benefit be obtained from lowering the bottom of this storage area to gain additional storage volume?

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

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

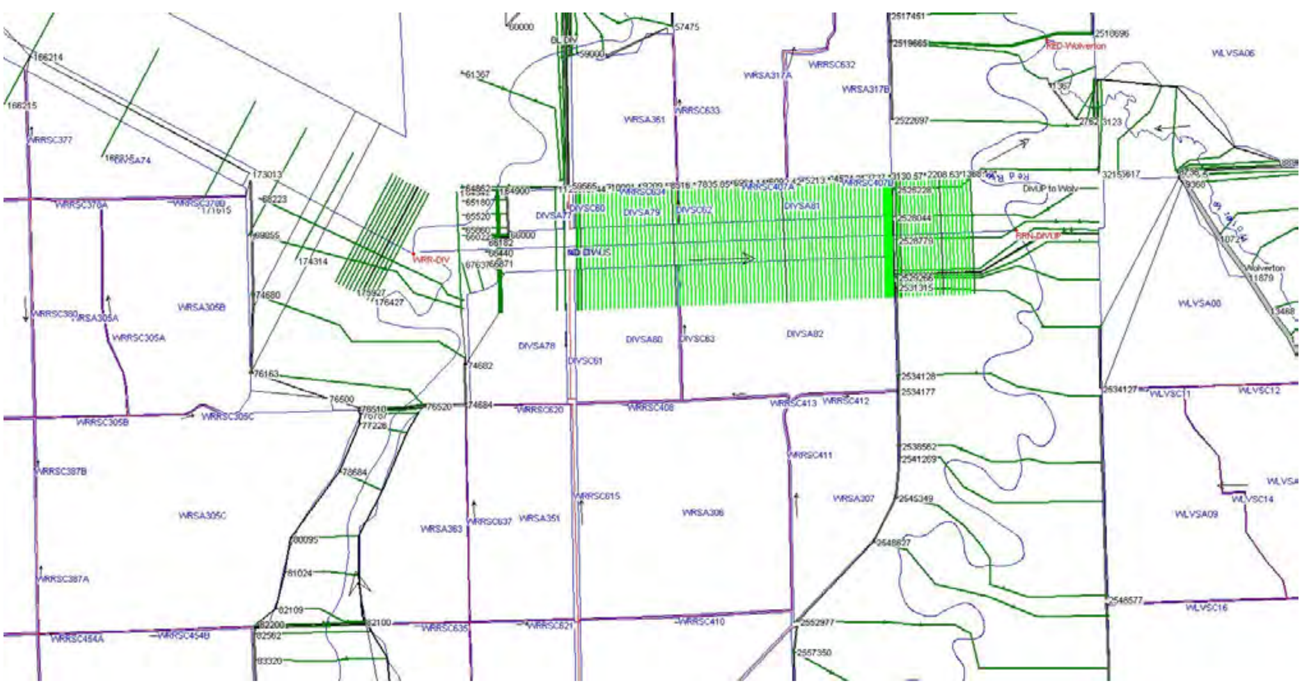

Figure 25 - Complex flow interactions at diversion features

49. A very large flow value is reported for several of the storage area connections such as DRASC33 (see Figure 26) and many of the OSLSC connections (Figure 27) that should be checked.

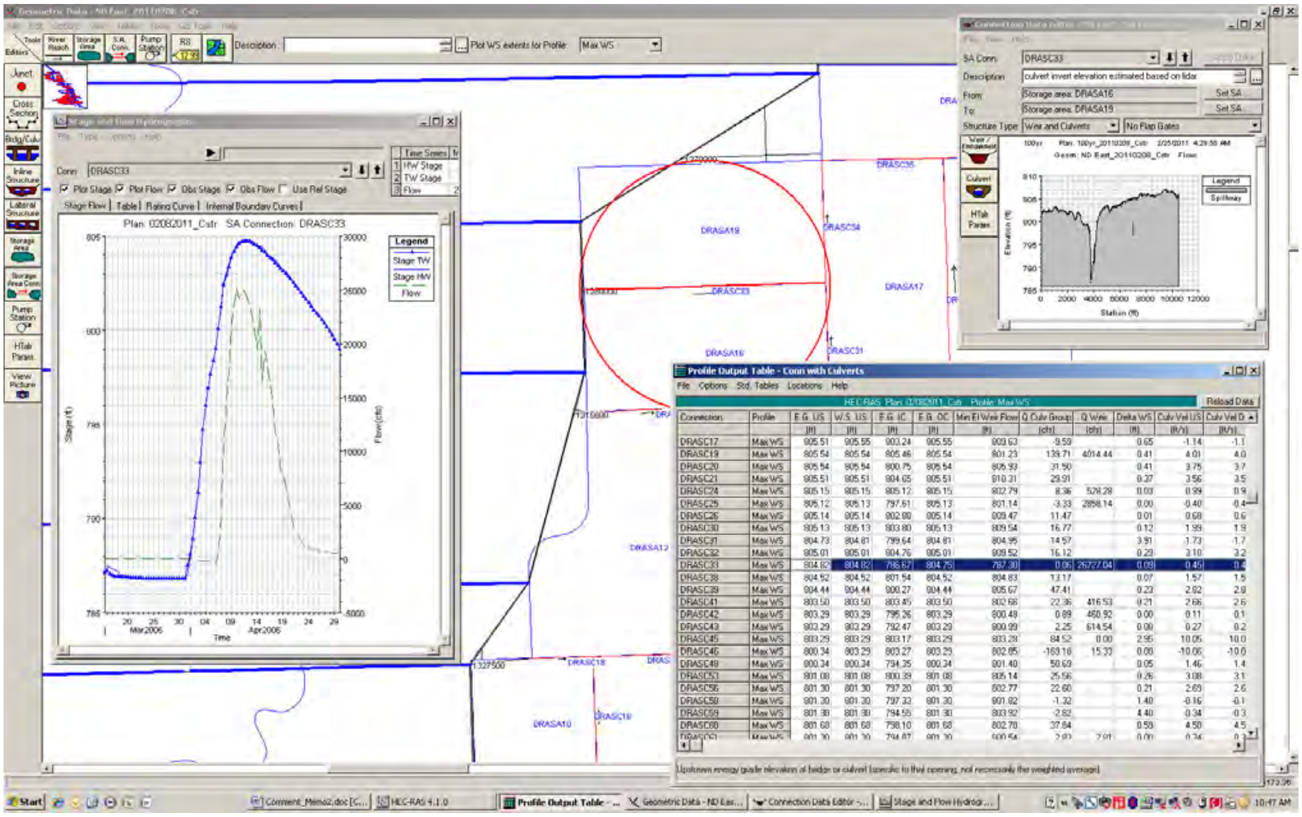

Figure 26 - Suspect Storage Area Connection Discharge Value

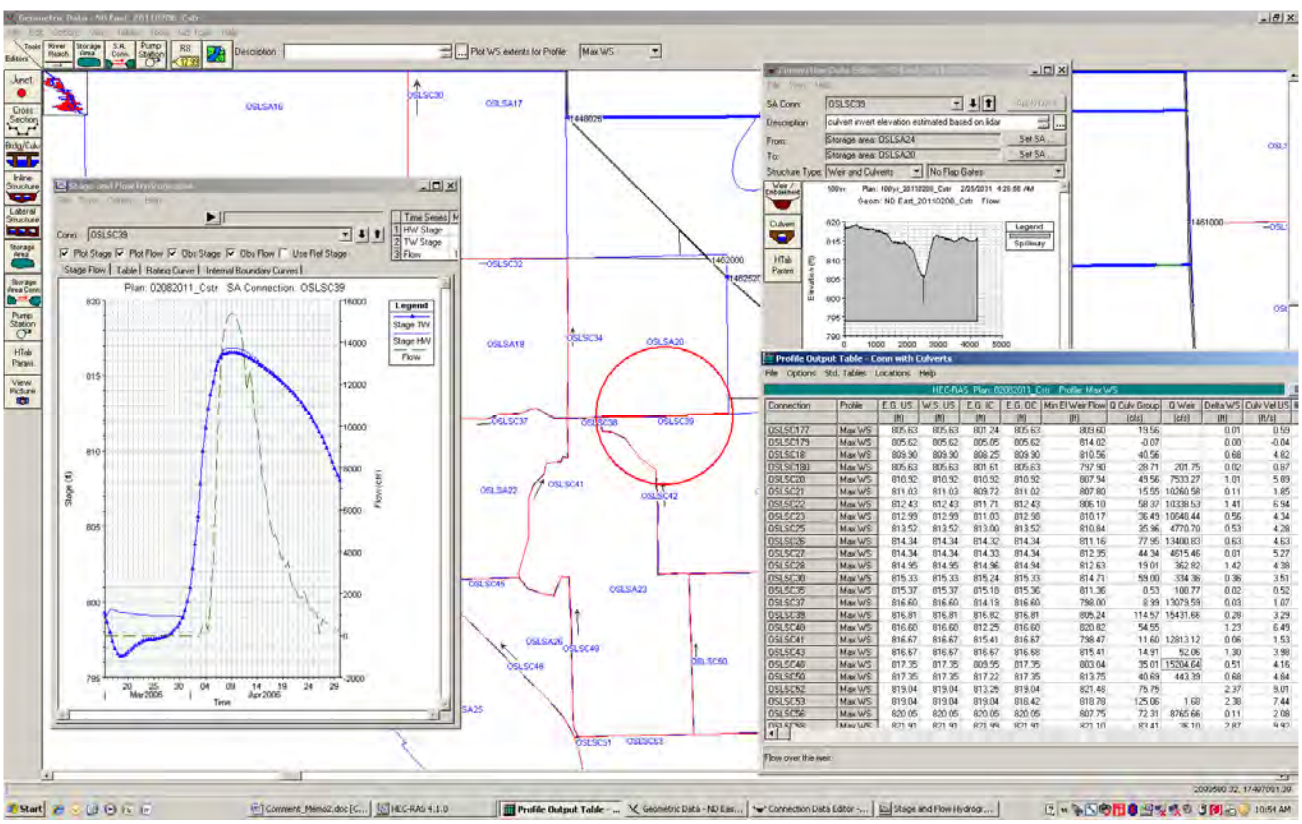

Figure 27 - Suspect Overflow Rates on OSOLC Connections

50. Attached are some locations in the post-processing of the results that appear to be suspect. In Figure 28 and 30, the abrupt edges either mean that you have a roadway or other "levee-like" structure that is impounding the water or there is overflow that is not being accounted for. If it is an artificial impoundment, how will we ultimately need to treat an embankment of unknown integrity?

51. In other locations, the mapping shows areas of isolated ponding with no apparent way for water to get there (Figure 29). Those locations should be verified.

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

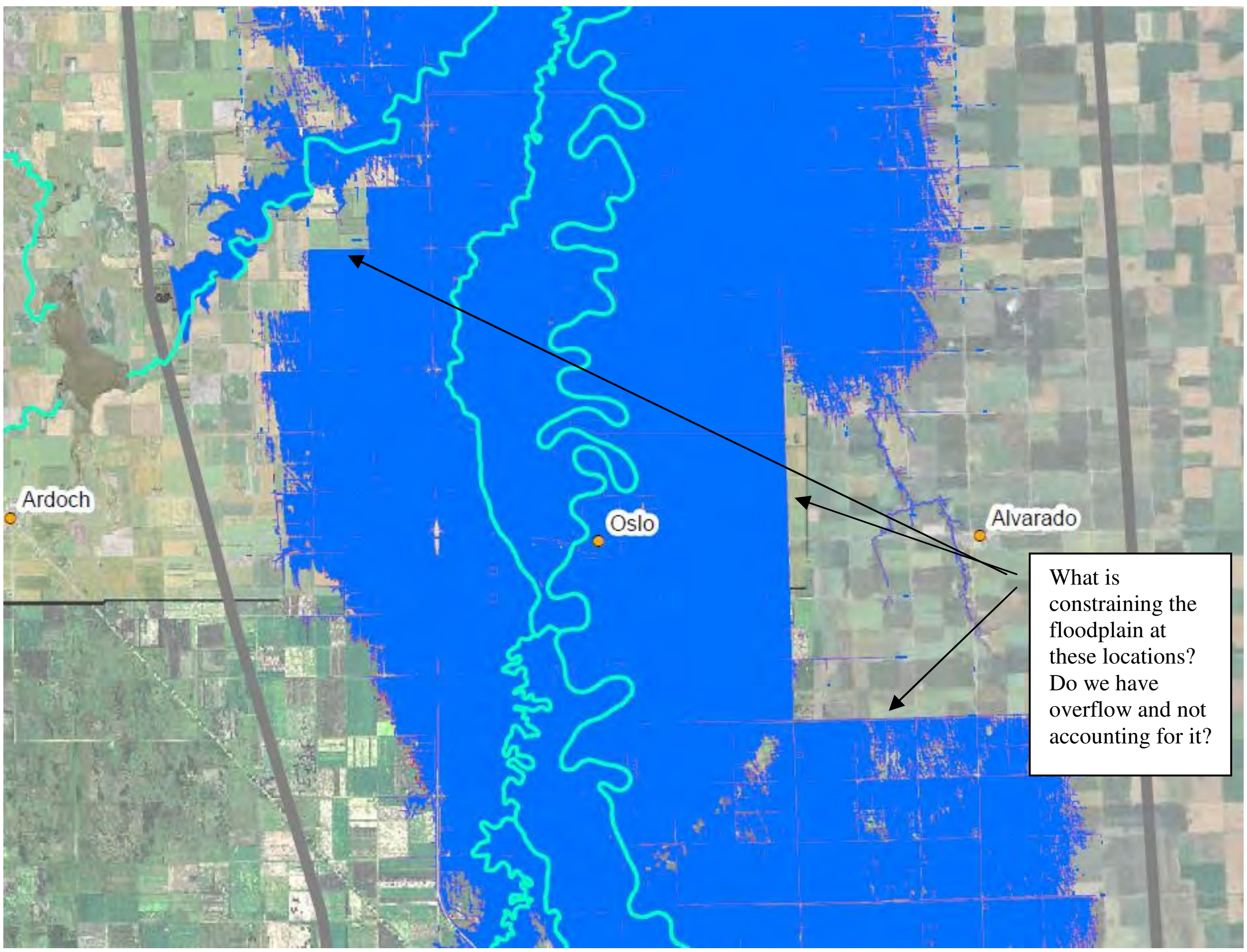

Figure 28 - 100-Year Inundation Limits

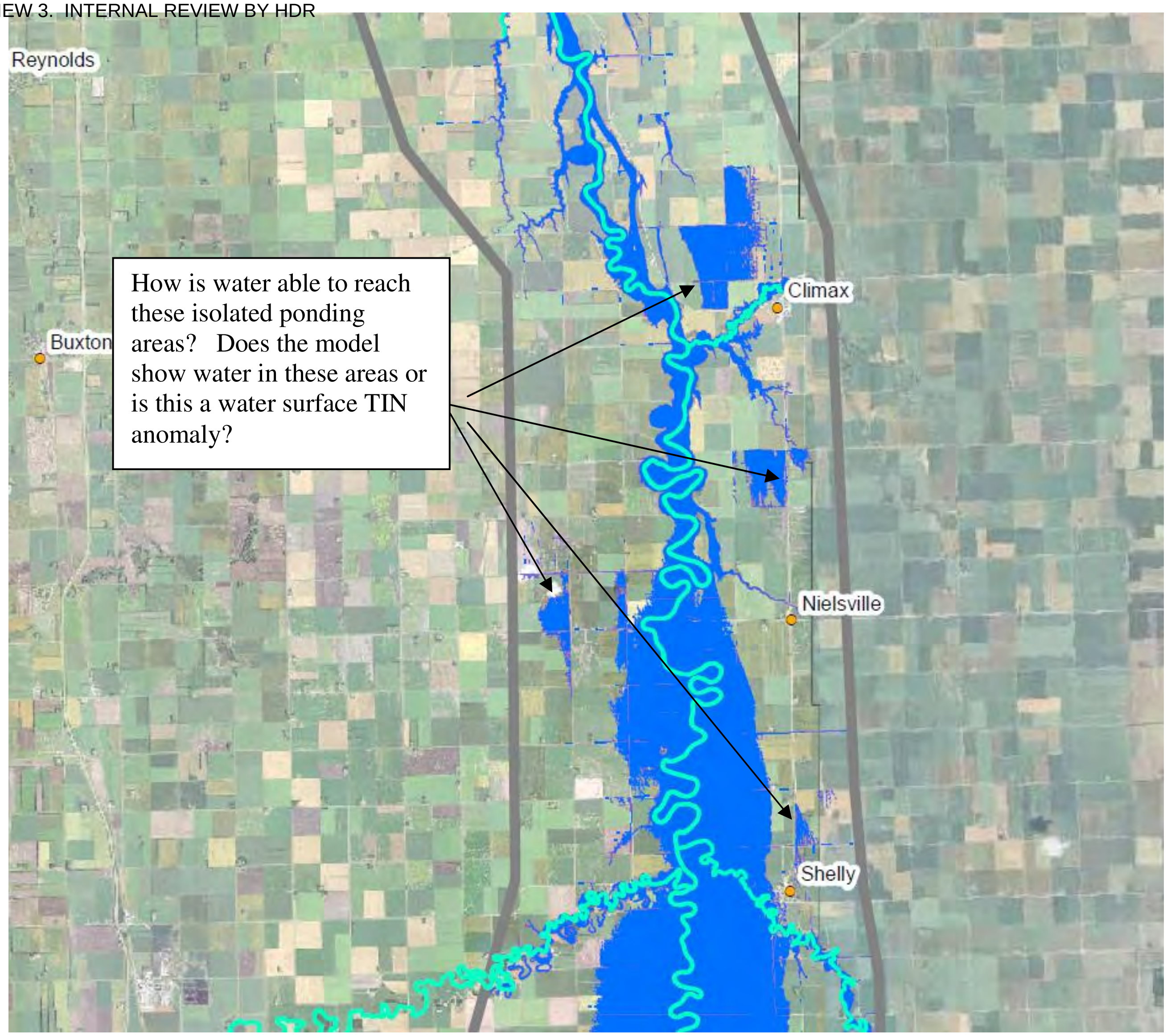

Figure 29 - 100-year Inundation Limits

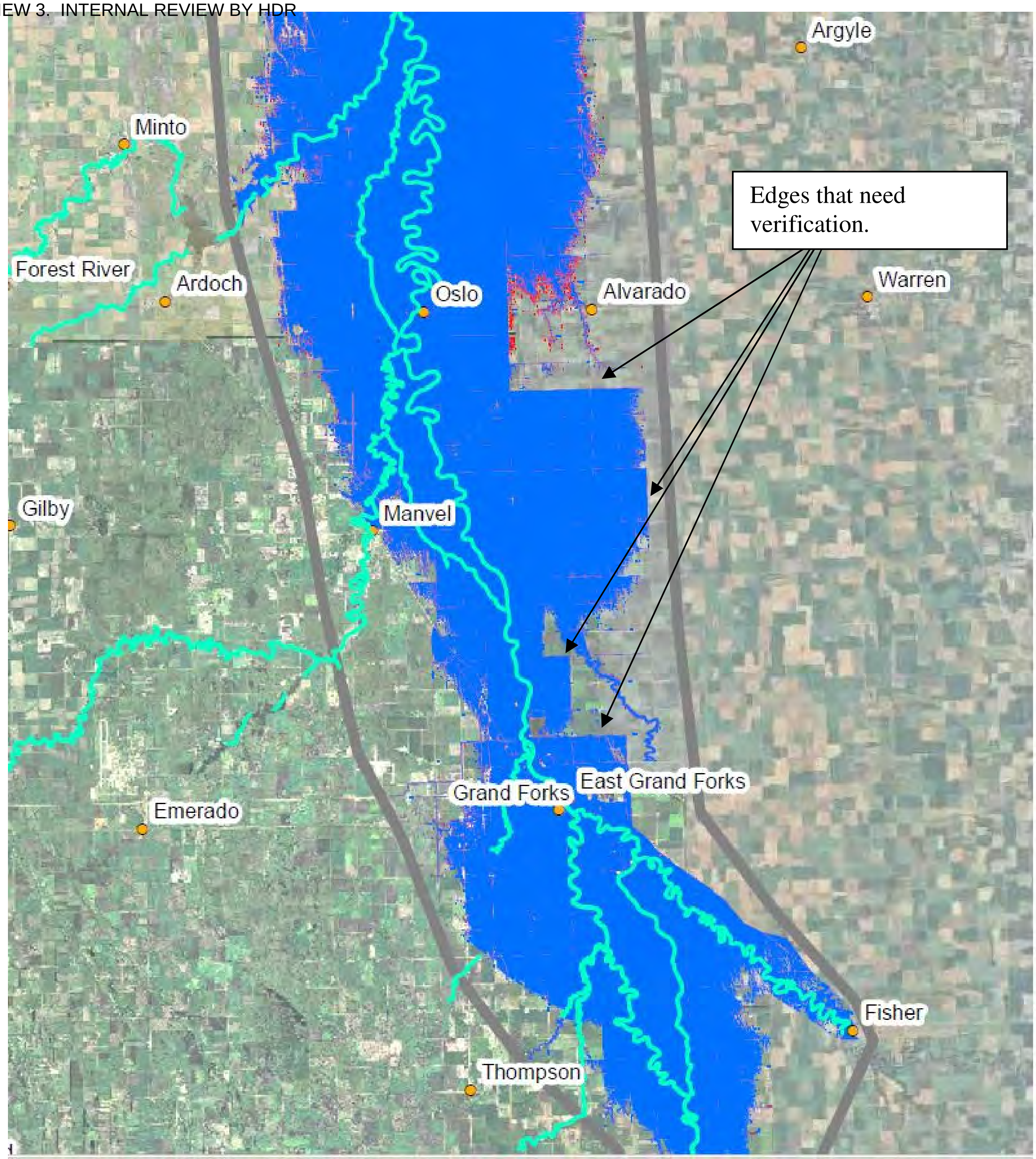

Figure 30 - 500-year Inundations Limits

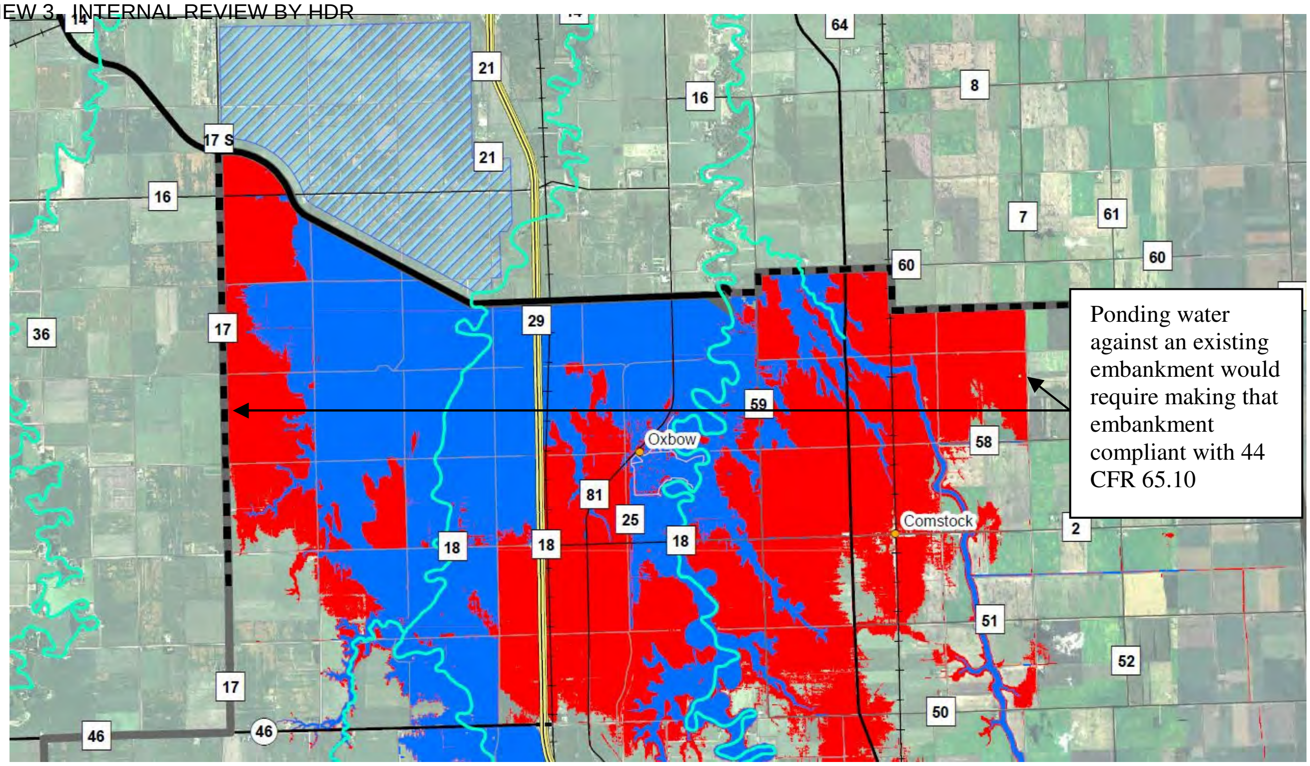

Figure 31 - 100-Upstream Post-Project Flood Limits