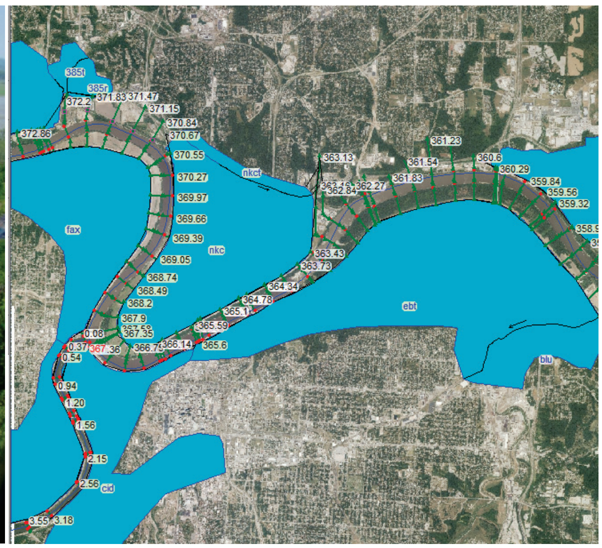
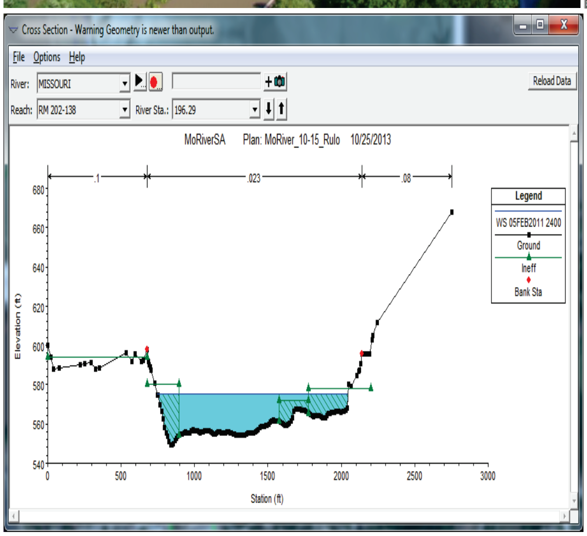
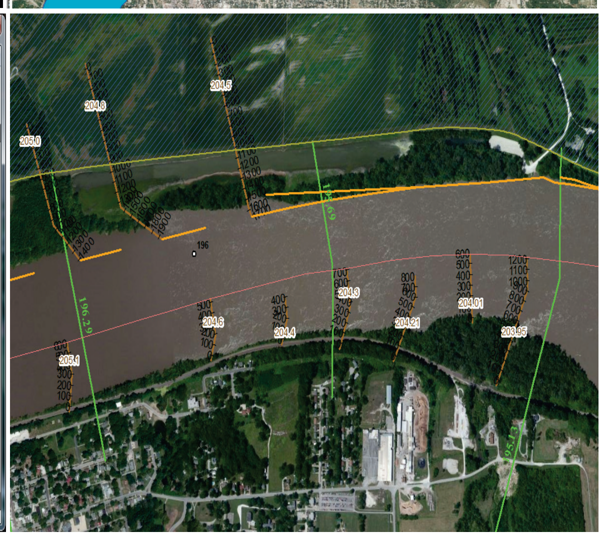
Missouri River Recovery Program Management Plan Environmental Impact Statement
Existing Conditions Unsteady HEC-RAS Model Calibration Report

US Army Corps of Engineers ®







July 2018

USACE, Northwestern Division
Omaha and Kansas City Districts

EXECUTIVE SUMMARY

A set of existing conditions HEC-RAS unsteady flow models was developed for the mainstem of the Missouri River as part of the larger Missouri River Recovery Program (MRRP). The models are designed to support modeling needs associated with the Management Plan (ManPlan) and integrated Environmental Impact Statement (EIS). The project involves the creation of a detailed suite of models for the Missouri River basin that will aid in evaluating alternative jeopardy avoidance strategies for the least tern, piping plover and pallid sturgeon. A key objective of the HEC-RAS model development was to simulate the full range of alternatives proposed for evaluation, while limiting complexity of the model(s) so that they could be developed in a reasonable time period yet have sufficient quality and accuracy to support a quantitative assessment of effects to human considerations and species ecological needs.

The HEC-RAS models documented in this report represent existing conditions on the Missouri River only. Modifications to the models to represent no action, future-without-project conditions and potential future-with-project river management alternatives will be made in futures phases of the ManPlan.

Five separate HEC-RAS models were developed for the the Missouri River between Ft. Peck Dam in Montana and the Mississippi River. Two Reaches were deferred due to the lack of riverine conditions between the dams. The HEC-RAS models have been designed to represent current river conditions and have been calibrated to river stages for time periods that are contemporaneous with those conditions.

Unsteady flow analysis was chosen as the method of hydraulic modeling due to the need under the ManPlan to analyze time series stage and flow data. Both the biological and human considerations are strongly impacted by the timing of river flows.

A large geographic scope, varied geographic conditions and complex system of river reaches, reservoirs, levees and navigation structures, coupled with a dynamic river system, present significant modeling challenges. The Missouri River is 2,341 miles long and drains one sixth of the contiguous United States, an area of 529,350 square miles. The Missouri River mainstem reservoir system, which became fully operational in 1967, includes six Corps of Engineers mainstem dams with a total storage capacity of 73.1 million acre-feet (MAF) and carry-over storage of 39 MAF of water, which makes it the largest reservoir system in North America.

Although the ManPlan is focused on the main stem Missouri River, the hydrologic and hydraulic response of the river is influenced by the watershed as a whole. Under the scope of the ManPlan studies not every sub-watershed or tributary needed to be evaluated explicitly in the HEC-RAS model. Large areas of the watershed, including some upstream reservoirs, do not have sufficient water management potential to significantly support proposed ManPlan jeopardy avoidance alternatives, or create significant social or economic impacts, within the scope of ManPlan. The hydrology of these parts of the Missouri River basin was captured as inputs to the HEC-RAS model through analysis of historic gage data and outputs from reservoir models. In

some cases portions of tributaries are included in the HEC-RAS models in order to more accurately route flows from gages to the mainstem to improve model calibration.

All HEC-RAS models were constructed based on the NAVD-88 vertical datum. The large number of river miles and the variability in quality and quantity of terrain, hydrographic and stream gage data make a totally consistent approach to model geometry development difficult across time and river reach. The age and precision of terrain data varies between the individual HEC-RAS model reaches based on data availability as well as on the magnitude of recent changes in the river due to large floods. Development of the model for the river below Rulo, Nebraska was already under way at the time of the 2011 flood. The modeling team determined that channel and overbank conditions on this reach of the river were not changed significantly by the 2011 flood and so the model was completed and calibrated using the pre-2011 bathymetry and flows. In contrast, the 2011 flood caused significant geometry changes in the channel, and less frequently, in the overbank terrain between Ft Peck Dam and Rulo, Nebraska. New bathymetry and LiDAR were collected following the flood for other purpose, that was available to use in development of the HEC-RAS model above Rulo. Overbank and bathymetry data have been merged in the models to give the most accurate representation of river conveyance conditions over a wide range of flows.

Models upstream of Rulo, Nebraska were calibrated to the measured 2011 and 2012 water surface profiles (WSP) and observed stage-gage data for the Missouri River. The computed water surface profile was within +/-1 ft along the entire reach and in the range of +/- 0.5 ft for about 50% to 75% of the reach. The river reach below Rulo, Nebraska was calibrated using USGS instantaneous gage data for a six year period from October 1, 2007 to September 30, 2013.

Because a longer calibration window was available below Rulo the model reproduces the present-day stage-discharge relationships at USGS gages on the Missouri River over a wider range of time and river conditions. On average, the model below Rulo has a mean stage error of 0.1 feet with a root mean square stage error of 0.8 feet, 86% of the time the computed stage is within 1-ft of observed, and 97% of the time it is within 2-ft of observed.

Model calibration is considered to be very good for a set of hydraulic models of this magnitude and is consistant with the objective of evaluating alternatives under the MRRP.

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ACRONYMS

BiOp.. .... Biological Opinion of the US Fish and Wildlife Service

BSNP.. .... Bank Stabilization and Navigation Project

EIS……………………Environmental Impact Statement

HEC .... Hydrologic Engineering Center

ManPlan………………Management Plan

MRRP .... Missouri River Recovery Program

NWK………………… Northwest Division Kansas City District

NWO………………… Northwest Division Omaha District

POR .... Period of Record

RAS.......................... River Analysis System

ResSim………………...Reservoir Simulation Software (by HEC)

UMRSFFS................ Upper Mississippi River System Flow Frequency Study

USACE……………… United States Army Corps of Engineers

USGS....................... United States Geological Survey

1 INTRODUCTION

The U.S Fish and Wildlife Service 2003 Amended Biological Opinion (BiOp) concluded that the Corps' operation of the Missouri River Mainstem Reservoir System, the Bank Stabalization and Navigation Project (BSNP) and the Kansas River Reservoir System jeopardizes the continued existence of the endangered pallid sturgeon, interior least tern and threatened piping plover. The Missouri River Recovery Program (MRRP) will address the environmental needs of the Missouri River as required for BiOp compliance while allowing the Corps to operate the Missouri River for all eight congressionally authorized purposes. The Missouri River Recovery Management Plan (ManPlan) and integrated Environmental Impact Statement (EIS) is being developed through the National Environmental Policy Act to address mitigation efforts, BiOp compliance, and cumulative effects of Corps actions along the river.

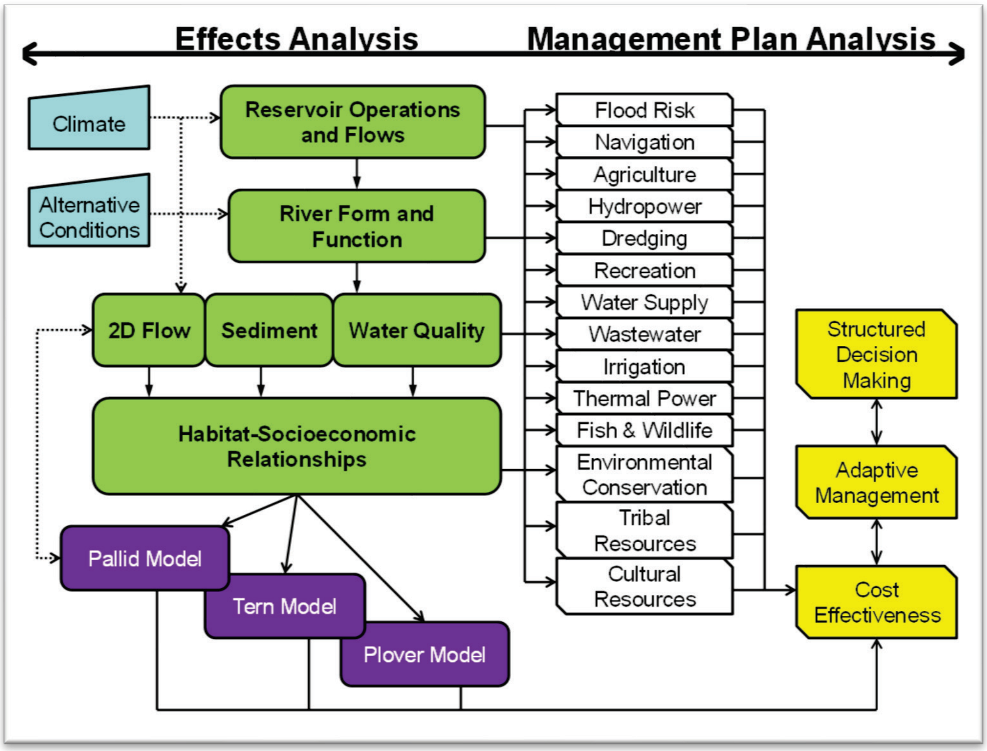
The Corps of Engineers Hydrologic Engineering Center's River Analysis System model (HEC-RAS Version 4.2 Beta and 5.0 Beta) is being used to model unsteady flow hydraulics for the Missouri River. The HEC-RAS model is part of a larger study effort using a variety of conceptual and quantitative models to simulate the effects of changes to river management under the ManPlan on species recovery as well as effects to human considerations. These changes in river management include both physical changes to the river channel as well as changes to reservoir and flow management. The end product of the HEC-RAS study component will be a set of hydraulic models for the Missouri River from Montana to the Mississippi River. These models will simulate how proposed alternatives and management measures will impact river stage and discharge over a wide range of basin hydrologic conditions.

Development of the hydrologic and hydraulic modeling component of the larger ManPlan and EIS consists of three parts:

1. Development of reservoir simulation (ResSim) models for managed federal reservoirs that impact management for the three species. This model will be used to assess the benefits and effects of changes in water management (reservoir operations) at these reservoirs. HEC-ResSim was chosen for this modeling**.**
2. Development of hydraulic models for free-flowing reaches of the river. Unsteady HEC-RAS was chosen for this modeling. HEC-RAS will be used to more accurately route discharges from reservoirs and tributaries to points downstream and to simulate impacts of mechanical changes in river channel geometry.
3. Development of a complete, sufficiently long period of gage records for the Missouri River and its principle tributaries, to be used in the hydrologic and hydraulic models. Regression methods were used to estimate missing data in older parts of the gage record. The goal was to have a record that realistically represents runoff conditions in the basin back to 1930. The record was also adjusted for depletions and other significant changes in the basin over time.

Outputs from the hydrologic and hydraulic modeling effort will support conceptual and quantitative ecological models for evaluating species responses to management actions in the Environmental Effects Analysis portion of the study, and evaluation of the effects to basin stakeholder interests and authorized purposes in the Management Plan Analysis. Figure 1 illustrates the modeling framework for the Effects Analysis and Management Plan Analysis.

Figure 1. Modeling Framework for Effects Analysis and Management Plan Analysis



(Fischenich, 2014)

1.1 OBJECTIVES

In order to meet the requirements of the MRRP, the USACE is undertaking an evaluation of proposed ManPlan actions and alternatives to be implemented over a 15 year horizon. Proposed ManPlan alternatives would be developed using a passive and active adaptive management framework to reduce uncertainty relative to specie-specific actions, aimed at avoiding jeopardy for the Least Tern, Piping Plover, and Pallid Sturgeon.These efforts are supported by an Effects Analysis and a Management Plan Analysis (Fischenich, 2014). HEC-RAS modeling effort provides time series Missouri River flow and stage data to support these analyses.

The very large geographic scope, varied geographic conditions and complex system of river reaches, reservoirs, levees and navigation structures, coupled with a dynamic river system, present significant modeling challenges. A key objective of the RAS model development was to simulate the full range of alternatives proposed for evaluation, while limiting complexity of the models so that they can be developed in a reasonable time period, with sufficient quality and accuracy to support the conceptual and quantitative human considerations and ecological models.

2 BACKGROUND

Development of an unsteady hydraulic model for a large portion of the Missouri River is a significant undertaking. Although there is a long history of model development along various reaches of the river for specific project purposes, this MRRP modeling effort is one the first attempts to tie a set of unsteady hydraulic models of riverine reaches with a set of reservoir management models, using widely distributed, publicly available modeling software. HEC-RAS was chosen as the hydraulic modeling software by a technical working group comprised of members from across the Corps' Northwest Division and the Bureau of Reclamation. In order to develop models of manageable sizes and with reasonable run times, yet with sufficient output detail for the ManPlan study, considerable care was required in defining the physical extent of the HEC-RAS models.

2.1 Basin Description

The Missouri River is 2,341 miles long and drains one sixth of the contiguous United States, an area of 529,350 square miles. Average annual rainfall varies from 8 inches a year to 40 inches a year across the basin, with a total average annual runoff of 25,000,000 acre-feet above Sioux City. The Missouri River mainstem reservoir system, which became fully operational in 1967, includes six Corps of Engineers mainstem dams with a total storage capacity of 73.1 million acre-feet (MAF) and carry-over storage of 39 MAF of water, which makes it the largest reservoir system in North America. Dozens of other Federal dams regulate flow on tributaries to the Missouri River and are managed in concert with the Mainstem dams.

The Missouri River system is managed by the U.S. Army Corps of Engineers to serve eight congressionally authorized project purposes; flood control, navigation, irrigation, hydropower, water supply, water quality, recreation, and fish and wildlife. Runoff from above the Mainstem reservoir system dams is stored in the six reservoirs where it is managed to serve these project purposes. Water is released from the mainstem reservoir system as prescribed by the system's master manual. Figure 2 shows the locations of the Missouri River Mainstem Dams.

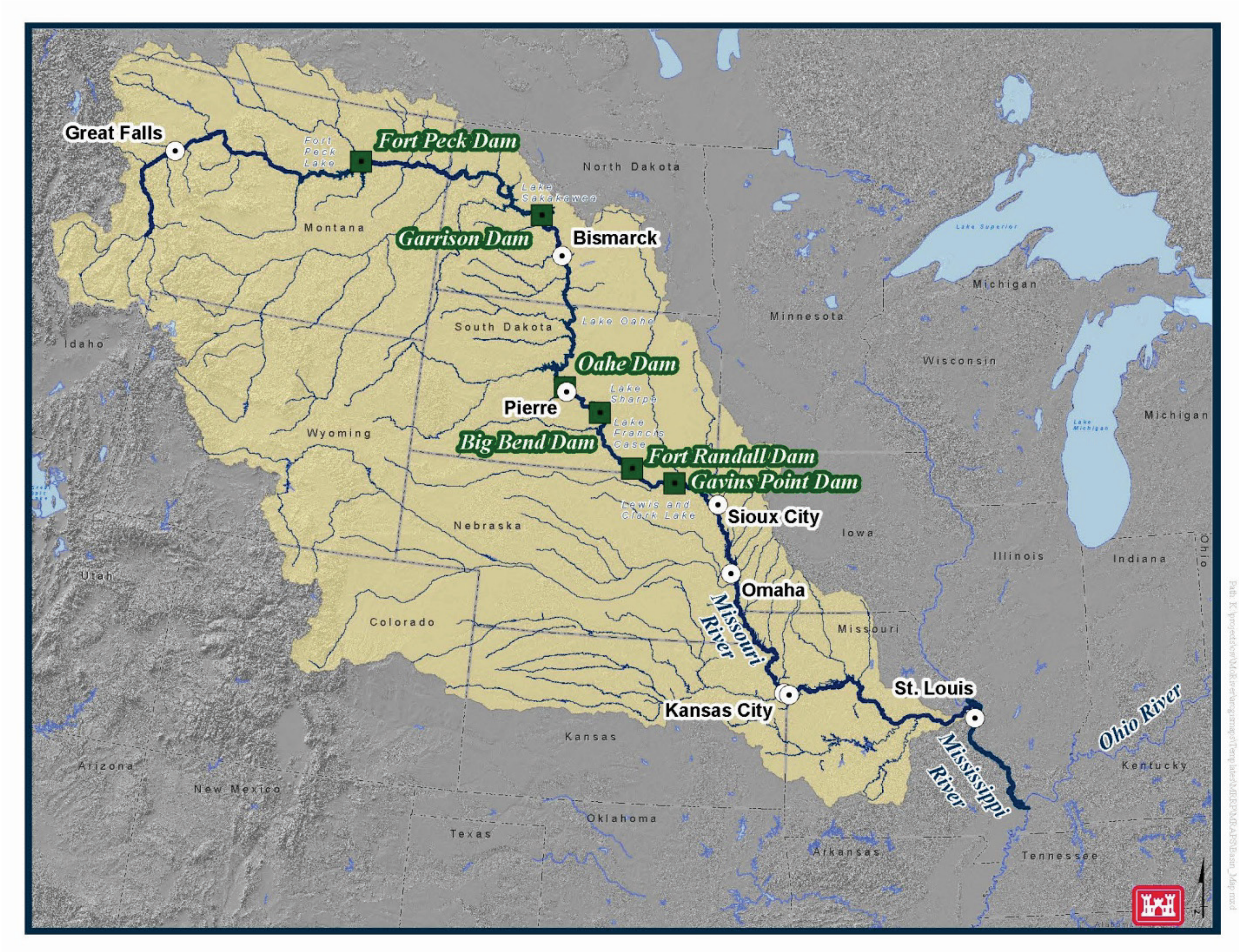


Figure 2. Missouri River Mainstem Reservoirs

2.2 PREVIOUS MODELING

Portions of the Missouri River mainstem and tributaries have been modeled over previous decades using a variety of modeling software developed for specific management purposes. In the mid twentyith Century the Missouri River was included in a physical model of the Mississippi River system built by the Corps' Waterways Experiment Station, which covered 200 acres near Jackson, Mississippi. (Coupe, 2013) This model is no longer operational. In past decades hydraulic models have been developed for various reaches of the river using HEC-2, UNET, HEC-RAS, and other programs developed by the individual Corps Districts. Reservoir management is currently implemented by the Northwestern Division's Reservoir Control Center with the aid of the Daily Routing Model and several other models.

From 1997 through 2003, a UNET model was developed for the Missouri River below Gavins Point, South Dakota as part of a flow frequency study for the Mississippi River System. The intent of the flow frequency study was to revise flood flow estimates for the river and to provide water surface profiles for various return-period floods. (U.S. Army Corps of Engineers, 2003) In the mid 2000's the Corps' Kansas City and Omaha Districts entered an agreement with FEMA Region VII to define a floodway for use in their Digital Flood Insurance Rate Map updates. Cross sections from the Flow Frequency Study UNET model were used to construct a steady flow HEC-RAS model below Sioux City, Iowa. The model was calibrated to the 1% annual return period profile from the UNET model before development of the floodway. Since development of the Floodway model, additional HEC-RAS, sediment and 2-dimensional models have been developed for limited reaches of river in support of specific flood damage reduction and fish and wildlife restoration projects. Knowledge of the Missouri River gained through development of these models has been incorporated in development of the current HEC-RAS model.

2.3 Hydraulic Model Selection for the Current Study

A hydrology and hydraulics technical working group, initiated under the Missouri River Ecosystem Restoration Program, recommended unsteady HEC-RAS as the preferred hydraulic modeling software for evaluating management strategies on the Missouri River. This working group was made up of representatives from the Corps, Kansas City and Omaha Districts, the Northwestern Division's Reservoir Control Center and the Bureau of Reclamation. The recommendations of the working group were coordinated with the district management chains. These recommendations have been carried forward to the MRRP ManPlan study. The HEC-RAS software was chosen based on: capability to model the large, complex river system, widespread use and acceptance both within and outside of the Corps of Engineers (transparency), compatibility with other HEC economic and ecological analysis software, thorough documentation, and availability of long term technical support.

The Hydrologic Engineering Center HEC-RAS web page describes the HEC-RAS software package as follows:

> *HEC-RAS is designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. The HEC-RAS system contains four one-dimensional river analysis components for: (1) steady flow water surface profile computations; (2) unsteady flow simulation; (3) movable boundary sediment transport computations; and (4) water quality analysis. A key element is that all four components use a common geometric data representation and common geometric and hydraulic computation routines. In addition to the four river analysis components, the system contains several hydraulic design features that can be invoked once the basic water surface profiles are computed.*
>
> *Steady Flow Water Surface Profiles -This component of the modeling system is intended for calculating water surface profiles for steady gradually varied flow. The system can handle a full network of channels, a dendritic system, or a single river reach. The steady flow component is capable of modeling subcritical, supercritical, and mixed flow regimes water surface profiles.*
>
> *Unsteady Flow Water Surface Profiles - This component of the HEC-RAS modeling system is capable of simulating one-dimensional unsteady flow through a full network of open channels. The unsteady flow equation solver was adapted from Dr. Robert L. Barkau's UNET model (Barkau, 1992 and HEC, 1997). The unsteady flow component was developed primarily for subcritical flow regime calculations. However, with the release of Version 3.1,*

> *the model can now performed mixed flow regime (subcritical, supercritical, hydraulic jumps, and draw downs) calculations in the unsteady flow computations module.*
>
> *The hydraulic calculations for cross-sections, bridges, culverts, and other hydraulic structures that were developed for the steady flow component were incorporated into the unsteady flow module. Special features of the unsteady flow component include: Dam break analysis; levee breaching and overtopping; Pumping stations; navigation dam operations; and pressurized pipe systems.* (HEC-RAS)

Only one-dimensional HEC-RAS analysis was chosen for this part of the ManPlan study. The use of 1-Dimensional models has been the standard of practice in modeling long stream reaches for decades. For most Corps of Engineers planning studies this is a technically adequate and economical approach. one-dimensional models do, however, have limitations that are relevant to habitat considerations that have bearing on ManPlan study. Corps of Engineers Engineer Manual 1110-2-1416, River Hydraulics, states that in one-dimensional models “Stage, velocity and discharge vary only in the streamwise directions…More detailed analysis of flow velocities and directions requires representation of flow physics in two an sometimes three dimensions.” (U.S. Army Corps of Engineers, 1993). Determination of flow direction and velocities for evaluation of habitat suitability may in some instances require detailed analysis beyond the capabilities of the 1-Dimentional HEC-RAS model. In these instances two-dimensional models may need to be constructed for selected river reaches where more detail is required.

2.4 Model Extents

The Missouri River Basin encompasses over one half million square miles. Although the MRRP is focused on the main stem Missouri River, the hydrologic and hydraulic response of the river is influenced by the watershed as a whole. Under the scope of the ManPlan Study not every sub-watershed or tributary needs to be evaluated explicitly in the HEC-RAS model. Large areas of the watershed, including some upstream reservoirs, do not have sufficient water management potential to significantly support restoration alternatives, create significant social or economic effects, or be significantly impacted by restoration alternatives within the scope of ManPlan. The hydrology of these parts of the basin is captured as inputs to the HEC-RAS model through analysis of historic gage data and outputs from reservoir models. In some cases portions of tributaries are included in the HEC-RAS models in order to more accurately route flows from the tributary gage to the mainstem and improve model calibration.

Reservoirs that have potential to impact management for the three listed species are modeled using the HEC Reservoir Simulation Model (HEC ResSim) and used as inputs to the HEC-RAS models. The ResSim modeling effort is described in separate modeling reports.

3 MODELING APPROACH

Unsteady flow analysis was chosen as the method of hydraulic modeling due project requirements under the ManPlan to analyze time series stage and flow data. Both the biological considerations (for example, seasonal habitat requirements) and the human considerations (for example, agricultural impacts) are effects by the timing of river flows.

Varying availability of terrain and bathymetric data, the presence of the Mainstem reservoirs, and the need to take advantage of local knowledge of river conditions led the staff in the Kansas City and Omaha Districts to develop 5 separate models for discrete reaches of the Missouri River. These reaches are; Fort Peck Dam Garrison Dam; Garrison Dam Oahe; Fort Randall Dam to Gavins Point Dam; Gavins Point Dam to Rulo, Nebraska (district boundary) and Rulo, Nebraska the mouth of the Missouri River at St. Louis, MO. Because the boundary between the Kansas City and Omaha Districts is at at Rulo Nebraska, the Gavins Point to the mouth overlap from Nebraska City, NE to St. Joseph, MO in order to facilitate calibration and a clean transition of flows between the two model reaches. Figure 3 lays out the locations of the individual HEC-RAS models. The Oahe to Big Bend and Big Bend to Randall reaches were not modeled in HEC-RAS due to the lack of riverine conditions between the dams.

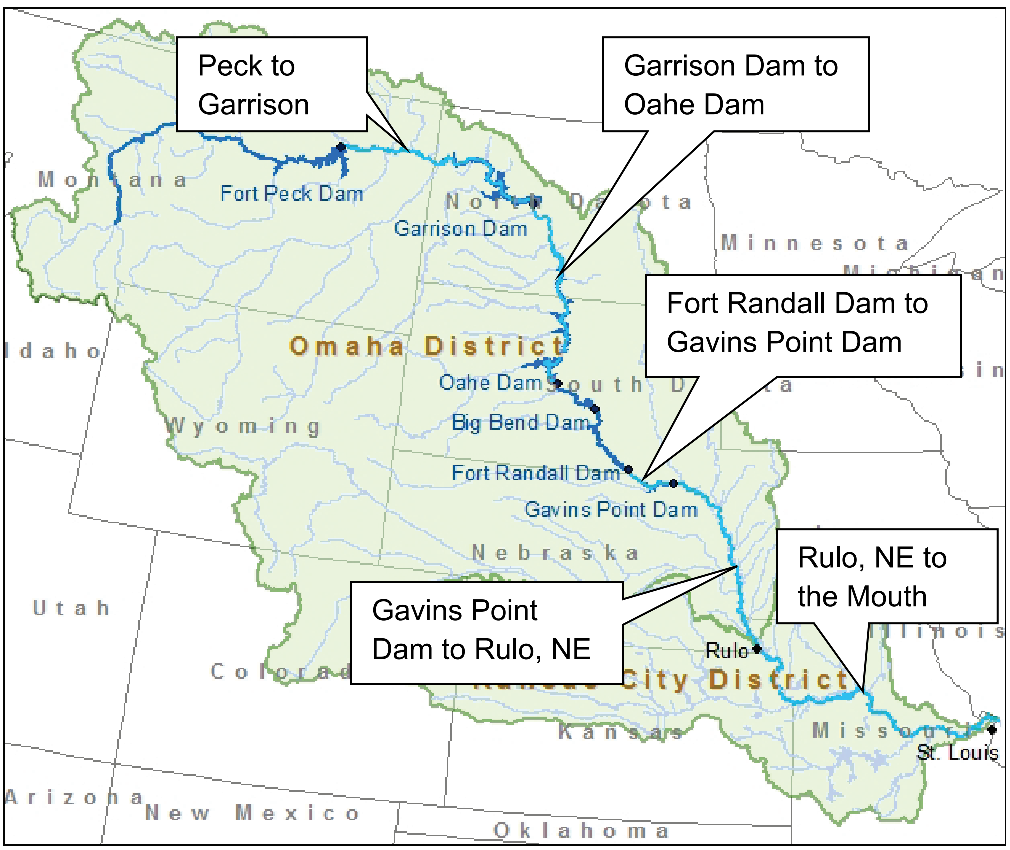


Figure 3. Location of HEC-RAS Modeled Reaches

3.1 Model Geometry

All HEC-RAS models are constructed based on the NAVD-88 vertical datum. Data from other sourcres that are used as imput to the models, such as boundary-condition water surface elevations that are in NGVD-29 have been converted to NAVD-88 using CorpsCon software.

The large number of river miles and the variability in quality and quantity of terrain, hydrographic and stream gage data makes a totally consistent approach to model geometry development across time and river reach difficult. In addition, river channel conditions have changed significantly over time due to construction of the mainstem dams and the Bank Stabilization and Navigation Project.Stage trends indicate that river conditions continue to change with seasonal and annual variations in the stage-flow relationship. As a result of this variability, historic river stages cannot be reliably compared to those measured under today's channel conditions.

The purpose of the HEC-RAS models was to create a baseline that closely represents current river conditions and to provide a tool to evaluate potential hydraulic changes resulting from proposed alternatives (changes in river channel geometry and flow management). As the study progresses the Baseline or Existing Conditions models will be modified to represent a future condition without the implementation of alternatives (Future without Project Model) and to evaluate alternative river management strategies for effectiveness in species recovery and effects on human considerations. At this stage in the project the HEC-RAS models have been constructed to represent modern river conditions and have been calibrated to river stages from time periods that are contemporaneous with those conditions. Modifications to the models made to represent changes under various management alternatives will be discussed in later documents.

Cross sections were digitized from digital terrain models and bathymetric surveys. Overbank areas of the floodplain, as well as higher portions of the channel that are above water under common flow conditions were developed from a variety of digital terrain models. Channel bed data was available from bathymetric survey data collected by the Corps of Engineers. Bathymetry was merged into overbanks to give the most accurate representation of river conveyance conditions over a wide range of flows. The age and precision of terrain data varies between the individual HEC-RAS model reaches based on data availability as well as on the magnitude of recent changes in the river due to large floods. Development of the model for the Rulo to Mouth reach was already well advanced at the time of the 2011 flood. The modeling team determined that channel and overbank conditions on this reach of the river were not changed significantly by the 2011 flood and the model was completed and calibrated using the pre-2011 bathymetry and flows. In contrast, the 2011 flood caused significant changes in the channel, and less frequently, in the overbank terrain between Ft Peck Dam and Rulo, Nebraska. New bathymetry and LiDAR were collected following the flood for other purpose, that was available to use in development of the HEC-RAS model above Rulo.

Downstream of Rulo Nebraska, where navigation structures have a significant impact on conveyance at low flows, ineffective flow areas were used to modify conveyance in the river channel caused by navigation structures.

Details concerning the sources and quality of the terrain data used to develop HEC-RAS models for each river reach, as well as methods for representing navigation structures are included in the calibration report appendices for each reach model.

Levees and overbank flow conditions behind levees during floods are represented in the models by lateral structures, storage areas and storage area connections. Lateral structures represent levees by blocking conveyance and storage of water behind the lateral structure until the level of the river is higher than the structure. At that point the lateral structure acts as a weir allowing flow out of the channel into the levee-protected area. The lateral structure can also be set up to represent a levee breach, increasing the rate of flow through the levee.

Please note that the term storage area is the name of the tool in HEC-RAS that accounts for flow that is taken out of the main channel due to levee overtopping or breach and does not represent a means to intentionally store water for flood control purposes. Once the leveed areas, represented by these storage areas have filled to capacity, water will return to the river either by flowing back over the lateral structure or by flow into an adjacent storage area before returning to the main channel. Other modeling tools have been incorporated to return flood waters to the river over a reasonable period following flood flows. Individual methods are described in the reach-specific appendicies. Transfer of water through very large leveed areas is regulated by storage area connections that act as intermediate weirs, controlling the timing of water movement behind levees and improving the over-all timing of flow volume and stage in the river.

3.2 CALIBRATION

As discussed in section 3.1, river conditions and data quality vary significantly along the Missouri River form the upstream model reach beginning in Montana to the downstream model reach ending on the Mississippi River. As a result, details of calibration for each model also vary depending on availability and quality of data and on the timing of historic high and low water events. In general, model calibration followed the following process. Detailed descriptions of calibration methods and precision are captured in the calibration report appendices for each reach model.

1. Model geometry was developed to represent the current physical characteristics of the channel and overbank conveyance areas. Features commonly represented included channel configuration, channel roughness (Manning's-n values), overbank configuration and vegetation (also represented by Manning's-n values), river structures where applicable, and lateral structures, storage areas and storage area connections to represent levees.
2. In some cases initial roughness values were adjusted based on steady flow runs of the model.
3. Once the model was running for a lower range of unsteady flows in a stable fashion, Manning's-n values and ineffective flow areas were adjusted for preliminary calibration at stream gages. The initial calibration model runs were based on in-channel flow events for time periods consistent with the channel geometry data.

4. Higher flows were next calibrated in an attempt to match the arrival of flow peaks at gages without regard to river stage.
5. When necessary for calibration purposes, ungaged inflows were estimated to improve timing and volume of flow at gages for calibration periods.
6. Final adjustments were made to river geometry features to adjust calibration for stage at river gages.
7. Due to the extremely uncertain nature of levee breach mechanisms, the inability to predict future levee performance within an HEC-RAS modeling context, and the large number of levee along some river reaches, the model calibration did not include any levee breaches. Consequently, locations that included levee breaches during the calibration period may have poor model stage / flow reproduction.

3.3 Sources of Uncertainty

Factors that contribute to uncertainty in the HEC-RAS models include the dynamic nature of the river system itself, river response to flood events and construction projects, the availability and quality of terrain data to represent the channel and floodplain geometry, and the quality of hydrologic data.

Each reach-specific model represents a snapshot in time on a dynamic river system. The Missouri River is a sand bed river in all of the model reaches. Channel depth varies with scour during high flow periods and deposition during low flow periods. Channel bed forms change in magnitude and migrate over time. In most instances the Missouri River channel varies within a fairly well know range of depth and magnitude of bed forms and the models are designed to be a reasonable representation of this dynamic equilibrium.

Significant change in river geometry continues to occur as a result of channel aggradation and degradation which in turn influences stage trends. This aggradation/degradation results from both natural variability in river morphology as well as from man made changes such as the historical construction of flood control projects, channel cutoffs, and channel and bank stability projects. Modeled reaches that include dams generally have a degrading reach below the dam and an aggrading reach in the headwaters of the downstream reservoir. The period of flow records available for calibration to current river geometry is limited in many areas due to ongoing stage trends and the extensive impact of the 2011 flood event. Specific details regarding stage trends and model limitations are presented in the reach specific appendices.

Another major source of uncertainty in the models results from the simplifying assumptions necessary for the construction of HEC-RAS models for a system of this scale and complexity. The HEC-RAS model relies on cross sections of the river channel and adjacent floodplain to represent conveyance conditions. For the ManPlan modeling effort, these cross sections are developed from bathymetric surveys of the channel and digital terrain models of the overbank areas. Although the age and level of precision of the bathymetry and terrain data vary, they are generally well matched to the precision level of the computation methods in the HEC-RAS model. Other parameters that affect river conveyance include floodplain and channel roughness, channel bed forms, river structures, levee overtopping conditions and flow behind levees during flooding. All of these conditions vary with either location along the river, with time or both. The impact of time variability will be addressed in the evaluation of alternatives in the study by normalizing river geometry in the future-without-project and alternative models for a specific point in time and the full period of record will be run through this geometry.

A final source of model uncertainty comes from the scarcity of stream gages and the limited length of record available at many existing gages. Only a limited number of Missouri River tributaries have continuous stream gage records. A significant percentage of the drainage area feeding each model is ungaged. Model calibration requires estimation of ungaged inflows in order to match flow and stage records at the Missouri River gages. Details of ungaged flow estimation for each model are discussed in the reach-specific appendices.

Once again it should be emphasized that the current modeling effort represents existing river conditions. Future river geometry conditions will be addressed during development of the Future Without Project and project alternatives for each reach specific model.

3.4 Quality Control

The quality control process for the RAS models is documented in the Kansas City Quality Management Plan (USACE, 2014) and Omaha District Design Quality Control Plan (USACE, 2013). Quality control has been an on-going process throughout model development. Team discussions were conducted through bi-weekly project calls involving the HEC-RAS modeling team and supervisory staff to resolve issues and maintain common standards. Periodic model peer reviews were conducted at key model development milestone such as low flow calibration, and occasional meetings were held with modeling experts from HEC. Formal District Quality Control (DQC) and Agency Technical Review (ATR) were conducted as prescribed in the Quality Management Plans and documented in Dr. Checks

4 CALIBRATION RESULTS

This section of the calibration report provides a brief summary of calibration results for each modeled reach. More detailed documentation of construction and calibration of each reach-specific model are included in the full calibration reports in Appendicies A through E.

Ft. Peck to Garrison

The Fort Peck to Garrison reach of the Missouri River begins with the regulated outflow from Fort Peck Dam in Montana. The reach extends approximately 365 miles downstream, encompassing a watershed of approximately 181,400 square miles, to just upstream of Garrison Dam on Lake Sakakawea, North Dakota. This reach was modeled in HEC-RAS Version 4.2 Beta and with the intent to update to Version 5.0 when it is released. The model was initially created in steady flow and then completed with unsteady modeling, and is now fully unsteady.

Inputs into the model are a flow hydrograph for Fort Peck Dam's release, flow hydrographs for the upstream boundaries of the major tributaries (Milk River, Poplar River, and Yellowstone River), and a stage hydrograph for the Garrison Pool (Lake Sakakawea). Output includes stage and flow hydrographs, as well as a number of additional calculated parameters such as average velocities, flow depth, etc.

The geometry was constructed using the most recent sediment range surveys from the Omaha District, which included topographic and hydrographic data. Additional cross sections were added between the sediment ranges using LiDAR data for the overbanks and interpolation of the sediment ranges for the bathymetry where hydrographic data was unavailable. The flow data for the tributaries were obtained from USGS gages. The observed Fort Peck releases and Garrison Pool elevations were obtained from the Omaha District CWMS database. The model includes the Williston, North Dakota levee. A lateral structure and storage area were used to model the levee.

The model reach includes a substantial degradation reach that extends downstream from Fort Peck Dam and a large aggradation zone in the headwaters of Lake Sakakawea. The extreme 2011 flow event significantly altered the river stage-flow relationship and model calibration to observed stages in flood years prior to 2011 is not valid in most areas. Therefore, due to impacts from the 2011 flood and long term changes within the aggradation and degradation areas, the hydraulic model , which is calibrated to current conditions, is not capable of reproducing observed stage-flow relationships prior to 2011.

The model was calibrated to the measured 2012 Water Surface Profile (WSP) and observed stage gage data for the Missouri River using ungaged inflows in HEC-RAS. The computed water surface profile was within +/- 1 ft along the entire reach and in the range of +/- 0.5 ft for about 50% to 75% of the reach. These were determined to be acceptable calibration targets. Comparison to observed hydrographs indicated that the model performed acceptably on timing of flood peaks within most areas.

Garrison Dam to Oahe Dam

The Garrison to Oahe reach of the Missouri River begins with the regulated outflow from Garrison Dam in North Dakota. The reach extends approximately 318 miles downstream, encompassing a watershed of approximately 243,490 square miles, to just upstream of Oahe Dam on Lake Oahe, South Dakota. This reach was modeled in HEC-RAS Version 4.2 Beta and with the intent to update to Version 5.0 when it is released. The model was initially created in steady flow and then completed with unsteady modeling, and is now fully unsteady.

Inputs into the model are a flow hydrograph for Garrison Dam’s release, flow hydrographs for the upstream boundaries of the major tributaries (Knife River, Square Butte Creek, Burnt Creek, Heart River, Apple Creek, Cannonball River, Beaver Creek, Oak Creek, Grand River, Moreau River, and Cheyenne River), and a stage hydrograph for the Oahe Pool (Lake Oahe). Output includes stage and flow hydrographs, as well as a number of additional calculated parameters such as average velocities, flow depth, etc.

The model geometry was constructed using the most recent sediment range surveys from the Omaha District, which included topographic and hydrographic data. Additional cross sections were added between the sediment ranges using LiDAR data for the overbanks and interpolation of the sediment ranges for the bathymetry where hydrographic data was unavailable. The flow and stage data for the tributaries were obtained from USGS gages. The observed Garrison releases and Oahe Pool elevations were obtained from the Omaha District CWMS database. The model reach includes a substantial degradation reach that extends downstream from Garrison Dam and a large aggradation zone in the headwaters of Lake Oahe. The extreme 2011 flow event significantly altered the river stage-flow relationship and model calibration to observed stages in flood years prior to 2011 is not valid in most areas. Therefore, due to impacts from the 2011 flood and long term changes within the aggradation and degradation areas, the hydraulic model , which is calibrated to current conditions, is not capable of reproducing observed stage-flow relationships prior to 2011.

The model was calibrated to the measured 2011 and 2012 Water Surface Profiles (WSP) and observed stage gage data for the Missouri River using ungaged inflows in HEC-RAS. The computed water surface profile was within +/-1 ft along the entire reach and in the range of +/- 0.5 ft for about 50% to 75% of the reach. These were determined to be acceptable calibration targets. Comparison to observed hydrographs indictated that the model performed acceptably on timing of flood peaks within most areas. Some minor calibration issues were noted with hydrograph timing in areas affected by the hourly flow peaking due to power releases from Garrison Dam.

Ft. Randall Dam to Gavins Point Dam

The Fort Randall to Gavins Point reach of the Missouri River begins with the regulated outflow from Fort Randall Dam in North Dakota. The reach then extends approximately 70 miles downstream, encompassing a watershed of approximately 279,480 square miles, to just upstream of Gavins Point Dam on Lewis and Clark Lake. This reach was modeled in HEC-RAS Version 4.2 Beta and with the intent to update to Version 5.0 when it is released. The model was initially created in steady flow and then completed with unsteady modeling, and is now fully unsteady.

Inputs into the model are a flow hydrograph for Fort Randall Dam's release, flow hydrographs for the upstream boundaries of the major tributaries including Ponca Creek, Niobrara River, Verdigre Creek (a Niobrara River tributary), and Bazile Creek, and a stage hydrograph for the Gavins Point Pool (Lewis and Clark Lake). Output includes stage and flow hydrographs, as well as a number of additional calculated parameters such as average velocities, flow depth, etc.

The geometry was constructed using the most recent sediment range surveys from the Omaha District, which included topographic and hydrographic data. Additional cross sections were added between the sediment ranges using LiDAR data for the overbanks and interpolation of the sediment ranges for the bathymetry where hydrographic data was unavailable. The flow and stage data were obtained from USGS gages. The observed Fort Randall releases and

Gavins Point Pool elevations were obtained from the Omaha District CWMS database.

The model reach includes a substantial degradation reach that extends downstream from Fort Randall Dam and a large aggradation zone in the headwaters of Lewis and Clark Lake. The extreme 2011 flow event significantly altered the river stage-flow relationship and model calibration to observed stages in flood years prior to 2011 is not valid in most areas. Therefore, due to impacts from the 2011 flood and long term changes within the aggradation and degradation areas, the hydraulic model , which is calibrated to current conditions, is not capable of reproducing observed stage-flow
relationships prior to 2011.

The model was calibrated to the measured 2011 and 2012 Water Surface Profiles (WSP) and observed stage gage data for the Missouri River. The computed water surface profile was within 1 ft along the entire reach and in the range of +/- 0.5 ft for about 50% to 75% of the reach. These were determined to be acceptable calibration targets. Comparison to observed stage hydrographs indictated that the model performed acceptably on timing of flood peaks within most areas.

Gavins Point Dam to Rulo, NE

The Gavins Point Dam to Rulo, NE, reach of the Missouri River begins with the regulated outflow from Gavins Point Dam in South Dakota at 1960 River Mile (RM) 811.1. The reach extends approximately 250 miles downstream to Rulo, NE at RM 498.0 which is the Omaha District boundary with Kansas City District. This reach was modeled in Hydrologic Engineering Center River Analysis System (HEC-RAS) Version 4.2 Beta with the intent to update to Version 5.0 when it is released. The model was initially created in steady flow and then completed with unsteady modeling, and is now fully unsteady.

Inputs into the model are a flow hydrograph for the Gavins Point Dam release and flow hydrographs for the upstream boundaries of the larger gaged tributaries within the Omaha District consisting of the James River, Vermillion River, Big Sioux River, Little Sioux River, Soldier River, Boyer River, Platte River, Weeping Water Creek, Nishnabotna River, Little Nemaha River, and Tarkio River. Output includes stage and flow hydrographs, as well as a number of additional calculated parameters such as average velocities, flow depth, and etc. available at specified locations.

The model extends downstream from Rulo, NE, to the St Joseph, MO, vicinity at RM 448.2, using data provided by Kansas City District to provide reasonable computation results for reporting at Rulo, NE. Therefore, the Omaha District and the Kansas City District models include an overlap reach at the Rulo, NE, boundary. The geometry was constructed using the most recent surveys from the Omaha District, which included topographic data from fall 2011 LiDAR. The LiDAR extent covered the active flow corridor. This data was supplemented when needed with state-provided LiDAR and 4 meter DEM data in some areas to extend cross section coverage within the wide floodplain or levee cell areas. Model geometry for the Missouri River channel was constructed from 2012 hydrographic cross section survey data from Ponca to

Rulo and 2013 data from Gavins Point Dam to Ponca at 250 foot spacing intervals. The flow data for the Gavins Point Dam release and inflow tributaries were obtained from the Omaha District database and USGS gages.

Levee storage areas and lateral structures were used to describe the federal levee system between Omaha and Rulo (RM 620 to RM 515). The complex network of private levees in the Rulo vicinity were also included as storage cells in the model. The levee and storage area connections were set to a very low weir coefficient to enhance model stability and also reflect the non-weir flow conditions with limited downstream conveyance. Efforts to evaluate the effect of the weir coefficient indicated some impact on peak stage elevations. A reasonable value was determined after comparison to some historic events. All levees were modeled with overtopping only, no levee breaches were included.

Valley wide cross sections were extracted from the topography and retained to allow for future alternative condition modeling in multiple configurations if necessary. Therefore, blocked obstructions were included to remove the levee protected area from the cross sections and prevent double counting of storage. Blocked obstructions were used rather than point deletion to allow for possible future modeling options. Blocked obstructions were also used in the area upstream of Omaha that does not include levee cells. These obstructions were necessary to limit the available storage, to allow the RAS coding of levee confinement near the main channel, and to eliminate the wide portions of the section from storage.

Model calibration was performed for recent flow events in 2011 and 2012. The extreme 2011 flow event significantly altered the river stage-flow relationship and comparison to observed stages in flood years prior to 2011 is not valid in most areas. The model reach includes a substantial degradation section that extends downstream from Gavins Point Dam that is noticeable from stage trends. Degradation that occurred during the 2011 event is also apparent. In addition, the 2011 extreme event model calibration within the federal levee reach is not possible at many locations due to multiple levee breaches that occurred. The model is constructed with post 2011 extreme flood geometry. This resulted in some notable calibration issues. For instance, the Nebraska City reach with the levee setback appears low in the model calibration, likely due to the geometry change. Since none of the levee breaches are included within the model, calibrating to observed flow / stage levels in areas highly impacted by levee breaches is not possible. Calibration data consists of observed hydrographs at gage station locations and measured water surface elevation profiles from both 2011 and 2012. The computed water surface profile was within +/- 1 ft along the entire reach and in the range of +/- 0.5 ft for approximately 50% to 75% of the reach. These were determined to be acceptable calibration targets based on accuracy attained during previous studies on the Missouri River. Comparison to observed hydrographs indicated that the model performed acceptably on timing of flood peaks within most areas. Poor calibration was noted in the downstream end of the model for the 2011 event for the areas affected by levee breaches.

HEC-RAS model construction differences occurred between the Omaha and Kansas City District modeling efforts due to changes in river features. Downstream of Rulo, NE, where the navigation structures are larger and have a significant impact on conveyance at low flows, ineffective flow areas were used to represent the navigation structure impact on channel conveyance. Other minor differences such as tieback modeling technique and calibration period also occurred. Refer to the model geometry and calibration discussion in each appendice for additional details.

Rulo to the Mouth

The Rulo to the Mouth reach includes the lower 498 miles miles of the Missour River contained within the boundary of the United States Army Corps of Engineers (USACE) Kansas City District. The model is fully unsteady. Inputs to the model are flow hydrographs, and outputs include stage and flow hydrographs at every cross section as well as a number of additional calculated parameters such as average channel velocity.

There are several geometry features that are unique to the Rulo to the Mouth reach. Fourteen of the largest tributaries are modeled as reaches in HEC-RAS, contributing a routed hydrograph from a USGS gage to the flow in the mainstem Missouri River. Leveed areas in the floodplain are represented in the model with lateral structures and storage areas. This is especially important near Rulo, NE, around Waverly, MO, and where the Missouri river flows into the Mississippi River north of St. Louis because a particularly wide floodplain, multiple levees, and high ground obstructions make flooding more difficult to model in these areas. In addition, navigation structures heavily influence low flows on this reach of the Missouri River so these structures are included in the model as permanent ineffective flow areas.

Calibration was performed using recent USGS instantaneous gage data for a six year period from October 1, 2007 to September 30, 2013. A longer calibration period was possible for the river below Rulo because flooding during 2011 had a much less significant impact on the channel geometry than on the river farther upstream. Between Rulo and the mouth, seven USGS stage-flow gages and three stage-only gages have reasonable record lengths during these six years. Calibration efforts focused on matching stages and flows at these gages for flows ranging from the low winter flows of 2012 to the significant floods of 2011 and 2013. Ungaged inflows were estimated by a combination of scaling up tributary flows by the basin area ratio and adding uniform monthly averaged missing flows. Additional calibration data included a low water profile collected in 2009 and high water marks collected after 2011 and 2013.

Calibration of this model is intended to reproduce, on average, both low and high flow conditions on the Missouri River. It was not calibrated tightly to any one event, but rather generally represents the present-day stage-discharge relationships at USGS gages on the Missouri River. On average, the model has a mean stage error of 0.1 feet with a root mean square stage error of 0.8 feet. Eighty six percent of the time the computed stage is within 1-ft of observed, and 97% of the time it is within 2-ft of observed. Model calibration is adequate for the objective of running a period of record to evaluate alternatives that may include operational and/or physical changes.

5 CONCLUSIONS

Five HEC-RAS models were developed for all reaches of the Missouri between Ft. Peck Reservoir in Montana to the mouth near St. Louis, Missouri. The purpose for developing the models was to simulate and compare a range of river management alternatives as part of the Missouri River Recovery Program (MRRP). Although variability in the quality and availability of terrain, hydrography and stream gage data required some differences in geometry development between models, final model calibration is considered to be very good for a set of hydraulic models of this magnitude.

Models upstream of Rulo, Nebraska were calibrated to measured 2011 and 2012 water surface profiles and observed stage-gage data. The computed water surface profiles were within +/-1 ft along the entire reach and in the range of +/- 0.5 ft for about 50% to 75% of the reach. The river reach below Rulo, Nebraska was calibrated using USGS instantaneous gage data for a six year period from October 1, 2007 to September 30, 2013. The longer calibration period was possible for the river below Rulo because flooding during 2011 had a less significant impact on the channel geometry so a longer calibration window was available for this reach of river. As a result this model reach was not calibrated as tightly to any one event, but represents the present-day stage-discharge relationships at USGS gages on the Missouri River over a wider range of time and river conditions. On average, the model below Rulo had a mean stage error of 0.1 feet with a root mean square stage error of 0.8 feet, 86% of the time the computed stage is within 1-ft of observed, and 97% of the time it is within 2-ft of observed. All HEC-RAS models are constructed based on the NAVD-88 vertical datum.

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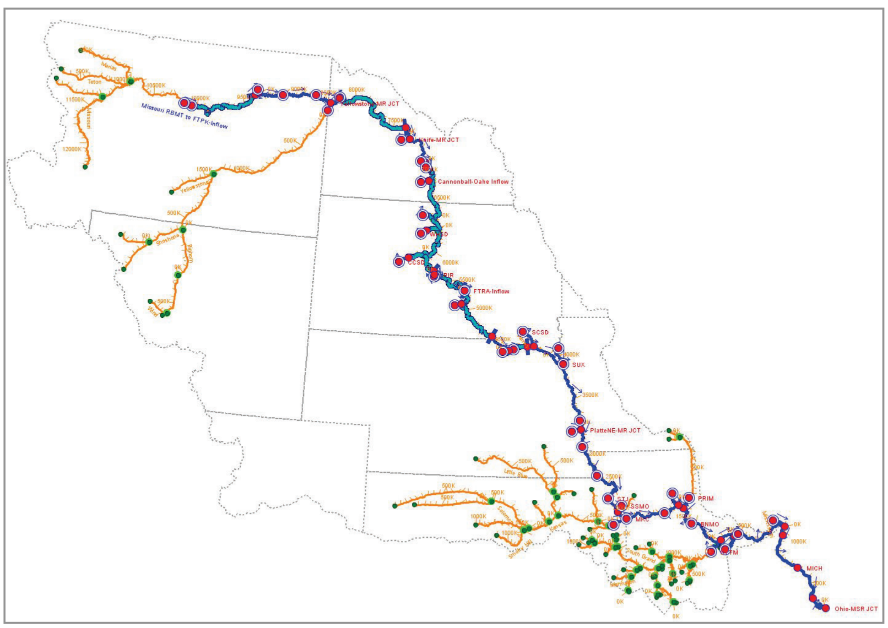
US Army Corps of Engineers ®

Omaha District

Missouri River Unsteady HEC-RAS Model
Calibration Report

Appendix A

Fort Peck Dam to Garrison Dam



July 2018

FINAL

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ATTACHMENTS

ACRONYMS

CFS…………………. Cubic Feet per Second

DEM…………………. Digital Elevation Model

DTM .... Digital Terrain Model

DSSVue…………….. Data Storage System (by HEC)

GIS .... Geographic Information System

HEC .... Hydrologic Engineering Center

LiDAR……………….Light Detection and Ranging

MAF………………….. Million acre-feet

NAD 1983………… ... North American Datum of 1983

NAVD 88……………. North American Vertical Datum of 1988

NGVD 29……………. National Geodetic Vertical Datum of 1929

MRBWM…………... .. Missouri River Basin Water Management Division (previously RCC)

NWK………………… Northwest Division Kansas City District

NWO………………… Northwest Division Omaha District

POR .... Period of Record

RAS.......................... River Analysis System Software (by HEC)

RCC .... Reservoir Control Center

ResSim……………...Reservoir Simulation Software (by HEC)

RM……………………1960 River Mile

System……………....Missouri River Mainstem System

USACE……………… United States Army Corps of Engineers

USGS .... United States Geological Survey

1 EXECUTIVE SUMMARY

The Fort Peck to Garrison reach of the Missouri River begins with the regulated outflow from Fort Peck Dam in Montana. The reach then extends approximately 365 miles downstream, encompassing a watershed of approximately 181,400 square miles, to just upstream of Garrison Dam on Lake Sakakawea, North Dakota. This reach was modeled in Hydrologic Engineering Center River Analysis System (HEC-RAS) Version 4.2 Beta and with the intent to update to Version 5.0 when it is released. The model was initially created in steady flow and then completed with unsteady modeling, and is now fully unsteady.

Inputs into the model are a flow hydrograph for Fort Peck Dam's release, flow hydrographs for the upstream boundaries of the major tributaries (Milk River, Poplar River, and Yellowstone River), and a stage hydrograph for the Garrison Pool (Lake Sakakawea). Output includes stage and flow hydrographs, as well as a number of additional calculated parameters such as average velocities, flow depth, etc. The latest version of HEC-RAS also has the ability to create inundation depth grids at various time-steps using RAS Mapper that can be exported for use in ecological and economic models.

The geometry was constructed using the most recent sediment range surveys from the Omaha District, which included topographic and hydrographic data. Additional cross sections were added between the sediment ranges using LiDAR data for the overbanks and interpolation of the sediment ranges for the bathymetry where hydrographic data was unavailable. The flow data for the tributaries were obtained from USGS gages. The observed Fort Peck releases and Garrison Pool elevations were obtained from the Omaha District CWMS database. The model includes the Williston, North Dakota levee. A lateral structure and storage area were used to model the levee.

The model reach includes a substantial degradation reach that extends downstream from Fort Peck Dam and a large aggradation zone in the headwaters of Lake Sakakawea. The extreme 2011 flow event significantly altered the river stage-flow relationship and model calibration to observed stages in flood years prior to 2011 is not valid in most areas. Therefore, due to impacts from the 2011 flood and long term changes within the aggradation and degradation areas, the hydraulic model is not capable of reproducing observed stage-flow relationships prior to 2011.

The model was calibrated to the measured 2012 Water Surface Profile (WSP) and observed stage gage data for the Missouri River using ungaged flows in HEC-RAS. The computed water surface profile was within +/- 1 ft along the entire reach and in the range of +/- 0.5 ft for about 50% to 75% of the reach. These were determined to be acceptable calibration targets. Comparison to observed hydrographs indicated that the model performed acceptably on timing of flood peaks within most areas.

2 INTRODUCTION

The Missouri River unsteady HEC-RAS model was created as a base model for planning studies which could be used to simulate and analyze broad scale watershed alternatives. The objective of this HEC-RAS model is to simulate current conditions on the Missouri River, with the intention of running period of record (POR) flows to compare alternatives. Future reports will address period of record runs, this report addresses model construction and calibration. This Appendix is for the Fort Peck to Garrison reach of the Missouri River as part of the Omaha District.

3 BACKGROUND

The Fort Peck to Garrison reach is the first upstream reach for the Omaha District's portion of the Missouri River.

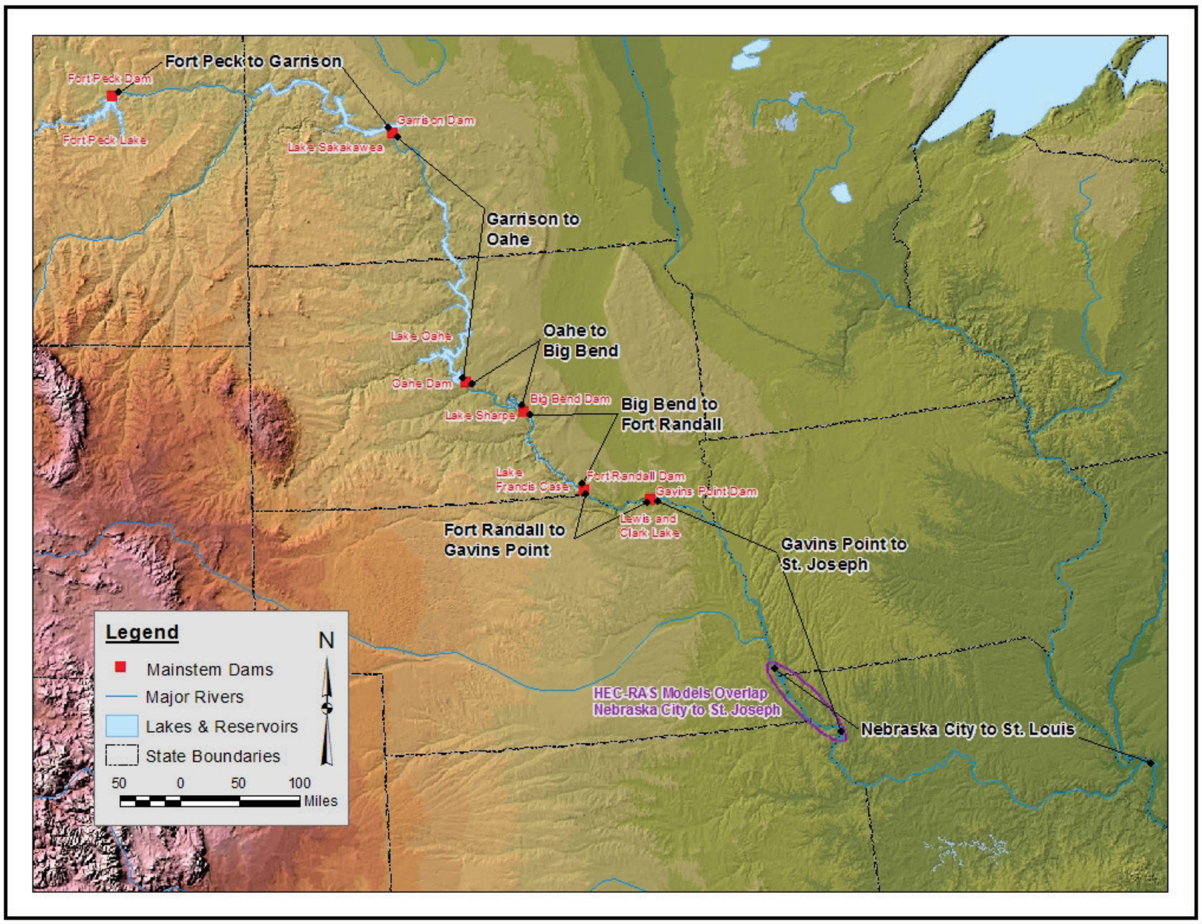

Figure 3-1: Model Extents

3.1 Model Extents

This is the most upstream portion of the Missouri River being modeled with HEC-RAS for the Omaha District, from River Mile 1769.04, located just downstream of Fort Peck Dam in Montana, to River Mile 1391.08, located just upstream of Garrison Dam in North Dakota (see Figure 3-1). Downstream of this reach, there are 3 other reaches of the Missouri River being modeled by Omaha District (see Appendices B – D) and the most downstream reach is being modeled by Kansas City District (see Appendix E).

3.2 Missouri River Mainstem System Description

The Missouri River Mainstem System (System) of dams is composed of six large earth embankments which impound a series of lakes that extend upstream for 1,257 river miles from Gavins Point Dam near Yankton, South Dakota to the head waters of Fort Peck Lake north of Lewiston, Montana. These dams were constructed by the Corps of Engineers for flood control, navigation, power production, irrigation, water supply, water quality, recreation, and fish and wildlife enhancement. Fort Peck Dam, the oldest of the six dams, was closed and began water storage in 1937. Fort Randall Dam was closed in 1952, followed by Garrison Dam in 1953, Gavins Point Dam in 1955, Oahe Dam in 1958, and Big Bend Dam in 1963. The current System of six projects first filled and began operating as a six-project System in 1967. At the top elevation of their normal operating pool level, the lakes behind these six dams provide about 1,146,000 acres of water surface area and extend a total length of 755 river miles. Only 325 miles of open river remain between the lakes, although there are 811 miles of open river downstream from Gavins Point Dam to the mouth of the Missouri River where it enters the Mississippi River at St. Louis, Missouri. The reservoirs contain an aggregate storage volume of approximately 73 million acre-feet (MAF) of which more than 16 MAF is for flood control.

Regulation of the System is according to the current Master Manual (USACE, 2006) and generally follows a repetitive annual cycle. Winter snows and spring and summer rains produce most of the year’s water supply, which results in rising pools and increasing storage accumulation. After reaching a peak reservoir level, usually during July, storage declines until late winter when the cycle begins anew. A similar pattern may be found in rates of releases from the System, with higher flows from mid-March to late November, followed by low rates of winter discharge from late November until mid-March, after which the cycle repeats.

Two primary high-risk flood seasons are the plains snowmelt season extending from late February through April and the mountain snowmelt period extending from May through July. Overlapping the two snowmelt flood seasons is the primary rainfall flood season, which includes both upper and lower basin regulation considerations.

Power generation is a component of System operation. The highest average power generation period extends from mid-April to mid-October with high peaking loads during the winter heating season (mid-December to mid-February) and the summer air conditioning season (mid-June to mid-August). The power needs during winter are supplied primarily with Fort Peck and Garrison releases and the peaking capacity of Oahe and Big Bend. During the spring and summer periods, releases are geared to navigation and flood control requirements and primary power loads are supplied using the four lower dams. During the fall when power needs diminish, Fort Randall pool is drawn down to permit generation during the winter period when the pool is refilled by Oahe and Big Bend peaking power releases. Gavins Point Dam, as the downstream-most reservoir, is operated at constant daily releases and is not used for daily power peaking.

Normally, the navigation season extends from April 1 through December 1 during which time reservoir releases are increased to meet downstream target flows in combination with downstream tributary inflows. Winter releases after the close of navigation season are much lower and vary depending on the need to conserve or evacuate system storage volumes, downstream ice conditions permitting. Minimum release restrictions and pool fluctuations for fish spawning management generally occur from April 1 through July. Endangered and threatened species, including the interior least tern and piping plover, nesting occurs from early May through August. During this period, special release patterns are made from Garrison, Fort Randall, and Gavins Point to avoid flooding nesting sites on low-lying sandbars and islands downstream from these projects.

Overall, the general regulation principles presented above provide the backbone philosophy for the Mainstem System regulation. Detailed operation plans are developed, followed, and adjusted as conditions warrant periodically as the System is monitored day-to-day. Beginning in 1953, projected operation of the Missouri River Mainstem Reservoir System for the year ahead was developed annually as a basis for advance coordination with the various interested Federal, State, and local agencies and private citizens. These regulation schedules are prepared by the Missouri River Basin Water Management Division, Northwest Division, Corps of Engineers and are reported in Annual Operating Plans (USACE, 2013b).

In addition to the six main stem projects operated by the Corps, 65 tributary reservoirs operated by the Bureau of Reclamation and the Corps provide over 15 million acre-feet of flood control storage.

Numerous reservoirs and impoundments constructed by different interests for flood control, irrigation, power production, recreation, water supply, and fish and wildlife are located throughout the basin on various tributaries. The Bureau of Reclamation and the Corps of Engineers have constructed the most significant of these structures. Although primarily constructed for irrigation and power production, the projects constructed by the Bureau of Reclamation do provide some limited flood control in the upper basin.

Table 3-1 lists pertinent data for the Missouri River Mainstem projects (USACE, 2013a).

Table 3-1: Pertinent Data for Missouri River Mainstem Projects

Description	Fort Peck	Garrison	Oahe	Big Bend	Fort Randall	Gavins Point
River Mile (1960 Mileage)	1771.5	1389.9	1072.3	987.4	880.0	811.1
Drainage Area (sq. mi.)	57,500	181,400	243,490	249,330	263,480	279,480
Incremental Drainage Area (sq. mi.)	57,500	123,900	62,090	5,840	14,150	16,000
Gross Storage (kAF)	18,463	23,451	22,983	1,798	5,293	428
Flood Storage (kAF)	3,675	5,706	4,315	177	2,293	133
Top of Dam* (ft NGVD29 (NAVD88))	2280.5 (2282.6)	1875.0 (1876.3)	1660.0 (1661.2)	1440.0 (1441.1)	1395.0 (1396.0)	1234.0 (1234.7)
Maximum Surcharge Pool** (ftNGVD29 (NAVD88))	2253.3 (2255.4)	1858.5 (1859.8)	1644.4 (1645.6)	1433.6 (1434.7)	1379.3 (1380.3)	1221.4 (1222.1)
Top of Exclusive FC Pool*** (ft NGVD29 (NAVD88))	2250.0 (2252.1)	1854.0 (1855.3)	1620.0 (1621.2)	1423.0 (1424.1)	1375.0 (1376.0)	1210.0 (1210.7)
Top of Annual FC Pool (ft NGVD29 (NAVD88))	2246.0 (2248.1)	1850.0 (1851.3)	1617.0 (1618.2)	1422.0 (1423.1)	1365.0 (1366.0)	1208.0 (1208.7)
Base of Flood Control Pool (ft NGVD29 (NAVD88))	2234.0 (2236.1)	1837.5 (1838.8)	1607.5 (1608.7)	1420.0 (1421.1)	1350.0 (1351.0)	1204.5 (1205.2)
Spillway Capacity (cfs)	275,000	827,000	304,000	390,000	633,000	584,000
Outlet Capacity (cfs)	45,000	98,000	111,000	n/a	128,000	n/a
Powerplant Capacity (cfs)	16,000	41,000	54,000	103,000	44,500	36,000
Date of Closure	Jun 1937	Apr 1953	Aug 1958	Jul 1963	Jul 1952	Jul 1955

* Operational elevations are referenced to the NGVD29 datum. They were converted to NAVD88 using CorpsCon conversion factors for use with model elevations.
**Maximum pool elevation with spillway gates opened.
***Maximum pool elevation with spillway gates closed.

3.3 Fort Peck and Garrison Dam and Reservoir Information

3.3.1 Fort Peck Dam and Fort Peck Lake

Fort Peck Dam and Fort Peck Lake are the largest dam and third largest reservoir in the Missouri River mainstem system. Fort Peck Dam is located on the Missouri River at RM 1771.50 in northeastern Montana. Closure of the dam occurred in 1937, and the project was placed in operation for navigation and flood control in 1938. Prior to 1956, Fort Peck was the only main stem project with a significant amount of accumulated storage. As a consequence, releases in the 28,000 cfs range were frequently required for navigation. Releases have seldom been in excess of power plant capacity which is about 15,000 cfs since the second power plant came on line in 1961. Table 3-2 through Table 3-4 shows the historical releases and release-duration and release-probability relationships for Fort Peck Dam (USACE, 2013a).

Table 3-2: Fort Peck Release Historical Records (1967-2011)

Month	Daily Release (cfs)		
	Maximum	Minimum	Mean
Jan	15,600	4,200	10,600
Feb	15,500	4,100	10,900
Mar	15,600	1,000	7,900
Apr	25,100	0	7,300
May	28,900	2,800	9,100
Jun	65,900	3,000	10,600
Jul	49,900	3,000	10,600
Aug	35,200	3,800	10,200
Sep	25,200	2,700	9,100
Oct	21,800	2,700	8,000
Nov	22,300	2,700	8,300
Dec	16,000	4,100	9,500
Annual	**65,900**	**0**	**9,300**

Table 3-3: Fort Peck Release-Duration Relationship

Percent of Time Equaled or Exceeded	Release (cfs)	
	Annual	May – Aug
Maximum	65,900	65,900
1	25,000	35,100
5	14,800	15,900
10	14,200	14,500
20	12,500	13,000
50	8,600	8,600
80	6,000	6,800
90	4,800	6,000
95	4,100	5,600
99	3,000	4,100
100	0	2,800

Table 3-4: Fort Peck Release-Probability Relationship

Annual Percent Chance Exceedance	Release (cfs)
50	15,000
20	17,000
10	25,000
2	48,000
1	60,000
0.2	95,000*

* Extrapolated: Max observed is 65,900 cfs, June 2011.

3.3.2 Garrison Dam and Lake Sakakawea

Garrison Dam is located in central North Dakota on the Missouri River at RM 1389.86, about 11 miles south of the town of Garrison, North Dakota. Construction of the project was initiated in 1946, closure was made in April 1953, and the navigation and flood control functions of the project were placed in operation in 1955. Lake Sakakawea is the largest USACE reservoir and contains almost a third of the total storage capacity of the System, nearly 24 MAF. The total drainage area of the Missouri River at Garrison Dam is 181,400 sq. miles. The incremental drainage area between Fort Peck Dam and Garrison Dam is 123,900 sq. miles. Table 3-5 through Table 3-7 shows the historical pool elevations and pool-duration and pool-probability relationships for Garrison Dam (USACE, 2013a).

Table 3-5: Garrison Pool Historical Records (1967-2011)

Month	Pool Elevation (ft, NGVD29 (NAVD88*))		
	Maximum	Minimum	Mean
Jan	1845.3 (1846.6)	1807.0 (1808.3)	1832.6 (1833.9)
Feb	1843.6 (1844.9)	1806.6 (1807.9)	1831.2 (1832.5)
Mar	1847.9 (1849.2)	1806.9 (1808.2)	1831.2 (1832.5)
Apr	1847.7 (1849.0)	1806.6 (1807.9)	1832.7 (1834.0)
May	1853.3 (1854.6)	1805.8 (1807.1)	1833.5 (1834.8)
Jun	1854.5 (1855.8)	1809.1 (1810.4)	1836.6 (1837.9)
Jul	1854.8 (1856.1)	1815.2 (1816.5)	1839.7 (1841.0)
Aug	1854.6 (1855.9)	1811.9 (1813.2)	1839.0 (1840.3)
Sep	1851.3 (1852.6)	1809.5 (1810.8)	1837.4 (1838.7)
Oct	1848.2 (1849.5)	1809.3 (1810.6)	1836.5 (1837.8)
Nov	1847.0 (1848.3)	1808.9 (1810.2)	1837.5 (1838.8)
Dec	1846.8 (1848.1)	1807.8 (1809.1)	1834.2 (1835.5)
Annual	**1854.8 (1856.1)**	**1805.8 (1807.1)**	**1834.9 (1836.2)**

*NGVD29 elevations were converted to NAVD88 using the conversion factor listed in Table 5-1.

Table 3-6: Garrison Pool-Duration Relationship

Percent of Time Equaled or Exceeded	Pool Elevation (ft, NGVD29 (NAVD88*))	
	Annual	May – Aug
Maximum	1854.8 (1856.1)	1854.8 (1856.1)
1	1851.9 (1853.2)	1854.1 (1855.4)
5	1848.6 (1849.9)	1850.6 (1851.9)
10	1846.9 (1848.2)	1849.0 (1850.3)
20	1844.5 (1845.8)	1847.5 (1848.8)
50	1838.8 (1840.1)	1840.3 (1841.6)
80	1823.8 (1825.1)	1825.7 (1827.0)
90	1816.9 (1818.2)	1817.7 (1819.0)
95	1812.4 (1813.7)	1815.7 (1817.0)
99	1807.6 (1808.9)	1808.8 (1810.1)
100	1805.8 (1807.1)	1805.7 (1807.0)

*NGVD29 elevations were converted to NAVD88 using the conversion factor listed in Table 5-1.

Table 3-7: Garrison Pool-Probability Relationship

Annual Percent Chance Exceedance	Pool Elevation (ft, NGVD29 (NAVD88*))
50	1845.0 (1846.3)
20	1850.5 (1851.8)
10	1852.0 (1853.3)
2	1854.0 (1855.3)
1	1854.5 (1855.8)
0.2	1855.5** (1856.8)

*NGVD29 elevations were converted to NAVD88 using the conversion factor listed in Table 5-1.
** Extrapolated: Max observed is 1854.8 ft NGVD29.

3.3.3 Survey History

Degradation and aggradation surveys are an integral part of the Omaha District's sediment data collection program. The survey work requires the periodic resurvey of the land surface and riverbed cross sections between permanently established survey control points called sediment ranges. There are forty-seven sediment ranges spaced an average of 3.7 miles apart below Fort Peck Dam. There are forty-five main stem sediment ranges spaced an average of 2.4 miles apart at Lake Sakakawea. Table 3-8 below provides a summary of the Fort Peck degradation and Garrison aggradation reaches. The break between survey ranges between the degradation and the aggradation reach is not representative of where degradation/aggradation is occurring but the point where the maximum pool elevation of Lake Sakakawea intersects the Missouri River thalweg profile.

Table 3-8: Sediment Range Information

Fort Peck Degradation Reach – Fort Peck Dam to Lake Sakakawea					
Fort Peck Dam River Mile (1960 RM)	Ending River Mile (1960 RM)	Reach Length (miles)	No. of Main Stem Sediment Ranges	Average Spacing of Ranges (miles)	Most Recent Survey Year
1771.50	1596.89	174.61	47	3.7	2011-2012
Garrison Aggradation Reach – Lake Sakakawea					
Beginning River Mile (1960 RM)	Garrison Dam River Mile (1960 RM)	Reach Length (miles)	No. of Main Stem Sediment Ranges	Average Spacing of Ranges (miles)	Most Recent Survey Year
1590.20	1389.86	200.34	45	2.4	2010-2012

3.4 REACH CHARACTERISTICS

The upstream end of the reach begins immediately downstream of Fort Peck Dam. The reach then extends approximately 365 miles downstream, encompassing a watershed of approximately 181,400 square miles, to just upstream of Garrison Dam on Lake Sakakawea, near Pick City, North Dakota, as shown in Plate 1.

This reach of the Missouri River flows through mostly agricultural land and sparsely populated areas. The two largest cities located near the Missouri River in this reach are Wolf Point, Montana and Williston, North Dakota. There is only one levee located on this reach, and that is near the town of Williston, North Dakota.

In addition to the modeling of the Missouri River, there are three tributaries modeled in HEC-RAS: 1) Milk River extending approximately 24 miles from the confluence with the Missouri River to Nashua, Montana. The Milk River watershed is approximately 22,300 square miles; 2) Poplar River near Poplar, Montana extending 14 miles upstream from the confluence with the Missouri River. The Poplar River watershed is approximately 3,200 square miles; 3) Yellowstone River extending approximately 62 miles from the confluence with the Missouri River to Sydney, Montana. The Yellowstone River watershed is approximately 69,100 square miles.

3.5 DEGRADATION AND AGGRADATION TRENDS

During the development of the Missouri River basin projects, significant change has occurred in channel conveyance as a result of aggradation and degradation. Missouri River natural variability and construction including flood control projects, channel cutoffs, channel and bank stability projects have all contributed to conveyance change. The release of essentially sediment-free water through the System dams has resulted in a lowering of the tailwater elevation. Two types of sediment deposits exist in the reservoirs: those occurring generally over the reservoir bottom, mostly composed of the finer material and those occurring in a characteristic delta formation at the head of the reservoir and where tributaries enter the reservoir, which include coarser material.

3.5.1 Degradation Trends – Downstream of Fort Peck Dam

Degradation in the reach downstream of Fort Peck Dam has been evaluated in a series of studies (USACE, 2012, 2013c). Degradation begins below Fort Peck Dam and gradually decreases in magnitude in the downstream direction to approximately RM 1597 which is about 15 miles upstream of the Yellowstone River confluence. At the Fort Peck Dam tailwater, degradation of about 0.5 to 1.5 feet has been observed since 1950 at normal flows of 10,000 to 30,000 cfs. The maximum amount of degradation in the reach since 1950 is about 5 feet at normal flows. The historic 2011 flood and period of sustained high flows led to degradation throughout the reach. Near the downstream end of the degradation reach, at the Culbertson gage (RM 1620.76), a normal flow water surface elevation decrease of 1 to 3 feet has been observed from 1950 to 2012 (USACE, 2013c).

3.5.2 Aggradation Trends – Lake Sakakawea Headwaters

A trend of aggradation due to the Lake Sakakawea headwaters has been seen in the reach below RM 1590.2, or about 12 miles upstream of the Yellowstone River confluence, and increases in the downstream direction. Sources of sediment in Lake Sakakawea are upstream Missouri River sediment load, sediment load from tributaries (including the Little Missouri River, Yellowstone River, and Milk River), overland sheet flow, and shoreline erosion. The storage capacity of Lake Sakakawea decreased approximately 1,000,000 acre-feet from 1953 to 2012 (USACE, 2014a).

3.6 FLOOD HISTORY

The largest flood prior to the construction of the System, above the Yellowstone River, was the flood of 1908. This flood was caused by heavy rainfall in the latter part of May and early days of June followed by the occurrence of a severe rainstorm on 3-6 June. This rain accompanied by the mountain snowmelt, caused basin-wide flooding and considerable damage. The estimated crest discharge in the reach from Fort Peck to the mouth of the Yellowstone River was 154,000 cfs.

The largest flood prior to the construction of the System, below the Yellowstone River, was the flood of 1952. Flooding was continuous from the Yellowstone River to the mouth due to flooding on most of the tributaries above Sioux City. The winter of 1951-52 had one of the heaviest snow covers in the upper plains with a high water content and an unusually cold winter. In late March, rapid melting of snow cover began. The Missouri River crested at Elbowoods, ND, below the mouth of the Little Missouri River, on April 5, 1952, establishing a record stage of 25.2 feet and discharge of 360,000 cfs.

Since the System first filled in 1967, the largest flood event was in 2011. During 2011, a record amount of runoff occurred due to melting snowpack and record rainfall over portions of the upper basin. Annual runoff into the System is estimated to be 60.8 MAF. As a result of the record runoff, record releases from all of the System dams occurred: 65,000 cfs at Fort Peck, 150,000 cfs at Garrison, 160,000 cfs at Oahe, 166,000 cfs at Big Bend, 160,000 cfs at Fort Randall, and 160,000 cfs at Gavins Point. A summary of the peak flows at the Culbertson, MT and Wolf Point, MT gages for each water year are shown in Figure 3-2 Figure and Figure 3-3.

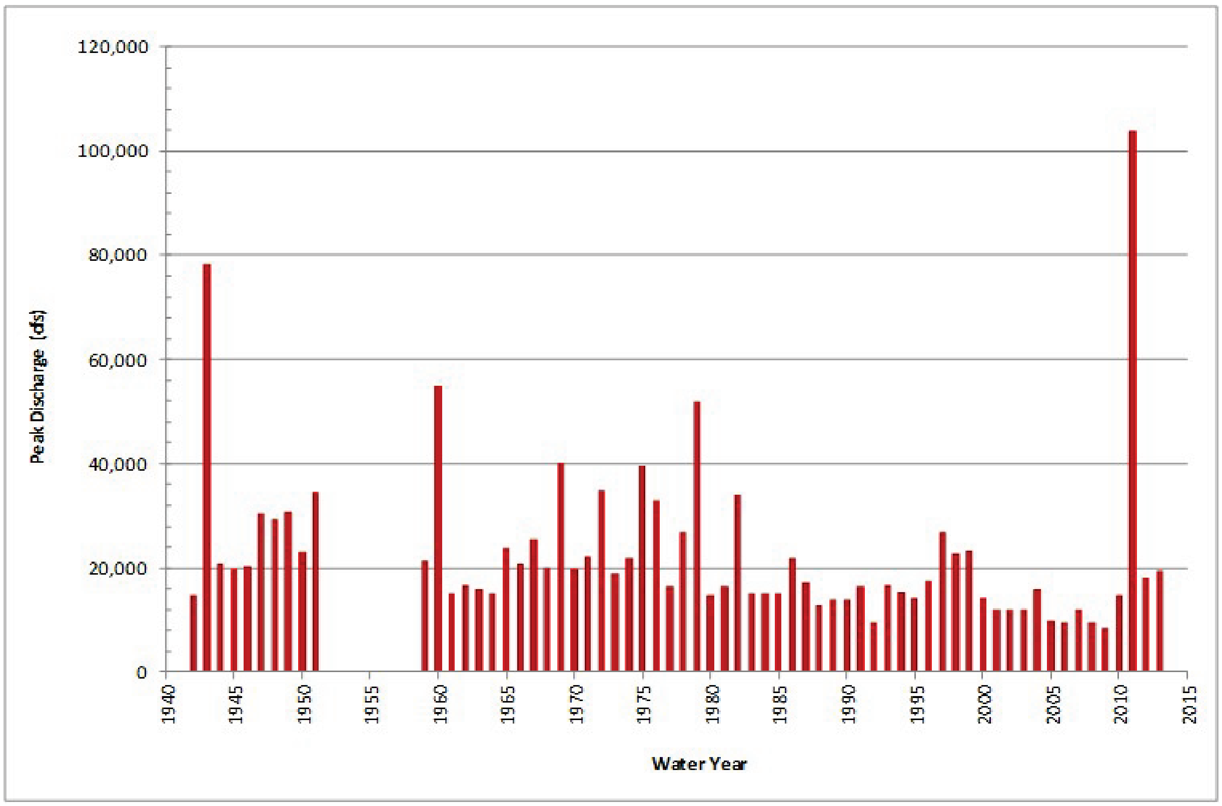

Figure 3-2: Missouri River at Culbertson, MT Annual Peak Flows

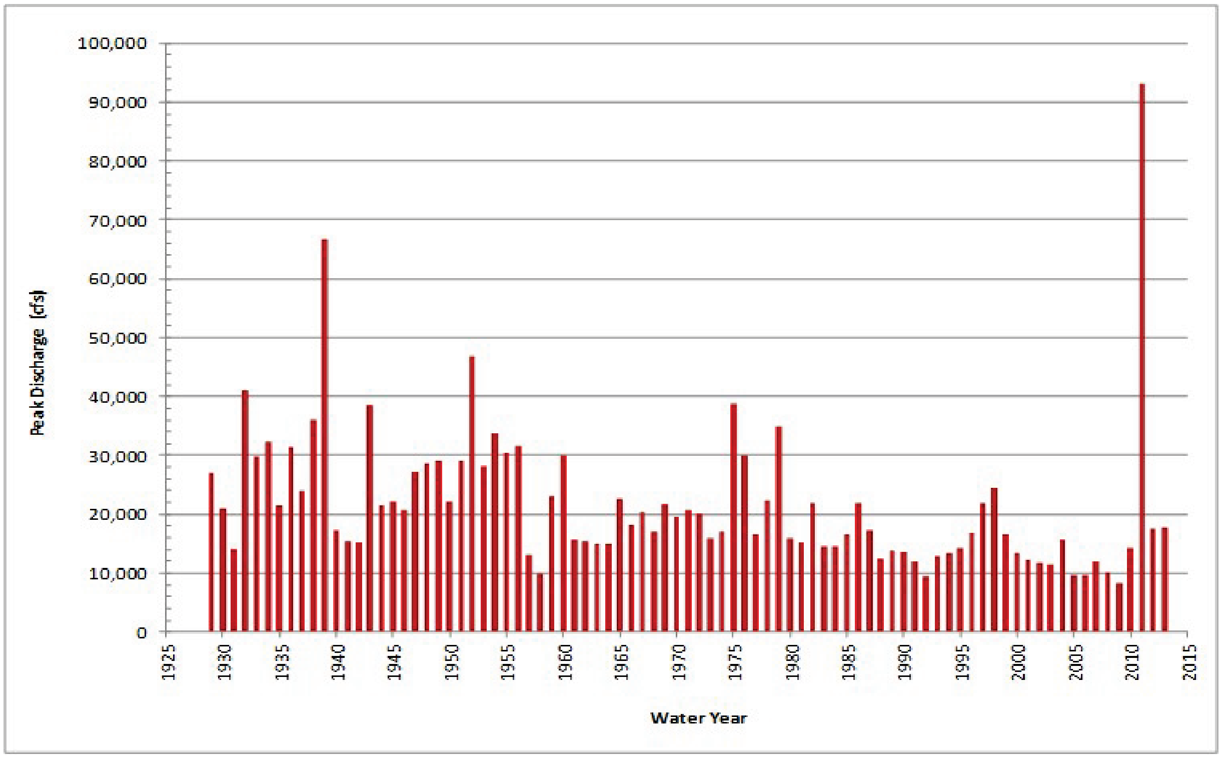

Figure 3-3: Missouri River at Wolf Point, MT Annual Peak Flows

4 DATA SOURCES

Primary data sources for construction of the unsteady HEC-RAS model includes terrain data, bathymetry data, and gage data. Terrain data encompasses everything from the bluffs to the riverbanks, defining the floodplain and overbanks, but does not often include data below the surface of the river. Bathymetry captures the cross section geometry below the water surface. Gage data provides the flow boundary conditions for the model and stage calibration targets. A summary of the data used in the model is provided in Table 4-1.

Table 4-1: Summary of Data Sources

Data Type	Data Title	Location Data Applied to Model	Collection Dates
Topographic Data			
Sediment Range Survey	Missouri River Hydrographic Surveys below Ft. Peck, Montana: River Miles 1865.7 to 1693.4	RM 1769.04 - 1597.17	Oct 2011 & Apr - May 2012
Sediment Range Survey	Lake Sakakawea and Tributaries located in west central North Dakota	RM 1594.24 - 1391.08	11-13 Sep 2011 & 1 May - 22 Aug 2012
Hydrographic Survey	Hydrographic Surveys of the Yellowstone River: Yellowstone River Miles 0 to 121.4 and Missouri River Miles 1552.6 to 1586.6	RM 1585.97 to 1552.61	Apr - Jun 2012
DEM – LiDAR	Fort Peck to Yellowstone LiDAR Mapping	Fort Peck Dam - RM 1586.74	10-12 Nov 2011
DEM – LiDAR	Yellowstone River Corridor - McKenzie County	RM 1585.28 - 1579.41	15 Oct 2007 - 2 Nov 2007
DEM – 4 m	NEXTMap	RM 1586.39 - 1585.97, RM 1578.98 - Garrison Dam	Apr - Dec 2007
Levee Profile	Williston Levee - Levee Profile (2009)	Williston Lateral Structure	2009
Land Cover			
Land Cover	National Land Cover Dataset 2006	All cross sections	2006
Flow Data			
Streamgage Data	Stage and Discharge	All cross sections	POR
Hydrologic Statistics	Release and Pool Duration for Fort Peck and Lake Sakakawea	All cross sections	POR
Water Surface Profile			
Water Surface Elevation Data	Missouri River Water Surface Profile from Fort Peck to Lake Sakakawea	All Sediment Ranges	11-13 Sep 2012

4.1 Terrain Development

A variety of terrain sources were available for this stretch of the Missouri River and floodplain. Described below are the source, collection methods and dates, and accuracy of each.

4.1.1 Sediment Range Surveys

Sediment range surveys for the main stem Missouri River from Fort Peck Dam to just upstream of Lake Sakakawea were collected in October 2011 and April to May 2012 by Eisenbraun and Associates, Inc for the Omaha District. The range surveys included topographic and hydrographic data. The data was collected using the horizontal coordinate system Montana

State-Plane Coordinate System North American Datum of 1893 (NAD 83), zone 2500, epoch 2002 in US survey feet. The elevations were surveyed in the vertical coordinate system North American Vertical Datum of 1988 (NAVD 88), geoid 2009 in US survey feet (Eisenbraun and Associates, Inc., 2012a).

Additionally, Eisenbraun and Associates, Inc. collected sediment range surveys of Lake Sakakawea in September 2011 and May to August 2012 for the Omaha District. This data was collected using the same methods described above with horizontal projection North Dakota State-Plane Coordinates System NAD 83, North Zone 3301 and vertical projection NAVD 88 (Eisenbraun and Associates, Inc., 2012b).

4.1.2 DEMs and LiDAR

Three LiDAR data sets were available for this stretch of the Missouri River. The first was a 1-meter Digital Elevation Model (DEM) collected in November 2011 from Fort Peck Dam to the Montana and North Dakota State line by Fugro Horizons, Inc. for the St. Louis District. The DEM is in the horizontal datum of Montana State-Plane NAD 83 and vertical datum of NAVD 88. The vertical accuracy of the LiDAR data was computed using proprietary software that compares the ground control coordinate with the surface the LiDAR data generates, and finds the residuals of the ground control points and calculates the RMS of the control. The RMS of the control compared to the LiDAR surface was calculated to be 10.0 cm in open areas (Fugro Horizons, Inc., 2012).

Another available LiDAR data set was collected in October through November 2007 in McKenzie County, North Dakota for Montana Department of Natural Resources and Conservation (DNRC). This is a 2.5-meter DEM in Montana State-Plane NAD 83 and NAVD 88 (Montana DNRC 2010). The horizontal accuracy meets or exceeds a 4.5 foot horizontal accuracy at 95 percent confidence level using RMSE(r) x 1.7308 as defined by the FGDC Geospatial Positioning Accuracy Standards. The vertical accuracy meets or exceeds a 0.6 foot vertical accuracy at 95 percent confidence level using RMSE(r) x 1.9600 as defined by the FGDC Geospatial Positioning Accuracy Standards.

Lastly, a NEXTMap 4-meter DEM was available that was collected in April through December 2007 by Intermap Technologies. This DEM was in Montana State-Plane NAD 83 and NAVD 88. The horizontal accuracy is 2 meters RMSE or better in areas of unobstructed flat ground. The vertical accuracy is 1 meter RMSE or better in areas of unobstructed flat ground (Intermap Technologies, Inc., 2008).

4.1.3 Land Cover

The United States Geographical Survey (USGS) National Land Cover Database 2006 (NLCD 2006) was used in the determination of appropriate Manning's n roughness values for overbank data. The NLCD 2006 is a 16-class land cover classification scheme at a spatial resolution of 30 meters and is based primarily on a 2006 Landsat satellite data. This is a raster digital data set (USGS, 2012).

4.1.4 Williston Levee Profile

The only levee in this stretch of the Missouri River is the Williston Levee located in Williston, North Dakota. A survey was taken of the elevation of the Williston Levee in 2009 (USACE, 2009).

4.2 BATHYMETRY

The bathymetry data available was a part of the sediment range survey data described in section 4.1.1. This hydrographic data was collected by Eisenbraun and Associates, Inc. in 2011 and 2012 (Eisenbraun and Associates, Inc., 2012a and 2012b) for the Omaha District. Additionally, hydrographic data was collected along the Yellowstone River and on a portion of the Missouri River centered around the confluence of the Yellowstone River in April to June 2012 by Eisenbraun and Associates, Inc. The hydrographic data was collected in Montana State-Plane NAD 83 and NAVD 88 (Eisenbraun and Associates, Inc., 2012c).

4.3 OBSERVED DATA

Water surface profiles are surveys periodically performed by the Omaha District Corps of Engineers that provide a water surface elevation for a given river mile. Stream stage and flow data available on the Missouri River include gages along the Missouri River main stem, and gages on many of the major tributaries. All gages are operated by the USGS and collect stage data remotely, usually at intervals of 15 minutes. Availability and quality of these datasets influenced the configuration of the model as well as the timeframe for calibration.

4.3.1 Water Surface Profile Data

Water surface profile elevation data was taken at every sediment range by Eisenbraun and Associates, Inc. using two boats between September 11-13, 2012, and this was used as the baseline for calibration of the model (Eisenbraun and Associates, Inc., 2012d).

4.3.2 USGS Gage Flow and Stage Data

Stream gage data was obtained through the USGS National Water Information System (NWIS) or, if not available online, from each state's USGS Water Science Center personnel for all applicable gages in this reach of the Missouri River and tributaries (USGS, 2012). Table 4-2 lists the main stem USGS gages and Table 4-3 lists the tributary USGS gages. Figure 4-1 is a map of the gage locations.

Table 4-2: USGS Missouri River Main Stem Gages

Gage Name	River Mile	Gage Number	Flow Data Dates	Stage Data Dates
Below Ft. Peck Dam, MT	1763.54	06132000	Apr 1934 - *	Aug 2011 - *
W Frazer Pump Plant	1750.99	06175100	n/a	July 2010 - *
Near Wolf Point, MT	1701.31	06177000	Oct 1928 - *	Oct 1989 - *
Near Culbertson, MT	1620.65	06185500	July 1941 - *	Oct 1989 - *
No. 4 near Nohly, MT	1597.40	06185600	n/a	Mar 1959 - *
No. 5 near Nohly, MT	1588.95	06185650	n/a	Apr 1959 - *
No. 5A at Buford, ND	1581.50	06329640	n/a	Apr 1960 - *
No. 6 near Buford, ND	1577.03	06329650	n/a	Apr 1959 - *
Near Williston, ND	1552.61	06330000	n/a	Apr 1966 - *
No. 9 at Williston, ND	1546.20	06330110	n/a	Apr 1959 – Sep 2006, May 2009 - *

* - indicates that this is a current gage

Table 4-3: USGS Tributary Gages

Gage Name	Gage Number	Confluence River Mile	Modeled or Lateral Inflow	Available Flow Data Dates
Milk River at Nashua, MT	06174500	1761.50	Modeled	Oct 1939 - *
Poplar River near Poplar, MT	06181000	1678.90	Modeled	Aug 1908 - *
Yellowstone River near Sidney, MT	06329500	1581.70	Modeled	Oct 1910 - *
Little Muddy River below Cow Creek near Williston, ND	06331000	1546.20	Lateral Inflow	Jun 1954 - *
Little Missouri River near Watford City, ND	06337000	1436.10	Lateral Inflow	Oct 1934 - *

* - indicates that this is a current gage

4.3.3 Fort Peck Dam Releases and Lake Sakakawea Pool Elevations

The observed Fort Peck releases and Lake Sakakawea (Garrison Pool) elevations were obtained from the NWO Corps Water Management System (CWMS) databaseand were used as the upstream and downstream boundary conditions.

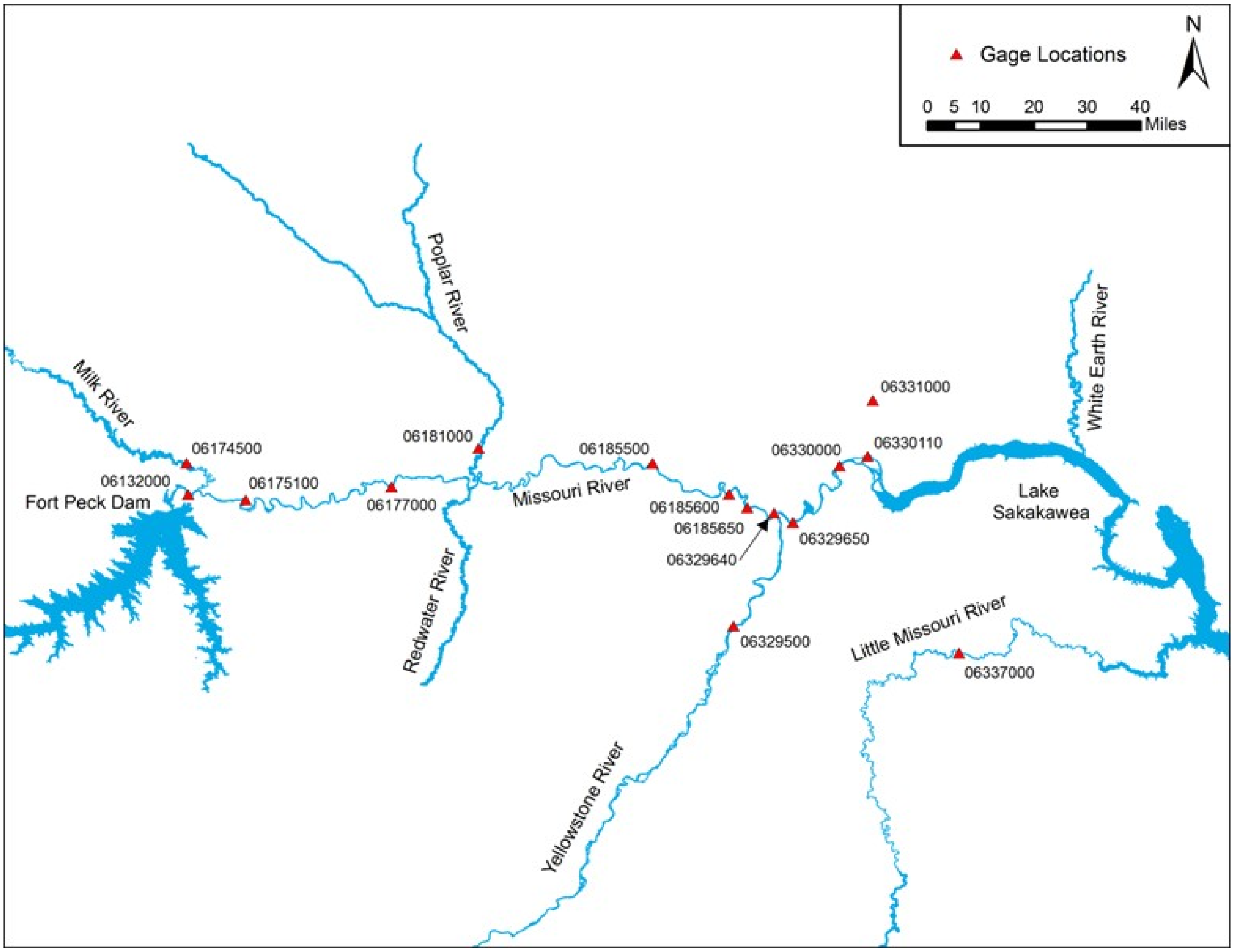

Figure 4-1: Gage Location Map

5 MODEL DEVELOPMENT

Model development includes the software version used, descriptions of the various geometry components of the model, and boundary conditions selected. The following sections outline the details of the model construction including fundamental assumptions, data sources for specific geometry features, techniques used, and justification for any unique parameters and decisions made during the process of building the model.

5.1 HEC-RAS

Unsteady computations in HEC-RAS version 4.2 Beta were used for this modeling effort. A computation interval of 4 hours was used because that was determined to be a stable time-step for the model and allowed model runs to be conducted in reasonable timeframes.

HEC-RAS has been significantly updated since version 4.1, and it is not recommended that the model be run in 4.1 or any earlier version.

HEC-RAS version 5.0 beta has been released and the model has been minimally tested in this version. The goal is to run the model in the newest version (not beta version), presumably version 5.0.

5.2 GEOMETRY

This section will discuss the development of the HEC-RAS model geometry for the Missouri River reach from Fort Peck to Garrison, including vertical datum and horizontal projection, the stream centerline and cross section geometry, Manning's n-values, and the modeling of structures such as bridges, dams, and levees. Geometries of the tributaries used in the model were developed outside of this project and were added after the completion of the Missouri River geometry. The Yellowstone River was modeled by Omaha District as a part of a different project. The remaining tributaries, the Milk River and Poplar River, were modeled by West Consultants(WEST, 2012).

5.2.1 Vertical Datum and Projection

The vertical datum for the Fort Peck to Garrison unsteady HEC-RAS model is NAVD88 to match the LiDAR data. Most of the other elevation data is referenced to the NGVD29 vertical datum; therefore a conversion factor was used to convert the data to NAVD88. See Table 5-1 for a list of vertical conversion factors used in the model. The program CorpsCon was used to obtain the conversion values based on the gage's coordinates. CorpsCon is a widely accepted standard practice for converting between NGVD29 and NAVD88 vertical datums. However, it has been found that discrepancies exist between the CorpsCon conversion values and actual re-survey of points in the NAVD88 datum.

The current horizontal projection is NAD 83 Montana State-Plane Coordinate System (US-Feet) as this is what most of the available terrain data was in. Re-projection to a nation-wide projection may be necessary after review and certification for compatibility with other HEC-RAS models and the ResSim models that are in UTM projections. Re-projecting a HEC-RAS model to a national projection is not difficult or time consuming, and there is a documented How-To procedure provided by HEC.

Table 5-1: Gage Vertical Datum Conversion Factors

Gage Number	Gage Name	Conversion Factor (from NGVD29 to NAVD88) (ft)
06132000	Missouri River below Ft. Peck Dam, MT	2.077
06175100	Missouri River at W Frazer Pump Plant	1.982
06177000	Missouri River near Wolf Point, MT	1.955
06185500	Missouri River near Culbertson, MT	1.732
06185600	Missouri River No. 4 near Nohly, MT	1.765
06185650	Missouri River No. 5 at Nohly, MT	1.778
06329640	Missouri River No. 5A at Buford, ND	1.732
06329650	Missouri River No. 6 near Buford, ND	1.749
06330000	Missouri River near Williston, ND	1.640
06330110	Missouri River No. 9 at Williston, ND	1.598
N/A	Lake Sakakawea (Garrison Pool)	1.309

*Conversion factor for Lake Sakakawea pool elevations used the location where the elevation is recorded. For this pool, that is at the intake structures.

5.2.2 Stream Centerline

One stream centerline for the Missouri River was developed in GIS for all of the Omaha District HEC-RAS models. A centerline from a previous study was modified to match the current state of the river, making sure to follow the center of mass of flow and avoiding crossing sandbars. It should be noted that the centerline defined in the model does not match the 1960 river miles line. Cross sections were named based on the 1960 river miles, therefore the reach lengths will not match up with the river miles.

5.2.3 Cross Section Geometry

The geometry of the cross sections were constructed using the most recent sediment range surveys, which included topographic and hydrographic data, in conjunction with the DEMs. The cross sections used survey data where possible and extended as necessary with DEM data. The sediment ranges are generally spaced 1 to 3 miles apart on this stretch of the Missouri River. It was determined that cross sections shall be spaced a maximum of 3000 feet apart on the river portion of the Missouri River. For Lake Sakakawea, the sediment range spacing was considered sufficient for modeling the impounded segment of the river. To obtain the desired spacing, additional cross sections were added between the sediment ranges using LiDAR or DEM data for the overbank extents and for the channel data, either RAS interpolated bathymetry or channel data from a nearby range was used. Attachment 1 provides a more detailed description of how the interpolated cross section's bathymetry was estimated. Additional hydrographic data, not from the range surveys, was available for the cross sections between RM 1585.97 to 1552.61, near the confluence with the Yellowstone River. Banklines for all the cross sections were set at approximately the 2-yr water surface elevation. Cross sections were named based on the 1960 river miles, since this is the primary method used to identify locations on the Missouri River. However, the 1960 river miles do not match up with the stream centerline, which produces reach lengths that do not match the river miles.

5.2.4 Manning's N-values

Manning's n-values in overbank areas were determined based on the land use classification from the NLCD 2006 data. The land cover values were condensed from the NLCD 2006 standards into 12 classes, as shown in Table 5-2. The land cover GIS shapefile was manually updated with the use of recent aerial images for changes to the river channel, mostly due to the 2011 flood event.

In the river channel, an initial Manning's n-value of 0.025 was used throughout the entire model. During calibration, n-values were modified between 0.022 to 0.031, which were determined to be reasonable channel roughness values for the Missouri River. Roughness values were generally changed in a reach wide manner of 10 to 30 mile long blocks. Final roughness values for the main channel are shown in Table 5-3. Manning's n-values for overbank areas were not modified in calibration.

Table 5-2: Land Use Reclassification and Initial Roughness Values

NLCD Number	NLCD Classification	Reclass Number	Reclassification for Model	Manning's N-Value
11	Open Water	11	Water[1]	0.025
		12	Channel Sandbar	0.032
		13	Channel Sandbar Light Vegetation	0.038
		14	Channel Sandbar Heavy Vegetation	0.052
		15	Channel Bank	0.056
21	Developed, Open Space	2	Urban	0.080
22	Developed, Low Intensity			
23	Developed, Med Intensity			
24	Developed, High Intensity			
31	Barren Land (Rock/Sand/Clay)	3	Sand	0.028
41	Deciduous Forest	4	Trees	0.070
42	Evergreen Forest			
43	Mixed Forest			
51	Dwarf Scrub	5	Scrub Brush	0.060
52	Shrub/Scrub			
71	Grassland/Herbaceous	6	Grass	0.035
72	Sedge/Herbaceous			
73	Lichens			
74	Moss			
81	Pasture/Hay			
82	Cultivated Crops	7	Crops	0.045
90	Woody Wetlands	8	Wetlands	0.055
95	Emergent Wetlands			

[1] Initial roughness value that was modified during the calibration process.

Table 5-3: Final Channel Roughness Values

Cross Section River Mile Range	Channel Manning's N-Value
1769.04 – 1760.74	0.031
1760.30 – 1752.92	0.025
1752.43 – 1742.60	0.022
1742.13 – 1732.58	0.029
1732.09 – 1720.19	0.025
1719.65 – 1703.07	0.022
1702.49 – 1671.30	0.027
1670.83 – 1662.57	0.025
1661.89 – 1638.31	0.03
1637.92 – 1631.78	0.025
1631.39 – 1613.32	0.022
1612.75 – 1567.86	0.025
1567.50 – 1554.18	0.022
1553.71 – 1539.93	0.031
1539.41 – 1528.05	0.022
1527.40 – 1391.08	0.025

5.2.5 Levee Modeling with Lateral Structures and Storage Areas

The Williston Levee and its protected area were modeled in HEC-RAS as a lateral structure and storage area. Not intended in any way to imply that these areas were designed to store water, the term refers to HEC-RAS features used to model flows affected by these features. Storage areas are described within the RAS model with lateral connections used to transfer flow from the main river channel into the storage area. The data for the levee crest elevation was obtained from the 2009 survey profile by USACE. The storage area's elevation-volume curve was calculated using GeoRAS and the best available terrain data.

5.2.6 Bridges

On the Missouri River main stem, cross sections representing bridge embankments are in the model, however the structures themselves are not. This was a simplification made to keep computation times shorter. In addition, all bridge deck low chords on the Missouri River are elevated higher than the floods of record, thus the only component other than the embankment that would impede water flow is the bridge columns, which likely have a local effect, but not global. This was determined to be sufficient for the Missouri River modeling. Bridges in the tributary models were left in the geometry.

5.2.7 Dams

This stretch of the Missouri River was modeled just downstream of Fort Peck dam to just upstream of Garrison dam, so the dams themselves are not in the model. The pool of Garrison Dam, Lake Sakakawea, is in this HEC-RAS model and is the downstream boundary condition.

5.2.8 Tributaries

Tributary reaches were included within the model to route flow from the gage station to the Missouri River and were not calibrated to stage. Three tributary routing reaches are included in the model as previously shown in Table 4-3.The modeling of the Yellowstone River was done by Omaha District for another study, while the remaining two tributaries were modeled by West Consultants (WEST, 2012). In general, the goal with the tributary routing reaches was to model travel time sufficiently well from the tributary gage station to the Missouri River and preserve timing for calibration purposes. No tributary computed stage information should be used from model results without carefully assessing the purpose and considering model construction limitations.

The tributary RAS models were converted to the correct vertical datum and horizontal projection and inserted into the Missouri River geometry with junctions. Junction lengths were assumed to represent the average distance that the water will travel from the last cross section in the reach to the first cross section of the following reach (USACE, 2010). For junction calculations, either the energy method or force equal water surfaces method was chosen based on model stability.

5.3 Boundary Conditions

The boundary conditions are the initial flows and stages used at the upstream and downstream extents of the HEC-RAS model. Below is a discussion of those boundary conditions.

5.3.1 Upstream Boundary Conditions

Upstream boundary conditions include the outflow from Fort Peck Dam and observed USGS flow hydrographs at the upstream end of each of the three tributary reaches. Hourly data was used when available and daily data was used to complete the flow record. To achieve stability, a minimum flow was used for each input, as shown in Table 5-4.

Table 5-4: Minimum Flows

Location	Minimum Flow (cfs)
Fort Peck Outflow	2,000
Milk River	50
Poplar River	50
Yellowstone River	500

5.3.2 Downstream Boundary Condition

The downstream boundary condition is the stage hydrograph of Garrison Dam's pool, Lake Sakakawea, from Omaha District's CWMS database.

5.3.3 Ungaged Inflow

Ungaged inflow refers to that portion of the flow that is not captured by the gage station records. Ungaged inflow computation has been automated within HEC-RAS and is fully described within the User's Manual (USACE, 2010). Ungaged calculations are made between two gages on the main stem which have a continuous record of both stage and flow.

The ungaged flow calculation is made by running the unsteady model with internal stage and flow boundaries at the gage locations mentioned above. At the endpoint, the calculated routed flow hydrograph is compared to the observed hydrograph, and the difference is calculated. The difference is put back into the model between the two gages at user specified locations with a backwards lag in time and the model is run again. This process is repeated until the flow at the endpoint either matches the flow convergence desired or meets the maximum number of iterations specified.

Lag time was input as the approximate travel time from the lateral inflow location to the gage station. For uniform lateral inflows, the travel time from the midpoint of the segment to the gage was used. Average velocity in the Fort Peck to Garrison reach of the Missouri River is about 3 ft/s, or 2 mi/hr. Simultaneous was selected as the optimization mode. The simultaneous option makes ungaged calculations for each reach independent of the others, whereas the sequential option runs calculations for each reach in order of upstream to downstream taking into account any lack in flow convergence that may have occurred in the upstream reach.

Execution of the ungaged inflow for the calibration period was problematic and had to be executed in several phases. In addition, HEC-RAS 4.2 Beta version contained a bug which did not allow for use of levee connections while computing ungaged. The Williston Levee was not overtopped or breached during the high 2011 flows. Therefore, ungaged inflow was determined with the levee connections removed.

Negative flows computed as ungaged are common. This is caused by a number of reasons including gaged inflow error, model timing, areas with significant water use or groundwater recharge, and similar. Ungaged inflow hydrographs were reviewed and determined as reasonable. Calibration accuracy was improved by using the determined ungaged inflows.

Ungaged inflow parameters are entered within the unsteady flow analysis options menu. Within the HEC-RAS model, flow / stage gage records are available at Wolf Point and Culbertson as shown in Table 4-2. Ungaged flow within each reach was distributed by prorating the remaining drainage area after the gage station drainage areas are removed. Input parameters for each of the ungaged inflow computation sections are shown in Figure 5-1 and Figure 5-2.

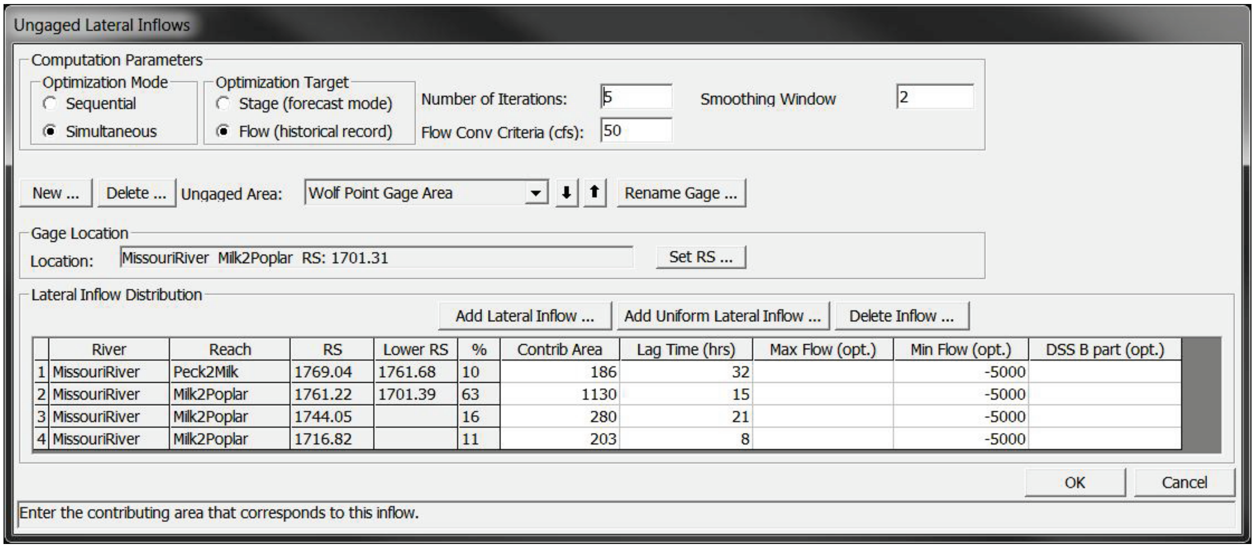

Figure 5-1: Ungaged Inflow Fort Peck to Wolf Point

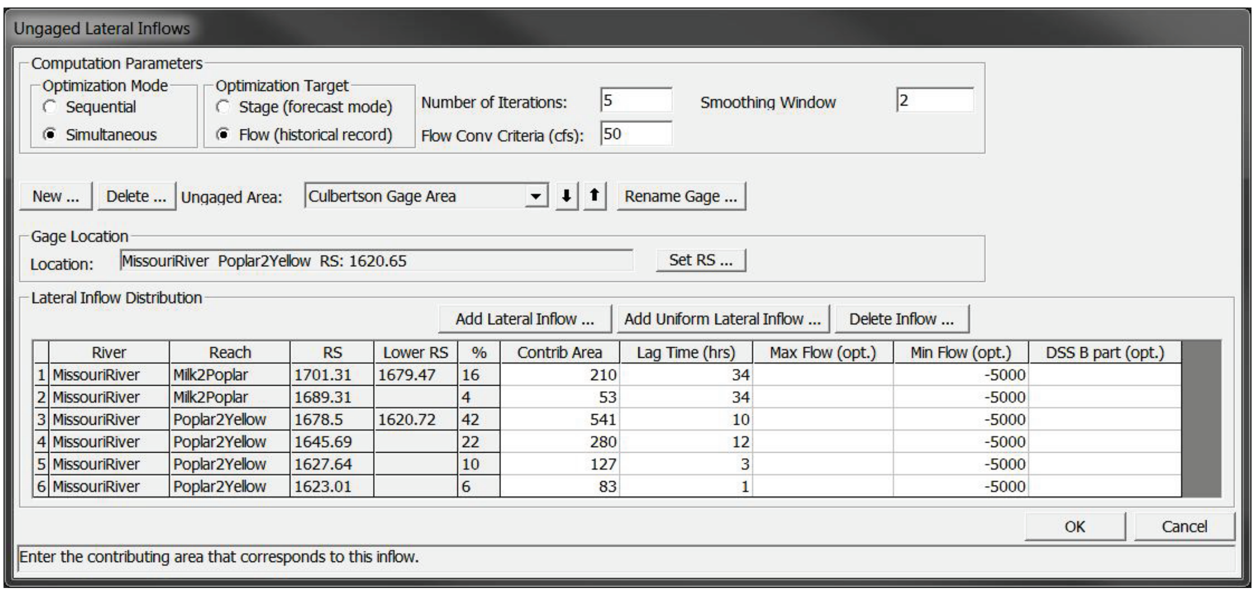

Figure 5-2: Ungaged Inflow Wolf Point to Culbertson

6 CALIBRATION

Model calibration was accomplished through several steps described in this section. Results as well as a discussion of level of calibration achieved and overall model performance are presented below.

6.1 MODEL CALIBRATION

Unlike previous modeling efforts on the Missouri river, a broad spectrum of flows from low flows to high flows were considered important to the project purposes. Calibration methods had to include a range of flows. The primary source of calibration data was observed stage and flow hydrographs on the main stem Missouri river gages and field measured water surface profile data that was surveyed in September 2012.

First, the model was calibrated in a steady state for geometry. A thorough check of the estimated bathymetry was performed. At various flows, output values were checked for consistency to avoid sudden changes from one cross section to the next. The output analyzed included flow distribution (overbanks and channel), top width, velocity, energy grade, and flow area. Cross section interpolations were revised based on this analysis. The steady state model was calibrated to the water surface profile collected in September 2012 by adjusting channel n-values. This was the only water surface profile of this reach available to use for calibration. The channel n-values were initially set at 0.025 and adjusted for steady state calibration to obtain a water surface elevation that was within a tolerance of the measured water surface profiles.

Second, the model was run in the unsteady state with steady flows to obtain a stable model. Then, one by one, tributary geometries were added into the model. The tributaries in the model were roughly calibrated and were inserted for the primary purpose of routing flows from the gage to the Missouri River for the unsteady model runs to preserve flow timing. Tributary computed stages will not be used in the analysis. Once the model was stable with all the tributaries added, the observed flows were added to the model as well as the computed ungaged flows. The model was run from January 2011 to December 2012 and results were compared to the September 11-13, 2012 observed water surface profile as well as stage and flow from gages, where available. Multiple iterations were required in this process with roughness values and ineffective flow locations.

Calibration philosophy was to primarily use the base roughness values to calibrate the model for normal flows and use the HEC-RAS option for flow roughness and adjustments to ineffective flow areas to calibrate for higher flow events. Flow roughness factors were used to calibrate to the 2011 high flow event as shown in Table 6-1.

Table 6-1: Flow Roughness Factors

U/S Cross Section	1761.22	1707.25	1678.5	1610.52	1594.24	1581.35
D/S Cross Section	1707.87	1679.47	1611.04	1594.64	1582.01	1391.08
Flow (cfs)	**Roughness Factor**					
0	1	1	1	1	1	1
20,000	1.2	1	1	1	1	1
25,000	1.3	1.2	1.15	1.1	1.1	1.1
30,000	1.4	1.25	1.15	1.15	1.15	1.15
50,000	1.4	1.25	1.1	1.15	1.2	1.2
70,000	1.4	1.1	1	1.1	1.2	1.2
90,000	1.4	1	0.95	1.1	1.2	1.2
110,000		1	0.95	1.1	1.2	1.2
130,000						1.2
150,000						1.2
180,000						1.2

The calibration goal was to achieve a water surface elevation within 1 ft for the entire reach and within 0.5 ft for most of the reach for both the measured water surface profiles and the observed gage data for 2011 and 2012, excluding periods of ice. The model does not account for ice. Ice causes much higher stages than would normally occur for an open water condition. Ice affected events typically occur from December to March. Plate 2 through Plate 21 are the hydrographs and computed minus observed stage vs flow plots for the gage locations. Plate 22 through Plate 38 show the computed profile vs the measured water surface profile. Multiple profiles are shown because due to the size of the reach, the water surface profile survey took several days to complete. Notes describing the survey schedule are included in the plots when the stage was not steady throughout the survey period.

6.2 Calibration Results

Model calibration results are within the desired range with most locations within 0.5 to 1 foot of observed stages.The results can be seen in Plate 2 through Plate 38. In general, comparison of model results to gage station hydrographs was reasonable. The measured profile calibration also provides confidence in model performance between the gage station locations. A comparison of peak stages for the 2011 flood are shown in Table 6-2.

Table 6-2: 2011 Flood Peak Stage Comparison

Location	Date	Peak Stage Difference (ft)
RM 1763.54 – blw Ft. Peck	M	M
RM 1750.99 – W Frazer Pump Plant	13Jun2011	-0.07
RM 1701.31 – Wolf Point	14Jun2011	-0.30
RM 1620.65 – Culbertson	21Jun2011	0.13
RM 1597.40 – No. 4 nr Nohly	22Jun2011	0.23
RM 1588.95 – No. 5 nr Nohly	22Jun2011	-0.08
RM 1581.50 – No. 5A at Buford	21Jun2011	0.35
RM 1577.03 – No. 6 nr Buford	M	M
RM 1552.61 – Williston	22Jun2011	0.21
RM 1546.20 – No. 9 at Williston	22Jun2011	0.13

*M – denotes gage peak stage data is missing

*Peak stages were manually estimated due to minor timing issues and bad data points.

6.2.1 Calibration Results Affected by Ice Conditions

Ice affected conditions including ice cover, ice breakup, and ice jams occur annually within the basin. Ice formation conditions typically occur in late November to late December with iceout typically occur in the early spring, usually in the March to April time frame. No ice parameters were included in the model development or calibration. Therefore, winter condition model calibration results should be viewed with caution and recognize that results do not reflect observed conditions.

6.2.2 Stage Trend Impacts

Degradation and aggradation conditions occur through the reach due to Fort Peck Dam at the upstream model boundary and Garrison Dam at the downstream model boundary. Due to the extreme 2011 event flows and the high degree of channel adjustment that occurred during the event, accurate stage calibration prior to 2011 using the post-2011 event model geometry is not possible. Model results for the rising portion of the event in May and June demonstrate how stage-flow relationships changed during the flood and also reduce calibration accuracy through this portion of the event.

7 CONCLUSIONS

The model performs well for the 2011 and 2012 observed gage data and is calibrated to the 2012 water surface profile. Significant points to consider with respect to model construction and calibration are as follows:

- Measured profile calibration in 2012 and gage hydrograph calibration for both 2011 and 2012 indicates that the model performs satisfactorily with a stage calibration accuracy of less than 1 foot at most locations.

- Incomplete hydrographic surveys were available to construct the model. Interpolation from hydrographic sections was used combined with LiDAR data to generate cross sections at the desired spacing of about 2,500 to 3,000 feet. Consequently, the interpolated sections within the model have reduced accuracy for the below water portion of the cross section. Normal flow calibration indicated that the model performs satisfactorily which implies the interpolation method was reasonable.
- Floodplain model geometry in the reach below Williston is limited due to the use of less accurate DEMs.
- No tributary computed stage information should be used from model results without carefully assessing the purpose and considering model construction limitations.
- Aggradation and degradation that occurred during the 2011 event reduces calibration accuracy for the flood hydrograph. This also prevents calibrating to flow events prior to 2011.
- Ungaged inflows are an important parameter in model calibration. Computation of ungaged inflow with HEC-RAS appeared to enhance model flow accuracy compared to observed flow at the gaging stations.

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APPENDIX A

FORT PECK DAM TO GARRISON DAM

PLATES

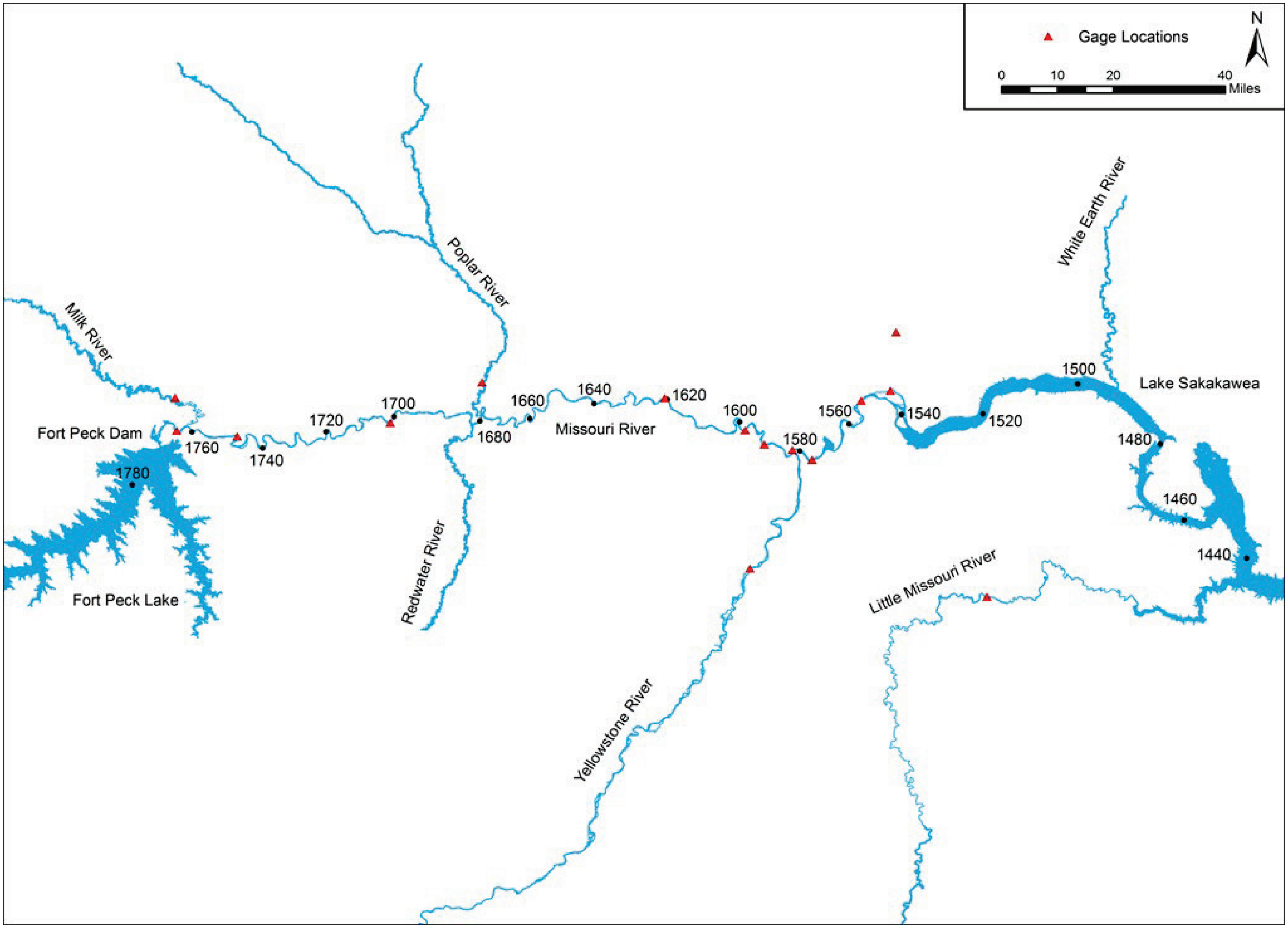


Plate 1: Overview Map

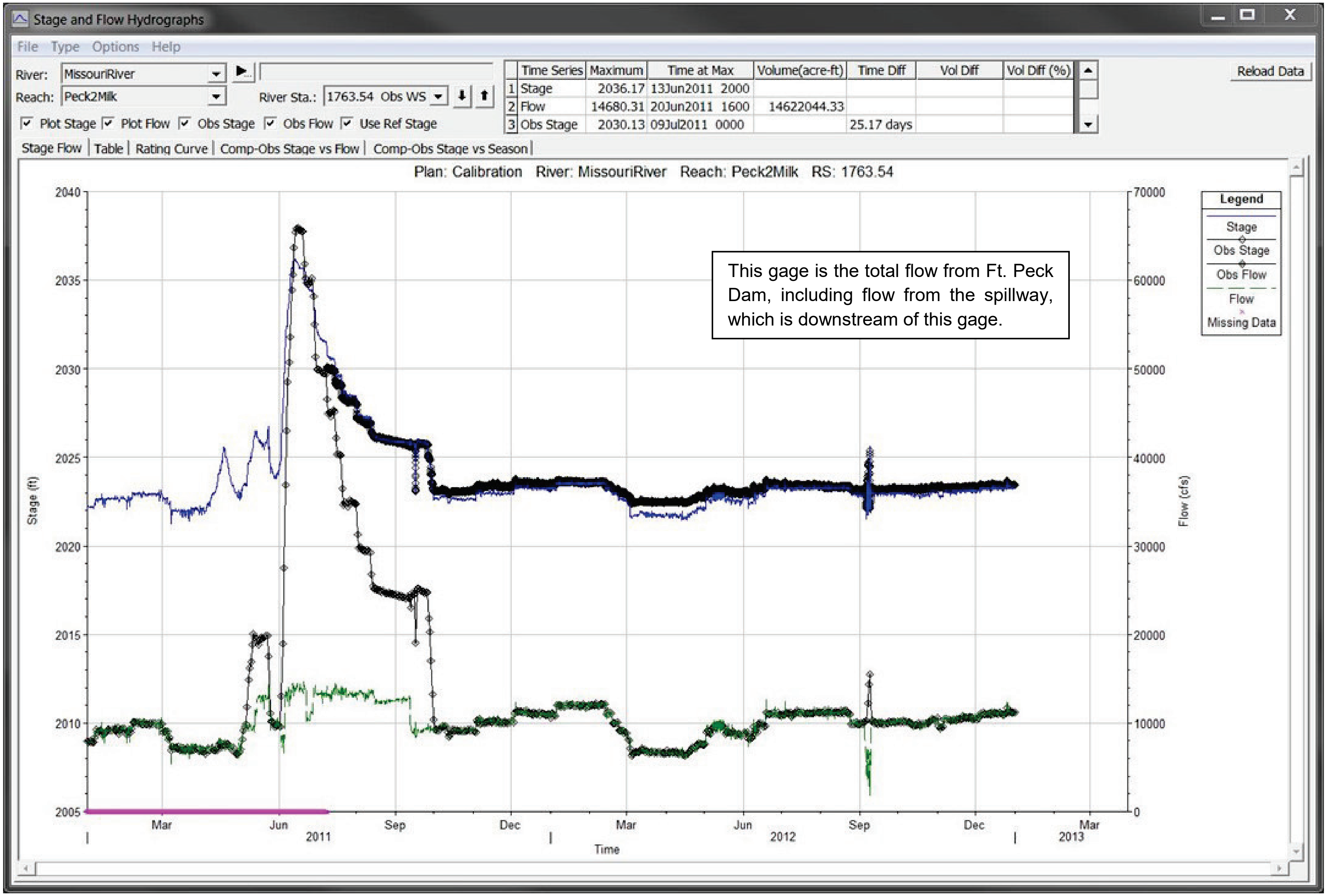


Plate 2: Missouri River below Fort Peck Dam, MT Hydrograph

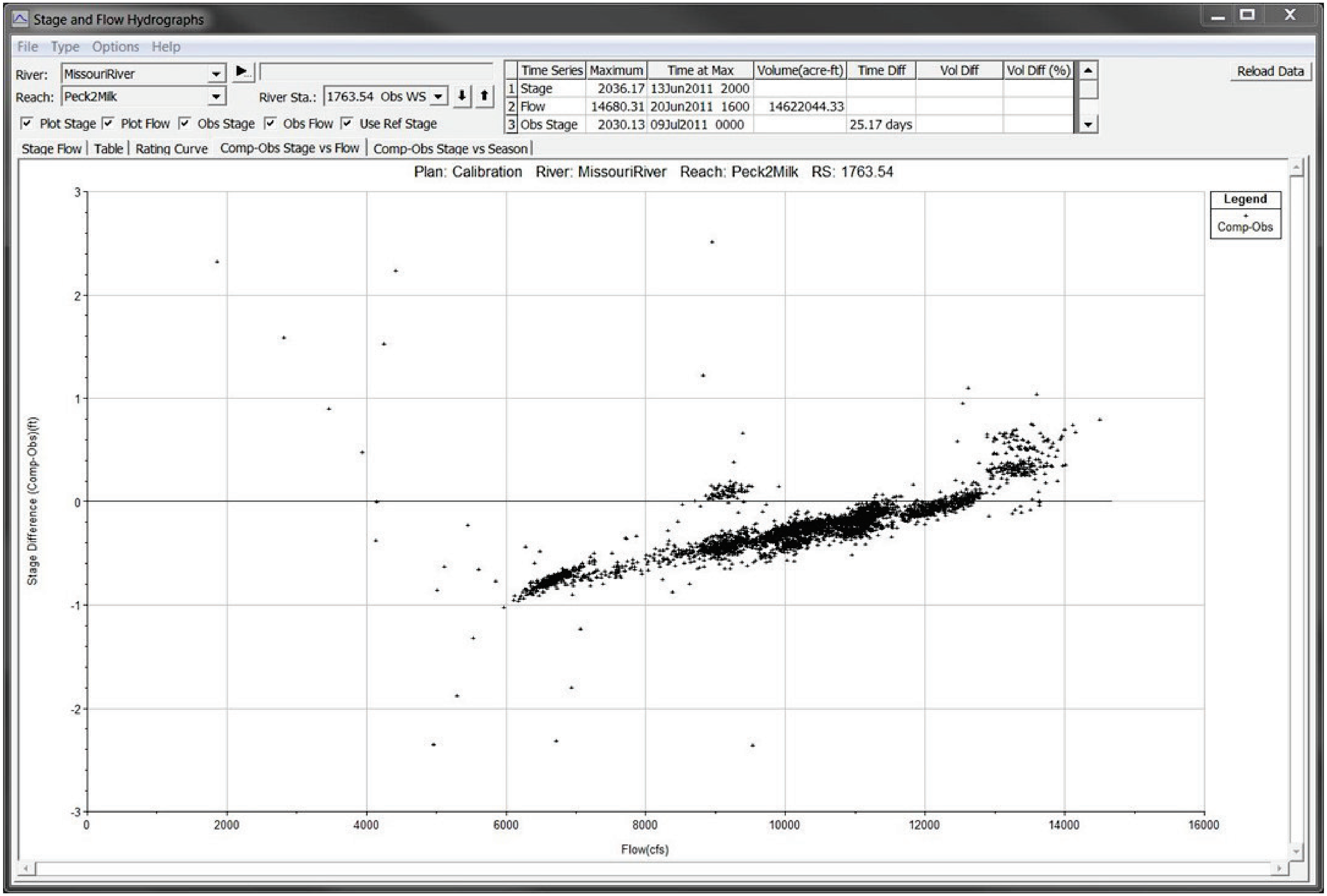


Plate 3: Missouri River below Fort Peck Dam Comp-Obs Stage vs Flow

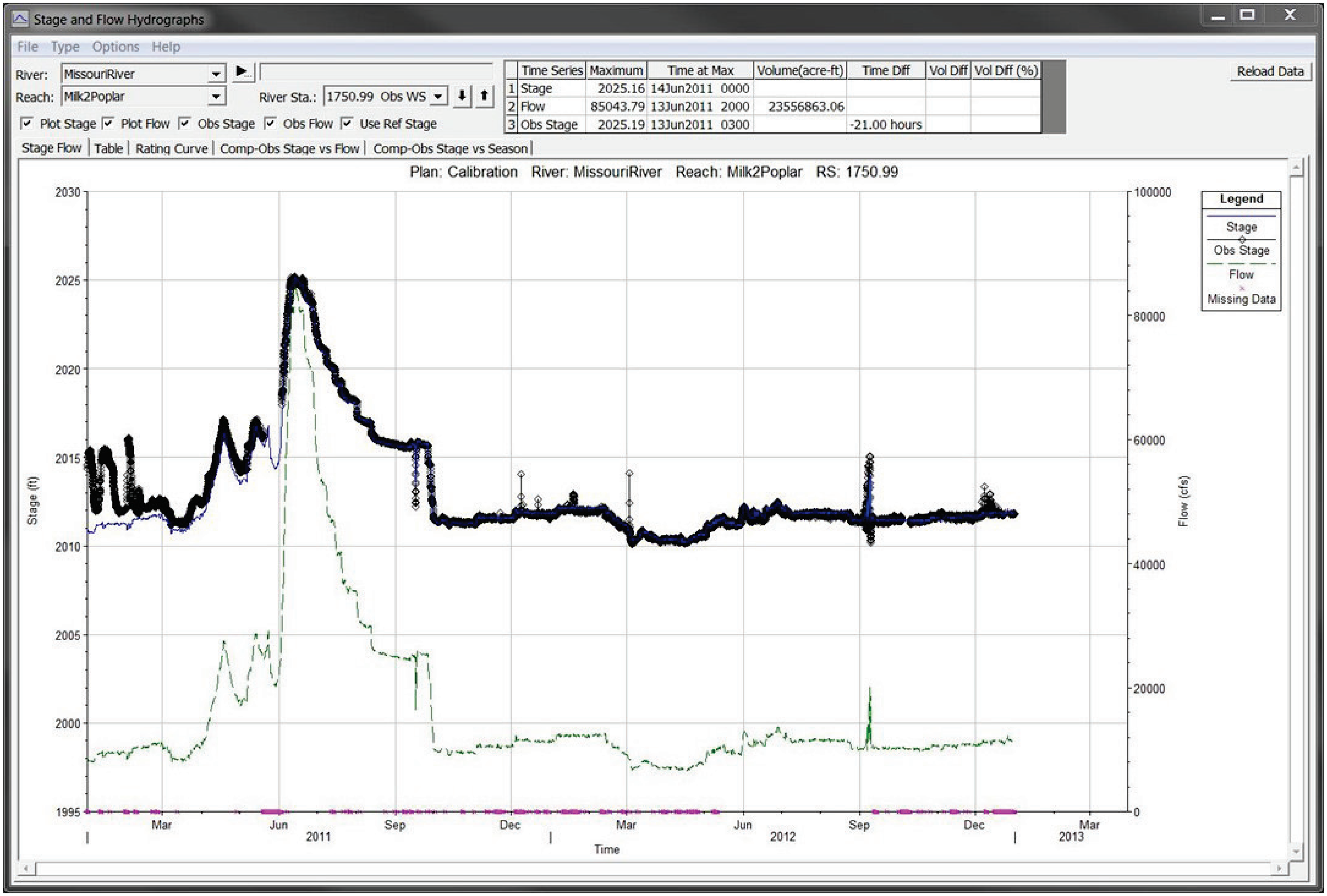


Plate 4: Missouri River near West Frazer Pump Plant, MT Hydrograph

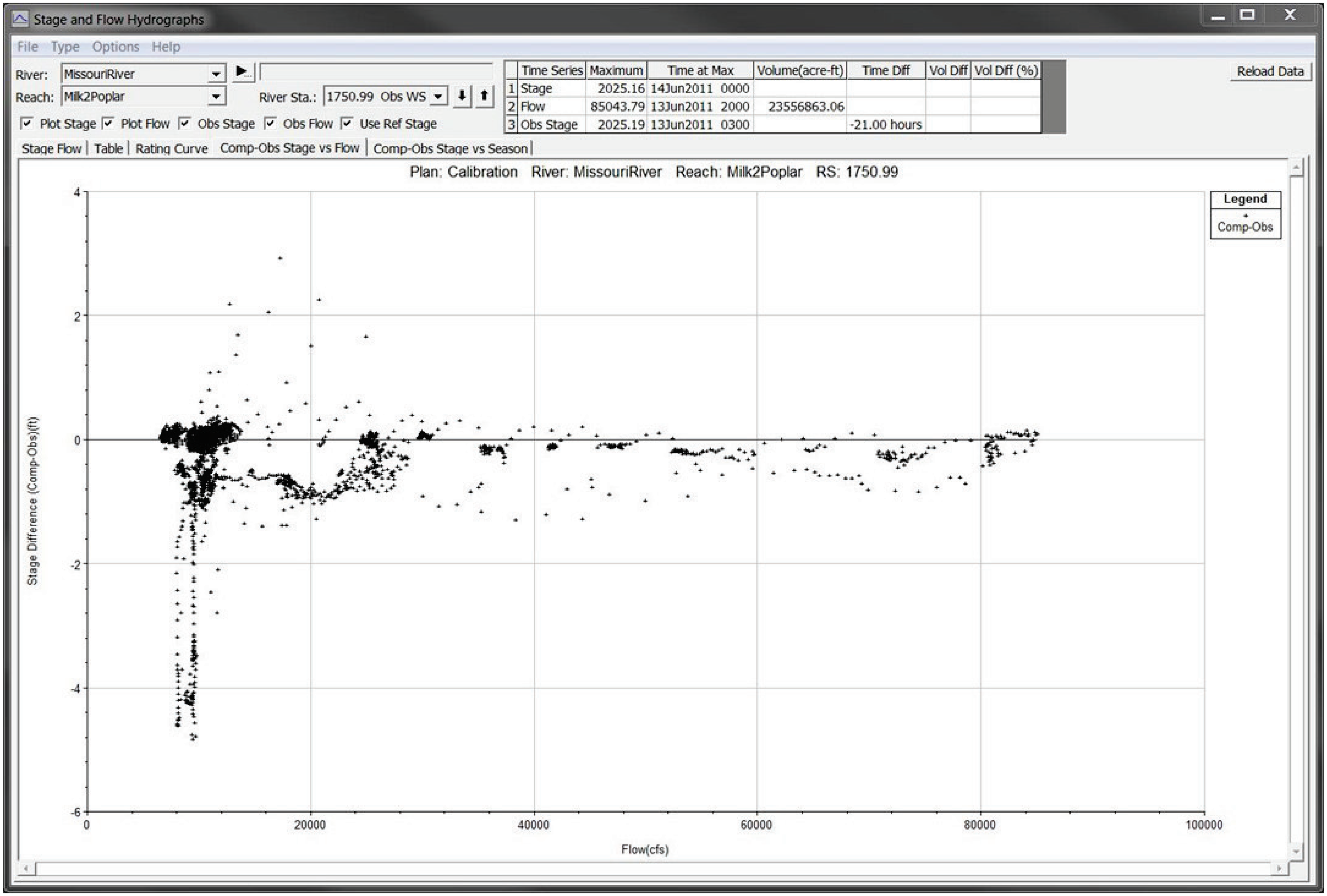


Plate 5: Missouri River near West Frazer Plant, MT Comp-Obs Stage vs Flow

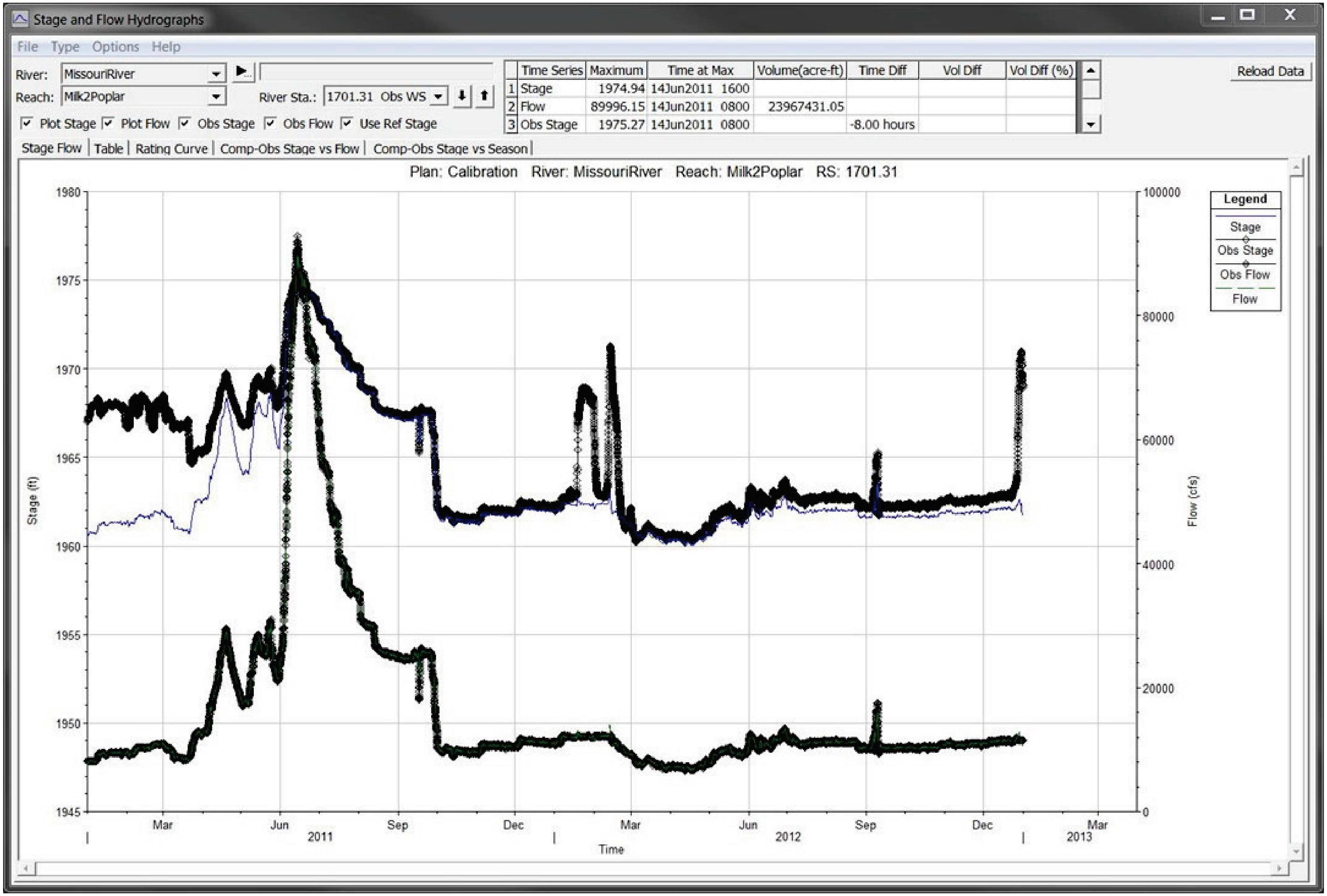


Plate 6: Missouri River near Wolf Point, MT Hydrograph

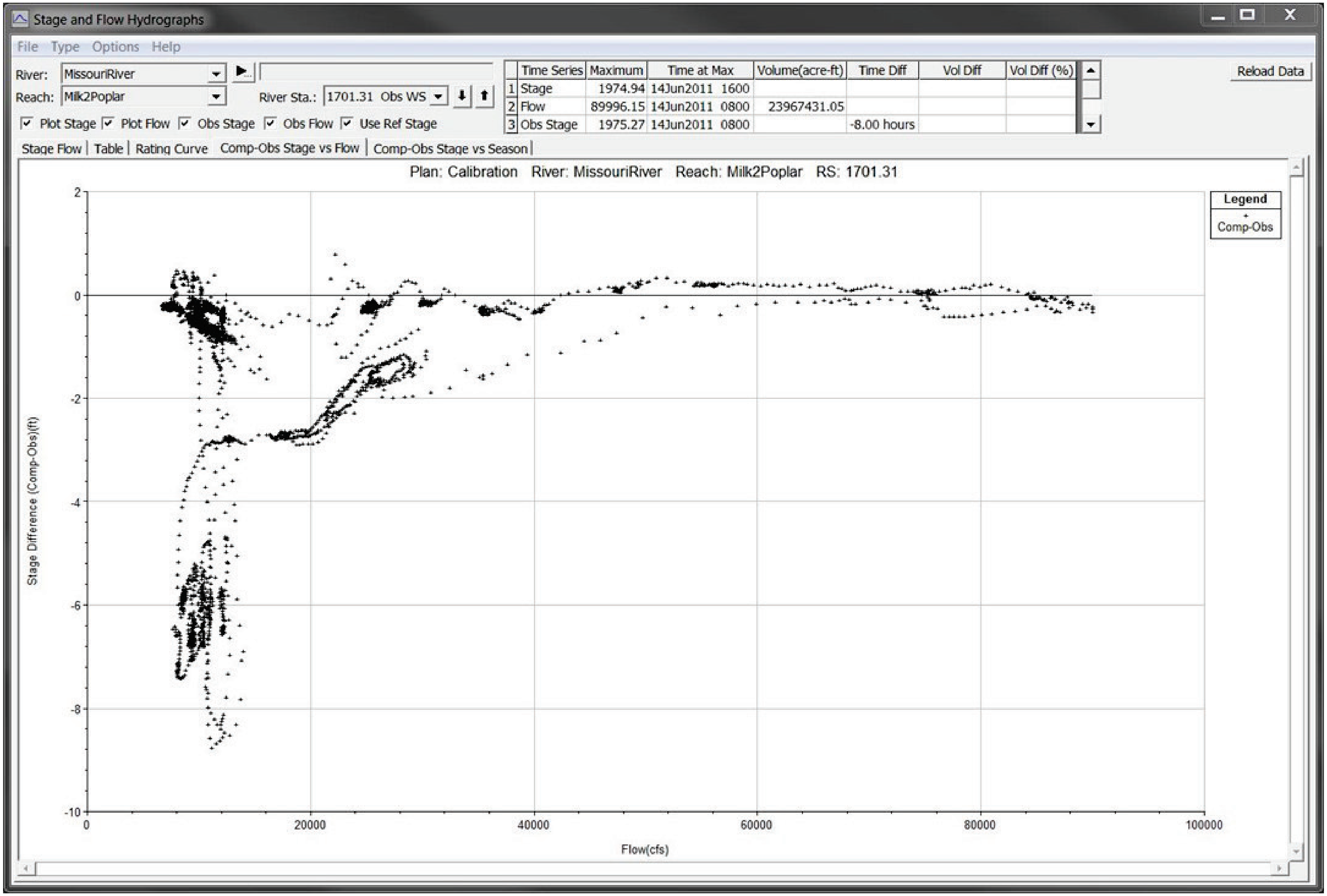


Plate 7: Missouri River near Wolf Point, MT Comp-Obs Stage vs Flow

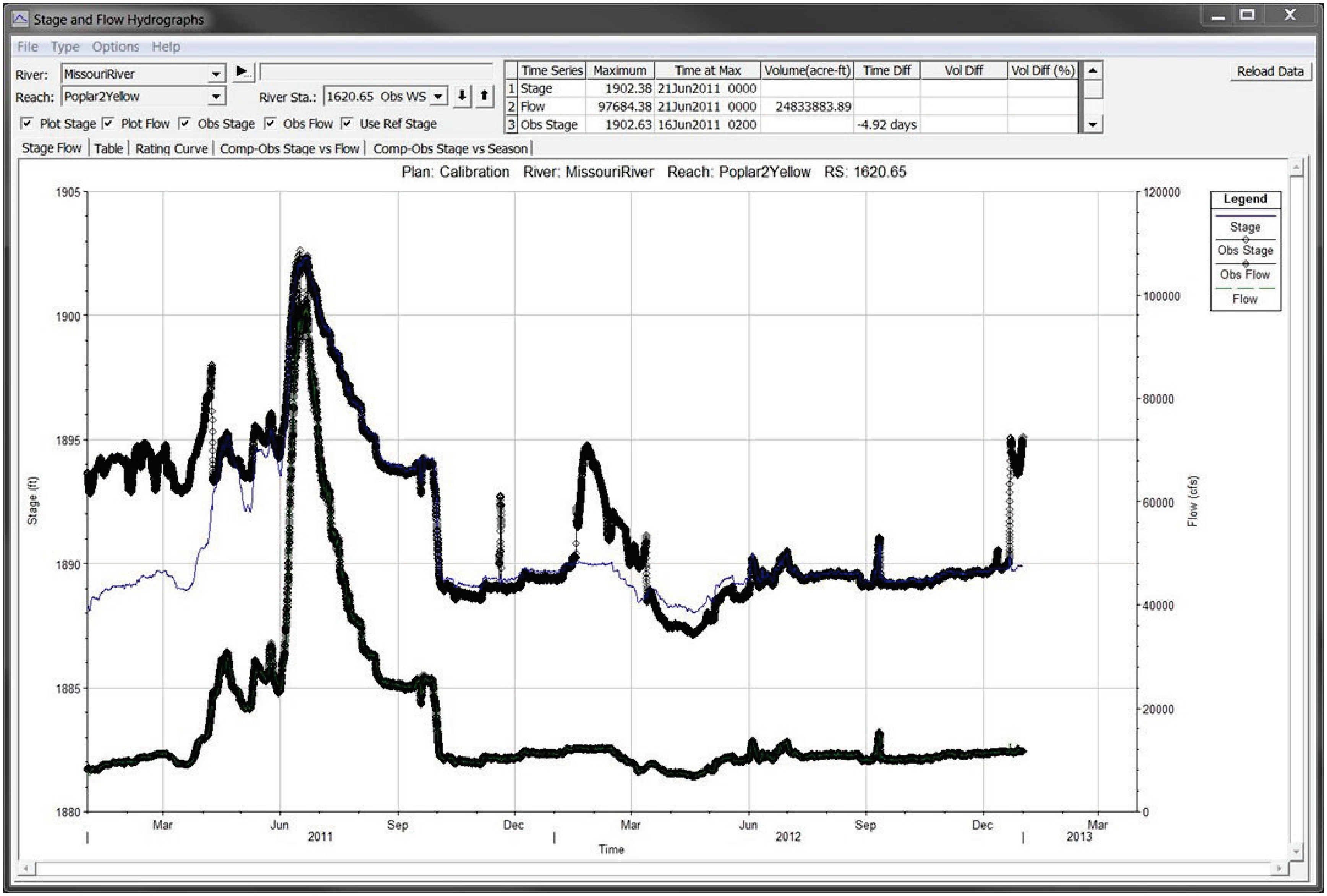


Plate 8: Missouri River near Culbertson, MT Hydrograph

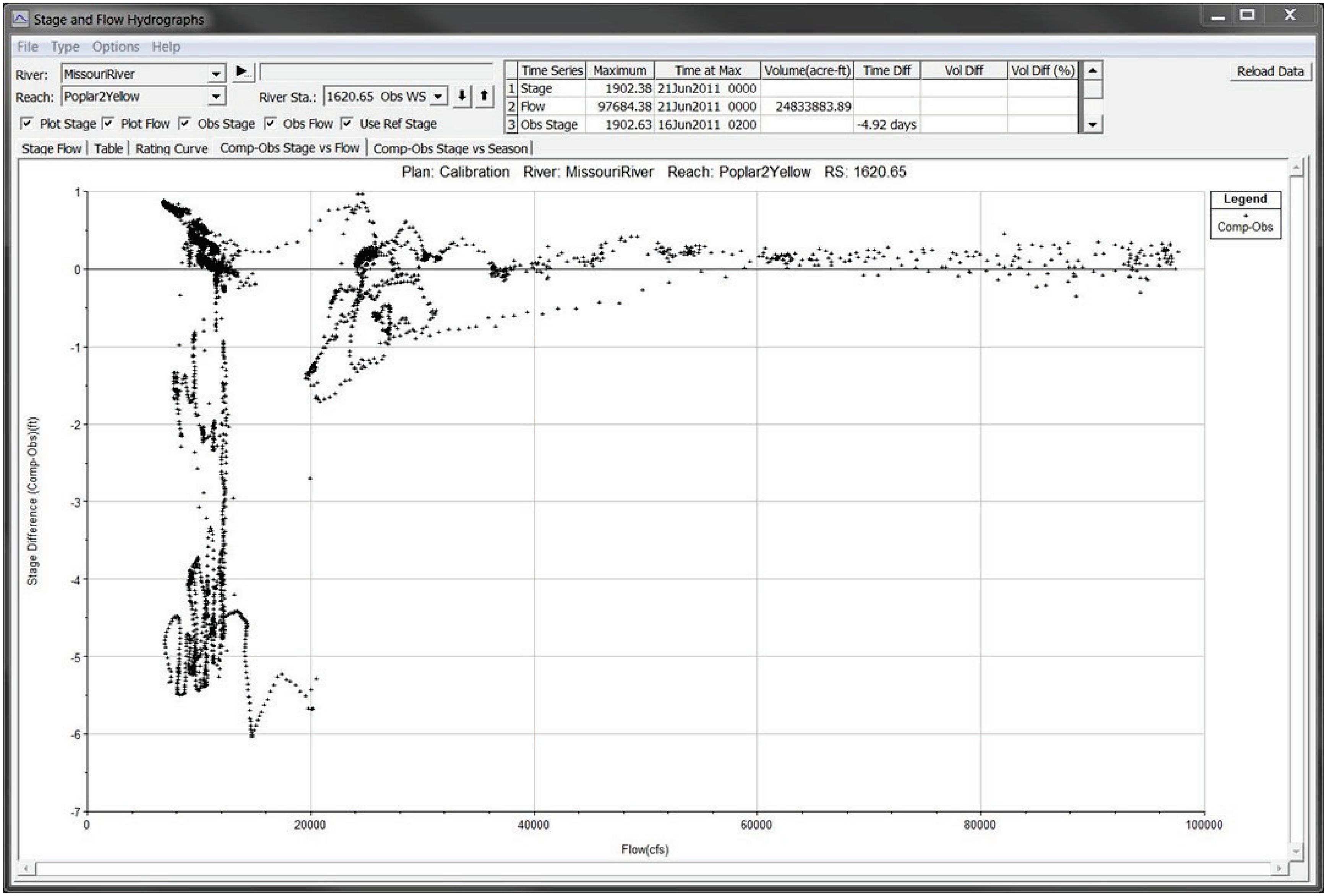


Plate 9: Missouri River near Culbertson, MT Comp-Obs Stage vs Flow

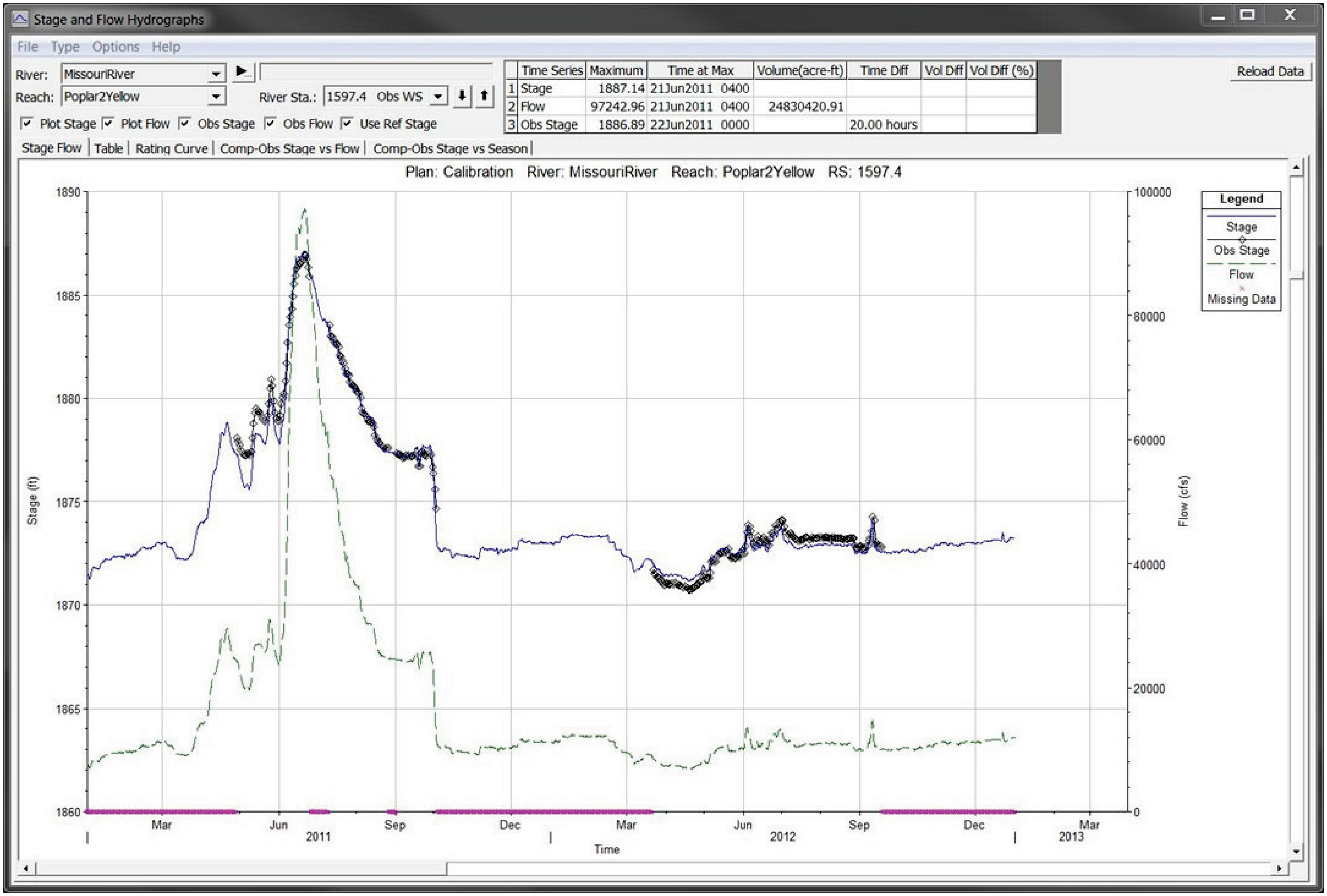


Plate 10: Missouri River at No. 4 near Nohly, MT Hydrograph

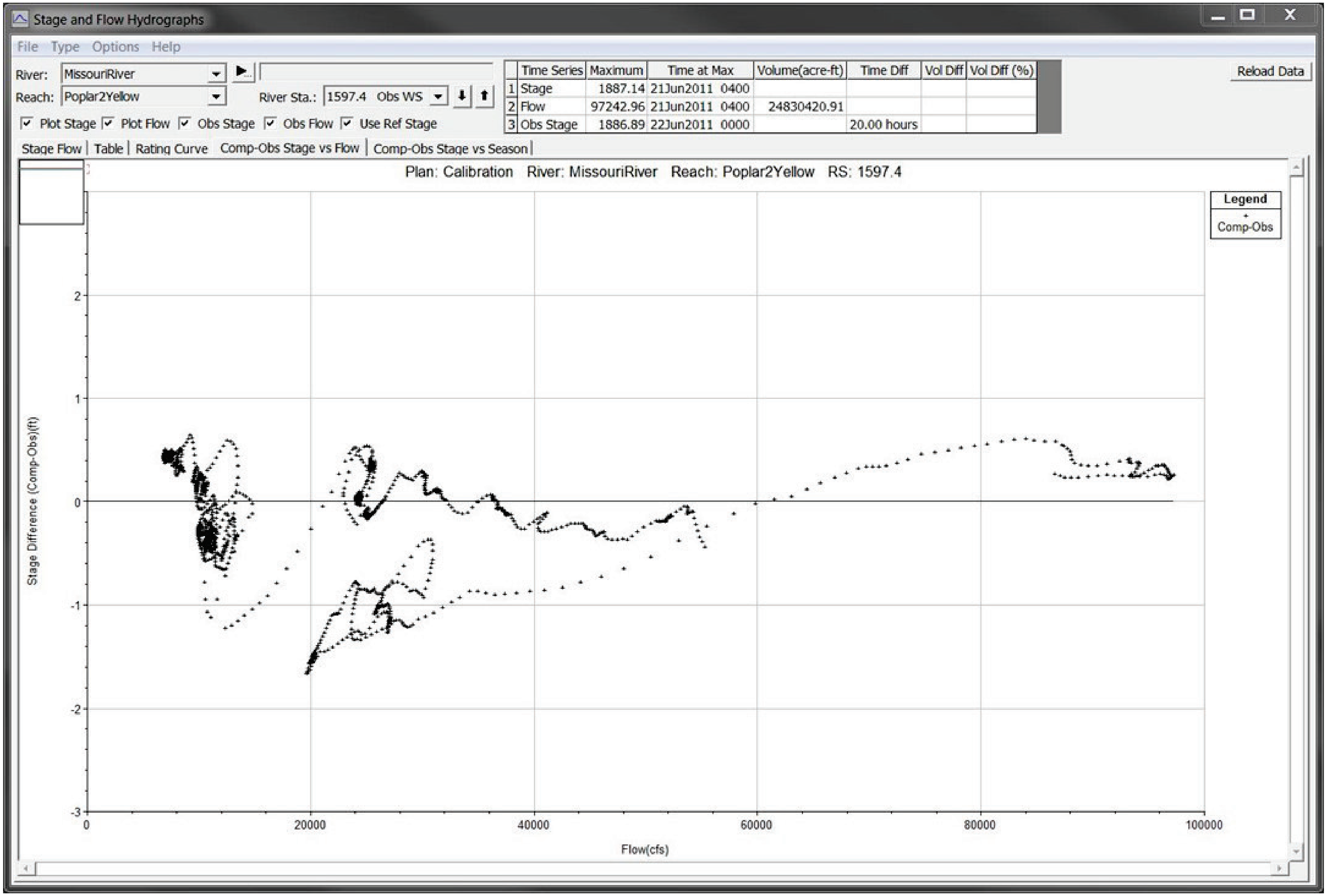


Plate 11: Missouri River at No. 4 near Nohly, MT Comp-Obs Stage vs Flow

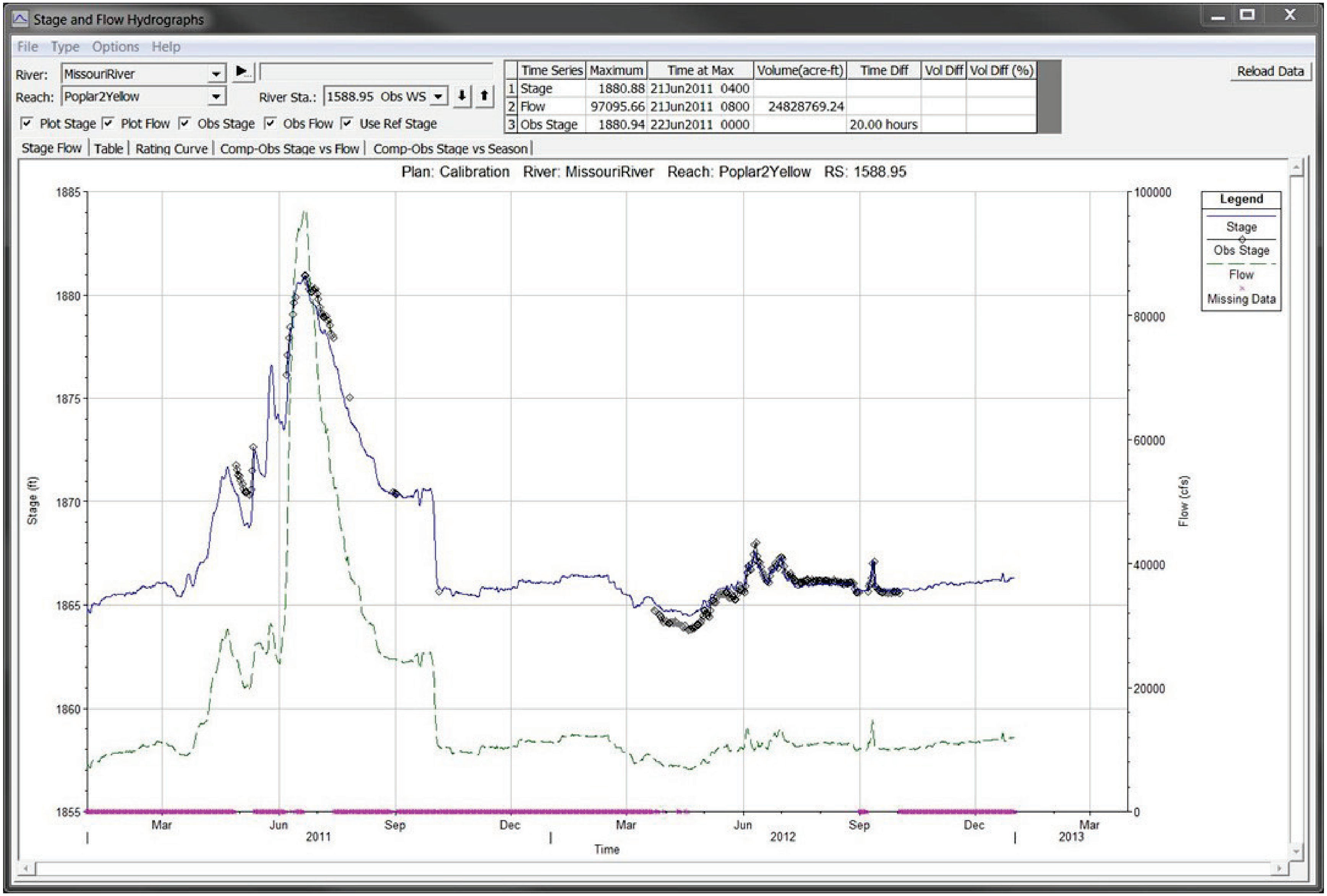


Plate 12: Missouri River at No. 5 near Nohly, MT Hydrograph

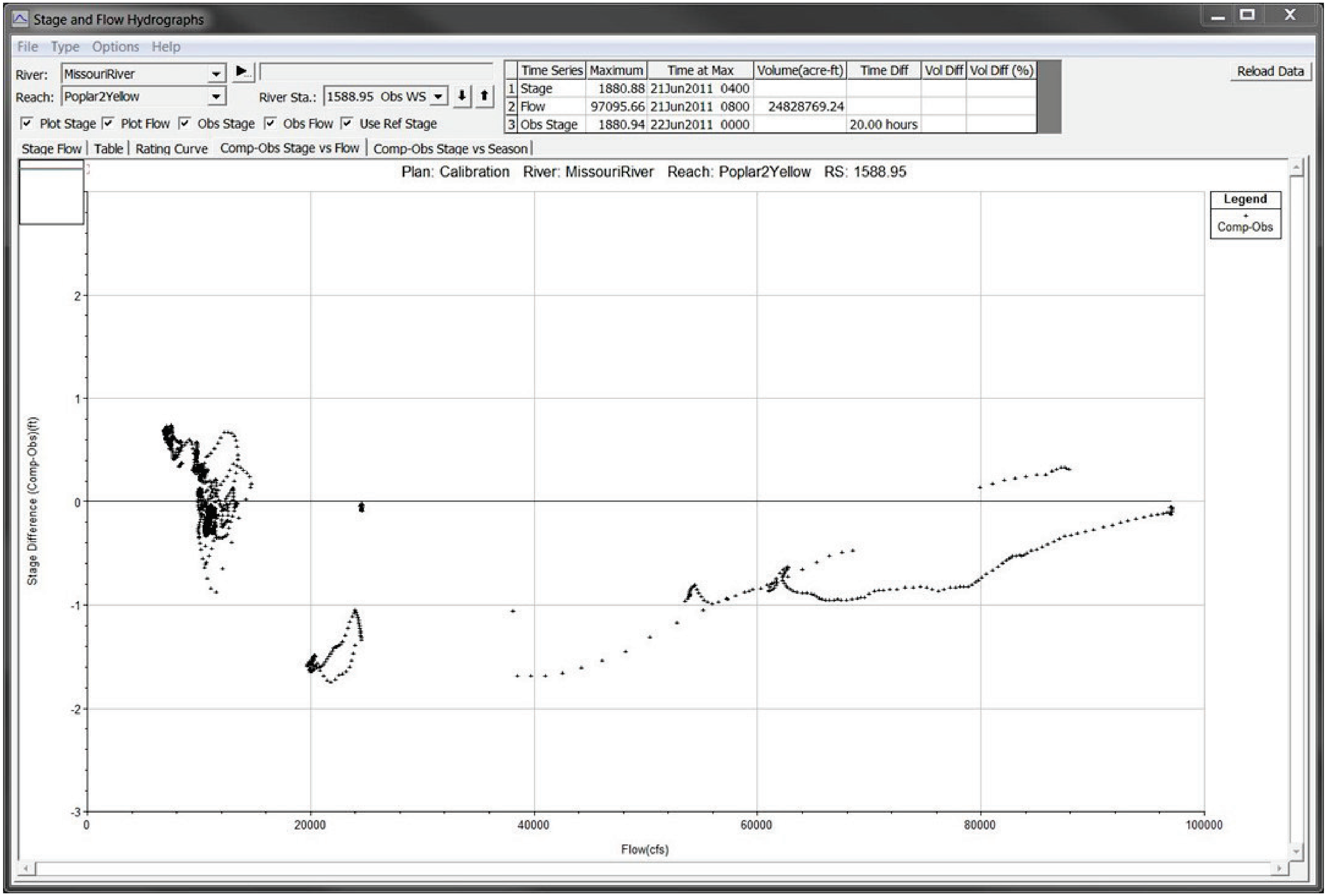
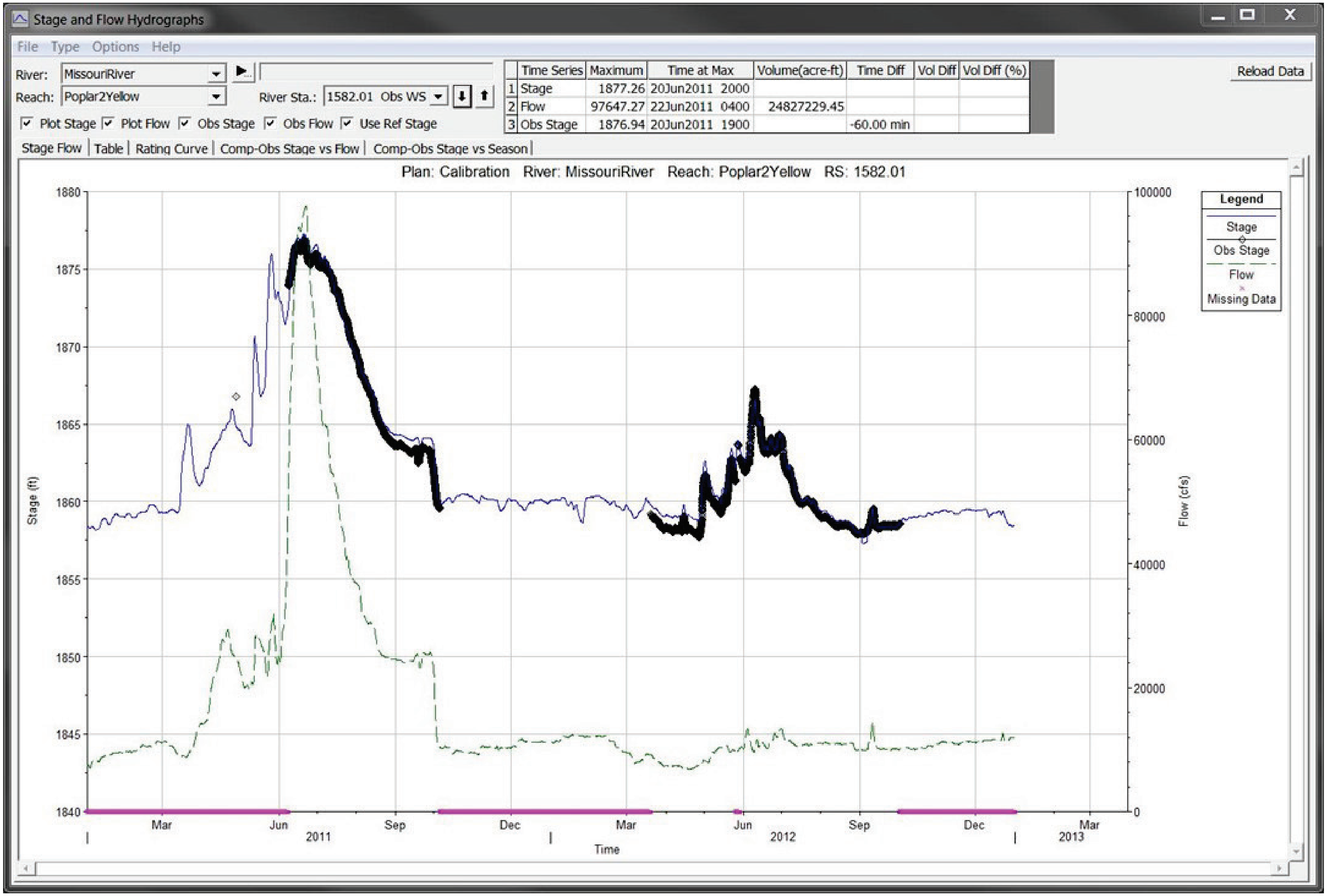


Plate 13: Missouri River at No. 5 near Nohly, MT Comp-Obs Stage vs Flow



	Time Series	Maximum	Time at Max	Volume(acre-ft)	Time Diff	Vol Diff	Vol Diff (%)
1	Stage	1877.26	20Jun2011 2000				
2	Flow	97647.27	22Jun2011 0400	24827229.45			
3	Obs Stage	1876.94	20Jun2011 1900		-60.00 min		

Plate 14: Missouri River at No. 5A near Buford, ND Hydrograph

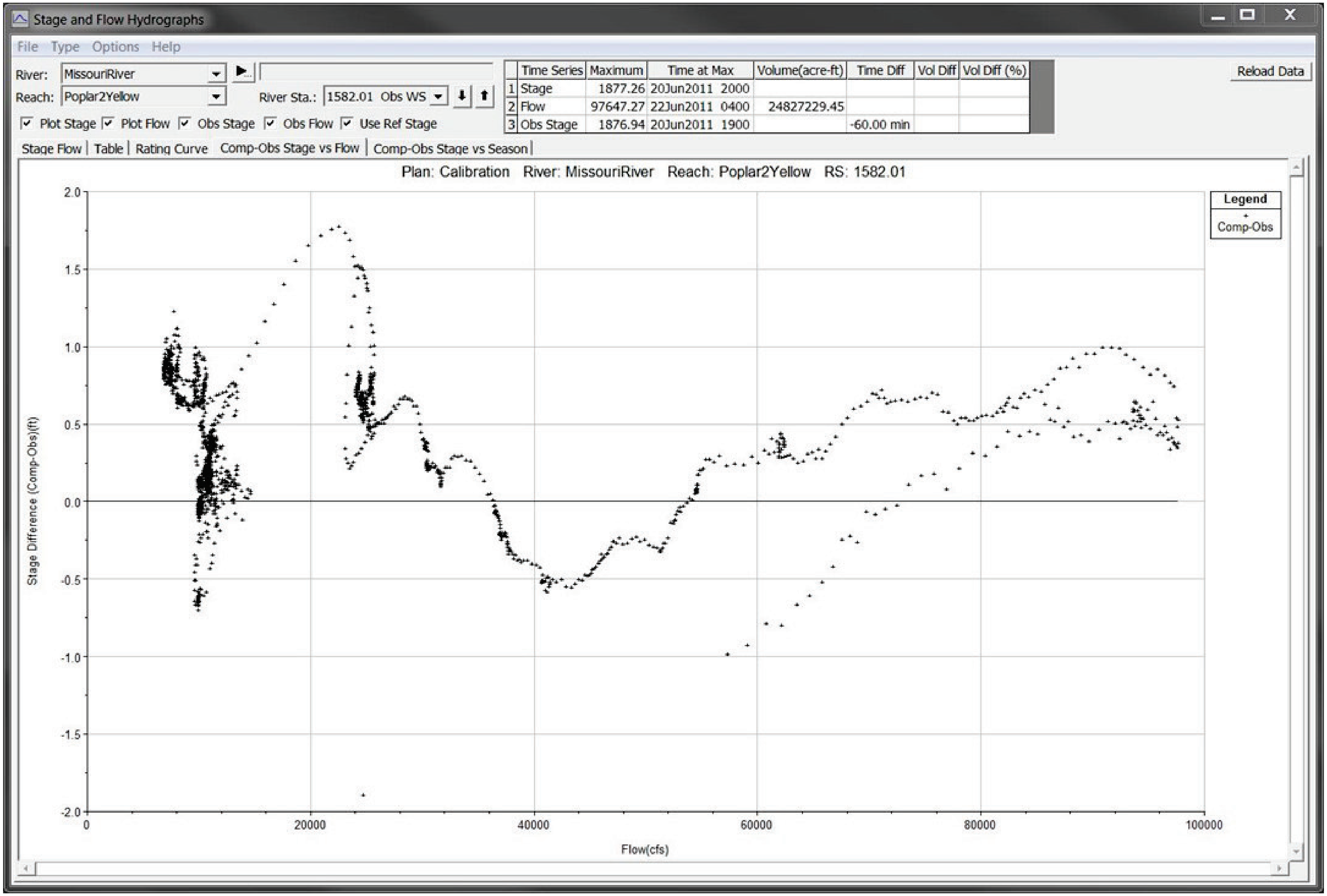
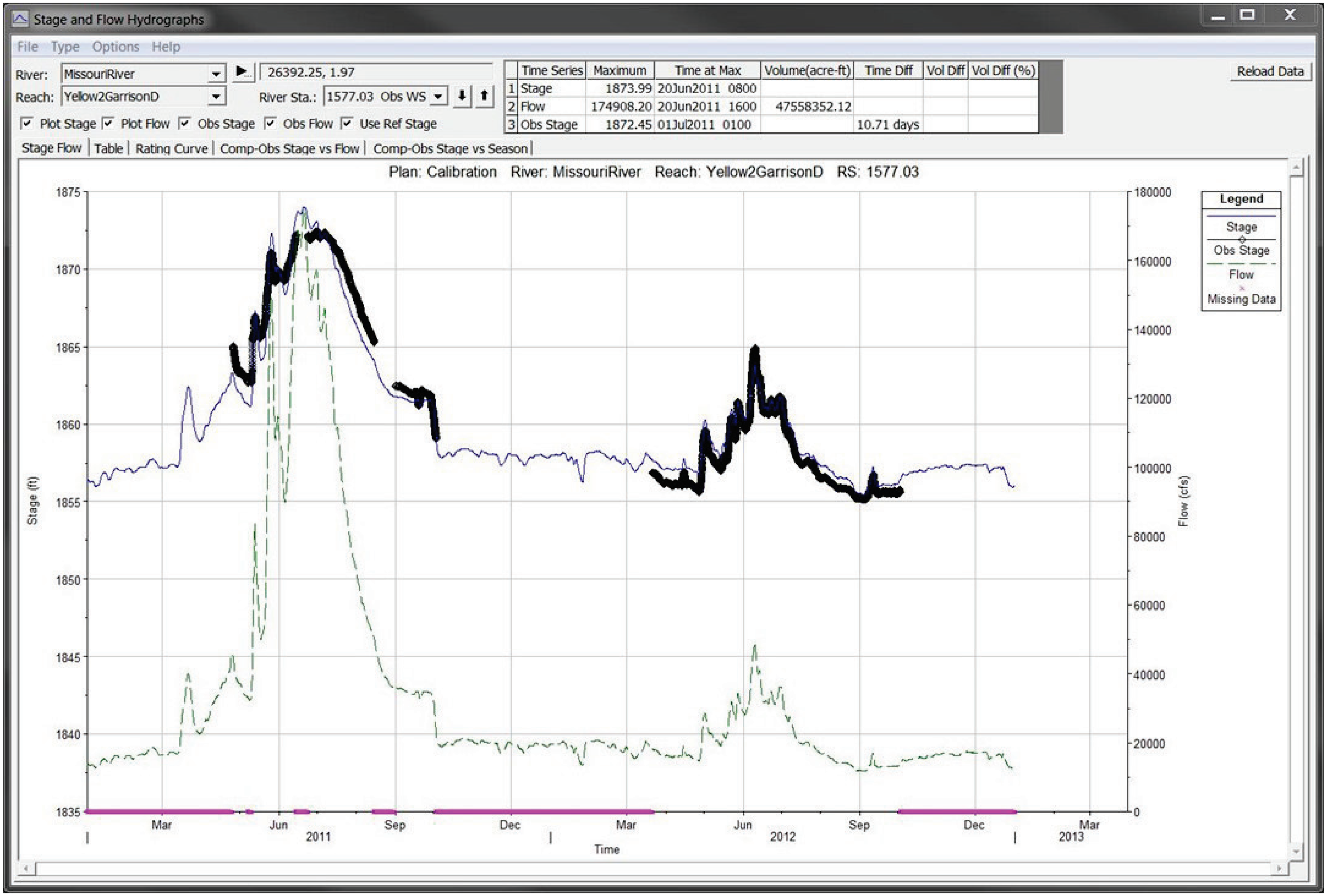


Plate 15: Missouri River at No. 5A near Buford, ND Comp-Obs Stage vs Flow



	Time Series	Maximum	Time at Max	Volume(acre-ft)	Time Diff	Vol Diff	Vol Diff (%)
1	Stage	1873.99	20Jun2011 0800				
2	Flow	174908.20	20Jun2011 1600	47558352.12			
3	Obs Stage	1872.45	01Jul2011 0100		10.71 days		

Plate 16: Missouri River at No. 6 near Buford, ND Hydrograph

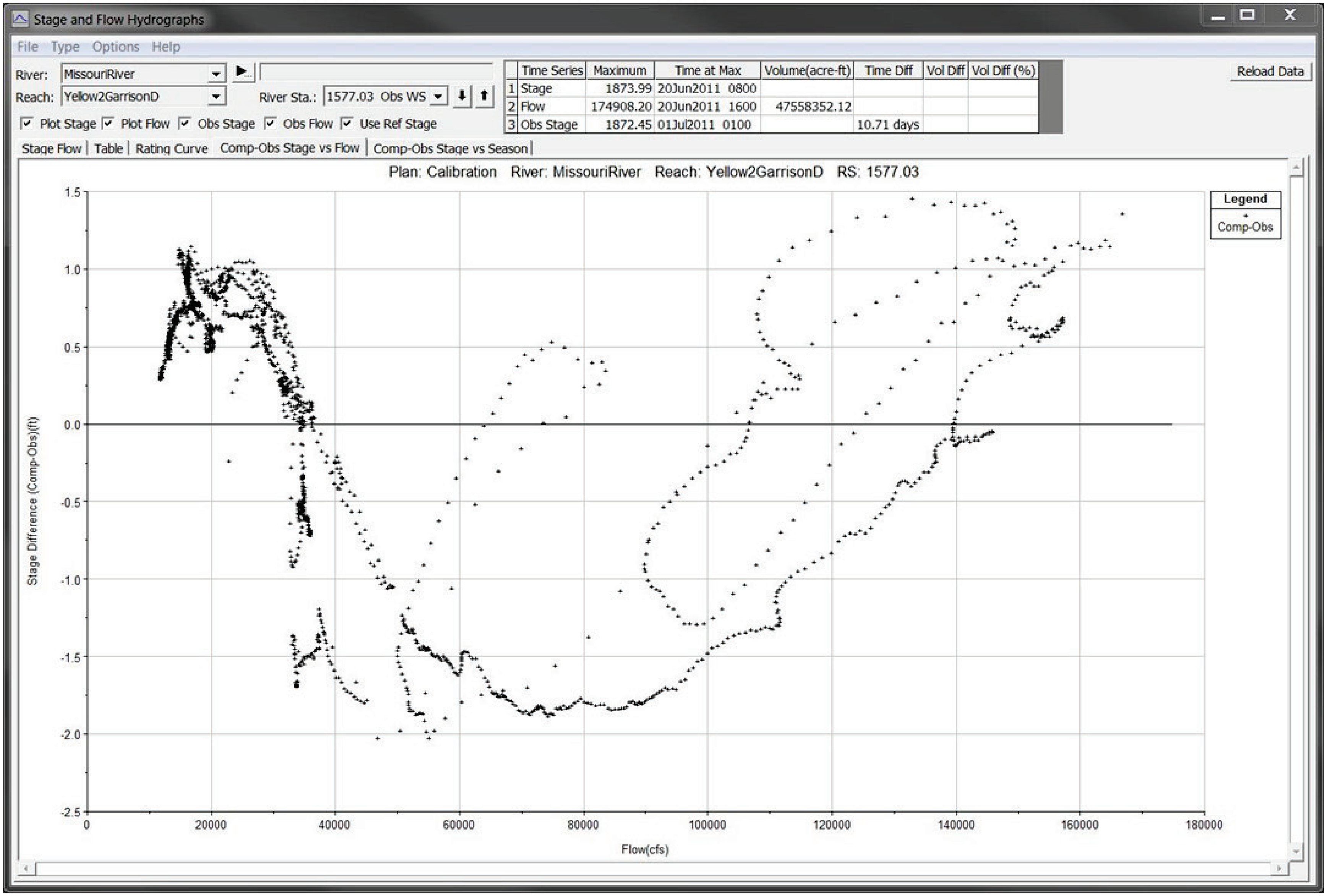


Plate 17: Missouri River at No. 6 near Buford, ND Comp-Obs Stage vs Flow

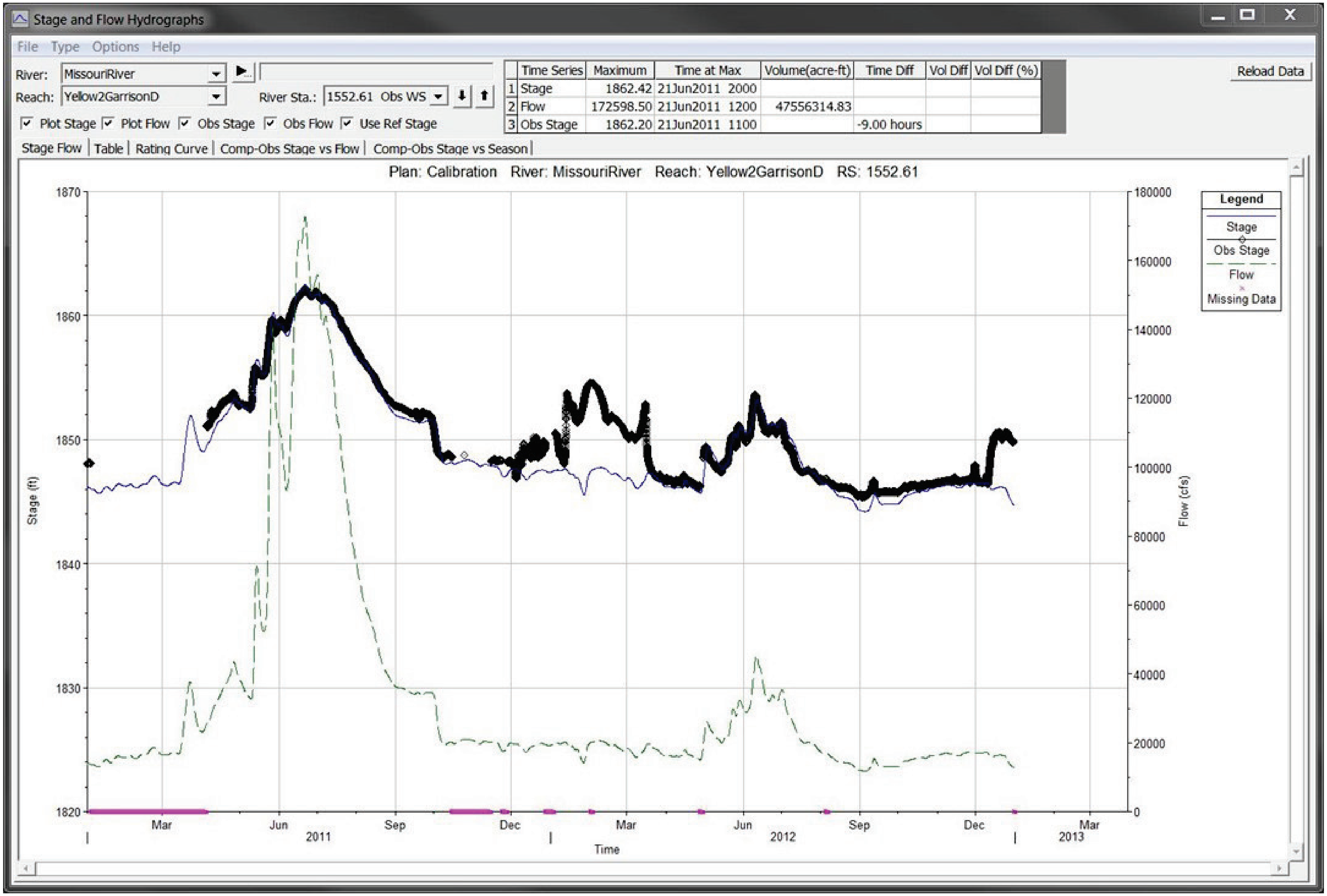


Plate 18: Missouri River near Williston, ND Hydrograph

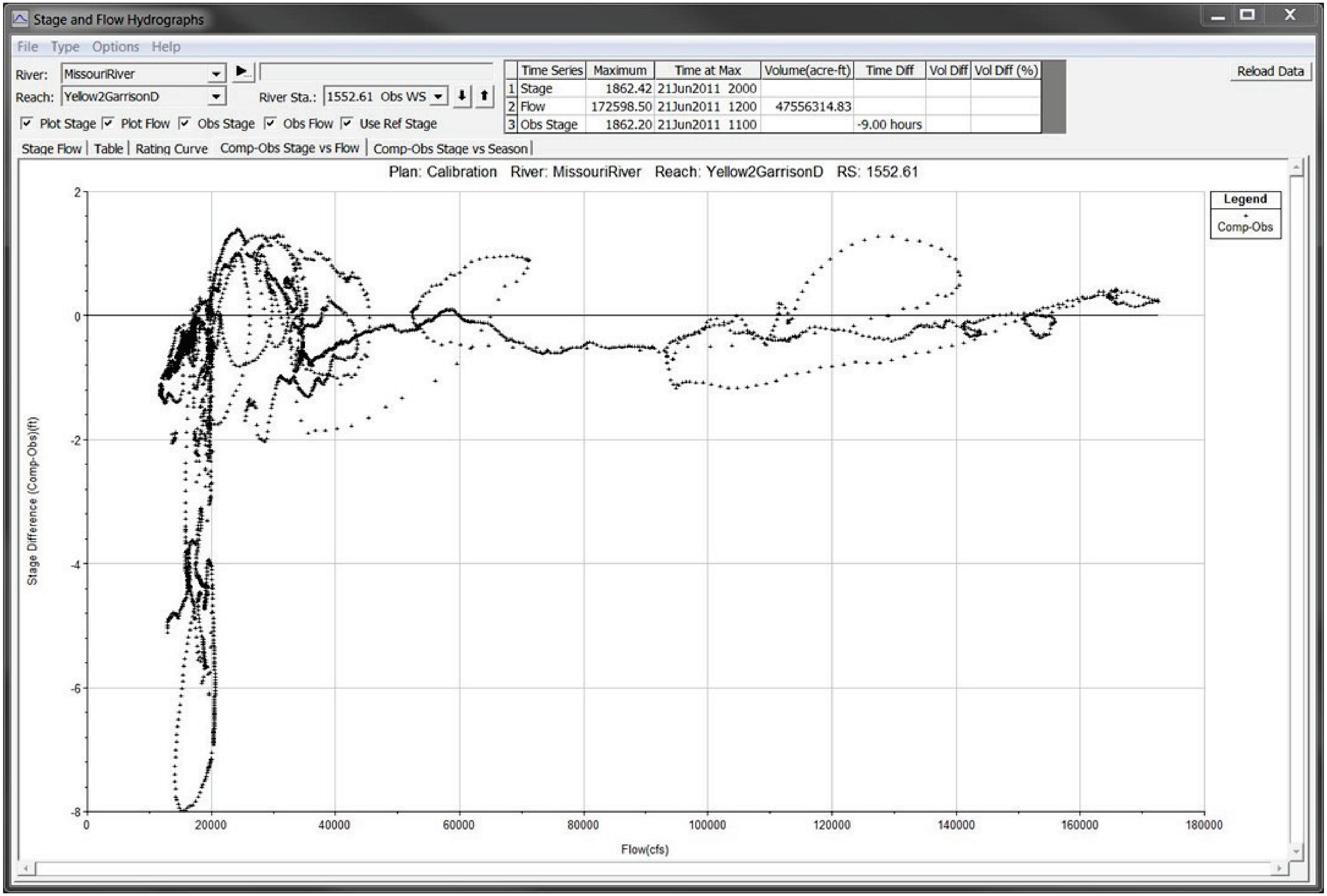


Plate 19: Missouri River near Williston, ND Comp-Obs Stage vs Flow

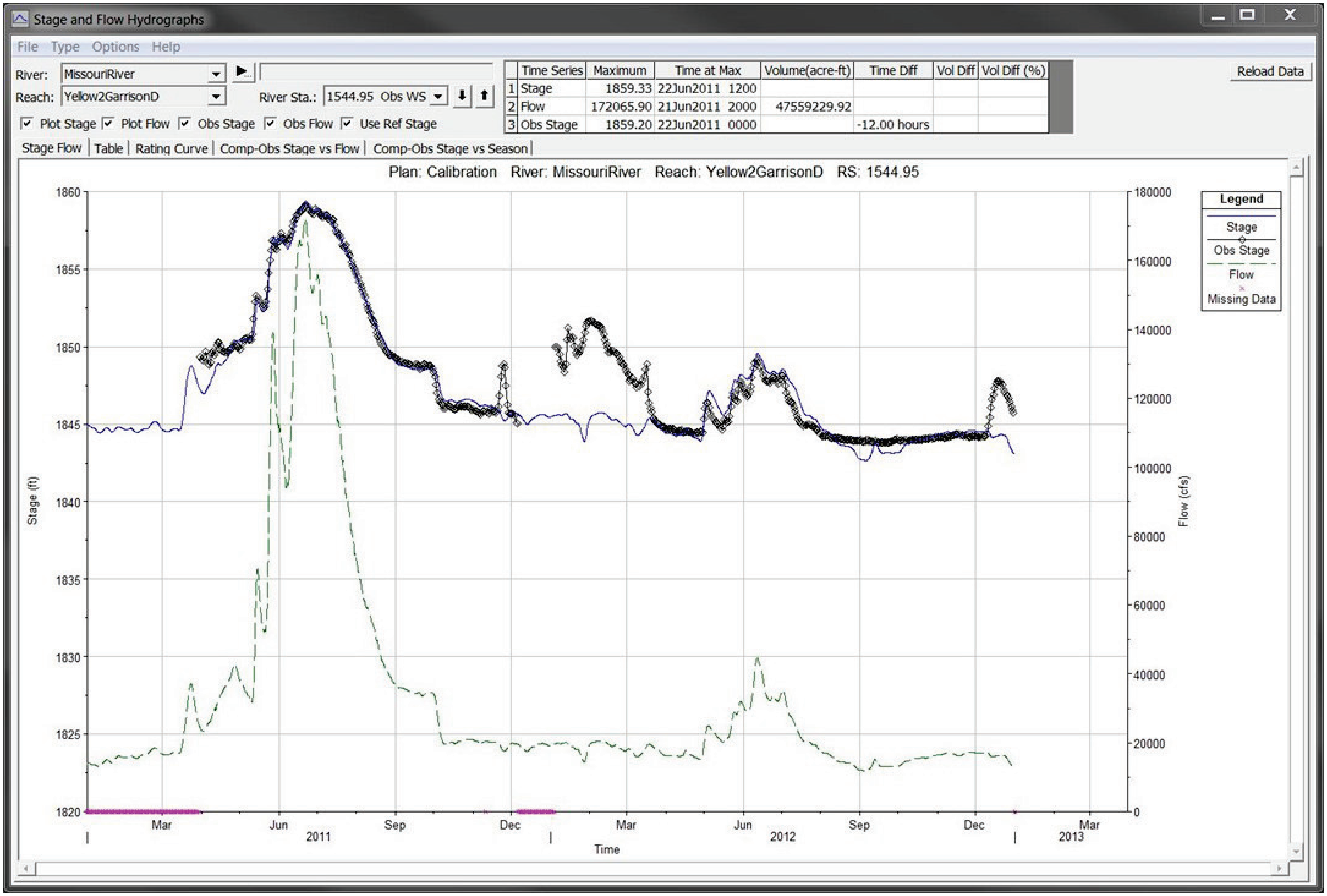


Plate 20: Missouri River No. 9 at Williston, ND Hydrograph

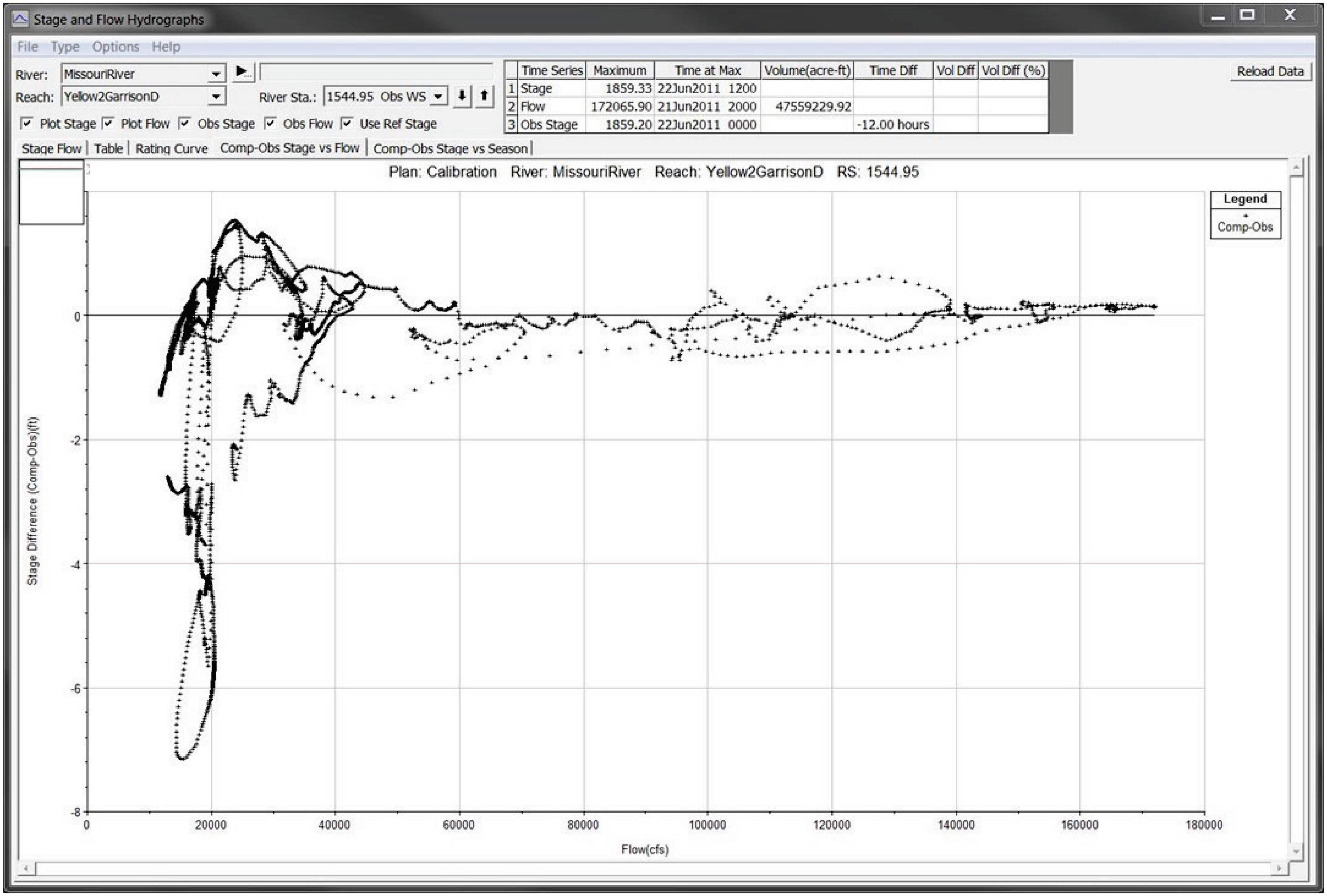


Plate 21: Missouri River No. 9 at Williston, ND Comp-Obs Stage vs Flow

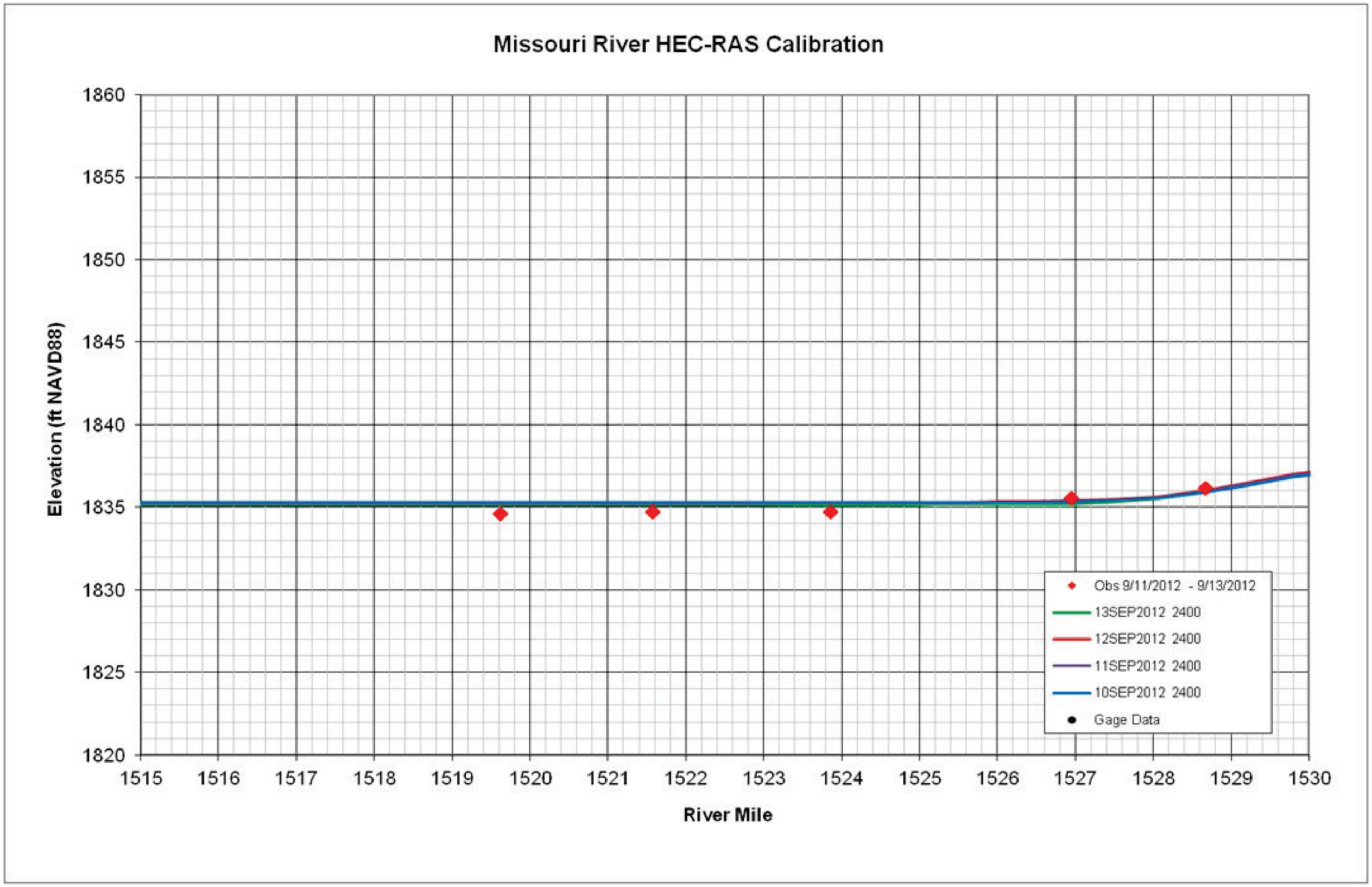


Plate 22: Measured WSP vs Computed Water Surface – RM 1515 to 1530

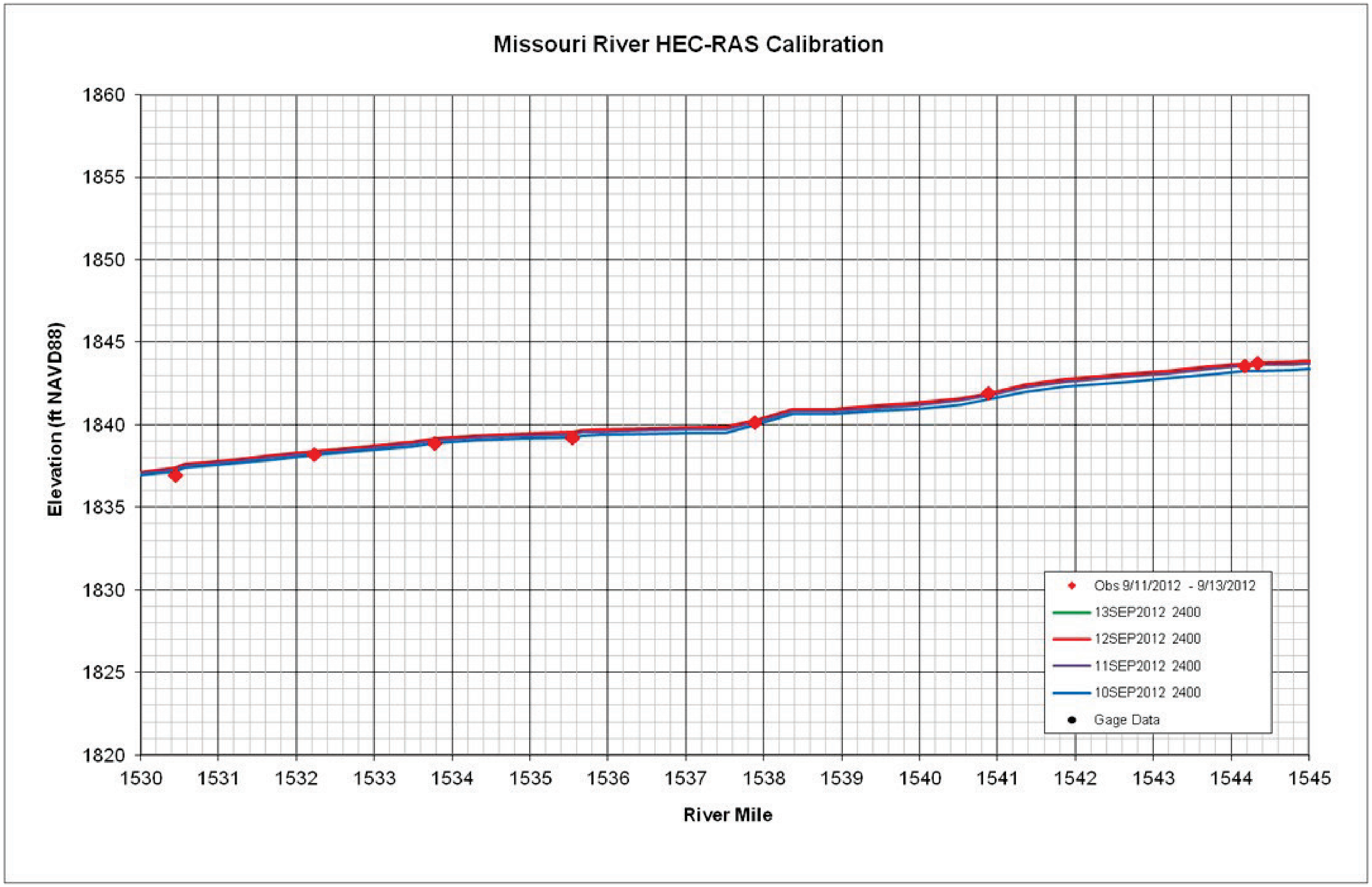


Plate 23: Measured WSP vs Computed Water Surface – RM 1530 to 1545

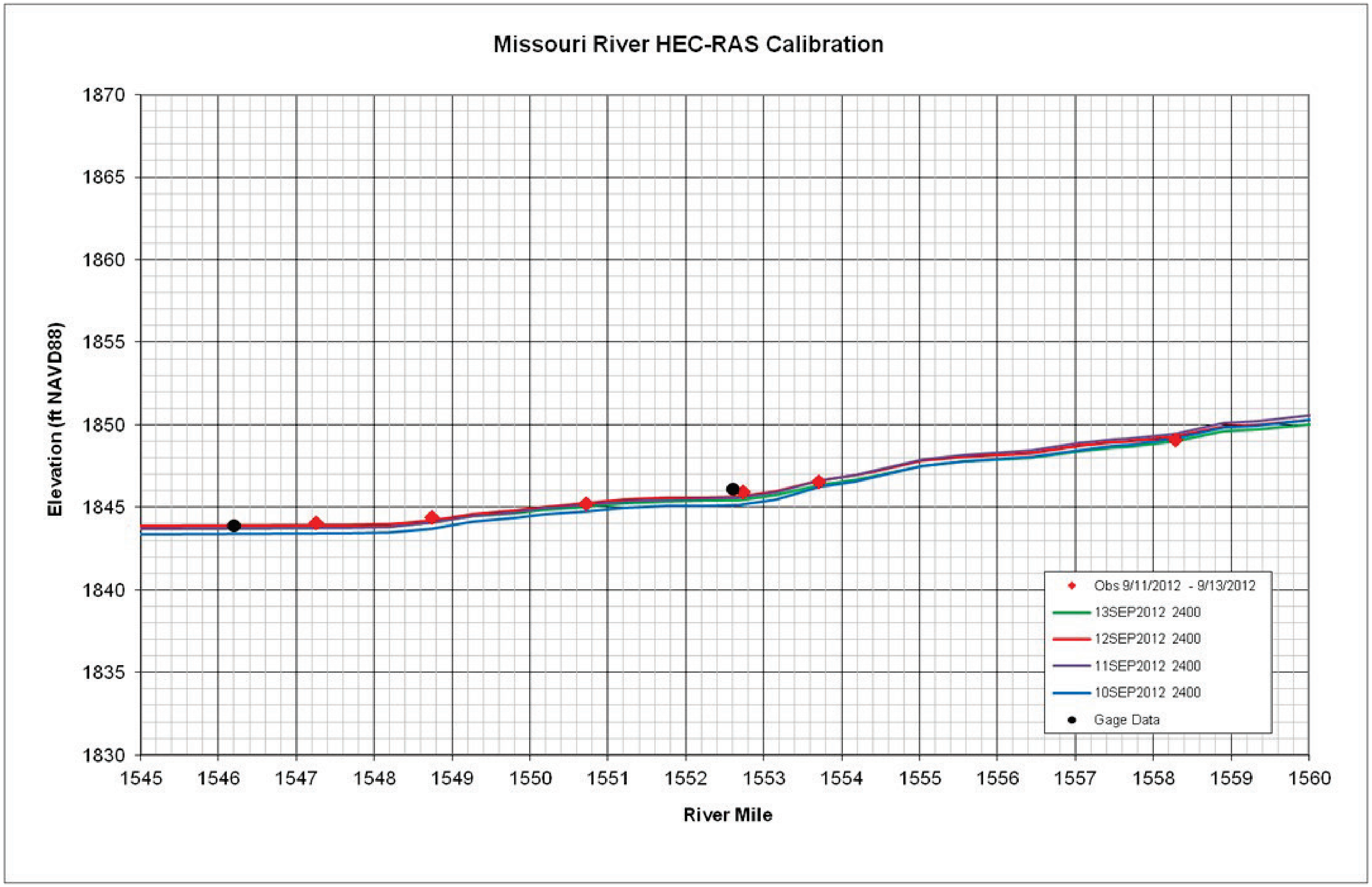


Plate 24: Measured WSP vs Computed Water Surface – RM 1545 to 1560

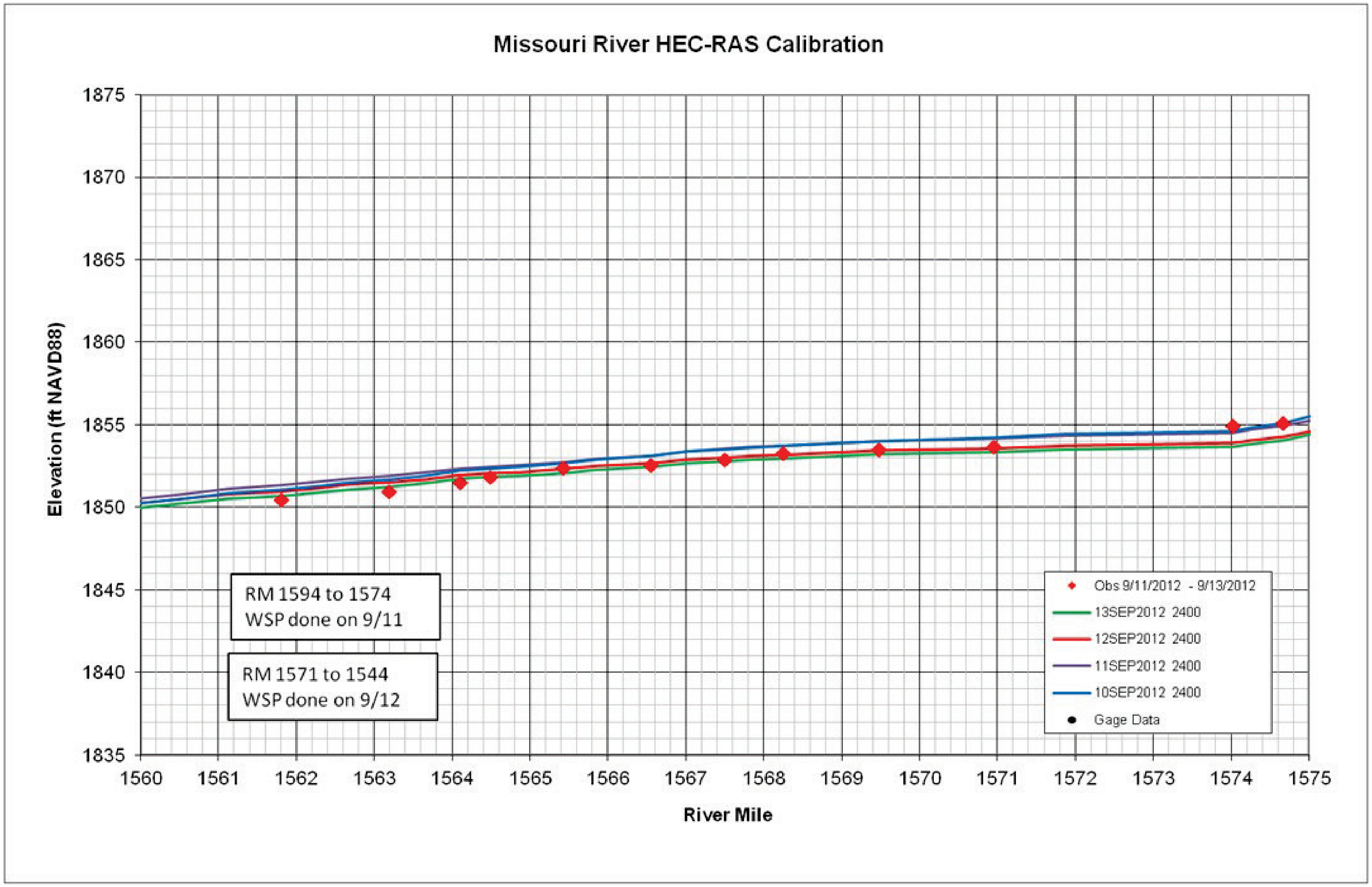


Plate 25: Measured WSP vs Computed Water Surface – RM 1560 to 1575

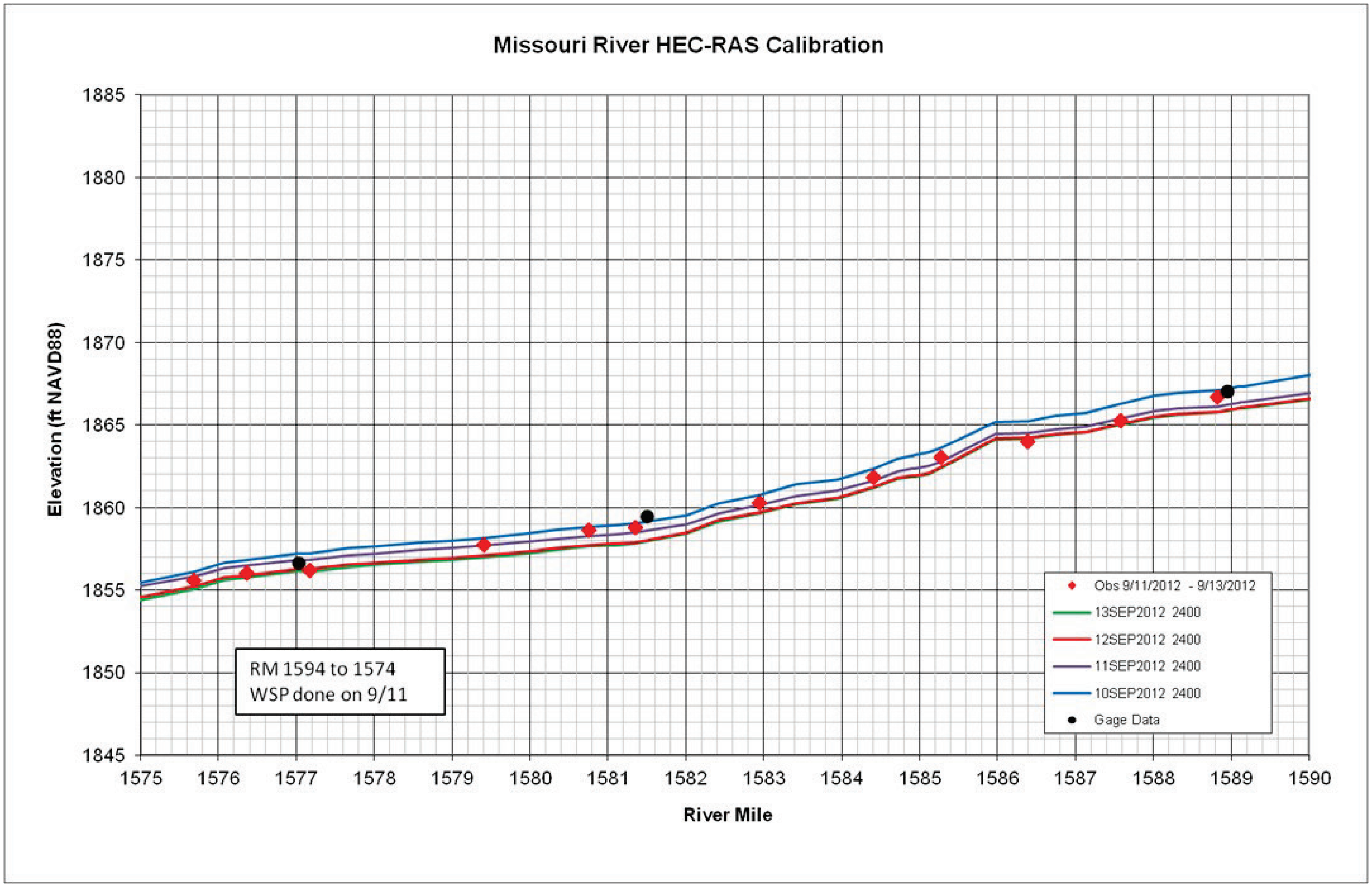


Plate 26: Measured WSP vs Computed Water Surface – RM 1575 to 1590

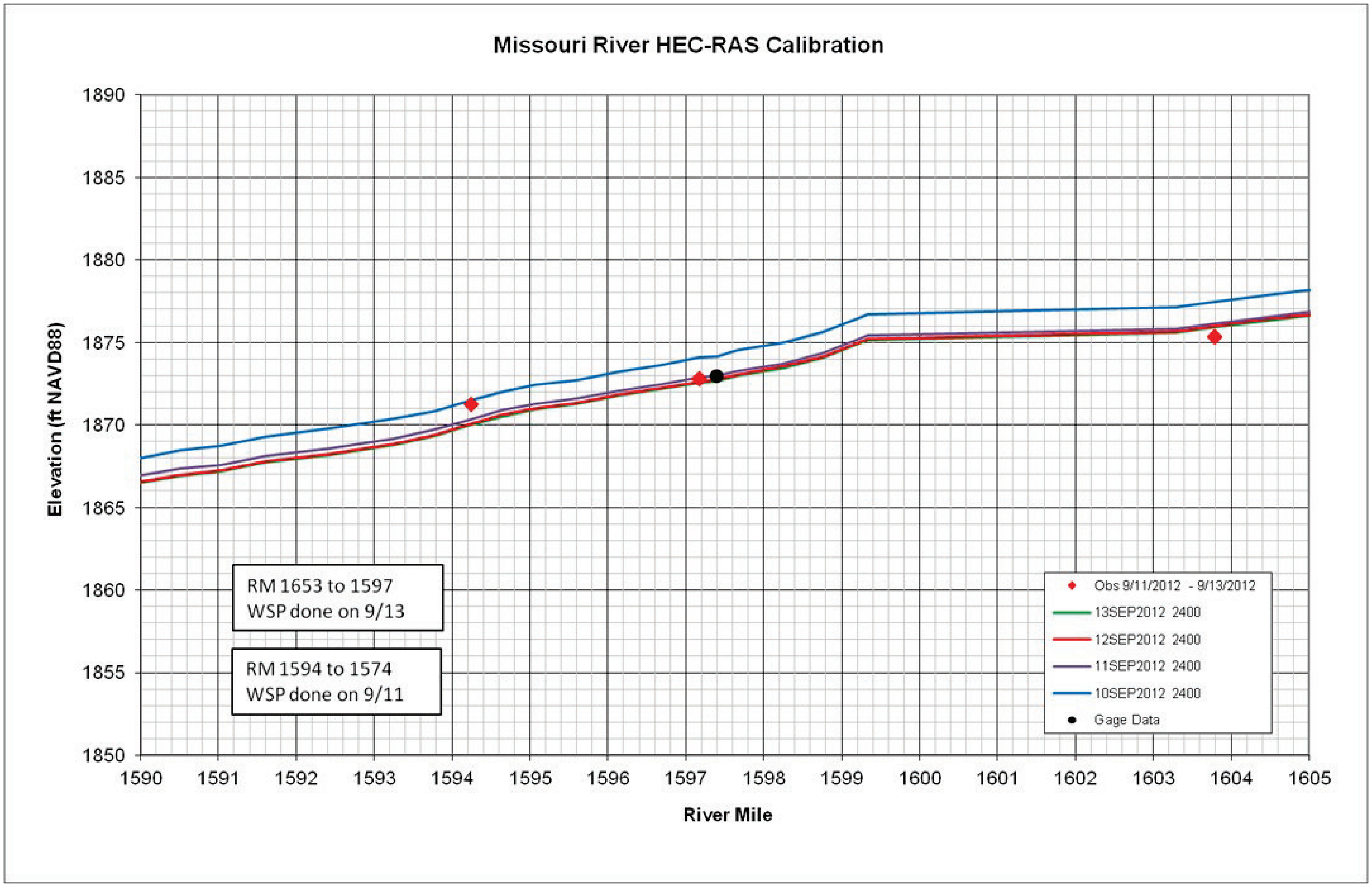


Plate 27: Measured WSP vs Computed Water Surface – RM 1590 to 1605

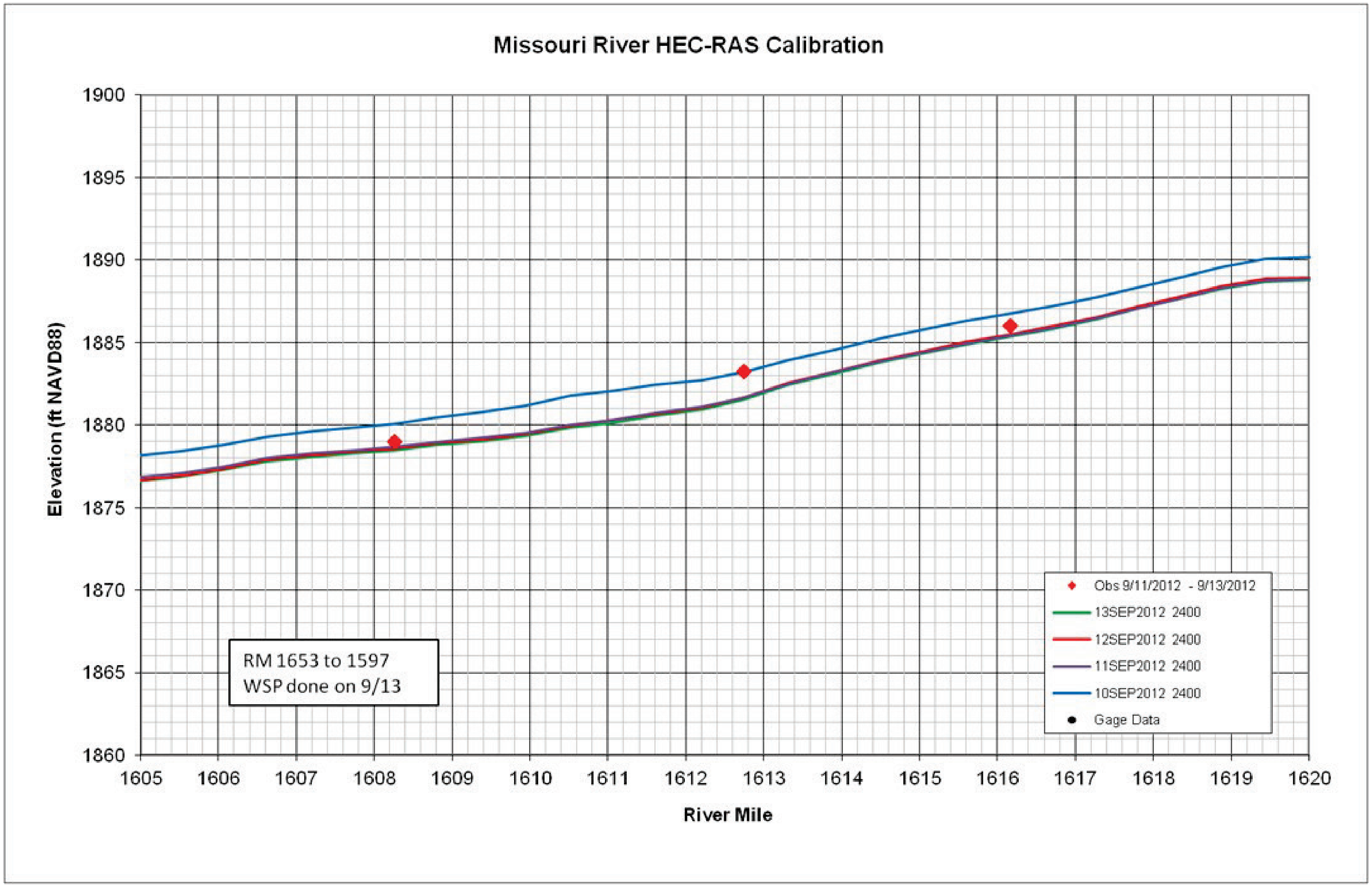


Plate 28: Measured WSP vs Computed Water Surface – RM 1605 to 1620

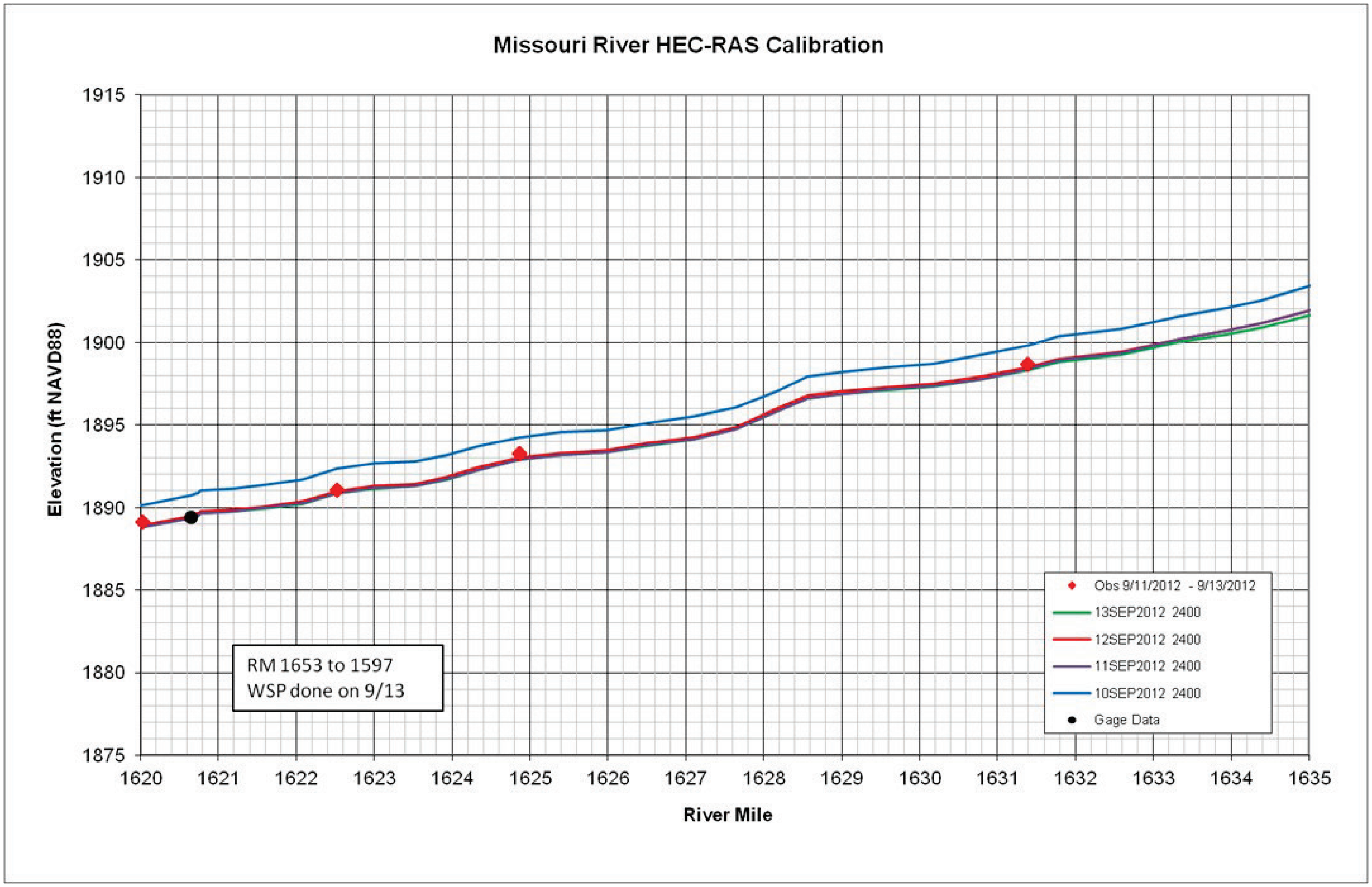


Plate 29: Measured WSP vs Computed Water Surface – RM 1620 to 1635

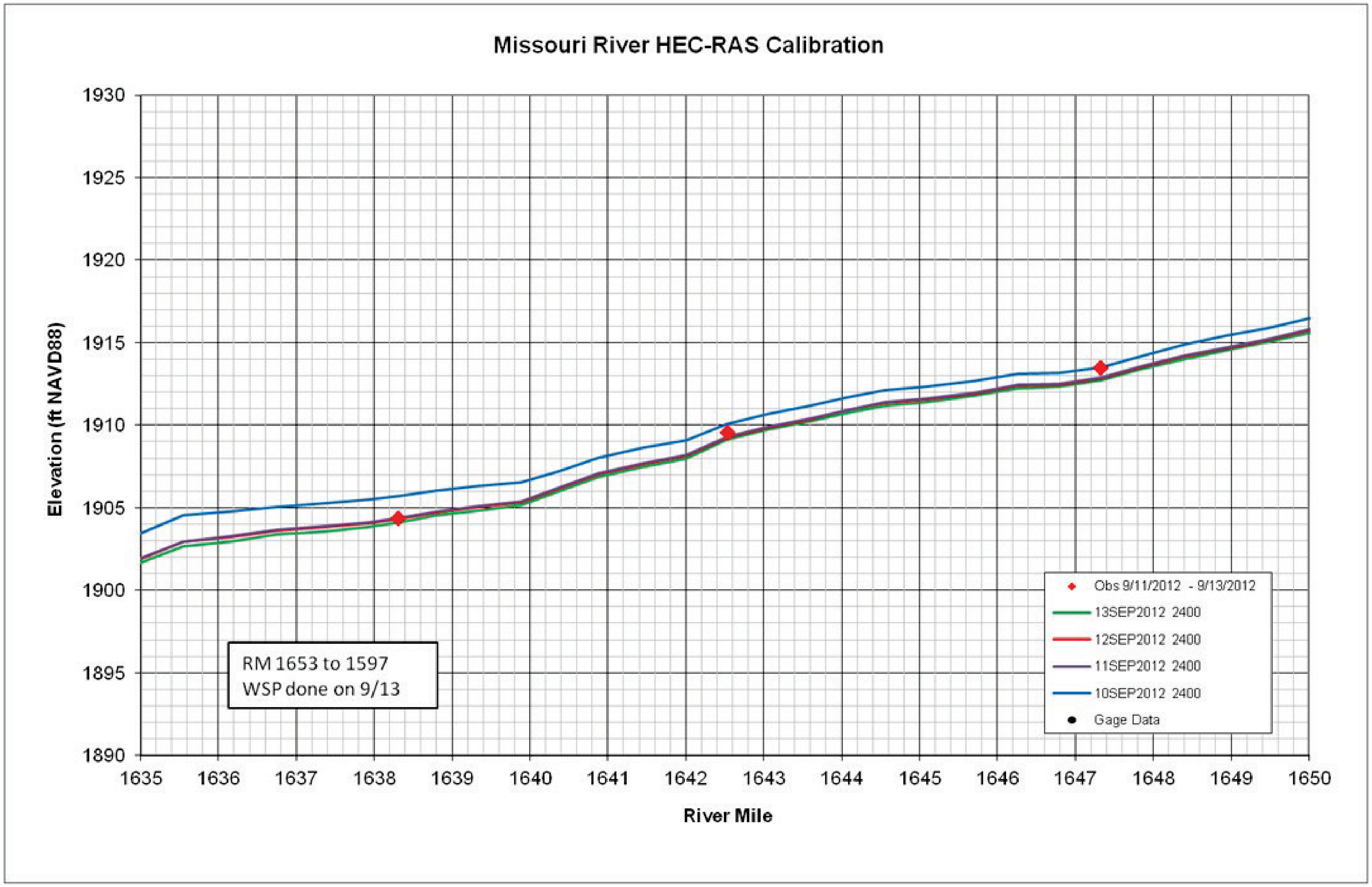


Plate 30: Measured WSP vs Computed Water Surface – RM 1635 to 1650

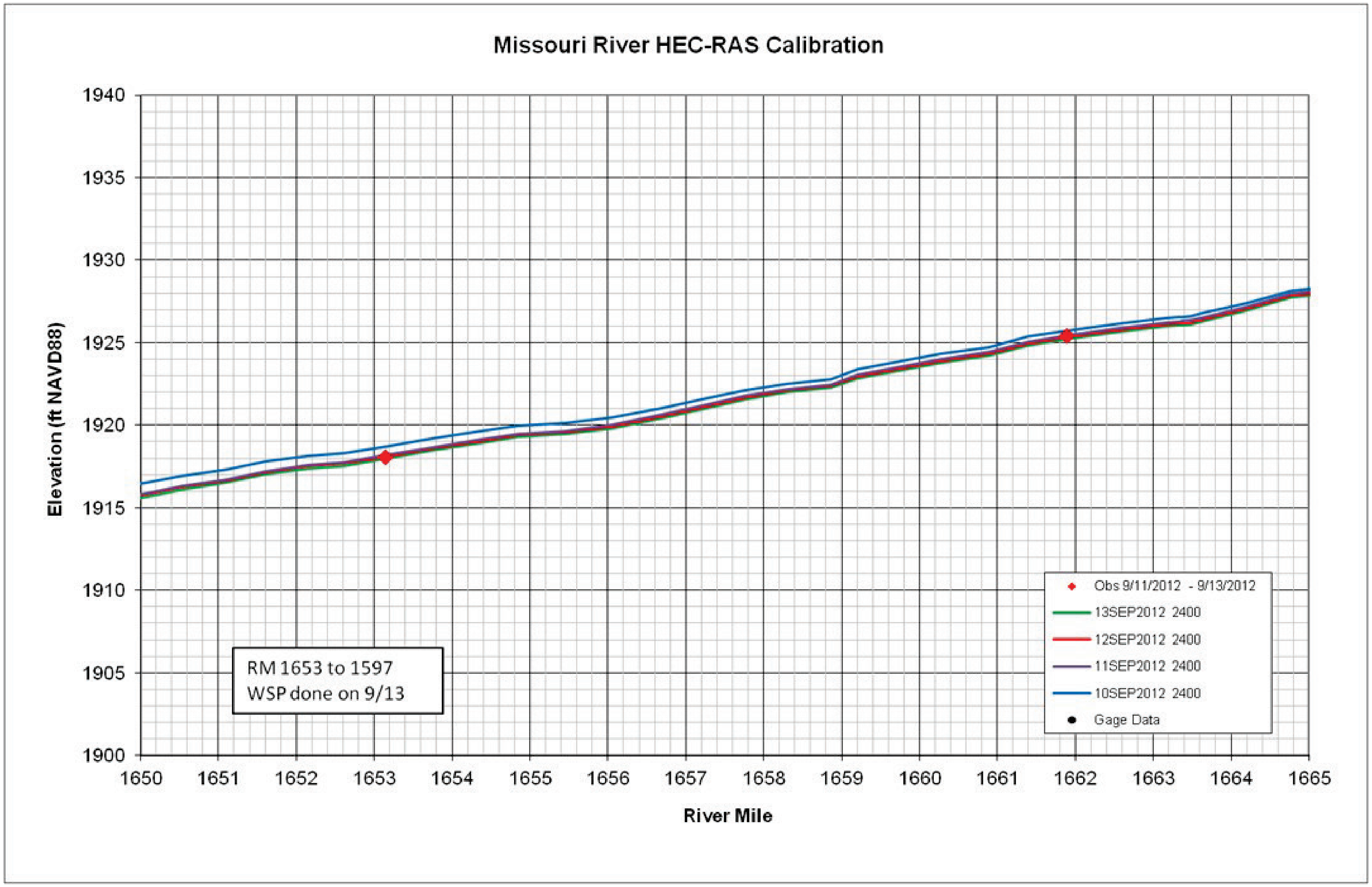


Plate 31: Measured WSP vs Computed Water Surface – RM 1650 to 1665

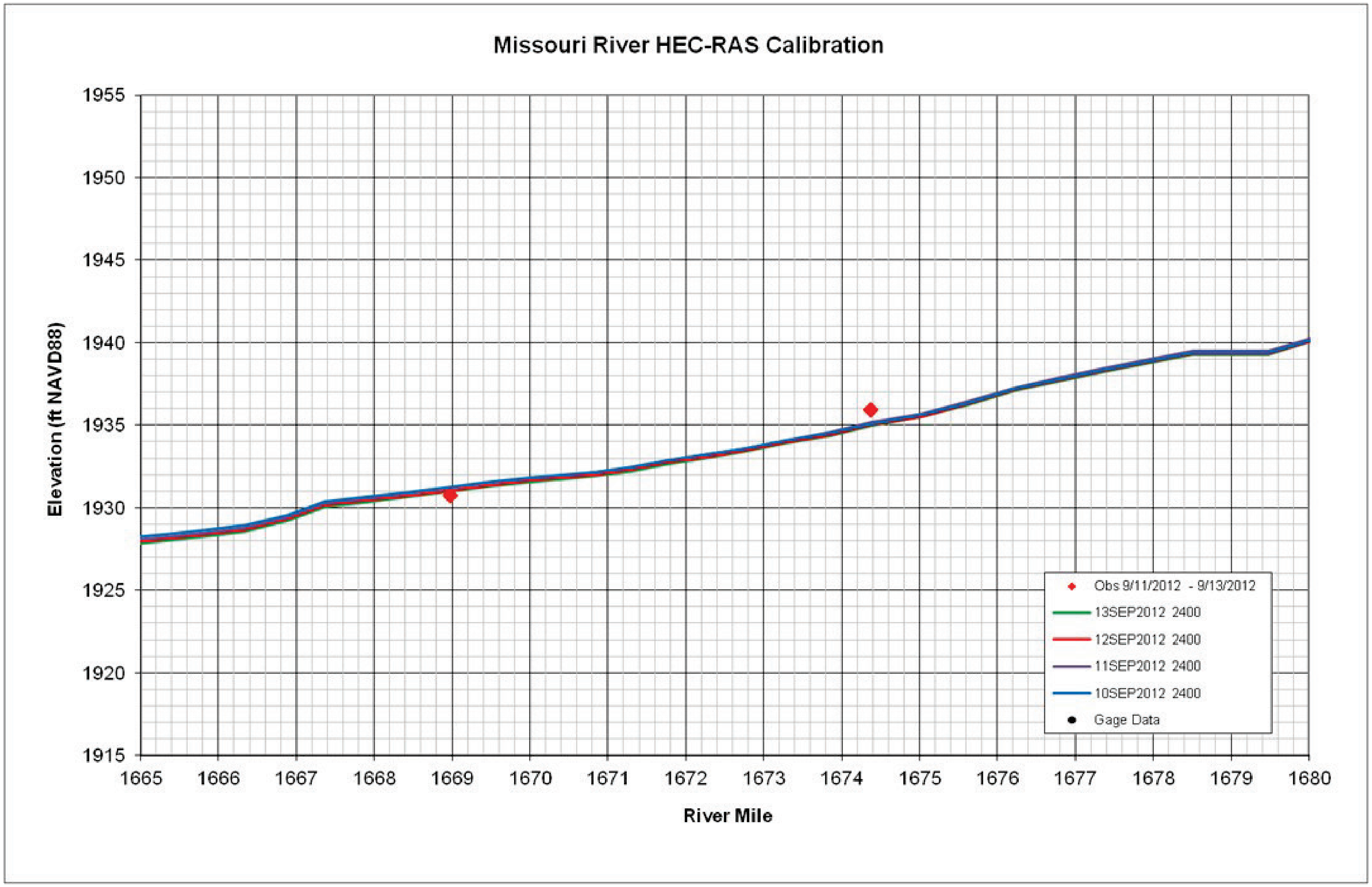


Plate 32: Measured WSP vs Computed Water Surface – RM 1665 to 1680

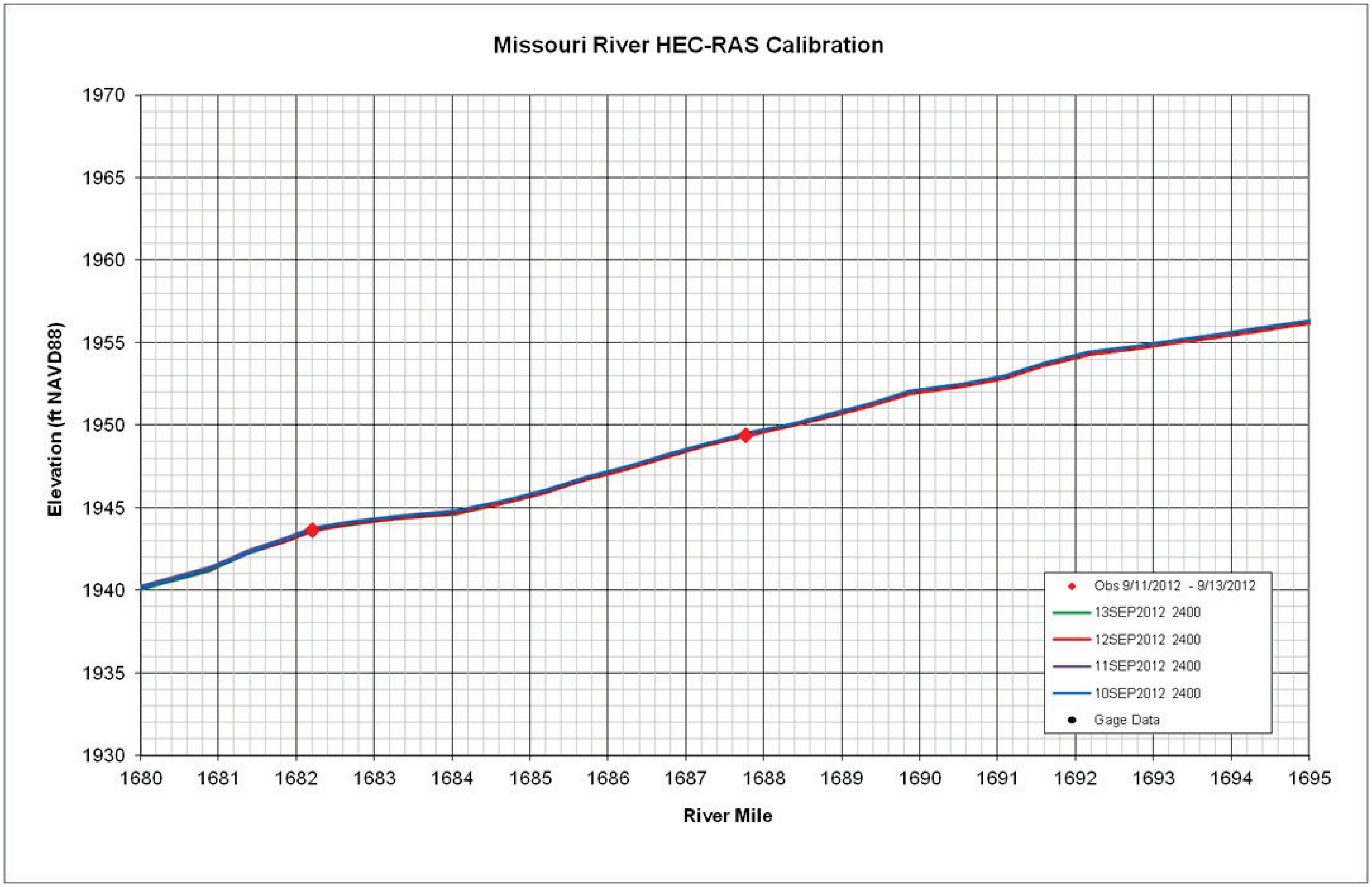


Plate 33: Measured WSP vs Computed Water Surface – RM 1680 to 1695

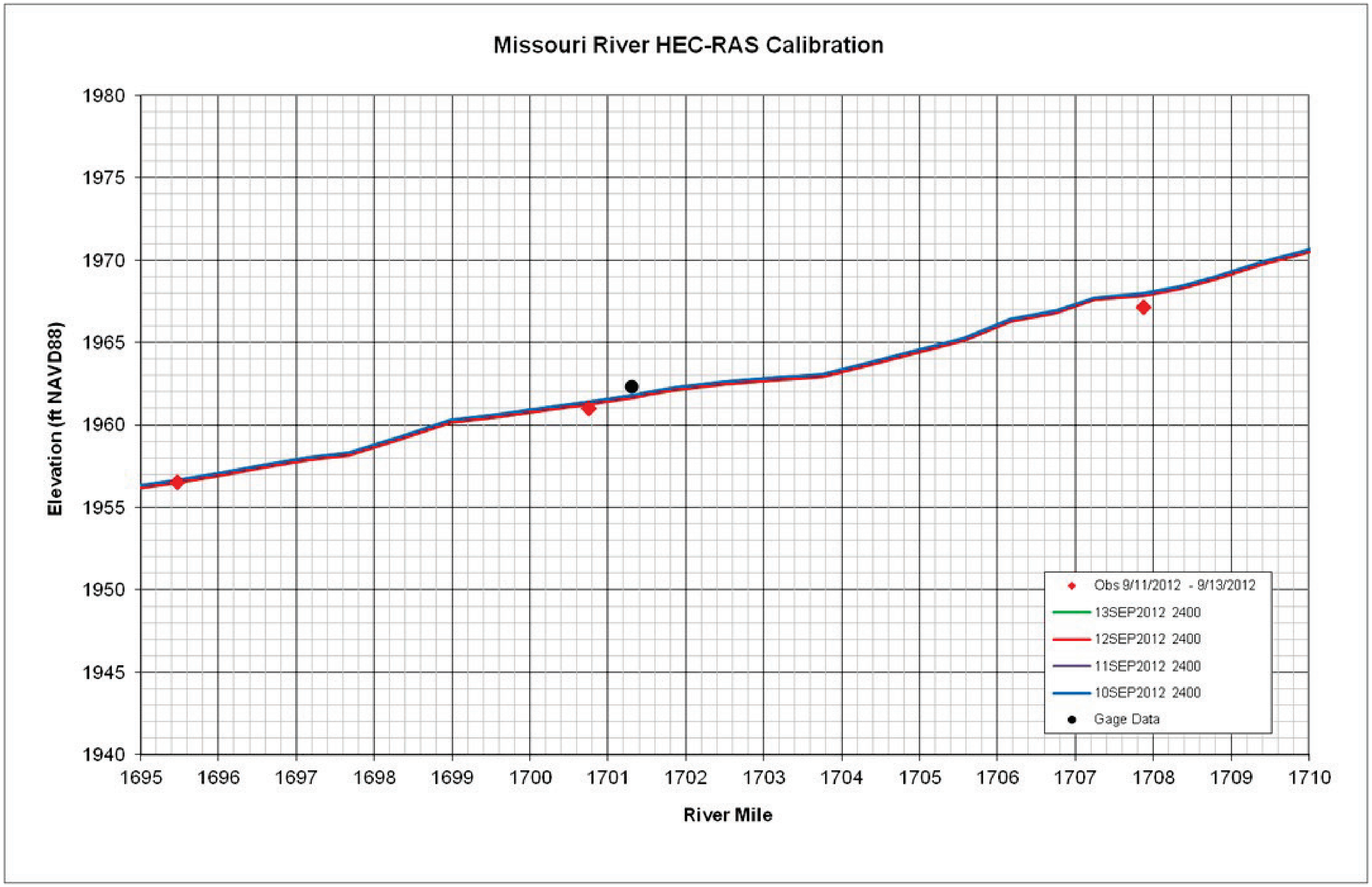


Plate 34: Measured WSP vs Computed Water Surface – RM 1695 to 1710

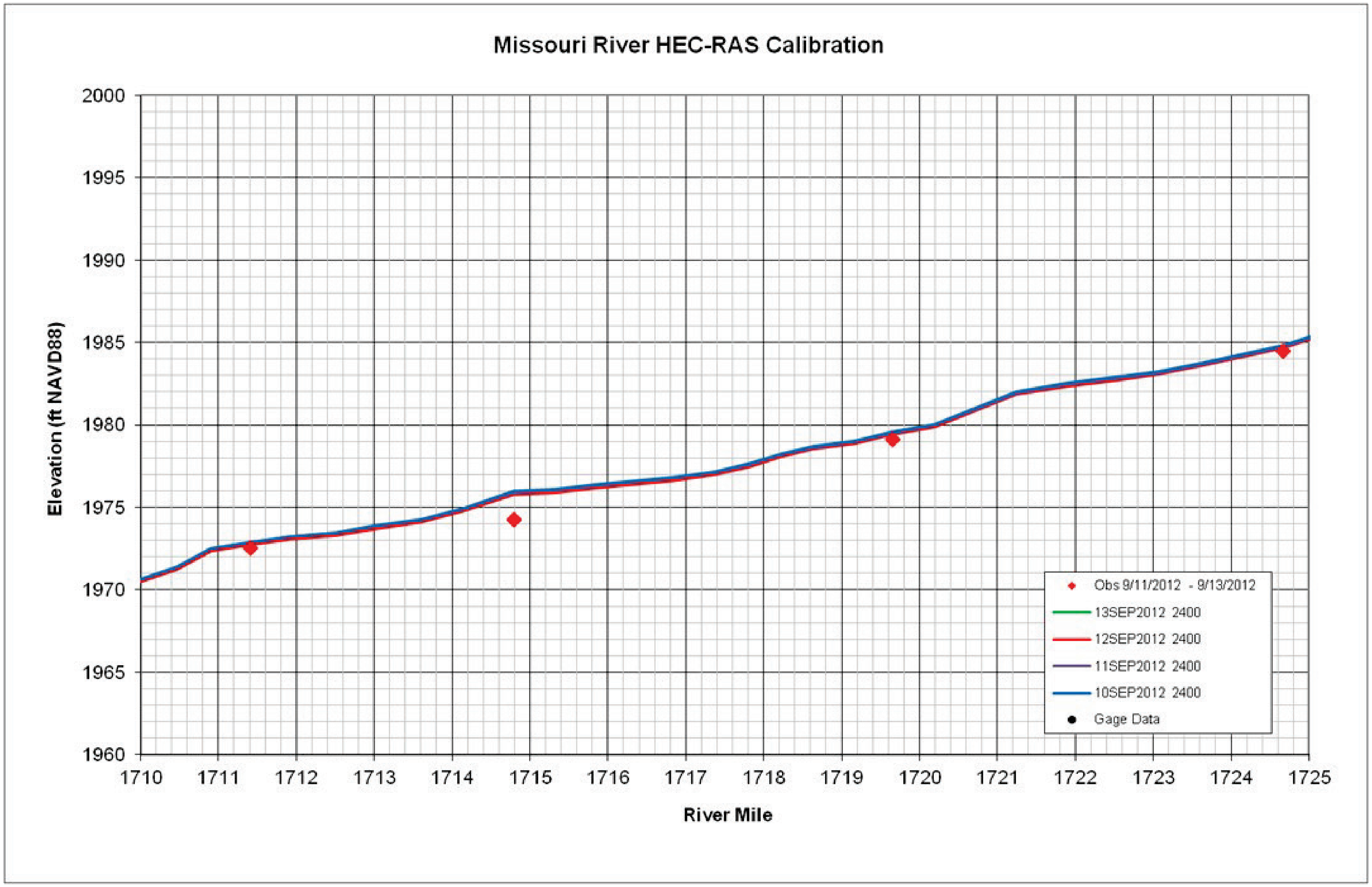


Plate 35: Measured WSP vs Computed Water Surface – RM 1710 to 1725

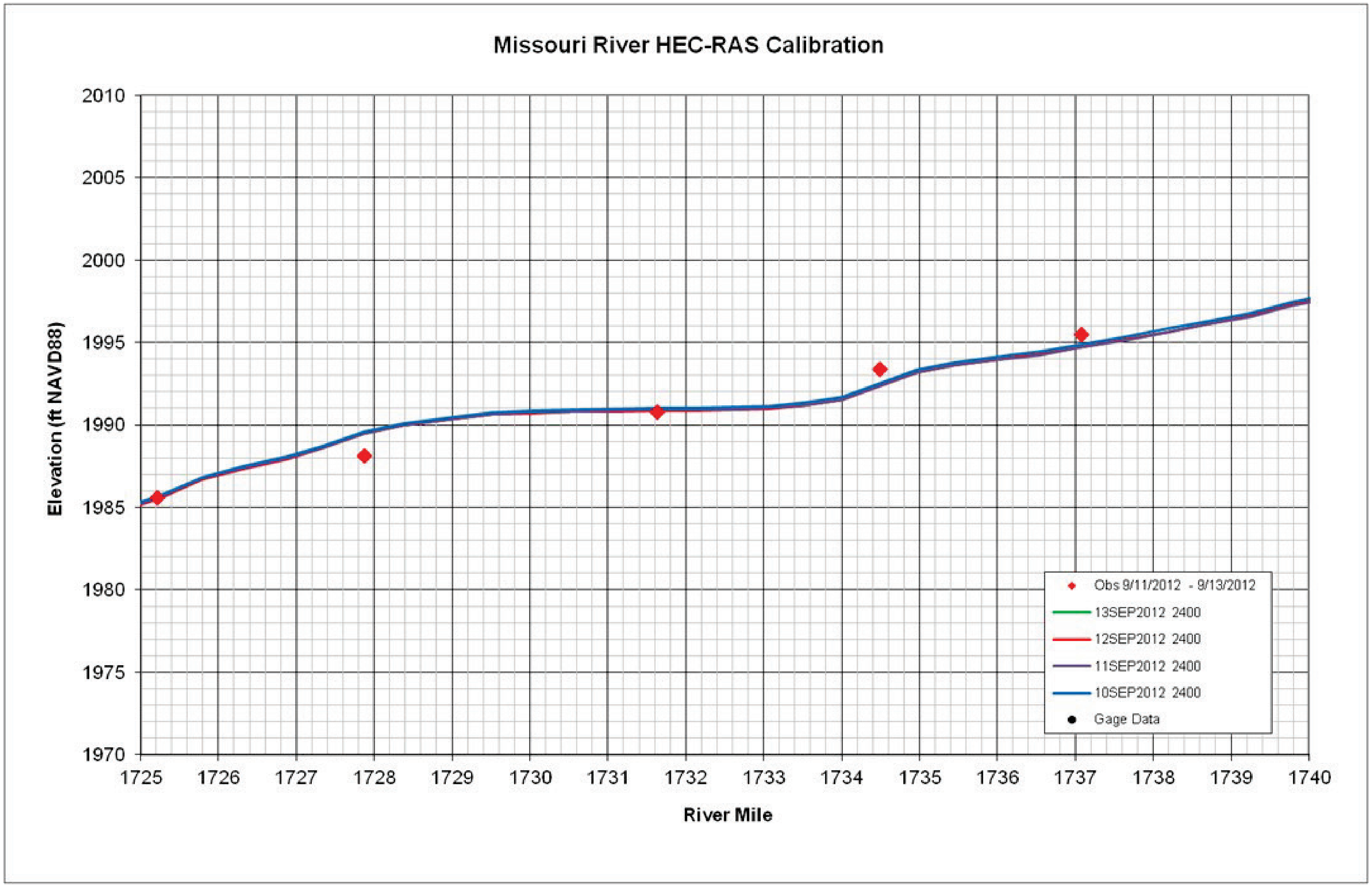


Plate 36: Measured WSP vs Computed Water Surface – RM 1725 to 1740

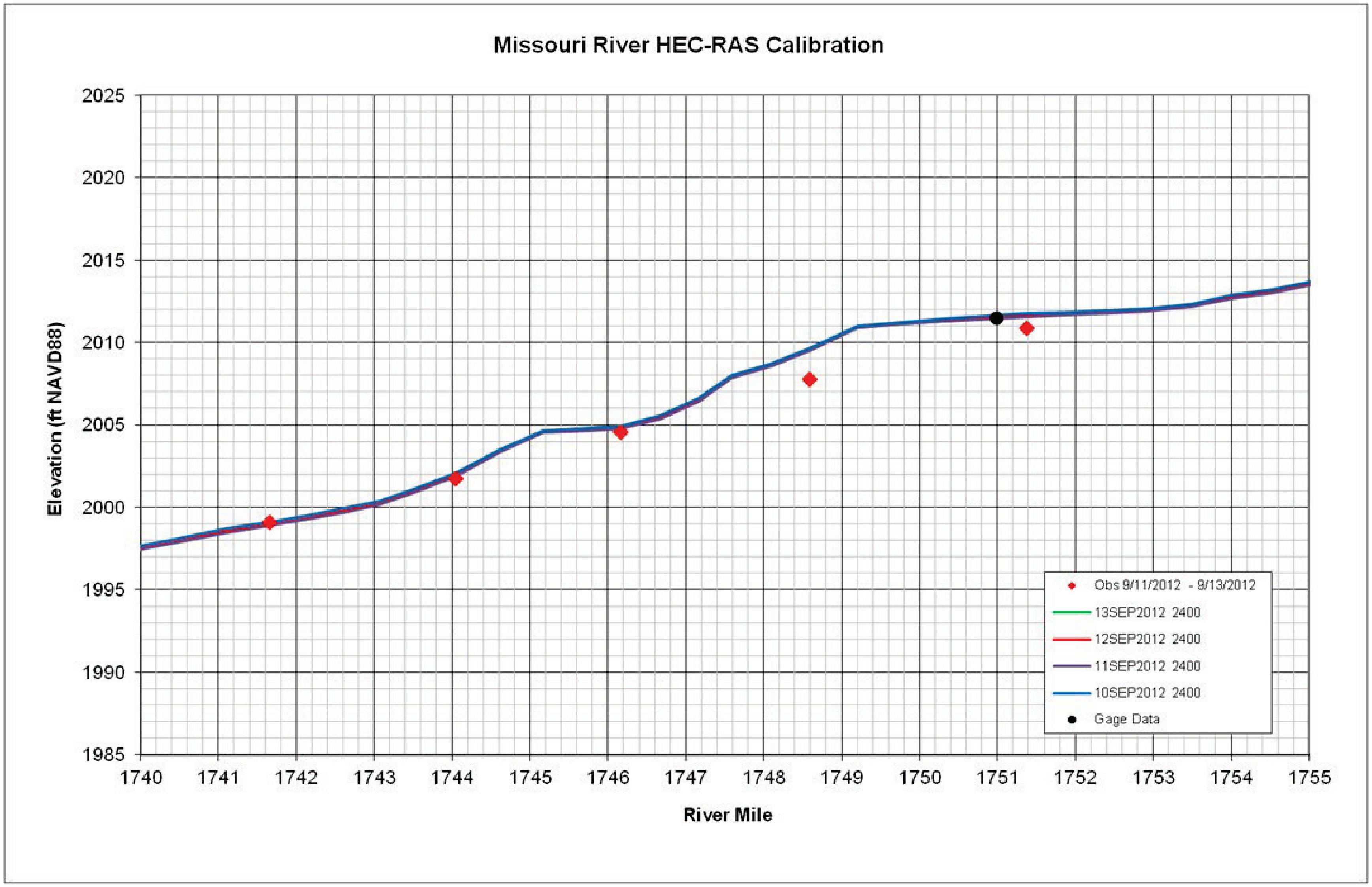


Plate 37: Measured WSP vs Computed Water Surface – RM 1740 to 1755

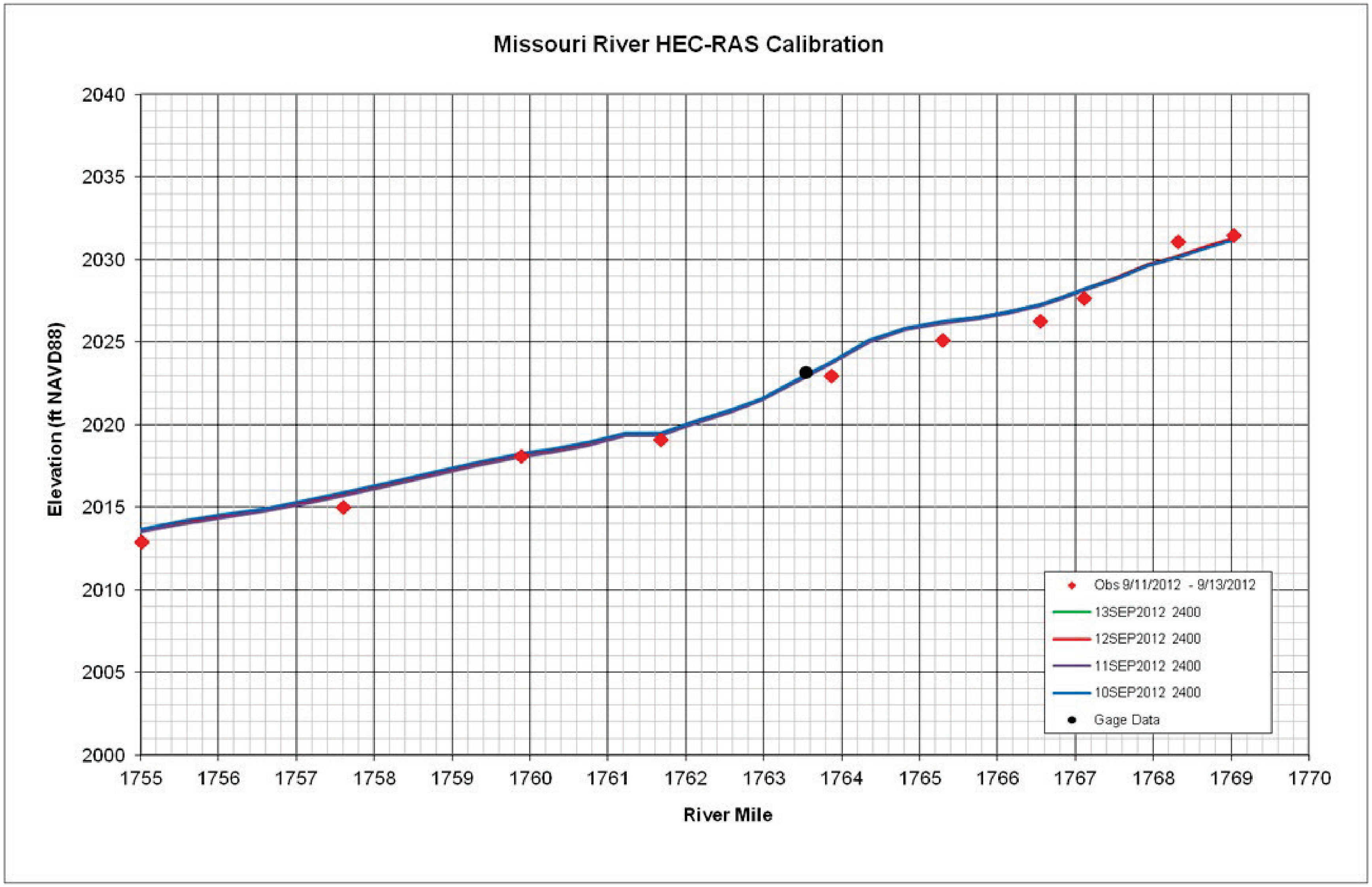


Plate 38: Measured WSP vs Computed Water Surface – RM 1755 to 1770

APPENDIX A

FORT PECK DAM TO GARRISON DAM

ATTACHMENT 1 – CROSS SECTION INTERPOLATION

Attachment 1
Missouri River RAS Modeling
Cross Section Interpolation
9 July 2014

Overview

The Missouri River RAS unsteady modeling project will construct unsteady flow models for the Missouri River from Ft Peck Dam, Montana, to St. Louis, Missouri. Upstream of Gavins Point Dam (near RM 811), the hydrographic data primarily consists of sediment range surveys used to monitor aggradation / degradation between the dams. Figure 1 illustrates the reach locations.

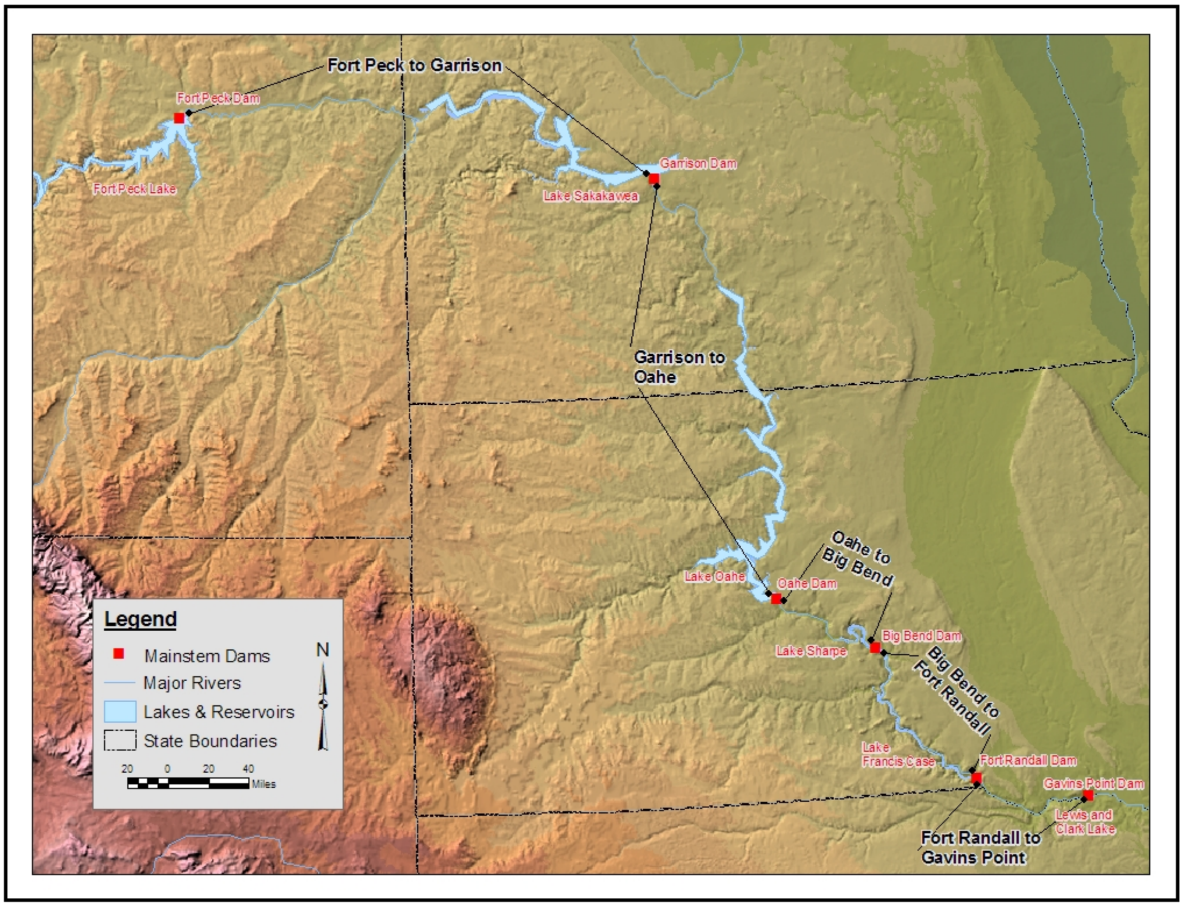


Figure 1. Mainstem Dam Modeling Reaches

Cross Section Interval

Model assembly principles and the goals of the study indicate that a cross section interval on the order of 2500 to 3000 feet would be appropriate. The sediment range spacing typically varies on the order of 1 to 3 miles so cross sections were interpolated in RAS to obtain estimated bathymetry.

Linear Interpolation

The between 2 cross sections option in the cross section interpolation tool in RAS was used to interpolate the underwater portion of the cross sections between the sediment ranges. Using the option **Generate for display as perpendicular segments to reach invert** places the interpolated cross sections along the stream centerline. A maximum distance of 3000 feet was used and additional cords were added where needed (the default cords are at the ends, banks, and channel invert). As can be seen in Figure 2 below, the RAS interpolated cross sections were imported into ArcMap and were adjusted to better represent the channel and floodplain. These new re-drawn cross sections were then used in GeoRAS to obtain elevation data along the correct alignment. The estimated (RAS interpolated) bathymetry was then merged into the re-drawn overbank cross section data.

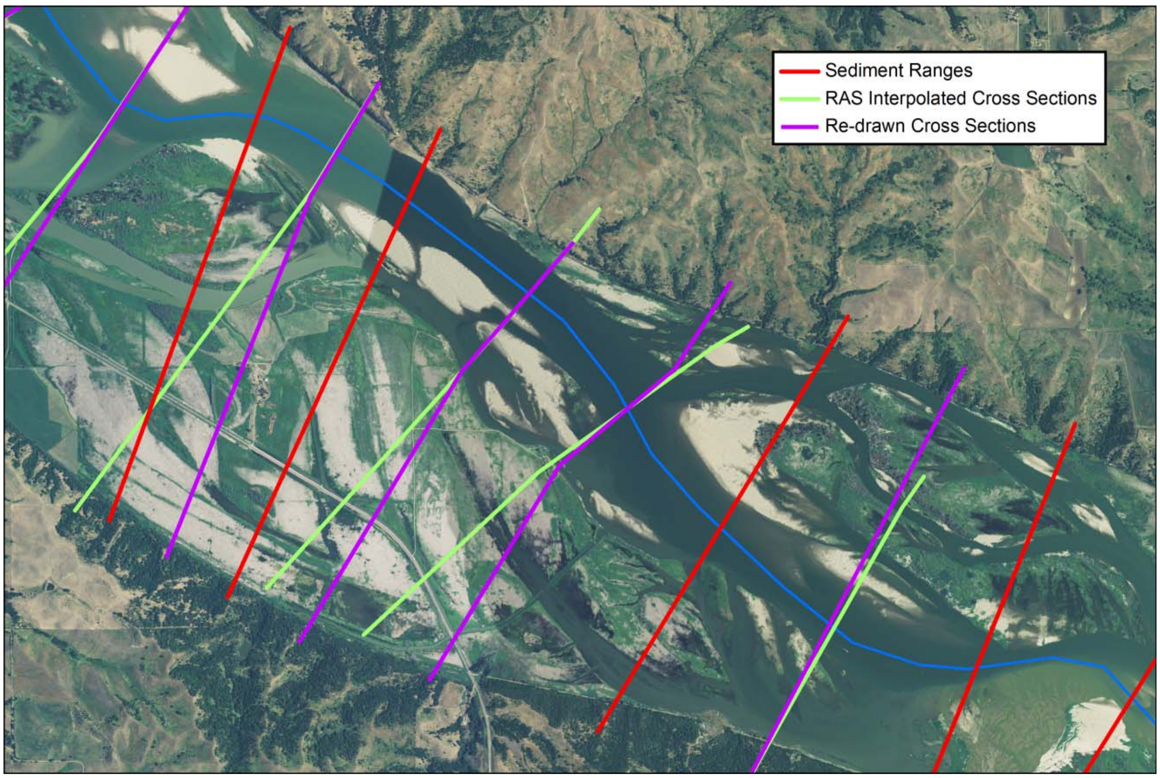


Figure 2. Comparison of RAS interpolated and re-drawn cross sections.

Estimating Bathymetry

After the new cross sections were re-cut in GeoRAS with LiDAR data, an underwater portion needed to be added to the cross section since the LiDAR does not penetrate below the water surface. Bathymetry was estimated by either using the RAS interpolated bathymetry or if that did not fit correctly with the overbank data, a nearby sediment range's bathymetry was vertically shifted and merged in. Differing widths and sandbar configurations presented a challenge to find another cross section that was similar.

When using a nearby range's bathymetry as an estimate a vertical shift was applied. The shift was based on the energy grade line slope (broken into several reaches) and the distance between the two cross

sections. See Figure 3 for an example of an energy grade line slope plot from a rough sediment range only model.

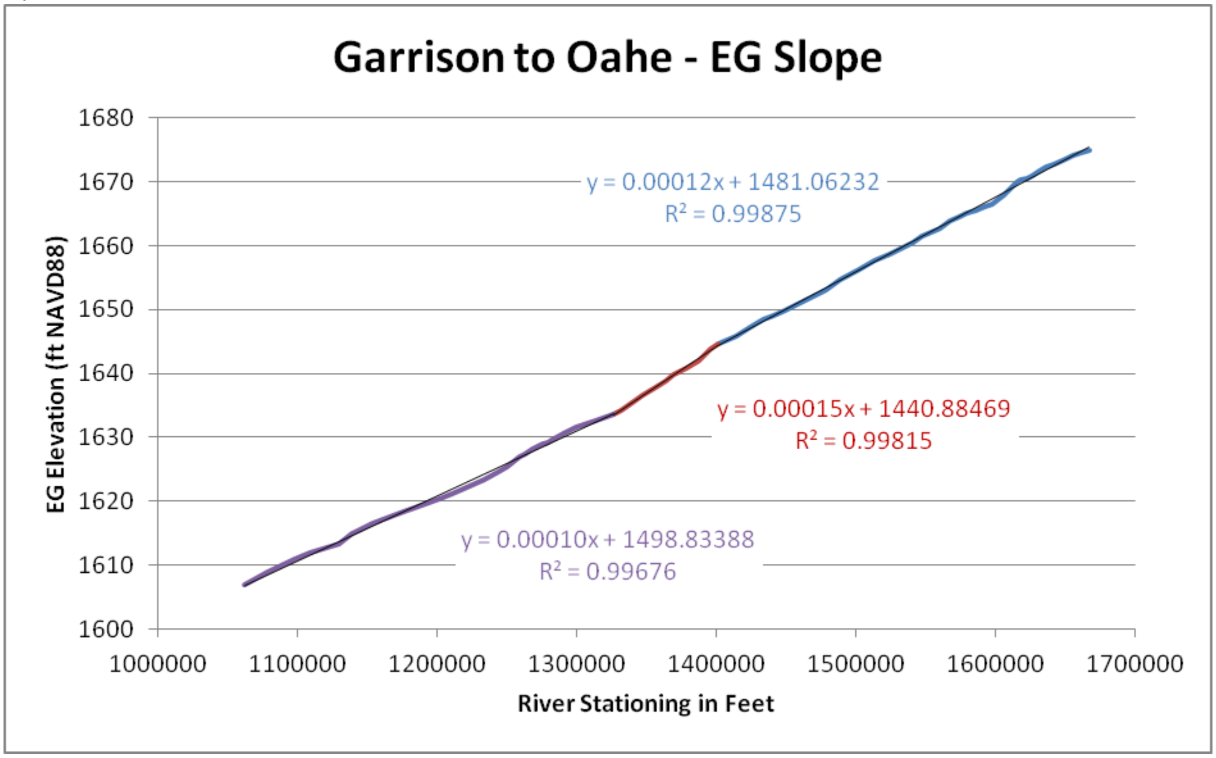


Figure 3. Example of Energy Grade Line Slope Plot

Examples of merging bathymetry into the cross sections are illustrated in Figures 4 – 9. The four example sites are shown in Figures 10 – 24.

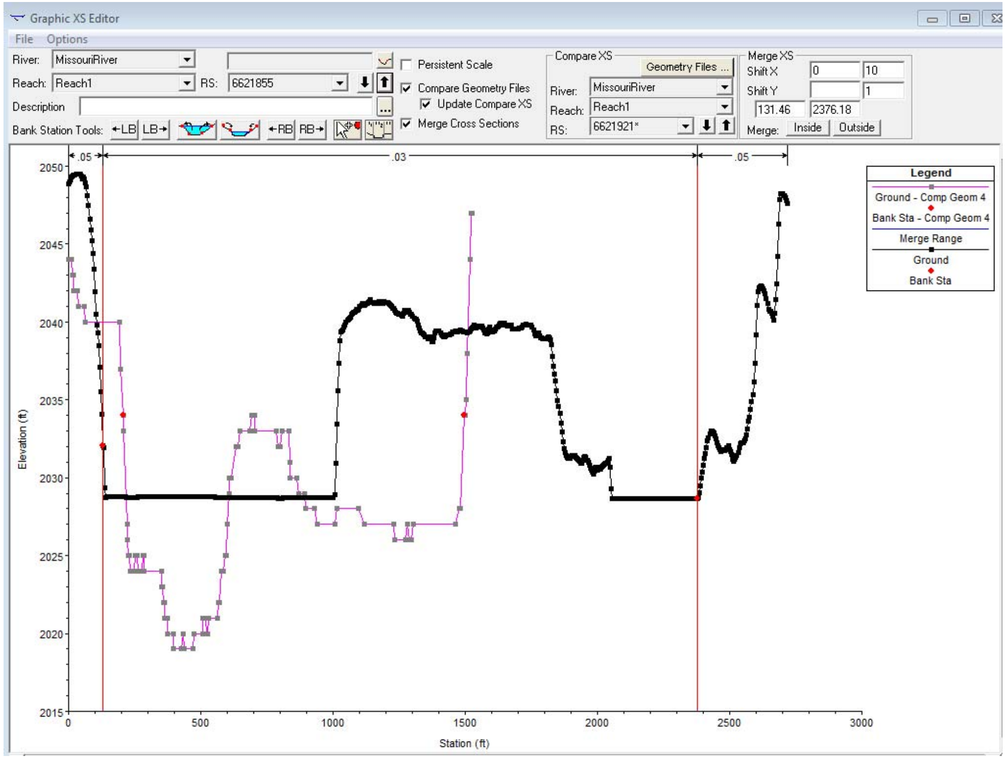


Figure 4. Paired XSs for merging, center island and changing channel widths cause alignment issues.

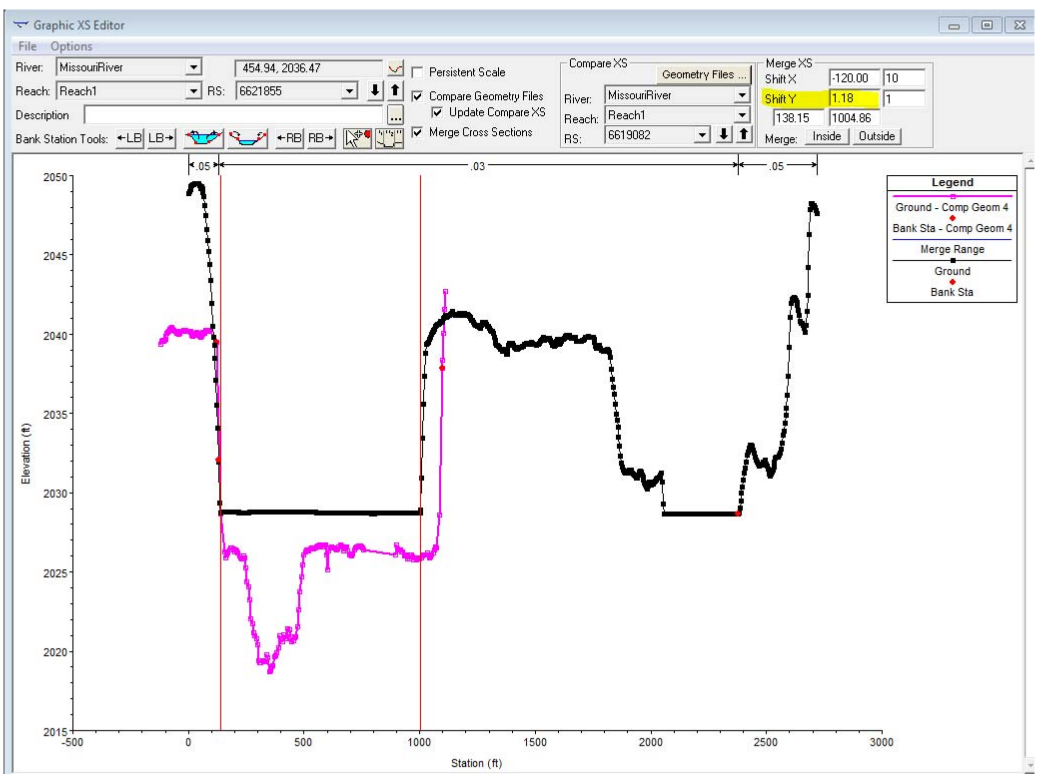


Figure 5. Same Cross-Section as in Figure with next downstream Rangeline vertically adjusted

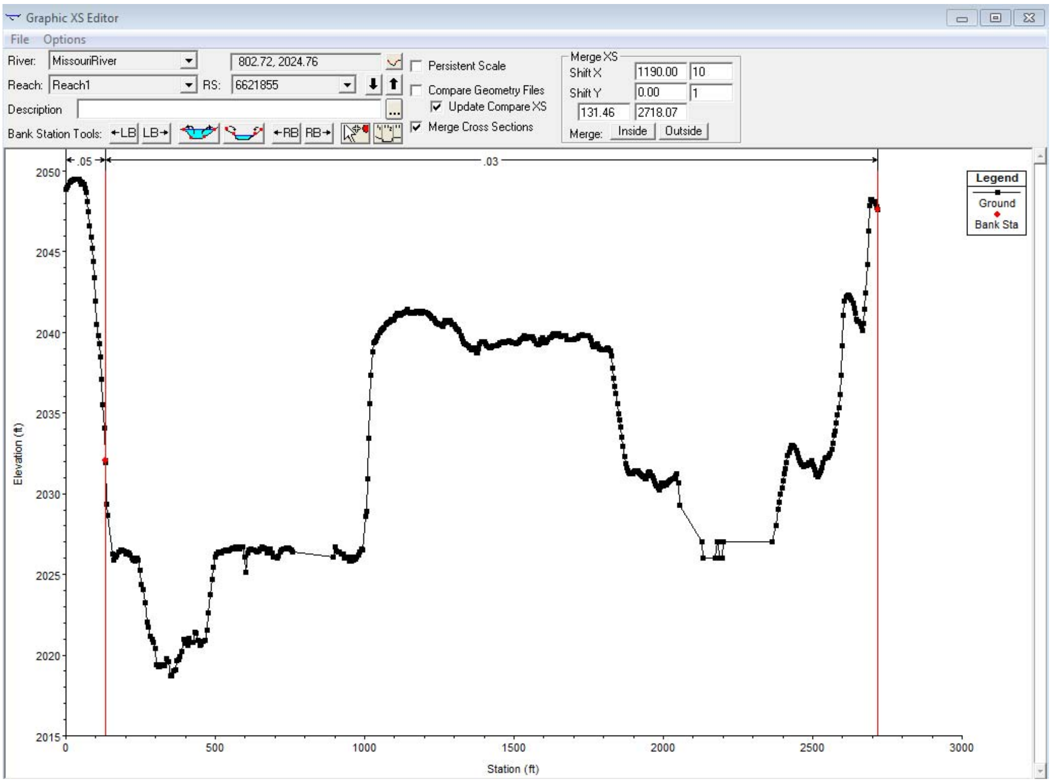


Figure 6. Composite Cross Section using the GeoRAS cut LiDAR data for above the WSE and the downstream rangeline and HEC-RAS interpolated cross sections for the channel data estimation.

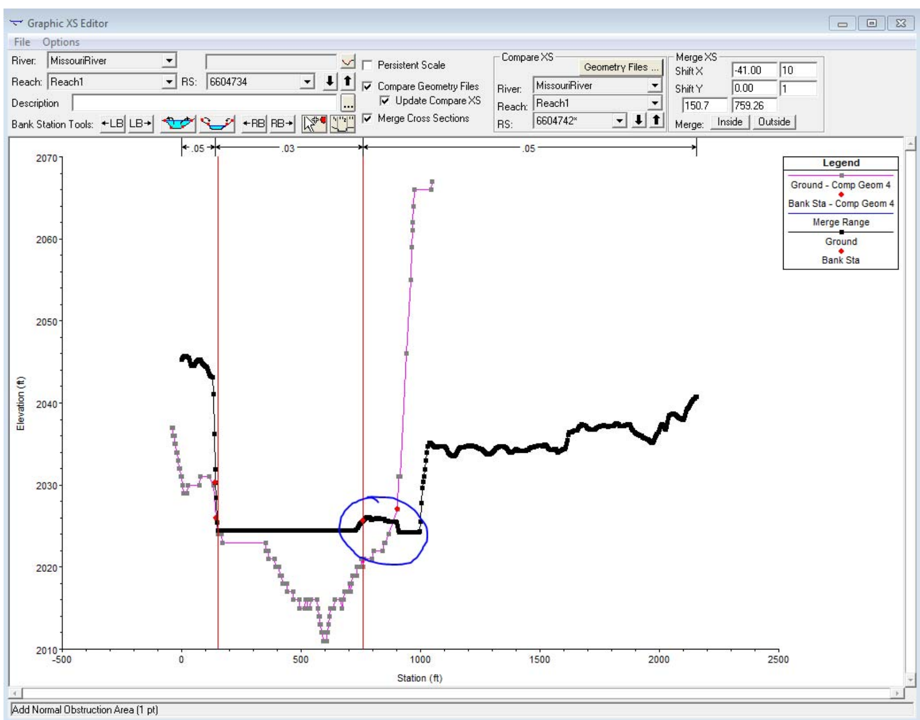


Figure 7. Sandbars in the channel present another challenge.

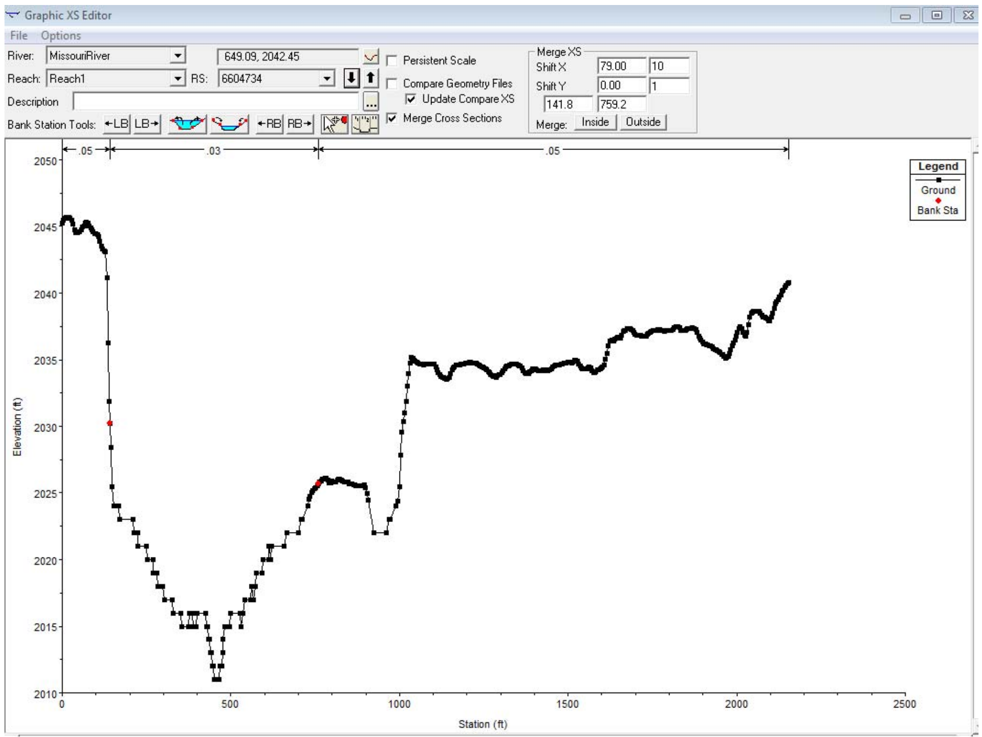


Figure 8. Composite Cross Section from Figure with sandbar

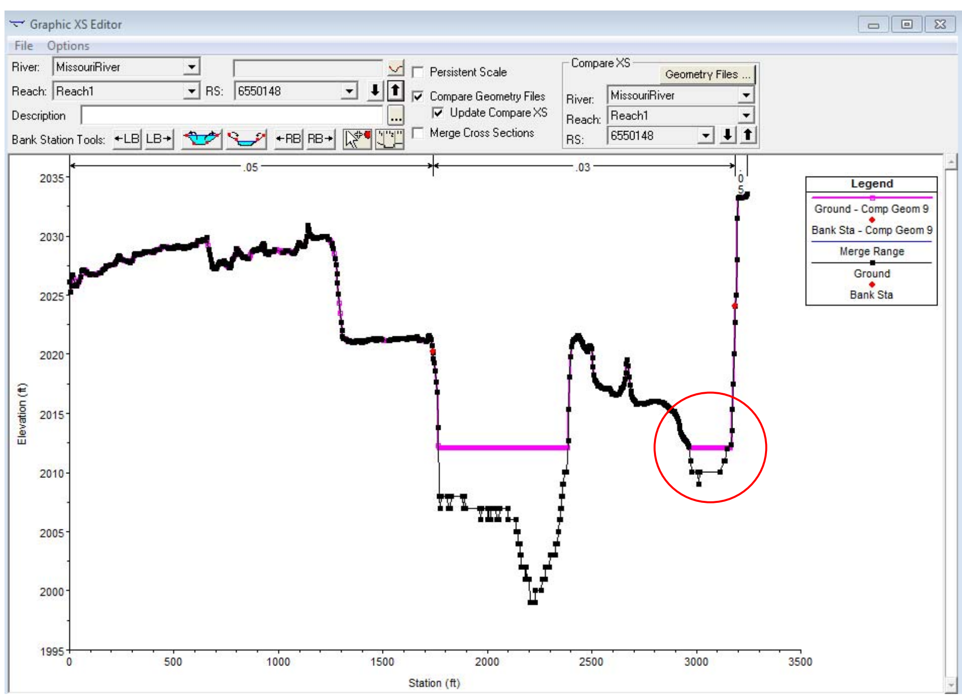


Figure 9. Before and After Merge - Small Channel inside of sandbar is Estimated

Example Sites

Comparison of RAS output built using Rangeline Interpolation and LiDAR data Merged with interpolated cross-sections. Images are at a Flow of 10,250 cfs which is about a normal annual flow in the Ft Peck Reach.

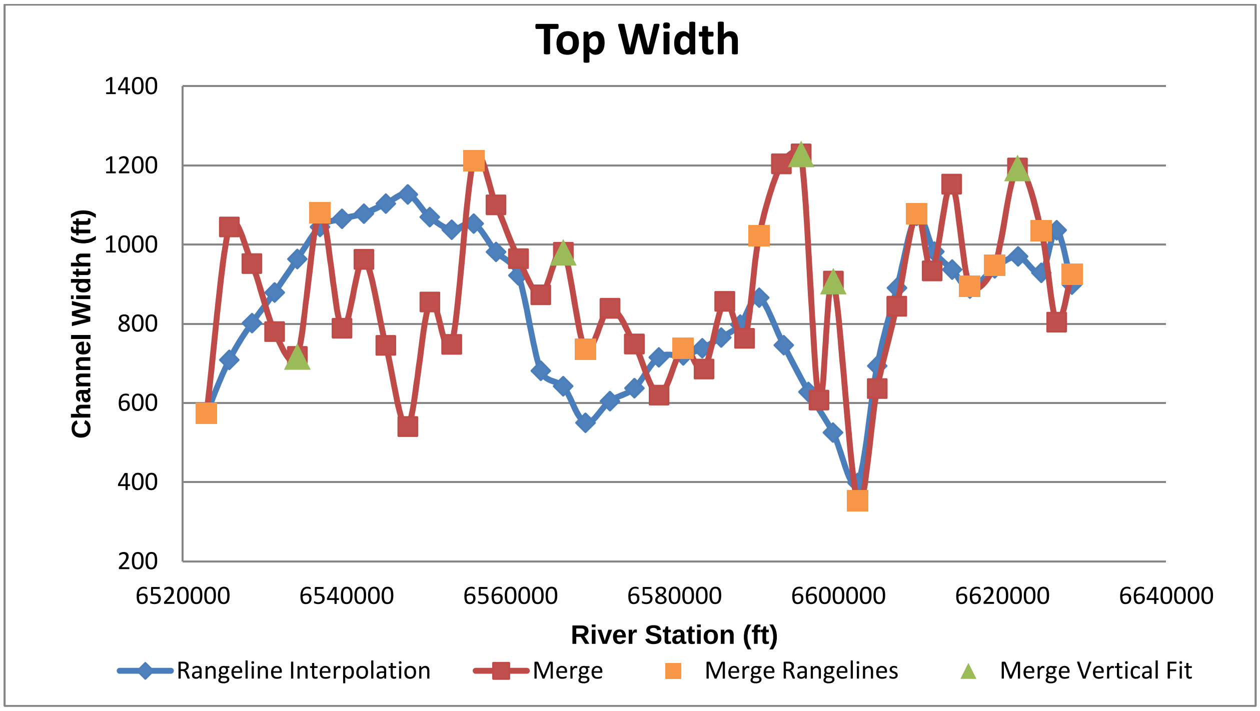


Figure10. Comparison of the RAS output for Top Width from the linearly interpolated rangeline XS model and the merged topo and rangeline model. The rangeline XSs based on survey data are orange. The green markers denote the topo XSs that didn't fit with the corresponding interpolated XS and used a more suitable nearby XS vertically fit to the local slope to merge the below water channel.

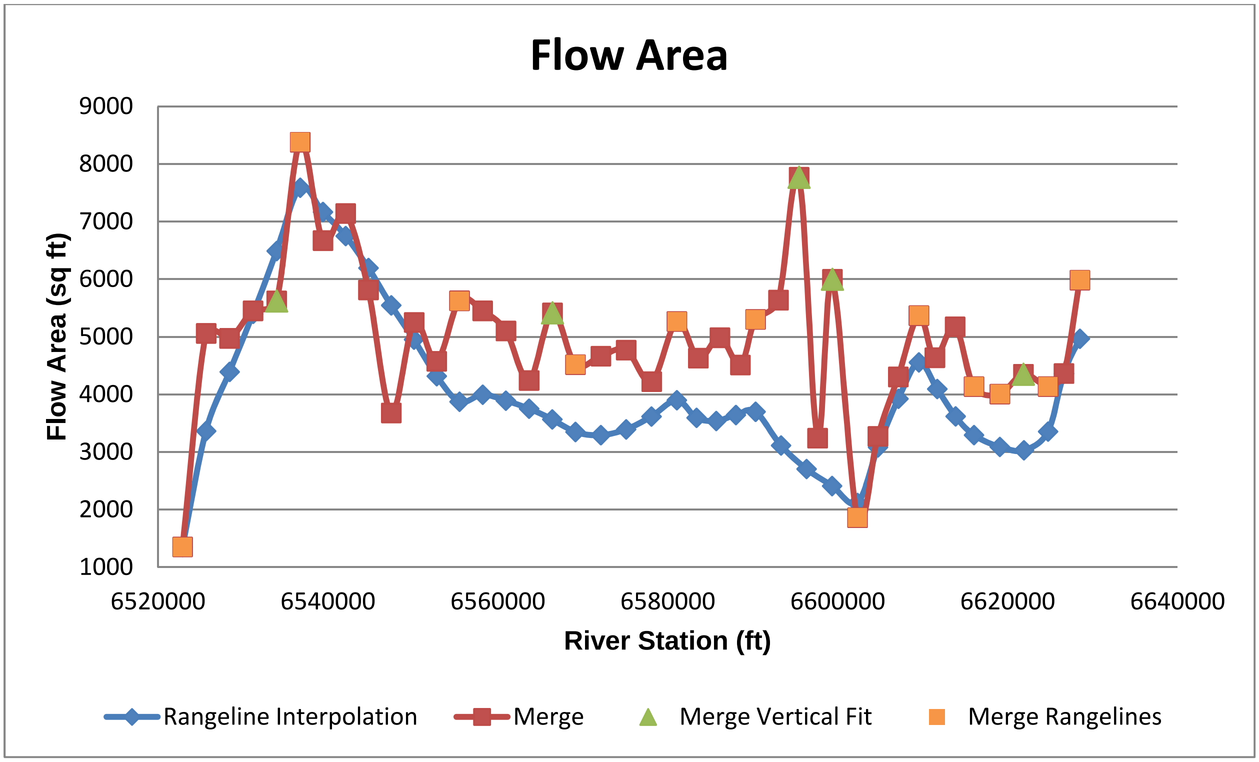


Figure11. Comparison of the RAS output for Flow Area from the linearly interpolated rangeline XS model and the merged topo and rangeline model.

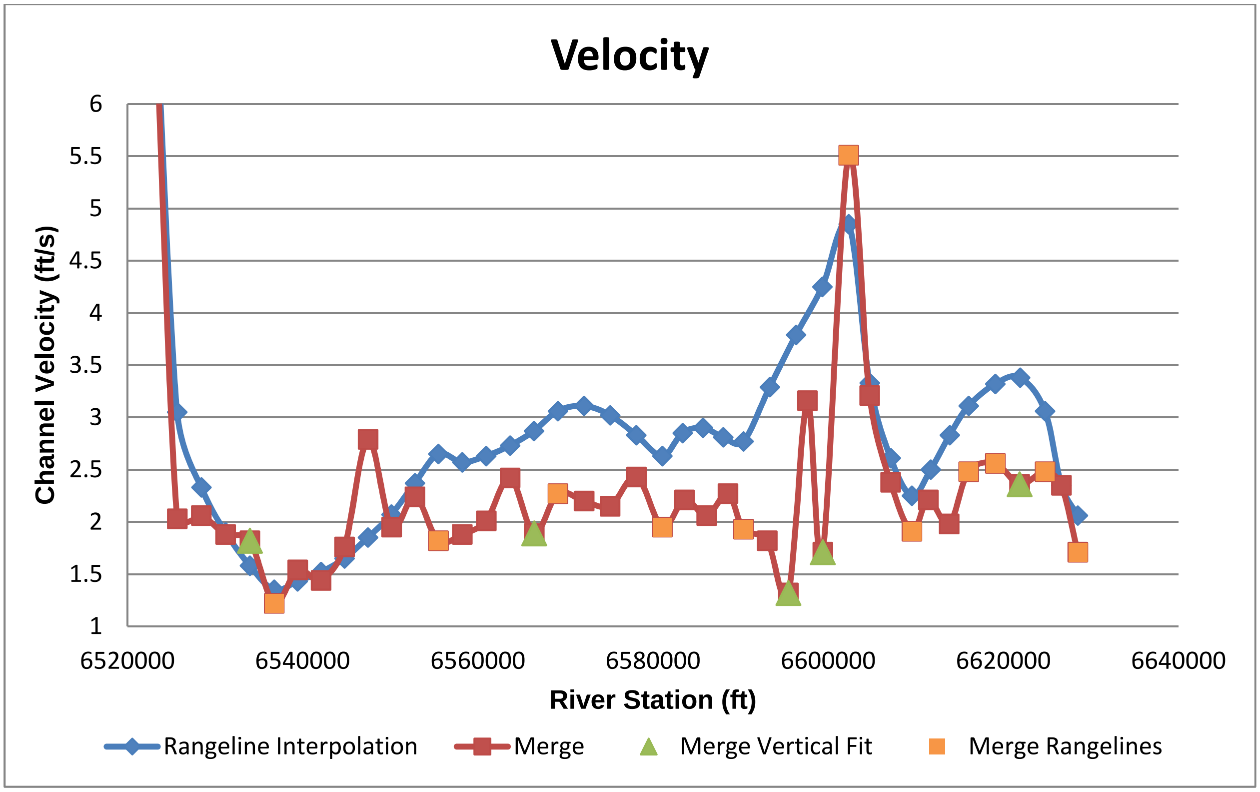


Figure 12. Comparison of the RAS output for Velocity from the linearly interpolated rangeline XS model and the merged topo and rangeline model.

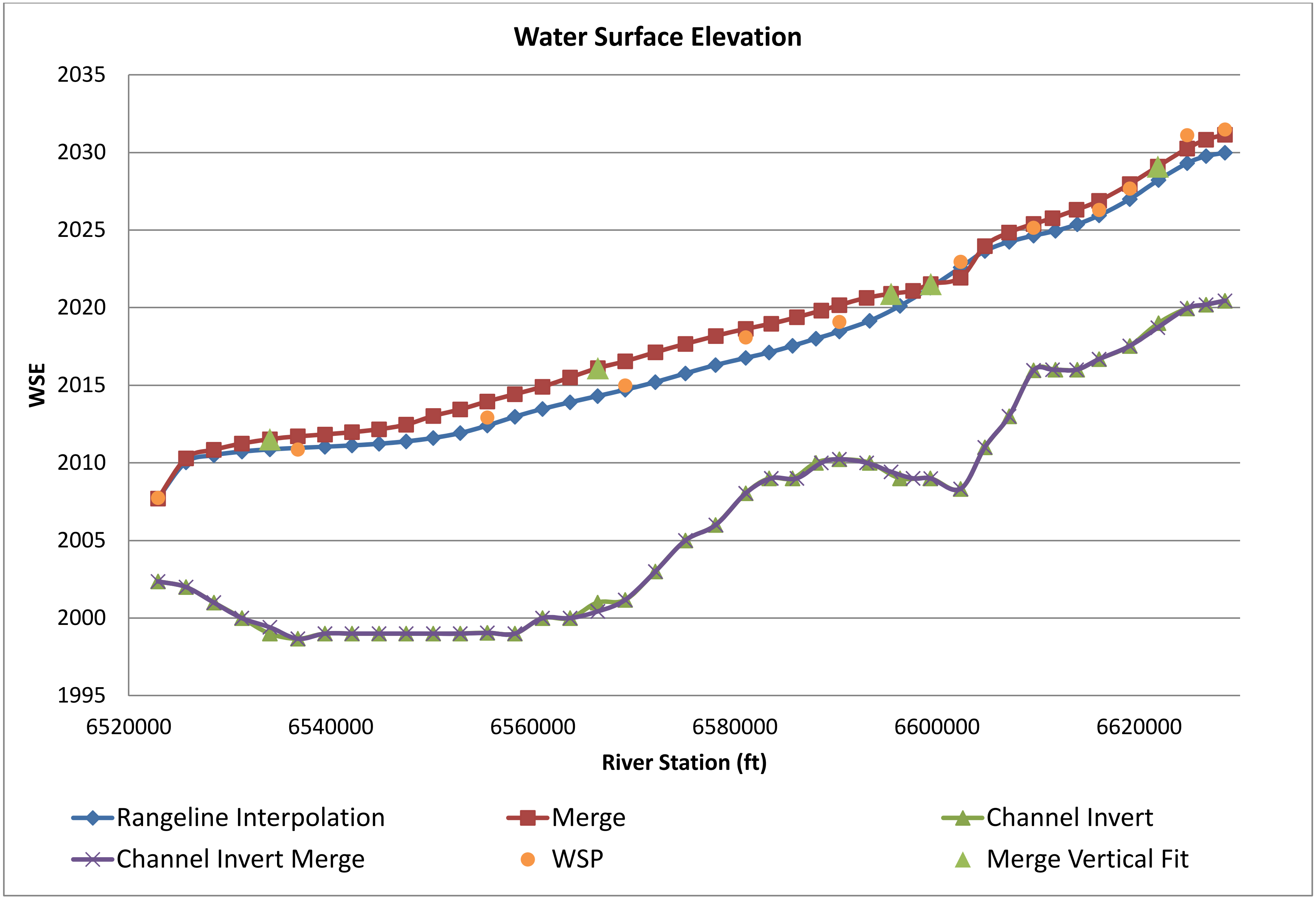
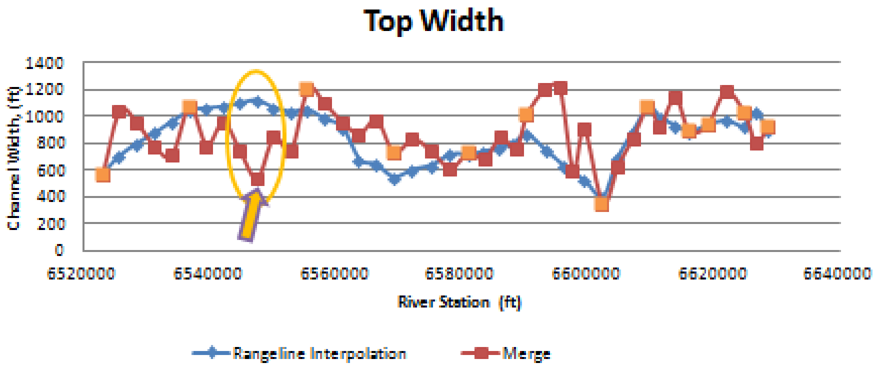


Figure 13. RAS Models compared to the 2012 Water Surface Profile (WSP) at Flow 10,250 cfs

Example 1



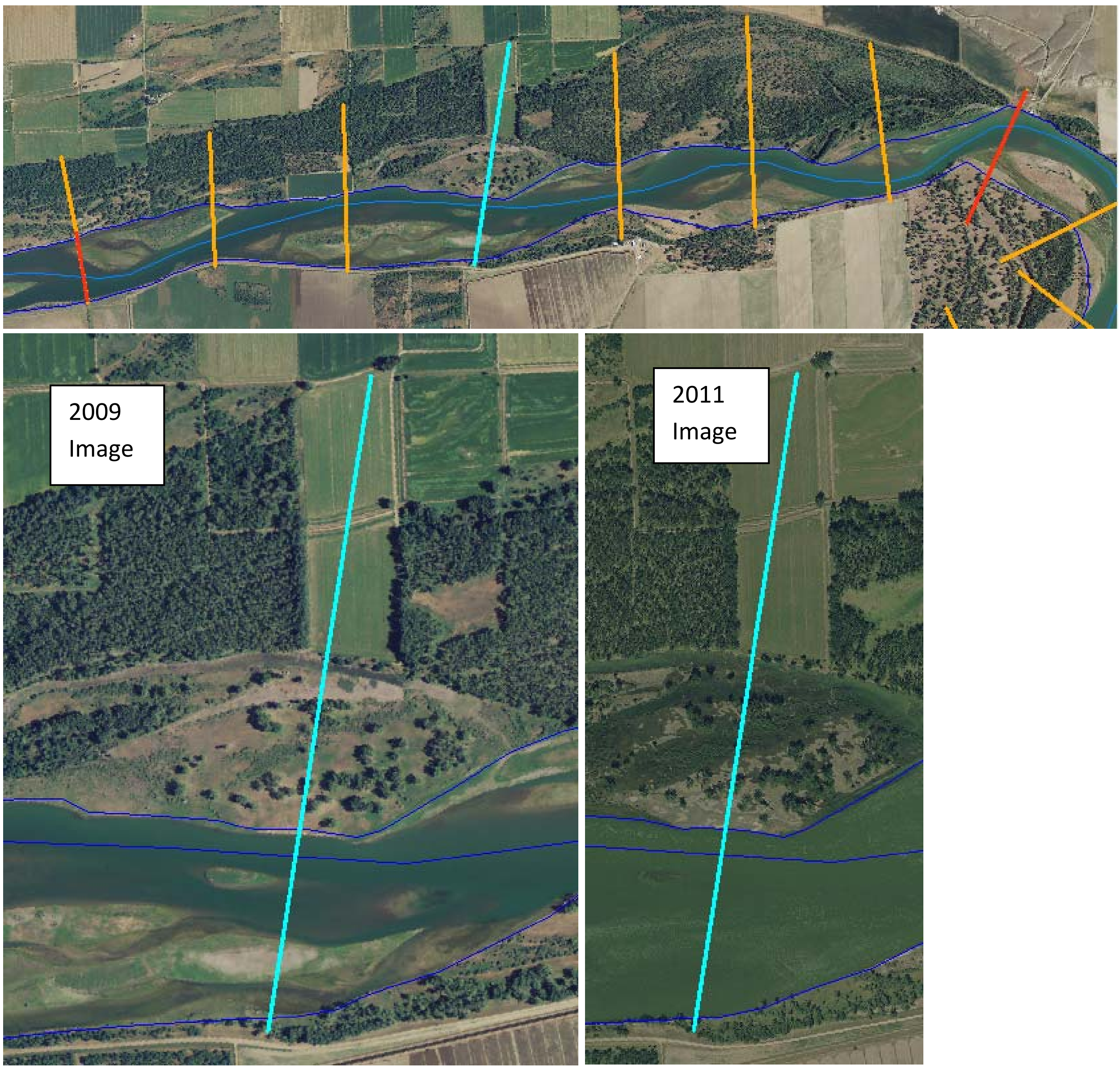


Figure 14. Aerial images of the XS, including the nearest Rangelines in the top image and closer view of the XS during a normal to low flow in 2009 and a high flow in 2011

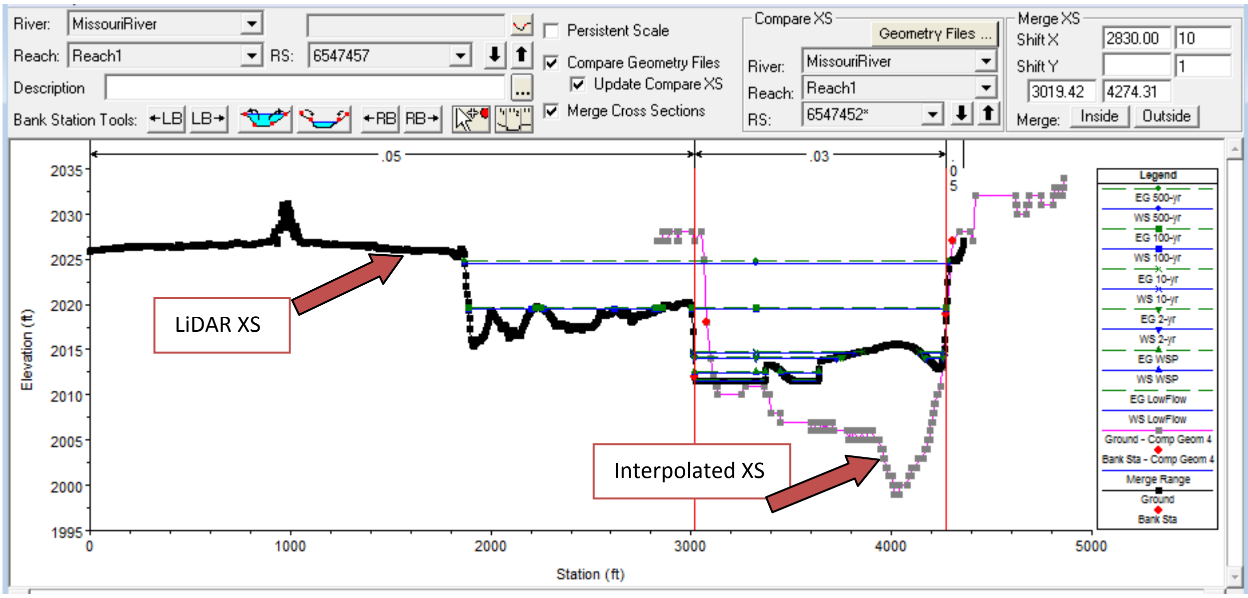


Figure 15. LiDAR XS (Black) and corresponding Interpolated XS (Grey/Pink) – note sandbars in channel

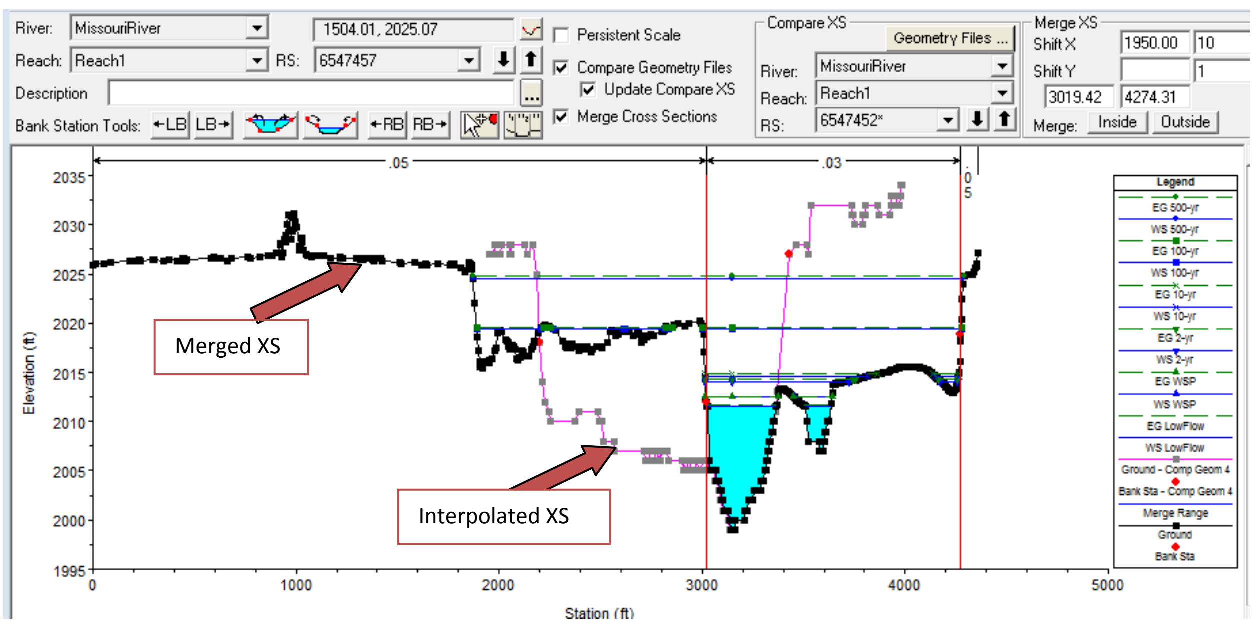


Figure 16. Merged XS (Black) and corresponding Interpolated XS (Grey/Pink)

Example 2

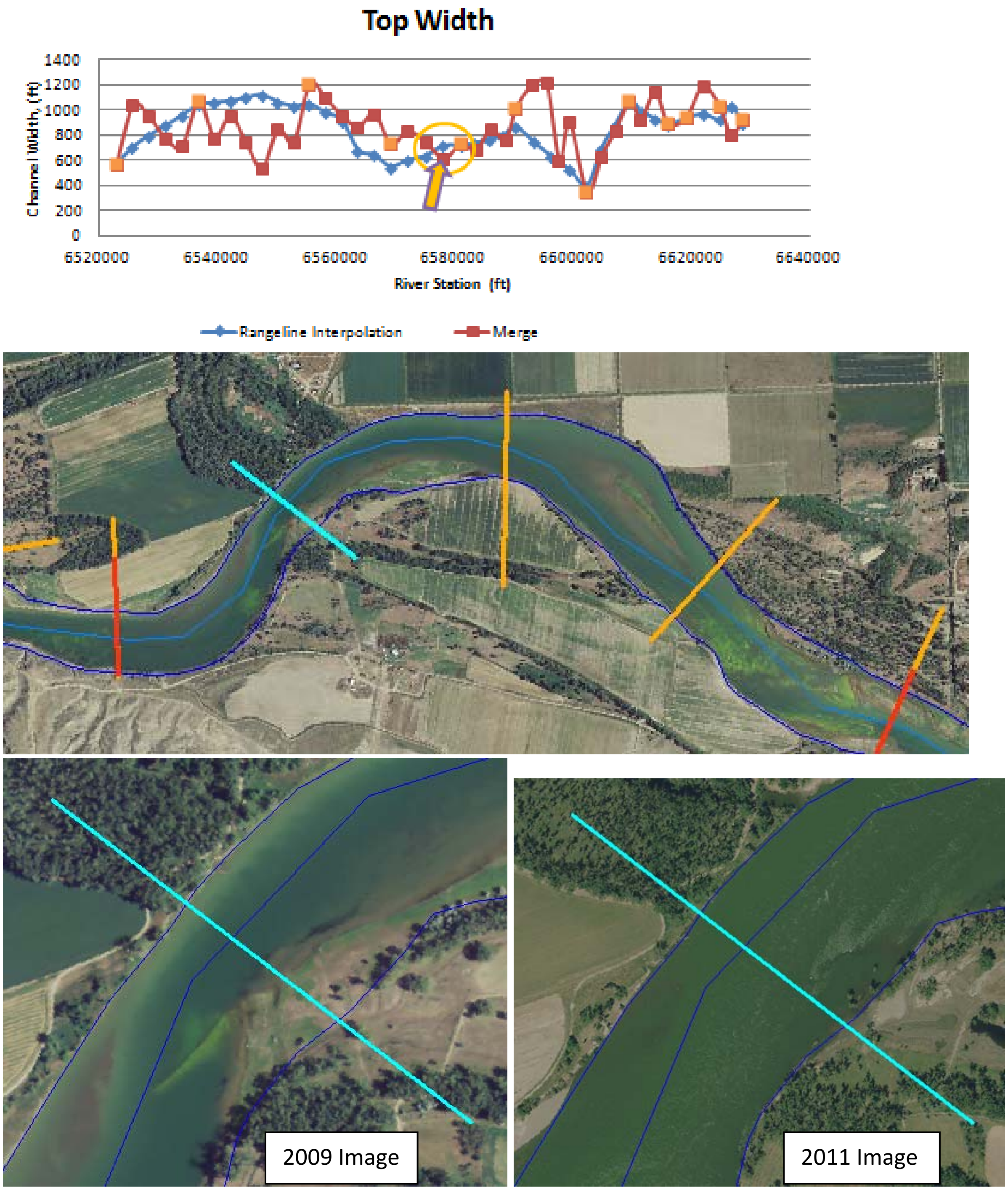


Figure 17. Aerial images of the XS, including the nearest Rangelines in the top image and closer view of the XS during a normal to low flow in 2009 and a high flow in 2011.

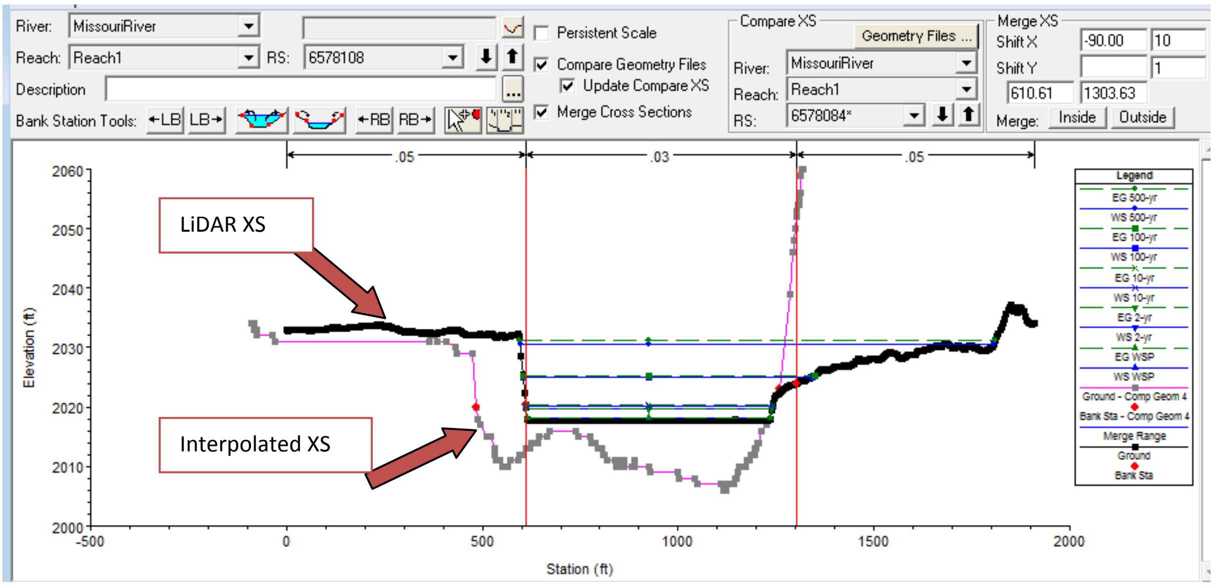


Figure 18. LiDAR XS (Black) and corresponding Interpolated XS (Grey/Pink)

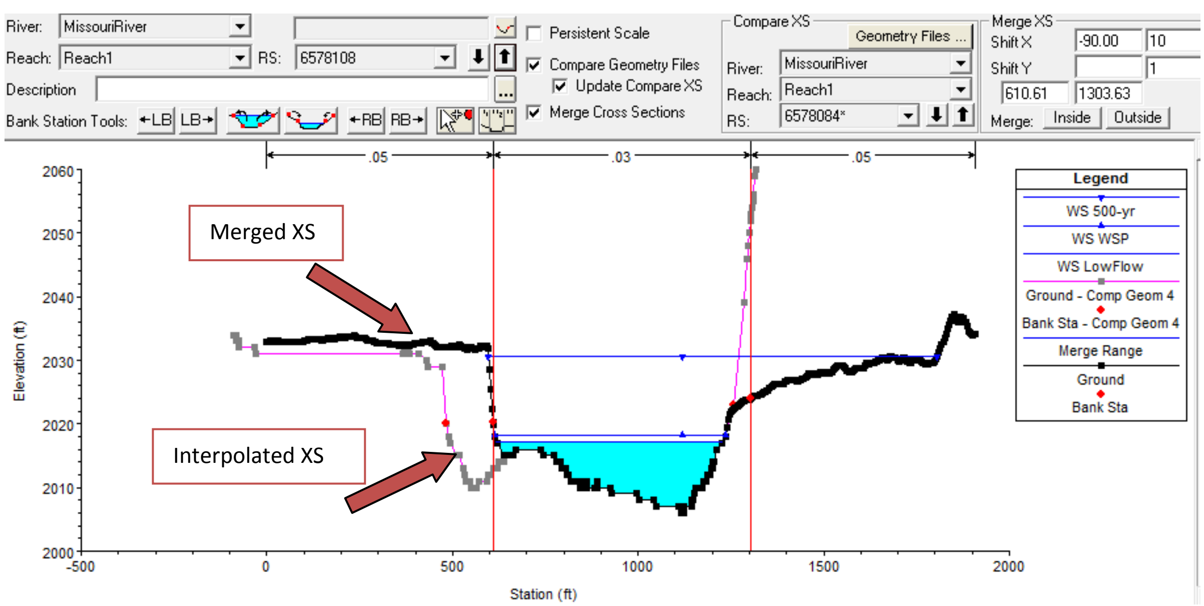
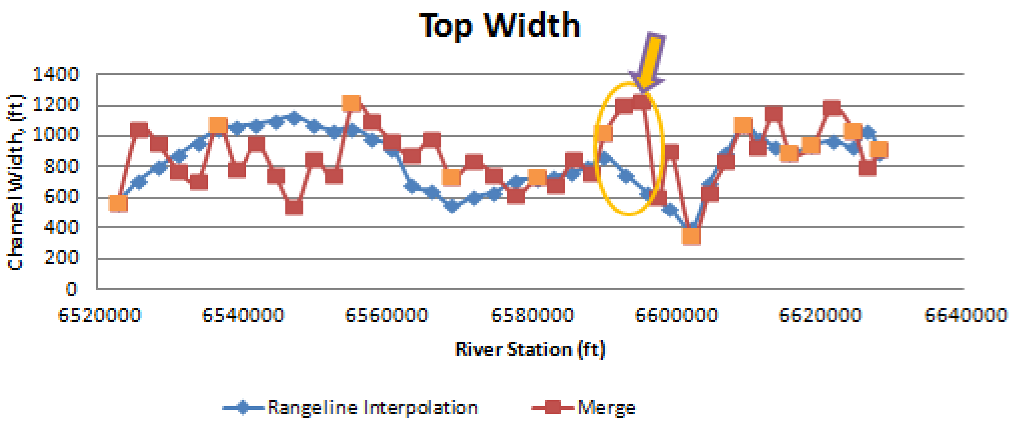


Figure 19. Merged XS (Black) and corresponding Interpolated XS (Grey/Pink)

Example 3

Used the downstream rangeline cross section with vertical adjustment to better fit cross section width



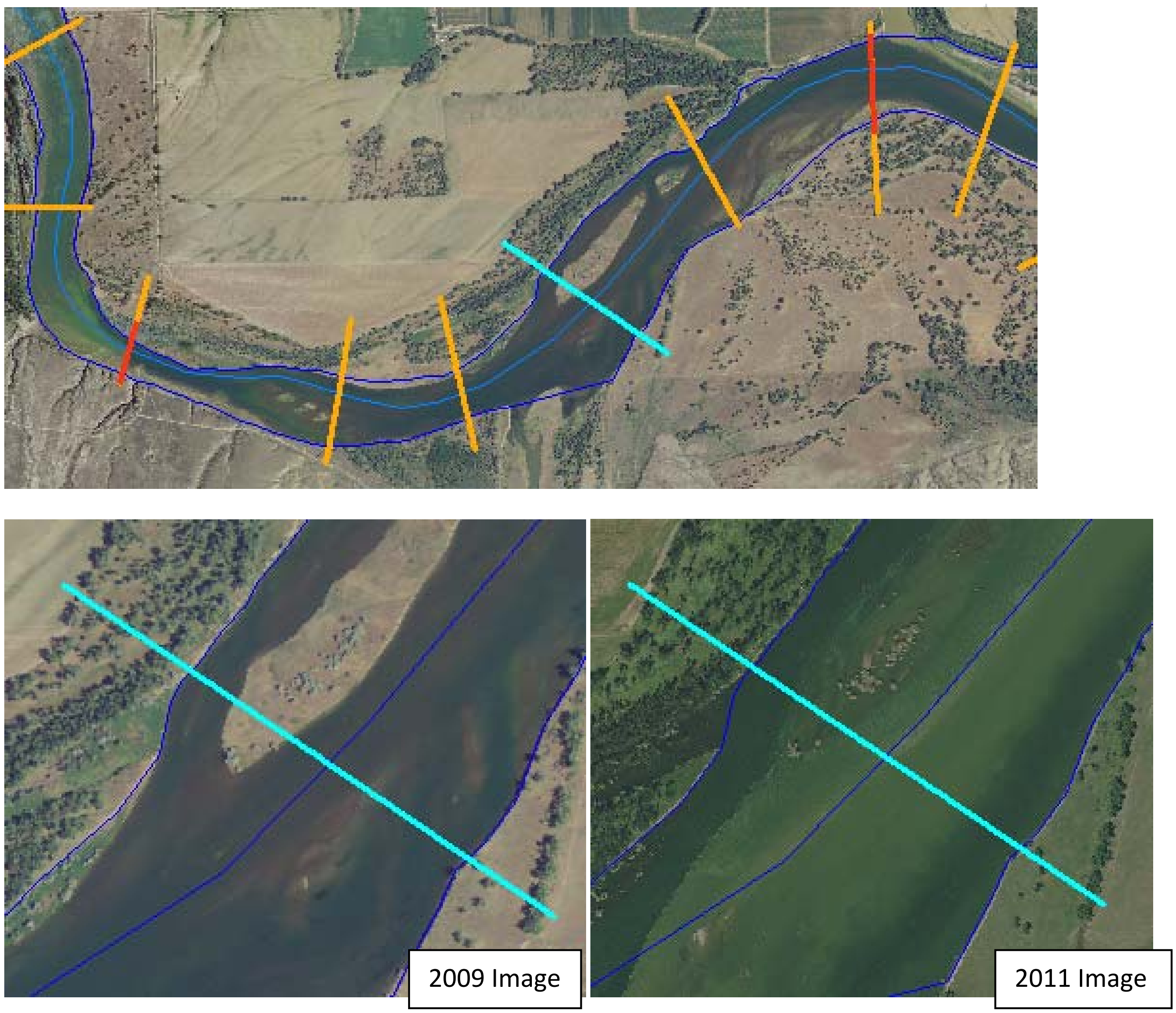


Figure 20. Aerial images of the XS, including the nearest Rangelines in the top image and closer view of the XS during a normal to low flow in 2009 and a high flow in 2011.

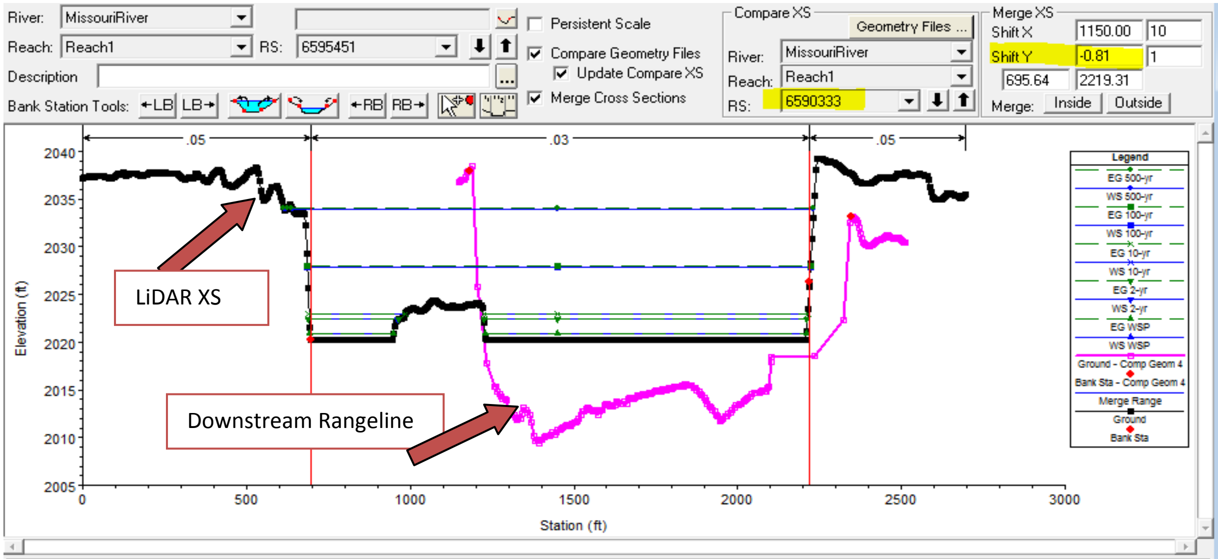


Figure 2. LiDAR XS (Black) and the next downstream Rangeline XS (Pink) vertically adjusted were used for merge due to the difference in channel width of the interpolated XS (see next figure).

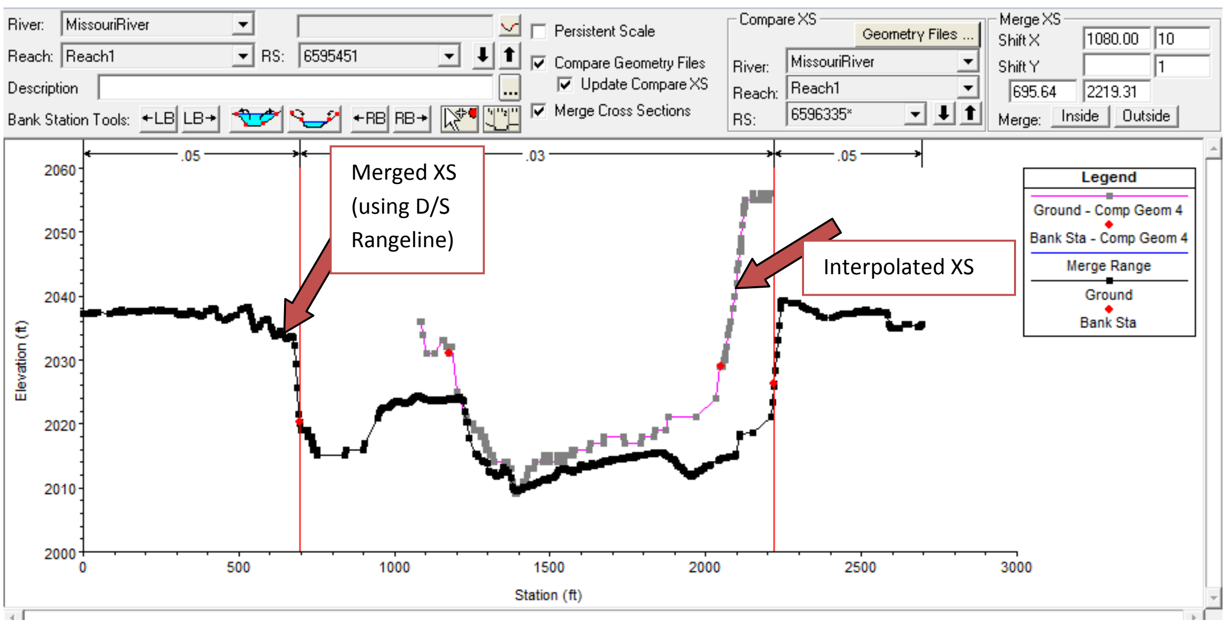
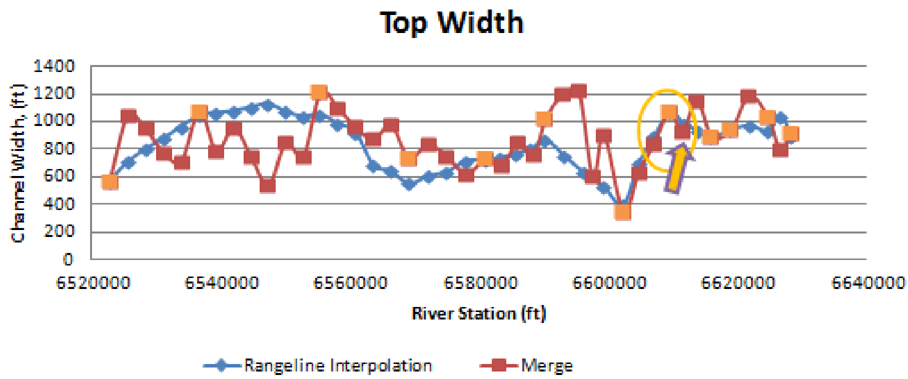


Figure 21. The final merged XS (Black) and the corresponding Interpolated XS (Pink/Grey) which was not used in this merge process.

Example 4



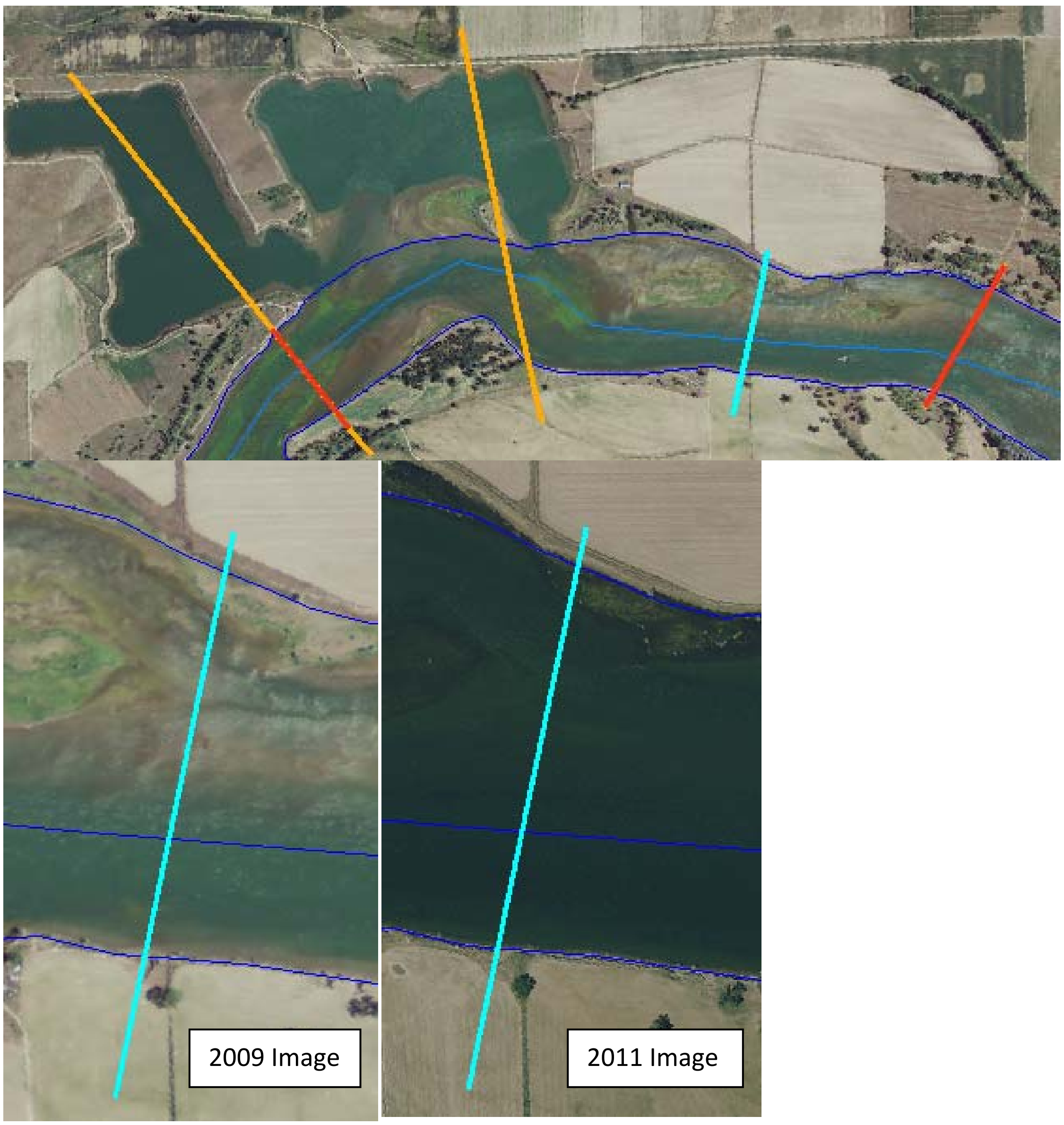


Figure 22. Aerial images of the XS, including the nearest Rangelines in the top image and closer view of the XS during a normal to low flow in 2009 and a high flow in 2011.

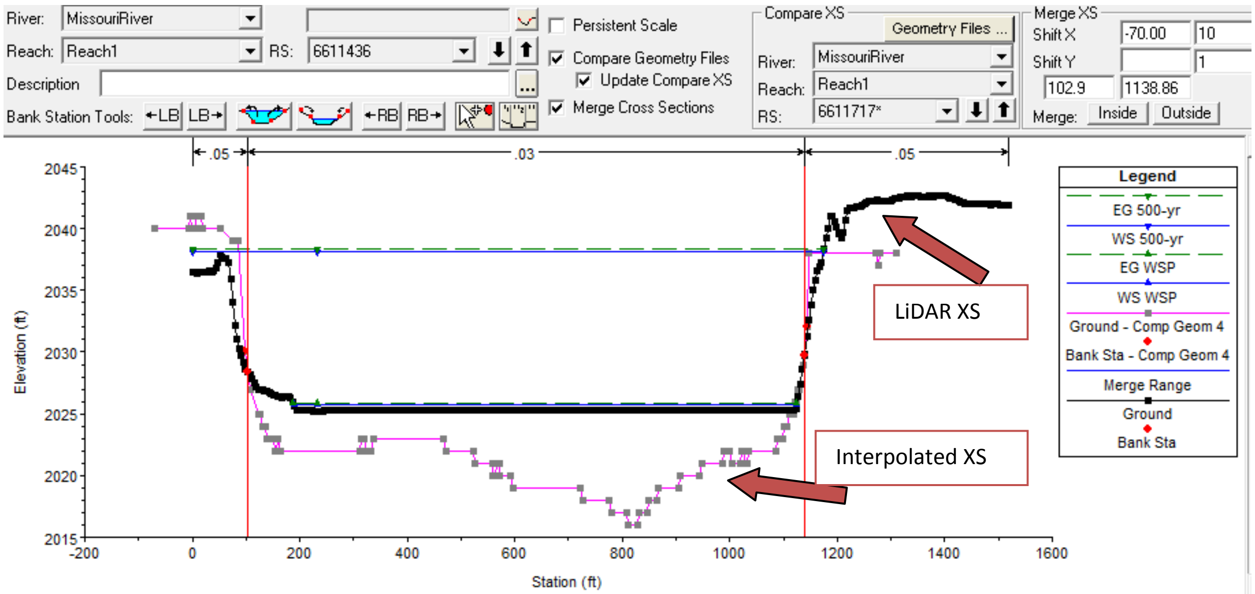


Figure 23. LiDAR XS (Black) and corresponding Interpolated XS (Grey/Pink) - XS had a good fit for channel width, note it is near a rangeline with little change between the Rangeline and XS locations.

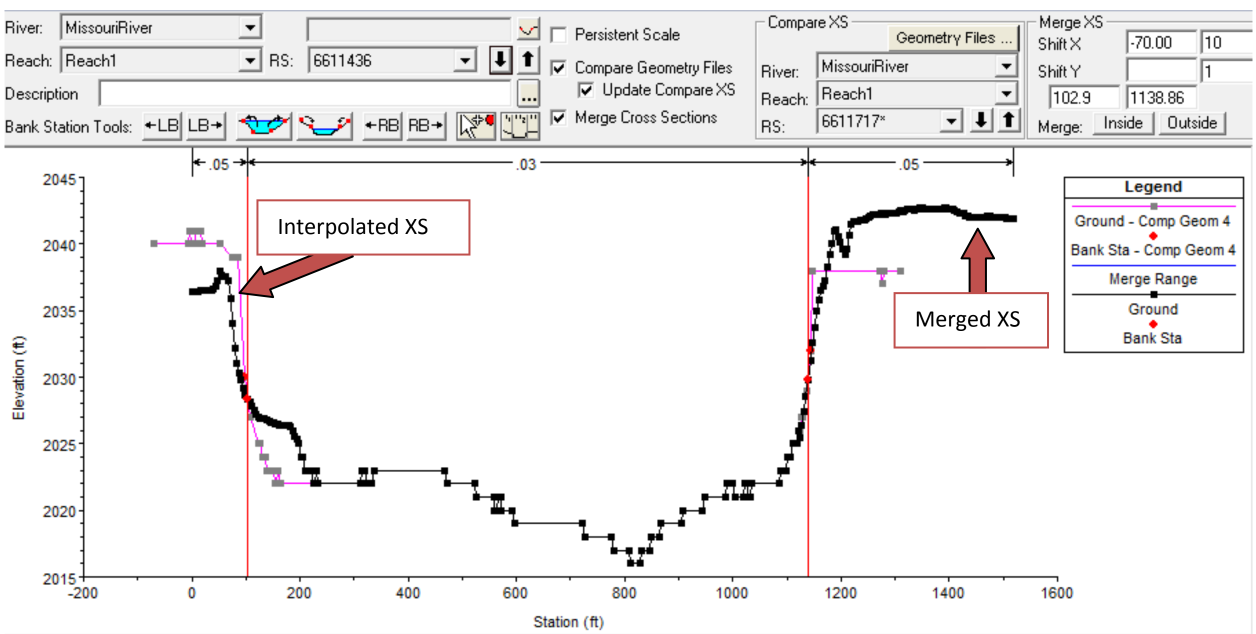


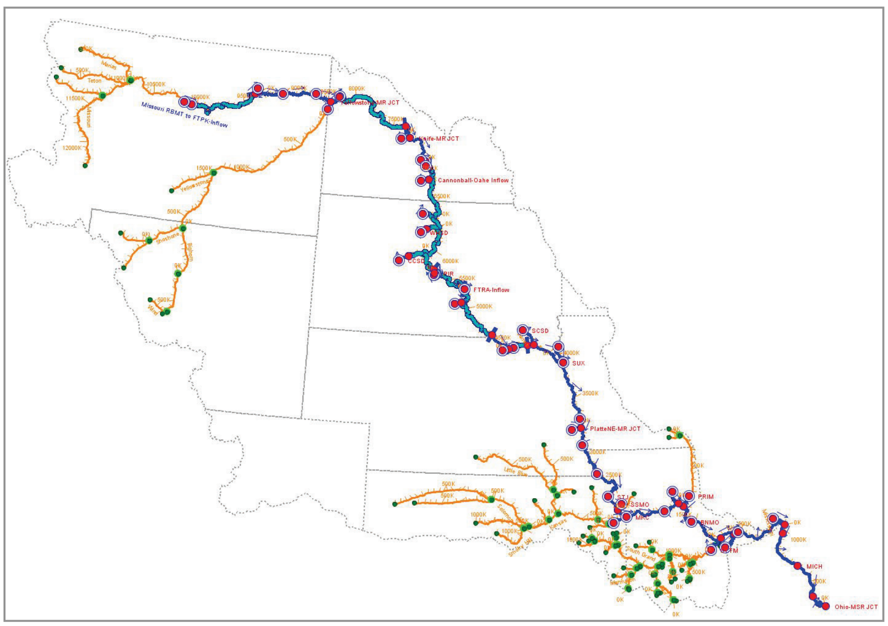
Figure 24. Merged XS (Black) and corresponding Interpolated XS (Grey/Pink).

US Army Corps of Engineers ®
Omaha District

Missouri River Unsteady HEC-RAS Model
Calibration Report

Appendix B

Garrison Dam to Oahe Dam



July 2018

FINAL

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ATTACHMENTS

ACRONYMS

CFS………………….. Cubic Feet per Second

DEM…………………. Digital Elevation Model

DTM .... Digital Terrain Model

DSSVue…………….. Data Storage System (by HEC)

GIS .... Geographic Information System

HEC .... Hydrologic Engineering Center

LiDAR………………..Light Detection and Ranging

MAF………………….. Million acre-feet

NAD 1983………… ... North American Datum of 1983

NAVD 88……………. North American Vertical Datum of 1988

NGVD 29……………. National Geodetic Vertical Datum of 1929

MRBWM…………... .. Missouri River Basin Water Management Division (previously RCC)

NWK………………… Northwest Division Kansas City District

NWO………………… Northwest Division Omaha District

POR .... Period of Record

RAS........................... River Analysis System Software (by HEC)

RCC .... Reservoir Control Center

ResSim……………...Reservoir Simulation Software (by HEC)

RM……………………1960 River Mile

System……………....Missouri River Mainstem System

USACE……………… United States Army Corps of Engineers

USGS .... United States Geological Survey

1 EXECUTIVE SUMMARY

The Garrison to Oahe reach of the Missouri River begins with the regulated outflow from Garrison Dam in North Dakota. The reach then extends approximately 318 miles downstream, encompassing a watershed of approximately 243,490 square miles, to just upstream of Oahe Dam on Lake Oahe, South Dakota. This reach was modeled in Hydrologic Engineering Center River Analysis System (HEC-RAS) Version 4.2 Beta and with the intent to update to Version 5.0 when it is released. The model was initially created in steady flow and then completed with unsteady modeling, and is now fully unsteady.

Inputs into the model are a flow hydrograph for Garrison Dam's release, flow hydrographs for the upstream boundaries of the major tributaries (Knife River, Square Butte Creek, Burnt Creek, Heart River, Apple Creek, Cannonball River, Beaver Creek, Oak Creek, Grand River, Moreau River, and Cheyenne River), and a stage hydrograph for the Oahe Pool (Lake Oahe). Output includes stage and flow hydrographs, as well as a number of additional calculated parameters such as average velocities, flow depth, etc. The latest version of HEC-RAS also has the ability to create inundation depth grids at various time-steps using RAS Mapper that can be exported for use in ecological and economic models.

The geometry was constructed using the most recent sediment range surveys from the Omaha District, which included topographic and hydrographic data. Additional cross sections were added between the sediment ranges using LiDAR data for the overbanks and interpolation of the sediment ranges for the bathymetry where hydrographic data was unavailable. The flow and stage data for the tributaries were obtained from USGS gages. The observed Garrison releases and Oahe Pool elevations were obtained from the Omaha District CWMS database.

The model reach includes a substantial degradation reach that extends downstream from Garrison Dam and a large aggradation zone in the headwaters of Lake Oahe. The extreme 2011 flow event significantly altered the river stage-flow relationship and model calibration to observed stages in flood years prior to 2011 is not valid in most areas. Therefore, due to impacts from the 2011 flood and long term changes within the aggradation and degradation areas, the hydraulic model is not capable of reproducing observed stage-flow relationships prior to 2011.

The model was calibrated to the measured 2011 and 2012 Water Surface Profiles (WSP) and observed stage gage data for the Missouri River using ungaged flows in HEC-RAS. The computed water surface profile was within +/-1 ft along the entire reach and in the range of +/-0.5 ft for about 50% to 75% of the reach. These were determined to be acceptable calibration targets. Comparison to observed hydrographs indictated that the model performed acceptably on timing of flood peaks within most areas. Some minor calibration issues were noted with hydrograph timing in areas affected by the hourly flow peaking due to power releases from Garrison Dam.

2 INTRODUCTION

The Missouri River unsteady HEC-RAS model was created as a base model for planning studies which could then be used to simulate and analyze broad scale watershed alternatives. The objective of this HEC-RAS model is to simulate current conditions on the Missouri River, with the intention of running period of record (POR) flows to compare alternatives. Future reports will address period of record runs, this report addresses model construction and calibration. This Appendix is for the Garrison Dam to Oahe Dam reach of the Missouri River as part of the Omaha District.

3 BACKGROUND

The Garrison Dam to Oahe Dam reach is the second reach for the Omaha District's portion of the Missouri River. The model includes about 315 River Miles (RM) of the Missouri River. Three tributary routing reaches are included to route flows from the USGS gage station location to the Missouri River.

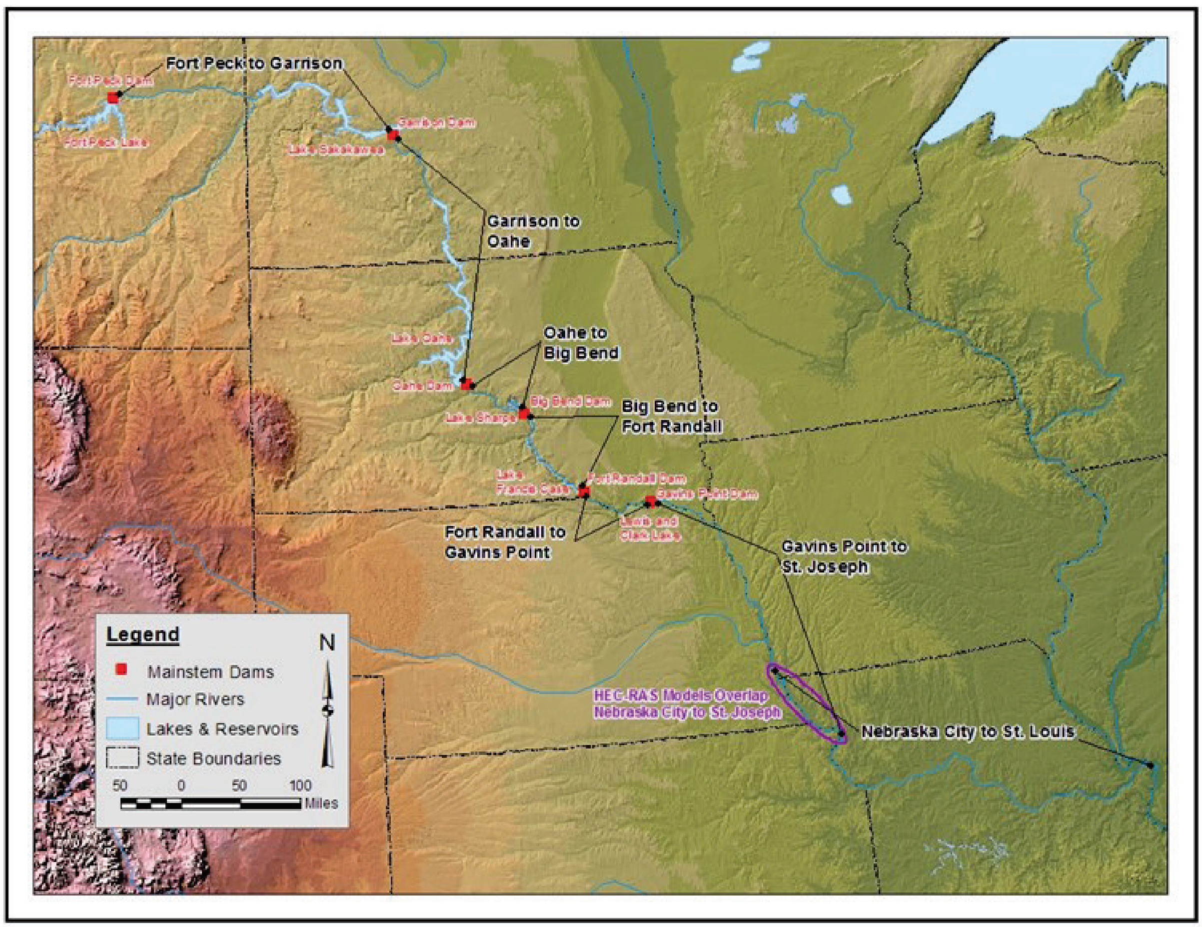

Figure 3-1: Model Extents

3.1 Model Extents

This is the second portion of the Missouri River being modeled with HEC-RAS for the Omaha District, from River Mile 1388.30, located just downstream of Garrison Dam in North Dakota, to River Mile 1073.04, located just upstream of Oahe Dam in South Dakota, as shown in Figure 3-1. Upstream of this reach, the Fort Peck to Garrison reach is being modeled (see Appendix A) and downstream of this reach, there are 2 other reaches of the Missouri River being modeled by Omaha District (see Appendices C & D) and the most downstream reach is being modeled by Kansas City District (see Appendix E).

3.2 Missouri River Mainstem System Description

The Missouri River Mainstem System (System) of dams is composed of six large earth embankments which impound a series of lakes that extend upstream for 1,257 river miles from Gavins Point Dam near Yankton, South Dakota to the head waters of Fort Peck Lake north of Lewiston, Montana. These dams were constructed by the Corps of Engineers for flood control, navigation, power production, irrigation, water supply, water quality, recreation, and fish and wildlife enhancement. Fort Peck Dam, the oldest of the six dams, was closed and began water storage in 1937. Fort Randall Dam was closed in 1952, followed by Garrison Dam in 1953, Gavins Point Dam in 1955, Oahe Dam in 1958, and Big Bend Dam in 1963. The current System of six projects first filled and began operating as a six-project System in 1967. At the top elevation of their normal operating pool level, the lakes behind these six dams provide about 1,146,000 acres of water surface area and extend a total length of 755 river miles. Only 325 miles of open river remain between the lakes, although there are 811 miles of open river downstream from Gavins Point Dam to the mouth of the Missouri River where it enters the Mississippi River at St. Louis, Missouri. The reservoirs contain an aggregate storage volume of approximately 73 million acre-feet (MAF) of which more than 16 MAF is for flood control.

Regulation of the System is according to the current Master Manual (USACE, 2006) and generally follows a repetitive annual cycle. Winter snows and spring and summer rains produce most of the year’s water supply, which results in rising pools and increasing storage accumulation. After reaching a peak reservoir level, usually during July, storage declines until late winter when the cycle begins anew. A similar pattern may be found in rates of releases from the System, with higher flows from mid-March to late November, followed by low rates of winter discharge from late November until mid-March, after which the cycle repeats.

Two primary high-risk flood seasons are the plains snowmelt season extending from late February through April and the mountain snowmelt period extending from May through July. Overlapping the two snowmelt flood seasons is the primary rainfall flood season, which includes both upper and lower basin regulation considerations.

Power generation is a component of System operation. The highest average power generation period extends from mid-April to mid-October with high peaking loads during the winter heating season (mid-December to mid-February) and the summer air conditioning season (mid-June to mid-August). The power needs during winter are supplied primarily with Fort Peck and Garrison releases and the peaking capacity of Oahe and Big Bend. During the spring and summer periods, releases are geared to navigation and flood control requirements and primary power loads are supplied using the four lower dams. During the fall when power needs diminish, Fort Randall pool is drawn down to permit generation during the winter period when the pool is refilled by Oahe and Big Bend peaking power releases. Gavins Point Dam, as the downstream-most reservoir, is operated at constant daily releases and is not used for daily power peaking.

Normally, the navigation season extends from April 1 through December 1 during which time reservoir releases are increased to meet downstream target flows in combination with downstream tributary inflows. Winter releases after the close of navigation season are much lower and vary depending on the need to conserve or evacuate system storage volumes, downstream ice conditions permitting. Minimum release restrictions and pool fluctuations for fish spawning management generally occur from April 1 through July. Endangered and threatened species, including the interior least tern and piping plover, nesting occurs from early May through August. During this period, special release patterns are made from Garrison, Fort Randall, and Gavins Point to avoid flooding nesting sites on low-lying sandbars and islands downstream from these projects.

Overall, the general regulation principles presented above provide the backbone philosophy for the Mainstem System regulation. Detailed operation plans are developed, followed, and adjusted as conditions warrant periodically as the System is monitored day-to-day. Beginning in 1953, projected operation of the Missouri River Mainstem Reservoir System for the year ahead was developed annually as a basis for advance coordination with the various interested Federal, State, and local agencies and private citizens. These regulation schedules are prepared by the Missouri River Basin Water Management Division, Northwest Division, Corps of Engineers and are reported in Annual Operating Plans (USACE, 2013b).

In addition to the six main stem projects operated by the Corps, 65 tributary reservoirs operated by the Bureau of Reclamation and the Corps provide over 15 million acre-feet of flood control storage.

Numerous reservoirs and impoundments constructed by different interests for flood control, irrigation, power production, recreation, water supply, and fish and wildlife are located throughout the basin on various tributaries. The Bureau of Reclamation and the Corps of Engineers have constructed the most significant of these structures. Although primarily constructed for irrigation and power production, the projects constructed by the Bureau of Reclamation do provide some limited flood control in the upper basin.

Table 3-1 lists pertinent data for the Missouri River Mainstem projects (USACE, 2013a).

Table 3-1: Pertinent Data for Missouri River Mainstem Projects

Description	Fort Peck	Garrison	Oahe	Big Bend	Fort Randall	Gavins Point
River Mile (1960 Mileage)	1771.5	1389.9	1072.3	987.4	880.0	811.1
Drainage Area (sq. mi.)	57,500	181,400	243,490	249,330	263,480	279,480
Incremental Drainage Area (sq. mi.)	57,500	123,900	62,090	5,840	14,150	16,000
Gross Storage (kAF)	18,463	23,451	22,983	1,798	5,293	428
Flood Storage (kAF)	3,675	5,706	4,315	177	2,293	133
Top of Dam* (ft NGVD29 (NAVD88))	2280.5 (2282.6)	1875.0 (1876.3)	1660.0 (1661.2)	1440.0 (1441.1)	1395.0 (1396.0)	1234.0 (1234.7)
Maximum Surcharge Pool** (ftNGVD29 (NAVD88))	2253.3 (2255.4)	1858.5 (1859.8)	1644.4 (1645.6)	1433.6 (1434.7)	1379.3 (1380.3)	1221.4 (1222.1)
Top of Exclusive FC Pool*** (ft NGVD29 (NAVD88))	2250.0 (2252.1)	1854.0 (1855.3)	1620.0 (1621.2)	1423.0 (1424.1)	1375.0 (1376.0)	1210.0 (1210.7)
Top of Annual FC Pool (ft NGVD29 (NAVD88))	2246.0 (2248.1)	1850.0 (1851.3)	1617.0 (1618.2)	1422.0 (1423.1)	1365.0 (1366.0)	1208.0 (1208.7)
Base of Flood Control Pool (ft NGVD29 (NAVD88))	2234.0 (2236.1)	1837.5 (1838.8)	1607.5 (1608.7)	1420.0 (1421.1)	1350.0 (1351.0)	1204.5 (1205.2)
Spillway Capacity (cfs)	275,000	827,000	304,000	390,000	633,000	584,000
Outlet Capacity (cfs)	45,000	98,000	111,000	n/a	128,000	n/a
Powerplant Capacity (cfs)	16,000	41,000	54,000	103,000	44,500	36,000
Date of Closure	Jun 1937	Apr 1953	Aug 1958	Jul 1963	Jul 1952	Jul 1955

*Operational elevations are referenced to the NGVD29 datum. They were converted to NAVD88 using CorpsCon conversion factors for use with model elevations.
**Maximum pool elevation with spillway gates opened.
***Maximum pool elevation with spillway gates closed.

3.3 Garrison and Oahe Dam and Reservoir Information

3.3.1 Garrison Dam and Lake Sakakawea

Garrison Dam is located in central North Dakota on the Missouri River at RM 1389.86, about 11 miles south of the town of Garrison, North Dakota. Construction of the project was initiated in 1946, closure was made in April 1953, and the navigation and flood control functions of the project were placed in operation in 1955. Lake Sakakawea is the largest USACE reservoir and contains almost a third of the total storage capacity of the System, nearly 24 MAF. The total drainage area of the Missouri River at Garrison Dam is 181,400 sq. miles. Table 3-2 through

Table 3-4 show the historical releases and release-duration and release-probability relationships for Garrison Dam (USACE, 2013a).

Table 3-2: Garrison Release Historical Records (1967-2011)

Month	Daily Release (cfs)		
	Maximum	Minimum	Mean
Jan	34,200	12,700	22,600
Feb	36,000	11,000	23,700
Mar	37,800	0	19,100
Apr	39,100	5,000	18,500
May	85,500	9,100	21,400
Jun	150,600	9,500	25,100
Jul	141,700	9,500	26,200
Aug	110,300	12,100	25,200
Sep	61,600	6,000	21,000
Oct	49,700	9,200	19,100
Nov	50,100	9,300	19,900
Dec	39,100	11,300	20,100
Annual	**150,600**	**0**	**21,700**

Table 3-3: Garrison Release-Duration Relationship

Percent of Time Equaled or Exceeded	Release (cfs)	
	Annual	May – Aug
Maximum	150,600	150,600
1	59,000	115,400
5	36,900	40,000
10	31,400	37,000
20	27,100	28,900
50	19,900	20,200
80	14,700	16,000
90	12,300	14,100
95	10,700	13,100
99	9,800	10,200
100	0	9,100

Table 3-4: Garrison Release-Probability Relationship

Annual Percent Chance Exceedance	Release (cfs)
50	39,000
20	42,000
10	48,000
2	72,000
1	85,000
0.2	150,000

3.3.2 Oahe Dam and Lake Oahe

Oahe Dam is located in central South Dakota on the Missouri River at RM 1072.3, about 6 miles northwest of Pierre, South Dakota. Construction of the project was initiated in September 1948, closure was made in August 1958, and deliberate accumulation of storage was begun in late 1961. Lake Oahe is the second largest USACE reservoir, with just over 23 MAF of storage capability. The total drainage area of the Missouri River at Oahe Dam is 243,490 sq. miles. The incremental drainage area between Garrison Dam and Oahe Dam is 62,090 sq. miles. Table 3-5 through Table 3-7 show the historical pool elevations and pool-duration and pool-probability relationships for Oahe Dam (USACE, 2013a).

Table 3-5: Oahe Pool Historical Records (1967-2011)

Month	Pool Elevation (ft, NGVD29 (NAVD88*))		
	Maximum	Minimum	Mean
Jan	1610.0 (1611.2)	1572.8 (1574.0)	1598.3 (1599.5)
Feb	1611.2 (1612.4)	1571.9 (1573.1)	1599.5 (1600.7)
Mar	1617.9 (1619.1)	1572.3 (1573.5)	1601.6 (1602.8)
Apr	1618.4 (1619.6)	1573.5 (1574.7)	1603.6 (1604.8)
May	1618.8 (1620.0)	1574.8 (1576.0)	1604.5 (1605.7)
Jun	1619.7 (1620.9)	1575.8 (1577.0)	1605.0 (1606.2)
Jul	1619.6 (1620.8)	1573.4 (1574.6)	1604.8 (1606.0)
Aug	1618.3 (1619.5)	1570.2 (1571.4)	1603.3 (1604.5)
Sep	1617.5 (1618.7)	1570.3 (1571.5)	1601.4 (1602.6)
Oct	1616.9 (1618.1)	1571.4 (1572.6)	1599.9 (1601.1)
Nov	1615.9 (1617.1)	1572.7 (1573.9)	1599.0 (1600.2)
Dec	1612.5 (1613.7)	1572.8 (1574.0)	1598.5 (1599.7)
Annual	**1619.7 (1620.9)**	**1570.2 (1571.4)**	**1601.6 (1602.8)**

*NGVD29 elevations were converted to NAVD88 using the conversion factor listed in Table 5-1.

Table 3-6: Oahe Pool-Duration Relationship

Percent of Time Equaled or Exceeded	Pool Elevation (ft, NGVD29 (NAVD88)*)	
	Annual	May – Aug
Maximum	1619.7 (1620.9)	1619.7 (1620.9)
1	1618.1 (1619.3)	1618.6 (1619.8)
5	1616.4 (1617.6)	1617.7 (1618.9)
10	1614.5 (1615.7)	1616.9 (1618.1)
20	1611.1 (1612.3)	1615.4 (1616.6)
50	1605.6 (1606.8)	1608.5 (1609.7)
80	1590.4 (1591.6)	1590.9 (1592.1)
90	1582.2 (1583.4)	1585.2 (1586.4)
95	1576.4 (1577.6)	1577.3 (1578.5)
99	1572.5 (1573.7)	1573.6 (1574.8)
100	1570.2 (1571.4)	1570.2 (1571.4)

*NGVD29 elevations were converted to NAVD88 using the conversion factor listed in Table 5-1.

Table 3-7: Oahe Pool-Probability Relationship

Annual Percent Chance Exceedance	Pool Elevation (ft, NGVD29 (NAVD88)*)
50	1613.0 (1614.2)
20	1617.0 (1618.2)
10	1618.1 (1619.3)
2	1619.5 (1620.7)
1	1620.0 (1621.2)
0.2	1621.0** (1622.2)

*NGVD29 elevations were converted to NAVD88 using the conversion factor listed in **Error! Reference source not found.**.

** Extrapolated: Max observed is 1854.8 ft NGVD29.

3.3.3 Survey History

Degradation and aggradation surveys are an integral part of the Omaha District's sediment data collection program. The survey work requires the periodic resurvey of the land surface and riverbed cross sections between permanently established survey control points called sediment ranges. There are 45 sediment range lines spaced an average of 1.4 miles apart below Garrison Dam. There are 88 main stem sediment ranges spaced an average of 3.0 miles apart at Lake Oahe. Table 3-8 below provides a summary of the Garrison degradation and Oahe aggradation reaches. The break between survey ranges between the degradation and the aggradation reach is not representative of where degradation/aggradation is occurring but the point where the maximum pool elevation of Lake Oahe intersects the Missouri River thalweg profile.

Table 3-8: Sediment Range Information

Garrison Degradation Reach – Garrison Dam to Lake Oahe					
Garrison Dam River Mile (1960 RM)	Ending River Mile (1960 RM)	Reach Length (miles)	No. of Main Stem Sediment Ranges	Average Spacing of Ranges (miles)	Most Recent Survey Year
1389.86	1326.69	63.17	45	1.4	2012
Oahe Aggradation Reach – Lake Oahe					
Beginning River Mile (1960 RM)	Oahe Dam River Mile (1960 RM)	Reach Length (miles)	No. of Main Stem Sediment Ranges	Average Spacing of Ranges (miles)	Most Recent Survey Year
1334.37	1072.30	262.07	88	3.0	2007/ 2010/ 2012

3.4 REACH CHARACTERISTICS

The upstream end of the reach begins immediately downstream of Garrison Dam. The reach then extends approximately 318 miles downstream, encompassing a watershed of approximately 243,490 square miles, to just upstream of Oahe Dam on Lake Oahe, near Pierre, South Dakota, as shown in Plate 1.

This reach of the Missouri River flows through mostly agricultural land and sparsely populated areas. Bismarck and Mandan, North Dakota are the largest cities located near the Missouri River in this reach.

In addition to the modeling of the Missouri River, there are three tributaries modeled in HEC-RAS: 1) The Knife River, extending approximately 26 miles from the confluence with the Missouri River to Hazen, North Dakota. The Knife River watershed is approximately 2,500 square miles; 2) The Heart River, extending approximately 11 miles from the confluence with the Missouri River to near Mandan, North Dakota. The Heart River watershed is approximately 3,300 square miles; 3) The Cannonball River, extending approximately 30 miles from the confluence with the Missouri River to Breien, North Dakota. The Cannonball River watershed is approximately 4,300 square miles.

3.5 DEGRADATION AND AGGRADATION TRENDS

During the development of the Missouri River basin projects, significant change has occurred in channel conveyance as a result of aggradation and degradation. Missouri River natural variability and construction including flood control projects, channel cutoffs, channel and bank stability projects have all contributed to conveyance change. The release of essentially sediment-free water through the System dams has resulted in a lowering of the tailwater elevation. Two types of sediment deposits exist in the reservoirs: those occurring generally over the reservoir bottom, mostly composed of the finer material and those occurring in a characteristic delta formation at the head of the reservoir and where tributaries enter the reservoir, which include coarser material.

3.5.1 Degradation Trends – Downstream of Garrison Dam

Degradation in the reach downstream of Garrsion Dam has been evaluated in a series of studies (USACE, 2012a, 2012b). Degradation begins at Garrison Dam and gradually decreases in magnitude in the downstream direction to approximately RM 1336 which is about 20 miles upstream of Bismarck, ND. At Garrison Dam tailwater, degradation of about 10 to 11 feet has been observed since dam closure in 1953 at normal flows of 20,000 to 30,000 cfs. The historic 2011 flood and period of sustained high flows led to degradation throughout the reach. Near the downstream end of the degradation reach, at the Price gage (RM 1338), a normal flow decrease of 2 to 4 feet has been observed from 1960 to 2010 (USACE, 2012a).

3.5.2 Aggradation Trends – Lake Oahe Headwaters

A trend of aggradation due to the Lake Oahe headwaters has been seen in the reach below RM 1337.3 and increases in the downstream direction. Most of the sediment currently entering Lake Oahe is from the tributaries. The major tributary streams contributing sediment to the lake are the Knife River, Heart River, Cannonball River, Grand River, Moreau River, and Cheyenne River. The storage capacity of Lake Oahe at the maximum pool elevation decreased approximately 488,340 acre-feet, or about 2 percent, from 1958 to 1989 (USACE, 1993).

3.6 FLOOD HISTORY

In the upper Missouri River, the largest flood prior to the construction of the System was the flood of 1952. Flooding was continuous from the Yellowstone River to the mouth due to flooding on most of the tributaries above Sioux City. The winter of 1951-52 had one of the heaviest snow covers in the upper plains with a high water content and an unusually cold winter. In late March, rapid melting of snow cover began. The Missouri River crested at Bismarck, ND on April 6, establishing a record discharge of 500,000 cfs, 152,000 cfs greater than the peak at the Garrison damsite. The large increase in peak flows resulted from severe Missouri River ice jams. Repetition of this event is not probable because this occurred when the System was not complete and ice jams would now be less severe due to the dams.

Since the System first filled in 1967, the largest flood event was in 2011. During 2011, a record amount of runoff occurred due to melting snowpack and record rainfall over portions of the upper basin. Annual runoff into the System is estimated to be 60.8 MAF. As a result of the record runoff, record releases from all of the System dams occurred: 65,000 cfs at Fort Peck, 150,000 cfs at Garrison, 160,000 cfs at Oahe, 166,000 cfs at Big Bend, 160,000 cfs at Fort Randall, and 160,000 cfs at Gavins Point. A summary of the peak flows for each water year is shown in Figure 3-2.

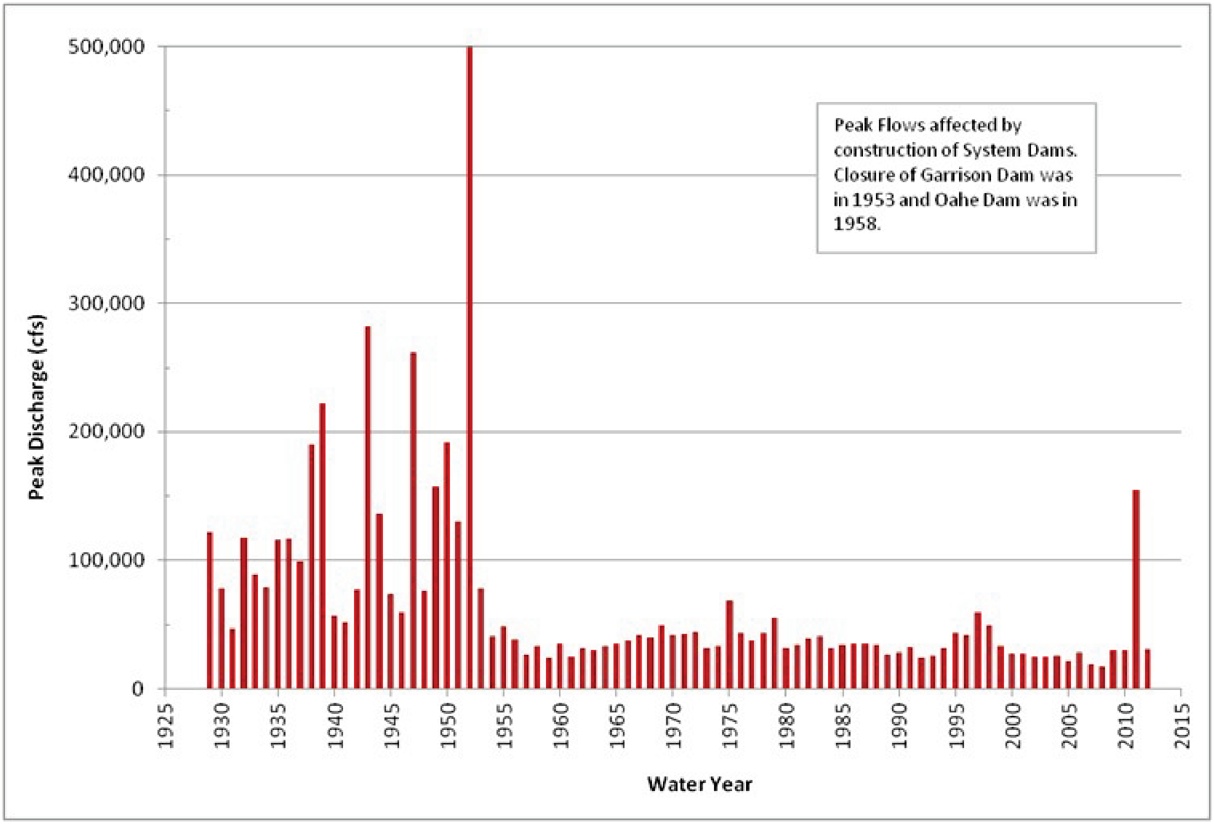

Figure 3-2: Missouri River at Bismarck, ND Annual Peak Flows

4 DATA SOURCES

Primary data sources for construction of the unsteady HEC-RAS model includes terrain data, bathymetry data, and gage data. Terrain data encompasses everything from the bluffs to the riverbanks, defining the floodplain and overbanks, but does not often include data below the surface of the river. Bathymetry captures the cross section geometry below the water surface. Gage data provides the flow boundary conditions for the model and stage calibration targets. A summary of the data used in the model is provided in Table 4-1.

Table 4-1: Summary of Data Sources

Data Type	Data Title	Location Data Applied to Model	Collection Dates
Topographic Data			
Sediment Range Survey	Garrison Degradation and Lake Oahe Sediment Range Surveys (Eisenbraun & Associates, Inc.)	RM 1388.3 – 1271.58	Aug – Oct 2012
Sediment Range Survey	Lake Oahe 2007 Sediment Range Survey (Ayres Associates)	RM 1268.01 – 1208.13	Jul – Aug 2007
Sediment Range Survey	Lake Oahe 2010 Sediment Range Survey (Eisenbraun & Associates, Inc.)	RM 1205.32 – 1198.96	12 Jul – 23 Sep 2010
Sediment Range Survey	Lake Oahe 2012 Sediment Range Survey (In-house)	RM 1195.22 – 1073.04	29 – 31 Oct 2012
DEM – LiDAR	Garrison to Bismarck LiDAR Mapping	Garrison Dam - RM 1292.58	1 Dec 2011 – 21 Mar 2012
DEM – 4 m	NEXTMap	Some Overbank: Garrison Dam – RM 1363.43, All: RM 1291.53 – Oahe Dam	May – Oct 2007
Land Cover			
Land Cover	National Land Cover Dataset 2006	All cross sections	2006
Flow Data			
Streamgage Data	Stage and Discharge	All cross sections	POR
Hydrologic Statistics	Release and Pool Duration for Garrison and Lake Oahe	All cross sections	POR
Water Surface Profile			
Water Surface Elevation Data	Missouri River Water Surface Profile from Garrison Dam to Lake Oahe	All cross sections	14 May 2012 and 21 – 22 Jun 2011

4.1 TERRAIN DEVELOPMENT

A variety of terrain sources were available for this stretch of the Missouri River and floodplain. Described below is the source, dates, and accuracy of each.

4.1.1 Sediment Range Surveys

Due to the size of the reach and funding availability, sediment range surveys for the main stem Missouri River from Garrison Dam to Oahe Dam were collected in four separate surveys: Lake Oahe in 2007 by Ayres Associates, Garrison degradation and Lake Oahe in 2010 and 2012 by Eisenbraun and Associates, Inc, and an in-house (Omaha District) Lake Oahe survey in 2012. See Table 4-1 for information on where each survey's data was used in the model. The sediment range surveys include topographic and hydrographic data. The sediment range alignments were originally set to be perpendicular to the flow of the river. Since then, the river has changed course while the sediment range alignments remain fixed. This produced cross sections that were not perpendicular to the current direction of flow in the river. For wide cross sections with dog legs, rather than skewing the entire cross section, which would shorten the entire cross section, the channel data was skewed and inserted back into the cross section.

4.1.2 DEMs and LiDAR

Two DEM data sets were available for this stretch of the Missouri River. The first was a 5-ft cell size GRID, LiDAR, Digital Elevation Model (DEM) collected from 1 Dec 2011 to 21 Mar 2012 extending from Garrison Dam to below Bismarck, ND. The horizontal and vertical accuracies are 1.25 ft RMSEr and 0.14 ft RMSEz, respectively.

Also, the NEXTMap 4-meter DEM was available that was collected from May through October 2007 by Intermap Technologies. This data set is available for the entire Omaha District. The LiDAR data did not completely cover the extents of the cross sections, so the NEXTMap data was used in the overbanks and downstream of where the LiDAR stopped, mostly in the lake. The horizontal accuracy is 2 meters RMSE or better in areas of unobstructed flat ground. The vertical accuracy is 1 meter RMSE or better in areas of unobstructed flat ground.

4.1.3 Land Cover

The United States Geographical Survey (USGS) National Land Cover Database 2006 (NLCD 2006) was used in the determination of appropriate Manning's n roughness values for overbank data. The NLCD 2006 is a 16-class land cover classification scheme at a spatial resolution of 30 meters and is based primarily on a 2006 Landsat satellite data. This is a raster digital data set (USGS, 2012).

4.2 BATHYMETRY

The bathymetry data available was a part of the sediment range survey data described in Section 4.1.1. This hydrographic data was collected in four separate surveys: Lake Oahe in 2007 by Ayres Associates, Garrison degradation and Lake Oahe in 2010 and 2012 by Eisenbraun and Associates, Inc, and an in-house (Omaha District) Lake Oahe survey in 2012.

4.3 OBSERVED DATA

Water surface profiles are surveys periodically performed by the Omaha District Corps of Engineers that provide a water surface elevation for a reach, usually collected approximately every 1 river mile. Stream stage and flow data available on the Missouri River includes gages along the Missouri River main stem, and gages on many of the major tributaries. All gages are operated by the USGS and collect stage data remotely, usually at intervals of 15 minutes. Availability and quality of these datasets influenced the configuration of the model as well as the timeframe for calibration.

4.3.1 Water Surface Profile Data

Water surface profile elevation data was collected from 21-22 June in 2011 and on 14 May 2012. Water surface elevations are collected approximately every river mile. This data was used as the baseline for calibration of the model.

4.3.2 USGS Gage Flow and Stage Data

Stream gage data was obtained through the USGS National Water Information System (NWIS) or, if not available online, from each state's USGS Water Science Center personnel for all applicable gages in this reach of the Missouri River and tributaries (USGS, 2012). Table 4-2 lists the main stem USGS gages and Table 4-3 lists the tributary USGS gages. Figure 4-1 is a map of the gage locations.

Table 4-2: USGS Missouri River Main Stem Gages

Gage Name	River Mile	Gage Number	Flow Data Dates	Stage Data Dates
Above Stanton, ND	1378.27	06339010	n/a	10/9/1976 - *
Near Stanton, ND	1372.52	06340700	n/a	10/28/1959 - *
Hensler, ND	1362.47	06340900	n/a	5/23/1959 - *
Washburn, ND	1355.07	06341000	n/a	8/20/1960 - *
Price, ND	1338.04	06342020	n/a	11/4/1959 - *
Eagle Park at Bismarck, ND[1]	1323.82	465522100 534300	n/a	5/28/2011 – 9/9/2011
Bismarck, ND	1314.65	06342500	10/1/1927 - *	1896 - *
Tavis Road at Bismarck, ND[1]	1311.59	464557100 485800	n/a	5/27/2011 – 11/2/2011
Schmidt, ND	1297.35	06349700	n/a	10/1/1966 - *

[1]Two temporary USGS gages were in operation during the 2011 event.
* - indicates that this is a current gage

Table 4-3: USGS Tributary Gages

Gage Name	Gage Number	Confluence River Mile	Modeled or Lateral Inflow	Available Flow Data Dates
Knife River at Hazen, ND	06340500	1374.50	Modeled	4/1/1929 - *
Square Butte Creek below Center, ND	06342260	1327.60	Lateral Inflow	6/1/1965 - *
Burnt Creek near Bismarck, ND	06342450	1320.90	Lateral Inflow	10/1/1967 - *
Heart River near Mandan, ND	06349000	1311.00	Modeled	4/1/1924 - *
Apple Creek near Menoken, ND	06349500	1300.50	Lateral Inflow	3/1/1905 - *
Cannonball River at Breien, ND	06354000	1269.50	Modeled	9/1/1934 - *
Beaver Creek below Linton, ND	06354580	1255.70	Lateral Inflow	10/1/1989 - *
Oak Creek near Wakpala, SD	06354882	1200.60	Lateral Inflow	10/1/1984 - *
Grand River at Little Eagle, SD	06357800	1197.80	Lateral Inflow	8/1/1958 - *
Moreau River near Whitehorse, SD	06360500	1175.50	Lateral Inflow	7/1/1954 - *
Cheyenne River near Plainview, SD	06438500	1110.70	Lateral Inflow	10/1/1950 - *

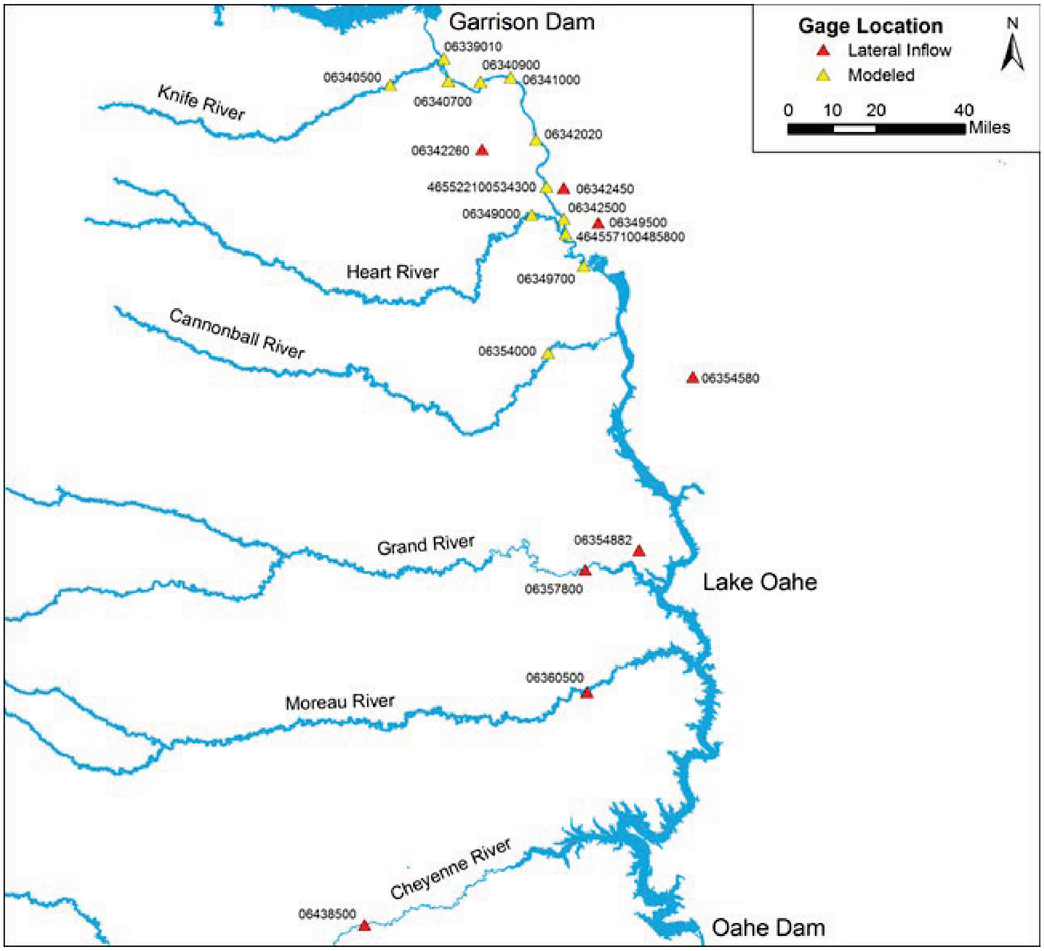

Figure 4-1: Gage Location Map

4.3.3 Garrison Dam Releases and Lake Oahe Pool Elevations

The observed Garrison releases and Lake Oahe (Oahe Pool) elevations were obtained from the Omaha District's Corps Water Management System (CWMS) database and were used as the upstream and downstream boundary conditions.

5 MODEL DEVELOPMENT

Model development includes the software version used, descriptions of the various geometry components of the model, and boundary conditions selected. The following sections outline the details of the model construction including fundamental assumptions, data sources for specific geometry features, techniques used, and justification for any unique parameters and decisions made during the process of building the model.

5.1 HEC-RAS

Unsteady computations in HEC-RAS version 4.2 Beta were used for this modeling effort. A computation interval of 3 hours was used because this was determined to be a stable time-step for the model and allowed model runs to be conducted in reasonable timeframes.

HEC-RAS has been significantly updated since version 4.1, and it is not recommended that the model be run in 4.1 or any earlier version.

HEC-RAS version 5.0 beta has been released but the model has not been tested in this version. The goal is to run the model in the newest version (not beta version), presumably version 5.0.

5.2 GEOMETRY

This section will discuss the development of the HEC-RAS model geometry for the Missouri River reach from Garrison to Oahe, including vertical datum and horizontal projection, the stream centerline and cross section geometry, the development of Manning's n-values, and the modeling of structures such as bridges and dams. The geometries of the tributaries used in the model were developed outside of this project and were added after the completion of the Missouri River geometry. The tributaries, the Knife River, Heart River, and Cannonball River, were modeled by West Consultants(WEST, 2012).

5.2.1 Vertical Datum and Projection

The vertical datum for the Garrison to Oahe unsteady HEC-RAS model is NAVD88 to match the LiDAR data. Most of the other elevation data is referenced to the NGVD29 vertical datum so a conversion factor was used to convert the data to NAVD88. See Table 5-1 for a list of vertical conversion factors used in the model. The program CorpsCon was used to obtain the conversion values based on the gage's coordinates. CorpsCon is a widely accepted standard practice for converting between NGVD29 and NAVD88 vertical datums. However, it has been found that discrepancies exist between the CorpsCon conversion values and actual re-survey of points in the NAVD88 datum.

The current horizontal projection is NAD 83 UTM 14 (US-Feet) as this is what most of the available terrain data was in. Re-projection to a nation-wide projection may be necessary after review and certification for compatibility with other HEC-RAS models and the ResSim models that are in UTM projections. Re-projecting a HEC-RAS model to a national projection is not difficult or time consuming, and there is a documented How-To procedure provided by HEC.

Table 5-1: Gage Vertical Datum Conversion Factors

Gage Number	Gage Name	Conversion Factor (from NGVD29 to NAVD88) (ft)
06339010	Missouri River above Stanton, ND	1.220
06340700	Missouri River near Stanton, ND	1.204
06340900	Missouri River near Hensler, ND	1.257
06341000	Missouri River at Washburn, ND	1.289
06342020	Missouri River at Price, ND	1.322
06342500	Missouri River at Bismarck, ND	1.342
06349700	Missouri River near Schmidt, ND	1.398
N/A	Lake Oahe (Oahe Pool)	1.240

*Conversion factor for Lake Oahe pool elevations used the location where the elevation is recorded. For this pool, that is at the intake structures.

5.2.2 Stream Centerline

One stream centerline for the Missouri River was developed in GIS for all of the Omaha District HEC-RAS models. A centerline from a previous study was modified to match the current state of the river, making sure to follow the center of mass of flow and avoiding crossing sandbars. It should be noted that the centerline does not match the 1960 river miles line. Cross sections were named based on the 1960 river miles so the reach lengths will not match up with the river miles.

5.2.3 Cross Section Geometry

The geometry of the cross sections were constructed using the most recent sediment range surveys, which included topographic and hydrographic data, in conjunction with the DEMs. The cross sections used survey data where possible and were extended as necessary with DEM data. The sediment ranges are generally spaced 1 to 3 miles apart on this stretch of the Missouri River. It was determined to have cross sections spaced no more than 3000 feet apart on the river portion of the Missouri River (for Lake Oahe the sediment range spacing was sufficient). To obtain the desired spacing, additional cross sections were added between the sediment ranges using LiDAR or DEM data for the overbank extents and for the channel data, either RAS interpolated bathymetry or channel data from a nearby range was used. Attachment 1 provides a more detailed description of how the interpolated cross section's bathymetry was estimated. Banklines for all the cross sections were set at approximately the 2-yr water surface elevation. Cross sections were named based on the 1960 river miles, since this is the primary method used to identify locations on the Missouri River. However, the 1960 river miles do not match up with the stream centerline, which produces reach lengths that do not match the river miles.

5.2.4 Manning's N-values

For the overbank areas, Manning's n values for roughness were set based on the land use classification from the NLCD 2006 data. The land cover values were condensed from the NLCD 2006 standards into 12 classes, as shown in Table 5-2. The land cover GIS shapefile was manually updated with the use of recent aerial images to reflect changes to the river channel, such as shifting sandbars, mostly due to the 2011 flood event.

Manning's n-values in the river channel were initially set to 0.025. During calibration, these were modified to between 0.022 to 0.025, which were determined to be reasonable channel roughness values for the Missouri River. Roughness values were generally changed in a reach wide manner of 10 to 30 mile long blocks. Final roughness values for the main channel are shown in Table 5-3. Manning's n-values for overbank areas were not modified in the calibration.

Table 5-2: Land Use Reclassification and Initial Roughness Values

NLCD Number	NLCD Classification	Reclass Number	Reclassification for Model	Manning's N-Value
11	Open Water	11	Water[1]	0.025
		12	Channel Sandbar	0.032
		13	Channel Sandbar Light Vegetation	0.038
		14	Channel Sandbar Heavy Vegetation	0.052
		15	Channel Bank	0.056
21	Developed, Open Space	2	Urban	0.080
22	Developed, Low Intensity			
23	Developed, Med Intensity			
24	Developed, High Intensity			
31	Barren Land (Rock/Sand/Clay)	3	Sand	0.028
41	Deciduous Forest	4	Trees	0.070
42	Evergreen Forest			
43	Mixed Forest			
51	Dwarf Scrub	5	Scrub Brush	0.060
52	Shrub/Scrub			
71	Grassland/Herbaceous	6	Grass	0.035
72	Sedge/Herbaceous			
73	Lichens			
74	Moss			
81	Pasture/Hay			
82	Cultivated Crops	7	Crops	0.045
90	Woody Wetlands	8	Wetlands	0.055
95	Emergent Wetlands			

[1]Initial roughness value that was modified during the calibration process.

Table 5-3: Final Channel Roughness Values

Cross Section River Mile Range	Channel Manning's N-Value
1388.3 – 1380.31	0.025
1379.94 – 1364.04	0.022
1363.43 – 1356.43	0.025
1355.85 – 1329.09	0.023
1328.66 – 1318.36	0.025
1317.81 – 1274.57	0.022
1271.58 – 1073.04	0.025

5.2.5 Bridges

On the Missouri River main stem, cross sections representing bridge embankments are in the model, but the structures themselves are not. This was a simplification made to keep computation times shorter. In addition, all bridge deck low chords on the Missouri River are elevated higher than the floods of record, so the only component other than the embankment that would impede water flow is the bridge columns, which likely have a local effect, but not global. This was determined to be sufficient for the Missouri River modeling. Bridges in the tributary models were left in the geometry unless they caused issues with model stability.

5.2.6 Dams

This stretch of the Missouri River was modeled just downstream of Garrison dam to just upstream of Oahe dam, so the dams themselves are not in the model. The pool of Oahe Dam, Lake Oahe, is in this HEC-RAS model and is the downstream boundary condition.

5.2.7 Tributaries

Tributary reaches were included within the model to route flow from the gage station to the Missouri River and were not calibrated to stage. Three tributary routing reaches are included in the model as previously shown in Table 4-3. Three major tributaries were modeled, the Knife River, Heart River, and Cannonball River. The tributary modeling was performed by West Consultants(WEST, 2012). In general, the goal with the tributary routing reaches was to model travel time sufficiently well from the tributary gage station to the Missouri River and preserve timing for calibration purposes. No tributary computed stage information should be used from model results without carefully assessing the purpose and considering model construction limitations.

The tributary RAS models were checked for the correct vertical datum and horizontal projection and were inserted into the Missouri River geometry with junctions. Junction lengths were assumed to represent the average distance that the water will travel from the last cross section in the reach to the first cross section of the following reach (Hydrologic Engineering Center, 2010). For junction calculations, either the energy method or force equal water surfaces method was chosen based on model stability.

5.3 Boundary Conditions

The boundary conditions are the initial flows and stages used at the upstream and downstream extents of the HEC-RAS model. Below is a discussion of those boundary conditions.

5.3.1 Upstream Boundary Conditions

Upstream boundary conditions include the outflow from Garrison Dam and observed USGS flow hydrographs at the upstream end of each of the three tributary reaches. Hourly data was used when available and daily data was used to complete the flow record. To achieve stability, a minimum flow was used for each input, as shown in Table 5-4.

Table 5-4: Minimum Flows

Location	Minimum Flow (cfs)
Garrison Outflow	2,000
Knife River	90
Heart River	50
Cannonball River	75

5.3.2 Downstream Boundary Condition

The downstream boundary condition used was the stage hydrograph for Oahe Dam's pool, Lake Oahe, from Omaha District's CWMS database.

5.3.3 Ungaged Inflow

Ungaged inflow refers to that portion of the flow that is not captured by the gage station records. Ungaged inflow computation has been automated within HEC-RAS and is fully described within the User's Manual (USACE, 2010). Ungaged calculations are made between two gages on the main stem which have a continuous record of both stage and flow.

The ungaged flow calculation is made by running the unsteady model with internal stage and flow boundaries at the gage locations mentioned above. At the endpoint, the calculated routed flow hydrograph is compared to the observed hydrograph, and the difference is calculated. The difference is put back into the model between the two gages at user specified locations with a backwards lag in time and given distribution and the model is run again. This process is repeated until the flow at the endpoint either matches the flow convergence desired or meets the maximum number of iterations specified.

Lag time was input as the approximate travel time from the lateral inflow location to the gage station. For uniform lateral inflows, the travel time from the midpoint of the segment to the gage was used. Average velocity in the Garrsion to Oahe reach of the Missouri River is about 3 ft/s, or 2 mi/hr. Simultaneous was selected as the optimization mode. The simultaneous option makes ungaged calculations for each reach independent of the others, whereas the sequential option runs calculations for each reach in order of upstream to downstream taking into account any lack in flow convergence that may have occurred in the upstream reach.

Negative flows computed as ungaged are common. This is caused by a number of reasons including gaged inflow error, model timing, areas with significant water use or groundwater recharge, and similar. Ungaged inflow hydrographs were reviewed and determined as reasonable. Calibration accuracy was improved by using the determined ungaged inflows.

Ungaged inflow parameters are entered within the unsteady flow analysis options menu. Within the HEC-RAS model, flow / stage gage records are available at Bismarck as shown in Table 4-2. Ungaged flow within each reach was distributed by prorating the remaining drainage area after the gage station drainage areas are removed. Input parameters for the ungaged inflow computation section is shown in Figure 5-1.

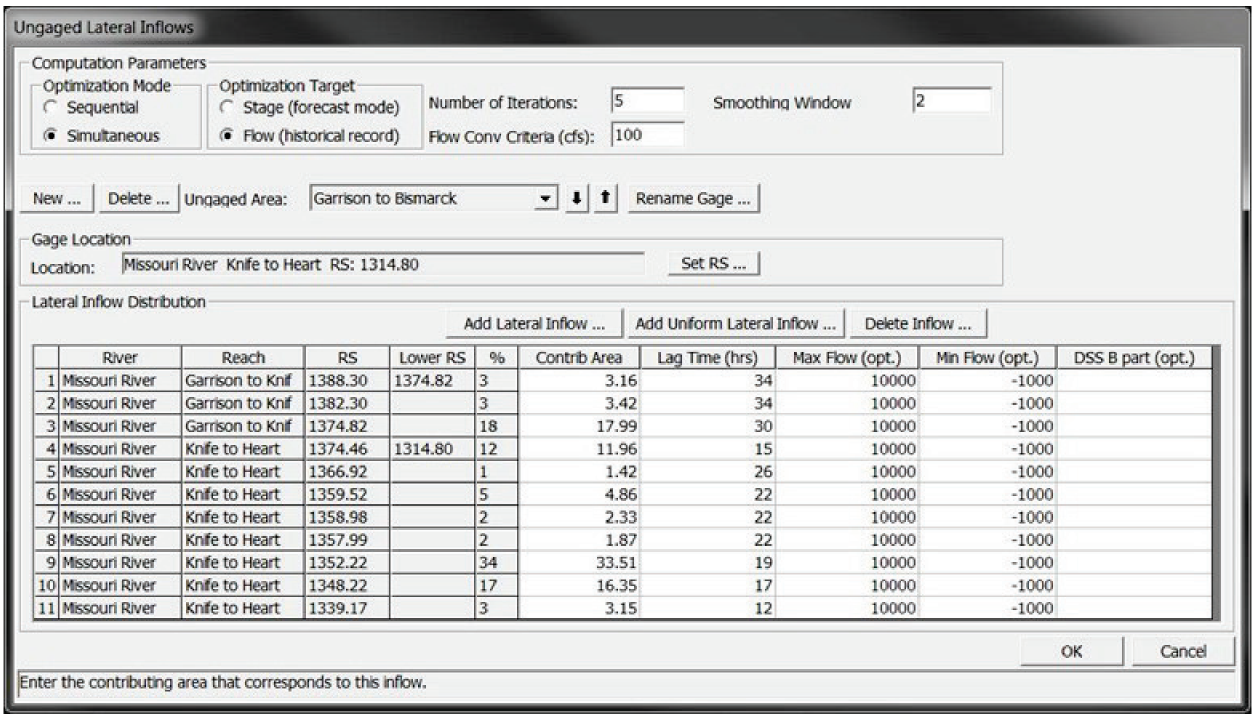

Figure 5-1: Ungaged Inflow Garrison to Bismarck

6 CALIBRATION

Model calibration was accomplished through several steps described in this section. Results as well as a discussion of level of calibration achieved and overall model performance are presented below.

6.1 MODEL CALIBRATION

Unlike previous modeling efforts on the Missouri River, a broad spectrum of flows from low flows to high flows were considered important to the project purposes. Calibration methods had to include a range of flows. The primary sources of calibration data were observed stage and flow hydrographs on the main stem Missouri River gages and field measured water surface profile data that was collected in June 2011 and May 2012.

First, the model was calibrated in a steady state for geometry. A thorough check of the estimated bathymetry was performed. At various flows, output values were checked for consistency to avoid sudden changes from one cross section to the next. The output analyzed included flow distribution (overbanks and channel), top width, velocity, energy grade, and flow area. Cross section interpolations were revised based on this analysis. The steady state model was calibrated to the water surface profiles collected in 2011 and 2012 by adjusting channel n-values. The channel n-values were initially set at 0.025 and were adjusted for steady state calibration to obtain a water surface elevation that was within a tolerance of the measured water surface profiles.

Second, the model was run in the unsteady state with steady flows to obtain a stable model. Then, one by one, tributary geometries were added into the model. The tributaries in the model were roughly calibrated and were inserted for the primary purpose of routing flows from the gage to the Missouri River for the unsteady model runs to preserve flow timing. Tributary computed stages will not be used in the analysis. Once the model was stable with all the tributaries added, the observed flows were put into the model as well as the computed ungaged flows. The model was run from January 2011 to December 2012 and results were compared to the water surface profile data for the time period it was collected and the observed stage and flow from the gages, where available. Multiple iterations were required in this process with roughness values and ineffective flow locations.

Calibration philosophy was to primarily use the base roughness values to calibrate the model for normal flows and use the HEC-RAS option for flow roughness and adjustments to ineffective flow areas to calibrate for higher flow events. As shown in Table 6-1, flow roughness factors were used to calibrate to the 2011 high flow event. In the delta region, from about RM 1285 to 1300, overbank n-values were increased during high flow calibration.

Several factors presented a challenge with the unsteady model calibration. A looped rating curve during the 2011 high flow event was difficult to calibrate to both the rising and falling limbs of the event. An attempt was made to match the stage at the peak of the event, recognizing that it would be difficult to calibrate to both the rising and falling limbs. Also, during the 20111 high flow event, advanced proctection measures were performed around the cities of Bismarck and Mandan, North Dakota, from approximately RM 1310 to 1315, on both the left and right bank. This model's geometry does not include these temporary measures and the model results reflect this with an amount of inundation that is greater than actually occured. Garrison Dam releases change throughout each day due to power peaking through the powerhouse. The releases usually range from approximately 15,000 to 30,000 cfs, although the power plant capacity is 41,000 cfs. This produces a stage difference of between 1.5 to 3 feet. Releases vary the most in the summer months. Timing in the model skews the results and may report that the model is not performing well while it is just off by a few hours.

The calibration goal was to achieve a water surface elevation within 1 ft for the entire reach and within 0.5 ft for most of the reach for both the measured water surface profiles and the observed gage data for 2011 and 2012, excluding periods of ice. The model does not account for ice. Ice causes much higher stages than would normally occur for an open water condition. Ice affected events typically occur from December to March. Plate 2 through Plate 19 are the hydrographs and computed minus observed stage vs flow plots for the gage locations. Plate 20 through Plate 28 show the computed profile vs the measured water surface profile.

Table 6-1: Flow Roughness Factors

U/S Cross Section	1388.30	1374.46	1350.32	1310.98
D/S Cross Section	1374.82	1350.80	1311.61	1271.58
Flow (cfs)	**Roughness Factor**			
0	1	1	1	1
20,000	1	1	1	1
40,000	1.05	1.1	1.05	1.2
60,000	1.05	1.2	1.1	1.2
80,000	1.05	1.2	1.1	1.2
100,000	1.05	1.3	1.2	1.3
120,000	1.05	1.2	1.1	1.25
140,000	1.06	1.1	1.1	1.15
160,000	1.07	1.1	1	1

6.2 Calibration Results

Model calibration results are within the desired range with most locations within 0.5 to 1 foot of observed stages. The results can be seen in Plate 2 through Plate 28. In general, comparison of model results to gage station hydrographs was reasonable. The measured profile calibration also provides confidence in model performance between the gage station locations. A comparison of peak stages for the 2011 flood are shown in Table 6-2.

Table 6-2: 2011 Flood Peak Stage Comparison

Location	Date	Peak Stage Difference (ft)
RM 1378.27 – above Stanton	M	M
RM 1372.52 – near Stanton	25Jun2011	-0.5
RM 1362.47 – Hensler	26Jun2011	0.1
RM 1355.07 – Washburn	27Jun2011	-0.1
RM 1338.04 – Price	26Jun – 12Jul 2011	0.6
RM 1323.82 – Eagle Park at Bismarck	26Jun – 13Jul 2011	-0.2
RM 1314.65 – Bismarck	26Jun – 12Jul 2011	-0.1
RM 1311.59 – Tavis Road at Bismarck	27Jun – 13Jul 2011	0.4
RM 1297.35 – Schmidt	24Jun – 13Jul 2011	-0.1

*M – denotes gage peak stage data is missing

*Peak stages were manually estimated due to minor timing issues and bad data points.

6.2.1 Calibration Results Affected by Ice Conditions

Ice affected conditions including ice cover, ice breakup, and ice jams occur annually within the basin. Ice formation conditions typically occur in late November to late December with iceout typically occur in the early spring, usually in the March to April time frame. No ice parameters were included in the model development or calibration. Therefore, winter condition model calibration results should be viewed with caution and recognize that results do not reflect observed conditions.

6.2.2 Stage Trend Impacts

Degradation and aggradation conditions occur through the reach due to Garrison Dam at the upstream model boundary and Oahe Dam at the downstream model boundary. Due to the extreme 2011 event flows and the high degree of channel adjustment that occurred during the event, accurate stage calibration prior to 2011 using the post-2011 event model geometry is not possible. Model results for the rising portion of the event in May and June demonstrate how stage-flow relationships changed during the flood and also reduce calibration accuracy through this portion of the event.

7 CONCLUSIONS

The model performs well for the 2011 and 2012 observed data and is calibrated to the 2011 and 2012 water surface profiles. Significant points to consider with respect to model construction and calibration are as follows:

- Measured profile and gage hydrograph calibration for both 2011 and 2012 indicates that the model performs satisfactorily with a stage calibration accuracy of less than 1 foot at most locations.
- Incomplete hydrographic surveys were available to construct the model. Interpolation from hydrographic sections was used combined with LiDAR data to generate cross sections at the desired spacing of about 2,500 to 3,000 feet. Consequently, the interpolated sections within the model have reduced accuracy for the below water portion of the cross section. Normal flow calibration indicated that the model performs satisfactorily which implies the interpolation method was reasonable.
- Floodplain model geometry in the reach below Williston is limited due to the use of less accurate DEMs.
- No tributary computed stage information should be used from model results without carefully assessing the purpose and considering model construction limitations.
- Aggradation and degradation that occurred during the 2011 event reduces calibration accuracy for the flood hydrograph. This also prevents calibrating to flow events prior to 2011.
- Ungaged inflows are an important parameter in model calibration. Computation of ungaged inflow with HEC-RAS appeared to enhance model flow accuracy compared to observed flow at the gaging stations.

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APPENDIX B

GARRISON DAM TO OAHE DAM

PLATES

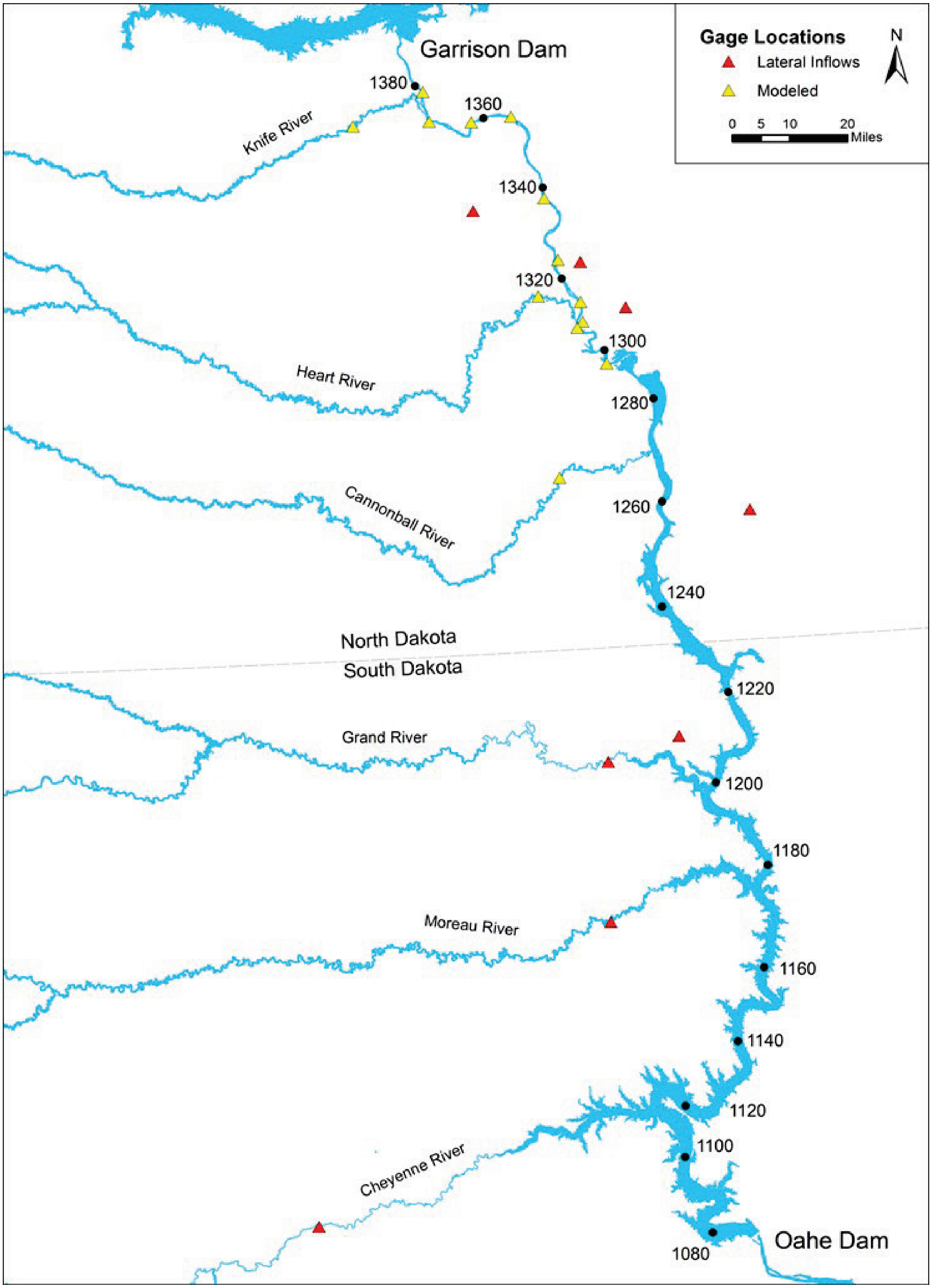


Plate 1: Overview Map

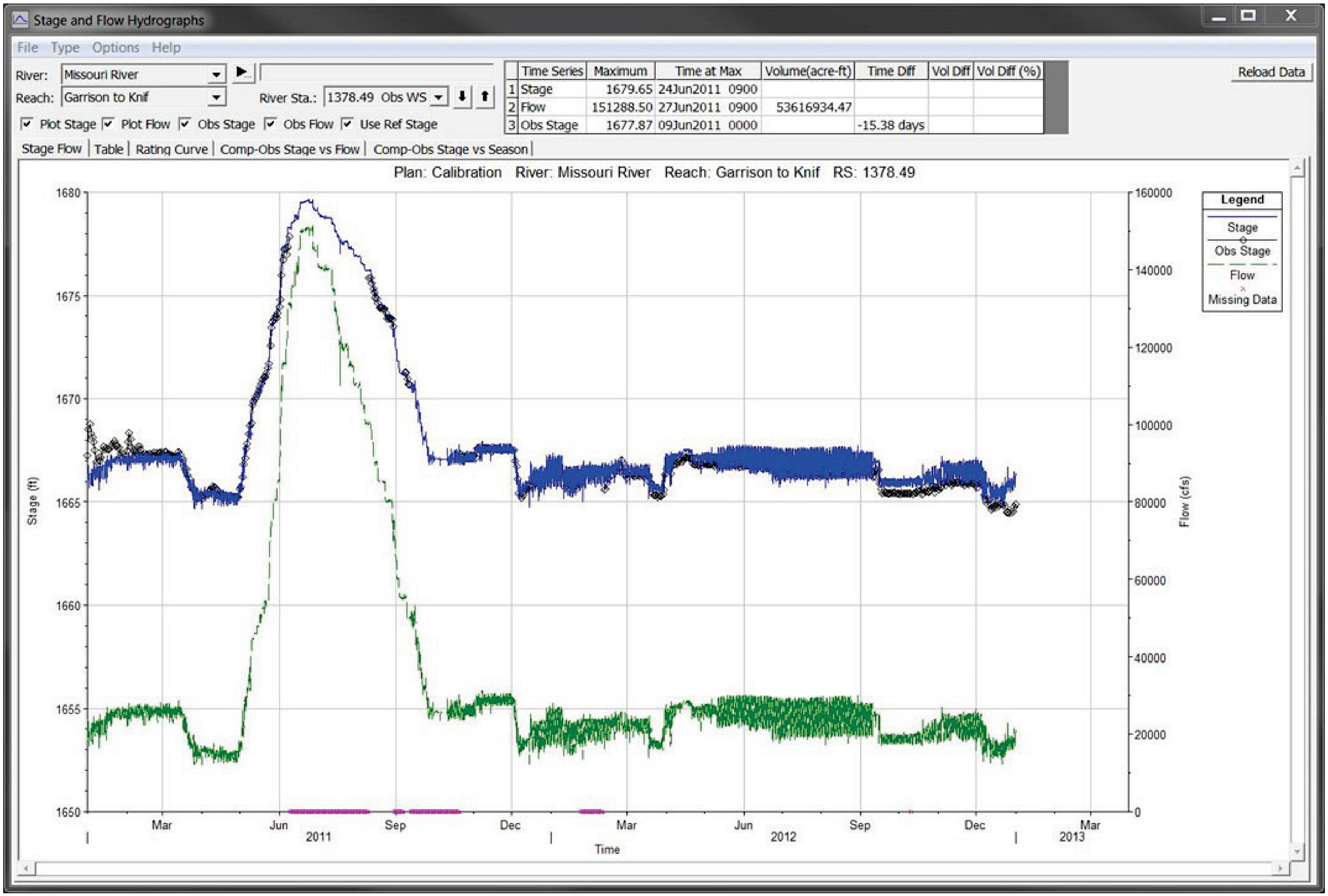


Plate 2: Missouri River above Stanton, ND Hydrograph

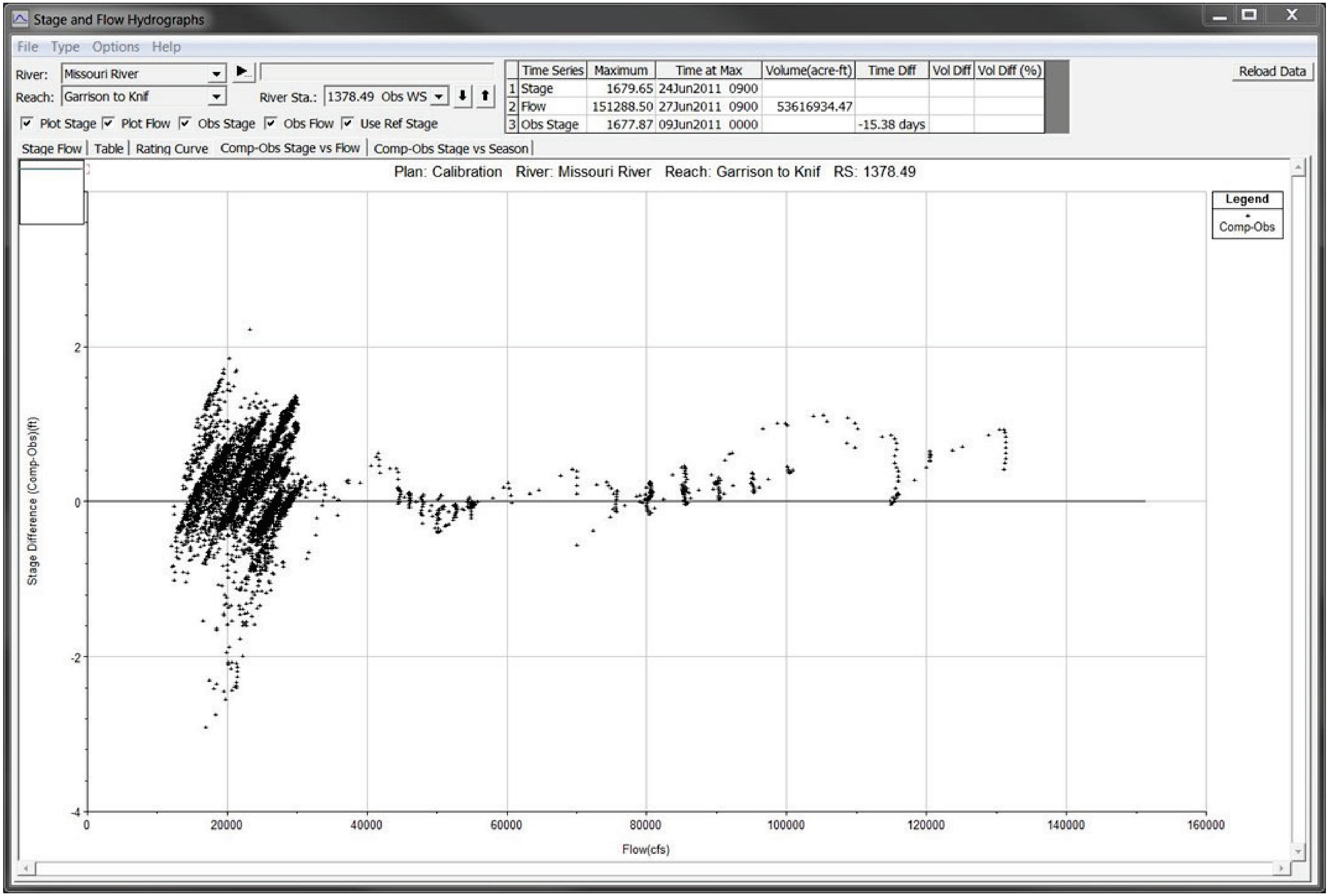


Plate 3: Missouri River above Stanton, ND Comp-Obs Stage vs Flow

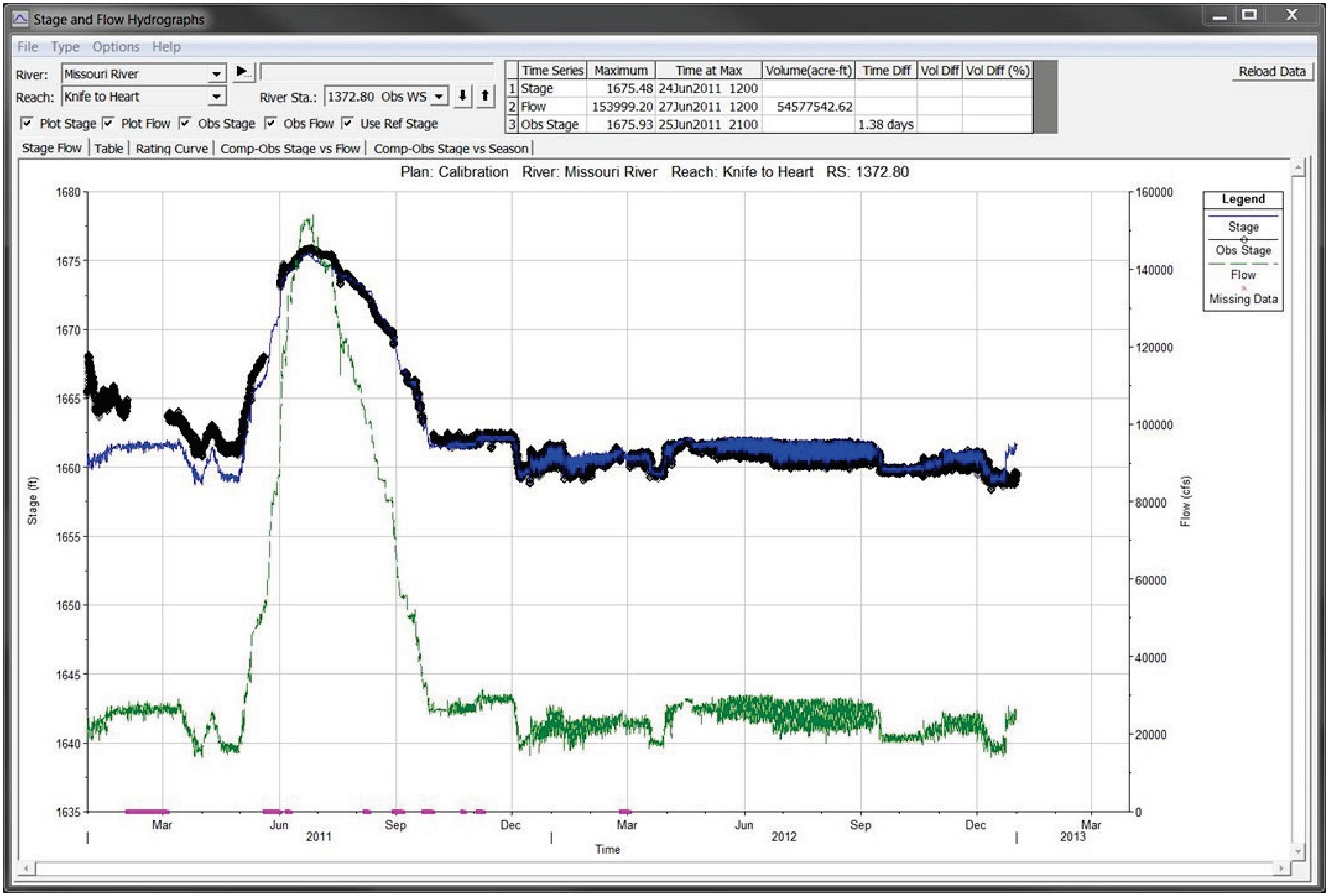


Plate 4: Missouri River near Stanton, ND Hydrograph

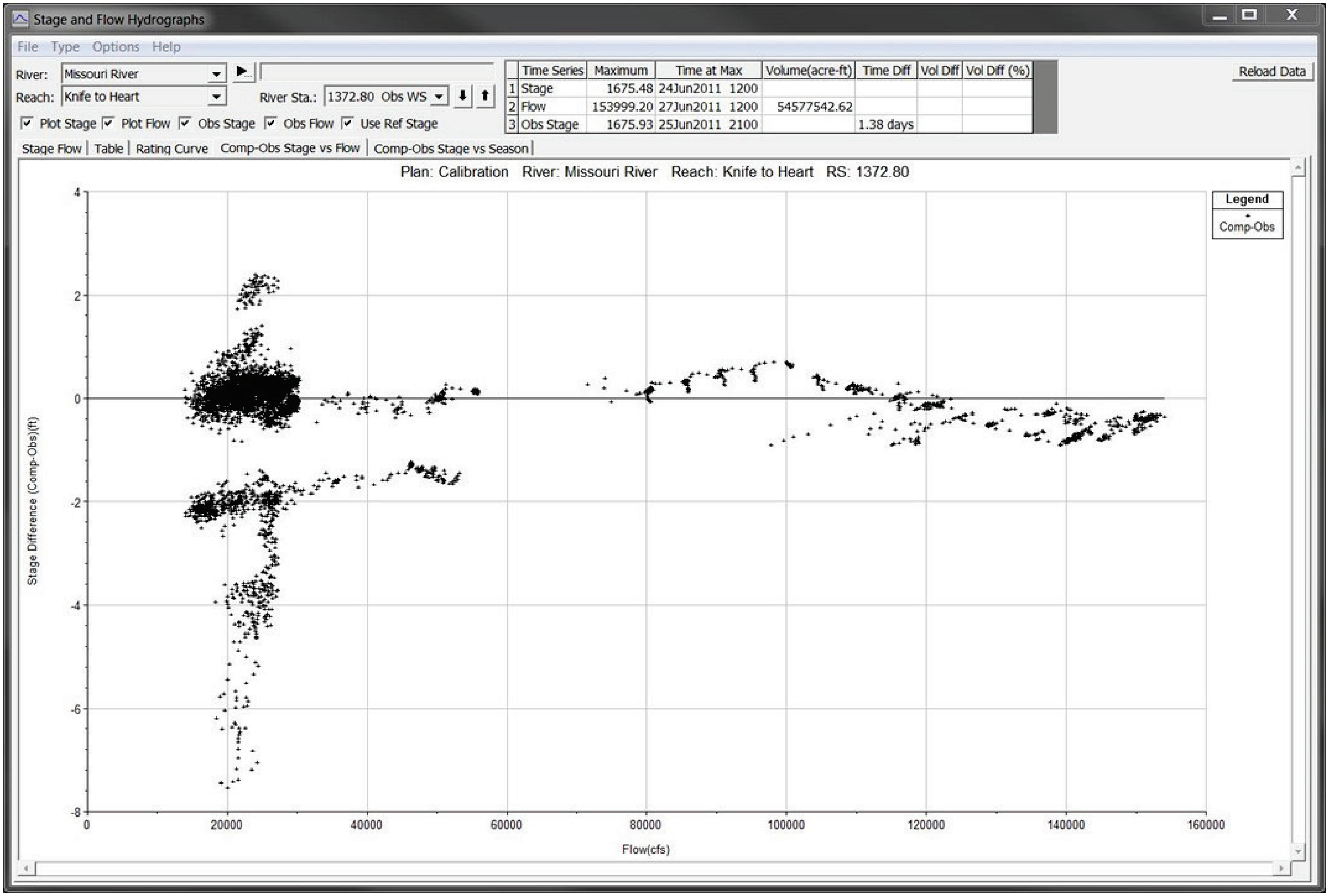
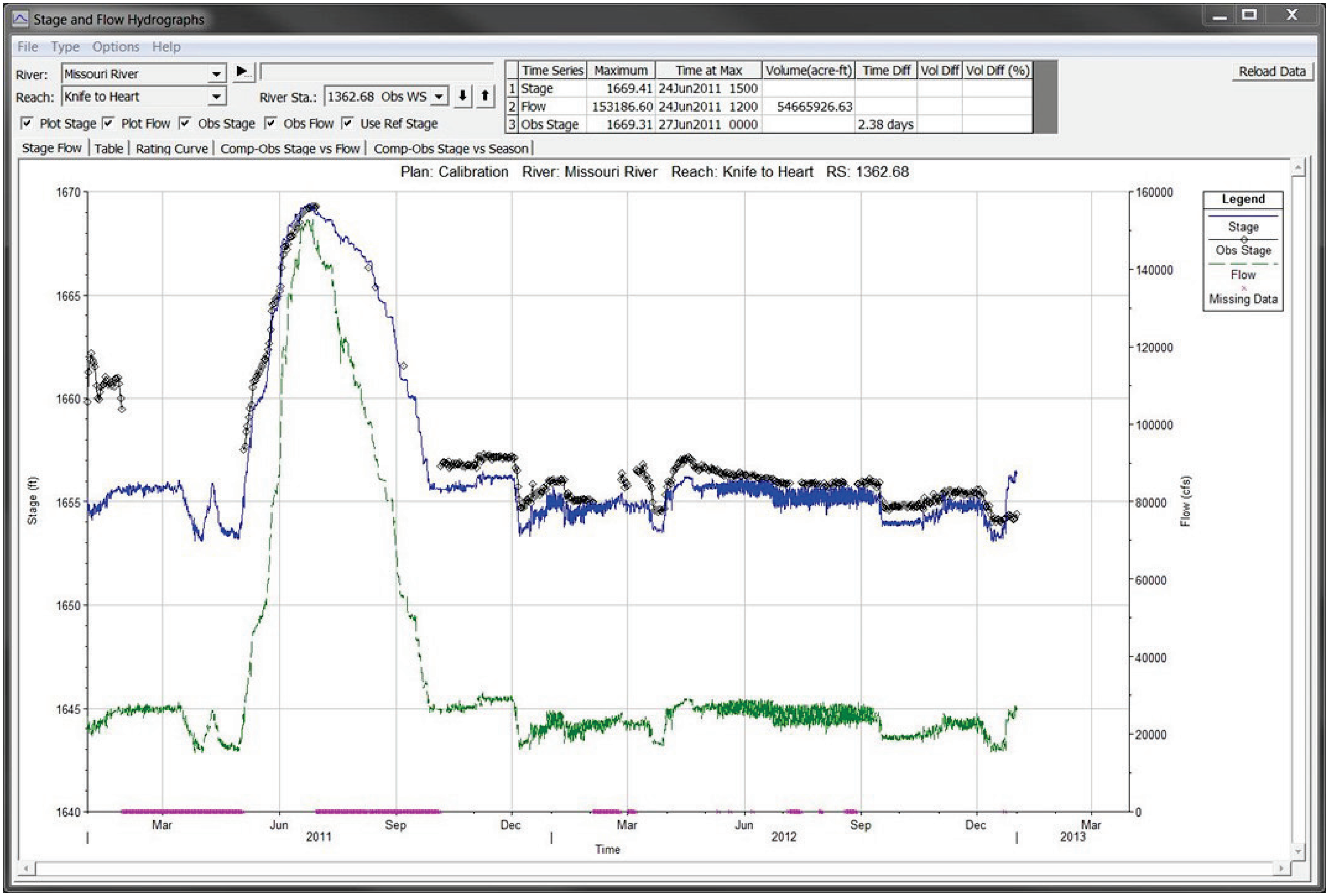


Plate 5: Missouri River near Stanton, ND Comp-Obs Stage vs Flow



	Time Series	Maximum	Time at Max	Volume(acre-ft)	Time Diff	Vol Diff	Vol Diff (%)
1	Stage	1669.41	24Jun2011 1500				
2	Flow	153186.60	24Jun2011 1200	54665926.63			
3	Obs Stage	1669.31	27Jun2011 0000		2.38 days		

Plate 6: Missouri River near Hensler, ND Hydrograph

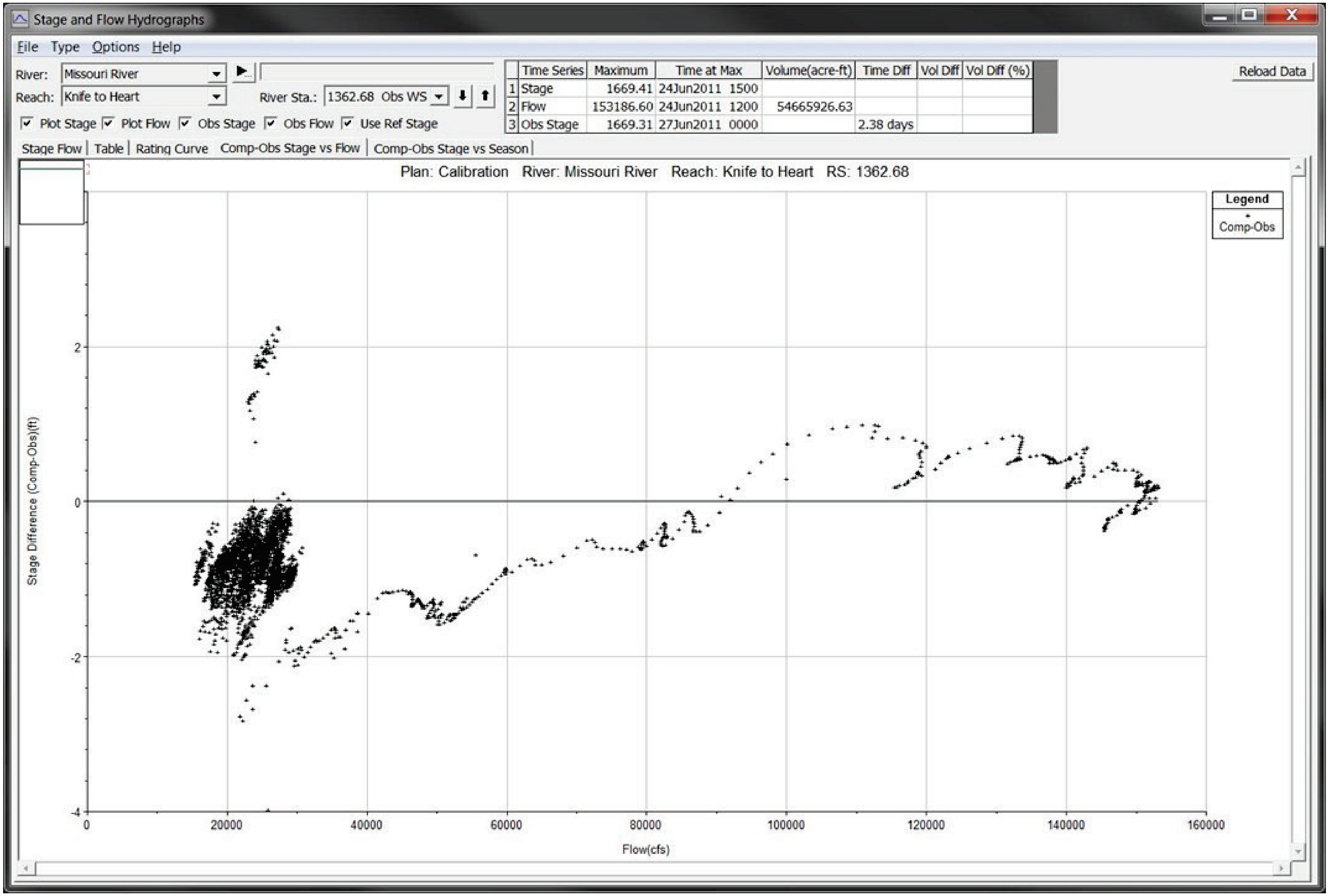


Plate 7: Missouri River near Hensler, ND Comp-Obs Stage vs Flow

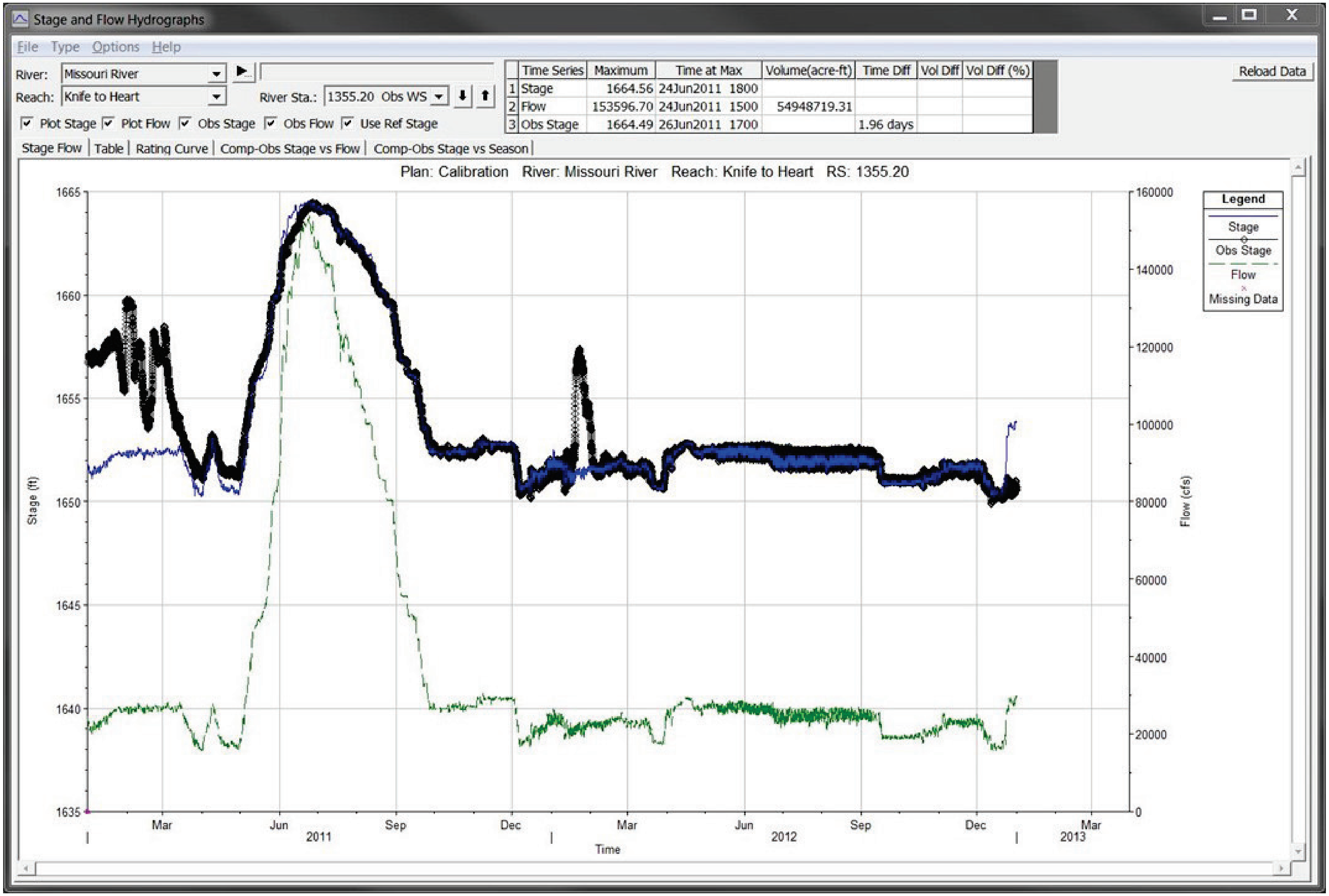


Plate 8: Missouri River at Washburn, ND Hydrograph

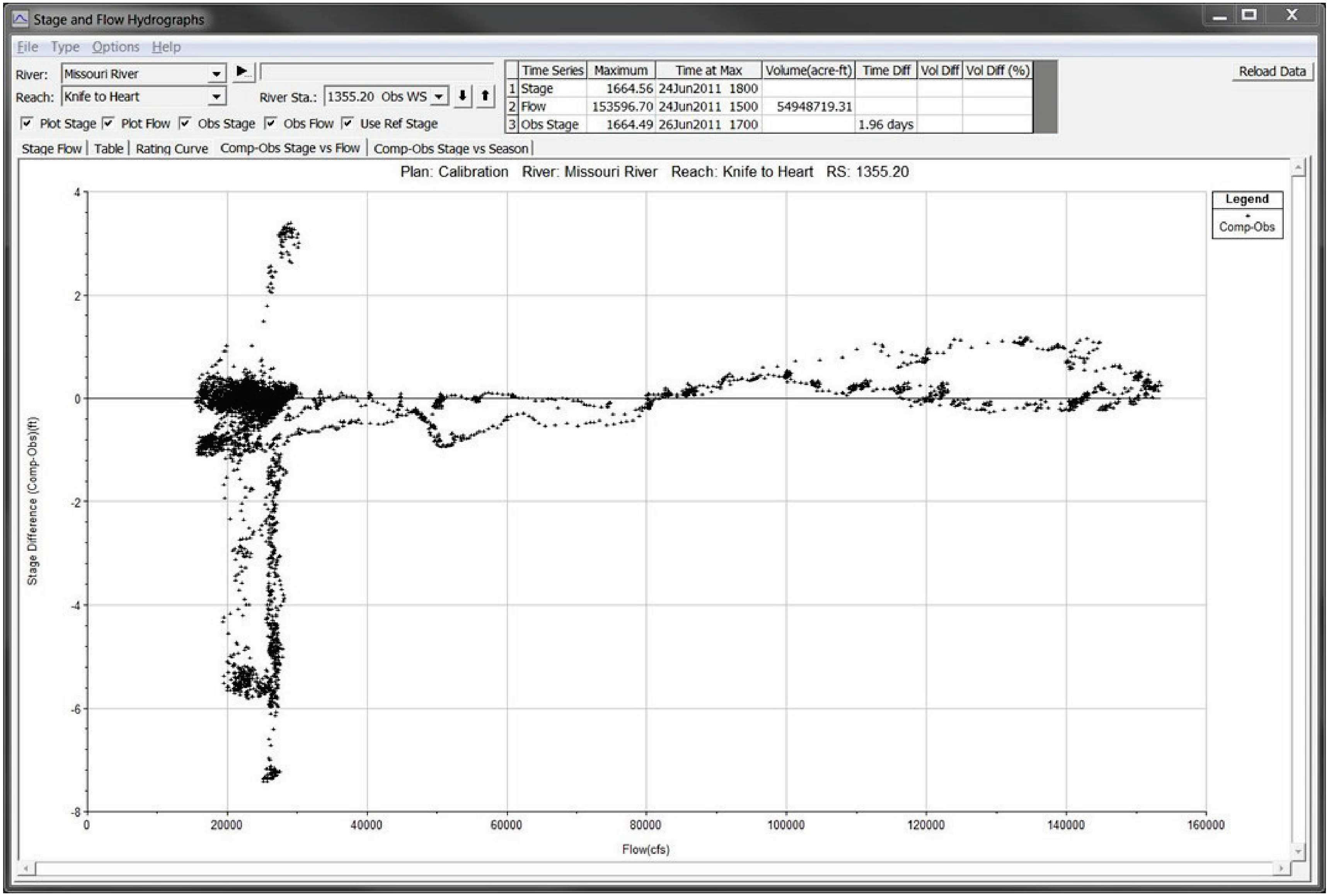


Plate 9: Missouri River at Washburn, ND Comp-Obs Stage vs Flow

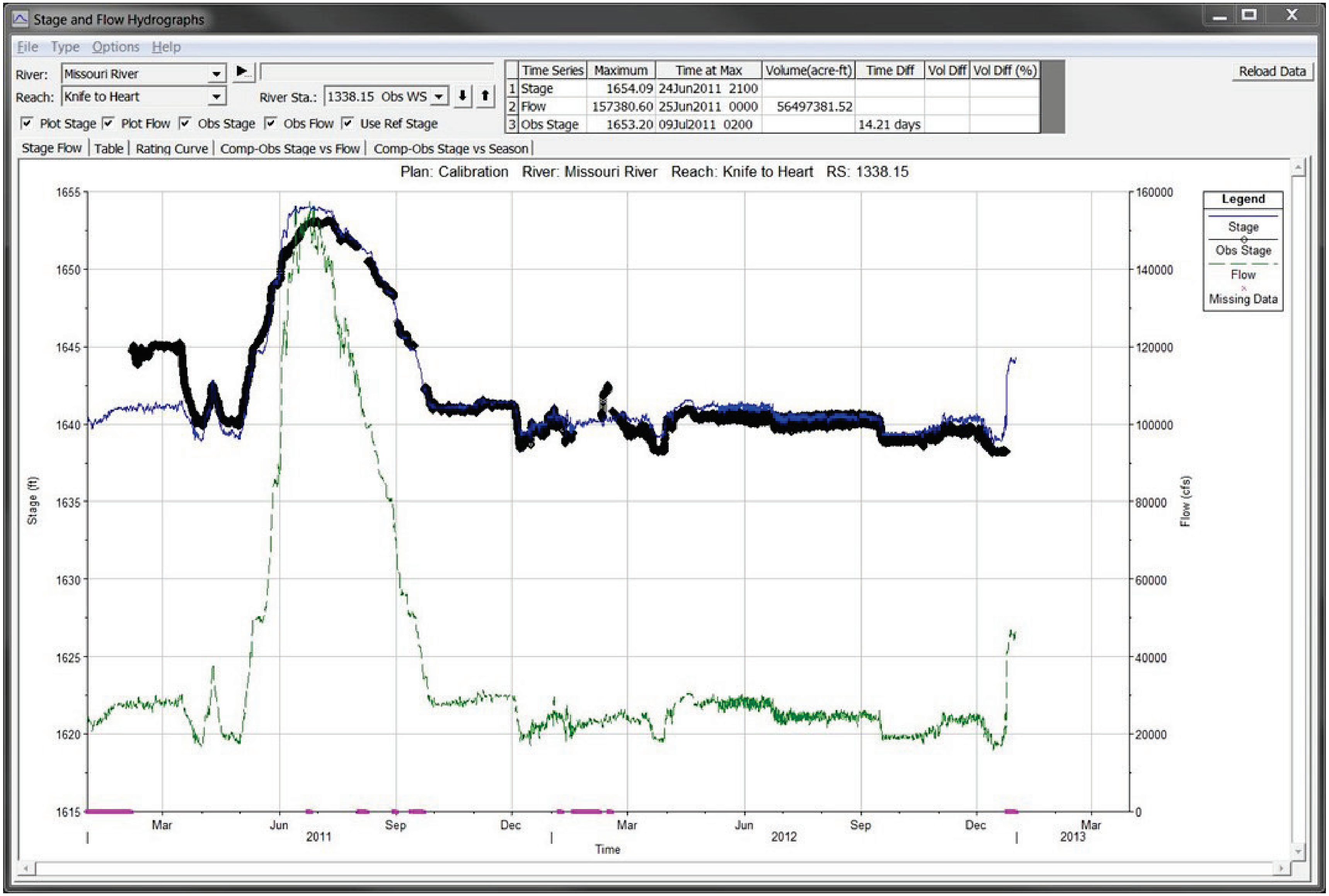


Plate 10: Missouri River at Price, ND Hydrograph

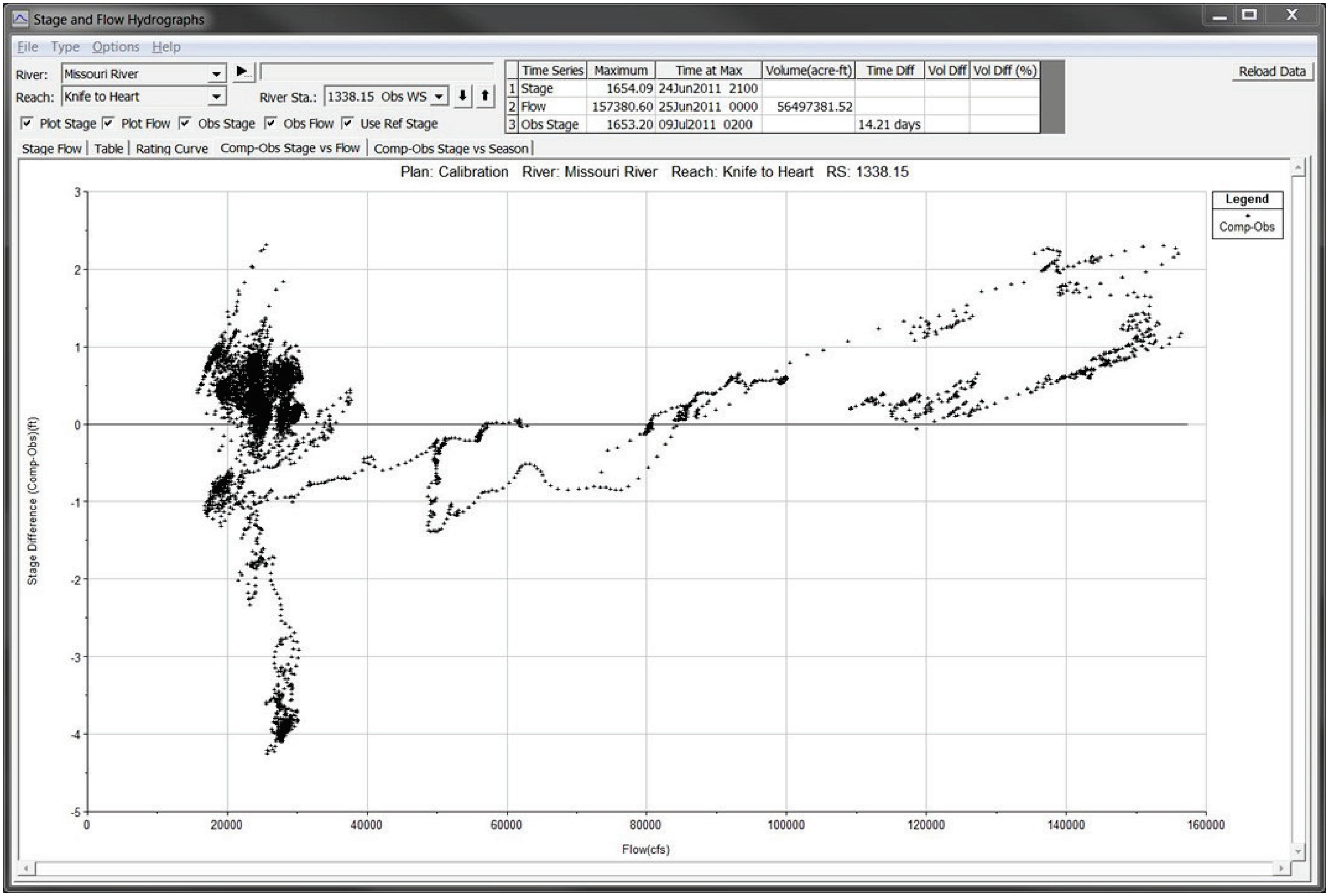


Plate 11: Missouri River at Price, ND Comp-Obs Stage vs Flow

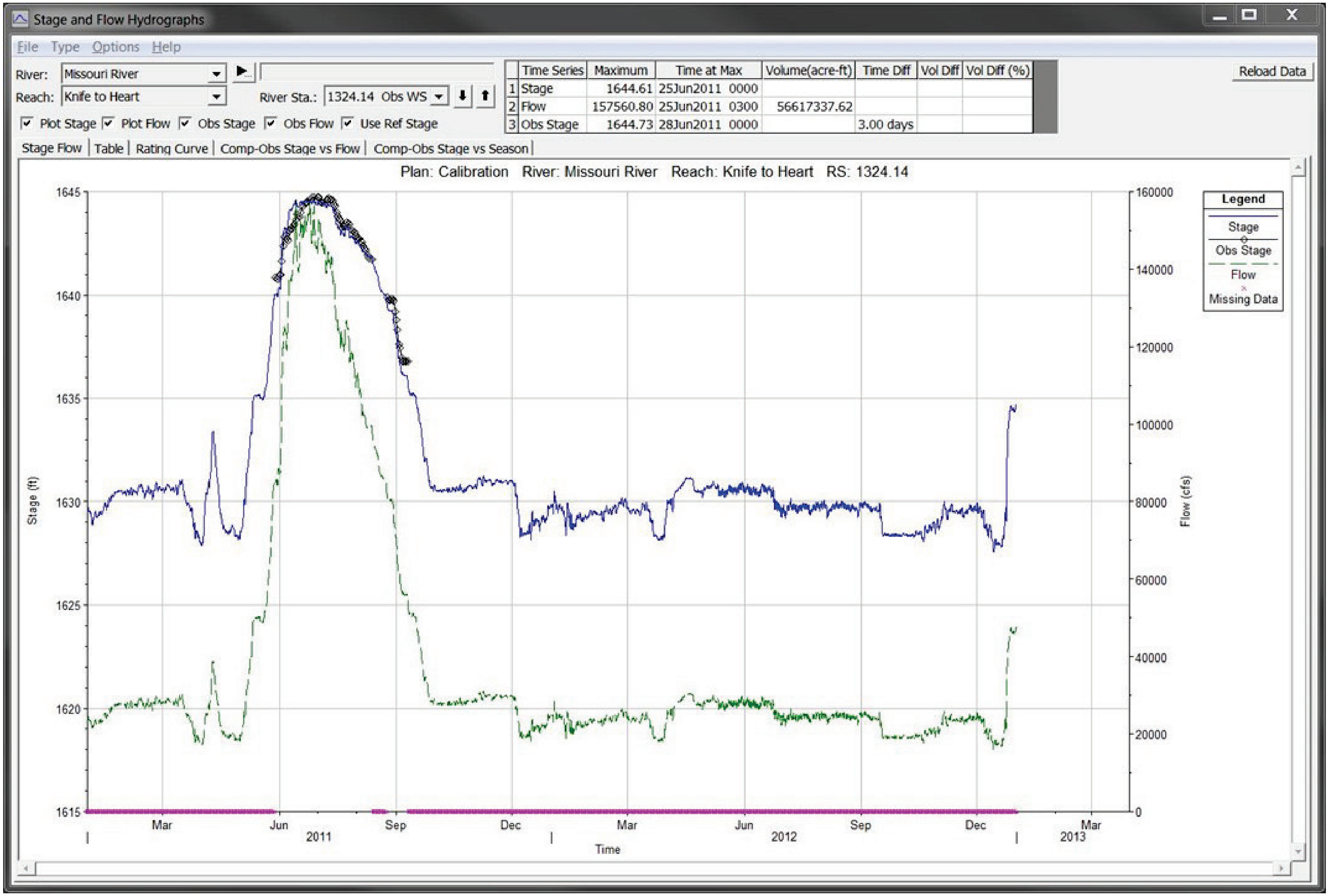


Plate 12: Missouri River at Eagle Park near Bismarck, ND Hydrograph

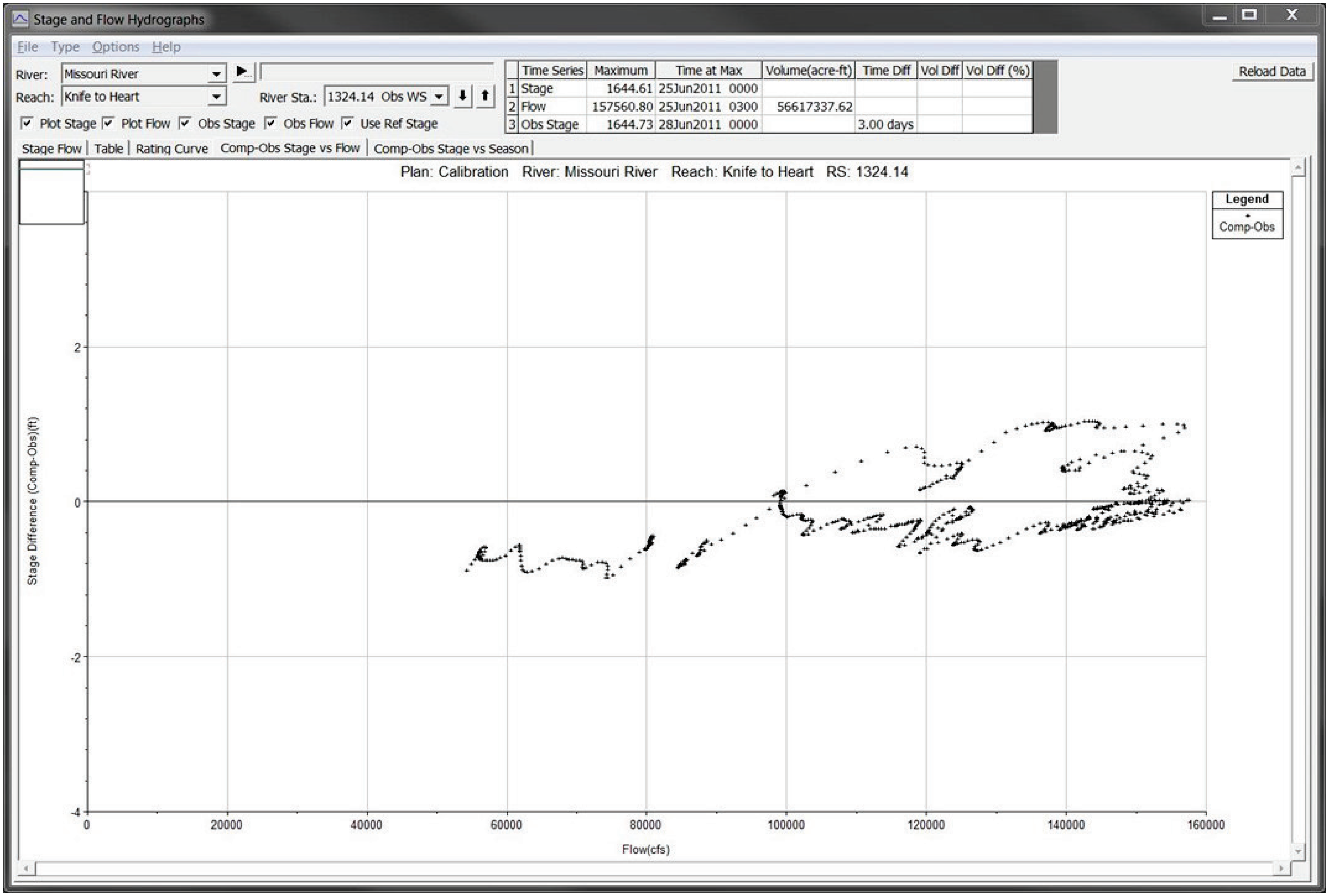


Plate 13: Missouri River at Eagle Park near Bismarck, ND Comp-Obs Stage vs Flow

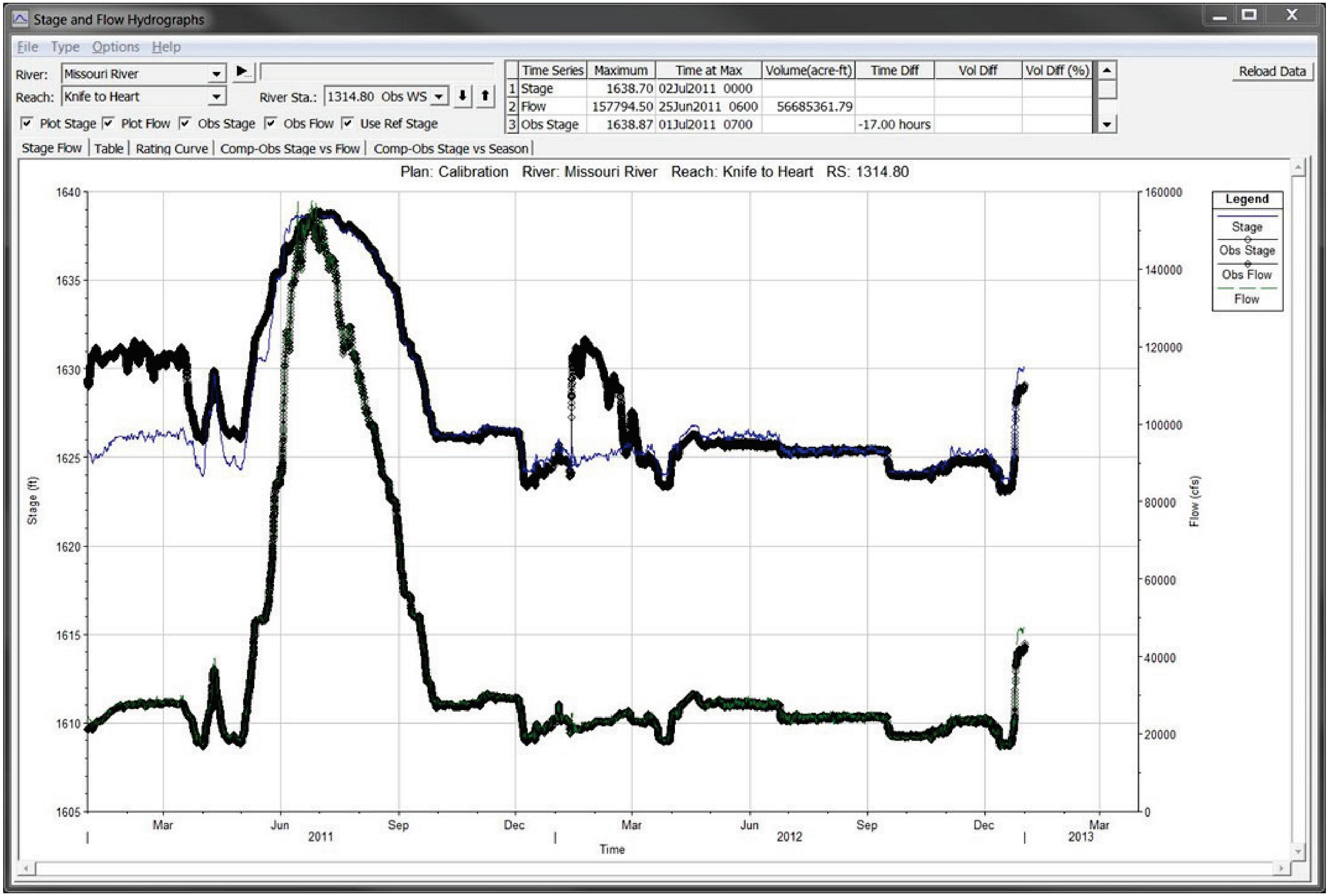


Plate 14: Missouri River at Bismarck, ND Hydrograph

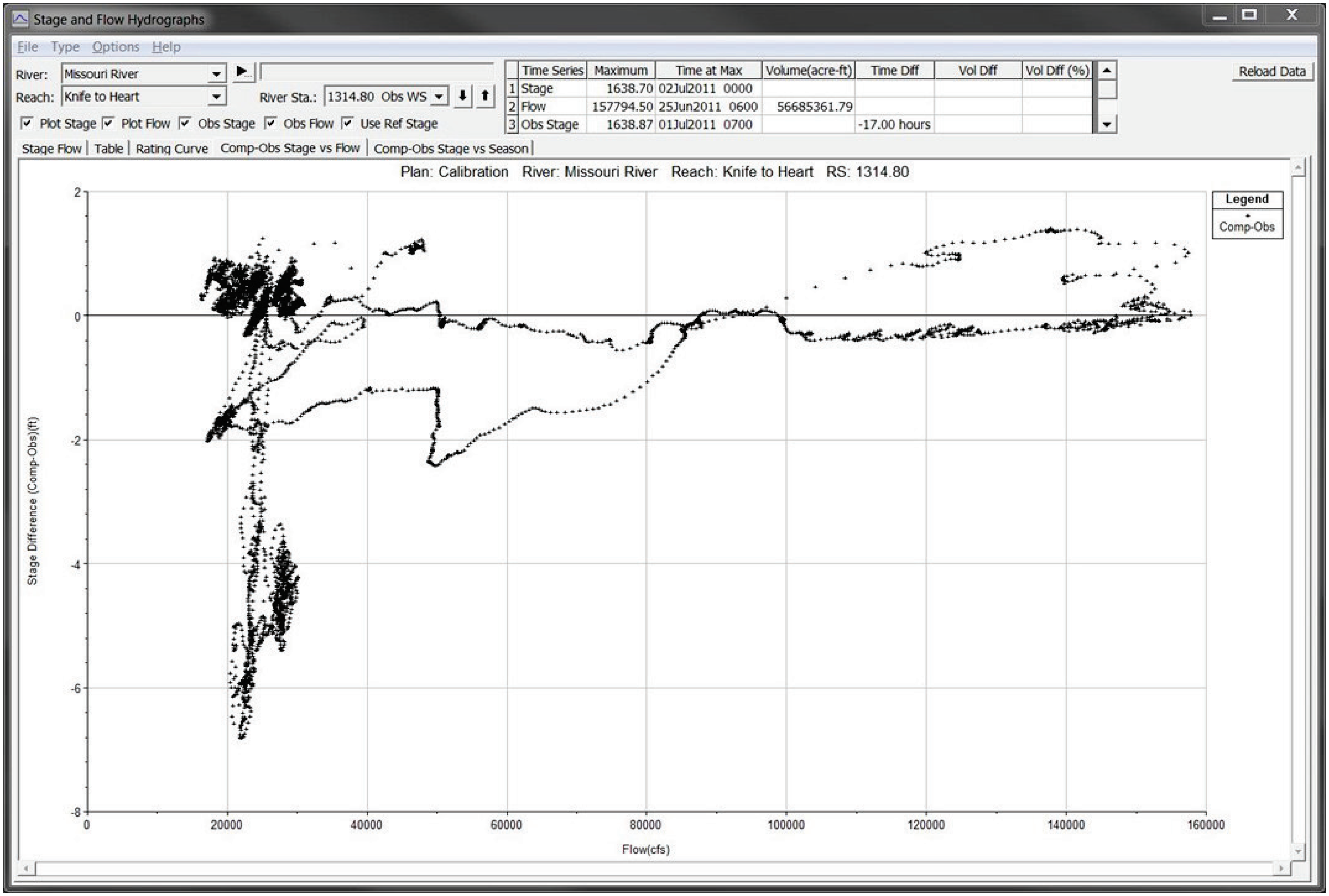


Plate 15: Missouri River at Bismarck, ND Comp-Obs Stage vs Flow

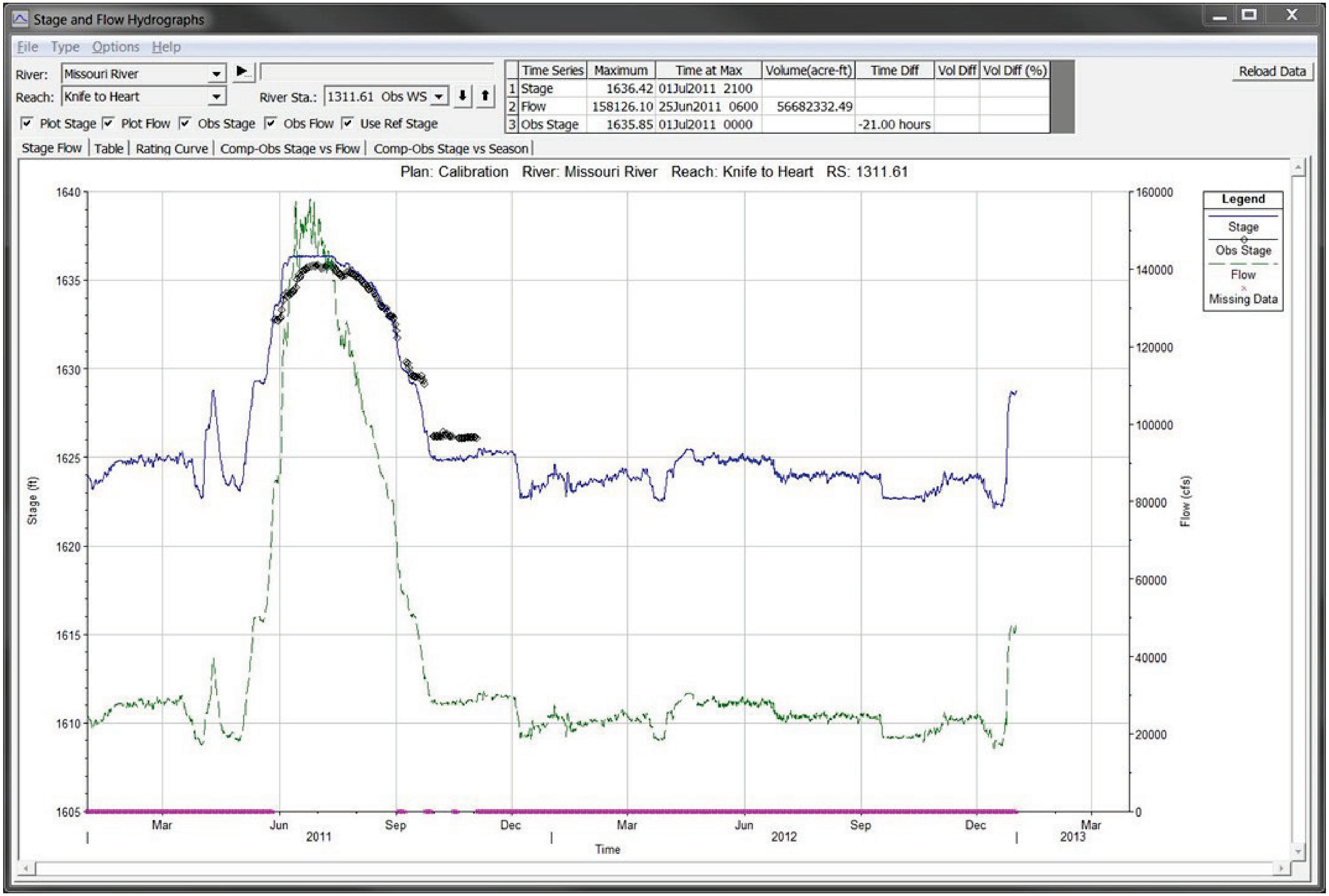
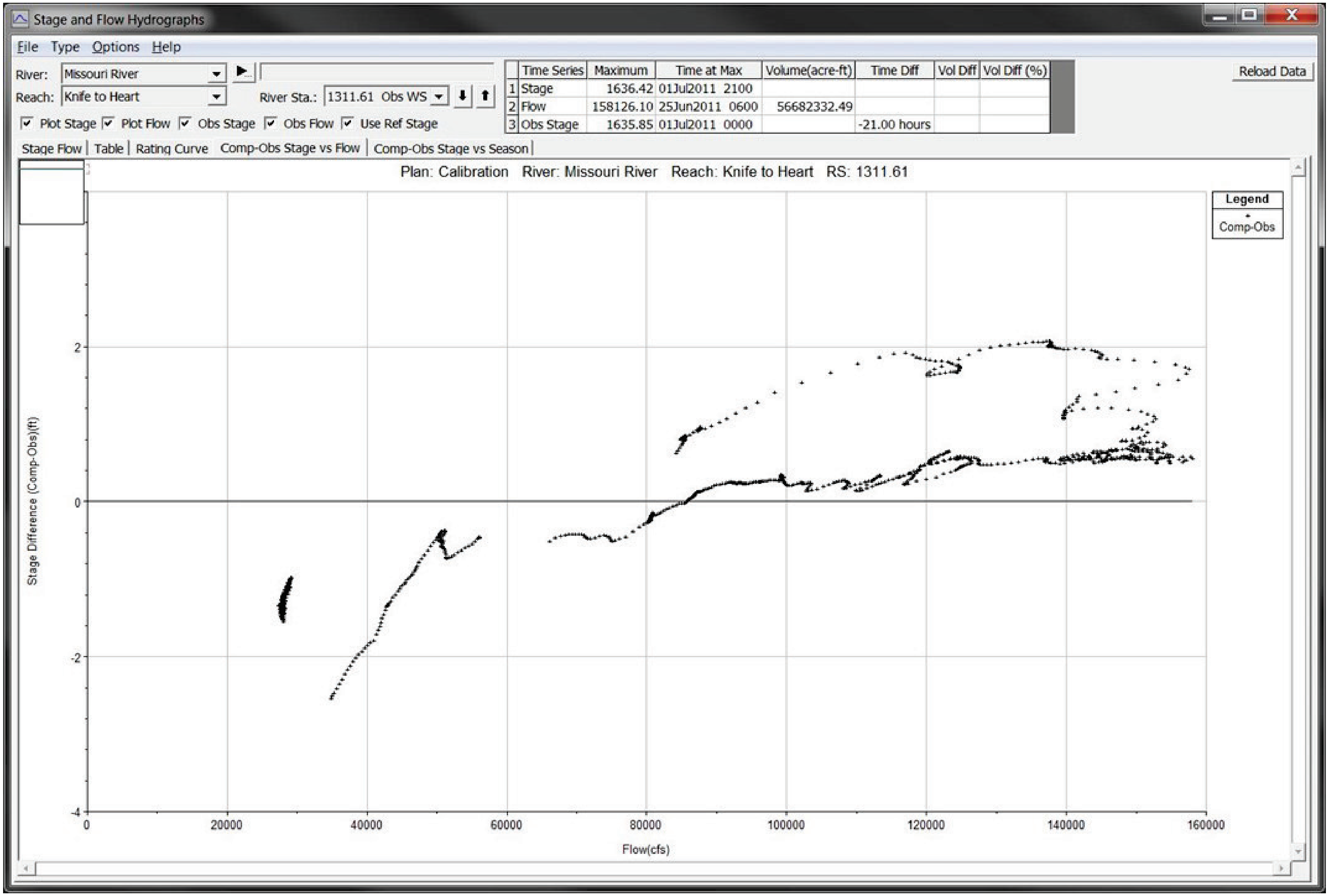


Plate 16: Missouri River on Tavis Road at Bismarck, ND Hydrograph



	Time Series	Maximum	Time at Max	Volume(acre-ft)	Time Diff	Vol Diff	Vol Diff (%)
1	Stage	1636.42	01Jul2011 2100				
2	Flow	158126.10	25Jun2011 0600	56682332.49			
3	Obs Stage	1635.85	01Jul2011 0000		-21.00 hours		

Plate 17: Missouri River on Tavis Road at Bismarck, ND Comp-Obs Stage vs Flow

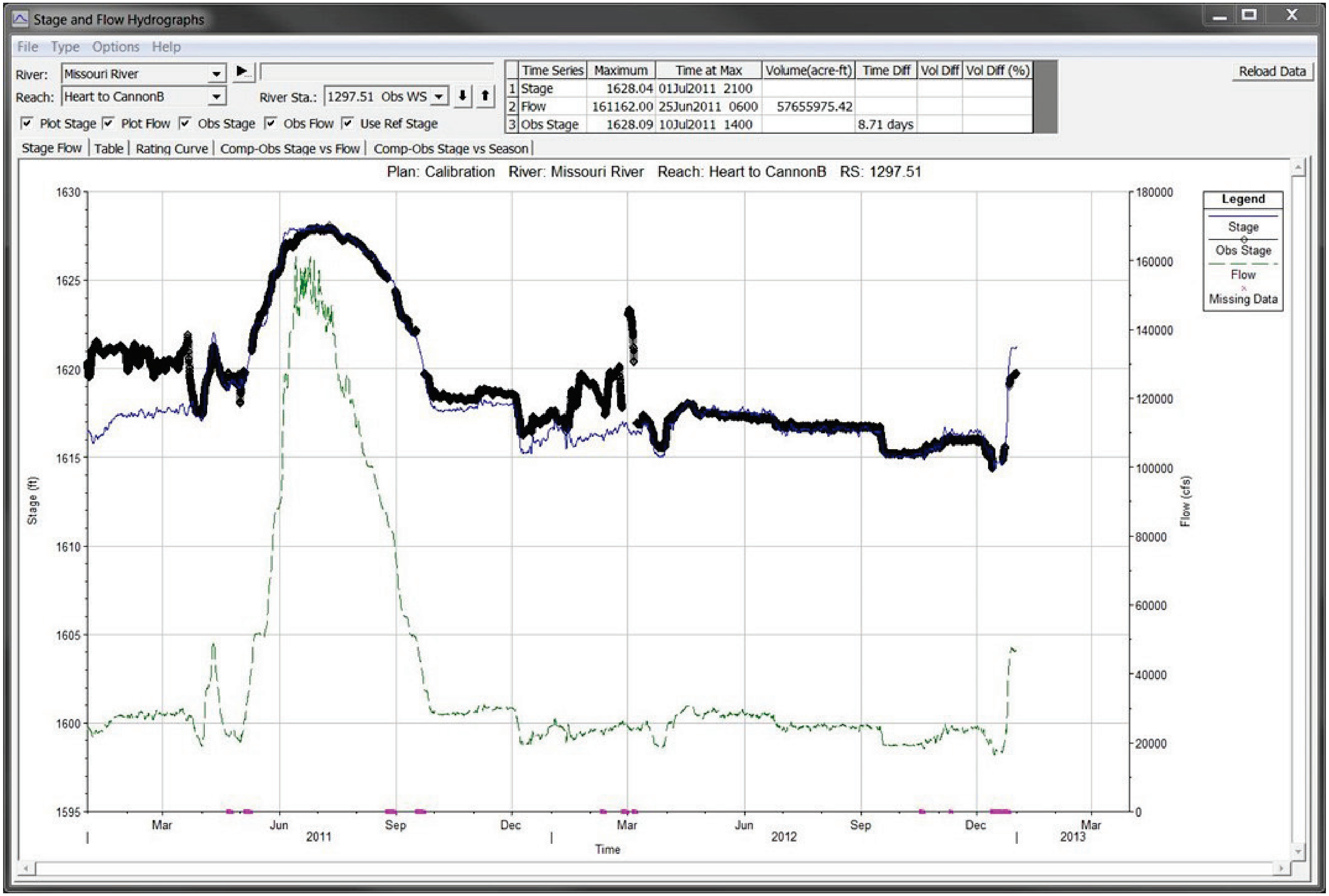


Plate 18: Missouri River near Schimdt, ND Hydrograph

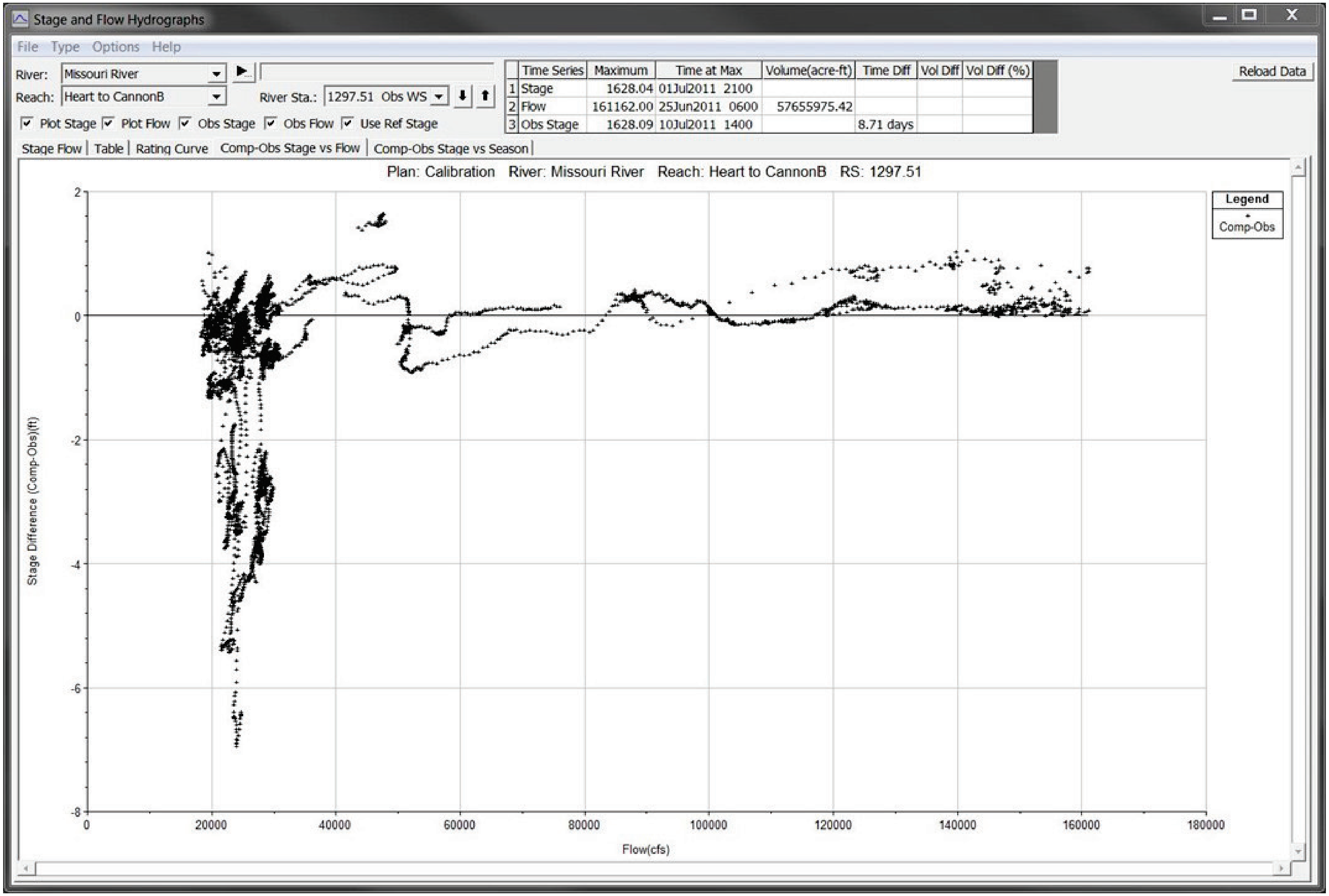


Plate 19: Missouri River near Schimdt, ND Comp-Obs Stage vs Flow

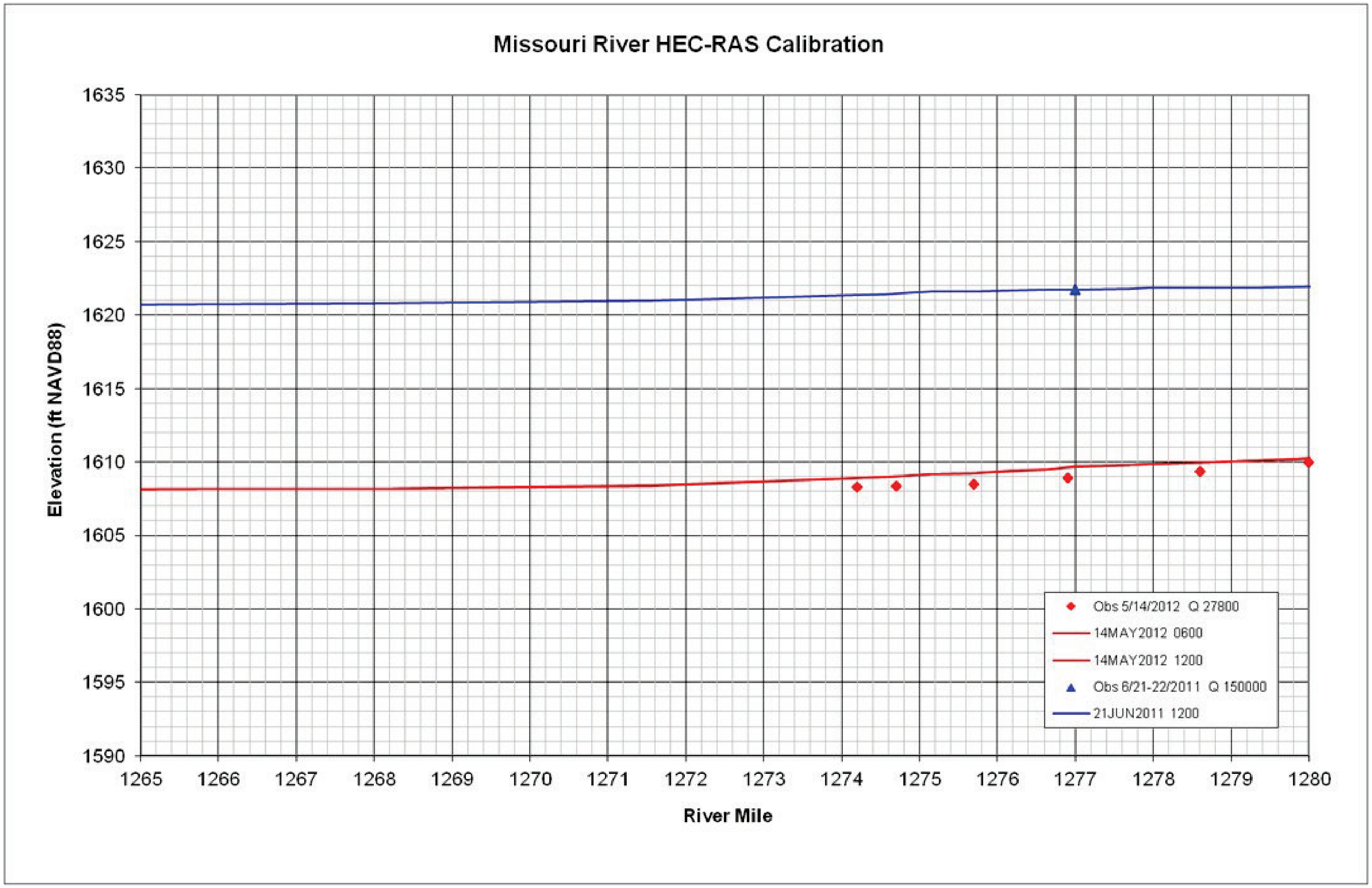


Plate 20: Measured WSP vs Computed Water Surface – RM 1265 to 1280

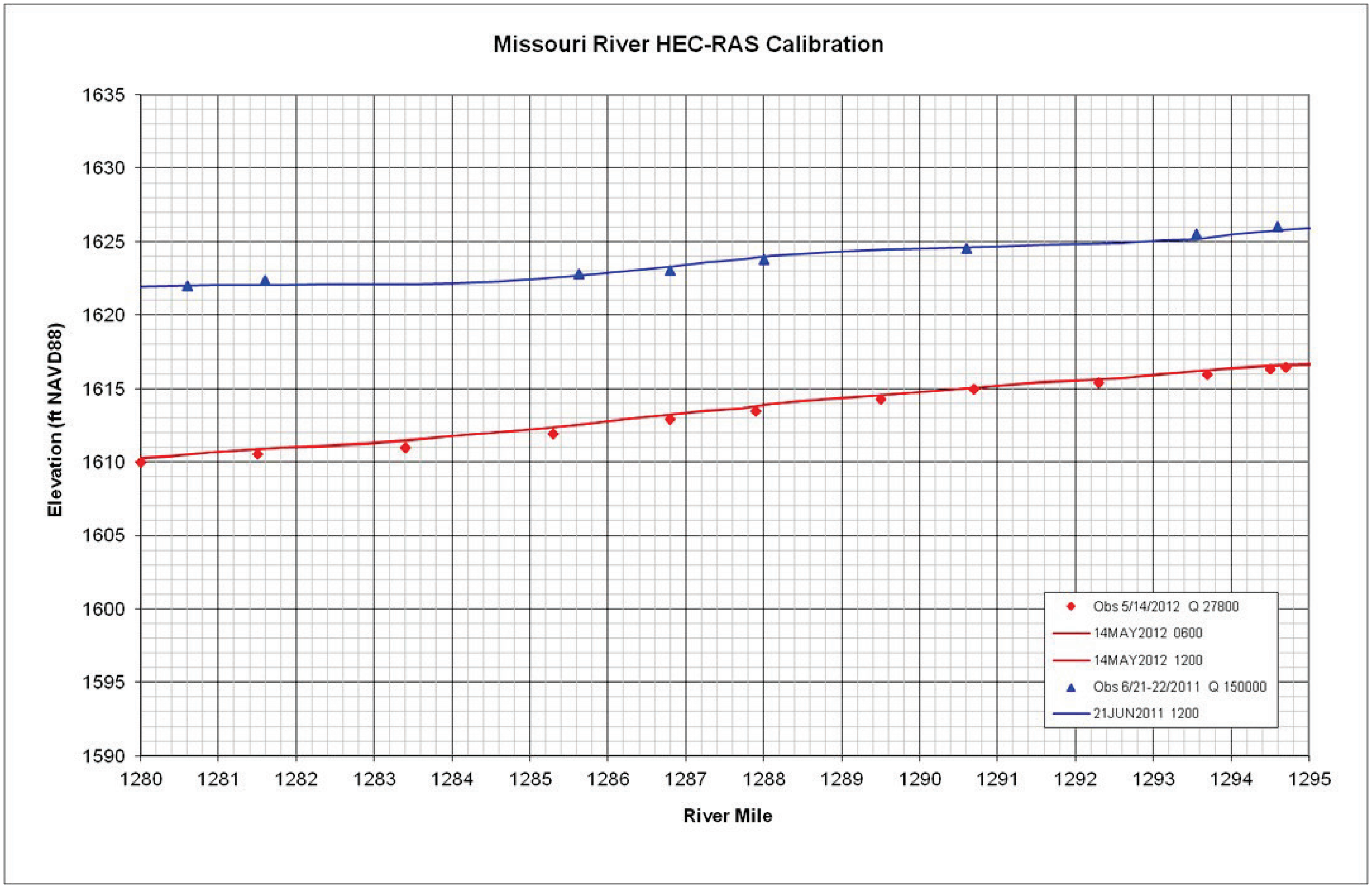


Plate 21: Measured WSP vs Computed Water Surface – RM 1280 to 1295

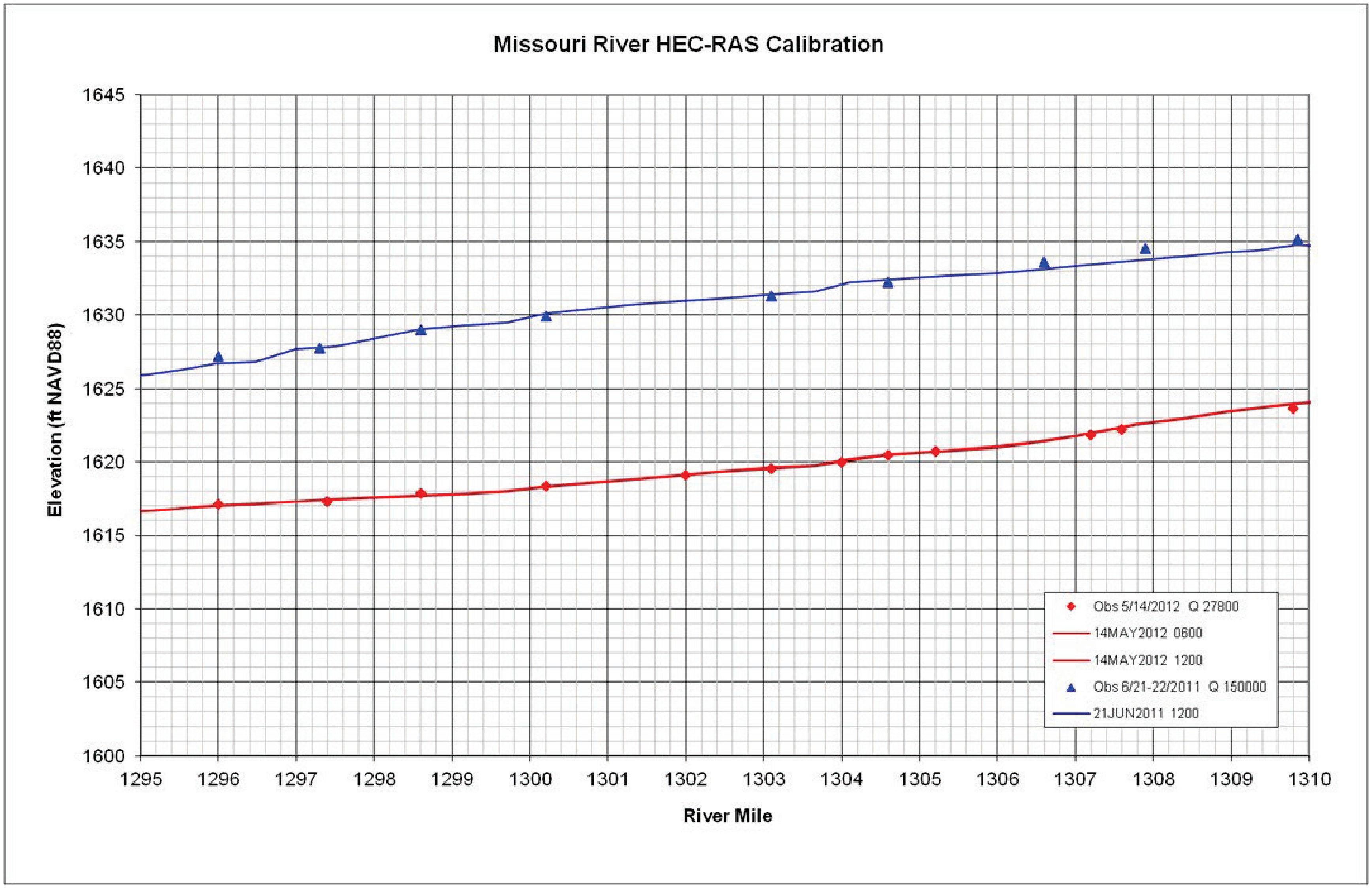


Plate 22: Measured WSP vs Computed Water Surface – RM 1295 to 1310

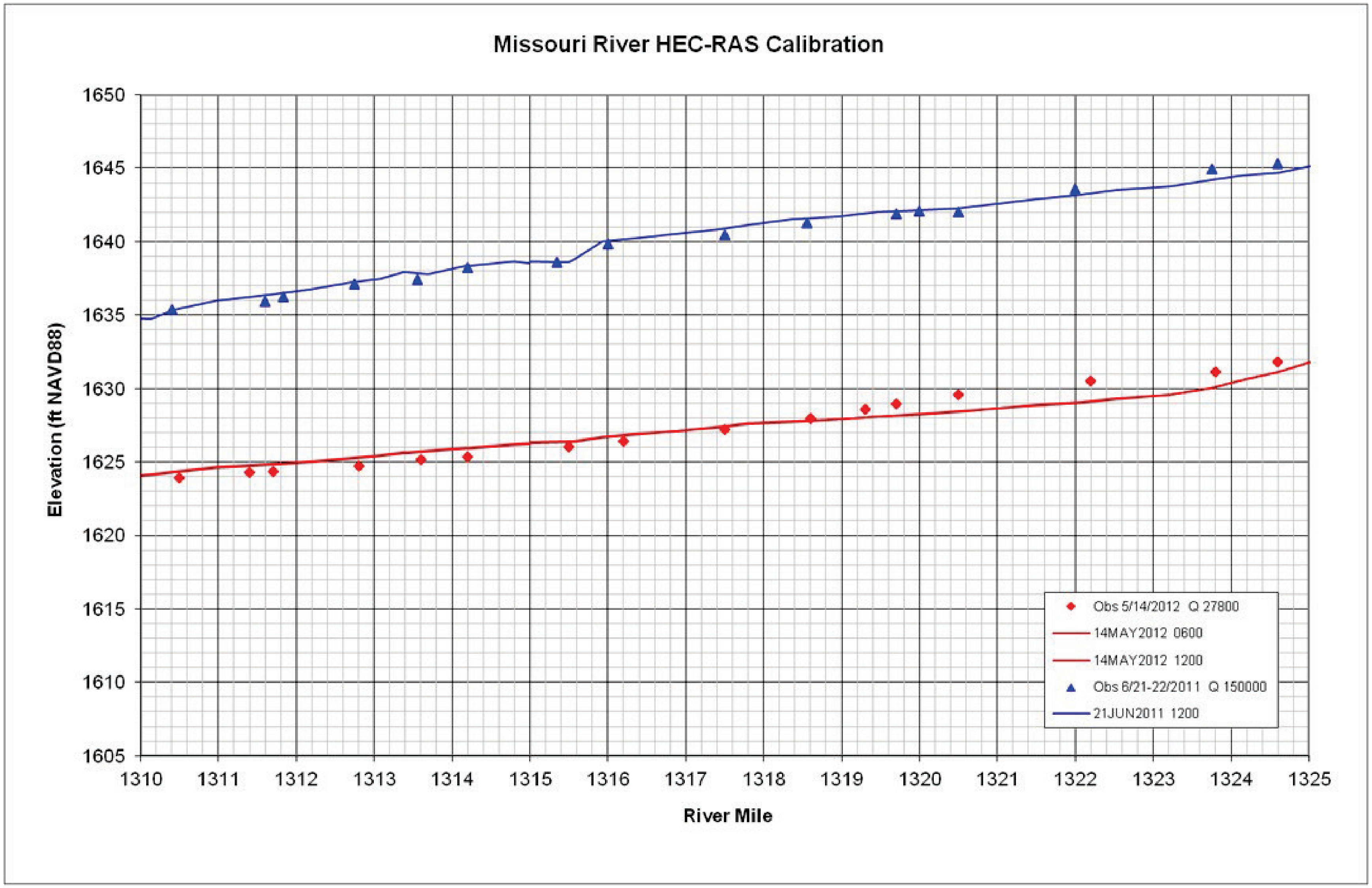


Plate 23: Measured WSP vs Computed Water Surface – RM 1310 to 1325

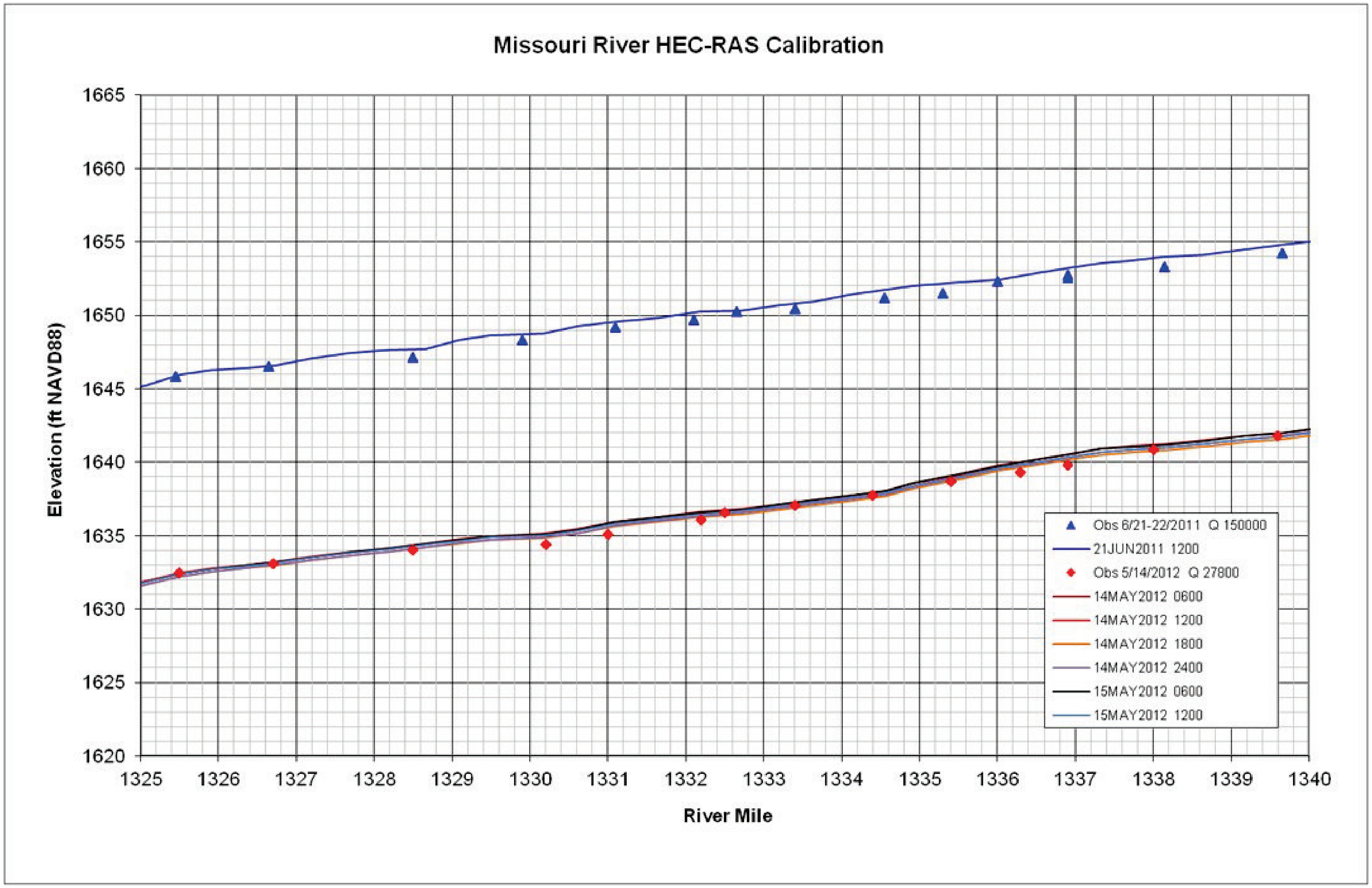


Plate 24: Measured WSP vs Computed Water Surface – RM 1325 to 1340

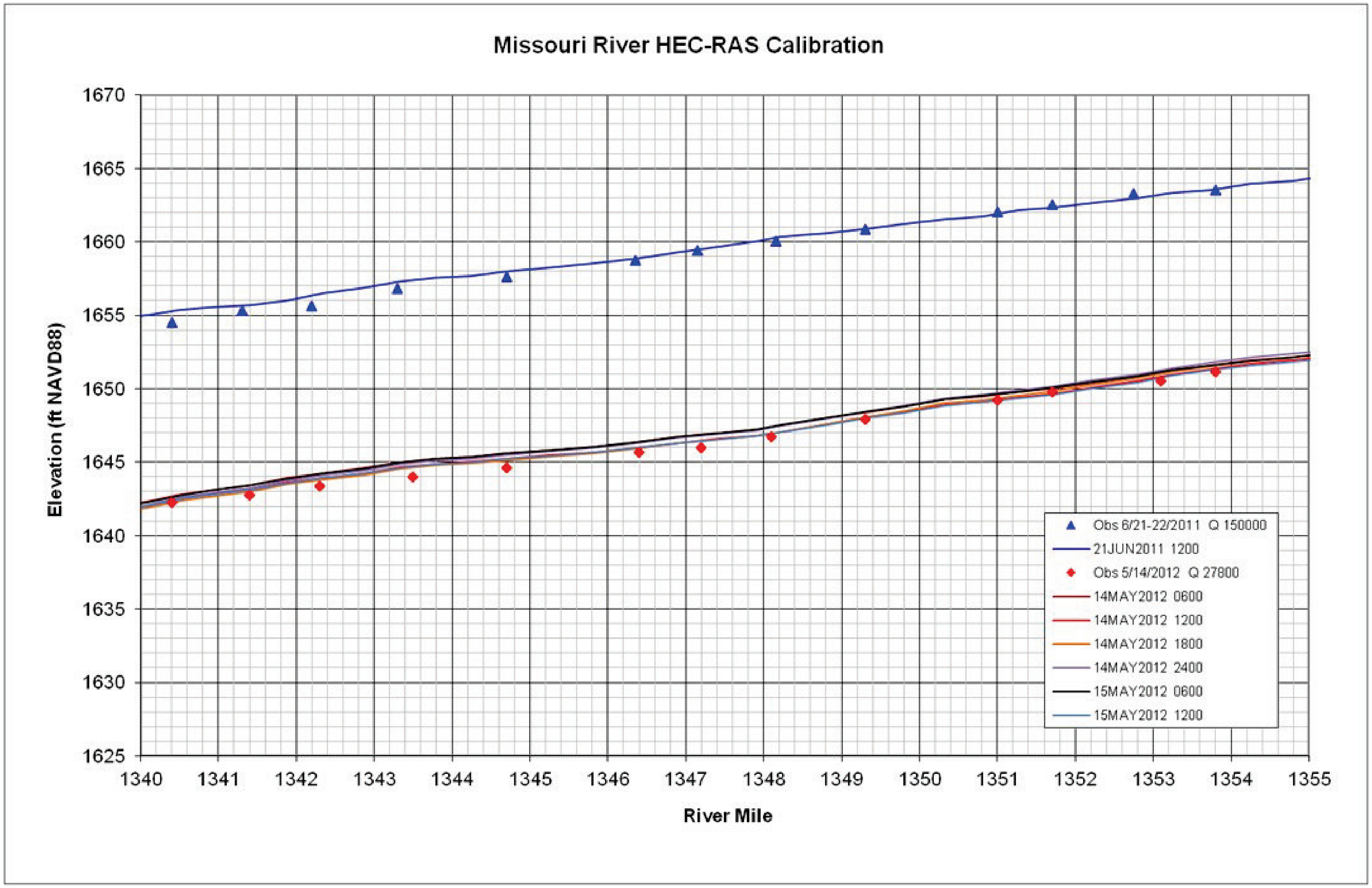


Plate 25: Measured WSP vs Computed Water Surface – RM 1340 to 1355

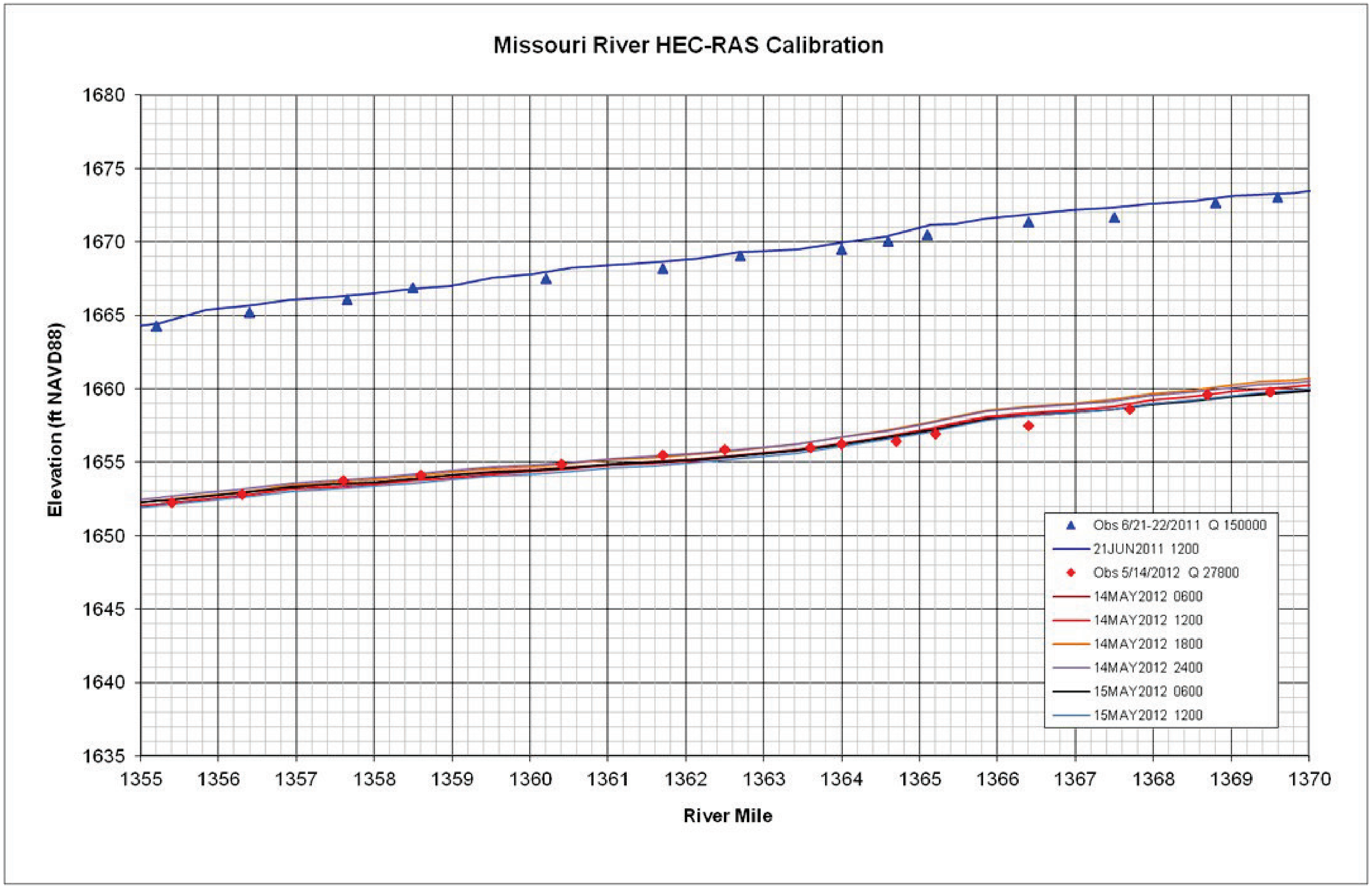


Plate 26: Measured WSP vs Computed Water Surface – RM 1355 to 1370

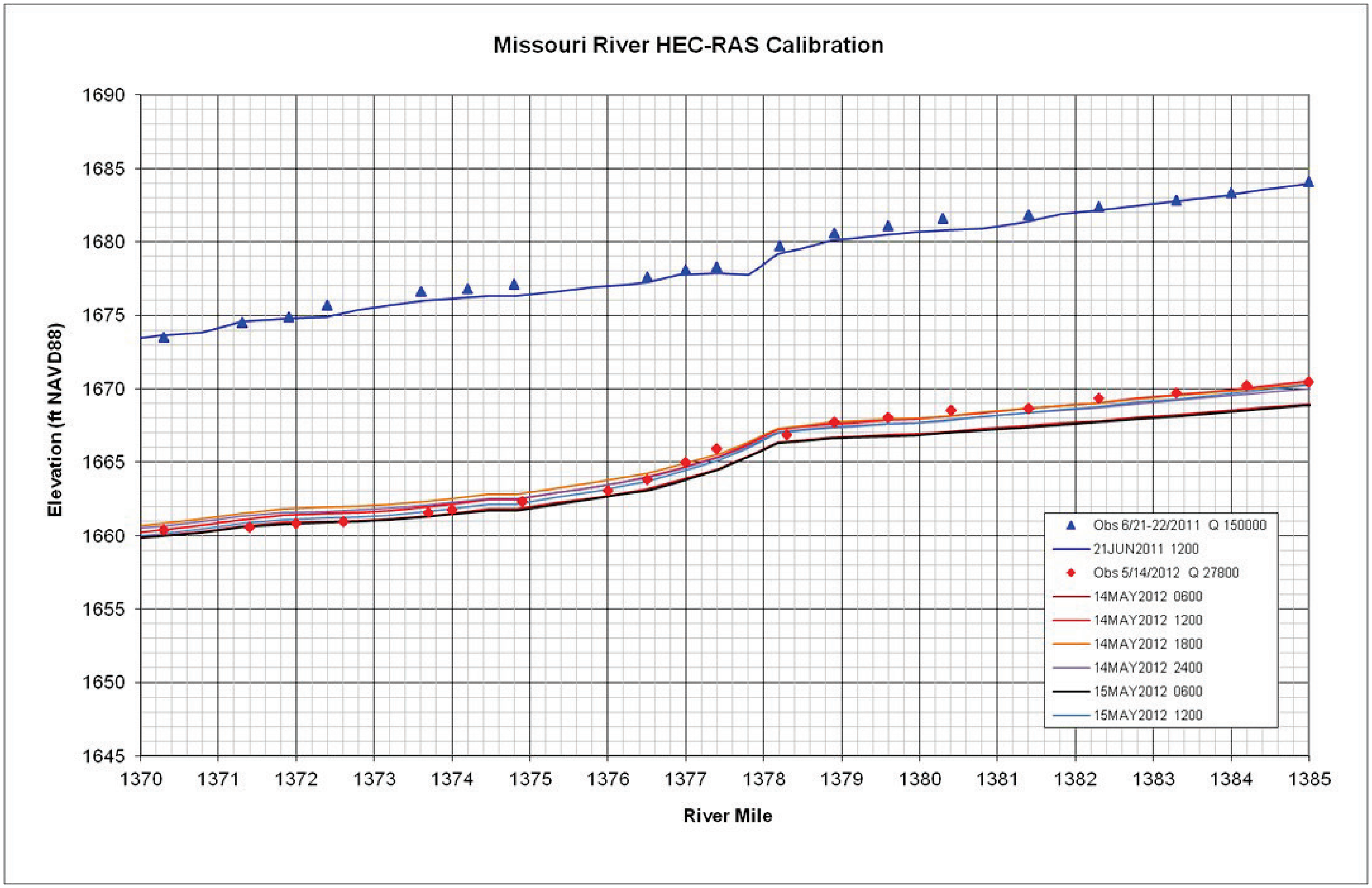


Plate 27: Measured WSP vs Computed Water Surface – RM 1370 to 1385

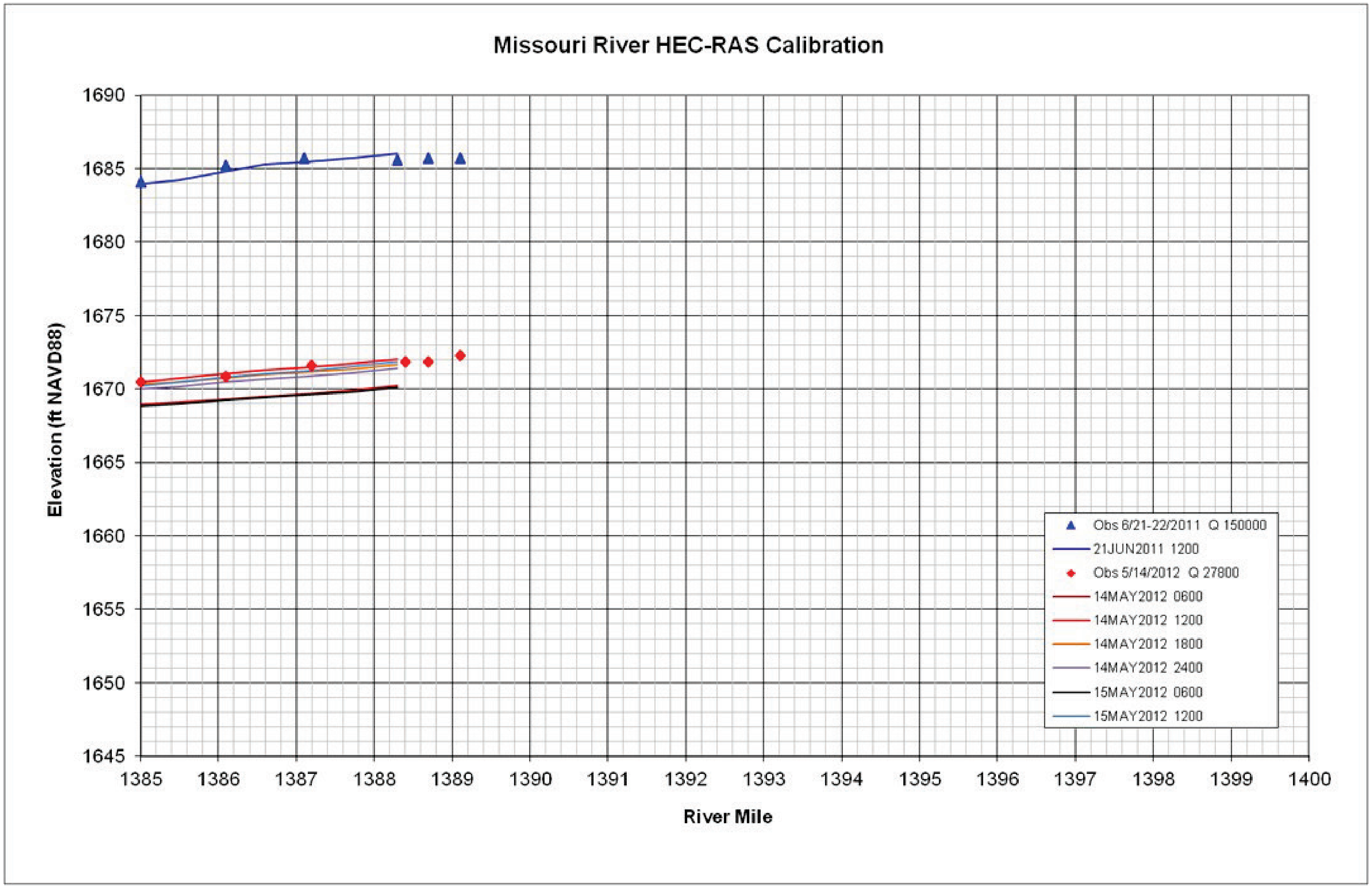


Plate 28: Measured WSP vs Computed Water Surface – RM 1385 to Garrison Dam

APPENDIX B

GARRISON DAM TO OAHE DAM

ATTACHMENT 1 – CROSS SECTION INTERPOLATION

Attachment 1
Missouri River RAS Modeling
Cross Section Interpolation
9 July 2014

Overview
The Missouri River RAS unsteady modeling project will construct unsteady flow models for the Missouri River from Ft Peck Dam, Montana, to St. Louis, Missouri. Upstream of Gavins Point Dam (near RM 811), the hydrographic data primarily consists of sediment range surveys used to monitor aggradation / degradation between the dams. Figure 1 illustrates the reach locations.

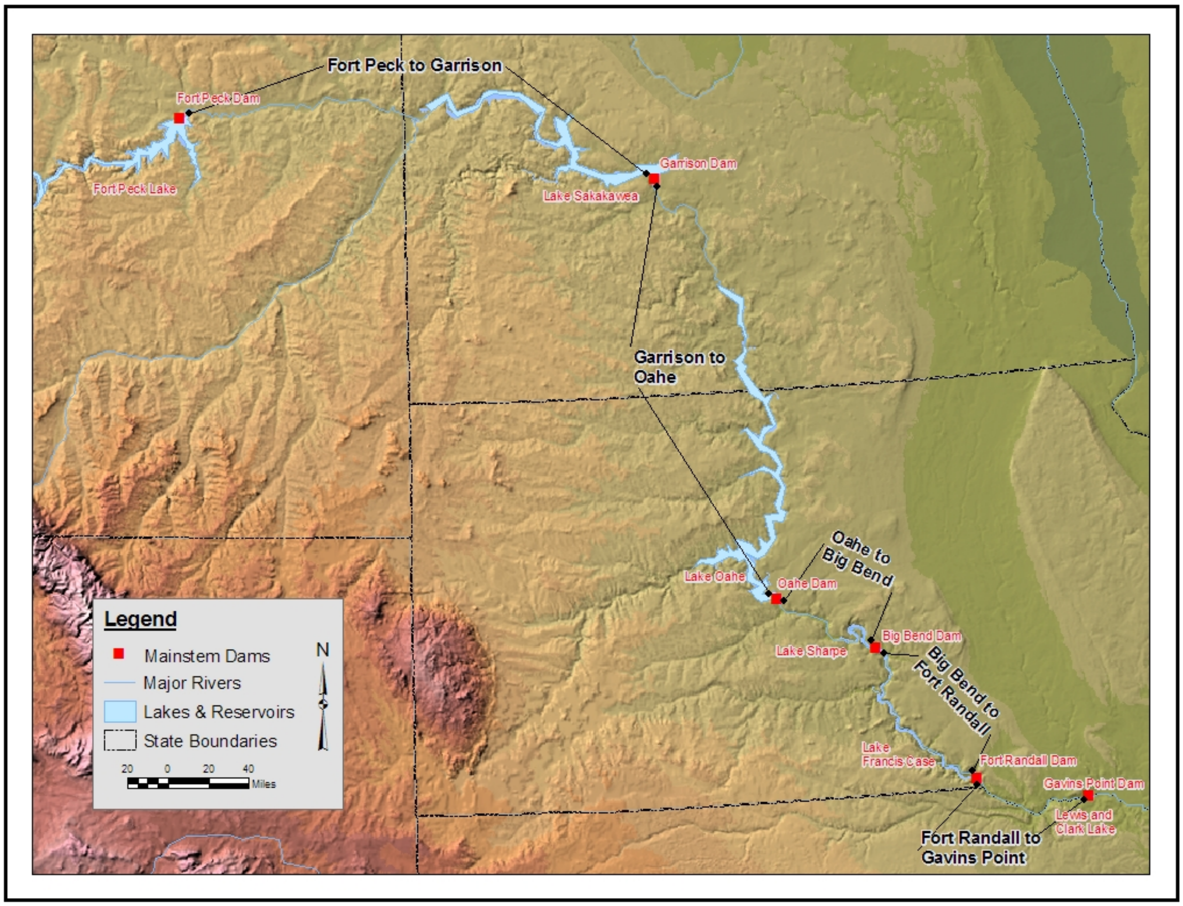


Figure 1. Mainstem Dam Modeling Reaches

Cross Section Interval
Model assembly principles and the goals of the study indicate that a cross section interval on the order of 2500 to 3000 feet would be appropriate. The sediment range spacing typically varies on the order of 1 to 3 miles so cross sections were interpolated in RAS to obtain estimated bathymetry.

Linear Interpolation

The between 2 cross sections option in the cross section interpolation tool in RAS was used to interpolate the underwater portion of the cross sections between the sediment ranges. Using the option **Generate for display as perpendicular segments to reach invert** places the interpolated cross sections along the stream centerline. A maximum distance of 3000 feet was used and additional cords were added where needed (the default cords are at the ends, banks, and channel invert). As can be seen in Figure 2 below, the RAS interpolated cross sections were imported into ArcMap and were adjusted to better represent the channel and floodplain. These new re-drawn cross sections were then used in GeoRAS to obtain elevation data along the correct alignment. The estimated (RAS interpolated) bathymetry was then merged into the re-drawn overbank cross section data.

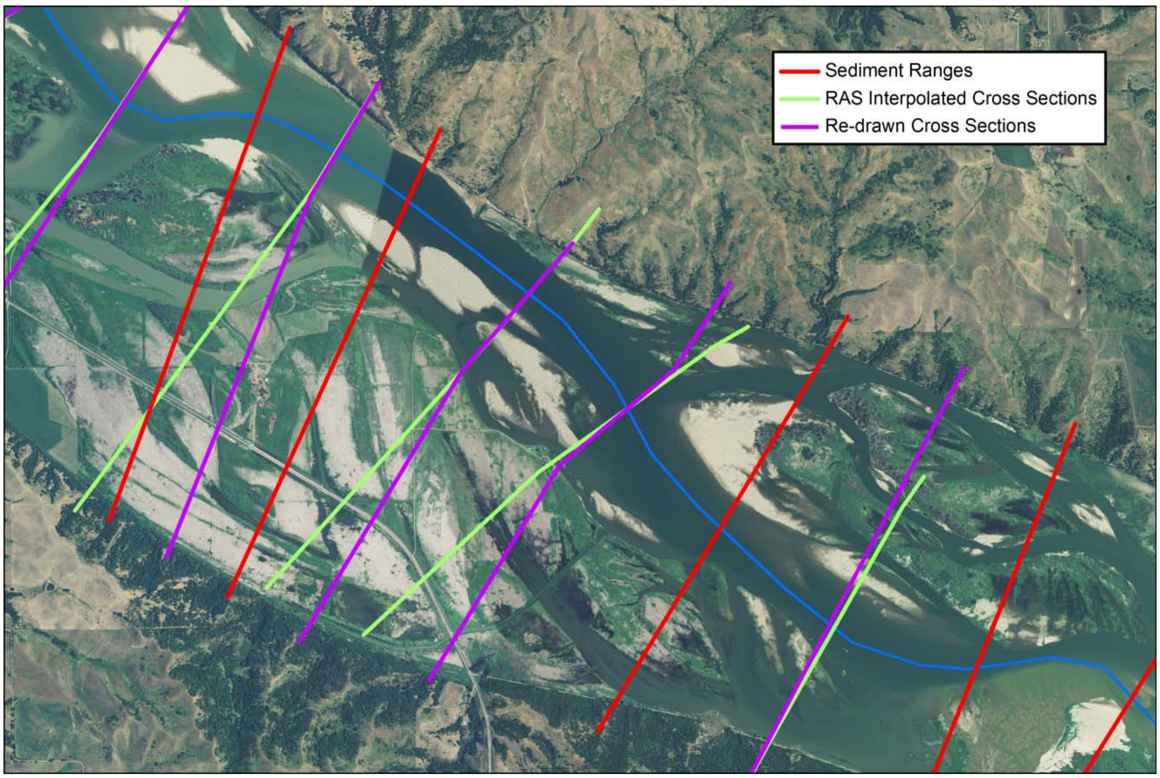


Figure 2. Comparison of RAS interpolated and re-drawn cross sections.

Estimating Bathymetry

After the new cross sections were re-cut in GeoRAS with LiDAR data, an underwater portion needed to be added to the cross section since the LiDAR does not penetrate below the water surface. Bathymetry was estimated by either using the RAS interpolated bathymetry or if that did not fit correctly with the overbank data, a nearby sediment range's bathymetry was vertically shifted and merged in. Differing widths and sandbar configurations presented a challenge to find another cross section that was similar.

When using a nearby range's bathymetry as an estimate a vertical shift was applied. The shift was based on the energy grade line slope (broken into several reaches) and the distance between the two cross

sections. See Figure 3 for an example of an energy grade line slope plot from a rough sediment range only model.

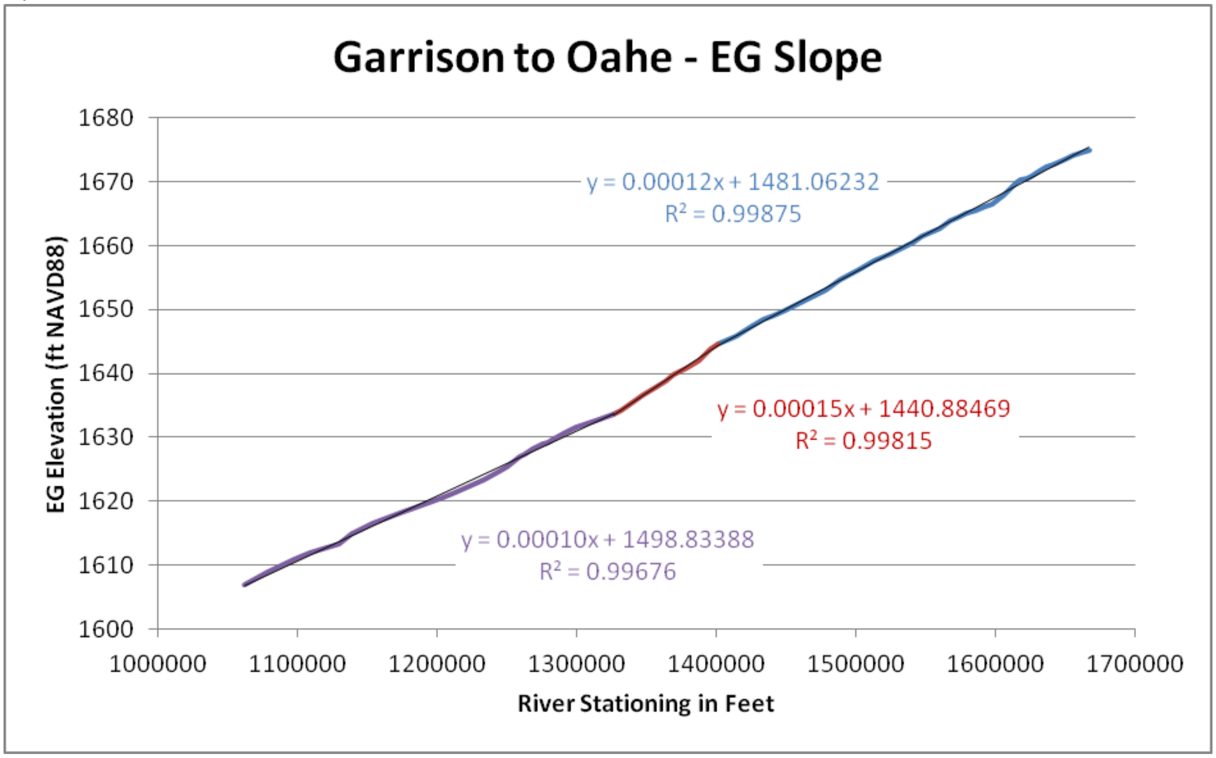


Figure 3. Example of Energy Grade Line Slope Plot

Examples of merging bathymetry into the cross sections are illustrated in Figures 4 – 9. The four example sites are shown in Figures 10 – 24.

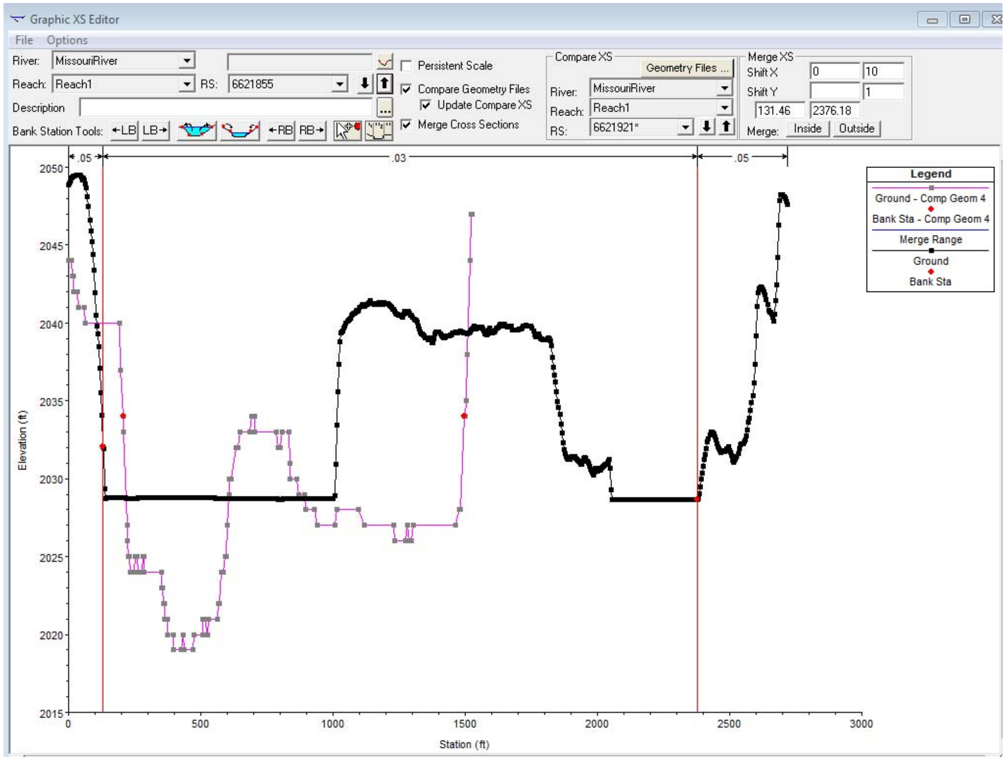


Figure 4. Paired XSs for merging, center island and changing channel widths cause alignment issues.

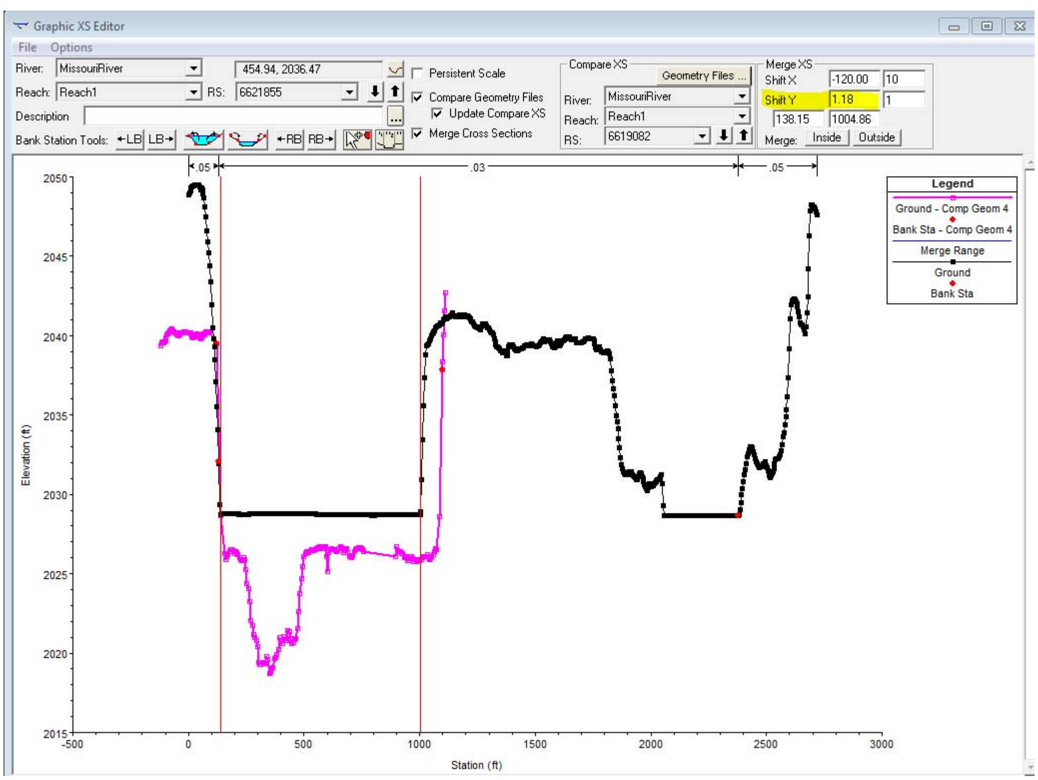


Figure 5. Same Cross-Section as in Figure with next downstream Rangeline vertically adjusted

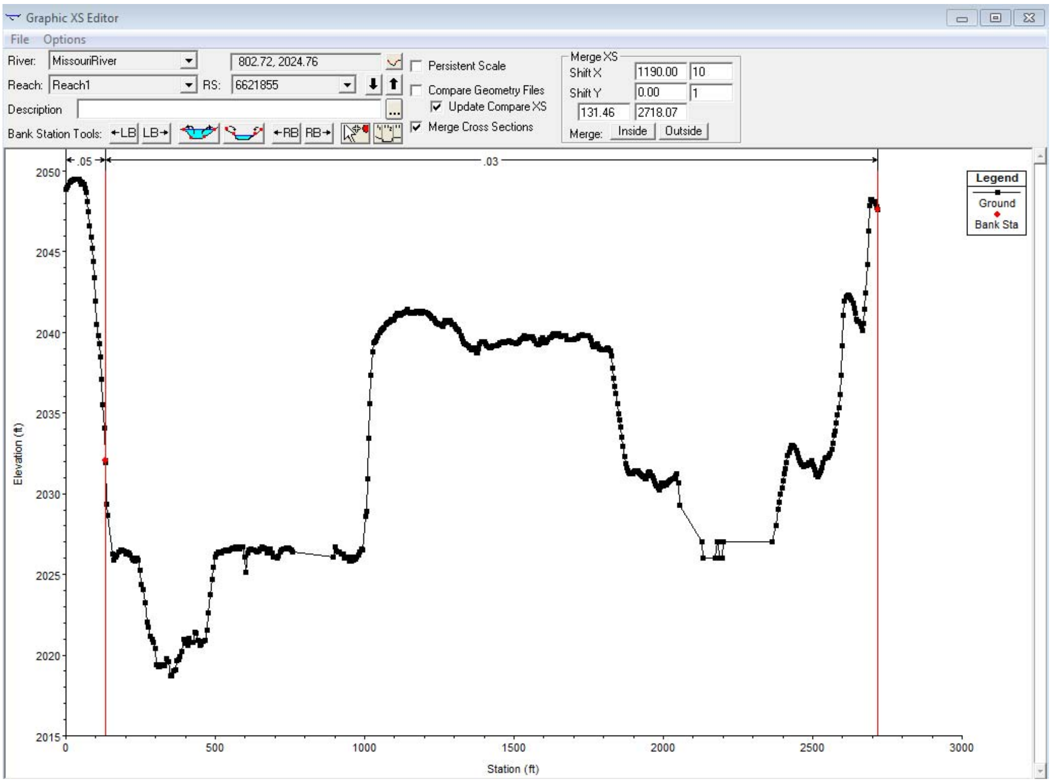


Figure 6. Composite Cross Section using the GeoRAS cut LiDAR data for above the WSE and the downstream rangeline and HEC-RAS interpolated cross sections for the channel data estimation.

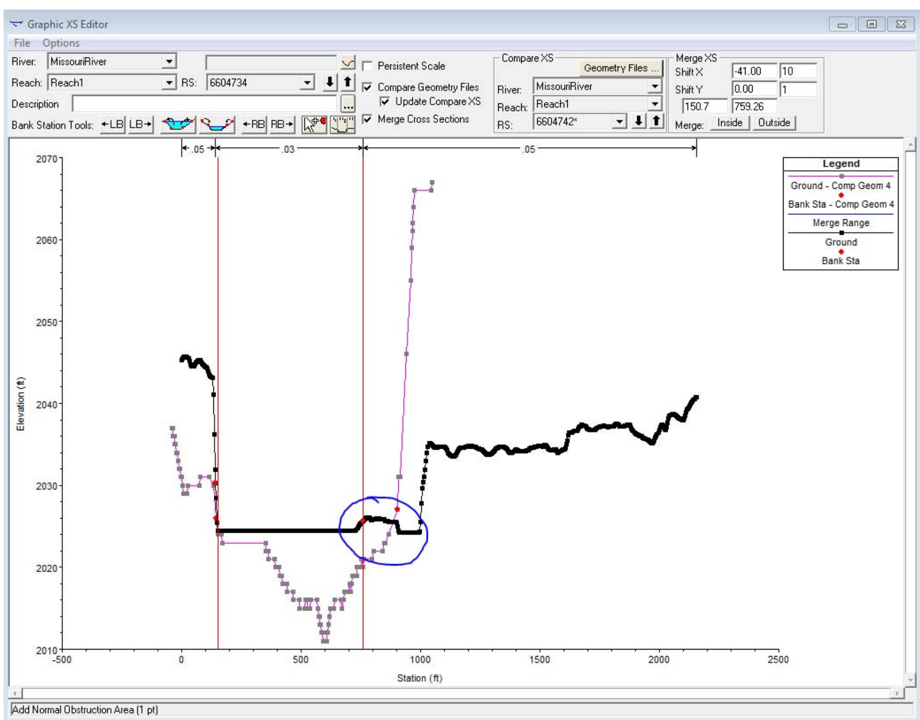


Figure 7. Sandbars in the channel present another challenge.

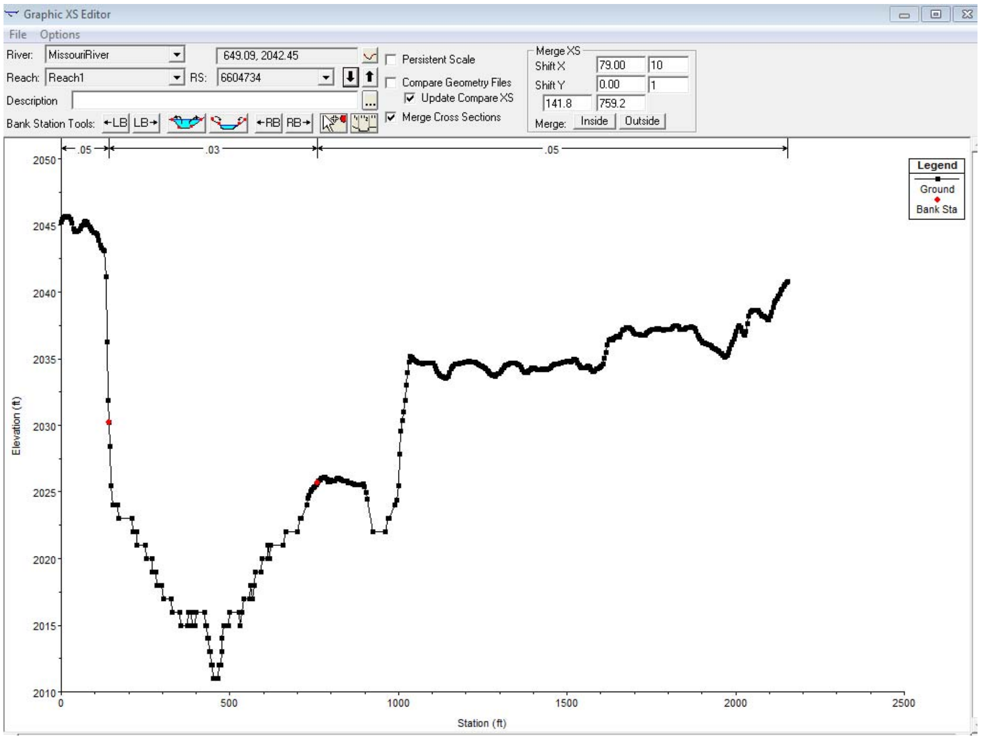


Figure 8. Composite Cross Section from Figure with sandbar

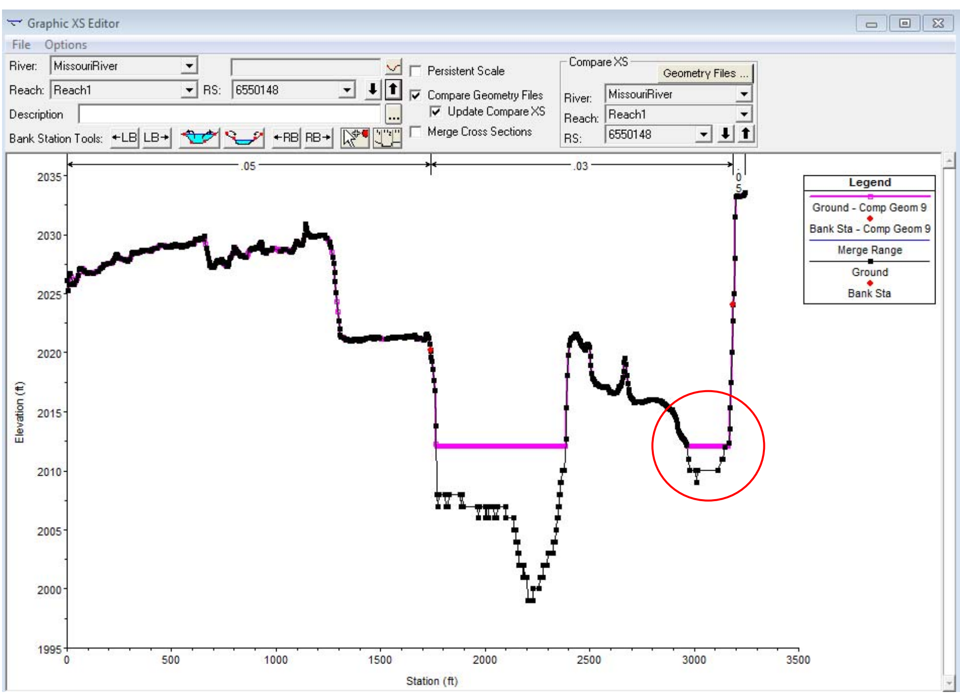


Figure 9. Before and After Merge - Small Channel inside of sandbar is Estimated

Example Sites

Comparison of RAS output built using Rangeline Interpolation and LiDAR data Merged with interpolated cross-sections. Images are at a Flow of 10,250 cfs which is about a normal annual flow in the Ft Peck Reach.

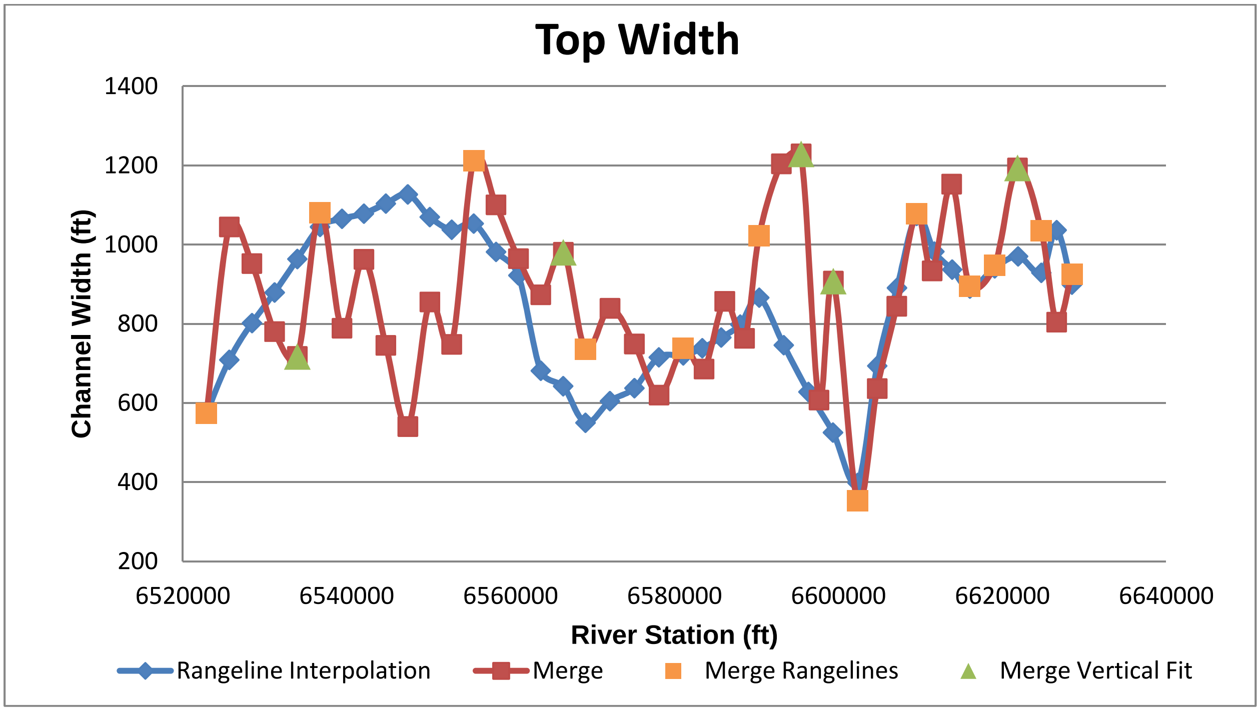


Figure10. Comparison of the RAS output for Top Width from the linearly interpolated rangeline XS model and the merged topo and rangeline model. The rangeline XSs based on survey data are orange. The green markers denote the topo XSs that didn't fit with the corresponding interpolated XS and used a more suitable nearby XS vertically fit to the local slope to merge the below water channel.

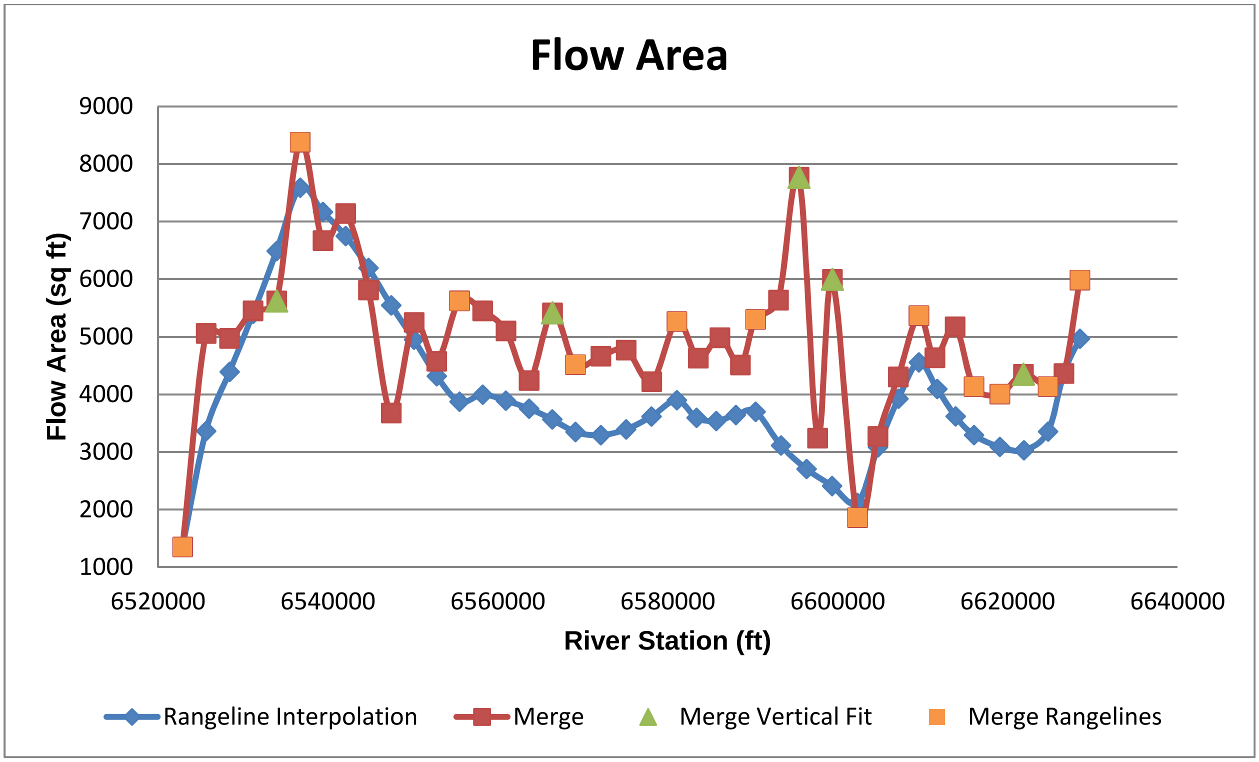


Figure11. Comparison of the RAS output for Flow Area from the linearly interpolated rangeline XS model and the merged topo and rangeline model.

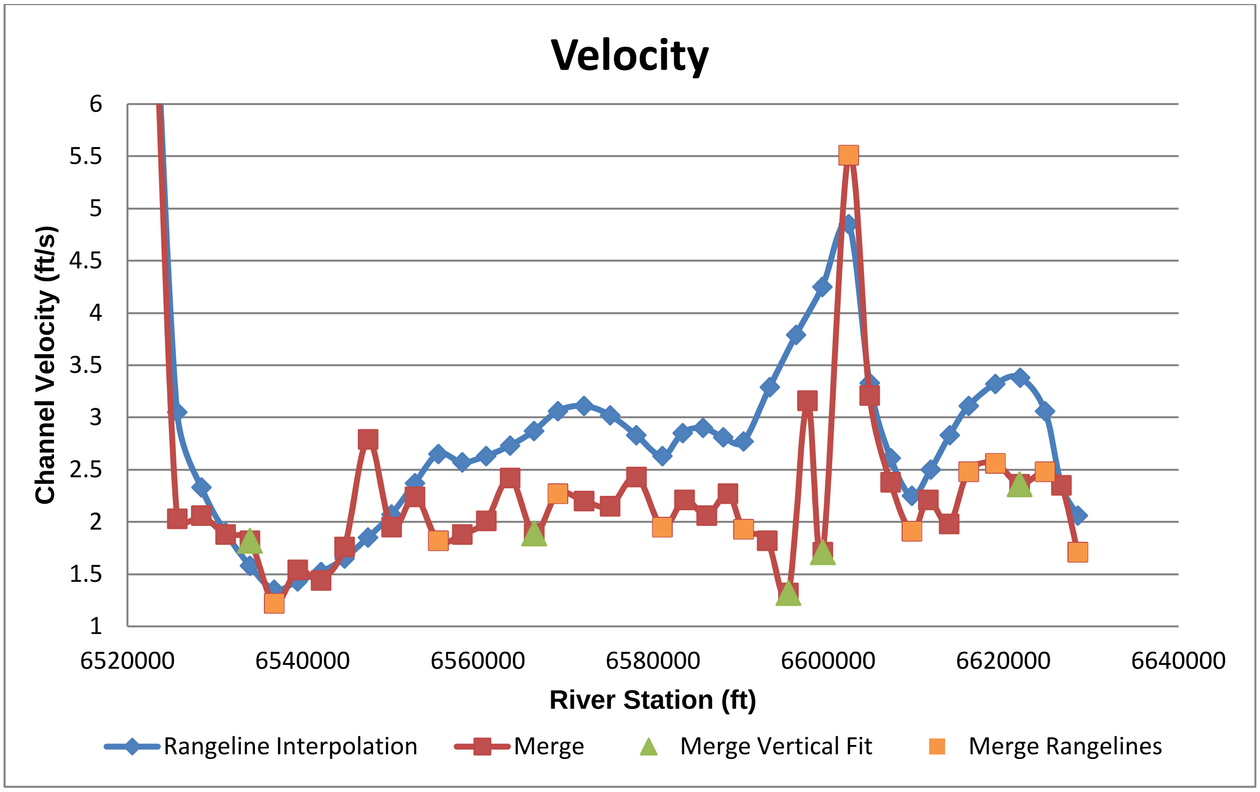


Figure 12. Comparison of the RAS output for Velocity from the linearly interpolated rangeline XS model and the merged topo and rangeline model.

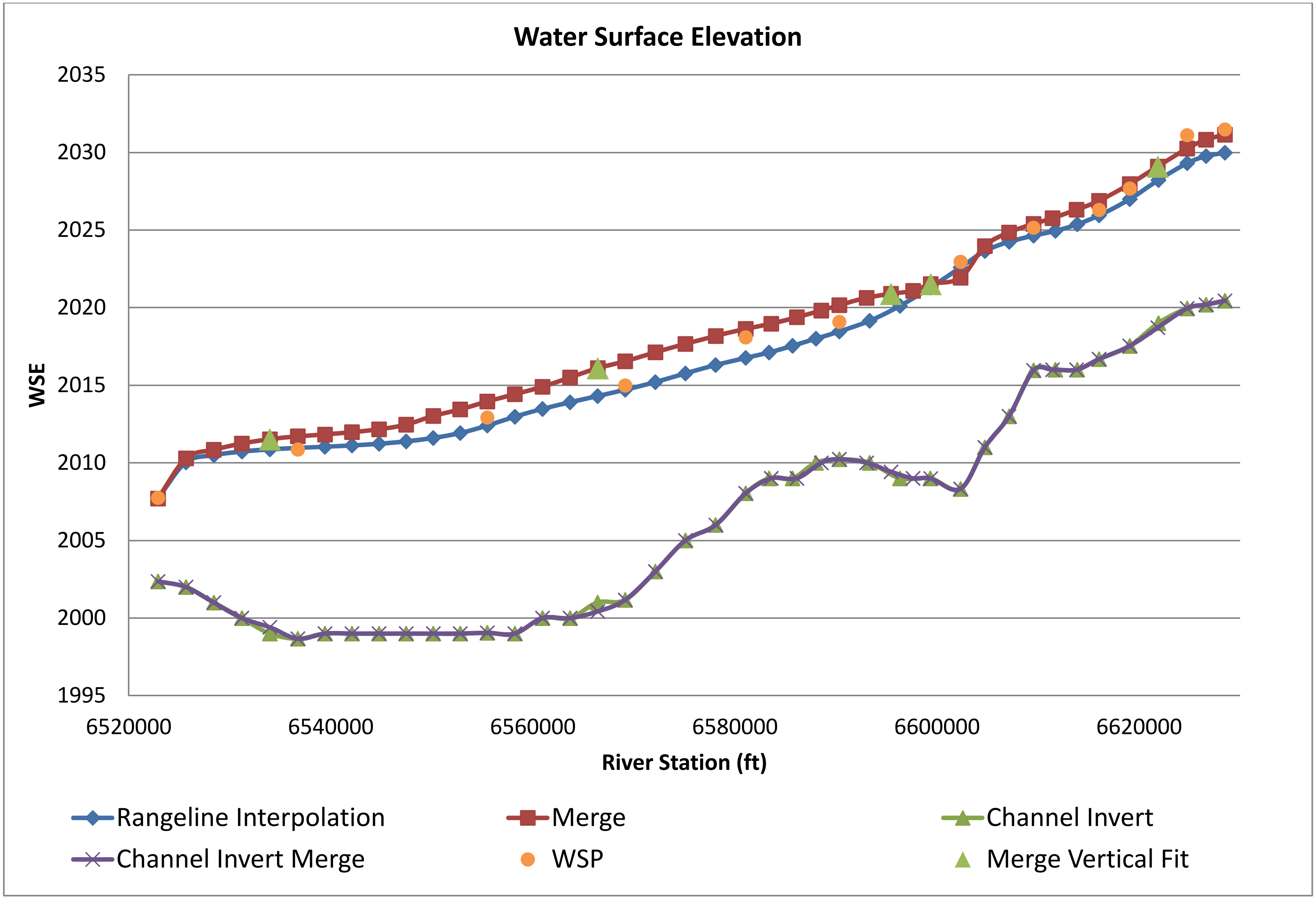
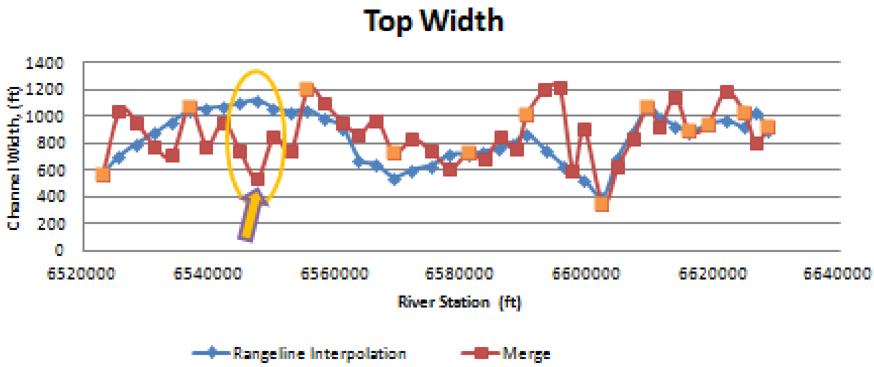


Figure 13. RAS Models compared to the 2012 Water Surface Profile (WSP) at Flow 10,250 cfs

Example 1



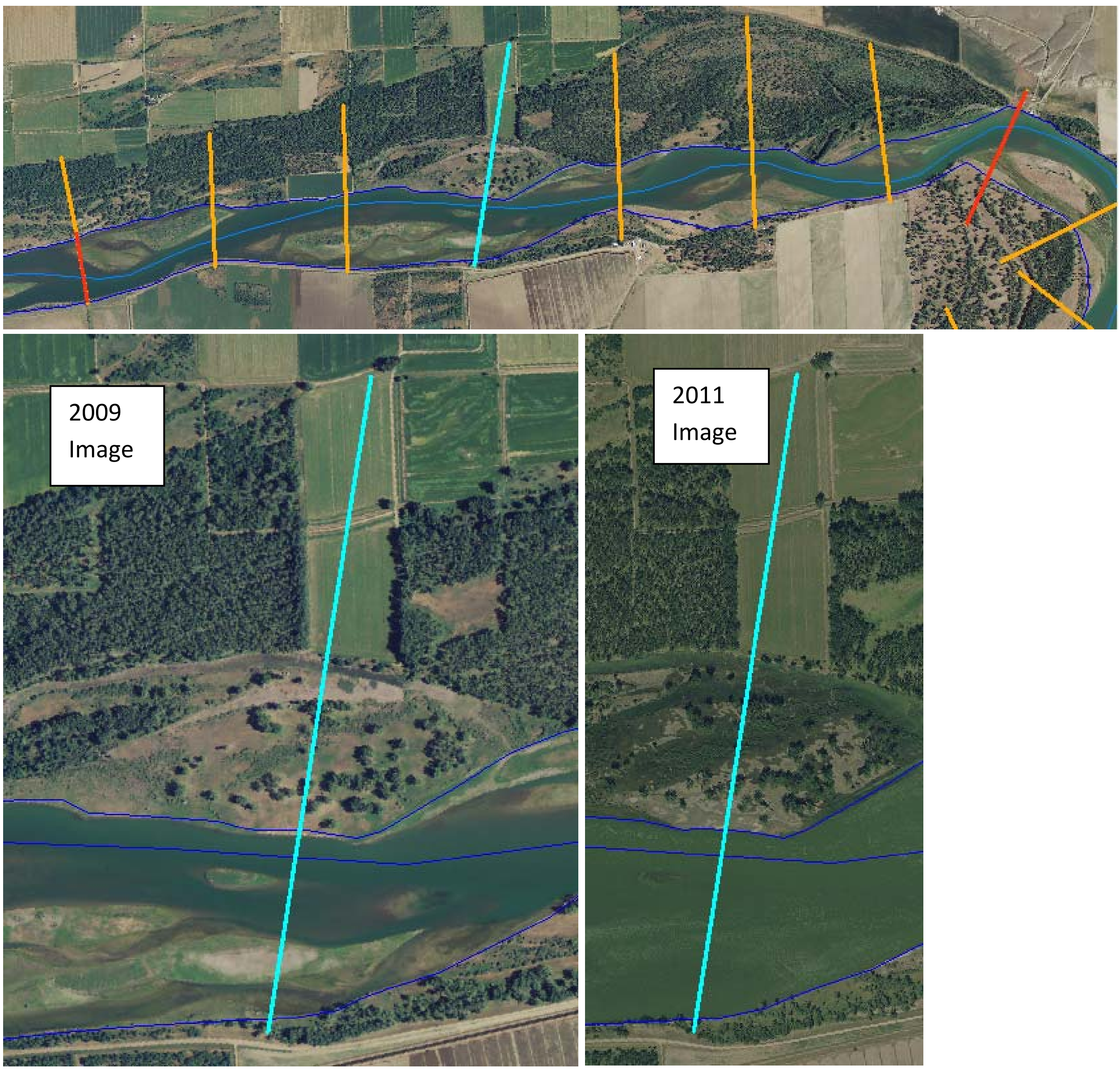


Figure 14. Aerial images of the XS, including the nearest Rangelines in the top image and closer view of the XS during a normal to low flow in 2009 and a high flow in 2011

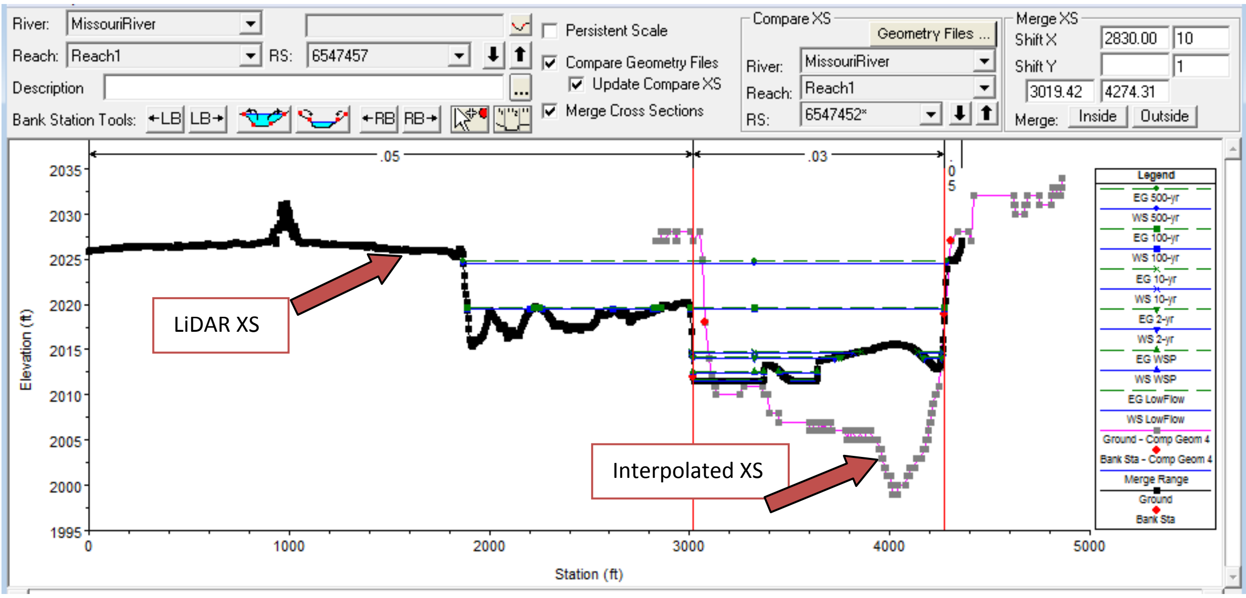


Figure 15. LiDAR XS (Black) and corresponding Interpolated XS (Grey/Pink) – note sandbars in channel

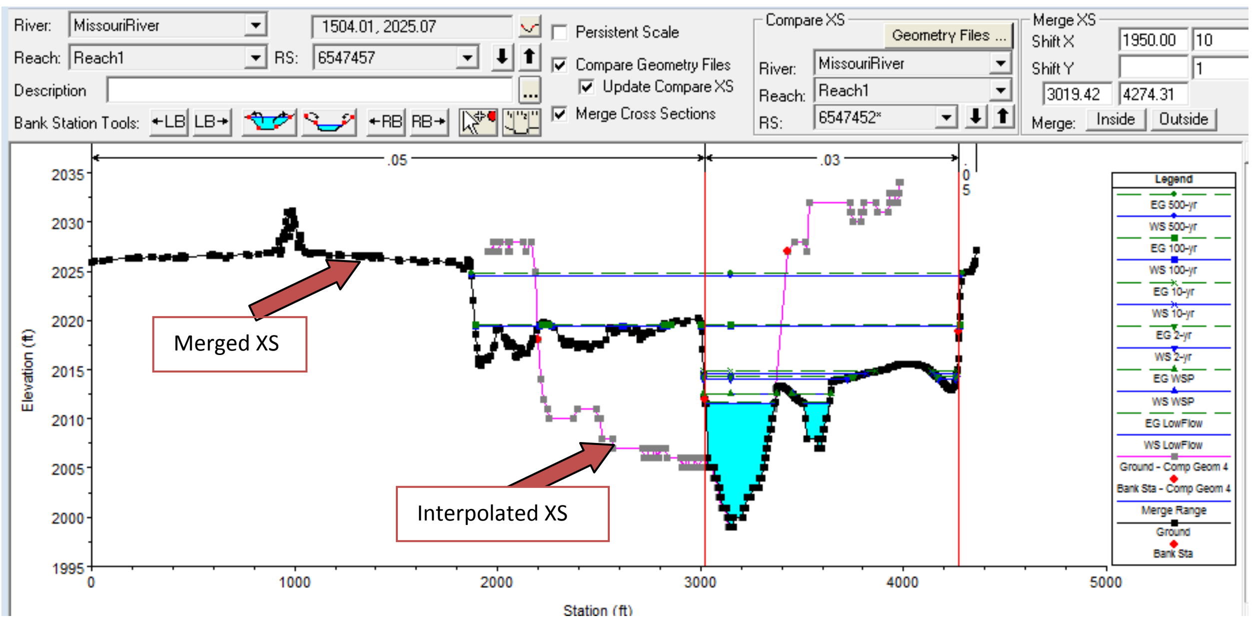
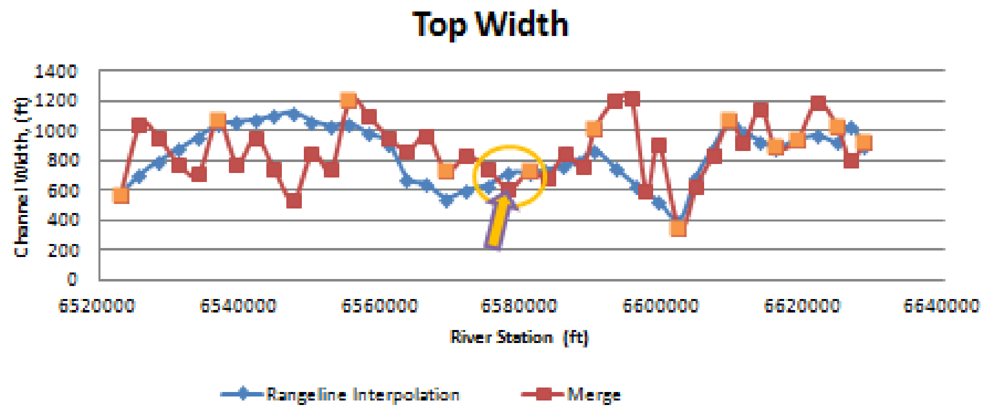


Figure 16. Merged XS (Black) and corresponding Interpolated XS (Grey/Pink)

Example 2



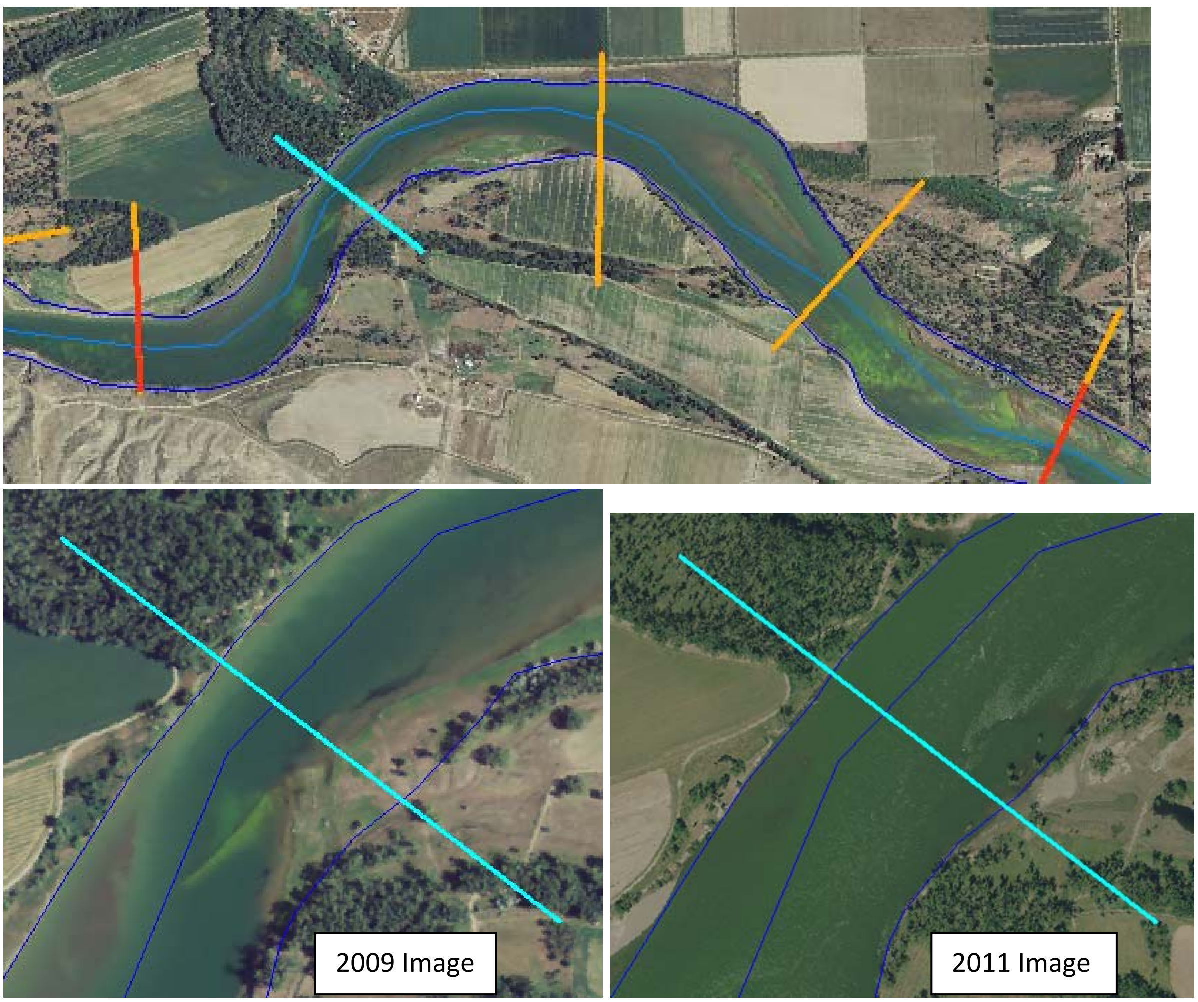


Figure 17. Aerial images of the XS, including the nearest Rangelines in the top image and closer view of the XS during a normal to low flow in 2009 and a high flow in 2011.

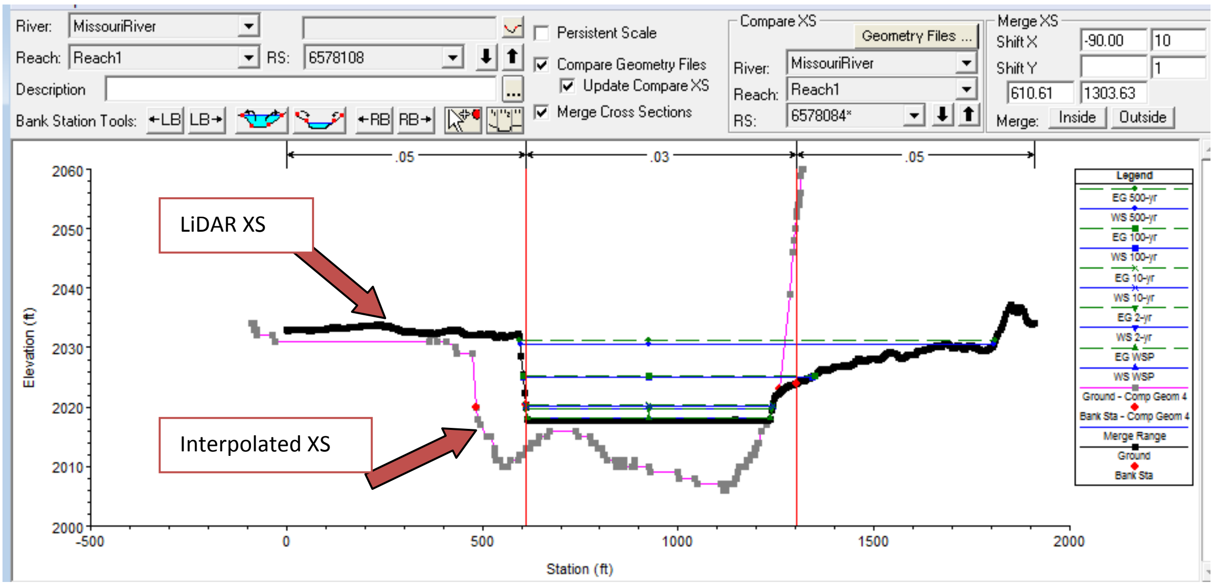


Figure 18. LiDAR XS (Black) and corresponding Interpolated XS (Grey/Pink)

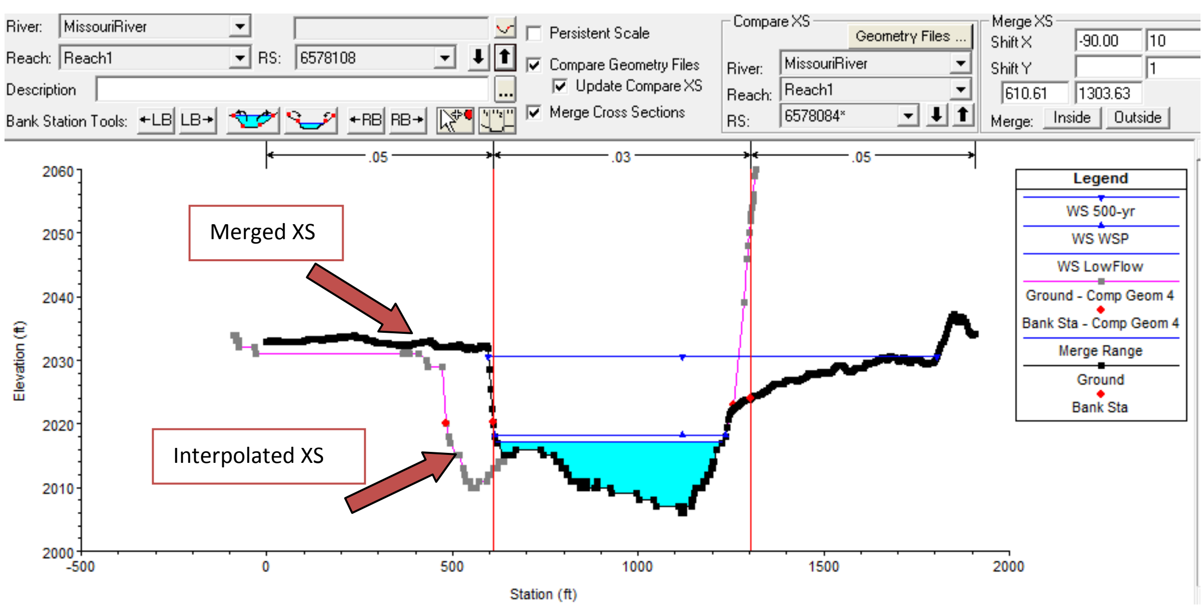
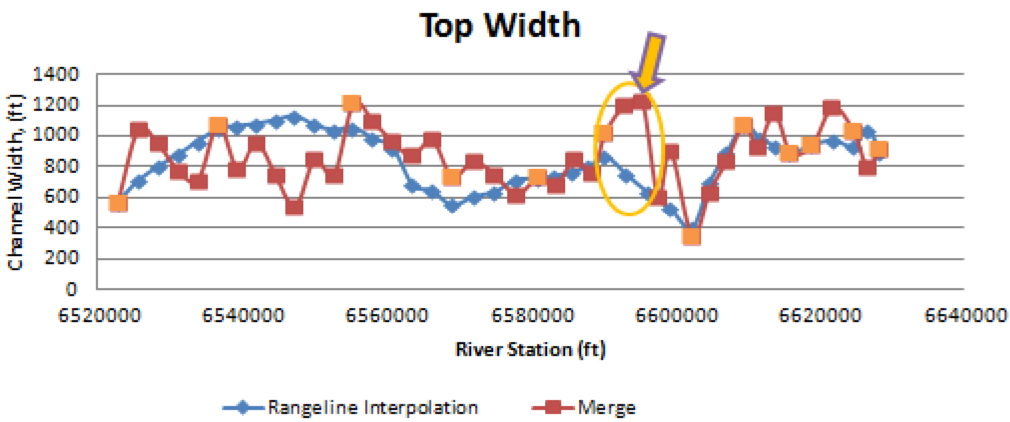
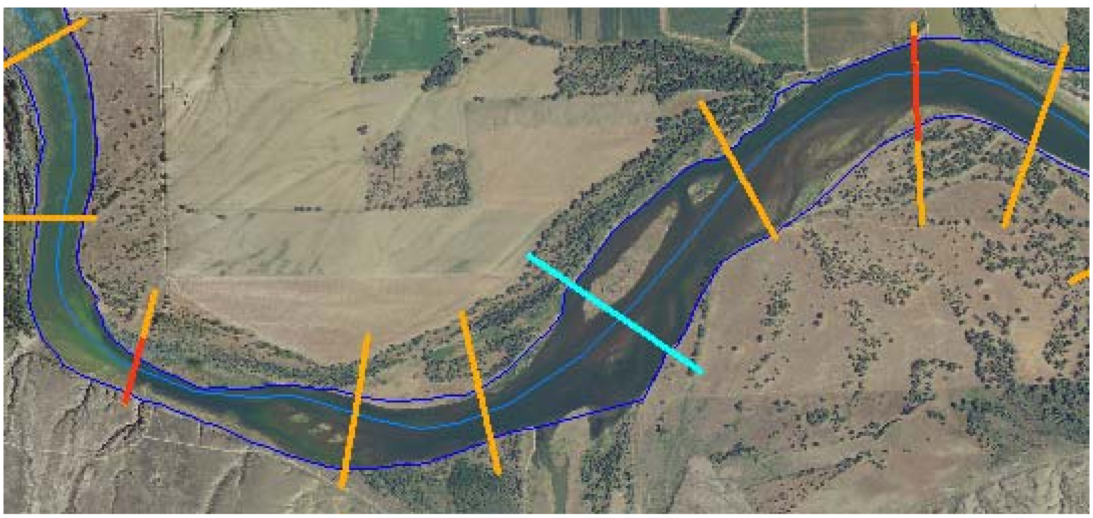


Figure 19. Merged XS (Black) and corresponding Interpolated XS (Grey/Pink)

Example 3

Used the downstream rangeline cross section with vertical adjustment to better fit cross section width





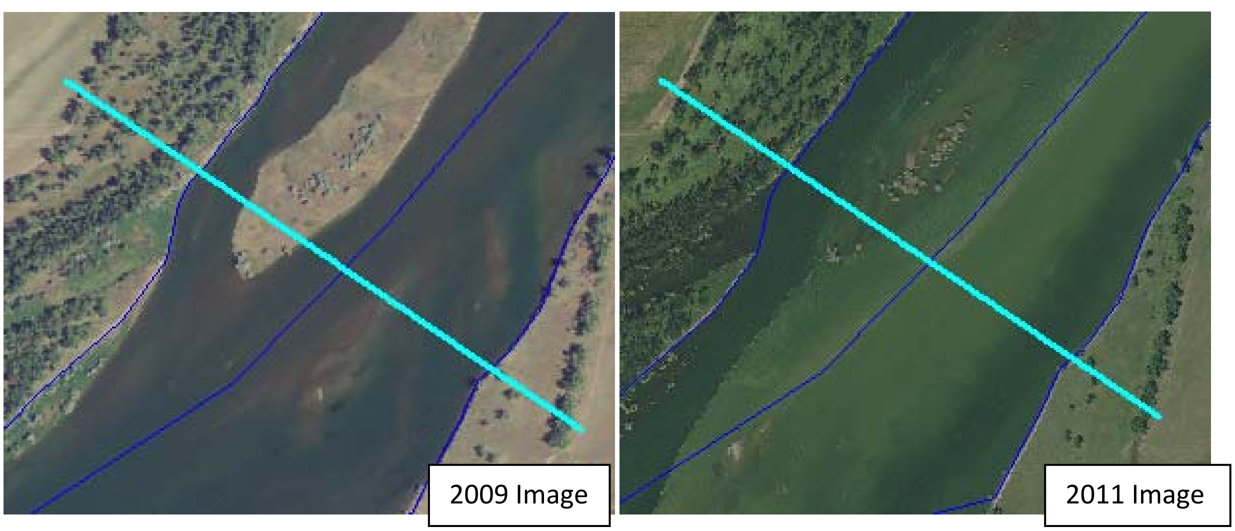


Figure 20. Aerial images of the XS, including the nearest Rangelines in the top image and closer view of the XS during a normal to low flow in 2009 and a high flow in 2011.

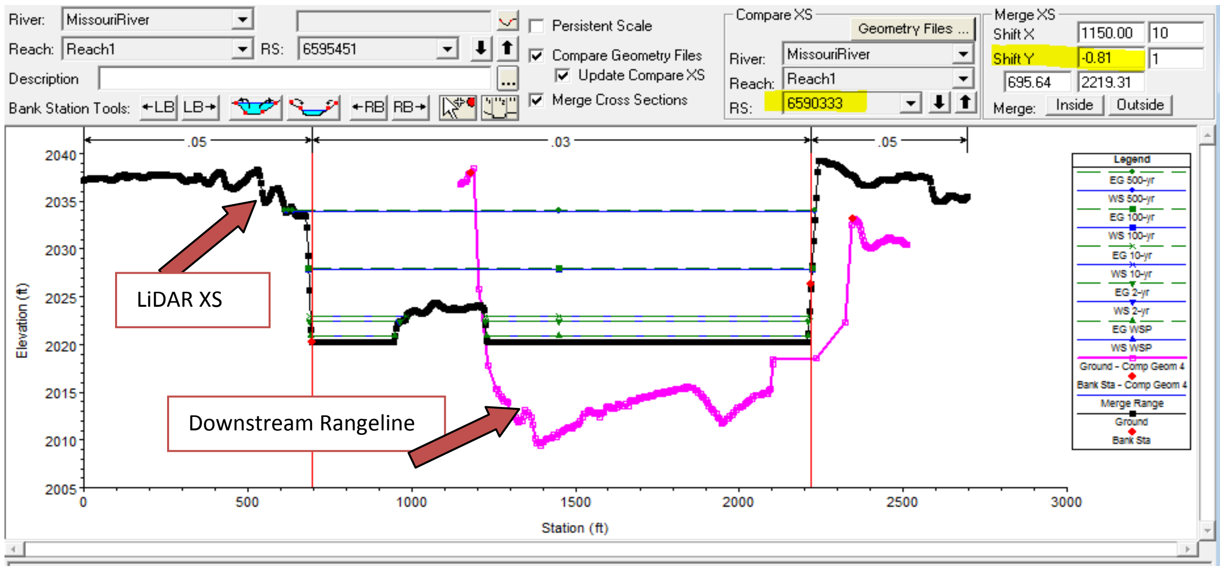


Figure 2. LiDAR XS (Black) and the next downstream Rangeline XS (Pink) vertically adjusted were used for merge due to the difference in channel width of the interpolated XS (see next figure).

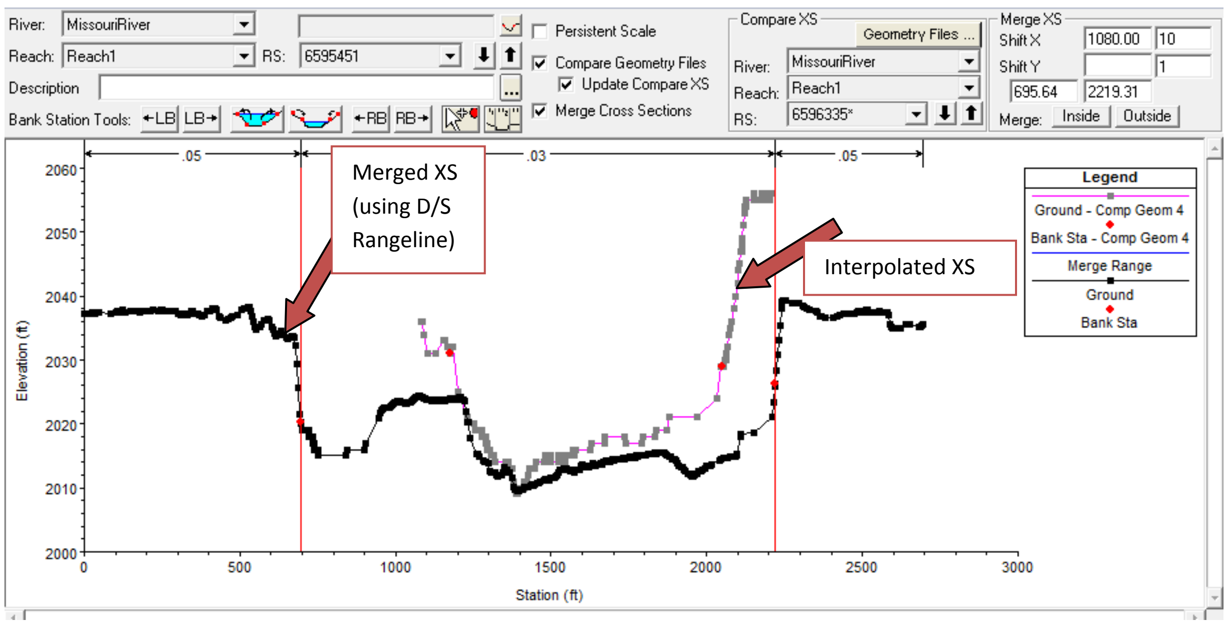
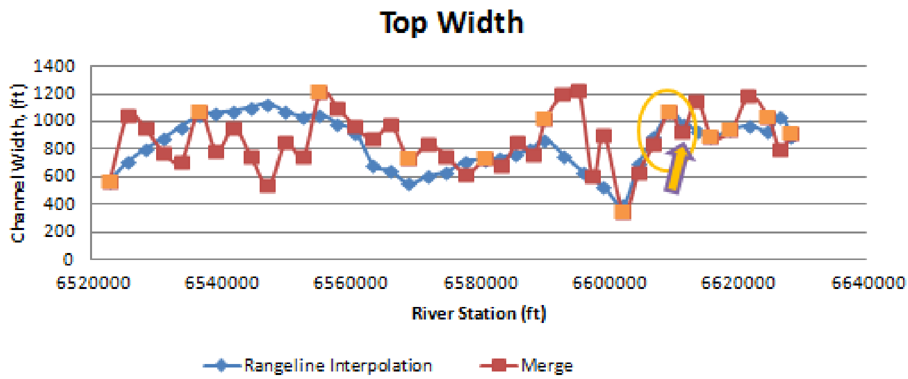


Figure 21. The final merged XS (Black) and the corresponding Interpolated XS (Pink/Grey) which was not used in this merge process.

Example 4



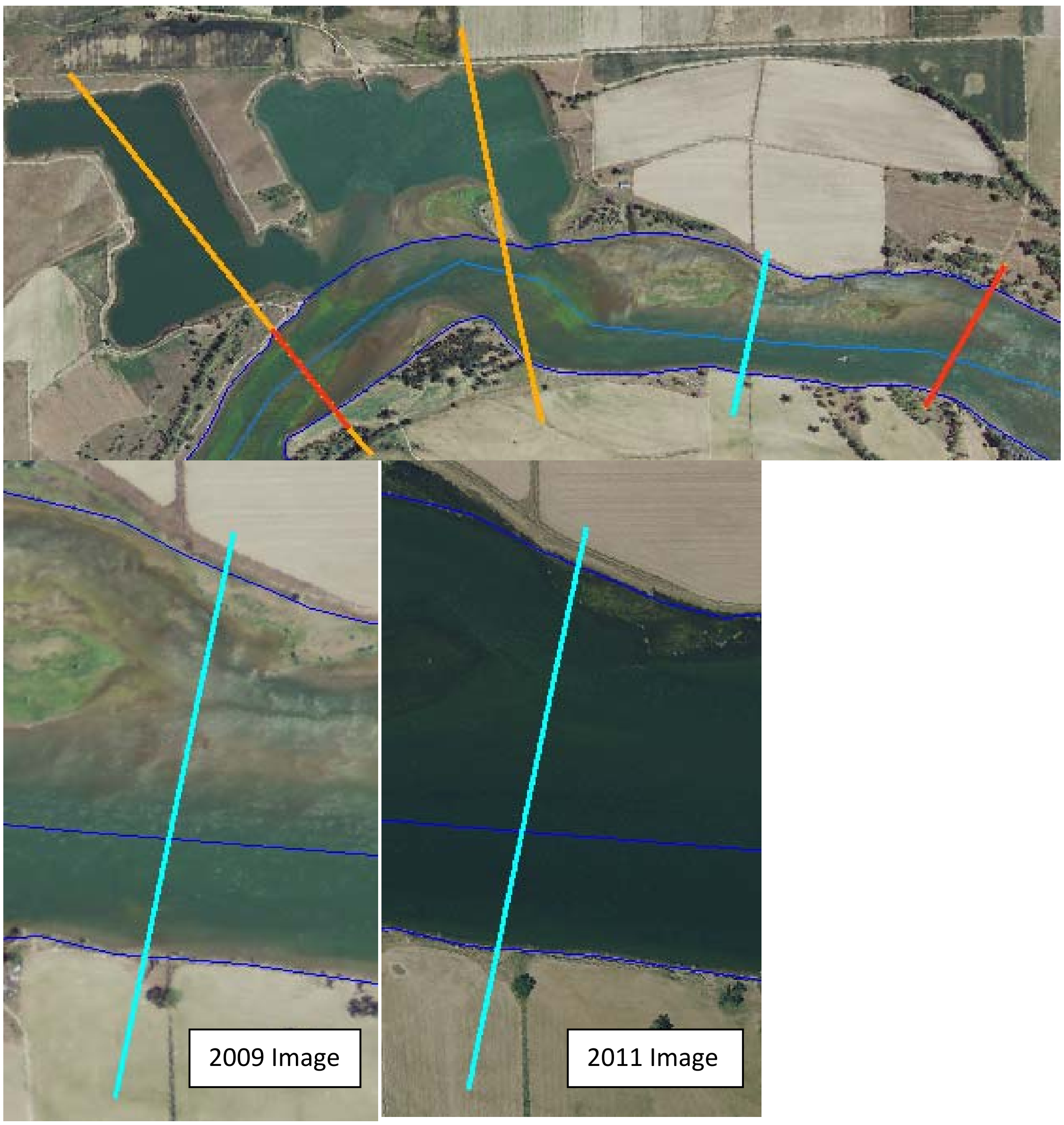


Figure 22. Aerial images of the XS, including the nearest Rangelines in the top image and closer view of the XS during a normal to low flow in 2009 and a high flow in 2011.

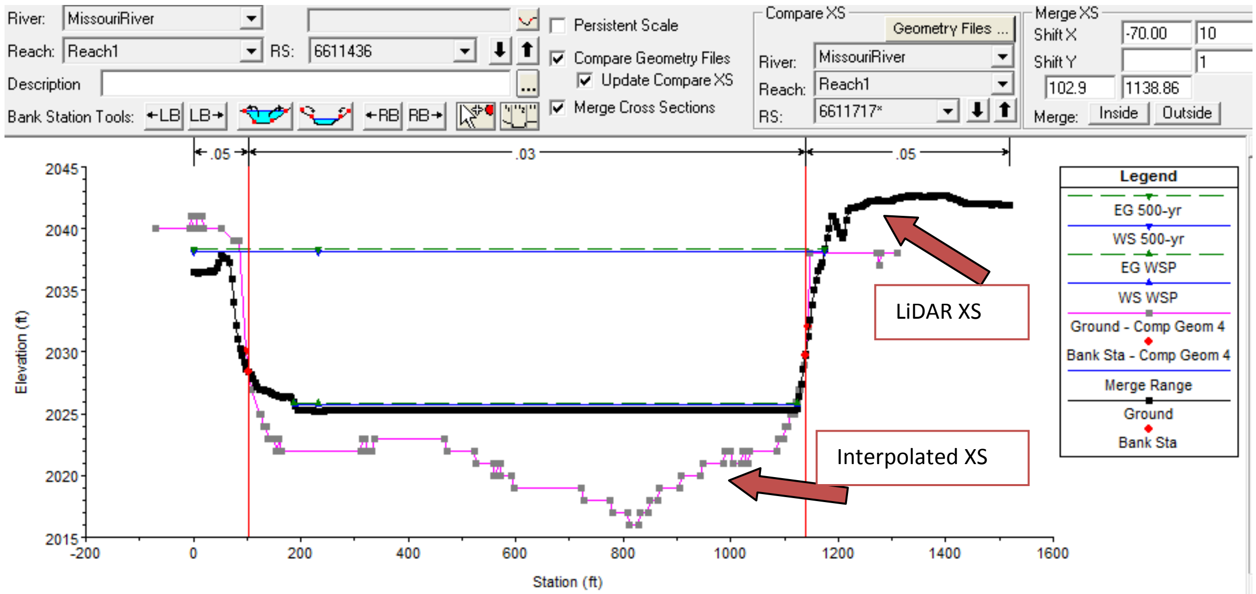


Figure 23. LiDAR XS (Black) and corresponding Interpolated XS (Grey/Pink) - XS had a good fit for channel width, note it is near a rangeline with little change between the Rangeline and XS locations.

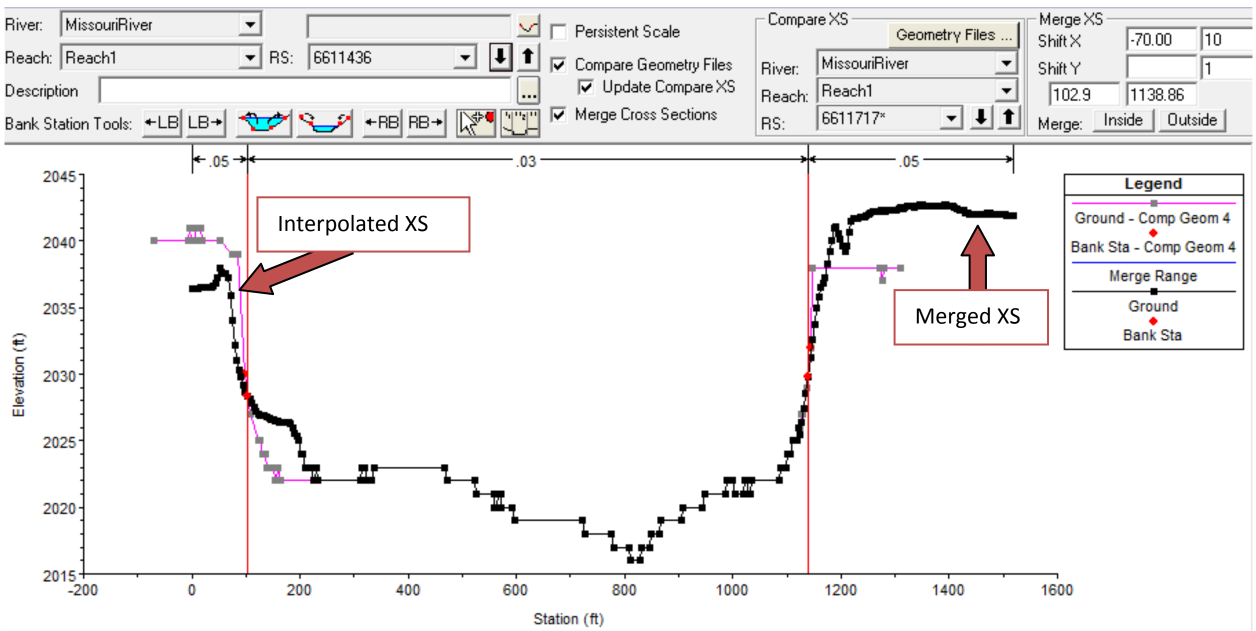


Figure 24. Merged XS (Black) and corresponding Interpolated XS (Grey/Pink).

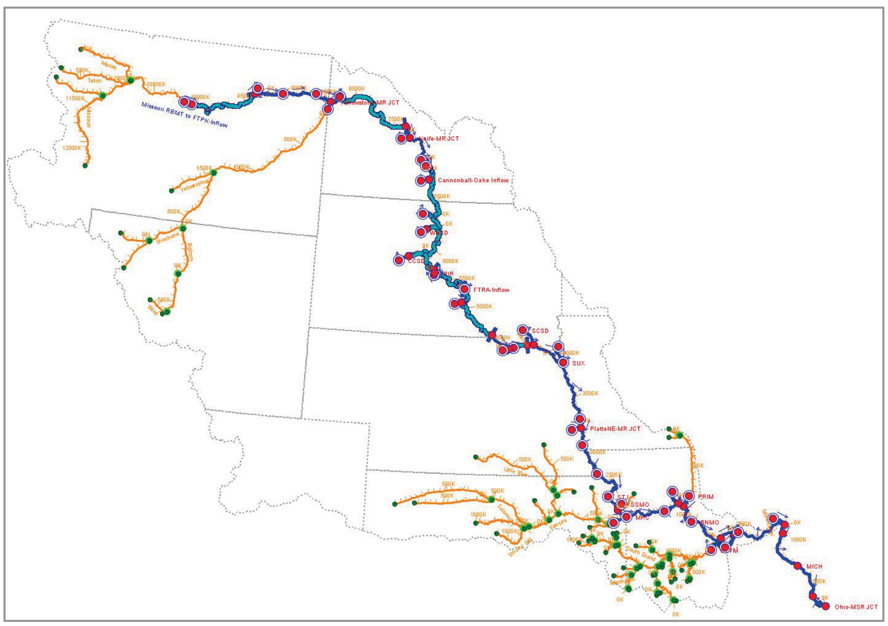
US Army Corps of Engineers ®

Omaha District

Missouri River Unsteady HEC-RAS Model
Calibration Report

Appendix C

Fort Randall Dam to Gavins Point Dam



July 2018

FINAL

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ATTACHMENTS

ACRONYMS

CFS…………………….. Cubic Feet per Second

DEM……………………. Digital Elevation Model

DTM .... Digital Terrain Model

DSSVue…………….. Data Storage System (by HEC)

GIS .... Geographic Information System

HEC .... Hydrologic Engineering Center

LiDAR………………..Light Detection and Ranging

MAF…………………..Million acre-feet

NAD 1983…………... North American Datum of 1983

NAVD 88……………. North American Vertical Datum of 1988

NGVD 29……………. National Geodetic Vertical Datum of 1929

MRBWM………….. .. Missouri River Basin Water Management Division (previously RCC)

NWK………………… Northwest Division Kansas City District

NWO………………… Northwest Division Omaha District

POR .... Period of Record

RAS........................... River Analysis System Software (by HEC)

RCC .... Reservoir Control Center

ResSim……………...Reservoir Simulation Software (by HEC)

RM……………………1960 River Mile

System……………....Missouri River Mainstem System

USACE……………… United States Army Corps of Engineers

USGS .... United States Geological Survey

1 EXECUTIVE SUMMARY

The Fort Randall to Gavins Point reach of the Missouri River begins with the regulated outflow from Fort Randall Dam in North Dakota. The reach then extends approximately 70 miles downstream, encompassing a watershed of approximately 279,480 square miles, to just upstream of Gavins Point Dam on Lewis and Clark Lake. This reach was modeled in Hydrologic Engineering Center River Analysis System (HEC-RAS) Version 4.2 Beta and with the intent to update to Version 5.0 when it is released. The model was initially created in steady flow and then completed with unsteady modeling, and is now fully unsteady.

Inputs into the model are a flow hydrograph for Fort Randall Dam's release, flow hydrographs for the upstream boundaries of the major tributaries (Ponca Creek, Niobrara River, Verdigre Creek(a Niobrara River tributary), and Bazile Creek), and a stage hydrograph for the Gavins Point Pool (Lewis and Clark Lake). Output includes stage and flow hydrographs, as well as a number of additional calculated parameters such as average velocities, flow depth, etc. The latest version of HEC-RAS also has the ability to create inundation depth grids at various time-steps using RAS Mapper that can be exported for use in ecological and economic models.

The geometry was constructed using the most recent sediment range surveys from the Omaha District, which included topographic and hydrographic data. Additional cross sections were added between the sediment ranges using LiDAR data for the overbanks and interpolation of the sediment ranges for the bathymetry where hydrographic data was unavailable. The flow and stage data were obtained from USGS gages. The observed Fort Randall releases and Gavins Point Pool elevations were obtained from the Omaha District CWMS database.

The model reach includes a substantial degradation reach that extends downstream from Fort Randall Dam and a large aggradation zone in the headwaters of Lewis and Clark Lake. The extreme 2011 flow event significantly altered the river stage-flow relationship and model calibration to observed stages in flood years prior to 2011 is not valid in most areas. Therefore, due to impacts from the 2011 flood and long term changes within the aggradation and degradation areas, the hydraulic model is not capable of reproducing observed stage-flow relationships prior to 2011.

The model was calibrated to the measured 2011 and 2012 Water Surface Profiles (WSP) and observed stage gage data for the Missouri River. The computed water surface profile was within 1 ft along the entire reach and in the range of +/- 0.5 ft for about 50% to 75% of the reach. These were determined to be acceptable calibration targets. Comparison to observed stage hydrographs indictated that the model performed acceptably on timing of flood peaks within most areas.

2 INTRODUCTION

The Missouri River unsteady HEC-RAS model was created as a base model for planning studies which could be used to simulate and analyze broad scale watershed alternatives. The objective of this HEC-RAS model is to simulate current conditions on the Missouri River, with the intention of running period of record (POR) flows to compare alternatives. Future reports will address period of record runs, this report addresses model construction and calibration. This Appendix is for the Fort Randall Dam to Gavins Point Dam reach of the Missouri River as part of the Omaha District.

3 BACKGROUND

The Fort Randall Dam to Gavins Point Dam reach is the third reach for the Omaha District's portion of the Missouri River. The model includes about 70 river miles of the Missouri River. One tributary routing reach is included to route flows from the USGS gage station location to the Missouri River.

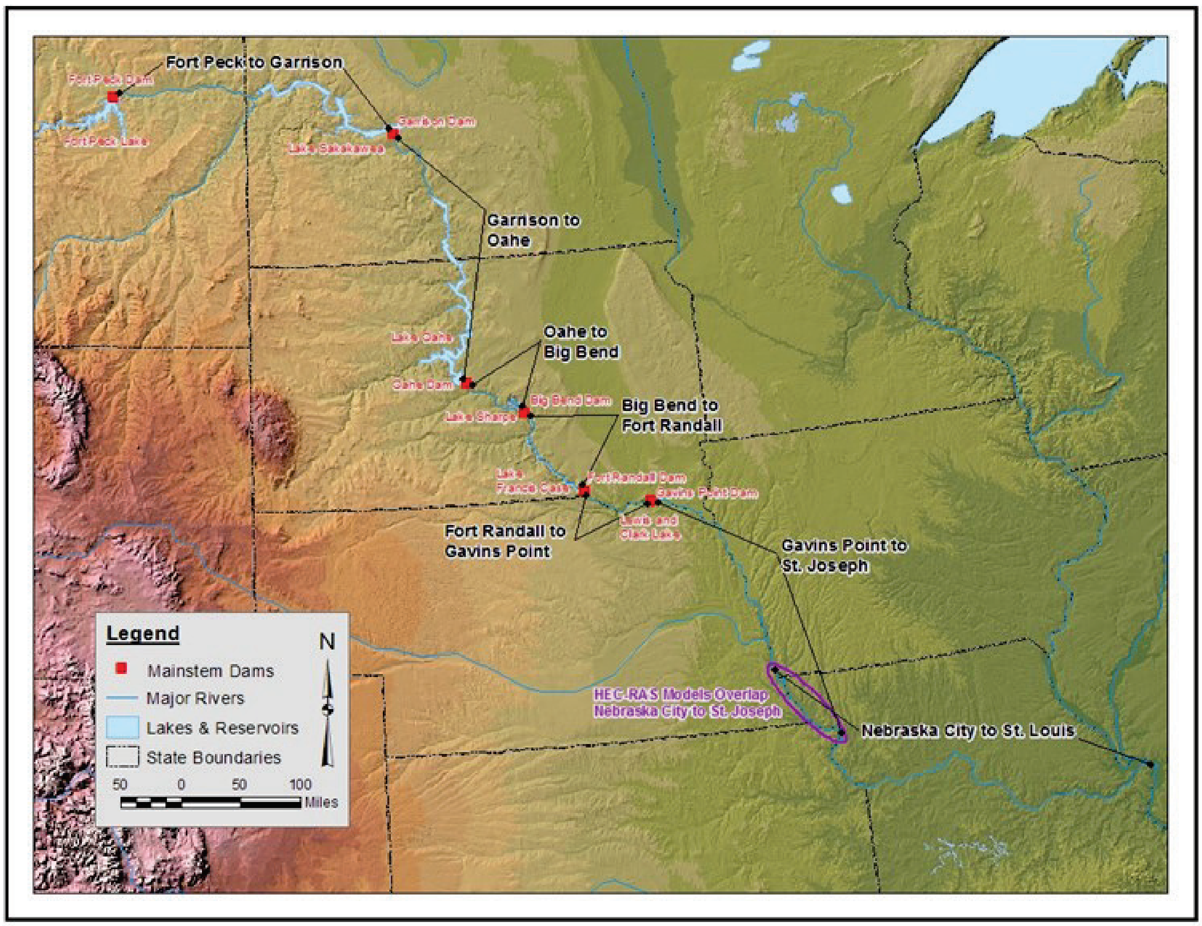

Figure 3-1: Model Extents

3.1 MODEL EXTENTS

This is the third portion of the Missouri River being modeled with HEC-RAS for the Omaha District, from River Mile (RM) 879.04, located just downstream of Fort Randall Dam in South Dakota, to RM 812.74, located just upstream of Gavins Point Dam in South Dakota and Nebraska, as shown in Figure 3-1. Upstream of this reach, the Fort Peck to Garrison and Garrison to Oahe reaches are being modeled (see Appendices A & B) and downstream of this reach, the Gavins Point to Rulo reach of the Missouri River is being modeled by Omaha District (see Appendix D) and the most downstream reach is being modeled by Kansas City District (see Appendix E).

3.2 MISSOURI RIVER MAINSTEM RESERVOIR SYSTEM DESCRIPTION

The Missouri River Mainstem System (System) of dams is composed of six large earth embankments which impound a series of lakes that extend upstream for 1,257 river miles from Gavins Point Dam near Yankton, South Dakota to the head waters of Fort Peck Lake north of Lewiston, Montana. These dams were constructed by the Corps of Engineers for flood control, navigation, power production, irrigation, water supply, water quality, recreation, and fish and wildlife enhancement. Fort Peck Dam, the oldest of the six dams, was closed and began water storage in 1937. Fort Randall Dam was closed in 1952, followed by Garrison Dam in 1953, Gavins Point Dam in 1955, Oahe Dam in 1958, and Big Bend Dam in 1963. The current System of six projects first filled and began operating as a six-project System in 1967. At the top elevation of their normal operating pool level, the lakes behind these six dams provide about 1,146,000 acres of water surface area and extend a total length of 755 river miles. Only 325 miles of open river remain between the lakes, although there are 811 miles of open river downstream from Gavins Point Dam to the mouth of the Missouri River where it enters the Mississippi River at St. Louis, Missouri. The reservoirs contain an aggregate storage volume of approximately 73 million acre-feet (MAF) of which more than 16 MAF is for flood control.

Regulation of the System is according to the current Master Manual (USACE, 2006) and generally follows a repetitive annual cycle. Winter snows and spring and summer rains produce most of the year’s water supply, which results in rising pools and increasing storage accumulation. After reaching a peak reservoir level, usually during July, storage declines until late winter when the cycle begins anew. A similar pattern may be found in rates of releases from the System, with higher flows from mid-March to late November, followed by low rates of winter discharge from late November until mid-March, after which the cycle repeats.

Two primary high-risk flood seasons are the plains snowmelt season extending from late February through April and the mountain snowmelt period extending from May through July. Overlapping the two snowmelt flood seasons is the primary rainfall flood season, which includes both upper and lower basin regulation considerations.

Power generation is a component of System operation. The highest average power generation period extends from mid-April to mid-October with high peaking loads during the winter heating season (mid-December to mid-February) and the summer air conditioning season (mid-June to mid-August). The power needs during winter are supplied primarily with Fort Peck and Garrison releases and the peaking capacity of Oahe and Big Bend. During the spring and summer periods, releases are geared to navigation and flood control requirements and primary power loads are supplied using the four lower dams. During the fall when power needs diminish, Fort Randall pool is drawn down to permit generation during the winter period when the pool is refilled by Oahe and Big Bend peaking power releases. Gavins Point Dam, as the downstream-most reservoir, is operated at constant daily releases and is not used for daily power peaking.

Normally, the navigation season extends from April 1 through December 1 during which time reservoir releases are increased to meet downstream target flows in combination with downstream tributary inflows. Winter releases after the close of navigation season are much lower and vary depending on the need to conserve or evacuate system storage volumes, downstream ice conditions permitting. Minimum release restrictions and pool fluctuations for fish spawning management generally occur from April 1 through July. Endangered and threatened species, including the interior least tern and piping plover, nesting occurs from early May through August. During this period, special release patterns are made from Garrison, Fort Randall, and Gavins Point to avoid flooding nesting sites on low-lying sandbars and islands downstream from these projects.

Overall, the general regulation principles presented above provide the backbone philosophy for the Mainstem System regulation. Detailed operation plans are developed, followed and adjusted as conditions warrant periodically as the System is monitored day-to-day. Beginning in 1953, projected operation of the Missouri River Mainstem Reservoir System for the year ahead was developed annually as a basis for advance coordination with the various interested Federal, State, and local agencies and private citizens. These regulation schedules are prepared by the Missouri River Basin Water Management Division, Northwest Division, Corps of Engineers and are reported in Annual Operating Plans (USACE, 2013b).

In addition to the six main stem projects operated by the Corps, 65 tributary reservoirs operated by the Bureau of Reclamation and the Corps provide over 15 million acre-feet of flood control storage.

Numerous reservoirs and impoundments constructed by different interests for flood control, irrigation, power production, recreation, water supply, and fish and wildlife are located throughout the basin on various tributaries. The Bureau of Reclamation and the Corps of Engineers have constructed the most significant of these structures. Although primarily constructed for irrigation and power production, the projects constructed by the Bureau of Reclamation do provide some limited flood control in the upper basin.

Table 3-1 lists pertinent data for the Missouri River Mainstem projects (USACE, 2013a).

Table 3-1: Pertinent Data for Missouri River Mainstem Projects

Description	Fort Peck	Garrison	Oahe	Big Bend	Fort Randall	Gavins Point
River Mile (1960 Mileage)	1771.5	1389.9	1072.3	987.4	880.0	811.1
Drainage Area (sq. mi.)	57,500	181,400	243,490	249,330	263,480	279,480
Incremental Drainage Area (sq. mi.)	57,500	123,900	62,090	5,840	14,150	16,000
Gross Storage (kAF)	18,463	23,451	22,983	1,798	5,293	428
Flood Storage (kAF)	3,675	5,706	4,315	177	2,293	133
Top of Dam* (ft NGVD29 (NAVD88))	2280.5 (2282.6)	1875.0 (1876.3)	1660.0 (1661.2)	1440.0 (1441.1)	1395.0 (1396.0)	1234.0 (1234.7)
Maximum Surcharge Pool** (ftNGVD29 (NAVD88))	2253.3 (2255.4)	1858.5 (1859.8)	1644.4 (1645.6)	1433.6 (1434.7)	1379.3 (1380.3)	1221.4 (1222.1)
Top of Exclusive FC Pool*** (ft NGVD29 (NAVD88))	2250.0 (2252.1)	1854.0 (1855.3)	1620.0 (1621.2)	1423.0 (1424.1)	1375.0 (1376.0)	1210.0 (1210.7)
Top of Annual FC Pool (ft NGVD29 (NAVD88))	2246.0 (2248.1)	1850.0 (1851.3)	1617.0 (1618.2)	1422.0 (1423.1)	1365.0 (1366.0)	1208.0 (1208.7)
Base of Flood Control Pool (ft NGVD29 (NAVD88))	2234.0 (2236.1)	1837.5 (1838.8)	1607.5 (1608.7)	1420.0 (1421.1)	1350.0 (1351.0)	1204.5 (1205.2)
Spillway Capacity (cfs)	275,000	827,000	304,000	390,000	633,000	584,000
Outlet Capacity (cfs)	45,000	98,000	111,000	n/a	128,000	n/a
Powerplant Capacity (cfs)	16,000	41,000	54,000	103,000	44,500	36,000
Date of Closure	Jun 1937	Apr 1953	Aug 1958	Jul 1963	Jul 1952	Jul 1955

*Operational elevations are referenced to the NGVD29 datum. They were converted to NAVD88 using CorpsCon conversion factors for use with model elevations.
**Maximum pool elevation with spillway gates opened.
***Maximum pool elevation with spillway gates closed.

3.3 Fort Randall and Gavins Point Dam and Reservoir Information

3.3.1 *Fort Randall Dam and Lake Francis Case*

Fort Randall Dam is located on the Missouri River at RM 880.0, about 6 miles south of Lake Andes, South Dakota. Lake Francis Case extends to Big Bend Dam. Construction of the project was initiated in August 1946, closure was made in July 1952, and initial power generation began in March 1954. The 39-mile reach of the Missouri River from Fort Randall Dam to Running Water, South Dakota has been designated a National Recreational River under the National Wild and Scenic Rivers Act. The total drainage area of the Missouri River at Fort

Randall Dam is 263,480 sq. miles. Table 3-2 through Table 3-4 show the historical releases and release-duration and release-probability relationships for Fort Randall Dam (USACE, 2013a).

Table 3-2: Fort Randall Release Historical Records (1967-2011)

Month	Daily Release (cfs)		
	Maximum	Minimum	Mean
Jan	27,600	4,500	15,100
Feb	25,400	900	13,300
Mar	40,500	500	15,500
Apr	53,000	1,400	21,200
May	76,600	0	25,400
Jun	155,300	500	29,100
Jul	160,000	600	33,300
Aug	149,500	3,100	35,600
Sep	90,200	8,600	34,800
Oct	67,000	3,200	32,300
Nov	67,500	3,200	28,700
Dec	63,000	900	17,400
Annual	**160,000**	**0**	**25,200**

Table 3-3: Fort Randall Release-Duration Relationship

Percent of Time Equaled or Exceeded	Release (cfs)	
	Annual	May – Aug
Maximum	160,000	160,000
1	65,700	145,000
5	49,500	52,000
10	42,000	45,600
20	33,500	36,700
50	24,000	28,200
80	13,500	21,900
90	9,800	17,000
95	7,100	11,600
99	2,900	2,600
100	0	0

Table 3-4: Fort Randall Release-Probability Relationship

Annual Percent Chance Exceedance	Release (cfs)
50	45,000
20	50,000
10	56,000
2	84,000
1	100,000
0.2	160,000

3.3.2 Gavins Point Dam and Lewis and Clark Lake

Gavins Point Dam is located on the Missouri River at RM 811.1 on the Nebraska-South Dakota border, 4 miles west of Yankton, South Dakota. Lewis and Clark Lake extends 37 miles to the vicinity of Niobrara, Nebraska. Construction of the project was initiated in 1952, closure was made in July 1955, and intial power generation began in September 1956. The total drainage area of the Missouri River at Gavins Point Dam is 279,480 sq. miles. The incremental drainage area between Fort Randall Dam and Gavins Point Dam is 16,000 sq. miles. Table 3-5 through

Table 3-7 show the historical pool elevations and pool-duration and pool-probability relationships for Gavins Point Dam (USACE, 2013a).

Table 3-5: Gavins Point Pool Historical Records (1967-2011)

Month	Pool Elevation (ft NGVD29)		
	Maximum	Minimum	Mean
Jan	1208.9	1204.1	1207.4
Feb	1209.2	1203.2	1206.9
Mar	1209.2	1199.8	1205.6
Apr	1208.2	1201.5	1205.8
May	1209.5	1204.2	1205.8
Jun	1209.7	1204.3	1206.0
Jul	1208.9	1204.4	1206.5
Aug	1209.4	1204.7	1207.0
Sep	1208.8	1203.9	1207.5
Oct	1209.2	1206.4	1207.7
Nov	1209.0	1204.5	1207.7
Dec	1209.1	1203.9	1207.4
Annual	**1209.7**	**1199.8**	**1206.8**

Table 3-6: Gavins Point Pool-Duration Relationship

Percent of Time Equaled or Exceeded	Pool Elevation (ft NGVD29)	
	Annual	May – Aug
Maximum	1209.7	1209.7
1	1208.5	1208.4
5	1208.2	1208.0
10	1208.1	1207.8
20	1207.9	1207.2
50	1206.9	1206.2
80	1205.7	1205.4
90	1205.2	1205.1
95	1205.0	1205.0
99	1204.4	1204.7
100	1199.8	1204.2

Table 3-7: Gavins Point Pool-Probability Relationship

Annual Percent Chance Exceedance	Pool Elevation (ft NGVD29)
50	1208.8
20	1209.0
10	1209.2
2	1209.6
1	1210.0
0.2	1211.0*

* Extrapolated: Max observed is 1209.7 ft NGVD29.

3.3.3 Survey History

Degradation and aggradation surveys are an integral part of the Omaha District's sediment data collection program. The survey work requires the periodic resurvey of the land surface and riverbed cross sections between permanently established survey control points called sediment ranges. There are 21 sediment ranges spaced an average of 1.2 miles apart below Fort Randall Dam. There are 32 main stem sediment ranges spaced an average of 1.3 miles apart at Lewis and Clark Lake. Table 3-8 below provides a summary of the Fort Randall degradation and Gavins Point aggradation reaches. The break between survey ranges between the degradation and the aggradation reach is not representative of where degradation/aggradation is occurring but the point where the maximum pool elevation of Lewis and Clark Lake intersects the Missouri River thalweg profile.

Table 3-8: Sediment Range Information

Fort Randall Degradation Reach – Fort Randall Dam to Lewis and Clark Lake					
Fort Randall Dam River Mile (1960 RM)	Ending River Mile (1960 RM)	Reach Length (miles)	No. of Main Stem Sediment Ranges	Average Spacing of Ranges (miles)	Most Recent Survey Year
880.00	855.26	24.74	21	1.2	2011
Gavins Point Aggradation Reach – Lewis and Clark Lake					
Beginning River Mile (1960 RM)	Gavins Point Dam River Mile (1960 RM)	Reach Length (miles)	No. of Main Stem Sediment Ranges	Average Spacing of Ranges (miles)	Most Recent Survey Year
853.37	811.05	42.32	32	1.3	2011

3.4 Reach Characteristics

The upstream end of the reach begins immediately downstream of Fort Randall Dam. The reach then extends approximately 69 miles downstream, encompassing a watershed of approximately 279,480 square miles, to just upstream of Gavins Point Dam on Lewis and Clark Lake, near Yankton, South Dakota, as shown in Plate 1.

This reach of the Missouri River flows through mostly agricultural land and sparsely populated areas. Niobrara, Nebraska and Springfield, South Dakota are the largest cities located near the Missouri River in this reach.

In addition to the modeling of the Missouri River, there is one tributary modeled in HEC-RAS. The Niobrara River model extends approximately 15 miles upstream from the confluence with the Missouri River to near Verdel, NE. The Niobrara River watershed is approximately 12,000 square miles.

3.5 Degradation and Aggradation Trends

During the development of the Missouri River basin projects, significant change has occurred in channel conveyance as a result of aggradation and degradation. Missouri River natural variability and construction including flood control projects, channel cutoffs, channel and bank stability projects have all contributed to conveyance change. The release of essentially sediment-free water through the System dams has resulted in a lowering of the tailwater elevation. Two types of sediment deposits exist in the reservoirs: those occurring generally over the reservoir bottom, mostly composed of the finer material and those occurring in a characteristic delta formation at the head of the reservoir and where tributaries enter the reservoir, which include coarser material.

3.5.1 Degradation Trends – Downstream of Fort Randall Dam

Degradation in the reach downstream of Fort Randall Dam has been evaluated in a series of studies (USACE, 2012a, 2012b). Degradation begins at Fort Randall Dam and gradually decreases in magnitude in the downstream direction to approximately RM 860, or just downstream of the Missouri River below Greenwood, SD gage, a distance of about 20 river miles. A transition zone extends from about RM 860 to RM 854. At Fort Randall Dam tailwater, degradation of about 6 to 8 feet has been observed since dam closure (1952) at normal flows of around 30,000 cfs. The historic 2011 flood and period of sustained high flows led to degradation throughout the reach. Near the downstream end of the degradation reach, at the Greenwood, SD gage, a normal flow decrease of 1 to 2 feet was observed during 2011.

3.5.2 Aggradation Trends – Lewis and Clark Lake Headwaters

A trend of aggradation has been seen in the reach below RM 855 and increases in the downstream direction. The Niobrara River enters the Missouri River at about RM 844.5. Due to the proximity of the mouth of the Niobrara River and its resulting delta formation to Lewis and Clark Lake, it is hard to differentiate between sources of sediment in this area. The 2011 sustained high flows caused a larger than normal amount of sediment to enter the reservoir as well as erosion and movement of sediment that had already been deposited. From 2009 to 2012, the visible delta front moved forward about 3.5 times faster than had been previously calculated, to about RM 826. Delta movement rates were calculated using aerial imagery. The storage capacity of Lewis and Clark Lake decreased by approximately 149,000 acre-feet, or 26 percent, from 1955 to 2011. As a result of the flooding in 2011, the rate of depletion between 2007 and 2011 was more than twice the longer term depletion rate between 1955 and 2011 (USACE, 2013c).

3.6 FLOOD HISTORY

In the upper Missouri River, the largest flood prior to the construction of the System was the flood of 1952. Flooding was continuous from the Yellowstone River to the mouth due to flooding on most of the tributaries above Sioux City. The winter of 1951-52 had one of the heaviest snow covers in the upper plains with a high water content and an unusually cold winter. In late March, rapid melting of snow cover began. The Missouri River crested at Fort Randall on April 12 at 447,000 cfs and Yankton, South Dakota on April 13 at 480,000 cfs.

Since the System first filled in 1967, the largest flood event was in 2011. During 2011, a record amount of runoff occurred due to melting snowpack and record rainfall over portions of the upper basin. Annual runoff into the System was estimated to be 60.8 MAF. As a result of the record runoff, record releases from all of the System dams occurred: 65,000 cfs at Fort Peck, 150,000 cfs at Garrison, 160,000 cfs at Oahe, 166,000 cfs at Big Bend, 160,000 cfs at Fort Randall, and 160,000 cfs at Gavins Point.

The Missouri River basin experiences numerous events with ice jams and ice covered river conditions. These events often result in much higher stages than would normally occur for an open water condition. Ice affected events typically occur in the early spring, usually in the March to April time frame, with ice cover, ice breakup, potential ice jams, snowmelt runoff and precipation events all contributing to spring event flows and stages.

4 DATA SOURCES

Primary data sources for construction of the unsteady HEC-RAS model includes terrain data, bathymetry data, and gage data. Terrain data encompasses everything from the bluffs to the riverbanks, defining the floodplain and overbanks, but does not often include data below the surface of the river. Bathymetry captures the cross section geometry below the water surface. Gage data provides the flow boundary conditions for the model and stage calibration targets. A summary of the data used in the model is provided in Table 4-1.

Table 4-1: Summary of Data Sources

Data Type	Data Title	Location Data Applied to Model	Collection Dates
Topographic Data			
Sediment Range Survey	Fort Randall Degradation Sediment Range Surveys (Eisenbraun & Associates, Inc.)	RM 879.04 – 855.37	Oct – Nov 2011
Sediment Range Survey	Gavins Point Aggradation Sediment Range Survey (Eisenbraun & Associates, Inc.)	RM 853.26 – 812.74	Oct – Dec 2011
Sediment Range Survey	Gavins Point Aggradation Sediment Range Survey – New Delta Ranges (Eisenbraun & Associates, Inc.)	RM 825.06 – 820.21	Aug 2012
DEM – LiDAR	Fort Randall to Niobrara LiDAR Mapping	Fort Randall Dam – RM 834.83, RM 827.52 – 826.04	1 Dec 2011 – 21 Mar 2012
DEM – 4 m	NEXTMap	Some Overbank: Fort Randall Dam – RM 834.83, All: RM 834.83 – Gavins Point Dam	May – Oct 2007
Land Cover			
Land Cover	National Land Cover Dataset 2006	All cross sections	2006
Flow Data			
Streamgage Data	Stage and Discharge	All cross sections	POR
Hydrologic Statistics	Release and Pool Duration for Garrison and Lake Oahe	All cross sections	POR
Water Surface Profile			
Water Surface Elevation Data	Missouri River Water Surface Profile from Fort Randall Dam to Lewis and Clark Lake	All cross sections	25 Jun 2012 and 19 Jul 2011

4.1 TERRAIN DEVELOPMENT

A variety of terrain sources were available for this stretch of the Missouri River and floodplain. Described below are the sources, dates, and accuracy of each.

4.1.1 Sediment Range Surveys

Sediment range surveys for the main stem Missouri River from Fort Randall Dam to Gavins Point Dam were collected in two surveys: Fort Randall degradation and Gavins Point Aggradation in 2011 and newly added delta ranges in 2012, both by Eisenbraun and Associates, Inc. See Table 4-1 for information on where each survey's data was used in the model. The sediment range surveys include topographic and hydrographic data.

4.1.2 DEMs and LiDAR

Two DEM data sets were available for this stretch of the Missouri River. The first was a 5-ft cell size GRID, LiDAR, Digital Elevation Model (DEM) collected from 1 Dec 2011 to 21 Mar 2012 extending from Fort Randall Dam to below Niobrara, Nebraska and another area between RM 827.52 and RM 826.04. The horizontal and vertical accuracies are 1.25 ft RMSEr and 0.14 ft RMSEz, respectively.

Also, the NEXTMap 4-meter DEM was available that was collected from May through October 2007 by Intermap Technologies. This data set is available for the entire Omaha District. The LiDAR data did not completely cover the extents of the cross sections, so the NEXTMap data was used in the overbanks and downstream of where the LiDAR stopped, mostly in the lake. The horizontal accuracy is 2 meters RMSE or better in areas of unobstructed flat ground. The vertical accuracy is 1 meter RMSE or better in areas of unobstructed flat ground.

4.1.3 Land Cover

The United States Geographical Survey (USGS) National Land Cover Database 2006 (NLCD 2006) was used in the determination of appropriate Manning's n roughness values for overbank data. The NLCD 2006 is a 16-class land cover classification scheme at a spatial resolution of 30 meters and is based primarily on a 2006 Landsat satellite data. This is a raster digital data set (USGS, 2012).

4.2 BATHYMETRY

The bathymetry data available was a part of the sediment range survey data described in Section 4.1.1. This hydrographic data was collected in two surveys: Fort Randall degradation and Gavins Point aggradation in 2011 and new delta ranges in 2012, both by Eisenbraun and Associates, Inc.

4.3 OBSERVED DATA

Water surface profiles are surveys periodically performed by the Omaha District Corps of Engineers that provide a water surface elevation for a reach, usually collected approximately every 1 river mile. Stream stage and flow data available on the Missouri River include gages along the Missouri River main stem, and gages on many of the major tributaries. All gages are operated by the USGS and collect stage data remotely, usually at intervals of 15 minutes. Availability and quality of these datasets influenced the configuration of the model as well as the timeframe for calibration.

4.3.1 Water Surface Profile Data

Water surface profile elevation data was collected on 19 July 2011 and 25 June 2012. Water surface elevations are collected approximately every river mile. This data was used as the baseline for calibration of the model.

4.3.2 USGS Gage Flow and Stage Data

Stream gage data was obtained through the USGS National Water Information System (NWIS) or, if not available online, from each state's USGS Water Science Center personnel for all applicable gages in this reach of the Missouri River and tributaries (USGS, 2012). Table 4-2 lists the main stem USGS gages and Table 4-3 lists the tributary USGS gages. Figure 4-1 is a map of the gage locations.

Table 4-2: USGS Missouri River Main Stem Gages

Gage Name	River Mile	Gage Number	Flow Data Dates	Stage Data Dates
Below Greenwood, SD	862.96	06453020	n/a	10/1/1989 - *
Below Ponca Creek near Verdel, NE (Old Location)	846.20[1]	06453620	n/a	10/15/1987 – 12/5/2011
Below Ponca Creek near Verdel, NE (New Location)	844.67[1]	06453620	n/a	1/6/2012 - *
Niobrara, NE	842.80	06466010	n/a	3/28/1996 - *
Lewis & Clark Lake at Springfield, SD	832.05	06466700	n/a	10/1/1980 - *

[1]The below Ponca Creek gage was destroyed twice in 2011 and was moved downstream 1.5 miles to its permanent location at the Niobrara State Park on 6 Jan 2012.
* - indicates that this is a current gage

Table 4-3: USGS Tributary Gages

Gage Name	Gage Number	Confluence River Mile	Modeled or Lateral Inflow	Available Flow Data Dates
Ponca Creek at Verdel, NE	06453600	849.90	Lateral Inflow	10/1/1957 - *
Niobrara River near Verdel, NE	06465500	843.90	Modeled	6/2/1958 - *
Verdigree Creek near Verdigre, NE[1]	06465700	5.05[1]	Lateral Inflow	5/1/2002 - *
Bazile Creek near Niobrara, NE	06466500	837.70	Lateral Inflow	6/1/1952 - *

[1]Verdigree Creek is a tributary of the Niobrara River that joins below the Niobrara River gage location.

* - indicates that this is a current gage

4.3.3 Lewis and Clark Lake Pool Elevations

The observed Lewis and Clark Lake (Gavins Point Pool) elevations were obtained from the Omaha District's Corps Water Management System (CWMS) database and were used as the downstream boundary condition.

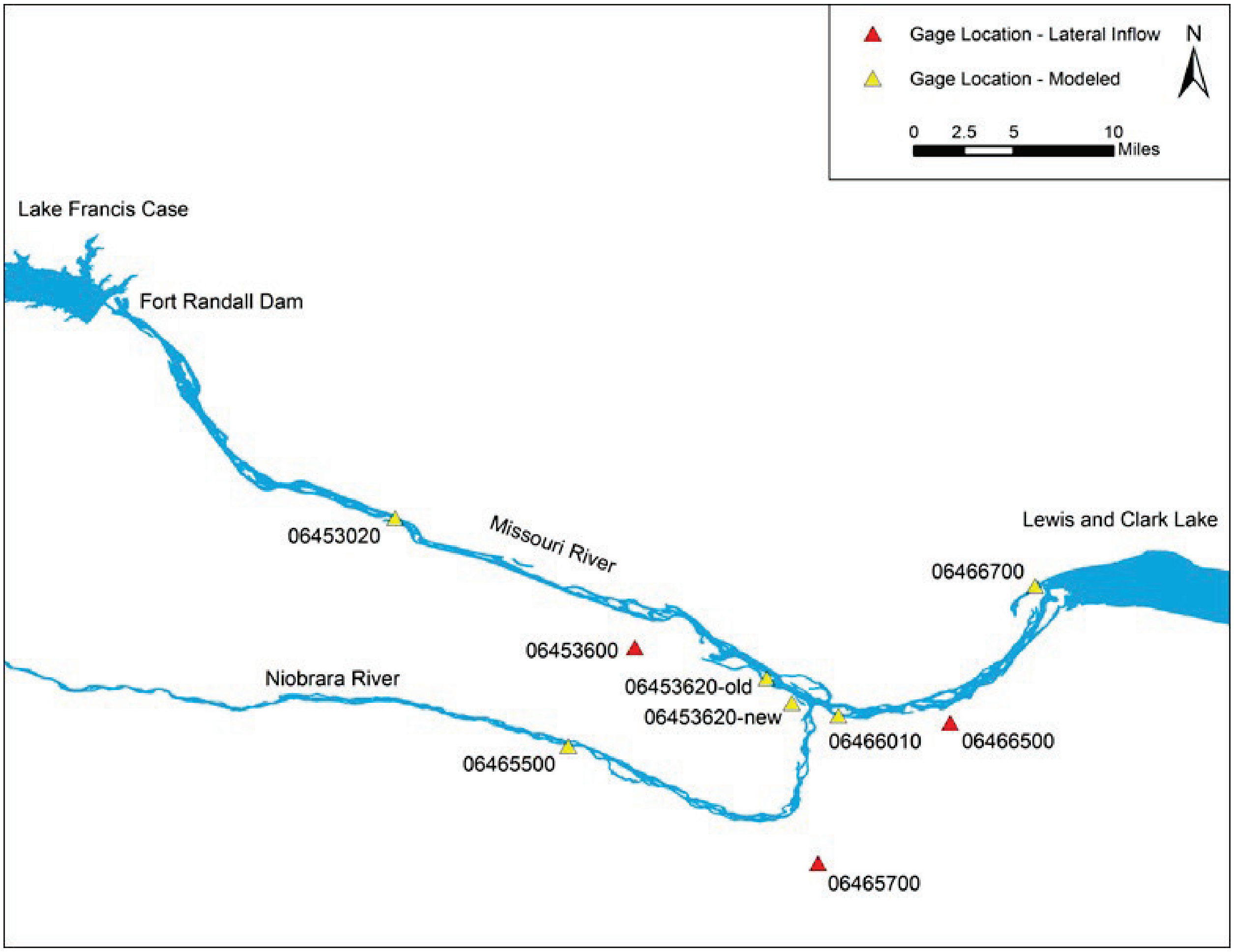

Figure 4-1: Gage Location Map

5 MODEL DEVELOPMENT

Model development includes the software version used, descriptions of the various geometry components of the model, and boundary conditions selected. The following sections outline the details of the model construction including fundamental assumptions, data sources for specific geometry features, techniques used, and justification for any unique parameters and decisions made during the process of building the model.

5.1 HEC-RAS

Unsteady computations in HEC-RAS version 4.2 Beta were used for this modeling effort. A computation interval of 2 hours was used because this was determined to be a stable time-step for the model and allowed model runs to be conducted in reasonable timeframes.

HEC-RAS has been significantly updated since version 4.1, and it is not recommended that the model be run in 4.1 or any earlier version.

HEC-RAS version 5.0 beta has been released but the model has not been tested in this version. The goal is to run the model in the newest version (not beta version), presumably version 5.0.

5.2 Geometry

This section will discuss the development of the HEC-RAS model geometry for the Missouri River reach from Fort Randall to Gavins Point, including vertical datum and horizontal projection, the stream centerline and cross section geometry, the development of Manning's n-values, and the modeling of structures such as bridges and dams. The geometry of the tributary used in the model was developed outside of this project and was added after the completion of the Missouri River geometry. The Niobrara River was modeled by West Consultants (WEST, 2010).

5.2.1 Vertical Datum and Projection

The current vertical datum for the Fort Randall to Gavins Point unsteady HEC-RAS model is NAVD88 to match the LiDAR data. Most of the other elevation data is referenced to the NGVD29 vertical datum so a conversion factor was used to convert that data to NAVD88. See Table 5-1 for a list of vertical conversion factors used in the model. The program CorpsCon was used to obtain the conversion values based on the gage's coordinates. CorpsCon is a widely accepted standard practice for converting between NGVD29 and NAVD88 vertical datums. However, it has been found that discrepancies exist between the CorpsCon conversion values and actual re-survey of points in the NAVD88 datum.

The current horizontal projection is NAD 83 UTM 14 (US-Feet) as this is what most of the available terrain data was in. Re-projection to a nation-wide projection may be necessary after review and certification for compatibility with other HEC-RAS models and the ResSim models that are in UTM projections. Re-projecting a HEC-RAS model to a national projection is not difficult or time consuming, and there is a documented How-To procedure provided by HEC.

Table 5-1: Gage Vertical Datum Conversion Factors

Gage Number	Gage Name	1960 River Mile	Conversion Factor (from NGVD29 to NAVD88) (ft)
06453020	Missouri River below Greenwood SD	862.96	0.863
06453620	Missouri River blw Ponca Creek near Verdel, NE (OLD Location)	846.2	0.728
06453620	Missouri River blw Ponca Creek near Verdel, NE (NEW Location)	844.67	0.712
06466010	Missouri River at Niobrara, NE	842.8	0.696
06466700	Lewis and Clark Lake at Springfield, SD	832.05	0.728
-	Lewis and Clark Lake (Gavins Pool)*	-	0.666

*Conversion factor for Lewis and Clark Lake pool elevations used the location where the elevation is recorded. For this pool, that is near the powerhouse.

5.2.2 Stream Centerline

One stream centerline for the Missouri River was developed in GIS for all of the Omaha District HEC-RAS models. A centerline from a previous study was modified to match the current state of the river, making sure to follow the center of mass of flow and avoiding crossing sandbars. It should be noted that the centerline does not match the 1960 river miles line. Cross sections were named based on the 1960 river miles so the reach lengths will not match up with the river miles.

5.2.3 Cross Section Geometry

The geometry of the cross sections were constructed using the most recent sediment range surveys, which included topographic and hydrographic data, in conjunction with the DEMs. The cross sections used survey data where possible and were extended as necessary with DEM data. The sediment ranges are generally spaced 1 to 2 miles apart on this stretch of the Missouri River. It was determined to have cross sections spaced no more than 3000 feet apart on the river portion of the Missouri River. For Lewis and Clark Lake, the sediment range spacing was considered sufficient for modeling the impounded segment of the river. To obtain the desired spacing, additional cross sections were added between the sediment ranges using LiDAR or DEM data for the overbank extents and for the channel data, either RAS interpolated bathymetry or channel data from a nearby range was used. Attachment 1 provides a more detailed description of how the interpolated cross section's bathymetry was estimated. Banklines for all the cross sections were set at approximately the 2-yr water surface elevation. Cross sections were named based on the 1960 river miles, since this is the primary method used to identify locations on the Missouri River. However, the 1960 river miles do not match up with the stream centerline, which produces reach lengths that do not match the river miles.

5.2.4 Manning's N-values

For the overbank areas, Manning's n values for roughness were set based on the land use classification from the NLCD 2006 data. The land cover values were condensed from the NLCD 2006 standards into 12 classes, as shown in Table 5-2. The land cover GIS shapefile was manually updated with the use of recent aerial images to reflect changes to the river channel, such as shifting sandbars, mostly due to the 2011 flood event.

Manning's n-values in the river channel were initially set to 0.025. During calibration, these were modified to between 0.025 to 0.031, which were determined to be reasonable channel roughness values for the Missouri River. Channel roughness values were generally changed in a reach wide manner of 10 to 30 mile long blocks. Final roughness values for the main channel are shown in Table 5-3. Manning's n-values for overbank areas were not modified during calibration.

Table 5-2: Land Use Reclassification and Initial Roughness Values

NLCD Number	NLCD Classification	Reclass Number	Reclassification for Model	Manning's N-Value
11	Open Water	11	Water[1]	0.025
		12	Channel Sandbar	0.032
		13	Channel Sandbar Light Vegetation	0.038
		14	Channel Sandbar Heavy Vegetation	0.052
		15	Channel Bank	0.056
21	Developed, Open Space	2	Urban	0.080
22	Developed, Low Intensity			
23	Developed, Med Intensity			
24	Developed, High Intensity			
31	Barren Land (Rock/Sand/Clay)	3	Sand	0.028
41	Deciduous Forest	4	Trees	0.070
42	Evergreen Forest			
43	Mixed Forest			
51	Dwarf Scrub	5	Scrub Brush	0.060
52	Shrub/Scrub			
71	Grassland/Herbaceous	6	Grass	0.035
72	Sedge/Herbaceous			
73	Lichens			
74	Moss			
81	Pasture/Hay			
82	Cultivated Crops	7	Crops	0.045
90	Woody Wetlands	8	Wetlands	0.055
95	Emergent Wetlands			

[1]Initial roughness value that was modified during the calibration process.

Table 5-3: Final Channel Roughness Values

Cross Section River Mile Range	Channel Manning's N-Value
879.04 – 864.09	0.025
863.47 – 855.37	0.024
854.84 – 840.56	0.031
840 – 812.74	0.025

5.2.5 Bridges

On the Missouri River main stem, cross sections representing bridge embankments are in the model, but the structures themselves are not. This was a simplification made to keep computation times shorter. In addition, all bridge deck low chords on the Missouri River are elevated higher than the floods of record, so the only component other than the embankment that would impede water flow is the bridge columns, which likely have a local effect, but not global. This was determined to be sufficient for the Missouri River modeling. Bridges in the tributary models were left in the geometry unless they caused issues with model stability.

5.2.6 Dams

This stretch of the Missouri River was modeled from just downstream of Fort Randall Dam to just upstream of Gavins Point Dam, so the dams themselves are not in the model. The pool of Gavins Point Dam, Lewis and Clark Lake, is in this HEC-RAS model and is the downstream boundary condition.

5.2.7 Tributaries

Tributary reaches were modeled to route flow from the gage station to the Missouri River and were not calibrated to stage. One tributary, the Niobrara River, was selected to model on this stretch of the Missouri River. The tributary modeling was performed by West Consultants (WEST, 2010). In general, the goal with the tributary routing reaches was to model travel time sufficiently well from the tributary gage station to the Missouri River and preserve timing for calibration purposes. No tributary computed stage information should be used from model results without carefully assessing the purpose and considering model construction limitations.

The tributary RAS model was checked for the correct vertical datum and horizontal projection and was inserted into the Missouri River geometry with a junction. Junction lengths were assumed to represent the average distance that the water will travel from the last cross section in the reach to the first cross section of the following reach (USACE, 2010). For junction calculations, either the energy method or force equal water surfaces method was chosen based on model stability. Due to stability issues encountered with the Niobrara River-Missouri River junction, the last cross section of the Niobrara River geometry was copied downstream to reduce the junction length. This may produce, to a minor degree, double counting of storage in the model.

5.3 Boundary Conditions

The boundary conditions are the initial flows and stages used at the upstream and downstream extents of the HEC-RAS model. Below is a discussion of those boundary conditions.

5.3.1 Upstream Boundary Conditions

Upstream boundary conditions include the outflow from Fort Randall Dam and observed USGS flow hydrographs at the upstream end of each of the tributary reaches. Hourly data was used when available and daily data was used to complete the flow record. To achieve stability, a minimum flow was used for each input, as shown in Table 5-4.

Table 5-4: Minimum Flows

Location	Minimum Flow (cfs)
Fort Randall Outflow	2,000
Niobrara River	800

5.3.2 Downstream Boundary Condition

The downstream boundary condition used was the stage hydrograph for Gavins Point Dam's pool, Lewis and Clark Lake, from Omaha District's CWMS database.

6 CALIBRATION

Model calibration was accomplished through several steps described in this section. Results as well as a discussion of level of calibration achieved and overall model performance are presented below.

6.1 Model Calibration

Unlike previous modeling efforts on the Missouri River, a broad spectrum of flows from low flows to high flows were considered important to the project purposes. Calibration methods had to include a range of flows. The primary source of calibration data was observed stage hydrographs on the main stem Missouri River gages and field measured water surface profile data that was collected in June 2011 and May 2012.

First, the model was calibrated in a steady state for geometry. A thorough check of the estimated bathymetry was performed. At various flows, output values were checked for consistency to avoid sudden changes from one cross section to the next. The output analyzed included flow distribution (overbanks and channel), top width, velocity, energy grade, and flow area. Cross section interpolations were revised based on this analysis. The steady state model was calibrated to the water surface profiles collected in 2011 and 2012 by adjusting channel n-values. The channel n-values were initially set at 0.025 and were adjusted for steady state calibration to obtain a water surface elevation that was within a tolerance of the measured water surface profiles.

Second, the model was run in the unsteady state with steady flows to obtain a stable model. Then, the tributary, the Niobrara River, geometry was added into the model. The tributary model was roughly calibrated and was inserted for the primary purpose of routing flows from the gage to the Missouri River for the unsteady model runs to preserve flow timing. Tributary computed stages will not be used in the analysis. Once the model was stable with the tributary added, the observed flows were put into the model. The model was run from January 2011 to December 2012 and the results were compared to the water surface profile data for the time period it was collected and the observed stage from the gages, where available. Multiple iterations were required in this process with roughness values and ineffective flow locations.

Calibration philosophy was to primarily use the channel roughness values to calibrate the model for normal flows and use the HEC-RAS option for flow roughness and adjustments to ineffective flow areas to calibrate for higher flow events. Flow roughness factors were used to calibrate to the 2011 high flow event as shown in Table 6-1.

Table 6-1: Flow Roughness Factors

U/S Cross Section	879.04	855.37
D/S Cross Section	859.31	844.16
Flow (cfs)	**Roughness Factor**	
0	1	1
40,000	1	1
60,000	1	1.05
80,000	1	1.05
100,000	1.1	1.05
120,000	1.1	1.05
140,000	1.2	1.05
160,000	1.3	1.1
180,000	1.3	1.1

Several factors presented a challenge with the unsteady model calibration. A looped rating curve during the 2011 high flow event was difficult to calibrate to both the rising and falling limbs of the event. As a result of the major degradation that occurred during the event, calibration on the rising side of the flood hydrograph using post flood data was not feasible. An example of the observed gage looped rating curve is shown in Figure 6-1. Fort Randall Dam releases change throughout each day due to power peaking through the powerhouse. The power releases usually range from approximately 10,000 to 35,000 cfs, although the power plant capacity is 36,000 cfs. This produces a stage difference of between 1 to 5 feet, or more. Releases vary the most in the summer months. Timing in the model skews the results and may report that the model is not performing well while it is just off by a few hours. Also, no flow gages are present on the Missouri River in this reach so no ungaged inflow computations could be performed.

The calibration goal was to achieve a water surface elevation within 1 ft for the entire reach and 0.5 ft for most of the reach for both the measured water surface profiles and the observed gage readings for 2011 and 2012, excluding periods of ice. The model does not account for ice. Ice causes much higher stages than would normally occur for an open water condition. Ice affected events typically occur from December to March. Plate 2 through Plate 11 are the hydrographs and computed minus observed stage vs flow plots from the 4 stage gage locations. Plate 12 through Plate 16 show the computed profile vs the measured water surface profile.

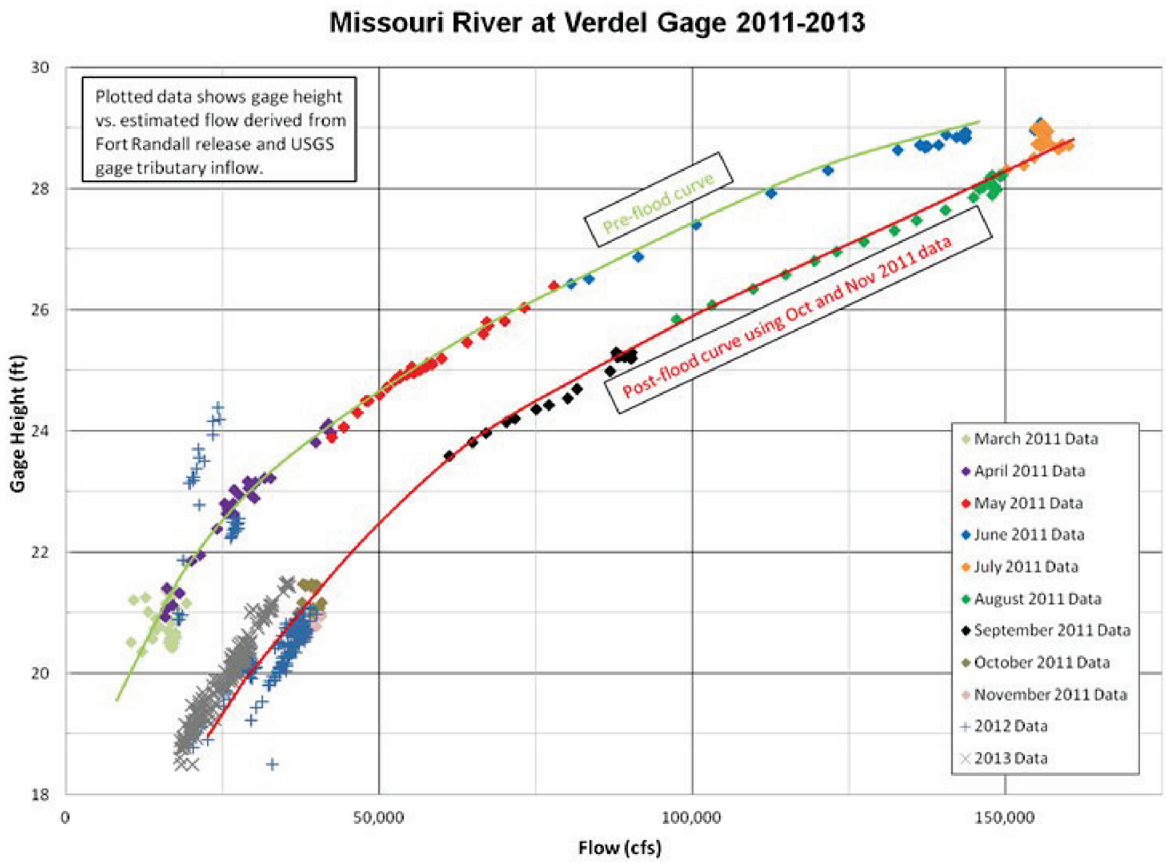

Figure 6-1: Example Observed Gage Looped Rating Curve for 2011 Event

6.2 Calibration Results

Model calibration results are within the desired range with most locations within 0.5 to 1 foot of observed stages.The results can be seen in Plate 2 through Plate 16. In general, comparison of model results to gage station hydrographs were reasonable. The measured profile calibration also provides confidence in model performance between the gage station locations. A comparison of peak stages for the 2011 flood are shown in Table 6-2.

Table 6-2: 2011 Flood Peak Stage Comparison

Location	Date	Peak Stage Difference (ft)
RM 862.96 – blw Greenwood	30 Jun – 29 Jul	-0.2
RM 846.20 – blw Ponca Crk near Verdel (Old)	30 Jun	-0.1
RM 844.67 – blw Ponca Crk nr Verdel (New)	M	M
RM 842.80 – at Niobrara	26 Jun – 01 Jul	0.1
RM 832.05 – at Springfield	22 Jun – 26 Jun	-0.8

*M – denotes gage peak stage data is missing

*Peak stages were manually estimated due to minor timing issues and bad data points.

6.2.1 Calibration Results Affected by Ice Conditions

Ice affected conditions including ice cover, ice breakup, and ice jams occur annually within the basin. Ice formation conditions typically occur in late November to late December with iceout typically occur in the early spring, usually in the March to April time frame. No ice parameters were included in the model development or calibration. Therefore, winter condition model calibration results should be viewed with caution and recognize that results do not reflect observed conditions.

6.2.2 Stage Trend Impacts

Degradation and aggradation conditions occur through the reach due to Fort Randall Dam at the upstream model boundary and Gavins Point Dam at the downstream model boundary. Due to the extreme 2011 event flows and the high degree of channel adjustment that occurred during the event, accurate stage calibration prior to 2011 using the post-2011 event model geometry is not possible. Model results for the rising portion of the event in May and June demonstrate how stage-flow relationships changed during the flood and also reduce calibration accuracy through this portion of the event.

7 CONCLUSIONS

The model performs well for the 2011 and 2012 observed data and is calibrated to the 2011 and 2012 water surface profiles.

- Measured profile and gage hydrograph calibration for both 2011 and 2012 indicates that the model performs satisfactorily with a stage calibration accuracy of less than 1 foot at most locations.
- Incomplete hydrographic surveys were available to construct the model. Interpolation from hydrographic sections was used combined with LiDAR data to generate cross sections at the desired spacing of about 2,500 to 3,000 feet. Consequently, the interpolated sections within the model have reduced accuracy for the below water portion of the cross section. Normal flow calibration indicated that the model performs satisfactorily which implies the interpolation method was reasonable.

- No tributary computed stage information should be used from model results without carefully assessing the purpose and considering model construction limitations.
- Aggradation and degradation that occurred during the 2011 event reduces calibration accuracy for the flood hydrograph. This also prevents calibrating to flow events prior to 2011.
- Ungaged flows exist in this reach but cannot be calculated due to the lack of mainstem flow gages. Some model calibration error can be attributed to this missing ungaged flow.

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APPENDIX C

FORT RANDALL DAM TO GAVINS POINT DAM

PLATES

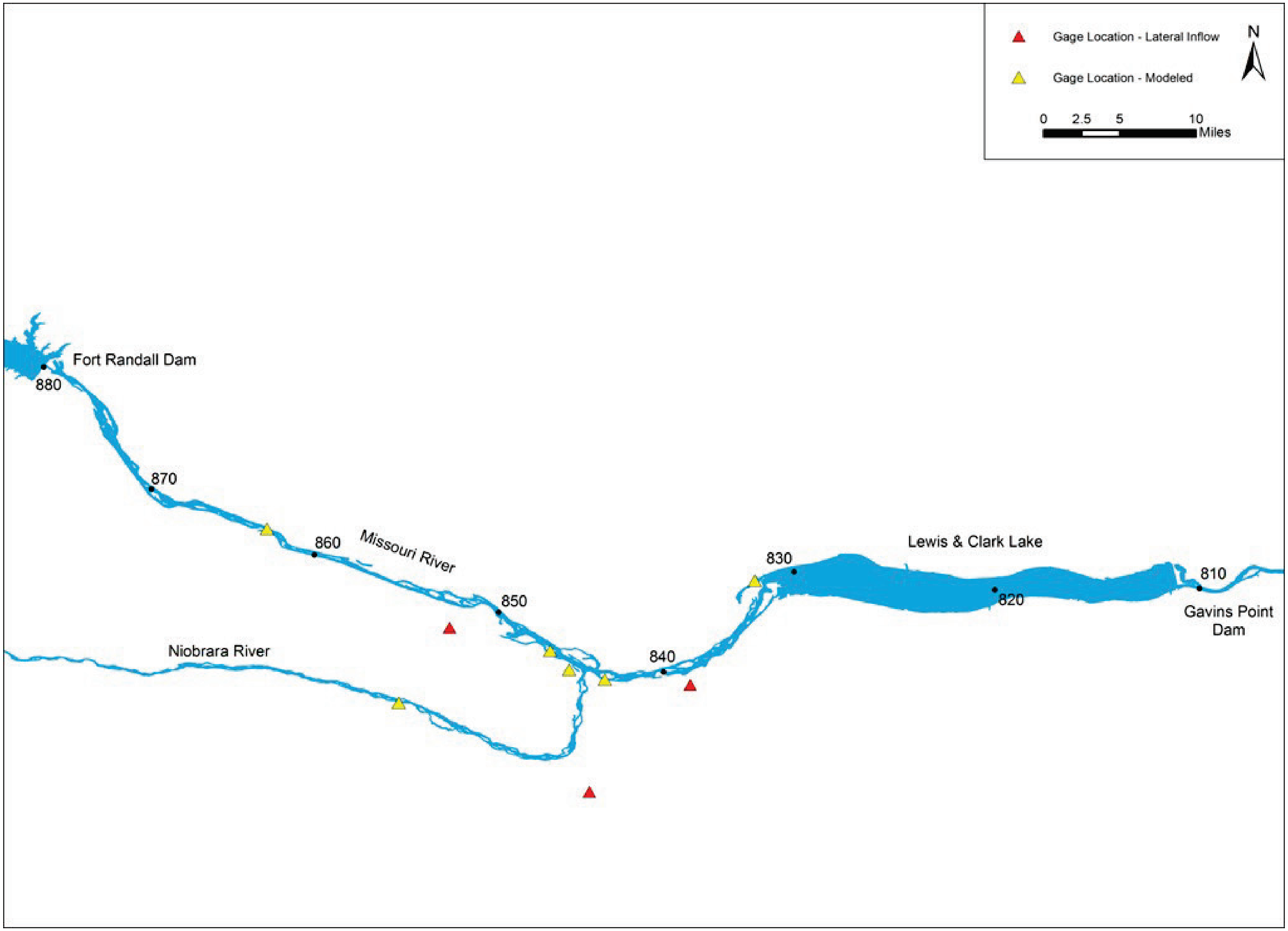
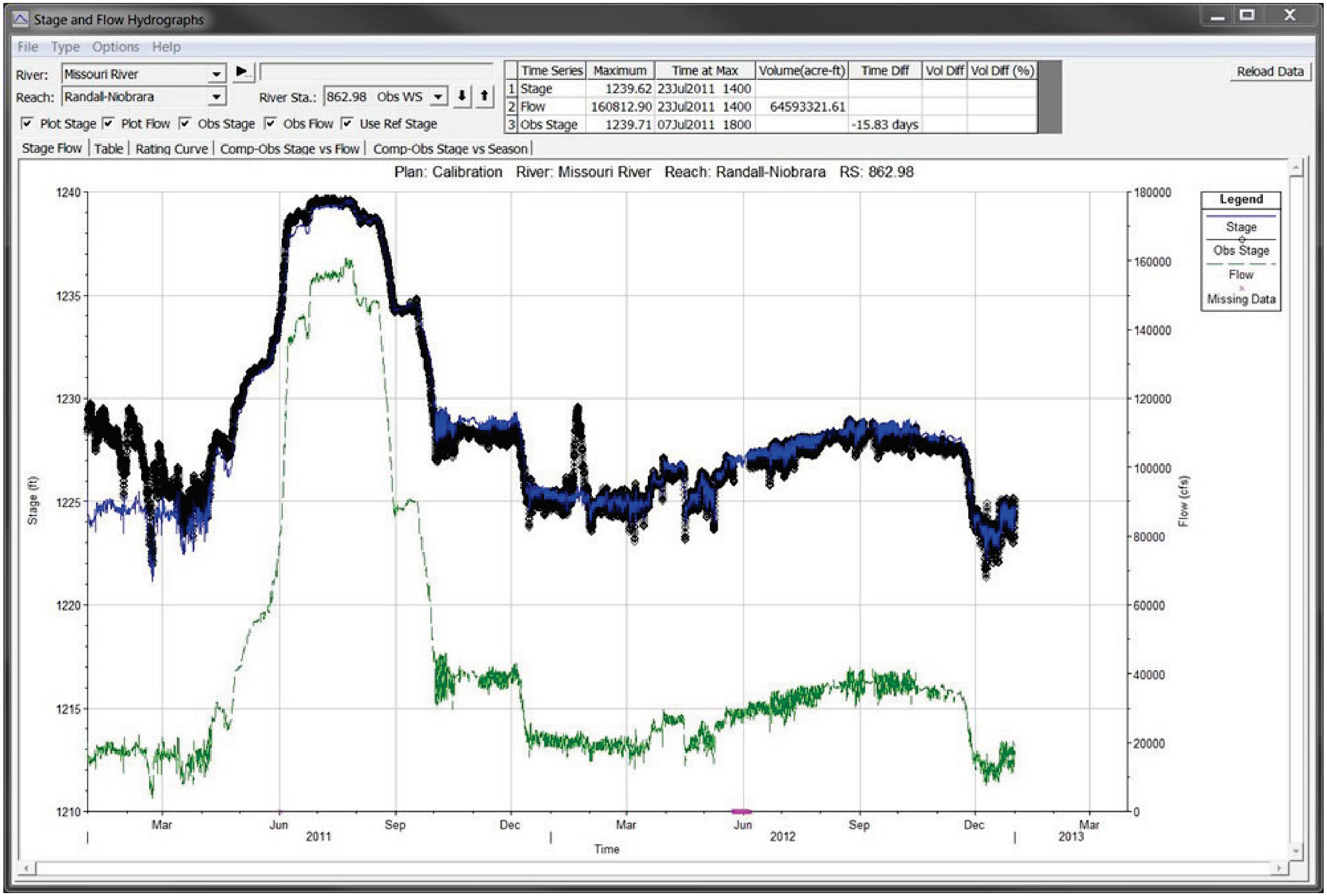


Plate 1: Overview Map



	Time Series	Maximum	Time at Max	Volume(acre-ft)	Time Diff	Vol Diff	Vol Diff (%)
1	Stage	1239.62	23Jul2011 1400				
2	Flow	160812.90	23Jul2011 1400	64593321.61			
3	Obs Stage	1239.71	07Jul2011 1800		-15.83 days		

Plate 2: Missouri River below Greenwood, SD Hydrograph

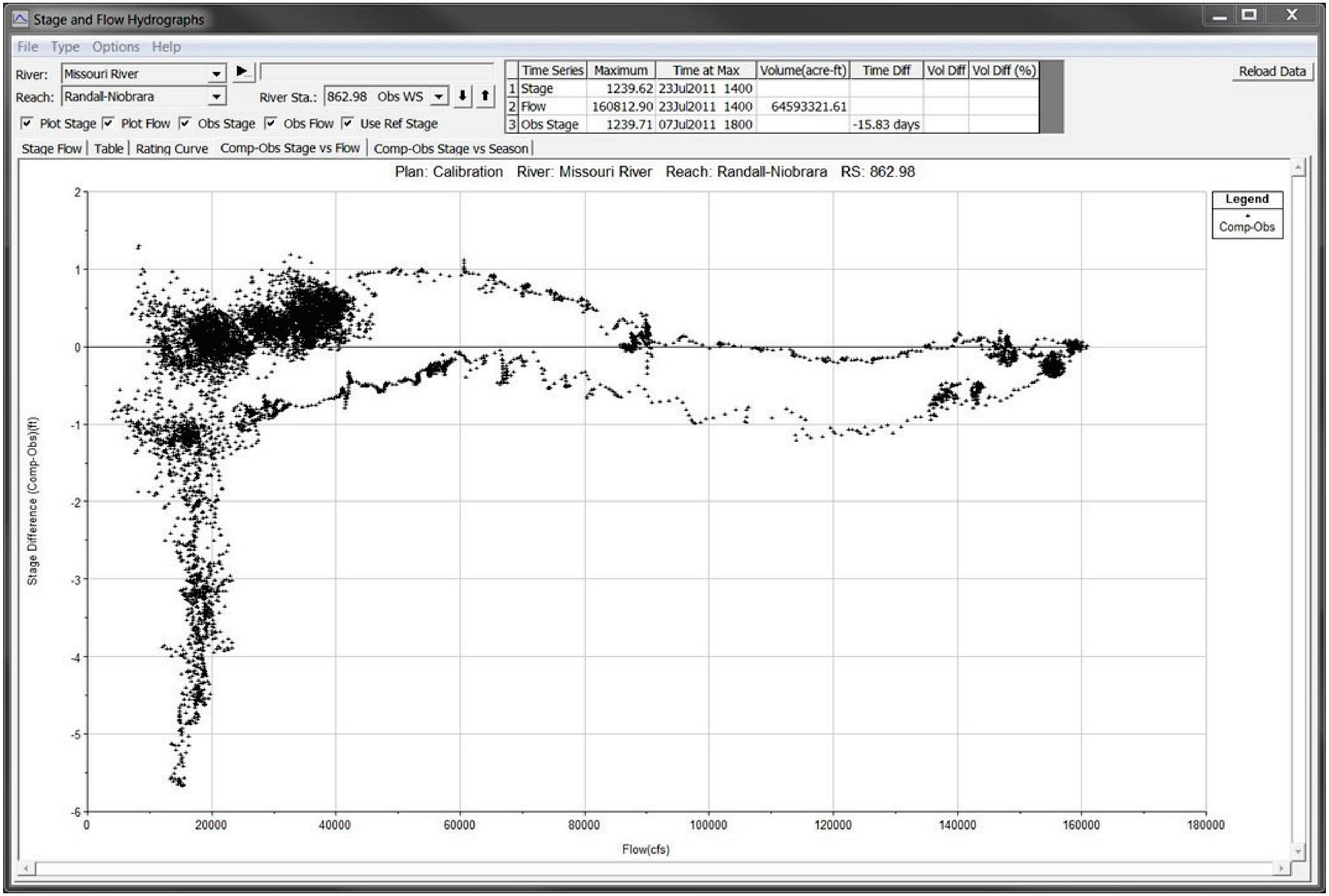


Plate 3: Missouri River below Greenwood, SD Comp-Obs Stage vs Flow

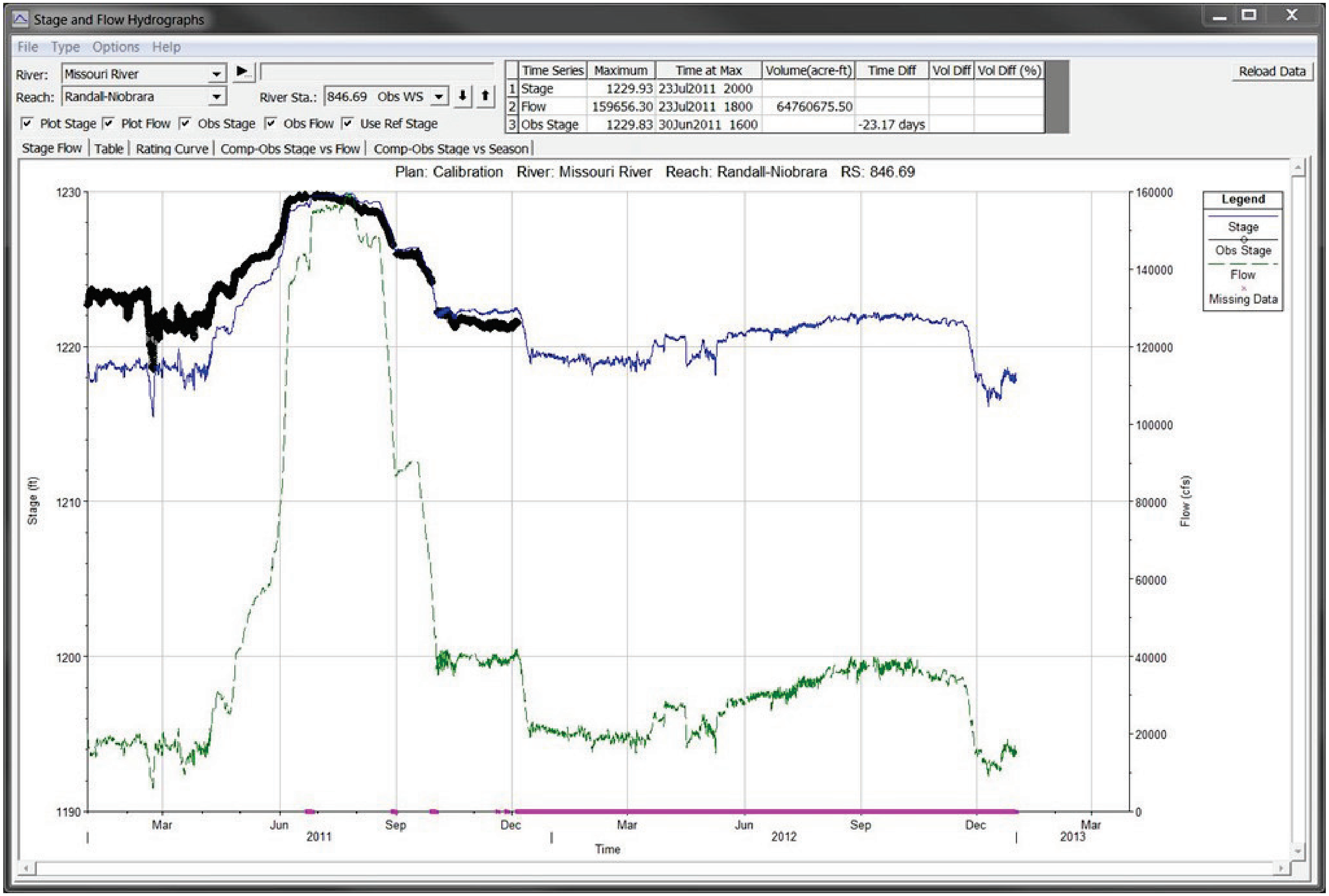


Plate 4: Missouri River below Ponca Creek near Verdel, NE (Old Location) Hydrograph

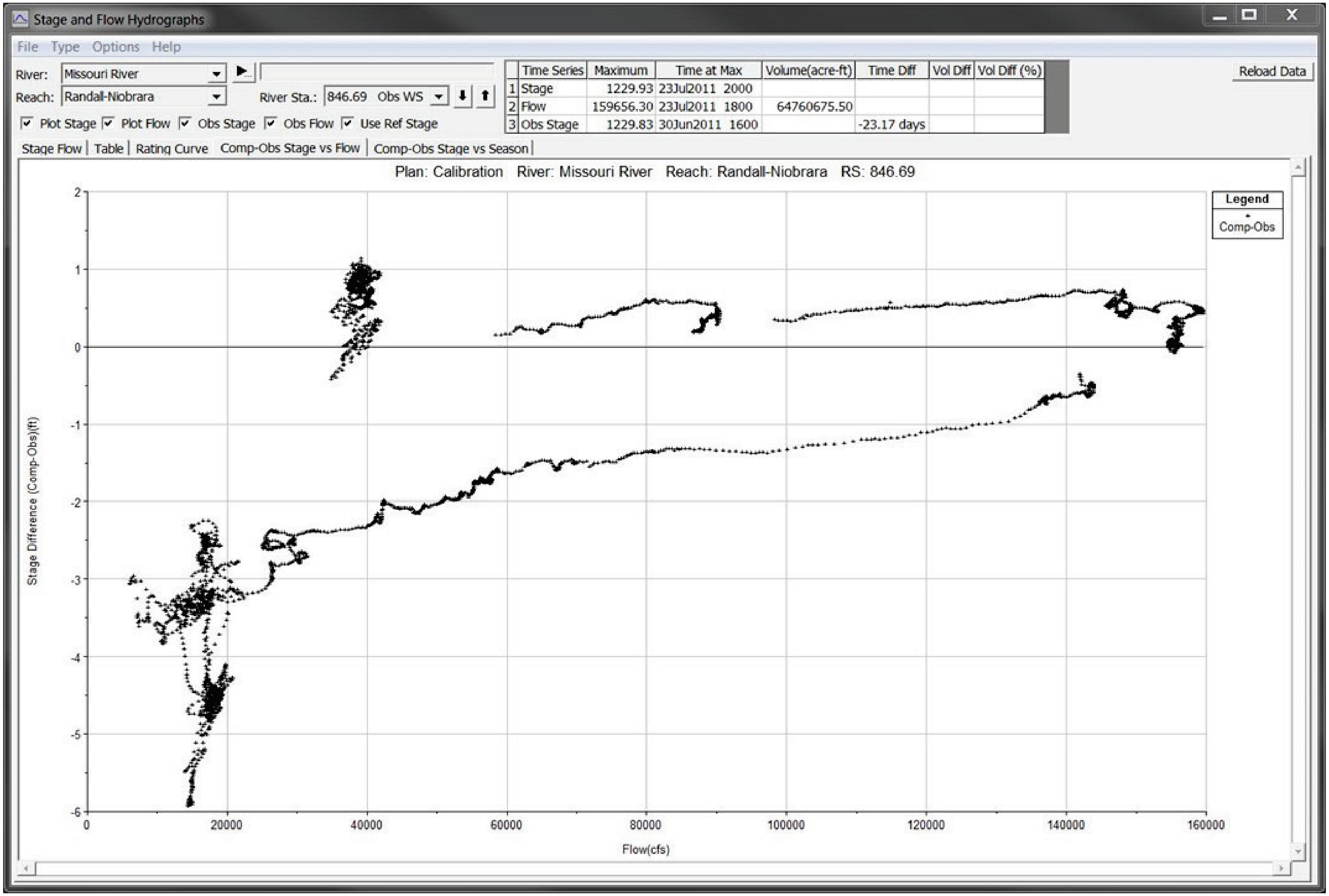


Plate 5: Missouri River below Ponca Creek near Verdel, NE (Old Location) Comp-Obs Stage vs Flow

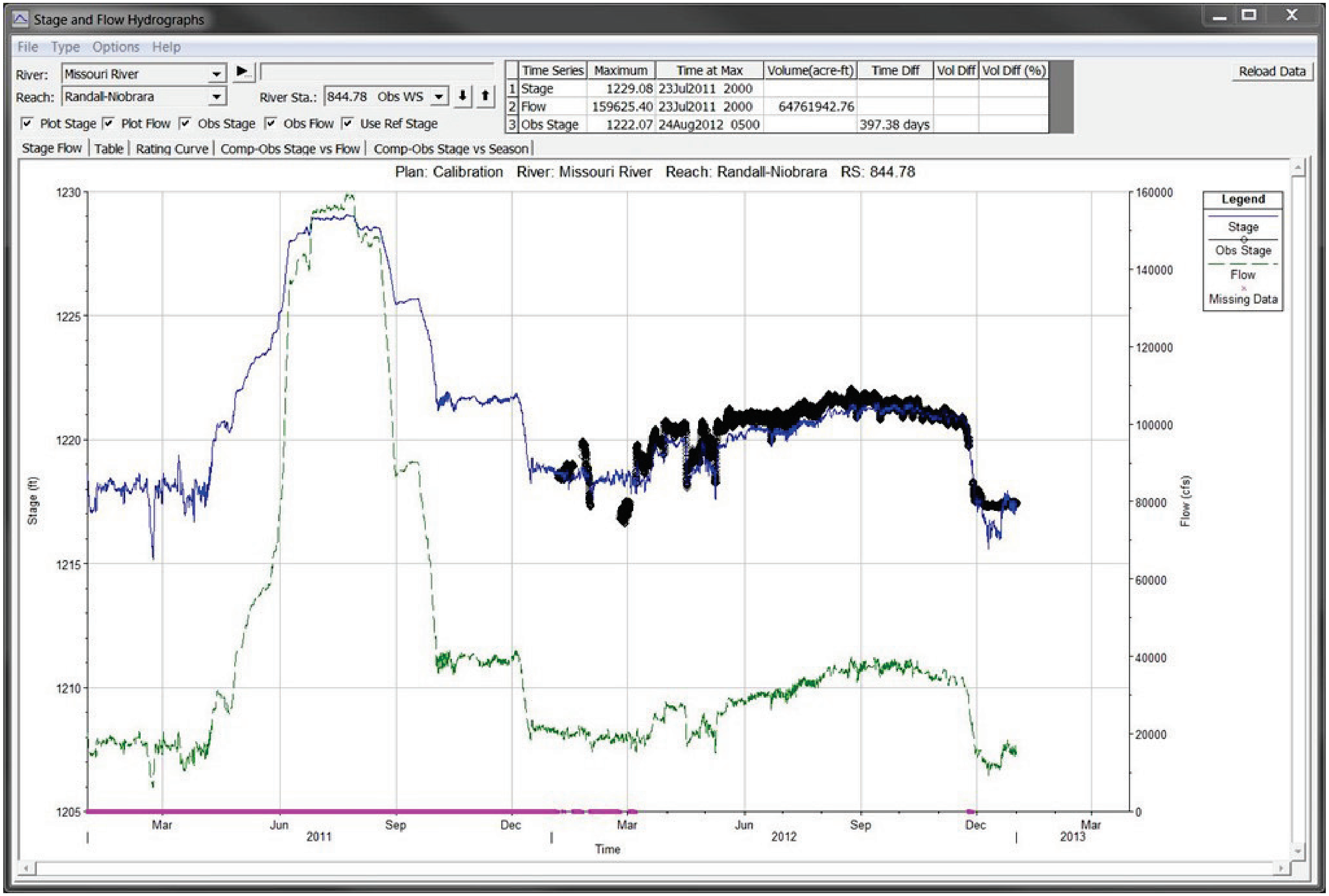


Plate 6: Missouri River below Ponca Creek near Verdel, NE (New Location) Hydrograph

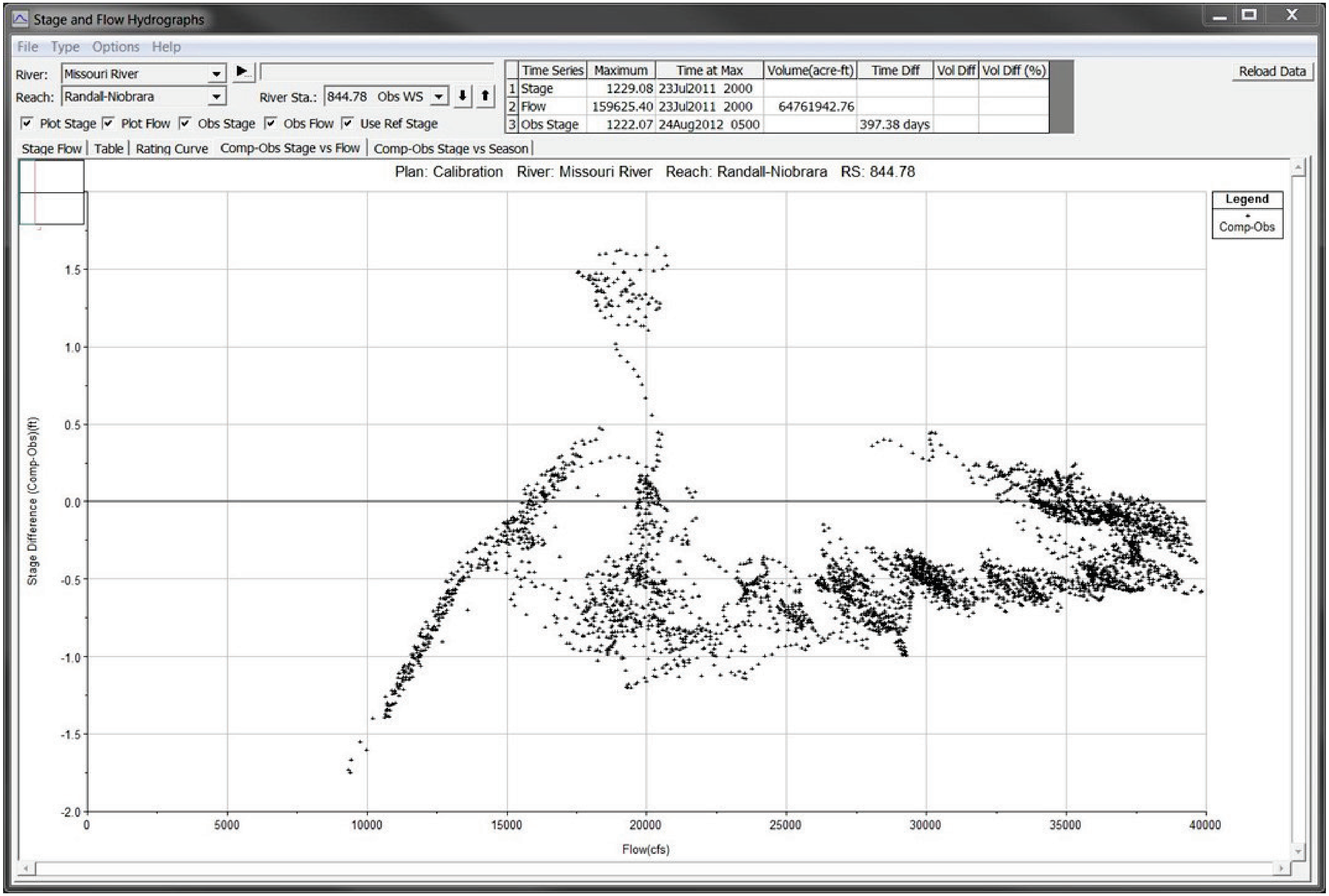


Plate 7: Missouri River below Ponca Creek near Verdel, NE (New Location) Comp-Obs Stage vs Flow

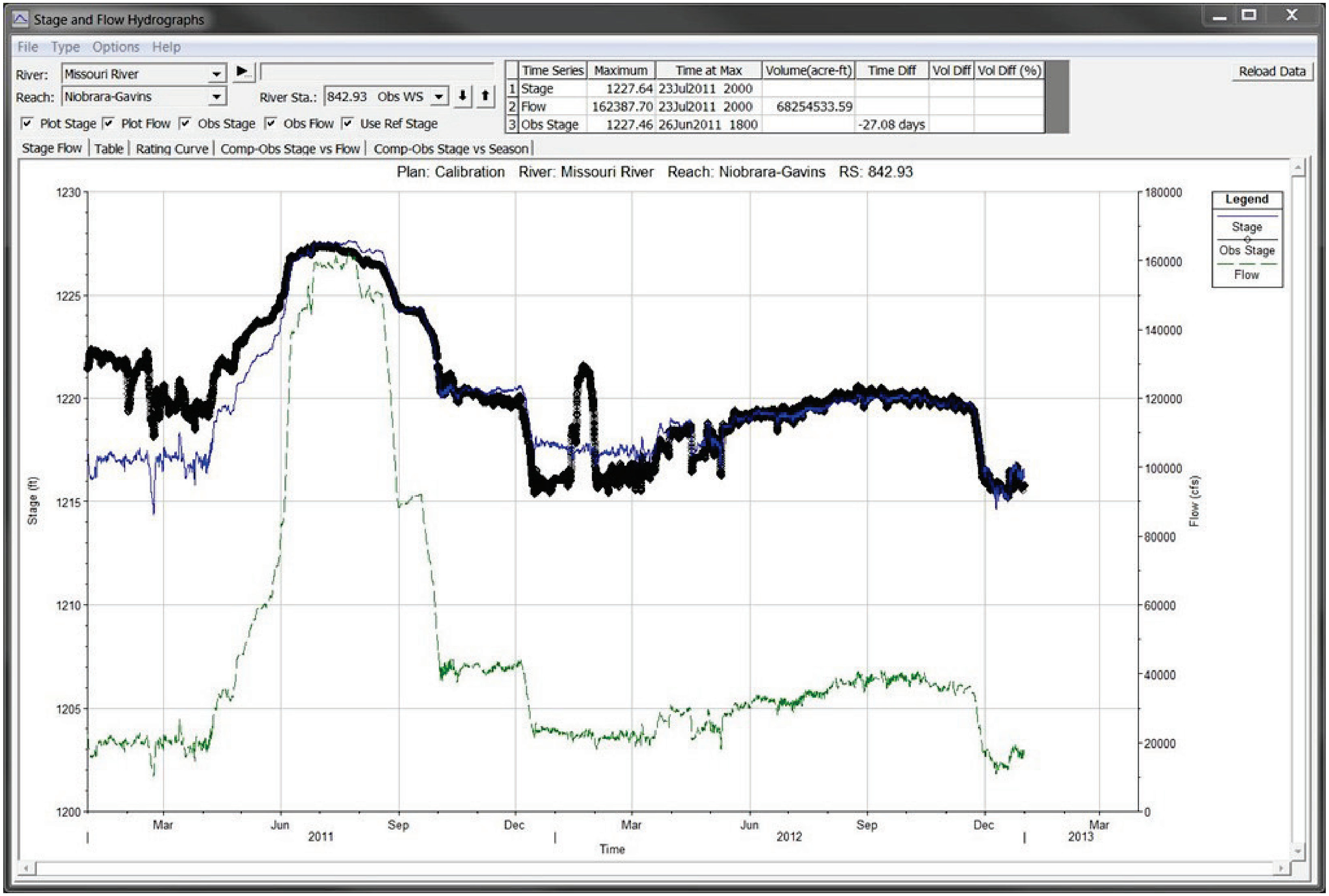


Plate 8: Missouri River at Niobrara, NE Hydrograph

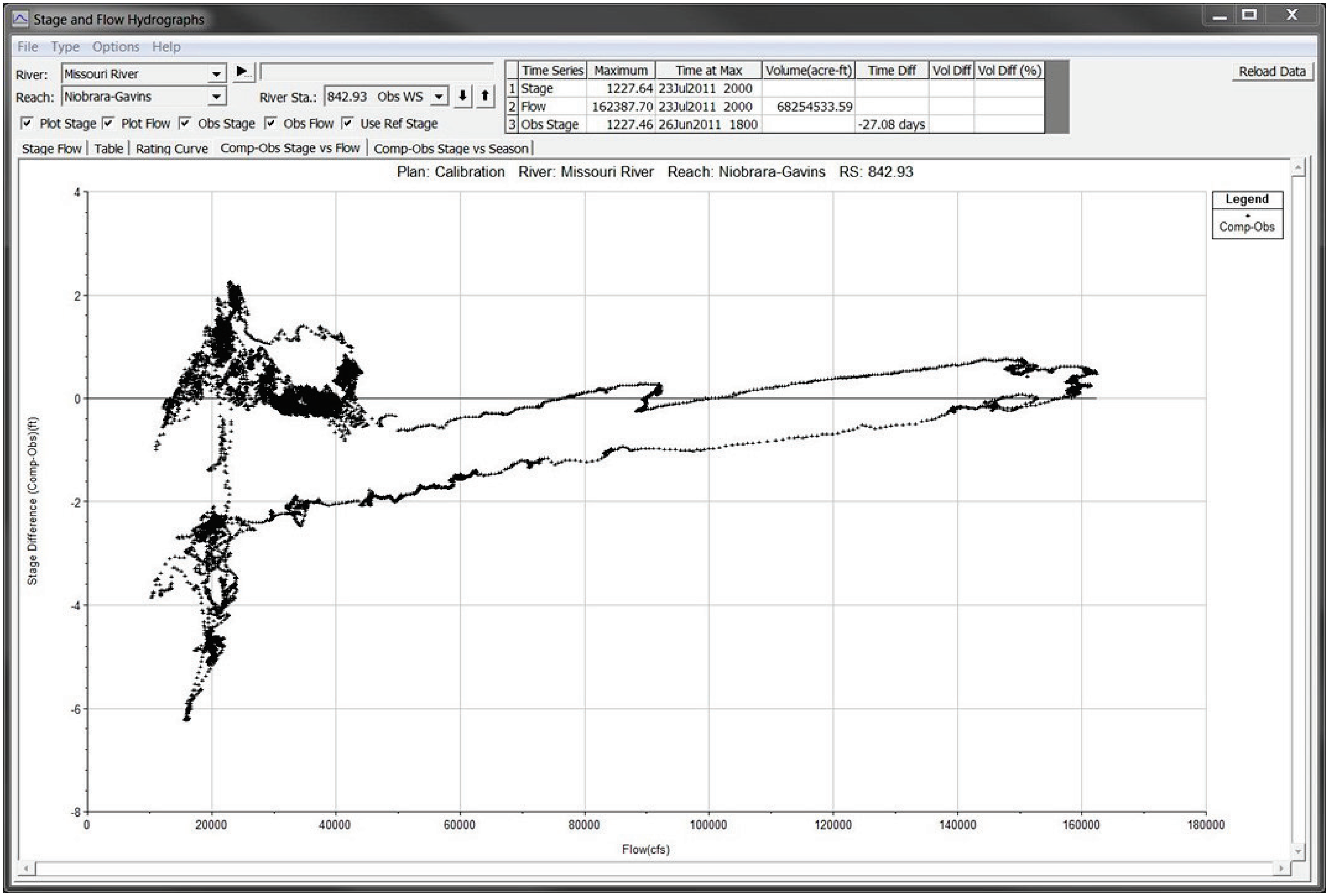


Plate 9: Missouri River at Niobrara, NE Comp-Obs Stage vs Flow

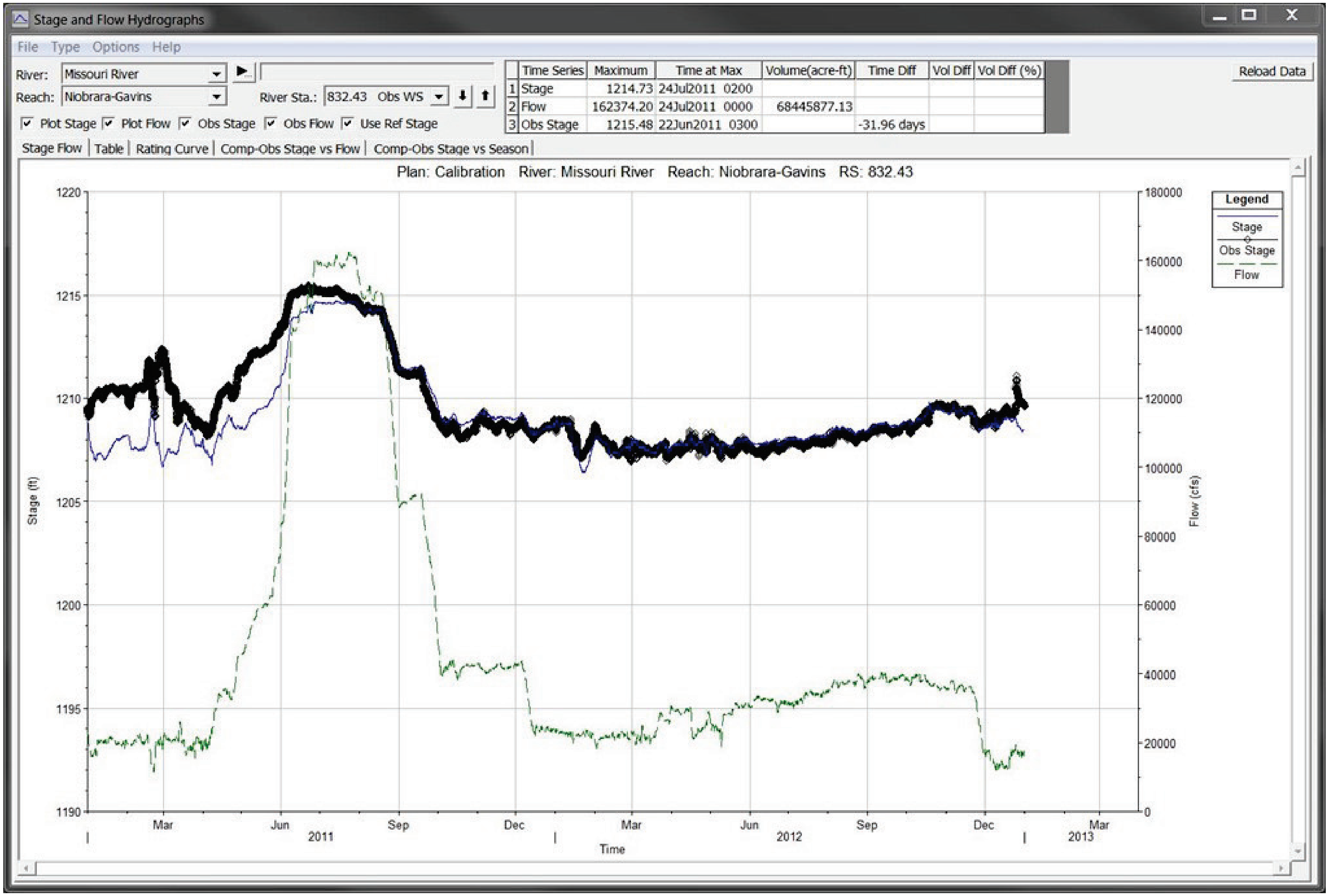
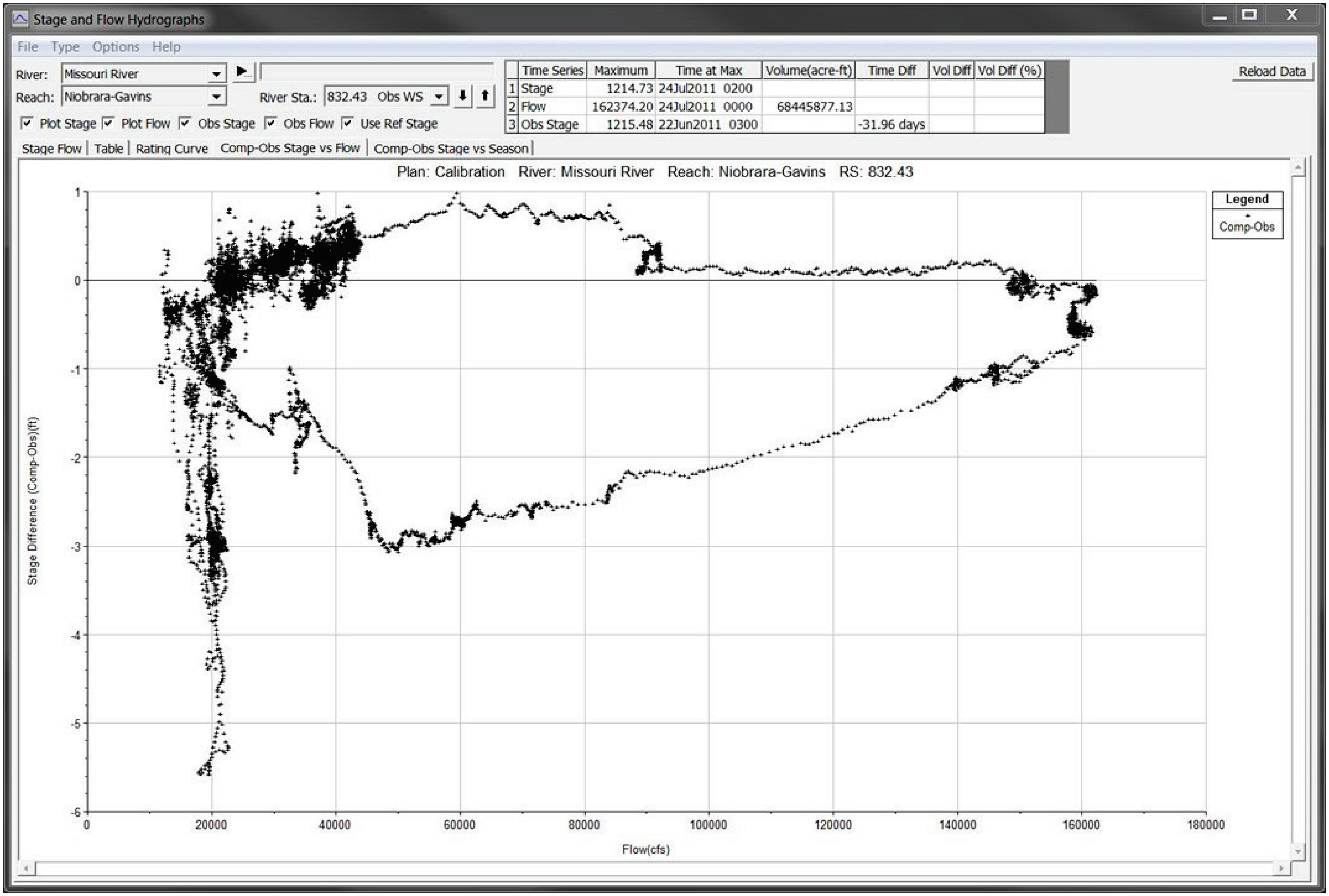


Plate 10: Lewis and Clark Lake at Springfield, SD Hydrograph



	Time Series	Maximum	Time at Max	Volume(acre-ft)	Time Diff	Vol Diff	Vol Diff (%)
1	Stage	1214.73	24Jul2011 0200				
2	Flow	162374.20	24Jul2011 0000	68445877.13			
3	Obs Stage	1215.48	22Jun2011 0300		-31.96 days		

Plate 11: Lewis and Clark Lake at Springfield, SD Comp-Obs Stage vs Flow

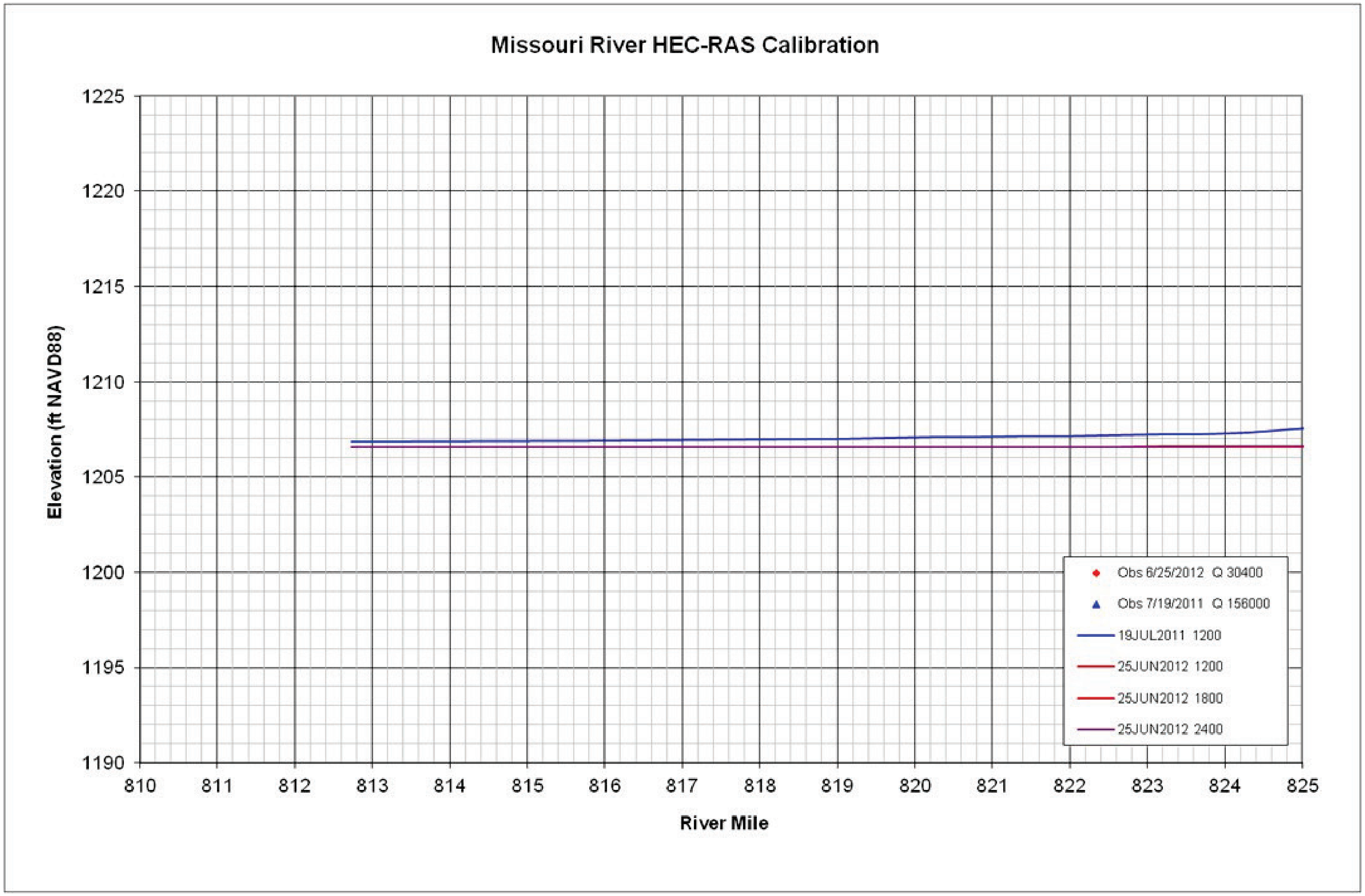


Plate 12: Measured WSP vs Computed Water Surface – Gavins Point Dam to RM 825

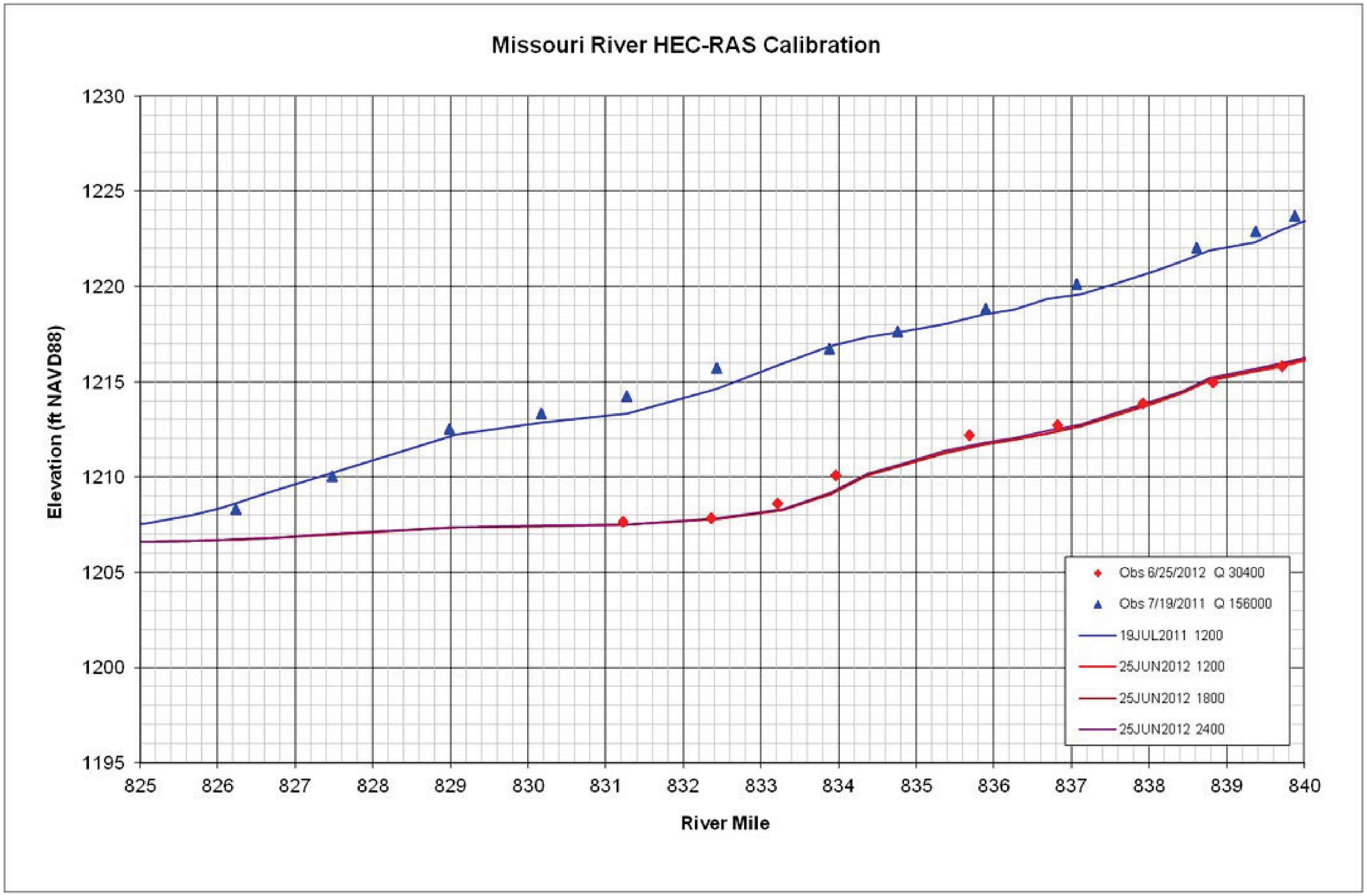


Plate 13: Measured WSP vs Computed Water Surface – RM 825 to 840

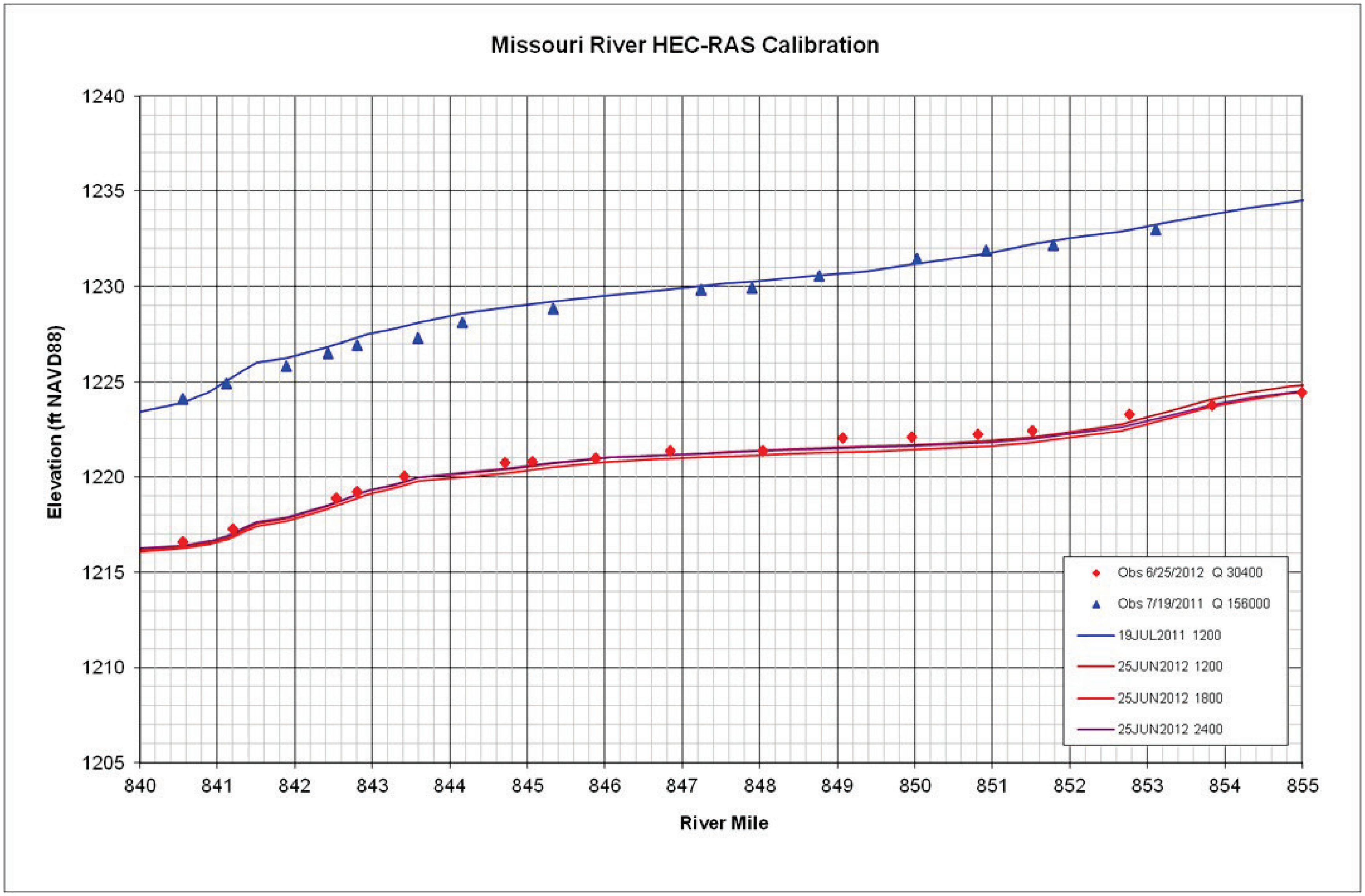


Plate 14: Measured WSP vs Computed Water Surface – RM 840 to 855

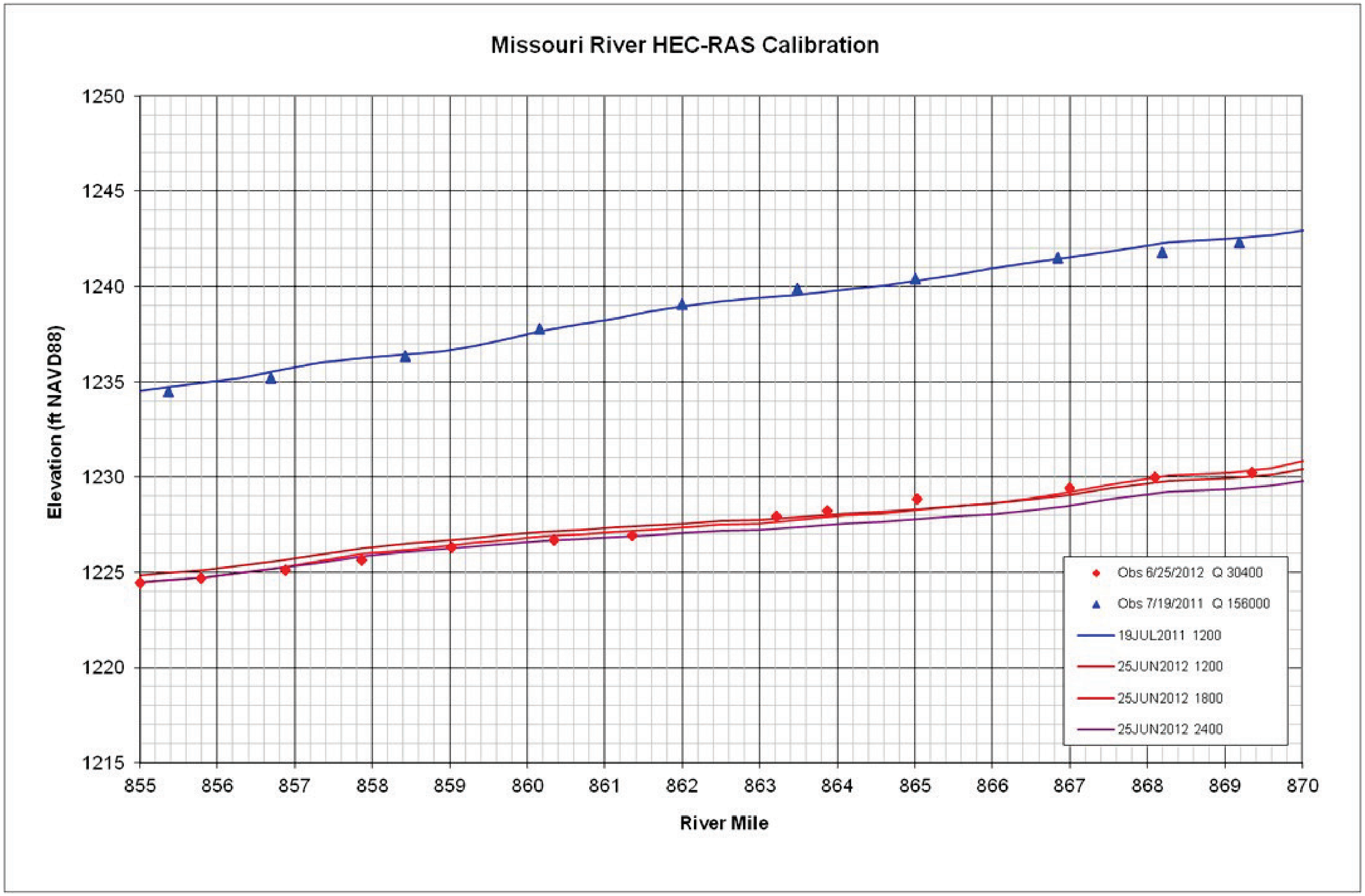


Plate 15: Measured WSP vs Computed Water Surface – RM 855 to 870

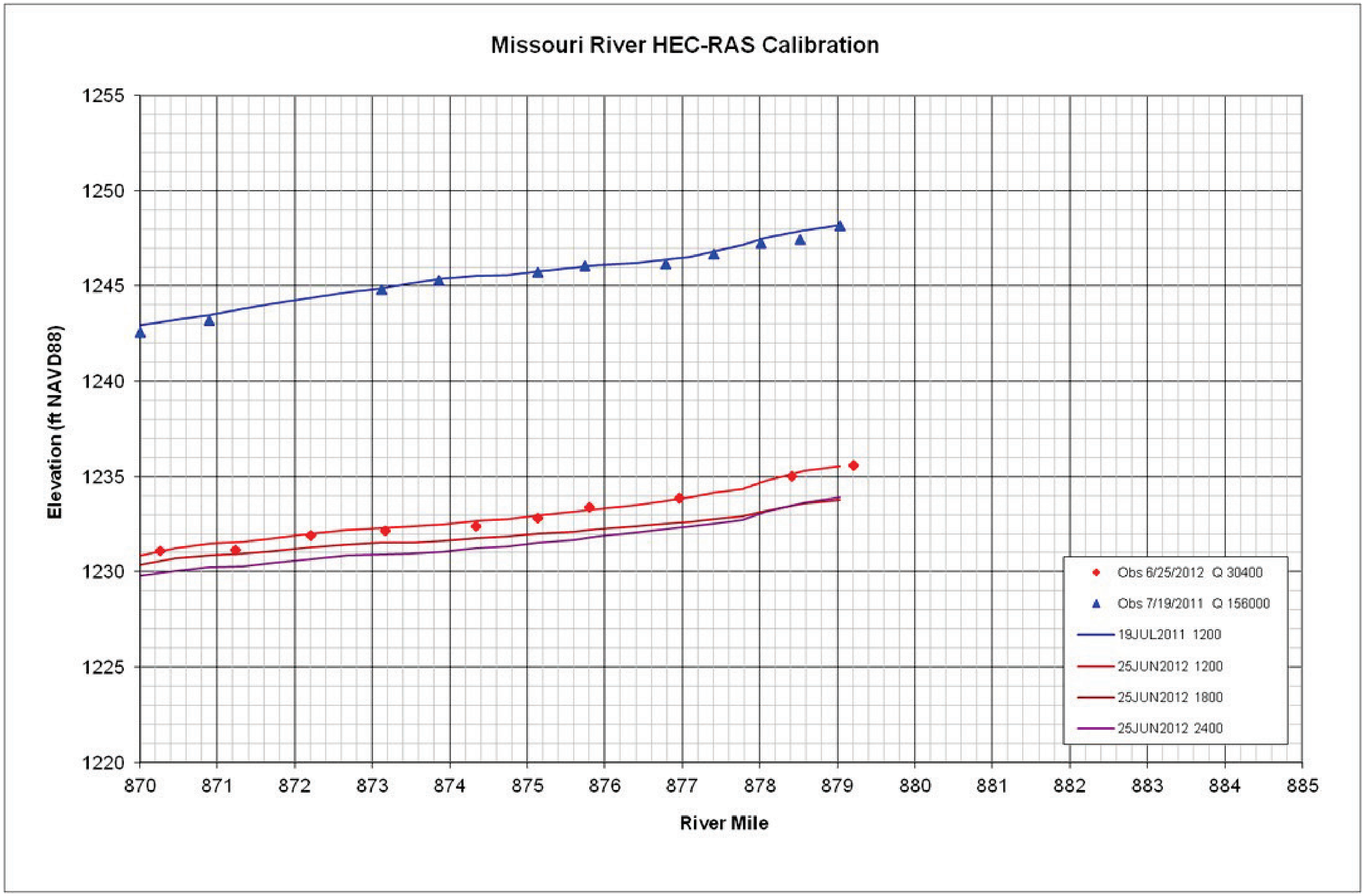


Plate 16: Measured WSP vs Computed Water Surface – RM 870 to Ft. Randall Dam

APPENDIX C

FORT RANDALL DAM TO GAVINS POINT DAM

ATTACHMENT 1 – CROSS SECTION INTERPOLATION

Attachment 1
Missouri River RAS Modeling
Cross Section Interpolation
9 July 2014

Overview

The Missouri River RAS unsteady modeling project will construct unsteady flow models for the Missouri River from Ft Peck Dam, Montana, to St. Louis, Missouri. Upstream of Gavins Point Dam (near RM 811), the hydrographic data primarily consists of sediment range surveys used to monitor aggradation / degradation between the dams. Figure 1 illustrates the reach locations.

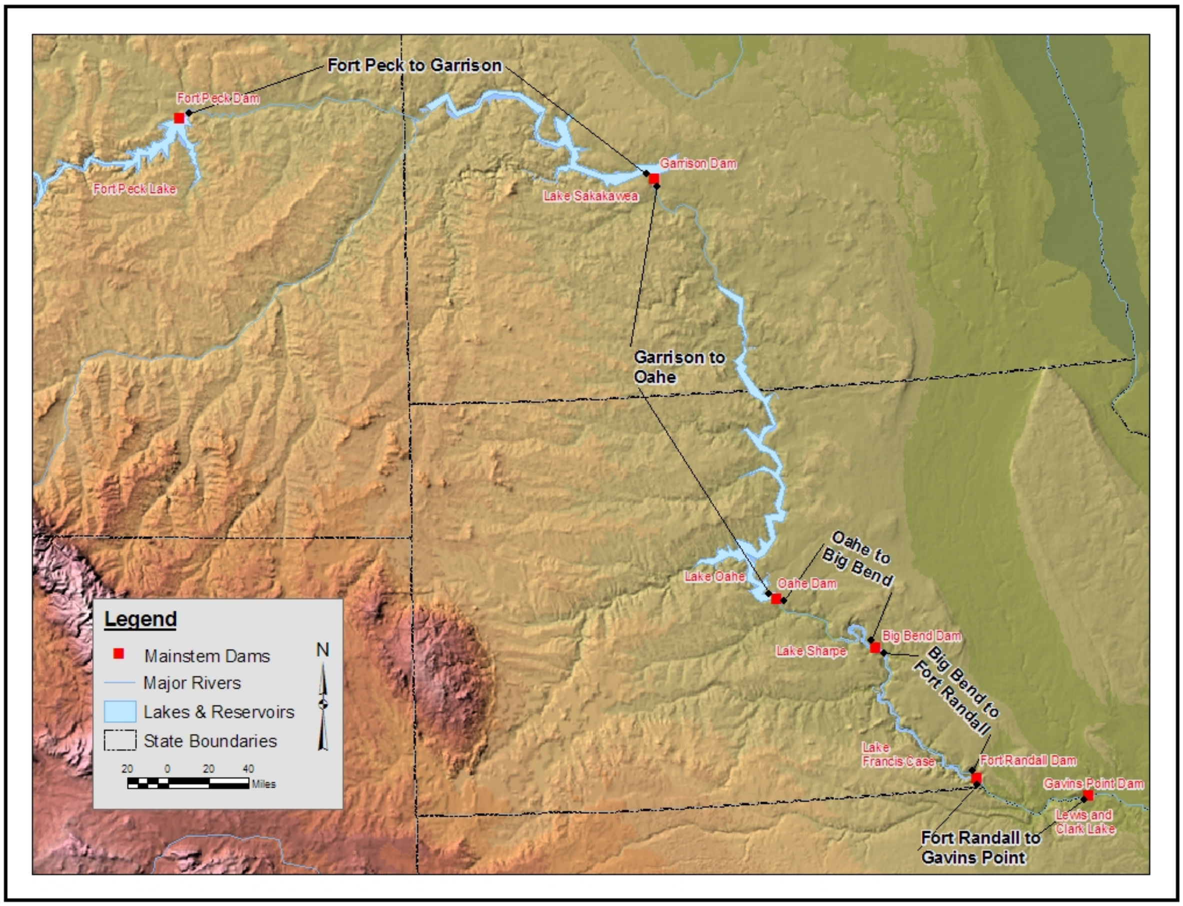


Figure 1. Mainstem Dam Modeling Reaches

Cross Section Interval

Model assembly principles and the goals of the study indicate that a cross section interval on the order of 2500 to 3000 feet would be appropriate. The sediment range spacing typically varies on the order of 1 to 3 miles so cross sections were interpolated in RAS to obtain estimated bathymetry.

Linear Interpolation

The between 2 cross sections option in the cross section interpolation tool in RAS was used to interpolate the underwater portion of the cross sections between the sediment ranges. Using the option **Generate for display as perpendicular segments to reach invert** places the interpolated cross sections along the stream centerline. A maximum distance of 3000 feet was used and additional cords were added where needed (the default cords are at the ends, banks, and channel invert). As can be seen in Figure 2 below, the RAS interpolated cross sections were imported into ArcMap and were adjusted to better represent the channel and floodplain. These new re-drawn cross sections were then used in GeoRAS to obtain elevation data along the correct alignment. The estimated (RAS interpolated) bathymetry was then merged into the re-drawn overbank cross section data.

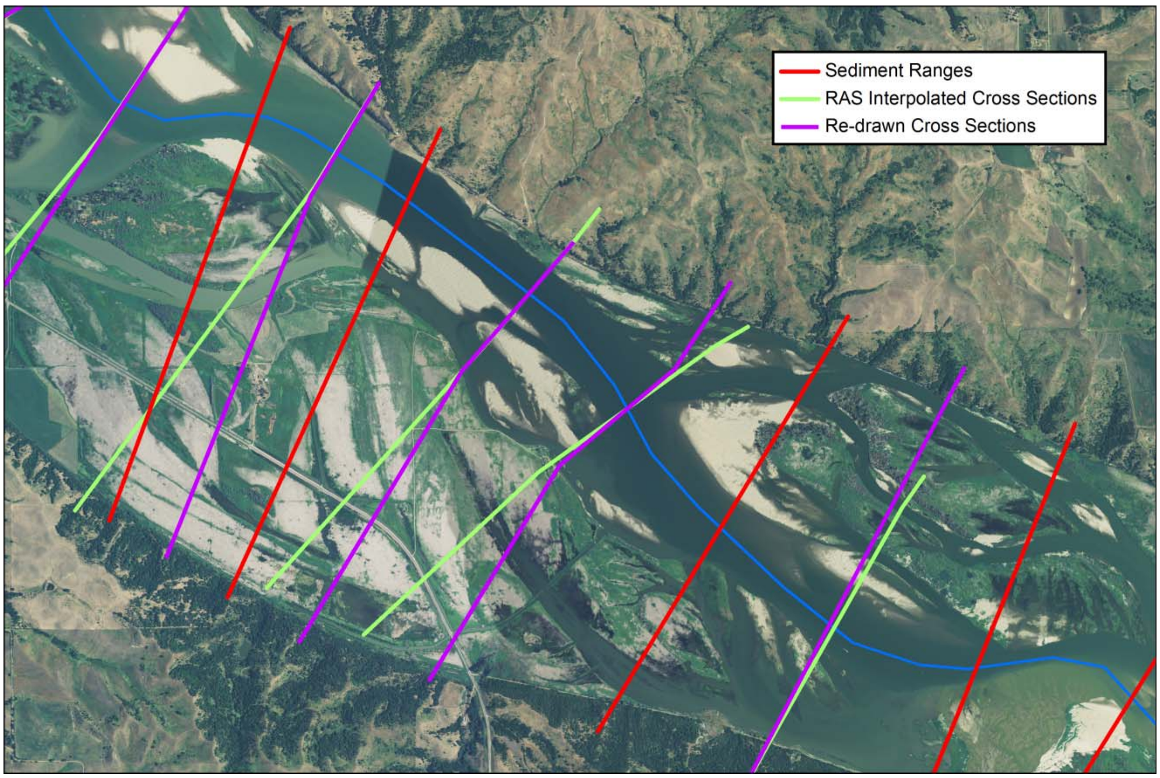


Figure 2. Comparison of RAS interpolated and re-drawn cross sections.

Estimating Bathymetry

After the new cross sections were re-cut in GeoRAS with LiDAR data, an underwater portion needed to be added to the cross section since the LiDAR does not penetrate below the water surface. Bathymetry was estimated by either using the RAS interpolated bathymetry or if that did not fit correctly with the overbank data, a nearby sediment range's bathymetry was vertically shifted and merged in. Differing widths and sandbar configurations presented a challenge to find another cross section that was similar.

When using a nearby range's bathymetry as an estimate a vertical shift was applied. The shift was based on the energy grade line slope (broken into several reaches) and the distance between the two cross

sections. See Figure 3 for an example of an energy grade line slope plot from a rough sediment range only model.

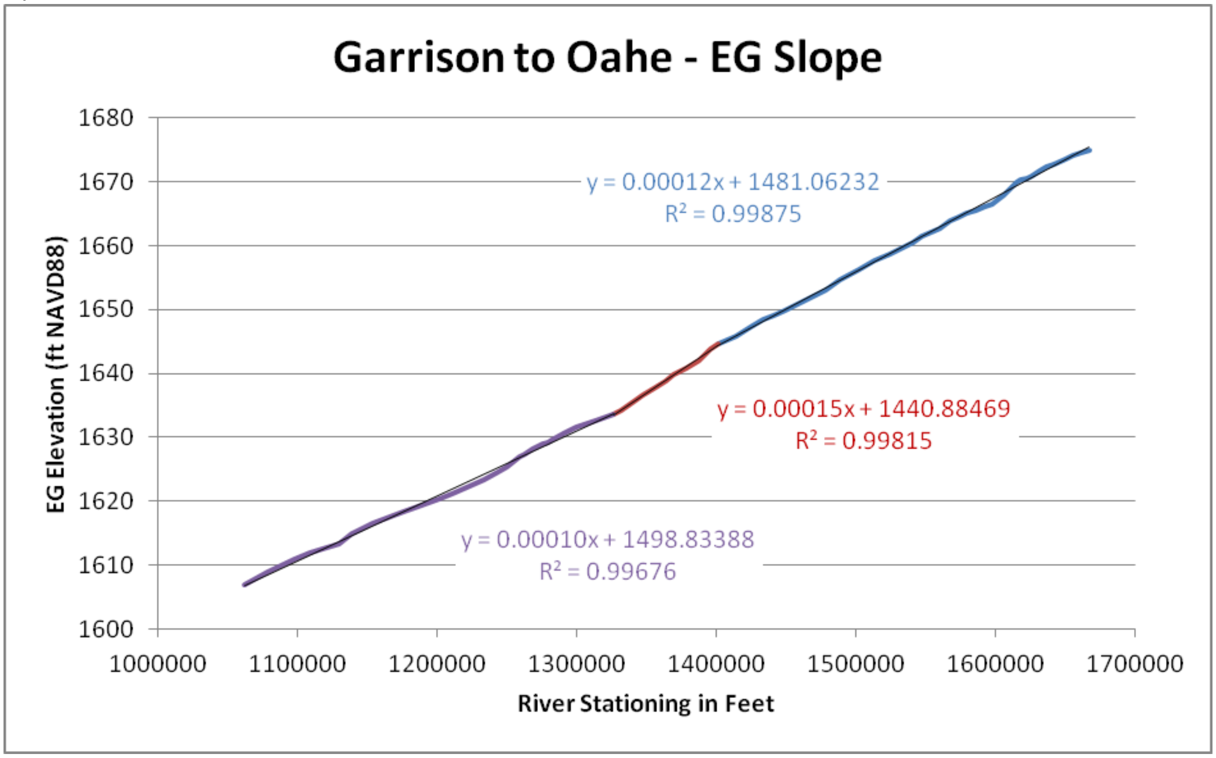


Figure 3. Example of Energy Grade Line Slope Plot

Examples of merging bathymetry into the cross sections are illustrated in Figures 4 – 9. The four example sites are shown in Figures 10 – 24.

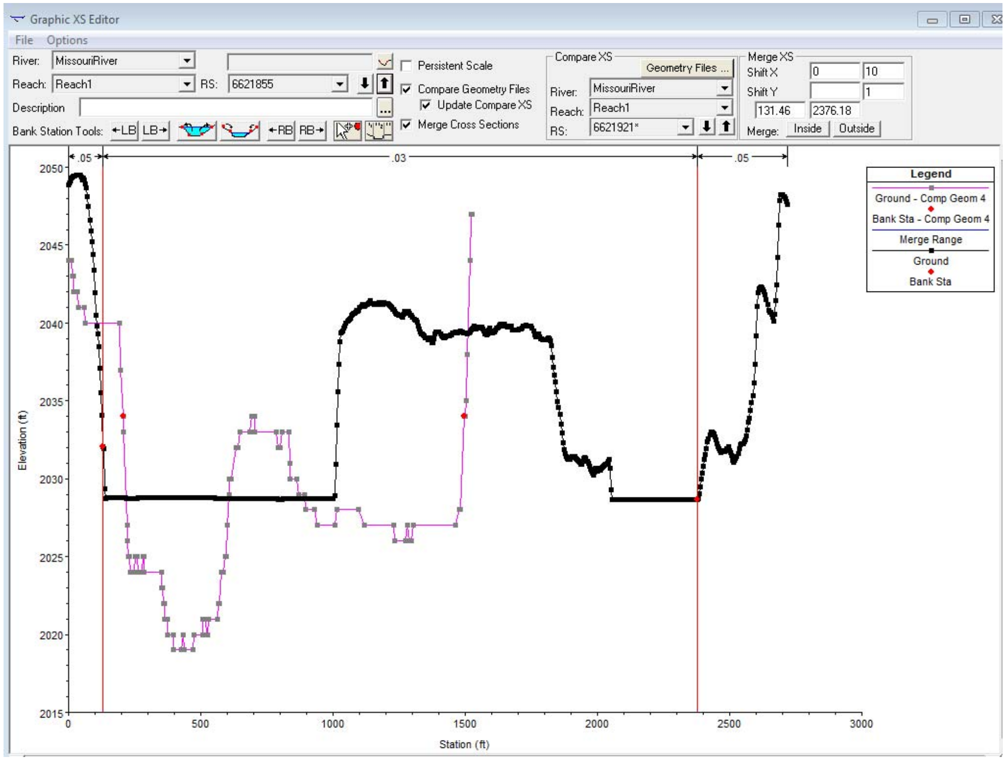


Figure 4. Paired XSs for merging, center island and changing channel widths cause alignment issues.

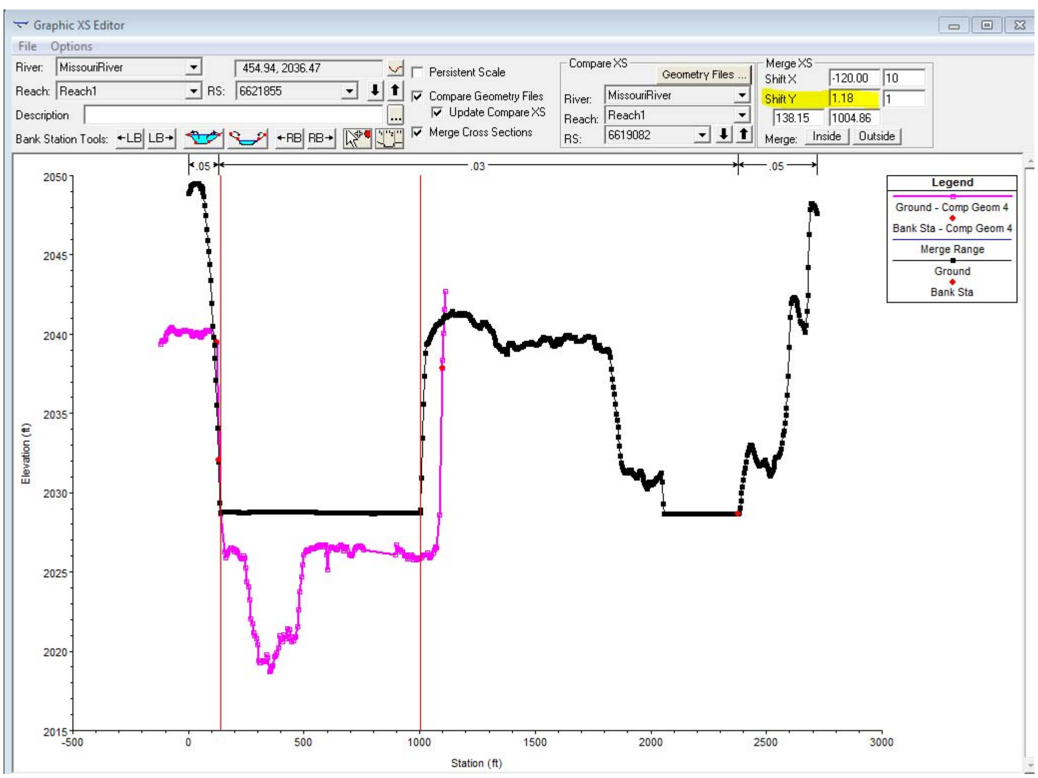


Figure 5. Same Cross-Section as in Figure with next downstream Rangeline vertically adjusted

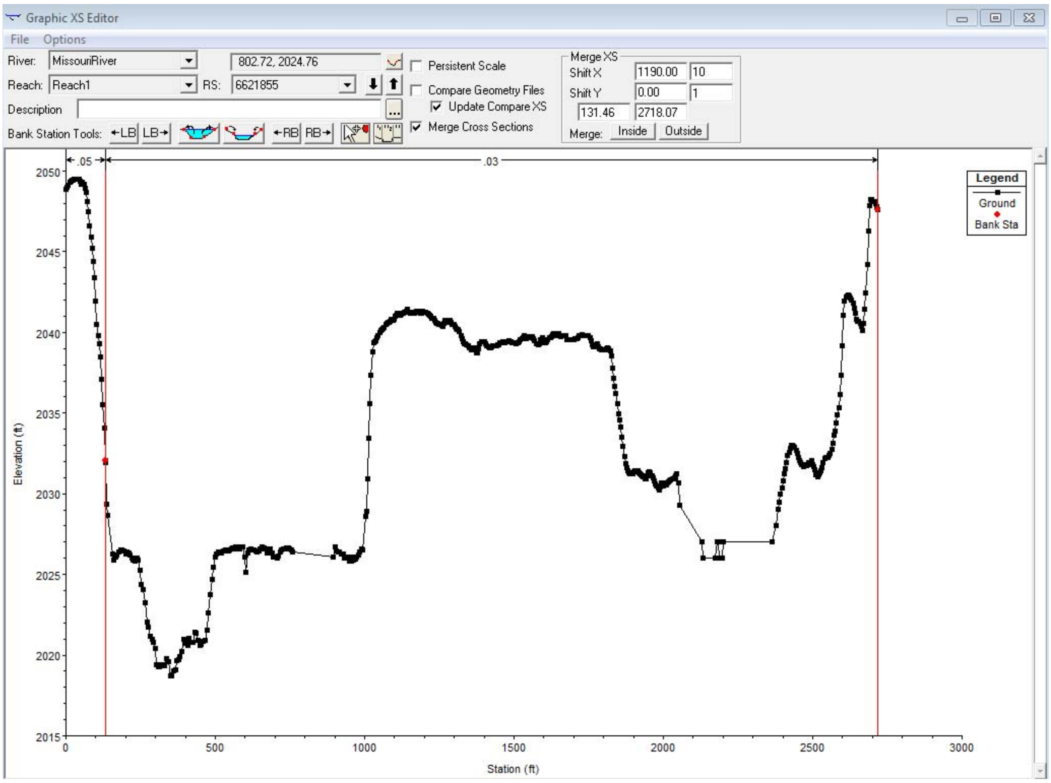


Figure 6. Composite Cross Section using the GeoRAS cut LiDAR data for above the WSE and the downstream rangeline and HEC-RAS interpolated cross sections for the channel data estimation.

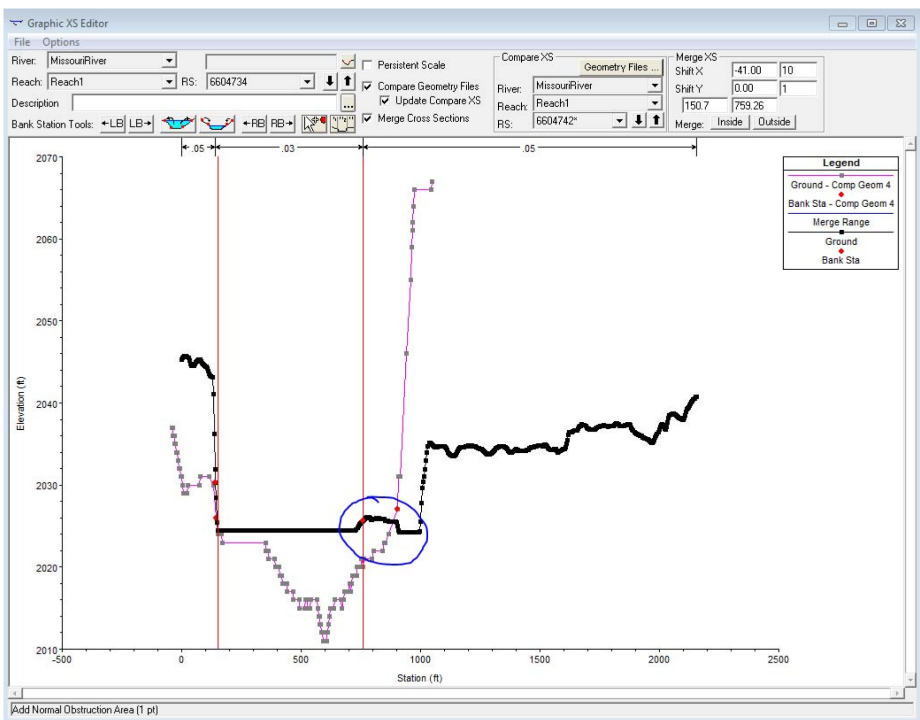


Figure 7. Sandbars in the channel present another challenge.

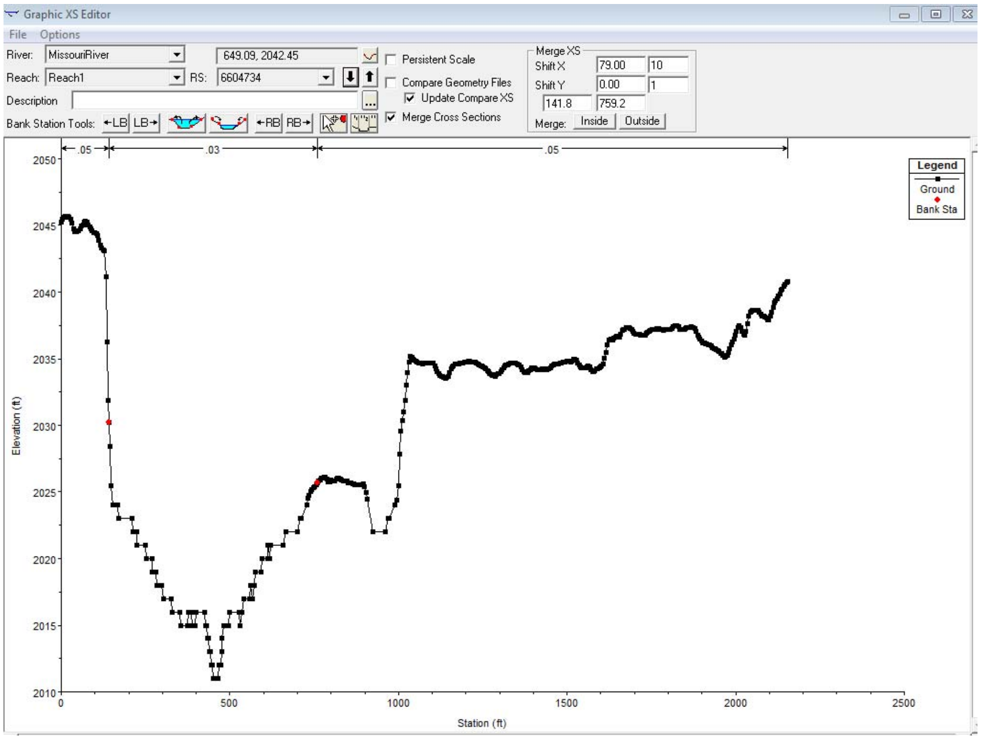


Figure 8. Composite Cross Section from Figure with sandbar

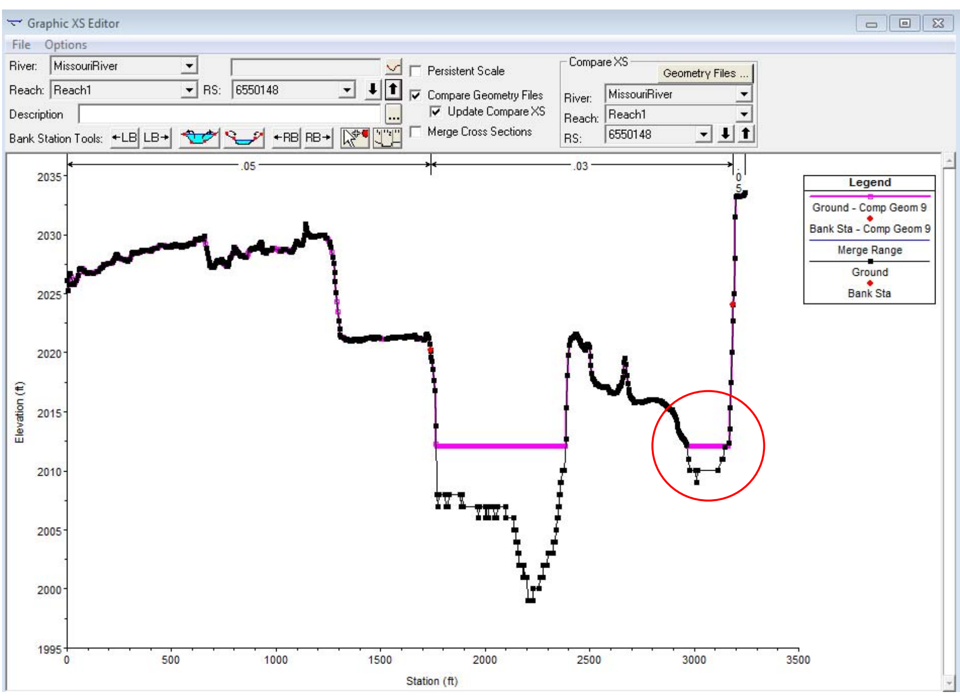


Figure 9. Before and After Merge - Small Channel inside of sandbar is Estimated

Example Sites

Comparison of RAS output built using Rangeline Interpolation and LiDAR data Merged with interpolated cross-sections. Images are at a Flow of 10,250 cfs which is about a normal annual flow in the Ft Peck Reach.

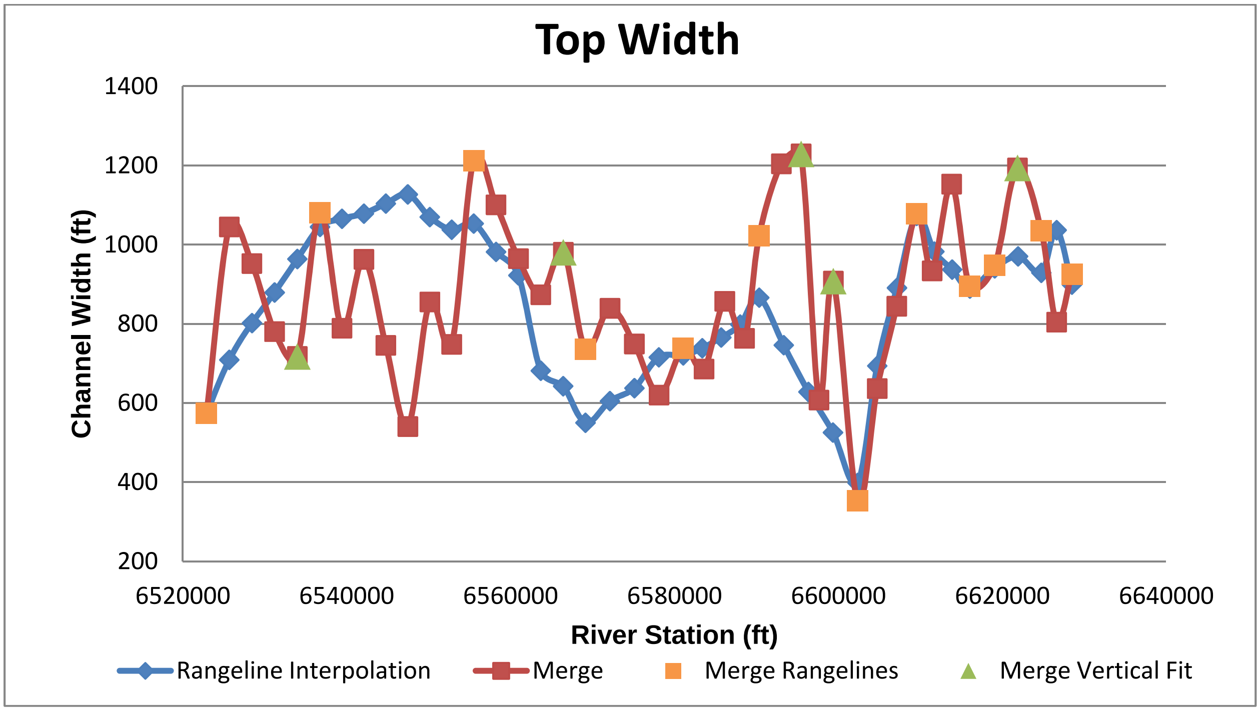


Figure10. Comparison of the RAS output for Top Width from the linearly interpolated rangeline XS model and the merged topo and rangeline model. The rangeline XSs based on survey data are orange. The green markers denote the topo XSs that didn't fit with the corresponding interpolated XS and used a more suitable nearby XS vertically fit to the local slope to merge the below water channel.

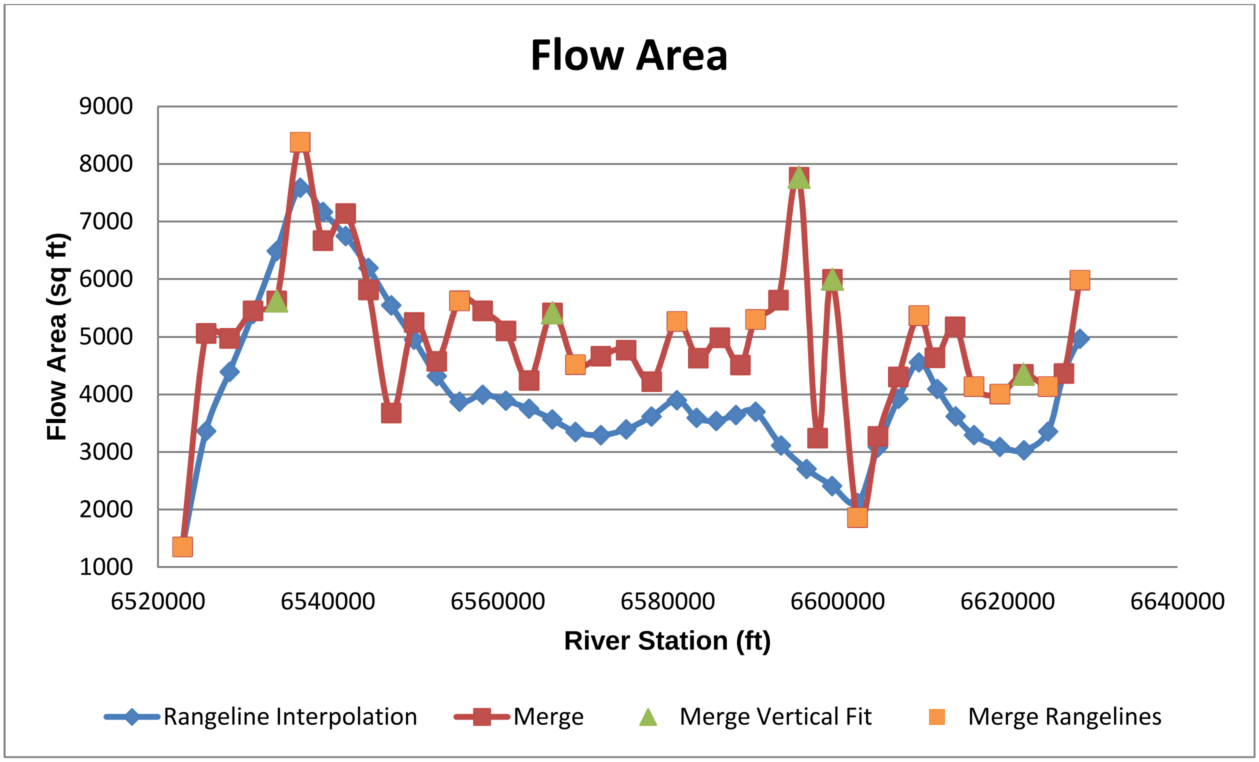


Figure11. Comparison of the RAS output for Flow Area from the linearly interpolated rangeline XS model and the merged topo and rangeline model.

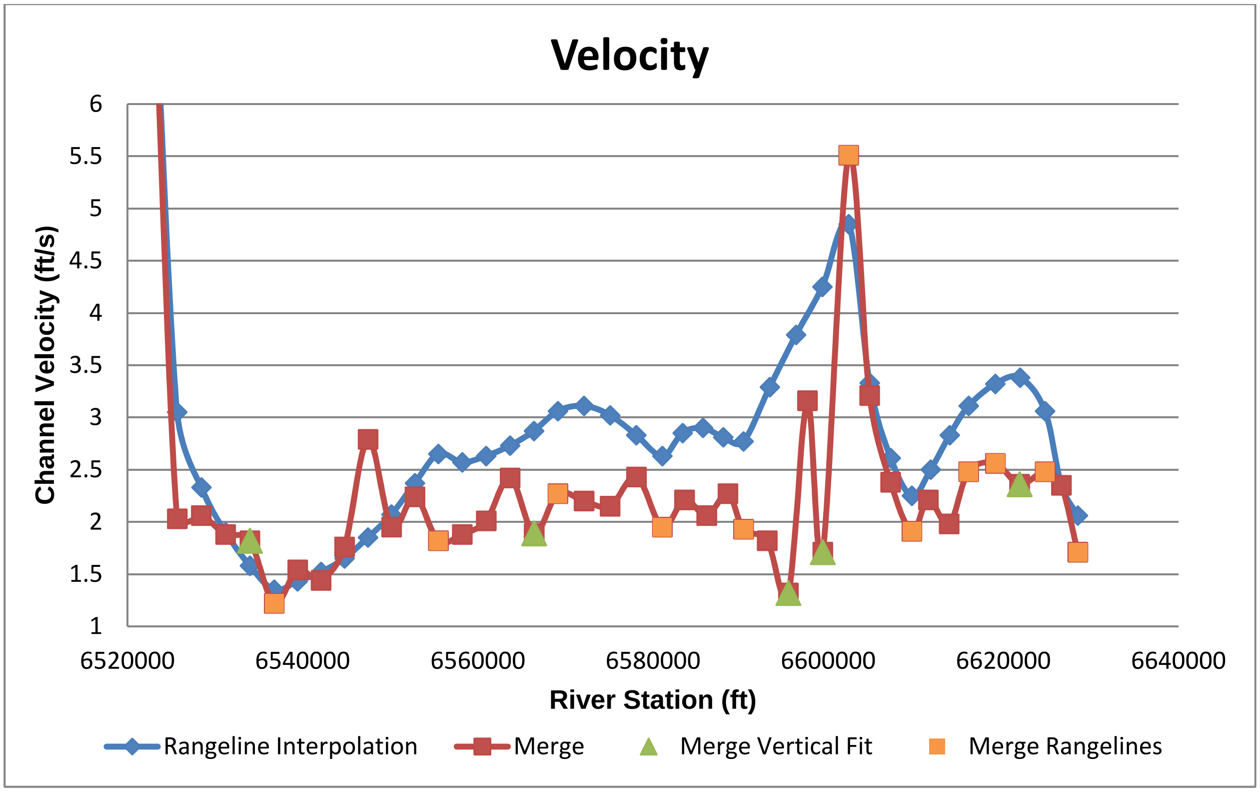


Figure 12. Comparison of the RAS output for Velocity from the linearly interpolated rangeline XS model and the merged topo and rangeline model.

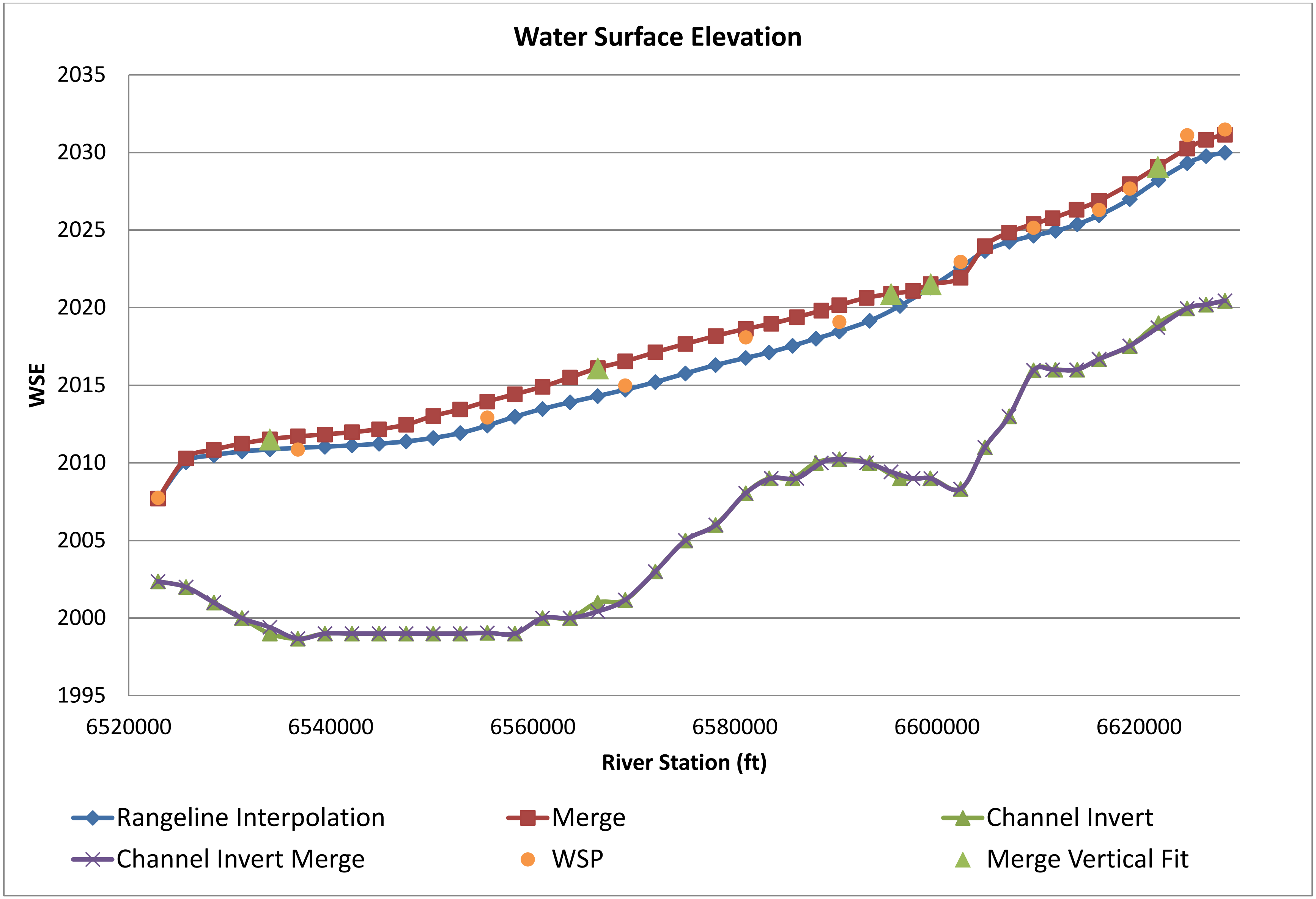
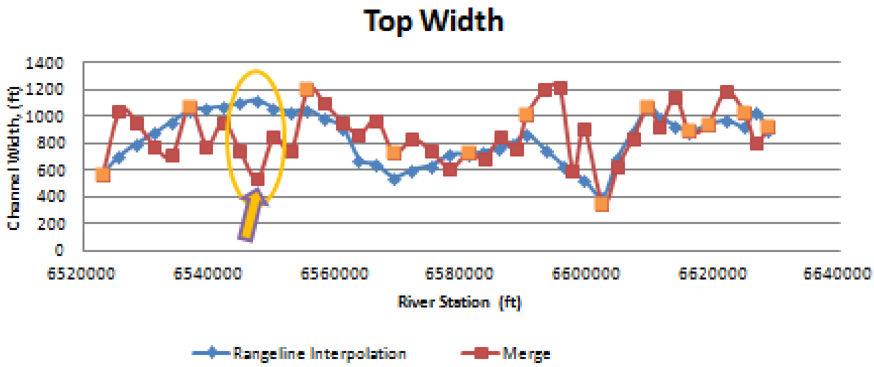


Figure 13. RAS Models compared to the 2012 Water Surface Profile (WSP) at Flow 10,250 cfs

Example 1



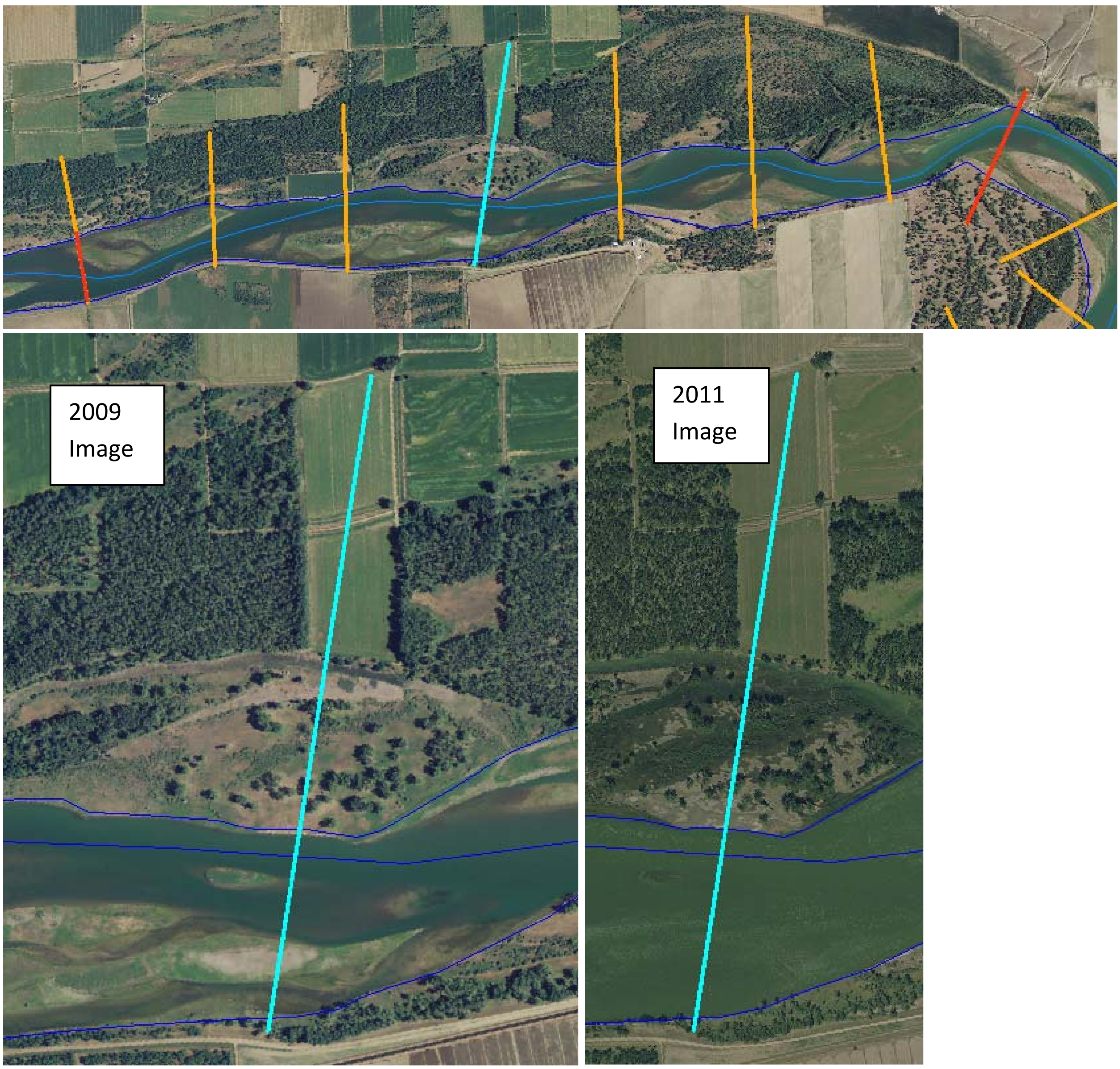


Figure 14. Aerial images of the XS, including the nearest Rangelines in the top image and closer view of the XS during a normal to low flow in 2009 and a high flow in 2011

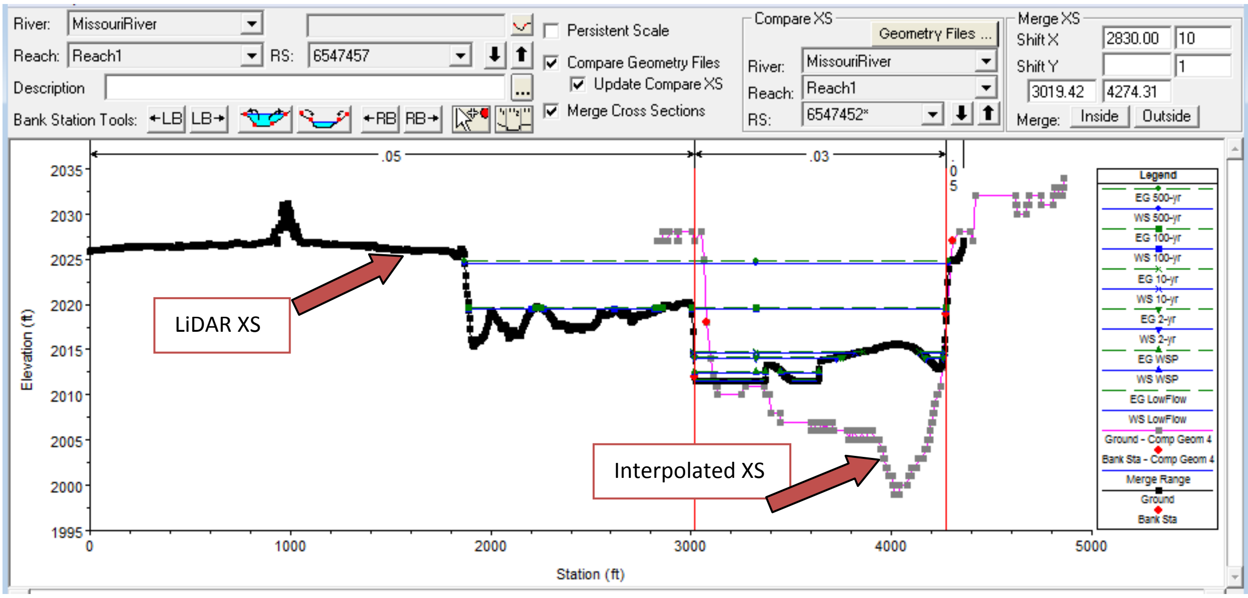


Figure 15. LiDAR XS (Black) and corresponding Interpolated XS (Grey/Pink) – note sandbars in channel

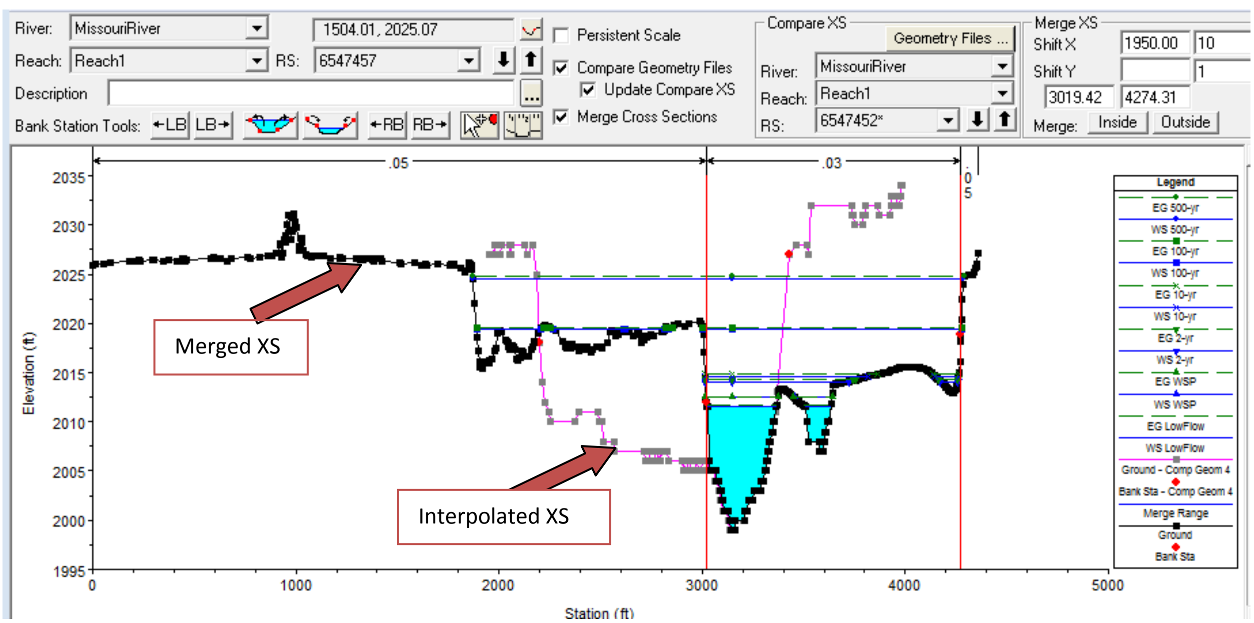
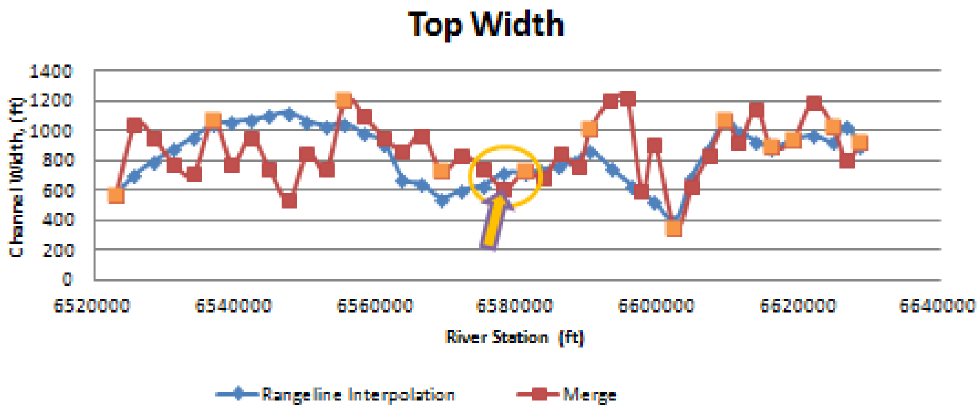


Figure 16. Merged XS (Black) and corresponding Interpolated XS (Grey/Pink)

Example 2



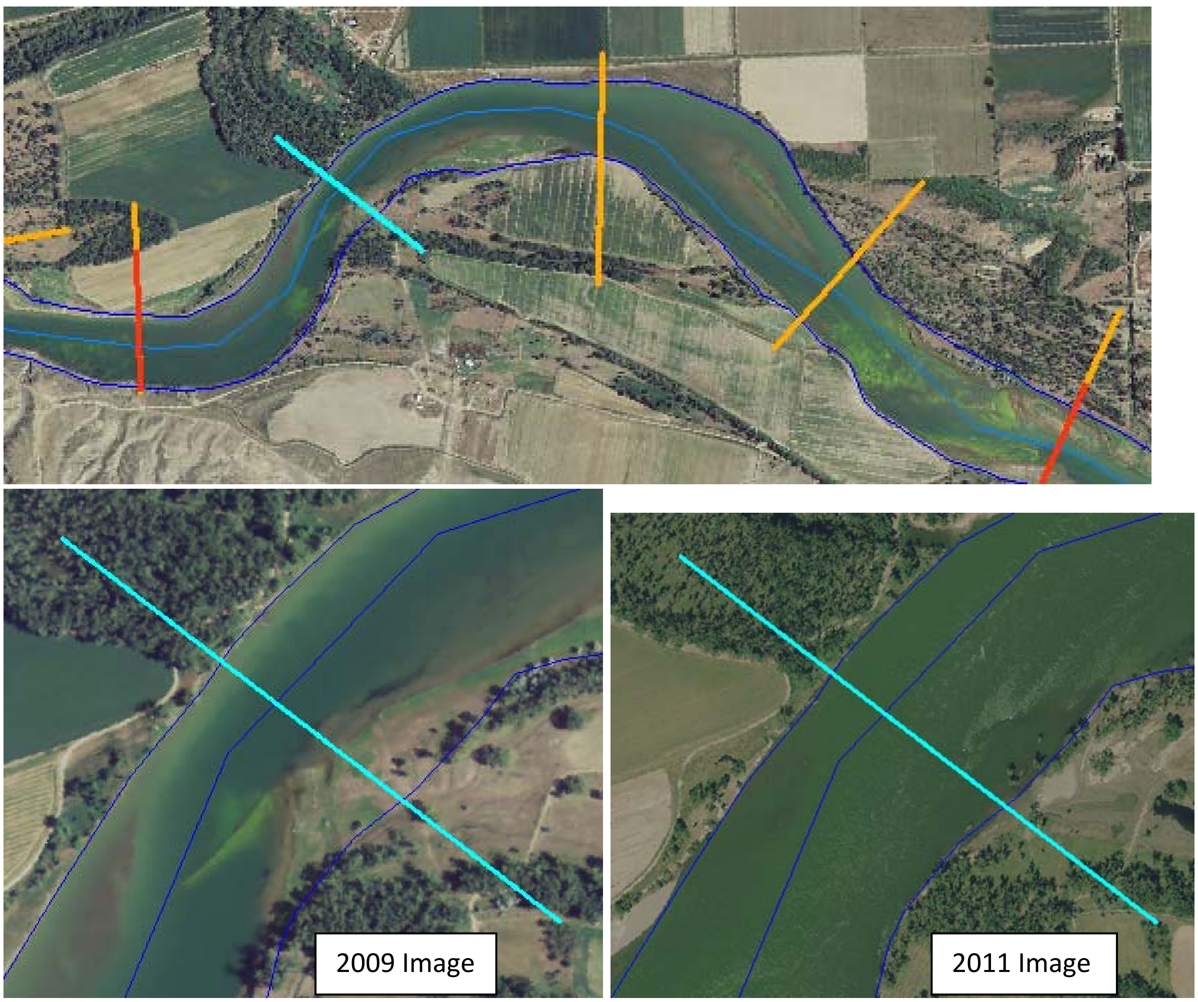


Figure 17. Aerial images of the XS, including the nearest Rangelines in the top image and closer view of the XS during a normal to low flow in 2009 and a high flow in 2011.

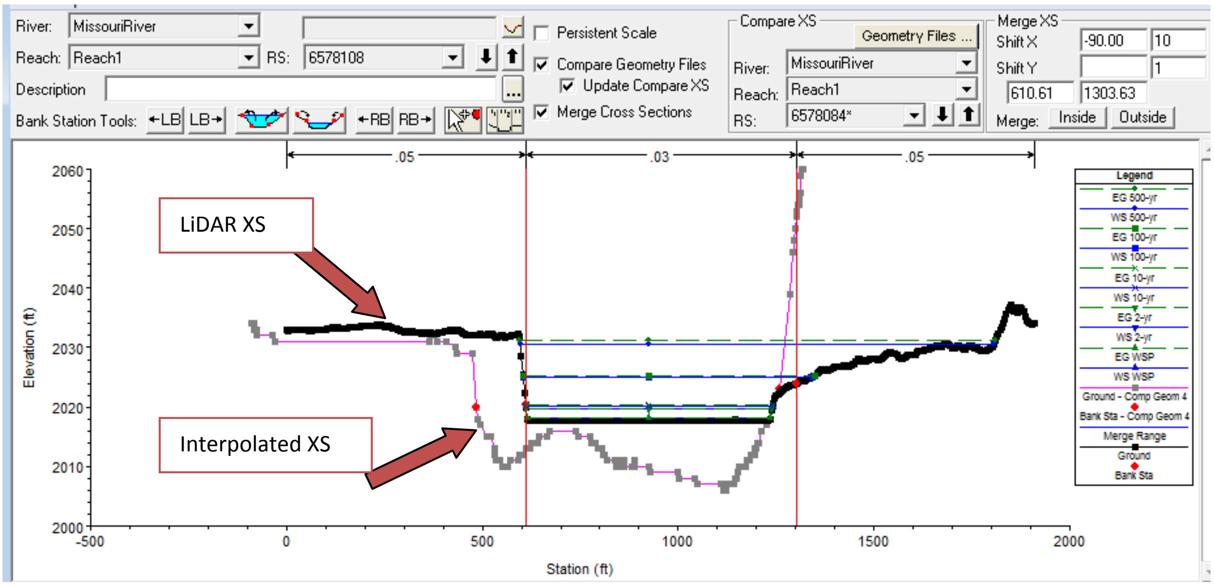


Figure 18. LiDAR XS (Black) and corresponding Interpolated XS (Grey/Pink)

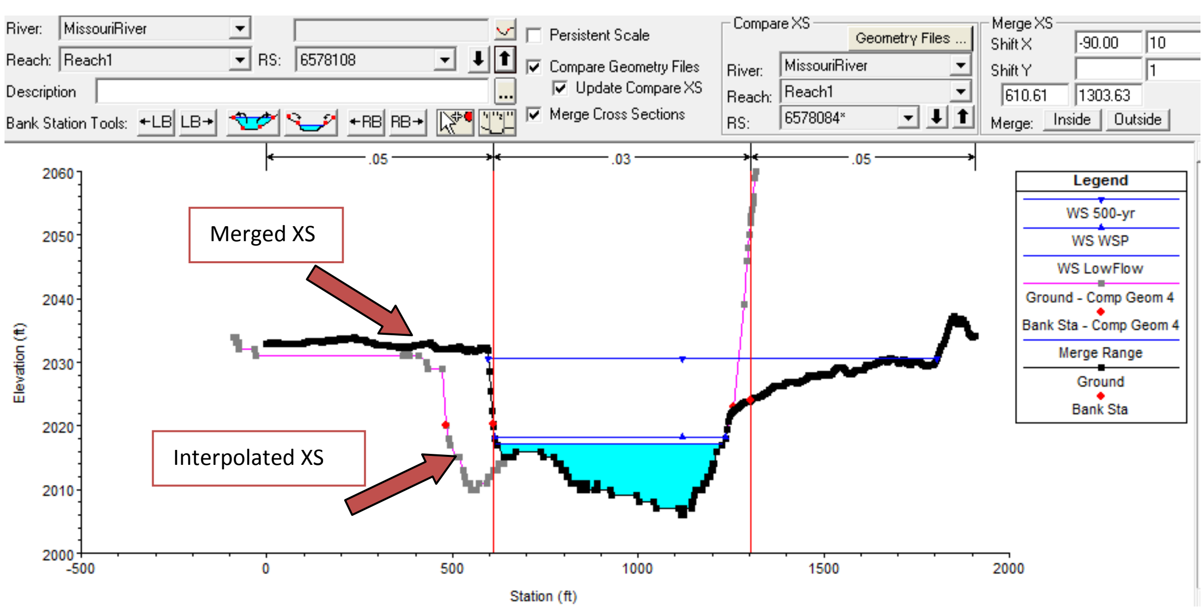
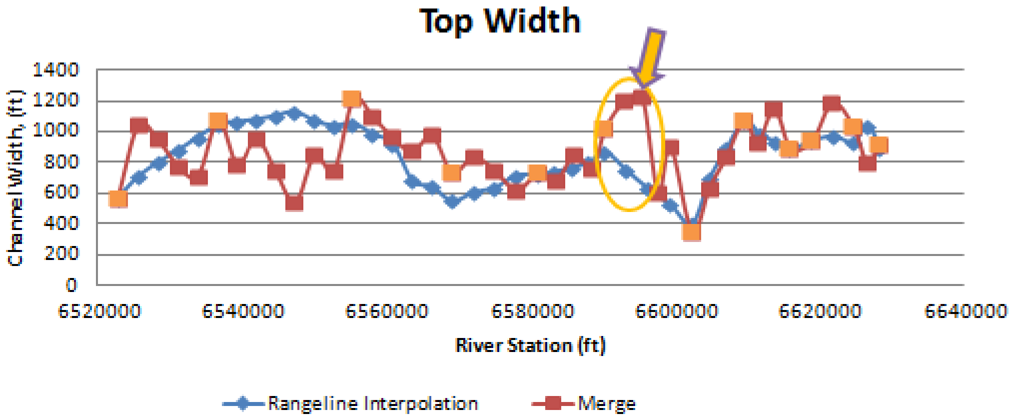


Figure 19. Merged XS (Black) and corresponding Interpolated XS (Grey/Pink)

Example 3

Used the downstream rangeline cross section with vertical adjustment to better fit cross section width



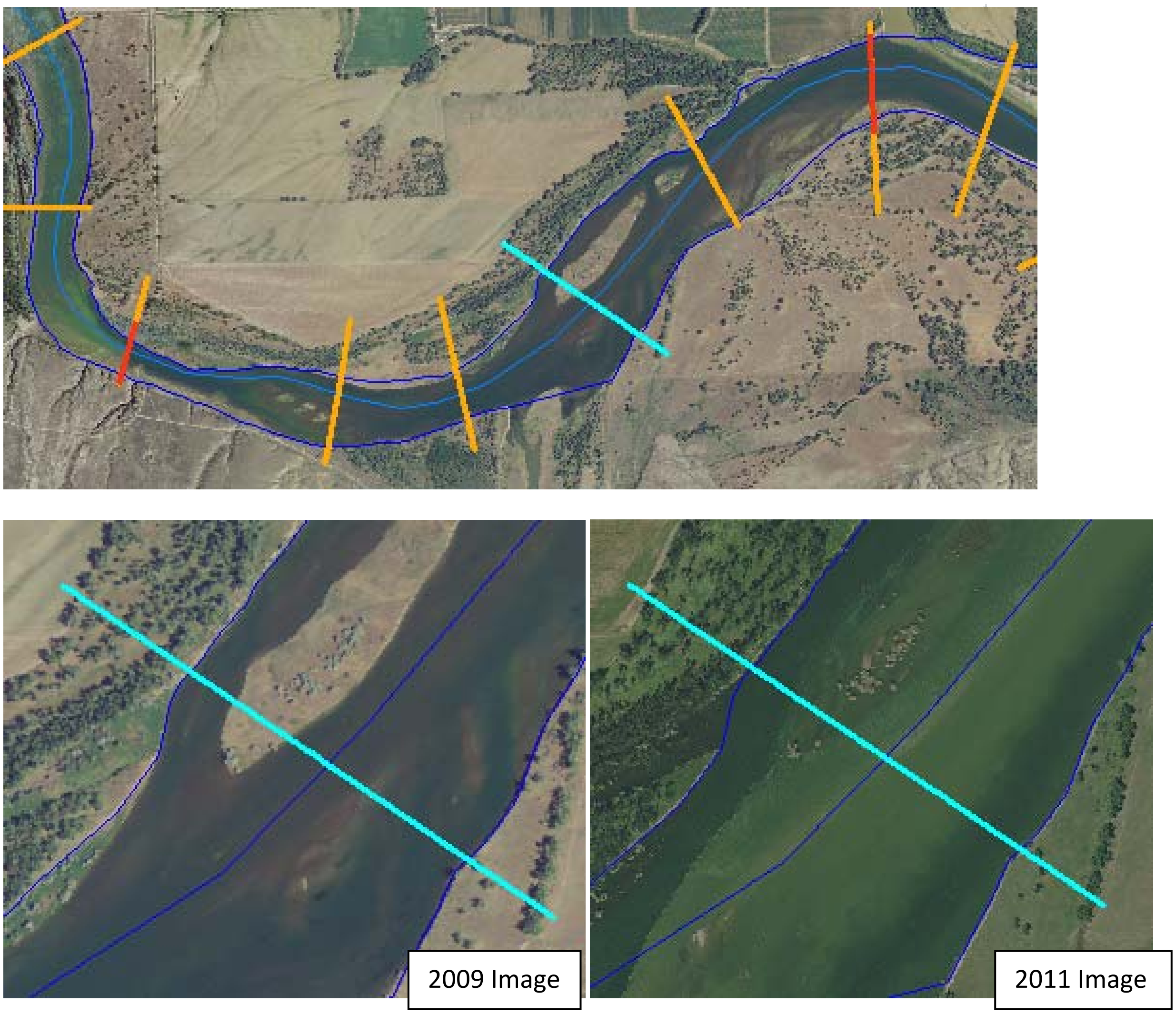


Figure 20. Aerial images of the XS, including the nearest Rangelines in the top image and closer view of the XS during a normal to low flow in 2009 and a high flow in 2011.

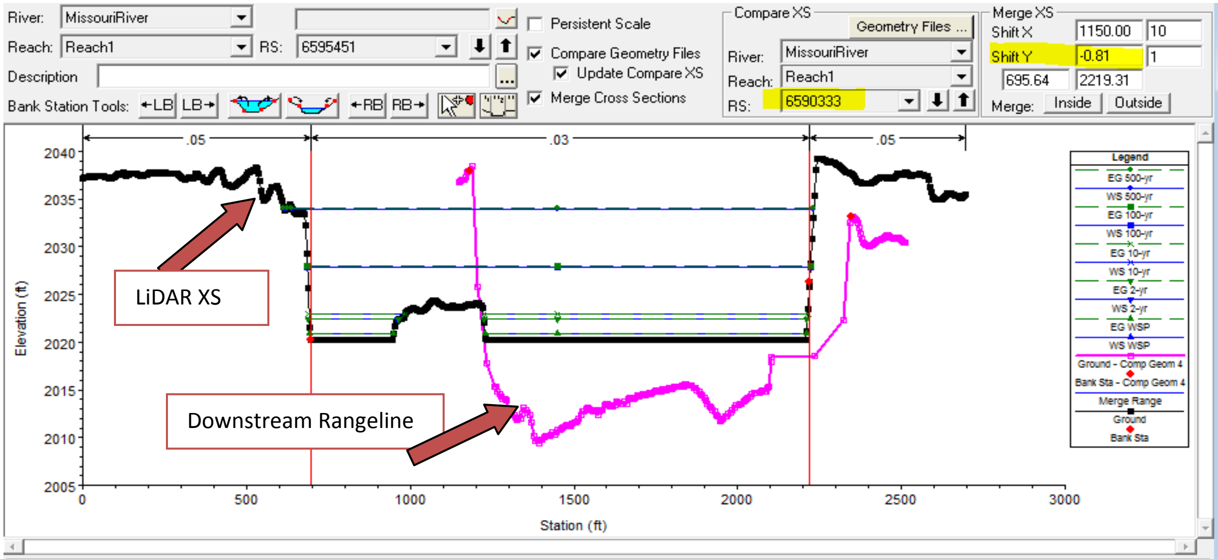


Figure 2. LiDAR XS (Black) and the next downstream Rangeline XS (Pink) vertically adjusted were used for merge due to the difference in channel width of the interpolated XS (see next figure).

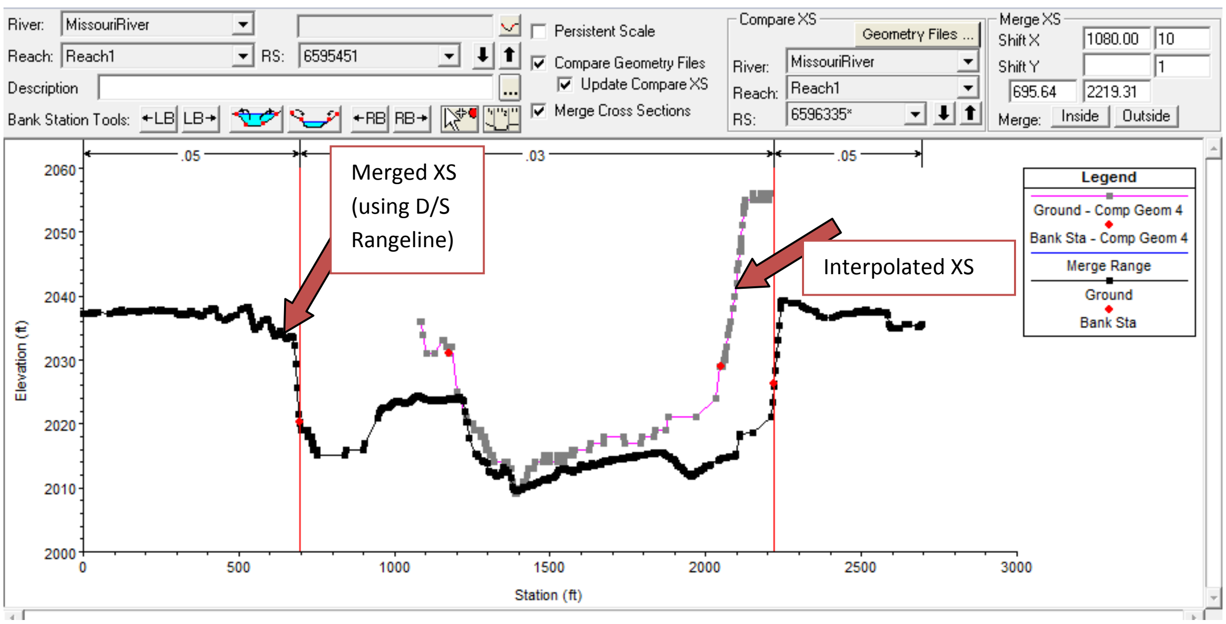
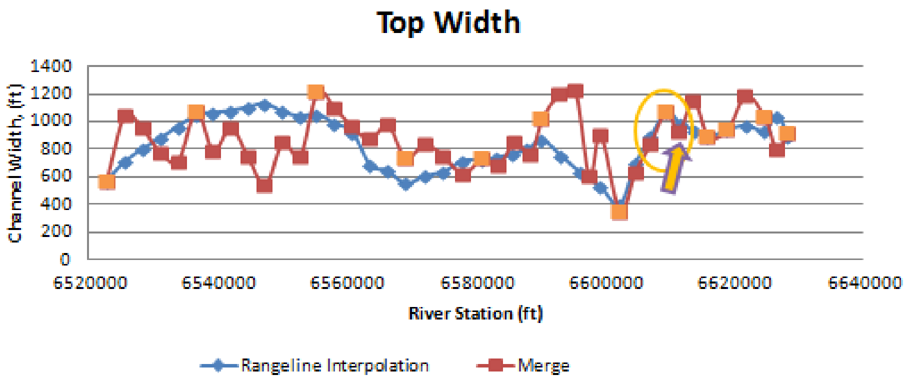


Figure 21. The final merged XS (Black) and the corresponding Interpolated XS (Pink/Grey) which was not used in this merge process.

Example 4



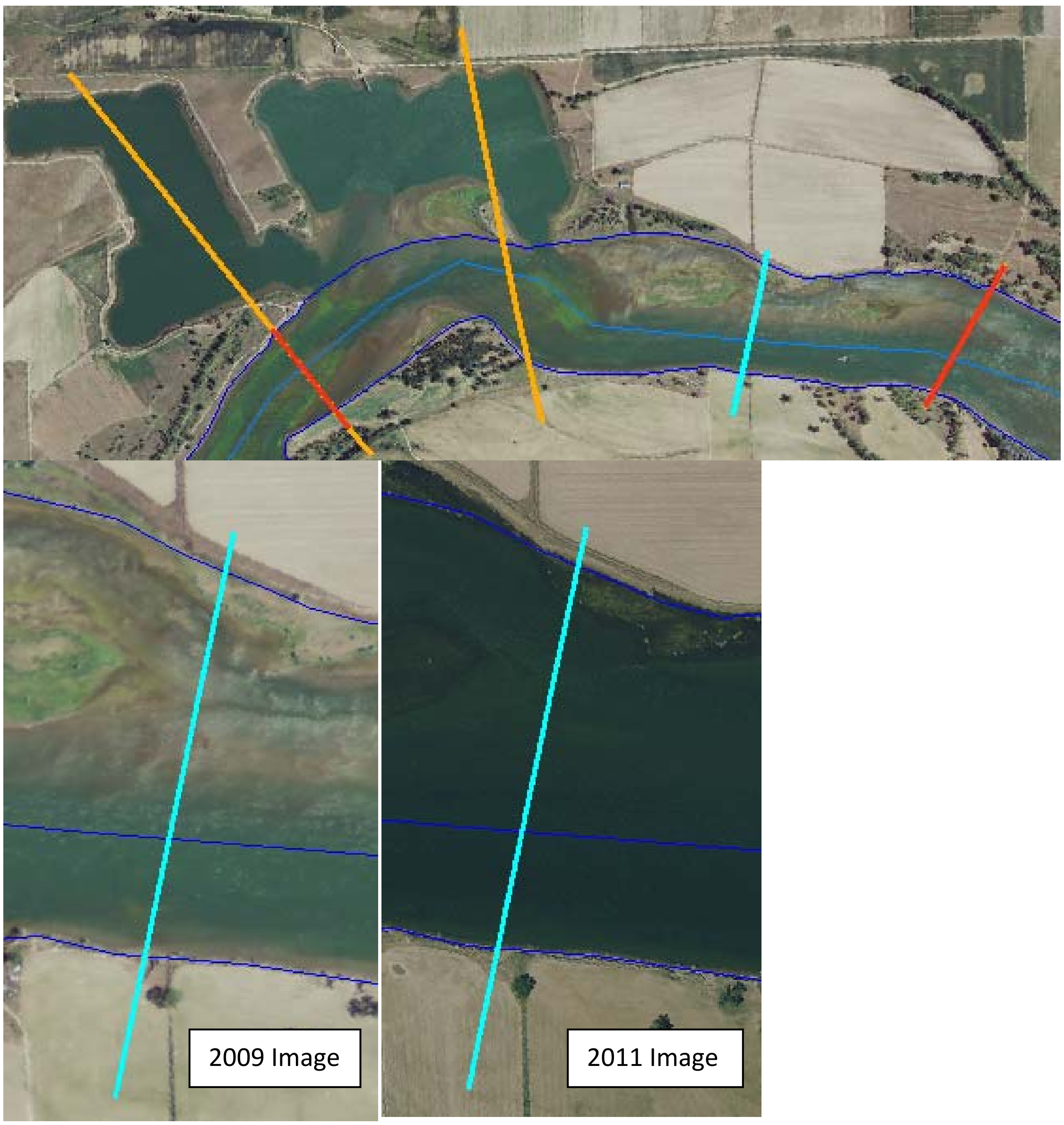


Figure 22. Aerial images of the XS, including the nearest Rangelines in the top image and closer view of the XS during a normal to low flow in 2009 and a high flow in 2011.

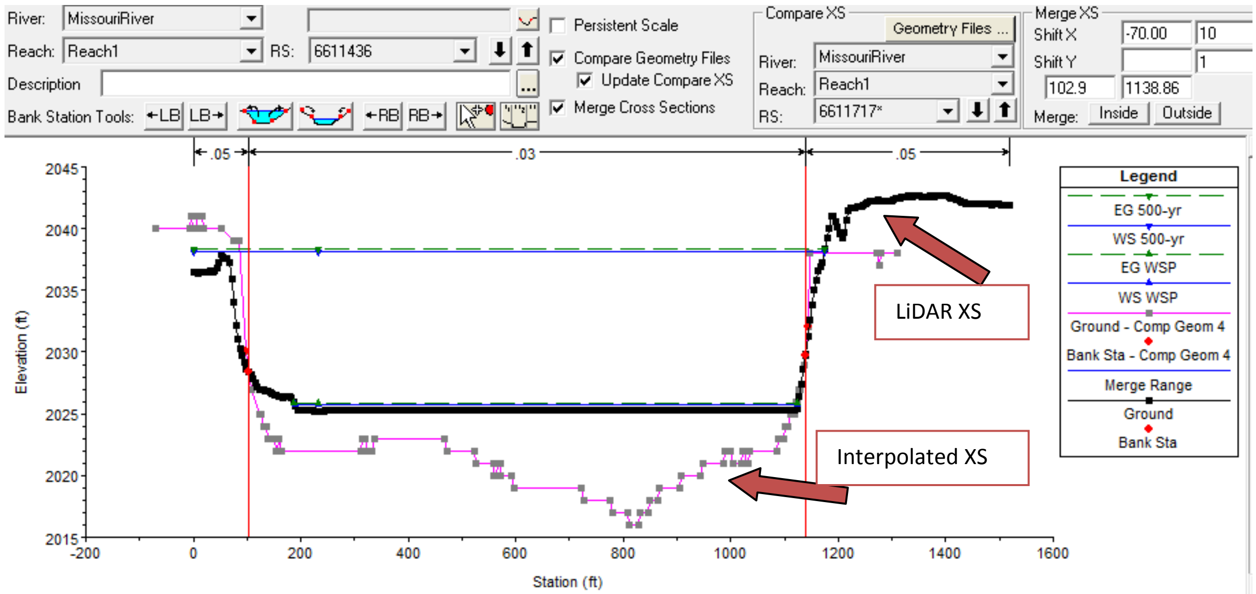


Figure 23. LiDAR XS (Black) and corresponding Interpolated XS (Grey/Pink) - XS had a good fit for channel width, note it is near a rangeline with little change between the Rangeline and XS locations.

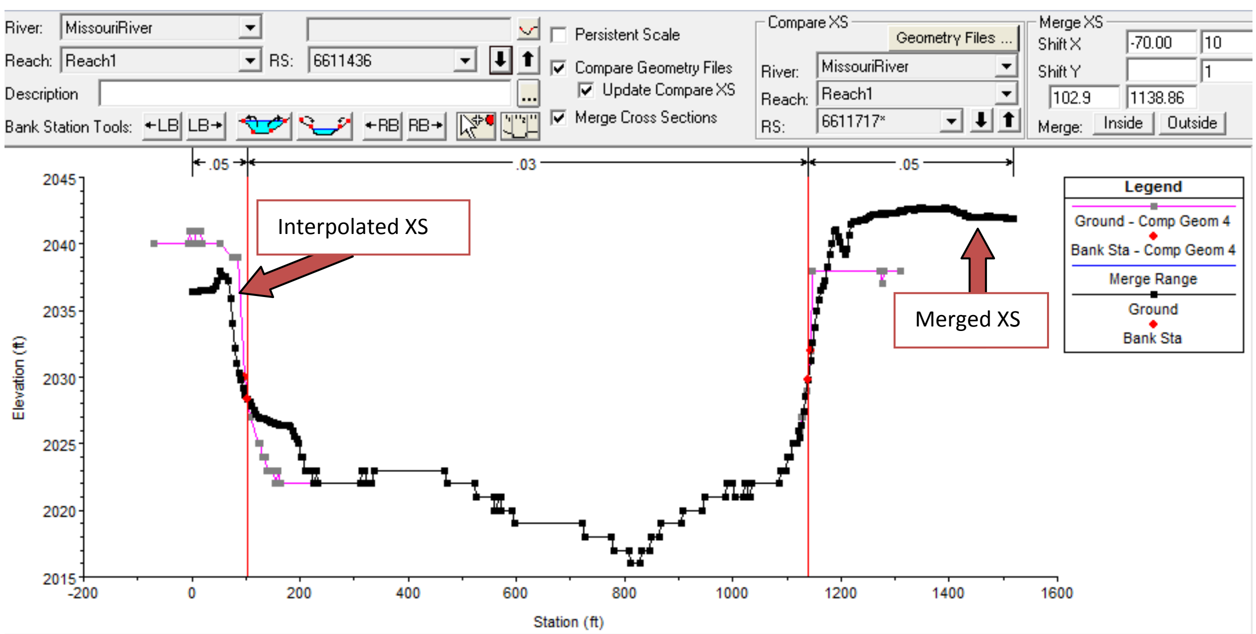


Figure 24. Merged XS (Black) and corresponding Interpolated XS (Grey/Pink).

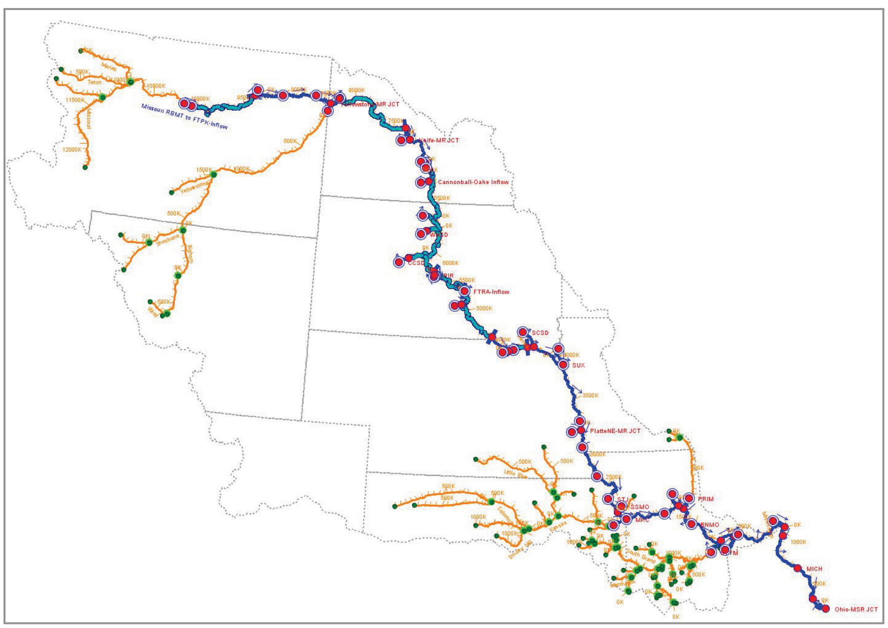
US Army Corps of Engineers®

Omaha District

Missouri River Unsteady HEC-RAS Model
Calibration Report

Appendix D

Gavins Point Dam to Rulo, NE



July 2018

FINAL

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ACRONYMS

CFS…………………... Cubic Feet per Second

DEM…………………. Digital Elevation Model

DTM .... Digital Terrain Model

DSSVue…………….. Data Storage System (by HEC)

GIS .... Geographic Information System

HEC .... Hydrologic Engineering Center

LiDAR………………..Light Detection and Ranging

MAF…………………..Million acre-feet

NAD 1983………… ... North American Datum of 1983

NAVD 88…………… . North American Vertical Datum of 1988

NGVD 29……………. National Geodetic Vertical Datum of 1929

MRBWM…………... .. Missouri River Basin Water Management Division (previously RCC)

NWK………………… Northwest Division Kansas City District

NWO………………… Northwest Division Omaha District

POR .... Period of Record

RAS........................... River Analysis System Software (by HEC)

RCC .... Reservoir Control Center

ResSim……………...Reservoir Simulation Software (by HEC)

RM……………………1960 River Mile

System……………....Missouri River Mainstem System

USACE……………… United States Army Corps of Engineers

USGS…………………United States Geological Survey

1 EXECUTIVE SUMMARY

The Gavins Point Dam to Rulo, NE, reach of the Missouri River begins with the regulated outflow from Gavins Point Dam in South Dakota at 1960 River Mile (RM) 811.1. The reach extends approximately 250 miles downstream to Rulo, NE at RM 498.0 which is the Omaha District boundary with Kansas City District. This reach was modeled in Hydrologic Engineering Center River Analysis System (HEC-RAS) Version 4.2 Beta with the intent to update to Version 5.0 when it is released. The model was initially created in steady flow and then completed with unsteady modeling, and is now fully unsteady.

Inputs into the model are a flow hydrograph for the Gavins Point Dam release and flow hydrographs for the upstream boundaries of the larger gaged tributaries within the Omaha District consisting of the James River, Vermillion River, Big Sioux River, Little Sioux River, Soldier River, Boyer River, Platte River, Weeping Water Creek, Nishnabotna River, Little Nemaha River, and Tarkio River.

Output includes stage and flow hydrographs, as well as a number of additional calculated parameters such as average velocities, flow depth, and etc. available at specified locations. The latest version of HEC-RAS also has the ability to create inundation depth grids at various time-steps using RAS Mapper that can be exported for use in ecological and economic models.

The model extends downstream from Rulo, NE, to the St Joseph, MO, vicinity at RM 448.2, using data provided by Kansas City District to provide reasonable computation results for reporting at Rulo, NE. Therefore, the Omaha District and the Kansas City District models include an overlap reach at the Rulo, NE, boundary. This report will focus only on the Omaha District portion of the model. The geometry was constructed using the most recent surveys from the Omaha District, which included topographic data from fall 2011 LiDAR. This data was supplemented with state and 4 meter data in some areas when needed to extend coverage within the wide floodplain or levee cell area but not within the active river flow corridor. Below water data was available from hydrographic cross section survey data from Ponca to Rulo in 2012 and from Gavins Point Dam to Ponca in 2013 for the main Missouri River channel at 250 foot spacing intervals. The flow data for the Gavins Point Dam release and inflow tributaries were obtained from the Omaha District database and USGS gages.

Levee storage areas and lateral structures were used to describe the federal levee system between Omaha and Rulo (RM 620 to RM 515). The complex network of private levees in the Rulo vicinity were also included as storage cells in the model. The levee and storage area connections were set to a very low weir coefficient to enhance model stability and also reflect the non-weir flow conditions with limited downstream conveyance. Efforts to evaluate the effect of the weir coefficient indicated some impact on peak stage elevations. A reasonable value was determined after comparison to some historic events. All levees were modeled with overtopping only, no breaches were included.

Valley wide cross sections were extracted from the topography and retained to allow for future alternative condition modeling in multiple configurations if necessary. Therefore, blocked obstructions were included to remove the levee protected area from the cross sections and prevent double counting of storage. Blocked obstructions were used rather than point deletion to allow for possible future modeling options. Blocked obstructions were also used in the area upstream of Omaha that does not include levee cells. These obstructions were necessary to limit the available storage, to allow the RAS coding of levee confinement near the main channel, and to eliminate the wide portions of the section from storage.

Model calibration was performed for recent flow events in 2011 and 2012. The extreme 2011 flow event significantly altered the river stage-flow relationship and comparison to observed stages in flood years prior to 2011 is not valid in most areas. The model reach includes a substantial degradation section that extends downstream from Gavins Point Dam that is noticeable from stage trends. Degradation that occurred during the 2011 event is also apparent. In addition, the 2011 extreme event model calibration within the federal levee reach is not possible at many locations due to multiple levee breaches that occurred. The model is constructed with post 2011 extreme flood geometry. This resulted in some notable calibration issues. For instance, the Nebraska City reach with the levee setback appears low in the model calibration, likely due to the geometry change. Since none of the levee breaches are included within the model, calibrating to observed flow / stage levels in areas highly impacted by levee breaches is not possible. Calibration data consists of observed hydrographs at gage station locations and measured water surface elevation profiles from both 2011 and 2012. The computed water surface profile was within +/- 1 ft along the entire reach and in the range of +/- 0.5 ft for approximately 50% to 75% of the reach. These were determined to be acceptable calibration targets based on accuracy attained during previous studies on the Missouri River. Comparison to observed hydrographs indicated that the model performed acceptably on timing of flood peaks within most areas. Poor calibration was noted in the downstream end of the model for the 2011 event for the areas affected by levee breaches.

HEC-RAS model construction differences occurred between the Omaha and Kansas City District modeling efforts due to changes in river features. Downstream of Rulo, NE, where the navigation structures are larger and have a significant impact on conveyance at low flows, ineffective flow areas were used to represent the navigation structure impact on channel conveyance. Other minor differences such as tieback modeling technique and calibration period also occurred. Refer to the model geometry and calibration discussion in each appendice for additional details.

2 INTRODUCTION

The Missouri River unsteady HEC-RAS model was created as a base model for planning studies which could be used to simulate and analyze broad scale watershed alternatives. The objective of this HEC-RAS model is to simulate current conditions on the Missouri River, with the intention of running period of record (POR) flows to compare alternatives. Future reports will address period of record runs, this report addresses model construction and calibration. This Appendix is for the Gavins Point Dam to Rulo, NE, reach of the Missouri River as part of the Omaha District. The Omaha District and the Kansas City District models include an overlap reach at the Rulo, NE, district boundary. This report will focus only on the Omaha District portion of the model. Refer to the Rulo, NE, to St. Louis Appendix E for details regarding the downstream model.

3 BACKGROUND

The Gavins Point Dam to Rulo, NE, model includes about 250 river miles of the Missouri River and an additional 50+ miles downstream of Rulo, NE, to establish the boundary condition.

3.1 MODEL EXTENT

Hydraulic modeling was performed along the Missouri River in a series of RAS models as shown in Figure 3-1. This report pertains to a portion of the model segment downstream of Gavins Point Dam to the confluence with the Mississippi River at St. Louis. The Omaha District portion of the hydraulic model extends from Gavins Point Dam, at river mile (RM) 811.1, downstream to Rulo, NE, at RM 498.0. Rulo, NE. The Rulo, NE, location corresponds with the Omaha District boundary with the Kansas City District. The Missouri River drainage area increases from 279,500 square miles at Gavins Point Dam to 414,900 square miles at Rulo, NE. In order to provide an accurate downstream boundary, the hydraulic model also includes an overlap section with the Kansas City District model from Rulo, NE to St. Joseph, MO. The Kansas City District model also includes an overlap reach for their upstream boundary from Nebraska City to Rulo. The overlap section is shown in Figure 3-1. Extending the model to St. Joseph, MO, adds an additional Missouri River length of about 50 miles to the hydraulic model. All features pertaining to the Missouri River downstream of Rulo, NE, are described within the Kansas City District section of the report, appendix E.

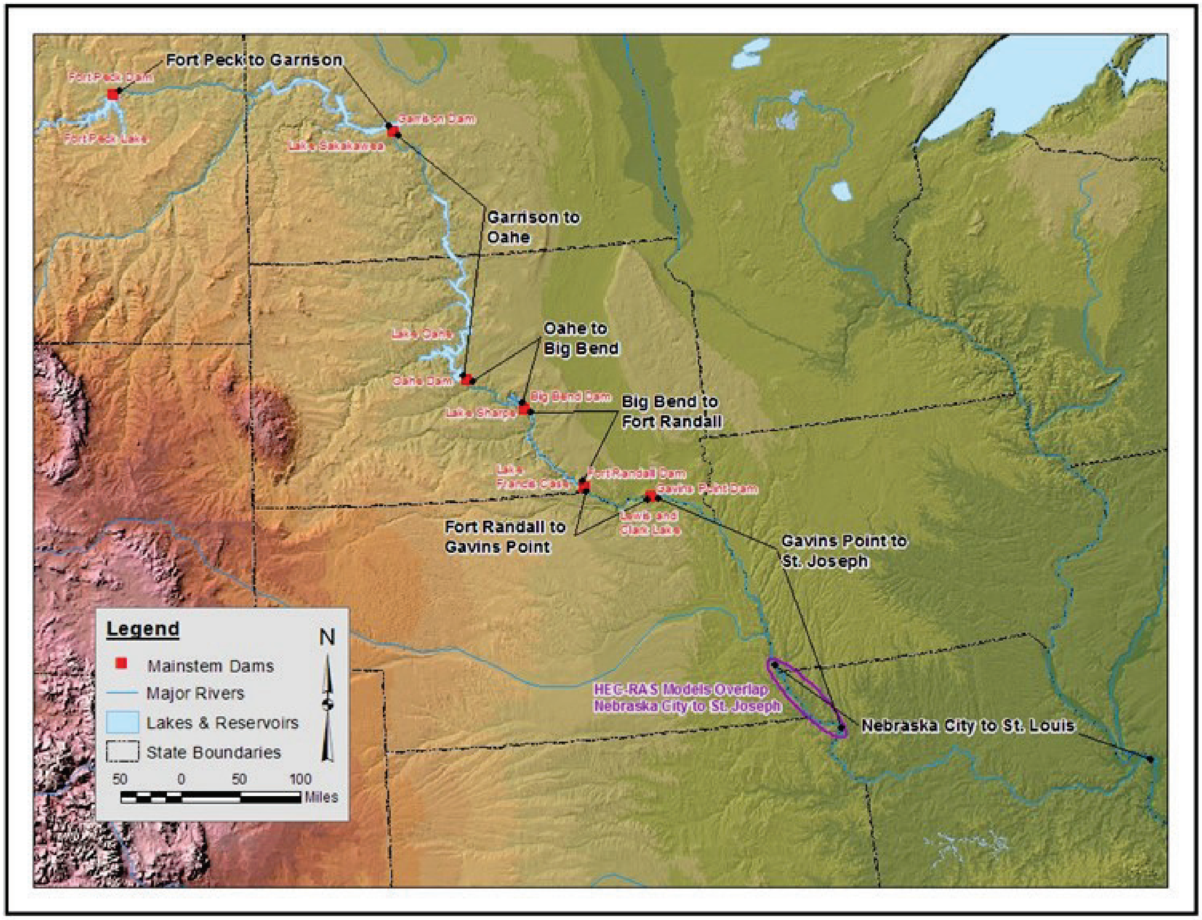


Figure 3-1. Model Extents

Shown in Figure 3-2 is a schematic of the Gavins to Rulo modeled area. The schematic illustrates the Missouri River gaging stations on the main stem, tributaries that are included as routing reaches, lateral inflows to the model, and the river mile location of hydrologic features. All river miles referenced in the Omaha District appendix use the 1960 mileage for the Missouri River.

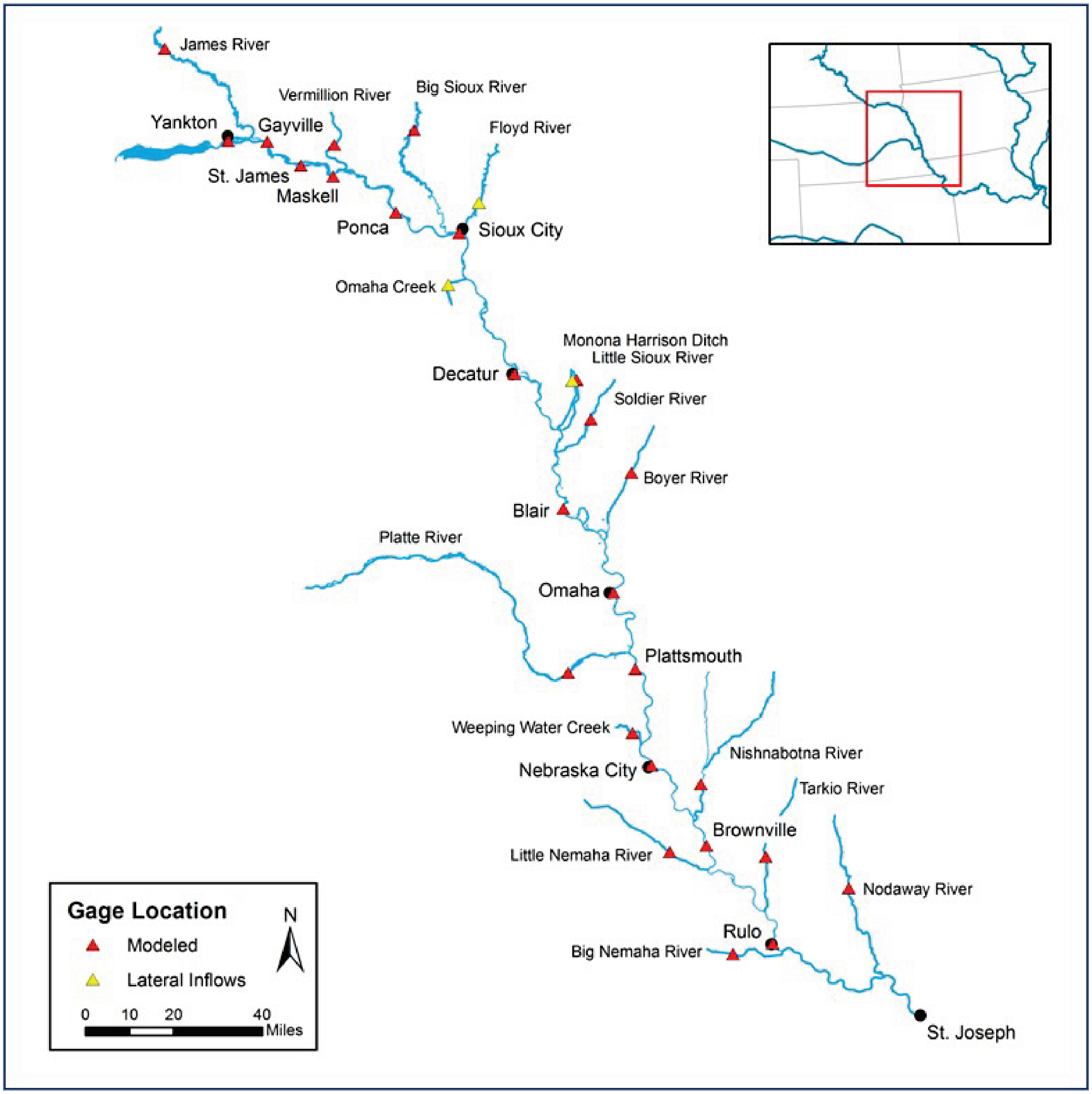


Figure 3-2. Model Schematic

3.2 MISSOURI RIVER MAINSTEM RESERVOIR SYSTEM DESCRIPTION

The Missouri River Mainstem System (System) of dams is composed of six large earth embankments which impound a series of lakes that extend upstream for 1,257 river miles from Gavins Point Dam near Yankton, South Dakota to the head waters of Fort Peck Lake north of Lewiston, Montana. These dams were constructed by USACE for flood control, navigation, power production, irrigation, water supply, water quality, recreation, and fish and wildlife enhancement. Fort Peck Dam, the oldest of the six dams, was closed and began water storage in 1937. Fort Randall Dam was closed in 1952, followed by Garrison Dam in 1953, Gavins Point Dam in 1955, Oahe Dam in 1958, and Big Bend Dam in 1963. The current System of six projects first filled and began operating as a six-project System in 1967. At the top elevation of their normal operating pool level, the lakes behind these six dams provide about 1,146,000 acres of water surface area and extend a total length of 755 river miles. Only 325 miles of open river remain between the lakes, although there are 811 miles of open river downstream from Gavins Point Dam to the mouth of the Missouri River where it enters the Mississippi River at St. Louis, Missouri. The reservoirs contain an aggregate storage volume of approximately 73 million acre-feet (MAF) of which more than 16 MAF is for flood control.

Regulation of the System is according to the current Master Manual (USACE, 2006) and generally follows a repetitive annual cycle. Winter snows and spring and summer rains produce most of the year's water supply, which results in rising pools and increasing storage accumulation. After reaching a peak reservoir level, usually during July, storage declines until late winter when the cycle begins anew. A similar pattern may be found in rates of releases from the System, with higher flows from mid-March to late November, followed by low rates of winter discharge from late November until mid-March, after which the cycle repeats.

Two primary high-risk flood seasons are the plains snowmelt season extending from late February through April and the mountain snowmelt period extending from May through July. Overlapping the two snowmelt flood seasons is the primary rainfall flood season, which includes both upper and lower basin regulation considerations.

Power generation is a component of System operation. The highest average power generation period extends from mid-April to mid-October with high peaking loads during the winter heating season (mid-December to mid-February) and the summer air conditioning season (mid-June to mid-August). The power needs during winter are supplied primarily with Fort Peck and Garrison releases and the peaking capacity of Oahe and Big Bend. During the spring and summer periods, releases are geared to navigation and flood control requirements and primary power loads are supplied using the four lower dams. During the fall when power needs diminish, Fort Randall pool is drawn down to permit generation during the winter period when the pool is refilled by Oahe and Big Bend peaking power releases. Gavins Point Dam, as the downstream-most reservoir, is operated at constant daily releases and is not used for daily power peaking.

Normally, the navigation season extends from April 1 through December 1 during which time reservoir releases are increased to meet downstream target flows in combination with downstream tributary inflows. Winter releases after the close of navigation season are much lower and vary depending on the need to conserve or evacuate system storage volumes, downstream ice conditions permitting. Minimum release restrictions and pool fluctuations for fish spawning management generally occur from April 1 through July. Endangered and threatened species, including the interior least tern and piping plover, nesting occurs from early May through August. During this period, special release patterns are made from Garrison, Fort Randall, and Gavins

Point to avoid flooding nesting sites on low-lying sandbars and islands downstream from these projects.

Overall, the general regulation principles presented above provide the backbone philosophy for the Missouri River Mainstem Reservoir System regulation. Detailed operation plans are developed, followed and adjusted as conditions warrant periodically as the System is monitored day-to-day. Beginning in 1953, projected operation of the System for the year ahead was developed annually as a basis for advance coordination with the various interested Federal, State, and local agencies and private citizens. These regulation schedules are prepared by the Missouri River Basin Water Management Division, Northwest Division, U.S. Army Corps of Engineers and are reported in Annual Operating Plans (USACE, 2013b).

In addition to the six main stem projects operated by USACE, 65 tributary reservoirs operated by the Bureau of Reclamation and USACE provide over 15 million acre-feet of flood control storage.

Numerous reservoirs and impoundments constructed by different interests for flood control, irrigation, power production, recreation, water supply, and fish and wildlife are located throughout the basin on various tributaries. The Bureau of Reclamation and USACE have constructed the most significant of these structures. Although primarily constructed for irrigation and power production, the projects constructed by the Bureau of Reclamation do provide some limited flood control in the upper basin.

Table 3-1 lists pertinent data for the Missouri River Mainstem projects (USACE, 2013a).

Table 3-1: Pertinent Data for Missouri River Mainstem Projects

Description	Fort Peck	Garrison	Oahe	Big Bend	Fort Randall	Gavins Point
River Mile (1960 Mileage)	1771.5	1389.9	1072.3	987.4	880.0	811.1
Drainage Area (sq. mi.)	57,500	181,400	243,490	249,330	263,480	279,480
Incremental Drainage Area (sq. mi.)	57,500	123,900	62,090	5,840	14,150	16,000
Gross Storage (kAF)	18,463	23,451	22,983	1,798	5,293	428
Flood Storage (kAF)	3,675	5,706	4,315	177	2,293	133
Top of Dam* (ft msl)	2280.5	1875.0	1660.0	1440.0	1395.0	1234.0
Maximum Surcharge Pool** (ft msl)	2253.3	1858.5	1644.4	1433.6	1379.3	1221.4
Top of Exclusive FC Pool*** (ft msl)	2250.0	1854.0	1620.0	1423.0	1375.0	1210.0
Top of Annual FC Pool (ft msl)	2246.0	1850.0	1617.0	1422.0	1365.0	1208.0
Base of Flood Control Pool (ft msl)	2234.0	1837.5	1607.5	1420.0	1350.0	1204.5
Spillway Capacity (cfs)	275,000	827,000	304,000	390,000	633,000	584,000
Outlet Capacity (cfs)	45,000	98,000	111,000	n/a	128,000	n/a
Powerplant Capacity (cfs)	16,000	41,000	54,000	103,000	44,500	36,000
Date of Closure	Jun 1937	Apr 1953	Aug 1958	Jul 1963	Jul 1952	Jul 1955

*Feet above mean sea level (ft msl). Elevations are referenced to the NGVD29 datum. Study output values are provided in 1988 NAVD. Refer to Table 5-1 for datum conversion values within the model reach.
**Maximum pool elevation with spillway gates opened.
***Maximum pool elevation with spillway gates closed.

3.3 Gavins Point Dam and Reservoir Information

Gavins Point Dam is located on the Missouri River at RM 811.1 on the Nebraska-South Dakota border, 4 miles west of Yankton, South Dakota. Lewis and Clark Lake extends 37 miles to the vicinity of Niobrara, Nebraska. Construction of the project was initiated in 1952, closure was made in July 1955, and initial power generation began in September 1956. The total drainage area of the Missouri River at Gavins Point Dam is 279,480 sq. miles.

3.3.1 Gavins Point Dam Flow History

The model uses Gavins Point Dam releases as the upstream boundary condition. Table 3-2 through Table 3-4 show the historical releases and release-duration and release-probability relationships for Gavins Point Dam (USACE, 2013a).

Table 3-2: Gavins Point Release Historical Records (1967 – 2011)

Month	Daily Release (cfs)		
	Maximum	Minimum	Mean
Jan	31,000	7,800	17,100
Feb	34,700	7,500	17,300
Mar	42,000	6,000	19,600
Apr	58,000	6,000	25,000
May	77,000	8,000	28,800
Jun	160,700	6,000	32,200
Jul	160,300	6,000	35,100
Aug	151,800	7,000	36,900
Sep	90,100	14,000	36,500
Oct	70,100	7,600	34,400
Nov	70,100	7,500	31,100
Dec	68,000	8,000	19,500
Annual	**160,700**	**6,000**	**27,800**

Table 3-3: Gavins Point Release-Duration Relationship

Percent of Time Equaled or Exceeded	Release (cfs)	
	Annual	May – Aug
Maximum	160,700	160,700
1	68,000	150,000
5	52,000	55,000
10	43,100	46,000
20	35,000	37,000
50	27,000	31,000
80	16,500	24,600
90	13,000	21,000
95	11,000	16,000
99	8,600	9,000
100	6,000	6,000

Table 3-4: Gavins Point Release-Probability Relationship

Annual Percent Chance Exceedance	Release (cfs)
50	38,000
20	47,000
10	57,000
2	84,000
1	100,000
0.2	160,000

3.3.2 Survey History

Degradation and aggradation surveys are an integral part of the Omaha District's sediment data collection program. The survey work requires the periodic resurvey of the land surface and riverbed cross sections between permanently established survey control points called sediment ranges. There are 43 sediment ranges spaced an average of 1.4 miles apart below Gavins Point Dam. Table 3-5 below provides a summary of the Gavins Point degradation reach.

Table 3-5: Sediment Range Information

Gavins Point Degradation Reach					
Gavins Point Dam River Mile (1960 RM)	Ending River Mile (1960 RM)	Reach Length (miles)	No. of Main Stem Sediment Ranges	Average Spacing of Ranges (miles)	Most Recent Survey Year
811.1	753.18	57.92	43	1.4	2013

3.4 REACH CHARACTERISTICS

The Missouri River in the model reach is highly impacted by reservoir operations and construction activities including the Main Stem dams, federal levee system, and the Bank Stabilization and Navigation Project (BSNP). The drainage area through this reach increases from 279,500 square miles at Gavins Point Dam to 414,900 square miles at Rulo, NE. The primary tributary in this reach is the Platte River with a drainage area of 85,370 square miles. The Platte River is also a major sediment contributor.

3.4.1 Recreational River and Kenslers Bend

The Gavins-to-Ponca reach (RM 811 to 752) of the Missouri River was designated a Recreational River pursuant to Section 707 of the National Parks and Recreation Act (PL 95-625) which amended the Wild and Scenic Rivers Act (PL 90-542). The river is channelized starting at the downstream end of the Recreational River, a segment known as "Kenslers Bend". Within the Recreational River, demonstration bank stabilization projects on the Missouri River were authorized under Section 32 of the Streambank Erosion Control Evaluation and Demonstration Act of 1974 (P.L. 93-251). Nine of these projects are located in the reach from Gavins Point Dam down to Ponca State Park.

The recreational river reach has been impacted by Gavins Point Dam including flow regulation and the capture of sediment. Within this reach, the riverbed has experienced significant degradation and the loss of high bank. Bank stabilization such as the Section 32 projects has greatly reduced the migration of the high banks. However, in many areas, the river is characterized by a dynamic channel with shifting islands and sand bars. The character of the river in this reach is very different than within the navigation channel. The typical flow area consists of a wider section with sandbars and islands more common.

3.4.2 Navigation Channel

The BSNP is designed as a self-scouring channel that uses the controlled erosive forces of flowing water to provide channel widths and depths, while providing stability to the river location and features. There were seven acts of Congress that provided for the construction, operation and maintenance of a navigation channel and bank stabilization works on the Missouri River. The most recent was authorized in 1945 and provided for bank stabilization combined with a 9-foot deep and not less than 300 feet wide navigation channel. The authorized project for the Missouri River extends from its confluence with the Mississippi River at St Louis, MO to Sioux City, IA for a total distance of 734.2 river miles. This was accomplished through revetment of banks, construction of permeable dikes, cutoff of oxbows, closing minor channels, removal of snags and dredging. In order to achieve the project objectives of bank stabilization and navigation, the river was shaped into a series of smoothly curved bends of the proper radii and channel width. Stabilization of the bank along the concave alignment of the design curve was accomplished with pile and stone fill revetments. Dikes were constructed along the convex bank, approximately perpendicular to the flow. These dikes were designed to prevent bank erosion and to promote accretion, forcing the channel to develop and maintain itself along the design alignment. In areas where the natural river channel did not conform to the design alignment, canals were excavated and natural channels blocked in order to force the river to flow along the design alignment.

BSNP structures are constantly attacked by river flows and therefore are continually degrading over time. Maintenance is conducted to ensure the structures provide river stability and channel dimensions necessary for commercial navigation and other authorized purposes.

Within the Omaha District portion of the navigation channel, the Missouri River has a top width generally between 600 and 700 feet. Dike spacing is also on the order of 600 to 700 feet. Typical dike length projecting from the bank is on the order of 50 to 100 feet. The typical BSNP components within the navigation channel are illustrated in Figure 3-3. Description of HEC-RAS modeling methods is provided in the geometry section of this report.

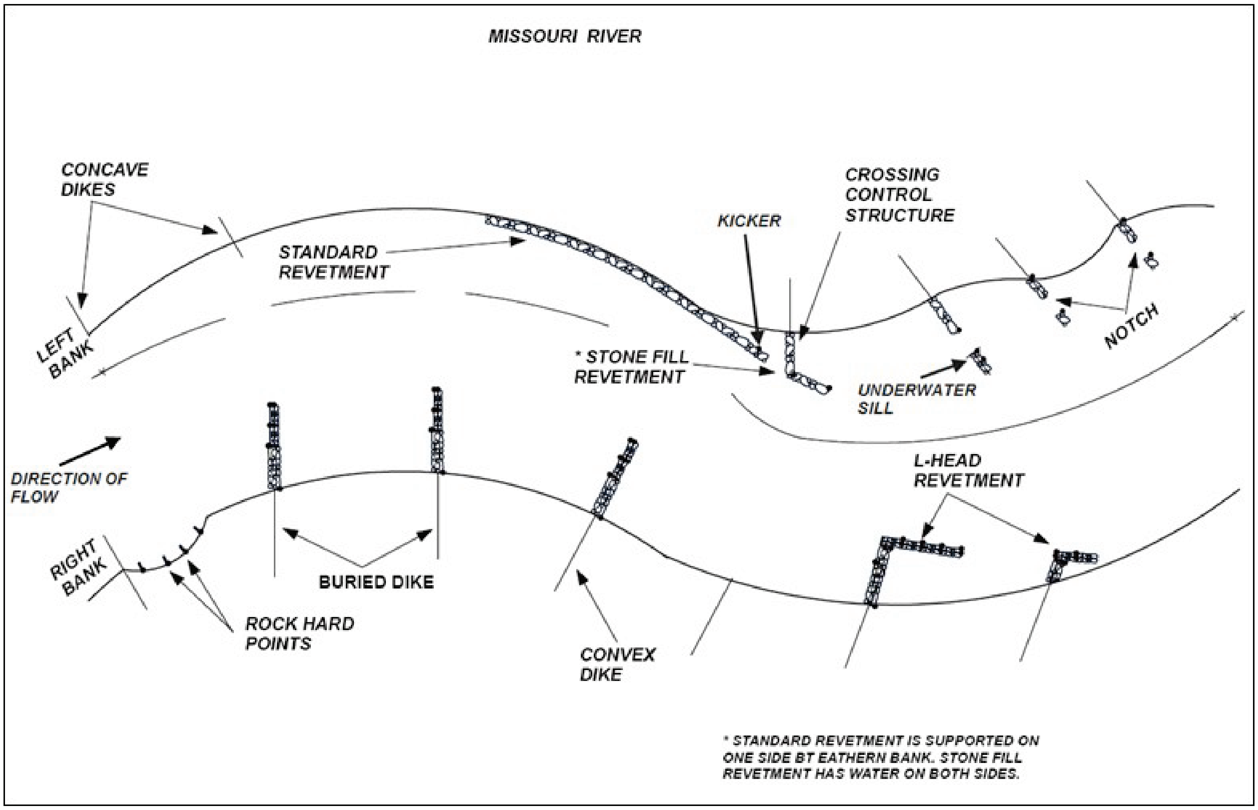


Figure 3-3. Typical Navigation Channel Plan View

3.4.3 Levees

The Missouri River levee system was authorized by the Flood Control Acts of 1941 and 1944 to provide protection to agricultural lands and communities from Sioux City, Iowa to the mouth at St. Louis, Missouri. No Federal levees have been constructed from Gavins Point Dam to the Omaha, Nebraska-Council Bluffs, Iowa, area due to the significant protection afforded this reach by the Missouri River mainstem reservoirs and due to gradual channel degradation through much of this reach. This reach does have non-Federal levees providing varying degrees of protection of largely unknown quality.

The Federal levee system begins in the Omaha-Council Bluffs metropolitan area, protecting a large urban area. Downstream of Omaha to Rulo, Nebraska, the Federal levee system protects agricultural lands and several small towns. All of these levee units were designed to operate in conjunction in with the six mainstem reservoirs to reduce flood damages. Most Federal levees were constructed in the 1950s and are generally set back from the riverbank 500 to 1500 feet. Federal levees provide left bank protection from river mile 515.2 to 619.7. Right bank levees are intermittent, as the river is often near the bluff. There are a total of 191 federally constructed levee miles from Omaha, Nebraska to Rulo, Nebraska, of which 133.5 miles are along the Missouri River and 57.5 miles are levee tiebacks.

Following construction of the Federal levee system, farming of the lands riverward of the Federal levees became more extensive. Farmers constructed secondary levees at or near the riverbank to prevent crop damages caused by flows above channel capacity, approximately a 2-year event in this reach, on the Missouri River. Private levees have also been built in those areas where Federal levees were not built. For example, the area near Rulo, NE, along the left bank reach from river mile 515.5 to 498.1 is protected solely by private levees.

3.4.4 Tributary Reaches

Numerous tributaries enter the Missouri River within the model reach. Refer to the model schematic shown in Figure 3-2 for the location of significant tributaries. Major tributaries were included as separate routing reaches within the model. Minor tributaries that have USGS gage data were included as lateral inflow to the model. Routing of the tributary flows from the gaging station location to the confluence with the Missouri River was found to increase the simulation accuracy. Tributary modeling efforts were of limited detail and intended for flow routing only. Data sources for many tributaries is aged and of questionable quality. Several tributaries relied only on channel distance and a few representative sections that were interpolated to provide reasonable cross section spacing. All structures were removed from tributaries to reduce stability problems. No storage areas were included with tributaries and those with levees (Boyer, Soldier, etc) are very simplified for stability reasons and only include ineffective flow areas. As a result of the coarse cross section data and no inclusion of levee systems, computed stage information on the tributaries is not accurate. Additional information for a few of the major tributaries is summarized in the below sections.

3.4.4.1 James River - RM 797.7

The James River is a major left bank tributary that enters the Missouri River downstream of Yankton, SD at river mile 797.7. The basin has a drainage area of approximately 20,942 square miles and includes portions of South Dakota and North Dakota. Federal projects on the James River include Pipestem and Jamestown Dams located near Jamestown, ND. The James River has a large drainage basin but an extremely flat channel gradient. The USGS gaging station #06478500 at Scotland, SD, is the upstream model inflow boundary and is located over 50 river miles from the Missouri River.

3.4.4.2 Big Sioux River - RM 734.0

The Big Sioux River is a left bank tributary that enters the Missouri River near Sioux City, IA at river mile 734.2. The basin has a drainage area of approximately 8424 square miles and includes portions of South Dakota, Minnesota and Iowa. The USGS gaging station #06485500 at Akron, IA, is the upstream model boundary and is located about 45 river miles measured along the channel from the Missouri River. The Big Sioux River floodplain length is slightly more than 30 miles measured from the Missouri River.

3.4.4.3 Platte River - RM 594.8

The Platte River is a major right bank tributary to the Missouri River draining an area of approximately 85,370 square miles of northeast Colorado, southeast Wyoming and most of central Nebraska. The Platte River joins the Missouri River approximately 21 miles downstream from

Omaha, NE at river mile 594.80. In eastern Nebraska, major tributaries to the Platte River are Salt Creek, the Elkhorn and Loup Rivers. The USGS gaging station #06805500 at Louisville, NE, is the upstream model inflow boundary and is located 16.5 river miles from the Missouri River.

3.4.4.4 Nishnabotna River - RM 542.1

The Nishnabotna River is a major left bank tributary to the Missouri River located approximately 20 miles downstream of Nebraska City, NE at river mile 542.1. It has a total drainage area of 2,806 square miles. Major changes within the basin include the construction of federal levees, private agricultural levees, channel changes and drainage improvements. The Nishnabotna River has federal levees along the right bank from the Missouri River confluence to Highway 275 located upstream from Hamburg, IA. The left bank also has federal levees from the Missouri River confluence upstream to Highway 275. The USGS gaging station #06810000 near Hamburg, IA, is the upstream model inflow boundary and is located 13.8 river miles from the Missouri River.

3.4.5 Ice Impacts on Peak Stage

The flood history within the Missouri River basin provides documentation of numerous events with ice jams and ice covered river conditions causing much higher stages than would normally occur for an open water condition. Ice affected events typically occur in the early spring, usually in the March to April time frame, with ice cover, ice breakup, potential ice jams, snowmelt runoff and precipitation events all contributing to spring event flows and stages. Ice jam occurrence in the study reach is rare, especially within the navigation channel.

No ice impacts or inclusion of typical ice parameters such ice jams or an ice cover were included in the existing condition modeling effort.

3.5 DEGRADATION AND AGGRADATION TRENDS

During the development of the Missouri River basin projects, significant change has occurred in channel conveyance as a result of aggradation and degradation. Missouri River natural variability and construction including flood control projects, channel cutoffs, channel and bank stability projects have all contributed to conveyance change.

3.5.1 Degradation Trends – Gavins Point Dam to Platte River

Degradation in the reach downstream of Gavins Point Dam has been evaluated in a series of studies (USACE, 2014a, 2014b). The degradation reach is generally considered as the reach from Gavins Point Dam downstream to the Platte River, a distance of over 200 river miles. Channel degradation has continued in the study reach, although the rate of degradation has generally decreased over time until 2010. At Gavins Point Dam tailwater, degradation of about 12 to 14 feet has been observed since dam closure at normal flows. The historic 2011 flood and period of sustained high flows led to degradation throughout the reach. Near the downstream end of the degradation reach at Omaha, NE, a decrease of over 2 feet was observed from pre-2011 to the post 2011flood water surface elevation at normal flow levels. Following previous extreme events, some flow elevation rebound has been observed. To date, only minor rebound has been observed.

The main findings and conclusions from the recent degradation studies (USACE, 2014a, 2014b) are summarized below:

Channel degradation has continued in the study reach from Gavins Point to the Platte River confluence. The rate of degradation has generally decreased over time.

The extreme 2011 flood produced 57.3 million acre-feet of runoff at the Sioux City gage, about 60 percent higher than the 1952 flood. The long period of sustained high flows led to degradation throughout the reach.

The most recent bed material data are from 2008. Although the number of samples is limited, there are some areas where the D90 has increased significantly, indicating the river was becoming more stable prior to the 2011 flood.

Water surface elevation trends for a specific discharge of 30,000 cfs showed that the rate of change (feet/year) of the water surface generally decreased over time until 2010. This was followed by a major decrease due to the 2011flood.

High discharges have historically increased degradation, followed by a rebound period and subsequent return to an overall trend. Only minor rebound has been observed following the 2011 extreme event at this time.

3.5.2 Aggradation Trends – Platte River to Rulo, NE

The reach between Omaha, NE (RM 616) and Rulo, NE (RM 498) has illustrated some aggradational trends at higher flow levels as illustrated by stage trend plots (USACE, 2012). Average bed slope at normal flow stages have remained relatively constant at 0.8 to 1.1 foot per mile. Floodplain flow trends have increased by about 1 to 3 feet since the 1980's. Within the reach, aggradation at and downstream from the Platte River confluence indicates that the Platte River continues to deliver significant sediment quantities. Based on gage station data, the sustained high flow of 2011 may have slightly altered the flood stage aggradational trend at the flood peak. However, the large number of levee breaches and substantial floodplain deposition on the receding limb of the hydrograph may have resulted in a net stage increase for the current floodplain flow levels in the post-flood condition. A detailed study of floodplain aggradation and post 2011 flood flow levels has not been performed.

While detailed evaluation has not been conducted, general observations have been made regarding floodplain stage trends in this reach. Sediment deposition within the floodplain near the channel is a common occurrence. In many river systems, natural levees are formed when deposition occurs outside of a channel during high flows (mainly during flood recessions) because vegetation traps sediment and increases hydraulic roughness, reducing velocities and sediment transport capacities. Another general characteristic of this phenomenon is the deposition of the larger size sediment particles immediately adjacent to the channel with a lateral reduction in grainsize down to finer materials such as silts and clays within low velocity settling areas away from the channel.

Floodplain features which affect flow patterns may be a factor which can contribute to higher floodplain deposition rates. Flow expansion and contraction zones within the floodplain may affect flow velocity and sediment transport. While site specific evaluations have not been conducted, a levee project may exacerbate the sediment deposition experienced during receding flows within the levee confined floodplain in some situations. For instance, although the federal levees are generally set back from the river bank, many areas include private levee systems between the federal levee and the river bank. These levee systems may act as sediment settling basins when the private levee elevation is exceeded, Missouri River flows flood the area between the river and the federal levee, and a quasi settling area with lower velocity is created during flood receding flows.

3.6 Flood History

The largest flood in recent history occurred in 2011. During the post dam construction period, previous extreme events included 1997, 1993, and 1984. A summary of peak flows at Sioux City and Nebraska City are included in Figure 3-4.

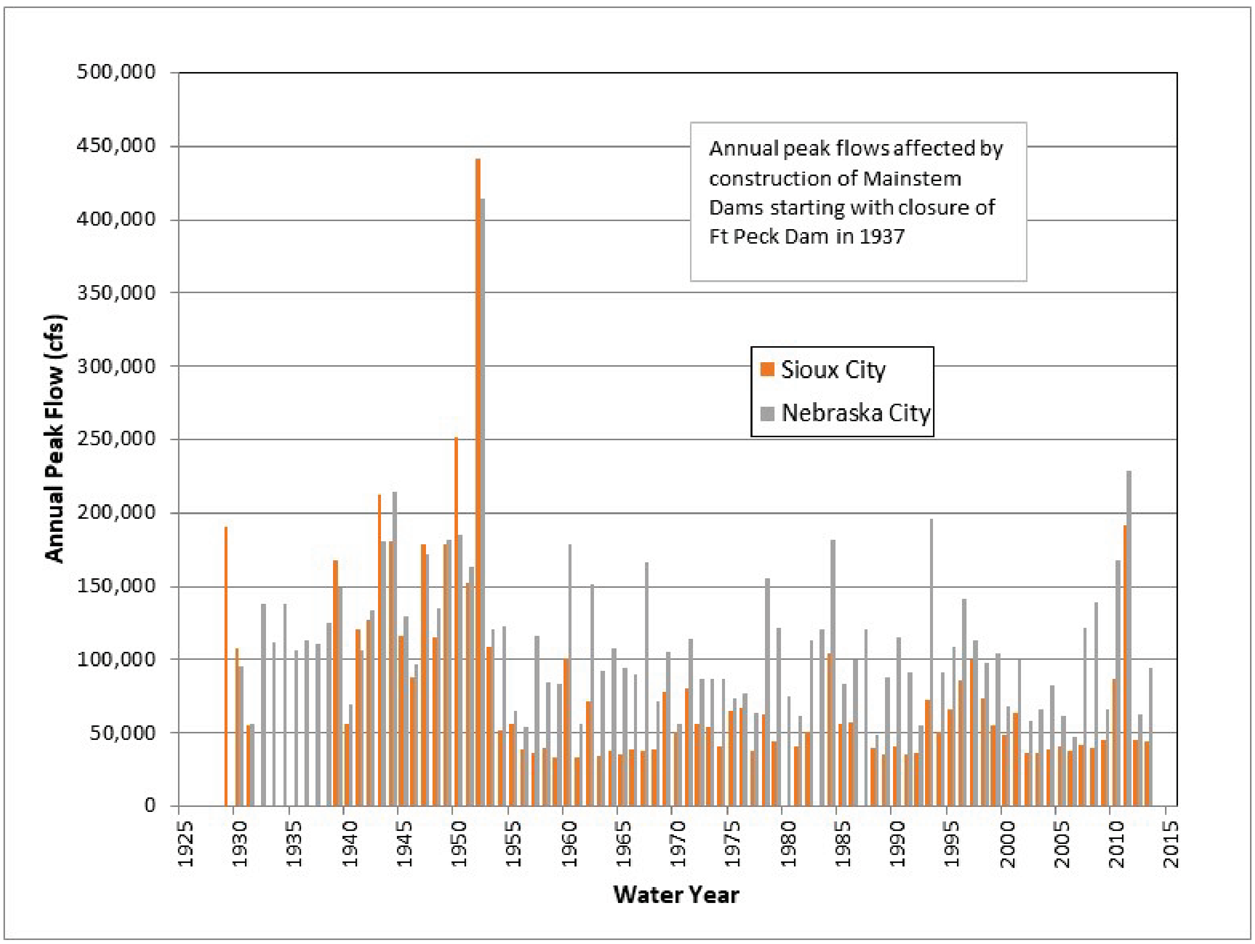


Figure 3-4. Missouri River at Sioux City and Nebraska City, Annual Peak Flow

3.6.1 Reservoir System

As previously discussed, the Missouri River Mainstem System includes six dams that provide a tremendous amount of storage volume. Historic event magnitude should be evaluated recognizing that conditions have changed significantly since dam closure with the primary construction period in the 1950's. System effect on flow decreases with distance downstream of Gavins Point Dam (RM 811) as tributary inflow increases. Refer to previous studies for a detailed description of system impacts on historic flows (USACE, 2003, 2006).

3.6.2 2011 Event

The duration and magnitude of the 2011 Missouri River flood event volume exceeds all other events in the recorded gage history of the river. The flow duration and energy acting on the river floodplain and projects within this environment were unprecedented. Within the floodplain corridor, the extreme high flood flows tended to travel across bends in the most energy efficient manner. Constructed projects and floodplain features in the path of this extreme flow zone were severely impacted as summarized below:

- Floodplain material dynamics occurred from the river extreme flows traveling linearly down the valley floodplain over the top of the meandering river
- Excess flood flow across the bends degraded dikes and revetments at most entry and exit points
- Sediment traveled with the flood flows, with extreme deposition depths observed throughout the floodplain
- Floodplain features and river dynamics that concentrated flows caused excessive scour at many locations
- Depending upon location and river dynamics, constructed chutes and backwaters in the floodplain experienced both scour and deposition

Historic events were reviewed to provide context on the magnitude of the 2011 event river flows. Bankfull flow, which is often described as the flow that correlates to the river stage at top of bank prior to significant floodplain flow, is often correlated to a 1.5 to 5-year event frequency. Flow days above bankfull indicate the potential for excess flow energy in the floodplain. Another river flow level of interest is near the top of levee that would indicate levels with the highest confined stage with associated increased flow depths and velocities within the floodplain and increased water depths for seepage and levee risk. Within the federal levee reach from Omaha to Rulo, the 4% Annual Chance Exceedance (ACE, 25-year) event correlates to an approximate flow level near the levee top (USACE, 2003).

In a simple method to compare the 2011 event flow energy to historic events, the number of days when the Missouri River flowed within the floodplain were tabulated for various events and compared to historic records. Sustained periods of high levels of floodplain flow indicate flow energy acting within the floodplain.

When comparing to historic events, it should be recognized that historic river flows were affected by main stem dam construction and reservoir filling, primarily in the period 1953 to 1967. All flow frequency values reported in the comparison are post dam construction (USACE, 2003). As a result, comparing to historic events is somewhat misleading since the reservoir system has significantly altered peak flows. Data from the USGS gage at Nebraska City is shown in Figure 3-5.

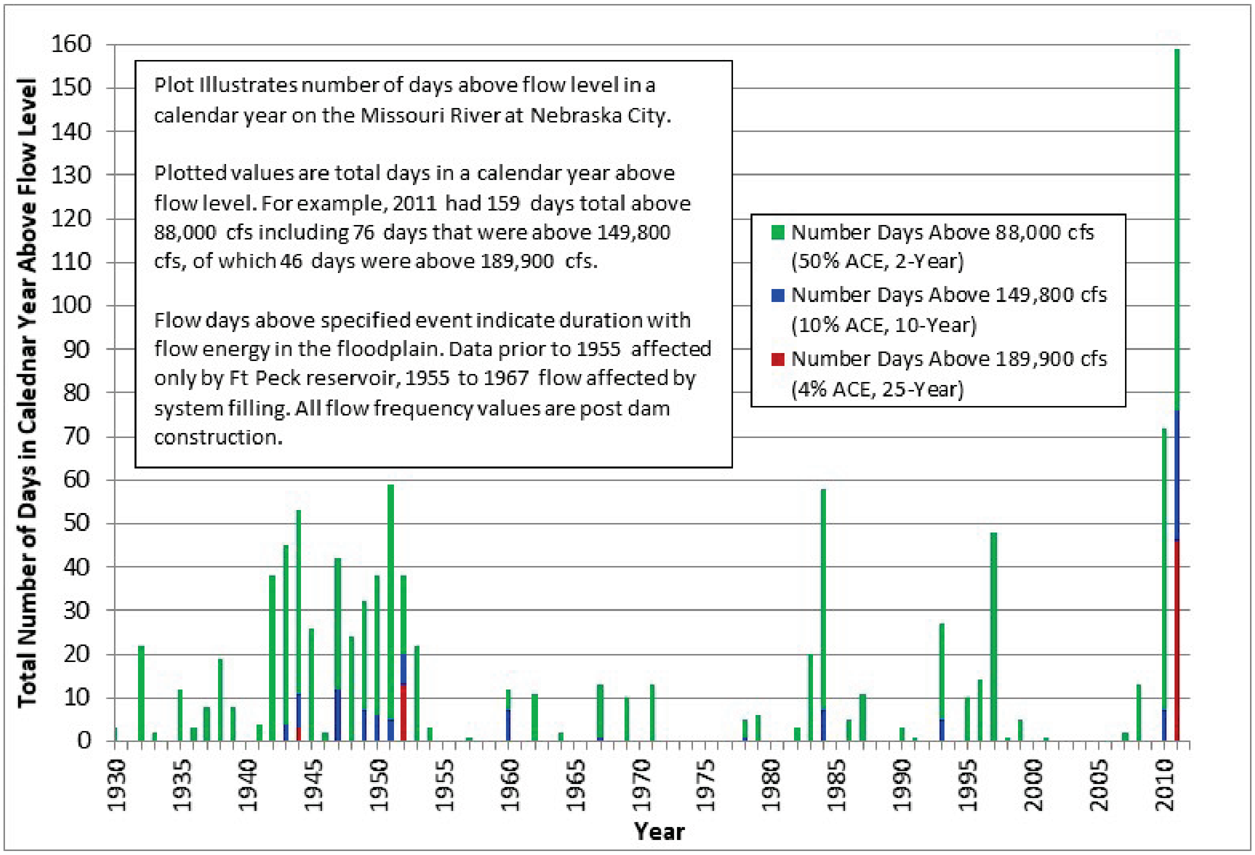


Figure 3-5. Nebraska City Days above Flow Value by Year

The above figure illustrates the severity of the 2011 event. Using the 189,900 cfs benchmark for comparison, the 2011 event dwarfs all other events which occurred on the historic Missouri River. This event was unique in the historic record with excess floodplain energy at a very high level for a prolonged duration.

4 DATA SOURCES

Primary data sources for construction of the unsteady HEC-RAS model includes terrain data, bathymetry data, and gage data. Terrain data encompasses everything from the bluffs to the riverbanks, defining the floodplain and overbanks, but does not often include data below the surface of the river. Bathymetry captures below the water surface. Gage data provides the boundary conditions for the model, and calibration benchmarks. A summary of the data used in the model is provided in Table 4-1.

Table 4-1: Summary of Data Sources

Data Type	Data Title	Location Data Applied to Model	Collection Dates
Topographic Data			
Bathymetry	Missouri River – Gavins Point to Ponca Hydrographic Survey	RM 810.87 – 749.20	Mar – May 2013
Bathymetry	Missouri River – Ponca to Sioux City Hydrographic Survey	RM 749.00 – 734.98	Nov 2011
Bathymetry	Missouri River – Sioux City to Rulo Hydrographic Survey	RM 734.20 – 497.91	Jul – Sep 2012
DEM – LiDAR	Gavins to Rulo LiDAR Mapping	RM 810.87 – 497.91	1 Dec 2011 – 21 Mar 2012
DEM – LiDAR	NRCS LiDAR – Nebraska DNR	Where needed on RB overbank (Nebraska)	2010 – 2011
DEM – 3 m	Iowa DNR Countywide LiDAR	Where needed on LB overbank (Iowa)	2008 - 2010
DEM – 4 m	NEXTMap DEMs	As needed in overbanks where USACE, NE, or IA DEMs didn't cover	Mar – Sep 2008
Land Cover			
Land Cover	National Land Cover Dataset 2006	All cross sections	2006
Flow Data			
Streamgage Data	Stage and Discharge	All cross sections	POR
Water Surface Profile			
Water Surface Elevation Data	Missouri River Water Surface Profile from Gavins Point to Rulo	All cross sections	24-26 Jun 2011 27-28 Jun 2012 13-14 Sep 2012

4.1 Terrain Development

A variety of terrain sources were available for this stretch of the Missouri River and floodplain. Past modeling efforts and much of the survey data collection used the 1929 NGVD vertical datum. This study used the 1988 vertical datum. Efforts continue to be performed using the 1929 vertical datum such as the USGS gage data collection and much of the historic data was converted for this study. Described below are the sources, dates, and accuracy of data used in the study.

4.1.1 DEMs and LiDAR

Several DEM data sets were available for this reach of the Missouri River. The first was a 5-ft cell size GRID, LiDAR, Digital Elevation Model (DEM) collected from 1 Dec 2011 to 21 Mar 2012 extending from Gavins Point Dam to Rulo, NE. The horizontal and vertical accuracies are 1.25 ft RMSEr and 0.14 ft RMSEz, respectively. State LiDAR was available for Nebraska and Iowa from the Nebraska and Iowa DNR websites. The collection dates and accuracy vary between individual datasets. NEXTMap 4-meter DEMs are available for the entire Omaha District. They were collected from March through September 2008 by Intermap Technologies. In some areas, the NEXTMap DEMs were the best available topographic data for the overbanks. The horizontal accuracy is 2 meters RMSE or better in areas of unobstructed flat ground. The vertical accuracy is 1 meter RMSE or better in areas of unobstructed flat ground.

The USACE post-flood LiDAR data did not completely cover the extents of the cross sections, so the state (NE and IA) DEMs were used for the overbanks. In areas outside of Nebraska and Iowa, the NEXTMap data was used in the overbanks. See Table 4-1 for general collection date ranges.

4.1.2 Land Cover

The United States Geographical Survey (USGS) National Land Cover Database 2006 (NLCD 2006) was used in the determination of appropriate Manning's n roughness values for overbank data. The NLCD 2006 is a 16-class land cover classification scheme at a spatial resolution of 30 meters and is based primarily on a 2006 Landsat satellite data. This is a raster digital data set (USGS, 2012).

4.1.3 Top of Levee Elevation Data.

The best available topographic information was desired for use with model levee crown elevation data. LiDAR survey data was supplemented with elevations from the National Levee Database (NLD). Within the NLD program, top of levee elevations were surveyed in NAVD88 datum and collected at an interval of 100-ft, or where noticeable horizontal or vertical alignment changes occurred. Several levee setbacks and repair have occurred following the 2011 flood. The levee setback alignment and new elevation were not available when the LiDAR survey information was collected and are not reflected in the DEM coverage. In addition to altering the levee elevation, setback areas would require modifying the floodplain to reflect the new grading during construction and levee features within the cross section. Therefore, the LiDAR survey information was used to extract the model cross sections. Flood breaches or damage to the levee system from the 2011 flood was not modeled within HEC-RAS. All levee top elevations reflect an intact condition. Model elevations used adjacent levee elevations since post construction repair information was not available. Note that only federal levees are included in the model as levee cells. All other private levees areas are modeled with elevation from the background topography.

4.2 BATHYMETRY

Hydrographic (bathymetric) surveys for the main stem Missouri River from Gavins Point Dam to Rulo, NE were collected in three surveys: high density data (250 ft spacing) from Gavins Point to Ponca, NE in 2013, whole river mile spaced data from Ponca, NE to Sioux City, IA in 2011, and high density data (250 ft spacing) from Sioux City, IA to Rulo, NE in 2012. See Table 4-1 for information on where each survey's data was used in the model.

4.3 OBSERVED DATA

Water surface profiles are surveys that are periodically collected, as resources allow, and are performed by the Omaha District that provide a water surface elevation for a reach. Elevations are usually collected approximately every river mile. Stream stage and flow data available on the Missouri River includes gages along the Missouri River main stem, and gages on many of the major tributaries. All gages are operated by the USGS and collect stage data remotely, usually at intervals of 15 minutes. Availability and quality of these datasets influenced the configuration of the model as well as the timeframe for calibration.

4.3.1 Water Surface Profile Data

Water surface profile elevation data was collected on 24-26 June 2011, 27-28 June 2012, 13-14 September 2012, and 7-8 July 2013. Water surface elevations are collected approximately every river mile. This data was used as the baseline for calibration of the model. The flow variability during the June 2011 collection period was significant. The data were collected on the following days starting below Gavins Point dam and proceeding downstream:

24 June 2011 Gavins Point Dam to below Sioux City, SD (Mile 810.2 to mile 722.0)

25 June 2011 Below Sioux City, SD to Blair, NE (Mile 721.9 to mile 646.6)

25 June 2011 Blair, NE to Council Bluff, IA (Mile 646.6 to Mile 607.5)

26 June 2011 Council Bluff, IA to Rulo, NE (Mile 607.4 to Mile 498)

4.3.2 USGS Gage Flow and Stage Data

Stream gage data was obtained through the USGS National Water Information System (NIWS) for all applicable gages in this reach of the Missouri River and tributaries (USGS, 2012). Table 4-2 lists the main stem USGS gages and Table 4-3 lists the tributary USGS gages. Figure 3-2 provides a map of the gage locations.

Table 4-2: USGS Missouri River Main Stem Gages

Gage Name	River Mile	Gage Number	Gage Datum (ft NAVD 88)	Flow Data Dates	Stage Data Dates
Yankton, SD	805.76	06467500	1140.35	10/1/1990–9/30/1995	5/22/1985 - *
Gayville, SD[1]	796.15	06478515	1100.36	n/a	10/1/1980–5/6/2012
St. James, NE[1]	784.92	06478523	1100.0	n/a	6/1/2012 - *
Maskell, NE	775.26	06478526	1100.65	n/a	10/15/1987 - *
Ponca, NE	751.17	06479097	1080.57	na	3/15/1987
Sioux City, IA	732.37	06486000	1057.53	10/1/1928-9/30/1931,10/1/1938 *	10/1/1988 - *
Decatur, NE	691.07	06601200	1010.47	10/1/1987 - *	10/30/1988 - *
Blair, NE	648.25	06609100	977.44	n/a	6/1/1977 - *
Omaha, NE	615.98	06610000	948.97	9/1/1928 - *	10/1/1987 - *
Plattsmouth, NE	591.50	06805600	929.48	n/a	10/15/1977 - *
Nebraska City, NE	562.60	06807000	905.56	8/11/1929 - *	10/1/1988 - *
Brownville, NE	535.25	06810070	859.95	n/a	10/15/1977 - *
Rulo, NE	498.04	06813500	837.44	10/1/1949 - *	10/1/1988 - *

[1] The Gayville gage was discontinued and replaced with the St. James gage due to becoming isolated from the main channel after the 2011 flood.
* - indicates a current gage

Table 4-3: USGS Tributary Gages

Gage Name	Gage Number	Gage River Mile	Confluence River Mile	Modeled or Lateral Inflow	Available Flow Data Dates
James River near Scotland, SD	06478500	55.0	800.5	Modeled	9/1/1928 - *
Vermillion River at Vermillion, SD	06479010	10.8	771.7	Modeled	10/1/1983 - *
Big Sioux River at Akron, IA	06485500	50.9	734.0	Modeled	10/1/1928 - *
Floyd River at James, IA	06600500	8.9	731.2	Lateral Inflow	12/8/1934 - *
Omaha Creek at Homer, NE	06601000	4.7	719.8	Lateral Inflow	10/1/1945 - *
Monona Harrison Ditch near Turin, IA	06602400	12.5	670.1	Lateral Inflow	5/7/1942 - *
Little Sioux River near Turin, IA	06607500	13.4	669.2	Modeled	5/7/1942 - *
Soldier River at Pisgah, IA	06608500	12.2	664.0	Modeled	3/5/1940 - *
Boyer River near Logan, IA	06609500	15.7	635.2	Modeled	11/4/1937 - *
Platte River at Louisville, NE	06805500	16.5	594.8	Modeled	6/1/1953 - *
Weeping Water Creek at Union, NE	06806500	6.2	568.7	Modeled	3/1/1950 - *
Nishnabotna River above Hamburg, IA	06810000	11.0	542.1	Modeled	10/1/1928 - *
Little Nemaha River at Auburn, NE	06811500	10.4	527.8	Modeled	9/1/1949 - *
Tarkio River at Fairfax, MO	06813000	13.4	507.6	Modeled	3/8/1922 - 12/31/1990, 6/27/2007 - *
Big Nemaha River at Falls City, NE	06815000	13.7	494.8	Modeled	4/1/1944 - *
Nodaway River near Graham, MO	06817700	28.9	463.0	Modeled	10/22/1982 - *

* - indicates a current gage

4.3.3 Gavins Point Dam Release

The upstream boundary for the model consists of the Gavins Point Dam release. For the 2011 and 2012 event calibration periods, the actual Gavins Point Dam release from COE records were used.

5 MODEL DEVELOPMENT

Model development includes the software version used, descriptions of the various geometry components of the model, and boundary conditions selected. The following sections outline the details of the model construction including fundamental assumptions, data sources for specific geometry features, techniques used, and justification for any unique parameters and decisions made during the process of building the model.

5.1 HEC-RAS

Unsteady computations in HEC-RAS version 4.2 Beta were used for this modeling effort. A computation interval of 2 hours was used for the non-levee overtopping event of 2012 and a time step of 6 minutes was used for the levee overtopping event of 2011. These time steps were determined to be stable for the model and allowed model runs to be conducted in reasonable timeframes.

HEC-RAS has been significantly updated since version 4.1. The model should not be run in 4.1 or earlier versions.

HEC-RAS version 5.0 beta has been released but the model has not been tested in this version. The model version will be updated to the next HEC-RAS version when it is released, presumably version 5.0.

5.2 Geometry

This section will discuss the development of the HEC-RAS model geometry for the Missouri River reach Gavins Point Dam to Rulo, NE, including vertical datum and horizontal projection, the stream centerline and cross section geometry, the development of Manning's n-values, and the modeling of structures such as bridges and dams. The geometry of the tributary used in the model was developed outside of this project and was added after the completion of the Missouri River geometry.

5.2.1 Vertical Datum and Projection

The current vertical datum for the unsteady HEC-RAS model is NAVD88 to match the LiDAR data. Most of the other elevation data is referenced to the NGVD29 vertical datum so a conversion factor was used to convert that data to NAVD88. See Table 5-1 for a list of vertical conversion factors used in the model. The program CorpsCon was used to obtain the conversion values based on the gage's coordinates. Conversion discrepancies have been noted with CorpsCon in some areas although it is regarded as the best available conversion software.

The model horizontal projection is NAD 83 UTM 15 (US-Feet) which is consistent with the majority of the available terrain data. Re-projection to a nation-wide projection may be necessary after review and certification for compatibility with other HEC-RAS models and the ResSim models that are in UTM projections.

Table 5-1: Gage Vertical Datum Conversion Factors

Gage Number	Gage Name	Conversion Factor (from NGVD29 to NAVD88) (ft)*
06467500	Missouri River at Yankton, SD	0.670
06478515	Missouri River near Gayville, SD	0.650
06478523	Missouri River near St. James, NE	-
06478526	Missouri River near Maskell, NE	0.653
06479097	Missouri River near Ponca, NE	0.574
06486000	Missouri River at Sioux City, IA	0.554
06601200	Missouri at Decatur, NE	0.466
06609100	Missouri at Blair, NE	-
06610000	Missouri at Omaha, NE	-
06805600	Missouri at Plattsmouth, NE	-
06807000	Missouri River at Nebraska City, NE	0.348
06810070	Missouri River at Brownville, NE	-
06813500	Missouri River at Rulo, NE	0.213

*Some of the gages used an NAVD88 datum so no conversion is listed.

5.2.2 Stream Centerline

One stream centerline for the Missouri River was developed in GIS for all of the Omaha District HEC-RAS models. A centerline from a previous study was modified to match the current state of the river, making sure to follow the center of mass of flow and avoiding crossing sandbars. It should be noted that the centerline does not match the 1960 river miles line alignment. However, in order to have Geo-RAS accurately determine the correct reach length, an accurate river centerline was required.

5.2.3 Cross Section Geometry

The geometry of the cross sections were constructed using the most recent bathymetric surveys in conjunction with the LiDAR DEMs. It was determined to have cross sections spaced at an interval between 2000 and 3000 feet apart with additional sections inserted as needed for bridge crossings. Bank points for all the cross sections were set at approximately the 2-yr water surface elevation. Cross sections were named based on the 1960 river miles to be consistent with previous studies and river documentation. The cross section naming to the 1960 river miles allows for comparative location with USGS gages and other river features. Since current channel centerline alignment does not match the 1960 alignment used to set river mileage, the HEC-RAS model centerline reach length will not match with the 1960 river mile cross section name.

The model cross sections were extracted valley wide from the available data sources. At the beginning of the study during model construction, the model purpose, methods to model federal levees, and possible alternatives were not known and cross sections were extracted valley wide. Insertion of the federal levees with lateral structures/storage areas required modifying the cross section to avoid double counting of flow area. Therefore, blocked obstructions were used to remove all areas landward of the federal levee.

Within the near bank region, both natural and privately constructed levees are common. In order to accurately model frequent flow events, levee points near the bank were included to remove floodplain flow area that is not available until the levee point location is overtopped.

Parameters for calculating the cross section hydraulic tables (HTab) were set to increments of 0.8 feet with 100 points for all mainstem cross sections. On the tributaries, the increment was generally reduced to between 0.5 and 0.8 feet. The number of points were increased above 100 at sections that required adjustment to exceed the maximum water surface elevation.

5.2.4 Navigation Structures.

As discussed in Section 3, the BSNP includes dikes, sills, revetments, and similar rock structures within the Missouri River channel. Refer to Figure 3-3 for a typical illustration of navigation channel structures.

Within the HEC-RAS model, calibration at normal flow levels was successful by adjusting channel roughness without any special geometry additions for the navigation structures. This method is different than that employed in the Kansas City District HEC-RAS model for the reach below Rulo, NE. The difference in modeling method is due to multiple factors including the variation in BSNP construction techniques, smaller navigation structure footprint within Omaha District, that Omaha District survey bathymetry includes structure impact on river elevations, the type and size of structure used varies between locations, and structure spacing between the upper portion of the Missouri River and the lower river downstream of Rulo, NE.

Cross sections were reviewed and a few sections were modified to include blocked obstructions. These locations were primarily at 2011 post flood repair sites where the survey data did not reflect September 2012 channel conditions.

5.2.5 Manning's n-values

For the overbank areas, Manning's n values for roughness were set based on the land use classification from the NLCD 2006 data. The land cover values were condensed from the NLCD 2006 standards into 12 classes, as shown in Table 5-2. The land cover GIS shapefile was manually updated with the use of recent aerial images to reflect changes to the river channel, such as shifting sandbars, mostly due to the 2011 flood event. Manning's n-values in the river channel were initially set to 0.025. These values were revised during calibration.

Table 5-2: Land Use Reclassification and Initial Roughness Values

NLCD Number	NLCD Classification	Reclass Number	Reclassification for Model	Initial Manning's N-Value*
11	Open Water	11	Water	0.025
		12	Channel Sandbar	0.032
		13	Channel Sandbar Light Vegetation	0.038
		14	Channel Sandbar Heavy Vegetation	0.052
		15	Channel Bank	0.056
21	Developed, Open Space	2	Urban	0.080
22	Developed, Low Intensity			
23	Developed, Med Intensity			
24	Developed, High Intensity			
31	Barren Land (Rock/Sand/Clay)	3	Sand	0.028
41	Deciduous Forest	4	Trees	0.070
42	Evergreen Forest			
43	Mixed Forest			
51	Dwarf Scrub	5	Scrub Brush	0.060
52	Shrub/Scrub			
71	Grassland/Herbaceous	6	Grass	0.035
72	Sedge/Herbaceous			
73	Lichens			
74	Moss			
81	Pasture/Hay			
82	Cultivated Crops	7	Crops	0.045
90	Woody Wetlands	8	Wetlands	0.055
95	Emergent Wetlands			

* Initial values only, revised during calibration

During calibration, the initial roughness values were modified to a range between 0.027 to 0.031, which were determined to be reasonable channel roughness values for the Missouri River. Manning's n-values for overbank areas were also modified in some areas during calibration. Typical floodplain roughness value change was in the range of 0.01 to 0.015 from the base condition. Roughness values were generally changed in a reach wide manner of 20 to 30 mile long blocks. Due to the use of horizontal roughness, tabulation of final roughness values was not feasible for the floodplain. Final roughness values for the main channel are tabulated below in Table 5-3. Roughness values in the non-navigation portion of the channel, from Gavins Point Dam downstream to Sioux City, also include variable roughness to reflect islands, sandbars, and similar vegetation that are not reflected in the below table.

Table 5-3: Final Channel Roughness Values

Cross Section River Mile Range	Channel Mannings N Value
810.87 – 800.6	0.027
800.58 – 781.66	.029
781.23 - 770.13	0.028
769.84 – 731.78	0.029
731.35 – 708.32	0.03
707.94 - 680.34	0.031
679.96 - 670.25	0.03
669.82 - 659.34	0.029
658.91 - 638.72	0.031
638.3 – 620.08	0.029
619.65 – 608.3	.03
607.77 – 601.35	0.028
601.3 – 595.02	.027
595.0 - 570.04	0.029
569.62 – 564.77	0.03
564.34 – 553.89	.031
553.5 – 553.02	.03
552.64 – 497.91	0.029
Below 497.91 (Overlap Reach)	KC Values

5.2.6 Federal Levee Area Modeling with Lateral Connections and Storage Areas

Federal levee protected areas within the floodplain were modeled within HEC-RAS using lateral connections and storage areas. Not intended in any way to imply that these areas were designed to store water, the term "storage area" refers to HEC-RAS features used to model flows affected by these features. Storage areas are described within the RAS model with lateral connections used to transfer flow from the main river channel into the storage area. Storage area connections are used to transfer flow between storage areas. In this way, flow conveyance is included within HEC-RAS to include the entire floodplain. The following sections provide further detail on the HEC-RAS model components used to describe the complex federal levee system with storage areas, lateral connections, and storage area connections. A schematic of the HEC-RAS modeled levee system is shown in Figure 5-1.

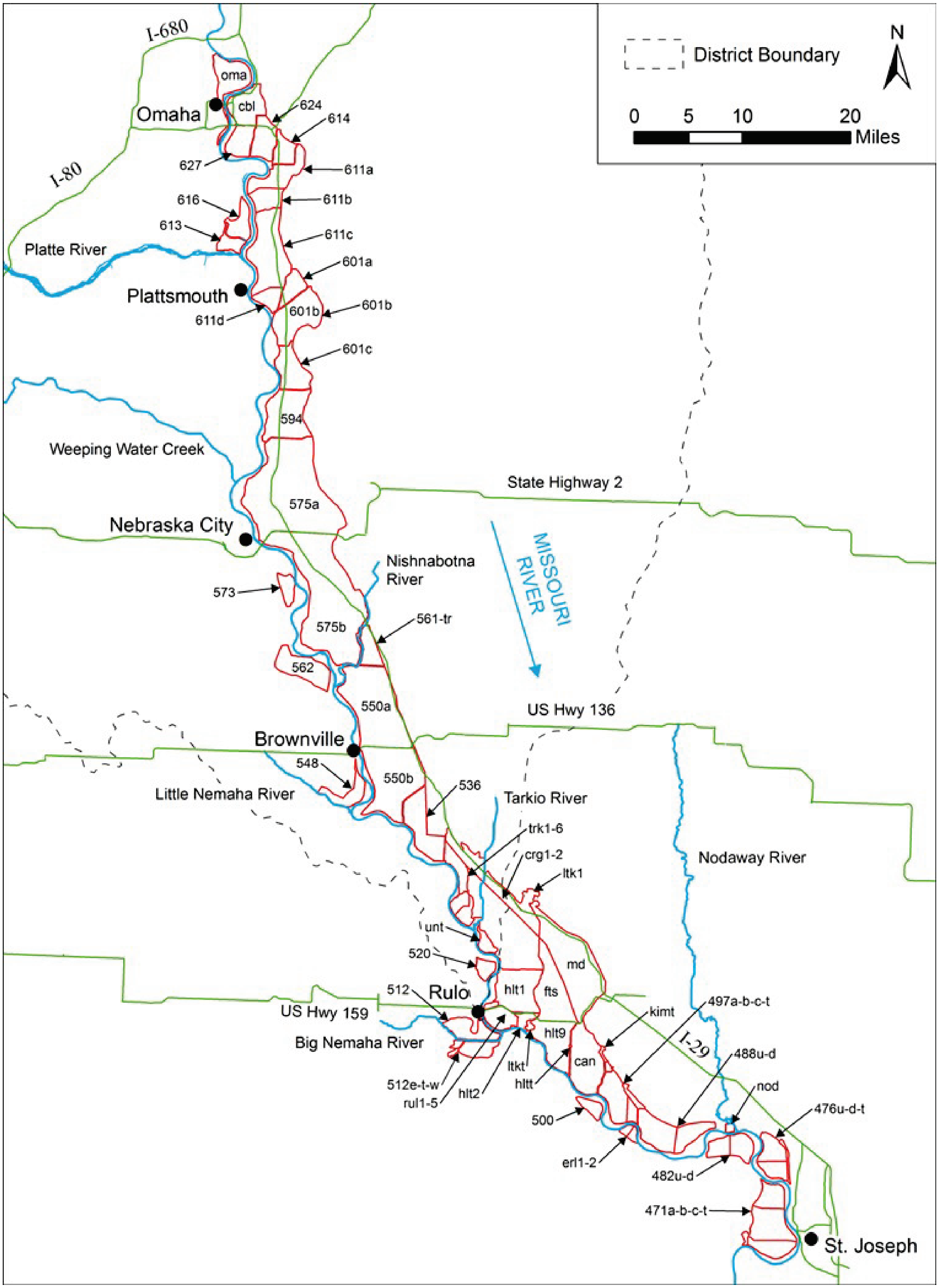


Figure 5-1. Federal Levee Modeling Area.

5.2.6.1 Levee Protected Areas.

All federal levee protected areas along the Missouri River were modeled in HEC-RAS using the storage cells as described within HEC-RAS. A storage area is visually represented in HEC-RAS with a polygon and numerically by an elevation-volume curve.

As previously stated, no storage areas were included with tributaries and those with levees (Boyer, Soldier, etc) are very simplified and modeled only with ineffective flow for stability reasons. Exceptions to this are the Nishnabotna, Big Nemaha, and Tarkio Rivers. These are all tributaries that enter the Missouri River within the federal levee reach that include very large tiebacks that are incorporated in the federal levee system.

A limitation of using storage areas to represent leveed area is that RAS assumes the entire storage area has the same water surface elevation. For larger levee protection units that span many miles along the river, this assumption is not accurate. This assumption affects both the water surface elevation within the levee protected area and also the travel time through the cell. For those reasons, the model input was created using intermediate breaks that were selected based on topography. Breaking the large levee protected areas into smaller areas within the model also helps to reduce the impact of the level water surface on travel time through the levee cell. No calibration data was available for these areas.

Elevation-volume curves for the storage areas were calculated in Geo-RAS (version 10.0), which incorporates the storage area slicer created by Don Duncan of MVS. Parameters were left at the default values of 20 slices and slice density of 0.2. The best available LiDAR surface was used in calculating the elevation-volume for all storage areas.

Naming convention for the storage areas is a shortened version of the levee unit name. Names were limited to 2-4 characters, and where multiple storage areas had the same name, either numbers or letters were added to identify unique areas.

When applicable, all storage areas were drawn back to high ground at the bluffs for more complete mapping. A typical volume elevation curve for a levee protected area is shown in Figure 5-2.

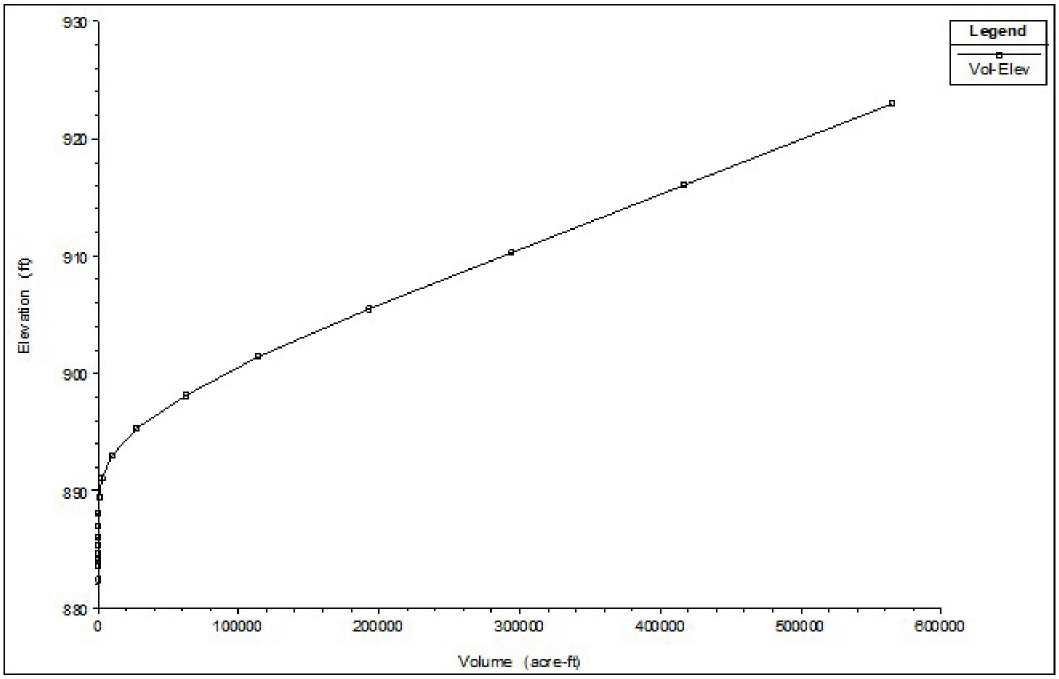


Figure 5-2. Typical Levee Area Elevation - Volume Curve

5.2.6.2 Lateral Structures

Lateral structures were defined within the HEC-RAS model to transfer levee overtopping flows from the adjacent river into the defined levee protected storage area. Lateral structure elevation is derived from the top of levee elevation as previously described. Flow rate over the lateral structure is computed by the model using the weir equation.

To locate the lateral structure within the model with respect to the river cross sections, user specified intersections were used to specify lateral structure stationing with respect to the cross section. This is necessary because the levee alignment and length is different than the river length. In this manner, model computed water surface elevations from each cross section are compared to the lateral structure / top of levee elevation. User specified intersections were calculated automatically in GIS using a linear routing tool and manually copy pasted into HEC-RAS.

Some lateral structures in the model that had excessive point density were filtered to maintain accuracy while reducing model run time. Filtering was accomplished with the tool in HEC-RAS, which minimizes change in weir flow area.

Naming convention for lateral structures matches the storage area to which they are attached. Each lateral structure can only link to one storage area, but multiple lateral structures can be attached to the same storage area. When this occurred, the lateral structure name was followed by a dash with a unique letter. A typical lateral structure profile and water surface elevation is shown in Figure 5-3.

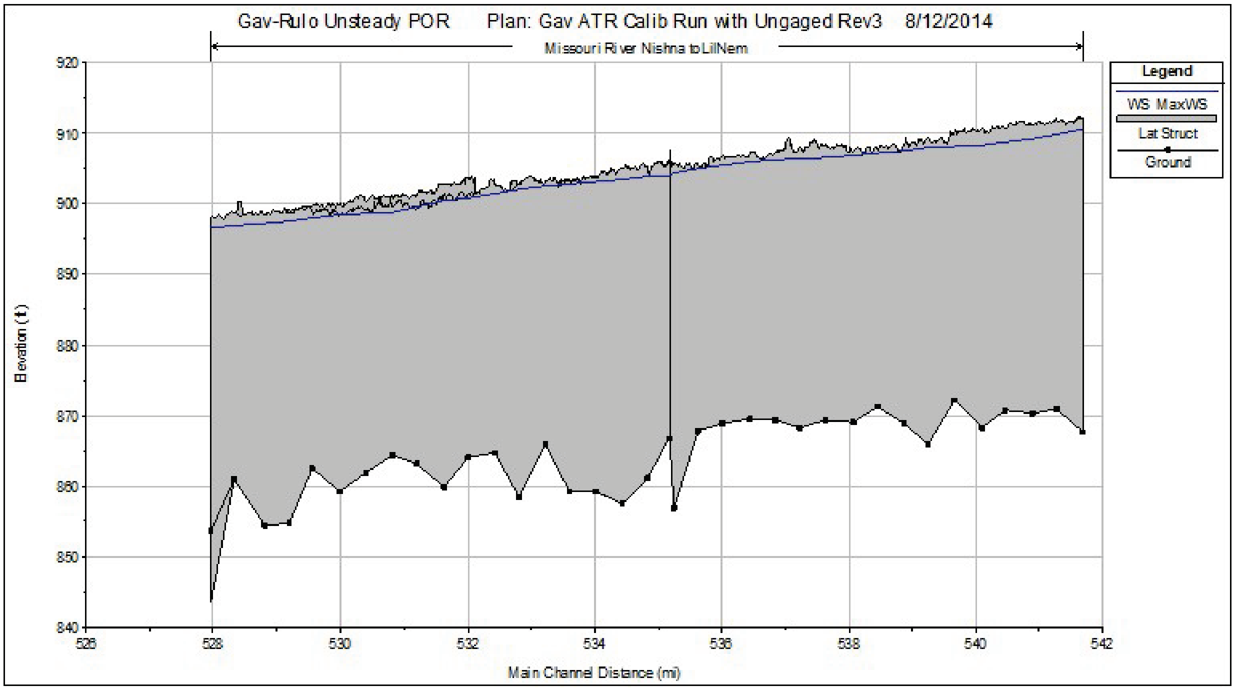


Figure 5-3. Typical Lateral Structure

5.2.6.3 Storage Area Connections

Storage area connections are used within HEC-RAS to connect two storage areas with a hydraulic structure and enable the transfer of flow. A storage area connection is described with station elevation points similar to a lateral structure and flow is calculated with the weir equation. Storage area connections were derived from features that separate flood plain conveyance such as high ground, road and railroad embankments, or levees.

Large protected areas behind a levee were split into multiple storage areas if necessary with storage area connections between adjacent areas. This was done to prevent the model from allowing flood waters overtopping at the upstream end to immediately fill the leveed area at the downstream end, ultimately short circuiting the reach of river in a few timesteps. Generally, if the protected area had a length along the river of longer than approximately five miles, it was split into multiple storage areas.

5.2.6.4 Weir Coefficients.

Weir coefficients are specified within HEC-RAS for both lateral structures and storage area connections. Since historic floods resulted in levee breaching as the primary method of flow transfer from the Missouri River to the levee area, historic data is not useful in calibrating weir coefficients.

The HEC-RAS user manual provides recommended ranges for weir coefficients. Values are impacted by factors such as material type (roads or vegetated earthen structures), direction of approach flow to the weir, approach depth to the weir, weir overflow depth, weir length, submergence, and similar. The HEC 2014 document, "Combined 1D and 2D Modeling with HEC-RAS", provides additional information on lateral structures with recommended weir coefficients as shown in Table 5-4 (HEC, 2014) for various weir flow components.

Table 5-4: Typical Weir Coefficients.

Weir Flow Component	Description	Typical Coefficient Range (USACE, 2014)
Levee / roadway – 3 ft or higher above natural ground	Broad crested weir shape, flow over Levee / road acts like weir flow	1.5 – 2.6
Levee / roadway – 1 to 3 ft elevated above ground	Broad crested weir shape, flow over Levee / road acts like weir flow but becomes submerged easily	1.0 – 2.0
Natural high ground barrier – 1 to 3 feet high	Does not really act like a weir, but water must flow over high ground to enter area	0.5 – 1.0
Non-elevated overbank terrain. Lateral structure not elevated above ground	Overland flow escaping the main river.	0.1 – 0.5

In addition to the table, the document also provides several items to consider when selecting a weir coefficient for a lateral structure. Although the document is focused on HEC-RAS 1D to 2D applications, the considerations are applicable to lateral structures in general:

> *In general, Lateral Structure weir coefficients should be lower than typical values used for inline weirs. Additionally, when a lateral structure (i.e. weir equation) is being used to transfer flow from the river (1D region) to the floodplain (2D Flow Area), then the weir coefficients that are used need to be very low, or too much flow will be transferred.*
> *Note: The number 1 problem HEC-RAS users have been having when interfacing 1D river reaches with 2D Flow Areas, is using to high of a weir coefficient for the situation being modeled. If the lateral structure is really just an overland flow interface between the 1D river and the 2D floodplain, then a weir coefficient in the range of 0.1 to 0.5 must be used to get the right flow transfer and keep the model stable.*
> *Note: A second issue is weir submergence. When a lateral structure gets highly submerged, HEC-RAS uses a weir submergence curve to compute the flow reduction over the weir. The curve is very steep (i.e. the flow reduction changes dramatically) between 95% and 100% submergence. This can cause oscillations and possible model stability issues. To reduce these oscillations, user can have HEC-RAS use a milder sloping submergence curve* by going to the 1D “Computational Options and Tolerances” and setting the field labeled “Weir flow submergence decay exponent” to 3.0. (HEC, 2014)

A sensitivity analysis was conducted for two large historic flood events in this reach, 2011 and 1952, to evaluate impact on of the selected weir coefficient on results. All weir coefficients for both lateral structures and storage area connections were set to a constant value. Separate model runs were performed with three different values of 0.3, 0.8, and 2.0. Stage and flow hydrographs model results at RM 507.49 are shown in Figure 5-4 and Figure 5-5. This location was selected near the downstream end of the model as showing the greatest difference between model results. Locations reviewed that are further upstream showed smaller impacts from changing the weir coefficient.

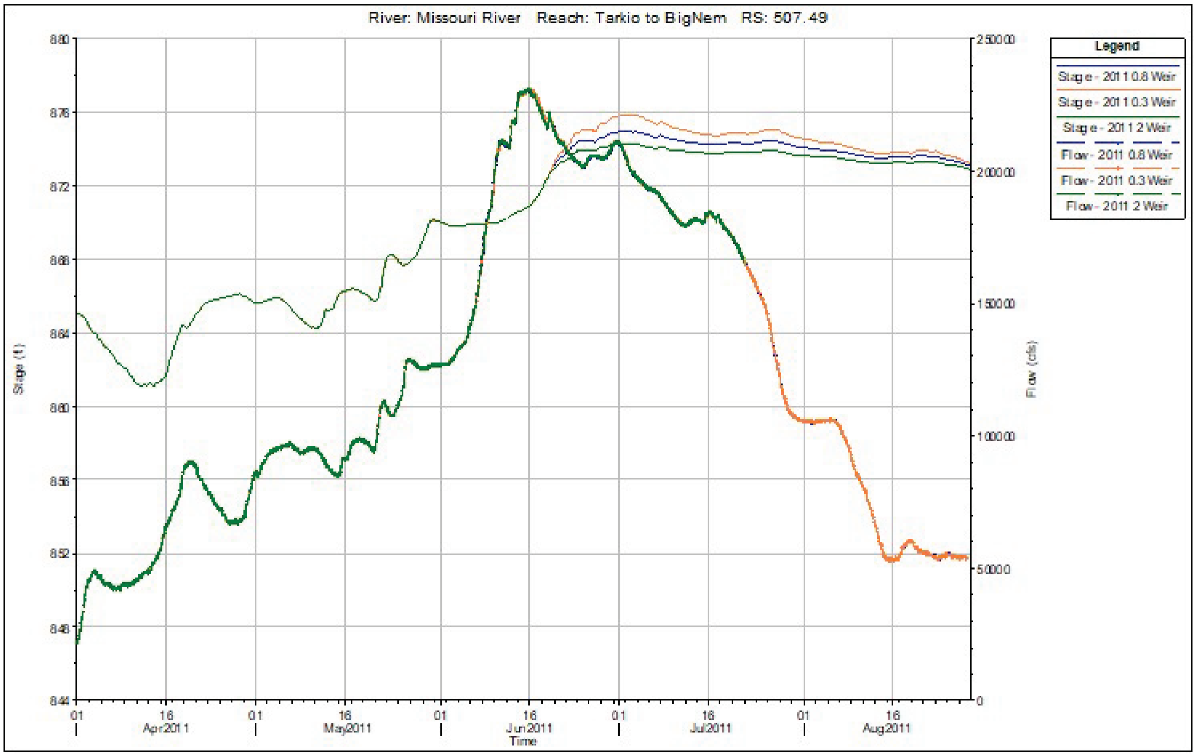


Figure 5-4. Weir Coefficient Sensitivity, 2011 Event

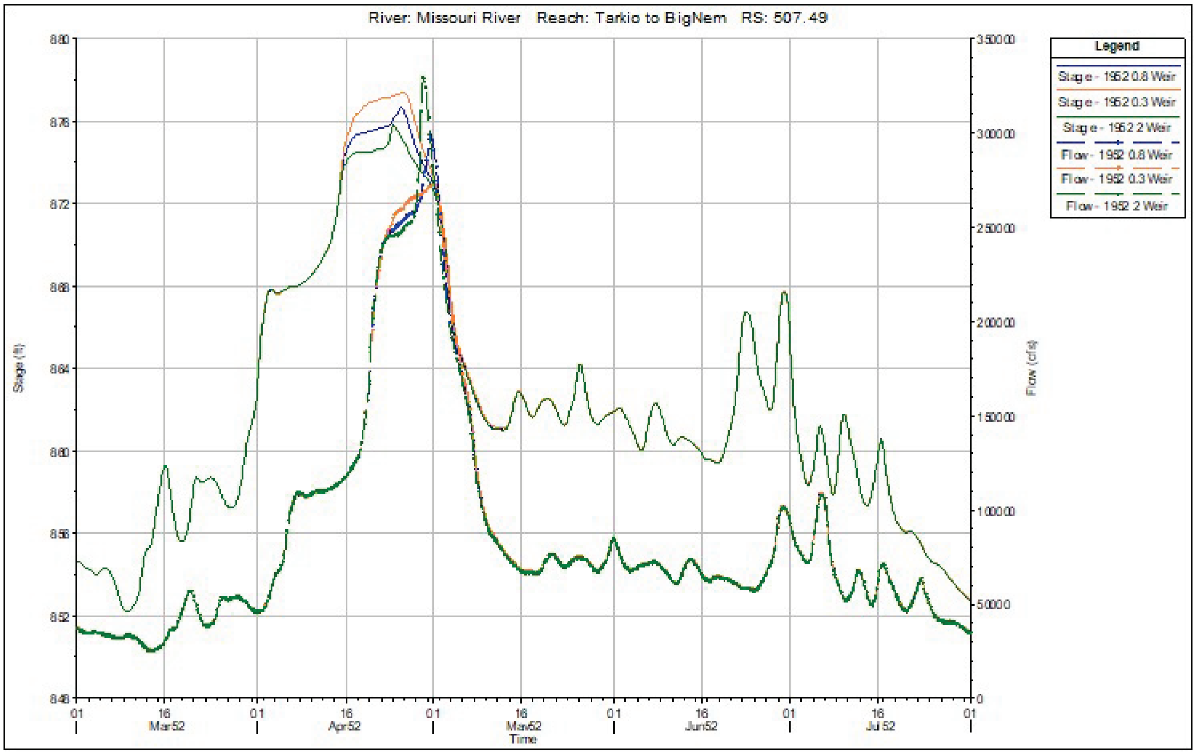


Figure 5-5. Weir Coefficient Sensitivity, 1952 Event

Results of the weir coefficient sensitivity analysis demonstrate that both peak flow and stage are dependent upon the weir coefficient. The duration of the impact of the weir coefficient on results will persist for the length of levee overtopping flow. After reviewing results and considering model stability performance, a coefficient of 0.8 was selected for all weir coefficients. This selection was based primarily on the parallel direction of most structures to the river, the mostly shallow overtopping depths, the recommendation from the modeling document (HEC, 2014) that lateral structure values are usually too high, and the model stability issues that occurred with a higher coefficient.

5.2.6.5 Tributary Tiebacks

Tributary tiebacks refer to the levee system components that are used to bracket tributary channels to convey flow into the main Missouri River channel while preventing Missouri River and tributary flooding within the Missouri River floodplain. Generally, the tiebacks are perpendicular to Missouri River flow as the tributary enters from the bluff and crosses the Missouri River floodplain. Most of the tributary tiebacks are very small drainage areas with minimal cross section flow area between the tieback levees, on the order of a few hundred feet in top width.

In contrast to the methodology employed in the Kansas City District model, tiebacks on the federal levees were modeled as a single storage area connection structure. This method was done to simplify computations and still accurately reflect the method in which flow is transferred within the floodplain levee cells parallel to the main channel. Observations and past flood performance indicates that once a federal levee overtops, the primary flow direction is within the leveed area parallel to the main channel. Flow proceeds downstream until reaching a ponding depth to return to the main Missouri River at the lowest levee elevation. This location usually occurs at the junction of the tieback and Missouri River levee. During a high flow event with prolonged levee overtopping such that the storage area fills, the model will connect flow from the upstream levee cell to either return to the river or over the tieback to the next downstream levee cell, depending upon elevation. This type of performance reflects actual observations of flood conveyance within Omaha District.

Larger rivers that form tiebacks are an exception to this method. For example, the Nishnabotna River, which is a large tributary modeled as a routing reach, includes separate lateral connections along the Nishnabotna River to the federal levee areas. By modeling in this manner, the tributary routing reach does not double count the available storage included with the tributary routing reach.

5.2.6.6 Emptying Storage Areas

Most extreme events that inundate the levee protected area also include levee overtopping or breaching that accommodates return flow to the main channel. Within HEC-RAS, the model must include a method to empty the storage area in advance of the next overtopping event. Breaching as a method to convey flow was considered but eliminated since breach location is highly speculative and also often results in model stability issues. At this point, model calibration did not require the need to empty the storage area. A suitable method to empty storage areas will be employed when necessary during multiple year simulations.

5.2.7 Sioux City to Omaha Modeling

Upstream of Omaha, a network of private levees exists with sparse information on construction method, elevation, location, and tieoff. As this reach is closer to Gavins Point Dam, flood flow frequency and flood damages have been reduced. The reach has also experienced some degradation. Experience during minor floods and even the 2011 flood indicated that HEC-RAS model construction could be simplified to use cross section geometry to adequately model the river. For all of these reasons, the decision was made to simplify the model in the reach from Sioux City to Omaha and private levees were not included as separate storage areas. The main channel and intermittent levees were modeled with cross sections and ineffective flow areas. Cross section levee points and blocked obstructions were also used to separate portions of the section to accurately describe flow conveyance. Since HEC-RAS only allows a single levee point on each side of the channel, blocked obstructions were also used to allow levee points along the main channel and to eliminate the wide portions of the section that had excessive storage.

The floodplain between Sioux City and Omaha is several miles wide. The 2011 flood event was used to provide information to assist with setting model effective flow width. In addition, evaluation of floodplain storage was performed using ineffective flow area (high storage) or blocked obstruction (reduced storage) to evaluate model geometry to best replicate storage during the 2011 event.. Multiple model geometries were created with different locations for levee section points and blocked obstructions. Comparison results from high storage and reduced storage options at Omaha are shown in Figure 5-6. It should be noted that these tests were done early in the model development process and do not illustrate final model calibration results.

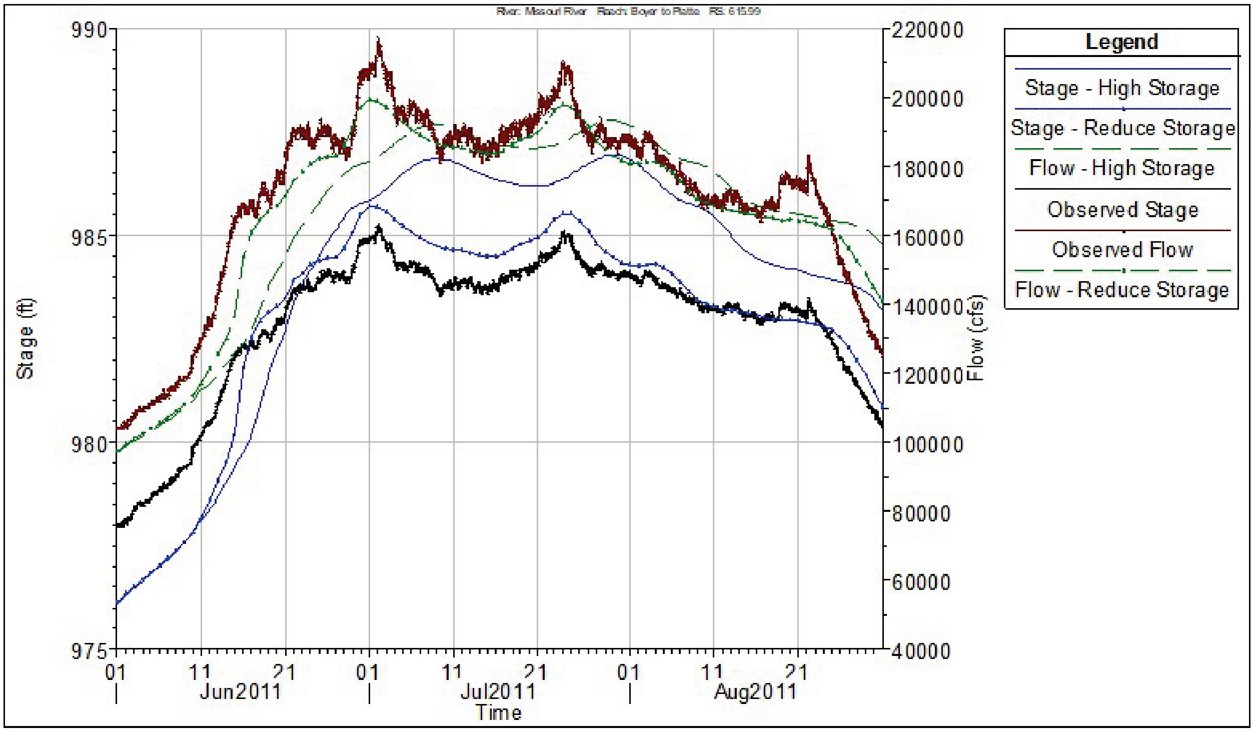


Figure 5-6. Missouri River at Omaha, Effect of Model Storage Upstream

Results illustrate that including additional cross section effective flow and storage in the reach upstream of Omaha did not match observed conditions. When cross section levee points and blocked obstructions were added to the model, hydrograph shape was much better and results matched both observed flow and stage values more closely.

5.2.8 Rulo, NE, Floodplain Area.

The Rulo, NE, floodplain area from about RM 515 downstream to RM 480 is a very wide floodplain of about 5 miles with multiple private levees and connections. The railroad and Highway 159 bridge embankments cross the floodplain and are major factors in flood flow conveyance. The geometry for this complex area was configured by Kansas City District. Refer to the Kansas City District appendix for further details regarding this reach.

5.2.9 Bridges

On the Missouri River main stem, cross sections representing bridge embankments are in the model, but the structures themselves are not. This was a simplification made to keep computation times shorter. In addition, all bridge deck low chords on the Missouri River are elevated higher than the floods of record, so the only component that would impede water flow is the bridge columns, which likely have a local effect, but not global. Bridges in the tributary models were left in the geometry unless they caused issues with model stability.

5.2.10 Dams

This stretch of the Missouri River was modeled from downstream of Gavins Point Dam to below Rulo, NE. The Gavins Point dam is not included in the model although dam releases are used for the upstream model boundary condition.

5.2.11 Tributaries

Tributary reaches were included within the model to route flow from the gage station to the Missouri River and were not calibrated to stage. Including the tributary geometries also accounts for backwater storage from the Missouri River main channel. Thirteen tributary routing reaches are included in the model as previously shown in Table 4-3. Two of these tributaries, the Big Nemaha and Nodaway Rivers, are within the model overlap reach downstream of Rulo, NE.

Tributary model geometry was developed from a mixture of data sources of limited accuracy and mostly dated surveys or else from a limited accuracy DEM. Most tributary models are primarily low quality from previous study efforts. Several tributaries required substantial alteration to the section for stability purposes including the Vermillion River, Soldier River, Boyer River, Platte River, and Weeping Water Creek. In general, the goal with the tributary routing reaches was to model travel time with sufficient detail from the tributary gage station to the Missouri River to preserve tributary timing for Missouri River calibration purposes. No tributary computed stage information should be used from model results without carefully assessing the purpose and considering model construction limitations.

5.2.12 Missouri River / Tributary Junctions

Missouri River and tributary junctions are defined within the junction editor within HEC-RAS. Modeling test runs indicated that the method selected to model the junction was a factor in model stability and computation results. Junction options include equal water surface and energy balance using either energy or momentum. While the equal water surface method reduced the number of model iterations and appeared the more stable method, the energy method was selected at all junctions with large length between the junction and adjacent cross sections. No significant difference in results was observed between the energy or momentum methods of computing energy loss.

5.2.13 Floodplain Chutes

Multiple natural and constructed chutes exist in the floodplain adjacent to the Missouri River. The conveyance areas of these chutes are represented in the model by the standard river cross sections. Due to the high number of chutes, it was not feasible to model these as split flow with junctions. These chutes were modeled with ineffective flow areas blocking the floodplain until river flow exceeded channel capacity and entered the floodplain. At some locations, levee points were used on the river bank to block the chute from flow if necessary to reflect conditions at time of model calibration. For the chute locations, revision to model geometry to open or close a chute will be made to future model efforts to coincide with alternative formulation. Normal flow calibration accuracy indicated that the selected chute modeling method was acceptable. A plan view to illustrate typical cross section geometry at a chute location is shown in Figure 5-7. A typical cross section is shown in Figure 5-8. At the model steady flow period in Sep 2012, the chute at this location was not fully connected and chuteflow through was not possible. To reflect this condition, the model cross section shows that the chute flow area is blocked with a cross section levee point.

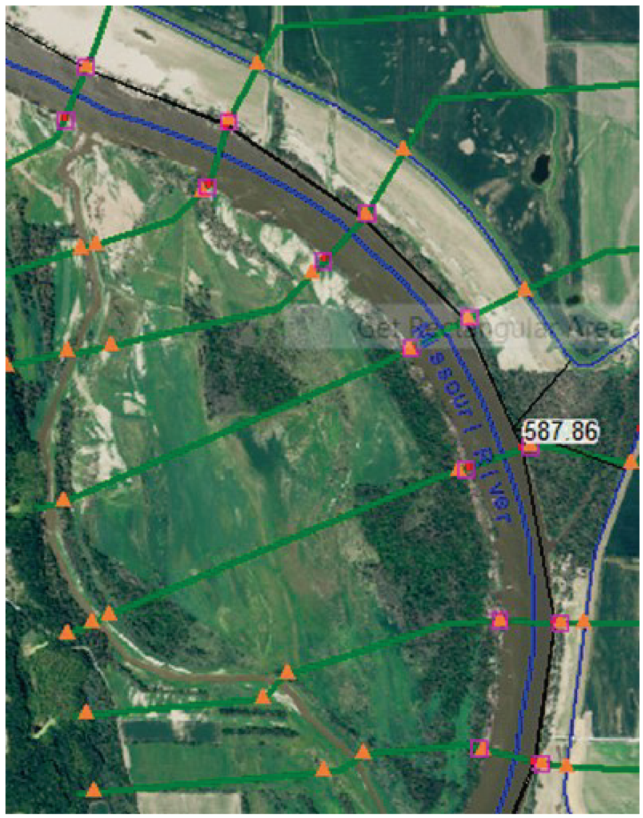


Figure 5-7. Plan View Location with Chute

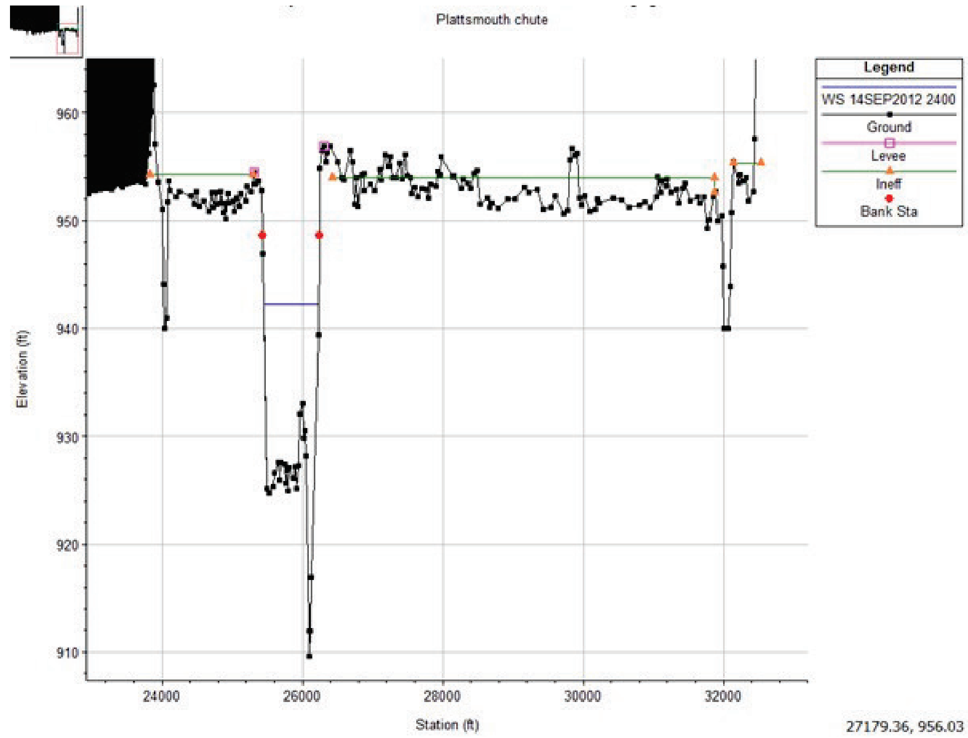


Figure 5-8. Model Geometry Example Floodplain Chute Cross Section

5.2.14 Model Ice Parameters

The hydraulic analysis does not include any input parameters or adjustment for ice conditions. Typically, flood events in the early spring will include floating ice with the potential for ice jams to occur. Installation of the mainstem dams has altered the frequency of spring floods and the accompanying ice jams.

5.2.15 Model Overlap Reach.

The HEC-RAS model was constructed with an overlap reach centered on the Kansas City / Omaha District boundary at Rulo, Nebraska (RM 498). The Kansas City model includes cross section geometry starting at Nebraska City (RM 562) which provides about 60 river miles of overlap while the Omaha District model contains geometry below Rulo to downstream of St Joseph (RM 448) which provides over 50 miles of overlap. Including the overlap reach between the two models assures that model results are not driven by boundary conditions. Within the Omaha District model, the geometry downstream of Rulo, NE, that was from the Kansas City District model geometry was modified to reduce computation stability issues and model complexity. Model results will be reported with the separation point at Rulo, NE, such that values in either District within the overlap reach will be ignored.

5.3 UNGAGED INFLOW

Ungaged inflow refers to that portion of the flow that is not captured by the gage station records. Ungaged inflow computation has been automated within HEC-RAS and is fully described within the User’s Manual (USACE, 2010). Ungaged calculations are made between two gages on the main stem which have a continuous record of both stage and flow. Model calibration accuracy was improved by using ungaged inflow to better replicate river flows.

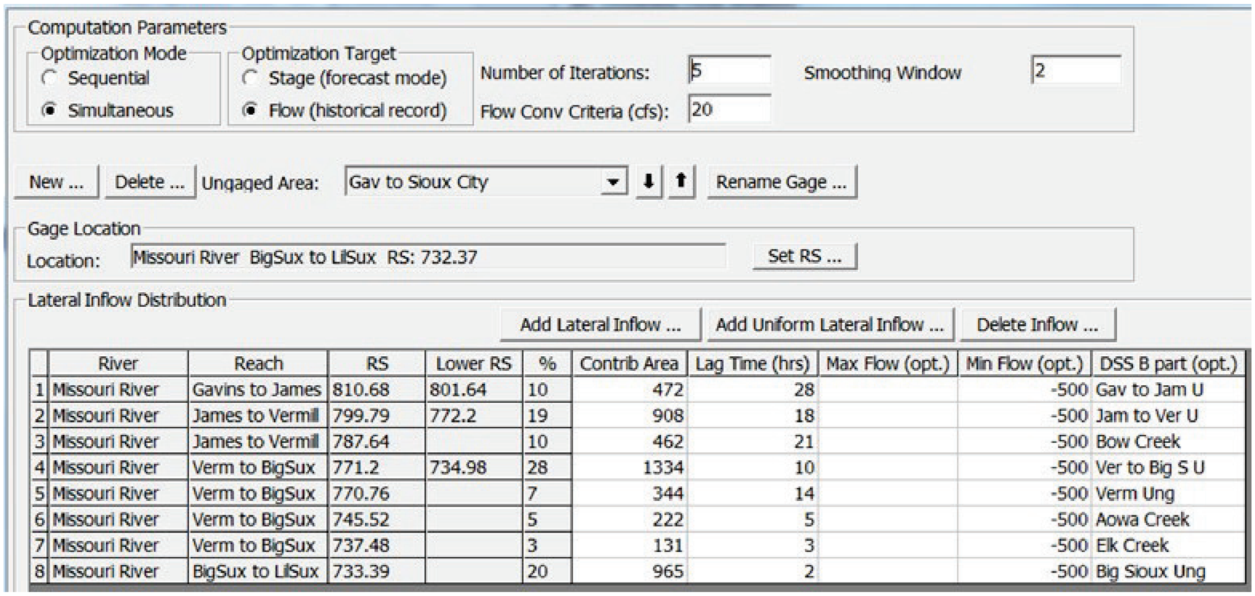
The ungaged flow calculation is made by running the unsteady model with internal stage and flow boundaries at the gage locations mentioned above. At the endpoint, the calculated routed flow hydrograph is compared to the observed hydrograph, and the difference is calculated. The difference is put back into the model between the two gages at user specified locations with a backwards lag in time and given distribution and the model is run again. This process is repeated until the flow at the endpoint either matches the flow convergence desired or meets the maximum number of iterations specified.

Lag time was input as the approximate travel time from the lateral inflow location to the gage station. For uniform lateral inflows, the travel time from the midpoint of the segment to the gage was used. Average velocity in the reach of the Missouri River is about 3 to 4 ft/s, or 2 to 2.5 mi/hr. Simultaneous was selected as the optimization mode. The simultaneous option makes ungaged calculations for each reach independent of the others, whereas the sequential option runs calculations for each reach in order of upstream to downstream taking into account any lack in flow convergence that may have occurred in the upstream reach.

Execution of the ungaged inflow for the calibration period (Mar 2011 thru Jul 2013) was problematic and had to be executed in several phases. In addition, HEC-RAS 4.2 beta version contained a bug which did not allow for use of levee connections while computing ungaged.The 2011 event included levee breaches with significant floodplain flow which limited the accuracy of the ungaged flow computations. The calibration period after the 2011 event did not include levee overtopping and ungaged computations were not affected.

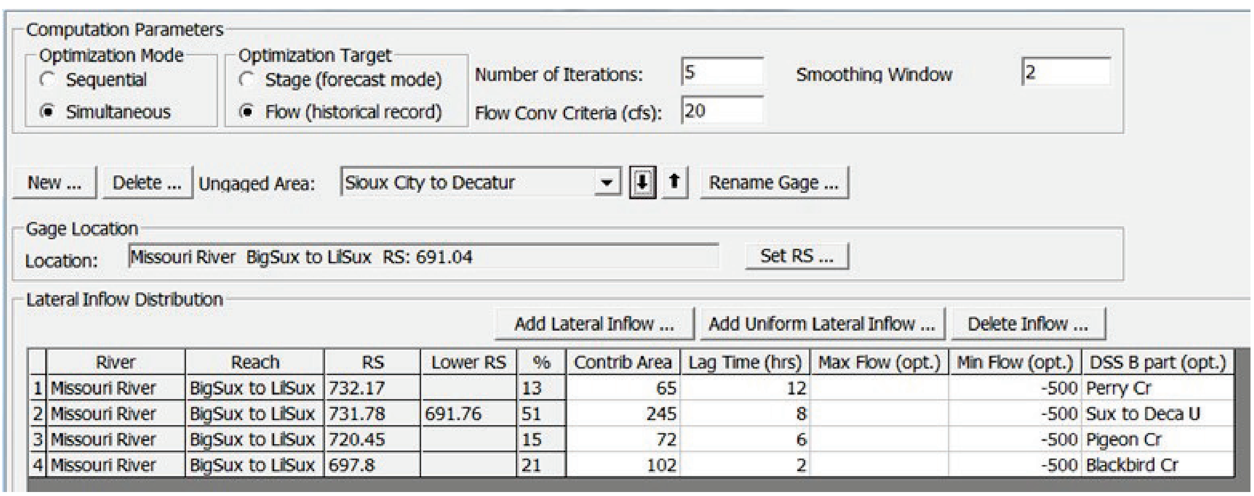
Negative flows computed during the ungaged process are common and were encountered. This is caused by a number of reasons including gaged inflow error, model timing, areas with significant water use or groundwater recharge, and similar. Ungaged inflow hydrographs were reviewed and determined as reasonable. Calibration accuracy was improved by using the determined ungaged inflows.

Ungaged inflow parameters are entered in the unsteady flow analysis options menu. Flow / stage gage records are available at Sioux City, Decatur, Omaha, Nebraska City, and Rulo as previously shown in Table 4-2. Ungaged flow within each reach was distributed by prorating the remaining drainage area after the gage station tributary drainage areas were removed. Input parameters for each of the ungaged inflow computation sections as specified within the RAS model interface are shown in Figure 5-9 through Figure 5-13.



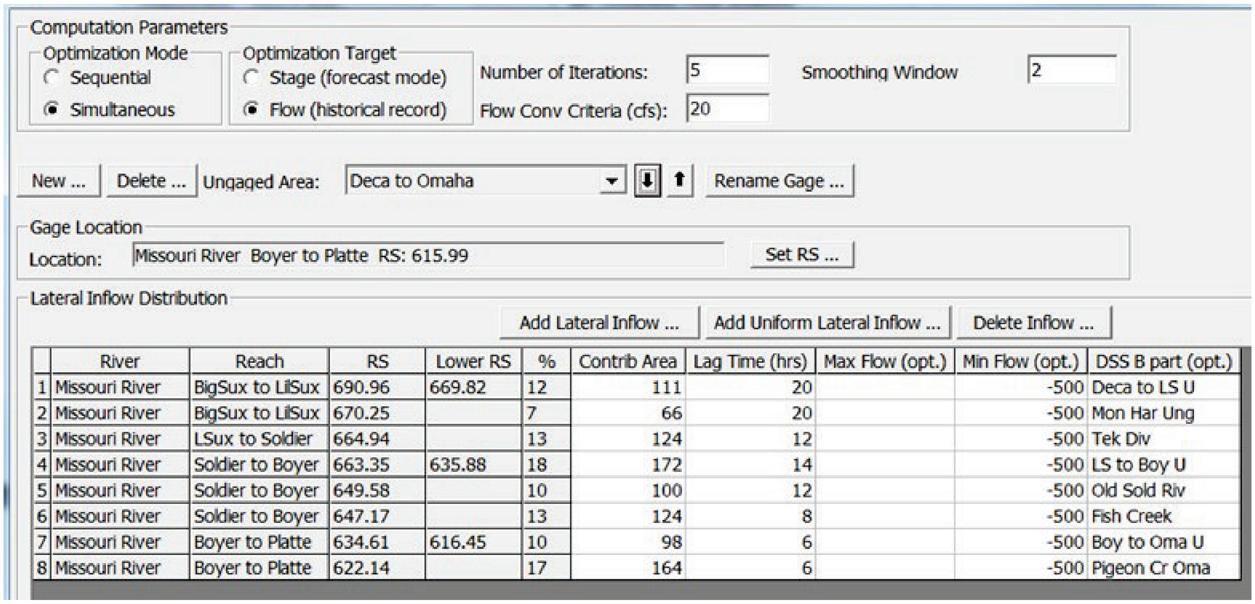
	River	Reach	RS	Lower RS	%	Contrib Area	Lag Time (hrs)	Max Flow (opt.)	Min Flow (opt.)	DSS B part (opt.)
1	Missouri River	Gavins to James	810.68	801.64	10	472	28		-500	Gav to Jam U
2	Missouri River	James to Vermil	799.79	772.2	19	908	18		-500	Jam to Ver U
3	Missouri River	James to Vermil	787.64		10	462	21		-500	Bow Creek
4	Missouri River	Verm to BigSux	771.2	734.98	28	1334	10		-500	Ver to Big S U
5	Missouri River	Verm to BigSux	770.76		7	344	14		-500	Verm Ung
6	Missouri River	Verm to BigSux	745.52		5	222	5		-500	Aowa Creek
7	Missouri River	Verm to BigSux	737.48		3	131	3		-500	Elk Creek
8	Missouri River	BigSux to LilSux	733.39		20	965	2		-500	Big Sioux Ung

Figure 5-9. Ungaged Inflow Gavins to Sioux City



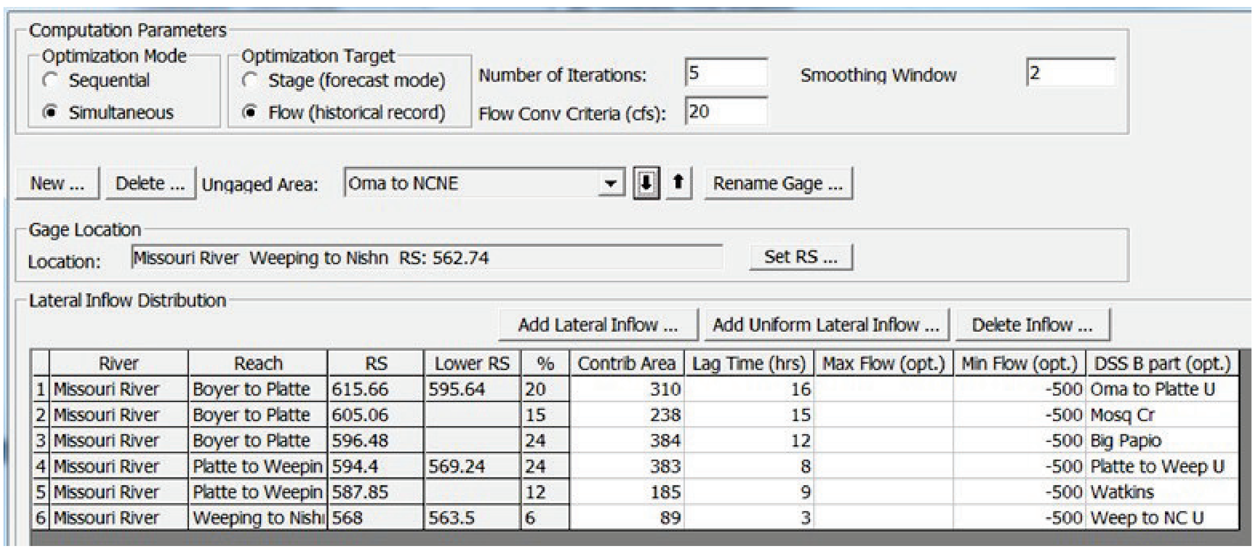
	River	Reach	RS	Lower RS	%	Contrib Area	Lag Time (hrs)	Max Flow (opt.)	Min Flow (opt.)	DSS B part (opt.)
1	Missouri River	BigSux to LilSux	732.17		13	65	12		-500	Perry Cr
2	Missouri River	BigSux to LilSux	731.78	691.76	51	245	8		-500	Sux to Deca U
3	Missouri River	BigSux to LilSux	720.45		15	72	6		-500	Pigeon Cr
4	Missouri River	BigSux to LilSux	697.8		21	102	2		-500	Blackbird Cr

Figure 5-10. Ungaged Inflow Sioux City to Decatur



	River	Reach	RS	Lower RS	%	Contrib Area	Lag Time (hrs)	Max Flow (opt.)	Min Flow (opt.)	DSS B part (opt.)
1	Missouri River	BigSux to LilSux	690.96	669.82	12	111	20		-500	Deca to LS U
2	Missouri River	BigSux to LilSux	670.25		7	66	20		-500	Mon Har Ung
3	Missouri River	LSux to Soldier	664.94		13	124	12		-500	Tek Div
4	Missouri River	Soldier to Boyer	663.35	635.88	18	172	14		-500	LS to Boy U
5	Missouri River	Soldier to Boyer	649.58		10	100	12		-500	Old Sold Riv
6	Missouri River	Soldier to Boyer	647.17		13	124	8		-500	Fish Creek
7	Missouri River	Boyer to Platte	634.61	616.45	10	98	6		-500	Boy to Oma U
8	Missouri River	Boyer to Platte	622.14		17	164	6		-500	Pigeon Cr Oma

Figure 5-11. Ungaged Inflow Decatur to Omaha



	River	Reach	RS	Lower RS	%	Contrib Area	Lag Time (hrs)	Max Flow (opt.)	Min Flow (opt.)	DSS B part (opt.)
1	Missouri River	Boyer to Platte	615.66	595.64	20	310	16		-500	Oma to Platte U
2	Missouri River	Boyer to Platte	605.06		15	238	15		-500	Mosq Cr
3	Missouri River	Boyer to Platte	596.48		24	384	12		-500	Big Papio
4	Missouri River	Platte to Weepin	594.4	569.24	24	383	8		-500	Platte to Weep U
5	Missouri River	Platte to Weepin	587.85		12	185	9		-500	Watkins
6	Missouri River	Weeping to Nishi	568	563.5	6	89	3		-500	Weep to NC U

Figure 5-12. Ungaged Inflow Omaha to Nebraska City

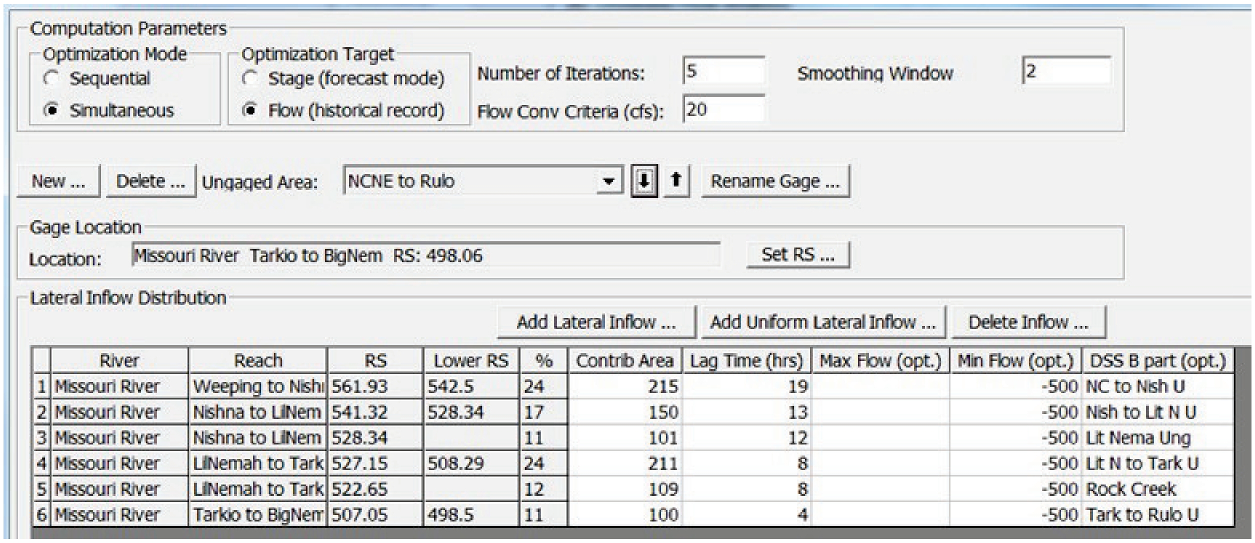


Figure 5-13. Ungaged Inflow Nebraska City to Rulo

5.4 Boundary Conditions

The boundary conditions are the flows and stages used at the upstream and downstream extents of the HEC-RAS model. Below is a discussion of those boundary conditions.

5.4.1 Upstream Boundary Conditions

Upstream boundary conditions include the outflow from Gavins Point Dam and observed USGS flow hydrographs at the top of each of the tributary reaches. Hourly data was used when available and daily data was used to complete the flow record. To achieve stability, a minimum flow was used for each input, as shown in Table 5-5.

Table 5-5: Minimum Flows

Location	Minimum Flow (cfs)
Gavins Point Outflow	5000
James River	50
Vermillion River	50
Big Sioux River	50
Little Sioux River	50
Soldier River	50
Boyer River	50
Platte River	500
Weeping Water Creek	50
Nishnabotna River	50
Little Nemaha River	50
Tarkio River	50
Big Nemaha River	50
Nodaway River	50

5.4.2 Downstream Boundary Condition

The downstream boundary condition used normal depth at cross section 422.56 which is about 20 miles downstream of St. Joseph, MO. After some iteration, a slope of 0.0002 ft/ft was selected.

5.4.3 Storage Areas

The initial elevation in all storage areas was set at the storage area invert such that all areas are dry as well as all lateral connections.

5.5 COMPUTATIONAL OPTIONS.

The HEC-RAS model includes numerous computational options that are accessed from the unsteady run options drop down menu, Calculation options and tolerances. These parameters are set when executing an unsteady flow model. Many of these options assist with model stability. Selected parameters are shown in Figure 5-14.

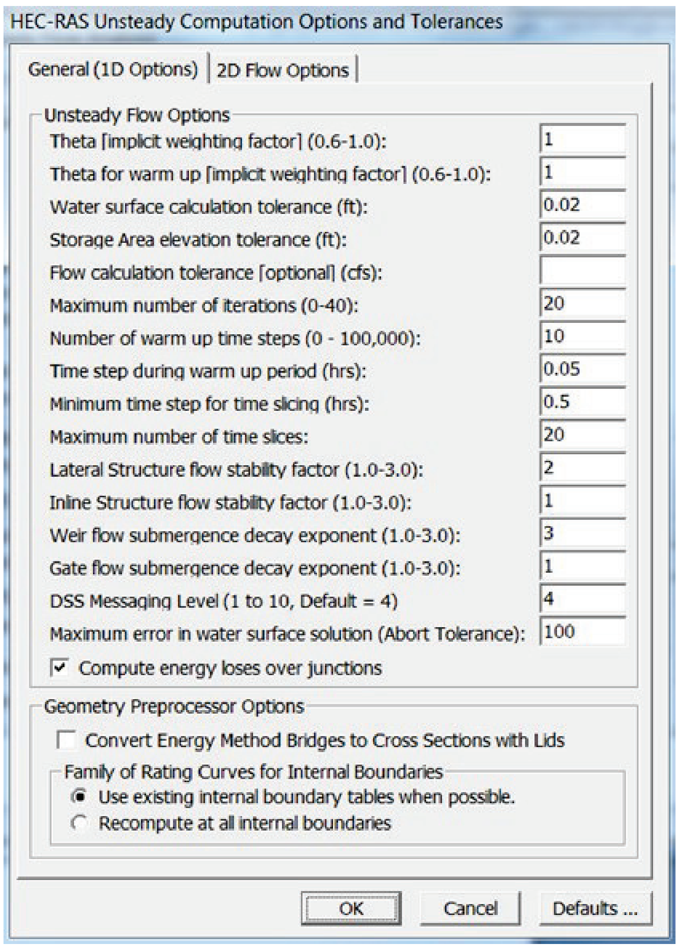


Figure 5-14. HEC-RAS Computation Options

6 CALIBRATION

Model calibration was accomplished through several steps described in this section. Results as well as a discussion of the level of calibration achieved and overall model performance are presented below.

6.1 MODEL CALIBRATION

Unlike typical modeling efforts that are focused on evaluation of flood damage reduction projects for extreme events, model development and calibration for this study was performed to provide reasonably accurate results for a broad spectrum of flows from normal flows to high flows The primary source of calibration data was observed stage and flow hydrographs on the main stem Missouri River gages and field measured water surface profile data that was collected for high flow in June 2011 and normal flow in June 2012 and Sep 2012. While 2012 was regarded as the calibration period, data from 2013 was also considered due to the ongoing river adjustment following the high 2011 flows.

First, the model was calibrated in a steady flow simulation mode. A thorough check of cross section geometry to minimize errors in computation was performed. This included typical hydraulic modeling review such as checking the change in flow distribution, velocity, top width, flow area, and energy grade elevation at various flows. The steady flow model was also calibrated to the water surface profiles by adjusting channel Manning's n-values. The channel Manning's n-values were initially set at 0.025 and were adjusted for steady flow calibration to obtain a water surface elevation that was within a tolerance of the measured water surface profiles with a desired accuracy in the range of 0.5 to 1 foot.

Second, the model was run in the unsteady state with steady flows to obtain a stable model. Then, one by one, tributary geometries were added into the model. The tributaries in the model were roughly calibrated and were inserted for the primary purpose of routing flows from the gage to the Missouri River. Once the model was stable with all the tributaries added, the observed flows were added into the model as well as the computed ungaged flows. The model was run from March 2011 to July 2013 and results were compared to the water surface profile data for the time period it was collected and the observed stage and flow from available gages. Multiple iterations were required in this process with roughness values and levee stations, and ineffective flow locations to obtain acceptable results.

6.2 FLOW ROUGHNESS FACTORS

Calibration philosophy was to primarily use the base roughness values to calibrate the model for normal flows and use the HEC-RAS option for flow roughness to calibrate for higher flow events. This was successful in most reaches except for a ten mile long reach near Gavins Point Dam that required additional flow roughness factors to match the 2012 measured profile. Determined flow roughness factors for the reaches used in calibration are shown in a series of tables, Table 6-1 through Table 6-3.

Table 6-1: Flow Roughness Factors (Upper Model Group)

Cross Section Range	810.87	Cross Section Range	800.21	780.45	771.77	738.79	733.78
	800.98		780.83	771.79	739.49	734.2	715.22
Flow (cfs)	**Roughness Factor**	**Flow (cfs)**	**Roughness Factor**				
0	1.	0	1	0.85	1.05	1.0	1.08
40,000	1	40,000	1	0.9	1.05	1.0	1.08
60,000	1.1	70,000	1.08	1.1	1.1	1.	1.05
80,000	1.18	100,000	1.12	1.1	1.1	1.	1.05
100,000	1.24	150,000	1.12	1.1	1.1	1.05	1.05
150,000	1.24	200,000	1.12	1.1	1.1	1.1	1.05

Table 6-2: Flow Roughness Factors (Middle Model Group)

Cross Section Range	714.79	669.02	663.76	635.02	616.08	595.0	568.42	541.73
	669.45	664.14	635.46	616.45	595.02	568.82	542.1	527.96
Flow (cfs)		**Roughness Factor**						
0	0.94	0.94	0.94	0.94	0.94	1.0	1.0	1.0
15,000	0.96	0.96	0.96	0.96	0.96	1.0	1.0	1.0
25,000	1.	1.	1.	1.	1.	1.0	1.0	1.0
40,000	1.	1.	1.	1.	1.	1.0	1.0	1.0
70,000	1.1	1.05	1.01	1.05	1.05	1.05	1.08	1.0
100,000	1.1	1.05	1.1	1.05	1.05	1.1	1.1	0.95
150,000	1.1	1.1	1.1	1.05	1.05	1.1	1.1	0.95
200,000	1.1	1.1	1.04	1.05	1.01	1.1	1.1	0.95
250,000	1.1	1.1	1.04	1.05	1.01	1.1	1.1	0.95
300,000	1.1	1.1	1.04	1.05	1.01	1.1	1.1	0.95
350,000	1.1	1.1	1.04	1.05	1.01	1.1	1.1	0.95

Table 6-3: Flow Roughness Factors (Lower Model Group)

Cross Section Range	527.55	507.49	Cross Section Range	498.08
	507.9	498.5		463.17
Flow (cfs)	**Roughness Factor**		**Flow (cfs)**	**Roughness Factor**
0	1.0	1.0	0	0.95
50,000	1.0	1.0	25,000	0.95
100,000	1.0	1.05	35,000	0.95
150,000	1.0	1.1	45,000	0.95
200,000	1.0	1.1	55,000	0.95
250,000	1.0	1.1	70,000	1.0
300,000	1.0	1.1	400,000	1.0

6.3 SEASONAL ROUGHNESS FACTORS

Seasonal roughness factors occur on natural rivers due to several factors including bed roughness and vegetation changes. The open-water stage flow relationship along the Missouri River are frequently seasonal in nature. Stages usually range from 0.5 to 1.0 foot or more higher in the summer (warm water) than in the winter (cold water) seasons.

A report on Missouri River temperature studies describes these effects (USACE, 1977). Temperatures affect hydraulic boundary resistance with near planar bed conditions observed in the winter compared to large dunes in the summer. Base model calibration was performed for the summer months using observed gage and water surface profile data. The HEC-RAS seasonal roughness capability was used to reduce roughness in the cold water season. Seasonal factors used within the model are tabulated in Table 6-4. The seasonal factors do not reflect ice jam conditions.

Table 6-4: Seasonal Flow Roughness Factors

Cross Sect. Range	810.87	Cross Sect. Range	773.78	669.02	Cross Sect. Range	635.02	Cross Sect. Range	595	Cross Sect. Range	541.73
	734.2		669.45	635.46		595.02		542.1		422.56
Date	**Factor**	**Date**	**Factor**	**Factor**	**Date**	**Factor**	**Date**	**Factor**	**Date**	**Factor**
1-Jan	0.95	1-Jan	0.92	0.95	1-Jan	0.95	1-Jan	0.95	1-Jan	0.95
1-Feb	0.95	1-Feb	0.92	0.95	1-Feb	0.95	1-Feb	0.95	1-Feb	0.95
1-Mar	0.97	1-Mar	0.97	0.97	1-Mar	0.97	1-Mar	0.97	1-Mar	0.97
1-Apr	1	1-Apr	1	1	1-Apr	1	1-Apr	1	1-Apr	1
1-May	1	1-May	1	1	1-May	1	1-May	1	1-May	1
1-Jun	1	1-Jun	1	1	1-Jun	1	1-Jun	1	1-Jun	1
1-Jul	1	1-Jul	1	1	1-Jul	1	1-Jul	1	1-Jul	1
1-Aug	1	1-Aug	1	1	1-Aug	1	1-Aug	1	1-Aug	1
1-Sep	1	1-Sep	1	1	1-Sep	1	1-Sep	1	1-Sep	1
1-Oct	0.95	15-Oct	0.95	0.95	1-Oct	1	1-Oct	1	1-Oct	1
1-Nov	0.95	1-Nov	0.95	0.95	1-Nov	0.95	15-Nov	0.95	1-Nov	1
1-Dec	0.95	1-Dec	0.92	0.95	1-Dec	0.95	1-Dec	0.95	1-Dec	0.95

6.4 2011 Flood Calibration Issues

Several factors presented a challenge with the unsteady model calibration. The observed rating curve during the 2011 high flow event was difficult to calibrate to both the rising and falling limbs of the event. As a result of the major degradation that occurred during the event, calibration on the rising side of the flood hydrograph using post flood data was not feasible with a fixed bed model. An example of the degradation that occurred during the event and the changed in the rating at a specific location is shown in Figure 6-1.

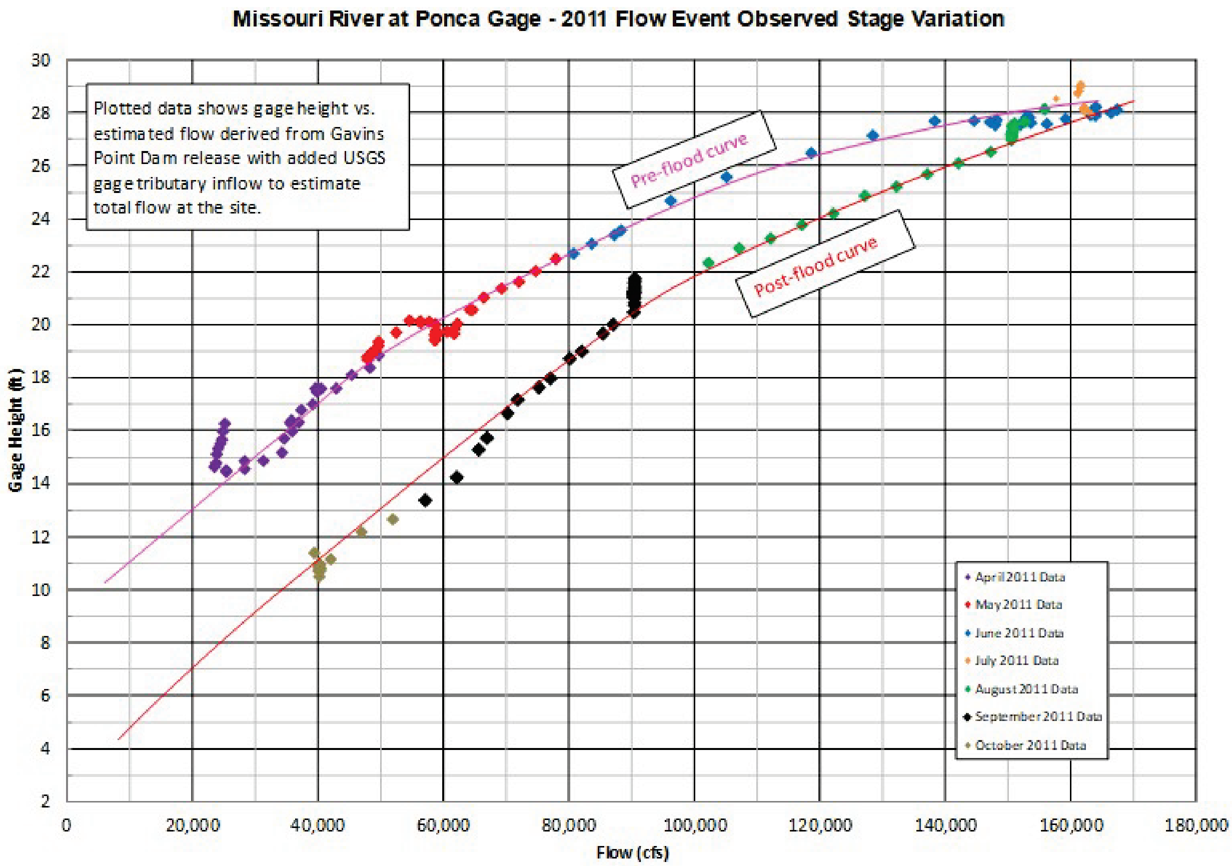


Figure 6-1. Example rating curve shift and degradation in 2011 event.

The calibration goal was to achieve a water surface elevation within 1 ft for the entire reach and less than 0.5 ft for most of the reach for both the measured water surface profiles and the observed gage data for 2011 and 2012. The calibration goal excludes periods affected by 2011 event geometry changes, ice conditions, and levee breaching. As previously stated, levee breaching and degradation that occurred during the flood were not considered. Either of these factors can affect river stage by several feet and as apparent in the observed 2011 event stage hydrographs. Calibration limitations for the 2011 event are further discussed in the following section.

6.5 Calibration Results

Plate 1 through Plate 18 present the hydrographs and computed minus observed stage vs flow plots for the gage locations. Plate 25 through Plate 45show the computed profile vs the measured water surface profile.

Model calibration results are within the desired range as computed stages at most locations are within 0.5 to 1 foot of observed stages. The measured profile calibration also provides confidence in model performance between the gage station locations. In summary, comparison of model results to gage station hydrographs and measured profiles show that the model performs with acceptable accuracy. A comparison of representative peak stages for the 2011 flood are shown in Table 6-5. Note that due to timing differences and levee breach impacts, the tabulated values are not good indicators of model performance in many locations. The peak stage difference was selected to represent the primary difference at tabulated locations. Refer to the gage hydrograph plots for modle performance.

Table 6-5: 2011 Flood Peak Stage Comparison

Location	Date[3]	Peak Stage Difference Model - Observed (ft)[4]
RM 805.76 – Yankton[1]	8 Jul 2011	+0.2
RM 775.26 – Maskell[1]	10 Jul 2011	+0.4
RM 751.17 – Ponca[1]	20 Jul 2011	+0.5
RM 732.37 – Sioux City[1]	20 Jul 2011	+0.3
RM 691.07 – Decatur[1]	20 Jul 2011	-0.5
RM 648.25 – Blair[1]	30 Jun 2011	+0.1
RM 615.98 – Omaha[1]	2 Jul 2011	+0.3
RM 591.50 – Plattsmouth[2]	1 Jul 2011	-0.1
RM 562.60 – Nebraska City[2]	28 Jun 2011	Not meaningful due to levee breaches (Plate 19)
RM 535.25 – Brownville[2]	23 Jun 2011	Not meaningful due to levee breaches (Plate 21)
RM 498.04 – Rulo[2]	27 Jun 2011	Not meaningful due to levee breaches (Plate 23)

1 Gage located in degradation reach during 2011 event that limited calibration accuracy.
2 Gaged located in levee breach and overtopping reach that limited calibration accuracy.
3 Peak date is approximate time of occurrence.
4 Difference was difficult to determine at some locations due to timing and levee breach

6.5.1 Stage Trend Impacts

Due to the extreme 2011 event flows and the high degree of channel adjustment that occurred during the event, accurate stage calibration prior to 2011 using the post-2011 event model geometry is not possible. Model results for the rising portion of the event in May and June demonstrate how stage-flow relationships changed during the flood and also reduce calibration accuracy through this portion of the event. Plotted data shown in Figure 6-1 illustrate stage change that occurred at some locations during the 2011 event. As a result, the model calibration for the 2011 event should be viewed understanding that changes in the stage-flow relationship during the flood prevent accurate model calibration for the entire flood event. However, model accuracy for simulating future events or performing alternative analysis is not affected.

6.5.2 Levee Breaching During the 2011 Event

During the 2011 flow event, the number of levee breaches downstream of Omaha prohibited calibration to observed stages without performing detailed modeling of multiple breaches. Not including the breaches limited calibration accuracy. These breaches occurred primarily during the period from mid-June through July. Calibration for 2011 focused on matching stage at the peak of the event, recognizing that it would be difficult to calibrate to both the rising pre-flood side of the hydrograph due to stage trend changes and the falling side of the hydrograph in areas affected by levee breaches. Since accurately modeling these breaches would, be very difficult, not helpful for calibration of future events, and that the alternative analysis will not include levee breaches, levee breach modeling was not performed. The high stage profile collected in 2011 was used for calibration to the extent possible.

6.5.1 Calibration Results Affected by Ice Conditions

Ice affected conditions including ice cover, ice breakup, and ice jams occur annually within the basin. Ice formation conditions typically occur in late November to late December with iceout typically occur in the early spring, usually in the March to April time frame. No ice parameters were included in the model development or calibration. Therefore, winter condition model calibration results between 2011 and 2013 on plotted hydrographs should be viewed with caution and recognize that results do not include any parameters to account for ice conditions.

7 CONCLUSIONS

The model performs well for the 2011 and 2012 observed data and is calibrated to the 2011 and 2012 water surface profiles. Significant points to consider with respect to model construction and calibration are as follows:

Measured profile calibration in 2012 and gage hydrograph calibration for both 2011 and 2012 indicates that the model performs satisfactorily with a stage calibration accuracy within 0.5 to 1 foot at most locations.

The HEC-RAS model was constructed with an overlap reach centered on the Kansas City / Omaha District boundary at Rulo, Nebraska (RM 498). Each model will be used to report results upstream and downstream of Rulo, NE.

No tributary computed stage information should be used from model results without carefully assessing the purpose and considering model construction limitations.

Aggradation and degradation that occurred during the 2011 event reduces calibration accuracy for the flood hydrograph. This also prevents calibrating to flow events prior to 2011.

Levee breaches are not included in the model. This limits model calibration accuracy during the period of significant levee breach flow. However, model accuracy for simulating future events or performing alternative analysis is not affected.

Ungaged inflows are an important parameter in model calibration. Computation of ungaged inflow with HEC-RAS appeared to enhance model flow accuracy compared to observed flow at the gaging stations.

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APPENDIX D

GAVINS POINT DAM TO RULO, NE

PLATES

Note: All elevations are NAVD 88 vertical datum

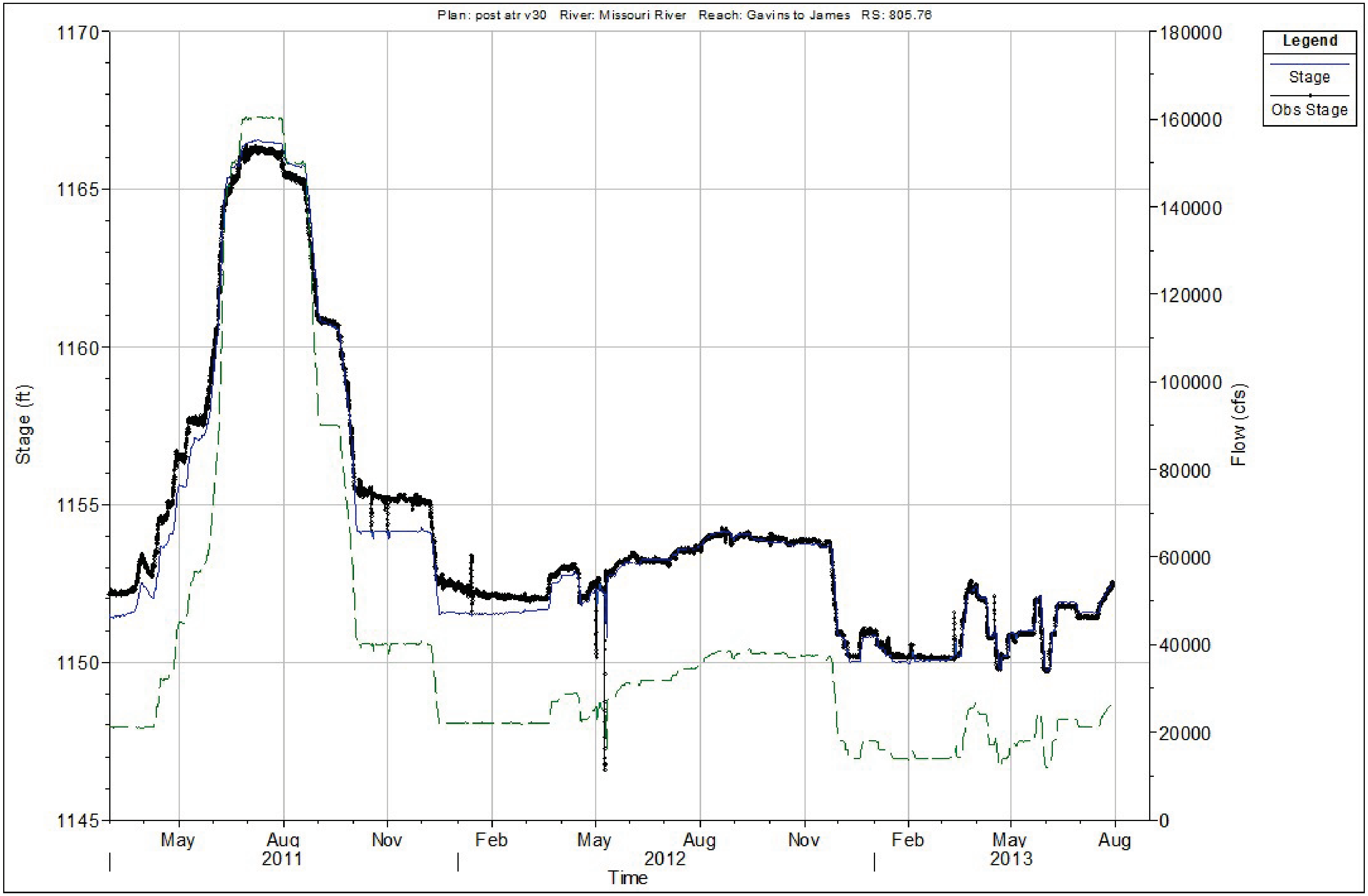


Plate 1: Missouri River at Yankton, SD Hydrograph

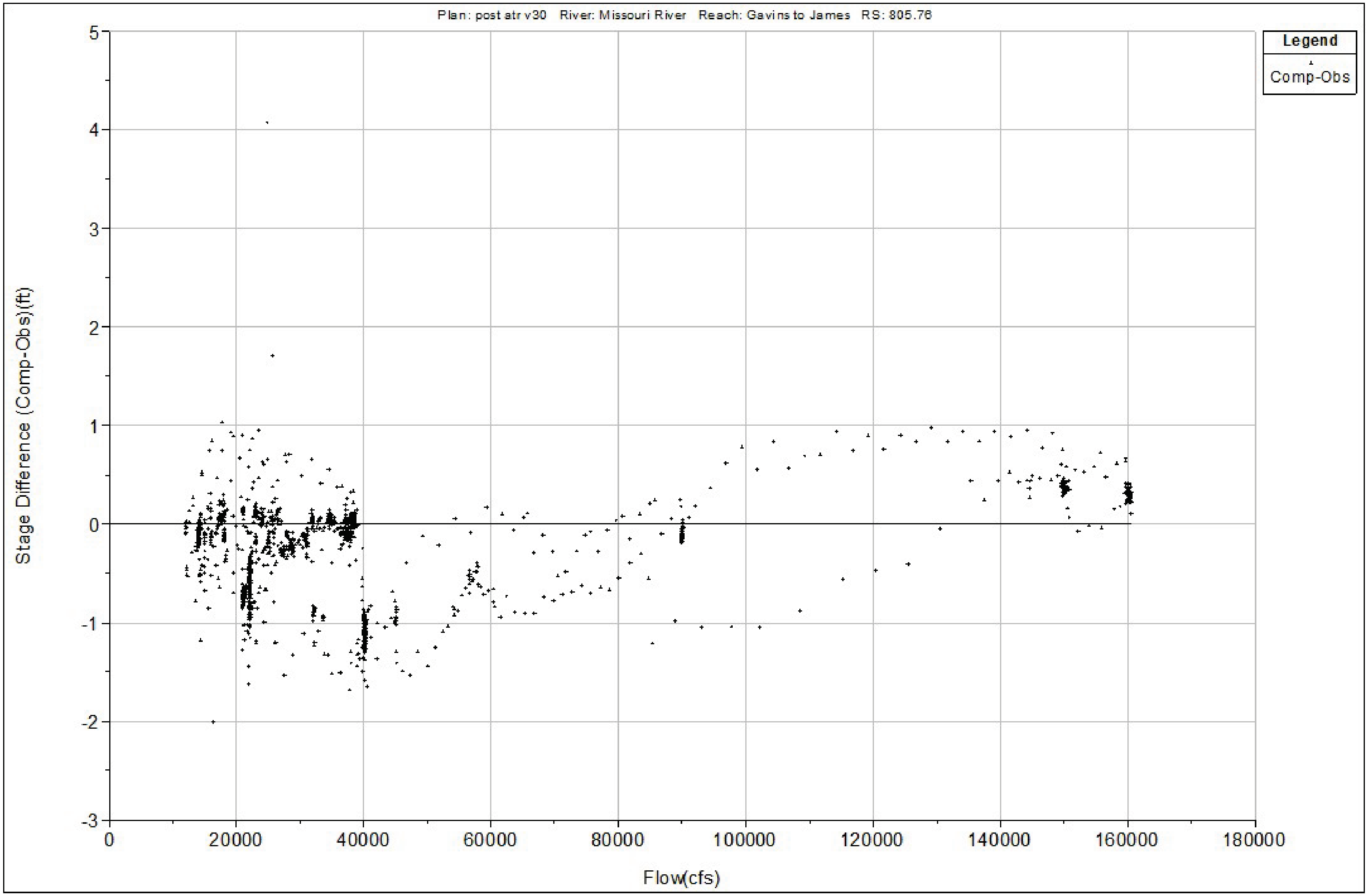


Plate 2: Missouri River at Yankton, SD Comp-Obs Stage vs Flow

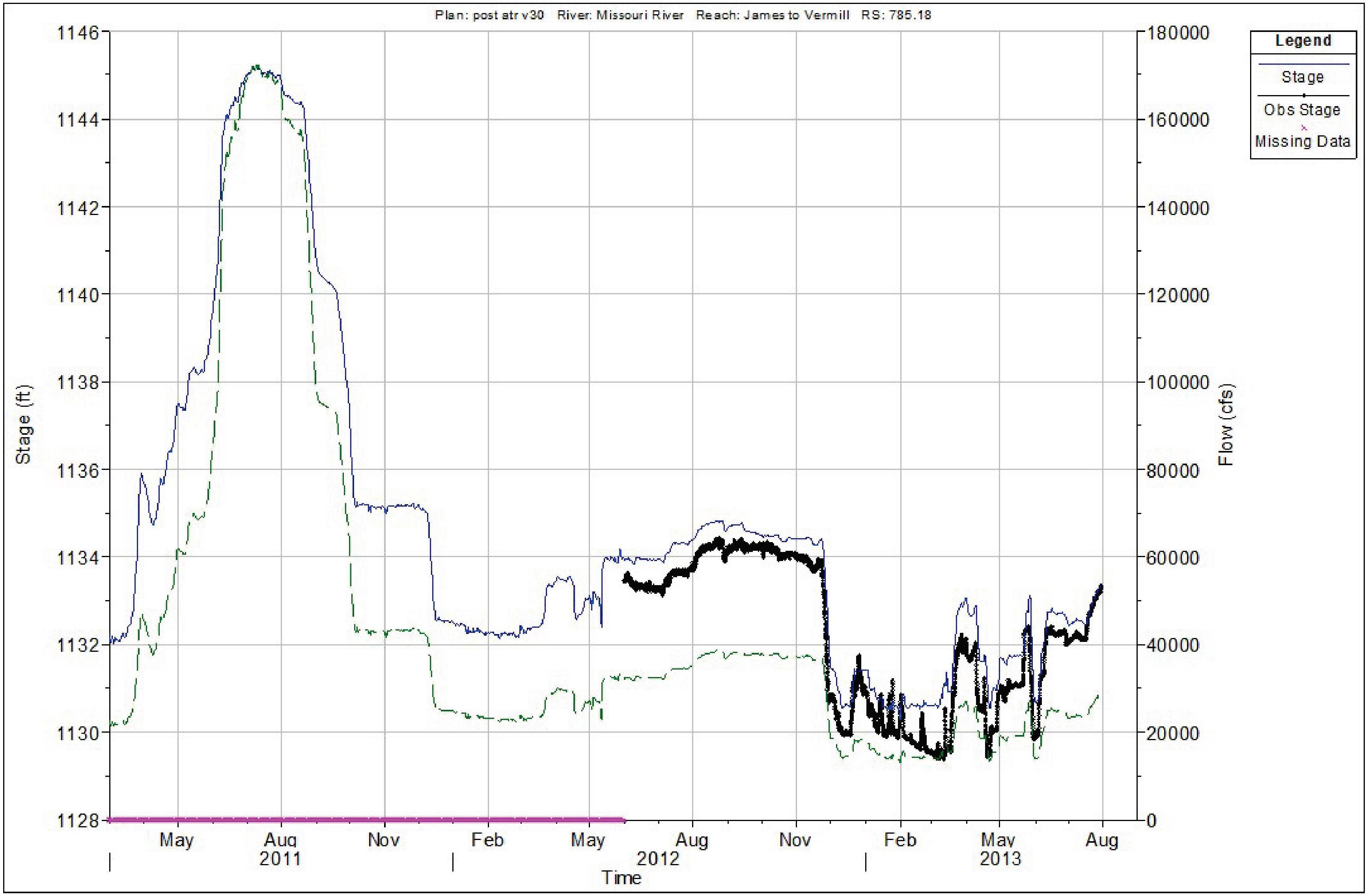


Plate 3: Missouri River near St. James, SD Hydrograph

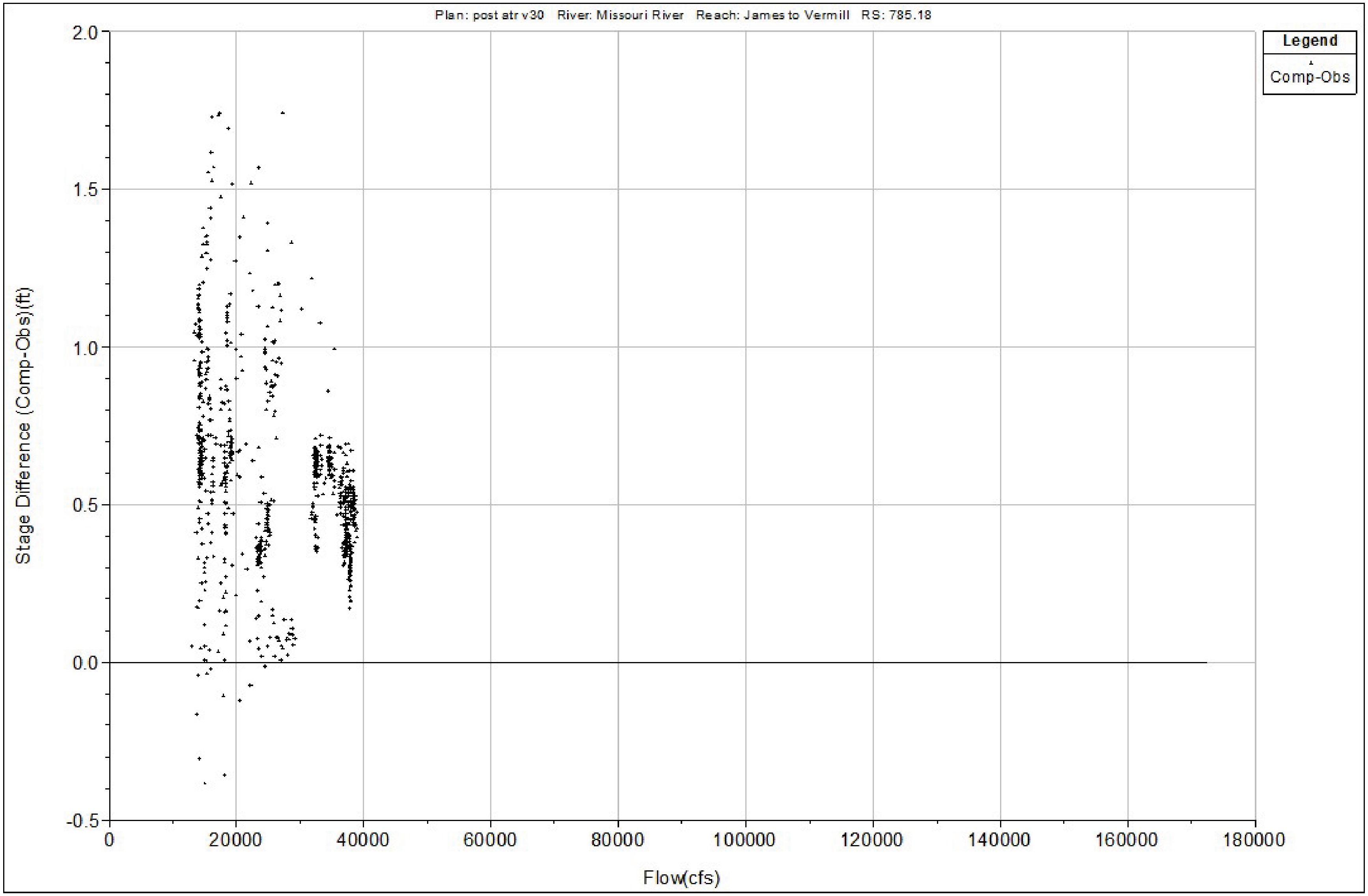


Plate 4: Missouri River near St. James, SD Comp-Obs Stage vs Flow

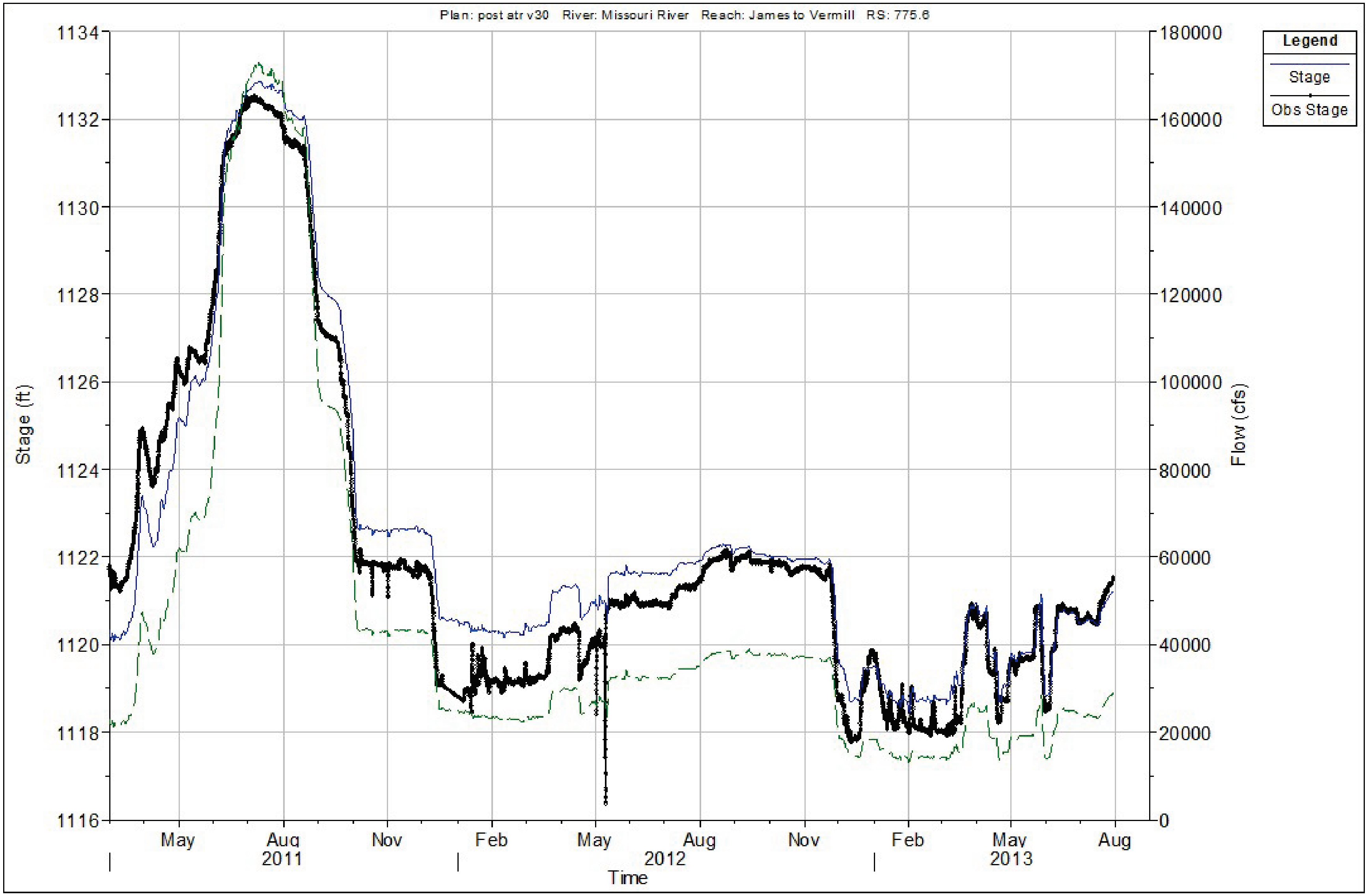


Plate 5: Missouri River near Maskell, NE Hydrograph

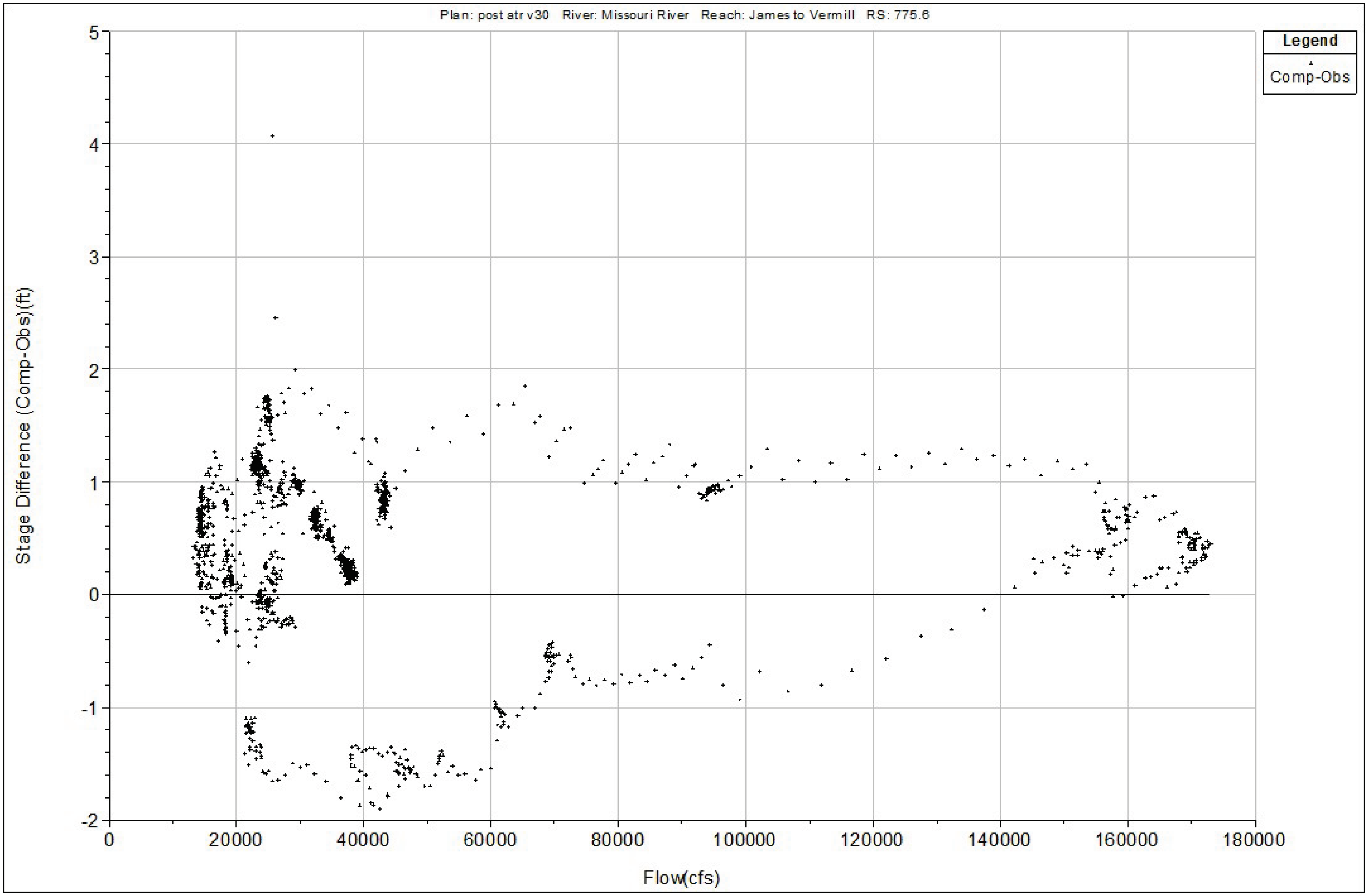


Plate 6: Missouri River near Maskell, NE Comp-Obs Stage vs Flow

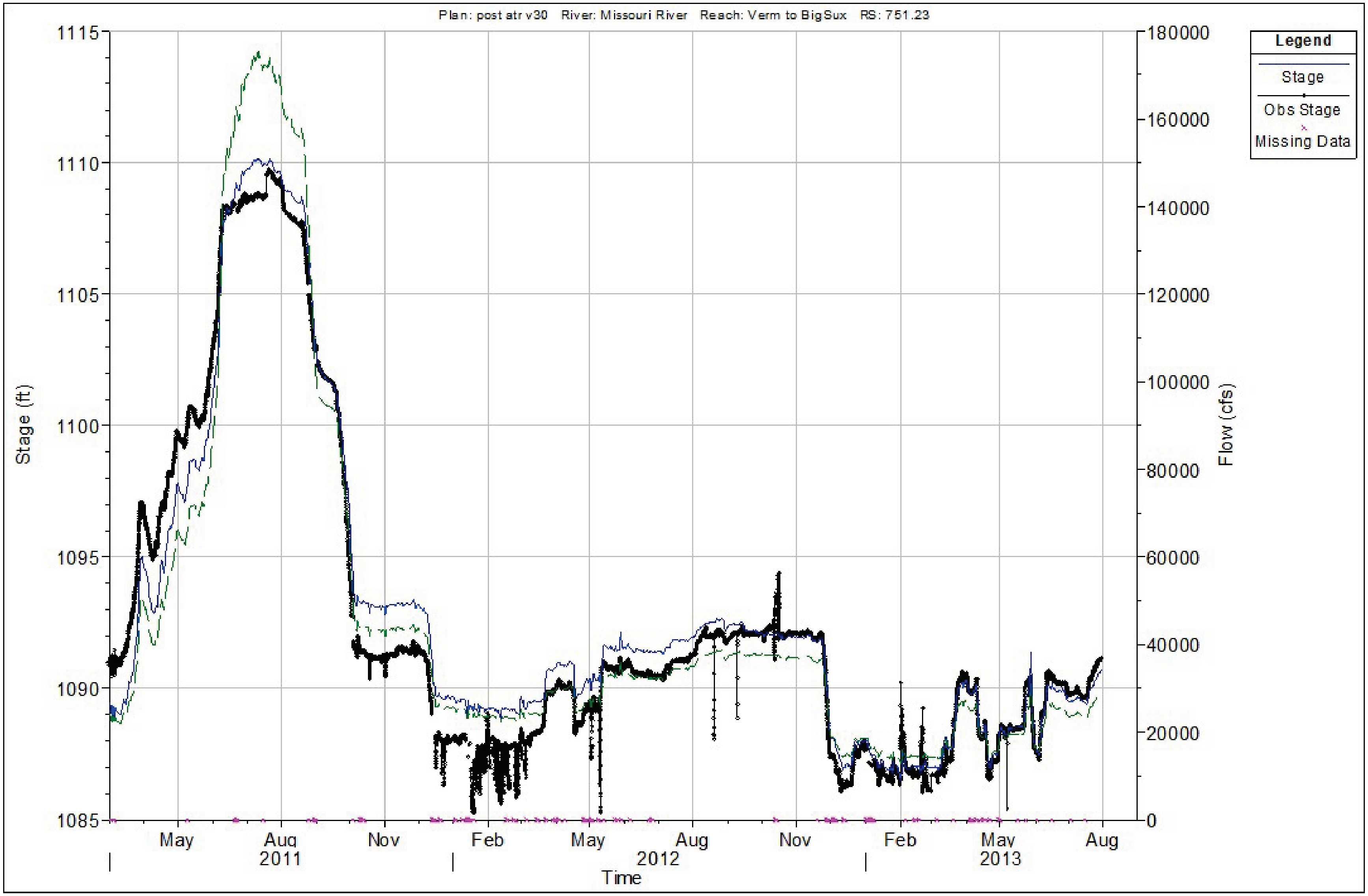


Plate 7: Missouri River at Ponca, NE Hydrograph

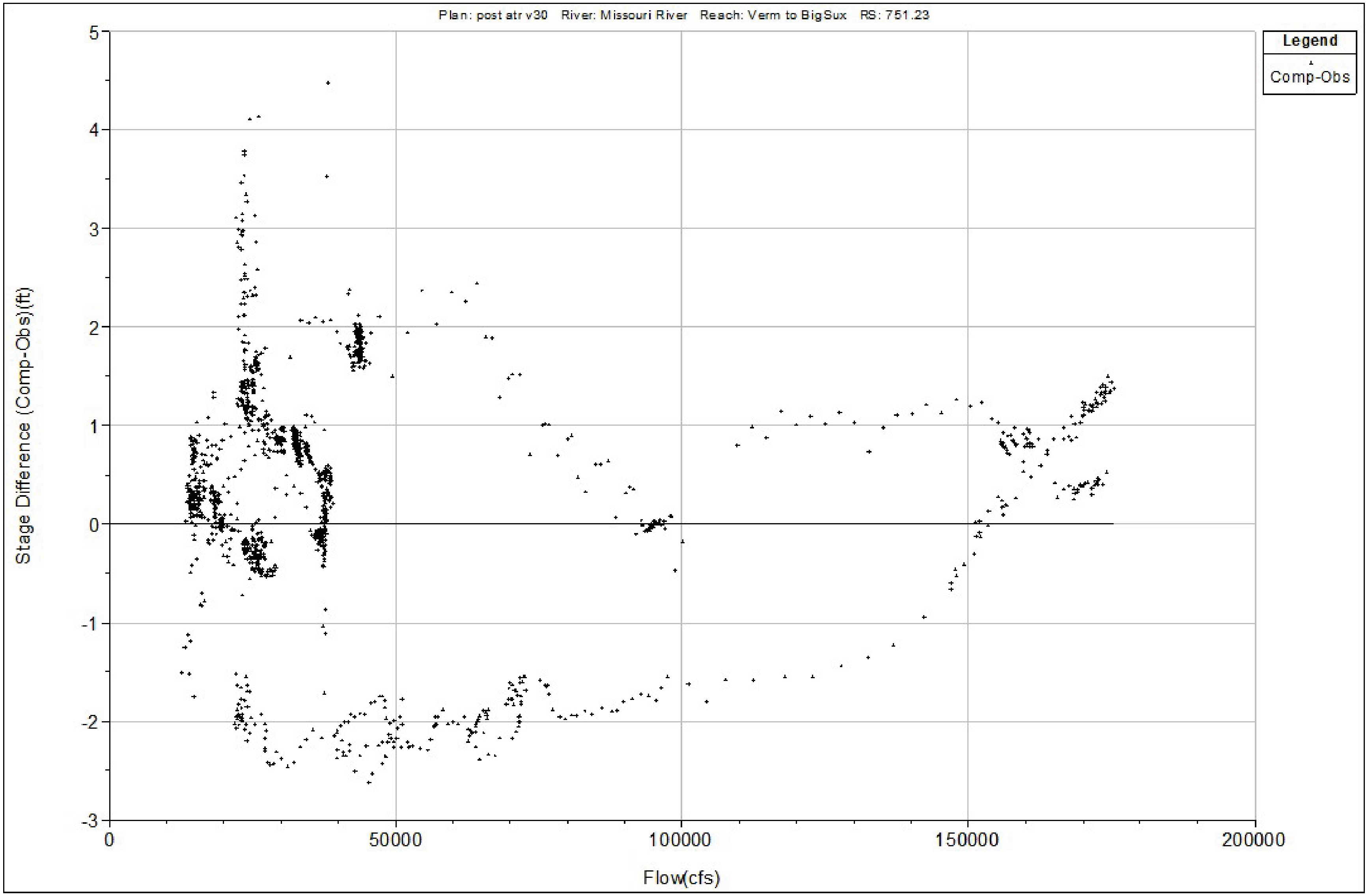


Plate 8: Missouri River at Ponca, NE Comp-Obs Stage vs Flow

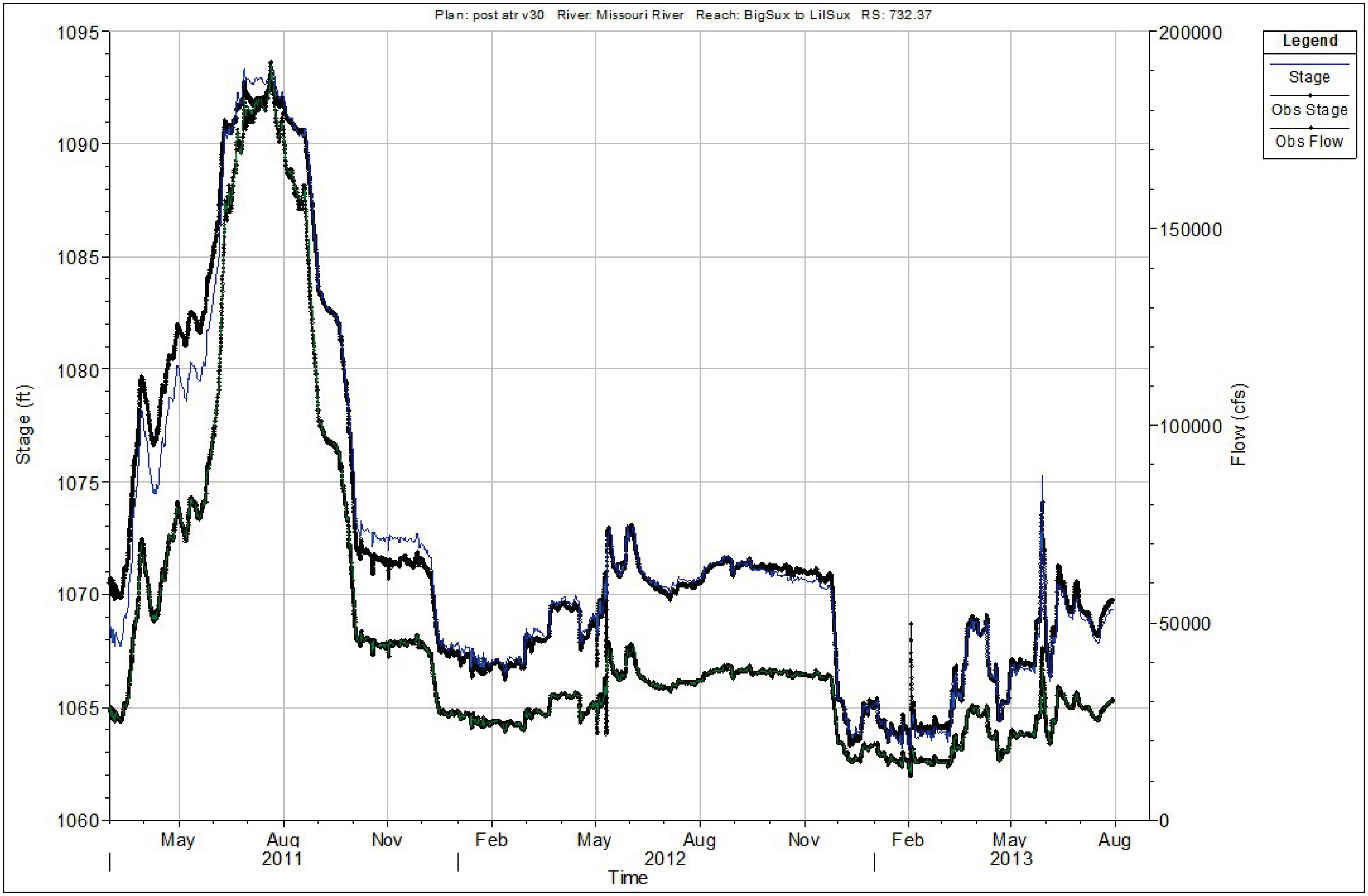


Plate 9: Missouri River at Sioux City, IA Hydrograph

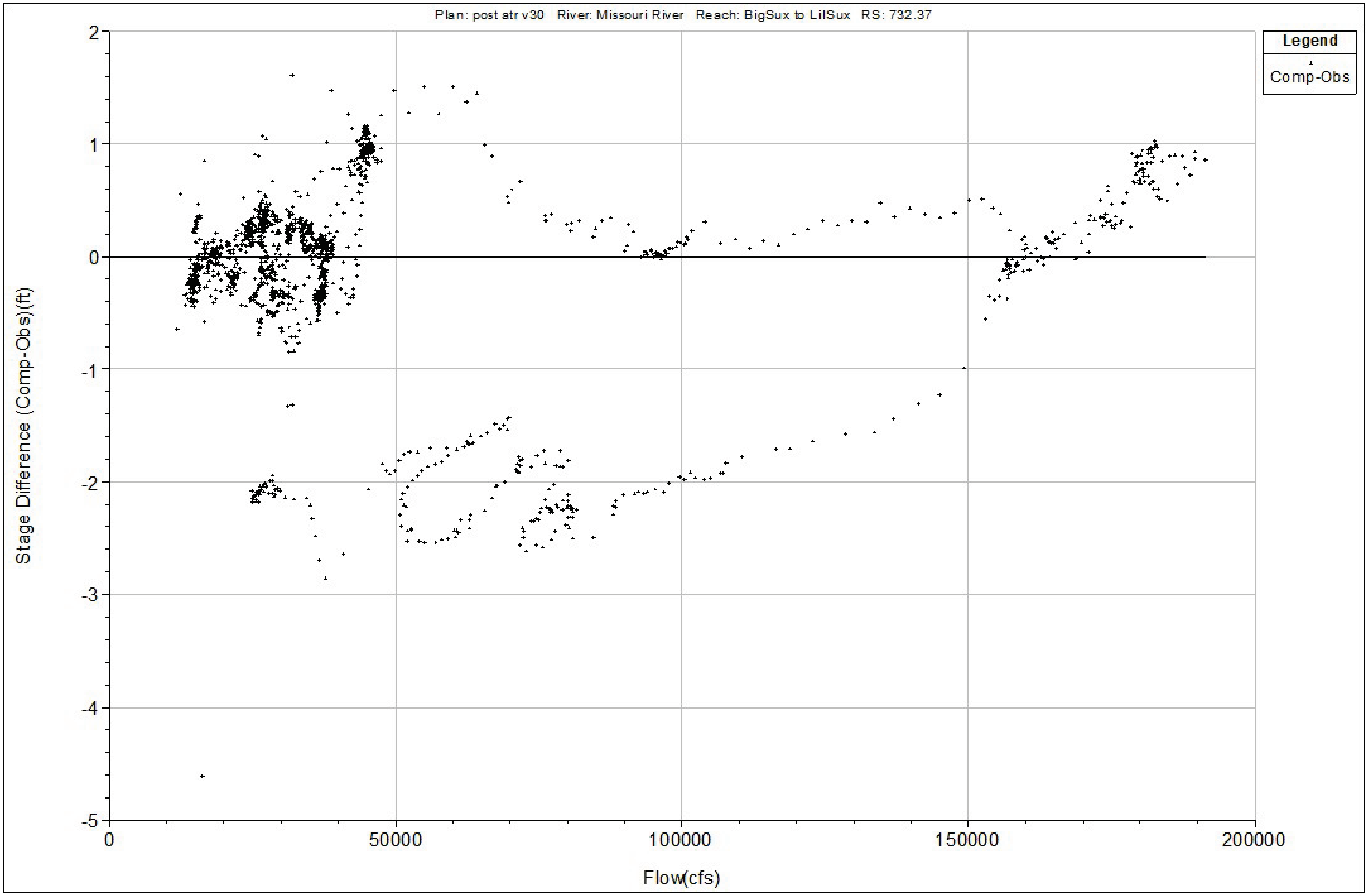


Plate 10: Missouri River at Sioux City, IA Comp-Obs Stage vs Flow

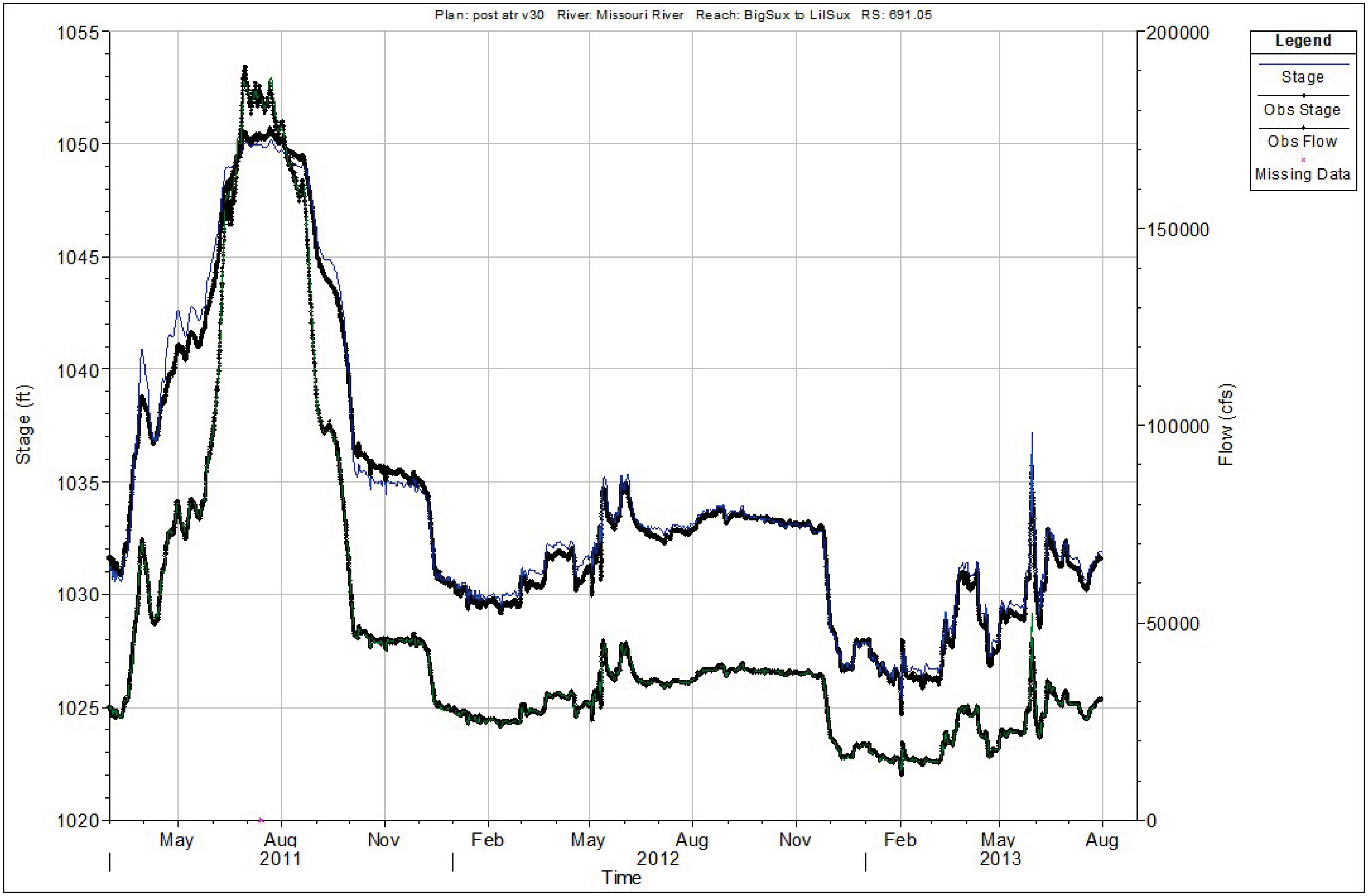


Plate 11: Missouri River at Decatur, NE Hydrograph

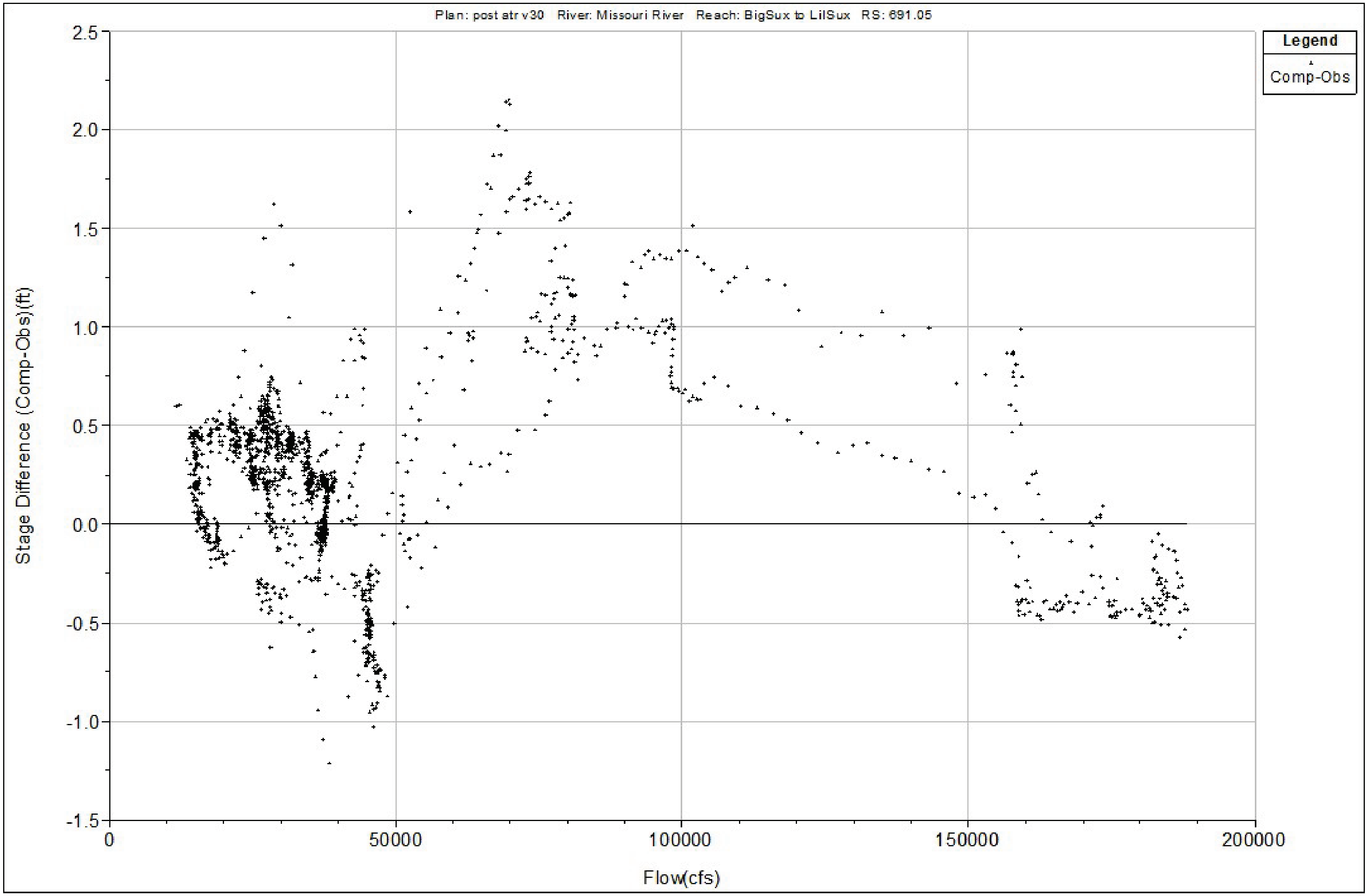


Plate 12: Missouri River at Decatur, NE Comp-Obs Stage vs Flow

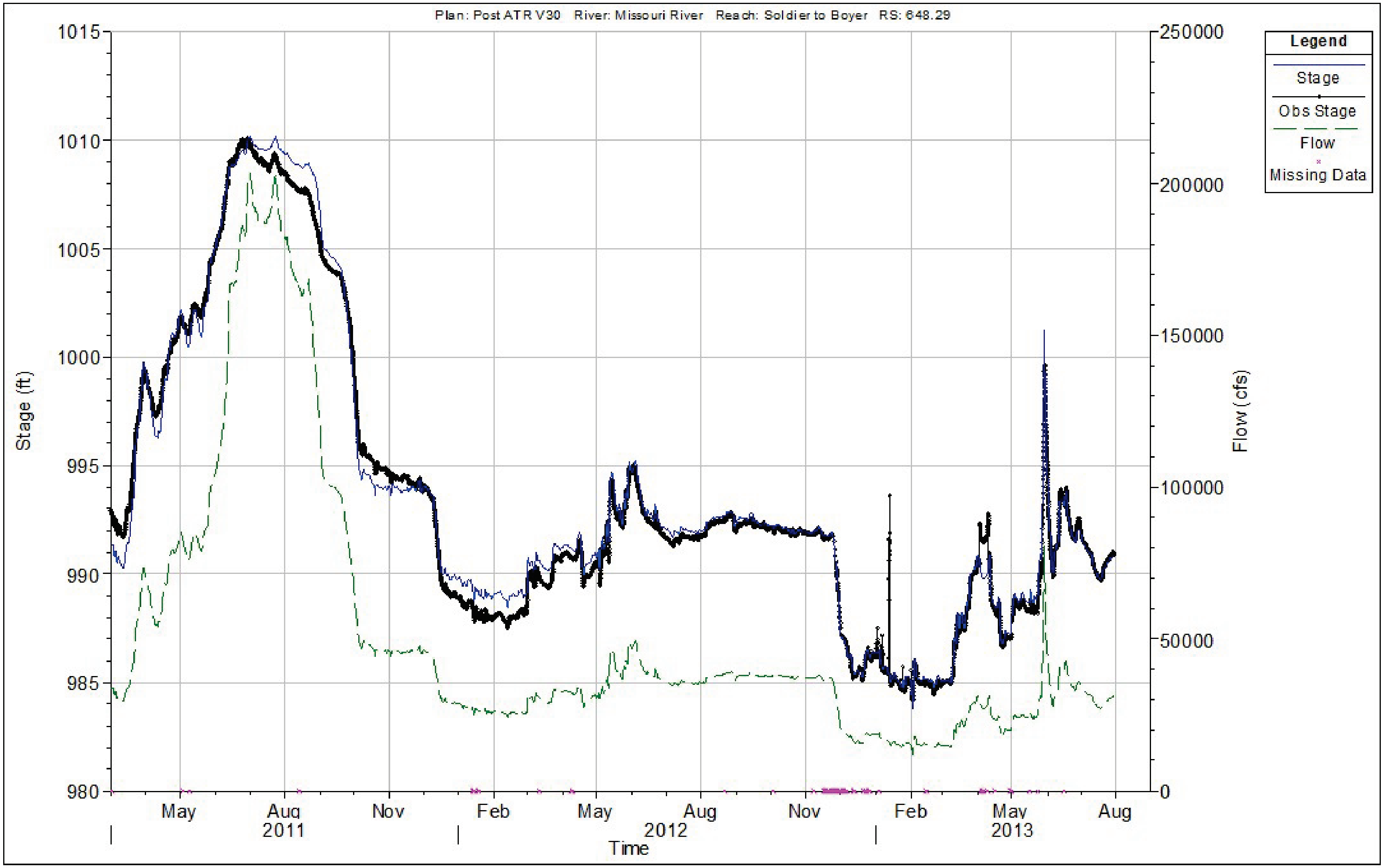


Plate 13: Missouri River at Blair, NE Hydrograph

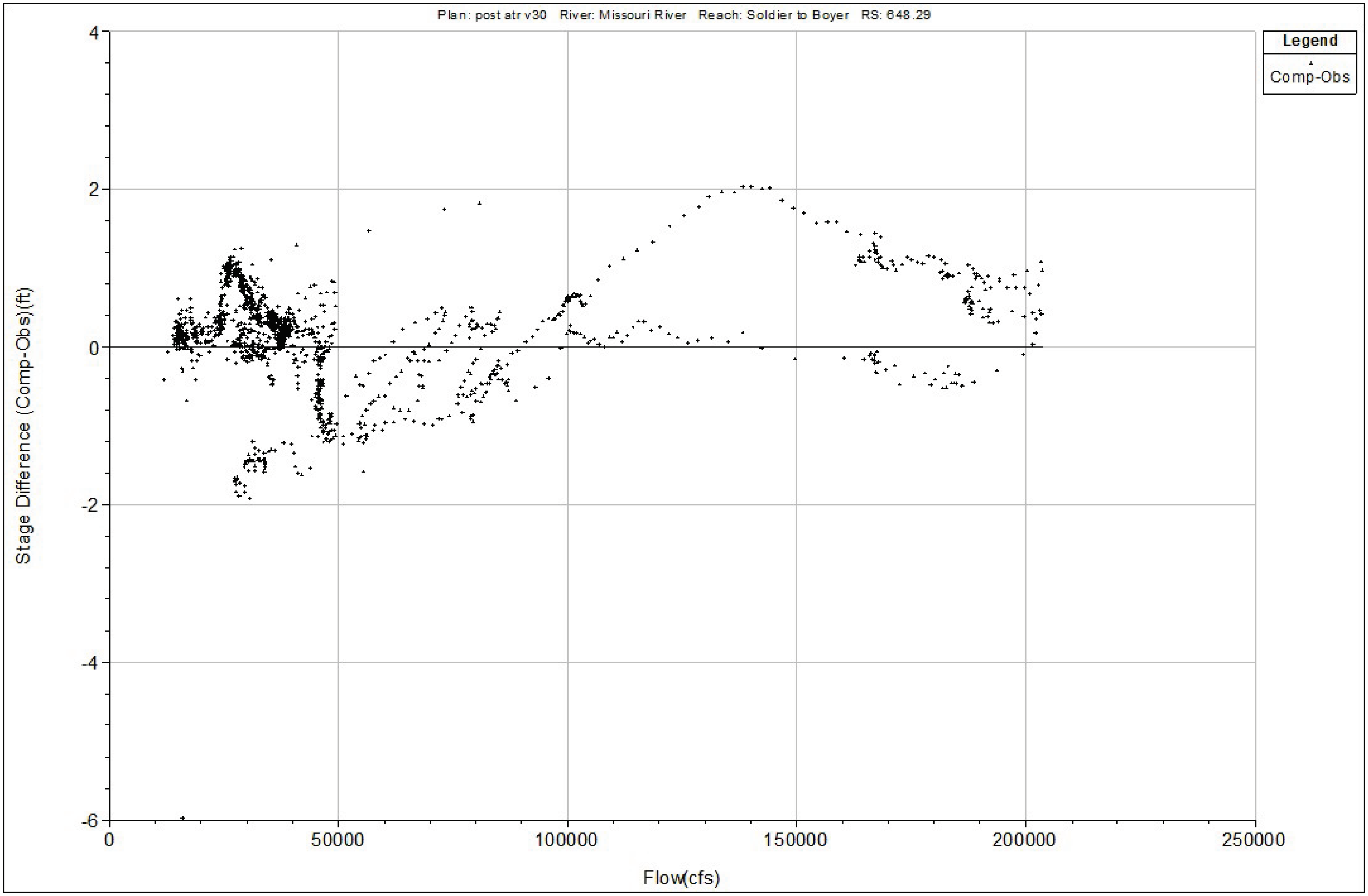


Plate 14: Missouri River at Blair, NE Comp-Obs Stage vs Flow

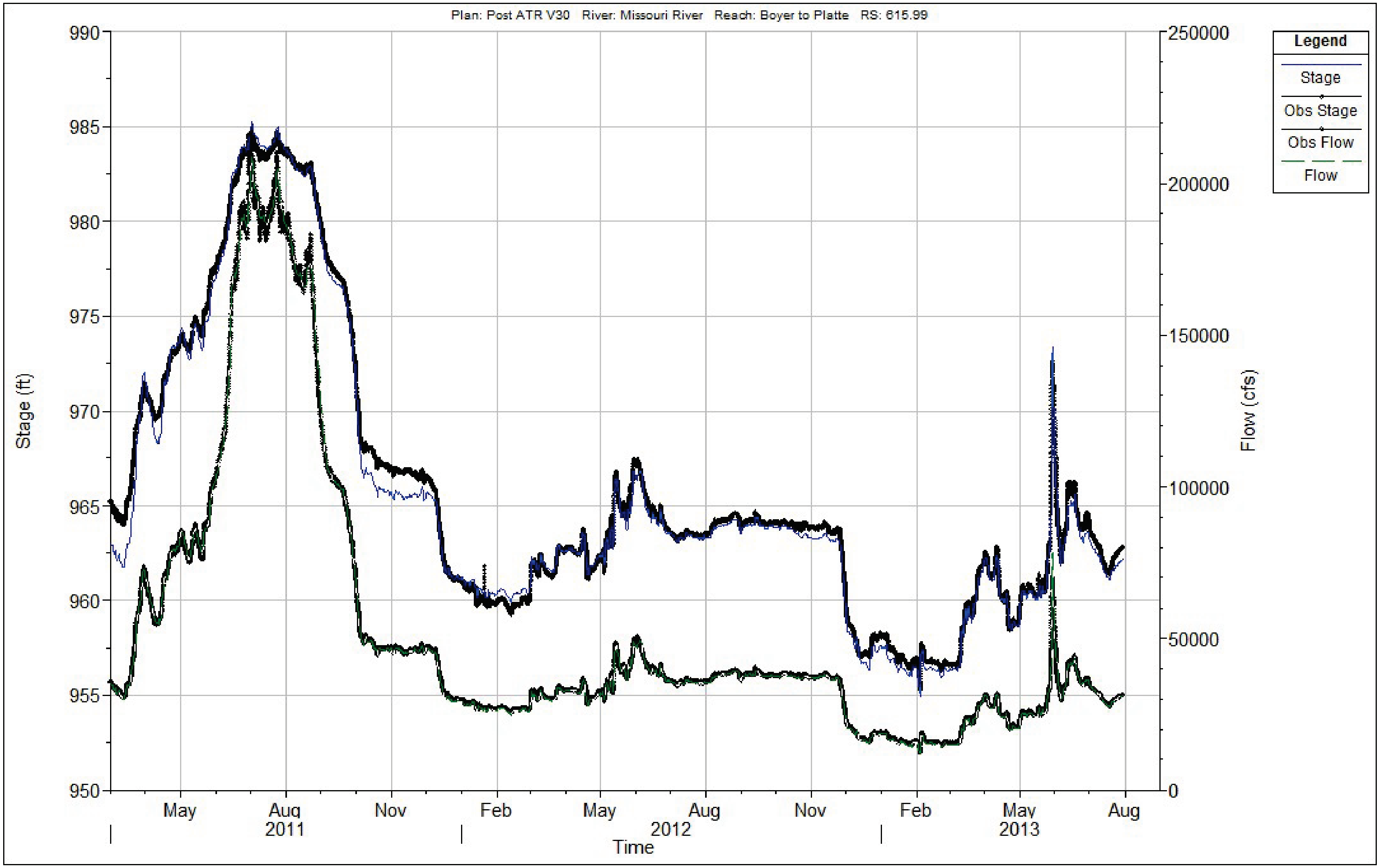


Plate 15: Missouri River at Omaha, NE Hydrograph

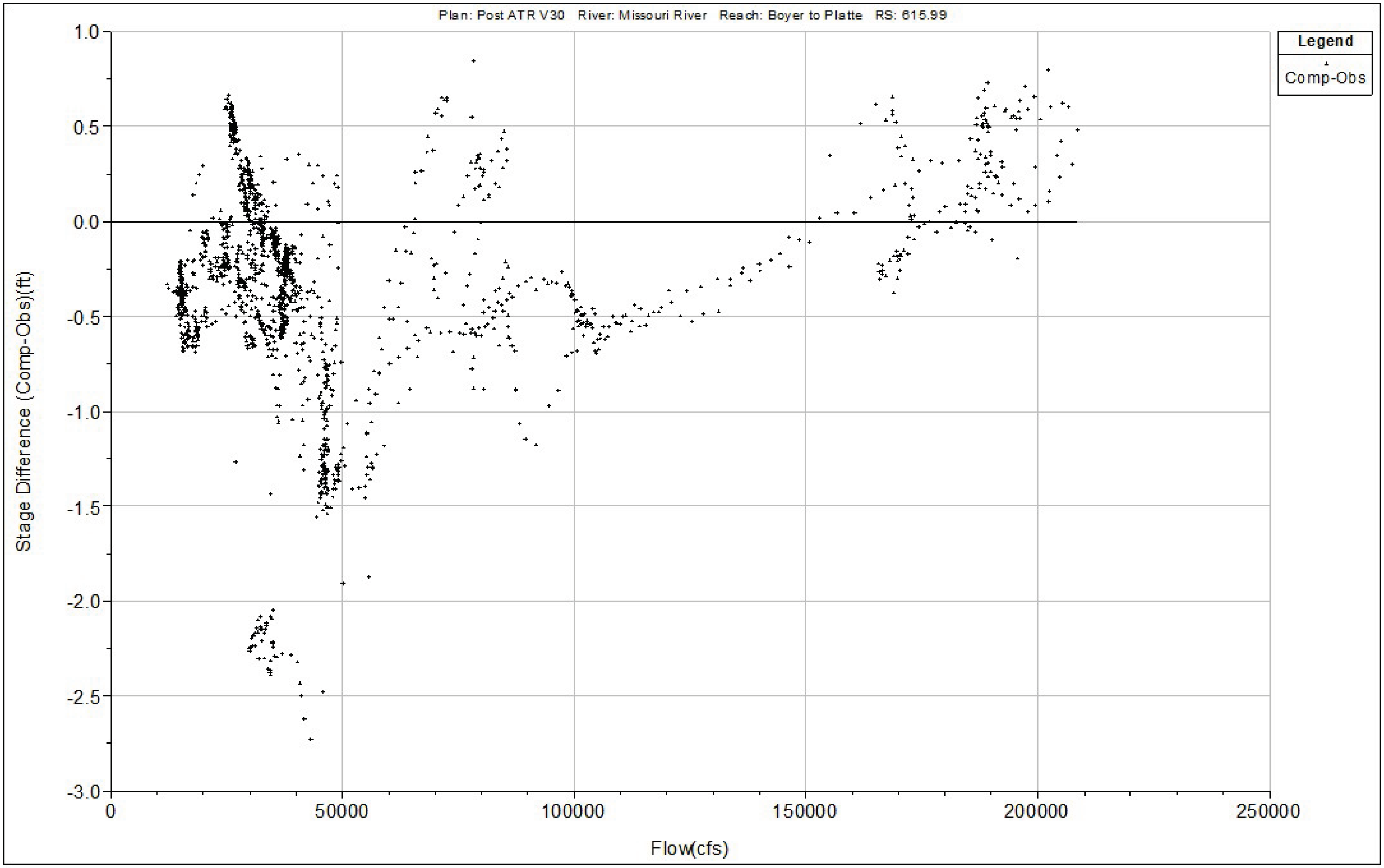


Plate 16: Missouri River at Omaha, NE Comp-Obs Stage vs Flow

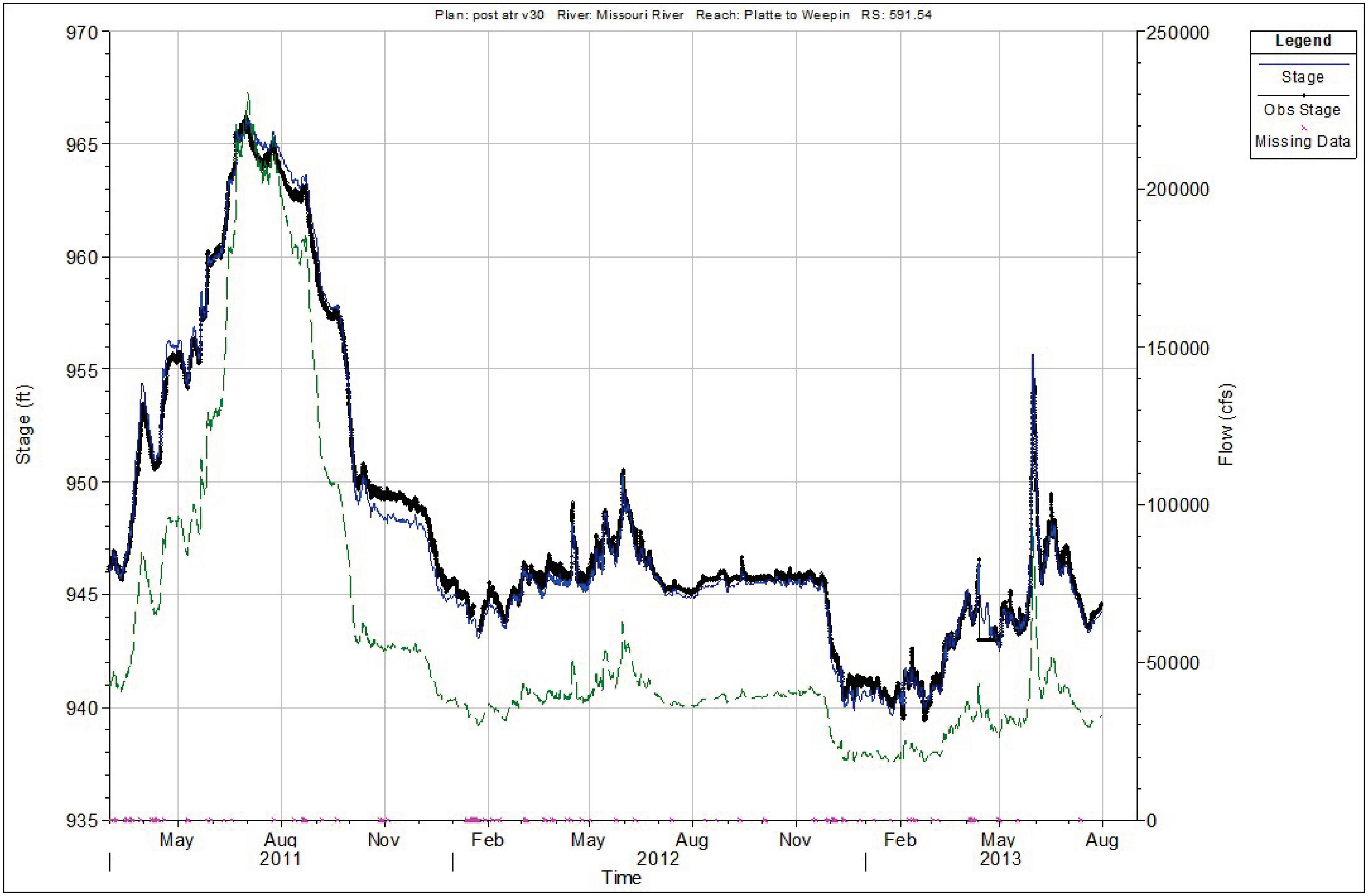


Plate 17: Missouri River at Plattsmouth, NE Hydrograph

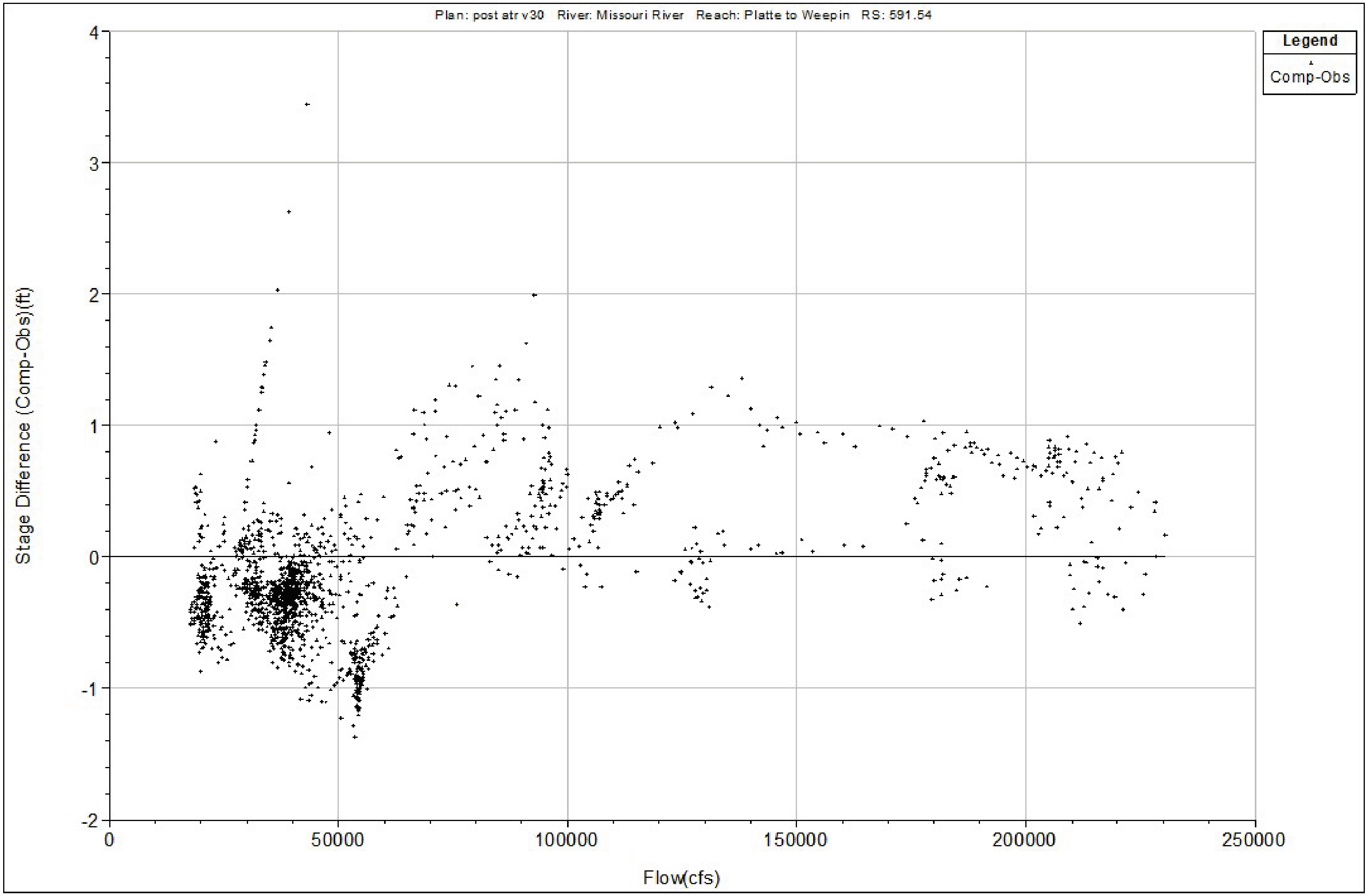


Plate 18: Missouri River at Plattsmouth, NE Comp-Obs Stage vs Flow

Plate 19: Missouri River at Nebraska City, NE Hydrograph

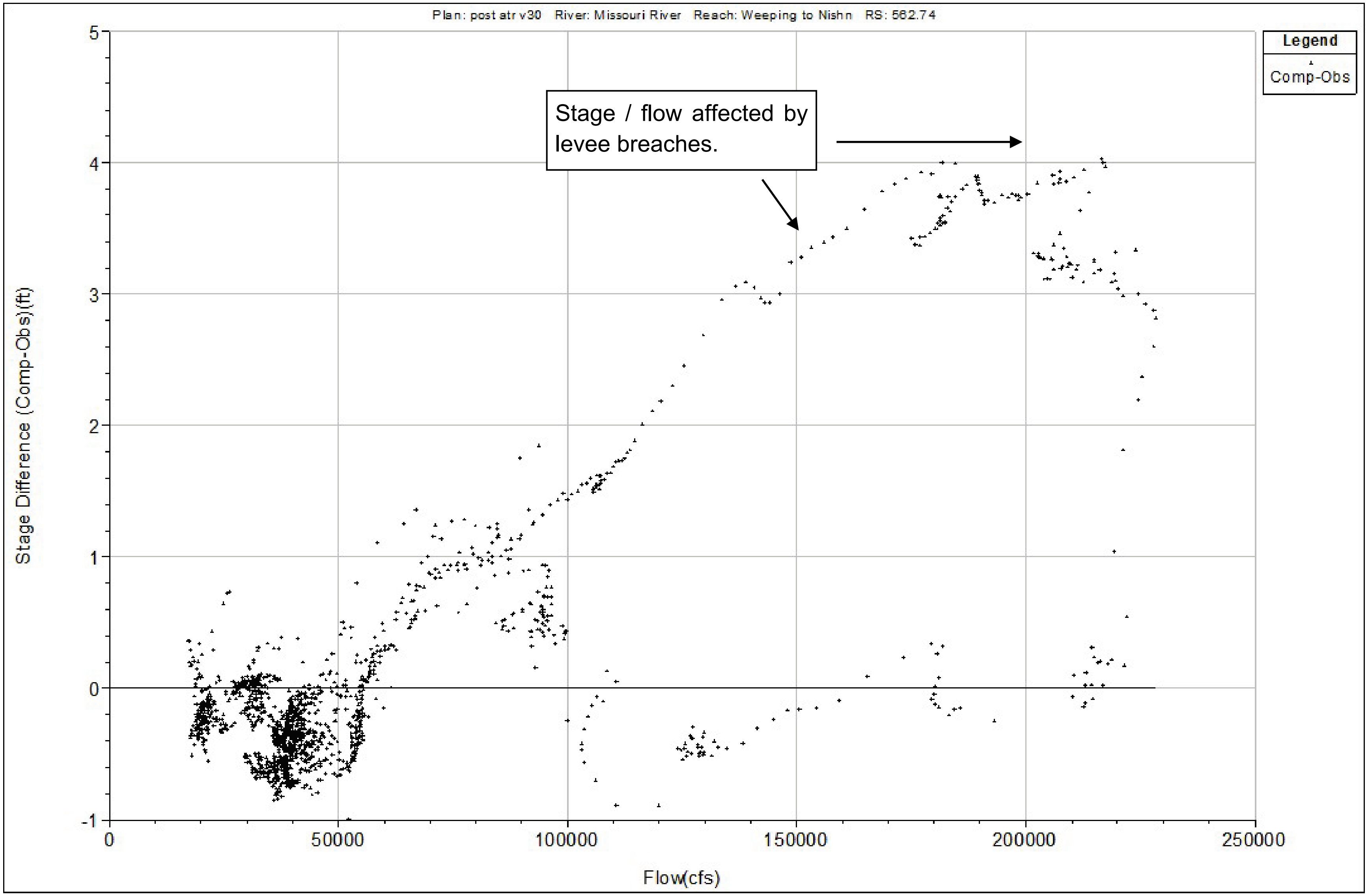
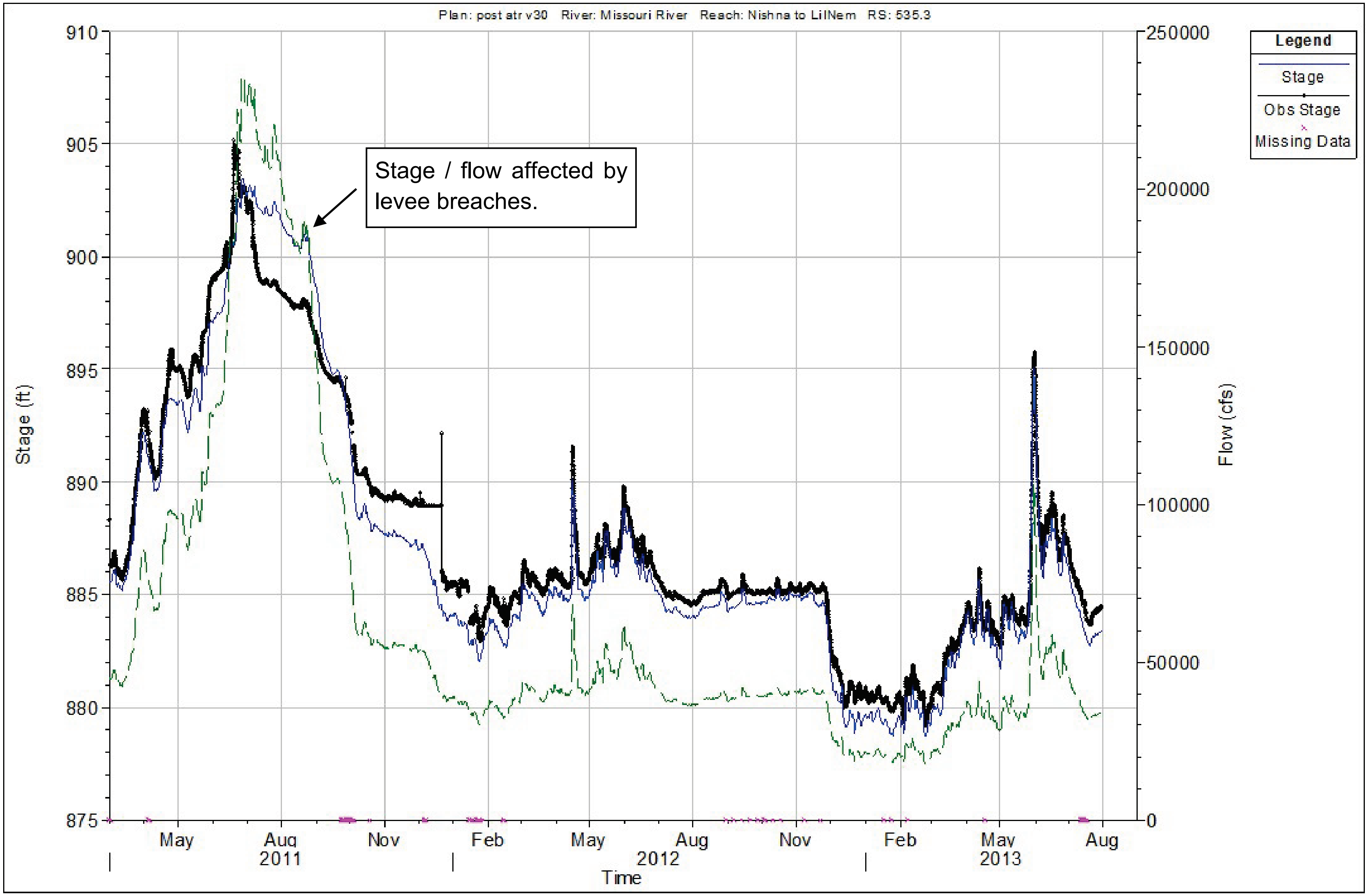


Plate 20: Missouri River at Nebraska City, NE Comp-Obs Stage vs Flow



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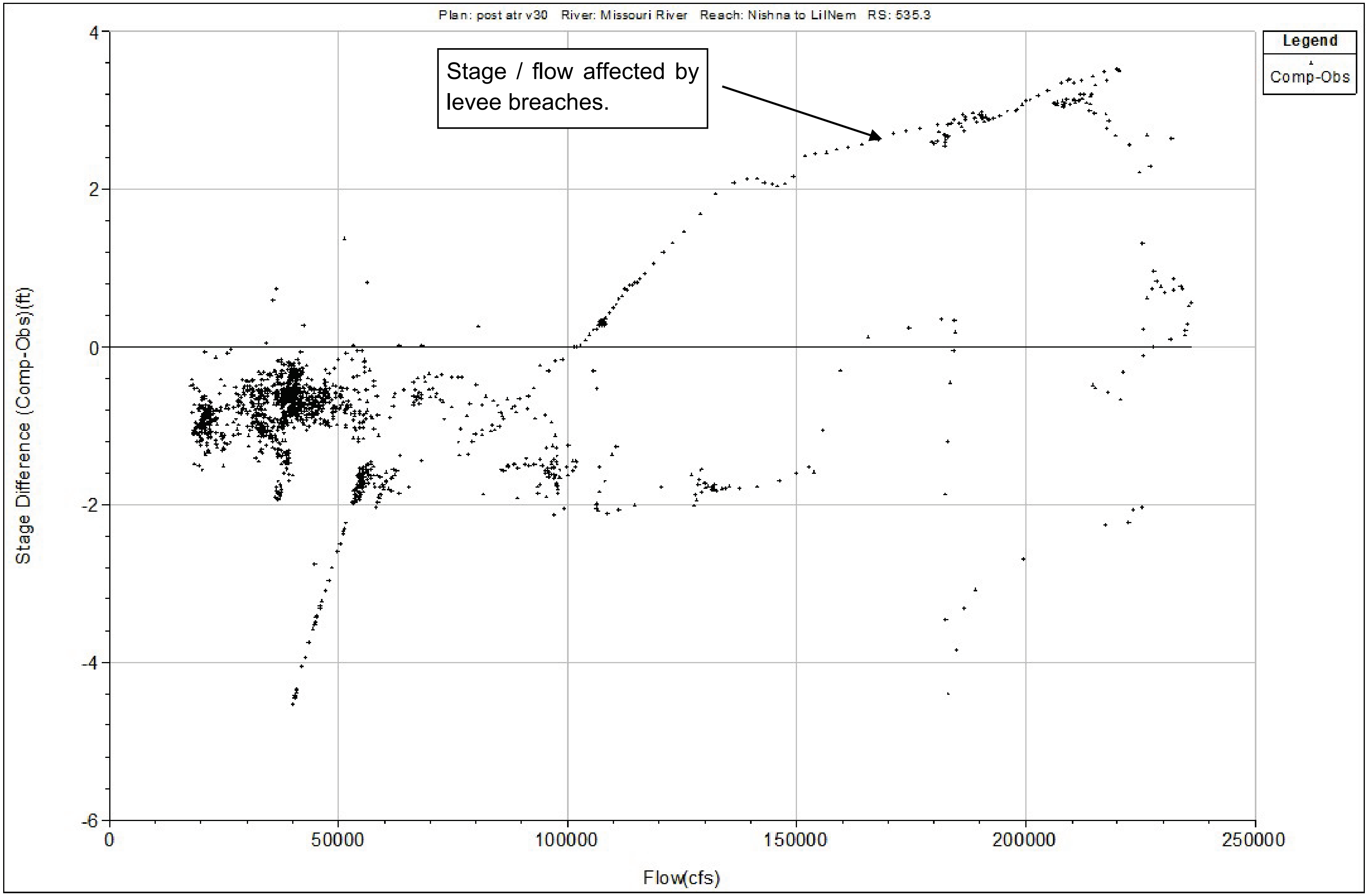

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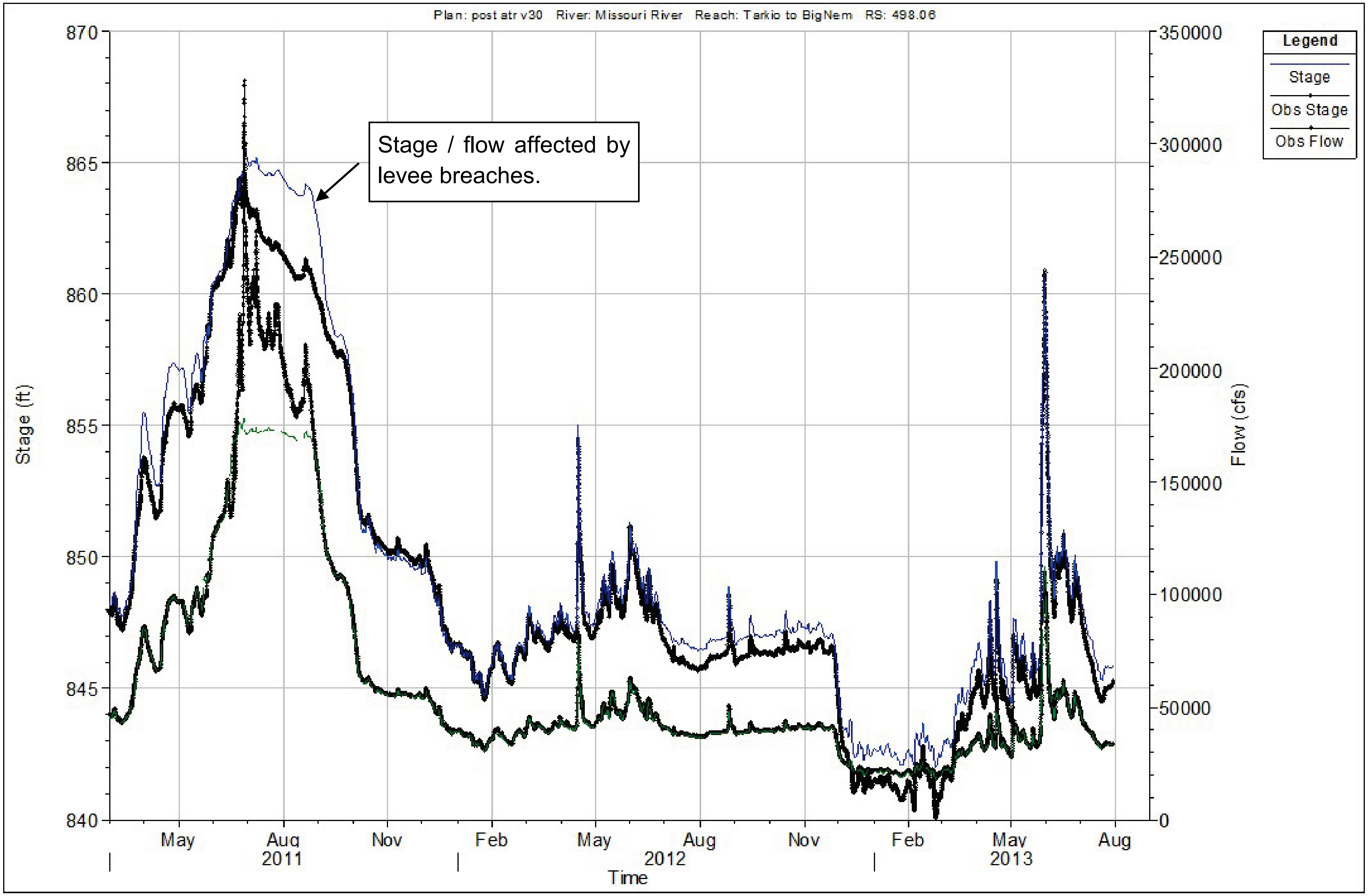

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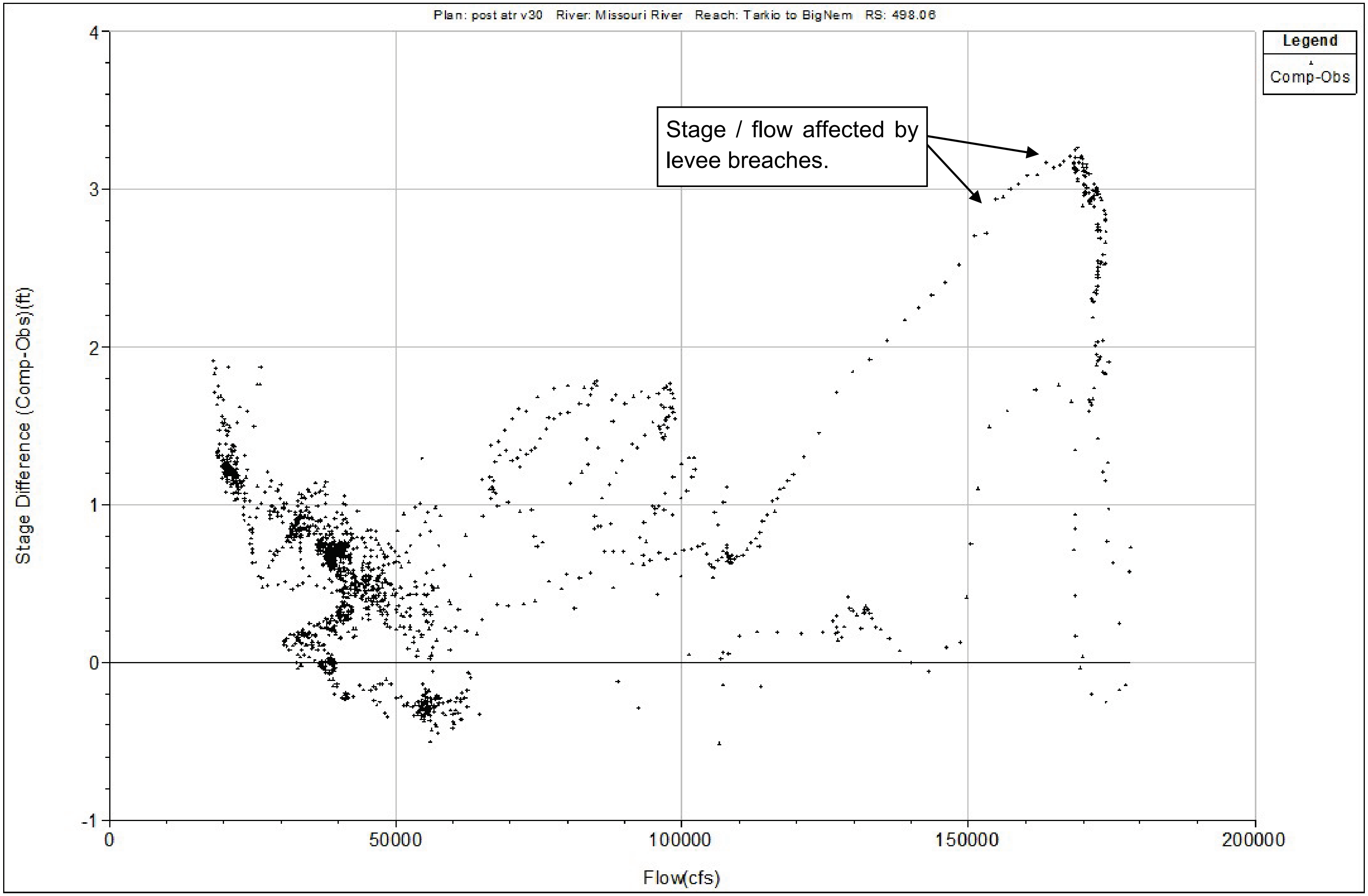

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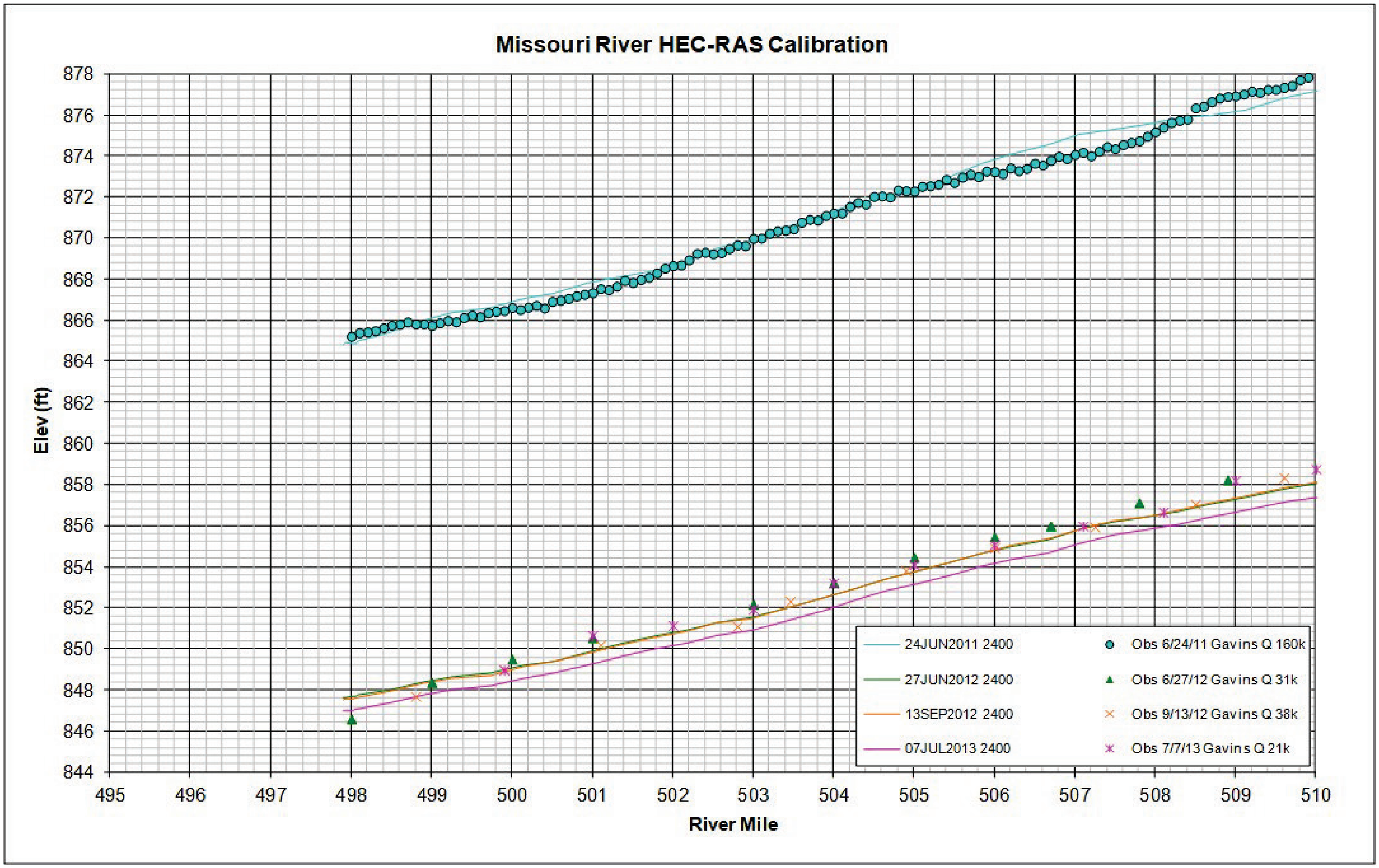

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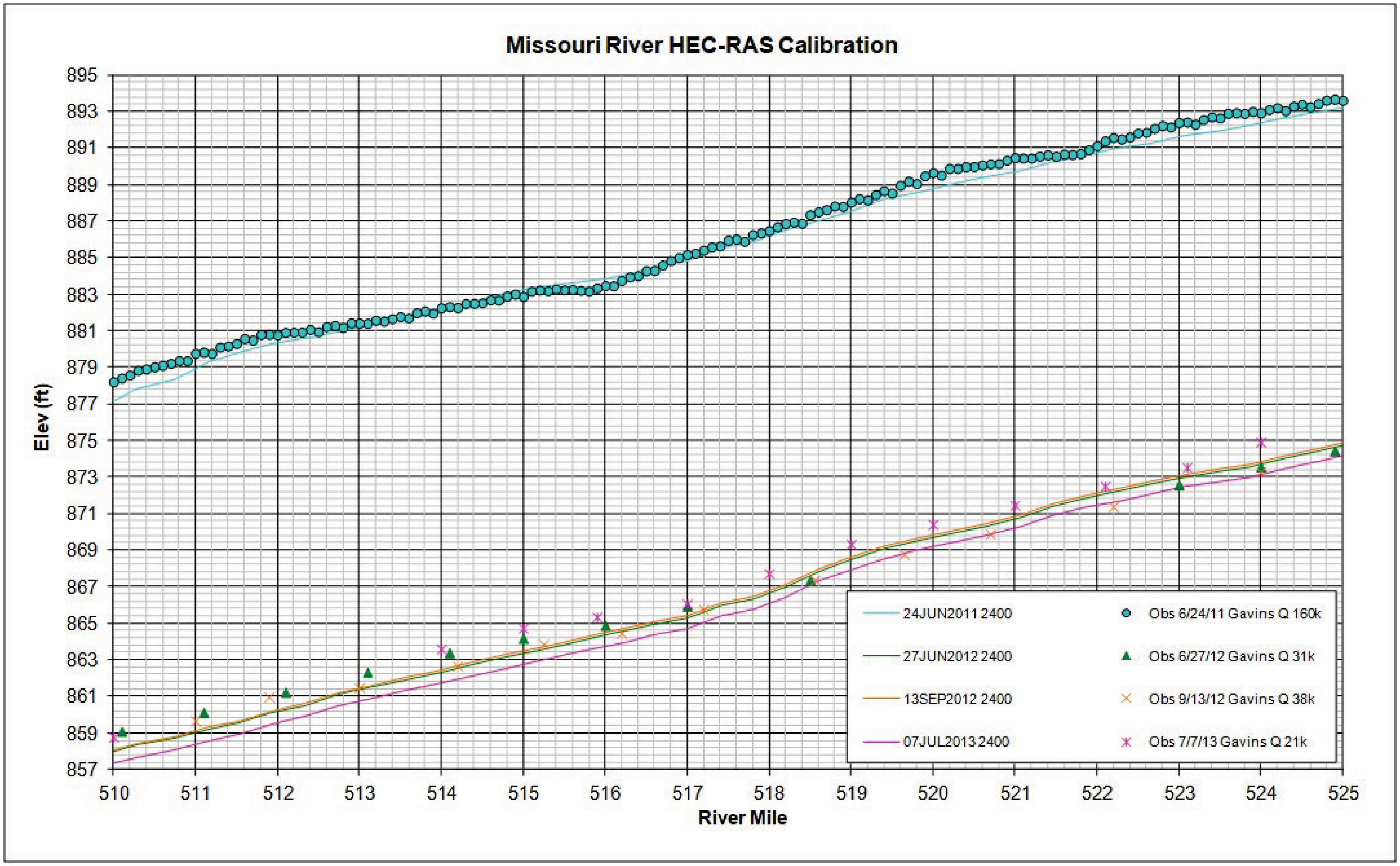

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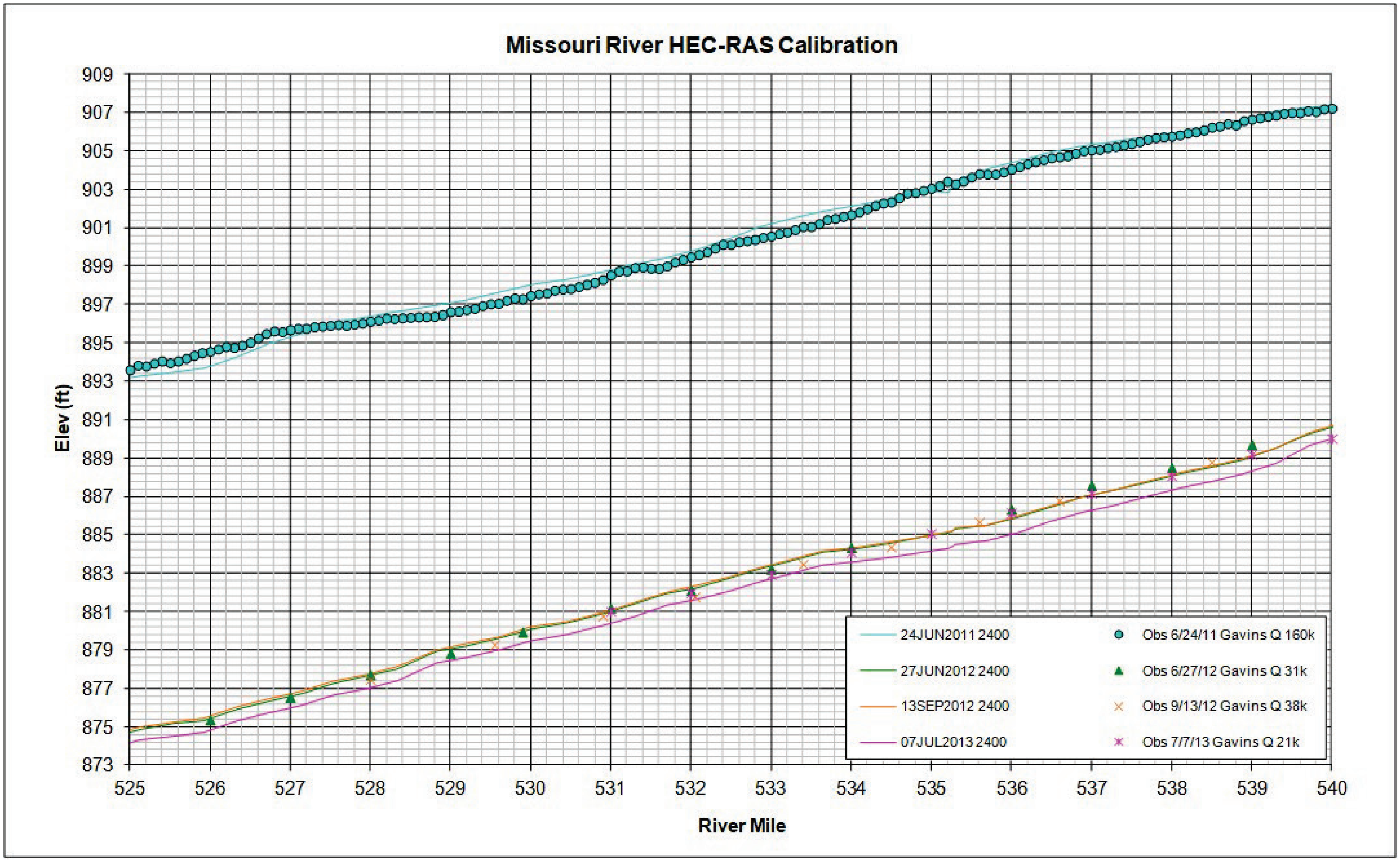

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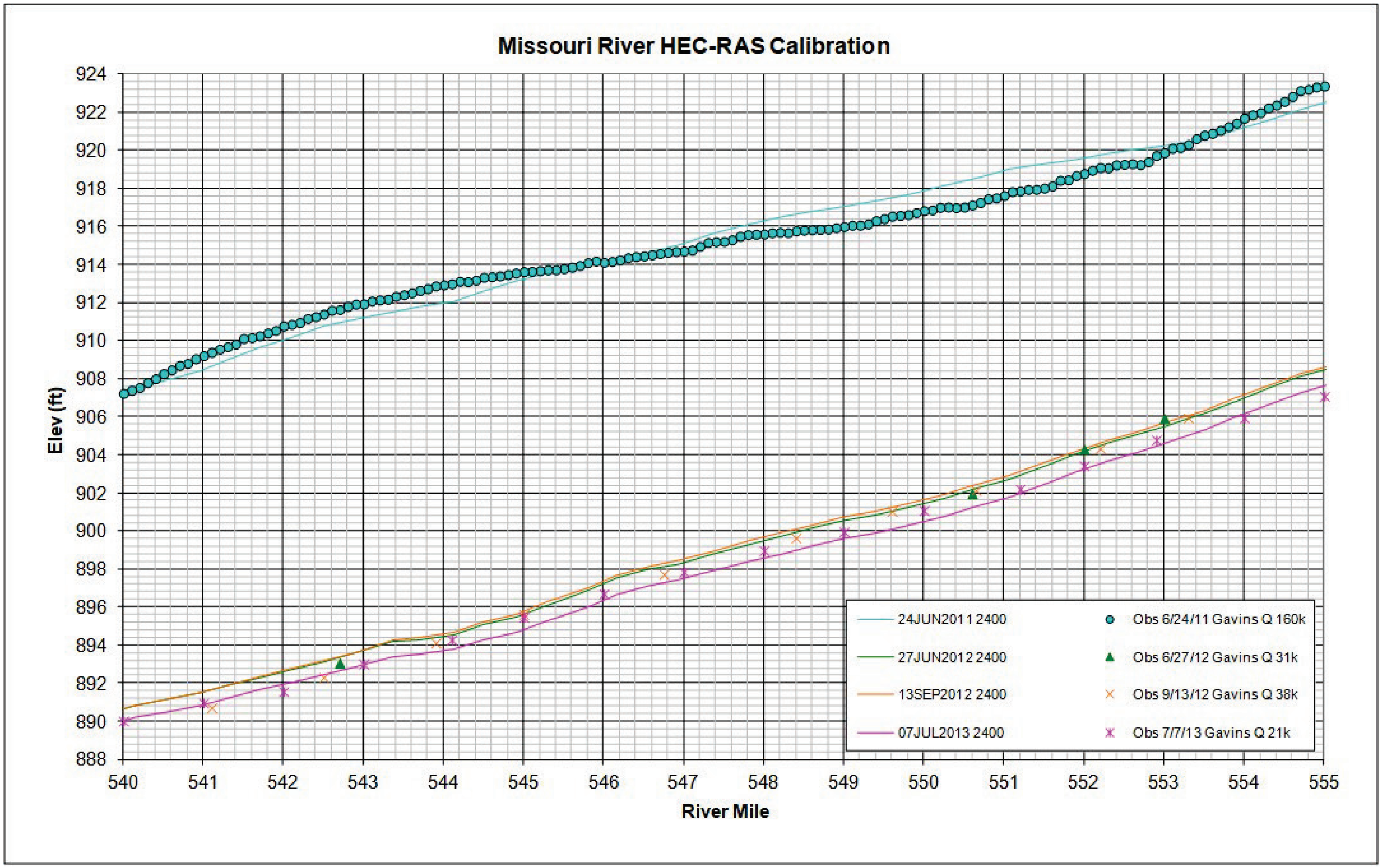

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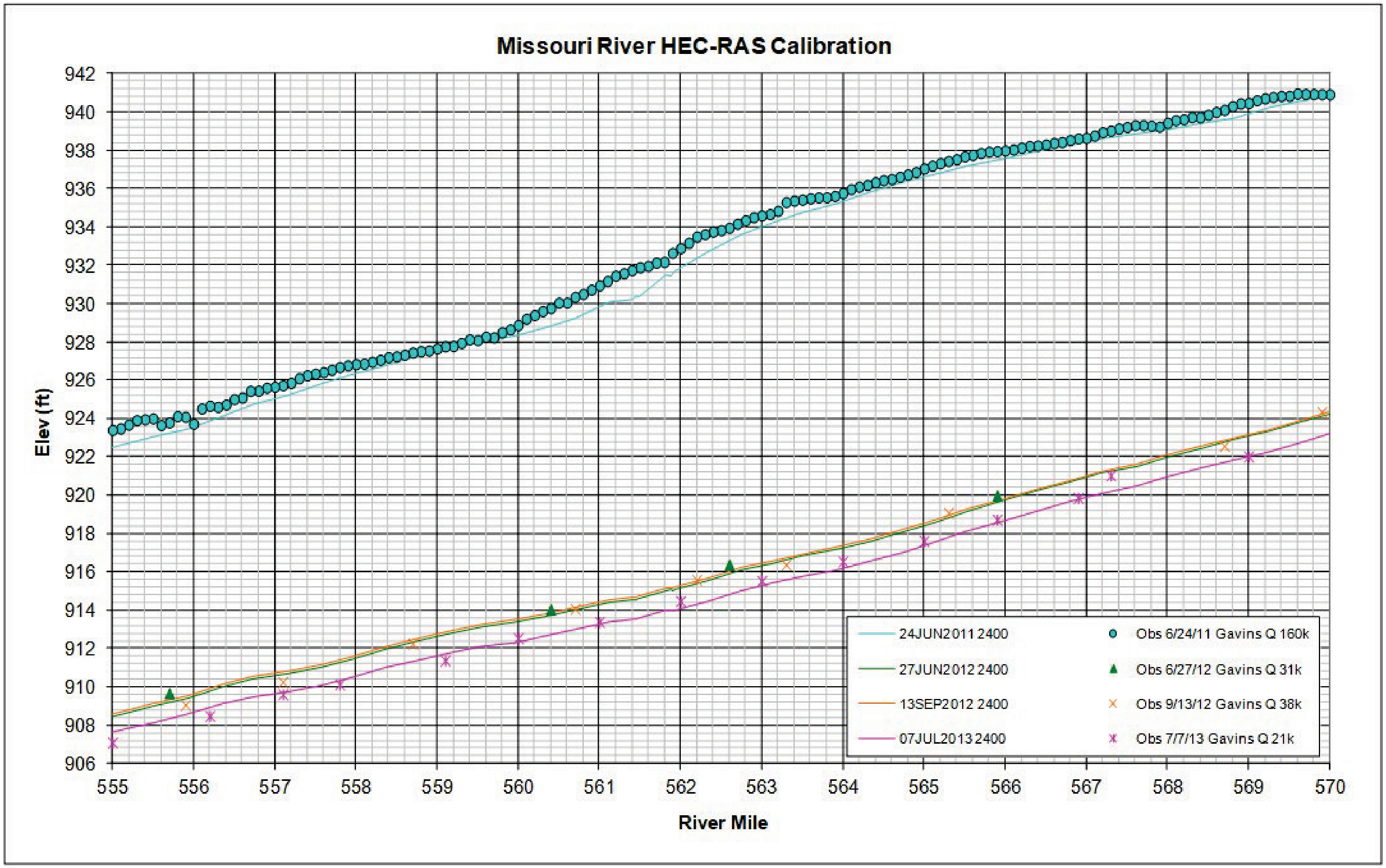

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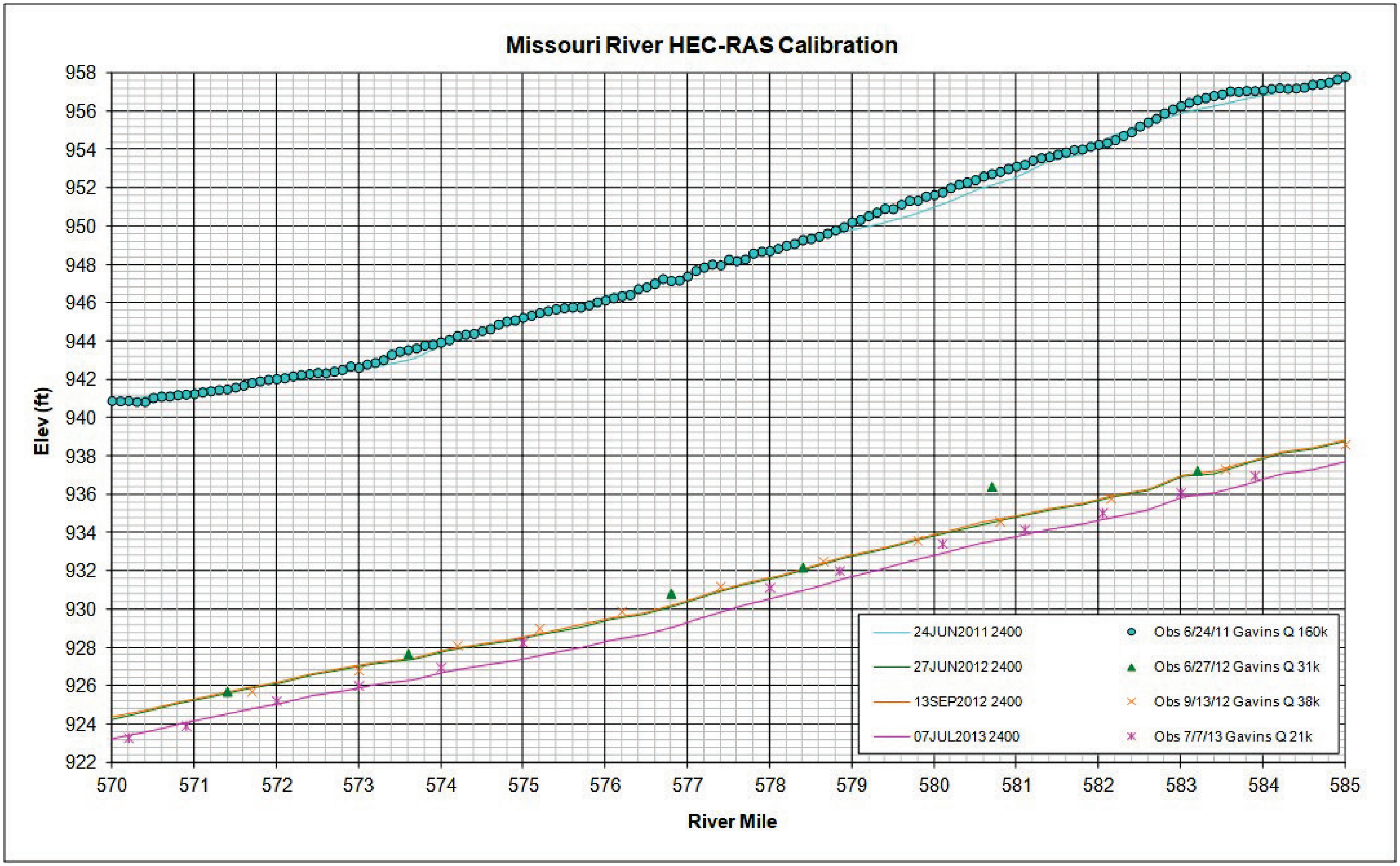

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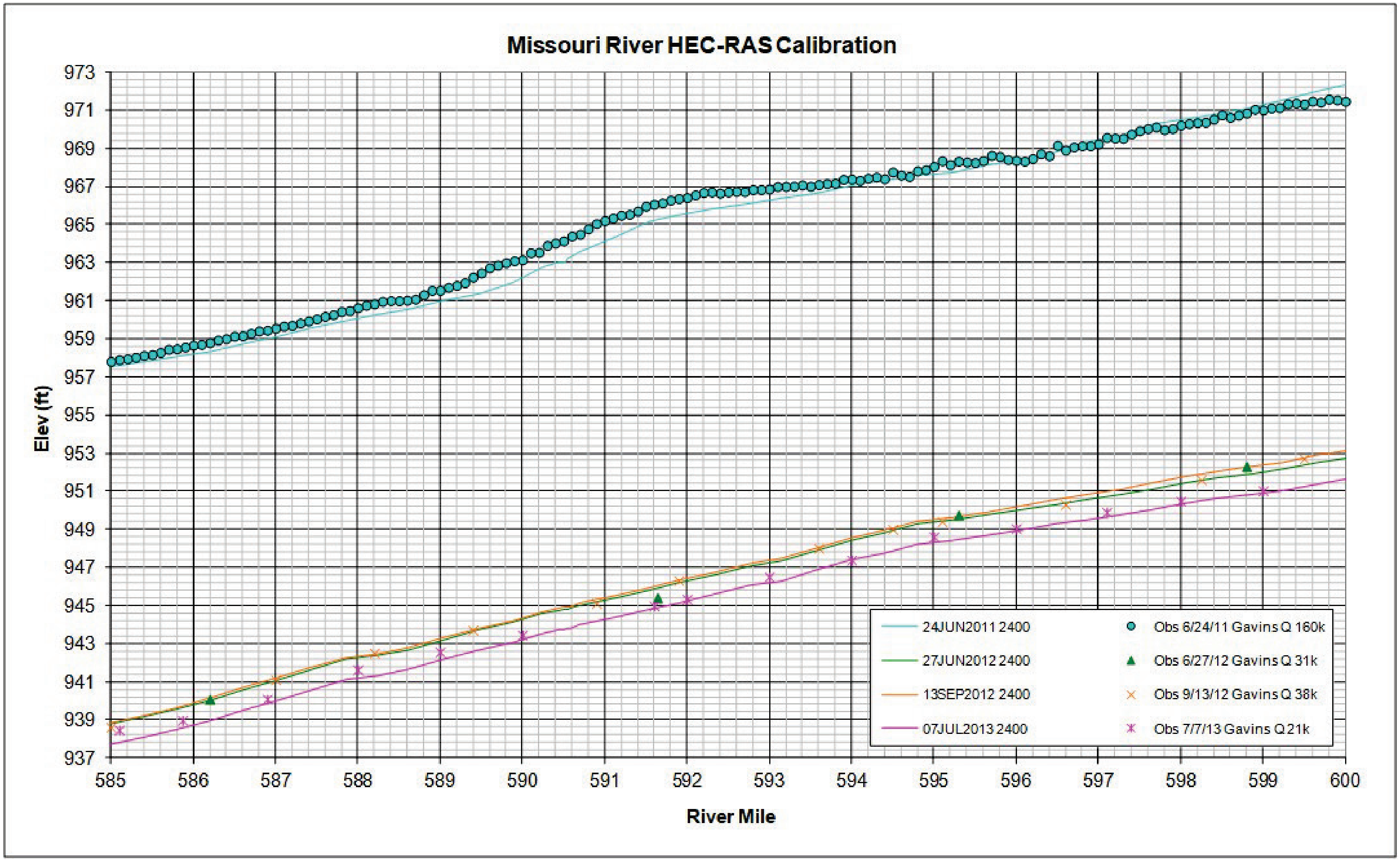

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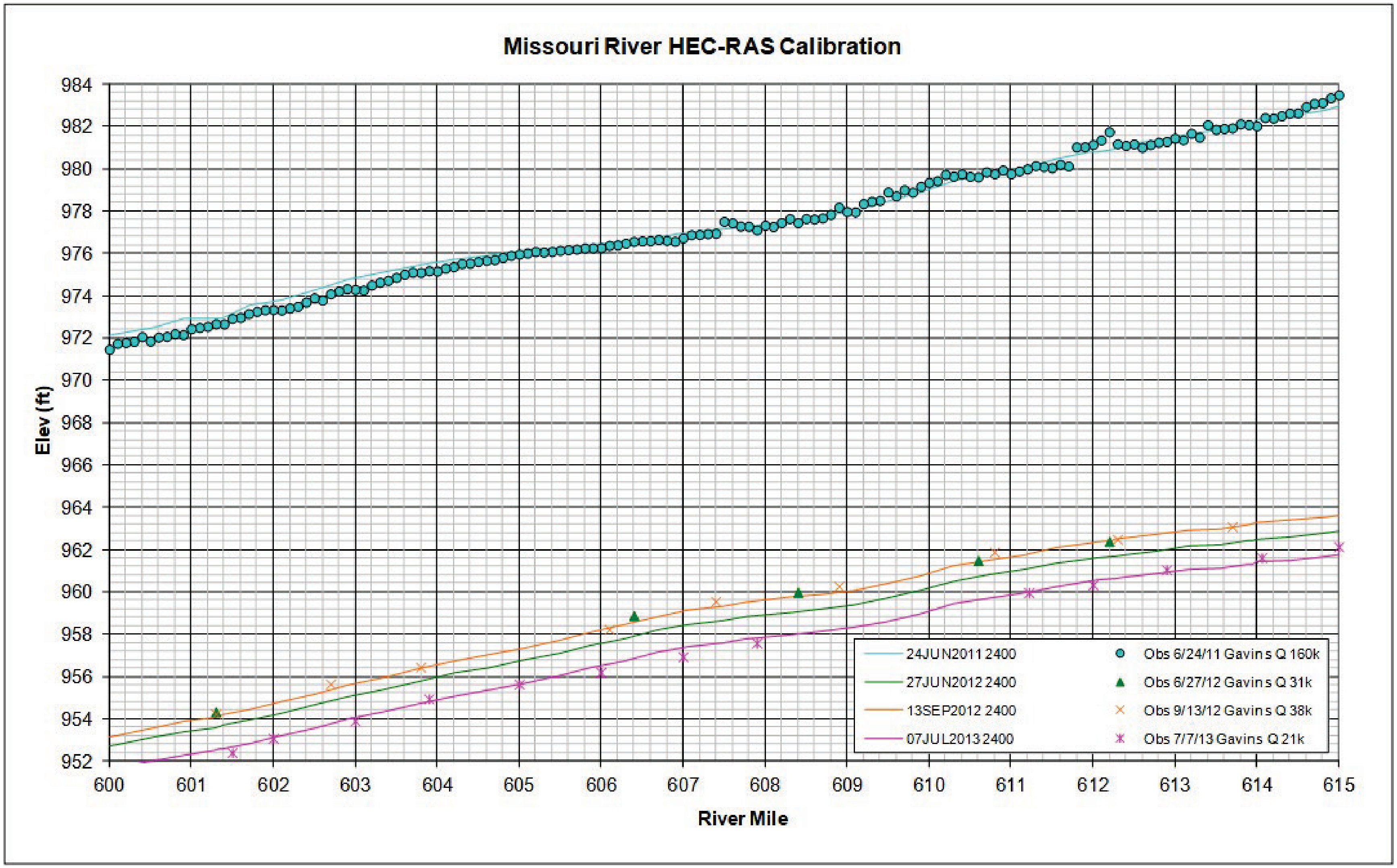

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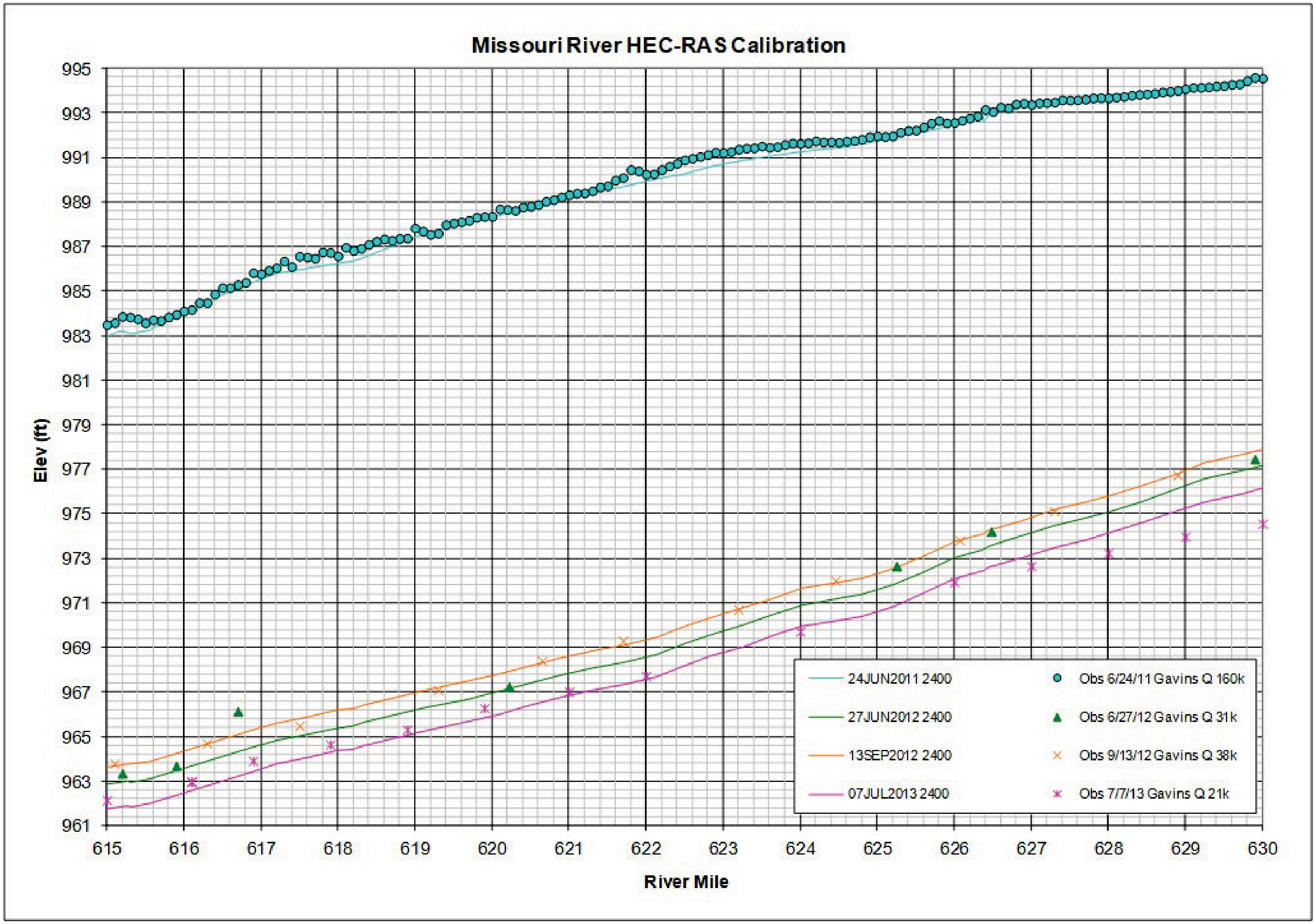

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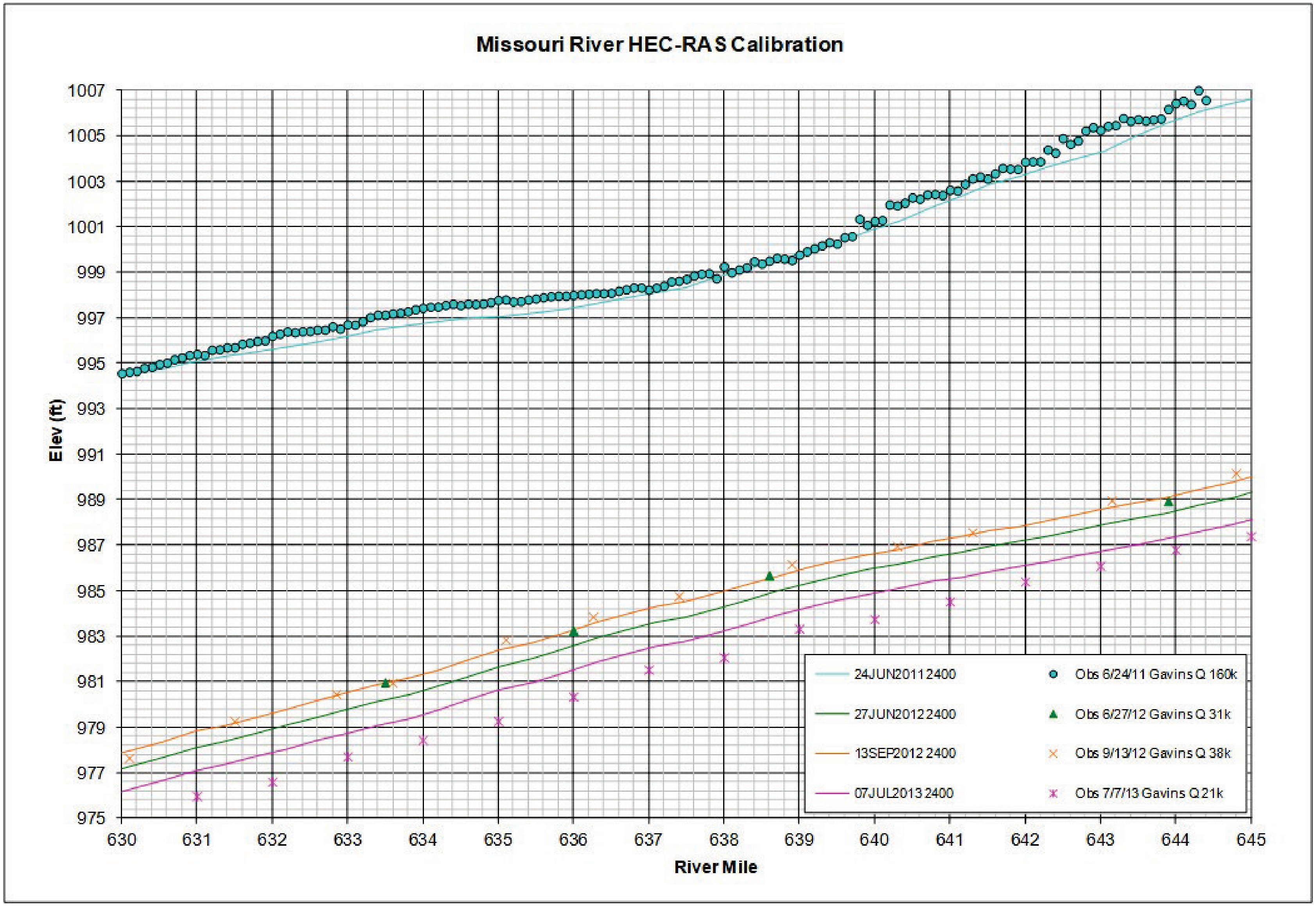

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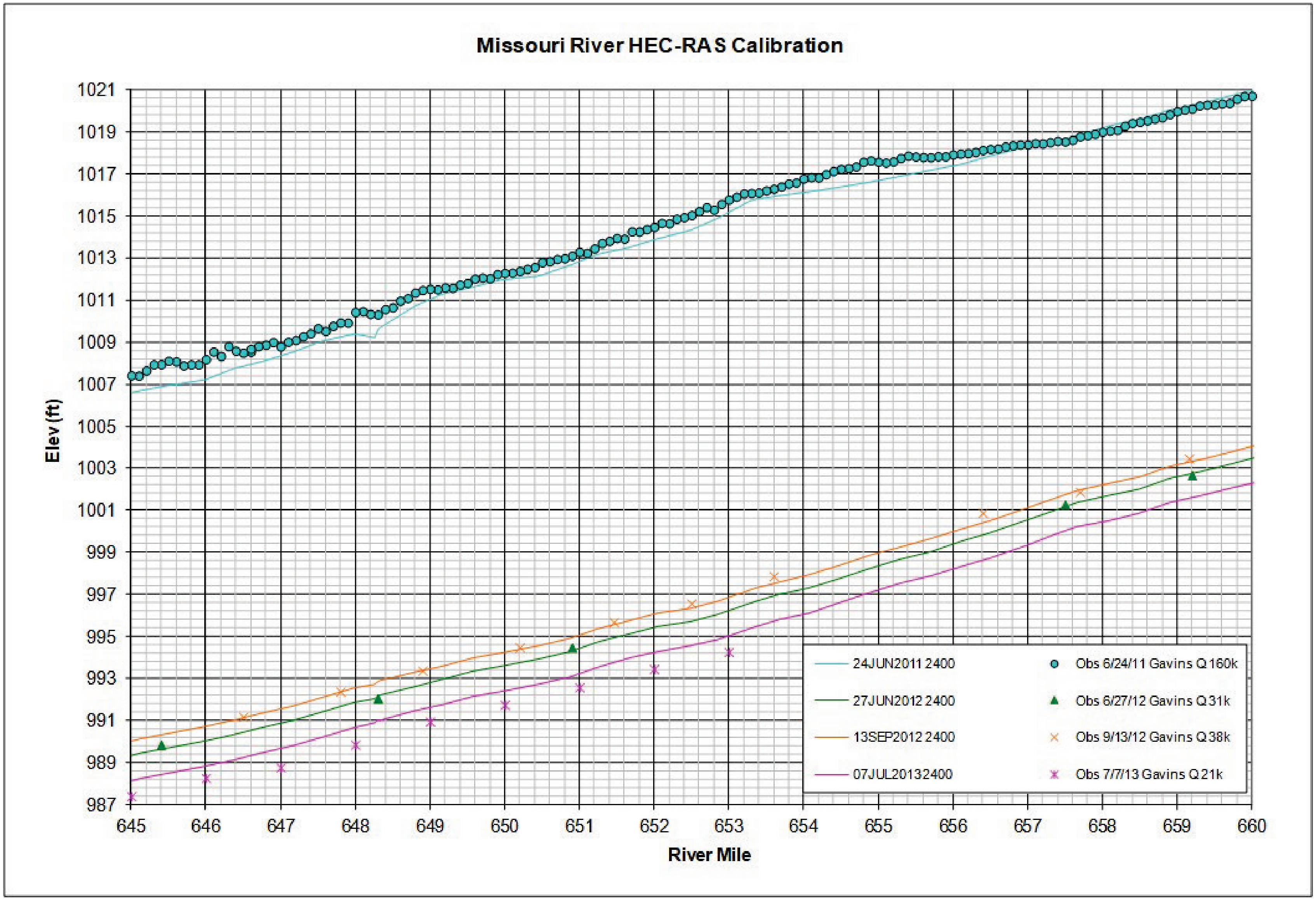

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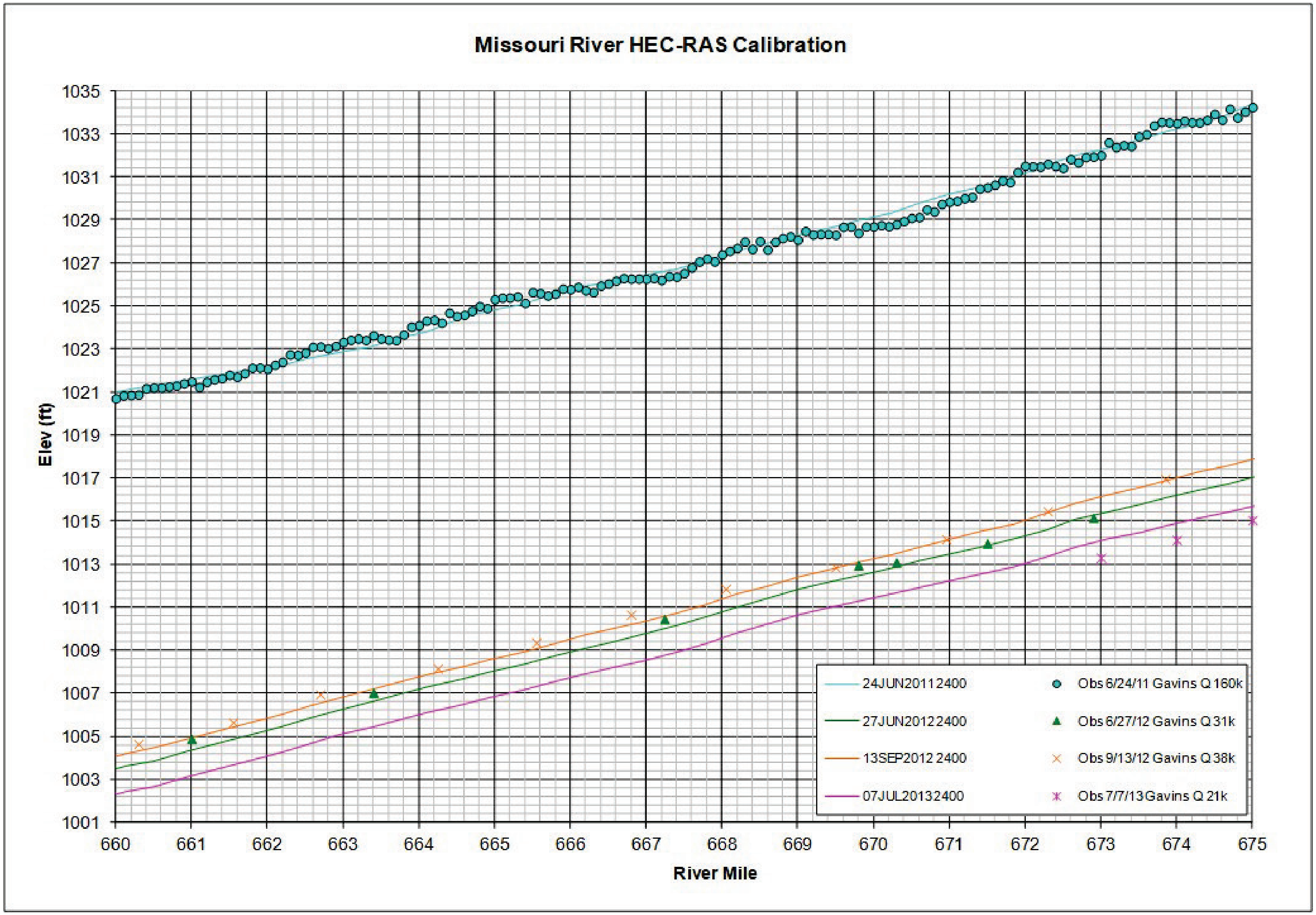

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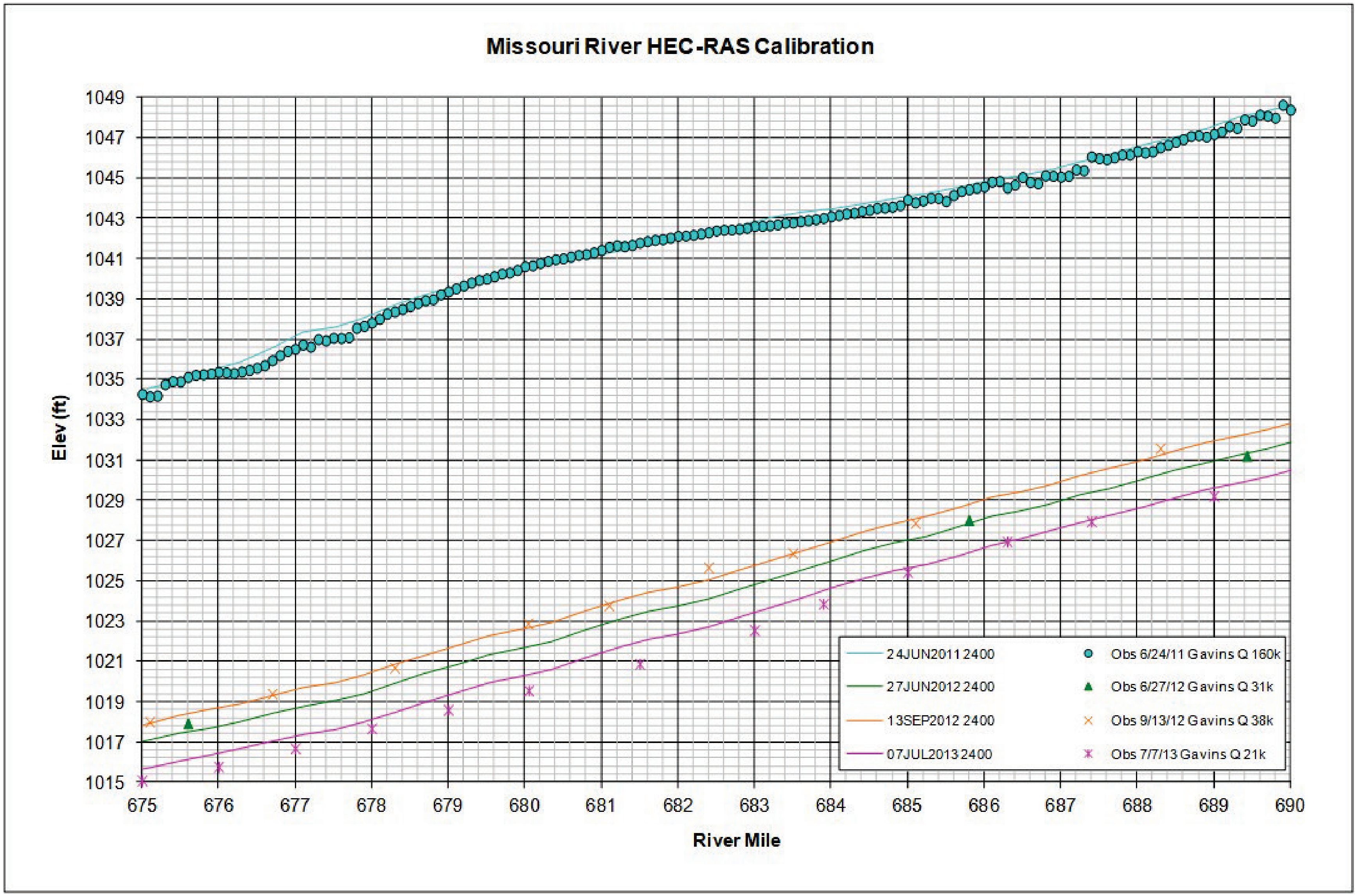

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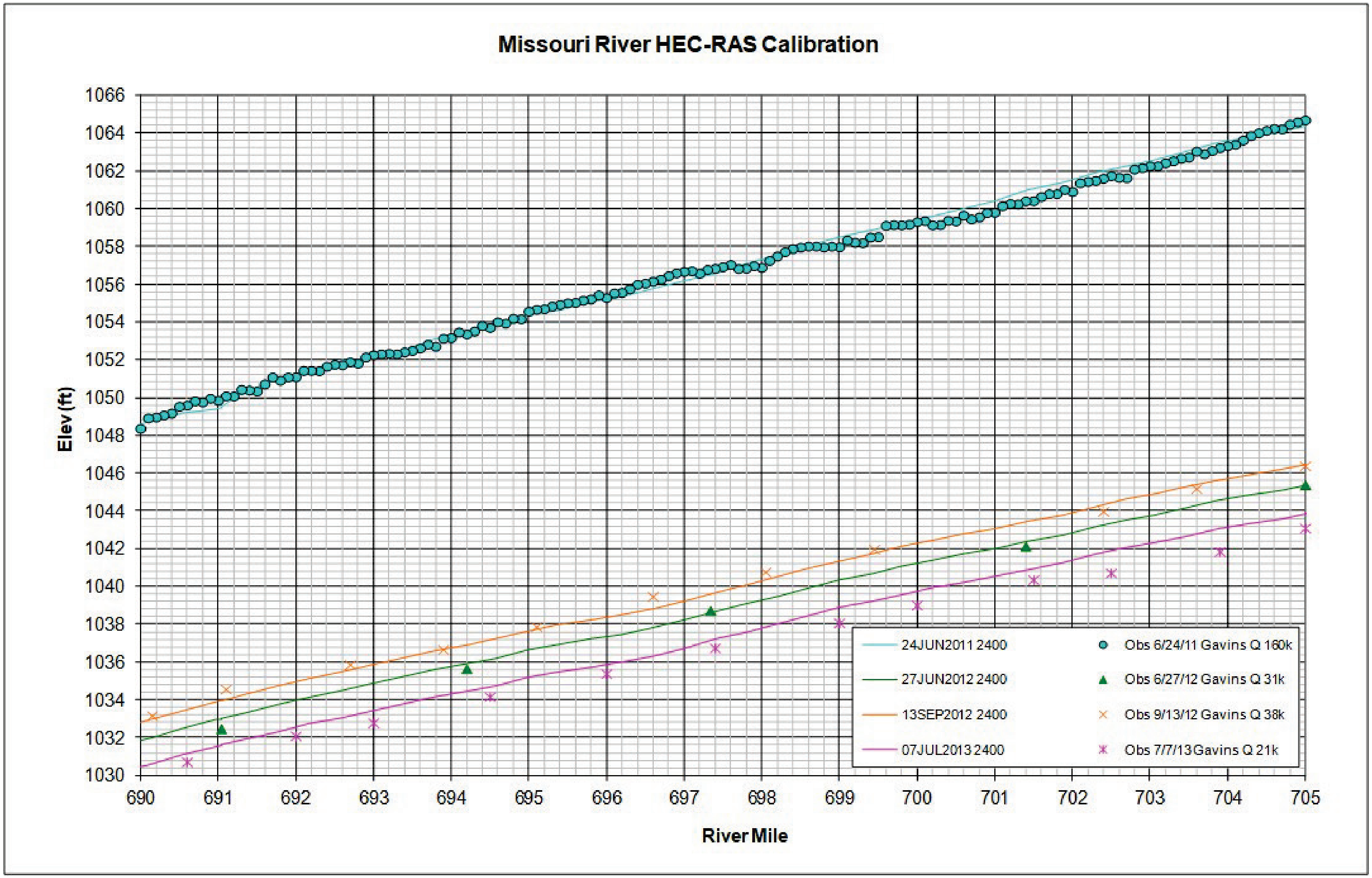

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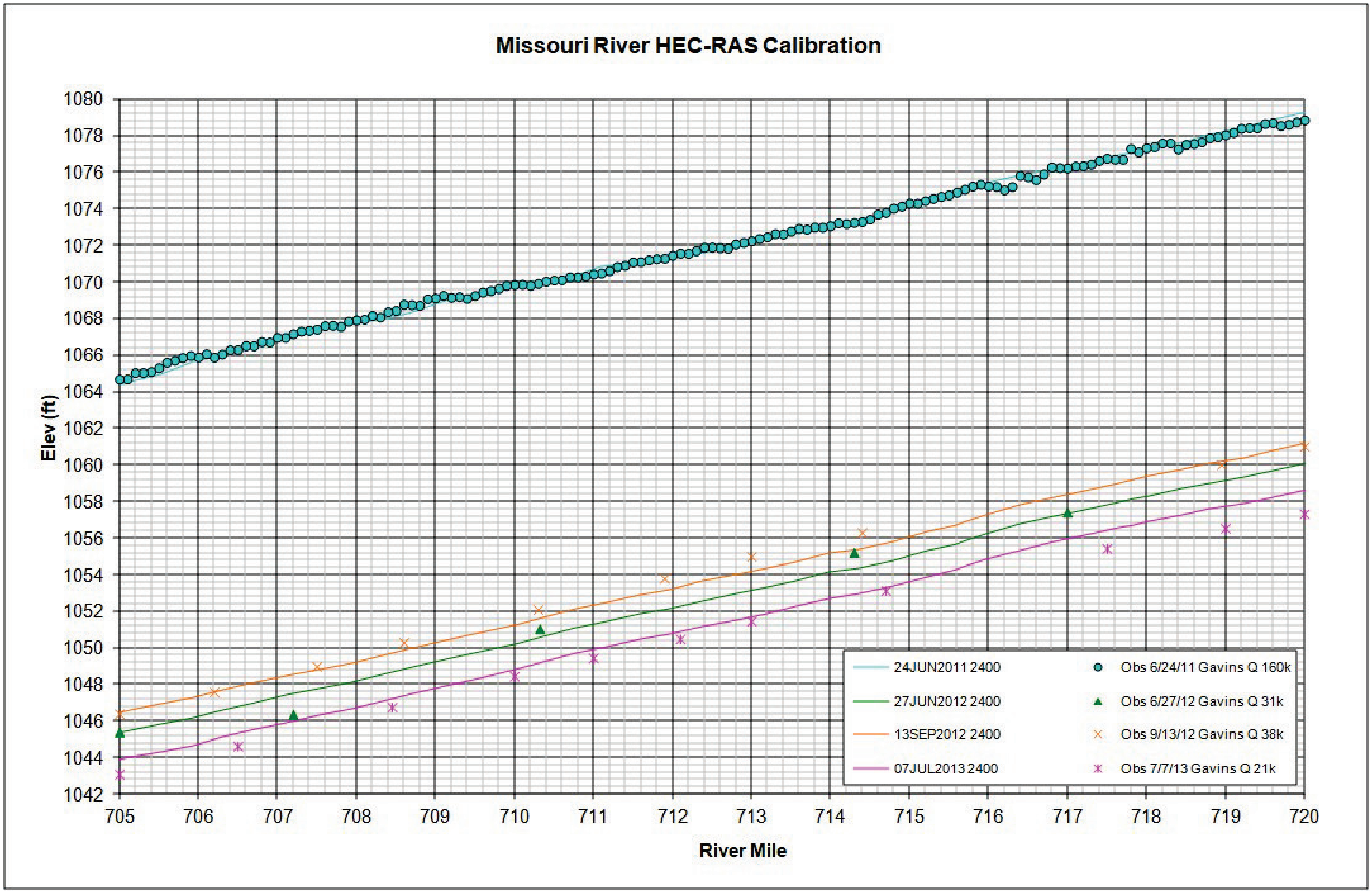

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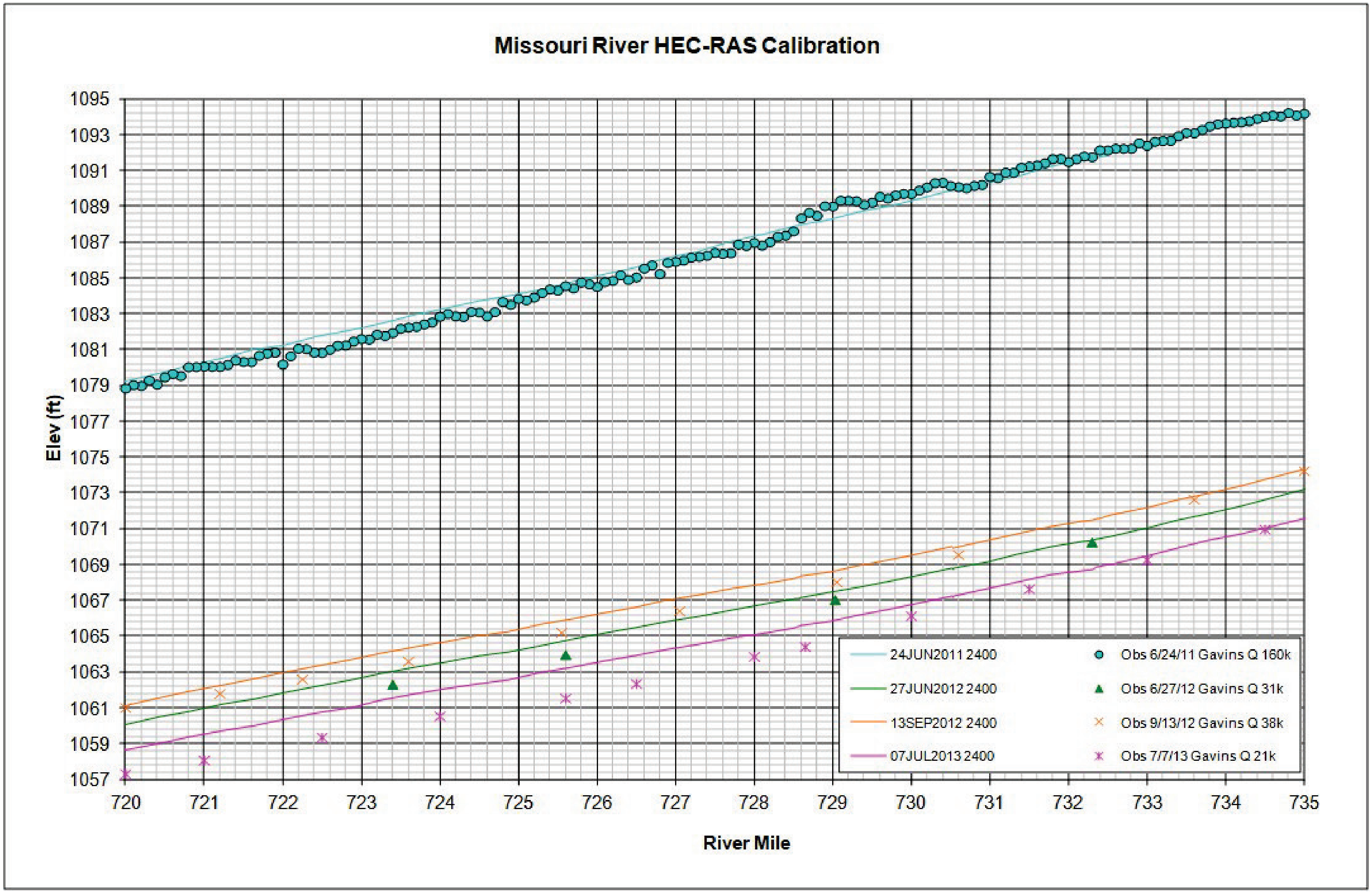

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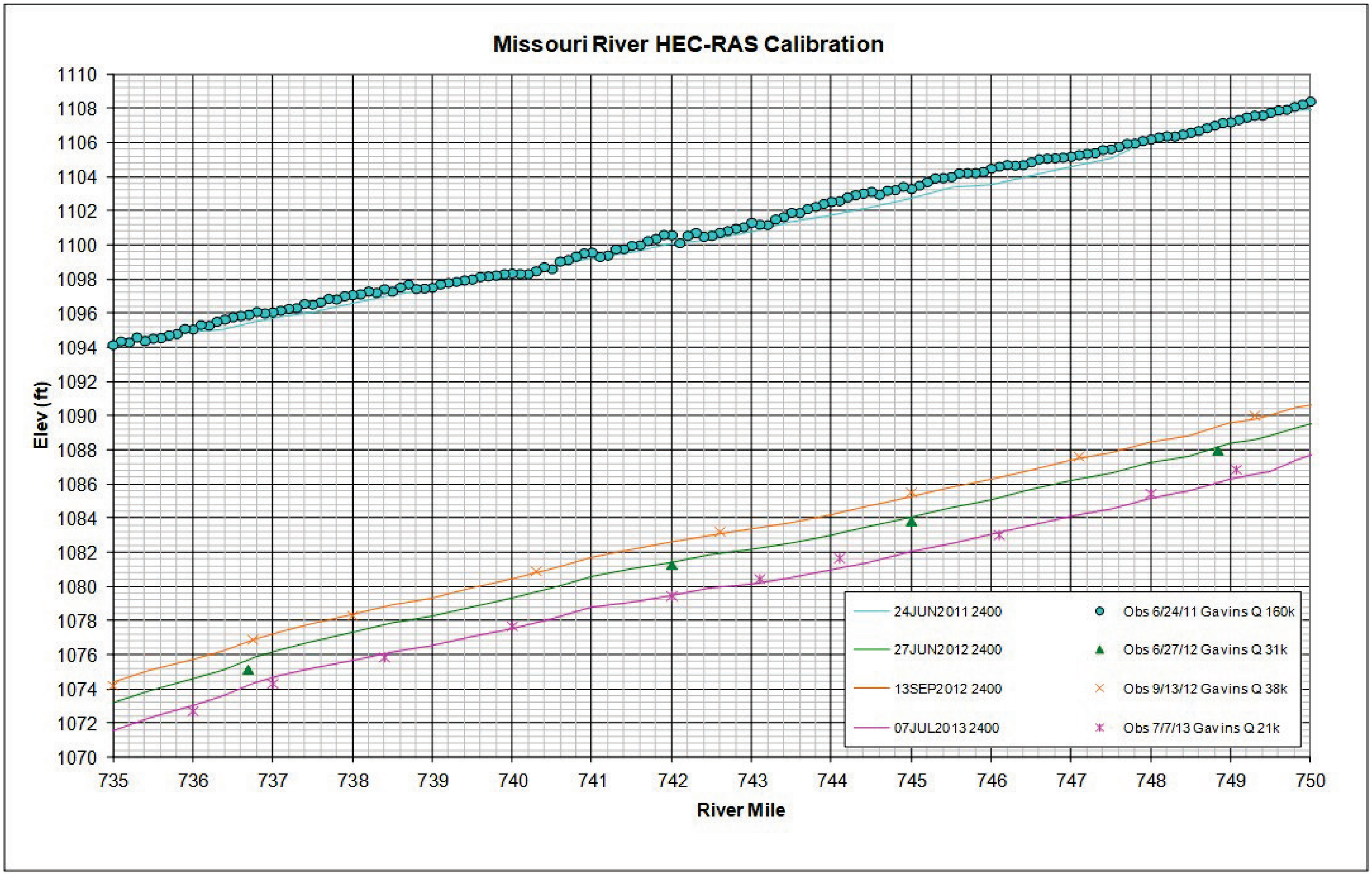

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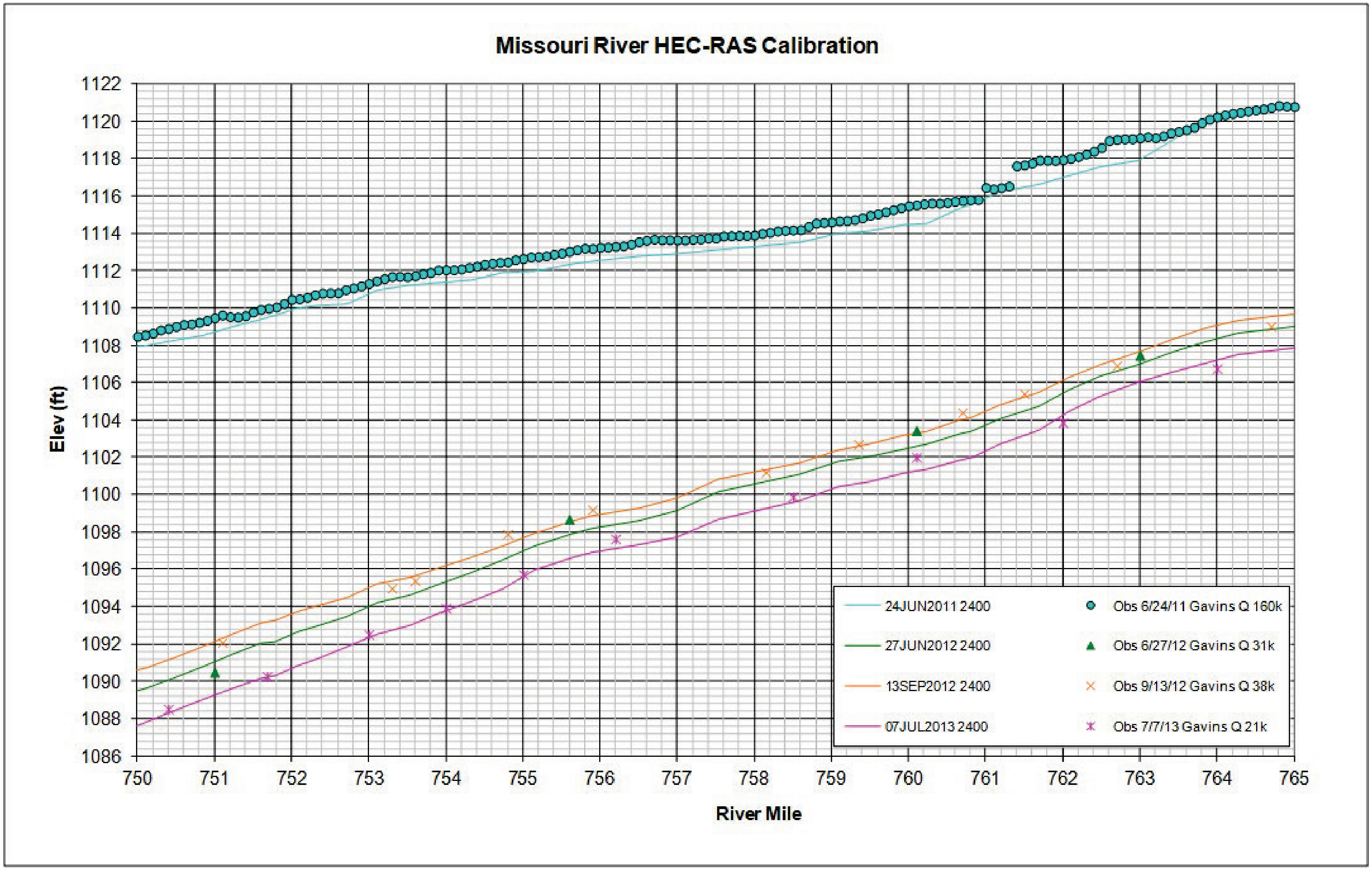

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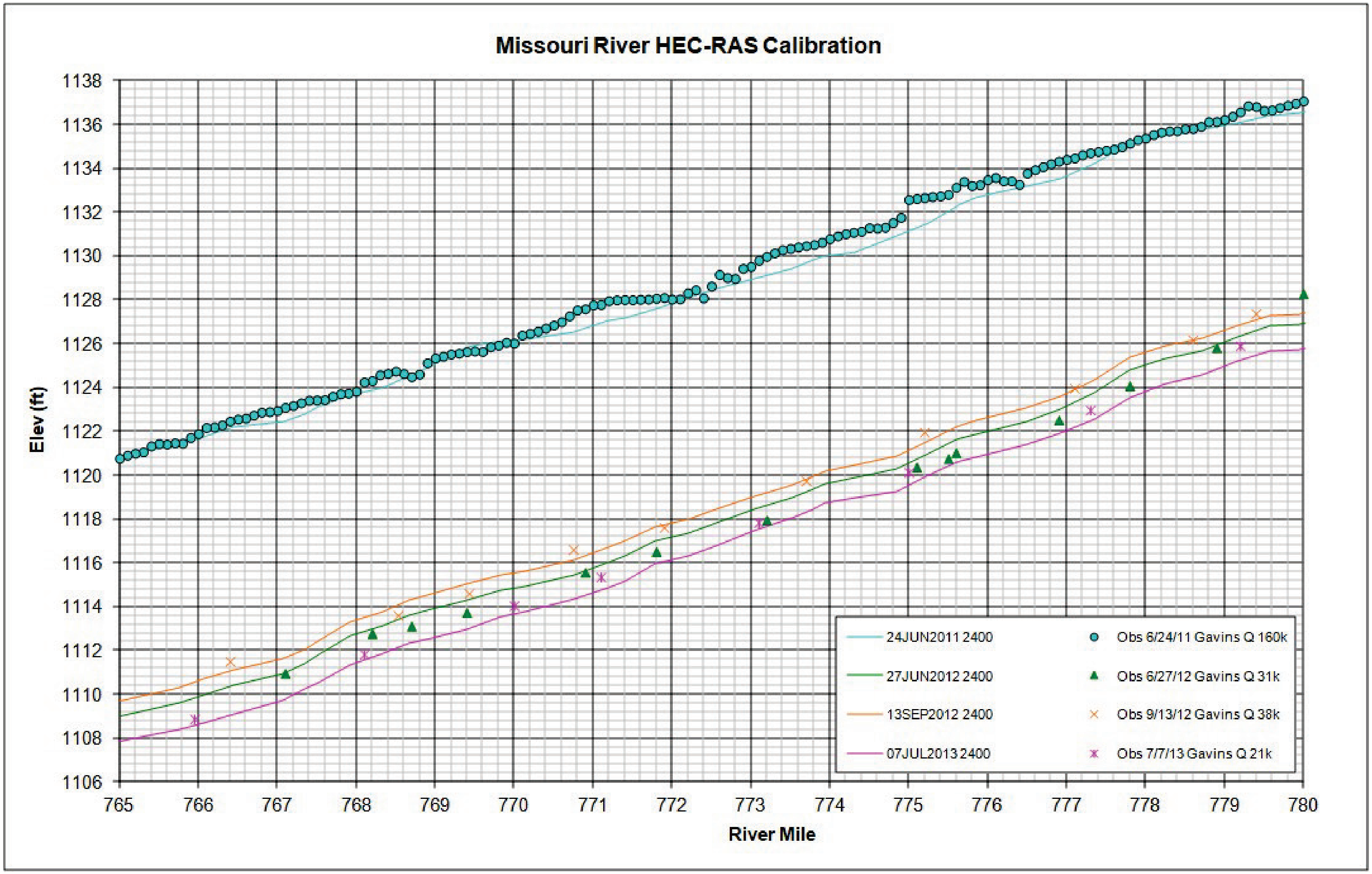

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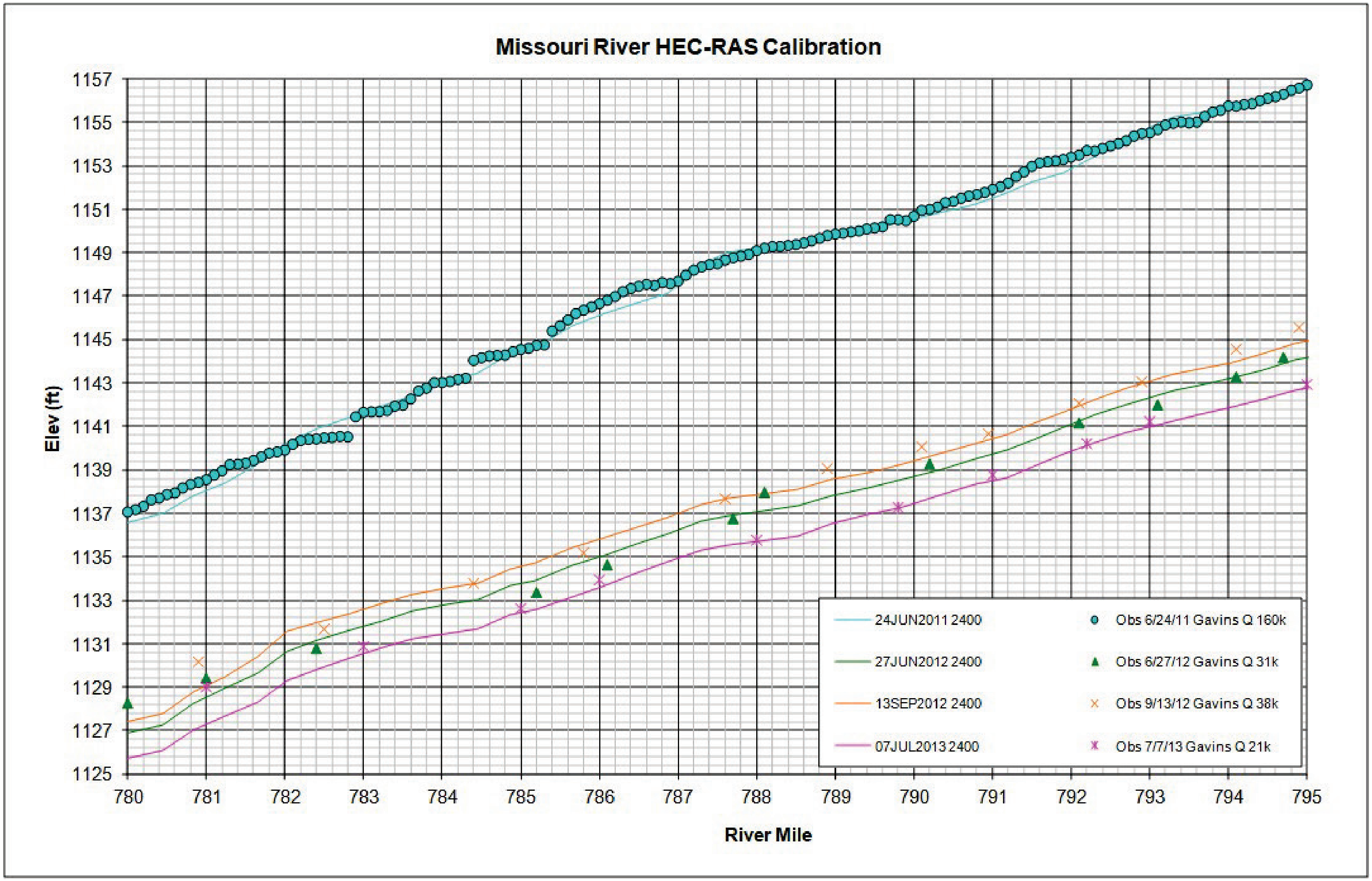

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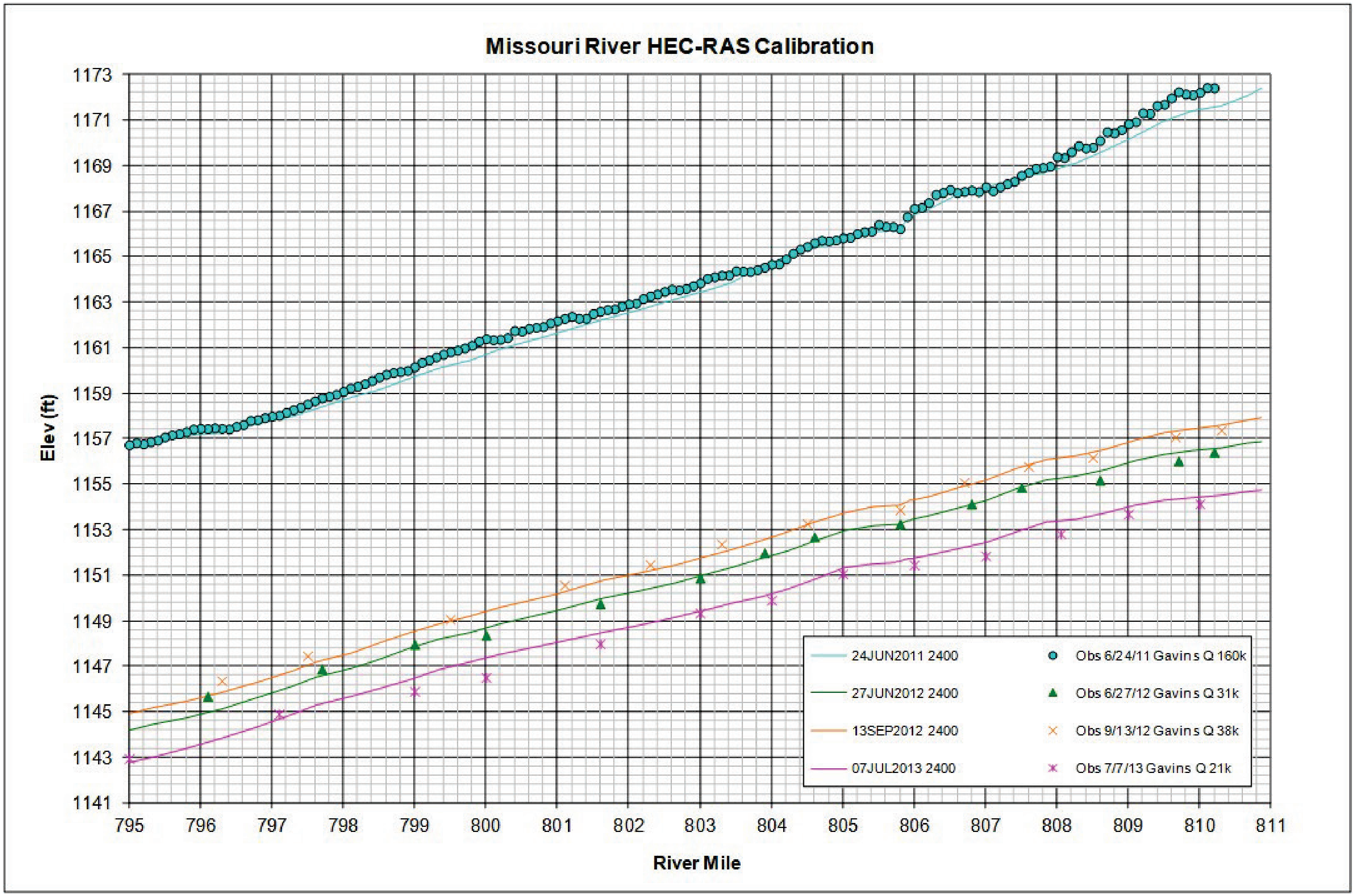

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Missouri River Unsteady HEC-RAS Model
Calibration Report

US Army Corps
of Engineers ®

Kansas City District

Appendix E

Rulo, NE to the Mouth

July 2018

COVER PHOTO:

The Missouri River at Kansas City looking upstream. Kaw point and the Kansas River confluence is at the top, the Charles B. Wheeler Downtown Airport is on the right, and the Broadway Bridge carrying Hwy 169 traffic over the Missouri River is on bottom. Taken from helicopter on September 7 during the descending limb of the 2011 flood event at an approximate flow rate of 140,000-cfs.

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ACRONYMS

AEP……………… ..... Annual Exceedance Probability

DEM………………….Digital Elevation Model

DTM………………… Digital Terrain Model

DSSVue…………….. Data Storage System (by HEC)

GIS........................... Geographic Information System

HAMP....................... Habitat Assessment Monitoring Program

HEC .... Hydrologic Engineering Center

MVS…………………. Mississippi Valley Division St Louis District

NAD 1983………… ... North American Datum of 1983

NAVD 88……………... North American Vertical Datum of 1988

NGVD 29……………. National Geodetic Vertical Datum of 1929

NED…………………. . National Elevation Dataset

NLD………………….. National Levee Database

NSRS……………… .. National Spatial Reference System

NWK………………… Northwest Division Kansas City District

NWO………………… Northwest Division Omaha District

NWS......................... National Weather Service

POR .... Period of Record

RAS.......................... River Analysis System

ResSim……………....Reservoir Simulation Software (by HEC)

RM…………………… River Mile

SWH......................... Shallow Water Habitat

UMRSFFS................ Upper Mississippi River System Flow Frequency Study

USACE……………… United States Army Corps of Engineers

USGS....................... United States Geological Survey

1 EXECUTIVE SUMMARY

The Rulo to the Mouth reach of the Missouri River Recovery Program Management Plan modeling efforts is the lower 498 mile stretch contained within the boundary of the United States Army Corps of Engineers (USACE) Kansas City District. Configured to run in Hydrologic Engineering Center River Analysis System (HEC-RAS) Version 5.0, the model is fully unsteady. Inputs into the model are flow hydrographs, and outputs are stage and flow hydrographs at every cross section as well as a number of additional calculated parameters such as average channel velocity. The latest version of HEC-RAS also has the ability to create inundation depth grids at various flow regimes using RAS Mapper that can be exported for use in ecological and economic models.

There are several geometry features that are unique to the Rulo to the Mouth reach, and unique to modeling efforts thus far completed on this stretch of the Missouri River. Fourteen of the largest tributaries are modeled as reaches in HEC-RAS, contributing a routed hydrograph from a USGS gage to the flow in the mainstem Missouri. Leveed areas in the floodplain are represented in the model with lateral structures and storage areas, which is an improved way to account for flooding verses full valley cross sections. This is especially important near Rulo, NE, around Waverly, MO, and at the confluence because a wide floodplain, multiple levees, and high ground obstructions make flooding complex in these reaches. In addition, navigation structures heavily influence low flows on the lower Missouri River so they were included in the model as permanent ineffective flow areas.

Calibration was performed using recent USGS instantaneous gage data for a six year block of time from October 1, 2007 to September 30, 2013. Between Rulo and the mouth, seven USGS stage-flow gages and three stage-only gages have reasonable record lengths during these six years. Calibration efforts focused on matching stages and flows at these gages for flows ranging from the low winter flows of 2012 to the significant floods of 2011 and 2013. Ungaged inflows were estimated by a combination of scaling up tributary flows by the basin area ratio and adding uniform monthly averaged missing flows. Additional calibration data included a low water profile collected in 2009 and high water marks collected after 2011 and 2013.

Calibration of this model is intended to reproduce on average the low and high conditions on the Missouri River. It was not calibrated tightly to any one event, but rather generally represents the present day stage-discharge relationships at USGS gages on the Missouri River. On average, the model has a mean stage error of 0.1 feet with a root mean square stage error of 0.8 feet, 86% of the time the computed stage is within 1-ft of observed, and 97% of the time it is within 2-ft of observed. Model calibration is adequate for the objective of running a period of record to evaluate alternatives that may include operational and/or physical changes.

2 INTRODUCTION

The Kansas City District portion of the Missouri River unsteady HEC-RAS model was created as a base model for planning studies which could be used to simulate and analyze broad scale basin management alternatives. The objective of this HEC-RAS model is to simulate current conditions on the Missouri River, with the intention of running period of record (POR) flows against which to compare various management plans. This report addresses model construction and calibration for a baseline condition that represents the river as closely as possible for present day conditions. Future reports will address the period of record and evaluation of alternative river management scenarios.

3 BACKGROUND

The lower Missouri River, below Rulo Nebraska, has some distinct characteristics that set it apart from the other reaches of the Missouri, as well as unique flood and drought years that have left their mark on the system.

3.1 Reach Characteristics

Basin area of the Kansas City District includes everything that drains to the Missouri River from Rulo, Nebraska at approximately river mile 498 to the confluence with the Mississippi near St. Louis Missouri. This territory encompasses most of the states of Missouri and Kansas as well as portions of Iowa, Nebraska, and Colorado for a total area of 110,445 sq-mi. In this reach the Missouri River meanders south through the dissected till planes of the central lowlands from Rulo to Kansas City, then traverses east along the northern border of the Osage Plains and Ozark Plateau until it empties into the Mississippi River north of St. Louis, MO (USACE, Kansas City District, 1994). Major tributaries include the Kansas, Grand, Chariton, Osage, and Gasconade.

Other than Kansas City and St. Louis, there are a handful of smaller cities along the lower Missouri River, the largest of which is Jefferson City. Most residential areas are on the high bluffs out of the floodplain, although there are some small populations and industrial areas residing in the floodplain behind levees. Nearly the entire floodplain of the Missouri River has at least a 5-year levee, behind which is primarily agricultural land. The channel itself is fixed in place by erosion protection measures of the Bank Stabilization and Navigation Project (BSNP), which also provide for a self scouring navigation channel.

3.2 Model Extent

Modeling responsibilities of the Kansas City District (NWK) are represented by the red line in Figure 1, from Rulo to the mouth at St. Louis. Limits of the NWK HEC-RAS model extend upstream and downstream of the district boundary for several reasons. First, it is poor modeling practice to have model boundaries close to the area being evaluated because the boundary conditions can introduce errors. Second, the complicated nature of modeling extreme floods such as 1993 and 2011 dictated that the model must extend upstream of Rulo and along the Mississippi River both upstream and downstream of the mouth. Third, backwater from the Mississippi River can influence water levels on several miles of the lower Missouri River. Fourth, passing flows between District models can be more easily checked at major stream gage stations, such as Nebraska City. Therefore, approximately 60 miles of the Mississippi River was included, from Lock & Dam 25 to the St. Louis USGS gage. Upstream, the model limits were extended approximately 60 miles to Nebraska City.

The Omaha District (NWO) is responsible for HEC-RAS modeling upstream of Rulo to Gavins Point (see Appendix D), and RAS models further upstream between the mainstem Reservoirs (see Appendix A – C). Cross sections at the Rulo bridge and upstream were provided by NWO and merged into the NWK model. NWK took responsibility for modeling leveed areas at Rulo and upstream to federal levee L-536. Federal levee L-536 and all other leveed areas upstream to Nebraska City were provided by NWO. Collaboration between the districts at the tie-in location at Rulo was ongoing during the modeling process.

Figure 1. Model extents

3.3 Recent Flood and Drought History

Calibration data included water years 2008 through 2013, as explained in more detail in Section 4.3.1, a time period where applicable 15 minute flow data is available. This six year block of time includes a diverse range of high and low flows. There were extreme floods in 2011 and 2013, moderate flooding experienced in 2008 and 2010, and drought conditions during the summer and winter of 2012. Figure 2 captures the general trends as seen in the observed flow hydrographs at an upper, middle, and lower river gage. Rulo is at the upstream boundary of the NWK district, Kansas City is just downstream of the Kansas River which is the largest tributary, and Hermann is about 100 miles upstream of St. Louis as well as below all of the major tributaries.

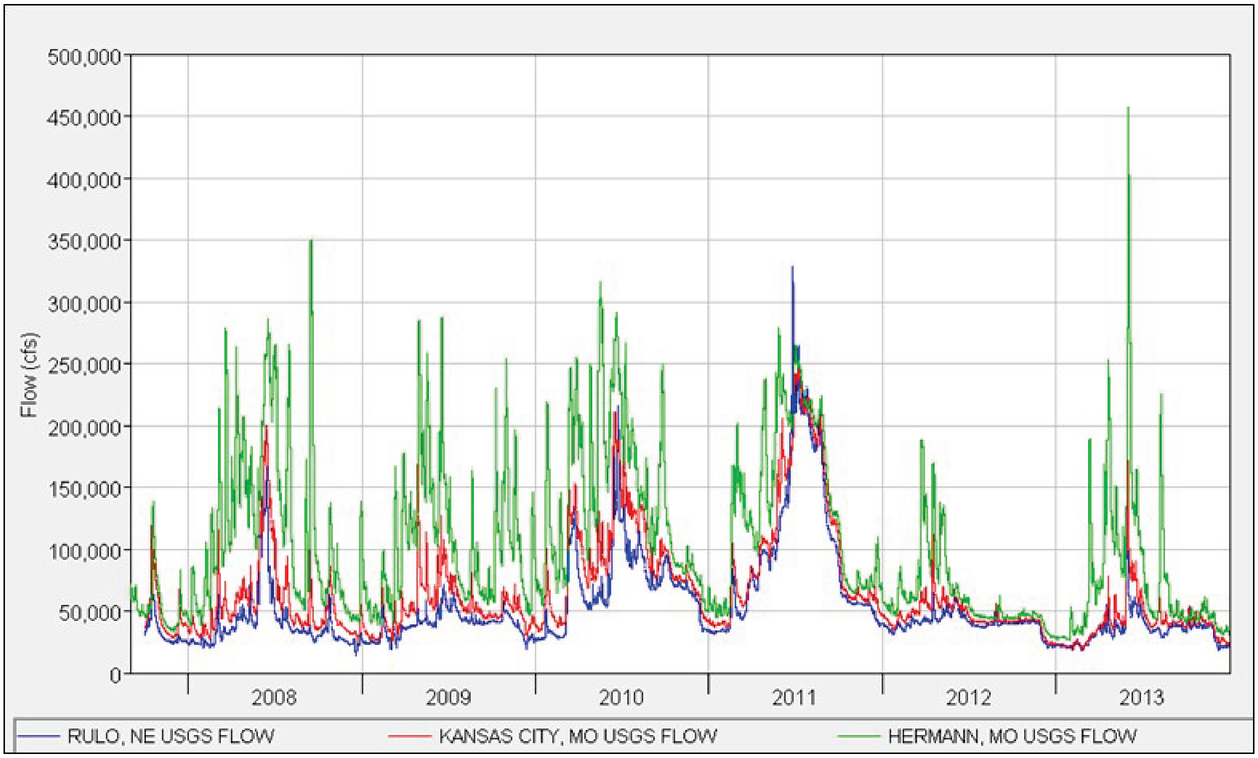

Figure 2. Recent flows on the lower Missouri

The summer of 2008 was generally considered wet with above average precipitation in the NWK basin area. A flood event in June overtopped two levees upstream of Kansas City, and overtopped and/or breached several levees on the Grand River. There were additional high water events in late July and in September that caused further flood damages, primarily on the Grand River and downstream.

In 2009 there were moderate flows but generally no damaging flood events other than slope failures primarily on a few levee tiebacks.

During the summer of 2010 there was a flood event in mid to late June which overtopped four levees upstream of Kansas City, and caused one levee to overtop/breach between Kansas City and the Grand River.

The summer of 2011 brought a considerable flood event, the worst in recent memory since the great flood of 1993. Heavy rain and snowmelt in the upper Missouri basin resulted in record releases from Gavin's Point dam from the end of May through mid-October. The event was more severe on the upper Missouri River than the lower. Approximately seventeen levees in the NWK PL 84-99 program overtopped and/or breached, all upstream of Miami, MO, and most of the levees down to the mouth were loaded to some extent. A record stage was set at Rulo, Nebraska, and the peak discharge at that location was approximately a 0.2% annual exceedance probability (AEP) (500-yr) according to Upper Mississippi River System Flow

Frequency Study (UMRSFFS), the most recent hydrologic analysis available on the Missouri River. At Kansas City the discharge was approximately a 10% AEP (10-yr).

The summer of 2012 was exceptionally dry. Drought conditions on the Mississippi River were worse than on the Missouri, and approached record low stage at some locations. Releases from the Missouri reservoir system were critical to supporting minimal service levels on the Mississippi until the end of navigation season in late November.

During the summer of 2013 there was a late June event in which heavy local rainfall on the lower river caused flooding primarily downstream of the Grand River. Nine levees in the NWK PL 84-99 program overtopped, two of which overtopped and breached. Several of the levees experienced overtopping on the tiebacks caused by the combination of high local inflows with a high Missouri River condition. Peak flows on the Missouri River downstream of the Grand were the largest observed since the May 1995 flood event with magnitudes exceeding a 10% AEP flood.

Calibration to this range of events over the entire length of the model is important because the ranges of management alternatives to be evaluated with the model have both habitat and human consideration impacts at both high and low flow conditions.

As an additional note, the flood of record for the Missouri River below Kansas City occurred in the summer of 1993. This flood was not included in the calibration period primarily because the geometry in the model represents present day conditions, and there have been significant changes to the overbanks and channel geometry. Many levees were raised, consolidated, set back, or taken out of commission as a result of the 1993 flood.

4 DATA SOURCES

Primary data sources for construction of the unsteady HEC-RAS model included terrain data, bathymetry data, and gage data. Terrain data encompasses everything from the bluffs to the riverbanks, defining the floodplain and overbanks, but does not often include data below the surface of the river. Bathymetry captures below the water surface. Gage data provides the boundary conditions for the model, and calibration benchmarks.

4.1 BATHYMETRY

Channel bed elevations for the lower Missouri River are maintained by the Kansas City District River section. Depth to the channel bed is collected by boat with a sonic depth sounder and collection software called HYPACK which utilizes GPS for collection of horizontal positioning and ground elevation. Depths are collected along pre-determined cross sections, and post-processed to calculate elevation based on distance to water surface measured at local benchmarks. Elevations are filtered to a sounding increment of approximately 5-ft, and snapped to the guiding cross section for analysis and use in software such as HEC-RAS. Uncertainty associated with the collected depths and elevations is due to high sediment concentrations, constantly moving bed forms, boat deviation, and variability in water surface. A full survey of the entire Missouri River is not completed every year. Table 1 gives details on the hydrosurvey data collected in the most recent five years. Collection dates for all years are summer to fall.

Table 1. Recent hydrographic surveys of the Missouri River

Year	Collected
2009	Full survey, approximately 10,551 cross sections from Rulo to St. Louis at 250-ft intervals
2010	None
2011	Multiple surveys from St Joseph to Waverly to monitor degradation in the Kansas City reach during the flood event
2012	Survey limited to 1310 cross sections at 2000-ft intervals
2013	Full survey, approximately 10,551 cross sections from Rulo to St. Louis at 250-ft intervals, final deliverable received in Jan 2014

4.2 TERRAIN

A variety of terrain sources are available for the Missouri River basin and floodplain. Described below are the source, collection methods and dates, and accuracy of each.

4.2.1 *3-Meter LiDAR*

In 2013 the Kansas City District Geospatial Branch (ED-S) compiled a mosaic of the latest available LiDAR covering the Missouri River Floodplain into a 3-meter digital elevation model

(DEM) in North American Vertical Datum of 1988 (NAVD 88). The LiDAR was primarily collected in 2010 to 2012 by Corps contractors and or the USGS, with the oldest data contained in the mosaic collected in 2006. All components compiled included hydroflattening, which applies downhill hydrological rules to rivers, water bodies, and other drainage paths. Original LiDAR data was collected at primarily 1-meter density and then was reduced in resolution to various densities using a bilinear re-sampling technique in order to better manage overall file size versus detail needed. Total size of the 3-meter LiDAR surface is 45 GB. Tests of the accuracy of top of levee information pulled from the DEM measured against original data and the National Levee Database (NLD) survey information have revealed that the 3-meter DEM can typically be considered within approximately 0.5-ft vertical accuracy of the more detailed survey methods. The mosaic surface covers the entire floodplain from bluff to bluff, at times extending slightly beyond, but within the NWK district boundary only.

3-meter LiDAR used for:

- Storage Areas (see Section 5.2.6)
- Lateral Structures (see Section 5.2.7)
- Storage Area Connections (see Section 5.2.8)

4.2.2 *NLD Top of Levee Elevations*

More precision was considered important for modeling the top of levees, so the 3-meter LiDAR was supplemented with elevations from the National Levee Database (NLD). For NWK, the NLD was populated in two phases, with Phase I in 2007 including all levees with level of protection at 1% annual chance exceedance or greater, and Phase II in 2010 including the remaining federal and non-federal projects. Top of levee elevations were surveyed in NAVD88 datum and collected at an interval of 100-ft, or where noticeable horizontal or vertical alignment changes occurred. Maintenance on the database includes updating the alignment centerline when there is a levee setback due to flood damages. At locations where this has occurred, the assumption is made that elevations for the realigned portions are carried through the realignment from either end of the repair.

NLD used for:

- Lateral Structures (see Section 5.2.7)

4.2.3 *10-Meter NED*

An additional terrain resource was the National Elevation Dataset (NED), maintained and updated by the USGS. The NED is the best available public domain raster elevation data encompassing the entire United States. Tiles are available online in units of meters, referenced to NAVD 88 vertical datum and resolution of 1/3 arc-second or approximately 10-meters. For purposes of this project, the most current tiles were downloaded in 2009 and then again in 2011, mosaiced together, converted to feet and re-projected to Universal Transverse Mercator (UTM) Zone 15 North. The coverage area for the mosaiced surface extends well beyond the floodplain of the Missouri River, incorporating the extents of the entire modeled area and beyond, including major tributaries.

10-meter NED used for:

1. Tributaries (see Section 5.2.11)

4.2.4 *UMRSFFS DTM / LMOR*

An older, but still relevant source of terrain data in the Missouri River floodplain is the Upper Mississippi River System Flow Frequency (UMRSFFS) Study Digital Terrain Model (DTM). Aerial photography, airborne global positioning system (GPS) control, ground survey control, and aero triangulation were used in development of the terrain. Aerial photography was taken in a combination of the years 1995 and 1998/1999. The DTM data is composed of mass points and break lines that adequately define elevated roads, railroads, levees, and other major topographic changes required for accurate DEM development. Ground surface elevations have a vertical datum of National Geodetic Vertical Datum of 1929 (NGVD 29) and are accurate to within 1.33 feet (U.S. Army Corps of Engineers, 2003). As a part of the Land Capability Index (LCPI) for the Lower Missouri River Valley (LMOR) investigation completed in 2007, the USGS combined bathymetry data from 1998-1999 with the DTM data to create a master DEM (Jacobson, Chojnacki, & Reuter, 2007). This surface covers from bluff to bluff only, but does extend from Gavins to the mouth and for some time was the only DEM available that included the bottom of the river integrated with the surrounding overbank terrain.

UMRSFFS DTM used for:

2. Cross Section Geometry (see Section 5.2.3)

4.2.5 *HAMP Terrain*

As a part of the Habitat Assessment Monitoring Program (HAMP) shallow water habitat (SWH) accounting effort, in the fall of 2014 a terrain was created to best represent the river banks and channel bed in the same surface. To represent the river banks, new low water LiDAR data was collected in the winter of 2013-2014. The 3-meter LiDAR mosaic was not used because in many locations the water surface is high, resulting in a gap between where the LiDAR ends and the hydrographic suvey begins, forcing a straight line assumption that may or may not actually represent the banks. The contract specified a 3000-ft swath centered on the river, wider when necessary to capture chute projects. Using GIS and a software called Global Mapper, the 2013 hydrographic surveyed cross sections were converted to a raster which was merged into the low water LiDAR. In locations where the two overlapped, LiDAR was given priority (USACE, Kansas City District, 2014). The resulting 3-meter combined terrain has many applications, including mapping HEC-RAS output at lower flows. Future efforts intend to merge the HAMP terrain with the 3-meter LiDAR to capture bluff to bluff in one terrain, but this is not complete at this time.

4.3 GAGE DATA

Stream stage and flow data available on the lower Missouri River includes gages along the Missouri River mainstem, and gages on many of the major tributaries. Most are operated by the USGS and collect stage data remotely at intervals of 15 minutes. Availability and quality of these datasets influenced the configuration of the model as well as the timeframe for calibration.

4.3.1 *Instantaneous Records*

For model calibration it was important to use instantaneous data rather than daily averages. Instantaneous data captures the peaks of flood events, while daily averages do not because they are primarily focused on conserving total volume. For example, Figure 3 shows the difference between instantaneous and daily average flows at the Kansas City gage just before the summer 2011 flood event. There is about a 10,000-cfs difference between the highest recorded instantaneous value and the daily average value, which translates to a stage difference of almost 1-ft.

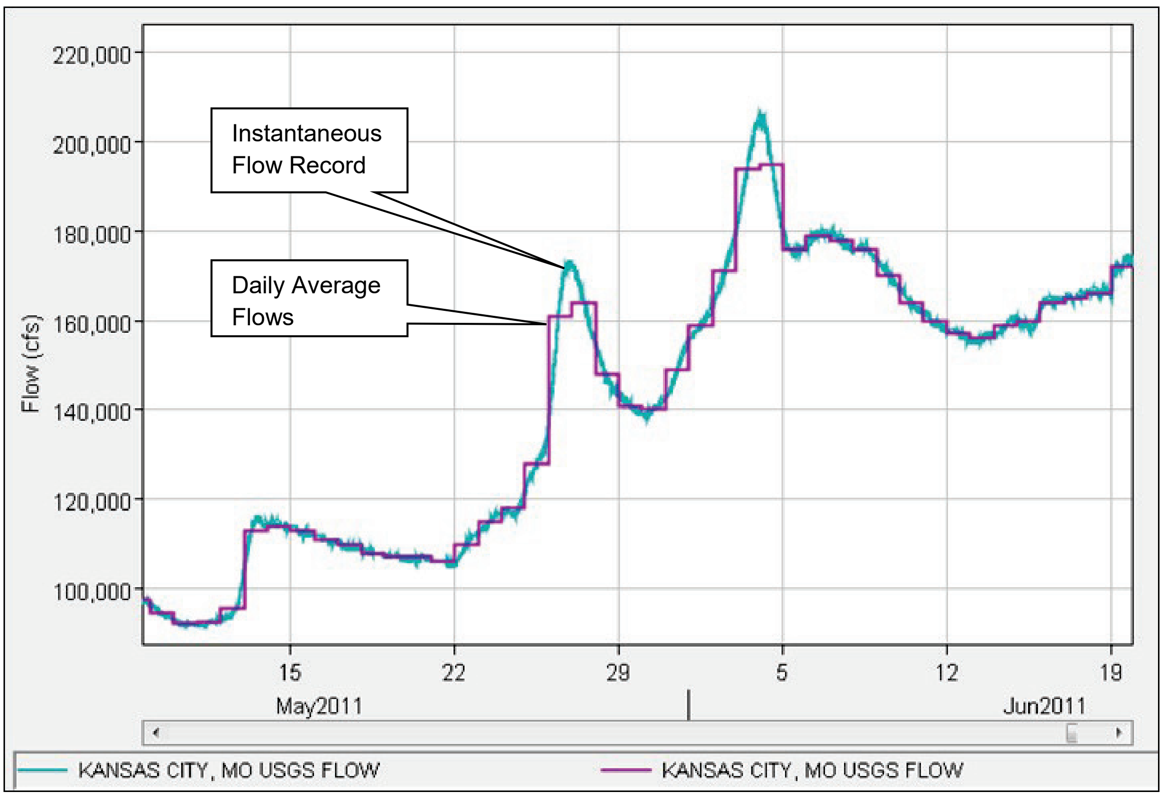

Figure 3. Comparison of instantaneous flows to daily average flows

It is important to note that instantaneous data has not been reviewed or published by the USGS; it is raw data that often contains uncorrected errors and gaps. This is a potential source of error during calibration, however, the risk was considered acceptable because it is the best available source of valuable peak and more precise timing information that is not captured by daily averages.

Within the model extents there are fourteen USGS and five NWS stream gages on the Missouri River, and there are two USGS gages and three USACE gages on the Mississippi. Figure 4 shows the location of the Missouri and Mississippi River gages. Gages at Leavenworth, KS, Napoleon, MO, Jefferson City, MO, and Washington, MO, and the Mississippi at Grafton, IL collect stage only. The USGS either does not take flow measurements or cannot develop a rating curve at these locations. Brownville, NE, Leavenworth, KS and Napoleon, MO came online recently and their records are only a few years long. The USACE gages are located at the Mississippi Lock and Dams (L&D) collect stage data in the pool and tailwater.

Figure 4. Mainstem Gages

Further details on mainstem gages can be found in Table 2. Dates in the table indicate the availability of instantaneous data. Raw instantaneous data is used by the USGS to develop published daily average flows, which are available back to the 1930s for several of the mainstem gages. However, instantaneous data before the early 1990's was not retained. NWS gages have intermittent stage data, also available back to the mid-90s. Readings are collected at irregular intervals by a local sponsor with either a wire weight or by reading a stage gage and then reported to the NWS, which means the potential for human error is higher than at the USGS gages.

Table 2. Mainstem Gages

Gage Name	USGS Gage	River Mile	Basin Area at Gage [1] (sq mi)	Stage Only	Instantaneous Data Start Date	Gage Zero Datum (ft) (NAVD88)
Missouri River						
Nebraska City, NE	x	562.8	410,000		1-Jan-1991	905.66
Brownville, NE	x	535.3			1-May-2010	859.94
Rulo, NE	x	498.0	414,900		1-Jan-1991	838.16
St. Joseph, MO	x	448.2	426,500		1-Apr-1993	789.27
Atchison, KS		422.6		x	1-Jun-1995	762.84
Leavenworth, KS	x	397.5	427,200	x	12-Sep-2012	742.47
Kansas City, MO	x	366.1	484,100		1-Oct-1992	706.68
Sibley, MO		336.5		x	1-Nov-1996	684.40
Napoleon, MO	x	329.1	485,100	x	18-Mar-2009	680.53
Waverly, MO	x	293.2	485,900		1-Oct-1993	646.17
Miami, MO		262.6		x	1-May-1996	621.73
Glasgow, MO	x	226.3	498,900		1-Oct-2000	586.65
Boonville, MO	x	196.6	500,700		1-Oct-1992	565.58
Jefferson City, MO	x	143.9	507,500	x	1-Oct-1994	520.18
Chamois, MO		117.7		x	4-Aug-2001	503.19
Gasconade, MO		105.2		x	10-Aug-1998	484.67
Hermann, MO	x	97.9	522,500		1-Oct-1987	481.50
Washington, MO	x	68.3	523,200	x	4-Sep-2008	457.27
St. Charles, MO	x	27.8	524,000		1-Apr-2000	413.47
Mississippi River						
L&D 25 Tailwater		241.3	142,000	x [2]	1-Jan-1992	406.47
Mississippi at Grafton	x	218.9	171,300	x	1-Oct-1992	403.40
L&D 26 (Mel Price) Pool		200.7	171,500	x	1-Feb-1990	395.04
L&D 26 (Mel Price) Tailwater		200.5	171,500	x	1-Jan-1990	395.04
L&D 27 (Chain of Rocks) Pool		193.8	696,910	x	1-Jan-1993	349.49
L&D 27 (Chain of Rocks) Tailwater		184.0	696,910	x	1-Oct-1985	349.49
Mississippi at St. Louis	x	179.5	697,000		1-Oct-1992	379.46 [3]

[1] Basin areas are from USGS online gage information pages when available

[2] Flows were provided by the St. Louis District, derived from a rating curve developed for internal forecasting purposes. While not the standard of USGS flow data, it was deemed this data was reasonable to use for the Mississippi flow since it will be seen as a constant from the perspective of the alternative analysis.

[3] Datum is preliminary, from survey accomplished by St. Louis District that has not been fully vetted and certified at this time.

For tributaries, twenty-three USGS stream gages measure inflow to the Missouri River within the model extents. Figure 5 shows the location of the tributary gages.

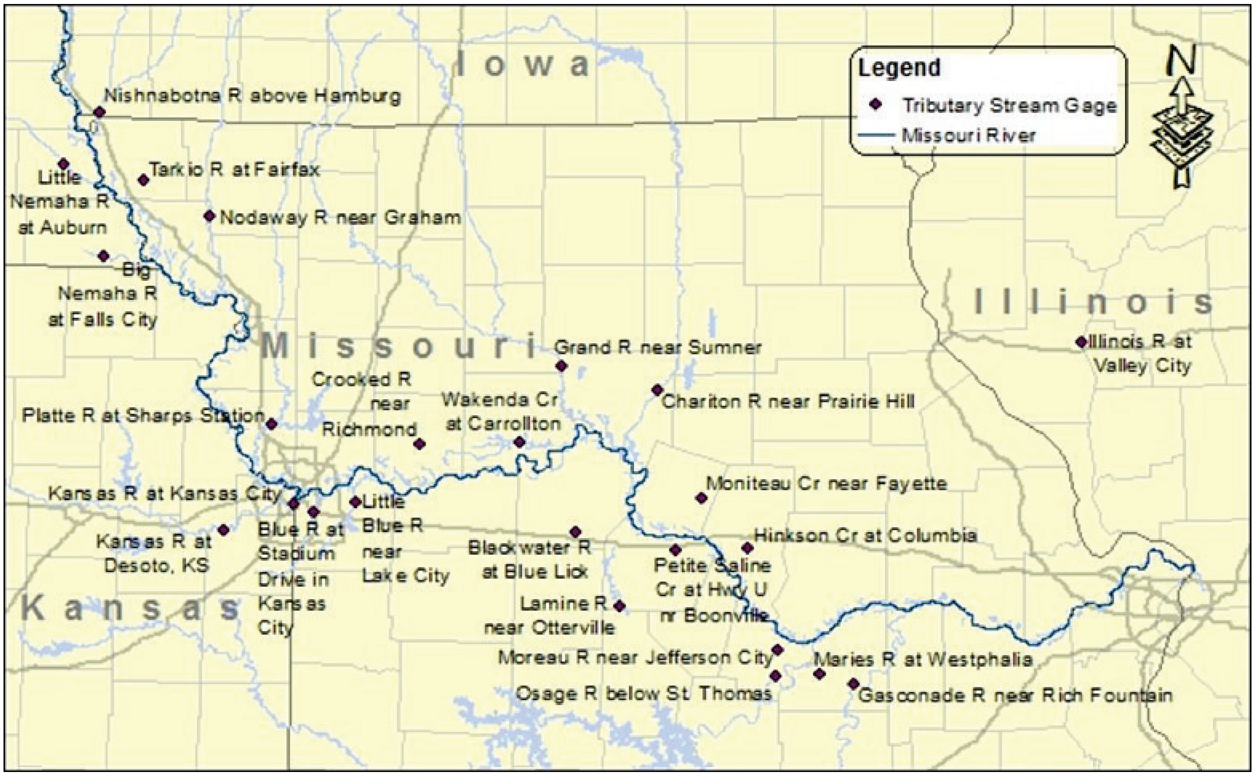

Figure 5. Tributary Gages

Further details on tributary gages can be found in Table 3. Length of record of instantaneous data for the tributaries is generally shorter than for the mainstem gages. Only a few have instantaneous data on record during the 1993 flood.

The Little Chariton River, Loutre River, and Auxvasse Creek were not included. For the Little Chariton and Loutre the USGS has some historical daily records, but there is no recent instantaneous data available. The Auxvasse only recently came online and has a short record. Several stage only gages were also not included, on the Kansas River at Kansas City, KS, the Blue River at 12th Street, and the Grand River at Brunswick, their records are limited and data was difficult to obtain from the operating agency.

Table 3. Tributary Gages

River	Gage Name	River Mile	Instantaneous Data Start Date	Gage Zero Datum (ft) (NAVD88)
Missouri River				
Nishnabotna	Nishnabotna River above Hamburg, IA	542.0	1-Oct-90	894.49
Little Nemaha	Little Nemaha River at Auburn, Nebr.	527.8	1-Oct-90	890.15
Tarkio	Tarkio River at Fairfax, MO	507.6	28-Jun-07	867.97
Big Nemaha	Big Nemaha River at Falls City, Nebr.	494.9	1-May-91	858.51
Nodaway	Nodaway River near Graham, MO	463.0	1-Feb-95	852.38
Platte	Platte River at Sharps Station, MO	391.1	1-Oct-94	754.54
Kansas	KANSAS R AT DESOTO, KS	367.4	1-Oct-90	754.19
Blue	Blue River at Stadium Drive in Kansas City, MO	358.0	1-Jul-02	718.56
Little Blue	Little Blue River near Lake City, MO	339.5	1-Oct-94	719.42
Crooked	Crooked River near Richmond, MO	313.6	1-Oct-07	706.65
Wakenda	Wakenda Creek at Carrollton, MO	262.8	1-Jan-08	641.40
Grand	Grand River near Sumner, MO	250.0	1-Nov-95	631.31
Chariton	Chariton River near Prairie Hill, MO	238.8	1-Oct-94	632.10
Blackwater	Blackwater River at Blue Lick, MO	202.5	1-Dec-02	594.05
Lamine	Lamine River near Otterville, MO	202.5	1-Oct-96	653.16
Moniteau	Moniteau Creek near Fayette, MO	186.5	1-Jul-02	607.99
Petite Saline	Petite Saline Creek at Hwy U nr Boonville, MO	177.5	14-Jun-07	600.17
Perche/Hinkson	Hinkson Creek at Columbia, MO	170.6	8-Mar-07	583.59
Moreau	Moreau River near Jefferson City, MO	138.3	13-Nov-00	546.46
Osage	Osage River below St. Thomas, MO	130.0	1-Nov-95	525.78
Maries	Maries River at Westphalia, MO	130.0	1-Oct-02	542.81
Gasconade	Gasconade River near Rich Fountain, MO	104.4	1-May-95	553.75
Mississippi River				
Illinois	Illinois River at Valley City, IL	200.0	8-Nov-89	417.65

HEC-RAS requires flow data to be in a regular timestep. Because instantaneous data is unreviewed, there are often gaps in recording for various reasons. Sometimes the gaps are small, less than an hour, and sometimes the gaps are large, several days or months. The larger gaps seemed to occur either during the winter, presumably because of ice, or on the rising limb of a hydrograph, possibly due to equipment malfunction or debris damage. For example, Figure 6 shows some of the gaps in flow on the Grand River at Sumner.

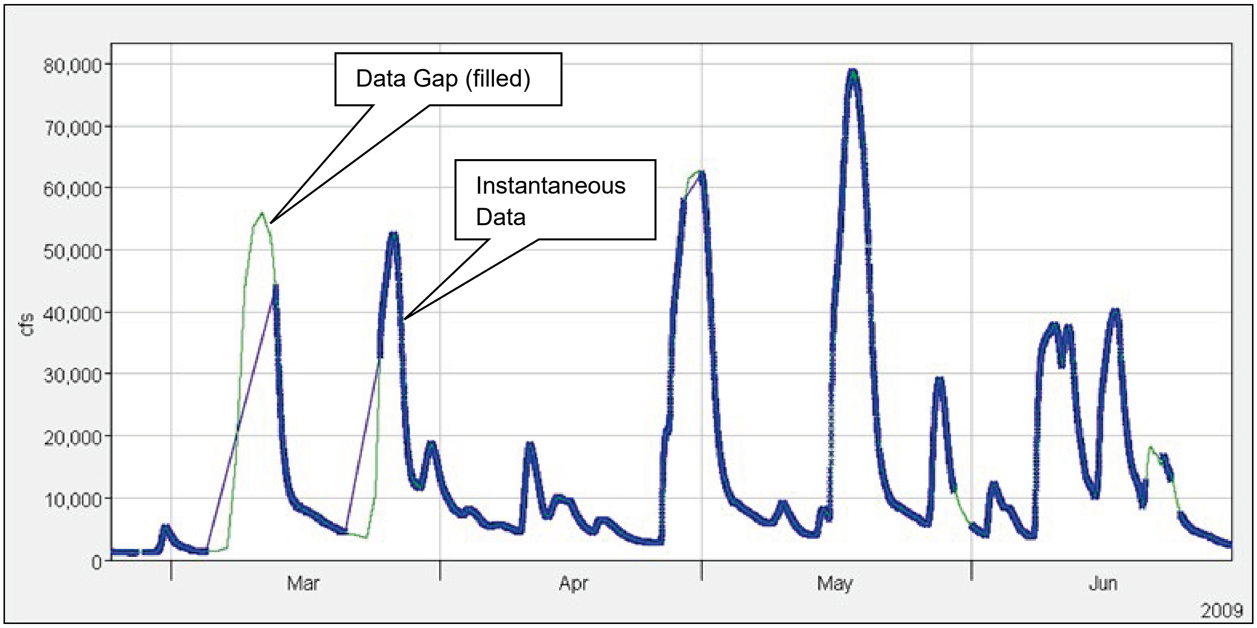

Figure 6. Instantaneous Gaps on the Grand River

Gaps in flow data had to be approximated before the calibration process, otherwise the incorrect boundary conditions would introduce significant errors making calibration difficult if not impossible. If there was stage data available during the flow gap, then the gap was filled by applying a rating curve. If both stage and discharge were missing, the flow data was filled with the daily average assumed as a point value at noon. **Attachment 1** contains more details on the gap fill process. For gaps that were short, less than a day or two, or gaps during a time where the flow was fairly uniform, the gap was filled with a straight line between the two bounding values. It was not necessary to fill gaps in stage data.

Instances of flow gaps occur more often before October 1, 2007. This data was also more difficult to obtain because it is stored on a USGS archive server and is not available online. Pre-2007 records often have unrealistic spikes, for example, a stage of over 1,000-ft recorded for one timestep in the middle of winter when the typical range is 1 to 2-ft. So, although the table shows available instantaneous data back into the 1990s, data before October 1, 2007 could not be used.

All finalized instantaneous mainstem and tributary stages and flows from October 1, 2007 through September 30, 2013 were compiled into one DSS file titled “MoRiverObs.dss”. Stages in this file have been converted to elevation by adding the elevation of the zero gage datum in NAVD88 for the gage. Mainstem gage zero datums are discussed in further detail in the following section.

4.3.2 *Datum Update*

During winter 2012-2013 the Kansas City District conducted work to re-established the elevation references of river gages along the Missouri River to comply with engineering regulation ER 1110-2-8160 (2009). This regulation specifies that inland flood risk management, navigation and water control systems are to be accurately referenced to the National Spatial Reference System (NSRS) and NAVD88. Survey work included determining the reference datum for each gage and identifying the benchmarks used to establish its datum. If no benchmarks were found at a gage, one was established. At least one benchmark at each gage was tied to the NSRS.

The spreadsheet in **Attachment 2** was developed to document the calculations of the revised gage zero in NAVD88. In cases where more than one reference mark is listed for a gage, the new calculated zero datum was calculated as the average. Gage zero elevations selected for use on the unsteady model are highlighted in yellow. Coordination was initiated with local USGS and NWS offices in November 2013. The Missouri USGS was conducting a similar effort simultaneously and most of their newly posted datums agree closely with the USACE effort. Further coordination may be necessary at the Rulo USGS gage and at NWS gages, which have not been updated to NAVD88 online at this time.

5 MODEL DEVELOPMENT

Model development includes software version, geometry components, and boundary conditions.

5.1 HEC-RAS

HEC-RAS Version 5.0 should be used to run the model. During the development of this model, ongoing updates were being made to the HEC-RAS software. As new 4.2 Beta versions were made available, the model was migrated to the newest version, and trial runs of this model contributed feedback to software updates. It is not recommended that the model be run in any version of 4.2 Beta, 4.1 or earlier versions of HEC-RAS. Specifically, the Navigation Dam rules were updated to improve calculations at the Mel Price Lock and Dam. If the model is run in 4.1 or some versions of 4.2 Beta it will not compute the same stages and flows on the Mississippi River. In addition, lateral structures cannot start after the first cross section of a reach in 4.1, whereas this is allowed in 5.0. Several lateral structures in the model start after the first cross section in a reach.

A timestep of 10 minutes was used because it was determined to be the most stable for the cross section spacing on the mainstem Missouri, and also stable for lateral structure computations.

5.2 GEOMETRY

Geometry features incorporated into the model include cross sections, storage areas, lateral structures, storage area connections, bridges, culverts, and inline structures. In total there are 2,206 cross sections, 1,051 on the mainstem Missouri and the remainder on major tributaries and the Mississippi River. Total number of reaches in the model is 33, with the Missouri broken into 14 reaches because of junctions at 13 tributaries confluences. There are 338 total storage areas, 342 storage area connections, and 354 lateral structures. Two bridges and two culverts were modeled near the Missouri Mississippi confluence. Four inline structures including the Water One Weir on the Kansas River, the unmanaged Lock and Dam on the Osage River, Melvin (Mel) Price Lock and Dam 26 on the Mississippi, and Chain of Rocks on the Mississippi were also included.

The following sections outline the details of the model construction including fundamental assumptions, data sources for specific geometry features, techniques used, and justification for any unique parameters and decisions made during the process of building the model.

5.2.1 *Vertical Datum and Projection*

The Vertical Datum for the NWK unsteady HEC-RAS model is NAVD88. The projection is NAD 1983 UTM Zone 15N (US_Feet). Re-projection to a nation-wide projection may be necessary after review and certification for compatibility with other HEC-RAS models and the Res-SIM models that are in different UTM Zones. Re-projecting a HEC-RAS model to a national projection is not difficult or time consuming, and there is a documented How-To procedure provided by HEC. However, the master version of the model will remain in UTM because this is the most useful projection locally and matches with important data sources such as the terrain data and the NLD.

5.2.2 *Rivers*

The configuration of the HEC-RAS river centerlines are shown in Figure 7. Fourteen of the twenty-two tributaries with gage data listed previously in Table 3 were modeled as reaches of river with cross sections in HEC-RAS. Of those, three are upstream of Rulo in the Omaha district's portion of the model. Tributaries will be discussed further in Section 5.2.11.

The centerline of the Missouri River in the model is the recommended route line maintained by Kansas City District Geospatial Branch (ED-S), which is also the recommended sailing line for navigation. Generally it matches the location of the thalweg. Vertices of the recommended route line have been calibrated to match the 1960 river miles, however the true distance of the line does not exactly measure to each mile. For tributaries, the centerline was digitized by hand based on the river location in the terrain surface, as there are no recognized standard stream alignments.

Figure 7. HEC-RAS Modeled Reaches

5.2.3 *Cross Section Geometry*

The base geometry for the current model is the Missouri River Floodway Model, created in 2007 for the purpose of modeling the 100-year water surface profile. New cross sections were not cut because at the initiation of the project in 2011 the 3-meter LiDAR surface described in Section 4.2.1 was not yet available. The source of the cross section station-elevation data in the floodway model was the UMRSFFS DTM described in Section 4.2.4. This was considered a reasonable assumption because in the Kansas City district the overbanks have not changed significantly in the last 10-15 years.

Early development of the model geometry was done in conjunction with the National Weather Service (NWS). The NWS extended many of the cross sections in the extreme overbank using elevations from the 2009 10-meter NED. The original floodway model at that time was in NGVD29 and the cross sections were extended with a surface in NAVD88, however, since this was in the extreme overbank and the difference between datums is at the most 0.3-ft, the impact on calculations was thought to be negligible. Cross sections were also trimmed at levee centerlines. During this process of extension and clipping the ratio of cross section to cut line length was not maintained. This has been addressed and repaired to some extent, however the georeferencing of many of the cross sections is still shifted slightly to one direction or the other. The NWS also re-named all the cross sections to match measured river mile along a centerline that did not match the recommended route line or the 1960 river miles. Cross sections have since been re-named to match the accepted 1960 river miles, which is still the current convention for locating features along the river present day.

Elevations in the channel were replaced by the Corps with elevations from the 2009 hydrographic surveys discussed in Section 4.1. At the time the cross sections were merged, the 2009 hydrographic surveys were in NGVD 29, consistent with the model datum. After this process was complete, the NWS converted the vertical datum from NGVD 29 to NAVD 88 using a HEC grid file, with a range of adjustment of approximately 0.0-0.3-ft.

Parameters for calculating the cross section hydraulic tables (HTab) were set to increments of 1-ft with 100 points for all mainstem cross sections. On the tributaries, the increment was generally set to 0.5-ft also with 100 points, unless the maximum water surface elevation exceeded the highest point. Then the increment was set large enough that the computations would not exceed the top of the curve.

For ease of viewing, gage locations have been labeled at the cross section which most closely matches the location of the gage. These can be viewed in the HEC-RAS model. In the profile plot dialogue, under options, landmarks, put a check next to node names.

5.2.4 *Mannings N-values*

Prior to calibration, n-values were set for the channel and the overbanks that reflect current land use. Land cover assessment was based off current Google Earth aerial imagery, which for the most part is reflective of 2011-2012 summertime conditions. Assigned n-values are listed by land cover type in Table 4, and accompanying pictures are in **Attachment 3**. Final n-values after flow and seasonally varied roughness factors were applied are also listed, and are within a reasonable tolerance to the original values.

Table 4. Manning's n-value ranges

Land Cover	Assigned n-value range	Final n-value range after applying calibration factors
Channel – mainstem Missouri River	0.020 – 0.030	0.017 – 0.030
Channel – tributaries	0.025 – 0.040	0.025 – 0.040
Channel – chutes	0.028	0.023 – 0.035
Overbank – grass/pasture/crops	0.04 – 0.05	0.033 – 0.062
Overbank – light to dense trees	0.07 – 0.15	0.058 – 0.16

5.2.5 *Navigation Structures*

Navigation structures were represented in the model by permanent ineffective flow areas in the channel. Structures represented include dikes, sills, and revetments, but does not include dike notches at this time. Cross section spacing is half a mile, which means most of the time cross sections are not located directly on a structure. Spacing of structures varies because they are location specific and every bend of the river is different, but typical dike fields usually have structures spaced about 500 to 1,000-ft apart. To represent the influence of navigation structures an estimated line of influence was drawn from tip to tip of the structures, as shown in Figure 8. Permanent ineffective flows were set where the line of influence intersects the cross section, as shown in Figure 9.

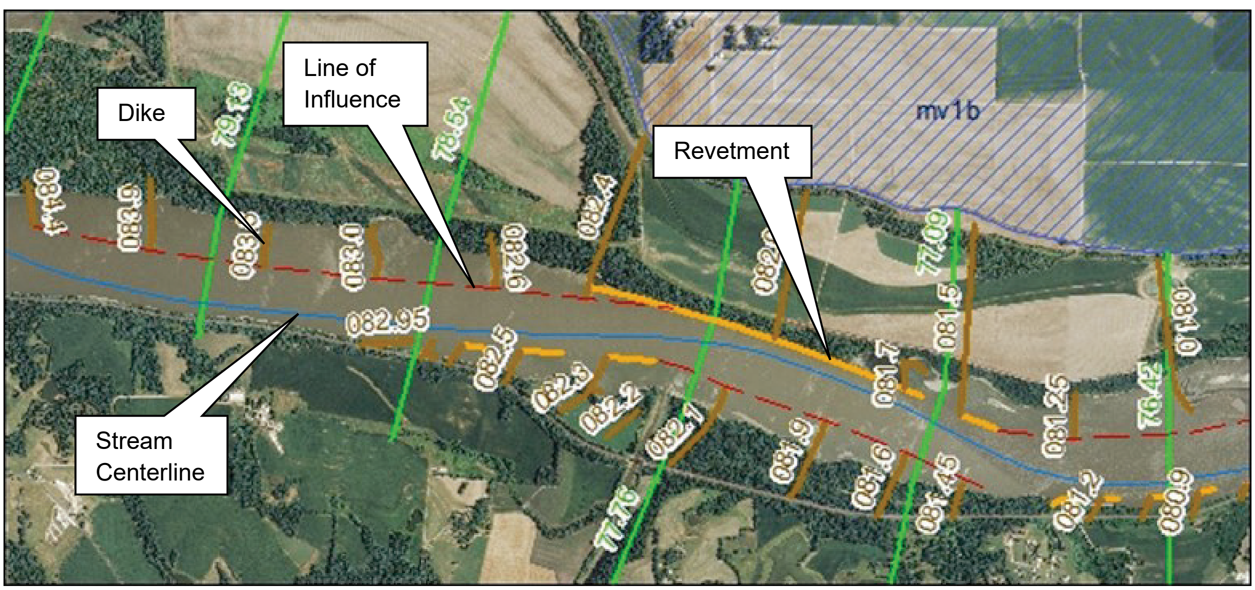

Figure 8. Navigation structure line of influence

Representing navigation structures offers the ability to evaluate approximate stage impacts as a result of alternatives that may include modifications to navigation structures, such as comprehensive or reach based lowering. However, there are several major limitations to modeling navigation structures to as permanent ineffective areas in a one dimensional model. First, the model is not valid for making conclusions on a without structure river, historic or otherwise. Looking into the past, the river and floodplain have changed significantly in form and location; model geometry as well as calibration would have to be revised to reflect that condition. Data to validate calibration for a historic condition is sparse and vague at best, or would require digitizing and importing geometry data from the desired time period. Looking into the future, the geometry of the banks and channel bed is fixed in the model, and is not able to account for scour or deposition interactions between the structures and river. If modifications to structures are significant enough that local sediment processes may be impacted these would need to be evaluated separately. For example, if dikes were removed from the model, the fixed bed boundary would likely become an invalid assumption as deposition in the main channel would be expected. Second, a one dimensional model cannot capture the intricacies of flow and around a navigation structure, which is complex and in multiple dimensions. A two dimensional model would be necessary to evaluate possible habitat in dike fields.

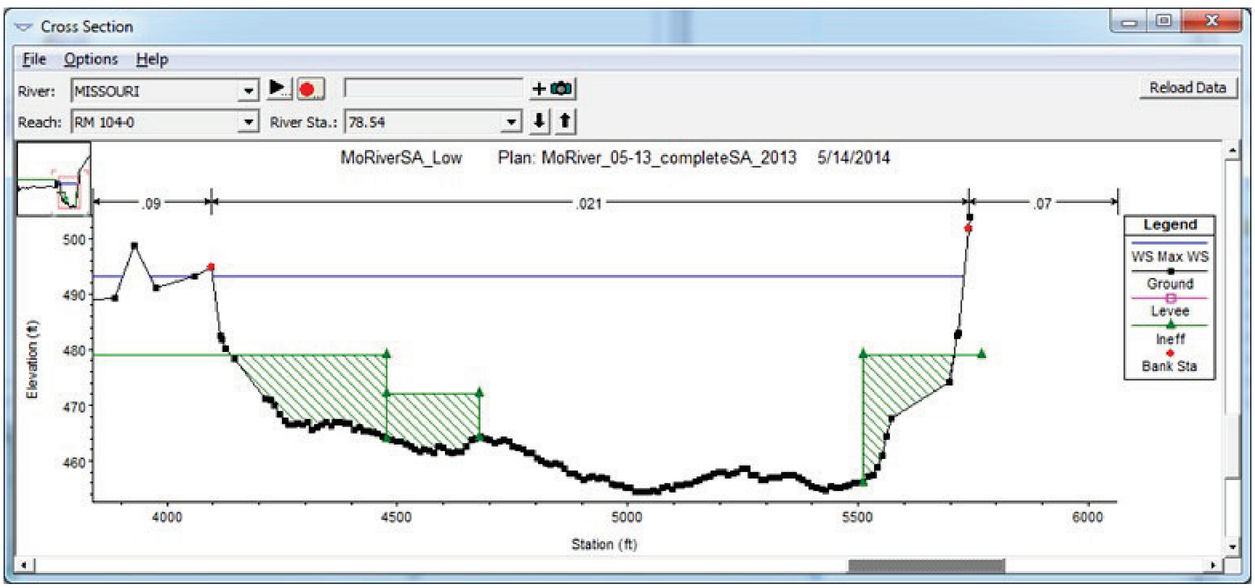

Figure 9. Navigation as permanent ineffective flow areas

Navigation structures were input into the model by contractor CDM Smith. Early in the process CDM Smith used a 100-mile test reach of the river to compare various methods for incorporating the navigation structures, including permanent and temporary ineffective flows and vertical and horizontal variation in manning's n. CDM also performed sensitivity testing to top of structure elevations and compared RAS calculated velocities for each method to Acoustic Doplar Current Profiler (ADCP) velocity data. The resulting technical memorandum is included in **Attachment 4**. From this test reach it was determined that permanent ineffective flows best match the ADCP velocities across the channel, with the added benefit of being a more user friendly tool than manning's n variation. A limitation of the CDM analysis is that while alternative methods to model the navigation structures were tested, the model was not recalibrated to match observed profiles for each alternative. The method selected was deemed reasonable for this study, but it must be acknowledged that the method may overpredict the influence of the dikes as the expansion and contraction occurring between the structures is not accounted for given the level of detail in the model. If more detailed dike modeling were to be conducted, the likely effect could be a shortening of the structures to account for the expansion and contraction between structures and a small increase in channel roughness to recalibrate. This navigation structure limitation should be considered in more detail if specific alternatives are formulated for dike modifications.

Elevations for top of structure were set to the elevations recorded in the 1994 Missouri River Hydrographic Survey (hydro-survey) book because it is the most comprehensive interpretation of dike modifications and adjustments up to 1994. Elevations in the 1994 hydro-survey book account for uniqueness of bends and specific reaches of river that were intentionally not set to design criteria. Another method for selecting structure heights would have been to set them to the design criteria with reference to the most recent CRP elevations, as set by the river section in 2010. The fundamental assumption with this method is that every dike top matches the newest CRP and design criteria, which is true for the Kansas City reach, but probably not for the rest of the river. The ultimate source for up to date structure elevations is stored in the Improvement and Erosion (IMERO) database, and is kept up to date by the Kansas City District River Engineering Section. However, this would involve incorporating construction records for each structure into the 1994 hydro-survey configuration, which would be a considerable effort and was beyond the scope of this project.

Major construction activities that the 1994 hydro-survey book does not account for are 1) notch cutting since 2000, 2) comprehensive lowering and notching of the Kansas City reach dikes in 2004 and 2009 in response to degradation, 3) sill extensions and raises associated with mitigation projects, and 4) post 2011 rehab, which would have used the design criteria with respect to 2010 CRP.

5.2.6 *Storage Areas*

Areas of the floodplain that are protected by levees are represented in the model with a HEC-RAS component called storage areas. This does not in any way imply that these areas were designed to store water, but it is the numerical method used to account for what happens when a leveed area floods. A storage area is an area that takes water away from the main flow in the river. In HEC-RAS it is visually represented with a polygon and numerically represented by an elevation-volume curve. At each timestep, HEC-RAS calculates the elevation of water in the storage area.

A limitation of using storage areas to represent leveed area is that RAS assumes the entire storage area has the same water surface elevation, which may or may not be true depending upon the flooding scenario. As soon as the storage area starts taking on water, HEC-RAS will immediately show the lowest elevation areas as flooded, whether or not it is hydraulically connected to the source of flooding. Despite this simplification, this is the best method available to account for water out of the channel in the lower Missouri River floodplain, which is divided and isolated into areas that flood somewhat independently from each other at varying levee overtopping elevations and/or because of breaches.

Elevation-volume curves for the storage areas were calculated in Geo-RAS (version 10.0), which incorporates the storage area slicer created by Don Duncan of MVS. Parameters were left at the default values of 20 slices and slice density of 0.2. The 3-meter LiDAR surface was used in calculating the elevation-volume for all storage areas with the exception of three storage areas that extended beyond the limits of the LiDAR surface and were instead created from the 10M NED.

Naming convention for the storage areas is a shortened version of the levee unit name. Names were limited to 2-3 characters, and where multiple storage areas had the same name, either numbers or letters were added to identify unique areas.

Figure 10 shows the storage area configuration near Waverly, MO. This area would be especially difficult to model without the use of storage areas because there is no full valley cross section configuration that can adequately represent the way this area floods during large events. For example, there is a NLD levee separating the storage areas labeled blt2 and sug1. The high terrace separating blt2 and blt4 is lower than the levee, so when flooding in blt2 gets high enough it floods sug1 from the back side, in addition to flooding blt5 and spilling into the Wakenda Creek.

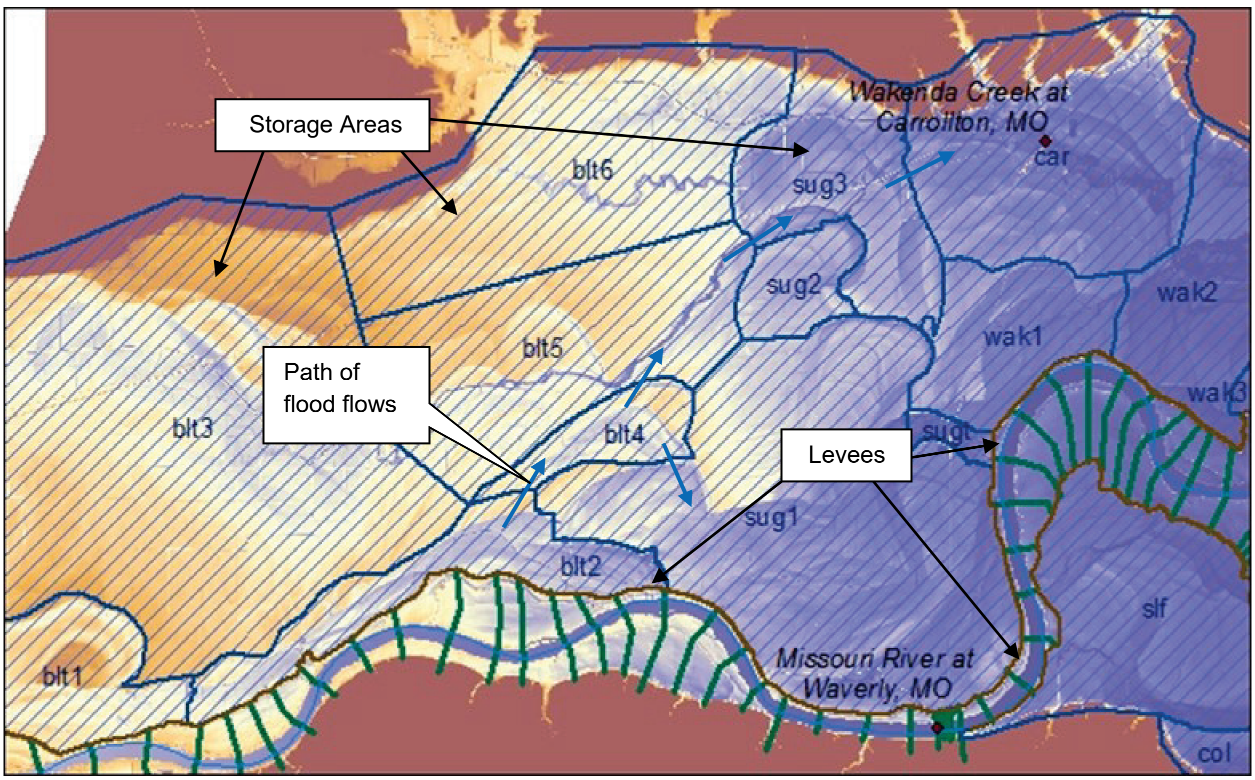

Figure 10. Wakenda Storage Areas

Another area that is especially difficult to model correctly with full valley cross sections is at Rulo, Nebraska. Figure 11 shows the storage area configuration at Rulo. The storage area labeled hlt1 contains the community of Big Lake, MO. During the 2011 flood, and during multiple previous floods, the levee protecting this area overtopped/breached, flooding this area.

State Highway 159 and a railroad embankment are the boundary between storage area hlt1 and storage areas hlt2 and rul5. Flood waters from hlt1 return to the river by flooding the road and railroad, bypassing the Rulo gage completely. The stage readings at the Rulo gage during these floods are therefore difficult to match. Storage areas more effectively model this area than do full valley cross sections.

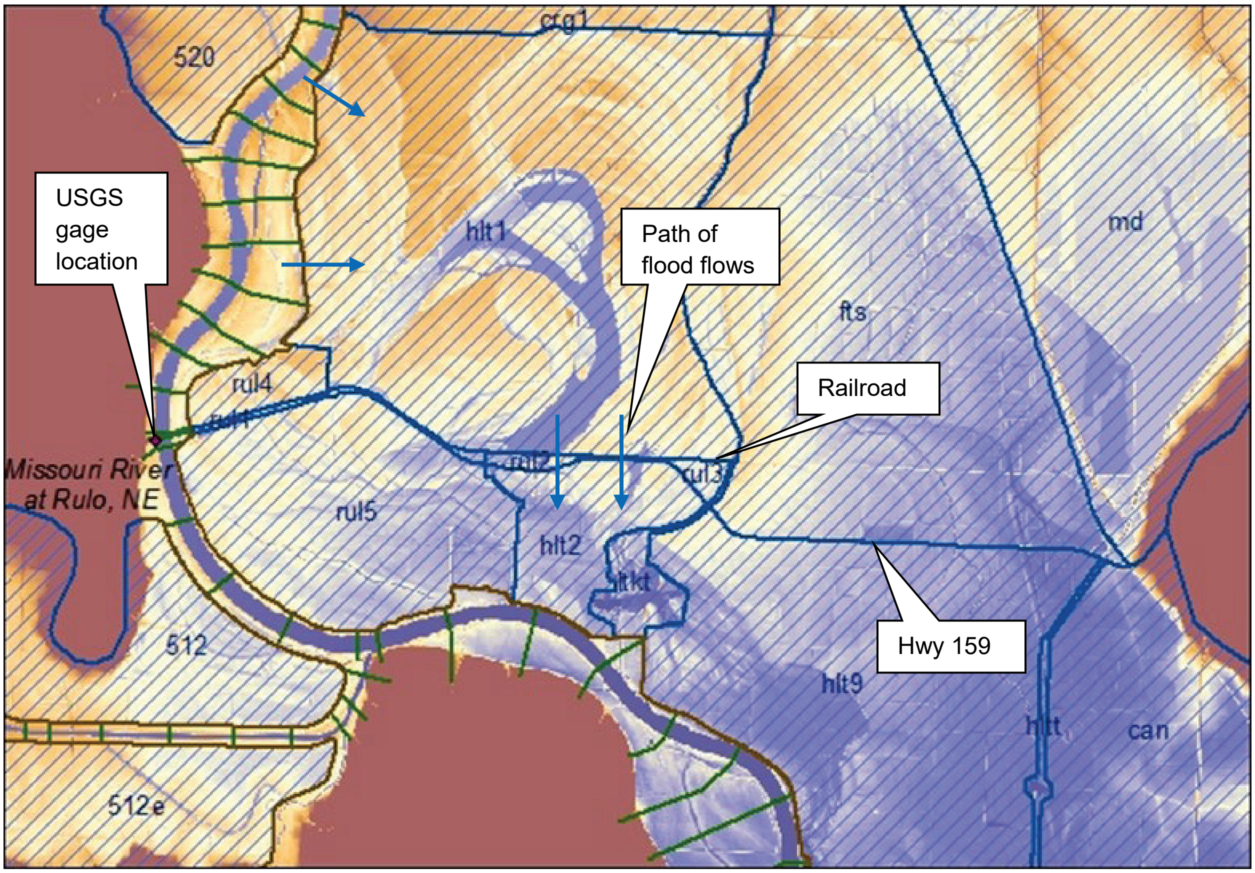

Figure 11. Rulo Storage Areas

As shown in Figure 10 and Figure 11, all storage areas were drawn back to high ground at the bluffs for more complete mapping. The user elevation option was selected in Geo-RAS, which allowed the elevation-volume curves to be capped to a reasonable elevation. Generally, a user elevation was selected that corresponded to about 10-ft higher than the 1993 high water marks.

Tiebacks and tributaries that are not represented by a river reach with cross sections were modeled as storage areas, for several reasons. This allows for overtopping/breaches from Missouri River backwater on tiebacks, which historically has happened fairly often on the lower Missouri River. This also allows for tieback flooding to be included in mapping, which will make for a more complete picture of inundation in the floodplain. Modeling the tiebacks as storage areas also allows for the ability to plug in a flow hydrograph as input into the storage area. This was not done for this modeling effort, but it is important to have this capability for modeling in more detail events such as the flood in June 2013, in which several levees flooded from large local inflows on tieback creeks, rather than from the mainstem Missouri River. In addition, tieback storage areas also add to the overall storage potential of the river in high water events that are large enough back water up into tributaries but don't necessarily flood many levees.

All storage areas in the model are stored in a common shapefile called MoRiverSA_Master. This master shapefile contains important information such as the full feature name for areas that are in the NLD protected area shapefile, as well as the node names. A master shapefile was necessary because the storage areas were created for HEC-RAS in several different Geo-RAS batches. All changes to storage area configuration in the model were first made to the master shapefile, and then copied into Geo-RAS for processing to RAS. The master shapefile should match exactly the storage areas in the model, and it is the intention that this file be maintained with any future modifications to the model.

5.2.7 *Lateral Structures*

Levees are modeled in HEC-RAS using a tool called Lateral Structures. Lateral structures are the connection between cross section flow in the river, and flooding in the storage areas. At each timestep the water surface elevation in the river is projected onto the lateral structure elevations, as shown in Figure 12. If the water surface profile is higher than the lateral structure the weir equation is used between each two points on the lateral structure to calculate flow over the levee.

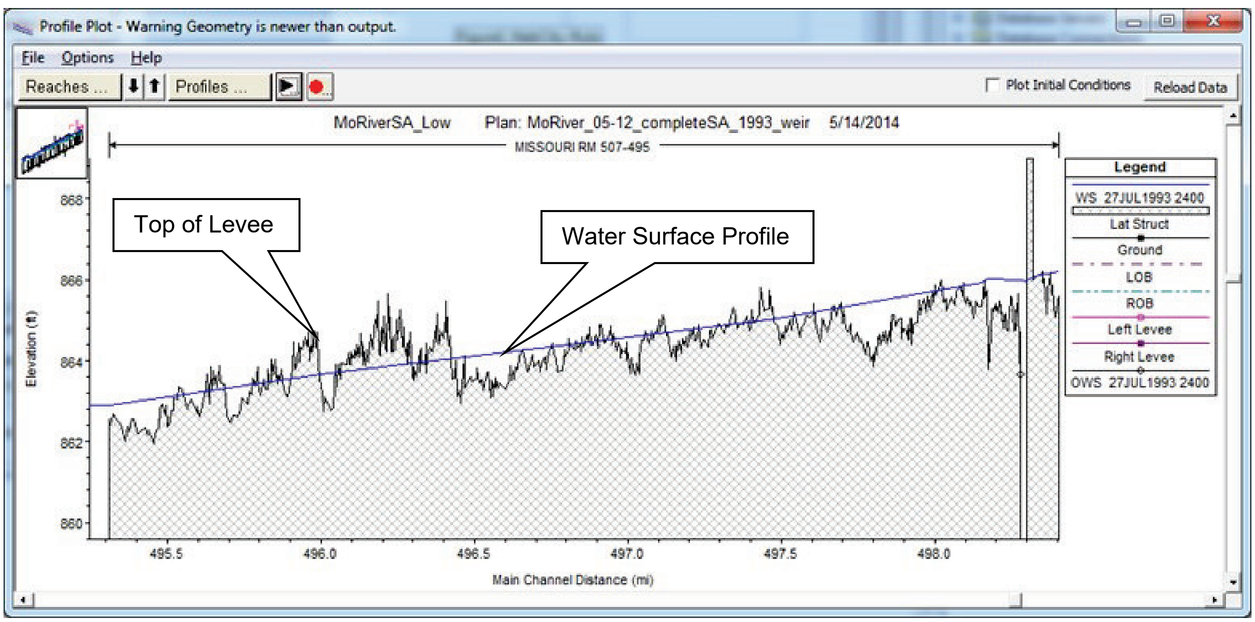

Figure 12. Lateral Structure

Federal levees and levees in the PL 84-99 program have lateral structure elevations from the NLD. All other lateral structures including non-program levees, high ground, and connections to tributary backwater areas, have elevations that were obtained in Geo-RAS by draping a digitized centerline over the 3-meter LiDAR. Cutting lateral structures from a DEM introduces some uncertainty to the model, in part because of the accuracy of the 3-meter DEM, but more so because it is difficult to sketch a line exactly on the high ground along the entire length of the lateral structure. It is possible that lateral structures cut from the LiDAR have spikes that are lower than actual elevations. There was no adjustment made because it was difficult to quantify the exact discrepancy.

Small and narrow non-program levees were modeled with levee points and ineffective flows. The theory behind this methodology is that until a certain elevation, the area behind the levee is dry. When the levee points overtop it may take some time for water to fill the protected area, it will pull water from the river rather than act as downstream conveyance for a time. Therefore, ineffective flow elevations are set 1-ft higher than the levee points. This way, as the water rises above the levee points, the ineffective area will act as storage, and when water gets above the top of ineffective the pre-processor will transition from storage to full conveyance. The difficult part in using this methodology is setting the levee points at consistent elevations, so that flow is not confined in one cross section and unconfined in the next.

Weir coefficients of 2 and 0.3 for levees and tiebacks, respectively, were used. This is lower than the typical broad crested weir coefficient of 3 because levees are parallel to flow rather than perpendicular and in the flow path. Weir coefficients are explained in further detail in the next section. A weir width of 10 was used for all lateral structures, which is for display purposes only as the weir equation is dependent only on the length of the weir, the depth of overtopping, and the weir coefficient. Weir width does not impact calculations.

User specified intersections were used for most levees in the model. User specified intersections describe the stationing along the levee at which the cross section intersects. This was important because often times the length of the levee does not match the reach length between cross sections and without the user intersections the water surface profile would not be correctly projected onto the levee. User specified intersections were calculated automatically in GIS using a linear routing tool and manually copy pasted into HEC-RAS.

All lateral structures in the model were filtered to 200 points or less. HEC-RAS allows a maximum of 500 points to describe the station elevation data of a lateral structure. However, at every timestep the model calculates the weir equation between each two points, so having less points should save time over a period of record model run. When filtering, every effort was made to maintain a level of accuracy that matched the confidence level of the source data. For example, levees that had NLD elevations had an original density of one point about every 100-ft or less and were generally filtered to one data point every 100 to 300-ft, depending on overall length. Lateral structures cut from the LiDAR generally have less confidence, because while the surface has a good accuracy there is additional error associated with sketching by the exact top of levee location on 3-meter pixels that represent a 10-ft top of levee. Therefore, lateral structures with LiDAR elevations were filtered to 100-points or less. And very short lateral structures that represented backwater connections to tiebacks were generally filtered to between 10 and 50 points. Filtering was accomplished with the tool in HEC-RAS, which minimizes change in weir flow area. Filtered elevations were visually checked to ensure they maintained the insipient overtopping location(s).

Naming convention for lateral structures matches the storage area they attach to. A lateral structure can only put water into one storage area, but multiple lateral structures can put water into the same storage area. When this occurred the lateral structures name was followed by a dash with a unique letter.

5.2.8 *Storage Area Connections*

Storage area connections are the mechanism by which water is transferred between storage areas. A flat water surface elevation from the storage area is projected onto the elevations of the storage area connection and flow is calculated with the weir equation. A storage area connection can represent a number of features such as high ground, additional interior levees, road embankments, railroad embankments, and/or simply at grade. Elevations for storage area connections along program levee tiebacks are from the NLD. All other storage area connections were cut from the 3-meter LiDAR.

Large protected areas behind a levee were split into multiple storage areas if necessary. This was done to prevent the model from allowing flood waters overtopping at the upstream end to immediately fill the leveed area at the downstream end, ultimately short circuiting the reach of river in a few timesteps. Generally, if the protected area had a length along the river of longer than approximately five miles, it was split into multiple storage areas. If the levee did not overtop in 1993 or 2011, the two most recent floods of record, than the protected area was left whole. When there was special anecdotal knowledge of how a leveed area floods, such as at Rulo, Wakenda, and L-471-460, this was taken into consideration so as to most appropriately model the path of flood waters.

Weir coefficients were selected based on the type of flow represented by the storage area connection. This is because water will flow very differently over a levee than over natural high ground or non-elevated terrain. Table 5 shows the recommended weir coefficients (Hydrologic Engineering Center, 2014) for various weir flow components, as well as the selected starting value used in the model.

Table 5. Weir Coefficients

Weir Flow Component	Appropriate Range of Weir Coefficients	Starting value for model
High levee/roadway	1.5 – 2.6	2.0
Low levee/roadway	1.0 – 2.0	1.5
Natural high ground	0.5 – 1.0	0.75
Non-elevated terrain	0.1 – 0.5	0.3

HEC-RAS convention is to draw storage area connections from left to right looking downstream, which is represented by an arrow that is displayed in HEC-RAS. Naming convention is the two connected storage areas separated by a dash. The first storage area listed is the one that orients the arrow pointing to the right when facing the storage area connection.

5.2.9 *Bridges*

On the Missouri River mainstem, cross sections representing bridge embankments are in the model, but the structures themselves are not. This was a simplification made to keep computation time shorter. In addition, all bridge deck low chords on the Missouri River are elevated higher than the floods of record, so the only component that would impede water flow is the bridge columns, which likely have a local effect, but not global. However, two bridges and two culverts were included in the floodplain reach at the Missouri Mississippi confluence called the "Crossover" in the model. This is because these structures are a significant component in accurately modeling flood flows in this area.

5.2.10 *Mississippi River Confluence*

Modeling of the confluence of the Missouri and Mississippi Rivers was accomplished independently by the St. Louis District, and the final model was merged into the Missouri River model. Local expertise was considered essential because there are several complex components to the way in which this area floods. Figure 13 shows the layout of the area in HEC-RAS and identifies several of the important landmarks.

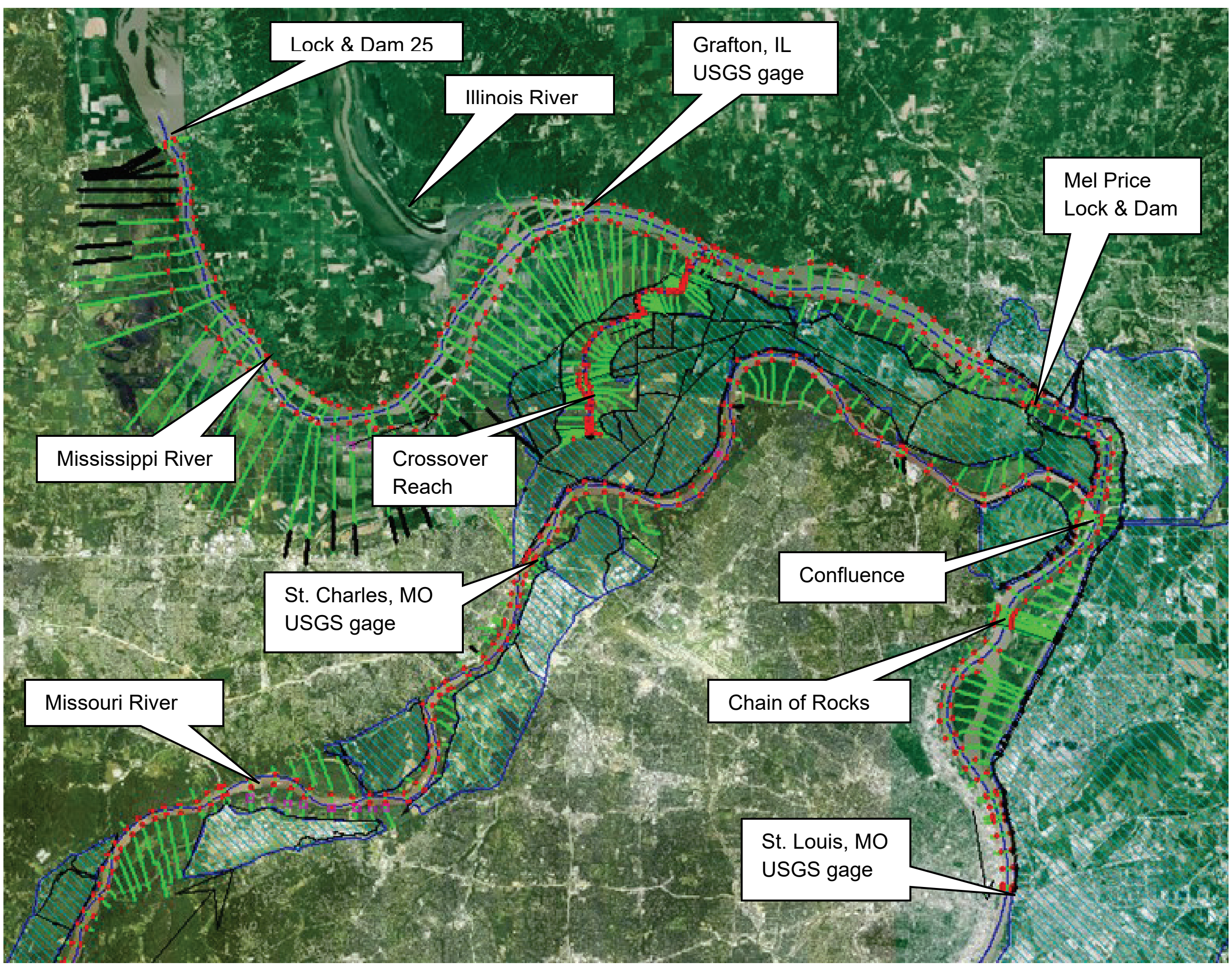

Figure 13. Mississippi and Missouri Rivers Confluence

The crossover reach is in the floodplain and is dry most of the time. During high flood events the Missouri River overflows into the vast network of leveed areas and connections and generally follows the path of the cross section layout of the crossover reach, joining the Mississippi River upstream of Mel Price Lock & Dam. Figure 14 shows the extent of inundation experienced during the flood of 1993. The complex nature of the crossover requires the use of storage areas and connections to adequately model flood flows. It has experienced flood events in 1973, 1993, 1995, and partially in 2013.

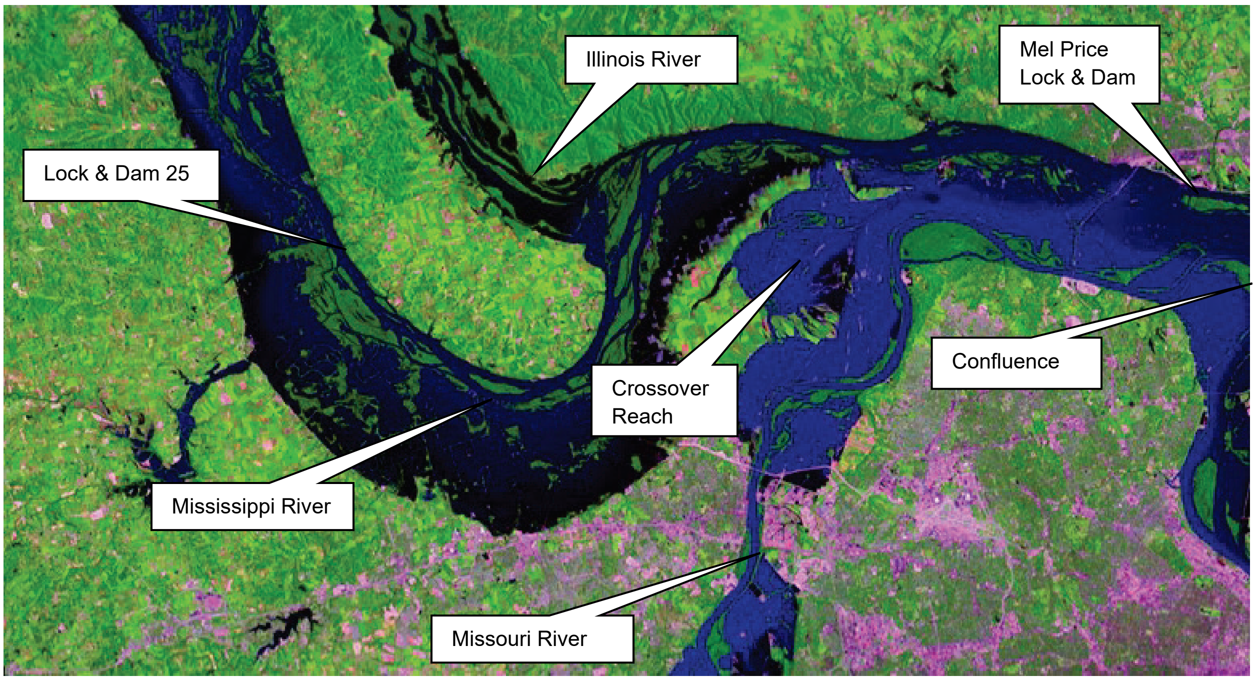

Figure 14. Mississippi and Missouri Rivers Confluence – 1993 Flood

Mel Price Lock & Dam was included in the model using the Navigation Dam operations in HEC-RAS. Flow releases from Mel Price are determined so as to maintain pool elevations at Grafton and at the headwaters of the Dam within certain elevation ranges. Further details on the Mel Price Navigation Dam Operation and the construction data sources and assumptions associated with the confluence model are included in **Attachment 5**.

5.2.11 *Tributaries*

Fourteen tributary reaches are modeled between Nebraska City and the mouth of the Missouri. The upstream boundary of each tributary reach is the most downstream gage on that river, as listed in Table 3. The primary purpose of the tributary reaches is to route flows from the gage to the confluence with the Missouri.

Tributary HEC-RAS models were created independently from the Missouri River model, using a preliminary stage hydrograph from the model as a downstream boundary condition. After construction and review of the model was complete, the tributary models were merged into the main model. Modeled tributaries are listed in Table 6 in order from upstream to downstream. Many of the tributary models were contracted to CDM Smith, and reviewed by the Kansas City

District at several milestones. A few of the larger tributaries were modeled in-house. Review documentation, which includes basic details on the model construction and assumptions, is included in **Attachment 6**.

Table 6. Tributary models

Tributary	River Mile	Model Length (miles)	Model Source	Terrain used for Cross Sections
Nishnabotna	542.0	12	Omaha	Best Available
Little Nemaha	527.8	10	CDM	10-meter NED (2009)
Tarkio	507.6	14	CDM	10-meter NED (2009)/ LiDAR
Big Nemaha	494.9	14	CDM	10-meter NED (2009)/ LiDAR
Nodaway	463.0	29	Kansas City	10-meter NED (2011)
Platte	391.1	25	CDM	10-meter NED (2009)/ LiDAR
Kansas	367.4	30	Kansas City	10-meter NED (2011)
Grand	250.0	35	Kansas City	LiDAR (2006/2007)
Chariton	238.8	20	Kansas City	LiDAR (2006/2007)
Blackwater	202.5	26	CDM	10-meter NED (2011)
Lamine	202.5	57	CDM	10-meter NED (2011)
Moreau	138.3	21	CDM	10-meter NED (2011)
Osage	130.0	34	Kansas City	10-meter NED (2011)
Gasconade	104.4	52	Kansas City	10-meter NED (2011)

Most of the tributary cross sections were created from the 2009 or 2011 10-meter NED because recent LiDAR was not always available. There were a few tributaries for which LiDAR was available within the bluffs of the Missouri River, which covered a few of the most downstream cross sections. The Grand and Chariton Rivers were cut entirely from LiDAR. Terrain sources are shown in Table 6.

For most tributaries, the surface that was used to cut the cross sections did not contain a distinct channel. Therefore a trapezoidal shaped channel was estimated based on comparing USGS measurements of top width and area to stage at the gage location and then all of the cross sections were manually modified to incorporate this channel shape from the gage to the mouth.

Model output was compared to observed stages, flows, and the USGS rating curve at the upstream gage locations. The goal of this effort was not necessarily to have stage calibrated tributary reaches, so this was used for a reality check rather than a calibration tool.

Junction lengths were assumed to represent the average distance that the water will travel from the last cross section in the reach to the first cross section of the following reach (Hydrologic Engineering Center, 2010). Junction lengths entered in the junction editor override the reach length in the cross section editor. For small tributaries entering a large river such as the Missouri, the junction length was assumed to hug the bank, rather than crossing the entire river to join the river centerline, because this better reflects the actual travel path of that water. For the mainstem Missouri, the junction length was measured along the river centerline. An example at the Nodaway River is shown in Figure 15.

Figure 15. Junction Lengths

Model stability on the tributaries when integrated with the greater Missouri River model was especially challenging. Techniques used to improve stability included adding additional cross sections closer to the junctions, using the minimum flow option, and lowering the thalweg of the last few tributary cross sections.

5.2.12 *Constructed River Chutes*

Constructed mitigation chutes are represented in the model by the standard river cross sections. Cross sections at chutes were modified if necessary to reflect appropriate conveyance and top width. If low flow conveyance through the chute is blocked by river structures this was represented with temporary ineffective flow at a consistent profile through all chute cross sections. Chutes constructed after 2012 were not included in the model geometry, but could easily be added if needed. A list of chutes included in the model and basic assumptions is included in **Attachment 7**.

A more refined method of modeling the chutes would be to have split flow reaches at each mitigation site. Additional cross sections would have to be cut, and the most current survey efforts incorporated. Split flow in unsteady HEC-RAS doesn't necessarily make the run time any longer as it does in steady flow, but it would increase and complicate the number of reaches on the Missouri River in the model. Another consideration is that split flows reaches work well for in channel flows, but could add additional errors during out of bank events depending on how cross sections are drawn. Split flow was considered to be too detailed for this project effort; added flow conveyance in cross sections is sufficient.

5.3 Boundary Conditions

Boundary conditions include flows input into the model, observed and ungaged, as well as the selected downstream boundary condition.

5.3.1 *Upstream Boundary Conditions*

Upstream boundary conditions include observed flow hydrographs at Nebraska City for the Missouri River, Lock & Dam 25 for the Mississippi River, and at the upstream extent of each of the fourteen tributary reaches. Figure 16 shows the location of Nebraska City and Lock & Dam 25 with respect to the overall HEC-RAS model layout.

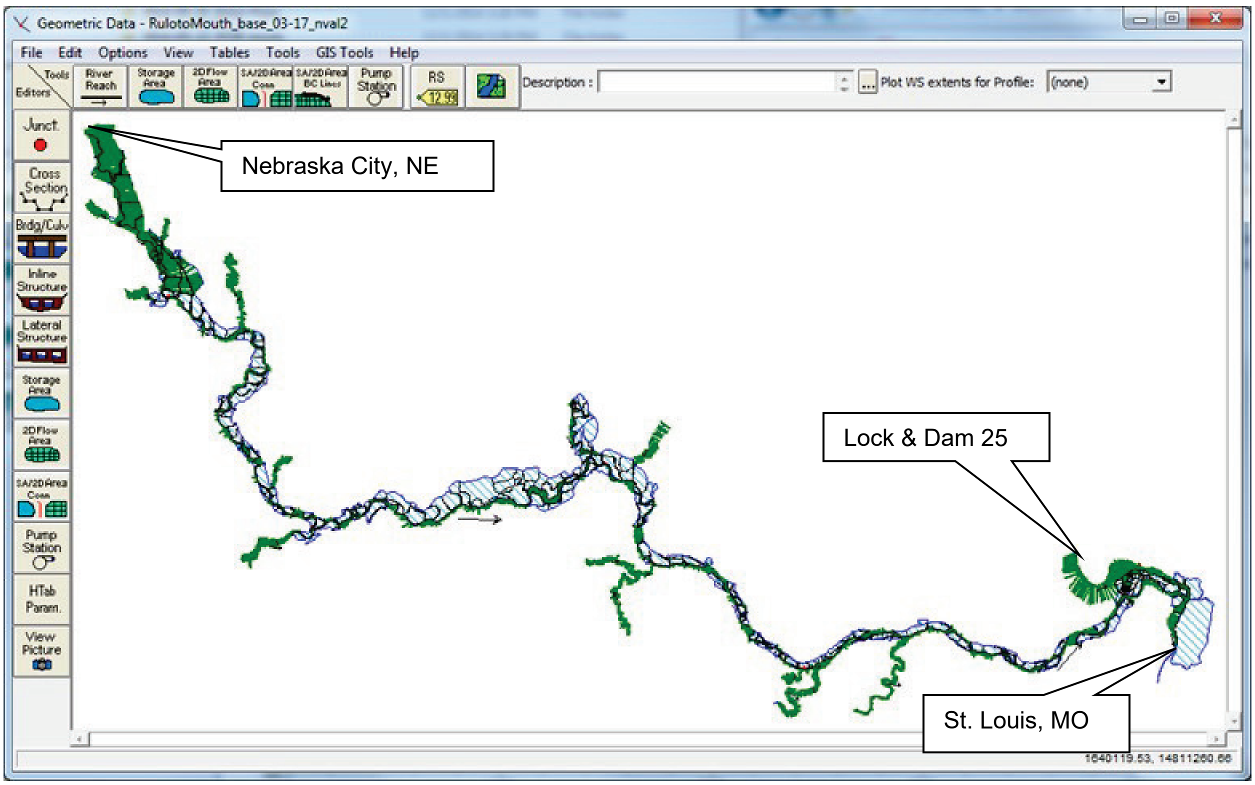

Figure 16. HEC-RAS Layout with Boundary Conditions

Nine additional gaged tributaries that were not included as reaches were input into the model as lateral inflow at the location which they enter the Missouri River. This likely introduces a small source of error in flow timing, as it assumes the flow hydrograph at the confluence is equal to the hydrograph as recorded at the upstream gage. Table 7 lists the tributaries in order of largest to smallest basin area, and identifies which were modeled as reaches and which were input as lateral inflow hydrographs. Not including the Illinois River, which is a tributary to the Mississippi River, the 8 lateral inflows represent approximately 1,400 total square miles or 0.3%

of the Missouri River basin area at St. Charles, small enough that timing errors were considered negligible.

Table 7. Tributary Boundary Conditions

No.	River	Basin Area at Gage (sq mi)	Modeled Reach	Lateral Inflow
1	Kansas	59,756	X	
2	Illinois	26,743		X
3	Osage	14,500	X	
4	Grand	6,880	X	
5	Gasconade	3,180	X	
6	Nishnabotna	2,806	X	
7	Platte	2,380	X	
8	Chariton	1,870	X	
9	Nodaway	1,520	X	
10	Big Nemaha	1,339	X	
11	Blackwater	1,120	X	
12	Little Nemaha	792	X	
13	Moreau	561	X	
14	Lamine	543	X	
15	Tarkio	508	X	
16	Blue	258		X
17	Maries	257		X
18	Wakenda	256		X
19	Little Blue	184		X
20	Crooked	159		X
21	Petite Saline	136		X
22	Moniteau	75		X
23	Perchee/Hinkson	70		X

5.3.2 *Downstream Boundary Condition*

The downstream boundary condition is a rating curve at the St. Louis gage received from the St. Louis District in fall of 2012. Gage zero datum applied to covert stage to elevation is listed in Table 2. The rating curve was approximated by 50 points for input directly in HEC-RAS. For the project purposes, this will allow for stage comparisons at the St. Louis gage between alternatives. If the geometry of the Mississippi River in the vicinity of the St. Louis gage were to be modified between alternatives, this would not be an appropriate choice for a downstream boundary condition. The assumption was made that model geometry modifications for alternatives analysis will be limited to the Missouri River.

5.3.3 *Ungaged Inflow*

Ungaged inflows were applied to the model as inflow boundary conditions in the unsteady flow editor. There are two components to ungaged flows: 1) the scaling of tributary flows and 2) evenly distributed uniform lateral inflows. Rationale for selection of this methodology is in the calibration discussion, Section 6.1.2.

First, flow inputs from all tributaries were scaled up by the ratio of basin area upstream of the gage to basin area at the confluence. Ratios were applied to tributary flow in modeled reaches as well as point lateral inflows. The square root of the basin area ratio was considered as an option, but as this tends to work better for a peak flow analysis, rather than a full range of flows. Table 8 lists the basin area ratio used at each tributary. Ratios were applied as a multiplier in the unsteady flow editor.

Table 8. Ungaged Flow – Basin Area Ratios

River	River Mile	Basin Area at Gage* (sq mi)	Basin Area at Confluence* (sq mi)	Ratio
Kansas	368	60,194	60,544	1.01
Osage	130	14,626	14,736	1.01
Grand	251	6,923	7,883	1.14
Gasconade	104	3,189	3,574	1.12
Nishnabotna	542	2,819	2,976	1.06
Platte	391	2,371	2,440	1.03
Chariton	239	1,889	2,370	1.25
Nodaway	463	1,516	1,794	1.18
Big Nemaha	495	1,342	1,922	1.43
Blackwater	203	1,118	1,543	1.38
Little Nemaha	528	793	894	1.13
Moreau	139	563	583	1.04
Lamine	203	546	1,111	2.04
Tarkio	508	479	514	1.07
Blue	358	255	270	1.06
Maries	130	257	293	1.14
Wakenda	263	257	344	1.33
Little Blue	340	195	225	1.15
Crooked	314	159	350	2.20
Petite Saline	178	141	239	1.69
Moniteau	187	75	148	1.98
Perche/Hinkson	171	70	401	5.75

** Basin areas were calculated in GIS using USGS HUCs and delineation by hand to the gage location and the confluence if necessary, see **Attachment 8** for maps. Calculated areas are similar to the areas listed in Table 7, but may not match exactly.*

Second, a uniform lateral inflow was added between gages. The amount of uniform lateral inflow varies by reach and varies on a monthly basis. All ungaged timeseries are in a DSS file titled “Ungaged.dss”. To calculate the amount of uniform lateral inflow, the observed flow records for all mainstem and tributary gages were converted to monthly average flows. Then, the ungaged amount was computed by subtracting the downstream gage minus the upstream gage and minus the gaged tributaries that enter in between. Specifics are laid out in Table 9. Location of inflow was selected as the reach in HEC-RAS which roughly corresponded to the area which would receive the most ungaged inflow. The multiplier was a calibration tool, used to match overall flow volume.

Table 9. Ungaged Flow – Uniform Lateral Inflows

Name of Reach (Part F in DSS)	Evenly Distributed Uniform Lateral Inflow Location	Multiplier	Tributaries
NECITY - RULO	527.55 - 507.90 (Little Nemaha - Tarkio)	0.25	Nishnabotna + Little Nemaha + Tarkio
RULO - STJOE	494.19 - 463.98 (Big Nemaha - Nodaway)	0.75	Big Nemaha + Nodaway
STJOE - KC	448.15 - 391.92 (St. Joseph - Platte)	0.5	Platte + Kansas
KC - WAV	366.06 - 293.22 (Kansas City - Waverly)	0.85	Blue + Little Blue + Crooked
WAV - BOON	238.52 - 202.97 (Chariton - Lamine)	0.4	Wakenda + Grand + Chariton + Blackwater + Lamine
BOON - HERM	129.29 - 105.21 (Osage - Gasconade)	0.1	Moniteau + Petite Saline + Hinkson + Moreau + Osage + Maries + Gasconade
HERM - STCH	97.84 - 28.1 (Hermann - St. Charles)	0.8	none

6 CALIBRATION

Model calibration was accomplished through several sequential steps. Results were compared to observed data and evaluated numerically and anecdotally. Model performance, primary sources of uncertainty, and model improvement will also be discussed in this section.

6.1 MODEL CALIBRATION

Unlike previous modeling efforts on the Missouri river, which concentrated on flood flows, a broad spectrum of flows from the record lows to the record highs are considered important to the project purposes. Calibration methods had to include a full range of flows. The primary source of calibration data was observed stage and flow hydrographs at the mainstem Missouri river gages. More weight was given to the USGS gages over NWS gages as the collection method is usually more precise and regular.

6.1.1 *Step 1 – Geometry and Boundary Conditions*

Step 1 in calibrating the model was to get the geometry and boundary conditions right. This could easily be considered the longest and most time consuming step in calibration. Boundary conditions were evaluated and flow gaps repaired. A thorough check of cross section geometry to minimize errors in computation was performed. This included revising bed data, navigation structures, ineffective flow areas and levee points to minimize large transitions or spikes in conveyance area, velocity, and top width at various flows and depths. Storage areas, connections, and lateral structures were input into the model, checked for performance and adjusted or re-drawn to better match to actual knowledge of flood scenarios. Tributary n-values were adjusted from those originally selected so as to better time the calculated to the observed hydrograph at the next downstream gage.

6.1.2 *Step 2 – Ungaged*

Step 2 in calibrating the model was to develop a simplified method of calculating ungaged inflows that could be applied over 6-yrs of calibration data and also applied to the 82-year period of record. This was necessary because without accounting for ungaged inflows the disparity between observed and calculated inflows is large on the lower reaches of the model. Refer to Table 10 for the percent of ungaged area and the impact on overall flow volume. For example, at St. Charles Missouri, 12% of the basin area in the Kansas City District is ungaged, resulting in missing 13% of the total flow volume. This is a significant amount and needed to be addressed before Manning's n values could be evaluated.

Table 10. Percent Ungaged

Gage Name	River Mile	Basin Area in NWK (sq mi)	Incremental Ungaged Area (sq mi)	Cumulative Ungaged Area (sq mi)	Percent Ungaged (%)	Missing Volume (%)
Rulo, NE	498.0	0	0	0	0%	-0.4%
St. Joseph, MO	448.2	11,600	1,827	1,827	2%	-3.2%
Kansas City, MO	366.1	69,200	1,337	3,164	3%	-3.5%
Waverly, MO	293.2	71,000	1,409	4,573	4%	-6.5%
Boonville, MO	196.6	85,800	3,763	8,336	8%	-10.7%
Hermann, MO	97.9	107,600	2,891	11,227	10%	-9.6%
St. Charles, MO	27.8	109,100	1,442	12,669	12%	-13.0%

Figure 17 is a visual representation of the ungaged basin areas on the Missouri River. Basin areas accounted for by stream gages are in blue, while red and yellow areas are ungaged. Red represents basin area that drains to one of the modeled tributary reaches, and yellow represents the remaining ungaged areas. Reach by reach maps are included in **Attachment 8**.

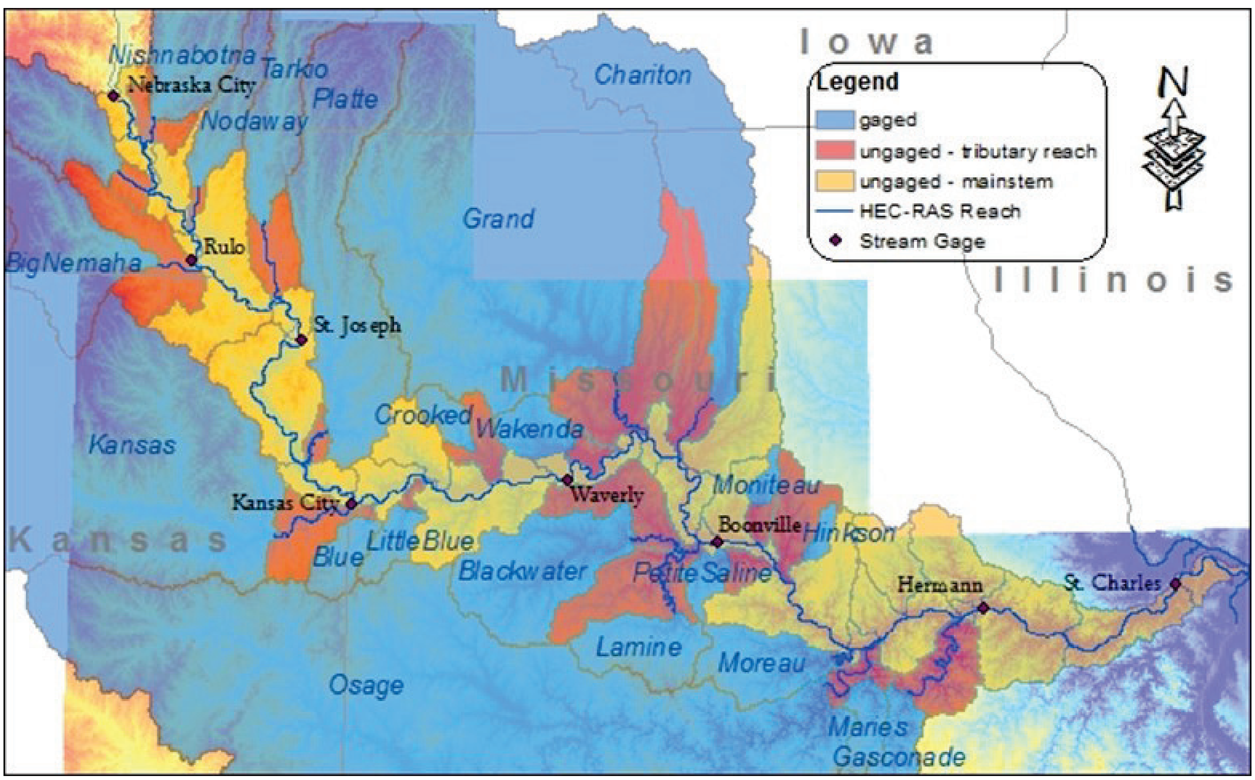

Figure 17. Ungaged Areas

Several different methods of accounting for ungaged inflows were considered during calibration. One option was to use the ungaged computation engine in HEC-RAS, which is based on a code that was developed by Bob Barkau for use on the lower Missouri River for the UMRSFFS. The method is incorporated as an option in the HEC-RAS plan window, and involves specific input data and boundary conditions, as well as iterative model runs. When added back in, computed ungaged inflows can result in close match of calculated to observed flow at the gages.

However, the ungaged calculation within HEC-RAS takes a significant amount of computing power and time. For example, with current computer hardware and software capabilities at the current model size, to calculated ungaged inflows using the computation within HEC-RAS for a two month time period takes approximately 8 hours. Therefore, it was not considered feasible to use this methodology to compute ungaged inflows for the entire 6-yr period of available calibration flows, much less for an entire period of record.

Therefore, a simplified method of approximating ungaged inflows was developed that could be calculated once for the calibration window, and was repeatable for the period of record. Three different versions were compared before the methodology described in Section 5.3.3 was selected. The criteria by which a method was selected was the overall best match to four parameters: 1) total flow volume over the 6-year calibration window, 2) 2012 winter low flows, 3) annual peak flows at USGS gages, and 4) observed flow duration curves.

Version 1 scaled up flow inputs from all tributaries by the basin area ratio, and a constant uniform lateral inflow was added between gages. Referring back to Figure 17, the basin area ratios could be thought of as a way to account for the red basin areas, whereas the constant inflows account for the yellow basin areas. Constant inflows could also be thought of as a contribution from groundwater or base flow. Basin area ratios alone reduced the overall missing flow volume at St. Charles, the most downstream gage, from 13% to 8%. Then, the amount of constant inflows was selected so as to match the overall 6-yr volume at each USGS gage. Or in other words, the total volume missing at St. Charles was calibrated to 0% from the 8% mentioned previously However, after evaluating the results it was determined that during drought years, where there is little to no rainfall, this methodology overestimated the amount of local inflow. To more closely match flows in the winter of 2012, the constant inflows had to be cut in half, resulting in missing 4% of the total volume at St. Charles. Therefore, version 1 of estimating ungaged matched well to low flows, compromised the total flow volume, as well as annual peak flows trended too low. Figure 18 compares the model calculated flows to observed during 2012 for the three different versions of estimating ungaged.

Version 2 involved selecting a representative nearby gaged basin for each ungaged basin area in Figure 17. The gaged basin's hydrograph was scaled by the ratio between the two basins and input into the model as a uniform lateral inflow along the reach of river, tributary or mainstem, closest to the ungaged area. Results were promising, but only on an event by event basis. Selecting one representative basin introduced a large amount of variability, for a select few flood events it worked well, but under or overshot the rest of the events. When the annual peaks were evaluated, the spread of errors was both more negative and more positive. Additionally, when comparing the output to observed flow duration curves, the method resulted in overpredicting the frequency of the higher flows. An appropriate selection is highly dependent on where the rain fell, which varies from event to event, making it difficult to select one basin that could represent on average every event over 6-years. Much of the contributing area is also floodplain, which would have much lower peak flows than the nearby tributaries. Substantial iterations would be necessary to select the most appropriate basins, and adjust the ratios such that flow volume matches at each of the USGS gages. Version 2 also didn't match well during 2012, as shown in Figure 18.

The last version of estimating ungaged inflows, and the one selected for use in the model is similar to version 1, but instead of adding constant inflows the uniform lateral inflow varied on a monthly basis. This allowed the dry times to stay dry and the missing flows were instead added during already wet times. In this way, the overall flow volume was matched without overestimating the 2012 drought and the errors in annual peaks were centered on zero rather than trending too low.

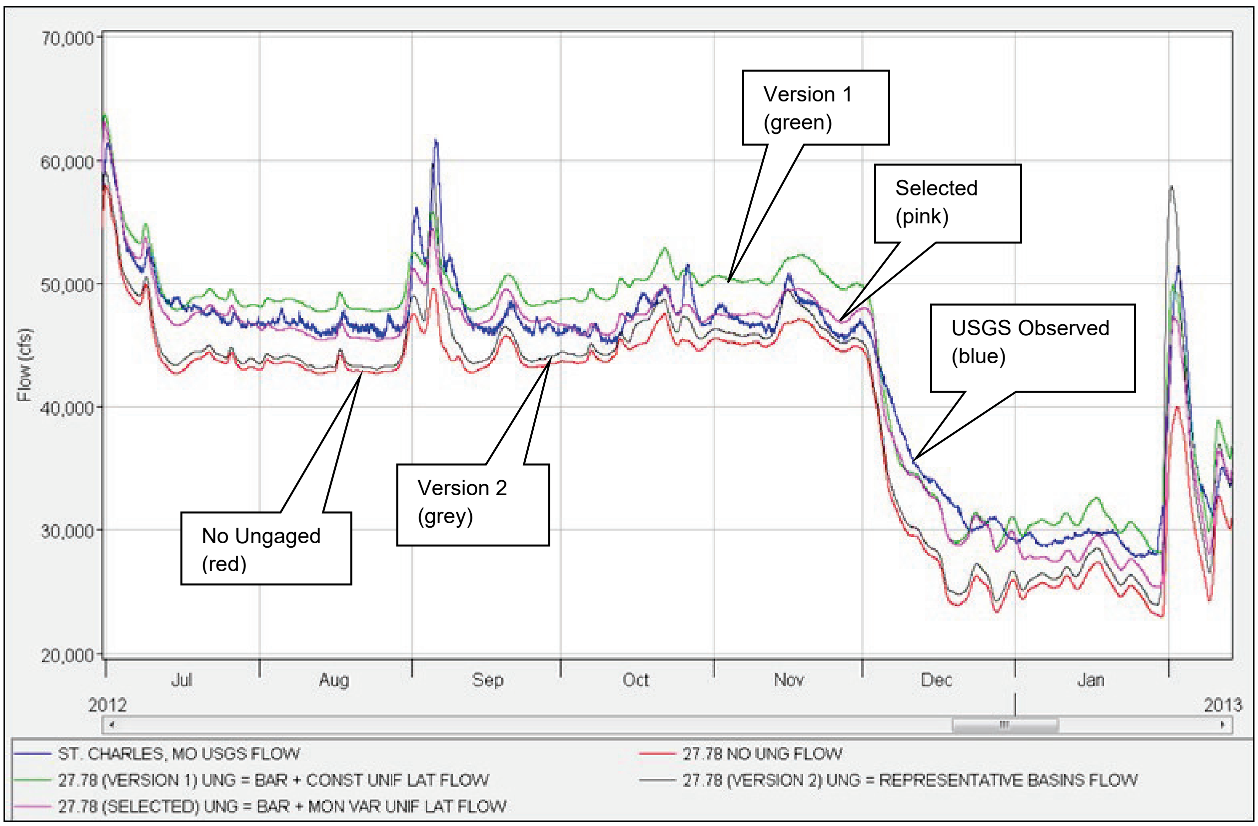

Figure 18. Comparison of ungaged methodology

Figure 18 demonstrates the impact of estimating ungaged flows by each version of ungaged on the late summer to winter flows in 2012, as well as no estimate of ungaged flows, compared to the observed data. No ungaged and version 2 both underestimated the amount of flow in the river during the drought. This could indicate some kind of base flow that is not represented by the main tributary gages. Version 1 matched flows or was slightly too high, but underestimated overall flow volume and underestimated times of higher flow. The selected methodology, basin area ratios with monthly varying inflows, was the best match to the low flows of 2012, as well as the best match to overall flow volume and annual peaks.

6.1.3 *Step 3 – Channel n-values*

Step 3 in calibrating the model was to set channel n-values so as to match a low water profile collected in August and September of 2009. The 2009 low water profile was selected because it matches the year of bathymetry data chosen for the model. Channel n-values were changed on a reach by reach basis, rather than cross section by cross section. The goal was to hit the middle range of the low water profile.

6.1.4 *Step 4 – Roughness Factors*

Flow and seasonally varied roughness factors were entered and adjusted to decrease error between modeled and observed water surface elevations. This was done by looking at the computed minus observed plots in HEC-RAS over the entire 6-yr calibration window and making adjustments based on trends. Flow factors were applied first, and then seasonal factors adjusted only if necessary. Factors were applied on a reach by reach basis, centered on the mainstem stream gages. Plots were evaluated at both USGS and NWS gages, although heavier emphasis was given to the USGS gages. Break points between reaches with different flow/seasonally factors were placed as much as possible at logical locations, such as at tributary confluences.

Generally, flow roughness factors were between 1.05 and 0.95. The largest factor used was 1.2 and smallest used was 0.85. Seasonal factors were generally only applied as a factor of 0.97, 0.98 or 0.95 during the winter months, although a positive factor of no more than 1.03 was applied during the summer months in two reaches.

For example, Figure 19 shows the computed minus observed vs. flow at the Kansas City gage for the 6-yr calibration period before flow roughness factors were applied. The model consistently calculates too high at flows above 70,000-cfs. Figure 20 shows the same plot after flow roughness factors were applied to account for this trend. The scatter plot is now centered around zero for all flows.

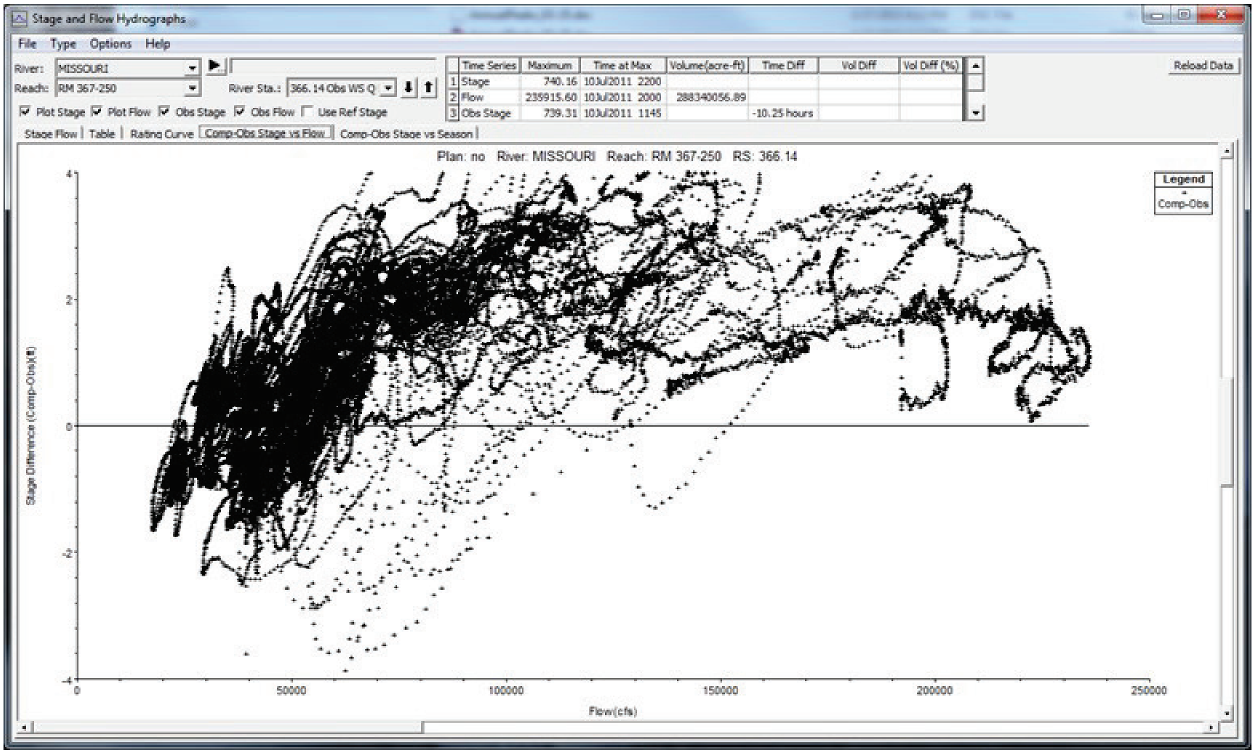

Figure 19. Before flow roughness factors at Kansas City gage

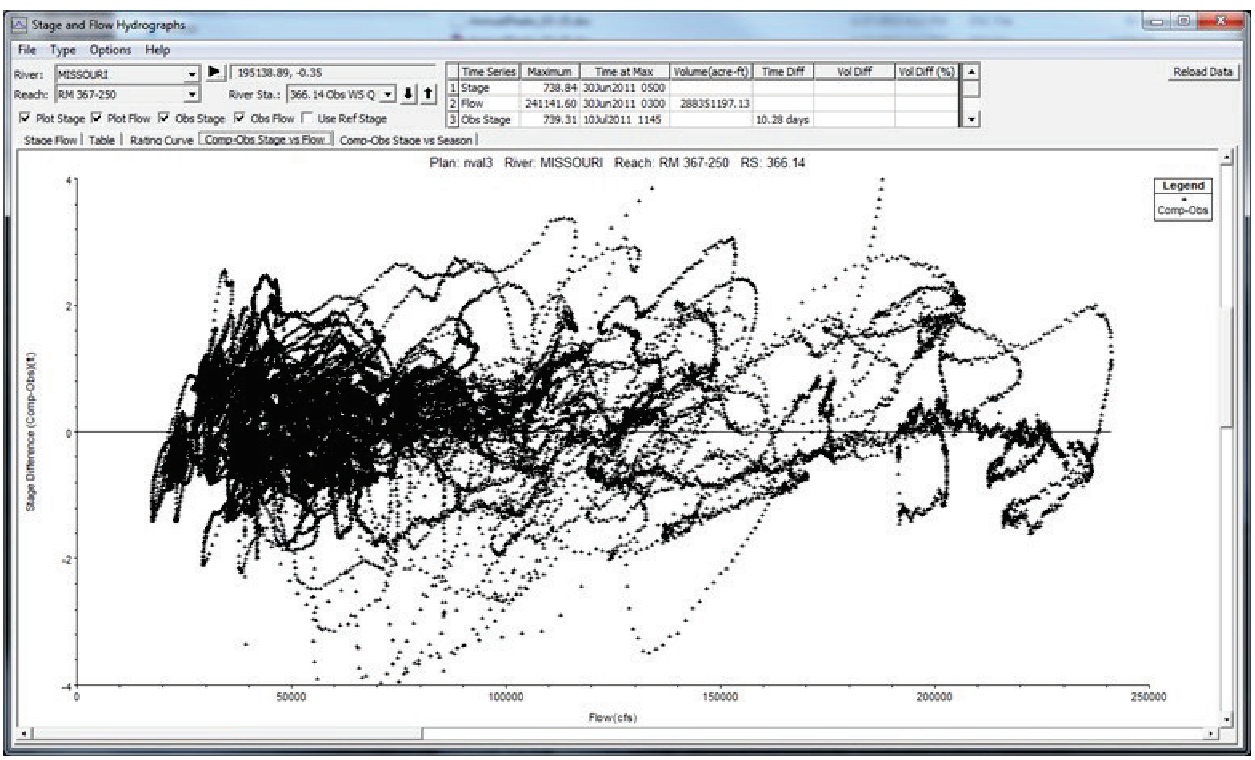

Figure 20. After flow roughness factors at Kansas City gage

Figure 21 shows the computed minus observed vs. season at the St. Charles gage for the 6-yr calibration period before seasonal roughness factors were applied. At this location the model consistently calculates too high in December through March. Figure 22 shows the same plot after seasonal roughness factors were adjusted to account for this trend.

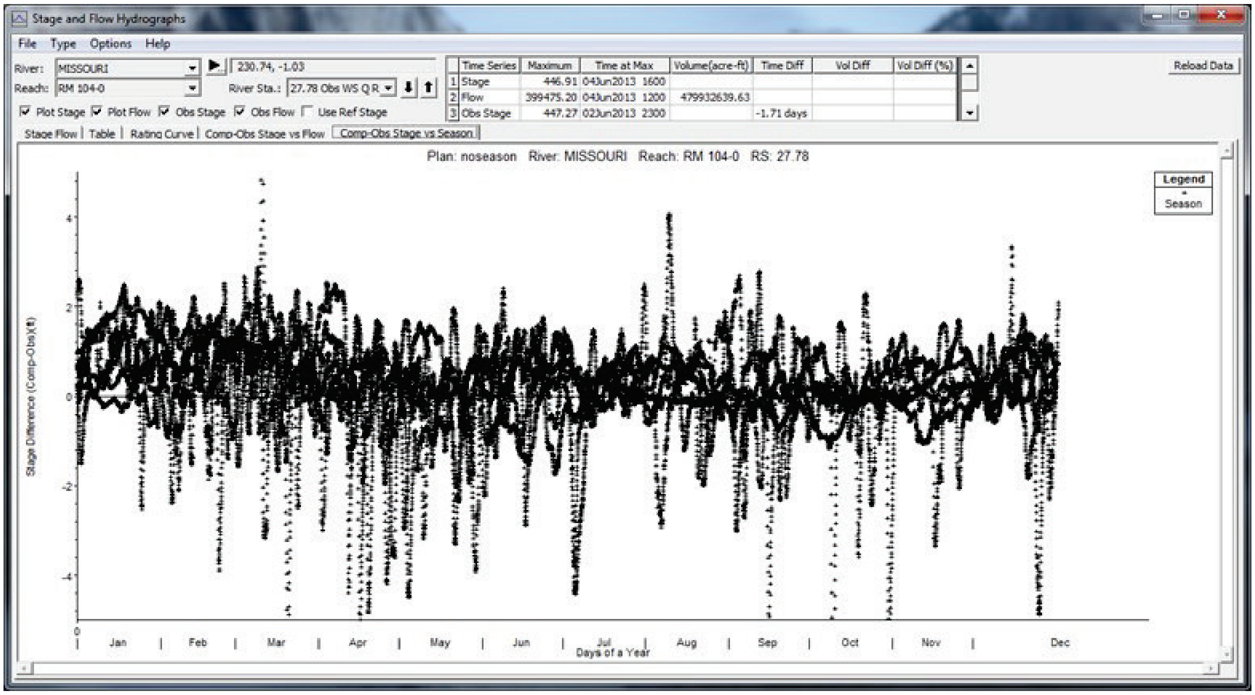

Figure 21. Before seasonally varied adjustments at St Charles gage

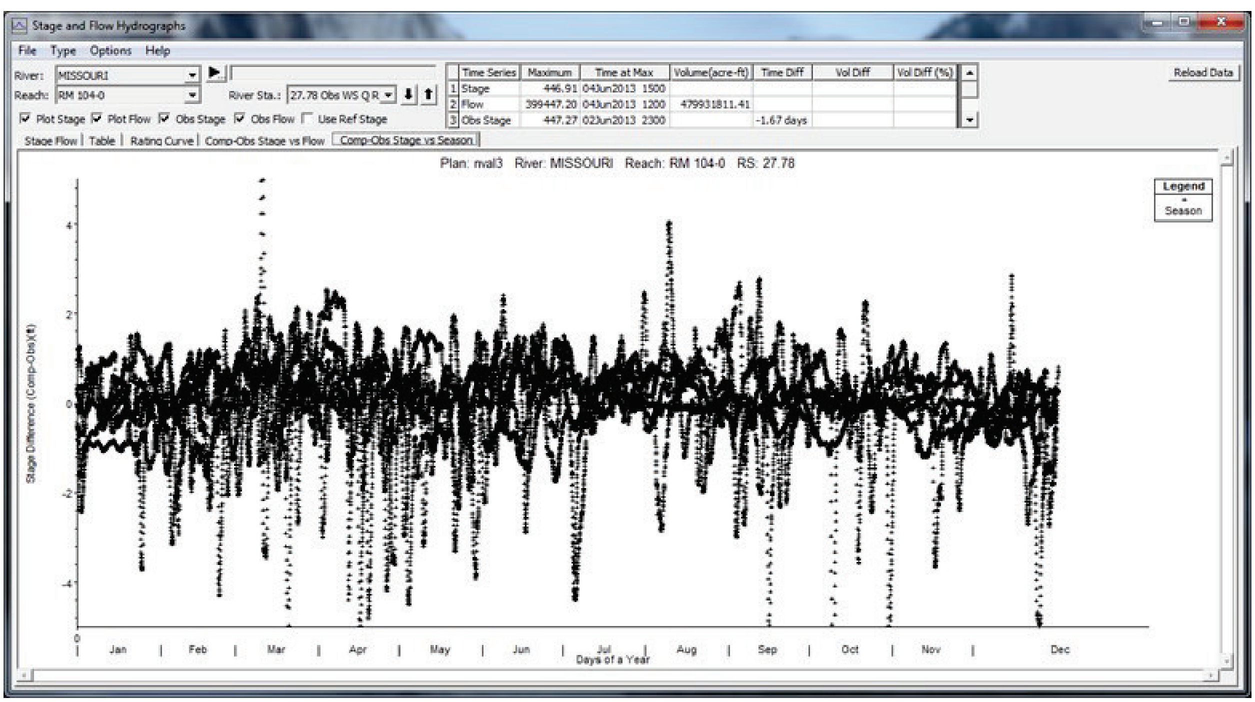

Figure 22. After seasonally varied adjustments at St Charles gage

6.1.5 *Step 5 – High Flow*

Two flood events that overtopped levees, 2011 and 2013, were run through the model to check the calibration and functionality of lateral structures and storage areas. The 2011 flood was a severe event primarily upstream Kansas City, and 2013 was the largest flood to occur downstream of the Grand River since the 1995 flood event. With the combination of these two events, high-flow calibration could be refined over a large percentage of the model. High water marks were also available for a check of the water surface profile between gages.

6.1.6 *Step 6 – Period of Record*

As an additional validation, the preliminary period of record flows, 1930 through 2013, were run through the model. Primary purpose for running the period of record was to test the capability of the model to run the lowest and highest flows on record, and the capability of HEC-RAS to manage 82 years of output.

6.2 CALIBRATION RESULTS

Model results for the 6-yr calibration period are best viewed within the HEC-RAS interface, and were not included in the report for size reasons. Observed stage and flow hydrographs, rating curves, as well as flood high water marks have been entered as observed data and can be viewed in the various HEC-RAS dialogues. To express the level of calibration achieved, statistics such as mean, median, root mean square, and histogram distributions were calculated on both stage and flow errors. Overall flow volume, annual peaks, and stage and flow duration curves, were also compared to observed data. Event specific performance was also evaluated for the 2011 and 2013 flood events.

6.2.1 *Stage Error*

Stage errors were calculated by comparing model computed stage hydrographs to observed stage hydrographs for the 6-yr calibration window at ten USGS gage locations between and including Rulo, NE and St. Charles, MO. Model calculated elevations were output at an hourly timestep. At every hour the observed value was subtracted from the model value to calculate the error. A positive error means the model calculation is too high, a negative error means the model calculation is too low. A total of 505,815 values were calculated, and distribution is summarized by the histogram in Figure 23. Errors were partitioned into 0.2-ft wide bins and converted into a percentage of the total computed errors. For example, 13.3% of all the stage errors were between 0 and 0.2-ft. Overall, 90% of the stage errors were between negative 1.3-ft and positive 1.0-ft. Stage errors are for the most part normally distributed, meaning there is not a significant skew to the positive or negative, and the average and median values are both 0.1-ft, a tenth of a foot higher than observed.

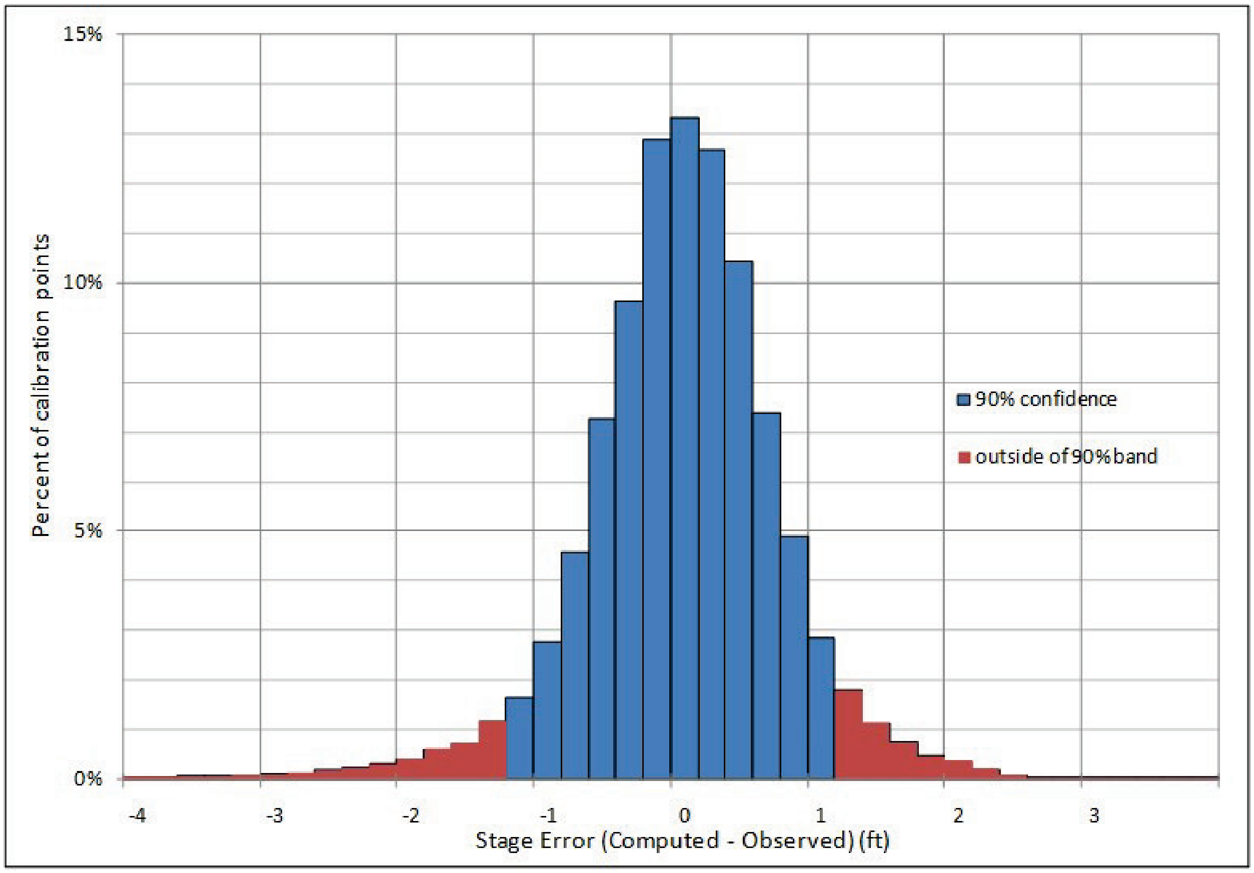

Figure 23. Stage Error Histogram

A breakdown of the errors by gage is shown in Figure 24 and Table 11. The mean and median errors are consistent with the overall mean and median, as slightly too high. Figure 24 is useful because it shows how the spread of errors vary by location on the river from upstream to downstream. Note that Waverly, at river mile 293, has a tighter calibration than Kansas City and St. Joseph. One of the contributors to stage error is that the model geometry is fixed, whereas in reality the bed of the river changed over the 6-year calibration window. Measured bed data analysis show degradation in the St. Joseph to Kansas City reach and Waverly as relatively stable, which could explain some of the trend seen in the figure. Degradation trends will be discussed more in Section 6.3.2.

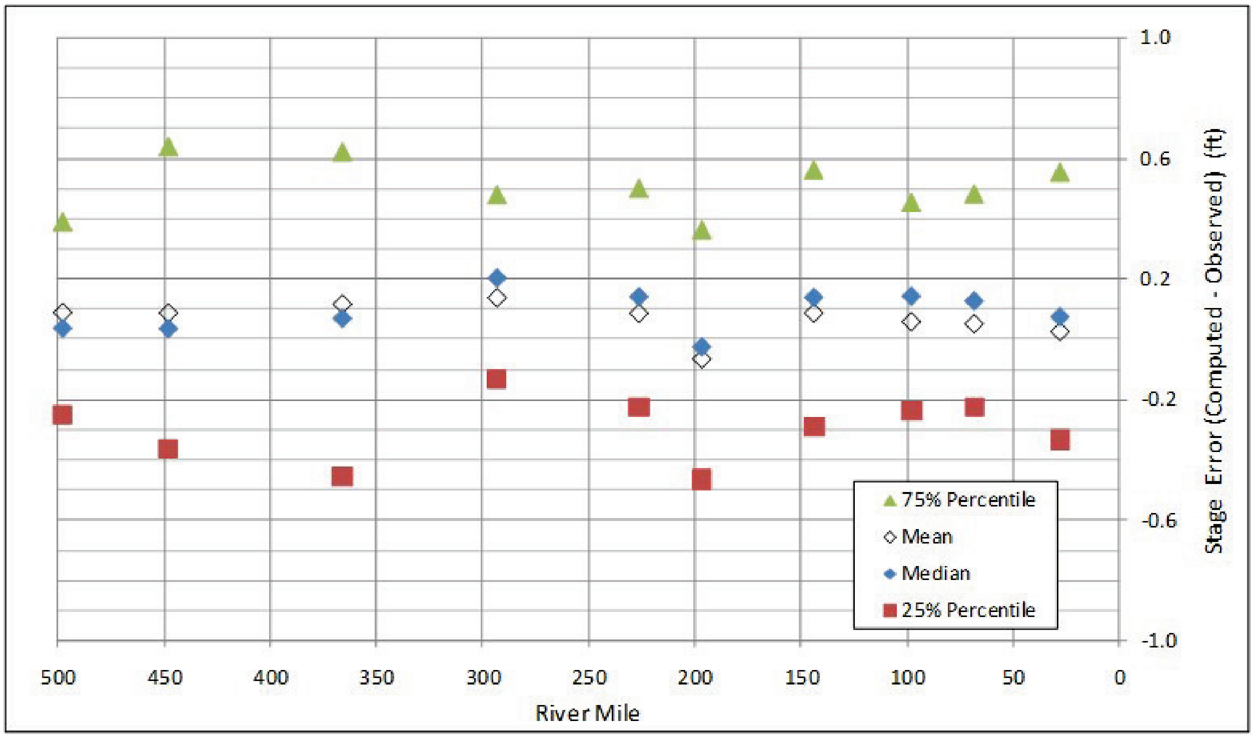

Figure 24. Stage Error by Location

Table 11. Calibration Results – Stage Error

Gage Name	River Mile	Mean Stage Error (ft)	Median Stage Error (ft)	RMS (ft)	Max Negative Error (ft)	Max Positive Error (ft)	% of the time stage is within 1-ft of obs	% of the time stage is within 2-ft of obs
Rulo	498.04	0.1	0.0	0.5	-4.2	2.5	95%	99%
St. Joseph	448.17	0.1	0.0	0.7	-5.0	3.8	91%	99%
Kansas City	366.14	0.1	0.1	0.8	-6.1	5.7	83%	98%
Waverly	293.22	0.1	0.2	0.7	-4.9	4.9	90%	99%
Glasgow	226.3	0.1	0.1	0.8	-5.6	3.5	83%	97%
Boonville	196.62	-0.1	0.0	0.8	-5.6	3.1	84%	97%
Jefferson City	143.86	0.1	0.1	0.8	-4.8	4.2	85%	97%
Hermann	97.93	0.1	0.1	0.9	-7.4	5.2	82%	96%
Washington	68.26	0.1	0.1	0.9	-8.5	4.7	82%	96%
St. Charles	27.78	0.0	0.1	0.9	-6.8	6.1	82%	96%
All		0.1	0.1	0.8	-8.5	6.1	86%	97%

Root mean square (RMS), or quadratic mean, describes the average absolute difference between the computed and observed points. When RMS is close to zero it indicates that the errors are overall close to zero. It is a similar statistical measure to the standard deviation, but the standard deviation measures the variability of the sample set around the mean, whereas RMS measures the variability around zero. RMS was computed with the following equation:

$$RMS = \sqrt{\frac{1}{n}(error_1^2 + error_2^2 + error_3^2 + \cdots + error_n^2)}$$

where

RMS	=	Root mean square
n	=	Total number of samples
$error_1$	=	Computed minus observed at sample point #1

On average, the RMS of all stage errors is 0.8-ft, which is also visually reflected in the histogram in Figure 23. Another way to look at the spread of errors is to compute a duration curve with the absolute value of all errors. Results of this analysis are in the last two columns of Table 11, listed as the percent of errors that are within 1-ft and 2-ft of observed at each gage. Overall, 86% of the time the model computed stage is within 1-ft of observed, and 97% of the time it is within 2-ft of observed.

The worst stage errors, ranging from 2 to 9-ft depending on the gage, are in many cases due to timing errors and flow errors. For example, at Kansas City the maximum negative error of 6.1-ft occurred on 13 September 2008 as shown in Figure 25.

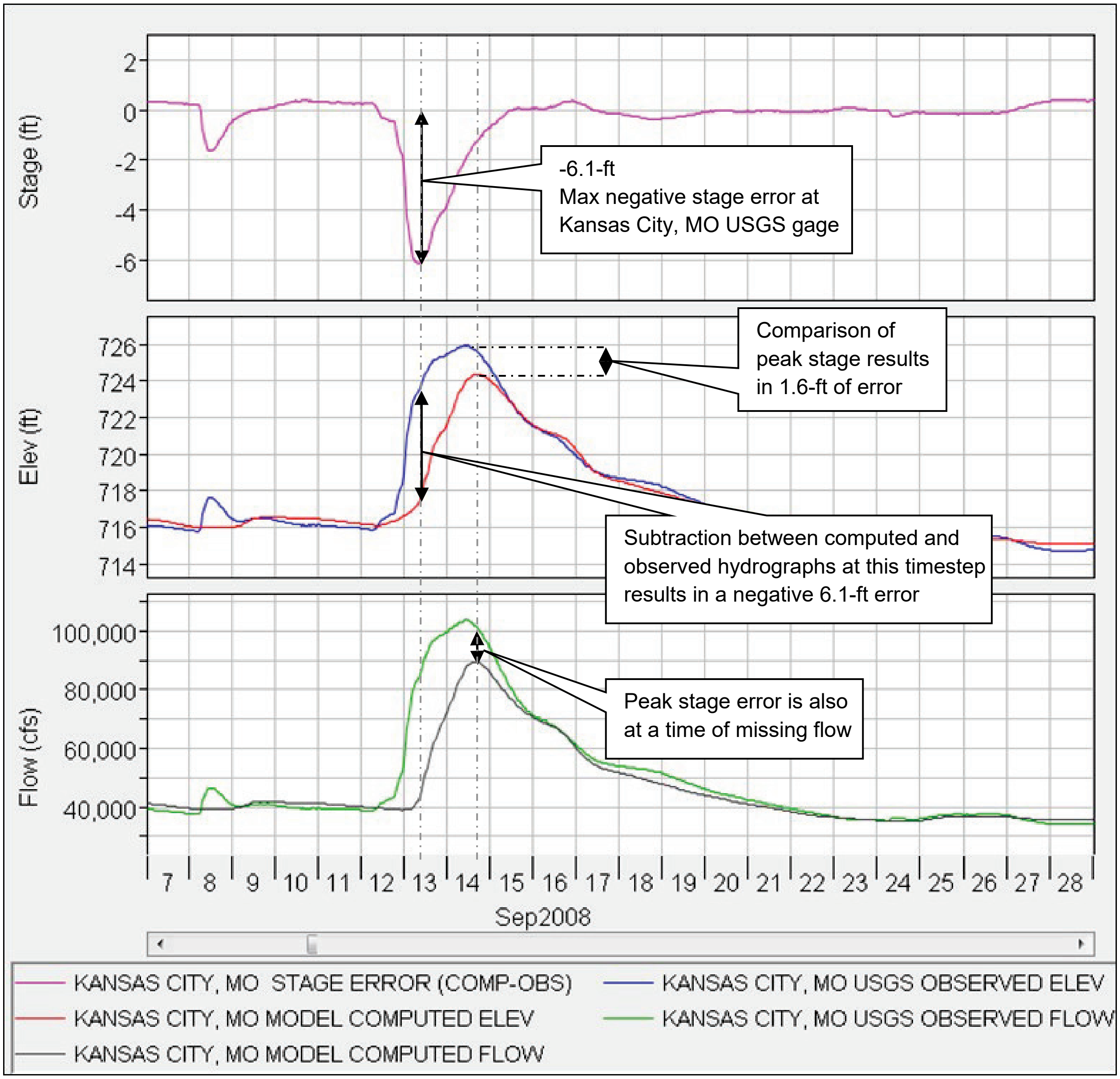

Figure 25. Max Stage Error

Because the error was calculated at each timestep, the 6.1-ft of difference actually occurred at a moment when the timing of the modeled hydrograph lagged behind the observed hydrograph on the rising limb of a small flood pulse. When comparing peaks, the model computed is low by only 1.6-ft, rather than 6.1-ft. Furthermore, when the flow hydrographs are compared, they follow the same trend as the stage hydrographs indicating that the error at the peak is primarily due to missing flow.

6.2.2 *Flow Error*

Flow errors were calculated by the same methodology as stage errors, by comparing model computed flow hydrographs to observed flow hydrographs for the 6-yr calibration window at each USGS gage. Gages at Jefferson City, MO and Washington, MO are stage only gages, and Glasgow, MO flow record had too many discontinuities, so while stage statistics were calculated at ten gages, flow statistics could only be calculated at seven gages. At an hourly timestep the observed value was subtracted from the model value to calculate the error. A positive error means the model calculates too much flow, a negative error means the model calculates too little flow. A total of 367,929 values were calculated, and distribution is summarized by the histogram in Figure 26. Errors were partitioned into 1,000-cfs wide bins and converted into a percentage of the total computed errors. For example, 21.4% of all flow errors were between 0 and 1,000-cfs. Overall, 90% of the stage errors were at flows between negative 11,200-cfs and positive 8,800-cfs. To give content to this amount, the median August flow in the Missouri River at Kansas City is 52,500-cfs and the flood of record is 625,000-cfs. Flow errors are also close to a normal distribution, meaning there is not a significant skew to the positive or negative, and the average and median values are close to zero. As discussed in Section 6.1.2, without accounting for ungaged inflows, the average flow error and general histogram skew would be negative.

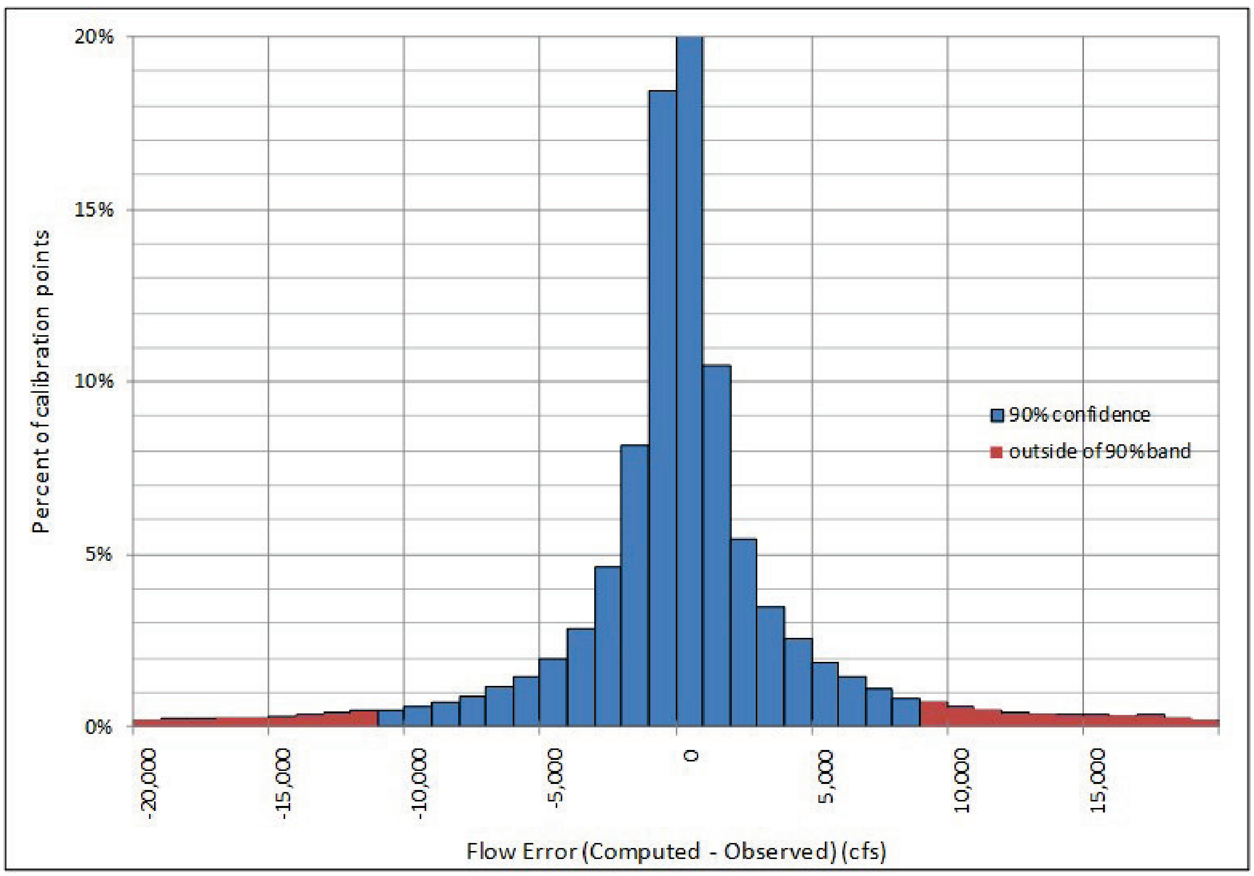

Figure 26. Flow Error Histogram

A breakdown of the errors by gage is shown in Figure 27 and Table 12. The mean and median errors are consistent with the overall mean and median, centered around zero or slightly high. Figure 27 is useful because it shows how the spread of errors vary by location on the river from upstream to downstream. While the mean and median stay relatively constant from upstream to downstream, the spread of error increases in the downstream direction. At the lower end of the river the cumulative effect of estimating ungaged inflows with a simplified method adds up.

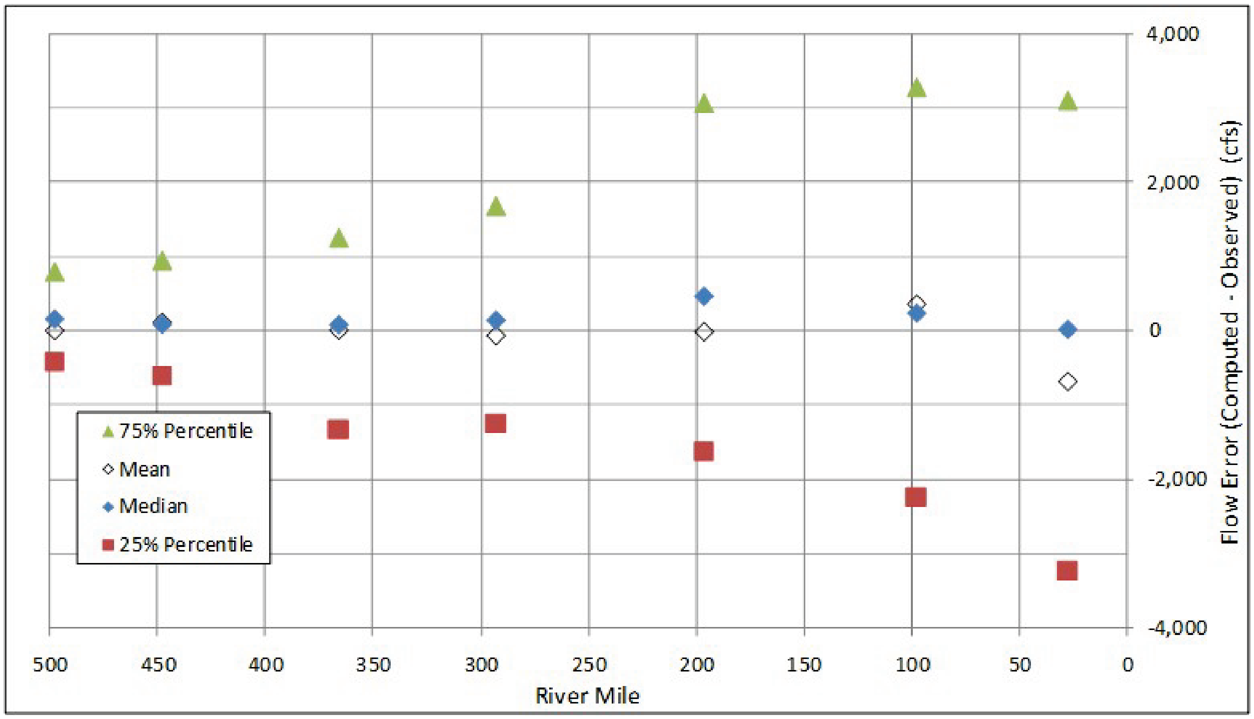

Figure 27. Flow Error by Location

Table 12. Calibration Results – Flow Error

Gage Name	River Mile	Mean Flow Error (cfs)	Median Flow Error (cfs)	RMS (cfs)	Max Negative Error (cfs)	Max Positive Error (cfs)
Rulo	498.0	11	170	4,077	-113,755	29,685
St. Joseph	448.2	122	98	4,191	-44,710	39,241
Kansas City	366.1	16	95	4,700	-44,666	55,994
Waverly	293.2	-57	155	5,410	-53,154	58,459
Boonville	196.6	-4	480	8,129	-75,171	38,849
Hermann	97.9	374	249	10,593	-83,454	91,797
St. Charles	27.8	-681	34	11,720	-116,662	81,456
All		-31	162	7,570	-116,662	91,797

On average, the RMS of all stage errors is approximately 8,000-cfs. RMS, along with the max negative and max positive errors generally increase in the downstream direction, again due to the cumulative effects of estimating missing ungaged inflows.

As was demonstrated with stage errors, the above statistical calculations also encompass timing errors, because the computed to observed comparison was made at an hourly timestep. The next three sections make comparisons between computed and observed that are not necessarily timing dependent.

6.2.3 *Flow Volume*

Percent flow volume difference between computed and observed is calculated by HEC-RAS at every location where there is an observed flow hydrograph for comparison. Flow volume is a measure of the total amount of flow that passed by that location over the 6-yr calibration time window, or in other words the integral of the flow hydrograph. Results are presented in Table 13. Overall flow volume was essentially matched by the simplified method of estimating ungaged inflows.

Table 13. Calibration Results – Flow Volume

Gage Name	River Mile	Flow Volume Difference (%)
Rulo	498.04	0.0%*
St. Joseph	448.17	0.2%
Kansas City	366.14	0.0%
Waverly	293.22	-0.1%
Boonville	196.62	0.0%
Hermann	97.93	0.4%
St. Charles	27.78	-0.6%

** Flow hydrograph at Rulo was compared to USGS channel measurements during the 2011 flood, rather than total flow which included overbank flow over Hwy 159.*

6.2.4 *Annual Peaks*

The maximum flow computed by the model in each year at each of the seven USGS flow gages was identified and compared to USGS published annual peaks. For example, see Figure 28 at the Kansas City gage, which compares the model in red to the observed in blue. Peaks are highlighted as squares on the overall 6-yr flow hydrographs.

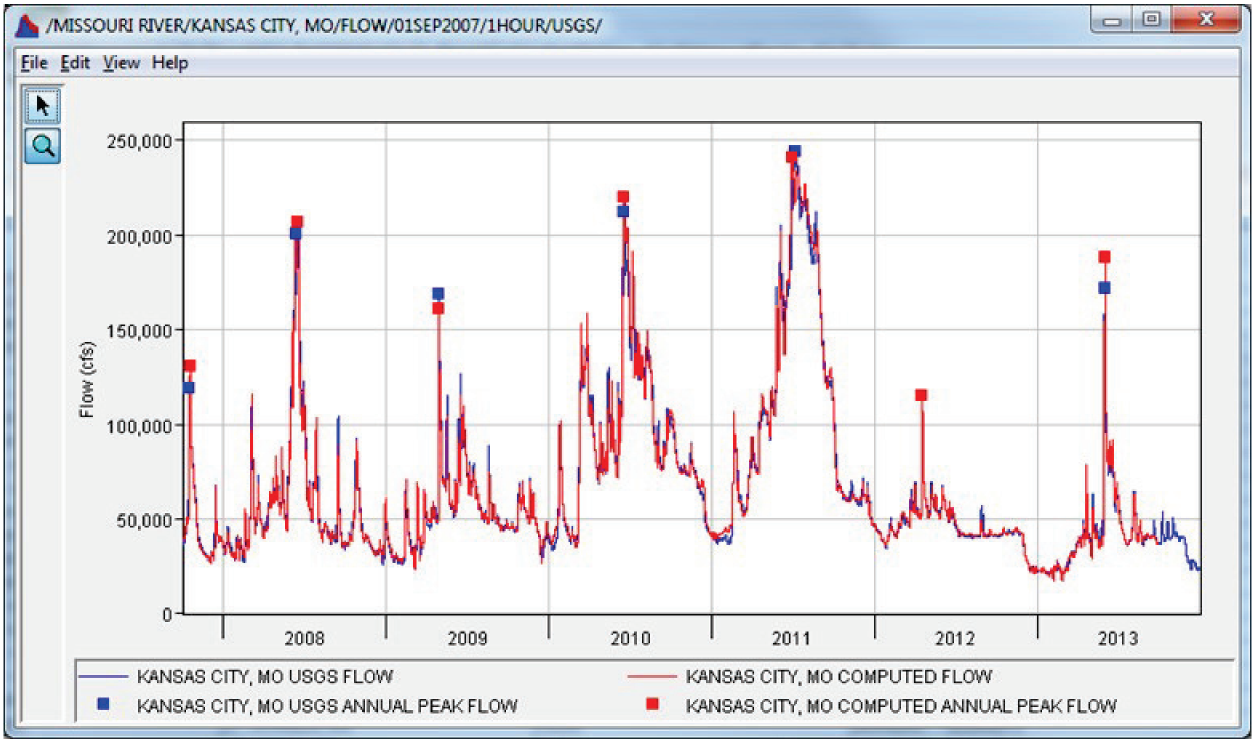

Figure 28. Annual Peaks at Kansas City, MO

At Kansas City, annual peaks in 2008, 2010, and 2013 are too high by 3%, 4% and 9% respectively. Peaks in 2009 and 2011 are too low by 3% and 4% respectively. The annual peak computed by the model in 2012 approximately matched USGS.

Annual peak results at all seven USGS flow gages are in **Attachment 9**, in graphical and tabular form, along with timing information. Generally, the trend is the same as at the Kansas City gage, some peaks are too high, some peaks are too low, but the overall average is zero. The worst negative was in 2011 at Rulo, the model underestimated the peak by 30% or 99,000-cfs, which will be discussed in more detail in Section 6.2.6. The worst positive was 25% too high or 34,000-cfs too high in 2013 at St. Joseph. Errors are primarily due to the method of estimating ungaged inflows.

6.2.5 *Stage and Flow Duration*

A duration analysis was conducted on the stage and flow hydrographs at each of the gages on the mainstem Missouri. Ten gages were evaluated for stage and seven for flow as explained previously. Many of the ecological and human consideration economic models will base their evaluations on statistical analysis of the output hydrographs so it was considered important to quantify the level of calibration in this way. Duration analysis will answer questions such as how long will a boat ramp be out of the water, and therefore unusable, during the recreation season.

Duration of time exceeding various stages and flow rates was calculated using the 15 minute data on both the observed hydrographs and the model computed hydrographs summarized in a 23-point curve. The curves were compared, and percent difference was computed at each point on the curve. Note that this analysis is different than a Bulletin 17B frequency analysis. Tables and plots are included in **Attachment 10** for stage and **Attachment 11** for flow.

Generally speaking, the model is able to reasonably reproduce flow duration curves with error less than 10% except for during the most severe flood events experienced during the calibration period. And the model is able to reasonably reproduce stage duration curves with error less than 1-ft, except on two occasions with error no more than 1.3-ft. Ecological models that conduct duration analysis on output hydrographs can be reasonably confident in results, especially at lower flows.

6.2.6 *2011 Flood*

Attachment 12 contains plots of the maximum profile of the 2011 flood compared against collected high water marks (HWM), as well as a comparison of the peak stage and flows at gage locations. Generally, the model tends to underestimate the maximum profile, especially between Rulo and St. Joseph and downstream of Leavenworth near the Platte River. The model also underestimates the peak flow at all USGS gages upstream of the Grand River.

At Rulo, the model computed flow hydrograph was short of the observed flow by almost 100,000-cfs. As shown in Figure 29, a large spike in observed USGS flows on 27 June is not present in the model results. This occurred during a time when the river was still adjusting to several levee breaches and significant ungaged flows were present due to localized rainfall as discussed in more detail later in this section. Other than this spike, the model matches fairly well to the observed data, especially considering the complex nature of flooding in this area, as presented in Section 5.2.6. With the configuration of the floodplain as a series of connected storage areas, the model is able to capture the flow in the river separate from the flow that bypasses the river over Highway 159.

During the flood event, USGS teams took flow measurements in the river and over the highway, as plotted in Figure 29. Model calculations show about 160,000-cfs of sustained flow in the river, whereas USGS measured about 130,000-cfs. This difference could be accounted for by modeling the breaches upstream of the Rulo bridge and successive breaches of the highway and levees downstream of the Rulo bridge, which circumvents the gage location and likely lowers the overall amount of flow in the river.

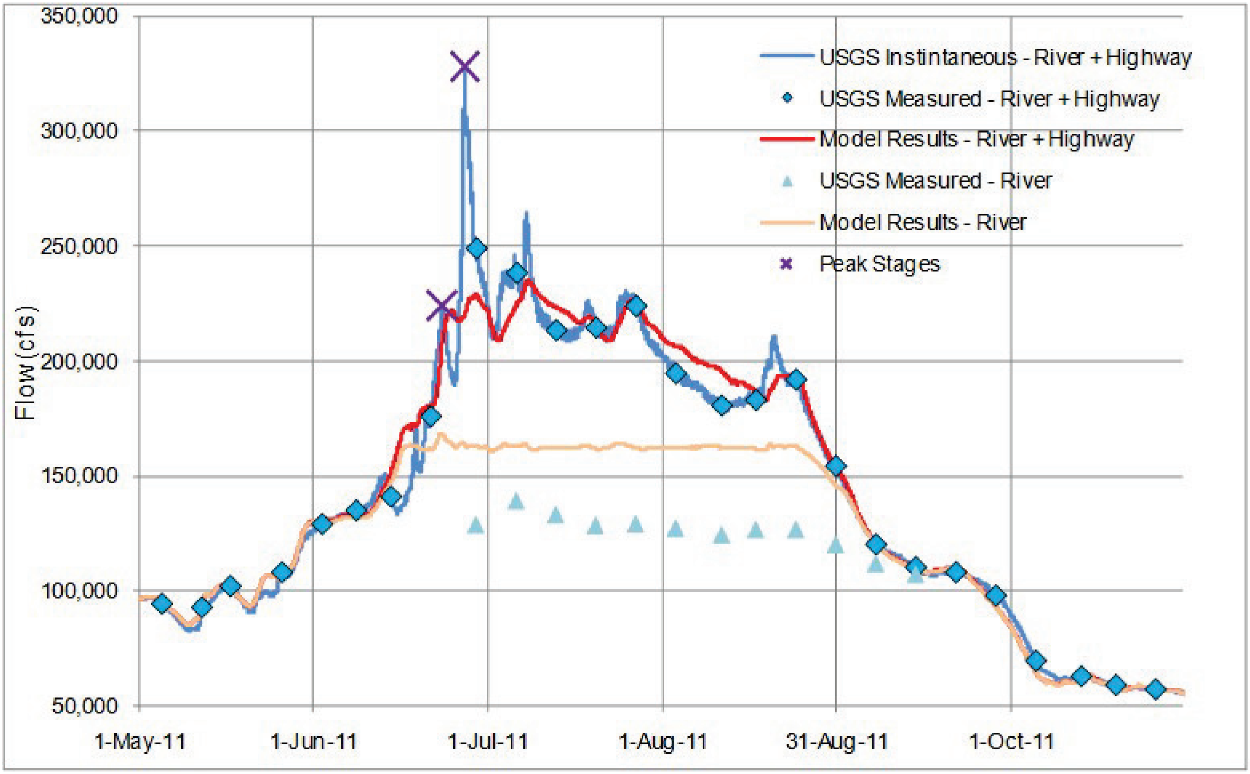

Figure 29. 2011 Flow Hydrograph at Rulo

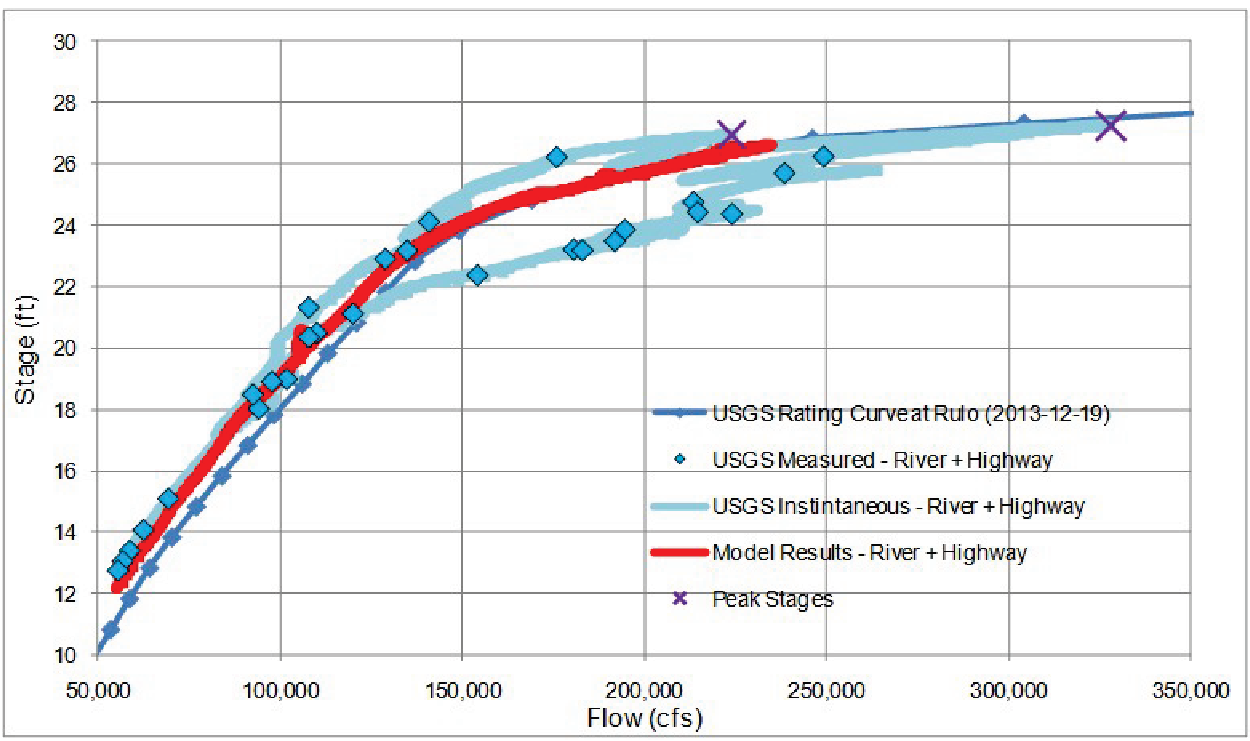

Figure 30. 2011 Rating Curve at Rulo

Figure 30 demonstrates the uncertainty involved reporting the highest flows at the Rulo gage because of the highway flow. Four days before the 27-June peak, the river crested at stage of 26.96, 0.3-ft lower than the peak stage of 27.26. The USGS converts stage measurements to flow based on measured data and a single value rating curve, shifted during events based on measurements. The first stage was converted to a flow of 224,000-cfs, while the second was converted to a flow of 328,000-cfs, about a 100,000-cfs difference. Missing information about breach timing and contribution to flow over Highway 159 contribute to the uncertainty involved in calculating a total flow. Union Township, Holt 10 upstream of the Rulo bridge and a Tarkio River levee behind Union Township all breached prior to the first peak, however it's likely that rising stages continued to enlarge breach dimensions as the event progressed. A flat rating curve, coupled with a shift in the rating curve during the event appear to explain the substantial difference between reported flows on 23-June and 27-June. In addition, the largest flow measured by the USGS during the event was 250,000-cfs on the descending limb. Because of the many sources of uncertainty, model results were considered acceptable at the Rulo gage even though the peak flows were significantly different.

Storage areas also facilitate much better inundation mapping of the Rulo area than could be obtained from cross sections. Figure 31 shows aerial photography of the Rulo floodplain taken shortly after the peak of the event. Figure 32 shows the max profile computed by the model, mapped as a depth grid in RasMapper.

Differences between model results and actual flooding illustrated in Figure 31 and Figure 32 could be reduced by including levee breaches in the model. For example, the model calculated a lower level of inundation than actually occurred in the storage areas labeled “hlt9” and “fts”. This is most likely because of the breach that occurred on the Little Tarkio tieback (“ltkt”) at Holt 10 (“hlt2”) and Holt County Levee District No 9 (“hlt9”). The model currently calculates the level of inundation in this area only based on water overtopping the levee from the mainstem Missouri, but the inundation level in the protected area was likely higher because of the breach as demonstrated by the sand splays present in the aerial photograph.

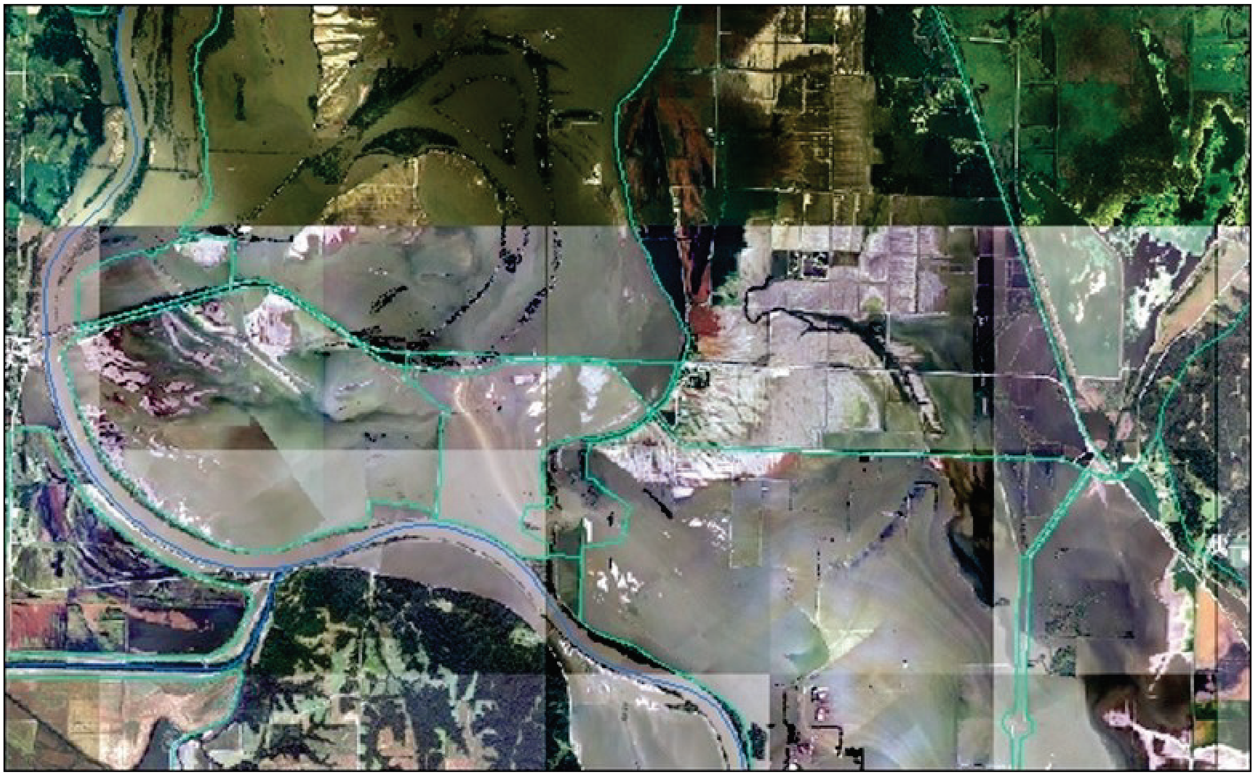

Figure 31. Flood Inundation at Rulo in 2011

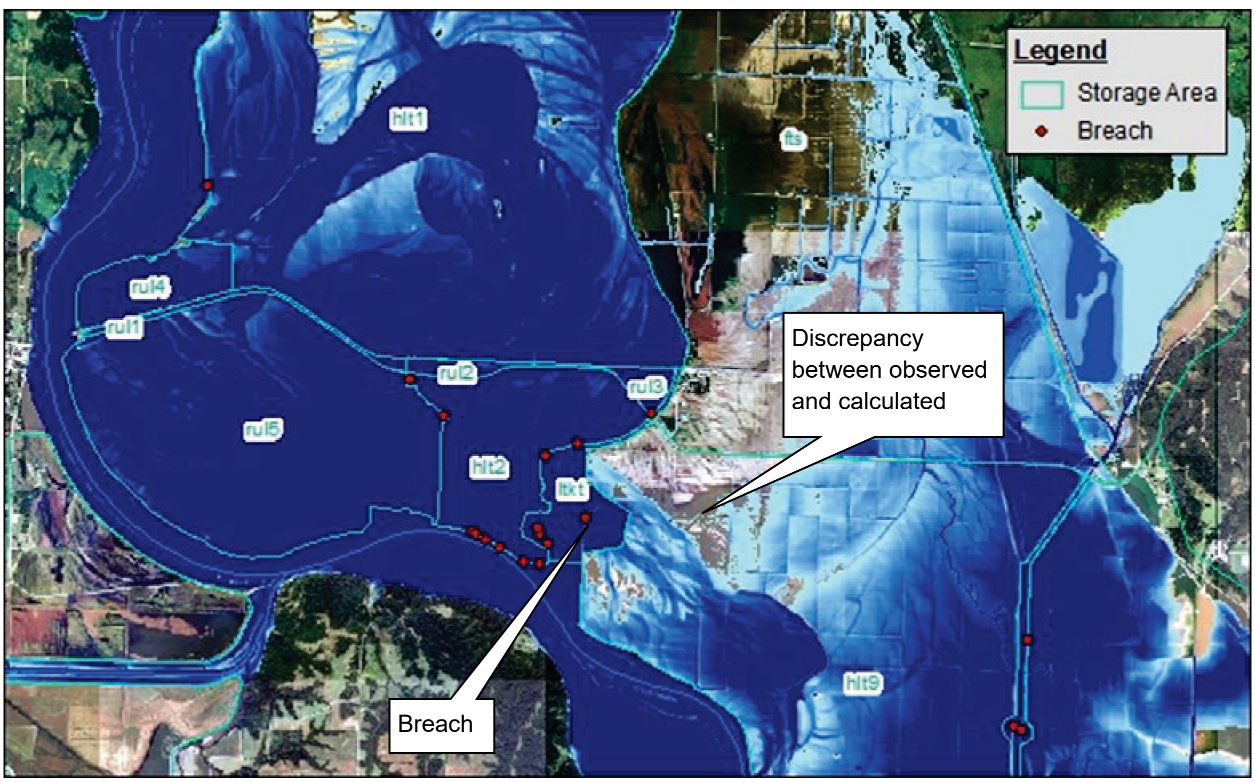

Figure 32. Flood Inundation at Rulo in 2011 – model results

While uncertainty in reported flow and levee breaches account for some of the discrepancy between computed and observed flow at Rulo, they don't fully explain why the model is still short 20,000-cfs from the highest measured value at Rulo, and missing 14% and 2% of the peak flow downstream at St. Joseph and Kansas City, respectively.

Overall the Kansas City District had lower than average precipitation during 2011. It was different than other historical events such as 1993 and 2007 in that upstream precipitation and subsequent releases from Gavin's Point Dam contributed to a much higher percentage of the peak flows. For example, peak releases of 160,000 cfs, assuming no losses due to evaporation or attenuation, made up approximately 49%, 58%, and 65% of the 2011 USGS peak flow estimates at the Rulo, St Joseph, and Kansas City streamgages, respectfully. In comparison, mainstem reservoir releases made up less than 5% of the peak flows during the 1993 flood event. However, near the peak of the 2011 flood the area between Nebraska City and Kansas City saw heavy precipitation. Figure 33 shows the 1-day precipitation from noon 26-June to noon 27-June, with the highest amounts of 5 to 6 inches centered on the Platte River and upper reaches of the Grand River basin. There was also 3 to 4 inches over the ungaged area between Rulo and the Nodaway River, as can be seen by comparing the radar against the basin area maps presented in Figure 17 and **Attachment 8**. In addition, all of northwest Missouri, Nebraska and Iowa had precipitation the previous two days in a row, so the antecedent moisture condition would have been such that much of this new precipitation would have been direct runoff into the streams and rivers. At Rulo the Missouri River peaked on 27-June, and at St. Joseph the river peaked on 28-June, within a day of this image.

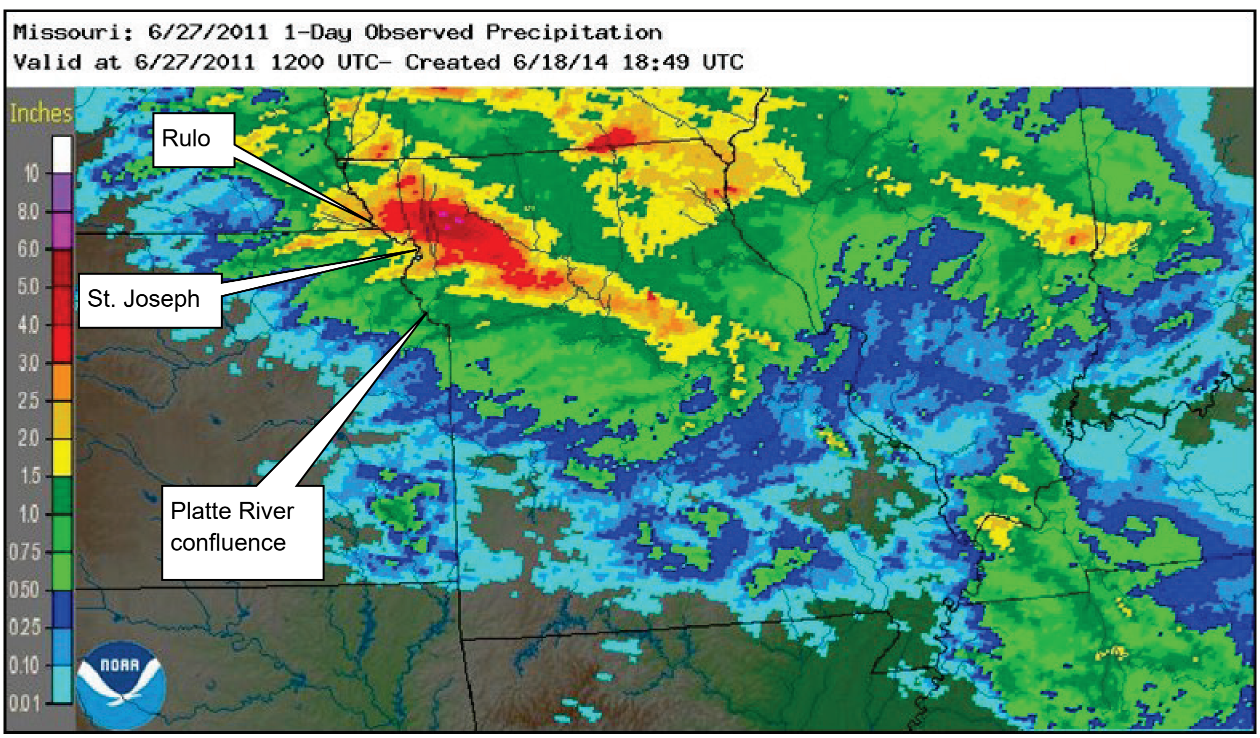

Figure 33. Precipitation near peak of the 2011 Flood

The method chosen for estimating ungaged inflows does a good job on average, but has no way of accounting for more extreme circumstances, and this is likely a contributing factor to missing flow at Rulo, St. Joseph, and Kansas City. Because missing flow was the reason the model was low on stage in some areas, the decision was made not to increase the n-values to match the 2011 high water marks. For the purposes of this project, it was considered more important to maintain the proper stage-flow relationship than to match one event.

6.2.7 *2013 Flood*

The lower Missouri River basin experienced flooding conditions in early June 2013 as a result of localized rainfall. Figure 34 shows the 1-day precipitation from noon 30-May to noon 1-June. The Missouri River peaked late on 1-June or early on 2-June at most locations. Non-federal levee sponsors within Kansas City District reported overtopping of 8 Missouri River levees and one Grand River levee. Additionally, levees immediately upstream and downstream of the confluence in St Louis District's area of responsibility also overtopped both from Mississippi and Missouri River flows. For reference, peak flows on the Missouri River downstream of the Grand River in June 2013 were the largest observed since the May 1995 flood event with magnitudes exceeding a 10% AEP (10-yr) flood (USACE, Kansas City District, 2014).

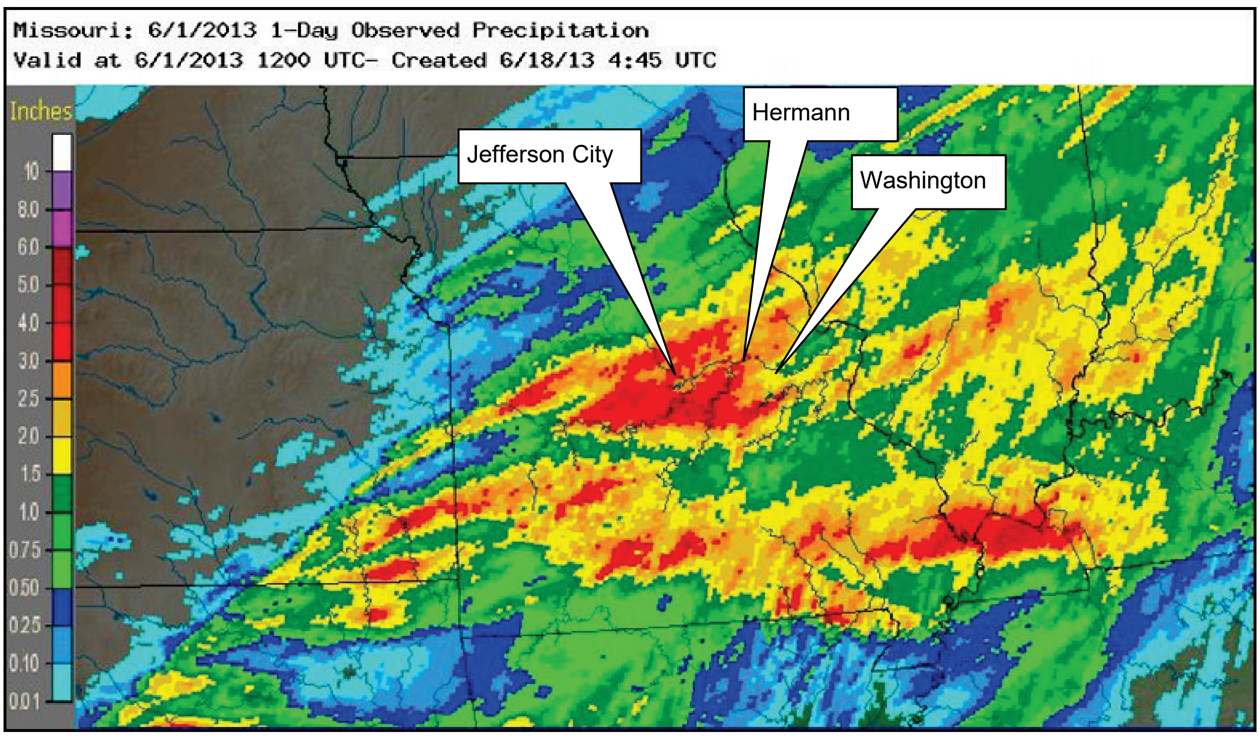

Figure 34. Precipitation near peak of the 2013 Flood

The highest precipitation amounts of 3 to 6 inches were centered on the Osage and Gasconade Rivers, as well as directly over the Missouri River, with significant heavy precipitation in ungaged areas. Figure 35 shows the flow hydrograph at Herman, MO, where the calculated hydrograph is lagged and missing 13% or 61,000-cfs at the peak.

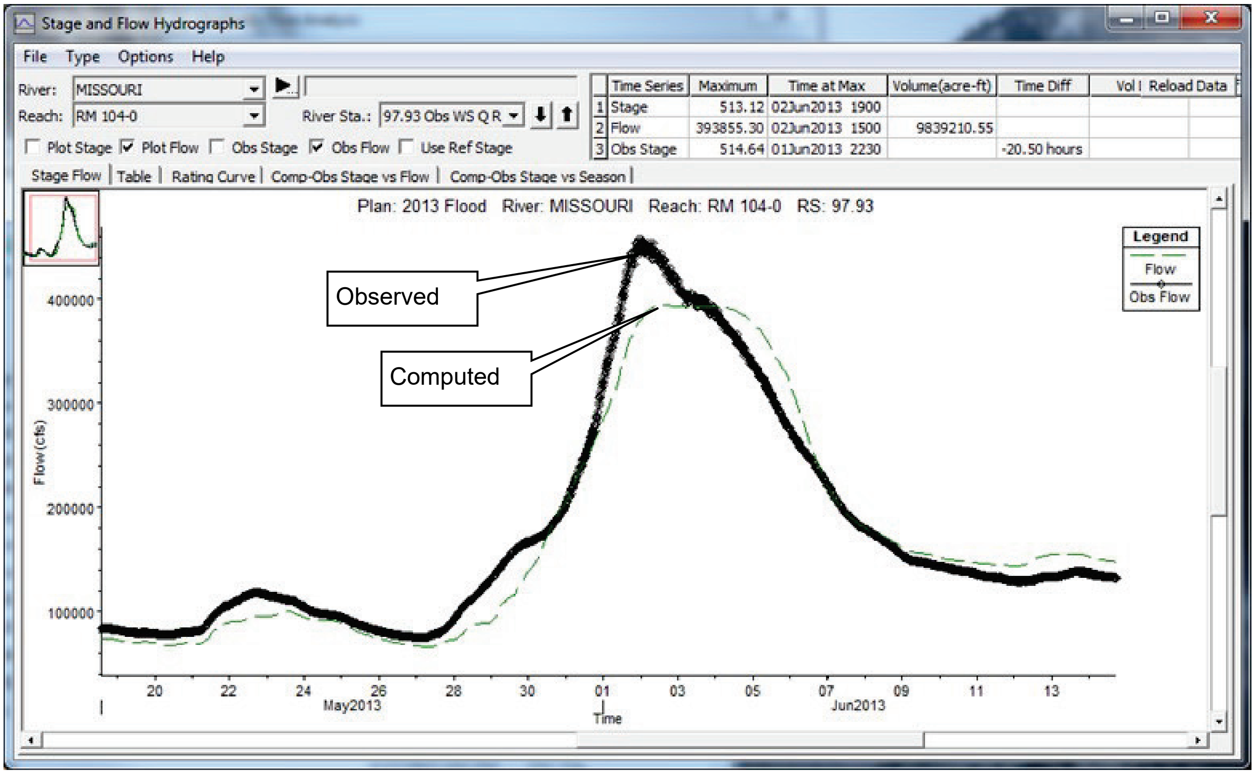

Figure 35. 2013 flow hydrograph at Hermann

Attachment 13 contains plots of the maximum profile of the 2013 flood compared against collected HWMs, as well as a comparison of the peak stage and flows at four gage locations. The worst discrepancy is at the Hermann gage; upstream at the Jefferson City gage and downstream at Washington and St. Charles gages results are better. Model results match well to HWMs near and just downstream of the Osage River, as well as downstream near Washington, with larger errors near the Gasconade River.

Based on model results coupled with radar information, it appears that local precipitation contributed a large amount of flow to the Missouri River that was not captured by the ungaged methodology. At Hermann, the difference of 63,000-cfs equates to 1.6-ft according to the gage rating curve, which corresponds closely to the amount of stage error between computed and observed. This indicates that the stage error is a result of missing flow, and that the model is appropriately reproducing the stage vs flow relationship.

6.3 MODEL PERFORMANCE

Calibration of this model is intended to, on average, reproduce both the low and high flow conditions on the Missouri River. It was not calibrated tightly to any one event, but rather is calibrated to match the present day stage-discharge relationships at USGS gages. The following model performance discussion identifies major sources of uncertainty for high and low flows, as well as general confidence in results. Time and data limitations as well as hardware and software limitations all play a role in the level of calibration attainable for this size of model. Given more resources and time, there are several improvements that could be made to the model to improve calibration.

6.3.1 *High Flow*

Many geometry factors influence the calibration of high flow events, including channel parameters, varying floodplain roughness coefficients, ineffective areas to capture effects of natural or man-made small levees directly on the river bank, and larger levees modeled as storage areas. The two largest factors that limit the capability of the model when it comes to flood events are 1) reproducing the locations and timing of levee breaches and subsequent repairs, and 2) estimating ungaged inflows.

With available LiDAR data and top of levee information from detailed surveys documented in the National Levee Database, levee heights and locations are considered to have high confidence. However, there is limited data on when and where breaches occurred during past floods, and almost no data on breach development rates, which are all parameters that would need to be input to HEC-RAS to properly model the breaches. Repair times and configurations would also need to be coded into the model geometry. Additionally, changes to the river geometry such as construction of new levees, changes in alignment, changes to the stage-discharge relationship over time, and retirement of some levees are often present after these historical floods. It would take a considerable effort to account for levee breaches in the 6-yr calibration window, and proportionally more over the entire period of record.

For this project, an assumption has been made that all levees overtop with no breaches. In addition, since the intended use of this model is the evaluation of management plan alternatives, a more consistent comparison can be achieved using levee overtopping events than with unpredictable levee breach events. Storage areas, storage area connections, and lateral structures have been configured in such a way that, if necessary, it would be possible to add levee breaches if additional time and resources are made available. Even without the use of levee breaches, the storage areas and lateral structures already in the model have the potential to be powerful tools in modeling complex flooding in the Missouri River floodplain that are not adequately represented by cross sections alone.

Ungaged inflows are the other, and more influential, source of uncertainty in model performance at high flows. Errors during flood events are for the most part a result of uncertainty introduced by the ungaged inflow methodology. Of the two components to the ungaged methodology, monthly varied uniform lateral inflow and basin area ratios, the ratios have a stronger influence on the computation of peak flows. Storms in the Kansas City District basin area are highly variable; sometimes widespread storm fronts move across entire states, other times areas of very heavy rainfall move quickly across smaller areas as isolated storm cells. Because of this variability, at times the basin area ratio doesn't reflect enough rainfall, other times it overestimates rainfall, and on occasion it matches well. During high flow events, errors in flow are closely tied to errors in stage. Figure 36 demonstrates the relationship between stage and flow errors at flows greater than a 50% AEP (2-yr) at St. Charles. St. Charles is the most downstream gage, so the cumulative effects of flow errors are most pronounced at this location. When the model doesn't have enough flow, stage computations are correspondingly too low. When the model has too much flow, stages are too high.

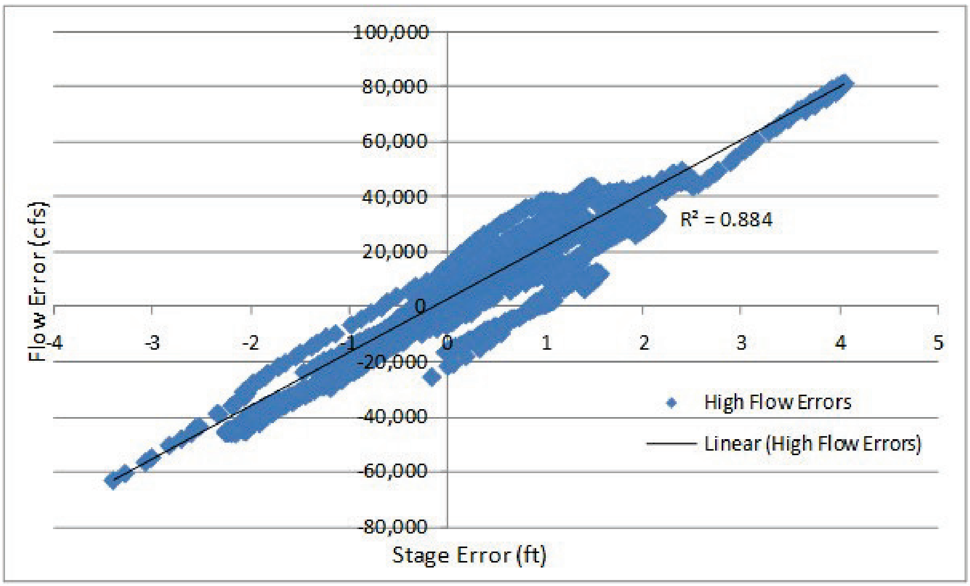

Figure 36. Stage Error vs. Flow error – St. Charles gage

Overall, high flow events were more difficult to calibrate to, due to the large amount of uncertainty and variability introduced by ungaged inflows and levee breaches. Therefore, calibration parameters were set to represent the stage flow relationship on average over the 6-yr window, rather than unrealistically weight the n-values to compensate for flow errors. To improve calibration to individual flood events, ungaged inflows could be calculated using an event specific methodology or the ungaged code built into the HEC-RAS interface. Considering the project purposes, calibration with the simplified method is considered adequate for the purpose of running period of record comparison of alternatives.

6.3.2 *Low Flow*

Confidence in the model results for low flows is about the same as for high flows, but without the large outlying errors. For purposes of this discussion, low flows are considered flows less than the median August discharge, well within the banks of the river but still including small rises. Overall, for low flow errors the root mean square is 0.6-ft, compared to 0.8-ft for all flows. And nearly all of the time the stage errors are within 2-ft of observed. Uncertainty at low flows has two major sources: 1) flow errors, and 2) changing bed conditions.

Similar to high flows, at low flows there is a correlation between flow errors and stage errors. If the right flow is not at the right location at the right time, it is difficult to match the stage. Rather than skew n-values, calibration parameters were set to match the stage flow relationship.

The more influential source of errors at low flows is changing bed conditions. Several reaches of the Missouri River have significant degradation/aggregation trends, historically and over the 6-yr calibration window. The model geometry is fixed in time with bed data from 2009, and has no mechanism to match water surface trends that are due to bed change trends. Figure 37 and Figure 38 shows the breakdown of stage errors by year at the St. Joseph and Kansas City gages, only for flows less than the median August. The average and median errors are centered around zero for all years, rather than matching exactly to stages in 2009. This is because during calibration the decision was made to match stage to the maximum extent over the calibration time window, rather than to match one particular year or event.

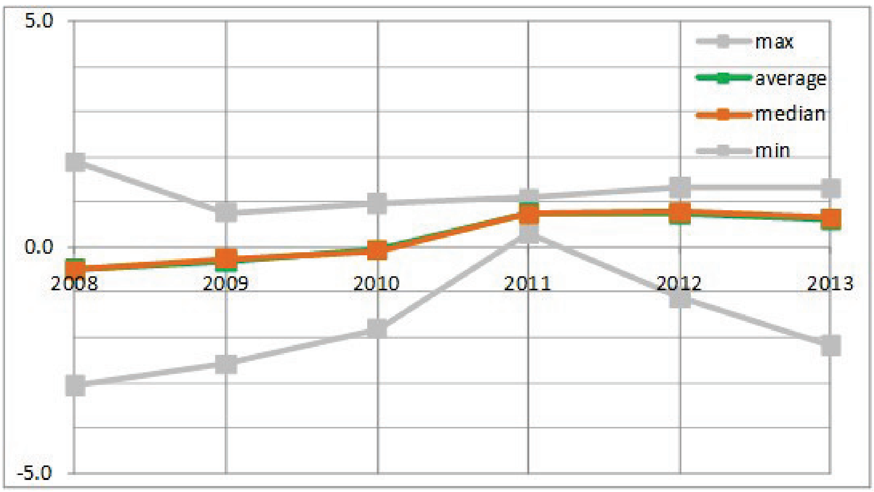

Figure 37. Low Flow Errors by Year – St Joseph gage

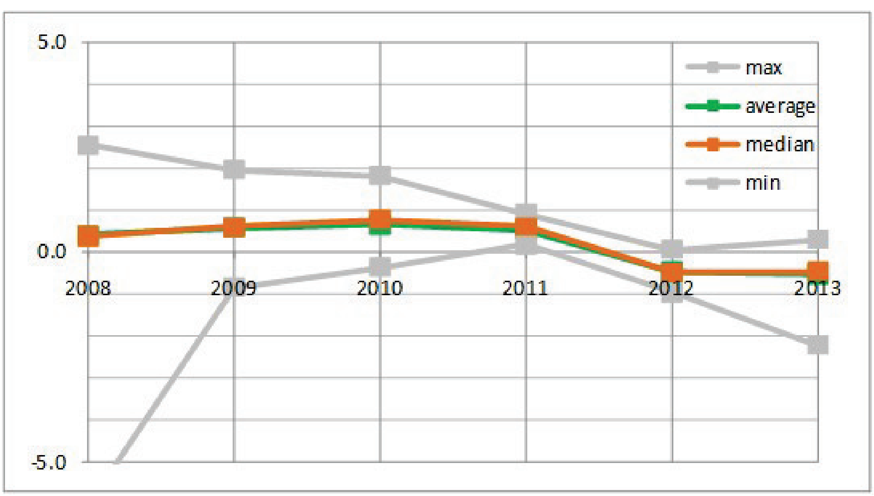

Figure 38. Low Flow Errors by Year – Kansas City gage

At St. Joseph, the model calculates on average too low before the 2011 flood, and too high after the 2011, for an overall difference of slightly over a foot. At Kansas City the trend is the opposite, the model calculates too low after the 2011 flood. Recent degradation trends at both these locations help explain these results.

Figure 39 and Figure 40 present trends in USGS field measurements at the St. Joseph and Kansas City gages. These measurements inform the rating curve applied to calculate the flow for a given stage reading. Changes in the rating curve over time are not the actual degradation, but a reflection of the impact of degredation on the water surface elevation in the river.

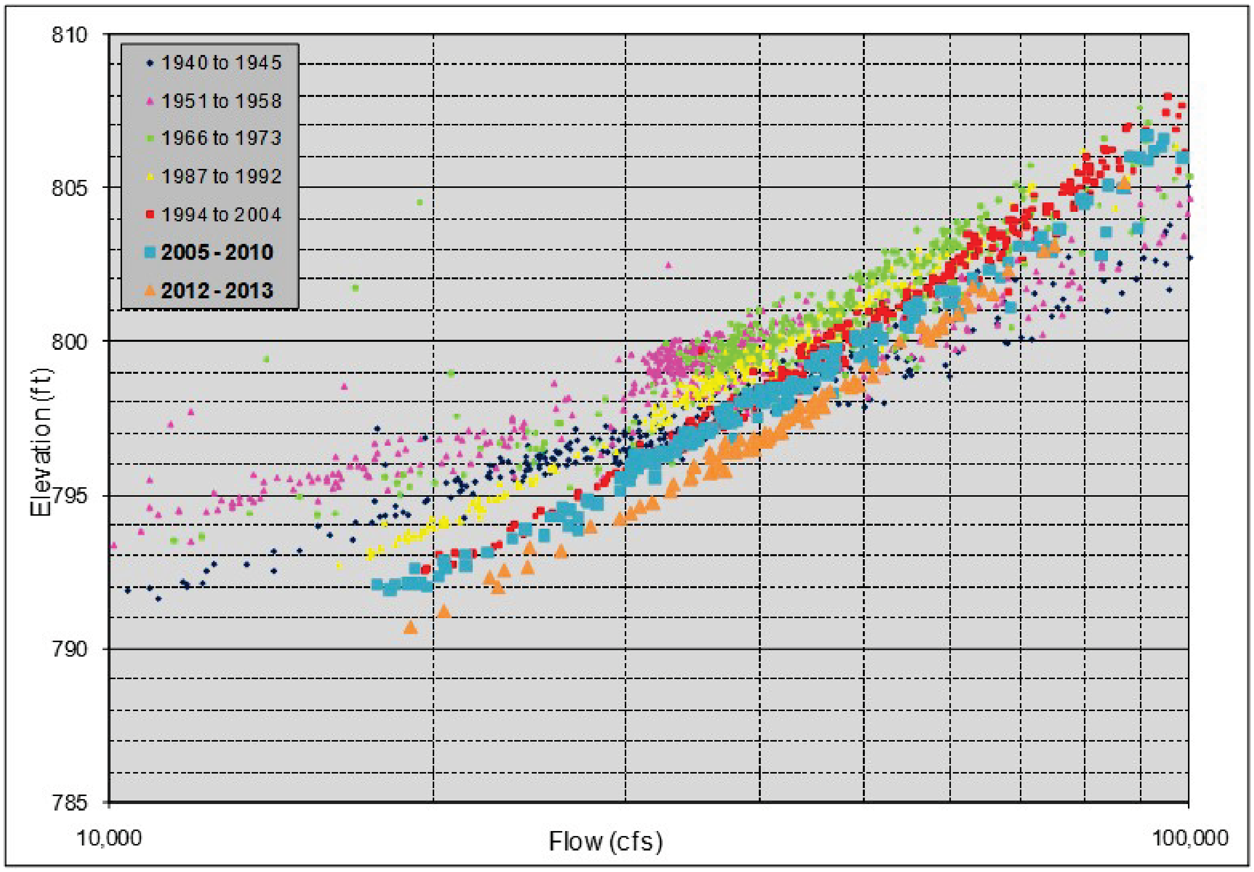

Figure 39. USGS Measured Flow vs. Stage – St. Joseph gage

At St. Joseph, for low flows, field measurements after the 2011 flood are consistently over a foot lower than before 2011 for the same flow. The model geometry parameters are fixed, so they can not reflect the general lowering of water surface elevation, therefore after the 2011 flood the model calculates stages that are too high. The difference between pre and post flood errors is about 1-ft, which matches the difference between pre and post flood stage measurements.

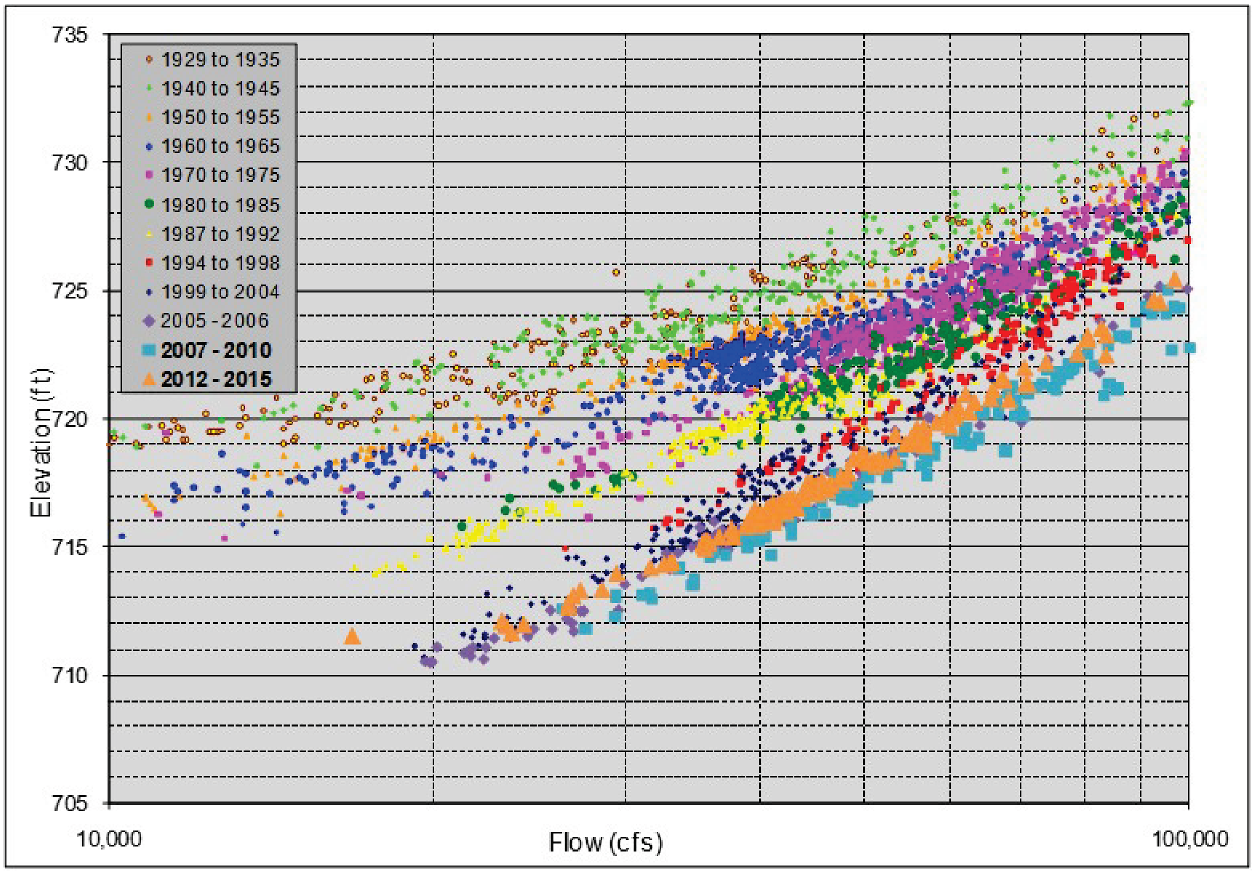

Figure 40. USGS Measured Flow vs. Stage – Kansas City gage

The Kansas City reach, from below Kansas City to St. Joseph has historically experienced significant degradation. Stage trend curves indicate that at the Kansas City gage a flow of 20,000-cfs measured a stage of 14-ft in 1930, 6-ft in 2000, and 4-ft in 2010 (USACE, Missouri River Basin Water Managment Division, 2012). However, in the most recent years the river bed appears to have rebounded slightly. Field measurements at the same flow rate correspond to stages that are consistently higher after the 2011 flood than before. One possible explanation is that aggregation occurred on the descending limb of the 2011 flood. The 2011 flood was more severe on the upper river and carried large sediment loads. A wider river at Kansas City along with less severe flow rates and velocities may have allowed some of the sediment load to deposit. Aggregation helps explain why the model consistently calculates too high before 2011, and too low after. From Rulo to the Mouth, Kansas City and St. Joseph have generally experienced more bed changes than other reaches, but there are a few other gages with calibration results that may show some impact from degradation/aggregation. Overall, calibrating to the entire 2007 to 2013 time window introduces uncertainty because of bed changes as a result of the 2011 flood.

There are also are some model stability issues associated with extreme low flows. HEC-RAS cannot complete a calculation with zero depth, but in reality some of the Missouri River tributaries do dry up to zero flow at times during the period of record. Minimum flows were used to avoid this scenario, which may cause the model to over predict flows during the lowest flow times during the period of record. Added together, the sum of all minimum flows is 1,040-cfs. This may mean that the model overpredicts the lowest flow in the period of record, but most of the time this is an insignificant amount compared to total Missouri River flow.

Model stability is also a challenge when flow in the Missouri and a tributary are simultaneously low. As unsteady calculations are solved as a matrix of every cross section at each timestep, these errors often ripple through the entire model and cause problems in unexpected places.

At flows that barely overtop navigation structures there are some minor instabilities, or perturbations, in the stage hydrograph, rating curve, and water surface profile. This is because of the choice to represent the navigation structures, primarily consisting of rock and or pile dikes, as permanent ineffective flows. The navigation structures are flat, so when they overtop by just a fraction of an inch, the top width in the cross section increases dramatically. There are also inconsistencies from one cross section to the next, when one structure overtops but the next downstream one doesn't. This is difficult to avoid because structure are constructed and maintained to the nearest foot. Even if the modeled elevations were adjusted to be more consistent with the exact water surface profile, it still would not be possible to have uniformity of overtopping at the same flow level, nor would it be true to reality. This is because in unsteady HEC-RAS the slope of the water surface profile tends to vary slightly over time, depending on if the river is rising or falling and how quickly.

Overall, confidence in model results when the river is below the top of bank is good. When water is within the banks, there are only a few parameters that influence the calculation of the water surface profile: channel geometry, the representation of navigation structures, and one channel Manning's roughness coefficient. However, none of these parameters can account for missing flow or bed changes over time, the two biggest contributing factors to uncertainty at low flows.

6.3.3 *Software and Hardware Limitations*

A model of this size and running for long periods of time pushes the limits of the software and standard computer hardware. The following are recommendations for computer hardware, as well as output limitations specific to this model.

The most important hardware recommendation is to have the newest available processor and the fastest available clock speed. The faster the processor speed, the faster the model will run. Multiple cores will, unfortunately, not speed up computations. Unsteady computations are single thread, which means they only use one core. More RAM will also not speed up computations, provided there is enough to run the unsteady computations. A standard 8 GB of RAM should be sufficient, depending on the operating system and other simultaneously running software. One unsteady model running uses less than 100 MB of RAM. The operating system should be 64-bit, the software will run in 32-bit but it will be considerably slower.

Model run time is the addition of the following steps: geometry pre-processor, unsteady flow calculations, write DSS hydrographs, and post processing. The unsteady flow calculations and post processing are by far the longest steps. Post processing time can be cut down by outputting only the maximum profile, or a longer detailed output interval such as 12 hr or 1 day. Post processing for a short interval such as 1 hr can take just as long as or longer than the unsteady calculations. On a computer with a 64-bit operating system, i7 processor and 2.8 GHz clock speed the 6-yr calibration period takes 8 hours to run, when only outputting the max profile. On a modeling computer with a Xeon processor and 3.40 GHz clock speed the same plan can run in 2 hours.

Detailed output is stored in a file with the extension *.O01, *.O02, *.O03, etc. Detailed output includes profiles, velocities, depths, and dozens of other parameters for cross sections, storage areas, and other components. Detailed output file size is limited to 2GB, once this limit is reached the profile and animation output as well as tables may not be able to be viewed. For this model, the 2GB limit is reached after about 1 month 10 days when requesting hourly data, and after 2 ½ years when requesting daily data.

Hydrographs are stored in a DSS output file. The DSS file stores stage and flow hydrographs for cross sections, storage areas, lateral structures, storage area connections, pumps, inline structures, etc. DSS file size limit is 8 GB. One run of the 6-yr calibration period outputting hydrographs at hourly intervals produces a DSS file size of approximately 1.2 GB. Therefore, only six different plans running this same time window and output interval can be saved to one DSS before reaching this limit. If the limit is reached, the model will successfully complete unsteady calculations, but not be able to move past the step of writing the DSS output, as well as the entire DSS file will be corrupt and all records previously stored in the DSS file will be lost.

6.3.4 *Model Improvements*

The unsteady HEC-RAS model in its current state meets the objectives of the project, however there were several tasks that could not be accomplished in the desired timeframe for completion. In addition, due to the nature of a river as a constantly evolving and changing system there will likely be newer and better terrain and bathymetry data and newer and better calibration data with every new season. The following captures several of the betterments that could be made to the model.

Before evaluating alternatives, a method to drain water from the storage areas after floods may need to be developed. Because of the decision to model all levees as overtopping without breaching, flooded levees will remain full of water until the end of the period of record as HEC-RAS does not have an automatic mechanism to remove water from storage areas. This causes obvious problems if there are two or more large events that flood similar levees over the period of record. There are two tools in HEC-RAS that could be used to drain water from the levees post-flood. One method would be to add a pump to each storage area, which would kick on after the river falls back down to within its banks after the flood event. Another method is to artificially breach the levee during the winter months, allow the levee to drain through the breach, and then repair it before the next flood season. HEC-RAS has a repair option in the breach tool. Both methods have advantages and disadvantages. Pumps are stored in the geometry file, but there is not an easy way to transfer pumps from one geometry to another once they are created. In addition, the lower elevations of the elevation-volume curves for many of the levees will need to be modified to stabilize pump operation when the remaining water volume becomes very small. Levee breaches are stored in the plan file, which means they can be easily applied to any alternative geometry once they are set up. However, the only way to trigger a breach is by time or water surface elevation. One of these methods will need to be implemented before making period of record runs.

An optional improvement that could be made would be to cut new cross sections. This would fix all georeferencing issues, bring the terrain and bathymetry data up to current conditions, and improve inundation mapping. The 3-meter LiDAR data was not available at the start of this effort, but is now available and in a mosaic form that is compatible with Geo-RAS. There have also been several full river hydrosurveys conducted, which would bring the bed data in the model up to current conditions. Cross section locations could generally remain the same. However it would be wise to re-evaluate the positioning of the overbank cut lines for multiple reasons. First, cut lines do not always extend all to the levee centerline, which causes slivers of mapping gaps on the riverside of levees. Second, cross sections are not always perpendicular to the lateral structures because the original cross section configuration was from bluff to bluff. In making revisions, mapping improvements could also be taken into account. Inundation mapping will end at the end of the bounding polygon, which is drawn from the end of one cross section to the end of the next downstream cross section. Sometimes this line does not encompass an area that would be flooded. By lengthening key cross sections or adding a few cross sections in strategic locations mapping could be greatly improved. Although worth the effort, n-values, navigation structures, ineffective flows, and base n-values may have to be re-evaluated, and the model would have to be re-calibrated.

Limited calibration data between gages means that the level of calibration at an area far from a USGS gage may not be as refined as at the gage. In order to improve this uncertainty the model would need to be calibrated to water surface profiles at several different flow levels, including bank full. Limited or no actual water surface data was available at this time for this level.

6.3.5 *Model Use*

Use of the model should take into consideration the limitations discussed in this report. Additional analysis or model refinement is recommended for the following model uses:

1. Detailed modeling of historic flood events.

   As discussed in Sections 6.2.6 and 6.2.7 on the 2011 and 2013 floods, to better replicate these historic events the ungaged inflow methodology should be re-considered and possibly revised to be specific to the rainfall distribution for each event. Breach data, including location, geometry, timing, and development rates, would need to be mined from flood event documentation, and likely many assumptions will need to be made to fill gaps of missing data. After ungaged inflow and breaches are input into the model and refined, additional adjustment of roughness factors and consideration of high water marks could be completed.

2. Predicting levee overtopping frequency in the period of record analysis.

As presented in **Attachments 12 and 13** comparing model results to high water marks collected after the 2011 and 2013 flood events, the model was shown to calculate 1 to 2 feet too low in areas where high peak ungaged inflows were occurring. Because of errors introduced by the simplified ungaged methodology, model calculations showing levees loaded within 1 to 2 feet of overtopping should be considered at risk of overtopping.

3. Without structure river, historic or otherwise.

   As discussed in Section 5.2.5, model calibration is only valid for present day conditions and does not evaluate sediment processes between navigation structures and river geometry.

4. Evaluating or updating flow frequency study.

   Updating the flow frequency study would require evaluation of an unregulated period of record. Earlier data back to 1898, more refined ungaged inflow calculations, and additional reservoir routings would have to be considered.

5. Evaluating event based profiles.

   No consideration was given to the 100-year or 500-year water surface profiles. The UMRSFFS profiles are still the best resource at this time.

7 CONCLUSIONS

The Rulo to the Mouth Missouri River HEC-RAS model is calibrated to match observed 15 minute stage data at the river gages for a six year time period between October 2007 and September 2013. On average, the model has a mean stage error of 0.1 feet with a root mean square stage error of 0.8 feet, 86% of the time the computed stage is within 1-ft of observed, and 97% of the time it is within 2-ft of observed. Flow volume for the 6 year window matches within a 0.6% error or less at all streamgages. Flow duration curves from the model are similar to observed data, generally matching within 10% for all frequency values. High flow calibration was also checked for the 2011 and 2013 flood events where reliable high water mark data was available. The model was shown to calculate up to 2-ft low in areas where high peak ungaged inflows were occurring, however, if these peak ungaged flows were accounted for the model was shown to appropriately reproduce the stage discharge relationship. Use of the model should take into consideration the limitations discussed in this report. Pending the final array of alternatives, some supplemental modeling may be required to further evaluate conditions beyond the limitations of the model. Generally, the stage-flow relationship produced by the model tracks well with current USGS rating curves at gage locations. The model is appropriate for comparing planning alternatives over a period of record.

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Attachment 1

Instantaneous Data Gap Fill Memorandum

Memorandum

To: *Jean Reed and Don Meier, United States Army Corps of Engineers (USACE) – Kansas City District*

From: *Matthew Scott*

Date: *October 24th, 2013*

Subject: *Missing USGS Gage Hydrograph Methodology*

This technical memorandum summarizes the methodology used to estimate missing flow data in United States Geological Survey (USGS) stream gage flow records. The methodology was developed by a collaborative effort between USACE and CDM Smith staff, and is completed in an Excel spreadsheet. The USGS records with missing flow data were instantaneous, with flows recorded at 15-minute intervals. The overall goal of this effort was to estimate missing flow values and preserve flow volumes. These records will be used to calibrate the HEC-RAS model of the Missouri River as part of the ongoing Missouri River Recovery Program.

The following bullets summarize the methodology.

- **Raw Data Collection.** Instantaneous flow and stage records were obtained from the USGS for the period from 2007 through 2012. These records were either downloaded from the USGS website or, when the records were not available on the website, obtained by the USACE directly from the USGS.
- **Gap Identification and Estimation of Gap Hydrographs.** Raw flow records were analyzed to find gaps in the flow record. These gaps were filled in based on stages recorded during the period of missing flow values using the rating curve for each gage.

  This was completed by first downloading the latest published rating curve from the USGS website. Because the USGS regularly updates rating curves, and because historical rating curves were not available on the USGS website, an approach which could use the latest published curve to estimate flows was developed. Because rating curves are updated by applying a shift to the curve such that flows are either raised or lowered, the latest published rating curve was shifted to approximate historical rating curves.

  A shift was calculated for each flow data gap by averaging the differences between observed flow values and flow values calculated using the latest published rating curve for each recorded value within the 24 hour period following each flow data gap. This shift was then applied to produce the estimated flow values used to fill in flow data gaps. Occasionally, application of the shift would result in negative flows, and in such cases the flow calculated from the latest published rating curve was used.

- **Estimation of Flows without a Stage Record.** Some gaps in the flow record coincided with gaps in the stage record. Where such concurrent gaps occurred, the rating curve could not be used to estimate missing flow data. Two approaches were developed to estimate flows for such a case. The first approach was to draw a straight line connecting the last observed flow and the next available observed flow. It was applied where the concurrent gap was relatively short, on the order of a few hours.

  The second approach was to estimate missing flows using the published daily flow value. For this approach, daily flows were assigned to the 1200 hour mark for each day of missing data, and a straight line drawn which connected the daily flows. This approach was taken when concurrent gaps in flow and stage data continued for over a day.

  While neither of these approaches preserved flow volumes as accurately as the rating curve approach, they produced reasonable results and provided flow estimates which had a basis in the observed flow record.

- **Creation of Final Flow Record and Quality Control.** The observed flow records were combined with estimated flows to produce a continuous record. These continuous records were then visually inspected for discontinuities or indications that estimated flows were reasonable as a quality control effort. Occasionally, use of the rating curve methodology produced flows which were not reasonable, and in such cases the two alternate approaches were used to replace them. The spreadsheets created to estimate flows for each gage include a summary table of methodology employed as a percentage of total hydrograph.

Attachment 2

Missouri River Gage Datum Verification Summary Table

Missouri River Gage Datum Verification - Summary Table

Gage Name	Gage Datum Selected for USACE Missouri River Model Efforts	Legacy Benchmark Surveyed by Powell and Associates (RM= Reference Mark, RP = Reference Point)	Survey Elevation of Legacy Benchmark (NAVD88)	Tie to Gage Zero (From Legacy Data)	USACE Calculated Gage Zero (NAVD88) (Survey Elev minus Tie to Gage Zero)	Legacy Gage Zero Elevation (NGVD29)	Legacy Gage Zero Elevation (NAVD88) (converted from 29 using Corpscon)	Recently posted USGS Gage Zero Elevation (NAVD88)	Difference between USACE calculated and most current Gage Zero (positive means current gage zero is lower)
Rulo, NE	**838.16**	LEGACY RP3	864.399	26.236	838.16	**837.23**	837.443		0.72
		LEGACY RM9	862.077	23.928	838.15				0.71
St Joseph, MO	**789.27**	LEGACY RM8	826.557	37.29	789.27			**789.27**	0.00
		LEGACY RM4-RR CROSS	828.880	39.59	789.29				0.02
Atchison, KS	**762.84**	GAGE STATION RM	786.837	23.995	762.84	**762.20**	762.525		0.32
Leavenworth, KS	**742.47**	LEGACY RM1	770.898	28.43	742.47			**742.48**	-0.01
		LEGACY RM2	770.892	28.43	742.46				-0.02
Kansas City, MO	**706.68**	LEGACY RM9 (GAGING STA RM)	765.610	58.93	706.68			**706.68**	0.00
		LEGACY RM11	770.620	63.94	706.68				0.00
		LEGACY RM13	772.540	65.87	706.67				-0.01
Sibley, MO	**684.40**	LEGACY RM 2	725.481	41.08	684.40	**683.92**	684.189		0.21
Napoleon, MO	**680.53**	A329	709.38	28.85	680.53	**680.24**	680.532		0.00
Waverly, MO	**646.17**	LEGACY RM 12	742.493	96.31	646.18			**646.17**	0.01
		LEGACY RM 13	741.779	95.60	646.18				0.01
		LEGACY RM 14	743.498	97.31	646.19				0.02
		LEGACY RM 15	739.952	93.77	646.18				0.01
Miami, MO	**621.73**	LEGACY RM-2	707.668	85.82	621.85	**621.35**	621.425		0.42
		LEGACY RM-3	710.603	88.81	621.79				0.37
		LEGACY RM-4	707.550	85.97	621.58				0.15
		LEGACY RM-5	710.550	88.87	621.68				0.25
Glasgow, MO	**586.65**	LEGACY RM4	622.029	35.378	586.65			**586.65**	0.00
		LEGACY RP3	617.258	30.59	586.67				0.02
Boonville, MO	**565.58**	LEGACY RM 14	599.208	33.65	565.56			**565.58**	-0.02
Jefferson City, MO	**520.18**	LEGACY BM 10	602.334	82.124	520.21			**520.18**	0.03
Chamois, MO	**503.19**	LEGACY BM TOP CURB	532.687	29.50	503.19	**502.5**	502.500		0.69
Gasconade, MO	**484.67**	31' MARK SLOPE GAGE	515.651	31.00	484.65	**484.80**	484.725		-0.07
		UPPER STAFF GAGE 40'	524.688	40.00	484.69				-0.04
Hermann, MO	**481.50**	LEGACY RM9	571.051	89.582	481.47			**481.50**	-0.03
		LEGACY RM10	568.780	87.292	481.49				-0.01
		LEGACY RM11	571.734	90.250	481.48				-0.02
Washington, MO	**457.27**	LEGACY RM1	491.698	34.422	457.28			**457.27**	0.01
St. Charles, MO	**413.47**	LEGACY RM8	447.270	33.808	413.46			**413.47**	-0.01
		LEGACY RM9	453.060	39.588	413.47				0.00

Attachment 3

Manning’s n-value Assignments

Channel – mainstem Missouri River – 0.02 to 0.03

Channel – tributaries – 0.025 to 0.040

Channel – Chutes – 0.028

Overbank – Grass/ pasture/ crops – 0.04 to 0.05

Overbank – Light to Dense Trees – 0.07 to 0.15

Attachment 4

Navigation Structure Test Reach Memorandum

Draft Technical Memorandum

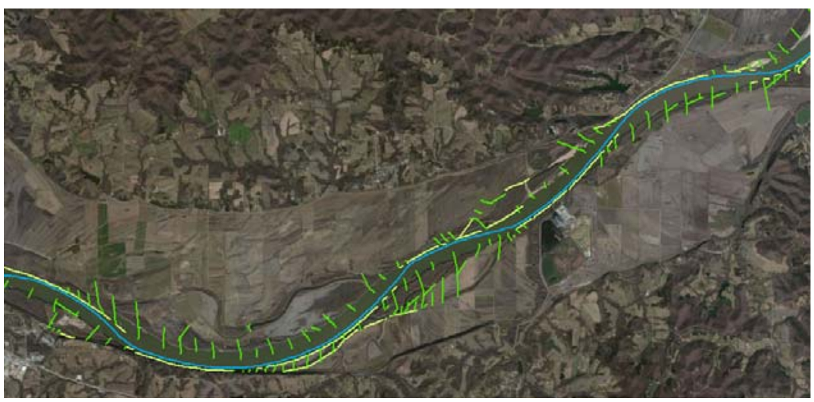
To: *Chance Bitner, USACE*
Don Meier, USACE
Jean Hilger, USACE

From: *Lisa Brink, CDM Smith*

Date: *August 2013*

Subject: *Contract Number WQ912DQ-08-D-0048, Task 1D*
Test Methods to Account for Missouri River Bank Stabilization and Navigation Project Structures Technical Memorandum (TM)

CDM Federal Programs Corporation (CDM Smith) was hired, contract number WQ912DQ-08-D-0048 (July 2012), to provide the Kansas City District support of various U.S. Army Corps of Engineers (USACE) projects under the Floodplain Management Services and Missouri River Recovery Programs. Task 1D of this contract, Test Methods to Account for Missouri River Bank Stabilization and Navigation Project Structures, includes testing Hydrologic Engineering Center River Analysis System (HEC-RAS) modeling methods to represent dikes, revetments, and related river training structures.



Dikes (in green) along the Missouri River.

Navigation structures are common along the Missouri River and were constructed as part of the Missouri River Bank Stabilization and Navigation Project completed in the early 1980s. The purpose of the project was to “train” the river and provide a much narrower and deeper channel for navigation than natural conditions allowed.

Determining a way to represent navigation structures in unsteady HEC-RAS will provide more accurate prediction of low stages and flows making the model a tool that is useful for a variety of projects.

Methods used for representing navigation structures are to be performed on an approximately 100-mile reach of the Missouri River from Boonville, Missouri, to Hermann, Missouri. These structure test methods include:

- Ineffective flow areas, both permanent and temporary.
- Manning's n vertical and horizontal variation by stage and/or flow.

The objective is to determine the most accurate method for considering the effects of these structures over a range of flows (low flow to minor flooding).

Missouri River navigation structure study reach.

Task 1D Assumptions

Assumptions for Task 1D as provided by the Kansas City District are:

- Location and dimensions of structures will be estimated based on aerial photography, record drawings, or other data provided.
- Three test methods will be identified and tested along the test reach.
- There are approximately 2 structures per river mile for the 100-mile reach (200 structures).

- Criteria for determining accuracy of the methods will be based on observed Missouri River gage data.

- Additional direction will be provided during meetings.

These assumptions and variations from these assumptions will be addressed in more detail throughout the remainder of this TM.

Background Data and Information

HEC-RAS version 4.2 Beta provided by the Kansas City District was used to complete Task 1D, HEC-RAS Test Methods to Account for Missouri River Bank Stabilization and Navigation Project Structures. An uncalibrated HEC-RAS 4.2 model of the study reach (Boonville to Hermann, MO) was provided for use as the starting point for this analysis.

The District provided information regarding navigations structures along the Missouri River, as well as a hard copy version of the *1994 Missouri River Hydrographic Survey*. A comprehensive list of data provided is provided in Table 1.

Table 1. Summary of data and information provided

Data	Provided by
GIS shapefiles and layers:	Kansas City District USACE
Dike_centerline	
Notch_line	
Revetment_centerline	
River2D	
XS_Cut_Lines	
XS_MO_R_Boon-Herm	
Rectified_channel_line (RCL)	
July (Marion) and August (Rocheport and Searcy) 2005 Acoustic Doppler Current Profiler (ADCP)	
Documents and other files:	
HEC-RAS 4.2 Model - MoRiverNAV	
1994 Missouri River Hydrographic survey (hard copy)	
June 2012 Lower Missouri River Hydrographic Survey.pdf	
1994_Jan BSNP (navigation project) Design Criteria.pdf	
Missouri River Navigation Structures.pptx	
Navigation_Structure_Typical_Drawings.pdf	

Navigation Structures

Navigation structures along the Missouri River include dikes, sills, and revetments. Dikes are constructed nearly perpendicular to flow and these structures contract the river, increasing velocity, encouraging scour and promoting a deeper and narrower channel. Sills are low elevation extensions from the end of dikes into the river and are used to control the shape of the channel cross section (XS). Revetments are constructed parallel to flow and assist in guiding flow and protecting channel banks.

The three methods tested for representing these structures in this study are non-permanent (temporary) and permanent ineffective flow areas and vertical/horizontal variation of Manning's n. Details of these analyses are presented below.

Navigation Structures as Ineffective Flow Areas

Ineffective flow areas are portions of the XS where flow is not actively being conveyed. In these areas water ponds, but has a velocity very near zero. These areas are included in storage calculations, but are not included as part of the active flow and conveyance area.

Ineffective flow areas can be either non-permanent (temporary) or permanent. Temporary ineffective flow areas are valid until water reaches a specified elevation at which the entire ineffective area is able to convey flow. Permanent ineffective flow areas do not convey flow even once the specified elevation is reached, and remain ineffective.

Temporary and permanent ineffective flow areas account for two of the three methods used for testing the effects of navigation structures in this study.

Temporary Ineffective Flow Areas

The approach used to model navigation structures as ineffective flow areas was discussed with the Kansas City USACE in advance of completing the modeling analysis. This approach uses the 1994 hydrographic survey to estimate dike and sill lengths and elevations for each river mile (RM) from Boonville, MO (MO R XS 196.62) to Hermann, MO (MO R XS 97.93).

Although GIS shapefiles of existing dikes and revetments have been provided, specific data related to structure elevations have not been included within these geographically referenced files. Therefore, this data was extracted from the hard copy. The 1994 hydrographic survey data was used because each plan view sheet of the river also provides detailed dike, sill, and revetment station and elevation data. The format of the 2012 hydrographic survey is different than the 1994 hydrographic survey sheets in that it lists all of the construction records on a separate sheet making the extraction and summary of this data more time and labor intensive. Limitations of the 1994 hydrographic survey include notch cutting, which started around the year 2000, and an overall lowering of the dikes through the Kansas City reach of the Missouri River.

Below is the step by step approach used to characterize navigation structures using ineffective areas in the HEC-RAS model.

Step 1: Create the following shapefiles separately for the left and right banks:

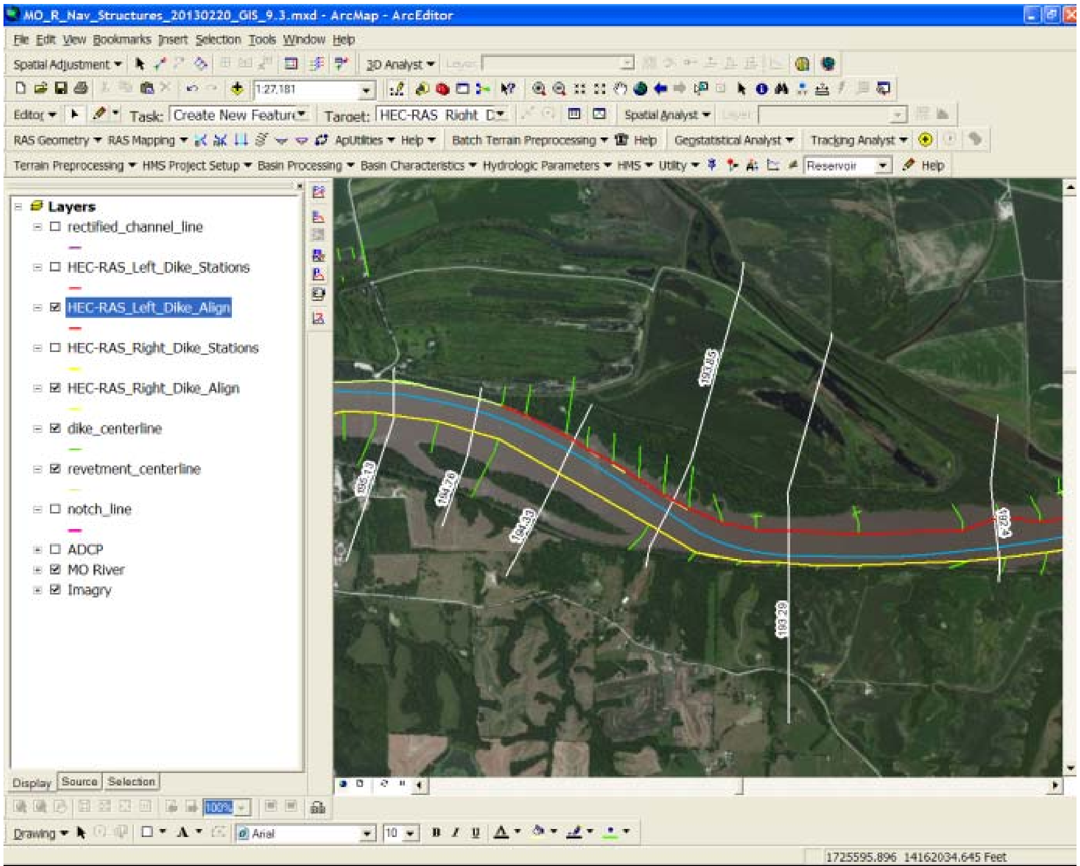
- "HEC-RAS_(LEFT OR RIGHT)_Dike_Align" with fields RM, APPROX_CRP, DIKE_ELEV, SILL_ELEV, and SILL_LENG.
- "HEC-RAS__(LEFT OR RIGHT)_Dike_Stations" with fields XS and FROM_STA (right overbank) or TO_STA (left overbank).

Step 2: Shapefile "...._Dike_Align": For each RM and separately for both the left and right banks, draw a line from end point to end point of each dike connecting several dikes. In this way, you can visually understand how the dikes (and other navigation structures) are affecting each XS and the reach between XS. If a sill extends from the end of the dike, the line is drawn to the end point of the sill. In this instance, the sill length is subtracted/added (left bank/right bank) from the end point to arrive at the correct station of the dike and sill.
(Note: If a rectified channel line shapefile is available for the river, this can be used in place of creating the "_Dike_Align" shapefiles. The rectified channel line represents the navigable channel created by the use of the dikes. Sills are not present for all dikes, however, when present; sills extend into the channel beyond the rectified channel line. In other words, the rectified channel corresponds to the end of the dike and not to the end of the sill.)

In this shapefile, estimate the following information (per RM) based on the information provided in the 1994 Missouri River Hydrographic Survey:

- Approximate C.R.P. (construction reference plane)(informational purpose only).
- Approximate dike elevation (for use in HEC-RAS).
- Approximate sill elevation (for use in HEC-RAS).
- Approximate sill length (for use in HEC-RAS).

These values are approximate because the information is being estimated for all navigation structures per RM. Each RM may have navigation structures of varying height. Some variability is also introduced by the size of rock used to construct the navigation structures.

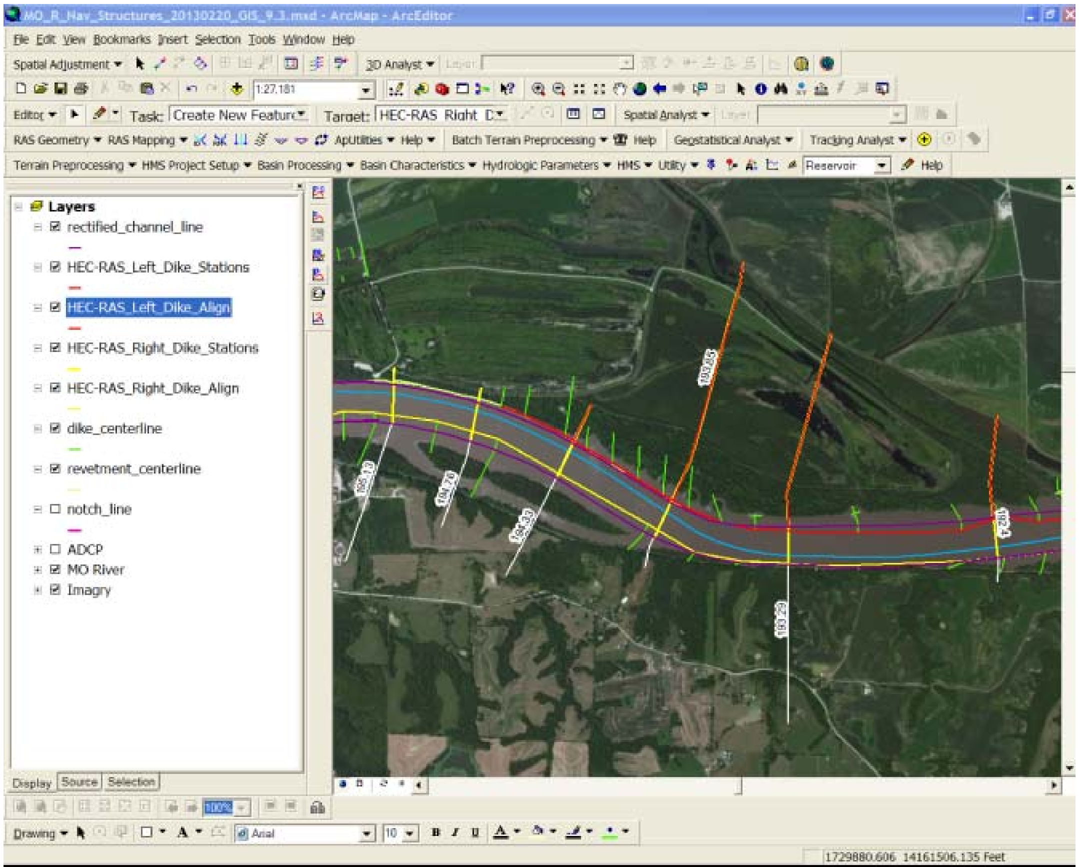


Step 2. Dike Alignment shapefiles.
Note: If available, rectified channel line shapefile can be used in place of creating Dike Alignment shapefiles.

Step 3: Shapefile "...Dike_Stations": At each XS (separately for the left and right), snap to and draw a line from the left side (looking downstream) of the XS to the point where the XS intersects the "_Dike_Align" or "rectified_channel_line" shapefile.

For the "HEC-RAS_Right_Dike_Stations", fill in the "FROM_STA" field, which is the calculated length of the line drawn and used as the HEC-RAS XS **starting** station for the right dike/ineffective flow area (navigation structure). The sill length will be subtracted from this **starting** station value, as the rectified channel line is the point at which the dike begins.

Follow the same procedure for the "HEC-RAS_Left_Dike Stations." However, for the left dike shapefile the "TO_STA" will be populated, which will be the HEC-RAS **end** station of the left dike/ineffective area (navigation structure). For sills attached to the left dike, the length of the sill will be added to the **end** station.



Step 3. Dike Station shapefiles.
Note: If available, rectified channel line shapefile can be used in place of using Dike Alignment shapefiles.

Step 4: There is no straightforward way to maintain existing ineffective flow areas within HEC-RAS when transitioning from “normal” to “multiple block”, so it is a manual process. Even with the "edit ineffective stations" box checked in HEC-RAS, there is no way to copy the river stations, though you can copy station and elevation information. Additionally, the ineffective flow table only shows those XS with ineffective flow areas.

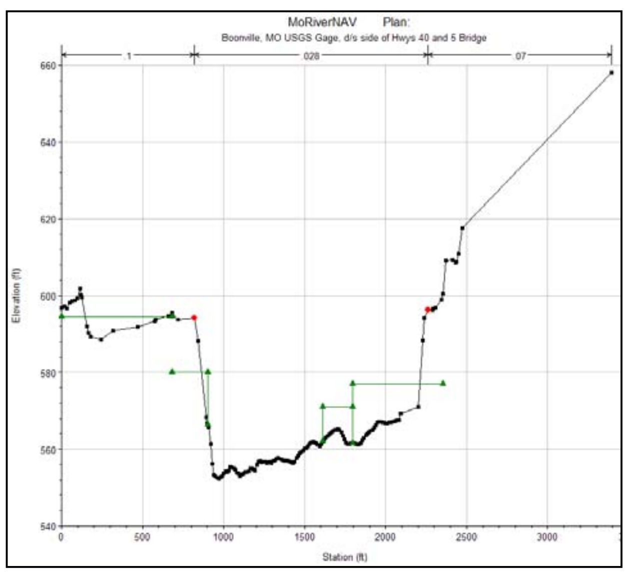
Two spreadsheets were created in Excel in the development of the process used to create "multiple block" ineffective flow areas from the data summarized in the shapefiles, as well as the ineffective flow areas presently existing within the HEC-RAS model.

The spreadsheet "Ext Ineff and Dike Info Summ" contains raw data from HEC-RAS and GIS. Existing ineffective areas were copied from HEC-RAS (Geometry Data Editor: Tables-Cross Section Ineffective Flow Area's Elevation Table) with the "Edit Ineffective Stations" box checked, so that both stations and elevations are shown. Since only those XS with ineffective flow areas are shown in the table and these stations cannot be copied from the ineffective area flow table, screenshots were captured and used in excel tab "Exst Ineffective" to associate XS with their ineffective flow areas.

For the GIS shapefiles, "HEC-RAS_Left/Right_Dike Stations" and "HEC-RAS_Left/Right_Dike_Align," that were created and completed in steps 1-3, the database files (.dbf) were opened and the tab copied into the "Exst Ineff and Dike Info Summ" spreadsheet. New Excel spreadsheet tabs "Left/Right Dike and Sill Info" were created, which summarize (using VLOOKUP) the data from the other tabs. In this way, the information is formatted such that it can be copied and pasted as HEC-RAS ineffective flow area multiple block format.

The information from existing ineffective areas was then combined into HEC-RAS multiple block format (in chronological station order) with dike information for each RM related to relevant XS.

River Station	Starting Station	Ending Station	Elevation
196.62	0	683	594.5
	683	902	580
	1613	1795.06	571
	1795.06	2351	577
196.54	0	682.26	594.3
	682.26	890	580
	1591	1802.12	571
	1802.12	2228	577
196.29	0	677	594
	677	879	580
	1570	1789.91	571
	1790	2191	577
195.69	0	509	593.8
	1451.2	1701	571
	1701	1800	577



Step 4. Multiple block formatting in Excel for use in HEC-RAS.

Permanent Ineffective Flow Areas
The permanent ineffective flow areas are the same as the temporary. The change is in the HEC-RAS ineffective flow areas model setting. In the ineffective flow - multiple blocks editor, the “n” for not permanent are changed to “y” for permanent ineffective flow areas. Unfortunately, the global “Set to Permanent Mode” could not be used in the analysis because existing ineffective flow areas are not permanent.

Navigation Structures as Manning’s n Values

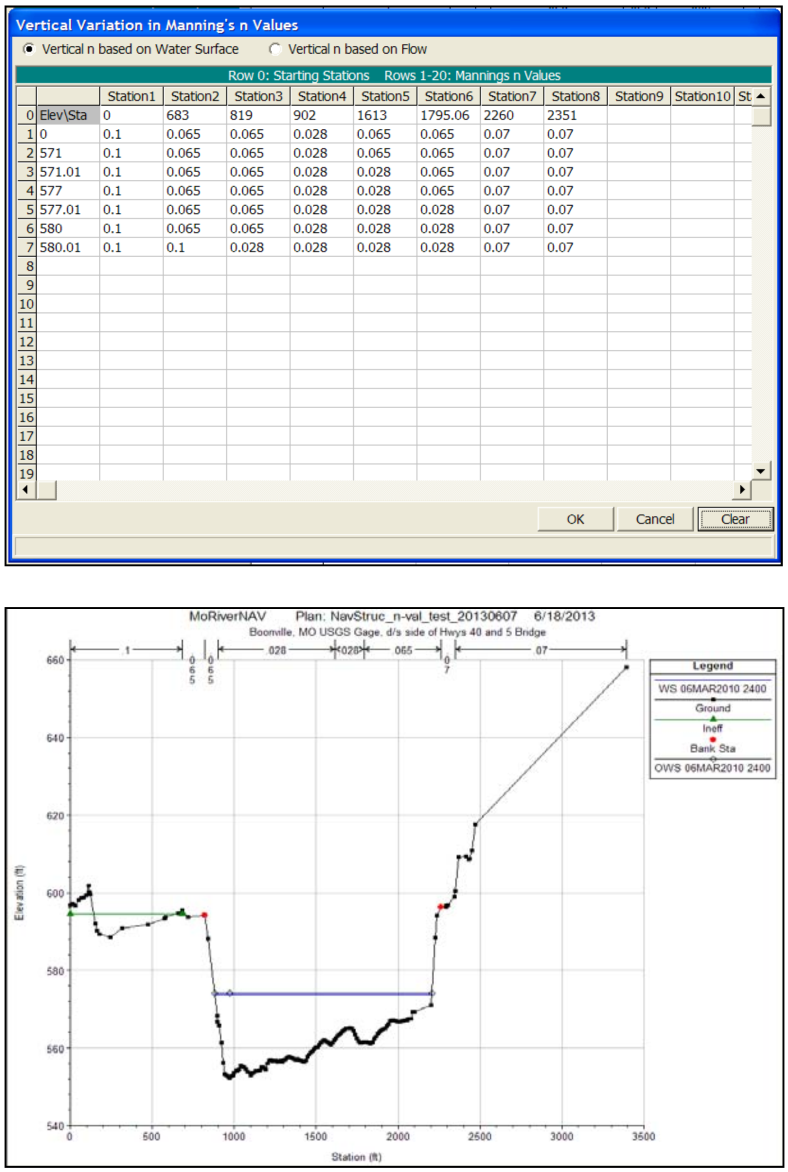
The second method used to represent navigation structures in HEC-RAS is Manning’s n values varied vertically and horizontally. In HEC-RAS, the n value can be varied by elevation or flow. For this analysis, n-values were varied by elevation since the ineffective flow area elevations had already been summarized.

For this method, Steps 1 through 3 from the ineffective flow method are the same and should be followed in order to summarize navigation structure data per RM. The process for copying in vertical and horizontal locations of varying Manning’s n-value requires some additional effort compared to the ineffective flow area multiple blocks simply because the HEC-RAS format is different for each method.

Vertical variation of Manning’s n-value, not only varies vertically, but horizontally as well (Figure 1). The “station” values across the top (column headings) are the same as those used for horizontally varied only Manning’s n values. In other words, the new specified Manning’s n-value will remain that value until a different value is specified at the next station. The rows are used to indicate the elevations at which the Manning’s n value changes. HEC-RAS will interpolate Manning’s n values whenever the actual water surface is between entered elevations. If the calculated water surface is less than the first elevation in the table, the n-value from that elevation will be used for all water surfaces below that elevation. In a similar way, if the calculated water surface is greater than the last elevation in the table, the program will use that n-value for all water surfaces greater than that elevation. No extrapolation is done beyond the first or last elevation in the table.

Although there is probably some transitional zone between the top of the navigation structure and where the structure no longer has an impact on stage and flow, the vertical variation Manning’s n table has clean breaks indicating the top of each navigation structure for testing this method. For example, if the top of the navigation structure is elevation 571, the value following it would be 571.01 indicating the top of the structure and a change in Manning’s n values, leaving little room for interpolation.

A Manning’s n value of 0.065 was used as the starting point for representing navigation structures, as this value is representative of the type of structure constructed along the Missouri River. In addition to including stations where navigation structures begin and end, it was also critical to include existing model bank stations, as they are oftentimes a location for transition from one Manning's n value to another.



Vertical Variation in Manning's n Values

Vertical n based on Water Surface   Vertical n based on Flow

Row 0: Starting Stations   Rows 1-20: Mannings n Values

		Station1	Station2	Station3	Station4	Station5	Station6	Station7	Station8	Station9	Station10	St
0	Elev\Sta	0	683	819	902	1613	1795.06	2260	2351			
1	0	0.1	0.065	0.065	0.028	0.065	0.065	0.07	0.07			
2	571	0.1	0.065	0.065	0.028	0.065	0.065	0.07	0.07			
3	571.01	0.1	0.065	0.065	0.028	0.028	0.065	0.07	0.07			
4	577	0.1	0.065	0.065	0.028	0.028	0.065	0.07	0.07			
5	577.01	0.1	0.065	0.065	0.028	0.028	0.028	0.07	0.07			
6	580	0.1	0.065	0.065	0.028	0.028	0.028	0.07	0.07			
7	580.01	0.1	0.1	0.028	0.028	0.028	0.028	0.07	0.07			
8												
9												
10												
11												
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19												

OK   Cancel   Clear

Figure 1. Manning's n - vertical variation input table and XS representation.

HEC-RAS Results - Modeling Navigation Structures

It is important to note again when discussing the results of this modeling analysis that the HEC-RAS 4.2 model provided to CDM Smith from the Kansas City USACE for use in this study was not a calibrated model and is only a 100-mile reach (Boonville to Hermann) of the unsteady HEC-RAS model to-date. Therefore, the stages and flows for the "No Structure" model have not been revised in order to best match observed values.

Since the model is not calibrated, the results of each of the three methods tested: Temporary and permanent ineffective flow areas and vertical variation in Manning's n-values, per the USACE request, will be compared to the no structures model.

The unsteady model that the Kansas City USACE has developed for the Missouri River from Nebraska City, NE, to St. Louis, MO, is already complicated; especially when simulating multiple years, therefore, some consideration must be given for qualitative modeling factors, such as level of effort, model stability and run times. Obviously, it is also important to understand how each method impacts model results and how these methods may be used independently or in conjunction with one another to yield the best quantitative results.

The remainder of this TM will focus on a combination of qualitative and quantitative indicators for the three methods instead of making a conclusive recommendation.

Qualitative Indicators

Based on discussion with the Kansas City USACE, there are several qualitative indicators, which are important for determining which method or combination of methods will provide adequate quantitative results while still practical to apply. These indicators include: level of effort for each methodology, model stability and model run times, which are, in effect, dictated by model stability.

For all three methods, steps 1-3 are a very similar level of effort. For this 100-mile reach of river (Boonville to Hermann), there were 184 XS. Steps 1-3 took approximately 50 hours or a little more than 15 minutes per XS. Step 4 is different for each approach in that information is entered either into the ineffective flow area or vertical Manning's n-value HEC-RAS data editor. Step 4 is somewhat tedious and took approximately 12 hours or about 5 minutes per XS. Practically, each method is approximately the same level of effort, which is estimated at 20 minutes per XS. However, the longer the process is applied by the same individual, the less time each step takes.

Model stability and model run times are associated factors. The less stable a model is, the greater number of iterations it must perform to arrive at a solution, and the longer it takes for the model to run. Initially, the results used to provide the quantitative comparison were provided for a test period, March 1-31, 2010. However, to understand on a practical level how long the entire unsteady HEC-RAS Missouri River model may take to run, all four years of provided gage data was used, October 2007 to October 2011. The run times are shown in Table 2 below. Run times are dependent on individual computers and additional applications that are open while the model is running. All conditions were consistent for the run times shown. Model computation settings for all are as follows:

- Computation Interval: 10 minutes
- Hydrograph Output Interval: 1 hour
- Detailed Output Interval: 6 hours

Table 2. Model run times

Method/Model Plan	Computation time (min)	Process time (min)	Total model run time (min)
4 YR N-VAL	7	15	23
4YR INEFF	8	15	24
4YR PERM INEFF	8	16	26
NO STRUC	12	14	28

*Rounded to the nearest minute, which is why total model run time may not match the sum of the other two.

It is interesting to note that run times are actually lower for those model runs that include a method for modeling navigation structures. Of the three methods, vertical Manning's n and temporary ineffective flow areas are essentially the same with permanent ineffective flow areas only taking a couple of additional minutes to run. In general, based on stability and run times, all methods are equal.

Quantitative Indicators

A quantitative indicator analysis was completed for model parameters used in the three test methods.

In general, this analysis was a sensitivity analysis to determine a quantified range of likely possible values given the uncertainty in the input data. In addition, a sensitivity analysis was also completed for the Manning's n-value of the navigation channel to quantify the potential changes in n-value that may occur for calibration.

For this reach of the Missouri River, the model includes observed stage and flow data for three locations including: Boonville (XS 196.62), Jefferson City (XS 143.86) and Hermann (XS 97.93). However, at this point, using an uncalibrated model, it is more critical to understand how the model reacts to the different approaches rather than how it compares to observed data. Therefore, nine

XS along the study reach were chosen for results comparison, including those locations for which there is observed data. Results in this section are compared to the "No Structure" (base) condition and are shown for a test period, March 1-31, 2010, which represents a range of stages and flows.

A series of plots, which include stage (solid) and flow (dashed) comparison for no structure, temporary and permanent ineffective and vertical Manning's n-values are attached at the end of this TM. The shaded areas on each plot represent the range of elevations to which navigation structures are constructed near that XS. It is important to understand the effect these structures potentially have on stage as well as flow and timing. The plots are ordered starting from the upstream (Boonville) to downstream (Hermann) with locations approximately every 20 RM. As shown on these plots, the permanent ineffective flow method had the most impact on the simulated river stages. The other methods only impact results at lower stages.

The plots generally show that navigation structures modeled as temporary ineffective flow and Manning's n-value have some variation over the range in stage, which are affected by structures (low stages). At these low stages, the temporary ineffective flow areas have a greater impact on stage results (results are more different from the base condition) than do changes in n-values. The permanent ineffective flow areas show an even greater difference from the base condition than the temporary ineffective flow areas, which is expected because the permanent ineffective areas are ineffective for any stage, whereas the temporary ineffective areas are discounted at any elevation above the elevation at which they are set.

Flow is not affected in the same way as stage. For the range of flows, there appears to be little difference in flow and timing for the base, temporary ineffective flow, and n-value results. However, the permanent ineffective flow areas do impact both flow and timing.

A summary table of peak stage and timing and peak flow and timing for each XS and method are shown in Table 3. In addition to the stage and flow hydrographs, this table illustrates that there is no significant change in peak stage or flow from the No Structure model results, except for the Permanent Ineffective flow areas.

Table 3. Peak Stage/Flow and Time for March 2010 Model Runs

Location	Modeling method	Peak stage (ft)	Peak stage time	Peak flow (cfs)	Peak flow time
196.62	No Struc	591.90	14Mar2010 2000	247,000	14Mar2010 1600
Boonville	Non-Perm Ineff	591.89	14Mar2010 2000	247,000	14Mar2010 1600
	Perm Ineff	594.68	14Mar2010 2000	247,000	14Mar2010 1600
	N-val	592.05	14Mar2010 2000	247,000	14Mar2010 1600
180.68	No Struc	578.67	14Mar2010 2300	244,923	14Mar2010 2200

Table 3. Peak Stage/Flow and Time for March 2010 Model Runs					
Location	Modeling method	Peak stage (ft)	Peak stage time	Peak flow (cfs)	Peak flow time
	Non-Perm Ineff	578.66	14Mar2010 2300	244,924	14Mar2010 2200
	Perm Ineff	580.7	14Mar2010 2300	244,791	14Mar2010 2200
	N-val	578.55	14Mar2010 2300	244,921	14Mar2010 2200
160.86	No Struc	560.81	15Mar2010 1300	241,734	15Mar2010 0400
	Non-Perm Ineff	560.77	15Mar2010 1300	241,754	15Mar2010 0400
	Perm Ineff	562.94	15Mar2010 1400	241,324	15Mar2010 1200
	N-val	560.68	15Mar2010 1300	241,763	15Mar2010 0400
143.86	No Struc	545.28	15Mar2010 1500	241,391	15Mar2010 1400
Jefferson City	Non-Perm Ineff	545.28	15Mar2010 1500	241,403	15Mar2010 1400
	Perm Ineff	548.63	15Mar2010 1700	241,020	15Mar2010 1500
	N-val	545.03	15Mar2010 1500	241,403	15Mar2010 1400
138.11	No Struc	539.96	15Mar2010 1700	241,325	15Mar2010 1400
	Non-Perm Ineff	539.96	15Mar2010 1700	241,338	15Mar2010 1400
	Perm Ineff	543.24	15Mar2010 2200	240,933	15Mar2010 1500
	N-val	539.75	15Mar2010 1700	241,338	15Mar2010 1400
133.12	No Struc	535.64	16Mar2010 0300	241,779	15Mar2010 1400
	Non-Perm Ineff	535.65	16Mar2010 0300	241,793	15Mar2010 1400
	Perm Ineff	539.82	16Mar2010 1000	241,211	15Mar2010 1600
	N-val	535.47	16Mar2010 0300	241,801	15Mar2010 1400
119.25	No Struc	523.9	16Mar2010 0800	257,533	16Mar2010 0600
	Non-Perm Ineff	523.9	16Mar2010 0800	257,539	16Mar2010 0600
	Perm Ineff	527.43	16Mar2010 1600	254,883	16Mar2010 1400
	N-val	523.77	16Mar2010 0800	257,529	16Mar2010 0600
103.19	No Struc	510.37	16Mar2010 1200	259,586	16Mar2010 1100
	Non-Perm Ineff	510.37	16Mar2010 1200	259,584	16Mar2010 1100
	Perm Ineff	512.79	16Mar2010 2000	256,711	16Mar2010 2000
	N-val	510.3	16Mar2010 1200	259,578	16Mar2010 1100
97.93	No Struc	506.92	16Mar2010 1200	259,567	16Mar2010 1200
Hermann	Non-Perm Ineff	506.92	16Mar2010 1200	259,563	16Mar2010 1200
	Perm Ineff	506.75	16Mar2010 2100	256,692	16Mar2010 2100
	N-val	506.92	16Mar2010 1200	259,556	16Mar2010 1200

ADCP Data and Velocity Comparison

Acoustic Doppler current profiler (ADCP) data was provided as GIS shapefiles for five locations; Rocheport, Searcy, Marion (2005), Boonville, and Hermann (2011-2012). While the data for Rocheport, Searcy, and Marion collected on one day only corresponds to a single flow, data at Boonville and Hermann was collected over a number of days and range of flows. ADCP meters measure water velocities over a range of depths using the Doppler effect by which sound waves are scattered back from particles within the water. This data provides another way in which the HEC-RAS model methods for representing navigation structures can be compared to measured data.

There are over a hundred locations of transecting ADCP data that vary by depth and flow. Gage data was provided for October 2007 to October 2011 for the HEC-RAS model navigation structure test methods. Since much of the ADCP data was collected prior to or after this time frame, it was important to find flows within this period that were comparable to the flows on the dates when the ADCP velocity data was collected. Information used for velocity comparison purposes are summarized in Table 4. The data collected at Hermann, Missouri was not used for comparison because it is the model downstream boundary condition and, at this point, is controlled by an uncalibrated rating curve.

Table 4. ADCP and HEC-RAS velocity comparison information

Location	Date ADCP data collected	USGS average daily flow*	HEC-RAS model representative XS	HEC-RAS model date and time used for comparable flow	Approximate HEC-RAS comparable flow
Marion	July 26, 2005	36,200	160.26	November 17, 2007 1200	36,300
Searcy	August 5, 2005	35,900	179.15	November 17, 2007 1200	36,200
Rocheport	August 23, 2005	59,700	182.7	January 13, 2008 0600	58,300
Boonville low flow	June 14, 2012	54,900	195.69	August 19, 2008 1200	54,400
Boonville mid flow	May 10, 2012	92,900	195.69	July 4, 2009 1200	93,400
Boonville high flow	September 1, 2011	186,000	195.69	August 31, 2011 0600	184,700

*USGS 06909000 (Boonville, MO) was the closest gage for each location.

The variation in velocity across the channel is of interest in understanding how navigation structures affect the navigable channel. The ADCP data is collected at numerous points across the channel and HEC-RAS has an option, which allows the user to break a specific XS(s) into a number of subsections for which the velocity is reported ("Unsteady flow editor"-"Flow distribution locations"). For this comparison, each HEC-RAS channel XS of interest was divided into 40 subsections. ADCP data nearest the XS of interest was used in the velocity versus station plots included at the end of this TM. It is important to note again while reviewing the results that the measured ADCP velocity data is being compared to uncalibrated HEC-RAS model results.

The plots include ADCP measured velocity data compared to model velocity results for the no Structure, n-value, temporary ineffective and permanent ineffective flow area runs. Generally, the permanent ineffective flow areas best represent the measured data velocity with the exception of the downstream location, Marion (XS 160.26). At this location, both the temporary and permanent ineffective flow area velocities are low. However, the results still fall within the range of measured ADCP values.

Sensitivity Analyses

A sensitivity analysis of the temporary ineffective flow area method and vertically varied n-values methods was completed to understand how changes made to parameters affect model results. The Kansas City USACE is currently calibrating the unsteady HEC-RAS Missouri River model and Manning's n values and even ineffective flow areas are often tools used to calibrate HEC-RAS models. Therefore, it is critical at this time to gain an understanding of how calibration may impact results when these parameters are modified.

Manning's n-value

The Manning's n-value for navigation structures was set to 0.065 for the initial evaluation. This is a Manning's n representative of shallow flow over riprap. To test the sensitivity of model results to a variation in navigation structure n-values, the original n-value, 0.065, was both increased and decreased by 25%, 0.08125 and 0.04875, respectively.

Plots similar to those used to illustrate the qualitative results comparison between methods are attached at the end of this TM for the +/- 25% change in Manning's n-value.

Temporary Ineffective Flow Area

A sensitivity analysis similar to that of the Manning's n value was completed for ineffective flow areas navigation structures. Each navigation structure elevation was increased and decreased by 2-ft. Navigation structures, including sills, along this 100-mile reach of the Missouri River have an average range in elevations of approximately 8-feet, so a 2-feet change in elevation represents an approximate 25% change in elevation, which makes the impact of changes between n-value and ineffective flow area sensitivity comparable.

Plots similar to those used to illustrate the qualitative results comparison between methods are attached at the end of this TM for the approximate +/- 25% change in ineffective flow area elevations.

Permanent Ineffective Flow Area

For this sensitivity analysis, the temporary ineffective flow areas navigation structures and navigation structures +/- 2-ft were set to permanent ineffective flow areas.

Plots similar to those used to illustrate the qualitative results comparison between methods are attached at the end of this TM for the approximate +/- 25% change in permanent ineffective flow area elevations.

Sensitivity Analyses Discussion

The results of the sensitivity analyses for navigation structures as temporary and permanent ineffective flow areas and Manning's n-values, as shown by the plots, indicates that when looking at stage, the n-value is more sensitive to change than are temporary ineffective flow areas. An example of this is particularly evident when looking at the XS 103.19 plot for both the n-value and temporary ineffective flow area sensitivity analysis results. At this location, the base, +25% and -25% temporary ineffective flow area stage results are 0.25-ft or less than one another over the range in stages of affective navigation structures (shaded area). However, the base, +25% and -25% Manning's n-value stage results are each separated by approximately 0.5-ft over the same range. Permanent ineffective flow areas, like temporary ineffective flow areas are not particularly sensitive at low stages, but are somewhat more sensitive at high stages with variation of approximately 0.5-ft for the +/- 25% change from the normal navigation structure elevations. While each of these methods does show some sensitivity, it is important to note that for a relatively large change in elevation (+/- 25%) there are not significant changes in the stage results.

Flow and timing do not seem to be significantly affected when temporary ineffective flow area elevations and Manning's n-values as navigation structures are increased or decreased by 25%. However, an increase or decrease in navigation structure elevations using permanent ineffective flow areas does show some variation in peak flow and timing. Again, however, the effect is relatively minor.

Additional Sensitivity Analysis – Manning's n-values

The results of the initial sensitivity analysis showed that variations made to a base navigation structure Manning's n-value impacts model results more than variations made to base navigation structure ineffective flow area elevations. Manning's n-values are often an effective way of calibrating models and also seems to be an effective and appropriate way of modeling navigation structures.

The Kansas City USACE indicated that since the model is not fully calibrated, there may still be some modifications made to channel Manning's n-values. This additional analysis was performed, in part, to more fully understand how modifications made to the channel Manning's n-values may affect model results.

Table 5 is the comprehensive list of model runs completed to test the impact of changing Manning's n-values.

Table 5. Manning's n-value Comprehensive Sensitivity Analysis		
Model Run	Channel n-value	Dike n-value
NO STRUC TEST	0.028	NA
NO STRUC CH N-VAL -25%	0.021	NA
NO STRUC CH N-VAL +25%	0.035	NA
N-VAL NS TEST	0.028	0.065
+25% N-VAL TEST	0.028	0.08125
-25% N-VAL TEST	0.028	0.04875
CH -25 EXST DIKE N NS	0.021	0.065
CH -25 DIKE+25%N NS	0.021	0.08125
CH -25 DIKE-25%N NS	0.021	0.04875
CH +25 EXST DIKE N NS	0.035	0.065
CH +25 DIKE+25%N NS	0.035	0.08125
CH +25 DIKE-25%N NS	0.035	0.04875

Additional Sensitivity Analysis – Manning's n-values Discussion

Of the 12 model runs shown (Table 5), the results of only 5 (Table 6) have been plotted for 5 locations along the 100-mile reach. These plots are attached at the end of this TM. The five model runs plotted bracket the high and low range of possible stage and flow results, as well as show some difference in sensitivity at low and high stages. Since calibration is relevant to this analysis and discussion, observed values (gage data) have also been plotted where available.

Table 6. Plotted Model Runs		
Model Run	Channel n-value	Dike n-value
NO STRUC CH N-VAL -25%	0.021	NA
NO STRUC CH N-VAL +25%	0.035	NA
N-VAL NS TEST	0.028	0.065
CH +25 DIKE+25%N NS	0.035	0.08125
CH -25 DIKE-25%N NS	0.021	0.04875

Stage and flow results for each of the five locations and five runs have been plotted separately.

Where observed stage data is available, the original Manning's n-value navigation structure model run (N-VAL NS TEST) show results in relative agreement with observed values for affective navigation structure stages (shaded area), but show some variation at higher stages. Generally, stage results for each location are as expected, relative to the original n-value navigation structure model run (N-VAL NS TEST).

At stages above affective navigation structures, the no structure model runs bracket the other stage model results. However, in the range of navigation structure stages (shaded area), the 25% increase in channel and navigation structure Manning's n-values (CH +25% DIKE +25%) result in greater stages than that of the no structure (NO STRUC CH +25%) model run. This is because the "CH +25% DIKE +25%" model run results in the highest possible combination of Manning's n-values tested for the affective navigation structures stages.

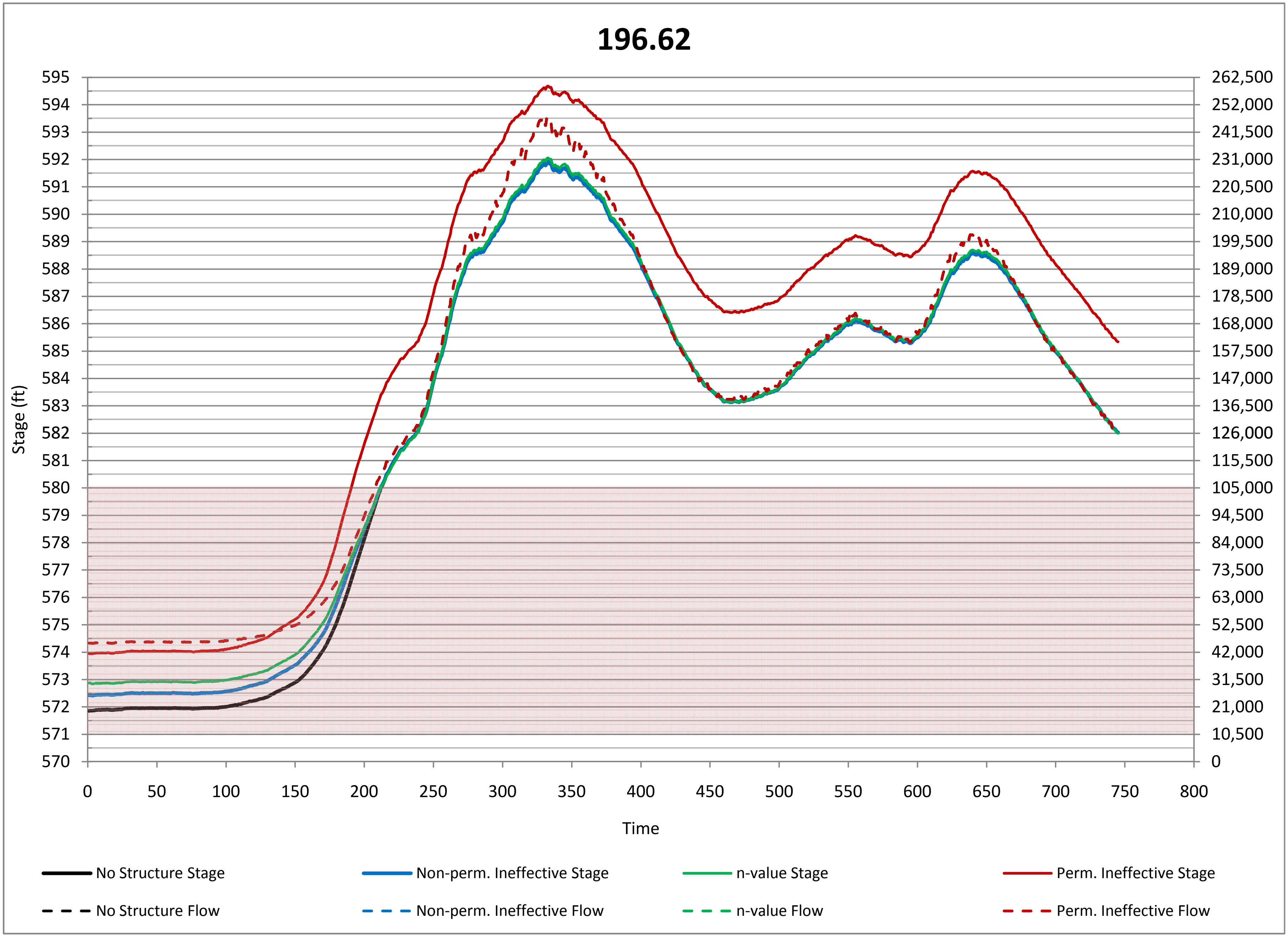
These stage results suggest that it may be possible to adjust the navigation structure Manning's n-values to better match low stages while the main channel Manning's n-values may be adjusted to better match at higher stages. Over the mid range of stages, it may be useful to allow some n-value interpolation between the top of the navigation structure and the elevation where the channel Manning's n-value becomes totally effective again.

The sensitivity analysis related only to navigation structure Manning's n-values showed little difference in flow between the original navigation structure Manning's n-values and the +/- 25% difference in the navigation structure n-values. This additional sensitivity analysis shows that change to the channel n-values does somewhat significantly affect flow timing and to a lesser extent, peak flows. The same upstream flow boundary condition was used for all model runs, therefore, flow results at Boonville do not show any difference in flow. However, all other locations plotted show some difference in flow timing and peak. Since change to the channel n-value affects timing, the general results from the flow plots are summarized in Table 7.

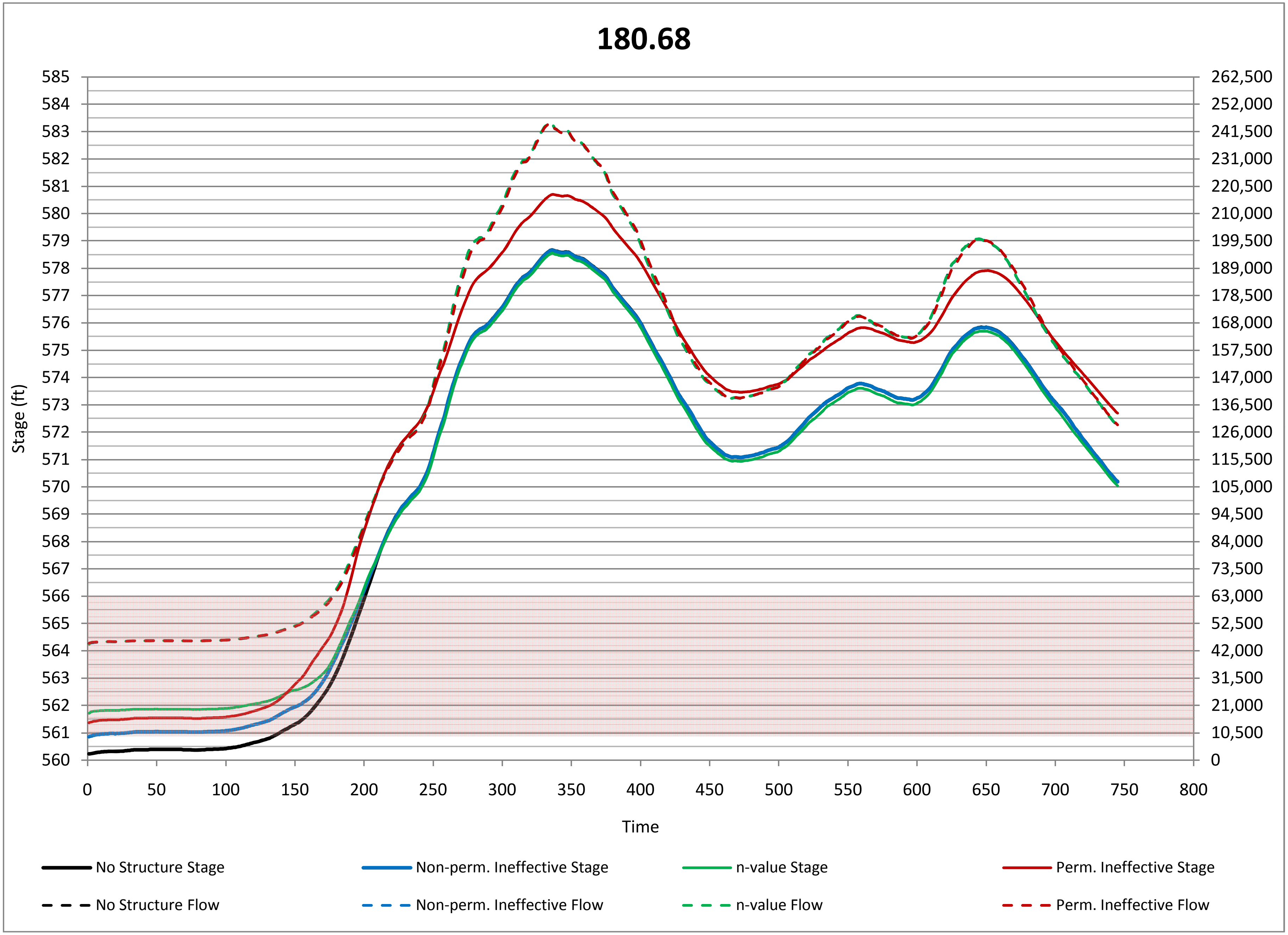
Table 7. Peak Flow and Time for Additional Sensitivity Model Runs

Location	Modeling method	Peak flow (cfs)	Peak flow time
160.86	CH -25%	244,929	15Mar2010 0000
	N-val	241,763	15Mar2010 0400
	CH +25%	240,857	15Mar2010 1300
143.86	CH -25%	244,379	15Mar2010 0200
Jefferson City	N-val	241,403	15Mar2010 1400
	CH +25%	240,315	15Mar2010 1600
133.12	CH -25%	244,760	15Mar2010 0300
	N-val	241,801	15Mar2010 1400
	CH +25%	240,210	15Mar2010 1700
103.19	CH -25%	257,680	16Mar2010 0500
	N-val	259,578	16Mar2010 1100
	CH +25%	256,355	16Mar2010 1500

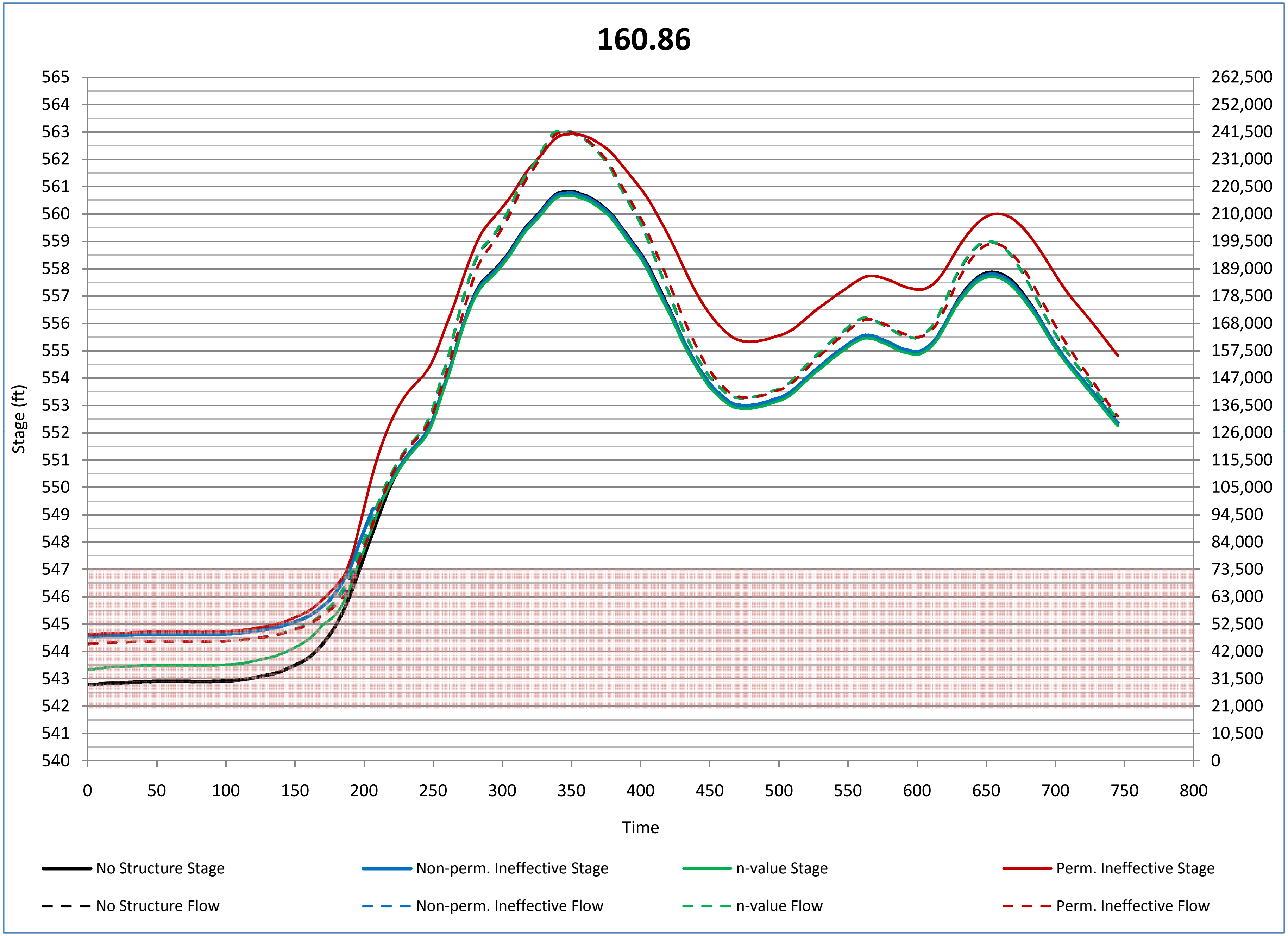
Quantitative Comparison of Navigation Structure Modeling Approaches



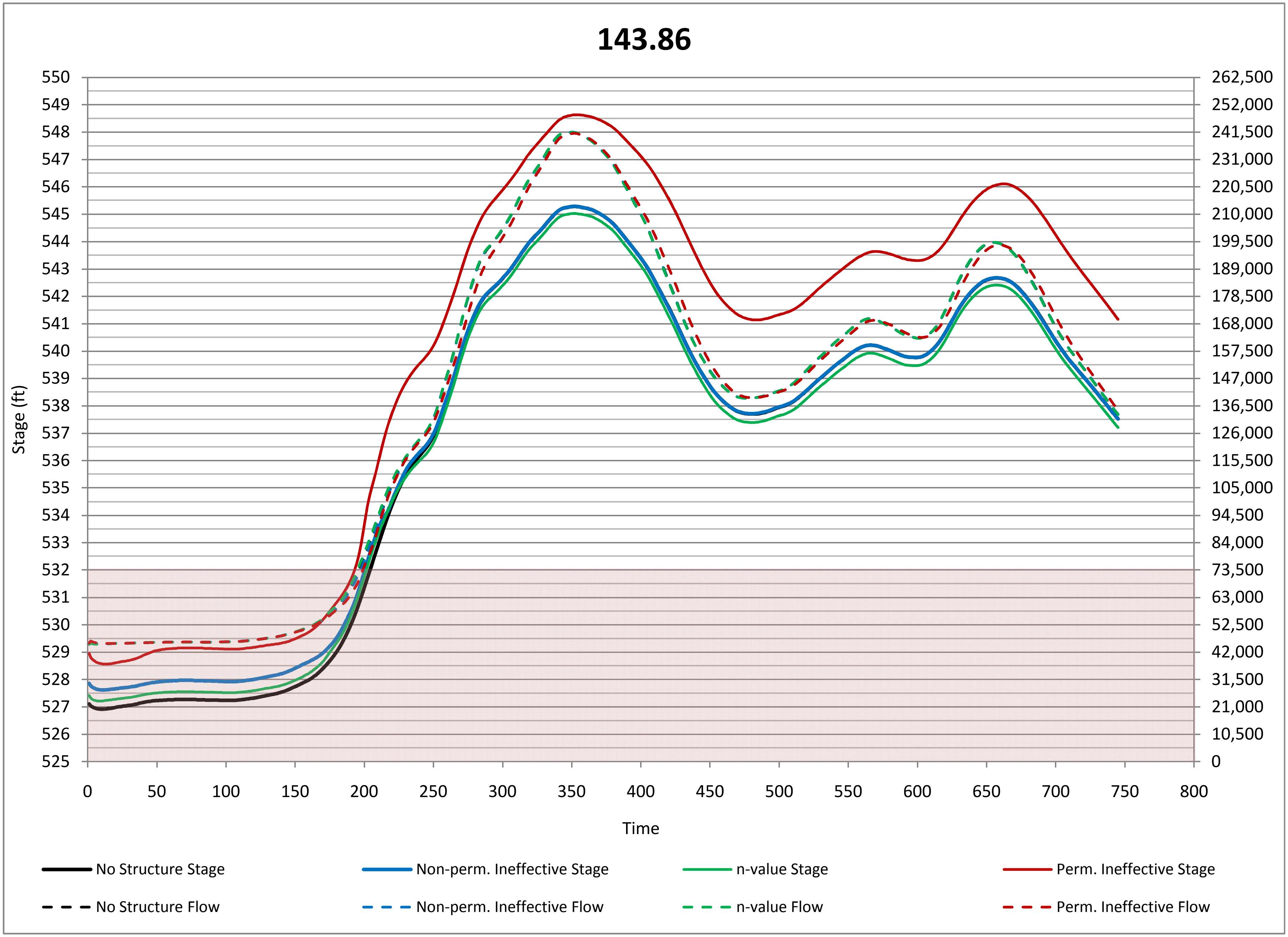
Quantitative Comparison - Nav Struc Methods



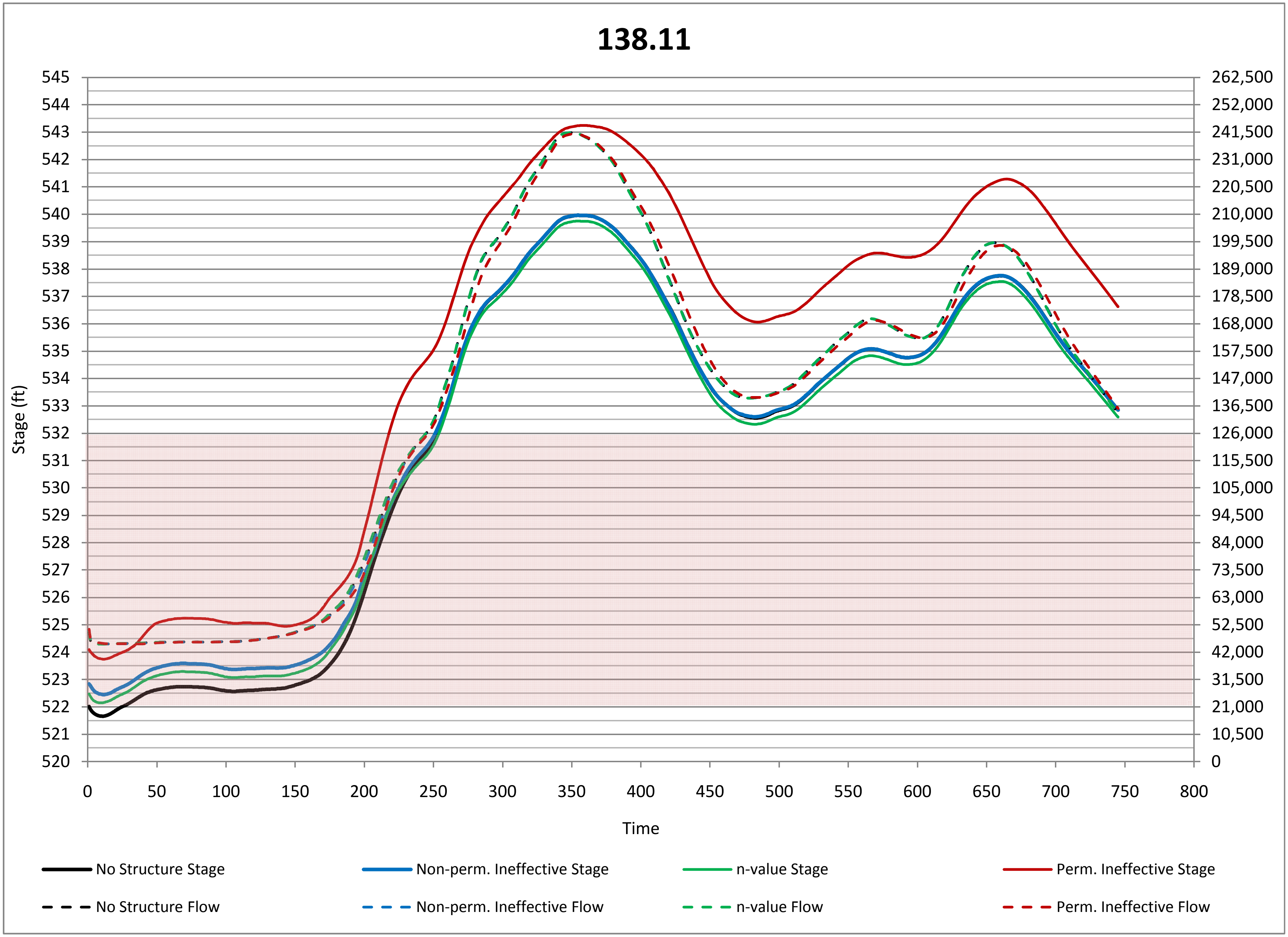
Quantitative Comparison - Nav Struc Methods



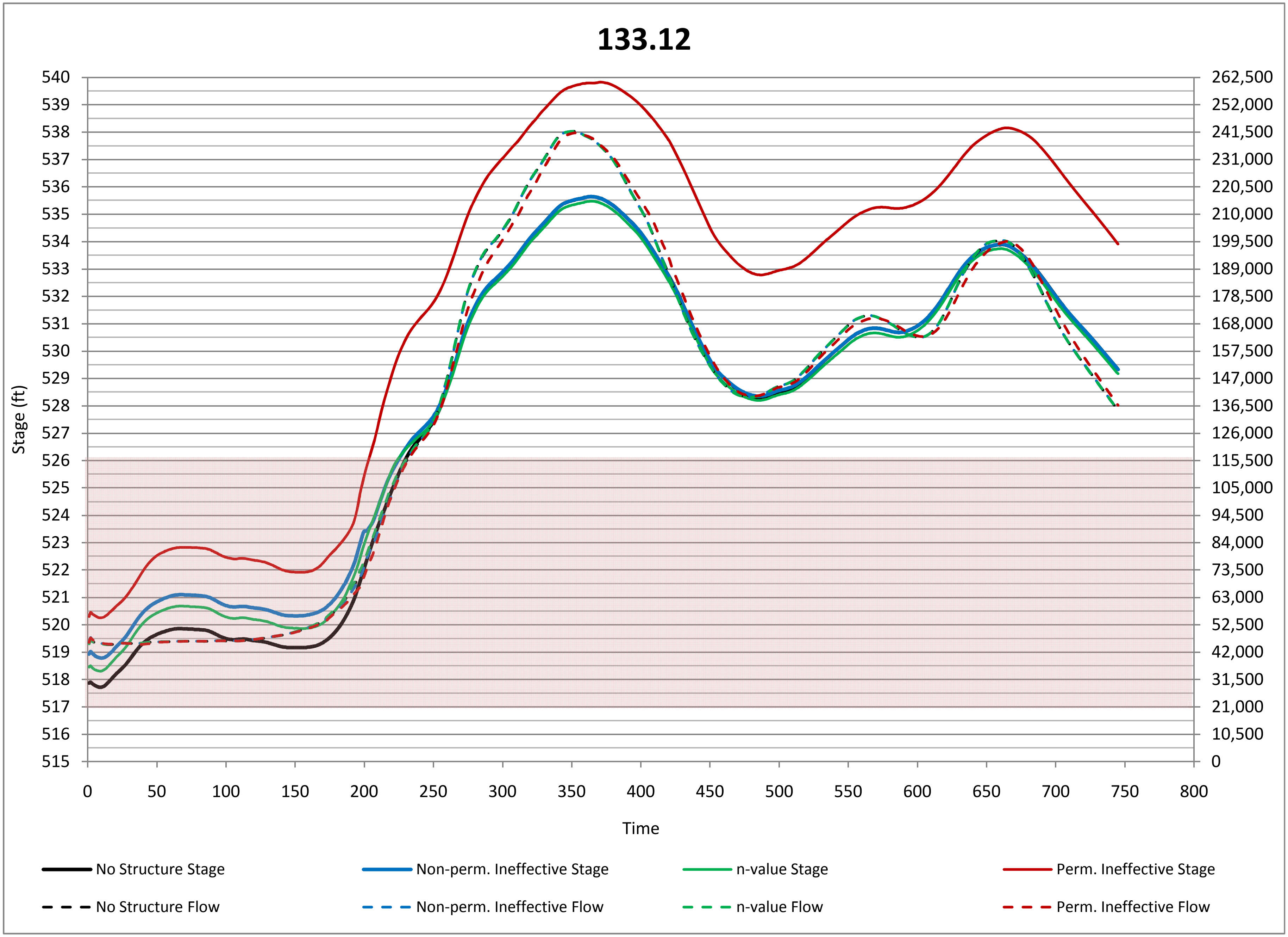
Quantitative Comparison - Nav Struc Methods



Quantitative Comparison - Nav Struc Methods



Quantitative Comparison - Nav Struc Methods



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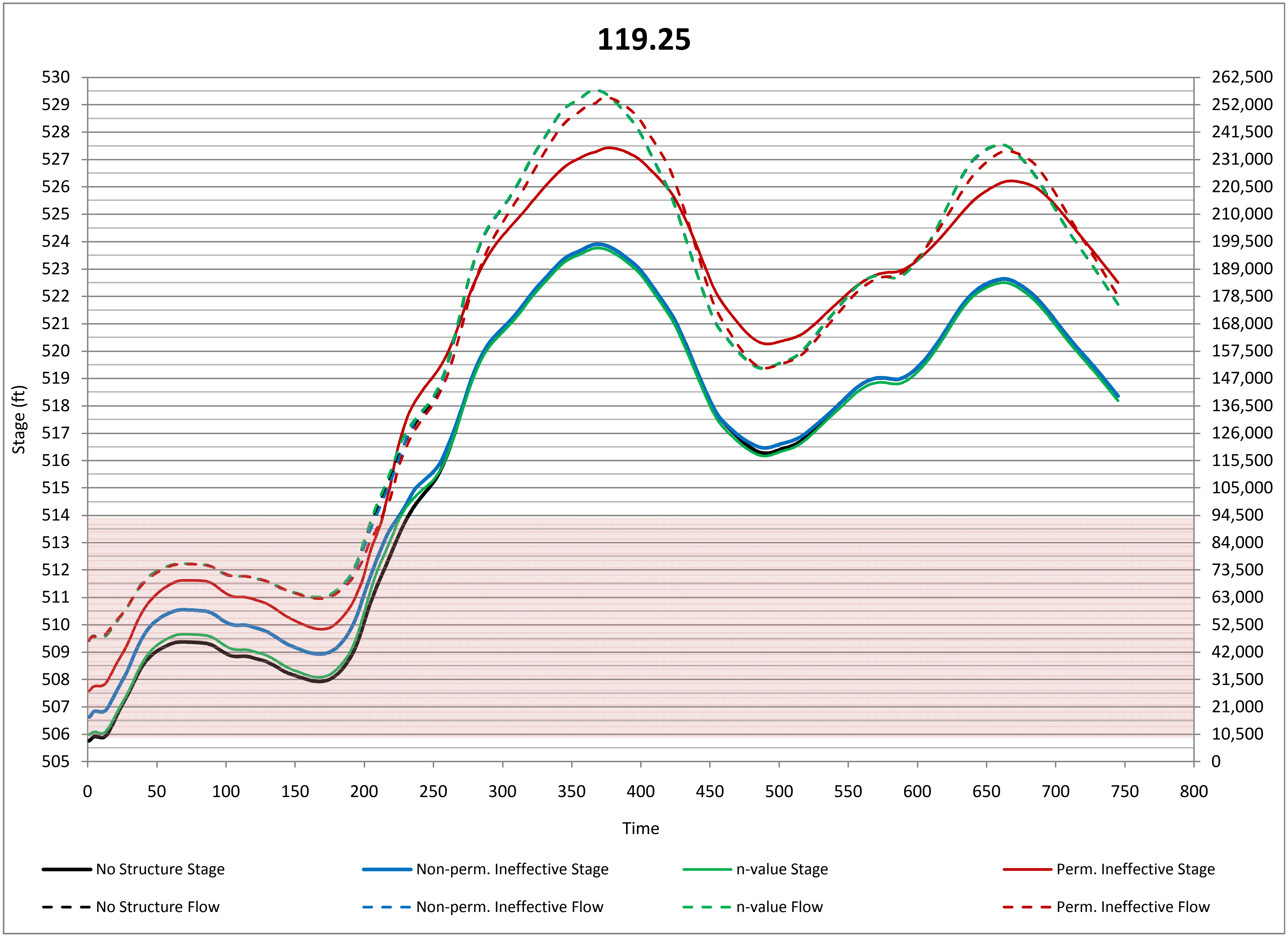
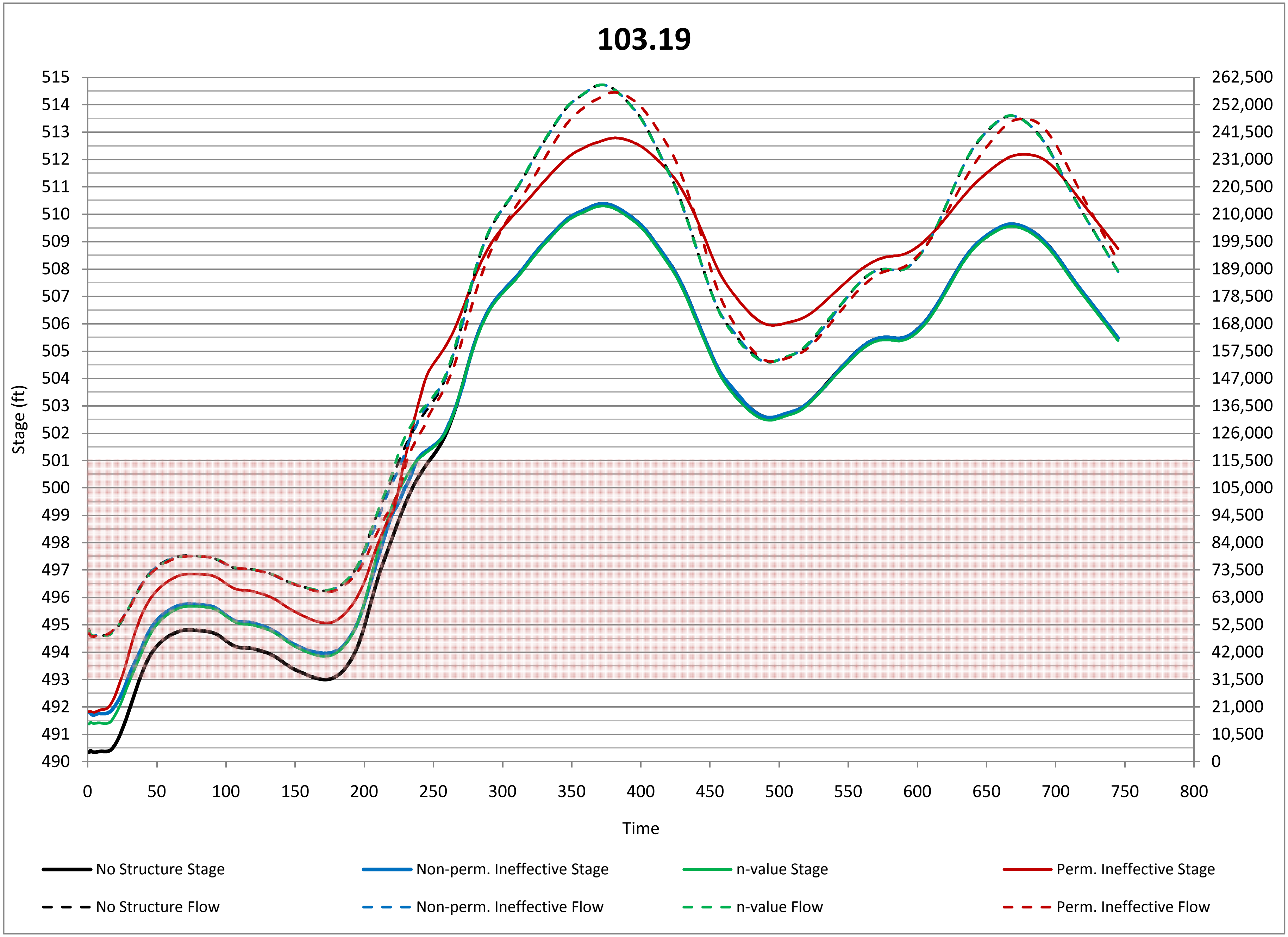


Quantitative Comparison - Nav Struc Methods



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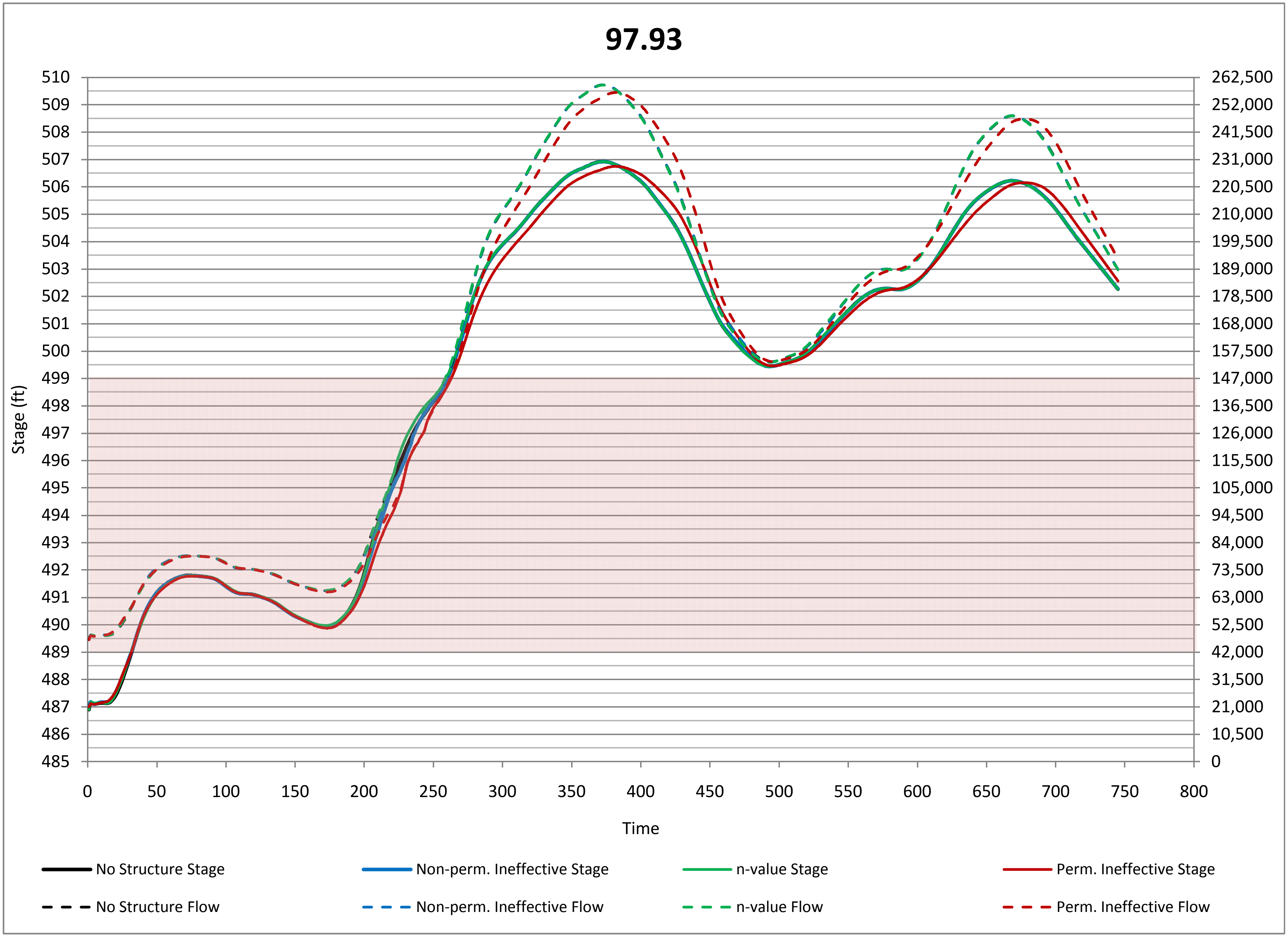
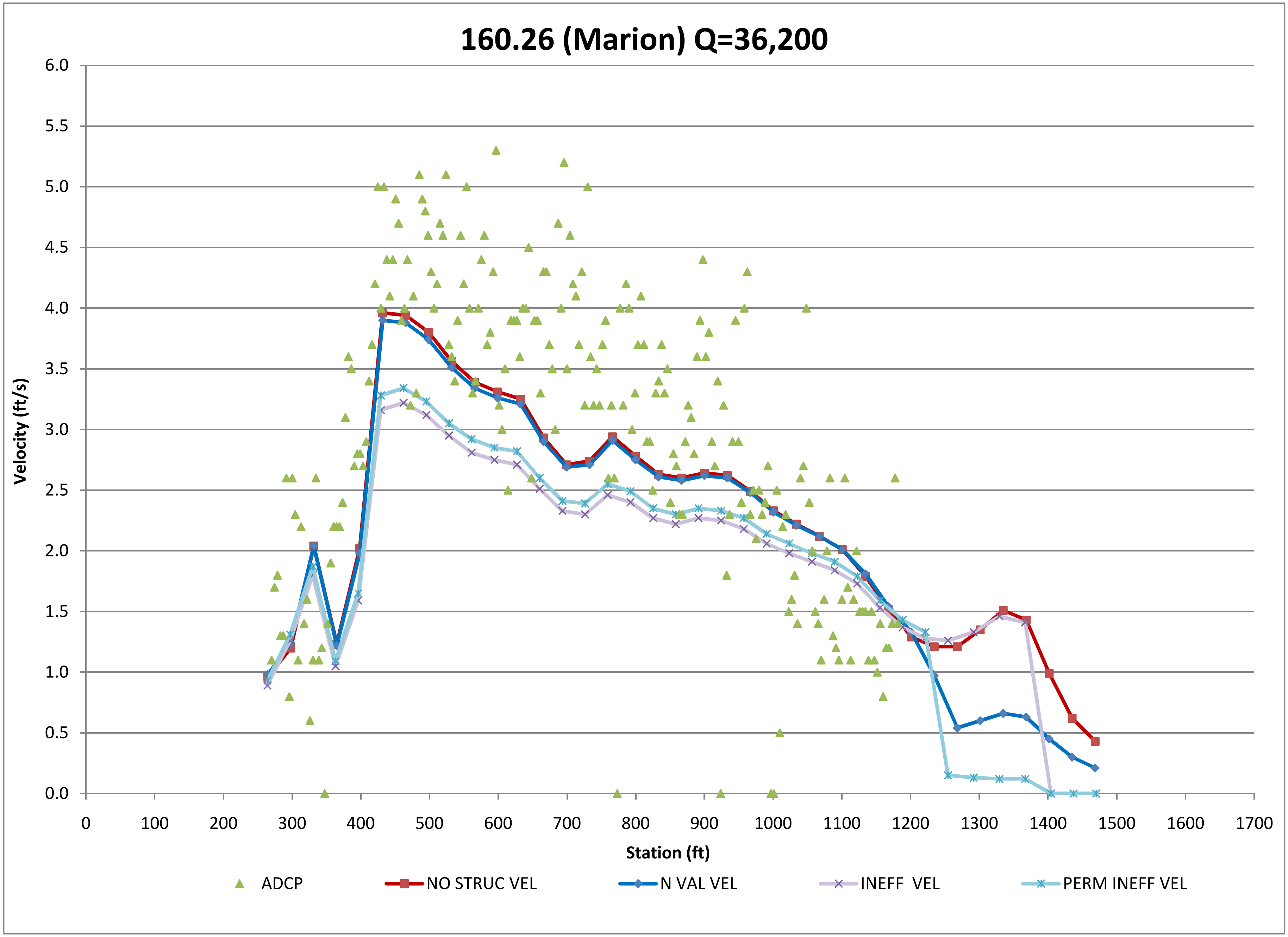
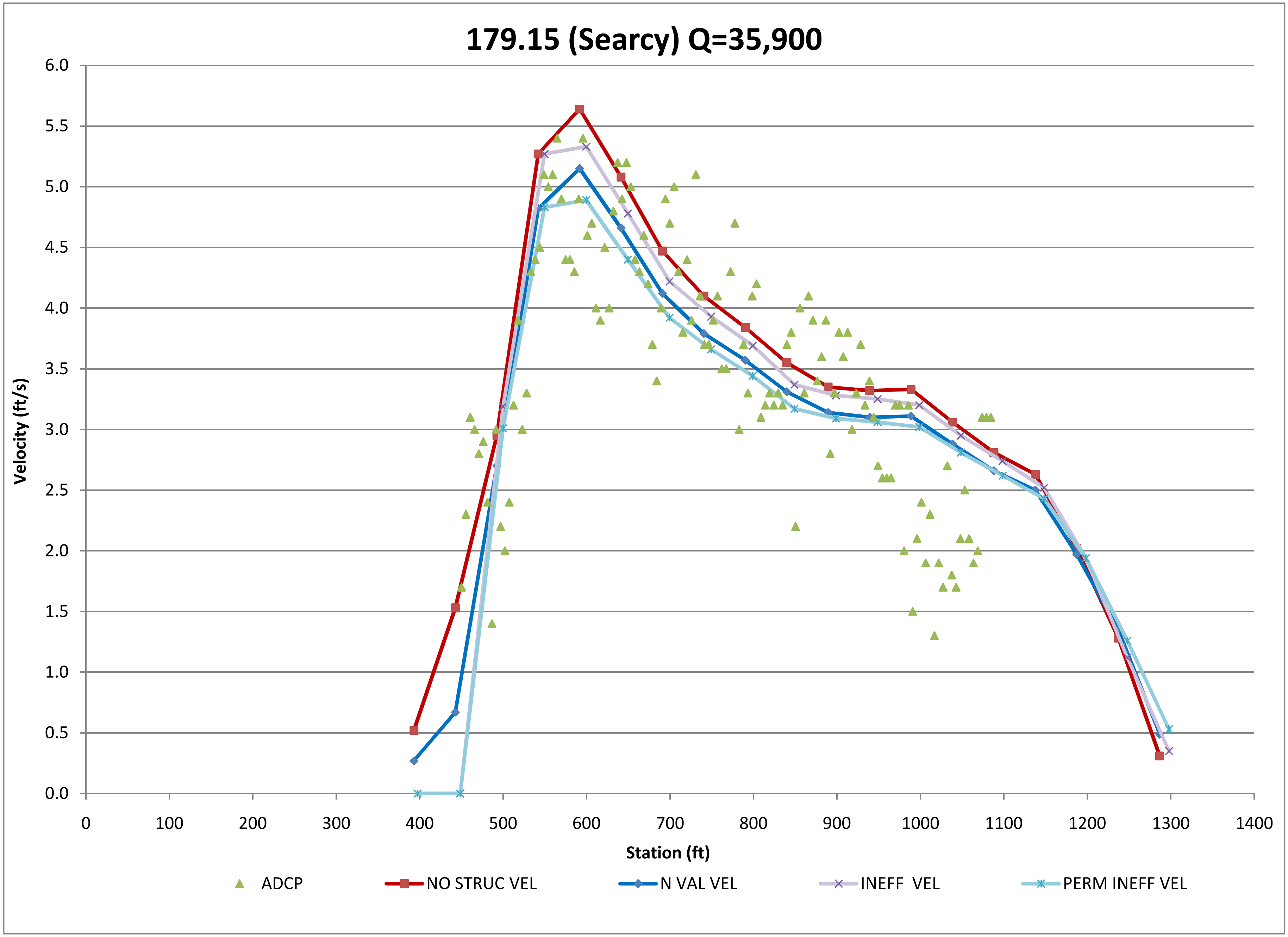


Quantitative Comparison - Nav Struc Methods

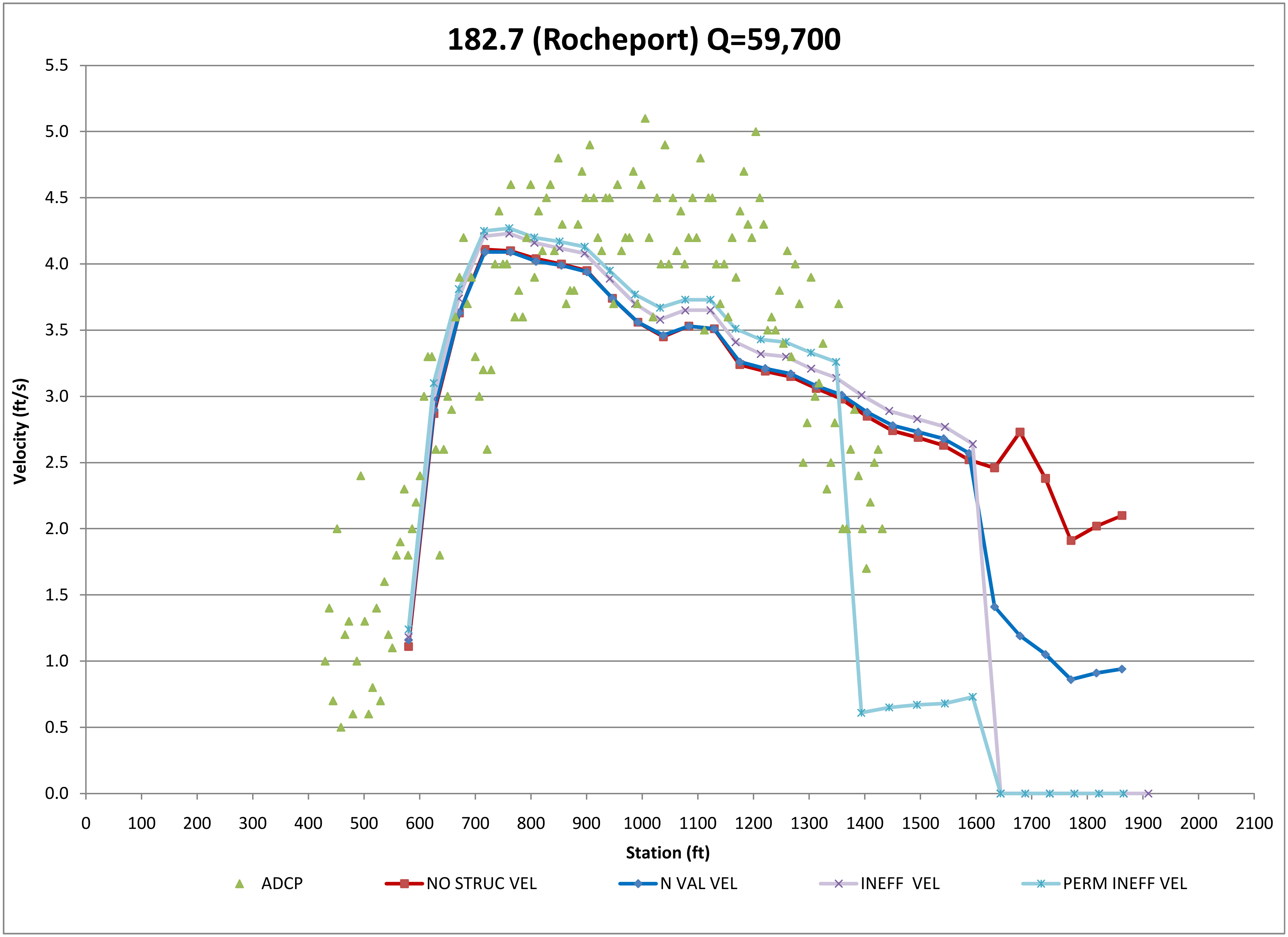
ADCP Data and Velocity Comparison

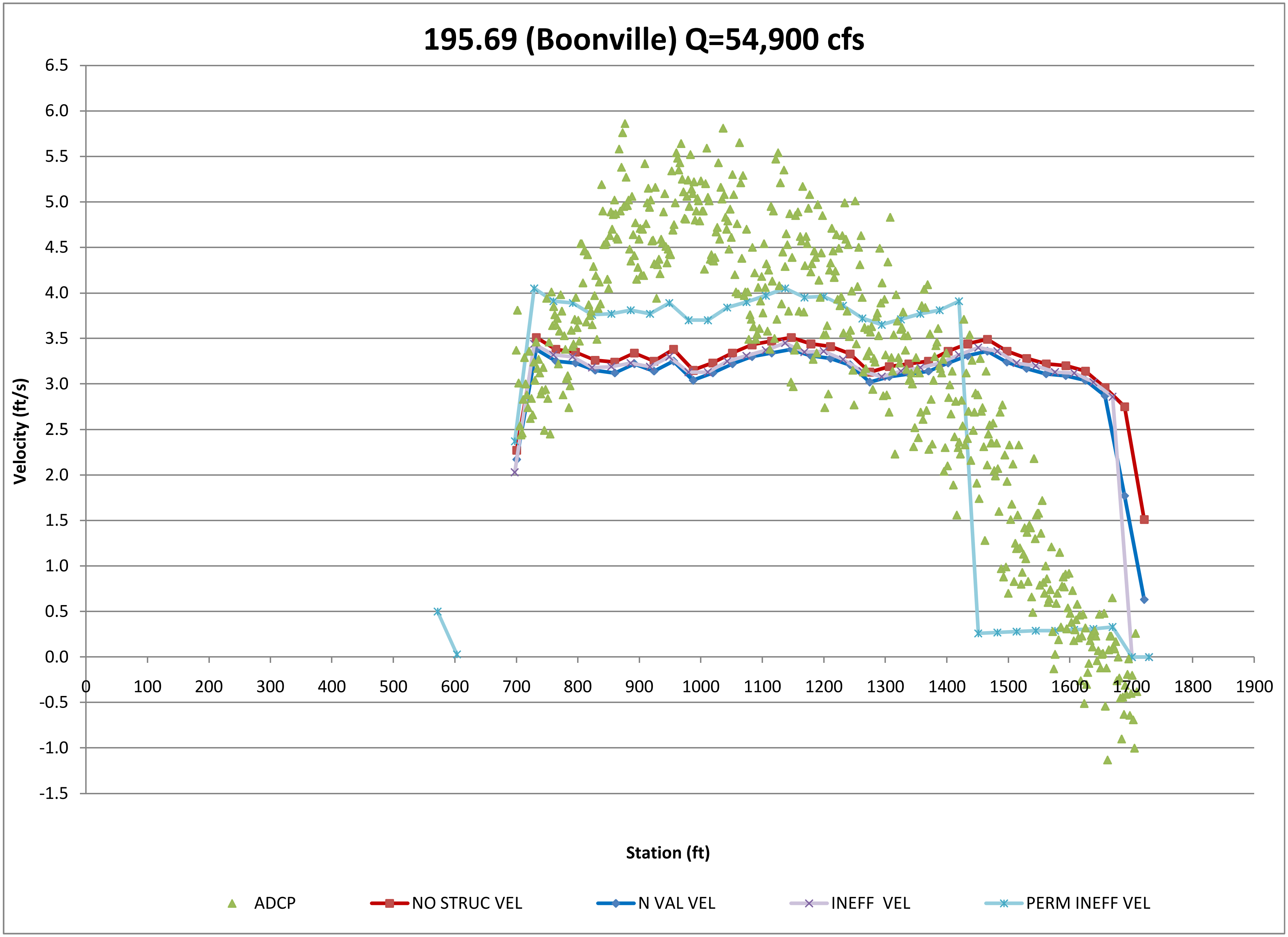


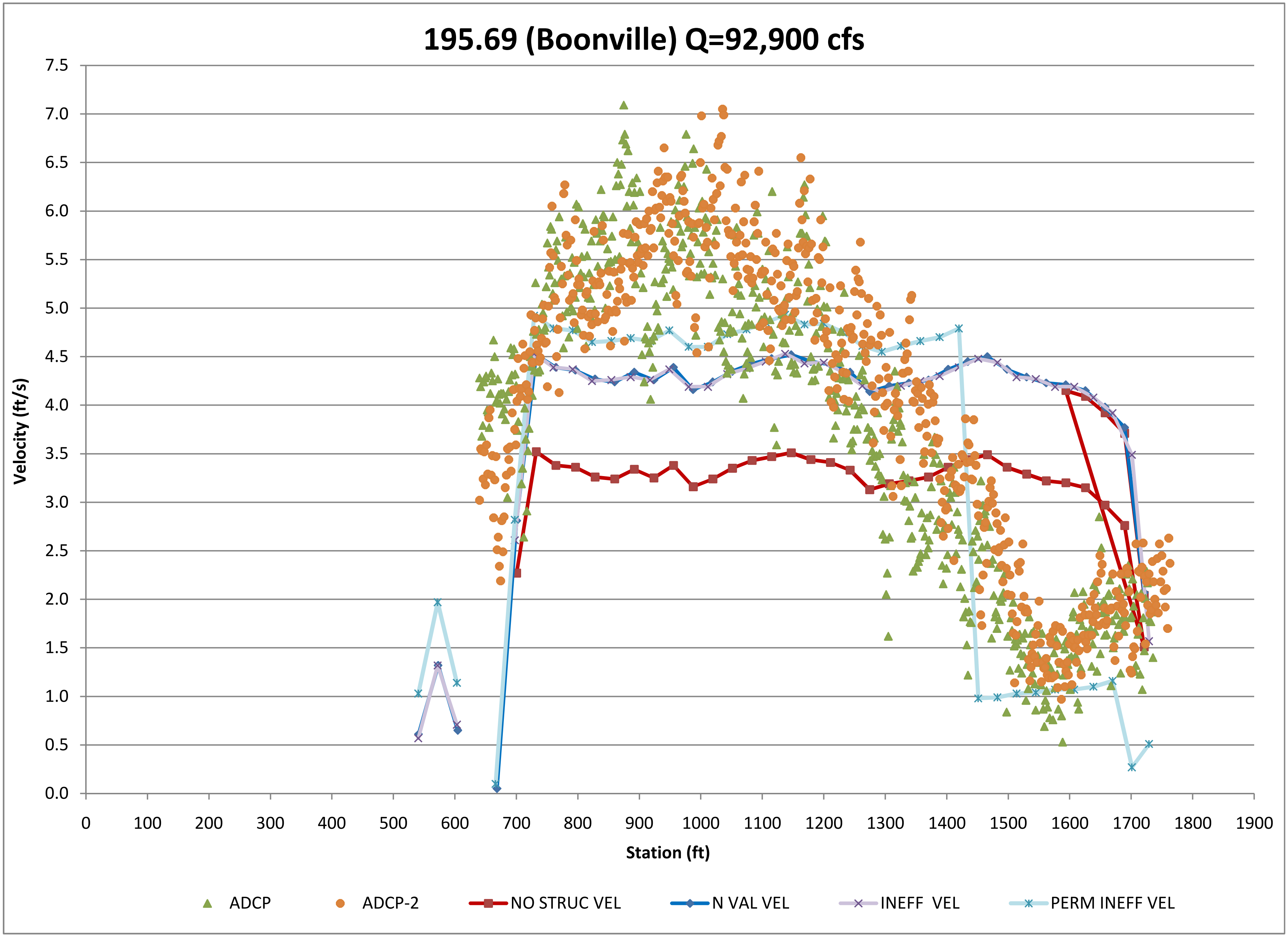
160.26 (Marion) Q=36,200
Velocity (ft/s)
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
Station (ft)
ADCP
NO STRUC VEL
N VAL VEL
INEFF VEL
PERM INEFF VEL



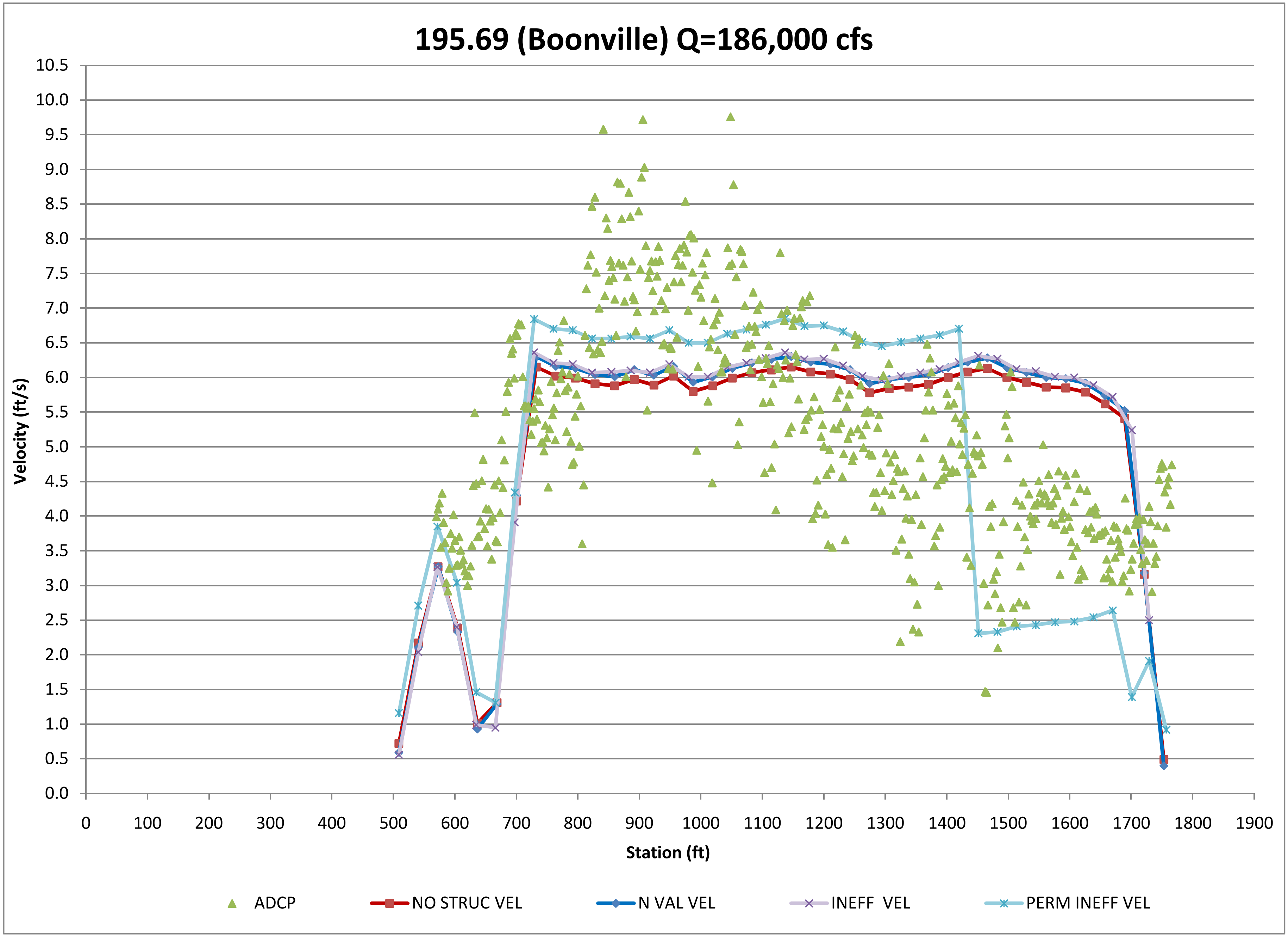
179.15 (Searcy) Q=35,900
Velocity (ft/s)
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
Station (ft)
ADCP
NO STRUC VEL
N VAL VEL
INEFF VEL
PERM INEFF VEL


182.7 (Rocheport) Q=59,700
Velocity (ft/s)
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
1900
2000
2100
Station (ft)
ADCP
NO STRUC VEL
N VAL VEL
INEFF VEL
PERM INEFF VEL


195.69 (Boonville) Q=54,900 cfs
Velocity (ft/s)
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
-0.5
-1.0
-1.5
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
1900
Station (ft)
ADCP
NO STRUC VEL
N VAL VEL
INEFF VEL
PERM INEFF VEL

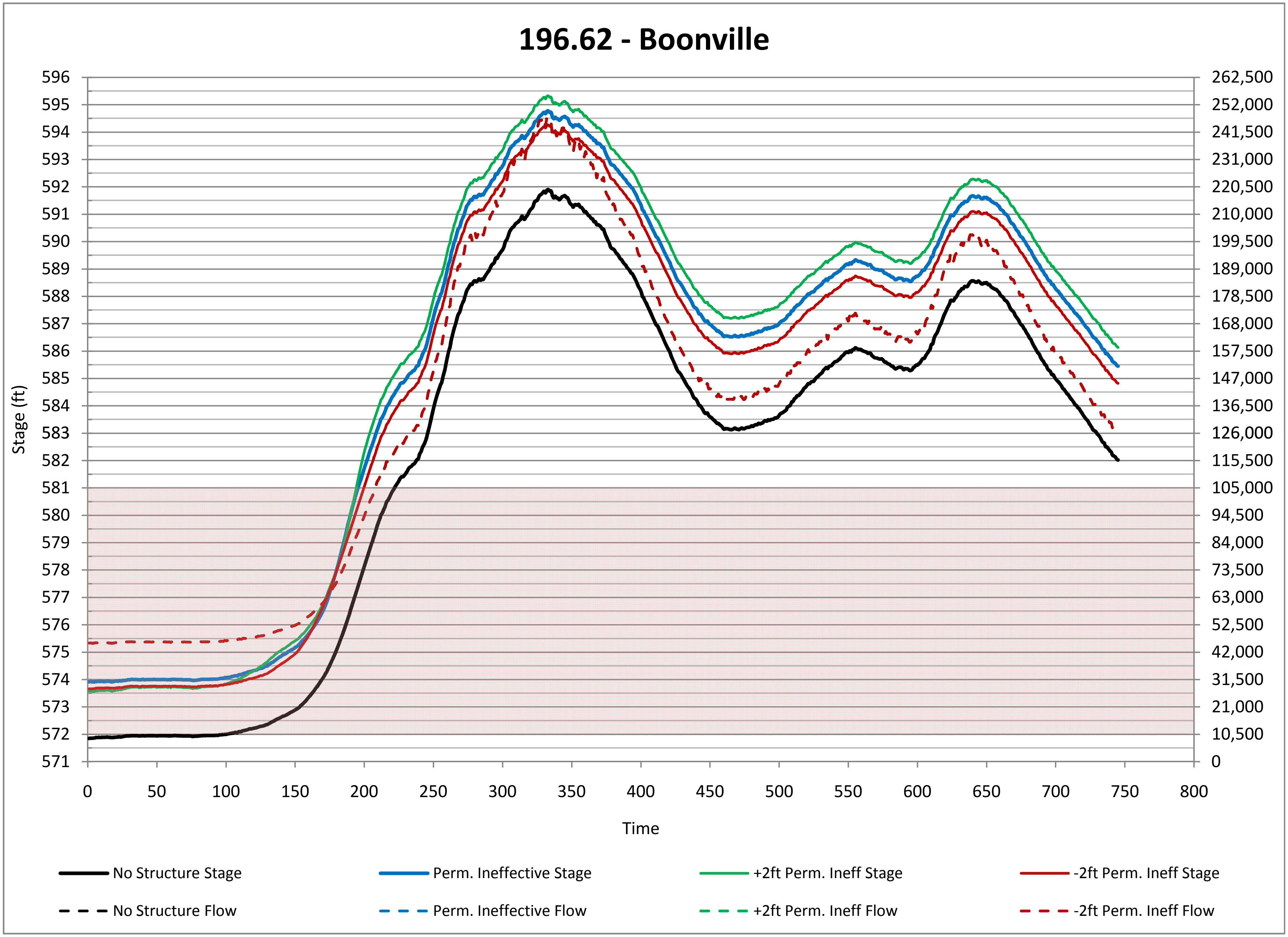


195.69 (Boonville) Q=92,900 cfs
Velocity (ft/s)
7.5
7.0
6.5
6.0
5.5
5.0
4.5
4.0
3.5
3.0
2.5
2.0
1.5
1.0
0.5
0.0
0
100
200
300
400
500
600
700
800
900
1000
1100
1200
1300
1400
1500
1600
1700
1800
1900
Station (ft)
ADCP
ADCP-2
NO STRUC VEL
N VAL VEL
INEFF VEL
PERM INEFF VEL

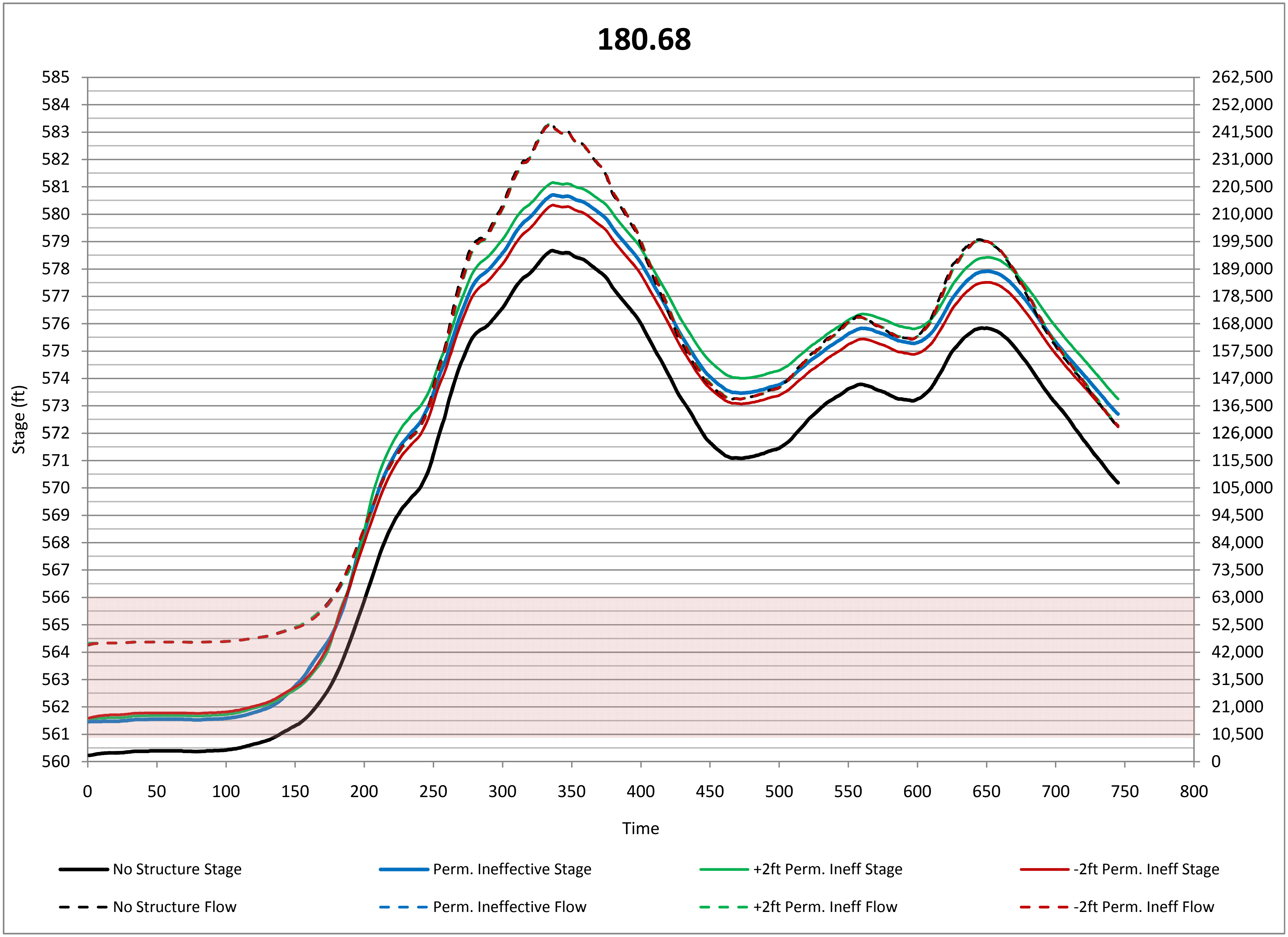

195.69 (Boonville) Q=186,000 cfs
Velocity (ft/s)
Station (ft)
ADCP
NO STRUC VEL
N VAL VEL
INEFF VEL
PERM INEFF VEL

Quantitative Sensitivity Analyses of Navigation Structures

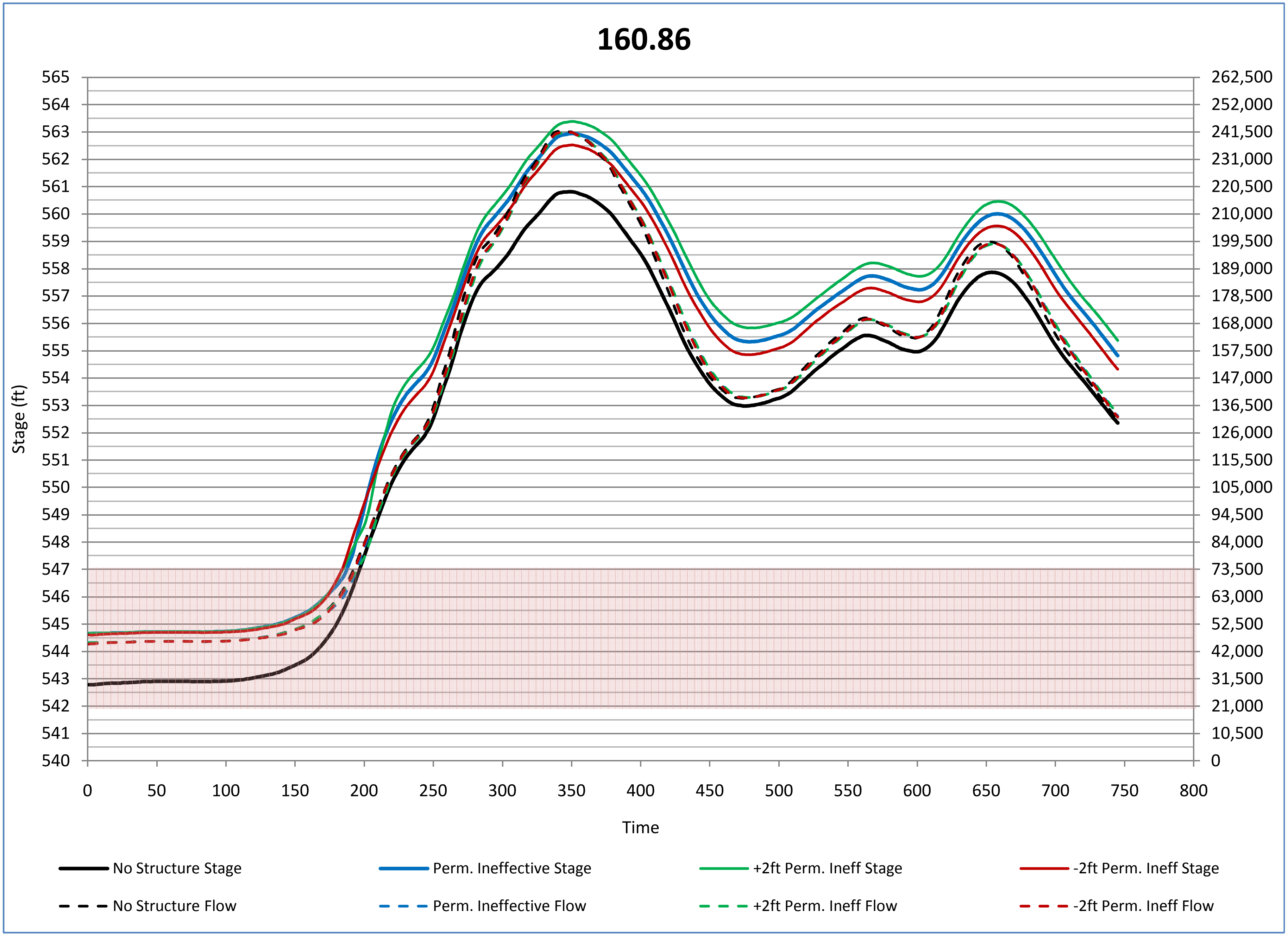
Permanent Ineffective Flow Areas



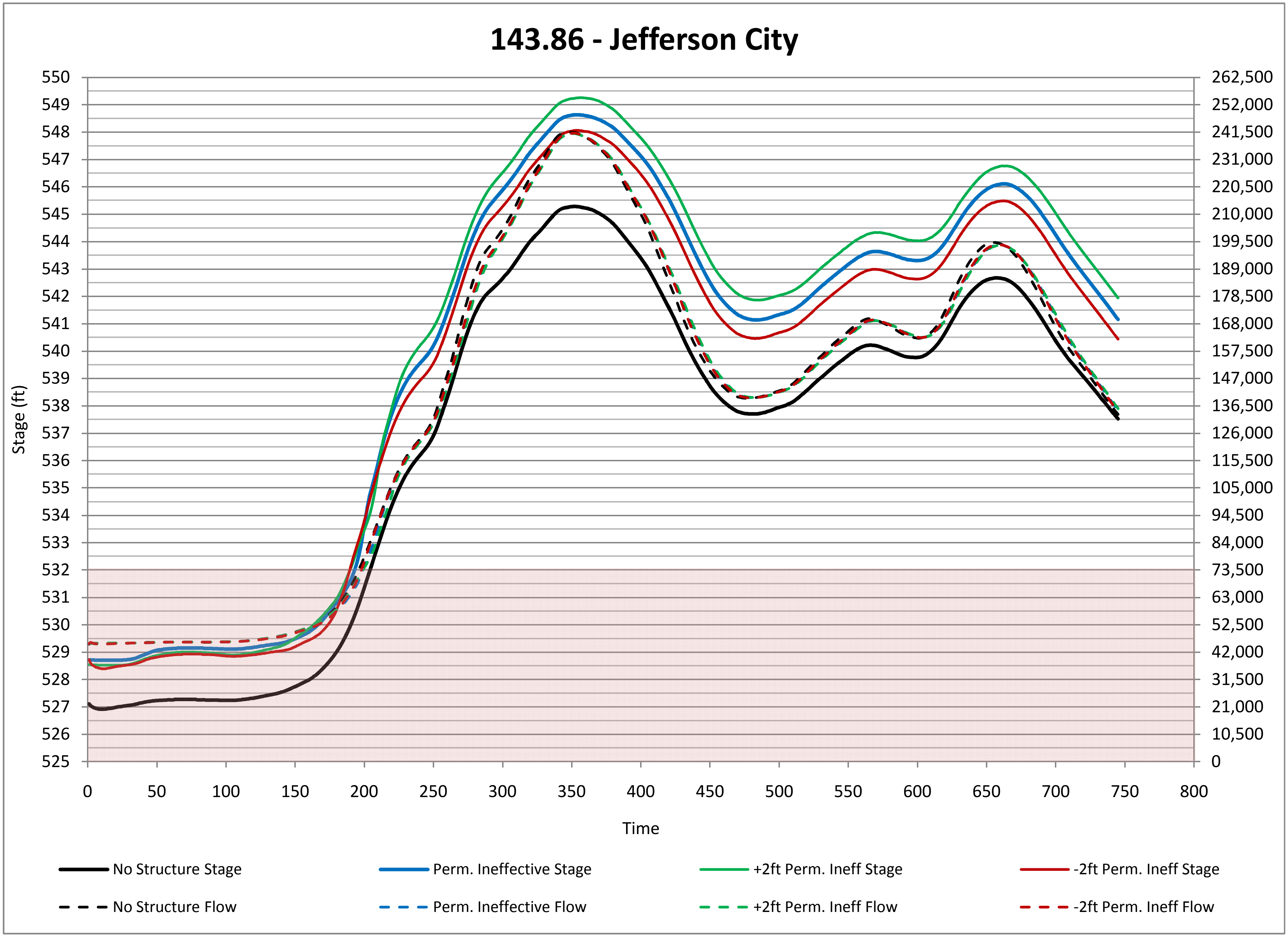
Quantitative Sensitivity - Ineffective Flow Areas



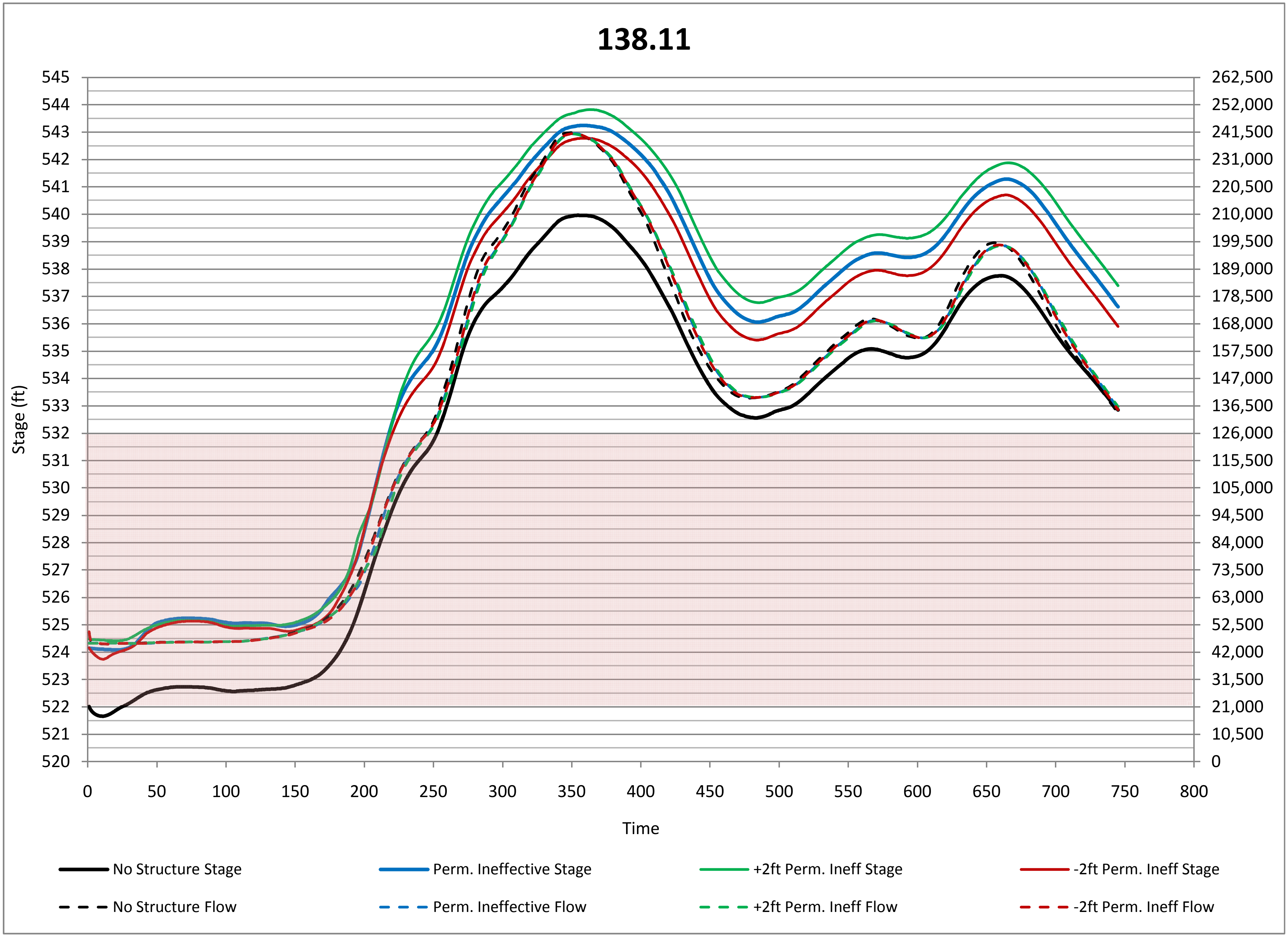
Quantitative Sensitivity - Ineffective Flow Areas



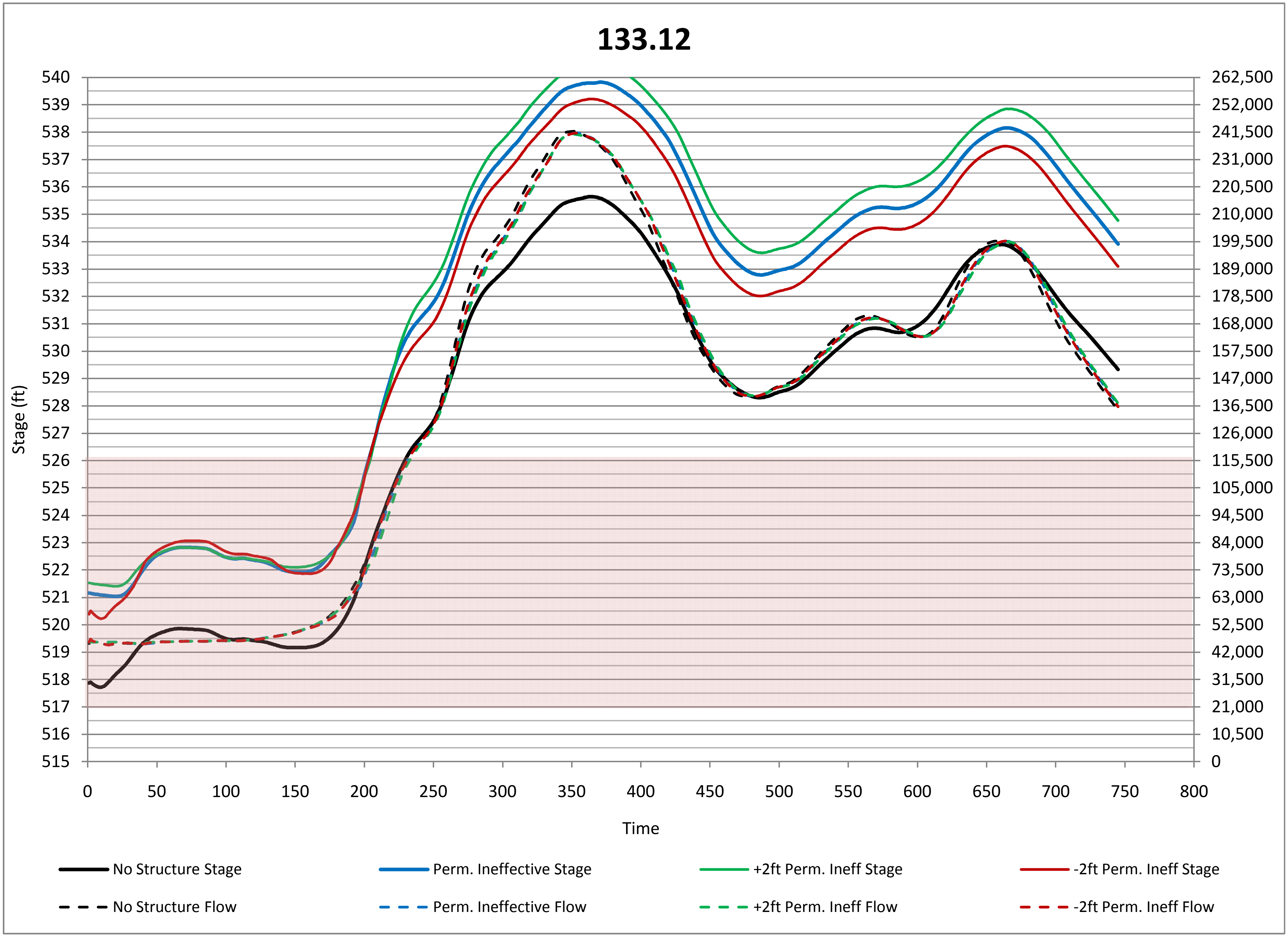
Quantitative Sensitivity - Ineffective Flow Areas



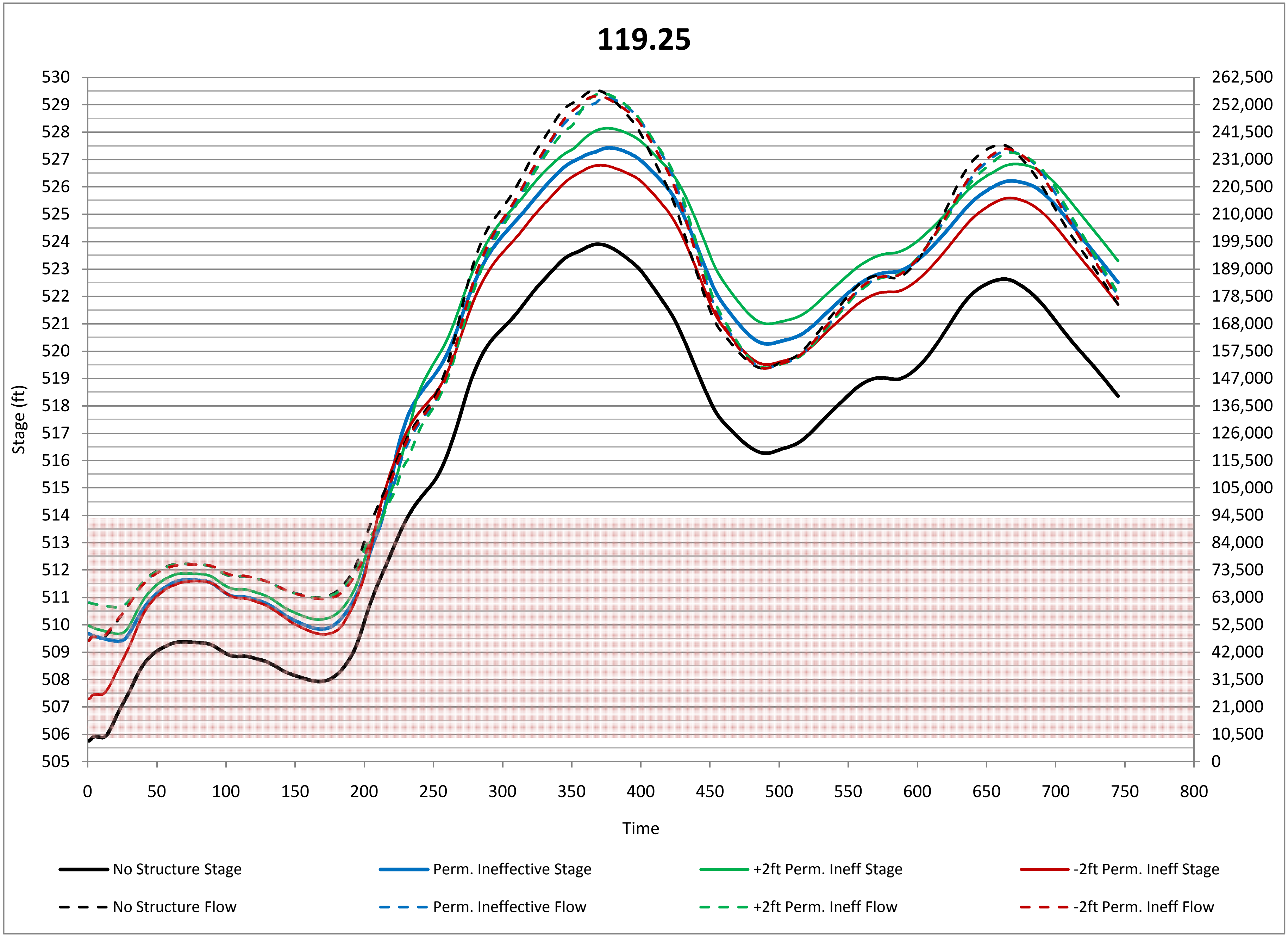
Quantitative Sensitivity - Ineffective Flow Areas



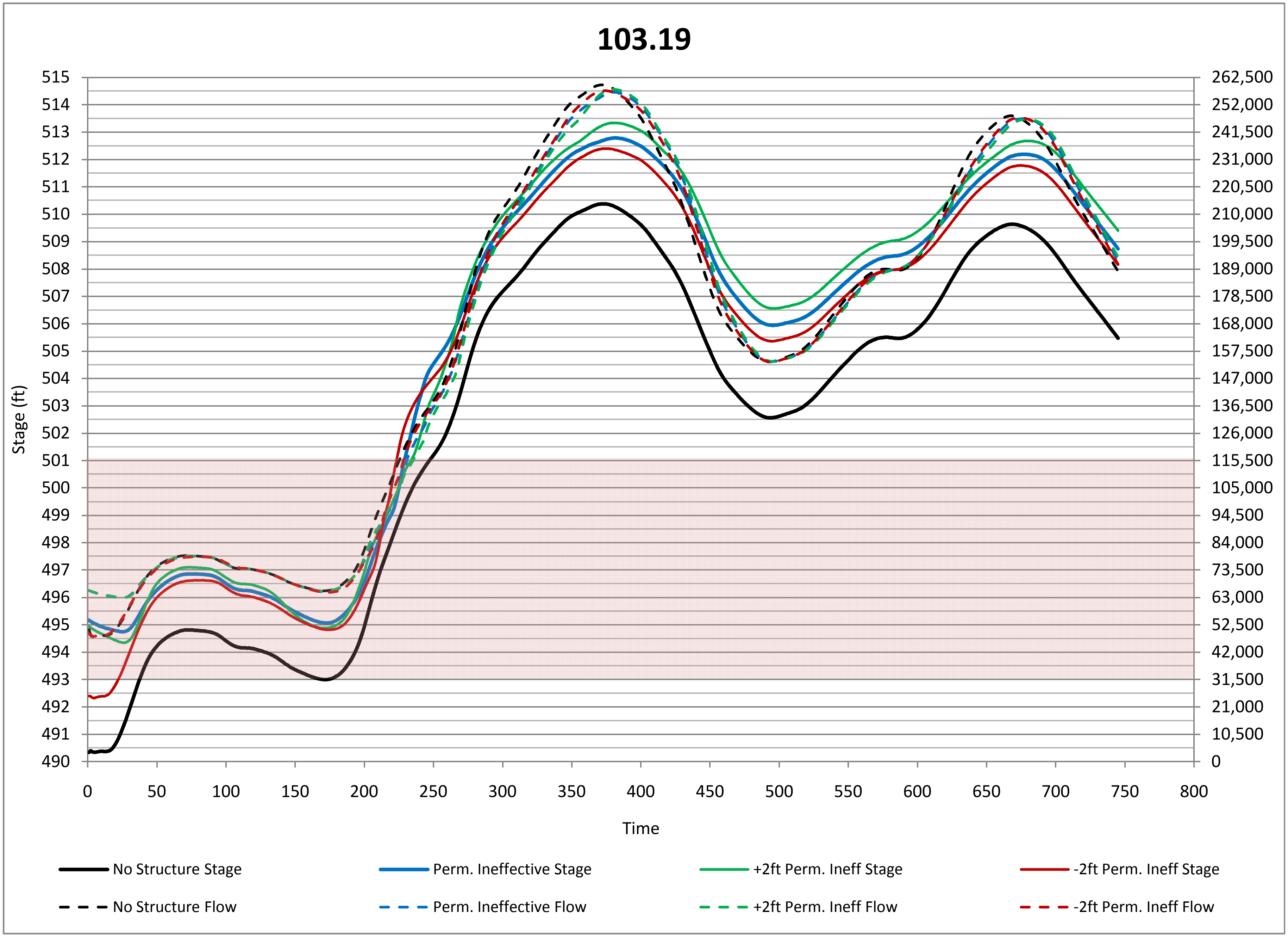
Quantitative Sensitivity - Ineffective Flow Areas



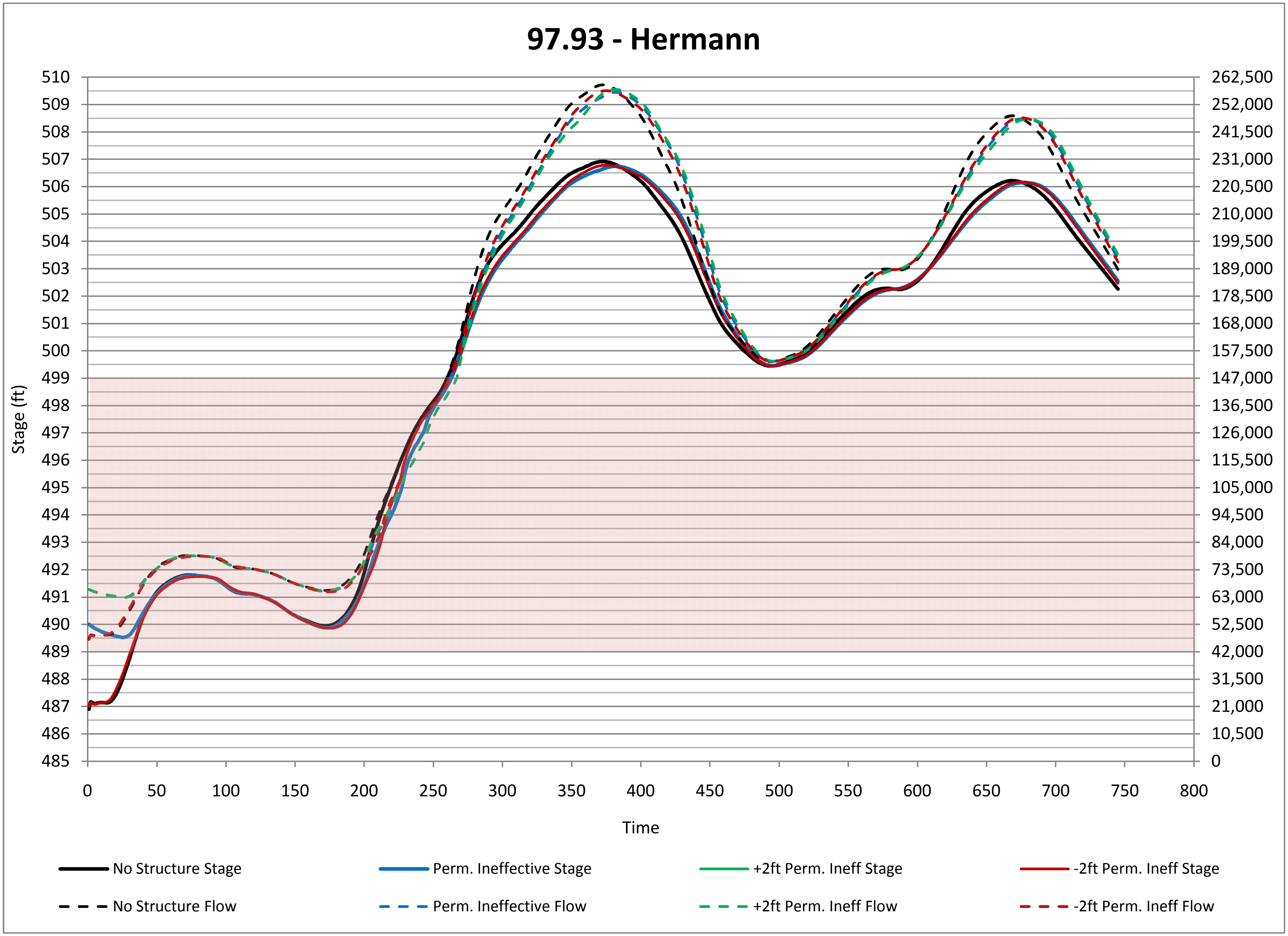
Quantitative Sensitivity - Ineffective Flow Areas



Quantitative Sensitivity - Ineffective Flow Areas



Quantitative Sensitivity - Ineffective Flow Areas



Quantitative Sensitivity - Ineffective Flow Areas

Attachment 5

Mississippi Missouri Crossover
Model Documentation

Mississippi River/Missouri River Confluence RAS Model

1.0 Background

This model was developed in conjunction with the Hydraulic Engineers at the Kansas City District. They are producing a HEC-RAS model of the Missouri River and asked the St. Louis District to assist them with the modeling of the Confluence of the Missouri and Mississippi Rivers. Engineers in St. Louis developed the portion of the model on the Mississippi River, starting just downstream of Lock and Dam 25 (river mile 241.4) and continuing south to river mile 168.8, approximately 11 miles downstream of St. Louis, Missouri. The stretch of the Missouri River in the model, between river miles 100 and 0, was provided by the Kansas City District. The St. Louis District also modeled the Crossover – the area between the Missouri and Mississippi Rivers that floods during high flow events. The model will be turned over to the Kansas City District for incorporation in to their final model.

2.0 Model Data

2.1 Cross Section Data

2.1a Mississippi River

Cross section layout was taken from a previous HEC-RAS model developed for floodway calculations on the Mississippi River. Overbank portions of the cross sections were taken from the SAST dataset that was collected after the flood of 1993. The SAST data has the horizontal accuracy required to generate 4 foot contours. Channel data was taken from a bathymetric survey conducted by the St. Louis District in the fall of 2010. The data was merged together in HEC-RAS using the Graphical Cross Section Editor tool.

2.1b Missouri River

Cross section layout was taken from a model provided by the Kansas City District. Overbank portions of the cross sections were taken from the SAST data. Channel data was taken from a bathymetric survey conducted by the Kansas City District in 2009. The data was merged together in HEC-RAS using the Graphical Cross Section Editor tool.

2.1c Crossover

The crossover reach was developed by the St. Louis District to model the proper connection of flows between the Missouri and Mississippi Rivers at high flows. The layout was developed off of a UNET model that was calibrated to the historic 1993 flood event. All data used for the cross sections comes from the SAST dataset. Some data surrounding some of the bridges comes from the UNET model and was incorporated into the HEC-RAS model sections.

2.2 Hydrograph Inputs

Inflows were requires for three major rivers in this model. The Missouri River flows were taken from the gage at Herman, MO (river mile 98). The Upper Mississippi River flows were taken from the gage at the Lock and Dam 25 tailwater (river mile 241.2). The Illinois River flows were taken from the gage at Valley City (river mile 61.3). Since the Illinois River was not included in the modeling, the flows were added to the model on the Mississippi River at river mile 220.02 near the mouth of the Illinois River.

2.3 Downstream Boundary

The downstream boundary condition used on the Mississippi River was normal depth, set to a slope of 0.0001 ft/ft, or roughly 0.5 ft/mile. The model was extended roughly 11 miles downstream of the St. Louis gage location to prevent any backwater issues from interfering with the calculations at the gage location.

2.4 Navigation Dams Operation

Mel Price Locks and Dam is included in the model. It is located on the Mississippi River at river mile 200.6, just a few miles upstream from the confluence of the Missouri River. In order to replicate the operation of the dam during pooled conditions, the Navigation Dams operation was used. The pool is held upstream of the dam using a hinge point operation schedule. This means that the water surface elevation at the dam is lowered as flows increase to keep the pool at an upstream location within regulated limits. The pool "hinges" off of a pivot point somewhere upstream until the flows increase enough to warrant the gates being pulled out of the water and allowing a natural flow to occur, otherwise known as "open river". The hinge point for Mel Price is located at Grafton, IL. The pool limits at the Mel Price pool gage are 412.5 and 419.0 ft (NGVD 29). The hinge point limits at the Grafton gage are 418.0 and 420.0 ft (NGVD 29). When holding a pool, the elevations must always be within these regulated bands.

Within the HEC-RAS model, the "Hinge Point and Min Pool Operations" setting was chosen. The hydraulic modelers felt that after some testing this option produced the most consistent results when compared to the historic data. Settings for the "Hinge Pool" and "Min Pool Control" tabs were set after running several iterations of the model and consulting with HEC staff. When comparing historic data, the model follows the trends but has a hard time replicating the results. Many factors play into the daily operations of the Dam. Time of year, rising or falling pool, environmental considerations, and even water control personnel can influence how the dam is operated. These factors cannot be taken into account by the model. The hydraulic modelers feel that the results from the model represent an average condition at the dam, within the operating limits set by the Water Control Manual.

2.5 Model Coordinates

The horizontal datum used for the RAS model is UTM 15 (feet) and the vertical datum is NAVD 88. Vertical Datum Shifts were calculated using the CorpsCon software package when necessary. Details of the datum shift are found in the appendix.

3.0 Model Calibration

Hydrologic data, both stage and flow, was gathered for the past three years (2010-2012) for various gage locations within the boundaries of the model. The locations are as follows: Herman and St. Charles on the Missouri River: Valley City on the Illinois River: and Lock and Dam 25 Tailwater, Grafton, Mel Price Pool and Tailwater, Lock 27 Pool, and St. Louis on the Mississippi River. All of these locations were used to aid in the calibration efforts of the model. The primary adjustment was made using the "Flow Roughness Factors" option in the Unsteady Flow Analysis plan window. Numerous calibration sets were established on each of the river reaches in the model. Roughness factors were adjusted based on observed flowrates at the gages until the hydraulic modeler was satisfied that the output from the model closely matched the observed data.

Attachment 6

Tributary Model Review Documentation

HEC-RAS Review Checklist

Tributary: **Little Nemaha**
MoRiver RM: **527**

Model Data/Major Assumptions

	Item	
1	Horizontal Projection	NAD_1983_UTM_Zone_15N
2	Vertical Datum	NAVD88
3	Surface(s) used to cut cross sections	USGS NED 2009, 1/3 arc
4	Channel data - if estimated, what were the dimensions chosen (width, side slopes, invert, slope, etc.) and what data source did you use to guide the estimates	4/11/2013 - Modification to channel data. After comparing the d/s thalweg elevation of the Little Nemaha to MoRiver tailwater elevations at low flow it was apparent that the d/s thalweg was too high (elev 877.4)... which by odd coincident is actually equal to MoRiver CRP at this location (RM 527.7). Lowered the elevations of the bottom 7 cross sections, which corresponds to 2 miles up the tributary, which seemed reasonable. Estimated that the thalweg in the last cross section should be approx negative 7 CRP (870.4). ADJ: In order to be more in line with Chance's approach the invert was modified to be the gage datum + the lowest gage reading. In this case, the invert was adjusted from 890.15 to 893.006. The results compared better to the rating curve using the adjusted invert. For this modification the channel dimensions used were: bw-100 feet and ss-5. Channel slope used to make modifications was 0.00031. Boundary conditions were based on the Trib BC DSS file provided by the USACE (JHilger). Spreadsheet "XS approximation based on USGS data_20120926" was used to determine channel shape and additional details can be found in that spreadsheet, which utilizes data taken directly from the USGS web site.
5	Other sources of data used	RC from USGS
6	Other assumptions and notes	

HEC-RAS Review Checklist

Tributary: Little Nemaha
MoRiver RM: 527

Review #1: Cross Section Layout

Designer/Date: Lisa Stahr (CDM Smith)/ 17 Oct 2012
Reviewer/Date: Jean Hilger/ 30 Oct 2012
Backcheck/Date: Lisa Stahr/ 21 Nov 2012

	Task	Reviewer	Backcheck
1	Cross section spacing less than 1/2 mile (less than approx 2600 feet)?	Yes. A few that are slightly more, but negligible.	X
2	Cross sections perpendicular to channel flow and perpendicular to floodplain flow?	Yes. Tweak a few locations. See map.	X
3	What is the largest flow of record on the gage? Are the cross sections high enough to contain more than this?	Yes. 164,000-cfs in 1950 and 105,000 in 1993. Cross sections are definitly high enough.	X
4	Is the cross section depth reasonably uniform on all cross sections?	Yes	X
5	Is the most upstream cross section drawn through the gage location?	Yes. Gage Location: on L bank @ u/s side of br on US Hwy 136. Check.	X
6	Cross sections at all road/bridge constrictions? Three cross sections - one through the centerline and one on either side outside the influence of embankment elevation.	Yes. Need to discuss cross section arangment at bridges/railroads. One cross section just to model the embankment? Or two so as to set the model up for bridge routine?	This was discussed. Three are appropriate to capture visible embankments. If embankments are not different 1 XS is appropriate. Some small roads which are not detectable on the DEM at all may be neglected.
7	Cross sections at any major geographical constriction? ie. narrow floodplain?	Yes.	X
8	Last cross section is cut as close to the confluence as possible without crossing Missouri River cross sections.	Yes, fix overlap.	X
9			X

	Reviewer	Backcheck
Other Comments:	There are levees on both sides of the Little Nemaha starting at RM 4. Discuss triming the cross sections to model these as storage areas.	In HEC-RAS these are trimmed to center of levee.
	Consider converting from feet to miles in GIS before exporting.	This is not necessary, the conversion is just as convenient in HEC-RAS.

HEC-RAS Review Checklist

Tributary: Little Nemaha
MoRiver RM: 527

Review #2: Geometry and Unsteady Run

Designer/Date: Lisa Stahr (CDM Smith)/ 28 Nov 2012
Reviewer/Date: Jean Hilger/ 30 Nov 2012
Backcheck/Date: Lisa Stahr (CDM Smith)/ 14 Feb 2013

	Task	Reviewer	Backcheck
1	Unsteady model runs without error.	Yes.	X
2	How do HTab Param (geometry property) look? Points set to 100? Reasonable increment?	Yes - points set to 100. Incriment on most x-sec is 0.5-ft, which is fine. Not all x-sections have H-Tab parameters to the top of the cross section. Stage of record is 27.65 (1950) + NAVD88 datum of 890.15 = 917.8. Most of the increments extend up to elev 940, I would think this would be plenty high. 3 are a little low... 4.70, 3.89, and 0.94. Was there a reason for smaller incriment? Consider using incriment of 0.5-ft for consistancy with all other x-sections.	HTab parameters have been modified for the XS in question.
3	How HT tables (calculated in pre-processor) look? Any sudden jumps or discontinuities?	Look fine.	X
4	Does the time period chosen for unsteady run include low flows, high flows, and the range in between?	Yes. Ranges from 400-cfs at the low to 41,000-cfs at the high.	X
5	Upstream boundary condition properly input?	Yes. 15 min flow readings at Auburn, NE.	X
6	Downstream boundary condition properly input?	DS bdry was interpolated stage, caused some wobbling at the bottom of the profile. Replaced with stage hydrograph output from mainstem model - no wobbles.	Interpolated stage BC replaced with real data provided by the USACE in the Trib_BC.DSS. June 2010 contains a wide range of flows, but not the max on record, so the June record was taken by a multiplier (3.93) to yield a flow of 164,000 cfs. The stage hydrograph BC was also multiplied by a factor (1.020) to better reflect stages at higher flows. This factor was determined based on the difference between stages for the June 2010 data and the max flow data.
7	Bed data captured by surface? If no, was the method used to estimate bed data reasonable and produce reasonable results at low flows?	Channel data was estimated using measured data at the USGS gage @ Auburn and extrapolated downstream.	X
8	Does the thalweg profile look reasonable?	Yes.	X
9	Overbank flowpaths reasonable?	Yes.	X
10	Are the bank stations for the channel reasonable? Reasonably consistant upstream and downstream? (No huge changes in width and elevations.)	Yes. Average 5-ft deep and 150-ft wide.	X
11	Are roughness coefficients reasonable?	Yes. Channel 0.035 and all overbanks as crop 0.05.	X

HEC-RAS Review Checklist

Tributary: **Little Nemaha**

MoRiver RM: **527**

12	Does the HEC-RAS Water Surface Profile look reasonable?	Yes, until flows of above 35,000. ie. timestep 21 June 2010 @ 1220. Spikes in WS profile in a few places. The one at x-sec 8.75 is because at x-sec 8.29 the top width is 400-ft, but u/s and d/s the top widths are wider than 4,000-ft. Adjust ineffective flow locations and elevations for better consistancy.	Current model does not have this issue.
13	Is the water surface profile contained within the cross sections?	Yes	X
14	Is there divided flow in any cross-sections? This is okay as long as it makes physical sense.	Is fine.	X
15	Does the calculated rating curve at the gage cross section come reasobably close to replicating the observed rating curve?	Yes. Low flows are a little too high and high flows are a little too low, but generally is close. Since there is some error in rating curves to begin with it should probably not be used as a tool for exact calibration anyway, just as a general guide.	X
16	Ineffective flows or levee points used?	May need to be adjusted up or down for consistancy in top width.	X
17	Are the velocities consistent through the reach? Are the velocities reasonable? Velocities that are much higher or lower than typical should be flagged for closer inspection.	During the peak of the June 2010 event there are big spikes in the channel velocities. This is probably also because of ineffective flow areas. Ie. 21 June 2010 @ 1220.	X
18	Is Freud's number equal to 1 at any of the cross sections? If yes, is this true to reality? If not it could be a red flag that there is something wrong with the model.	Nope. Okay.	X

Other Comments:

HEC-RAS Review Checklist

Tributary: **Tarkio**
MoRiver RM: **507**

Model Data/Major Assumptions

	Item	
1	Horizontal Projection	NAD_1983_UTM_Zone_15N
2	Vertical Datum	NAVD88
3	Surface(s) used to cut cross sections	USGS NED 2009, 1/3 arc and LiDAR data downstream as provided by the KC USACE.
4	Channel data - if estimated, what were the dimensions chosen (width, side slopes, invert, slope, etc.) and what data source did you use to guide the estimates	The Tarkio model is somewhat complicated in that it seemed to have a topo data discrepancy , which broke at XS 11.35. Upstream of XS 11.35, gage data was used to estimate the invert upstream, which was projected downstream to XS 11.74. The upstream invert used was the gage datum + lowest gage reading = 872.13 feet. LiDAR data was collected and available from the downstream end to XS 8.29. However, 2 additional XS were within (but not fully within) the LiDAR data, XS 8.81 and 9.32, so the inverts here could also be used estimated. This adjusted data using LiDAR was smoothly projected for remaining XS 9.7 up to 11.35. XS 11.74-13.56 sideslopes: 5 bottom width:~80 feet XS: 0.06-11.11 sideslopes: 2-2.5 bottom width: 40-45 feet LiDAR data appeared to capture the WS, so 6.5 feet was subtracted from the LiDAR data inverts to account for the gage stage reading in January 2012 when the LiDAR was collected. Downstream invert is elevation 847.53 Tarkio XS do overlap with some MO River XS, but this was discussed with the KC USACE and the decision was made to have them overlap and MO River XS or Tarkio XS will be adjusted accordingly. Bottom line: better to have too much data than too little.
5	Other sources of data used	RC provided on the USGS gage website.
6	Other assumptions and notes	Channel Manning's n value was modified from 0.035 to 0.04. Min Flow file created to complete the test of a range of flows. Basically, the June 08 data was multiplied by a factor to decrease the lowest flow in this record to 23 cfs (factor 0.03129). The BC was also reduced to reflect the reduced flow and was also multiplied by a factor based on the difference in stage between the max flows and June 2008 flows (factor 0.99496). The same formula was applied to create the max flow file The June 08 data was multiplied by a factor to increase the highest flow in this record to 16,300 cfs (factor 1.3). The BC was also increased to reflect the increased flow and was multiplied by a factor based on the difference in stage between the max flows and June 2008 flows (factor 1.0030).

HEC-RAS Review Checklist

Tributary: Tarkio
MoRiver RM: 507

Review #1: Cross Section Layout

Designer/Date: Lisa Stahr (CDM Smith)/ 17 Oct 2012
Reviewer/Date: Jean Hilger/ 01 Nov 2012
Backcheck/Date: Lisa Stahr/ 20 Nov 2012

	Task	Reviewer	Backcheck
1	Cross section spacing less than 1/2 mile (less than approx 2600 feet)?	Consider adding another x-sec between 11.83 and 11.16? Reach length is 3511. That's not too far off, though. Otherwise okay.	XS have been revised.
2	Cross sections perpendicular to channel flow and perpendicular to floodplain flow?	Yes. Few tweaks. Avoid large bends in the middle of the floodplain as this probaby will give a larger floodplain width than the water sees.	Adjustments made.
3	What is the largest flow of record on the gage? Are the cross sections high enough to contain more than this?	16,300 in 1942, and yes.	X
4	Is the cross section depth reasonably uniform on all cross sections?	Is okay. May want to give some consideration to this, ie. RS 6.24 on the left goes all the way up to 1050, may not need quite so long/high of cross sections if the max period of record is probably way down at elev 890. But wide extents are good for inundation maps. Also, noted that within the levees and extreemly flat areas it will not be possible to contain the water. This wil be accounted for with lateral structures and storage areas.	Adjustments made.
5	Is the most upstream cross section drawn through the gage location?	No - move to downstream side of the bridge and roadway.	Corrected.
6	Cross sections at all road/bridge constrictions? Three cross sections - one through the centerline and one on either side outside the influence of embankment elevation.	Need cross section (one will do) at roadways within the leveed reach? Add x-sec @ CLs of all crossings.	Done.
7	Cross sections at any major geographical constriction? ie. narrow floodplain?	If possible, within the levees, snap the end of the cross section to the boundary of the levee (no overlap) so the cross section ends at the centerline of the levee, this way we don't overcount conveyance or storage.	Done.
8	Last cross section is cut as close to the confluence as possible without crossing Missouri River cross sections.	Yes. However, not sure how best to approach this area, once the levee ends, since it's so flat! Should the cross sections be a little longer? How did you choose where to end them? We may be able to adjust MoRiver x-sections if that ends up working best.	XS at the DS were cut a little longer, but as stated above, adjustment will have to be made either to the Tarkio XS or the MO R XS.
9	Is the river centerline reasonable?	I would sketch your own rather than using the shapefile we gave you, which doesn't seem to line up with the center of the river very well between the levees.	X

HEC-RAS Review Checklist

Tributary: Tarkio
MoRiver RM: 507

Other Comments:

There's almost no chanel definition at the downstream end (for example RS 0.67), this can't be right. Hopefully Lidar and estimating channel data will improve this.

LiDAR did improve channel definition at the downstream.

Review #2: Geometry and Unsteady Run

Designer/Date: Lisa Stahr (CDM Smith)/ 28 Nov 2012
Reviewer/Date: Jean Hilger/ 03 Dec 2012
Backcheck/Date: Lisa Stahr/ 14 Feb 2013

	Task	Reviewer	Backcheck
1	Unsteady model runs without error.	Yes.	X
2	How do HTab Param (geometry property) look? Points set to 100? Reasonable increment?	100 points, 0.5-ft incriment. Looks good.	X
3	How HT tables (calculated in pre-processor) look? Any sudden jumps or discontinuities?	All x-secs with levee points have huge jumps in flow area @ the top of the levee. Add ineffective flow areas 1-ft abov the levee top for transition.	X
4	Does the time period chosen for unsteady run include low flows, high flows, and the range in between?	12,600 in June 2008, 50-150-cfs in Sept 2009	
5	Upstream boundary condition properly input?	Yes.	X
6	Downstream boundary condition properly input?	Yes.	X
7	Bed data captured by surface? If no, was the method used to estimate bed data reasonable and produce reasonable results at low flows?	Yep!	X
8	Does the thalweg profile look reasonable?	Yes! Outstanding accomplishment considering the raw data.	X
9	Overbank flowpaths reasonable?	Yes	X
10	Are the bank stations for the channel reasonable? Reasonably consistant upstream and downstream? (No huge changes in width and elevations.)	Yes. Seems a little small at the top (3-ft deep, 110-ft wide), really small at x-sec 11.35. Please explain reasoning for bank station position.	Bank stations adjusted. Consistently set near normal Tarkio gage ht. reading of 5 feet.

HEC-RAS Review Checklist

Tributary: **Tarkio**
MoRiver RM: **507**

11	Are roughness coefficients reasonable?	Yes. 0.04 for channel 0.045 for overbanks. (At a few of the cross sections the 0.04 n-value got off centered from the bank stations for some reason.)	X
12	Does the HEC-RAS Water Surface Profile look reasonable?	Yes.	X
13	Is the water surface profile contained within the cross sections?	Yes	X
14	Is there divided flow in any cross-sections? This is okay as long as it makes physical sense.	No. Blocked by levee points.	X
15	Does the calculated rating curve at the gage cross section come reasobably close to replicating the observed rating curve?	Looks really excellent in the 1,000-9,000 cfs range. And in the low flows. Above 10,000-cfs the calculated curve flattens out while the observed continues to increase. This is because there are a few cross sections where levees are overtopping. If we add ineffective flow areas 1-ft above levee elevations I think this will improve.	X
16	Ineffective flows or levee points used appropriately?	Cross section 10.02 should probably have levee poitns around the height of the upstream and downstream levee poitns. Cross section happened to be drawn on a tie-back. I think that's why no obvious levee shows up in the x-sec geometry.	Correction made.
17	Are the velocities consistent through the reach? Are the velocities reasonable? Velocities that are much higher or lower than typical should be flagged for closer inspection.	Velocities are all right around 6 ft/s except for in the MoRiver backwater area. Which is very reasonable. With the exception of x-sec 10.02 and the x-sec where levees overtop the velocity drops way down. Levee/ineff fixes will likely correct this.	X
18	Is Freud's number equal to 1 at any of the cross sections? If yes, is this true to reality? If not it could be a red flag that there is something wrong with the model.	No.	X

Other Comments:	Is there a reason the cross sections in the leveed area do not line up with the CL of levee/perimeter of storage area?	Most of them are very close. Discussed this: decided the few that didn't extend quite far enough would have negligible effect on the results.
	Still need to address transition at the MoRiver confluence. Esp. maybe look for a smoother transition between the narrow leveed area and the wide confluence x-sections.	This is probably something that must be considered as the MO River unsteady model development progresses.

HEC-RAS Review Checklist

Tributary: **Big Nemaha**

MoRiver RM: **495**

Model Data/Major Assumptions

	Item	
1	Horizontal Projection	NAD_1983_UTM_Zone_15N
2	Vertical Datum	NAVD88
3	Surface(s) used to cut cross sections	USGS NED 2009, 1/3 arc
4	Channel data - if estimated, what were the dimensions chosen (width, side slopes, invert, slope, etc.) and what data source did you use to guide the estimates	4/11/2013 - Modification to channel data. After comparing the d/s thalweg elevation of the Big Nemaha to MoRiver tailwater elevations at low flow it was apparent that the d/s thalweg was too high (elev 841-842) 2009 Hydrosruvey up the tribs also confirmed this (elevs 833-836). A few other pieces of data supported this change. 1 - the overall WS profile at the max flow is steeper than the bed of the river, and 2 - CRP is 843.4 at the mouth (which is just 1-ft above the bottom x-sec elevation, completly unrealistic). The channel bottom of the four d/s cross sections was dropped. Although, I think this might have been a quick fix, and will need to re-evaluate later. Lisa Stahr's notes as follows --- The NED2009 data actually fit reasonably with the channel as estimated from the USGS data. LiDAR data was provided from XS 6.36 downstream and was merged with the NED2009 data to represent the channel. Channel dimensions were generally as follows: The sideslope used in the HEC-RAS model was 4 and bottom width of closer to 50 ft. In addition, some slight modifications were made to XS where this general fit was not quite right. Channel slope used to make modifications was 0.00029 (XS 13.66 to 6.36) and 0.00018 (XS 6.36 to ds). Also, the channel profile between XS 6.36 - 11.20 was adjusted to be "filled" when channel geometry changes were made, as these XS were much deeper than the others. This helped smooth the profile (and results) out. Boundary conditions were based on the Trib BC DSS file provided by the USACE (JHilger). Spreadsheet "XS approximation based on USGS data_20120926" was used to determine channel shape and additional details can be found in that spreadsheet, which utilizes data taken directly from the USGS web site.
5	Other sources of data used	Rating Curve from the USGS website.

HEC-RAS Review Checklist

Tributary: **Big Nemaha**

MoRiver RM: **495**

6	Other assumptions and notes	Channel Manning's n revised from 0.035 to 0.025.

Review #1: Cross Section Layout

Designer/Date: **Lisa Stahr (CDM)/ 17 Oct 2012**

Reviewer/Date: **Jean Hilger/ 02 Nov 2012**

Backcheck/Date: **Lisa Stahr/ 30 Jan 2013**

	Task	Reviewer	Backcheck
1	Cross section spacing less than 1/2 mile (less than approx 2600 feet)?	A few that are more than this. Check and perhaps adjust.	x
2	Cross sections perpendicular to channel flow and perpendicular to floodplain flow?	Yes	x
3	What is the largest flow of record on the gage? Are the cross sections high enough to contain more than this?	Yep. 71,600-cfs in 1974.	x
4	Is the cross section depth reasonably uniform on all cross sections?	Yes	x
5	Is the most upstream cross section drawn through the gage location?	Yes. Don't need cross sections through the centerline and upstream of the bridge, only downstream.	x
6	Cross sections at all road/bridge constrictions? Three cross sections - one through the centerline and one on either side outside the influence of embankment elevation.	No crossings.	x
7	Cross sections at any major geographical constriction? ie. narrow floodplain?	Yes.	x
8	Last cross section is cut as close to the confluence as possible without crossing Missouri River cross sections.	Cut closer, we will adjust the mainstem cross section so there is no crossing.	x
9	Is the river centerline reasonable?	Yes.	x

HEC-RAS Review Checklist

Tributary: Big Nemaha

MoRiver RM: 495

Other Comments: There are levees at the bottom end of this tributary (noticing a trend….) trim cross sections to the protected area shapefile. x

Review #2: Geometry and Unsteady Run

Designer/Date: Lisa Stahr (CDM)/ 28 Nov 2012

Reviewer/Date: Jean Hilger/ 03 Dec 2012

Backcheck/Date: Lisa Stahr/ 30 Jan 2013

	Task	Reviewer	Backcheck
1	Unsteady model runs without error.	Yes	x
2	How do HTab Param (geometry property) look? Points set to 100? Reasonable increment?	Yes. 100 points, 0.5-ft increment.	x
3	How HT tables (calculated in pre-processor) look? Any sudden jumps or discontinuities?	Ok.	x
4	Does the time period chosen for unsteady run include low flows, high flows, and the range in between?	27,000 in June 2010	x
5	Upstream boundary condition properly input?	Yes.	x
6	Downstream boundary condition properly input?	Yes.	x
7	Bed data captured by surface? If no, was the method used to estimate bed data reasonable and produce reasonable results at low flows?	Yes.	x
8	Does the thalweg profile look reasonable?	Yes. The profile is pretty smooth, except for a bump at 6.73. Was there a reason for this?	x
9	Overbank flowpaths reasonable?	Yes	x
10	Are the bank stations for the channel reasonable? Reasonably consistant upstream and downstream? (No huge changes in width and elevations.)	Yes.	x

HEC-RAS Review Checklist

Tributary: **Big Nemaha**

MoRiver RM: **495**

11	Are roughness coefficients reasonable?	0.025 for channel 0.045 for overbank. Is there a reason for such a low choice for channel roughness? Overbanks are mostly ag so 0.045 is fine.	Lowered to help better match the RC.
12	Does the HEC-RAS Water Surface Profile look reasonable?	Yes.	x
13	Is the water surface profile contained within the cross sections?	Yes.	x
14	Is there divided flow in any cross-sections? This is okay as long as it makes physical sense.	Ineffectives used.	x
15	Does the calculated rating curve at the gage cross section come reasobably close to replicating the observed rating curve?	Yes. A little low at lows and a little high at highs. But not unreasonable.	x
16	Ineffective flows or levee points used appropriately?	Yes.	x
17	Are the velocities consistent through the reach? Are the velocities reasonable? Velocities that are much higher or lower than typical should be flagged for closer	Yes. A little spikey in the are of RM 6-12, but doesn't appear to be any major issue causing this.	x
18	Is Freud's number equal to 1 at any of the cross sections? If yes, is this true to reality? If not it could be a red flag that there is something wrong with the model.	Ok.	x

Other Comments:

LiDAR (provided by USACE) data used from XS 6.36 downstream.

HEC-RAS Review Checklist

Tributary: Nodaway
MoRiver RM: 463

Model Data/Major Assumptions

	Item	
1	Horizontal Projection	NAD_1983_UTM_Zone_15N
2	Vertical Datum	NAVD88, Foot_US
3	Surface(s) used to cut cross sections	morivnedutm
4	Channel data - if estimated, what were the dimensions chosen (width, side slopes, invert, slope, etc.) and what data source did you use to guide the estimates	The USGS measured data (top width vs. stage, and flow area vs. stage) was used to approximate the channel data mssing from the NED surface. http://waterdata.usgs.gov/mo/nwis/uv?site_no=06817700
5	Other sources of data used	Storage areas and lateral structures were cut from the 2011 NED
6	Other assumptions and notes	The selected geometry for integrating into the model, for now, is up to the Graham gage. However, this gage has only been in opration since 1982, wheras the next upstream gage @ Burlington Junction has records back to 1992. Therefore, we created geometry up to Burlington Junction but decided only to put to Graham in the model. I think we'd prefer to have a consistant geometry for use in calibration AND running the period of record, so we considered leaving up to Burlington Juncion in the model and running a pilot/bogus flow (ie. 1-cfs) just to have water in the model, and putting the Graham flows in as a lateral point inflow, however, could not get this to run stable. We found that this scenario could be stable by bumping up the pilot flow (to 25 or 30 or higher) or by using time slicing (down to 5 minute), but prefered not to have to do either of these two things because they are not as transparent as having a stable, consistant model. Later, while running the period of record, the decsion still could be made to exend the model to Burlington.

Review #1: Cross Section Layout

Designer/Date: Adam Jones / 18 Dec 2012
Reviewer/Date: Jean Hilger / 27 Dec 2012
Backcheck/Date: Adam Jones/ 28 Dec 2012

	Task	Reviewer	Backcheck
1	Cross section spacing less than 1/2 mile (less than approx 2600 feet)?	For the most part, yes. A few that are around 3,000, but close enough.	-

HEC-RAS Review Checklist

Tributary: **Nodaway**

MoRiver RM: **463**

2	Cross sections perpendicular to channel flow and perpendicular to floodplain flow?	Yes. Good job with this.	-
3	What is the largest flow of record on the gage? Are the cross sections high enough to contain more than this?	In 1993 peak flow @ Graham was 90,700-cfs.	-
4	Is the cross section depth reasonably uniform on all cross sections?	No… many cross sections in the downstream half of this model are not drawn to high enough ground. Please double check this and extend cross sections where necessary.	Revised.
5	Is the most upstream cross section drawn through the gage location?	Yes. (Burlington Junction)	Will shorten model.
6	Cross sections at all road/bridge constrictions? Three cross sections - one through the centerline and one on either side outside the influence of embankment elevation.	Yes, good job with this. Only missing one downstream of Skidmore (Hwy 113)	Revised.
7	Cross sections at any major geographical constriction? ie. narrow floodplain?	Yep.	
8	Last cross section is cut as close to the confluence as possible without crossing Missouri River cross sections.	Downstream of the bridge can add more cross sections, trim to centerline of levee. (May need you to create accurate storage areas for this area because they don't look quite right in the protected area shapefile.)	Revised downstream XS's.
9	Is the river centerline reasonable?	Looks like Bing Maps and the DEM differ in some locaitons. You followed Bing Maps, which is likely the most current. Do we see this causing any problems since it is different than the surface we are cutting the cross sections from, or will it be okay?	I believe it will be okay. HEC-RAS may throw up an error that the bank stations are not in-line with the river centerline, but the computations should continue just the same. HEC-RAS runs off of the rating curves it computes for the XS's, so assuming the basic parameters of the XS (channel width, depth, area) haven't changed much, the only problem is with the visual representation.

Other Comments:	End model @ old Burlington Junction gage, no need to go all the way to Clarinda, Iowa. The period of record @ Graham is 1982 - current, and the period of record @ Burlington Juncion is 1922 - 1983. Clarinda's record is 1918-1925 and 1936 - current. The dranage area is less than half the area at Graham (762 compared to 1,520) and and since the gage at Burlington goes all the way back to 1922 it doesn't seem like much value will be added to the model by going all the way to Clarinda. Thoughts? Agree or disagree?	Will shorten model to Burlington Junction. Was unsure of protocol, and Clarinda gives one more calibration point.
	Several cross sections (ie. 45.77, 45.77, 44.29, 42.85 just to name a few) are drawn up low lying tributary areas instead of finding high ground, eggagerating the conveyance and floodplain width. What is the reason for this? If there is a good hydraulic reason, than explain. If not, please find high ground sooner.	This is a gray area to me. The two benefits I see to doing this: (1) Mapping; and (2) Modeling. (1) Mapping: should this data be mapped, it will represent the extent and depth accurately; (2) Modeling: there are multiple small tribs that enter the Nodaway along its length, and by creating ineffective flow areas along these backwater trib XS's, I theorize that it economizes both in terms of model creation time and stability by representing some portion of the otherwise unrepresented backwater storage area along the Nodaway River mainstem, and minimizes the need to create storage areas, which can lead to issues with model stability. Since it appears to be protocol to not represent the tributary backwaters with extended cross-sections and ineffective flow areas, I have removed them from the geometry or altered according to recommendation.

HEC-RAS Review Checklist

Tributary: Nodaway

MoRiver RM: 463

Other Comments:

There are four cross sections (2.48, 2.18, 1.80, 1.39) drawn at an extreem bend in the river… can we find a way to make these also work for high flows? Not sure this is the best cross section layout for that. May need to re-draw.

Revised.

SIDE NOTE (AQJ): I am not certain what percentage of the time that the Nodaway spends out of its banks, but just off the top of my head, I would estimate ~95+% is spent within banks (annually). Since the low flows are usually the most difficult to calibrate to, I decided to largely trace the bank lines. I can't quote a particular document, but in the classes I have taken, the folks at HEC have said that overbank flow lengths are not particularly sensitive, and I would still wager to guess that the centroid of overbank flow is very close to the main channel for almost all but the highest events. Therefore, I largely traced the top-of-bank as the overbank flowpaths. Additionally, for models built to evaluate mainly (low-frequency) design events, I would advocate sticking with flowpaths at approximately 1/3 of the overbank distance to the left or right of the channel on the respective overbank; however, this is not the case, with this model expected to reproduce a period of record.

Review #2: Geometry and Unsteady Run

Designer/Date: Adam Jones / 11 Jan 2013

Reviewer/Date: Jean Hilger / 13 Feb 2013

Backcheck/Date: Adam Jones / 14 Feb 2013

	Task	Reviewer	Backcheck
1	Unsteady model runs without error.	Yes.	
2	How do HTab Param (geometry property) look? Points set to 100? Reasonable increment?	Points are all @ 100. Incriment varries from 0.3 to 0.5, which I assume was troubleshooting. Looks fine as long as the cross sections with smaller incriments reach high enough elev.	This was done to hopefully aid in computations and possibly accuracy. The HEC-RAS generated spacing went well above the top of overbanks, whereas the increment values were lowered to keep the computations to the applicable stages and assist in generating detailed rating curves for each XS, as opposed to interpolating.
	How HT tables (calculated in pre-processor) look? Any sudden jumps or discontinuities?	Ok.	-
3	Does the time period chosen for unsteady run include low flows, high flows, and the range in between?	Ran the range 150-cfs to 52,000-cfs. Created another plan called "Max" to test the flow of record (90,700). Ran Grahm flows for May and June 2008 from Burlington with a multiplier of 1.75 so that the peak of the June event would match the flood of record. Model seems to handle this fine.	-
4	Upstream boundary condition properly input?	Yes. Flow hydrograph from DSS. Plan 1, @ Grahm (x-sec 28.90). Plan 2 @ Burlingotn Junction.	-
5	Downstream boundary condition properly input?	Yes. Stage hydrograph for Plan 1, normal depth for plan 2 since there is no model runs for 1982.	-

HEC-RAS Review Checklist

Tributary: **Nodaway**

MoRiver RM: **463**

6	Bed data captured by surface? If no, was the method used to estimate bed data reasonable and produce reasonable results at low flows?	Please describe how you did this, basic assumptions, etc. To Document. CDM Smith has also been including a geometry in their submittals that is the original cut from the DEM. --- Looks reasonable.	The bed data was developed according to a method described by CDM for this effort. The methodology is described in the worksheet tab within this document titled "XS development methodology". I still have the GIS exports of the "original" geometry within the new HEC-RAS folder that will be submitted for final review, as well as a GISImport geometry
7	Does the thalweg profile look reasonable?	Yes	-
8	Overbank flowpaths reasonable?	Yes. Explanation above.	-
9	Are the bank stations for the channel reasonable? Reasonably consistant upstream and downstream? (No huge changes in width and elevations.)	Yes. They were a little higher than I expected and there are a few odd spikes (plot on profile to check it out), but produce good results, so ok.	-
10	Are roughness coefficients reasonable?	channel n-value 0.03, overbank n-values 0.07. These were tweaked to better match the rating curve. Within the reasonable range - ok.	-
11	Does the HEC-RAS Water Surface Profile look reasonable?	Yes. Very good.	-
12	Is the water surface profile contained within the cross sections?	Yes.	-
13	Is there divided flow in any cross-sections? This is okay as long as it makes physical sense.	Ok.	-
14	Does the calculated rating curve at the gage cross section come reasobably close to replicating the observed rating curve?	A little too high at the low flows and a little too low at the high flows. Most likely, these differences are due to bad/incomplete DEM surface data, therefore inaccuracies in our geometry. This is tracking farily well for the data that we do have.	-
15	Ineffective flows or levee points used appropriately?	Ineff @ x-secs 18.92, 19.41, 19.88…. Should these be a little higher so they don't turn off during the highest flows in June? This is a big constriction in the floodplain, and I think we do need some ineffective areas. Also -- do we need some ineffectives or levee points @ 0.93 and 0.89 just upstream of the bridge?	The 18.92-19.88 series of ineffective flow areas (IFA's) has been adjusted from 855 to 865 so that they are not overtopped. XS's 0.80, 0.89, and 0.93 had IFA's added.
16	Are the velocities consistent through the reach? Are the velocities reasonable? Velocities that are much higher or lower than typical should be flagged for closer inspection.	Velocities look really reasonable 2-3 ft/s during low flows, 4-8 ft/s during peaks of floods. Nice.	-
17	Is Freud's number equal to 1 at any of the cross sections? If yes, is this true to reality? If not it could be a red flag that there is something wrong with the model.	Yes, @ x-sec 23.07 on 07JUN2008 at 1000. I think this is because this is it is a constriction. If we wanted to, we probably could solve this with a better transition, but I think it's ok.	-
	Other Comments:	2 plans reviewed. Same geometry, different flows. #1 - Interm Review: 20-cfs from Burlington Junction (insignificant ammount, only there so that the model doesn't give error) and observed flows input @ the Grahm Gage as a lateral inflow. March - Sept 2008.	-

HEC-RAS Review Checklist

Tributary:	**Nodaway**
MoRiver RM:	**463**

Other Comments:	#2 - 1982-1983 data overlap: For one year, both the Burlington Junction and Grahm gage were in operation, flow input was observed @ Burlington Junction and used observed flow and Stage @ Grahm to validate the model calculations.	-
	Oops, looks like some of our flow inputs (Nodaway.dss) got mixed in with the outputs, also Nodaway.dss (since this is the name of the HEC-RAs project). It's best practice to keep these separate. Create a Nodaway_Obs.dss with the USGS daily data @ Burlington, Clarinda, and Grahm, and point to that instead.	Revised.
	I added observed stage @ Grahm. HEC-RAS does not recongize time series from DSS unless it is regular. Had to convert the observed stage from IR-MONTH to 15 min. That's my bad, I should have had this done for you already.	-
	Storage areas and lateral structures @ the d/s end look really good. Looks like the only overtop during the peak of the June 2008 event, which seems reasonable.	-
	One quick question about the LS… did you ened the lateral structure on the ROB @ the confluence with the Missouri, or did you loop it around as is in the geodatabase?	Once the model is joined in, I intend to import the LS, as in the geodatabase. The current Nodaway River model does not include the Missouri River portion of the LS, though.
	Check it out! Some photos! K:\MissionProjects\sec\ed-h\MoRiver_Models\500_Rivers\512_Data\Photos\2012-10-18_MoRiver-Helicopter\RM463_NodawayRiver	

HEC-RAS Review Checklist

Tributary: **Platte**

MoRiver RM: **391**

Model Data/Major Assumptions

	Item	
1	Horizontal Projection	NAD_1983_UTM_Zone_15N (Foot US)
2	Vertical Datum	NAVD88
3	Surface(s) used to cut cross sections	USGS NED 2009, 1/3 arc, and LiDAR provided by the KC USACE.
4	Channel data - if estimated, what were the dimensions chosen (width, side slopes, invert, slope, etc.) and what data source did you use to guide the estimates	The NED2009 data actually fit reasonably with the channel as estimated from the USGS data. LiDAR data was provided from XS 6.34 downstream and was merged with the NED2009 data to represent the channel. Channel dimensions were generally as follows: The sideslope used in the HEC-RAS model was 3 and bottom width of closer to 60 ft from the US-XS 6.34; from XS 6.34p-DS, sideslope was 4 and channel bottom width was 65 ft. In addition, some slight modifications were made to XS where this general fit was not quite right. Channel slope used to make modifications was 0.00019 and 0.00012 (DS 2 miles). Boundary conditions were based on the Trib BC DSS file provided by the USACE (JHilger). Spreadsheet "XS approximation based on USGS data_20120926" was used to determine channel shape and additional details can be found in that spreadsheet, which utilizes data taken directly from the USGS web site.
5	Other sources of data used	RC used for comparison came from the USGS website.
6	Other assumptions and notes	OB Manning's n revised from 0.045 to 0.05.

HEC-RAS Review Checklist

Tributary: **Platte**
MoRiver RM: **391**

Review #1: Cross Section Layout

Designer/Date: **Lisa Stahr (CDM)/ 17 Oct 2012**
Reviewer/Date: **Jean Hilger/ 05 Nov 2012**
Backcheck/Date: **Lisa Stahr/ 20 Nov 2012**

	Task	Reviewer	Backcheck
1	Cross section spacing less than 1/2 mile (less than approx 2600 feet)?	Because the river is so sinuous it's hard to get cross sections spacing this close in some areas. Try your best to adjust this if possible. This is important for unsteady model stability. Not sure how important it will be (may end up being a non-issue), but once we start calibrating I'd rather not have to go back and add more cross sections because our spacing was too far apart. Better safe than sorry.	Added a few XS.
2	Cross sections perpendicular to channel flow and perpendicular to floodplain flow?	Yes.	X
3	What is the largest flow of record on the gage? Are the cross sections high enough to contain more than this?	Yes. May 2007, 41,800-cfs.	X
4	Is the cross section depth reasonably uniform on all cross sections?	Yes.	X
5	Is the most upstream cross section drawn through the gage location?	First cross section should be drawn at the downstream face of the bridge. Don't worry about the 3 cross sections for this bridge.	X
6	Cross sections at all road/bridge constrictions? Three cross sections - one through the centerline and one on either side outside the influence of embankment elevation.	Three cross sections and add cross sections at a few roadways.	X
7	Cross sections at any major geographical constriction? ie. narrow floodplain?	Fine.	X
8	Last cross section is cut as close to the confluence as possible without crossing Missouri River cross sections.	Could be a little closer	X
9	Is the river centerline reasonable?	Yes.	X

Other Comments: Trim cross sections to centerline of levees.

HEC-RAS Review Checklist

Tributary: Platte
MoRiver RM: 391

Review #2: Geometry and Unsteady Run

Designer/Date: Lisa Stahr (CDM)/ 28 Nov 2012
Reviewer/Date: Jean Hilger/ 03 Dec 2012
Backcheck/Date: Lisa Stahr/ 13 Feb 2013

	Task	Reviewer	Backcheck
1	Unsteady model runs without error.	Yes.	X
2	How do HTab Param (geometry property) look? Points set to 100? Reasonable increment?	Invert elevations appear to be set at the old channel invert. I'm not sure if this makes a differnce in calculations, but I would think it would.	This was an oversight. Should be set to invert. Correction made.
3	How HT tables (calculated in pre-processor) look? Any sudden jumps or discontinuities?	Ok.	X
4	Does the time period chosen for unsteady run include low flows, high flows, and the range in between?	Ok. As low as 700-cfs and as high as 21,000-cfs. (Interesting fact: Today the Platte is at 43-cfs… exceptionally low.)	X
5	Upstream boundary condition properly input?	Yes.	X
6	Downstream boundary condition properly input?	Yes.	X
7	Bed data captured by surface? If no, was the method used to estimate bed data reasonable and produce reasonable results at low flows?	Yes.	X
8	Does the thalweg profile look reasonable?	Yes.	X
9	Overbank flowpaths reasonable?	Yes.	X
10	Are the bank stations for the channel reasonable? Reasonably consistant upstream and downstream? (No huge changes in width and elevations.)	Yes. Explain thought process for height and width of bank stations.	Generally, bank stations are set near the normal water surface. For the Platte, that is a gage height reading between 4-5 feet, which is how the bank stations were set.
11	Are roughness coefficients reasonable?	Yes. 0.035 for channel and 0.05 on overbanks. Based on observed stage and rating curve perhaps channel n-value should be decreased and overbanks bumped up. Also, check out aerial photos. Lots of trees in the platte valley.	Aerials checked and n-values adjusted accordingly. Channel n-value is still 0.035.
12	Does the HEC-RAS Water Surface Profile look reasonable?	Yes.	X
13	Is the water surface profile contained within the cross sections?	Yes.	X
14	Is there divided flow in any cross-sections? This is okay as long as it makes physical sense.	Ok.	X

HEC-RAS Review Checklist

Tributary: **Platte**

MoRiver RM: **391**

15	Does the calculated rating curve at the gage cross section come reasobably close to replicating the observed rating curve?	Calculating too high on the lows and too low on the highs, but still looks very reasonable.	X
16	Ineffective flows or levee points used appropriately?	Check out the top width at the peak of the May 2010 flood. It bounces around a TON. I think this is in part because of some ineffective flows being overtopped and others not. Personally I've always had a hard time determining the best way to approach this. The only reason I think it may need to be addressed is because the stage at the gage flattens out during the peak... possibly because of the set up of ineffective flows.	Did not address
17	Are the velocities consistent through the reach? Are the velocities reasonable? Velocities that are much higher or lower than typical should be flagged for closer inspection.	Relativly consistant. Within the reasonable range of 4-6 ft/s.	X
18	Is Freud's number equal to 1 at any of the cross sections? If yes, is this true to reality? If not it could be a red flag that there is something wrong with the model.	Ok.	X

Other Comments:

HEC-RAS Review Checklist

Tributary: **Kansas River**
MoRiver RM: **367.45**

Model Data/Major Assumptions

	Item	
1	Horizontal Projection	NAD_1983_UTM_Zone_15N
2	Vertical Datum	NAVD88, Foot_US
3	Surface(s) used to cut cross sections	2011 National Elevation Dataset, morivnedutm; Levee elevations are surveyed NLD elevations extracted from 2009 survey data
4	Channel data - if estimated, what were the dimensions chosen (width, side slopes, invert, slope, etc.) and what data source did you use to guide the estimates	Channel data was developed from 2012 Kansas River survey data by NWK and KWO. Channel design modification editor was used in HEC-RAS in between locations where survey data was available. Channel design modification editor geometry consisted of varying bottom widths, according to the cross-section, with a 25' channel depth, 3H:1V side slopes, and 0.03 Manning's n-values. Channel bottoms were adjusted to slopes interpolated from surveyed data cross-sections. See screenshots at right.
5	Other sources of data used	Overbank n-values were developed from aerial imagery from the Bing maps ESRI basemap service. Inline structure information was provided by Tom Schrempp of WaterOne. The dimensions of the Water One weir were estimated from the plan set. In general the weir is 55-ft in width (as the water flows over it) with an estimated weir coefficient of 2.6, and an overall wier length of 1275-ft, with a 410-ft wide low flow in the center @ elev 736, and a 75-ft lower flow on the right @ elev 735 with a 15-ft wide fish passage @ elev 732.
6	Other assumptions and notes	Channel n-values chosen were 0.035 for upstream of the water one weir, and 0.03 for downstream of the water one weir. In general this slightly over estimates the stage at DeSoto, the mid-flows are off by about a foot too high, but the highest flows are matched. At the KCK gage, the low to mid stages for the most part cluster around +/-1-ft, but at the highest flows are too low... it was assumed that the boundary condition on the MoRiver will have an inpact on this stage, probably is too low in the current model, so when that is corrected, it will perhaps correct this difference.

HEC-RAS Review Checklist

Tributary: Kansas River
MoRiver RM: 367.45

Review #1: Cross Section Layout

Designer/Date: Adam Jones/ 27 February 2
Reviewer/Date: Jean Hilger/ 07 Mar 2013
Backcheck/Date: Adam Jones/19 March 2013

	Task	Designer notes	Reviewer	Backcheck
1	Cross section spacing less than 1/2 mile (less than approx 2600 feet)?	Generally, yes. Approximately a half-dozen XS's have spacing of about 3000 feet or more.	Ok.	
2	Cross sections perpendicular to channel flow and perpendicular to floodplain flow?	Generally, yes. A few XS's were cut in line with bridge alignments with skew across the river. The widths of the channel were adjusted with ineffective flow areas.	I see why you did this, but I think we should remove any cross section that is drawn across the river at a skew. Esp those in the KC Metro. ie. 0.31, 0.90, 1.47, 4.97, 5.89, 7.24 etc. The whole theory behind why we were cutting 3 cross sections @ bridges was to accout for the constriction caused by the roadway embankment in the floodplain, which does not apply in any of these cases.	Revised.
3	What is the largest flow of record on the gage? Are the cross sections high enough to contain more than this?	Peak measured at Topeka (U/S of DeSoto) was 469k cfs. Since 1980, the approximate conclusion of dam construction along the Kansas River, the maximum measured flow at DeSoto was 170k cfs.	Ok.	
4	Is the cross section depth reasonably uniform on all cross sections? (meaning…. highest elev minus thalweg…. ie. just looking for this to be around the same, no crazy outliers)	Yes. A few anomalies at bridge abutments and other locations.	Ran a steady flow of the 469k and 170k thorugh the model and some of the cross sections were not high enough. Check this out and decide if it's worth the effort to extend them to higher ground.	Revised widths on XS's 10.15, 11.05, 12.16, 13.05, 20.79, 26.68, 27, 27.44, 27.73, 27.99, 28.13, 28.24, and 30.01
5	Is the most upstream cross section drawn through the gage location?	Yes. DeSoto, Kansas is the gauge location and upstream-most XS (30.42)	Ok.	
6	Cross sections at all road/bridge constrictions? Three cross sections - one through the centerline and one on either side outside the influence of embankment elevation.	Yes.	Ok.	
7	Cross sections at any major geographical constriction? ie. narrow floodplain?	Yes. A large channel bend and near the WaterOne weir near I-435 in the Kansas City metro (XS 14.78)	Ok.	
8	Last cross section is cut as close to the confluence as possible without crossing Missouri River cross sections.	Yes.	Ok.	

HEC-RAS Review Checklist

Tributary: Kansas River

MoRiver RM: 367.45

9	Is the river centerline reasonable?	Yes. A little coarse, but remains within the watercourse.	Ok.	
	Other Comments:		Another big geometry note about bridges: x-sec 9.27 should be drawn along the roadway centerline and the upstream and downstream x-sections should be adjusted accordingly so they outside of the extents of the roadway embankment.	XS 9.27 was realigned and XS's upstream and downstream were relocated well away from the roadway embankment
			Same w/ x-sec 14.19, it should not be drawn through the embankment.	XS 14.90 was realigned.

Review #2: Geometry and Unsteady Run

Designer/Date: Adam Jones/ 27 February 2

Reviewer/Date: Jean Hilger/ 07 Mar 2013

Backcheck/Date: Adam Jones/19 March 2013

	Task	Designer notes	Reviewer	Backcheck
1	Unsteady model runs without error.	Yes.	Ok.	
2	How do HTab Param (geometry property) look? Points set to 100? Reasonable increment?	Look good. Lower end is confined to within levee channel with fewer than 100 points.	Please make the increment 0.5-ft on all. That's what we've done on the rest of the tributaries. On the levee bounded cross sections, it probably would be best to exent the points to above the top of cross section by a few feet. See Flood of Record steady flow run and extend to above that.	Revised to 0.5 to 0.8 for a 5x multiplier of 2011 flows (about 400k). Each XS should be capable of passing the 469k flow with reasonable accuracy. HEC-GeoRAS mapping may require the widening of some XS's.
	How HT tables (calculated in pre-processor) look? Any sudden jumps or discontinuities?	Generally good. A couple of XS's cut on bridge skews hae weird jumps.	Ok.	
3	Does the time period chosen for unsteady run include low flows, high flows, and the range in between?	Based upon period of record data for the DeSoto gauge, yes. In 2011, a flow of 80k cfs was measured.	Ok.	
4	Upstream boundary condition properly input?	Yup. DeSoto gauge has data from the 2008 water year.	Ok.	

HEC-RAS Review Checklist

Tributary: **Kansas River**

MoRiver RM: **367.45**

5	Downstream boundary condition properly input?	For the purposes of the initial development, a normal depth B/C was set. Model was inserted into larger MO River model, negating need for D/S B/C.	Would a normal depth of 0.0002 (2.0E-04) be more appropriate than 1.19E-02?	Selected 1.9x10^-4 assuming that the MO River slope is about 1 foot per mile. This washes out when inserted into the MO River model - was just used for model assembly.
6	Bed data captured by surface? If no, was the method used to estimate bed data reasonable and produce reasonable results at low flows?	Partially. Where available, bathymetry data was used. In between, either interpolated XS's or XS's from the design modification editor within HEC-RAS were used. These likely overestimate water conveyance area, but results seem to represent measured data well.	Probably not the best assumption for low flows, a more trapezoidal or triangular shape for the channel bed would have been better.	Revised to triangular that matches for the most part surveyed cross section shapes upstream and downstream.
7	Does the thalweg profile look reasonable?	Flat due to interpolated channel data, but generally, yes. Thalweg was interpolated from Kansas River survey in 2012.	Ok.	
8	Overbank flowpaths reasonable?	Yes. A little jagged, but indicate overbank flow paths.	Oops. Looks like the flowpath missed x-sec 14.47 and 14.9 ROB reach length is WAY too long.	Revised in GIS. Measured by hand and inserted into HEC-RAS.
9	Are the bank stations for the channel reasonable? Reasonably consistant upstream and downstream? (No huge changes in width and elevations.)	Yes. Widen and higher at XS's cut at bridges.	Should this be the case or should the bank stations stay uniform thorugh the bridge sections? Also, 12.16 bank sta too high, 15.36 & 15.65 bank sta too low, 26.21 too wide (1,500-ft seems more approproate b/c this is at least where the trees start).	Revised 12.16, 15.36, and 15.65. XS 26.21 looks okay to me. Set bank stations according to aerial photography and topography.
10	Are roughness coefficients reasonable?	Yes.	What's up with the varriation @ 18.12 & 18.70? Also, on all the other tribs we set n-values based on the landuse. 0.045 for crops, 0.07-0.1 for trees, 0.1 for urban. The methodolgy we followed was where possible to pick a single n-value that described most (80%) of the floodplain, and if necessary break into no more than 3 sections. This should be an hour task at most.	XS's 18.12 and 18.70 were anomalies of the channel design editor. It sets n-values over the top of whatever you've set in your geometry. These XS's were revised. N-values were revised.
11	Does the HEC-RAS Water Surface Profile look reasonable?	A few dips and jumps along portion upstream of WtaerOne weir to DeSoto.	Ok.	
12	Is the water surface profile contained within the cross sections?	Yes.	Not for the flood of record, which we need to be prepared to run. Check x-sections 28.24 - 26.68, 12,16, 11.05, 10.15 - all need to be higher.	XS's were extended to high banks, and IFA's were added as applicable.

HEC-RAS Review Checklist

Tributary: **Kansas River**

MoRiver RM: **367.45**

13	Is there divided flow in any cross-sections? This is okay as long as it makes physical sense.	There is an island that splits the channel in one section, but does not significantly impact the result.	X-sec 13.05 - 11.05 there is a RR in the right bank that probably is a barrier to KS river flows. Treat with levee points and/or ineffective?	IFA's were added. I believe that water would permeate the ballast, so a levee point would not be an accurate representation of the flow.
14	Does the calculated rating curve at the gage cross section come reasobably close to replicating the observed rating curve?	Yes.	Looks pretty darn good @ DeSoto!	:-)
15	Ineffective flows or levee points used appropriately?	Generally, yes. Used in a couple of locations due to bridge skew alignment of XS. These were used to implement approximately appropriate top widths and channel areas for those XS's.	Like the combo use of levee pts and perm ineffetive for the holes in the overbank. -- Need ineffective flows for bridges @ 14.99, 19.70, 9.27, upstream and downstream.	Revised.
16	Are the velocities consistent through the reach? Are the velocities reasonable? Velocities that are much higher or lower than typical should be flagged for closer inspection.	Yes. XS 25.69 appears to have high velocity, but is a narrow section downstream of a wider upstream section.	Ok.	
17	Is Froude's number equal to 1 at any of the cross sections? If yes, is this true to reality? If not it could be a red flag that there is something wrong with the model.	Highest observed value was 0.49 at XS 25.69	Ok.	
18	How does the thalweg elevation at the downstream end compare to other sources of data? 1999 hydrosurvey data at the mouth of each tributary, and low stages on the Missouri River at the mouth.	Data was used from 2012 survey for D/S XS.	Ok.	

Other Comments:		Perm. Ineffective flows. We use at quite a few locaitons. How does this impact calculations? What does it mean and are we using it appropriately?	Mostly using permanent IFA's for overbank ponds/quarries/lakes that are already filled by water or that will not represent an effective conveyance path for overbank flows. Water may be displaced during a flood, but these areas will not represent an active and continuous downstream path for
		I'm wondering if the inline structure dimensions need to be on the skew rather than projected to perpendicular. At low flows the water probably makes the turn and the flow coming over is equal to the weir eqn off of the true opening. Thoughts?	Revised. Not perfect, but closer to reality. I generally agree with the comment. Permanent structures redirect flow perpendicular to their orientation. Thus, flow is reoriented according to the weir, and should be represented with a XS oriented in a similar manner. This is probably a better estimate for low & mid flows, may eggagerate high flows.

HEC-RAS Review Checklist

Tributary: **Grand River**

MoRiver RM: **250**

Model Data/Major Assumptions

	Item	
1	Horizontal Projection	NAD_1983_UTM_Zone_15N
2	Vertical Datum	NAVD88, Foot_US
3	Surface(s) used to cut cross sections	Chariton and Carroll county Lidar, flown for the USACE during the winter of 2006/2007.
4	Channel data - if estimated, what were the dimensions chosen (width, side slopes, invert, slope, etc.) and what data source did you use to guide the estimates	Lidar data did not include the channel bottom because the method of collection does not reach underwater, especially in murky rivers like the ones in northwest Missouri. The flat surface, representative of the WSE, that shows up at the Sumner gage cross section (34.87) is elev 636.35-ft, which correspons to a gage stage of 5.04-ft. Based on USGS measured flow areas this is approx 256 sq-ft. (Which would be a trapezoidal shape 170-ft wide by 3-ft deep.) The bank full flow area is approx 9,000 sq-ft, 256 sq-ft is less than 3% of this area, and was therefore considered insignificant to the conveyance of the channel. Did not modify cross sections for this missing area. Downstream of cross section 7.10, however, it was obvious that due to backwater from the Missouri River there was significantly more area missing from the cross section because it was covered by water during the Lidar collection. The 1999 Hydrosurvey was used to estimate the missing channel shape at the two downstream most cross sections (0.68 and 1.80) and a straight line approximation was used to estimate the invert for the cross sections between 7.10 and 1.80. The shape missing was assumed to be a triangle with the deeper portion on the outside of the bend.
5	Other sources of data used	Gage data from USGS gage 06902000 near Sumner, MO, 1999 Hydrosurvey, 2011 Bing Maps were used to set prelimineary n-values.
6	Other assumptions and notes	There have been some levee breaches and setbacks in the area since the lidar was flown (floods in 2007, 2008, and 2010). Two locations impact cross section geomoetry: 24.92 & 11.67. The current levee location is at the edge of the cross section, even though the geometry shows the levee closer to the channel. Did not update the geometry.

HEC-RAS Review Checklist

Tributary: Grand River
MoRiver RM: 250

Review #1: Cross Section Layout

Designer/Date: Jean Hilger / 18 Mar 2013
Reviewer/Date: Adam Jones/19 March 2013
Backcheck/Date: Jean Reed / 30 May 2013

	Task	Designer notes	Reviewer	Backcheck
1	Cross section spacing less than 1/2 mile (less than approx 2600 feet)?		Stream centerline spacing varies greatly - from 2000ish feet to 14000+	Not ideal, but the model was so far progressed before we came up with this criteria that it was not worth the effort to go back and cut cross sections more often. There is only on x-section that has 14,000-ft reach length (and it is on an awkward bend in the river), the rest are no more than 9,000-ft and on average, just less than 1 mile.
2	Cross sections perpendicular to channel flow and perpendicular to floodplain flow?		Yes.	Ok
3	What is the largest flow of record on the gage? Are the cross sections high enough to contain more than this?	The 1993 flow was the second highest at 166,000-cfs, The higest flow of record is 180,000-cfs in 1947.	Ran at 2x the submittal flows (~90k cfs) and usual errors, but appeared capable of passing. For flows of record magnitude, lack of volume behind levees	(because no storage areas exist in the model as of yet) Ok
4	Is the cross section depth reasonably uniform on all cross sections? (meaning…. highest elev minus thalweg…. ie. just looking for this to be around the same, no crazy outliers)		Depths are very consistent - mid-20's at bottom end and low-to mid-30's the rest of the way.	Ok
5	Is the most upstream cross section drawn through the gage location?		Yes. Sumner.	Ok
6	Cross sections at all road/bridge constrictions? Three cross sections - one through the centerline and one on either side outside the influence of embankment elevation.	Only did one through the centerline of each bridge to capture the roadway constriction.	Verified designer notes.	Ok
7	Cross sections at any major geographical constriction? ie. narrow floodplain?		Bridges and leveed reaches are represented with narrow XS's.	Ok
8	Last cross section is cut as close to the confluence as possible without crossing Missouri River cross sections.		Yes.	Ok
9	Is the river centerline reasonable?		Aerial imagery indicates that river centerline follows river geometry.	Ok

HEC-RAS Review Checklist

Tributary: Grand River
MoRiver RM: 250

Review #2: Geometry and Unsteady Run

Designer/Date: Jean Hilger / 18 Mar 2013
Reviewer/Date: Adam Jones/20 Mar 2013
Backcheck/Date: Jean Reed / 30 May 2013

	Task	Designer notes	Reviewer	Backcheck
1	Unsteady model runs without error.	Just a few erros, all are smaller than 0.01, all are at timesteps where the WS profile has reached the top of the levee	See at right. Verified designer notes.	Ok
2	How do HTab Param (geometry property) look? Points set to 100? Reasonable increment?		Htab parameters frequently exceed the XS height significantly. Model runs stably, so no need to reduce.	Just because the Htab cacluates above the XS height, doesn't mean the WSE reaches that high. Most of these are due to cross sections being clipped at levees (Grand in this stretch is heavily leveed)
	How HT tables (calculated in pre-processor) look? Any sudden jumps or discontinuities?		Tables reflect geometry.	Ok
3	Does the time period chosen for unsteady run include low flows, high flows, and the range in between?		Doesn't hit highest peaks, but includes good variety, and near-highest in recent history.	Ok
4	Upstream boundary condition properly input?		Flow hydrograph at Sumner, MO.	Ok
5	Downstream boundary condition properly input?	Daily stage data from the main model @ the MoRiver cross section just d/s of the Grand. Think that this is why the back side of the hydrograph does not match, that if the model was calibrated correctly and at a smaller timestep, the impact from MoRiver backwater would probably raise the stage on the decending limb.	Yes.	Ok
6	Bed data captured by surface? If no, was the method used to estimate bed data reasonable and produce reasonable results at low flows?	Explanation in the top section for how this was estimated.	Yes	Ok
7	Does the thalweg profile look reasonable?		Yes	Ok
8	Overbank flowpaths reasonable?		Yes	Ok

HEC-RAS Review Checklist

Tributary: Grand River

MoRiver RM: 250

9	Are the bank stations for the channel reasonable? Reasonably consistant upstream and downstream? (No huge changes in width and elevations.)	Set the bank stations just above the water surface profile created by approx 4-5,000-cfs flow. This corresponds to the annual mean flow calculated by the USGS for the life of the gage (1925-2012). The idea was, if this is the mean flow in the Grand River, the vegitation line probably corresponds somewhat to this flow.	Looks good. Appears to pay dividends on comparison against measured data at Sumner.	Ok
10	Are roughness coefficients reasonable?		Seems a little low for main channel. Aerial imagery masks shoals because of flooding. Excerpted from HEC-RAS ref. manual at right.	0.03 is in the middle of the clean channel range. At high flows, the shoals and pools probably offer almost no resistance. The Grand is one of the largest tributaries to the MoRiver, I would think it would have a lower n-value than other smaller tributaries. Leave at 0.03 because this gives good results.
11	Does the HEC-RAS Water Surface Profile look reasonable?		Yes.	Ok.
12	Is the water surface profile contained within the cross sections?		Generally, yes. Lateral structures remove water from system.	Will add storage areas later.
13	Is there divided flow in any cross-sections? This is okay as long as it makes physical sense.		There appear to be several lateral creeks/streams along the overbanks. Most have IFA's; some don't: 27.27, 24.92, 20.89, 18.04, 16.16, 13.10,	Added permanent ineffectve areas for these.
14	Does the calculated rating curve at the gage cross section come reasobably close to replicating the observed rating curve?		Replicates rating curve pretty well. See at right. Filling and emptying of wide floodplain likely contributes to divergence from computed curve at higher stages.	Ok.
15	Ineffective flows or levee points used appropriately?	On the upstream most levees (x-secs 30.01 - 34.87), set these @ the high ground- t/levee shown in the cross section, then adjusted them so they followed the water surface profile. Ended up lowering all by 2.5-ft because this helped match the bend in the USGS rating curve better.	Appear to generally activate well for extreme events on XS's attached to same leveed area. Well done!	Ok!
16	Are the velocities consistent through the reach? Are the velocities reasonable? Velocities that are much higher or lower than typical should be flagged for closer inspection.		Yes.	Ok

HEC-RAS Review Checklist

Tributary: **Grand River**

MoRiver RM: **250**

17	Is Froude's number equal to 1 at any of the cross sections? If yes, is this true to reality? If not it could be a red flag that there is something wrong with the model.		Froude numbers are all well bnelow 1.	Ok
18	How does the thalweg elevation at the downstream end compare to other sources of data? 1999 hydrosurvey data at the mouth of each tributary, and low stages on the Missouri River at the mouth.	The 1999 Hydrosruvey places the thalweg of the downstream most cross section (0.068) at approx elev 605-606.	Fine	Ok
	Other Comments:	A note about the ineffective flows for x-sec 30.01-34.87. They are 3.5-ft above the levees rather than the typical 1-ft. Without this, there is an incredible amount of flow area in the overbanks that chops off the top of the stage hydrograph.	Fine	Ok

HEC-RAS Review Checklist

Tributary: Chariton River

MoRiver RM: 239

Model Data/Major Assumptions

	Item	
1	Horizontal Projection	NAD_1983_UTM_Zone_15N
2	Vertical Datum	NAVD88, Foot_US
3	Surface(s) used to cut cross sections	Chariton and Carroll county Lidar, flown for the USACE during the winter of 2006/2007.
4	Channel data - if estimated, what were the dimensions chosen (width, side slopes, invert, slope, etc.) and what data source did you use to guide the estimates	Revision 2/11/2014 - Lowered the thalweg of the Chariton at the confluence because of instablitities during the POR run with very low flows in the MoRiver during 1930s. Thalweg elevation at the last cross section (0.33) before lowering was 603.83, since CRP is 609.8 this was only -6 CRP. Lowered to 589 (or approx -21 CRP), which is approximately equal to the pool elevations in the MoRiver at this location, and gradually sloped up to tie back in with the Chariton slope over the lower 5 x-sections. The thalweg elevation at the gage cross section 19.63 as cut from the lidar was 632.98. The datum at the Prairie Hill gage is 632.10 (NAVD88), which means the bottom of the channel corresponds to a gage stage of 0.88-ft. The USGS measured data shows that the flow area in the Chariton at the gage location is zero below stages of 1-ft. Therefore it was assumed that the Chariton was dry when the Lidar was flown and no channel estimation was needed. The three most downstream cross sections had unusually high elevations compared with the rest of the thalweg profile (presumably from Missouri River backwater), so a channel similar in shape and size to the next upstream cross sections was manually added and a straight line approximation coninuing the upstream slope of ~ 0.0002 ft/ft to the confluence.
5	Other sources of data used	Gage data from USGS gage 06905500 near Prairie Hill, MO, 1999 Hydrosurvey, 2012 NAIP aerial photos were used to set preliminary n-values.
6	Other assumptions and notes	

HEC-RAS Review Checklist

Tributary: Chariton River
MoRiver RM: 239

Review #1: Cross Section Layout

Designer/Date: Jean Hilger / 20 Mar 2013
Reviewer/Date: Adam Jones/29 March 2013
Backcheck/Date: Jean Reed / 30 May 2013

	Task	Designer notes	Reviewer	Backcheck
1	Cross section spacing less than 1/2 mile (less than approx 2600 feet)?	No. Almost every cross section is spaced 3000 - 5000 feet. I think this is why we get all the little errors when the model is run, the timestep is not suitable for this cross section spacing. However, since the max error is 0.12 and we are not calibrating to stage on the Chariton (only care about the d/s hydrograph), chose not to update.	Agreed.	Ok
2	Cross sections perpendicular to channel flow and perpendicular to floodplain flow?		Look good.	Ok
3	What is the largest flow of record on the gage? Are the cross sections high enough to contain more than this?	38,400-cfs in July 2008. Yes! This event is included in the unsteady period used to calibrate the model.	Agreed.	Ok
4	Is the cross section depth reasonably uniform on all cross sections? (meaning…. highest elev minus thalweg…. ie. just looking for this to be around the same, no crazy outliers)		Very uniform. All depths are in the 20-22 foot range.	Ok
5	Is the most upstream cross section drawn through the gage location?		Yes. Drawn just downstream of the Missouri Highway 129 bridge in Prairie Hill, MO.	Ok
6	Cross sections at all road/bridge constrictions? Three cross sections - one through the centerline and one on either side outside the influence of embankment elevation.		No. County Road W is missing XS's just SW of MO Hwy 129; MicDonald Road, the next bridge SW; Dooley Ford Rd; State Hwy VV	These bridges/roadways were small and did not appear to offer a major constriction to the Chariton River. Therefore cross sections were not included for these bridges.
7	Cross sections at any major geographical constriction? ie. narrow floodplain?		Good. Narrow through leveed floodplain portion near Missouri River.	Ok
8	Last cross section is cut as close to the confluence as possible without crossing Missouri River cross sections.		XS 0.33 looks good.	Ok
9	Is the river centerline reasonable?		Follows aerial photography quite nicely.	Ok

HEC-RAS Review Checklist

Tributary: Chariton River
MoRiver RM: 239

Review #2: Geometry and Unsteady Run

Designer/Date: Jean Hilger / 20 Mar 2013
Reviewer/Date: Adam Jones/29 March 2013
Backcheck/Date: Jean Reed / 30 May 2013

	Task	Designer notes	Reviewer	Backcheck
1	Unsteady model runs without error.	Few erros but none are larger than 0.1	I got one greater than 0.1. XS 13.02 appears to be either a source or a source or in the viciinty of a problem. Seven of the errors at right originate with this XS.	Bumped the ineffective flow up @ this cross section, get's rid of the 0.1 error. Other errors still remain .
2	How do HTab Param (geometry property) look? Points set to 100? Reasonable increment?		All 0.5 feet with 100 values per XS.	Ok
	How HT tables (calculated in pre-processor) look? Any sudden jumps or discontinuities?		Look good with a review of the XS's.	Ok
3	Does the time period chosen for unsteady run include low flows, high flows, and the range in between?	Peak of 38,300-cfs, min of 641-cfs.	See screenshot at right. Looks like an excellent selection to me!	Ok
4	Upstream boundary condition properly input?		U/S hydrograph from measured USGS data.	Ok
5	Downstream boundary condition properly input?		Stage hydrograph from modeling output along the Missouri River for the same time period.	Ok
6	Bed data captured by surface? If no, was the method used to estimate bed data reasonable and produce reasonable results at low flows?		See above. Appears LIDAR was flow during very low water, which provided excellent channel bed resolution.	Ok
7	Does the thalweg profile look reasonable?		Stable slope for the entire reach.	Ok
8	Overbank flowpaths reasonable?		Consistent with channel reach lengths, accounting for bends in the river.	Ok
9	Are the bank stations for the channel reasonable? Reasonably consistant upstream and downstream? (No huge changes in width and elevations.)		Yes.	Ok

HEC-RAS Review Checklist

Tributary: Chariton River

MoRiver RM: 239

10	Are roughness coefficients reasonable?	Used 0.028 for the channel. Is a little lower than expected, but still reasonable, and best matches the rating curve @ Prairie Hill.	River appears to have been manually straightened through a sandy river bottom, but is relatively straight with few shoals and little in-channel veg or grade control. 0.028 seems a little low, but very reasonable. Prairie Hill RC comparison seems to indicate there may be more room for lowering n-values.	0.028 was as low as I was willing to go. Any lower than that seems outside of the reasonable range.
11	Does the HEC-RAS Water Surface Profile look reasonable?		Yeah. It's mentioned below in No. 15, but may need to work on coordinating levee overtoppings a little better if not using lateral structure/storage area configuration if this set up doesn't work.	Spent a considerable ammount of time trying and ierating for the perfect combination of levee heights + ineffective elevations that was consistant and matched the rating curve @ Prairie Hill. This was the best/most stable version I could come up with. It has weaknesses, but the ammount of time to improve upon these I do not think would be profitable.
12	Is the water surface profile contained within the cross sections?		Yes. Contained throughout the model.	Ok
13	Is there divided flow in any cross-sections? This is okay as long as it makes physical sense.		None that hampers the modeling output.	Ok
14	Does the calculated rating curve at the gage cross section come reasobably close to replicating the observed rating curve?		Near Prairie Hill, the match is excellent - most likely because the measured data from the USGS gauge is used as the upstream B/C. The downstream XS has no measured rating curve, and would be highly variable due to Missouri River backwater.	Only the flow is used as the u/s bounary condition. The stage is entirely dependent on model geometry, and is a measure of how well the model is calbrated.
15	Ineffective flows or levee points used appropriately?	Not really. Tried several versions of arraging the levee points and ineffectives so that they made logical sense from upstream to downstream, but ended up in the end worse of and with a wackier hydrograph than when they weren't consistant. Add this to the 80% list. Not worth the time to fix now, but if we find issue with the Chariton input into the mainstem this is something we could consider re-looking at.	Noticed that on July 28th @ 0600, that XS 17.75 is inundated bluff-to-bluff, while the upstream and downstream XS's are dry on one or both overbanks. Seems like it should be consistent or at least relatively consistent within a protected area.	Same - tried to get rid of this inconsistancy but it causes other inconsistancies to crop up. Do not think storage areas would be worth the effort for this area.

HEC-RAS Review Checklist

Tributary: **Chariton River**

MoRiver RM: **239**

16	Are the velocities consistent through the reach? Are the velocities reasonable? Velocities that are much higher or lower than typical should be flagged for closer inspection.		Velocities are kind of variable. Because of IFA's, levees, and different floodplain features, there are a couple of drops by approximately one-half. See at right below.	Not much we can do about this. Without fixing the said issues.
17	Is Froude's number equal to 1 at any of the cross sections? If yes, is this true to reality? If not it could be a red flag that there is something wrong with the model.		Froude numbers look good.	Ok.
18	How does the thalweg elevation at the downstream end compare to other sources of data? 1999 hydrosurvey data at the mouth of each tributary, and low stages on the Missouri River at the mouth.	The elevations of the 1999 hydrosurvey at the confluence of the the Chariton are in the 604-605 range. The downstream most x-sec (0.33) has a thalweg of 603.83, which is comparable.	See Section 4 above in Model Data/Major Assumptions.	Ok.
	Other Comments:		In looking at the geometry editor for the 28 July 2008 event, it is a little surprising that the downstream XS's remain entirely in-bank, while virtually every XS is flooded to some depth over most overbanks. The Missouri River must have been relatively low at this time, because this seems unlikely otherwise.	Seems reasonable to me to assume the bottom few cross sections are incised from continuous MoRiver backwater.

HEC-RAS Review Checklist

Tributary: **Blackwater**

MoRiver RM: **202 (Trib to Lamine)**

Model Data/Major Assumptions

	Item	
1	Horizontal Projection	NAD_1983_UTM_Zone_15N
2	Vertical Datum	NAVD88
3	Surface(s) used to cut cross sections	USGS NED 2011, 1/3 arc
4	Channel data - if estimated, what were the dimensions chosen (width, side slopes, invert, slope, etc.) and what data source did you use to guide the estimates	The NED2011 data matched USGS data well for the upstream portion of the Blackwater. The inverts were also very comparable 600.646 USGS to 600.6 NED2011. For this reason, the channel was left as is from US-XS 9.80, at which point there appears to be a bust in the DEM. However, modifications were made to the channel from XS 9.80 at slope 0.00019, which is in line with the upstream portion of the channel and which also meets the estimated downstream invert of ~574 feet. The estimated downstream invert was predicted from the Lamine River near the confluence of the Blackwater and Lamine. Other parameters related to the modified channel from XS 9.80-DS include: bw: 40 feet and sideslope: 4. Bank stations were set at approximately 7 feet higher than the invert which seems to reflect general flow conditions based on gage heights. Since the Blackwater flows into the Lamine, the BC was not readily available and was instead developed by running the Lamine model for September 2010 (time of max flow on Blackwater) and using the stage hydrograph at Lamine XS 9.28 as the Blackwater downstream boundary condition. Spreadsheet "XS approximation based on USGS data_20121119" was used to determine channel shape and additional details can be found in that spreadsheet, which utilizes data taken directly from the USGS web site.
5	Other sources of data used	USGS website rating curve.
6	Other assumptions and notes	Overbank Manning's n values to reflect those areas with trees (0.07-0.1).

HEC-RAS Review Checklist

Tributary: Blackwater
MoRiver RM: 202 (Trib to Lamine)

Review #1: Cross Section Layout

Designer/Date: Lisa Stahr/ 21 Dec 2012
Reviewer/Date: Jean Hilger/ 07 Jan 2013
Backcheck/Date: Lisa Stahr/16 Jan 2013

	Task	Reviewer	Backcheck
1	Cross section spacing less than 1/2 mile (less than approx 2600 feet)?	Yes.	X
2	Cross sections perpendicular to channel flow and perpendicular to floodplain flow?	Yes.	X
3	What is the largest flow of record on the gage? Are the cross sections high enough to contain more than this?	54,000 in 1929, cross sections are big enough regardless.	X
4	Is the cross section depth reasonably uniform on all cross sections?	Yes.	X
5	Is the most upstream cross section drawn through the gage location?	Yes. Keep in mind what we learned on the Moreau. Do we need to draw a cross section through the bridge centerline? Maybe or maybe not. I think it will depend on closer inspection of the measured data and the rating curve.	Did not additional XS through bridge CL.
6	Cross sections at all road/bridge constrictions? Three cross sections - one through the centerline and one on either side outside the influence of embankment elevation.	Yes. Did not include 2, but this is because they are small. No roadway embankments.	X
7	Cross sections at any major geographical constriction? ie. narrow floodplain?	Fine.	X
8	Last cross section is cut as close to the confluence as possible without crossing Missouri River cross sections.	Lamine in this case rather than the MoRiver. Best practice is not to have crossing cross sections, so will need to either mod the Blackwater or the Lamine cross sections in this area because right now they overlap. Looks like even with the highest flow the blackwater doesn't get as high as that Lamine cross section, so could just trim all your blackwater x-sections.	XS were not cut. Will leave this up to the USACE as modeling continues.
9	Is the river centerline reasonable?	Yes. If you could snap the Blackwater centerline at the downstream end to the Lamine in GIS, that will make creating a junction in HEC-RAS easier when we merge the two.	X

HEC-RAS Review Checklist

Tributary: Blackwater
MoRiver RM: 202 (Trib to Lamine)

Review #2: Geometry and Unsteady Run

Designer/Date: Lisa Stahr/ 23 Jan 2013
Reviewer/Date: Jean Hilger/ 29 Jan 2013
Backcheck/Date: Lisa Stahr/ 30 Jan 2013

	Task	Reviewer	Backcheck
1	Unsteady model runs without error.	Yes	X
2	How do HTab Param (geometry property) look? Points set to 100? Reasonable increment?	Yes - 100 points, incriment of 0.5-ft	X
	How HT tables (calculated in pre-processor) look? Any sudden jumps or discontinuities?	Fine.	X
3	Does the time period chosen for unsteady run include low flows, high flows, and the range in between?	No. Ran range of 64-cfs to 16,500-cfs in Sept of 2010, but please run the Max Flow plan with a multiplier of 3.3 to test the model's capeability to run the max flow of record (54,000-cfs). The max flow you tested, 22,000-cfs, must be the max flow that they have been able to measure flow data at for the rating curve - not necessarily the flood of record.	Done.
4	Upstream boundary condition properly input?	Yes. Blackwater @ Blue Lick.	X
5	Downstream boundary condition properly input?	Yes. Stage data from the Lamine model.	X
6	Bed data captured by surface? If no, was the method used to estimate bed data reasonable and produce reasonable results at low flows?	Very good. Agree that the channel in the DEM u/s of RM 9 is reasonable, but d/s of there is bogus so your only modifications to geometry were d/s.	X
7	Does the thalweg profile look reasonable?	Yes.	X
8	Overbank flowpaths reasonable?	Yes	X

HEC-RAS Review Checklist

Tributary: **Blackwater**

MoRiver RM: **202 (Trib to Lamine)**

9	Are the bank stations for the channel reasonable? Reasonably consistant upstream and downstream? (No huge changes in width and elevations.)	Yes, definitly	X
10	Are roughness coefficients reasonable?	Yes. This is just about the right ammount of detail.	X
11	Does the HEC-RAS Water Surface Profile look reasonable?	Yes.	X
12	Is the water surface profile contained within the cross sections?	Yes.	X
13	Is there divided flow in any cross-sections? This is okay as long as it makes physical sense.	Used ineffective flows.	X
14	Does the calculated rating curve at the gage cross section come reasobably close to replicating the observed rating curve?	Yes. In general, slightly too high at the lows and too low at the highs. This could indicate that we were missing a little bit of channel data in the upstream cross sections, but not a big enough concern to revise the model now. Will route fine I think.	X
15	Ineffective flows or levee points used appropriately?	Yes.	X
16	Are the velocities consistent through the reach? Are the velocities reasonable? Velocities that are much higher or lower than typical should be flagged for closer inspection.	Yes, at the peak in Sept 2010 the highest velocity is about 8 ft/s. This seems very reasonable.	X
17	Is Freud's number equal to 1 at any of the cross sections? If yes, is this true to reality? If not it could be a red flag that there is something wrong with the model.	No. Ok.	X
18	How does the thalweg elevation at the downstream end compare to other sources of data? 1999 hydrosurvey data at the mouth of each tributary, and low stages on the Missouri River at the mouth.	Cross section thalwag is 574.0 in the most d/s blackwater x-section. Confluence with Lamie just d/s of x-sec 9.87 which has a thalweg of 573.82. This matches up fine.	X

Other Comments:

HEC-RAS Review Checklist

Tributary: **Lamine**
MoRiver RM: **202**

Model Data/Major Assumptions

	Item	
1	Horizontal Projection	NAD_1983_UTM_Zone_15N (Foot US)
2	Vertical Datum	NAVD88
3	Surface(s) used to cut cross sections	United States Geological Survey (USGS) National Elevation Dataset (NED) 2011, 1/3 arc-second
4	Channel data - if estimated, what were the dimensions chosen (width, side slopes, invert, slope, etc.) and what data source did you use to guide the estimates	The USGS surface water measurments of top with vs. stage @ Otterville were used to estimate the channel bathyemtry missing from the 2011 NED. The channel "cut" into the geometry generally had these characteristics: bottom width: 90 feet, ss: 5, and channel slope of 0.00039 from US-XS 22.19 and 0.00008 from XS 22.19-DS.
5	Source of calibration data (rating curve, flow events chosen, etc.)	USGS rating curve @ Otterville
6	Other assumptions and notes	Overbank Manning's n value was modified slightly n=0.045 to 0.05 to get a better RC fit, which is justified considering the sinuosity and river characteristics.

Review #1: Cross Section Layout

Designer/Date: **Lisa Stahr/ 28 Nov 2012**
Reviewer/Date: **Jean Hilger/ 11 Dec 2012**
Backcheck/Date: **Lisa Stahr/ 21 Dec 2012**

	Task	Reviewer	Backcheck
1	Cross section spacing less than 1/2 mile (less than approx 2600 feet)?	Yes.	X

HEC-RAS Review Checklist

Tributary: **Lamine**

MoRiver RM: **202**

2	Cross sections perpendicular to channel flow and perpendicular to floodplain flow?	Don't forget to stay perpendicular to the bluff-to bluff flow as well. Also - try to avoid near 90 degree bends in cross section lines as this is pretty unrealistic and sometimes tends to skew the cross section length unreasonably. I realize that sometimes this is not possible, but if it is possible to avoid, please do so. Adjust cross sections if necessary to meet this.	XS adjusted and now perpindicular to FP flow as well.
3	What is the largest flow of record on the gage? Are the cross sections high enough to contain more than this?	84,900-cfs in May 1995. Need to extend a few. 40.57, 40.04, 39.77, 38.41, 9.93, 9.89, 9.87, 8.83. Any others?	XS modified
4	Is the cross section depth reasonably uniform on all cross sections?	Yes. For the most part.	X
5	Is the most upstream cross section drawn through the gage location?	Yes.	X
6	Cross sections at all road/bridge constrictions? Three cross sections - one through the centerline and one on either side outside the influence of embankment elevation.	Yes. The one @ 43.51 needs a centerline.	Centerline added.
7	Cross sections at any major geographical constriction? ie. narrow floodplain?	Yes	X
8	Last cross section is cut as close to the confluence as possible without crossing Missouri River cross sections.	Get a little closer.	X
9	Is the river centerline reasonable?	Yes	X

Other Comments: Keep in mind when drawing x-sections that HEC-RAS will consider the direction of flow to be perpendicular to the cross section. This is why it is better to avoid sharp bends and make sure to draw the cross section lines perpendicular to the bluff to bluff as well as to the river channel.

Review #2: Geometry and Unsteady Run

Designer/Date: **Lisa Stahr/ 21 Dec 2012**

Reviewer/Date: **Jean Hilger/ 28 Dec 2012**

Backcheck/Date: **Lisa Stahr/10 Jan 2013**

	Task	Reviewer	Backcheck
1	Unsteady model runs without error.	Yes.	x
2	How do HTab Param (geometry property) look? Points set to 100? Reasonable increment?	Incriment is 0.3 or 0.4 on most, I assume this was to improve wobbling profile. Some of the max water surface elevations are within the last imcriment - but the max that was run through the model was the period of record flow so should be ok.	Should all be set to 0.4. Yes, the purpose was in hopes of smoothing the profile out just a bit.

HEC-RAS Review Checklist

Tributary: Lamine

MoRiver RM: 202

	How HT tables (calculated in pre-processor) look? Any sudden jumps or discontinuities?	Noticed at a few cross sections conveyance goes down. ie. Cross section 49.38 @ elevation 661. Doesn't appear to be any obvious explanation for this and it doesn't appear to cause any problems. Just making a note.	x
3	Does the time period chosen for unsteady run include low flows, high flows, and the range in between?	Yes. From 80-cfs to 84,000-cfs.	x
4	Upstream boundary condition properly input?	Yes. Ran with a multiplier of 2.4 to get a flow equiv to the flow of record.	x
5	Downstream boundary condition properly input?	Used normal depth - looks like MoRiver model probably is low in this area and needs calibration. Please doument how downstream flowline was chosen in case adjustments need to be made.	Originally only ran normal depth for max flow because the BC was making the profile a little weird, which is to be expected, as I was still using the Sept08 BC. In HEC-RAS (as far as I know) there is not a way to use a multiplier with the a stage hdyrograph. The Sept08 BC could have been manipulated in the DSS to more accurately represent the stage hydrograph at max flow. However, at this point, it did not seem necessary.
6	Bed data captured by surface? If no, was the method used to estimate bed data reasonable and produce reasonable results at low flows?	Yes, seems reasonable.	x
7	Does the thalweg profile look reasonable?	Yes, is reasonable to think that the downstream portion has a flatter slope. Although, likely this change occurs at the confluence with the Blackwater.	Generally, slopes used to modify channel geometry reflect the slopw of the NED 2011 data. In some cases, it is obvious that the slope is capturing a water surface as a result of backwater (completely flat surface). When this occurs, either the slope from the upstream portion is carried through to the invert at the confluence with the Missouri, or if better data is available (hydrosurvey, etc...this information is used to determine an invert and the slope computed from that point back up to a point where there does not appear to be backwater. In this case, hydrosurvey was available and the downstream invert (and channel slope) were revised to reflect this information.
8	Overbank flowpaths reasonable?	Yes.	x
9	Are the bank stations for the channel reasonable? Reasonably consistant upstream and downstream? (No huge changes in width and elevations.)	Yes.	x
10	Are roughness coefficients reasonable?	Please assign n-values that represent the majority of landuse in each overbank.	Manning's n-values were revised based on the internet based Bing aerial maps. Adjusted very generally mostly to reflect areas of heavy trees.
11	Does the HEC-RAS Water Surface Profile look reasonable?	About the wobbles - I think this is due to differences in the height of the floodplain bench with respect to the channel. Bad DEM data, basically. If you check the cross sections where there are velcotiy spikes (of sometimes up to 15-ft/s) they're all at cross sections where the floodplain is barely wet (less than a foot of water) and the bulk of the flow is shoved into the channel. And at the surrounding cross sections there is a lot of water in the overbanks, which slows the water down and is almost acting like a backwater cross section. Since this doesn't seem to impact stability or results, probably we should just leave it.	x
12	Is the water surface profile contained within the cross sections?	Yes.	x

HEC-RAS Review Checklist

Tributary: **Lamine**

MoRiver RM: **202**

13	Is there divided flow in any cross-sections? This is okay as long as it makes physical sense.	Fine.	x
14	Does the calculated rating curve at the gage cross section come reasobably close to replicating the observed rating curve?	Yes.	x
15	Ineffective flows or levee points used appropriately?	Yes.	x
16	Are the velocities consistent through the reach? Are the velocities reasonable? Velocities that are much higher or lower than typical should be flagged for closer inspection.	Lots of spikes, for the same reasons as described under WS profile. Probably we could treat each case individually if we wanted to fix this. Tweaking n-values and adding strategic ineffective flows.	Looked into these, but ultimately did not fix the spikes. Generally, many of these are constrictions captured by road XS. Removing some of the XS across the road (which capture the embankment constriction) may smooth out the velocity profile.
17	Is Freud's number equal to 1 at any of the cross sections? If yes, is this true to reality? If not it could be a red flag that there is something wrong with the model.	Yes. This happens @ 32.04, 35.10, and 54.90 at the peak of the flood. Let's see what we can do to fix this.	Current model does not seem to have any Froude # > 1.
18	How does the thalweg elevation at the downstream end compare to other sources of data? 1999 hydrosurvey data at the mouth of each tributary, and low stages on the Missouri River at the mouth.		Based on hydrsosurvey, revised downstream invert to reflect this information at approximate elevation 565'.

Other Comments:	Why did we remove the cross sections at the I-70 bridge? Even though there is no constriction because of the interstate, it is easily the largest and most significant crossing that any of the tributaries will have.	Did not add this back in, as it is not visible in the DEM. However, there is a XS very near I-70.

HEC-RAS Review Checklist

Tributary: **Moreau**
MoRiver RM: **138**

Model Data/Major Assumptions

	Item	
1	Horizontal Projection	NAD_1983_UTM_Zone_15N
2	Vertical Datum	NAVD88
3	Surface(s) used to cut cross sections	USGS NED 2011, 1/3 arc
4	Channel data - if estimated, what were the dimensions chosen (width, side slopes, invert, slope, etc.) and what data source did you use to guide the estimates	Update to the downstream thalweg 6/25/2013. The thalweg at the downstream most x-section was lowered from 517.68 to an elevation of 515-ft, which corresponds to about 8.5-ft below CRP (approx 523.56 at the Moreau mouth) because otherwise the downstream cross section dries up when the Missouri River is low. This also corresponds better to hydrosurvey elevations of 513-517. The most downstream 2 miles of the Moreau was also altered to transition to this depth. Channel detail appeared much better for the Moreau, as a result of using the 2011 surface, however, the overbank (OB) elevations were markedly different. The data from the USGS web site (2007-2011) has the Moreau channel OB starting at approximately 580 ft, while the 2011 NED data shows the overbanks starting at 565 ft, which does not necessarily affect model results at lower flows (those below elevation 565), but does impact model results at higher flows. Once this had been discovered, additional modifications were made to the model geometry. Adjustments to the geometry before making the OB discovery include adjusting the channel to better match USGS data: bottom width: 90 feet; ss: 5; channel slope 0.00021 (to get a channel invert close to 522, which would be a value just less than the lowest stage value in the DS MO River BC stage hydrgrograph). The channel geometry changes were only made from XS 13.63 to the downstream end. No changes were made to the US XS downstream to XS 13.63 cut from the 2011 NED data, as the US invert was reasonable compared to the gage datum at 546.46. Once the XS was plotted and the model was run, the impact of the OB was evident and additional changes were made to the Moreau geometry file. The following changes are not in the geometry used to merge into the MoRiver Model. [~~Because the USGS data indicated that OB should be at elevation 580 ft and the 2011 NED showed OB at 565 ft all XS in the geometry from US to downstream were raised by 15 feet. After doing this, the channel had to be "cut" into the geometry to reflect appropriate invert elevations. This channel had dimensions: bottom width: 70 feet; ss: 4; and slope 0.00025. Spreadsheet "XS approximation based on USGS data_20121119" was used to determine channel shape. Additional details can be found in that spreadsheet, which utilizes data taken directly from the USGS web site.~~] An unadjusted OB model geometry was also created, which reflects information from the hydrosurvey at the downstream end. Generally, channel dimensions are as follows: bottom width: 70 feet; ss: l - 3, r - 3.5; and slope 0.00028.
5	Other sources of data used	USGS website rating curve.
6	Other assumptions and notes	

HEC-RAS Review Checklist

Tributary: **Moreau**
MoRiver RM: **138**

Review #1: Cross Section Layout

Designer/Date: **Lisa Stahr/ 28 Nov 2012**
Reviewer/Date: **Jean Hilger/ 11 Dec 2012**
Backcheck/Date: **Lisa Stahr/ 21 Dec 2012**

	Task	Reviewer	Backcheck
1	Cross section spacing less than 1/2 mile (less than approx 2600 feet)?	Yes.	X
2	Cross sections perpendicular to channel flow and perpendicular to floodplain flow?	Yes.	X
3	What is the largest flow of record on the gage? Are the cross sections high enough to contain more than this?	33,000-cfs in 1985 (just out of curiosity, where did you get 47,000-cfs?) Let's get on the same page with where we get this number from.	As discussed, had simply been taking the largest flow from the USGS RC.
4	Is the cross section depth reasonably uniform on all cross sections?	Yes.	X
5	Is the most upstream cross section drawn through the gage location?	Yes.	X
6	Cross sections at all road/bridge constrictions? Three cross sections - one through the centerline and one on either side outside the influence of embankment	Missing a few. Check for this and add cross sections.	XS added where requested.
7	Cross sections at any major geographical constriction? ie. narrow floodplain?	Yes.	X
8	Last cross section is cut as close to the confluence as possible without crossing Missouri River cross sections.	Yes.	X
9	Is the river centerline reasonable?	Yes.	X

Other Comments: Cut new cross sections from updated surface. Looks quite a bit different in this area, I think it will make a significant difference!

Review #2: Geometry and Unsteady Run

Designer/Date: **Lisa Stahr/ 28 Dec 2012**
Reviewer/Date: **Jean Hilger/ 04 Jan 2012**
Backcheck/Date: **Lisa Stahr/ 14 Jan 2012**

	Task	Reviewer	Backcheck
1	Unsteady model runs without error.	Yep	x
2	How do HTab Param (geometry property) look? Points set to 100? Reasonable increment?	Fine	x
	How HT tables (calculated in pre-processor) look? Any sudden jumps or discontinuities?	Fine	x
3	Does the time period chosen for unsteady run include low flows, high flows, and the range in between?	Yes. Range from 20-cfs to 30,700-cfs runs without error.	x
4	Upstream boundary condition properly input?	Yes.	x

HEC-RAS Review Checklist

Tributary: **Moreau**

MoRiver RM: **138**

5	Downstream boundary condition properly input?	Yes. Stage from the MoRiver uncalibrated model @ cross sections near mouth.	x
6	Bed data captured by surface? If no, was the method used to estimate bed data reasonable and produce reasonable results at low flows?	Is it possible that the USGS measurments were taken at the bridge abutments during high flows? The bridge abutment are @ elev 579. And in the USGS measured data it lists the location where the flow measurment was taken as 0 (@ the bridge). This would mean that the cross sections do not all need to be raised by 15-ft, and instead a cross section just needs to be cut at the CL of the bridge to match the rating curve. I find it unlikely that there is a 15-ft bust in the DEM data, especially considering this area has been recently enhanced by lidar. Also, raising the overbanks by 15-ft causes one of the largest flows this river has seen (30,000-cfs) to be almost entirely contained in the channel. This also is unlikely.	A similar issue was discovered on the Gasconade, but in the case of the Gasconade, it was clear that there was a DEM OB issue between 2009 and 2011 NED data. This discrepancy was not evident as in the case of Gasconade, but there is reason to believe that the DEM could be causing these OB on the Moreau.For this reason, two geometries were created for the Moreau. 1. The first geometry has OB adjusted +15', so that HEC-RAS model XS looks similar to USGS developed XS and so that model results, specifically the RC, matches that of the USGS RC. After doing this, the channel had to be "cut" into the geometry to reflect appropriate invert elevations. This channel had dimensions: bottom width: 70 feet; ss: 4; and slope 0.00025. (The second geometry was merged into the MoRiver model, since we are not trying to acheive accuracy in stage on the tributaires, only timing, it was decided that there is not enough evidence to make such a wholistic change to the geometry.) 2. The model run using the second geometry does not match the USGS RC well for the aforementioned reason. The OB are located in the original location, but the channel has been cut to better match the upstream (USGS data) and downstream (hydrosurvey data) inverts. Generally, channel dimensions are as follows: bottom width: 70 feet; ss: l - 3, r - 3.5; and slope 0.00028.
7	Does the thalweg profile look reasonable?	Yes. Just to document… what has been your standard method for determining what slope you use?	Slope generally follows the ground slope of the original NED data, which may change slope throughout the reach. This is part of the reason that the original geometry has been left intact within the model.
8	Overbank flowpaths reasonable?	Yes	x
9	Are the bank stations for the channel reasonable? Reasonably consistant upstream and downstream? (No huge changes in width and elevations.)	Yes	x
10	Are roughness coefficients reasonable?	0.035 for the channel and 0.045 for all overbanks. Would prefer a base n-value for each overbank that matches the bulk of the landuse. ie. cross sections 9.10 through 6.78 on the left overbank have mostly trees. N-value should be 0.07-0.1 as discussed early on in the modeling process.	x
11	Does the HEC-RAS Water Surface Profile look reasonable?	Yes. Excepting the downstream bdry. Possible indicator that thalweg should be lower?	
12	Is the water surface profile contained within the cross sections?	Yes.	x
13	Is there divided flow in any cross-sections? This is okay as long as it makes physical sense.	No.	x
14	Does the calculated rating curve at the gage cross section come reasobably close to replicating the observed rating curve?	This is very good. I can see why you made the changes you did.	Yes, for OB adjusted geometry. No, for unadjusted OB case.
15	Ineffective flows or levee points used appropriately?	None.	x

HEC-RAS Review Checklist

Tributary: **Moreau**

MoRiver RM: **138**

16	Are the velocities consistent through the reach? Are the velocities reasonable? Velocities that are much higher or lower than typical should be flagged for closer inspection.	Yes. Except pretty unreasonable @ the downstream end (05 Sept)…. 25-cfs??	Value is still higher at the downstream end (14 ft/2), but this is probably due to the fact that the very downstream has very high steep overbanks. In other words, the final XS is more constricted than those upstream of it.
17	Is Freud's number equal to 1 at any of the cross sections? If yes, is this true to reality? If not it could be a red flag that there is something wrong with the model.	Yes at the downstream end. Same location and time as the high velocities.	The value is not > 1. Value is still higher at the downstream end (0.6-0.7), but this is probably due to the fact that the very downstream has very high steep overbanks. In other words, the final XS is more constricted than those upstream of it.
18	How does the thalweg elevation at the downstream end compare to other sources of data? 1999 hydrosurvey data at the mouth of each tributary, and low stages on the Missouri River at the mouth.	Thalweg elev at the d/s x-section is 520.88, and although the lowest stage in the time period we have chosen is 522.3 (13 Aug 2008) the lowest stage at this cross section in the 2007-2011 time frame is below 520 (this happens about 3 times during a four year period). 1999 hydrosurvey showed the channel in this area ranging from 513 to 517. I will get this data to you. Consider lowering the thalweg elevation.	This was adjusted to more closely match the hydrosurvey data. Currently, the invert is approximately 518 for the unadjusted OB geometry and 521 for the OB adjusted geometry.

Other Comments:

HEC-RAS Review Checklist

Tributary: **Osage River**

MoRiver RM: **130**

Model Data/Major Assumptions

	Item	
1	Horizontal Projection	NAD_1983_UTM_Zone_15N, Foot_US
2	Vertical Datum	NAVD88
3	Surface(s) used to cut cross sections	United States Geological Survey (USGS) National Elevation Dataset (NED) 2011, 1/3 arc-second
4	Channel data - if estimated, what were the dimensions chosen (width, side slopes, invert, slope, etc.) and what data source did you use to guide the estimates	Single beam bathymetry cross sections were surveyed in August of 2007 from the mouth to the St. Thomas gage at about 1 mile spacing. Model cross sections were drawn at the bathymetry cross section locations, and in between to get approx half-mile spacing. Where these cross sections lined up, the bathymetry was merged into the cross section with the merge tool in HEC-RAS. For the intermediate cross sections the bathymetry was interpolated and then merged into the model cross sections. Bathymetry elevations are in NAVD88.
5	Other sources of data used	2012 NAIP imagery to set n-values. The lock dimensions were estimated as follows. 30-ft wide weir measured from 2012 NAIP, from the record drawings dated January 1892 the lock opening is 52' wide and 27' deep with a lift of dam at 10.5'. Because there was no way to tie the elvations on the plan set to current datum (NAVD 88), best judgment was used to set the elvation of the dam at an elevation of 521.5-ft. In the cross setion directly upstream of the structure (12.08), the bed elevation was approx 515.5, it was assumed that this corresponded to the upper sill elevation on the lock (on the plan set, section AB,CD). The rest of the elevations of the structure were set relative to this. The lock was assumed to be open and free flowing. The dam overtops at about a flow of 1,800-cfs, which is lower than we know to be true (from C Bitner's analysis of historical aerial photos and correlation with known discharges), but gives ok results at St. Thomas. Also, we have evidence that there is flow through the
6	Other assumptions and notes	The current gage location (06926510 Osage River below St. Thomas, MO) has only been in operation since 1996. Before that, the gage was located approx 8.5 miles upstream (06926500 Osage River near St. Thomas, MO) and has records for 1931 - 1996. The cross sections end at the current gage location, since the model will be calibrated to current conditions, and for the period of record flows from the old gage location will be the flow input in at the new gage location. The basin areas differ by less than 1% (14,500 vs. 14,584 square miles) and therefore the flows should be almost identical. And for the period of record the error will be inside of the tolerance with which we will be able to calculate.

HEC-RAS Review Checklist

Tributary: **Osage River**

MoRiver RM: **130**

Other Comments:

Also, a little Osage River history. Lock and Dam No. 1 (at approx river mile 12) was constructed around 1906 by the USACE, and was transferred to private ownership in 1960 and is now a degraded historic structure. Bagnel Dam is at approx river mile 82 and was constructed in 1930. It is a private lake and is also used to provide hydropower to the area, so the flows on the Osage river are higher during peak demand for electricity. Truman lake is upstream of Bagnell. The minimum flow out of Bagnell was recently changed to 900 cfs.

#1 Cross Section Layout

Designer/Date: **Jean Reed / 22 May 2013**

	Task	Designer notes
1	Cross section spacing less than 1/2 mile (less than approx 2600 feet)?	X
2	Cross sections perpendicular to channel flow and perpendicular to floodplain flow?	X
3	What is the largest flow of record on the gage? Are the cross sections high enough to contain more than this?	82,600-cfs in 2005 for the current gage (below St. Thomas), and, the old gage (near St. Thomas) had recorded a peak flow of 216,000-cfs in 1943
4	Is the cross section depth reasonably uniform on all cross sections? (meaning…. highest elev minus thalweg…. ie. just looking for this to be around the same, no crazy outliers)	X
5	Is the most upstream cross section drawn through the gage location?	Through the current gage (below St. Thomas)
6	Cross sections at all road/bridge constrictions? Three cross sections - one through the centerline and one on either side outside the influence of embankment elevation.	Only one bridge - Hwy 63 (x-sec 5.61). Drew one cross section through the centerline of the roadway.
7	Cross sections at any major geographical constriction? ie. narrow floodplain?	X
8	Last cross section is cut as close to the confluence as possible without crossing Missouri River cross sections.	X
9	Is the river centerline reasonable?	X

HEC-RAS Review Checklist

Tributary: **Osage River**

MoRiver RM: **130**

#2 Geometry and Unsteady Run

Designer/Date: **Jean Reed / 22 May 2013**

	Task	Designer notes
1	Unsteady model runs without error.	X
2	How do HTab Param (geometry property) look? Points set to 100? Reasonable increment?	Set to 100 w/ incriment of 0.5-ft except for one especially deep cross section.
	How HT tables (calculated in pre-processor) look? Any sudden jumps or discontinuities?	X
3	Does the time period chosen for unsteady run include low flows, high flows, and the range in between?	Ran Spring 2008, and then ran September 1999 (min flow of 280-cfs) and May 1943 (max flow of 216,000-cfs) to test the lowest and highest flows.
4	Upstream boundary condition properly input?	Flow @ St. Thomas, mostly instintanous (15 min) flows, but used daily avg for 1943 run.
5	Downstream boundary condition properly input?	Ran the mainstem model to get a stage boundary @ the mouth of the Osage.
6	Bed data captured by surface? If no, was the method used to estimate bed data reasonable and produce reasonable results at low flows?	Merged hydrosurvey from Aug 2007. Limitation was that the cross sections were spaced at 1-mile.
7	Does the thalweg profile look reasonable?	X
8	Overbank flowpaths reasonable?	X
9	Are the bank stations for the channel reasonable? Reasonably consistant upstream and downstream? (No huge changes in width and elevations.)	X
10	Are roughness coefficients reasonable?	Used overbank n-values of 0.1 for heavy trees, 0.07 for light/partial trees, 0.045 for crops.
11	Does the HEC-RAS Water Surface Profile look reasonable?	X

HEC-RAS Review Checklist

Tributary: **Osage River**

MoRiver RM: **130**

12	Is the water surface profile contained within the cross sections?	X
13	Is there divided flow in any cross-sections? This is okay as long as it makes physical sense.	X
14	Does the calculated rating curve at the gage cross section come reasobably close to replicating the observed rating curve?	X
15	Ineffective flows or levee points used appropriately?	Used pernament ineffective flows to represent three L head dikes that are in this reach of the Osage River. To estimate the height, checked NAIP 2012 imagery which showed the dikes out of the water or barely overtopped. The imagery was collected on 6/24/2012 and the avg flow on that day was 1,610 cfs. Therefore, set the ineffective flows to be overtopped at about this flow.
16	Are the velocities consistent through the reach? Are the velocities reasonable? Velocities that are much higher or lower than typical should be flagged for closer inspection.	X
17	Is Froude's number equal to 1 at any of the cross sections? If yes, is this true to reality? If not it could be a red flag that there is something wrong with the model.	X
18	How does the thalweg elevation at the downstream end compare to other sources of data? 1999 hydrosurvey data at the mouth of each tributary, and low stages on the Missouri River at the mouth.	The thalweg elevations were set by merging hydrosurvey data. Should be ok.

Other Comments:

Also ran Aug 2007 and Sept 1996 because an observed water surface profile was avalable to compare to. It was difficult to this, I think both because of error in the model (ie. hydrosurvey data interpolated where there was none, and 10M DEM surface topography), and error in the water surface collection method.

Was able to match the calculated with observed within about 1 - 1.5-ft of difference.

HEC-RAS Review Checklist

Tributary: **Gasconade**

MoRiver RM: **104**

Model Data/Major Assumptions

	Item	
1	Horizontal Projection	NAD 1983 UTM Zone 15N (Foot US)
2	Vertical Datum	NAVD 88
3	Surface(s) used to cut cross sections	USGS National Elevation Dataset 10M DEM (2011)
4	Channel data - if estimated, what were the dimensions chosen (width, side slopes, invert, slope, etc.) and what data source did you use to guide the estimates	USGS Field measurments were used to approximate channel data at the Rich Fountain gage cross section. The 1999 hydrosurvey was used to approximate channel data at the confluence with the Missouri River. Intermediate cross sections were interpolated between the Gage cross section and the Mouth cross section. The slope was set at 0.00023 ft/ft based on the original profile.
5	Other sources of data used	Big Maps aerial photograhy, and USGS topographic maps
6	Other assumptions and notes	The rating curve on this gage calculates too high at mid flows and too low at high flows. There is at least one time consuming, modification that could be made to possibly improve this calculation: Adjust overbank elevations at x-section 26.12 and downstream. Upstream of 26.12 has been lidar enhanced in the 2011 NED, downtream has not. If we compare the 2009 and 2011 NED upstream of x-sec 26.12 in the 2009 NED there is an obvious stair step trend in the overbank elevations with drops of 20-feet that is smoothed in the 2011 NED. We could use this as a template for hand modifying overbanks downstream of 26.12.

HEC-RAS Review Checklist

Tributary: Gasconade
MoRiver RM: 104

Review #1: Cross Section Layout

Designer/Date: Carolyn Pearson/ 02-Nov-2012
Reviewer/Date: Jean Hilger/ 02-Nov-2012
Backcheck/Date: Carolyn Pearson/19-Nov-2012

	Task	Reviewer	Backcheck
1	Cross section spacing less than 1/2 mile (less than approx 2600 feet)?	Yes	Yes
2	Cross sections perpendicular to channel flow and perpendicular to floodplain flow?	For the most part, yes. Tweak a few cross sections that have sharp angles (doesn't make physical sense). Also visually check that line is perpendicular to channel flow, since cross sections were drawn automatically with tool then adjusted some of them are perpendicular to the flowline but the flowline doesn't perfectly parallel banks	Completed
3	What is the largest flow of record on the gage? Are the cross sections high enough to contain more than this?	134,00-cfs in Dec 1982 and 119,000-cfs in Mar 2008	Ran 134,000 cfs and it is contained in all of the cross sections.
4	Is the cross section depth reasonably uniform on all cross sections?	Not yet, working on it	Tweaked cross section starting and ending points for this.
5	Is the most upstream cross section drawn through the gage location?	Yes	Yes
6	Cross sections at all road/bridge constrictions? Three cross sections - one through the centerline and one on either side outside the influence of embankment elevation.	Need 3 cross sections at the roads, also need to add cross sections for the RR bridge near the confluence.	Competed
7	Cross sections at any major geographical constriction? ie. narrow floodplain?	Yes	Yes
8	Last cross section is cut as close to the confluence as possible without crossing Missouri River cross sections.	Try to get one closer if possible, see map.	Completed
9	Is the river centerline reasonable?	Yes.	Yes, was updated to reflect most recent Bing Imagry.

HEC-RAS Review Checklist

Tributary: Gasconade
MoRiver RM: 104

Review #2: Geometry and Unsteady Run

Designer/Date: Carolyn Pearson/ 21-Dec-2012
Reviewer/Date: Jean Hilger/ 10-Jan-2013
Backcheck/Date: Carolyn Pearson/ 16January201

	Task	Reviewer	Backcheck
1	Unsteady model runs without error.	Yes.	X
2	How do HTab Param (geometry property) look? Points set to 100? Reasonable increment?	Points are set to 100 but incriment is 1-ft. This means the table depth is 100-ft… try incriment of 0.5-ft, may help the lower elevations have smoother curves and reduce some of the wobbles in the profile.	X
3	How HT tables (calculated in pre-processor) look? Any sudden jumps or discontinuities?	Fine.	X
4	Does the time period chosen for unsteady run include low flows, high flows, and the range in between?	Yep. 118,000-cfs high and 750-cfs low. No multiplier was necessary.	X
5	Upstream boundary condition properly input?	Yes.	X
6	Downstream boundary condition properly input?	Yes.	X
7	Bed data captured by surface? If no, was the method used to estimate bed data reasonable and produce reasonable results at low flows?	The method was reasonable, and low flows run stable.	X
8	Does the thalweg profile look reasonable?	Yes.	X
9	Overbank flowpaths reasonable?	Yes.	X

HEC-RAS Review Checklist

Tributary: Gasconade

MoRiver RM: 104

10	Are the bank stations for the channel reasonable? Reasonably consistant upstream and downstream? (No huge changes in width and elevations.)	Yes. A little less reasonable downstream of 26.12 for the overbank reasons discussed in Model Data/Major Assumptions.	X
11	Are roughness coefficients reasonable?	0.018 is not reasonable for a channel n-value, especially considering the MoRiver is probably about 0.027 or 0.028 and it is larger and less rough. Adjusted n-values for 0.03 in the channel and simplified overbank n-values.	I agree. The rating curve might have matched slightly better with a value of 0.018, but this is not a reasonable value for this river.
12	Does the HEC-RAS Water Surface Profile look reasonable?	Some large wobbles, due to stair step nature of overbank on the lower half of this model. Also a large channel constriction around RM 35-ish. No action.	X
13	Is the water surface profile contained within the cross sections?	Yes.	X
14	Is there divided flow in any cross-sections? This is okay as long as it makes physical sense.	Fine.	X
15	Does the calculated rating curve at the gage cross section come reasobably close to replicating the observed rating curve?	Is still a little high at mid and a little low at the peaks, but everything reasonable has been done to try to match this at this point.	I concur.
16	Ineffective flows or levee points used appropriately?	Yes.	X
17	Are the velocities consistent through the reach? Are the velocities reasonable? Velocities that are much higher or lower than typical should be flagged for closer inspection.	Check out 21Mar2008 @ 1200. Some BIG spikes. Ie. 34.53, 25.62, 11.51 - velocities of about 15 ft/s. Unreasonably high. Looks like it's due to overbank height changes @ the downstream two	X
18	Is Freud's number equal to 1 at any of the cross sections? If yes, is this true to reality? If not it could be a red flag that there is something wrong with the model.	Yes. Same locations as above.	X

Other Comments:	Computation interval should be 10 minutes, Output interval can be 1 hour	X

HEC-RAS Review Checklist

Tributary: **Gasconade**

MoRiver RM: **104**

Other Comments:	Appears that river centerline was updated to better match Bing Maps but flowpaths were not re-calculated, so rech lenghts in the model match the old river centerline and don't exactly match the stationing. However, the difference is small (hundreths of a mile) so did not change.	x

Attachment 7

Mitigation Chutes Basic Assumptions

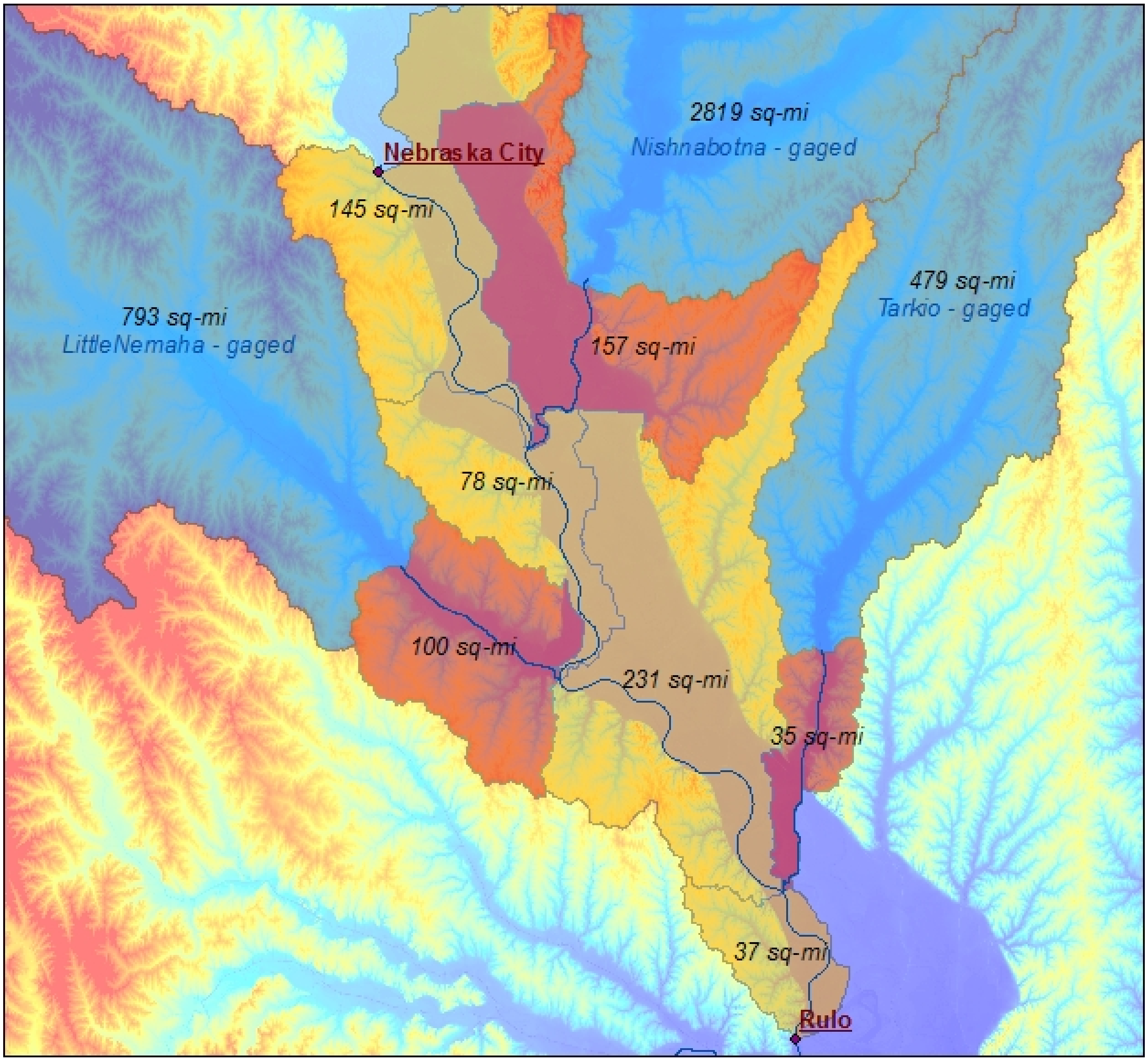
First cross section in Model	Chute Name	Natural	Geometry Assumptions	Ineffective flow assumptions
458.71	Worthwine		150-ft wide at CRP (2012 google earth), triangular chnl from CRP to -10 CRP, 0.5:1 sides	All effective
281.14	Cranbury	x	Floodway model geometry	All effective
217.69	Lisbon	x	Floodway model geometry	All effective
214.09	Jameson		300-ft wide at CRP (2012 google earth), triangular chnl from CRP to -10 CRP, 0.5:1 sides	All effective
194.76	Franklin Island	x	Floodway model geometry	All effective
186.98	Overton North		200-ft wide at top of bank, trapezoid chnl at -10 CRP	All effective
179.9	Tadpole		200-ft wide at top of bank, trapezoid chnl at -10 CRP	All effective
133.12	Smokey Waters		200-ft wide at top of bank, trapezoid chnl at -10 CRP, 2:1 side slopes	Flow blocked by revetment, assume +4 CRP based on design criteria
124.47	St Aubert	x	Floodway model geometry	All effective
112.88	Tate Island	x	Trapezoid, 220-ft wide at top of bank, -10 CRP, 3:1 sides, based on floodway geom and arial photos	Flow blocked to +3 CRP per Chapman
93.08	Lunch Island	x	Trapezoid, 550-ft wide at top of bank, -13 CRP, 5:1 sides, based on floodway geom and arial photos	Flow blocked to -2 CRP because of sand bar

60.4	no name (near Labadie Bottoms)	x	Floodway model geometry	Flows at approx +7 CRP and up, per Chapman est.
57.85	no name	x	3-m LiDAR geometry	Ineffective to t/bank b/c silted
49.57	Centaur Chute/Howell Island	x	Floodway model geometry	Massive control structure, no flow until +7 CRP, per Chapman est.
42.96	Johnson Island	x	Trapezoid, 250-ft wide at top of bank, -4 CRP, 1:1 sides, based on floodway geom and arial photos	Flow at -3 CRP, per Chapman, old structure was removed, new structure built to -3
41.34	Bonhomme Chute	x	Floodway model geometry, same n-value as overbank b/c silted and vegitated	All effective
36.63	no name	x	Floodway model geometry	Ineffective to t/bank b/c silted
33.61	no name	x	Floodway model geometry	Silt and debris because of bridge, flows only at high flows, per Chapman
26.35	Bryan Island	x	Floodway model geometry (w/ slight tweaks for consistant 300-400-ft top width)	Substantial entrance structure, flows at +7 CRP and above, per Chapman survey
15.83	Car of Commerce/Pelican Island	x	Floodway model geometry	Effective flow at +5 CRP, per Chapman est.
10.67	Littles	x	Floodway model geometry (w/ slight tweak at one x-sec for consistant top width)	Effective flow at +3 CRP, per Chapman est.

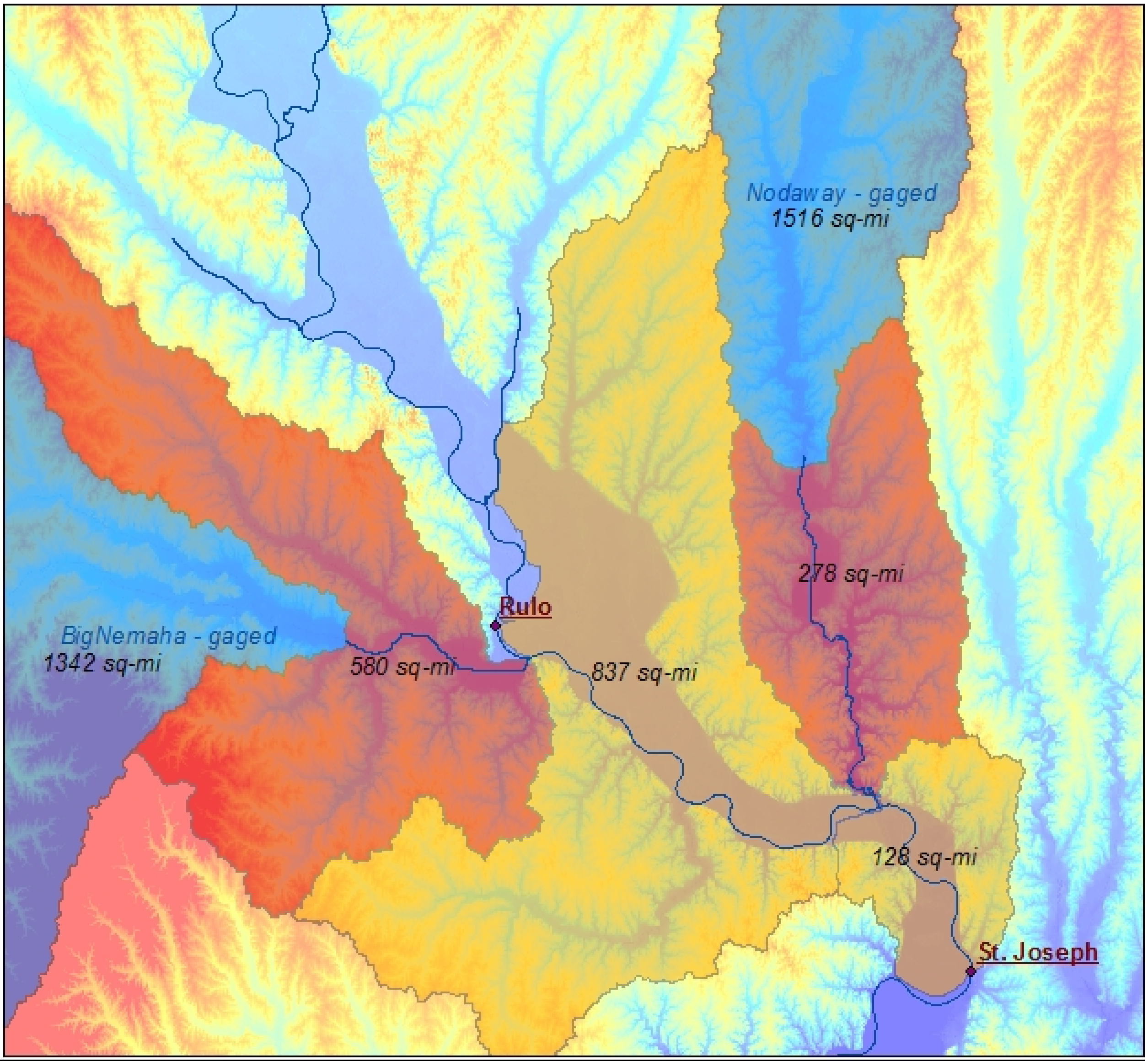
Attachment 8

Ungaged Basin Area Maps

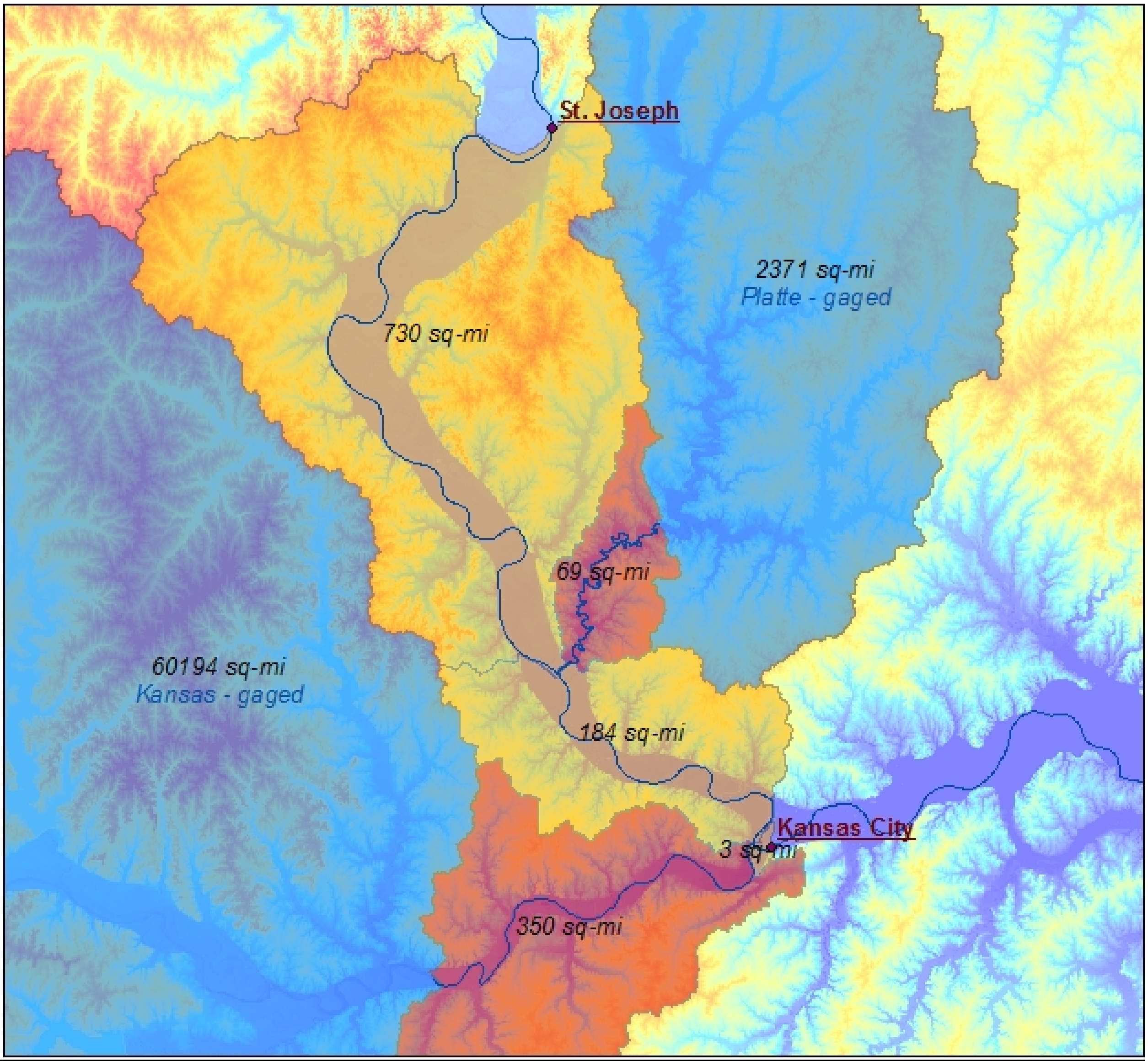
Nebraska City to Rulo



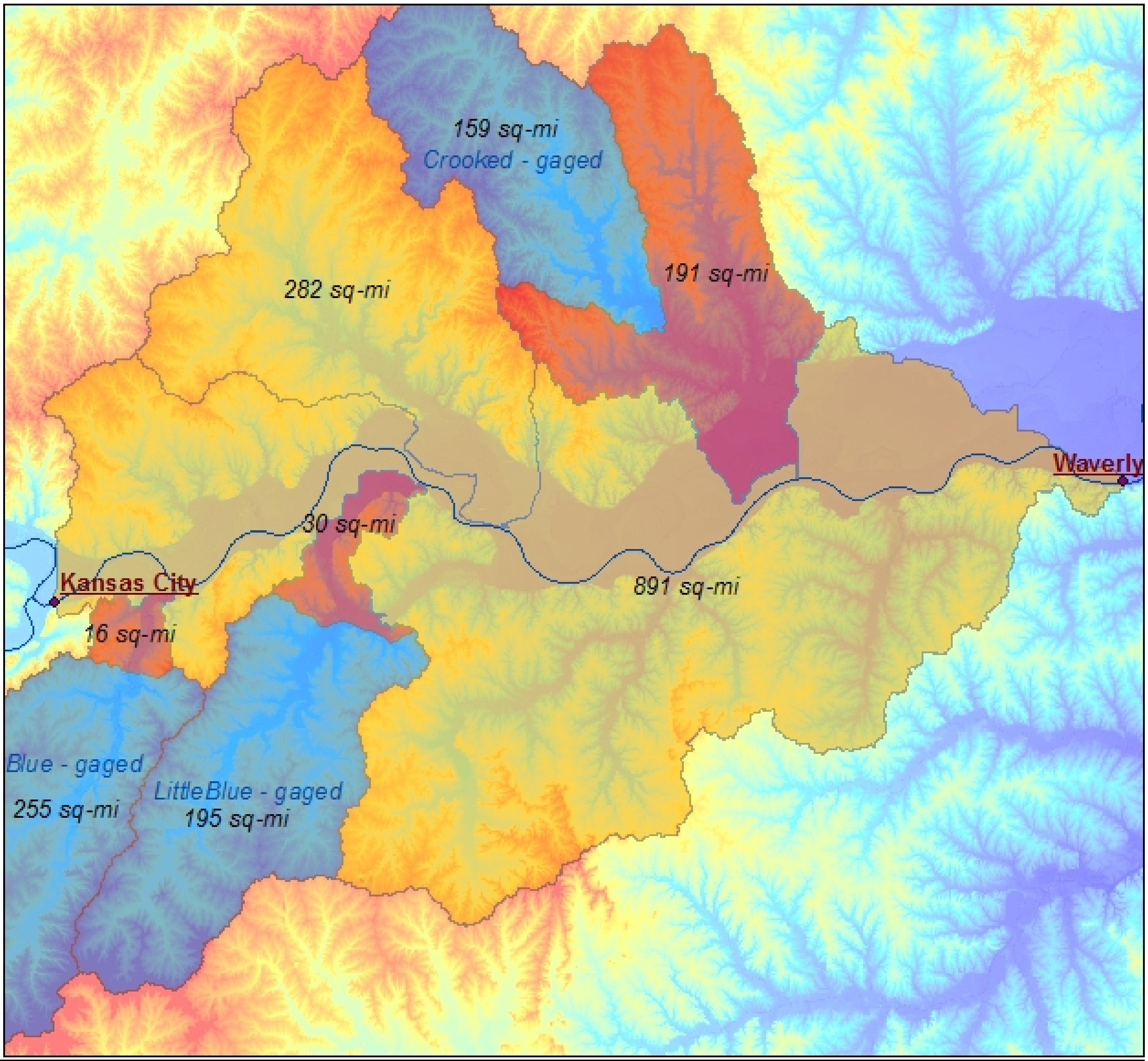
Rulo to St. Joseph



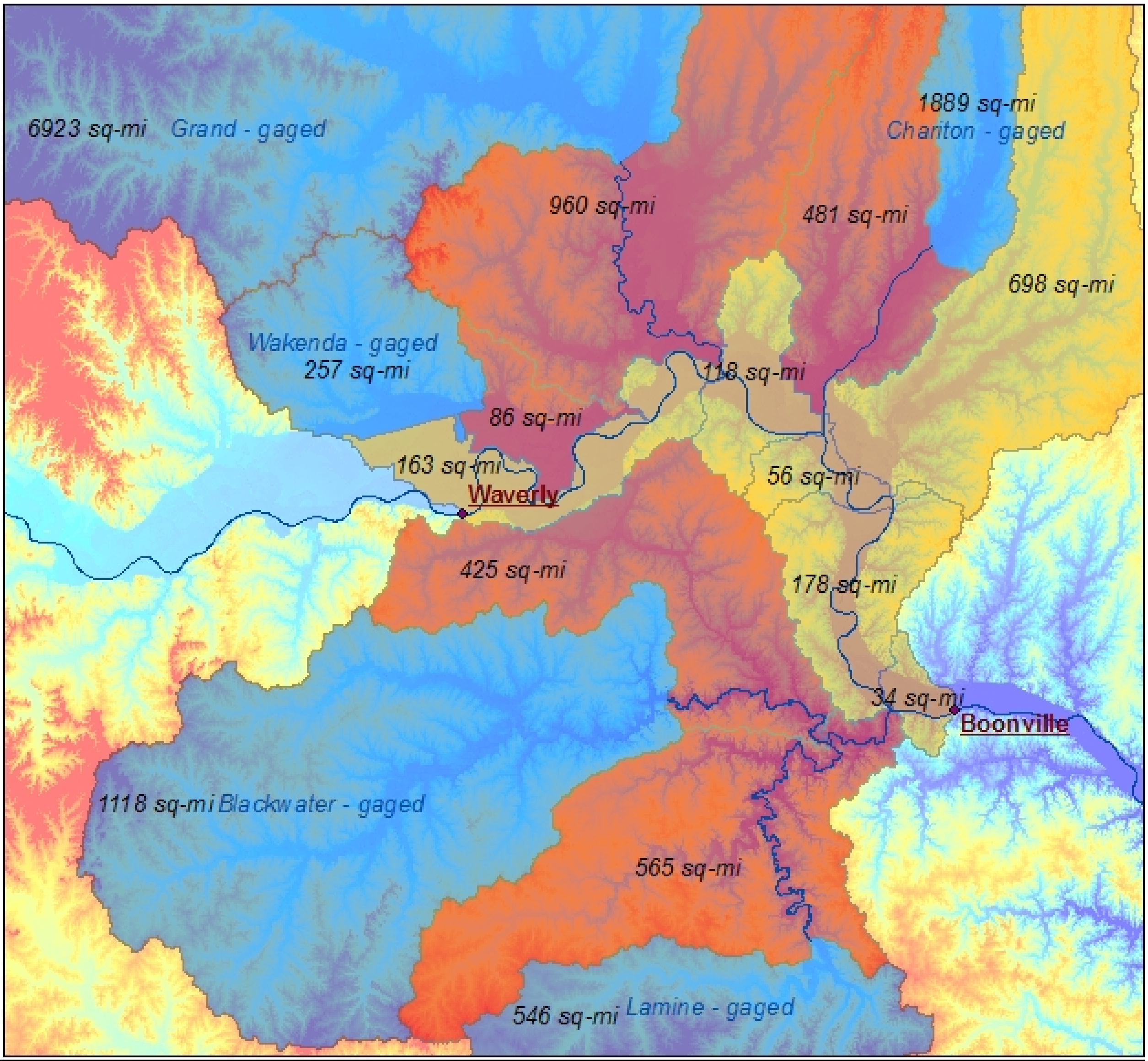
St. Joseph to Kansas City



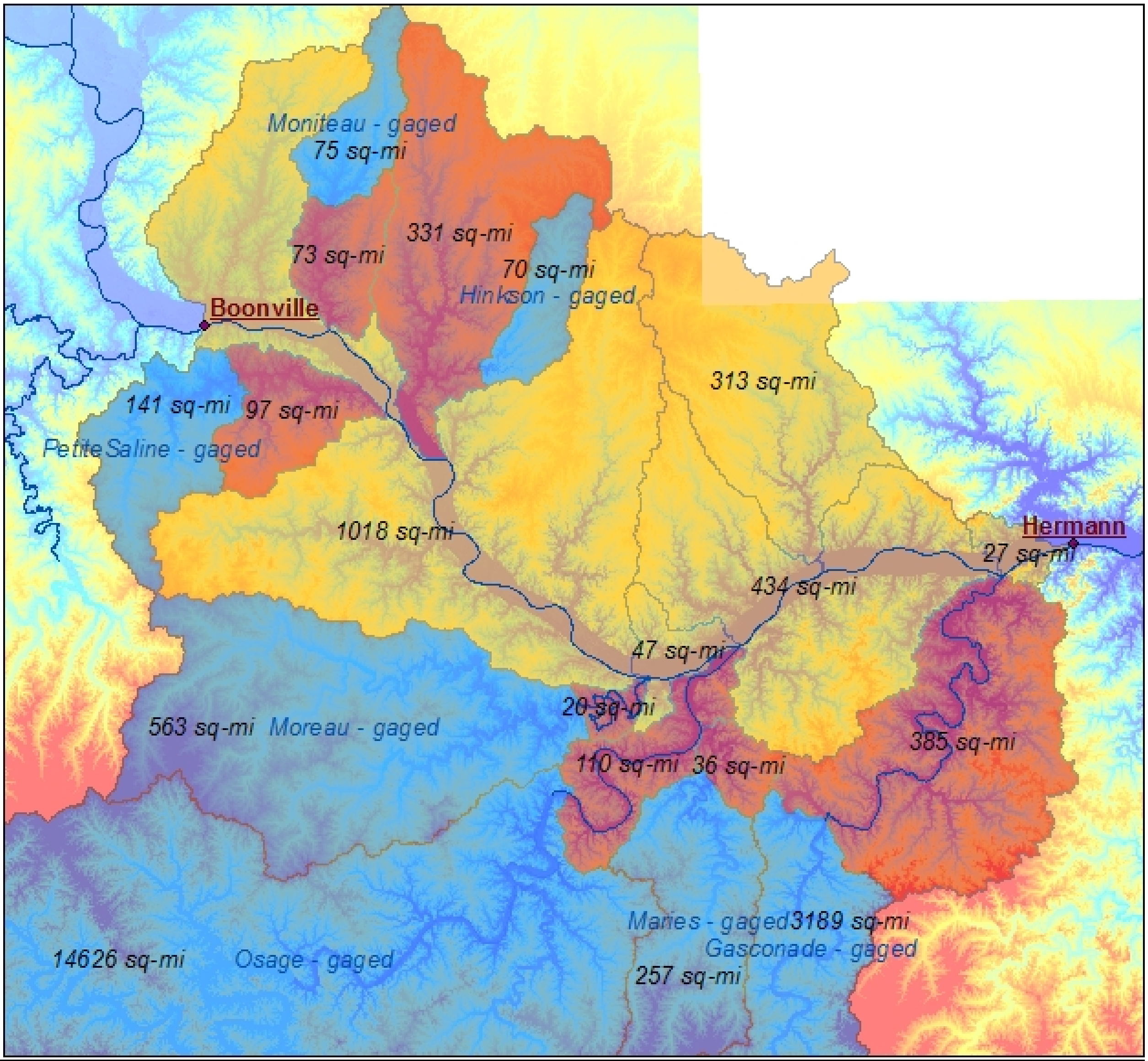
Kansas City to Waverly



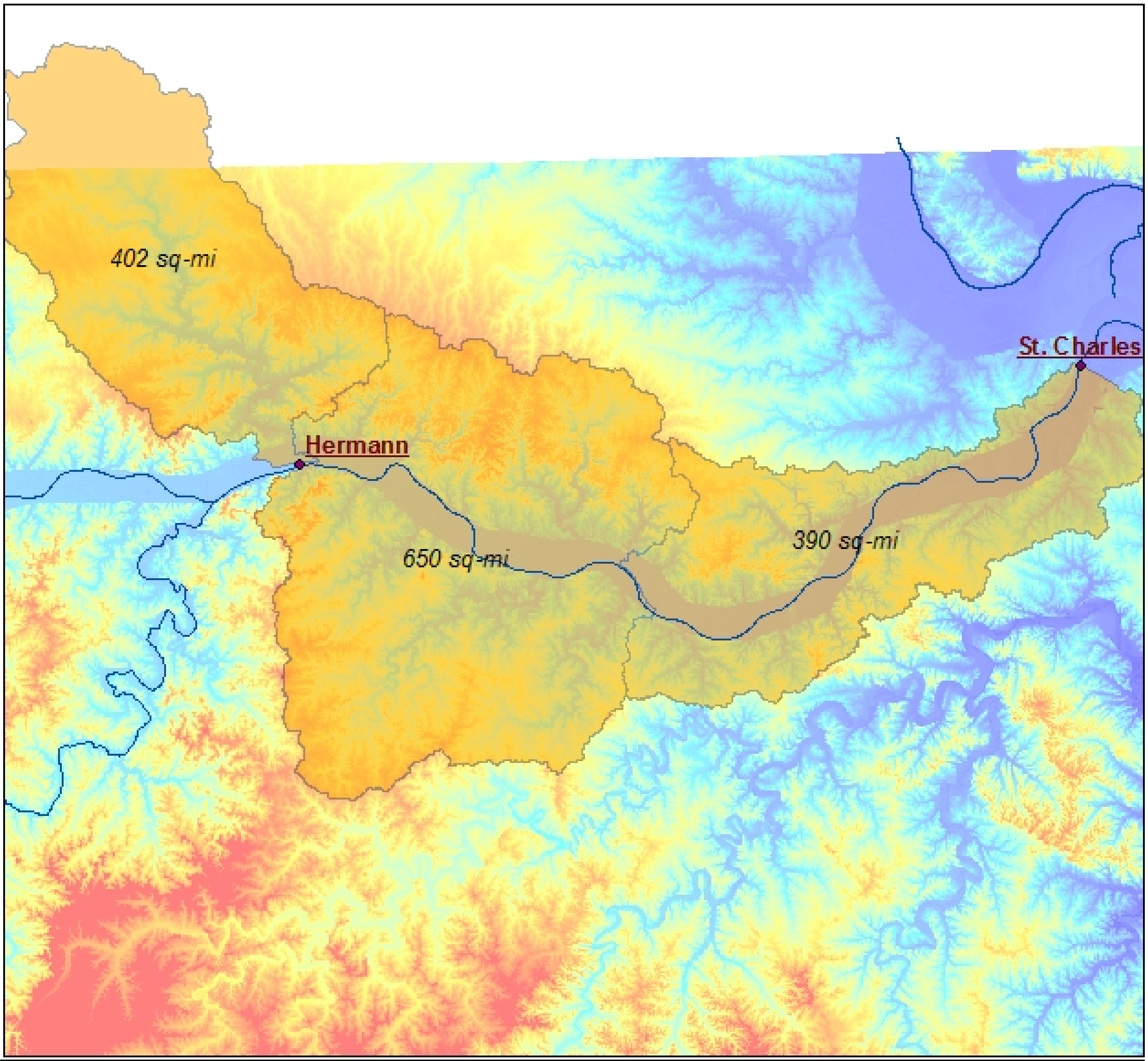
Waverly to Boonville



Boonville to Hermann

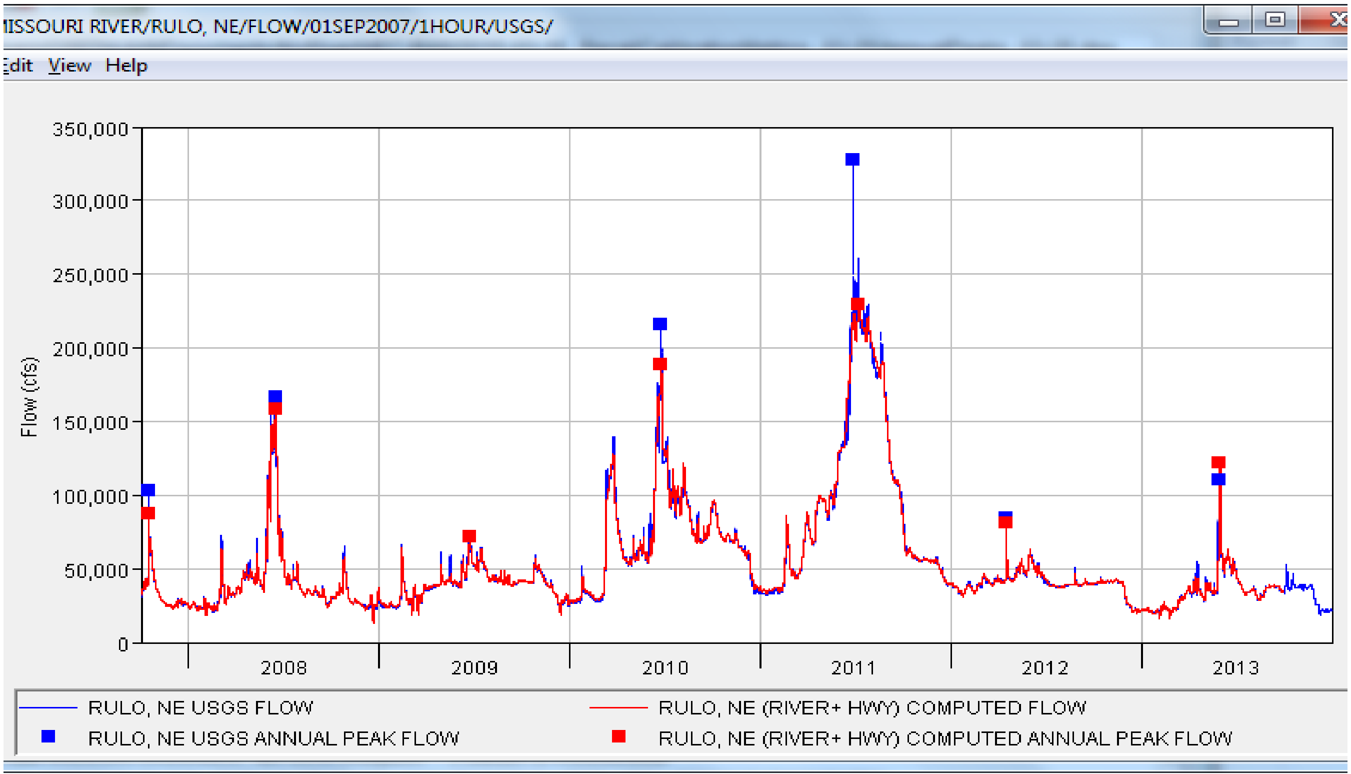


Hermann to St. Charles



Attachment 9

Annual Peak Calibration Results

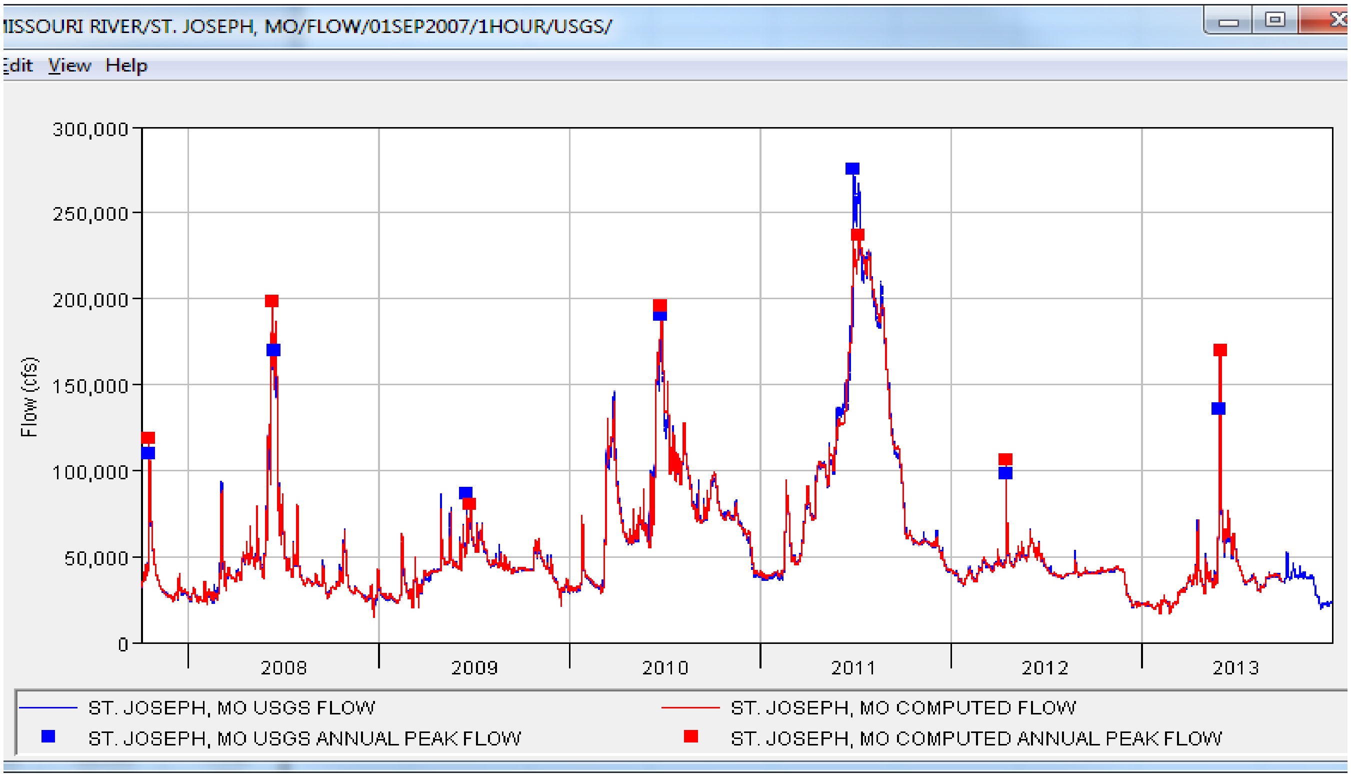


Gage Name	Year	USGS Observed Flow (cfs) [2]	USGS Observed Date [2]	Model Computed Flow (cfs)	Model Computed Date/Time	Flow Error (comp-obs) (cfs)	Percent Difference
Rulo	2008	167,000	14 Jun 08	158,000 [1]	15 Jun 08, 08:00	-9,000	-5%
	2009	71,700	21 Jun 09	72,000	23 Jun 09, 03:00	300	0%
	2010	216,000	24 Jun 10	188,000	22 Jun 10, 18:00	-28,000	-13%
	2011	328,000	27 Jun 11	229,000	23 Jun 11, 06:00	-99,000	-30%
	2012	84,000	15 Apr 12	81,000	16 Apr 12, 02:00	-3,000	-4%
	2013	110,000	30 May 13	122,000	31 May 13, 17:00	12,000	11%

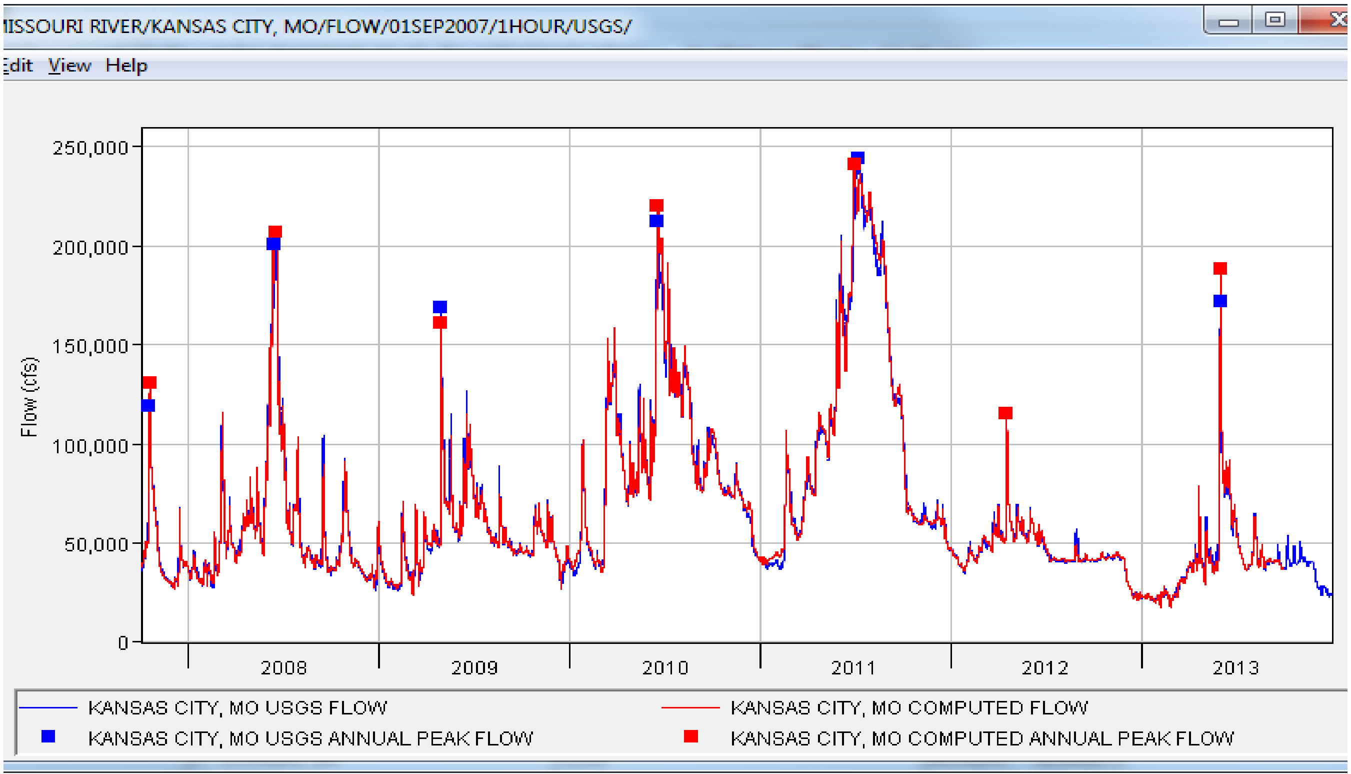
Notes:

[1] Model computed flow in 2011 includes flow over Hwy 159, computed by adding storage area connection hydrographs to the flow at the Rulo gage cross section

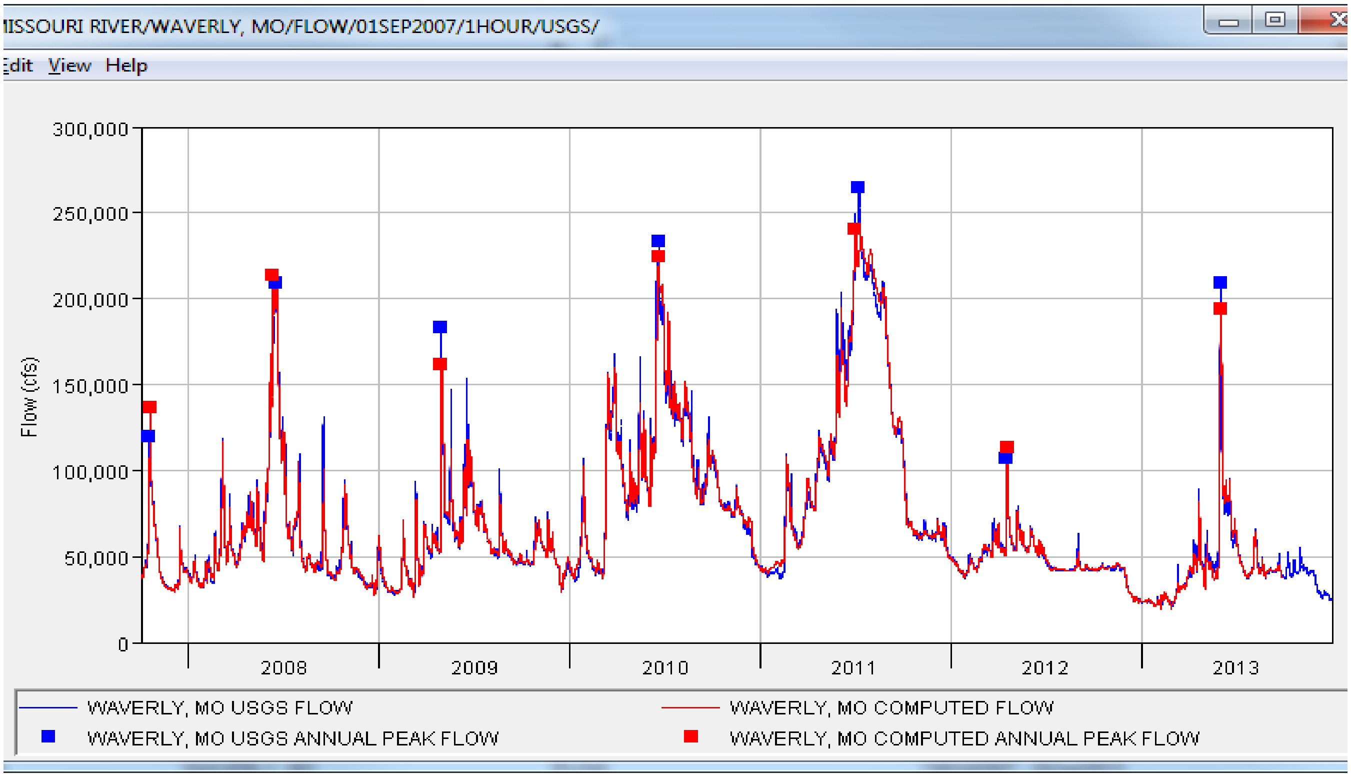
[2] USGS observed data is as pubished on the USGS website, may differ sligtly from flow/date in raw instintaneous data, which is shown in the graph



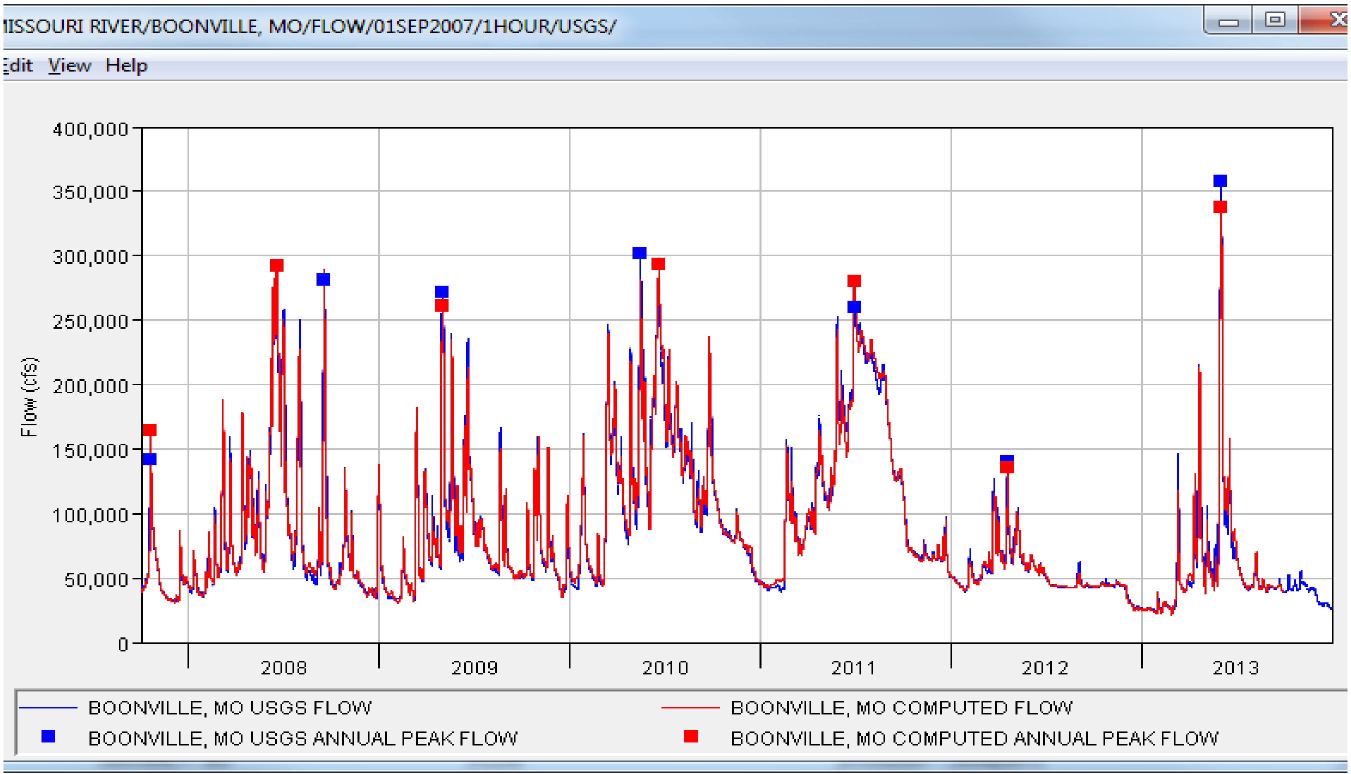
Gage Name	Year	USGS Observed Flow (cfs) [2]	USGS Observed Date [2]	Model Computed Flow (cfs)	Model Computed Date/Time	Flow Error (comp-obs) (cfs)	Percent Difference
St. Joseph	2008	171,000	13 Jun 08	199,000	08 Jun 08, 06:00	28,000	16%
	2009	86,800	16 Jun 09	80,000	22 Jun 09, 08:00	-6,800	-8%
	2010	190,000	25 Jun 10	196,000	24 Jun 10, 08:00	6,000	3%
	2011	277,000	28 Jun 11	237,000	09 Jul 11, 09:00	-40,000	-14%
	2012	98,400	16 Apr 12	106,000	16 Apr 12, 14:00	7,600	8%
	2013	136,000	31 May 13	170,000	31 May 13, 16:00	34,000	25%



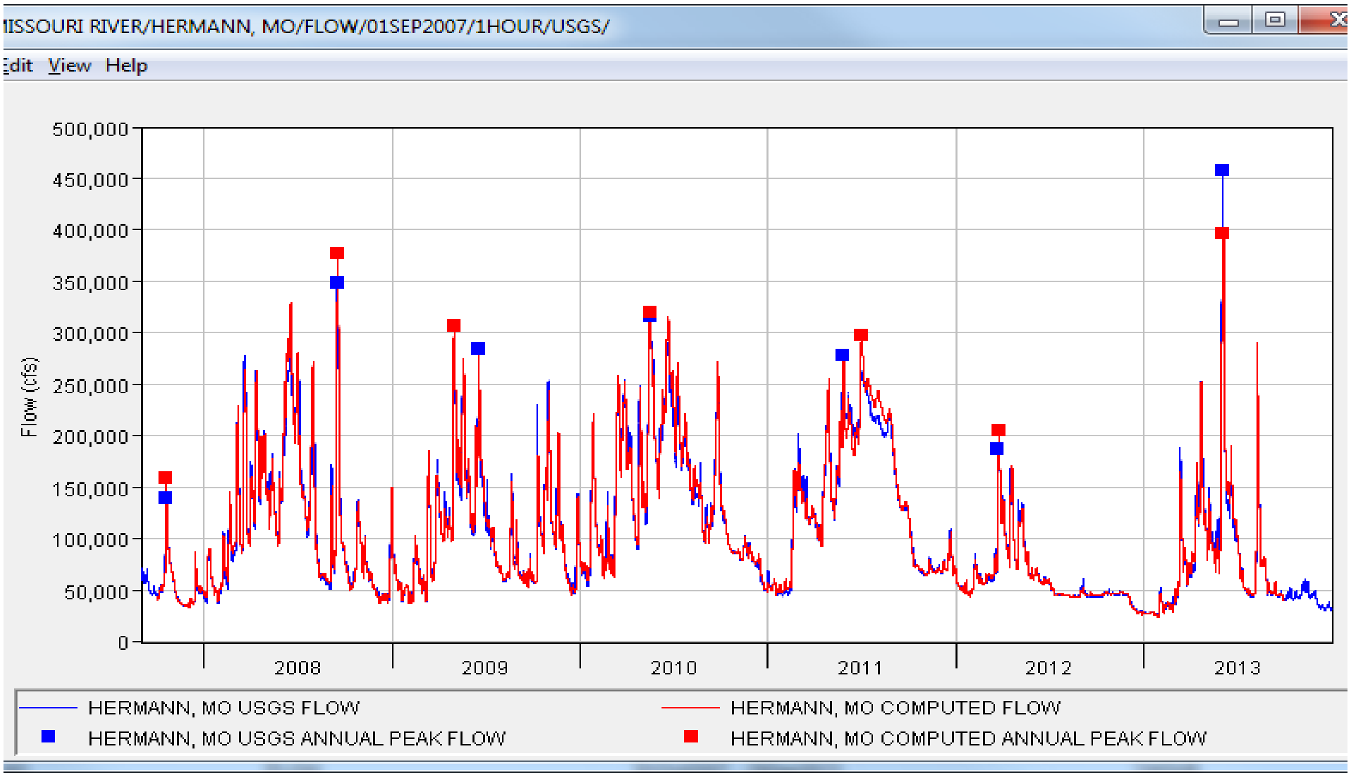
Gage Name	Year	USGS Observed Flow (cfs) [2]	USGS Observed Date [2]	Model Computed Flow (cfs)	Model Computed Date/Time	Flow Error (comp-obs) (cfs)	Percent Difference
Kansas City	2008	201,000	13 Jun 08	207,000	16 Jun 08, 03:00	6,000	3%
	2009	169,000	28 Apr 09	161,000	28 Apr 09, 18:00	-8,000	-5%
	2010	212,000	18 Jun 10	220,000	18 Jun 10, 12:00	8,000	4%
	2011	245,000	10 Jul 11	241,000	30 Jun 11, 03:00	-4,000	-2%
	2012	115,000	16 Apr 12	115,000	17 Apr 12, 09:00	0	0%
	2013	172,000	01 Jun 13	188,000	01 Jun 13, 24:00	16,000	9%



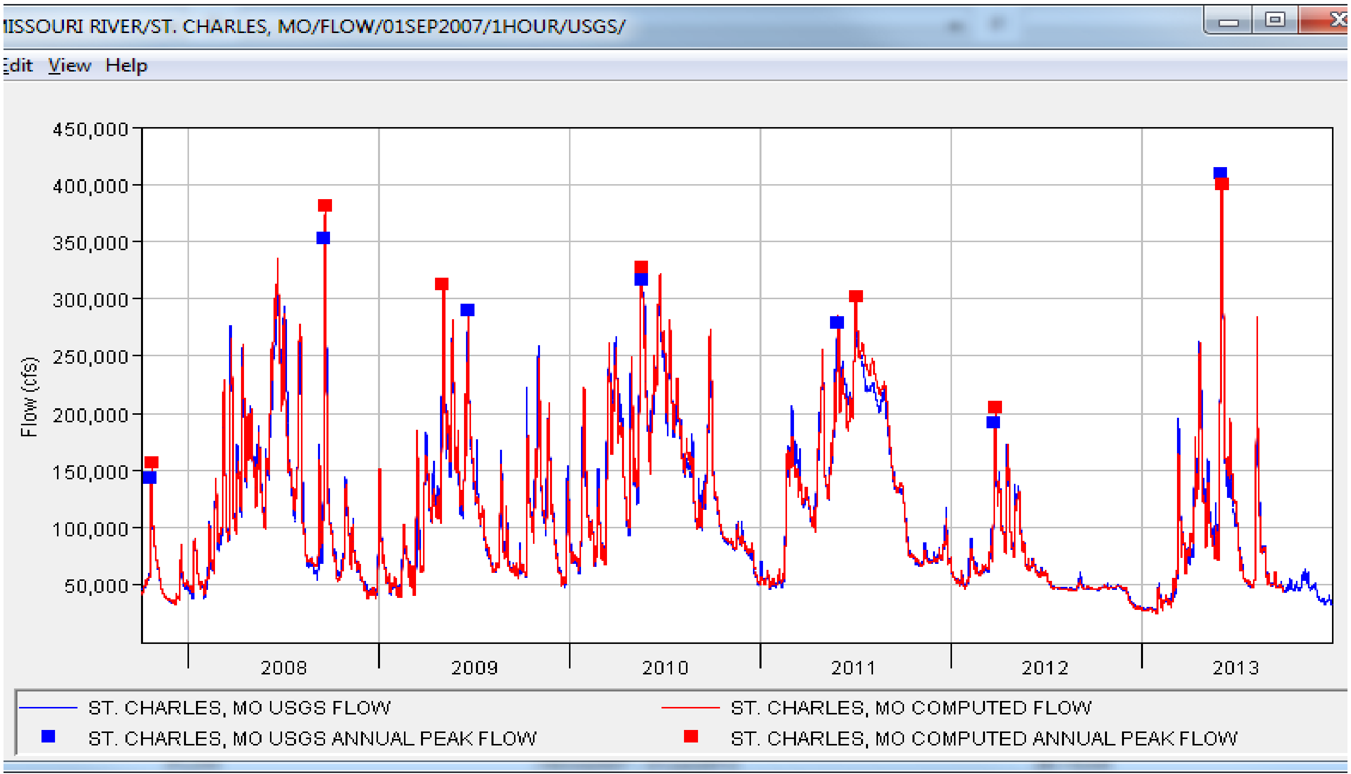
Gage Name	Year	USGS Observed Flow (cfs) [2]	USGS Observed Date [2]	Model Computed Flow (cfs)	Model Computed Date/Time	Flow Error (comp-obs) (cfs)	Percent Difference
Waverly	2008	211,000	14 Jun 08	213,000	10 Jun 08, 14:00	2,000	1%
	2009	183,000	28 Apr 09	161,000	29 Apr 09, 11:00	-22,000	-12%
	2010	233,000	19 Jun 10	225,000	19 Jun 10, 16:00	-8,000	-3%
	2011	265,000	09 Jul 11	240,000	01 Jul 11, 13:00	-25,000	-9%
	2012	108,000	17 Apr 12	113,000	18 Apr 12, 01:00	5,000	5%
	2013	209,000	01 Jun 13	194,000	02 Jun 13, 13:00	-15,000	-7%



Gage Name	Year	USGS Observed Flow (cfs) [2]	USGS Observed Date [2]	Model Computed Flow (cfs)	Model Computed Date/Time	Flow Error (comp-obs) (cfs)	Percent Difference
Boonville	2008	281,000	16 Sep 08	292,000	17 Jun 08, 17:00	11,000	4%
	2009	275,000	30 Apr 09	260,000	01 May 09, 13:00	-15,000	-5%
	2010	302,000	15 May 10	293,000	20 Jun 10, 10:00	-9,000	-3%
	2011	260,000	01 Jul 11	280,000	30 Jun 11, 10:00	20,000	8%
	2012	140,000	18 Apr 12	136,000	18 Apr 12, 16:00	-4,000	-3%
	2013	358,000	02 Jun 13	338,000	02 Jun 13, 14:00	-20,000	-6%



Gage Name	Year	USGS Observed Flow (cfs) [2]	USGS Observed Date [2]	Model Computed Flow (cfs)	Model Computed Date/Time	Flow Error (comp-obs) (cfs)	Percent Difference
Hermann	2008	350,000	15 Sep 08	376,000	17 Sep 08, 08:00	26,000	7%
	2009	287,000	18 Jun 09	307,000	01 May 09, 20:00	20,000	7%
	2010	316,000	17 May 10	321,000	17 May 10, 19:00	5,000	2%
	2011	279,000	27 May 11	297,000	02 Jul 11, 17:00	18,000	6%
	2012	188,000	23 Mar 12	205,000	26 Mar 12, 04:00	17,000	9%
	2013	457,000	01 Jun 13	396,000	03 Jun 13, 20:00	-61,000	-13%



Gage Name	Year	USGS Observed Flow (cfs) [2]	USGS Observed Date [2]	Model Computed Flow (cfs)	Model Computed Date/Time	Flow Error (comp-obs) (cfs)	Percent Difference
St. Charles	2008	353,000	16 Sep 08	381,000	16 Sep 08, 20:00	28,000	8%
	2009	289,000	19 Jun 09	312,000	18 Jun 09, 24:00	23,000	8%
	2010	317,000	18 May 10	327,000	18 May 10, 20:00	10,000	3%
	2011	279,000	28 May 11	301,000	28 May 11, 15:00	22,000	8%
	2012	192,000	24 Mar 12	205,000	24 Mar 12, 05:00	13,000	7%
	2013	409,000	02 Jun 13	399,000	02 Jun 13, 19:00	-10,000	-2%

All Gages

	Flow Error (cfs)	Percent Difference
Max Pos	34,000	25%
Max Neg	-99,000	-30%
Average	-1,164	0%
Count	42	42
RMS	24,498	9%

Attachment 10

Stage Duration Calibration Results

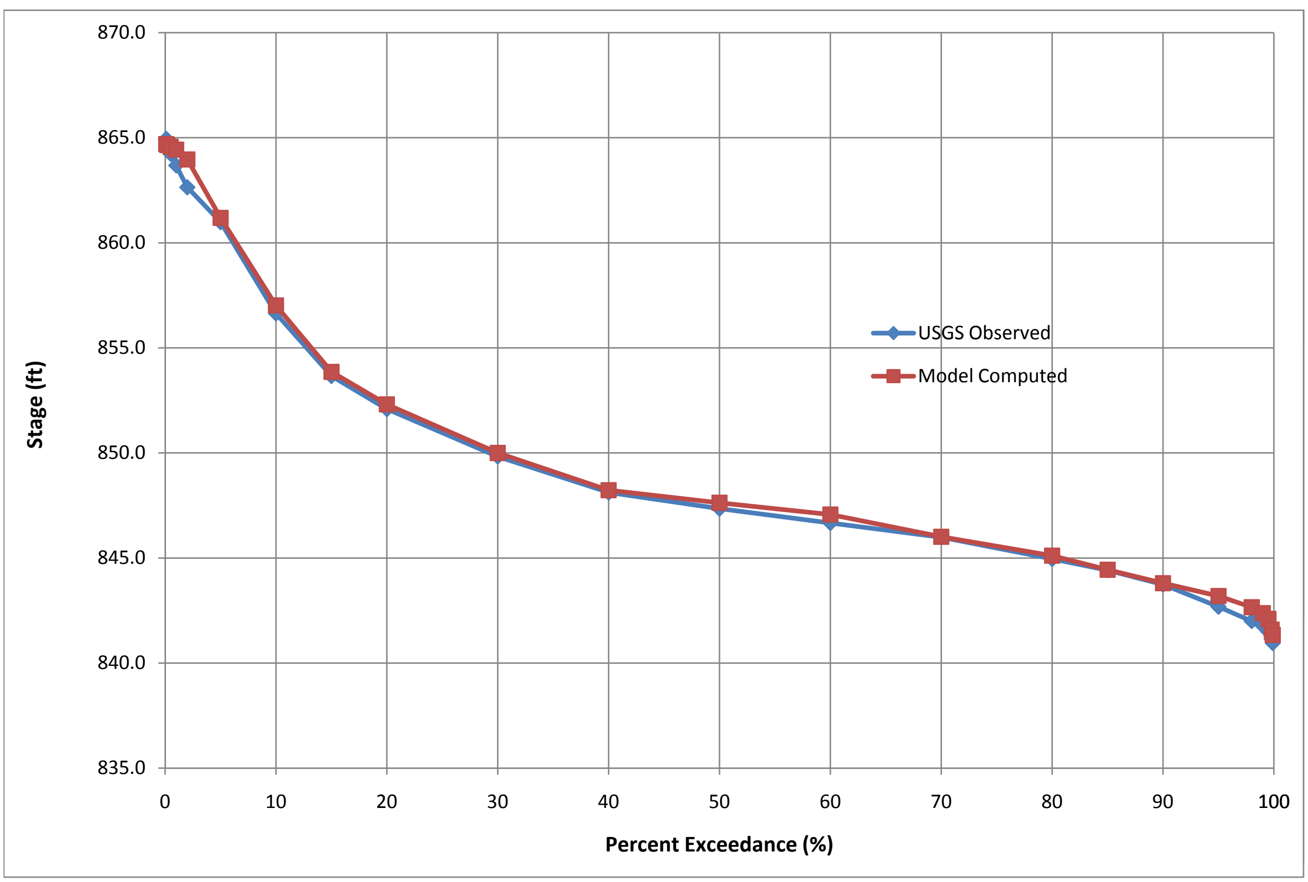
Gage Name	Percent Exceedance (%)	Stage - USGS Observed (ft)	Stage- Model Computed (ft)	Error (Comp-Obs) (ft)
Rulo, NE	0.1	864.9	864.7	-0.2
	0.2	864.7	864.7	-0.1
	0.5	864.2	864.6	0.3
	1	863.7	864.4	0.8
	2	862.6	864.0	1.3
	5	861.0	861.2	0.2
	10	856.6	857.0	0.4
	15	853.7	853.9	0.2
	20	852.1	852.3	0.2
	30	849.8	850.0	0.2
	40	848.1	848.2	0.1
	50	847.4	847.6	0.3
	60	846.7	847.1	0.4
	70	846.0	846.0	0.0
	80	845.0	845.1	0.1
	85	844.4	844.4	0.0
	90	843.7	843.8	0.1
	95	842.7	843.2	0.5
	98	842.0	842.7	0.7
	99	841.8	842.4	0.6
	99.5	841.5	842.1	0.6
	99.8	841.1	841.6	0.5
	99.9	841.0	841.3	0.4

All gages

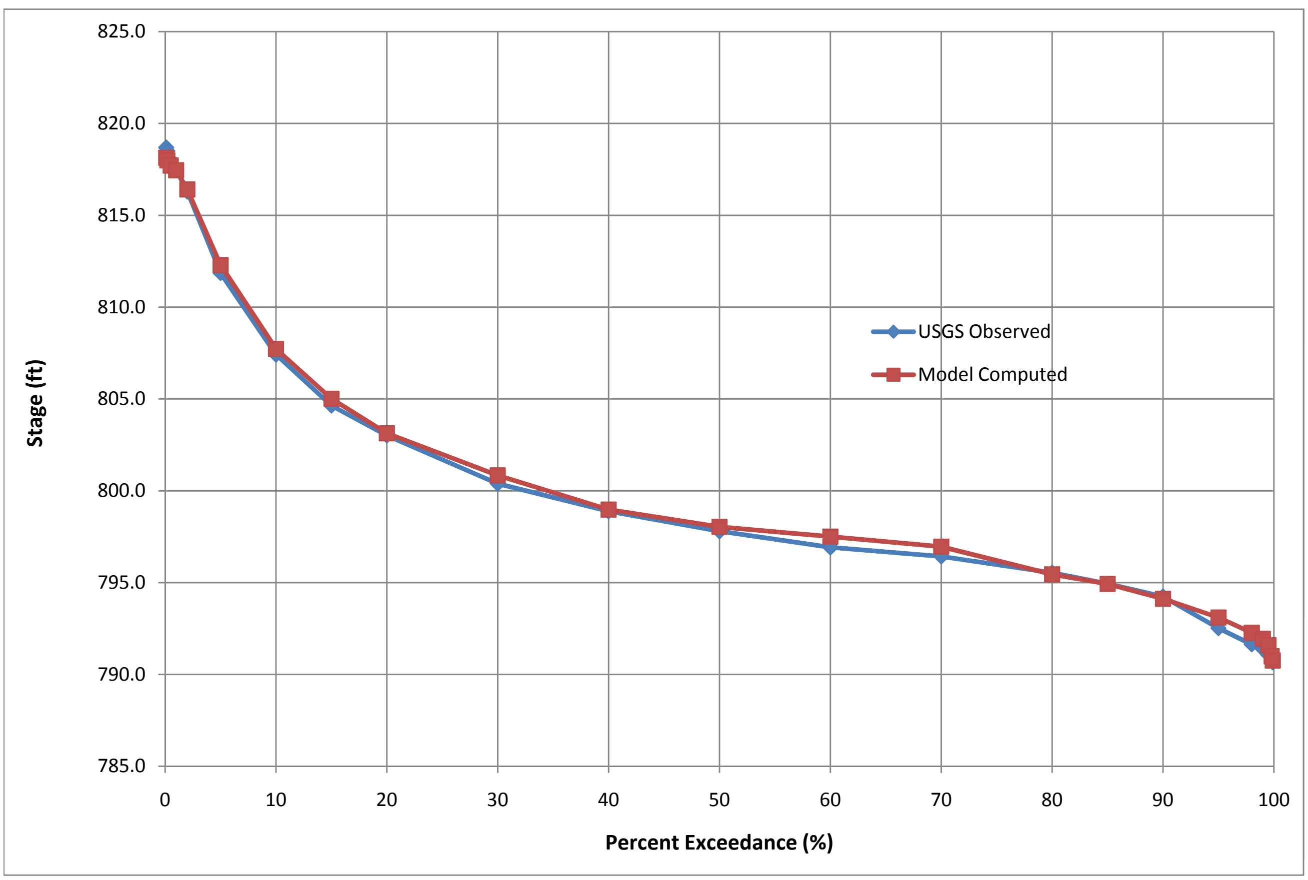
	Error (cfs)
average	0.1
median	0.2
max	1.3
min	-1.8

Greater than 1% Exeedance
(not including major floods)

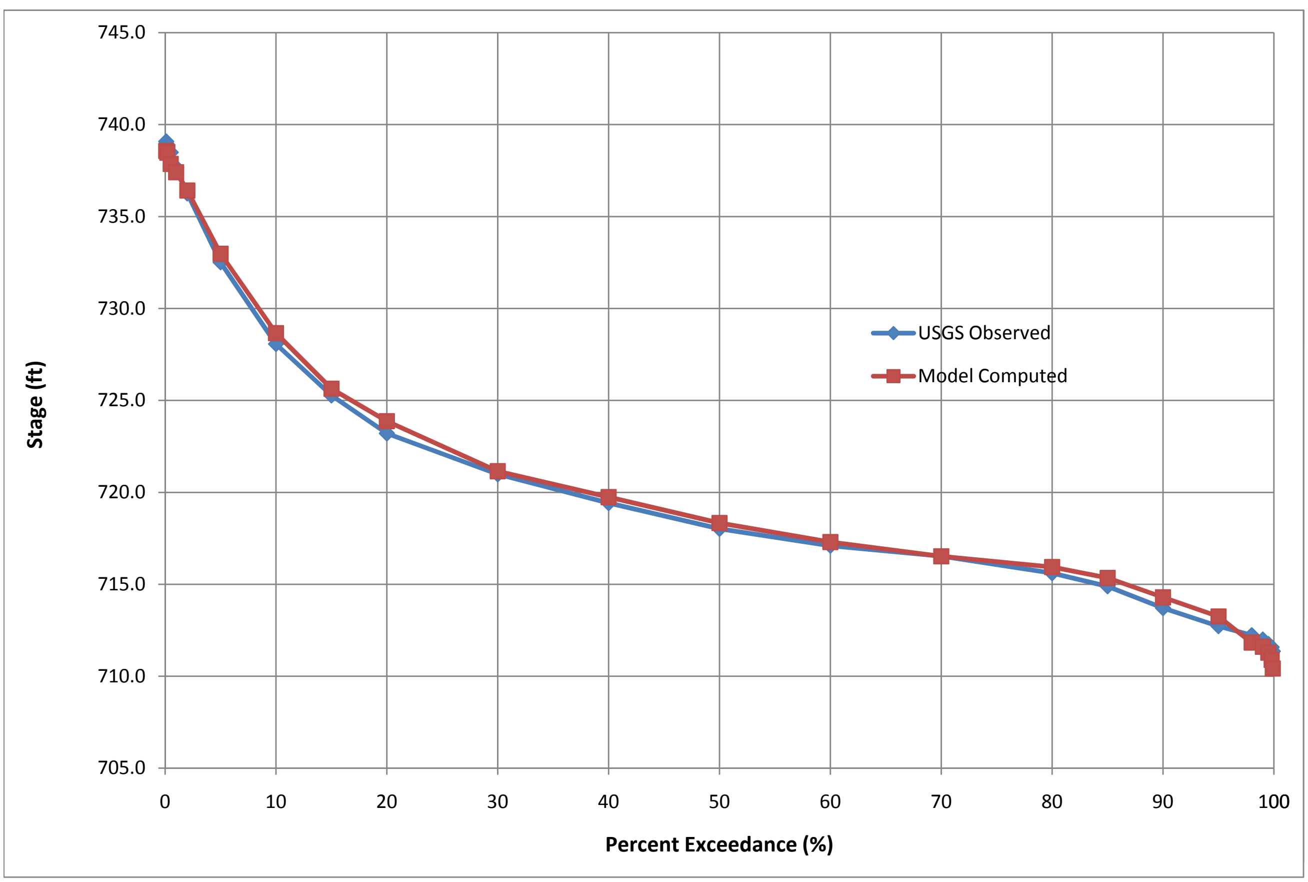
	Error (cfs)
average	0.2
median	0.2
max	1.3
min	-0.9



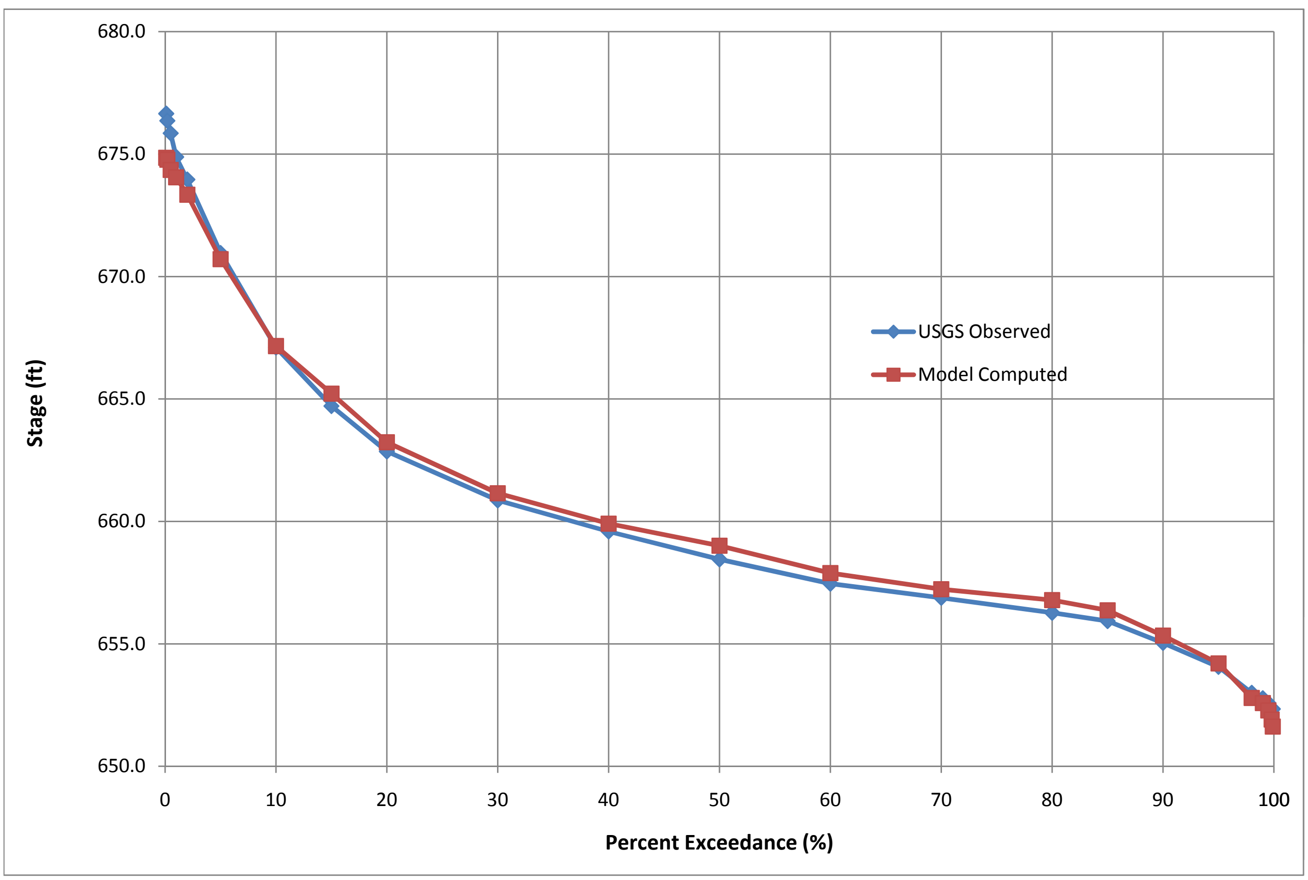
Gage Name	Percent Exceedance (%)	Stage - USGS Observed (ft)	Stage- Model Computed (ft)	Error (Comp-Obs) (ft)
St. Joseph, MO	0.1	818.7	818.1	-0.6
	0.2	818.5	818.0	-0.5
	0.5	818.0	817.7	-0.3
	1	817.5	817.5	0.0
	2	816.3	816.4	0.1
	5	811.9	812.3	0.4
	10	807.4	807.7	0.3
	15	804.6	805.0	0.4
	20	803.0	803.1	0.1
	30	800.4	800.8	0.5
	40	798.9	799.0	0.1
	50	797.8	798.0	0.2
	60	796.9	797.5	0.6
	70	796.4	797.0	0.5
	80	795.5	795.5	-0.1
	85	794.9	794.9	0.0
	90	794.2	794.1	-0.1
	95	792.5	793.1	0.6
	98	791.6	792.3	0.6
	99	791.4	791.9	0.6
	99.5	791.1	791.6	0.5
	99.8	790.8	791.0	0.2
	99.9	790.7	790.8	0.1



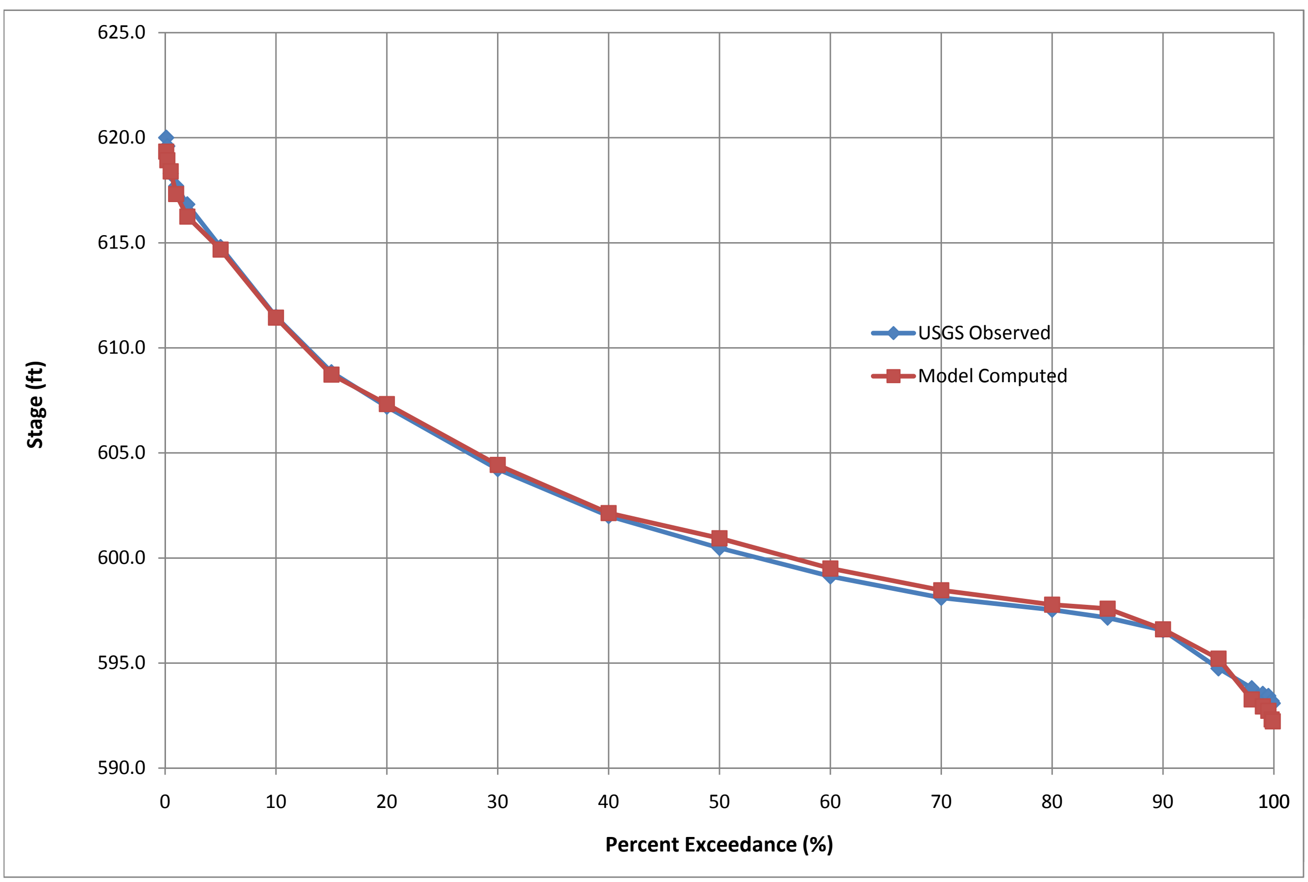
Gage Name	Percent Exceedance (%)	Stage - USGS Observed (ft)	Stage- Model Computed (ft)	Error (Comp-Obs) (ft)
Kansas City, MO	0.1	739.1	738.6	-0.5
	0.2	738.9	738.5	-0.4
	0.5	738.5	737.9	-0.6
	1	737.7	737.4	-0.3
	2	736.3	736.4	0.2
	5	732.5	733.0	0.5
	10	728.1	728.7	0.6
	15	725.3	725.6	0.4
	20	723.2	723.9	0.7
	30	721.0	721.2	0.2
	40	719.4	719.7	0.3
	50	718.0	718.3	0.3
	60	717.1	717.3	0.2
	70	716.5	716.5	0.0
	80	715.6	715.9	0.3
	85	714.9	715.4	0.5
	90	713.7	714.3	0.6
	95	712.7	713.3	0.5
	98	712.2	711.8	-0.4
	99	712.0	711.6	-0.4
	99.5	711.8	711.3	-0.5
	99.8	711.6	710.9	-0.7
	99.9	711.4	710.4	-0.9



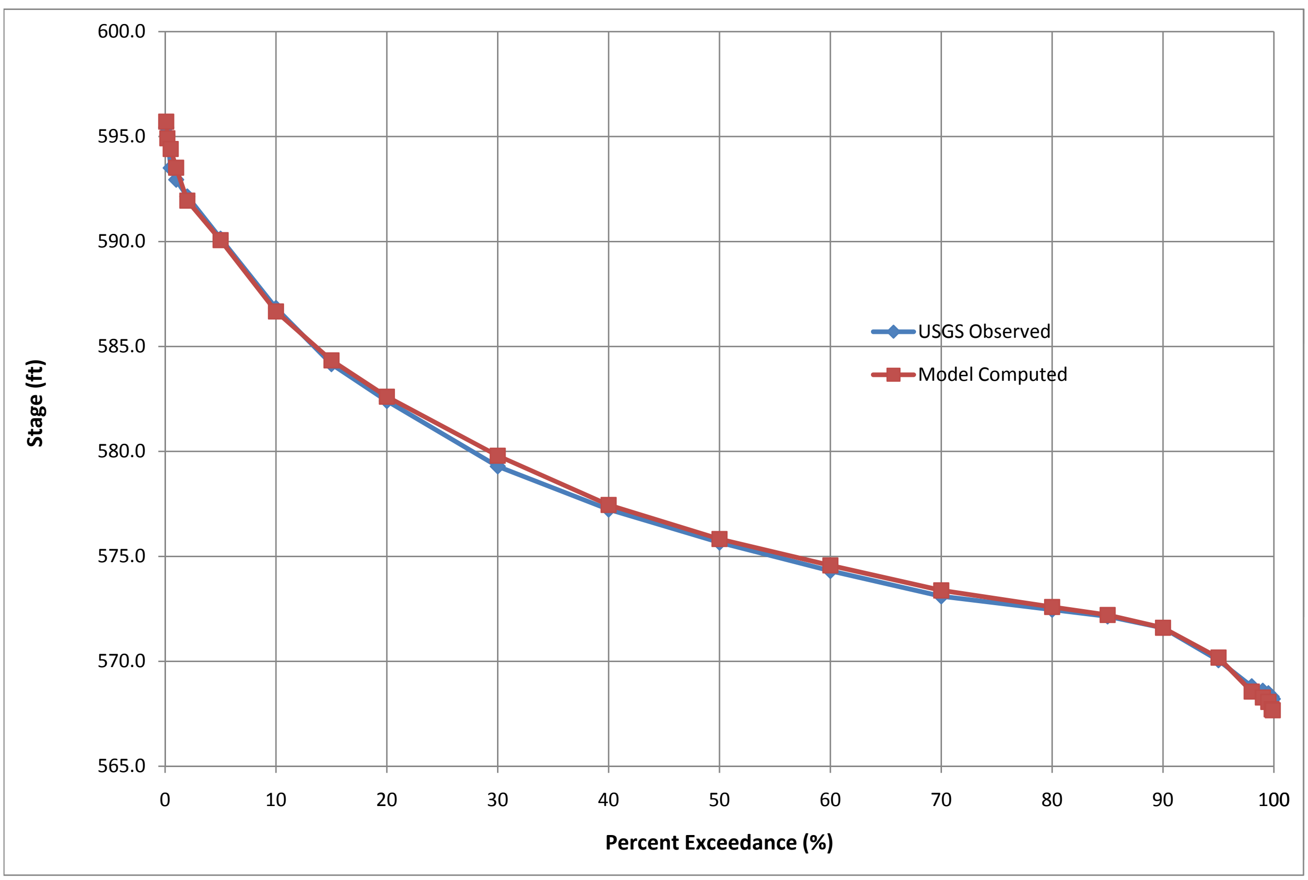
Gage Name	Percent Exceedance (%)	Stage - USGS Observed (ft)	Stage- Model Computed (ft)	Error (Comp-Obs) (ft)
Waverly, MO	0.1	676.7	674.9	-1.8
	0.2	676.4	674.8	-1.6
	0.5	675.9	674.4	-1.5
	1	674.9	674.1	-0.8
	2	674.0	673.3	-0.6
	5	671.0	670.7	-0.3
	10	667.1	667.2	0.1
	15	664.7	665.2	0.5
	20	662.9	663.2	0.4
	30	660.9	661.2	0.3
	40	659.6	659.9	0.3
	50	658.5	659.0	0.6
	60	657.5	657.9	0.4
	70	656.9	657.2	0.4
	80	656.3	656.8	0.5
	85	655.9	656.4	0.4
	90	655.0	655.3	0.3
	95	654.1	654.2	0.1
	98	653.0	652.8	-0.2
	99	652.8	652.6	-0.2
	99.5	652.6	652.3	-0.3
	99.8	652.4	651.9	-0.5
	99.9	652.3	651.6	-0.7



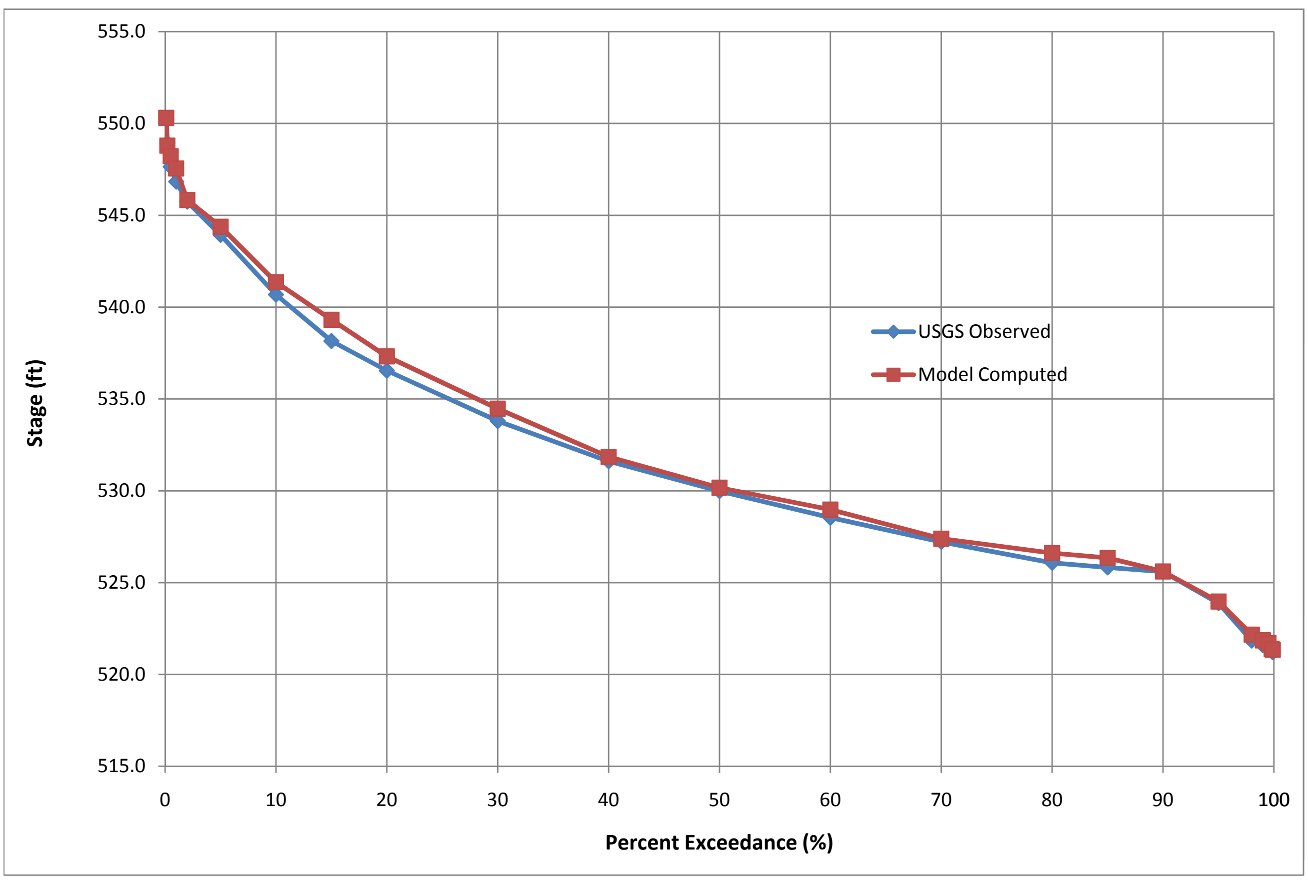
Gage Name	Percent Exceedance (%)	Stage - USGS Observed (ft)	Stage- Model Computed (ft)	Error (Comp-Obs) (ft)
Glasgow, MO	0.1	620.0	619.3	-0.7
	0.2	619.6	618.9	-0.7
	0.5	618.3	618.4	0.1
	1	617.7	617.3	-0.4
	2	616.8	616.3	-0.6
	5	614.8	614.7	-0.1
	10	611.5	611.4	0.0
	15	608.8	608.7	-0.1
	20	607.2	607.3	0.1
	30	604.2	604.4	0.2
	40	602.0	602.1	0.1
	50	600.5	600.9	0.5
	60	599.1	599.5	0.4
	70	598.1	598.5	0.4
	80	597.5	597.8	0.2
	85	597.2	597.6	0.4
	90	596.6	596.6	0.1
	95	594.8	595.2	0.5
	98	593.8	593.3	-0.5
	99	593.5	592.9	-0.6
	99.5	593.4	592.7	-0.7
	99.8	593.2	592.3	-0.9
	99.9	593.1	592.2	-0.9



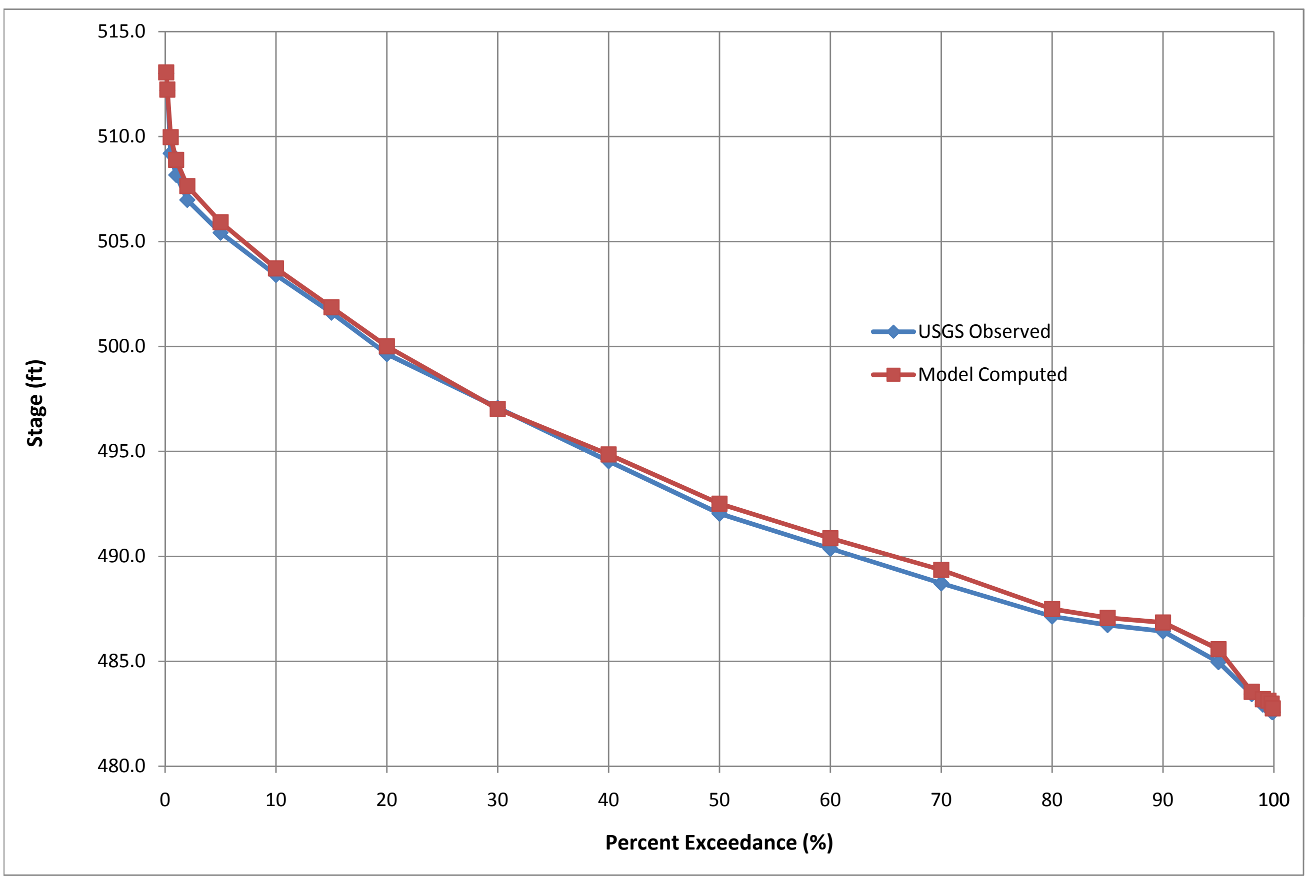
Gage Name	Percent Exceedance (%)	Stage - USGS Observed (ft)	Stage- Model Computed (ft)	Error (Comp-Obs) (ft)
Boonville, MO	0.1	595.4	595.7	0.3
	0.2	595.1	594.9	-0.2
	0.5	593.5	594.4	0.9
	1	593.0	593.5	0.6
	2	592.2	592.0	-0.2
	5	590.1	590.1	-0.1
	10	586.8	586.7	-0.2
	15	584.2	584.3	0.2
	20	582.4	582.6	0.2
	30	579.3	579.8	0.5
	40	577.2	577.5	0.2
	50	575.7	575.8	0.2
	60	574.3	574.6	0.3
	70	573.1	573.4	0.3
	80	572.5	572.6	0.1
	85	572.1	572.2	0.1
	90	571.6	571.6	0.0
	95	570.1	570.2	0.1
	98	568.8	568.6	-0.3
	99	568.6	568.3	-0.3
	99.5	568.5	568.1	-0.4
	99.8	568.3	567.7	-0.6
	99.9	568.2	567.7	-0.5



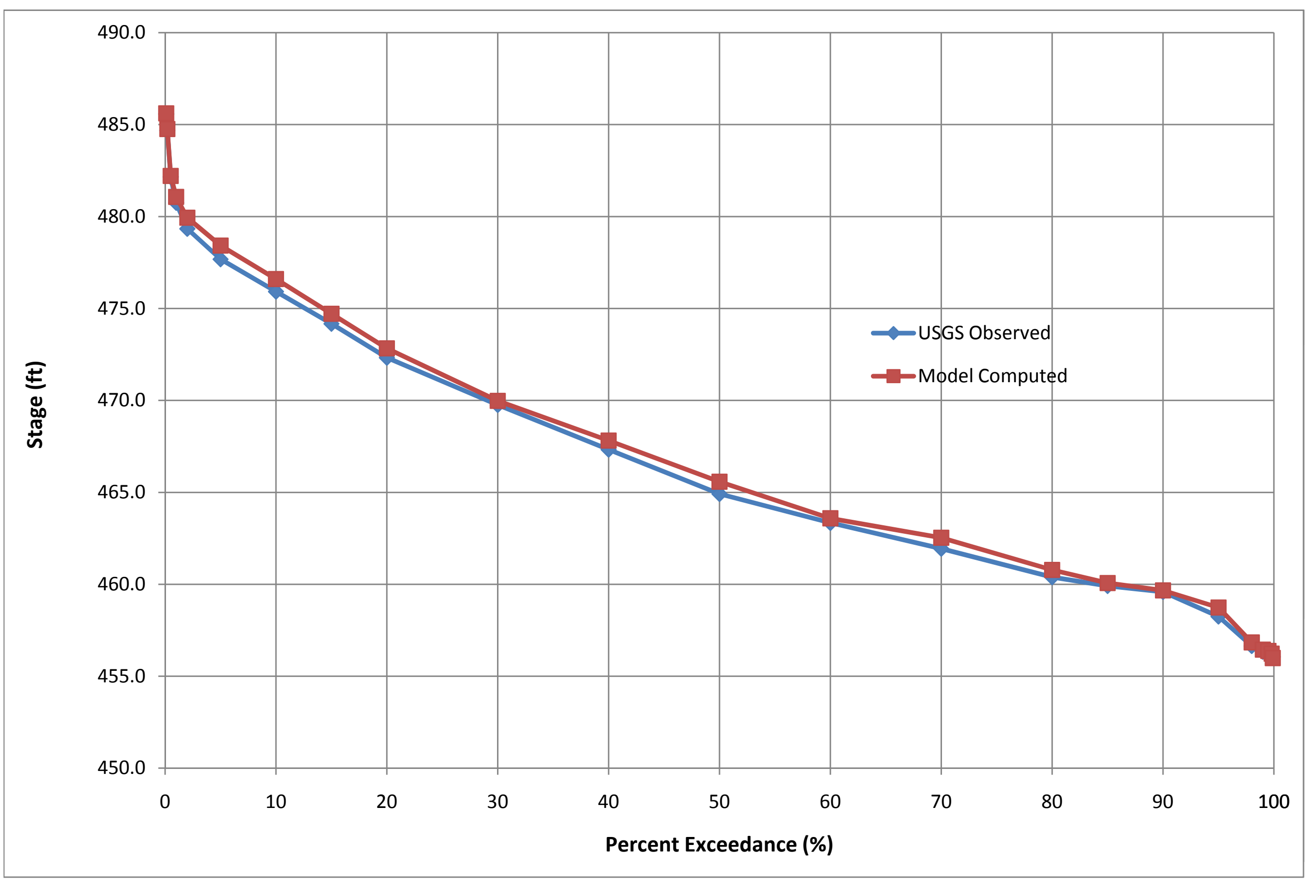
Gage Name	Percent Exceedance (%)	Stage - USGS Observed (ft)	Stage- Model Computed (ft)	Error (Comp-Obs) (ft)
Jefferson City, MO	0.1	550.3	550.3	0.0
	0.2	548.9	548.8	-0.1
	0.5	547.6	548.2	0.6
	1	546.8	547.6	0.7
	2	545.7	545.8	0.1
	5	543.9	544.4	0.5
	10	540.7	541.4	0.7
	15	538.2	539.3	1.2
	20	536.5	537.3	0.8
	30	533.8	534.5	0.7
	40	531.6	531.9	0.2
	50	530.0	530.2	0.2
	60	528.5	529.0	0.4
	70	527.2	527.4	0.2
	80	526.1	526.6	0.5
	85	525.8	526.4	0.5
	90	525.6	525.6	0.0
	95	523.9	524.0	0.1
	98	521.8	522.2	0.3
	99	521.6	521.9	0.3
	99.5	521.4	521.7	0.3
	99.8	521.3	521.4	0.1
	99.9	521.2	521.3	0.1



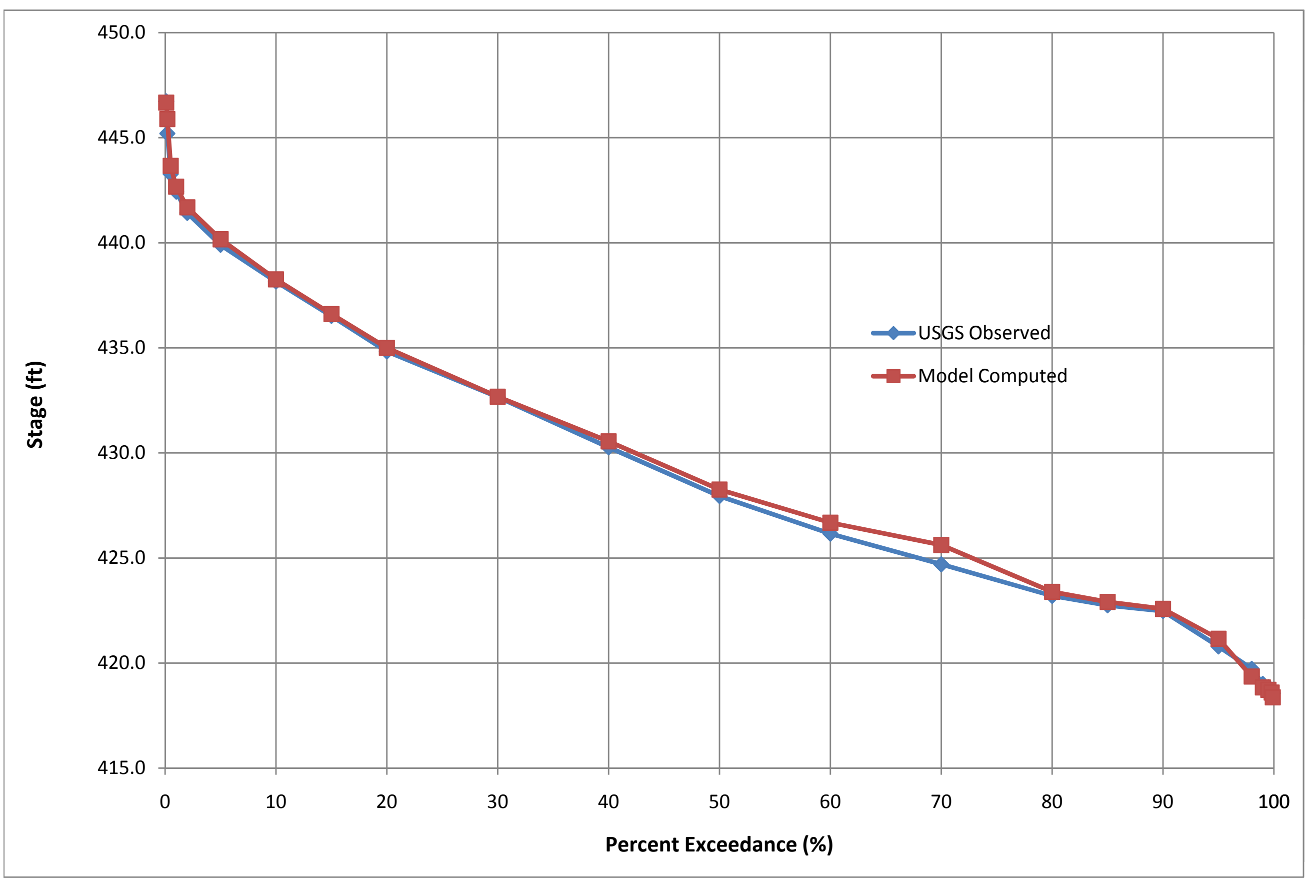
Gage Name	Percent Exceedance (%)	Stage - USGS Observed (ft)	Stage- Model Computed (ft)	Error (Comp-Obs) (ft)
Hermann, MO	0.1	513.0	513.1	0.0
	0.2	512.2	512.2	0.0
	0.5	509.2	510.0	0.8
	1	508.2	508.9	0.7
	2	507.0	507.6	0.7
	5	505.4	505.9	0.5
	10	503.4	503.7	0.3
	15	501.6	501.9	0.3
	20	499.6	500.0	0.4
	30	497.1	497.0	-0.1
	40	494.5	494.9	0.3
	50	492.0	492.5	0.5
	60	490.4	490.9	0.5
	70	488.7	489.4	0.6
	80	487.1	487.5	0.4
	85	486.7	487.1	0.3
	90	486.4	486.9	0.4
	95	485.0	485.6	0.6
	98	483.4	483.6	0.1
	99	482.9	483.2	0.3
	99.5	482.8	483.1	0.4
	99.8	482.6	483.0	0.4
	99.9	482.6	482.8	0.2



Gage Name	Percent Exceedance (%)	Stage - USGS Observed (ft)	Stage- Model Computed (ft)	Error (Comp-Obs) (ft)
Washington, MO	0.1	485.6	485.6	0.0
	0.2	484.7	484.8	0.1
	0.5	482.2	482.2	0.0
	1	480.8	481.1	0.3
	2	479.3	479.9	0.6
	5	477.7	478.4	0.7
	10	475.9	476.6	0.7
	15	474.2	474.7	0.5
	20	472.3	472.8	0.5
	30	469.8	470.0	0.2
	40	467.3	467.8	0.5
	50	464.9	465.6	0.7
	60	463.3	463.6	0.3
	70	461.9	462.5	0.6
	80	460.4	460.8	0.4
	85	459.9	460.1	0.2
	90	459.6	459.7	0.1
	95	458.3	458.7	0.5
	98	456.7	456.8	0.2
	99	456.3	456.5	0.1
	99.5	456.2	456.4	0.2
	99.8	456.1	456.2	0.2
	99.9	456.1	456.0	-0.1



Gage Name	Percent Exceedance (%)	Stage - USGS Observed (ft)	Stage- Model Computed (ft)	Error (Comp-Obs) (ft)
St. Charles, MO	0.1	446.7	446.7	-0.1
	0.2	445.2	445.9	0.7
	0.5	443.3	443.7	0.4
	1	442.4	442.7	0.2
	2	441.4	441.7	0.3
	5	439.9	440.2	0.3
	10	438.2	438.3	0.1
	15	436.5	436.6	0.1
	20	434.8	435.0	0.2
	30	432.7	432.7	0.0
	40	430.3	430.6	0.3
	50	428.0	428.3	0.3
	60	426.2	426.7	0.5
	70	424.7	425.6	0.9
	80	423.2	423.4	0.2
	85	422.8	422.9	0.2
	90	422.5	422.6	0.1
	95	420.8	421.2	0.4
	98	419.7	419.4	-0.4
	99	419.0	418.8	-0.2
	99.5	418.9	418.7	-0.1
	99.8	418.6	418.6	0.0
	99.9	418.6	418.4	-0.2



Attachment 11

Flow Duration Calibration Results

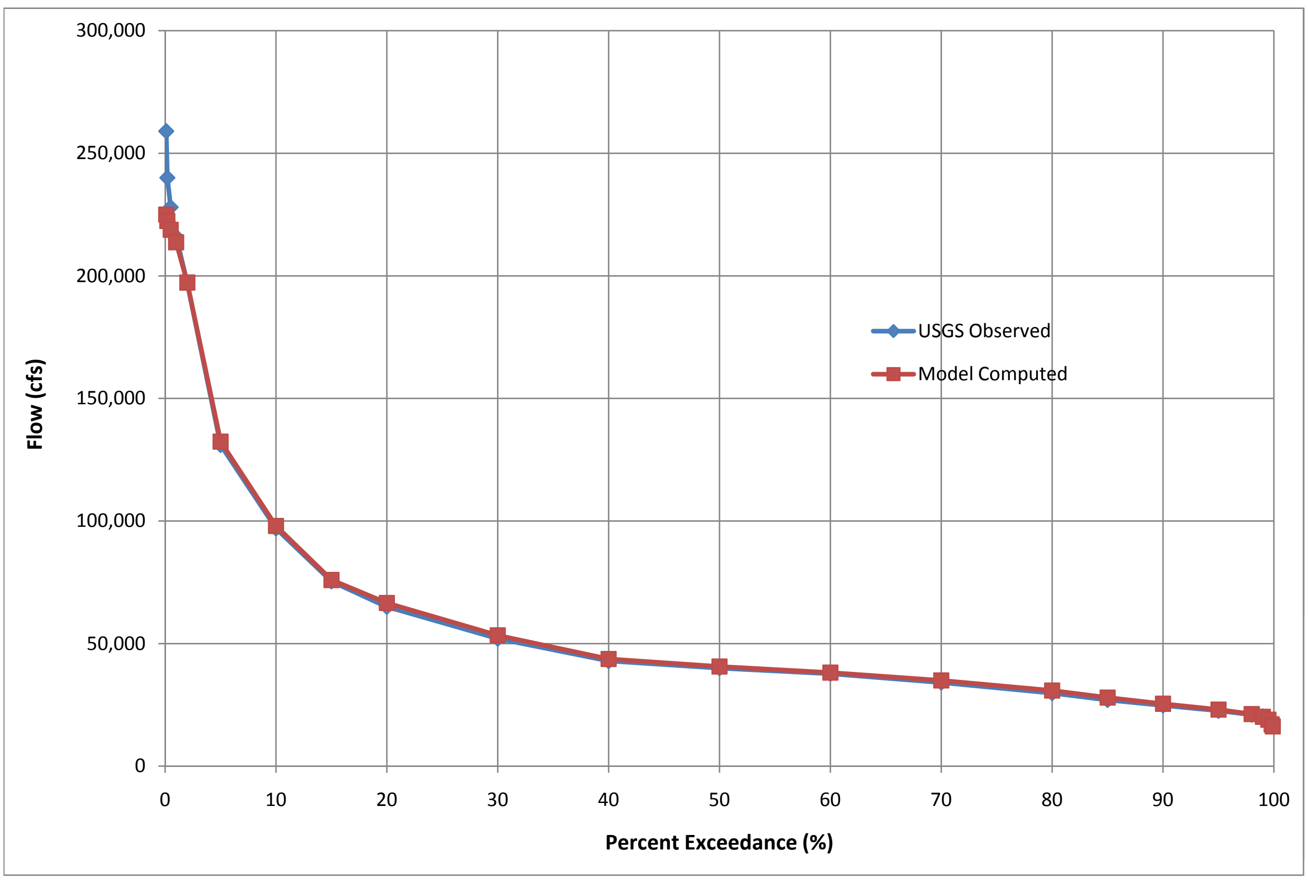
Gage Name	Percent Exceedance (%)	Flow - USGS Observed (cfs)	Flow - Model Computed (cfs)	Error (Comp-Obs) (cfs)	Percent Difference (%)
Rulo, NE	0.1	259,000	224,996	-34,004	-13%
	0.2	240,000	222,347	-17,653	-7%
	0.5	228,000	218,772	-9,228	-4%
	1	216,000	213,696	-2,304	-1%
	2	197,000	197,259	259	0%
	5	131,000	132,393	1,393	1%
	10	97,000	97,998	998	1%
	15	75,400	75,942	542	1%
	20	65,000	66,588	1,588	2%
	30	52,000	53,369	1,369	3%
	40	42,900	43,724	824	2%
	50	40,000	40,667	667	2%
	60	37,700	38,180	480	1%
	70	34,100	34,963	863	3%
	80	29,800	30,873	1,073	4%
	85	27,000	27,986	986	4%
	90	24,800	25,464	664	3%
	95	22,600	23,104	504	2%
	98	21,000	21,232	232	1%
	99	20,400	20,132	-268	-1%
	99.5	19,600	18,976	-624	-3%
	99.8	18,500	17,044	-1,456	-8%
	99.9	16,263	16,235	-28	0%

All gages

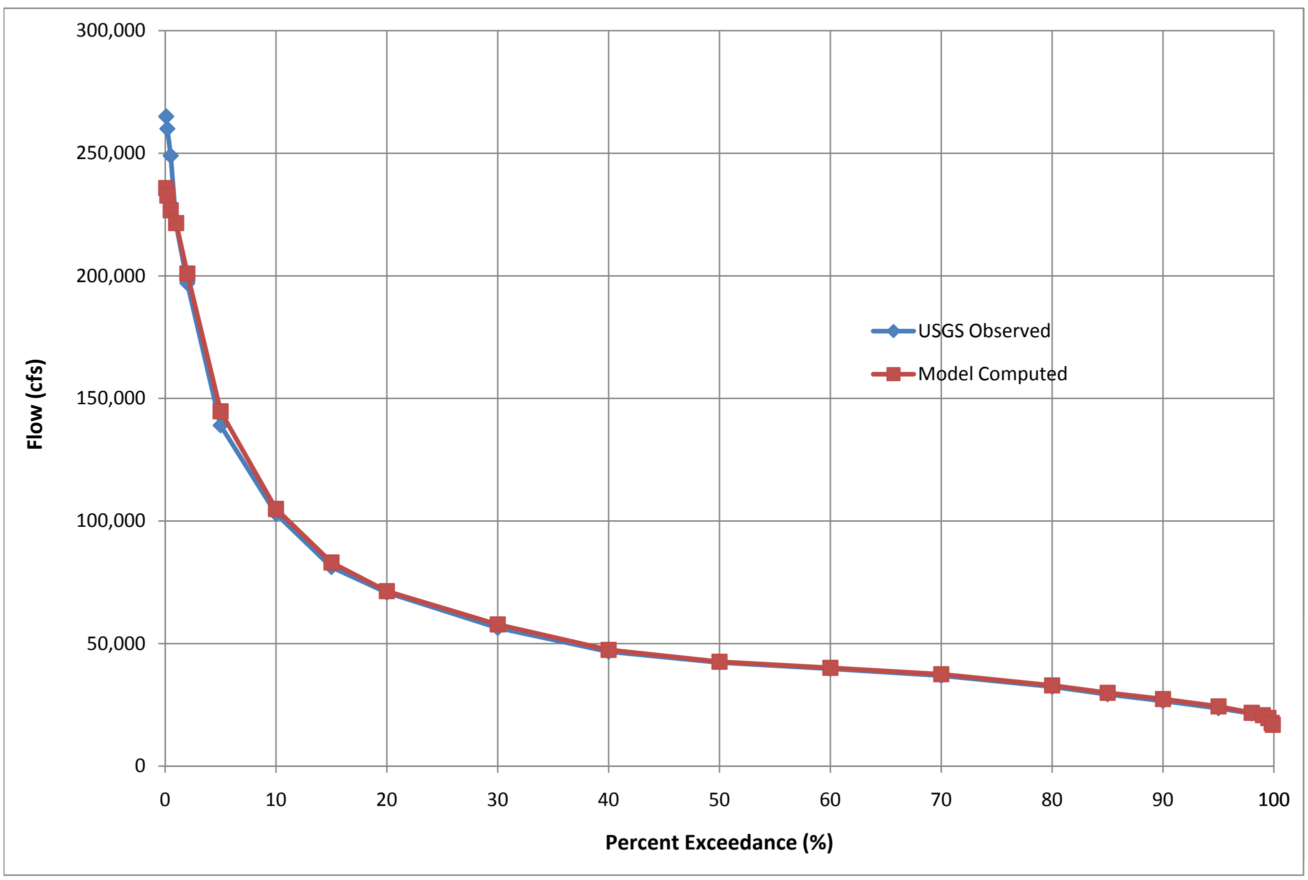
	Error (cfs)	Difference (%)
average	671	1%
median	891	2%
max	30,560	9%
min	-34,004	-13%

Greater than 1% Exeedance
(not including major floods)

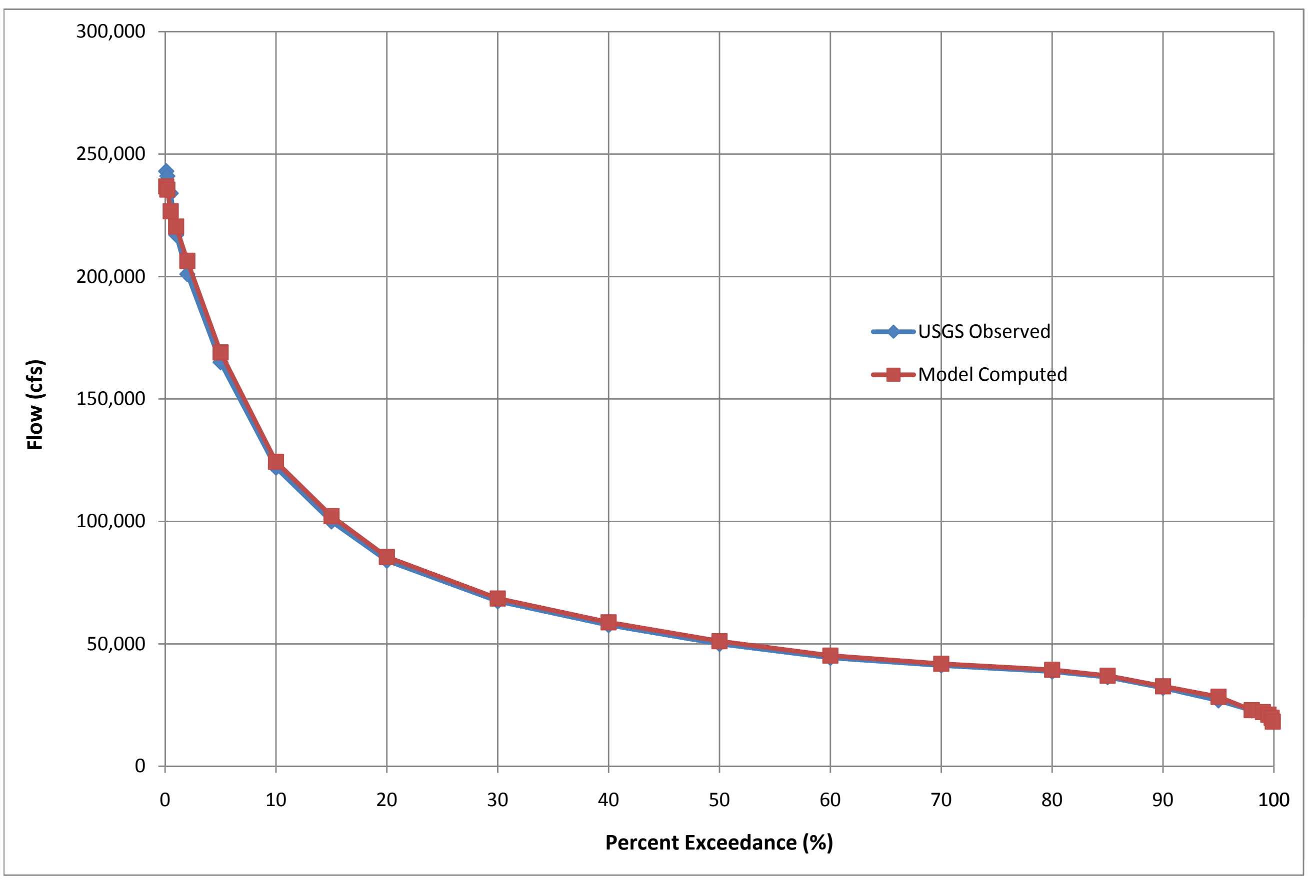
	Error (cfs)	Difference (%)
average	1,440	1%
median	992	2%
max	18,463	7%
min	-5,117	-9%



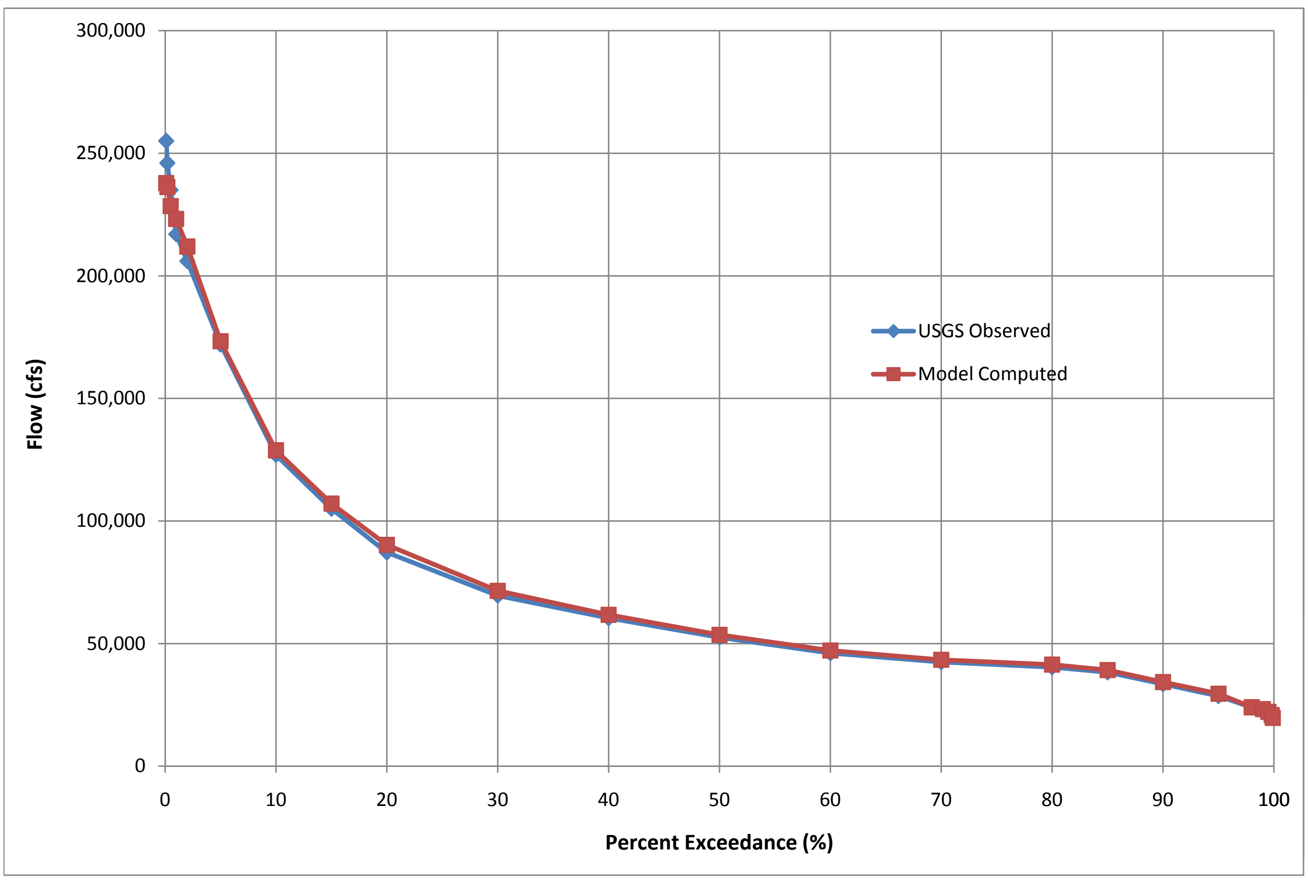
Gage Name	Percent Exceedance (%)	Flow - USGS Observed (cfs)	Flow - Model Computed (cfs)	Error (Comp-Obs) (cfs)	Percent Difference (%)
St. Joseph, MO	0.1	265,000	235,799	-29,201	-11%
	0.2	260,000	232,761	-27,239	-10%
	0.5	249,000	226,754	-22,246	-9%
	1	221,000	221,521	521	0%
	2	197,000	200,918	3,918	2%
	5	139,000	144,720	5,720	4%
	10	103,000	104,933	1,933	2%
	15	81,200	83,087	1,887	2%
	20	70,800	71,381	581	1%
	30	56,400	57,815	1,415	3%
	40	46,700	47,449	749	2%
	50	42,300	42,590	290	1%
	60	39,700	40,099	399	1%
	70	36,900	37,506	606	2%
	80	32,400	32,920	520	2%
	85	29,300	29,952	652	2%
	90	26,600	27,368	768	3%
	95	23,600	24,369	769	3%
	98	21,500	21,773	273	1%
	99	20,600	20,776	176	1%
	99.5	19,800	19,648	-152	-1%
	99.8	18,900	17,644	-1,256	-7%
	99.9	18,100	16,853	-1,247	-7%



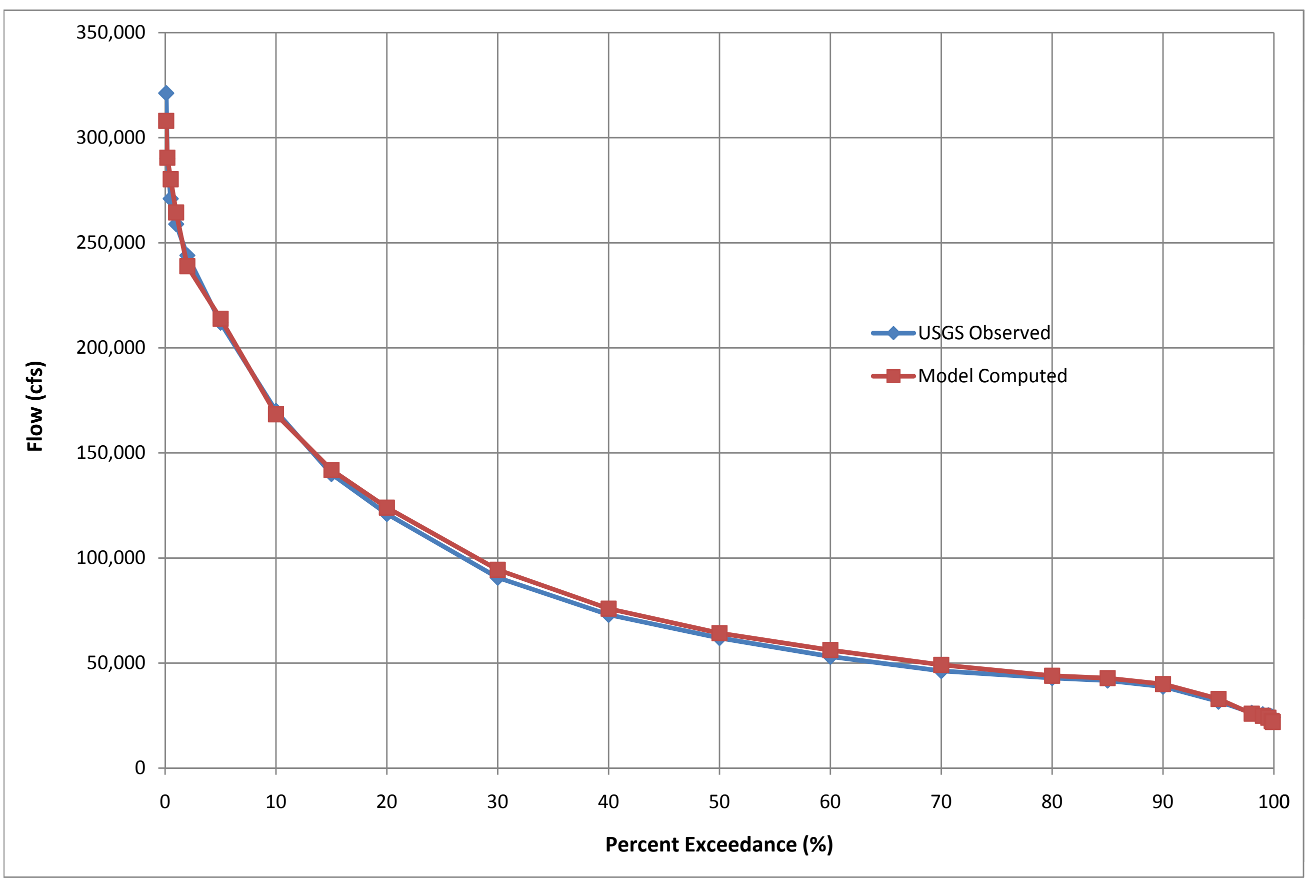
Gage Name	Percent Exceedance (%)	Flow - USGS Observed (cfs)	Flow - Model Computed (cfs)	Error (Comp-Obs) (cfs)	Percent Difference (%)
Kansas City, MO	0.1	243,000	236,793	-6,207	-3%
	0.2	241,000	235,506	-5,494	-2%
	0.5	234,000	226,668	-7,332	-3%
	1	217,000	220,389	3,389	2%
	2	201,000	206,400	5,400	3%
	5	165,000	169,042	4,042	2%
	10	122,000	124,343	2,343	2%
	15	100,000	102,144	2,144	2%
	20	84,000	85,503	1,503	2%
	30	67,400	68,512	1,112	2%
	40	57,700	58,818	1,118	2%
	50	50,000	51,114	1,114	2%
	60	44,300	45,191	891	2%
	70	41,200	41,879	679	2%
	80	38,700	39,418	718	2%
	85	36,400	36,965	565	2%
	90	32,000	32,669	669	2%
	95	26,900	28,403	1,503	6%
	98	22,800	22,974	174	1%
	99	21,900	22,168	268	1%
	99.5	21,000	21,089	89	0%
	99.8	20,262	19,786	-476	-2%
	99.9	19,500	18,268	-1,232	-6%



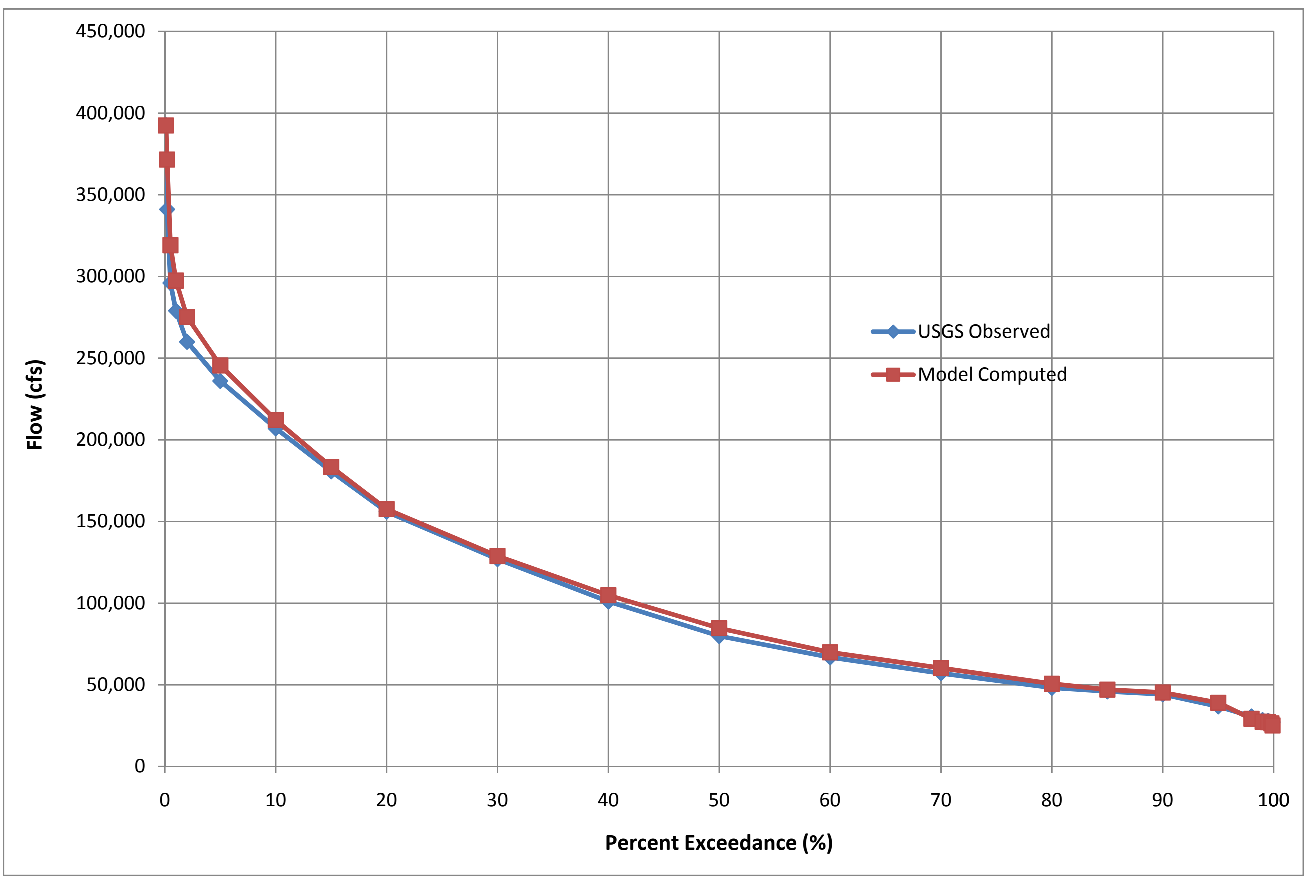
Gage Name	Percent Exceedance (%)	Flow - USGS Observed (cfs)	Flow - Model Computed (cfs)	Error (Comp-Obs) (cfs)	Percent Difference (%)
Waverly, MO	0.1	255,000	237,822	-17,178	-7%
	0.2	246,000	236,224	-9,776	-4%
	0.5	235,000	228,450	-6,550	-3%
	1	217,000	223,219	6,219	3%
	2	206,000	211,955	5,955	3%
	5	172,000	173,336	1,336	1%
	10	127,000	128,827	1,827	1%
	15	105,000	107,145	2,145	2%
	20	87,200	90,283	3,083	4%
	30	69,600	71,558	1,958	3%
	40	60,400	61,731	1,331	2%
	50	52,500	53,553	1,053	2%
	60	46,100	47,177	1,077	2%
	70	42,500	43,403	903	2%
	80	40,400	41,435	1,035	3%
	85	38,300	39,212	912	2%
	90	33,500	34,290	790	2%
	95	28,600	29,534	934	3%
	98	23,900	24,033	133	1%
	99	23,000	23,275	275	1%
	99.5	22,300	22,160	-140	-1%
	99.8	21,500	20,862	-638	-3%
	99.9	21,400	19,695	-1,705	-8%



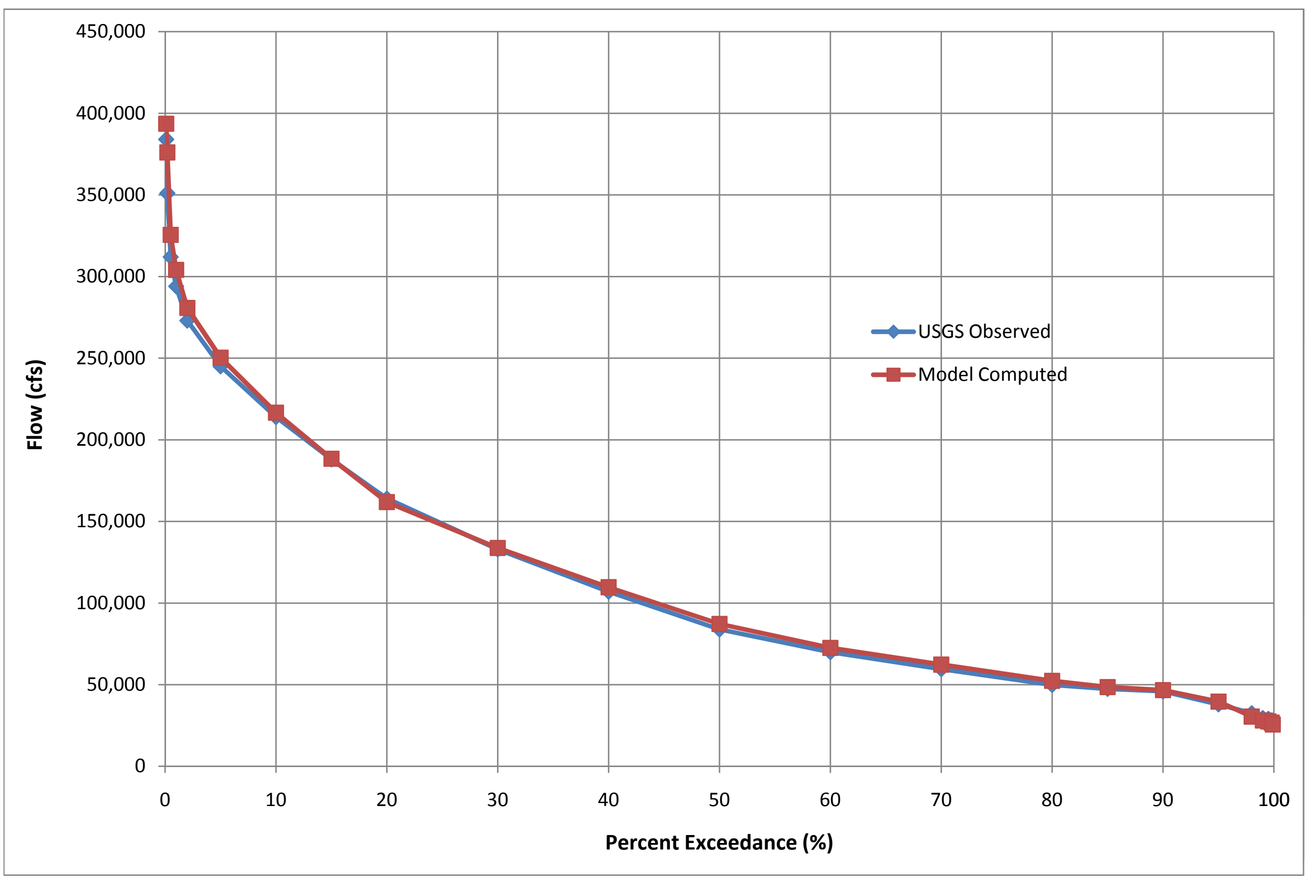
Gage Name	Percent Exceedance (%)	Flow - USGS Observed (cfs)	Flow - Model Computed (cfs)	Error (Comp-Obs) (cfs)	Percent Difference (%)
Boonville, MO	0.1	321,189	308,027	-13,162	-4%
	0.2	290,000	290,541	541	0%
	0.5	271,000	280,269	9,269	3%
	1	258,890	264,431	5,541	2%
	2	244,000	238,883	-5,117	-2%
	5	212,000	213,891	1,891	1%
	10	170,000	168,428	-1,572	-1%
	15	140,000	141,884	1,884	1%
	20	121,000	124,006	3,006	2%
	30	90,700	94,388	3,688	4%
	40	73,031	75,903	2,872	4%
	50	62,000	64,273	2,273	4%
	60	53,100	56,217	3,117	6%
	70	46,300	49,140	2,840	6%
	80	42,900	44,012	1,112	3%
	85	41,700	42,862	1,162	3%
	90	38,800	40,026	1,226	3%
	95	31,700	32,953	1,253	4%
	98	26,400	25,956	-444	-2%
	99	25,700	24,959	-741	-3%
	99.5	25,200	24,012	-1,188	-5%
	99.8	24,700	22,482	-2,218	-9%
	99.9	24,300	22,011	-2,289	-9%



Gage Name	Percent Exceedance (%)	Flow - USGS Observed (cfs)	Flow - Model Computed (cfs)	Error (Comp-Obs) (cfs)	Percent Difference (%)
Hermann, MO	0.1	392,481	392,392	-89	0%
	0.2	341,000	371,560	30,560	9%
	0.5	296,000	319,106	23,106	8%
	1	279,000	297,463	18,463	7%
	2	260,000	275,159	15,159	6%
	5	236,000	245,474	9,474	4%
	10	207,000	212,114	5,114	2%
	15	180,644	183,379	2,735	2%
	20	156,000	157,552	1,552	1%
	30	127,000	128,804	1,804	1%
	40	101,000	104,853	3,853	4%
	50	79,900	84,674	4,774	6%
	60	66,800	69,860	3,060	5%
	70	57,100	60,300	3,200	6%
	80	48,200	50,696	2,496	5%
	85	46,000	47,068	1,068	2%
	90	44,200	45,294	1,094	2%
	95	36,800	39,047	2,247	6%
	98	30,600	29,233	-1,367	-4%
	99	28,519	27,421	-1,098	-4%
	99.5	27,900	27,096	-804	-3%
	99.8	27,200	26,380	-820	-3%
	99.9	27,200	25,148	-2,052	-8%



Gage Name	Percent Exceedance (%)	Flow - USGS Observed (cfs)	Flow - Model Computed (cfs)	Error (Comp-Obs) (cfs)	Percent Difference (%)
St. Charles, MO	0.1	384,000	393,633	9,633	3%
	0.2	351,000	376,018	25,018	7%
	0.5	312,000	325,538	13,538	4%
	1	294,000	304,064	10,064	3%
	2	273,000	280,735	7,735	3%
	5	245,000	250,281	5,281	2%
	10	214,000	216,623	2,623	1%
	15	188,000	188,418	418	0%
	20	164,000	161,840	-2,160	-1%
	30	133,000	133,712	712	1%
	40	107,000	109,693	2,693	3%
	50	83,900	87,213	3,313	4%
	60	70,000	72,525	2,525	4%
	70	59,600	62,296	2,696	5%
	80	49,900	52,373	2,473	5%
	85	47,500	48,555	1,055	2%
	90	46,000	46,633	633	1%
	95	38,000	39,580	1,580	4%
	98	32,500	30,463	-2,037	-6%
	99	29,600	28,051	-1,549	-5%
	99.5	29,100	27,506	-1,594	-5%
	99.8	28,200	26,785	-1,415	-5%
	99.9	28,000	25,587	-2,413	-9%



Attachment 12

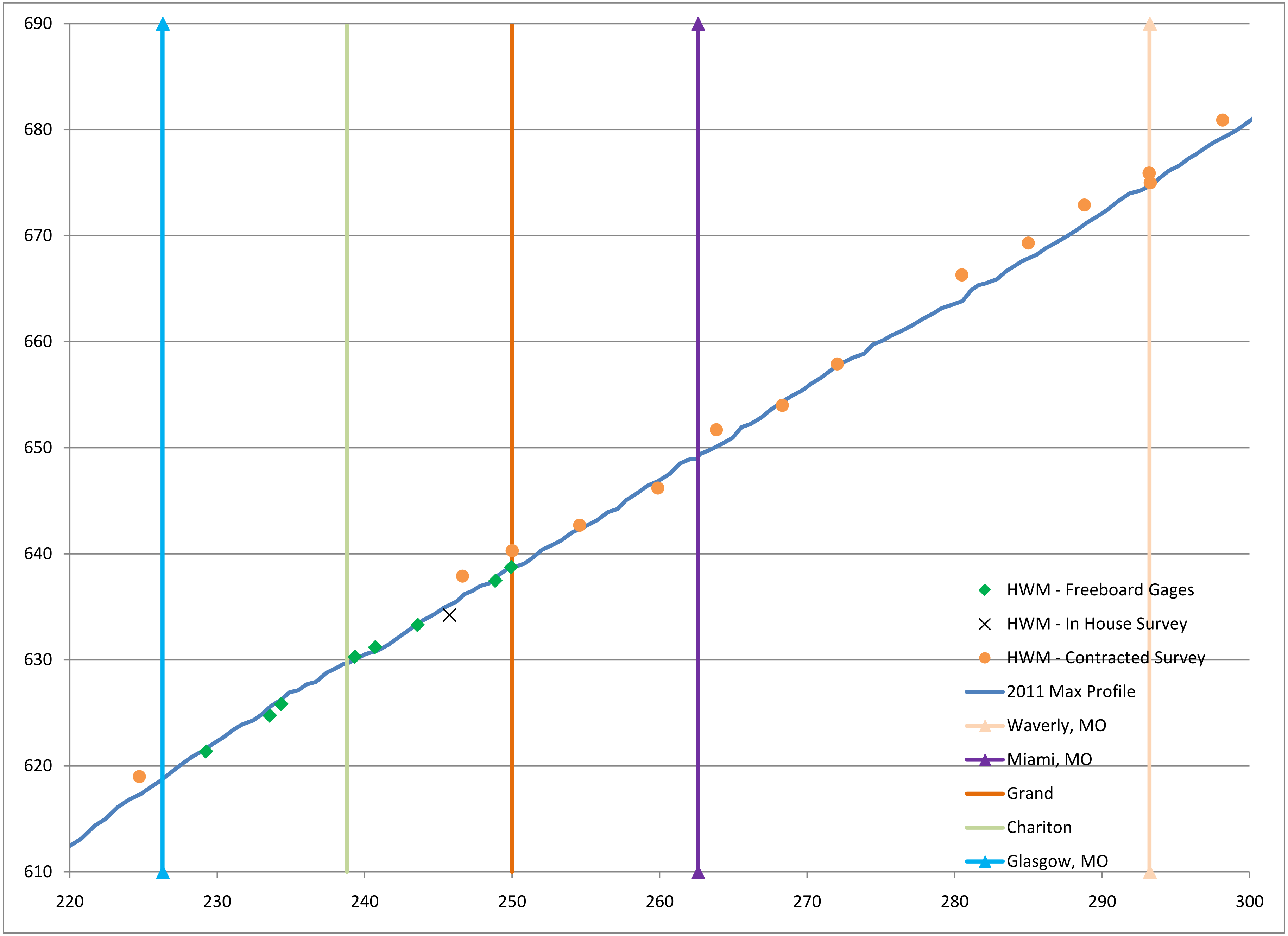
2011 Flood Calibration Results

2011 Calibration
Summary at Gages

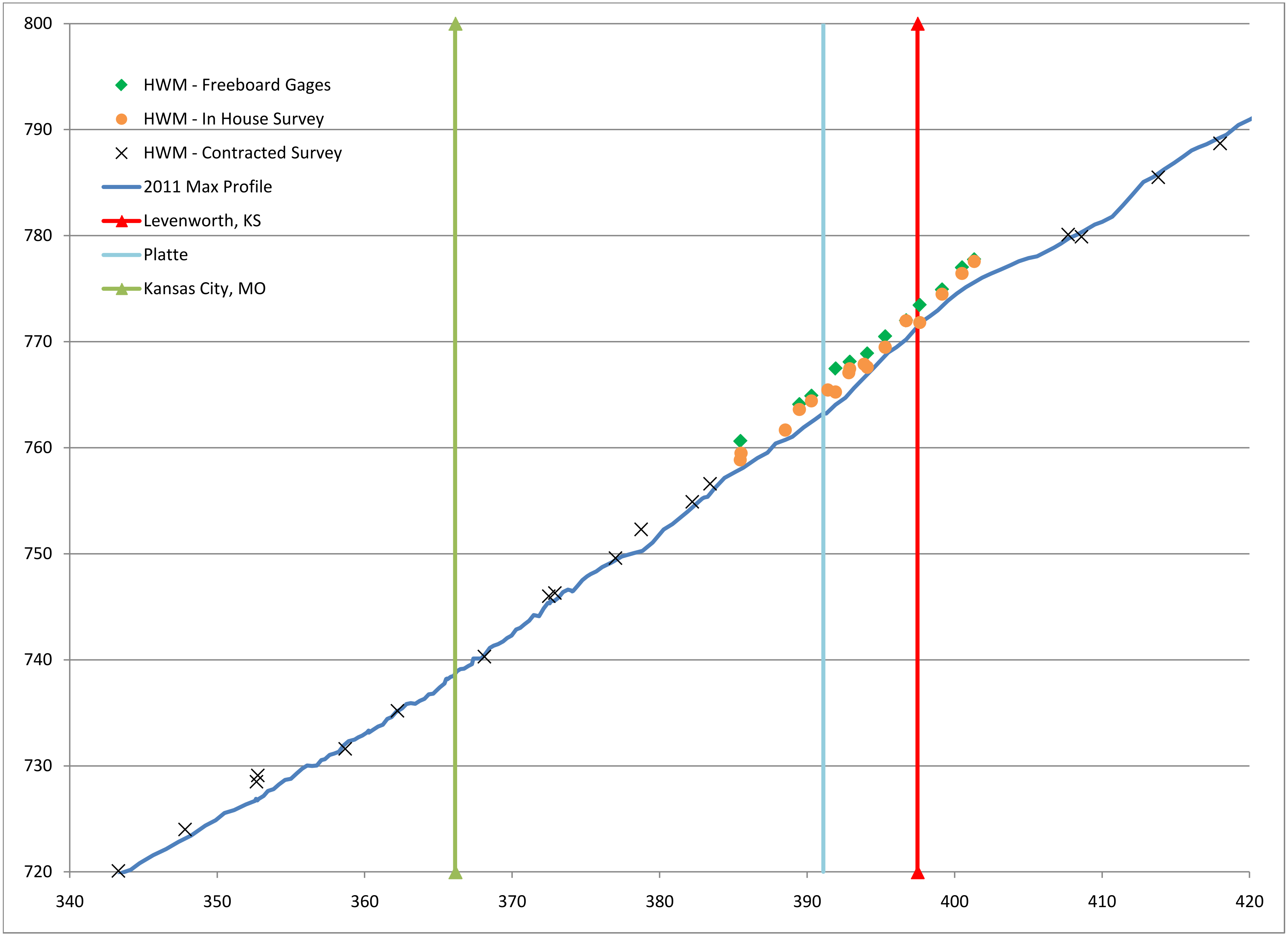
				Peak Stage (ft)		Stage Error (ft)	Peak Flow (cfs)			Flow Error (cfs)	Flow Error
Gage	River Mile	Computed Elev	Datum	Computed	Obs*	(Comp-Obs)	Computed	Obs*	Day*	(Comp-Obs)	(%)
Rulo, NE	498.04	864.79	838.16	26.63	27.26	-0.63	229,100**	328,000	6/27/2011	-98,900	-30%
St. Joseph, MO	448.17	818.20	789.27	28.93	29.97	-1.04	237,200	277,000	6/28/2011	-39,800	-14%
Atchiston, KS	422.58	793.36	762.84	30.52	31.00	-0.48	227,800	255,000	6/29/2011	-27,200	-11%
Leavenworth, KS	397.48	771.43	742.47	28.96	30.80	-1.84	232,100	249,000	6/30/2011	-16,900	-7%
Kansas City, MO	366.14	738.72	706.68	32.04	32.65	-0.61	239,300	245,000	7/10/2011	-5,700	-2%
Sibley, MO	336.50	714.49	684.40	30.09	31.10	-1.01	236,600	256,000	7/7/2011	-19,400	-8%
Napoleon, MO	329.05	706.67	680.53	26.14	27.60	-1.46	236,700	262,000	7/10/2011	-25,300	-10%
Waverly, MO	293.22	674.89	646.17	28.72	30.75	-2.03	238,600	265,000	7/9/2011	-26,400	-10%
Miami, MO	262.60	649.07	621.73	27.34	28.80	-1.46	238,200	259,000	7/10/2011	-20,800	-8%
Glasgow, MO	226.30	618.74	586.65	32.09	31.78	0.31	276,500	255,000	6/30/2011	21,500	8%

* Observed stage, flow, and date are from the 2011 Post Flood Report
** Computed flow at Rulo includes the flow in the River + flow over Hwy 159
*** Downstream of Glasgow the 2011 Flood did not rank significantly, no assessment in Post Flood Report

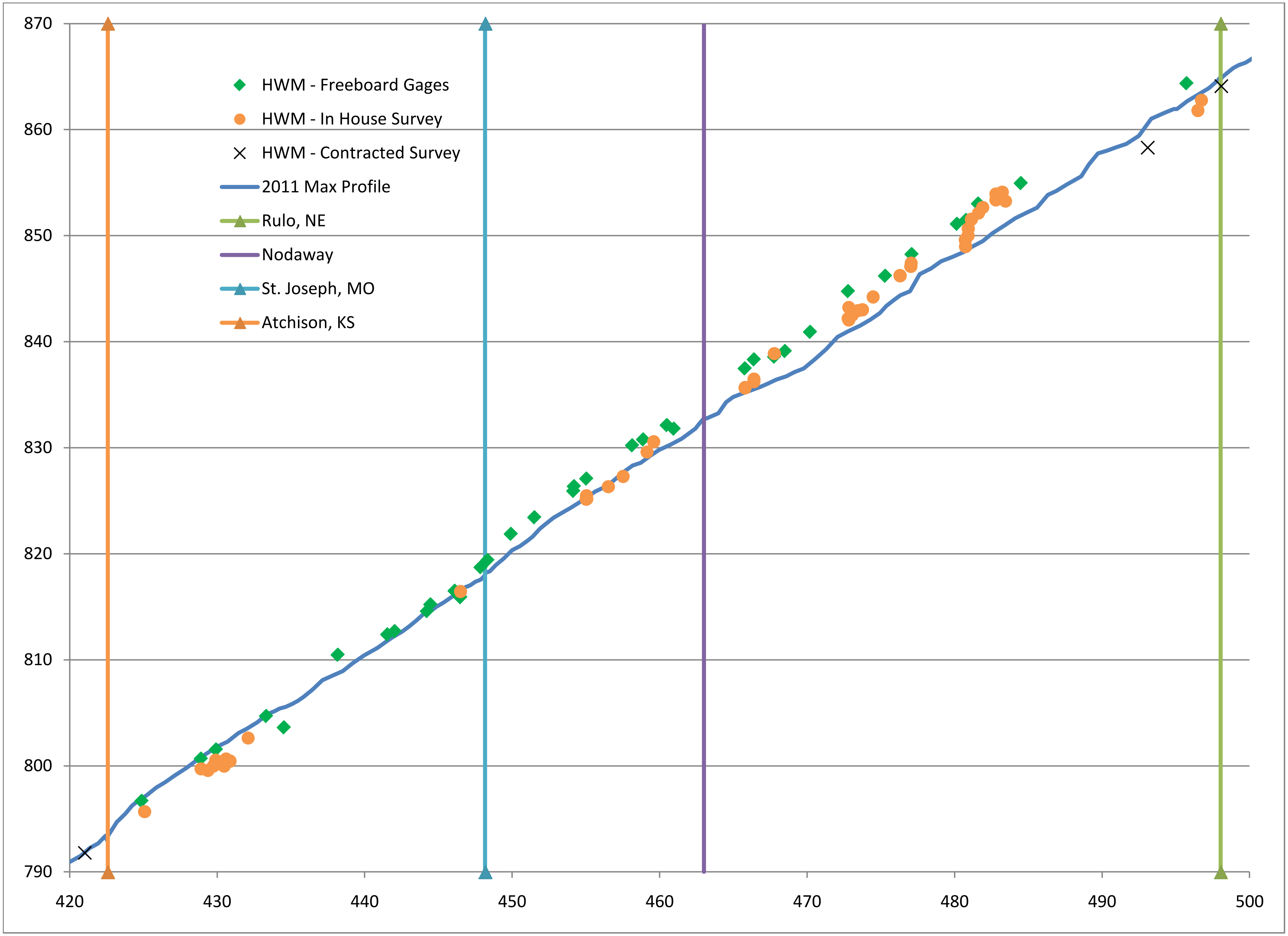
				All Gages						All Gages	
						Error				Error	Error
						(ft)				(cfs)	(%)
					Average	-1.0			Average	-25,890	-9%
					Std Dev	0.7			Std Dev	30,420	10%
					Max Positive	0.3			Max Positive	21,500	8%
					Max Negative	-2.0			Max Negative	-39,800	-30%



690
680
670
660
650
640
630
620
610
220
230
240
250
260
270
280
290
300
HWM - Freeboard Gages
HWM - In House Survey
HWM - Contracted Survey
2011 Max Profile
Waverly, MO
Miami, MO
Grand
Chariton
Glasgow, MO



800
790
780
770
760
750
740
730
720
340
350
360
370
380
390
400
410
420
HWM - Freeboard Gages
HWM - In House Survey
HWM - Contracted Survey
2011 Max Profile
Levenworth, KS
Platte
Kansas City, MO



HWM - Freeboard Gages
HWM - In House Survey
HWM - Contracted Survey
2011 Max Profile
Rulo, NE
Nodaway
St. Joseph, MO
Atchison, KS
870
860
850
840
830
820
810
800
790
420
430
440
450
460
470
480
490
500

Attachment 13

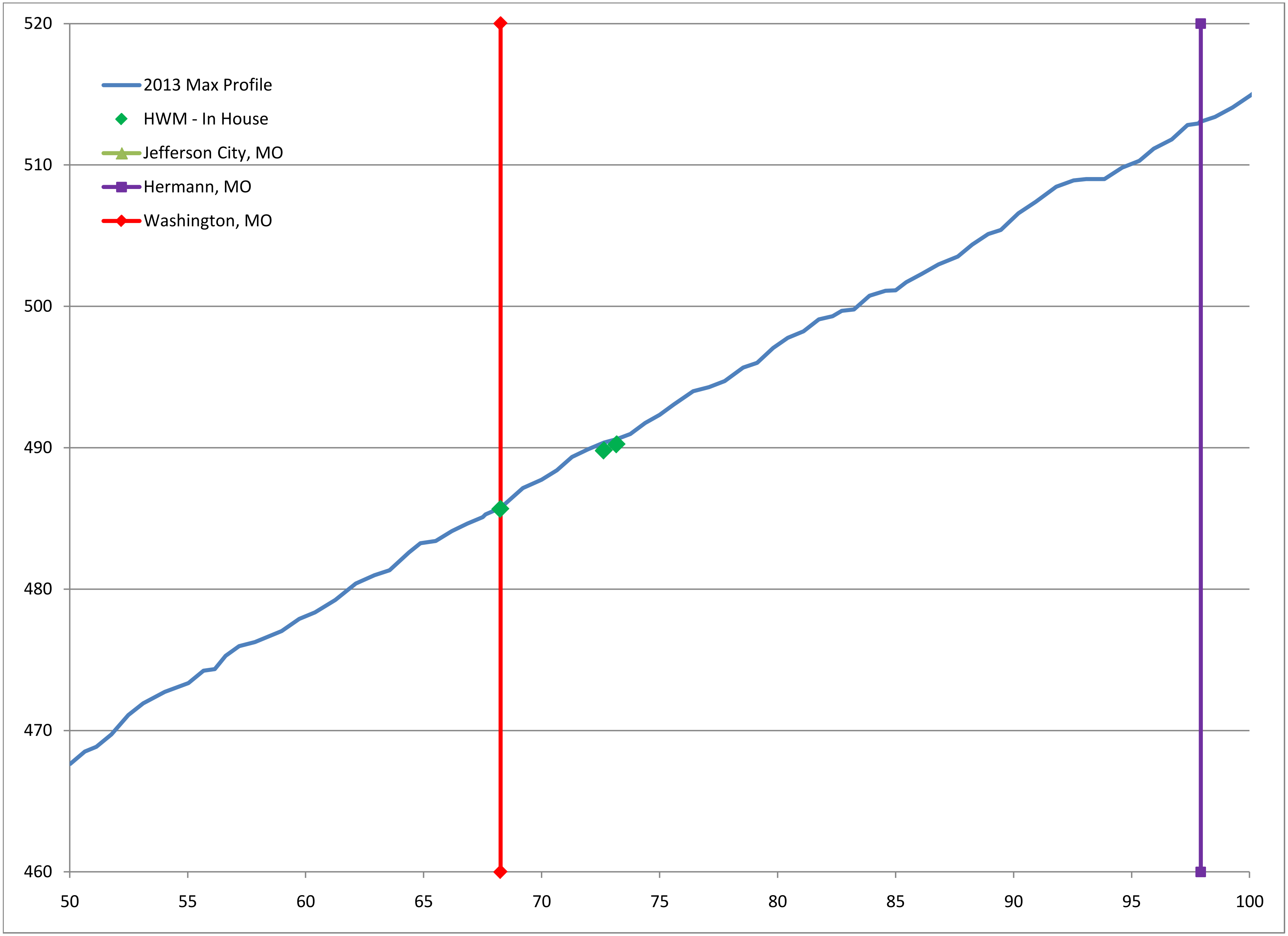
2013 Flood Calibration Results

2013 Calibration									
Summary at Gages									
		Peak Stage (ft)		Stage Error (ft)	Peak Flow (cfs)			Flow Error (cfs)	Flow Error
Gage	River Mile	Computed	Obs*	(Comp-Obs)	Computed	Obs*	Day/Time*	(Comp-Obs)	(%)
Jefferson City, MO (stage only)	143.86	550.97	551.01	0.0		n/a	6/2/13 5:30 PM		
Hermann, MO	97.93	513.12	514.64	-1.5	393,676	457,000	6/1/13 11:30 PM	-63,324	-14%
Washington, MO (stage only)	68.26	485.74	486.26	-0.5		n/a	6/2/13 9:30 AM		
St. Charles, MO	27.78	446.84	447.27	-0.4	397,681	409,000	6/2/13 7:00 PM	-11,319	-3%

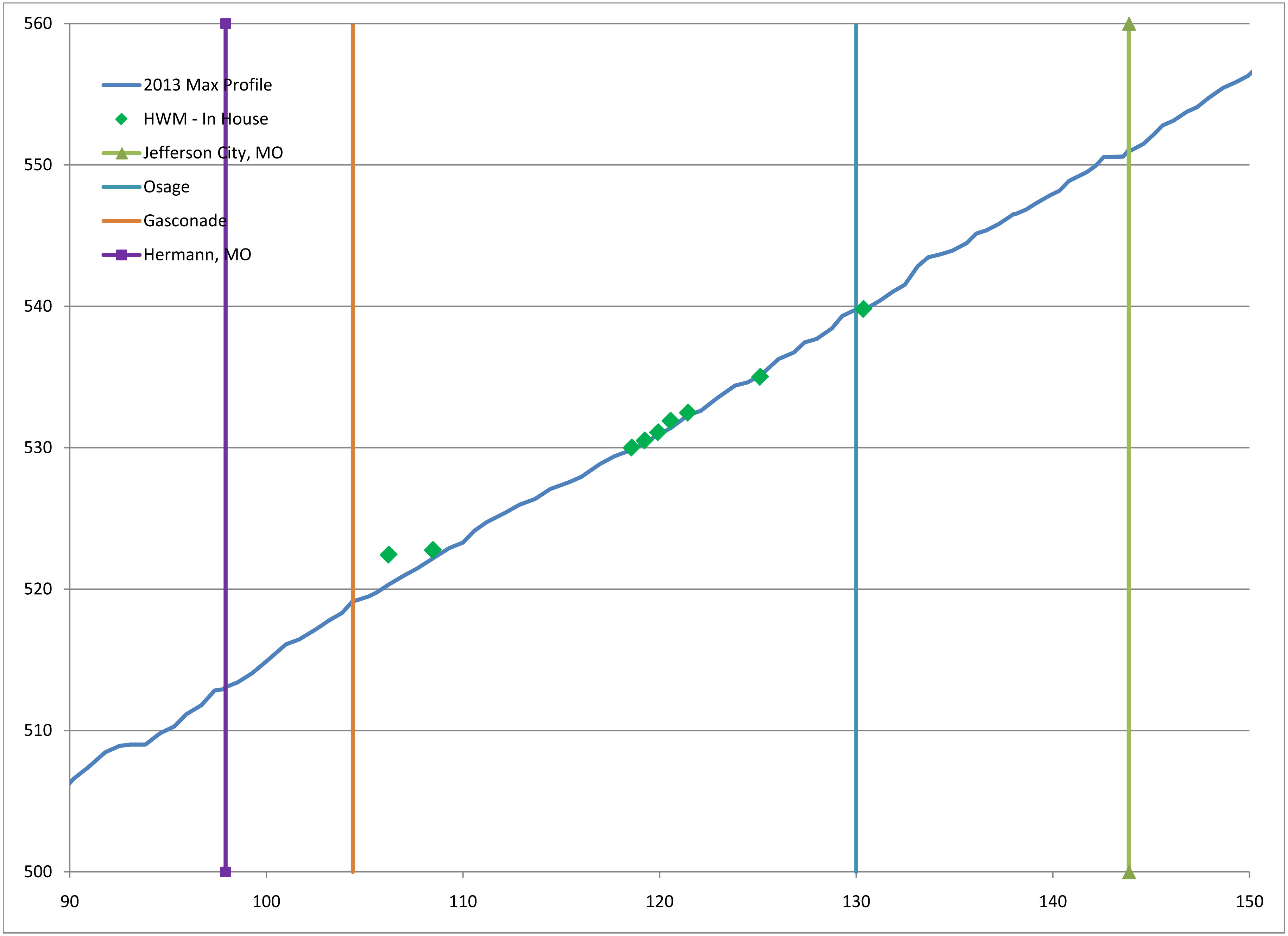
* Observed value read from USGS instintaneous data (as contained in MoRiverObs.dss)

** Upstream of Jefferson City the 2013 Flood was not as severe, no high water marks collected

	All Gages
	Error (ft)
Average	-0.63
Std Dev	0.63
Max Positive	-0.04
Max Negative	-1.52



2013 Max Profile
HWM - In House
Jefferson City, MO
Hermann, MO
Washington, MO
520
510
500
490
480
470
460
50
55
60
65
70
75
80
85
90
95
100


2013 Max Profile
HWM - In House
Jefferson City, MO
Osage
Gasconade
Hermann, MO
560
550
540
530
520
510
500
90
100
110
120
130
140
150

2011 High Water Marks						All HWMs
Summary						
					Average	-0.2
					Std Dev	0.7
					Max Positive	0.6
					Max Negative	-2.1
RM	HWM	Source	Levee	Sta	Computed WSE	Error (Comp-Obs)
130.4	539.8	In House	Jacobs/Tebbets		539.71	-0.1
125.1	535.0	In House	Tebbets		535.09	0.1
121.5	532.5	In House	Chamois 1		532.31	-0.2
120.6	531.9	In House	Chamois 2		531.38	-0.5
119.9	531.1	In House	Chamois 2		530.92	-0.2
119.3	530.5	In House	Chamois 2		530.37	-0.1
118.6	530.0	In House	Chamois 2		529.85	-0.1
108.5	522.8	In House	Morrison		522.18	-0.6
106.2	522.4	In House	Morrison		520.33	-2.1
73.2	490.2	In House	Holtmeier		490.61	0.4
72.6	489.8	In House	Holtmeier		490.37	0.6
68.3	485.7	In House	Tuque		485.74	0.1