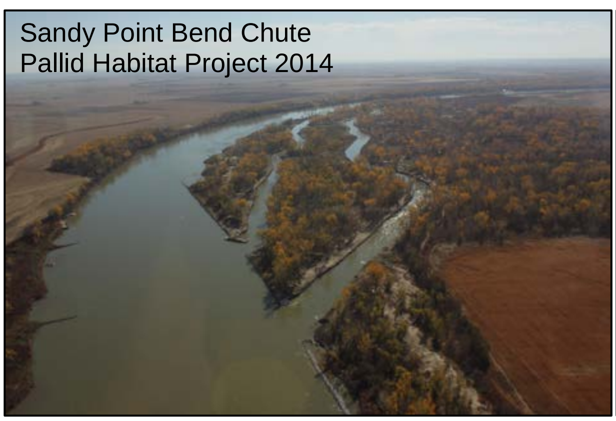
Missouri River Recovery Program Management Plan Environmental Impact Statement
HEC-RAS Modeling Alternatives Report
FINAL

US Army Corps of Engineers ®

Gavins Point Dam
Spillway Release
June 2011



Sandy Point Bend Chute
Pallid Habitat Project 2014

Deer Island Widening During
Construction Oct 2014
Pallid Habitat Project

July 2018

USACE, Northwestern Division
Omaha and Kansas City Districts

EXECUTIVE SUMMARY

Hydrologic analysis was performed to support the Missouri River Recovery Program (MRRP) evaluation for the Management Plan (ManPlan) and integrated Environmental Impact Statement (EIS). This document summarizes alternative analysis performed using HEC-RAS (RAS).

In a previous effort (USACE 2015b), five separate HEC-RAS models were developed for the Missouri River reaches from downstream of Ft. Peck Dam to the mouth at St. Louis (Ft Peck Dam to Lake Sakakawea, Garrison Dam to Lake Oahe, Ft Randall Dam to Lewis and Clark Lake, Gavins Point Dam to Rulo, NE, and Rulo, NE, to the mouth at St. Louis). The model geometry and calibration efforts vary for each of the model reaches. Models were constructed from the best available geometry and calibrated. Each model can generally be thought of as representative of 2012 conditions as documented in USACE 2015b.

Hydrologic analysis utilized a period of record (POR) methodology to evaluate alternative conditions. Flows were developed for the Missouri River basin for the period of record used in the RAS analysis from March 1930 through December 2012. Therefore, the POR consists of 82 full years of record from 1931 through 2012 with a model stability and initial startup period in 1930. All flows were corrected to current level depletions to reflect basin water development. Therefore, comparison of hydrologic model results from either HEC-ResSim (ResSim) or RAS to observed conditions is not possible.

Alternative conditions were simulated through the reservoir system using ResSim. The results were used to provide reservoir releases to the RAS models. ResSim discussion is provided in a separate report.

Alternative analysis was performed using the five separate HEC-RAS calibrated current condition models. The HEC-RAS models were revised for each alternative by adding pallid sturgeon habitat to the current condition models downstream of Gavins Point Dam. All HEC-RAS modeling efforts are for a static geomorphic condition only, the model does not adjust geometry during the 82 year POR. A qualitative Year 15 analysis was performed that determined the relative performance of alternatives in the future would be similar to that for the current condition.

Results from the HEC-RAS models were provided to Human Considerations (HC) team members for the comparison of alternative conditions. All model results are based on a simulated period of record routed through reservoir models to test reservoir rule changes and alternative condition river geometries. POR flows represent a hypothetical condition with all flows corrected to current water development levels within the basin. All model results are based on the stage-flow relationship developed from the model calibration period and differences in future habitat quantities between alternatives. Dynamic conditions in the future such as variations in the stage-flow relationship due to past and future aggradation / degradation are not included. Model geometry for all alternatives, including the no action, has been altered from the current condition. In summary, ***none of the alternative analysis hydrologic model results should be used to estimate flow-frequency, stage-frequency, or stage-flow relationships. Model results are***

suitable for comparison between study alternatives, comparison to historic events or observed conditions is not meaningful.

Statistical comparisons of results between alternatives were performed for the POR. Critical factors that should be considered when comparing results between alternatives include:

1) Small number of pulses
2) Reservoir releases from the Mainstem System are calculated by ResSim
3) The habitat additions to the river geometry
4) Storage changes within the river and floodplain
5) Timing of reservoir releases and downstream inflows, combined with HEC-RAS routing, during the POR
6) HEC-RAS output interval may mask minor differences
7) Comparison of results on a POR basis can mask impacts due to flow changes. For example, an alternative with high benefits over the POR could have larger impacts than other alternatives in a single year.

The performed hydrologic evaluation of the complex Missouri River System provides a powerful alternative analysis tool for assessing differences between alternatives. However, comparison between alternatives should recognize that minor and insignificant differences can occur due to many factors that were identified for RAS modeling analysis uncertainty and limitations summarized as:

1) The dynamic Missouri River, with significant sediment transport, could affect future geometry and constructed habitat
2) Climate change and natural climate variability could affect historic POR inflows
3) POR has limited of number of implemented pulses for several alternatives which restricts the ability to evaluate possible impacts
4) No analysis was performed to assess the potential for change in flood risk on the Missouri River System (reservoirs and levees)
5) Riverbed and floodplain aggradation / degradation trends are not included in static HEC-RAS modeling
6) POR flow record required the estimation of ungaged inflow, also flow sources could be different
7) Levee performance and interior drainage analysis was simplified
8) Simplifying assumptions were necessary when constructing HEC-RAS models

An interior drainage evaluation was also performed at four selected locations using the HEC-RAS alternative condition model and HEC-HMS models that were used to develop inflows. Uncertainties regarding the interior drainage pump operation and culvert sediment levels were evaluated with a sensitivity analysis. Analysis demonstrated that these inputs have a high impact on results.

A brief channel capacity evaluation was performed to provide an indication of the flow rate at which flow begins to leave the main channel and enter the floodplain. The estimated channel capacity varies considerably within the reach and may change over time. Following the channel capacity estimates, an evaluation of flood potential was conducted at two locations. Analysis determined that alternative flow pulse releases exceed channel capacity within the Fort Randall reach downstream of the Niobrara River confluence. Within the Nebraska City area, the change in flood potential varies with some flow pulse alternatives showing a change in flood potential.

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ACRONYMS

AM……………………Adaptive Management

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

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

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

ESH…………………..Emergent Sandbar Habitat

HC .... Human Considerations

HEC......................... Hydrologic Engineering Center

IRC .... Interception Rearing Complex

ManPlan………………Management Plan

MRRMP-EIS………… Missouri River Recovery Management Plan and Environmental Impact Statement

MRRP .... Missouri River Recovery Program

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

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

POR......................... Period of Record

RAS .... River Analysis System (computer model distributed by HEC, referred to as HEC-RAS)

ResSim……………....Reservoir Simulation Software (computer model distributed by HEC, referred to as HEC-ResSim)

RPA…………………..Reasonable and Prudent Actions

SWH………………….Shallow Water Habitat

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

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

USFWS……………….United States Fish and Wildlife Service

USGS .... United States Geological Survey

1 INTRODUCTION

A series of hydrologic evaluations were conducted in support of the Missouri River Recovery Program (MRRP) evaluation for the Management Plan (ManPlan) and Integrated Environmental Impact Statement (EIS). The various hydrologic evaluations are summarized in the *Missouri River Recovery Program Management Plan Environmental Impact Statement, Summary of Hydrologic Engineering Analysis* (USACE 2018e). This report provides documentation on the HEC-RAS (RAS) alternative analysis.

The Missouri River unsteady RAS model was created as a base model for planning studies which could be used to simulate and analyze broad scale watershed alternatives. Model geometry development and calibration for the existing conditions is documented in *Missouri River Unsteady RAS Model Calibration Report* (USACE 2015b).

The objective of the RAS alternative modeling was to simulate the Management Plan alternatives which include both geometry and reservoir operation flow changes.

Outputs from the RAS modeling effort were used by 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. Evaluation of the RAS modeling outputs was performed by the Human Considerations (HC) team.

2 STUDY ALTERNATIVES

Management actions, and the intended environmental effects, are described in detail within the EIS study documentation. These actions are briefly summarized in this document to provide context for those aspects of the alternatives that were implemented with RAS modeling.

Numerous management actions were developed to benefit the least tern and piping plover by providing suitable emergent sandbar habitat (ESH) within the reservoir and open river reaches. Omaha District has an existing ESH Program and has mechanically constructed ESH in the Gavins Point river reach and upper Lewis and Clark Lake at various locations during the period from 2004 to 2010. Methods to create new ESH (flow manipulation and mechanical construction) are conducted by redistributing sand within the existing river cross section. No stabilization of sandbars is included. Therefore, sandbar habitat tends to decay with time as sandbar elevation decreases due to normal sediment processes. Experience has also shown that the conveyance of the section is the same with no net change in flow area. For these reasons, ESH related actions of the alternatives were not included in the RAS models and are not included in the brief alternative summary within this document.

- Multiple management actions, consisting of revisions to reservoir flow releases intended to create emergent sandbar habitat, were evaluated with HEC-ResSim.
- Since the sediment processes are dynamic, the ESH creation actions were not evaluated with the RAS models.

- ResSim computed flow releases were evaluated with the RAS models.

Management actions were also developed to add varying rates of additional pallid habitat in the Missouri River between Ponca, NE, and the mouth. Consequently, the new pallid habitat only affects the downstream two RAS models (Gavins to Rulo and Rulo to St. Louis). Pallid habitat, also known as shallow water habitat (SWH), was assumed to consist of interception and rearing complexes (IRC habitat) and backwater areas. Pallid habitat is created by removing river alluvial material. SWH constructed via chutes and IRC habitat are flow through and will alter current river conveyance. Backwater habitat consists of small areas that are not flow through and do not alter conveyance.

- Pallid habitat geometry changes were evaluated with the RAS models.

The following sections describe the alternatives in additional detail. Note that only those portions of each alternative that pertain to the RAS models is included in the description. Additional actions, such as ESH creation, pallid propagation, and etc. are not included in this brief description.

2.1 Background Information From 2003 BiOp

Omaha and Kansas City Districts have conducted numerous actions to implement reasonable and prudent alternatives from the 2003 BiOp (USFWS 2003). A brief background is summarized in this section to provide relevant information. Refer to the Affected Environment section of the Missouri River Recovery Management Plan and Environmental Impact Statement (MRRMP-EIS) for additional information regarding historic actions.

2.1.1 Shallow Water Habitat

SWH was defined as areas on the Missouri River downstream of Ponca, NE, (RM 752) with flow depths less than 5 feet and velocities less than 2 fps measured at the flow defined as the 50% exceedance discharge from the August flow duration curve(s). The SWH restoration goal stated in the 2003 BiOp is to achieve an average of 20-30 acres of SWH per mile of river. Historically, over 100 acres of SWH per river mile existed prior to Corps construction activities for the BSNP and mainstem Missouri River dams. The 2003 BiOp estimated that approximately 3,000 acres of SWH existed on the system. This goal would require the restoration of 12,035 acres to 19,565 acres (20-30 acres per mile). SWH has been constructed by both Omaha and Kansas City District within the main channel of the river via structure modifications and off-channel using side channel chutes or backwaters in the adjacent floodplain. The most recent accounting of Missouri River SWH was conducted with 2012 / 2013 survey data and is reported in the *2014 Shallow Water Habitat Accounting Report* (USACE 2014).

2.1.2 Emergent Sandbar Habitat

ESH acreage targets were established to meet least tern and piping plover biological metrics (population and productivity) for the open river reaches downstream of Garrison, Fort Randall, and Gavins Point dams and also Lewis and Clark Lake. To meet these targets, intermittent mechanical construction and vegetative management actions were performed within the Omaha District in the period through 2010. Since the 2011 flood event, which resulted in significant sandbar creation, no ESH mechanical construction actions have been performed.

2.2 Alternative 1 – No Action

Under the No-Action Alternative, the MRRP would continue to be implemented as it is currently. The current program does not implement all Reasonable and Prudent Actions (RPA) included in the 2003 Amended BiOp (USFWS 2003).

Reservoir operations and basin conditions have changed throughout the period of record (e.g. reservoirs were closed at different times, operational criteria has been updated, etc.). In order to estimate impacts that occur due to the operational changes in the various alternatives, each alternative, including the No Action, was evaluated with the same operational criteria when not operating for an alternative's specific criteria (e.g. ESH release, spawning cue, etc.). To accomplish this, a No Action simulation representing the current reservoir operations under the current basin conditions was evaluated. Although the modeled results for each alternative will not capture all of the real-time decisions and adjustments, the impacts provide an assessment of the differences between modeled alternatives.

2.2.1 Pallid Habitat

Pallid habitat construction would continue to occur as part of the SWH program. The SWH restoration goal as outlined in the 2003 Amended BiOp (USFWS, 2003) is to achieve an average of 20–30 acres of SWH per river mile. Under the No-Action Alternative, the USACE would achieve the low end of this acreage target (i.e., 20 acres per river mile between Ponca, Nebraska, and the mouth). This equates to a total of 15,060 acres of SWH. Existing habitat on the system was used to determine the additional pallid habitat required. Since acreage above the target goal by segment is not required, adjusting for excess acreage in the Osage River to the mouth reach and acreage change since the 2012 RAS model geometry creation due to construction actions, 3,999 acres of additional habitat is to be created (Table 2-1). For purposes of evaluating potential impacts to the human environment, modeling assumed that the additional SWH acreage would be created as follows (Table 2-2):

- Approximately 3,519 acres of in-channel SWH created through channel or top-width widening. A conceptual width of 250 feet was assumed for projects between Ponca and Rulo (20 projects encompassing 48 river miles) and 300 feet for projects downstream of Rulo (24 projects encompassing 57 river miles). Actual project width and size will vary by site.
- Approximately 480 acres of off-channel backwaters, assuming 8 new backwaters with each creating 60 acres of SWH.

Table 2-1. Summary of Projected Shallow Water Habitat Creation Under the No-Action Alternative

River Reach	RM Start	RM End	Miles in Reach	20 acres per mile of SWH	Existing Acreage[1]	Acreage Change 2012 to 2015[2]	2015 Target Acres of SWH[3]
Ponca to Sioux City	753	735	18	360	120	0	240
Sioux City to Platte River	735	595	140	2,800	1,682	97	1,021
Platte River to Rulo	595	498	97	1,940	1,290	-22	672
Rulo to Kansas River	498	367	131	2,620	1,270	221	1,129
Kansas River to Osage River	367	130	237	4,740	3,710	93	937
Osage River to Mouth	130	0	130	2,600	2600[4]	NA[4]	0
Total			**753**	**15,060**	NA[4]	**389[5]**	**3,999[5]**

1 Existing acreage estimate derived from 2014 SWH Accounting Report (USACE 2014).
2 Includes additional chute habitat added between 2012 RAS model geometry creation and 2015 to include acreage change due to construction actions. Acreage reduction results due to estimated habitat change and response to construction.
3 Target acreage included in RAS models See appendices D & E for further details.
4 Acreage above 2600 ignored, no new acreage required in this segment since existing exceeds the 20 ac/mi goal.
5 Excludes acreage in the Osage River to mouth reach for purposes of demonstrating alternative 1 acreage increase.

Table 2-2. Projected Composition of Shallow Water Habitat Creation Type Under the No-Action Alternative

River Reach	Target Acres of SWH	Channel Widening[1]			Backwaters[2]	
		Acres	Miles	# of Projects	Acres	# of Projects
Ponca to Sioux City	240	180	5.9	2	60	1
Sioux City to Platte River	1,021	601	19.8	9	420	7
Platte River to Rulo	672	672	22.2	9	0	0
Rulo to Kansas River	1,129	1,129	31.1	14	0	0
Kansas River to Osage River	937	937	25.8	10	0	0
Osage River to Mouth	0	0	0	0	0	0
Total	**3,999**	**3,519**	**105**	**44**	**480**	**8**

1 Acreage amounts assume a top width of 250 feet for projects between Ponca and Rulo and 300 feet for projects downstream of Rulo.

2 Assumes 60 acres of SWH are created by each project.

Table 2-3 summarizes the amount of land acquisition that was assumed to be required to implement the identified amount of SWH. Land acquisition was estimated by comparing additional acreage with existing public lands. This estimate required a rough layout of potential sites using the exclusion criteria that considers aspects such as surrounding infrastructure. Refer to Appendix D and E for a thorough discussion of the excluded area criteria. Land acquisition estimates were provided to the HC team for use with preparing alternative cost estimates.

Table 2-3. Land Acquisition Requirements to Implement Early Life History Pallid Sturgeon Habitat Under Alternative 1

River Reach	Target Acres of SWH	Additional Land Required – Habitat Only (acres)	Additional Land Required – Total (acres)*
Ponca to Sioux City	240	240	1,848
Sioux City to Platte River	1,021	0	0
Platte River to Rulo	672	0	0
Rulo to Kansas River	1,129	675	5,198
Kansas River to Osage River	937	0	0
Osage River to Mouth	0	0	0
Total	**3,999**	**915**	**7,046**

* For estimating purposes, it was assumed that 7.7 acres of land acquisition are required for every 1 acre of habitat needed. This is based on historic implementation data and accounts for factors such as parcel size and other real estate acquisition considerations.

2.2.2 Reservoir Operations

Under Alternative 1 (Alt 1), the Missouri River Mainstem Projects would continue to be operated as they are currently. Operations within the ResSim model were set up to closely follow the Master Manual that is used during real-time operations of the System; however, the model does have limitations and cannot capture all real-time decisions that occur.

For the No-Action Alternative, the USACE assumed implementation of the plenary spring pulse as described in the Master Manual (USACE 2006). The bimodal Gavins Point spring pulse plan includes flow pulses in March and May. The magnitude of both the March and May Gavins Point spring pulses would be constrained by the Gavins Point spring pulse downstream flow limits. Pulse magnitude varies with James River inflows and is also constrained by multiple factors. Refer to the ResSim Alternatives report (USACE 2018a) for additional details regarding the reservoir operations changes.

2.3 ALTERNATIVE 2 – U.S. FISH AND WILDLIFE SERVICE 2003 BIOLOGICAL OPINION PROJECTED ACTIONS

Alternative 2 represents the USFWS interpretation of the management actions that would be implemented as part of the 2003 Amended BiOp RPA (USFWS 2003). Whereas the No-Action Alternative only includes the continuation of management actions the USACE has implemented to date for BiOp compliance, Alternative 2 includes additional iterative actions and expected actions that the USFWS anticipates would ultimately be implemented through Adaptive Management (AM) and as impediments to implementation were removed.

2.3.1 Pallid Habitat

Pallid habitat construction would occur as part of the SWH program. Under Alternative 2, the USACE would achieve the upper end of the 20–30 acres of SWH per river mile acreage target (i.e., 30 acres per river mile between Ponca, Nebraska, and the mouth). This equates to a total of 22,590 acres of SWH. Existing SWH projects and natural habitat resulted in a total of 11,832 acres (11,325 plus 507), leaving 10,758 acres to be created (Table 2-4). For the purposes of evaluating potential impacts to the human environment, modeling assumed that the additional SWH acreage would be created as follows (Table 2-5):

- Approximately 9,858 acres of in-channel SWH would be created through channel widening. A conceptual width of 250 feet was assumed for projects between Ponca and Rulo (60 projects encompassing 118.2 river miles) and 450 feet for projects downstream of Rulo (48 projects encompassing 115 river miles). Actual project width and size will vary by site.
- Approximately 900 acres of off-channel backwaters, assuming 15 new backwaters with each creating 60 acres of SWH.

Table 2-4. Summary of Projected Shallow Water Habitat Creation Under Alternative 2

River Reach	RM Start	RM End	Miles in Reach	30 acres per mile of SWH	Existing Acreage[1]	Acreage Change 2012 to 2015[2]	2015 Target Acres of SWH[3]
Ponca to Sioux City	753	735	18	540	120	0	420
Sioux City to Platte River	735	595	140	4,200	1,682	97	2,421
Platte River to Rulo	595	498	97	2,910	1,290	-22	1,642
Rulo to Kansas River	498	367	131	3,930	1,270	221	2,439
Kansas River to Osage River	367	130	237	7,110	3,710	93	3,307
Osage River to Mouth	130	0	130	3,900	3,253	118	529
		Total	**753**	**22,590**	**11,325**	**507**	**10,758**

1 Existing acreage estimate derived from 2014 SWH Accounting Report (USACE 2014).
2 Includes additional chute habitat added between 2012 RAS model geometry creation and 2015 to include acreage change due to construction actions. Acreage reduction results due to estimated habitat change and response to construction.
3 Target acreage included in RAS models for alternative. See appendices D & E for further details.

Table 2-5. Projected Composition of Shallow Water Habitat Creation Type Under Alternative 2

River Reach	Target Acres of SWH	Channel Widening[1]			Backwaters[2]	
		Acres	Miles	# of Projects	Acres	# of Projects
Ponca to Sioux City	420	240	7.9	4	180	3
Sioux City to Platte River	2,421	1,761	58.1	32	660	11
Platte River to Rulo	1,642	1,582	52.2	24	60	1
Rulo to Kansas River	2,439	2,439	44.7	19	0	0
Kansas River to Osage River	3,307	3,307	60.6	25	0	0
Osage River to Mouth	529	529	9.7	4	0	0
Total	**10,758**	**9,858**	**233**	**108**	**900**	**15**

1 Acreage amounts assume a top width of 250 feet for projects between Ponca and Rulo and 450 feet for projects downstream of Rulo.

2 Assumes 60 acres of SWH are created by each project.

Land acquisition to implement the SWH requirements described is summarized in Table 2-6.

Table 2-6. Land Acquisition Requirements to Implement Early Life History Pallid Sturgeon Habitat Under Alternative 2

River Reach	Target Acres of SWH	Additional Land Required – Habitat Only (acres)	Additional Land Required – Total (acres)*
Ponca to Sioux City	420	420	3,234
Sioux City to Platte River	2,421	925	7,123
Platte River to Rulo	1,642	675	5,198
Rulo to Kansas River	2,439	1,985	15,285
Kansas River to Osage River	3,307	1,932	14,876
Osage River to Mouth	529	0	0
Total	**10,758**	**5,937**	**45,716**

* For estimating purposes, it was assumed that 7.7 acres of land acquisition are required for every 1 acre of habitat needed. This is based on historic implementation data and accounts for factors such as parcel size and other real estate acquisition considerations.

2.3.1 Reservoir Operations

Alternative 2 (Alt 2) represents the USFWS interpretation of the management actions that would be implemented as part of the 2003 Amended BiOp RPA (USFWS 2003). Operational criteria include different early and late spring spawning cues (March and May), low summer flows, and a maximum winter release limit.

The USFWS determined in the 2003 Amended BiOp (USFWS 2003) that restoration of a normalized river hydrograph below Gavins Point Dam was necessary to avoid jeopardizing the continued existence of the pallid sturgeon. Several biologically relevant features were identified for a flow action below Gavins Point Dam including (1) flows to cue spawning that are sufficiently high for an adequate duration; and (2) flows that provide for connection of low-lying lands adjacent to the channel. The spawning cue release from Gavins Point Dam would be bimodal (i.e., consisting of two separate flow pulses) and would be implemented in every year if conditions are met. The USFWS 2003 Amended BiOp (USFWS 2003) also called for modification to System operations to allow for summer flows that are sufficiently low to provide SWH as rearing, refugia, and foraging areas for larval, juvenile, and adult pallid sturgeon. Refer to the ResSim Alternatives report (USACE 2018b) for additional details regarding the reservoir operations changes.

2.3.2 Floodplain Connectivity

The BiOp geometry integrates floodplain connectivity along with SWH criteria set forth in the 2003 Amended BiOp. Coordination with the USFWS produced a Planning Aid Letter (USFWS 2015) detailing the modeling assumptions for the BiOp alternative. A total of 100,000 acres of SWH and floodplain connectivity were assumed for both districts. To calculate the goal amount of only floodplain connectivity, 22,590 acres (30 ac/mi * 753 mi) was subtracted from the 100,000 acres to obtain 77,410 acres.

Mapping of existing floodplain connectivity was performed by using a RAS model calibrated to 2012 conditions to calculate a water surface profile for the 20% annual chance exceedance event (20% ACE or 5-year). The 5-year flow input for the model was obtained from the 2003 Upper Mississippi River System Flow Frequency Study (USACE 2003). Existing floodplain connectivity acres (147,652 acres) surpass the total acres available for floodplain connectivity (100,000 acres) therefore, no changes were made to the RAS models. Refer to Appendix D and E for specific details in each of the RAS modeling reaches regarding the floodplain connectivity analysis.

2.4 PALLID HABITAT ACTIONS COMMON TO ALTERNATIVES 3–6

Under Alternatives 3–6, the USACE would create three high-quality spawning habitat sites. The spawning sites are at locations separate from the IRC habitat. Although spawning habitat specific characteristics are undefined, it was assumed for the purposes of this analysis that these areas do not significantly modify conveyance and were not included in the RAS model.

Under Alternatives 3–6, construction of pallid habitat would occur following the IRC concept. Best available science indicates that future acreage required to construct IRCs would most likely be achieved through channel widening. For the purposes of evaluating potential impacts to the human environment, modeling assumed that about 3,380 acres of channel widening would be implemented to create IRCs under Alternatives 3–6 (Table 2-7). A conceptual width of 250 feet was assumed for projects between Ponca and Rulo and 300 feet for projects downstream of Rulo.

Table 2-7. Summary of Projected IRC Creation Under Alternatives 3–6

River Reach	River Mile Start	River Mile End	Miles in Reach	Target Acres of IRC habitat[1]
Ponca to Sioux City	753	735	18	0
Sioux City to Platte River	735	595	140	276
Platte River to Rulo	595	498	97	585
Rulo to Kansas River	498	367	131	670
Kansas River to Osage River	367	130	237	1,389
Osage River to Mouth	130	0	130	460
			Total	**3,380**

1 All acreage achieved through channel widening. Acreage amounts assume a top width of 250 feet for projects between Ponca and Rulo and 300 feet for projects downstream of Rulo.

Land acquisition to implement the requirements described is summarized in Table 2-8.

Table 2-8. Land Acquisition Requirements to Implement IRC Under Alternatives 3–6

River Reach	Target Acres of SWH	Existing Public Lands Available for Habitat Development (acres)[1]	Additional Land Required – Habitat Only (acres)	Additional Land Required – Total (acres)[2]
Ponca to Sioux City	0	420	0	0
Sioux City to Platte River	276	276	0	0
Platte River to Rulo	585	585	0	0
Rulo to Kansas River	670	454	216	1,664
Kansas River to Osage River	1,389	1,375	14	108
Osage River to Mouth	460	460	0	0
Total	**3,380**	**3,150**	**230**	**1,772**

1 Existing public lands includes USACE, USFWS, and state conservation owned lands. Acreage was based on identifying government owned lands that may be appropriate for habitat development; however, these areas do not necessarily represent actual locations of future habitat development.

2 For estimating purposes, it was assumed that 7.7 acres of land acquisition are required for every 1 acre of habitat needed. This is based on historic implementation data and accounts for factors such as parcel size and other real estate acquisition considerations.

2.5 Alternative 3 – Mechanical Construction Only

Alternative 3 (Alt 3) consists of mechanical construction of emergent sandbar habitat (ESH). Operational criteria consist of removing the early and late spring spawning cues in Alt 1. Pallid habitat construction is as described in Table 2-7.

2.6 ALTERNATIVE 4 – SPRING HABITAT-FORMING FLOW RELEASE

Under Alternative 4 (Alt 4), the early and late spring spawning cues in Alt 1 are removed from the operational criteria and a spring ESH-creating reservoir release from Gavins Point and Garrison is added. During the period of flow releases for ESH-creation release, ResSim downstream flood targets are increased to increase the potential for ESH-creation releases to be implemented.

Alternative 4 reservoir operations would be similar to Alternative 1 (current operations), with the addition of a high spring release designed to create ESH for the least tern and piping plover. If implementation conditions are met (USACE 2018b), the habitat-forming flow release would be implemented on April 1 with a release of up to 60 kcfs out of Gavins Point Dam, and as often as every four years. To achieve the Gavins Point Dam release, Fort Randall Dam releases would be increased a similar amount as Gavins Point and releases from Garrison Dam would be approximately 17.5 kcfs less than the Gavins Point release.

Implementation conditions (system storage, downstream flood restrictions) could result in partial flow releases with truncated duration. The duration of the release would increase as release magnitude is decreased to meet habitat forming objectives.

Pallid habitat construction for Alternative 4 is as described in Table 2-7.

2.7 ALTERNATIVE 5 – FALL HABITAT-FORMING FLOW RELEASE

Alternative 5 (Alt 5) removes the early and late spring spawning cues in Alt 1 and adds a fall ESH-creating reservoir release from Gavins Point and Garrison to the operational criteria. While the ESH-creation release is occurring from Gavins Point, flood targets are increased to allow the ESH-creation release the opportunity to run.

Alt 5 reservoir operations would be similar to Alternative 1 (current operations), with the addition of a high fall release designed to create ESH for the least tern and piping plover. If implementation conditions are met (USACE 2018a), the habitat-forming flow release would be implemented on October 17 with a release of up to 60 kcfs out of Gavins Point Dam, and as often as every four years. To achieve the Gavins Point Dam release, Fort Randall Dam releases would be increased a similar amount as Gavins Point and releases from Garrison Dam would be approximately 17.5 kcfs less than the Gavins Point release.

Implementation conditions (system storage, downstream flood restrictions) could result in partial flow releases with truncated duration. The duration of the release would increase as release magnitude is decreased to meet habitat forming objectives.

Pallid habitat construction for Alternative 5 is as described in Table 2-7.

2.8 ALTERNATIVE 6 – PALLID STURGEON SPAWNING CUE

Alternative 6 (Alt 6) replaces the early and late spring spawning cues in Alt 1 with different spawning cues. The early spring spawning cue in Alt 6 occurs at the same time as the early spring spawning cue in Alt 1 but with a higher peak release. The late spring spawning cue in Alt 6 occurs later in May than the late spring spawning cue in Alt 1 and has a larger peak release.

Under Alternative 6, the USACE would attempt a spawning cue release every 3 years consisting of a bimodal pulse in March and May. For the March pulse, the peak Gavins Point release is double the navigation release that occurs on the day the pulse is initiated. For the May pulse, the peak Gavins Point release would be double the steady release that occurs on the day the pulse is initiated (May 18). Both pulses would be formed by increasing flows by 2,200 cfs /day to the peak, maintain the peak for two days, and then receded by 1,700 cfs/day to the navigation target flow. These spawning cue releases would not be started or would be terminated whenever flood targets are exceeded.

Pallid habitat construction for Alternative 6 is as described in Table 2-7.

3 PERIOD OF RECORD ANALYSIS

Hydrologic analysis utilized a period of record (POR) methodology to evaluate alternative conditions with the RAS and ResSim modeling effort for the MRRMP-EIS. As used in hydrologic models for flood-runoff analysis, period of record analysis refers to applying a hydrologic model to simulate a continuous period of record of streamflow. Detailed documentation of the data development methods and data sources conducted to create the POR for all hydrologic models is provided in *Missouri River Recovery Management Plan Time Series Data Development for Hydrologic Modeling* (USACE 2018c).

3.1 METHODOLOGY

Flows for all required inflow points to all hydrologic models were developed for the Missouri River basin for the period of record used in the ManPlan Study analysis for the period from March 1930 through December 2012. When gage data was unavailable, other methods were used to determine inflow for the entire POR. Estimated daily flow for the POR was used to provide all RAS model inflows. Although the hydrologic models provided results from a portion of 1930, HC team analysis was only performed for the 82 year period from January 1, 1931 through December 31, 2012.

POR development requires relatively sophisticated hydrologic models capable of simulating all extremes of the hydrologic cycle, including detailed simulation of flood events, drought years, and seasonal fluctuations. Due to study needs, the POR was assembled using daily flow values. Assembling the immense data set within the large Missouri River basin study area to accurately include all inflows, evaporation, and other consumptive water use required extensive data collection and processing from multiple sources. The final POR input data set allows accurate simulation of the MRRMP-EIS base condition and alternative conditions.

3.2 RESULTS

Summary results are presented in the POR documentation report (USACE 2018d). Regarding the POR flow data set:

- Various methods were used to assemble the POR flow record for each model.

- All flows were corrected to current level depletions to reflect water use within the basin. Therefore, comparison of hydrologic model results from either ResSim or RAS to observed conditions is not possible.
- Although the hydrologic models provide results from a portion of 1930, an 82 year POR was used for HC analysis from 1931 through 2012.

4 HEC-RAS MODELING OF ALTERNATIVES

Hydrologic model development was conducted to create a robust suite of models suitable for study use. Outputs of the RAS models were used in concert with other modeling programs such as HEC-Ecosystem Functions Model (HEC-EFM) and HEC-Flood Impact Analysis (HEC-FIA) to perform impacts analysis.

RAS is designed to perform one-dimensional hydraulic calculations for a full network of natural and constructed channels. Common outputs include stage, duration/timing of inundation, water velocities, flow areas/routes, water temperature, and sediment loads. Unsteady flow analysis was chosen as the method of hydraulic modeling due to the need to analyze time series stage and flow data. Both the biological considerations (e.g., seasonal habitat requirements) and the human considerations (e.g., potential changes in flows that could affect flood risk, agricultural practices, and etc.) are affected by the timing of river flows. RAS was used to more accurately route discharges from reservoirs and tributaries to points downstream and to simulate impacts of mechanical changes in river channel geometry. These models simulate how proposed alternatives and management actions would impact river stage and discharge over a wide range of basin hydrologic conditions.

The purpose of the 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 management actions or alternatives (e.g. channel reconfiguration and/or flow management). HEC-RAS is used extensively throughout the world and is an appropriate model for this EIS. The baseline or existing conditions models were modified to represent a future condition under the No Action and action alternatives.

An overview of the steps necessary to complete the alternative condition modeling is summarized as:

a) Revise ResSim model for each flow alternative
b) Revise RAS models to reflect new pallid habitat
c) Using the POR flows, simulate ResSim for each alternative
d) Using the POR flows, ResSim model reservoir releases and pool levels, and the revised RAS model geometries, simulate each alternative through the suite of RAS models
e) Provide summary output at key locations from the ResSim and RAS models for use with the HC team analysis

The Missouri River is a dynamic system that is changing constantly within the study area, which extends from Ft. Peck Dam downstream to the Missouri River mouth at St. Louis. Some areas have experienced continued degradation since 2012 while other areas have experienced aggradation. Regardless, all Alternatives were modeled with HEC-RAS using the same geometry and the comparison between the Alternatives is valid.

The HEC-RAS models were based on the best available channel survey information and calibrated to 2012 conditions. Local effects on stage due to temporary changes in river conditions, including ice jams, ice cover, and transient sandbar dynamics, are not included within the HEC-RAS model. These temporary effects often cause river stage changes of several feet. However, for the purposes of alternative comparison, including transient effects is not relevant (e.g. the formation of an ice jam has the same effect on all alternatives). Calibration accuracy within the HEC-RAS models varies by location but is generally within 0.5 to 1 foot accuracy for normal and low flows.

4.1 Previously Created HEC-RAS Models

RAS models were previously created for the study with the intent to revise the models in the future for alternative condition analysis. Refer to the *Missouri River Recovery Program Management Plan Environmental Impact Statement Existing Conditions Unsteady RAS Model Calibration* Report (USACE 2015b) for a detailed description of HEC- RAS model development and calibration. The RAS models for the calibration condition within each reach were based on geometry and calibration representative of the current, or approximately 2012, river conditions.

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 RAS models for discrete reaches of the Missouri River. These reaches are: Fort Peck Dam to Garrison Dam; Garrison Dam to Oahe Dam; 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. The boundary between the Kansas City and Omaha Districts is at Rulo, NE, therefore the Gavins Point to the mouth reach has an overlap from Nebraska City, NE to St. Joseph, MO in order to provide an accurate transition of flows between the two model reaches.

Figure 4-1 provides the locations of the individual RAS models. The Oahe Dam to Big Bend Dam and Big Bend Dam to Fort Randall Dam reaches were not modeled in RAS due to the lack of riverine conditions between the dams.

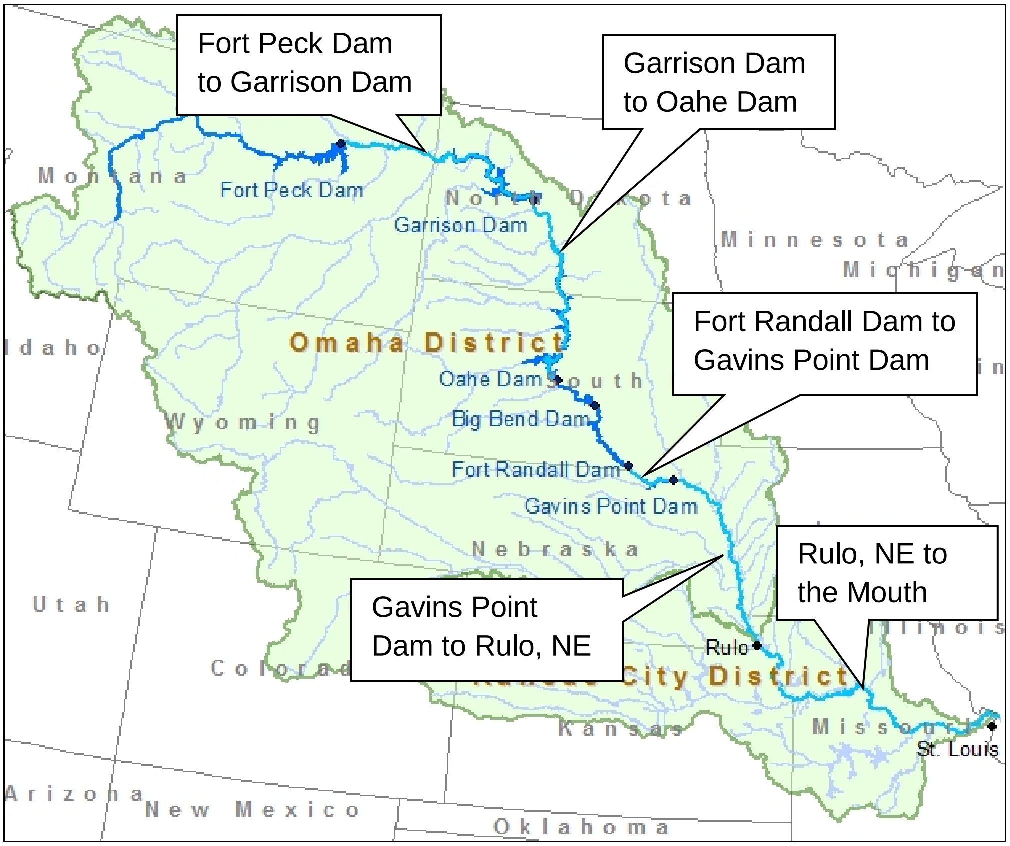

Figure 4-1. Location of RAS Modeled Reaches

Modifications were performed to implement RAS modeling of alternatives. Study alternatives were applied to the RAS models using the following guidance:

- Since the sediment processes are dynamic with generally no net change in cross sectional area, the ESH creation actions were not evaluated with the RAS models.
- ResSim computed flow releases were evaluated with all five of the RAS models for each alternative.
- Pallid habitat geometry changes were evaluated in the two RAS models downstream of Gavins Point Dam only
- Backwater pallid habitat was not modeled, as these areas, with a single river connection and an orientation that is typically not aligned with flow, do not significantly alter overall floodplain conveyance

4.2 Geometry Changes for Alternative Analysis

Three geometries were created for the alternatives analysis within the suite of RAS models. Since no pallid habitat actions are necessary upstream of Ponca, NE, only the two RAS models used for study analysis that are located downstream of Gavins Point Dam (Gavins to Rulo, NE and Rulo, NE to the mouth) were modified with geometry revisions.

4.2.1 Changes to Existing (2012) Calibration Model

During RAS model development, several intermediate geometries were constructed in order to obtain the final base geometry before adding in the various pallid habitat alternatives. These modifications were necessary to develop an updated current condition model that includes changes to the 2012 model geometry that occurred in both Omaha and Kansas City Districts. Changes consisted of adding habitat projects constructed since the 2012 model creation and revising existing chute invert and width elevations to reflect estimations of future modifications. Refer to the RAS modeling Appendices D and E for specific details on changes within each model.

4.2.2 Pallid Habitat

The pallid habitat configurations that were modeled to implement the alternatives, previously described in the study alternative section, are summarized as:

1. **No Action** - Assumes habitat construction activities follow current practices to achieve 20 acres/mile of SWH, the minimum target specified in the 2003 Amendment to the 2000 Biological Opinion.
2. **Biological Opinion as Projected (BiOp)** - Guidance from the USFWS was provided to create a geometry which represents an ideal implementation of the 2003 Biological Opinion. It assumes habitat construction accomplishes 30 acres/mile of SWH, and performs at a wider range of flows including a summer low, median August, and spring pulse. Floodplain connectivity was evaluated, but the requirement was met so no changes to the RAS geometry were necessary. A thorough description of the floodplain connectivity evaluation is contained within Appendices D and E.
3. **Interception-Rearing Complexes (IRC)** - SWH construction activities proceed based on findings made by the Effects Analysis (EA) team. It assumes habitat construction accomplishes 260 acres/year based on current annual habitat construction rates.

Pallid habitat was only added to the Gavins to Rulo (Omaha District) and Rulo to the Mouth (Kansas City District) models. All other RAS models upstream of Gavins Point Dam do not have habitat construction geometry changes.

4.2.3 Areas Excluded from New Pallid Habitat Construction

Locations selected for habitat construction in the RAS model are theoretical and do not reflect actual locations of future mitigation projects. However, the following areas were intentionally avoided when making modifications to RAS models. For the BiOp as Projected geometry in the Gavins to Rulo reach, satisfying all of these criteria was not always possible due to the amount of habitat added. Implementation within the RAS models varied somewhat between Omaha and Kansas City Districts based on reach specific issues and are stated within Appendix D and E. A general summary of common exclusion criteria used is:

1) Reaches of river within a 10,000-ft radius of an airport (per FAA AC 150/5200-33, (FAA, Aug 2007)

2) Areas within 1/4 mile upstream or downstream of small town infrastructure along the river bank, on that side of the river only
3) Areas within 1/4 mile upstream or downstream of power plant or municipal water intakes, on both sides of the river
4) Areas within 1/4 mile upstream or downstream of barge loading facilities and other river related industrial infrastructure along the river bank, on that side of the river only
5) Areas within 1/4 mile upstream or downstream of bridges, on both sides of the river
6) Areas within 1/4 mile upstream and downstream of Federal/PL 84-99 levees lying close to the river bank (approximately 1000 feet or less), on that side of the river only.
7) Reaches adjacent to larger cities, particularly where the channel is confined by urban levees
8) Widening projects were not located in the same bend as new or existing chutes or widening to avoid excessive navigation channel flow loss
9) The outside bend within Kansas City District based on past performance experience

4.2.4 Implementation within HEC-RAS

Creation of the new habitat areas within HEC-RAS required defining habitat elevation and size. Elevations of pallid habitat are defined using reference flows such as the August 50% exceedance level and median monthly flows. Reference flows were implemented with steady flow RAS modeling runs to determine profiles for use with setting habitat elevations.

Top width widening was the primary means of adding pallid habitat (both SWH and IRC) to all three geometries. Widening width, depth, and design invert elevation varied by alternative and by spatial location on the river. Widening areas alter geometry within the RAS model that affects computed conveyance and results. This also impacted model parameters: flow roughness, overbank ineffective areas, levee points, and permanent ineffective areas in the channel. An example of how habitat creation was implemented within the RAS model from Rulo to the Mouth is shown in Figure 4-2. Implementation was slightly different between the two RAS models (Gavins to Rulo and Rulo to the Mouth). Refer to Appendix D and E for additional details regarding RAS model implementation.

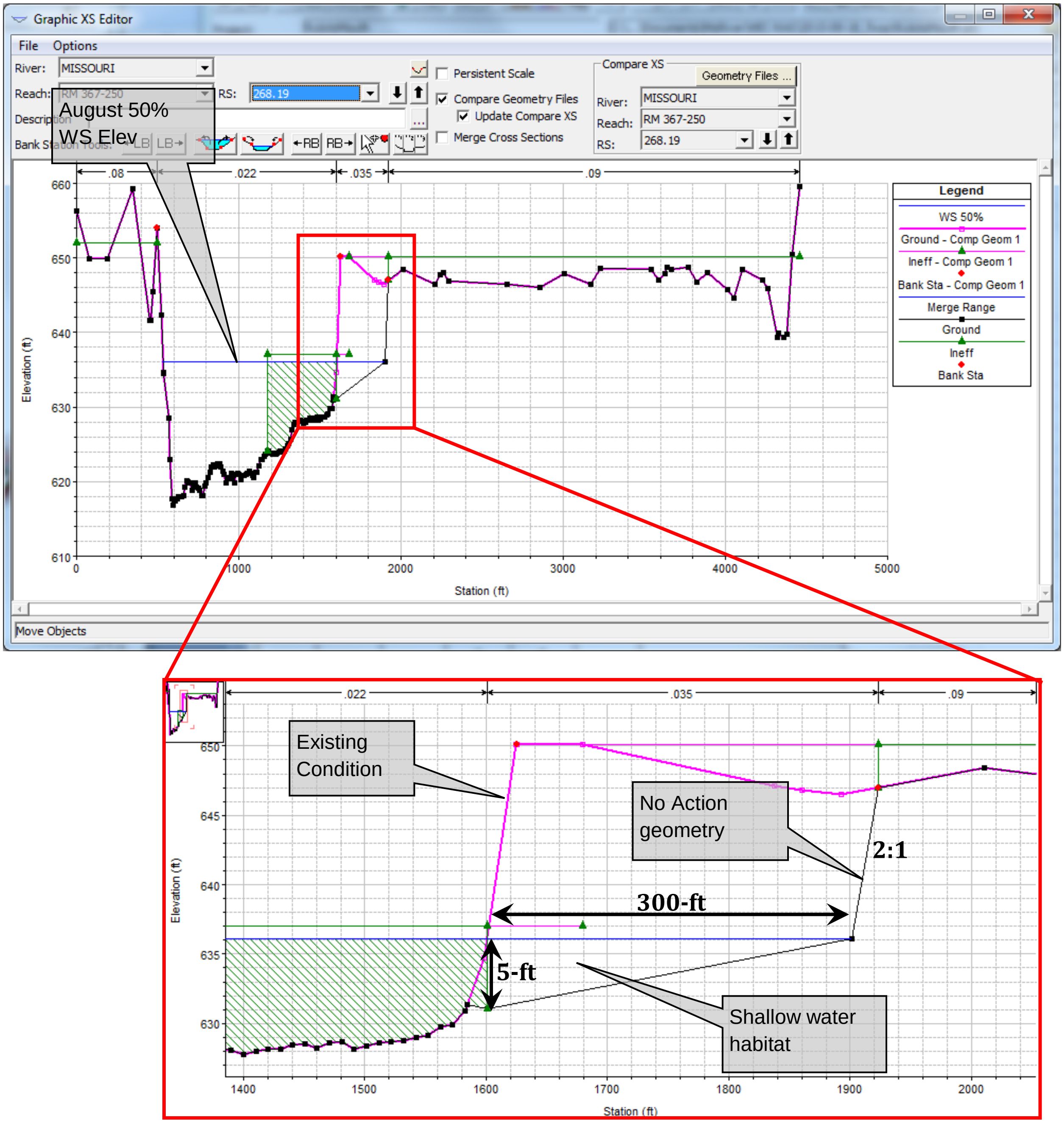

Figure 4-2. Widened cross section – No Action

A summary of the created geometries, habitat target metric, and the reference flow used in design are provided in Table 4-1.

Table 4-1. Geometry Summary

Geometry	Target Habitat	Reference Flow (cfs)
No Action	20 acre/mile	August 50% exceedance
BiOp	30 acre/mile + floodplain connectivity	Summer low, Median August, & Spring Pulse
IRC	260 acre/year for 13 years	Median June

Geometry revisions previously discussed in this section are summarized as:

- Revisions were necessary to transition the model from the 2012 calibration version to a new current condition model that reflects modifications already constructed or in construction.
- The no action and alternative conditions all include geometry changes from the calibration condition model to reflect variations in future habitat.
- Geometry changes for alternatives with new pallid habitat are only necessary for the two RAS models located downstream of Gavins Point Dam.
- All geometry changes are assumed to occur instantaneously without consideration of a construction time window.
- All RAS modeling efforts are for a static geomorphic condition only. The model does not adjust geometry during the POR such as what would be expected with future aggradation / degradation.
- Several of the alternatives are intended to create ESH via flow releases. ESH dynamic changes during the POR simulation are not included within the RAS models.

4.3 Flow Changes

Revisions to reservoir releases were a primary component of all six alternatives. The flow changes were modeled in ResSim as described in the ResSim alternative modeling detailed documentation *Mainstem Missouri River Reservoir Simulation Alternatives Technical Report* (USACE 2018a). Flow alternatives are conducted for the purposes of ESH creation and pallid spawning benefit. The reservoir pool elevations and dam outflows determined with the ResSim model were used as input for the various RAS models for each of the six flow alternatives as previously described in Section 2.

Table 4-2 summarizes the ranges of releases from Gavins Point, as well as the number of years in which the release was eliminated, partially or fully completed within ResSim. Evaluation of the ResSim output is informative when examining the changes that occurred in the RAS model results.

Table 4-2. Summary of Management Action Flows Simulated over the Period of Record

Alternative	Month	Frequency during 82-year Period of Record (1931-2012)			
		Eliminated[1]	Partial Completion[2]	Full Completion/Duration[3]	
		No. of Years	No. of Years	No. of Years	Percent Years of POR
1 – No Action	March	48	4	30	37
	May	53	8	21	26
	Both months			**16**	**20**
2 – USFWS 2003 BiOp Projected Actions	March	40	24	18	22
	May	42	25	15	18
	Both months			**10**	**12**
3 – Mechanical Construction Only		not applicable, no flow management action included			
4 – Spring Habitat-Forming Flow Release		67	7	**10** [4]	**12**
5 – Fall Habitat-Forming Flow Release		73	2	**7** [4]	**8**
6 – Pallid Sturgeon Spawning Cue	March	39	26	17	21
	May	65	6	11	13
	Both months			**11**	**13**

[1] Eliminated: Hydrological conditions in these years would not have been appropriate for any release.

[2] Partial Completion: Releases would have occurred but not at the full planned volume or duration (1 day minimum)

[3] Full Completion/Duration: Releases would have occurred for the full planned volume and duration.

[4] Shown values for spring (Alternative 4) and fall (Alternative 5) are deliberate releases, do not include events when targeted flow release levels would have been achieved "naturally" during normal operations.

4.4 ALTERNATIVE GEOMETRY AND FLOW PAIRING

Each flow alternative was paired with a geometry alternative to produce six total alternatives that were run through the RAS models. The No Action geometry was paired with the No Action flow for Alternative 1. The BiOp geometry was combined with the BiOp flow for Alternative 2. The IRC geometry was paired with flow alternatives 3 through 6 to produce Alternatives 3, 4, 5, and 6. Table 4-3 provides a listing of the geometry and flow pairings for each alternative.

Table 4-3. Alternative Geometry and Flow Pairings

Alternative	Geometry	Flow
Alternative 1	No Action	No Action
Alternative 2	BiOp	BiOp Projected
Alternative 3	IRC	Mechanical Only
Alternative 4	IRC	Spring Habitat Forming Release
Alternative 5	IRC	Fall Habitat Forming Release
Alternative 6	IRC	Pallid Sturgeon Spawning Cue

4.5 FUTURE CONDITION YEAR 15 EVALUATION

Degradation and aggradation of the Missouri River channel bed and sedimentation in the reservoirs are ongoing processes, which have the potential to effect virtually all economic resources and human considerations. Therefore, additional modeling was performed to provide estimates of how ongoing sedimentation processes may affect the performance of alternatives in the future. The designation “Year 15” comes from the timeframe for implementation.

Alterations were made to both the ResSim and RAS models to represent conditions at the end of the 15 year implementation period. Within ResSim, reservoir sediment depletion rates were used to develop a Year 15 future condition reservoir storage capacity relationship. Within RAS, an HEC-RAS with sediment modeling effort was conducted to develop an estimate of Year 15 geometry changes. The geometry changes were incorporated into the current condition alternative model geometry. Refer to the RAS modeling Appendices B, C, D, and E for specific details on changes within each model. Year 15 modeling was not performed for the Ft Peck to Garrison reach, described in Appendix A, since none of the alternatives affect reservoir releases in this reach.

After developing the Year 15 models, all six alternatives were simulated for the 82 year POR, and results were compared between alternatives and to the base condition. Results from the Year 15 analysis were provided to human considerations teams for qualitative evaluation, versus the full quantitative evaluation that was performed on the base 2012 condition (also referred to as Year 0).

4.6 HEC-RAS MODEL VERSION

All alternatives runs were performed in RAS 5.0 Beta (21-August-2015) because that was the most current version as of October and November 2015, when most of the runs were completed. Official release of RAS 5.0 was on 4-March-2016 and 5.0.1 on 22-Apr-2016, at which point evaluation of results by the HC teams was well underway and it would have unnecessarily compromised schedule to re-run the simulations. Due to improvements made to the ResSim model while preparing for running the Year 15 simulations, the decision was made to re-run the Year 0 in the updated ResSim model. The RAS Year 0 models were re-run on 25-Aug-2017 using HEC-RAS 5.0.3All Year 15 alternatives runs were performed in HEC-RAS 5.0.3 which was available when model runs were performed in October 2017.

5 HEC-RAS MODEL RESULTS

Hydrologic model results, consisting of computed daily flow and elevation information, was compiled from the RAS models at key locations throughout the basin. Results from the hydrologic modeling were provided to the HC team members for the comparison of alternative conditions. The model results at key locations were developed in summary form for comparison between alternatives.

The HC team member analysis used the daily (instantaneous 2400 value for each day) flow and water surface elevation output to analyze effects to various resources that include: hydropower, cultural resources, fish & wildlife (exclusive of listed species), flood risk, irrigation, recreation, thermal power, and water supply. The HC team performed an extensive analysis on each of the alternatives for all of the resources and provide a detailed comparison of results. For this report, only the hydraulic model output is presented.

To express the changes compared with the No Action alternative, the model results were evaluated by 1) statistical evaluation comparing min, max, and percentiles, and 2) duration analysis plots. Due to the basin size, number of models, model complexity, and the number of alternatives developed, typical model outputs such as profiles and flood area mapping were not developed.

Refer to the HEC-ResSim and the appendices to this report for detailed documentation of the results for each modeling effort (USACE 2018a.

5.1 Comparison of Alternative Results

For the comparison of results, flow and water surface elevation were analyzed to compare the differences between the No Action Alternative and the remaining five alternatives. Tables comparing min, max, and percentile flows and stages at key locations were developed for each RAS model and are contained within the respective model appendix.

However, caution should be used when trying to draw conclusions only from the statistics tables. The FIA models used by the HC team that compute structural and agricultural damages from flood events will provide a more complete alternative impact assessment because they incorporate all cross sections along the river in addition to the single site statistics. In addition, tabulating the changes for the entire period of record can mask impacts of the alternative on individual events within the POR.

An example of results comparison at St. Joseph, derived from the Rulo to the mouth RAS model output, is provided in Table 5-1 and Table 5-2.

Table 5-1. Flow (cfs) statistics on the period of record at St. Joseph

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	7,416	7,537	7,416	7,416	7,416	7,416
10%	18,181	18,170	18,250	18,020	18,231	18,094
25%	26,258	25,180	26,443	25,719	26,317	25,931
50%	39,282	38,478	39,156	38,950	39,046	38,988
75%	50,589	52,358	50,527	51,032	50,703	51,328
90%	68,612	69,893	68,614	70,557	69,439	69,001
Max	292,224	293,577	297,961	297,991	297,994	297,977

Table 5-2. Elevation (NAVD 88 ft) statistics on the period of record at St. Joseph

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	787.8	787.8	787.8	787.8	787.8	787.8
10%	791.3	791.3	791.3	791.2	791.3	791.3
25%	793.8	793.4	793.8	793.6	793.8	793.7
50%	797.4	797.1	797.4	797.3	797.4	797.3
75%	799.6	799.7	799.6	799.7	799.7	799.8
90%	802.6	802.4	802.6	802.9	802.8	802.6
Max	820.3	819.2	820.8	820.8	820.8	820.8

In the above tables, the min and max are the lowest daily flow or elevation and the highest daily flow or elevation output for each alternative over the period of record.

Calculated flow changes do not necessarily occur on the same date from alternative to alternative. For example, the minimum flow shown in Table 5-1 at St. Joseph is 7,416 cfs for Alternatives 1, 3, 4, 5, and 6. In the model output, this occurred on 3 Dec 1955, as shown by the hydrograph output in Figure 5-1. The low summer flow Gavins Point release rule during the months of July and August allowed the navigation season to extend one to two weeks longer in Alternative 2 than the other Alternatives, with the result of increasing the lowest few days of flow at St. Joseph.

Figure 5-1. Alternatives Flows at St. Joseph – 1955

Seasonal duration analysis was also performed at key locations using POR results. Seasonal dates chosen for the duration analysis coincide with the current System operational seasons: spring (1Mar to 30Apr), summer (1May to 31Aug), fall (1Sep to 30Nov), and winter (1Dec to 28Feb). The greatest difference can usually be seen closer to the reservoir release point and in the spring and winter durations due to the spring pulses and resulting lower winter flows. An example duration analysis for the spring period (1 Mar to 30 Apr) at Sioux City comparing alternatives plot is shown in Figure 5-2. In this example, alternatives 2, 4, and 6 illustrate changes due to flow modifications during this period.

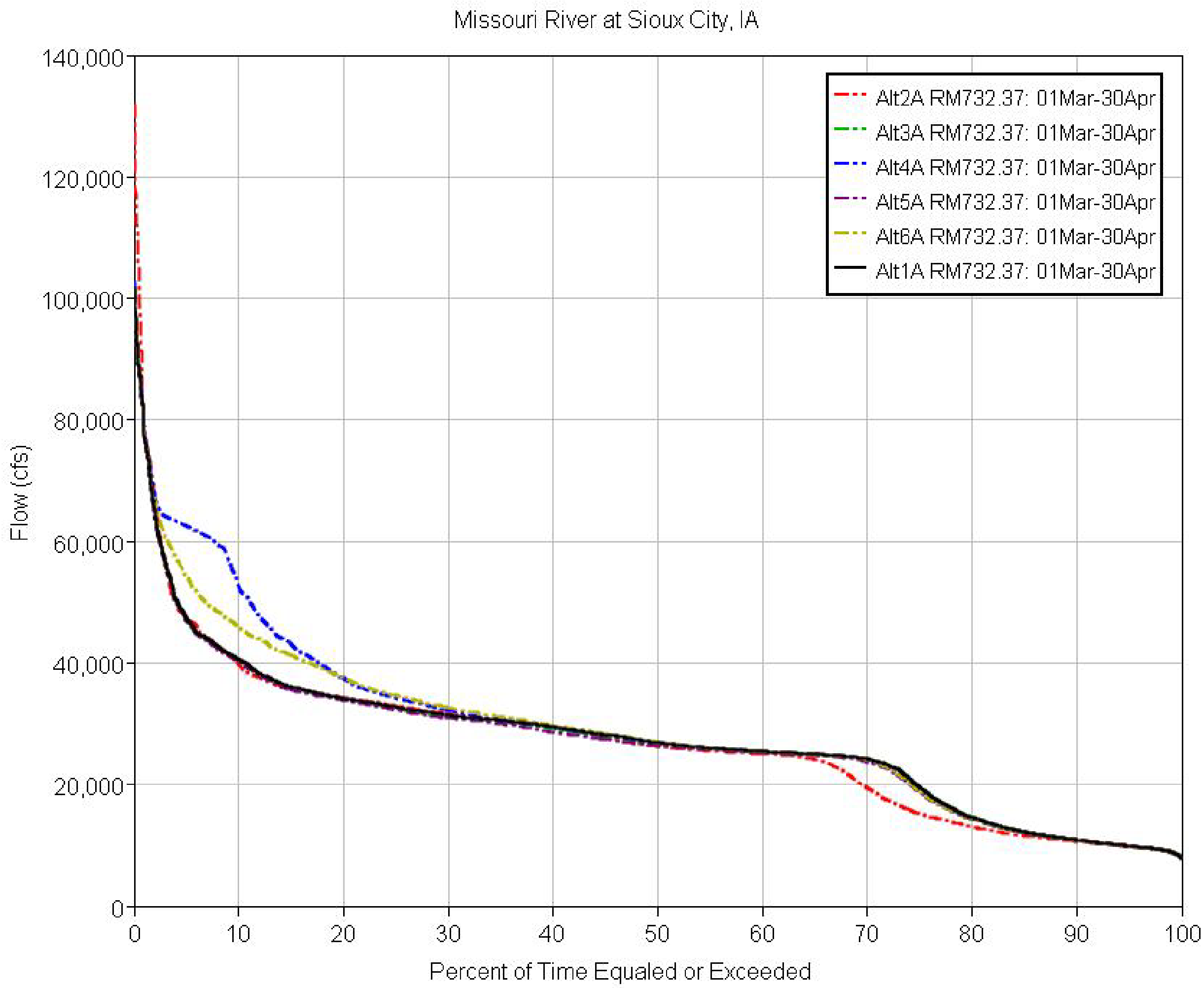

Figure 5-2. Sioux City Spring (1 Mar to 30 Apr) Duration for POR

5.2 YEAR 15 ANALYSIS RESULTS

Output from the Year 15 analysis was evaluated by comparing the Year 15 alternatives to the Year 15 No Action to indicate how alternative performance may vary in the future for consideration with the selection of a preferred alternative. In addition, comparison of the Year 15 alternatives to the base condition was performed to provide a sense of how future channel bed and reservoir sedimentation conditions may impact Missouri River flows and stages. The Year 15 analysis results were qualitatively evaluated using the HydroViz tool (Long 2017). Refer to the separate appendices to this report for additional detail.

While not intended to represent detailed estimates of future reservoir and channel conditions, the results do provide an alternative comparison methodology. The comparison to the base condition (Year 0) analysis results is limited due to the analysis methodology. The Year 15 results are directly influenced by multiple modeling assumptions that were made about an unknown future using historic sediment data and the 82 year POR flow data set. Sediment yield and sediment transport are highly dependent on factors such as flow magnitude, flow sequence, sediment input source, and sediment input material size.

Visual and statistical evaluation of the Year 15 output indicates that regardless of changed conditions, the alternatives compare similarly to each other. Therefore, the Year 15 results do not indicate a significant change in the relative difference between alternatives in the future. While not intended to represent detailed estimates of future reservoir and channel conditions, the results do provide an alternative comparison methodology. Results from the Year 15 analysis were also provided to the human considerations teams for qualitative evaluation, versus the full quantitative evaluation that was performed on the base condition (also referred to as Year 0).

5.3 Bank Erosion Analysis Downstream of Gavins Point Dam

In order to assess potential variation in bank erosion rates as a result of change in flow releases from study alternatives, a bank erosion model was assembled for evaluation in the 60 mile long reach downstream of Gavins Point Dam. Refer to Appendix D, Attachment 3, Missouri River Unsteady HEC-RAS Model Sediment Analysis, for additional details.

The bank erosion analysis determined that the bank erosion total volume percent change from the base condition alternative 1 varied by less than 1 percent for all alternatives. Alternatives that included flow changes (2, 4, 5, and 6) resulted in slightly increased bank erosion volumes while the alternative 3 change was about 0.1% less than the alternative 1 base condition. While results indicate that bank erosion rates in the Gavins to Ponca reach are slightly sensitive to the Gavins Point Dam releases, the small change computed by the model indicates that the variation in bank erosion rates as a result of the flow change alternatives is projected to be minor. Note that model results are summarized for the entire reach, localized variation may occur.

5.4 Factors to Consider in Analysis of Results

The analysis of flow and stage changes between alternatives at a certain location is influenced by an array of variables. For example, even when the reservoir release has no change, the flow or stage calculated from alternative to alternative at a downstream location may change because of minor variations in the RAS unsteady flow routing and the habitat additions to the river geometry. Differences in the unsteady flow routing, levee storage cells, and the overall amount of constructed habitat are causes for variation in peak flow and stage when an alternative does not include a change in reservoir releases.

From alternative to alternative the primary factors that should be considered when evaluating changes between alternatives include:

a. Small number of pulses
b. Reservoir releases from the Mainstem System as calculated by HEC-ResSim
c. The pallid sturgeon habitat additions to the river geometry
d. Storage changes in the river and floodplain
e. Timing of reservoir releases and downstream inflows, combined with RAS routing, during the POR
f. RAS output interval may mask minor differences
g. Comparison of results on a POR basis can mask impacts due to flow changes

5.4.1 Small Number of Pulses

Each of the flow alternatives were developed with a specific set of rules that restrict implementation. The number of times a pulse occurs for each alternative is illustrated within Table 4-2. Several of the flow alternatives are implemented less than ten times during the POR simulation. Downstream impacts are highly dependent on local inflows that combine with reservoir releases. Since the number of implemented pulses is small, results analysis is limited by the downstream tributary conditions of these few occurrences.

5.4.2 HEC-ResSim Flow Change

Flow calculated by the RAS model at a downstream location not only depends on how the reservoir System releases changed, but also how those changes carry downstream and were combined with inflow during the POR. Minor variation can occur in HEC-ResSim flow releases due to model scripting (USACE, 2018a). These flow changes would carry forward into RAS model results.

5.4.3 Change in Results – Habitat Additions

There is evidence that habitat construction tends to lower the river stages in the project vicinity and slightly upstream of the habitat location and generally has a minor dampening effect on the hydrograph, lowering peak flows downstream of the habitat location (Jacobson et al 2015). However, other experiences such as observations during the 2010 and 2011 flow events indicate that habitat project changes to peak river levels may be variable and minor (USACE 2012 and USACE 2013) due to factors such as the overall river flow distribution within the floodplain and sediment transport. The dynamic nature of the Missouri River and sediment transport contribute to the uncertainty regarding habitat construction impacts. For this study, the RAS modeling effort does not attempt to address Missouri River channel dynamics or sediment transport changes with a static model geometry for the entire POR. Therefore, the most likely explanation for variation in peak flow and stage within the two RAS models downstream of Gavins Point Dam is the difference in constructed habitat and RAS model geometry change.

Pallid habitat is not added equally within the alternatives and thus the impact on results will vary. For instance, the difference in added habitat between Alt 1 to Alts 3-6 that affects conveyance, excluding backwater habitat, is relatively small (3,519 acres vs. 3,380 acres, Table 2-2 and Table 2-7). However, the distribution of added habitat changes significantly with habitat upstream of Rulo, NE, decreasing from 1,453 ac to 861 ac while habitat downstream of Rulo, NE, increases from 2,066 ac to 2,519 ac (Table 2-2 and Table 2-7). Because of the habitat distribution variation, differences between alternatives varies with location and is not consistent.

Results variation from constructed habitat is shown by the increase in the maximum flow and stage at St. Joseph for Alternatives 3-6 (which have less habitat construction) when compared to Alternative 1, as shown in Table 5-1 and Table 5-2. The maximum flow for the period of record at St. Joseph occurred in all Alternatives during the simulated flood of 1993. Releases out of Gavins Point Dam from the ResSim model are identical for all the alternatives during this year, which means all differences observed at downstream locations are due to habitat additions to the river geometry or slight variations in unsteady flow routing in combination with levee storage areas.

Alternatives 3-6 all utilize the IRC geometry configuration, whereas Alternative 1 uses the No Action geometry configuration which results in more new pallid habitat (3,519 acres vs. 3,380 acres, Table 2-2 and Table 2-7). Within the Rulo to Kansas City reach, less habitat overall was added to the Alt 3-6 geometry (670 ac) than the Alt 1 No Action geometry (1,129 ac). Upstream habitat changes, unsteady flow computations, and timing with levee storage areas, may also contribute to results differences.

Examination of other events illustrate inconsistencies and difficulty when examining results between alternatives with variable added habitat and flow changes. For example, the BiOp geometry had the most habitat added, both upstream of Rulo and in the Rulo to Kansas City reach. The maximum stage for Alt 2 at St Joseph, as shown in Table 5-2, was significantly lower than the other alternatives, however the flow was slightly increased as compared to Alt 1, see Table 5-1. Unsteady flow routing introduces the effects of the looped rating curve, where the stage-flow relationship varies on the rising and falling limb of the event hydrograph. Multiple factors that affect the stage-flow relationship may be occurring within the model results.

5.4.4 Change in Results - Storage

Storage within the RAS model is a factor that influences the timing and magnitude of flows at all levels is the interaction of downstream conveyance features that represent storage. Storage interacts with the conveyance in the main river by taking on water when the river is rising, and returning water when the river is falling. Only within the Rulo to the mouth model, this interaction happens at low flow with the navigation structures represented by permanent ineffective areas in the channel. Within all models at bank full flows, this interaction happens with storage areas that represent tributaries and tiebacks with low lying connection to the river, and also in low lying floodplain areas between the high banks and adjacent levees or bluffs. At the highest flow levels, this interaction happens with transfer of flow from the Missouri River channel to the large protected areas behind the levees that were modeled in RAS with storage areas. Additional flow area from habitat projects may seem small compared to a fully inundated floodplain, but even slight differences in the river water surface elevation could change the interaction of the river with the storage areas and alter the timing and magnitude of water to reach a certain location. Without extensive testing and sensitivity runs it is difficult to determine the magnitude of storage changes on alternative result comparison.

5.4.5 Change in Results – Unsteady Routing

Coupling the ResSim and RAS models provides a powerful alternative analysis tool that provides a high quality basis for assessing differences. Comparison between alternatives should recognize that minor and insignificant differences can occur due to the nuances of unsteady flow computations. Flow calculated by the RAS model at a downstream location not only depends on how the reservoir System releases changed, but also how those changes carry downstream and were combined with inflow during the POR. Minor variation can occur in the ResSim flow releases due to model scripting (USACE 2018a). Even when the reservoir release has no change, the flow or stage calculated from alternative to alternative at a downstream location may change because of minor variations in the RAS unsteady flow routing, timing of downstream inflows, and flow attenuation due to the habitat additions to the river geometry.

5.4.6 HEC-RAS Output Interval

Minor differences were noted in model output that were in the range of a few hundred cfs and a few tenths foot of stage. Review of model results indicated that some of this difference is due to the RAS alternative models output interval that were configured to report one value per 24 hour period. In contrast to ResSim, the single value is not the daily average. The RAS model computes stage and flow at the computation interval (varies from ten to 30 minutes within each individual RAS model) for the entire period of record, but only reports the instantaneous value that occurs at hour 2400 on each day. Due to the watershed size and long duration Missouri River flow events, number of hydrographs and locations required for HC team economic evaluations, and the POR length, the daily reporting time was deemed appropriate. This means that slight shifts in timing from alternative to alternative can carry over into the results as small fluctuations in the reported peak flow. A brief evaluation was conducted as reported within Appendix D and E. Changes were determined to be a small factor on the order of 0.1 foot difference. This level is not likely to significantly impact the HC results evaluation which uses annual peak stage for damage computations.

5.4.7 Results Based on POR

Comparison of results is based on the POR methodology. Due to the large number of daily values, the limited number of pulse occurrences for many of the alternatives, and the limited pulse duration, the POR methodology that relies on comparing annual peaks or flow duration statistics for the entire 82 years may not identify change in risk to human considerations. Figure 5-3 illustrates how flows may change between alternatives.

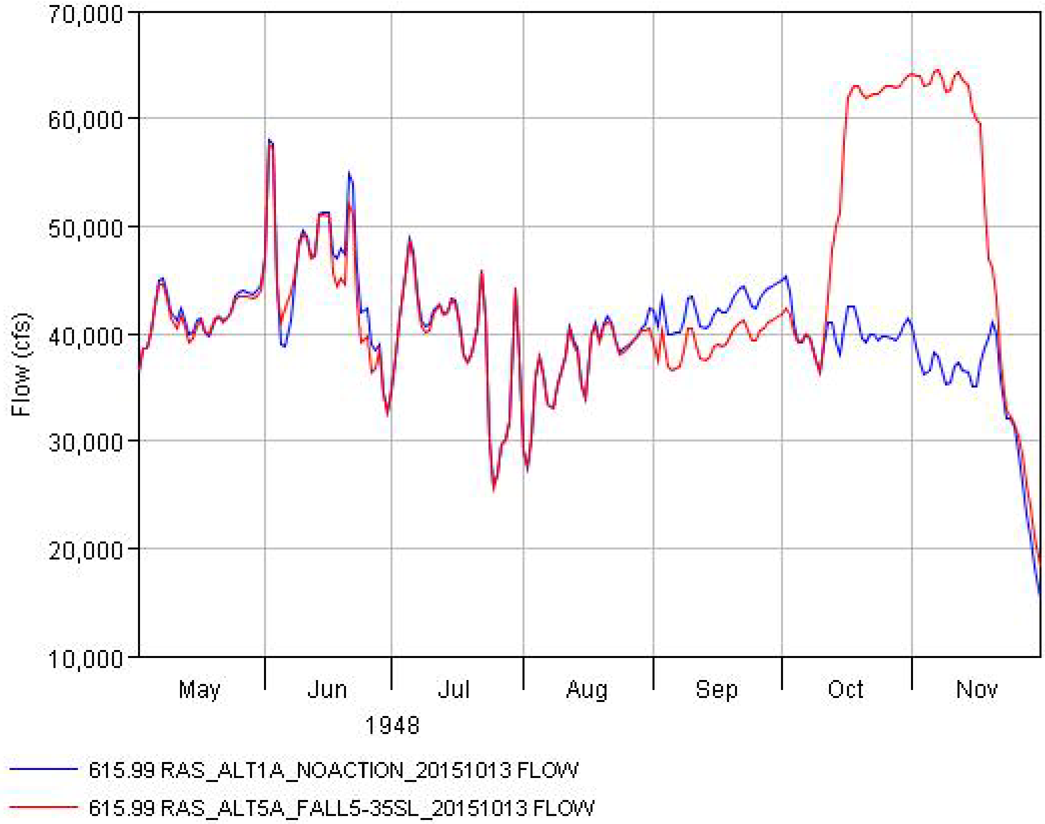

Figure 5-3. Example Comparison of Alt 1 and 5 flows at Omaha, NE

Although the 82 year POR length is statistically significant, alternatives that include flow release changes for a relatively few number of years, often less than ten, can be obscured when averaged over the entire POR. In addition, this small sample size limits possible combinations with downstream inflow. Further, proximity to the upstream dam is a factor when considering the relative magnitude of the pulse change to normal river flow.

If the statistics are compared during the pulse year instead of for the entire POR, results are significantly different. As an example, the change in statistics between Alternatives 1 and 5 using the entire POR and only 7 months in 1948, a year that included a flow pulse release for alternative 5, is shown in Table 5-3.

Table 5-3. Comparison of Omaha Flow (cfs), Alt. 1 and 5, POR and Flow Pulse Duration

	82 Year POR Length			1 May to 20 Nov, 1948		
	Alt 1	**Alt 5**	**Difference (Alt 5-1)**	**Alt 1**	**Alt 5**	**Difference (Alt 5-1)**
Min	7,029	7,029	0	25,676	25,623	-53
10%	13,218	13,243	25	35,511	36,330	818
25%	18,073	18,173	99	38,576	39,003	427
50%	31,424	31,185	-239	40,815	41,113	298
75%	38,558	38,538	-20	43,191	46,952	3,761
90%	49,388	49,925	537	45,836	62,681	16,845
Max	206,235	206,262	27	58,056	64,444	6,389

6 INTERIOR DRAINAGE EVALUATION

Interior drainage refers to the conveyance of flow from interior, or landward side, of the levee to the Missouri River channel. Typical Missouri River levee systems have gravity flow culverts or pump stations to allow local drainage to exit the interior of the levee and drain to the river. Each culvert typically would include one or more closures, such as a flap gate or sluice gate, to prevent river water from backing up into the leveed area. When river levels are higher than the culvert outlets and this coincides with heavy local rainfall, ponding water can cause flooding on the interior of the levee. Additionally, when river levels are above the interior ground level, seepage through the ground under the levee can also cause flooding on the interior. Refer to the RAS alternative condition model report for a detailed discussion of interior drainage model creation and results within Appendix D and E.

The flow change alternatives have the potential to significantly impact interior drainage. Using a flow of 75,000 cfs, which is just below the flood target of 82,000 cfs for Nebraska City in the ResSim model, Figure 6-1 illustrates that the RAS model computed profile is either partially or fully over all of the flap gates on L-536 and L-575 levee units (circled in red). The steady flow profile analysis illustrates that the risk for interior drainage impact is high for any flow pulse since the downstream flood target elevation results in interior drainage blockage at some locations.

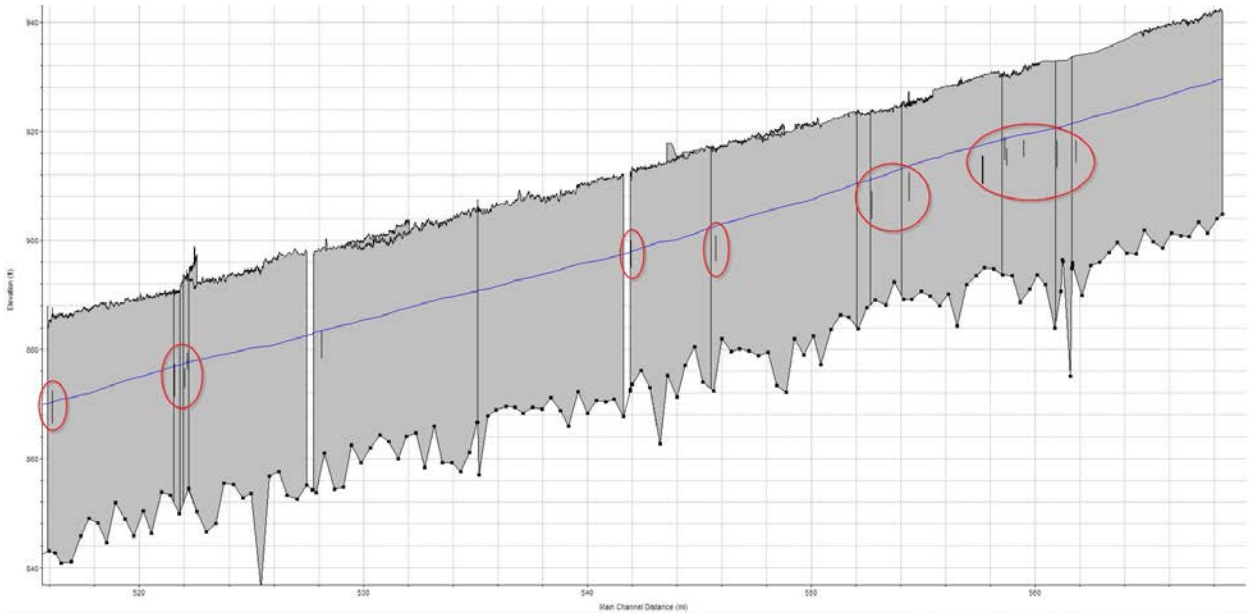

Figure 6-1: 75,000 cfs HEC-RAS Steady Profile and Flap Gates

6.1 CONDUCTED EVALUATION

To evaluate interior drainage flooding, and measure differences between the proposed alternatives and Alt 1, a sub-set of the seven sites evaluated for the Master Manual (USACE 1998) were modeled in detail. Four sites were selected, L-575 and L-536 in the Omaha District and L-488 and L-246 in the Kansas City District.

The interior drainage evaluation was conducted using the alternative condition RAS models. All sites are located downstream of Omaha, NE, within the reach in which federal levees were constructed. Consequently, only the Gavins to Rulo and Rulo to the mouth RAS models were used in the interior drainage analysis. Figure 6-2 shows an area map with the locations of the four sites on the river.

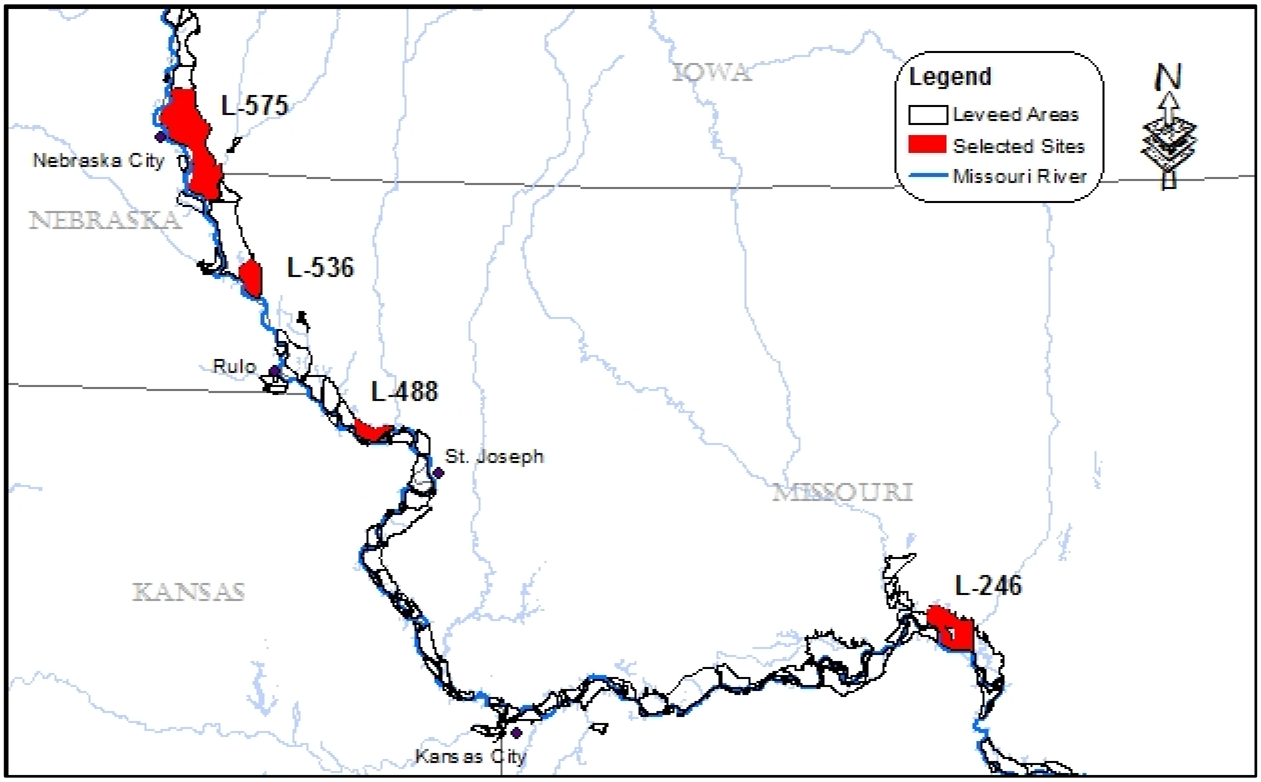

Figure 6-2. Interior Drainage Sites

6.2 Interior Drainage Data Assembly

Drainage basins for each unit were drawn using the best available LiDAR data and previously delineated basins from the Master Manual were used as a guide (USACE 1998). Lateral structures were used to model levee alignments. Culvert and flapgate data was assembled from the best available survey information and the levee unit O&M manuals. For the two units within the Omaha District (L-575 and L-536), the best available pump data was obtained from the updated O&M manual. However, it is noted that pumps are not always maintained and operated by the levee drainage districts so there is very little current information known. RAS requires a pump capacity curve including losses. Since such little information was known about the pumps, pump capacity curves were developed using manufacturer's information or representative curves. Friction and minor losses were included in the final pump capacity curves.

Seepage into the interior area during periods of high river stage was accounted for by use of a lateral inflow hydrograph boundary condition to each storage area in the RAS interior drainage model. A reference cross section was chosen to calculate the seepage into each lateral structure piece. Missouri River stage output from each alternative was used to calculate the seepage with a daily time step. For each day in the POR, the differential head was calculated and was multiplied by the seepage rate and the length of levee or number of relief wells. The final interior drainage model contained lateral inflow hydrographs of total seepage for each storage area for each alternative.

6.3 Interior Area Rainfall

An HEC-HMS model provided rainfall inflow hydrographs to each storage area for the POR for the levee unit interior drainage basins. Each storage area inflow dataset was input into the RAS model as a lateral inflow hydrograph. Unlike the seepage, which had alternative specific inflows, the same rainfall input was used for all of the alternative runs.

6.4 Interior Drainage Evaluation results

Interior drainage analysis was performed by modifying the respective RAS alternative condition model with the above parameters. Since the interior drainage model significantly effects unsteady flow computations, results may be compared only with other interior drainage model results.

To express the changes compared with the No Action alternative, the water surface elevation model results were evaluated by the same statistical evaluations made on the full models. Output tables are available in Appendix D and E. All of the alternatives show minor water surface elevation differences. Alternative 2 has the greatest impact (higher water surface elevations) among the alternatives.

The interior drainage models provide a powerful tool to assess the complicated interaction between reservoir releases upstream of a levee unit on interior ponding resulting from rainfall and/or seepage. However, limited conclusions can be extrapolated to the entire river. With such slight differences compared to Alt 1, it is difficult to separate the impact of flow changes from the reservoirs from the impact of added habitat to make global conclusions. Changes are highly localized, depending upon factors such as how low the culvert outlet is and the interior area available for ponding before damages occur.

Interior drainage analysis was also performed using the Year 15 RAS modeling results. The relative comparison between alternatives for Year 15 produces very similar results to the base condition analysis. This result is expected since the primary factors that would alter interior drainage analysis results between alternatives, different river levels that would affect seepage rates and gravity flow through interior drainage connections, were also similar in Year 15.

6.5 Sensitivity Analysis and Results

A series of sensitivity runs were conducted for the interior drainage analysis within the Omaha District as presented in Appendix D. They included three actions either alone or in combination: all pumps removed, culverts filled with sediment halfway, and a simulated pulse every year (spring or fall, depending on the alternative). Reasons for running each sensitivity are as follows:

- All of the pumps removed - conducted due to the uncertainty with the pumps' physical and operational information.
- Culverts half filled with sediment - conducted because of a comparison to limited temporary gage data revealed that the model's water surface elevations were generally low.

- Simulated pulse every year - conducted to analyze if the timing and amount of pulses had a large effect on the results.

Sensitivity analyses indicated that changes in assumptions about the interior drainage structures have a large impact on results. Removing all pumps had the most effect on the storage area water surface elevations. The culvert sediment and pulse every year runs produced minimal differences compared to the no pumps run. The risk of flooding from interior drainage is much more a function of the conditions of the interior drainage structures (culverts and pumps) at specific sites. Alternative 2 has relatively larger impacts compared with Alternatives 3-6.

- Interior drainage analysis determined that alternative 2 had the largest impact
- A steady flow profile analysis at Nebraska City determined that the risk for interior drainage impact is raised for any flow pulse since the downstream flood target elevation at Nebraska City corresponds to river water levels above culvert invert elevations. River levels at this level or higher can reduce capacity of gravity flow levee drainage structures.
- Uncertainties regarding the interior drainage pump operation and culvert sediment levels were evaluated with a sensitivity analysis. Analysis demonstrated that these inputs have a high impact on results.

7 CHANNEL CAPACITY ANALYSIS

Channel capacity estimates were performed to provide an indication of the flow rate at which bank elevations are overtopped and flow begins to leave the main channel and enter the floodplain. Channel capacity estimates were performed with the one-dimensional RAS models calibrated to 2012 conditions by comparing steady flow profiles with top of bank elevations at each cross section combined with reviewing the best available floodplain topography. Floodplain flow connectivity was not assessed. The estimated channel capacity does not necessarily correlate with the onset of flood damage. In addition, channel capacity is typically highly variable along the channel bank due to wide variation in bank elevations. The quality of the channel capacity estimate is affected by numerous factors including how representative the model cross sections are of river geometry, local channel geometry variation, low spots in bank elevations, and the floodplain topography accuracy. Within the reservoir delta areas where the river enters the downstream lake, the channel capacity estimate is not meaningful.

A Monte Carlo risk analysis methodology capable of assessing impacts to dam safety and flood risk as a result of flow release changes would be required to fully assess how an alternative impacts potential flood risk. Refer to the *Summary of Hydrologic Engineering Analysis* (USACE 2018e) for additional details on the risk analysis methodology.

7.1 CHANNEL CAPACITY ESTIMATES AT SELECT LOCATIONS

Channel capacity estimates were performed to provide an indication of reaches susceptible to flooding and if any of the alternatives may alter flood risk. Within selected model reaches, the minimum flow that exceeded bank elevations was determined at a representative area. These areas were selected based on flood potential. For instance, the degradation reach immediately downstream of Gavins Point Dam has much higher channel capacity than areas further downstream. For that reason, the Nebraska City vicinity was selected as more representative of flood risk when evaluating alternatives. Refer to the individual appendices for each RAS model reach for additional information regarding reach characteristics and the channel capacity estimates. A summary of the channel capacity estimates is provided in Table 7-1. Due to the large distance downstream of Gavins Point Dam, a channel capacity estimate was not performed for the river between Rulo, NE and the mouth at St. Louis.

Table 7-1. Channel Capacity Estimates

Location	Representative Area[1]	From RM	To RM[2]	Channel Capacity (kcfs) [3]
Fort Peck Dam to Lake Sakakawea	Below the Yellowstone River	1771.5	-	35 to 40
Garrison Dam to Bismarck, ND	Bismarck, ND	1389.9	1314.6	55 to 60
Downstream of Bismarck, ND to Lake Oahe	Vicinity of Schmidt, ND	1314.6	-	35 to 40
Fort Randall Dam to Lewis & Clark Lake	Below the Niobrara River	880.0	-	35 to 40
Gavins Point Dam to Rulo, NE	Nebraska City, NE	811.1	498.0	80 to 85

1 Since the model reaches are up to several hundred miles in length, the estimated capacity should be regarded as an indication for the onset of flooding at the representative area, not the entire reach.

2 Downstream boundaries that are reservoir pools are not a static location and change with pool elevations.

3 The channel capacity estimate is based on an evaluation of hydraulic model results. The estimated channel capacity refers to the flow level at which significant water levels exceed bank elevations (may represent ponding water and not necessarily flow through connectivity). Values vary considerably within the reach and may change over time.

7.2 Flood Potential Evaluation

Following the channel capacity estimate, an additional brief evaluation was conducted of two areas to provide an indication of the potential for an alternative to change flood risk. From the channel capacity estimates shown in Table 7-1, two areas were selected that appeared to have the highest potential for a change in flood risk: 1) the Fort Randall Dam reach downstream of the Niobrara River; 2) the Nebraska City area downstream of Gavins Point Dam.

7.2.1 Downstream of the Niobrara River

For the reach downstream of the Niobrara River between Fort Randall Dam and Lewis and Clark Lake, it was determined that any of the proposed flow releases to provide spawning cues or ESH habitat exceed the existing channel capacity and would result in 100% flood risk potential for that area.

7.2.2 Nebraska City Area

For the Nebraska City area, a simplified evaluation was conducted to compare the number of days that each alternative has the potential to exceed the estimated channel capacity of 80,000 to 85,000 cfs. This was performed by comparing the number of days in the POR that the sum of the downstream inflow between Gavins Point and Nebraska City exceeded the capacity remaining after accounting for the Gavins Point Dam release.

A simplified example to present the evaluation method is as follows:

Alt 3 Scenario:

Total inflow downstream (Gavins release minus Nebraska City flow) was 30,000 cfs
Alt 3 normal Gavins Point Dam daily release of 35,000 cfs
Flood potential not counted (80,000 capacity is greater than the 35,000 release plus the 30,000 cfs downstream inflow)

Alt 4 Scenario:

Total inflow downstream (Gavins release minus Nebraska City flow) was 30,000 cfs
Alt 4 ESH Gavins Point Dam daily release of 60,000 cfs
Flood potential day counted (80,000 capacity is less than the 60,000 cfs release plus the 30,000 cfs downstream inflow)

The above computation was performed for all alternatives at Nebraska City. The percent exceedance for each alternative was calculated from the days above capacity compared to the total number of days. In order to simplify the computation, flow release periods were reduced to month long lengths. Values were calculated only for the month time period during the alternative specific pulse flow release period. Flood potential was computed as the number of days that downstream inflow plus Gavins Point Dam release exceeds channel capacity. The flood potential was expressed as a percent time exceeded that was computed from the total number of days exceeding capacity compared to the total number of pulse release days.

Results determined that most of the alternatives have the potential to alter flood risk at Nebraska City with most of the alternatives resulting in a greater than 10% change in flood potential during the flow release period. Results are illustrated in Figure 7-1.

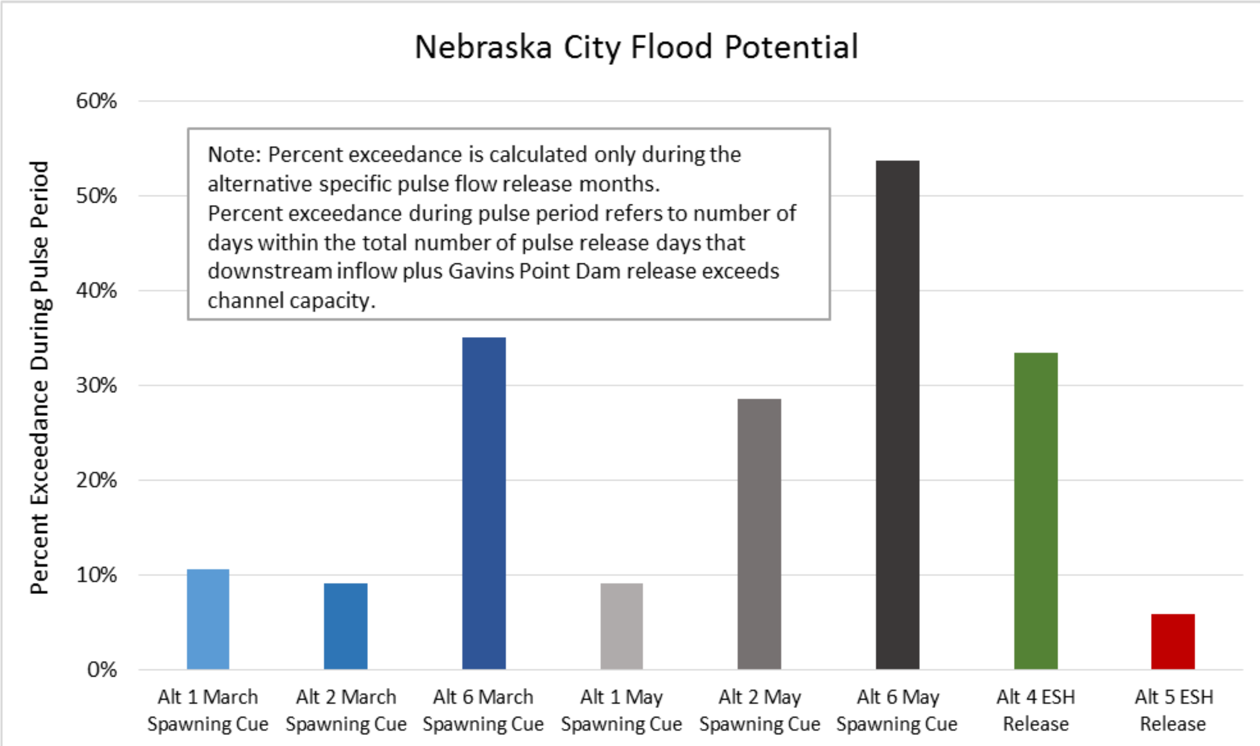

Figure 7-1. Nebraska City Flood Potential

7.3 Summary

Channel capacity estimates were performed to provide an indication the flow rate at which bank elevations are overtopped and flow begins to leave the main channel and enter the floodplain. The channel capacity estimate is based on an evaluation of hydraulic model results and vary considerably within the reach and may change over time. Following the channel capacity estimates, an evaluation of flood potential was conducted at two locations.

- Capacity estimates refer to the minimum capacity within a large reach, are highly variable, and may change over time.
- Within the Fort Randall reach downstream of the Niobrara River confluence, flow release above channel capacity is included in all alternatives. Flood severity variation between alternatives occurs due to changes in flow increase magnitude and duration.
- Within the Nebraska City area, the flood potential varies with most of the alternatives showing a greater than 10% increase in flood potential.
- All analysis was performed for the existing condition. No future condition analysis was performed of channel capacity estimates.

8 SOURCES OF UNCERTAINTY AND LIMITATIONS

The performed hydrologic evaluation of the complex Missouri River System provides a powerful alternative analysis tool for assessing differences between alternatives. Comparison between alternatives should recognize that minor and insignificant differences can occur due to many sources. Multiple factors contribute to the uncertainty in the ResSim and RAS model results including 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. In addition, the analysis relies on the simulation of the 82-year period of record using daily average outflows from a ResSim model input into a fixed bed RAS model, with stage and flow output. Sources of uncertainty and limitations of model output include:

1. Dynamic River - Each reach-specific model represents a snapshot in time on a dynamic river system that has experienced variance in the stage discharge relationship over time. 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. River banks can expand or contract depending on river flows. Channel bed forms change in magnitude and migrate over time. In most instances the Missouri River channel varies within a fairly well known range of depth and magnitude of bed forms and the models are designed to be a reasonable representation of this dynamic equilibrium. Evaluating dynamic change that may occur in response to specific project construction of Missouri River habitat projects was beyond the scope of this analysis and not attempted. This limitation was addressed to some extent by the Year 15 future condition evaluation.
2. Climate Change – The POR evaluation used the historic inflows with adjustment to current level of depletions. The historic inflows may not be comparable to future conditions. A climate change assessment of the Missouri River basin indicates increases to both temperature and precipitation along with increasing trends in extreme floods and droughts (USACE 2018c).
3. POR Limited Sample Size - Several of the alternatives include flow changes. Within the POR, the downstream impact of the flow change is only evaluated with the inflow that occurred historically. The conditions during a pulse year in the future could vary greatly from the small sample of pulse events included in the POR analysis.
4. Risk Analysis - The Missouri River System as currently operated provides substantial flood damage reduction and benefits to the entire basin. The current ResSim and RAS analysis, which employs an 82 year period of record simulation, shows the potential for negative impacts to flood damage reduction and dam safety for alternatives that include changes in reservoir flow releases. The current study methodology does not simulate a sufficient number of events and possible runoff combinations within the large Missouri River basin to evaluate potential change in downstream flood risk and dam safety or to quantify the associated uncertainties.

   Scoping efforts were conducted to determine a Monte Carlo risk analysis methodology capable of assessing impacts to dam safety and flood damage reduction as a result of flow release changes. The risk analysis primary components include further development of the period of record flow data set, ResSim and RAS model modifications, development of levee fragility curves, assignment of uncertainty, assembly and debugging of models, Monte Carlo simulation, analysis of results, and reporting. The Monte Carlo methodology properly assesses the effects of the alternative operation changes because it increases the sample size of flow data and number of combinations of flow periods that may occur in the future so that impacts can be characterized with greater confidence. Without such analysis, the impacts of operational changes will only be known for a limited number of flow pulse and downstream inflow combinations. Potential impacts to flood risk management were identified by evaluation of the outputs from the ResSim and RAS analysis that could be quantified with the risk analysis.

5. Bed and Floodplain Aggradation / Degradation Processes - Significant change in Missouri River geometry continues to occur as a result of channel aggradation and degradation. The observed aggradation/degradation trends, as discussed in the *Missouri River Stage Trends* study (USACE 2012c), result 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. Although alternative analysis is performed for the 82 year POR, the model geometry is static and does not adjust during the simulation period for aggradation / degradation processes. Additionally, the analysis does not try to project where sediment may accumulate in the floodplain or include projections of future change in floodplain roughness. For habitat alternatives, it was assumed that any bed or floodplain changes would be either negligible, similar between each alternative, or mitigated during more detailed design of river widening projects. The Year 15 qualitative analysis provides a limited view of future conditions that provides further understanding for the near term.
6. Limited Stream Gage Records and Flow Sources in the POR - The Missouri River, major tributaries, and ungaged inflows included in the RAS in the POR model are the only flow sources. During the early portion of the POR (1930 to about 1950), many of the stream gages had not been installed. Therefore, a significant percentage of the drainage area feeding each model is ungaged during the early years of the POR. Modeling 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 and POR report. In addition, the inflow sources, rates, and volumes are limited to those that occurred in the historic record as modeled within RAS. In reality, a wide variety of downstream flows should be expected due to localized rainfall and subsequent runoff.
7. Levee Performance and Interior Drainage – The limited scope of the interior drainage evaluation identified that many culverts are at a low flow level that are easily impacted by high Missouri River flow levels. Additional flooding of the interior can occur through seepage under levee foundations during prolonged high water, or through failure of levees prior to overtopping. Levee breach formation has occurred in the past on the Missouri River prior to levee overtopping. Assumptions regarding levee performance and how repairs occur after each event were greatly simplified for the POR evaluation. Detailed interior drainage calculations, which account for only river levels, localized rainfall and runoff, and underseepage was conducted at only four levee systems.

8. RAS Model Simplifying Assumptions - Simplifying assumptions were necessary for the construction of RAS models for a system of this scale and complexity. Bridges were not included. Floodplain flow areas were all considered as one-dimensional flow areas. Levee breaches without overtopping were not included. The RAS model construction methodology, which relies on cross sections of the river channel and adjacent floodplain to represent conveyance, was assumed to be not significantly altered by any of the alternatives. Alternatives were also assumed to have no effect on channel bed forms and flow roughness.

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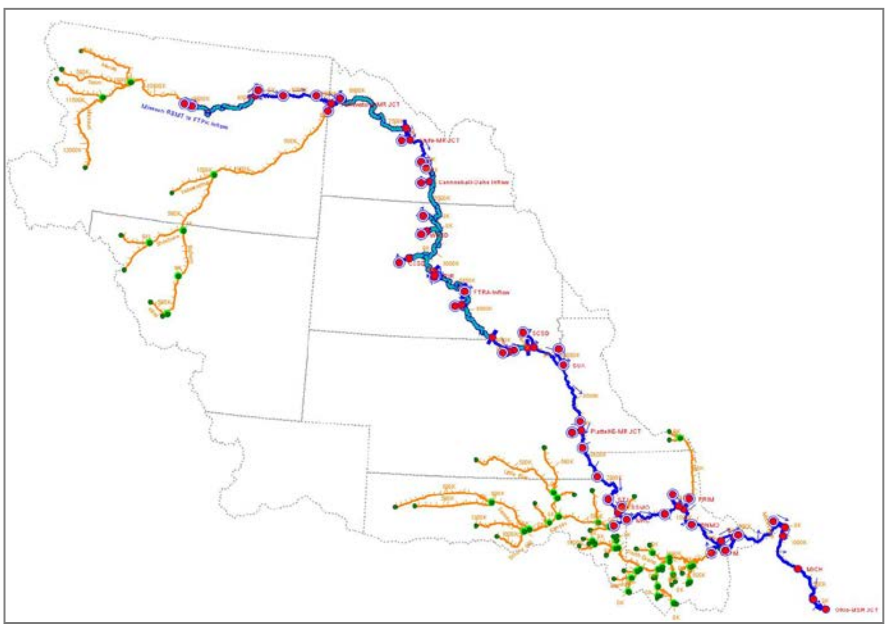
US Army Corps of Engineers
Omaha District

Missouri River Unsteady HEC-RAS Model
Alternatives Analysis

FINAL

Appendix A

Fort Peck Dam to Garrison Dam



July 2018

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ACRONYMS

BiOp…………………..Biological Opinion

CFS…………………. Cubic Feet per Second

ESH…………………..Emergent Sandbar Habitat

HC……………………Human Considerations

HEC .... Hydrologic Engineering Center

IRC……………………Interception-Rearing Complexes

MAF………………….. Million acre-feet

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

MRRPMP-EIS….……Missouri River Recovery Program Management Plan Environmental Impact Statement

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

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

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

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

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

POR .... Period of Record

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

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

RM……………………1960 River Mile

SWH………………....Shallow Water Habitat

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

USFWS……………….United States Fish and Wildlife Service

USGS .... United States Geological Survey

1 INTRODUCTION

The Missouri River unsteady HEC-RAS (RAS) model was developed for the Missouri River Recovery Program Management Plan and Integrated Environmental Impact Statement (MRRPMP-EIS) to assist in the assessment of a suite of actions to meet Endangered Species Act (ESA) responsibilities for the piping plover, the interior least tern, and the pallid sturgeon using USACE authorities. Model geometry development and calibration for the existing conditions is documented in *Missouri River Unsteady HEC-RAS Model Calibration Report Appendix A Fort Peck Dam to Garrison Dam* (USACE 2015). The objective of the HEC-RAS modeling is to simulate the Management Plan alternatives which include both geometry and flow changes relative to the No Action alternative. The Human Considerations (HC) team performed an extensive analysis on each of the alternatives for each of the resources (hydropower, cultural resources, fish & wildlife (exclusive of listed species), flood risk, irrigation, recreation, thermal power, and water supply) and provide a detailed comparison of results. For this report, only the hydraulic model output is presented; there is no alternative selection or discussion. This Appendix is for the Fort Peck Dam to Garrison Dam reach of the Missouri River as part of the Omaha District.

Six alternatives, including the No Action alternative, were simulated in RAS from March 1930 to December 2012, however the HC team only used complete year data for their analysis from January 1, 1931 to December 31, 2012. Development of inflow records at current depletion levels to use as boundary conditions for the HEC-ResSim (ResSim) and RAS models is documented in the report, *Missouri River Recovery Management Plan Time Series Data Development for Hydrologic Modeling* (USACE 2018c). Each alternative has unique flow releases from the reservoirs, as simulated by the ResSim model.

2 GEOMETRY

For the Fort Peck Dam to Garrison Dam model, no geometry changes were modeled. All alternative runs used the current conditions (2012) calibrated geometry.

3 FLOW ALTERNATIVES

A total of six flow alternatives were modeled in ResSim. Reservoir pool elevations and dam outflow output from the ResSim model was used as input for the RAS model for each of the six flow alternatives. A brief summary of the flow alternatives is provided below. For more details, see the ResSim technical report, *Mainstem Missouri River Reservoir Simulation Alternatives Technical Report* (USACE 2018a).

Tributary and ungaged inflows were kept consistent between alternatives. More details on the Period of Record (POR) flow dataset used can be found in the report *Missouri River Recovery Management Plan Time Series Data Development for Hydrologic Modeling* (USACE 2018c).

3.1 Alternative 1 - No Action

Under Alternative 1 (Alt 1), the Missouri River Mainstem Projects would continue to be operated as they are currently. Operations within the ResSim model were set up to closely follow the Master Manual that is used during real-time operations of the System; however, the model does have limitations and cannot capture all real-time decisions that occur.

3.2 Alternative 2 - U.S. Fish & Wildlife Service 2003 Biological Opinion Projected Actions

Alternative 2 (Alt 2) represents the U.S. Fish and Wildlife Service (USFWS) interpretation of the management actions that would be implemented as part of the 2003 Amended BiOp RPA (USFWS 2003). Operational criteria include different early and late spring spawning cues, low summer flows, and a maximum winter release limit.

3.3 Alternative 3 - Mechanical Construction Only

Alternative 3 (Alt 3) consists of mechanical construction of emergent sandbar habitat (ESH). Operational criteria consist of removing the early and late spring spawning cues in Alt 1.

3.4 Alternative 4 - Spring Habitat-Forming Flow Release

Under Alternative 4 (Alt 4), the early and late spring spawning cues in Alt 1 are removed from the operational criteria and a spring ESH-creating reservoir release from Gavins Point and Garrison is added. While the ESH-creation release is occurring from Gavins Point, flood targets are increased to allow the ESH-creation release the opportunity to run.

3.5 Alternative 5 - Fall Habitat-Forming Flow Release

Alternative 5 (Alt 5) removes the early and late spring spawning cues in Alt 1 and adds a fall ESH-creating reservoir release from Gavins Point and Garrison to the operational criteria. While the ESH-creation release is occurring from Gavins Point, flood targets are increased to allow the ESH-creation release the opportunity to run.

3.6 Alternative 6 - Pallid Sturgeon Spawning Cue

Alternative 6 (Alt 6) replaces the early and late spring spawning cues with different spawning cues. The early spring spawning cue in Alt 6 occurs at the same time as the early spring spawning cue in Alt 1 but with a higher peak release. The late spring spawning cue in Alt 6 occurs later in May than the late spring spawning cue in Alt 1 and has a larger peak release. Please note that the former name of this alternative was Alternative 7, which may correspond to some RAS model runs and file names.

4 SIMULATION OF ALTERNATIVES

Each flow alternative from ResSim was run through RAS with the current conditions geometry. Alternative names match in both ResSim and RAS.

5 RESULTS

All alternative runs were performed in HEC-RAS 5.0.3. Model output contains a considerable amount of information, not easily condensed to simple conclusions. Each of the six alternative runs produced 82 years (March 1930 – December 2012) of stage and flow hydrographs. To express the changes compared with the No Action alternative, the model results were evaluated by statistical evaluation and duration analysis plots.

Results from the 82-year runs for the six alternatives were provided to the HC team for analysis. They used the daily (instantaneous 2400 value for each day) flow and water surface elevation output to analyze effects to various resources that include: hydropower, cultural resources, fish & wildlife (exclusive of listed species), flood risk, irrigation, recreation, thermal power, and water supply. The HC team performed an extensive analysis on each of the alternatives for all of the resources and provide a detailed comparison of results. For this report, only the hydraulic model output is presented.

5.1 STATISTICS

For the statistical evaluation, daily flow and water surface elevation results were analyzed to compare the differences between the No Action Alternative and the remaining five alternatives. All of the alternatives show minor changes, while Alternative 4 shows the most difference with respect to both flow and water surface elevation. Tables showing the differences between calculated statistics for both flow and water surface elevation for below Fort Peck Dam, Wolf Point, Culbertson, Williston, and Lake Sakakawea (elevation only) can be seen in Plate 1 and Plate 2. The statistics calculated include: the 10^{th}, 25^{th}, 50^{th}, 75^{th}, and 90^{th}- Percentiles, and the Minimum and Maximum. It should be noted that the percentile statistics calculated are from a duration analysis and not a Bulletin 17B flow frequency analysis.

The min and max are the lowest daily flow or stage and the highest daily flow or stage output for each alternative over the period of record. For model stability, a minimum flow of 2,500 cfs was used for Fort Peck outflow in RAS. As seen in the tables, the minimum flow varies slightly between alternatives while the maximum flow shows greater differences. Caution should be used when trying to draw conclusions from the statistics alone. The economic models (HEC-FIA) provide a more complete analysis of how high flows effect total damages for each alternative because they incorporate all of the cross section output, whereas these tabular statistics only capture one location.

Stage statistics have been rounded to the nearest tenth of a foot, which is equivalent to 1.2 inches. This helps demonstrate how flow changes impact river elevations, which is the more tangible result. For example, even though the 50^{th} percentile flow for Williston in Alternative 6 was 132-cfs higher than in Alternative 1, there is less than an inch of impact to the water surface elevation of the river, and therefore zero reported change.

It is also important to note that the RAS alternative models, although they have a 30 minute computation interval, have been configured to report one value per 24 hour period, and unfortunately that one value is not a daily average. The RAS model reports the value that lands on 2400 of each day. The most reasonable output interval was chosen as daily due to the size of watershed being modeled, POR length, and the number of hydrograph locations necessary for HC analysis. This means that slight shifts in timing from alternative to alternative can carry over into the results as small fluxuations in the reported flow. Changes in timing are a small factor, not likely to significantly impact any results evaluation, but should be kept in mind when making comparison at a precise level such as in the statistics tables.

5.2 Seasonal Duration Plots

A duration analysis was also performed for the alternative output. Seasonal duration plots for key main stem locations including below Fort Peck Dam, Wolf Point, Culbertson, Williston, and Lake Sakakawea (elevation only) are shown in Plate 3 through Plate 22. Seasonal dates chosen for the duration analysis coincide with the current System operational seasons: spring (1Mar to 30Apr), summer (1May to 31Aug), fall (1Sep to 30Nov), and winter (1Dec to 28Feb). There are minimal changes in all seasons for most of the reach. The most notable differences can be seen in the pool elevations in Lake Sakakawea and the winter flows for alternative 2.

5.3 Limitations

The analysis relies on the simulation of the 82-year period of record using daily average outflows from a ResSim model input into a fixed bed RAS model, with stage and flow output. While the analysis coupled with species and human considerations models can be used to show relative benefits and potential impacts based on historic flows, there are limitations in the conclusions that can be drawn based on some of the simplifying assumptions.

1. **POR Methodology** - An 82-year period of record, adjusted to current level of depletions, was used and may not be comparable to future conditions. A climate change assessment of the Missouri River basin indicates increases to both temperature and precipitation along with increasing trends in extreme floods and droughts (USACE 2018b). The conditions during a pulse year in the future could vary greatly from the small sample of pulse events included in the POR analysis.

2. **No Risk Analysis** - The Missouri River system as currently operated provides substantial flood damage reduction and benefits to the entire basin. The current ResSim and RAS analysis, which employs an 82-year period of record simulation, shows the potential for negative impacts to flood damage reduction and dam safety for alternatives that include changes in reservoir flow releases. The current study methodology does not simulate a sufficient number of events and possible runoff combinations within the large Missouri River basin to evaluate potential change in downstream flood risk and dam safety.

   Scoping efforts were conducted to determine a Monte Carlo risk analysis methodology capable of assessing impacts to dam safety and flood damage reduction as a result of flow release changes. The risk analysis primary components include further development of the period of record flow data set, ResSim and RAS model modifications, development of levee fragility curves, assignment of uncertainty, assembly and debugging of models, Monte Carlo simulation, analysis of results, and reporting. The Monte Carlo methodology properly assesses the effects of the alternative operation changes because it increases the sample size of flow data and number of combinations of flow periods that may occur in the future so that impacts can be characterized with greater confidence. Without such analysis, the impacts of operational changes will only be known for events and combinations of events that have already occurred. Statistics calculated based on the 82-years of record should therefore be used with caution, and with the understanding of the consequences of using only a small sample of years.

3. **Stable Bed and Floodplain** - The hydraulic modeling to date is based on the existing conditions geometry. The analysis does not account for how the bed of the Missouri River may respond to flow changes. Additionally, the analysis does not try to project where sediment may accumulate in the floodplain or include projections of future change in floodplain roughness that could occur during the POR simulation. This carries with it the necessary assumptions that any bed and floodplain changes would be either negligible or similar between each alternative.

5.4 CHANNEL CAPACITY ANALYSIS

Channel capacity estimates were performed to provide an indication of the flow rate at which bank elevations are overtopped and flow begins to leave the main channel and enter the floodplain. Channel capacity estimates were performed with the one-dimensional RAS model calibrated to 2012 conditions by comparing steady flow profiles with top of bank elevations at each cross section combined with reviewing the best available floodplain topography. Floodplain flow connectivity was not assessed. The estimated channel capacity does not necessarily correlate with the onset of flood damage. In addition, channel capacity is typically highly variable along the channel bank due to wide variation in bank elevations. The quality of the channel capacity estimate is affected by numerous factors including how representative the model cross sections are of river geometry, local channel geometry variation, low spots in bank elevations, and the floodplain topography accuracy. Within the reservoir delta areas where the river enters the downstream lake, the channel capacity estimate is not meaningful. While channel capacity varies within the reach and through time, a range for the entire Fort Peck to Lake Sakakawea reach is 35,000 to 40,000 cfs.

A Monte Carlo risk analysis methodology capable of assessing impacts to dam safety and flood risk as a result of flow release changes would be required to fully assess how an alternative impacts potential flood risk. Refer to the *Summary of Hydrologic Engineering Analysis* (USACE 2018d) for additional details on the risk analysis methodology.

6 CONCLUSIONS

The unsteady RAS model analysis gives a means to systematically evaluate differences in river elevations for various reservoir and habitat alternatives given the limitations presented in Section 5.3. These results can be fed into additional species and human considerations models, such as HEC-FIA, to screen alternatives for relative benefits and potential economic impacts. The outputs should be carefully examined with an eye towards the model limitations and judgment applied where needed to mitigate any potential pitfalls of the hydraulic analysis.

If flow change alternatives are considered for implementation, additional risk and uncertainty analysis is recommended to more comprehensively quantify risk of spring or fall pulse flows.

The analysis presented in this report is based off of the existing conditions RAS model. All of the other reach models had future condition (Year 15) projections made and run through the model. The Year 15 model was deemed unnecessary for this reach due to the lack of flow changes included in the alternatives.

7 REFERENCES

USACE (2018a). *Mainstem Missouri River Reservoir Simulation Alternatives Technical Report.* Omaha, NE, U.S. Army Corps of Engineers.

USACE (2018b). *Missouri River Recovery Management Plan - Climate Change.* Omaha, NE, U.S. Army Corps of Engineers.

USACE (2018c). *Missouri River Recovery Management Plan Time Series Data Development for Hydrologic Modeling.* U.S. Army Corps of Engineers, Northwestern Division, Omaha and Kansas City Districts.

USACE (2018d). *Missouri River Recovery Program Management Plan Environmental Impact Statement, Summary of Hydrologic Engineering Analysis.* U.S. Army Corps of Engineers, Northwestern Division, Omaha and Kansas City Districts.

USACE (2015). *Missouri River Unsteady HEC-RAS Model Calibration Report Appendix A Fort Peck Dam to Garrison Dam.* Omaha, NE, U.S. Army Corps of Engineers.

USFWS (2003). *U.S. Fish and Wildlife Service 2003 Amendment to the 2000 Biological Opinion on the Operation of the Missouri River Main Stem Reservoir System, Operation and Maintenance of the Missouri River Bank Stabilization and Navigation Project, and Operation of the Kansas River Reservoir System.* U.S. Fish and Wildlife Service.

APPENDIX A

FORT PECK DAM TO GARRISON DAM

PLATES

Flow (cfs)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Fort Peck - XS 1769.04							
Alt 1A	5,000	6,100	7,800	10,000	12,900	3,000	77,000
Alt 2A	5,000	6,100	7,700	10,100	12,900	3,000	76,000
Change from Alt 1A	0	0	-100	100	0	0	-1,000
Alt 3A	5,000	6,100	7,900	9,900	12,700	3,000	77,000
Change from Alt 1A	0	0	100	-100	-200	0	0
Alt 4A	5,000	6,100	7,800	10,200	12,900	3,000	77,000
Change from Alt 1A	0	0	0	200	0	0	0
Alt 5A	5,000	6,100	7,800	9,900	12,800	3,000	77,000
Change from Alt 1A	0	0	0	-100	-100	0	0
Alt 6A	5,000	6,100	7,900	10,200	12,700	3,000	77,000
Change from Alt 1A	0	0	100	200	-200	0	0
Wolf Point - XS 1701.31							
Alt 1A	5,187	6,407	8,407	10,820	13,585	2,348	100,221
Alt 2A	5,186	6,370	8,425	10,921	13,635	2,348	99,106
Change from Alt 1A	-1	-38	18	101	49	0	-1,114
Alt 3A	5,187	6,399	8,418	10,802	13,565	2,348	100,221
Change from Alt 1A	0	-9	11	-17	-21	0	0
Alt 4A	5,210	6,427	8,482	10,849	13,608	2,348	99,510
Change from Alt 1A	23	20	75	29	22	0	-711
Alt 5A	5,252	6,449	8,328	10,818	13,657	2,348	99,510
Change from Alt 1A	65	42	-79	-1	72	0	-710
Alt 6A	5,175	6,433	8,455	10,915	13,544	2,348	100,221
Change from Alt 1A	-12	25	47	95	-42	0	0
Culbertson - XS 1620.65							
Alt 1A	5,369	6,543	8,475	10,997	14,088	1,860	106,511
Alt 2A	5,367	6,501	8,543	11,084	14,103	1,860	105,527
Change from Alt 1A	-2	-42	68	87	15	0	-984
Alt 3A	5,368	6,541	8,463	10,954	14,064	1,860	106,511
Change from Alt 1A	-1	-2	-12	-43	-23	0	0
Alt 4A	5,382	6,607	8,552	10,982	14,042	1,860	106,329
Change from Alt 1A	14	64	78	-15	-45	0	-182
Alt 5A	5,427	6,613	8,419	10,976	14,122	1,860	106,330
Change from Alt 1A	58	70	-55	-21	34	0	-181
Alt 6A	5,337	6,566	8,541	11,058	13,992	1,860	106,511
Change from Alt 1A	-32	23	66	62	-96	0	0

Plate 1: Alternative Flow Statistics from POR Duration

Flow (cfs)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Williston - XS 1552.61							
Alt 1A	10,891	13,158	16,779	23,272	37,286	4,747	182,471
Alt 2A	10,948	13,192	17,002	23,272	36,719	4,747	181,327
Change from Alt 1A	57	33	222	1	-568	0	-1,144
Alt 3A	10,893	13,153	16,793	23,294	37,253	4,747	182,473
Change from Alt 1A	2	-5	14	22	-33	0	2
Alt 4A	10,946	13,198	16,904	23,227	37,043	4,747	182,287
Change from Alt 1A	55	40	124	-45	-243	0	-184
Alt 5A	10,881	13,223	16,950	23,258	37,087	4,747	182,285
Change from Alt 1A	-10	65	171	-13	-199	0	-186
Alt 6A	10,930	13,254	16,912	23,197	37,068	4,747	182,469
Change from Alt 1A	39	96	132	-74	-219	0	-2

Plate 1 cont'd: Alternative Flow Statistics from POR Duration

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Fort Peck - XS 1769.04							
Alt 1A	2028.9	2029.5	2030.3	2031.2	2032.3	2027.6	2043.0
Alt 2A	2028.9	2029.5	2030.3	2031.3	2032.3	2027.6	2042.9
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	-0.1
Alt 3A	2028.9	2029.5	2030.4	2031.2	2032.3	2027.6	2043.0
Change from Alt 1A	0.0	0.0	0.0	0.0	-0.1	0.0	0.0
Alt 4A	2028.9	2029.5	2030.3	2031.3	2032.3	2027.6	2043.0
Change from Alt 1A	0.0	0.0	0.0	0.1	0.0	0.0	0.0
Alt 5A	2028.9	2029.5	2030.3	2031.2	2032.3	2027.6	2043.0
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Alt 6A	2028.9	2029.5	2030.4	2031.3	2032.3	2027.6	2043.0
Change from Alt 1A	0.0	0.0	0.0	0.1	-0.1	0.0	0.0
Wolf Point - XS 1701.31							
Alt 1A	1959.2	1959.9	1960.8	1961.9	1962.9	1956.9	1975.8
Alt 2A	1959.2	1959.8	1960.9	1961.9	1962.9	1956.9	1975.7
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	-0.1
Alt 3A	1959.2	1959.9	1960.9	1961.9	1962.9	1956.9	1975.8
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Alt 4A	1959.2	1959.9	1960.9	1961.9	1962.9	1956.9	1975.8
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Alt 5A	1959.2	1959.9	1960.8	1961.9	1962.9	1956.9	1975.8
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Alt 6A	1959.1	1959.9	1960.9	1961.9	1962.9	1956.8	1975.8
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Culbertson - XS 1620.65							
Alt 1A	1887.4	1887.9	1888.7	1889.6	1890.6	1885.5	1903.2
Alt 2A	1887.4	1887.9	1888.7	1889.6	1890.6	1885.5	1903.1
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	-0.1
Alt 3A	1887.4	1887.9	1888.7	1889.6	1890.6	1885.5	1903.2
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Alt 4A	1887.4	1887.9	1888.7	1889.6	1890.6	1885.5	1903.1
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Alt 5A	1887.4	1887.9	1888.7	1889.6	1890.6	1885.5	1903.1
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Alt 6A	1887.4	1887.9	1888.7	1889.6	1890.6	1885.5	1903.2
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.0

Plate 2: Alternative Water Surface Elevation Statistics from POR Duration

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Williston - XS 1552.61							
Alt 1A	1843.8	1844.9	1846.5	1849.1	1852.1	1839.2	1863.1
Alt 2A	1843.8	1844.9	1846.5	1849.0	1851.9	1839.2	1862.8
Change from Alt 1A	0.0	0.0	0.1	0.0	-0.1	0.0	-0.3
Alt 3A	1843.8	1844.9	1846.5	1849.1	1852.1	1839.2	1863.1
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Alt 4A	1843.8	1844.9	1846.5	1849.0	1852.0	1839.2	1863.0
Change from Alt 1A	0.0	0.0	0.0	-0.1	0.0	0.0	-0.1
Alt 5A	1843.8	1844.9	1846.5	1849.1	1852.0	1839.2	1863.0
Change from Alt 1A	0.0	0.0	0.0	0.0	-0.1	0.0	-0.1
Alt 6A	1843.8	1844.9	1846.5	1849.1	1852.0	1839.2	1863.1
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Lake Sakakawea - XS 1391.08							
Alt 1A	1805.8	1817.3	1832.4	1839.3	1843.4	1775.4	1856.8
Alt 2A	1805.0	1817.1	1832.4	1838.8	1842.8	1774.0	1855.8
Change from Alt 1A	-0.9	-0.2	0.0	-0.5	-0.6	-1.3	-1.1
Alt 3A	1806.1	1817.7	1832.4	1839.3	1843.5	1775.5	1856.9
Change from Alt 1A	0.3	0.4	0.1	0.0	0.1	0.1	0.0
Alt 4A	1804.2	1814.6	1830.6	1838.8	1842.7	1773.4	1856.6
Change from Alt 1A	-1.7	-2.7	-1.8	-0.5	-0.7	-1.9	-0.2
Alt 5A	1806.0	1817.4	1831.7	1838.9	1843.0	1775.5	1856.5
Change from Alt 1A	0.2	0.1	-0.7	-0.4	-0.4	0.1	-0.3
Alt 6A	1804.2	1814.8	1831.7	1839.1	1843.0	1773.0	1857.0
Change from Alt 1A	-1.6	-2.5	-0.7	-0.2	-0.5	-2.3	0.2

Plate 2 cont’d: Alternative Water Surface Elevation Statistics from POR Duration

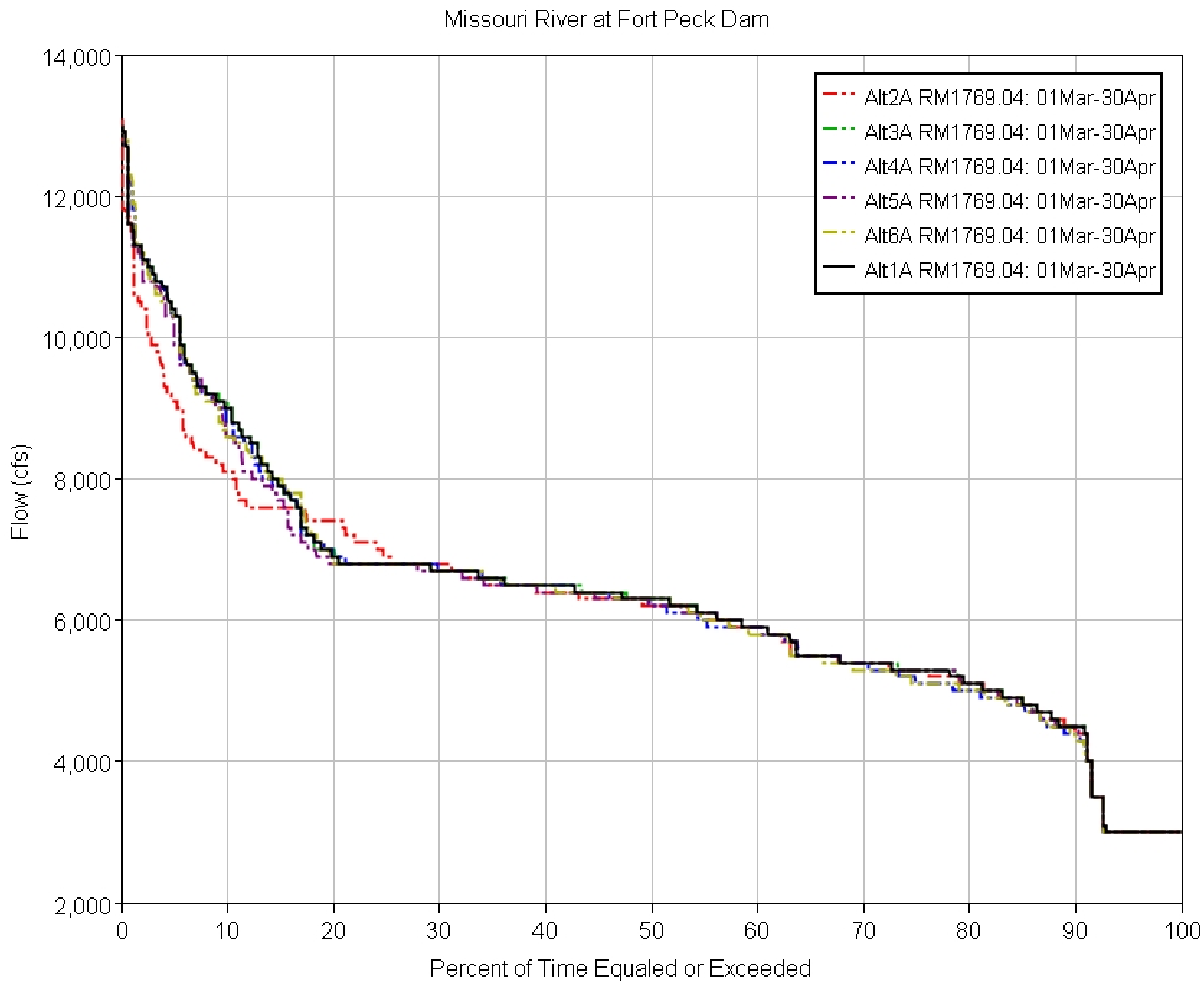


Plate 3: Fort Peck Spring Duration

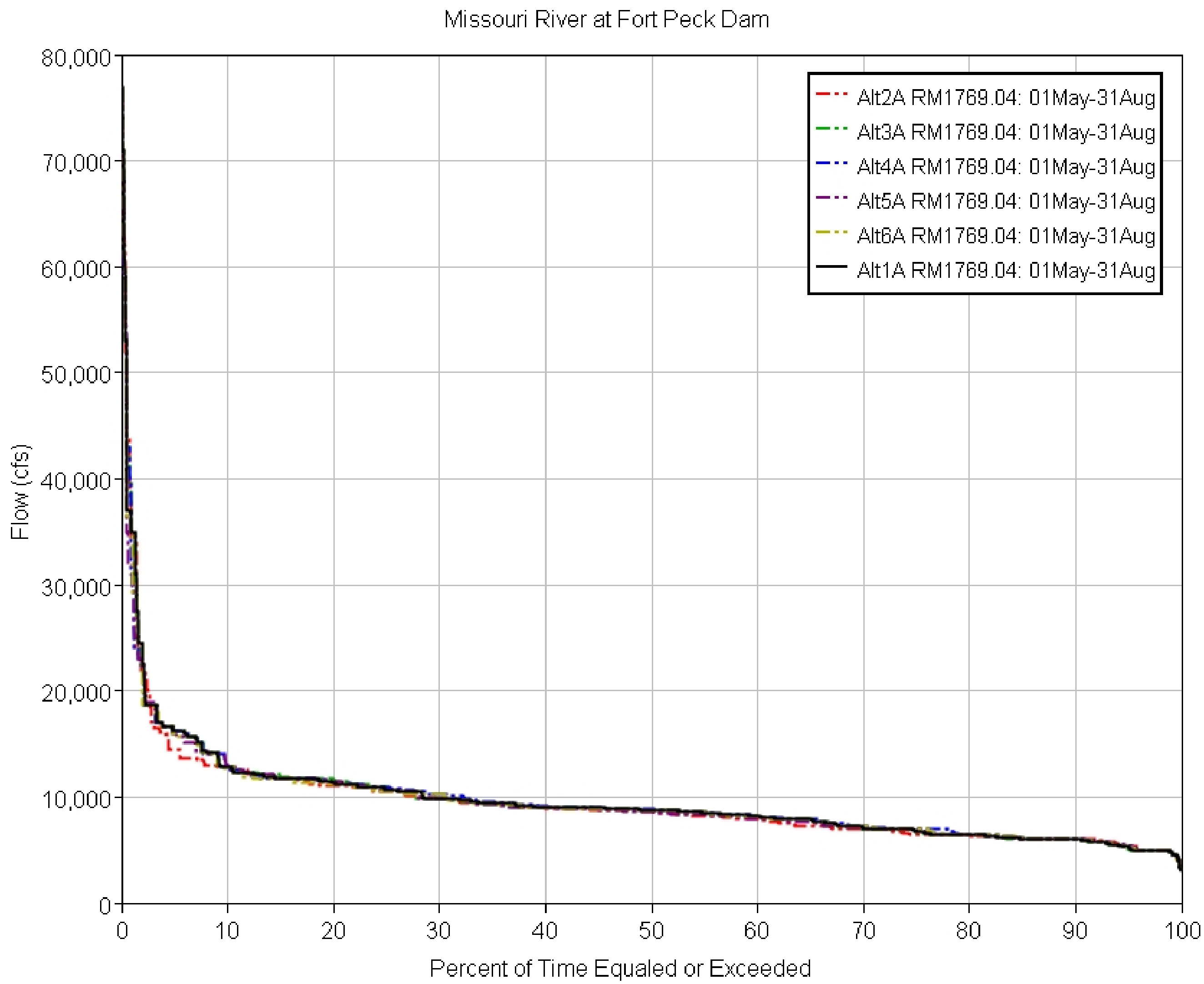


Plate 4: Fort Peck Summer Duration

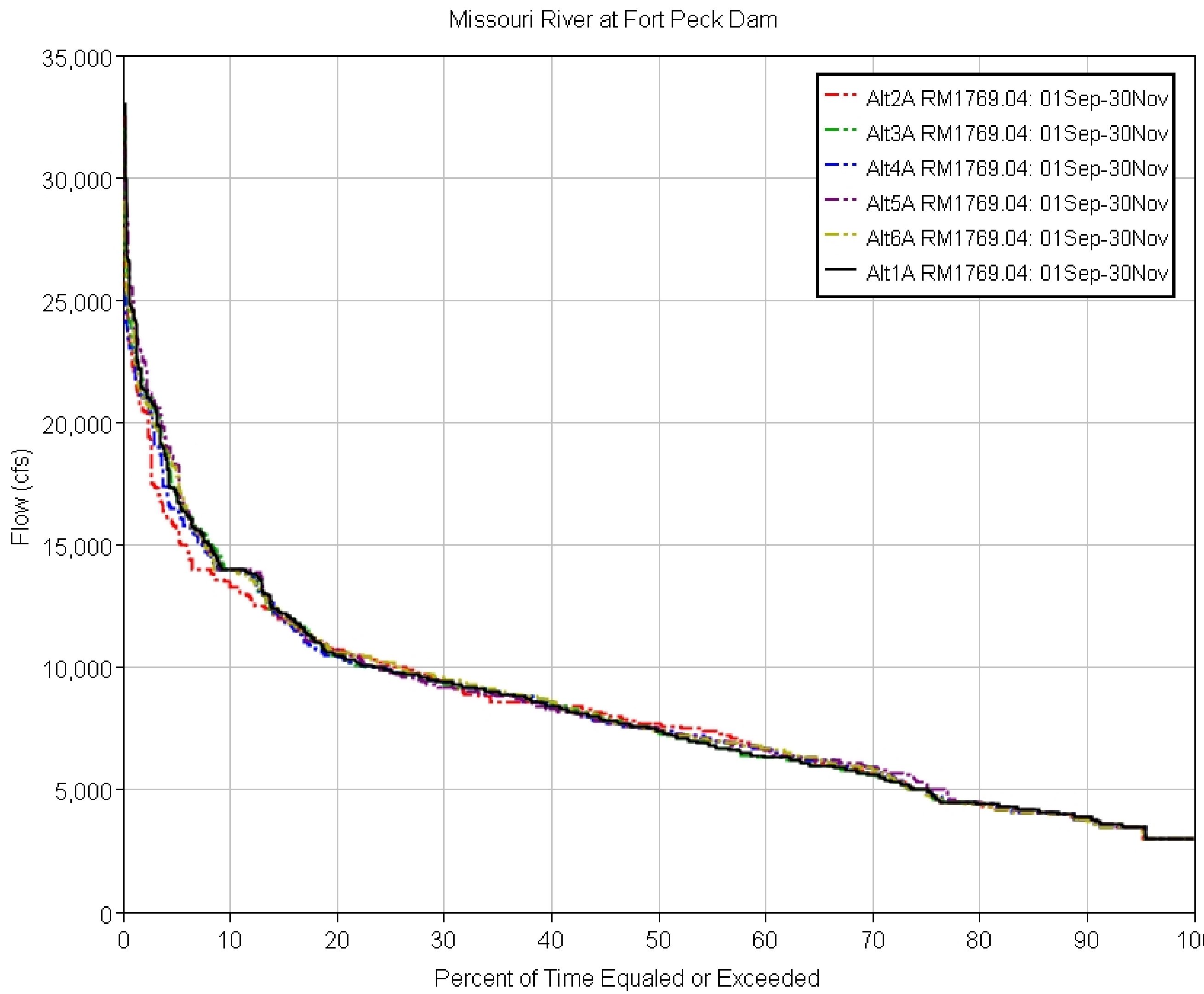


Plate 5: Fort Peck Fall Duration

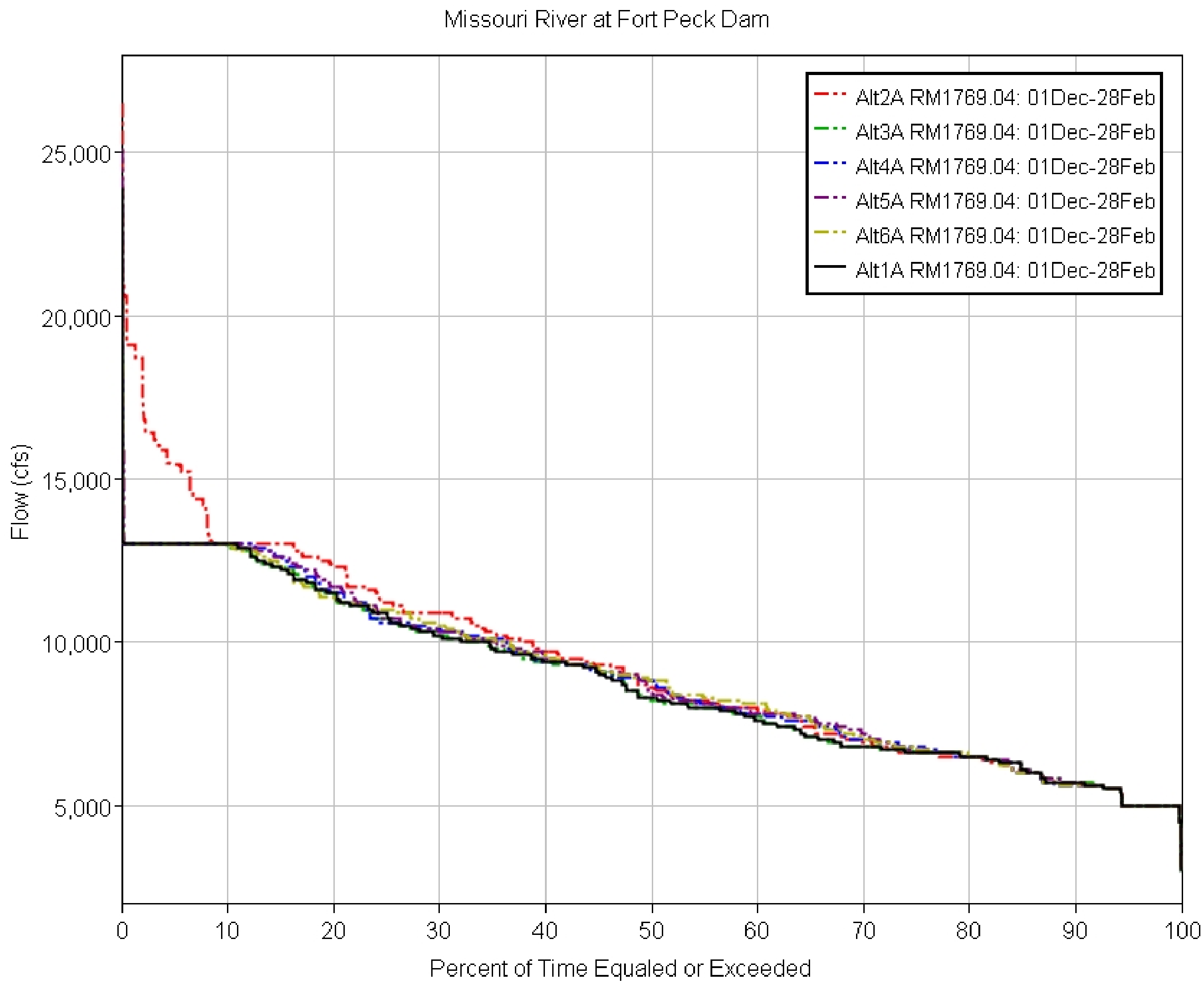


Plate 6: Fort Peck Winter Duration

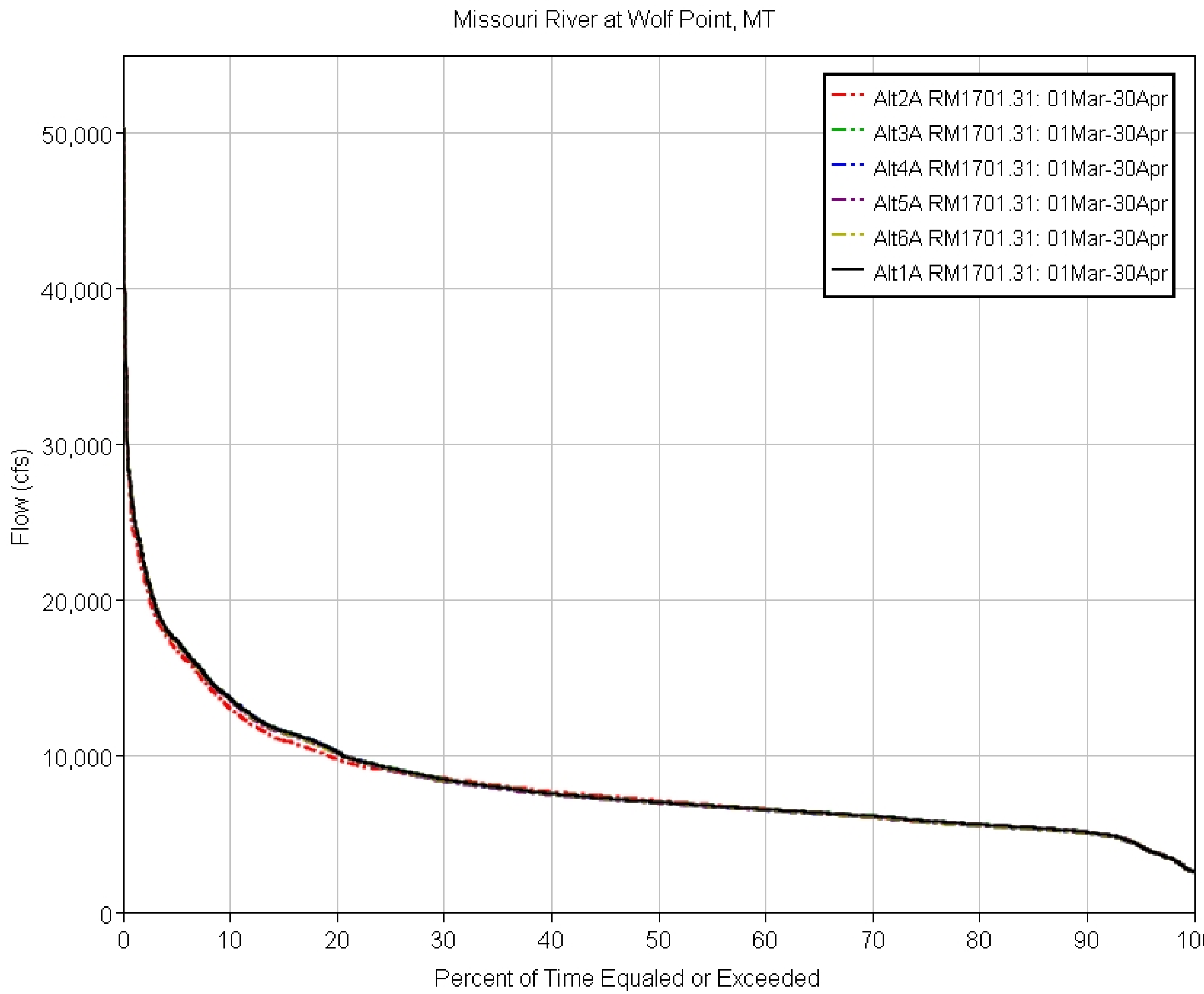


Plate 7: Wolf Point Spring Duration

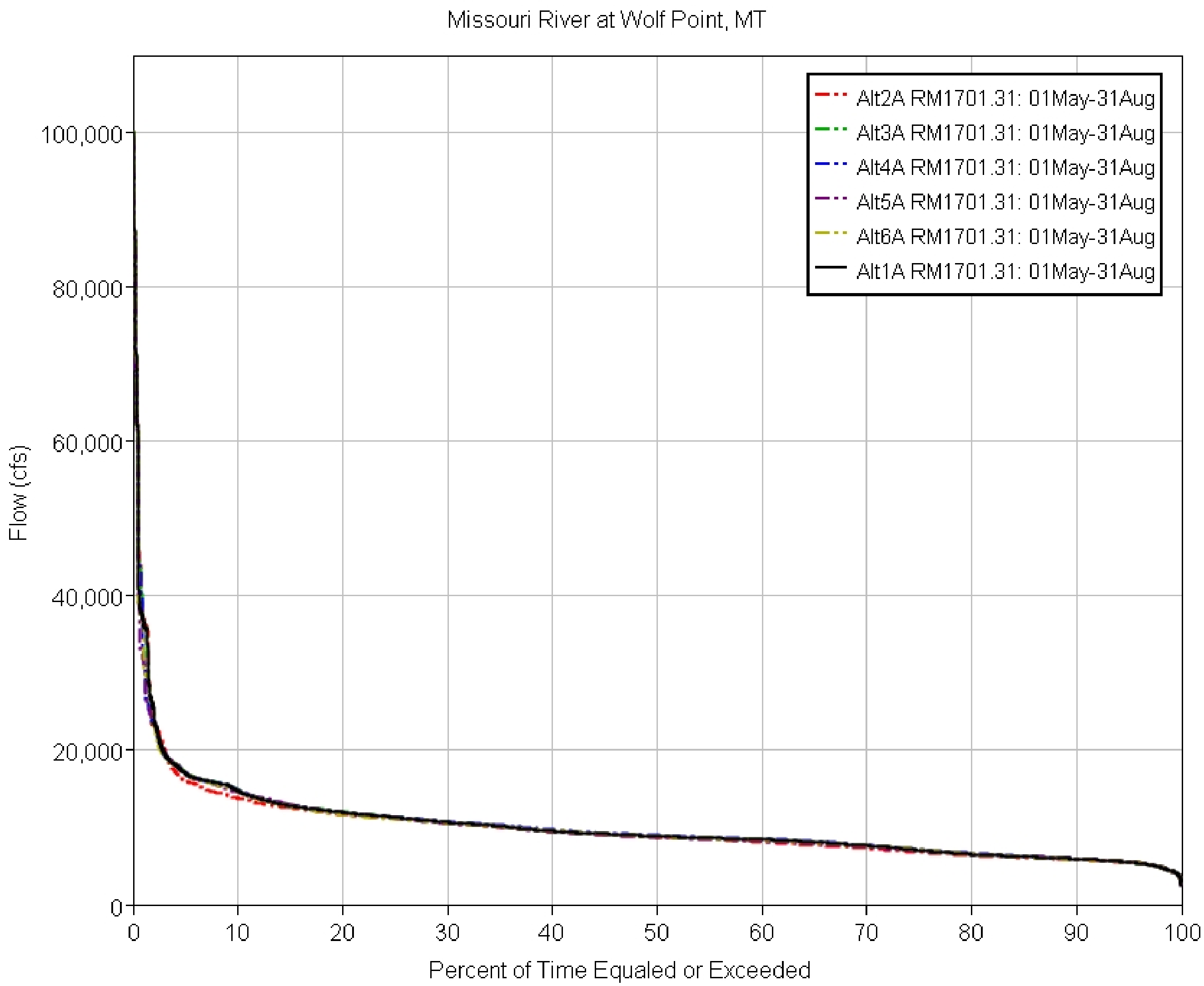


Plate 8: Wolf Point Summer Duration

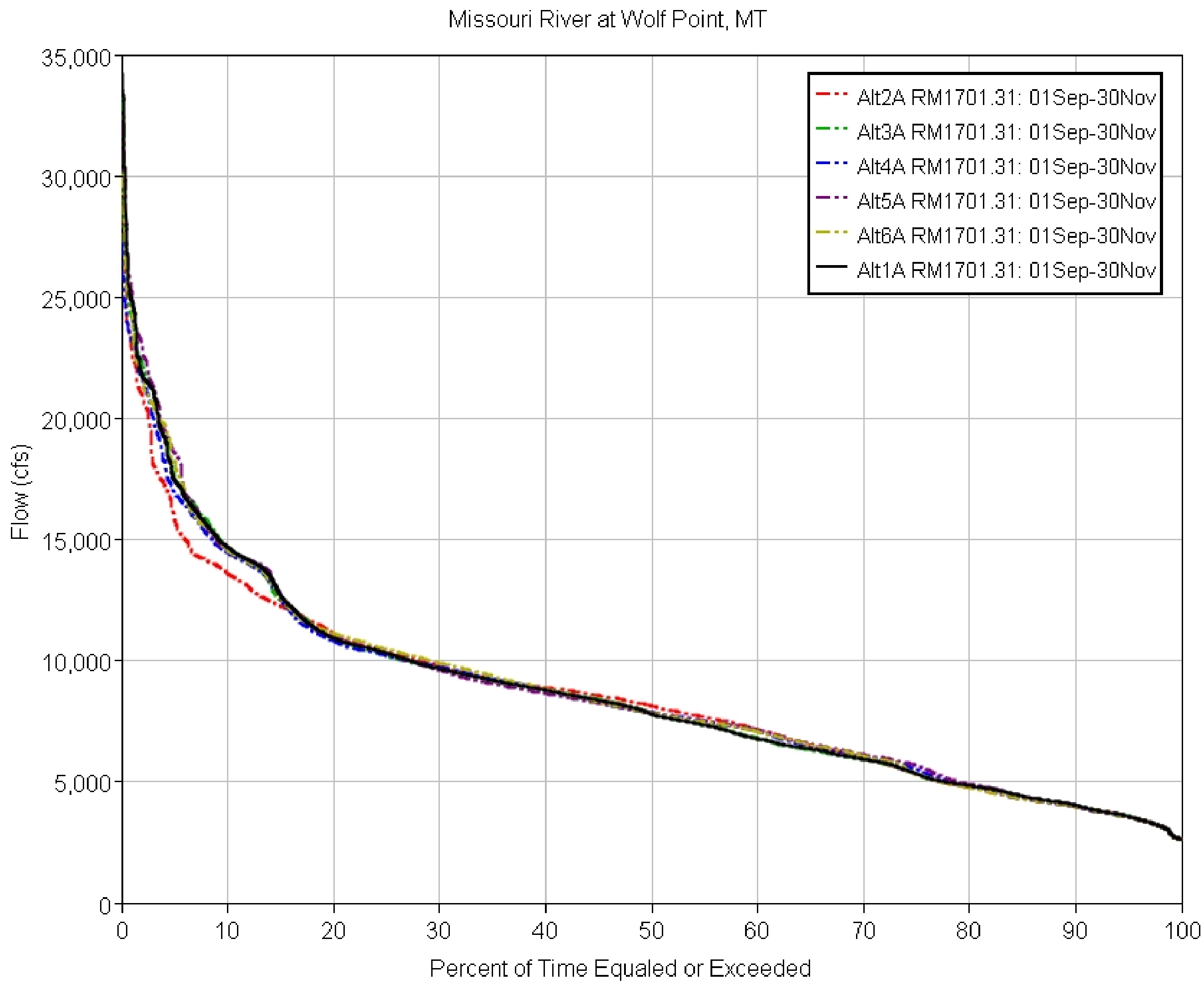


Plate 9: Wolf Point Fall Duration

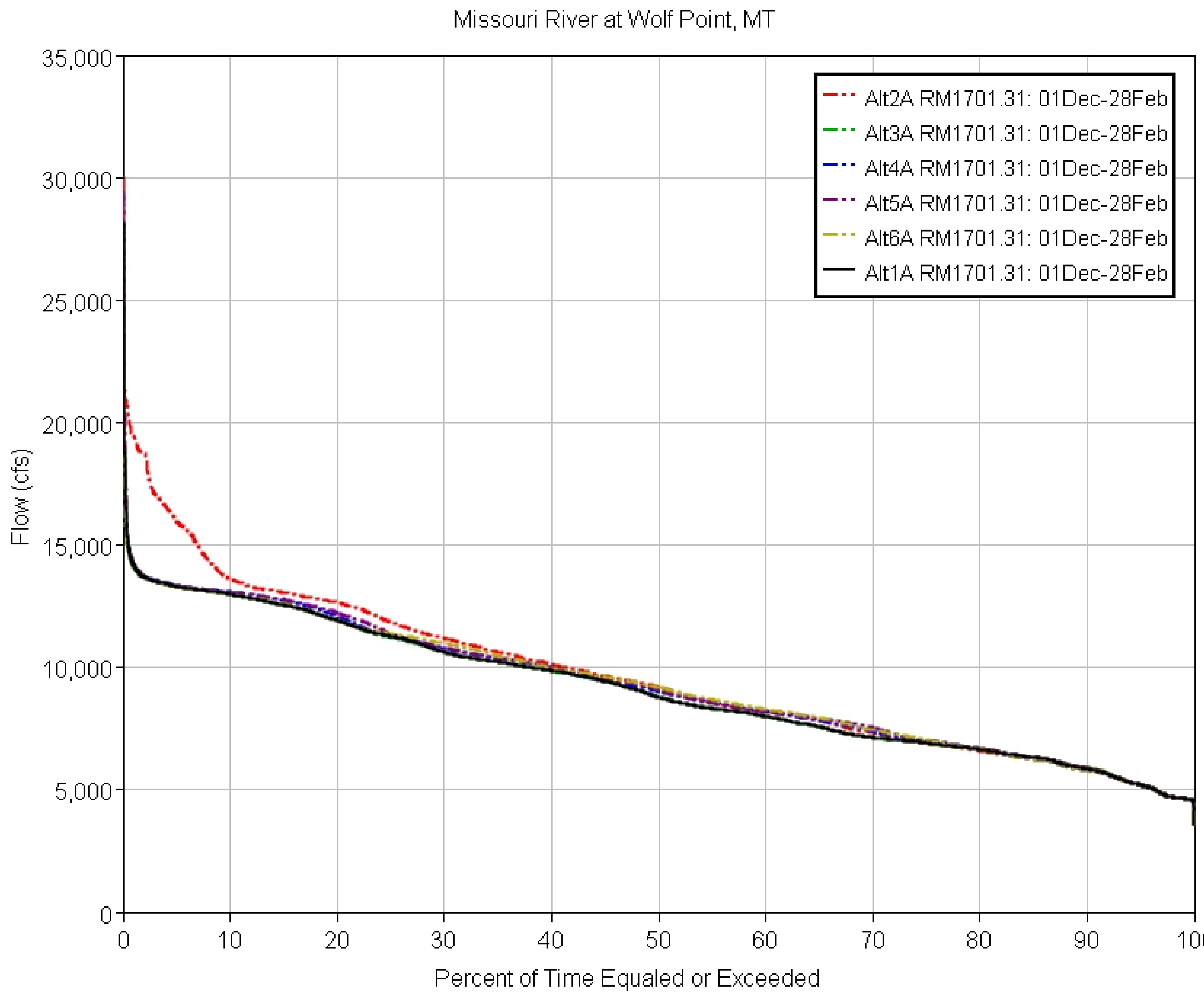


Plate 10: Wolf Point Winter Duration

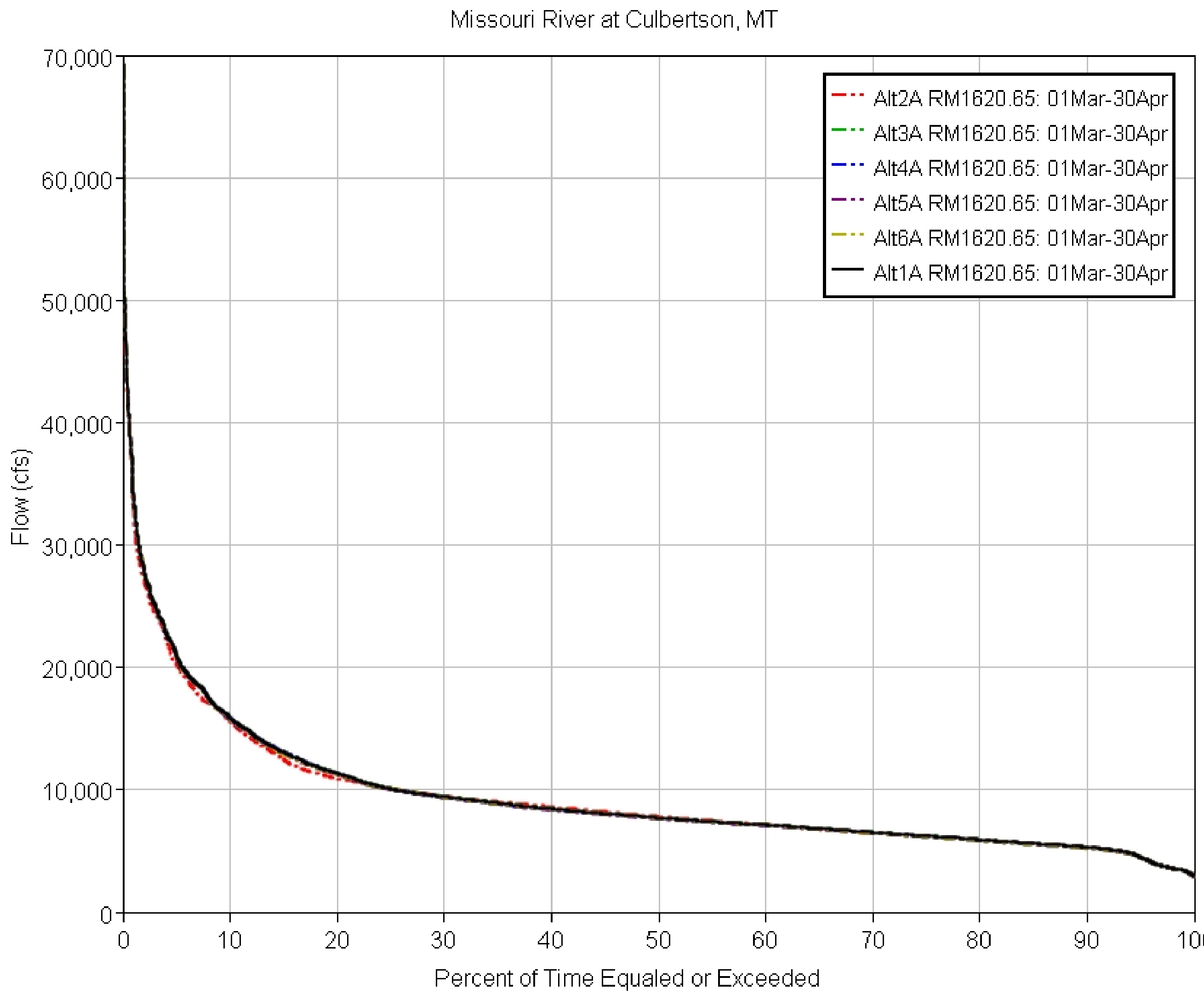


Plate 11: Culbertson Spring Duration

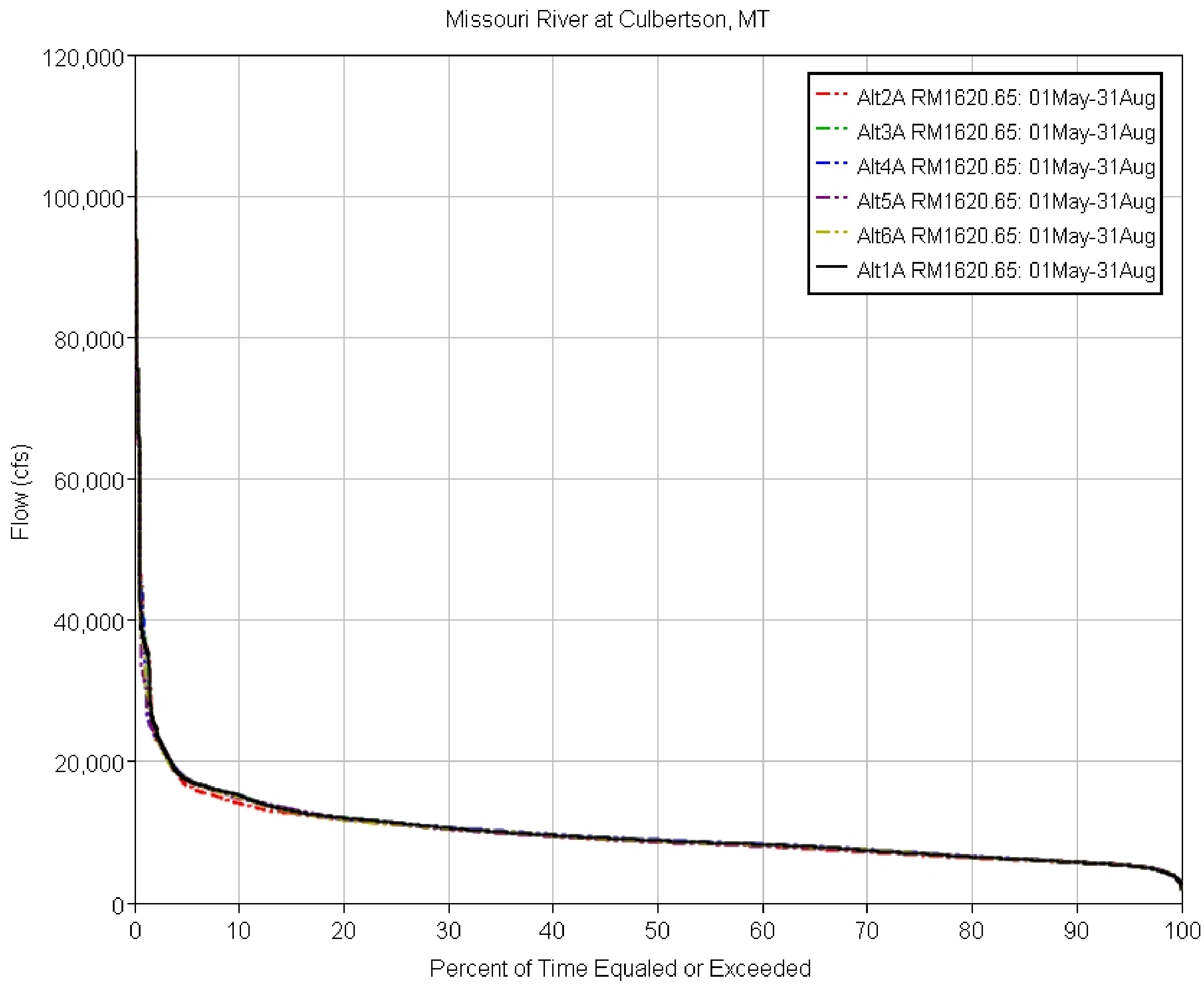


Plate 12: Culbertson Summer Duration

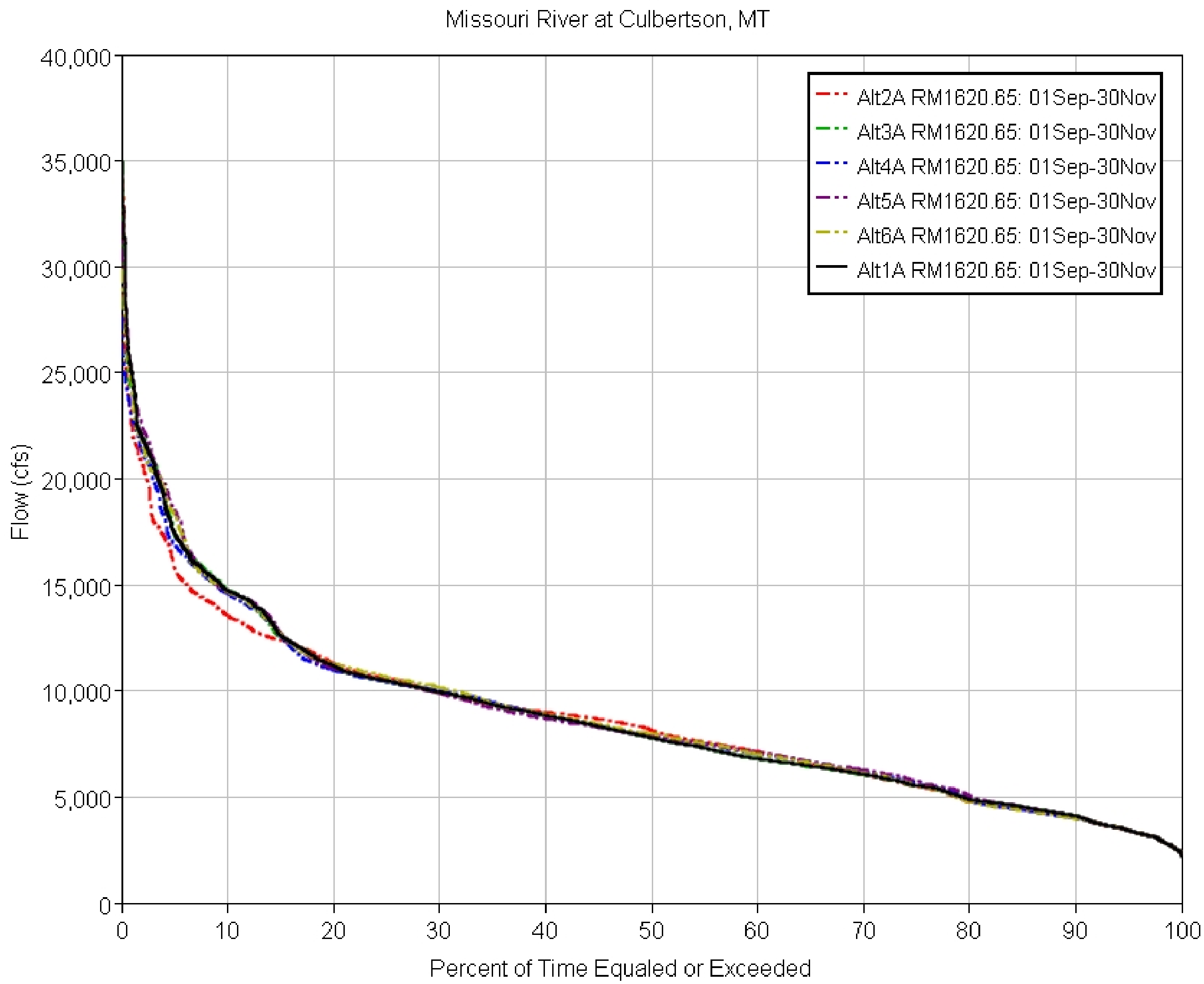


Plate 13: Culbertson Fall Duration

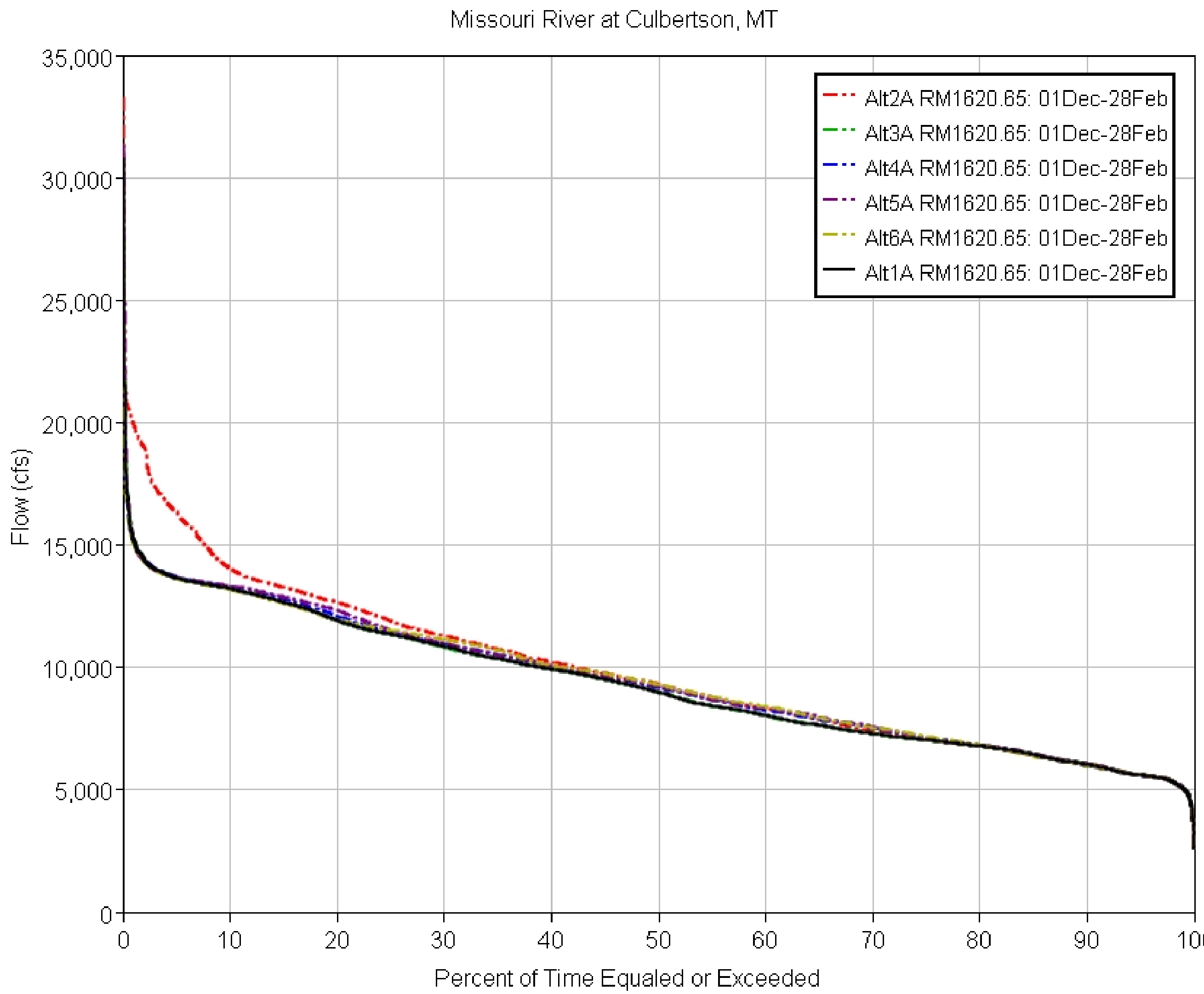


Plate 14: Culbertson Winter Duration

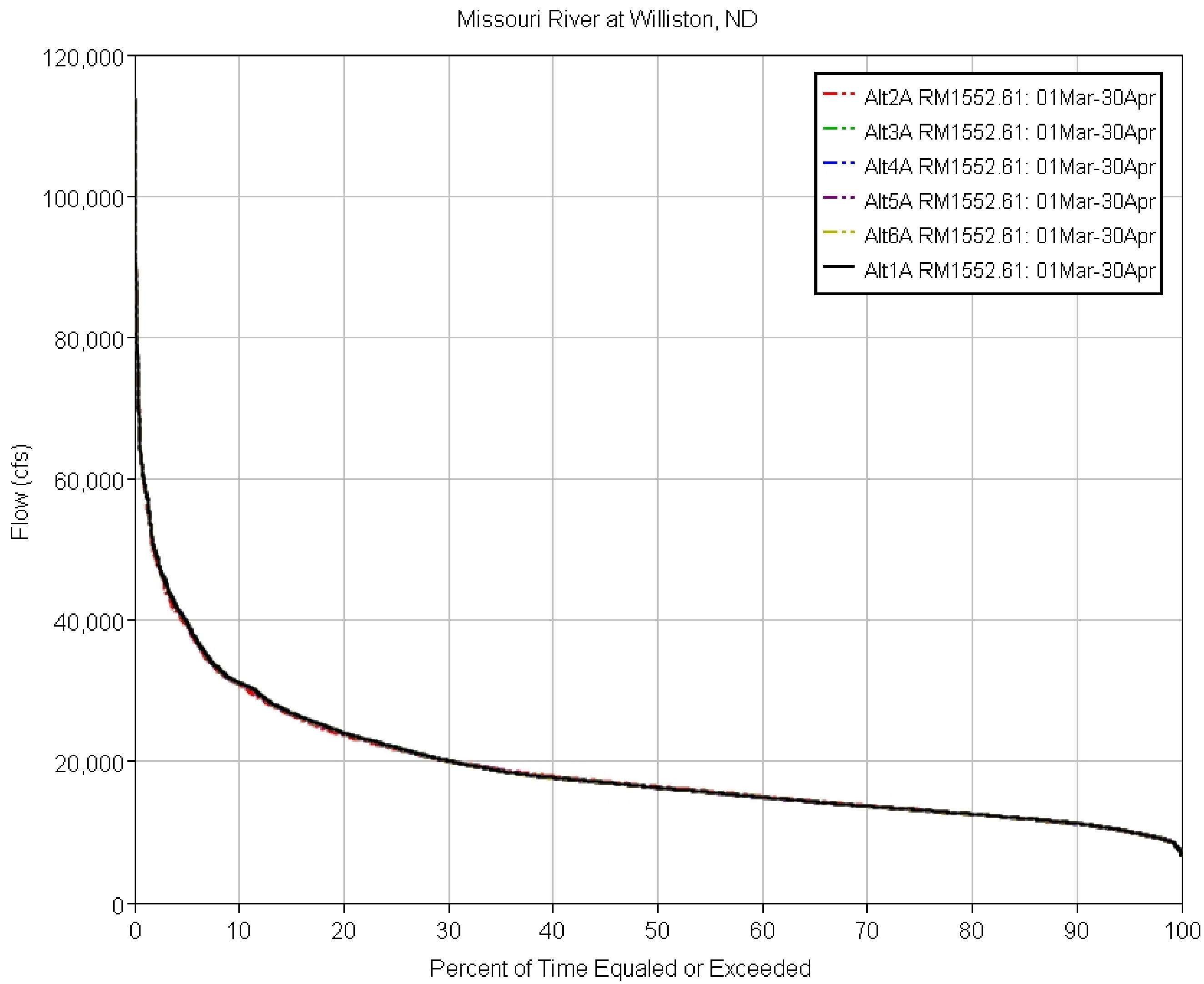


Plate 15: Williston Spring Duration

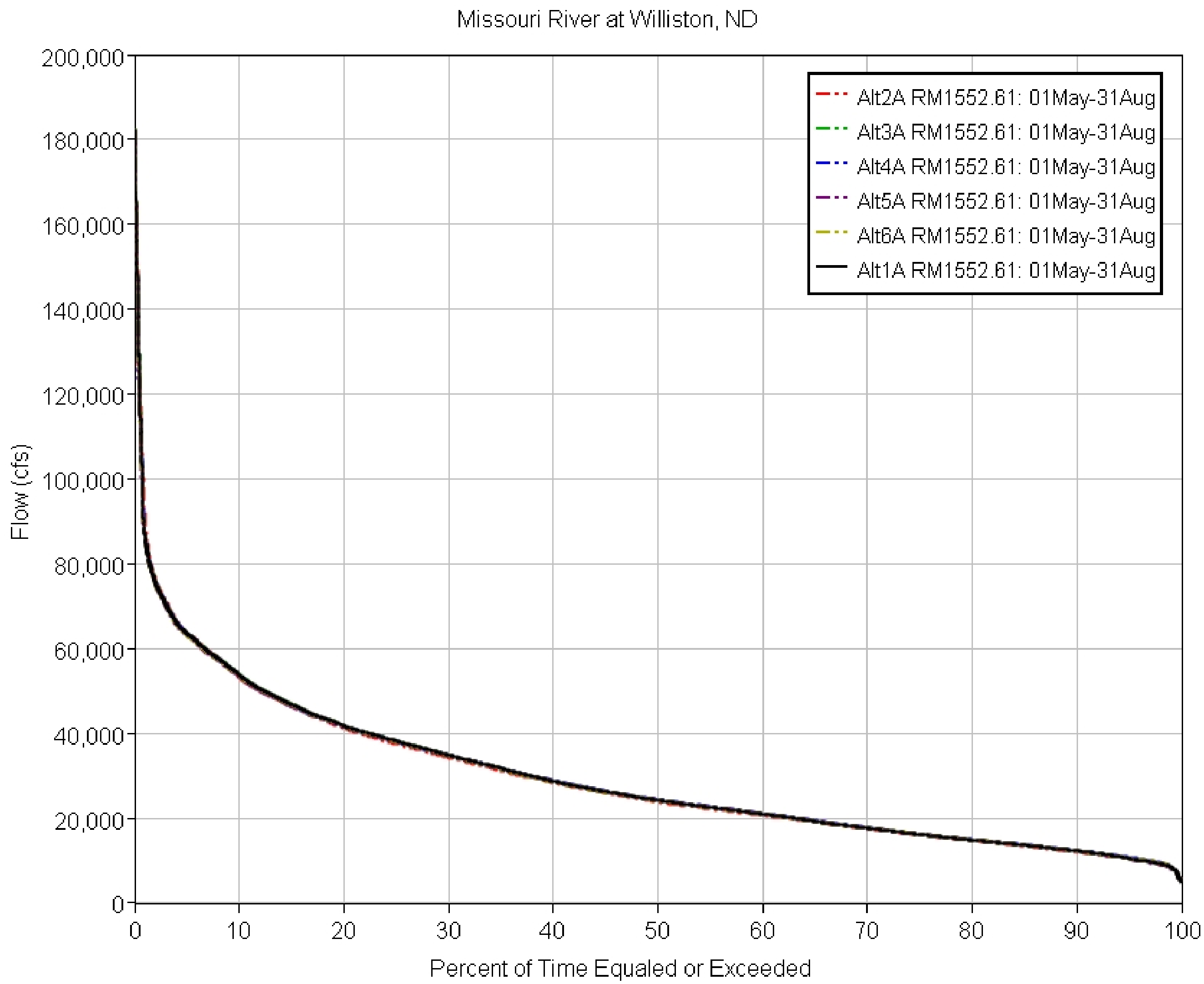


Plate 16: Williston Summer Duration

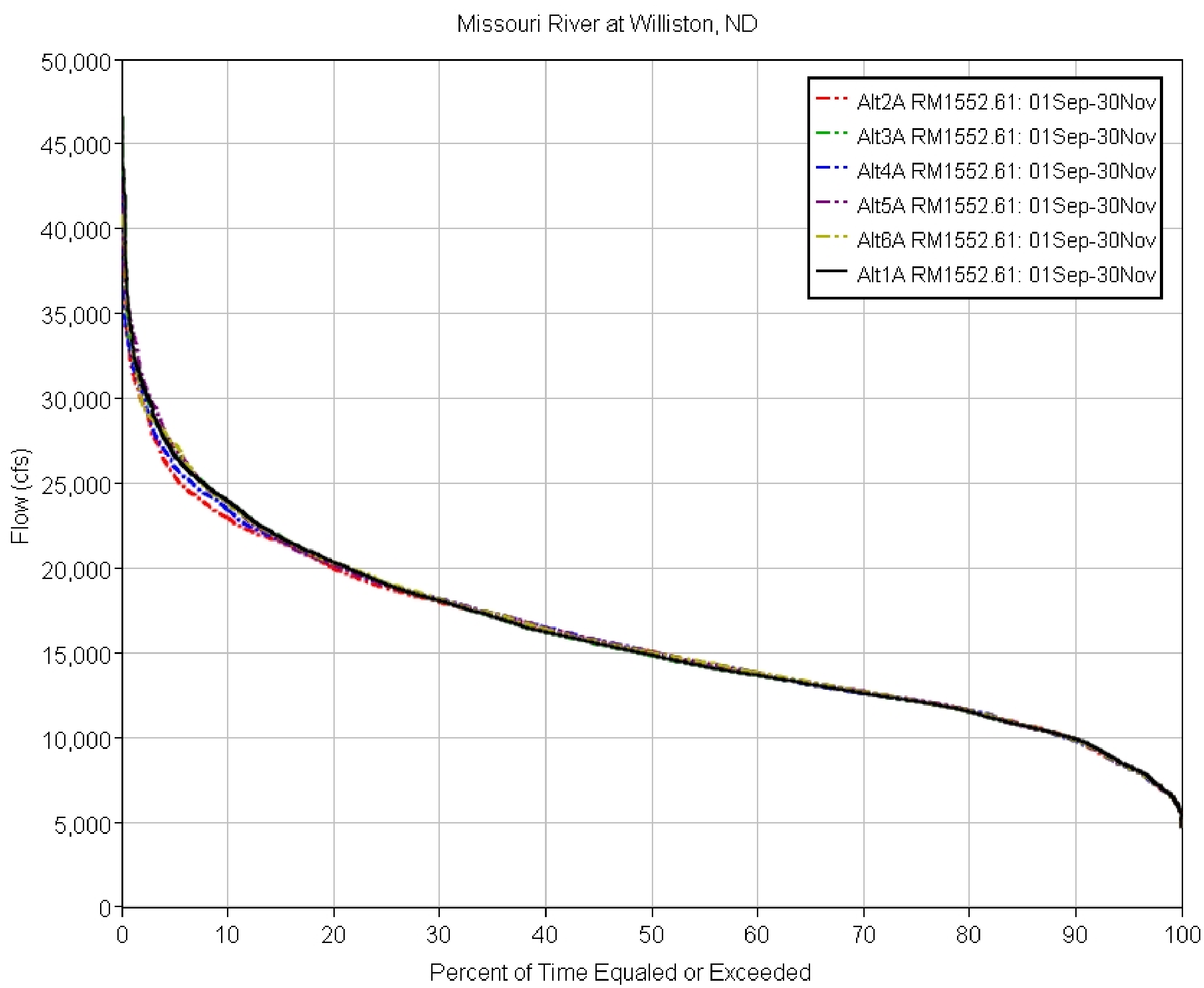


Plate 17: Williston Fall Duration

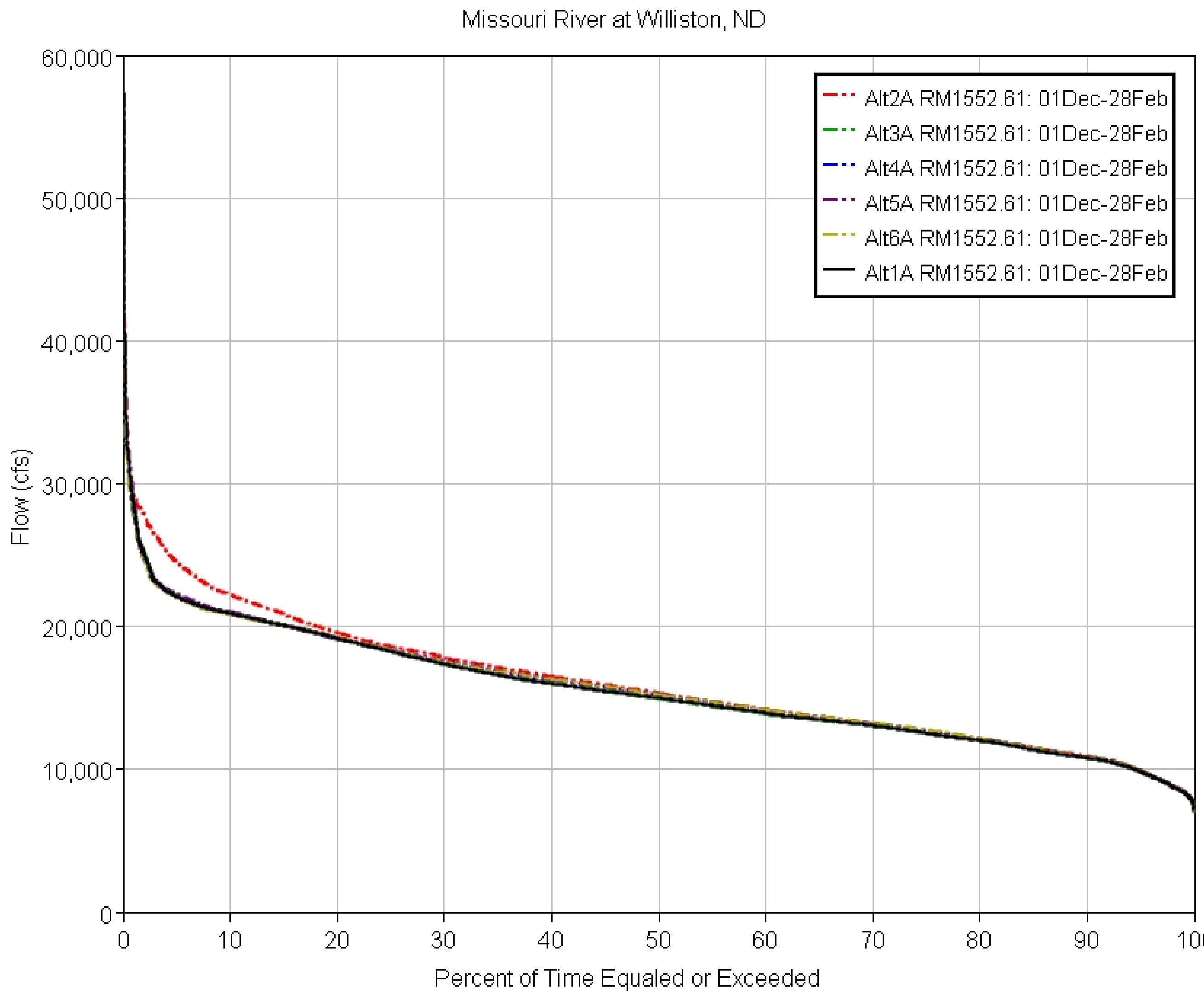


Plate 18: Williston Winter Duration

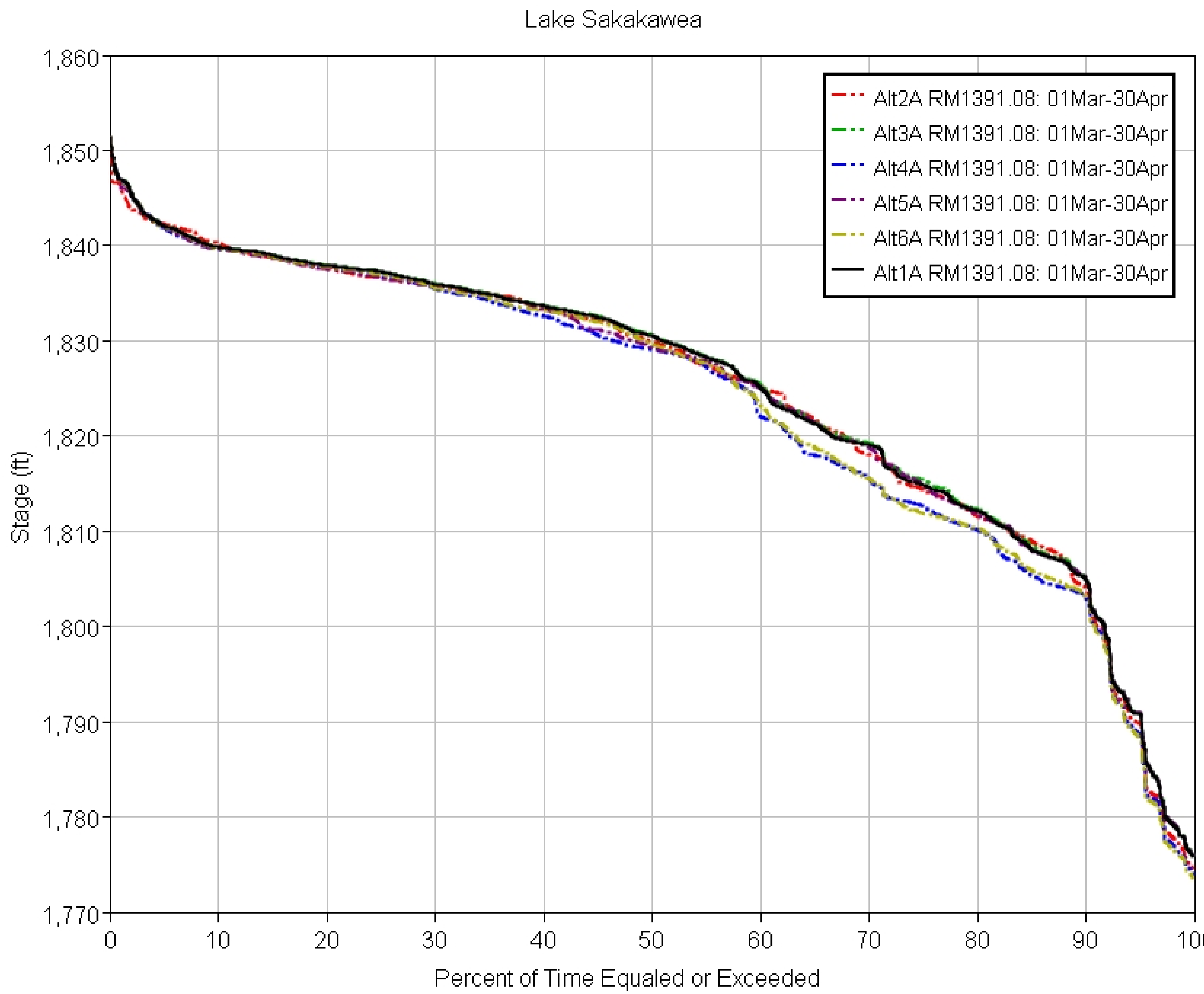


Plate 19: Lake Sakakawea Spring Duration

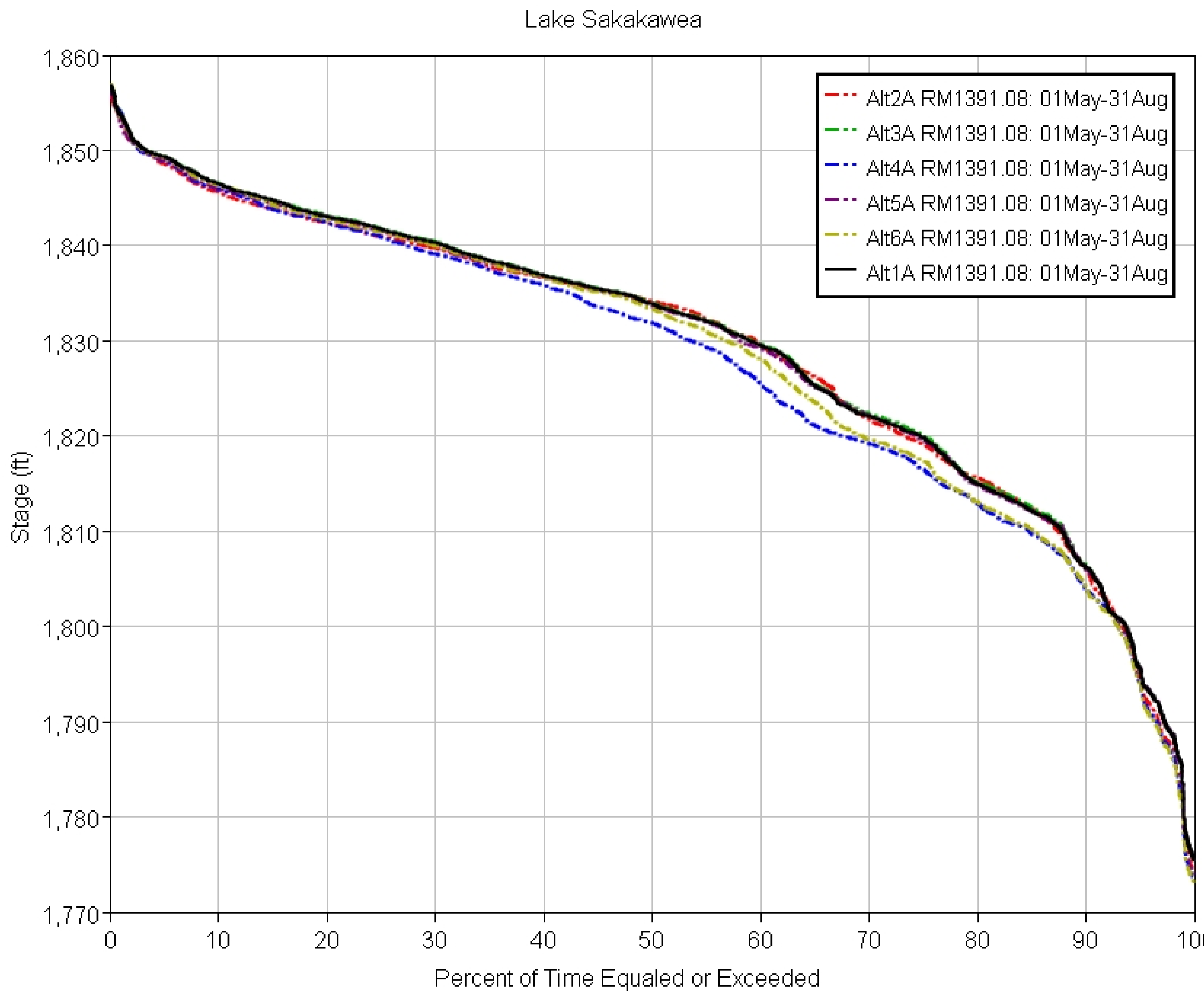


Plate 20: Lake Sakakawea Summer Duration

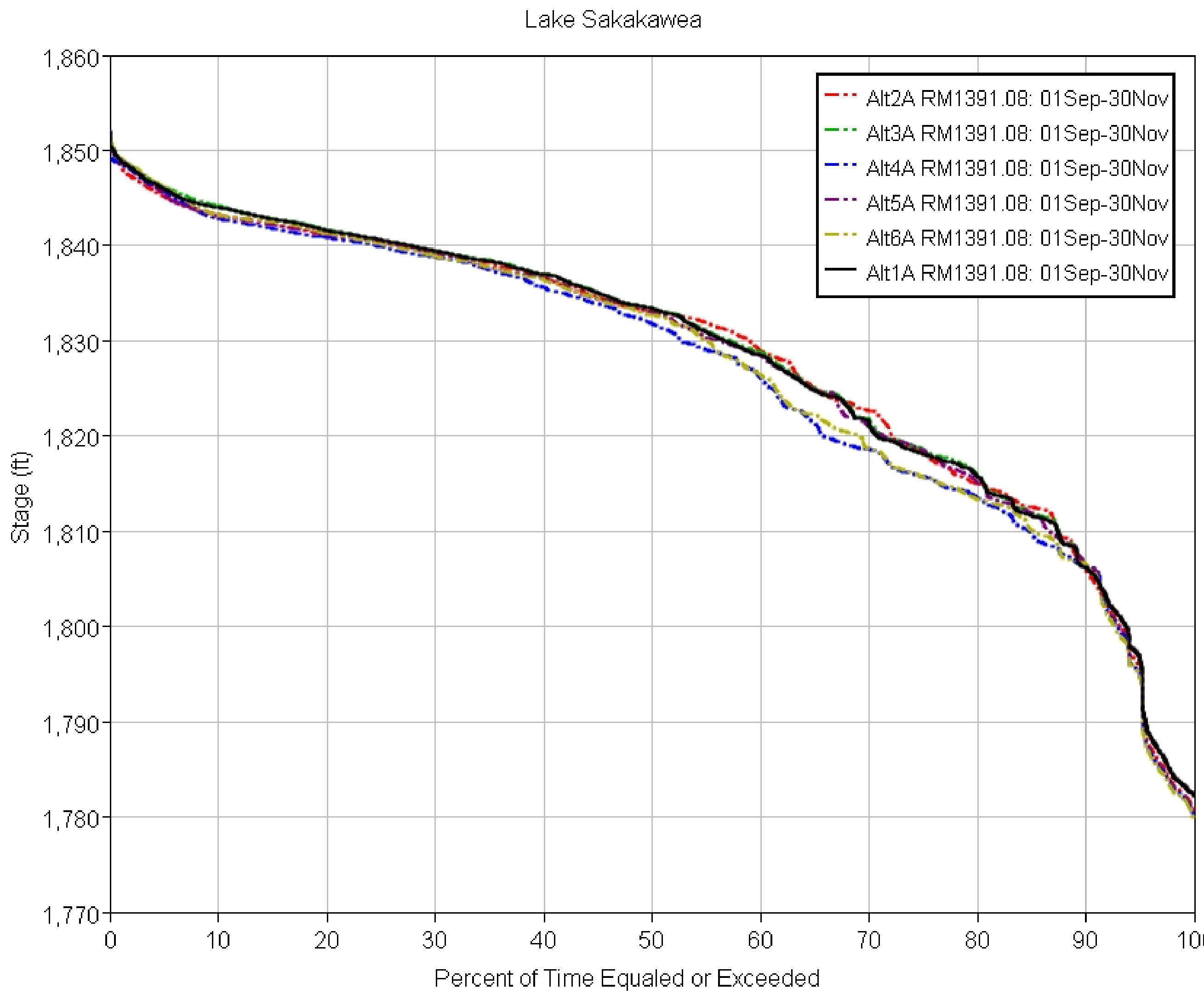


Plate 21: Lake Sakakawea Fall Duration

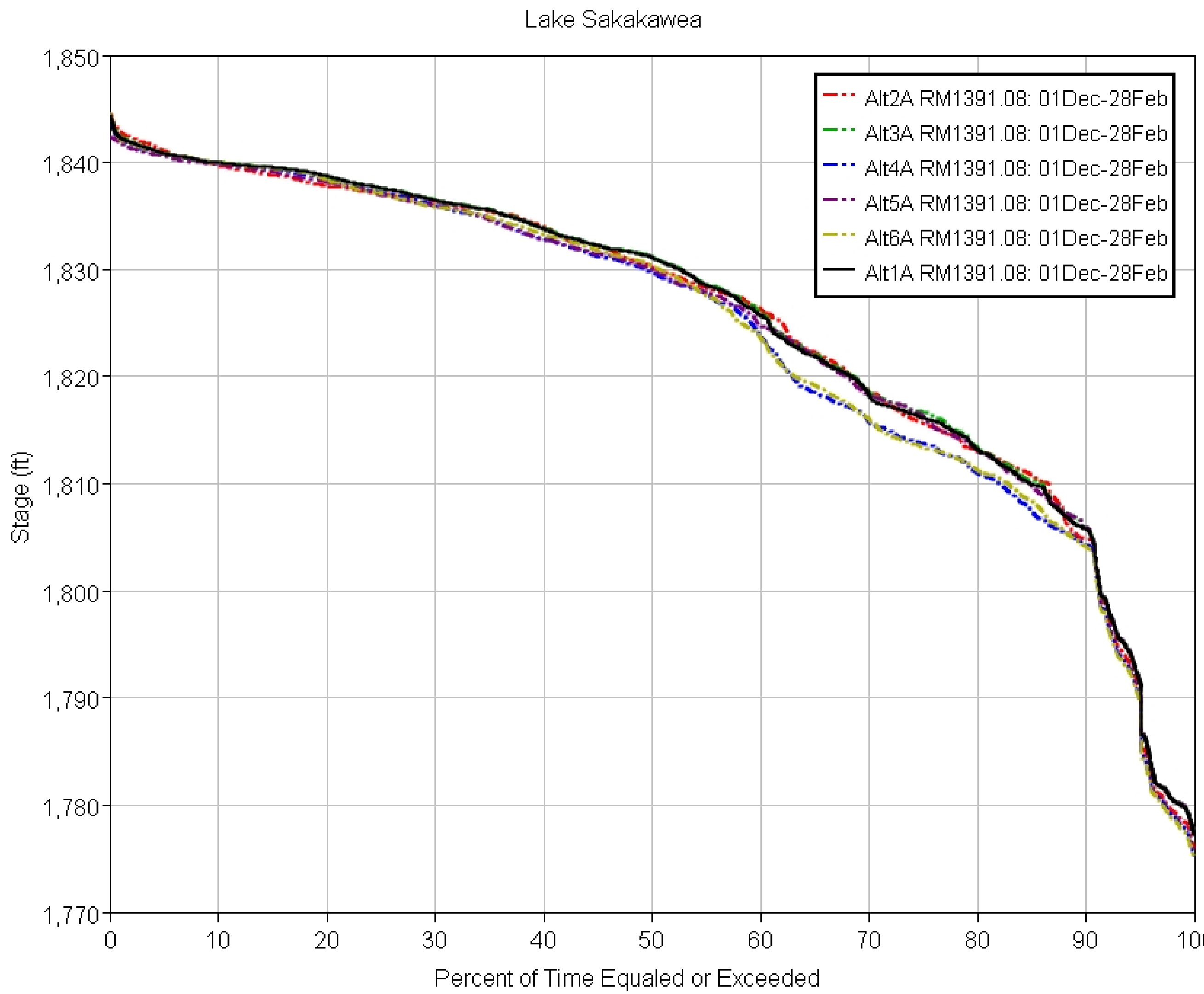


Plate 22: Lake Sakakawea Winter Duration

US Army Corps of Engineers ®

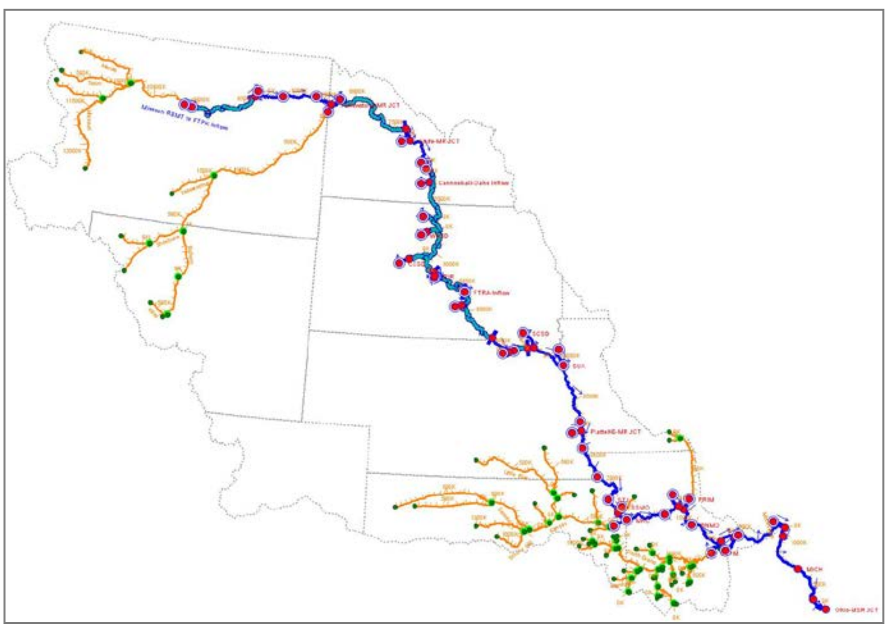
Omaha District

Missouri River Unsteady HEC-RAS Model
Alternatives Analysis

FINAL

Appendix B

Garrison Dam to Oahe Dam



July 2018

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ATTACHMENTS

ACRONYMS

BiOp…………………..Biological Opinion

CFS…………………. Cubic Feet per Second

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

ESH…………………..Emergent Sandbar Habitat

HC…………………….Human Considerations

HEC .... Hydrologic Engineering Center

IRC……………………Interception-Rearing Complexes

MAF………………….. Million acre-feet

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

MRRPMP-EIS….……Missouri River Recovery Program Management Plan Environmental Impact Statement

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

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

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

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

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

POR .... Period of Record

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

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

RM……………………1960 River Mile

ROD…………………...Record of Decision

SWH………………......Shallow Water Habitat

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

USFWS……………….United States Fish and Wildlife Service

USGS .... United States Geological Survey

1 INTRODUCTION

The Missouri River unsteady HEC-RAS (RAS) model was developed for the Missouri River Recovery Program Management Plan and Integrated Environmental Impact Statement (MRRPMP-EIS) to assist in the assessment of a suite of actions to meet Endangered Species Act (ESA) responsibilities for the piping plover, the interior least tern, and the pallid sturgeon using USACE authorities. Model geometry development and calibration for the existing conditions is documented in *Missouri River Unsteady HEC-RAS Model Calibration Report Appendix B Garrison Dam to Oahe Dam* (USACE 2015). The objective of the RAS modeling is to simulate the Management Plan alternatives which include flow changes relative to the No Action alternative. The Human Considerations (HC) team performed an extensive analysis on each of the alternatives for each of the resources (hydropower, cultural resources, fish & wildlife (exclusive of listed species), flood risk, irrigation, recreation, thermal power, and water supply) and provide a detailed comparison of results. For this report, only the hydraulic model output is presented; there is no alternative selection or discussion. This Appendix is for the Garrison Dam to Oahe Dam reach of the Missouri River as part of the Omaha District.

Six alternatives, including the No Action alternative, were simulated in RAS from March 1930 to December 2012, however the HC team only used complete year data for their analysis from January 1, 1931 to December 31, 2012. Development of inflow records at current depletion levels to use as boundary conditions for the HEC-ResSim (ResSim) and RAS models is documented in the report, *Missouri River Recovery Management Plan Time Series Data Development for Hydrologic Modeling* (USACE 2018c). Each alternative has unique flow releases from the reservoirs, as simulated by the ResSim model.

2 GEOMETRY

For the Garrison Dam to Oahe Dam model, no geometry changes were modeled. All alternative runs used the current conditions (2012) calibrated geometry.

3 FLOW ALTERNATIVES

A total of six flow alternatives were modeled in ResSim. Reservoir pool elevations and dam outflow output from the ResSim model was used as input for the RAS models for each of the six flow alternatives. A brief summary of the flow alternatives is provided below. For more details, see the ResSim technical report, *Mainstem Missouri River Reservoir Simulation Alternatives Technical Report* (USACE 2018a).

Tributary and ungaged inflows were kept consistent between alternatives. More details on the Period of Record (POR) flow dataset used can be found in the report *Missouri River Recovery Management Plan Time Series Data Development for Hydrologic Modeling* (USACE 2018c).

3.1 ALTERNATIVE 1 - NO ACTION

Under Alternative 1 (Alt 1), the Missouri River Mainstem Projects would continue to be operated as they are currently. Operations within the ResSim model were set up to closely follow the Master Manual that is used during real-time operations of the System; however, the model does have limitations and cannot capture all real-time decisions that occur.

3.2 ALTERNATIVE 2 - U.S. FISH & WILDLIFE SERVICE 2003 BIOLOGICAL OPINION PROJECTED ACTIONS

Alternative 2 (Alt 2) represents the U.S. Fish and Wildlife Service (USFWS) interpretation of the management actions that would be implemented as part of the 2003 Amended BiOp RPA (USFWS 2003). Operational criteria include different early and late spring spawning cues, low summer flows, and a maximum winter release limit.

3.3 ALTERNATIVE 3 - MECHANICAL CONSTRUCTION ONLY

Alternative 3 (Alt 3) consists of mechanical construction of emergent sandbar habitat (ESH). Operational criteria consist of removing the early and late spring spawning cues in Alt 1.

3.4 ALTERNATIVE 4 - SPRING HABITAT-FORMING FLOW RELEASE

Under Alternative 4 (Alt 4), the early and late spring spawning cues in Alt 1 are removed from the operational criteria and a spring ESH-creating reservoir release from Gavins Point and Garrison is added. While the ESH-creation release is occurring from Gavins Point, flood targets are increased to allow the ESH-creation release the opportunity to run.

3.5 ALTERNATIVE 5 - FALL HABITAT-FORMING FLOW RELEASE

Alternative 5 (Alt 5) removes the early and late spring spawning cues in Alt 1 and adds a fall ESH-creating reservoir release from Gavins Point and Garrison to the operational criteria. While the ESH-creation release is occurring from Gavins Point, flood targets are increased to allow the ESH-creation release the opportunity to run.

3.6 ALTERNATIVE 6 - PALLID STURGEON SPAWNING CUE

Alternative 6 (Alt 6) replaces the early and late spring spawning cues with different spawning cues. The early spring spawning cue in Alt 6 occurs at the same time as the early spring spawning cue in Alt 1 but with a higher peak release. The late spring spawning cue in Alt 6 occurs later in May than the late spring spawning cue in Alt 1 and has a larger peak release. Please note that the former name of this alternative was Alternative 7, which may correspond to some RAS model runs and file names.

4 SIMULATION OF ALTERNATIVES

Each flow alternative from ResSim was run through RAS with the current conditions geometry. Alternative names match in both ResSim and RAS.

5 RESULTS

All alternative runs were performed in HEC-RAS 5.0.3. Model output contains a considerable amount of information, not easily condensed to simple conclusions. Each of the six alternative runs produced 82 years (March 1930 – December 2012) of stage and flow hydrographs. Responses to the ResSim flow changes in combination with habitat geometry changes are complex. To express the changes compared with the No Action alternative, the model results were evaluated by statistical evaluation and duration analysis plots.

Results from the 82-year runs for the six alternatives were provided to the HC team for analysis. They used the daily (instantaneous 2400 value for each day) flow and water surface elevation output to analyze effects to various resources that include: hydropower, cultural resources, fish & wildlife (exclusive of listed species), flood risk, irrigation, recreation, thermal power, and water supply. The HC team performed an extensive analysis on each of the alternatives for all of the resources and provide a detailed comparison of results. For this report, only the hydraulic model output is presented.

5.1 STATISTICS

For the statistical evaluation, daily flow and water surface elevation results were analyzed to compare the differences between the No Action Alternative and the remaining five alternatives. Tables showing the differences between calculated statistics for both flow and water surface elevation for below Garrison Dam, Hensler, Price, Bismarck, Schmidt, and Lake Oahe (elevation only) can be seen in Plate 1 and Plate 2. The statistics calculated include: the 10^{th}, 25^{th}, 50^{th}, 75^{th}, and 90^{th}- Percentiles, and the Minimum and Maximum. It should be noted that the percentile statistics calculated are from a duration analysis and not a Bulletin 17B flow frequency analysis.

The min and max are the lowest daily flow or stage and the highest daily flow or stage output for each alternative over the period of record. For model stability, a minimum flow of 2,000 cfs was used for Garrison Dam outflow. As seen in the tables, the minimum and maximum flow or stage for the period of record remains unchanged from alternative to alternative. Caution should be used when trying to draw conclusions from the statistics alone. The economic models (HEC-FIA) provide a more complete analysis of how high flows effect total damages for each alternative because they incorporate all of the cross section output, whereas these tabular statistics only capture one location.

Stage statistics have been rounded to the nearest tenth of a foot, which is equivalent to 1.2 inches. This helps demonstrate how flow changes impact river elevations, which is the more tangible result. For example, even though the 10^{th} percentile flow for Bismarck in Alternative 2 was 204-cfs higher than in Alternative 1, there is less than an inch of impact to the water surface elevation of the river, and therefore zero reported change.

It is also important to note that the RAS alternative models, although they have a 30 minute computation interval, have been configured to report one value per 24 hour period, and unfortunately that one value is not a daily average. The RAS model reports the value that lands on 2400 of each day. The most reasonable output interval was chosen as daily due to the size of watershed being modeled, POR length, and the number of hydrograph locations necessary for HC analysis. This means that slight shifts in timing from alternative to alternative can carry over into the results as small fluxuations in the reported flow. Changes in timing are a small factor, not likely to significantly impact any results evaluation, but should be kept in mind when making comparison at a precise level such as in the statistics tables.

5.2 Seasonal Duration Plots

A duration analysis was also performed for the alternative output. Seasonal duration plots for key main stem locations including below Garrison Dam, Hensler, Price, Bismarck, Schmidt, and Lake Oahe (elevation only) are shown in Plate 3 through Plate 26. Seasonal dates chosen for the duration analysis coincide with the current System operational seasons: spring (1Mar to 30Apr), summer (1May to 31Aug), fall (1Sep to 30Nov), and winter (1Dec to 28Feb). Alternative 4 shows the most difference in the spring duration due to the spring pulses in that alternative. Alternative 5 shows the most difference in the fall duration due to the fall pulses in that alternative.

5.3 Limitations

The analysis relies on the simulation of the 82-year period of record using daily average outflows from a ResSim model input into a fixed bed RAS model, with stage and flow output. While the analysis coupled with species and human considerations models can be used to show relative benefits and potential impacts based on historic flows, there are limitations in the conclusions that can be drawn based on some of the simplifying assumptions.

1. **POR Methodology** - An 82-year period of record, adjusted to current level of depletions, was used and may not be comparable to future conditions. A climate change assessment of the Missouri River basin indicates increases to both temperature and precipitation along with increasing trends in extreme floods and droughts (USACE 2018b). The conditions during a pulse year in the future could vary greatly from the small sample of pulse events included in the POR analysis.

2. **No Risk Analysis** - The Missouri River system as currently operated provides substantial flood damage reduction and benefits to the entire basin. The current ResSim and RAS analysis, which employs an 82-year period of record simulation, shows the potential for negative impacts to flood damage reduction and dam safety for alternatives that include changes in reservoir flow releases. The current study methodology does not simulate a sufficient number of events and possible runoff combinations within the large Missouri River basin to evaluate potential change in downstream flood risk and dam safety.

   Scoping efforts were conducted to determine a Monte Carlo risk analysis methodology capable of assessing impacts to dam safety and flood damage reduction as a result of flow release changes. The risk analysis primary components include further development of the period of record flow data set, ResSim and RAS model modifications, development of levee fragility curves, assignment of uncertainty, assembly and debugging of models, Monte Carlo simulation, analysis of results, and reporting. The Monte Carlo methodology properly assesses the effects of the alternative operation changes because it increases the sample size of flow data and number of combinations of flow periods that may occur in the future so that impacts can be characterized with greater confidence. Without such analysis, the impacts of operational changes will only be known for events and combinations of events that have already occurred. Statistics calculated based on the 82-years of record should therefore be used with caution, and with the understanding of the consequences of using only a small sample of years.

3. **Stable Bed and Floodplain** - The hydraulic modeling to date is based on the existing conditions geometry. The analysis does not account for how the bed of the Missouri River may respond to flow changes. Additionally, the analysis does not try to project where sediment may accumulate in the floodplain or include projections of future change in floodplain roughness that could occur during the POR simulation. This carries with it the necessary assumptions that any bed and floodplain changes would be either negligible or similar between each alternative.

5.4 Channel Capacity Analysis

Channel capacity estimates were performed to provide an indication of the flow rate at which bank elevations are overtopped and flow begins to leave the main channel and enter the floodplain. Channel capacity estimates were performed with the one-dimensional RAS model calibrated to 2012 conditions by comparing steady flow profiles with top of bank elevations at each cross section combined with reviewing the best available floodplain topography. Floodplain flow connectivity was not assessed. The estimated channel capacity does not necessarily correlate with the onset of flood damage. In addition, channel capacity is typically highly variable along the channel bank due to wide variation in bank elevations. The quality of the channel capacity estimate is affected by numerous factors including how representative the model cross sections are of river geometry, local channel geometry variation, low spots in bank elevations, and the floodplain topography accuracy. Within the reservoir delta areas where the river enters the downstream reservoir, the channel capacity estimate is not meaningful. While channel capacity varies within the reach and through time, a range for the Bismarck, ND area is 55,000 to 60,000 cfs and for the Schmidt, ND area is 35,000 to 40,000 cfs.

A Monte Carlo risk analysis methodology capable of assessing impacts to dam safety and flood risk as a result of flow release changes would be required to fully assess how an alternative impacts potential flood risk. Refer to the *Summary of Hydrologic Engineering Analysis* (USACE 2018d) for additional details on the risk analysis methodology.

6 ADDITIONAL ANALYSIS FOR YEAR 15

Degradation and aggradation of the Missouri River channel bed and sedimentation in the reservoirs are ongoing processes, which have the potential to effect virtually all economic resources and human considerations. Therefore, additional modeling was performed to provide estimates of how ongoing sedimentation processes may affect the performance of future alternatives. The future without and with project condition modeling is based on a number of critical assumptions regarding historic trends, flows, and sediment inputs. While not intended to represent detailed estimates of future channel conditions, the results do provide an alternative comparison methodology. The designation “Year 15” comes from the timeframe for implementation; the Record of Decision (ROD) for the MRRPMP-EIS is expected to be signed in 2018, with a construction completion date of 2033, resulting in an implementation period of 15 years. Results from the Year 15 analysis were provided to economists and human considerations teams for qualitative evaluation, versus the full quantitative evaluation that was performed on the base condition (also referred to as Year 0).

6.1 Year 15 Future Conditions

To project river bed aggradation and degradation trends to the year 2033, moveable bed sediment models of the mainstem Missouri River reaches were created in RAS. Results from the sediment modeling provided a projected change in water surface at a normal flow for each of the six alternatives. Full details on construction and calibration of the sediment model are in Attachment 1, Missouri River Unsteady HEC-RAS Model Sediment Analysis, Garrison Dam to Oahe Dam.

6.2 Alternative Analysis

Alterations were made to both the ResSim and RAS models to represent conditions at the end of the implementation period. All six alternatives were re-run, and results were compared between alternatives and to the base condition.

6.2.1 Geometry

In RAS, the Missouri River channel bed was adjusted to represent the aggradation or degradation that was estimated for the end of the implementation period. Water surface change at a normal flow of 30,000 cfs was used to estimate bed change. Since the forecasted water surface change between alternatives was generally within 0.1 ft, an average of all of the alternatives was used and only one future geometry was created and used for all alternatives. For areas with forecasted degradation, the channel points between the banks were adjusted vertically downward while for areas with forecasted aggradation, the fixed sediment elevation tool was used to produce the forecasted water surface increase. Figure 6-1 shows a cross section that was adjusted for anticipated degradation and Figure 6-2 shows a cross section adjusted for anticipated aggradation.

Figure 6-1. Cross Section Vertical Adjustment for Year 15 - Degradation

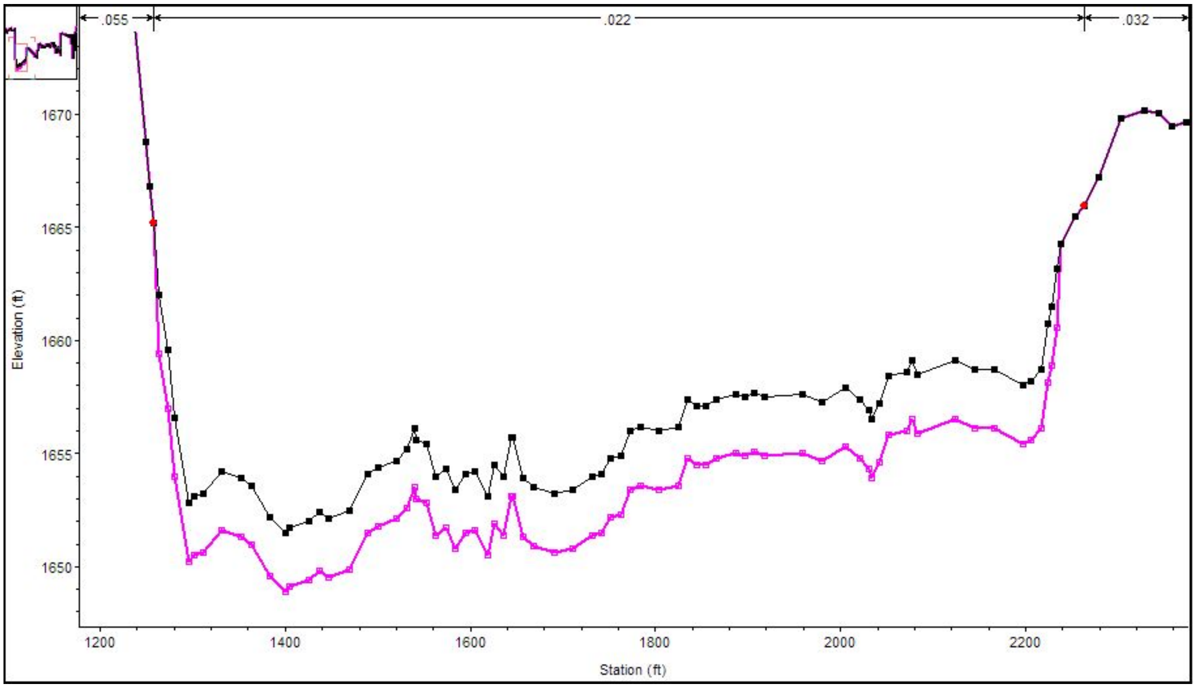
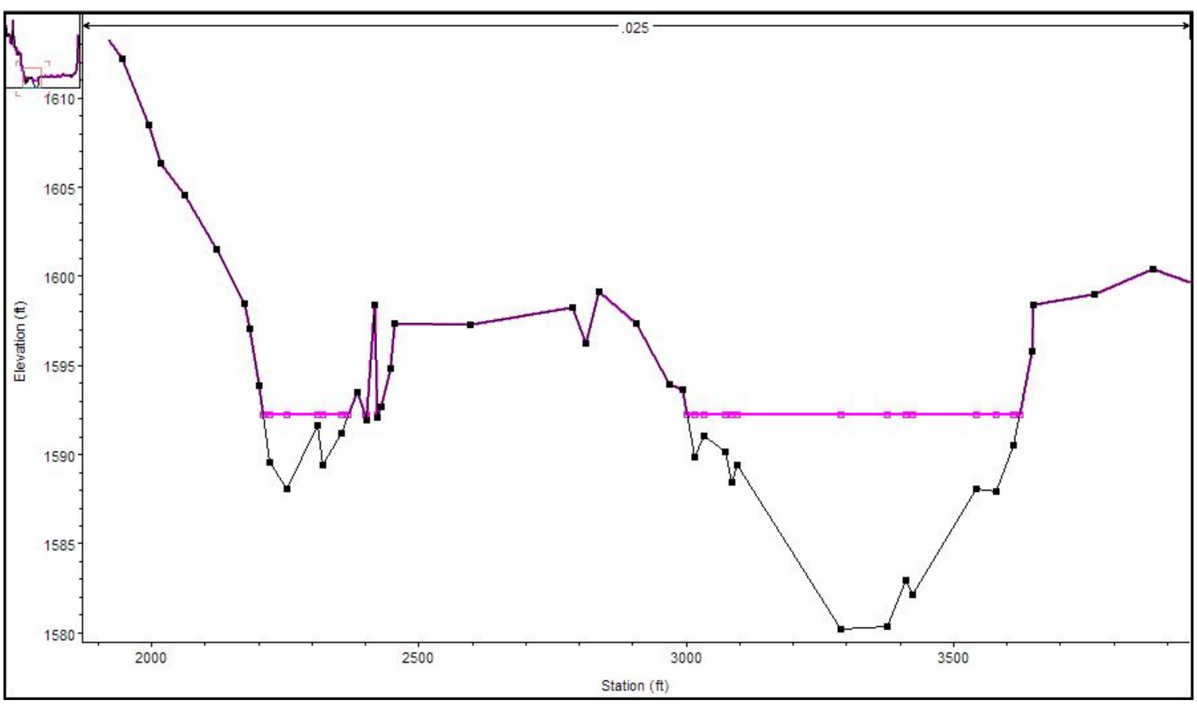
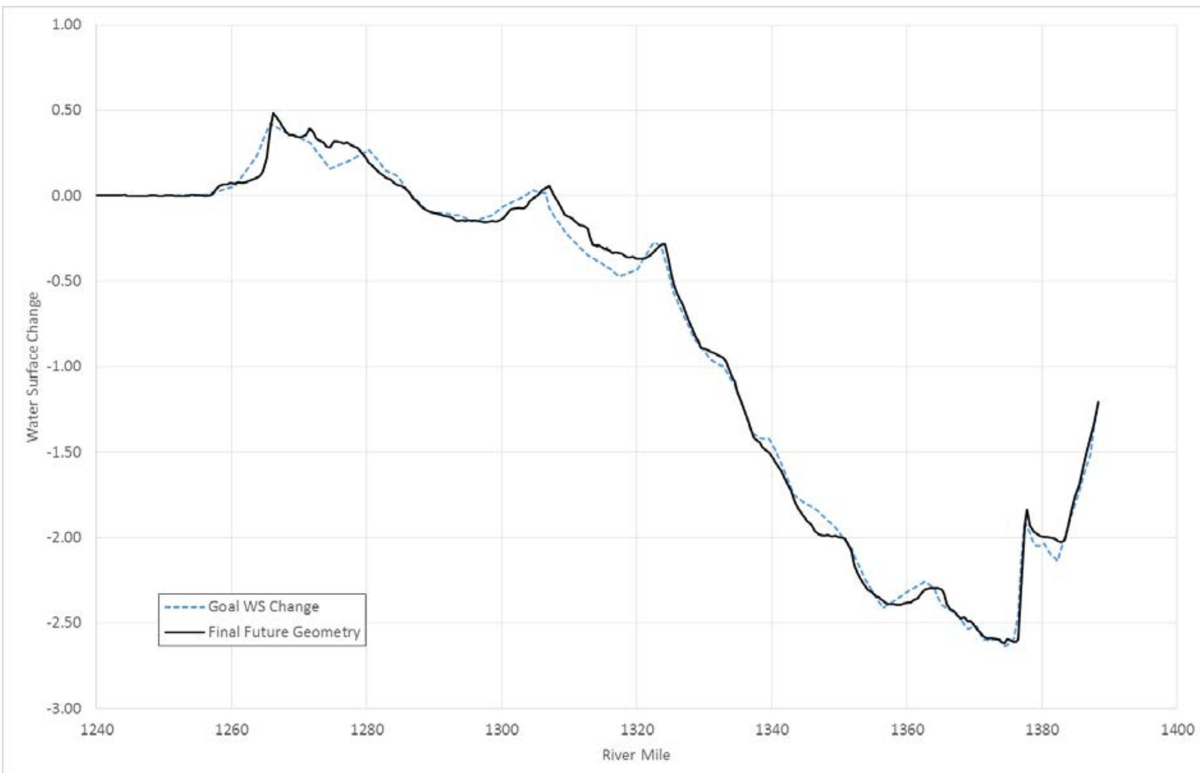


Figure 6-2. Cross Section Vertical Adjustment for Year 15 - Aggradation



For areas of forecasted degradation, a simplified RAS sediment model was used to produce the desired water surface change at a normal flow of 30,000 cfs. Minimum channel elevations were adjusted until the model changed the water surface the desired amount. Similarly, for areas of forecasted aggradation, the fixed sediment elevation tool was used to produce a water surface change. For both aggradation and degradation, water surface changes were usually pretty close to the estimate but due to sediment model limitations, some are within 0.2 ft of the goal. A plot of the water surface change estimate vs what the future geometry actually produces is shown in Figure 6-3. The aggradation reach stops at the river mile of the normal pool level.

Figure 6-3. Year 15 Water Surface Change Estimates vs. Actual



6.2.2 Flows

Reservoir storage volumes were adjusted in ResSim to represent sedimentation in the reservoirs that could be expected by the end of the implementation period. Reservoir operation rules were left the same as the base set of alternatives. All six alternatives were re-run in ResSim for the period of record, resulting in slightly different release decisions from the System. Refer to *Mainstem Missouri River Reservoir Simulation Alternatives Technical Report* (USACE 2018a) for details regarding the ResSim modeling.

6.2.3 Results

Output from the Year 15 analysis can be evaluated in two ways. First, comparison of the Year 15 alternatives to the Year 15 No Action provides additional data to inform on how alternative performance may vary in the future for consideration with the selection of a preferred alternative. Second, comparison of the Year 15 alternatives to the base condition provides a sense of how future channel bed and reservoir sedimentation conditions may impact Missouri River flows and stages. The second evaluation is limited in the useful information it can provide to the decision making process, as the results are directly tied to modeling assumptions that were made about an unknown future using historic data. The first evaluation parallels the comparisons made amongst alternatives in the base condition analysis. Visual and statistical evaluation of the Year 15 output indicates that regardless of changed conditions, the alternatives compare similarly to each other.

Statistical whisker plots for Garrison Dam outflow, Bismarck stage and flow, and Lake Oahe pool elevation are shown in Figure 6-4 through Figure 6-6. The figures present the statistics of minimum, maximum, 5th percentile, 95th percentile, median and mean for all six alternatives for the base condition (Year 0) and Year 15 (Long 2017).

Figure 6-4. Flow Statistics at Garrison Dam

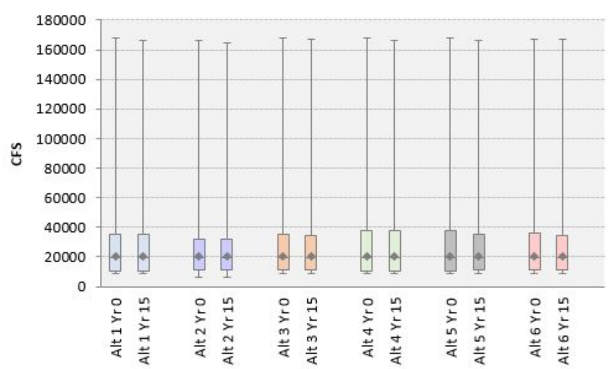


Figure 6-5. Flow and Stage Statistics at Bismarck, ND

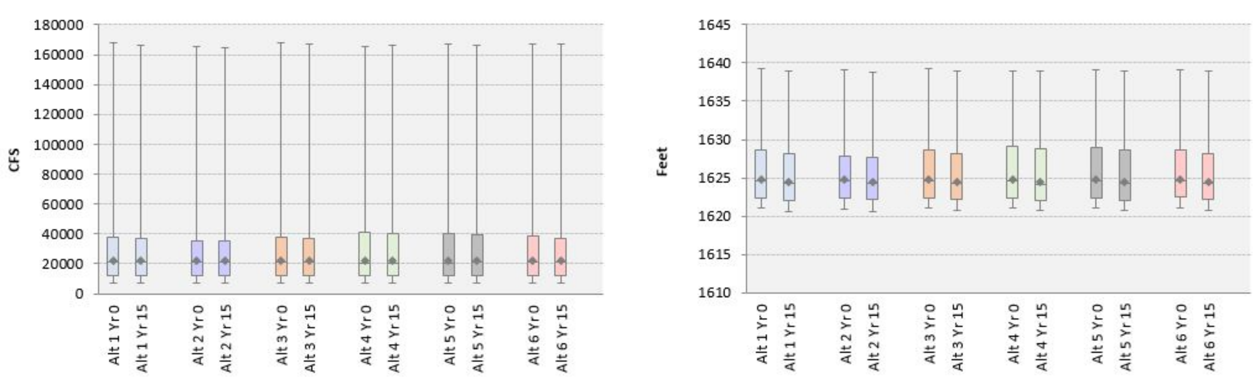
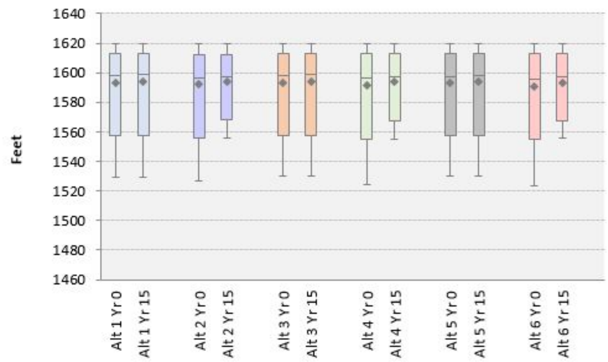


Figure 6-6. Elevation Statistics at Lake Oahe



Although comparisons between Year 15 and Year 0 have limited usefulness in alternative selection, a few trends are worth pointing out. First, differences in flow statistics between Year 0 and Year 15 are almost negligible. Any differences that do show up can be attributed to release changes due to different storage volumes in the reservoir system. These differences decrease moving downstream. Other flow differences compared to Year 0 could be due to slightly different routing because of changed channel capacities. These would accumulate and be greater moving downstream, but they are slight in magnitude and pretty difficult to discern from model noise. Second, differences in stage statistics between Year 0 and Year 15 are directly related to the vertical bed change applied locally. As stated previously, the relative comparison between alternatives for Year 15 produces very similar results to the base condition analysis.

Previously discussed limitations for the alternative analysis also apply to the Year 15 analysis. Although the Year 15 model geometry includes an adjustment to the river bed, the model geometry is static and does not change during the 82 year POR.

7 CONCLUSIONS

The unsteady RAS model analysis gives a means to systematically evaluate differences in river elevations for various reservoir and habitat alternatives given the limitations presented in Section 5.3. These results can be fed into additional species and human considerations models, such as HEC-FIA, to screen alternatives for relative benefits and potential economic impacts. The outputs should be carefully examined with an eye towards the model limitations and judgement applied where needed to mitigate any potential pitfalls of the hydraulic analysis. Additional modeling was performed to provide estimates of how ongoing sedimentation processes may affect the performance of future alternatives. While not intended to represent detailed estimates of future channel conditions, the Year 15 results do provide an alternative comparison methodology that was evaluated qualitatively rather than quantitatively for economic and human consideration impacts. If flow change alternatives are considered for implementation, additional risk and uncertainty analysis is recommended to more comprehensively quantify risk of spring or fall pulse flows.

8 REFERENCES

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APPENDIX B

GARRISON DAM TO OAHE DAM

PLATES

Flow (cfs)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Garrison - XS 1388.30							
Alt 1A	12,200	15,500	19,600	23,600	27,500	9,000	168,000
Alt 2A	12,500	15,800	19,800	23,600	28,000	6,500	166,000
Change from Alt 1A	300	300	200	0	500	-2,500	-2,000
Alt 3A	12,200	15,300	19,600	23,600	28,400	9,000	168,000
Change from Alt 1A	0	-200	0	0	900	0	0
Alt 4A	12,000	15,100	19,300	23,600	28,300	9,000	168,000
Change from Alt 1A	-200	-400	-300	0	800	0	0
Alt 5A	12,000	15,300	19,400	23,200	28,500	9,000	168,000
Change from Alt 1A	-200	-200	-200	-400	1,000	0	0
Alt 6A	12,200	15,600	19,600	23,700	26,900	9,000	167,500
Change from Alt 1A	0	100	0	100	-600	0	-500
Hensler - XS 1362.68							
Alt 1A	12,739	15,986	20,193	24,124	28,936	8,776	168,699
Alt 2A	12,941	16,176	20,535	24,370	29,017	7,871	166,681
Change from Alt 1A	202	191	342	246	82	-905	-2,018
Alt 3A	12,746	15,931	20,154	24,082	29,400	8,796	168,699
Change from Alt 1A	7	-54	-39	-42	464	20	0
Alt 4A	12,620	15,619	19,965	24,075	29,558	8,909	166,987
Change from Alt 1A	-119	-367	-228	-49	622	133	-1,712
Alt 5A	12,656	15,895	20,081	23,867	29,753	8,796	168,693
Change from Alt 1A	-83	-91	-112	-257	818	20	-6
Alt 6A	12,737	16,080	20,302	24,378	28,503	8,909	168,199
Change from Alt 1A	-2	94	110	254	-432	133	-500
Price - XS 1338.15							
Alt 1A	13,102	16,350	20,776	24,786	30,390	7,366	167,664
Alt 2A	13,289	16,599	21,125	25,150	30,195	7,366	165,665
Change from Alt 1A	187	249	348	364	-194	0	-1,999
Alt 3A	13,139	16,341	20,697	24,714	30,673	7,366	167,663
Change from Alt 1A	38	-9	-79	-72	283	0	0
Alt 4A	12,915	16,061	20,574	24,787	30,919	7,366	165,186
Change from Alt 1A	-187	-289	-202	2	529	0	-2,477
Alt 5A	13,008	16,305	20,634	24,578	30,850	7,366	167,616
Change from Alt 1A	-93	-45	-143	-208	460	0	-47
Alt 6A	13,037	16,467	20,903	24,996	30,070	7,366	167,176
Change from Alt 1A	-65	117	126	210	-320	0	-487

Plate 1: Alternative Flow Statistics from POR Duration

Flow (cfs)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Bismarck - XS 1314.80							
Alt 1A	13,173	16,476	20,974	25,021	30,829	7,296	167,685
Alt 2A	13,377	16,794	21,292	25,362	30,634	7,296	165,689
Change from Alt 1A	204	317	317	341	-195	0	-1,996
Alt 3A	13,210	16,473	20,897	24,944	31,058	7,293	167,685
Change from Alt 1A	37	-3	-77	-77	228	-2	1
Alt 4A	12,958	16,205	20,767	25,050	31,313	7,293	165,802
Change from Alt 1A	-215	-272	-207	29	484	-3	-1,882
Alt 5A	13,098	16,418	20,820	24,857	31,235	7,291	167,502
Change from Alt 1A	-75	-59	-155	-164	406	-5	-182
Alt 6A	13,100	16,563	21,114	25,211	30,531	7,293	167,189
Change from Alt 1A	-73	87	140	190	-299	-3	-496
Schmidt - XS 1297.51							
Alt 1A	13,363	16,648	21,165	25,276	31,449	7,953	173,045
Alt 2A	13,534	16,962	21,513	25,679	31,179	7,746	171,074
Change from Alt 1A	171	313	348	403	-270	-207	-1,971
Alt 3A	13,398	16,656	21,090	25,221	31,680	8,095	173,046
Change from Alt 1A	35	8	-75	-55	231	142	0
Alt 4A	13,092	16,406	20,982	25,303	31,978	8,091	167,266
Change from Alt 1A	-272	-242	-183	27	529	137	-5,779
Alt 5A	13,287	16,597	21,024	25,124	31,859	8,095	169,759
Change from Alt 1A	-77	-51	-141	-153	410	141	-3,287
Alt 6A	13,274	16,744	21,329	25,505	31,198	8,436	172,532
Change from Alt 1A	-90	96	164	229	-252	483	-514

Plate 1 cont'd: Alternative Flow Statistics from POR Duration

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Garrison - XS 1388.30							
Alt 1A	1668.1	1669.0	1670.0	1670.8	1671.7	1667.0	1687.4
Alt 2A	1668.2	1669.0	1670.0	1670.9	1671.8	1666.2	1687.3
Change from Alt 1A	0.1	0.1	0.1	0.0	0.1	-0.9	-0.2
Alt 3A	1668.1	1668.9	1670.0	1670.9	1671.8	1667.0	1687.4
Change from Alt 1A	0.0	0.0	0.0	0.0	0.1	0.0	0.0
Alt 4A	1668.0	1668.9	1669.9	1670.9	1671.9	1667.0	1687.4
Change from Alt 1A	0.0	-0.1	-0.1	0.0	0.1	0.0	0.0
Alt 5A	1668.0	1668.9	1669.9	1670.8	1671.9	1667.0	1687.4
Change from Alt 1A	0.0	0.0	0.0	-0.1	0.2	0.0	0.0
Alt 6A	1668.1	1669.0	1670.0	1670.9	1671.6	1667.0	1687.4
Change from Alt 1A	0.0	0.0	0.0	0.0	-0.1	0.0	0.0
Hensler - XS 1362.68							
Alt 1A	1652.3	1653.2	1654.2	1655.1	1656.2	1650.8	1670.4
Alt 2A	1652.3	1653.2	1654.3	1655.2	1656.2	1650.5	1670.2
Change from Alt 1A	0.1	0.1	0.1	0.1	0.0	-0.3	-0.1
Alt 3A	1652.3	1653.2	1654.2	1655.1	1656.3	1650.8	1670.4
Change from Alt 1A	0.0	0.0	0.0	0.0	0.1	0.0	0.0
Alt 4A	1652.2	1653.1	1654.1	1655.1	1656.3	1650.8	1670.2
Change from Alt 1A	0.0	-0.1	-0.1	0.0	0.1	0.0	-0.2
Alt 5A	1652.2	1653.2	1654.2	1655.0	1656.3	1650.8	1670.4
Change from Alt 1A	0.0	0.0	0.0	-0.1	0.2	0.0	0.0
Alt 6A	1652.3	1653.2	1654.2	1655.2	1656.1	1650.8	1670.3
Change from Alt 1A	0.0	0.0	0.0	0.1	-0.1	0.0	0.0
Price - XS 1338.15							
Alt 1A	1638.1	1638.8	1639.7	1640.5	1641.5	1635.8	1654.6
Alt 2A	1638.1	1638.8	1639.8	1640.6	1641.5	1635.8	1654.5
Change from Alt 1A	0.0	0.1	0.1	0.1	0.0	0.0	-0.1
Alt 3A	1638.1	1638.8	1639.7	1640.5	1641.6	1635.8	1654.6
Change from Alt 1A	0.0	0.0	0.0	0.0	0.1	0.0	0.0
Alt 4A	1638.0	1638.7	1639.7	1640.5	1641.6	1635.8	1654.5
Change from Alt 1A	0.0	-0.1	0.0	0.0	0.1	0.0	-0.1
Alt 5A	1638.0	1638.8	1639.7	1640.5	1641.6	1635.8	1654.6
Change from Alt 1A	0.0	0.0	0.0	0.0	0.1	0.0	0.0
Alt 6A	1638.0	1638.8	1639.7	1640.5	1641.5	1635.8	1654.6
Change from Alt 1A	0.0	0.0	0.0	0.0	-0.1	0.0	0.0

Plate 2: Alternative Water Surface Elevation Statistics from POR Duration

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Bismarck - XS 1314.80							
Alt 1A	1622.8	1623.6	1624.6	1625.6	1626.9	1621.0	1639.2
Alt 2A	1622.8	1623.7	1624.7	1625.6	1626.9	1620.9	1639.1
Change from Alt 1A	0.0	0.1	0.1	0.1	-0.1	-0.1	-0.1
Alt 3A	1622.8	1623.6	1624.6	1625.5	1627.0	1621.0	1639.2
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Alt 4A	1622.7	1623.5	1624.5	1625.6	1627.1	1621.0	1639.0
Change from Alt 1A	-0.1	-0.1	0.0	0.0	0.1	0.0	-0.2
Alt 5A	1622.8	1623.6	1624.6	1625.5	1627.0	1621.0	1639.1
Change from Alt 1A	0.0	0.0	0.0	0.0	0.1	0.0	-0.1
Alt 6A	1622.8	1623.6	1624.6	1625.6	1626.9	1621.1	1639.2
Change from Alt 1A	0.0	0.0	0.0	0.0	-0.1	0.1	0.0
Schmidt - XS 1297.51							
Alt 1A	1613.6	1614.6	1615.7	1616.9	1618.6	1611.6	1628.2
Alt 2A	1613.6	1614.6	1615.8	1617.0	1618.4	1611.5	1628.1
Change from Alt 1A	0.0	0.0	0.0	0.1	-0.2	-0.1	-0.1
Alt 3A	1613.6	1614.6	1615.7	1616.9	1618.6	1611.8	1628.2
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.1	0.0
Alt 4A	1613.5	1614.5	1615.7	1616.9	1618.7	1611.8	1628.1
Change from Alt 1A	-0.1	-0.1	-0.1	0.0	0.2	0.2	-0.1
Alt 5A	1613.6	1614.6	1615.7	1616.9	1618.7	1611.8	1628.1
Change from Alt 1A	0.0	0.0	0.0	0.0	0.2	0.1	-0.1
Alt 6A	1613.5	1614.6	1615.8	1616.9	1618.5	1611.8	1628.2
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.2	0.0
Lake Oahe - XS 1073.04							
Alt 1A	1,570.3	1,582.9	1,599.1	1,607.7	1,611.8	1,530.7	1,621.0
Alt 2A	1,568.5	1,582.7	1,597.6	1,606.5	1,610.8	1,528.2	1,621.0
Change from Alt 1A	-1.8	-0.3	-1.5	-1.2	-1.0	-2.5	-0.1
Alt 3A	1,570.4	1,583.3	1,599.2	1,607.7	1,611.8	1,531.1	1,621.1
Change from Alt 1A	0.1	0.4	0.1	0.0	0.0	0.4	0.1
Alt 4A	1,567.8	1,580.5	1,597.7	1,607.3	1,611.4	1,525.9	1,621.0
Change from Alt 1A	-2.5	-2.4	-1.4	-0.4	-0.4	-4.9	-0.1
Alt 5A	1,570.4	1,582.9	1,598.7	1,607.5	1,611.7	1,531.1	1,621.1
Change from Alt 1A	0.1	0.0	-0.4	-0.2	-0.1	0.4	0.1
Alt 6A	1,567.2	1,579.8	1,597.3	1,607.1	1,611.2	1,524.9	1,620.9
Change from Alt 1A	-3.1	-3.1	-1.8	-0.6	-0.6	-5.8	-0.2

Plate 2 cont'd: Alternative Water Surface Elevation Statistics from POR Duration

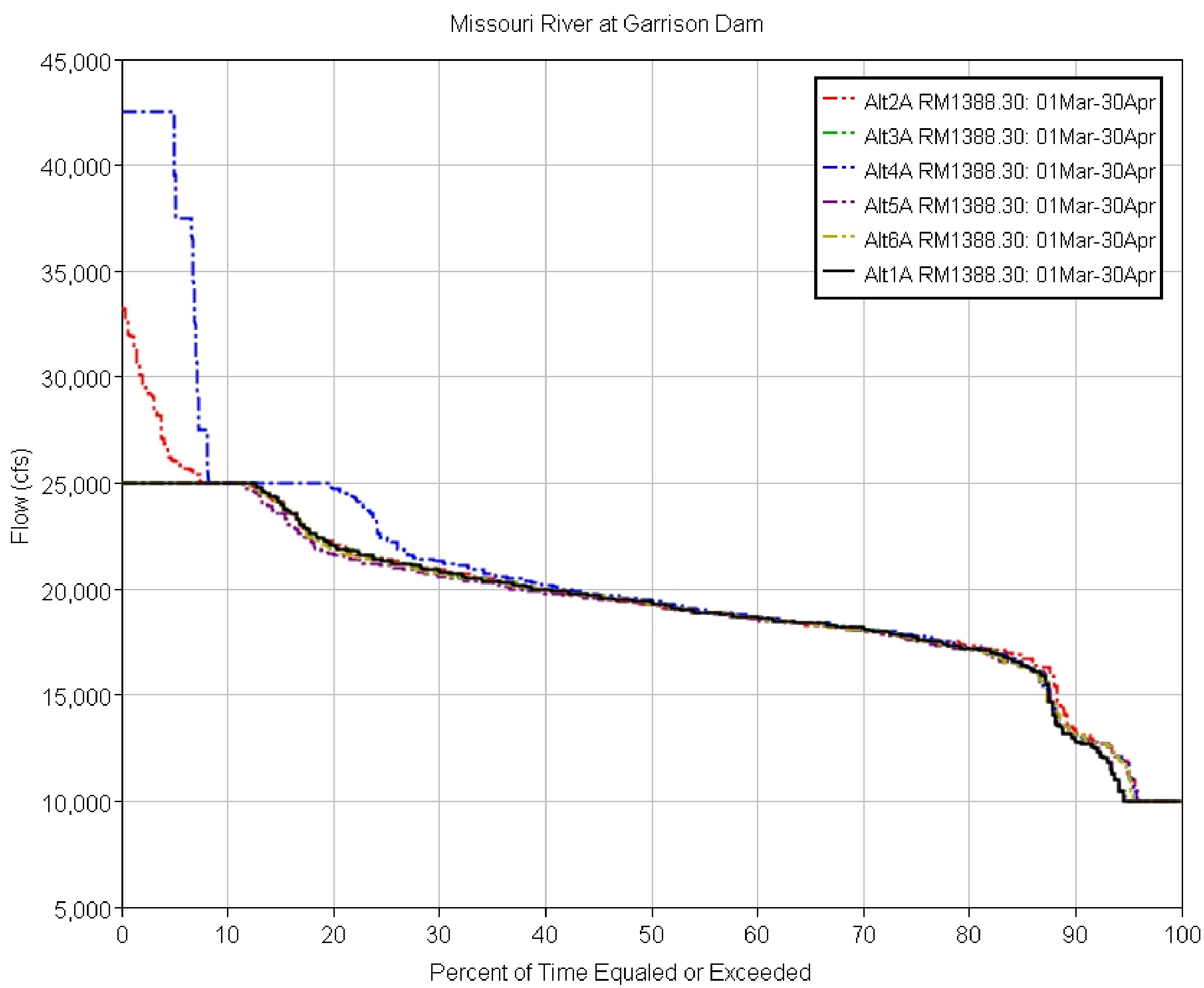


Plate 3: Garrison Spring Duration

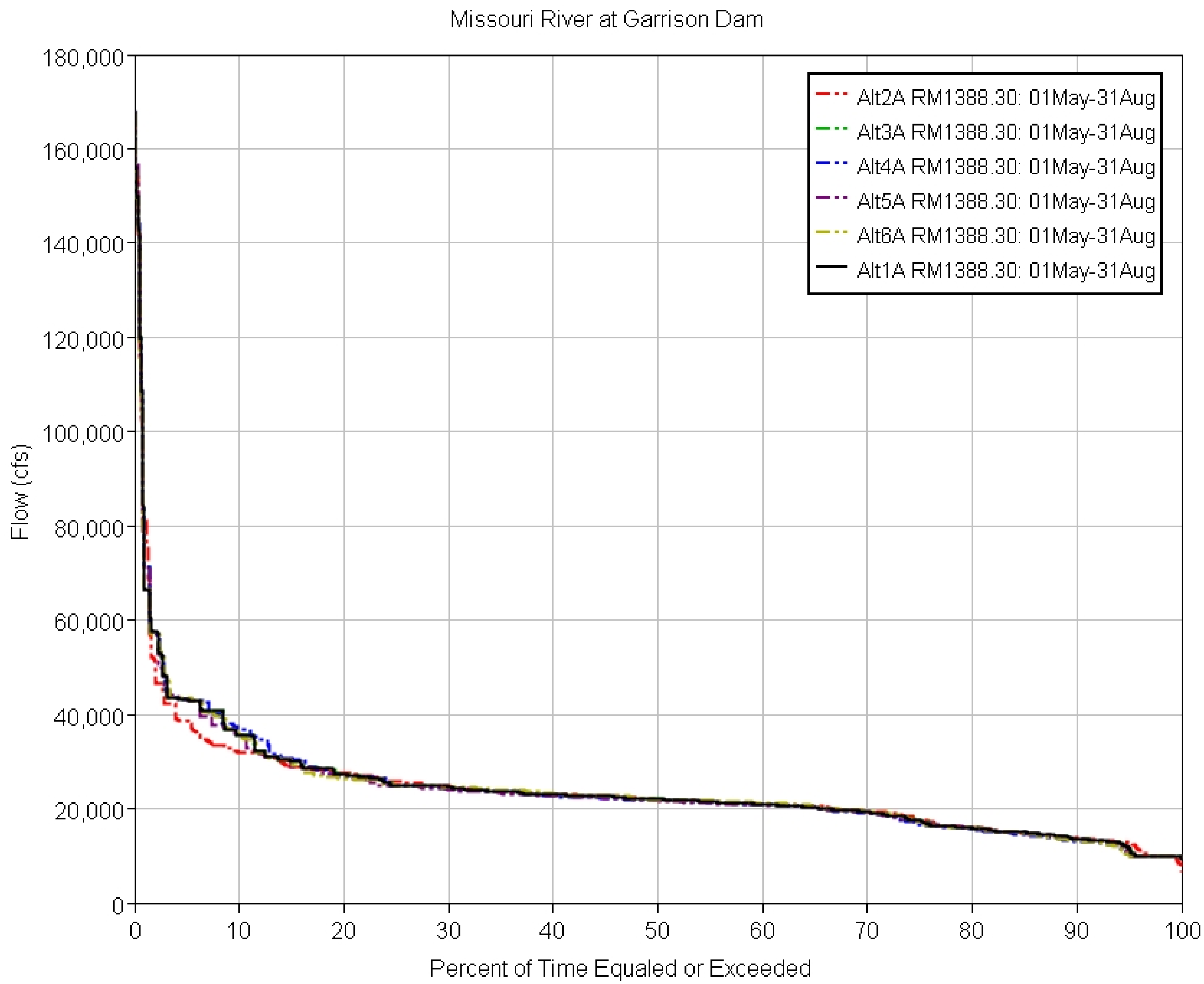


Plate 4: Garrison Summer Duration

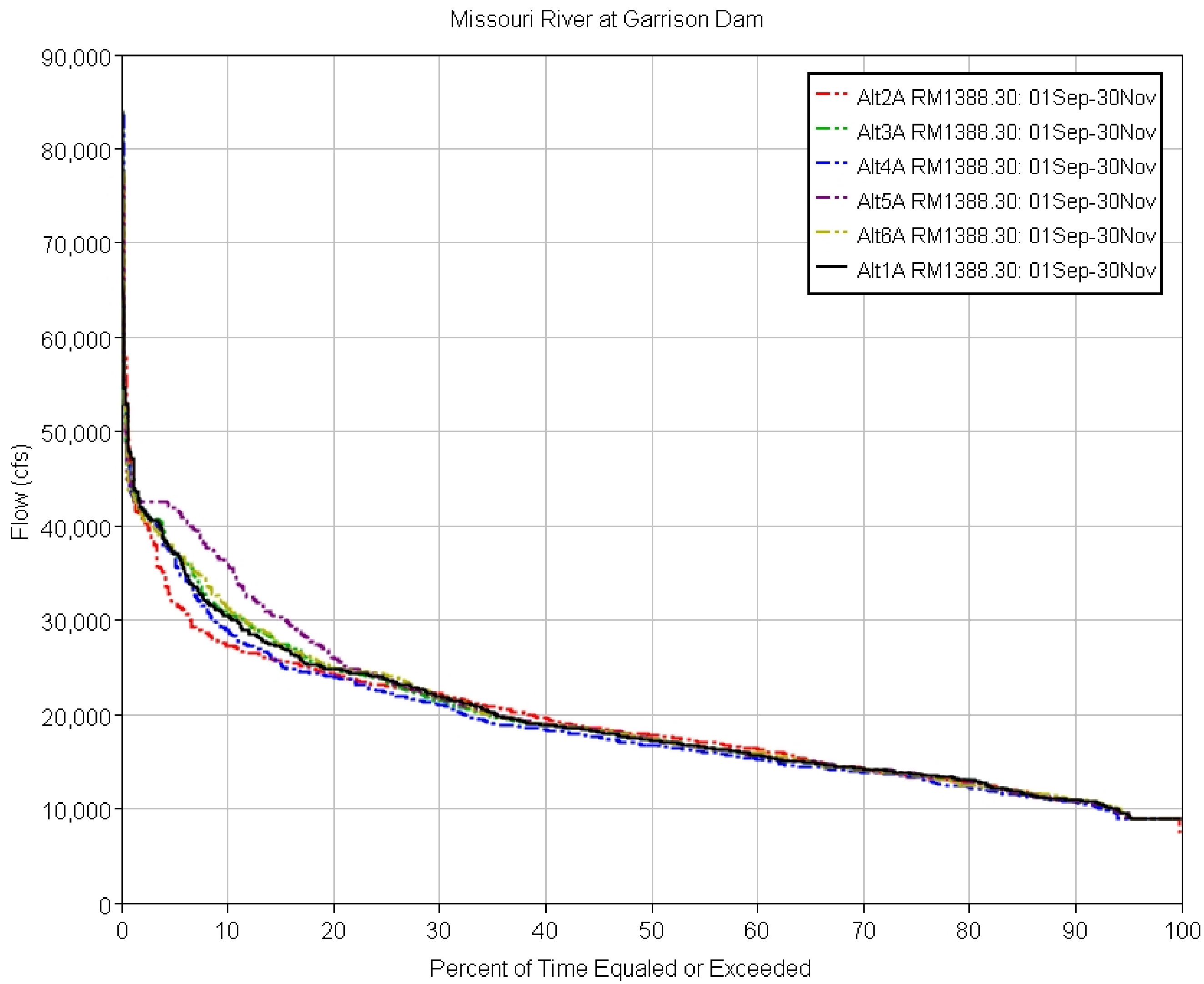


Plate 5: Garrison Fall Duration

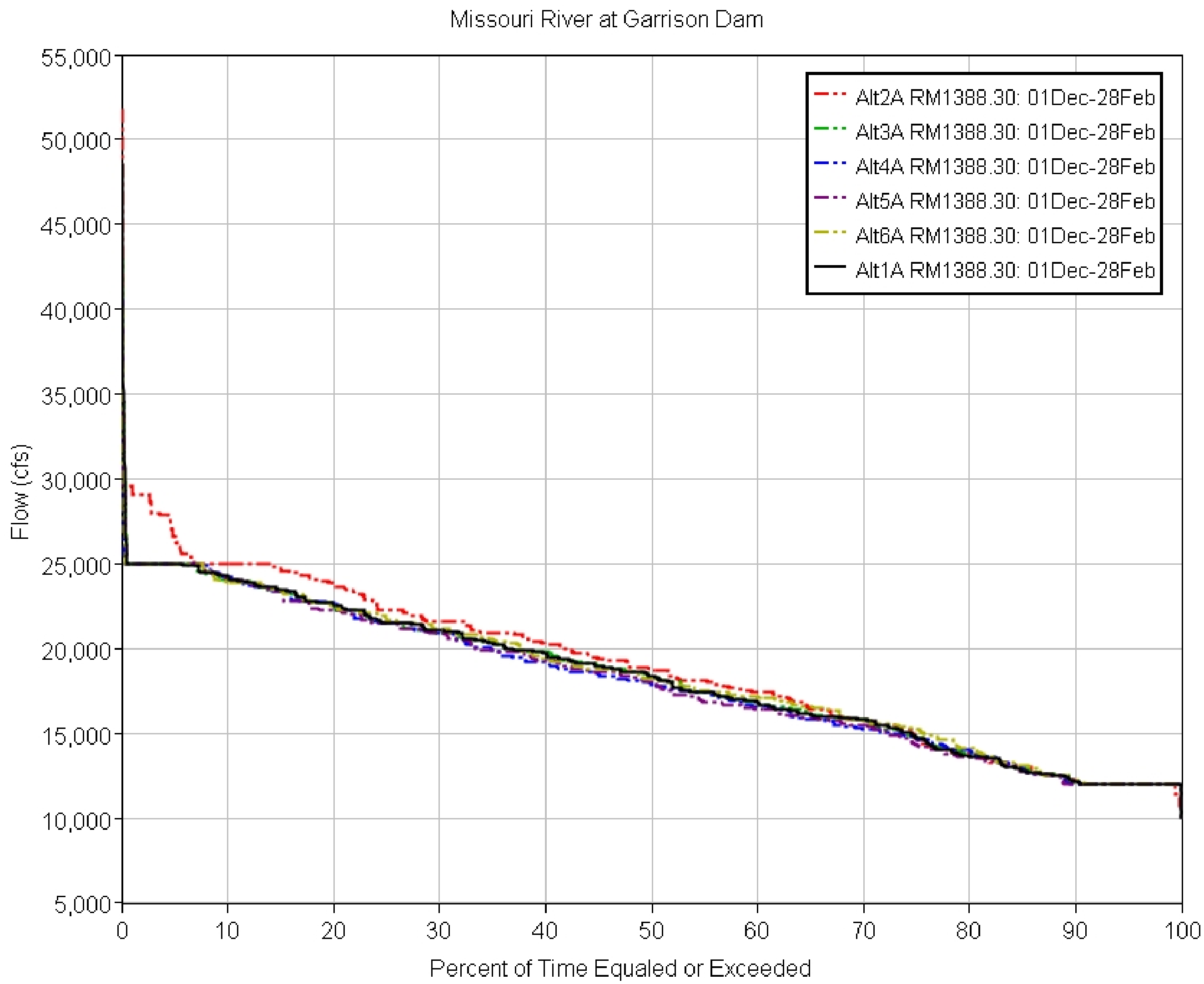


Plate 6: Garrison Winter Duration

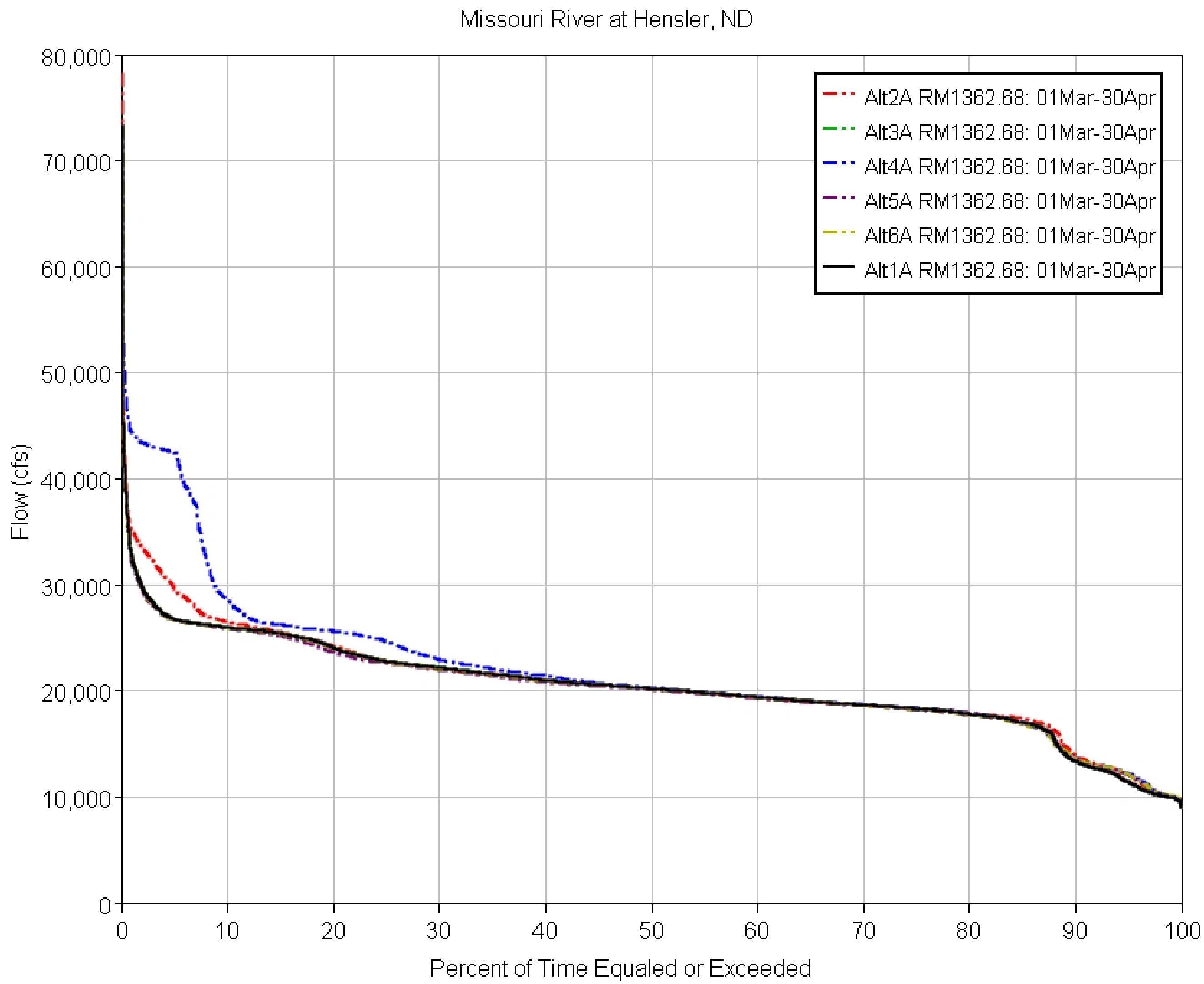


Plate 7: Hensler Spring Duration

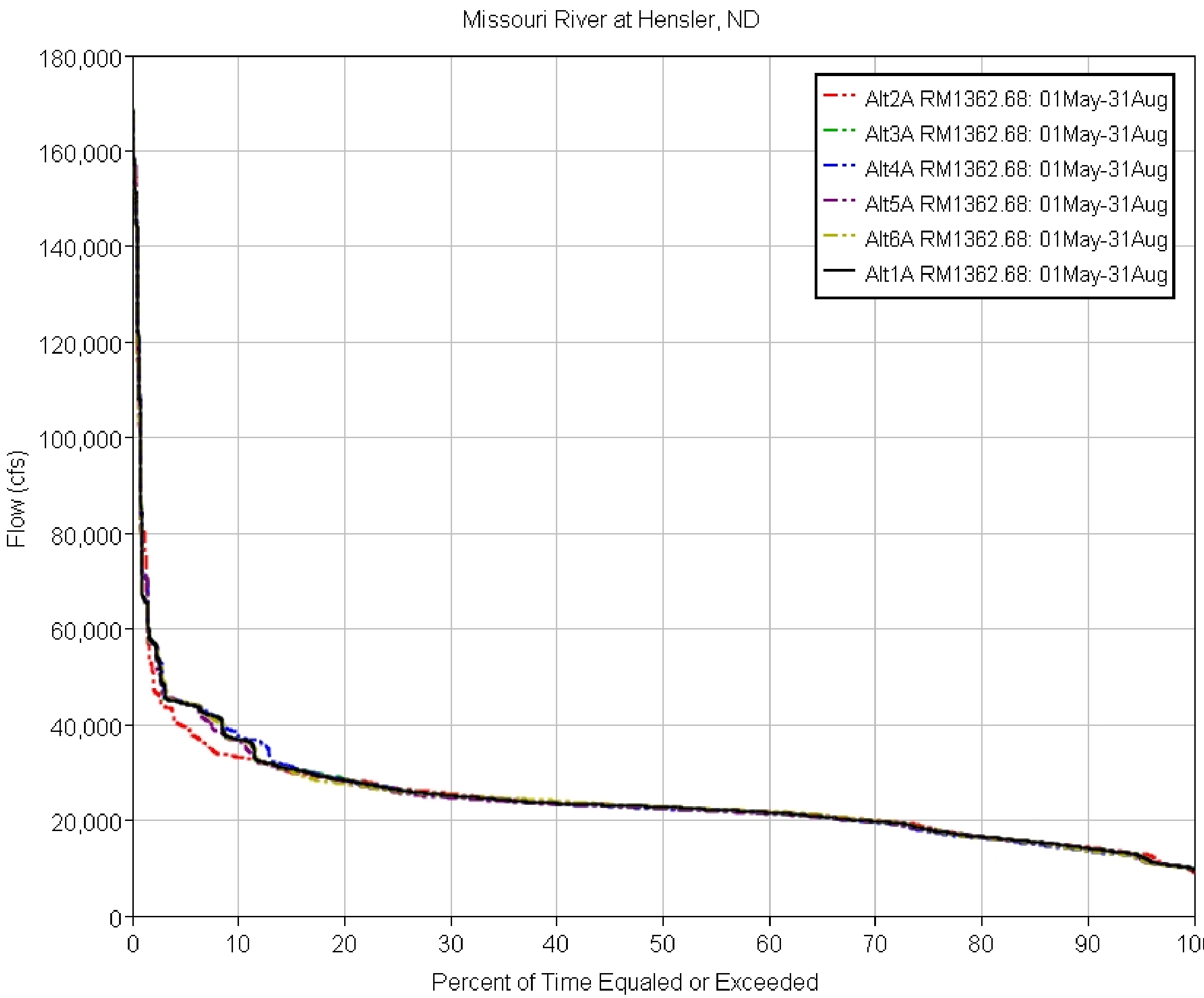


Plate 8: Hensler Summer Duration

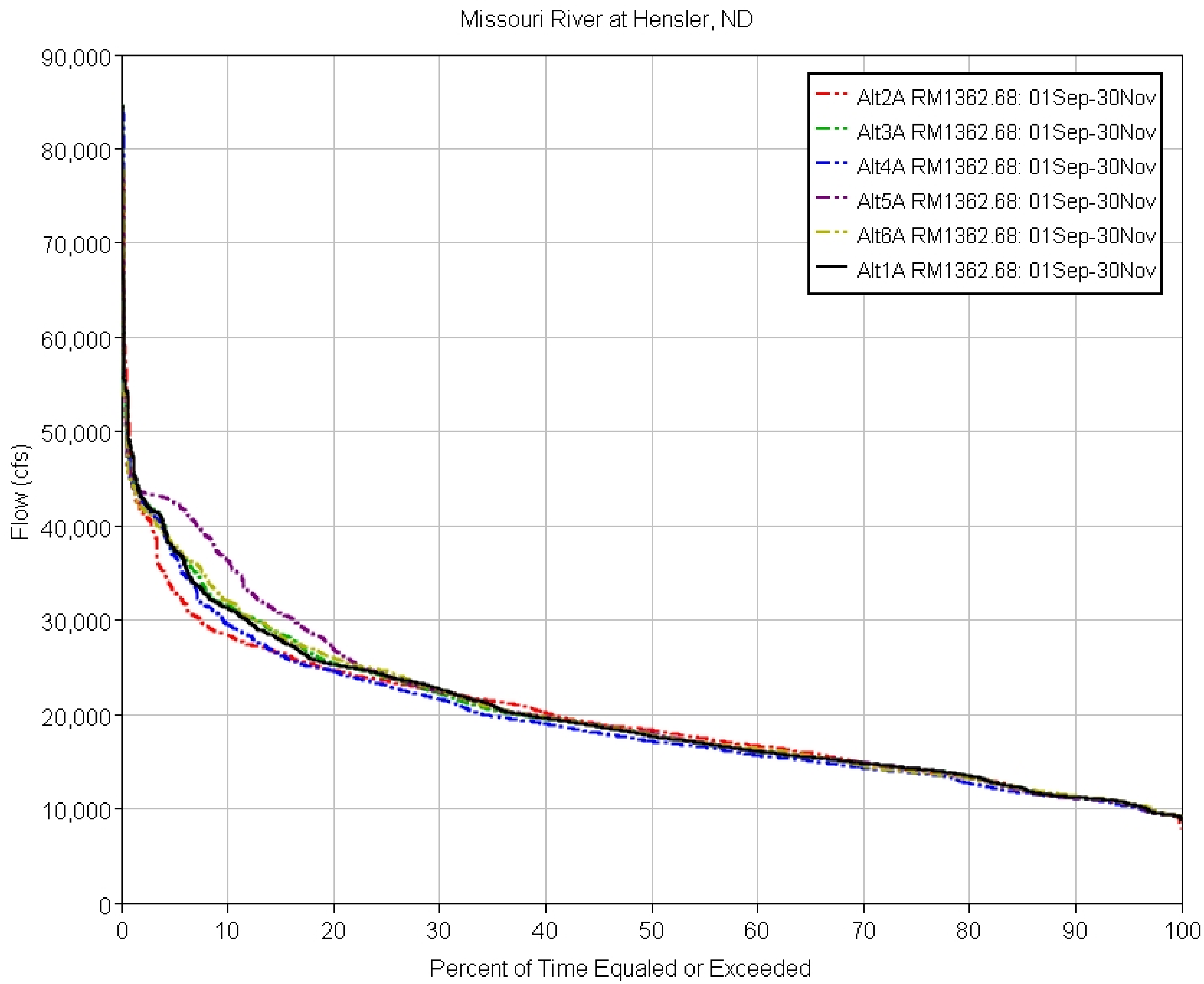


Plate 9: Hensler Fall Duration

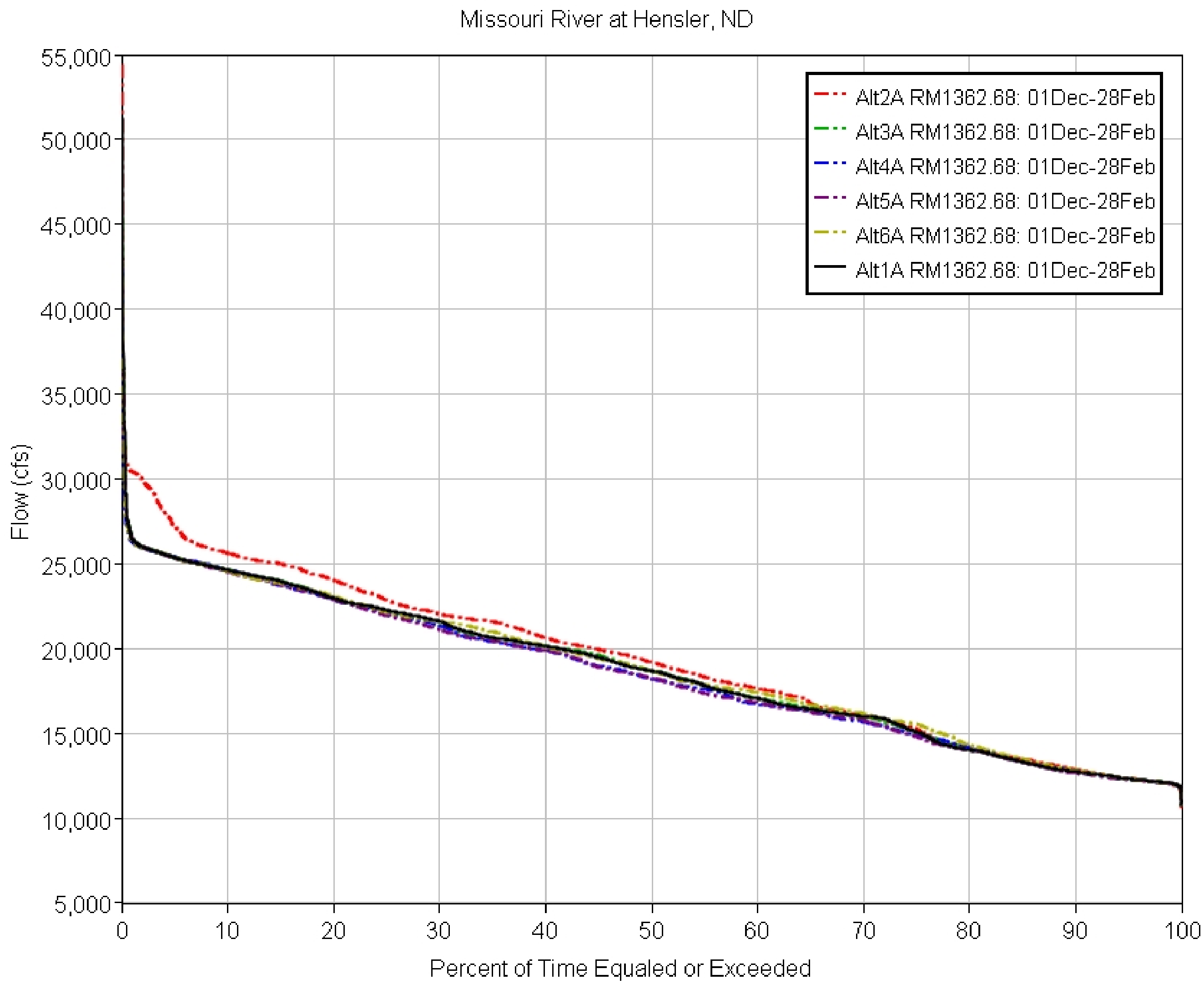


Plate 10: Hensler Winter Duration

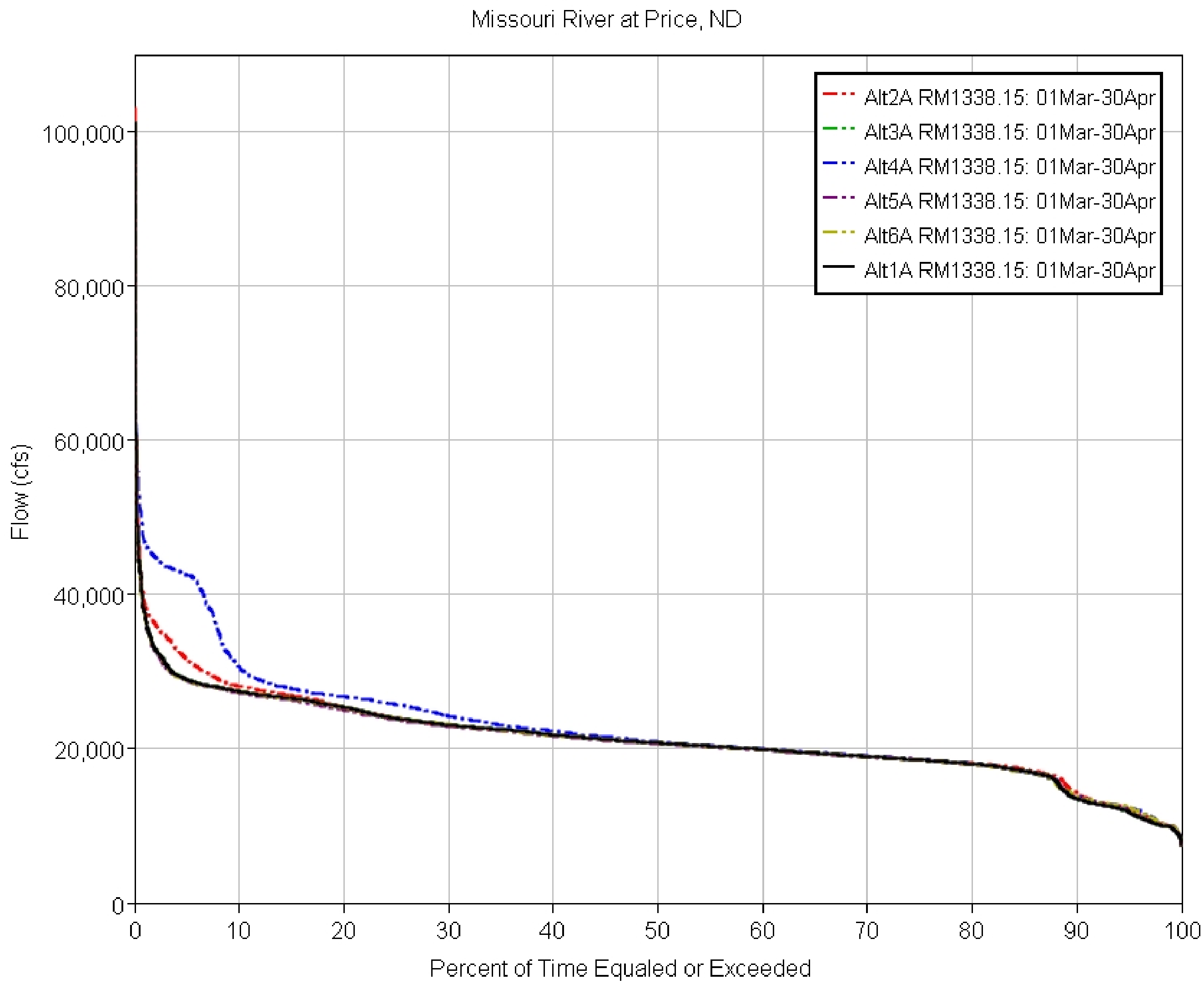


Plate 11: Price Spring Duration

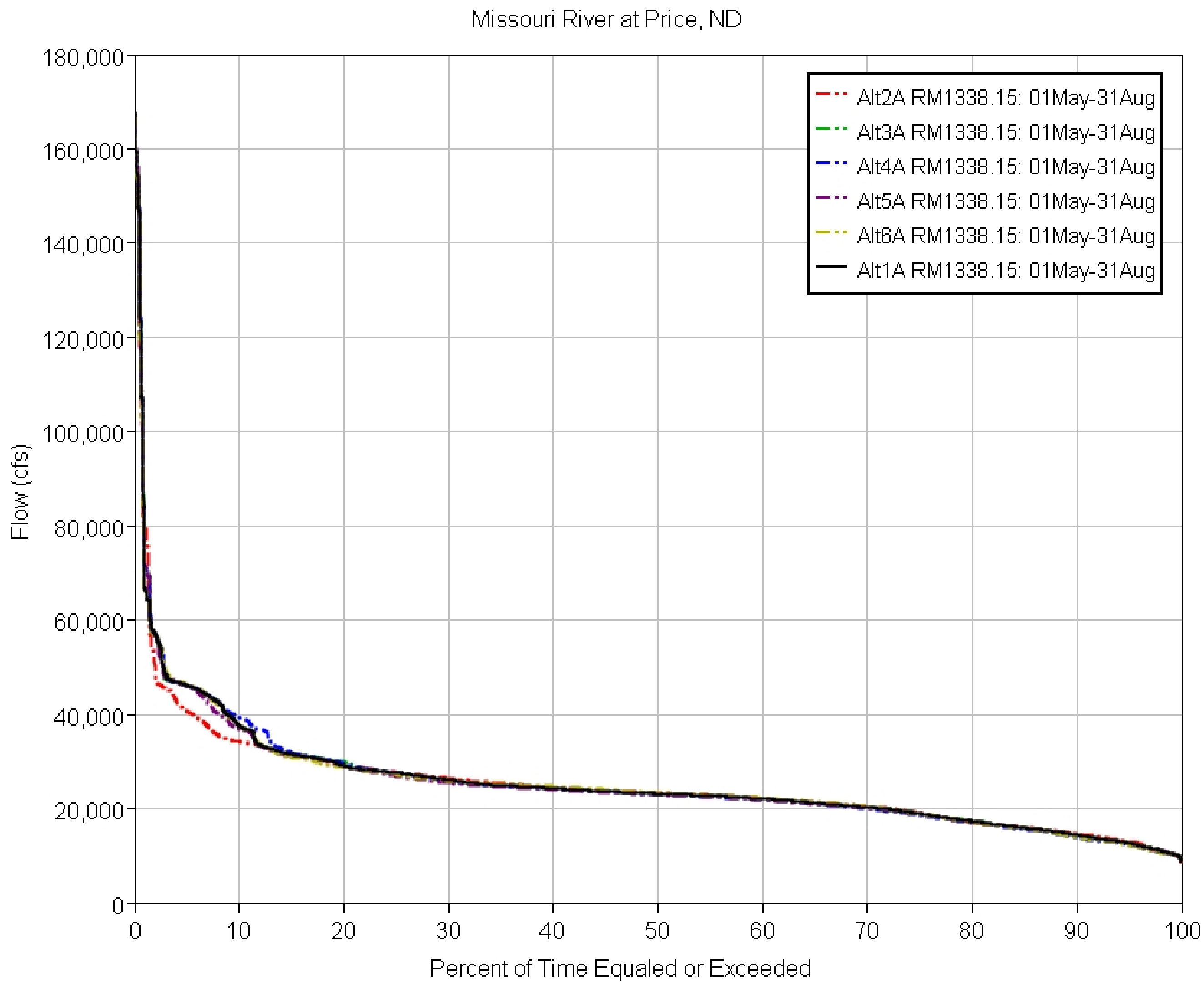


Plate 12: Price Summer Duration

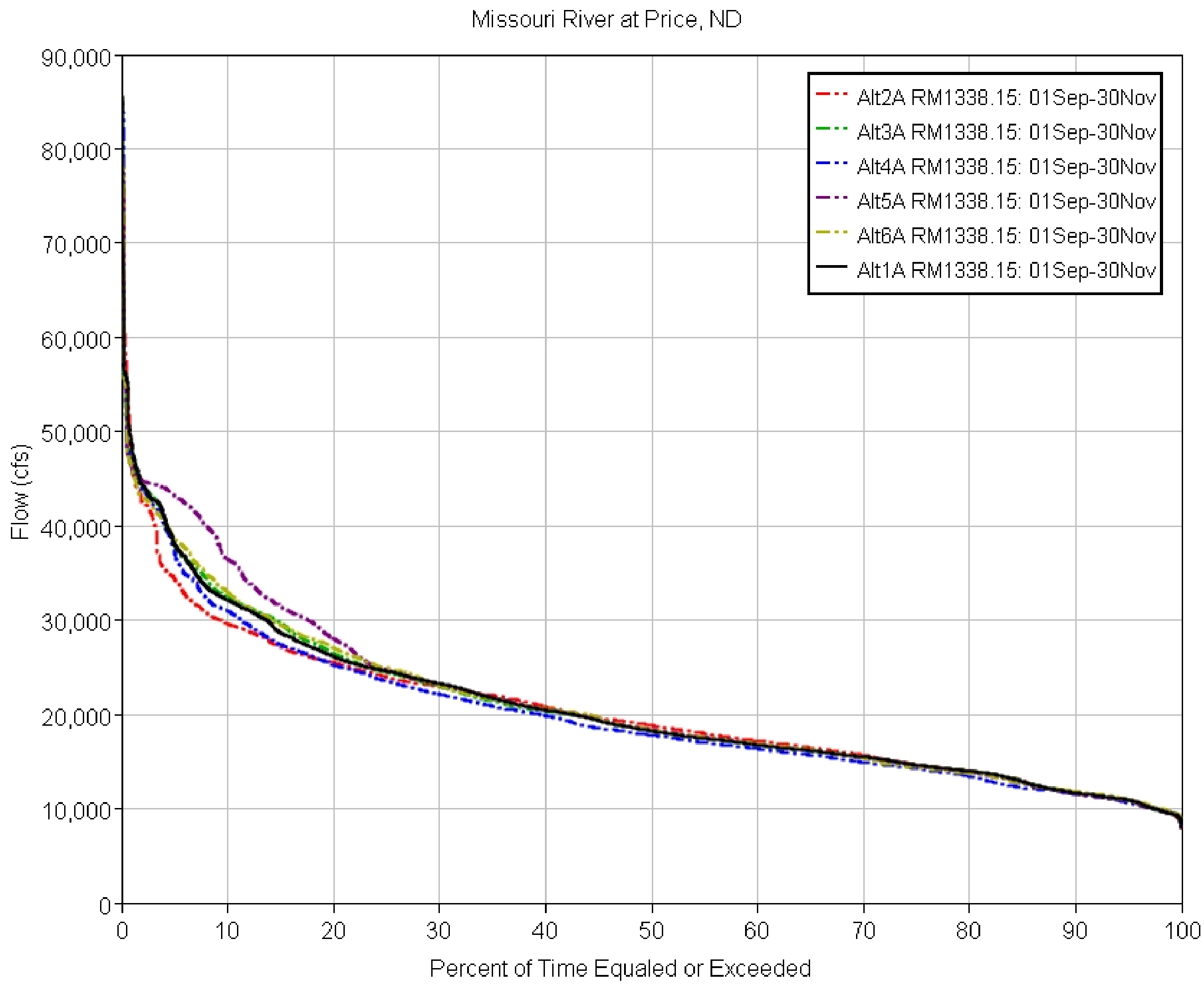


Plate 13: Price Fall Duration

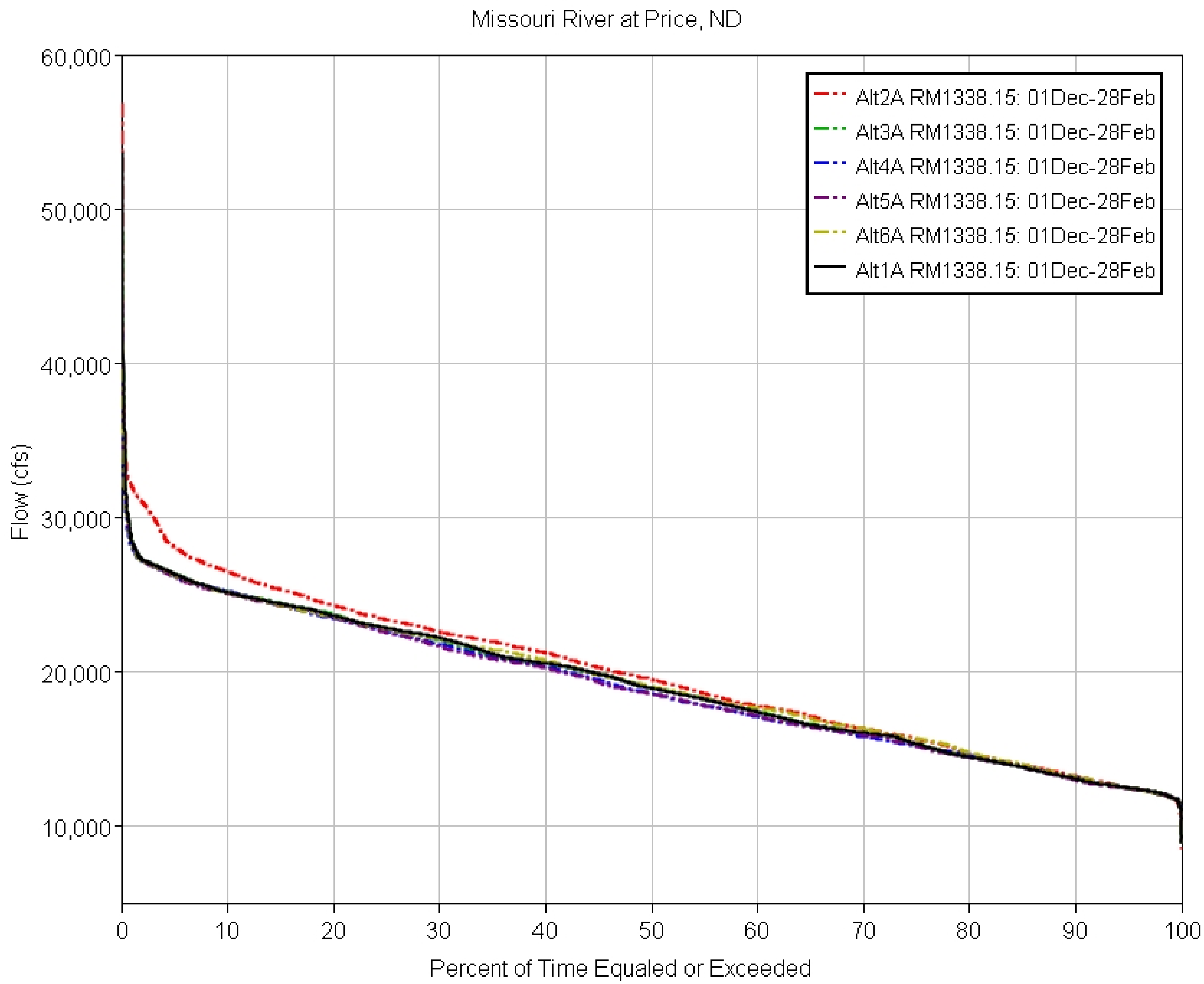


Plate 14: Price Winter Duration

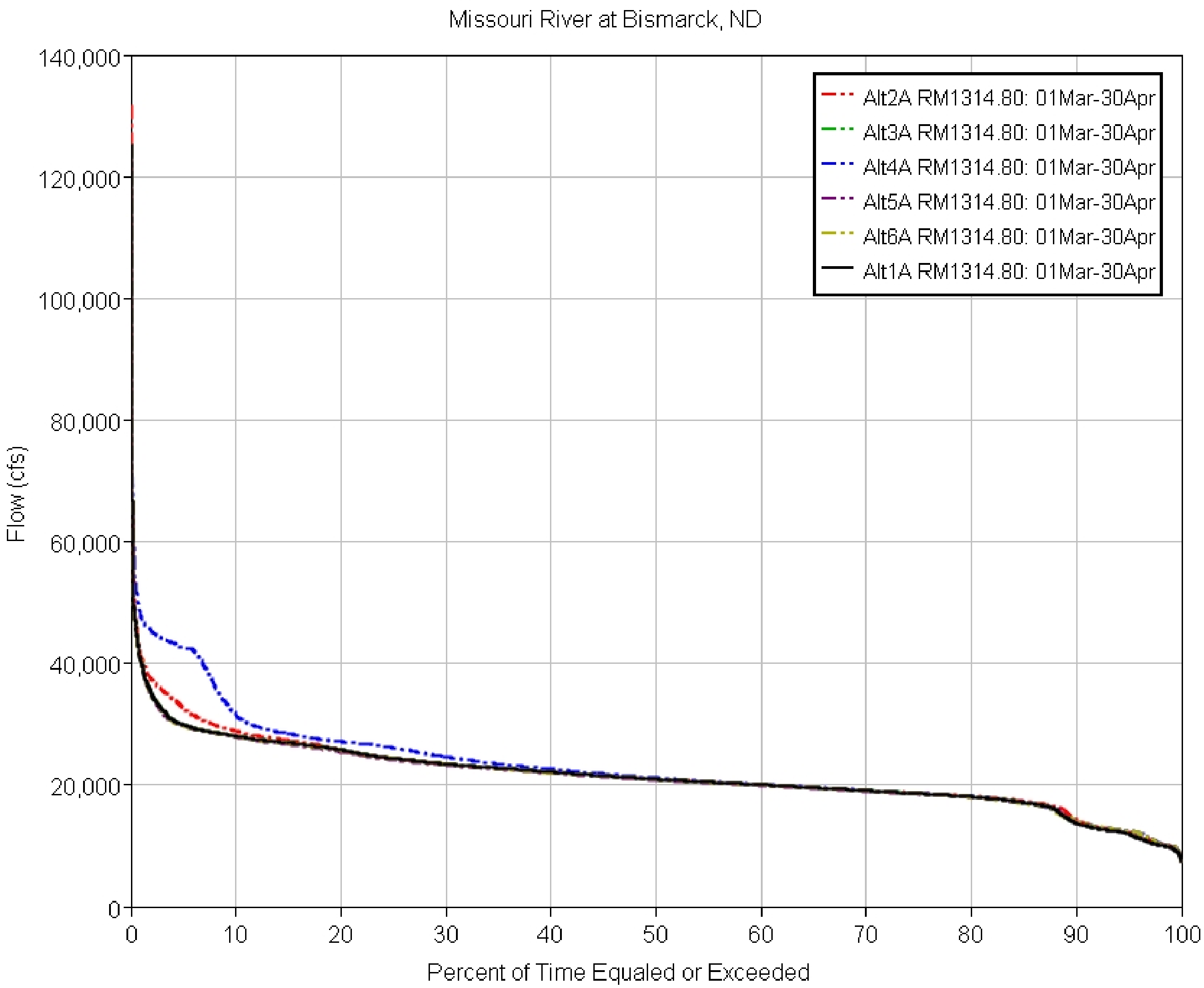


Plate 15: Bismarck Spring Duration

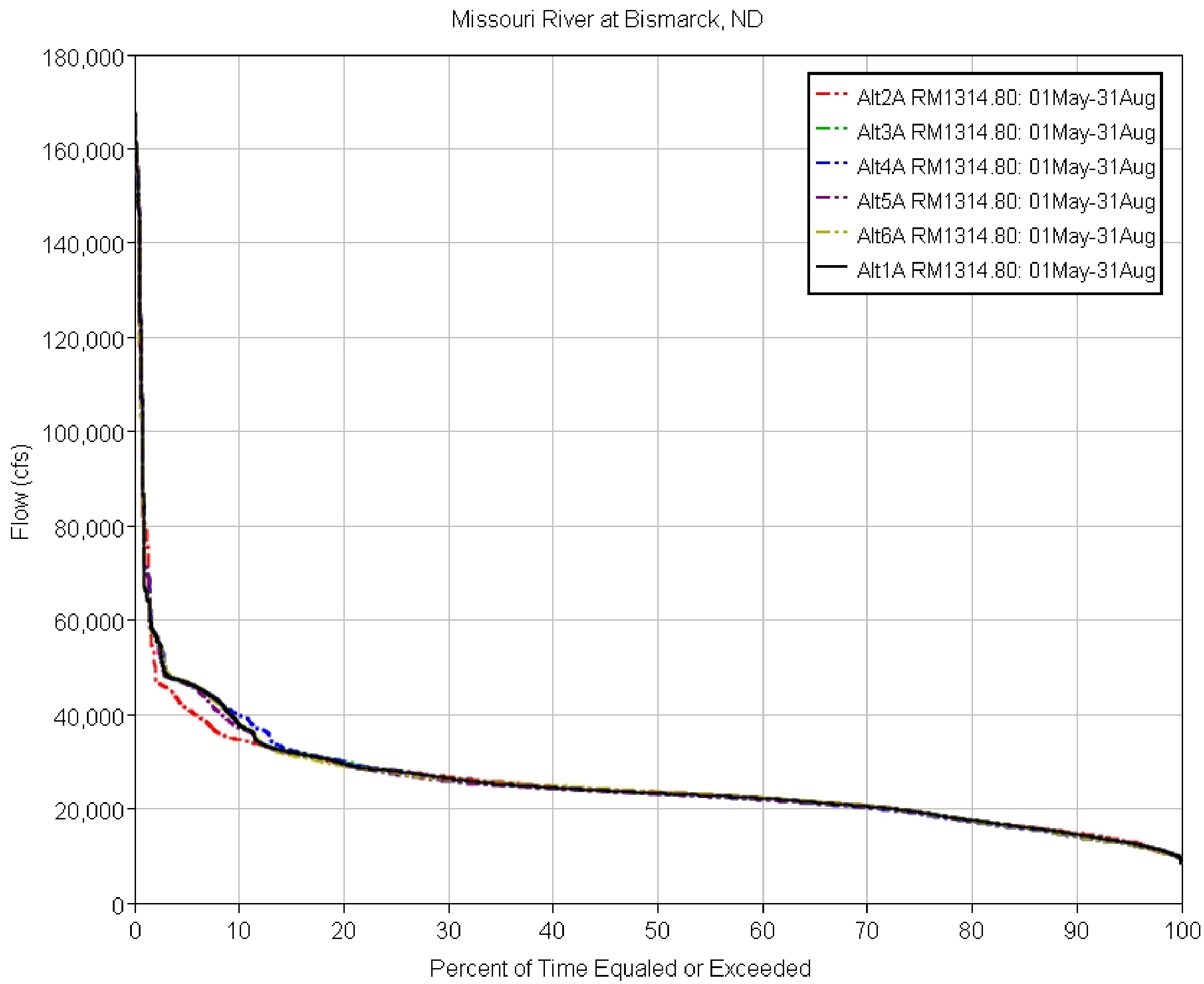


Plate 16: Bismarck Summer Duration

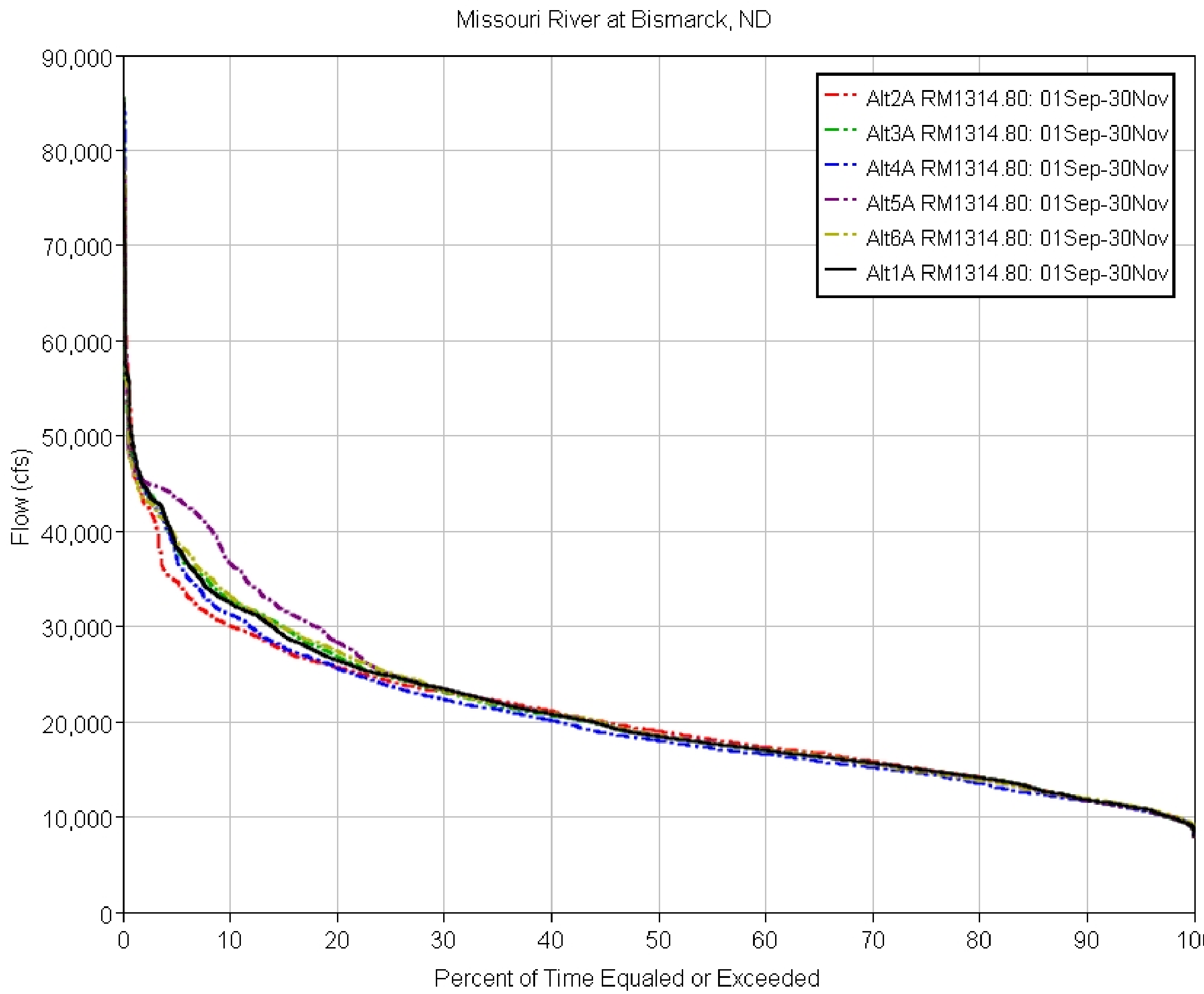


Plate 17: Bismarck Fall Duration

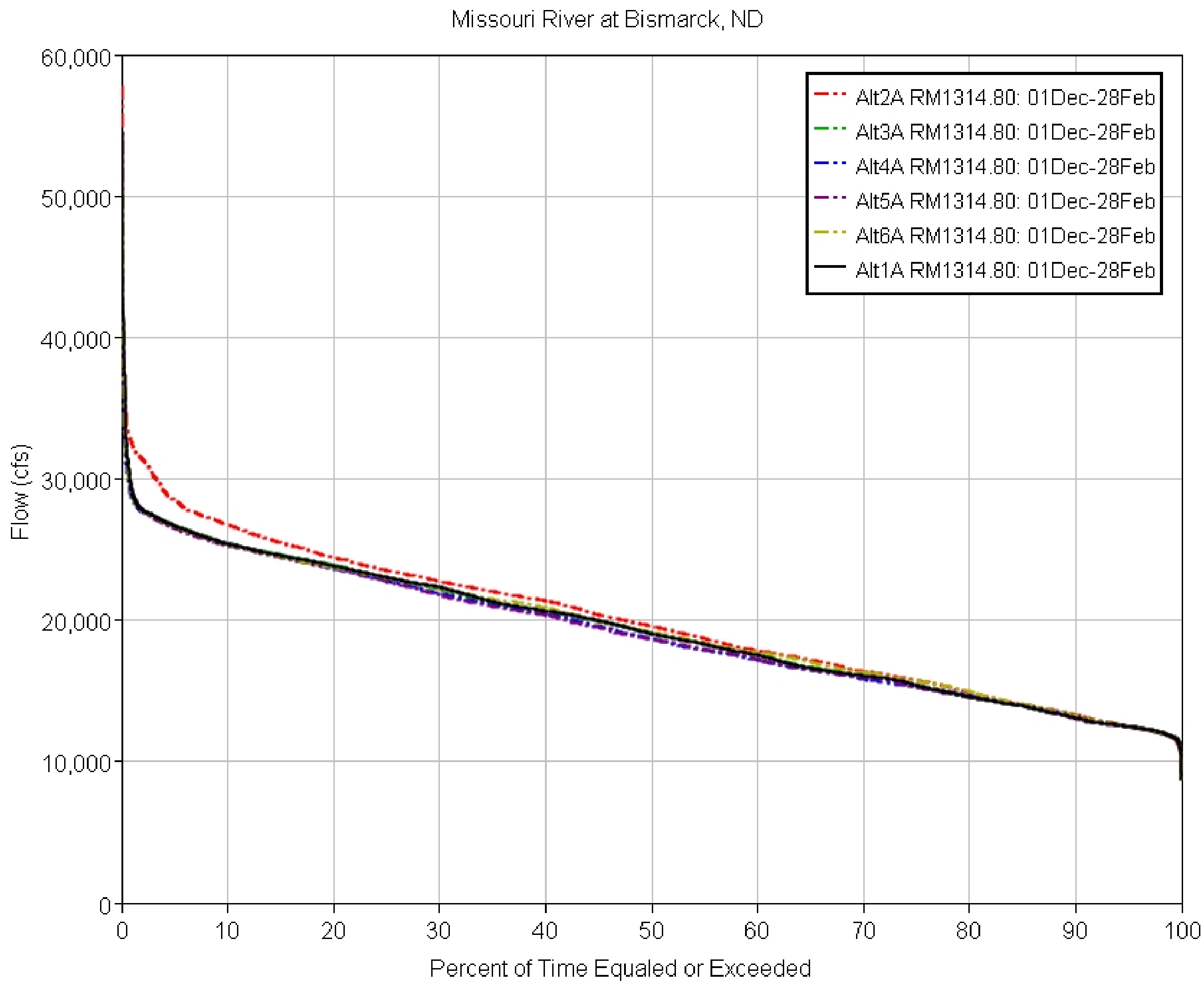


Plate 18: Bismarck Winter Duration

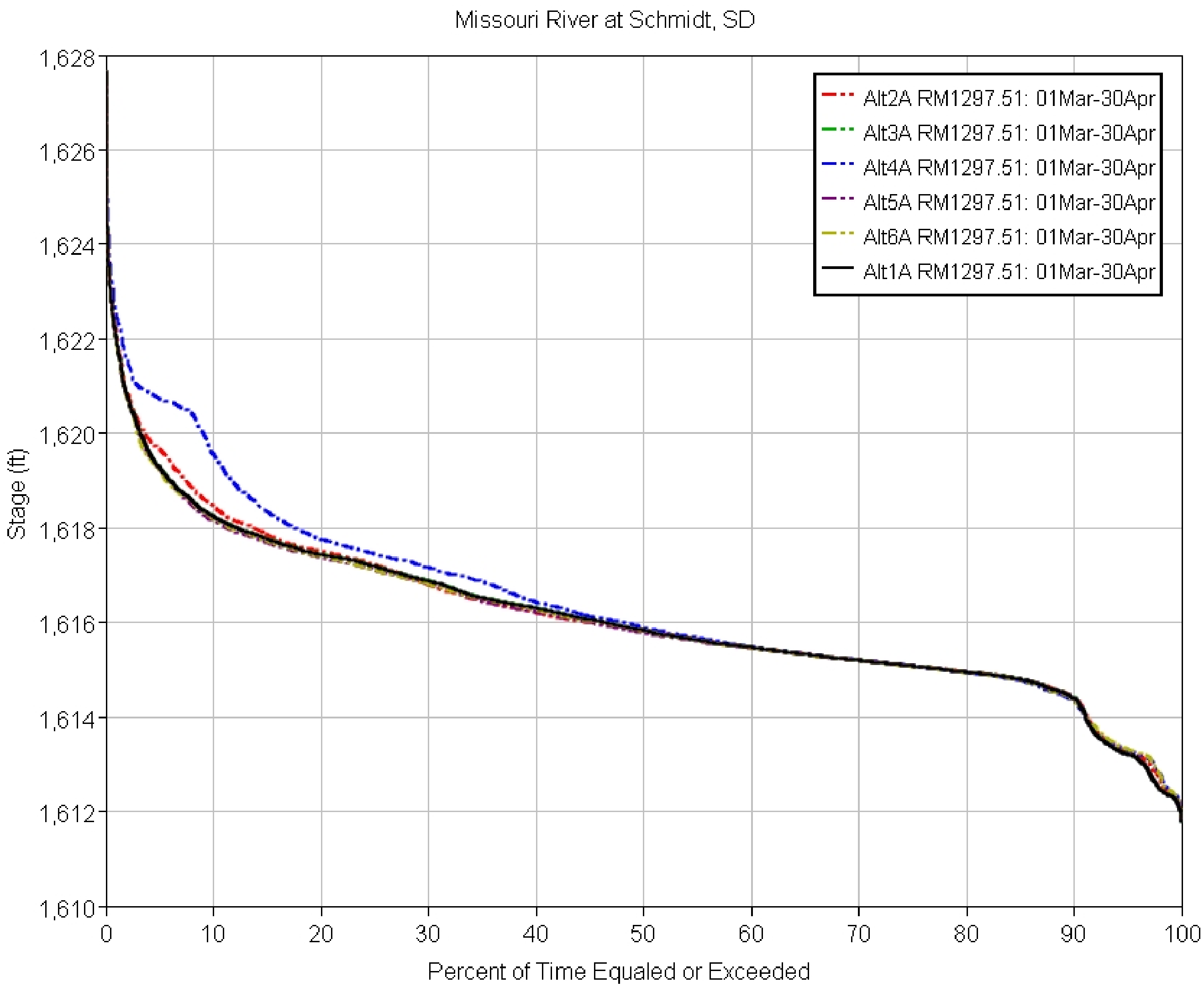


Plate 19: Schmidt Spring Duration

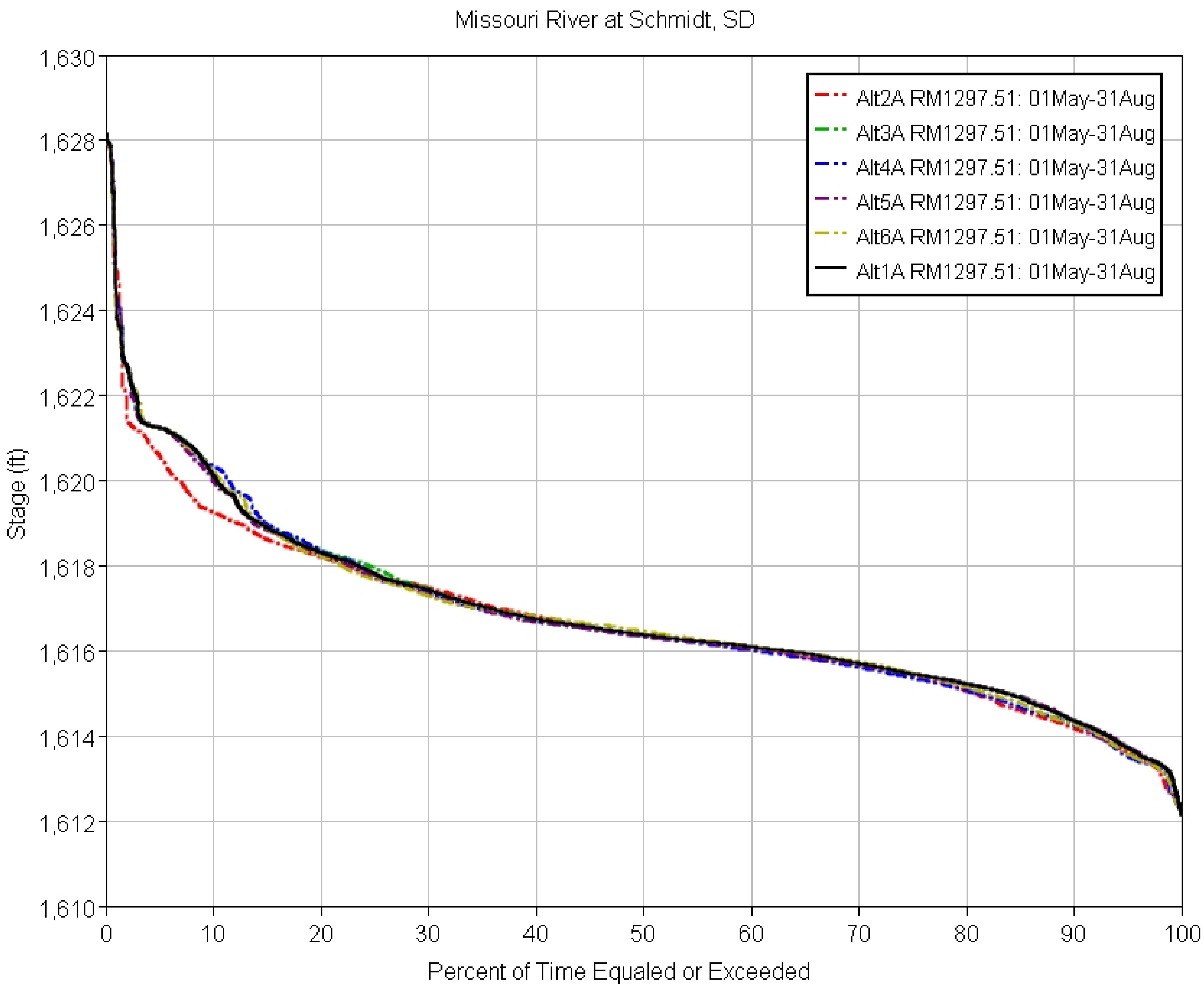


Plate 20: Schmidt Summer Duration

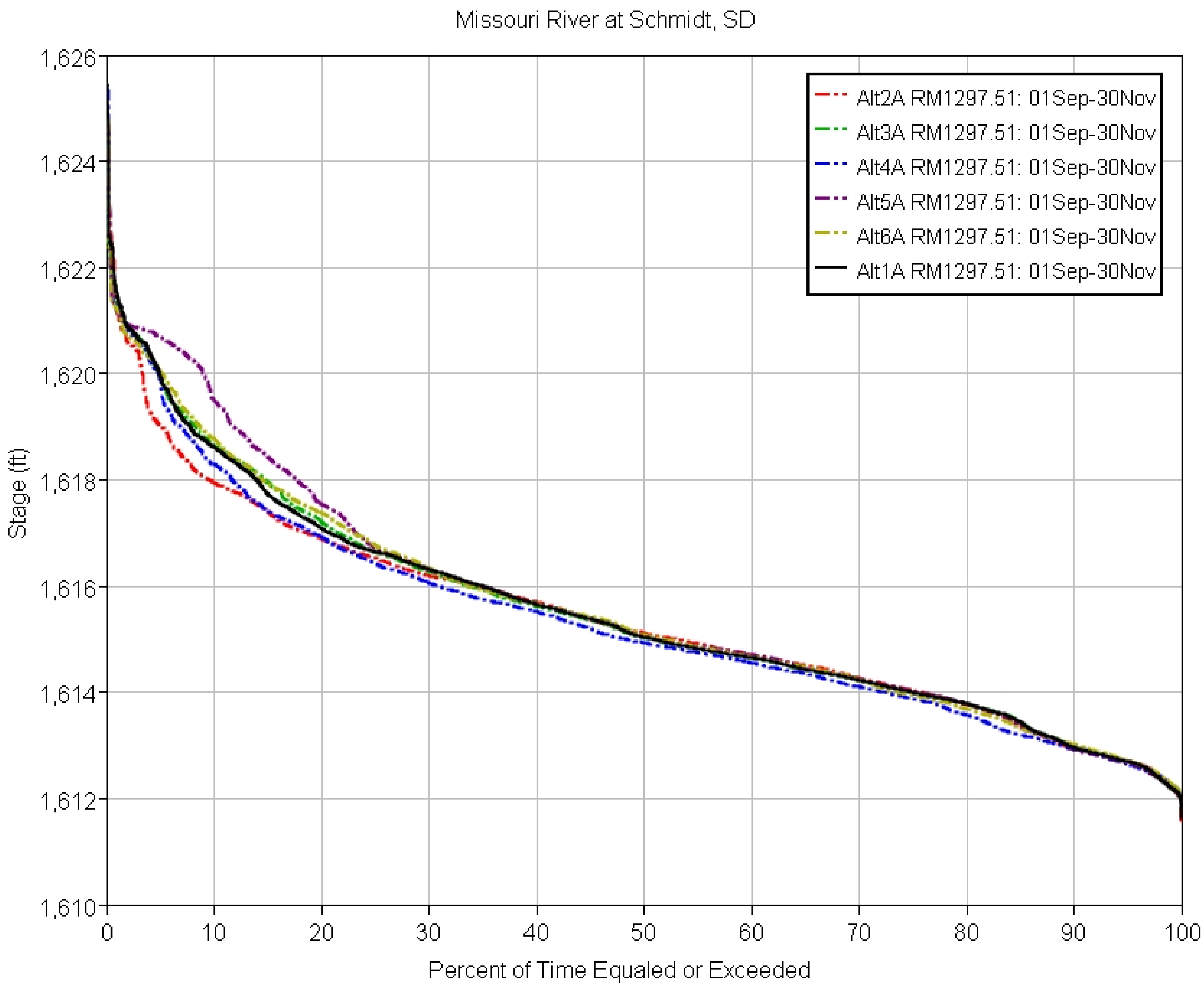


Plate 21: Schmidt Fall Duration

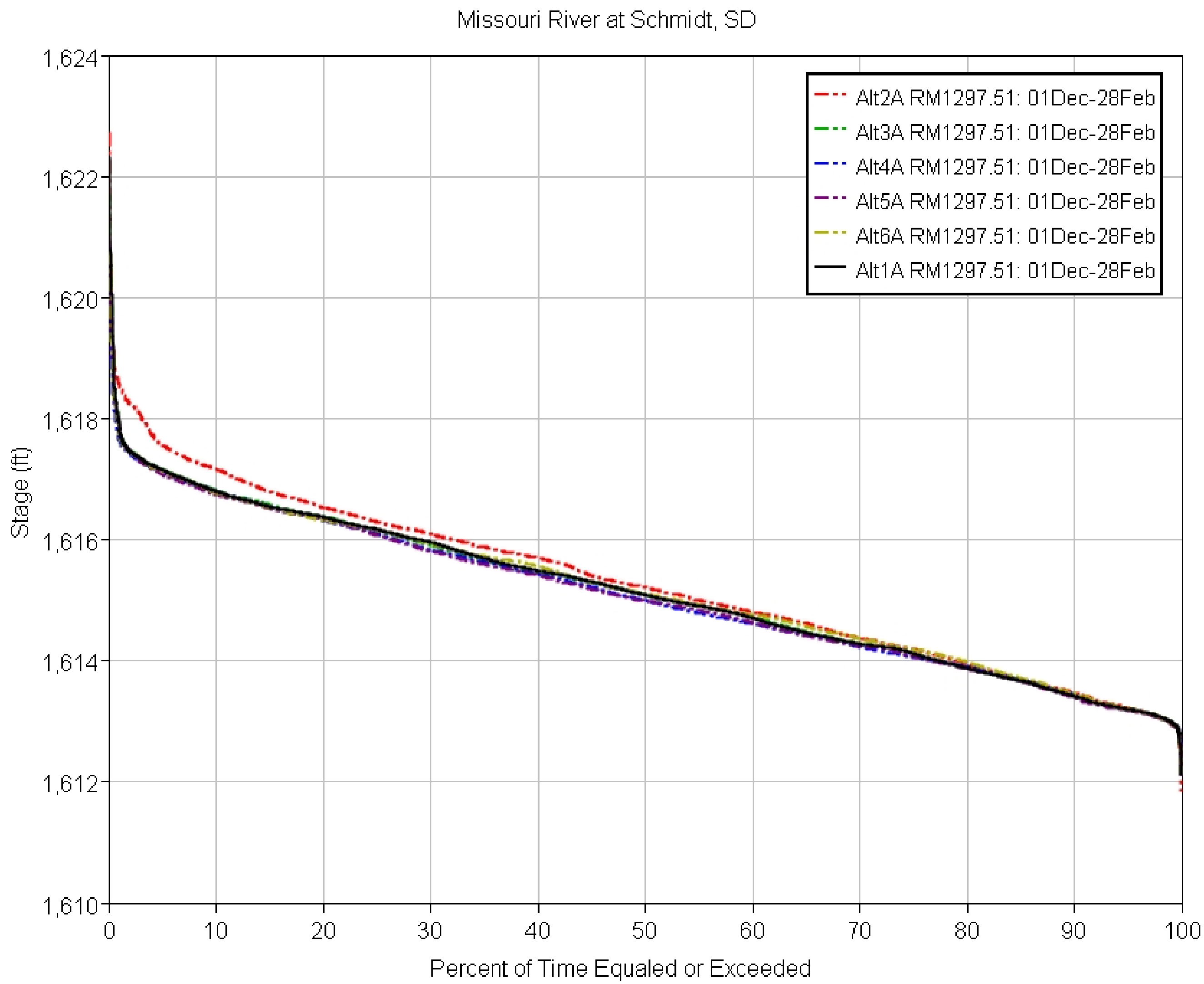


Plate 22: Schmidt Winter Duration

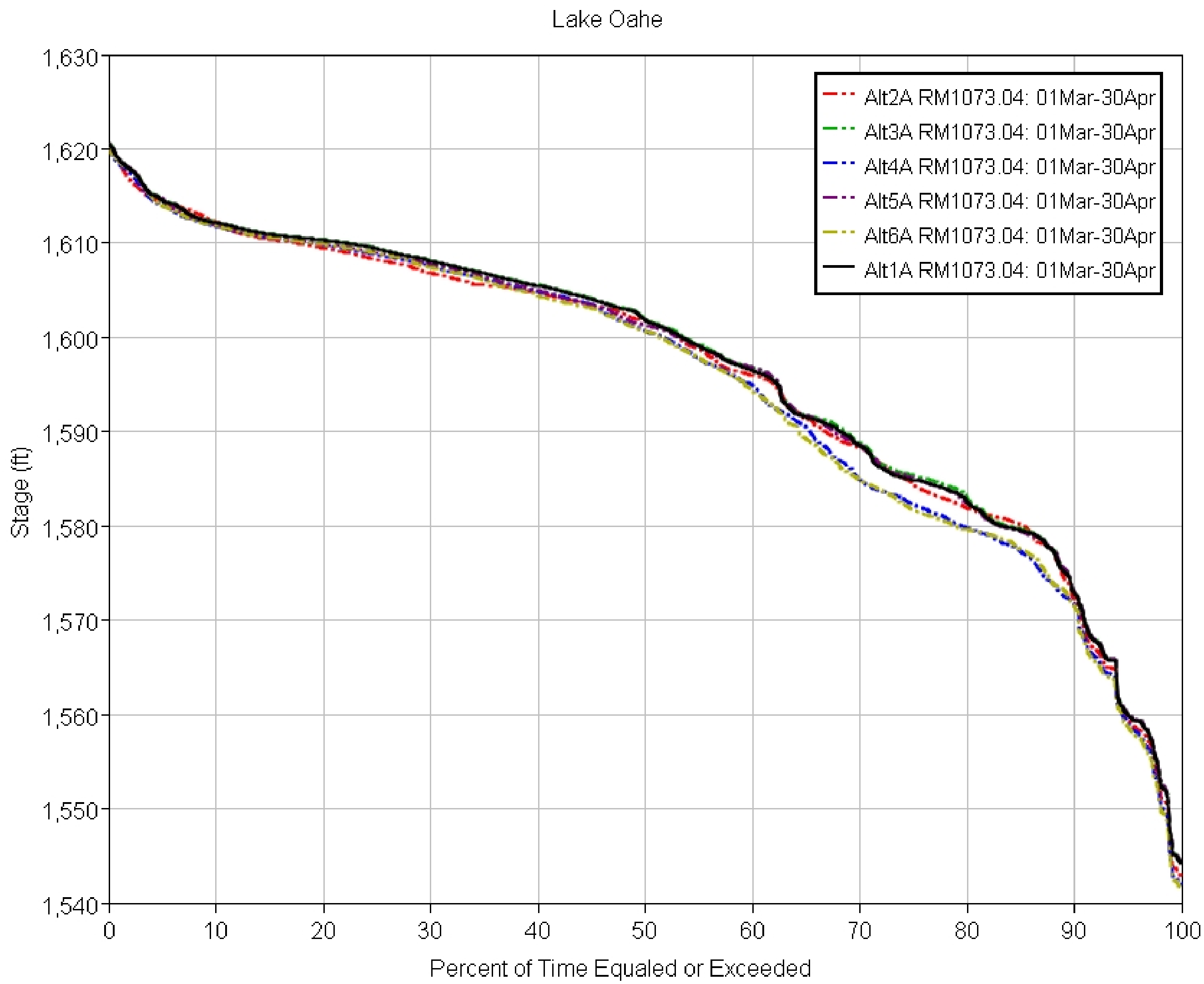


Plate 23: Lake Oahe Spring Duration

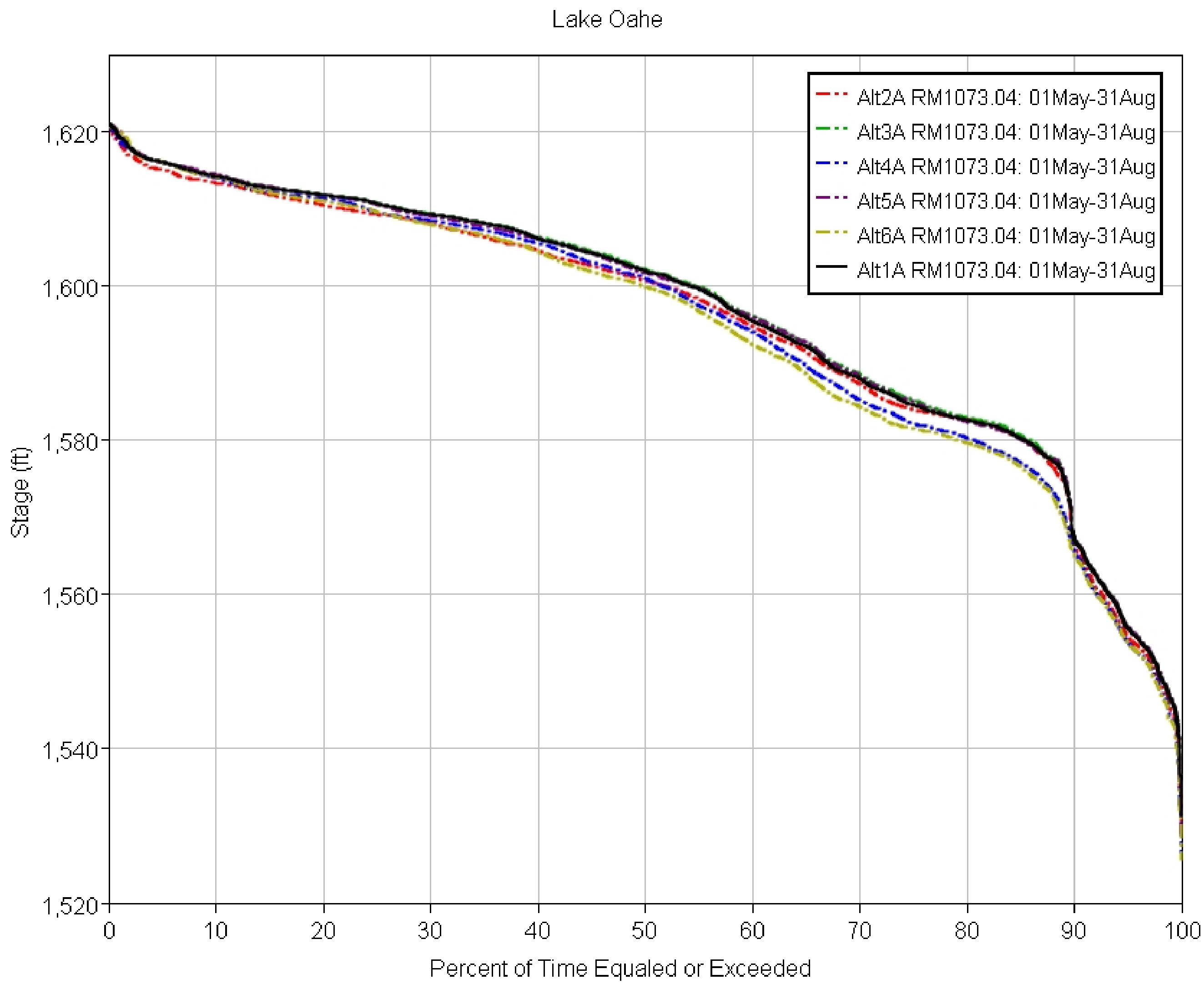


Plate 24: Lake Oahe Summer Duration

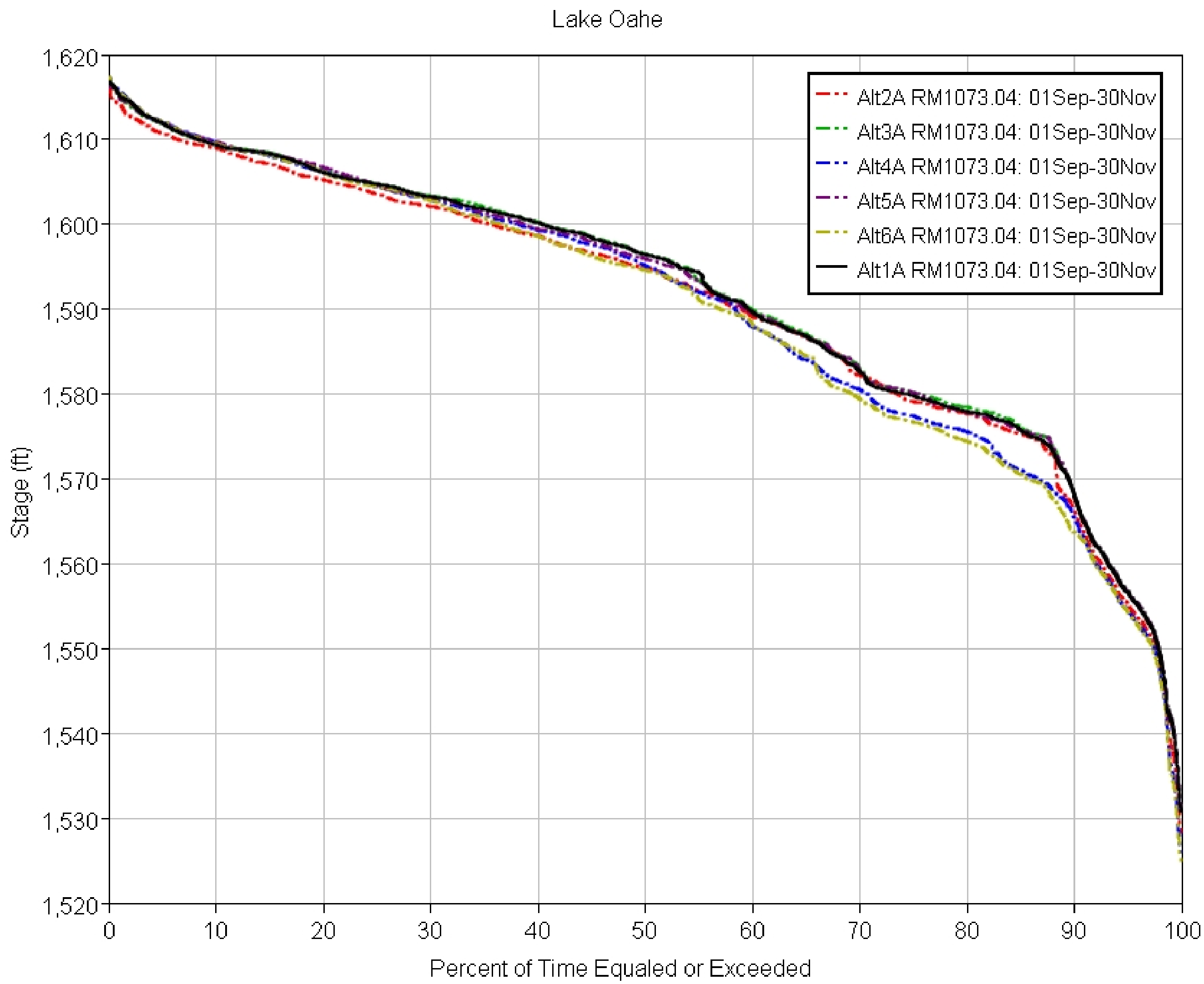


Plate 25: Lake Oahe Fall Duration

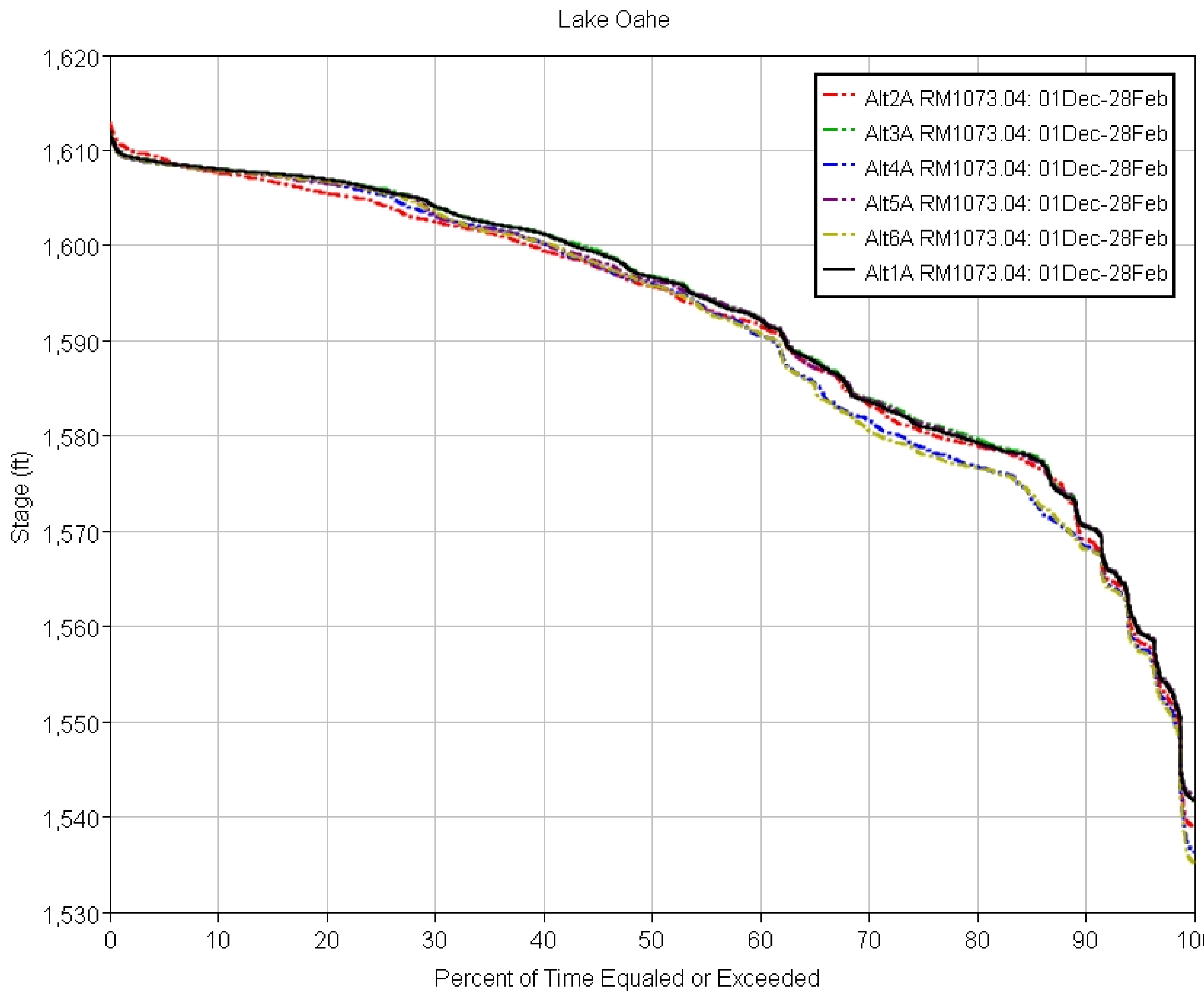


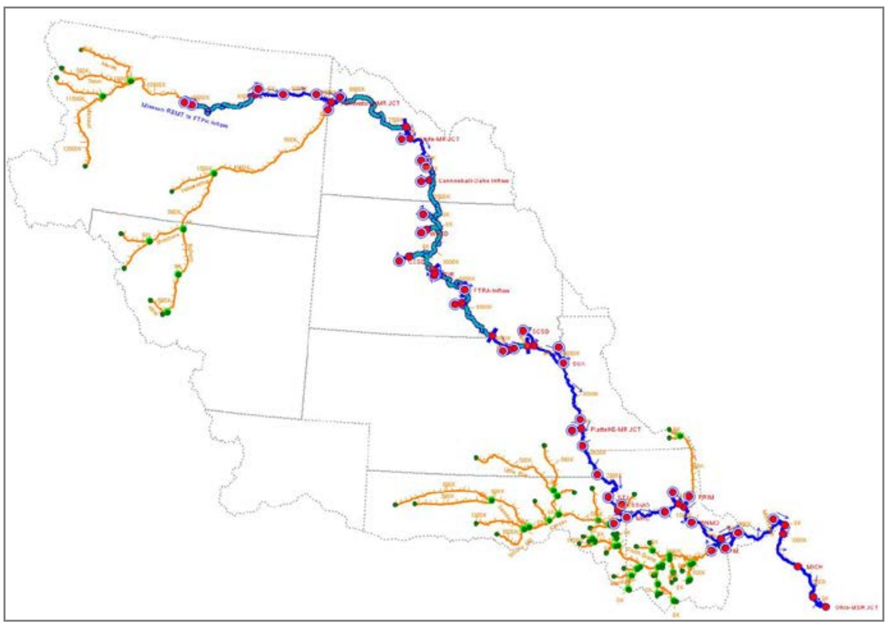
Plate 26: Lake Oahe Winter Duration

US Army Corps of Engineers ®
Omaha District

Missouri River Unsteady HEC-RAS Model
Sediment Analysis

FINAL

Garrison Dam to Oahe Dam



July 2018

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ACRONYMS

CFS…………………. Cubic Feet per Second

HC……………………Human Considerations

HEC .... Hydrologic Engineering Center

MRRIC………………..Missouri River Recovery Implementation Comittee

MRRPMP-EIS……….Missouri River Recovery Program Management Plan Environmental Impact Statement

POR......................... Period of Record

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

RM……………………1960 River Mile

ROD…………………...Record of Decision

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

USGS .... United States Geological Survey

WSP .... Water Surface Profile

1 INTRODUCTION

Future condition sedimentation evaluations were performed to help support questions and comments from Missouri River Recovery Implementation Committee (MRRIC) members, review panels, and the public related to sedimentation. Future conditions, that include aggradation and degradation within the reservoir reaches and navigation channel, may change the river stage - flow relationship and consequently could affect alternative condition performance. HEC-RAS (RAS) with sediment modeling was used to evaluate future condition channel conditions. While the current condition modeling effort is an informative tool, the Missouri River is dynamic with ongoing aggradation and degradation processes.

1.1 Background

Historically the Missouri River was characterized as a free-flowing, highly dynamic, multi-channel river, consisting of highly variable flows, high turbidity conditions, with many channel sandbars throughout the river channel and floodplain, which provided a diversity of habitat and food resources for many terrestrial and aquatic organisms. Since the late 1800's, the Missouri River has been modified by reservoirs, bank stabilization, construction of the navigation channel, and many other water resources development projects that have affected basin sediment yield and sediment transport within the mainstem Missouri River and tributaries. Navigation channel and bank stabilization structures have altered the historic multi-channeled, highly variable river system into a predominantly deep and swift, single channel in the lower river, downstream of Sioux City. Reservoir construction has created a series of alternating pools and open river reaches. These actions have combined to result in noticeable aggradation and degradation trends on the mainstem Missouri River. In summary, aggradation and degradation trends are documented, recent, and known to be ongoing.

1.2 Year 15 Future Condition

The future condition modeling was referred to as "Year 15". The designation "Year 15" comes from the timeframe for implementation; the Record of Decision (ROD) for the Missouri River Recovery Program, Management Plan Environmental Impact Statement (MRRPMP-EIS) is expected to be signed in 2018, with a construction completion date of 2033, resulting in an implementation period of 15 years. All alternatives were evaluated for the Year 15 future condition. While not intended to represent detailed estimates of future reservoir and channel conditions, the results do provide an alternative comparison methodology. Results from the Year 15 analysis were provided to the human considerations teams for qualitative evaluation, versus the full quantitative evaluation that was performed on the base condition (also referred to as Year 0).

1.3 Limitations

The future condition modeling was based on a number of critical assumptions regarding historic trends, flows, and sediment inputs. Model methodology is consistent for use with the MRRMP-EIS study using the period of record (POR) flow record. Calibration of the model was limited by available data.

Results are qualitative. Determination of future condition variation between alternatives and application with Human Considerations (HC) evaluation may by limited due to RAS model assumptions and accuracy.

2 METHODOLOGY

The methodology used a calibrated historic condition sediment model to derive reasonable sediment modeling parameters for the POR simulation. These parameters are used with the current condition RAS model available from the MRRMP-EIS study to simulate the 82-year period of record and develop a rate of change throughout the model length. HEC-RAS 5.0.3 was used for all of the model runs.

2.1 SIMPLIFYING ASSUMPTIONS

- The RAS model was simplified to only the Missouri River mainstem without any tributary routing reaches.
- Flow input locations were reduced to only tributaries with sediment loads. Small ungaged tributaries and the ungaged uniform lateral flows were not included.

2.2 MODELING OVERVIEW

- Assemble a historic model for the Missouri River
- Calibrate the historic model with steady flow and available water surface data (gages and profiles)
- Assemble sediment data and simulate from the historic period to current
- Assess sediment model performance with volume and water surface change
- Apply the sediment parameters to the current condition simplified Missouri River model and perform sediment computations using the 82-year POR flows
- Compute the water surface elevation change at the end of the 82-year period. Calculate the average annual rate and multiply by 15 years to produce a future "Year 15" water surface change estimate.
- Modify the model to produce a profile equivalent to the projected change over the 15 year period.

2.3 ASSEMBLE THE SEDIMENT MODEL INPUT DATA

- Each model starts at the upstream dam and assumes no sediment input from the reservoir release.
- Assemble Missouri River bed gradation from historic bed samples. Review data to ensure that the bed material includes larger gradations to reflect armoring in the immediate reach downstream of the reservoir.

- For tributary sediment input, assemble sediment load information using best available USGS gage data and previous studies. Focus on the major drainage area tributaries using the drainage area accounting table for the reach.
- Calibrate the historic condition RAS model water surface elevation to the warm season (mid-April through mid-October), which is of primary interest.
- Set movable bed limits using the bank stations.
- Select Toffaleti, Laursen Copeland, or Yang as the transport function (initial that will be verified in calibration).
- Select Copeland (Exner 7) or Exner 5 as the sorting method (initial that will be verified in calibration).
- Select Report 12 or Ruby fall velocity method (initial that will be verified in calibration).

2.4 Sediment Model Startup

Perform a stability check of the assembled sediment model and initial sediment parameter review for low, medium, and high flows.

- Create 30 day constant flow files of low (10k), medium (near bank full), and high flows (above bank in the non-degradation reach).
- Simulate sediment model for each flow condition, start with the medium flow condition for initial evaluation and debugging.
- Review performance at each flow, revise model levee stations / encroachments / ineffective flows / cross-section spacing to achieve reasonable sediment response.
- Check bed level change and debug problem areas.
- Perform initial adjustment of sediment input parameters to achieve 30 day model stability.

2.5 Calibration and Simulation of Historic Period

- Calibrate historic condition model roughness to water surface profiles at the start of the historic period.
- Simulate the flow period for the time period between surveys (e.g. 1975 thru 2012).
- Compare model computed volume change to actual on a reach basis for the simulation period.
- Compare water surface elevation at the end of the simulation period (e.g. 2012) to observed.
- Review and refine model sediment inputs.

2.6 Future Conditions Sediment Runs Using Parameters from the Calibration Model

The primary product from the development and calibration of the historic sediment model are the sediment modeling parameters. These parameters were used for the future conditions sediment model.

3 COMPUTATIONAL ANALYSIS SEDIMENT MODEL DEVELOPMENT

Available geometry, flow, and sediment data was used to assemble a historic model.

3.1 Sediment Model Computational Studies

Following the hydraulic model calibration, additional model calibration is required for modeling of sedimentation processes. Sediment modeling computational studies may be defined in two general categories (1) computational model studies and (2) computational analysis studies (Thomas and Chang 2008). Calibration is the process of arriving at sediment model parameters (e.g. hydraulic parameters such as roughness, sediment loads, sediment material size, sediment transport function, etc.) that will allow the model to calculate values that agree with measured prototype values. A sediment model computational study involves both calibration and verification. Verification refers to the demonstration that a calibrated model will match the prototype during a period of time not used in calibration. Due to the limited data set, model verification was not possible for the sediment model. Therefore, the performed evaluation is referred to as a computational analysis. Such studies are useful and allow evaluation of the study area and comparaison of calculated results between alternatives.

3.2 Historic Model Geometry

For consistency in the sediment modeling, the 1976 channel survey data was merged into the calibrated 2012 geometry from the previous modeling. Cross sections from the 2012 model that were not surveyed in 1976 were removed.

3.3 Hydrologic Data

Observed flow data from 1976 to 2012 was used for the sediment model computational analysis. Garrison Dam realeases and Lake Oahe pool elevations were used as well as tributary inflows, which included the Knife River, Heart River, and Cannonball River. The hydrologic data is summarized in Table 3-1.

Table 3-1. Hydrologic Data

Location	RS	Boundary Condition
Garrison Dam	1388.30	Flow Series
Knife River	1375.83	Lateral Flow Series
Heart River	1312.70	Lateral Flow Series
Cannonball River	1271.58	Lateral Flow Series
Lake Oahe (Oahe Dam)	1073.04	Stage Series

3.4 QUASI-UNSTEADY FLOW AND TEMPERATURE DATA

The quasi-unsteady flow application was selected for use with sediment modeling for this exercise instead of full unsteady. Quasi-unsteady flow allows the user to enter a time series of flow but the model does not alter the flow with computational routing such as would occur in full unsteady. Thus, channel and reservoir storage is not modeled with quasi-unsteady flow. Each inflow is entered into the quasi-unsteady flow editor within RAS as shown in Figure 3-1.

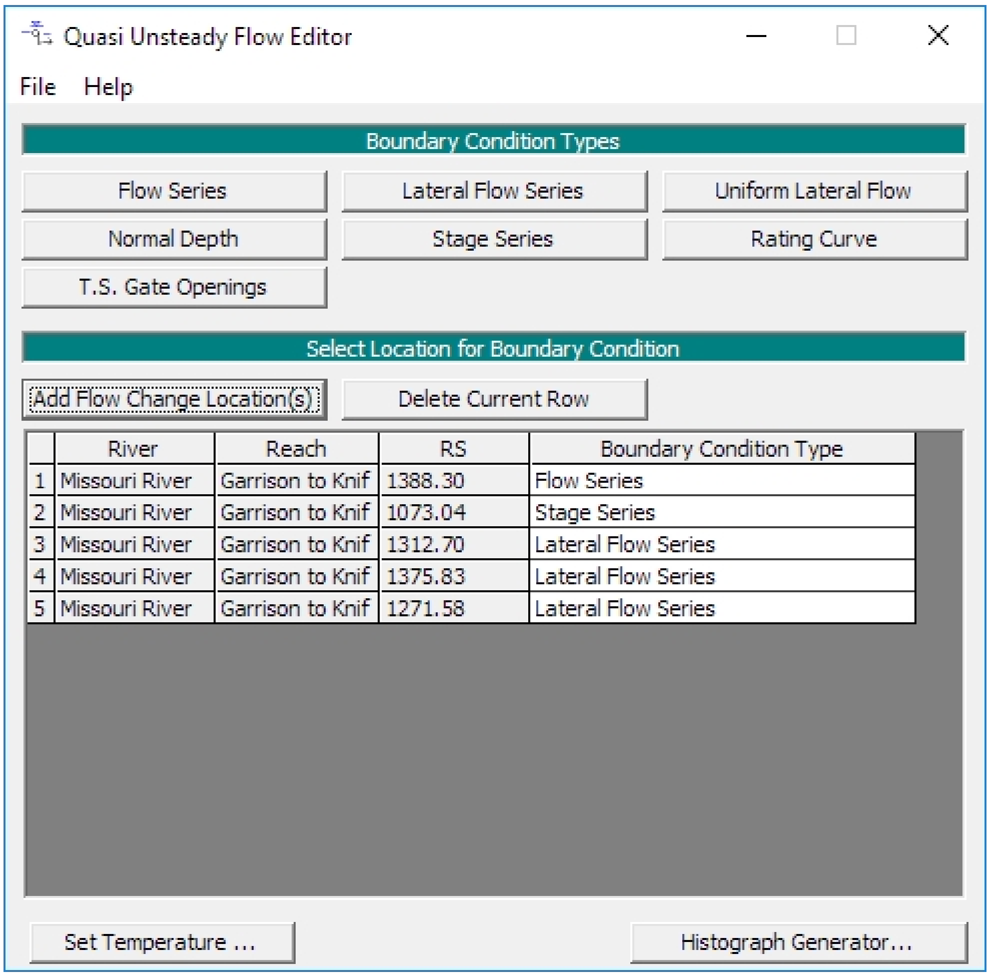
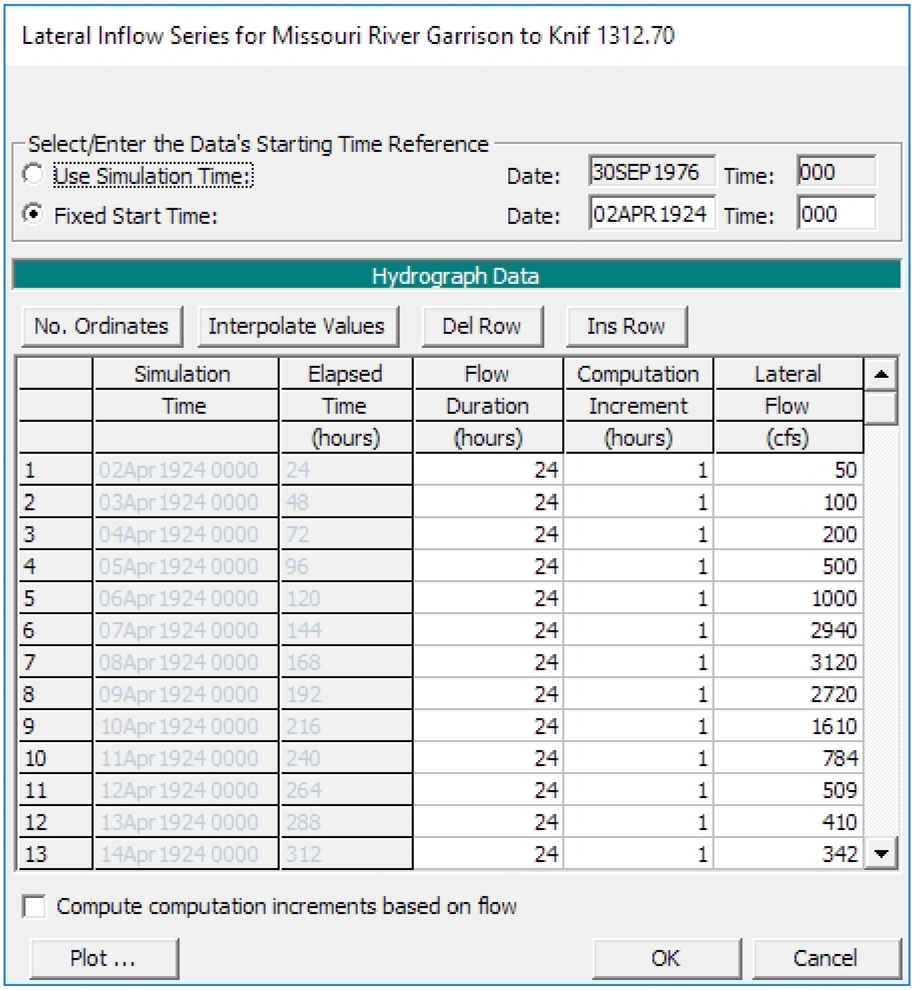


Figure 3-1. Quasi Unsteady Flow Editor

Within the quasi-unsteady flow editor, the user defines the boundary condition type for each inflow point. The furthest downstream cross section requires a downstream boundary condition which was set to a stage series of daily Lake Oahe pool elevations.

For this modeling application, all flow data was entered with a 24 hour duration and a computation increment of 1 hour. An example lateral inflow data input screen is shown in Figure 3-2.



	Simulation Time	Elapsed Time (hours)	Flow Duration (hours)	Computation Increment (hours)	Lateral Flow (cfs)
1	02Apr1924 0000	24	24	1	50
2	03Apr1924 0000	48	24	1	100
3	04Apr1924 0000	72	24	1	200
4	05Apr1924 0000	96	24	1	500
5	06Apr1924 0000	120	24	1	1000
6	07Apr1924 0000	144	24	1	2940
7	08Apr1924 0000	168	24	1	3120
8	09Apr1924 0000	192	24	1	2720
9	10Apr1924 0000	216	24	1	1610
10	11Apr1924 0000	240	24	1	784
11	12Apr1924 0000	264	24	1	509
12	13Apr1924 0000	288	24	1	410
13	14Apr1924 0000	312	24	1	342

Figure 3-2. Lateral Inflow Data Input Example

Within the editor, the user also enters temperature data. For this exercise, the temperature data did not cover the entire POR, therfore a monthly average temperature at Bismarck, ND (based on available data) was repeated for each year of the POR.

3.5 Sediment Modeling Parameters

Sediment transport modeling parameters are specified within the sediment boundary condition editor. The model used the Laursen(Copeland) sediment transport equation, the Copeland (Exner 7) bed mixing algorithm, and the Report 12 fall velocity method. The Copeland mixing method was selected because it often performs better than the alternatives for large sand bed rivers, and simulates erosion better in these systems (alternative mixing methods tend to over-predict armoring and, therefore, under predict erosion). Typical input within the RAS model is shown in Figure 3-3.

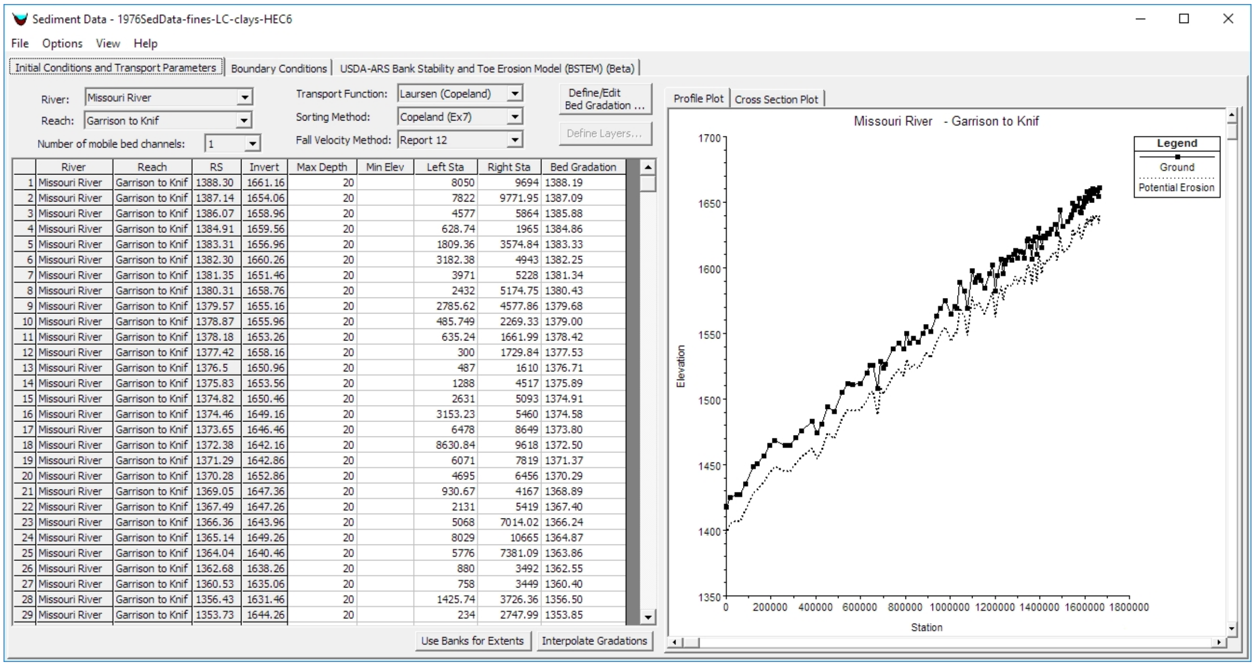


Figure 3-3. Sediment Data Example

3.6 Movable Bed Limits

Movable bed limits are used within the model to define how the cross section elevation points are adjusted in response to computed mass erosion or deposition. The movable bed limits were originally set at the channel banks but were changed during the calibration process. RAS has different bed change options and the option to not allow deposition outside of the moveable bed limits was selected. A typical cross section with the movable bed limits is shown in Figure 3-4.

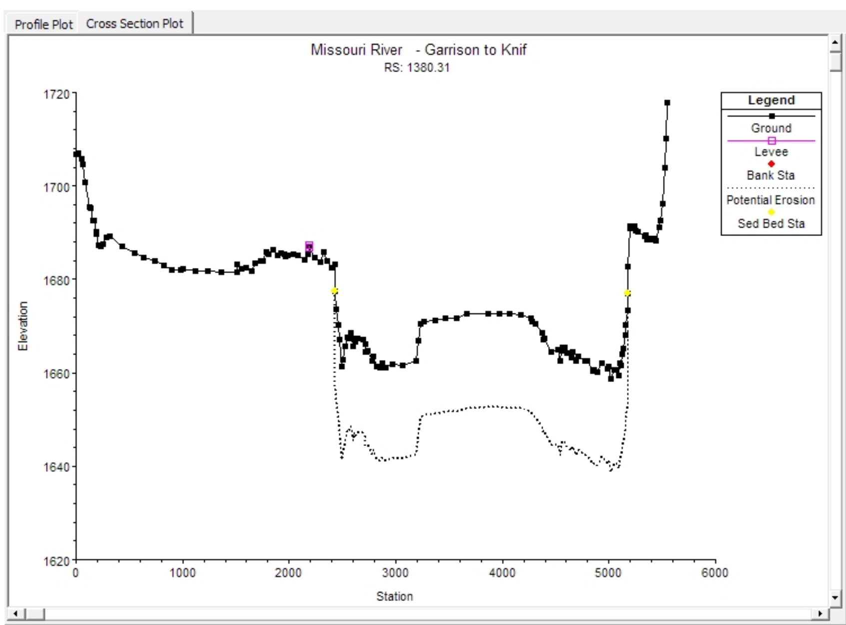


Figure 3-4. Moveable Bed Limits Example

3.7 Bed Gradation Data

A bed gradation is specified at each cross section or can be interpolated between cross sections. Bed gradation data within the model is from field measurements taken in 1976. An example bed gradation curve is shown in Figure 3-5.

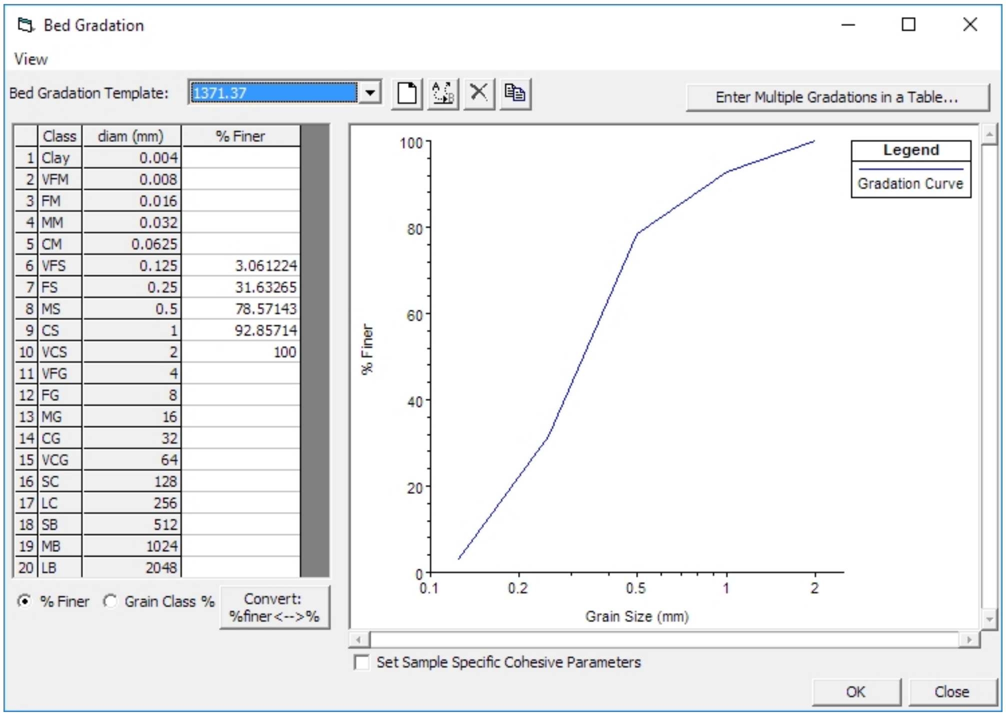


Figure 3-5. Bed Gradation Example

3.8 Sediment Boundary Condition: Flow-Load Rating Curve

Sediment models require sediment load boundary conditions at all locations where sediment inflow occurs. Locations may include the upstream model boundary and each of the flow input locations for the major tributaries and uniform lateral inflows. Sediment input is specified for each computational time step in the simulation. Because sediment time series data are rare, sediment models often use flow-load rating curves to compute sediment load boundary conditions from the flow boundary conditions.

The sediment loading curve is typically expressed as:

$$Q_s = a \times Q^b \qquad \textbf{Eq. 1}$$

Where

Q_s = Suspended sediment (tons/day)

Q = Discharge (ft^3/sec)

a = the intercept

b = the slope, exponent typically in the range between 1.5 and 2.5

The typical format of the sediment load relationship recognizes that the formulation of the sediment load power function as a linear model requires a logarithmic transformation to linearize the function and subsequently correct for subunity bias in the retransformation of sediment-discharge or –concentration estimates (Crawford 1991). The degree to which constituent discharges are underestimated as a result of retransformation is a function of the goodness-of-fit of the regression line. Generally, increasing the data scatter around the regression line results in decreasing estimates of the value of the dependent variable (Gray and Simões 2008). Therefore, best practice for fitting a rating curve to flow-load data involves computing an "unbiased corrector" to account for these biases. The Duan (1983) "smearing factor" approach is a method often employed to yield an unbiased flow load rating curve relationship that conforms with theory and experience. However, for this modeling exercise, the tributary sediment load data was sparse and an unbiased correction factor was not necessary.

A direct relation between Q and Q_s in streams is rarely present. A lack of synchronization between the peaks of water discharge and sediment concentration over a flood hydrograph is more the rule than the exception. That means that in parts of the hydrograph where sediment discharge is increasing, sediment concentration may be decreasing, and vice versa. In some cases, a piece-wise relationship, with different flow-load relationships for high and moderate flows might be appropriate. Piece-wise flow-load relationships (sometimes called "bent rating curves") are common in sediment analysis, because the highest flows are often supply limited, delivering less sediment than the capacity predicts because less sediment is available.

For input to the RAS model, the Q_s relationship shown in Eq. 1 was used to generate a rating curve for each input location. The input rating curves are illustrated in Table 3-2 through Table 3-5. During the study, HEC released a bug report indicating that sediment loads were doubled during model simulation. All values shown inare the actual tributary sediment loads. Report values were halved when entered in RAS.

Table 3-2. Garrison Sediment Rating Curve

Flow (cfs)	0.1	200,000
Load (tons/day)	0.2	2

Table 3-3. Knife River Sediment Rating Curve

Flow (cfs)	0	2,000	5,000	11,000	18,000
Load (tons/day)	0	4,757	19,913	68,263	147,361

Table 3-4. Heart River Sediment Rating Curve

Flow (cfs)	0	2,000	6,000	13,000	21,000	29,000
Load (tons/day)	0	3,888	20,543	66,290	137,097	223,577

Table 3-5. Cannonball River Sediment Rating Curve

Flow (cfs)	0	6,000	15,000	29,000	45,000	64,000
Load (tons/day)	0	63,362	269,407	763,231	1,527,794	2,664,960

3.9 SEDIMENT BOUNDARY CONDITION: LOAD GRADATIONS

Multiple grain size, sediment transport models do not just require upstream sediment load boundary conditions, but also require users to subdivide boundary loads by grain class. Additionally, load-gradation relationships can vary as a function of flow, sometimes in complicated or counterintuitive ways (Gibson and Cai 2017). Sometimes the load coarsens at higher flows and sometimes it fines. Very limited load gradation data was available for the tributary input locations. Load gradations were edited during the calibration process.

3.10 DATA GAPS AND DATA QUALITY

Data gaps for the RAS sediment modeling include tributary sediment load and gradation data, historic river geometry, and locations of bedrock.

3.10.1 Tributary Sediment Load and Gradation

Limited tributary sediment sampling information was available.

3.10.2 Channel Geometry

During model calibration and sediment simulation, it became apparent that historic surveys were limited in cross section. Volume change computations demonstrated the lack of quality information for the historic period.

3.10.3 Bedrock Controls

Very few bedrock controls are known to exist along the Missouri River. However, bedrock outcrops may be present in some areas within the degradation reach that acts to limit channel degradation. No bedrock controls were used within the model.

4 HISTORICAL MODEL STEADY FLOW CALIBRATION

The historic model from 1976 was calibrated to steady flow conditions using available profile data and observed gage flows. The calibration was performed with steady flow conditions. Water surface profiles from 1975, 1980, and 1985 were used to calibrate the geometry. The water surface profiles gave a wide range of flows, from 18,000 cfs to 50,000 cfs, to calibrate the geometry. Although some of the profile data points look suspect, for most of the model, the calibration was within 1 foot.

5 SEDIMENT SIMULATIONS

Sediment simulations were performed for the period from 1976 through 2012. Sediment parameters were adjusted based on volume change. Extensive sensitivity analyses were performed for all of the major sediment parameters.

5.1 VOLUME CHANGE

The primary data used for calibration of the sediment model was volume change. A steady model was used to compare volumes for the 1976 and 2012 geometries. A profile was set at the bank height of each cross section for both geometries. This was done to exclude the effects of bank failure in the longitudinal volume change. A comparison of using the channel only, overbank only, or the whole cross section is shown in Figure 5-1.

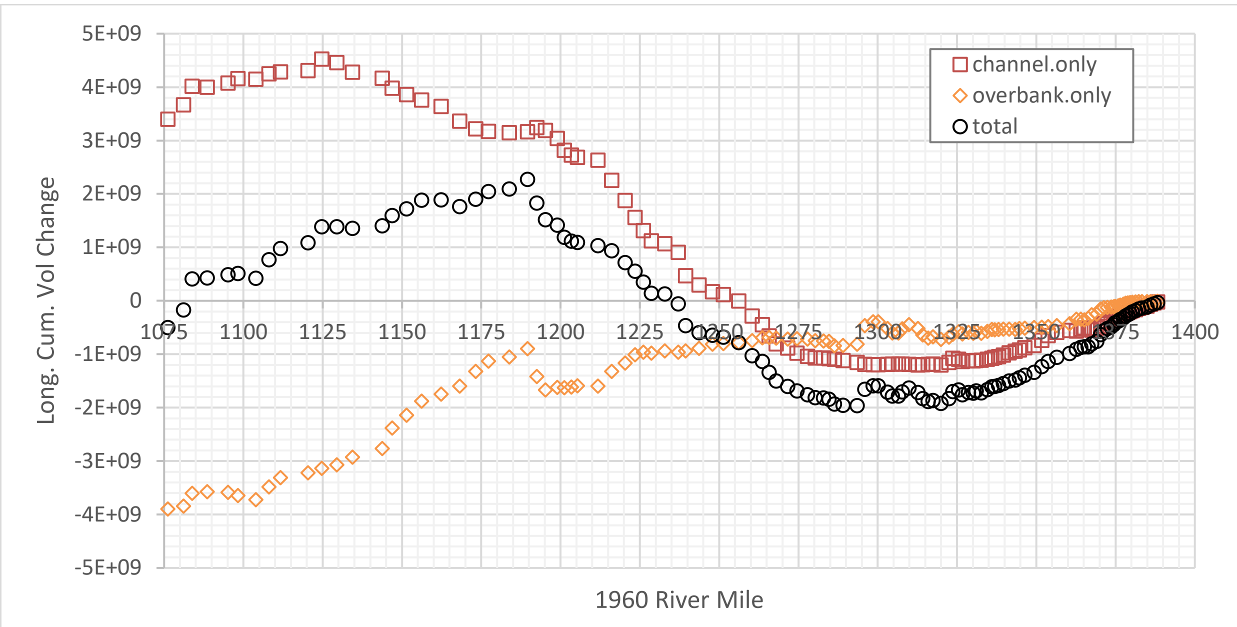


Figure 5-1. Volume Change Data Comparison

A range of sensitivities were run that included: transport function, fall velocity, sorting method, cohesive options, and computation time step. For the transport function, Acker-White, Laursen (Copeland), and Toffaleti-Meyer-Peter-Muller all performed similarly while Yang and (to a lesser extent) Toffaleti under-predicted erosion in the upper reach and Engelund-Hansen over-predicted erosion in the upper reach. For the fall velocity, Dietrich, Ruby, Toffaleti, and Van Rijn are virtually indistinguishable while Report 12 provided the best prediction. For the sorting method, Exner-5 predicted greater erosion and Exner-7 outperformed Exner-5 in the upper reach. For the cohesive options, a larger (more realistic) unit-weight for fines reduces net erosion in the upper reach. The results were insensitive to shear stress threshold. The effect of the HEC6T capacity method were less noticeable in the upper reach. The default unit weights for silt and clay were increased to 90 lbs/ft^3 and 80 lbs/ft^3, respectively, which reduced the net erosion in the upper reach. Example plots of the sensitivities for the transport function and sorting method are shown in Figure 5-2 and Figure 5-3. It should be noted that the RAS model does not reproduce the transport of fines far into the reservoir well so the plots only show from the dam down to river mile 1225, which is well into the reservoir.

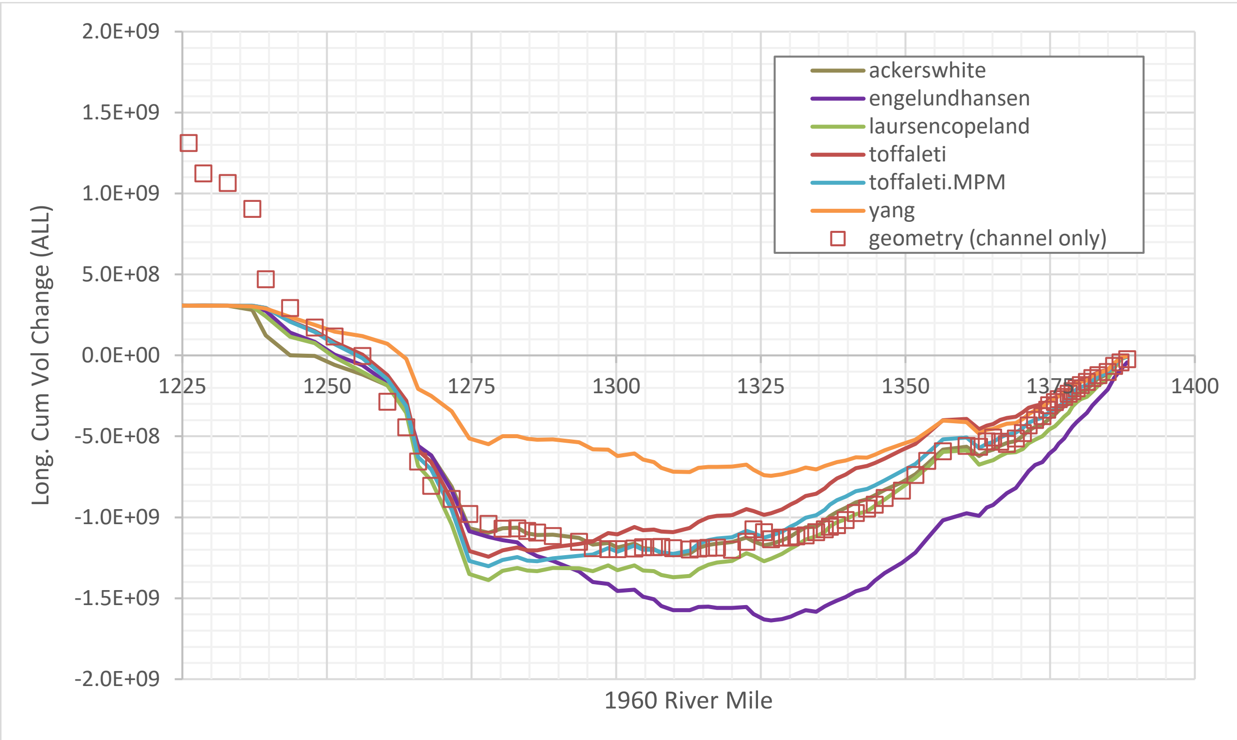


Figure 5-2. Transport Function Sensitivity

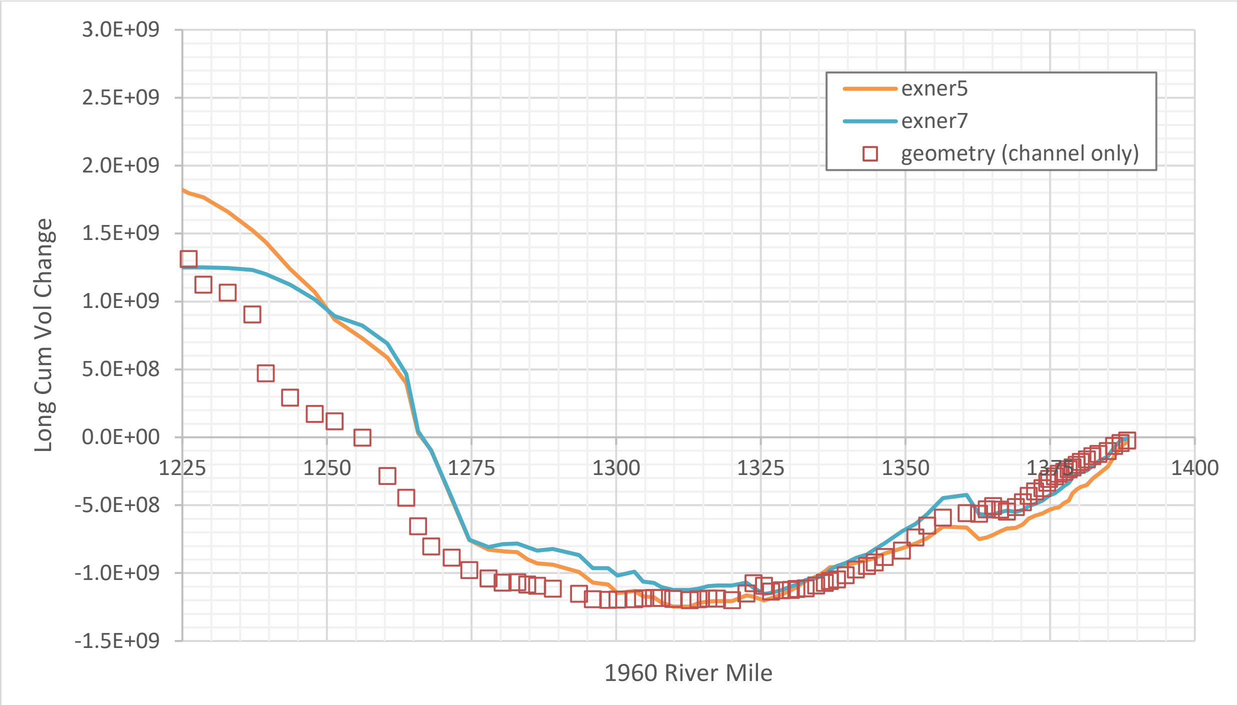


Figure 5-3. Sorting Method Sensitivity

For the final calibration the parameters chosen as the best fit to historic data are shown in Table 5-1. A plot of the longitudinal cumulative volume change for the final run is also shown in Figure 5-4.

Table 5-1. Final Calibration Sediment Parameters

Transport Function	Laursen (Copeland)
Fall Velocity	Report 12
Sorting Method	Copeland (Exner 7)
Cohesive Option	Krone/Partheniades HEC 6T Capacity Method
Unit Weight	Sand/Gravel: 93 lbs/ft^3 (default)
	Silt: 90 lbs/ft^3 (default 65 lbs/ft^3)
	Clay: 80 lbs/ft^3 (default 30 lbs/ft^3)

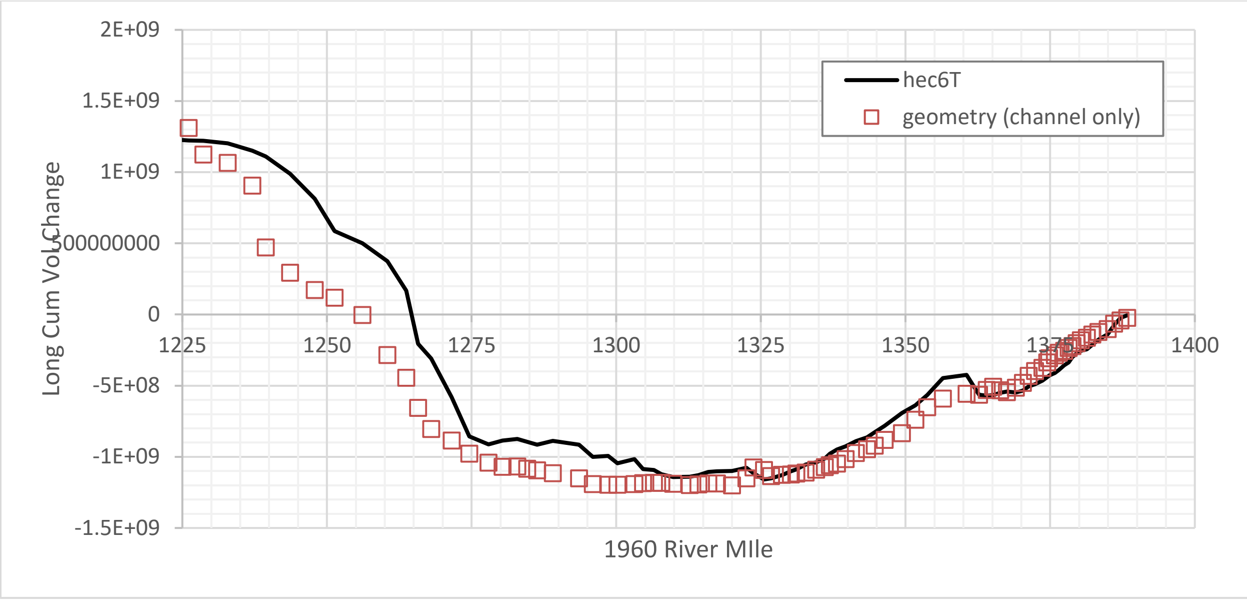


Figure 5-4. Longitudinal Cumulative Volume Change - Final Calibration

6 FUTURE CONDITIONS SEDIMENT RUNS

Future condition sediment runs used the sediment parameters from the calibration sediment model. For the future condition sediment runs, the geometry was updated to the 2012 survey and the previously used POR flows from 1930 to 2012 were used to model 82 years into the future for the six alternatives. The average water surface change for that period was then used to estimate the Year 15 geometry.

6.1 GEOMETRY

The 2012 (sediment range only) geometry was used in the future conditions sediment runs.

6.2 FLOW

Simulations used the Management Plan study alternative flows that were previously developed for the existing condition model. The POR flows include ungaged inflow and current level depletions. Alternative simulations were performed for the POR from 1930 to 2012. Unsteady flows from the POR were converted to quasi-unsteady for use with the future conditions sediment model. Since there are different rules for unsteady and quasi-unsteady flows, some uniform lateral flows had to be adjusted so that they did not start at the same cross section as another flow input. More details on the POR flow dataset can be found in the report, *Missouri River Recovery Management Plan Time Series Data Development for Hydrologic Modeling* (USACE 2018c).

6.3 Sediment Parameters

Sediment parameters from the calibration sediment model were used for the future conditions sediment runs. Some parameters were updated such as bed data was updated to 2014. The moveable bed limits were also updated due to using a different geometry with different stationing.

7 SUMMARY

A sediment model was constructed for the Garrison to Oahe reach of the Missouri River t for the purposes of evaluating future conditions in support of the Missouri River Management Plan study. Sediment model simulations for the period from 1976 to 2012 were used to calibrate sediment parameters. These parameters were then used to run a prediction of the future conditions, starting in 2012. These results were then averaged to produce a water surface change estimate for the year 2032 (Year 15). Details on how the Year 15 future geometry was created can be found in the *Missouri River Recovery Management Plan Environmental Impact Statement HEC-RAS Modeling Alternatives Report* (USACE 2018b).

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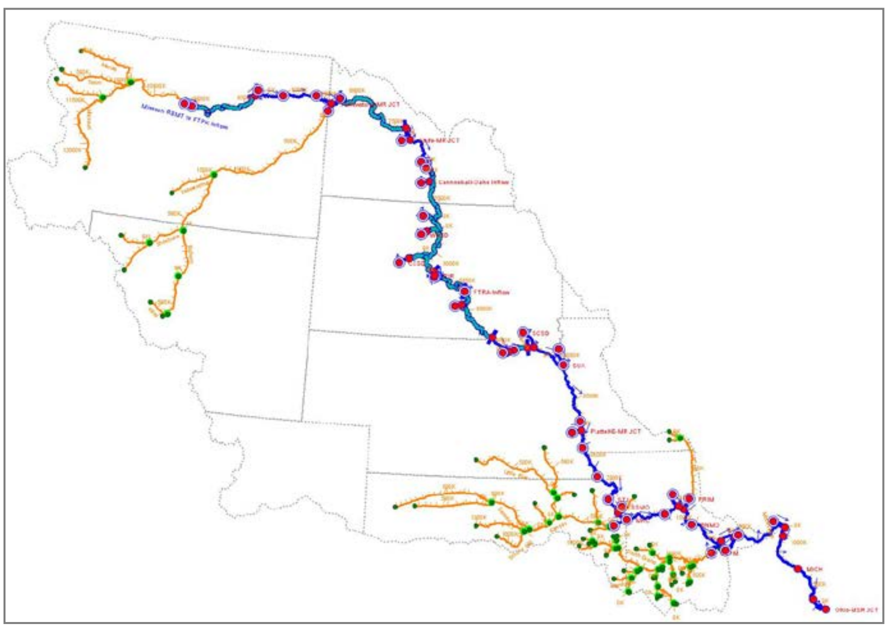
Omaha District

Missouri River Unsteady HEC-RAS Model
Alternatives Analysis

FINAL

Appendix C

Fort Randall Dam to Gavins Point Dam



July 2018

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ATTACHMENTS

ACRONYMS

BiOp…………………..Biological Opinion

CFS…………………. Cubic Feet per Second

ESH…………………..Emergent Sandbar Habitat

HC……………………Human Considerations

HEC .... Hydrologic Engineering Center

IRC……………………Interception-Rearing Complexes

MAF………………….. Million acre-feet

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

MRRPMP-EIS…...……Missouri River Recovery Program Management Plan Environmental Impact Statement

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

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

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

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

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

POR .... Period of Record

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

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

RM……………………1960 River Mile

ROD…………………..Record of Decision

SWH………………....Shallow Water Habitat

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

USFWS……………….United States Fish and Wildlife Service

USGS .... United States Geological Survey

1 INTRODUCTION

The Missouri River unsteady HEC-RAS (RAS) model was developed for the Missouri River Recovery Program Management Plan and Integrated Environmental Impact Statement (MRRPMP-EIS) to assist in the assessment of a suite of actions to meet Endangered Species Act (ESA) responsibilities for the piping plover, the interior least tern, and the pallid sturgeon using USACE authorities. Model geometry development and calibration for the existing conditions is documented in *Missouri River Unsteady HEC-RAS Model Calibration Report Appendix C Fort Randall Dam to Gavins Point Dam* (USACE 2015). The objective of the RAS modeling is to simulate the Management Plan alternatives which include flow changes relative to the No Action alternative. The Human Considerations (HC) team performed an extensive analysis on each of the alternatives for each of the resources (hydropower, cultural resources, fish & wildlife (exclusive of listed species), flood risk, irrigation, recreation, thermal power, and water supply) and provide a detailed comparison of results. For this report, only the hydraulic model output is presented; there is no alternative selection or discussion. This Appendix is for the Fort Randall Dam to Gavins Point Dam reach of the Missouri River as part of the Omaha District.

Six alternatives, including the No Action alternative, were simulated in RAS from March 1930 to December 2012, however the HC team only used complete year data for their analysis from January 1, 1931 to December 31, 2012. Development of inflow records at current depletion levels to use as boundary conditions for the HEC-ResSim (ResSim) and RAS models is documented in the report, *Missouri River Recovery Management Plan Time Series Data Development for Hydrologic Modeling* (USACE 2018c). Each alternative has unique flow releases from the reservoirs, as simulated by the ResSim model.

2 GEOMETRY

For the Fort Randall Dam to Gavins Point Dam model, no geometry changes were modeled. All alternative runs used the current conditions (2012) calibrated geometry.

3 FLOW ALTERNATIVES

A total of six flow alternatives were modeled in ResSim. Reservoir pool elevations and dam outflow output from the ResSim model was used as input for the RAS models for each of the six flow alternatives. A brief summary of the flow alternatives is provided below. For more details, see the ResSim technical report, *Mainstem Missouri River Reservoir Simulation Alternatives Technical Report* (USACE 2018a).

Tributary inflows were kept consistent between alternatives. Due to the lack of flow gages in this reach, no ungaged inflows were calculated. More details on the Period of Record (POR) flow dataset used can be found in the report *Missouri River Recovery Management Plan Time Series Data Development for Hydrologic Modeling* (USACE 2018c).

3.1 ALTERNATIVE 1 - NO ACTION

Under Alternative 1 (Alt 1), the Missouri River Mainstem Projects would continue to be operated as they are currently. Operations within the ResSim model were set up to closely follow the Master Manual that is used during real-time operations of the System; however, the model does have limitations and cannot capture all real-time decisions that occur.

3.2 ALTERNATIVE 2 - U.S. FISH & WILDLIFE SERVICE 2003 BIOLOGICAL OPINION PROJECTED ACTIONS

Alternative 2 (Alt 2) represents the U.S. Fish and Wildlife Service (USFWS) interpretation of the management actions that would be implemented as part of the 2003 Amended BiOp RPA (USFWS 2003). Operational criteria include different early and late spring spawning cues, low summer flows, and a maximum winter release limit.

3.3 ALTERNATIVE 3 - MECHANICAL CONSTRUCTION ONLY

Alternative 3 (Alt 3) consists of mechanical construction of emergent sandbar habitat (ESH). Operational criteria consist of removing the early and late spring spawning cues in Alt 1.

3.4 ALTERNATIVE 4 - SPRING HABITAT-FORMING FLOW RELEASE

Under Alternative 4 (Alt 4), the early and late spring spawning cues in Alt 1 are removed from the operational criteria and a spring ESH-creating reservoir release from Gavins Point and Garrison is added. While the ESH-creation release is occurring from Gavins Point, flood targets are increased to allow the ESH-creation release the opportunity to run.

3.5 ALTERNATIVE 5 - FALL HABITAT-FORMING FLOW RELEASE

Alternative 5 (Alt 5) removes the early and late spring spawning cues in Alt 1 and adds a fall ESH-creating reservoir release from Gavins Point and Garrison to the operational criteria. While the ESH-creation release is occurring from Gavins Point, flood targets are increased to allow the ESH-creation release the opportunity to run.

3.6 ALTERNATIVE 6 - PALLID STURGEON SPAWNING CUE

Alternative 6 (Alt 6) replaces the early and late spring spawning cues with different spawning cues. The early spring spawning cue in Alt 6 occurs at the same time as the early spring spawning cue in Alt 1 but with a higher peak release. The late spring spawning cue in Alt 6 occurs later in May than the late spring spawning cue in Alt 1 and has a larger peak release. Please note that the former name of this alternative was Alternative 7, which may correspond to some RAS model runs and file names.

4 SIMULATION OF ALTERNATIVES

Each flow alternative from ResSim was run through RAS with the current conditions geometry. Alternative names match in both ResSim and RAS.

5 RESULTS

All alternative runs were performed in HEC-RAS 5.0.3. Model output contains a considerable amount of information, not easily condensed to simple conclusions. Each of the six alternative runs produced 82 years (March 1930 – December 2012) of stage and flow hydrographs. To express the changes compared with the No Action alternative, the model results were evaluated by statistical evaluation and duration analysis plots.

Results from the 82-year runs for the six alternatives were provided to the HC team for analysis. They used the daily (instantaneous 2400 value for each day) flow and water surface elevation output to analyze effects to various resources that include: hydropower, cultural resources, fish & wildlife (exclusive of listed species), flood risk, irrigation, recreation, thermal power, and water supply. The HC team performed an extensive analysis on each of the alternatives for all of the resources and provide a detailed comparison of results. For this report, only the hydraulic model output is presented.

5.1 STATISTICS

For the statistical evaluation, daily flow and water surface elevation results were analyzed to compare the differences between the No Action Alternative and the remaining five alternatives. All of the alternatives show minor changes, while Alternative 2 shows the most difference with respect to both flow and water surface elevation. Tables showing the differences between calculated statistics for both flow and water surface elevation for below Fort Randall Dam, Greenwood, Verdel, Niobrara, Springfield (elevation only), and Lewis and Clark Lake (elevation only) can be seen in Plate 1 and Plate 2. The statistics calculated include: the 10th, 25th, 50th, 75th, and 90th- Percentiles, and the Minimum and Maximum. It should be noted that the percentile statistics calculated are from a duration analysis and not a Bulletin 17B flow frequency analysis.

The min and max are the lowest daily flow or stage and the highest daily flow or stage output for each alternative over the period of record. For model stability, a minimum flow of 2,000 cfs was used for Fort Randall Dam outflow in RAS. The ResSim model output for Fort Randall Dam produces daily outflows below the RAS minimum flow of 2,000 cfs, however the impacts to the HC analysis should be minimal. As seen in the tables, the minimum flow varies slightly between alternatives while the maximum flow shows greater differences. The dramatic reduction in the minimum flow for Verdel may have been caused by relatively low Missouri River flows coupled with a backwater effect of high Niobrara River flows, just downstream of Verdel. Caution should be used when trying to draw conclusions from the statistics alone. The economic models (HEC-FIA) provide a more complete analysis of how high flows effect total damages for each alternative because they incorporate all of the cross section output, whereas these tabular statistics only capture one location.

Stage statistics have been rounded to the nearest tenth of a foot, which is equivalent to 1.2 inches. This helps demonstrate how flow changes impact river elevations, which is the more tangible result. For example, even though the 90th percentile flow for Greenwood in Alternative 5 was 277-cfs higher than in Alternative 1, there is less than an inch of impact to the water surface elevation of the river, and therefore zero reported change.

It is also important to note that the RAS alternative models, although they have a 30 minute computation interval, have been configured to report one value per 24 hour period, and unfortunately that one value is not a daily average. The HEC-RAS model reports the value that lands on 2400 of each day. The most reasonable output interval was chosen as daily due to the size of watershed being modeled, POR length, and the number of hydrograph locations necessary for HC analysis. This means that slight shifts in timing from alternative to alternative can carry over into the results as small fluxuations in the reported flow. Changes in timing are a small factor, not likely to significantly impact any results evaluation, but should be kept in mind when making comparison at a precise level such as in the statistics tables.

5.2 Seasonal Duration Plots

A duration analysis was also performed for the alternative output. Seasonal duration plots for key main stem locations including below Fort Randall Dam, Greenwood, Verdel, Niobrara, Springfield (elevation), and Lewis and Clark Lake (elevation) are shown in Plate 3 through Plate 22. Seasonal dates chosen for the duration analysis coincide with the current System operational seasons: spring (1Mar to 30Apr), summer (1May to 31Aug), fall (1Sep to 30Nov), and winter (1Dec to 28Feb). Alternatives 2, 4, and 6 have the most notable differences during the spring duration due to the spring pulses included in those alternatives. Lower winter releases can be seen for Alternative 2 and are caused by storage usage due to the spring pulses.

5.3 Limitations

The analysis relies on the simulation of the 82-year period of record using daily average outflows from an ResSim model input into a fixed bed RAS model, with stage and flow output. While the analysis coupled with species and human considerations models can be used to show relative benefits and potential impacts based on historic flows, there are limitations in the conclusions that can be drawn based on some of the simplifying assumptions.

1. **POR Methodology** - An 82-year period of record, adjusted to current level of depletions, was used and may not be comparable to future conditions. A climate change assessment of the Missouri River basin indicates increases to both temperature and precipitation along with increasing trends in extreme floods and droughts (USACE 2018b). The conditions during a pulse year in the future could vary greatly from the small sample of pulse events included in the POR analysis.

2. **No Risk Analysis** - The Missouri River system as currently operated provides substantial flood damage reduction and benefits to the entire basin. The current ResSim and RAS analysis, which employs an 82-year period of record simulation, shows the potential for negative impacts to flood damage reduction and dam safety for alternatives that include changes in reservoir flow releases. The current study methodology does not simulate a sufficient number of events and possible runoff combinations within the large Missouri River basin to evaluate potential change in downstream flood risk and dam safety.

   Scoping efforts were conducted to determine a Monte Carlo risk analysis methodology capable of assessing impacts to dam safety and flood damage reduction as a result of flow release changes. The risk analysis primary components include further development of the period of record flow data set, ResSim and RAS model modifications, development of levee fragility curves, assignment of uncertainty, assembly and debugging of models, Monte Carlo simulation, analysis of results, and reporting. The Monte Carlo methodology properly assesses the effects of the alternative operation changes because it increases the sample size of flow data and number of combinations of flow periods that may occur in the future so that impacts can be characterized with greater confidence. Without such analysis, the impacts of operational changes will only be known for events and combinations of events that have already occurred. Statistics calculated based on the 82-years of record should therefore be used with caution, and with the understanding of the consequences of using only a small sample of years.

3. **Stable Bed and Floodplain** - The hydraulic modeling to date is based on the existing conditions geometry. The analysis does not account for how the bed of the Missouri River may respond to flow changes. Additionally, the analysis does not try to project where sediment may accumulate in the floodplain or include projections of future change in floodplain roughness that could occur during the POR simulation. This carries with it the necessary assumptions that any bed and floodplain changes would be either negligible or similar between each alternative.

5.4 Channel Capacity Analysis

Channel capacity estimates were performed to provide an indication of the flow rate at which bank elevations are overtopped and flow begins to leave the main channel and enter the floodplain. Channel capacity estimates were performed with the one-dimensional RAS model calibrated to 2012 conditions by comparing steady flow profiles with top of bank elevations at each cross section combined with reviewing the best available floodplain topography. Floodplain flow connectivity was not assessed. The estimated channel capacity does not necessarily correlate with the onset of flood damage. In addition, channel capacity is typically highly variable along the channel bank due to wide variation in bank elevations. The quality of the channel capacity estimate is affected by numerous factors including how representative the model cross sections are of river geometry, local channel geometry variation, low spots in bank elevations, and the floodplain topography accuracy. Within the reservoir delta areas where the river enters the downstream lake, the channel capacity estimate is not meaningful. While channel capacity varies within the reach and through time, a range for the area below the Niobrara River is 35,000 to 40,000 cfs.

A Monte Carlo risk analysis methodology capable of assessing impacts to dam safety and flood risk as a result of flow release changes would be required to fully assess how an alternative impacts potential flood risk. Refer to the *Summary of Hydrologic Engineering Analysis* (USACE 2018d) for additional details on the risk analysis methodology.

6 ADDITIONAL ANALYSIS FOR YEAR 15

Degradation and aggradation of the Missouri River channel bed and sedimentation in the reservoirs are ongoing processes, which have the potential to effect virtually all economic resources and human considerations. Therefore, additional modeling was performed to provide estimates of how ongoing sedimentation processes may affect the performance of future alternatives. The future without and with project condition modeling is based on a number of critical assumptions regarding historic trends, flows, and sediment inputs. While not intended to represent detailed estimates of future channel conditions, the results do provide an alternative comparison methodology. The designation “Year 15” comes from the timeframe for implementation; the Record of Decision (ROD) for the MRRPMP-EIS is expected to be signed in 2018, with a construction completion date of 2033, resulting in an implementation period of 15 years. Results from the Year 15 analysis were provided to economists and human considerations teams for qualitative evaluation, versus the full quantitative evaluation that was performed on the base condition (also referred to as Year 0).

6.1 YEAR 15 FUTURE CONDITIONS

To project river bed aggradation and degradation trends to the year 2033, moveable bed sediment models of the mainstem Missouri River reaches were created in RAS. Results from the sediment modeling provided a projected change in water surface at a normal flow for each of the six alternatives. Full details on construction and calibration of the sediment model are in Attachment 1, Missouri River Unsteady HEC-RAS Model Sediment Analysis, Fort Randall Dam to Gavins Point Dam.

6.2 ALTERNATIVE ANALYSIS

Alterations were made to both the ResSim and RAS models to represent conditions at the end of the implementation period. All six alternatives were re-run, and results were compared between alternatives and to the base condition.

6.2.1 Geometry

In RAS, the Missouri River channel bed was adjusted to represent the aggradation or degradation that was estimated for the end of the implementation period. Water surface change at a normal flow of 30,000 cfs was used to estimate bed change. Since the forecasted water surface change between alternatives was generally within 0.1 ft, an average of all of the alternatives was used and only one future geometry was created and used for all alternatives. For areas with forecasted degradation, the channel points between the banks were adjusted vertically downward while for areas with forecasted aggradation, the fixed sediment elevation tool was used to produce the forecasted water surface increase. Figure 6-1 shows a cross section that was adjusted for anticipated degradation and Figure 6-2 shows a cross section adjusted for anticipated aggradation.

Figure 6-1. Cross Section Vertical Adjustment for Year 15 - Degradation

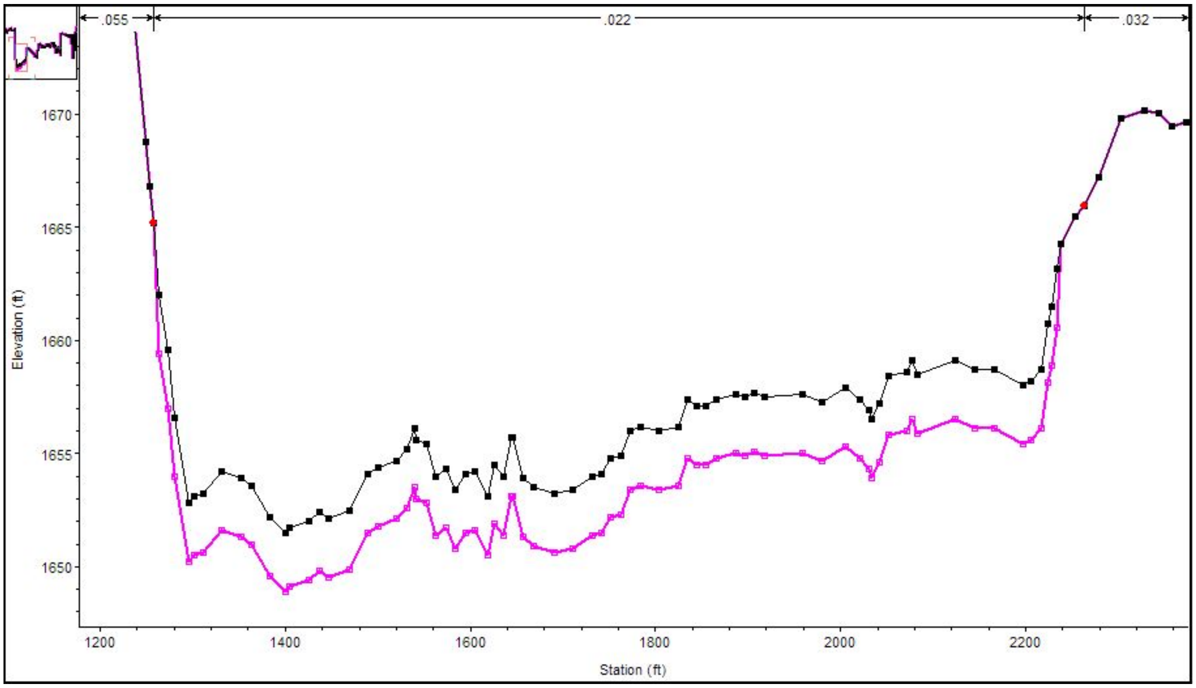
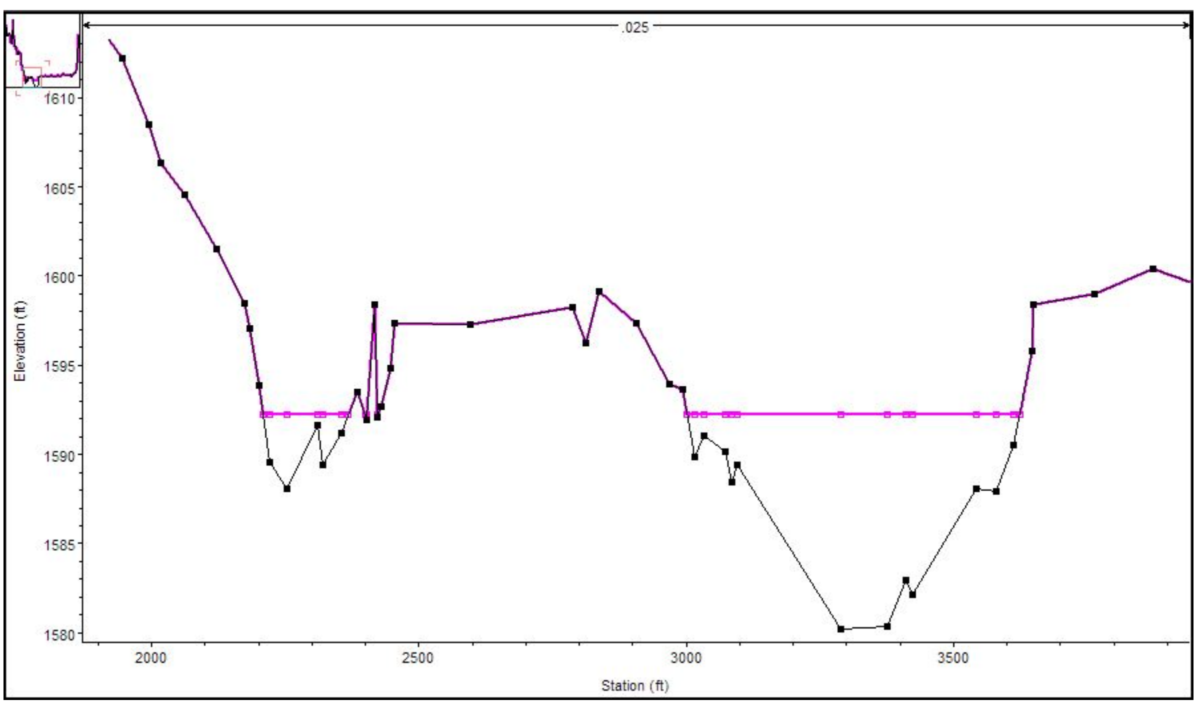
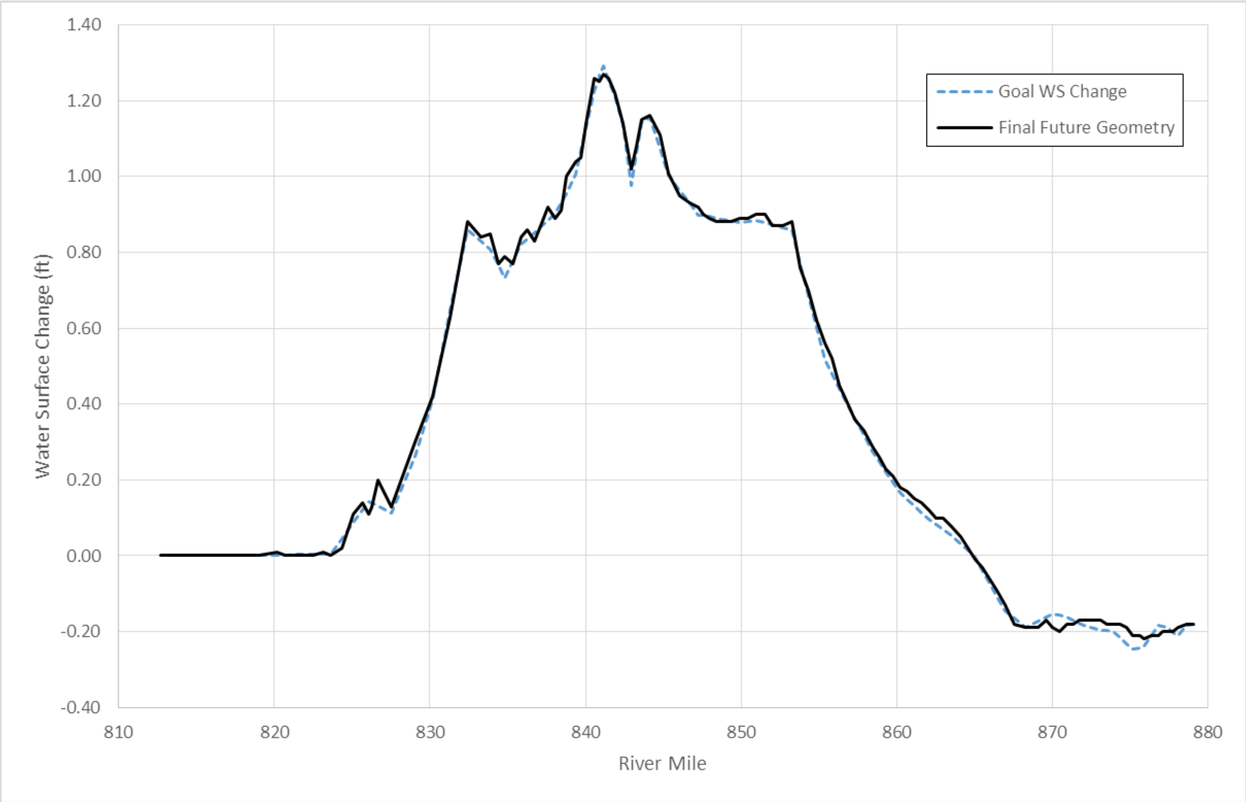


Figure 6-2. Cross Section Vertical Adjustment for Year 15 - Aggradation



For areas of forecasted degradation, a simplified RAS sediment model was used to produce the desired water surface change at a normal flow of 30,000 cfs. Minimum channel elevations were adjusted until the model changed the water surface the desired amount. Similarly, for areas of forecasted aggradation, the fixed sediment elevation tool was used to produce a water surface change. For both aggradation and degradation, water surface changes were generally within 0.05 ft of the estimate. A plot of the water surface change estimate vs what the future geometry actually produces is shown in Figure 6-3. The aggradation reach stops at the river mile of the normal pool level.

Figure 6-3. Year 15 Water Surface Change Estimates vs. Actual



6.2.2 Flows

Reservoir storage volumes were adjusted in ResSim to represent sedimentation in the reservoirs that could be expected by the end of the implementation period. Reservoir operation rules were left the same as the base set of alternatives. All six alternatives were re-run in ResSim for the period of record, resulting in slightly different release decisions from the System. Refer to *Mainstem Missouri River Reservoir Simulation Alternatives Technical Report* (USACE 2018a) for details regarding the ResSim modeling.

6.2.3 Results

Output from the Year 15 analysis can be evaluated in two ways. First, comparison of the Year 15 alternatives to the Year 15 No Action provides additional data to inform on how alternative performance may vary in the future for consideration with the selection of a preferred alternative. Second, comparison of the Year 15 alternatives to the base condition provides a sense of how future channel bed and reservoir sedimentation conditions may impact Missouri River flows and stages. The second evaluation is limited in the useful information it can provide to the decision making process, as the results are directly tied to modeling assumptions that were made about an unknown future using historic data. The first evaluation parallels the comparisons made amongst alternatives in the base condition analysis. Visual and statistical evaluation of the Year 15 output indicates that regardless of changed conditions, the alternatives compare similarly to each other.

Statistical whisker plots for Fort Randall Dam outflow and Lewis and Clark Lake pool elevation are shown in Figure 6-4 and Figure 6-5. The figures present the statistics of minimum, maximum, 5th percentile, 95th percentile, median and mean for all six alternatives for the base condition (Year 0) and Year 15 (Long 2017).

Figure 6-4. Flow Statistics at Fort Randall Dam

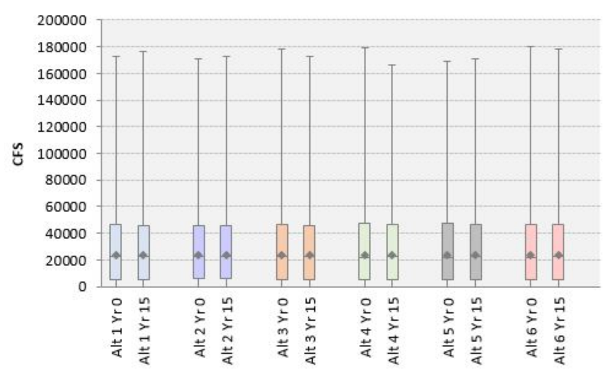
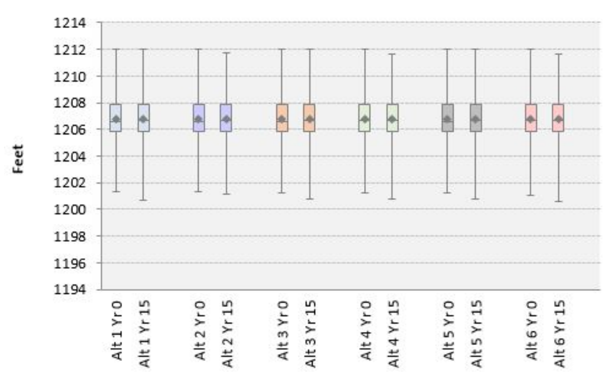


Figure 6-5. Elevation Statistics at Lewis and Clark Lake



Although comparisons between Year 15 and Year 0 have limited usefulness in alternative selection, a few trends are worth pointing out. First, differences in flow statistics between Year 0 and Year 15 are almost negligible. Any differences that do show up can be attributed to release changes due to different storage volumes in the reservoir system. These differences decrease moving downstream. Other flow differences compared to Year 0 could be due to slightly different routing because of changed channel capacities. These would accumulate and be greater moving downstream, but they are slight in magnitude and pretty difficult to discern from model noise. Second, differences in stage statistics between Year 0 and Year 15 are directly related to the vertical bed change applied locally. As stated previously, the relative comparison between alternatives for Year 15 produces very similar results to the base condition analysis.

Previously discussed limitations for the alternative analysis also apply to the Year 15 analysis. Although the Year 15 model geometry includes an adjustment to the river bed, the model geometry is static and does not change during the 82 year POR.

7 CONCLUSIONS

The unsteady RAS model analysis gives a means to systematically evaluate differences in river elevations for various reservoir and habitat alternatives given the limitations presented in Section 5.3. These results can be fed into additional species and human considerations models, such as HEC-FIA, to screen alternatives for relative benefits and potential economic impacts. The outputs should be carefully examined with an eye towards the model limitations and judgement applied where needed to mitigate any potential pitfalls of the hydraulic analysis. Additional modeling was performed to provide estimates of how ongoing sedimentation processes may affect the performance of future alternatives. While not intended to represent detailed estimates of future channel conditions, the Year 15 results do provide an alternative comparison methodology that was evaluated qualitatively rather than quantitatively for economic and human consideration impacts. If flow change alternatives are considered for implementation, additional risk and uncertainty analysis is recommended to more comprehensively quantify risk of spring or fall pulse flows. If flow change alternatives are considered for implementation, additional risk and uncertainty analysis is recommended to more comprehensively quantify risk of spring or fall pulse flows.

8 REFERENCES

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APPENDIX C

FORT RANDALL DAM TO GAVINS POINT DAM

PLATES

Flow (cfs)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Fort Randall - XS 879.04							
Alt 1A	7,500	11,000	22,500	34,000	41,500	2,000	173,000
Alt 2A	7,500	11,000	23,000	33,500	41,500	2,000	170,500
Change from Alt 1A	0	0	500	-500	0	0	-2,500
Alt 3A	7,500	11,000	22,500	34,000	41,500	2,000	178,000
Change from Alt 1A	0	0	0	0	0	0	5,000
Alt 4A	7,500	11,000	22,500	34,000	42,000	2,000	179,000
Change from Alt 1A	0	0	0	0	500	0	6,000
Alt 5A	7,500	11,000	22,500	33,500	42,000	2,000	169,000
Change from Alt 1A	0	0	0	-500	500	0	-4,000
Alt 6A	7,500	11,000	22,500	34,000	42,000	2,000	180,000
Change from Alt 1A	0	0	0	0	500	0	7,000
Greenwood - XS 862.98							
Alt 1A	7,504	10,995	22,700	33,681	41,360	1,830	171,922
Alt 2A	7,718	11,088	23,213	33,295	41,168	1,935	167,528
Change from Alt 1A	214	94	513	-385	-192	104	-4,393
Alt 3A	7,506	11,003	22,676	33,647	41,423	1,830	174,840
Change from Alt 1A	2	8	-24	-34	63	0	2,918
Alt 4A	7,438	10,795	22,314	33,786	41,787	1,831	175,821
Change from Alt 1A	-66	-200	-386	106	428	0	3,899
Alt 5A	7,495	10,924	22,315	33,551	41,636	1,830	166,086
Change from Alt 1A	-9	-71	-385	-130	277	0	-5,836
Alt 6A	7,491	10,823	22,326	34,027	41,813	1,831	176,815
Change from Alt 1A	-13	-172	-374	346	454	0	4,893
Verdel - XS 844.78							
Alt 1A	7,601	11,025	22,720	33,662	41,230	1,548	171,296
Alt 2A	7,839	11,127	23,308	33,253	41,063	1,561	166,757
Change from Alt 1A	238	102	588	-409	-167	13	-4,539
Alt 3A	7,609	11,049	22,691	33,638	41,347	1,543	174,219
Change from Alt 1A	8	24	-28	-25	117	-5	2,924
Alt 4A	7,531	10,866	22,309	33,752	41,620	1,547	175,080
Change from Alt 1A	-70	-159	-411	90	390	-1	3,785
Alt 5A	7,580	11,002	22,346	33,535	41,489	1,541	165,531
Change from Alt 1A	-21	-23	-374	-127	259	-7	-5,765
Alt 6A	7,563	10,911	22,321	34,004	41,724	1,545	176,393
Change from Alt 1A	-38	-114	-398	341	494	-3	5,098

Plate 1: Alternative Flow Statistics from POR Duration

Flow (cfs)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Niobrara - XS 842.93							
Alt 1A	9,479	12,657	24,679	35,030	42,595	3,664	175,049
Alt 2A	9,679	12,781	25,102	34,761	42,417	3,663	169,669
Change from Alt 1A	200	125	423	-270	-177	-1	-5,380
Alt 3A	9,478	12,684	24,649	34,997	42,661	3,663	177,130
Change from Alt 1A	-1	28	-30	-33	66	-1	2,082
Alt 4A	9,394	12,447	24,285	35,092	43,025	3,664	178,721
Change from Alt 1A	-85	-209	-395	61	431	0	3,672
Alt 5A	9,446	12,612	24,320	34,873	42,791	3,663	168,851
Change from Alt 1A	-33	-45	-359	-157	196	-1	-6,198
Alt 6A	9,417	12,506	24,283	35,363	43,080	3,664	180,147
Change from Alt 1A	-62	-151	-397	332	485	0	5,099

Plate 1 cont'd: Alternative Flow Statistics from POR Duration

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Fort Randall - XS 879.04							
Alt 1A	1229.1	1230.3	1233.0	1235.0	1236.0	1226.4	1249.5
Alt 2A	1229.2	1230.3	1233.1	1234.9	1236.0	1226.5	1249.2
Change from Alt 1A	0.0	0.0	0.1	0.0	0.0	0.0	-0.2
Alt 3A	1229.1	1230.3	1233.0	1235.0	1236.0	1226.4	1249.7
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.3
Alt 4A	1229.1	1230.3	1232.9	1235.0	1236.1	1226.4	1249.8
Change from Alt 1A	0.0	0.0	-0.1	0.0	0.1	0.0	0.3
Alt 5A	1229.1	1230.3	1232.9	1235.0	1236.1	1226.4	1249.1
Change from Alt 1A	0.0	0.0	-0.1	0.0	0.0	0.0	-0.3
Alt 6A	1229.1	1230.3	1232.9	1235.0	1236.1	1226.5	1249.8
Change from Alt 1A	0.0	0.0	-0.1	0.1	0.1	0.0	0.4
Greenwood - XS 862.98							
Alt 1A	1222.2	1223.1	1225.7	1227.9	1229.1	1219.7	1240.2
Alt 2A	1222.2	1223.1	1225.8	1227.9	1229.1	1219.8	1239.9
Change from Alt 1A	0.1	0.0	0.1	-0.1	0.0	0.1	-0.3
Alt 3A	1222.2	1223.1	1225.7	1227.9	1229.2	1219.7	1240.3
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.1
Alt 4A	1222.2	1223.1	1225.7	1228.0	1229.2	1219.7	1240.3
Change from Alt 1A	0.0	-0.1	-0.1	0.0	0.1	0.0	0.1
Alt 5A	1222.2	1223.1	1225.7	1227.9	1229.2	1219.7	1239.9
Change from Alt 1A	0.0	0.0	-0.1	0.0	0.0	0.0	-0.3
Alt 6A	1222.2	1223.1	1225.7	1228.0	1229.2	1219.7	1240.4
Change from Alt 1A	0.0	0.0	-0.1	0.1	0.1	0.0	0.2
Verdel - XS 844.78							
Alt 1A	1215.0	1216.1	1219.1	1220.8	1221.7	1212.1	1229.6
Alt 2A	1215.0	1216.1	1219.1	1220.7	1221.7	1212.1	1229.4
Change from Alt 1A	0.1	0.0	0.1	0.0	0.0	0.0	-0.2
Alt 3A	1215.0	1216.1	1219.1	1220.8	1221.7	1212.1	1229.7
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.1
Alt 4A	1214.9	1216.0	1219.0	1220.8	1221.8	1212.1	1229.8
Change from Alt 1A	0.0	-0.1	-0.1	0.0	0.0	0.0	0.2
Alt 5A	1215.0	1216.1	1219.0	1220.8	1221.7	1212.1	1229.4
Change from Alt 1A	0.0	0.0	-0.1	0.0	0.0	0.0	-0.3
Alt 6A	1214.9	1216.1	1219.0	1220.8	1221.8	1212.1	1229.8
Change from Alt 1A	0.0	0.0	-0.1	0.0	0.1	0.0	0.2

Plate 2: Alternative Water Surface Elevation Statistics from POR Duration

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Niobrara - XS 842.93							
Alt 1A	1214.0	1215.1	1218.0	1219.6	1220.5	1210.8	1228.2
Alt 2A	1214.1	1215.2	1218.1	1219.6	1220.5	1210.8	1227.9
Change from Alt 1A	0.1	0.0	0.1	0.0	0.0	0.0	-0.2
Alt 3A	1214.0	1215.1	1218.0	1219.6	1220.5	1210.8	1228.2
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.1
Alt 4A	1214.0	1215.1	1217.9	1219.6	1220.5	1210.8	1228.3
Change from Alt 1A	0.0	-0.1	-0.1	0.0	0.0	0.0	0.1
Alt 5A	1214.0	1215.1	1217.9	1219.6	1220.5	1210.8	1227.9
Change from Alt 1A	0.0	0.0	-0.1	0.0	0.0	0.0	-0.3
Alt 6A	1214.0	1215.1	1217.9	1219.6	1220.5	1210.8	1228.4
Change from Alt 1A	0.0	-0.1	-0.1	0.0	0.1	0.0	0.2
Springfield - XS 832.43							
Alt 1A	1207.3	1207.7	1208.3	1208.6	1209.1	1204.7	1215.3
Alt 2A	1207.3	1207.7	1208.3	1208.7	1209.1	1205.7	1215.1
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	1.0	-0.2
Alt 3A	1207.3	1207.7	1208.3	1208.6	1209.1	1204.7	1215.4
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.1
Alt 4A	1207.3	1207.7	1208.3	1208.6	1209.1	1204.7	1215.4
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.1
Alt 5A	1207.3	1207.7	1208.3	1208.6	1209.1	1204.7	1215.1
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	-0.2
Alt 6A	1207.3	1207.7	1208.3	1208.6	1209.1	1205.7	1215.5
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	1.0	0.2
Lewis & Clark Lake - XS 812.74							
Alt 1A	1,206.6	1,206.8	1,207.3	1,208.2	1,208.4	1,202.0	1,212.7
Alt 2A	1,206.6	1,206.8	1,207.3	1,208.2	1,208.4	1,202.0	1,212.7
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Alt 3A	1,206.6	1,206.8	1,207.3	1,208.2	1,208.4	1,201.9	1,212.7
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	-0.1	0.0
Alt 4A	1,206.6	1,206.8	1,207.3	1,208.2	1,208.4	1,201.9	1,212.7
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	-0.1	0.0
Alt 5A	1,206.6	1,206.8	1,207.3	1,208.2	1,208.4	1,201.9	1,212.7
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	-0.1	0.0
Alt 6A	1,206.6	1,206.8	1,207.3	1,208.2	1,208.4	1,201.7	1,212.7
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	-0.3	0.0

Plate 2 cont'd: Alternative Water Surface Elevation Statistics from POR Duration

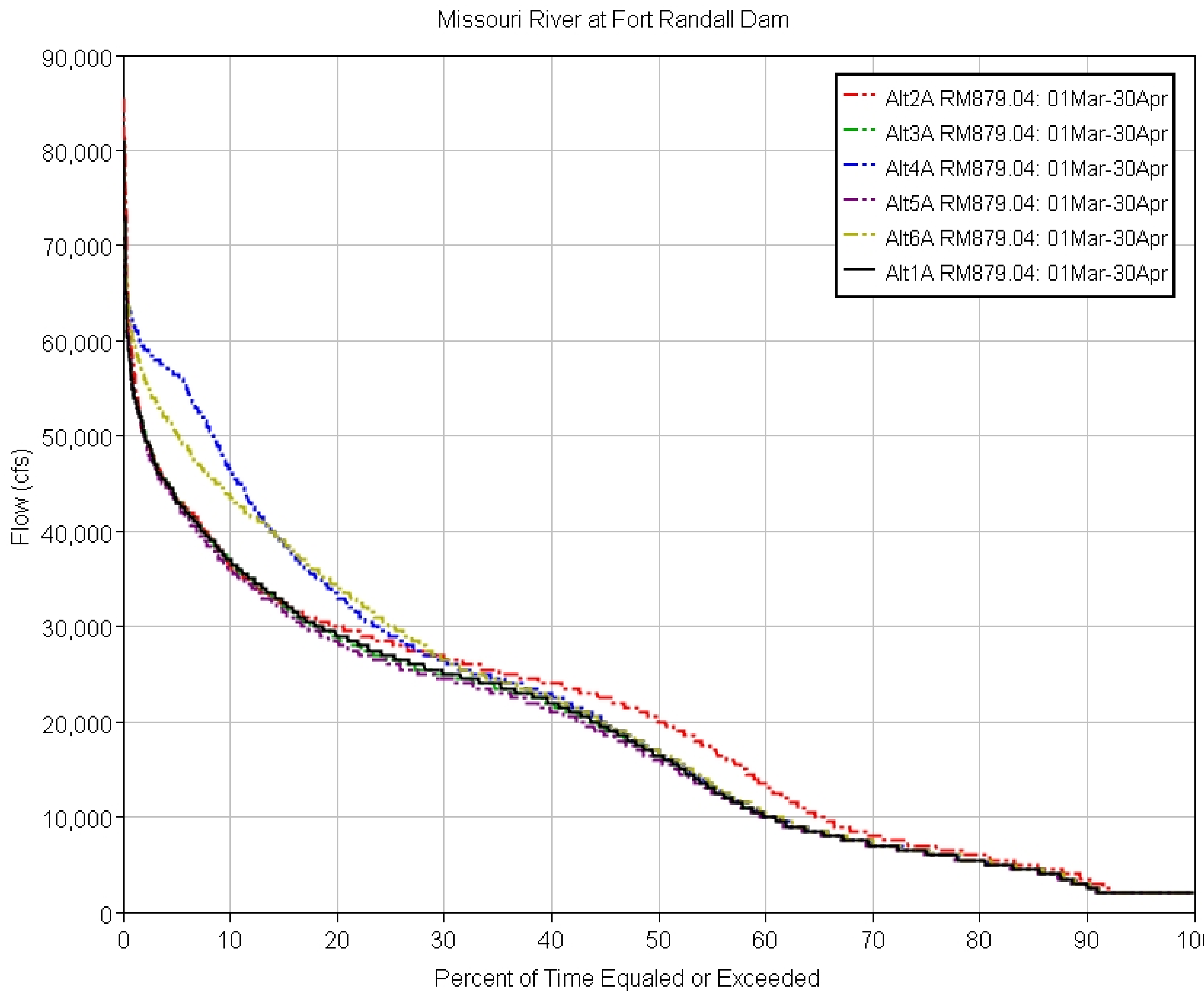


Plate 3: Fort Randall Spring Duration

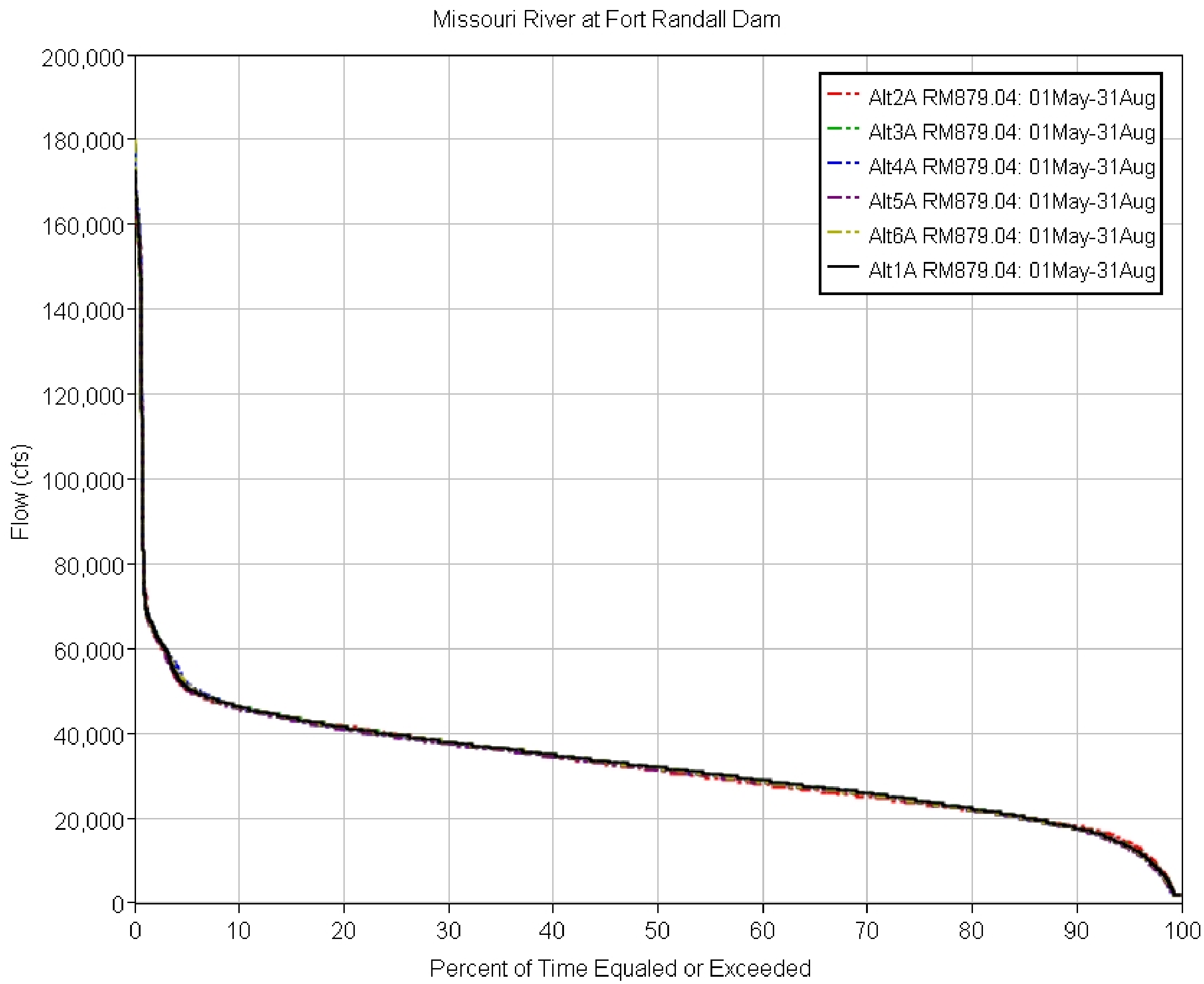


Plate 4: Fort Randall Summer Duration

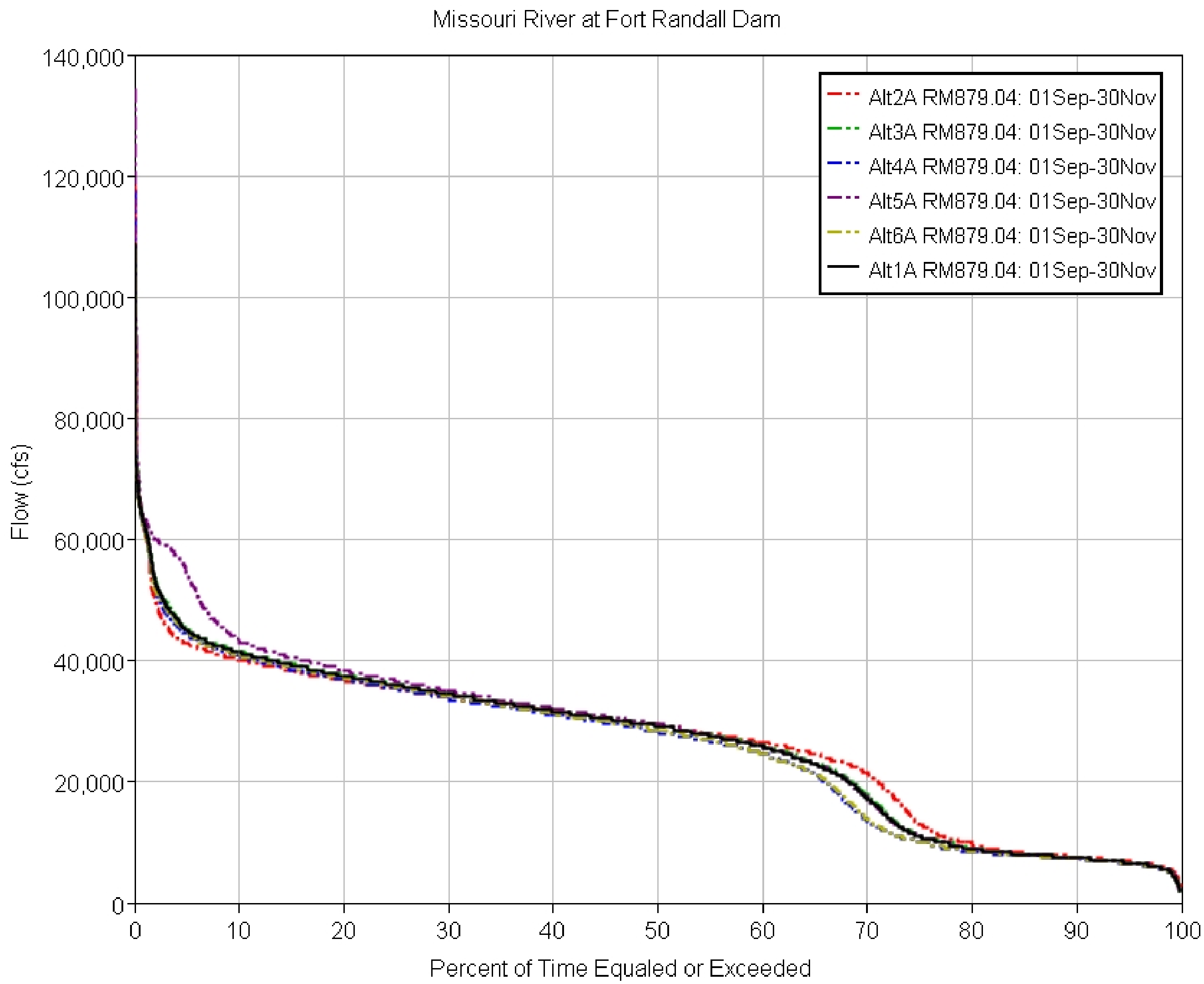


Plate 5: Fort Randall Fall Duration

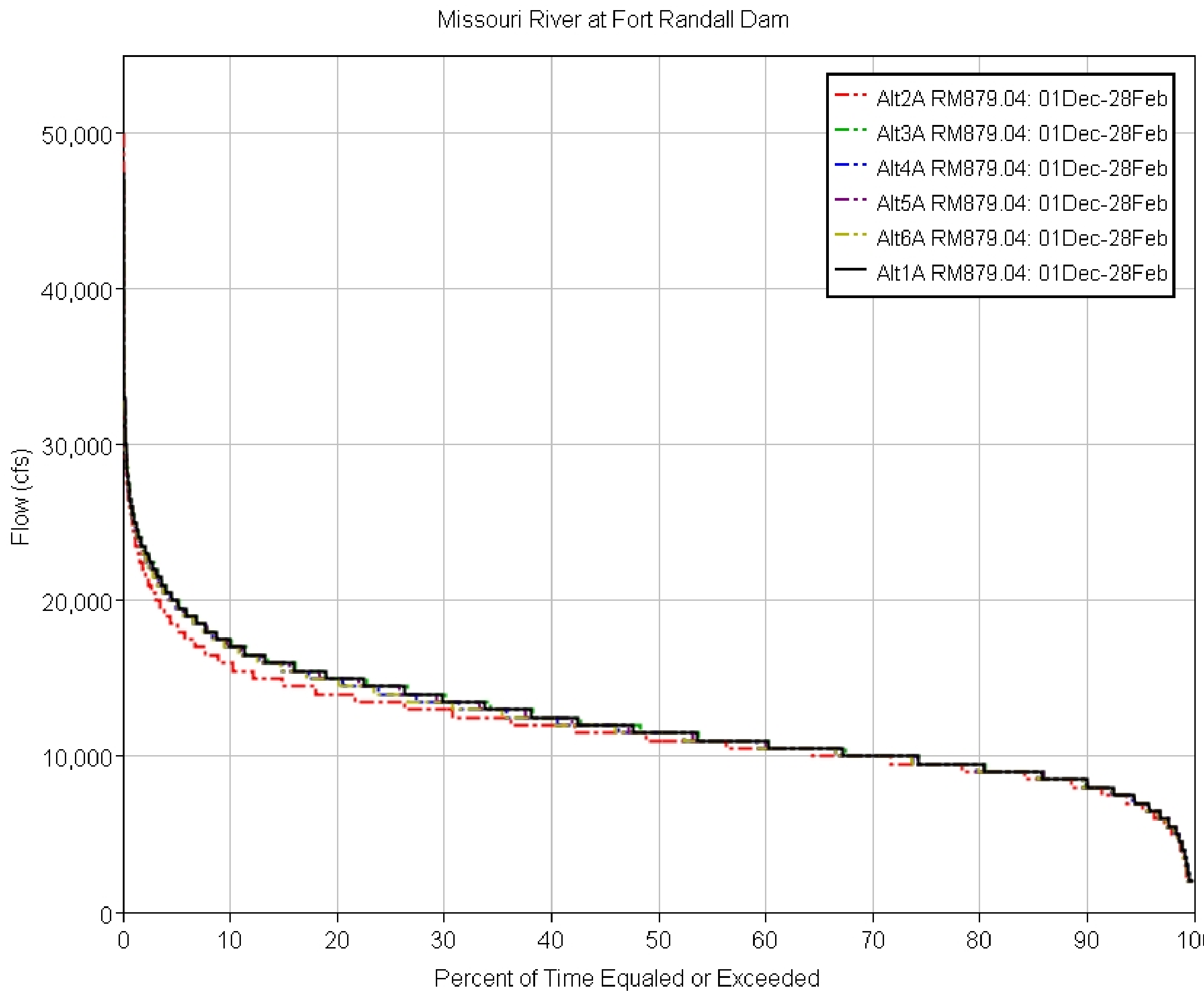


Plate 6: Fort Randall Winter Duration

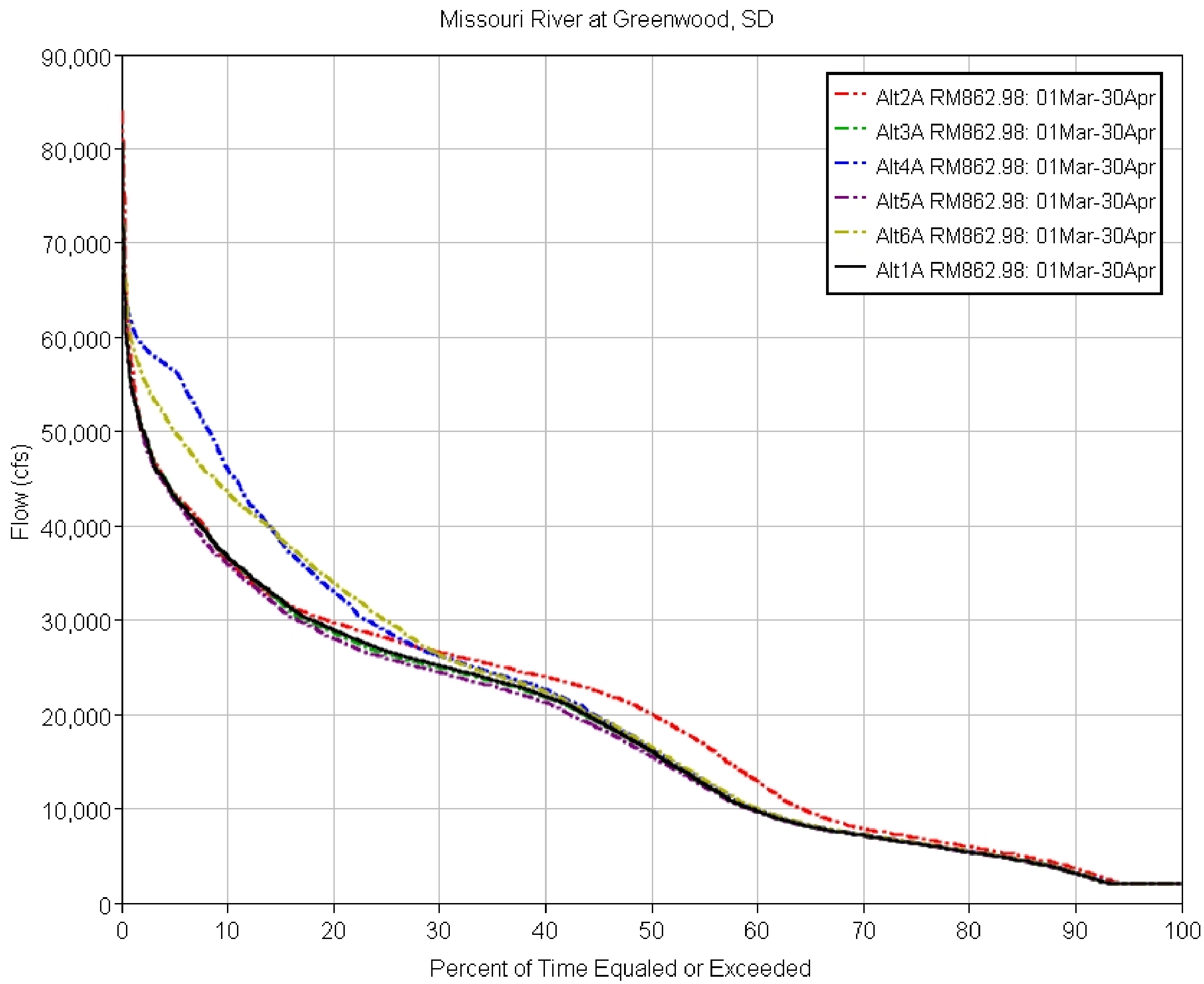


Plate 7: Greenwood Spring Duration

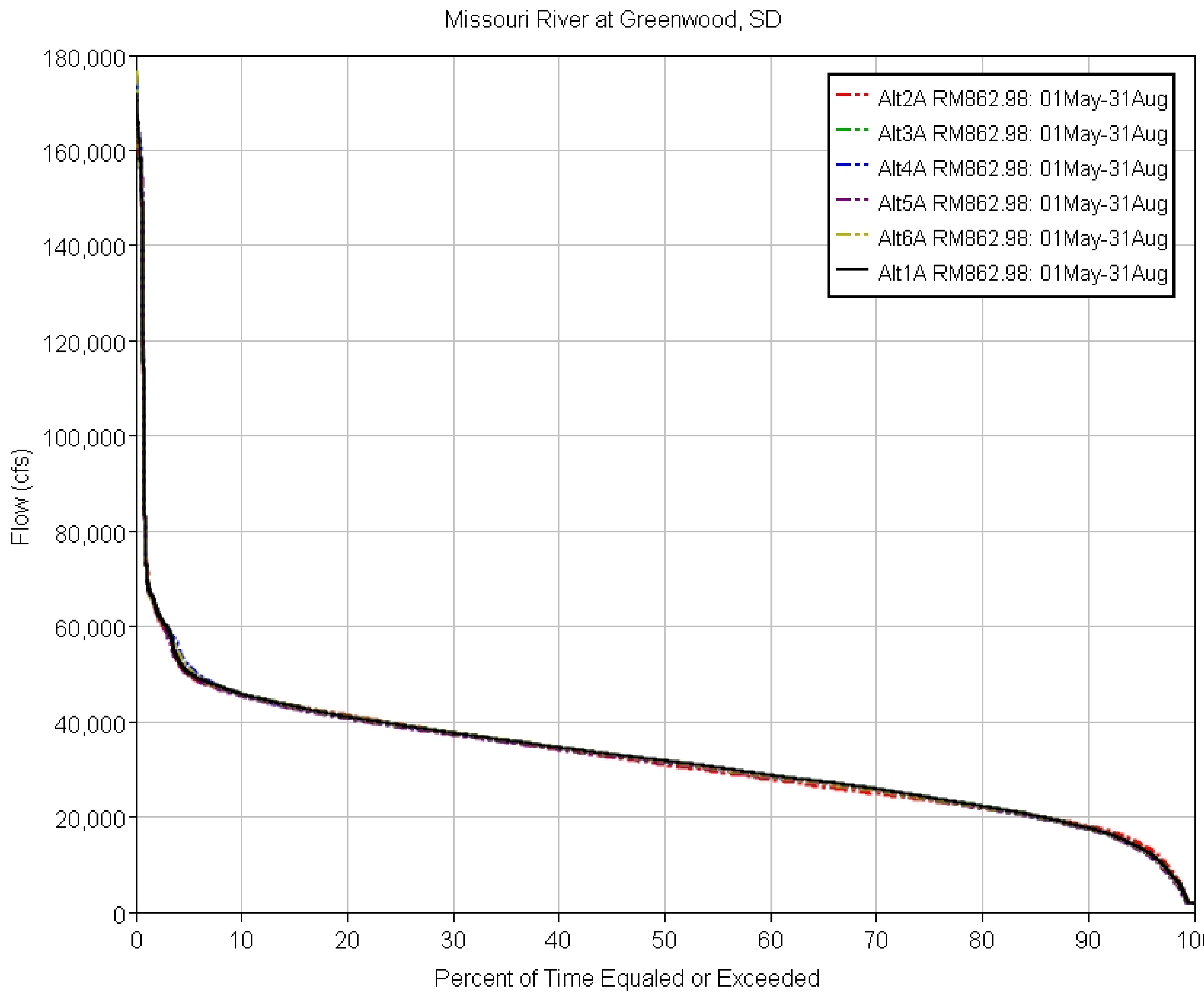


Plate 8: Greenwood Summer Duration

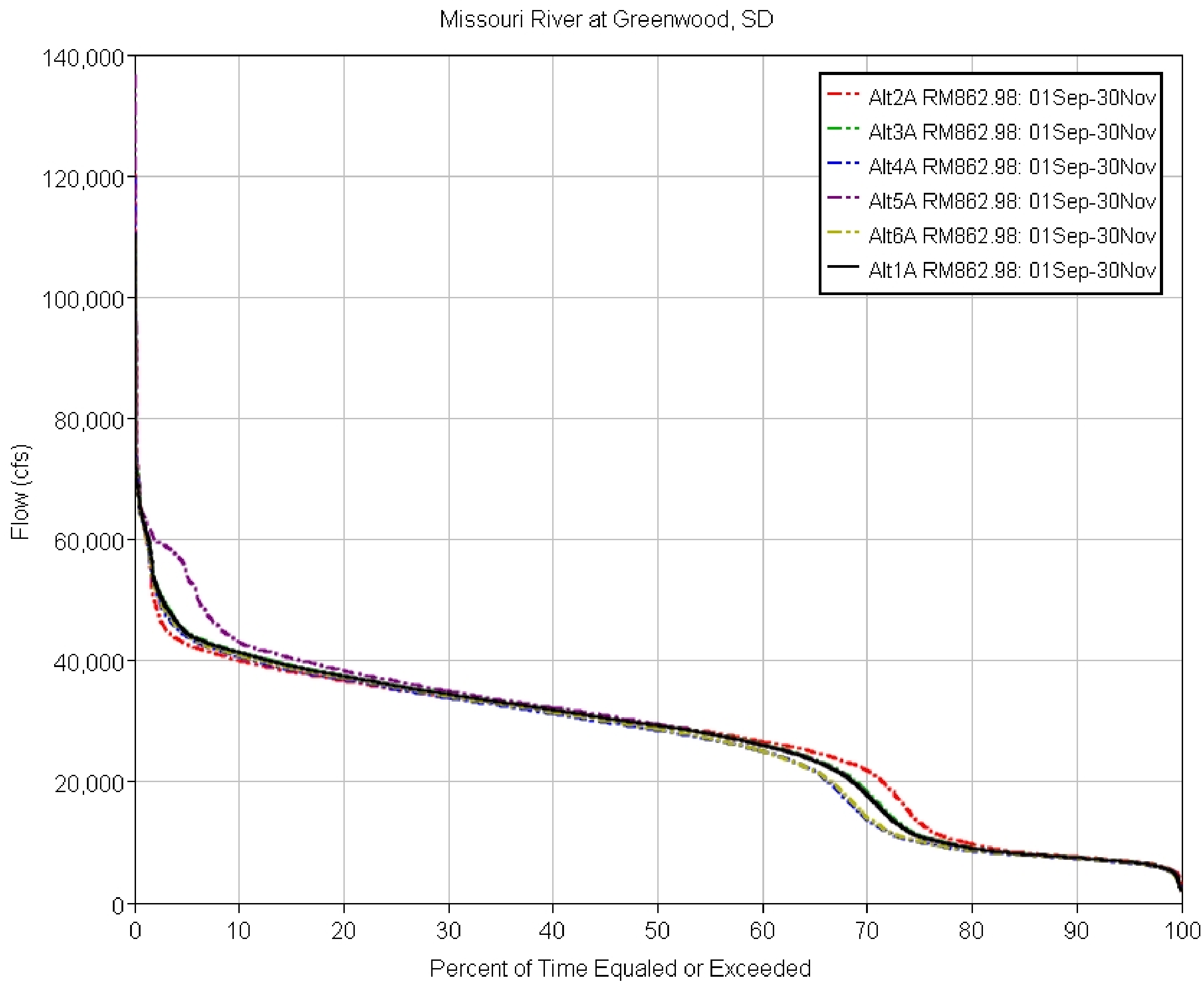


Plate 9: Greenwood Fall Duration

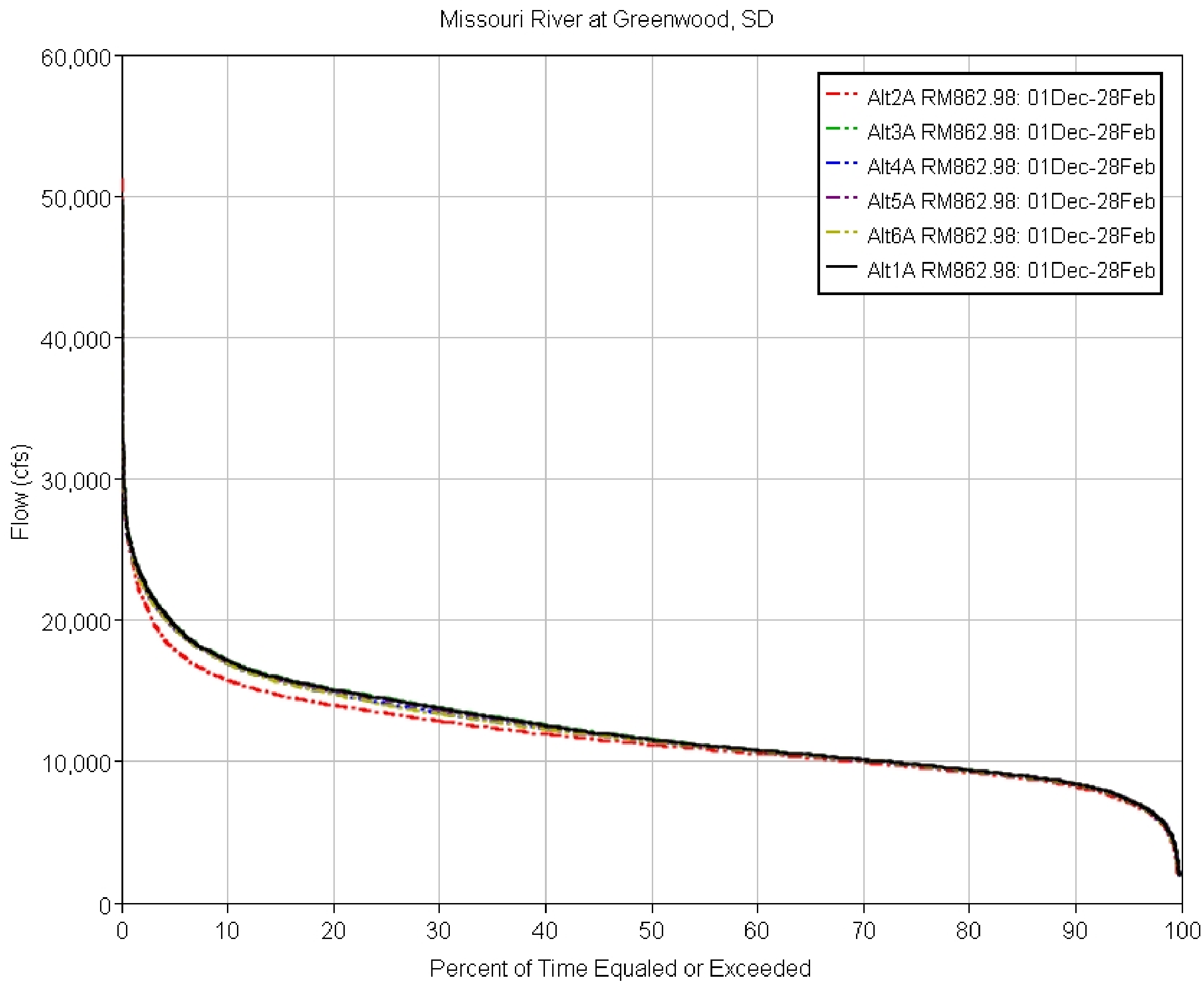


Plate 10: Greenwood Winter Duration

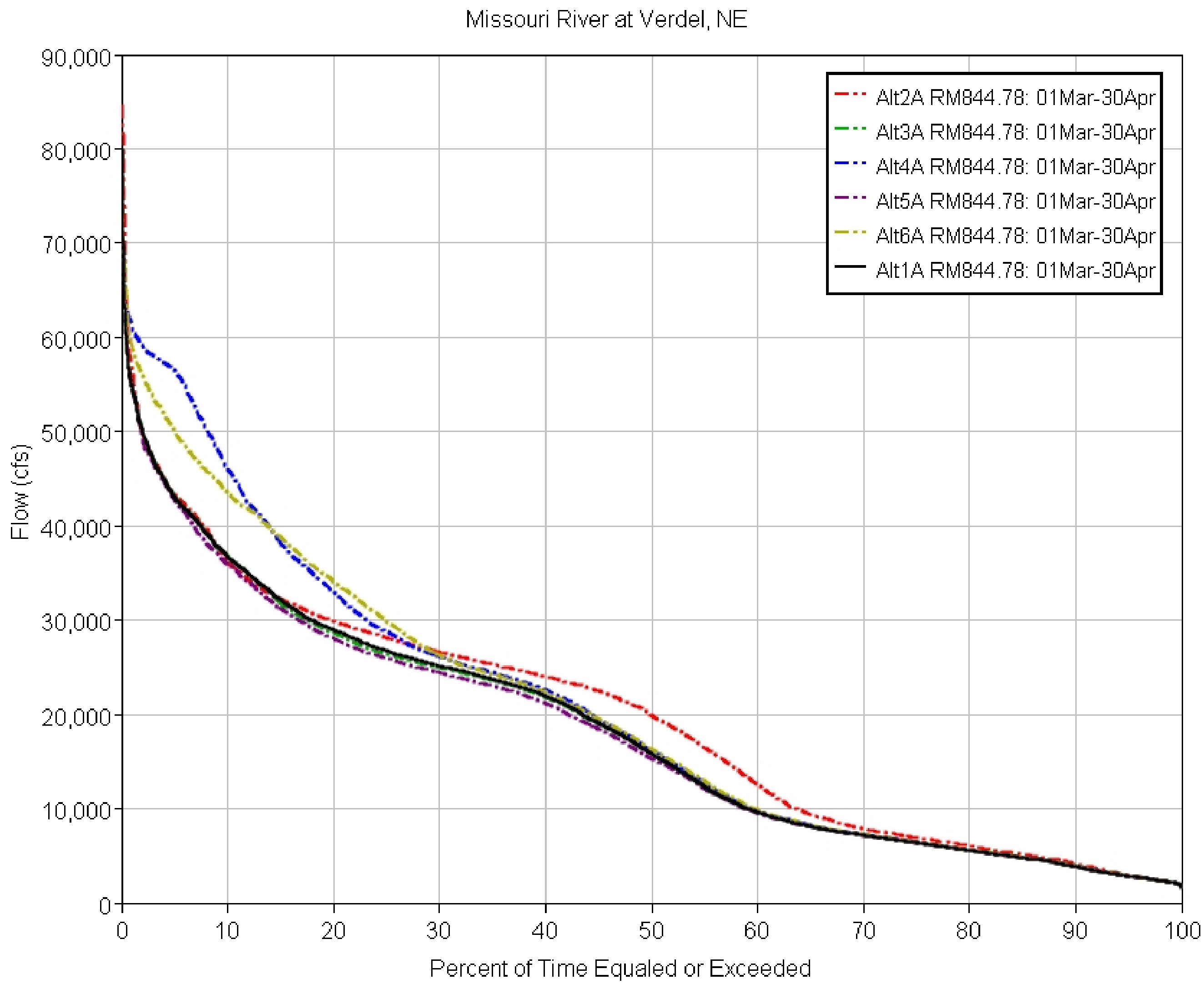


Plate 11: Verdel Spring Duration

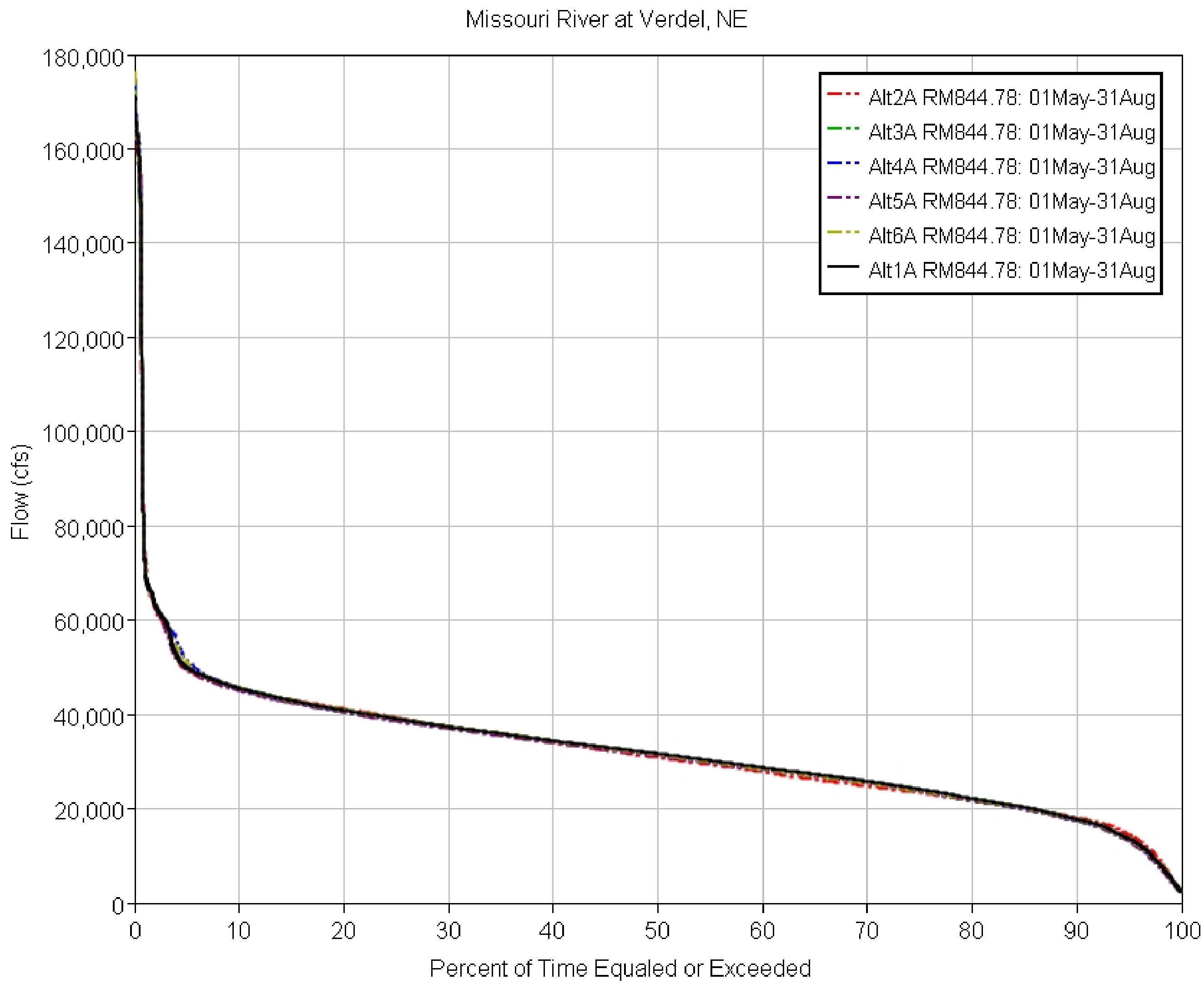


Plate 12: Verdel Summer Duration

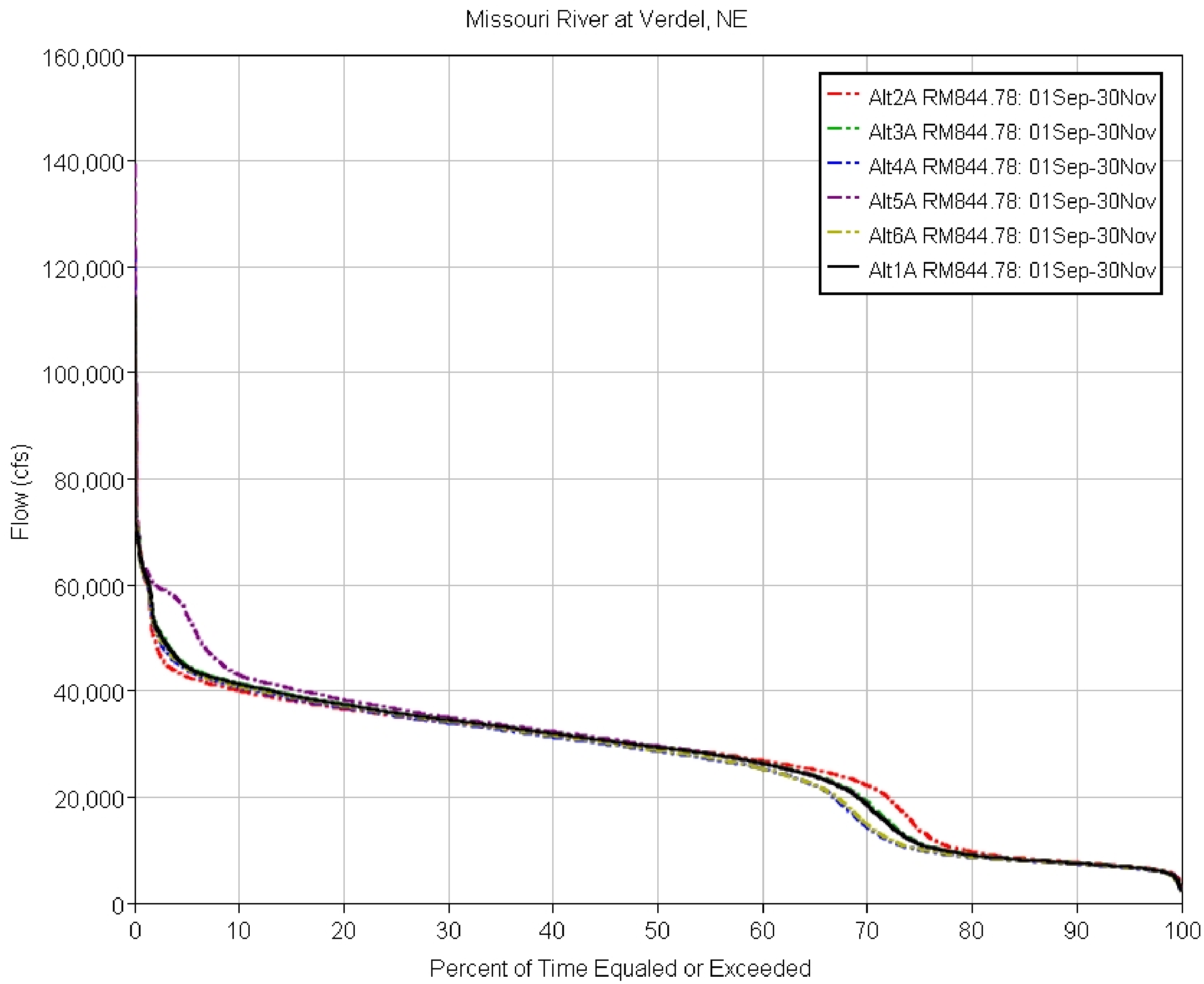


Plate 13: Verdel Fall Duration

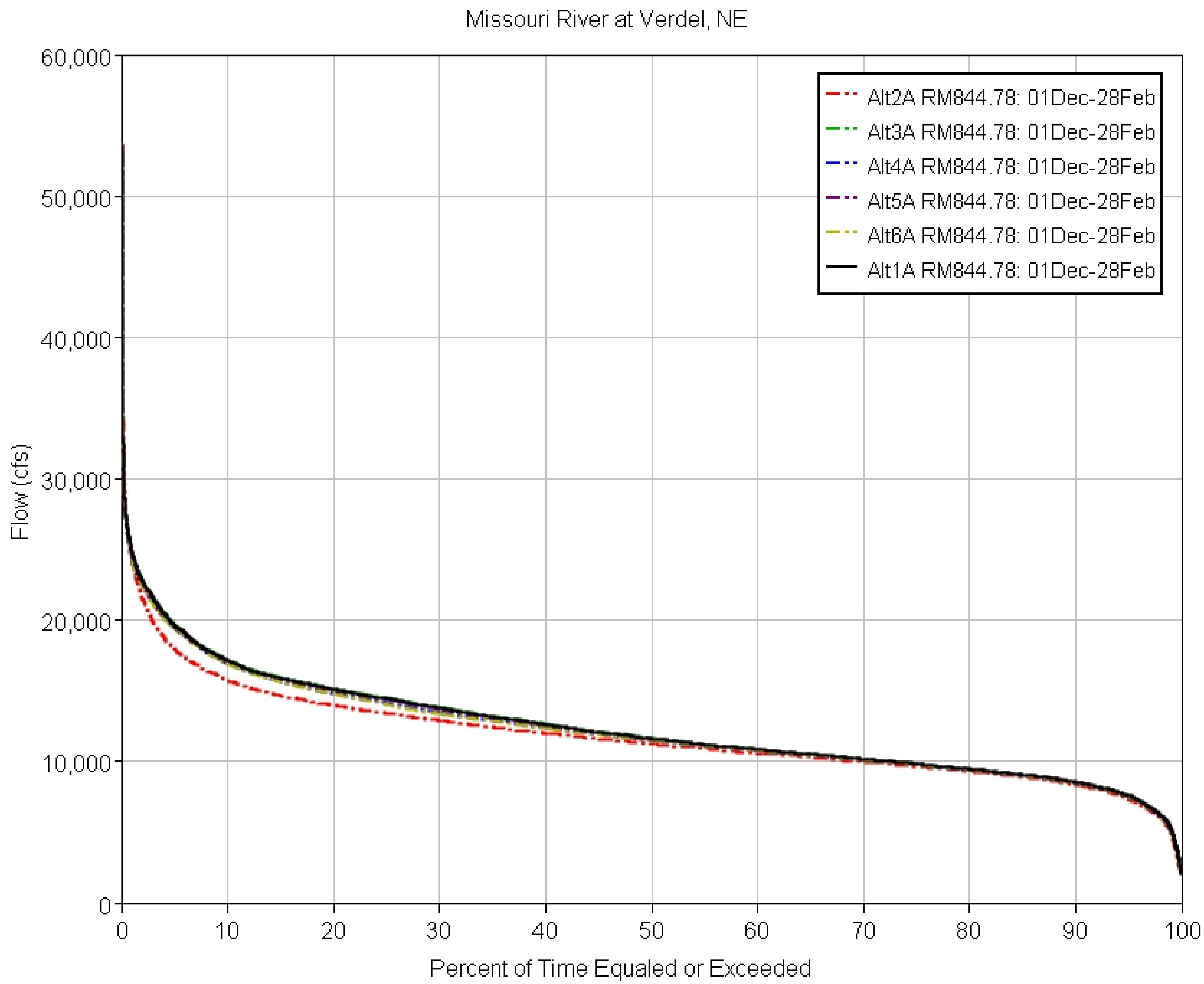


Plate 14: Verdel Winter Duration

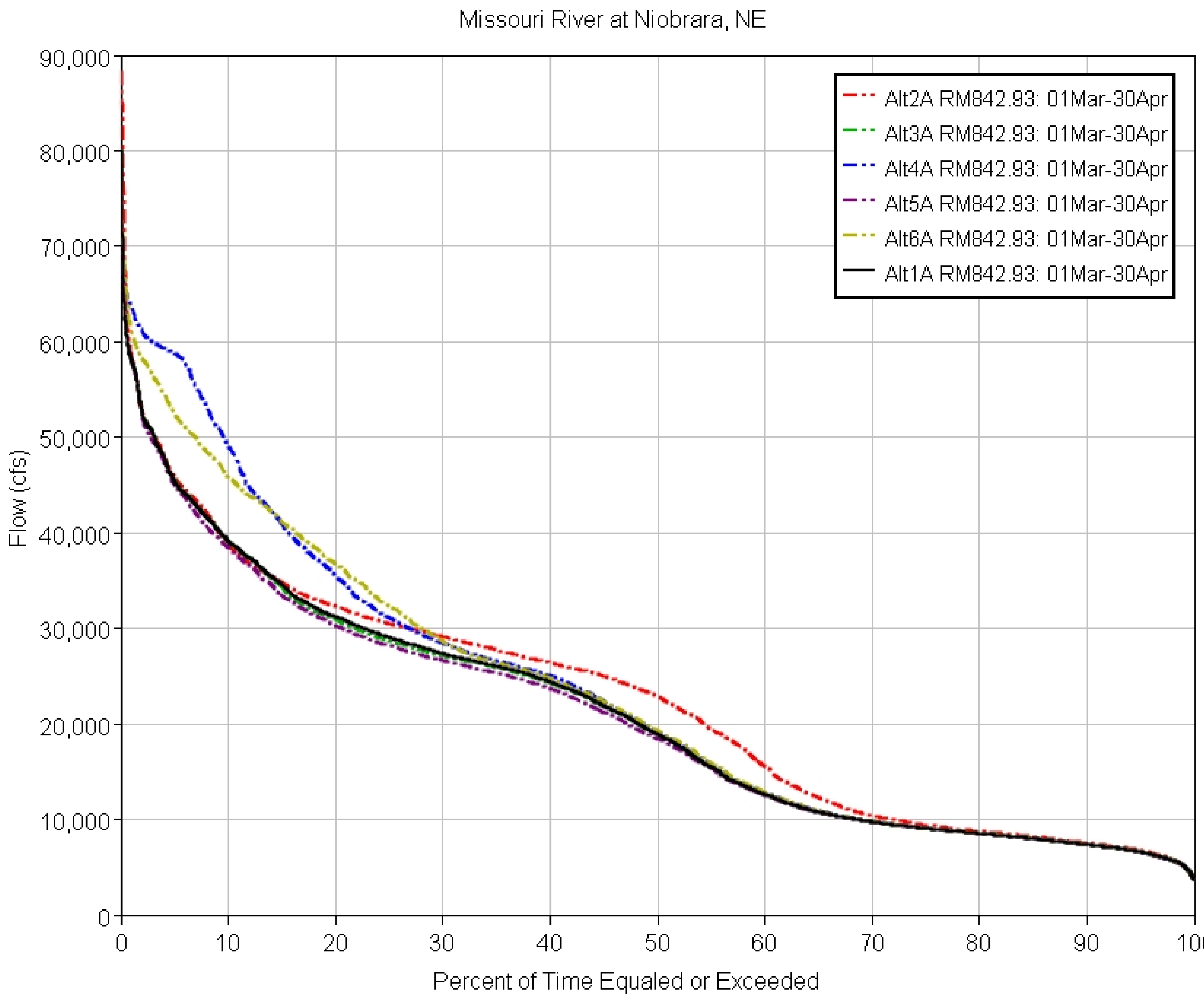


Plate 15: Niobrara Spring Duration

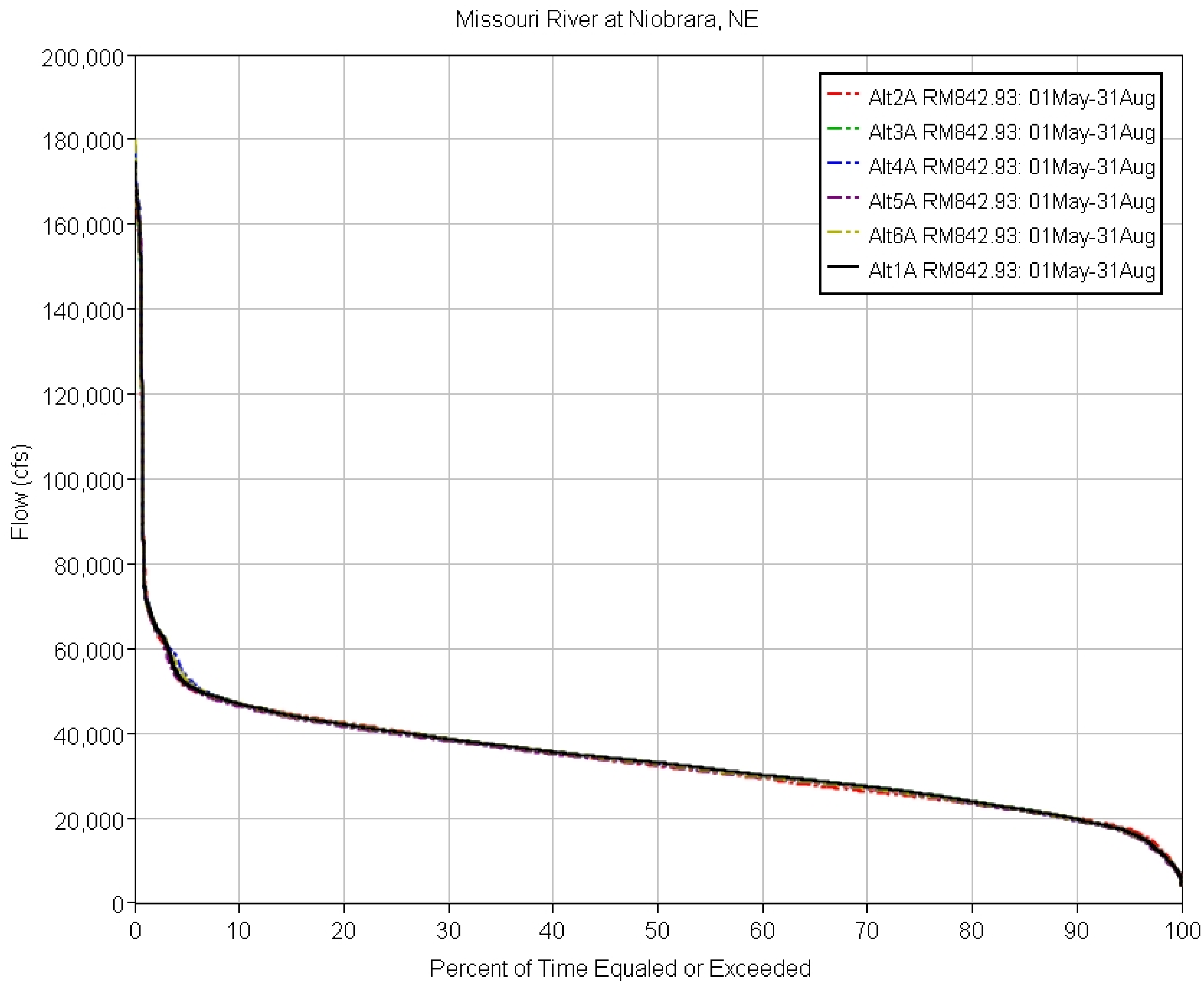


Plate 16: Niobrara Summer Duration

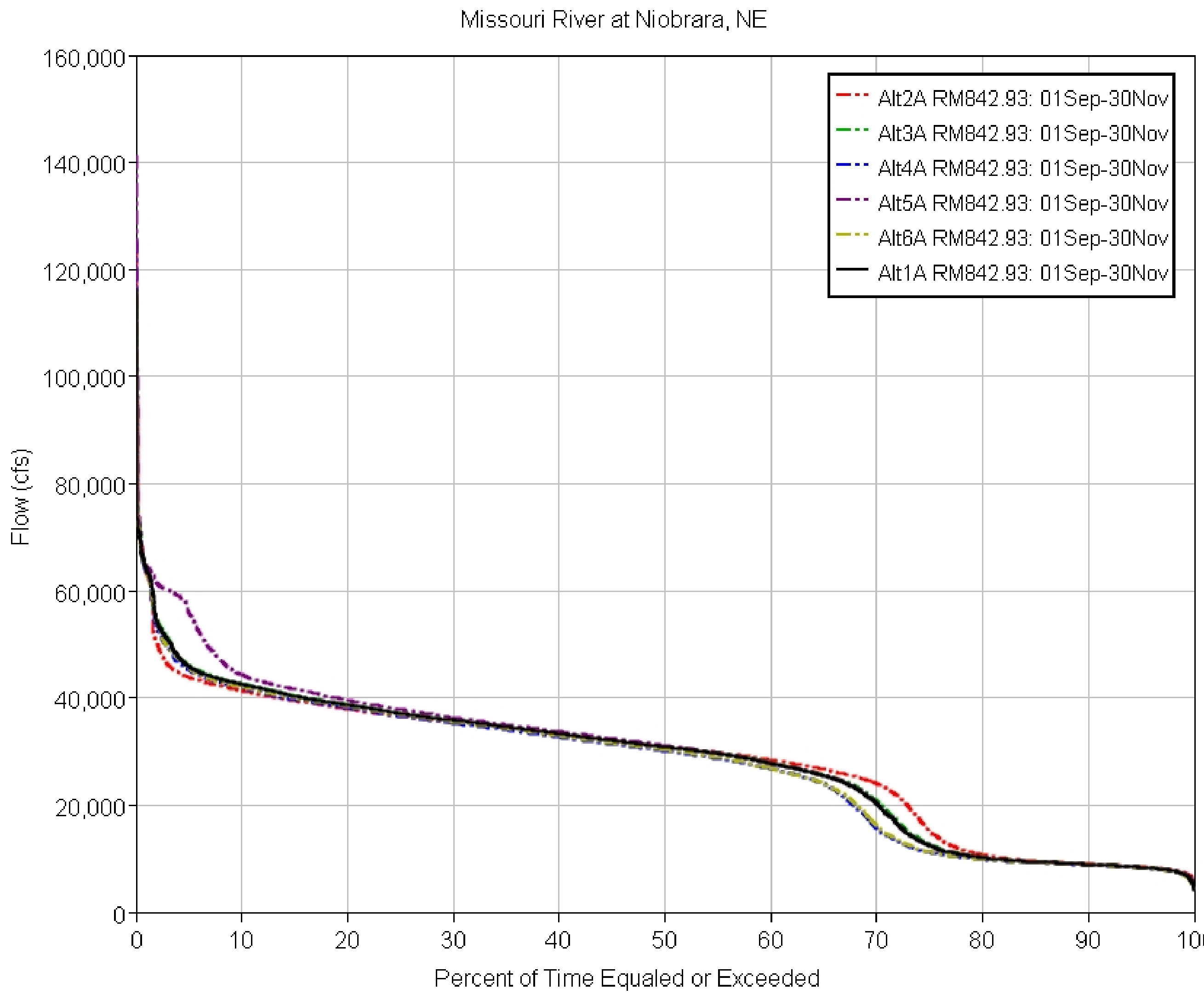


Plate 17: Niobrara Fall Duration

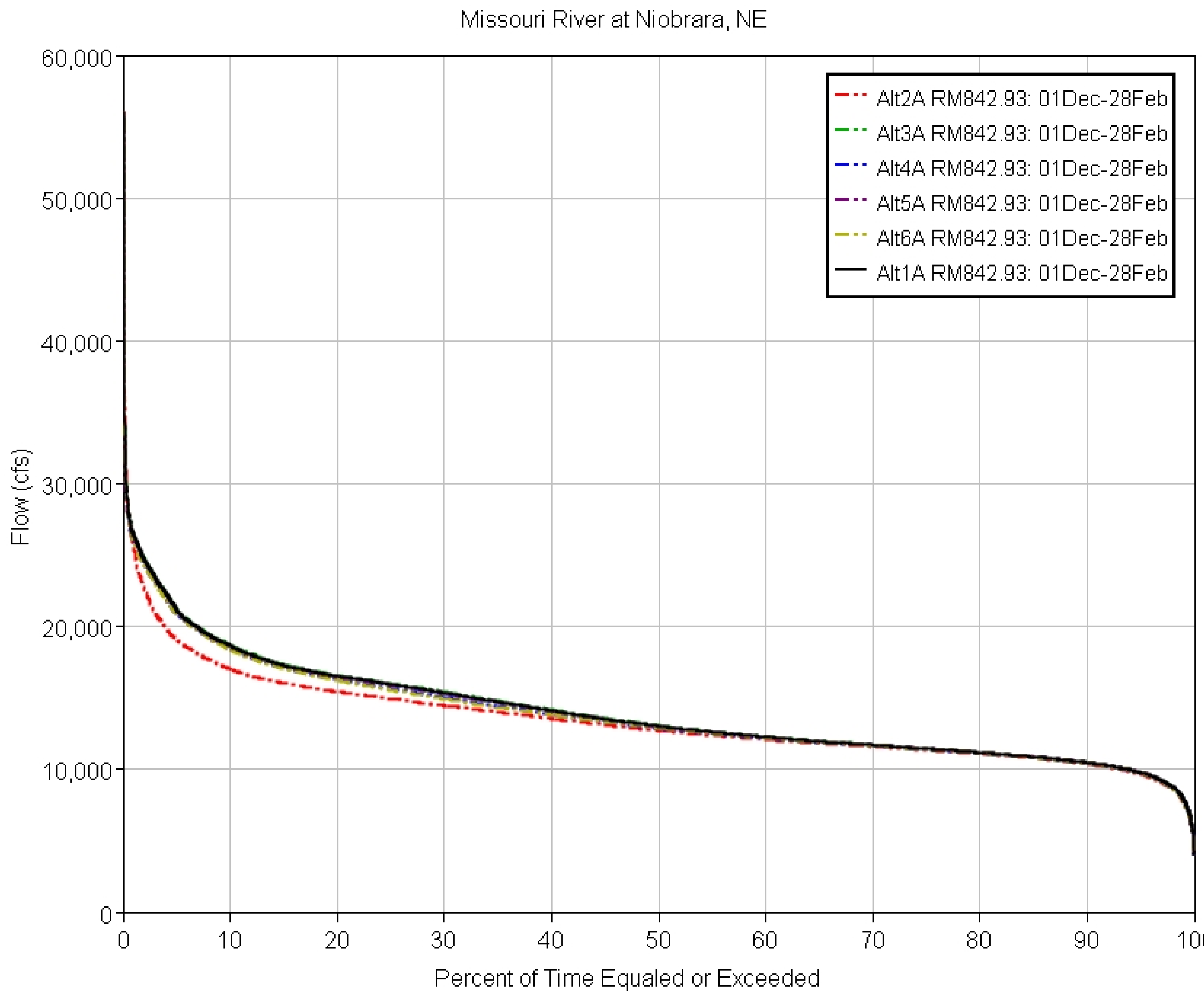


Plate 18: Niobrara Winter Duration

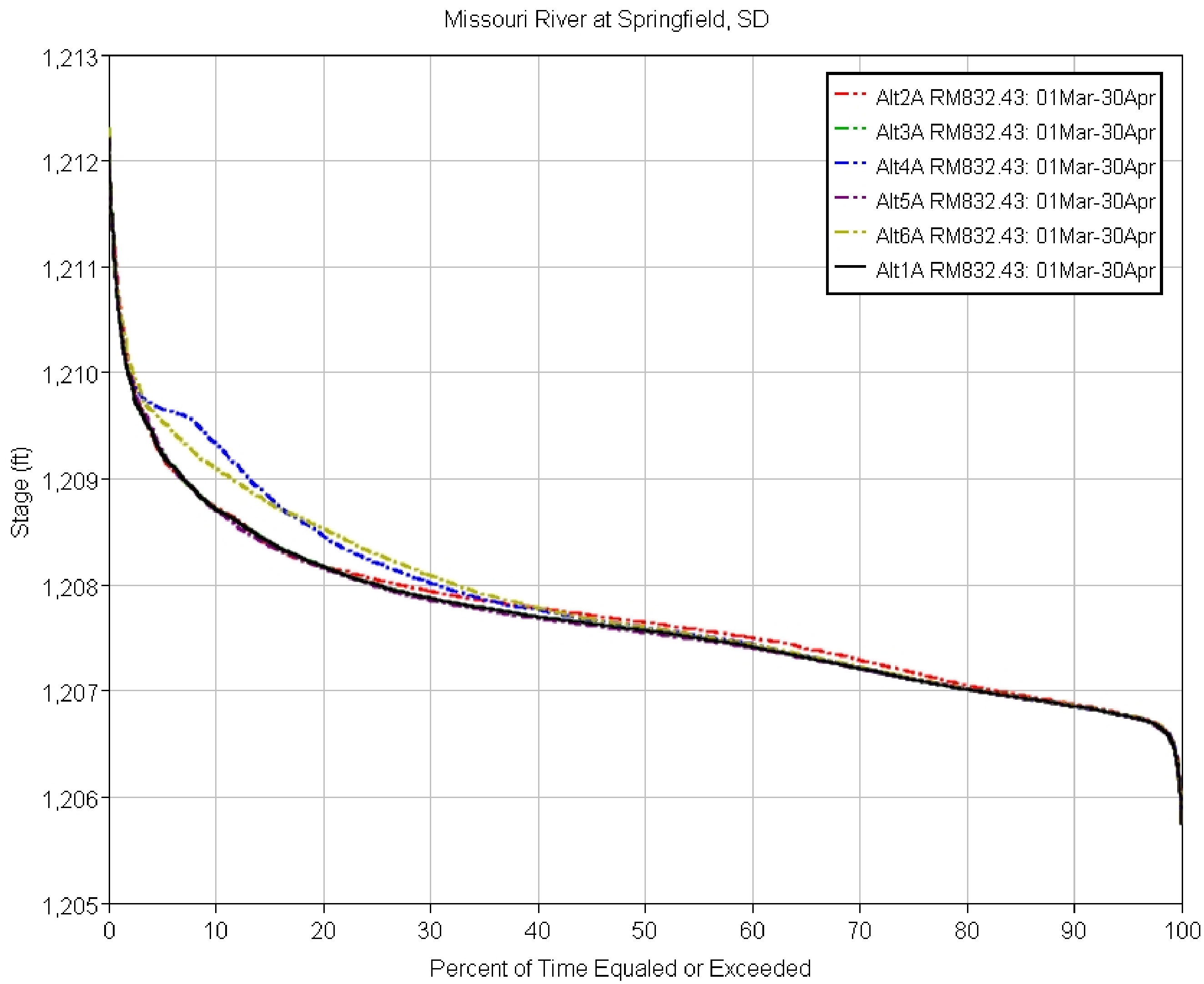


Plate 19: Springfield Spring Duration

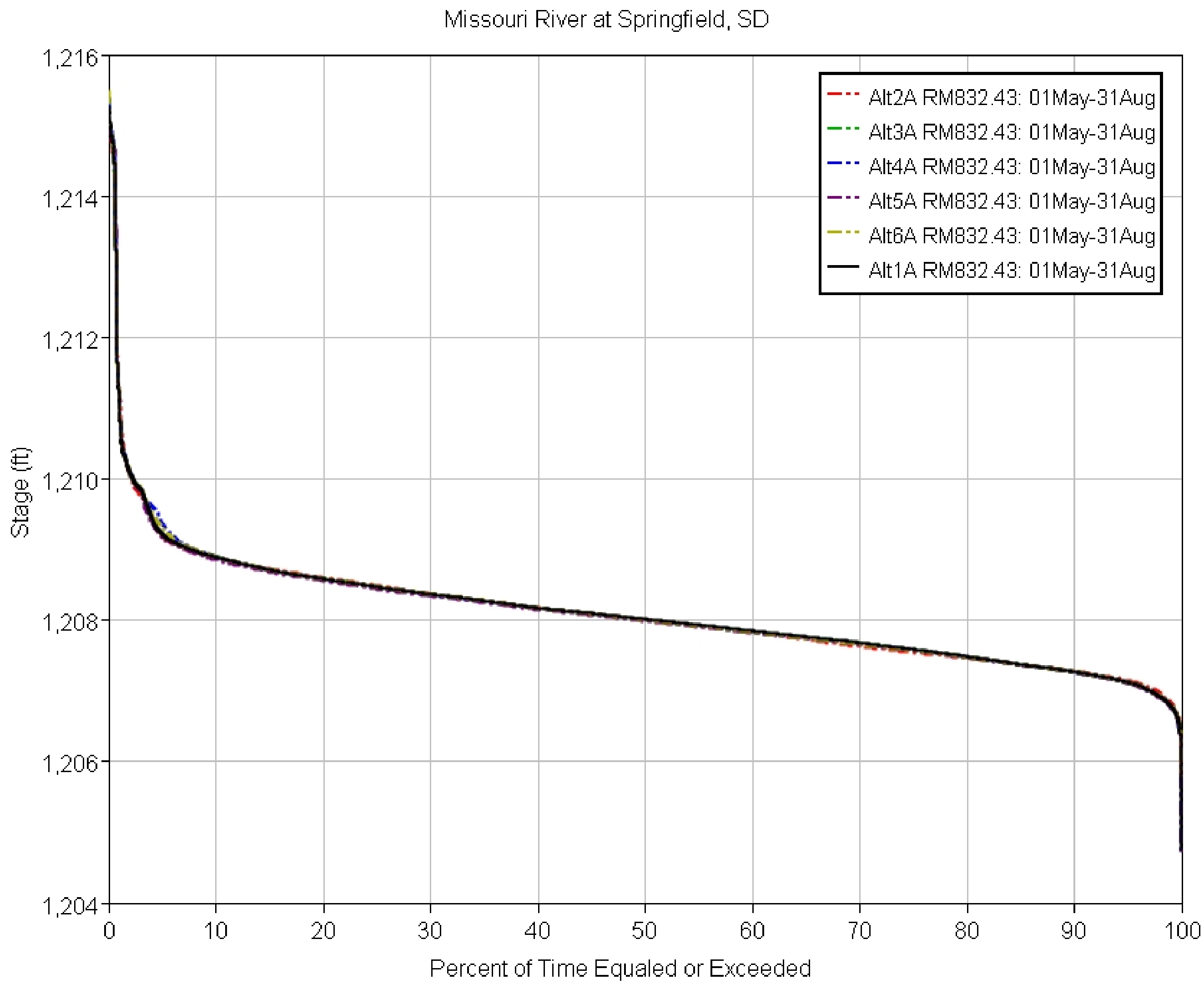


Plate 20: Springfield Summer Duration

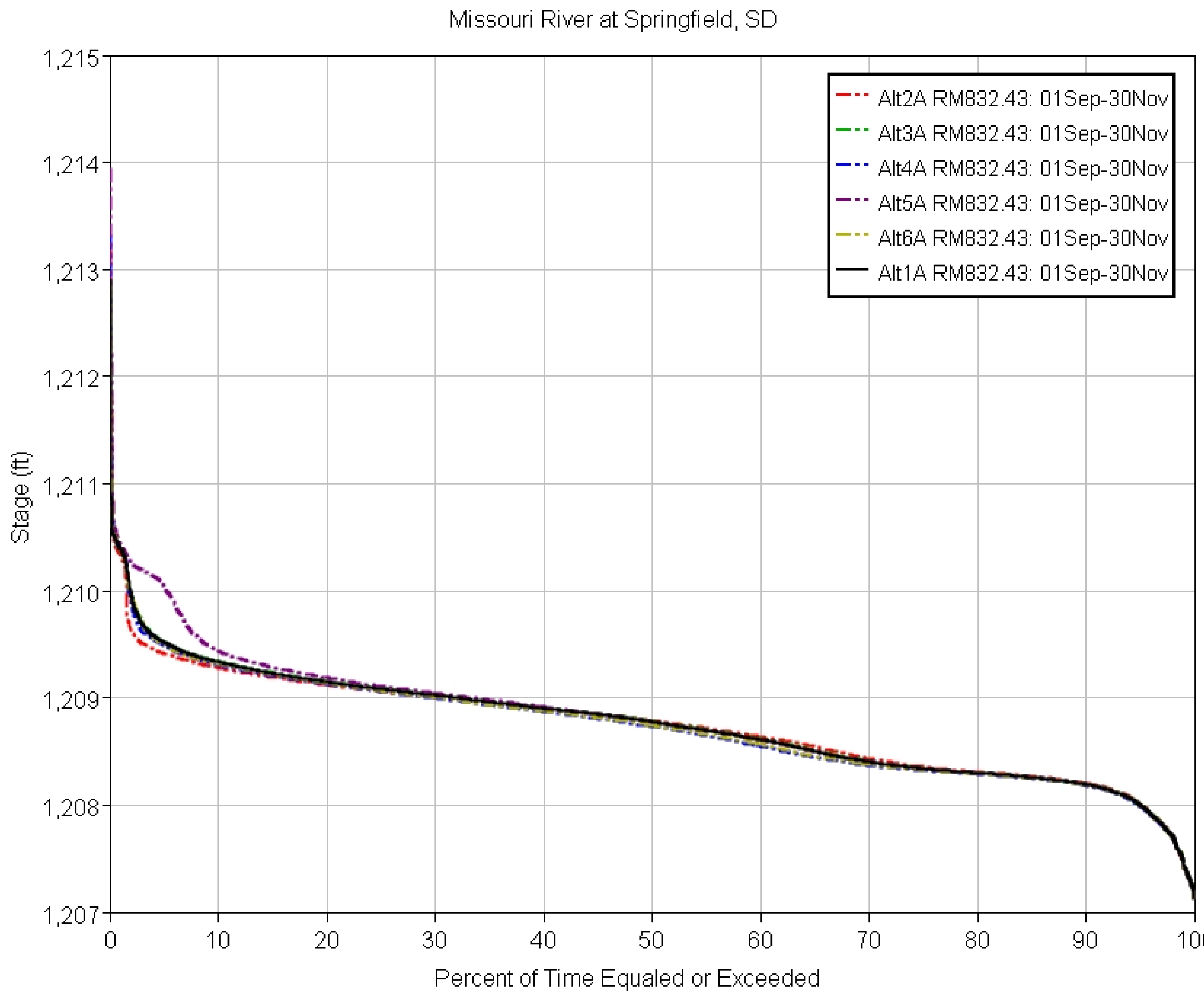


Plate 21: Springfield Fall Duration

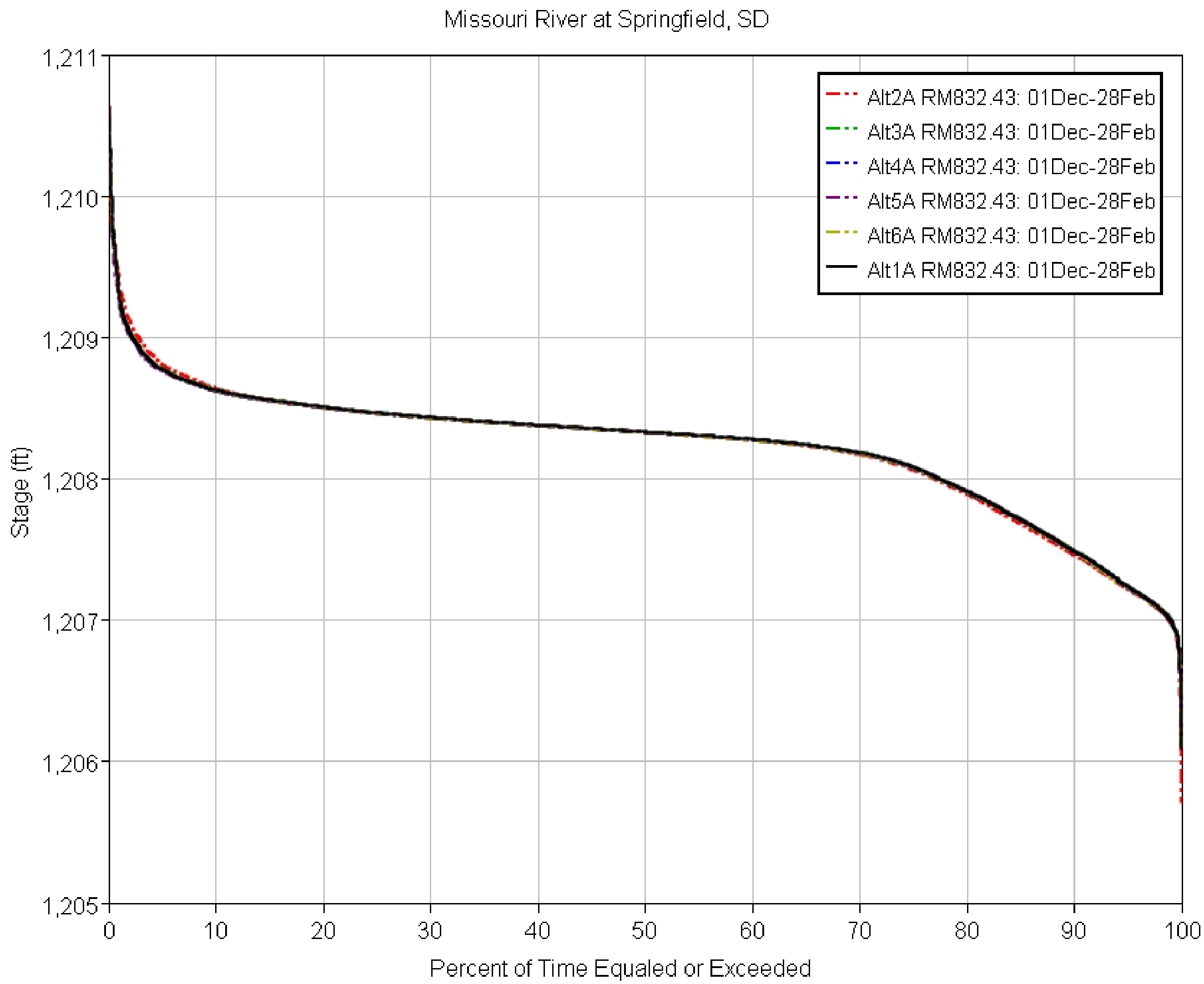


Plate 22: Springfield Winter Duration

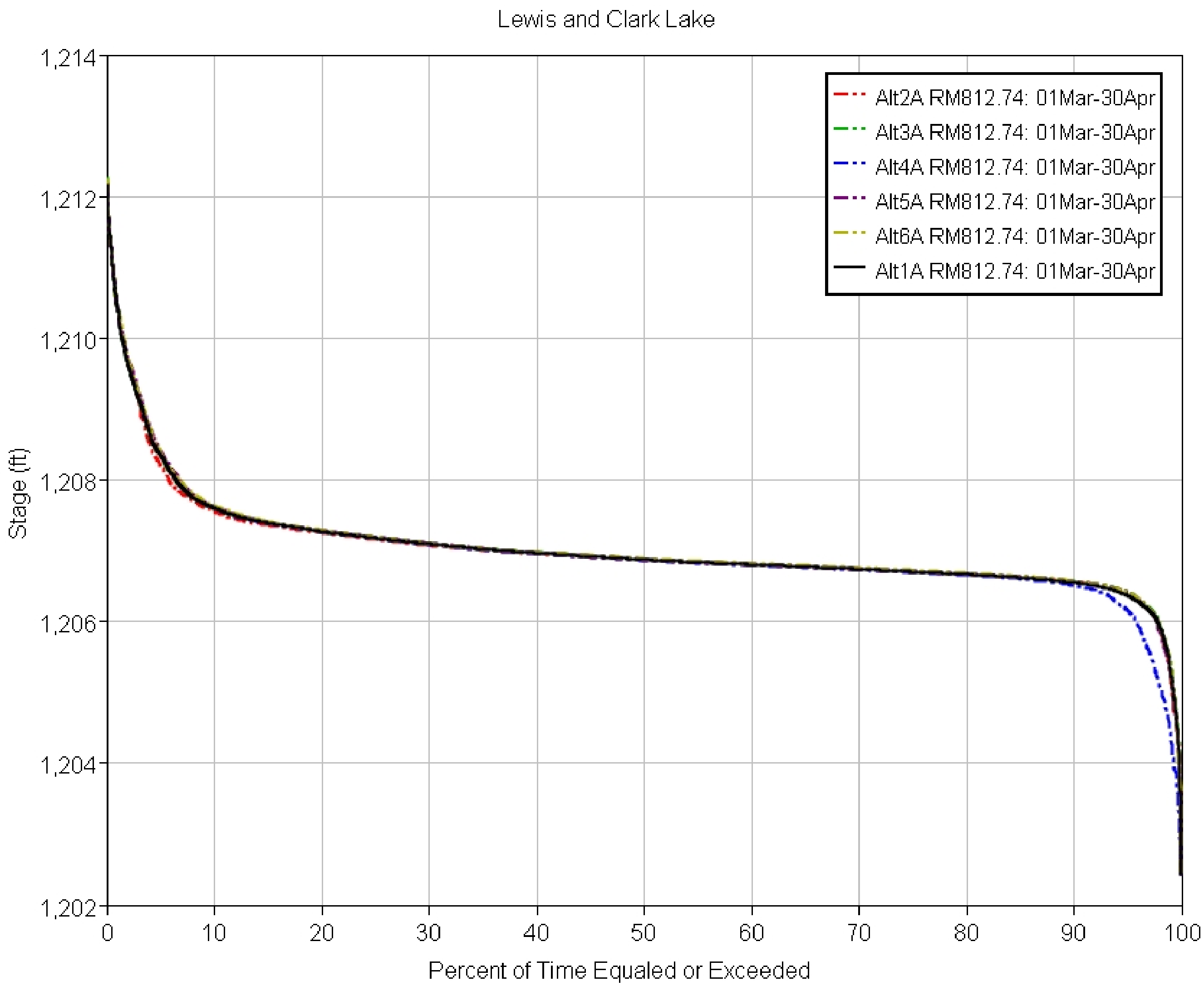


Plate 23: Lewis and Clark Lake Spring Duration

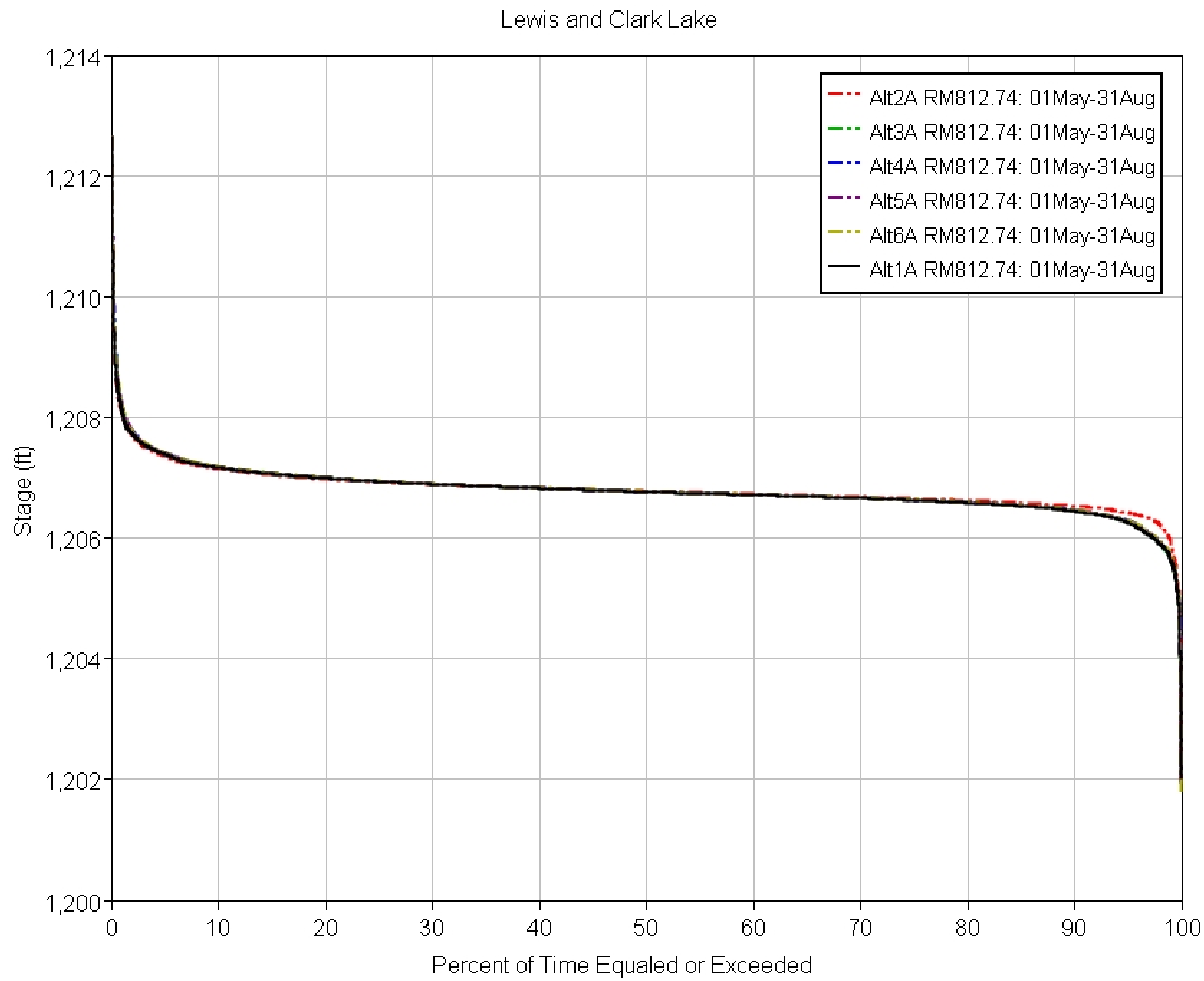


Plate 24: Lewis and Clark Lake Summer Duration

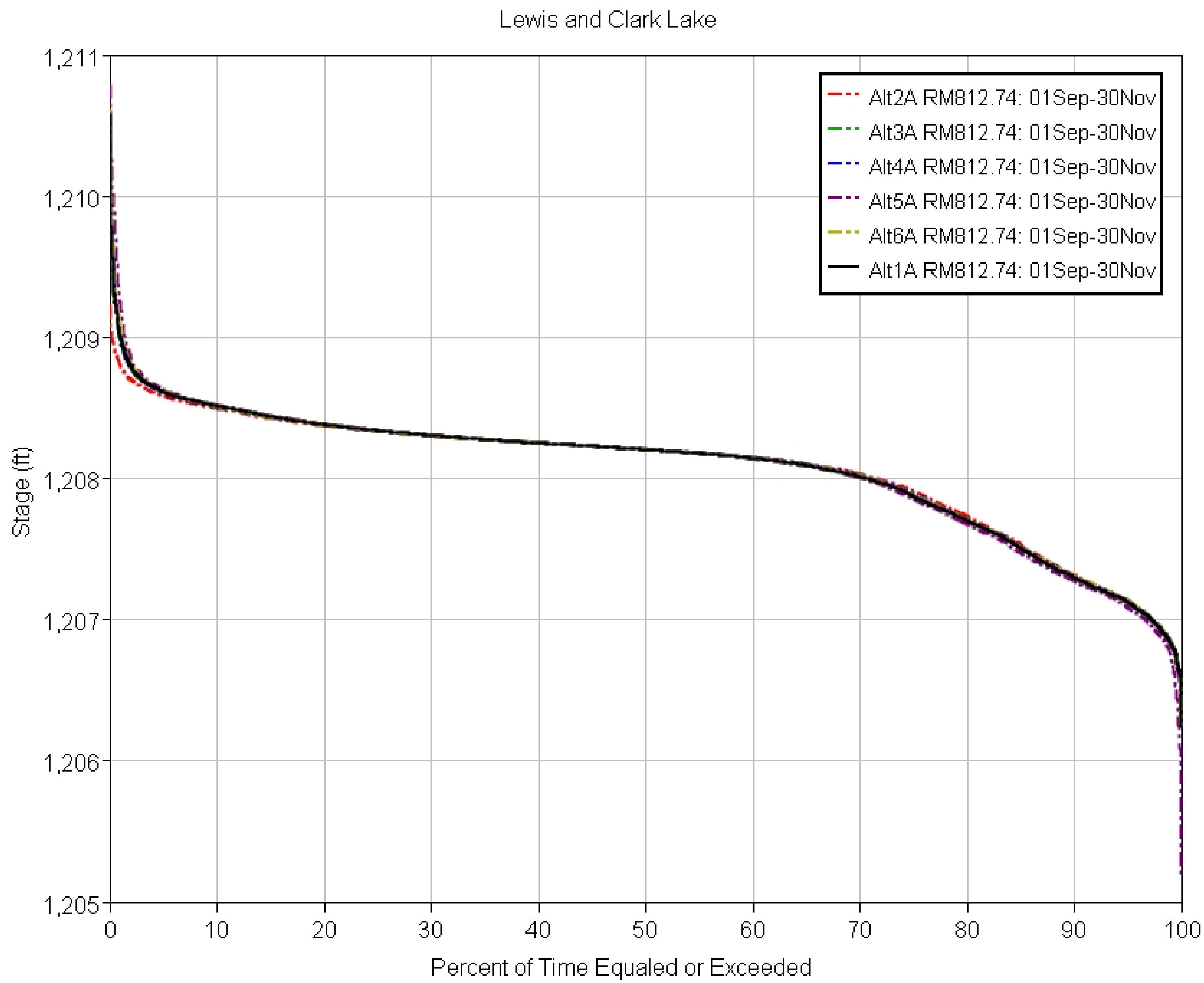


Plate 25: Lewis and Clark Lake Fall Duration

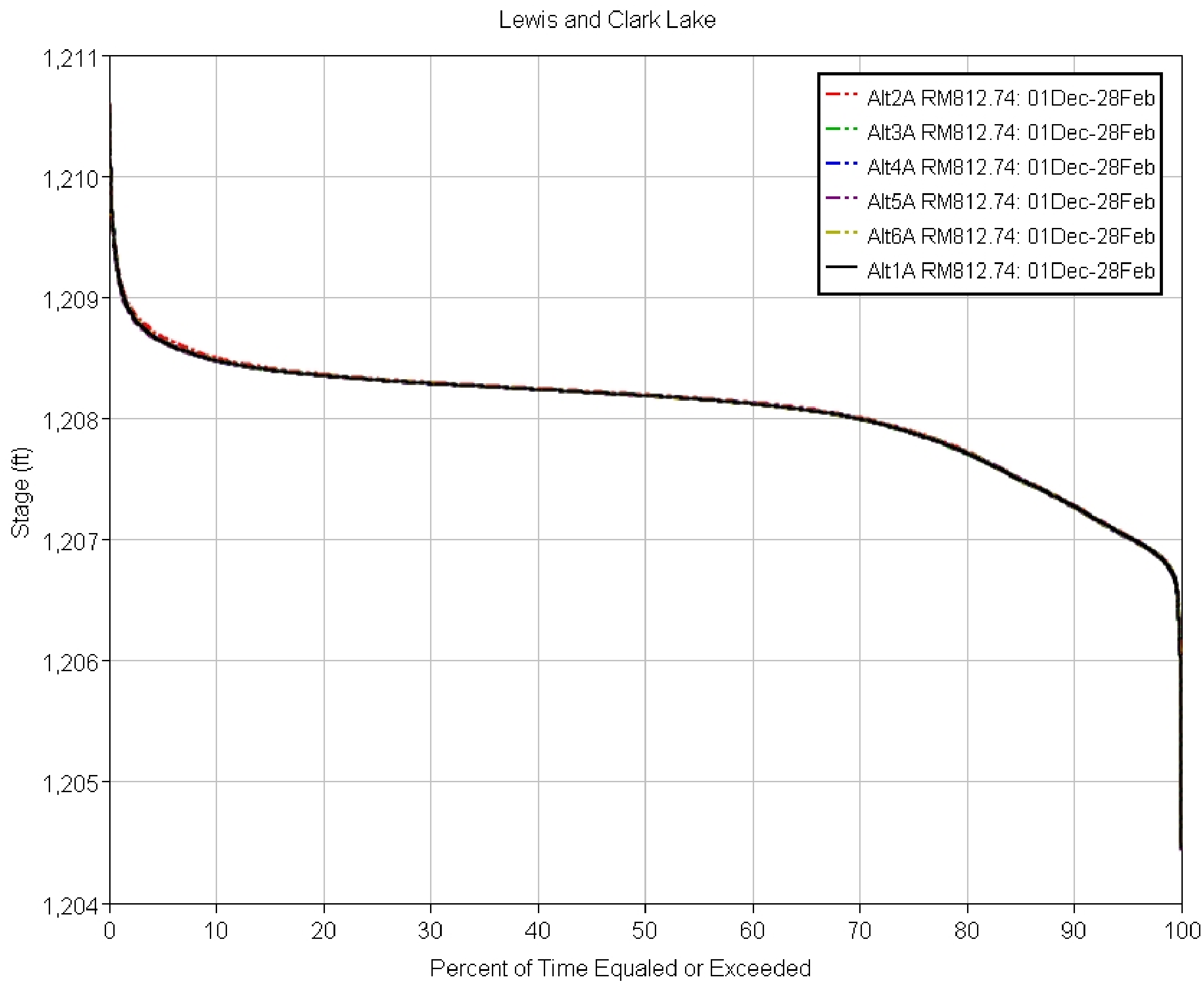


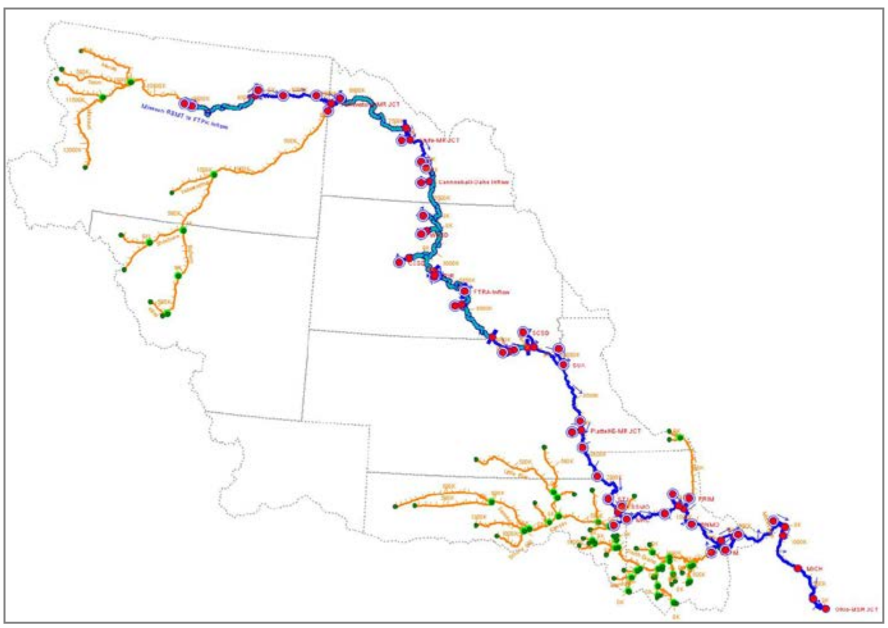
Plate 26: Lewis and Clark Lake Winter Duration

US Army Corps of Engineers ®
Omaha District

Missouri River Unsteady HEC-RAS Model
Sediment Analysis

FINAL

Fort Randall Dam to Gavins Point Dam



July 2018

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ACRONYMS

CFS…………………. Cubic Feet per Second

HC……………………Human Considerations

HEC .... Hydrologic Engineering Center

MRRIC………………..Missouri River Recovery Implementation Committee

MRRPMP-EIS……….Missouri River Recovery Program Management Plan Environmental Impact Statement

POR......................... Period of Record

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

RM……………………1960 River Mile

ROD…………………...Record of Decision

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

USGS .... United States Geological Survey

WSP .... Water Surface Profile

1 INTRODUCTION

Future condition sedimentation evaluations were performed to help support questions and comments from Missouri River Recovery Implementation Committee (MRRIC) members, review panels, and the public related to sedimentation. Future conditions, that include aggradation and degradation within the reservoir reaches and navigation channel, may change the river stage - flow relationship and consequently could affect alternative condition performance. HEC-RAS (RAS) with sediment modeling was used to evaluate future condition channel conditions. While the current condition modeling effort is an informative tool, the Missouri River is dynamic with ongoing aggradation and degradation processes.

1.1 BACKGROUND

Historically the Missouri River was characterized as a free-flowing, highly dynamic, multi-channel river, consisting of highly variable flows, high turbidity conditions, with many channel sandbars throughout the river channel and floodplain, which provided a diversity of habitat and food resources for many terrestrial and aquatic organisms. Since the late 1800's, the Missouri River has been modified by reservoirs, bank stabilization, construction of the navigation channel, and many other water resources development projects that have affected basin sediment yield and sediment transport within the mainstem Missouri River and tributaries. Navigation channel and bank stabilization structures have altered the historic multi-channeled, highly variable river system into a predominantly deep and swift, single channel in the lower river, downstream of Sioux City. Reservoir construction has created a series of alternating pools and open river reaches. These actions have combined to result in noticeable aggradation and degradation trends on the mainstem Missouri River. In summary, aggradation and degradation trends are documented, recent, and known to be ongoing.

1.2 YEAR 15 FUTURE CONDITION

The future condition modeling was referred to as "Year 15". The designation "Year 15" comes from the timeframe for implementation; the Record of Decision (ROD) for the Missouri River Recovery Program, Management Plan Environmental Impact Statement (MRRPMP-EIS) is expected to be signed in 2018, with a construction completion date of 2033, resulting in an implementation period of 15 years. All alternatives were evaluated for the Year 15 future condition. While not intended to represent detailed estimates of future reservoir and channel conditions, the results do provide an alternative comparison methodology. Results from the Year 15 analysis were provided to the human considerations teams for qualitative evaluation, versus the full quantitative evaluation that was performed on the base condition (also referred to as Year 0).

1.3 LIMITATIONS

The future condition modeling was based on a number of critical assumptions regarding historic trends, flows, and sediment inputs. Model methodology is consistent for use with the MRRMP-EIS study using the period of record (POR) flow record. Calibration of the model was limited by available data.

Results are qualitative. Determination of future condition variation between alternatives and application with Human Considerations (HC) evaluation may by limited due to RAS model assumptions and accuracy.

2 METHODOLOGY

The methodology used a calibrated historic condition sediment model to derive reasonable sediment modeling parameters for the POR simulation. These parameters are used with the current condition RAS model available from the MRRMP-EIS study to simulate the 82-year period of record and develop a rate of change throughout the model length. HEC-RAS 5.0.3 was used for all of the model runs.

2.1 SIMPLIFYING ASSUMPTIONS

- The RAS model was simplified to only the Missouri River mainstem without any tributary routing reaches.
- Flow input locations were reduced to only tributaries with sediment loads. Small ungaged tributaries and the ungaged uniform lateral flows were not included.

2.2 MODELING OVERVIEW

- Assemble a historic model for the Missouri River.
- Calibrate the historic model with steady flow and available water surface data (gages and profiles).
- Assemble sediment data and simulate from the historic period to current.
- Assess sediment model performance with volume and water surface change.
- Apply the sediment parameters to the current condition simplified Missouri River model and perform sediment computations using the 82-year POR flows.
- Compute the water surface elevation change at the end of the 82-year period. Calculate the average annual rate and multiply by 15 years to produce a future “Year 15” water surface change estimate.
- Modify the model to produce a profile equivalent to the projected change over the 15 year period.

2.3 ASSEMBLE THE SEDIMENT MODEL INPUT DATA

- Each model starts at the upstream dam and assumes no sediment input from the reservoir release.
- Assemble Missouri River bed gradation from historic bed samples. Review data to ensure that the bed material includes larger gradations to reflect armoring in the immediate reach downstream of the reservoir.

- For tributary sediment input, assemble sediment load information using best available USGS gage data and previous studies. Focus on the major drainage area tributaries using the drainage area accounting table for the reach.
- Calibrate the historic condition RAS model water surface elevation to the warm season (mid-April through mid-October), which is of primary interest.
- Set movable bed limits using the bank stations.
- Select Toffaleti, Laursen Copeland, or Yang as the transport function (initial that will be verified in calibration).
- Select Copeland (Exner 7) or Exner 5 as the sorting method (initial that will be verified in calibration).
- Select Report 12 or Ruby fall velocity method (initial that will be verified in calibration).

2.4 SEDIMENT MODEL STARTUP

Perform a stability check of the assembled sediment model and initial sediment parameter review for low, medium, and high flows.

- Create 30 day constant flow files of low (10k), medium (near bank full), and high flows (above bank in the non-degradation reach).
- Simulate sediment model for each flow condition, start with the medium flow condition for initial evaluation and debugging.
- Review performance at each flow, revise model levee stations / encroachments / ineffective flows / cross-section spacing to achieve reasonable sediment response.
- Check bed level change and debug problem areas.
- Perform initial adjustment of sediment input parameters to achieve 30 day model stability.

2.5 CALIBRATION AND SIMULATION OF HISTORIC PERIOD

- Calibrate historic condition model roughness to water surface profiles at the start of the historic period.
- Simulate the flow period for the time period between surveys (e.g. 1975 thru 2012).
- Compare model computed volume change to actual on a reach basis for the simulation period.
- Compare water surface elevation at the end of the simulation period (e.g. 2012) to observed.
- Review and refine model sediment inputs.

2.6 Future Conditions Sediment Runs Using Parameters from the Calibration Model

The primary product from the development and calibration of the historic sediment model are the sediment modeling parameters. These parameters were used for the future conditions sediment model.

3 COMPUTATIONAL ANALYSIS SEDIMENT MODEL DEVELOPMENT

Available geometry, flow, and sediment data was used to assemble a historic model.

3.1 Sediment Model Computational Studies

Following the hydraulic model calibration, additional model calibration is required for modeling of sedimentation processes. Sediment modeling computational studies may be defined in two general categories (1) computational model studies and (2) computational analysis studies (Thomas and Chang 2008). Calibration is the process of arriving at sediment model parameters (e.g. hydraulic parameters such as roughness, sediment loads, sediment material size, sediment transport function, etc.) that will allow the model to calculate values that agree with measured prototype values. A sediment model computational study involves both calibration and verification. Verification refers to the demonstration that a calibrated model will match the prototype during a period of time not used in calibration. Due to the limited data set, model verification was not possible for the sediment model. Therefore, the performed evaluation is referred to as a computational analysis. Such studies are useful and allow evaluation of the study area and comparaison of calculated results between alternatives.

3.2 Historic Model Geometry

1975 survey data was used for the historic geometry. Overbanks were filled in with either 1995 or 2007 data where needed.

3.3 Hydrologic Data

Observed flow data from 1975 to 2012 was used for the sediment model computational analysis. Fort Randall Dam realeases and Lewis and Clark Lake elevation were used as well as tributary inflows, which included Ponca Creek, the Niobrara River, and Bazile Creek. The hydrologic data is summarized in Table 3-1.

Table 3-1. Hydrologic Data

Location	RS	Boundary Condition
Fort Randall Dam	879.01	Flow Series
Ponca Creek	849.9	Lateral Flow Series
Niobrara River	843.63	Lateral Flow Series
Bazile Creek	837.08	Lateral Flow Series
Lewis & Clark Lake (Gavins Point Dam)	812.54	Stage Series

3.4 QUASI-UNSTEADY FLOW AND TEMPERATURE DATA

The quasi-unsteady flow application was selected for use with sediment modeling for this exercise instead of full unsteady. Quasi-unsteady flow allows the user to enter a time series of flow but the model does not alter the flow with computational routing such as would occur in full unsteady. Thus, channel and reservoir storage is not modeled with quasi-unsteady flow. Each inflow is entered into the quasi-unsteady flow editor within RAS as shown in Figure 3-1.

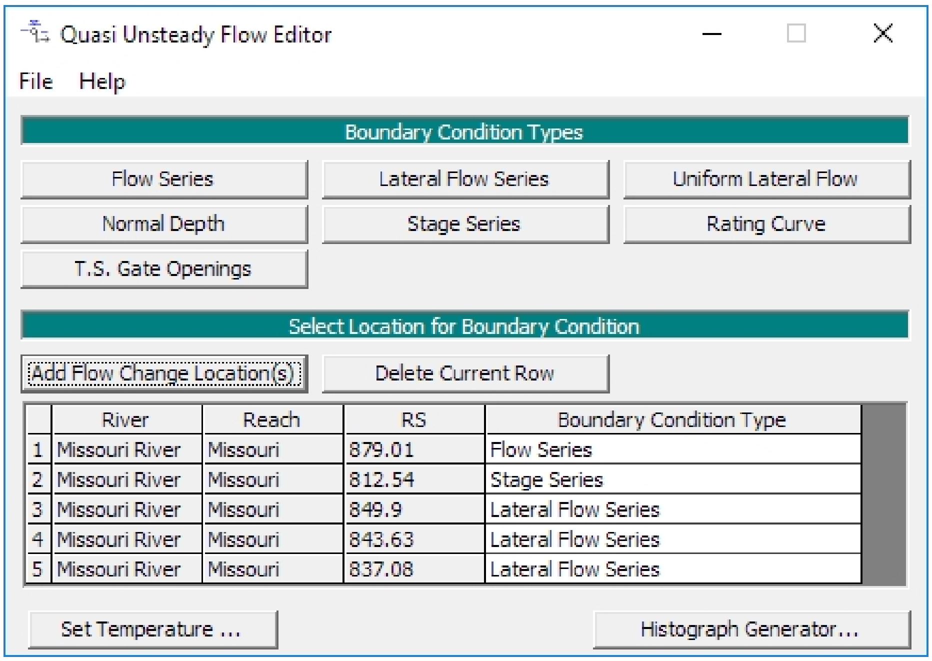


Figure 3-1. Quasi Unsteady Flow Editor

Within the quasi-unsteady flow editor, the user defines the boundary condition type for each inflow point. The furthest downstream cross section requires a downstream boundary condition which was set to a stage series of daily Lewis and Clark Lake pool elevations.

For this modeling application, all flow data was entered with a 24 hour duration and a computation increment of 24 hours. An example lateral inflow data input screen is shown in Figure 3-2.

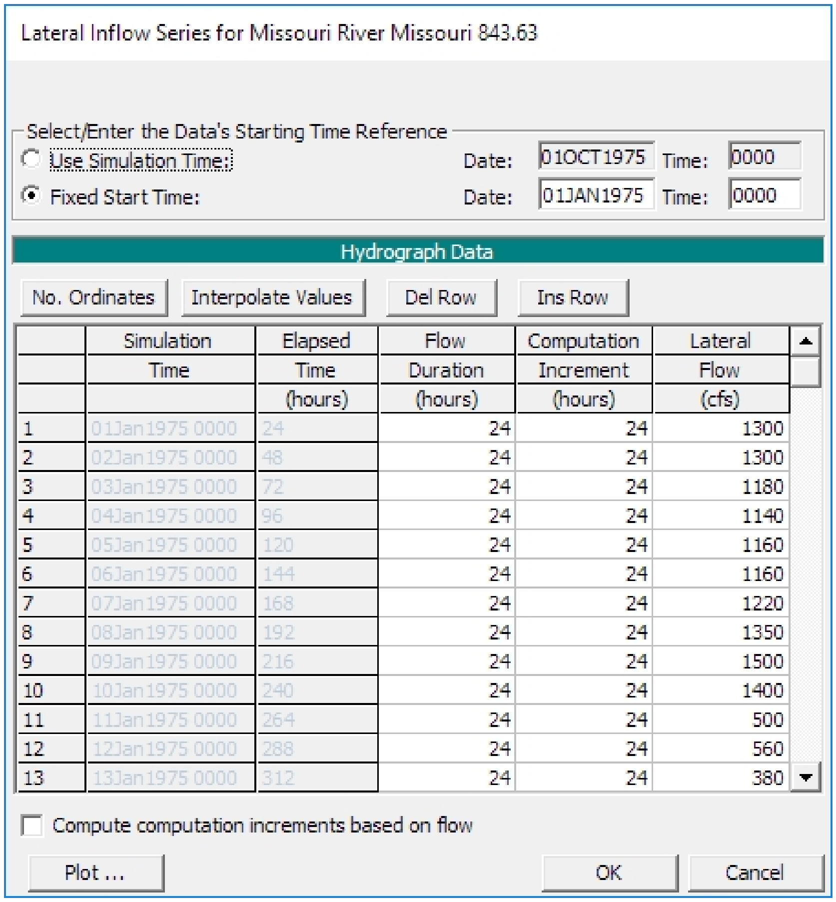
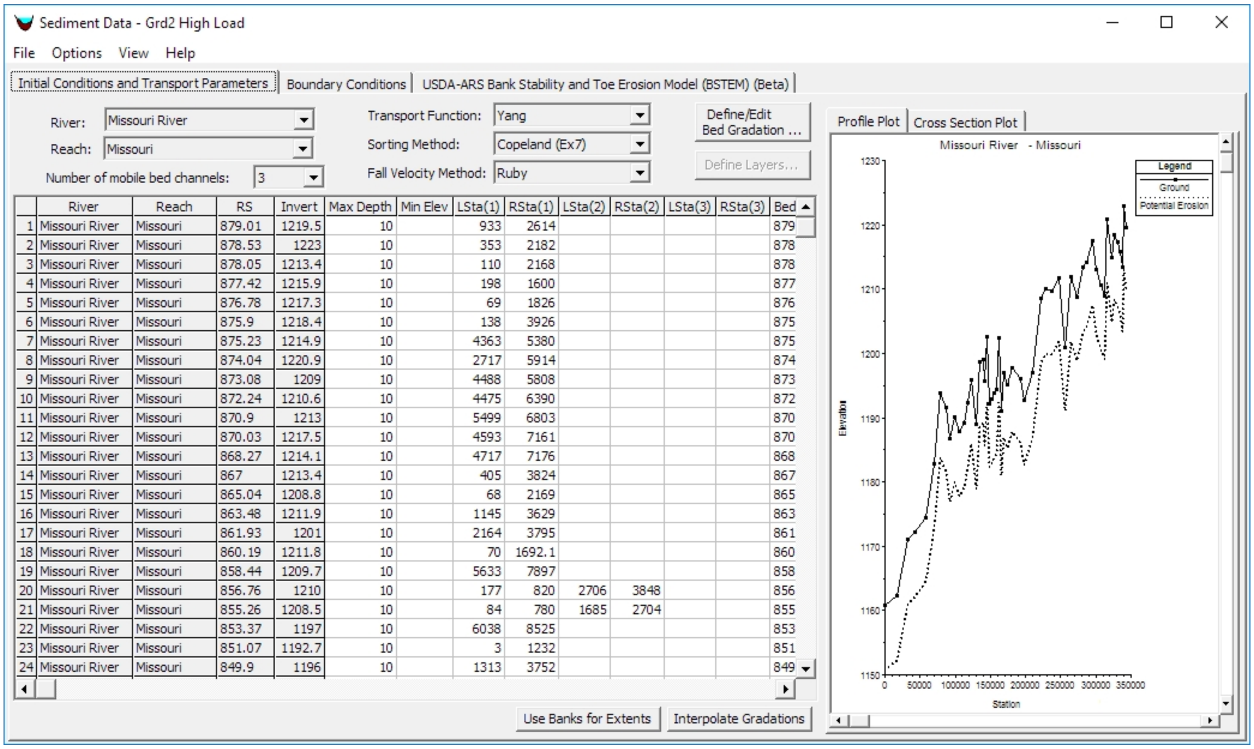


Figure 3-2. Lateral Inflow Data Input Example

Within the editor, the user also enters temperature data. For this exercise, daily temperature data from Fort Randall Dam was used.

3.5 Sediment Modeling Parameters

Sediment transport modeling parameters are specified within the sediment boundary condition editor. The model used the Yang sediment transport equation, the Copeland (Exner 7) bed mixing algorithm, and the Ruby fall velocity method. The Copeland mixing method was selected because it often performs better than the alternatives for large sand bed rivers, and simulates erosion better in these systems (alternative mixing methods tend to over-predict armoring and, therefore, under predict erosion). Typical input within the RAS model is shown in Figure 3-3.



	River	Reach	RS	Invert	Max Depth	Min Elev	LSta(1)	RSta(1)	LSta(2)	RSta(2)	LSta(3)	RSta(3)	Bed
1	Missouri River	Missouri	879.01	1219.5	10		933	2614					879
2	Missouri River	Missouri	878.53	1223	10		353	2182					878
3	Missouri River	Missouri	878.05	1213.4	10		110	2168					878
4	Missouri River	Missouri	877.42	1215.9	10		198	1600					877
5	Missouri River	Missouri	876.78	1217.3	10		69	1826					876
6	Missouri River	Missouri	875.9	1218.4	10		138	3926					875
7	Missouri River	Missouri	875.23	1214.9	10		4363	5380					875
8	Missouri River	Missouri	874.04	1220.9	10		2717	5914					874
9	Missouri River	Missouri	873.08	1209	10		4488	5808					873
10	Missouri River	Missouri	872.24	1210.6	10		4475	6390					872
11	Missouri River	Missouri	870.9	1213	10		5499	6803					870
12	Missouri River	Missouri	870.03	1217.5	10		4593	7161					870
13	Missouri River	Missouri	868.27	1214.1	10		4717	7176					868
14	Missouri River	Missouri	867	1213.4	10		405	3824					867
15	Missouri River	Missouri	865.04	1208.8	10		68	2169					865
16	Missouri River	Missouri	863.48	1211.9	10		1145	3629					863
17	Missouri River	Missouri	861.93	1201	10		2164	3795					861
18	Missouri River	Missouri	860.19	1211.8	10		70	1692.1					860
19	Missouri River	Missouri	858.44	1209.7	10		5633	7897					858
20	Missouri River	Missouri	856.76	1210	10		177	820	2706	3848			856
21	Missouri River	Missouri	855.26	1208.5	10		84	780	1685	2704			855
22	Missouri River	Missouri	853.37	1197	10		6038	8525					853
23	Missouri River	Missouri	851.07	1192.7	10		3	1232					851
24	Missouri River	Missouri	849.9	1196	10		1313	3752					849

Figure 3-3. Sediment Data Example

3.6 MOVABLE BED LIMITS

Movable bed limits are used within the model to define how the cross section elevation points are adjusted in response to computed mass erosion or deposition. The movable bed limits were originally set at the channel banks but were changed during the calibration process. Some areas near the Niobrara River confluence were very sensitive to the moveable bed limit location. RAS has different bed change options and the option to allow deposition outside of the moveable bed limits was selected. In some areas, especially in the delta area downstream of the Niobrara River where there are two to three main channels, multiple moveable bed limits were set. A typical cross section with the movable bed limits is shown in Figure 3-4.

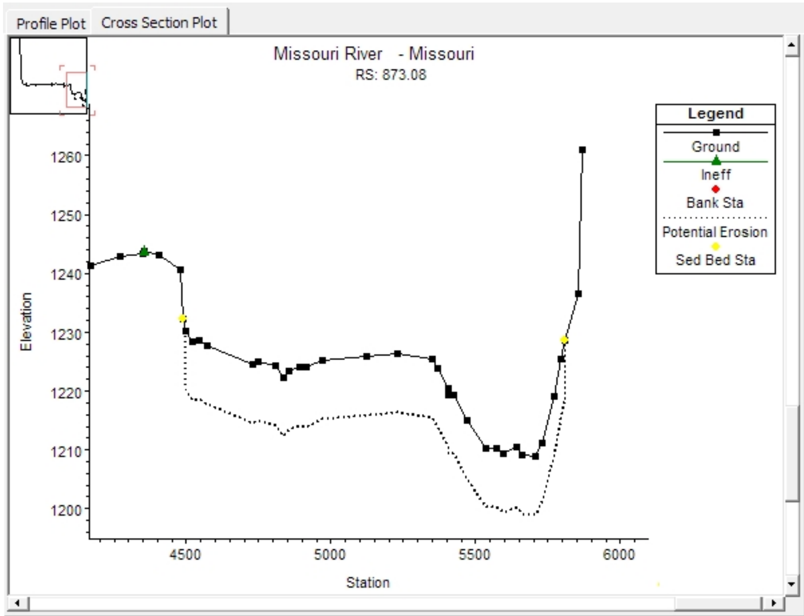


Figure 3-4. Moveable Bed Limits Example

3.7 BED GRADATION DATA

A bed gradation is specified at each cross section or can be interpolated between cross sections. Bed gradation data within the model is from field measurements taken in 1975. Bed data from cross section 827.5 was used for the remaining six downstream reservoir cross sections in the model due to inconsistent field data. An example bed gradation curve is shown in Figure 3-5.

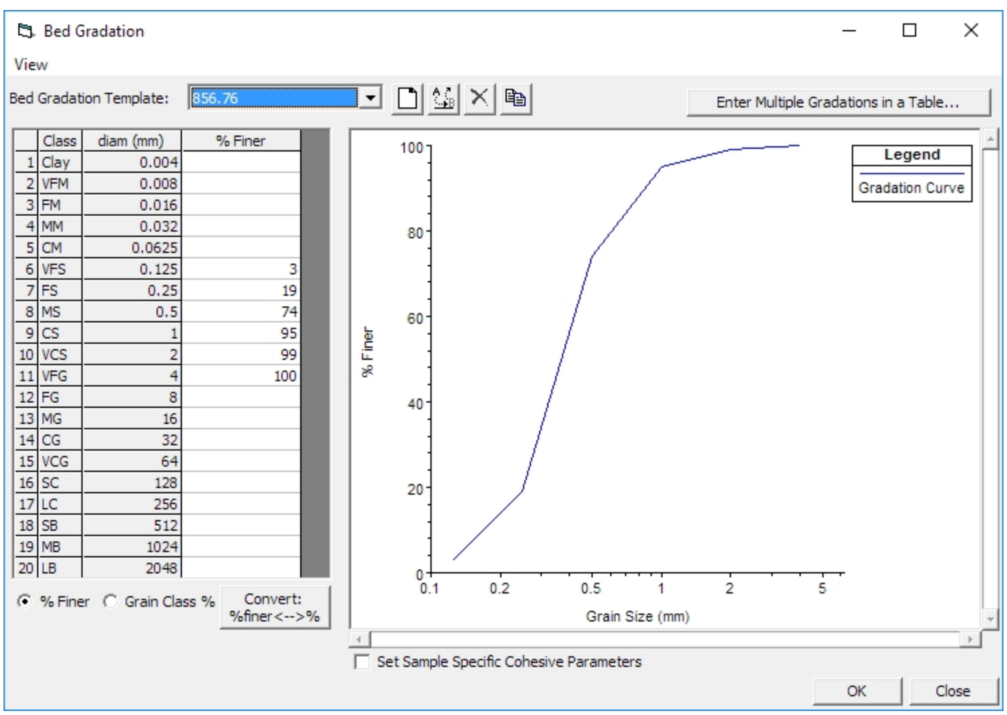


Figure 3-5. Bed Gradation Example

3.8 Sediment Boundary Condition: Flow-Load Rating Curve

Sediment models require sediment load boundary conditions at all locations where sediment inflow occurs. Locations may include the upstream model boundary and each of the flow input locations for the major tributaries and uniform lateral inflows. Sediment input is specified for each computational time step in the simulation. Because sediment time series data are rare, sediment models often use flow-load rating curves to compute sediment load boundary conditions from the flow boundary conditions.

The sediment loading curve is typically expressed as:

$$Q_s = a \times Q^b \qquad \textbf{Eq. 1}$$

Where

Q_s = Suspended sediment (tons/day)

Q = Discharge (ft^3/sec)

a = the intercept

b = the slope, exponent typically in the range between 1.5 and 2.5

The typical format of the sediment load relationship recognizes that the formulation of the sediment load power function as a linear model requires a logarithmic transformation to linearize the function and subsequently correct for subunity bias in the retransformation of sediment-discharge or –concentration estimates (Crawford 1991). The degree to which constituent discharges are underestimated as a result of retransformation is a function of the goodness-of-fit of the regression line. Generally, increasing the data scatter around the regression line results in decreasing estimates of the value of the dependent variable (Gray and Simões 2008). Therefore, best practice for fitting a rating curve to flow-load data involves computing an “unbiased corrector” to account for these biases. The Duan (1983) “smearing factor” approach is a method often employed to yield an unbiased flow load rating curve relationship that conforms with theory and experience. However, for this modeling exercise, the tributary sediment load data was sparse and an unbiased correction factor was not necessary.

A direct relation between Q and Q_s in streams is rarely present. A lack of synchronization between the peaks of water discharge and sediment concentration over a flood hydrograph is more the rule than the exception. That means that in parts of the hydrograph where sediment discharge is increasing, sediment concentration may be decreasing, and vice versa. In some cases, a piece-wise relationship, with different flow-load relationships for high and moderate flows might be appropriate. Piece-wise flow-load relationships (sometimes called “bent rating curves”) are common in sediment analysis, because the highest flows are often supply limited, delivering less sediment than the capacity predicts because less sediment is available.

For this purposes of this computational modeling exercise, the derivation of the sediment load rating curve at various tributary inflow points was performed as part of the calibration process.

For input to the RAS model, the Q_s relationship shown in Eq. 1 was used to generate a rating curve for each input location. The input rating curves are illustrated in Table 3-2 through Table 3-5. During the study, HEC released a bug report indicating that sediment loads were doubled during model simulation. All values shown inare the actual tributary sediment loads. Report values were halved when entered in RAS.

Table 3-2. Gavins Point Sediment Rating Curve

Flow (cfs)	300	500	2,000	7,000	10,000	20,000	50,000	100,000	300,000
Load (tons/day)	0.2	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1

Table 3-3. Ponca Creek Sediment Rating Curve

Flow (cfs)	100	1,000	2,000
Load (tons/day)	300	4,000	15,000

Table 3-4. Niobrara River Sediment Rating Curve

Flow (cfs)	100	3,000	6,000	10,000	15,000	20,000	26,000
Load (tons/day)	14	20,000	90,000	200,000	400,000	600,000	800,000

Table 3-5. Bazile Creek Sediment Rating Curve

Flow (cfs)	100	5,000	18,000
Load (tons/day)	6	70,000	1,000,000

3.9 SEDIMENT BOUNDARY CONDITION: LOAD GRADATIONS

Multiple grain size, sediment transport models do not just require upstream sediment load boundary conditions, but also require users to subdivide boundary loads by grain class. Additionally, load-gradation relationships can vary as a function of flow, sometimes in complicated or counterintuitive ways (Gibson and Cai 2017). Sometimes the load coarsens at higher flows and sometimes it fines. Very limited load gradation data was available for the tributary input locations. Load gradations were edited during the calibration process.

3.10 DATA GAPS AND DATA QUALITY

Data gaps for the RAS sediment modeling include tributary sediment load and gradation data, historic river geometry, and locations of bedrock.

3.10.1 Tributary Sediment Load and Gradation

Limited tributary sediment sampling information was available.

3.10.2 Channel Geometry

During model calibration and sediment simulation, it became apparent that historic surveys were limited in cross section. Volume change computations demonstrated the lack of quality information for the historic period.

3.10.3 Bedrock Controls

Very few bedrock controls are known to exist along the Missouri River. However, bedrock outcrops may be present in some areas within the degradation reach that acts to limit channel degradation. No bedrock controls were used within the model.

4 HISTORICAL MODEL STEADY FLOW CALIBRATION

The 1975 geometry was taken from the *Lewis and Clark Lake Sediment Management Study, Phase II, Part II* (USACE 2018a).

5 SEDIMENT SIMULATIONS

Sediment simulations were performed for the period from 1975 through 2012. Sediment parameters were adjusted based on volume change and water surface elevation change. The main calibration parameters were the Niobrara River's sediment load and gradation. The other two tributaries (Ponca Creek and Bazile Creek) have such small sediment input, that they were deemed insignificant during calibration and the intial values were not changed. The moveable bed limits were also adjusted during calibration, especially in the area around the Niobrara River.

5.1 Volume Change

The primary data used for calibration of the sediment model was volume change. A steady model was used to compare volumes for the 1975 and 2007 geometries. A profile of 100,000 cfs was run through the 1975 geometry and the resulting water surface elevations were set for the 2007 geometry. Incremental volume differences were converted to cumulative (upstream to downstream) to match the output of the sediment model for comparison. The 2007 data was used instead of the 2011/2012 geometry because it is more representative of the long term historic trend. The flood of 2011 was a unique event that was not included in the calibration.

A range of gradations and loads for the Niobrara River were simulated. As seen in Table 5-1 and Figure 5-1, three Niobrara River gradations of varied coarseness were simulated. The load was kept the same thoughout. Then the Grd2 gradation was run with a higher and lower load. As seen in Figure 5-1, the RAS simulations fail to capture the movement of fine sediments into the reservoir pool however, the goal of the sediment modeling was to produce an estimated average water surface change and the backwater effect of the pool negates the need for this estimate in the reservoir area. The Grd2 high load was chosen as the best calibration run based upon its ability to match observed water surface changes at Niobrara.

Table 5-1. Niobrara River Gradation Sensitivity Runs

Grain Class	Grain Size (mm)	Grd1	Grd2	Grd3
Clay	0.002-0.004	5	1	3
VFM	0.004-0.008	5	1	10
FM	0.008-0.016	10	5	15
MM	0.016-0.032	10	5	15
CM	0.032-0.0625	15	5	10
VFS	0.0625-0.125	15	9	10
FS	0.125-0.25	20	36	20
MS	0.25-0.5	10	25	10
CS	0.5-1	7	10	5
VCS	1-2	2	2	1
VFG	2-4	1	1	1

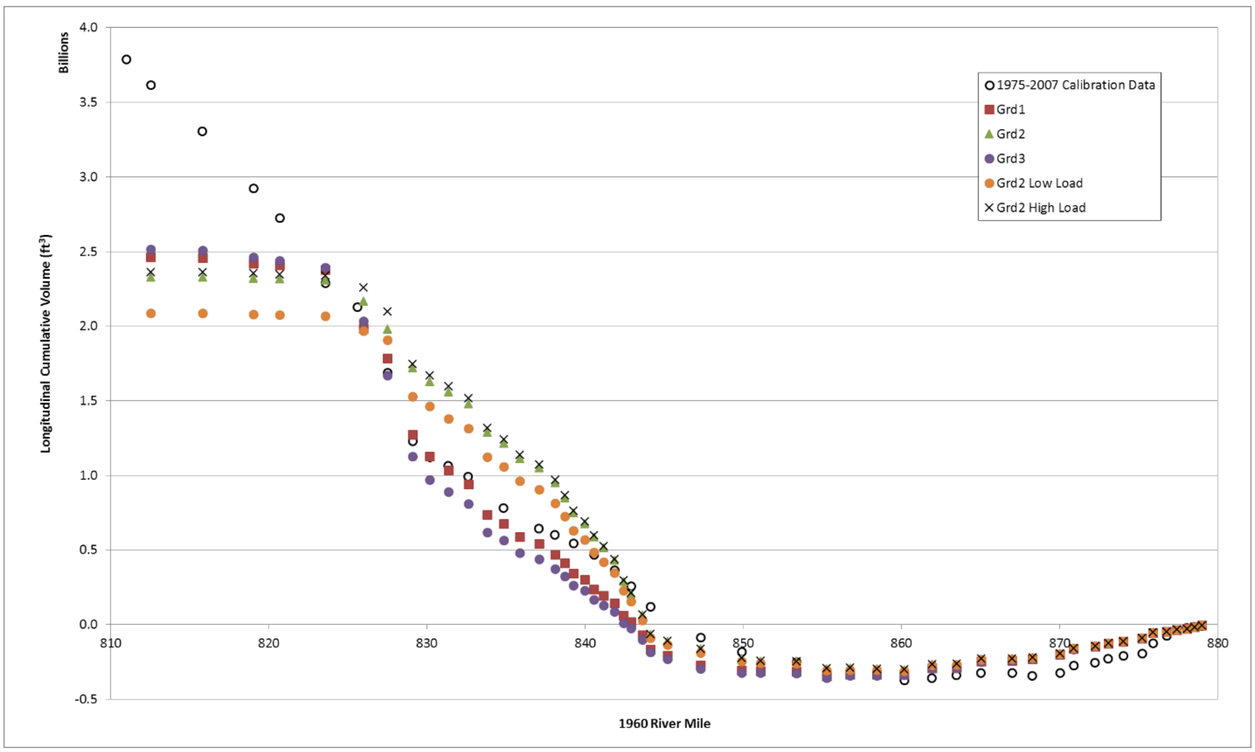


Figure 5-1. Longitudinal Cumulative Volume Change

5.2 Water Surface Elevation

The 2009 Water Surface Profile (WSP) was used to compare model water surface output through the range of model runs. The 2012 WSP was not used due to it being after the 2011 flood and the powerplant at Fort Randall Dam was peaking during the profile survey. Usually, dam releases are kept steady during the water surface profile survey. Gage data (Missouri River at Niobrara and Springfield) was also used to confirm the water surface profile elevations. As seen in Figure 5-2, the Grd2 and Grd2 high load matched the water surface at Niobrara. However, none of the runs could reproduce the water surface at Springfield, which is well into the delta of the reservoir. The model underpredicts the amount of fines making it to the delta and reservoir.

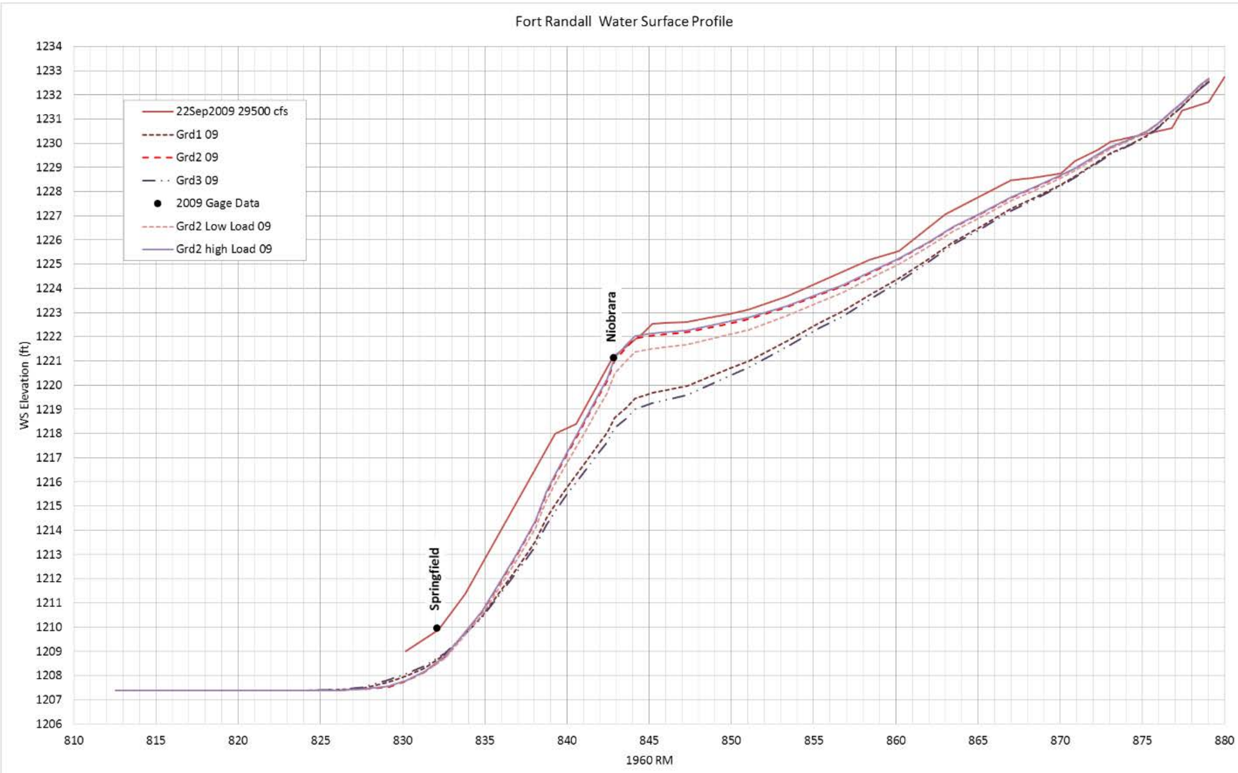


Figure 5-2. Water Surface Profile Comparison

6 FUTURE CONDITIONS SEDIMENT RUNS

Future condition sediment runs used the sediment parameters from the calibration sediment model. For the future condition sediment runs, the geometry was updated to the 2012 survey and the previously used POR flows from 1930 to 2012 were used to model 82 years into the future. The average water surface change for that period was then used to estimate the Year 15 geometry.

6.1 GEOMETRY

The 2012 (sediment range only) geometry was used in the future conditions sediment runs.

6.2 FLOW

Unsteady flows for the six alternatives from 1930 to 2012 from the previous Management Plan modeling were converted to quasi-unsteady for use with the future conditions sediment model. Since there are different rules for unsteady and quasi-unsteady flows, some uniform lateral flows had to be adjusted so that they did not start at the same cross section as another flow input. More details on the POR flow dataset can be found in the report, *Missouri River Recovery Management Plan Time Series Data Development for Hydrologic Modeling* (USACE 2018c).

6.3 Sediment Parameters

Sediment parameters from the calibration sediment model were used for the future conditions sediment runs. Bed data was updated to represent 2012 data. The moveable bed limits were also updated due to using a different geometry with different stationing. The temperature data was extended to cover the period from 1930 to 1970 with an estimated daily average, while there was observed daily data from 1970-2012.

7 SUMMARY

A sediment model was constructed for the Fort Randall to Gavins Point reach of the Missouri River for the purposes of evaluating future conditions in support of the Missouri River Management Plan study. Sediment model simulations used modeling from the period 1975 to 2012 to develop sediment parameters. These parameters were then used with the POR flows and sediment simulations to develop a prediction of the future conditions, starting with 2012 geometry. These results were then averaged to produce a water surface change estimate for the year 2032 (Year 15). Details on how the Year 15 future geometry was created can be found in the *Missouri River Recovery Management Plan Environmental Impact Statement HEC-RAS Modeling Alternatives Report* (USACE 2018b).

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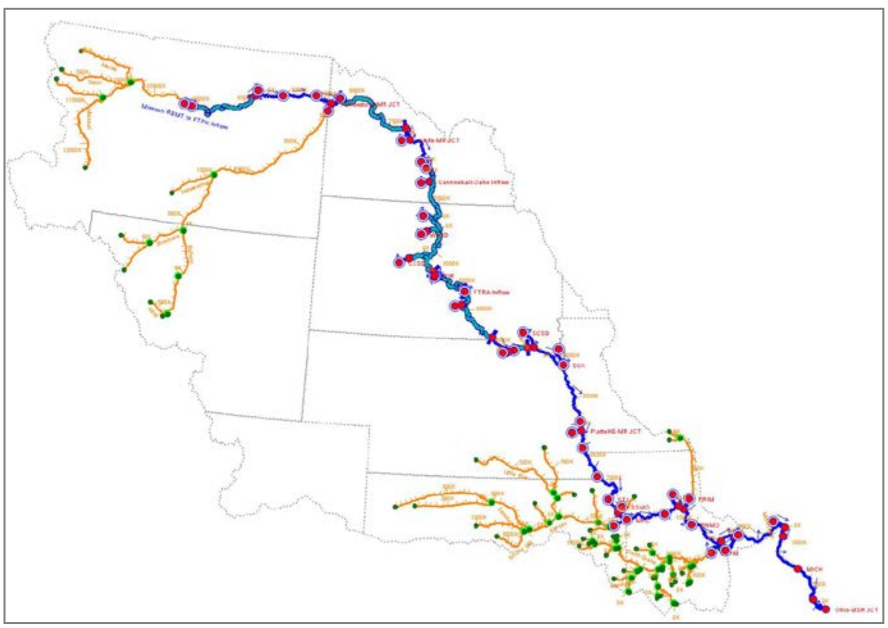
US Army Corps of Engineers ®
Omaha District

Missouri River Unsteady HEC-RAS Model
Alternatives Analysis

FINAL

Appendix D

Gavins Point Dam to Rulo, NE



July 2018

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ACRONYMS

BiOp…………………..Biological Opinion

CFS…………………. Cubic Feet per Second

ESH…………………..Emergent Sandbar Habitat

HC…………………….Human Considerations

HEC .... Hydrologic Engineering Center

IRC……………………Interception-Rearing Complexes

MAF………………….. Million acre-feet

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

MRRPMP-EIS…..……Missouri River Recovery Program Management Plan Environmental Impact Statement

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

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

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

NLD…………………...National Levee Database

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

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

POR .... Period of Record

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

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

RM…………………….1960 River Mile

ROD…………………..Record of Decision

SWH………………….....Shallow Water Habitat

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

USFWS……………….United States Fish and Wildlife Service

USGS .... United States Geological Survey

1 INTRODUCTION

The Missouri River unsteady HEC-RAS (RAS) model was developed for the Missouri River Recovery Program Management Plan and Integrated Environmental Impact Statement (MRRPMP-EIS) to assist in the assessment of a suite of actions to meet Endangered Species Act (ESA) responsibilities for the piping plover, the interior least tern, and the pallid sturgeon using USACE authorities. The objective of the RAS modeling is to simulate the Management Plan alternatives which include both geometry and flow changes relative to the No Action alternative. The Human Considerations (HC) team performed an extensive analysis on each of the alternatives for each of the resources (hydropower, cultural resources, fish & wildlife (exclusive of listed species), flood risk, irrigation, recreation, thermal power, and water supply) and provide a detailed comparison of results. For this report, only the hydraulic model output is presented; there is no alternative selection or discussion. 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, the district boundary. This report will focus only on the Omaha District portion of the model. Refer to Kansas City District's report (Appendix E) for details regarding the downstream model from Rulo, NE to St. Louis, MO.

Six alternatives, including the No Action alternative, were simulated in RAS from March 1930 to December 2012, however the HC team only used complete year data for their analysis from January 1, 1931 to December 31, 2012. Development of inflow records at current depletion levels to use as boundary conditions for the HEC-ResSim (ResSim) and RAS models is documented in the report, *Missouri River Recovery Management Plan Time Series Data Development for Hydrologic Modeling* (USACE 2018c). Each alternative has unique flow releases from the reservoirs, as simulated by the ResSim model. Three RAS geometries were developed, representing three proposed habitat construction configurations on the river.

2 GEOMETRY

Model geometry development and calibration for the existing conditions is documented in *Missouri River Unsteady HEC-RAS Model Calibration Report Appendix D Gavins Point Dam to Rulo, NE* (USACE 2015a). The alternative geometries contain all of the major features from the calibration models including: levee and ineffective points, roughness values, and lateral structures representing levee systems.

Three geometries were created for the alternatives analysis. Modifications generally included the addition of shallow water habitat (SWH) in the form of river widening at a specified reference flows. Modifications to previously constructed habitat were also included in the geometries while backwaters were not and are only accounted for in the acreage goals. The configurations modeled included:

1. **No Action -** Assumes habitat construction activities follow current practices to achieve 20 acres/mile of SWH, the minimum target specified in the 2003 Amendment to the 2000 Biological Opinion.

2. **Biological Opinion as Projected (BiOp) -** Guidance from the U.S. Fish & Wildlife (USFWS) was provided to create a geometry which represents an ideal implementation of the 2003 Biological Opinion (2003 BiOp) (USFWS 2003). It assumes habitat construction accomplishes 30 acres/mile of SWH, and performs at a wider range of flows including a summer low, median August, and spring pulse. Floodplain connectivity was evaluated, but the requirement was met so no changes to the RAS geometry were necessary.
3. **Interception-Rearing Complexes (IRC) -** SWH construction activities proceed based on findings made by the Effects Analysis team. It assumes habitat construction accomplishes 260 acres/year based on current annual habitat construction rates.

SWH was only added to the Gavins to Rulo (Omaha District) and Rulo to the Mouth (Kansas City District) models. All other RAS models upstream of Gavins Point Dam do not have SWH geometry changes.

Table 2-1: Geometry Summary

Geometry	Target SWH	Reference Flow (cfs)
No Action	20 acre/mile	August 50% exceedance
BiOp	30 acre/mile + floodplain connectivity	Summer low, Median August, & Spring Pulse
IRC	260 acre/year	Median June

2.1 Changes to Existing (2012) Model

For the Gavins to Rulo reach model, several geometries were constructed in order to obtain a final base geometry to apply the modifications to for each of the alternative geometries. Figure 2-1 is a flow chart of the RAS geometries created. First, the existing (2012) geometry was modified to include projects that were constructed or awarded between 2012 and 2015. The August 50% exceedance flows from the 2007 update along with the 2012 geometry was used to assign inverts to the added projects. The current design criteria for SWH in chutes is 5 ft below the August 50% exceedance profile. Widths were assigned based on the 2014 SWH Accounting Report (USACE 2015b) and/or 2014 aerial imagery. This new geometry is named “2015 Base” and coupled with the August 50% exceedance profile was used for assigning inverts to all other models. The “2015 Base” model was then modified to reflect fully evolved chutes and top width widening for all of the projects; this geometry is called “Built-Out SWH”. With a few exceptions, all chutes were modeled with a 200 ft top width, invert elevation of 5 ft below August 50% exceedance (from the 2015 Base geometry), and 2H:1V side slopes. Top width widening projects were modeled with a 250 ft top width and inverts were 0 - 5 ft below the August 50% exceedance. All chutes used an n-value of 0.043. The “Built-Out SWH” geometry was used as the starting point for all three of the alternative geometries: No Action, BiOp, and IRC. The following sections describe the changes made to each of the alternative geometries in more detail.

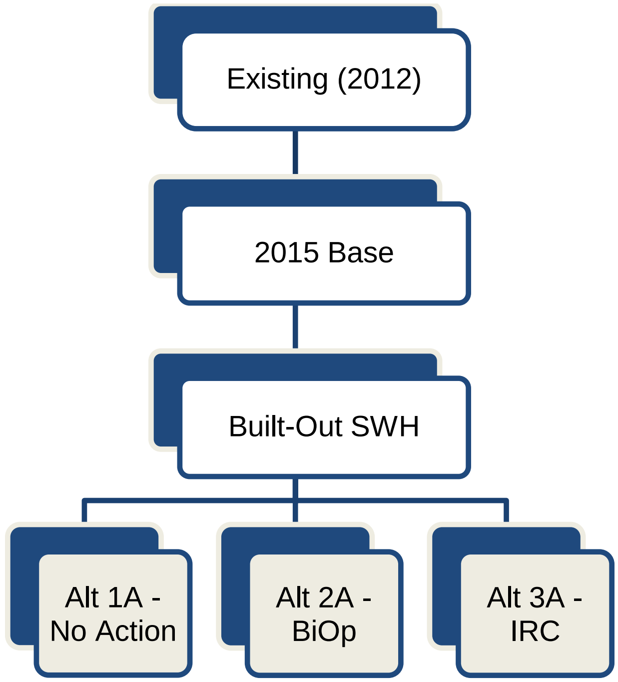


Figure 2-1: RAS Geometry Flow Chart

No new chutes were added to the RAS models, primarily because recent emphasis has been on top width widening projects. All chutes included in the 2014 SWH Acreage Accounting were counted as existing SWH. Some chutes were not altered whereas some were set to a reduced top width at sites where 200 feet would be unrealistic. See Table 2-2 for a list of chute top widths. The existing geometry was modified to produce these dimensions. At chutes with multiple channels, such as Sandy Point Chute, the combined top width of both channels was limited to 200 feet.

Table 2-2: Chute Top Widths

Site Name	US River Mile	DS River Mile	Chute Length (Feet)	2014 GIS Measured Area (Acres)	Built-out Area (Acres)	Built-out Top Width (Feet)
Glovers Point	713.4	711.2	11,100	5	5	20
Middle Decatur Bend	688.2	687.4	4,640	4	21	200
Lower Decatur Chute*	687.3	684.9	2,400	0	0	0
Fawn Island	674.1	673.3	2,979	16	16	234
Little Sioux Bend	668.3	666.8	7,400	19	34	200
Sandy Point Bend	658	656.4	6,505	30	30	200
Tyson Chute (w/o upper end)	655.5	653.1	9,234	16	16	75
California Bend - NE	650.1	648.5	9,230	14	14	66
California Bend - IA	650.3	649.6	4,000	8	8	87
Boyer Chute	637.8	633.7	16,760	63	77	200
Lower Calhoun Bend*	637.6	637.1	2,750	0	0	0
Council Bend	617.8	616.8	5,630	26	26	200
Plattsmouth	594.5	592.1	12,070	0	55	200
Tobacco Island	588.4	586.3	16,300	46	75	200
Upper Hamburg	555.9	552.2	15,950	152	73	200
Lower Hamburg	553.3	550.7	12,960	79	60	200
Kansas Bend	546.4	544.4	7,180	42	33	200
Nishnabotna	543.6	542.5	5,780	24	27	200
Deroin Bend	520.4	516.4	18,140	172	83	200
Rush Bottoms	501.5	500.1	8,400	28	39	200

*Note that Lower Decatur Chute and Lower Calhoun Bend projects were filled in from the 2011 flood event. They are not expected to be restored and will therefore have 0 acres of habitat.

2.2 ALTERNATIVE GEOMETRY OVERVIEW

The "Built-Out SWH" geometry was modified to produce the three alternative geometries: No Action, BiOp, and IRC. Amounts of habitat added to each geometry were based on the 2003 Amended BiOp (USFWS 2003), discussions with USFWS, and calculations of existing SWH from the 2014 SWH Accounting Report (USACE 2015b).

2.2.1 No Action Geometry

The No Action geometry assumes habitat construction activities follow current practices to achieve 20 acres/mile of SWH, the minimum target specified within the 2003 BiOp (USFWS 2003) of 20 – 30 ac/mi. Habitat was distributed by the 2003 BiOp reaches, and the 2014 Shallow Water Habitat Accounting Report (USACE 2015b) was used to determine the acreage deficit within each reach to attain the 20 ac/mi goal. Habitat was placed to provide 0 – 5 ft of depth at August 50% exceedance flows, which is the current design criteria for SWH. Most of the SWH added to reach the goal was in the form of top width widening while backwaters were incorporated as well.

Backwaters were not included in the model geometry because they provide limited conveyance that cannot be evaluated by the one-dimensional RAS model.

2.2.2 BiOp Geometry

The BiOp as projected geometry assumes the maximum habitat goal of 30 acres/mile of SWH from the 2003 BiOp. Similar to the No Action geometry, habitat was distributed by 2003 BiOp reaches, and the 2014 Shallow Water Habitat Accounting Report was used to determine the acreage deficit within each reach to attain the 30 ac/mi goal. Habitat was placed to provide 0 – 5 ft of depth at three different flow levels: low summer flow, mid-August, and Spring Pulse. Section 2.4.2 provides further details of the reference flows. Most of the SWH added to reach the goal was in the form of top width widening while backwaters were incorporated as well. Backwaters were not included in the model geometry because they provide limited conveyance that cannot be evaluated by the one-dimensional RAS model. Part of the BiOp requirement was achieving floodplain connectivity, and a separate analysis was conducted as documented in Section 2.7 to ensure the requirements were met, although no changes to the RAS model were necessary.

2.2.3 IRC Geometry

Assumes SWH construction activities proceed based on findings made by the Effects Analysis team. The total amount of habitat was based on the current SWH implementation rate of about 130 acres/year per district for a total of 260 acres/year. Assuming a 15-year implementation period and that IRC habitat construction will begin at no more than 2 years after the Record of Decision (ROD) is signed, this results in a 13-year construction window. Therefore, the total amount of IRC habitat for both districts is 3,380 acres (260 acre/year * 13 years = 3,380 acres).

Distribution was based on conversations with the Effects Analysis team, who specified upper and lower boundaries based on their knowledge of larval pallid spawning locations, drift rates, and timing of interception. Sioux City is the upstream threshold for IRC placement, with the area between the Nebraska Platte River and the Osage River more likely to be successful. Chutes and existing habitat may be modified to meet IRC habitat criteria, but for purposes of the RAS modeling these were not counted toward the target acreage. Habitat was placed to provide 0 – 6 ft of depth at median June flows. Section 2.4.3 provides further details of the reference flows. All of the SWH added to the geometry was in the form of top width widening.

2.3 ACRES OF HABITAT

Acres of habitat added to each of the three alternative geometries was based on the 2003 Amended BiOp, discussions with USFWS, or calculations of existing SWH from the 2014 SWH Accounting Report. Three components of shallow water habitat were taken into account when calculating how much habitat to add: top width widening, modifications to previously constructed projects, and backwaters. The Gavins to Rulo reach underwent significant changes during the 2011 flood, and the adjustment of existing chutes, top width widening, and backwater projects to sustainable dimensions required numerous changes to the RAS geometry.

2.3.1 No Action Alternative

Acres of SWH added to HEC-RAS to meet 20 ac/mi for the No Action geometry were broken down by 2003 BiOp segment for even distribution of habitat along the river. Table 2-3 shows the acres needed to reach the goal acreage by segment. The total goal acres needed for the Gavins to Rulo reach was 2,008 acres.

Table 2-3: Acres of SWH - No Action Geometry

Reach	Segment	RM Start	RM End	Miles	Required Acres (20 ac/mi)	SWH Acres Existing (2014 SWH Report)	SWH Acres Needed to Reach Goal
Ponca to Sioux City	11	753	735	18	360	120	240
Sioux City to Platte River	12	735	595	140	2,800	1,682	1,118
Platte River to Rulo (NWO)	13	595	498	97	1,940	1,290	650
Rulo to Kansas River (NWK)	13	498	367	131	2,620	1,270	1,350
Kansas River to Osage River	14	367	130	237	4,740	3,710	1,030
Osage River to Mouth	15	130	0	130	2,600	2,600*	0
Omaha District				255	5,100	3,092	2,008
Kansas City District				498	9,960	7,580	2,380
Total				753	15,060	10,672	4,388

*Existing acres for segment 15 (3,253 acres) exceeds 20 ac/mi.

For the No Action Alternative geometry, the breakdown of habitat type for the Omaha District is shown in Table 2-4 and is about 72% top width widening, 24% backwaters, and 4% changes to existing SWH.

Table 2-4: SWH Type - No Action Geometry

Reach	Segment	Widening (acres)	Widening (miles) [1]	Number of Widening Projects	Backwaters (acres)	Number of Backwater Projects	Changes to Existing SWH (acres)[3]
Ponca to Sioux City	11	180	5.9	2	60	1	0
Sioux City to Platte	12	601	19.8	9	420	7	97
Platte to Rulo (NWO)	13[2]	672	22.2	9	0	0	-22
Total		1,453	48	20	480	8	75

1 Assumes average widening width of 250-ft in the Omaha District.
2 Segment 13 is from the Platte River to the Kansas River and is divided at Rulo by the District boundary.
3 Changes to Existing SWH includes repairs to projects following the 2011 flood which decreased acreage in some projects.

Miles were converted to acres by multiplying the length of top width widening in miles by width and converting units to acres. For widening, the length measurement started and ended about halfway between cross sections, since this most appropriately reflects the interpolation between cross sections made by RAS during computations.

2.3.2 BiOp Alternative

Acres of SWH added to HEC-RAS to meet 30 ac/mi for the BiOp as Projected geometry were broken down by 2003 BiOp segment for even distribution of habitat along the river. Table 2-5 shows the acres needed to reach the goal acreage by segment. The total goal acres needed for the Gavins to Rulo reach was 4,558 acres.

Table 2-5: Acres of SWH - BiOp as Projected Geometry

Reach	Segment	RM Start	RM End	Miles	Required Acres (30 ac/mi)	SWH Acres Existing (2014 SWH Report)	SWH Acres Needed to Reach Goal
Ponca to Sioux City	11	753	735	18	540	120	420
Sioux City to Platte River	12	735	595	140	4,200	1,682	2,518
Platte River to Rulo (NWO)	13	595	498	97	2,910	1,290	1,620
Rulo to Kansas River (NWK)	13	498	367	131	3,930	1,270	2,660
Kansas River to Osage River	14	367	130	237	7,110	3,710	3,400
Osage River to Mouth	15	130	0	130	3,900	3,253	647
Omaha District				255	7,650	3,092	4,558
Kansas City District				498	14,940	8,233	6,707
Total				753	22,590	11,325	11,265

For the BiOp Alternative geometry, the breakdown of habitat type for the Omaha District is shown in Table 2-6 and is about 79% top width widening, 20% backwaters, and 2% changes to existing SWH.

Table 2-6: SWH Type - BiOp Geometry

Reach	Segment	Widening (acres)	Widening (miles) [1]	Number of Widening Projects	Backwaters (acres)	Number of Backwater Projects	Changes to Existing SWH (acres)
Ponca to Sioux City	11	240	7.9	4	180	3	0
Sioux City to Platte	12	1,761	58.1	32	660	11	97
Platte to Rulo (NWO)	13[2]	1,582	52.2	24	60	1	-22
Total		3,583	118	60	900	15	75

1 Assumes average widening width of 250-ft in the Omaha District.
2 Segment 13 is from the Platte River to the Kansas River and is divided at Rulo by the District boundary.
3 Changes to Existing SWH includes repairs to projects following the 2011 flood which decreased acreage in some projects.

Miles were converted to acres by multiplying the length of top width widening in miles by width and converting units to acres. For widening, the length measurement started and ended about halfway between cross sections, since this most appropriately reflects the interpolation between cross sections made by RAS during computations.

2.3.3 IRC Alternative

The total amount of habitat for the IRC alternative geometry was based on current SWH implementation rates, which equates to about 130 acres/year per district for a total of 260 acres/year. Assuming a 15-year implementation period and that IRC habitat construction will begin at no more than 2 years after the ROD is signed, this results in a 13-year construction window. Therefore, the total amount of IRC habitat for both districts is 3,380 acres (260 acre/year * 13 years = 3,380 acres).

Due to larval drift rates, among other factors, IRC habitat distribution does not extend upstream of Sioux City (RM 735) and is concentrated more heavily below the Platte River (RM 595). A few projects were located between Sioux City and the Platte River to determine if these areas are viable IRC locations. Approximate IRC habitat rates were used to set the total amount of acres for each reach as seen in Table 2-7. Existing habitat sites may be modified to meet IRC criteria as a part of this alternative, however, this will not be represented in RAS because of minimal impacts to water surface elevations, and will not be counted towards the target acres as a conservative assumption.

Table 2-7: Acres of Habitat - IRC Geometry

Reach	Segment	RM Start	RM End	Miles	IRC Habitat Rate (ac/mi)	Target Acres*
Ponca to Sioux City	11	753	735	18	0	0
Sioux City to Platte River	12	735	595	140	2.0	276
Platte River to Rulo (NWO)	13	595	498	97	6.0	585
Rulo to Kansas River (NWK)	13	498	367	131	5.1	670
Kansas River to Osage River	14	367	130	237	5.9	1,389
Osage River to Mouth	15	130	0	130	3.5	460
Omaha District				255	3.4	861
Kansas City District				498	5.1	2,519
Total				753	4.5	3,380

*As a conservative assumption, existing habitat sites were not counted towards the goal acreage.

As shown in Table 2-8, top width widening was the only type of habitat added to the RAS model.

Table 2-8: Habitat Type - IRC Geometry

Reach	Segment	Widening (acres)	Widening (miles) *	Number of Widening Projects
Ponca to Sioux City	11	0	0	0
Sioux City to Platte	12	276	9.1	4
Platte to Rulo (NWO)	13**	585	19.3	7
Total		861	28.4	11

*Assumes average widening width of 250-ft in the Omaha District.
**Segment 13 is from the Platte River to the Kansas River and is divided at Rulo by the District boundary.

2.3.4 Public Ownership Acreage Analysis

To assist the economic analysis team, an analysis was performed to calculate the amount of existing publicly owned lands available for new habitat construction. This information was used to determine the amount and cost of additional lands that would need to be purchased. Existing public lands includes USACE, USFWS, and state conservation owned lands suitable for habitat development, and was accumulated by counting publicly owned river front acres available in each BiOp segment, so existing public land availability is a constant between alternatives. Values in the tables were capped at the target. Suitability of particular sites for the type of habitat to be constructed was not considered. Furthermore, the additional lands required acreage is habitat only, a factor would need to be used to convert from habitat land to total real estate tract purchase size. See Table 2-9, Table 2-10, and Table 2-11 for a listing of public ownership acreage availability for each geometry.

Table 2-9: Public Ownership Acreage - No Action Geometry

		Widening (ac)			Backwaters (ac)			Total[1]
Reach	Seg	Target (ac)	Exist Public Lands (ac)[2]	Addt'l Lands Req'd (ac)	Target (ac)	Exist Public Lands (ac)[2]	Addt'l Lands Req'd (ac)	Addt'l Lands Req'd (ac)
Ponca to Sioux City	11	180	0	180	60	0	60	**240**
Sioux City to Platte River	12	601	601	0	420	420	0	**0**
Platte River to Rulo (NWO)	13	672	672	0	0	0	0	**0**
Rulo to Kansas River (NWK)	13	1,129	454	675	-	-	-	**675**
Kansas River to Osage River	14	937	937	0	-	-	-	**0**
Osage River to Mouth	15	0	0	0	-	-	-	**0**
Omaha District		1,453	1,273	180	480	420	60	**240**
Kansas City District		2,066	1,391	675	-	-	-	**675**
Total		3,519	2,664	855	480	420	60	**915**
Total Percentage		-	76%	24%	-	87.5%	12.5%	**-**

1 Note that chutes (NWK) and changes to existing SWH (NWO) are not shown because they reside on existing public lands. Also, this acreage is habitat only and does not include a factor for total real estate tract purchase size.
2 Includes existing public lands that are available for habitat placement. This number is capped at the target acres and is not the total acres of existing public lands.

Table 2-10: Public Ownership Acreage - BiOp Geometry

		Widening (ac)			Backwaters (ac)			Total[1]
Reach	Seg	Target (ac)	Exist Public Lands (ac)[2]	Addt'l Lands Req'd (ac)	Target (ac)	Exist Public Lands (ac)[2]	Addt'l Lands Req'd (ac)	Addt'l Lands Req'd (ac)
Ponca to Sioux City	11	240	0	240	180	0	180	**420**
Sioux City to Platte River	12	1,761	836	925	660	660	0	**925**
Platte River to Rulo (NWO)	13	1,582	907	675	60	60	0	**675**
Rulo to Kansas River (NWK)	13	2,439	454	1,985	-	-	-	**1,985**
Kansas River to Osage River	14	3,307	1,375	1,932	-	-	-	**1,932**
Osage River to Mouth	15	529	529	0	-	-	-	**0**
Omaha District		3,583	1,743	1,840	900	720	180	**2,020**
Kansas City District		6,275	2,358	3,917	-	-	-	**3,917**
Total		9,858	4,101	5,757	900	720	180	**5,937**
Total Percentage		-	42%	58%	-	80%	20%	**-**

1 Note that chutes (NWK) and changes to existing SWH (NWO) are not shown because they reside on existing public lands. Also, this acreage is habitat only and does not include a factor for total real estate tract purchase size.
2 Includes existing public lands that are available for habitat placement. This number is capped at the target acres and is not the total acres of existing public lands.

Table 2-11: Public Ownership Acreage - IRC Geometry

		Widening (ac)			Backwaters (ac)			Total[1]
Reach	Seg	Target (ac)	Exist Public Lands (ac)[2]	Addt'l Lands Req'd (ac)	Target (ac)	Exist Public Lands (ac)[2]	Addt'l Lands Req'd (ac)	Addt'l Lands Req'd (ac)
Ponca to Sioux City	11	0	0	0	-	-	-	**0**
Sioux City to Platte River	12	276	276	0	-	-	-	**0**
Platte River to Rulo (NWO)	13	585	585	0	-	-	-	**0**
Rulo to Kansas River (NWK)	13	670	454	216	-	-	-	**216**
Kansas River to Osage River	14	1,389	1,375	14	-	-	-	**14**
Osage River to Mouth	15	460	460	0	-	-	-	**0**
Omaha District		861	861	0	-	-	-	**0**
Kansas City District		2,519	2,289	230	-	-	-	**230**
Total		3,380	3,150	230	-	-	-	**230**
Total Percentage		-	93%	7%	-	-	-	**-**

1 Note that chutes (NWK) and changes to existing SWH (NWO) are not shown because they reside on existing public lands. Also, this acreage is habitat only and does not include a factor for total real estate tract purchase size.
2 Includes existing public lands that are available for habitat placement. This number is capped at the target acres and is not the total acres of existing public lands.

2.3.5 Excluded Areas

Locations selected for habitat construction in the RAS model are theoretical and do not reflect actual locations of future mitigation projects. However, the following areas were intentionally avoided when making modifications to RAS models. For the BiOp Alternative geometry, satisfying all of the below criteria was not always possible due to the amount of habitat added.

1) Reaches of river within a 10,000-ft radius of an airport (per FAA AC 150/5200-33) (FAA 2007)
2) Areas within 1/4 mile upstream or downstream of small town infrastructure along the river bank, on that side of the river only
3) Areas within 1/4 mile upstream or downstream of power plant or municipal water intakes, on both sides of the river
4) Areas within 1/4 mile upstream or downstream of barge loading facilities and other river related industrial infrastructure along the river bank, on that side of the river only
5) Areas within 1/4 mile upstream or downstream of bridges, on both sides of the river
6) Areas within 1/4 mile upstream and downstream of Federal/PL 84-99 levees lying close to the river bank, on that side of the river only.

7) Reaches adjacent to larger cities (e.g. Omaha and Sioux City), particularly where the channel is confined by urban levees

8) Widening projects were not located in the same bend as new or existing chutes or widening to avoid excessive navigation channel flow loss.

2.4 Reference Flows

Reference flows were used to set design elevations for SWH added to the geometries. Reference flows were run through the "2015 Base" geometry in steady flow, and the resulting water surface profile along with the specified depth was used to set the invert for added SWH.

2.4.1 No Action Geometry

August 50% exceedance flows from the 2007 SWH Profile Report (USACE 2007) were run through the "2015 Base" geometry to obtain invert elevations 5 feet below the profile. Flows for the mainstem gage locations are shown in Table 2-12 and a full list of flows used in RAS can be seen in Plate 1.

Table 2-12: No Action August 50% Duration Flows - Mainstem Gages

Location	River Mile	August 50% Duration Flow (cfs)
Gavins Point Dam	810.9	32,630
Sioux City	732.37	33,430
Decatur	691.03	34,220
Omaha	616.03	36,630
Nebraska City	562.6	40,050
Rulo	498.04	42,170

2.4.2 BiOp Geometry

Three reference flows were used for the BiOp geometry: low summer flow, mid-August, and Spring Pulse. Flows were calculated by starting with the Gavins Point release (from MRBWMD *Hydrologic Statistics Technical Report* (USACE 2013)) and adding incremental flows at major tributaries. Incremental flows were calculated by the MRBWMD in the *Missouri River Incremental Flows Below Gavins Point Technical Report* (USACE 2014a) for the pre-dam and post-dam time periods, at statistical levels of minimum, median, and maximum, lower and upper decile and quartiles. Median pre-dam records were used because they incorporate the drought of the 30's and are therefore slightly lower than the post-dam statistic. Incremental flows for the months of July, August, and May, were used for summer low, median August, and spring pulse, respectively. July incremental flows were added downstream of the summer low Gavins Release (21,000 cfs), because the summer low condition centers primarily on this month. May incremental flows were added downstream of the spring pulse because the second spring pulse specified occurs during this month. Flows for the mainstem gage locations and the incremental flows between them are listed in Table 2-13 below. A full list of flows used in the model is shown in Plate 1.

Table 2-13: BiOp Reference Flows - Mainstem Gages

Location	River Mile	Low Summer Flow (cfs)	Mid-August Flow (cfs)	Spring Pulse Flow (cfs)
Gavins Point Dam	810.9	21,000	36,900	48,800
Incremental Flow		2,200	1,700	3,200
Sioux City	732.37	23,200	38,600	52,000
Incremental Flow		2,600	1,700	2,400
Omaha	616.03	25,800	40,300	54,400
Incremental Flow		5,000	3,400	7,300
Nebraska City	562.6	30,800	43,700	61,700
Incremental Flow		1,900	1,300	2,200
Rulo	498.04	32,700	45,000	63,900

2.4.3 IRC Geometry

The reference flow for the IRC geometry was the median June flow, which includes the median June release from Gavins Point Dam (from MRBWMD *Hydrologic Statistics Technical Report* (USACE 2013)) plus June incremental flows from the *Missouri River Incremental Flows Below Gavins Point Technical Report* (USACE 2014a). Flows for the mainstem gage locations and the incremental flows between them are listed in Table 2-14 below. A full list of flows used in the model is shown in Plate 1.

Table 2-14: IRC Mid-June Reference Flows - Mainstem Gages

Location	River Mile	Mid-June Flow (cfs)
Gavins Point Dam	810.9	32,200
Incremental Flow		3,400
Sioux City	732.37	35,600
Incremental Flow		3,200
Omaha	616.03	38,800
Incremental Flow		8,300
Nebraska City	562.6	47,100
Incremental Flow		2,700
Rulo	498.04	49,800

2.5 Top Width Widening

Top width widening was a SWH component added to all three alternative geometries. Cross sections were widened by 250 ft upstream of Rulo (Omaha District) and 300 ft downstream of Rulo (Kansas City District) by use of the channel design/modification editor in RAS. Depths and reference flow water surface profiles both vary between alternatives. The widened area was assigned a Manning's n-value of 0.035 which is rougher than the main channel because it accounts for depth variability and additional structures that are likely to be included in a typical project. This value was selected based on past experience with top width widening projects' varying topography however, future projects may perform differently. Levee and ineffective points were adjusted to match the new high bank station but remained at the same elevation. Keeping the levee and ineffective point elevations the same assumes that the overbank topography would remain similar to existing conditions. This also minimizes the difference between the 2012 calibration geometry. Figure 2-2 is a comparison between the top width widening for all three alternative geometries at a sample cross section.

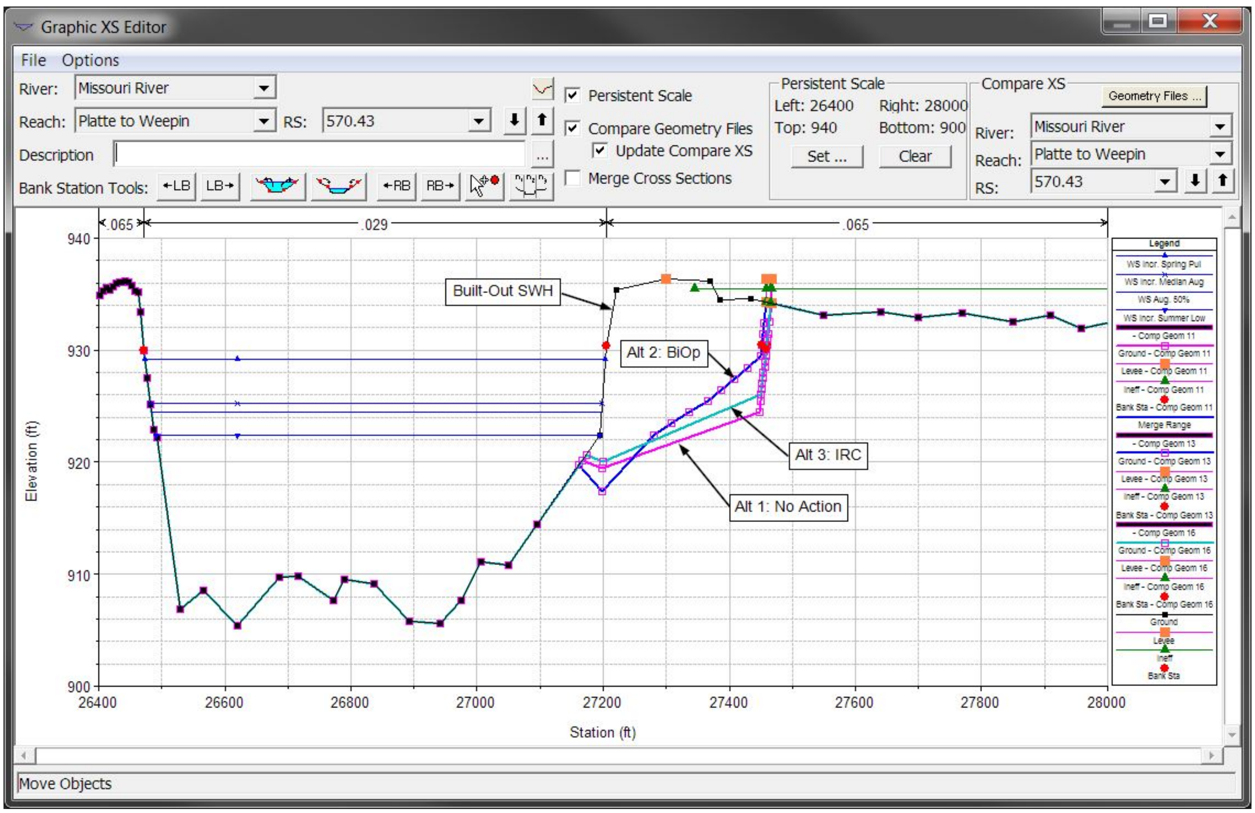


Figure 2-2: Comparison of Top Width Widening - All Geometries

2.5.1 No Action Geometry

Top width widening was added to cross sections in the RAS model based on the calculated habitat that needed to be added to the 2003 BiOp reaches. Depths range from 0 to 5 ft with respect to the August 50% water surface elevation at each cross section. An example of the top width widening cross-section modification for the No Action alternative geometry is shown in Figure 2-3.

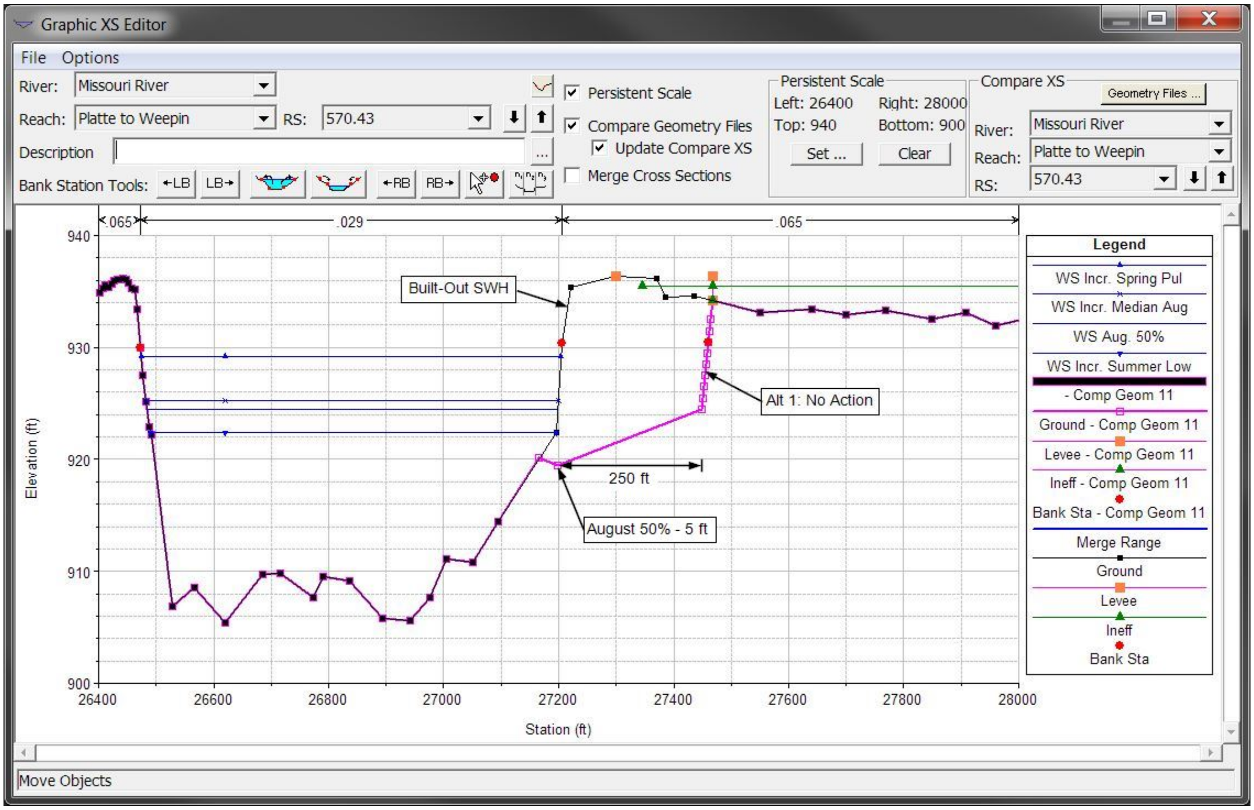


Figure 2-3: Example Top Width Widening for No Action Geometry

2.5.2 BiOp Geometry

Top width widening was added to cross sections in the RAS model based on the calculated habitat that needed to be added to the 2003 BiOp reaches. Depths range from 0 - 5 ft at three flow levels: low summer flow, mid-August, and Spring Pulse. A series of five templates was used to expedite the insertion of top width widening. The water surface elevation difference was calculated between the spring pulse and median August profiles and between the median August and the summer low flow profiles. The numbers were then rounded to the nearest foot and a template was selected with that combination of elevation differences. The templates always start 5 ft below the low summer flow, rise to the calculated template distance to the mid-August flow, and then rise to the calculated template distance to the Spring Pulse flow. Each flow level is approximately a third of the width or about 83 ft. If multiple templates were calculated for a proposed project, the most dominant template was chosen for all cross sections in the project. The number of projects that used each template is shown in Table 2-15. An example of the top width widening cross-section modification for the BiOp alternative geometry is shown in Figure 2-4.

Table 2-15: BiOp Geometry Top Width Widening Templates

Template	Number of Projects
Low Summer to Mid-August Difference: 3 ft Mid-August to Spring Pulse Difference: 3 ft	11
Low Summer to Mid-August Difference: 4 ft Mid-August to Spring Pulse Difference: 3 ft	33
Low Summer to Mid-August Difference: 4 ft Mid-August to Spring Pulse Difference: 4 ft	1
Low Summer to Mid-August Difference: 3 ft Mid-August to Spring Pulse Difference: 4 ft	14
Low Summer to Mid-August Difference: 2 ft Mid-August to Spring Pulse Difference: 3 ft	1

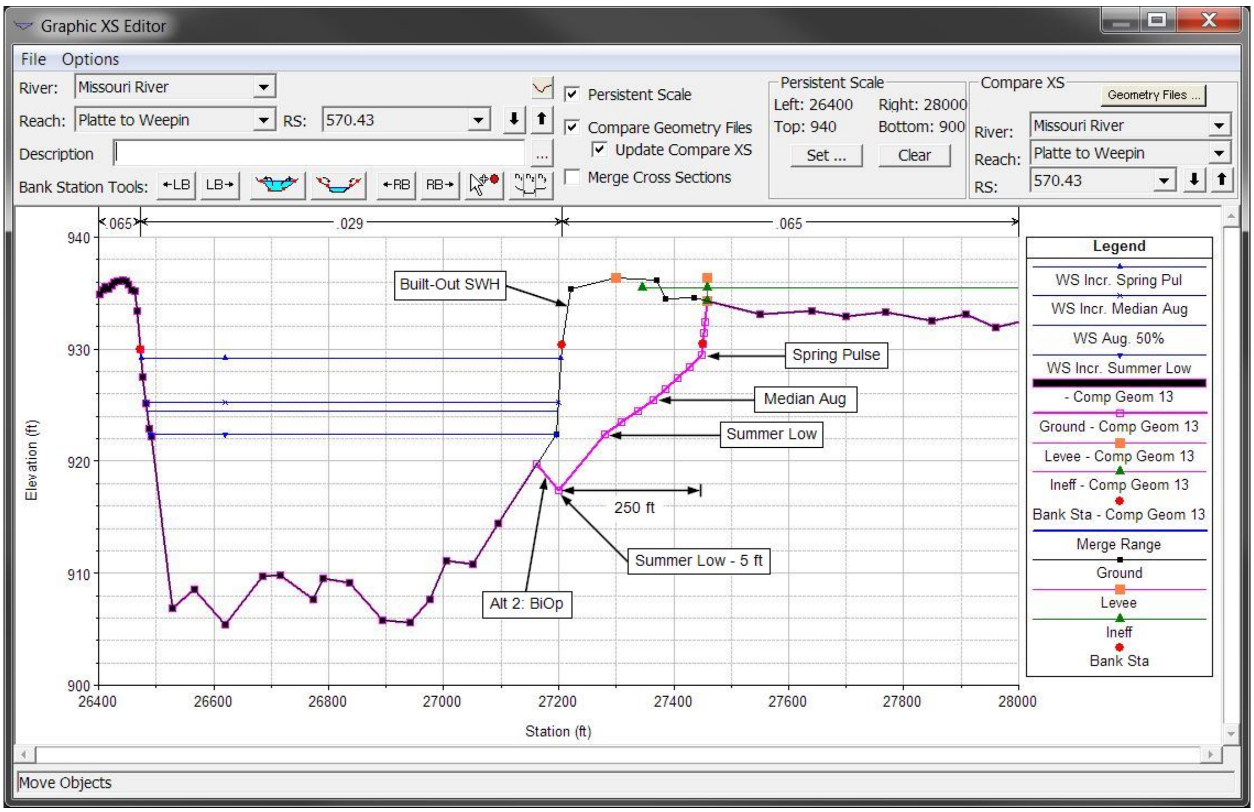


Figure 2-4: Example Top Width Widening for BiOp Geometry

2.5.3 IRC Geometry

Top width widening was added to cross sections in the RAS model based on the calculated habitat that needed to be added to the 2003 BiOp reaches. Depths range from 0 to 6 ft with respect to the mid-June water surface elevation at each cross section. The widened area was assigned a Manning's n-value of 0.035. An example of the top width widening cross-section modification for the IRC alternative geometry is shown in Figure 2-5.

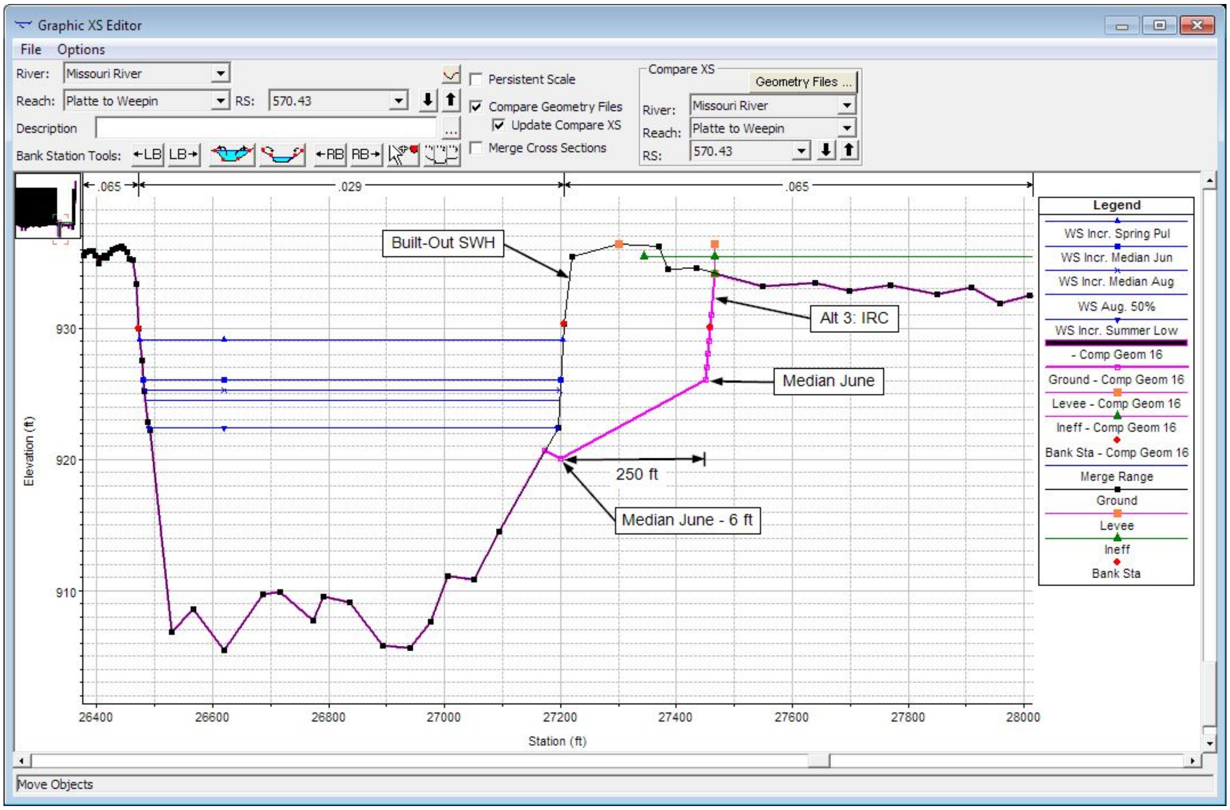


Figure 2-5: Example Top Width Widening for IRC Geometry

2.6 Backwaters

Backwaters provide limited conveyance that cannot be evaluated by the one-dimensional RAS model. Therefore, while the acreage was accounted for, modifications to the RAS model were not made. See Table 2-4 and Table 2-6 for the total number of backwaters added to each segment for each alternative. No backwater acres were added for the IRC alternative.

2.7 Floodplain Connectivity

The BiOp geometry integrates floodplain connectivity along with SWH criteria set forth in the 2003 Amended BiOp. Coordination with the USFWS produced a Planning Aid Letter (USFWS 2015) detailing the modeling assumptions for the BiOp alternative. A total of 100,000 acres of SWH and floodplain connectivity were assumed for both districts. To calculate the goal amount of only floodplain connectivity, 22,590 acres (30 ac/mi * 753 mi) was subtracted from the 100,000 acres to obtain 77,410 acres.

Mapping of existing floodplain connectivity was performed by using the RAS model calibrated to 2012 conditions to calculate a water surface profile for the 20% annual chance exceedance event (20% ACE or 5-year). Table 2-16 lists the flow and stage for the 20% ACE or 5-year flow used in the model. The 5-year flow input into the model was obtained from the 2003 Upper Mississippi River System Flow Frequency Study (USACE 2003). Calculated floodplain connectivity acres by state are listed for each district and in total in Table 2-17. Existing floodplain connectivity acres

(147,652 acres) surpass the total acres available for floodplain connectivity (100,000 acres) therefore, no changes were made to the RAS model.

The calculated acres of existing floodplain inundation with connectivity includes:

- Areas lower in elevation than the computed 20% ACE water surface and judged to be connected to the main channel.
- Private lands not protected by levees, including fringe areas between levees and river bank and areas without any discernable protection that would be inundated at the reference flow.

Areas excluded from the existing floodplain connectivity acres:

- Area behind all active/maintained levees, including federal levees, levees in the PL84-99 program and smaller agricultural levees often found between the federal/ program levees and the river bank. No distinction was made as to levee reliability or performance risk.
- Inundated area well outside the bluff line or in tributary backwater areas.
- Missouri River main channel as determined by the boundary of the August 50% duration flow extent (from the 2014 SWH Accounting report).

Table 2-16: 20% ACE (5-yr) Flow and Stage

Location	River Mile	20% ACE (5-year) Flow (cfs)	20% ACE (5-year) Stage (ft, NAVD88)
Ponca	751	64,100	1097.4
Sioux City	734	66,800	1079.4
Decatur	691	70,500	1041.2
Omaha	616	85,300	975.1
Nebraska City	563	118,600	928.1
Rulo	498	132,300	861.4

The 100,000 inundation acreage includes both the main channel and connected floodplain area (shallow water habitat). The 77,410 inundation acreage includes only the connected floodplain. The calculated acres of floodplain connectivity shown in Table 2-17 includes some forms of shallow water habitat (chutes), but excludes others (areas between the river banks). For this reason, the calculated acres were compared to the full portion of authorized acres, and was considered acceptable because existing exceeded authorized even without counting some of the shallow water habitat acres.

Table 2-17: Acres of Floodplain Connectivity by State

State	Acres of Existing Floodplain Connectivity (20% ACE inundation)[2]			Authorized Acres Available for SWH & Floodplain Connectivity	Acres to be added to HEC-RAS
	NWO	NWK	Total[1]		
Iowa	16,120	0	16,120	14,228	0
Kansas	0	8,565	8,565	6,976	0
Missouri	8,020	83,130	91,150	62,813	0
Nebraska	31,550	267	31,817	15,983	0
Total	**55,690**	**91,962**	**147,652**	**100,000[3]**	**0**

1) Does not imply ownership, includes both public and private lands.
2) Does not include the main channel acres defined by median August flows.
3) This total includes both SWH and floodplain connectivity. Floodplain connectivity only is 77,410 acres.

3 FLOW ALTERNATIVES

A total of six flow alternatives were modeled in ResSim. Reservoir pool elevations and dam outflow output from the ResSim model was used as input for the unsteady RAS model for each of the six flow alternatives for the Period of Record (POR). A brief summary of the flow alternatives is provided below. For more details, see the ResSim technical report, *Mainstem Missouri River Reservoir Simulation Alternatives Technical Report* (USACE 2018a).

Tributary and ungaged inflows were kept consistent between alternatives. More details on the POR flow dataset used can be found in the report *Missouri River Recovery Management Plan Time Series Data Development for Hydrologic Modeling* (USACE 2018c).

3.1 ALTERNATIVE 1 - NO ACTION

Under Alternative 1 (Alt 1), the Missouri River Mainstem Projects would continue to be operated as they are currently. Operations within the ResSim model were set up to closely follow the Master Manual that is used during real-time operations of the System; however, the model does have limitations and cannot capture all real-time decisions that occur. This includes a plenary bimodal spawning cue attempt each year, one in March and one in May.

3.2 ALTERNATIVE 2 - U.S. FISH & WILDLIFE SERVICE 2003 BIOLOGICAL OPINION PROJECTED ACTIONS

Alternative 2 (Alt 2) represents the USFWS interpretation of the management actions that would be implemented as part of the 2003 Amended BiOp RPA (USFWS 2003). Operational criteria include different early and late spring spawning cues (March and May), low summer flows, and a maximum winter release limit.

3.3 ALTERNATIVE 3 - MECHANICAL CONSTRUCTION ONLY

Alternative 3 (Alt 3) consists of mechanical construction of emergent sandbar habitat (ESH). Operational criteria consist of removing the early and late spring spawning cues in Alt 1.

3.4 ALTERNATIVE 4 - SPRING HABITAT-FORMING FLOW RELEASE

Under Alternative 4 (Alt 4), the early and late spring spawning cues in Alt 1 are removed from the operational criteria and a spring ESH-creating reservoir release from Gavins Point and Garrison is added. While the ESH-creation release is occurring from Gavins Point, flood targets are increased to allow the ESH-creation release the opportunity to run.

3.5 ALTERNATIVE 5 - FALL HABITAT-FORMING FLOW RELEASE

Alternative 5 (Alt 5) removes the early and late spring spawning cues in Alt 1 and adds a fall ESH-creating reservoir release from Gavins Point and Garrison to the operational criteria. While the ESH-creation release is occurring from Gavins Point, flood targets are increased to allow the ESH-creation release the opportunity to run.

3.6 ALTERNATIVE 6 - PALLID STURGEON SPAWNING CUE

Alternative 6 (Alt 6) replaces the early and late spring spawning cues with different spawning cues. The early spring spawning cue in Alt 6 occurs at the same time as the early spring spawning cue in Alt 1 but with a higher peak release. The late spring spawning cue in Alt 6 occurs later in May than the late spring spawning cue in Alt 1 and has a larger peak release. Please note that the former name of this alternative was Alternative 7, which may correspond to some RAS model runs and file names.

4 SIMULATION OF ALTERNATIVES

Each flow alternative was paired with a geometry alternative to produce six total alternatives that were run through RAS. The No Action geometry was paired with the No Action flow for Alternative 1. The BiOp geometry was combined with the BiOp flow for Alternative 2. The IRC geometry was paired with flow alternatives 3 through 6 to produce Alternatives 3, 4, 5, and 6. See Table 4-1 for a complete listing of the geometry and flow pairings for each alternative.

Table 4-1: Alternative Geometry and Flow Pairings

Alternative	Geometry	Flow
Alternative 1	No Action	No Action
Alternative 2	BiOp	BiOp Projected
Alternative 3	IRC	Mech Only
Alternative 4	IRC	Spring Habitat Forming Release
Alternative 5	IRC	Fall Habitat Forming Release
Alternative 6*	IRC	Pallid Sturgeon Spawning Cue

*Formerly known as Alternative 7

5 RESULTS

All alternative runs were performed in HEC-RAS 5.0.3, except the interior drainage sensitivity runs. Model output contains a considerable amount of information, not easily condensed to simple conclusions. Each of the six alternative runs produced 82 years (March 1930 – December 2012) of stage and flow hydrographs. Responses to the ResSim flow changes in combination with habitat geometry changes are complex. To express the changes compared with the No Action alternative, the model results were evaluated by statistical evaluation and duration analysis plots.

Results from the 82-year runs for the six alternatives were provided to the HC team for analysis. They used the daily (instantaneous 2400 value for each day) flow and water surface elevation output to analyze effects to various resources that include: hydropower, cultural resources, fish & wildlife (exclusive of listed species), flood risk, irrigation, recreation, thermal power, and water supply. The HC team performed an extensive analysis on each of the alternatives for all of the resources and provide a detailed comparison of results. For this report, only the hydraulic model output is presented.

5.1 STATISTICS

For the statistical evaluation, flow and water surface elevation results were analyzed to compare the differences between the No Action Alternative and the remaining five alternatives. Tables showing the differences between calculated statistics for both flow and water surface elevation for below Gavins Point Dam, Sioux City, Omaha, Nebraska City, and Rulo can be seen in Plate 2 and Plate 3. The statistics calculated include: the 10th, 25th, 50th, 75th, and 90th- Percentiles, and the Minimum and Maximum. It should be noted that the percentile statistics calculated are from a duration analysis and not a Bulletin 17B flow frequency analysis.

The min and max are the lowest daily flow or stage and the highest daily flow or stage output for each alternative over the period of record. For model stability, a minimum of 5,000 cfs was used for Gavins Point Dam outflow in RAS. As seen in the tables, the minimum flow or stage for the period of record remains unchanged from alternative to alternative. The maximum flow or stage differences are also essentially unchanged (within 1%) and increase in the downstream direction. Caution should be used when trying to draw conclusions from the statistics alone, especially for peak flows. The economic models (HEC-FIA) provide a more complete analysis of how high flows effect total damages for each alternative because they incorporate all of the cross section output, whereas these tabular statistics only capture one location.

Flow and stage changes between alternatives at a location are influenced by an array of variables. From alternative to alternative the two primary changes to the hydraulic model were flow out of the Mainstem Missouri River reservoir system as calculated by ResSim, and the habitat additions to the river geometry. Flow calculated by the RAS model at a downstream location not only depends on how the Gavins Point release changed, but also how those changes carry downstream. Even when the Gavins Point release has no change, the flow or stage calculated from alternative to alternative at a downstream location may change because of the habitat additions to the river geometry.

It is also important to note that the RAS alternative models, although they have a 30 minute computation interval, have been configured to report one value per 24 hour period, and unfortunately that one value is not a daily average. The RAS model reports the value that lands on 2400 of each day. The most reasonable output interval was chosen as daily due to the size of watershed being modeled, POR length, and the number of hydrograph locations necessary for HC analysis. This means that slight shifts in timing from alternative to alternative can carry over into the results as small fluxuations in the reported flow. Changes in timing are a small factor, not likely to significantly impact any results evaluation, but should be kept in mind when making comparison at a precise level such as in the statistics tables.

Stage statistics have been rounded to the nearest tenth of a foot, which is equivalent to 1.2 inches. This helps demonstrate how flow changes impact river elevations, which is the more tangible result. For example, even though the 50th percentile flow for Nebraska City in Alternative 4 was 193-cfs lower than in Alternative 1, there is less than an inch of impact to the water surface elevation of the river, and therefore zero reported change.

5.2 Seasonal Duration Plots

A duration analysis was also performed for the alternative output. Seasonal duration plots for key main stem locations including Gavins Point, Sioux City, Omaha, Nebraska City, and Rulo are shown in Plate 4 through Plate 23. Seasonal dates chosen for the duration analysis coincide with the current System operational seasons: spring (1Mar to 30Apr), summer (1May to 31Aug), fall (1Sep to 30Nov), and winter (1Dec to 28Feb). The greatest difference can usually be seen in the spring and winter durations due to the spring pulses and resulting lower winter flows.

5.3 Limitations

The analysis relies on the simulation of the 82-year period of record using daily average outflows from a ResSim model input into a fixed bed RAS model, with stage and flow output. While the analysis coupled with species and human considerations models can be used to show relative benefits and potential impacts based on historic flows, there are limitations in the conclusions that can be drawn based on some of the simplifying assumptions.

1. **POR Methodology -** An 82-year period of record, adjusted to current level of depletions, was used and may not be comparable to future conditions. A climate change assessment of the Missouri River basin indicates increases to both temperature and precipitation along with increasing trends in extreme floods and droughts (USACE 2018b). The conditions during a pulse year in the future could vary greatly from the small sample of pulse events included in the POR analysis.
2. **No Risk Analysis -** The Missouri River system as currently operated provides substantial flood damage reduction and benefits to the entire basin. The current ResSim and RAS analysis, which employs an 82 year period of record simulation, shows the potential for negative impacts to flood damage reduction and dam safety for alternatives that include changes in reservoir flow releases. The current study methodology does not simulate a sufficient number of events and possible runoff combinations within the large

Missouri River basin to evaluate potential change in downstream flood risk and dam safety.

Scoping efforts were conducted to determine a Monte Carlo risk analysis methodology capable of assessing impacts to dam safety and flood damage reduction as a result of flow release changes. The risk analysis primary components include further development of the period of record flow data set, ResSim and RAS model modifications, development of levee fragility curves, assignment of uncertainty, assembly and debugging of models, Monte Carlo simulation, analysis of results, and reporting. The Monte Carlo methodology properly assesses the effects of the alternative operation changes because it increases the sample size of flow data and number of combinations of flow periods that may occur in the future so that impacts can be characterized with greater confidence. Without such analysis, the impacts of operational changes will only be known for events and combinations of events that have already occurred. Statistics calculated based on the 82 years of record should therefore be used with caution, and with the understanding of the consequences of using only a small sample of years.

3. **Stable Bed and Floodplain -** The alternative condition hydraulic model geometry included revisions to the existing conditions hydraulic model to account for varying amounts and distributions of habitat through river widening and continued development of SWH projects such as chutes. However, the analysis does not account for how the bed of the Missouri River may respond to river widening activities during the POR simulation. Additionally, the analysis does not try to project where sediment may accumulate in the floodplain or include projections of future change in floodplain roughness that could also occur during the POR simulation. This carries with it the necessary assumptions that any bed and floodplain changes would be either negligible, similar between each alternative, or mitigated during more detailed design of river widening projects.

4. **Flood Source -** The Missouri River, major tributaries, and ungaged inflows included in the RAS model are the only flood sources, water floods areas protected by levees from levee overtopping. In reality, flooding can also occur due to localized rainfall and subsequent runoff, through seepage under levee foundations during prolonged high water, or through failure of levees prior to overtopping. While the level of effort required to model all levees systems with detailed rainfall runoff and underseepage flow calculations was considered prohibitive, more detailed analysis was conducted for four levee systems, two in Omaha District and two in Kansas City District. The two Omaha District interior drainage models are discussed in Section 6.

   Past experience on the Missouri River indicates that a majority of levee breaches occur following levee overtoppings, when water flows into or exits the levee system. However, breaches prior to overtopping have occurred and may occur again in the future. Predicting breach formation through the period of record simulation was considered infeasible due to the unknown location of levee breaches during future events and factors such as repairs and improvements made by levee sponsors or USACE after each event. Levee breaches, and the corresponding increase in depth within the levee protected area, may affect potential flood damages.

5.4 Channel Capacity Analysis

Channel capacity estimates were performed to provide an indication of the flow rate at which bank elevations are overtopped and flow begins to leave the main channel and enter the floodplain. Channel capacity estimates were performed with the one-dimensional RAS model calibrated to 2012 conditions by comparing steady flow profiles with top of bank elevations at each cross section combined with reviewing the best available floodplain topography. Floodplain flow connectivity was not assessed. The estimated channel capacity does not necessarily correlate with the onset of flood damage. In addition, channel capacity is typically highly variable along the channel bank due to wide variation in bank elevations. The quality of the channel capacity estimate is affected by numerous factors including how representative the model cross sections are of river geometry, local channel geometry variation, low spots in bank elevations, and the floodplain topography accuracy. The degradation reach immediately downstream of Gavins Point Dam has a much higher channel capacity than areas further downstream. For that reason, the Nebraska City vicinity was selected as more representative of flood risk for the Gavins Point to Rulo, NE reach. For the Nebraska City area the channel capacity estimate is from 80,000 to 85,000 cfs. Due to the large distance downstream of Gavins Point Dam, a channel capacity estimate was not performed for the river between Rulo, NE and the mouth at St. Louis, MO.

A Monte Carlo risk analysis methodology capable of assessing impacts to dam safety and flood risk as a result of flow release changes would be required to fully assess how an alternative impacts potential flood risk. Refer to the *Summary of Hydrologic Engineering Analysis* (USACE 2018d) for additional details on the risk analysis methodology.

6 INTERIOR DRAINAGE ANALYSIS

An interior drainage analysis was performed for two federal levee units in Omaha District, L-536 and L575. The No Action alternative RAS model geometry (Alternative 1) was modified for these two levee units and the six flow alternatives were run. The geometry modifications included dividing storage areas into smaller sub-basins, sub-dividing lateral structures and adding storage area connections, and adding culverts and pumps. Two lateral inflow hydrographs representing rainfall and levee seepage were also added. A sensitivity analysis was also performed to determine if pumps, culvert sediment, or flow pulses had a large effect on the results.

6.1 Levee Unit L-575 Description

Levee Unit L-575 is located along the left bank of the Missouri River in Fremont County, Iowa; Atchison County, Missouri; and Nemaha County, Nebraska between River Miles 574 and 544. The upstream end of the levee unit includes a left bank tieback on Plum Creek and the downstream end includes a right bank tieback on the Nishnabotna River.

6.2 LEVEE UNIT L-536 DESCRIPTION

Levee Unit L-536 is located along the left bank of the Missouri River in Atchison and Holt Counties, Missouri between River Miles 522 and 516. On the upstream end of the levee unit, there is a tieback along the left bank of Rock Creek and the downstream end includes a tieback on the right bank of Mill Creek.

6.3 BASIN DELINEATION & LATERAL STRUCTURE MODIFICATION

The L-575 drainage area was separated into eight basins and L-536 into four basins for this analysis. Basins were drawn using the best available LiDAR data and previously delineated basins from the Master Manual were used as a guide (USACE 1998). Maps of the L-575 and L-536 basins are shown in Plate 24 and Plate 25.

Lateral structures had to be subdivided in the RAS geometry because lateral structures can only be connected to one storage area. The ArcMap extension GeoRAS was used to extract the data. The lateral structures were updated with the post-2011 flood setback levee alignments, elevations, and culvert information. The National Levee Database (NLD) did not contain the latest information from the setback levees so design elevations were used. Storage area connections were also required for between each of the storage areas. The LiDAR surface was used to extract elevations for the storage area connections.

6.4 CULVERT DATA

The L-536 O&M manual was updated in 2014 and the L-575 O&M manual was in the process of being updated at the time of this study (USACE 2016c). Tables containing updated culvert data were used in this analysis. Several structures had been removed or moved to new setback levees. Structures in place as of 2015 were compiled and input into the HEC-RAS geometry. The list of L-536 and L-575 culverts and the parameters used in RAS are shown in Plate 30 and Plate 31. Culvert locations are shown on the basin map on Plate 24 and Plate 25.

6.5 PUMP DATA

The best available pump data was contained in the updated O&M manual's table of culverts. However, it is noted that pumps are not always maintained and operated by the levee drainage districts so there is very little current information known. Pump information was gathered from the original O&M manual, periodic inspection reports, and previous efforts from the Master Manual Interior Drainage Analysis (USACE 1998). As was done in the Master Manual Interior Drainage Analysis, the pump on elevations were set to the top of the culvert and pump off elevations were set 1 foot below the on elevation. Pump data for both L-575 and L-536 are shown in Plate 32.

HEC-RAS requires a pump capacity curve including losses. Since such little information was known about the pumps, Design Branch, Mechanical Section was tasked with providing pump capacity curves for this analysis. Curves were provided for seven of the eighteen pumps. Multiple curves were very similar and effective duplicates were removed, leaving four total pump curves used in the analysis. The pump capacity curve used for each pump was assigned based on the size given in the O&M manual table. Friction and minor losses were included in the final pump capacity curves. Due to the pumps being at permanent pump stations that go through the levee, the “highest elevation in the pump line” option in RAS was left blank. The pump curves and pump curve loss parameters are shown in Plate 33 and Plate 34.

6.6 Seepage Calculations

Seepage into the interior area during periods of high river stage was accounted for by use of a lateral inflow hydrograph boundary condition to each storage area in the RAS interior drainage model. Each piece of lateral structure bounding the storage areas was analyzed. Longer stretches were further sub-divided into sections of 10,000 ft or less. A reference cross section was chosen to calculate the seepage into each lateral structure piece. Missouri River stage output from each alternative was used to calculate the seepage with a daily time step. Note that higher river levels affect interior groundwater levels, and is not accounted for in this analysis.

A geotechnical engineer provided maximum (at the levee crest) seepage rates for both levee units. The seepage rate follows a linear relationship from the maximum (levee crest) to the zero point (usually the levee toe). For relief wells in levee unit L-536, a head of 2.5 feet was considered the zero point. See Attachment 1, the Geotechnical Seepage Tech Report, *L575 and L536 Seepage Flow Determinations* (USACE 2016a) for further details. For L-575, the seepage rates given were separated into berm, non-berm, and relief well. For L-536, the seepage rates given were separated into levee/berm and relief well. Total lengths of each seepage rate category were calculated for each lateral structure piece.

For each day in the POR, the differential head was calculated and was multiplied by the seepage rate and the length of levee or number of relief wells. The first seepage iteration used the river stage output from Alternative 1. The second seepage iteration used the river stage output from each alternative. The seepage calculations for all of the lateral structure pieces connected to a storage area were added together. The final interior drainage model contained lateral inflow hydrographs of total seepage for each storage area for each alternative.

6.7 Rainfall Data

An HEC-HMS model provided rainfall inflow hydrographs to each storage area for the POR for the two levee unit basins. Each storage area inflow dataset was input into the RAS model as a lateral inflow hydrograph. Unlike the seepage, which had alternative specific inflows, the same rainfall input was used for all of the alternative runs. For more information on the HEC-HMS model and its components, see Attachment 2, *Missouri River Interior Drainage Hydrologic Analysis* (USACE 2016b).

6.8 Results

All interior drainage alternative runs were performed in HEC-RAS 5.0.3, the same version as the full alternative runs. Results from the 82-year runs for the six alternatives were provided to the HC team for analysis. They used the daily storage area elevation output to analyze effects to flood risk. The HC team performed an extensive analysis on each alternative and provide a detailed comparison of results. For this report, only the hydraulic model output is presented.

To express the changes compared with the No Action alternative, the water surface elevation model results were evaluated by the same statistical evaluations made on the full models. All of the alternatives show minor differences between water surface elevation. Tables showing differences between the calculated statistics for water surface elevation for the twelve storage areas can be seen in Plate 26. The statistics calculated include: the 10th, 25th, 50th, 75th, and 90th-Percentiles, and the Minimum and Maximum. Alternative 2 generally has the greatest impact (higher water surface elevations) among the alternatives.

An example output can be seen in Plate 28. It shows the interior storage area (575A) and exterior (cross section 571.2) water surface elevations. Higher river water surface elevations cause flap gates to close and produce higher water surface elevations in the interior area.

The interior drainage models provide a powerful tool to assess the complicated interaction between reservoir releases upstream of a levee unit on interior ponding resulting from rainfall and/or seepage. However, limited conclusions can be extrapolated to the entire river. With such slight differences compared to No Action, it is difficult to separate the impact of flow changes from the reservoirs from the impact of added habitat to make global conclusions. Changes are highly localized, depending upon factors such as how low the culvert outlet is and the interior area available for ponding before damages occur.

6.9 Sensitivity Analysis and Results

A series of sensitivity runs were conducted for the interior drainage analysis. They included three actions either alone or in combination: all pumps removed, culverts filled with sediment halfway, and a simulated pulse every year (spring or fall, depending on the alternative). Unlike the rest of the models, the sensitivity runs were not re-run in HEC-RAS 5.0.3 due to time constraints and the assumption that the relative results would be similar. Reasons for running each sensitivity are as follows:

- All of the pumps removed - conducted due to the uncertainty with the pumps' physical and operational information.
- Culverts half filled with sediment - conducted because of a comparison to limited temporary gage data revealed that the model's water surface elevations were generally low.
- Simulated pulse every year - conducted to analyze if the timing and amount of pulses had a large effect on the results.

An example output plot from the interior drainage sensitivity is shown in Plate 29. Adding the culvert sediment increased the minimum elevation that the storage area could drain down to while taking out all of the pumps increased the water surface elevation when the flap gates were blocked.

The simulated pulse every year was achieved by setting the flow below the Platte River to be 75,000 cfs during the pulse timeframes (either spring or fall) with appropriate ramp up and ramp down time. 75,000 cfs was chosen because it is just below the flood target of 82,000 cfs for

Nebraska City in the ResSim model. As seen in Figure 6-1, a flow of 75,000 cfs produces a profile that either partially or fully covers all of the flap gates on L-536 and L-575 (circled in red).

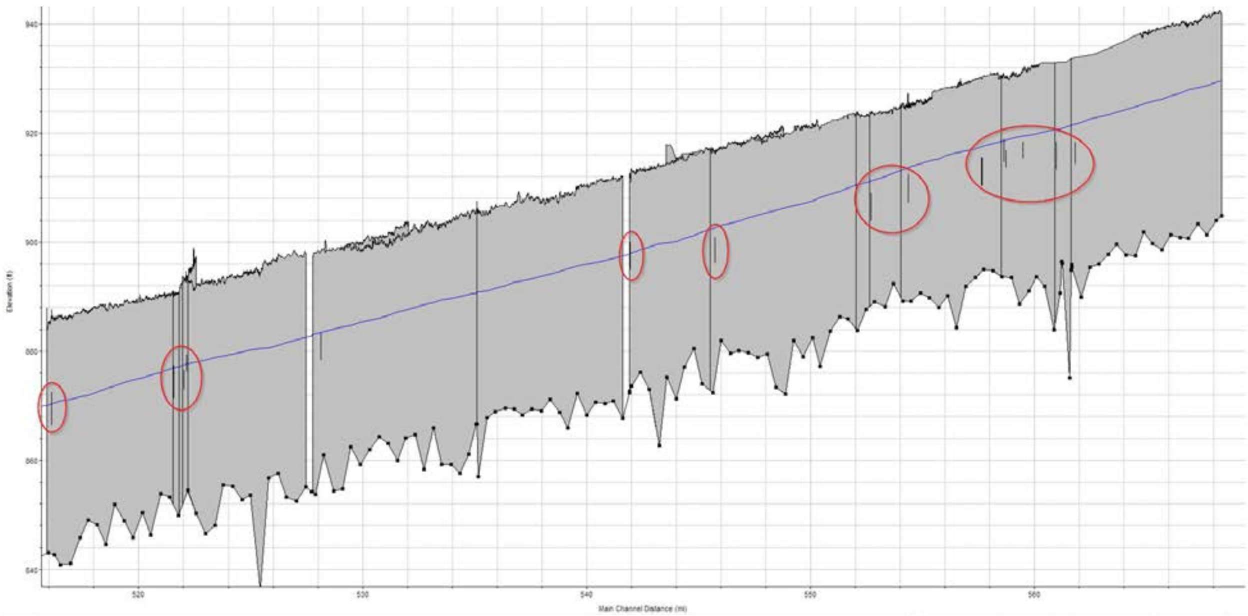


Figure 6-1: 75,000 cfs Profile and Flap Gates

Sensitivity analyses indicated that changes in assumptions about the interior drainage structures have much larger effects than the differences between the alternatives. The results from the sensitivity runs were analyzed and revealed that taking out all of the pumps had the most effect on the storage area water surface elevations. The culvert sediment and pulse every year runs produced minimal differences compared to the no pumps run. Statistics of the results and the difference between them and the matching alternative's results can be found in Plate 27. The risk of flooding from interior drainage is much more a function of the conditions of the interior drainage structures (culverts and pumps) at specific sites. Alternative 2 has relatively larger impacts compared with Alternatives 3-6.

A timestep sensitivity analysis was also performed for the interior drainage model. The timestep used for the alternatives and interior drainage models was 30 minutes. A sensitivity run using a 2-minute timestep was conducted and compared to the original output. A comparison of the annual max stage timeseries was performed since the flood risk analysis (HEC-FIA) uses peak stages only. As seen in Table 6-1, for the cross sections and storage areas, the annual max results were generally within 0.05 ft. There is a max difference of 0.1 ft for the cross sections and -0.35 ft for the storage areas. Another factor considered with the timestep sensitivity is the total model runtime. The original interior drainage model (30-min timestep) took 6.5 hours to run while the 2-min timestep model took almost 10 times longer at 63 hours. This was not a feasible run time to conduct numerous runs in a reasonable amount of time.

Table 6-1: Timestep Sensitivity Results

	Stage - Annual Max (ft)		
XS	**Min**	**Max**	**Mean**
591.15	-0.0005	0.002	0.0006
580.98	-0.0007	0.01	0.002
570.83	-0.0009	0.02	0.003
562.74	0.00006	0.02	0.005
560.32	0.00006	0.02	0.005
550.61	-0.001	0.04	0.008
540.15	-0.002	0.04	0.009
530.01	-0.002	0.06	0.01
520.25	-0.002	0.09	0.02
510.03	-0.003	0.1	0.02
500.15	-0.0007	0.1	0.02
SA			
575 A	-0.05	0.05	0.003
575 B	-0.18	0.05	-0.005
575 C	-0.009	0.02	0.002
575 D	-0.02	0.02	0.002
575 E	-0.06	0.07	0.002
575 F	-0.11	0.05	0.0004
575 G	-0.02	0.06	0.007
575 H	-0.35	0.09	-0.009
536 A	-0.03	0.07	0.005
536 B	-0.02	0.05	0.003
536 C	-0.03	0.11	0.006
536 D	-0.04	0.05	0.003

7 ADDITIONAL ANALYSIS FOR YEAR 15

Degradation and aggradation of the Missouri River channel bed and sedimentation in the reservoirs are ongoing processes, which have the potential to effect virtually all economic resources and human considerations. Therefore, additional modeling was performed to provide estimates of how ongoing sedimentation processes may affect the future performance of alternatives. The future without and with project condition modeling is based on a number of critical assumptions regarding historic trends, flows, and sediment inputs. While not intended to represent detailed estimates of future channel conditions, the results do provide an alternative comparison methodology. The designation "Year 15" comes from the timeframe for implementation; the ROD for the MRRPMP-EIS is expected to be signed in 2018, with a construction completion date of 2033, resulting in an implementation period of 15 years. Results from the Year 15 analysis were provided to economists and human considerations teams for qualitative evaluation, versus the full quantitative evaluation that was performed on the base condition (also referred to as Year 0).

7.1 YEAR 15 FUTURE CONDITIONS

To project river bed aggradation and degradation trends to the year 2033, moveable bed sediment models of the mainstem Missouri River reaches were created in RAS. Results from the sediment modeling provided a projected change in water surface at a normal flow for each of the six alternatives. Full details on construction and calibration of the sediment model are in Attachment 3, Missouri River Unsteady HEC-RAS Model Sediment Analysis, Gavins Point Dam to Rulo.

7.2 ALTERNATIVE ANALYSIS

Alterations were made to both the ResSim and RAS models to represent conditions at the end of the implementation period. All six alternatives were re-run, and results were compared between alternatives and to the base condition. All Year 15 alternatives runs were performed using HEC-RAS version 5.0.3 in October 2017.

7.2.1 Geometry

In RAS, the Missouri River channel bed was adjusted to represent the degradation that was estimated for the end of the implementation period. Water surface change at a normal flow of 30,000 cfs was used to estimate bed change. Alternative 2 produced slightly different future water surface change estimates than the rest of the alternatives so two estimates were used, one for alternative 2 and one for alternatives 1 and 3 through 6. The differences were above Sioux City or RM 733. The sediment models erroneously projected aggradation below RM 536, so the estimated water surface change was set to zero from this point to the end of the reach. To produce the aniticipated degradation, the channel points between the banks were adjusted vertically downward. Figure 7-1 shows a cross section that was adjusted for anticipated degradation.

For areas of forecasted degradation, a simplified RAS sediment model was used to produce the desired water surface change at a normal flow of 30,000 cfs. Minimum channel elevations were adjusted until the model changed the water surface the desired amount. The water surface changes were generally within 0.1 ft of the estimate. A plot of the water surface change estimate vs what the future geometry actually produces is shown in Figure 7-2.

Figure 7-1. Cross Section Vertical Adjustment for Year 15 - Degradation

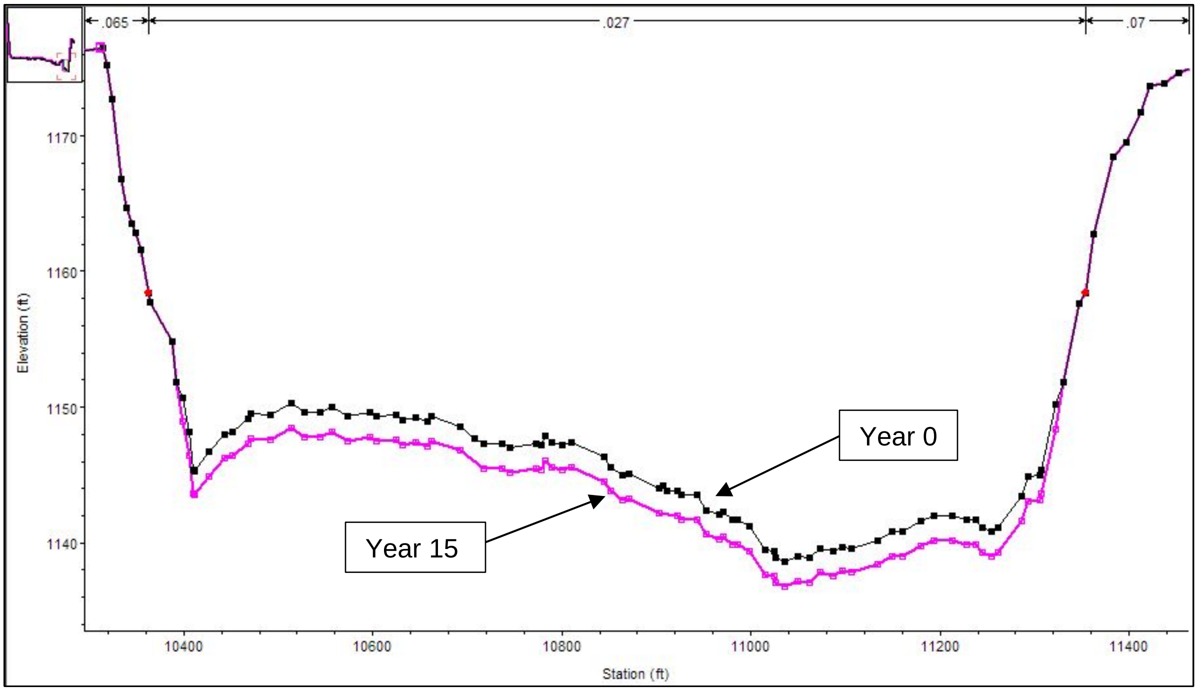
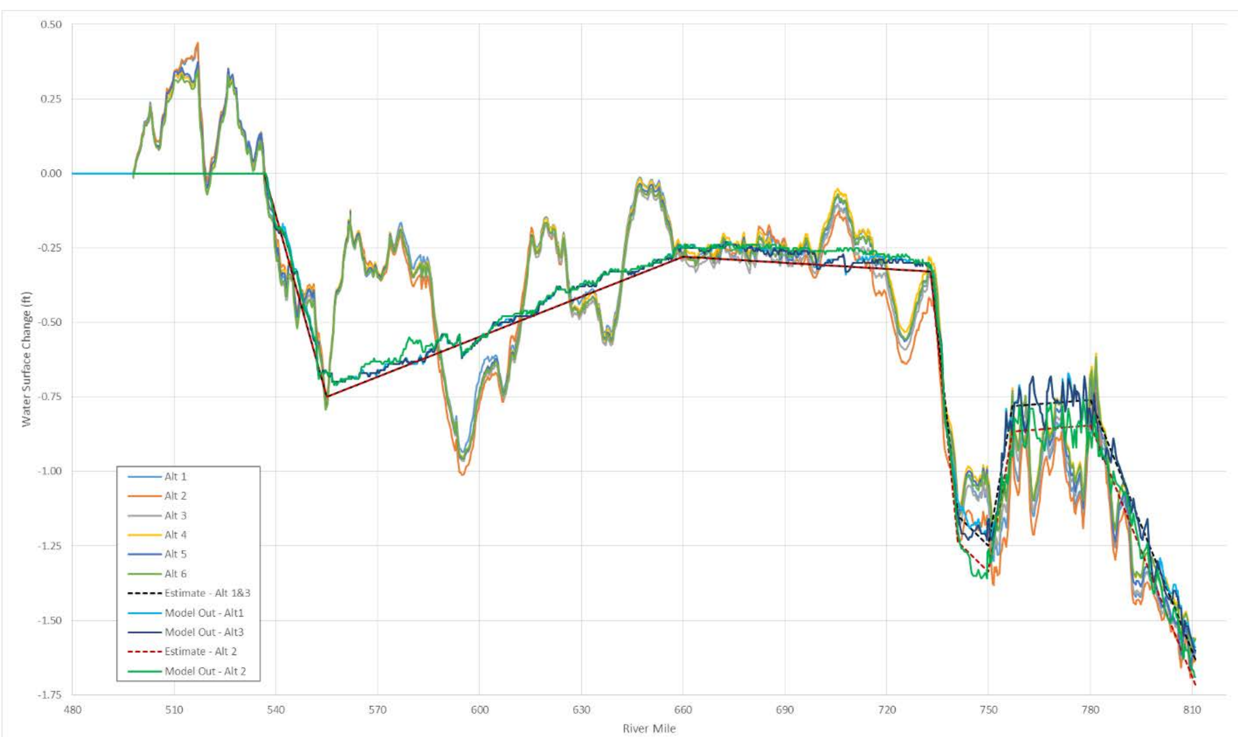


Figure 7-2. Year 15 Water Surface Change Estimates vs. Actual



7.2.2 Flows

Reservoir storage volumes were adjusted in ResSim to represent sedimentation in the reservoirs that could be expected by the end of the implementation period. Reservoir operation rules were left the same as the base set of alternatives. All six alternatives were re-run in ResSim for the period of record, resulting in slightly different release decisions from the System. Refer to *Mainstem Missouri River Reservoir Simulation Alternatives Technical Report* (USACE 2018a) for details regarding the ResSim modeling.

7.2.3 BSTEM Analysis and Results

In order to assess potential variation in bank erosion rates as a result of change in flow releases from study alternatives, a bank erosion model was assembled for evaluation in the 60 mile long reach downstream of Gavins Point Dam. The downstream channel has intermittent bank protection features and has experienced significant bank erosion rates since data collection began in the 1930's. Downstream of Ponca, NE, the channel transitions to the Missouri River Bank Stabilization and Navigation Project (BSNP) and banks are protected with rock structures.

Bank erosion modeling was performed using the Bank-Stability and Toe Erosion Model (BSTEM) within HEC-RAS. BSTEM was developed by the National Sediment Laboratory of the USDA's Agricultural Research Station and is a physically based model that accounts for the dominant stream bank processes. The bank erosion analysis was performed with a separate model from the sediment modeling effort due to the model complexity and run time length. Bank erosion computed with the model was used as a sediment input to the reach wide sediment modeling effort. Refer to Attachment 3, Missouri River Unsteady HEC-RAS Model Sediment Analysis, for additional details on assembly of the BSTEM model, results, and how bank erosion rates were incorporated into the Gavins to Rulo sediment model.

The POR flows for the various alternatives were simulated with the BSTEM model and bank erosion rates were compared in the reach from Gavins Point Dam downstream to Ponca, NE which is a distance of about 60 river miles.

The bank erosion analysis determined that the bank erosion total volume percent change from the base condition alternative 1 varied by less than 1 percent for all alternatives. Alternatives that included flow changes (2, 4, 5, and 6) resulted in slightly increased bank erosion volumes while the alternative 3 change was about 0.1% less than the alternative 1 base condition. While results indicate that bank erosion rates in the Gavins to Ponca reach are slightly sensitive to the Gavins Point Dam releases, the small change computed by the model indicates that the variation in bank erosion rates as a result of the flow change alternatives is projected to be minor. The comparison of bank erosion volume from model results are summarized for the entire reach, localized variation may occur.

7.2.4 Sediment Model Results

Output from the Year 15 analysis can be evaluated in two ways. First, comparison of the Year 15 alternatives to the Year 15 No Action provides additional data to inform on how alternative performance may vary in the future for consideration with the selection of a preferred alternative.

Second, comparison of the Year 15 alternatives to the base condition provides a sense of how future channel bed and reservoir sedimentation conditions may impact Missouri River flows and stages. The second evaluation is limited in the useful information it can provide to the decision making process, as the results are directly tied to modeling assumptions that were made about an unknown future using historic data. The first evaluation parallels the comparisons made amongst alternatives in the base condition analysis. Visual and statistical evaluation of the Year 15 output indicates that regardless of changed conditions, the alternatives compare similarly to each other.

Statistical whisker plots for Gavins Point Dam, Sioux City, Omaha, and Nebraska City are shown in Figure 7-3 through Figure 7-6. The figures present the statistics of minimum, maximum, 5th percentile, 95th percentile, median and mean for all six alternatives for the base condition (Year 0) and Year 15 (Long 2017).

Figure 7-3. Flow Statistics at Gavins Point Dam

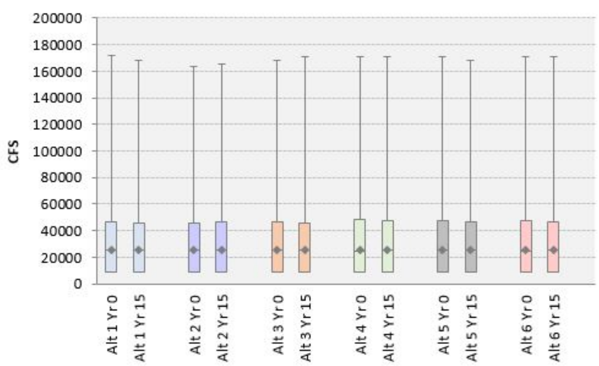


Figure 7-4. Flow and Stage Statistics at Sioux City

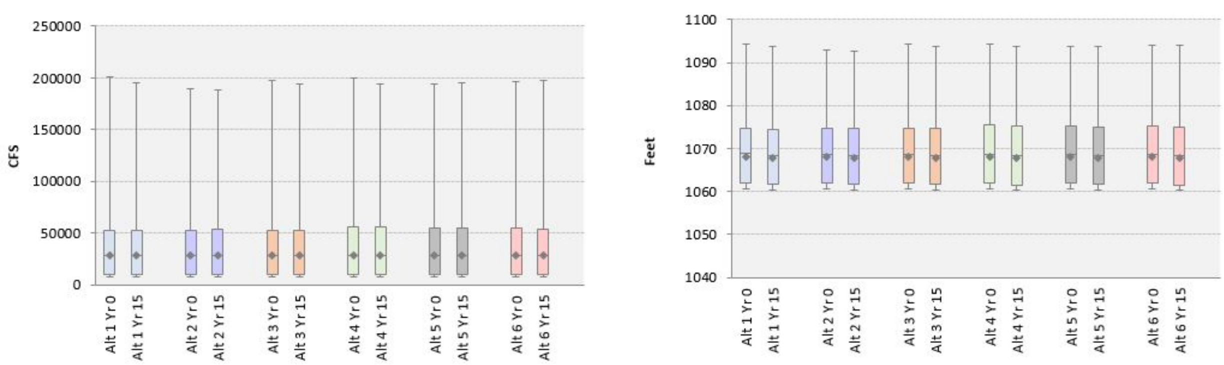


Figure 7-5. Flow and Stage Statistics at Omaha

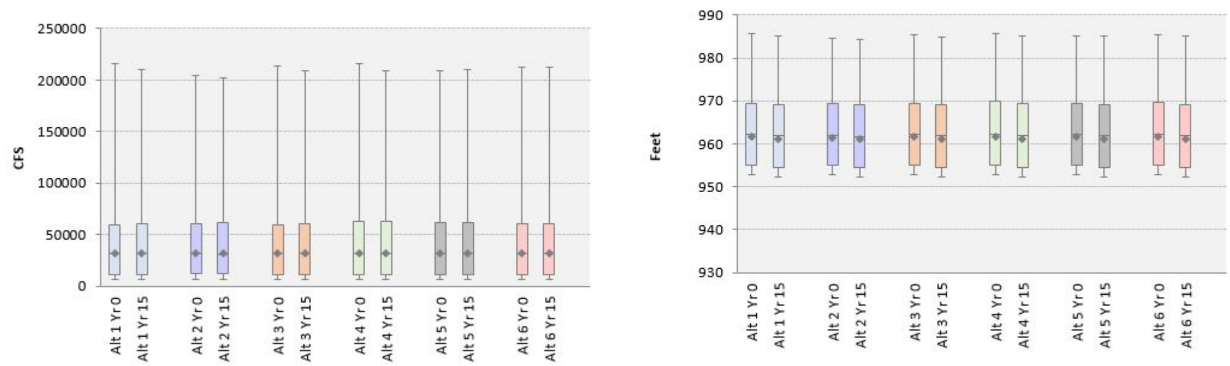
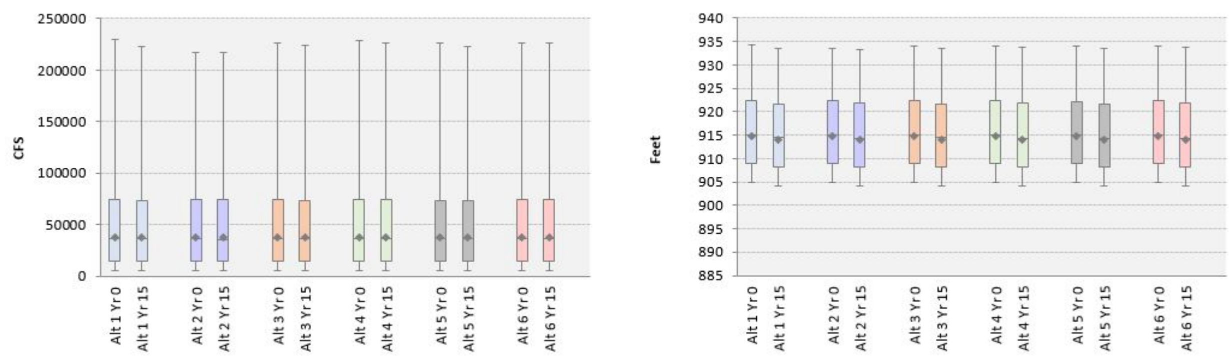


Figure 7-6. Flow and Stage Statistics at Nebraska City



Although comparisons between Year 15 and Year 0 have limited usefulness in alternative selection, a few trends are worth pointing out. First, differences in flow statistics between Year 0 and Year 15 are almost negligible. Any differences that do show up can be attributed to release changes due to different storage volumes in the reservoir system. These differences decrease moving downstream. Other flow differences compared to Year 0 could be due to slightly different routing because of changed channel capacities. These would accumulate and be greater moving downstream, but they are slight in magnitude and pretty difficult to discern from model noise. Second, differences in stage statistics between Year 0 and Year 15 are directly related to the vertical bed change applied locally. As stated previously, the relative comparison between alternatives for Year 15 produces very similar results to the base condition analysis.

Previously discussed limitations for the alternative analysis also apply to the Year 15 analysis. Although the Year 15 model geometry includes an adjustment to the river bed, the model geometry is static and does not change during the 82 year POR.

7.3 Interior Drainage Analysis

Flow hydrographs and modified cross sections from the Year 15 analysis were incorporated into the interior drainage model. The model was re-run for the period of record, seepage was re-calculated and the period of record was re-run including seepage. All Year 15 alternatives runs were performed in HEC-RAS 5.0.3 in December 2017. Model output was compared between alternatives and to the base condition. Cross sections in the vicinity of both levee units were modified for degradation, resulting in lower water surface levels along the levee, and therefore lower ponding levels in the leveed area for the No Action and all alternatives. However, the relative comparison between alternatives for Year 15 produces very similar results to the base condition analysis. This result is expected since the primary factors that would alter interior drainage analysis results between alternatives, different river levels that would affect seepage rates and gravity flow through interior drainage connections, were also similar in Year 15.

8 CONCLUSIONS

The unsteady RAS model analysis gives a means to systematically evaluate differences in river elevations for various reservoir and habitat alternatives given the limitations presented in Section 5.3. These results can be fed into additional species and human considerations models, such as HEC-FIA, to screen alternatives for relative benefits and potential economic impacts. The outputs should be carefully examined with an eye towards the model limitations and judgment applied where needed to mitigate any potential pitfalls of the hydraulic analysis. An advantage to the alternative modeling in this study is the ability to account for differences in flow routings and river stages with varying amounts and distributions of habitat.

Additional modeling was performed to provide estimates of how ongoing sedimentation processes may affect the performance of future alternatives. While not intended to represent detailed estimates of future channel conditions, the Year 15 results do provide an alternative comparison methodology that was evaluated qualitatively rather than quantitatively for economic and human consideration impacts. If flow change alternatives are considered for implementation, additional risk and uncertainty analysis is recommended to more comprehensively quantify risk of spring or fall pulse flows. If flow change alternatives are considered for implementation, additional risk and uncertainty analysis is recommended to more comprehensively quantify risk of spring or fall pulse flows.

As part of the sedimentation process modeling, a bank erosion analysis was conducted in the reach downstream of Gavins Point Dam. Analysis was conducted to determine if bank erosion rates would vary between alternatives. Sensitivity analysis was included to determine if altering bank erosion rate parameters could affect the alternative relative rank. The BSTEM analysis determined that all alternatives differed from the no action by less 1 percent. With respect to erosion rates, alternatives that included flow changes (2, 4, 5, and 6) resulted in increased bank erosion rates while alternative 3 was about 0.1% less than the alternative 1 base condition.

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APPENDIX D

GAVINS POINT DAM TO RULO, NE

PLATES

River	Reach	RS	(Alt 1) Aug 50% Flow (cfs)	(Alt 2) Summer Low Flow (cfs)	(Alt 2) Mid-August Flow (cfs)	(Alt 2) Spring Pulse (cfs)	(Alt 3-6) Mid-June Flow (cfs)
Big Nemaha	River	13.66	201	574	399	599	674
Big Sioux River	Gage to Mouth	50.93	643	148	115	216	229
Boyer River	Gage to Mouth	15.74	299	152	100	141	187
James River	combined	55.606	273	1331	1029	1936	2057
Little Nemaha	Gage to Mouth	10.52	241	203	139	235	289
Little Sioux	REACH # 8	13.51	765	1119	732	1033	1378
Missouri River	Gavins to James	810.87	32627	21000	36900	48800	32200
Missouri River	Gavins to James	805.77	32607	21001	36901	48802	32202
Missouri River	James to Vermill	800.58	32880	22332	37930	50738	34259
Missouri River	James to Vermill	787.64	32884	22375	37963	50800	34325
Missouri River	James to Vermill	779.17	32852	22472	38038	50941	34474
Missouri River	Verm to BigSux	771.77	32874	22556	38103	51064	34605
Missouri River	Verm to BigSux	762.98	32839	22582	38123	51102	34645
Missouri River	Verm to BigSux	753.93	32804	22608	38144	51140	34686
Missouri River	Verm to BigSux	745.52	32802	22633	38163	51176	34724
Missouri River	Verm to BigSux	737.11	32791	23048	38483	51778	35365
Missouri River	BigSux to LilSux	734.1	33434	23196	38598	51994	35594
Missouri River	BigSux to LilSux	732.37	33428	23200	38600	52000	35600
Missouri River	BigSux to LilSux	732.17	33438	23221	38614	52020	35626
Missouri River	BigSux to LilSux	730.98	33582	23504	38799	52281	35975
Missouri River	BigSux to LilSux	724.87	33678	23551	38830	52324	36033
Missouri River	BigSux to LilSux	719.6	33775	23591	38856	52362	36083
Missouri River	BigSux to LilSux	713.98	33858	23615	38872	52384	36113
Missouri River	BigSux to LilSux	706.3	33978	23648	38893	52414	36153
Missouri River	BigSux to LilSux	697	34123	23688	38919	52451	36202
Missouri River	BigSux to LilSux	691.03	34221	23707	38931	52468	36225
Missouri River	BigSux to LilSux	683.21	34338	23825	39008	52577	36370
Missouri River	BigSux to LilSux	676.28	34441	23929	39076	52673	36498
Missouri River	BigSux to LilSux	669.82	34673	24026	39140	52763	36618
Missouri River	LSux to Soldier	669.32	35438	25145	39872	53796	37996
Missouri River	LSux to Soldier	664.94	35527	25191	39902	53839	38053
Missouri River	Soldier to Boyer	664.03	35628	25322	39987	53960	38214
Missouri River	Soldier to Boyer	654.88	35765	25356	40009	53991	38256
Missouri River	Soldier to Boyer	649.16	35869	25377	40023	54011	38282
Missouri River	Soldier to Boyer	647.55	35916	25419	40050	54049	38333
Missouri River	Soldier to Boyer	641.55	36011	25564	40144	54182	38511

Plate 1: RAS Reference Flow Input

River	Reach	RS	(Alt 1) Aug 50% Flow (cfs)	(Alt 2) Summer Low Flow (cfs)	(Alt 2) Mid-August Flow (cfs)	(Alt 2) Spring Pulse (cfs)	(Alt 3-6) Mid-June Flow (cfs)
Missouri River	Boyer to Platte	635.22	36310	25716	40244	54323	38698
Missouri River	Boyer to Platte	627.3	36430	25760	40272	54363	38752
Missouri River	Boyer to Platte	621.75	36543	25790	40292	54391	38789
Missouri River	Boyer to Platte	616.03	36632	25800	40300	54400	38800
Missouri River	Boyer to Platte	609.89	36665	25813	40309	54419	38822
Missouri River	Boyer to Platte	605.49	36725	25822	40315	54432	38837
Missouri River	Boyer to Platte	596.48	36833	25852	40335	54475	38886
Missouri River	Platte to Weepin	595	39806	30749	43665	61624	47014
Missouri River	Platte to Weepin	587.4	39874	30766	43676	61648	47042
Missouri River	Platte to Weepin	580.98	39910	30776	43683	61663	47059
Missouri River	Platte to Weepin	574.1	39949	30787	43690	61679	47077
Missouri River	Weeping to Nishn	568.7	40015	30796	43696	61691	47091
Missouri River	Weeping to Nishn	562.35	40049	30800	43700	61700	47100
Missouri River	Weeping to Nishn	555.14	40201	31215	43984	62181	47690
Missouri River	Weeping to Nishn	547.81	40354	31637	44273	62670	48291
Missouri River	Nishna to LilNem	542.02	41298	31971	44501	63056	48765
Missouri River	Nishna to LilNem	535.7	41426	32134	44612	63244	48996
Missouri River	LilNemah to Tark	527.8	41667	32337	44751	63479	49285
Missouri River	LilNemah to Tark	522.29	41798	32400	44794	63552	49375
Missouri River	LilNemah to Tark	517.42	41896	32487	44854	63653	49499
Missouri River	LilNemah to Tark	511.94	42009	32585	44921	63766	49638
Missouri River	Tarkio to BigNem	507.68	42165	32661	44973	63854	49746
Missouri River	Tarkio to BigNem	498.04	42362	32700	45000	63900	49800
NISHNABOTNA	Nishnabotna	61570	944	334	228	386	474
Nodaway	Nodaway	28.91	100	100	100	100	101
Platte River	Platte River	16.74	2973	4897	3330	7149	8128
SOLDIER	REACH # 10	12.28	101	131	85	121	161
Tarkio	Gage to Mouth	13.56	156	76	52	88	108
VERMILLION	REACH # 4	10.75	22	84	65	123	131
WEEPING	REACH # 16	6.5	66	9	6	12	14

Plate 1 cont'd: RAS Reference Flow Input

Flow (cfs)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Gavins Point - XS 810.87							
Alt 1A	9,000	12,500	25,500	35,000	42,000	9,000	171,500
Alt 2A	9,500	13,000	25,500	34,500	42,000	9,000	163,500
Change from Alt 1A	500	500	0	-500	0	0	-8,000
Alt 3A	9,000	13,000	25,000	35,000	42,000	9,000	168,500
Change from Alt 1A	0	500	-500	0	0	0	-3,000
Alt 4A	9,000	12,500	25,000	35,000	42,500	9,000	171,000
Change from Alt 1A	0	0	-500	0	500	0	-500
Alt 5A	9,000	12,500	25,000	34,500	42,000	9,000	170,500
Change from Alt 1A	0	0	-500	-500	0	0	-1,000
Alt 6A	9,000	12,500	25,000	35,000	42,500	9,000	170,500
Change from Alt 1A	0	0	-500	0	500	0	-1,000
Sioux City - XS 732.37							
Alt 1A	12,090	14,942	28,747	37,180	45,288	7,690	200,765
Alt 2A	12,161	14,976	28,319	37,326	45,970	7,690	189,936
Change from Alt 1A	71	34	-428	146	683	0	-10,829
Alt 3A	12,102	14,979	28,653	37,156	45,338	7,690	197,819
Change from Alt 1A	12	37	-94	-24	51	0	-2,946
Alt 4A	12,018	14,613	28,423	37,105	46,439	7,690	200,273
Change from Alt 1A	-72	-329	-324	-75	1,151	0	-492
Alt 5A	12,088	14,925	28,459	36,991	45,344	7,690	193,967
Change from Alt 1A	-2	-17	-288	-189	56	0	-6,798
Alt 6A	12,015	14,658	28,405	37,442	46,313	7,690	196,820
Change from Alt 1A	-75	-284	-342	262	1,025	0	-3,945
Omaha - XS 615.99							
Alt 1A	13,138	17,045	31,903	39,871	49,615	6,914	216,232
Alt 2A	13,212	16,997	31,413	39,985	50,270	6,914	204,264
Change from Alt 1A	74	-49	-490	114	654	0	-11,968
Alt 3A	13,156	17,102	31,821	39,900	49,636	6,914	213,321
Change from Alt 1A	18	57	-82	29	20	0	-2,911
Alt 4A	13,052	16,668	31,642	39,776	51,259	6,914	215,735
Change from Alt 1A	-86	-377	-261	-95	1,643	0	-497
Alt 5A	13,131	17,039	31,685	39,817	49,766	6,914	209,176
Change from Alt 1A	-7	-6	-218	-55	150	0	-7,057
Alt 6A	13,049	16,612	31,727	40,089	50,922	6,914	212,175
Change from Alt 1A	-89	-433	-176	218	1,307	0	-4,058

Plate 2: Alternative Flow Statistics from POR Duration

Flow (cfs)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Nebraska City - XS 562.74							
Alt 1A	16,584	22,283	36,551	45,221	59,525	5,549	229,324
Alt 2A	16,686	22,173	36,094	45,261	59,906	5,549	217,215
Change from Alt 1A	102	-110	-457	40	381	0	-12,108
Alt 3A	16,605	22,337	36,494	45,158	59,612	5,549	226,313
Change from Alt 1A	21	54	-57	-63	86	0	-3,010
Alt 4A	16,460	21,995	36,358	45,163	61,514	5,549	228,941
Change from Alt 1A	-124	-287	-193	-58	1,989	0	-382
Alt 5A	16,566	22,272	36,382	45,157	60,159	5,549	226,145
Change from Alt 1A	-18	-11	-169	-64	633	0	-3,179
Alt 6A	16,450	21,942	36,448	45,423	60,734	5,549	226,145
Change from Alt 1A	-134	-341	-103	203	1,209	0	-3,179
Rulo - XS 498.07							
Alt 1A	17,126	23,381	37,787	47,383	63,704	5,435	275,149
Alt 2A	17,243	23,224	37,318	47,484	63,986	5,436	276,485
Change from Alt 1A	118	-157	-468	101	282	0	1,336
Alt 3A	17,154	23,416	37,713	47,331	63,743	5,435	274,189
Change from Alt 1A	29	36	-74	-52	39	0	-961
Alt 4A	16,979	23,055	37,589	47,645	65,329	5,436	274,180
Change from Alt 1A	-147	-325	-198	262	1,625	0	-969
Alt 5A	17,117	23,334	37,612	47,394	64,594	5,435	274,182
Change from Alt 1A	-9	-47	-175	10	890	0	-968
Alt 6A	16,971	23,022	37,677	47,763	64,631	5,436	274,178
Change from Alt 1A	-155	-359	-110	380	927	0	-971

Plate 2 cont'd: Alternative Flow Statistics from POR Duration

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Gavins Point - XS 810.87							
Alt 1A	1151.6	1152.6	1155.6	1157.3	1158.5	1151.4	1173.6
Alt 2A	1151.7	1152.7	1155.6	1157.2	1158.4	1151.4	1172.9
Change from Alt 1A	0.1	0.0	0.0	0.0	0.0	0.0	-0.7
Alt 3A	1151.6	1152.6	1155.6	1157.3	1158.5	1151.4	1173.3
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	-0.3
Alt 4A	1151.6	1152.5	1155.5	1157.3	1158.6	1151.4	1173.5
Change from Alt 1A	0.0	-0.1	-0.1	0.0	0.1	0.0	0.0
Alt 5A	1151.6	1152.6	1155.5	1157.3	1158.5	1151.4	1173.4
Change from Alt 1A	0.0	0.0	-0.1	0.0	0.0	0.0	-0.1
Alt 6A	1151.6	1152.5	1155.5	1157.3	1158.6	1151.4	1173.4
Change from Alt 1A	0.0	-0.1	-0.1	0.0	0.2	0.0	-0.1
Sioux City - XS 732.37							
Alt 1A	1062.6	1063.9	1068.8	1071.1	1073.1	1060.6	1094.4
Alt 2A	1062.7	1063.9	1068.6	1071.0	1073.1	1060.6	1093.0
Change from Alt 1A	0.0	0.0	-0.2	-0.1	0.0	0.0	-1.4
Alt 3A	1062.6	1063.9	1068.8	1071.1	1073.1	1060.6	1094.2
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	-0.2
Alt 4A	1062.6	1063.7	1068.7	1071.1	1073.4	1060.6	1094.4
Change from Alt 1A	0.0	-0.1	-0.1	0.0	0.3	0.0	0.0
Alt 5A	1062.6	1063.9	1068.7	1071.1	1073.1	1060.6	1093.9
Change from Alt 1A	0.0	0.0	-0.1	0.0	0.0	0.0	-0.5
Alt 6A	1062.6	1063.7	1068.7	1071.2	1073.4	1060.6	1094.1
Change from Alt 1A	0.0	-0.1	-0.1	0.1	0.3	0.0	-0.3
Omaha - XS 615.99							
Alt 1A	955.5	957.1	962.3	964.4	966.9	952.6	985.7
Alt 2A	955.5	957.0	962.0	964.3	966.8	952.6	984.6
Change from Alt 1A	0.0	0.0	-0.3	-0.2	-0.1	0.0	-1.2
Alt 3A	955.5	957.1	962.3	964.4	966.9	952.6	985.5
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	-0.2
Alt 4A	955.5	956.9	962.3	964.4	967.3	952.6	985.7
Change from Alt 1A	0.0	-0.1	-0.1	0.0	0.4	0.0	0.0
Alt 5A	955.5	957.0	962.3	964.4	967.0	952.6	985.2
Change from Alt 1A	0.0	0.0	-0.1	0.0	0.1	0.0	-0.5
Alt 6A	955.5	956.9	962.3	964.5	967.2	952.6	985.4
Change from Alt 1A	0.0	-0.2	-0.1	0.1	0.4	0.0	-0.3

Plate 3: Alternative Water Surface Elevation Statistics from POR Duration

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Nebraska City - XS 562.74							
Alt 1A	909.6	911.4	915.0	916.9	919.7	904.9	934.2
Alt 2A	909.6	911.4	914.9	916.9	919.8	904.9	933.5
Change from Alt 1A	0.0	0.0	-0.1	0.0	0.1	0.0	-0.6
Alt 3A	909.6	911.4	915.0	916.9	919.8	904.9	934.1
Change from Alt 1A	0.0	0.0	0.0	0.0	0.1	0.0	-0.1
Alt 4A	909.6	911.3	915.0	916.9	920.1	904.9	934.2
Change from Alt 1A	0.0	-0.1	0.0	0.0	0.4	0.0	0.0
Alt 5A	909.6	911.4	915.0	916.9	919.9	904.9	934.0
Change from Alt 1A	0.0	0.0	0.0	0.0	0.2	0.0	-0.1
Alt 6A	909.6	911.3	915.0	916.9	920.0	904.9	934.0
Change from Alt 1A	0.0	-0.1	0.0	0.1	0.3	0.0	-0.1
Rulo - XS 498.07							
Alt 1A	841.7	843.2	846.8	848.7	851.8	837.5	865.8
Alt 2A	841.7	843.2	846.7	848.7	852.0	837.5	865.9
Change from Alt 1A	0.0	0.0	-0.1	0.0	0.2	0.0	0.0
Alt 3A	841.7	843.2	846.8	848.7	851.8	837.5	865.8
Change from Alt 1A	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Alt 4A	841.7	843.2	846.8	848.7	852.5	837.5	865.8
Change from Alt 1A	0.0	-0.1	-0.1	0.0	0.7	0.0	0.0
Alt 5A	841.7	843.2	846.8	848.7	852.2	837.5	865.8
Change from Alt 1A	0.0	0.0	0.0	0.0	0.5	0.0	0.0
Alt 6A	841.7	843.2	846.8	848.8	852.1	837.5	865.8
Change from Alt 1A	0.0	-0.1	0.0	0.1	0.4	0.0	0.0

Plate 3 cont'd: Alternative Water Surface Elevation Statistics from POR Duration

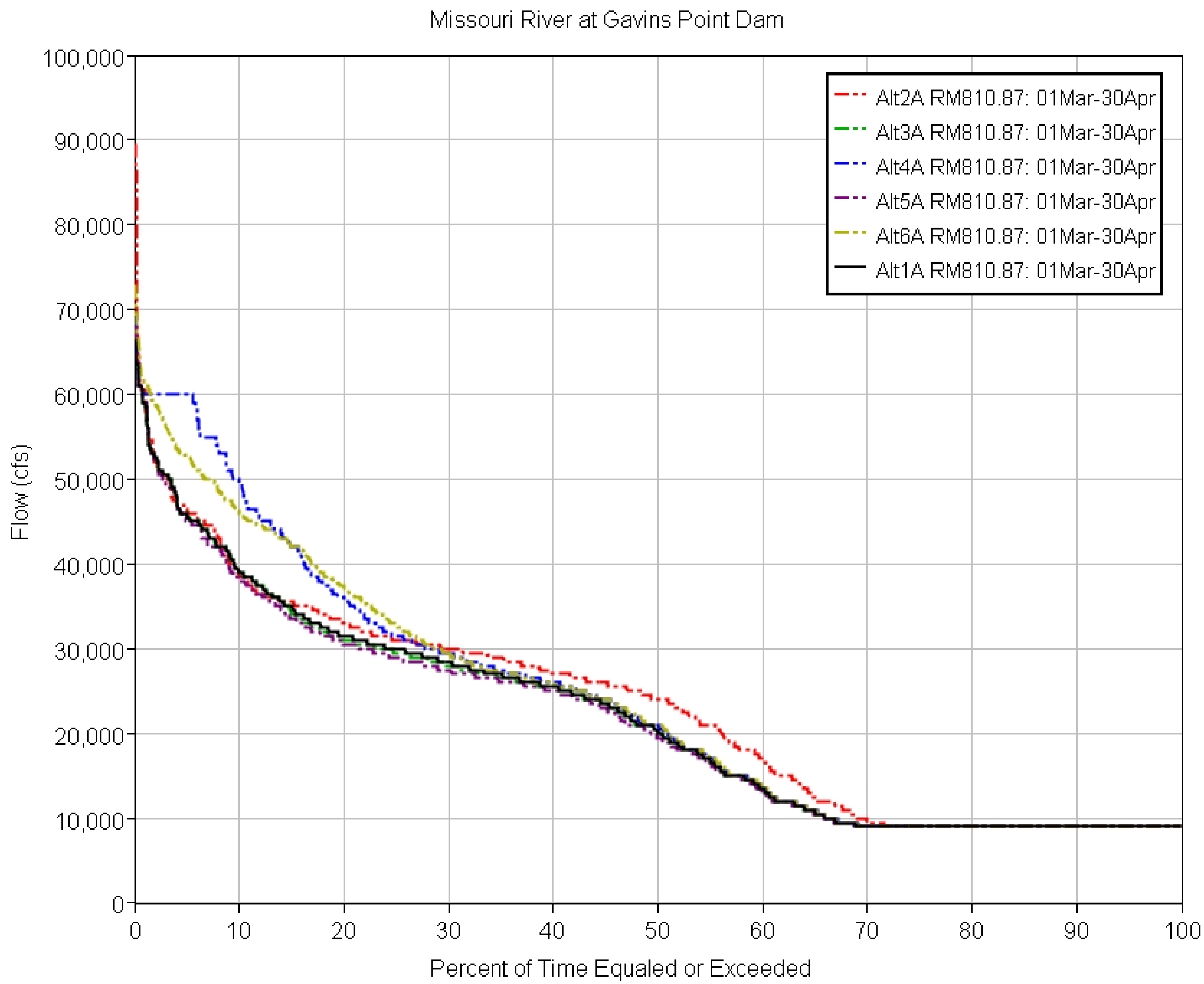


Plate 4: Gavins Point Spring Duration

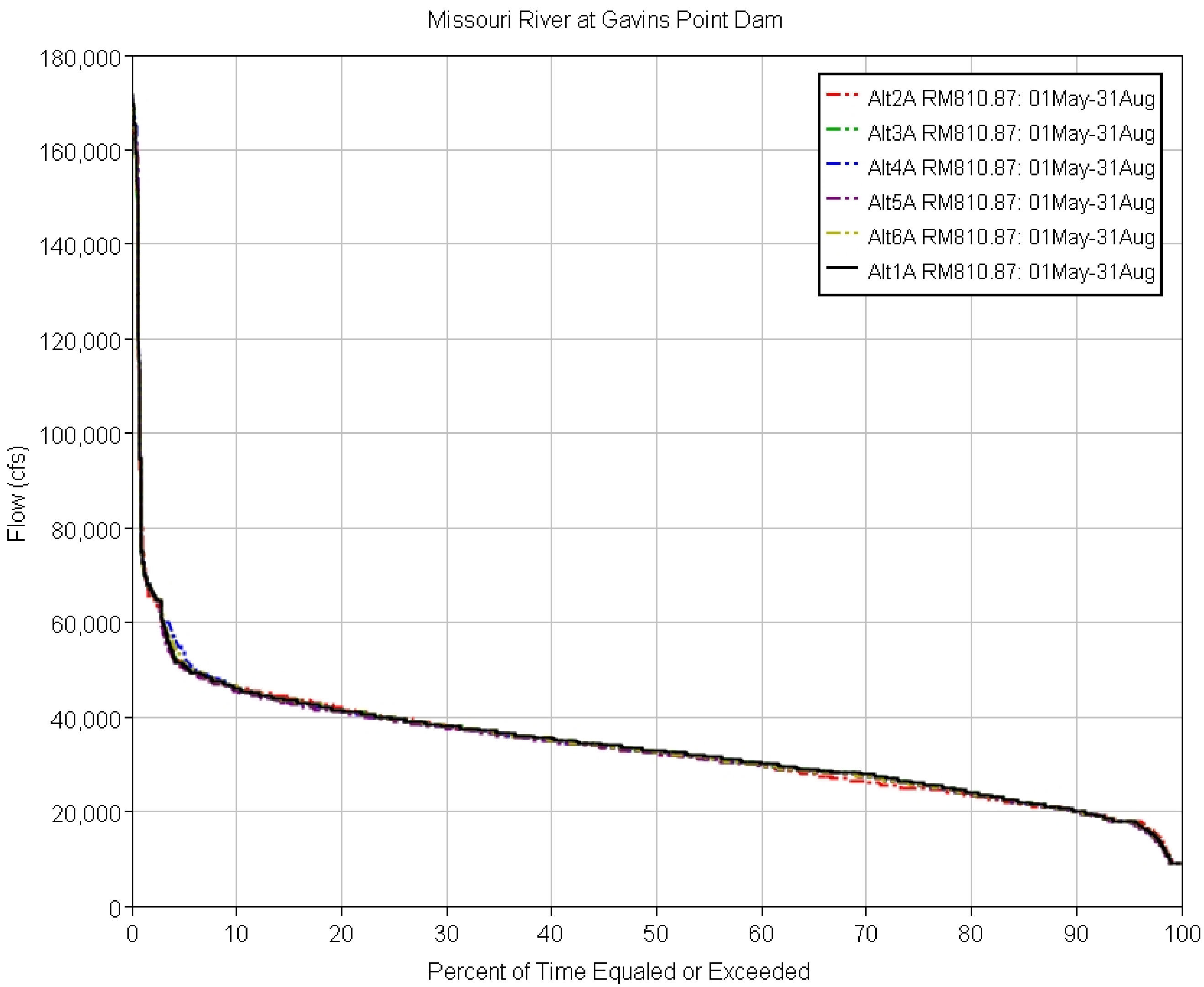


Plate 5: Gavins Point Summer Duration

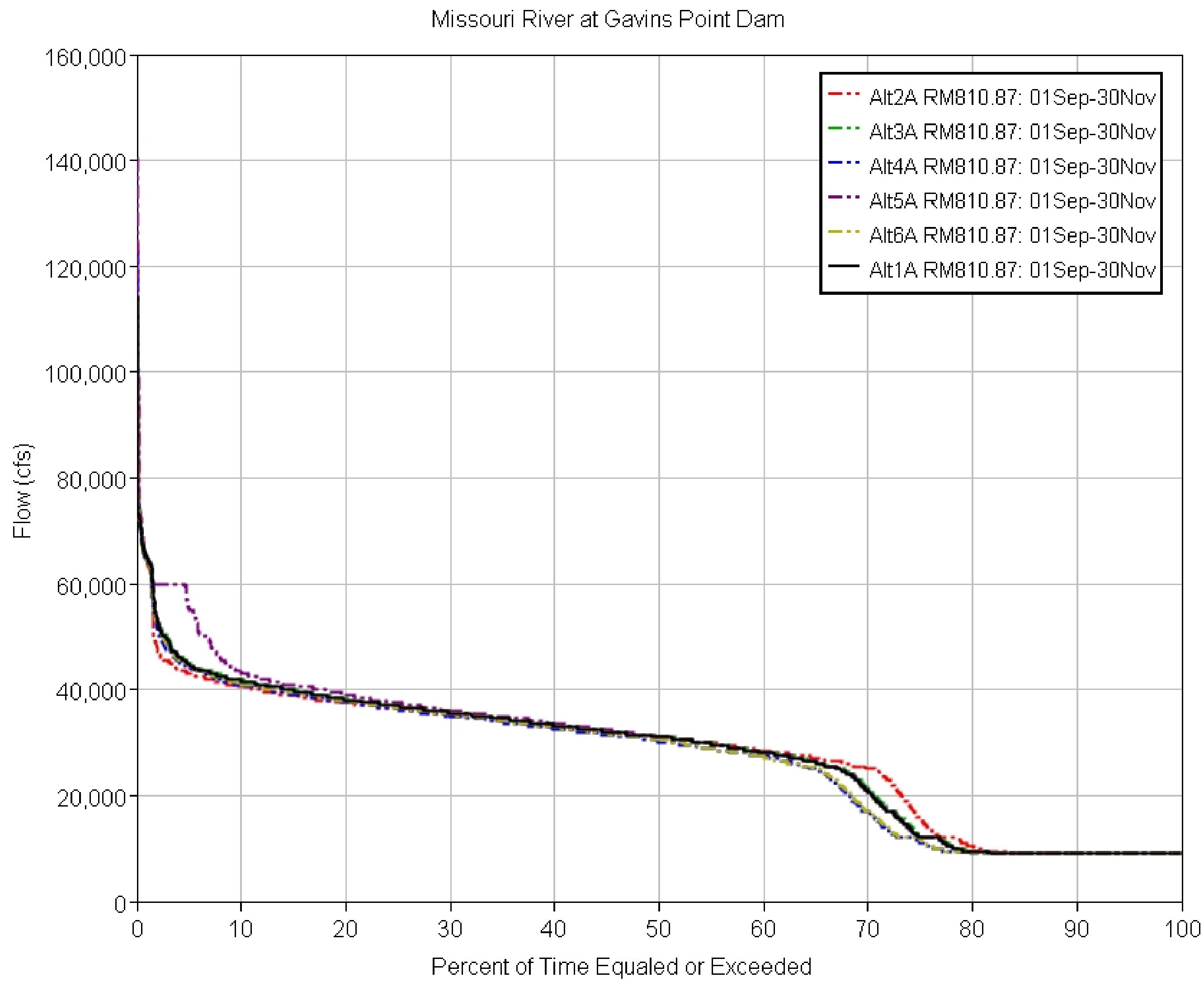


Plate 6: Gavins Point Fall Duration

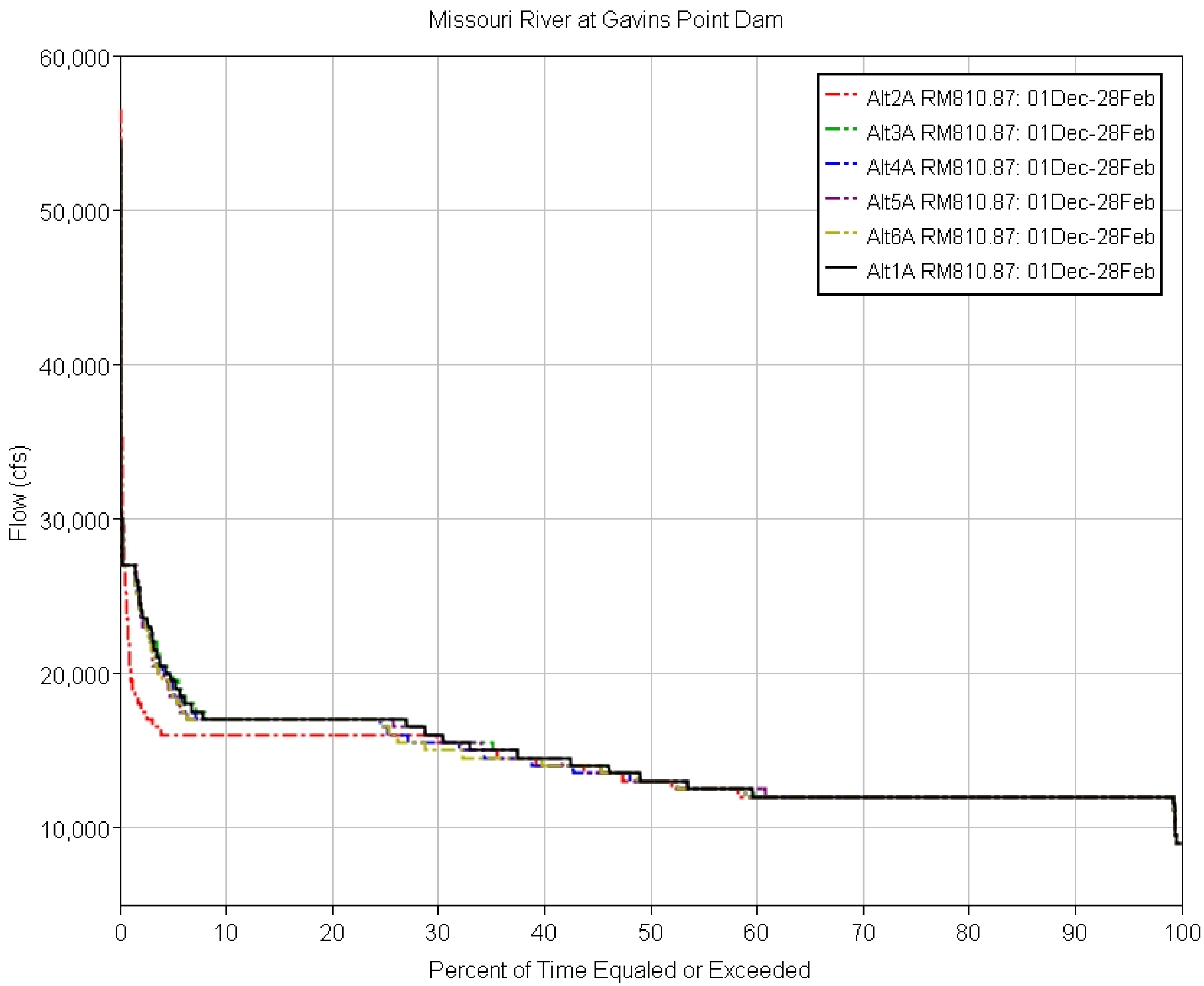


Plate 7: Gavins Point Winter Duration

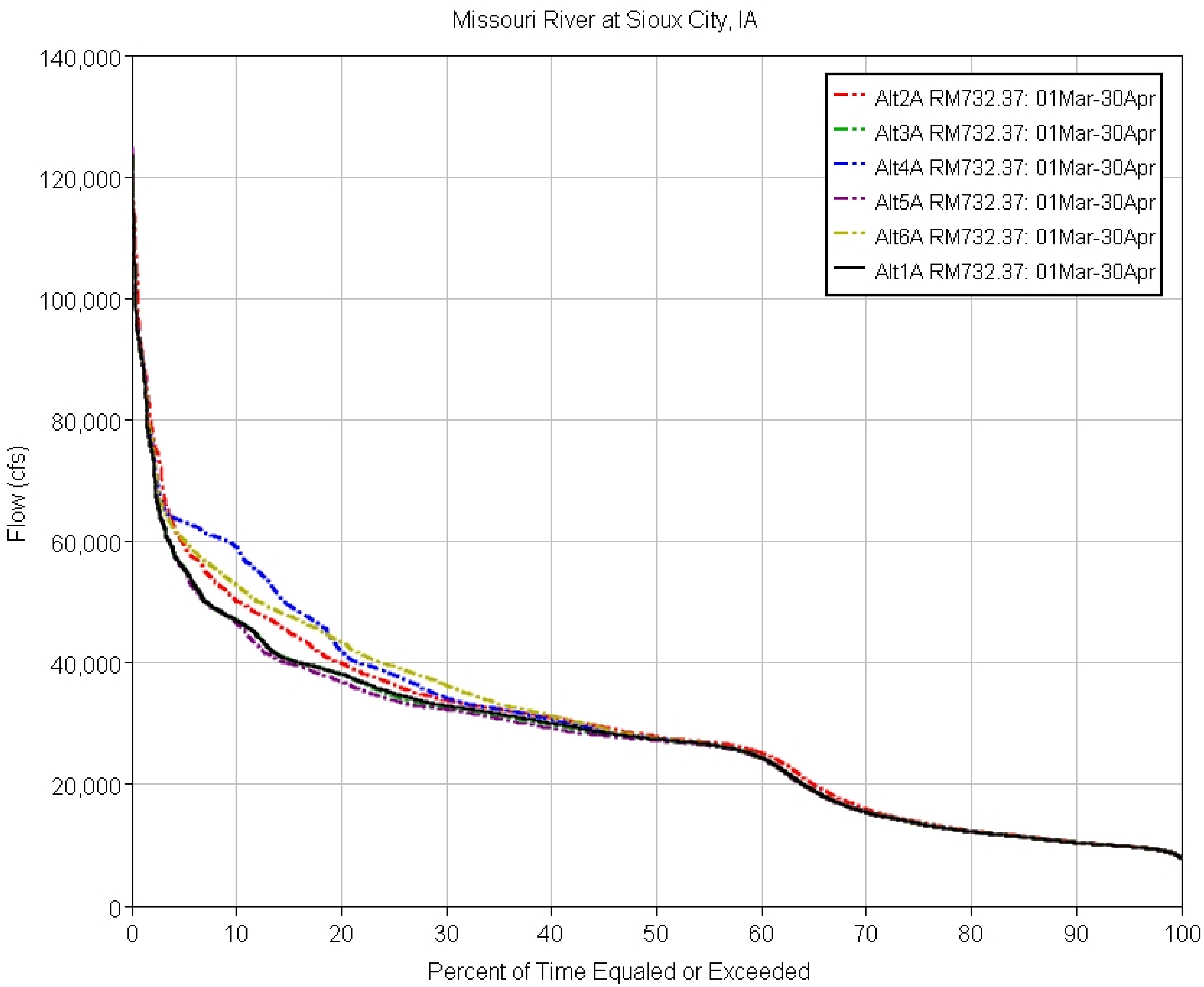


Plate 8: Sioux City Spring Duration

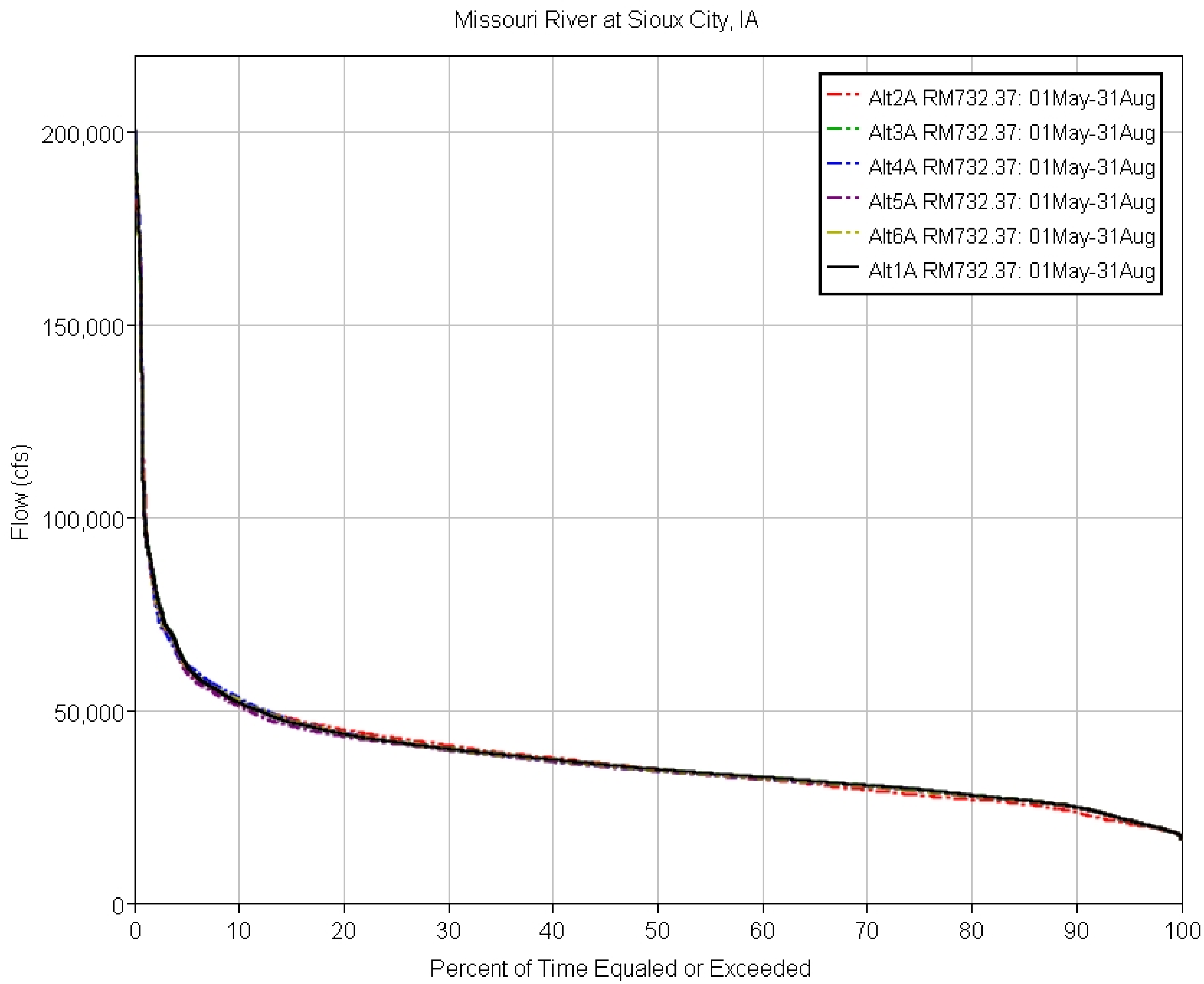


Plate 9: Sioux City Summer Duration

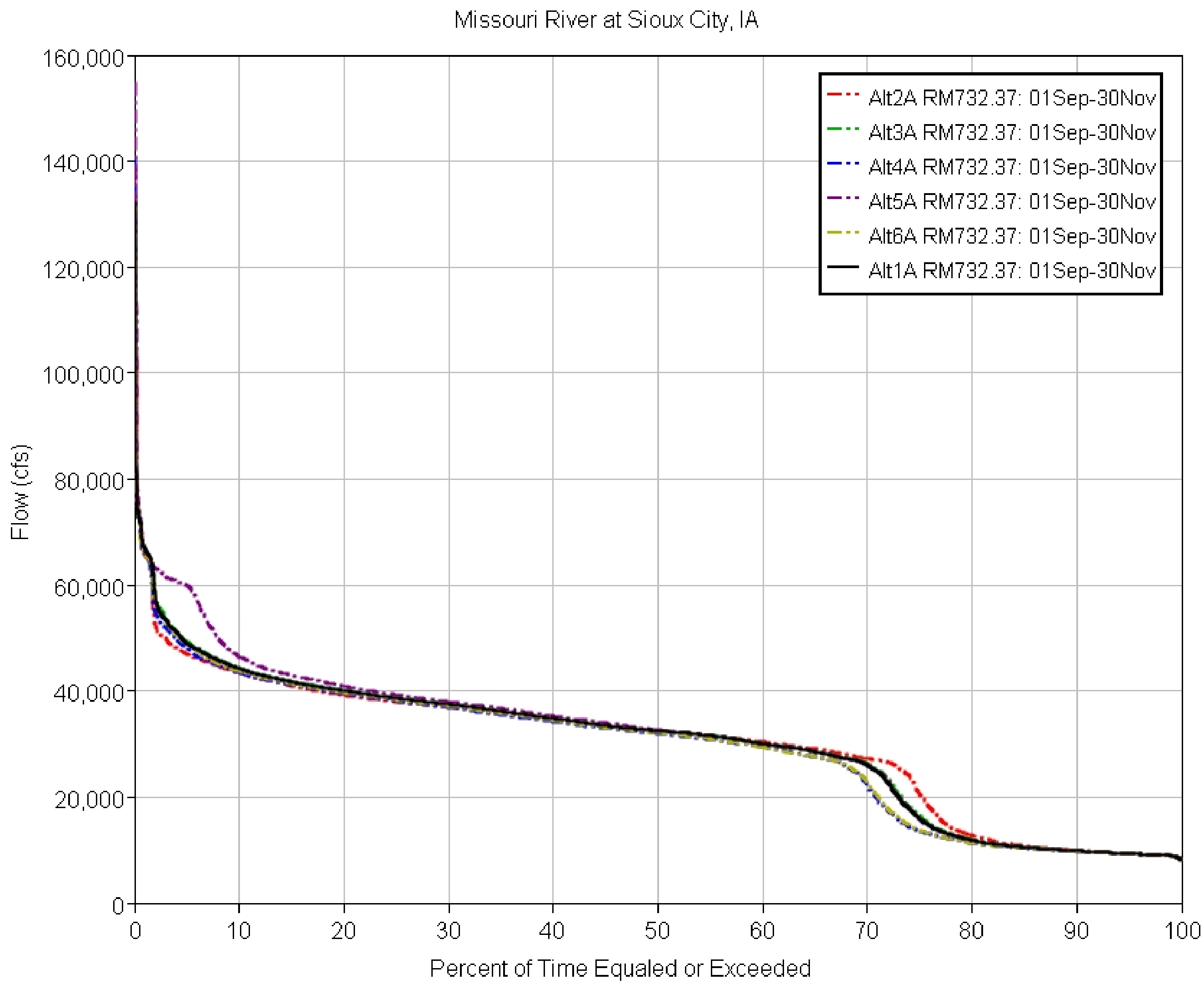


Plate 10: Sioux City Fall Duration

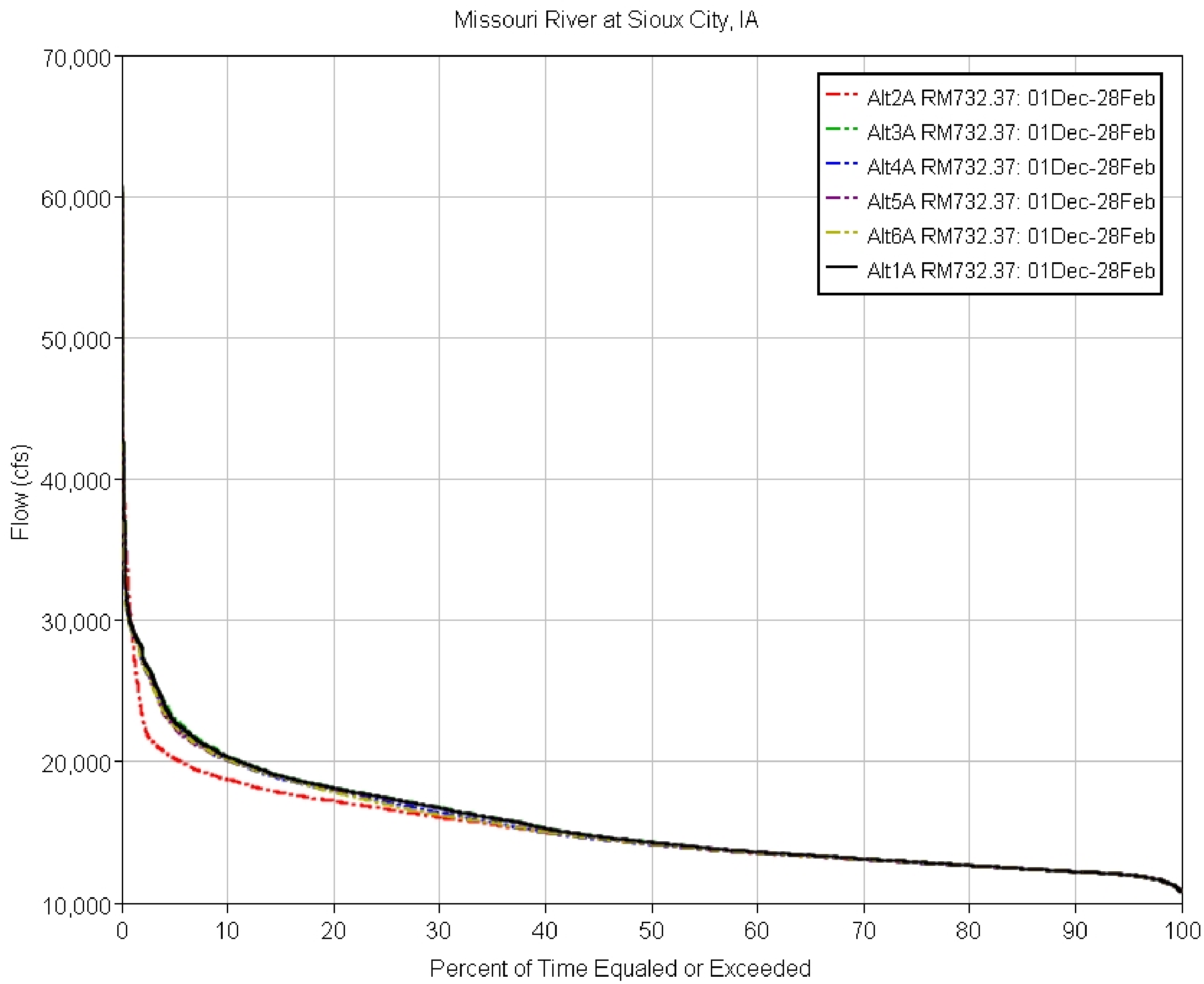


Plate 11: Sioux City Winter Duration

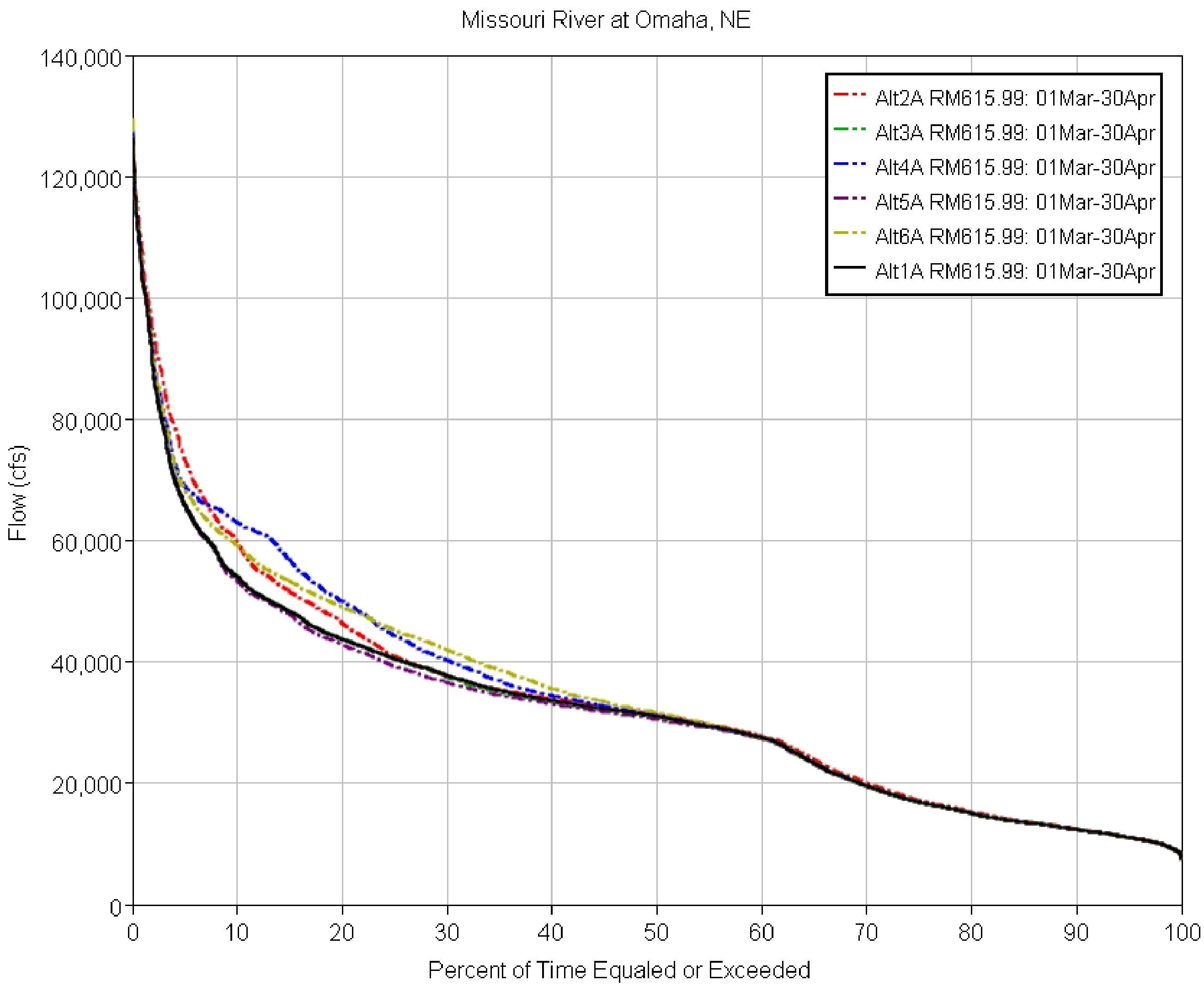


Plate 12: Omaha Spring Duration

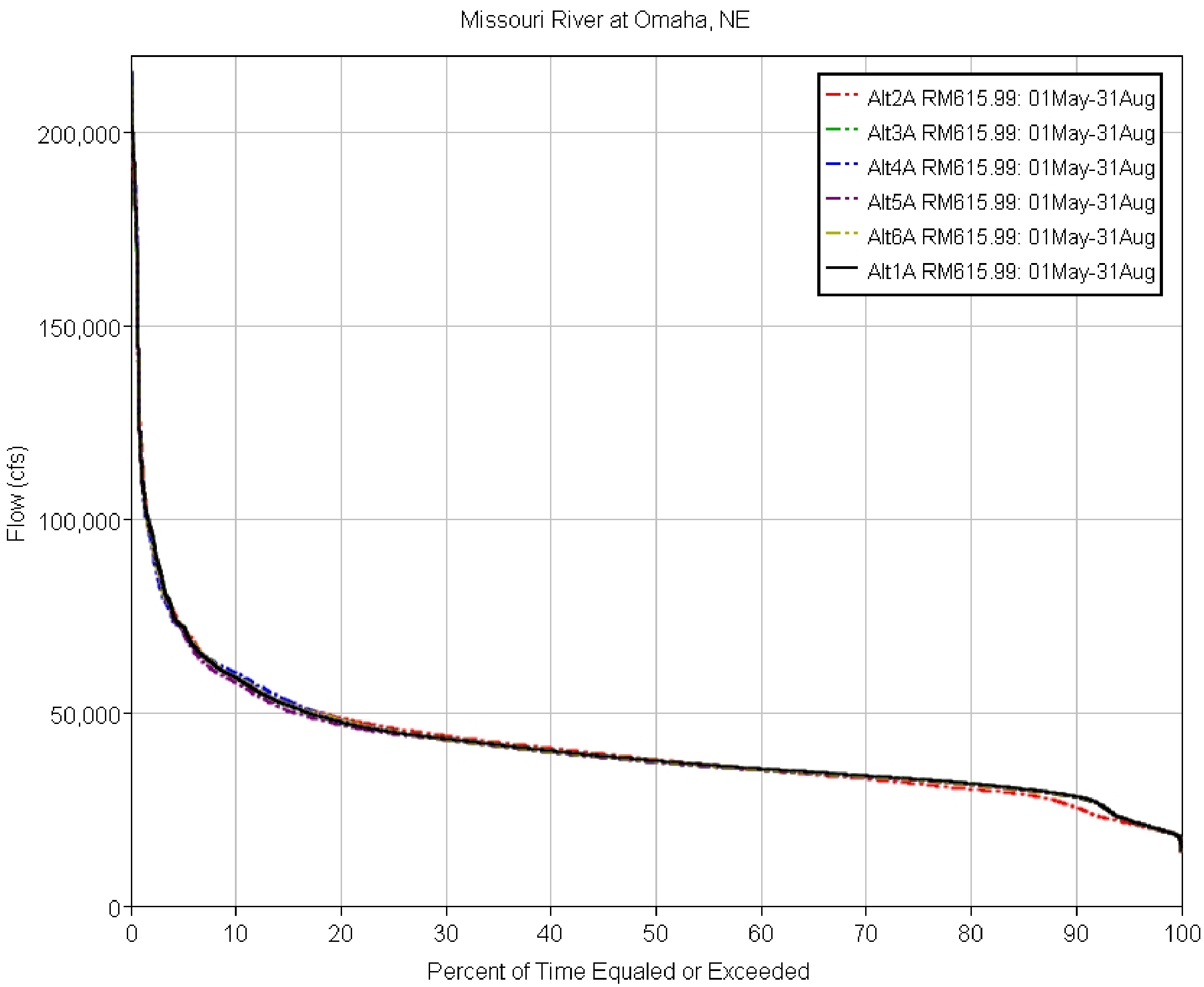


Plate 13: Omaha Summer Duration

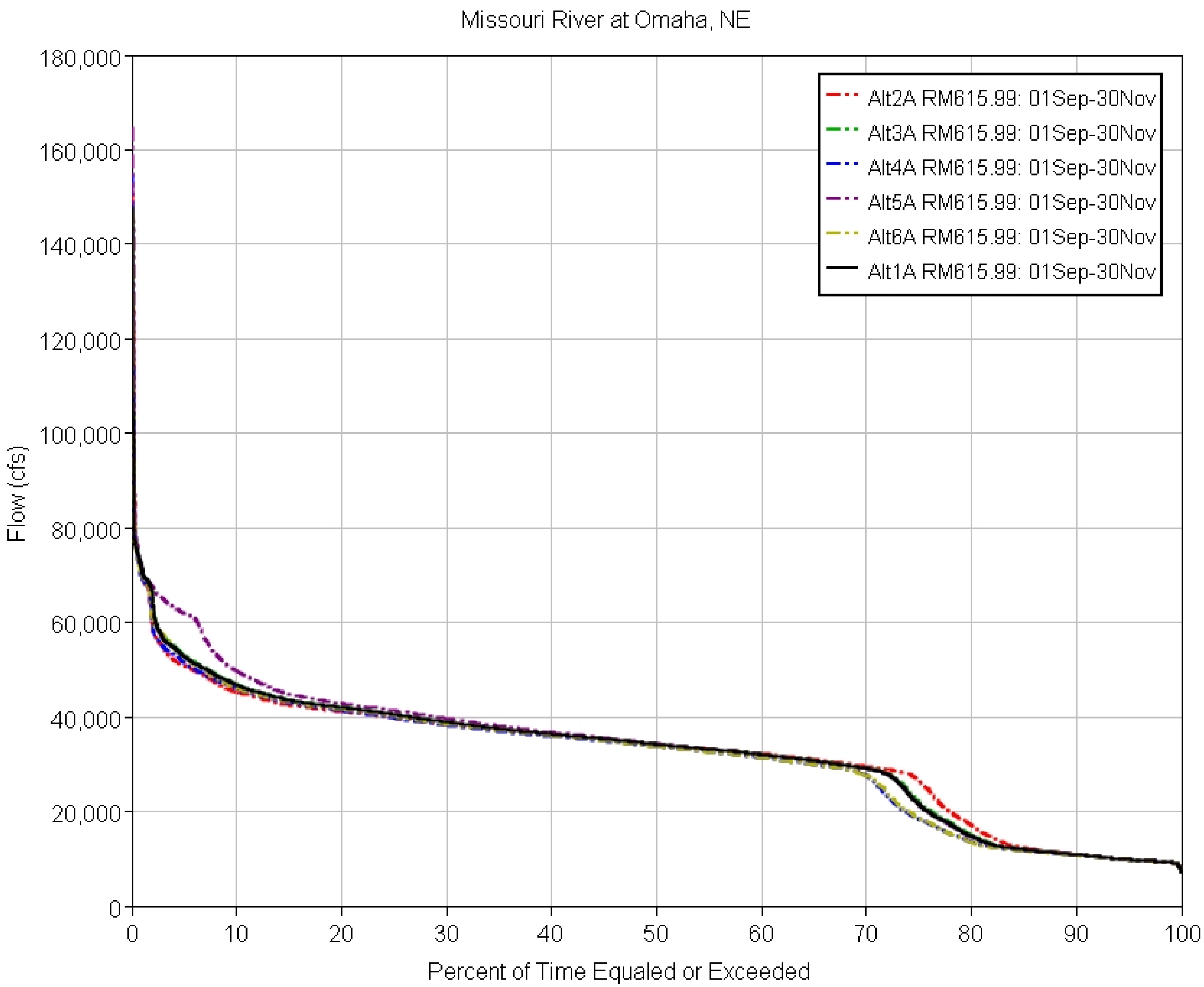


Plate 14: Omaha Fall Duration

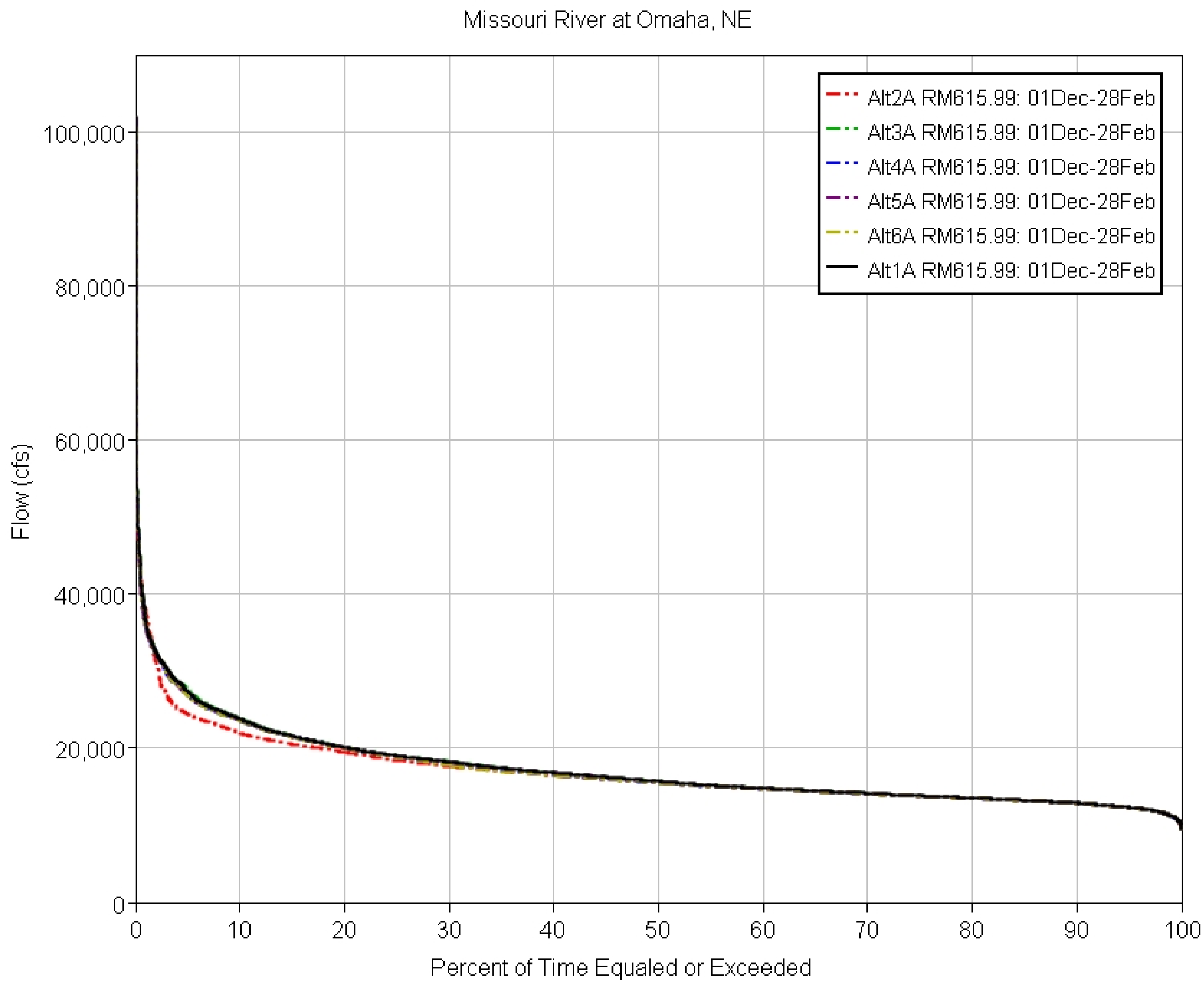


Plate 15: Omaha Winter Duration

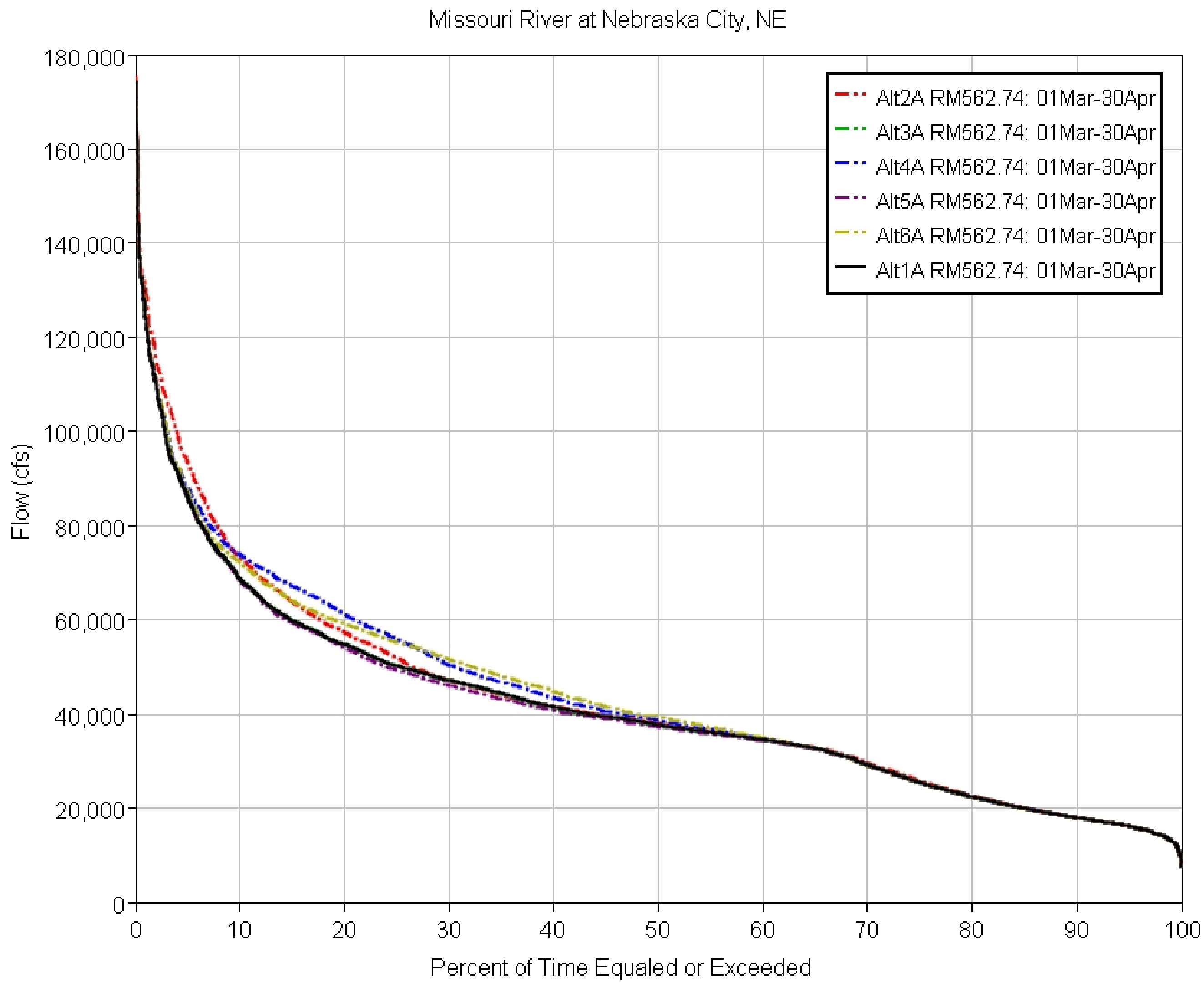


Plate 16: Nebraska City Spring Duration

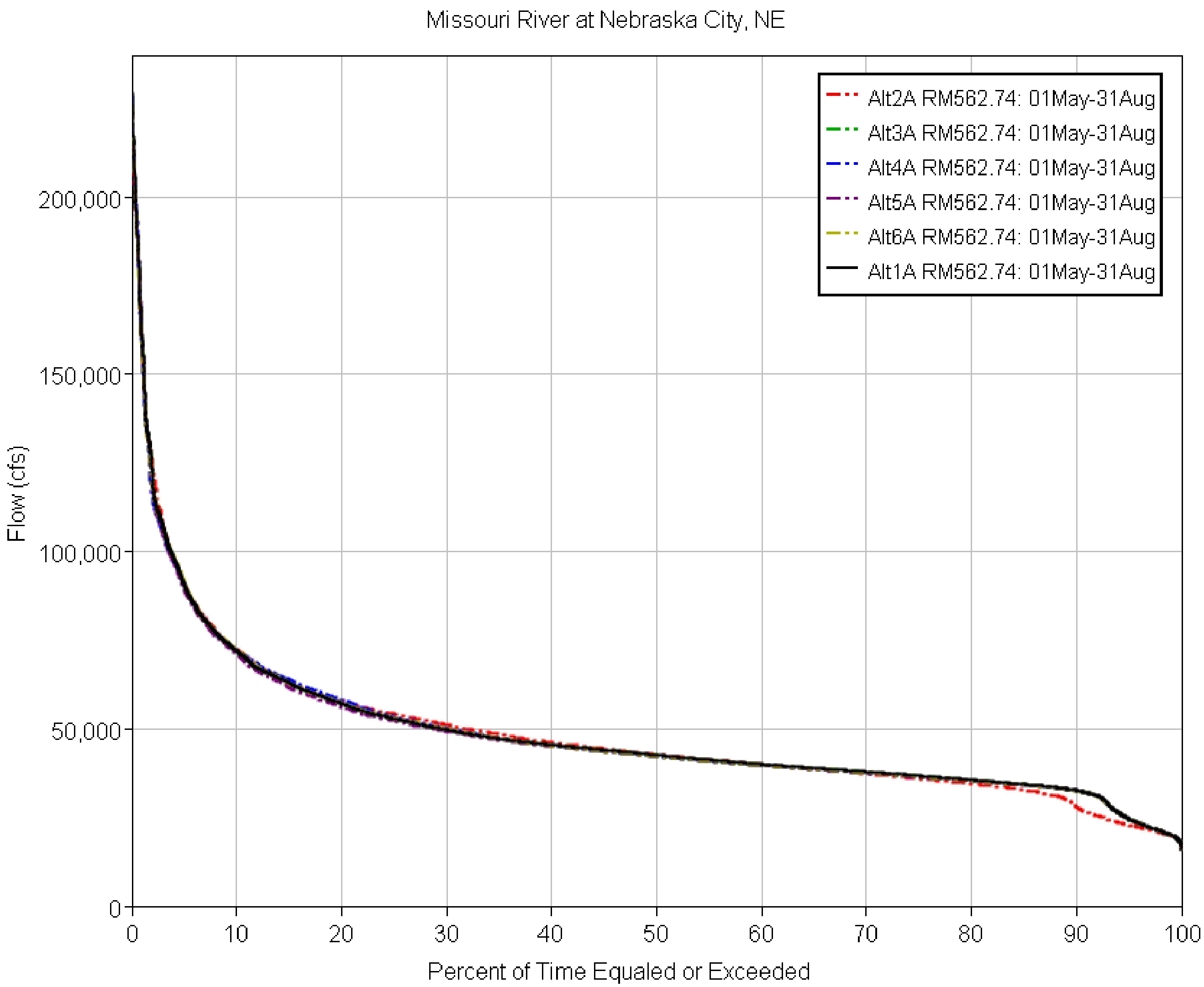


Plate 17: Nebraska City Summer Duration

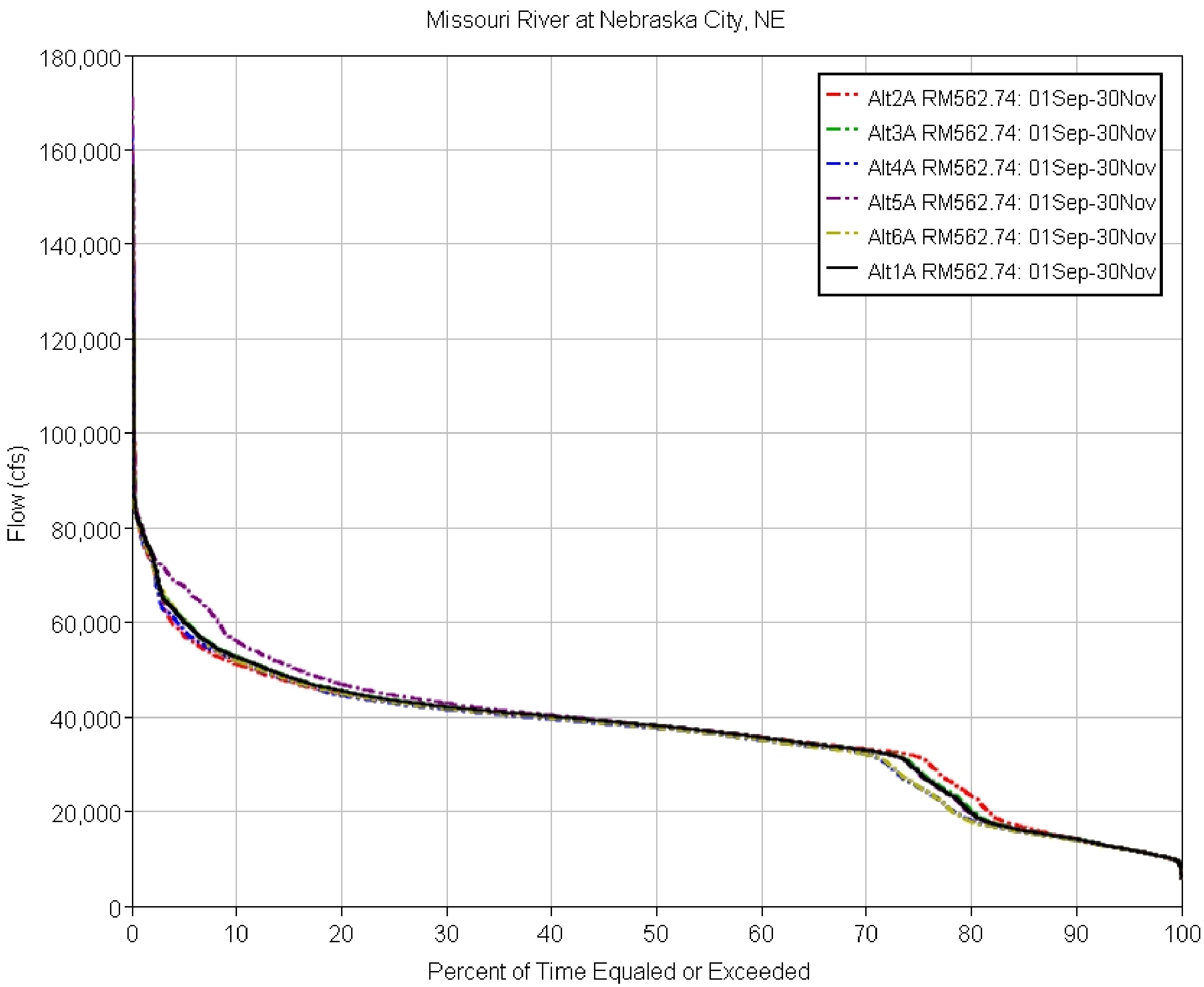


Plate 18: Nebraska City Fall Duration

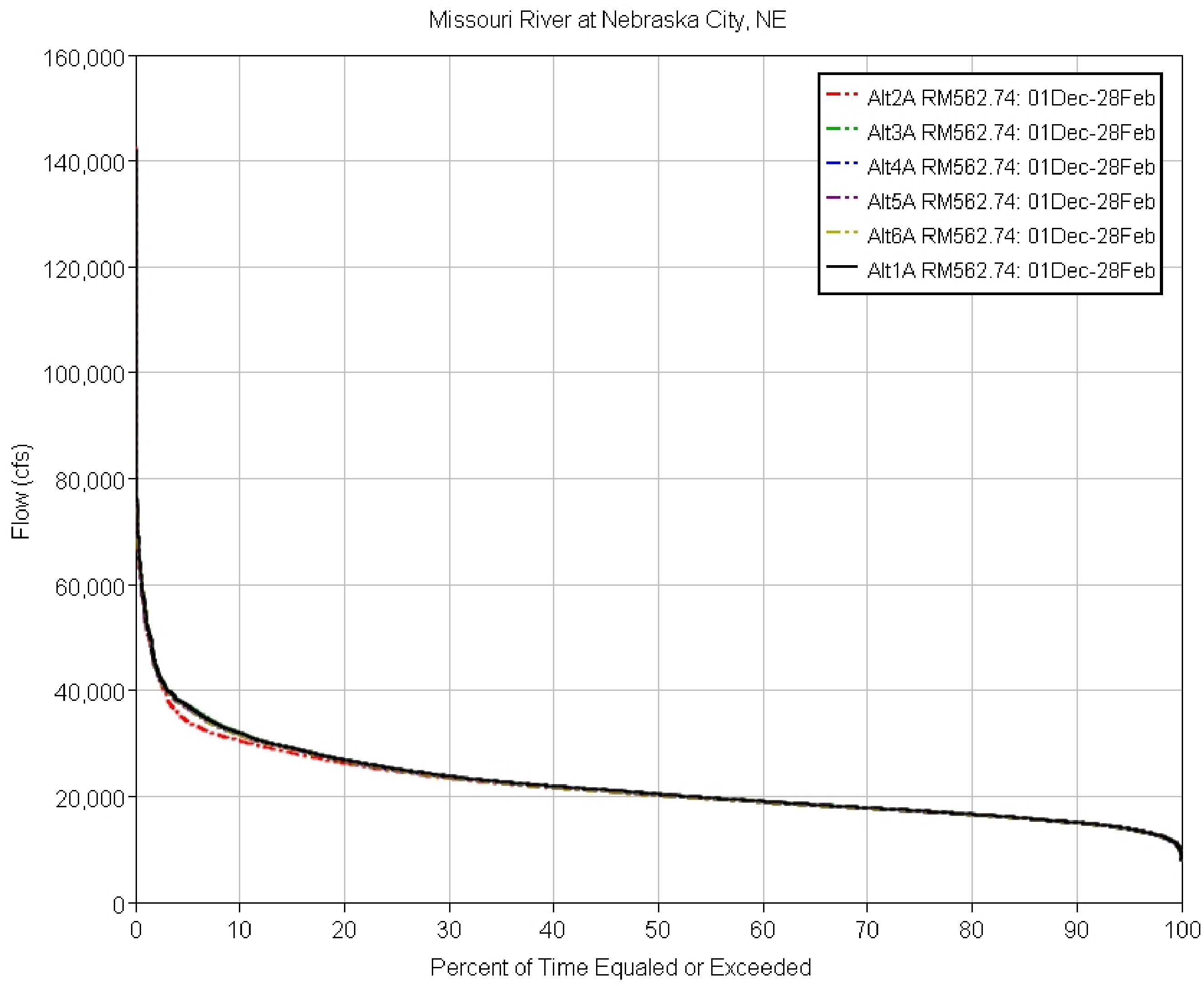


Plate 19: Nebraska City Winter Duration

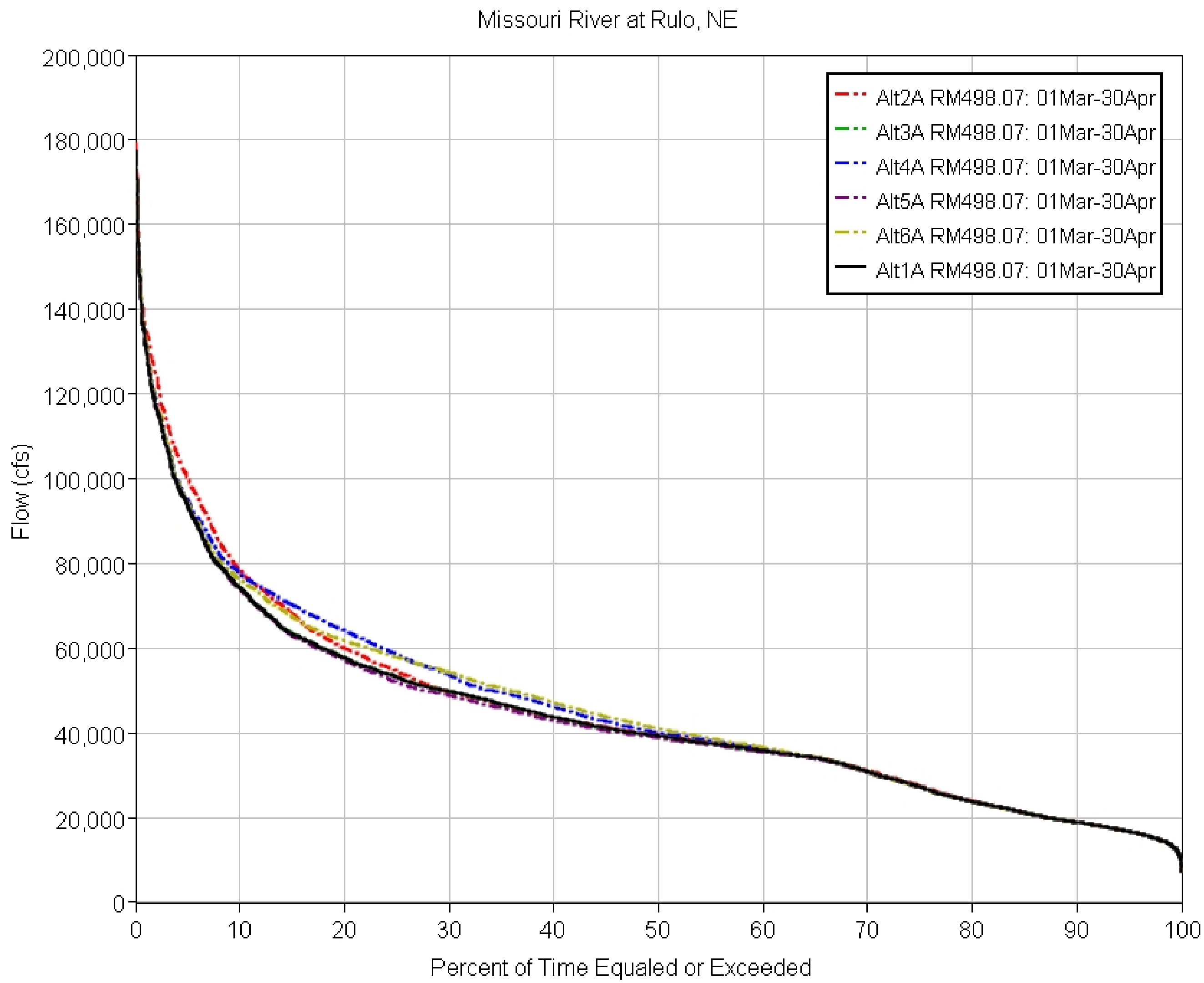


Plate 20: Rulo Spring Duration

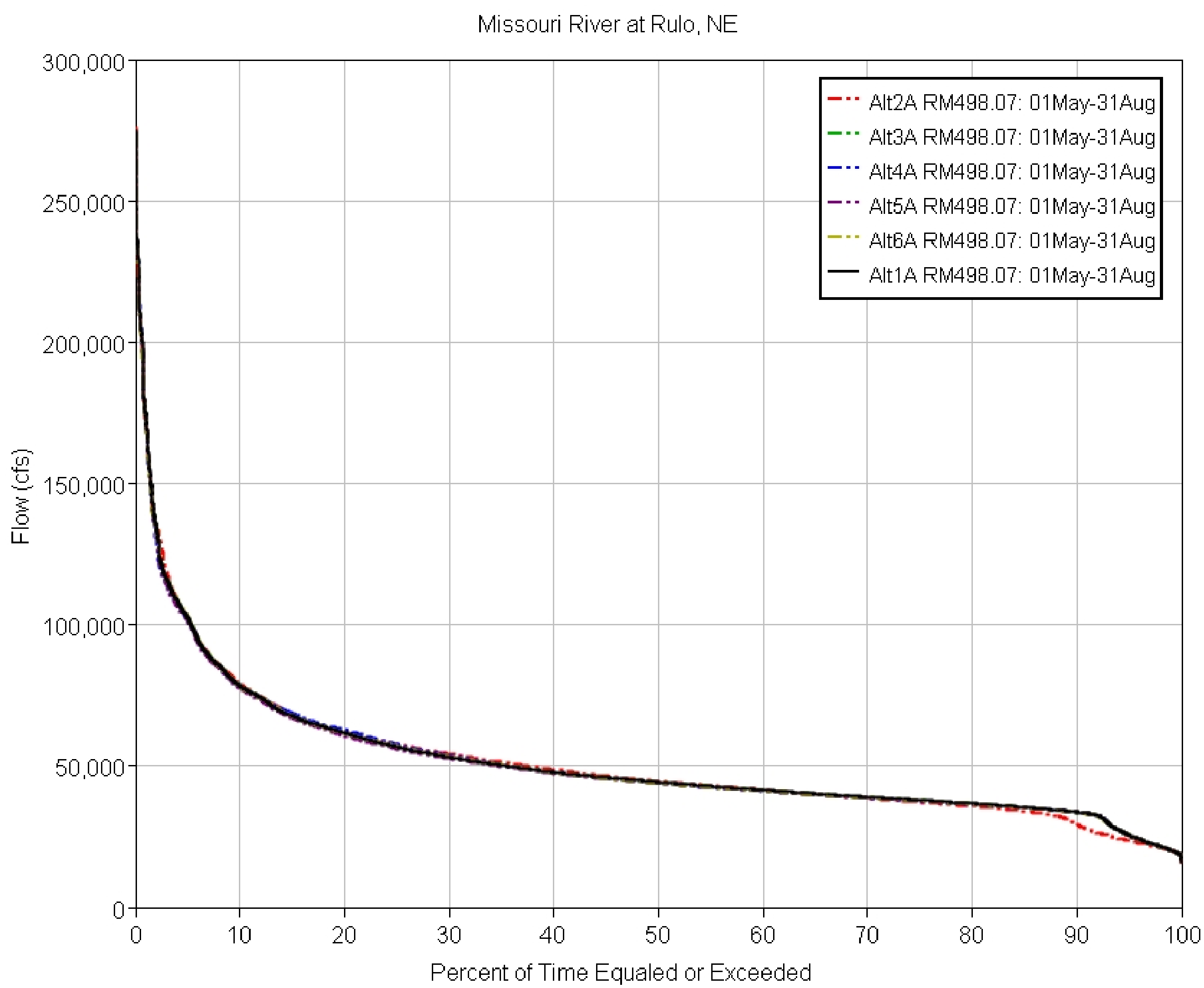


Plate 21: Rulo Summer Duration

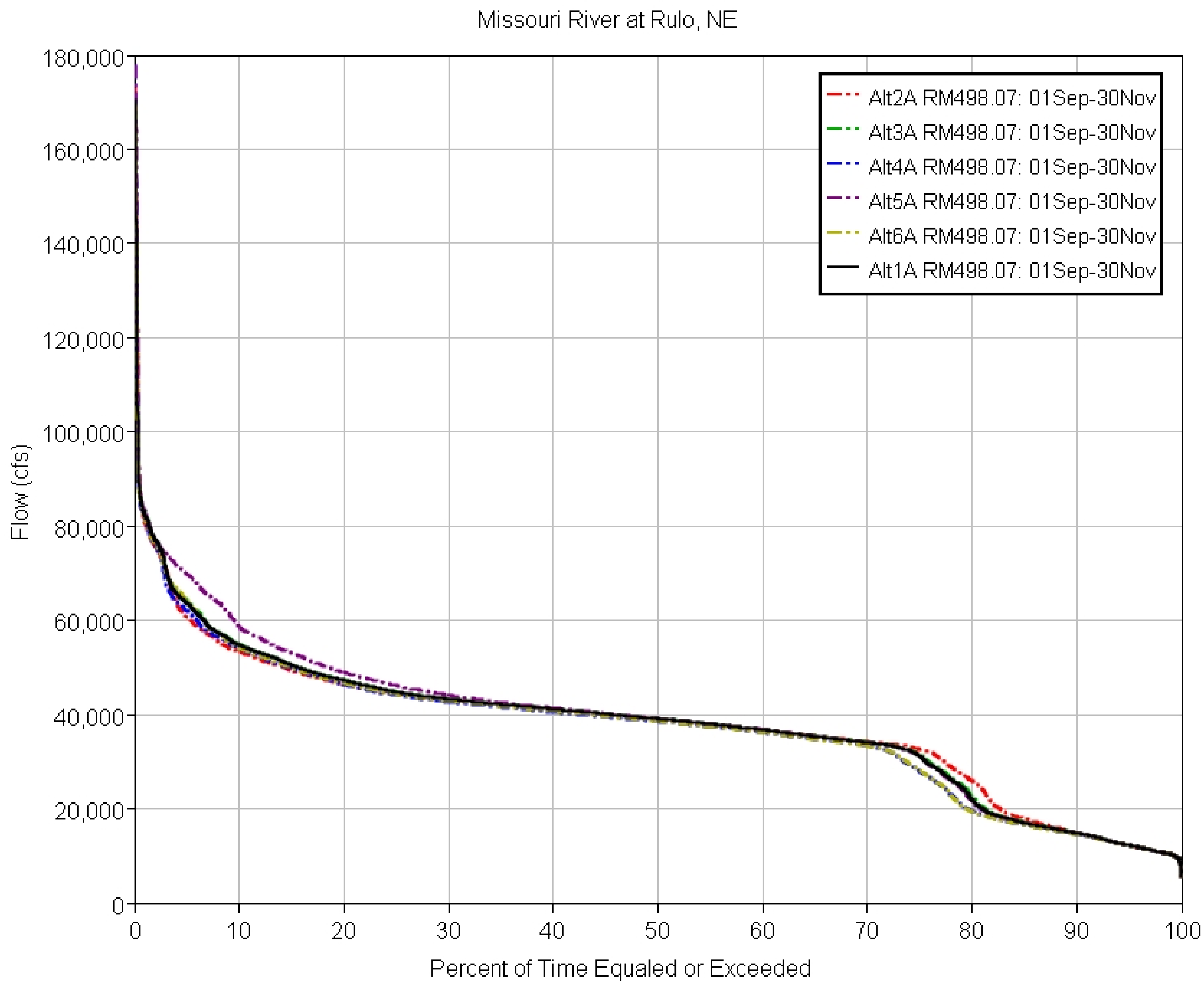


Plate 22: Rulo Fall Duration

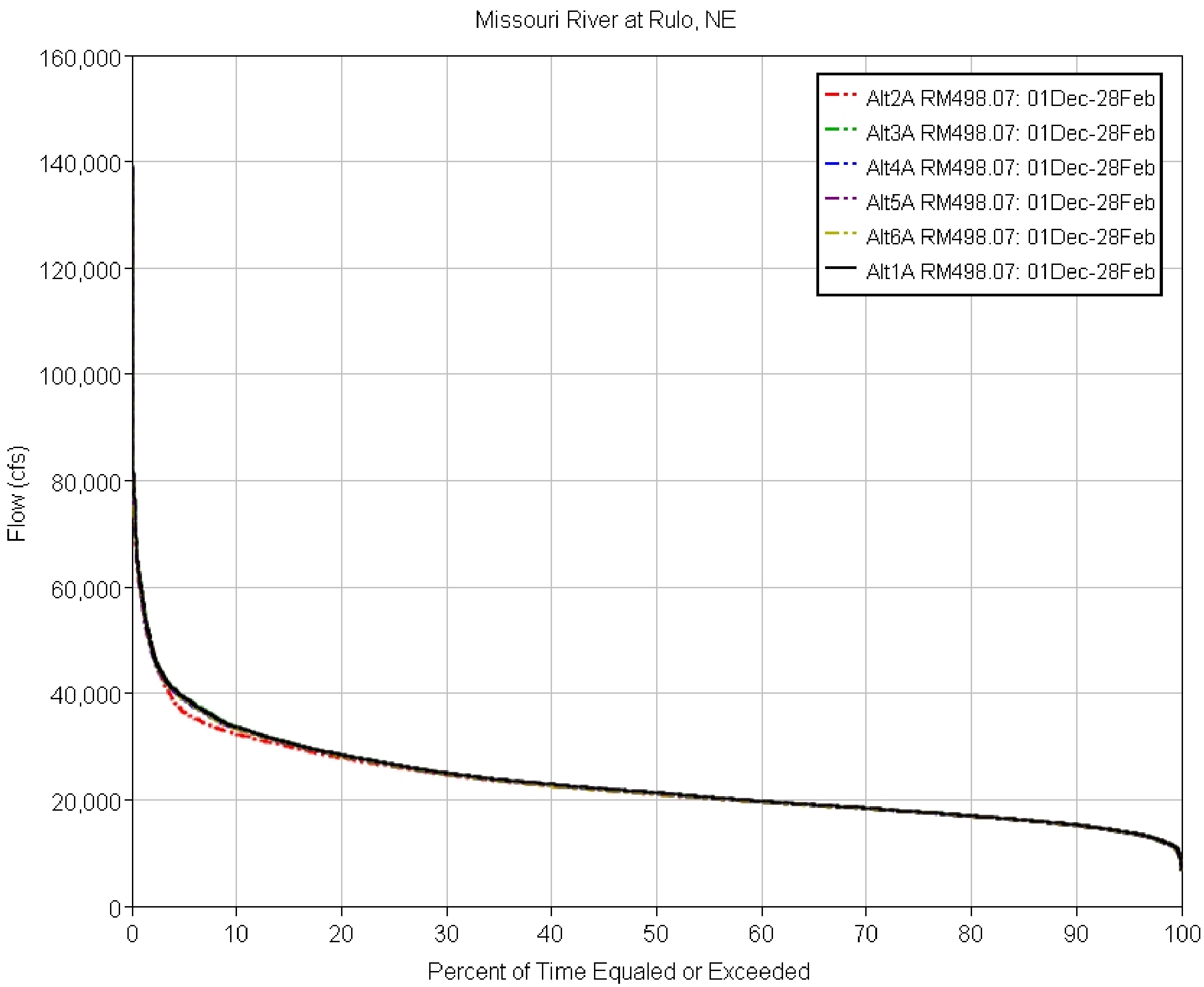


Plate 23: Rulo Winter Duration

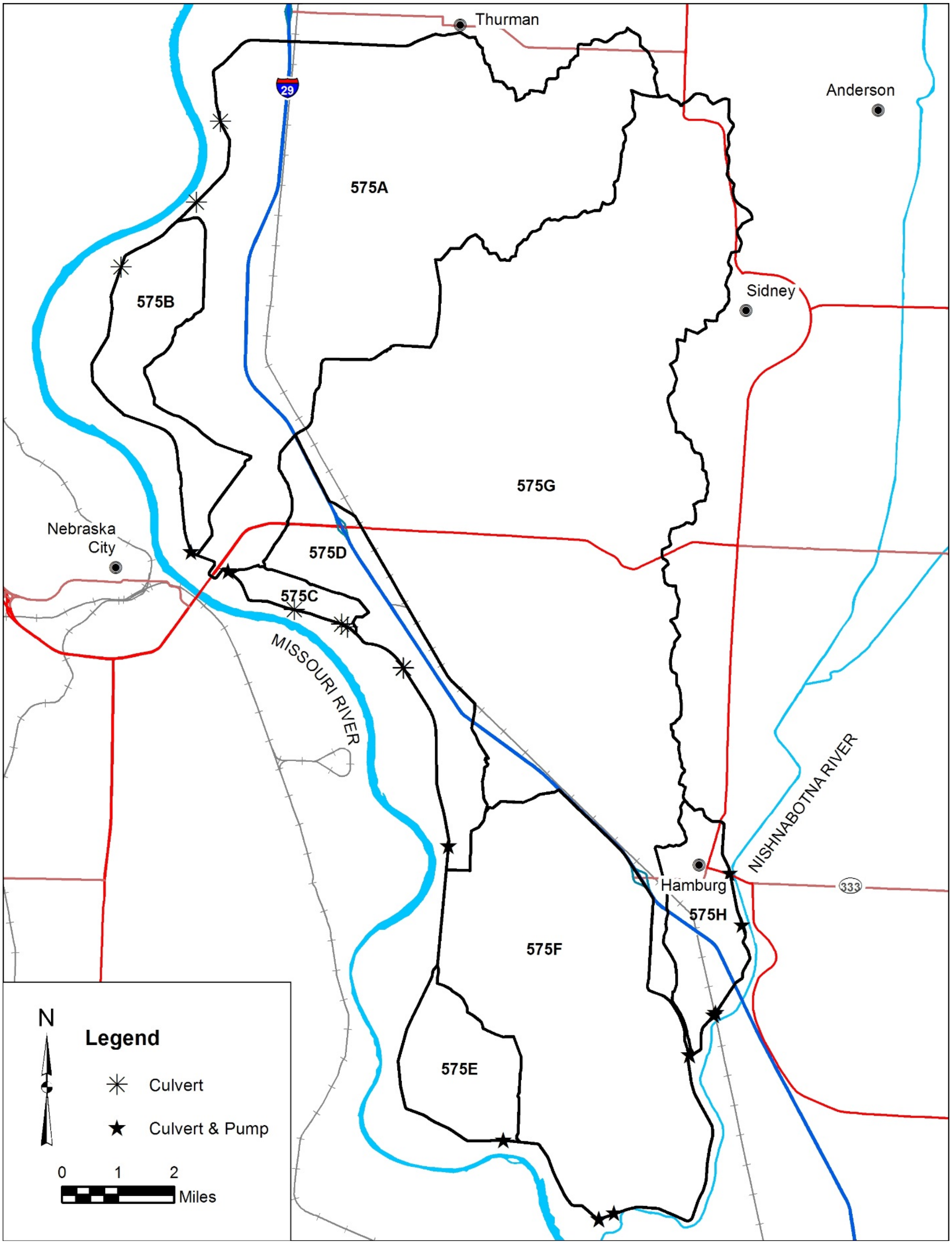


Plate 24: Levee Unit L-575 Basin Map

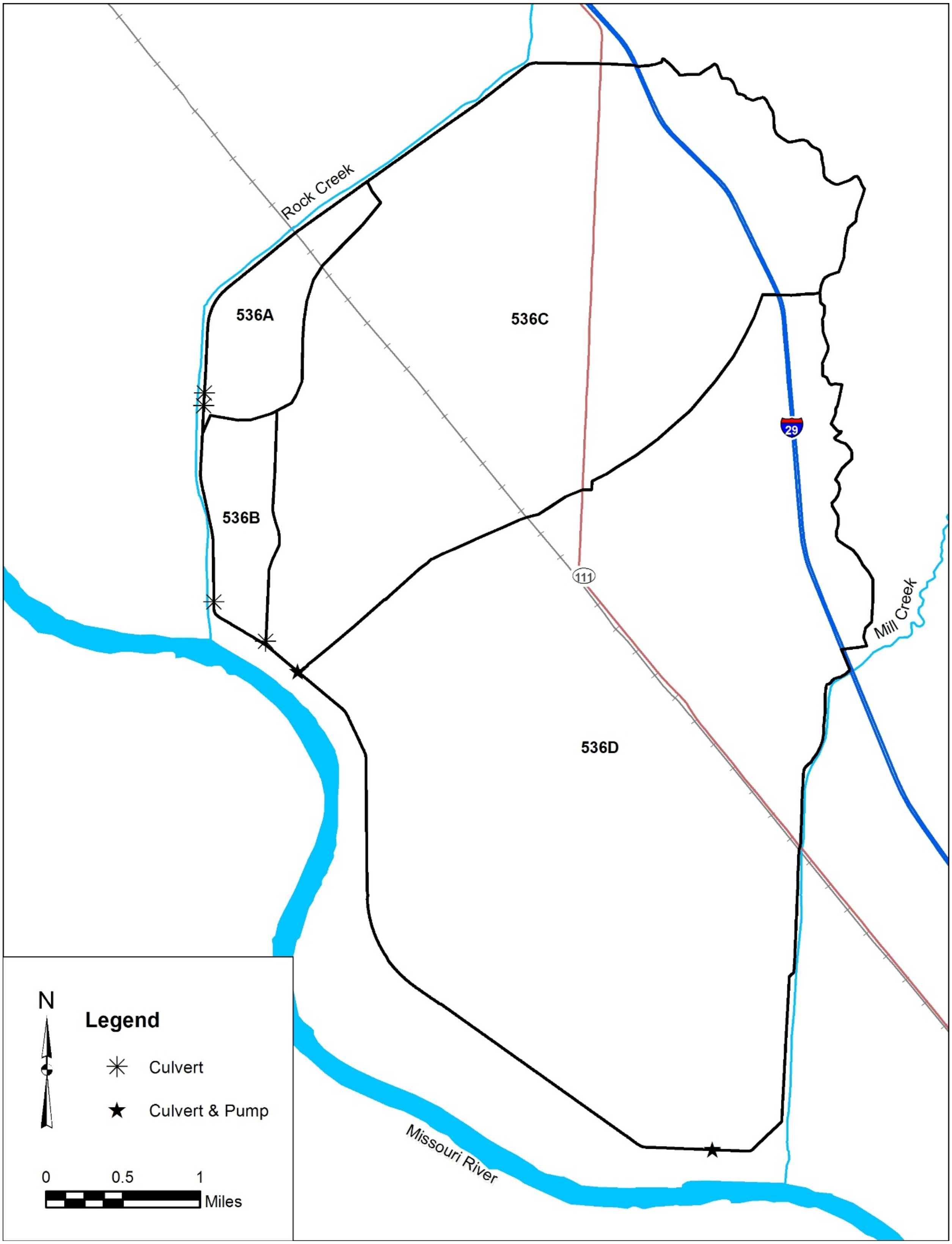


Plate 25: Levee Unit L-536 Basin Map

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Storage Area 575A							
Alt 1A	913.43	913.52	913.82	914.66	916.33	912.28	921.32
Alt 2A	913.42	913.52	913.82	914.64	916.41	912.28	921.32
Change from Alt 1A	0.00	0.00	0.00	-0.02	0.08	0.00	0.00
Alt 3A	913.43	913.53	913.83	914.69	916.36	912.28	921.32
Change from Alt 1A	0.00	0.00	0.00	0.02	0.03	0.00	0.00
Alt 4A	913.43	913.52	913.82	914.66	916.35	912.28	921.32
Change from Alt 1A	0.00	0.00	0.00	0.00	0.02	0.00	0.00
Alt 5A	913.43	913.52	913.83	914.69	916.31	912.28	921.32
Change from Alt 1A	0.00	0.00	0.00	0.03	-0.02	0.00	0.00
Alt 6A	913.43	913.52	913.83	914.68	916.32	912.28	921.32
Change from Alt 1A	0.00	0.00	0.01	0.02	-0.01	0.00	0.00
Storage Area 575B							
Alt 1A	914.35	914.41	914.64	915.66	917.48	912.94	922.53
Alt 2A	914.35	914.41	914.63	915.62	917.54	912.94	922.51
Change from Alt 1A	0.00	0.00	-0.01	-0.04	0.06	0.00	-0.02
Alt 3A	914.35	914.41	914.64	915.67	917.52	912.94	922.53
Change from Alt 1A	0.00	0.00	0.00	0.01	0.04	0.00	0.00
Alt 4A	914.35	914.40	914.63	915.67	917.58	912.94	922.53
Change from Alt 1A	0.00	0.00	0.00	0.01	0.10	0.00	0.00
Alt 5A	914.35	914.41	914.64	915.68	917.51	912.94	922.53
Change from Alt 1A	0.00	0.00	0.00	0.02	0.03	0.00	0.00
Alt 6A	914.35	914.40	914.64	915.68	917.53	912.94	922.53
Change from Alt 1A	0.00	0.00	0.00	0.02	0.05	0.00	0.00
Storage Area 575C							
Alt 1A	913.83	913.85	913.91	914.02	914.66	912.26	919.68
Alt 2A	913.83	913.85	913.91	914.02	914.87	912.26	919.68
Change from Alt 1A	0.00	0.00	0.00	0.00	0.21	0.00	0.00
Alt 3A	913.83	913.85	913.91	914.02	914.72	912.26	919.68
Change from Alt 1A	0.00	0.00	0.00	0.00	0.06	0.00	0.00
Alt 4A	913.83	913.85	913.91	914.03	914.72	912.26	919.68
Change from Alt 1A	0.00	0.00	0.00	0.01	0.06	0.00	0.00
Alt 5A	913.83	913.85	913.91	914.02	914.65	912.26	919.68
Change from Alt 1A	0.00	0.00	0.00	0.00	-0.01	0.00	0.00
Alt 6A	913.83	913.85	913.91	914.03	914.73	912.26	919.68
Change from Alt 1A	0.00	0.00	0.00	0.01	0.07	0.00	0.00

Plate 26: Storage Area Alternative Statistics

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Storage Area 575D							
Alt 1A	907.36	907.42	907.66	908.45	910.13	905.63	911.97
Alt 2A	907.36	907.42	907.65	908.43	910.27	905.63	911.96
Change from Alt 1A	0.00	0.00	-0.01	-0.02	0.14	0.00	0.00
Alt 3A	907.36	907.42	907.66	908.45	910.15	905.63	911.97
Change from Alt 1A	0.00	0.00	0.00	0.00	0.02	0.00	0.00
Alt 4A	907.36	907.42	907.66	908.46	910.36	905.63	911.97
Change from Alt 1A	0.00	0.00	0.00	0.01	0.23	0.00	0.00
Alt 5A	907.36	907.42	907.66	908.45	910.17	905.63	911.97
Change from Alt 1A	0.00	0.00	0.00	0.00	0.04	0.00	0.00
Alt 6A	907.36	907.42	907.67	908.47	910.21	905.63	911.97
Change from Alt 1A	0.00	0.00	0.00	0.02	0.07	0.00	0.00
Storage Area 575E							
Alt 1A	896.29	896.31	896.41	897.02	898.68	895.18	904.54
Alt 2A	896.29	896.31	896.41	896.97	898.83	895.18	904.52
Change from Alt 1A	0.00	0.00	0.00	-0.04	0.15	0.00	-0.01
Alt 3A	896.29	896.31	896.41	897.02	898.73	895.18	904.55
Change from Alt 1A	0.00	0.00	0.00	0.00	0.05	0.00	0.01
Alt 4A	896.29	896.31	896.41	896.99	898.64	895.18	904.54
Change from Alt 1A	0.00	0.00	0.00	-0.03	-0.04	0.00	0.00
Alt 5A	896.29	896.31	896.41	897.02	898.63	895.18	904.55
Change from Alt 1A	0.00	0.00	0.00	0.01	-0.05	0.00	0.01
Alt 6A	896.29	896.31	896.41	897.01	898.67	895.18	904.55
Change from Alt 1A	0.00	0.00	0.00	-0.01	-0.01	0.00	0.01
Storage Area 575F							
Alt 1A	893.54	893.62	893.80	894.32	895.96	892.00	904.54
Alt 2A	893.54	893.62	893.79	894.30	895.96	892.00	904.53
Change from Alt 1A	0.00	0.00	0.00	-0.02	0.00	0.00	-0.01
Alt 3A	893.54	893.62	893.80	894.33	896.00	892.00	904.55
Change from Alt 1A	0.00	0.00	0.00	0.01	0.04	0.00	0.01
Alt 4A	893.54	893.62	893.80	894.36	896.18	892.00	904.54
Change from Alt 1A	0.00	0.00	0.00	0.04	0.22	0.00	0.00
Alt 5A	893.54	893.62	893.80	894.33	896.08	892.00	904.55
Change from Alt 1A	0.00	0.00	0.00	0.01	0.12	0.00	0.01
Alt 6A	893.54	893.62	893.80	894.35	896.05	892.00	904.55
Change from Alt 1A	0.00	0.00	0.01	0.03	0.09	0.00	0.01

Plate 26 cont'd: Storage Area Alternative Statistics

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Storage Area 575G							
Alt 1A	898.09	898.17	898.39	899.67	901.58	890.61	909.78
Alt 2A	898.09	898.17	898.39	899.67	901.60	890.61	909.78
Change from Alt 1A	0.00	0.00	0.00	0.00	0.01	0.00	0.00
Alt 3A	898.09	898.17	898.39	899.67	901.58	890.61	909.78
Change from Alt 1A	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Alt 4A	898.09	898.17	898.39	899.69	901.60	890.61	909.78
Change from Alt 1A	0.00	0.00	0.00	0.02	0.02	0.00	0.00
Alt 5A	898.09	898.17	898.39	899.68	901.59	890.61	909.78
Change from Alt 1A	0.00	0.00	0.00	0.01	0.00	0.00	0.00
Alt 6A	898.09	898.17	898.39	899.67	901.58	890.61	909.78
Change from Alt 1A	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Storage Area 575H							
Alt 1A	898.26	898.42	899.13	900.37	900.81	896.86	906.75
Alt 2A	898.26	898.42	899.13	900.37	900.81	896.86	906.74
Change from Alt 1A	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Alt 3A	898.26	898.42	899.13	900.37	900.81	896.86	906.75
Change from Alt 1A	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Alt 4A	898.26	898.42	899.13	900.37	900.81	896.86	906.75
Change from Alt 1A	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Alt 5A	898.26	898.42	899.13	900.37	900.81	896.86	906.75
Change from Alt 1A	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Alt 6A	898.26	898.42	899.13	900.37	900.81	896.86	906.75
Change from Alt 1A	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Storage Area 536A							
Alt 1A	875.92	875.93	875.96	876.04	876.32	874.00	883.55
Alt 2A	875.92	875.93	875.96	876.04	876.35	874.00	883.55
Change from Alt 1A	0.00	0.00	0.00	0.00	0.02	0.00	0.00
Alt 3A	875.92	875.93	875.96	876.04	876.34	874.00	883.55
Change from Alt 1A	0.00	0.00	0.00	0.00	0.02	0.00	0.00
Alt 4A	875.92	875.93	875.96	876.04	876.38	874.00	883.55
Change from Alt 1A	0.00	0.00	0.00	0.00	0.05	0.00	0.00
Alt 5A	875.92	875.93	875.96	876.04	876.36	874.00	883.55
Change from Alt 1A	0.00	0.00	0.00	0.00	0.03	0.00	0.00
Alt 6A	875.92	875.93	875.96	876.04	876.35	874.00	883.55
Change from Alt 1A	0.00	0.00	0.00	0.00	0.02	0.00	0.00

Plate 26 cont'd: Storage Area Alternative Statistics

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Storage Area 536B							
Alt 1A	873.07	873.08	873.12	873.26	873.81	871.12	881.99
Alt 2A	873.07	873.08	873.12	873.25	873.84	871.12	881.85
Change from Alt 1A	0.00	0.00	0.00	-0.02	0.03	0.00	-0.14
Alt 3A	873.07	873.08	873.12	873.27	873.88	871.12	882.05
Change from Alt 1A	0.00	0.00	0.00	0.01	0.07	0.00	0.05
Alt 4A	873.07	873.08	873.12	873.26	873.86	871.12	882.05
Change from Alt 1A	0.00	0.00	0.00	0.00	0.05	0.00	0.05
Alt 5A	873.07	873.08	873.12	873.26	873.85	871.12	882.05
Change from Alt 1A	0.00	0.00	0.00	0.00	0.04	0.00	0.05
Alt 6A	873.07	873.08	873.12	873.27	873.81	871.12	882.05
Change from Alt 1A	0.00	0.00	0.00	0.01	0.00	0.00	0.05
Storage Area 536C							
Alt 1A	871.42	871.44	871.59	872.34	874.38	869.20	881.94
Alt 2A	871.42	871.44	871.58	872.30	874.40	869.20	881.79
Change from Alt 1A	0.00	0.00	-0.01	-0.04	0.01	0.00	-0.15
Alt 3A	871.42	871.44	871.60	872.37	874.45	869.20	881.99
Change from Alt 1A	0.00	0.00	0.00	0.03	0.07	0.00	0.05
Alt 4A	871.42	871.44	871.59	872.38	874.50	869.20	881.99
Change from Alt 1A	0.00	0.00	0.00	0.04	0.12	0.00	0.05
Alt 5A	871.42	871.44	871.60	872.37	874.44	869.20	881.99
Change from Alt 1A	0.00	0.00	0.00	0.03	0.06	0.00	0.05
Alt 6A	871.42	871.44	871.60	872.40	874.50	869.20	881.99
Change from Alt 1A	0.00	0.00	0.00	0.06	0.11	0.00	0.05
Storage Area 536D							
Alt 1A	866.58	866.61	866.72	866.98	867.92	864.58	881.12
Alt 2A	866.58	866.61	866.72	866.98	867.96	864.58	880.90
Change from Alt 1A	0.00	0.00	0.00	0.00	0.04	0.00	-0.23
Alt 3A	866.58	866.61	866.72	867.00	868.20	864.58	881.64
Change from Alt 1A	0.00	0.00	0.00	0.02	0.28	0.00	0.51
Alt 4A	866.58	866.61	866.72	867.02	868.32	864.58	881.64
Change from Alt 1A	0.00	0.00	0.00	0.04	0.40	0.00	0.51
Alt 5A	866.58	866.61	866.72	867.01	868.23	864.58	881.64
Change from Alt 1A	0.00	0.00	0.00	0.03	0.32	0.00	0.51
Alt 6A	866.58	866.61	866.72	867.02	868.25	864.58	881.64
Change from Alt 1A	0.00	0.00	0.00	0.04	0.33	0.00	0.51

Plate 26 cont'd: Storage Area Alternative Statistics

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Storage Area 575A							
Alt 1	913.43	913.52	913.82	914.66	916.33	912.28	921.32
Alt 1: C[1]	915.40	915.46	915.63	915.91	916.55	912.28	921.36
Change from Alt 1	1.98	1.94	1.80	1.25	0.22	0.00	0.04
Alt 1: P[2]	913.43	913.52	913.81	914.61	916.45	912.28	921.47
Change from Alt 1	0.00	0.00	-0.01	-0.05	0.12	0.00	0.15
Alt 1: C + P	915.40	915.46	915.63	915.92	916.82	912.28	921.47
Change from Alt 1	1.98	1.94	1.80	1.26	0.49	0.00	0.15
Alt 2	913.42	913.52	913.82	914.64	916.41	912.28	921.32
Alt 2: C + P	915.40	915.47	915.63	915.97	916.99	912.28	921.47
Change from Alt 2	1.98	1.94	1.81	1.32	0.59	0.00	0.16
Alt 3	913.43	913.53	913.83	914.69	916.36	912.28	921.32
Alt 3: C + P	915.40	915.46	915.63	915.92	916.84	912.28	921.47
Change from Alt 3	1.98	1.94	1.80	1.23	0.48	0.00	0.15
Alt 4	913.43	913.52	913.82	914.66	916.35	912.28	921.32
Alt 4: PEY[3]	913.43	913.53	913.85	914.96	916.52	912.28	921.36
Change from Alt 4	0.00	0.01	0.03	0.30	0.17	0.00	0.04
Alt 4: C + PEY	915.41	915.47	915.65	916.03	916.71	912.28	921.36
Change from Alt 4	1.98	1.95	1.83	1.37	0.36	0.00	0.04
Alt 4: C + P	915.40	915.46	915.63	915.93	916.81	912.28	921.47
Change from Alt 4	1.98	1.94	1.81	1.27	0.46	0.00	0.15
Alt 4: C + P + PEY	915.41	915.47	915.65	916.07	917.21	912.28	921.47
Change from Alt 4	1.98	1.95	1.83	1.40	0.86	0.00	0.15
Alt 5	913.43	913.52	913.83	914.69	916.31	912.28	921.32
Alt 5: C + P	915.40	915.47	915.63	915.92	916.81	912.28	921.47
Change from Alt 5	1.98	1.94	1.80	1.23	0.50	0.00	0.15
Alt 5: C + P + PEY	915.41	915.48	915.67	916.04	916.97	912.28	921.47
Change from Alt 5	1.98	1.96	1.84	1.35	0.65	0.00	0.15
Alt 6	913.43	913.52	913.83	914.68	916.32	912.28	921.32
Alt 6: C + P	915.40	915.47	915.63	915.93	916.80	912.28	921.47
Change from Alt 6	1.98	1.94	1.80	1.24	0.48	0.00	0.15

1) C = Culverts filled half-full of sediment sensitivity
2) P = Pumps removed sensitivity
3) PEY = Simulated pulse every year sensitivity

Plate 27: Interior Drainage Sensitivity - Storage Area Statistics

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	**25th Percentile**	**50th Percentile**	**75th Percentile**	**90th Percentile**	**Min**	**Max**
Storage Area 575B							
Alt 1	914.35	914.41	914.64	915.66	917.48	912.94	922.53
Alt 1: C	916.35	916.37	916.44	916.65	917.57	912.94	922.61
Change from Alt 1	2.00	1.96	1.80	0.99	0.09	0.00	0.09
Alt 1: P	914.35	914.40	914.63	915.58	917.71	912.94	922.81
Change from Alt 1	0.00	0.00	-0.01	-0.08	0.23	0.00	0.28
Alt 1: C + P	916.35	916.37	916.44	916.66	918.11	912.94	922.81
Change from Alt 1	2.00	1.96	1.81	1.00	0.63	0.00	0.28
Alt 2	914.35	914.41	914.63	915.62	917.54	912.94	922.51
Alt 2: C + P	916.35	916.37	916.45	916.71	918.29	912.94	922.81
Change from Alt 2	2.00	1.96	1.82	1.09	0.75	0.00	0.30
Alt 3	914.35	914.41	914.64	915.67	917.52	912.94	922.53
Alt 3: C + P	916.35	916.37	916.44	916.66	918.12	912.94	922.81
Change from Alt 3	2.00	1.96	1.80	0.99	0.60	0.00	0.28
Alt 4	914.35	914.40	914.63	915.67	917.58	912.94	922.53
Alt 4: PEY	914.35	914.41	914.68	916.11	917.75	912.94	922.61
Change from Alt 4	0.00	0.01	0.04	0.44	0.17	0.00	0.09
Alt 4: C + PEY	916.35	916.37	916.45	916.80	917.84	912.94	922.61
Change from Alt 4	2.00	1.97	1.82	1.12	0.26	0.00	0.09
Alt 4: C + P	916.35	916.37	916.44	916.68	918.09	912.94	922.81
Change from Alt 4	2.00	1.96	1.81	1.00	0.50	0.00	0.28
Alt 4: C + P + PEY	916.35	916.37	916.46	916.85	918.82	912.94	922.81
Change from Alt 4	2.00	1.97	1.82	1.18	1.24	0.00	0.28
Alt 5	914.35	914.41	914.64	915.68	917.51	912.94	922.53
Alt 5: C + P	916.35	916.37	916.44	916.67	918.11	912.94	922.81
Change from Alt 5	2.00	1.96	1.81	0.99	0.60	0.00	0.28
Alt 5: C + P + PEY	916.35	916.37	916.46	916.80	918.37	912.94	922.81
Change from Alt 5	2.00	1.97	1.83	1.12	0.86	0.00	0.28
Alt 6	914.35	914.40	914.64	915.68	917.53	912.94	922.53
Alt 6: C + P	916.35	916.37	916.44	916.67	918.04	912.94	922.81
Change from Alt 6	2.00	1.96	1.81	0.99	0.51	0.00	0.28

1) C = Culverts filled half-full of sediment sensitivity
2) P = Pumps removed sensitivity
3) PEY = Simulated pulse every year sensitivity

Plate 27 cont'd: Interior Drainage Sensitivity - Storage Area Statistics

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Storage Area 575C							
Alt 1	913.83	913.85	913.91	914.02	914.66	912.26	919.68
Alt 1: C	915.29	915.31	915.34	915.40	915.66	912.26	919.70
Change from Alt 1	1.46	1.46	1.43	1.38	1.00	0.00	0.02
Alt 1: P	913.83	913.85	913.91	914.02	914.75	912.26	919.75
Change from Alt 1	0.00	0.00	0.00	0.00	0.09	0.00	0.07
Alt 1: C + P	915.29	915.31	915.34	915.40	915.66	912.26	919.75
Change from Alt 1	1.46	1.46	1.43	1.38	1.00	0.00	0.07
Alt 2	913.83	913.85	913.91	914.02	914.87	912.26	919.68
Alt 2: C + P	915.29	915.31	915.34	915.41	915.71	912.26	919.75
Change from Alt 2	1.46	1.46	1.44	1.38	0.84	0.00	0.07
Alt 3	913.83	913.85	913.91	914.02	914.72	912.26	919.68
Alt 3: C + P	915.29	915.31	915.34	915.40	915.66	912.26	919.75
Change from Alt 3	1.46	1.46	1.43	1.38	0.95	0.00	0.07
Alt 4	913.83	913.85	913.91	914.03	914.72	912.26	919.68
Alt 4: PEY	913.83	913.86	913.92	914.09	915.15	912.26	919.70
Change from Alt 4	0.00	0.00	0.01	0.06	0.43	0.00	0.02
Alt 4: C + PEY	915.29	915.31	915.35	915.43	915.99	912.26	919.70
Change from Alt 4	1.46	1.46	1.44	1.40	1.26	0.00	0.02
Alt 4: C + P	915.29	915.31	915.34	915.40	915.67	912.26	919.75
Change from Alt 4	1.46	1.46	1.43	1.38	0.94	0.00	0.07
Alt 4: C + P + PEY	915.29	915.31	915.35	915.43	915.99	912.26	919.75
Change from Alt 4	1.46	1.46	1.44	1.40	1.27	0.00	0.07
Alt 5	913.83	913.85	913.91	914.02	914.65	912.26	919.68
Alt 5: C + P	915.29	915.31	915.34	915.40	915.66	912.26	919.75
Change from Alt 5	1.46	1.46	1.43	1.38	1.02	0.00	0.07
Alt 5: C + P + PEY	915.29	915.31	915.35	915.42	915.73	912.26	919.75
Change from Alt 5	1.47	1.46	1.45	1.40	1.08	0.00	0.07
Alt 6	913.83	913.85	913.91	914.03	914.73	912.26	919.68
Alt 6: C + P	915.29	915.31	915.34	915.40	915.65	912.26	919.75
Change from Alt 6	1.46	1.46	1.43	1.37	0.93	0.00	0.07

1) C = Culverts filled half-full of sediment sensitivity
2) P = Pumps removed sensitivity
3) PEY = Simulated pulse every year sensitivity

Plate 27 cont'd: Interior Drainage Sensitivity - Storage Area Statistics

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Storage Area 575D							
Alt 1	907.36	907.42	907.66	908.45	910.13	905.63	911.97
Alt 1: C	909.88	909.91	909.99	910.14	910.65	905.63	911.98
Change from Alt 1	2.52	2.49	2.33	1.69	0.52	0.00	0.01
Alt 1: P	907.36	907.42	907.65	908.41	910.15	905.63	912.02
Change from Alt 1	0.00	0.00	-0.01	-0.04	0.02	0.00	0.05
Alt 1: C + P	909.88	909.91	909.99	910.14	910.65	905.63	912.02
Change from Alt 1	2.52	2.49	2.33	1.69	0.52	0.00	0.05
Alt 2	907.36	907.42	907.65	908.43	910.27	905.63	911.96
Alt 2: C + P	909.88	909.91	909.99	910.16	910.71	905.63	912.02
Change from Alt 2	2.52	2.49	2.34	1.73	0.44	0.00	0.05
Alt 3	907.36	907.42	907.66	908.45	910.15	905.63	911.97
Alt 3: C + P	909.88	909.91	909.99	910.14	910.65	905.63	912.02
Change from Alt 3	2.52	2.49	2.33	1.69	0.50	0.00	0.05
Alt 4	907.36	907.42	907.66	908.46	910.36	905.63	911.97
Alt 4: PEY	907.36	907.43	907.70	908.96	910.74	905.63	911.98
Change from Alt 4	0.00	0.01	0.04	0.50	0.38	0.00	0.01
Alt 4: C + PEY	909.88	909.92	910.00	910.22	910.81	905.63	911.98
Change from Alt 4	2.52	2.49	2.35	1.76	0.45	0.00	0.01
Alt 4: C + P	909.88	909.91	909.99	910.15	910.66	905.63	912.02
Change from Alt 4	2.52	2.49	2.33	1.69	0.30	0.00	0.05
Alt 4: C + P + PEY	909.88	909.92	910.00	910.22	910.82	905.63	912.02
Change from Alt 4	2.52	2.49	2.35	1.76	0.45	0.00	0.05
Alt 5	907.36	907.42	907.66	908.45	910.17	905.63	911.97
Alt 5: C + P	909.88	909.91	909.99	910.15	910.64	905.63	912.02
Change from Alt 5	2.52	2.49	2.33	1.70	0.47	0.00	0.05
Alt 5: C + P + PEY	909.88	909.92	910.01	910.20	910.73	905.63	912.02
Change from Alt 5	2.53	2.50	2.35	1.75	0.55	0.00	0.05
Alt 6	907.36	907.42	907.67	908.47	910.21	905.63	911.97
Alt 6: C + P	909.88	909.91	909.99	910.15	910.63	905.63	912.02
Change from Alt 6	2.52	2.49	2.32	1.68	0.43	0.00	0.05

1) C = Culverts filled half-full of sediment sensitivity
2) P = Pumps removed sensitivity
3) PEY = Simulated pulse every year sensitivity

Plate 27 cont'd: Interior Drainage Sensitivity - Storage Area Statistics

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Storage Area 575E							
Alt 1	896.29	896.31	896.41	897.02	898.68	895.18	904.54
Alt 1: C	898.53	898.53	898.55	898.61	899.09	895.18	904.73
Change from Alt 1	2.24	2.22	2.14	1.59	0.41	0.00	0.19
Alt 1: P	896.29	896.31	896.41	896.96	898.82	895.18	908.60
Change from Alt 1	0.00	0.00	0.00	-0.06	0.14	0.00	4.06
Alt 1: C + P	898.53	898.53	898.55	898.61	899.14	895.18	909.03
Change from Alt 1	2.24	2.22	2.14	1.59	0.46	0.00	4.49
Alt 2	896.29	896.31	896.41	896.97	898.83	895.18	904.52
Alt 2: C + P	898.53	898.53	898.55	898.62	899.30	895.18	909.03
Change from Alt 2	2.24	2.22	2.15	1.65	0.47	0.00	4.51
Alt 3	896.29	896.31	896.41	897.02	898.73	895.18	904.55
Alt 3: C + P	898.53	898.53	898.55	898.61	899.14	895.18	909.03
Change from Alt 3	2.24	2.22	2.14	1.59	0.41	0.00	4.48
Alt 4	896.29	896.31	896.41	896.99	898.64	895.18	904.54
Alt 4: PEY	896.29	896.31	896.42	897.28	899.24	895.18	904.72
Change from Alt 4	0.00	0.00	0.01	0.28	0.59	0.00	0.18
Alt 4: C + PEY	898.53	898.53	898.55	898.65	899.57	895.18	904.70
Change from Alt 4	2.24	2.22	2.15	1.66	0.92	0.00	0.16
Alt 4: C + P	898.53	898.53	898.55	898.61	899.16	895.18	909.03
Change from Alt 4	2.24	2.22	2.14	1.62	0.51	0.00	4.49
Alt 4: C + P + PEY	898.53	898.53	898.55	898.65	899.62	895.18	909.03
Change from Alt 4	2.24	2.22	2.15	1.66	0.97	0.00	4.49
Alt 5	896.29	896.31	896.41	897.02	898.63	895.18	904.55
Alt 5: C + P	898.53	898.53	898.55	898.61	899.12	895.18	909.03
Change from Alt 5	2.24	2.22	2.14	1.59	0.49	0.00	4.48
Alt 5: C + P + PEY	898.53	898.53	898.56	898.65	899.29	895.18	909.03
Change from Alt 5	2.24	2.22	2.15	1.62	0.66	0.00	4.48
Alt 6	896.29	896.31	896.41	897.01	898.67	895.18	904.55
Alt 6: C + P	898.53	898.53	898.55	898.61	899.12	895.18	909.03
Change from Alt 6	2.24	2.22	2.14	1.60	0.45	0.00	4.48

1) C = Culverts filled half-full of sediment sensitivity
2) P = Pumps removed sensitivity
3) PEY = Simulated pulse every year sensitivity

Plate 27 cont'd: Interior Drainage Sensitivity - Storage Area Statistics

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Storage Area 575F							
Alt 1	893.54	893.62	893.80	894.32	895.96	892.00	904.54
Alt 1: C	895.73	895.76	895.82	895.94	896.47	892.00	904.73
Change from Alt 1	2.19	2.15	2.02	1.62	0.51	0.00	0.19
Alt 1: P	893.54	893.62	893.80	894.32	895.97	892.00	908.60
Change from Alt 1	0.00	0.00	0.00	0.00	0.01	0.00	4.06
Alt 1: C + P	895.73	895.76	895.82	895.95	896.70	892.00	909.03
Change from Alt 1	2.19	2.15	2.03	1.62	0.73	0.00	4.49
Alt 2	893.54	893.62	893.79	894.30	895.96	892.00	904.53
Alt 2: C + P	895.73	895.76	895.82	895.95	896.78	892.00	909.03
Change from Alt 2	2.19	2.15	2.03	1.65	0.82	0.00	4.51
Alt 3	893.54	893.62	893.80	894.33	896.00	892.00	904.55
Alt 3: C + P	895.73	895.76	895.82	895.95	896.70	892.00	909.03
Change from Alt 3	2.19	2.15	2.02	1.61	0.70	0.00	4.48
Alt 4	893.54	893.62	893.80	894.36	896.18	892.00	904.54
Alt 4: PEY	893.54	893.63	893.84	894.62	896.87	892.00	904.72
Change from Alt 4	0.00	0.01	0.04	0.26	0.69	0.00	0.18
Alt 4: C + PEY	895.73	895.77	895.83	896.01	897.05	892.00	904.70
Change from Alt 4	2.19	2.15	2.03	1.65	0.87	0.00	0.16
Alt 4: C + P	895.73	895.76	895.82	895.95	896.78	892.00	909.03
Change from Alt 4	2.19	2.15	2.02	1.59	0.59	0.00	4.49
Alt 4: C + P + PEY	895.73	895.77	895.83	896.02	898.04	892.00	909.03
Change from Alt 4	2.19	2.15	2.03	1.66	1.85	0.00	4.49
Alt 5	893.54	893.62	893.80	894.33	896.08	892.00	904.55
Alt 5: C + P	895.73	895.76	895.82	895.95	896.72	892.00	909.03
Change from Alt 5	2.19	2.15	2.02	1.62	0.64	0.00	4.48
Alt 5: C + P + PEY	895.73	895.77	895.84	896.01	897.22	892.00	909.03
Change from Alt 5	2.19	2.15	2.04	1.68	1.14	0.00	4.48
Alt 6	893.54	893.62	893.80	894.35	896.05	892.00	904.55
Alt 6: C + P	895.73	895.76	895.82	895.95	896.68	892.00	909.03
Change from Alt 6	2.19	2.15	2.02	1.60	0.63	0.00	4.48

1) C = Culverts filled half-full of sediment sensitivity
2) P = Pumps removed sensitivity
3) PEY = Simulated pulse every year sensitivity

Plate 27 cont'd: Interior Drainage Sensitivity - Storage Area Statistics

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Storage Area 575G							
Alt 1	898.09	898.17	898.39	899.67	901.58	890.61	909.78
Alt 1: C	901.05	901.08	901.14	901.28	901.98	890.61	909.79
Change from Alt 1	2.96	2.92	2.75	1.60	0.40	0.00	0.01
Alt 1: P	898.09	898.17	898.39	899.69	901.56	890.61	910.14
Change from Alt 1	0.00	0.00	0.00	0.02	-0.03	0.00	0.36
Alt 1: C + P	901.05	901.08	901.14	901.28	902.01	890.61	910.17
Change from Alt 1	2.96	2.92	2.75	1.61	0.43	0.00	0.39
Alt 2	898.09	898.17	898.39	899.67	901.60	890.61	909.78
Alt 2: C + P	901.05	901.08	901.14	901.28	902.02	890.61	910.17
Change from Alt 2	2.96	2.92	2.75	1.61	0.42	0.00	0.39
Alt 3	898.09	898.17	898.39	899.67	901.58	890.61	909.78
Alt 3: C + P	901.05	901.08	901.14	901.28	902.01	890.61	910.17
Change from Alt 3	2.96	2.92	2.75	1.61	0.42	0.00	0.39
Alt 4	898.09	898.17	898.39	899.69	901.60	890.61	909.78
Alt 4: PEY	898.10	898.17	898.42	899.76	901.61	890.61	909.79
Change from Alt 4	0.00	0.01	0.03	0.07	0.01	0.00	0.01
Alt 4: C + PEY	901.05	901.08	901.14	901.28	902.01	890.61	909.79
Change from Alt 4	2.96	2.92	2.75	1.59	0.41	0.00	0.01
Alt 4: C + P	901.05	901.08	901.14	901.28	902.01	890.61	910.17
Change from Alt 4	2.96	2.92	2.75	1.59	0.41	0.00	0.39
Alt 4: C + P + PEY	901.05	901.08	901.14	901.28	902.05	890.61	910.18
Change from Alt 4	2.96	2.92	2.75	1.60	0.45	0.00	0.40
Alt 5	898.09	898.17	898.39	899.68	901.59	890.61	909.78
Alt 5: C + P	901.05	901.08	901.14	901.28	902.01	890.61	910.17
Change from Alt 5	2.96	2.92	2.75	1.60	0.42	0.00	0.39
Alt 5: C + P + PEY	901.05	901.08	901.14	901.28	902.02	890.61	910.17
Change from Alt 5	2.96	2.92	2.75	1.60	0.43	0.00	0.39
Alt 6	898.09	898.17	898.39	899.67	901.58	890.61	909.78
Alt 6: C + P	901.05	901.08	901.14	901.28	902.01	890.61	910.17
Change from Alt 6	2.96	2.92	2.75	1.61	0.42	0.00	0.39

1) C = Culverts filled half-full of sediment sensitivity
2) P = Pumps removed sensitivity
3) PEY = Simulated pulse every year sensitivity

Plate 27 cont'd: Interior Drainage Sensitivity - Storage Area Statistics

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Storage Area 575H							
Alt 1	898.26	898.42	899.13	900.37	900.81	896.86	906.75
Alt 1: C	899.72	899.89	900.24	900.62	900.97	896.86	906.81
Change from Alt 1	1.46	1.47	1.11	0.25	0.15	0.00	0.07
Alt 1: P	898.26	898.42	899.15	900.62	902.61	896.86	910.15
Change from Alt 1	0.00	0.00	0.02	0.25	1.80	0.00	3.40
Alt 1: C + P	899.72	899.90	900.36	901.41	903.25	896.86	910.18
Change from Alt 1	1.46	1.48	1.23	1.04	2.44	0.00	3.43
Alt 2	898.26	898.42	899.13	900.37	900.81	896.86	906.74
Alt 2: C + P	899.72	899.90	900.36	901.41	903.25	896.86	910.18
Change from Alt 2	1.46	1.48	1.23	1.04	2.44	0.00	3.44
Alt 3	898.26	898.42	899.13	900.37	900.81	896.86	906.75
Alt 3: C + P	899.72	899.90	900.36	901.41	903.25	896.86	910.18
Change from Alt 3	1.46	1.48	1.22	1.04	2.44	0.00	3.43
Alt 4	898.26	898.42	899.13	900.37	900.81	896.86	906.75
Alt 4: PEY	898.26	898.44	899.17	900.38	900.82	896.86	906.81
Change from Alt 4	0.00	0.02	0.04	0.01	0.01	0.00	0.06
Alt 4: C + PEY	899.72	899.89	900.24	900.62	900.97	896.86	906.80
Change from Alt 4	1.46	1.47	1.11	0.25	0.15	0.00	0.05
Alt 4: C + P	899.72	899.90	900.36	901.41	903.25	896.86	910.18
Change from Alt 4	1.46	1.48	1.23	1.04	2.44	0.00	3.43
Alt 4: C + P + PEY	899.72	899.90	900.37	901.43	903.28	896.86	910.19
Change from Alt 4	1.46	1.48	1.23	1.05	2.47	0.00	3.44
Alt 5	898.26	898.42	899.13	900.37	900.81	896.86	906.75
Alt 5: C + P	899.72	899.90	900.36	901.41	903.25	896.86	910.18
Change from Alt 5	1.46	1.48	1.23	1.03	2.43	0.00	3.43
Alt 5: C + P + PEY	899.72	899.90	900.36	901.42	903.25	896.86	910.18
Change from Alt 5	1.46	1.48	1.23	1.04	2.44	0.00	3.43
Alt 6	898.26	898.42	899.13	900.37	900.81	896.86	906.75
Alt 6: C + P	899.72	899.90	900.36	901.41	903.25	896.86	910.18
Change from Alt 6	1.46	1.48	1.23	1.04	2.43	0.00	3.43

1) C = Culverts filled half-full of sediment sensitivity
2) P = Pumps removed sensitivity
3) PEY = Simulated pulse every year sensitivity

Plate 27 cont'd: Interior Drainage Sensitivity - Storage Area Statistics

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Storage Area 536A							
Alt 1	875.92	875.93	875.96	876.04	876.32	874.00	883.55
Alt 1: C	877.42	877.42	877.42	877.46	877.55	874.00	883.55
Change from Alt 1	1.50	1.49	1.46	1.43	1.23	0.00	0.00
Alt 1: P	875.92	875.93	875.96	876.04	876.28	874.00	883.55
Change from Alt 1	0.00	0.00	0.00	0.00	-0.04	0.00	0.00
Alt 1: C + P	877.42	877.42	877.42	877.46	877.55	874.00	883.55
Change from Alt 1	1.50	1.49	1.46	1.43	1.23	0.00	0.00
Alt 2	875.92	875.93	875.96	876.04	876.35	874.00	883.55
Alt 2: C + P	877.42	877.42	877.42	877.46	877.56	874.00	883.55
Change from Alt 2	1.50	1.49	1.46	1.43	1.22	0.00	0.00
Alt 3	875.92	875.93	875.96	876.04	876.34	874.00	883.55
Alt 3: C + P	877.42	877.42	877.42	877.46	877.55	874.00	883.55
Change from Alt 3	1.50	1.49	1.46	1.43	1.21	0.00	0.00
Alt 4	875.92	875.93	875.96	876.04	876.38	874.00	883.55
Alt 4: PEY	875.92	875.93	875.97	876.08	877.07	874.00	883.55
Change from Alt 4	0.00	0.00	0.01	0.04	0.70	0.00	0.00
Alt 4: C + PEY	877.42	877.42	877.42	877.48	877.94	874.00	883.55
Change from Alt 4	1.50	1.49	1.46	1.44	1.56	0.00	0.00
Alt 4: C + P	877.42	877.42	877.42	877.46	877.56	874.00	883.55
Change from Alt 4	1.50	1.49	1.46	1.42	1.18	0.00	0.00
Alt 4: C + P + PEY	877.42	877.42	877.42	877.48	877.94	874.00	883.55
Change from Alt 4	1.50	1.49	1.46	1.44	1.56	0.00	0.00
Alt 5	875.92	875.93	875.96	876.04	876.36	874.00	883.55
Alt 5: C + P	877.42	877.42	877.42	877.46	877.55	874.00	883.55
Change from Alt 5	1.50	1.49	1.46	1.42	1.20	0.00	0.00
Alt 5: C + P + PEY	877.42	877.42	877.42	877.47	877.74	874.00	883.55
Change from Alt 5	1.50	1.49	1.46	1.44	1.38	0.00	0.00
Alt 6	875.92	875.93	875.96	876.04	876.35	874.00	883.55
Alt 6: C + P	877.42	877.42	877.42	877.46	877.55	874.00	883.55
Change from Alt 6	1.50	1.49	1.46	1.42	1.21	0.00	0.00

1) C = Culverts filled half-full of sediment sensitivity
2) P = Pumps removed sensitivity
3) PEY = Simulated pulse every year sensitivity

Plate 27 cont'd: Interior Drainage Sensitivity - Storage Area Statistics

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Storage Area 536B							
Alt 1	873.07	873.08	873.12	873.26	873.81	871.12	881.99
Alt 1: C	874.79	874.79	874.80	874.84	875.10	871.12	882.82
Change from Alt 1	1.72	1.71	1.68	1.58	1.29	0.00	0.83
Alt 1: P	873.07	873.08	873.12	873.27	873.82	871.12	883.08
Change from Alt 1	0.00	0.00	0.00	0.01	0.01	0.00	1.09
Alt 1: C + P	874.79	874.79	874.80	874.84	875.11	871.12	883.08
Change from Alt 1	1.72	1.71	1.68	1.58	1.30	0.00	1.09
Alt 2	873.07	873.08	873.12	873.25	873.84	871.12	881.85
Alt 2: C + P	874.79	874.79	874.80	874.84	875.11	871.12	882.69
Change from Alt 2	1.72	1.71	1.68	1.60	1.27	0.00	0.83
Alt 3	873.07	873.08	873.12	873.27	873.88	871.12	882.05
Alt 3: C + P	874.79	874.79	874.80	874.84	875.11	871.12	883.12
Change from Alt 3	1.72	1.71	1.68	1.57	1.22	0.00	1.08
Alt 4	873.07	873.08	873.12	873.26	873.86	871.12	882.05
Alt 4: PEY	873.07	873.08	873.13	873.33	874.10	871.12	882.87
Change from Alt 4	0.00	0.00	0.01	0.07	0.24	0.00	0.82
Alt 4: C + PEY	874.79	874.79	874.80	874.86	875.31	871.12	882.87
Change from Alt 4	1.72	1.71	1.68	1.60	1.45	0.00	0.82
Alt 4: C + P	874.79	874.79	874.80	874.84	875.10	871.12	883.12
Change from Alt 4	1.72	1.71	1.68	1.58	1.24	0.00	1.08
Alt 4: C + P + PEY	874.79	874.79	874.80	874.86	875.32	871.12	883.12
Change from Alt 4	1.72	1.71	1.68	1.60	1.45	0.00	1.08
Alt 5	873.07	873.08	873.12	873.26	873.85	871.12	882.05
Alt 5: C + P	874.79	874.79	874.80	874.84	875.10	871.12	883.12
Change from Alt 5	1.72	1.71	1.68	1.58	1.26	0.00	1.08
Alt 5: C + P + PEY	874.79	874.79	874.80	874.85	875.16	871.12	883.12
Change from Alt 5	1.72	1.71	1.68	1.59	1.31	0.00	1.08
Alt 6	873.07	873.08	873.12	873.27	873.81	871.12	882.05
Alt 6: C + P	874.79	874.79	874.80	874.84	875.11	871.12	883.12
Change from Alt 6	1.72	1.71	1.68	1.57	1.29	0.00	1.08

1) C = Culverts filled half-full of sediment sensitivity
2) P = Pumps removed sensitivity
3) PEY = Simulated pulse every year sensitivity

Plate 27 cont'd: Interior Drainage Sensitivity - Storage Area Statistics

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Storage Area 536C							
Alt 1	871.42	871.44	871.59	872.34	874.38	869.20	881.94
Alt 1: C	874.42	874.42	874.45	874.54	874.95	869.20	882.82
Change from Alt 1	3.00	2.98	2.85	2.20	0.56	0.00	0.89
Alt 1: P	871.42	871.44	871.59	872.32	874.43	869.20	883.08
Change from Alt 1	0.00	0.00	0.00	-0.02	0.05	0.00	1.15
Alt 1: C + P	874.42	874.42	874.45	874.54	874.95	869.20	883.08
Change from Alt 1	3.00	2.98	2.85	2.20	0.57	0.00	1.15
Alt 2	871.42	871.44	871.58	872.30	874.40	869.20	881.79
Alt 2: C + P	874.42	874.42	874.45	874.55	875.04	869.20	882.69
Change from Alt 2	3.00	2.98	2.86	2.25	0.65	0.00	0.90
Alt 3	871.42	871.44	871.60	872.37	874.45	869.20	881.99
Alt 3: C + P	874.42	874.42	874.45	874.54	874.95	869.20	883.12
Change from Alt 3	3.00	2.98	2.85	2.17	0.50	0.00	1.13
Alt 4	871.42	871.44	871.59	872.38	874.50	869.20	881.99
Alt 4: PEY	871.42	871.44	871.61	873.02	875.70	869.20	882.87
Change from Alt 4	0.00	0.00	0.02	0.64	1.20	0.00	0.88
Alt 4: C + PEY	874.42	874.42	874.46	874.60	876.08	869.20	882.87
Change from Alt 4	3.00	2.98	2.86	2.22	1.57	0.00	0.88
Alt 4: C + P	874.42	874.42	874.45	874.55	875.03	869.20	883.12
Change from Alt 4	3.00	2.98	2.85	2.16	0.53	0.00	1.13
Alt 4: C + P + PEY	874.42	874.42	874.46	874.60	876.09	869.20	883.12
Change from Alt 4	3.00	2.98	2.86	2.22	1.58	0.00	1.14
Alt 5	871.42	871.44	871.60	872.37	874.44	869.20	881.99
Alt 5: C + P	874.42	874.42	874.45	874.54	874.99	869.20	883.12
Change from Alt 5	3.00	2.98	2.85	2.17	0.54	0.00	1.13
Alt 5: C + P + PEY	874.42	874.42	874.46	874.60	875.65	869.20	883.12
Change from Alt 5	3.00	2.98	2.86	2.23	1.21	0.00	1.14
Alt 6	871.42	871.44	871.60	872.40	874.50	869.20	881.99
Alt 6: C + P	874.42	874.42	874.45	874.54	874.97	869.20	883.12
Change from Alt 6	3.00	2.98	2.85	2.14	0.47	0.00	1.13

1) C = Culverts filled half-full of sediment sensitivity
2) P = Pumps removed sensitivity
3) PEY = Simulated pulse every year sensitivity

Plate 27 cont'd: Interior Drainage Sensitivity - Storage Area Statistics

Water Surface Elevation (ft, NAVD88)							
	10th Percentile	25th Percentile	50th Percentile	75th Percentile	90th Percentile	Min	Max
Storage Area 536D							
Alt 1	866.58	866.61	866.72	866.98	867.92	864.58	881.12
Alt 1: C	869.58	869.59	869.63	869.73	869.97	864.58	882.82
Change from Alt 1	3.00	2.98	2.91	2.75	2.05	0.00	1.70
Alt 1: P	866.58	866.61	866.72	866.98	867.93	864.58	883.08
Change from Alt 1	0.00	0.00	0.00	0.00	0.01	0.00	1.96
Alt 1: C + P	869.58	869.59	869.63	869.73	869.98	864.58	883.08
Change from Alt 1	3.00	2.98	2.91	2.75	2.06	0.00	1.96
Alt 2	866.58	866.61	866.72	866.98	867.96	864.58	880.90
Alt 2: C + P	869.58	869.59	869.63	869.73	870.01	864.58	882.68
Change from Alt 2	3.00	2.98	2.91	2.75	2.05	0.00	1.79
Alt 3	866.58	866.61	866.72	867.00	868.20	864.58	881.64
Alt 3: C + P	869.58	869.59	869.63	869.73	869.98	864.58	883.12
Change from Alt 3	3.00	2.98	2.91	2.73	1.78	0.00	1.48
Alt 4	866.58	866.61	866.72	867.02	868.32	864.58	881.64
Alt 4: PEY	866.58	866.61	866.74	867.16	869.37	864.58	882.87
Change from Alt 4	0.00	0.00	0.02	0.14	1.06	0.00	1.23
Alt 4: C + PEY	869.58	869.59	869.64	869.77	870.40	864.58	882.87
Change from Alt 4	3.00	2.98	2.91	2.75	2.08	0.00	1.23
Alt 4: C + P	869.58	869.59	869.63	869.73	869.99	864.58	883.12
Change from Alt 4	3.00	2.98	2.90	2.71	1.68	0.00	1.48
Alt 4: C + P + PEY	869.58	869.59	869.64	869.77	870.44	864.58	883.12
Change from Alt 4	3.00	2.98	2.91	2.75	2.12	0.00	1.49
Alt 5	866.58	866.61	866.72	867.01	868.23	864.58	881.64
Alt 5: C + P	869.58	869.59	869.63	869.73	869.98	864.58	883.12
Change from Alt 5	3.00	2.98	2.91	2.72	1.75	0.00	1.48
Alt 5: C + P + PEY	869.58	869.59	869.64	869.77	870.25	864.58	883.12
Change from Alt 5	3.00	2.98	2.92	2.76	2.01	0.00	1.48
Alt 6	866.58	866.61	866.72	867.02	868.25	864.58	881.64
Alt 6: C + P	869.58	869.59	869.63	869.73	869.98	864.58	883.12
Change from Alt 6	3.00	2.98	2.90	2.71	1.73	0.00	1.48

1) C = Culverts filled half-full of sediment sensitivity
2) P = Pumps removed sensitivity
3) PEY = Simulated pulse every year sensitivity

Plate 27 cont'd: Interior Drainage Sensitivity - Storage Area Statistics

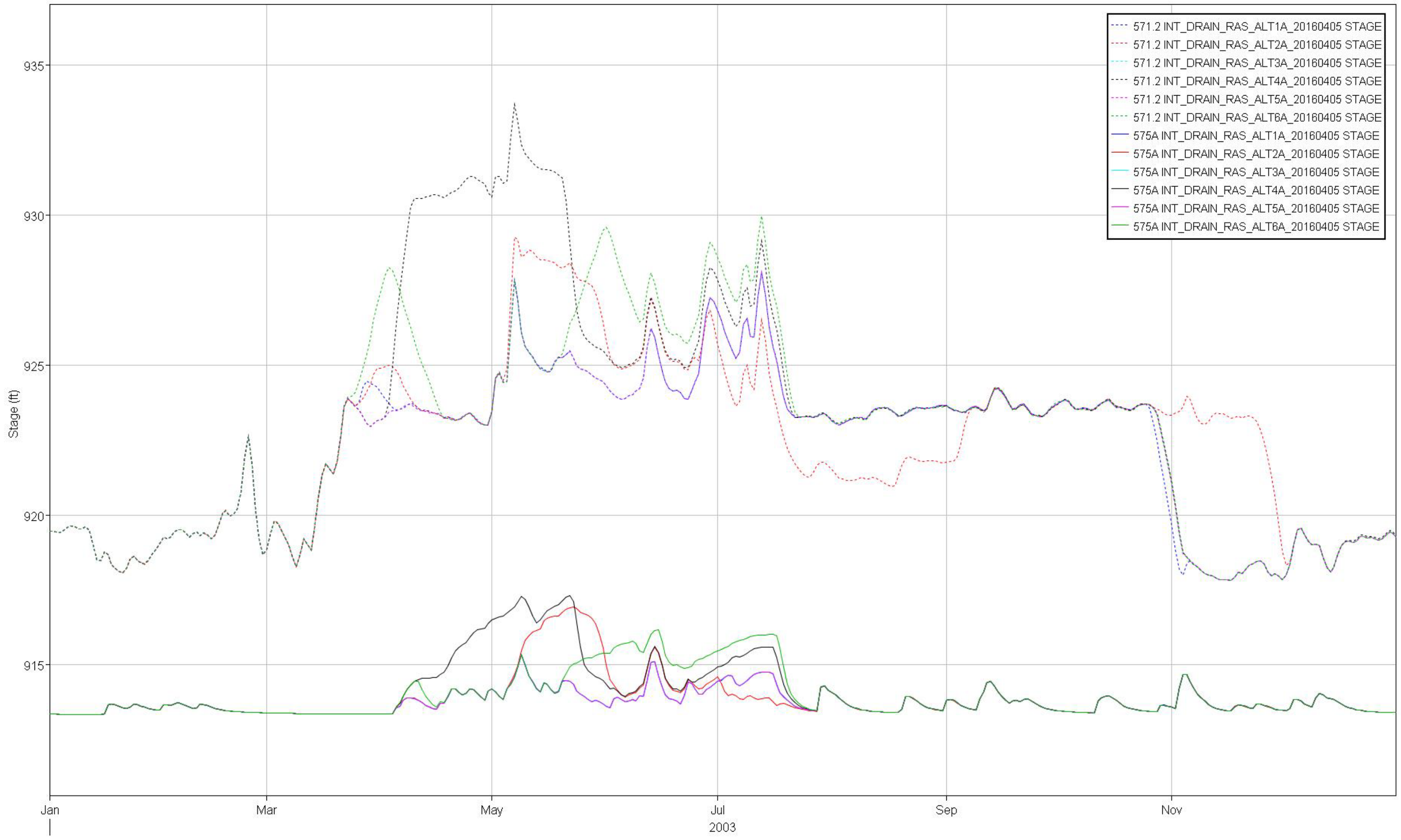


Plate 28: Interior Drainage Example Output

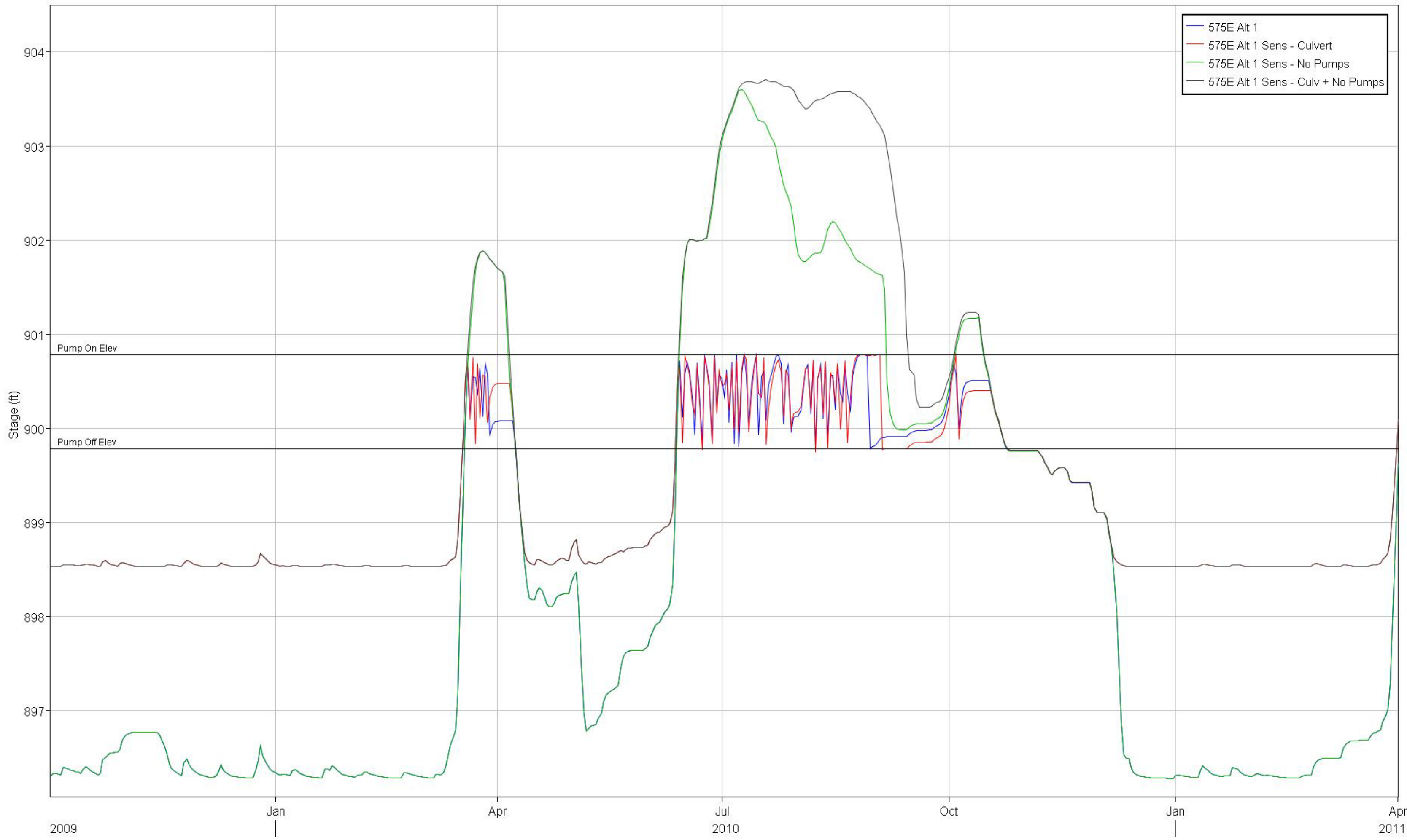


Plate 29: Interior Drainage Sensitivity Example Output

Levee Station	Lateral Structure/ Storage Area Connection	Lateral Structure Station(s)	# of Pipes	Solution Criteria	Diameter (ft)	Chart	Scale #	US Invert Elev (NAVD88, ft)	DS Invert Elev (NAVD88, ft)	Culvert Length (ft)	Entrance Loss Coefficient	Exit Loss Coefficient	N-Value for Top	N-Value for Bottom	Depth to Use Bottom	Depth Blocked
495+35	575a1-m	5565	1	Highest US EG	4	2	1	930.09	928.84	135	0.5	1	0.024	0.024	0	0
24+91	575a1-m	14560	1	Highest US EG	4	1	1	928.95	927.05	374	0.5	1	0.013	0.013	0	0
684+80	575b1-m	7105	1	Highest US EG	3.5	2	1	924.42	922.74	169	0.5	1	0.024	0.024	0	0
172+00	575b2-m	29510	1	Highest US EG	4	1	1	914.35	912.81	309	0.5	1	0.013	0.013	0	0
320+35 - #1	575a2-m	5491	1	Highest US EG	4	1	1	913.35	912.21	237	0.5	1	0.013	0.013	0	0
320+35 - #2	575a2-m	5500	1	Highest US EG	5	1	1	913.35	912.21	237	0.5	1	0.013	0.013	0	0
320+35 - #3	575a2-m	5509	1	Highest US EG	4	1	1	913.35	912.21	237	0.5	1	0.013	0.013	0	0
1114+50	575c-m	7475	1	Highest US EG	3	2	1	915.35	913.86	153	0.5	1	0.024	0.024	0	0
1161+40	575c-m	12165	1	Highest US EG	3	2	1	913.79	912.27	153	0.5	1	0.024	0.024	0	0
1167+50	575c-m	12775	1	Highest US EG	4	2	1	914.80	913.71	143	0.5	1	0.024	0.024	0	0
450+00	575d-m	5950 5980	2	Highest US EG	5	1	1	910.45	909.33	238	0.5	1	0.013	0.013	0	0
1404+00	575d-m	23708 23698	2	Highest US EG	5	2	1	907.34	905.59	159	0.5	1	0.024	0.024	0	0
NR 2165+00 R	575g-t	324 335 347 359 373	5	Highest US EG	6	2	1	898.05(1)	896.69(1)	173	0.5	1	0.024	0.024	0	0
NR 2213+54 R	575h-t	14890	1	Highest US EG	4	2	1	902.35	900.98	113	0.5	1	0.024	0.024	0	0
NR 254+30 R	575h-t	14590	1	Highest US EG	3	2	1	898.22	897.51	116	0.5	1	0.024	0.024	0	0
1769+35	575e-m	24370 34380	2	Highest US EG	4.5	2	1	896.28	895.11	179	0.5	1	0.024	0.024	0	0
NR 1900+32 R	575f-t	25500 25510	2	Highest US EG	4	2	1	893.81	892.61	126	0.5	1	0.024	0.024	0	0
NR 1915+30 R	575f-t	24005 24015	2	Highest US EG	4.5	2	1	893.47	892.50	141	0.5	1	0.024	0.024	0	0
NR 155+40 R	575h-t	5000	1	Highest US EG	3	2	1	901.39(1)	900.82(1)	574	0.5	1	0.024	0.024	0	0
NR 102+29 R	575h-t	5	1	Highest US EG	4.5	2	1	907.09	902.94	294	0.5	1	0.024	0.024	0	0

(1) - Inverts were shifted up 5 feet because they were near or below the Nishnabotna River invert.

Plate 30: L-575 Culvert Information

Levee Station	Lateral Structure/ Storage Area Connection	Lateral Structure Station(s)	# of Pipes	Solution Criteria	Diameter (ft)	Chart	Scale #	US Invert Elev (NAVD88, ft)	DS Invert Elev (NAVD88, ft)	Culvert Length (ft)	Entrance Loss Coefficient	Exit Loss Coefficient	N-Value for Top	N-Value for Bottom	Depth to Use Bottom	Depth Blocked
RC 412+70 L	536a-t	9850	1	Highest US EG	3	2	1	875.92	875.37	90	0.5	1	0.024	0.024	0	0
RC 417+00	536a-t	10285	1	Highest US EG	3	2	1	876.32	875.49	88	0.5	1	0.024	0.024	0	0
RC 485+00 L	536b-t	5851	1	Highest US EG	3.5	2	1	873.04	871.95	112.5	0.5	1	0.024	0.024	0	0
520+80[(1)]	536b-c	8031	1	Highest US EG	2.5	2	1	877.77	877.29	48	0.5	1	0.024	0.024	0	0
535+10	536c-m	1420 1430	2	Highest US EG	6	2	1	871.42	869.41	128.5	0.5	1	0.024	0.024	0	0
777+20	536d-m	24120 24130	2	Highest US EG	6	2	1	866.58	865.24	118.5	0.5	1	0.024	0.024	0	0

(1) - 520+80 is a culvert under an interior road (G Ave) adjacent to the levee.

Plate 31: L-536 Culvert Information

				Pump Connection Data			Pump Group Data					
Levee Station	Pump Line Size and Type	Curve # Used	Culvert Length (ft)	Pump From (SA)	Pump To (XS)	Distance from US RS to the pump outlet	Group Name	# of Pumps in Group	Startup (min)	Shutdown (min)	WS Elev On (ft)	WS Elev Off (ft)
L-575												
172+00	30" INLINE WATER HOG	3	309	575B	562.35	835	Group #1	1	0	0	918.35	917.35
320+35	2-30" INLINE WATER HOG	3	237	575A	561.4	700	Group #1	2	0	0	917.35	916.35
1404+00	24" STEEL	2	159	575D	554.7	870	Pump Group #1	1	0	0	912.34	911.34
1404+00	48" STEEL	4	159	575D	554.7	922	Pump Group #2	1	0	0	912.34	911.34
NR 2165+00 R	24" STEEL	2	173	575F	41975	600	Pump Group #1	1	0	0	904.05	903.05
NR 2165+00 R	30" STEEL	3	173	575F	41975	575	Pump Group #2	1	0	0	904.05	903.05
NR 2165+00 R	2-48" STEEL	4	173	575G	41975	340	Group #1	2	0	0	904.05	903.05
NR 2213+54 R	18" STEEL	1	113	575H	45994	170	Pump Group #1	1	0	0	906.35	905.35
NR 2213+54 R	24" STEEL	2	113	575H	45994	170	Pump Group #2	1	0	0	906.35	905.35
NR 254+30 R	18" STEEL	1	116	575H	46284	50	Group #1	1	0	0	901.22	900.22
1769+35	2-24" STEEL	2	179	575E	546.15	1837	Group #1	2	0	0	900.78	899.78
NR 1900+32 R	24" STEEL	3	126	575F	14635	130	Group #1	1	0	0	897.81	896.81
NR 1915+30 R	36" STEEL	4	141	575F	18586	1700	Group #1	1	0	0	897.97	896.97
NR 155+40 R	18" STEEL	1	574	575H	56606	165	Group #1	1	0	0	904.39	903.39
NR 102+29 R	15" STEEL	1	294	575H	61096	0	Group #1	1	0	0	911.59	910.59
L-536												
535+10	18" STEEL	1	128.5	536C	521.85	1260	Group #1	1	0	0	877.42	876.42
777+20	24" STEEL	2	118.5	536D	516.3	604	Pump Group #1	1	0	0	872.56	871.56
777+20	30" STEEL	3	118.5	536D	516.3	639	Pump Group #2	1	0	0	872.56	871.56

Plate 32: Pump Data

Curve #1		Curve #2		Curve #3		Curve #4	
Fairbanks Morse - 18”		Fairbanks Morse - 24”		Cascade 24P - 24”		Cascade 42P - 42”	
Head (ft)(1)	**Flow (cfs)**	**Head (ft)(1)**	**Flow (cfs)**	**Head (ft)(1)**	**Flow (cfs)**	**Head (ft)(1)**	**Flow (cfs)**
30.94	12.71	25.29	21.08	27.00	21.54	28.18	48.09
29.86	13.15	23.93	22.45	25.67	22.77	27.76	52.58
27.28	14.23	22.57	23.54	24.10	24.22	27.07	57.09
24.63	15.29	20.98	24.64	22.26	25.70	26.18	62.17
21.90	16.28	19.05	25.73	19.72	27.57	24.86	68.18
19.20	17.16	17.13	26.82	17.35	29.40	23.32	74.09
16.50	17.98	14.92	27.91	14.49	31.30	20.52	82.29
13.76	18.67	12.47	29.00	11.76	33.10	17.72	88.52
11.00	19.31	9.99	30.06	9.04	34.71	14.01	94.56
8.28	19.90	7.25	31.01	6.32	36.01	10.62	99.69
5.52	20.41	4.62	31.81	3.60	36.96	6.85	104.27
2.84	20.89	1.98	32.46	1.11	37.62	3.97	106.84
0.16	21.37					1.32	109.18

(1) Head values include friction and minor losses.

Plate 33: Pump Curve Data

	Curve #1	Curve #2	Curve #3	Curve #4
Average Pipe Length (ft)	245.1	148.5	192.7	157.7
N-Value	0.012	0.012	0.012	0.012
Diameter (ft)	1.5	2	2	3.5
Friction Head Loss (ft)	0.01887	0.00247	0.00320	0.00013
Sum of Loss Coefficients, K	1.5	1.5	1.5	1.5

Plate 34: Pump Curve Loss Parameters

L575 AND L536 SEEPAGE FLOW DETERMINATIONS

MISSOURI RIVER LEVEE SYSTEM

U.S. ARMY CORPS OF ENGINEERS

OMAHA DISTRICT

1616 CAPITOL AVENUE

OMAHA, NEBRASKA 68102-4901

FINAL

July 2018

LEVEE UNDERSEEPAGE AND RELIEF WELL FLOWS.

1. General. CENWO-ED-GA was tasked with determining seepage flows from underseepage through the levee foundation, and from relief wells. This was performed for levee systems L575 and L536.

1.1. Underseepage Flows. The underseepage flows were determined from flows in SEEP/W models created after the 2011 flood. SEEP/W is a numerical model that was developed by GEO-SLOPE International Ltd. These models were chosen because they are recent and developed from recent soil investigations (MER, CPT, borings). The flows were obtained from within the model at the flux boundary and location at the levee centerline. This is discussed in detail in later paragraphs.

1.2. Relief Well Flows. The relief wells flows were determined using USACE Mobile District spreadsheet developed from equations in EM 1110-2-1914 Design, Construction, and Maintenance of Relief Wells. Soil parameters, stratigraphies, and geometries were obtained from the SEEP/W models. The river level elevations were varied from the top of levee to the toe of levee and relief well flows recorded.

L536 DETERMINATION.

1. Project Description. Levee System L-550/536 is located in Atchison and Holt Counties in Missouri along the left (east) bank on the Missouri River. The total length of the system is 15.8 miles. The system extends from the Rock Creek tieback south of Rock Port, Missouri down the Missouri River and Mill Creek tieback to high ground west of Corning, Missouri. The county line is located near the southern end of the mainline levee being Holt 100/Atchison Holt Road located about one mile north of Corning, Missouri. The Rock Creek tieback segment is 5.5 miles long and is officially part of the L-550 levee even though it is part of the L-536 system. The length of the L-536 segments of the system is 10.3 miles as described below: L-536 begins at Station 503+45 where it continues stationing from L-550 at the Rock Creek tieback, approximately six miles south of Rock Port, Missouri. L-536 runs downstream along the Missouri River for a distance of 8.1 miles (Station 801+00) where it joins the right bank of the Mill Creek tieback. The levee continues up the Mill Creek right bank for a distance of 2.2 miles where the system ties into high ground. This upstream tieback is located just west of Corning, Missouri. See Figure 1 for a partial map of L536, mainly the area of seepage during 2011 flooding.

2. Underseepage Berms. Landside and riverside underseepage berms were constructed along the entire reach of the Missouri River levee. It was determined that no berms were needed along the tiebacks. The landside berms generally extend 100 feet from the centerline of the levee with the exception of the berm between Station 503+45-535+50 which extended 20'. The thickness of the berms, measured at the berm shoulders, vary from 2.5 to 4.5 feet. Additional underseepage control methods (relief wells) were required at various reaches of the levee where conditions of relatively large, thin blanket on the landward side, or close proximity of the levee to the river bank, or a combination of these, render inadequate the use of berms alone. Riverside berms extend 25' from the centerline of the levee along the below intervals. All berms were sloped at 1:50. The table below describes locations of landside and riverside berms.

3. Relief Wells. Fifty-seven pressure relief wells were installed along the landside toe of the Missouri River reach of the levee system. Relief wells were constructed with 8-inch inside diameter slotted wood stave screens. An 8-inch inside diameter plastic riser pipe extends from the screens to slightly above the surrounding natural ground. A relief valve assembly (check valve) was attached to the top of each riser pipe to prevent ponded water from entering the well. At the surface, the wells were finished in a concrete well pit with a landside opening at the ground line and a trash guard in the opening. Some wells had shallow excavations adjacent to the overflow opening to facilitate flow into ponding areas.

Phase 1 repairs were considered time critical and needed to be completed prior to the 2012 flood season. The Phase 1 construction contract was awarded on February 21, 2012. The Phase 1 design was based upon engineering judgment without the aid of additional field surveys or geotechnical data. Concurrent with the construction of the Phase 1 repairs, extensive geotechnical investigations were conducted. The investigations included profiling the levee embankment ant foundation stratigraphy with geophysics, specifically multiple point electric resistivity. Cone penetrometers were then pushed to target areas of concern and to correlate the MER data. Borings were then drilled to target areas of concern and to correlate the CPT data. All components of the geotechnical investigations were then surveyed to locate the horizontal location and vertical elevation of the data points. The purpose of the study was to assess the need for, and the design of, additional seepage control measures in areas that sustained damages during the flood event. The purpose of the underseepage report was to utilize the completed geotechnical investigation data to evaluate the adequacy of the Phase 1 repairs and to design the Phase 2 repairs.

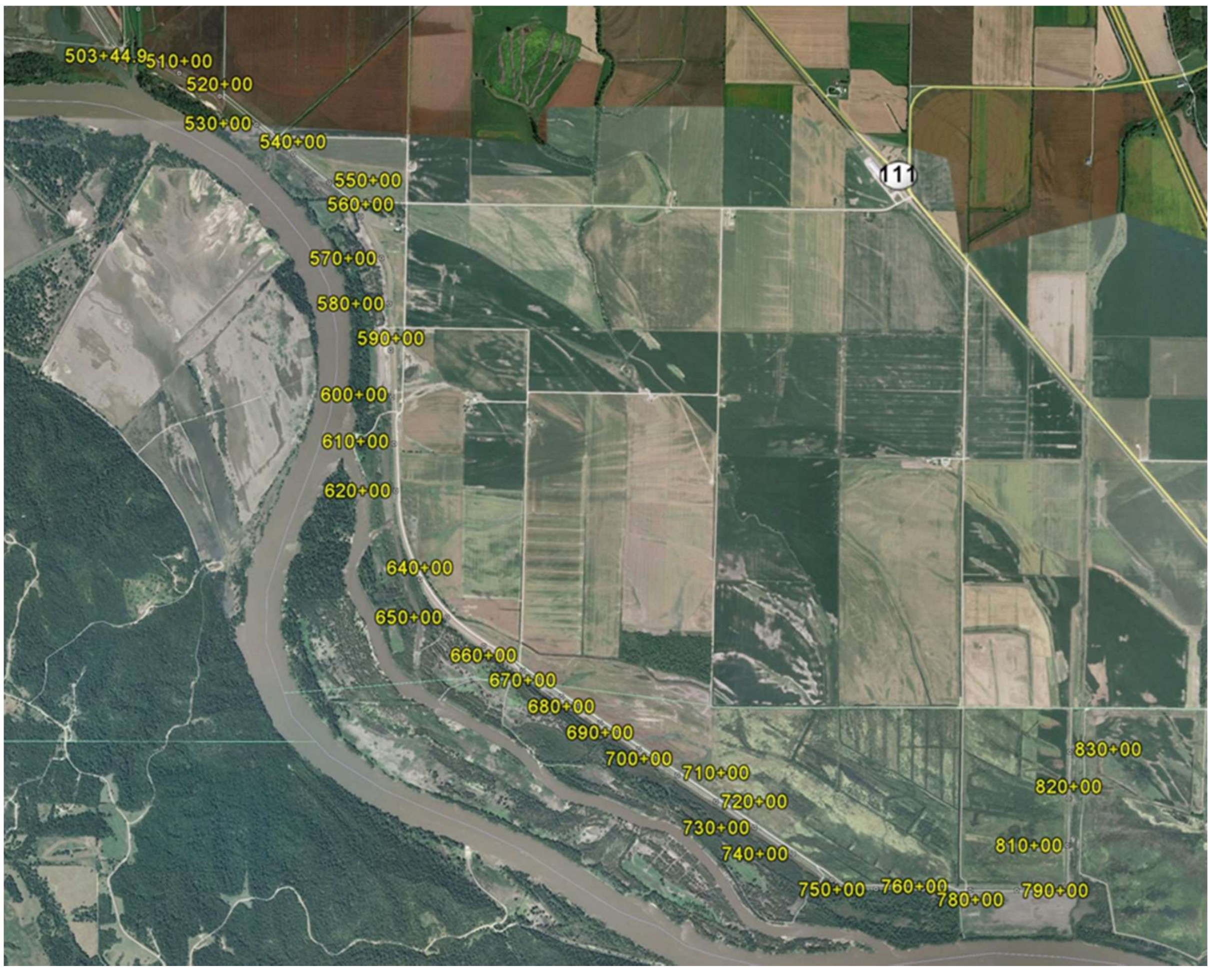


Figure 1. L536 South Portion Along Missouri River

4. Flow Determination. The underseepage and relief flow determination, and the values to use are discussed in the following paragraphs. The L536 boundary was divided into four areas by ED-H and designated as 536A, 536B, 536C and 536D. Areas 536B, 536C and 536D bound the levee along the Missouri River. Areas 536A, 536B and 536C bound the Rock Creek tieback levee. Area 536D bound the Mill Creek tieback levee.

4.1. Underseepage Flow Determination. The underseepage flows, as stated earlier, were determined from existing recent post-2011 Flood SEEP/W modeling. L536 models were performed at Stations 568+00, 677+25, 728+25, and 800+00. The areas were designated in the modeling as, and to cover Area 1 between Stations 790+00 and 810+00, Area 2A between Stations 730+00 and 747+00, Area 2B between Stations 647+00 and 690+00, and Area 2C between Stations 555+00 and 570+00.

Landside seepage berms are located from Stations 555+00 to 570+00, 601+45, 640+00 to 642+00, 647+00 to 690+00, and 710+00 to 809+00. The four seepage models all yielded a similar flow rate and it was decided to use one value. A seepage rate with the river at the levee crest for the tiebacks and berm-only areas was obtained from the SEEP/W model as 0.24 gallons per minute/linear foot of levee for the river level at the crest. The seepage rate with river levels at the toe would be 0 and any seepage rates for other river levels should be determined by interpolating between the crest and toe values. This rate should be used for berm-only areas station 555+00 to 565+50, 710+20 to 776+45, and 777+40 to 809+00; and the tieback levees (stationing adjusted for areas containing both wells and berms). Table 1 and the succeeding three paragraphs should be used for flows for the entire system.

Table 1 – L536 Seepage Flow Rates

Stationing Range		Reach Length	Q_{BERM} gpm/lf	Q_{WELL}
535+10	555+00	[2]1990	0.24	0
555+00	565+50	[2]1050	0.24	0
565+50	710+20	[1]14470	0	See Table 2
710+20	776+45	[2]6625	0.24	0
776+45	777+40	[1]95	0	See Table 2
777+40	809+00	[2]3160	0.24	0

[1]Use curve provided, should use desired stage and pick rate from curve and multiply by number of wells. The duration of the event should be considered by ramping up to a peak, using the peak length, and ramping down from peak. There are a total of 68 wells in these two station groups.

[2]A linear curve of 0.24 gpm/linear foot of levee at the crest to 0 at the levee toe should be used for these station reaches. The rate should be picked using desired stage and multiplied by the total linear feet of levee. The duration of the event should be considered by ramping up to a peak, using the peak length, and ramping down from peak.

The other three drainage boundaries would use just the berm seepage rates and methodology as in note [2]. Two of the drainage boundaries border the Missouri River and should use the berm seepage rates.

4.2. Relief Well Flow Determination. The relief well flows, as stated earlier, were determined using a USACE Mobile District spreadsheet. There are a total of 68 wells in the reach along the Missouri River, the south drainage Area 536D. The next two northern drainage area boundaries (536B & 536C) that are along the Missouri River are from station 503+45 to 520+80, and 520+80 to 535+10; these do not have wells. One well flow rate analysis was used for L536, see Table 2 and Figure 2 for the values. It should be noted that the flows are linear for the range of elevations presented.

Table 2 Relief Well Flow Rates	
Elev.	**GPM**
887*	93.3
885	76.6
883	61.3
880	36.8
878	20.4
876	4.1
875.5	0
873**	0

*Crest of Levee
**Base of Levee

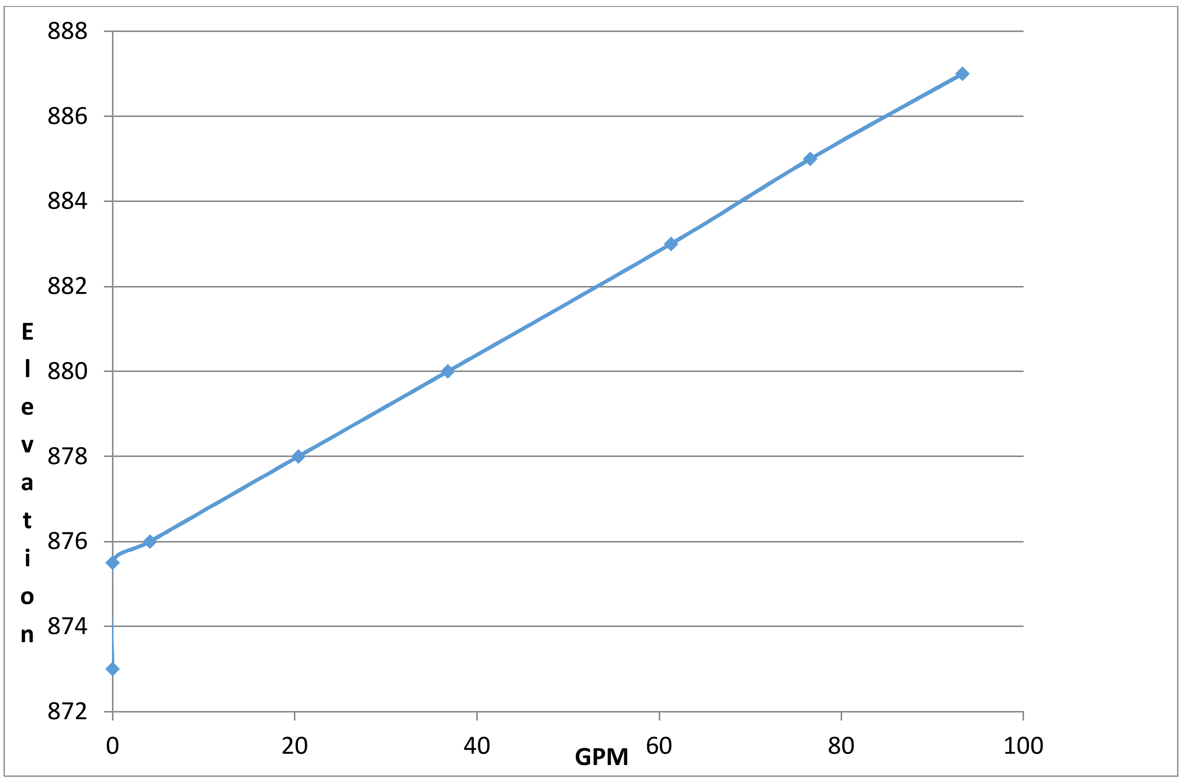


Figure 2. Chart of Relief Well Flows

<u>L575 DETERMINATIONS.</u>

1. Project Description. The L-575 levee system consists of five segments totaling approximately 41.8 miles in length which protects nearly 70,000 acres of land within the flood plain of the Missouri River. The system protects small communities and agricultural land against high stages of the Missouri River, Plum Creek, and the Nishnabotna River. The construction of the levees began May 1947 and was completed in August 1949, while the installation of the underseepage control facilities continued until November 1950. All five levee segments were originally designed to provide flood risk protection for a two percent annual chance (50 year return period) flood with two feet of freeboard.

The Plum Creek and Missouri River (Benton-Washington) levee segment stretches approximately 24.5 miles along the left descending bank of Plum Creek and the Missouri River. The levee embankment begins on the left bank of Plum Creek at Station 222+90 near the town of Thurman, Iowa, extends west to Station 445+00, near Missouri River mile 573.5, and continues south along the left bank of the Missouri River to Station 1490+00.

The Missouri River and Nishnabotna River (NW Atchison) levee segment includes four separate sections of the Missouri River and the Nishnabotna River levees. The Northwest Atchison County Levee District includes segments along the Missouri River between approximate Stations 1490+00 to 40+43 (on the Middle Breach Setback levee) and 1783+00 to 1834+50. The district also controls segments along the Nishnabotna River between approximate Stations 1960+00 to 2069+67, 2126+13 to 2213+80, and 260+20 to 176+75 (Note that levee stationing changes at 2213+80 to 260+20, at which point stationing decreases upstream toward Hamburg). Overall, the district controls 7.4 miles of levee.

The Missouri River (Buchanan) levee segment begins at approximate Station 40+43 on the Middle Breach Setback and extends south approximately 4.4 miles to Station 1783+00.

The Nishnabotna River (McKissock Island) levee segment includes two separate sections of levee totaling approximately 3.1 miles. The district controls levee along the right bank of the Nishnabotna River from approximate Station 1834+50 to 1960+00 and from Station 2069+67 to Station 2126+13.

The Nishnabotna River (Hamburg) levee segment begins at Station 177+00 and extends north for approximately 2.3 miles to tie-in to high ground at Station 49+96 in Hamburg, Iowa.

See Figure 3 for a map of L575.

2. Underseepage Berms. Landside seepage berms were constructed as part of the original levee design along portions of the Missouri River in the Benton-Washington, NW Atchison, Buchanan, and McKissock Island districts. Flooding in 2011 caused breaches on portions of the levee system, and the levee embankments and seepage berms in these areas were abandoned. Repairs to the damage occurred in 2012 and 2013 and included new setback levees constructed in breach locations and new underseepage control systems, including landside seepage berms, relief wells, and toe drain systems. Landside and riverside underseepage berms were constructed along the entire reach of the Missouri River levee. It was determined that no berms were needed along the tiebacks. Additional underseepage control methods (relief wells) were required at various reaches of the levee where conditions of relatively large, thin blanket on the landward side, or close proximity of the levee to the river bank, or a combination of these, render inadequate the use of berms alone.

3. Relief Wells. There were originally 230 relief wells installed on the L-575 levee system. Several relief wells located in areas that were breached and damaged in the 2011 flood event were abandoned or unable to be located and new relief wells were installed.

On the Benton-Washington levee segment, there were originally 128 relief wells. During the 2012-2013 repairs, 84 of these wells were abandoned, 19 wells were replaced, and 39 new wells were installed. Currently there are 83 relief wells on this segment. There were 17 new relief wells installed on the NW Atchison segment on the new setback levee and seepage berm constructed during the 2012-2013 repairs. On the Buchanan segment, there were originally 42 relief wells. During the 2012-2013 repairs, 13 of these wells were abandoned, 11 wells were replaced, and 15 new wells were installed. Currently there are 44 relief wells on this segment. On the McKissock Island levee segment, there were originally 60 relief wells. During the 2012-2013 repairs, 23 of these wells were abandoned or not found, 22 wells were replaced, and 14 new wells were installed. Currently there are 51 relief wells on this segment. There are no relief wells located on the Hamburg segment.

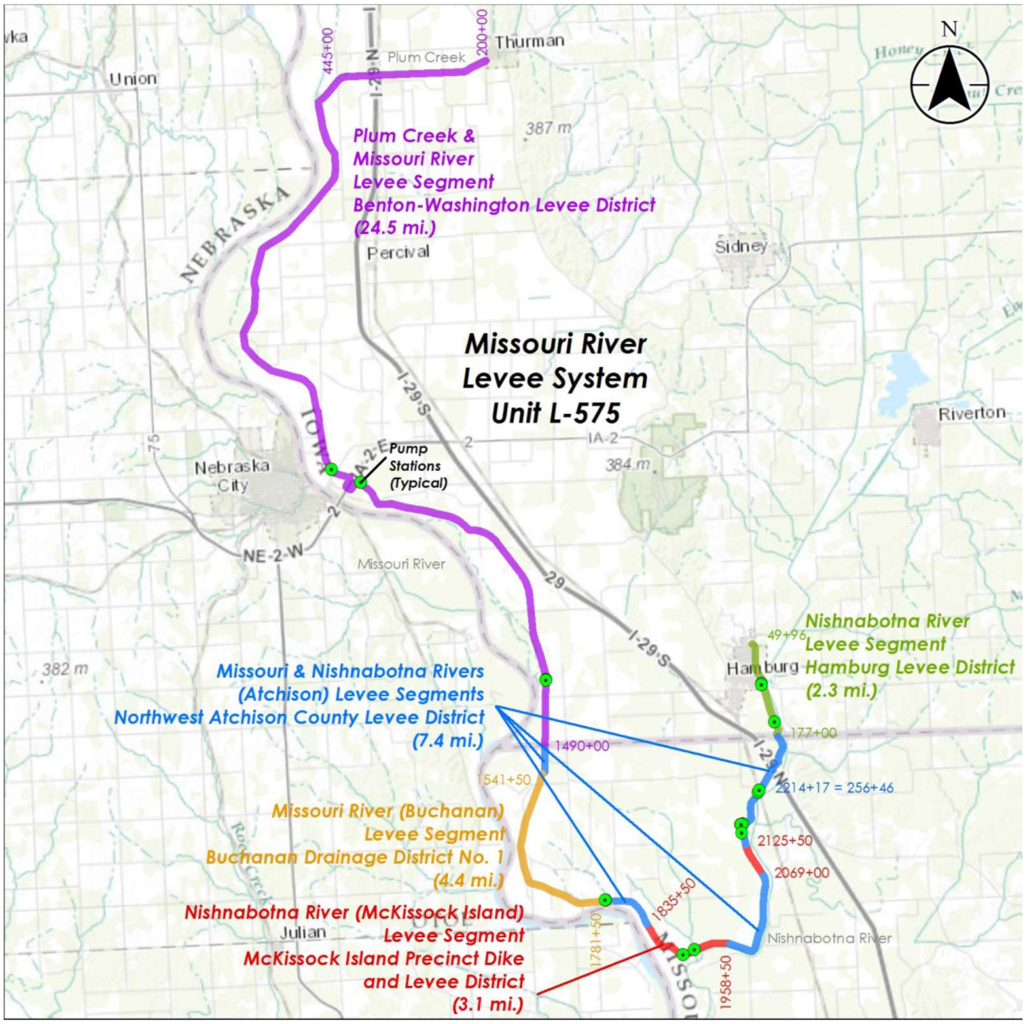


Figure 3. L536 South Portion Along Missouri River

4. Flow Determination. The underseepage and relief flow determination, and the values to use are discussed in the following paragraphs. The L575 boundary was divided into eight areas by ED-H and designated as 575A, 575B, 575C, 575D, 575E, 575F, 575G and 575H. Areas 575B, 575C, 575D, 575E and 575F bound the levee along the Missouri River. Areas 575A and 575B bound the Plum Creek tieback levee. Areas 575F and 575H bound the Nishnabotna River tieback levee.

4.1. Underseepage Flow Determination. The underseepage flows, as stated earlier, were determined from existing recent post-2011 Flood SEEP/W modeling. L575 models were performed at the 1185, Upper Breach, Middle Breach and Lower Breach Setbacks. 1185 was modeled at four locations, the Upper Breach at three locations, the Middle Breach at three locations, and the Lower Breach at two locations. The flows through the aquifer at the centerline of levee with the river level at the crest for each location is shown in Table 3.

Table 3 – Underseepage Flows per SEEP/W			
	Stations	**cfs/lf**	**gpm/lf**
1185 Setback			
	405+00	0.00007537	0.034
	435+00	0.00007372	0.033
	470+00	0.00004378	0.020
	496+00	0.00005070	0.023
Upper Breach			
	34+00	0.00717000	3.218
	34+00	0.00680200	3.053
	34+00	0.00676700	3.037
	40+00 to 65+00	0.00242700	1.089
	40+00 to 65+00	0.00260700	1.170
	40+00 to 65+00	0.00298700	1.341
	52+00	0.00169500	0.761
	52+00	0.00141400	0.635
	52+00	0.00169500	0.761
Middle Breach			
	19+00	0.00800300	3.592
	19+00	0.00800900	3.595
	35+00	0.00441100	1.980
	35+00	0.00442500	1.986
	37+00 to 63+00	0.00331600	1.488
Lower Breach			
	10+00	0.00637000	2.859
	10+00	0.00821500	3.687
	22+50	0.00423700	1.902
	22+50	0.00583200	2.618

4.2. Relief Well Flow Determination. The relief well flows, as stated earlier, were determined using a USACE Mobile District spreadsheet. There are a total of 191 wells in the L575 system. Relief wells are located between Stations 55+200 and 600+00, 734+00 and 743+80, 984+00 and 1044+00, 1111+00 and 1162+40, 1384+00 and 1404+50, 1424+75 and 1425+00, 1530+00 and 1592+00, 1596+45 and 1646+00, 1687+90 and 1703+00, 1756+15 and 1758+85, 1857+00 and 1864+50, 1866+00 and 1900+00, 1903+90 and 1951+80, and 661+00. Each location was analyzed with one relief well flow spreadsheet specifics reflective of the reach. Fourteen well locations were analyzed in L575 due to scoping changes after the L536 determinations. Table 4 lists the relief well reaches and the flow determined for all wells in the reach.

Table 4 – Relief Well Flow Rates	
Station Reach	**gpm/well**
535+00 to 620+00	77.0
656+50 to 665+50	67.4
725+00 to 751+78	124.3
837+00 to 1072+00	98.6
1072+00 to 1170+00	118.4
1344+00 to 1422+00	64.1
1422+00 to 1482+00	64.1
1525+00 to 1592+00	79.1
1592+00 to 1646+00	79.1
1651+00 to 1729+90	115.7
1750+00 to 1783+00	115.7
1834+52 to 1864+90	47.7
1864+90 to 1900+00	63.7
1900+00 to 1933+00	47.0

Table 5 presents the seepage flow values to be used for both relief well reaches and non-well reaches; the values presented are maximum values. The seepage rate with river levels at the toe would be 0 and any seepage rates for other river levels should be determined by interpolating between the crest and toe values as was done for L536; this would also apply to the relief well values. This is considered reasonable and proper because the resulting flow functions are linear. In relief well reaches, the relief well value would be used in lieu of the underseepage value. For stations 837+00 to 1072+00, 1072+00 to 1170+00 and 1185+00 to 1344+00, where there are no wells, a berm value of 2.197 should be used instead of the listed non-berm value of 0.034. Final underseepage flow rates shown in Table 5 were adjusted from those presented in Table 4, the low values were usually ignored and an average of the remaining values were used and some adjusted up or down using engineering judgement. The underseepage flow values presented in Table 4 may have been high being they were obtained beneath the levee centerline, and actual flow exiting the ground surface would be reduced to loss of head. Values outside of the seepage model areas were selected based on foundation stratigraphy.

Table 5 – Underseepage and Relief Well Flows

Berm Locations-sta		Reach length	Actual length	Berm width		well start station	well end station	No. of Wells	Q_{berm}	Q_{rw}	
									gpm/LF	gpm/well	
22290	44000	21710			No Berm				0.034		
44000	44700	700		80					2.197		
44700	45400	700			No Berm				0.034		
45400	46000	600		80					2.197		575a
46000	49000	3000			No Berm				0.034		
49000	49750	750		90					2.197		
49750	53500	3750			No Berm				0.034		
53500	62000	8500	6454	150	Upper Breach Setback	55200	60000	32	2.197	77.0	
62000	63800	1800			No Berm				0.034		
63800	65450	1650		90					2.197		
65450	65650	200		90-105					2.197		
65650	66550	900		105		66100		1	2.197	67.4	
66550	68750	2200		105-90					2.197		
68750	69050	300		90					2.197		
69700	72500	2800		75-110					2.197		575b
72500	75178	2678		110		73400	74380	7	2.197	124.3	
75178	77000	1822		110					2.197		
77000	80300	3300		110-75					2.197		
80300	83700	3400		75-100					2.197		
83700	107200	23500	34645	150	Highway 2 Setback	98400	104400	4	0.034	98.6	575a
107200	117000	9800		100		111100	116240	26	0.034	118.4	
117000	118500	1500			No Berm				0.034		575d
118500	134400	15900	16764	150	1185 Setback				0.034		
134400	142200	7800		100		138400	140450	9	2.965	64.1	
142200	148200	6000		100		142475	142500	2	2.965	64.1	
148200	149005	805		150					2.965		575c
149005	152200	3195		90-70					2.965		
152200	152500	300		100					2.965		
152500	159200	6700	6262	150	Middle Breach Setback	153000	159200	32	2.965	79.1	
159200	164600	5400		95		159645	164600	16	2.965	79.1	575e
164600	165100	500		95-100					2.673		
165100	172990	7890		100		168790	170300	10	2.673	115.7	
172990	175000	2010		105-110					2.673		
175000	178300	3300		110-90		175615	175885	3		115.7	
178300	180000	1700		110-90					2.673		
180000	183452	3452		90-110					2.673		
183452	186490	3038		90-110		185700	186450	6	2.380	47.7	
186490	190000	3510	3424	150	Lower Breach Setback	186600	190000	14	2.380	63.7	575f
190000	193300	3300			No Berm	190390	195180	29	0.034	47	
193300	195800	2500		100					2.380		
195800	221354	25554			No Berm				0.034		
											575g
5000	25634	20634			No Berm				0.034		575h

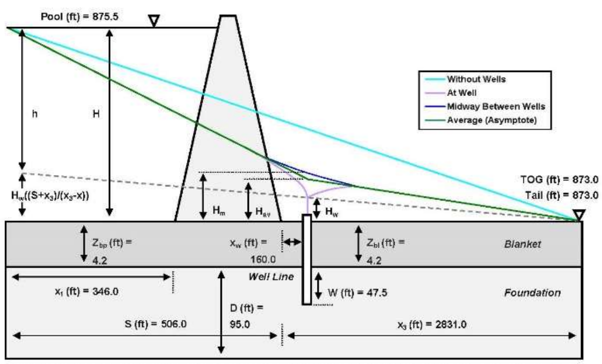
APPENDIX A

USACE Mobile District Relief Well Spreadsheet

Relief Well System Design and Analysis Program			*vs 9.8.09*
Project	L536		
Program is Applicable for Blanket Aquifer Foundation Case with an Infinite Line of Wells Parallel to an Infinite Line Source			
Parameter	*Expected Value*	*Units*	*Definition*
Load and Underseepage Geometric Parameters			
Pool	875.50	ft	Pool elevation
Tail	873.00	ft	Landside tail water elevation
H	2.50	ft	Net head on well system
TOG	873.00	ft	Landside top of ground elevation
L_1	1275.0	ft	Distance from poolside embankment toe to an open seepage entrance; enter "NA" if not applicable
	NA	ft	Distance from poolside embankment toe to a seepage block; enter "NA" if not applicable
L_2	160.0	ft	Base width of impervious embankment
L_3	NA	ft	Distance from landside embankment toe to an open seepage entrance; enter "NA" if not applicable
	NA	ft	Distance from landside embankment toe to a seepage block; enter "NA" if not applicable
x_w	160.0	ft	Line of wells distance downstream from the embankment toe; if line of wells is located at the embankment toe enter "0.0"
x_1	346.0	ft	Distance from poolside embankment toe to effective seepage entry
	346.0	ft	x_1 determined from hydraulic grade line measured by aquifer foundation piezometeric data; enter "NA" if not applicable
S	506.0	ft	Distance from effective source of seepage entry into foundation to the landside embankment toe (fig. 11, TM 3-424)
x_3	2831.0	ft	Distance from landside embankment toe to effective seepage exit
	NA	ft	x_3 determined from hydraulic grade line measured by aquifer foundation piezometric data; enter "NA" if not applicable
Blanket Strata Parameters			
Z_{bp-1}	4.2	ft	Actual thickness of poolside blanket stratum 1 *Classification: Sandy Silt*
Z_{bp-2}	0.0	ft	Actual thickness of poolside blanket stratum 2 *Classification: -*
Z_{bp}	4.2	ft	Total actual thickness of poolside blanket strata
K_{bp-1}	8.60E-03	ft/d	K_y (vertical hydraulic conductivity) of poolside blanket stratum 1
$\underline{K_{bp}}$	8.6E-03	ft/d	Effective K_y of poolside blanket strata (eq. 3, TM 3-424)
c_{bp}	0.0004		Constant for poolside blanket strata (eq. B-4, EM 1110-2-1914)
Z_{bl-1}	4.2	ft	Actual thickness of landside blanket stratum 1 *Classification: Sandy Silt*
Z_{bl}	4.2	ft	Total actual thickness of landside blanket strata
K_{bl-1}	8.60E-03	ft/d	K_y (vertical hydraulic conductivity) of landside blanket stratum 1
$\underline{K_{bl}}$	8.6E-03	ft/d	Effective K_y of landside blanket strata (eq. 3, TM 3-424)
c_{bl}	0.0004		Constant for landside blanket strata (eq. B-4, EM 1110-2-1914)
$\gamma_{sat-bl-1}$	115.0	pcf	Saturated unit weight (γ_{sat}) of landside blanket stratum 1
$\underline{\gamma_{sat-bl}}$	115.0	pcf	Effective saturated unit weight of landside blanket strata
$\underline{\gamma_{b-bl}}$	52.6	pcf	Effective buoyant unit weight (γ_b) of landside blanket strata
Aquifer Foundation Strata Parameters			
D_1	95.0	ft	Actual thickness of foundation stratum 1 *Classification: Sand*
D_2	0.0	ft	Actual thickness of foundation stratum 2 *Classification: -*
D_3	0.0	ft	Actual thickness of foundation stratum 3 *Classification: -*
D_4	0.0	ft	Actual thickness of foundation stratum 4 *Classification: -*
D_5	0.0	ft	Actual thickness of foundation stratum 5 *Classification: -*
D_6	0.0	ft	Actual thickness of foundation stratum 6 *Classification: -*
D_7	0.0	ft	Actual thickness of foundation stratum 7 *Classification: -*
D_8	0.0	ft	Actual thickness of foundation stratum 8 *Classification: -*
D_9	0.0	ft	Actual thickness of foundation stratum 9 *Classification: -*
D	95.0	ft	Total actual thickness of foundation strata
$\underline{D}$	95.0	ft	Thickness of transformed (homogeneous, isotropic) foundation (eq. 4-19, EM 1110-2-1914)
K_{x-1}	1.73E+02	ft/d	K_x (horizontal hydraulic conductivity) of foundation stratum 1
K_{y-1}	1.73E+02	ft/d	K_y (vertical hydraulic conductivity) of foundation stratum 1
K_{x-2}	0.00E+00	ft/d	K_x of foundation stratum 2
K_{y-2}	0.00E+00	ft/d	K_y of foundation stratum 2
K_{x-3}	0.00E+00	ft/d	K_x of foundation stratum 3
K_{y-3}	0.00E+00	ft/d	K_y of foundation stratum 3
K_{x-4}	0.00E+00	ft/d	K_x of foundation stratum 4
K_{y-4}	0.00E+00	ft/d	K_y of foundation stratum 4
K_{x-5}	0.00E+00	ft/d	K_x of foundation stratum 5
K_{y-5}	0.00E+00	ft/d	K_y of foundation stratum 5
K_{x-6}	0.00E+00	ft/d	K_x of foundation stratum 6
K_{y-6}	0.00E+00	ft/d	K_y of foundation stratum 6
K_{x-7}	0.00E+00	ft/d	K_x of foundation stratum 7
K_{y-7}	0.00E+00	ft/d	K_y of foundation stratum 7
K_{x-8}	0.00E+00	ft/d	K_x of foundation stratum 8
K_{y-8}	0.00E+00	ft/d	K_y of foundation stratum 8
K_{x-9}	0.00E+00	ft/d	K_x of foundation stratum 9
K_{y-9}	0.00E+00	ft/d	K_y of foundation stratum 9
$\underline{K_x}$	1.73E+02	ft/d	Effective horizontal hydraulic conductivity of foundation
$\underline{K_y}$	1.73E+02	ft/d	Effective vertical hydraulic conductivity of foundation
$\underline{K_x}/\underline{K_y}$	1.0		Ratio of effective horizontal and vertical hydraulic conductivity of foundation
$\underline{K_f}$	1.73E+02	ft/d	Effective hydraulic conductivity of transformed (homogenous, isotropic) foundation (eq. 4-20, EM 1110-2-1914)
Relief Well System Design and Analysis Program			*vs 9.8.09*

Well Parameters			
d_w	0.667	ft	Diameter of screen for relief wells
T_{fp}	0.500	ft	Thickness of filter pack (between outside of well screen and natural formation) for relief wells
r_w	0.833	ft	Effective well radius (well screen radius plus filter pack thickness)
W/D	50.0%		Effective well screen penetration into transformed foundation
If continuous well screen is not used, manually adjust below values for W_1, W_2, and ...W_n until the calculated W/D ' is equal to the above user specified W/D			
W/D '	50.0%		Effective well screen penetration into transformed foundation (eq. 4-21, EM 1110-2-1914)
W_1	47.5	ft	Actual screen length of well screen in foundation stratum 1
W_2	0.0	ft	Actual screen length of well screen in foundation stratum 2
W_3	0.0	ft	Actual screen length of well screen in foundation stratum 3
W_4	0.0	ft	Actual screen length of well screen in foundation stratum 4
W_5	0.0	ft	Actual screen length of well screen in foundation stratum 5
W_6	0.0	ft	Actual screen length of well screen in foundation stratum 6
W_7	0.0	ft	Actual screen length of well screen in foundation stratum 7
W_8	0.0	ft	Actual screen length of well screen in foundation stratum 8
W_9	0.0	ft	Actual screen length of well screen in foundation stratum 9
W	47.5	ft	Total actual screen length of well in foundation strata
W/D	50.0%		Actual penetration into foundation strata of well screen
$RW_{TOR/TOW}$	875.00	ft	Relief well riser elevation (or elevation of backed-up well discharge water in well housing if > than the riser elevation)
	No		Will discharge ($RW_{TOR/TOW}$) elevation remain constant for all tail water elevations below TOG via pumping of discharge line?
H_{el}	2.00	ft	Elevation head loss
E_w	80.0%		Well efficiency (theoretical/actual drawdown) for projected Q_w *0.0 gpm = projected Q_w*
Enter E_w measured from pump-testing existing system, or E_w to be maintained if designing new system; alternatively, E_w can be adjusted to yield estimated H_h value			
H_h	0.00	ft	Equivalent well (e.g. entrance, friction, and velocity) losses (excluding potential partial penetration losses) for specified E_w

Input Parameters Conceptual Diagram



Drawing not to scale; head curves shown for illustrative purposes; calculated values based on all inputs are given below; under certain conditions H_{av} may exceed H_m

Deterministic Design Requirements			
$h_{excess-base (ft)}$	2.12	0.50	Base condition excess head (at embankment toe) above TOG (or tail water elevation if > than TOG)
FS_{base}	1.67		Base condition (effective stress) factor of safety against uplift (heave) at embankment toe
$FS_{required-HHD}$	1.57		Required (effective stress) factor of safety against uplift (heave) at well line (based on 6/11/09 HHD criteria for 25 and 30 pools)
$FS_{required}$	1.00		User-specified design (effective stress) factor of safety against uplift (heave) at well line
$h_{excess-allowed}$	3.54	ft	Allowable excess head above TOG (or tail water elevation if > than TOG) at well line
$i_{v-bl-allowed}$	0.84		Allowable vertical gradient through blanket at well line

Well Spacing Parameters			
$a_{approximate}$	771430.0	ft	Well spacing predicted from automated trials to approximately meet $FS_{required}$ (if prediction result is an error, increase W/D)
a	75.0	ft	Final design well spacing; enter value which yields adequate $FS_{uplift-a}$ and reliability if designing a new well system
a/r_w	90.0		Ratio used for determination of θav and θm parameters
D/a	1.27		Ratio used for determination of θav and θm parameters
θav	1.001		Average uplift parameter (derived from table 5-1, EM 1110-2-1914; based on theoretical well factors solutions)
$h_{(iteration\ 1)}$	0.00	ft	Net head on well system corrected for elevation head loss (iteration 1)
$h_{av\ (iteration\ 1)}$	0.00	ft	Average net head in plane of wells corrected for elevation loss (derived from fig. 5-4b, eq. 5-23, EM 1110-2-1914); (iteration 1)
$H_{w\ (iteration\ 1)}$	2.00	ft	Total well losses (elevation and efficiency); (iteration 1)
$h_{(iteration\ 2)}$	0.00	ft	Net head on well system corrected for total well losses (iteration 2)
$h_{av\ (iteration\ 2)}$	0.00	ft	Average net head in plane of wells corrected for total well losses (derived from fig. 5-4b, eq. 5-23, EM 1110-2-1914); (iteration 2)
$H_{w\ (iteration\ 2)}$	2.00	ft	Total well losses (elevation and efficiency); (iteration 2)
$h_{(iteration\ 3)}$	0.00	ft	Net head on well system corrected for total well losses (iteration 3)
$h_{av\ (iteration\ 3)}$	0.00	ft	Average net head in plane of wells corrected for total well losses (derived from fig. 5-4b, eq. 5-23, EM 1110-2-1914); (iteration 3)
$H_{w\ (iteration\ 3)}$	2.00	ft	Total well losses (elevation and efficiency); (iteration 3)
H_{av}	2.00	ft	Average net head in plane of wells (eq. 43, TM 3-424)
θm	0.982		Midwell uplift parameter (derived from table 5-1, EM 1110-2-1914; based on theoretical well factors solutions)
h_m	0.00	ft	Net head midway between wells corrected for total well losses (derived from fig. 5-4b, eq. 5-22, EM 1110-2-1914)
H_m	2.00	ft	Net head midway between wells (eq. 45, TM 3-424)
> H_{av},H_m	875.00	ft	Total head elevation equivalent of the greater of H_{av} and H_m at well line resulting from well spacing (a)

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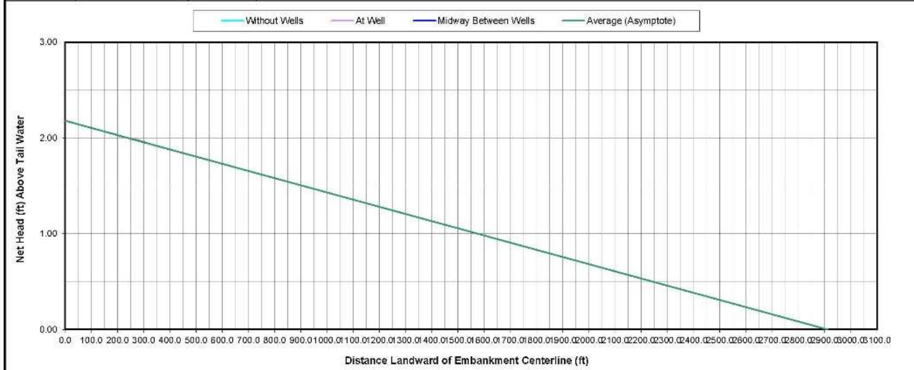
Deterministic Design Results			
$FS_{uplift-a}$	1.77		Effective stress factor of safety against uplift (heave) at well line resulting from well spacing (a)
Check to ensure $FS_{uplift-a}$ is close to, but >= to $FS_{required}$, which indicates well system design is appropriate; if not, then adjust well system design			
$h_{excess-a}$	2.00	ft	Excess head above TOG (or tail water elevation if > than TOG) at well line
i_{v-bl-a}	0.48		Vertical gradient through blanket at well line
ΔM	0.00000		Net seepage gradient toward the well line (eq. 7-6, EM 1110-2-1914; Darcy's law eq.)
Q_w	2.1	ft^3/d	Discharge from a single well (eq. 7-7, EM 1110-2-1914)
	0.0	gpm	

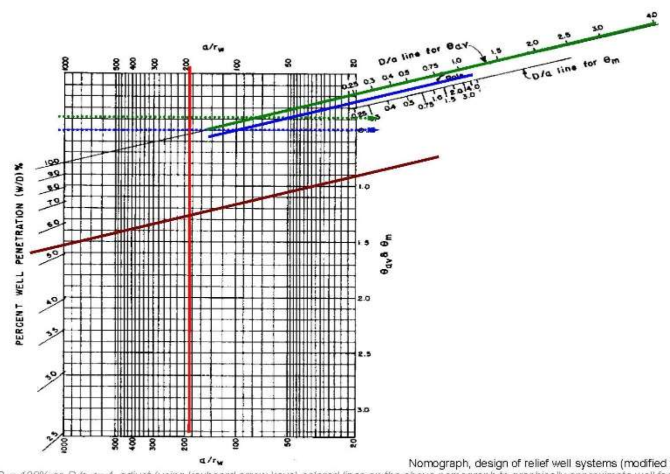


Plot assumes Midway and Average curves converge U/S & D/S from well line at a distance equal to well spacing; At Well curve U/S & D/S projections are approximate

$H_{av}, H_{m(D/S\ Toe)}$	875.12	ft	Projected total head elevation at the downstream (D/S) embankment toe resulting from well spacing (a)
$_{x-D/S}$	0.0	ft	Specified distance landward of well line to project total head elevation; enter value between 0 and (x_3 - x_w)
$H_{av}, H_{m(x-D/S)}$	875.00	ft	Projected total head elevation at specified distance ($_{x-D/S}$) downstream of well line

Check to ensure projected heads are tolerable; if not then adjust well system design or implement additional necessary seepage control measure(s)

Theoretical Uplift Parameters Check



Nomograph, design of relief well systems (modified fig. 60, TM 3-424)

If W/D = 100% or D/a <= 4, adjust (using keyboard arrow keys) colored lines on the above nomograph to graphically approximate well factors solutions

W/D	50%		Design value for effective well screen penetration into transformed foundation (maroon line on nomograph)
a/r_w	90.0		Ratio used for determination of θav and θm parameters (red line on nomograph)
D/a	1.27		Ratio used for determination of θav and θm parameters
θav	1.001		Average uplift parameter (derived from table 5-1, EM 1110-2-1914; based on theoretical well factors solutions)
$θav_{nomograph}$	1.10		Average uplift parameter (estimated graphically/manually from above nomograph - green lines; fig. 60, TM 3-424)
θm	0.982		Midwell uplift parameter (derived from table 5-1, EM 1110-2-1914; based on theoretical well factors solutions)
$θm_{nomograph}$	1.20		Midwell uplift parameter (estimated graphically/manually from above nomograph - blue lines; fig. 60, TM 3-424)

Comparison can be made (if W/D = 100% or D/a <= 4) between well factors estimated from above nomograph to those calculated from theoretical solutions

Relief Well System Design and Analysis Program vs 9.8.09

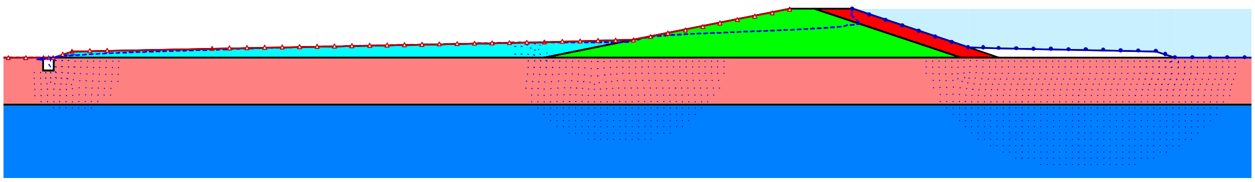
Summary of Well Installation Parameters			
W	47.5	ft	Screen length (in foundation strata) for each well
d_w	0.67	ft	Diameter of each well screen
T_f	0.50	ft	Thickness of filter pack (between outside of well screen and natural foundation) for each well
a	75.0	ft	Spacing for wells
E_w	75%		Well efficiency (theoretical/actual drawdown) for projeced Q_w *0.0 gpm = projected Q_w*

Notes

1. Relief well system analysis completed by Erich D. Guy, USACE Huntington District
2. Program applicable for blanket aquifer foundation case with an infinite line of wells parallel to an infinite line source. It is based on methods contained in:
   U.S. Army Corps of Engineers (USACE), 1992, Design, Construction, and Maintenance of Relief Wells (Engineer Manual 1110-2-1914)
   USACE, 1956, Investigation of Underseepage and its Control: Lower Mississipp River Levees (Technical Memorandum 3-424)
   Mansur and Kaufman, 1962, Dewatering, in Foundation Engineering edited by Leonards, McGraw-Hill Book Company
   Department of The Army, 1983, Dewatering and Groundwater Control (Army Technical Manual 5-818-5)
3. Program modifications to/advancements beyond existing methods include: stratified blanket and foundation consideration, well losses (elevation, efficiency) consideration, automated uplift parameters solution and appropriate well spacing prediction, variable tail water (i.e. not necessarily at prevailing ground surface elevation) consideration, variable well line position (e.g. downstream of toe) consideration, well discharge backup/discharge line pumping considerations, and automated reliability module inclusion.
4. Program assumes laterally continuous strata, and doesn't account for topographical variations along the analysis cross-section (e.g. low-lying areas downstream of the well line which may affect projected stability, nor does it account for possible well spacing adjustments necessary to account for ends of reach and/or finite reach length effects.

Relief Well System Design and Analysis Program *vs 9.8.09*

USACE MOBILE DISTRICT RELIEF WELL SPREADSHEET



SEEP/W TYPICAL SECTION

US Army Corps of Engineers ®
Omaha District

Missouri River Interior Drainage Hydrologic Analysis

FINAL

USACE Omaha District
Hydrologic Engineering Branch
July 2018

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OVERVIEW

An interior drainage analysis was performed as part of the Missouri River Recovery Program (MRRP). Two levee units within the Omaha District were analyzed, L536 and L575. The purpose of the interior drainage hydrologic analysis was to model flows at each outlet structure to be used in assessing flood damage. L575 and L536 are about 10 miles apart on the left bank of the Missouri River. Both interior drainage basins are shown in Figure 1.

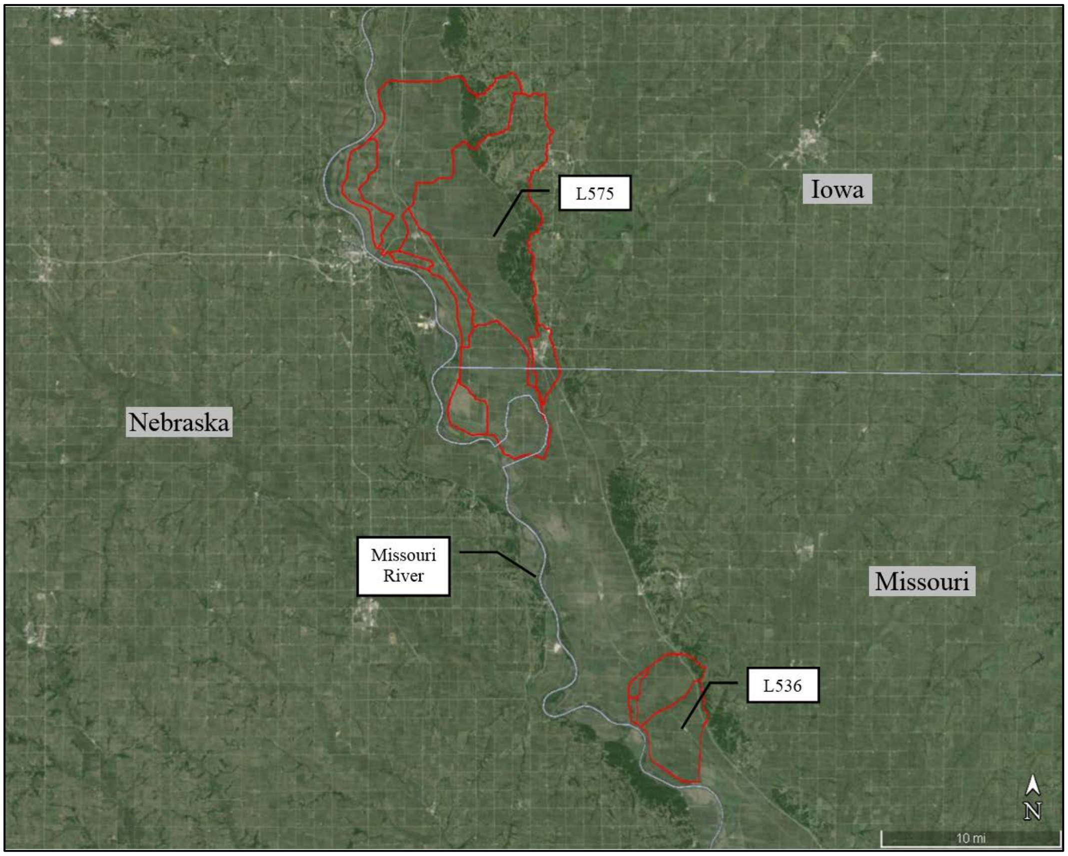

Figure 1. L536 and L575 basins modeled in the interior drainage basin hydrologic analysis.

The Missouri River Master Water Control Manual (USACE, 1998) included a volume about the Interior Drainage Study performed on both levee basins. Much of the information in that study was used to create the new interior drainage analysis with updated hydrologic modeling methods. Subbasin delineation was updated based on changes in the outlets of each drainage basin.

Calibration of both levee interior drainage basin models was limited due to availability of data. Therefore, a pilot study, within close proximity and with similar basin characteristics, was utilized in calibration. Several small basins with similar characteristics were considered for calibration, but, ultimately, Mill Creek, near Oregon, MO, was used. Mill Creek is relatively small in drainage area and has similar basin characteristics to those of both interior drainage basins. The intent of the Mill Creek pilot study was to calibrate that model with available streamgage data and apply the same parameter estimation techniques and adjustments to each interior drainage model. Details of the Mill Creek pilot study are provided in Attachment A.

LEVEE UNIT L536

SITE DESCRIPTION

Levee Unit L536 is located in Atchison and Holt Counties in northwestern Missouri. The levee unit spans along the left bank of the Missouri River. Rock Port and Langdon, Missouri are located just north of the basin, and Corning, Missouri is located south of the basin. Bluffs are located in the northeastern section of the basin that are approximately 1.2 mi^2 in area. The L536 drainage basin is approximately 22 mi^2 in area and composed of 4 subbasins. The L536 drainage basin is shown in Figure 2 below.

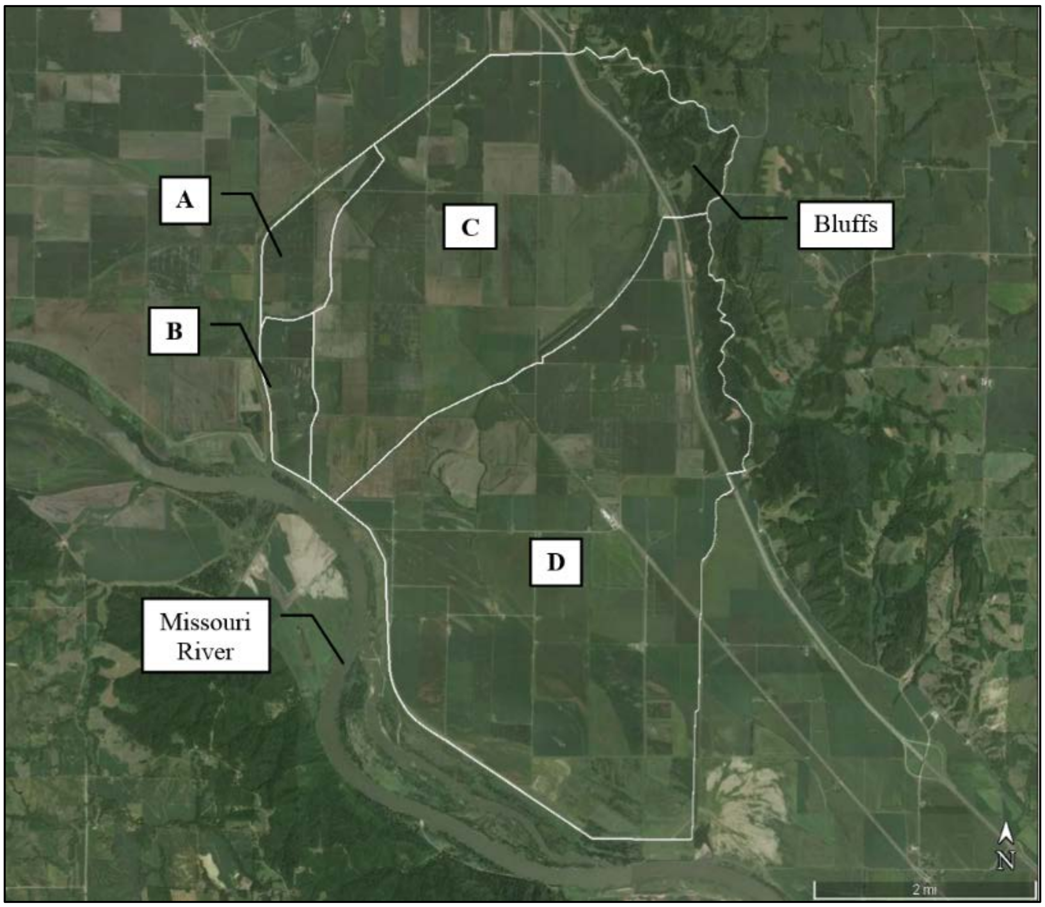

Figure 2. L536 drainage basin.

A site visit to the L536 interior drainage basin was used to obtain hydrologic information about the watershed. The primary purpose of the visit was to determine the location of streams and the amount of baseflow in the streams. Streams A and B were not accessible because of their locations. The following photos show streams in the L536 drainage basin.

Figure 3. Flow path in basin C with very little flow. (40.337402°N, 95.540044°W)

Figure 4. Main flow path in basin C looking east. (40.316878°N, 95.521348°W)

Figure 5. Stream D looking east. (40.290419°N, 95.502614°W)

Figure 6. Basin D flow path in the bluffs. (40.345658°N, 95.506015°W)

HYDROLOGIC MODEL

Hydrologic modeling was performed using HEC-HMS version 4.1 developed by the US Army Corps of Engineers Hydrologic Engineering Center (USACE, 2015). A continuous simulation was created from 1930-2012 with an hourly time step in order to obtain period of record flow at various outlets. Precipitation was combined at Nebraska City, Nebraska, and Hamburg, Iowa, to create an hourly rainfall record from 1930-2012. The hydrologic model utilized the deficit and constant loss method, Snyder's unit hydrograph runoff transformation, and recession baseflow. Parameter estimates were made based on techniques used in the Mill Creek pilot study in Attachment A and adjusted based on calibration of the Mill Creek model.

Precipitation

Hourly precipitation data was obtained from NCDC Climate Data Online (2016) at Hamburg, Iowa, from 1948-2009. In order to complete the hourly precipitation record from 1930-2012, daily precipitation at Nebraska City, Nebraska, was converted to hourly precipitation using hourly rainfall distributions computed using Hamburg hourly data. Hourly factors were computed by dividing the rainfall in one hour by the cumulative daily rainfall at the observed rain gage. This technique was used so the daily precipitation distribution would mimic that of the available hourly data. Hourly distributions were determined for the following four types of precipitation.

1. No precipitation the day before or after
2. Precipitation the day before but not the day after
3. Precipitation the day after but not the day before
4. Precipitation the day before and the day after

Each precipitation type produced a different hourly distribution to be used in the conversion of daily precipitation. The hourly distributions for each daily precipitation type are shown in Figure 7 below.

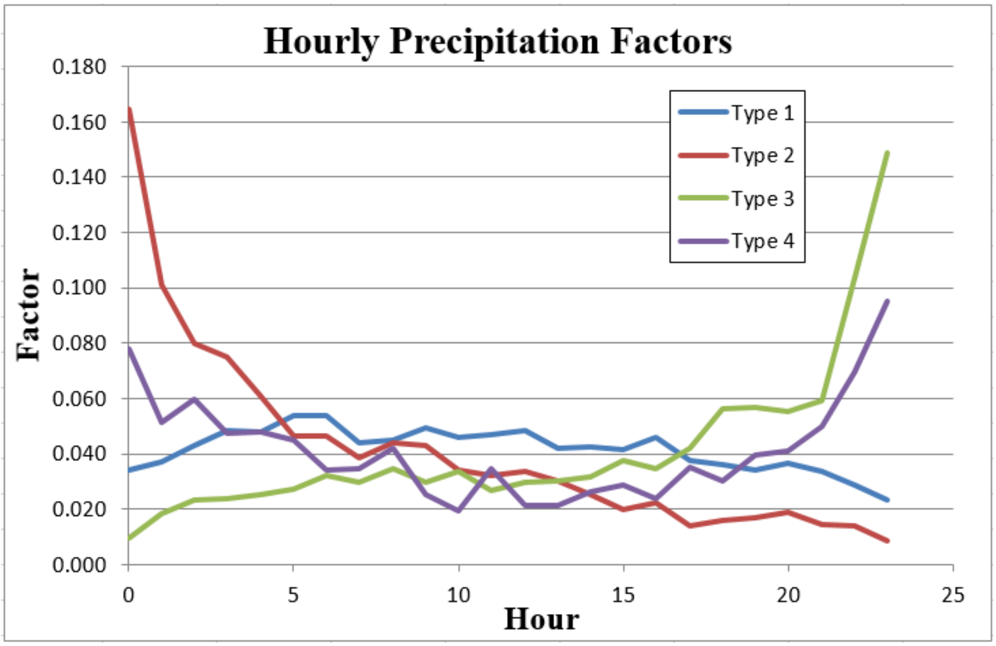

Figure 7. Hourly precipitation factor for each daily precipitation type.

Daily precipitation was then distributed over each day based of the type of precipitation and the calculated average hourly factors. Converted daily Nebraska City precipitation was used in place of any missing data in the Hamburg precipitation period of record.

Although the Hamburg and Nebraska City precipitation records are different, they were compared to check for any large discrepancies. In August 1987 the Hamburg rainfall record showed 6.6 inches of rainfall in one hour. The Nebraska City record showed no rainfall at that time. It was assumed that the 6.6 inches was recorded incorrectly and that there was no rainfall because it is a very large amount of precipitation (larger than a 1000-year frequency storm) in one hour and there was no precipitation the day before or after. In March 1987, 5.5 inches of precipitation were recorded at Hamburg. Again, there was no precipitation recorded in Nebraska City on that day, and there was no precipitation recorded the day before or after at Hamburg, so it was again assumed that there was no rainfall in the time period. There were no other large discrepancies between the two rainfall records. The final precipitation used in the levee unit L536 model is shown in Figure 8 below.

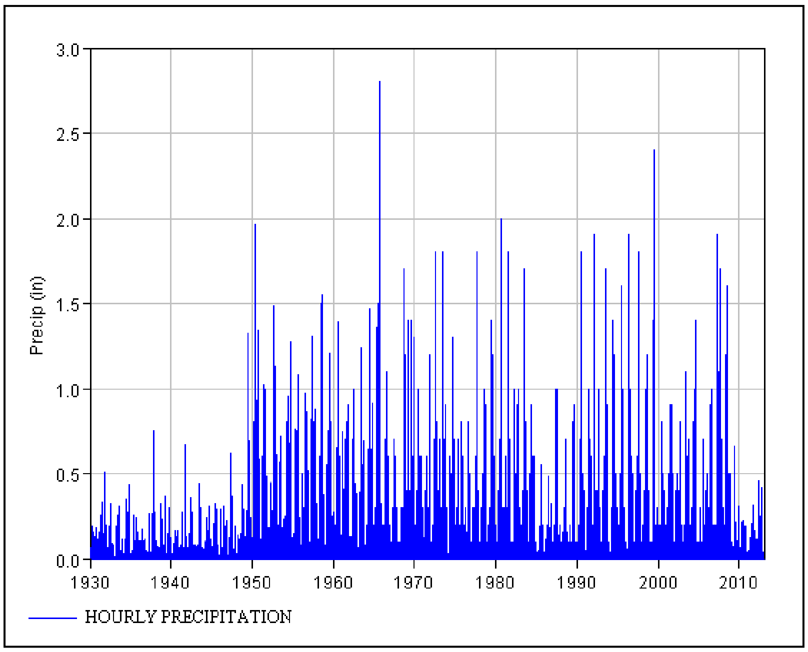

Figure 8. Hourly precipitation at Hamburg, Iowa combined with converted daily precipitation at Nebraska City, Nebraska.

Meteorologic Model

The meteorologic model utilized the described precipitation from 1930-2012.The hourly precipitation used in the hydrologic model is shown in Figure 8. The monthly average method was used to model evapotranspiration in conjunction with the meteorologic model. The monthly average method combines monthly average pan evaporation and a coefficient to convert to evapotranspiration. Monthly average pan evaporation was estimated using evaporation data from NCDC Climate Data Online (2016) at Shenandoah, Iowa, which was the closest available evaporation data from NCDC Climate Data Online (2016). Determination of the coefficient, 0.75, is described in more detail in the Mill Creek Pilot Study in Attachment A. Monthly pan evaporation is shown in Table 1.

Table 1. Monthly average evaporation at Shenandoah, IA. A factor of 0.75 was used.

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Evaporation (in)	0	0	0	5.19	5.96	7.06	7.76	6.42	5.04	3.54	0	0

Delineation

Levee unit L536 subbasins were delineated based on the drainage basins in the Master Manual Interior Drainage Study (1998). Basin delineations were updated based on changes to outlets in the levee, which resulted in the merging of two subbasins into subbasin A. Subbasin delineations are shown in Figure 2. The HEC-HMS schematic is shown in Figure 9 below. Subbasin areas are shown in Table 2.

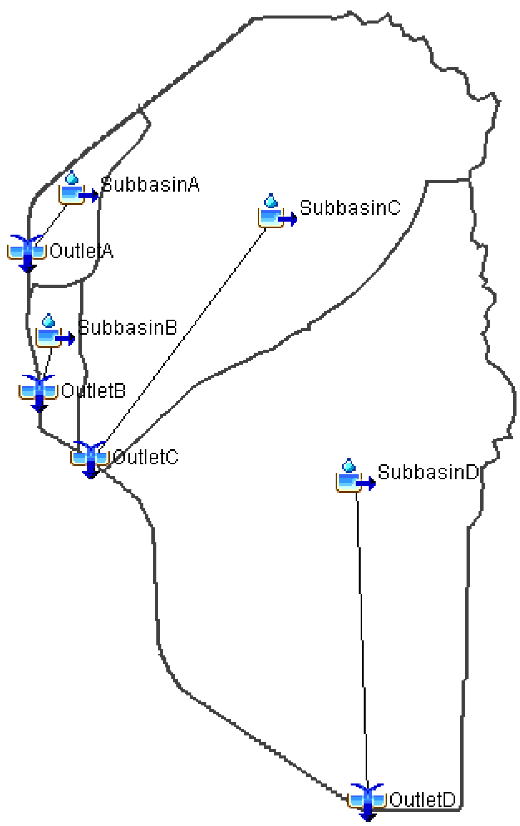

Figure 9. HEC-HMS L536 basin schematic.

Table 2. Subbasin areas computed using HEC-GeoHMS processing.

Subbasin	A	B	C	D
Area (mi^2)	0.75	0.58	8.47	12.11

Canopy

The simple canopy method was used in conjunction with monthly average evapotranspiration and rainfall loss methods. The canopy method models the interception of rainfall by plants in the landscape (USACE, 2013). It was assumed that the interception of rainfall by plants is negligible in the L536 drainage basin. The simple uptake method was required by HMS in order to properly model evapotranspiration losses in the basin.

Rainfall Losses

The deficit and constant method was used to model rainfall losses. Parameters of the deficit and constant method are initial deficit, maximum storage, constant loss rate, and percent imperviousness. The initial deficit is an estimate of the moisture required to saturate the soil at the start of the simulation. The maximum deficit represents the maximum amount of water it would take to saturate the soil. The constant loss is represented by the hydraulic conductivity of the soil. Soil properties were determined based on gridded SSURGO soils data (NRCS, 2012) and associated properties from Rawls, et al (1999). L536 is composed mostly of clay soils with silt loam soils in the bluffs. Soil parameter maps are shown in Figure 10 below.

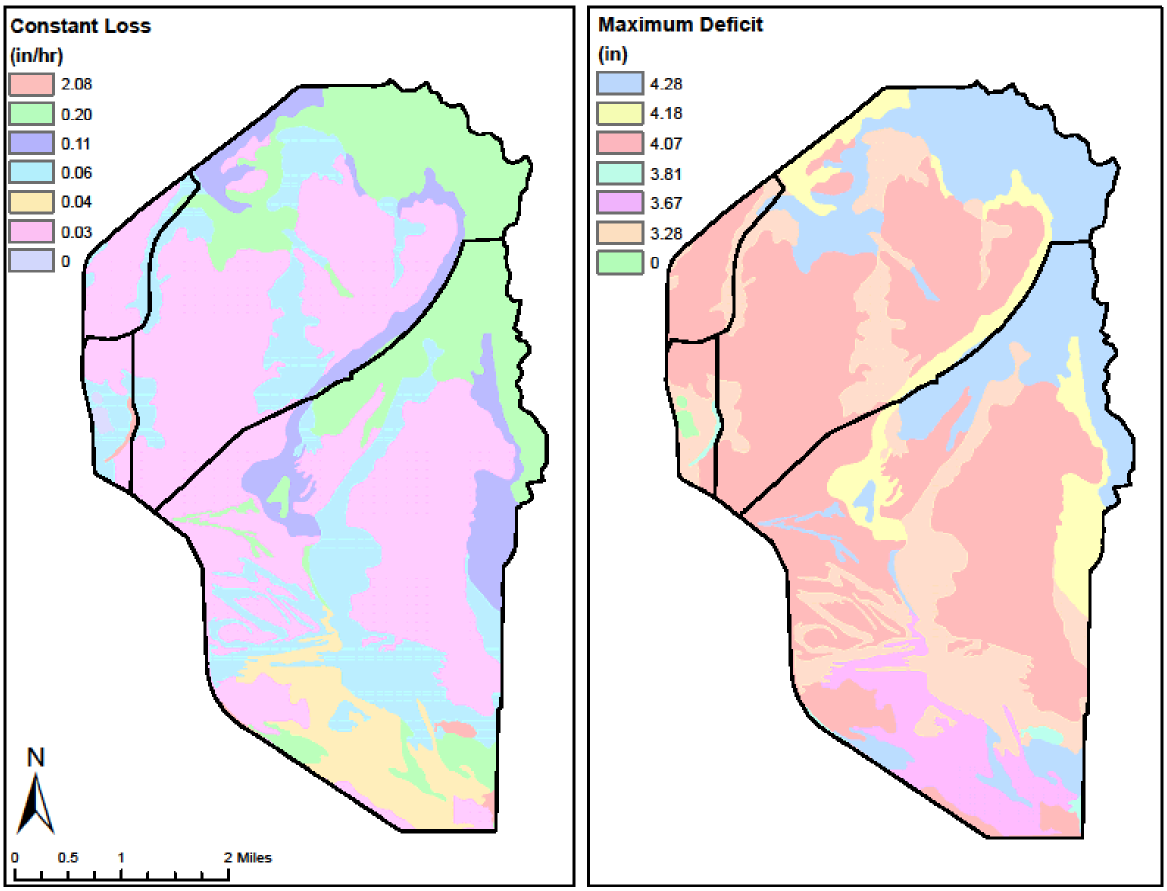

Figure 10. Deficit and constant rainfall loss parameters used to find subbasin average parameters.

Percent imperviousness was determined from the National Land Cover Database, which assigns an average percent imperviousness to each 100’ x 100’ grid cell (NLCD, 2011). The average percent imperviousness of each subbasin was calculated using HEC-GeoHMS processing. Deficit and constant loss parameters are shown in Table 3.

Table 3. Deficit and constant loss parameters.

Subbasin	Initial Deficit (in)	Maximum Storage (in)	Constant Loss (in/hr)	Impervious (%)
A	1.5	4.0	0.10	0.54
B	1.5	4.0	0.10	0.38
C	1.5	4.1	0.15	1.01
D	1.5	4.0	0.15	1.13

Runoff Transformation

Snyder's unit hydrograph method was used to model runoff transformation for L536. Snyder's utilizes lag time and a peaking coefficient (C_p). Lag time (t_l) is calculated from a lag time coefficient (C_t), the longest flowpath, and the centroidal longest flowpath. Flowpath measurements were estimated using Google Earth Pro and HEC-GeoHMS processing. C_t was estimated at 0.6 based on Veismann and Lewis (2002) and the Mill Creek pilot study. The peaking coefficient was estimated at 0.5 based on the Mill Creek pilot study. Parameters used in the Snyder's unit hydrograph runoff transform are shown in Table 4 below.

Table 4. Snyder's unit hydrograph runoff transform parameters.

Subbasin	C_p	C_t	Centroidal Longest Flowpath (mi)	Length (mi)	Lag Time (hr)
A	0.5	0.6	0.65	1.92	0.64
B	0.5	0.6	0.65	1.25	0.56
C	0.5	0.6	2.16	5.70	1.27
D	0.5	0.6	2.58	6.24	1.38

Baseflow

The recession baseflow method was used in modeling L536. The recession method utilizes an initial discharge, threshold discharge, and recession constant. The initial discharge and threshold discharge were estimated based on flow observations on the site visit. The recession constant was estimated at 0.34 for the two smaller subbasins, A and B, and 0.4 for the two larger subbasins, C and D. Recession baseflow parameters used to model L536 are shown in Table 5 below.

Table 5. Recession baseflow parameters used to model L536.

Subbasin	Initial Discharge (cfs)	Recession Constant	Threshold Flow (cfs)
A	10	0.34	15
B	5	0.34	10
C	15	0.4	30
D	15	0.4	30

CALIBRATION

Calibration of the L536 interior drainage model was completed using the Mill Creek pilot study described in Attachment A and a USGS runoff publication. The intent of the Mill Creek pilot study was to determine how calibration altered the starting parameter estimates, and to use the same parameter adjustments as calibration for the interior drainage hydrologic model. Calibration of the Mill Creek pilot study included adjustments of the monthly average evapotranspiration coefficient, baseflow recession constant, and the runoff transformation peaking and lag time coefficients. Parameters that were not adjusted in the Mill Creek pilot study were estimated using the same methods and were not adjusted, in order to parallel calibration techniques. For example, the deficit and constant loss parameters were not adjusted in calibration of the Mill Creek pilot study; therefore, those same parameters were estimated using the same technique as the Mill Creek pilot study and were not adjusted from original estimates.

In the Mill Creek pilot study, the monthly average evapotranspiration coefficient was initially estimated to be 0.6 and adjusted in calibration to 0.75. Because the same monthly average evaporation values were used in the L536 model, the same coefficient, 0.75, was used, as well.

The baseflow recession constant for Mill Creek was initially estimated to be 0.5 and reduced to 0.34 for each subbasin to decrease runoff volume per unit area, and to account for the very small area of each subbasin. The same initial estimate was used for the L536 subbasin because the two basins are very similar. The recession constant for the two smaller subbasins, A and B, was reduced to 0.34. Subbasins C and D were assigned a recession constant of 0.4 to account for the larger area which usually results in a higher recession constant.

The peaking coefficient was initially estimated at 0.8 in the Mill Creek pilot study and was adjusted to 0.6 in calibration. L536 is in the same region as Mill Creek, so the initial estimate was the same for Mill Creek and L536; however, the final calibrated peaking coefficient was 0.5 to account for much flatter slopes. The lag time coefficient for Mill Creek was initially estimated at 0.4 and adjusted to 0.6 in calibration. The same starting parameter value and final calibrated value were used for L536 because of the basin similarities.

The L536 model was also calibrated using the USGS published document showing average annual runoff throughout the United States from 1951 to 1980 in the form of a contour map (Gebert, 1987). According to the map of average annual runoff, L536 would produce approximately 7 inches of runoff, which is similar to results of the L536 hydrologic model. Overall the L536 basin averages about a 3% higher runoff volume per unit area (7.2 inches) over the USGS observation period. Subbasin runoff volumes per unit area and USGS runoff publication volume are shown in Table 6.

Table 6. Comparison of runoff volume per unit area from 1951-1980.

Source	Runoff Volume per unit area (inches)
Subbasin A	9.7
Subbasin B	10.3
Subbasin C	4.5
Subbasin D	4.2
USGS Runoff Publication	7.0

RESULTS

The L536 interior drainage basin was calibrated based on parameter adjustments of the Mill Creek pilot study. Runoff volumes per unit area were compared to USGS average runoff volumes (Gebert, 1987) as another method of calibration. Flows at each one of the outlets are shown in Figure 11, Figure 12, Figure 13, and Figure 14. Yearly runoff volume per unit area, the total runoff from each subbasin over each year in the period of record, at each outlet as well as a period of record annual average are shown in Table 7. Higher runoff volume per unit area in subbasins A and B are the results of lower constant loss rates of those basins.

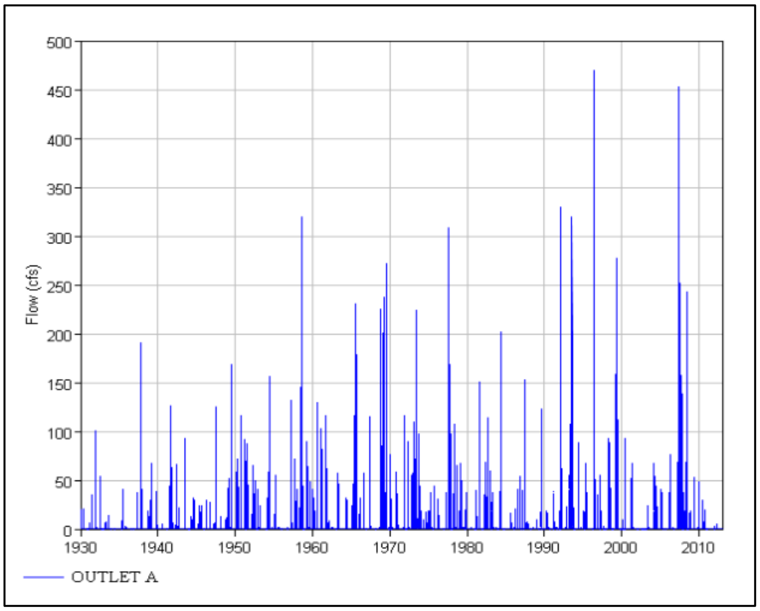

Figure 11. Outflows from L536 outlet A.

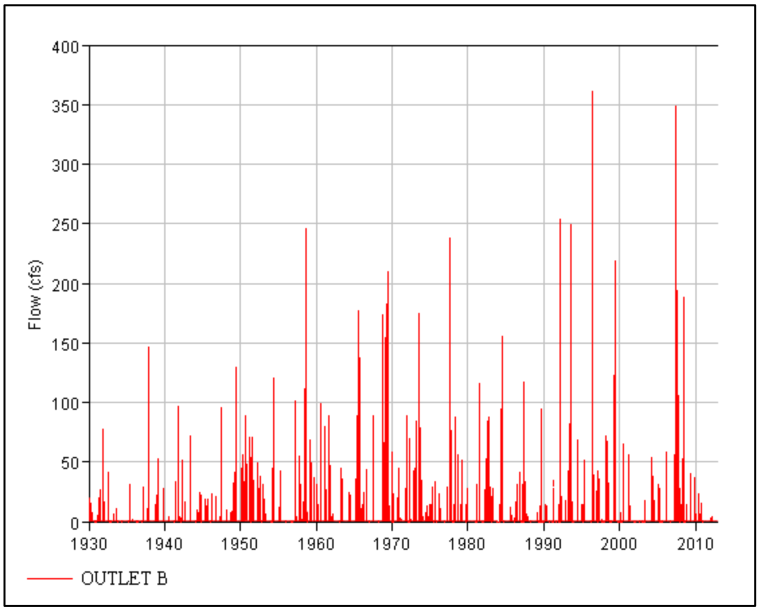

Figure 12. Outflows from L536 outlet B.

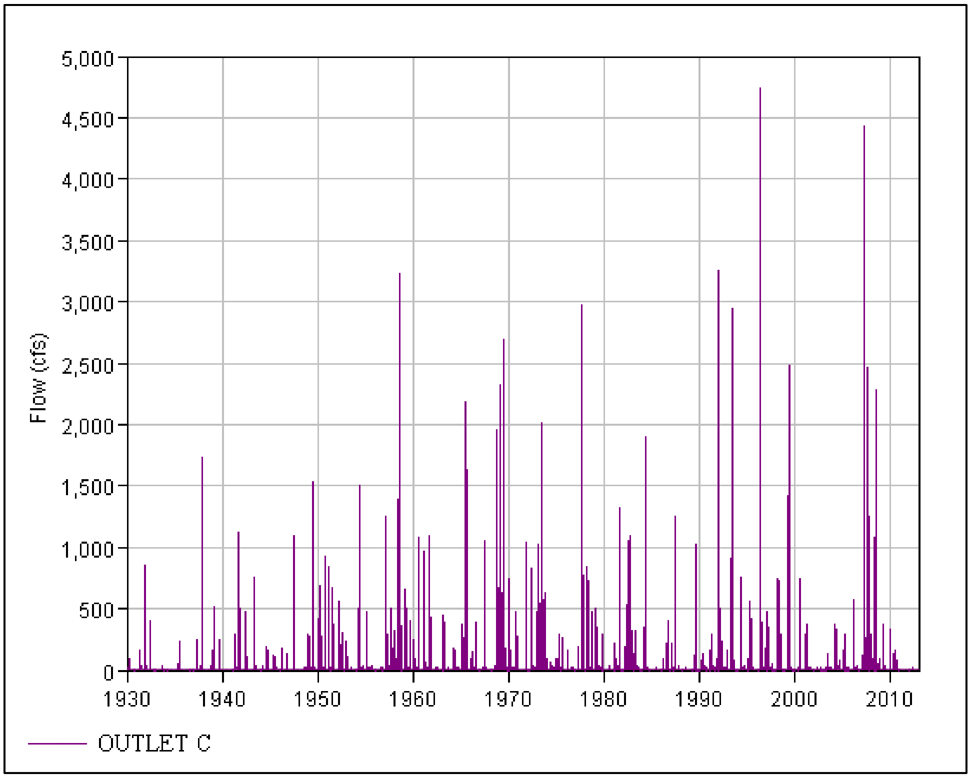

Figure 13. Outflows from L536 outlet C.

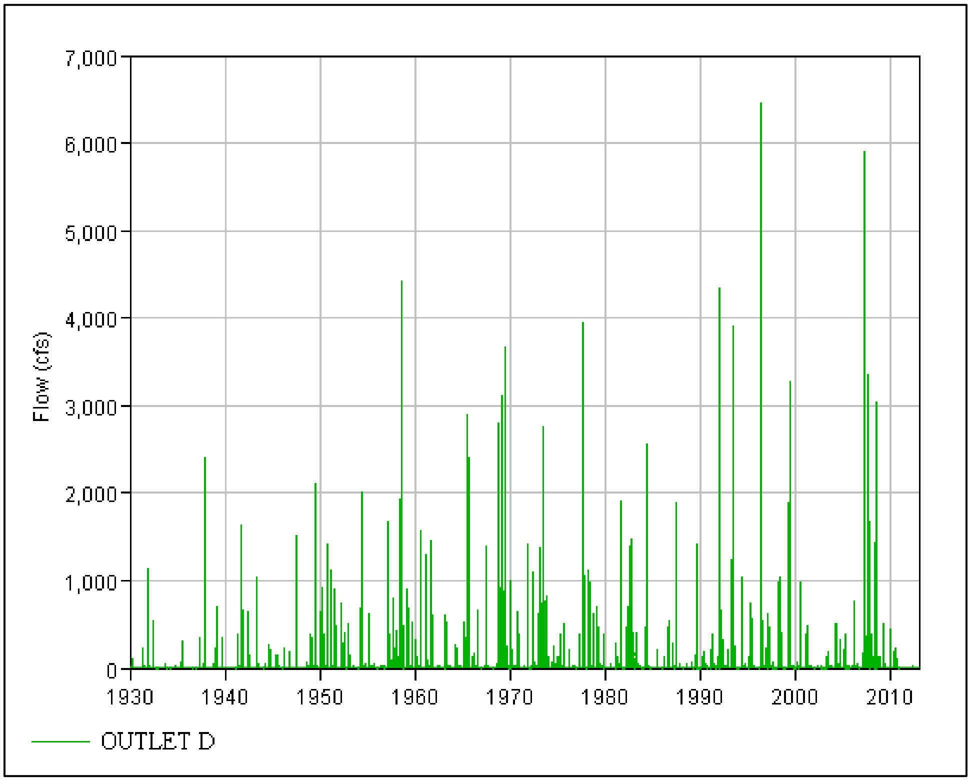

Figure 14. Outflows from L536 outlet D.

Table 7. Yearly runoff volume per unit area, in inches, at each outlet in L536. Yearly averages are also shown.

Year	A	B	C	D	Year	A	B	C	D
1930	1.5	1.8	0.9	0.9	**1972**	7.4	8.4	3.7	3.4
1931	5.0	3.8	2.6	2.3	**1973**	25.9	30.0	10.4	9.5
1932	2.2	2.6	1.1	1.0	**1974**	1.7	1.4	1.5	1.8
1933	0.3	0.2	0.6	0.6	**1975**	3.6	1.8	2.0	2.1
1934	0.2	0.1	0.5	0.5	**1976**	5.7	5.5	1.8	1.7
1935	2.4	2.7	1.2	1.2	**1977**	27.3	30.9	14.1	13.2
1936	0.3	0.3	0.4	0.5	**1978**	11.4	12.8	5.4	4.9
1937	9.8	7.6	3.9	3.6	**1979**	11.9	13.4	4.5	4.1
1938	0.6	0.5	1.0	1.0	**1980**	0.8	0.5	1.3	1.4
1939	4.1	3.0	1.4	1.3	**1981**	7.6	8.6	3.4	3.2
1940	0.3	0.2	0.5	0.6	**1982**	16.2	14.7	6.4	5.9
1941	6.3	7.6	2.2	2.1	**1983**	4.8	3.0	2.6	2.3
1942	4.5	4.8	1.7	1.6	**1984**	16.4	15.9	7.6	6.7
1943	3.6	3.9	1.5	1.5	**1985**	0.9	0.8	0.7	0.8
1944	3.9	2.2	1.8	1.7	**1986**	3.8	4.1	1.6	1.6
1945	4.7	4.5	1.7	1.5	**1987**	5.6	6.3	2.6	2.6
1946	0.8	0.7	1.5	1.4	**1988**	0.4	0.3	0.7	0.7
1947	6.2	6.7	2.6	2.5	**1989**	3.0	3.3	1.8	1.7
1948	0.4	0.3	0.6	0.7	**1990**	1.9	1.6	1.9	1.9
1949	11.6	13.1	4.9	4.5	**1991**	9.5	8.0	3.6	3.2
1950	10.8	12.0	4.3	4.3	**1992**	10.6	11.1	6.4	6.0
1951	19.2	20.7	7.5	6.6	**1993**	33.5	35.1	15.7	14.7
1952	7.6	6.7	2.9	2.7	**1994**	2.9	3.1	1.9	1.9
1953	2.7	2.5	1.4	1.4	**1995**	14.3	16.0	5.9	5.0
1954	6.4	7.1	3.4	3.3	**1996**	18.4	18.7	9.6	9.1
1955	2.6	2.9	1.7	1.6	**1997**	4.5	5.3	2.4	2.2
1956	0.8	0.6	1.4	1.5	**1998**	9.5	10.9	4.7	4.3
1957	10.3	11.5	4.4	4.2	**1999**	18.6	19.7	8.5	7.8
1958	20.9	23.5	10.6	9.9	**2000**	2.8	3.1	1.8	1.9
1959	20.0	20.4	6.7	6.0	**2001**	9.1	8.3	3.5	3.2
1960	13.7	15.1	4.8	4.4	**2002**	0.7	0.5	1.1	1.2
1961	14.6	13.1	5.6	5.3	**2003**	1.6	1.3	1.5	1.6
1962	2.0	1.7	1.6	1.7	**2004**	9.6	10.7	3.8	3.5
1963	4.8	5.5	2.3	2.2	**2005**	4.3	5.0	1.6	1.5
1964	2.8	2.5	1.8	1.9	**2006**	6.1	6.5	2.6	2.6
1965	20.5	23.3	9.1	8.5	**2007**	28.3	32.2	14.6	14.0
1966	2.8	1.9	2.0	1.8	**2008**	14.9	16.9	6.7	6.1
1967	7.4	8.4	3.8	3.6	**2009**	5.1	5.8	1.8	1.7
1968	5.6	6.2	3.2	3.2	**2010**	5.0	4.9	2.1	1.9
1969	18.6	21.4	9.4	8.8	**2011**	0.3	0.2	0.6	0.7
1970	7.2	6.2	3.7	3.2	**2012**	0.5	0.4	0.6	0.6
1971	3.6	3.8	2.6	2.6	**Yearly Average**	**7.7**	**8.1**	**3.6**	**3.4**

LEVEE UNIT L575

SITE DESCRIPTION

The L575 drainage basin is located in parts of Montgomery County in southwestern Iowa, Atchison County in northwestern Missouri, and Nemaha County in southeastern Nebraska. Hamburg, Iowa is located in the southeastern part of the drainage basin in subbasin H. Nebraska City, Nebraska, is located just west of L575 and the Missouri River. The L575 basin is approximately145 mi^2 in area and broken up into 10 subbasins ranging in size from 1.2 mi^2 to 36 mi^2. The L575 interior drainage basin and its subbasins are shown in Figure 15 below.

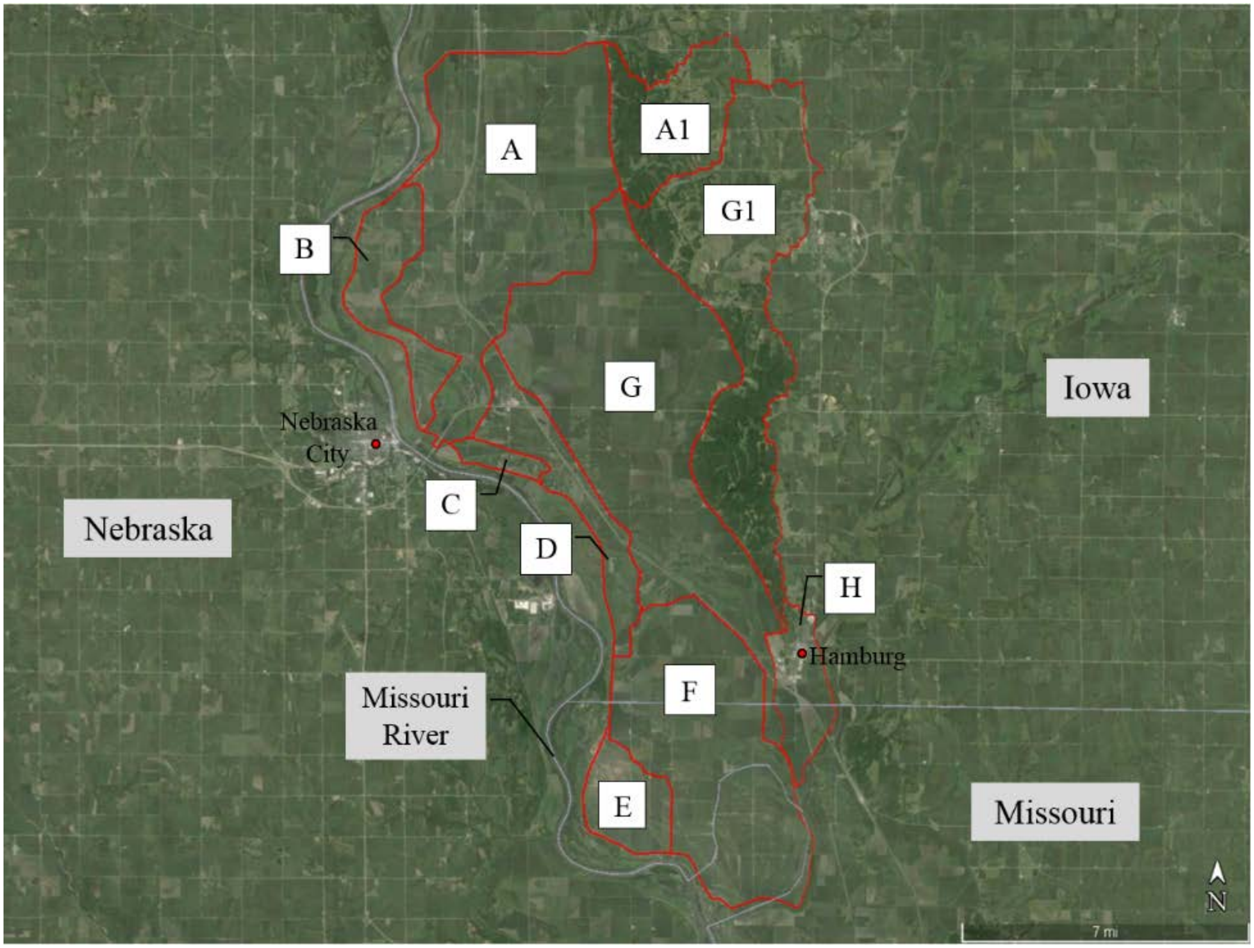

Figure 15. L575 drainage basins.

Hydrologic information about the watershed was obtained on a site visit to the interior drainage basin. The purpose of the site visit was to observe the location of the streams and the amount of baseflow, and to make general observations of the basin. Eight point cross-sections were observed in basins A and G. The following photos show observations of streams in the L575 drainage basin.

Figure 16. Basin G flow path west of Hamburg. (40.60086°N, 95.674128°W)

Figure 17. Basin A1 flow path looking north from Bluff Rd. (40.766921°N, 95.70520°W)

Figure 18. Basin A flow path looking south. (40.758872°N, 95.771166°W)

Figure 19. Basin B flow path looking south. There was no flow in this particular flow path location. (40.758336°N, 95.838822°W)

HYDROLOGIC MODEL

Hydrologic modeling of basin L575 was performed using HEC-HMS version 4.1 developed by the US Army Corps of Engineers Hydrologic Engineering Center (USACE, 2015). A continuous simulation was created from 1930-2012 with an hourly time step. Precipitation used for basin model L536 was also used for L575 due to the proximity of the basins to each other and the precipitation gages. Parameters values were estimated based on estimation and calibration of the Mill Creek pilot study parameters.

Precipitation

The same precipitation used to model L536 was also used to model L575. It is the best available data for the basin location. Hourly precipitation at Hamburg, IA and converted hourly precipitation at Nebraska City, NE were combined and are shown in Figure 20.

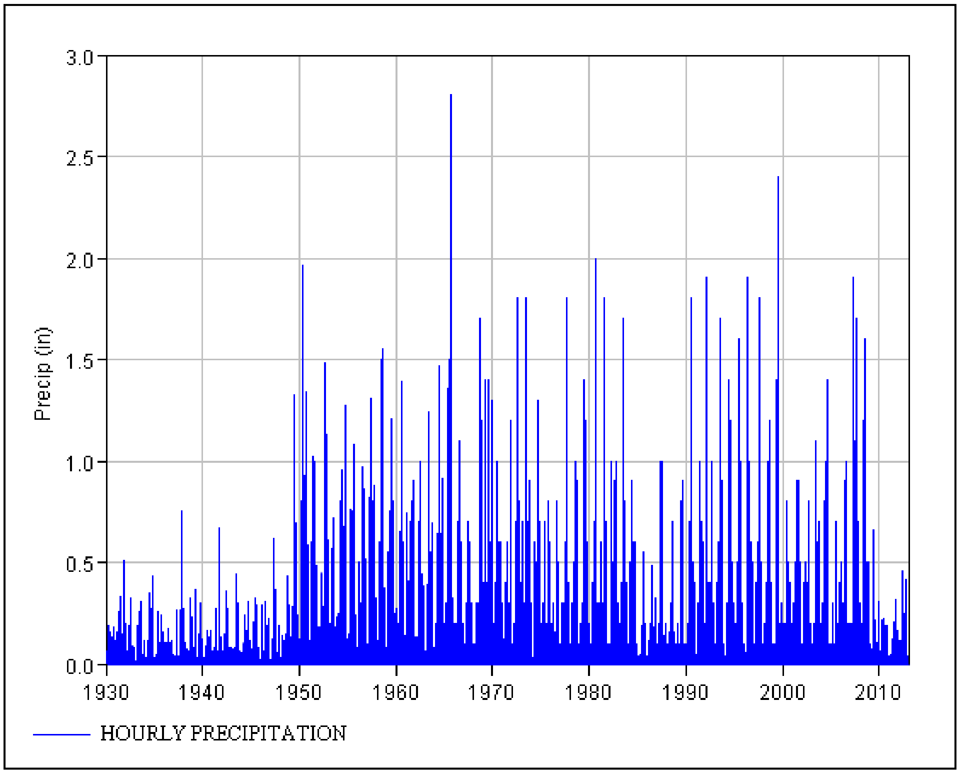

Figure 20. Hourly precipitation combined at Nebraska City and Hamburg.

Meteorologic Model

The meteorologic model utilized the described precipitation, also used in the L536 model, from 1930 through 2012. The hourly precipitation used in the hydrologic model is shown in Figure 20. The monthly average evapotranspiration method was used in conjunction with the meteorologic model. The monthly average method combines monthly average pan evaporation and a coefficient that converts pan evaporation to evapotranspiration. Monthly average evaporation was estimated using pan evaporation data from NCDC Climate Data Online (2016) at Shenandoah, Iowa, which was the closest available data. A coefficient of 0.75 was estimated based on the pilot study. Determination of evapotranspiration parameters is described in more detail in the Mill Creek pilot study in Attachment A. Monthly pan evaporation is shown in Table 8.

Table 8. Monthly average pan evaporation at Shenandoah, IA. A factor of 0.75 was used.

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Evaporation (in)	0	0	0	5.19	5.96	7.06	7.76	6.42	5.04	3.54	0	0

Delineation

Levee unit L575 subbasins were delineated based on the drainage basins in the Master Manual Interior Drainage Study (USACE, 1998). Updates were made to the delineations based on changes to levee outlets. The delineations are shown in Figure 15. The HEC-HMS schematic is shown in Figure 21. Levee unit L575 subbasin areas were determined using HEC-GeoHMS processing and are shown in Table 9.

Table 9. L575 subbasin areas.

Subbasin	A	A1	B	C	D	E	F	G	G1	H
Area (mi²)	30.82	9.37	5.88	1.21	6.85	4.46	23.19	35.29	23.92	4.25

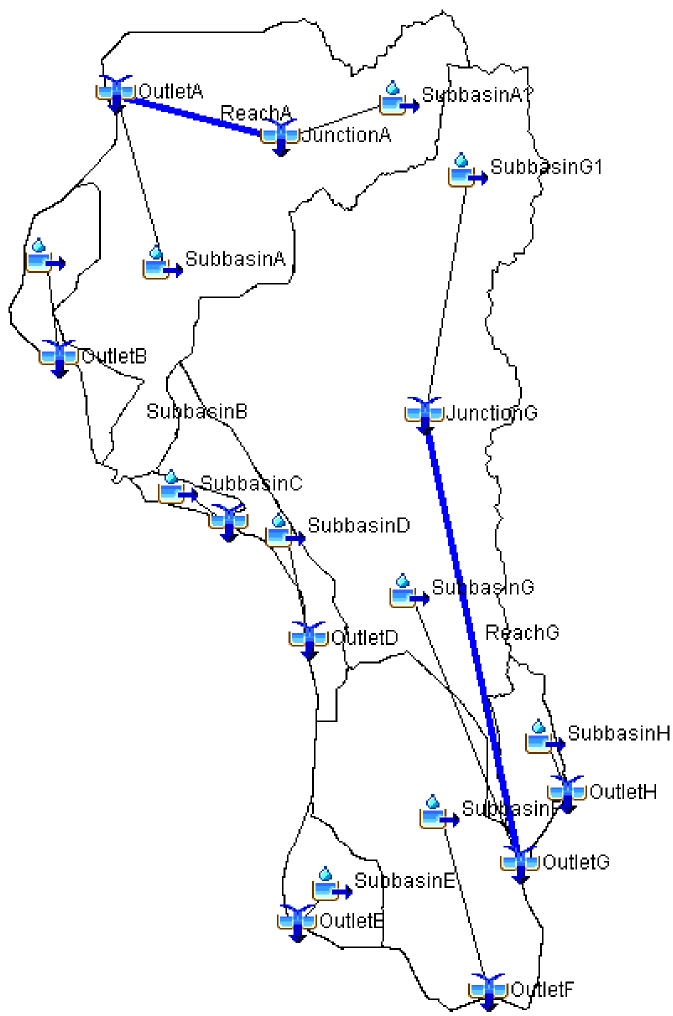

Figure 21. L575 HEC-HMS schematic.

Canopy

The simple canopy method was used in conjunction with monthly average evapotranspiration and rainfall loss methods. The canopy method models the interception of rainfall by plants in the landscape (USACE, 2013). It was assumed that there is no canopy storage in the L575 interior drainage basin. The simple uptake method was required by HMS in order to properly model evapotranspiration losses in the basin.

Rainfall Losses

The deficit and constant method was used to model rainfall losses. Parameters used to model deficit and constant losses are initial deficit, maximum storage, constant loss rate, and percent imperviousness. The initial deficit is an estimate of the moisture required to saturate the soil at the start of the simulation. The maximum deficit represents the maximum amount of moisture it would take to saturate the soil. The constant loss is represented by the hydraulic conductivity of the soil. Soil properties were determined based on gridded SSURGO soils data from NRCS (2012) and associated properties from Rawls, et al (1999). L575 is composed of a mix of clay loam and sandy soils, which accounts for higher constant loss rates than the L536 interior drainage basin. Soil parameter maps are shown in Figure 22 below.

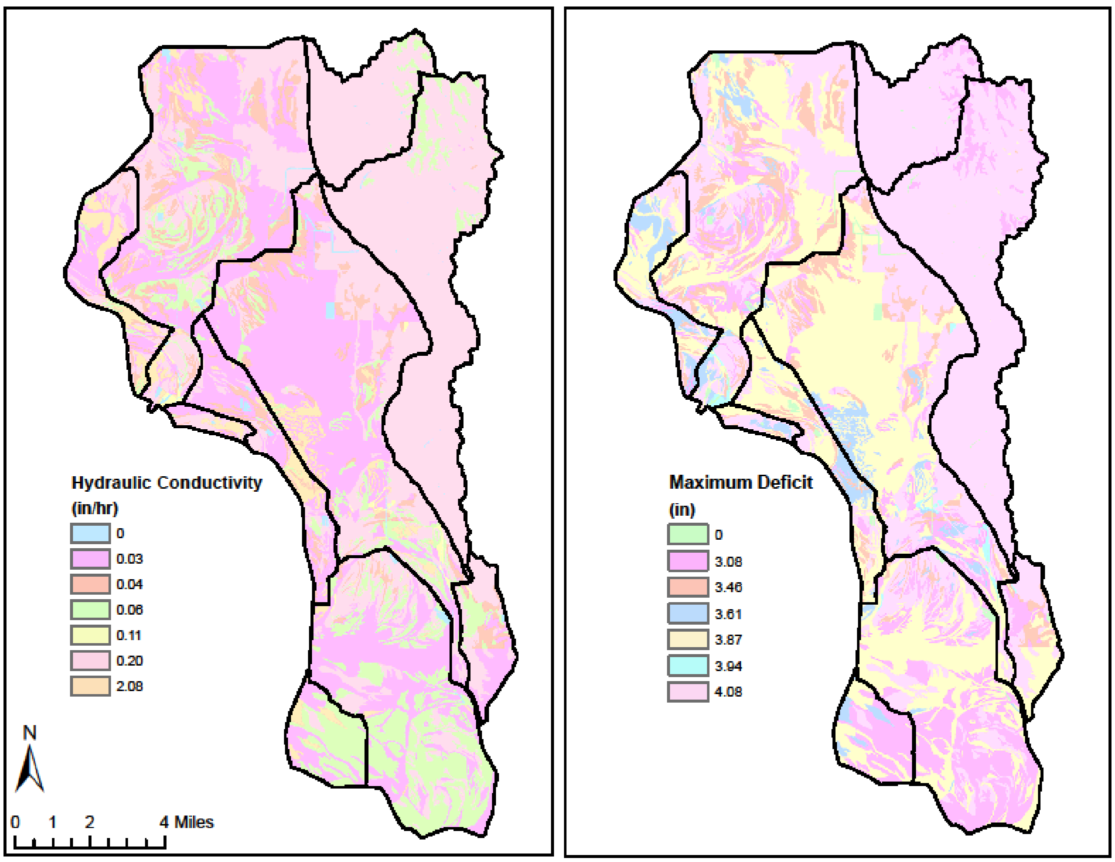

Figure 22. Deficit and Constant rainfall loss parameters used to find average subbasin parameters.

Percent imperviousness was determined from the National Land Cover Database, which assigns an average percent imperviousness to each 100' x 100' grid cell (NLCD, 2011). Percent imperviousness was calculated using HEC-GeoHMS processing. Hamburg is located in subbasin H, which accounts for the higher percent imperviousness. The Mill Creek pilot study indicated no change in rainfall loss parameters from initial estimates, so L575 deficit and constant parameters were not adjusted from the initial estimates. Deficit and constant loss parameters are shown in Table 10 below.

Table 10. Deficit and constant loss parameters. Hamburg is located in subbasin H, which accounts for the higher percent impervious of that subbasin.

Subbasin	Initial Deficit (in)	Max Deficit (in)	Constant Rate (in/hr)	Impervious (%)
A	1.5	3.8	0.60	1.24
A1	1.5	3.8	0.30	0.26
B	1.5	3.9	0.60	1.09
C	1.5	3.9	0.48	0.85
D	1.5	3.8	0.32	2.54
E	1.5	3.8	0.32	0.37
F	1.5	3.8	0.32	0.60
G	1.5	3.9	0.48	0.71
G1	1.5	3.9	0.32	0.25
H	1.5	3.8	0.48	7.91

Runoff Transformation

The Snyder unit hydrograph method was used to model the runoff transformation for L575. Snyder's utilizes lag time and a peaking coefficient (C_p). Lag time (t_l) is calculated from a lag time coefficient (C_t), the longest flowpath, and the centroidal longest flowpath. Flowpath measurements were estimated using Google Earth Pro and HEC-GeoHMS. C_t was estimated at 0.6 based on Veismann and Lewis (2002) and the Mill Creek pilot study. The peaking coefficient was estimated at 0.55 for subbasins A1 and G1 in the bluffs, and 0.5 for all other subbasins, because the subbasins have flatter slopes than the Mill Creek basin. Parameters used to model the Snyder unit hydrograph runoff transformation are shown in Table 11 below.

Table 11. Snyder unit hydrograph transform parameters.

Subbasin	C_p	C_t	Length (mi)	Centroidal Longest Flowpath (mi)	Lag Time (hr)
A	0.50	0.6	9.7	7.3	2.2
A1	0.55	0.6	4.8	2.4	1.3
B	0.50	0.6	7.5	4.6	1.7
C	0.50	0.6	2.3	1.1	0.8
D	0.50	0.6	5.0	1.3	1.1
E	0.50	0.6	3.1	1.7	1.0
F	0.50	0.6	7.8	3.9	1.7
G	0.50	0.6	18.5	8.1	2.7
G1	0.55	0.6	4.9	2.6	1.3
H	0.50	0.6	2.3	1.0	0.8

Channel Routing

The Muskingum-Cunge method was used to model channel routing in the L575 interior drainage basin. Two channels, A and G, were modeled in the L575 basin. Channel length, slope, shape, and Manning's n roughness coefficient are used in channel routing. Channel length and slope were determined using Google Earth Pro and HEC-GeoHMS. An eight-point cross-section was used to model both streams. The shape was estimated based on observations from the site visit. The observed shape of each channel is shown in Figure 23. Manning's n roughness coefficient was estimated at 0.04 for the channel and 0.045 for each bank of the channel based on the channel type (Gupta, 2008). The streams and overbank areas in basin A and basin G are very similar in roughness, so the same Manning's n coefficients were used for each. Muskingum-Cunge channel routing parameters are shown in Table 12.

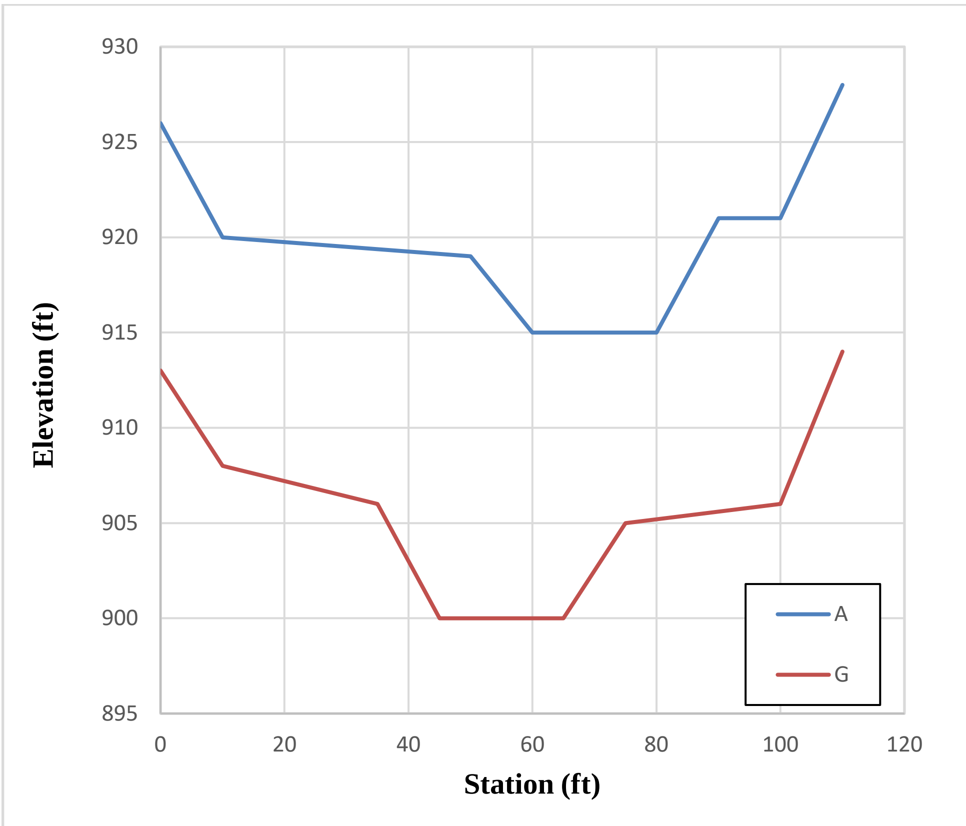

Figure 23. Channel cross-sections used in Muskingum-Cunge routing. Shapes were determined from observations of each channel.

Table 12. Muskingum-Cunge channel routing parameters.

Reach	Length (ft)	Slope (ft/ft)	Manning's n	Right Bank Manning's n	Left Bank Manning's n
A	25524	0.002	0.04	0.045	0.045
G	97908	0.001	0.04	0.045	0.045

Baseflow

The recession baseflow method was used in modeling L575. The recession method utilizes an initial discharge, threshold flow, and recession constant. The initial discharge and threshold discharge were estimated based on flow observations from the site visit. The recession constant was estimated based on the Mill Creek pilot study. Because the L575 drainage basin is much larger in area and has flatter slopes than the Mill Creek basin, it was assumed that the recession constant would be greater than that of the Mill Creek model. Recession baseflow parameters used to model L575 are shown in Table 13 below.

Table 13. Recession baseflow parameters

Subbasin	Initial Discharge (cfs)	Recession Constant	Threshold Flow (cfs)
A	20	0.50	35
A1	10	0.45	20
B	5	0.45	15
C	5	0.40	15
D	10	0.45	20
E	10	0.45	20
F	10	0.50	20
G	20	0.50	35
G1	5	0.50	15
H	10	0.45	20

CALIBRATION

Calibration of the L575 interior drainage basin was completed using the Mill Creek pilot study in Attachment A, the USGS runoff volume publication, and spring pulse gage data. The intent of the Mill Creek pilot study was to determine how calibration altered the starting parameter estimates and to use the same parameter adjustments as calibration for the interior drainage hydrologic model. Calibration of the Mill Creek pilot study included adjustments of the monthly average evapotranspiration coefficient, baseflow recession constant, and the runoff transformation peaking and lag time coefficients. Parameters that were not adjusted in the Mill Creek pilot study were estimated using the same methods and were not adjusted from initial estimates so calibration was paralleled in the L575 model.

In the Mill Creek pilot study, the monthly average evapotranspiration coefficient was initially estimated to be 0.6 and adjusted in calibration to 0.75. Because the same monthly average evaporation values were used in the L575 model, the same coefficient, 0.75, was used, as well.

The baseflow recession constant for Mill Creek was initially estimated to be 0.5 and reduced to 0.34 for each subbasin to decrease runoff volume per unit area, and to account for the very small area of each subbasin. The same initial estimate was used for the L575 subbasins because the two basins are very similar. L575 subbasin areas are much larger than the Mill Creek and L536 subbasins, so the recession constant was reduced to 0.4 for subbasin C, 0.45 for subbasins A1, B, D, E, and H, and 0.5 for the rest of the subbasins.

The peaking coefficient was initially estimated at 0.8 in the Mill Creek pilot study and was adjusted to 0.6 in calibration. L575 is in the same region as Mill Creek, so the initial estimate is the same; however, the final calibrated peaking coefficient was adjusted to 0.5 to account for flatter slopes in the basin. A peaking coefficient of 0.55 was used for the two subbasins in the bluffs, A1 and G1, to account for steeper slopes. The lag time coefficient for Mill Creek was initially estimated at 0.4 and adjusted to 0.6 in calibration. The same starting lag time coefficient estimate and calibration adjustment were applied to the L575 model.

The L575 model was also calibrated using the USGS published document showing average annual runoff throughout the United States from 1951 to 1980 (Gebert, 1987). According to the map of average annual runoff, L575 would produce approximately 6.5 inches of runoff volume per unit area from 1951-1980. Over the same observation period, the L575 HMS model produced approximate 3.5 inches of runoff volume per unit area, which is significantly less than the USGS publication. The significant difference can be attributed to the higher loss rates in the basin. Loss parameters could be adjusted to better calibrate runoff volumes to USGS estimates; however, the USGS document generalized runoff amounts for large areas and variability at a watershed scale is to be expected, so loss parameters were not altered from original estimates. Table 14 shows a comparison of runoff volume per unit area over the USGS publication observed period of 1951-1980.

Table 14. Comparison of runoff volume per unit area from 1951-1980.

Source	Runoff Volume per unit area (inches)
Subbasin A	1.9
Subbasin B	2.0
Subbasin C	2.5
Subbasin D	5.2
Subbasin E	2.0
Subbasin F	1.9
Subbasin G	1.4
Subbasin H	11.3
USGS Runoff Publication	6.5

The final method of calibration of the L575 interior drainage basin utilized spring pulse gage data. Spring pulse data was available in 2008, 2009, and 2010. Spring pulse data was compared to modeled data resulting from the L575 hydraulic model. The original estimates of loss rates, as well as alternatives of 75%, 50%, and 25% of original estimates, were analyzed. The results of the comparison showed that, while there were peak flows that were better modeled using one of the three alternatives, initial loss rate estimates ultimately provided the most valuable results, in terms of runoff volume, over the entire modeled period of record (1930-2012).

RESULTS

The L575 interior drainage basin was calibrated based on parameter adjustments of the Mill Creek pilot study and runoff volumes per unit area from 1951-1980 provided in the USGS runoff publication. Results are shown from 1930-2012. Flows at each one of the outlets are shown in Figure 24 to Figure 31. Yearly runoff volume per unit area, the total runoff from each subbasin over each year in the period of record, at each outlet as well as a period of record annual average are shown in Table 15. Yearly averages are much lower in the L575 subbasins than in L536, which can be attributed to higher constant losses. Subbasin H produced a higher runoff volume per unit area because Hamburg is located in that subbasin which resulted in a higher percent imperviousness and greater direct runoff.

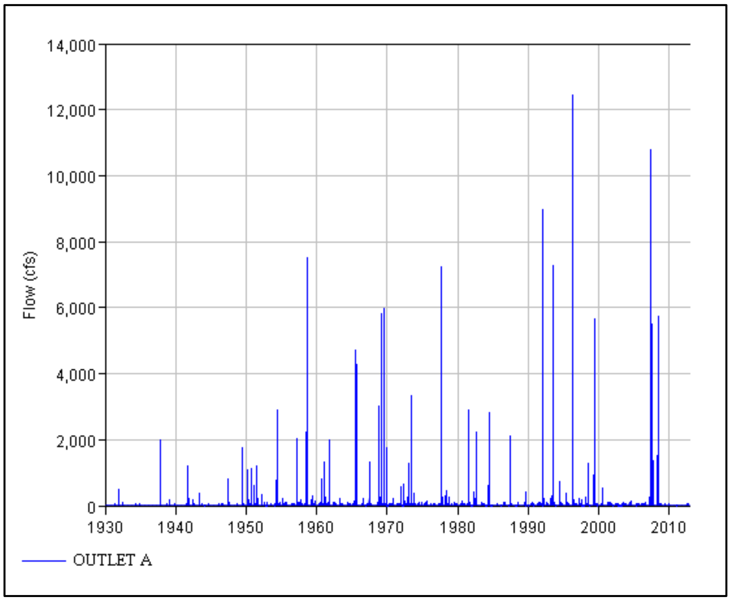

Figure 24. Subbasin A outflow.

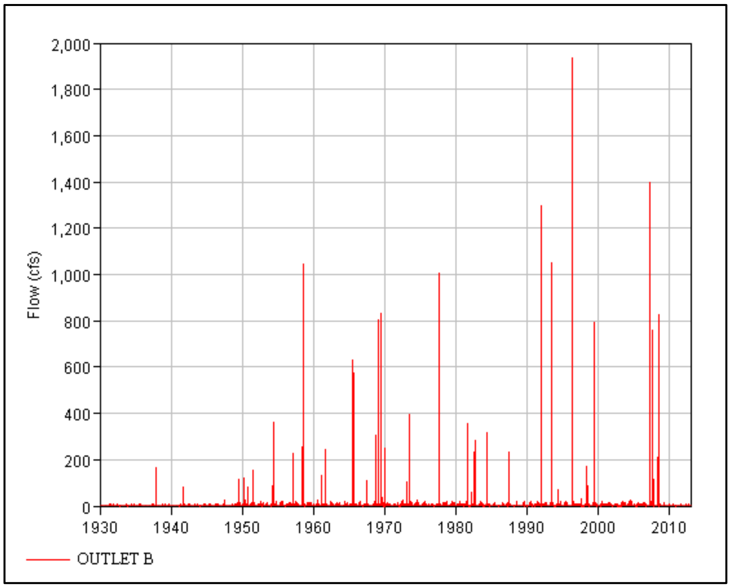

Figure 25. Subbasin B outflow.

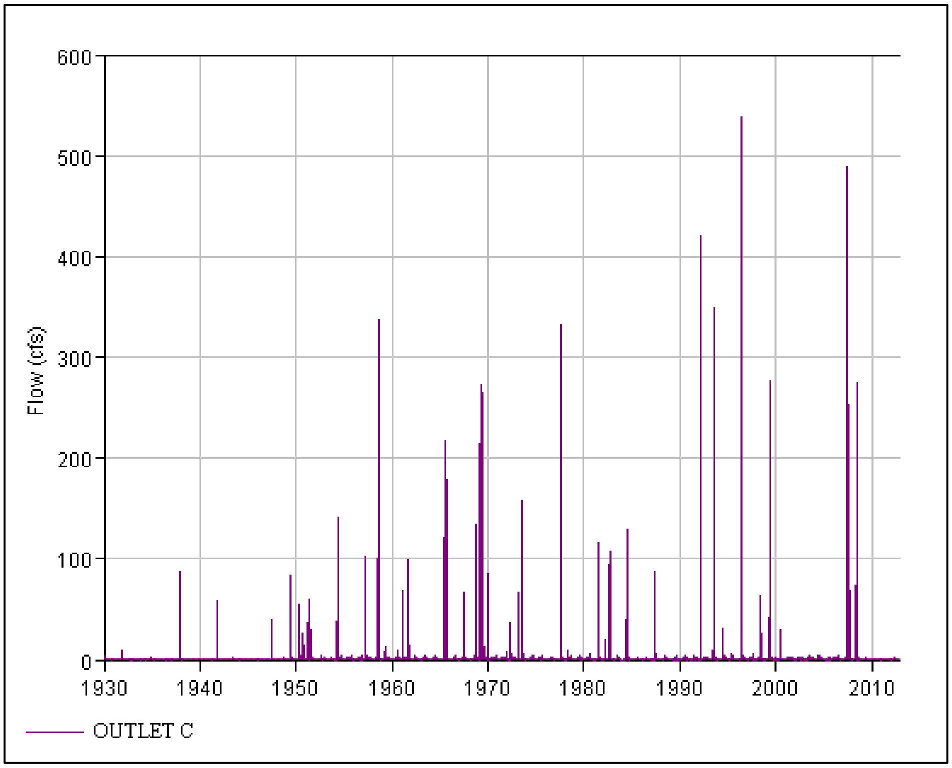

Figure 26. Subbasin C outflow.

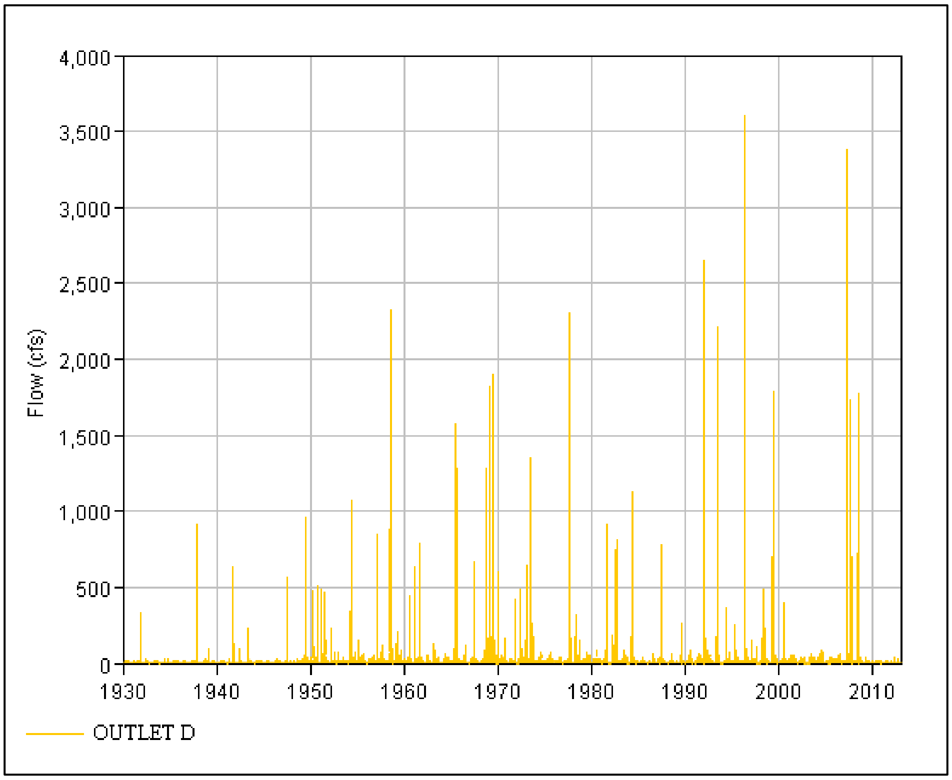

Figure 27. Subbasin D outflow.

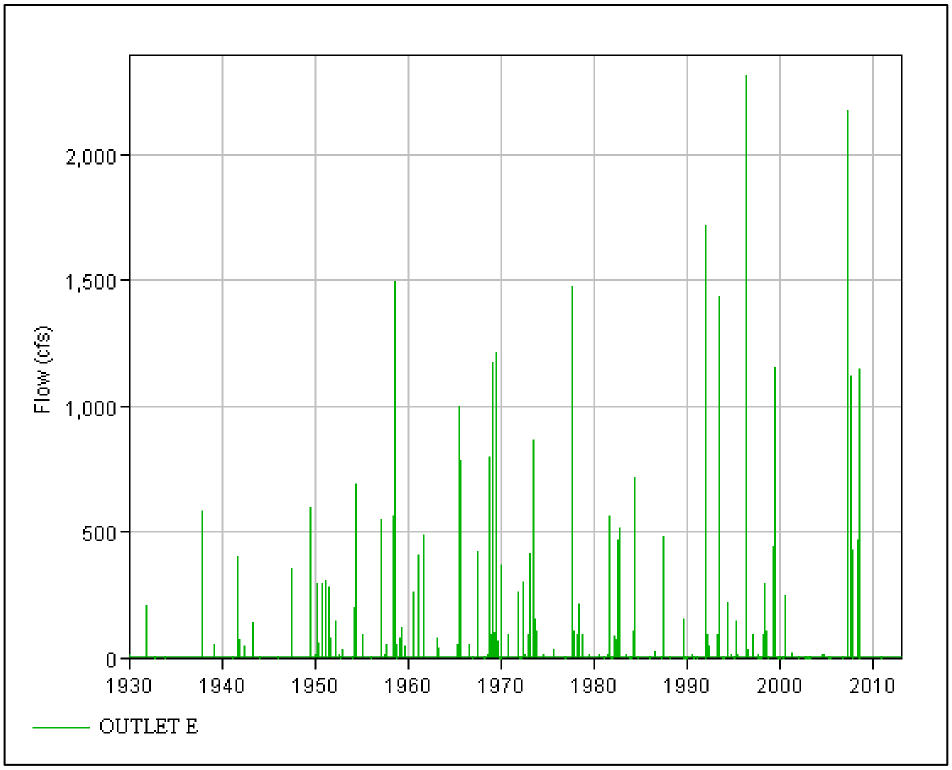

Figure 28. Subbasin E outflow

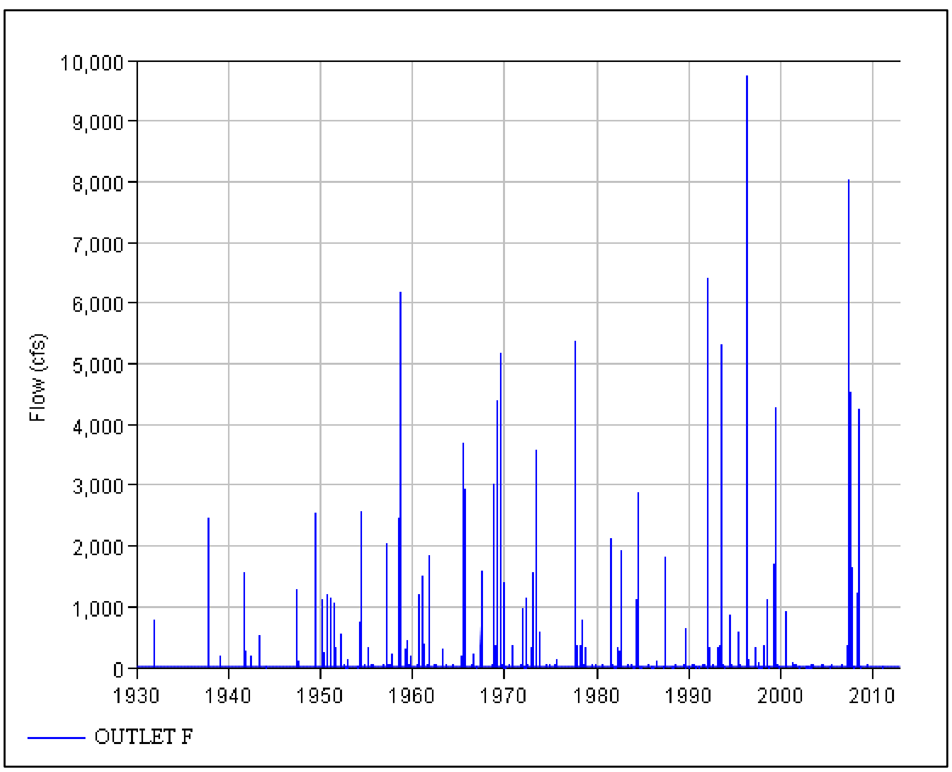

Figure 29. Subbasin F outflow.

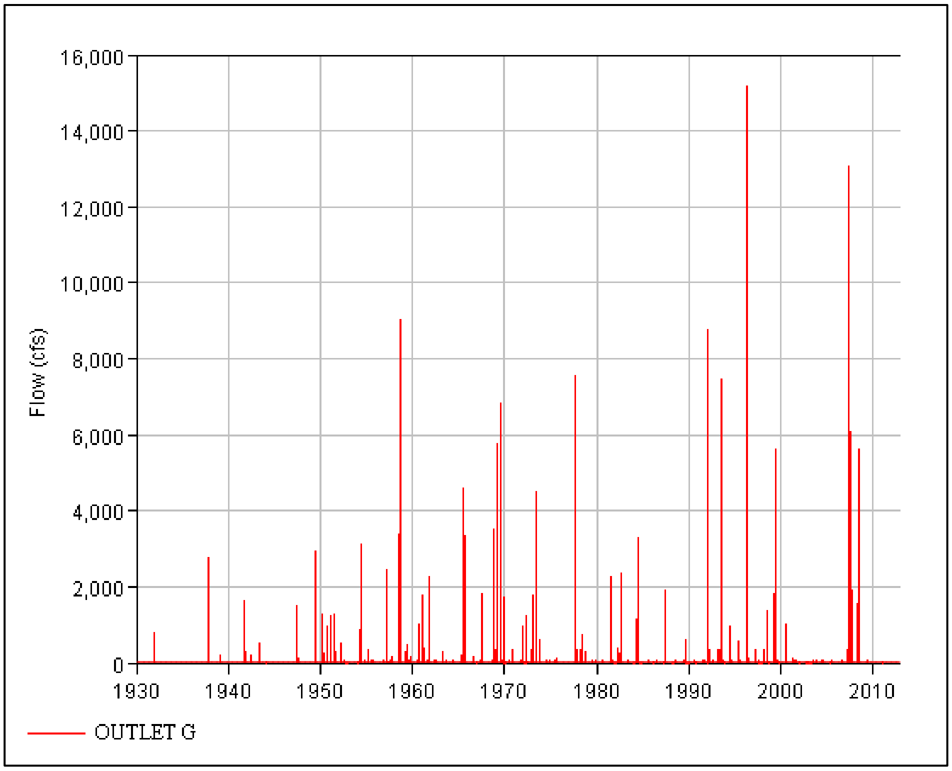

Figure 30. Subbasin G outflow.

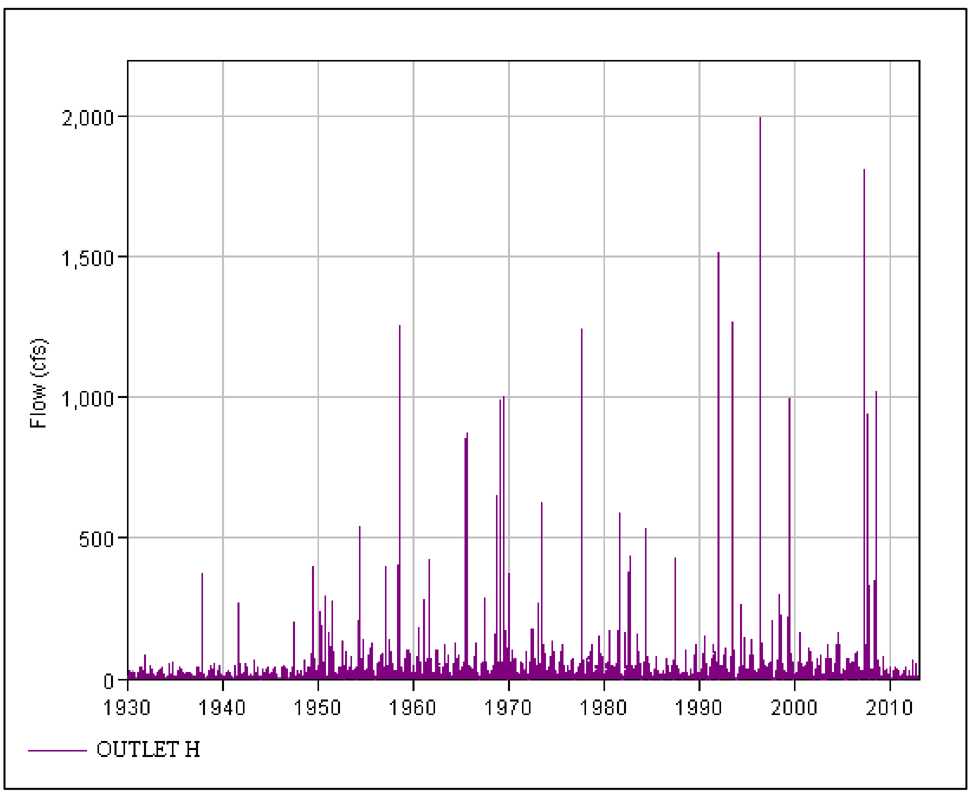

Figure 31. Subbasin H outflow.

Table 15. Runoff volume per unit area, in inches, at each outlet in the L575 interior drainage basin. A yearly average for each outlet is also provided. Higher constant loss rates account for much lower yearly average runoff volume than that of L536.

Year	A	B	C	D	E	F	G	H
1930	0.65	0.66	0.55	1.31	0.29	0.40	0.33	4.45
1931	0.94	0.93	0.85	2.07	0.63	0.75	0.54	6.10
1932	0.61	0.62	0.35	1.22	0.17	0.38	0.29	3.85
1933	0.64	0.67	0.42	1.41	0.19	0.41	0.32	4.37
1934	0.49	0.53	0.29	1.12	0.14	0.33	0.26	3.30
1935	0.72	0.75	0.45	1.58	0.22	0.46	0.36	5.26
1936	0.49	0.49	0.36	1.17	0.17	0.30	0.24	3.77
1937	1.02	0.98	1.22	2.23	1.03	1.11	0.82	5.33
1938	0.62	0.65	0.38	1.31	0.18	0.40	0.31	4.23
1939	0.62	0.61	0.37	1.47	0.41	0.42	0.33	4.00
1940	0.59	0.61	0.43	1.38	0.20	0.37	0.30	4.47
1941	0.92	0.90	1.15	2.11	1.05	0.94	0.67	5.17
1942	0.88	0.89	0.54	1.88	0.49	0.59	0.45	5.75
1943	0.72	0.72	0.46	1.79	0.54	0.56	0.40	4.75
1944	1.00	1.04	0.66	2.25	0.31	0.64	0.50	7.03
1945	0.83	0.84	0.57	1.89	0.28	0.51	0.41	5.64
1946	0.79	0.82	0.47	1.65	0.22	0.50	0.39	5.04
1947	0.91	0.88	1.10	2.19	0.98	0.88	0.63	5.41
1948	0.71	0.72	0.44	1.55	0.22	0.43	0.36	4.94
1949	1.90	2.02	2.74	5.64	1.92	1.97	1.43	12.74
1950	2.14	2.30	2.83	5.75	2.12	1.88	1.29	11.66
1951	2.42	2.48	3.86	6.87	2.85	2.42	1.72	14.14
1952	1.33	1.40	1.11	3.55	1.08	1.01	0.68	8.82
1953	1.00	1.06	1.02	3.33	0.50	0.67	0.49	8.68
1954	1.83	1.93	2.61	4.83	1.73	1.75	1.30	10.33
1955	1.08	1.32	1.11	3.43	0.81	0.85	0.62	8.40
1956	1.20	1.40	1.32	3.85	0.65	0.88	0.65	8.83
1957	1.93	2.14	2.33	5.20	1.67	1.63	1.24	11.34
1958	4.50	4.37	6.42	9.11	6.01	5.49	4.42	14.43
1959	2.14	2.18	2.65	6.16	2.31	1.74	1.22	13.79
1960	1.77	1.90	1.65	5.30	1.49	1.49	0.96	11.92
1961	2.08	2.37	2.90	5.65	2.18	2.09	1.58	12.02
1962	1.39	1.71	1.53	4.20	0.76	1.01	0.75	10.10
1963	1.31	1.46	1.36	4.24	1.09	1.01	0.69	9.70
1964	1.33	1.44	1.34	4.30	0.66	0.90	0.68	10.40
1965	4.23	4.17	6.39	8.68	5.30	4.56	3.69	15.75
1966	1.21	1.32	1.19	3.75	0.81	0.91	0.61	9.48
1967	1.76	1.79	2.36	5.44	1.63	1.65	1.14	12.10
1968	1.61	1.62	2.18	4.64	1.87	1.69	1.23	11.13
1969	4.59	4.70	6.74	8.46	5.98	5.12	4.34	13.20
1970	1.46	1.47	1.50	4.57	1.12	1.12	0.78	11.13

1971	1.18	1.11	1.35	3.67	0.98	0.99	0.68	9.17
1972	1.48	1.47	1.93	4.70	1.40	1.28	0.92	11.23
1973	2.72	2.87	3.91	7.69	3.49	3.03	2.21	16.04
1974	1.13	1.37	1.24	3.73	0.61	0.82	0.60	8.82
1975	1.26	1.51	1.26	4.11	1.02	0.97	0.68	10.89
1976	0.87	0.88	0.70	2.45	0.34	0.58	0.43	6.60
1977	5.13	4.71	7.06	10.86	7.38	6.53	5.09	16.76
1978	1.66	1.83	1.69	5.25	1.77	1.50	0.97	12.36
1979	1.55	1.78	1.71	4.97	0.84	1.14	0.82	11.90
1980	1.15	1.25	1.20	3.65	0.59	0.79	0.54	9.59
1981	1.84	2.06	2.36	4.82	1.69	1.58	1.17	10.61
1982	2.56	2.56	4.18	6.44	2.98	2.54	2.01	13.70
1983	1.13	1.23	1.17	3.68	0.58	0.79	0.57	9.97
1984	2.10	1.96	3.09	5.77	3.30	2.58	1.79	11.08
1985	0.74	0.81	0.50	1.68	0.24	0.48	0.38	5.16
1986	0.95	0.96	0.54	1.89	0.46	0.60	0.46	5.74
1987	1.40	1.56	1.90	3.67	1.34	1.34	1.00	8.95
1988	0.57	0.69	0.57	1.57	0.27	0.41	0.31	4.32
1989	0.76	0.86	0.79	2.29	0.70	0.70	0.49	5.38
1990	1.39	1.72	1.55	4.53	0.76	1.02	0.69	11.85
1991	1.28	1.45	1.27	3.95	0.62	0.92	0.66	9.58
1992	3.00	3.16	3.65	6.97	3.12	2.87	2.36	13.92
1993	6.16	5.61	7.96	12.01	8.52	7.40	6.10	18.36
1994	1.18	1.30	1.70	3.59	1.13	1.01	0.69	8.62
1995	1.56	1.80	1.78	5.12	1.33	1.28	0.84	13.10
1996	4.46	4.34	5.58	9.27	5.21	4.88	4.03	15.61
1997	0.97	0.95	0.97	3.04	0.75	0.72	0.49	7.46
1998	1.73	1.95	2.65	5.03	1.71	1.42	1.08	11.93
1999	3.23	3.12	5.65	7.26	4.61	3.81	2.99	13.13
2000	1.02	1.15	1.47	3.33	0.86	0.90	0.67	8.63
2001	1.28	1.57	1.38	4.24	0.84	0.94	0.70	10.58
2002	0.91	1.06	1.03	3.14	0.51	0.69	0.49	8.07
2003	1.23	1.36	1.19	3.83	0.58	0.90	0.61	9.66
2004	1.28	1.64	1.55	3.99	0.76	0.92	0.70	10.05
2005	0.74	0.81	0.74	2.43	0.36	0.52	0.40	6.46
2006	1.30	1.41	1.29	4.30	0.63	0.90	0.66	11.03
2007	5.61	5.30	8.37	11.38	8.21	7.39	5.79	17.30
2008	3.10	3.26	5.46	6.78	3.91	3.34	2.74	12.87
2009	0.80	0.82	0.50	1.72	0.24	0.49	0.40	5.40
2010	0.87	0.90	0.56	1.83	0.27	0.55	0.42	6.09
2011	0.68	0.69	0.43	1.45	0.21	0.41	0.33	4.85
2012	0.60	0.67	0.41	1.38	0.18	0.40	0.31	4.10
Yearly Average	**1.60**	**1.67**	**1.96**	**4.12**	**1.54**	**1.50**	**1.14**	**9.28**

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ATTACHMENT A: MILL CREEK PILOT STUDY

BACKGROUND

The Mill Creek pilot study was used in calibration of the levee unit interior drainage models. The intent of the pilot study was to use the same parameter estimation techniques and calibration adjustments for each levee unit model. There are no gages located in the interior drainage basins, so Mill Creek was selected to calibrate the levee unit models. After considering several different stream locations, Mill Creek near Oregon, MO was selected because of its similarities to the interior drainage basins and relatively small size (5.2 mi^2). USGS operated a gage (6816000) on Mill Creek from 1950-1976. Gage records include yearly peak flows and average daily flows. The hydrologic model was calibrated based on runoff volume per unit area over the USGS gage period of record.

HYDROLOGIC MODEL

Meteorologic Model

The Mill Creek meteorologic model was developed using hourly precipitation at St. Joseph, Missouri, and daily precipitation at Oregon, Missouri. Hourly precipitation was available at St. Joseph, MO, and daily precipitation was available at Oregon, MO. Each hour of the St. Joseph precipitation record was assigned a factor, which is the precipitation in one hour divided by the cumulative precipitation over that day. Those factors were then multiplied by the daily precipitation at Oregon, MO to convert to hourly precipitation. On days when there was precipitation in Oregon and none in St. Joseph, average hourly factors, computed from hourly factors at St Joseph over the gage period of record, were used to convert daily precipitation to hourly precipitation. The converted hourly precipitation at Oregon is shown in Figure 32 below.

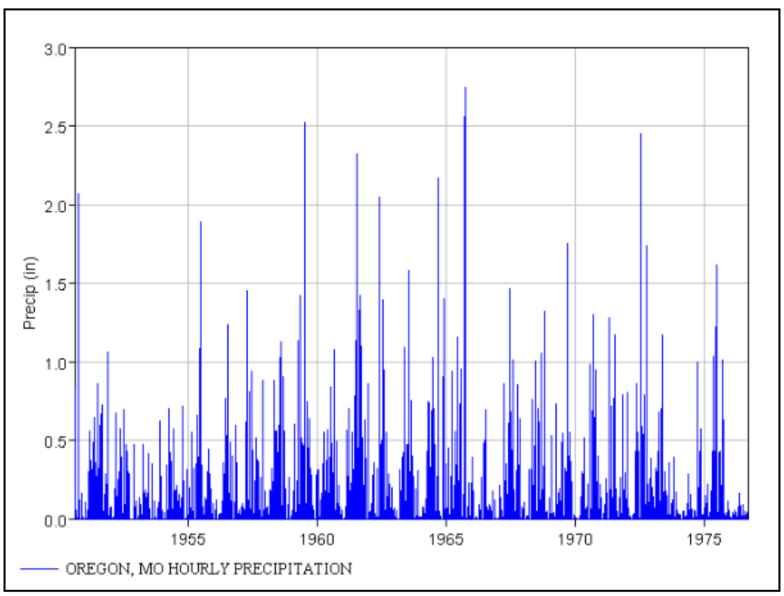


Figure 32. Oregon hourly precipitation converted using hourly factors from St. Joseph.

The monthly average method was used to model evapotranspiration in the Mill Creek drainage basin. The monthly average method uses a monthly pan evaporation rate and a pan coefficient to convert pan evaporation to evapotranspiration. Coefficients to convert pan evaporation to evapotranspiration can range from 0.6 to 0.8 for a Class A land pan (Gupta, 2008). An initial estimate of 0.6 was assumed. Runoff volume per unit area prior to calibration was too high, so the coefficient was adjusted to 0.75 to account for more losses in the basin. Evaporation was estimated using monthly pan evaporation at

Shenandoah, Iowa from 1966-1986, and monthly averages were calculated. Shenandoah provided the closest available data obtained from NCDC Climate Data Online (2016). The monthly average pan evaporation at Shenandoah, Iowa is shown in Table 16 below.

Table 16. Monthly evaporation at Shenandoah, Iowa used to model monthly average evapotranspiration.

Month	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Evaporation (in)	0	0	0	5.19	5.96	7.06	7.76	6.42	5.04	3.54	0	0

Delineation

Mill Creek subbasins were delineated using HEC-GeoHMS. The Mill Creek basin delineation and the HEC-HMS schematic are shown in Figure 33 below. Subbasin areas are shown in Table 17.

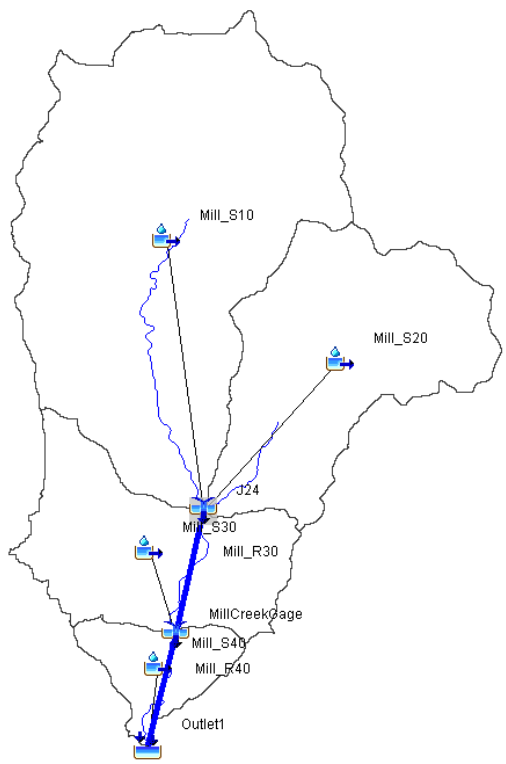


Figure 33. Mill Creek HEC-HMS schematic.

Table 17. Mill Creek subbasin areas.

Subbasin	**Mill_S10**	**Mill_S20**	**Mill_S30**	**Mill_S40**
Area (mi2)	2.70	1.28	0.90	0.28

Canopy

The simple canopy method was used in conjunction with monthly average evapotranspiration and rainfall loss methods. The canopy method models the interception of rainfall by plants in the landscape (USACE, 2013). It was assumed that there is no canopy storage in the Mill Creek basin; however, the simple uptake method was required by HMS in order to properly model evapotranspiration losses in the basin.

Rainfall Losses

The deficit and constant loss method was used to model rainfall losses. The method uses an initial deficit, maximum deficit, constant loss rate, and percent imperviousness. Gridded SSURGO soils data was used to determine soil properties (NRCS, 2012). The starting loss parameters were determined based on soil type, mostly silt loam soils, and the associated deficit and constant parameters (Rawls, et al, 1983). The National Land Cover Database (NLCD) was used to determine average percent imperviousness for each subbasin (2012). There was no adjustment of rainfall loss parameters during calibration of the Mill Creek model. Deficit and constant parameters are shown in Table 18 below.

Table 18. Deficit and constant loss parameters used in the Mill Creek model.

Subbasin	Initial Deficit (in)	Maximum Storage (in)	Constant Rate (in/hr)	Impervious (%)
Mill_S10	1.0	3.6	0.20	0.2
Mill_S20	1.0	3.6	0.20	0.3
Mill_S30	1.0	3.4	0.16	3.9
Mill_S40	1.0	3.6	0.20	2.3

Runoff Transformation

Snyder's unit hydrograph method was used to model the runoff transformation. This method utilizes a peaking coefficient (C_p) and a lag time coefficient (C_t). According to Viesmann and Lewis (2002), the lag time coefficient for the area of Mill Creek usually ranges from 0.2 to 0.6, with an average value of 0.4. Data was not available for the specific location of Mill Creek, so it was assumed that parameters would be similar to those of Western Iowa, which is the location of the provided range of values. The starting value was assumed to be 0.4 for all subbasins. That lag time coefficient was adjusted to 0.6 to account for relatively mild slopes and to reflect the timing of flows at the USGS streamflow gage. The lag time was calculated using the coefficient, longest flowpath, and centroidal longest flowpath.

Peaking coefficients in the Mill Creek region, again assumed to be similar to those of Western Iowa, are generally in the range of 0.7 to 1.0, with an average value of 0.8 (Viesmann & Lewis, 2002). A starting value of 0.8 was chosen for the Mill Creek basin. During calibration, that value was adjusted to 0.6 to account for milder slopes and the relatively small size of the basin. Although 0.6 is slightly outside the suggested range, it produces the most accurate results in respect to volume per unit area. Snyder's unit hydrograph parameters are shown in Table 19 below.

Table 19. Snyder's unit hydrograph parameters used to Mill Creek runoff transformation. Initial and final calibrated parameters are shown.

Iteration	Parameter	Mill_R10	Mill_R20	Mill_R30	Mill_R40
Initial	**C_t**	0.4	0.4	0.4	0.4
	Lag Time (hrs)	0.57	0.38	0.31	0.30
	C_p	0.8	0.8	0.8	0.8
Final	**C_t**	0.6	0.6	0.6	0.6
	Lag Time (hrs)	0.85	0.57	0.47	0.46
	C_p	0.6	0.6	0.6	0.6

Channel Routing

The Muskingum-Cunge method was used to route flow through channels. The method uses channel shape, slope, length, and Manning's n roughness coefficient. Lengths, slopes, and channel shapes were determined using Google Earth Pro and ArcMap 10.0. The Manning's n roughness coefficient used in modeling channel routing was 0.04 (Gupta, 2008). Channel routing parameters were not adjusted during calibration. Muskingum-Cunge parameters are shown below in Table 20.

Table 20. Muskingum-Cunge channel routing parameters.

Reach	Length (ft)	Slope (ft/ft)	Manning's n	Width (ft)	Side Slope (H:V)
Mill_R30	3800	0.002	0.04	12	4
Mill_R40	4020	0.002	0.04	12	4

Baseflow

The recession method was used to model baseflow. Baseflow parameters were estimated using the USGS streamflow gage. Initial discharge, recession constant, and threshold flow are used in the recession baseflow method. According to the HEC-HMS technical reference manual, the recession constant for surface runoff can range from 0.3 to 0.8 with smaller watersheds generally producing lower values. The starting value for the recession constant was estimated as 0.5. Through calibration, the value was adjusted to 0.34. The small value is justified by the very small size of the watershed and its subbasins. The initial discharge and threshold flows were adjusted to match flows at the USGS streamgage. Baseflow parameters are shown in Table 21 below.

Table 21. Recession baseflow parameters.

Subbasin	Initial Recession Coefficient	Final Recession Coefficient	Initial Discharge (cfs)	Threshold Flow (cfs)
Mill_S10	0.5	0.34	10	18
Mill_S20	0.5	0.34	10	18
Mill_S30	0.5	0.34	10	18
Mill_S40	0.5	0.34	10	18

CALIBRATION

The Mill Creek pilot study was calibrated using runoff volumes from the HMS model, the USGS Mill Creek streamflow gage, and the USGS runoff publication. In calibration, the recession baseflow constant, Snyder's Unit Hydrograph peaking and lag time coefficients, and monthly average evapotranspiration coefficient were all adjusted. Average yearly runoff volume per unit area, determined prior to any calibration adjustments, was approximately 7.2 inches. After adjustments of the aforementioned parameters, the resulting yearly average runoff volume per unit area was approximately 5.8 inches.

Runoff volumes in acre-feet were calculated using HEC-DSSVue 2.0.1 and were converted to inches of runoff based on the area of the basin. The USGS streamflow gage produced an average annual runoff of 5.7 inches during the observation period of 1950-1976 (2016). USGS published a document that provides average annual runoff throughout the United States in the form of a contour map from 1951-1980 (Gebert, 1987). The runoff publication showed a runoff volume per unit area of approximately 7.5 inches in the area of Mill Creek. The adopted Mill Creek HMS model produced an average annual runoff volume per unit area of 5.8 inches, which is about 2% greater than the USGS streamflow gage and about 30% less than the USGS runoff publication. That significant difference in runoff volumes between the USGS runoff publication and the HMS runoff can be attributed to using varying periods of record and also that the USGS document generalized runoff amounts for large areas and variability at a watershed scale is to be expected. Runoff volumes per unit area in inches are shown in Table 22.

Table 22. Runoff volumes of the Mill Creek HMS model and calibration sources.

Source	Observation Period (yrs)	Runoff Volume (in)
HMS Model	1950-1976	5.8
USGS Gage	1950-1976	5.7
USGS Runoff Publication	1951-1980	7.5

RESULTS

The Mill Creek pilot study was calibrated based on runoff volumes of the HMS model, the streamflow gage, and the USGS runoff publication. Table 23 shows the yearly runoff volume per unit area of the Mill Creek pilot study. The yearly average runoff volume per unit area of Mill Creek is approximately 5.8 inches, which is about 2% greater than the 5.7 inches of average yearly runoff volume of the USGS streamflow gage. The USGS runoff publication showed approximately 7.5 inches of runoff. The difference can be attributed to reducing the significant scale of the publication to a watershed scale. Flows resulting from the HMS model are shown in Figure 34.

Table 23. Yearly runoff volume per unit area of the Mill Creek pilot study.

Year	**Runoff Volume (in)**
1950	1.7
1951	13.7
1952	4.2
1953	1.9
1954	4.0
1955	2.9
1956	1.3
1957	5.7
1958	9.2
1959	9.6
1960	3.9
1961	12.5
1962	4.8
1963	2.6
1964	10.1
1965	14.3
1966	0.7
1967	8.3
1968	2.7
1969	4.5
1970	5.1
1971	4.0
1972	5.3
1973	8.6
1974	2.1
1975	8.3
Yearly Average	**5.8**

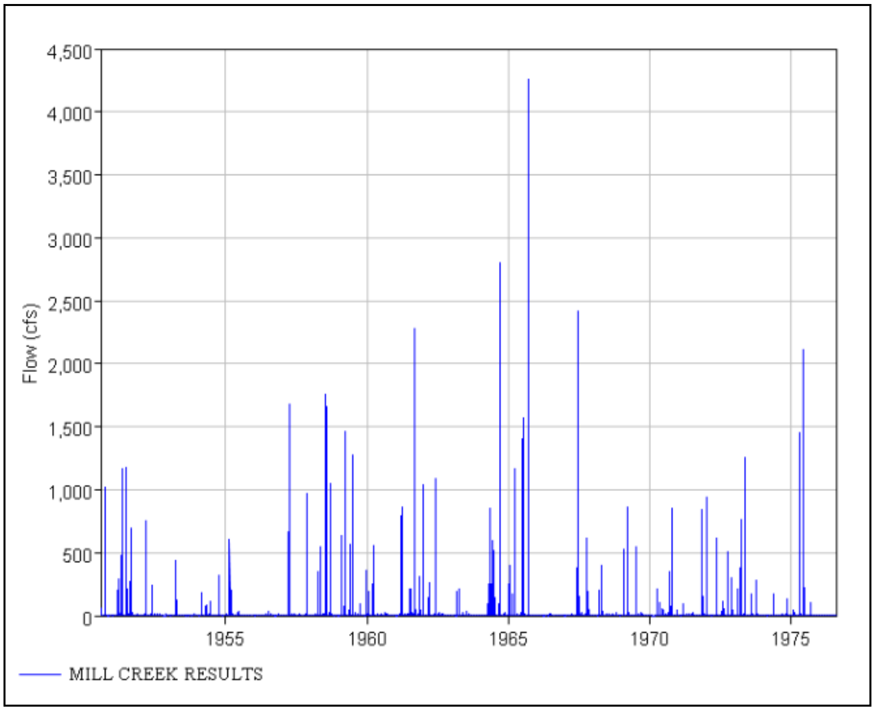


Figure 34. Mill Creek HMS flow results shown from 1950 to 1976.

CONCLUSIONS

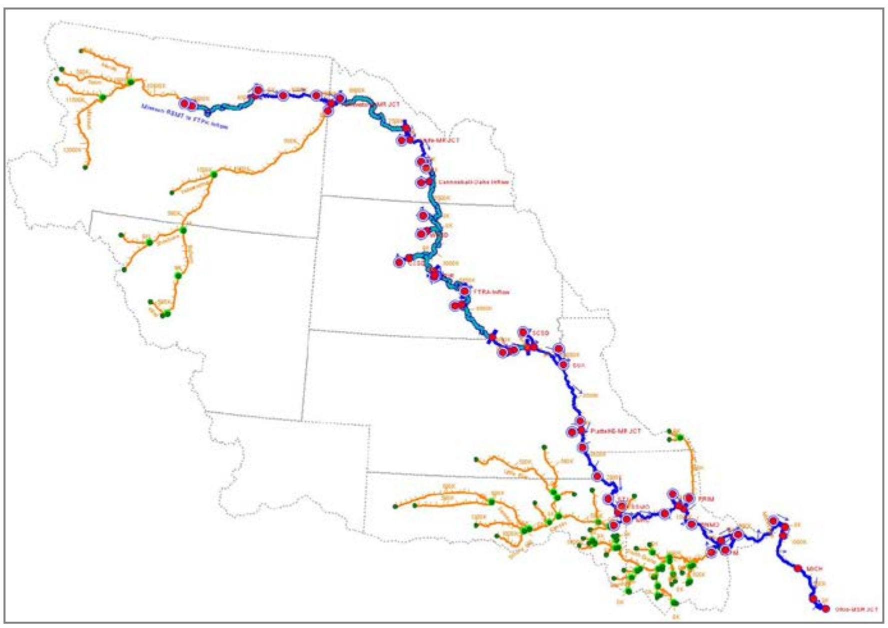
The intent of the Mill Creek pilot study was to use the described methods to determine initial parameter values and adjust the values for each levee unit according to the adjustments made in calibration. Calibration of the Mill Creek pilot study included adjusting Snyder's Unit Hydrograph runoff transformation and recession baseflow parameters; therefore, those same parameters should be adjusted in the levee unit studies. In the case of Snyder's Unit Hydrograph parameters, the interior drainage basins have milder slopes than that of Mill Creek so the peaking coefficient and lag time coefficients should be adjusted accordingly. Baseflow parameter adjustments should be similar, and the same method of initial estimation should be used. Because there was no adjustment in deficit and constant loss parameters calculated using gSSURGO soils data, it can be assumed that no adjustments need to be made to levee unit loss parameters calculated using the same method.

US Army Corps of Engineers ®
Omaha District

Missouri River Unsteady HEC-RAS Model
Sediment Analysis

FINAL

Gavins Point Dam to Rulo, NE



July 2018

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ACRONYMS

BiOp…………………..Biological Opinion

CFS…………………. Cubic Feet per Second

ESH…………………..Emergent Sandbar Habitat

HC……………………Human Considerations

HEC .... Hydrologic Engineering Center

IRC……………………Interception-Rearing Complexes

MAF………………….. Million acre-feet

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

MRRIC………………..Missouri River Recovery Implementation Committee

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

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

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

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

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

POR .... Period of Record

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

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

RM……………………1960 River Mile

SWH………………....Shallow Water Habitat

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

USFWS………………United States Fish and Wildlife Service

USGS .... United States Geological Survey

1 INTRODUCTION

Future condition sedimentation evaluations were performed to help support questions and comments from Missouri River Recovery Implementation Committee (MRRIC) members, review panels, and the public related to sedimentation. Future conditions, that include aggradation and degradation within the reservoir reaches and navigation channel, may change the river stage - flow relationship and consequently could affect alternative condition performance. HEC-RAS (RAS) with sediment modeling was used to evaluate future condition channel conditions. While the current condition modeling effort is an informative tool, the Missouri River is dynamic with ongoing aggradation and degradation processes.

1.1 BACKGROUND

Historically the Missouri River was characterized as a free-flowing, highly dynamic, multi-channel river, consisting of highly variable flows, high turbidity conditions, with many channel sandbars throughout the river channel and floodplain, which provided a diversity of habitat and food resources for many terrestrial and aquatic organisms. Since the late 1800's, the Missouri River has been modified by reservoirs, bank stabilization, construction of the navigation channel, and many other water resources development projects that have affected basin sediment yield and sediment transport within the mainstem Missouri River and tributaries. Navigation channel and bank stabilization structures have altered the historic multi-channeled, highly variable river system into a predominantly deep and swift, single channel in the lower river downstream of Sioux City. Reservoir construction has created a series of alternating pools and open river reaches. These actions have combined to result in noticeable aggradation and degradation trends on the mainstem Missouri River. For instance, degradation downstream of Gavins Point dam has been observed in the range of 10 to 12 feet since dam closure in 1955. During the extreme 2011 event, many areas on the Missouri River experienced single event degradation and aggradation in the range of two to four feet (USACE 2012). In summary, aggradation and degradation trends are documented, recent, and known to be ongoing.

1.2 YEAR 15 FUTURE CONDITION

The future condition modeling was referred to as "Year 15". The designation "Year 15" comes from the timeframe for implementation; the Record of Decision (ROD) for the Missouri River Recovery Program, Management Plan Environmental Impact Statement (EIS) is expected to be signed in 2018, with a construction completion date of 2033, resulting in an implementation period of 15 years. All alternatives were evaluated for the Year 15 future condition. While not intended to represent detailed estimates of future reservoir and channel conditions, the results do provide an alternative comparison methodology. Results from the Year 15 analysis were provided to the human considerations teams for qualitative evaluation, versus the full quantitative evaluation that was performed on the base condition (also referred to as Year 0).

1.3 LIMITATIONS

The future condition modeling was based on a number of critical assumptions regarding historic trends, flows, and sediment inputs. Model methodology is consistent for use with the MRRMP-EIS study using the period of record (POR) flows. Limitations are summarized as:

- Calibration of the model was limited by available data.
- Results are qualitative. Determination of future condition variation between alternatives and application with Human Considerations evaluation may by limited due to RAS model assumptions and accuracy.

2 METHODOLOGY

The methodology used a calibrated historic condition sediment model to derive reasonable sediment modeling parameters for the POR simulation. These parameters are used with the current condition HEC-RAS model available from the MRRMP-EIS study to simulate the 82-year period of record and develop a rate of change throughout the model length. Simulations were performed using HEC-RAS version 5.0.3.

2.1 SIMPLYING ASSUMPTIONS

- The model will only utilize the sand fraction of the sediment load (material greater than 0.0625 mm). Silts and clays, which act as wash load and remain suspended in water column, will not be included within the sediment model.
- The unsteady RAS model created for the calibration analysis will be simplified to only the Missouri River mainstem without any tributary routing reaches.
- Flow input locations will be reduced to only tributaries with sediment loads. Small ungaged tributaries and the ungaged uniform lateral flows are not included.

2.2 MODELING OVERVIEW

- Assemble a historic model for the Missouri River
- Calibrate the historic model with steady flow and available water surface data (gages and profiles)
- Assemble sediment data and simulate from the historic period to current
- Assess sediment model performance with volume change, water surface change, compare to available sediment load and budget data
- Apply the sediment parameters to the current condition simplified Missouri River model and perform sediment computations using the 82-year POR flows
- Compute the water surface elevation change at the end of the 82 year period. Pro rate that change to an annual rate, apply over 15-year period

- Modify the Management Plan model to produce a profile equivalent to the projected change over the 15-year period.

2.3 Assemble The Sediment Model Input Data

- Each model starts at the upstream dam, assume no sediment input from the reservoir release
- Assemble Missouri River bed gradation from historic bed samples. Review data to ensure that the bed material includes larger gradations to reflect armoring in the immediate reach downstream of the reservoir.
- For tributary sediment input, assemble sediment load information using best available USGS gage data and previous studies. Focus on the major drainage area tributaries using the drainage area accounting table for the reach.
- Calibrate the historic condition RAS model water surface elevation to the warm season (mid-April through mid-October), which is of primary interest.
- Set mobile bed limits using the bank stations.
- Select Toffaleti, Laursen Copeland, or Yang as the transport function (initial that will be verified in calibration).
- Select Copeland (Exner 7) or Exner 5 as the sorting method (initial that will be verified in calibration).
- Select Report 12 or Ruby fall velocity method (initial that will be verified in calibration).

2.4 Sediment Model Startup

Perform a stability check of the assembled sediment model and initial sediment parameter review for low, medium, and high flows.

- Create 30 day constant flow files of low (10k), medium (near bank full, e.g. 70,000 for Gavins), and high flows (above bank in the non-degradation reach, e.g. 100,000 for Gavins).
- Perform sediment computations with the model for each steady flow condition, start with the medium flow condition for initial evaluation and debugging.
- Review performance at each flow, revise model levee stations / encroachments / ineffective flows / cross-section spacing to achieve reasonable sediment response.
- Review model sediment inputs are reasonable by comparing to gage station sediment load rating curves.
- Perform initial adjustment of sediment input parameters to achieve 30 day model stability.
- Check bed level change and debug problem areas.

2.5 Calibration and Simulation of Historic Period

- Calibrate historic condition model roughness to water surface profiles at the start of the historic period.
- Simulate the flow period for the time period between surveys (e.g. 1995 thru 2012). Use daily unsteady flow with tributary inflow and ungaged as developed for the ManPlan calibration model (USACE 2015).
- Compare model computed volume change to actual on a reach basis for the simulation period.
- Compare water surface elevation at the end of the simulation period (e.g. 2012) to observed
- Review and refine model sediment inputs. Compare model results to available suspended sediment concentration data during the historic period at gage locations (e.g. Omaha).

2.6 Tabulate Sediment Model Calibration Parameters

The primary product from the development and calibration of the historic sediment model are the sediment modeling parameters. These parameters will be tabulated and reviewed for use with the 2012 condition sediment model.

3 COMPUTATIONAL ANALYSIS SEDIMENT MODEL DEVELOPMENT

Available data was used to assemble a historic model for the simulation with sediment. Historic model creation and calibration followed the same processes as previously document in the calibration model. Refer to the *Missouri River Recovery Program Management Plan Environmental Impact Statement Existing Conditions Unsteady RAS Model Calibration* Report (USACE 2015b) for a detailed description of HEC-RAS model development and calibration.

3.1 Sediment Model Computational Studies

Following the hydraulic model calibration, additional model calibration is required for modeling of sedimentation processes. Sediment modeling computational studies may be defined in two general categories (1) computational model studies and (2) computational analysis studies (Thomas and Chang, 2008). Calibration is the process of arriving at sediment model parameters (e.g. hydraulic parameters such as roughness, sediment loads, sediment material size, sediment transport function, etc.) that will allow the model to calculate values that agree with measured prototype values. A sediment model computational study involves both calibration and verification. Verification refers to the demonstration that a calibrated model will match the prototype during a period of time not used in calibration. Due to the limited data set, model verification was not possible for the sediment model. Therefore, the performed evaluation is referred to as a computational analysis. Such studies are useful and allow evaluation of the study area and comparison of calculated results between alternatives.

3.2 Model Geometry

Model geometry was assembled for use with the sedimentation modeling effort.

3.2.1 Historic Geometry

Two periods of historic hydrographic surveys were available for possible sediment modeling, consisting of data from 1975 and 1995. Both data sets were assembled into a RAS model geometry by importing data from previous study hydraulic model files. The 1995 era model was imported from the 2003 study (USACE 2003) and the 1975 model was imported from the 1970 era HEC-2 model files used in the Missouri River Flood Hazard Study (USACE, 1981). When working with the data, several issues were noted as follows:

- Within the 1995 data set, the Gavins to Ponca portion of the model reach, from RM 752 to 811, was collected in 1994. Modeling quickly illustrated that the Gavins to Ponca data from 1994 was suspect with an apparent water surface elevation discrepancy of several feet.
- The 1975 era data set had very few cross section data points.
- Volume change computations with both the 1975 or 1995 model data were suspect.
- Neither the 1995 nor the 1975 hydrographic surveys RAS models had xyz format for the cross section. Location was assigned by river mile label.

3.2.2 Geometry Assembly and Modifications

The initial model geometry included the same flow-roughness and seasonal calibration factors as developed for the calibration model (USACE 2015b). Several simplifications were necessary when transitioning from the previously developed calibration model to the sedimentation model including:

- Remove all tributary reaches; develop a single Missouri River mainstem model.
- Remove all storage areas and storage area connections.
- Remove geometry used in the unsteady flow model to reflect the sediment model does not include storage or flow attenuation: delete all portions of the model cross section landward of the federal levee (Omaha to Rulo); reduce the cross section length upstream of Omaha to reflect 2011 flood extents.
- Revise the ineffective flow areas within the floodplain for use with the sediment model to limit the effective flow area and prevent massive change in model hydraulics when flows above channel capacity occur.
- Set bank stations within the navigation channel reach at about the 2-year flow event level; set bank stations in the Gavins to Ponca reach (RM 811-752) to include the mid-channel bars and allow use of the bank stations as the moveable limits.

3.3 HYDROLOGIC DATA

Flow data used within the model was the same as that developed previously for the calibration model and POR analysis. The model employed daily flow data for all tributaries and ungaged flow inputs. Refer to the calibration model report (USACE 2015b) for additional information.

Sediment modeling should include sediment input associated with each flow input. Many of the flow inputs used in the calibration model such as the ungaged inflow and the minor tributaries do not have any information to define sediment input. These flow input locations were not included within the sediment model. A listing of the included sediment and flow inputs along with drainage area is included in Table 1.

Table 1. Flow and Sediment Inputs for Gavins Point Dam to Rulo, NE

Stream	Station	Missouri River Mile	Missouri Drainage Area (sq. mi)	Tributary Drainage Area (sq. mi)	Sediment Model Input
Gavins	**Gavins**	**811.10**			Yes
Missouri River	**Yankton, SD**	805.8	279,500		
James River	Yankton-Scot.	797.7		20,942	Yes
Bow Creek	St James, NE	787.6		304	
Vermillion River	Vermill-Wak.	771.9		2,302	Yes
Aowa Creek	Ponca, NE	745.2		222	
Elk Creek		737.3		132	
Big Sioux River	Akron, IA	734.0		8,424	Yes
Unknown	Ungaged			2,774	
Percent Ungaged	7.9%				
Missouri River	**Sioux City, IA**	732.3	314,600		
Perry Creek	Sioux City, IA	732.1		65	
Floyd River	James, IA	731.3		886	Yes
Omaha Creek	Homer, NE	719.9		174	
Blackbird Creek		697.6		106	
Unknown	Ungaged			369	
Percent Ungaged	23.1%				
Missouri River	**Decatur, NE**	691.0	316,200		
Monona Har. Ditch	Turin, IA	670.0		900	Yes
Little Sioux River	Turin, IA	669.2		3,526	Yes
Tekamah Dv. Ditch		665.0		124	
Soldier River	Pisgah, IA	664.0		407	Yes
Old Soldier R.Ditch		649.3		100	
Fish Creek		647.9		124	
Boyer River	Logan, IA	635.2		871	Yes
Pigeon Creek		622.0		166	
Unknown	Ungaged			382	
Percent Ungaged	5.8%				
Missouri River	**Omaha, NE**	615.9	322,800		
Mosquito Cr		605.8		238	
Big Papillion Cr	Fort Crook	596.6		384	
Platte River	Ashland-Louis.	594.8		85,370	Yes
Watkins Ditch		587.5		185	
Weeping Water Cr	Union	568.7		241	Yes
Unknown				782	
Percent Ungaged	0.9%				
Missouri River	**Nebraska City**	562.6	410,000		
Nishnabotna River	Hamburg	542.1		2,806	Yes
Little Nemaha River	Auburn	527.8		793	Yes

Stream	Station	Missouri River Mile	Missouri Drainage Area (sq. mi)	Tributary Drainage Area (sq. mi)	Sediment Model Input
Rock Creek		522.2		104	
Tarkio River	Fairfax, MO	507.6		520	
Unknown				677	
Percent Ungaged	13.8%				
Missouri River	**Rulo**	498.0	414,900		

3.4 QUASI-UNSTEADY FLOW AND TEMPERATURE DATA

The Quasi-unsteady flow application was selected for use with sediment modeling for this exercise instead of full unsteady. Quasi-unsteady flow allows the user to enter a time series of flow but the model does not alter the flow with computational routing such as would occur in full unsteady. Thus, channel and reservoir storage are not modeled with Quasi-unsteady flow. Each inflow is entered into the Quasi Unsteady flow editor within HEC-RAS as shown in Figure 1.

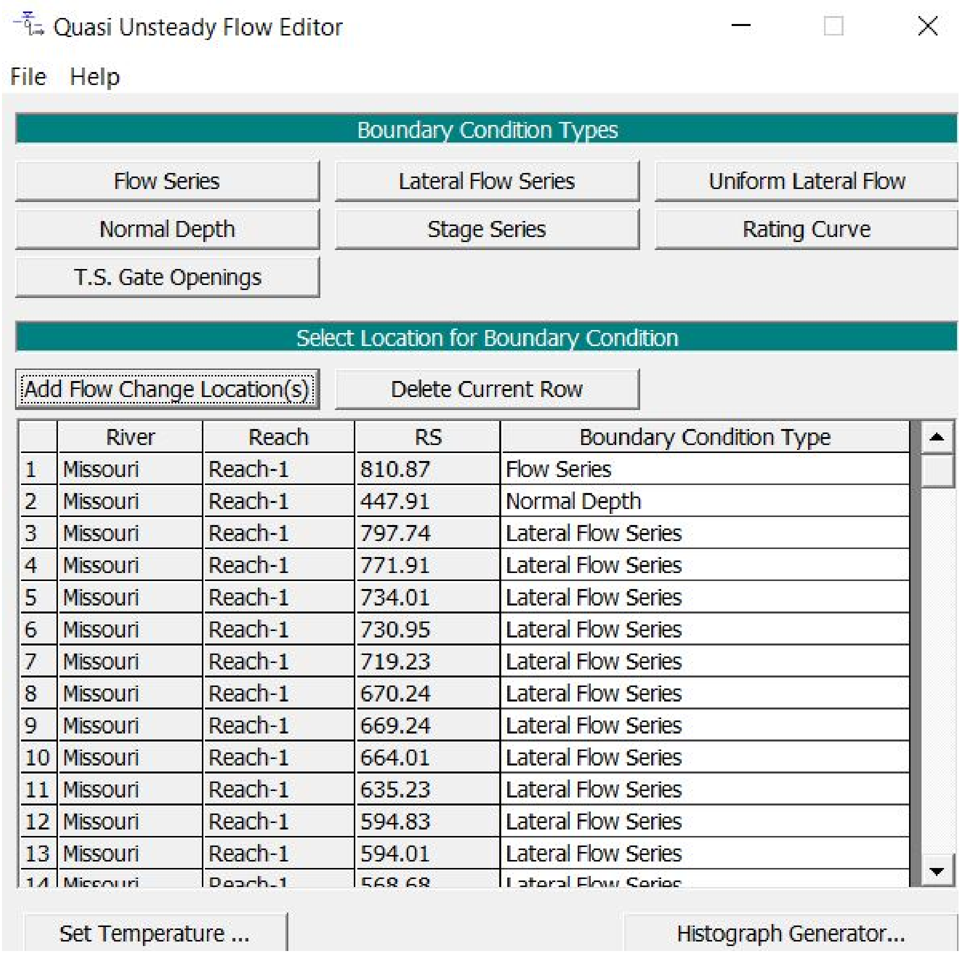


Figure 1. Quasi-unsteady Flow Input

Within the Quasi-Unsteady flow editor, the user defines the upstream cross section which is entered with a flow series. The furthest downstream cross section requires a downstream boundary which was set at normal depth using a slope of 0.0002.

For this modeling application, all flow data was entered with a 24 hour duration and a computation increment of 12 hours. An example HEC-RAS input screen and hydrograph is shown in Figure 2.

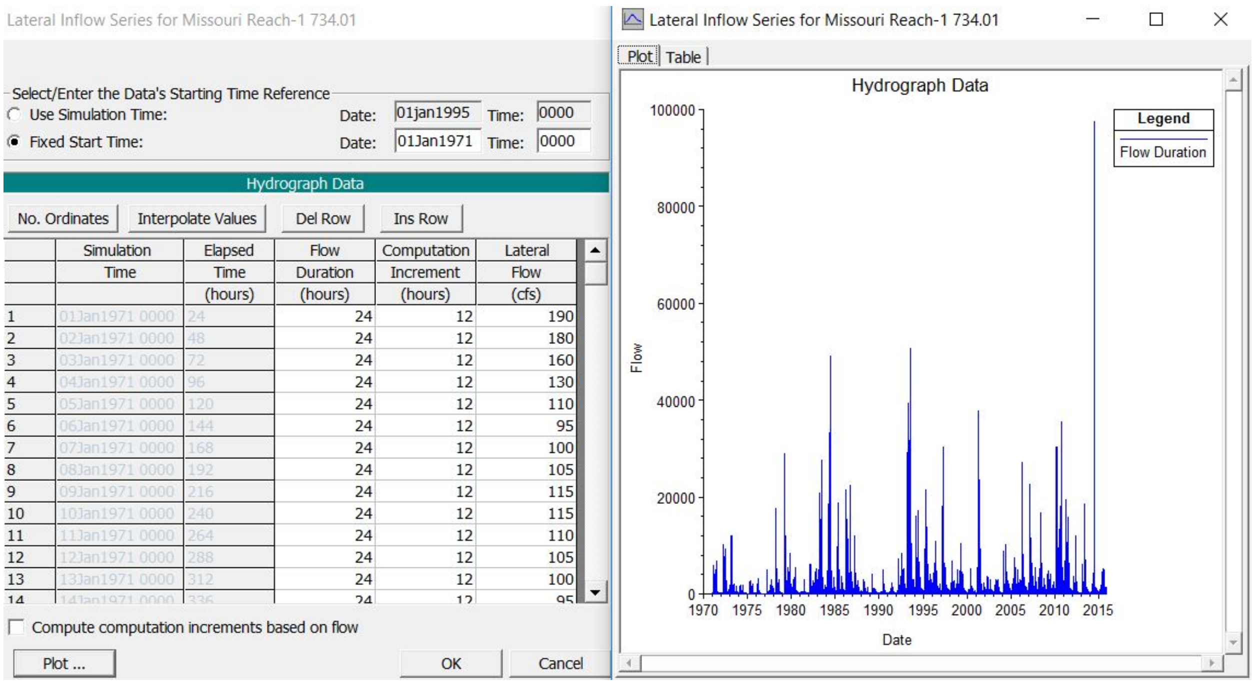


Figure 2. Lateral Inflow Series with HEC-RAS Entry

Within the editor, the user also enters temperature data. Water temperature was based on the monthly average values developed from the USGS gage data at Omaha, NE. Values that were input to the HEC-RAS model are shown in Table 2.

Table 2. Monthly Temperature Values Used in HEC-RAS

Temp (°F)	Month
34.3	Jan
35.6	Feb
41.2	Mar
50.6	Apr
61.9	May
72	Jun
78.5	Jul
77.5	Aug
70.3	Sep
57.5	Oct
44.8	Nov
35.8	Dec

Average temperature values developed from Omaha daily gage data (Oct 1971-Dec 2016)

Within HEC-RAS, all temperature data was entered as average monthly values as shown in Figure 3. Values were repeated on an annual cycle for the modeling simulation period.

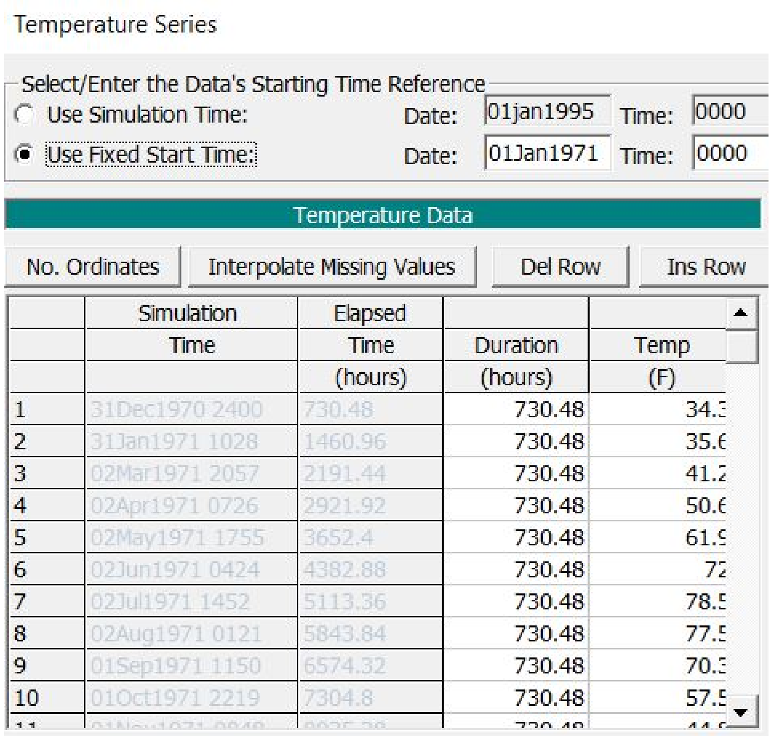


Figure 3. Quasi Unsteady Temperature Data

3.5 BANK INPUT, GAVINS POINT DAM TO PONCA

Large scale bank movement and sediment input has been observed within the Gavins Point Dam (RM 811) to Ponca (RM 751) reach along the Missouri River. This area is upstream of the navigation channel and within the degradation reach downstream of Gavins Point Dam.

3.5.1 Previous Studies

Observed bank erosions rates, as measured from aerial photography and other sources that were tabulated in the study *Missouri River Gavins Point Dam Degradation Trends Study, MRB Sediment Memorandum 24A* (USACE 2014) are shown in Table 3.

Table 3. Gavins to Ponca Bank Erosion Rates (Table 10-1, USACE 2014)

Status	Period	Acres Lost	Number of Years	Acres Lost / Year	Cumulative Acres Lost	Cumulative Acres Lost / Year	Volume (Cu ft/yr)[2]	Mass (tons/yr)[3]
Pre-Dam	1930 – 1946	3,062.4	16	191.4	3,062.4	191.4	1.33E+09	6.34E+07
	1946 – 1956	2,213.6	10	221.4	5,276.0	202.9	9.64E+08	4.58E+07
Post-Dam[1]	1956 – 1959	237.8	3	79.3	5,513.8	190.1	1.04E+08	4.92E+06
	1959 – 1969	1,661.9	10	166.2	7,175.7	184.0	7.24E+08	3.44E+07
	1969 – 1972	783.5	3	261.2	7,959.2	189.5	3.41E+08	1.62E+07
	1972 – 1974	451.4	2	225.7	8,410.6	191.2	1.97E+08	9.34E+06
	1974 – 1975	277.1	1	277.1	8,687.7	193.1	1.21E+08	5.73E+06
	1975 – 1979	404.3	4	101.1	9,092.0	185.6	1.76E+08	8.37E+06
	1979 – 1985	474.3	6	79.1	9,566.3	173.9	2.07E+08	9.81E+06
	1985 – 1995	559.9	10	56.0	10,126.2	155.8	2.44E+08	1.16E+07
	1995 – 1997	549.4	2.0	274.7	10,675.6	159.3	2.39E+08	1.14E+07
	1998 – 2003	475.9	5.0	95.2	11,151.5	154.9	2.07E+08	9.85E+06
	2003 – 2006	116.0	2.8	41.4	11,267.5	150.6	5.05E+07	2.40E+06
	2006 – 2008	117.5	2.5	47.0	11,385.0	147.3	5.12E+07	2.43E+06
	2008 – 2011	440.7	3.1	142.2	11,825.7	147.1	1.92E+08	9.12E+06

1 Does not include data from 1997 – 1998.

2 Column is an addition to the Degradation Report Table 10-1, the tabulated volume provides a gross approximation that is estimated using an average 10 foot bank height for the entire reach. Volume estimate includes all grain size classes (silts and clays, also sands)

3 Mass computed from volume using an average sediment unit weight of 95 lbs/cu ft

3.5.2 BSTEM Analysis

The bank erosion rates shown in Table 3 illustrate a high correlation with flow. In order to assess potential variation in bank erosion rates as a result of change in flow releases from study alternatives, a bank erosion model was assembled for evaluation. The Bank-Stability and Toe Erosion Model (BSTEM) developed by the National Sediment Laboratory of the USDA's Agricultural Research Station is a physically based model that accounts for the dominant stream bank processes that was implemented within HEC-RAS. The flows for the various alternatives were simulated with the model and bank erosion rates were compared. The BSTEM evaluation is summarized in Attachment 1.

Although the BSTEM model calibration started in the 1960's, only the more recent bank erosion rate from the post 1980 period was selected as representative of future bank erosion rates. The 1960 to 1980 period exhibits an accelerated bank erosion rate that is affected by the initial response to Gavins Point Dam closure in 1955. Annual bank erosion volume computed by BSTEM was fit to a curve for the post 1980 period of the following form:

Post-1980 Bank Loss Rate: $Y = m\,x$ + b

Y = Annual bank erosion (ft^3)

X = annual release volume from Gavins Point Dam (ft^3)

m = 2.39321e-05 (ft^3 sediment/ft^3 water)

b = -9.56485e+06 ft^3 sediment

The annual volumes of BSTEM computed bank erosion for sand only were correlated to the Gavins Point Dam annual release flow volume for the two periods as shown in Figure 4.

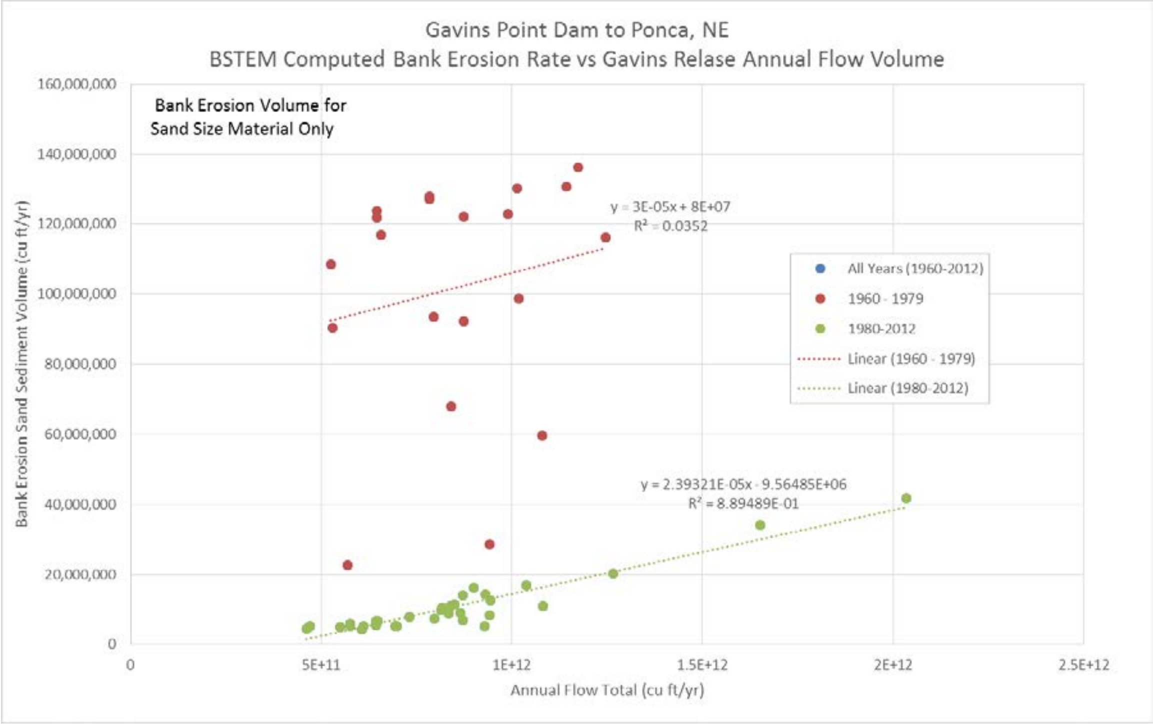


Figure 4. Annual Flow Volume and BSTEM Bank Erosion Volume

The BSTEM modeling results were compared to the bank erosion volumes previously shown from the Degradation Report study (USACE 2014) as shown in Table 4.

Table 4. BSTEM Bank Erosion Volume Comparison

Period	Degradation Report (USACE 2014) Acres Lost/Year	Volume[1] (cu ft/yr)	BSTEM Bank Erosion Volume (cu ft/yr)
1979 – 1985	79.1	34,455,960	15,295,516
1985 – 1995	56	24,393,600	8,175,155
1995 – 1997*	274.7	119,659,320	18,570,923
1998 – 2003	95.2	41,469,120	7,086,471
2003 – 2006	41.4	18,033,840	
2006 – 2008	47	20,473,200	
2008 – 2011	142.2	61,942,320	7,323,086
Average 1980-2011		52,315,560	11,195,740
Ratio BSTEM Vol/Deg. Report Vol			21%

1 Volume computed from acres using an assumed average bank height of 10 feet.

Table 4 illustrates that the BSTEM erosion rates are less than those measured from aerial photography in the *Gavins Point Dam Degradation Trends Study* (USACE 2014). However, the 10 foot average bank height used to determine volume may not be representative of the entire reach. In addition, BSTEM volumes are for sand only. Silts and clays would be included in the Degradation Report volume. The BSTEM calibration (Attachment 1) showed a good match to channel widening computed from repeat cross-section surveys. Discrepancies between these two method of calculating bank erosion volume are expected due to the different method. A significant portion the discrepancy may be due to 1) the assumption of an "average" bank height for computing volume from the imagery that does not capture the complexity of channel bank height along the reach; and 2) the simplification of the channel geometry that is inherent to cross section differencing (change is interpolated between cross sections),

Due to the difference between volumes, the bank erosion input volume was evaluated in a sensitivity analysis during sediment model calibration.

3.5.3 Bank Input to RAS Model

Results from the BSTEM analysis were used to derive a daily sediment bank input separated into three locations. This input was formulated from Gavins flow releases for both the calibration period and the POR simulation period. Sediment was input to the model using a daily flow and a rating curve. Data was input in this manner for ease of use and also to allow rapid sensitivity assessment of the effect of bank sediment input on the model.

In order to enter bank erosion volume within RAS as a daily sediment input, the above relationship was approximated with a uniform lateral inflow that was based on a flow ratio of the Gavins release and a rating curve. The flow ratio was set at 1/15,000 of the actual Gavins Point Dam release flow to avoid adding unnecessary inflow to the RAS model. The sediment input for each day was determined as a ratio of the daily flow to the annual flow. The derived bank relationship at each of the three inflow reaches is shown in Table 5.

Sediment grain size was based on the bank material samples reported in a previous study (Biedenharn et al, 2001, Table 5.20). Reported values included a D_{50} of 0.19 mm and D_{90} of 0.45 mm. Selected grain size was slightly smaller than those reported. Both sediment loads and grain size were adjusted during the calibration process. The sediment loads shown in Table 5 are input to the RAS model in combination with a uniform lateral inflow that was computed with the ratio of 1/15,000 of the actual Gavins Point Dam flow. At normal flow releases from Gavins Point Dam in the range of the maximum power plant capacity of 35,000 cfs, the flow input from each reach is about 2.3 cfs with a total of 7 cfs from all three reaches.

Table 5. Bank Erosion Load Input to RAS

Flow (cfs)	0	0.2	0.5	0.8	1	2	3
Load (tons/day)	0	1275	3000	12300	21000	35000	45000
VFS (0.0625 - 0.125 mm)	0.5	0.5	0.5	0.5	0.5	0.5	0.5
FS (0.125-0.25 mm)	0.4	0.4	0.4	0.4	0.4	0.4	0.4
MS (0.25-0.5 mm)	0.1	0.1	0.1	0.1	0.1	0.1	0.1

The uniform flow for each reach was further altered based on the distribution of the total bank erosion volume that was determined with the BSTEM model. The bank erosion volume distribution ratios determined with BSTEM are shown Table 6.

Table 6. Bank Erosion Volume Reach Sediment Distribution

Sediment Distribution Ratio From BSTEM 1980 - 2012	**Gavins to James River**	**James River To Vermillion River**	**Vermillion River to Ponca, NE**
	0.47	0.22	0.31

3.6 FLOODPLAIN SEDIMENT STORAGE

Previous studies and observations have indicated that sediment deposition occurred within the floodplain during the 2011 event. Unfortunately, no reliable relationship or quantification of sediment deposition volume was available. Therefore, an empirical relationship was derived for each during the calibration process to represent floodplain deposition of the form $Q_s = a\,Q^b$ (see Equation 1). During the 82 year POR, the empirical relationship results in a few years with sediment deposition in the pre-dam era (1930 – 1955). In the post-dam era after 1955, no other significant sediment deposition occurs in any year other than 2011.

Floodplain sediment deposition was based on an initial flow at which deposition initiates and then an exponential relationship for flows above the initial flow. Since the floodplain sediment removal was keyed to gage flow, some very minor variation occurs between alternatives. Floodplain deposition was derived using the Sioux City, Omaha, and Nebraska City flows as shown in Table 7.

Table 7. Floodplain Sediment Summary

	Sioux City		Omaha					Nebraska City	
River Mile	**745.19**	**725.71**	**700.2**	**680.36**	**655.29**	**640.73**	**610.3**	**553.89**	**540.93**
Slope a	5.0 e -6	5.0 e -6	5.0 e -6	5.0 e -6	5.0 e -6	5.0 e -6	5.0 e -6	9.0 e -6	9.0 e -6
Exp b	2	2	2	2	2	2	2	2	2
Flow Min (cfs)	80000	80000	110000	110000	110000	110000	110000	110000	110000
					Alternative 1				
Days above Min Flow[1]	300	300	149	149	149	149	149	385	385
Max Daily (tons/day)[2]	71,173	71,173	54,836	54,836	54,836	54,836	54,836	128,997	128,997
2011 Event (tons)[3]	3,428,095	3,428,095	2,065,574	2,065,574	2,065,574	2,065,574	2,065,574	5,416,129	5,416,129
Total Sediment (tons)[4]	3,660,930	3,660,930	2,153,633	2,153,633	2,153,633	2,153,633	2,153,633	7,922,015	7,922,015
					Alternative 2				
Days above Min Flow	309	309	177	177	177	177	177	437	437
Max Daily (tons/day)	63,746	63,746	47,429	47,429	47,429	47,429	47,429	109,183	109,183
2011 Event (tons)	3,304,195	3,304,195	1,924,037	1,924,037	1,924,037	1,924,037	1,924,037	5,075,743	5,075,743
Total Sediment (tons)	3,644,216	3,644,216	2,005,980	2,005,980	2,005,980	2,005,980	2,005,980	7,620,677	7,620,677
					Alternative 3				
Days above Min Flow	302	302	150	150	150	150	150	387	387
Max Daily (tons/day)	70,585	70,585	54,357	54,357	54,357	54,357	54,357	127,989	127,989
2011 Event (tons)	3,479,111	3,479,111	2,096,508	2,096,508	2,096,508	2,096,508	2,096,508	5,478,230	5,478,230
Total Sediment (tons)	3,712,816	3,712,816	2,183,481	2,183,481	2,183,481	2,183,481	2,183,481	7,987,280	7,987,280
					Alternative 4				
Days above Min Flow	276	276	149	149	149	149	149	361	361
Max Daily (tons/day)	72,327	72,327	55,899	55,899	55,899	55,899	55,899	127,319	127,319
2011 Event (tons)	3,329,577	3,329,577	1,990,761	1,990,761	1,990,761	1,990,761	1,990,761	5,248,646	5,248,646
Total Sediment (tons)	3,558,234	3,558,234	2,078,112	2,078,112	2,078,112	2,078,112	2,078,112	7,498,478	7,498,478
					Alternative 5				
Days above Min Flow	294	294	148	148	148	148	148	365	365
Max Daily (tons/day)	68,234	68,234	52,198	52,198	52,198	52,198	52,198	121,405	121,405
2011 Event (tons)	3,277,628	3,277,628	1,954,721	1,954,721	1,954,721	1,954,721	1,954,721	5,171,772	5,171,772
Total Sediment (tons)	3,507,283	3,507,283	2,037,816	2,037,816	2,037,816	2,037,816	2,037,816	7,632,555	7,632,555
					Alternative 6				
Days above Min Flow	293	293	153	153	153	153	153	389	389
Max Daily (tons/day)	71,775	71,775	55,373	55,373	55,373	55,373	55,373	126,190	126,190
2011 Event (tons)	3,507,137	3,507,137	2,118,385	2,118,385	2,118,385	2,118,385	2,118,385	5,527,364	5,527,364
Total Sediment (tons)	3,737,152	3,737,152	2,206,960	2,206,960	2,206,960	2,206,960	2,206,960	7,976,063	7,976,063

1 Days above minimum flow threshold, no floodplain sediment deposition occurs until the reference gage exceeds this flow.

2 Maximum daily sediment removal from Missouri River to the floodplain during the entire POR (1930-2012) in tons/day

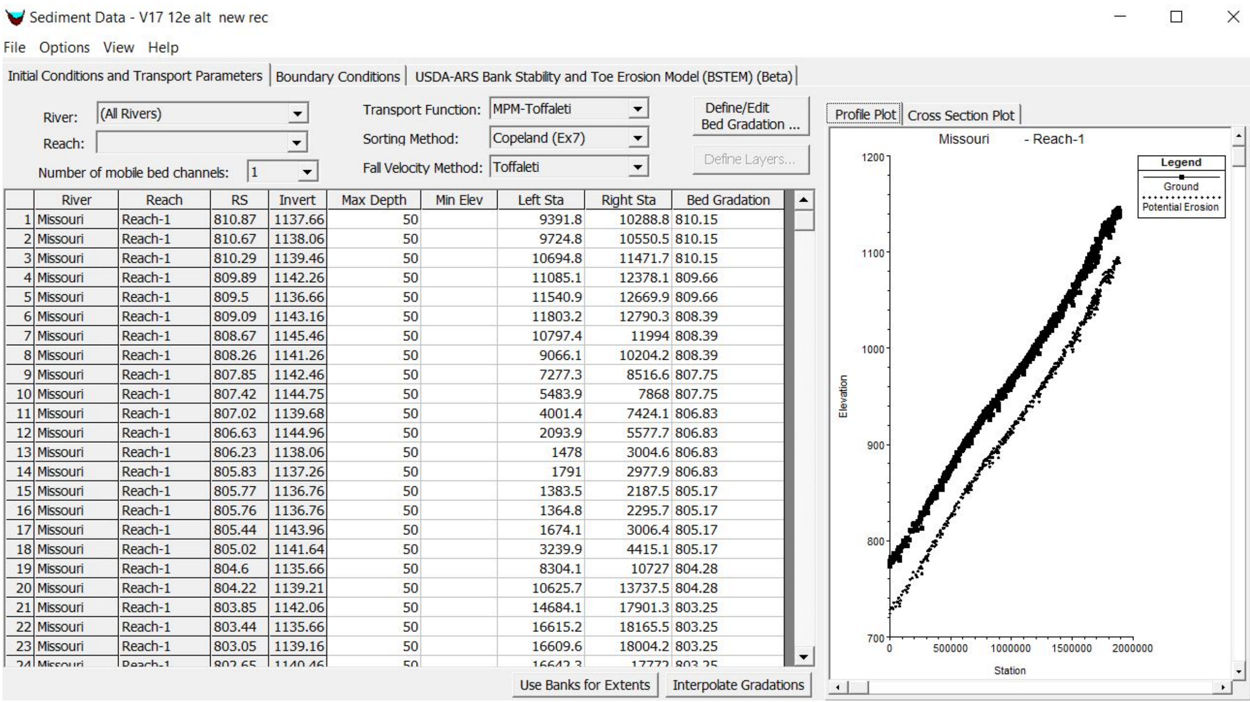
3 Total sediment removed from Missouri River to the floodplain for 2011, tons/day

4 Total sediment removed from Missouri River to the floodplain during the entire POR (1930-2012) in tons

The daily sediment removal was input to the RAS model as a sediment time series consisting of tons/day removed from the river for each day during the POR (1930-2012). A separate time series was entered at river mile location shown in Table 7.

3.7 Sediment Modeling Parameters

Sediment transport modeling parameters are specified within the sediment boundary condition editor. The model used the Meyer-Peter Muller - Toffaleti sediment transport equation and the Copeland (Exner 7) bed mixing algorithm. These algorithms were selected because of the larger bed material size and basin slopes. The Meyer-Peter Muller has a wide applicability for larger bed material and the Toffaleti function has applicability for suspended sediment transport with sand sized material. The Copeland mixing method was selected because it often performs better than the alternatives for large sand bed rivers, and simulates erosion better in these systems (alternative mixing methods tend to over-predict armoring and, therefore, under predict erosion). Typical input within the RAS model is shown in Figure 5.



	River	Reach	RS	Invert	Max Depth	Min Elev	Left Sta	Right Sta	Bed Gradation
1	Missouri	Reach-1	810.87	1137.66	50		9391.8	10288.8	810.15
2	Missouri	Reach-1	810.67	1138.06	50		9724.8	10550.5	810.15
3	Missouri	Reach-1	810.29	1139.46	50		10694.8	11471.7	810.15
4	Missouri	Reach-1	809.89	1142.26	50		11085.1	12378.1	809.66
5	Missouri	Reach-1	809.5	1136.66	50		11540.9	12669.9	809.66
6	Missouri	Reach-1	809.09	1143.16	50		11803.2	12790.3	808.39
7	Missouri	Reach-1	808.67	1145.46	50		10797.4	11994	808.39
8	Missouri	Reach-1	808.26	1141.26	50		9066.1	10204.2	808.39
9	Missouri	Reach-1	807.85	1142.46	50		7277.3	8516.6	807.75
10	Missouri	Reach-1	807.42	1144.75	50		5483.9	7868	807.75
11	Missouri	Reach-1	807.02	1139.68	50		4001.4	7424.1	806.83
12	Missouri	Reach-1	806.63	1144.96	50		2093.9	5577.7	806.83
13	Missouri	Reach-1	806.23	1138.06	50		1478	3004.6	806.83
14	Missouri	Reach-1	805.83	1137.26	50		1791	2977.9	806.83
15	Missouri	Reach-1	805.77	1136.76	50		1383.5	2187.5	805.17
16	Missouri	Reach-1	805.76	1136.76	50		1364.8	2295.7	805.17
17	Missouri	Reach-1	805.44	1143.96	50		1674.1	3006.4	805.17
18	Missouri	Reach-1	805.02	1141.64	50		3239.9	4415.1	805.17
19	Missouri	Reach-1	804.6	1135.66	50		8304.1	10727	804.28
20	Missouri	Reach-1	804.22	1139.21	50		10625.7	13737.5	804.28
21	Missouri	Reach-1	803.85	1142.06	50		14684.1	17901.3	803.25
22	Missouri	Reach-1	803.44	1135.66	50		16615.2	18165.5	803.25
23	Missouri	Reach-1	803.05	1139.16	50		16609.6	18004.2	803.25

Figure 5. HEC-RAS Sediment Input Table

The model also requires the specification of sediment properties. Selected values are shown in Figure 6

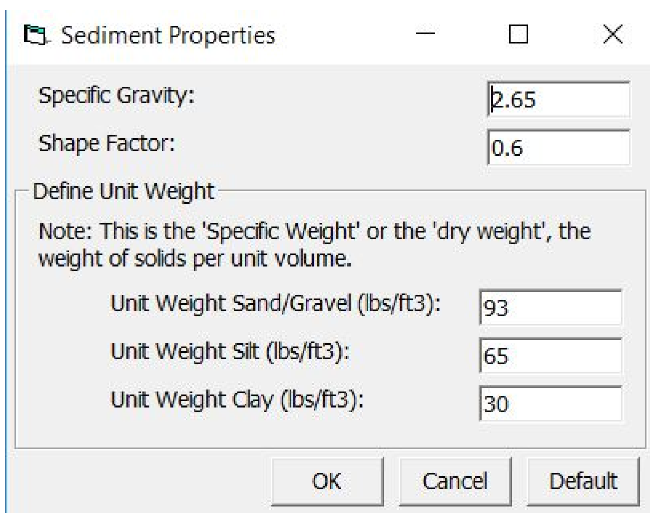


Figure 6. Sediment Properties

3.8 MOVABLE BED LIMITS

Movable bed limits are used within the model to define how the cross section station / elevation points are adjusted in response to computed mass erosion or deposition. For the Gavins model, the movable bed limits were set at the channel banks. HEC-RAS has different options that may be specified within the sediment modeling window. For the Gavins model, the option was selected as no bed change allowed outside of the movable bed limits. A typical cross section with the movable bed limits illustrated is shown in Figure 7.

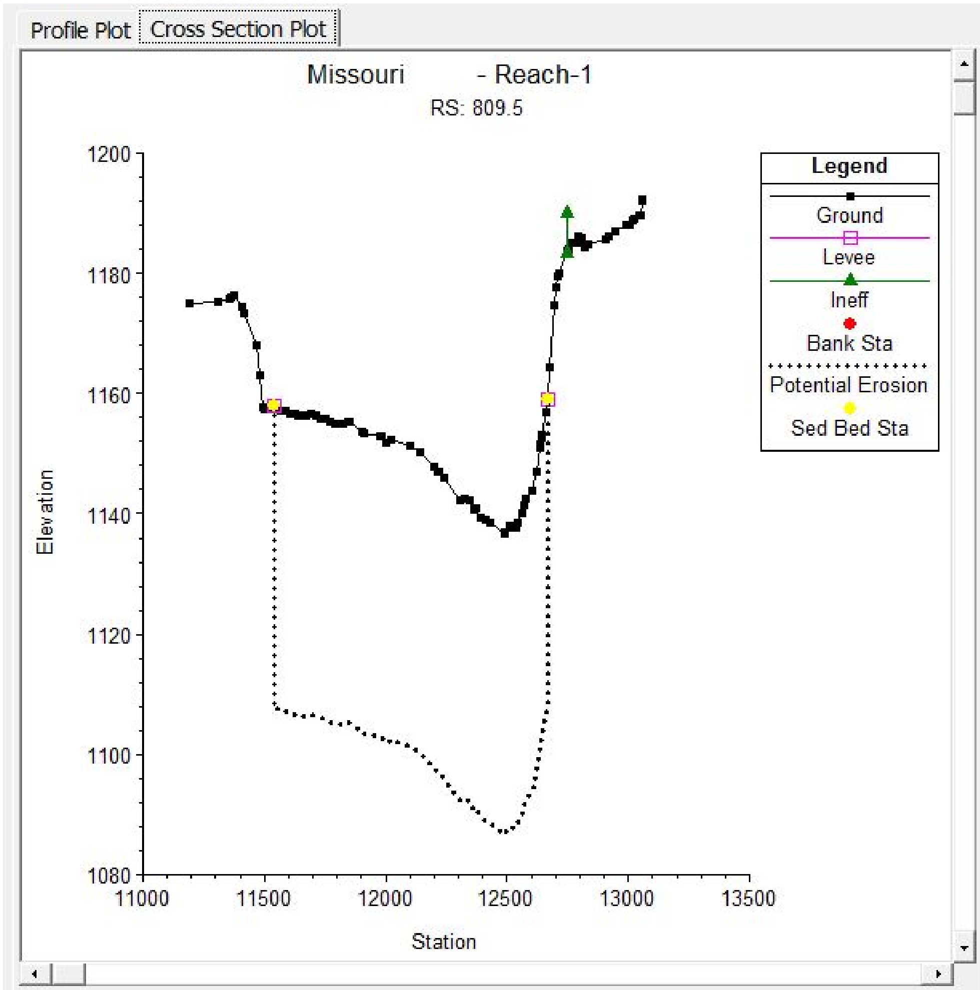


Figure 7. Moveable Bed Limits at a Cross Section

3.9 Bed Gradation Data

Bed gradation data is input by specifying at each individual cross section. Cross section density within the HEC-RAS model is much greater than the bed material data. Therefore, adjacent sections used the same bed gradation data as best available. A portion of the HEC-RAS gradation input table is shown in Figure 5. Bed gradation data within the model was initially based on the field measurements. Smoothing of bed gradation data was performed to reduce field data inconsistencies. Bed gradation data was also refined during the calibration process. An example bed gradation curve is shown in Figure 8.

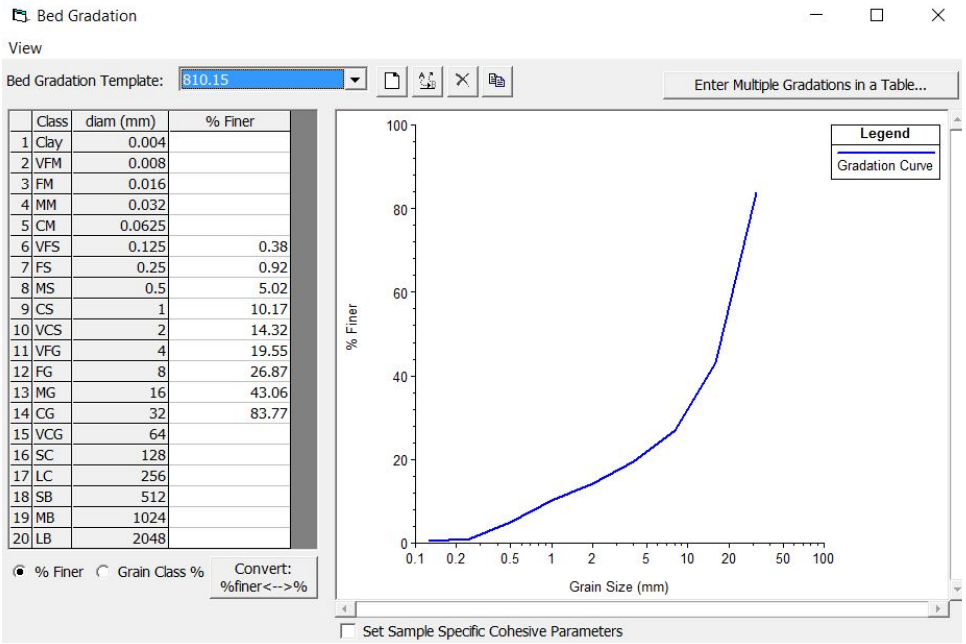


Figure 8. Bed Gradation Input to HEC-RAS

3.10 Sediment Boundary Condition: Flow-Load Rating Curve

Sediment models require sediment load boundary conditions at all locations where sediment inflow occurs. Locations include the upstream model boundary and each of the flow input locations for the major tributaries and uniform lateral inflows. Sediment input is specified for each computational time step in the simulation. Because sediment time series data are rare, sediment models often use flow-load rating curves to compute sediment load boundary conditions from the flow boundary conditions.

The sediment loading curve is typically expressed as:

$$Q_s = a \times Q^b \qquad \textbf{Equation 1}$$

Where

Q_s = Suspended sediment (tons/day)

Q = Discharge (ft^3/sec)

a = the intercept

b = the slope, exponent typically in the range between 1.5 and 2.5

The typical format of the sediment load relationship recognizes that the formulation of the sediment load power function as a linear model requires a logarithmic transformation to linearize the function and subsequently correct for subunity bias in the retransformation of sediment-discharge or –concentration estimates (Crawford 1991). The degree to which constituent discharges are underestimated as a result of retransformation is a function of the goodness-of-fit of the regression line. Generally, increasing the data scatter around the regression line results in decreasing estimates of the value of the dependent variable (Gray and Simões, 2008). Therefore, best practice for fitting a rating curve to flow-load data involves computing an "unbiased corrector" to account for these biases. The Duan (1983) "smearing factor" approach is a method often employed to yield an unbiased flow load rating curve relationship that conforms with theory and experience. However, for this modeling exercise, the tributary sediment load data was sparse and an unbiased correction factor was not necessary.

A direct relation between Q and Q_s in streams is rarely present. A lack of synchronization between the peaks of water discharge and sediment concentration over a flood hydrograph is more the rule than the exception. That means that in parts of the hydrograph where sediment discharge is increasing, sediment concentration may be decreasing, and vice versa. In some cases, a piece-wise relationship, with different flow-load relationships for high and moderate flows might be appropriate. Piece-wise flow-load relationships (sometimes called "bent rating curves") are common in sediment analysis, because the highest flows are often supply limited, delivering less sediment than the capacity predicts because less sediment is available.

Due to the scarcity of the data, employing an unbiased corrector or developing a refined sediment load piece-wise relationship was not feasible. For this purposes of this computational modeling exercise, the derivation of the sediment load rating curve at various tributary inflow points was performed as part of the calibration process.

For input to the HEC-RAS model, the Q_s relationship shown in Equation 1 was used to generate a rating curve for each input location. During the study, HEC released a bug report indicating that sediment loads were doubled during model simulation. All report tabulated values are the estimated tributary sediment loads. Report values were halved when entered in RAS.

The tributary input rating curves are illustrated in Table 8.

Table 8. Sediment Rating Curves to HEC-RAS Model by Location

Tributary	Flow (cfs)		Sediment Load (tons/day)[1]					
James	10	2,000	5,000	10,000	20,000	28,000		
XS 797.74	6	1,200	3,000	6,000	12,000	16,800		
Vermillion	1	100	2,000	6,000	14,000	21,000		
XS 771.91	1	6	206	766	2,112	3,430		
Big Sioux	100	2,000	5,000	15,000	28,000	60,000	98,000	
XS 734.01	10	460	1,472	6,038	13,460	35,838	67,310	
Floyd	100	2,000	5,000	10,000	17,000	25,000	33,000	
XS 730.95	8	2,240	12,866	46,712	125,342	256,838	430,476	
Monona-Harrison	100	400	2,000	5,000	10,000	30,000		
XS 670.24	6	72	1,000	4,500	14,000	80,000		
Little Sioux	100	1,000	5,000	10,000	15,000	23,000	30,000	
XS 669.24	1	82	1,680	6,100	13,200	29,300	48,200	
Soldier	1	100	4,000	9,000	15,000	23,000	30,000	
XS 664.01	0	3	1,408	6,372	16,496	36,556	59,946	
Boyer	100	1,000	5,000	10,000	15,000	20,000	26,000	
XS 635.23	4	600	17,600	77,416	183,900	339,762	594,716	
Platte	100	5,000	9,000	16,000	22,000	30,000	50,000	80,000
XS 594.83	8	8,000	19,600	60,000	112,000	190,000	380,000	700,000
Weeping Water	1	100	3,000	8,000	15,000	25,000	35,000	
XS 568.68	1	16	4,000	32000	84000	180,000	300,000	
Nishnabotna	200	8,000	17,000	27,000	40,000	54,000		
XS 542.11	4	15,580	80,982	222,702	440,000	800,000		
Little Nemaha	1	100	1,000	5,000	10,000	50,000		
XS 527.81	1	70	2000	16,000	37,500	160,000		

The plotted tributary sediment load curves are shown in Figure 9 and Figure 10. Sediment model runs were performed with HEC-RAS version 5.0.3. During the study, HEC released a bug report indicating that sediment loads were doubled during model simulation. All values shown in Table 8, Figure 9, and Figure 10 are the actual tributary sediment loads. Report values were halved when entered in RAS.

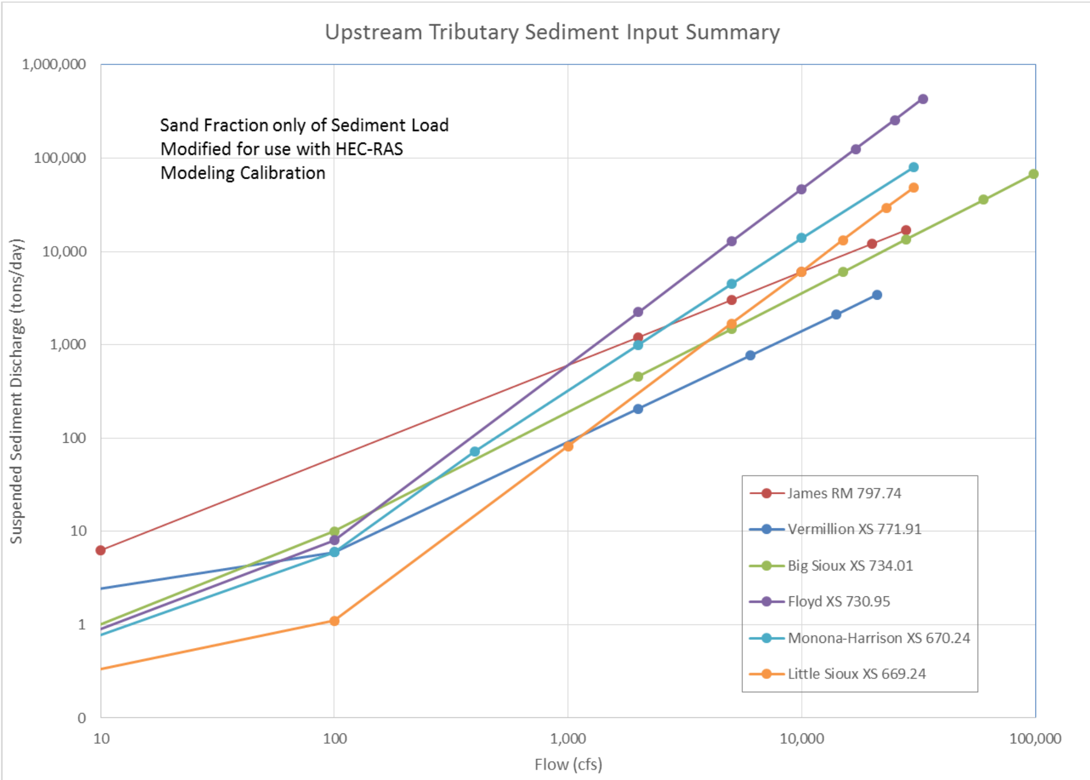


Figure 9. Upstream Tributary Sediment Load Rating Curves

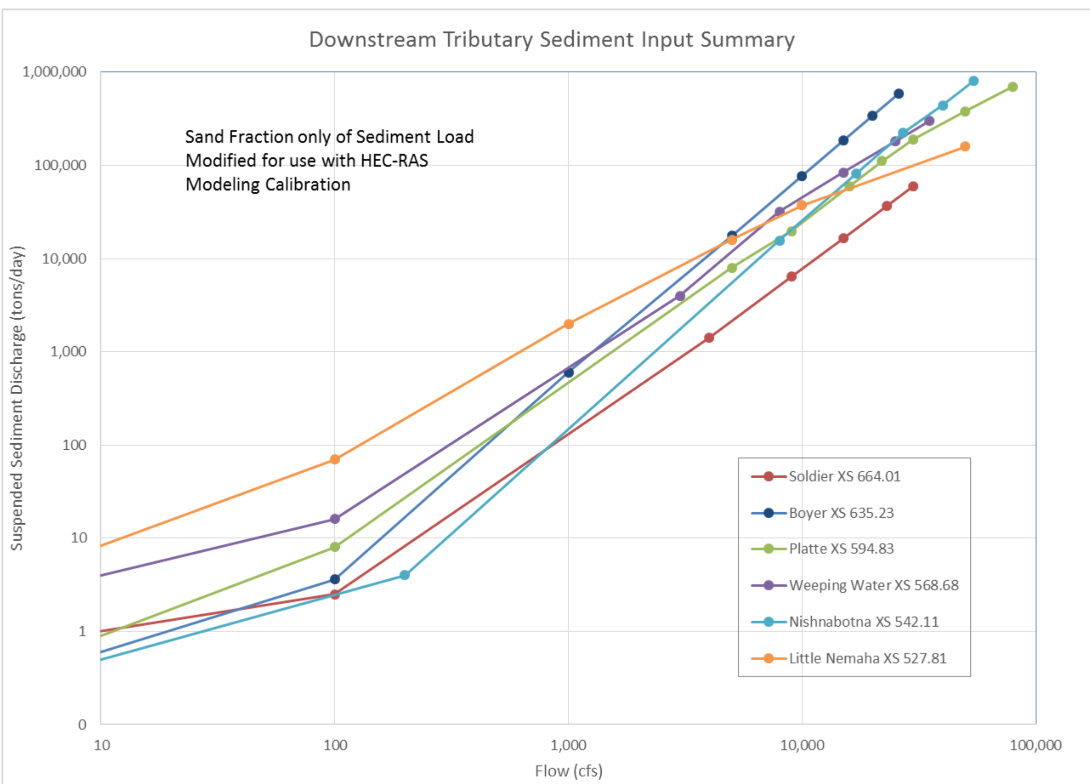
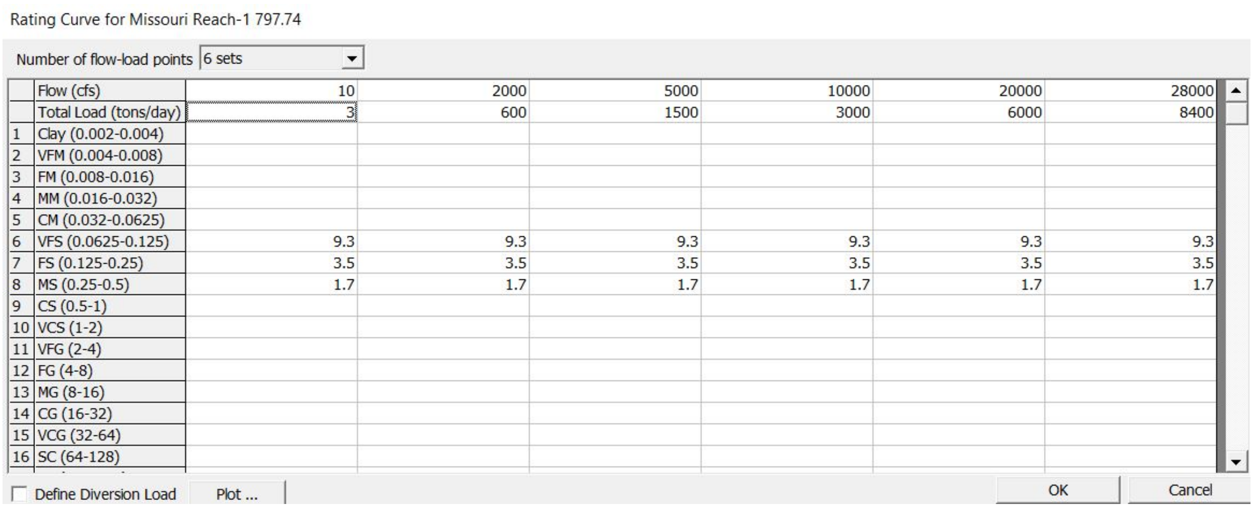


Figure 10. Downstream Tributary Sediment Load Rating Curves

3.11 SEDIMENT BOUNDARY CONDITION: LOAD GRADATIONS

Multiple grain size, sediment transport models do not just require upstream sediment load boundary conditions, but also require users to subdivide boundary loads by grain class. Additionally, load-gradation relationships can vary as a function of flow, sometimes in complicated or counterintuitive ways (Gibson and Cai, 2017). Sometimes the load coarsens at higher flows and sometimes it fines.

Very limited load gradation data was available for the tributary input locations. Due to the scarcity of data, the assumed load gradation was kept constant with flow. An example assumed gradation for a typical sediment input location is shown in Figure 11.


Rating Curve for Missouri Reach-1 797.74

Number of flow-load points 6 sets

	Flow (cfs)	10	2000	5000	10000	20000	28000
	Total Load (tons/day)	3	600	1500	3000	6000	8400
1	Clay (0.002-0.004)						
2	VFM (0.004-0.008)						
3	FM (0.008-0.016)						
4	MM (0.016-0.032)						
5	CM (0.032-0.0625)						
6	VFS (0.0625-0.125)	9.3	9.3	9.3	9.3	9.3	9.3
7	FS (0.125-0.25)	3.5	3.5	3.5	3.5	3.5	3.5
8	MS (0.25-0.5)	1.7	1.7	1.7	1.7	1.7	1.7
9	CS (0.5-1)						
10	VCS (1-2)						
11	VFG (2-4)						
12	FG (4-8)						
13	MG (8-16)						
14	CG (16-32)						
15	VCG (32-64)						
16	SC (64-128)						

Define Diversion Load Plot ... OK Cancel

Figure 11. Example Load Gradation at Tributary Inflow Location

Tributary load gradations generally consisted of very fine sand, fine sand, and medium sand. Sensitivity analysis on the tributary load gradation illustrated very little influence on results.

3.12 DATA GAPS AND DATA QUALITY

Data gaps for HEC-RAS sediment modeling include tributary sediment load and gradation data, historic river geometry, and locations of bedrock.

3.12.1 Tributary Sediment Load and Gradation.

Limited tributary sediment sampling information was available. In addition, no sediment gradation information was available for either the tributary bed material or suspended load.

3.12.2 Channel Geometry.

During model calibration and sediment simulation, it became apparent that historic surveys were limited in cross section. Volume change computations demonstrated the lack of quality information for the historic period.

3.12.3 Bedrock Controls.

Very few bedrock controls are known to exist along the Missouri River. However, bedrock outcrops may be present in some areas within the degradation reach that acts to limit channel degradation. No bedrock controls were used within the model.

4 HISTORICAL MODEL STEADY FLOW CALIBRATION

The historic model from 1995 was calibrated to steady flow conditions using available profile data and observed gage flows. Calibration was performed with steady flow conditions.

4.1 CALIBRATION FLOW

The calibration period focused on the mid-1990's to coincide with the 1995 geometry date. The most significant flow occurred in 1997. Gage data illustrates degradation at many locations as a result. The 2011 flow, the largest on record since mainstem dam construction, is also included for illustration purposes even though the 1995 model period is significantly prior to 2011. Flow profiles that were used from the mid1990's calibration period are presented in Figure 12.

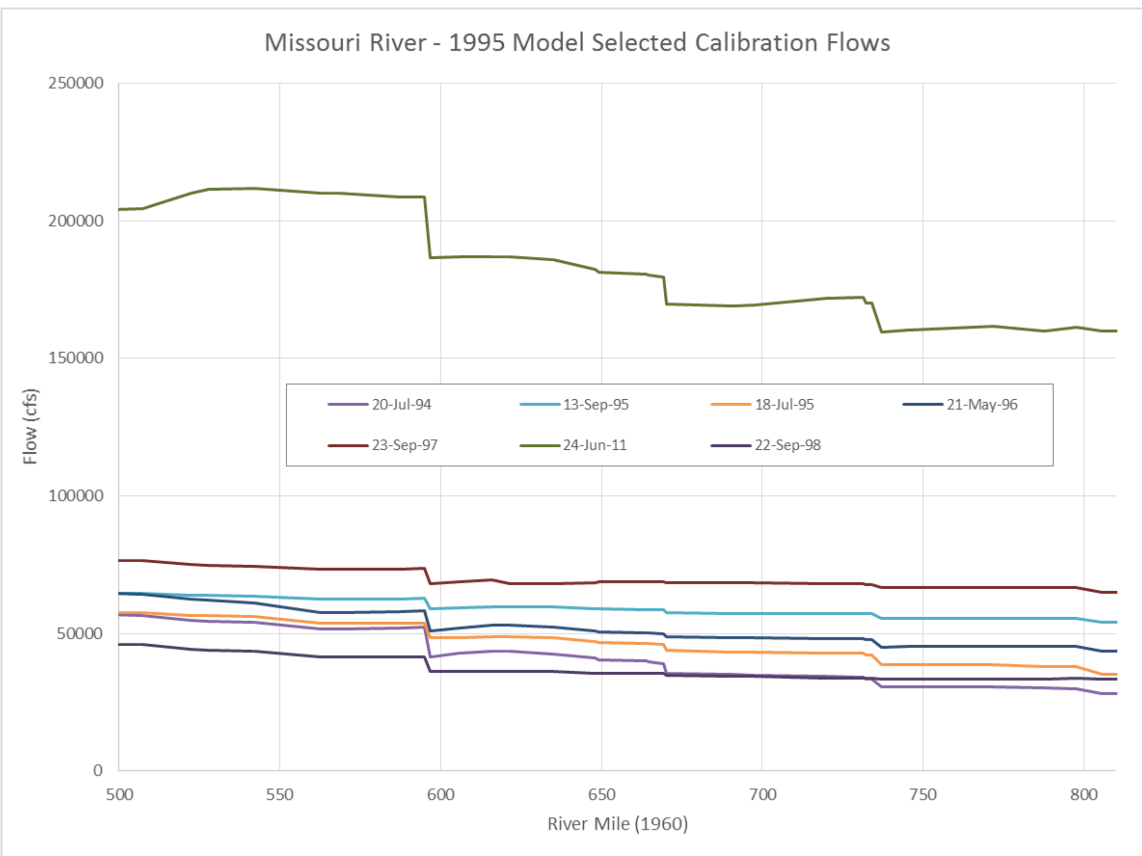


Figure 12. 1995 Model Selected Calibration Flows

4.2 CALIBRATION RESULTS

With the exception of the upstream 25 river miles (RM 811 to 785), computed water surface elevation was generally within 1 foot or less at all locations within the HEC-RAS model. An illustration of the model steady flow calibration is provided in Plate 1 through 10. The 2011 measured profile is included for reference only. Calibration to the 2011 profile using the 1995 model sections was not performed.

5 SEDIMENT SIMULATIONS

Sediment simulations were performed for the period from the historic model surveys in 1995 through the new channel surveys in 2012. Sediment parameters were adjusted based on volume change and water surface elevation change. The main calibration parameters were the sediment load for the Platte River, the bank erosion volume input and size, and the moveable bed limits and bed gradation size in the Gavins to Sioux City portion of the model.

Calibration examined volume change between surveys, water surface elevation at various periods computed with the sediment model, sediment concentration at the mainstem gage stations, and a sediment budget balance using gage station data. During the calibration process, it quickly became apparent that several factors limited model calibration accuracy:

- The steady flow water surface calibration indicated that the cross section data for the upper 25 river miles of the model were likely in error. This reduced sediment model calibration capability.
- Model computed bed erosion for the Gavins to Ponca reach, which includes sediment input for bank erosion volumes, is affected by the increasing channel width and the effect on hydraulics.
- Simultaneous calibration to sediment mass at the gage stations, volume change between surveyed cross sections, and water surface elevation during the calibration was not feasible. Volume change comparisons determined very large differences. However, correcting the model volume change would severely degrade the quality of both the sediment mass and water surface elevation results. For this study purposes, sediment model calibration to water surface was selected as the most critical parameter.
- Simultaneous calibration to parameters for pre- and post-2011 event were a challenge. Due to the unique nature of the 2011 event, the post-2011 event river showed significantly different sediment transport capability. Variation from pre- and post-2011 event flow vs. sediment load at Nebraska City is shown in Figure 13. Computed sediment load difference from the regression values determined with the annual gage data is shown in Table 9.

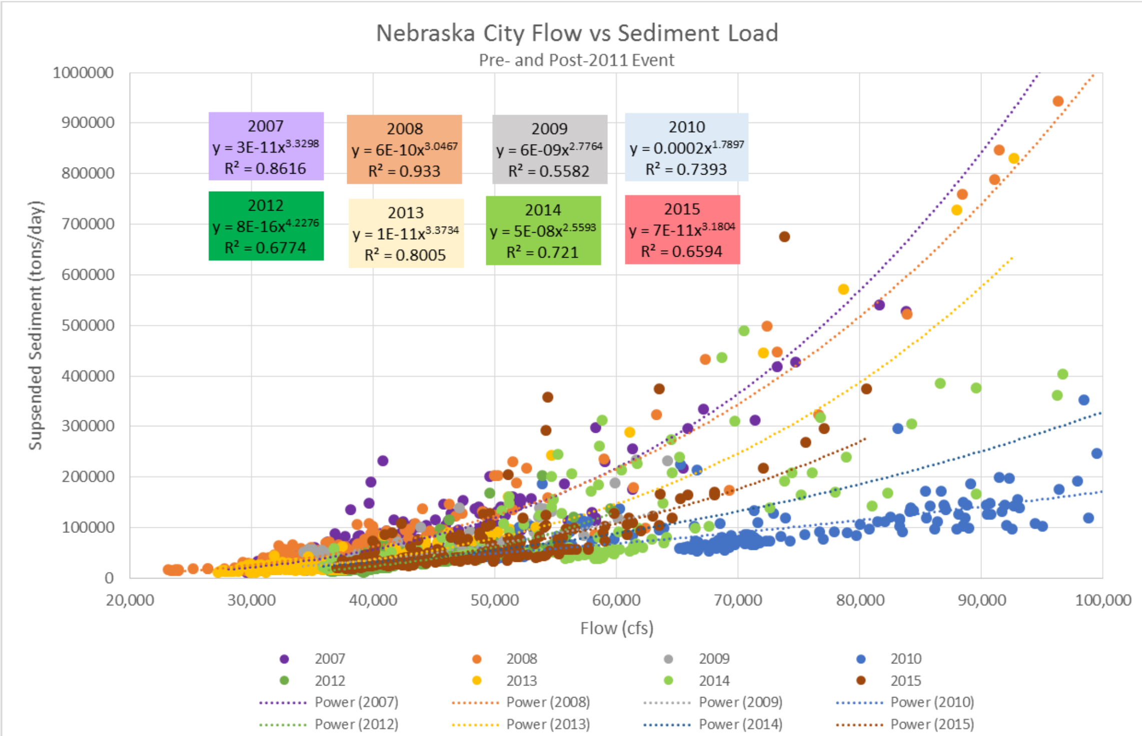


Figure 13. Nebraska City Flow vs. Sediment Load (pre- and post-2011 event)

Table 9. Sediment Load Difference Pre- and Post-2011 Event

	2007	2008	2009	2010	2012	2013	2014	2015
a	3.00E-11	6.00E-10	6.00E-09	2.00E-04	8.00E-16	1.00E-11	5.00E-08	7.00E-11
b	3.3298	3.0467	2.7764	1.7897	4.2276	3.3734	2.5593	3.1804
Q_w	35,000	35,000	35,000	35,000	35,000	35,000	35,000	35,000
Q_s	40,547	41,934	24,791	27,136	12,989	21,328	21,311	19,818
Q_w	60,000	60,000	60,000	60,000	60,000	60,000	60,000	60,000
Q_s	244,011	216,643	110,716	71,201	126,824	131,406	84,663	110,035

	2007-2010 Avg	2012-2015 Avg	% Change Sediment Load Change Post-2011
Q_w	35,000	35,000	
Qs	33,602	18,862	56%
Q_w	60,000	60,000	
Qs	160,643	113,232	70%

5.1 Water Surface Elevation

Water surface elevation data is collected two or three times annually on the Missouri River between Gavins Point Dam (RM 811) and Rulo, NE (RM 498). Data from these observed elevation profiles was compared to model water surface elevation at selected locations. This data was used to improve sediment model performance throughout the model reach.

Model simulations performed with the final calibration parameters and also several variables to illustrate sensitivity including transport function (MPM-Toffaleti vs. Laursen Copeland), fall velocity (Toffaleti vs. Report 12), tributary sediment load inflow (normal vs. 50% increase), and bed gradation (final calibration vs. original field data).

Calibration objectives were to obtain model accuracy within 1 foot of the observed data. Past studies have shown that steady flow calibration is usually feasible within 0.5 feet throughout the model reach. Field data collection from Gavins Point Dam to Rulo, NE, usually requires 3 days. Missouri River flow changes can occur during the data collection period. Profiles were selected to avoid significant periods of ungaged inflow. Referenced profile dates refer to the model profile output date. Observed data collection is within 3 or 4 days of the reported model profile date. Model computed water surface elevation represents the model results on that date after sediment simulation starting 1 January 1995. No corrections were made to model flow to match observed flows. Statistics were computed for both the average difference from observed and the sum of squares error from the observed. A summary of model results is provided in Table 10.

Table 10. Model Calibration to Water Surface Elevation Summary

	Steady Flow Initial Model Results		**Final Sediment Model Calibration**		**Sensitivity Fall Velocity Report 12**		**Sensitivity Ackers White Transport Function**		**Sensitivity Increase Trib Sed Load 50%**		**Sensitivity Original Bed Gradation**	
Profile	Avg. Error[1]	Sum Sq Error[1]	Avg. Error[1]	Sum Sq Error[1]	Avg. Error[1]	Sum Sq Error[1]	Avg. Error[1]	Sum Sq Error[1]	Avg. Error[1]	Sum Sq Error[1]	Avg. Error[1]	Sum Sq Error[1]
13 Sep 1995	-0.19	12.52	-0.37	16.22	-0.39	16.28	-0.39	16.71	-0.33	15.62	-0.46	19.35
27 August 2008			0.00	17.52	-0.18	22.35	-0.44	59.29	0.34	26.10	-0.34	42.61
9 Sep 2009			-0.35	19.73	-0.52	26.84	-0.77	63.14	0.00	24.46	-0.68	51.38
24 Jun 2011			0.27	38.01	0.14	39.17	-0.21	58.65	0.48	43.68	0.09	47.46
13 Sep 2012			0.33	18.49	0.08	13.41	0.33	18.49	0.76	34.04	-0.10	60.41
Average/Sum	NA	NA	-0.03	109.97	-0.17	118.06	-0.29	216.28	0.25	143.89	-0.30	221.20

1 All units based on feet

A summary of model results from the 13 September 2012 profile illustrating the river mile of model cross section locations used in the comparison, the observed water surface elevation, and the difference between the model and observed data is shown in Table 11.

Table 11. Comparison of Model Results to Observed Profile on 13 Sep 2012

Model Cross Section	Observed WSEL	Final Sediment Model Calibration		Sensitivity Fall Velocity Report 12		Sensitivity Ackers White Transport Function		Sensitivity Increase Trib Sed Load 50%		Sensitivity Original Bed Gradation	
805.77	1153.85	1153.75	-0.09	1153.30	-0.55	1153.75	-0.09	1154.00	0.16	1153.50	-0.34
796.3	1146.35	1146.57	0.22	1146.16	-0.19	1146.57	0.22	1146.78	0.43	1145.89	-0.46
788.96	1139.50	1139.83	0.33	1139.37	-0.13	1139.83	0.33	1139.92	0.42	1139.02	-0.48
778.7	1126.31	1127.49	1.17	1127.14	0.83	1127.49	1.17	1127.52	1.21	1126.90	0.59
773.35	1119.88	1121.17	1.29	1120.70	0.82	1121.17	1.29	1121.23	1.35	1120.60	0.72
768.34	1113.41	1115.40	1.99	1114.88	1.47	1115.40	1.99	1115.50	2.09	1115.18	1.77
759.96	1103.45	1104.41	0.96	1103.86	0.41	1104.41	0.96	1104.59	1.14	1104.59	1.13
753.52	1095.27	1095.89	0.62	1095.56	0.29	1095.89	0.62	1096.03	0.76	1096.40	1.12
739.85	1080.44	1080.23	-0.21	1079.79	-0.65	1080.23	-0.21	1080.53	0.09	1082.62	2.18
732.37	1071.38	1072.33	0.94	1071.78	0.40	1072.33	0.94	1072.59	1.20	1074.34	2.96
722.07	1062.48	1062.05	-0.43	1061.78	-0.69	1062.05	-0.43	1062.18	-0.30	1064.41	1.94
712.37	1054.31	1054.26	-0.05	1053.86	-0.45	1054.26	-0.05	1054.41	0.10	1055.42	1.12
702.68	1044.26	1044.32	0.07	1044.14	-0.12	1044.32	0.07	1044.47	0.21	1044.68	0.42
691.04	1033.88	1033.82	-0.06	1033.43	-0.45	1033.82	-0.06	1034.00	0.12	1034.15	0.27
681.21	1023.92	1024.16	0.24	1024.00	0.08	1024.16	0.24	1024.48	0.56	1023.54	-0.37
669.83	1013.17	1014.01	0.84	1013.55	0.38	1014.01	0.84	1014.32	1.16	1013.35	0.19
659.31	1003.63	1003.44	-0.19	1003.33	-0.29	1003.44	-0.19	1003.75	0.13	1002.64	-0.98
652.48	996.55	996.39	-0.16	996.43	-0.12	996.39	-0.16	996.73	0.17	995.21	-1.34
645.56	990.62	990.77	0.16	990.74	0.12	990.77	0.16	991.20	0.58	989.81	-0.81
640.34	986.99	986.20	-0.79	986.27	-0.72	986.20	-0.79	986.65	-0.34	985.16	-1.83
632.59	980.23	980.09	-0.14	979.97	-0.25	980.09	-0.14	980.59	0.37	979.25	-0.97
624.42	971.96	972.98	1.02	972.91	0.96	972.98	1.02	973.56	1.60	971.85	-0.11
616.04	964.49	965.67	1.18	965.32	0.83	965.67	1.18	966.54	2.05	963.91	-0.58
610.74	961.83	961.68	-0.14	961.48	-0.35	961.68	-0.14	962.73	0.91	959.63	-2.20
601.31	954.23	954.96	0.73	954.95	0.72	954.96	0.73	955.83	1.60	952.45	-1.79
591.55	945.89	946.14	0.25	945.77	-0.12	946.14	0.25	947.18	1.29	944.16	-1.73
578.52	932.37	932.69	0.32	932.79	0.42	932.69	0.32	933.64	1.27	931.36	-1.00
568.02	921.85	921.04	-0.81	920.91	-0.94	921.04	-0.81	922.33	0.48	919.46	-2.39
562.74	915.94	915.78	-0.16	915.46	-0.48	915.78	-0.16	916.86	0.92	913.88	-2.06
551.08	902.74	902.14	-0.60	901.95	-0.79	902.14	-0.60	902.91	0.17	901.48	-1.26
542.51	892.31	892.26	-0.06	892.07	-0.25	892.26	-0.06	892.99	0.67	892.24	-0.07
529.19	878.84	879.90	1.06	879.64	0.80	879.90	1.06	880.30	1.46	879.66	0.82
520.65	869.82	870.89	1.07	870.61	0.80	870.89	1.07	871.19	1.37	870.88	1.06
510.75	859.43	860.40	0.98	860.62	1.20	860.40	0.98	860.54	1.11	860.43	1.00
498.89	847.82	847.76	-0.06	847.79	-0.03	847.76	-0.06	847.77	-0.05	847.78	-0.04
Sum Squares Error			18.49		13.41		18.49		34.04		60.41
Average Error			0.33		0.08		0.33		0.76		-0.10

A comparison of the difference between the model water surface elevation and the observed elevation for all profile data is shown in

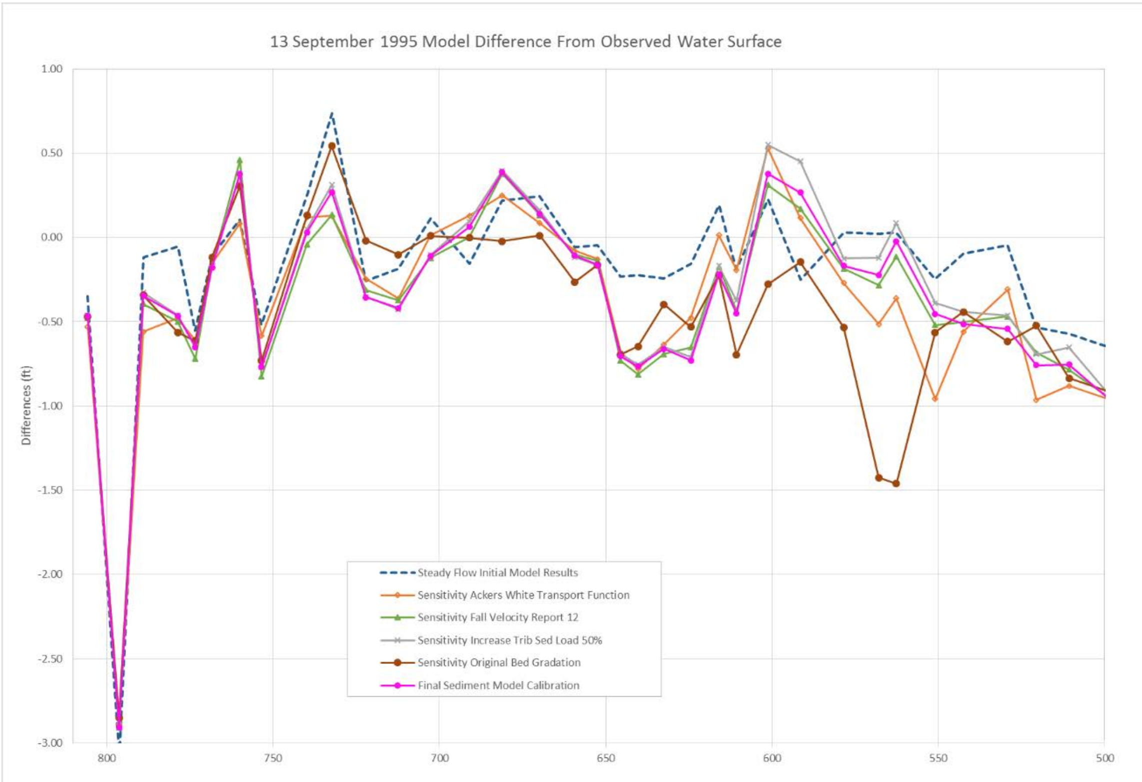


Figure 14. 13 September 1995 Model Difference From Observed Water Surface

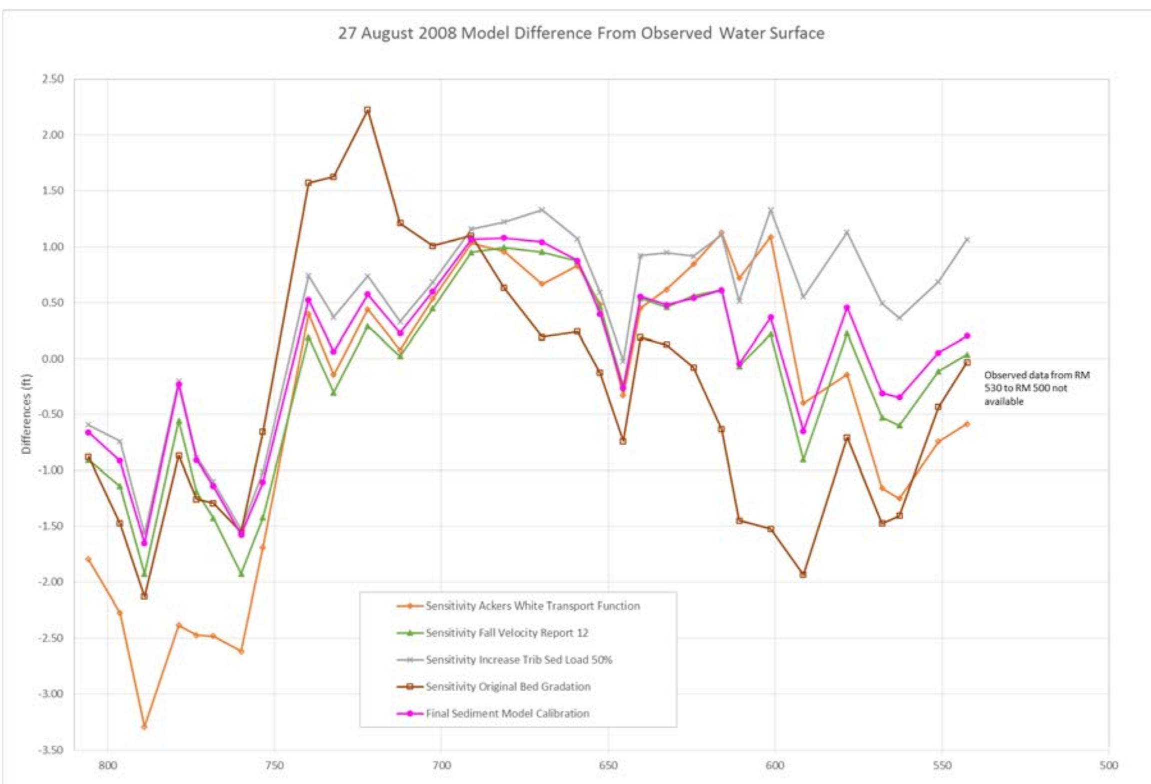


Figure 15. 27 August 2008 Model Difference From Observed Water Surface

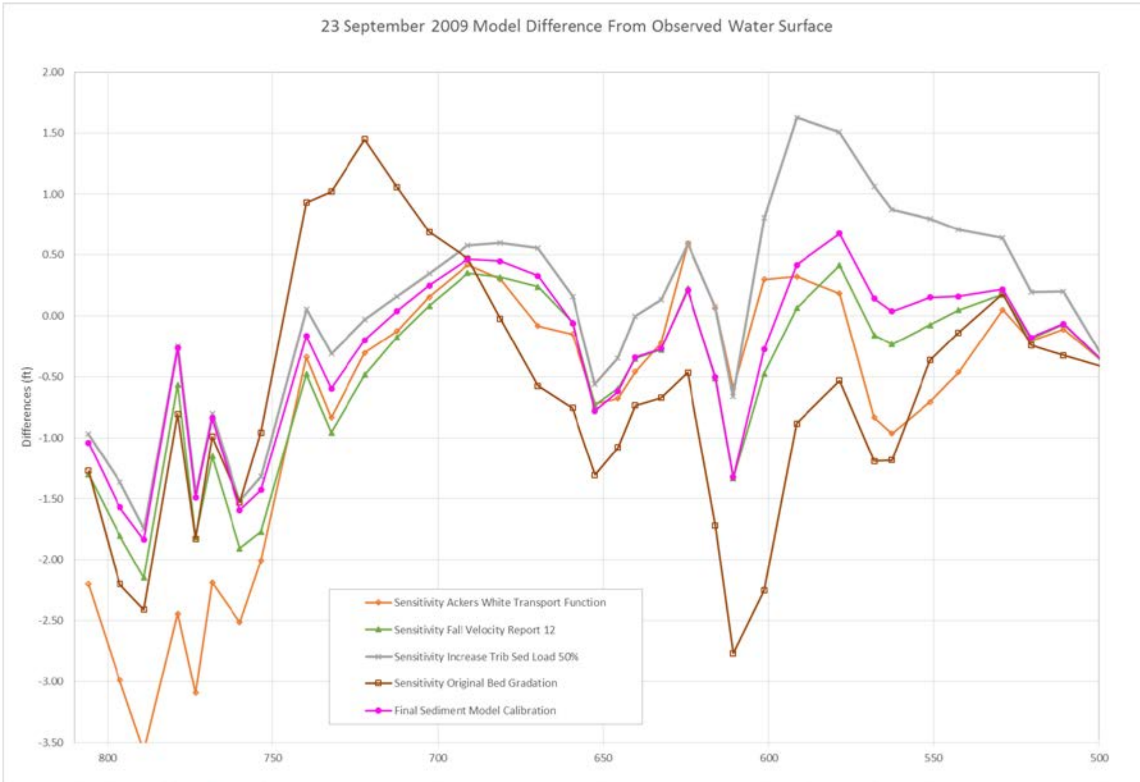


Figure 16. 23 September 2009 Model Difference From Observed Water Surface

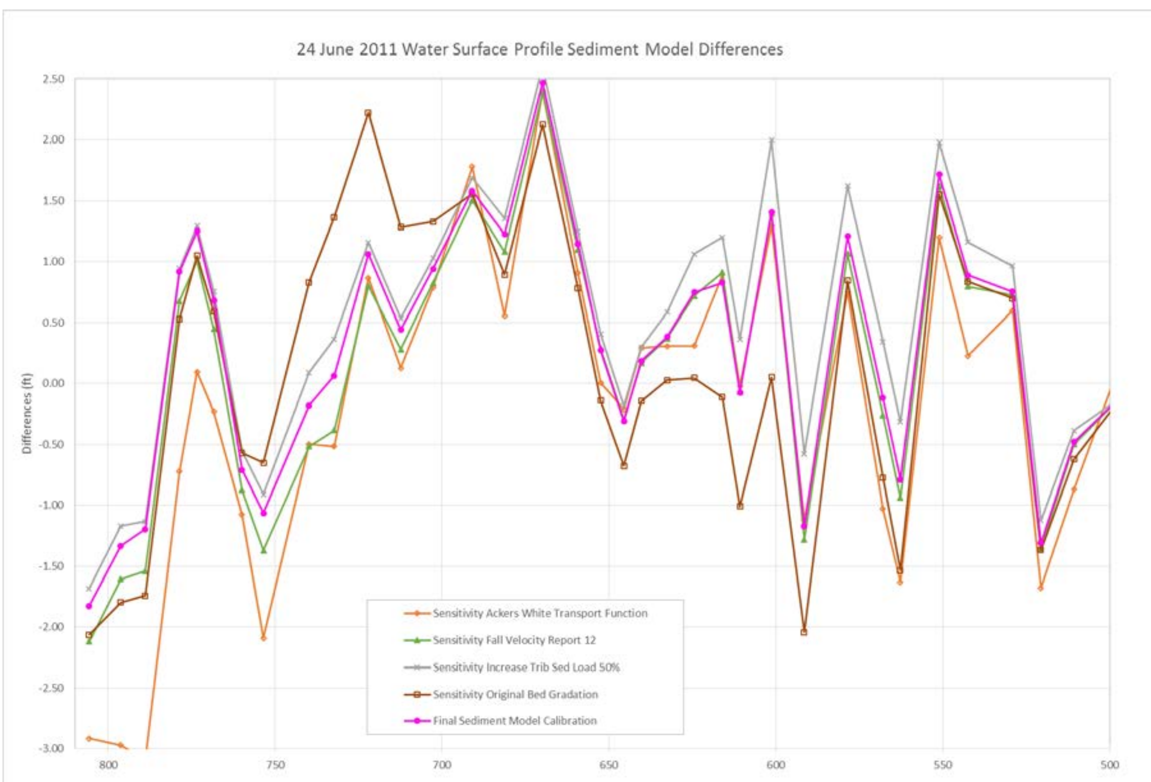


Figure 17. 24 June 2011 Model Difference From Observed Water Surface

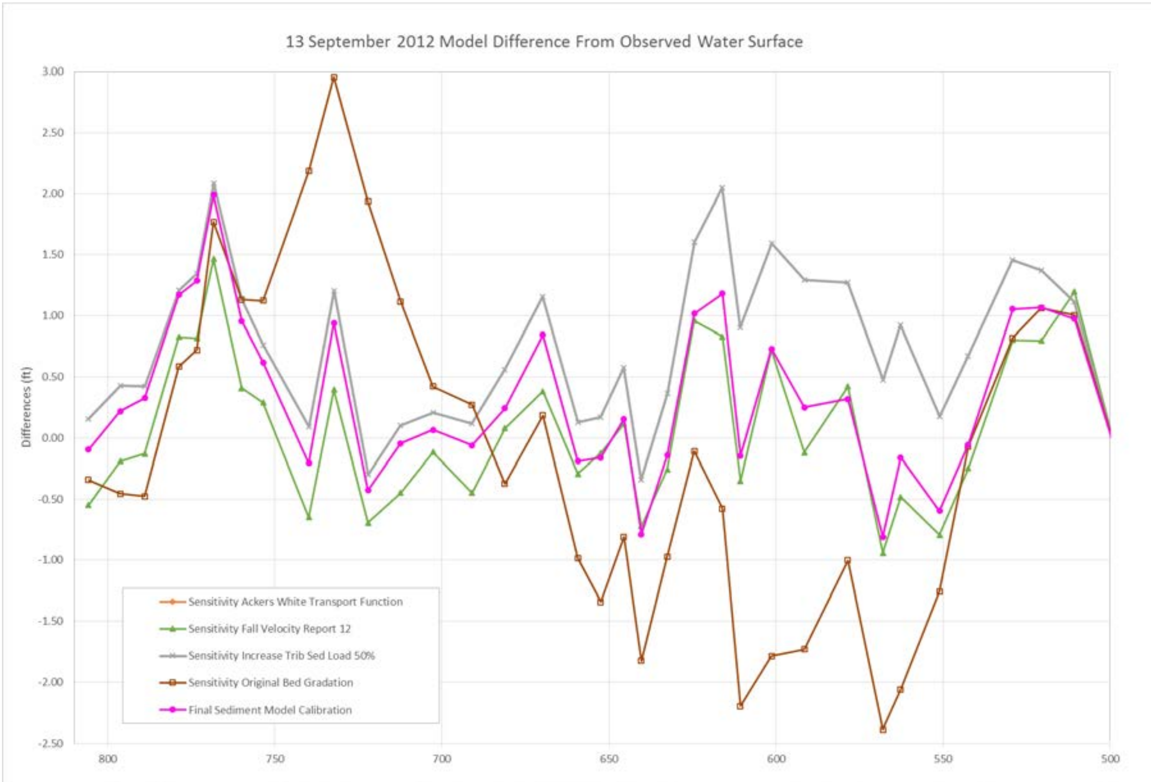


Figure 18. 13 September 2012 Model Difference From Observed Water Surface

5.2 CUMULATIVE MASS FROM USGS SUSPENDED SEDIMENT GAGE RECORDS

Observed estimation of the sand mass in the suspended load at Missouri River gage stations used information presented in the report *Generalized Sediment Budgets of the Lower Missouri River* (Heimann 2016). Variation in the sand fraction by flow occurs as shown in Figure 19.

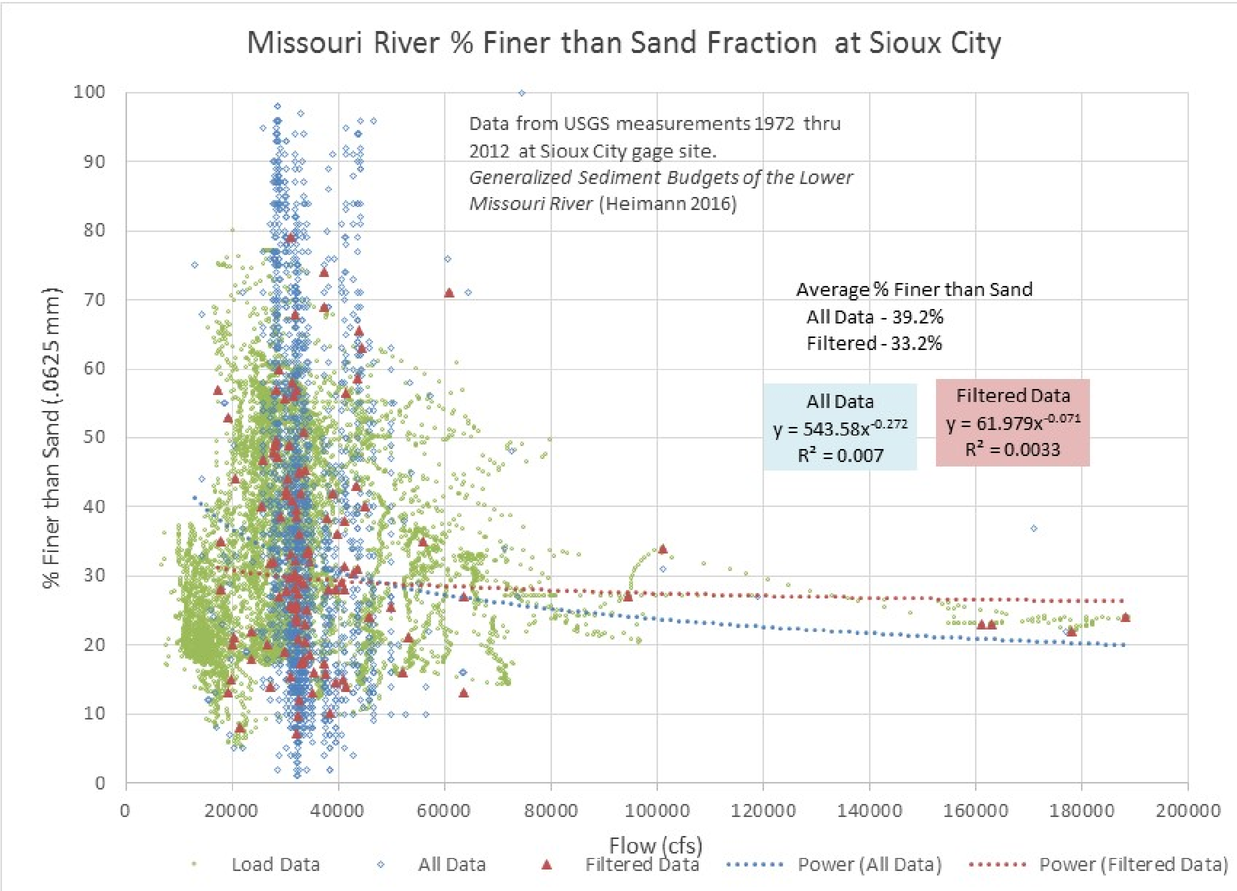


Figure 19. Missouri River % Finer than Sand at Sioux City

Model results were compared to the USGS gage data at Sioux City, Omaha, and Nebraska City. The results from the final calibration and sensitivity analysis HEC-RAS simulations were compared to the sand fraction USGS gage data at Sioux City, Omaha, and Nebraska City as shown in Figure 20, Figure 21, and Figure 22. In general, the mass calibration at the three gage stations was acceptable.

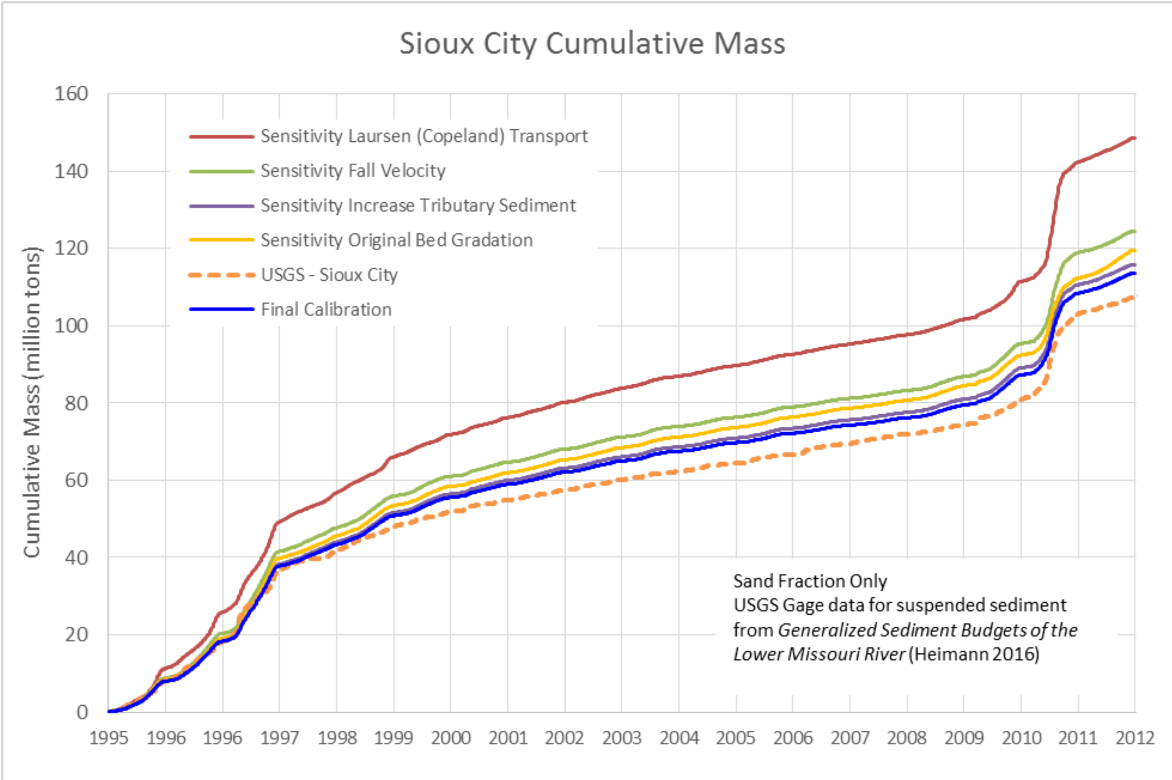


Figure 20. Sioux City USGS Suspended Sediment Mass vs. Model Simulations

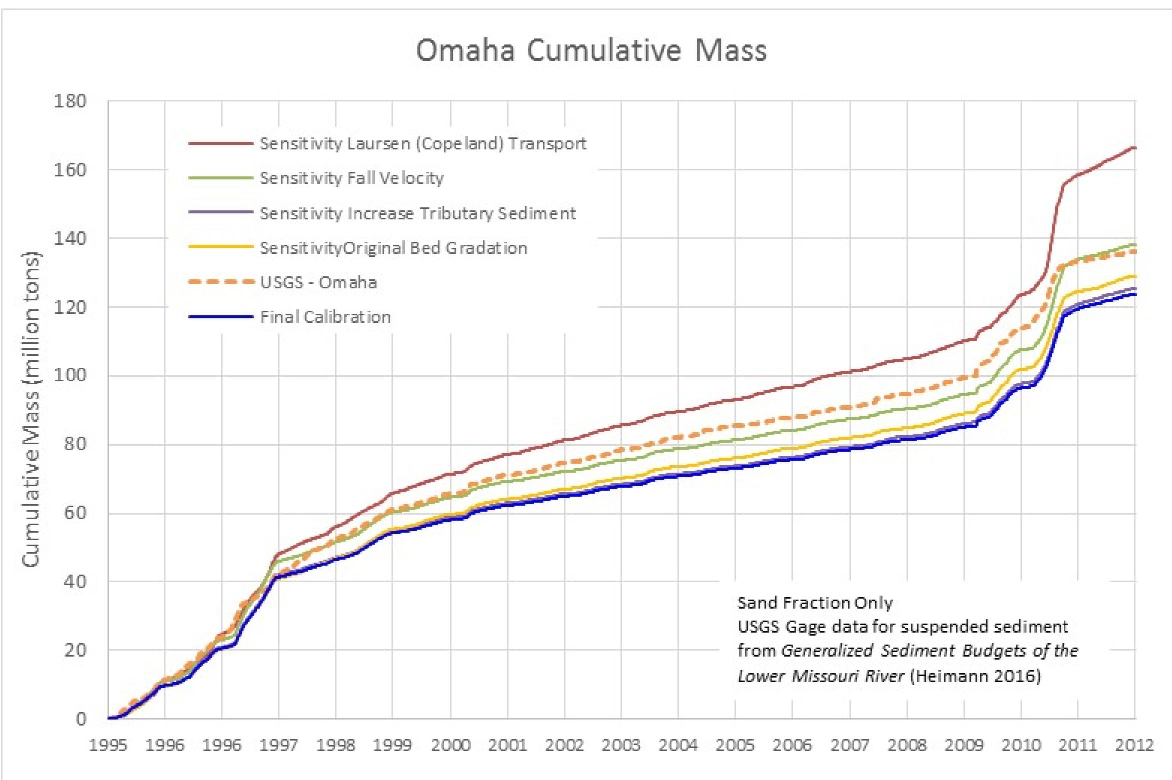


Figure 21. Omaha USGS Suspended Sediment Mass vs. Model Simulations

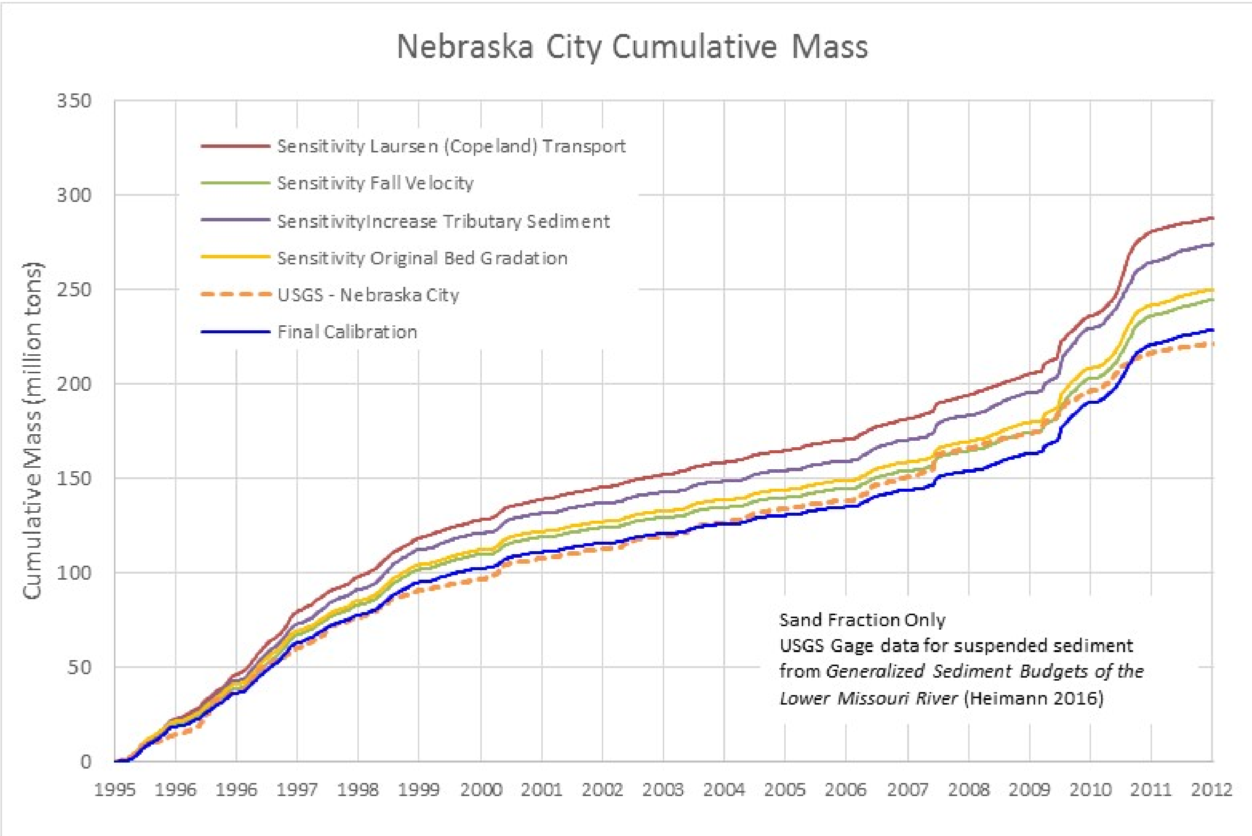


Figure 22. Nebraska City USGS Suspended Sediment vs. Model Simulations

5.3 SEDIMENT BUDGET

A rudimentary sediment budget was constructed to illustrate the primary sediment inputs and comparison to USGS data for each reach between gage stations. The sediment budget model to USGS gage data difference for all years from 1995 – 2012 is +6% for the Gavins Point Dam to Sioux City reach, -9% for the Sioux City to Omaha reach, and +3% for the Omaha to Nebraska City reach. Results are illustrated in Table 12, Table 13, and Table 14.

Table 12. Gavins Point Dam to Sioux City Summary Budget

Sand Fraction Only. All units tons.

Calendar Year	Gavins to James Bank	James	James to Vermillion Bank	Vermillion	Big Sioux	Vermillion to Sioux City Bank	Reach Total Sediment Input	USGS Gage Data at Sioux City	Final Calibration at Sioux City	% Final Cal. \ USGS	Mass Balance USGS D/S - U/S - Input (tons)
1995	289,594	366,979	235,296	13,493	202,399	275,936	1,383,696	8,096,551	7,909,310	98%	-6,712,855
1996	354,378	149,921	280,875	4,962	92,817	328,159	1,211,112	10,104,549	10,384,312	103%	-8,893,437
1997	533,183	454,249	602,729	12,952	216,454	480,405	2,299,972	18,343,565	19,470,414	106%	-16,043,593
1998	187,192	162,624	77,786	4,714	76,715	126,066	635,097	5,138,594	5,601,336	109%	-4,503,497
1999	219,914	207,129	148,225	6,990	90,203	148,825	821,285	6,309,222	7,437,676	118%	-5,487,937
2000	164,755	61,966	58,978	639	21,349	105,735	413,423	3,785,076	4,736,488	125%	-3,371,653
2001	116,920	293,450	7,575	8,337	214,051	77,549	717,883	3,119,837	3,396,072	109%	-2,401,954
2002	156,377	22,302	9,456	1,152	33,389	77,684	300,360	2,593,837	3,135,284	121%	-2,293,477
2003	165,371	25,317	6,827	3,070	24,953	76,122	301,660	2,741,003	2,864,732	105%	-2,439,344
2004	178,482	47,330	6,708	2,502	58,769	61,334	355,125	2,031,257	2,534,500	125%	-1,676,132
2005	182,231	72,881	615	3,874	72,097	52,654	384,352	2,197,613	2,244,792	102%	-1,813,261
2006	186,356	41,213	5,409	1,897	107,088	51,061	393,025	2,317,179	2,452,840	106%	-1,924,154
2007	166,748	236,804	545	6,734	132,018	48,660	591,508	2,720,748	2,084,712	77%	-2,129,240
2008	187,326	78,227	125	6,175	96,441	35,826	404,121	2,433,164	1,910,712	79%	-2,029,043
2009	157,012	324,757	21,203	3,920	63,818	77,467	648,178	2,510,494	3,323,848	132%	-1,862,317
2010	196,672	620,230	185,690	27,017	328,871	217,002	1,575,482	6,571,660	7,802,432	119%	-4,996,177
2011	590,204	687,967	858,125	16,296	299,482	527,534	2,979,606	22,152,857	21,182,920	96%	-19,173,251
2012	100,557	75,146	59,588	1,504	59,036	92,685	388,516	4,450,466	5,122,880	115%	-4,061,950
Sum 1995-2012	4,133,273	3,928,492	2,565,754	126,229	2,189,948	2,860,705	15,804,402	107,617,673	113,595,260	106%	-91,813,272
Average	229,626	218,250	142,542	7,013	121,664	158,928	878,022	5,978,760	6,310,848		-5,100,737
% Reach Mass Inflow	4%	4%	2%	0%	2%	3%	15%				

Table 13. Sioux City to Omaha Summary Budget

Sand Fraction Only. All units tons.

Calendar Year	USGS Gage Data at Sioux City	Floyd	Monona Harrison	Little Sioux	Soldier	Boyer	Reach Total Sediment Input	USGS Gage Data at Omaha	Final Calibration at Omaha	% Final Cal. \ USGS	Mass Balance USGS D/S - U/S - Input (tons)
1995	8,096,551	52,452	22,210	104,462	1,761	23,510	204,395	11,398,610	9,650,604	85%	-3,097,664
1996	10,104,549	73,796	45,634	99,067	30,278	471,823	720,599	12,517,930	10,887,897	87%	-1,692,782
1997	18,343,565	52,971	13,377	58,979	3,330	47,831	176,488	18,747,576	20,880,720	111%	-227,522
1998	5,138,594	21,067	17,541	49,916	4,001	114,822	207,349	10,014,320	5,100,648	51%	-4,668,377
1999	6,309,222	23,780	14,407	64,525	4,762	55,487	162,960	8,373,870	7,676,360	92%	-1,901,687
2000	3,785,076	1,420	2,406	2,312	398	1,040	7,575	4,680,237	3,832,216	82%	-887,586
2001	3,119,837	45,462	16,686	100,696	3,997	68,343	235,185	5,427,013	4,180,536	77%	-2,071,991
2002	2,593,837	4,643	5,656	11,269	801	7,572	29,942	3,492,756	2,653,216	76%	-868,978
2003	2,741,003	13,343	9,493	49,459	1,065	31,306	104,665	3,759,628	2,875,612	76%	-913,959
2004	2,031,257	46,541	19,909	77,687	981	31,079	176,197	3,744,982	2,964,312	79%	-1,537,528
2005	2,197,613	18,864	6,715	46,798	638	16,371	89,386	3,338,641	2,358,584	71%	-1,051,643
2006	2,317,179	14,159	2,830	64,123	230	3,127	84,469	2,407,316	2,492,680	104%	-5,667
2007	2,720,748	56,029	29,783	168,394	15,052	249,169	518,427	3,015,458	2,960,824	98%	223,716
2008	2,433,164	41,585	22,641	114,812	7,741	697,999	884,779	3,808,302	2,800,912	74%	-490,359
2009	2,510,494	23,939	16,388	42,872	3,988	51,601	138,789	4,700,532	3,698,792	79%	-2,051,249
2010	6,571,660	210,992	56,414	379,519	6,009	233,618	886,551	14,522,678	11,419,448	79%	-7,064,467
2011	22,152,857	98,690	19,020	179,245	3,569	68,526	369,050	19,445,253	23,097,560	119%	3,076,653
2012	4,450,466	10,182	5,263	22,846	551	2,746	41,587	2,779,584	4,128,152	149%	1,712,469
Sum 1995-2012	107,617,673	809,915	326,377	1,636,981	89,151	2,175,970	5,038,393	136,174,686	123,659,073	91%	-23,518,620
Average	5,978,760	44,995	18,132	90,943	4,953	120,887	279,911	7,565,260	6,869,948		-1,306,590
% Reach Mass Inflow		3%	1%	6%	0%	8%	18%				

Table 14. Omaha to Nebraska City Summary Budget

Sand Fraction Only. All units tons.

Calendar Year	USGS Gage Data at Omaha	Platte	Weeping Water	Reach Total Input	USGS Gage Data at Nebraska City	Final Calibration at Nebraska City	% Final Cal. \ USGS	Mass Balance USGS D/S -U/S - Input (tons)
1995	11,398,610	4,485,440	9,753	4,495,193	14,494,760	18,687,551	129%	-1,399,043
1996	12,517,930	3,829,395	12,522	3,841,917	22,596,089	17,835,650	79%	-18,754,173
1997	18,747,576	3,183,200	3,919	3,187,119	23,305,062	26,723,556	115%	-20,117,943
1998	10,014,320	4,182,676	52,206	4,234,882	16,293,569	14,660,652	90%	-12,058,686
1999	8,373,870	4,729,381	73,089	4,802,470	14,124,854	17,189,136	122%	-9,322,384
2000	4,680,237	1,701,947	13,258	1,715,205	6,174,488	7,429,368	120%	-4,459,283
2001	5,427,013	2,544,195	16,816	2,561,011	10,828,963	8,626,856	80%	-8,267,952
2002	3,492,756	1,212,051	8,609	1,220,660	5,143,830	4,701,912	91%	-3,923,170
2003	3,759,628	1,317,392	718	1,318,110	6,371,257	4,928,224	77%	-5,053,147
2004	3,744,982	1,454,803	8,862	1,463,665	7,558,792	5,058,792	67%	-6,095,128
2005	3,338,641	1,583,929	2,492	1,586,421	7,158,627	4,819,584	67%	-5,572,206
2006	2,407,316	1,176,996	679	1,177,675	4,383,941	4,315,336	98%	-3,206,266
2007	3,015,458	3,721,262	48,280	3,769,543	12,350,068	8,908,448	72%	-8,580,526
2008	3,808,302	4,978,393	51,579	5,029,972	15,421,522	10,059,664	65%	-10,391,551
2009	4,700,532	2,745,024	5,867	2,750,891	7,908,084	9,349,744	118%	-5,157,193
2010	14,522,678	7,998,683	134,954	8,133,638	22,346,482	27,046,688	121%	-14,212,845
2011	19,445,253	4,458,560	26,185	4,484,745	20,118,101	30,737,168	153%	-15,633,356
2012	2,779,584	1,369,438	4,095	1,373,532	5,102,844	7,484,256	147%	-3,729,311
Sum 1995-2012	136,174,686	56,672,766	473,883	57,146,648	221,681,334	228,562,585	103%	-155,934,161
Average	7,565,260	3,148,487	26,327	3,174,814	12,315,630	12,697,921		-8,663,009
% Reach Mass Inflow		66%	1%	67%				

5.4 VOLUME CHANGE

Volume change during the model simulations were compared to that computed between historic river channel surveys. A steady flow RAS model was used to compare volumes for the 1995, 2008, and 2012 hydrographic surveys. A profile of 35,000 cfs was run through the 1995 geometry and the resulting water surface elevations were set for the 2008 and 2012 geometries. This method provides a constant plane. RAS output provides the volume “beneath” the plane to indicate how the river cross section is changing between surveys. Incremental volume differences between survey periods were converted to cumulative (upstream to downstream) to match the output of the sediment model for comparison. As previously discussed, the flood of 2011 was a unique event that altered the flow vs. sediment load relationship in 2012. Cumulative volume differences for the model, sensitivity simulations, and the volume computed from the surveys in shown in Figure 23 and Figure 24.

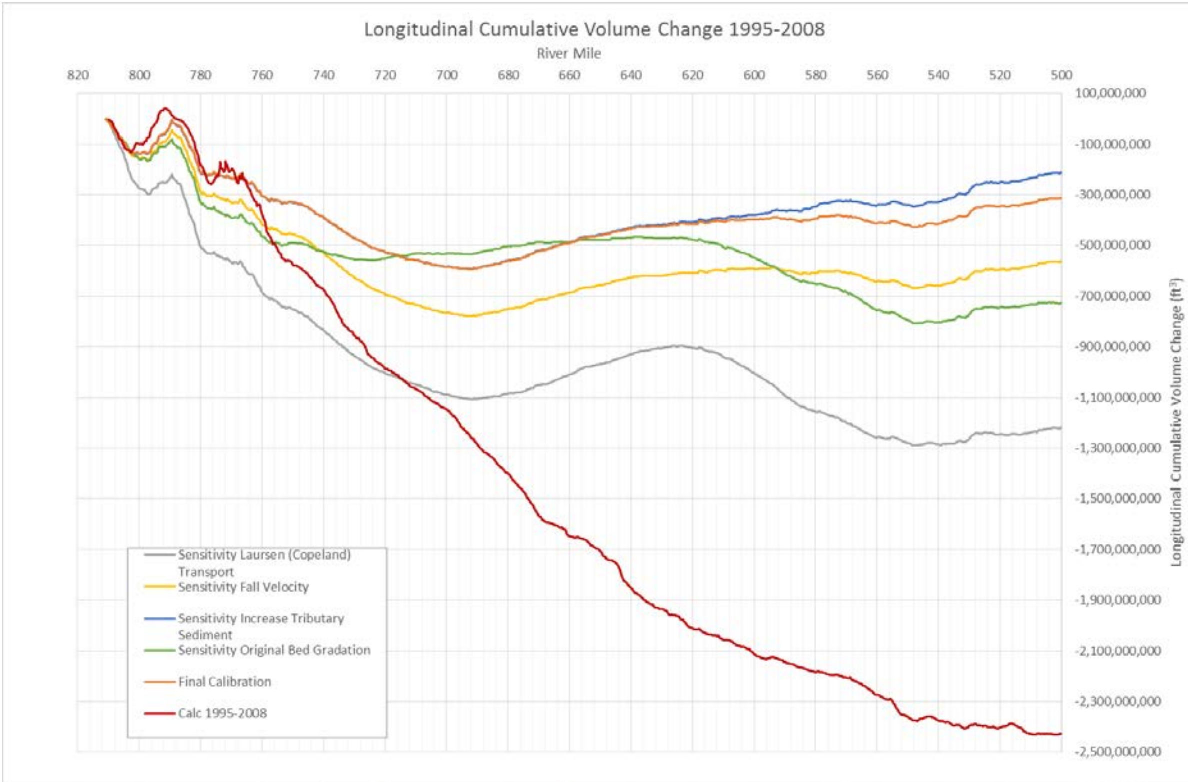


Figure 23. Longitudinal Cumulative Volume Change 1995-2008

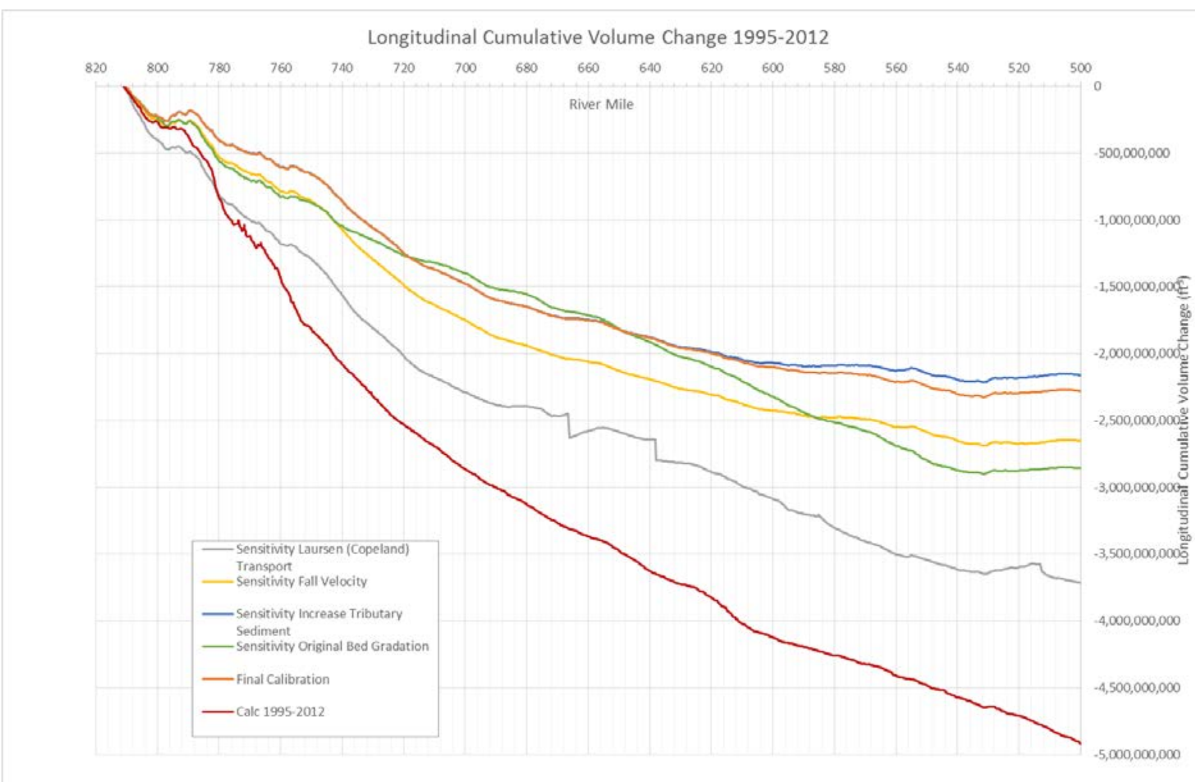


Figure 24. Longitudinal Cumulative Volume Change 1995-2012

The comparison between model results and computed survey cumulative volume differences illustrate a large difference. Sensitivity results also indicate that minor variation in the typical calibration parameters does not alter the poor fit between model results and survey computed volume change.

During the calibration process, it became apparent that calibrating to mass and water surface elevation change was feasible but that also calibrating to volume change while maintaining mass and water surface elevation was not. The differences between the volume computed from surveys and the model are summarized in Table 15. Results shown in the table illustrate that the volume ratio from 1995-2008 is only 13% while the ratio is 79% from 2008-2012. This would indicate that issues in volume change primarily occur in the period from 1995-2008.

Table 15. Longitudinal Volume Difference Summary at River Mile 500

Time Period	Survey Computed (ft^3)	Final Calibration (ft^3)	% Calibrated/Survey
1995-2008	-2,436,440,633	-319,476,500	13%
1995-2012	-4,941,427,234	-2,287,710,000	46%
2008-2012	-2,504,986,600	-1,968,233,500	79%

6 FUTURE CONDITION SEDIMENT SIMULATIONS

Future condition sediment runs used the sediment parameters from the calibration sediment model. Some parameters were updated such as bed data, moveable bed limits (due to different geometry), temperature (was estimated with monthly average observed data repeated for 82 year POR) and flow data (used the Management Plan study alternative flows, ungaged, and depletions). The average water surface change for that period was then used to estimate the Year 15 geometry.

6.1 GEOMETRY

For the future condition sediment runs, the geometry was updated to use the previously calibration existing condition model that used the 2012 survey. In addition, the alternatives used the various habitat construction models that were previously developed for each of the alternatives to reflect the construction of pallid habitat.

6.2 FLOW INPUT

Simulations used the Management Plan study alternative flows that were previously developed for the existing condition model. The POR flows include ungaged inflow and current level depletions. Alternative simulations were performed for the POR from 1930 to 2012. Unsteady flows from the POR were converted to quasi-unsteady for use with the future conditions sediment model. Since there are different rules for unsteady and quasi-unsteady flows, some uniform lateral flows had to be adjusted so that they did not start at the same cross section as another flow input. More details on the POR flow dataset can be found in the report, *Missouri River*

Recovery Management Plan Time Series Data Development for Hydrologic Modeling (USACE 2018b).

6.3 SEDIMENT PARAMETERS

The sediment parameters from the calibration sediment model were used for the future conditions sediment runs. Some parameters were updated such as bed data was revised to the latest survey data from 2012. The moveable bed limits were also updated due to using a different geometry with different cross section stationing and bank stations.

7 SUMMARY

A sediment model was constructed of the Gavins Point Dam to Rulo, NE, reach of the Missouri River for the purposes of evaluating future conditions in support of the Missouri River Management Plan study. Sediment model simulations used modeling from the period 1995 to 2012 to develop sediment parameters. These parameters were added to the previously developed alternative conditions RAS model to allow simulation with sediment and the POR flows to develop a prediction of future conditions. Simulations started with 2012 geometry. Results from the simulations were then averaged to produce a water surface change estimate for the year 2032 (Year 15 future condition) as shown in the main section of this report.

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GAVINS POINT DAM TO RULO

PLATES

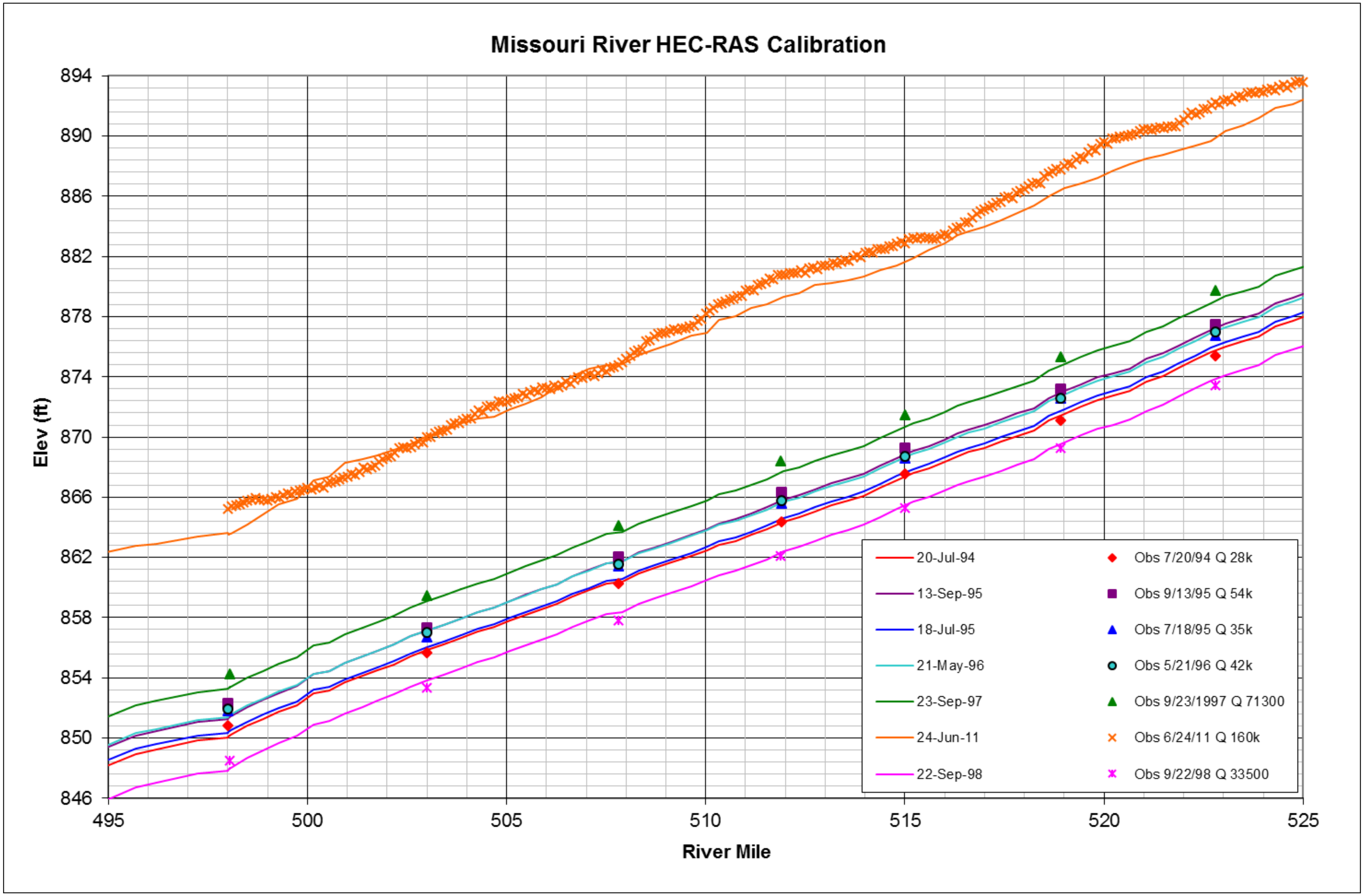


Plate 1. 1995 RAS Model Calibration, RM 495-525

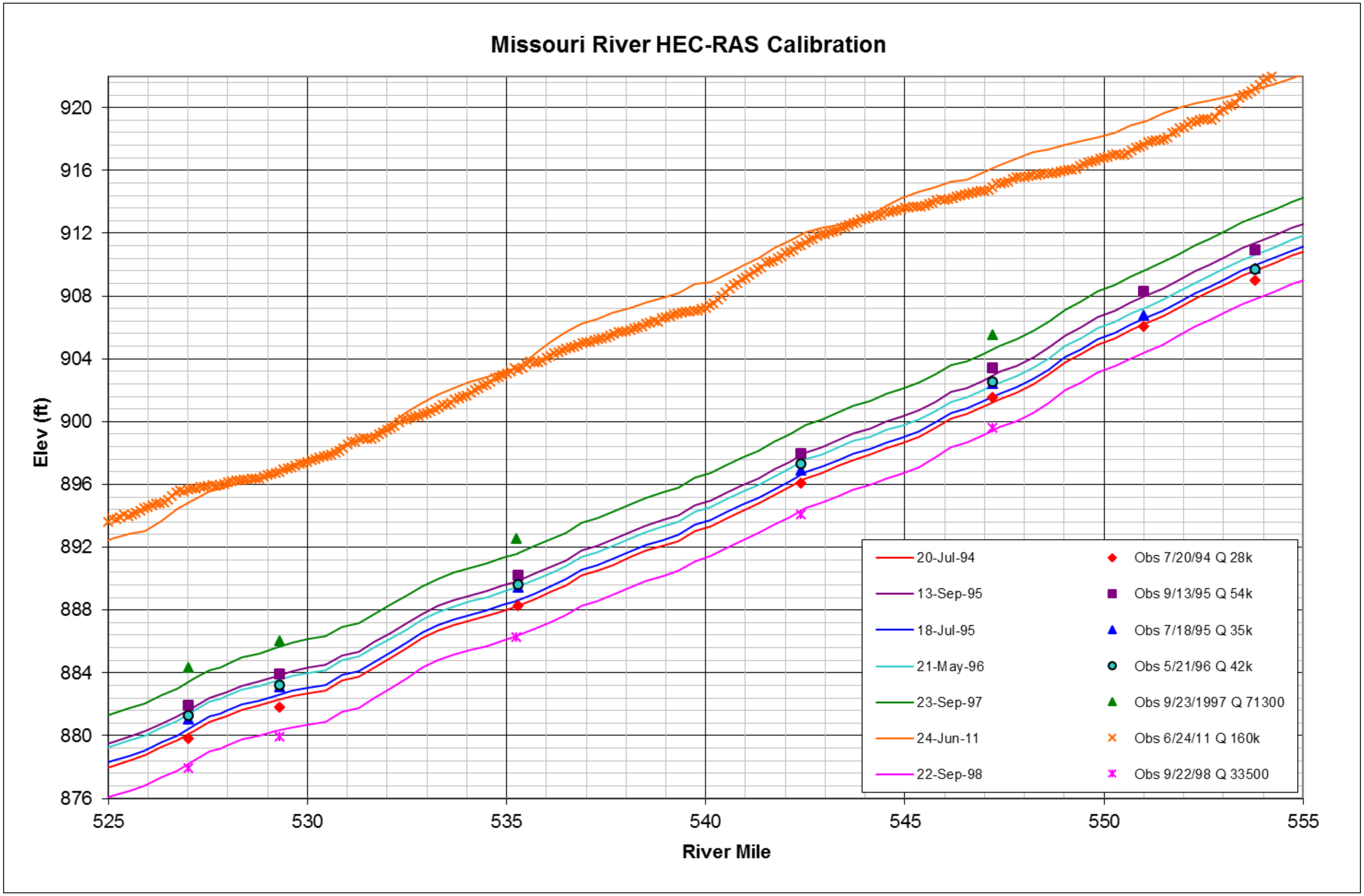


Plate 2. 1995 RAS Model Calibration, RM 525-555

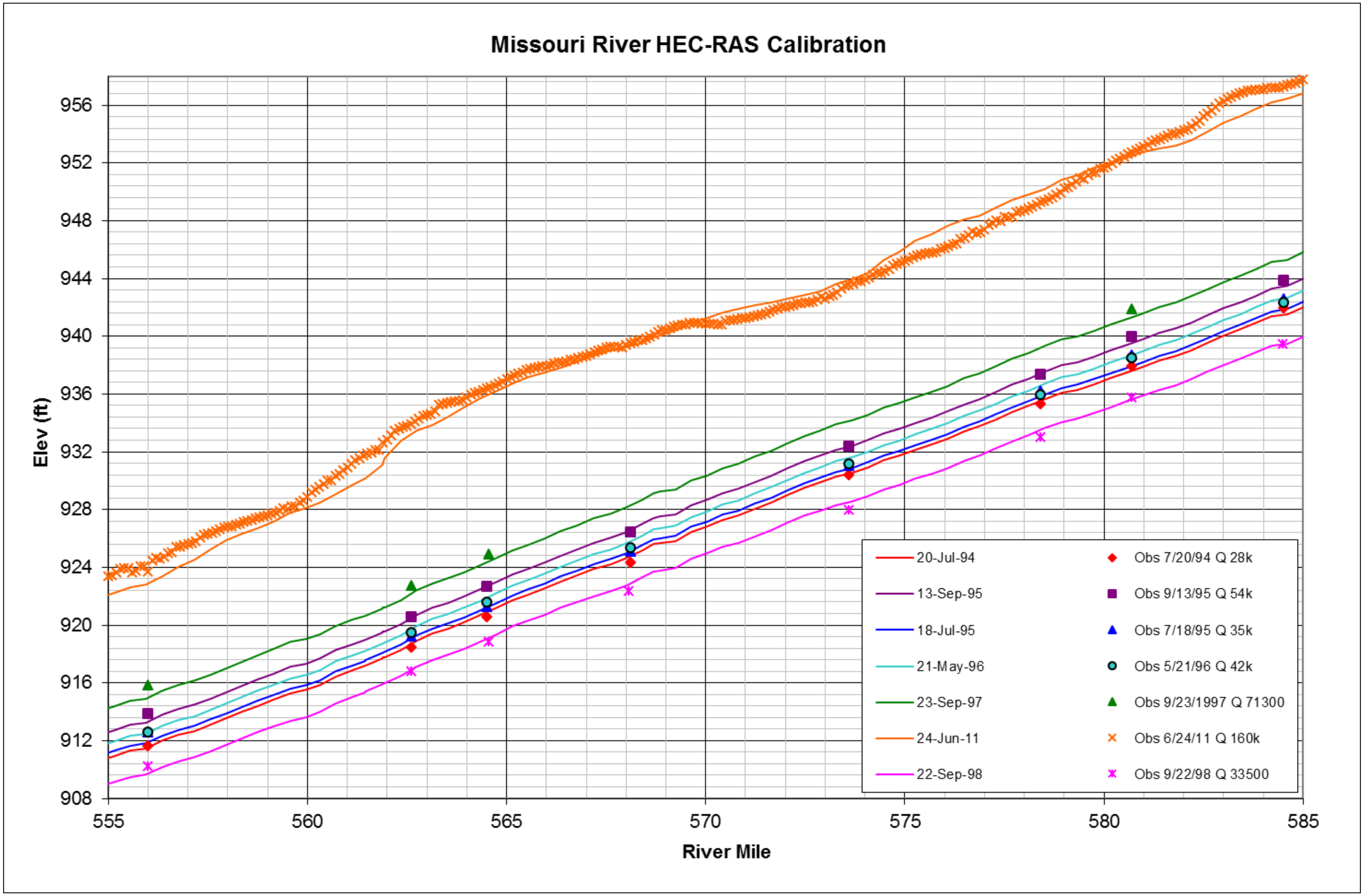


Plate 3. 1995 RAS Model Calibration, RM 555-585

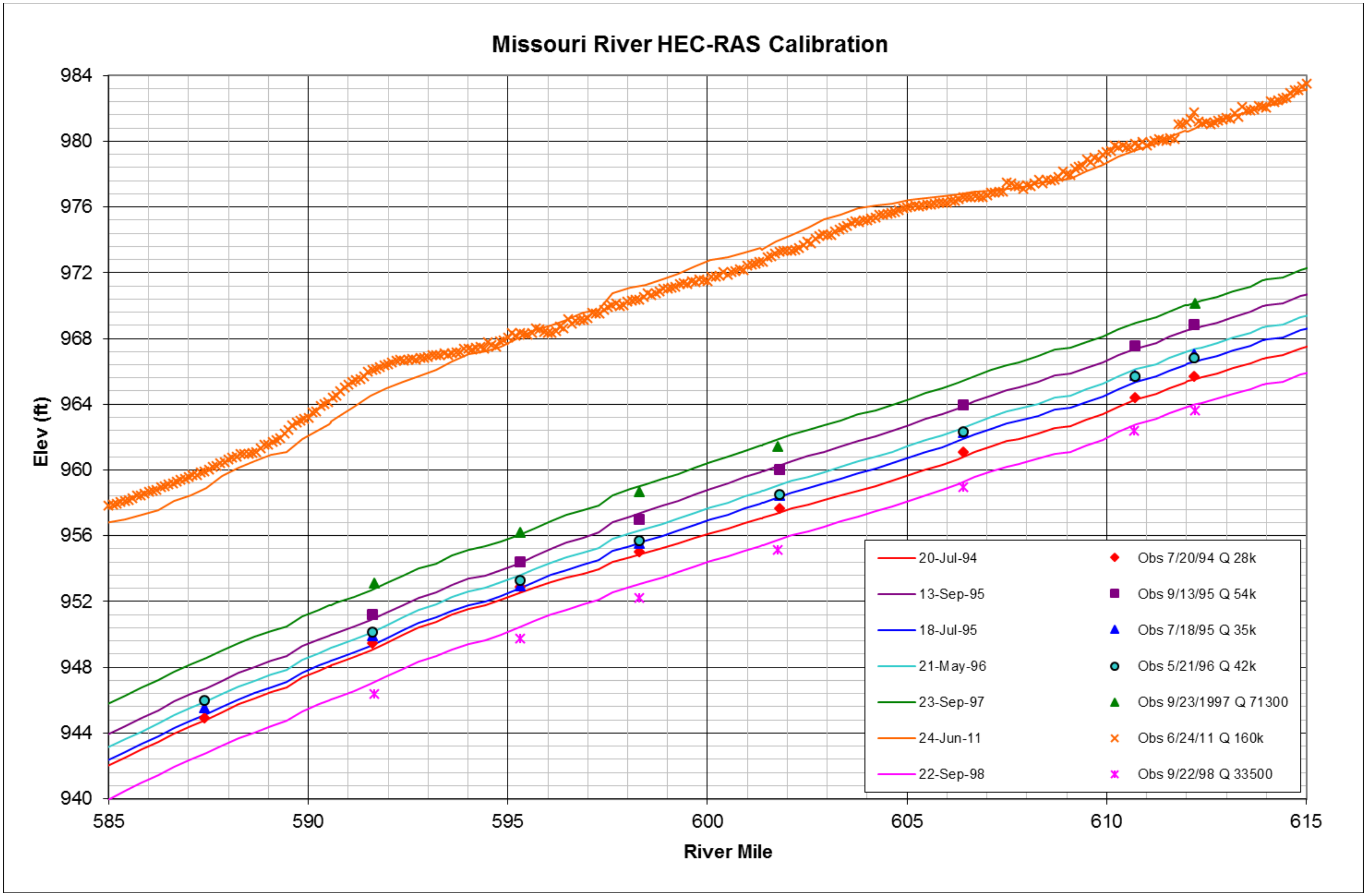


Plate 4. 1995 RAS Model Calibration, RM 585-615

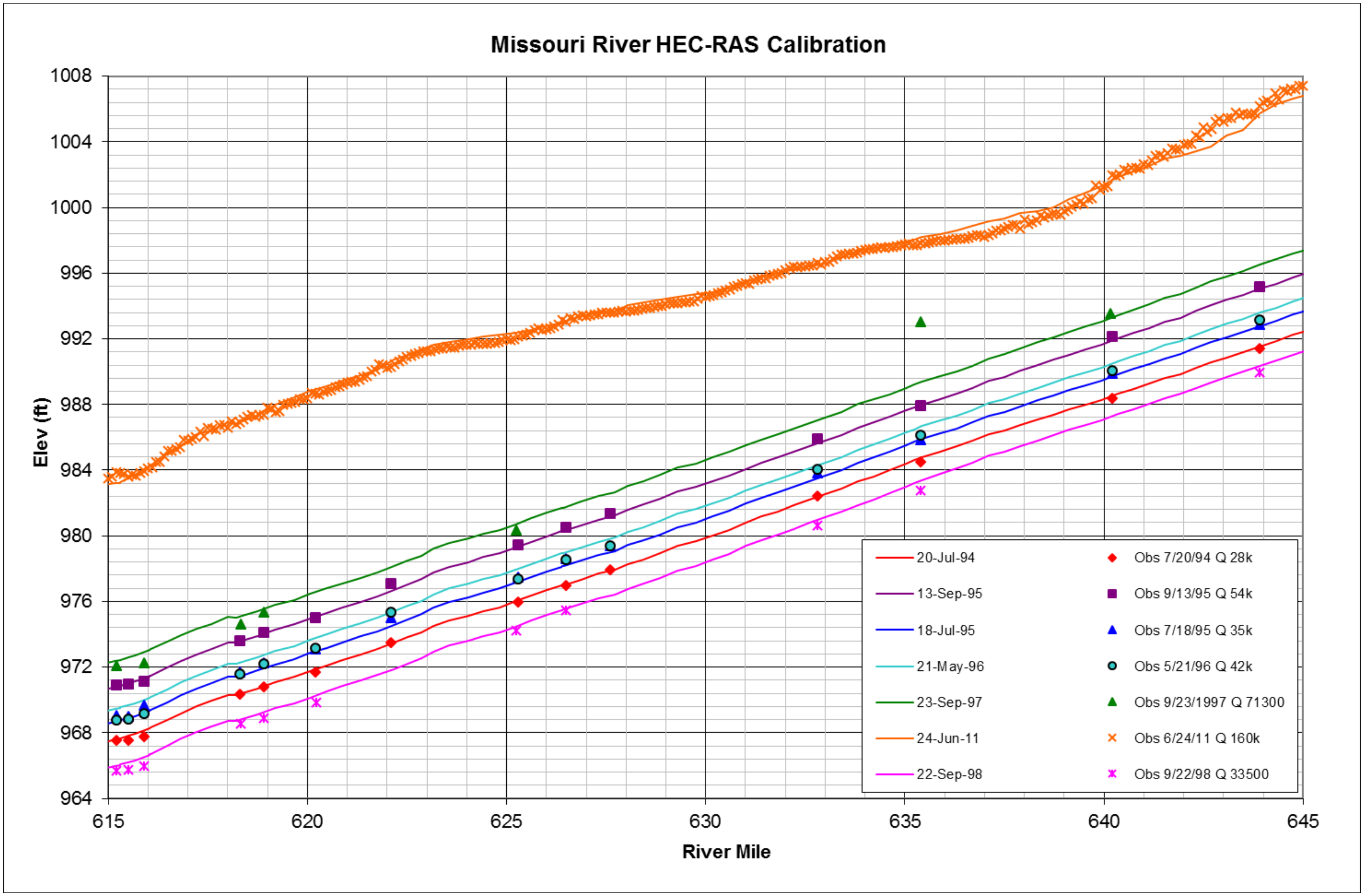


Plate 5. 1995 RAS Model Calibration, RM 615-645

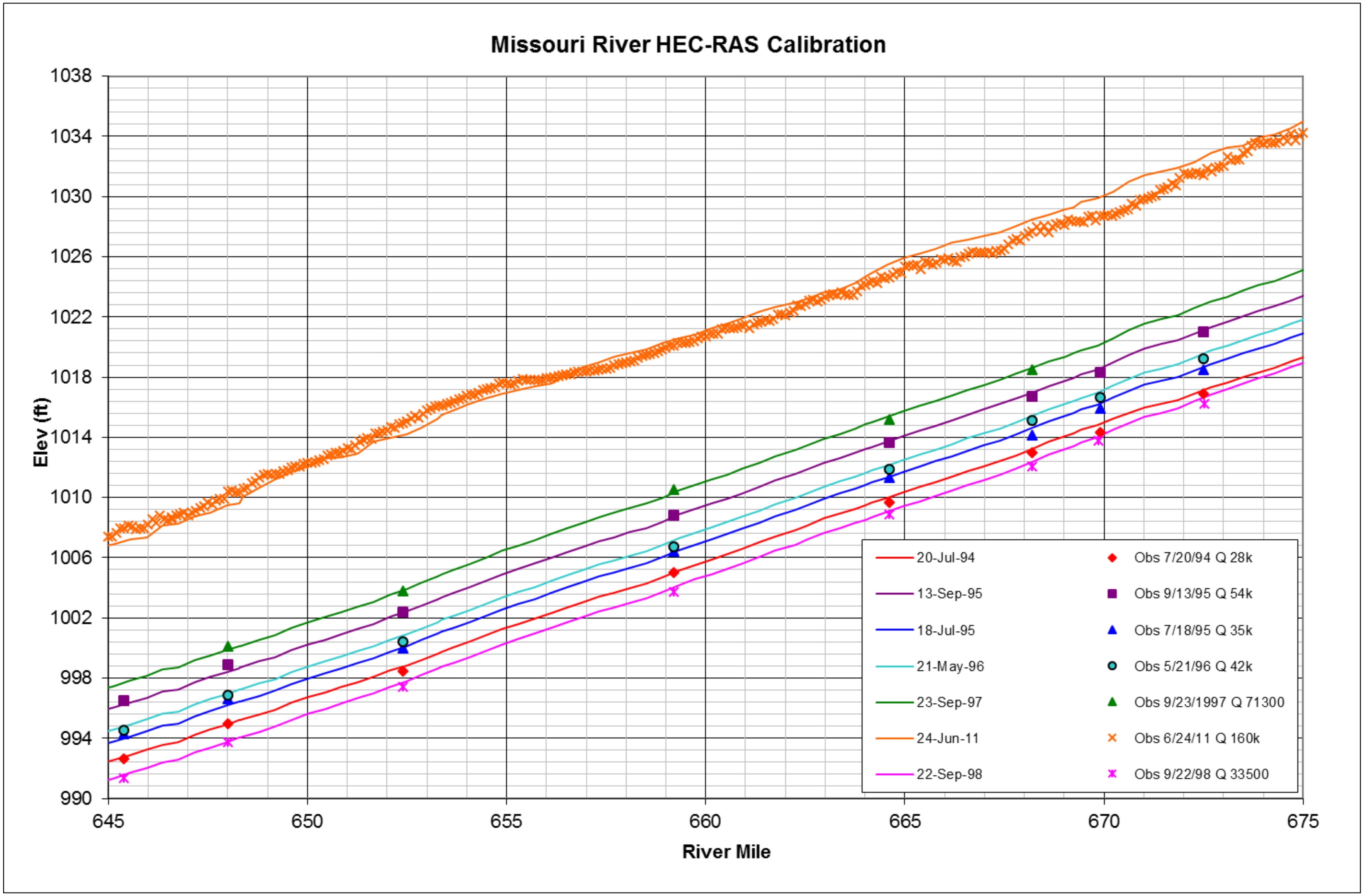


Plate 6. 1995 RAS Model Calibration, RM 645-675

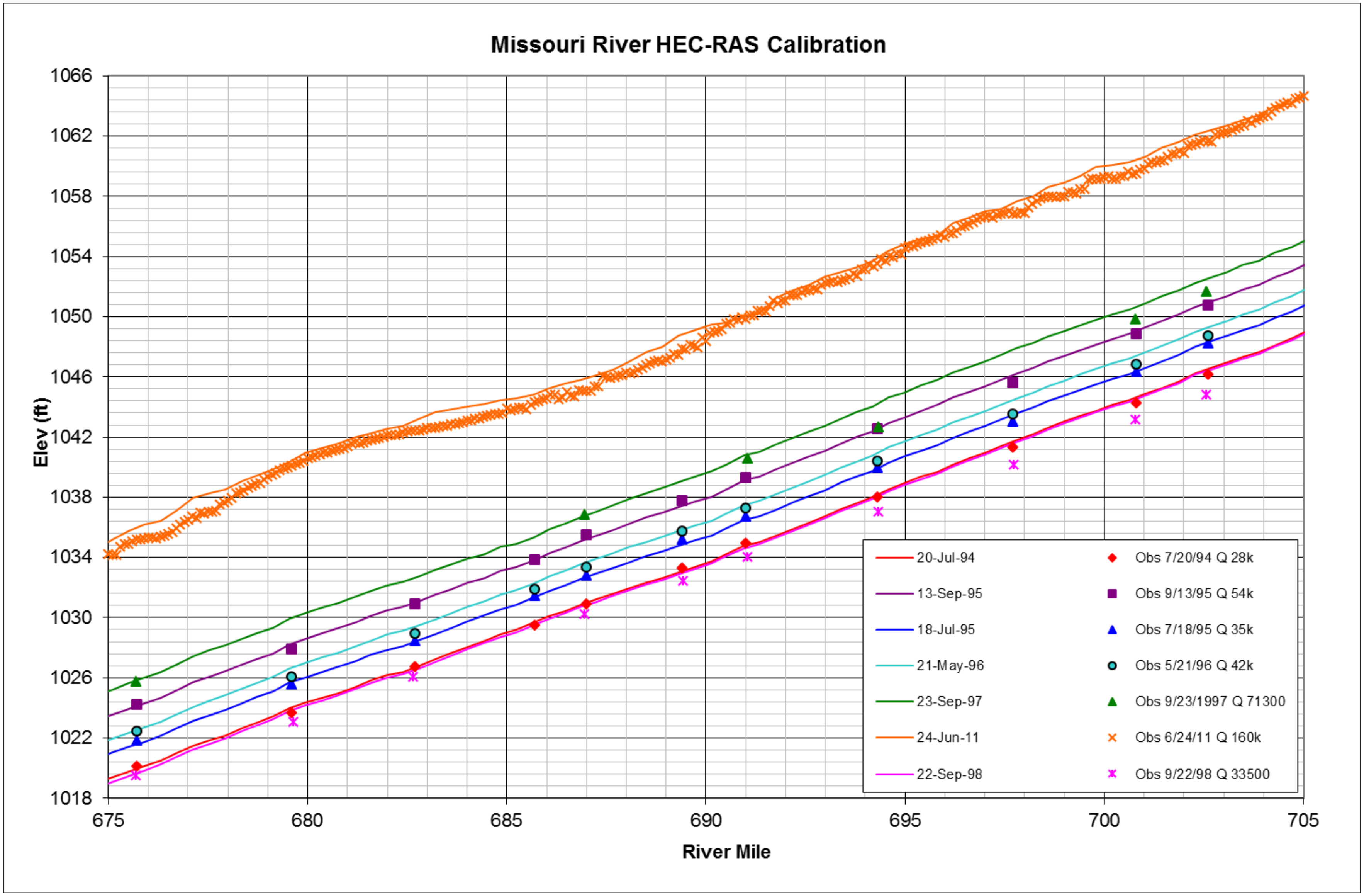


Plate 7. 1995 RAS Model Calibration, RM 675-705

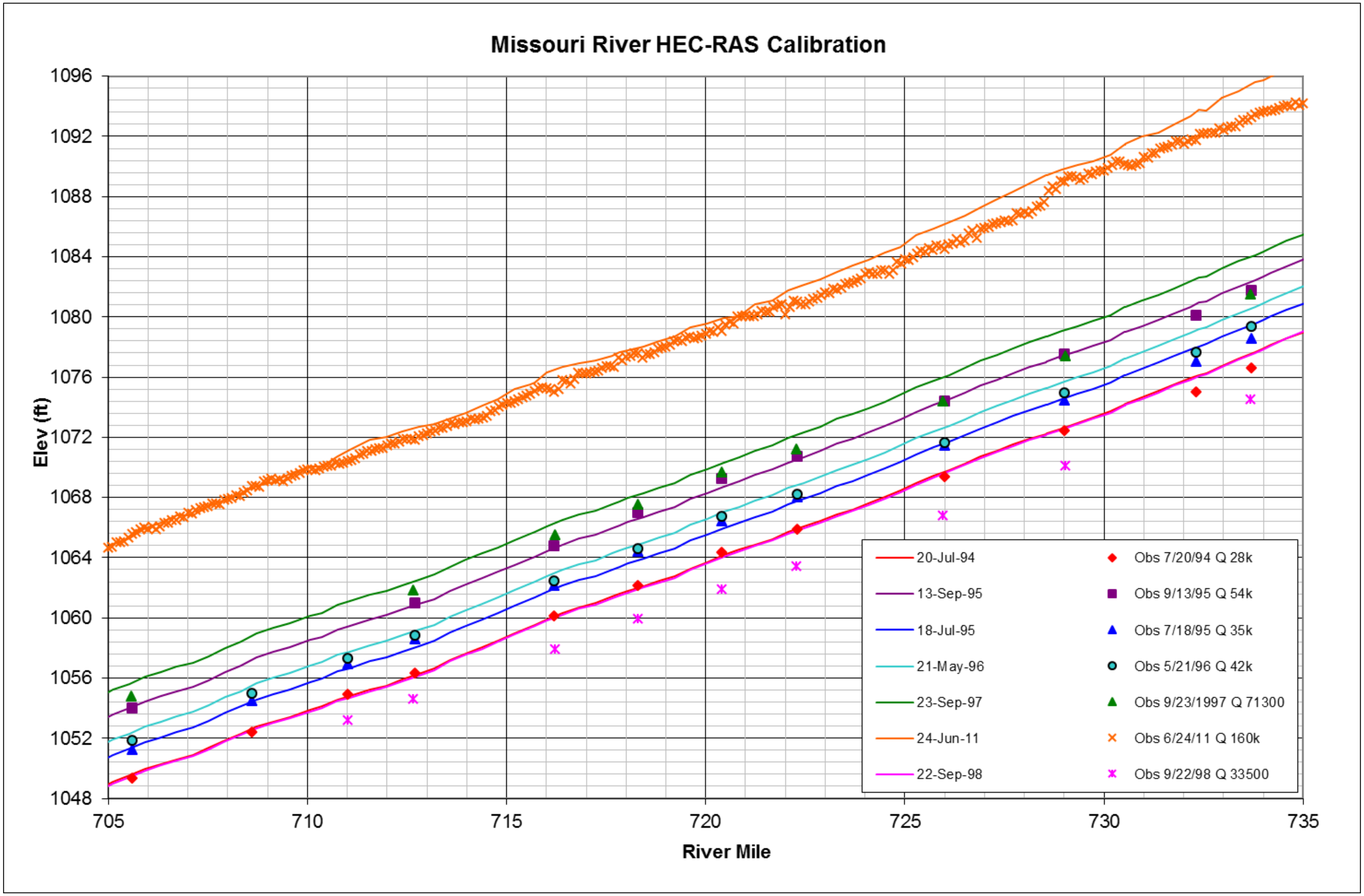


Plate 8. 1995 RAS Model Calibration, RM 705-735

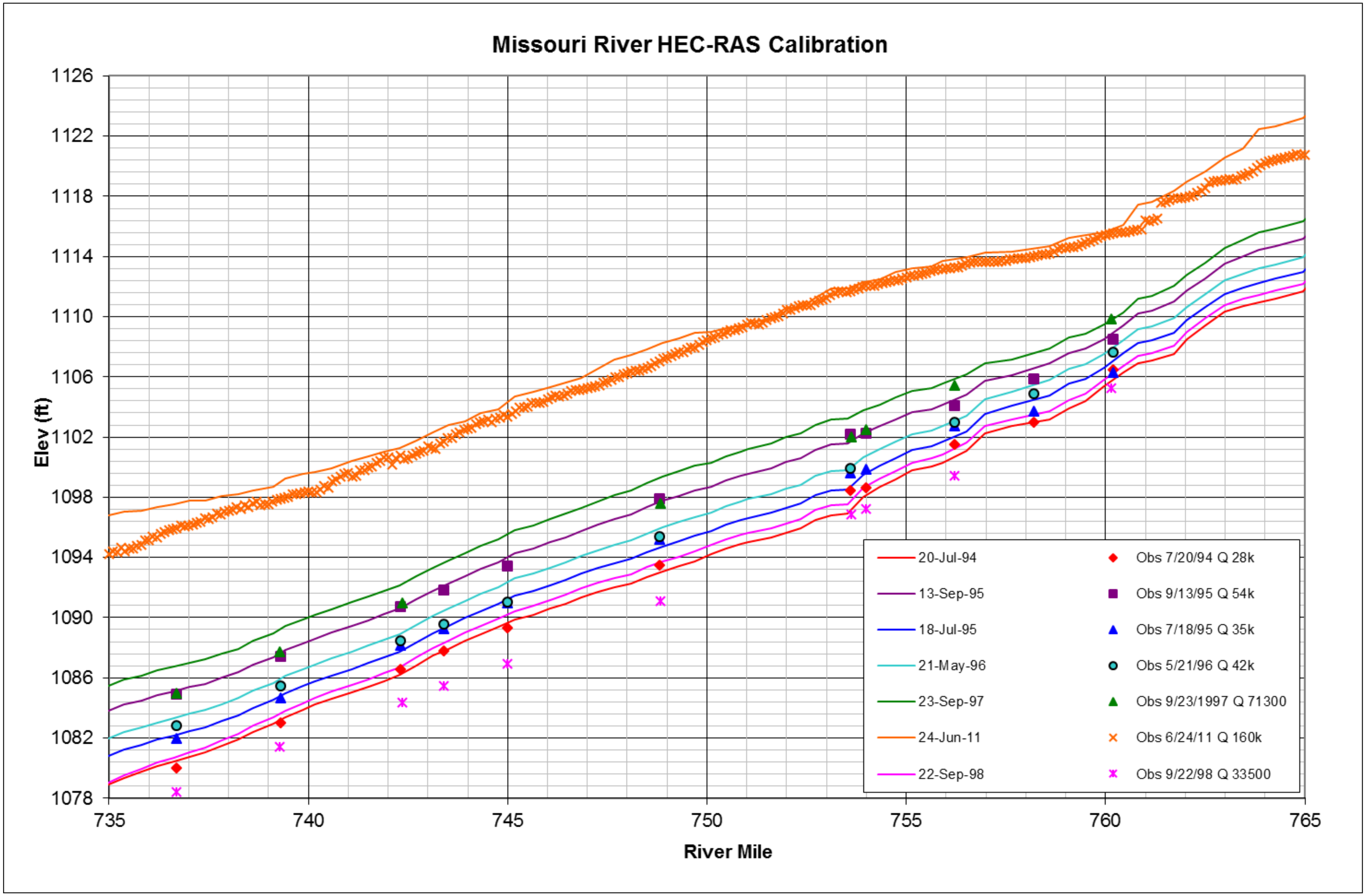


Plate 9. 1995 RAS Model Calibration, RM 735-765

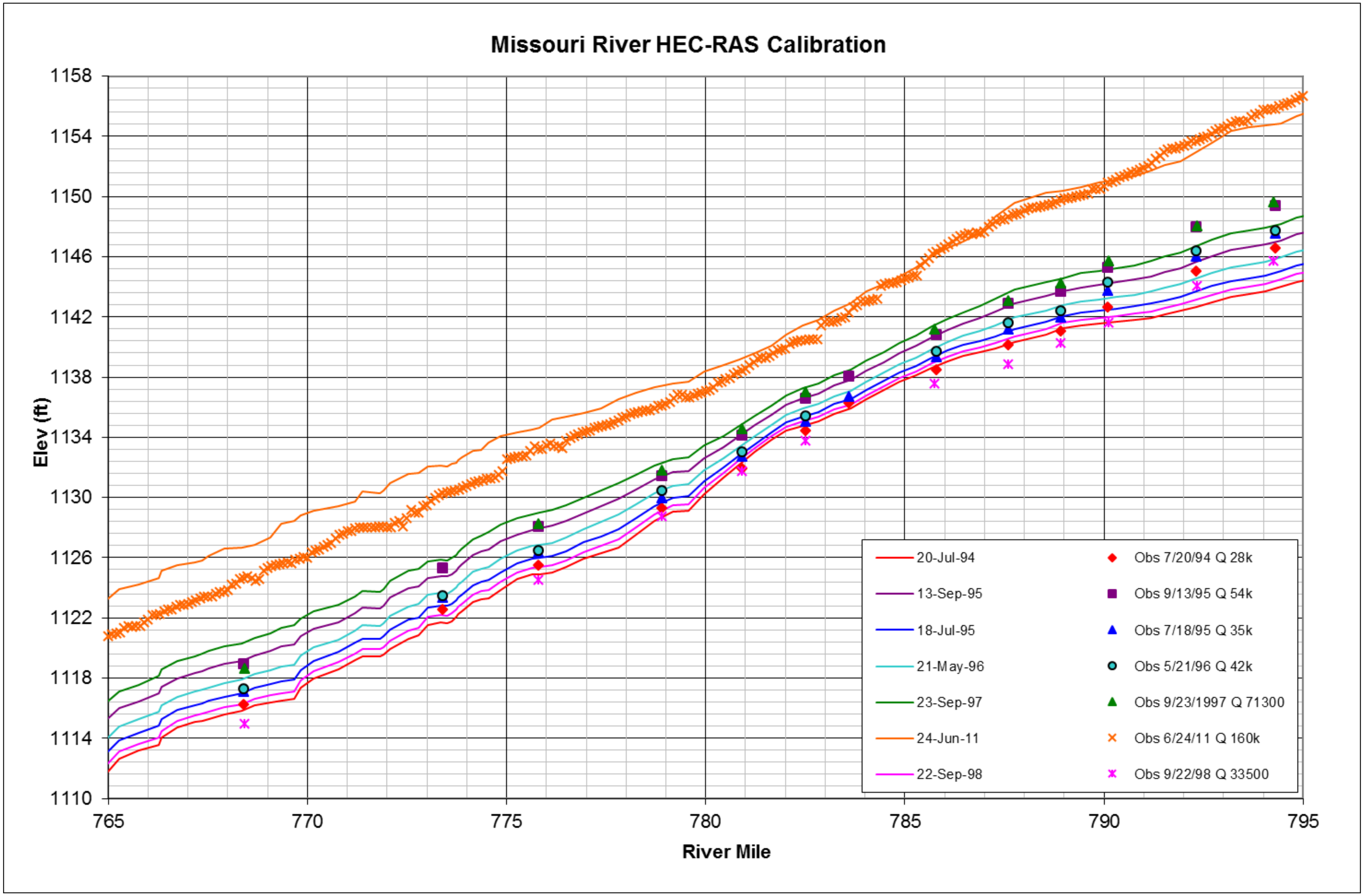


Plate 10. 1995 RAS Model Calibration, RM 765-795

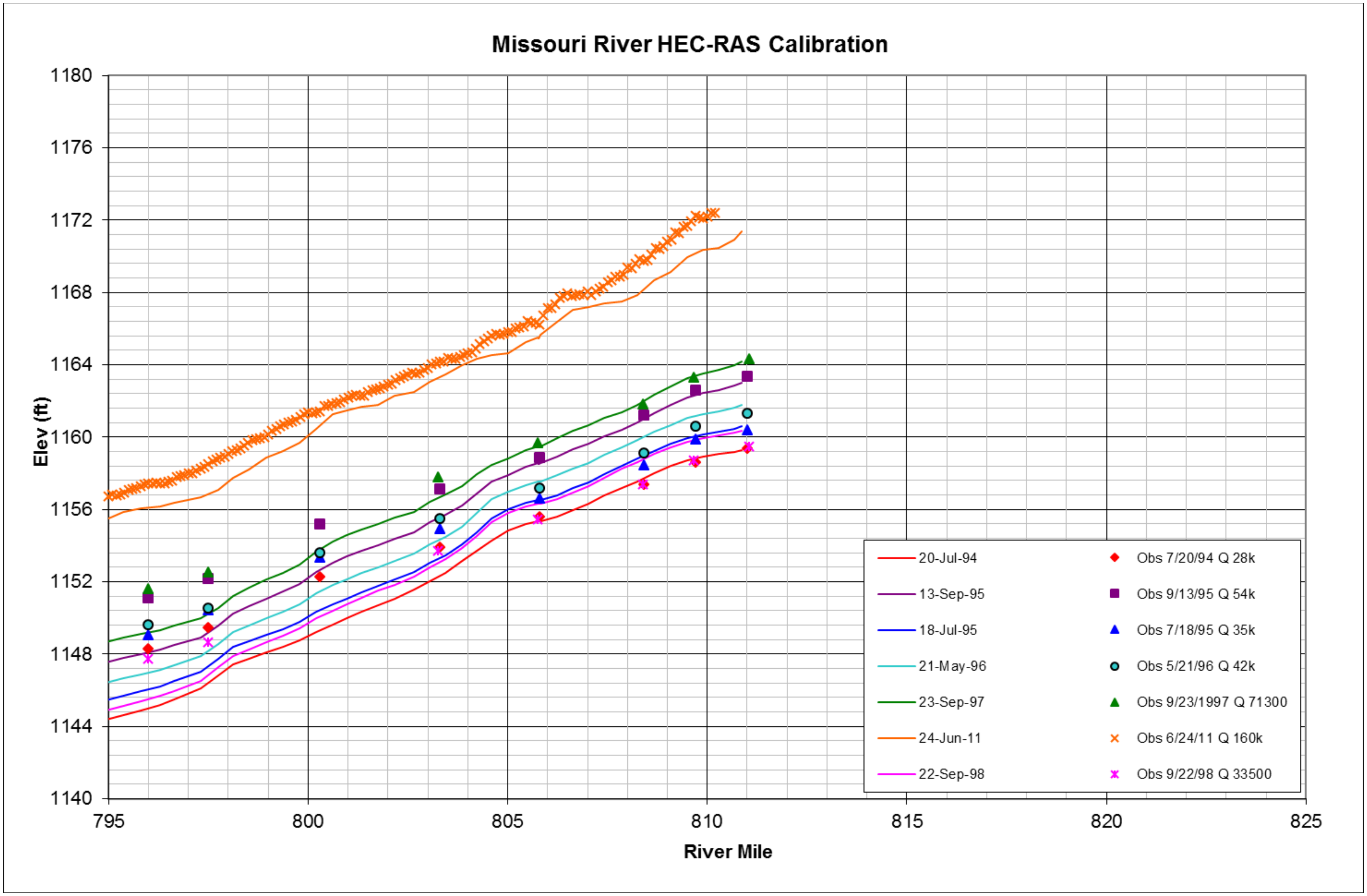


Plate 11. 1995 RAS Model Calibration, RM 795-815

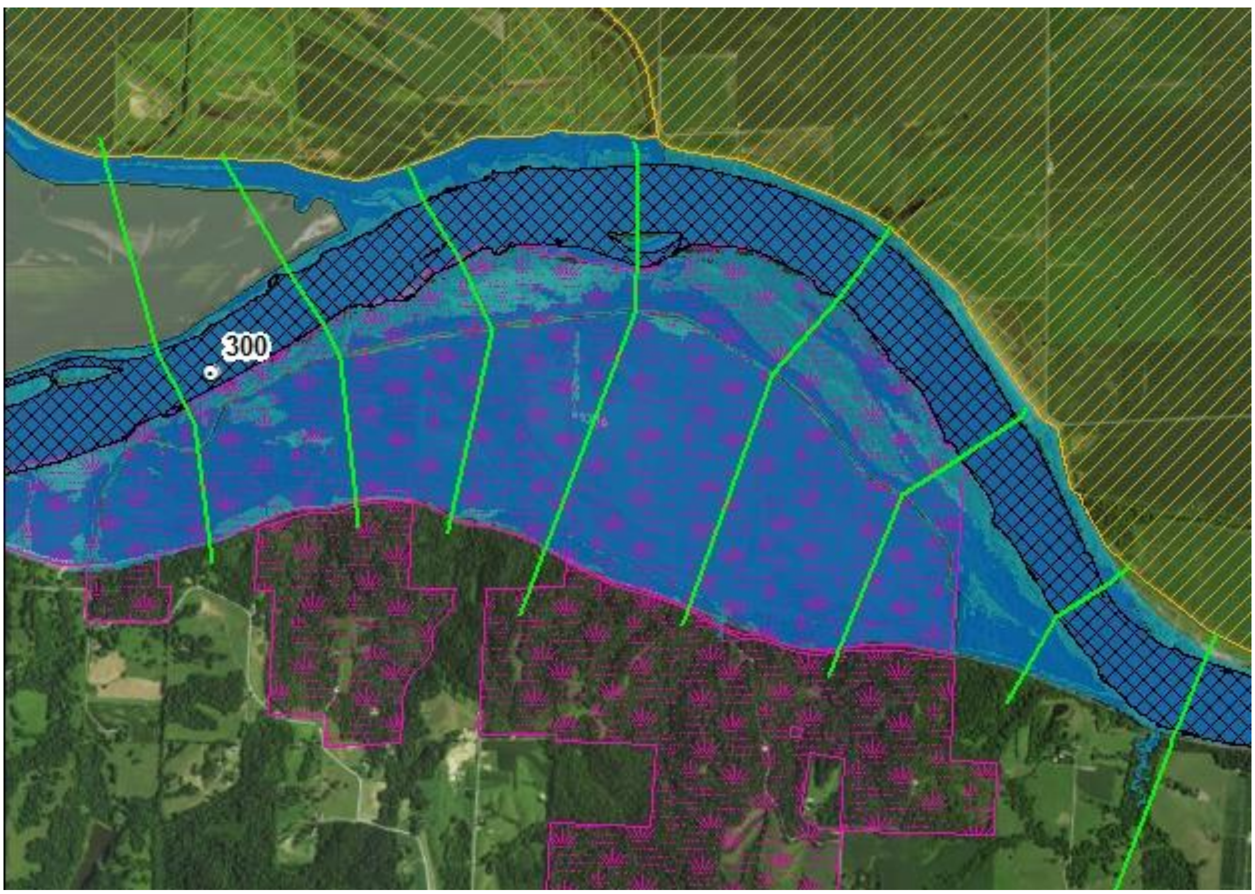
Missouri River Unsteady HEC-RAS Models
Alternatives Analysis

FINAL

US Army Corps of Engineers
Kansas City District

Appendix E

Rulo to Mouth



August 2018

COVER PHOTO:

Baltimore Bend, the site of an interception habitat project currently undergoing design as a part of the Missouri River Recovery Program (MRRP).

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Attachment 6 – Interior Drainage HMS Model Documentation

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Attachment 9A – Lower Missouri River Sediment Model Calibration Report

Attachment 9B – Lower Missouri River Sediment Model MRRP Alternatives

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ACRONYMS

2003 BiOp……………2003 Amendments to the 2000 Biological Opinion

DRM…………………..Daily Routing Model

HAMP....................... Habitat Assessment Monitoring Program

MRBWMD……………Missouri River Basin Water Management Division

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

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

RAS.......................... Hydrologic Engineering Center River Analysis System Software

ROD…………………..Record of Decision

ResSim……………..... Hydrologic Engineering Center Reservoir Simulation Software

SWH......................... Shallow Water Habitat

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

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

USFWS………………United States Fish and Wildlife Service

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

1 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. Model geometry development and calibration for the existing conditions is documented in *Missouri River Unsteady HEC-RAS Model Calibration Report Appendix E Rulo to the Mouth* (USACE, May 2015). The objective of the HEC-RAS modeling documented in this Appendix is to simulate the Management Plan alternatives which include both geometry and flow changes relative to the No Action alternative for the reach of the Missouri River in Kansas City District extending from Rulo, NE to the Mississippi River. Model extents overlap with the Omaha District on the Missouri River upstream of Rulo, Nebraska to Nebraska City, Nebraska, and with St. Louis District on the Mississippi River from L&D 25 to the St. Louis, Missouri gage.

Six alternatives, including the No Action alternative, were simulated in HEC-RAS from March 1930 to December 2012, however the subsequent economic evaluation only used complete year data for their analysis from January 1, 1931 to December 31, 2012. Development of inflow records at current depletion levels to use as boundary conditions for the Res-Sim and HEC-RAS models is documented in *Missouri River Recovery Management Plan Time Series Data Development for Hydrologic Modeling Report* (USACE, 2016a). Each alternative has unique flow releases from the reservoirs, as simulated by the Res-Sim models. Output flow hydrographs from Res-Sim were routed through the Gavins to Rulo HEC-RAS model, and then passed to the Rulo to Mouth model. Refer to **Appendix D** for details regarding the upstream model between Gavins Point Dam and Rulo, Nebraska. Three HEC-RAS geometries were developed, representing three proposed habitat construction configurations on the river. See Table 1-1 for a complete list of the geometry and flow pairings for each alternative. No Action Alternative 1 is the baseline simulation to which all other Alternatives will be compared for evaluation.

Table 1-1. Alternative Geometry and Flow Pairings

Alternative	River Geometry (RAS)	Reservoir Flow (Res-Sim)	Adopted Alternative Short Name
Alternative 1	No Action	No Action	No Action
Alternative 2	BiOp	BiOp	BiOp
Alternative 3	IRC	All Mechanical	Mech
Alternative 4	IRC	Spring Bird Flow 2, 42 MAF	Spring 2
Alternative 5	IRC	Fall Bird Flow 5, 35kcfs service level	Fall 5
Alternative 6 *	IRC	Pallid Spawning Cue	Spawn Cue

* Former name was Alternative 7, which may corresponds to some HEC-RAS model runs and file names

Refer to Section 2 for detailed descriptions of the geometries, and Section 3 for a summary of the flows.

2 GEOMETRY

Three geometry configurations were developed in HEC-RAS. Modifications generally included the addition of shallow water habitat (SWH) in the form of river widening at a specified reference flow plus the addition of a few chutes. Backwaters and modifications to previously constructed projects were accounted for by the Omaha District only. Assumptions for each individual geometry, as well as assumptions that apply to all three are detailed in the following subsections.

2.1 MODIFICATIONS TO CALIBRATION MODEL

The starting point for all alternative geometry changes was the calibrated geometry documented in the *Missouri River Unsteady HEC-RAS Model Calibration Report Appendix E Rulo to the Mouth* (USACE, May 2015). From hereafter this geometry will be referred to as Existing Conditions 2012 geometry because it generally represents river conditions and calibrated water surface elevations as of the year 2012.

A handful of geometry changes had to be made to run the period of record. All changes from the Existing Conditions 2012 geometry and calibration runs are documented in **Attachment 1 – Revision Log**. Many of the modifications were small corrections to mistakes found later and adjustments to weir coefficients for stability that are not likely to impact water surfaces significantly. Significant modifications that are worth mentioning include:

1) **Hydrosurvey data corrected** – Bed data was corrected to account for Temporary Bench Mark (TBM) errors discovered during a spring 2015 quality control check of the hydrosurvey spreadsheet. 8 TBMs were found to be in error, impacting 64 model cross sections. The maximum positive shift in bed data was 1.27 ft and the maximum negative shift was -1.31 ft. Cross sections were only modified if the impact was greater than two tenths of a foot. Modifications were between the upstream limit of cross section 449.44 near the St. Joseph gage, and downstream limit of cross section 286.16 near Waverly. Calibration was checked at each gage from St. Joseph to Waverly. Calibration at St. Joseph was improved after the changes, calibration at Kansas City remained unchanged, and at Waverly the flow varied factors were adjusted to return the calibration to Post-ATR model accuracy. Calibration metrics were not re-calculated because the changes were so small it is not likely to significantly change the calibration precision of the model.

2) **RAS version** – Calibration was performed in the 01-Oct-2014 version of HEC-RAS 5.0 Beta. All alternatives runs were performed in HEC-RAS 5.0.3. Modifications to the model for the software upgrade included: 1) lateral structures adjusted slightly so they did not extend into junction zones, 2) initial conditions removed on most storage areas, these are now automatically calculated and 3) HTab minimum elevations adjusted where they were accidently set below the channel invert. HEC-RAS 5.0.3 is the recommended version for running the Missouri River model.

3) **Mechanism to drain storage areas** – In HEC-RAS once a storage area is overtopped it holds water until the end of the simulation, unless a mechanism is implemented to drain

the water from the area. This weakness was acknowledged in the calibration report. Possible solutions were to add pumps, add breaches, or add culverts.

Pumps were tested, but were tedious to input into the model and had stability challenges. Additionally, pumps were inadvertently left out of the import wizard, making them difficult to transfer between geometries.

Breaches were also tested but not ultimately used for multiple reasons. First and foremost, breach variability in the real world makes it very difficult to pick one location and one overtopping depth per unit that would appropriately replicate the breach process for 82 years of simulated record. Breaches are unpredictable and vary considerably from event to event even on the same levee unit. Man-hours required to set and test appropriate breach parameters for each of the 160+ leveed areas on the lower Missouri River downstream of Rulo would be considerable, without adding much value to the project objectives. Additionally, due to a coding oversight in HEC-RAS the breach and repair function only occurs one time per run. Many levees on the lower river have breached dozens of times over the period of record, making this option only feasible to implement if the period of record simulations were re-started after any levee breach, which is not practical for running a period of record simulation for multiple alternatives.

Culverts with flap gates were quick to input, easy to stabilize, and easily transferable from geometry to geometry. Culverts input into the model to drain storage areas was an independent and simplified effort as compared to the detailed interior drainage modeling analysis. Refer to Section 5 for full documentation of the site-specific analysis performed to evaluate interior flooding from rainfall runoff and seepage. For the primary model runs, actual on the ground culvert configuration at each levee was not input into the model, instead a blanket assumption for number and size of culverts was made based on protected area size. For areas less than 5,000-acres, one pipe per 700-acres was created in the model, with a minimum of at least 1-36” pipe. For areas over 5,000 acres 1-60” pipe was created per 1,100-ac of protected area. Culverts were created at or slightly above the lowest elevation in the storage area curve. Leaving a little room at the bottom provided stability on the receding limb and left a small amount of water in the area for stability on the next overtopping event. Water remaining in the areas was checked to make sure it was an insignificant amount and was contained in ditch areas. The flap gate option was used in HEC-RAS to prevent river water from entering the area prior to overtopping, but allows water from levee overtopping to drain from the area after the river recedes.

2.2 Alternative Geometry Overview

Each of the three geometries created for alternatives analysis has a unique set of assumptions and goals with regard to SWH. Target acres, distribution on the river, and depth criteria at a specified statistical reference flow were defined for each geometry. Geometries were constructed to represent the end state at the completion of the ROD implementation period. Table 2-1 summarizes the descriptions below.

1) **No Action** – Assumes habitat construction activities follow current practices to achieve 20 acres/mile of SWH, the minimum target specified within the 2003 Amendments to the 2000 Biological Opinion (2003 BiOp) of 20 – 30 ac/mi (USFWS, Dec 2003). Habitat was distributed by 2003 BiOp reaches, and the 2014 *Shallow Water Habitat Accounting Report* (USACE, Sept 2014) was used to determine the acreage deficit within each reach to attain the 20 ac/mi goal. Habitat was placed to provide 0 – 5 ft of depth at August 50% exceedance flows. Most of the SWH added to the geometry was in the form of top width widening, the remainder accomplished with chutes.

2) **BiOp As Written/ As Projected (BiOp)** – Guidance from the US Fish & Wildlife (USFW) documented in *Planning Aid Letter Regarding the Missouri River Recovery Management Plan-EIS: USFWS 2003 BiOp Projected Alternative* was provided to create a geometry which represents an ideal implementation of the 2003 BiOp. It assumes habitat construction accomplishes 30 ac/mi of SWH, and performs at a wider range of flows. Similar to the No Action geometry, habitat was distributed by 2003 BiOp reaches, and the 2014 *Shallow Water Habitat Accounting Report* was used to determine the acreage deficit within each reach to attain the 30 ac/mi goal. One third of the habitat was set provide shallow water at summer low flows, one third at median August, and one third at a spring pulse flows. Most of the SWH added to the geometry was in the form of top width widening, the remainder accomplished with chutes. Part of the BiOp requirement was maximizing floodplain connectivity, and a separate analysis was conducted as documented in Section 2.8 to ensure the requirements were met, although no changes to the HEC-RAS model were necessary.

3) **Interception Rearing Complex (IRC)** – Assumes SWH construction activities proceed based on findings made by the Effects Analysis (Jacobson, et al., 2016). Total amount of habitat was based on a current annual SWH implementation rate of about 130 ac/year per district for a total of 260 ac/year for 13 years. Distribution was based on conversations with the Effects Analysis team, who specified upper and lower boundaries based on their knowledge of larval pallid spawning locations, drift rates, and timing of interception. Sioux City is the upstream threshold for IRC placement, with the area between the Nebraska Platte River and the Osage River more likely to be successful. Chutes and existing habitat may be modified to meet IRC habitat criteria, but for purposes of HEC-RAS modeling these were not counted toward the target acreage. Habitat was placed to provide 0 – 6 ft of depth at median June flows. All of the SWH goal acreage was accomplished by top width widening.

Table 2-1. Geometry Summary

	No Action	BiOp	IRC
Target acres of SWH	20 ac/mi	30 ac/mi	260 ac/yr for 13 years
Basis for SWH target	minimum 2003 BiOp target	full 2003 BiOp target	current annual SWH implementation rate
Existing Habitat	counted, used 2014 SWH Report	counted, used 2014 SWH Report	not counted
Chutes[1]	Modeled & counted	Modeled & counted	Modeled but not counted
Distribution	20 ac/mi per BiOp reach	30 ac/mi per BiOp reach	Located for optimal interception/retention
Reference flow	August 50% Exceedance	1/3 Summer low, 1/3 median August, and 1/3 spring pulse	Median June
SWH depth criteria	0 – 5 ft	0 – 5 ft	0 – 6 ft
Additional requirements	-	Floodplain connectivity	-

Note:

1) All three geometries include the same 5 chute projects that were in construction or were completed soon after the calibration period (Benedictine Bottoms, Dalbey, Cranberry Bend, Jameson Extension, and Cora Island).

2.3 Acres of Habitat

Breakdown of target acres of SWH by reach and by habitat type for each geometry is summarized below. No Action and BiOp targets take into account existing acres of SWH on the river, whereas IRC only considered new construction. Two types of SWH construction in the Kansas City District were added, top width widening and chutes. Emphasis was heavy on top width widening. Chutes added were the same for all geometries, but were only counted toward the target acres for the No Action and BiOp geometries.

Widening and chutes were accumulated by a length along the river in miles, and then converted to acres by multiplying by a uniform top width and converting units to acres. For widening, the length measurement started and ended about halfway between cross sections, since this most appropriately reflects the interpolation between cross sections made by HEC-RAS during computations.

2.3.1 *No Action*

Acres of SWH added to HEC-RAS to meet 20 ac/mi for the No Action geometry were broken down by 2003 BiOp segment for even distribution of habitat along the river. Table 2-2 shows the acres added to HEC-RAS by segment. Total acres added to the Rulo to Mouth HEC-RAS model was 2,380 acres.

For the Kansas City District, the No Action acreage was achieved by widening and chutes only. The breakdown of habitat type for Rulo to the mouth is shown in and is equivalent to 13% chutes and 87% top width widening. Total number of cross sections modified downstream of Rulo, Nebraska was 128 (94 for widening and 34 for chutes) of the 879 Missouri River cross sections.

Table 2-2. Acres of SWH – No Action Geometry

Reach	Segment	RM Start	RM End	Miles	Required Acres (20 ac/mi)	Existing Acres of SWH (2014 SWH Report)	SWH Acres Needed to Reach Goal
Ponca to Sioux City	11	753	735	18	360	120	240
Sioux City to Platte River	12	735	595	140	2,800	1,682	1,118
Platte River to Rulo (Omaha)	13	595	498	97	1,940	1,290	650
Rulo to Kansas River (KC)	13	498	367	131	2,620	1,270	1,350
Kansas River to Osage River	14	367	130	237	4,740	3,710	1,030
Osage River to Mouth	15	130	0	130	2,600	2,600 [1]	0
Omaha District				255	5,100	3,092	2,008
Kansas City District				498	9,960	7,580	2,380
Total				753	15,060	10,672	4,388

Note:

1) Existing acres for segment 15 (3,253 acres) exceeds 20 ac/mi. For purposes of Alternative 1 No Action, no new SWH will be added to this segment with the exception of Cora Island Chutes A & C which are currently under construction (summer/fall 2015). Cora Island was not shown to be counted toward the target acres in this table.

2) No SWH will be added to reaches upstream of Gavins, these calculations only apply to the Gavins to Rulo and Rulo to the Mouth HEC-RAS models.

Table 2-3. SWH Type – No Action Geometry

Reach	Segment	Widening (acres)	Widening (miles) [1]	Number of Widening Projects	Chutes (acres)	Chutes (miles)[2]	Number of Chute Projects
Rulo to Kansas R	13	1,129	31.1	14	221	6.1	2
Kansas R to Osage R	14	937	25.8	10	93	2.6	2
Osage R to Mouth	15	0	0	0	0	0	1[3]
Total		2,066	57	24	314	9	5

Note:

1) Assumes approx widening width on average of 300-ft in the Kansas City district.

2) Assumes ultimate chute width of approximately 300-ft in the Kansas City district.

3) Even though the target acres to accomplish 20 ac/mi = 0, Cora Island was still added to the No Action geometry because it is currently under construction.

2.3.2 *BiOp*

Acres of SWH added to HEC-RAS to meet 30 ac/mi for the BiOp geometry were broken down by 2003 BiOp segment, same as the No Action, for even distribution of habitat along the river.

Table 2-4 shows the acres added to HEC-RAS by segment. Total acres added to the Rulo to Mouth HEC-RAS model was 6,707 acres.

For the Kansas City District, the BiOp acreage was achieved by widening and chutes only. The breakdown of habitat type for the Rulo to the mouth only is shown in Table 2-5 and is equivalent to 6% chutes and 94% top width widening. Total number of cross sections modified downstream of Rulo, Nebraska was 222 (188 for widening and 34 for chutes) of the 879 Missouri River cross sections.

Table 2-4. Acres of SWH – BiOp Geometry

Reach	Segment	RM Start	RM End	Miles	Required Acres (30 ac/mi)	Existing Acres of SWH (2014 SWH Report)	SWH Acres Needed to Reach Goal
Ponca to Sioux City	11	753	735	18	540	120	420
Sioux City to Platte River	12	735	595	140	4,200	1,682	2,518
Platte River to Rulo (Omaha)	13	595	498	97	2,910	1,290	1,620
Rulo to Kansas River (KC)	13	498	367	131	3,930	1,270	2,660
Kansas River to Osage River	14	367	130	237	7,110	3,710	3,400
Osage River to Mouth	15	130	0	130	3,900	3,253	647
Omaha District				255	7,650	3,092	4,558
Kansas City District				498	14,940	8,233	6,707
Total				753	22,590	11,325	11,265

Table 2-5. SWH Type – BiOp Geometry

Reach	Segment	Widening (acres)	Widening (miles) [1]	Number of Widening Projects	Chutes (acres)	Chutes (miles)[2]	Number of Chute Projects
Rulo to Kansas R	13	2,439	44.7	19	221	6.1	2
Kansas R to Osage R	14	3,307	60.6	25	93	2.6	2
Osage R to Mouth	15	529	9.7	4	118	3.2	1
Total		6,275	115	48	432	12	5

Note:

1) Assumes approx widening width on average of 450-ft in the Kansas City district.

2) Assumes ultimate chute width of approximately 300-ft in the Kansas City district.

2.3.3 *IRC*

Total acres of SWH added to HEC-RAS to meet 260 ac/yr for the IRC geometry were calculated by assuming a 13 year construction window. This is based on a 15-year implementation period, and the assumption that habitat construction will begin no more than 2 years after the ROD is signed. Distribution was based on conversations with the Effects Analysis team. The highest density of habitat was located between the Nebraska Platte River and Osage Rivers (RM 595 to 130), with the remainder spread between the Sioux City to the Platte River and Osage to the Mouth Reaches. Table 2-6 shows the acres added to HEC-RAS by BiOp segment, for easy comparison to the other two geometries. Between both districts, the total target acres of habitat for the IRC configuration is about 1,000 acres less compared to the No Action, but the distribution is more heavily targeted on the lower river. For the Kansas City district, total acres added to the Rulo to Mouth HEC-RAS model was 2,519 acres, slightly more than the No Action geometry and with more habitat downstream of Kansas City.

For both districts, the IRC target SWH acreage was achieved by widening only. The breakdown of habitat type for the Rulo to the mouth only is shown in Table 2-7. Chutes are still shown in the table because they were added to the HEC-RAS geometry, even though they were not counted toward the target acreage. The total number of cross sections modified downstream of Rulo, Nebraska was 148 (114 for widening and 34 for chutes) of the 879 Missouri River cross sections.

Table 2-6. Acres of SWH – IRC Geometry

Reach	Segment	RM Start	RM End	Miles	Acres of SWH added to HEC-RAS
Ponca to Sioux City	11	753	735	18	0
Sioux City to Platte River	12	735	595	140	276
Platte River to Rulo (Omaha)	13	595	498	97	585
Rulo to Kansas River (KC)	13	498	367	131	670
Kansas River to Osage River	14	367	130	237	1,389
Osage River to Mouth	15	130	0	130	460
Omaha District				255	861
Kansas City District				498	2,519
Total				753	3,380

Table 2-7. SWH Type – IRC Geometry

Reach	Segment	Widening (acres)	Widening (miles) [1]	Number of Widening Projects	Chutes (acres)	Chutes (miles)[2]	Number of Chute Projects
Rulo to Kansas R	13	670	18.4	8	221	6.1	2
Kansas R to Osage R	14	1,389	38.2	15	93	2.6	2
Osage R to Mouth	15	460	12.7	6	118	3.2	1
Total		2,519[3]	69	29	432	12	5

Note:

1) Assumes approx widening width on average of 300-ft in the Kansas City district.

2) Assumes ultimate chute width of approximately 300-ft in the Kansas City district.

3) Target acres entirely accomplished by top width widening, even though chutes were still added to the model.

2.3.4 *Public Ownership Acreage Analysis*

To assist the economic analysis team, an analysis was performed to calculate the amount of existing publicly owned lands available for new habitat construction. This information was used to determine the amount and cost of additional lands that would need to be purchased. Existing public lands includes USACE, USFWS, and state conservation owned lands suitable for habitat development, and was accumulated by counting publicly owned river front acres available in each BiOp segment, so existing public land availability is a constant between alternatives. Values in the table were capped at the target. Suitability of particular sites for the type of habitat to be constructed was not considered. Furthermore, the additional lands required acreage is habitat only, a factor would need to be used to convert from habitat land to total real estate tract purchase size. See Table 2-8, Table 2-9, and Table 2-10 for a listing of public ownership acreage availability for each alternative geometry.

Table 2-8: Public Ownership Acreage - No Action Alternative

		Widening (ac)			Backwaters (ac)			Total
Reach	Seg	Target (ac)	Exist Public Lands (ac)[2]	Addt'l Lands Req'd (ac)	Target (ac)	Exist Public Lands (ac)[2]	Addt'l Lands Req'd (ac)	Addt'l Lands Req'd (ac)[1]
Ponca to Sioux City	11	180	0	180	60	0	60	**240**
Sioux City to Platte River	12	601	601	0	420	420	0	**0**
Platte River to Rulo (NWO)	13	672	672	0	0	0	0	**0**
Rulo to Kansas River (NWK)	13	1,129	454	675	-	-	-	**675**
Kansas River to Osage River	14	937	937	0	-	-	-	**0**
Osage River to Mouth	15	0	0	0	-	-	-	**0**
Omaha District		1,453	1,273	180	480	420	60	**240**
Kansas City District		2,066	1,391	675	-	-	-	**675**
Total		3,519	2,664	855	480	420	60	**915**
Total Percentage		-	76%	24%	-	87.5%	12.5%	**-**

Note:

1) Note that chutes (NWK) and changes to existing SWH (NWO) are not shown because they reside on existing public lands.

2) Includes existing public lands that are available for habitat placement. This number is capped at the target acres and is not the total acres of existing public lands.

Table 2-9: Public Ownership Acreage - BiOp Alternative

		Widening (ac)			Backwaters (ac)			Total
Reach	**Seg**	**Target (ac)**	**Exist Public Lands (ac)[2]**	**Addt'l Lands Req'd (ac)**	**Target (ac)**	**Exist Public Lands (ac)[2]**	**Addt'l Lands Req'd (ac)**	**Addt'l Lands Req'd (ac)[1]**
Ponca to Sioux City	11	240	0	240	180	0	180	**420**
Sioux City to Platte River	12	1,761	836	925	660	660	0	**925**
Platte River to Rulo (NWO)	13	1,582	907	675	60	60	0	**675**
Rulo to Kansas River (NWK)	13	2,439	454	1,985	-	-	-	**1,985**
Kansas River to Osage River	14	3,307	1,375	1,932	-	-	-	**1,932**
Osage River to Mouth	15	529	529	0	-	-	-	**0**
Omaha District		3,583	1,743	1,840	900	720	180	**2,020**
Kansas City District		6,275	2,358	3,917	-	-	-	**3,917**
Total		9,858	4,101	5,757	900	720	180	**5,937**
Total Percentage		-	42%	58%	-	80%	20%	**-**

Note:

1) Note that chutes (NWK) and changes to existing SWH (NWO) are not shown because they reside on existing public lands.

2) Includes existing public lands that are available for habitat placement. This number is capped at the target acres and is not the total acres of existing public lands.

Table 2-10: Public Ownership Acreage - IRC Alternative

		Widening (ac)			Backwaters (ac)			Total
Reach	Seg	Target (ac)	Exist Public Lands (ac)[2]	Addt'l Lands Req'd (ac)	Target (ac)	Exist Public Lands (ac)[2]	Addt'l Lands Req'd (ac)	Addt'l Lands Req'd (ac)[1]
Ponca to Sioux City	11	0	0	0	-	-	-	**0**
Sioux City to Platte River	12	276	276	0	-	-	-	**0**
Platte River to Rulo (NWO)	13	585	585	0	-	-	-	**0**
Rulo to Kansas River (NWK)	13	670	454	216	-	-	-	**216**
Kansas River to Osage River	14	1,389	1,375	14	-	-	-	**14**
Osage River to Mouth	15	460	460	0	-	-	-	**0**
Omaha District		861	861	0	-	-	-	**0**
Kansas City District		2,519	2,289	230	-	-	-	**230**
Total		3,380	3,150	230	-	-	-	**230**
Total Percentage		-	93%	7%	-	-	-	**-**

Note:

1) Note that chutes (NWK) and changes to existing SWH (NWO) are not shown because they reside on existing public lands.

2) Includes existing public lands that are available for habitat placement. This number is capped at the target acres and is not the total acres of existing public lands.

2.4 Excluded Areas

Locations selected for habitat construction in the HEC-RAS model are theoretical and do not reflect actual locations of future mitigation projects. However, the following areas were intentionally avoided when making modifications to HEC-RAS models:

1) Reaches of river within a 10,000-ft radius of an airport (FAA, Aug 2007)

2) Areas within 1/4 mile upstream or downstream of small town infrastructure along the river bank, on that side of the river only

3) Areas within 1/4 mile upstream or downstream of power plant or municipal water intakes, on both sides of the river

4) Areas within 1/4 mile upstream or downstream of barge loading facilities and other river related industrial infrastructure along the river bank, on that side of the river only

5) Areas within 1/4 mile upstream or downstream of bridges, on both sides of the river

6) Riverfront property near Federal/PL 84-99 levees within 1,000-ft of the river bank, on that side of the river only.

7) Reaches adjacent to larger cities where the channel is confined by urban levees

8) The outside edge of river bends, because based on past experience most widening and or chute projects in the Kansas City District have been constructed on the inside of river bends.

9) Widening projects were not located in the same bend as new or existing chutes or widening to avoid excessive navigation channel flow loss.

2.5 Reference Flows

Reference flows were used to set construction elevations for SWH and chutes added to the geometries. Reference flows were run through the 2012 existing condition geometry in steady flow, and the resulting water surface profile along with the specified depth was used to set the invert for added SWH.

2.5.1 *No Action*

Reference flow for the No Action geometry was the 50% exceedance flow in the month of August. August 50% exceedance was evaluated for the *2014 Shallow Water Accounting Report* at mainstem Missouri River gages as well as the Kansas River at DeSoto and the Osage River at St. Thomas (USACE, Sept 2014). Flow changes in HEC-RAS were made at major tributaries based on the *Upper Mississippi River System Flow Frequency Study* (UMRSFFS) accumulated basin size to the confluence (USACE, 2003). Table 2-11 lists the flows used by reach.

Table 2-11. Reference Flows – No Action

From	To	Gage Evaluated	From (RM)	To (RM)	August 50% Exceedance Flow (cfs)
Tarkio	Big Nemaha	Rulo	507	495	41,900
Big Nemaha	Nodaway		495	463	43,000
Nodaway	Platte	St Joseph	463	391	44,800
Platte	Kansas		391	367	45,200
Kansas	Waverly	Kansas River at DeSoto / Kansas City	367	293	52,500
Waverly	Grand	Waverly	293	250	52,900
Grand	Chariton		250	239	53,500
Chariton	Lamine		239	202	55,300
Lamine	Moreau	Boonville	202	138	57,700
Moreau	Osage		138	130	58,100
Osage	Gasconade	Osage at St Thomas	130	104	60,300
Gasconade	Mouth	Hermann	104	0	66,900

2.5.2 *BiOp*

Three different reference flows were used for the BiOp geometry: 1) summer low, 2) median August, and 3) spring pulse. Flows were calculated by starting with the Gavins Point release and adding incremental flows at major tributaries. Incremental flows were calculated by the MRBWMD in the *Missouri River Incremental Flows Below Gavins Point Technical Report* (USACE, July 2014) for the pre-dam and post-dam time periods, at statistical levels of minimum, median, and maximum, lower and upper decile and quartiles. Median pre-dam records were used because they incorporate the drought of the 30's and are therefore slightly lower than the post-dam statistic. Incremental flows for the months of July, August, and May, were used for summer low, median August, and spring pulse, respectively. July incremental flows were added downstream to the summer low Gavins Release, because the summer low condition centers primarily on this month. May incremental flows were added downstream to the spring pulse because the second spring pulse specified occurs during this month. Changes to the reservoir operations for each alternative as modeled in Res-Sim are discussed in more detail in Section 3.

Release from Gavins for the summer low and spring pulse were selected to be in concert with the Res-Sim model rules, with the intention of constructing habitat at an effective level for those conditions. Release from Gavins for the Median August reference flow was pulled from the

MRBWMD *Hydrologic Statistics Technical Report* (USACE, Sept 2013). Table 2-12 lists the Kansas City District flows by reach, as well as the release from Gavins for reference.

Table 2-12. Reference Flows – BiOp

From	To	USGS Gage Location	Summer Low [1] (cfs)	Median August [2] (cfs)	Spring Pulse [3] (cfs)
Gavins Release			21,000	36,900	48,800
Tarkio	Big Nemaha	Rulo	32,700	45,000	63,900
Big Nemaha	Nodaway		33,553	45,594	64,790
Nodaway	Platte	St Joseph	35,000	46,600	66,300
Platte	Kansas		35,451	46,890	66,777
Kansas	Crooked	Kansas City	43,700	52,200	75,500
Crooked	Grand	Waverly	44,000	52,400	75,900
Grand	Chariton		45,929	54,037	80,577
Chariton	Lamine		46,505	54,525	81,973
Lamine	Moreau	Boonville	47,300	55,200	83,900
Moreau	Osage		48,223	55,823	85,868
Osage	Gasconade		53,958	59,692	98,098
Gasconade	Mouth	Hermann	55,600	60,800	101,600

Note:
1) Flows for summer low were increased downstream of Gavins by adding median July incremental flows as calculated by the MRBWMD
2) Flows for Median August were increased by adding median August
3) Flows for Spring Pulse were increased by adding median May

Median August differs slightly from the August 50% exceedance used for No Action, this is because August 50% exceedance was based off an independent evaluation of daily flow records at each major gage location, whereas the incremental flows used to for the median August flow were produced by the Daily Routing Model (DRM), applying a routing coefficient to daily flow records and comparing two different major gage locations.

2.5.3 *IRC*

Reference flow for the IRC geometry was the median June, with release from Gavins and incremental flows as calculated in the MRBWMD technical reports (USACE, July 2014) (USACE, Sept 2013). Median June flows in the Kansas City reaches tend to be higher compared with July and August. Table 2-13 lists the Kansas City District flows by reach and the release from Gavins.

Table 2-13. Reference Flows – IRC

From	To	USGS Gage Location	Median June Flow (cfs)
Gavins Release			32,200
Tarkio	Big Nemaha	Rulo	49,800
Big Nemaha	Nodaway		50,802
Nodaway	Platte	St Joseph	52,500
Platte	Kansas		53,158
Kansas	Crooked	Kansas City	65,200
Crooked	Grand	Waverly	65,600
Grand	Chariton		69,517
Chariton	Lamine		70,686
Lamine	Moreau	Boonville	72,300
Moreau	Osage		73,923
Osage	Gasconade		84,012
Gasconade	Mouth	Hermann	86,900

2.6 TOP WIDTH WIDENING

Some form of top width widening was the primary means of adding SWH to all three geometries. Widening width, depth, and design invert elevation varied by alternative and by position on the river.

Widening areas offer full conveyance for flow in the model. This impacted three model parameters: overbank ineffective areas, levee points, and permanent ineffective areas in the channel. 1) Ineffective areas in the overbank impacted by the widening were moved back to the limit of widening at the same elevation, making the assumption that the widening would not affect the elevation at which the overbank conveys flow. 2) If there were levee points in the overbank impacted by the widening, these were also moved back to the limit of widening and left at the same elevation, making the assumption that any minor agricultural levees (levees not modeled using lateral weirs and storage areas) impacted by the widening would be set back and offer the same level of protection. 3) Permanent Ineffective flow areas in the channel that represent navigation structures (only applies to the Rulo to Mouth model) were truncated, assuming that during construction navigation structures would be modified to provide some level of conveyance through the widening area. An example cross section where all three parameters were adjusted is shown in Figure 2-1. These assumptions were carried through all three geometries.

Widened areas in all three geometries were assigned a Manning's n-value of 0.035, which is rougher than the main channel because it accounts for structure modifications that may be necessary to maintain the habitat and adjacent navigation channel to the desired dimensions. This assumed Manning's n-value is lower than the roughness in the overbanks.

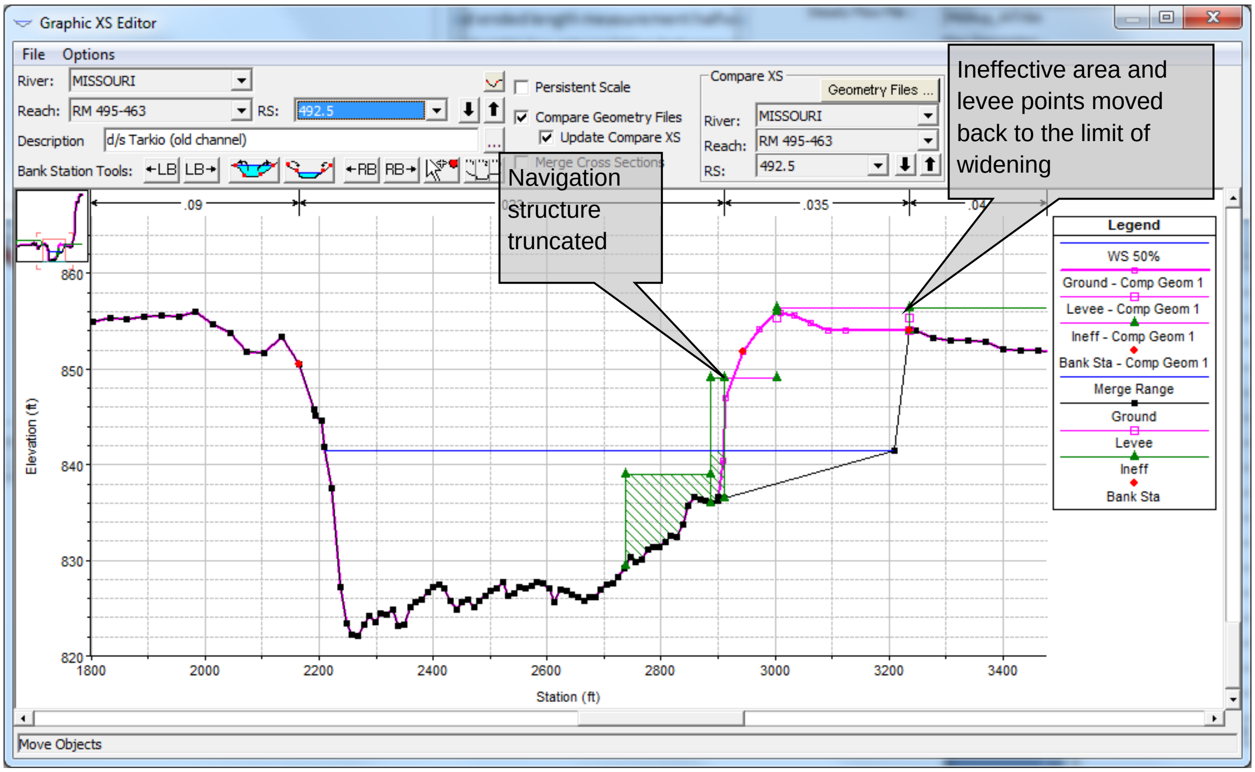


Figure 2-1. Ineffective flows and levee points in widened cross section

Preliminary modeling suggests widening could cause local aggradation in the main channel. This potential localized aggradation of the bed, if it occurred, could be a concern if it results in the loss of the authorized channel dimensions or was significant enough to offset the added flow conveyance from the river widening during flood flows. However, this would be prevented on a project by project basis with detailed hydraulic modeling and adjustments to structures incorporated into the design. A full geomorphic sediment model would be necessary to determine if considerable widening could cause system wide changes.

Example widened cross sections for each geometry are shown in the following sections.

2.6.1 *No Action*

Top width widening for the No Action geometry was added to the HEC-RAS model in each affected cross section as shown in Figure 2-2. Cross sections were widened 300-ft downstream of Rulo (Kansas City District). Depths range from 5 to 0-ft with respect to the August 50% water surface elevation at each cross section.

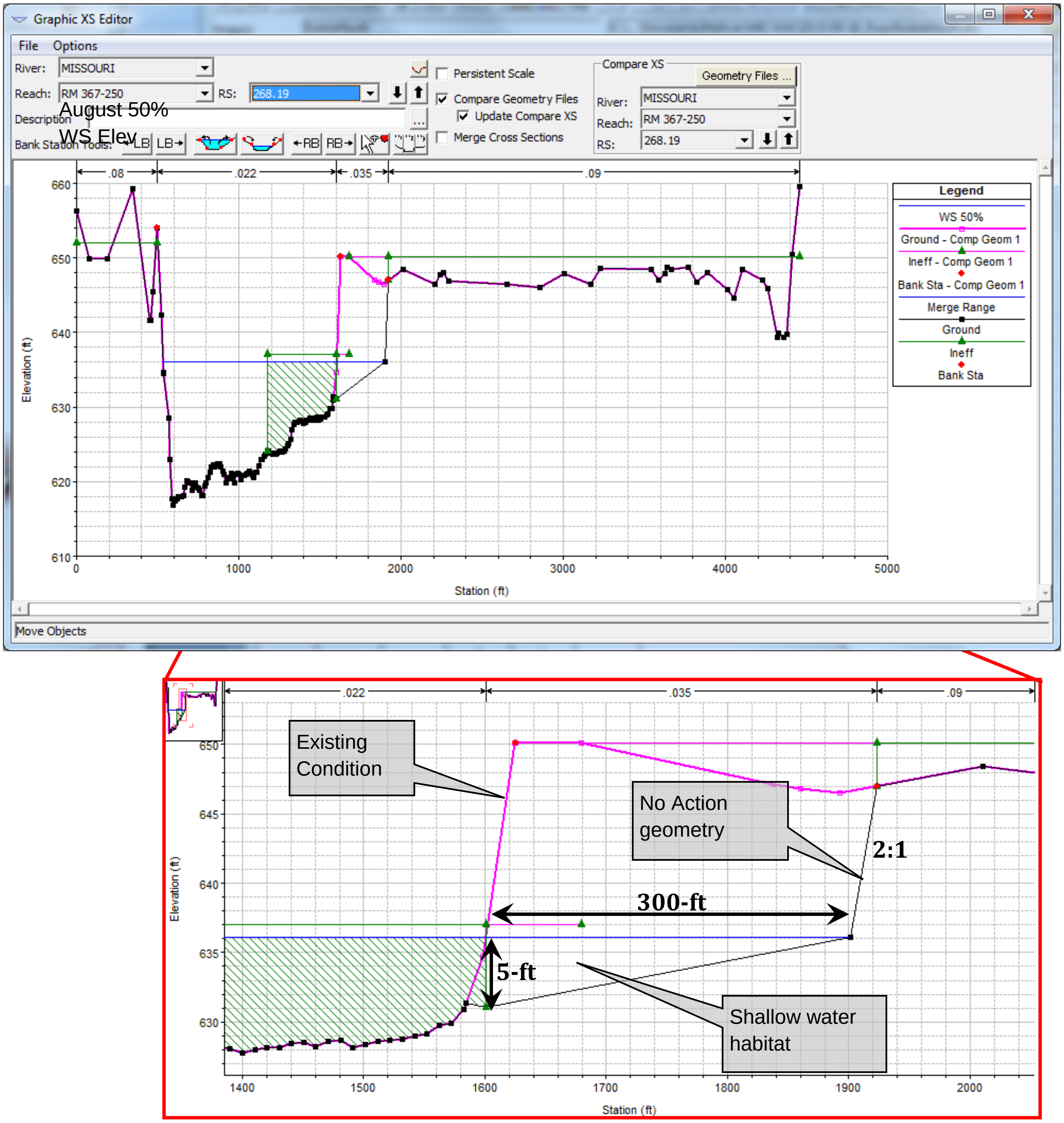


Figure 2-2. Widened cross section – No Action

2.6.2 *BiOp*

Top width widening for the BiOp geometry was added to the HEC-RAS model in each affected cross section as shown in Figure 2-3. Downstream of Rulo (Kansas City District), cross sections were widened a total of 450-ft, allowing for three 150-ft benches of SWH for each of the three flow regimes. Depths range from 0 to no more than 5-ft on each bench with respect to the reference water surface elevation.

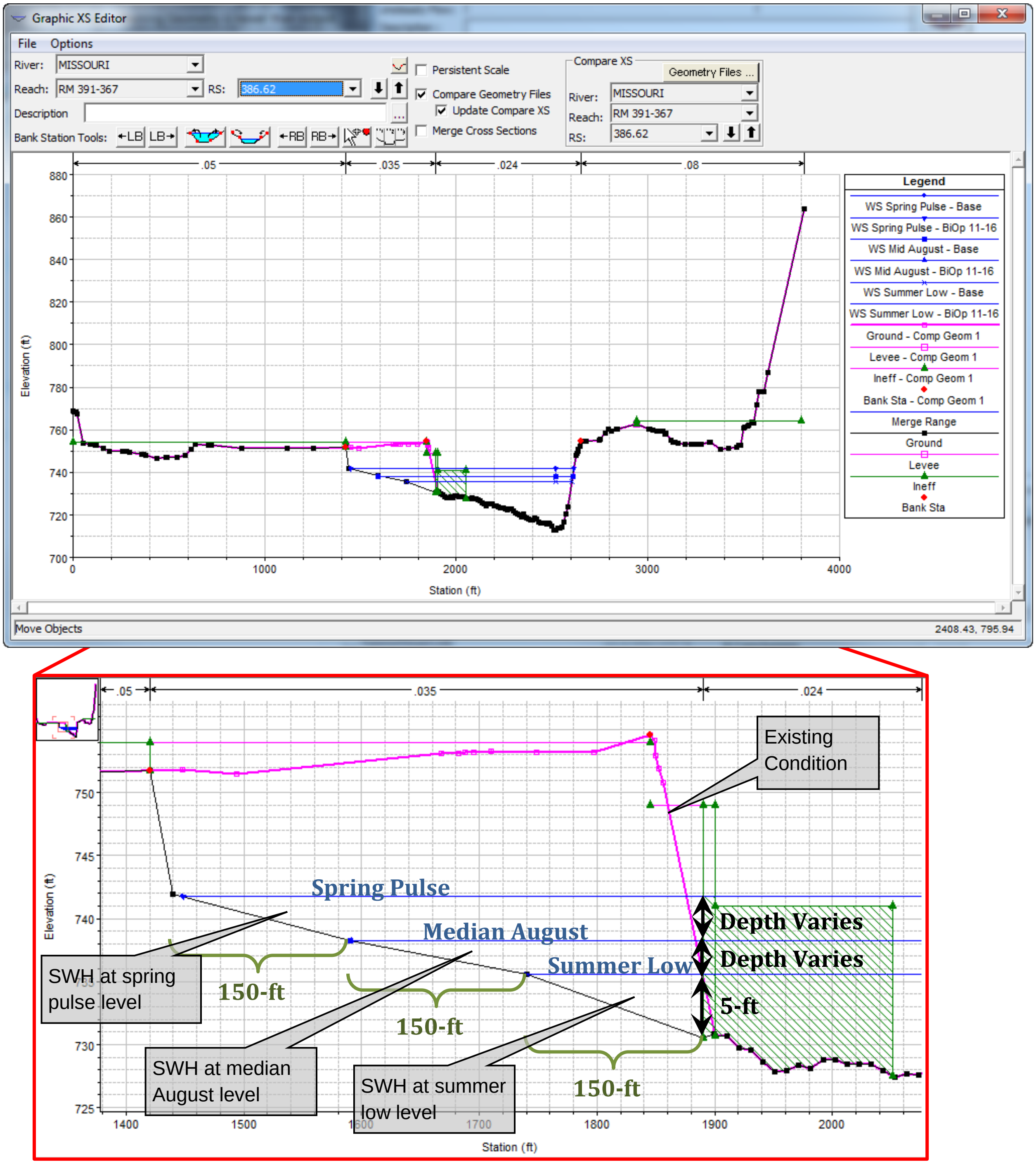


Figure 2-3. Widened cross section – BiOp

Water surface profiles for each of the three reference flows did not exactly parallel each other due to the variability of cross section shape and flow changes. HEC-RAS's template tool was used to add the widening to the model, so it was necessary to select a uniform template with identical widths and depths, even though the depths between profiles vary. After examining the difference between profiles, shown in Figure 2-4, three different templates were created for each BiOp reach. Depth selected were based on the average distance between profiles.

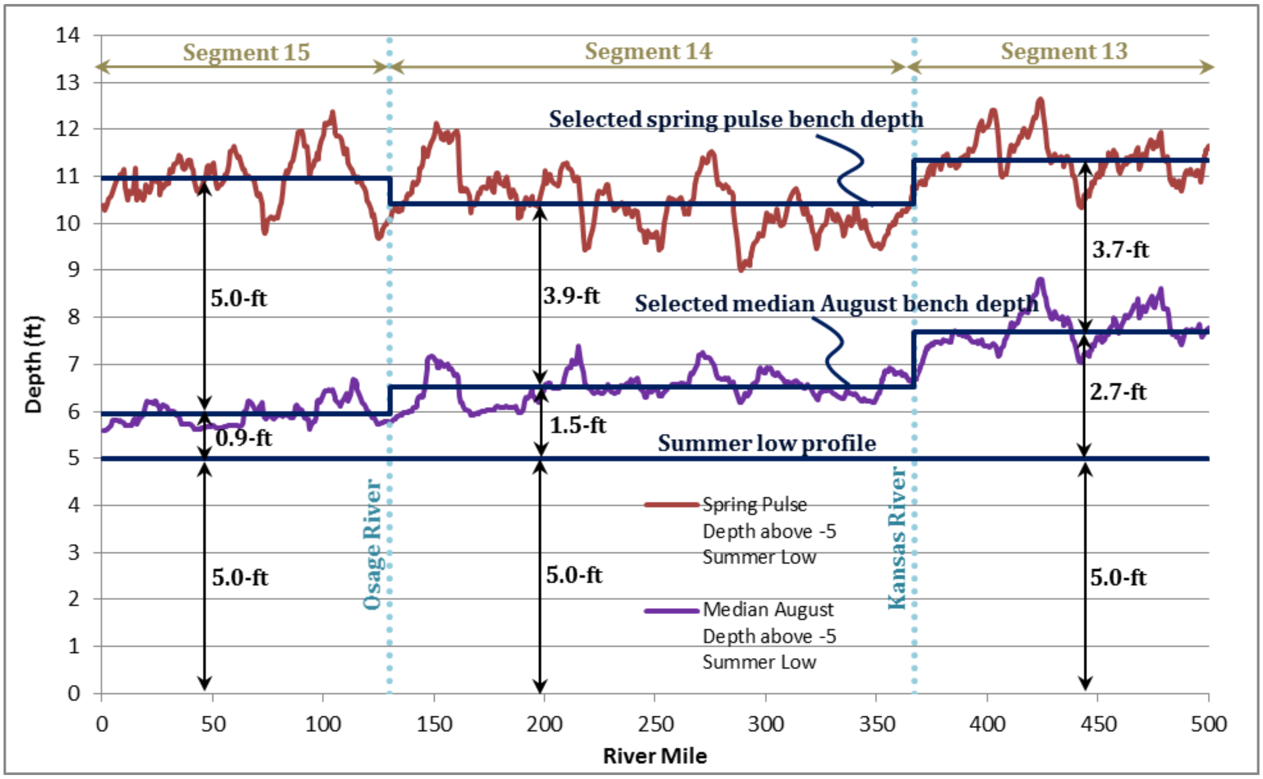


Figure 2-4. Widening Depths - BiOp

An average bench depth for each BiOp reach means that sometimes the reference profile was shallower than the top of bench and sometimes deeper than the top of bench. An exact method would have been time intensive, requiring manual editing of each of the 188 cross sections by hand to exactly match the reference profile. Considering the natural variability of the flows seen by the Missouri River from year to year, an exact methodology would not necessarily add value to the analysis. Summer low, median August, and spring profiles themselves are based on a statistical approximation of median flow conditions, any given year the actual flows for the specified time period could be higher or lower than the statistic. The methodology chosen should hit the average of conditions over the period of record.

Each habitat bench is sloped, so the depth over the 150-ft width varies from 0-ft to the depth shown in Figure 2-4 and repeated in tabular form in Table 2-14.

Table 2-14. Widening Depths - BiOp

	Segment 13		Segment 14		Segment 15	
	Width (ft)	**Depth (ft)**	**Width (ft)**	**Depth (ft)**	**Width (ft)**	**Depth (ft)**
Spring pulse bench	150	3.7	150	3.9	150	5.0
Median August bench	150	2.7	150	1.5	150	0.9
Summer low bench	150	5	150	5	150	5
Total	**450**	**11.4**	**450**	**10.4**	**450**	**11.0**

2.6.3 *IRC*

Top width widening for the IRC geometry was added to the HEC-RAS model in each affected cross section as shown in Figure 2-5. Cross sections were widened 300-ft downstream of Rulo (Kansas City District). Depths range from 6 to 0-ft with respect to the Median June water surface elevation at each cross section.

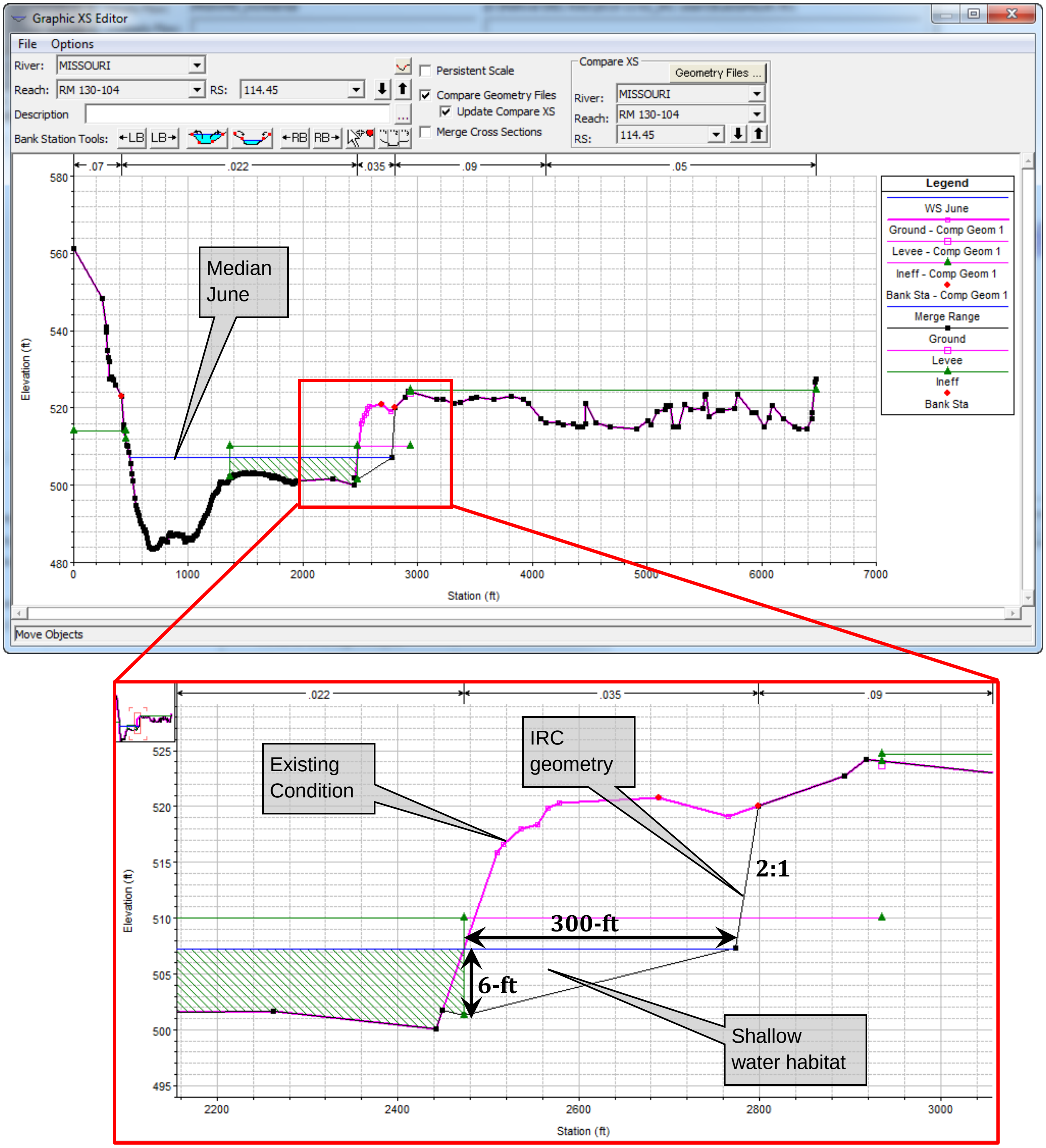


Figure 2-5. Widened cross section – IRC

2.7 Chutes

Changes to chutes in the HEC-RAS models were identical for all three geometries. Only chutes constructed or awarded to construction between the existing condition 2012 geometry and present day 2015 were added to HEC-RAS, because recent emphasis has been on top width widening projects rather than chute projects. Table 2-15 lists all known chutes in the Kansas City District. The list includes natural chutes, constructed chutes, chutes that have been designed but may never be constructed, as well as the modeling approach selected for alternatives.

Table 2-15. Chutes

Chute	Origin		Modeling Approach				Notes
	Natural	USACE Project	In Existing Condition 2012 Geometry	Added new chute in Alter-natives Geometry	Evolved in Alter-natives Geometry	Did not Add	
Wolf Creek		x				x	Design is on hold as of Fall 2015
Worthwine		x	x		x		
Benedictine		x		x			Under construction 2015 - 2016
Dalbey		x		x			Constructed 2013
Baltimore		x				x	Design is on hold as of Fall 2015
Cranberry		x		x			Under construction 2015 - 2016
Cranberry (existing)	x		x				
Lisbon	x		x				
Jameson		x	x		x		
Jameson Extension		x		x			
Franklin Island	x		x				Chutes have partially silted in
Overton North		x	x		x		
Tadpole		x	x		x		
Smokey Waters		x	x				No future development anticipated
St Aubert	x		x				
Tate Island	x		x				
Lunch Island	x		x				
No name (RM 60)	x		x				
No name (RM 57)	x		x				

Howell Island/Centaur Chute	x		x				
Johnson Island	x		x				
Bonhomme Chute	x		x				
No name (RM 36)	x		x				
Bryan Island	x		x				
Pelican	x		x				
Littles	x		x				
Cora Island		x		x			Under construction 2015 - 2016

New chutes that were added to the Rulo to Mouth reach for all alternatives included: Benedictine, Dalbey, Cranberry, the Jameson Extension, and Cora Island. Wolf Creek and Baltimore have some level of design completed but are currently on hold (as of fall 2015) and therefore were not included. Dalbey was added because construction was completed in 2013 so it was not included in the 2012 existing condition geometry.

Chutes are represented in the HEC-RAS model as added overbank conveyance in cross sections that intersect the chute alignment. A single chute shows up in anywhere from 1 to 6 cross sections, depending on the length of the chute and cross section arrangement. Chutes were given a triangular shaped channel bottom 300-ft wide ranging in depth from 0 to 5-ft at August 50% for all three geometries as shown in Figure 2-6. Chute areas were assigned a Manning's n-value of 0.028, which is generally slightly rougher than the main channel, but not as rough as the 0.035 selected for widened habitat.

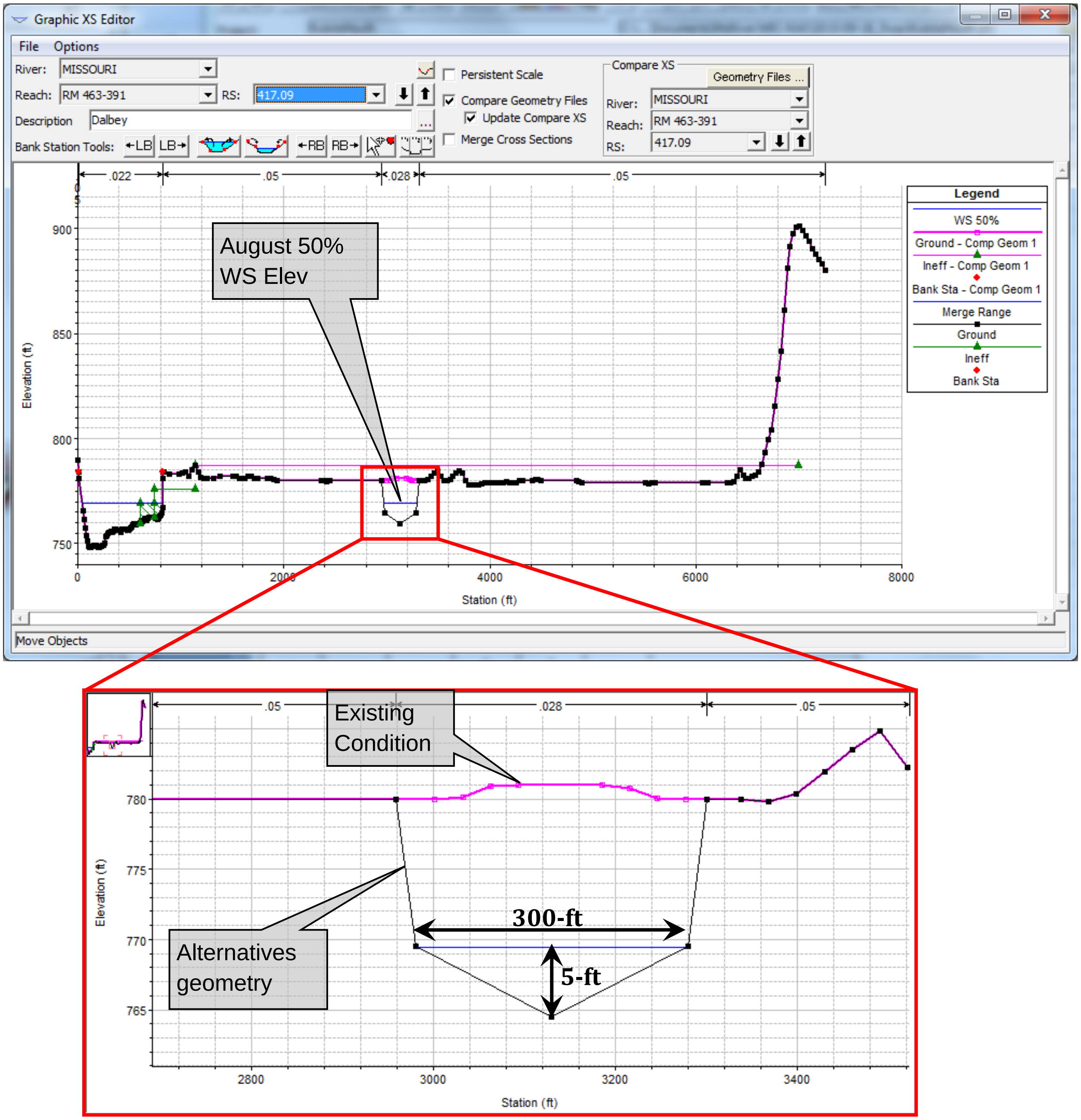


Figure 2-6. Chute cross section

Assumptions with regards to levee points and ineffective areas at cross sections where chutes were added were site specific and reflect best as possible the on-the-ground post construction conditions.

All previously constructed chutes that are in the baseline geometry were widened to the same fully developed dimensions as the new chutes, 300-ft wide, triangular shaped channel bottom with depths ranging from 0-ft to 5-ft. This only applies to four chutes from Rulo to the Mouth: Worthwine, Jameson, Overton North, and Tadpole. All three are in the existing conditions geometry with inverts at -10 CRP, based on approximations from surveyed depths. However, the assumption was still made to essentially raise the bottom of the chutes by 5-ft, assuming that the chutes will be continue to be managed until they achieve the original design goal of shallower depths on average.

Natural chutes on the river (21 total represented in existing conditions model) were not evolved because future conditions would be difficult to forecast or they are assumed to have already reached a relatively stable state.

2.8 FLOODPLAIN CONNECTIVITY

Floodplain connectivity at a 20% annual chance exceedance (ACE) (or 5-yr) was assessed as a requirement of Alternative 2. A mapping analysis was conducted, although no changes to the HEC-RAS model were necessary.

Coordination with the USFWS produced a Planning Aid Letter detailing the modeling assumptions for the BiOp alternative. Per the criteria set forth in the 2003 Amended BiOp (USFWS, Dec 2003), the total authorized acreage for the Missouri River Fish and Wildlife Mitigation Program is 166,750-ac. In the 2003 BiOp and Planning Aid Letter, it was assumed that 100,000 of the 166,750-ac would be utilized for SWH and floodplain connectivity. Per the USFWS Draft Program Management Plan (USACE, Apr 2007), the total authorized acreage was split by state, and as applied to floodplain connectivity numbers are reported in the first column of Table 2-16.

Mapping of existing floodplain connectivity was performed in HEC-GeoRAS. A steady profile was run in HEC-RAS for the 20% ACE (5-yr), and also for the 50% ACE (2-yr) for comparison, using flows from UMRSFFS (USACE, 2003). GIS data was exported from HEC-RAS, and Geo-RAS was used to create an inundation boundary. The bounding polygon had to be edited for a clean mapping product, and other manual edits were made to exclude areas that could not be counted as floodplain connectivity. Full procedure followed to count acres of floodplain connectivity is included in **Attachment 2 – Floodplain Connectivity Metadata**. Calculated floodplain connectivity acres by state are listed for each district and total in Table 2-16. Existing floodplain connectivity acres of 147,650-ac surpasses the 100,000-ac goal of floodplain connectivity, therefore no changes were made to the HEC-RAS model.

Table 2-16. Acres of Floodplain Connectivity by State Downstream of Sioux City, IA

State	Portion of authorized acres available for floodplain connectivity and SWH [1]	Existing Acres of floodplain connectivity (20% ACE inundation) [2, 3]			Additional acres of floodplain habitat to add to HEC-RAS models
		NWO	NWK	Total	
Nebraska	15,983	31,550	270	31,820	0
Iowa	14,228	16,120	-	16,120	0
Kansas	6,976	-	8,560	8,560	0
Missouri	62,813	8,020	83,130	91,150	0
Total	100,000[1]	55,690	91,960	147,650	0

Note:

1) The 100,000 authorized acreage goal includes both SWH and floodplain connectivity.

2) Does not imply ownership, includes both public and private land.

3) The calculated acres of floodplain connectivity shown in the table above includes chutes but excludes main channel acres defined by median August flows, both defined as SWH. Because of this inconsistency, the calculated acres were compared to the full portion of authorized acres, rather than trying to separate SWH from floodplain connectivity, and was considered acceptable because existing exceeded authorized even without counting some of the SWH acres.

In the Kansas City district, total floodplain connectivity at the 5-yr was calculated at 91,960 acres. For comparison, the floodplain connectivity at 2-yr is 32,050-ac in the Kansas City District, approximately one third of the 5-yr inundation acres.

The calculated acres of existing floodplain inundation with connectivity includes:

- Areas lower in elevation than the computed 5-yr water surface and judged to be connected to the main channel
- Private lands not protected by levees, including fringe areas between levees and river bank and areas without any discernable protection that would be inundated at the reference flow.
- Existing chutes as of the year 2012

Areas excluded from the existing floodplain connectivity acres:

- Area behind all active/maintained levees, including federal levees, levees in the PL84-99 program and smaller agricultural levees often found between the federal/ program levees and the river bank. No distinction was made as to levee reliability or performance risk.

- Disconnected areas and areas judged not to have obvious connection to the main channel at the 5-yr
- Inundated area well outside the bluff line or in tributary backwater areas
- Missouri River main channel as determined by the boundary of the August 50% duration flow extent shapefile obtained from the *2014 HAMP SWH Accounting Report* (USACE, Sept 2014). The total amount of existing habitat for the Kansas City District alone is 8,232-ac, which does include chutes. Chute acres were added by hand for the report, and this calculation was not easily transferable to the floodplain analysis.
- Current BSNP mitigation land behind levees, namely the downstream portion of Eagle Bluffs, where the site is actively managed to provide connectivity via structures.

Selected locations that demonstrate the inundation mapping as well as included and excluded areas are shown in the following figures.

Figure 2-7 is the area at and just upstream of the Nodaway River confluence at where Federal levees 482-R and 488-L are located. The area has a mix of both major and minor levees that were excluded, and also demonstrates that the Nodaway River backwater area was not included beyond the bluff line.

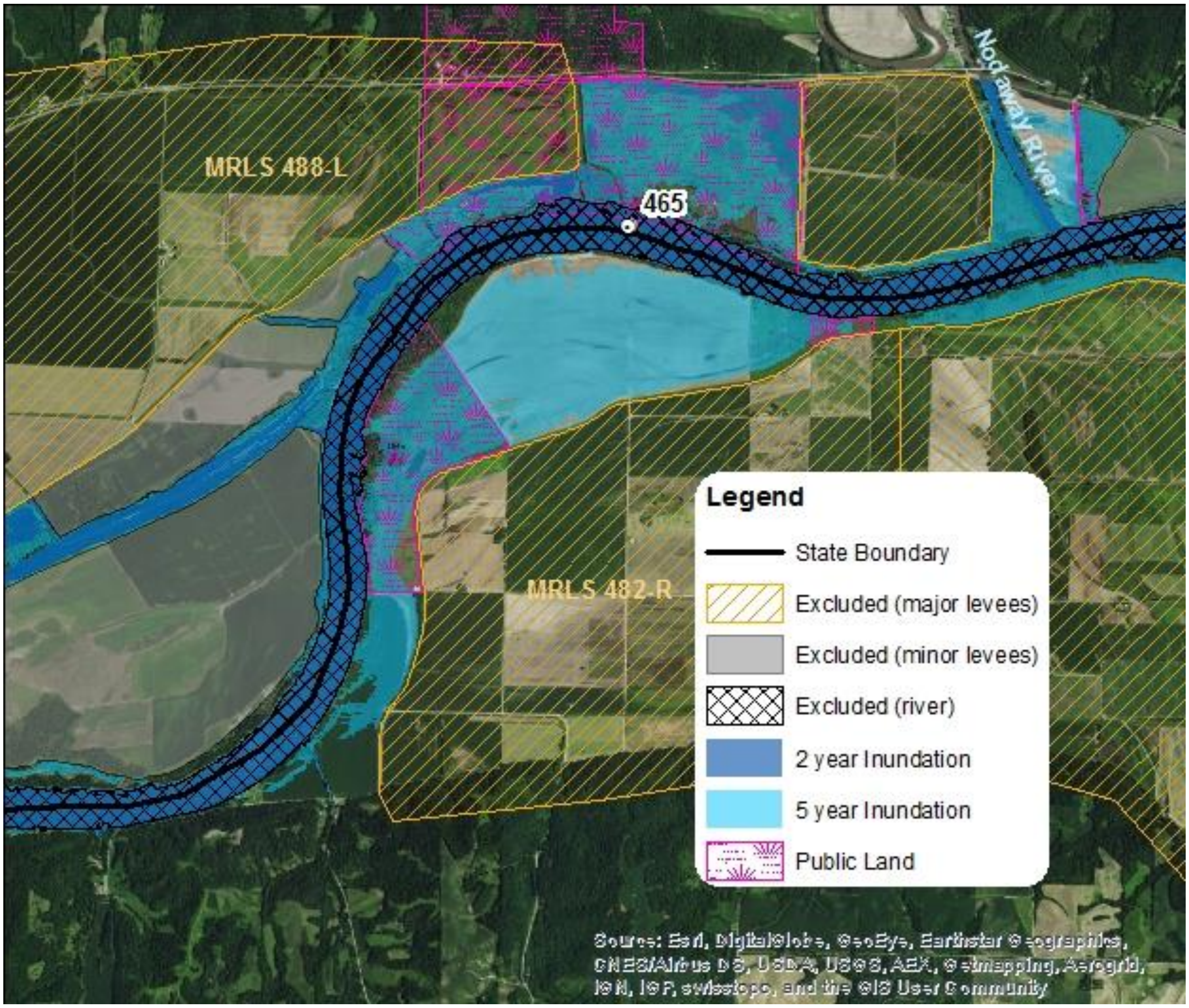


Figure 2-7. Floodplain Connectivity

Figure 2-8 shows the Baltimore Bottoms Unit of the Big Muddy National Fish and Wildlife Refuge, an area owned by the USFWS. At this location the old levees surrounding the refuge have been cut or left unrepaired following the May 2007 flood event to allow for floodplain connectivity. This area appears to be inundated at a 2-yr level with slivers of connection to the main channel.

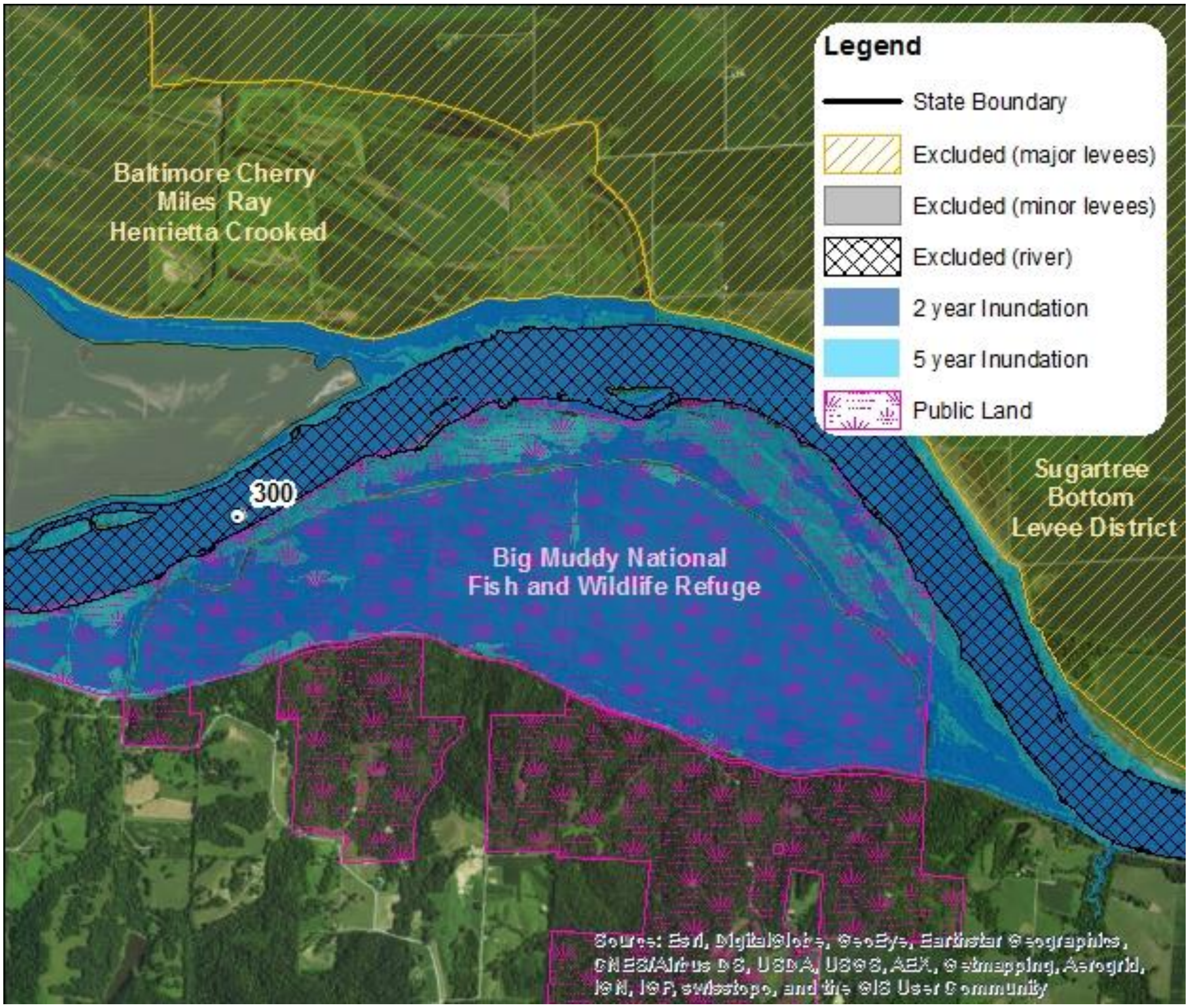


Figure 2-8. Floodplain Connectivity

Figure 2-9 shows the Lisbon (north / left bank) and Jameson Island (south / right bank) Units of the Big Muddy National Fish and Wildlife Refuge. A naturally formed chute is present at Lisbon, whereas USACE constructed a chute at Jameson Island. Note that while the river was excluded in the floodplain calculation, the chute areas were counted. Also note the tieback area between the two Howard County PL84-99 program leveed areas was excluded and is labeled as a major levee. All tiebacks between levees were modeled as storage areas, and to simplify the floodplain calculation no storage areas were included in the inundation mapping, even though these tieback areas are technically connected to the river.

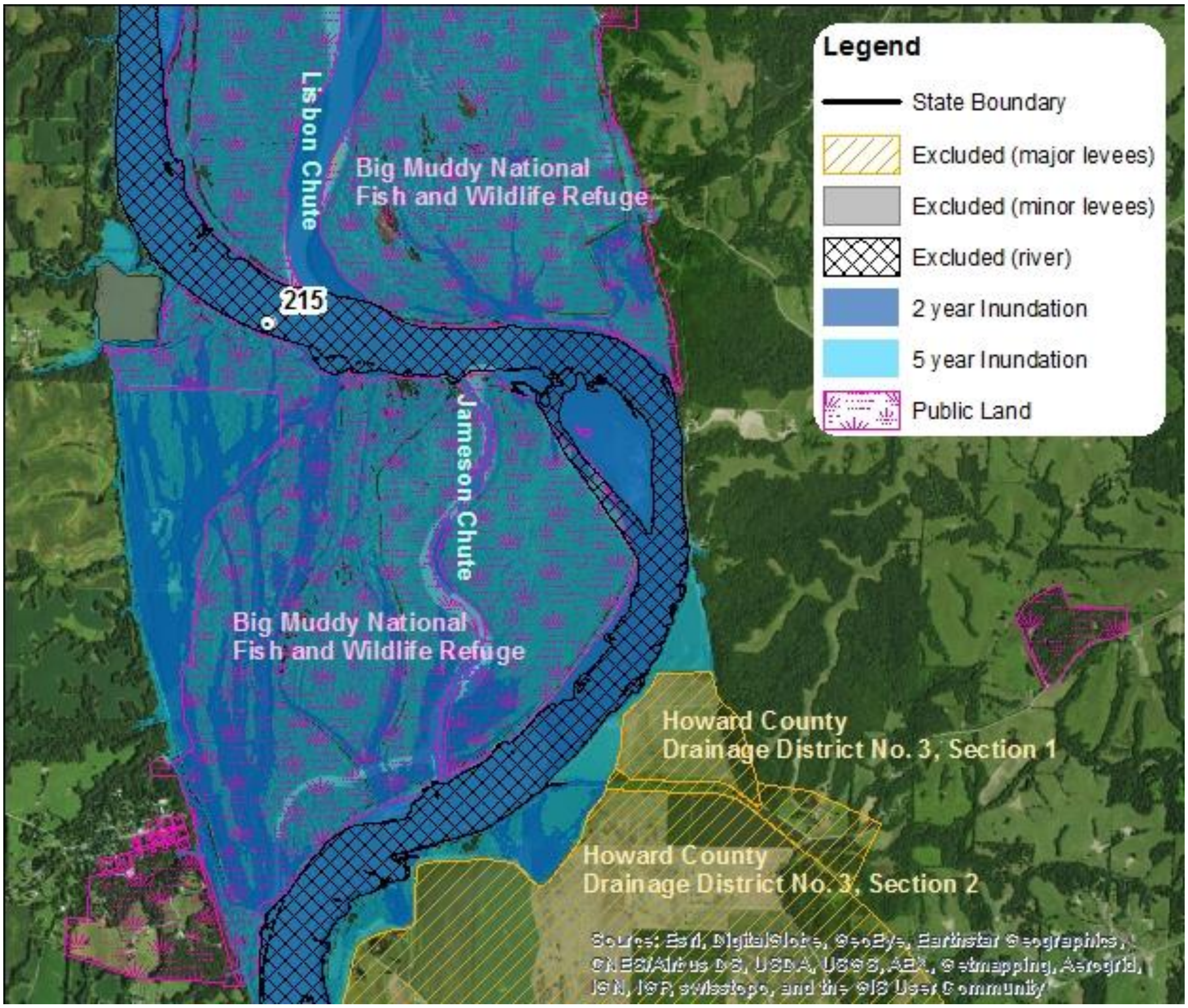


Figure 2-9. Floodplain Connectivity

Figure 2-10 is at Pelican Island (right bank state owned Conservation Area) and Little's chutes (left bank in private ownership). There is an unintentional mapping gap over Pelican Island, due to a gap in the terrain data. This figure provides another example of where the chute areas were included while the river was excluded.

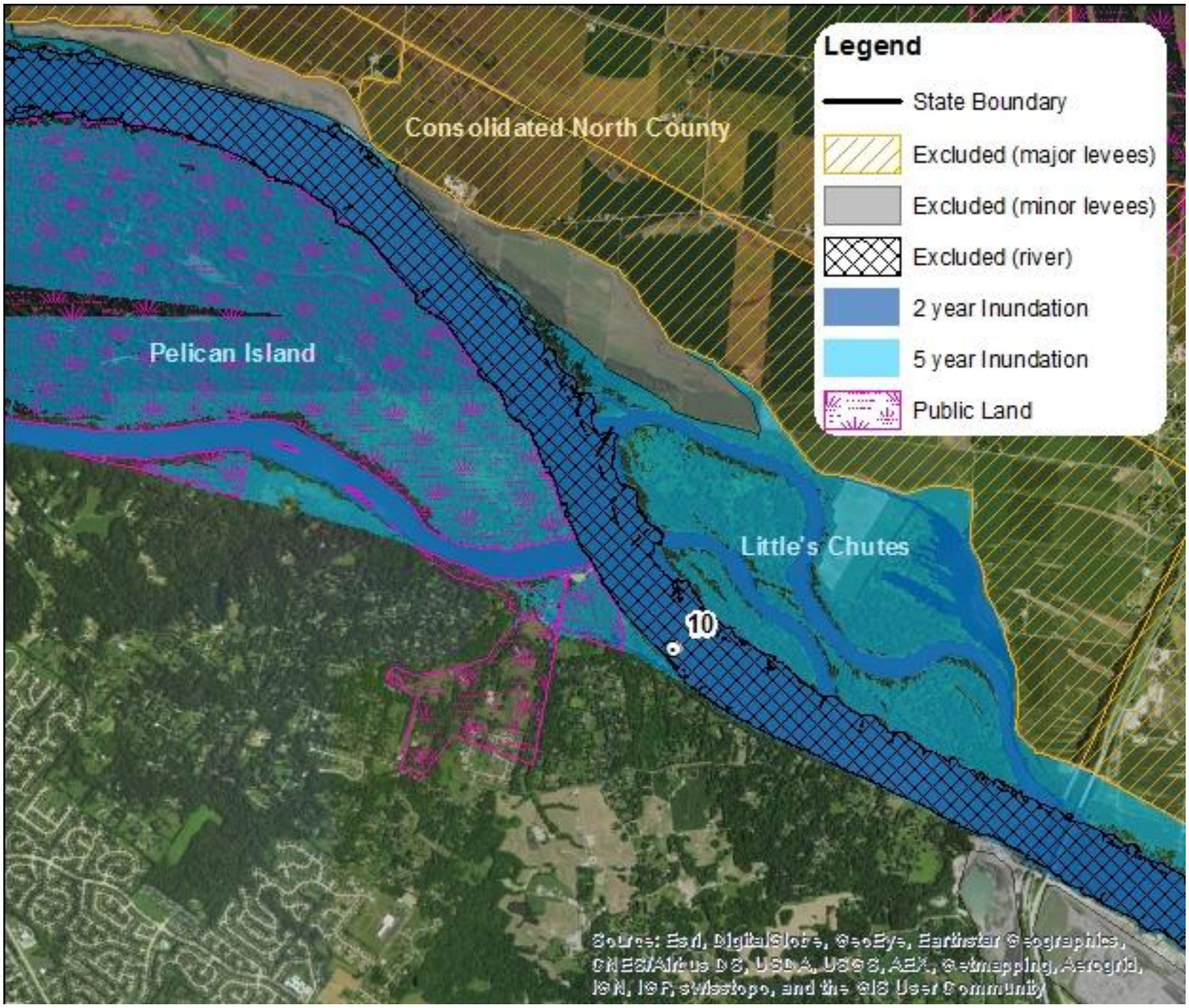


Figure 2-10. Floodplain Connectivity

The full 5-yr and 2-yr inundation boundaries are available in the electronic files accompanying the report.

3 FLOW CHANGES

Changes to the reservoir operations for each alternative were modeled using rule changes in the Res-Sim model(s). The resulting daily flow hydrograph out of Gavins was then run through the appropriate Gavins to Rulo HEC-RAS model for each alternative before being run though the corresponding Rulo to Mouth model. The hydrograph handoff location was at Nebraska City because of the overlap between the two districts. One period of record run was made for each model for each of the six alternatives. Refer to *Mainstem Missouri River Reservoir Simulation Alternatives Technical Report* (USACE, 2018) for details regarding the Res-Sim modeling.

For quick reference, the following list and table summarizes the flow releases/pulses as they impact the lower river and the Kansas City district reaches. The full Res-Sim document should be referenced for complete details as there are many complexities and layers to the Res-Sim rules. Each simulation makes specific checks on system storage, runoff forecasts, flood control targets, navigation targets, navigation season length, water supply targets, and other parameters, all of which can limit or eliminate the releases/pulse in any given year. Table 3-1 summarizes the ranges of releases from Gavins point, as well as the number of years in which the release was eliminated, partially or fully completed.

1) **Alt 1 No Action –** Master Manual flows including a plenary bimodal spawning cue attempted each year, one in March and one in May

2) **Alt 2 BiOp –** two spawning cues attempted each year, one in March and one in May, plus a summer low flow from June 23 lasting as late as September 1 that only occurs in the two years following complete March & May pulses

3) **Alt 3 Mech** – same rules as No Action except no plenary bimodal spawning cue (no pulse)

4) **Alt 4 Spring 2** – high spring release starting in April to create sandbar habitat, duration ranging from 35 to 175 days depending on release magnitude

5) **Alt 5 Fall 5** – high fall release starting October 15 to create sandbar habitat, duration ranging from 35 to 175 days depending on release magnitude

6) **Alt 6 Spawn Cue** – two spawning cutes attempted every 3 years, one in March and one in May

Table 3-1. Gavins Releases per Alternative

<table>
<tr><th>Alternative</th><th>Start Month</th><th>Release from Gavins (kcfs) [1]</th><th>Eliminated (# of years)</th><th>Partial (# of years)</th><th colspan="2">Full/ completed (# of years)</th></tr>
<tr><td rowspan="2">Alt 1 No Action</td><td>March</td><td>23 – 35</td><td>55</td><td>9</td><td>19</td><td rowspan="2">11</td></tr>
<tr><td>May</td><td>25 – 41</td><td>57</td><td>10</td><td>16</td></tr>
<tr><td rowspan="2">Alt 2 BiOp</td><td>March</td><td>31</td><td>69</td><td>9</td><td>5</td><td rowspan="2">4</td></tr>
<tr><td>May</td><td>38 – 56</td><td>55</td><td>7</td><td>21</td></tr>
<tr><td>Alt 3 Mech</td><td>-</td><td>-</td><td>-</td><td>-</td><td colspan="2">-</td></tr>
<tr><td>Alt 4 Spring 2</td><td>April</td><td>45 – 60</td><td>63</td><td>7</td><td colspan="2">17</td></tr>
<tr><td>Alt 5 Fall 5</td><td>October</td><td>45 – 60</td><td>74</td><td>2</td><td colspan="2">14</td></tr>
<tr><td rowspan="2">Alt 6 Spawn Cue</td><td>March</td><td>39 – 61</td><td>47</td><td>16</td><td>20</td><td rowspan="2">6</td></tr>
<tr><td>May</td><td>50 – 67</td><td>70</td><td>5</td><td>8</td></tr>
</table>

Note:

1) Based on Res-Sim POR simulations

4 RESULTS

All alternatives runs were performed in HEC-RAS 5.0.3 in August 2017. All of the results included in this report have been updated with the most current output.

Model output contains a considerable amount of information, not easily condensed to simple conclusions. Each of the six alternative runs produced 82 years (March 1930 – December 2012) of stage and flow hydrographs at 1,192 cross sections and stage hydrographs at 338 storage areas between Rulo and the Mouth. Responses to the Res-Sim flow changes in combination with habitat geometry changes are complex.

To express the changes compared with the No Action alternative, the model results were evaluated by 1) statistical evaluation comparing min, max, and percentiles, and 2) duration analysis plots. Statistical and duration evaluations were made at five locations along the river, the Missouri at St. Joseph, Kansas City, Boonville, and Hermann, and the Mississippi River at St. Louis. Each is fairly representative of hydraulic reaches bounded by the major tributaries to the Missouri. St Joseph represents the reach upstream of the Kansas River, Kansas City represents downstream of the Kansas River, Boonville captures the effects of flow out of the Grand/Chariton Rivers, and Hermann is representative of the lower river, below the Osage and Gasconade Rivers.

Stage and flow hydrographs at all cross sections and all storage areas were passed to economists and biologists for evaluation of impacts to human considerations and habitat interests.

4.1 STATISTICS

Tables comparing min, max, and percentile flows and stages at the five major locations are provided in **Attachment 3 – Rulo to Mouth Alternative Statistics**. Significance of the statistics are explained below using St. Joseph as an example. St. Joseph statistics are repeated in the body of the text and in the attachment.

In the attachment, the top two tables provide the minimum, maximum, 10, 25, 50, 75 and 90 percentile statistical flows on the period of record hydrographs. Flow statistics are on the left, stage statistics are on the right. Table 4-1 and

Table 4-2 show the statistics at St. Joseph.

Table 4-1. Flow (cfs) statistics on the period of record at St. Joseph

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	6,969	6,968	6,969	6,969	6,969	6,969
10%	18,007	18,097	18,029	17,887	18,007	17,859
25%	25,043	24,819	25,096	24,688	25,003	24,665
50%	39,546	39,203	39,456	39,303	39,336	39,416
75%	50,485	50,555	50,457	50,760	50,533	51,109
90%	68,611	69,488	68,735	70,548	69,742	69,657
Max	289,014	292,309	283,954	283,928	283,930	283,933

Table 4-2. Stage (ft) statistics on the period of record at St. Joseph

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	787.7	787.7	787.7	787.7	787.7	787.7
10%	791.2	791.3	791.2	791.2	791.2	791.2
25%	793.4	793.3	793.4	793.3	793.4	793.3
50%	797.4	797.3	797.4	797.4	797.4	797.4
75%	799.6	799.4	799.6	799.7	799.7	799.7
90%	802.5	802.3	802.6	802.8	802.8	802.7
Max	820.3	819.1	820.2	820.2	820.2	820.2

Min and max are the lowest daily flow/stage and the highest daily flow/stage output by the model in each alternative over the period of record. The lowest flow/stage in the period of record, and how the alternatives may impact this for the better or worse is an important result for several interest groups, including navigation and water supply. The highest flood flow/stage, and how the alternatives may impact these at various locations along the river is of interest to agricultural and flood damage interests. However, caution should be used when trying to draw conclusions from this statistics table alone, especially for maximum and peak flood flows in the Rulo to Mouth reach because of the interaction of river flow with leveed areas. The FIA models produced by the economists that compute structural and agricultural damages from flood events will provide a more complete picture of how the alternatives impact the lowest and highest flows/stages because they incorporate all cross sections and all storage areas along the river, whereas these tabular statistics only capture one location.

Flow and stage changes between alternatives at a certain location are influenced by an array of variables. From alternative to alternative the two primary changes to the hydraulic model were 1) flow out of the Mainstem Missouri River reservoir system as calculated by HEC-ResSim, and 2) the habitat additions to the river geometry. Flow calculated by the HEC-RAS model at a downstream location not only depends on how the Gavins Point release changed, but also how those changes carry downstream. Even when the Gavins Point release has no change, the flow or stage calculated from alternative to alternative at a downstream location may change because of the habitat additions to the river geometry.

There is evidence that habitat construction on a reach of river tends to lower the river stages in the vicinity and slightly upstream of the habitat location and generally have a minor dampening effect on the hydrograph, lowering peak flows downstream of the habitat location (Jacobson, Linder, & Bitner, 2015). However, this simple correlation between habitat and flow/stage does not always hold exactly true, both because of the complexities of the Missouri River system and the nature of unsteady HEC-RAS modeling. Flow changes appear in the results in unexpected and often unexplainable ways.

One factor that influences the timing and magnitude of flows at all levels is the interaction of downstream conveyance features that represent storage. Storage interacts with the conveyance in the main river by taking on water when the river is rising, and returning water when the river is falling. At low flows, this interaction happens with the navigation structures represented by permanent ineffective areas in the channel. At bank full flows, this interaction happens with storage areas that represent tributaries and tiebacks with low lying connection to the river. And at the highest of flows, this interaction happens at a more extreme magnitude with large protected areas behind levees modeled in HEC-RAS with storage areas. River widening for habitat may seem small compared to a fully inundated floodplain, but even slight differences in the river water surface elevation could change the interaction of the river with the storage areas, and alter the timing and magnitude of water to reach a certain location. Without extensive testing and sensitivity runs it is difficult to get a handle on the magnitude of influence of storage on the system.

It is also important to note that the HEC-RAS alternative models have been configured to report one value per 24 hour period, and unfortunately that one value is not a daily average as in HEC-ResSim. The HEC-RAS model computes stage and flow every ten minutes for the entire period of record, but only reports the value that lands on 2400 of each day. The most reasonable output interval was chosen as daily due to the size of watershed being modeled, POR length, and the number of hydrograph locations necessary for HC analysis. This means that slight shifts in timing from alternative to alternative can carry over into the results as small fluxuations in the reported peak flow. Changes in timing are a small factor, not likely to significantly impact any results evaluation, but should be kept in mind when making comparison at a precise level such as in the statistics tables. In the attachment the second table down for each gage calculates the difference between the alternative and the No Action Alternative 1 for each statistic. As an example, the different tables for St. Joseph are shown below as Table 4-3 and Table 4-4. If the value in this table is positive, the alternative resulted in an increase from No Action. If the value in this table is negative, the alternative resulted in a decrease from No Action.

Table 4-3. Change in Flow (cfs) statistic compared to No Action at St. Joseph

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-1	0	0	0	0
10%	-	90	21	-120	0	-148
25%	-	-224	53	-355	-40	-378
50%	-	-343	-90	-244	-211	-130
75%	-	70	-28	275	49	624
90%	-	877	125	1,937	1,131	1,046
Max	-	3,295	-5,060	-5,086	-5,084	-5,081

Table 4-4. Change in Stage (cfs) statistic compared to No Action at St. Joseph

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	-0.1	0.0	-0.1	0.0	-0.1
50%	-	-0.2	0.0	0.0	0.0	0.0
75%	-	-0.2	0.0	0.1	0.0	0.1
90%	-	-0.3	0.1	0.3	0.2	0.2
Max	-	-1.1	0.0	-0.1	-0.1	-0.1

Stage statistics have been rounded to the nearest tenth of a foot, which is equivalent to 1.2 inches. This helps demonstrate how flow changes impact river elevations, which is the more tangible result.

The third and last table for each gage is quite a bit different than the first two tables. For the entire period of record, the difference from alternative to No Action on each day was calculated, and then the statistics were calculated on that new dataset. As an example, same day change tables are shown for St. Joseph as Table 4-5 and Table 4-6. The minimum and maximum in this table should be thought of as the biggest 1 day reduction or 1 day increase in flow or stage over the period of record.

Table 4-5. One day change in Flow (cfs) at St. Joseph

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-38,478	-11,844	-28,804	-44,309	-32,537
10%	-	-2,089	-80	-1,127	-506	-1,129
25%	-	-700	-17	-102	-47	-137
50%	-	-16	0	-1	0	-1
75%	-	35	22	6	13	6
90%	-	646	132	79	116	100
Max	-	43,838	16,986	42,686	61,366	44,249

Table 4-6. One day change in Stage (ft) at St. Joseph

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-5.6	-2.4	-6.5	-8.0	-7.8
10%	-	-0.9	0.0	-0.2	-0.1	-0.2
25%	-	-0.3	0.0	0.0	0.0	0.0
50%	-	-0.2	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.1	0.1	0.1	0.1
Max	-	6.8	2.9	7.1	6.9	8.1

The magnitudes of the maximum and minimum flows in the one day change tables look extreme compared to the period of record statistics, but the numbers alone are almost meaningless without knowledge of where in the spectrum of low to high these flows occurred. As an example, both Alternatives 4 & 5 had daily changes in flow compared with Alternative 1 of magnitudes exceeding 20-30,000-cfs for many days during the year 1994. Flow hydrograph output at St. Joseph in 1994 are presented in Figure 4-1.

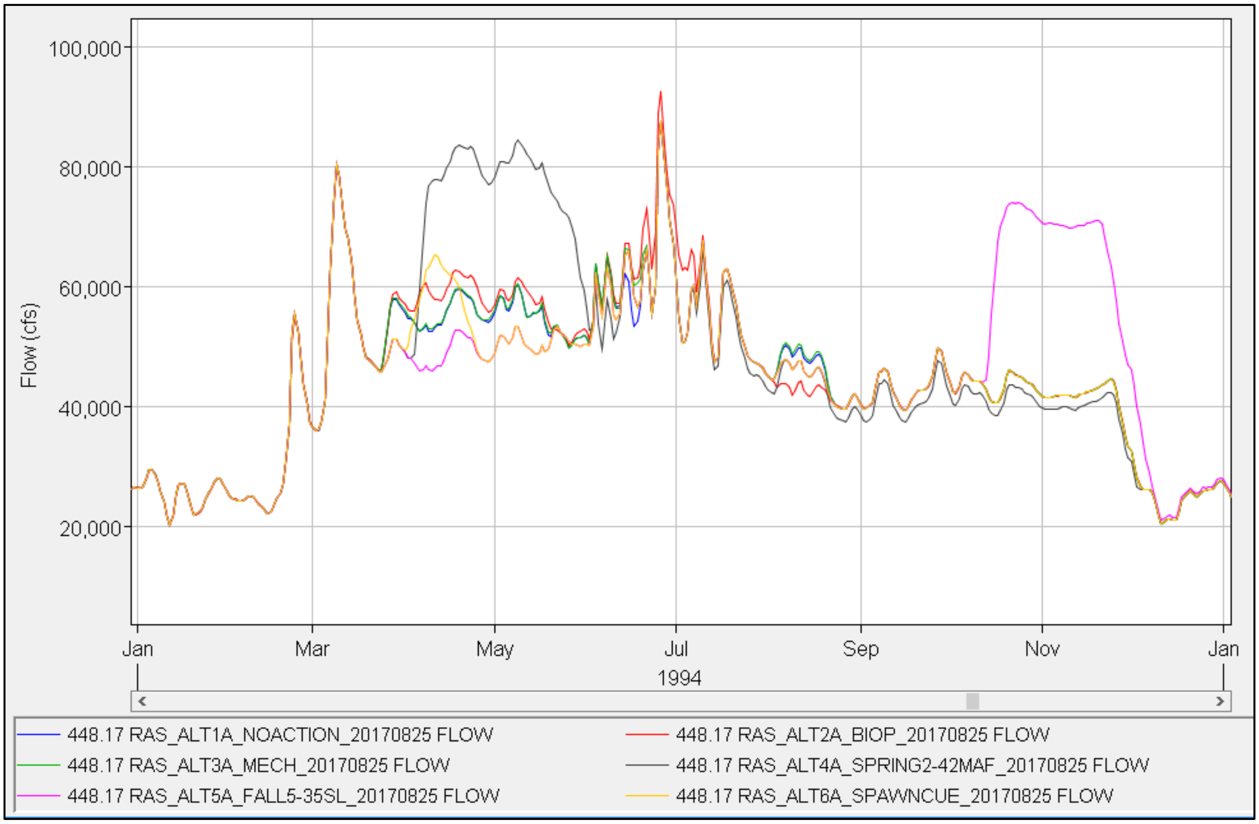


Figure 4-1. Alternatives Flows at St. Joseph – 1994

For context, at St. Joseph the 2-yr flow from UMRSFFS is 109,000-cfs, which is about bank full, and 50% exceedance during the month of August is 44,800-cfs. The 20-30,000-cfs flow changes during 1994 seen in Alternatives 4 & 5 occur well within the confines of the bank of the river. Significance of these changes with regards to stakeholders is addressed within the economic evaluations.

4.2 Seasonal Duration Plots

Plots comparing percent of time alternative flows and stage equal or exceed a range of values are provided in **Attachment 4 – Rulo to Mouth Alternative Flow Duration Plots** and **Attachment 5 – Rulo to Mouth Alternative Stage Duration Plots**. Plots compare alternatives at the five major locations by season. Seasons are defined as spring (01-Mar to 30-Apr), summer (01-May to 31-Aug), fall (01-Sep to 30-Nov), and winter (01-Dec to 28-Feb), and coincide with the seasons as defined for the reservoir system operational rules.

For the flow plots, the difference between the alternative curves and No Action are the largest at St. Joseph, and become smaller moving downstream as the effects of release changes out of Gavins Point are masked by incremental flows from tributaries. Winter flows decrease in the alternatives with respect to the No Action, whereas spring and summer seasons see increases, and fall has a little bit of both.

There is more variability in the stage plots, as this incorporates both flow timing/ magnitude changes intertwined with river widening habitat changes. However, a similar trend was observed as flows with the largest variability between alternatives being at the upstream gage and decreasing with distance downstream from Gavins Point.

4.3 Limitations

The analysis relies on the simulation of the 82 year period of record using daily average outflows from a HEC-ResSim model input into fixed bed HEC-RAS models, with stage and flow values pulled at the same time each day from 10 minute computation. While the analysis coupled with species and human considerations models can be used to show relative benefits and potential impacts based on historic flows, there are limitations in the conclusions that can be drawn based on some of the simplifying assumptions.

1) **Uncertainty Analysis –** A probabilistic risk and uncertainty assessment was not performed on the combined HEC-ResSim and HEC-RAS model output. Pulses and release changes only occur in 12% of the years on the low end (Alternative 2) to 21% of the years on the high end (Alternative 4) in the period of record, as noted in Table 3-1. The potential flood damages from coincident downstream flows occurring during the spring or fall pulses can only be considered for this smaller subset of the period of record of 10 to 18 years. Without a Monte Carlo type 1,000-year plus simulation, it is difficult to express with even moderate confidence the risk associated with the flow and river geometry changes made for the alternatives. Statistics and percentages calculated based on the 82 years of record should therefore be used with caution, and with the understanding of the consequences of using only a small sample of years.

2) **Stationarity of the flow record –** The period of record ranges from the extreme drought of the 1930s, to the floods of 1993 and 2011, however, it's unknown what the flow conditions of the next 50-years will bring, and whether or not they will be comparable to the conditions experienced over the last 82 years. Climate change analysis performed for the Missouri River basin indicates that earlier snowmelt can be expected along with increasing trends in extreme floods and droughts (USACE, 2016b). Given the same criteria for releasing a spring or fall pulse, the percentage of years in which a pulse is released could be quite different, and in turn the economic impact could be quite different as well.

3) **Stable Bed and Floodplain –** The hydraulic modeling to date is based on revisions to the existing conditions hydraulic model to account for varying amounts and distributions of habitat through river widening and continued development of SWH projects such as chutes. The analysis does not account for how the bed of the Missouri River may respond to river widening activities. Additionally, the analysis does not try to project where sediment may accumulate in the floodplain or include projections of future change

in floodplain roughness. This carries with it the necessary assumptions that any bed and floodplain changes would be either negligible, similar between each alternative, or mitigated during more detailed design of river widening projects.

4) **Flood source –** The Missouri River, major tributaries, and ungaged inflows included in the HEC-RAS model are the only flood sources, water only floods areas protected by levees from levee overtopping, and levees do not breach. In reality, flooding can also occur due to localized rainfall and subsequent runoff, through seepage under levee foundations during prolonged high water, or through failure of levees prior to overtopping. Past experience on the Missouri River indicates that a majority of levee breaches occur following levee overtoppings when water flows into or exits the levee system. However, breaches prior to overtopping have occurred and may occur again in the future. Predicting breach formation through the period of record simulation was considered infeasible in part due to ongoing repairs and improvements made by levee sponsors or USACE after each event. Not including breaches can underestimate potential flood damages in some cases. While the level of effort required to model all levees systems with detailed rainfall runoff and underseepage flow calculations was considered prohibitive, more detailed analysis was conducted for four levee systems, two in Omaha District and two in Kansas City District. The two Kanas City District interior drainage models are documented in Section 5.

5) **Level of detail** – The HEC-RAS model is a one dimensional unsteady flow calculation, which means flow only moves in one direction and velocity components in directions other than the direction of flow are not accounted for. Additionally, cross-sections were spaced a half mile apart on the Missouri River. This means any changes in the river between cross-sections is not captured by the model.

5 INTERIOR DRAINAGE ANALYSIS

Crop damage as a result of levee overtopping was accounted for in the primary modeling effort, but damage to crops behind levees can be caused by other mechanisms. Typical Missouri River levee systems have culverts to allow local drainage to exit the interior of the levee and drain to the river. Each culvert typically would include one or more closures, such as a flap gate or sluice gate, to prevent river water from backing up into the leveed area. When river levels are higher than the culvert outlets and this coincides with heavy local rainfall, ponding water can cause flooding on the interior of the levee. Additionally, when river levels are above the interior ground level, seepage through the ground under the levee can also cause flooding on the interior.

To simulate these types of flooding, and measure differences between the proposed alternatives and the No Action alternative, a sub-set of the seven sites evaluated for the Master Manual (USACE, Aug 1998) were modeled in detail. Four sites were selected, L-575 and L-536 in the Omaha district and L-488 and L-246 in the Kansas City district. Figure 5-1 shows an area map with the locations of the four sites on the river.

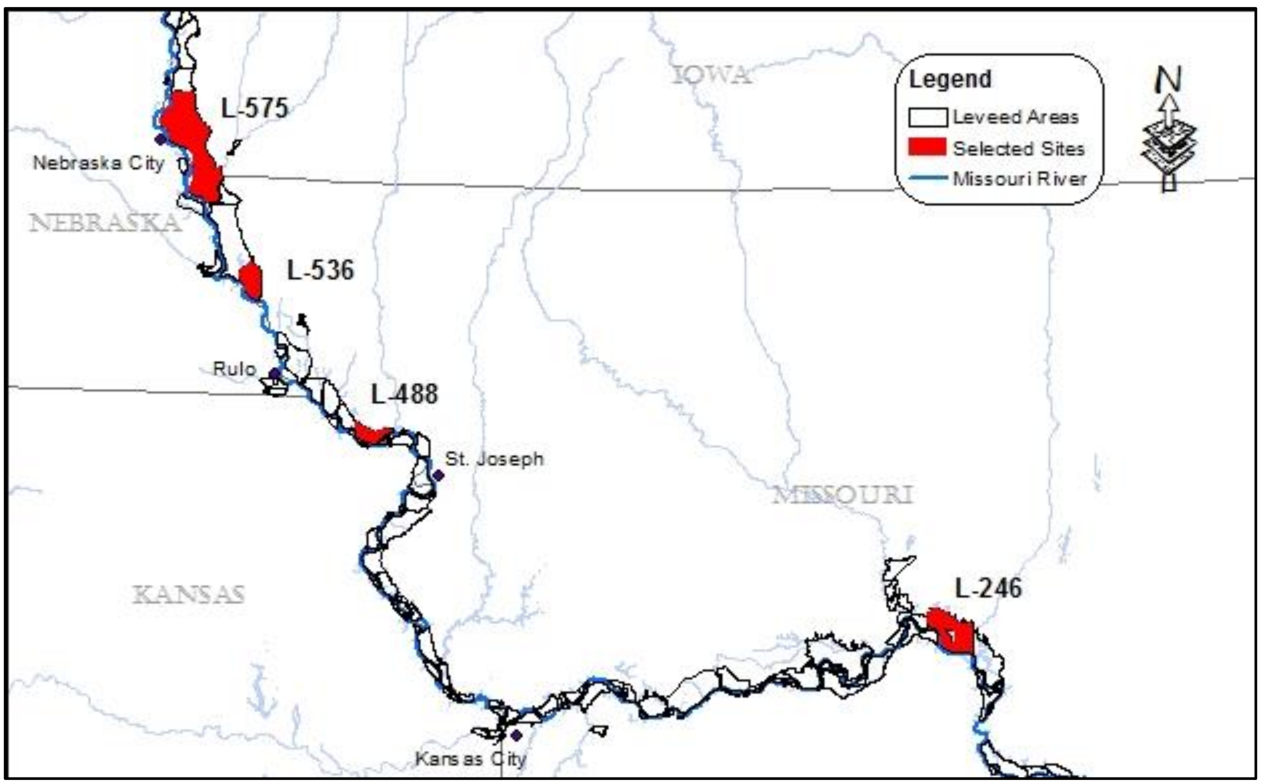


Figure 5-1. Interior Drainage Sites

Detailed modeling included a HEC-HMS model to simulate runoff, a HEC-RAS model to simulate the water surface profile of the river and water elevation on the levee interior, and a HEC-FIA model to calculate damages. Because the models are quite complex and time consuming to set up, it was not feasible to model every levee on the Missouri River to this level of detail. HEC-HMS model documentation can be found in **Attachment 6 – Interior Drainage**

HMS Model Documentation. Construction, calibration, and results of HEC-RAS model are documented in the following sections.

5.1 Model Construction

Two smaller trimmed HEC-RAS models were created to model L-488 and L-246. The boundaries were selected to be well upstream and downstream of the zone of influence on the levee of concern, as well as at a clean boundary condition with regard to floodplain flow through storage areas. Upstream flow boundary conditions, and the downstream stage boundary condition were taken from the primary models for each alternative period of record run. Cross sections in the trimmed models are from each of the alternative geometries, No Action, BiOp and IRC, and reflect the habitat assumptions in that reach per each alternative geometry.

To accurately model the changing stage on the interior of the levee with regard to local rainfall, culvert drainage, and seepage, the protected area behind L-488 and L-246 were delineated into multiple independent pooling areas. Area delineation considered the route of rainfall runoff from the bluffs into ditches in the floodplain, the most dominate culvert outlet locations, and natural high ground divisions between pooling areas. Each area has at least one associated lateral structure, to which was added the corresponding culvert(s) that primarily drain the area. No pumps were considered in the models at this time.

5.1.1 *L-488*

Delineation of the protected area behind L-488 is shown in Figure 5-2. The area was subdivided into five storage areas based on the five primary culvert outlets. Selected delineation is very comparable to the delineation in the Master Manual, with the exception of 448c and 448d, which were combined in the old evaluation, and a few slight differences on the high ground division which can be attributed to better quality present day terrain data.

New storage area curves for the five areas were cut from 1-meter LiDAR data flown in the winter of 2011-2012. Storage area curves were created in HEC-GeoRAS using 0.1 slice density and 50 points, capped at an elevation of 855-ft. Observed data from spring pulse monitoring conducted in 2009 was available for model calibration at one of the culverts in L-488. Figure 5-2 presents a schematic of the L-488 detailed hydraulic model including the locations of the observed stage hydrographs.

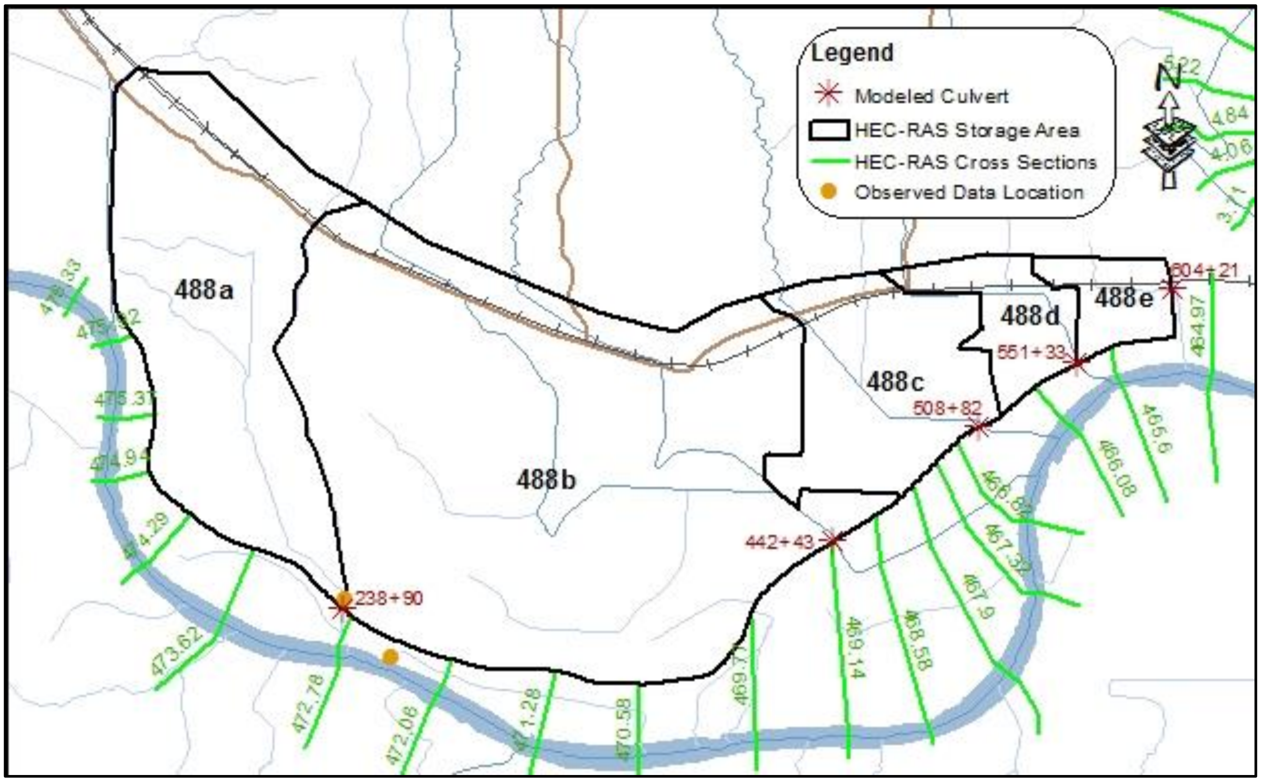


Figure 5-2. L-488 Layout

An HEC-HMS model was created to simulate the localized runoff to each culvert using hourly rainfall data assembled from nearby available precipitation gages for the full period of record. Details of the HEC-HMS model set up and calibration are contained in **Attachment 6**.

Each area in HEC-RAS was paired with a flow hydrograph produced by HEC-HMS, representing the rainfall runoff from the bluffs and over the floodplain area, routed to the respective outlet location, identified by a node name. Pairings, as well as drainage area and floodplain areas are in Table 5-1. Flow hydrographs from HEC-HMS were input as a flow boundary condition to each storage area in HEC-RAS.

Table 5-1. L-488 Links

Storage Area	Storage Area Size (sq mi)	Associated HMS Node	Drainage Area (sq mi)
488a	3.66	30c	4.02
488b	8.47	24c	18.34
488c	1.71	25c	1.92
488d	0.64	4c	2.52
488e	0.43	28c	0.63

One lateral structure representing the levee was associated with each storage area. There are nine total drainage structures on the levee system, however only one primary outlet location was added to each lateral structure. Secondary culvert outlets were all small and appeared to prevent localized ponding on the floodplain rather than provide a drainage path for bluff rainfall. Including these may have actually over-represented the ability of the interior drainage system to exit water from the interior, as ponding water would have to be high enough to spill over high ground on the interior to utilize these outlets. Selected culverts and their key hydraulic parameters are listed in Table 5-2. All pipe information was taken from the Missouri River Levee System Unit 488-L 2010 Periodic Inspection Report.

Table 5-2. L-488 Culverts

Levee Station	Associated Storage Area	Size and Conduit	Manning's n-value	Gates	Entrance Type	Ent Loss Coeff	Length (ft)	Inlet Elev (NAVD 88)	Outlet Elev (NAVD 88)
238+90	488a	2-72" RCP*	0.013	Sluice	Headwall	0.5	116	825.39	823.94
442+43	488b	5-72"CMP	0.024	Sluice	Headwall	0.5	102	821.32	819.84
508+82	488c	5-72" CMP	0.024	Sluice	Headwall	0.5	124	821.31	819.70
551+33	488d	3-54" CMP	0.024	Sluice	Projected	0.9	132	819.80	817.62
604+21	488e	1-48" CMP	0.024	Sluice	Headwall	0.5	134	820.80	818.86

Notes
* Original pipe was two CMPs, replaced by two RCPs

5.1.2 *L-246*

Delineation of the protected area behind L-246 is shown in Figure 5-3. The area is significantly more complicated than L-488, with a ring of levees surrounding an interior lake and drainage ditch called Palmer Creek. Storage area delineation went through several different versions, with the final configuration based on the best match to observed data. Selected delineation is comparable to the delineation in the Master Manual, but with the addition of 246n.

New storage area curves for the five areas were cut from 1-meter LiDAR data flown in 2006, the most recent available at this time. Storage area curves were created in Geo-RAS using slice density of 0 and 50 points, capped at an elevation of 645-ft. Observed data from spring pulse monitoring conducted between 2008 and 2010 was available for model calibration at three of the culverts in L-246. Figure 5-3 presents a schematic of the L-246 detailed hydraulic model including the locations of the observed stage hydrographs.

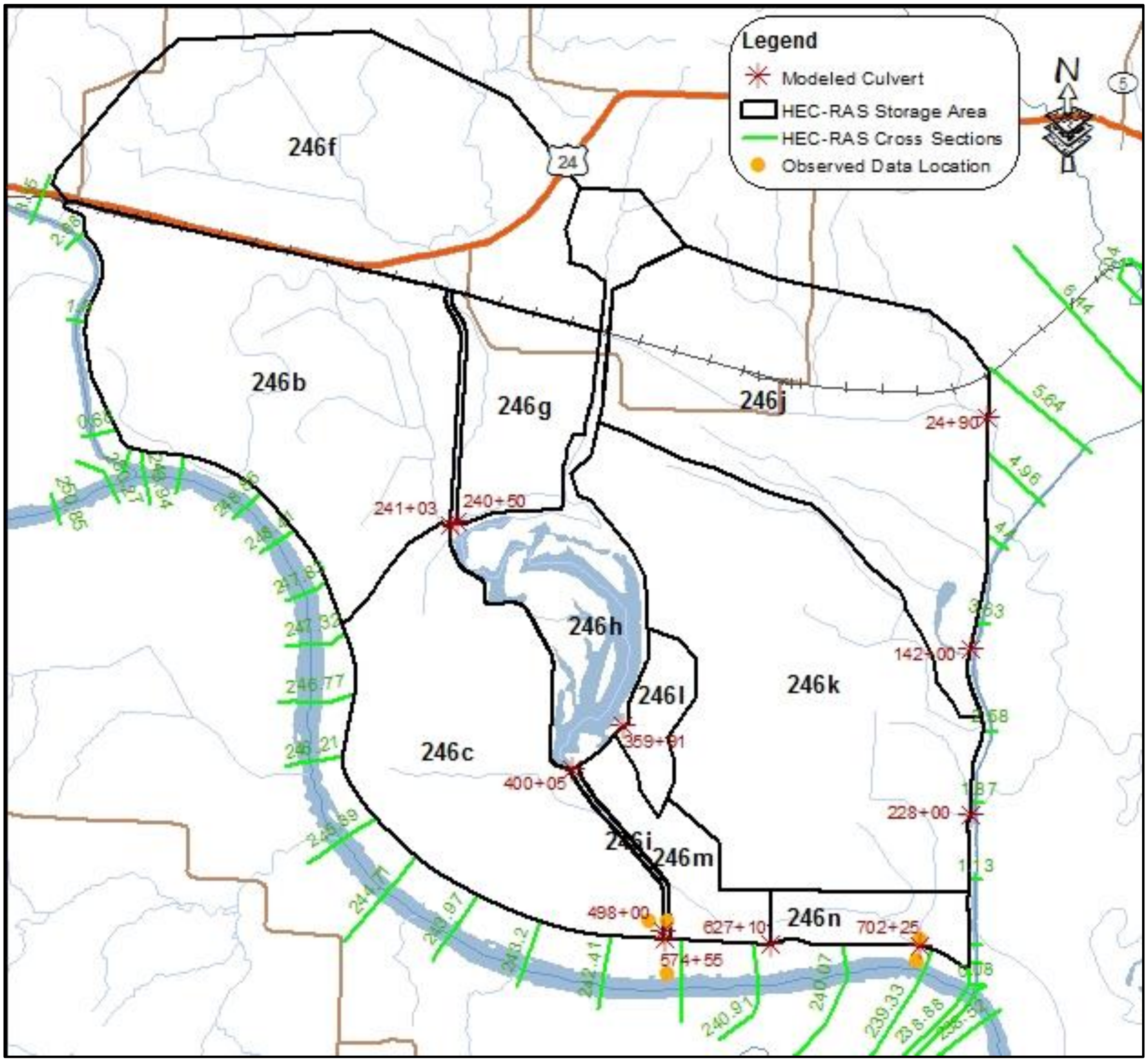


Figure 5-3. L-246 Layout

An HEC-HMS model was created to simulate the localized runoff to each culvert using hourly rainfall data assembled from nearby available precipitation gages for the full period of record. Details of the HEC-HMS model set up and calibration are contained in **Attachment 6**.

Each area in HEC-RAS was paired with a flow hydrograph produced by HEC-HMS, representing the rainfall runoff from the bluffs and over the floodplain area, routed to the respective outlet location, identified by a node name. Pairings, as well as drainage area and floodplain areas are in Table 5-3. Flow hydrographs from HEC-HMS were input as a flow boundary condition to each storage area in HEC-RAS.

Table 5-3. L-246 Links

Storage Area	Storage Area size (sq mi)	Associated HMS Node	Drainage Area (sq mi)
246b	8.55	29c + 34c	9.9
246c	6.52	49c + 36c + 47c	6.7
246f	8.93	-	-
246g	2.22	32c	2.2
246h	3.72	45c + 41c	47.7
246i	0.12	-	-
246j	8.35	27c + 25c	10.5
246k	10.02	33c	10.1
246l	0.72	40c	0.42
246m	1.29	39c	1.91
246n	0.98	-	-

One lateral structure representing the exterior levee was associated with each storage area. Interior levees and high ground divisions between areas were represented by storage area connections. There are thirty eight total drainage structures on the levee system, providing drainage from the interior to the Grand, Chariton, and Missouri Rivers, as well as through the interior levees to the lake, and from the lake to the Missouri River. One drainage structure was identified as the controlling structure for each storage area and added to either the relevant lateral structure or storage area connection, with the exception of 246j for which there were two fairly large structures. Selected culverts and their key hydraulic parameters are listed in Table 5-4. All pipe information was taken from the Missouri River Levee System Unit 246-L 2013 Periodic Inspection Report.

Table 5-4. L-246 Culverts

Levee Station	Alignment	Associated Storage Area	Size and Conduit	Manning's n-value	Gates	Entrance Type	Ent Loss Coeff	Length (ft)	Inlet Elev (NAVD 88)	Outlet Elev (NAVD 88)
241+03	Palmer RB	246b	1-60" CMP	0.024	sluice	headwall	0.5	53.6	621.19	620.60
			1-48" CMP	0.024	sluice	headwall	0.5	53.6	621.28	620.65
498+00	Palmer RB	246c	1-36" CMP	0.024	flap	headwall	0.5	124.7	616.08	615.50
240+50	Palmer ULB	246g	2-42" CMP	0.024	flap	headwall	0.5	56.5	621.23	620.84
400+05	Palmer RB	246h	3-6'x6' RCB	0.013	sluice	wingwall, square	0.5	68.7	619.10	613.60
574+55	Main	246i	1-5'x10' RCB	0.013	sluice/ flap	wingwall, rounded	0.2	122.3	611.85	610.10
			2-5'x5' RCB	0.013	sluice/ flap	wingwall, rounded	0.2	122.4	616.90	614.97
24+90	Chariton RB	246j	2-5'x5' RCB	0.013	sluice/ flap	headwall	0.5	94	621.84	620.51
142+00	Chariton RB	246j	1-72" CMP	0.024	sluice/ flap	headwall	0.5	164	614.10	612.47
228+00	Chariton RB	246k	2- 5'x5' RCB	0.013	sluice/ flap	wingwall, rounded	0.2	176	613.80	612.44
702+25	Main	246n	1-54" CMP	0.024	sluice/ flap	headwall	0.5	170.69	613.02	-
627+10	Main	246m	1-30" CMP	0.024	flap	headwall	0.5	125.2	613.89	-
359+91	Palmer LLB	246l	1-36" CMP	0.024	flap	headwall	0.5	75.9	620.45	620.00

5.2 Simulation and Calibration

All calibration and alternative period of record runs were made in HEC-RAS version 5.0 Release Candidate 2 (21 August 2015) for consistency with the primary alternative runs. Before running the alternatives the models were calibrated using data collected by the MRRP in 2008, 2009, and 2010 to evaluate the impacts of the spring pulse and rises on interior drainage (USACE, Jan 2011). Calibration data included riverside and landside stages that were compared to model calculations.

5.2.1 *Seepage*

Seepage was incorporated for both the calibration and alternative runs. To calculate seepage the models were run twice. In the first run, ponding on the interior was simulated based only on the contribution of rainfall runoff from HMS routed through the culverts in HEC-RAS. Seepage rate estimates from the Master Manual were reviewed by the Geotechnical Engineering Section and were deemed reasonable for this purpose. Recent seepage studies at L-497 showed slightly lower seepage rates than those utilized in the Master Manual study. The seepage rates vary, increasing proportionally with river head, and are expressed in units of cfs per foot of levee. Results of the first model run were used to estimate the driving head as a difference between the interior ponding elevation and the river elevation, and a daily average seepage rate was then calculated for the length of the levee corresponding to each storage area. For each area the levee length along the river was measured, as seepage is identified as a flow rate per unit levee length. A representative cross section was selected near the mid-point of the levee length for comparison to the landside ground and ponding elevation. Each day the head difference between river and landside was calculated. Head was defined as the river stage compared to the ground or ponded water on the landside, whichever was higher. If the head was negative, meaning the river was lower than the ponded water on the landside, or lower than the lowest point on the landside, seepage was set to zero. If the head was positive, a daily seepage rate was calculated based on the values in Table 5-5 and Table 5-6. The models were then run again incorporating the seepage as inflow into the area. No further iterations were performed, as these would have only served to slightly reduce the seepage contribution.

Table 5-5. L-488 Seepage

	Storage Area	448a	448b	448c	448d	448e
	Levee Length [1] (ft)	23,964	23,022	4,688	3,353	5,556
Head (ft)	Seepage per unit length [2] (cfs/ft)	Seepage applied to HEC-RAS Storage Areas per Head per Day (cfs)				
0	0	0.0	0.0	0.0	0.0	0.0
2	0.00048	11.5	11.1	2.3	1.6	2.7
4	0.00096	23.0	22.1	4.5	3.2	5.3
6	0.00144	34.5	33.2	6.8	4.8	8.0
8	0.00192	46.0	44.2	9.0	6.4	10.7
12.5	0.003	71.9	69.1	14.1	10.1	16.7
15	0.00359	86.0	82.6	16.8	12.0	19.9
	Representative River Cross Section	474.94	470.58	467.32	466.08	465.6

Notes
1 Levee length calculated in GIS
2 Seepage per unit length as per Master Manual values

Table 5-6. L-246 Seepage

	Storage Area	246b	246c	246m	246n	246k	246j
	Levee Length [1] (ft)	28,089	26,713	5,181	13,921	9,000	17,512
Head (ft)	Seepage per unit length [2] (cfs/ft)	Seepage applied to HEC-RAS Storage Areas per Head per Day (cfs)					
0	0	0.0	0.0	0.0	0.0	0.0	0.0
2	0.00047	13.2	12.6	2.4	6.5	4.2	8.2
4	0.00093	26.1	24.8	4.8	12.9	8.4	16.3
6	0.0014	39.3	37.4	7.3	19.5	12.6	24.5
8	0.00187	52.5	50.0	9.7	26.0	16.8	32.7
15	0.0035	98.3	93.5	18.1	48.7	31.5	61.3
	Representative River Cross Section	248.86	244.71	241.63	240.07	1.87[3]	4.4[3]

Notes
1 Levee length calculated in GIS
2 Seepage per unit length as per Master Manual values
3 Chariton River cross-section

On L-246, seepage was only calculated along the exterior levee units along the Grand, Missouri, and Chariton Rivers. Seepage from the Missouri River to the lake, or from the lake to the fields was not accounted for.

5.2.2 *L-488*

Observed data for calibration at L-488 was available at the locations identified in Figure 5-2 near culvert 238+90. Measurements were only taken in the year 2009, on the landside and riverside near the culvert. HEC-RAS results for this year are plotted verses the observed data in Figure 5-4 and Figure 5-5.

Although model results don't match observed data exactly, the interior ponding generally reaches similar depths at a similar occurrence frequency. The Spring Pulse report (USACE, Jan 2011) recognized that there was considerable noise/chatter in the landside dataset compared to measured data at other units. The variability at L-488 may be influenced by the sponsors pumping activities at this location, which was not accounted for in this analysis due to not enough information on how pumps are operated during events. River stage computed by HEC-RAS could be tracking a little low because the NoAction rather than the existing condition cross section geometry was used for calibration runs. Downstream habitat placement may have a general lowering effect on the water surface profile in this vicinity.

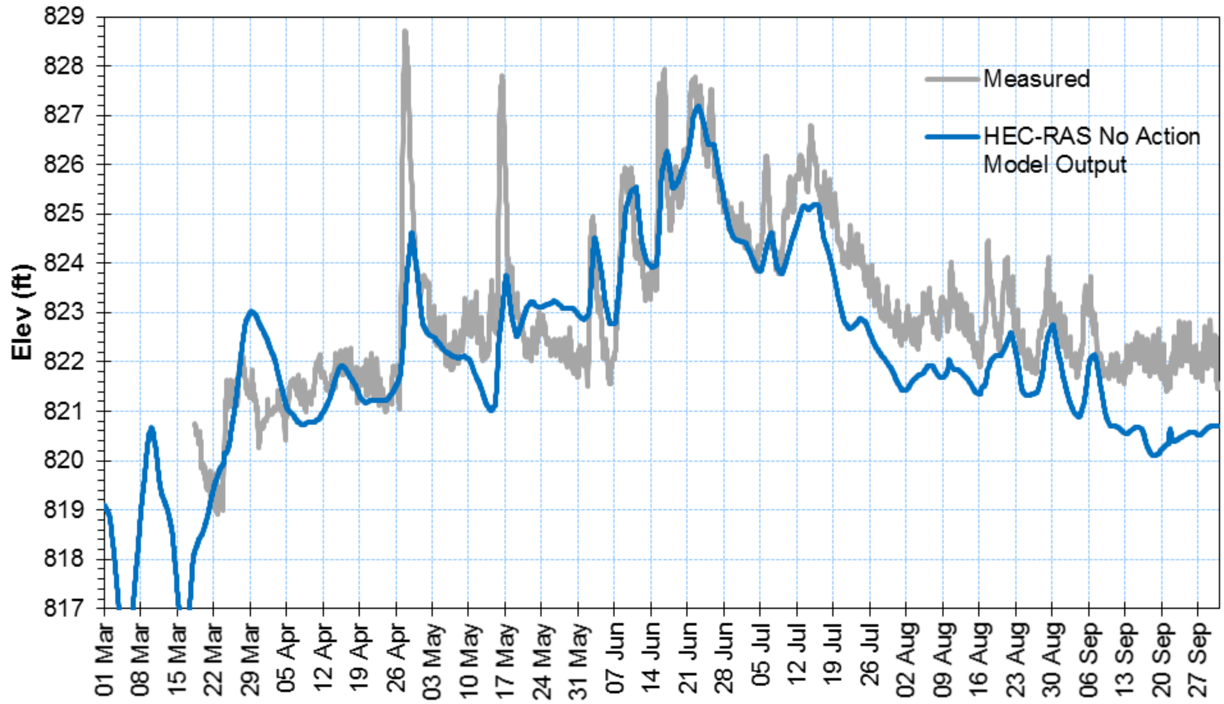


Figure 5-4. L-488 River Stage 2009 – measured vs modeled

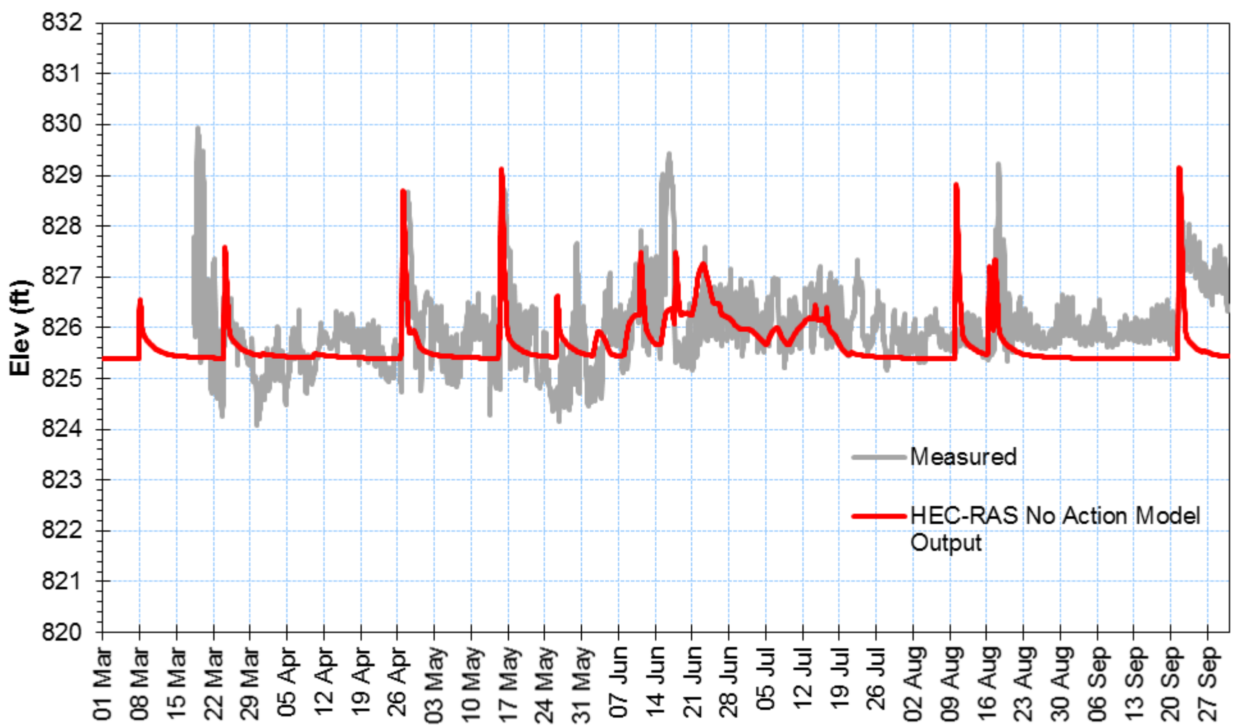


Figure 5-5. L-488 Interior Stage 2009 – measured vs modeled

As an additional validation, 2011 and 1993 floods were run through the model and RAS mapper inundation extents were compared to actual inundation extents.

Figure 5-6 shows the aerial photography collected during the 2011 flood event with HEC-RAS storage areas at L-488 outlined in black. Figure 5-7 overlays model results for comparison. L-488 did not overtop or breach during the 2011 flood, which means all of the ponding on the landside of the levee is due to seepage and/or bluff rainfall runoff that was unable to drain through the culvert because of high river profiles all summer long. Model results generally appear to overestimate the ponding in the interior during 2011, but not by much. The aerial photography was collected at the end of August, when flood waters were beginning to recede. Inundation shown in Figure 5-7 represents the inundation at the maximum ponding depth experienced during the 2011 flood simulation. The gap between water in the photo and maximum inundation is large, but the match between the brown areas that indicate dead crops and the maximum inundation is much closer.

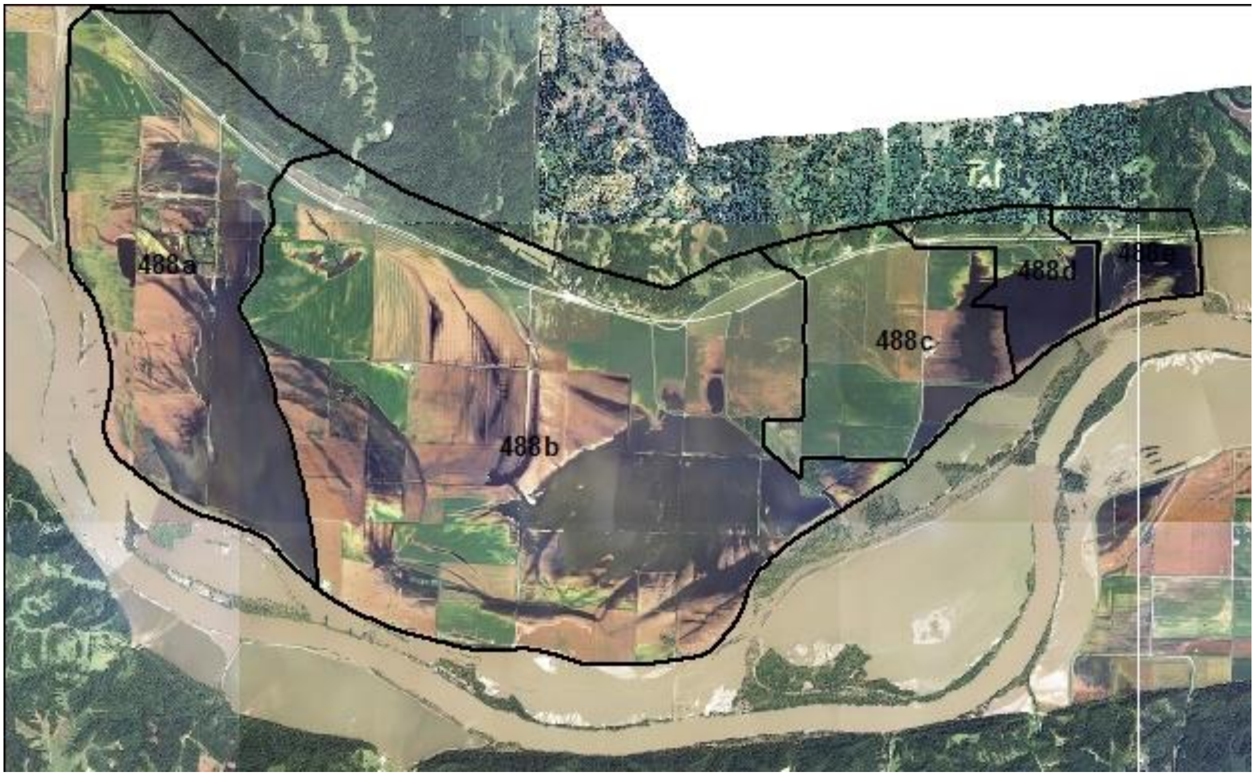


Figure 5-6. Flood inundation in 2011 at L-488

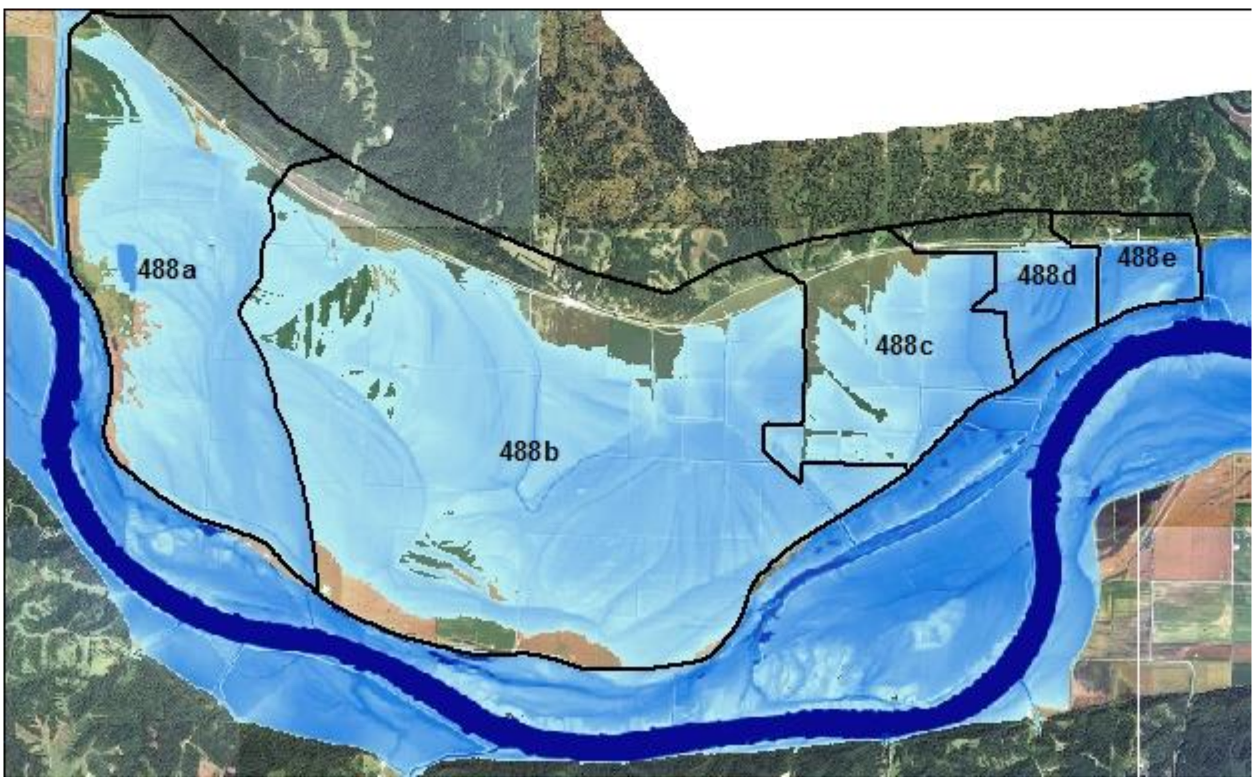


Figure 5-7. Flood inundation in 2011 at L-488 – model results

Figure 5-8 overlays the model results with the 1993 flood extents digitized during the post flood report from aerial photography. L-488 was reported as overtopped but not breached in the 1993 post flood report (USACE, 1994). However, the profile of the river at the maximum moment of the 1993 flood is about 2-ft lower than the L-488 top of levee in the No Action alternative model run. Reasons for this were discussed in the calibration report, the use of daily data and the ungaged flow methodology made it difficult to match annual peaks in all years in all locations. Regardless, the model results still overestimate the interior flooding during 1993 based on local rainfall and seepage contributions alone.

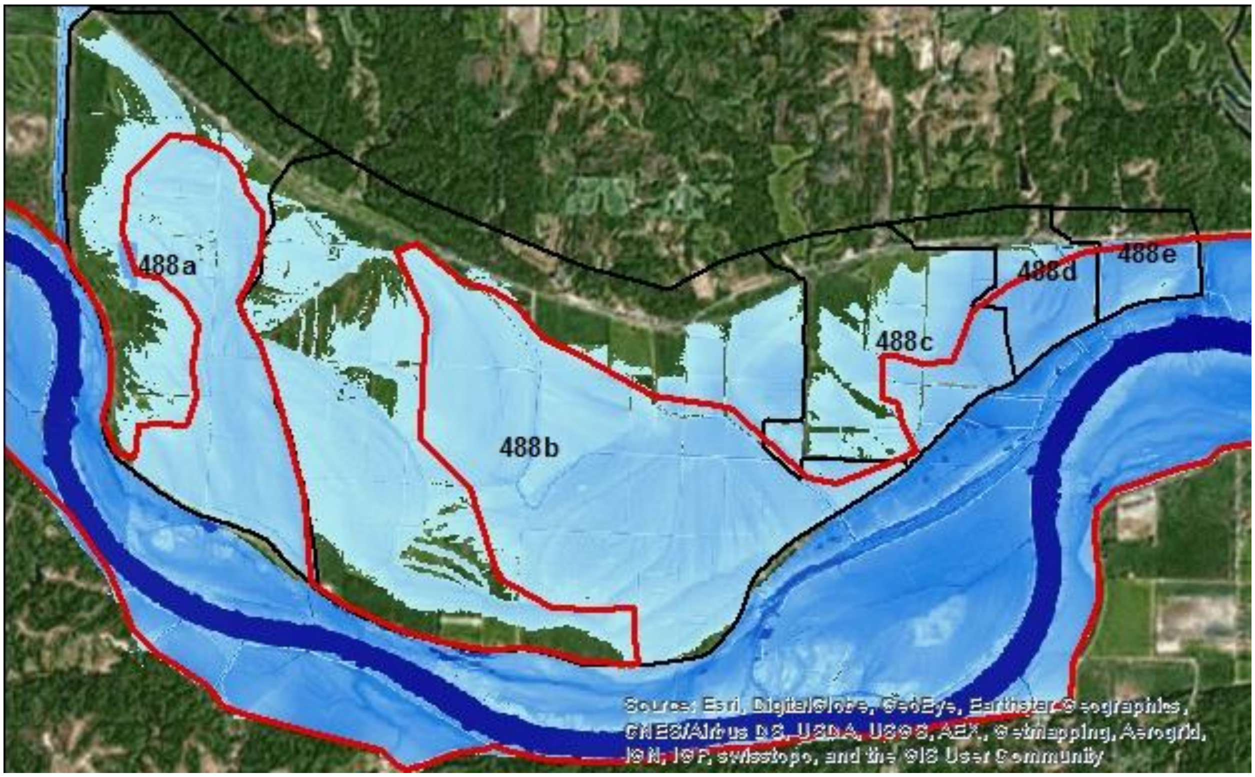


Figure 5-8. Flood inundation in 1993 at L-488 – model results

Overall, calibration and validation of the L-488 interior drainage model indicates that the model is acceptable for re-producing pooling water in the interior of the levee due to high river stages, seepage, and local bluff rainfall.

5.2.3 *L-246*

Observed data for calibration at L-246 was available for the locations identified in Figure 5-3. Measurements were taken in 2008 and 2009 on the riverside and landside at culvert 574+55 where Palmer Ditch drains water from the lake to the Missouri River. In 2009 an additional data collector was added in the landside fields to the west of the ditch at the inlet of culvert 498+00. In 2010, stages were collected at culvert 702+25 known as Dalton Ditch, instead of at the Palmer Ditch outlet. HEC-RAS results for 2008, 2009, and 2010 are plotted verses the observed data in Figure 5-9 through Figure 5-13. Model results track well with the peak ponding recorded by the instrumentation both in frequency and magnitude.

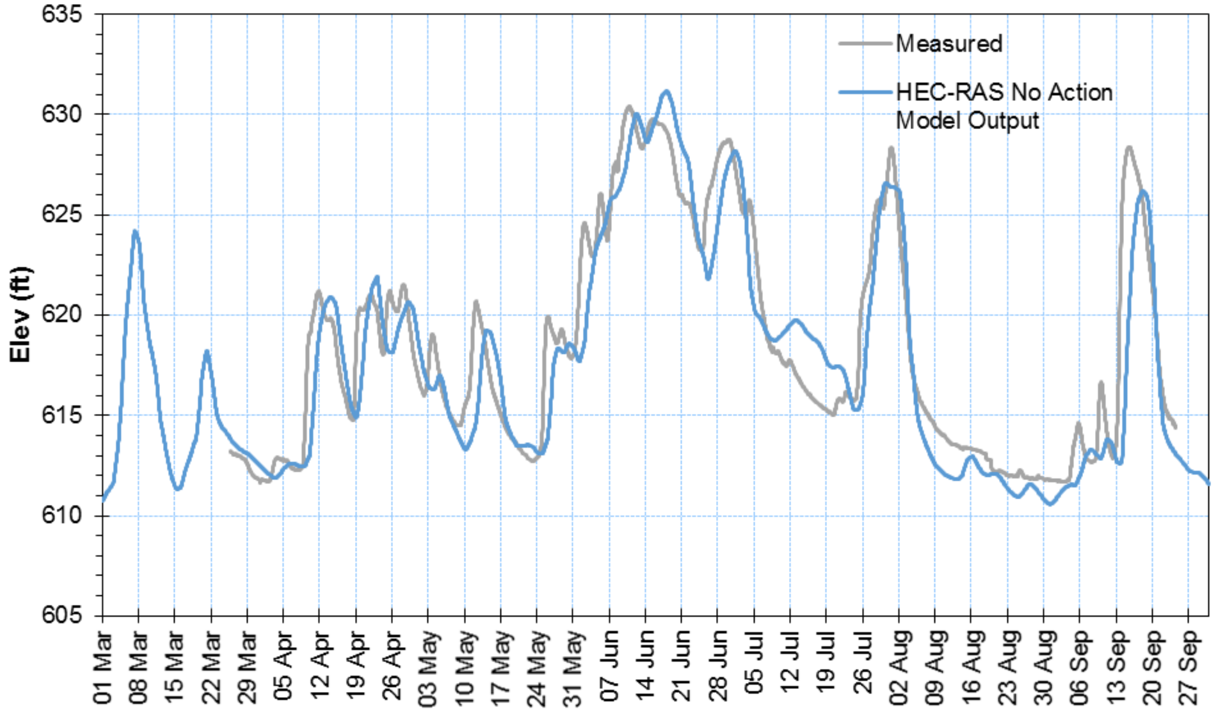


Figure 5-9. L-246 River Stage 2008 – measured vs modeled

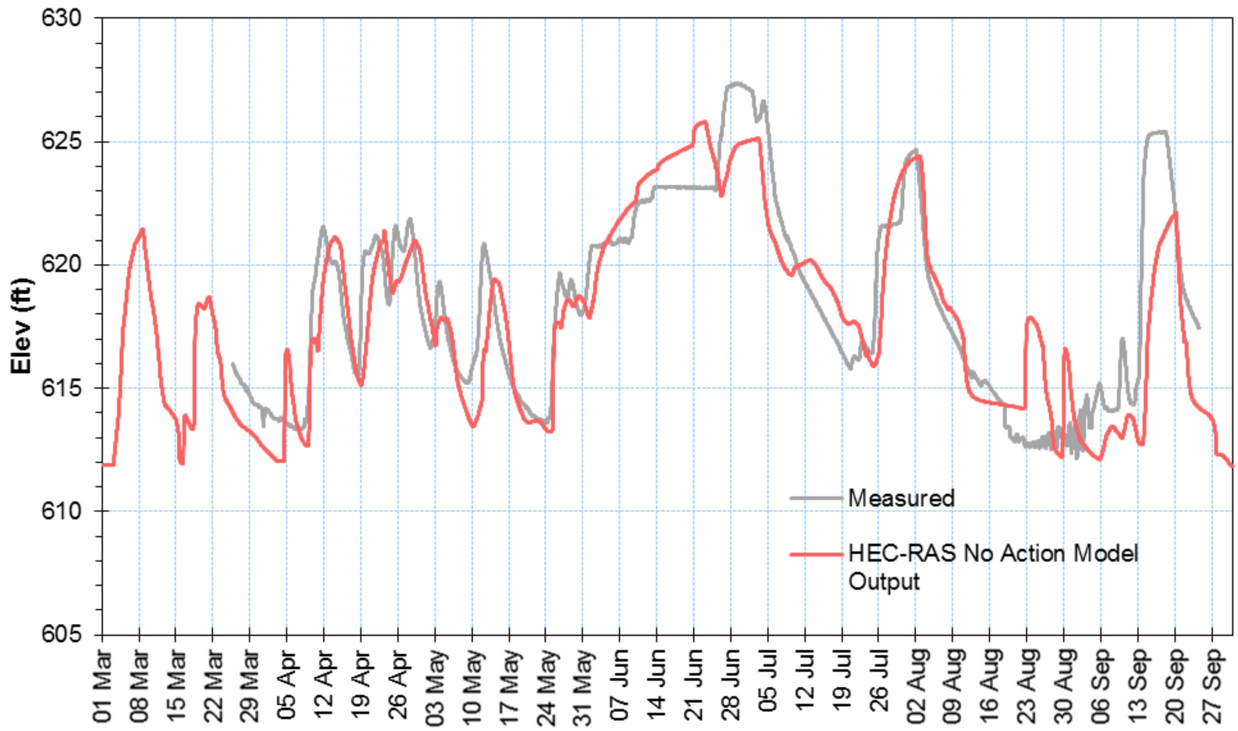


Figure 5-10. L-246 Palmer Ditch Stage 2008 – measured vs modeled

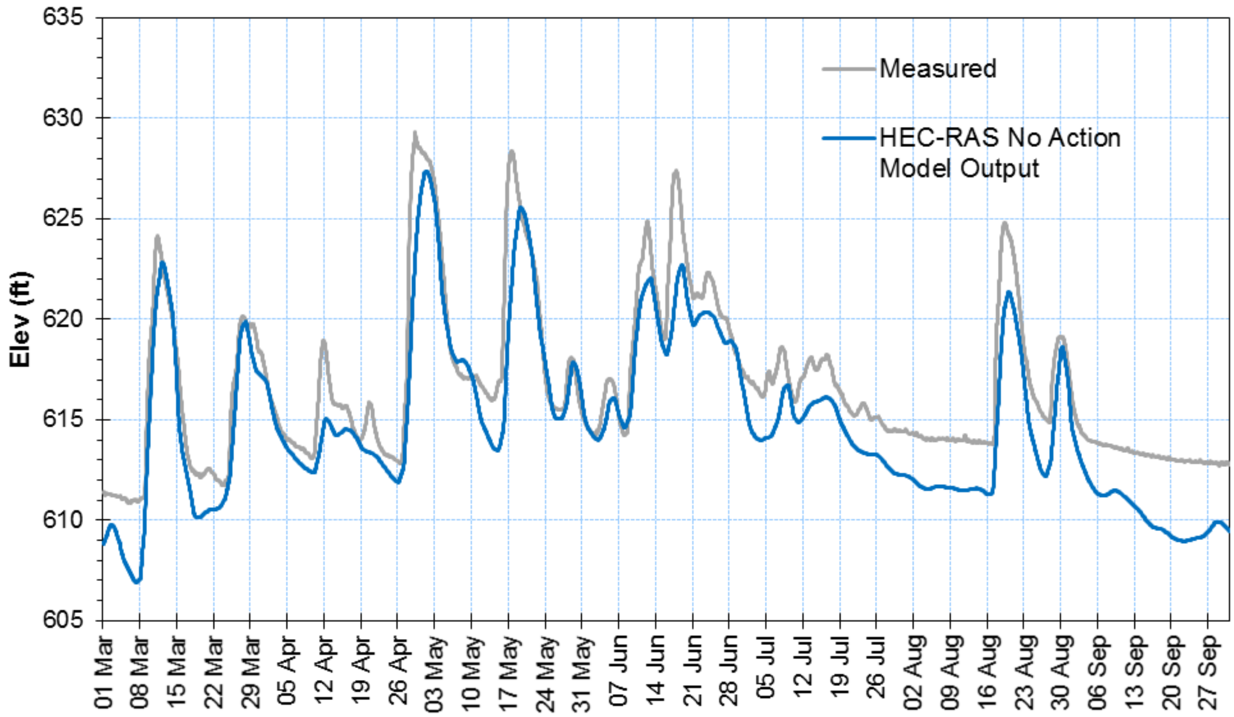


Figure 5-11. L-246 River Stage 2009 – measured vs modeled

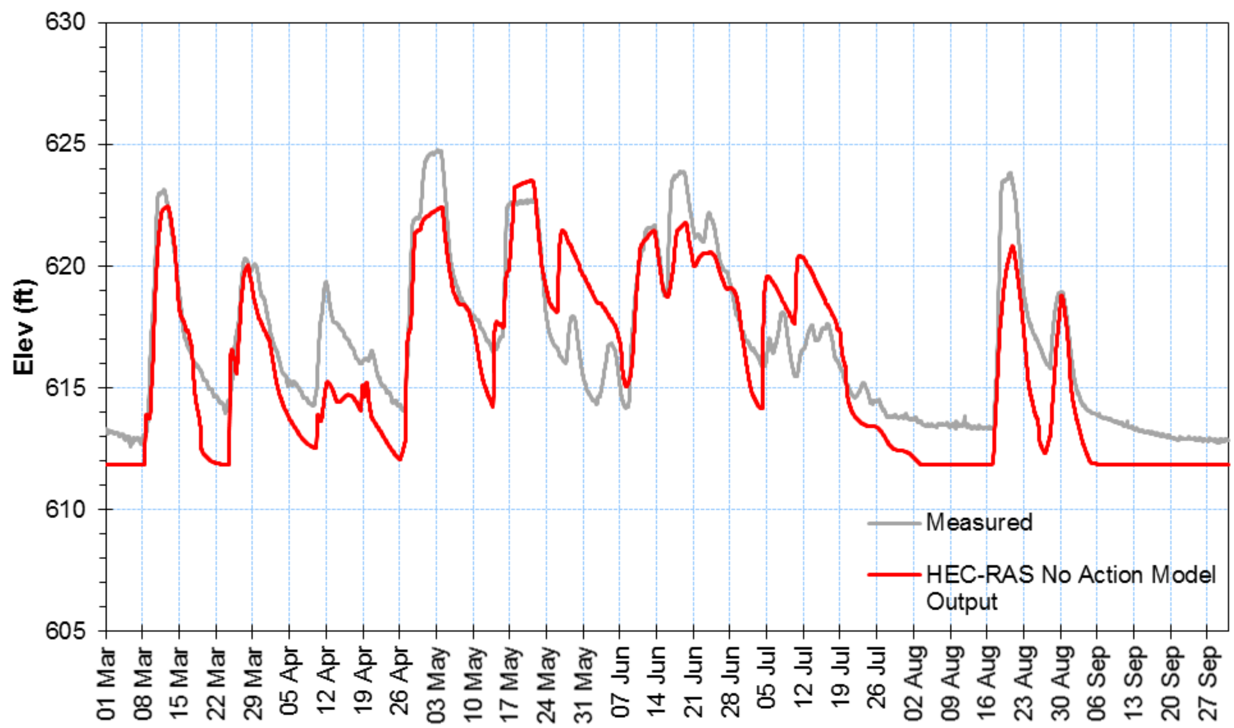


Figure 5-12. L-246 Palmer Ditch Stage 2009 – measured vs modeled

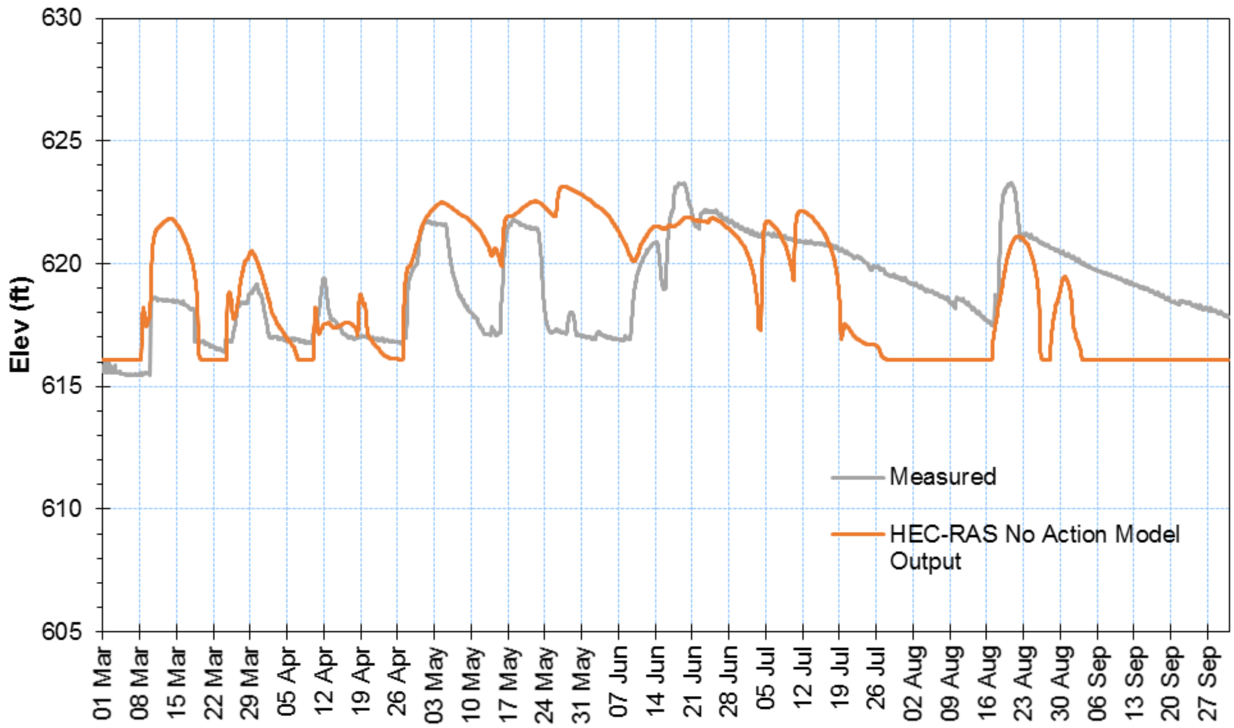


Figure 5-13. L-246 West of Palmer Ditch Stage 2009 – measured vs modeled

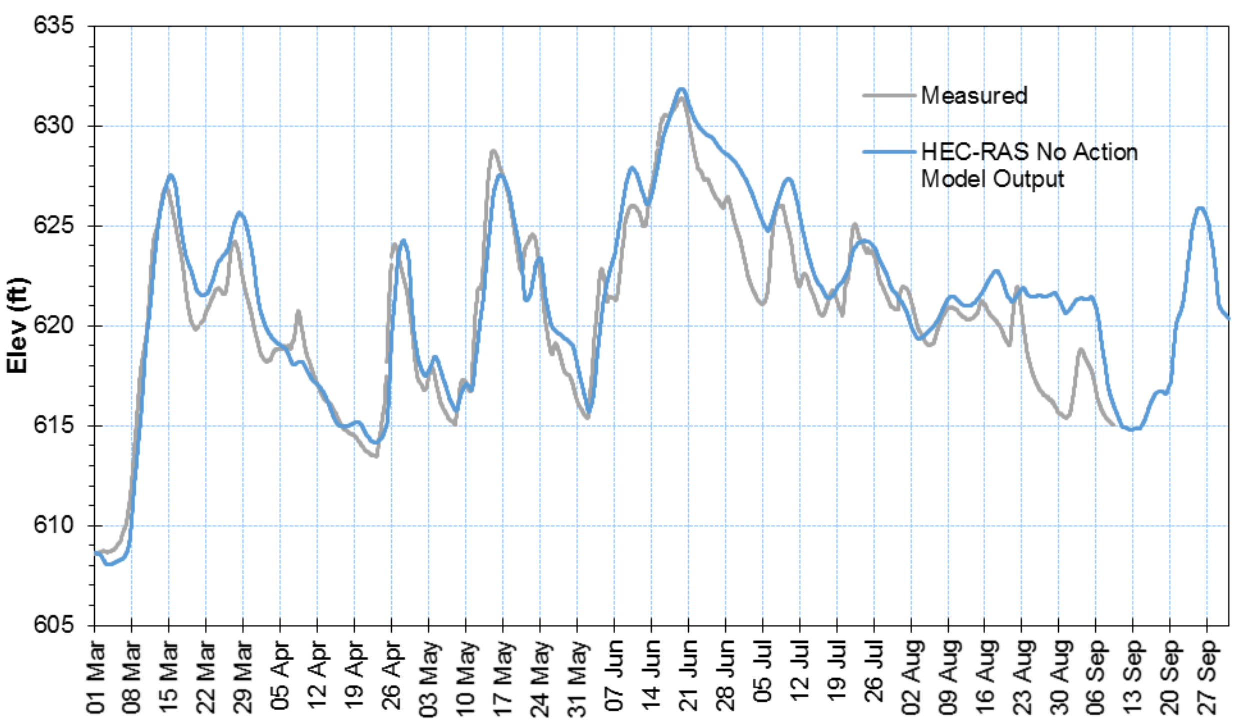


Figure 5-14. L-246 River Stage 2010 – measured vs modeled

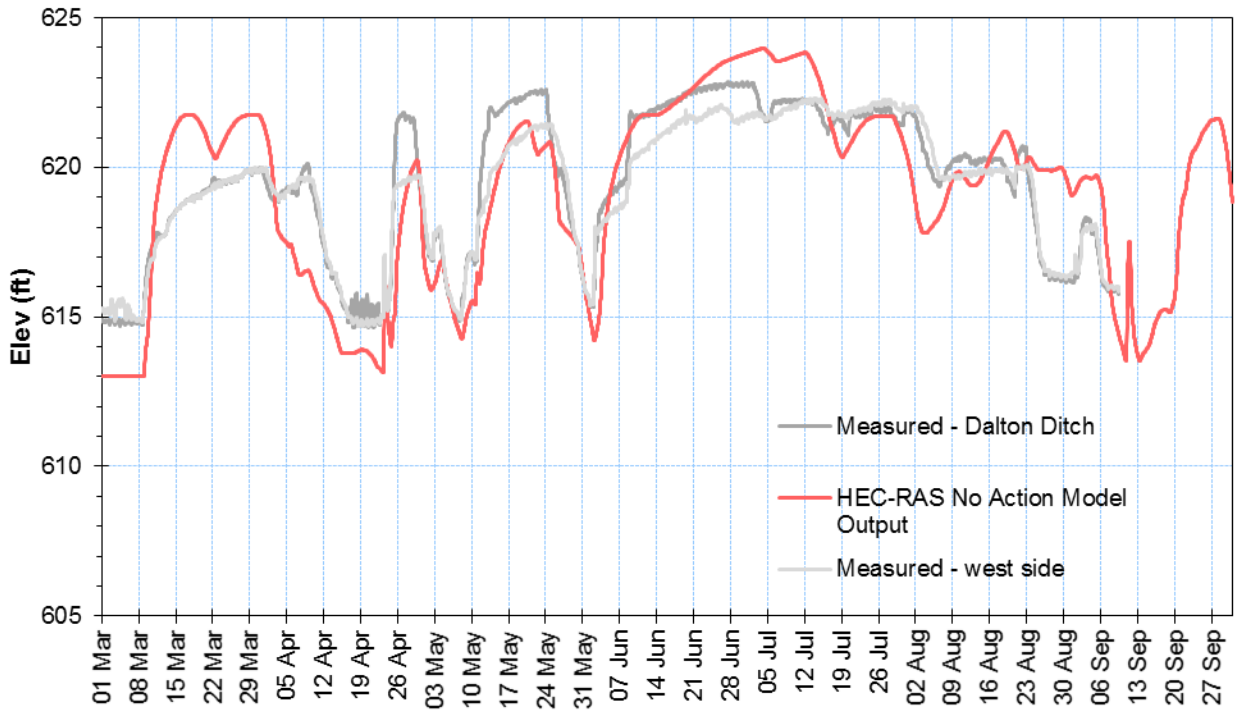


Figure 5-15. Dalton Ditch Stage 2010 – measured vs modeled

Note that in 2009, Figure 5-13, the model drains the area west of Palmer ditch too efficiently in June and August events. River stages on the Missouri, and correspondingly in Palmer ditch are also several feet too low during this same timeframe, which could be contributing to the discrepancy.

As an additional validation, 2011 and 1993 floods were run through the model and RAS mapper inundation extents were compared to actual inundation extents.

Figure 5-16 shows the aerial photography collected during the 2011 flood event with HEC-RAS storage areas at L-246 outlined in black. Figure 5-17 overlays model results for comparison. The 2011 event was not as severe on the lower river downstream of Kansas City. Interior flooding shown in the image is due to prolonged high river stages preventing rainfall runoff from draining out culverts, as well as the contribution of seepage. Model results overestimate the ponding in the interior during 2011. However, this could partially be from the model not accounting for pumping done by local land owners during the 2011 flood.

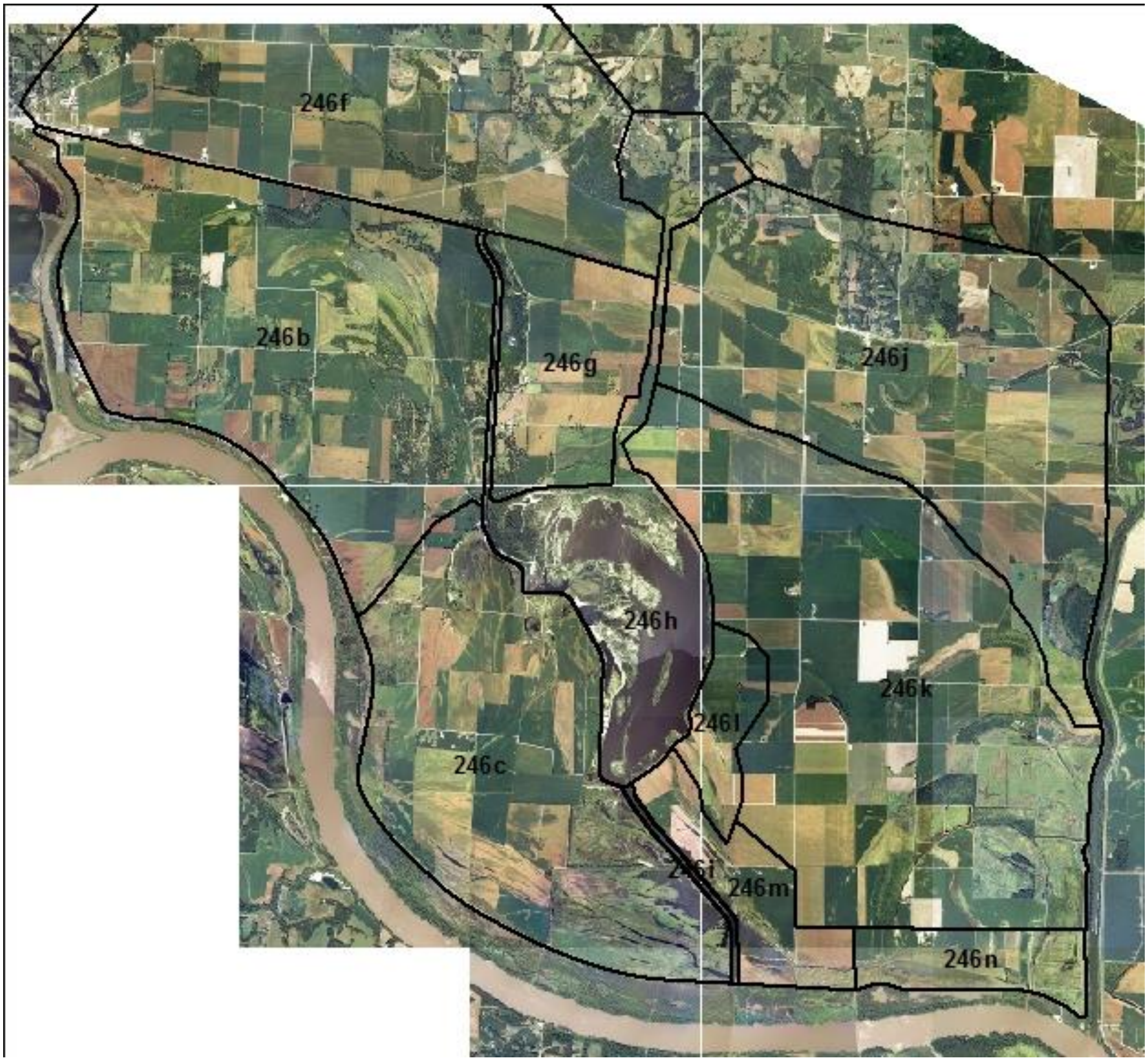


Figure 5-16. Flood inundation in 2011 at L-246

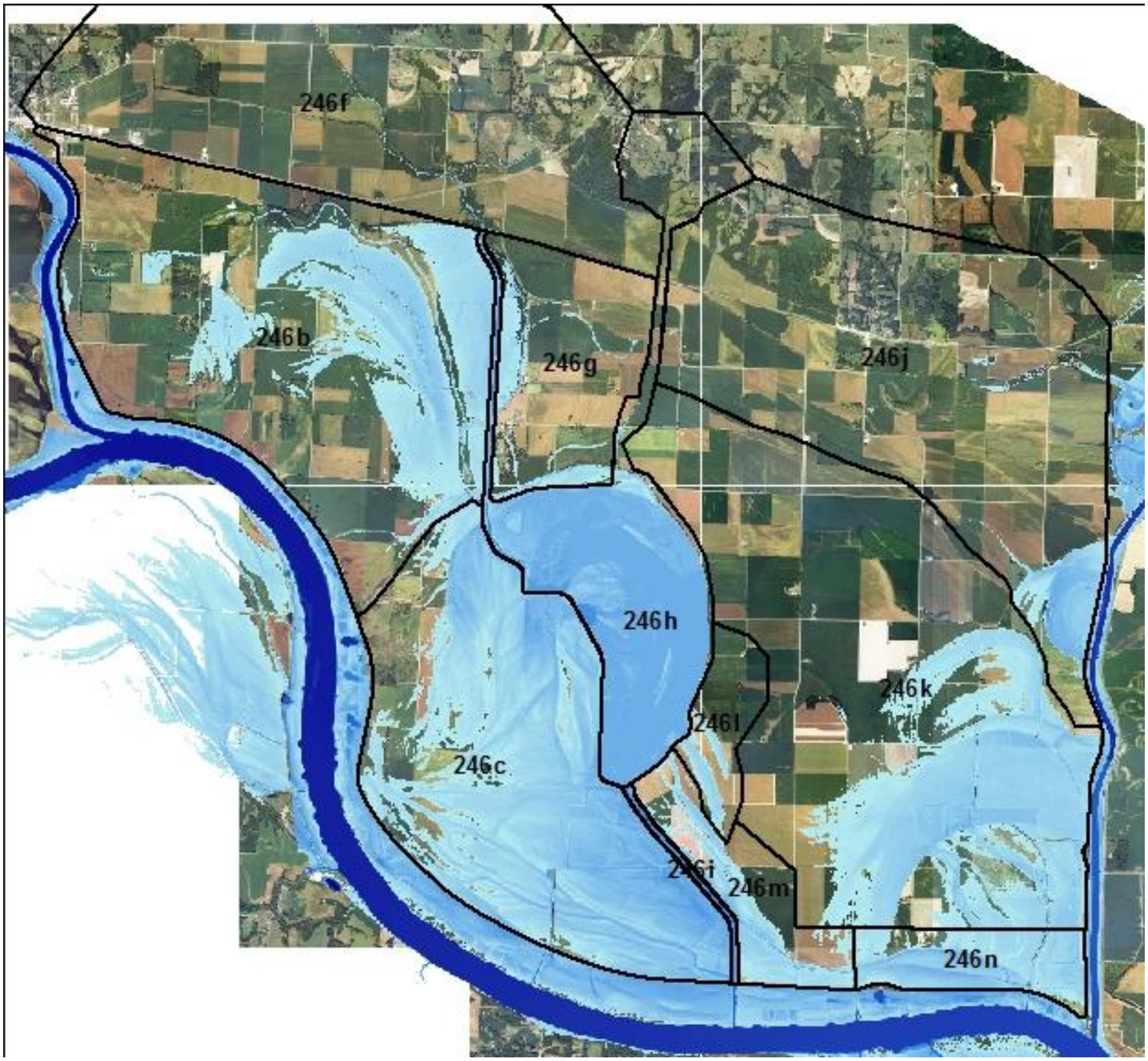


Figure 5-17. Flood inundation in 2011 at L-246 – model results

As at L-488, the aerial photography is from late August and the model results shown are the inundation at the maximum ponding depth experienced during the 2011 flood simulation. At this location on the river it is possible that the duration of time that flood waters were at a level high enough to cause crop damage on the levee interior was significantly less than on the upper river.

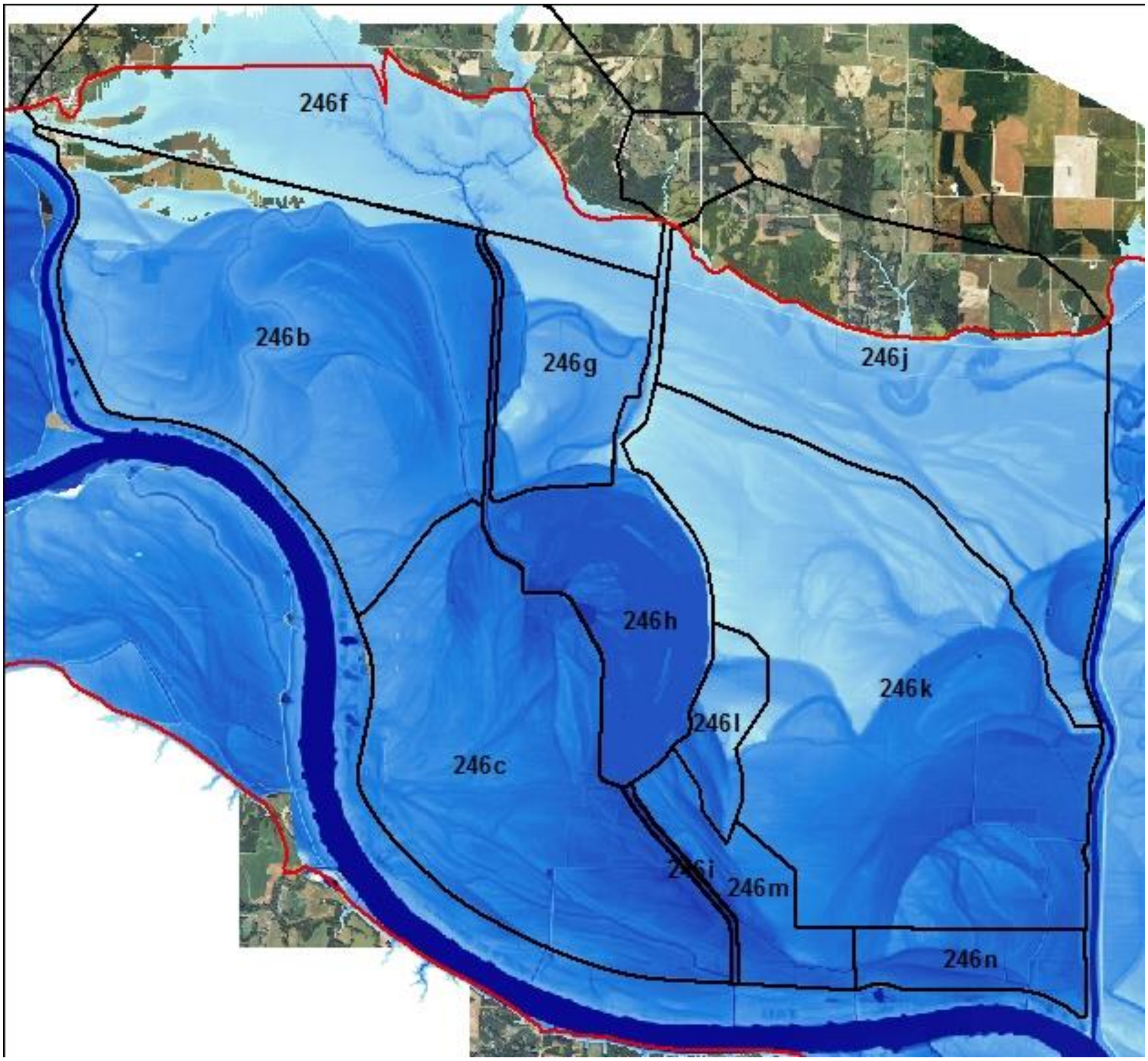


Figure 5-18. Flood inundation in 1993 at L-246 – model results

Figure 5-18 overlays the model results with the 1993 flood extents digitized during the post flood report from aerial photography. L-246 overtopped and breached at half a dozen locations, therefore validation from this particular event does not inform much about the performance of seepage and runoff calculations. However, as explained in the calibration report and in section 2.1, breaches were not accounted for in the model simulations, only overtopping. Validation of the 1993 inundation demonstrates that the model produces a realistic maximum inundation boundary with overtopping alone.

Overall, calibration and validation of the L-246 interior drainage model indicates that the model is acceptable for re-producing pooling water in the interior of the levee due to high river stages, seepage, and local bluff rainfall.

5.3 RESULTS

All interior drainage runs were performed in HEC-RAS 5.0.3 in July 2017. All of the results included in this report have been updated with the most current output. Period of record for the interior drainage models was limited by availability of rainfall data for the HMS models. For L-488 the record length was 64 years, from Aug 1948 to Dec 2012. For L-488 the record length was 63 years from Aug 1948 to Dec 2011. Stage and flow hydrographs for the interior areas of L-488 and L-246 were input into HEC-FIA models for evaluation of impacts to agricultural damage. Results are documented in the Environmental Consequences section of the EIS and the Flood Risk Management Technical Report.

To express the changes compared with the No Action alternative, the model results were evaluated by the same statistical evaluations made on the full models. Tables comparing min, max, and percentiles for each individual storage area are provided in **Attachment 7 – L-488 Interior Drainage Alternative Statistics** and **Attachment 8 – L-246 Interior Drainage Alternative Statistics**.

Most of the time the interior areas do not have water, as shown by the 10, 25, 50, and sometimes 75 percentile statistics being equal to the minimum elevation. Additionally, it appears that most of the time the alternatives didn't cause any change to the interior ponding compared to No Action, as shown by the 10 through 90 percentile daily change being equal to zero as well.

Seepage appears to be a large factor in changes from No Action. Take for example the year 1982. Alternative 4 had a complete pulse released during the month of April, which caused river stages near the mouth of the Chariton to increase by about five feet, as shown in Figure 5-19. The result was an increase of about 4 feet of ponding depth on the interior of area 246K as shown in Figure 5-20 compared to no Action, one of the larger increases over the period of record, and entirely due to seepage as there was no rainfall inflow from HMS during the month of August in this year. Additionally, it is important to keep the ponding depth in context. An increase of 3-4 feet sounds like a lot, but the interior flooding was modeled to have been higher than that in March and again in May and multiple other times during the same year.

Figure 5-19. Alternatives Flows at L-246 – 1982

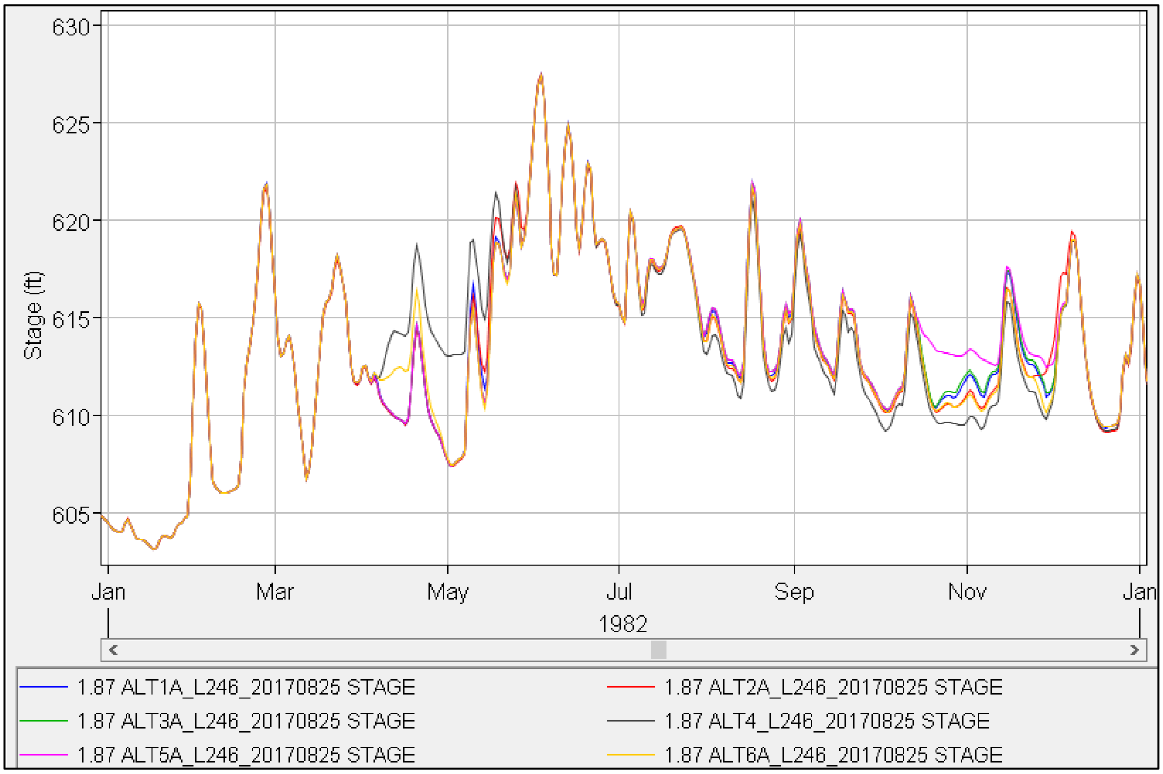
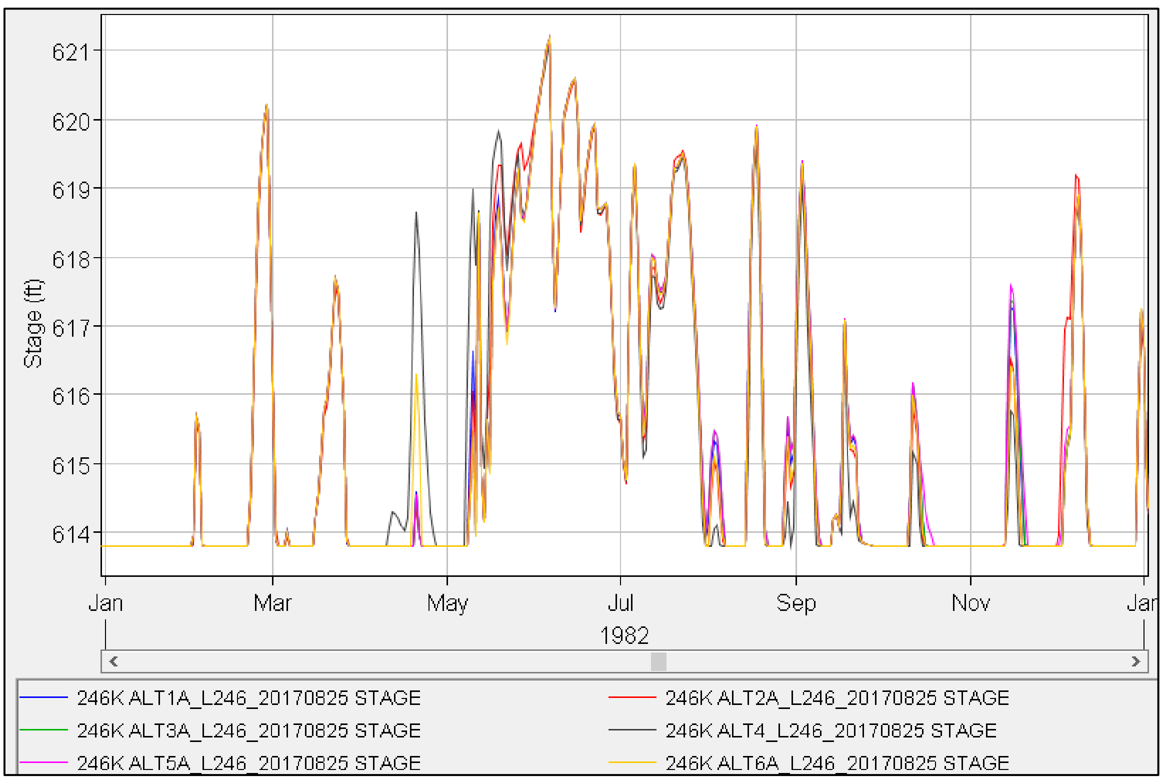


Figure 5-20. Alternative Stages in 246K – 1982



The interior drainage models provide a powerful tool to assess the complicated interaction between reservoir releases hundreds of miles upstream of a levee unit on interior ponding resulting from rainfall and/or seepage. However, limited conclusions can be extrapolated to the entire river. With such slight differences compared to No Action, it is difficult to separate the impact of flow changes from the reservoirs from the impact of added habitat to make global conclusions. Changes are highly localized, depending upon factors such as how low the culvert outlet is and the interior area available for ponding before damages occur.

The Omaha district ran a handful of sensitivity simulations at L-575 and L-536, comparing pumps to no pumps, half clogged culverts, and pulse level water surface elevations along the levee every single year. For results refer to the Gavins to Rulo **Appendix D**. An additional sensitivity simulation would be to hold the river geometry at the constant baseline condition, instead of accounting for habitat. Omaha used the No Action river geometry for all simulations, while the Kansas City district varied the river geometry according to the alternative assumptions. These sensitivity runs help separate the influence of the many contributing factors, as well as identify an upper threshold of influence based on the release rules of the pulses given the 62-years of downstream rainfall conditions.

6 ADDITIONAL ANALYSIS FOR YEAR 15

Degradation and aggradation of the Missouri River channel bed and sedimentation in the reservoirs are ongoing processes, which have the potential to effect virtually all economic resources and human considerations. Therefore, additional modeling was performed to provide estimates of how ongoing sedimentation processes may affect the performance of future alternatives. The future without and with project condition modeling is based on a number of critical assumptions regarding historic trends, flows, and sediment inputs. While not intended to represent detailed estimates of future channel conditions, the results do provide an alternative comparison methodology. The designation "Year 15" comes from the timeframe for implementation; the Record of Decision (ROD) for the Missouri River Recovery Program, Management Plan Environmental Impact Statement (EIS) is expected to be signed in 2018, with a construction completion date of 2033, resulting in an implementation period of 15 years. Results from the Year 15 analysis were provided to economists and human considerations teams for qualitative evaluation, versus the full quantitative evaluation that was performed on the base condition (also referred to as Year 0).

6.1 YEAR 15 FUTURE CONDITIONS

To project river bed aggradation/ degradation trends to year 2033, moveable bed sediment models of the mainstem Missouri reaches were created in HEC-RAS. The Kansas City district model covers from Rulo, NE to the mouth and is called the Lower Missouri River Sediment Model (LMRSM). Results from the sediment modeling provided a projected change in area of the channel bed for each mile of the river for each of the six alternatives. Full details on construction and calibration of the LMRSM are in **Attachment 9A – Lower Missouri River Sediment Model Calibration Report**. Modeling of the alternatives is described in **Attachment 9B – Lower Missouri River Sediment Model MRRP Alternatives**. Sediment model output from the February 2018 version (actual model runs performed in October 2017) was used for the Year 15 analysis. As a result of the review process, changes were made and the sediment models were re-run in July 2018. Because the updated results follow the same basic trends as the earlier output, the unsteady models were not updated with new numbers. Refer to Section 9 in **Attachment 9B** for a comparison of the February 2018 and July 2018 outputs.

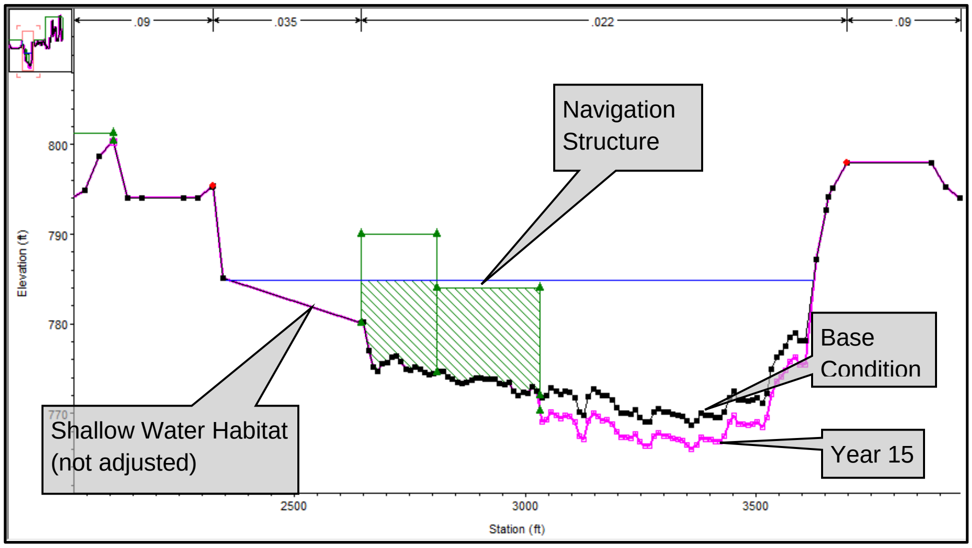
6.2 ALTERNATIVE ANALYSIS

Alterations were made to both the Res-Sim and HEC-RAS models to represent conditions at the end of the implementation period. All six alternatives were re-run, and results were compared between alternatives and to the base condition. All Year 15 alternatives runs were performed in HEC-RAS 5.0.3 in October 2017.

6.2.1 *Geometry*

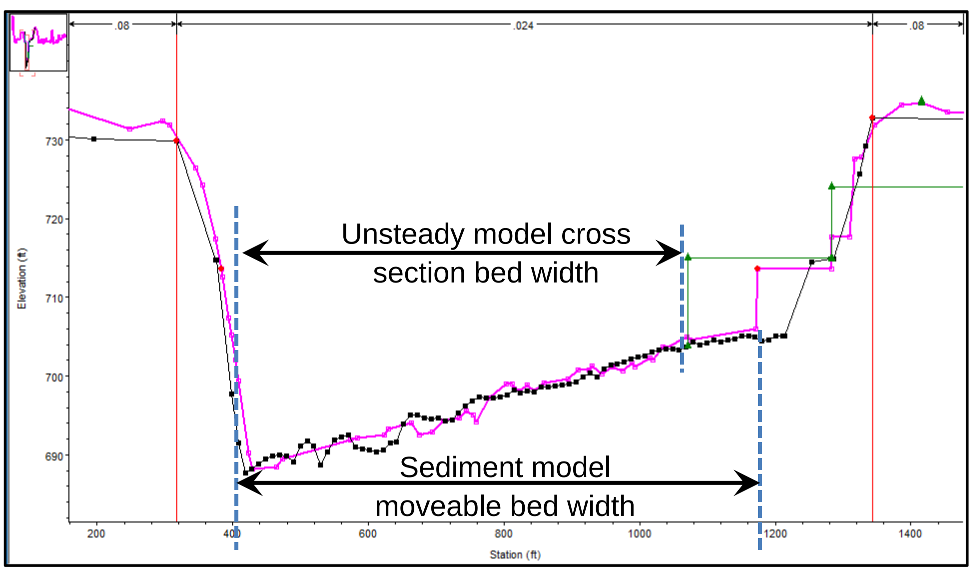
In HEC-RAS, the Missouri River channel bed was adjusted to represent the aggradation or degradation that could be expected by the end of the implementation period. Only the bottom of the channel between the end of the navigation structure and the opposite bank was adjusted vertically, up in areas of forecasted aggradation, down in areas of forecasted degradation. Figure 6-1 shows a cross section that was adjusted for anticipated degradation.

Figure 6-1. Cross Section vertical adjustment for Year 15



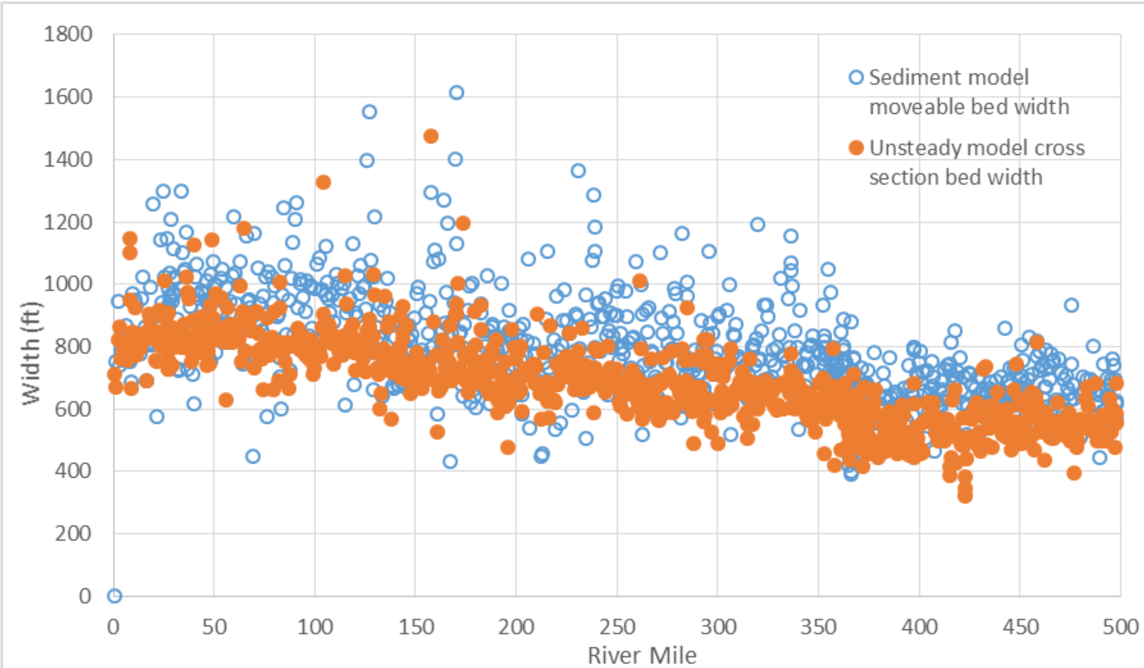
Vertical adjustment applied to the HEC-RAS cross sections varied by river mile and by alternative, depending on the results output from the LMRSM. Area change from the LMRSM was provided, rather than vertical adjustment to the bed, so as not introduce additional errors in translating sediment modeling bed change predictions into the unsteady HEC-RAS model. The two models are of the same river, and have cross sections at nearly the same locations, but have different active bed widths due to the technique with which the navigation structures were modeled, as shown in Figure 6-2.

Figure 6-2. Unsteady vs Sediment model – cross section comparison



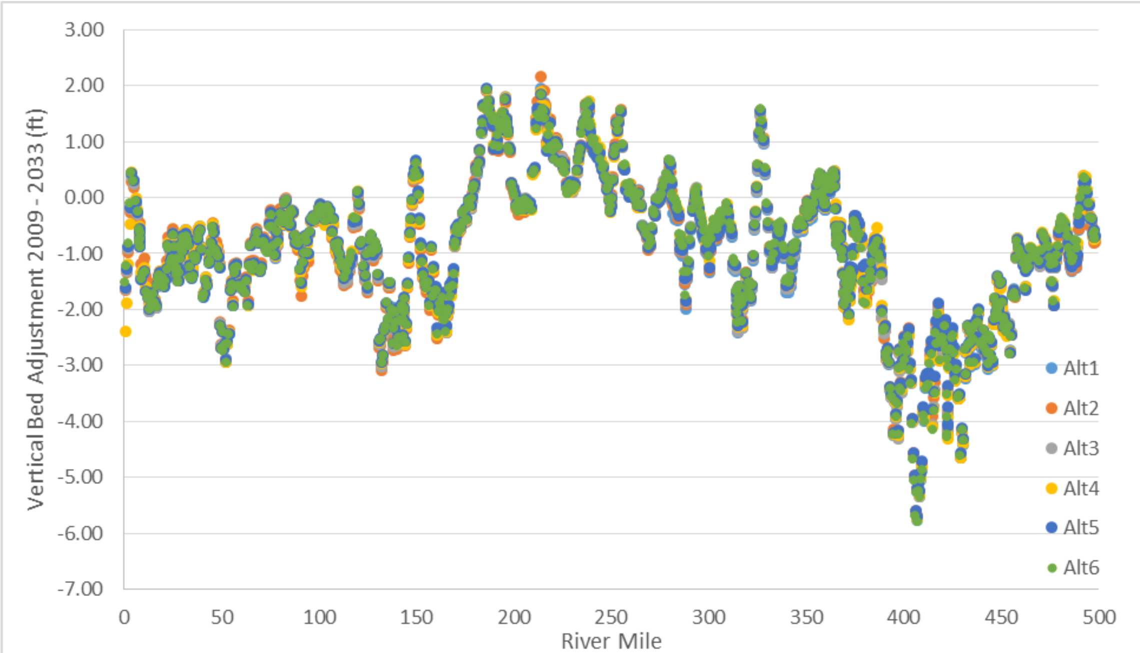
Navigation structures in the sediment model were represented with ground points, and at a reduced length to account for contraction/ expansion of flows between structures. Navigation structures in the unsteady HEC-RAS model were represented with permanent ineffective flow to the full length of the structure. Therefore, the unsteady HEC-RAS model generally has narrower cross section bed widths on the entire river, as shown in Figure 6-3.

Figure 6-3. Unsteady vs Sediment model – bed widths



To calculate aggradation/ degradation the area change (in square feet) was divided by the unsteady model cross section bed width (in feet) to provide a vertical bed adjustment (in feet) for the Year 15 geometry. A positive adjustment is aggradation, a negative adjustment is degradation. Vertical adjustments applied to the unsteady HEC-RAS model geometries for all alternatives are shown in Figure 6-4.

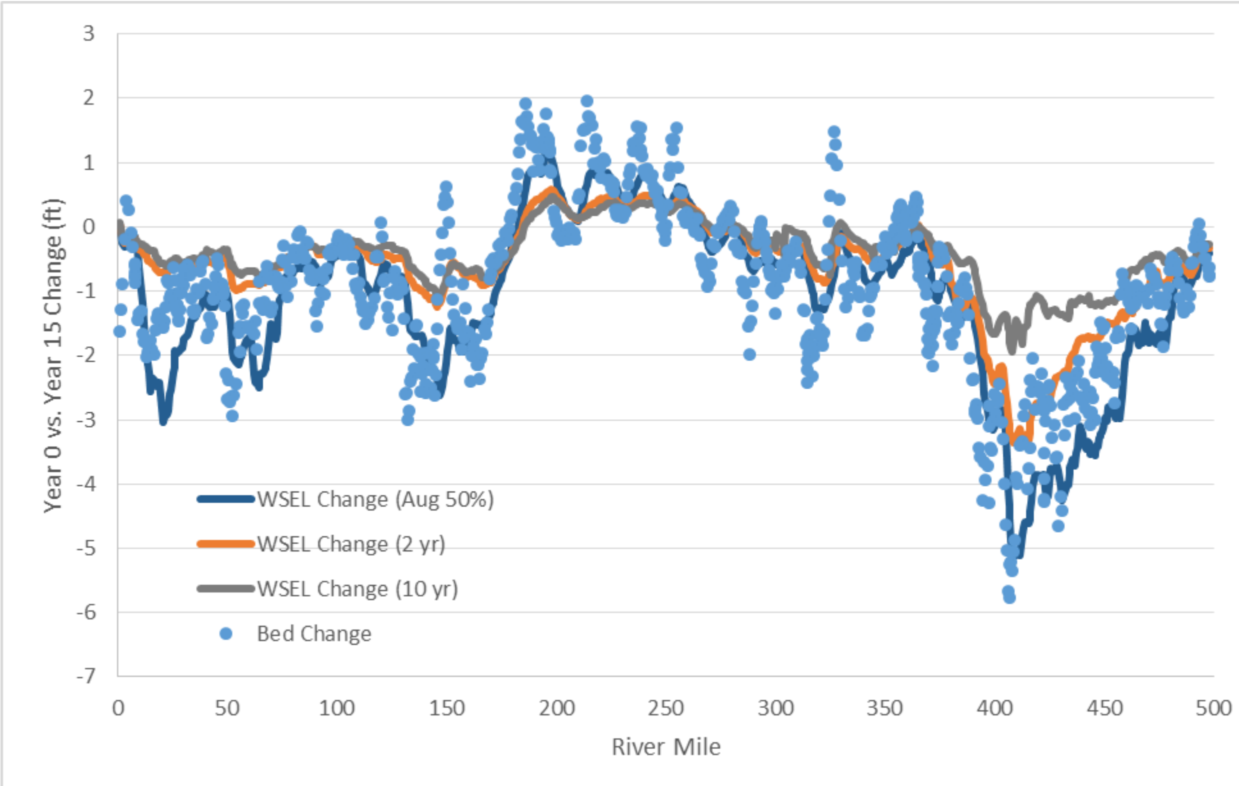
Figure 6-4. Year 15 Aggradation/ Degradation applied to Alternative Geometries



All six alternatives had nearly identical aggradation/ degradation trends along the river, ranging from -5.8 feet of degradation downstream of St. Joseph to +2.8 feet of aggradation in the vicinity of Lisbon/Jameson. Different vertical adjustments were applied to each alternative, however the differences between alternatives were minor, only varying by +/- 0.1 foot on the average, with isolated locations varying by no greater than 0.4-ft.

A simple steady flow water surface comparison was made between the Year 0 geometry and Year 15 geometry for three profiles: August 50%, 2 year, and 10 year. Change in water surface verses the bed change applied to the Alternative 1 geometry are shown in Figure 6-5. The water surface response is slightly lagged in the upstream direction but generally always tracks with the direction and magnitude of bed change. August 50% flows are well within the banks of the river and very sensitive to bed changes, the water surface response magnitude is nearly one to one in some areas with the bed change applied. Higher flows that are at or above the river banks are less sensitive to the bed changes, as would be expected. Alternatives 2-6 show the same trends. A steady flow comparison does not take into account the flow changes out of the reservoir, for that the full period of record evaluation is needed.

Figure 6-5. Bed change vs. Water Surface Elevation (WSEL) change



6.2.2 *Flows*

Reservoir storage volumes were adjusted in Res-Sim to represent sedimentation in the reservoirs that could be expected by the end of the implementation period. Reservoir operation rules were left the same as the base set of alternatives. All six alternatives were re-run in Res-Sim for the period of record, resulting in slightly different release decisions from Gavins Point in some years. The Gavins Point output is the input for the HEC-RAS models, resulting in slightly different flows in the Missouri River mainstem for the Year 15 analysis. Refer to *Mainstem Missouri River Reservoir Simulation Alternatives Technical Report* (USACE, 2018) for details regarding the Res-Sim modeling.

6.2.3 *Results*

Output from the Year 15 analysis can be evaluated in two ways. First, comparison of the Year 15 alternatives to the Year 15 No Action provides additional data to inform on how alternative performance may vary in the future for consideration with the selection of a preferred alternative. Second, comparison of the Year 15 alternatives to the base condition provides a sense of how future channel bed and reservoir sedimentation conditions may impact Missouri River flows and stages. The second evaluation is limited in the useful information it can provide to the decision making process, as the results are directly tied to modeling assumptions that were made about an unknown future using historic data. The first evaluation parallels the comparisons made amongst alternatives in the base condition analysis. Visual and statistical evaluation of the Year

15 output indicates that regardless of changed conditions, the alternatives compare similarly to each other.

Statistical whisker plots for the five major locations from Graham Long's Hydrology Visualization Tool (Long, 2017) are in Figure 6-6 through Figure 6-10. The figures present the statistics of minimum, maximum, 5th percentile, 95th percentile, median and mean for all six alternatives for the Year 0 base condition and Year 15.

Figure 6-6. Flow/ stage statistics at St. Joseph, MO

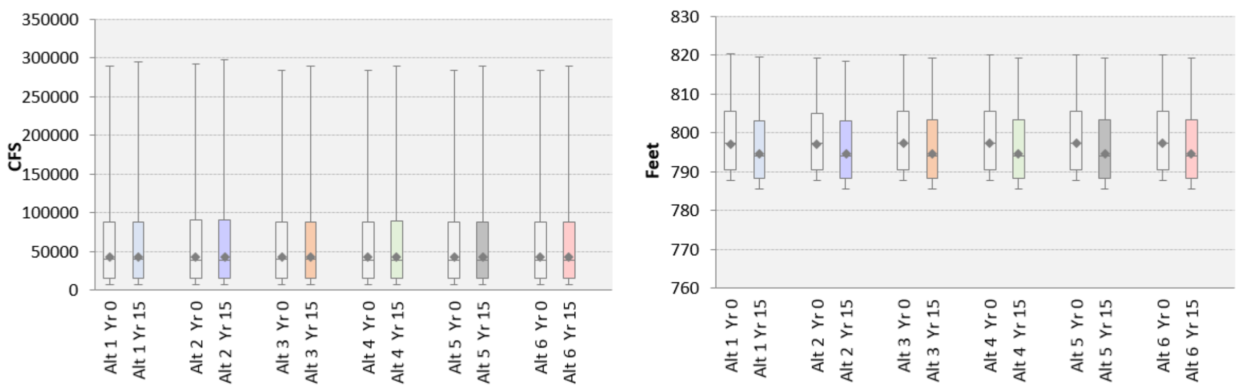


Figure 6-7. Flow/ stage statistics at Kansas City, MO

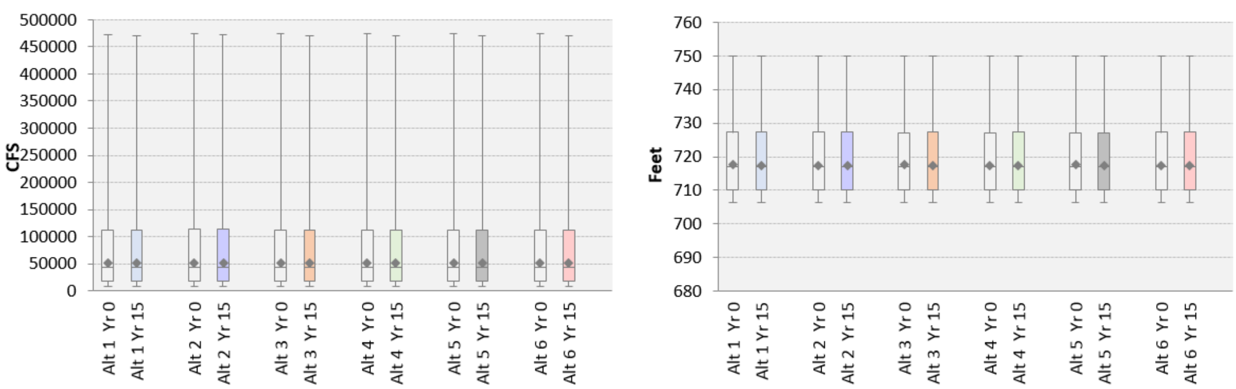


Figure 6-8. Flow/ stage statistics at Boonville, MO

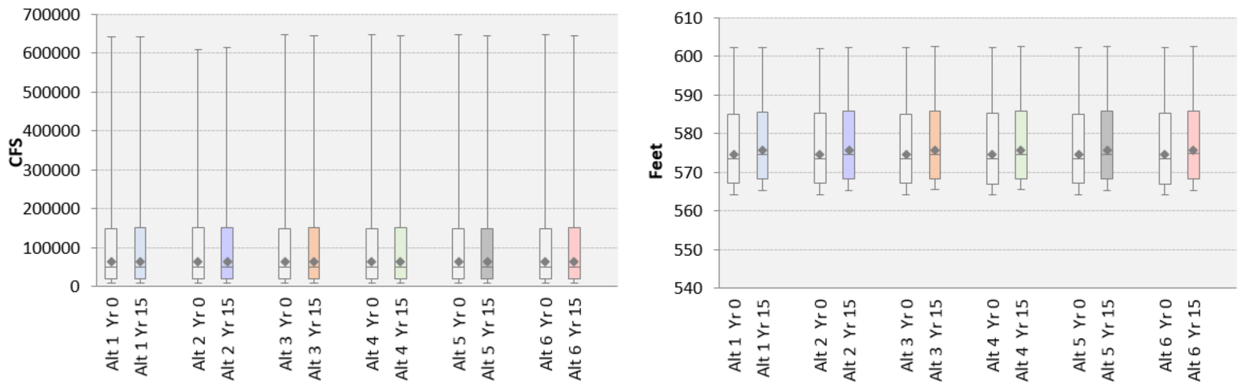


Figure 6-9. Flow/ stage statistics at Hermann, MO

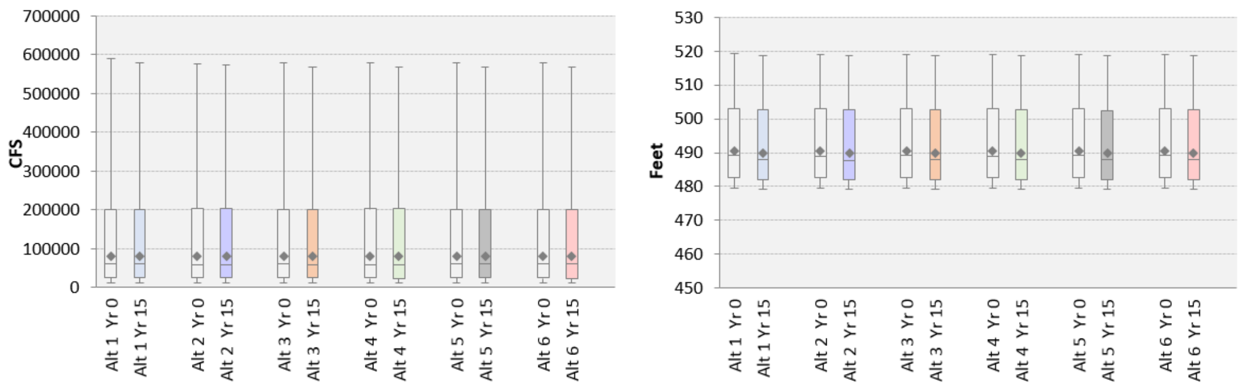
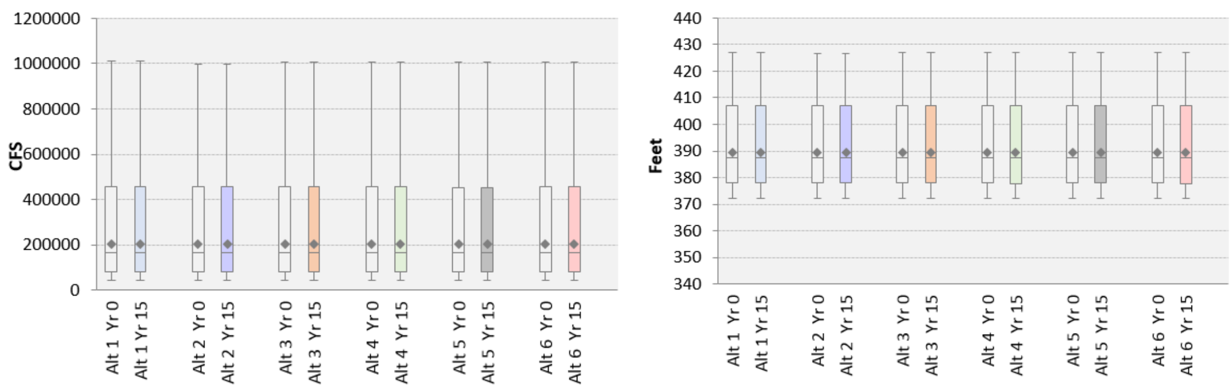


Figure 6-10. Flow/ stage statistics at St. Louis, MO



Although comparisons between Year 15 and Year 0 have limited usefulness in alternative selection, a few trends are worth pointing out. First, differences in flow statistics between Year 0 and Year 15 on the lower river are almost negligible. Any differences that do show up can be attributed to release changes out of Gavins due to different storage volumes in the reservoirs. These differences decrease moving downstream. Other flow differences compared to Year 0 could be due to slightly different routing because of changed channel capacities. These would accumulate and be greater moving downstream, but they are slight in magnitude and pretty difficult to discern from model noise. Second, differences in stage statistics between Year 0 and Year 15 on the lower river are directly related to the vertical bed change applied locally. Water surfaces are lower in areas of degradation and higher in areas of aggradation, as can be seen in the whisker plots. For example, significant degradation applied in the St. Joseph reach for all alternatives results in lower stage statistics for all six alternatives. As stated previously, the relative comparison between alternatives for Year 15 produces very similar results to the base condition analysis.

6.3 Interior Drainage Analysis

Flow hydrographs and modified cross sections from the Year 15 analysis were incorporated into the L-488 and L-246 interior drainage models. The models were re-run for the period of record, seepage was re-calculated and the period of record was re-run including seepage. All Year 15 alternatives runs were performed in HEC-RAS 5.0.3 in December 2017. Model output was compared between alternatives and to the base condition. Cross sections in the vicinity of L-488 were modified for degradation, resulting in lower water surface levels along the levee unit, and therefore lower ponding levels in the leveed area for the No Action and all alternatives. Cross sections in the vicinity of L-246 were modified for aggradation, resulting in higher water surface levels along the levee unit, therefore increasing ponding levels in the leveed area for the No Action and all alternatives. However, the relative comparison between alternatives for Year 15 produces very similar results to the base condition analysis.

7 CONCLUSIONS

The unsteady HEC-RAS model analysis gives a means to systematically evaluate differences in river elevations for various reservoir and habitat alternatives given the limitations presented in Section 4.3. These results can be fed into additional species and human considerations models, such as HEC-FIA, to screen alternatives for relative benefits and potential economic impacts. The outputs should be carefully examined with an eye towards the model limitations and judgment applied where needed to mitigate any potential pitfalls of the hydraulic analysis. An advantage to the alternative modeling in this current study compared to the Master Manual is the ability to account for differences in flow routings and river stages with varying amounts and distributions of habitat. Additional modeling was performed to provide estimates of how ongoing sedimentation processes may affect the performance of future alternatives. While not intended to represent detailed estimates of future channel conditions, the Year 15 results do provide an alternative comparison methodology that was evaluated qualitatively rather than quantitatively for economic and human consideration impacts.

Flow change alternatives show the largest change in flow and stage duration statistics relative to No Action in upstream reaches. Downstream reaches, such as downstream of major tributaries such as the Kansas, Grand, Chariton, Osage and Gasconade Rivers show increasingly less change as drainage area increases. If flow change alternatives are considered for implementation, additional risk and uncertainty analysis is recommended to more comprehensively quantify risk of spring or fall pulse flows.

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Appendix E

Attachment 1 – Revision Log

Missouri River Unsteady HEC-RAS Model
Revision Log

Revision Number	Date	Description of changes	Distribution
Rev 1	6/9/2015	Bed data was corrected to account for Temporary Bench Mark (TBM) errors discovered during a spring 2015 quality control check of the hydrosurvey spreadsheet. 8 TBMs were found to be in error, impacting 64 model cross sections. The maximum positive shift in bed data was 1.27 and the maximum negative shift was -1.31. Cross sections were only modified if the impact was greater than two tenths of a foot. Modifications were between the upstream limit of cross section 449.44 near the St. Joseph gage, and downstream limit of cross section 286.16 near Waverly. Calibration was checked at each gage from St. Joseph to Waverly. Calibration at St. Joseph was improved after the changes, calibration at Kansas City remained unchanged, and at Waverly the flow varied factors were adjusted to return the calibration to Post-ATR model accuracy. Calibration Metrics were not re-calculated because the changes were so small it is not likely to significantly change the calibration precision of the model.	(1) National Weather Service, (2) Riverside for IDEKER lawsuit
		Calibration of the Mississippi River at gage locations was re-evaluated and slightly improved.	
		Lateral structure and storage area connection weir coefficients in the crossover area were double checked for consistency and revised. All SAC weir coefficients were lowered to 0.3, except the railroad (1.5) and the interstate (2). All lateral structure weir coefficients on the crossover reach were lowered to 0.3, except a few that were obviously high ground (0.5).	
		Storage area connections at Rulo representing the Railroad and Highway 159 have been renamed to: rr1, rr2, rr3, rr4 and hwy1-2, hwy3, hwy4, and hwy5.	
		Corrections made to SAs/LSs/SACs after examining period of record leveed area flooding: removed closure gaps on lateral structures: 455a, 385q, 385r, arm, cid-k, sal making the assumption that the gaps will be closed properly prior to flooding. Connected LS 220.1 how3 to the proper SA. Fixed bogus low spot at the beginning of how4.	
		Corrections made to bank sta xsec 14.01 and 15.1. Discovered while making modifications to base model for alternatives evaluation.	
Rev 2	9/11/2015	Lateral Structures were tweaked so that they did not extend into junction zones, which are downstream of the last cross section in a reach. This was necessary for the upgrade to the new version of RAS 5.0.	None
		Corrections to cross sections 479.8, 479.09, 478.4, first ineffective area representing navigation structure was switched from temporary to permanent.	
		Correction to cross section 472.06, ineffective & levee point stationing were not aligned.	
		Adjusted levee point at cross section 311.07 to elev 690.6	
	10/21/2015	Confirmed run compatibility with RAS 5.0 Release Candidate 2	

Revision Number	Date	Description of changes	Distribution
Rev 2 (cont'd)		Removed initial conditions on all SAs, except a few leveed areas that need a starting water surface for stability at initial overtopping. In previous versions of HEC-RAS, if a lateral structure had an open connection to the river (such as a storage area that represents a tieback) the initial water surface in the receiving storage area had to be close to the water surface in the river or the model would go unstable in the first timestep. A spreadsheet was developed to auto populate the initial conditions. However, in the new version of RAS 5.0, if a SA initial condition is left blank the default starting water surface is equal to the river if water is over the lateral structure (if not, the default starting water surface is set to empty). Therefore, the model is more versatile for distribution with the initial conditions set to blank.	
Rev 3	10/28/2015	Added culverts with flap gates to drain the storage areas post floods SAC chrt-tuqt was accidently connected to tuq not tuqt LS blt1 and crk swapped river stationing on blt1 and crk lateral structures SAC sug1-2 removed ditch that was draining sug1 out the north side	
	11/4/2015	xsec 177.28 repaired at sandbar LS wol3 and wol3-t lowered weir coeffs from 1.5 to 1 LS cnc01 lowered weir coeff from 2 to 1.5 raised culverts to be equal to starting ws elev in wol3 (745.5), gr03 (651), gr04 (650), gr05 (649.7), cnc07 (427.6)	(1) Final Run of No Action Alternative for Draft EIS
	11/16/2015	SAC capt-2 lowered weir coeff from 1.5 to 1 LS cnc01 lowered first weir coeff (27.871) from 1.5 to 1 SAC gr09 and gr08 lowered weir coeffs from 1.5 to 1 SAC ckl lowered weir coeff from 1.5 to 1 SAC teb3-mokt lowered weir coeff (0.75 to 0.5) SAC mokt-mok lowered weir coeff (2 to 1.5) LS teb3 lowered weir coeff from 0.75 to 0.5 LS mok lowered weir coeff from 2 to 1.5	(2) Final Run BiOp and IRC Alternatives for Draft EIS (3) Year 15 (2032deg) runs

Appendix E

Attachment 2 – Floodplain Connectivity Metadata

Process:

1. Run HEC-RAS steady flow plan for 2-yr and 5-yr UMRSFFS flows
2. Export GIS data for GeoRAS: cross sections and profiles only, not including storage areas
3. In GIS, using GeoRAS commands, follow the steps to create inundation area for 5-yr
4. Widened bounding polygon to include areas that did not get captured by cross section extents only, but should be mapped as wet in the 5-yr [boundary.shp]
5. Identified (excluded) areas that should not be counted as floodplain connectivity (even though they may be inundated at the 5-yr level) [excluded.shp]
6. Re-computed inundation area for 5-yr and computed inundation area for 2-yr with new bounding polygon

   Repeat steps 7-16 for both 5-yr and 2-yr:

7. Create a new file geodatabase [Math5yr.gdb] and import inundation boundary [b5yr_raw]
8. Use Repair Geometry tool on boundary [b5yr_raw]
9. Erase SAs (SAs included tributaries/tiebacks, however, when mapping for future purposes may actually want to create a new SA set for this step without tribs and high ground connections so these areas are mapped … wish list item) [b5yr_eraseSA]
10. Erase Excluded areas [b5yr_eraseExcluded]
11. Select the largest polygon (assume this represents all inundation areas with connectivity to the river) and export to new shapefile [b5yr_one]
12. Import state boundaries [states.shp] and BiOp segment boundaries [biopsegments.shp] into the file geodatabase
13. Split by state [Kansas, Missouri, Nebraska] and split by BiOp Segment [segment13, segment14, segment15]
14. Combine split feature classes into one and compute acres [b5yr_acres]
15. Export to final directory [5yr_existing.shp]
16. Subtract acres of river [River.gdb] (see next process) in each state for final acres of floodplain connectivity
17. To view in Google Earth use the dice tool (20,000 verticies) and then in Google Earth select File > Import [5yr_dice]

Process for calculating river:

1. Obtained depth grid of August 50% from RAS Mapper computed as a part of the HAMP SWH accounting effort (three segments, all .vrt file type) [1Depth (50%).vrt, 2Depth (50%).vrt, 3Depth (50%).vrt]
2. Used Reclassify tool in GIS to make an img with cell value of 1 where there was water and cell value of NoData where there was no water (one for each segment) [reclass1, reclass2, reclass3]
3. Used Raster to Polygon tool to create polygon with a perimeter at the bounds of the img (one for each segment) [poly1, poly2, poly3]
4. Merge the polygons into one shapefile [poly_all]

5. Use the clip tool to get rid of extra stuff (started with Ben's HAMP clip polygon and modified it to also remove all wet areas in chutes and Omaha area) [poly_all_clip]
6. Select the largest shapefile(s) to minimize size and complexity and hopefully remove some of the SWH areas
7. Created a new file geodatabase [River.gdb] and imported the largest shapefile from previous step for the split calculation [poly_one]
8. Import state boundaries [states.shp] and BiOp segment boundaries [biopsegments.shp] into the file geodatabase
9. Split by state [Kansas, Missouri, Nebraska] and split by BiOp Segment [segment13, segment14, segment15]
10. Combine split feature classes into one and compute acres [river]
11. Use dice tool for viewing in google earth [river_dice]

Appendix E

Attachment 3 – Rulo to Mouth Alternative Statistics

St. Joseph, MO

Flow | **Stage**

Min, Max, Percentile Statistics on the period of record (March 1930 - Dec 2012) hydrographs

Flow

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	6,969	6,968	6,969	6,969	6,969	6,969
10%	18,007	18,097	18,029	17,887	18,007	17,859
25%	25,043	24,819	25,096	24,688	25,003	24,665
50%	39,546	39,203	39,456	39,303	39,336	39,416
75%	50,485	50,555	50,457	50,760	50,533	51,109
90%	68,611	69,488	68,735	70,548	69,742	69,657
Max	289,014	292,309	283,954	283,928	283,930	283,933

Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	787.7	787.7	787.7	787.7	787.7	787.7
10%	791.2	791.3	791.2	791.2	791.2	791.2
25%	793.4	793.3	793.4	793.3	793.4	793.3
50%	797.4	797.3	797.4	797.4	797.4	797.4
75%	799.6	799.4	799.6	799.7	799.7	799.7
90%	802.5	802.3	802.6	802.8	802.8	802.7
Max	820.3	819.1	820.2	820.2	820.2	820.2

Min, max, percentile change from No Action

Flow

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-1	0	0	0	0
10%	-	90	21	-120	0	-148
25%	-	-224	53	-355	-40	-378
50%	-	-343	-90	-244	-211	-130
75%	-	70	-28	275	49	624
90%	-	877	125	1,937	1,131	1,046
Max	-	3,295	-5,060	-5,086	-5,084	-5,081

Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	-0.1	0.0	-0.1	0.0	-0.1
50%	-	-0.2	0.0	0.0	0.0	0.0
75%	-	-0.2	0.0	0.1	0.0	0.1
90%	-	-0.3	0.1	0.3	0.2	0.2
Max	-	-1.1	0.0	-0.1	-0.1	-0.1

Min, max, percentile on the daily change from No Action

Flow

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-38,478	-11,844	-28,804	-44,309	-32,537
10%	-	-2,089	-80	-1,127	-506	-1,129
25%	-	-700	-17	-102	-47	-137
50%	-	-16	0	-1	0	-1
75%	-	35	22	6	13	6
90%	-	646	132	79	116	100
Max	-	43,838	16,986	42,686	61,366	44,249

Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-5.6	-2.4	-6.5	-8.0	-7.8
10%	-	-0.9	0.0	-0.2	-0.1	-0.2
25%	-	-0.3	0.0	0.0	0.0	0.0
50%	-	-0.2	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.1	0.1	0.1	0.1
Max	-	6.8	2.9	7.1	6.9	8.1

Kansas City, MO

Flow / **Stage**

Min, Max, Percentile Statistics on the period of record (March 1930 - Dec 2012) hydrographs

Flow

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	7,930	7,930	7,930	7,930	7,930	7,930
10%	20,831	20,962	20,854	20,627	20,797	20,597
25%	30,388	29,745	30,440	30,007	30,366	29,949
50%	44,207	43,992	44,144	43,996	44,209	44,220
75%	62,233	62,096	62,206	63,021	62,646	62,963
90%	89,421	89,907	89,440	89,907	89,273	89,270
Max	472,894	473,919	473,742	473,698	473,676	473,719

Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	706.4	706.4	706.4	706.4	706.4	706.4
10%	711.2	711.2	711.2	711.1	711.2	711.1
25%	713.7	713.6	713.7	713.6	713.7	713.6
50%	717.1	717.0	717.0	717.0	717.1	717.1
75%	720.4	720.3	720.4	720.5	720.5	720.5
90%	724.3	724.3	724.3	724.4	724.3	724.3
Max	750.0	750.1	750.0	750.0	750.0	750.0

Min, max, percentile change from No Action

Flow

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-1	0	0	0	0
10%	-	132	24	-204	-34	-234
25%	-	-643	52	-381	-22	-438
50%	-	-215	-63	-211	2	13
75%	-	-137	-27	788	413	730
90%	-	485	18	485	-148	-151
Max	-	1,026	848	805	783	826

Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	-0.1	0.0	-0.1
25%	-	-0.2	0.0	-0.1	0.0	-0.1
50%	-	-0.1	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.1	0.1	0.1
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	0.0	0.0	0.0	0.0	0.0

Min, max, percentile on the daily change from No Action

Flow

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-40,509	-11,084	-30,907	-46,258	-32,434
10%	-	-2,147	-127	-1,154	-519	-1,132
25%	-	-719	-31	-154	-77	-193
50%	-	-18	0	-3	0	-4
75%	-	50	32	10	20	9
90%	-	739	185	122	174	149
Max	-	44,850	18,394	45,708	61,449	54,790

Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-4.8	-2.2	-5.8	-6.3	-6.9
10%	-	-0.4	0.0	-0.2	-0.1	-0.2
25%	-	-0.1	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.1	0.0	0.0	0.0	0.0
Max	-	6.7	2.3	6.3	6.6	7.7

Boonville, MO

Flow

Min, Max, Percentile Statistics on the period of record (March 1930 - Dec 2012) hydrographs

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	9,284	9,285	9,284	9,284	9,284	9,284
10%	23,435	23,543	23,460	23,211	23,389	23,178
25%	36,048	35,233	36,112	35,536	35,985	35,514
50%	49,468	49,325	49,417	49,311	49,510	49,461
75%	75,151	75,240	75,334	76,149	76,009	76,335
90%	114,802	115,564	114,857	115,398	114,609	114,750
Max	642,384	610,192	646,186	646,168	646,149	646,198

Min, max, percentile change from No Action

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	1	0	0	0	0
10%	-	108	25	-224	-46	-257
25%	-	-816	63	-512	-64	-534
50%	-	-143	-51	-157	42	-7
75%	-	88	183	997	858	1,183
90%	-	762	55	596	-193	-52
Max	-	-32,192	3,802	3,784	3,765	3,813

Min, max, percentile on the daily change from No Action

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-74,982	-31,384	-31,788	-46,011	-32,018
10%	-	-2,385	-138	-1,174	-549	-1,148
25%	-	-777	-28	-171	-75	-217
50%	-	-25	0	-2	0	-2
75%	-	117	44	16	29	17
90%	-	1,066	201	136	190	171
Max	-	45,686	19,186	46,656	63,530	47,348

Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	564.1	564.1	564.1	564.1	564.1	564.1
10%	568.0	568.0	568.0	567.9	568.0	567.9
25%	570.9	570.8	570.9	570.8	570.9	570.8
50%	573.5	573.5	573.5	573.5	573.5	573.5
75%	577.5	577.5	577.6	577.6	577.6	577.6
90%	581.8	581.8	581.9	581.9	581.9	581.9
Max	602.3	602.1	602.4	602.4	602.4	602.4

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	-0.1
25%	-	-0.2	0.0	-0.1	0.0	-0.1
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.1	0.1	0.1	0.1
90%	-	0.0	0.1	0.1	0.1	0.1
Max	-	-0.2	0.1	0.1	0.1	0.1

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-4.4	-1.9	-5.0	-5.3	-5.6
10%	-	-0.3	0.0	-0.1	0.0	-0.1
25%	-	-0.1	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.1	0.1	0.1	0.1
90%	-	0.1	0.2	0.2	0.2	0.2
Max	-	5.5	2.0	5.6	5.6	6.5

Hermann, MO

Flow / Stage

Min, Max, Percentile Statistics on the period of record (March 1930 - Dec 2012) hydrographs

Flow

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	10,712	10,711	10,712	10,712	10,712	10,712
10%	29,093	29,192	29,121	28,729	29,081	28,716
25%	43,399	42,828	43,366	43,242	43,252	43,220
50%	59,497	59,286	59,442	59,207	59,544	59,456
75%	97,001	97,247	96,988	97,315	97,507	97,439
90%	153,014	153,537	152,931	153,440	152,644	152,986
Max	591,123	575,093	578,332	578,131	578,051	578,317

Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	479.6	479.6	479.6	479.6	479.6	479.6
10%	483.5	483.6	483.6	483.5	483.5	483.5
25%	486.5	486.4	486.5	486.5	486.5	486.5
50%	489.1	489.0	489.1	489.0	489.1	489.1
75%	493.9	493.9	493.9	493.9	494.0	494.0
90%	499.7	499.7	499.7	499.7	499.6	499.7
Max	519.1	518.9	518.9	518.9	518.9	518.9

Min, max, percentile change from No Action

Flow

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-1	0	0	0	0
10%	-	99	28	-364	-12	-377
25%	-	-571	-34	-157	-147	-179
50%	-	-211	-54	-289	47	-41
75%	-	246	-13	314	505	437
90%	-	523	-83	426	-370	-28
Max	-	-16,030	-12,791	-12,992	-13,072	-12,806

Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	-0.1	0.0	-0.1
25%	-	-0.1	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	-0.1	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	-0.2	-0.2	-0.2	-0.2	-0.2

Min, max, percentile on the daily change from No Action

Flow

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-49,851	-13,912	-34,681	-45,999	-34,678
10%	-	-2,434	-192	-1,235	-667	-1,184
25%	-	-795	-37	-211	-92	-257
50%	-	-27	0	-2	0	-2
75%	-	146	64	30	47	32
90%	-	1,172	257	191	248	240
Max	-	44,525	18,290	52,713	62,269	46,515

Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-3.5	-2.1	-4.4	-4.8	-5.1
10%	-	-0.3	0.0	-0.2	-0.1	-0.2
25%	-	-0.1	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.1	0.0	0.0	0.0	0.0
Max	-	5.5	2.0	5.4	5.5	6.1

St Louis, MO

Flow

Min, Max, Percentile Statistics on the period of record (March 1930 - Dec 2012) hydrographs

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	44,333	44,334	44,333	44,333	44,333	43,916
10%	94,340	94,394	94,372	94,050	94,334	93,988
25%	122,269	121,770	122,289	121,909	122,644	122,230
50%	166,420	165,941	166,410	166,409	166,477	166,475
75%	246,564	246,448	246,473	246,965	246,656	247,114
90%	372,373	373,350	372,434	373,652	371,533	373,042
Max	1,011,086	998,200	1,007,790	1,007,736	1,007,724	1,007,782

Min, max, percentile change from No Action

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0	0	0	0	-417
10%	-	54	32	-290	-6	-352
25%	-	-499	20	-360	375	-39
50%	-	-480	-11	-12	57	55
75%	-	-116	-90	401	92	550
90%	-	977	61	1,279	-840	669
Max	-	-12,886	-3,296	-3,350	-3,362	-3,305

Min, max, percentile on the daily change from No Action

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-51,895	-12,517	-33,295	-46,093	-33,226
10%	-	-2,415	-268	-1,221	-682	-1,175
25%	-	-791	-64	-271	-148	-297
50%	-	-43	1	-8	-1	-11
75%	-	182	99	55	79	58
90%	-	1,170	321	267	312	311
Max	-	44,281	15,884	51,168	62,642	45,980

Stage

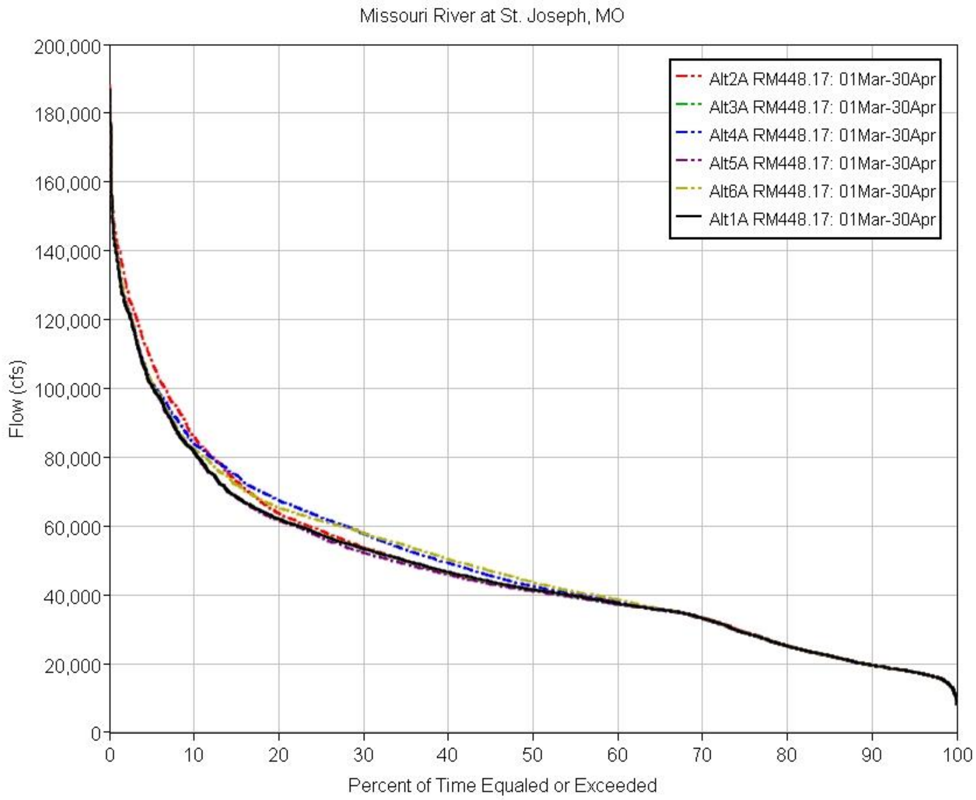
	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	372.3	372.3	372.3	372.3	372.3	372.2
10%	379.8	379.8	379.8	379.8	379.8	379.8
25%	383.0	383.0	383.0	383.0	383.1	383.0
50%	387.5	387.5	387.5	387.5	387.5	387.5
75%	394.1	394.1	394.1	394.1	394.1	394.1
90%	402.4	402.5	402.4	402.5	402.4	402.5
Max	427.0	426.6	426.9	426.9	426.9	426.9

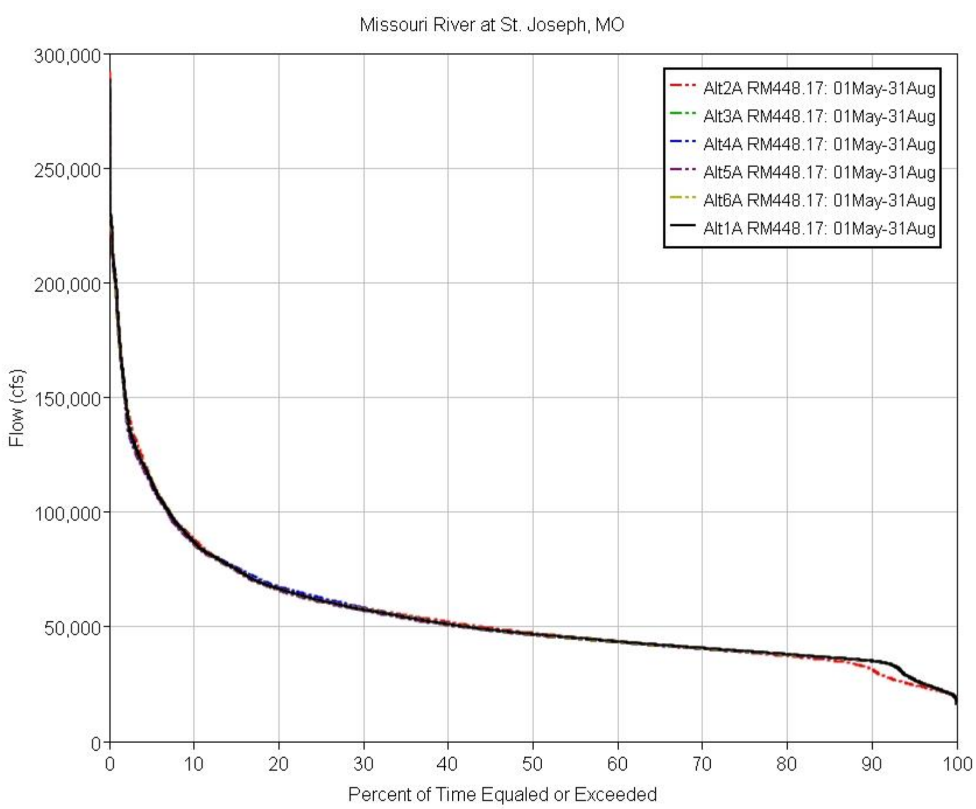
	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	-0.1
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	-0.1	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.1	0.0	0.1	-0.1	0.0
Max	-	-0.3	-0.1	-0.1	-0.1	-0.1

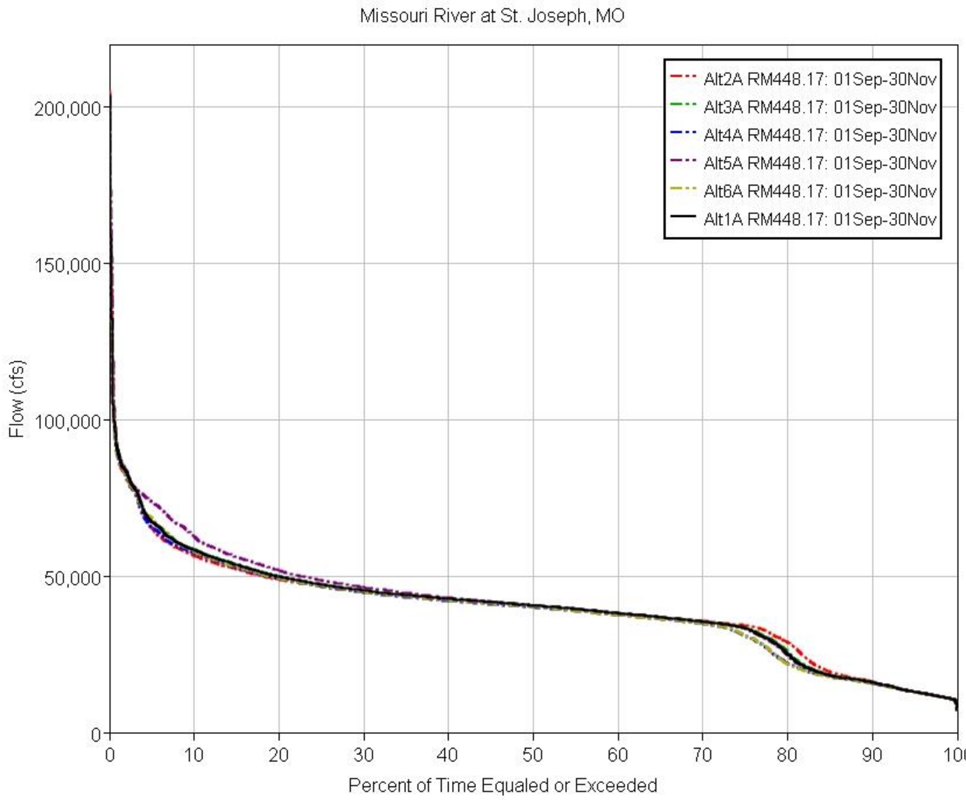
	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-3.1	-1.3	-3.0	-2.8	-3.5
10%	-	-0.2	0.0	-0.1	-0.1	-0.1
25%	-	-0.1	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.1	0.0	0.0	0.0	0.0
Max	-	3.9	1.5	3.5	5.4	3.5

Appendix E

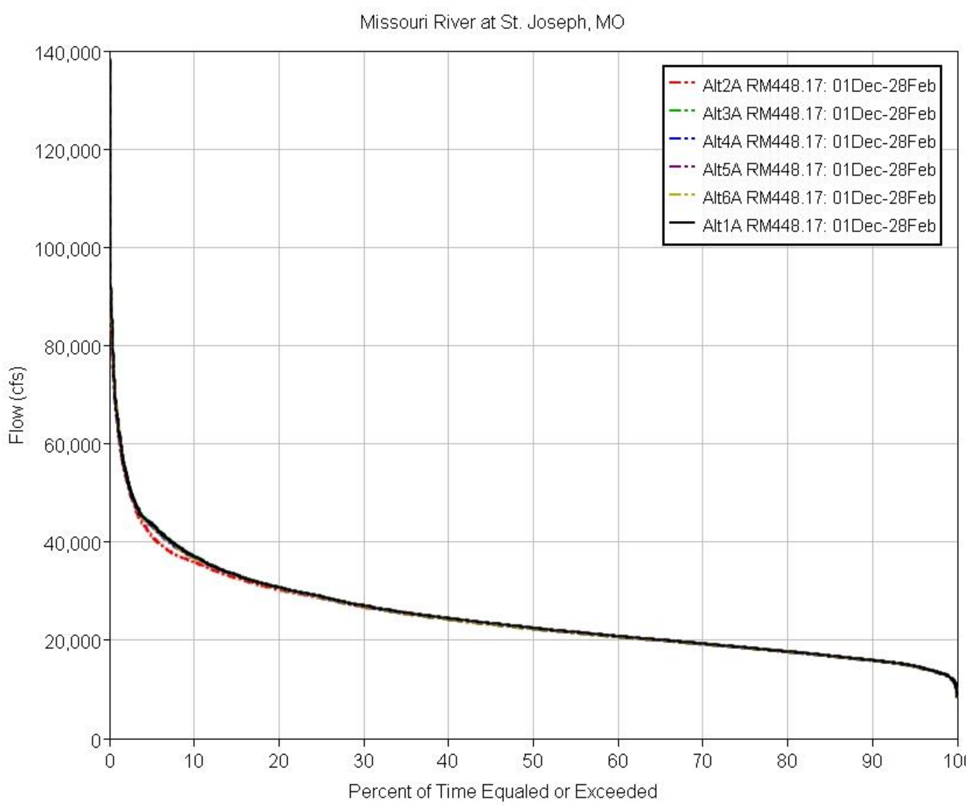
Attachment 4 – Rulo to Mouth Alternative Flow Duration Plots


Missouri River at St. Joseph, MO
Flow (cfs)
Percent of Time Equaled or Exceeded
Alt2A RM448.17: 01Mar-30Apr
Alt3A RM448.17: 01Mar-30Apr
Alt4A RM448.17: 01Mar-30Apr
Alt5A RM448.17: 01Mar-30Apr
Alt6A RM448.17: 01Mar-30Apr
Alt1A RM448.17: 01Mar-30Apr
0
20,000
40,000
60,000
80,000
100,000
120,000
140,000
160,000
180,000
200,000
0
10
20
30
40
50
60
70
80
90
100

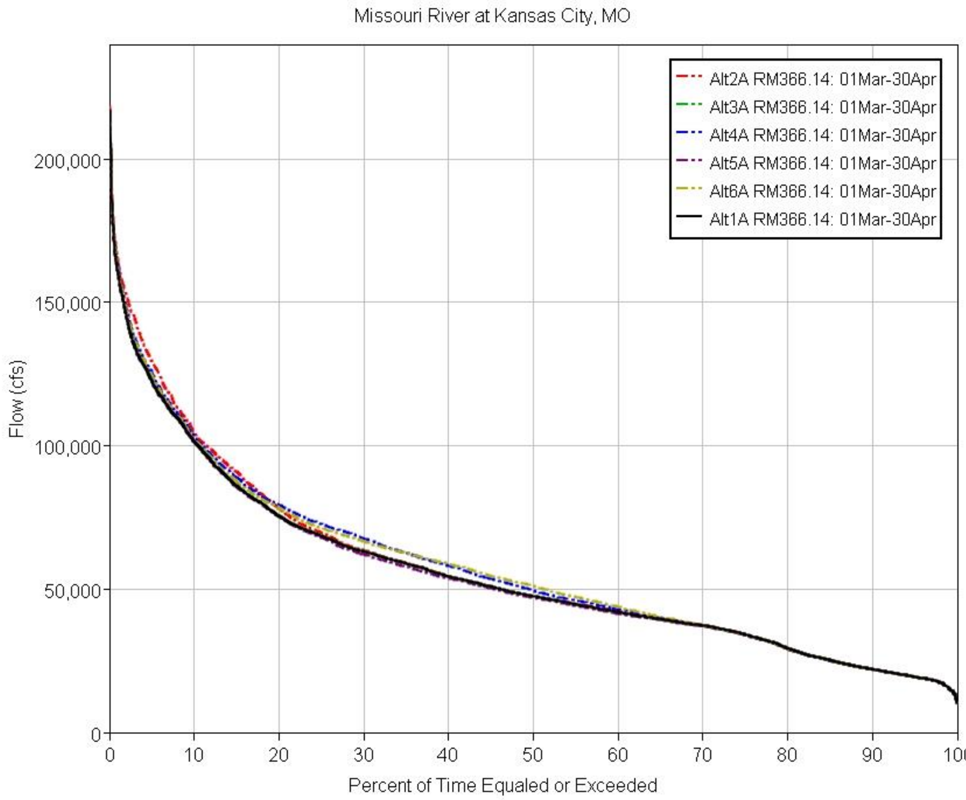

Missouri River at St. Joseph, MO
Flow (cfs)
Percent of Time Equaled or Exceeded
Alt2A RM448.17: 01May-31Aug
Alt3A RM448.17: 01May-31Aug
Alt4A RM448.17: 01May-31Aug
Alt5A RM448.17: 01May-31Aug
Alt6A RM448.17: 01May-31Aug
Alt1A RM448.17: 01May-31Aug
0
50,000
100,000
150,000
200,000
250,000
300,000
0
10
20
30
40
50
60
70
80
90
100

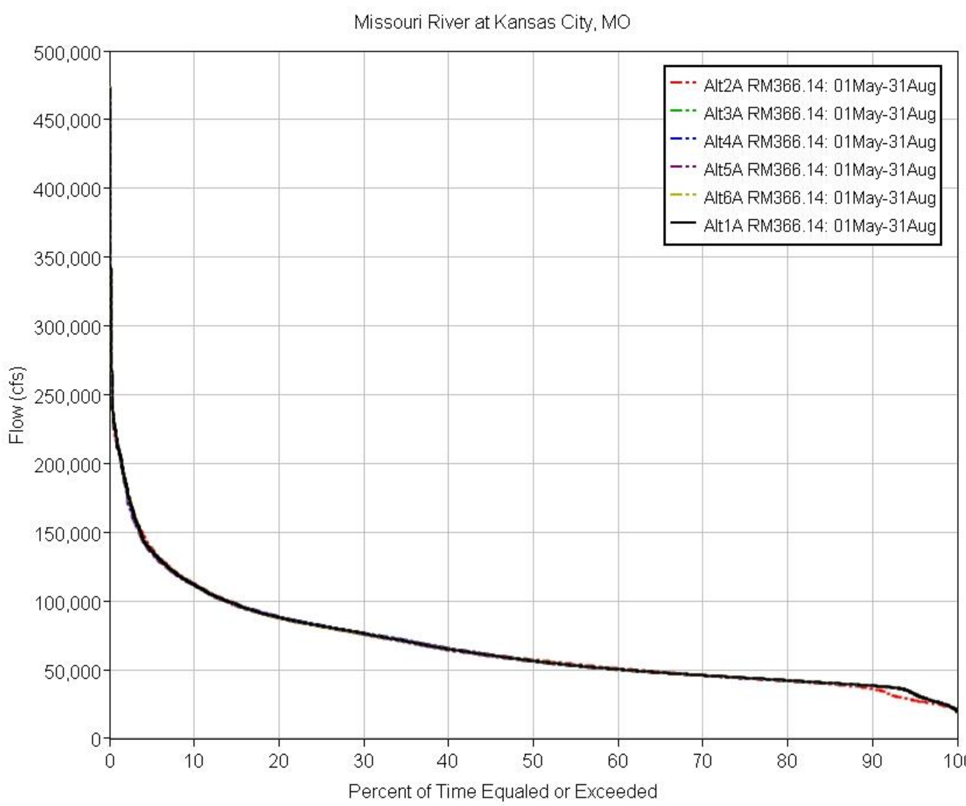


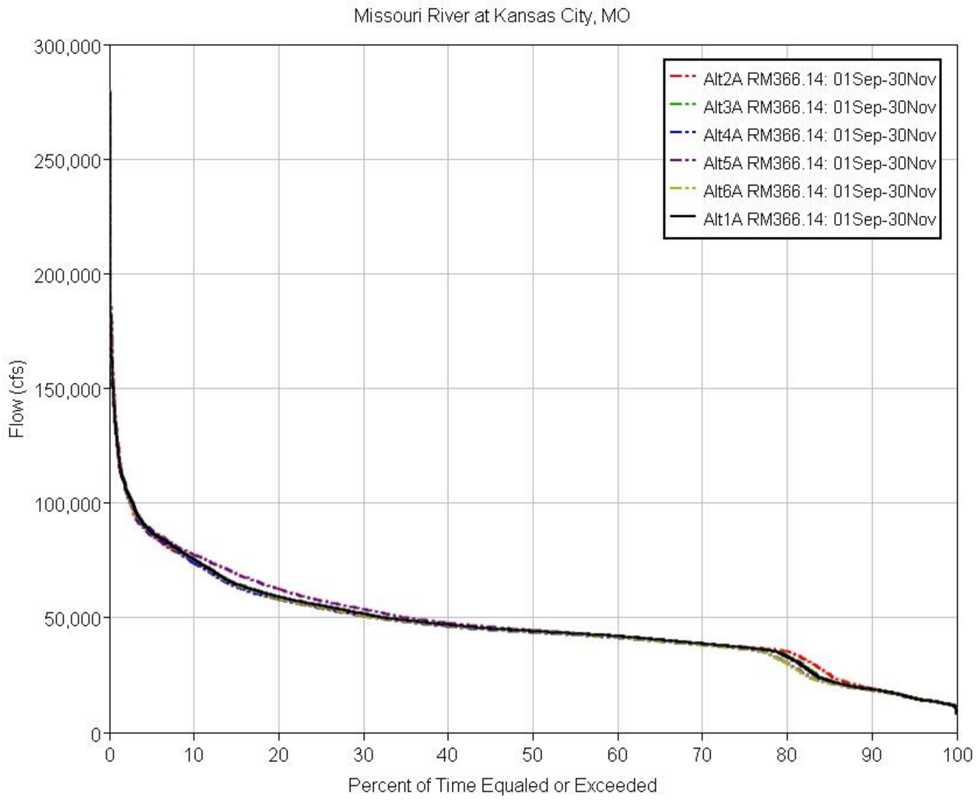
Missouri River at St. Joseph, MO
Alt2A RM448.17: 01Sep-30Nov
Alt3A RM448.17: 01Sep-30Nov
Alt4A RM448.17: 01Sep-30Nov
Alt5A RM448.17: 01Sep-30Nov
Alt6A RM448.17: 01Sep-30Nov
Alt1A RM448.17: 01Sep-30Nov
Flow (cfs)
200,000
150,000
100,000
50,000
0
0
10
20
30
40
50
60
70
80
90
100
Percent of Time Equaled or Exceeded

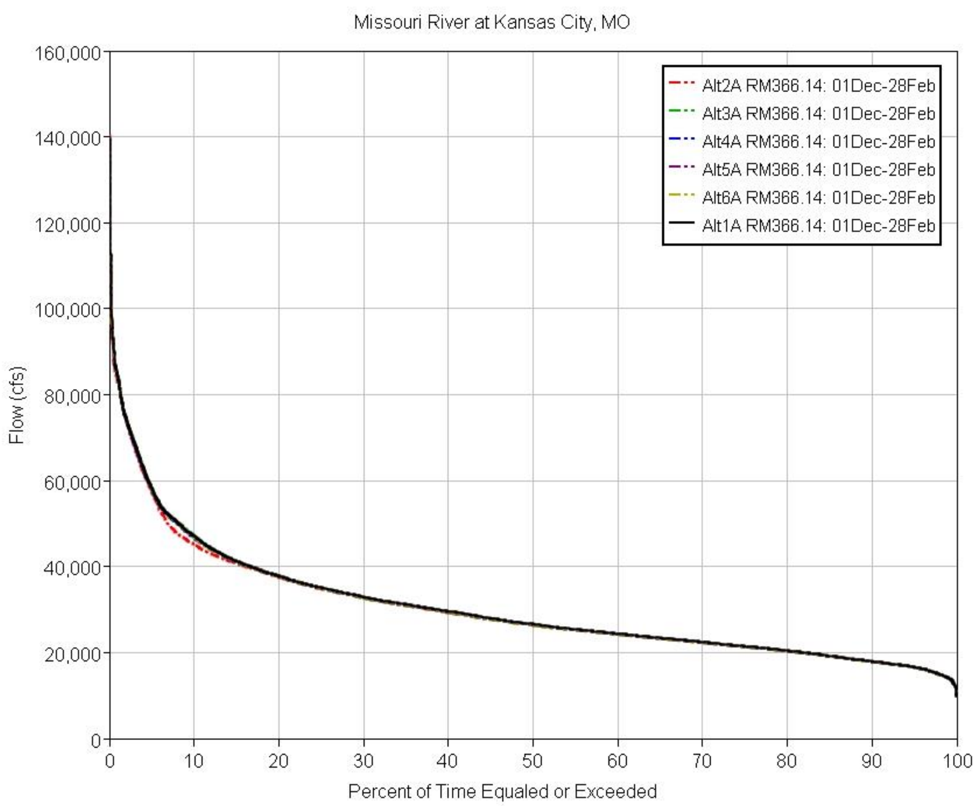


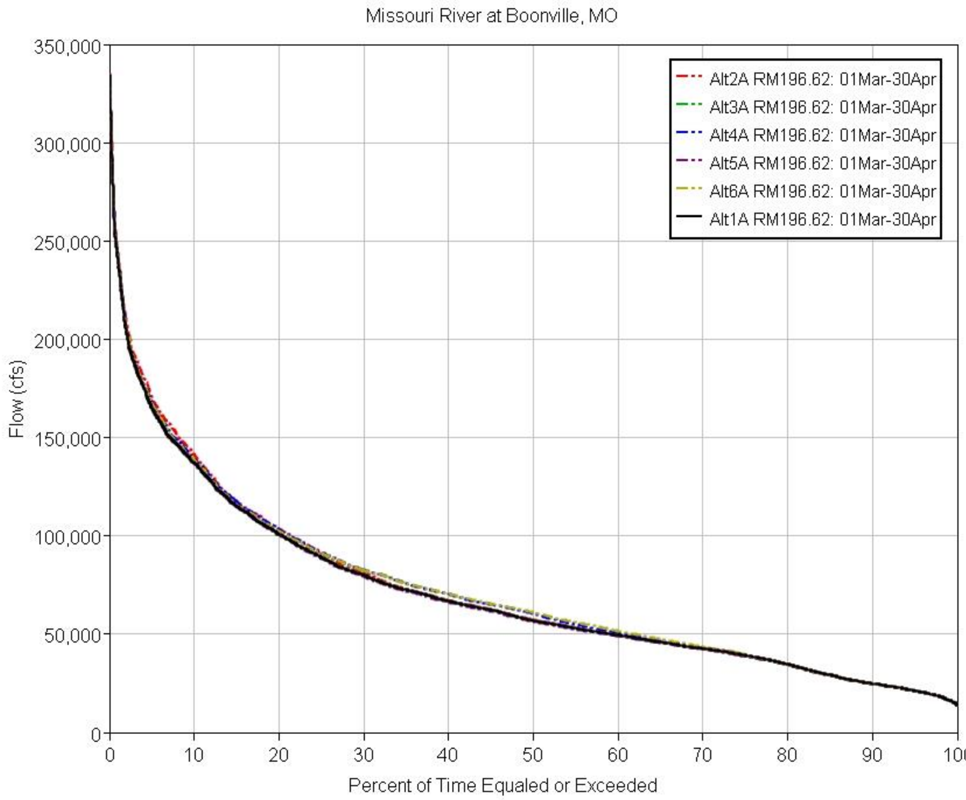
Missouri River at St. Joseph, MO
Alt2A RM448.17: 01Dec-28Feb
Alt3A RM448.17: 01Dec-28Feb
Alt4A RM448.17: 01Dec-28Feb
Alt5A RM448.17: 01Dec-28Feb
Alt6A RM448.17: 01Dec-28Feb
Alt1A RM448.17: 01Dec-28Feb
Flow (cfs)
140,000
120,000
100,000
80,000
60,000
40,000
20,000
0
0
10
20
30
40
50
60
70
80
90
100
Percent of Time Equaled or Exceeded

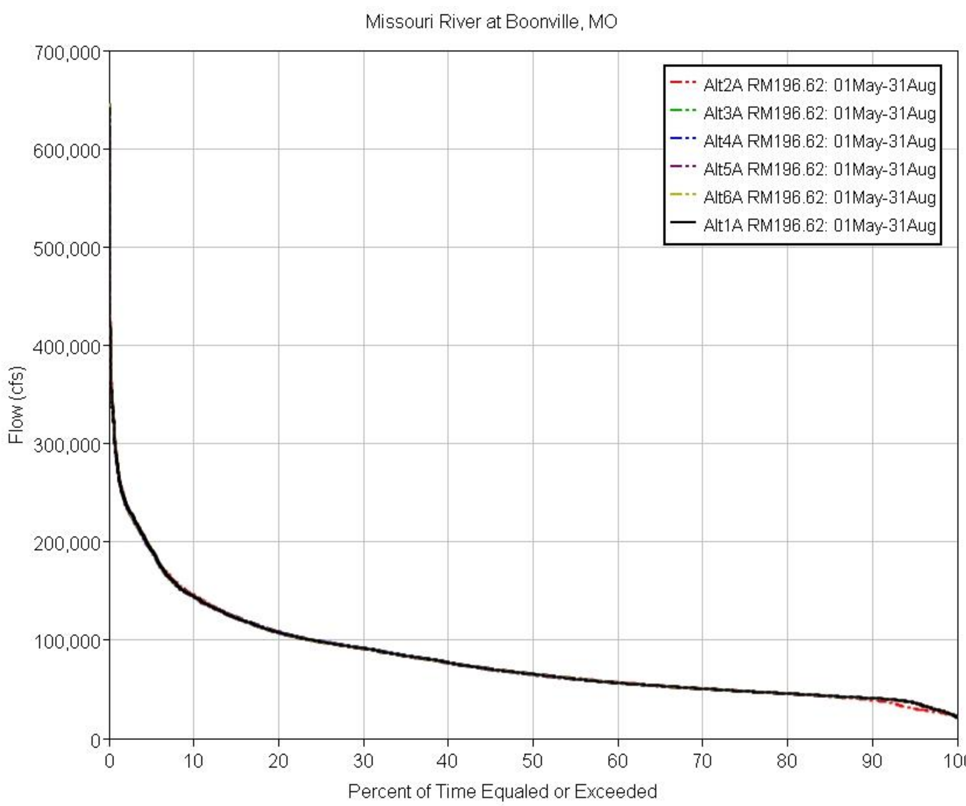

Missouri River at Kansas City, MO
Alt2A RM366.14: 01Mar-30Apr
Alt3A RM366.14: 01Mar-30Apr
Alt4A RM366.14: 01Mar-30Apr
Alt5A RM366.14: 01Mar-30Apr
Alt6A RM366.14: 01Mar-30Apr
Alt1A RM366.14: 01Mar-30Apr
Flow (cfs)
0
50,000
100,000
150,000
200,000
Percent of Time Equaled or Exceeded
0 10 20 30 40 50 60 70 80 90 100

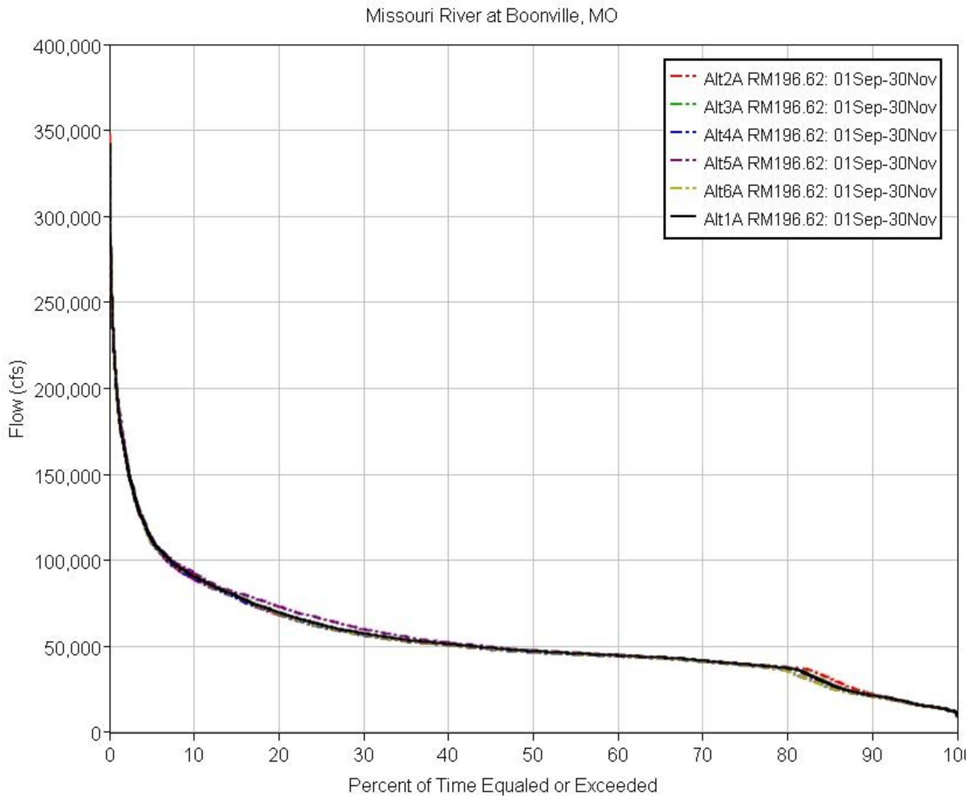

Missouri River at Kansas City, MO
Alt2A RM366.14: 01May-31Aug
Alt3A RM366.14: 01May-31Aug
Alt4A RM366.14: 01May-31Aug
Alt5A RM366.14: 01May-31Aug
Alt6A RM366.14: 01May-31Aug
Alt1A RM366.14: 01May-31Aug
Flow (cfs)
0
50,000
100,000
150,000
200,000
250,000
300,000
350,000
400,000
450,000
500,000
Percent of Time Equaled or Exceeded
0 10 20 30 40 50 60 70 80 90 100

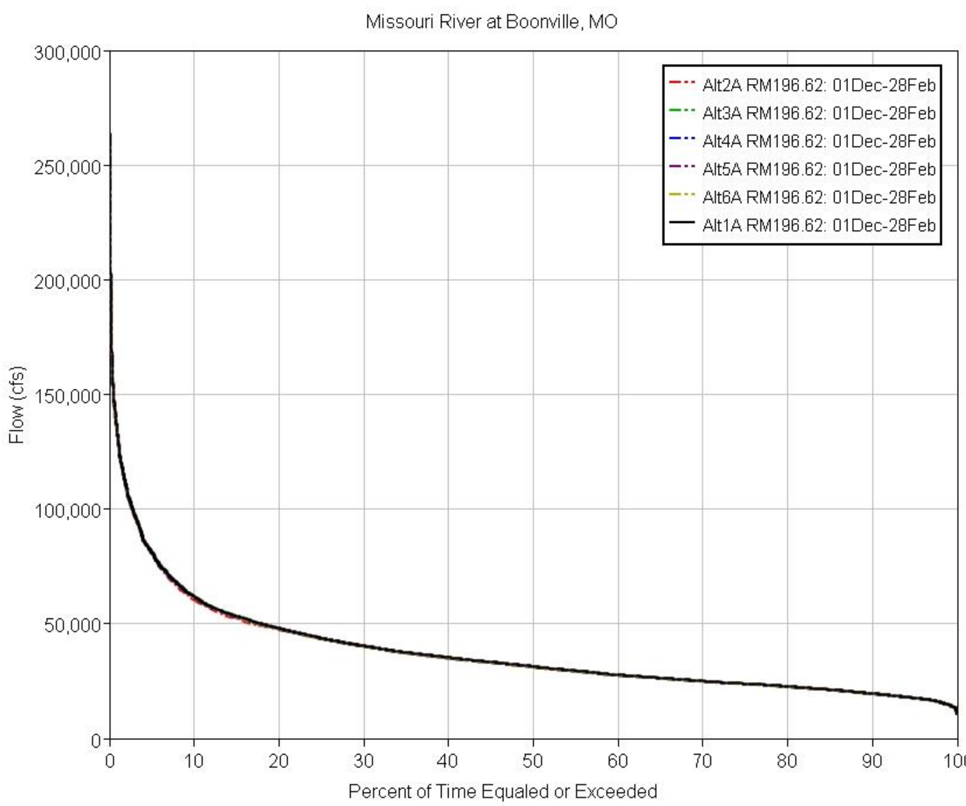

Missouri River at Kansas City, MO
Alt2A RM366.14: 01Sep-30Nov
Alt3A RM366.14: 01Sep-30Nov
Alt4A RM366.14: 01Sep-30Nov
Alt5A RM366.14: 01Sep-30Nov
Alt6A RM366.14: 01Sep-30Nov
Alt1A RM366.14: 01Sep-30Nov
Flow (cfs)
300,000
250,000
200,000
150,000
100,000
50,000
0
0
10
20
30
40
50
60
70
80
90
100
Percent of Time Equaled or Exceeded

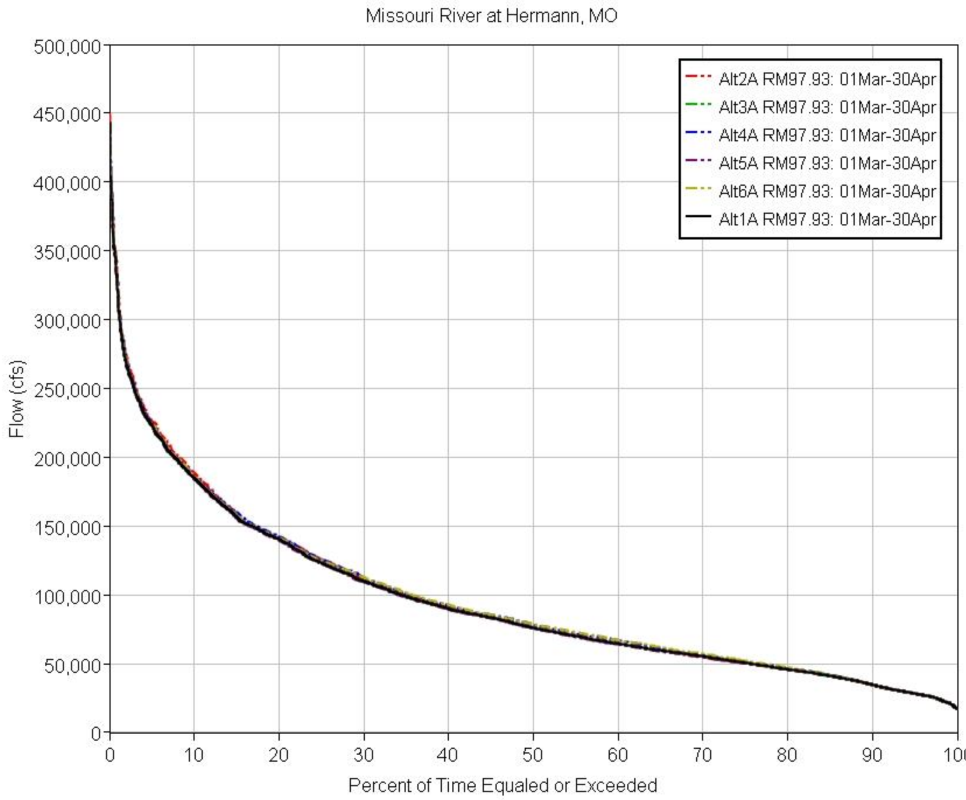

Missouri River at Kansas City, MO
Alt2A RM366.14: 01Dec-28Feb
Alt3A RM366.14: 01Dec-28Feb
Alt4A RM366.14: 01Dec-28Feb
Alt5A RM366.14: 01Dec-28Feb
Alt6A RM366.14: 01Dec-28Feb
Alt1A RM366.14: 01Dec-28Feb
Flow (cfs)
160,000
140,000
120,000
100,000
80,000
60,000
40,000
20,000
0
0
10
20
30
40
50
60
70
80
90
100
Percent of Time Equaled or Exceeded

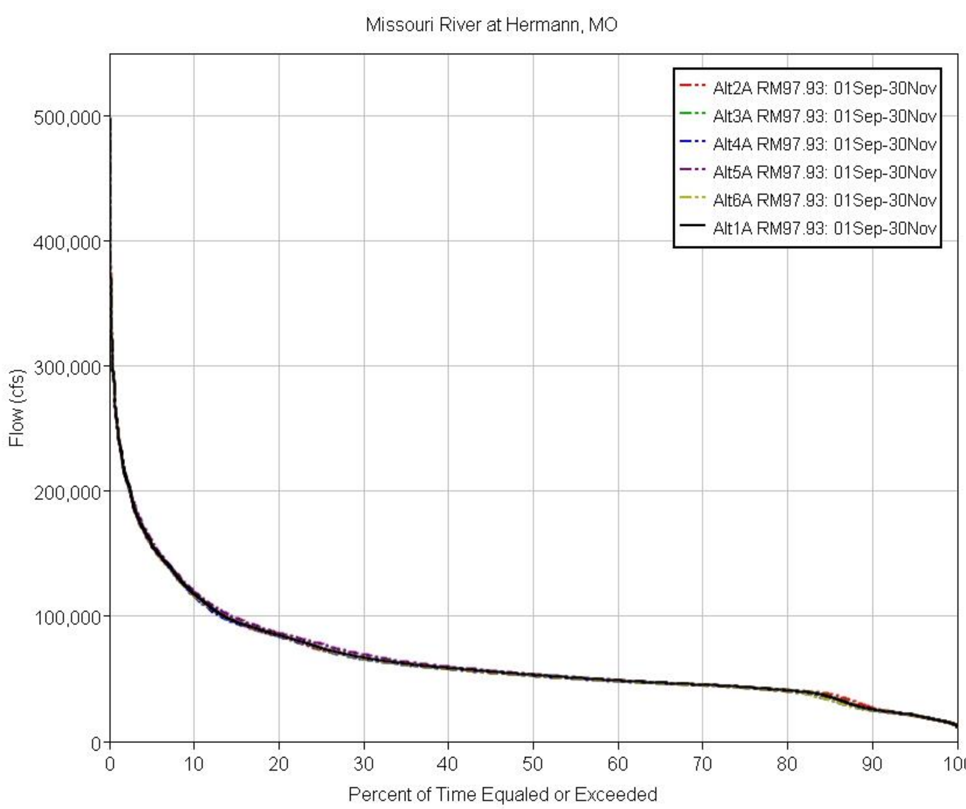

Missouri River at Boonville, MO
Flow (cfs)
350,000
300,000
250,000
200,000
150,000
100,000
50,000
0
0
10
20
30
40
50
60
70
80
90
100
Percent of Time Equaled or Exceeded
Alt2A RM196.62: 01Mar-30Apr
Alt3A RM196.62: 01Mar-30Apr
Alt4A RM196.62: 01Mar-30Apr
Alt5A RM196.62: 01Mar-30Apr
Alt6A RM196.62: 01Mar-30Apr
Alt1A RM196.62: 01Mar-30Apr

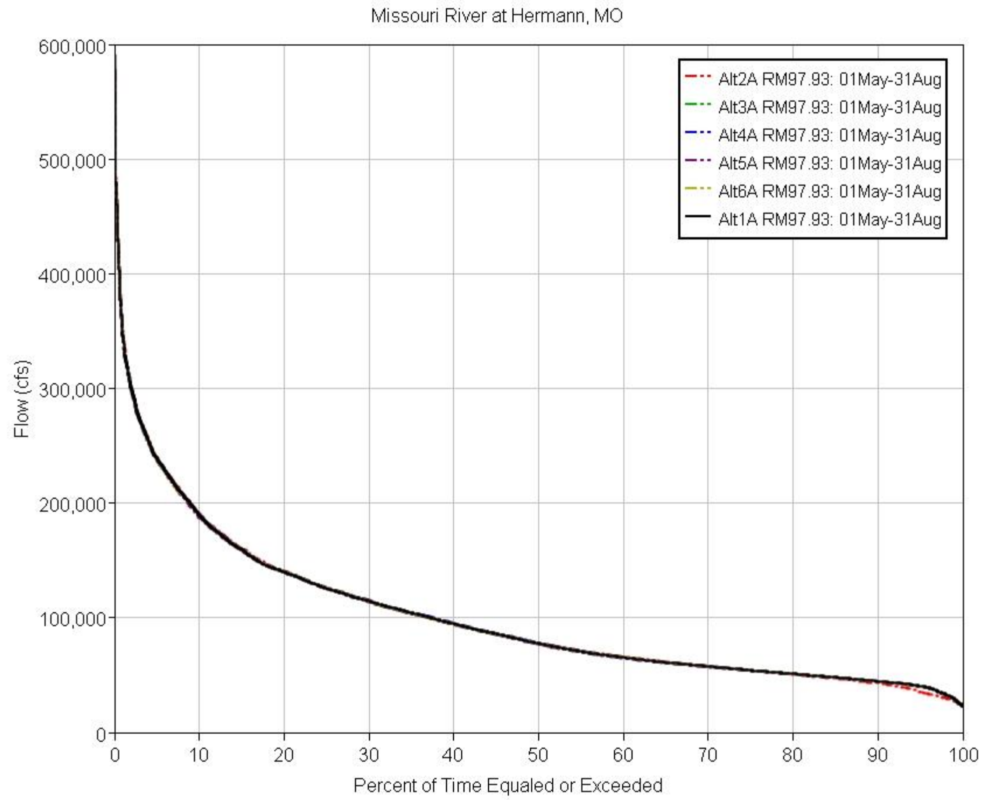

Missouri River at Boonville, MO
Flow (cfs)
700,000
600,000
500,000
400,000
300,000
200,000
100,000
0
0
10
20
30
40
50
60
70
80
90
100
Percent of Time Equaled or Exceeded
Alt2A RM196.62: 01May-31Aug
Alt3A RM196.62: 01May-31Aug
Alt4A RM196.62: 01May-31Aug
Alt5A RM196.62: 01May-31Aug
Alt6A RM196.62: 01May-31Aug
Alt1A RM196.62: 01May-31Aug

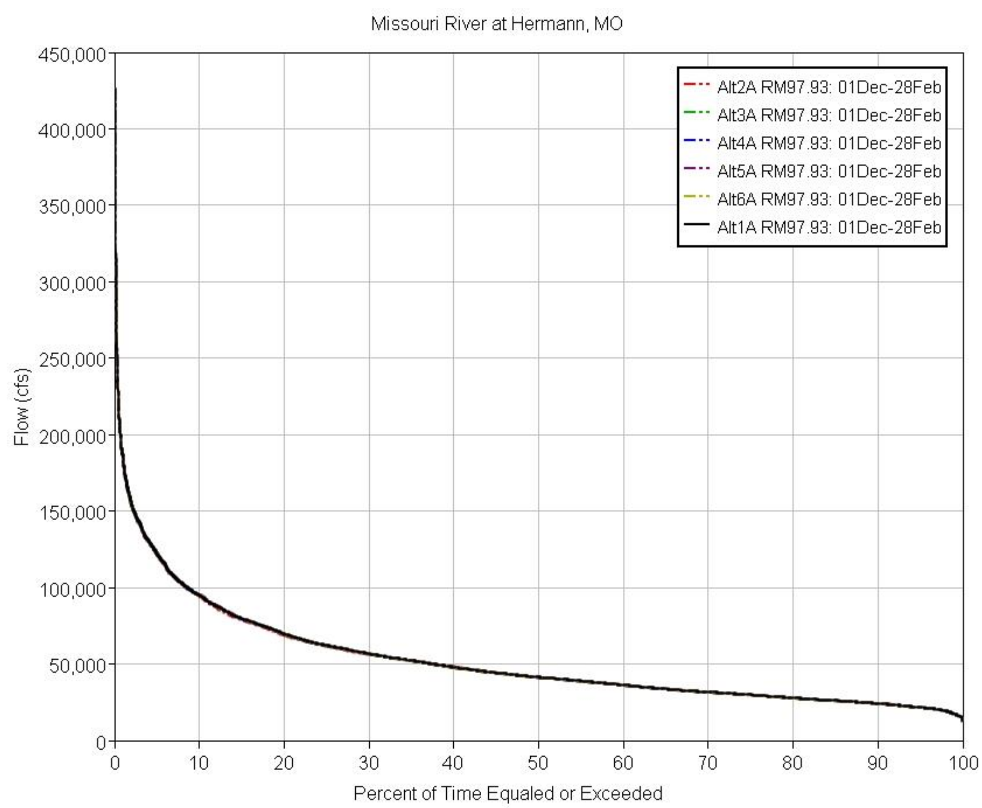

Missouri River at Boonville, MO
400,000
350,000
300,000
250,000
200,000
150,000
100,000
50,000
0
Flow (cfs)
0
10
20
30
40
50
60
70
80
90
100
Percent of Time Equaled or Exceeded
Alt2A RM196.62: 01Sep-30Nov
Alt3A RM196.62: 01Sep-30Nov
Alt4A RM196.62: 01Sep-30Nov
Alt5A RM196.62: 01Sep-30Nov
Alt6A RM196.62: 01Sep-30Nov
Alt1A RM196.62: 01Sep-30Nov

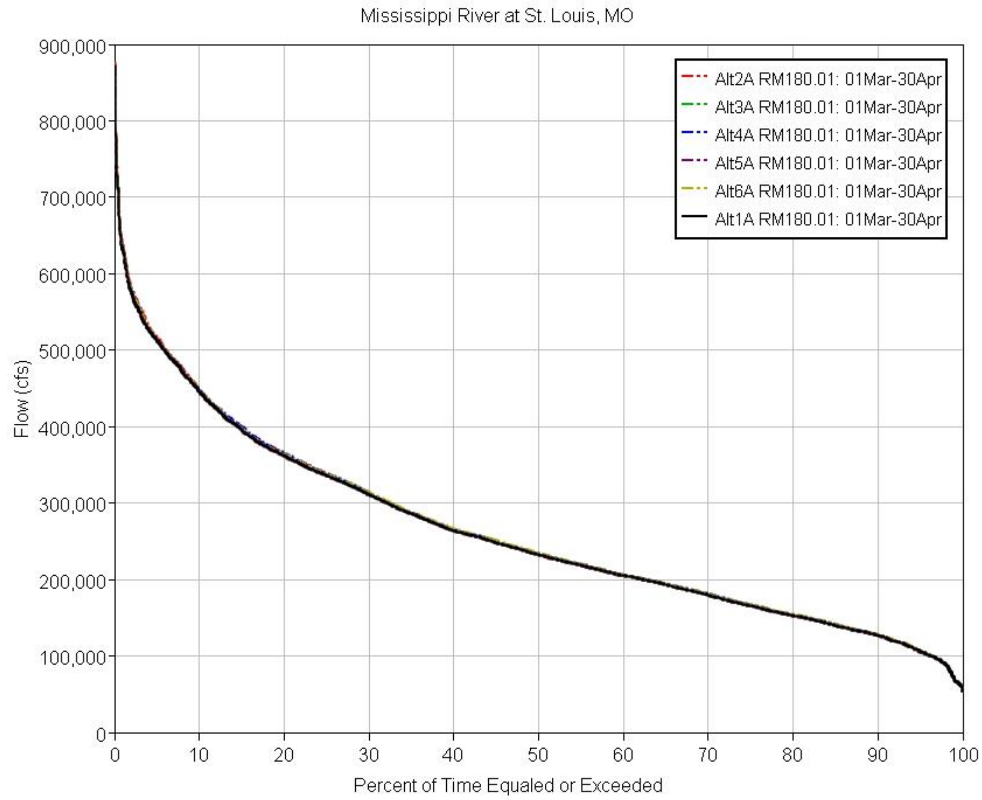

Missouri River at Boonville, MO
300,000
250,000
200,000
150,000
100,000
50,000
0
Flow (cfs)
0
10
20
30
40
50
60
70
80
90
100
Percent of Time Equaled or Exceeded
Alt2A RM196.62: 01Dec-28Feb
Alt3A RM196.62: 01Dec-28Feb
Alt4A RM196.62: 01Dec-28Feb
Alt5A RM196.62: 01Dec-28Feb
Alt6A RM196.62: 01Dec-28Feb
Alt1A RM196.62: 01Dec-28Feb

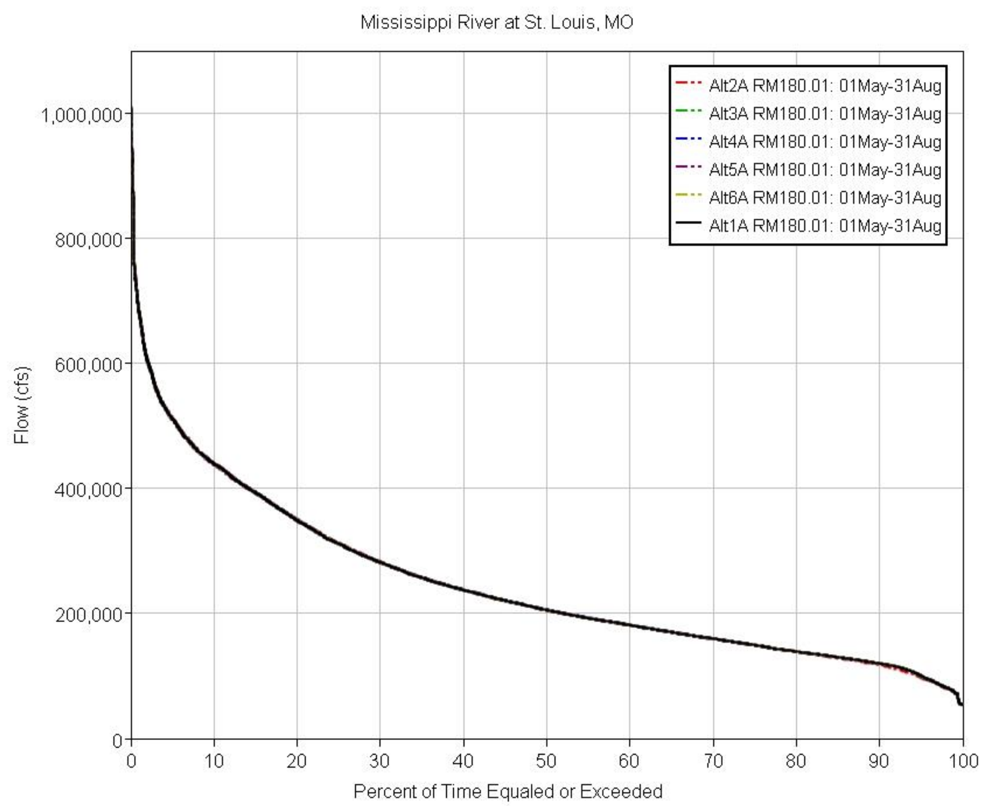

Missouri River at Hermann, MO
Flow (cfs)
Percent of Time Equaled or Exceeded
Alt2A RM97.93: 01Mar-30Apr
Alt3A RM97.93: 01Mar-30Apr
Alt4A RM97.93: 01Mar-30Apr
Alt5A RM97.93: 01Mar-30Apr
Alt6A RM97.93: 01Mar-30Apr
Alt1A RM97.93: 01Mar-30Apr
0
50,000
100,000
150,000
200,000
250,000
300,000
350,000
400,000
450,000
500,000
0
10
20
30
40
50
60
70
80
90
100

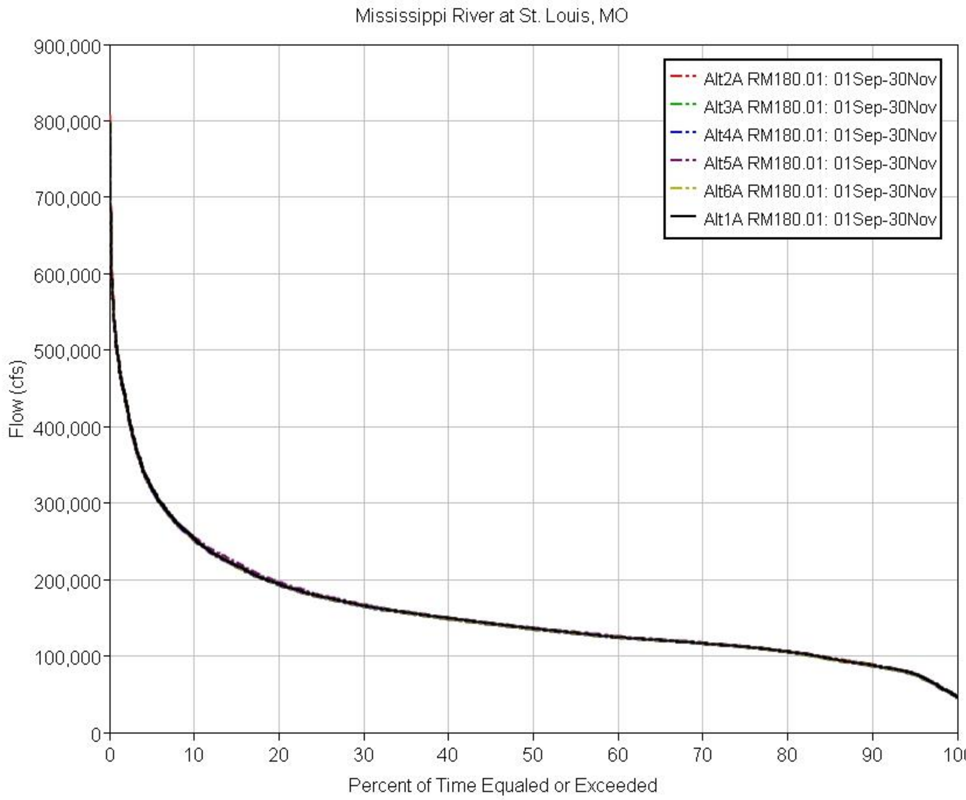

Missouri River at Hermann, MO
Flow (cfs)
Percent of Time Equaled or Exceeded
Alt2A RM97.93: 01Sep-30Nov
Alt3A RM97.93: 01Sep-30Nov
Alt4A RM97.93: 01Sep-30Nov
Alt5A RM97.93: 01Sep-30Nov
Alt6A RM97.93: 01Sep-30Nov
Alt1A RM97.93: 01Sep-30Nov
0
100,000
200,000
300,000
400,000
500,000
0
10
20
30
40
50
60
70
80
90
100

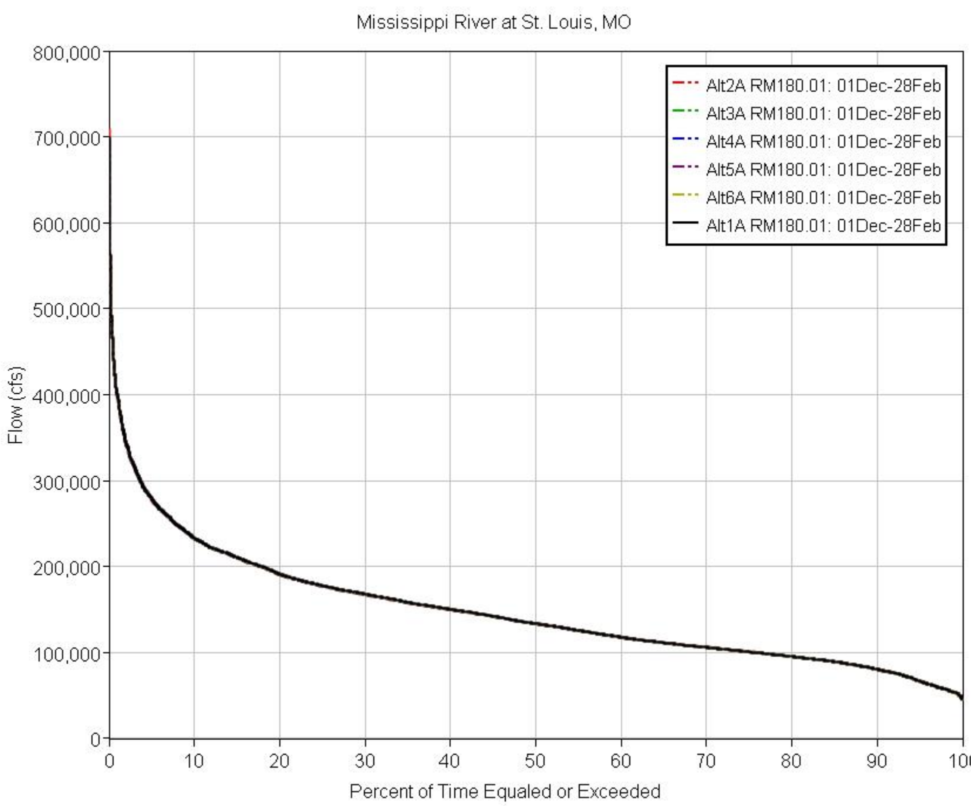

Missouri River at Hermann, MO
Alt2A RM97.93: 01May-31Aug
Alt3A RM97.93: 01May-31Aug
Alt4A RM97.93: 01May-31Aug
Alt5A RM97.93: 01May-31Aug
Alt6A RM97.93: 01May-31Aug
Alt1A RM97.93: 01May-31Aug
Flow (cfs)
600,000
500,000
400,000
300,000
200,000
100,000
0
0
10
20
30
40
50
60
70
80
90
100
Percent of Time Equaled or Exceeded


Missouri River at Hermann, MO
Alt2A RM97.93: 01Dec-28Feb
Alt3A RM97.93: 01Dec-28Feb
Alt4A RM97.93: 01Dec-28Feb
Alt5A RM97.93: 01Dec-28Feb
Alt6A RM97.93: 01Dec-28Feb
Alt1A RM97.93: 01Dec-28Feb
Flow (cfs)
450,000
400,000
350,000
300,000
250,000
200,000
150,000
100,000
50,000
0
0
10
20
30
40
50
60
70
80
90
100
Percent of Time Equaled or Exceeded


Mississippi River at St. Louis, MO
Alt2A RM180.01: 01Mar-30Apr
Alt3A RM180.01: 01Mar-30Apr
Alt4A RM180.01: 01Mar-30Apr
Alt5A RM180.01: 01Mar-30Apr
Alt6A RM180.01: 01Mar-30Apr
Alt1A RM180.01: 01Mar-30Apr
Flow (cfs)
Percent of Time Equaled or Exceeded

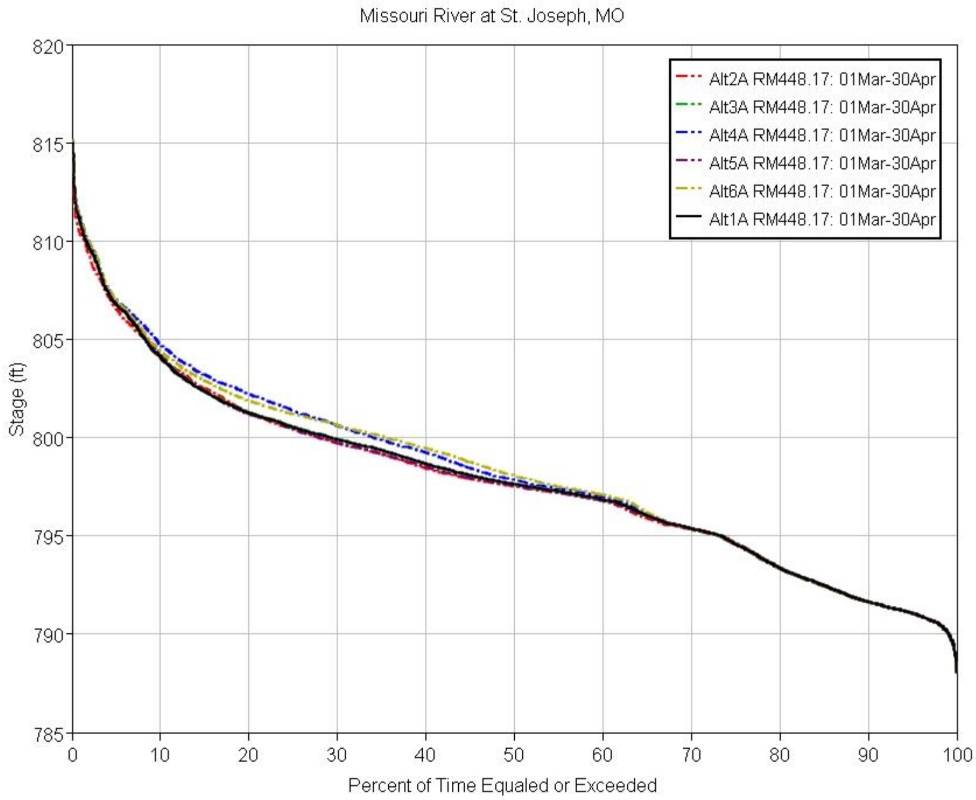

Mississippi River at St. Louis, MO
Alt2A RM180.01: 01May-31Aug
Alt3A RM180.01: 01May-31Aug
Alt4A RM180.01: 01May-31Aug
Alt5A RM180.01: 01May-31Aug
Alt6A RM180.01: 01May-31Aug
Alt1A RM180.01: 01May-31Aug
Flow (cfs)
Percent of Time Equaled or Exceeded

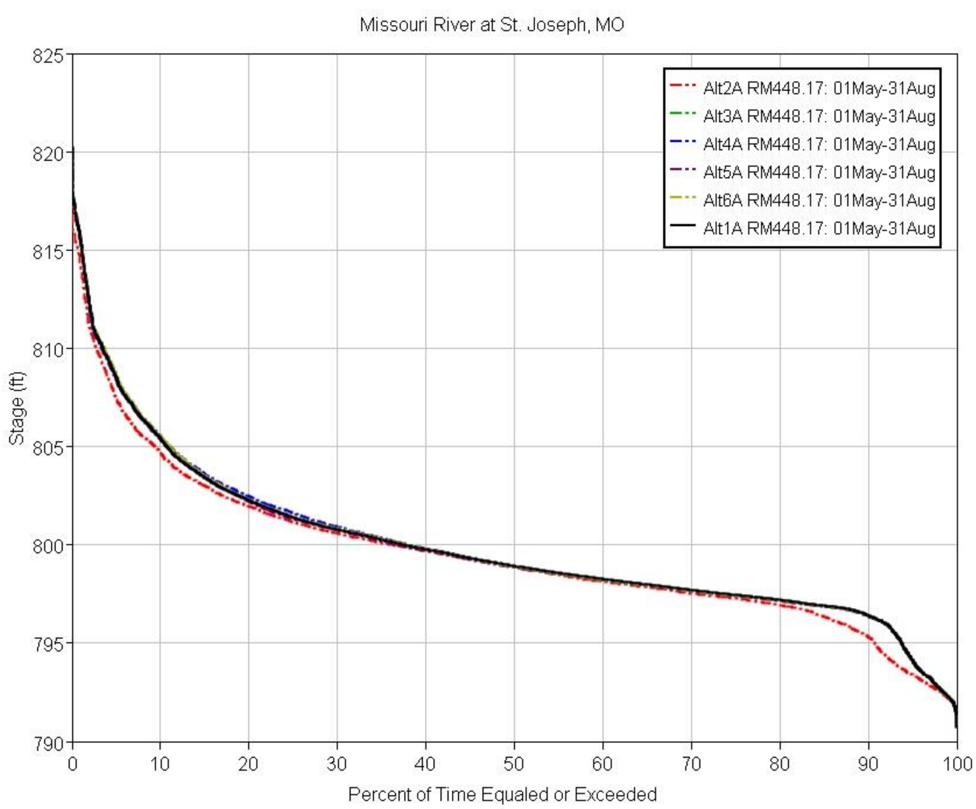

Mississippi River at St. Louis, MO
Flow (cfs)
Percent of Time Equaled or Exceeded
0
100,000
200,000
300,000
400,000
500,000
600,000
700,000
800,000
900,000
0
10
20
30
40
50
60
70
80
90
100
Alt2A RM180.01: 01Sep-30Nov
Alt3A RM180.01: 01Sep-30Nov
Alt4A RM180.01: 01Sep-30Nov
Alt5A RM180.01: 01Sep-30Nov
Alt6A RM180.01: 01Sep-30Nov
Alt1A RM180.01: 01Sep-30Nov

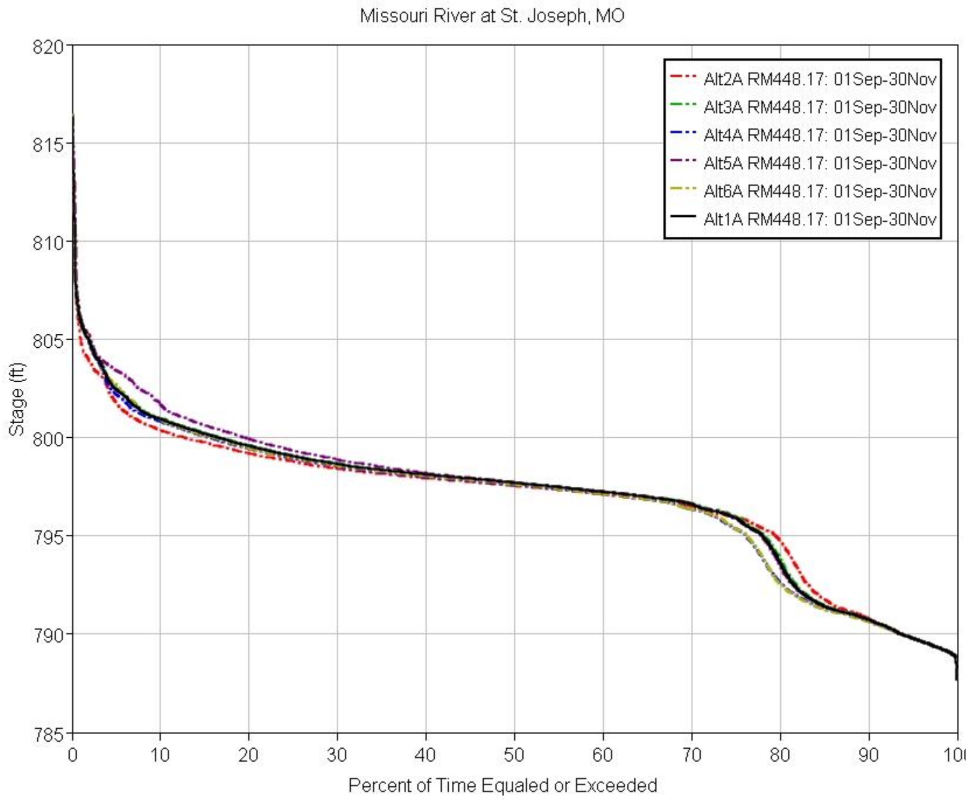

Mississippi River at St. Louis, MO
Flow (cfs)
Percent of Time Equaled or Exceeded
0
100,000
200,000
300,000
400,000
500,000
600,000
700,000
800,000
0
10
20
30
40
50
60
70
80
90
100
Alt2A RM180.01: 01Dec-28Feb
Alt3A RM180.01: 01Dec-28Feb
Alt4A RM180.01: 01Dec-28Feb
Alt5A RM180.01: 01Dec-28Feb
Alt6A RM180.01: 01Dec-28Feb
Alt1A RM180.01: 01Dec-28Feb

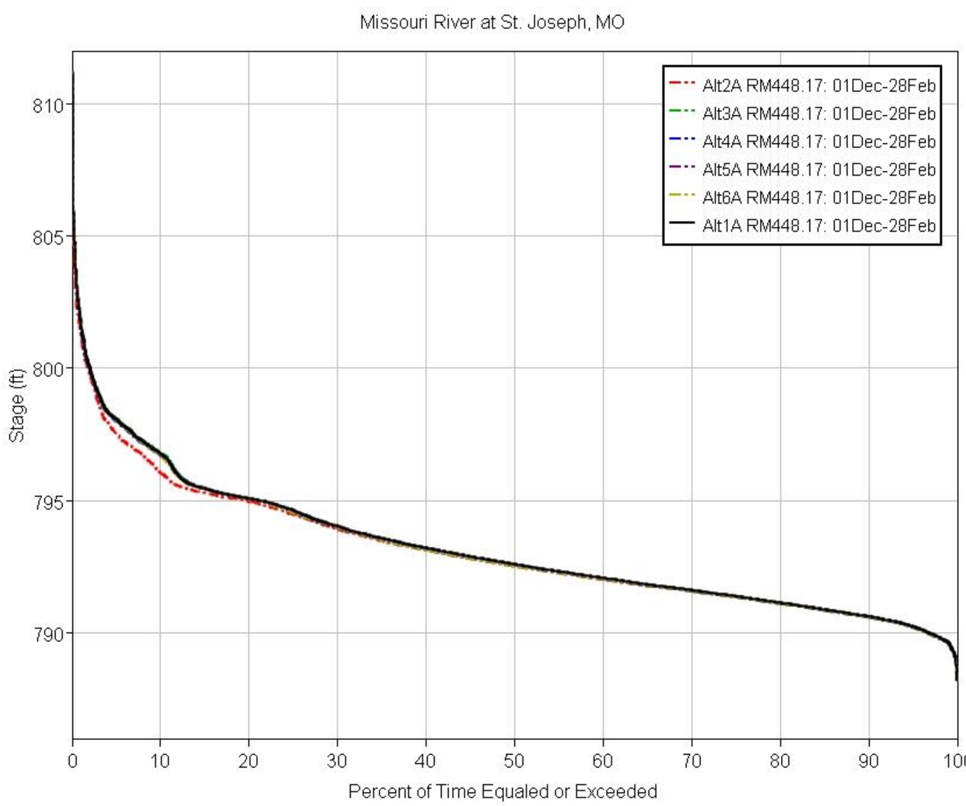
Appendix E

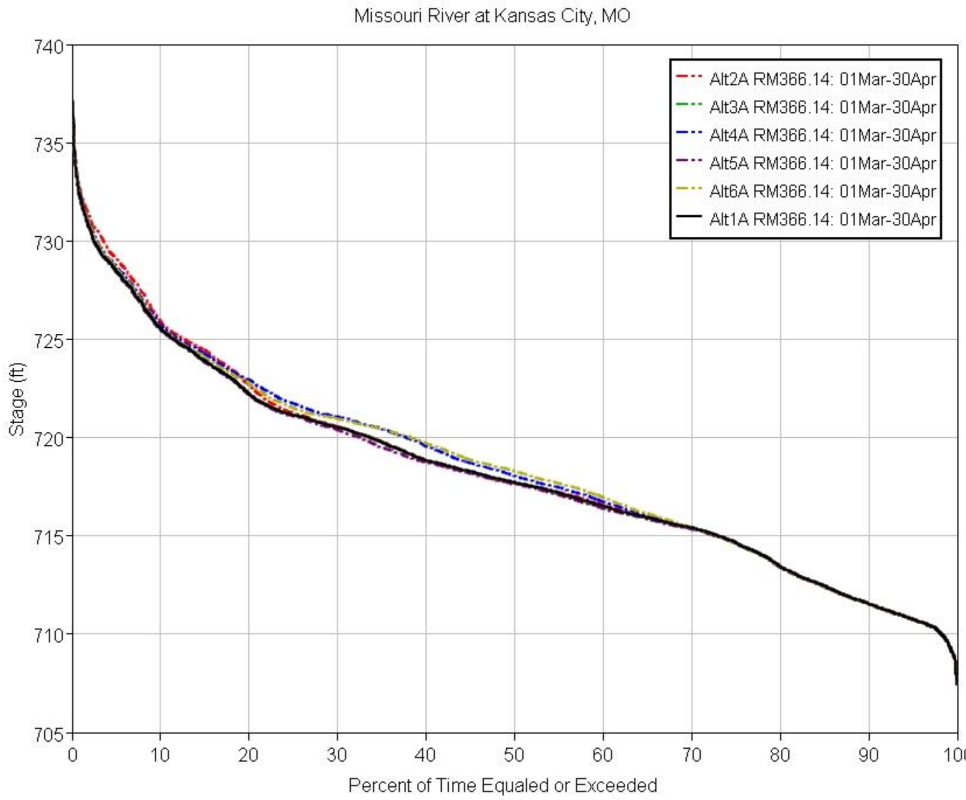
Attachment 5 – Rulo to Mouth Alternative Stage Duration Plots

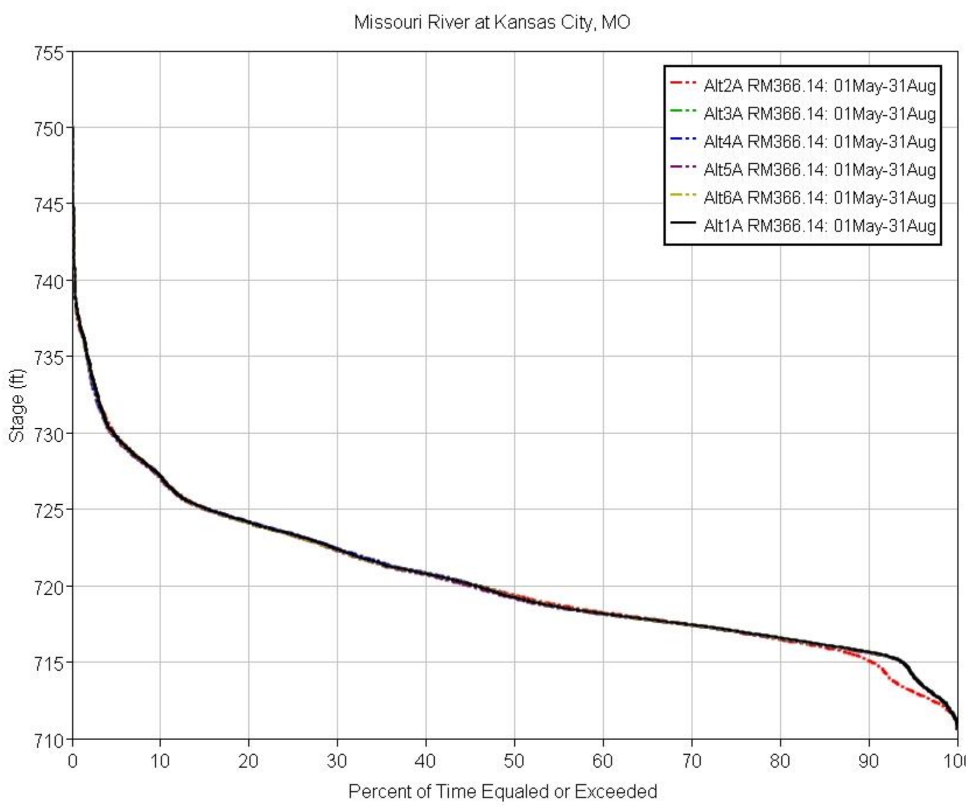

Missouri River at St. Joseph, MO
Stage (ft)
Percent of Time Equaled or Exceeded
Alt2A RM448.17: 01Mar-30Apr
Alt3A RM448.17: 01Mar-30Apr
Alt4A RM448.17: 01Mar-30Apr
Alt5A RM448.17: 01Mar-30Apr
Alt6A RM448.17: 01Mar-30Apr
Alt1A RM448.17: 01Mar-30Apr

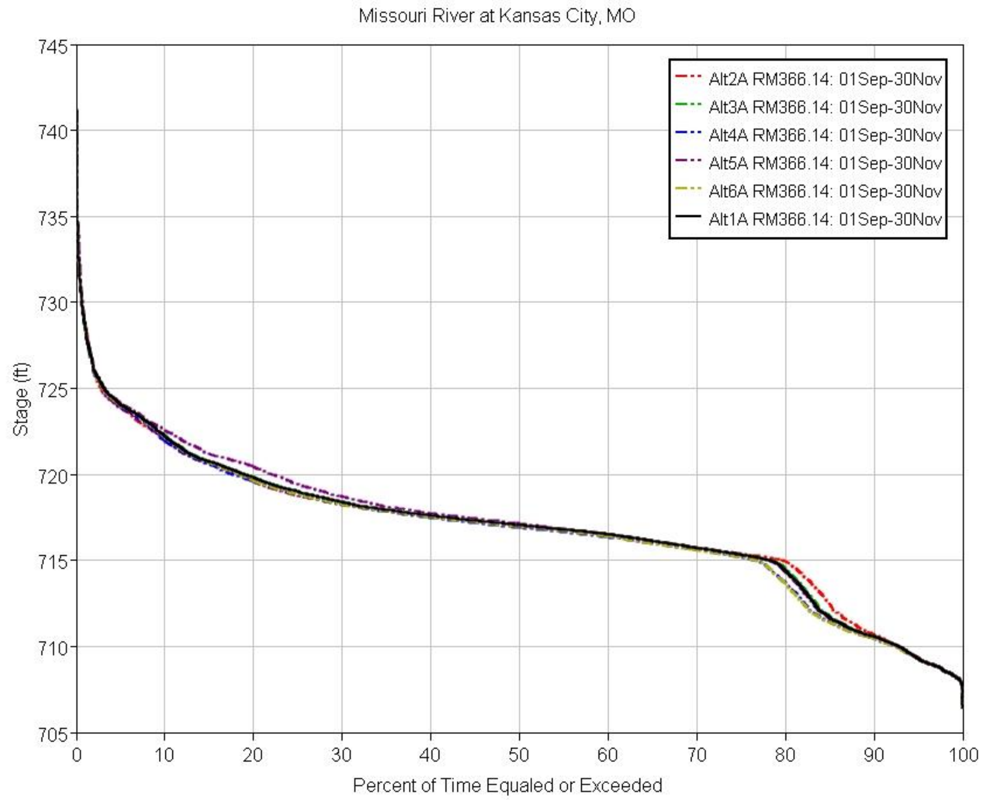

Missouri River at St. Joseph, MO
Stage (ft)
Percent of Time Equaled or Exceeded
Alt2A RM448.17: 01May-31Aug
Alt3A RM448.17: 01May-31Aug
Alt4A RM448.17: 01May-31Aug
Alt5A RM448.17: 01May-31Aug
Alt6A RM448.17: 01May-31Aug
Alt1A RM448.17: 01May-31Aug


Missouri River at St. Joseph, MO
Stage (ft)
Percent of Time Equaled or Exceeded
Alt2A RM448.17: 01Sep-30Nov
Alt3A RM448.17: 01Sep-30Nov
Alt4A RM448.17: 01Sep-30Nov
Alt5A RM448.17: 01Sep-30Nov
Alt6A RM448.17: 01Sep-30Nov
Alt1A RM448.17: 01Sep-30Nov


Missouri River at St. Joseph, MO
Stage (ft)
Percent of Time Equaled or Exceeded
Alt2A RM448.17: 01Dec-28Feb
Alt3A RM448.17: 01Dec-28Feb
Alt4A RM448.17: 01Dec-28Feb
Alt5A RM448.17: 01Dec-28Feb
Alt6A RM448.17: 01Dec-28Feb
Alt1A RM448.17: 01Dec-28Feb


Missouri River at Kansas City, MO
Stage (ft)
Percent of Time Equaled or Exceeded
Alt2A RM366.14: 01Mar-30Apr
Alt3A RM366.14: 01Mar-30Apr
Alt4A RM366.14: 01Mar-30Apr
Alt5A RM366.14: 01Mar-30Apr
Alt6A RM366.14: 01Mar-30Apr
Alt1A RM366.14: 01Mar-30Apr


Missouri River at Kansas City, MO
Stage (ft)
Percent of Time Equaled or Exceeded
Alt2A RM366.14: 01May-31Aug
Alt3A RM366.14: 01May-31Aug
Alt4A RM366.14: 01May-31Aug
Alt5A RM366.14: 01May-31Aug
Alt6A RM366.14: 01May-31Aug
Alt1A RM366.14: 01May-31Aug


Missouri River at Kansas City, MO
Stage (ft)
Percent of Time Equaled or Exceeded
745
740
735
730
725
720
715
710
705
0
10
20
30
40
50
60
70
80
90
100
Alt2A RM366.14: 01Sep-30Nov
Alt3A RM366.14: 01Sep-30Nov
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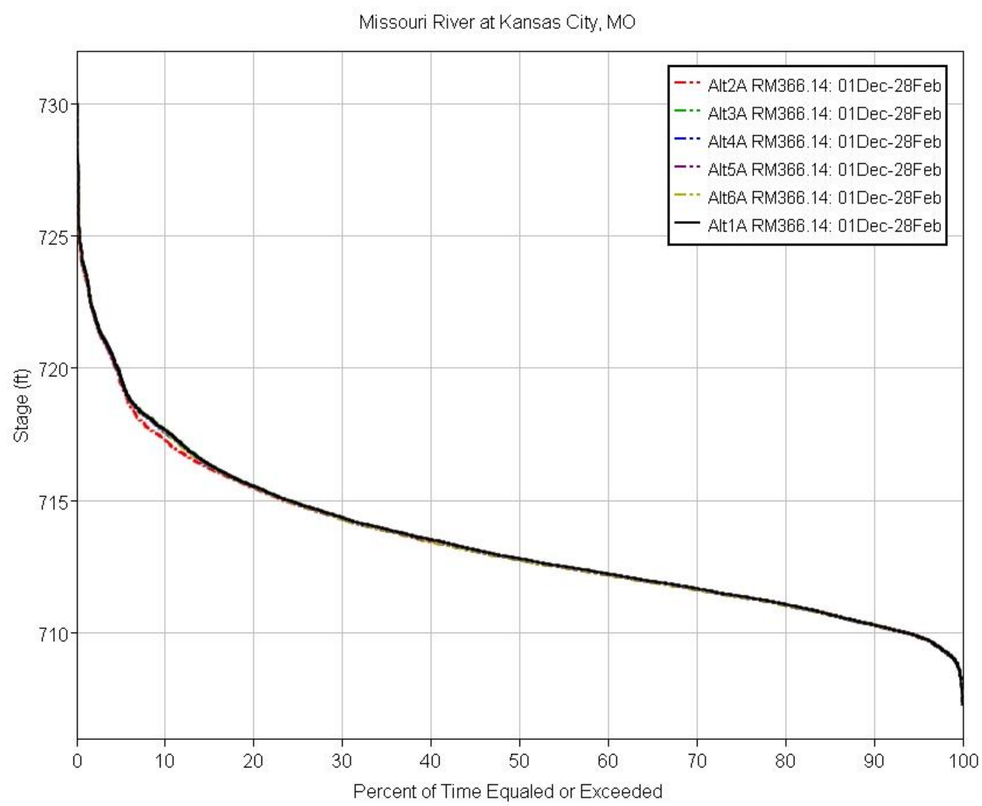
Alt5A RM366.14: 01Sep-30Nov
Alt6A RM366.14: 01Sep-30Nov
Alt1A RM366.14: 01Sep-30Nov


Missouri River at Kansas City, MO
Stage (ft)
Percent of Time Equaled or Exceeded
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710
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10
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70
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90
100
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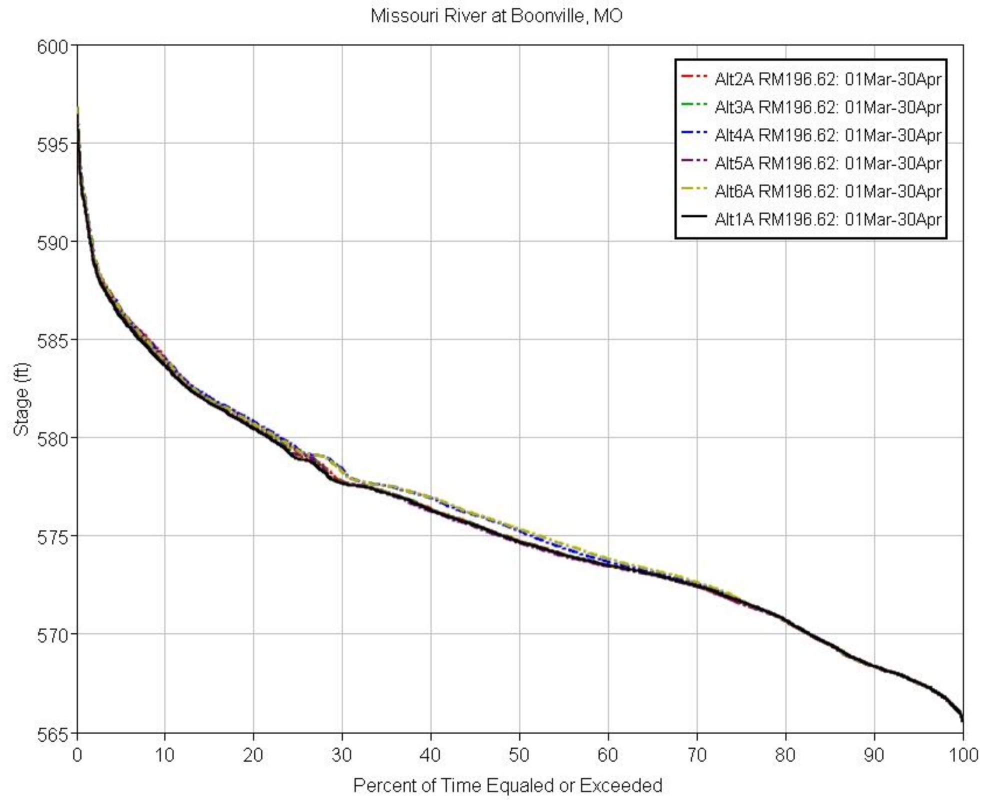
Missouri River at Boonville, MO
Stage (ft)
Percent of Time Equaled or Exceeded
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Alt5A RM196.62: 01Mar-30Apr
Alt6A RM196.62: 01Mar-30Apr
Alt1A RM196.62: 01Mar-30Apr
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585
580
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570
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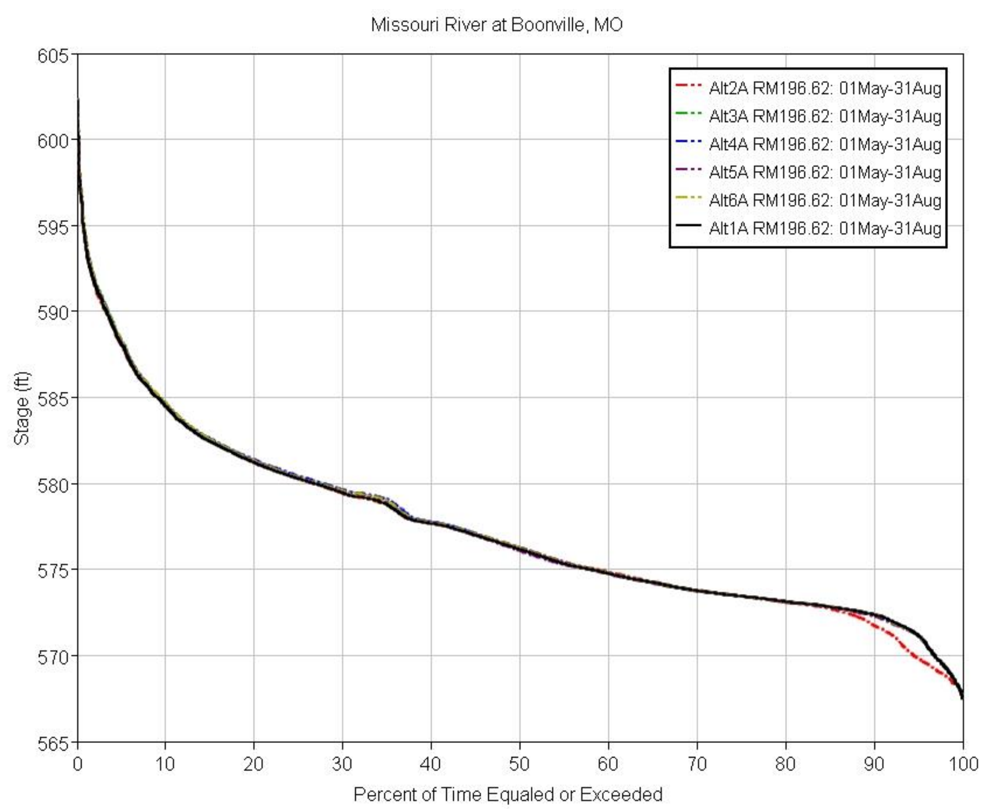
Missouri River at Boonville, MO
Stage (ft)
Percent of Time Equaled or Exceeded
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Alt5A RM196.62: 01May-31Aug
Alt6A RM196.62: 01May-31Aug
Alt1A RM196.62: 01May-31Aug
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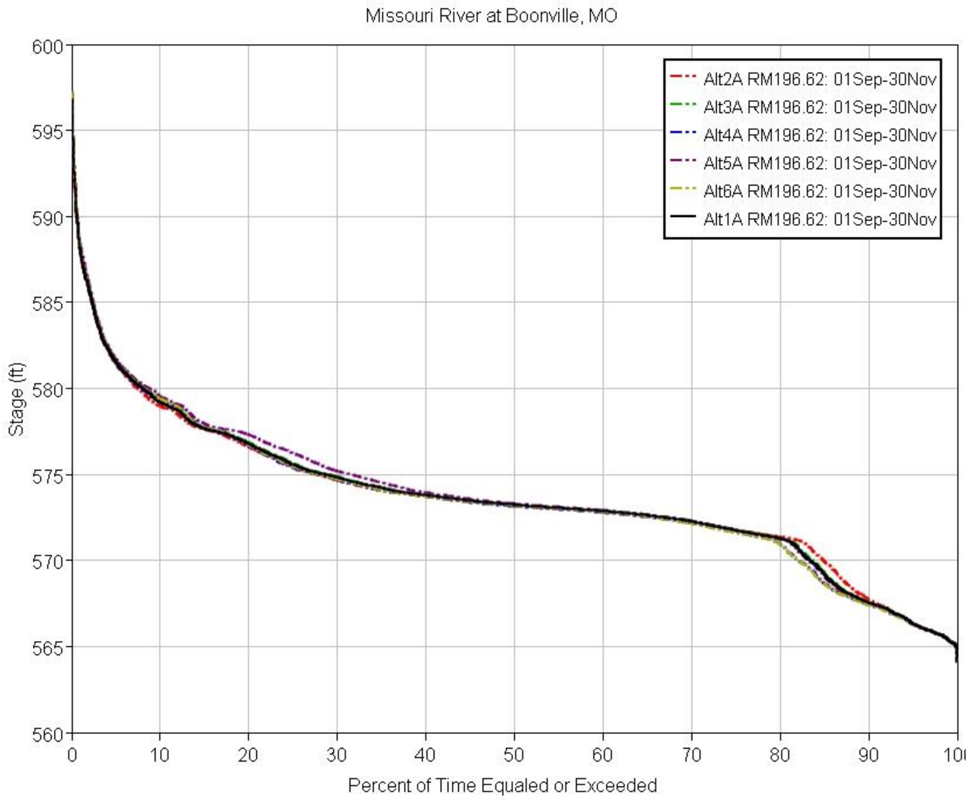
Missouri River at Boonville, MO
Stage (ft)
Percent of Time Equaled or Exceeded
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Alt2A RM196.62: 01Sep-30Nov
Alt3A RM196.62: 01Sep-30Nov
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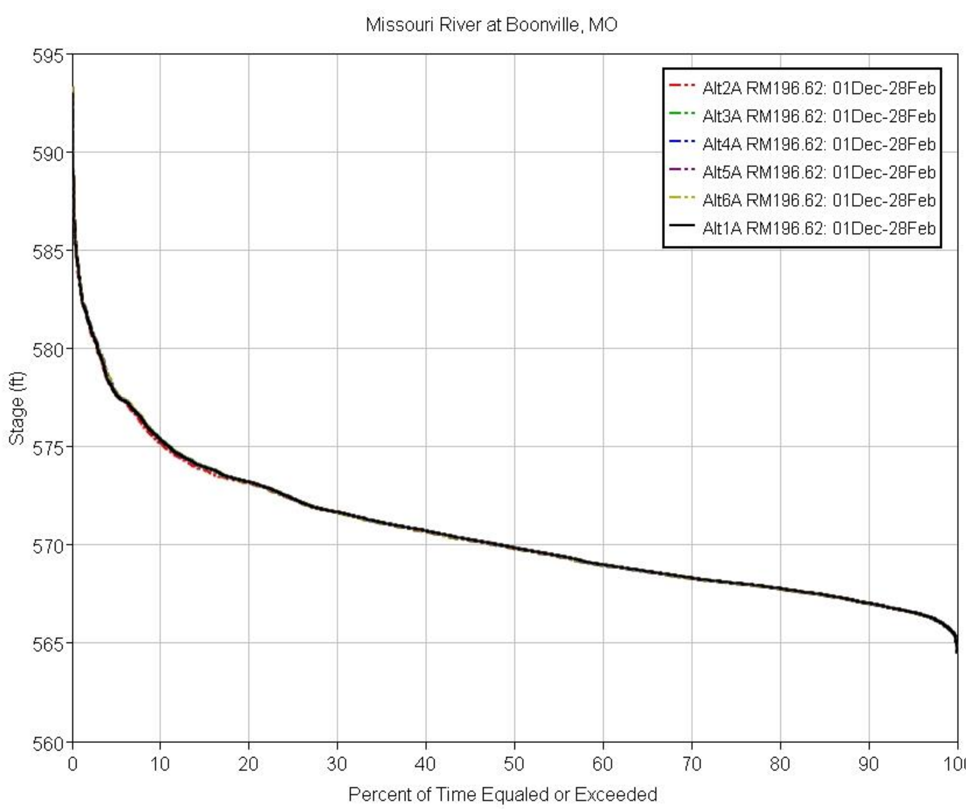
Missouri River at Boonville, MO
Stage (ft)
Percent of Time Equaled or Exceeded
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Alt2A RM196.62: 01Dec-28Feb
Alt3A RM196.62: 01Dec-28Feb
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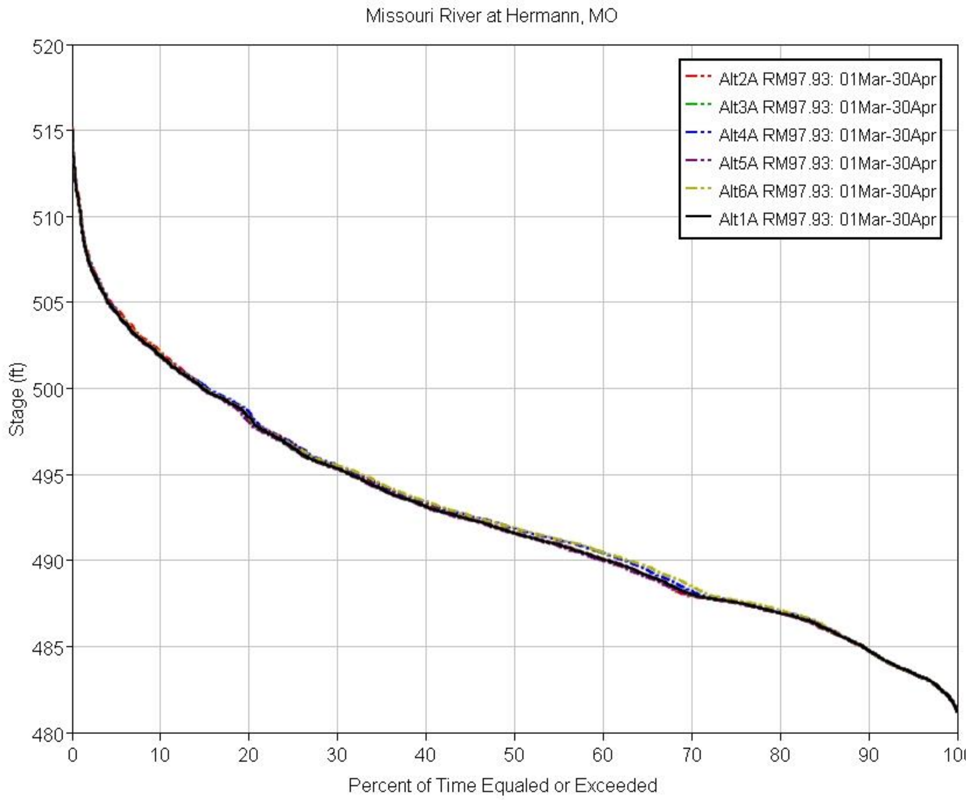
Missouri River at Hermann, MO
Stage (ft)
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505
500
495
490
485
480
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20
30
40
50
60
70
80
90
100
Percent of Time Equaled or Exceeded
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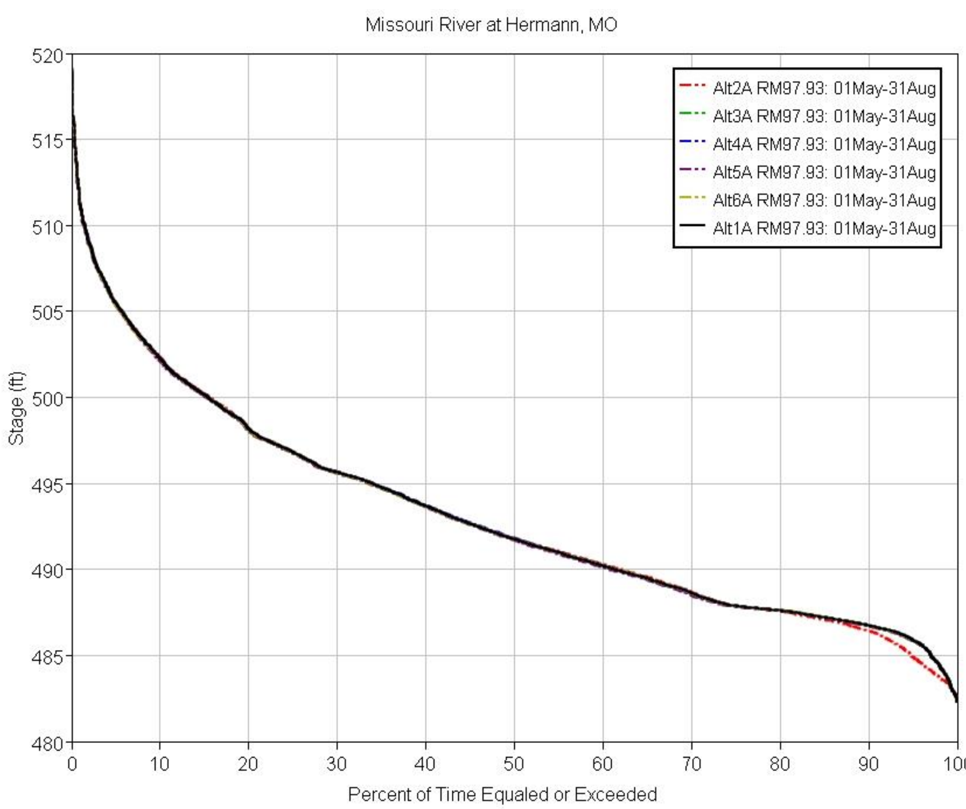
Missouri River at Hermann, MO
Stage (ft)
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Percent of Time Equaled or Exceeded
Alt2A RM97.93: 01May-31Aug
Alt3A RM97.93: 01May-31Aug
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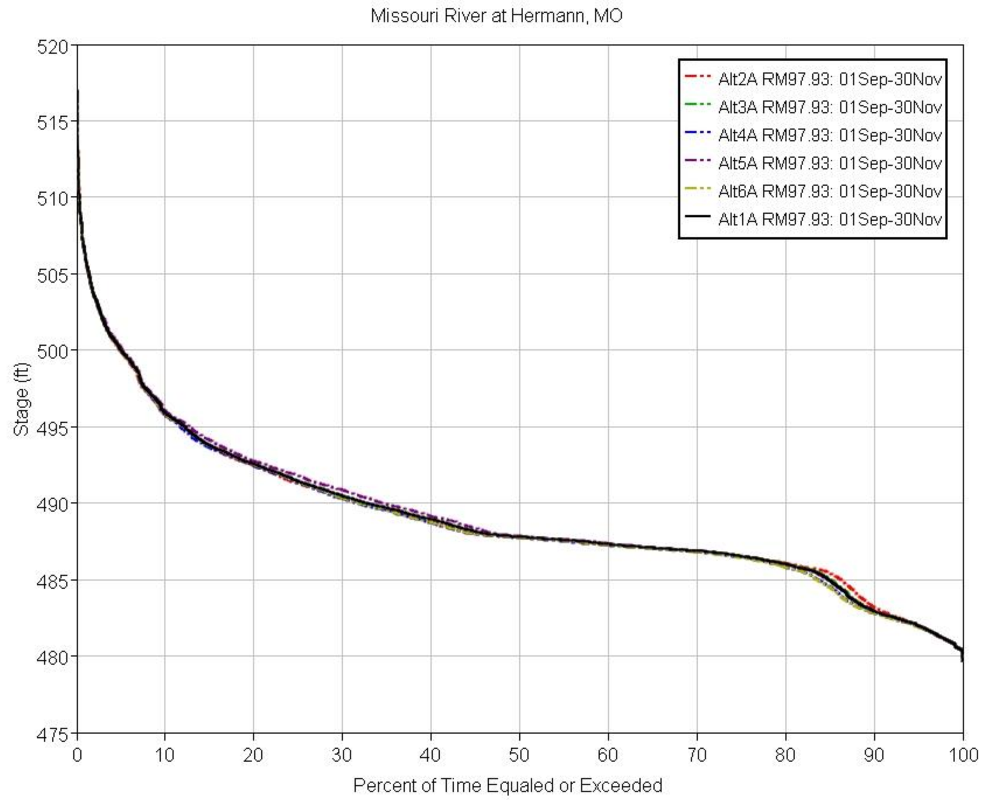

Missouri River at Hermann, MO
Stage (ft)
Percent of Time Equaled or Exceeded
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Alt5A RM97.93: 01Sep-30Nov
Alt6A RM97.93: 01Sep-30Nov
Alt1A RM97.93: 01Sep-30Nov
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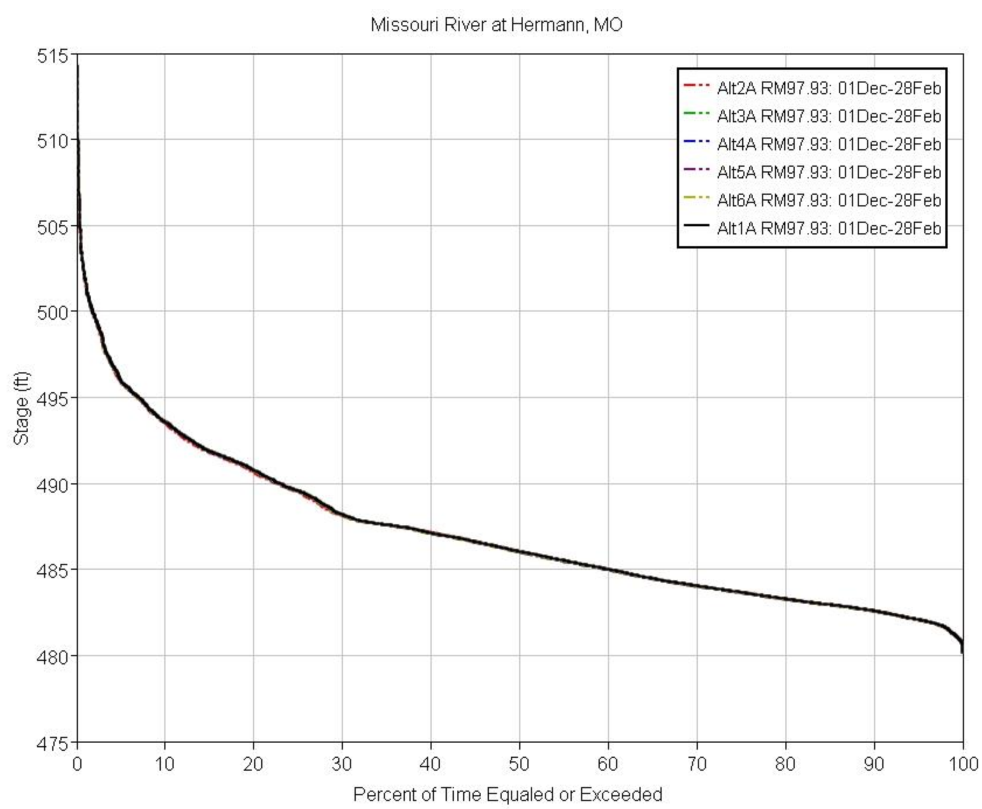

Missouri River at Hermann, MO
Stage (ft)
Percent of Time Equaled or Exceeded
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Alt5A RM97.93: 01Dec-28Feb
Alt6A RM97.93: 01Dec-28Feb
Alt1A RM97.93: 01Dec-28Feb
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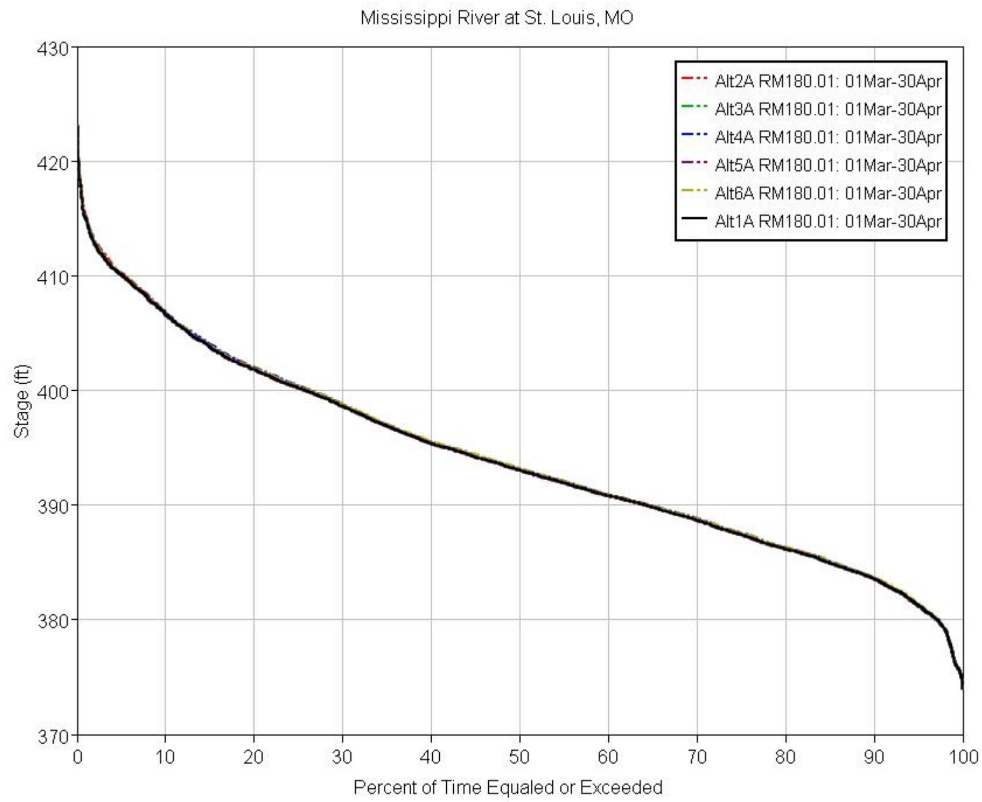

Mississippi River at St. Louis, MO
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Alt3A RM180.01: 01Mar-30Apr
Alt4A RM180.01: 01Mar-30Apr
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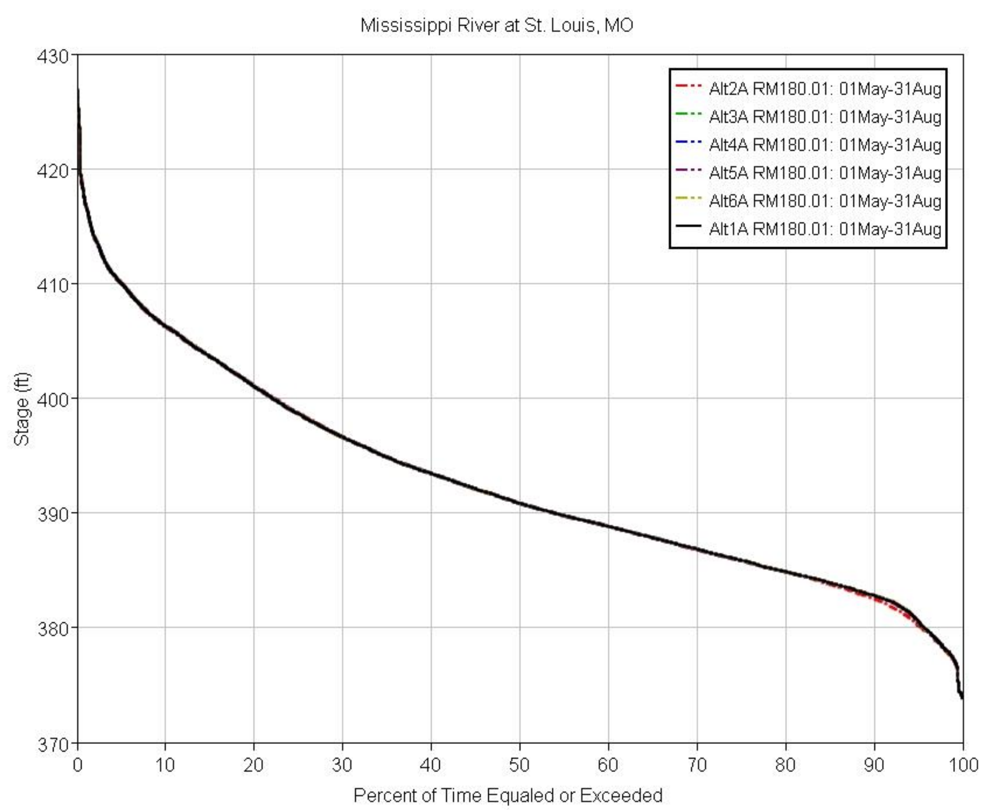
Stage (ft)
Percent of Time Equaled or Exceeded



Mississippi River at St. Louis, MO
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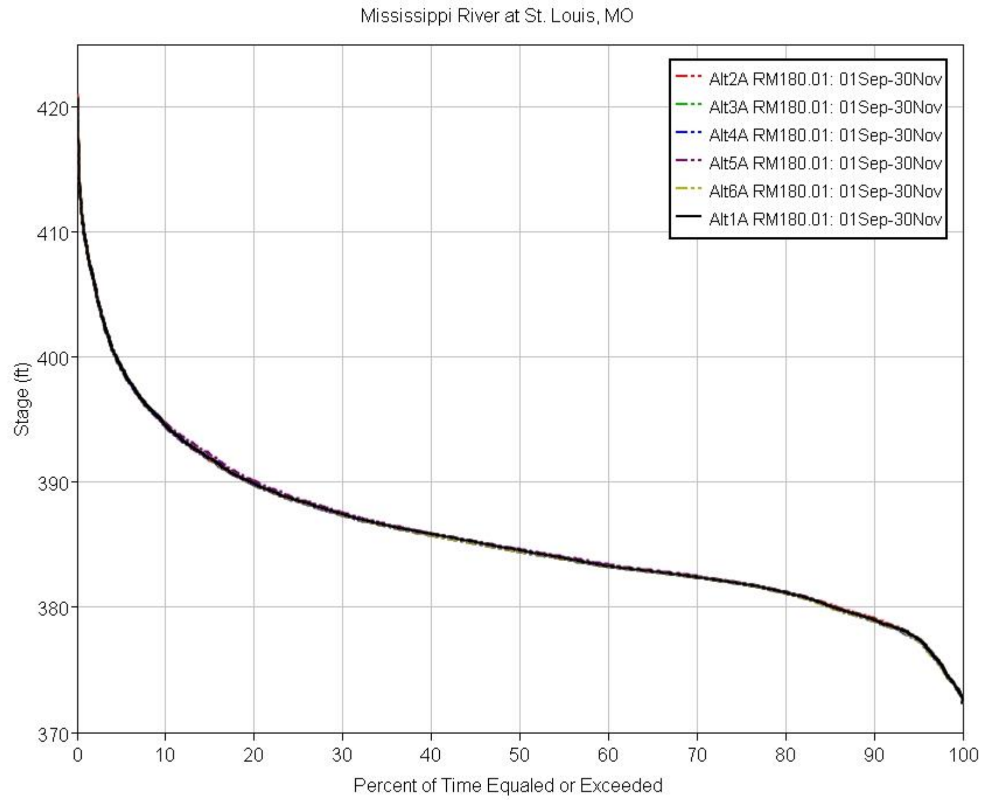
Stage (ft)
Percent of Time Equaled or Exceeded


Mississippi River at St. Louis, MO
Stage (ft)
Percent of Time Equaled or Exceeded
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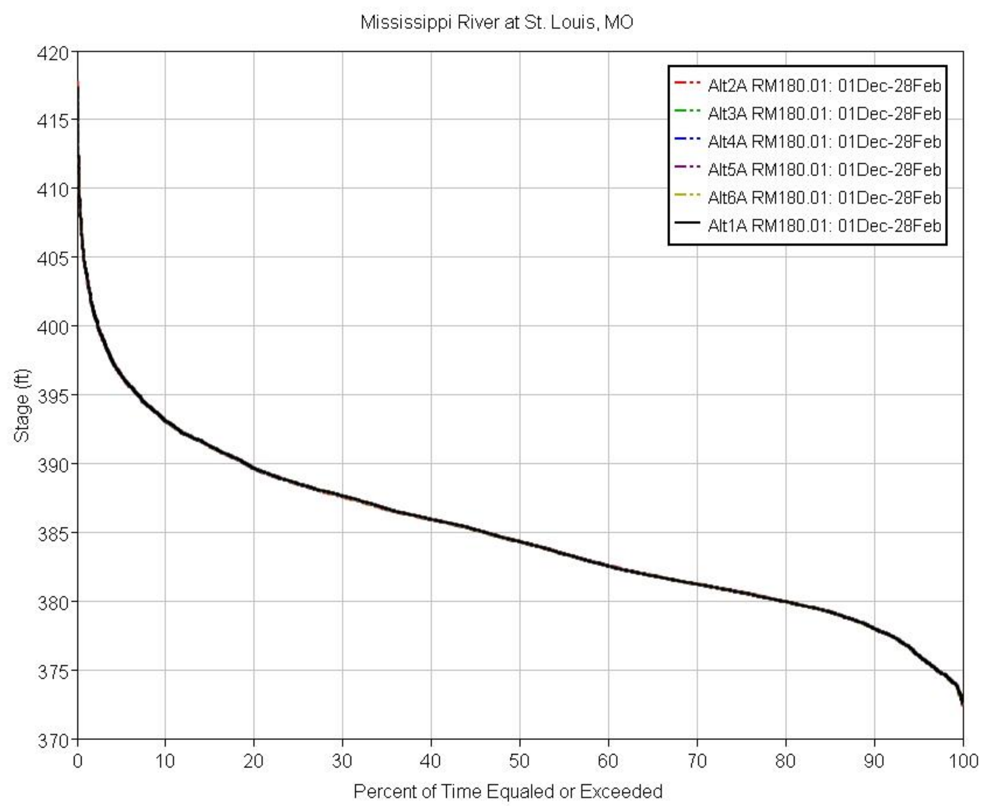
Mississippi River at St. Louis, MO
Stage (ft)
Percent of Time Equaled or Exceeded
Alt2A RM180.01: 01Dec-28Feb
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Alt4A RM180.01: 01Dec-28Feb
Alt5A RM180.01: 01Dec-28Feb
Alt6A RM180.01: 01Dec-28Feb
Alt1A RM180.01: 01Dec-28Feb

Attachment 6

Interior Drainage Hydrologic Analysis

Missouri River Recovery Program Management Plan

United States Army Corps of Engineers

Kansas City District

FINAL – July 2018

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Acronyms

DEM	Digital Elevation Model
ET	Evapotranspiration
HSG	Hydrologic Soil Group
Ksat	Saturated Hydraulic Conductivity
LIDAR	Light Detection and Ranging
MRLS	Missouri River Levee System
NAIP	National Agricultural Imagery Program
NED	National Elevation Database
POR	Period of Record
USGS	United States Geological Survey
WMS	Watershed Modeling System

1 INTRODUCTION

Interior drainage analyses for four Missouri River levee systems, two in Omaha District and two in Kansas City District, were conducted as part of the Missouri River Recovery Program's Management Plan. The selected sites represent a sub sample of locations modeled in 1998 during the Master Manual review and update (REF). The interior drainage analysis involved simulating period of record (POR) inflows using a rainfall runoff model to provide flow inputs to an unsteady hydraulic analysis to simulate inundation in leveed areas. Hourly rainfall-runoff hydrologic simulations between 1948-2012 was conducted for the interior drainage at the Missouri River Levee System (MRLS) L488 and the MRLS L246 levee systems. This work was conducted in general accordance with USACE Engineering Manual (EM) 1110-2-1413 "Hydrologic Analysis of Interior Areas" and EM 1110-2-1417 "Flood-Runoff Analysis." The purpose of this report is to document the supporting engineering analyses of the interior drainage hydrologic modeling for the two levee systems in the Kansas City District. Close coordination of the modeling was conducted with Omaha District to ensure reasonably consistent methodology, including both districts utilizing model calibration of a historic stream gage on Mill Creek to inform the model development for the interior drainage sites (USACE, 2016).

2 MRLS L488 INTERIOR DRAINAGE

The MRLS L488 levee, described as L488, interior drainage analysis has been developed to provide POR runoff estimates that are produced by the contributing drainage basin of the levee system. The leveed area generally surrounds Forbes, Missouri and protects agricultural lands along the Missouri River floodplain. The contributing drainage area consists of about 15 square miles of flood plain and about 13 square miles of area above the bluff line. Figure 1 shows a general layout of the interior drainage area.

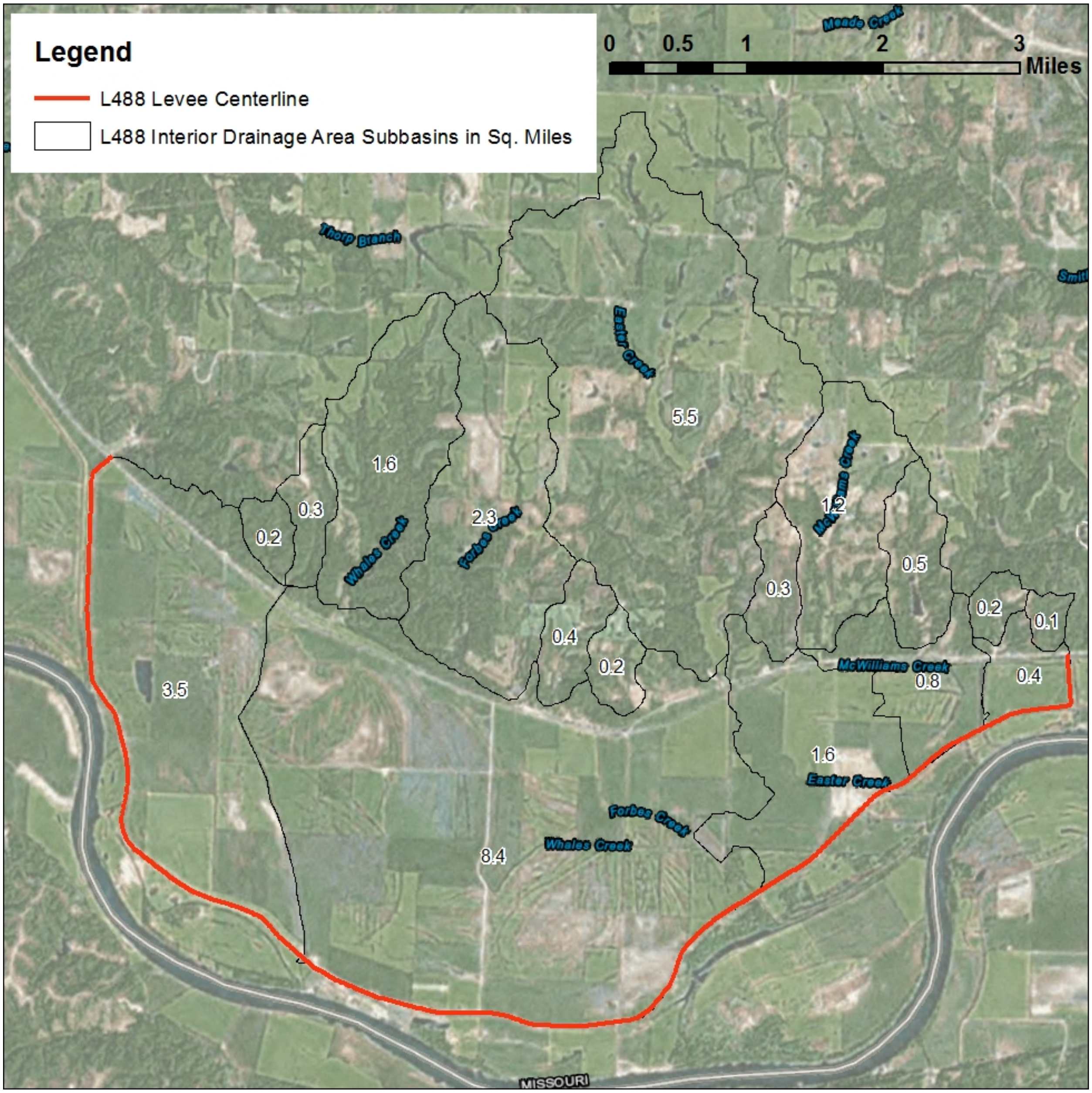

Figure 1: L488 Interior Drainage Area and Subbasin Boundaries

2.1 HYDROLOGIC MODEL DEVELOPMENT

A hydrologic analysis was developed with parameters shown in Table 1. The basin model development involved estimating terrain, soil, and land use properties as well as calibrating the basin to meet the objectives of the study.

Table 1: L488 Hydrologic Model Parameters

Model Parameter	**Adopted Method**
Hydrologic Modeling Software	HEC-HMS v 4.2
Basin delineation and physical terrain analysis	A 10-meter terrain grid obtained from the USGS National Elevation Database (NED) was processed with Aquaveo WMS software with refinements based on 1-meter LIDAR and recent NAIP aerial photography
Precipitation Data	Hourly point precipitation data at several gages
Loss Methods	Deficit and Constant
Routing Methods	Muskingum-Cunge
Transform methods	Snyder Unit Hydrograph
Canopy Methods	none
Calibration approach	Adapted from the Mill Creek pilot study basin parameters, input flows into the hydraulic analysis and compared results to modeled interior ponding elevations
HEC-HMS working File Location	K:\MissionProjects\sec\ed-h\MoRiver_Models\500_Rivers\592_Alternatives_HC\Interior Drainage\L488\HMS

Basin delineations and river centerlines were first established from a 10-meter digital elevation model (DEM) and refined in the floodplain with a higher definition 1-meter DEM that is based on light detection and ranging (LIDAR) data collected in 2012. These refinements were needed because many drainage boundaries and canals within the floodplain were not adequately represented in the 10-meter DEM. In many floodplain locations with a relatively small amount of elevation relief, the relatively coarse terrain data in the 10-meter LIDAR did not adequately represent the hydraulic parameters needed to route discharge to each outlet. Historical National Agriculture Imagery Program (NAIP) aerial photography was also used to verify the delineations and river centerlines. The basin model layout is shown in Figure 2.

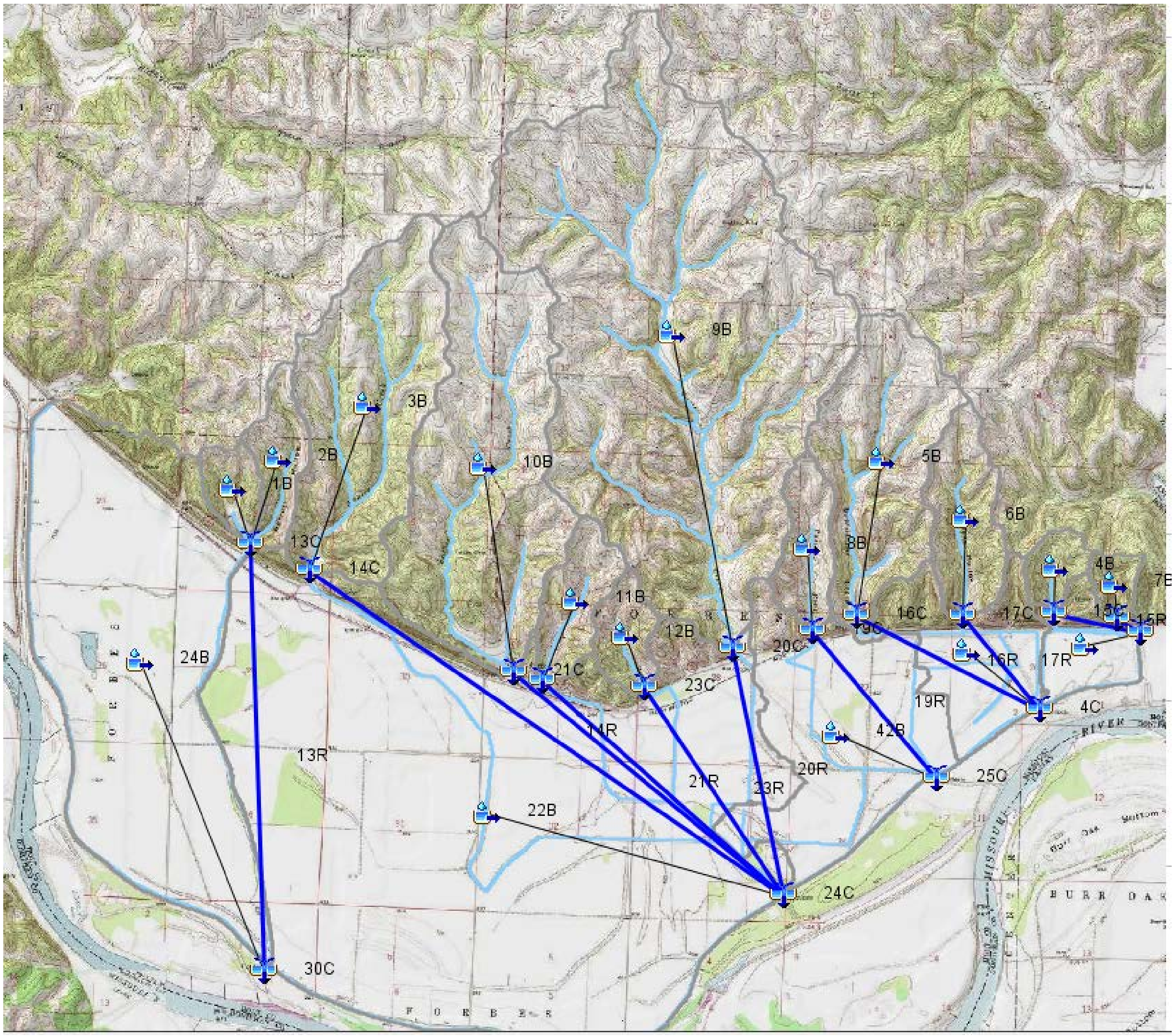

Figure 2: L488 Basin Schematic

2.2 BASIN MODEL COMPONENTS

The basin model consisted of estimating several parameters that were required to route flow to each outlet structure in the levee. Five outlet structures are shown in Table 2 that have contributing subbasins summarized in Table 2.

Table 2: L488 Hydrologic Index for each Outlet

Outlet Levee Station	Junction	Cumulative Drainage Area (mi^2)
442+43	24C	18.34
508+82	25C	1.92
604+21	28C	0.63
238+90	30C	4.02
551+33	4C	2.52

Hydrologic parameters of each subbasin are shown in Table 3 that include hydrologic connections, outlets through the levee, basin area, and lag time. Uniform parameters for all basins included the Snyder's transform method with a peaking coefficient of 0.6, the recession baseflow initial discharge of 0.5 cfs/square mile, and initial/constant loss methods with an initial deficit of 1-inch. Basins located in the floodplain do not have downstream reaches because they are directly connected to the outlet structures. Variation of parameters such as loss rates and recession constants is discussed in the subsequent sections.

Table 3: L488 Subbasin Parameters

Basin Name	Downstream Reach	Outlet Structure	Basin Location	Basin Area (mi^2)	Lag Time (hours)	Recession Constant	Threshold Flow (cfs)	Maximum Storage (in)	Constant Rate (in/hr)
6B	17R	4C	above bluffs	0.515	0.67	0.4	0.515	4.1	0.21
5B	16R	4C	above bluffs	1.207	0.92	0.45	1.207	4.1	0.24
44B	--	4C	Floodplain	0.800	0.96	0.4	0.8	3.9	0.27
2B	13R	30C	above bluffs	0.298	0.57	0.4	0.298	4.0	0.26
1B	13R	30C	above bluffs	0.183	0.37	0.4	0.183	4.0	0.23
24B	--	30C	Floodplain	3.536	3.51	0.45	3.536	3.6	0.41
7B	18R	28C	above bluffs	0.112	0.27	0.4	0.112	3.9	0.22
4B	15R	28C	above bluffs	0.154	0.35	0.4	0.154	4.0	0.20
17B	18R	28C	Floodplain	0.360	0.67	0.4	0.36	4.0	0.25
8B	19R	25C	above bluffs	0.326	0.46	0.4	0.326	4.3	0.34
42B	--	25C	Floodplain	1.593	2.08	0.45	1.593	3.5	0.21
12B	23R	24C	above bluffs	0.232	0.45	0.4	0.232	4.1	0.21
11B	22R	24C	above bluffs	0.350	0.59	0.4	0.35	4.1	0.25
10B	21R	24C	above bluffs	2.258	1.46	0.45	2.258	4.1	0.23
9B	20R	24C	above bluffs	5.493	1.66	0.45	5.493	4.1	0.26
3B	14R	24C	above bluffs	1.620	1.15	0.45	1.62	4.1	0.29
22B	--	24C	Floodplain	8.385	4.86	0.45	8.385	3.3	0.20

2.2.1 Loss methods

Hydrologic losses were established with considerations to published saturated hydraulic conductivity (Ksat) and hydrologic soil groups (HSGs) estimates within the modeled basins. These values were compared to the Mill Creek pilot study parameters to identify possible differences in runoff losses between each basin. The comparison of soil properties for the two basins was used to improve the site investigations for the L488 basin, which has no gaged information with which could be used for calibration. Aquaveo's Watershed Modeling System (WMS) v 10.0 software was used to acquire geospatial layers of soil parameters from the SSURGO database that are shown in Figure 3 and Figure 4. The analysis indicates the Mill Creek basins have Ksat values of of 2-4 um/sec and mostly soil types B. For comparison purposes it is noted that 1um/sec = 0.142inches/hour. The L488 basins above the bluff line had KSAT values of 1-3 um/sec and soil types B&C with some D. The L488 flood plain basins have lower values of KSAT no more than 1 um/sec with soil types D&C.

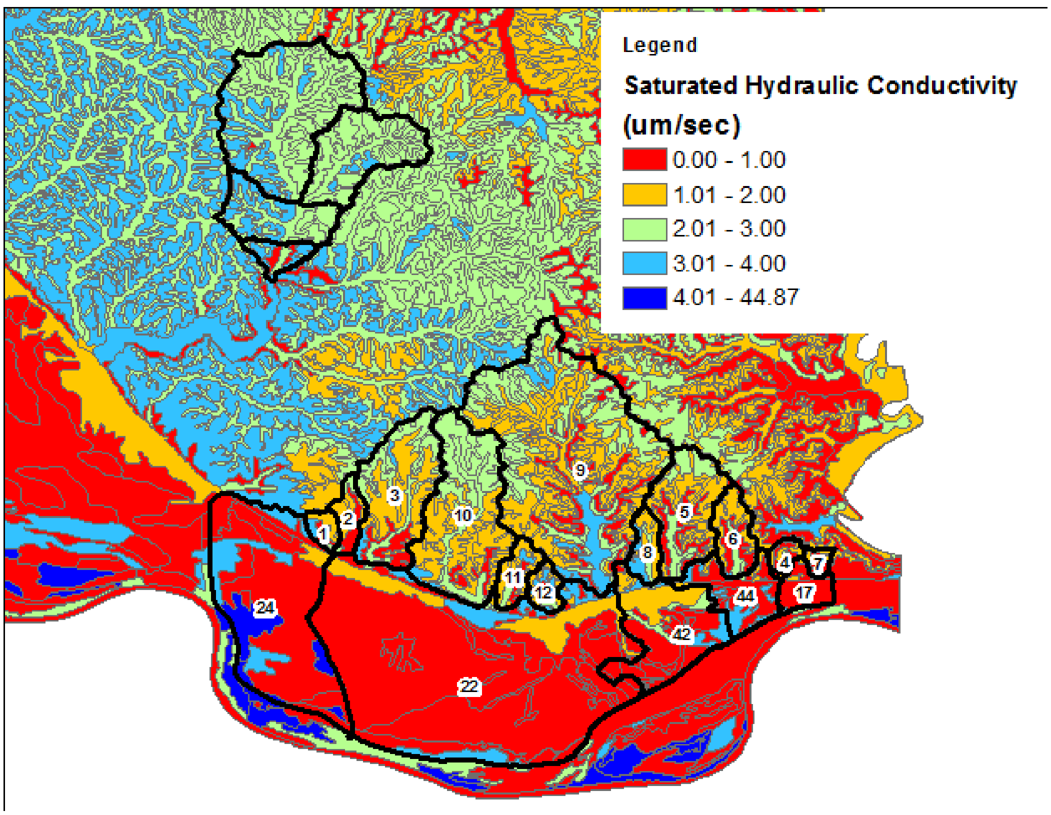

Figure 3: SSURGO Saturated Ksat in each sub-basin of L488 and Mill Creek Basins

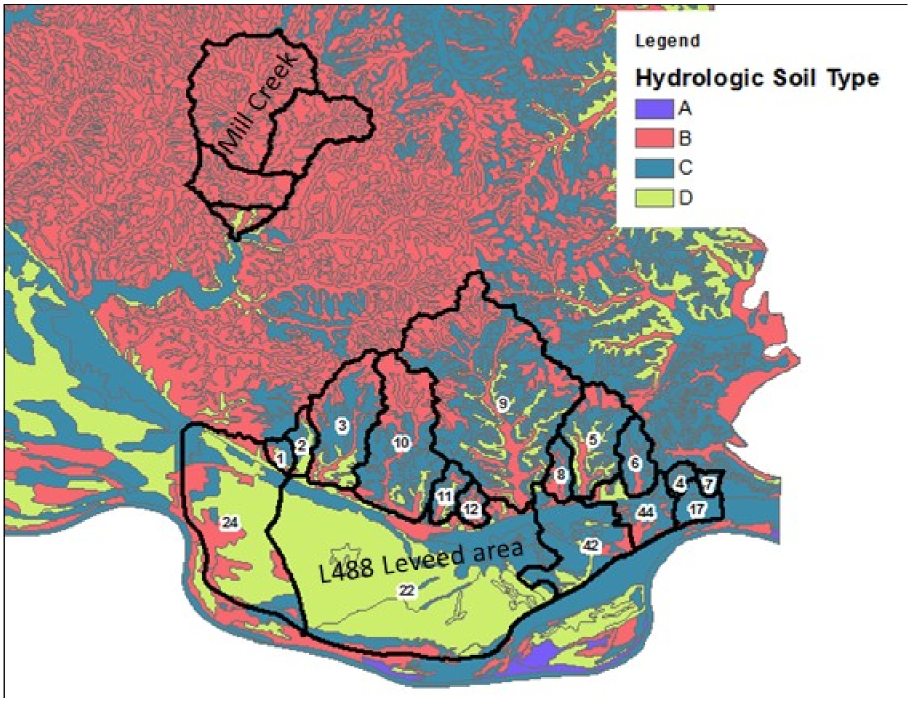

Figure 4: SSURGO Hydrologic Soils in each sub-basin of Mill Creek and L488

Rainfall losses were modeled using the deficit and constant loss method. This method uses the following parameters: initial deficit, maximum storage, constant rate, and percent impervious. The initial deficit is an estimate of the moisture required to saturate the soil at the start of the simulation. The maximum storage (or maximum deficit) is the amount of moisture needed to saturate the soil. The constant loss rate is similar to the hydraulic conductivity of the soil and is an important calibration parameter. Soil properties were determined based on the SSURGO soils data and associated properties from Rawls, et al (1983). L488 is composed of mostly silt loam and silty clay soils. The percent impervious was left at zero as it was considered negligible due to this being mostly agriculture land.

2.2.1 Baseflow

Baseflow was modeled by scaling the baseflow as a function of the basin area for each subbasin. A baseflow of 0.5 cfs per square mile was used. This compares to the Mill Creek pilot study, where the median flow from the gage was 0.53 cfs per square mile. The recession constant was also estimated as a function of basin area. Recession constants of 0.4, 0.45, and 0.5 were applied to basins with less than 1 square mile, 1-10 square miles, and 10-20 square miles, respectively.

2.2.2 Routing Reaches

Routing reaches were modeled using the Muskingum-Cunge method with an automatic fixed interval, a manning's n-value of 0.035, and a trapezoidal channel shape with 40-ft widths and 2H:1V side slopes. Computed values of slope and length for each modeled reach is shown in Table 4.

Table 4: L488 Routing Reach Parameters

Routing Reach	Final Outlet	Length (ft)	Slope (ft/ft)
17R	4C	6577.478	0.00319
16R	4C	10154.31	0.00251
13R	30C	15540.52	0.00136
18R	28C	1067.984	0.00763
15R	28C	3383.138	0.00383
19R	25C	9778.543	0.00169
23R	24C	9918.963	0.00186
22R	24C	12493.28	0.00153
21R	24C	13362.53	0.00146
20R	24C	11032.43	0.00076
14R	24C	26089.8	0.00096

2.3 Meteorological Model Components

2.3.1 Evapotranspiration and Canopy Loss Considerations

The meteorological model considered the Mill Creek study's evapotranspiration (ET) with a monthly average rate and coefficient as shown in the Table 5. Efforts to model ponding area elevations during the calibration process resulted in not modeling ET or canopy losses.

Table 5: Evapotranspiration Rates and Coefficients

Month	Average Rate (inches/month)	Coefficient
January	0	0.75
February	0	0.75
March	0	0.75
April	5.19	0.75
May	5.96	0.75
June	7.06	0.75
July	7.76	0.75
August	6.42	0.75
September	5.04	0.75
October	3.54	0.75
November	0	0.75
December	0	0.75

2.3.2 Precipitation

Hourly precipitation since 1948 was assembled and reviewed at the nearest 7 weather stations to L488 from the NCDC – CDO database to ensure the best quality precipitation dataset could be adopted for the study. Since continuous coverage was not available at any of the stations, the measurement coverage in each year at each gage was evaluated. Precipitation data input to the HEC-HMS model was assembled into a single time series by prioritizing measurement coverage and minimizing station distance from L488 and shown in Figure 5. Gage data sources are shown in Exhibit 1.

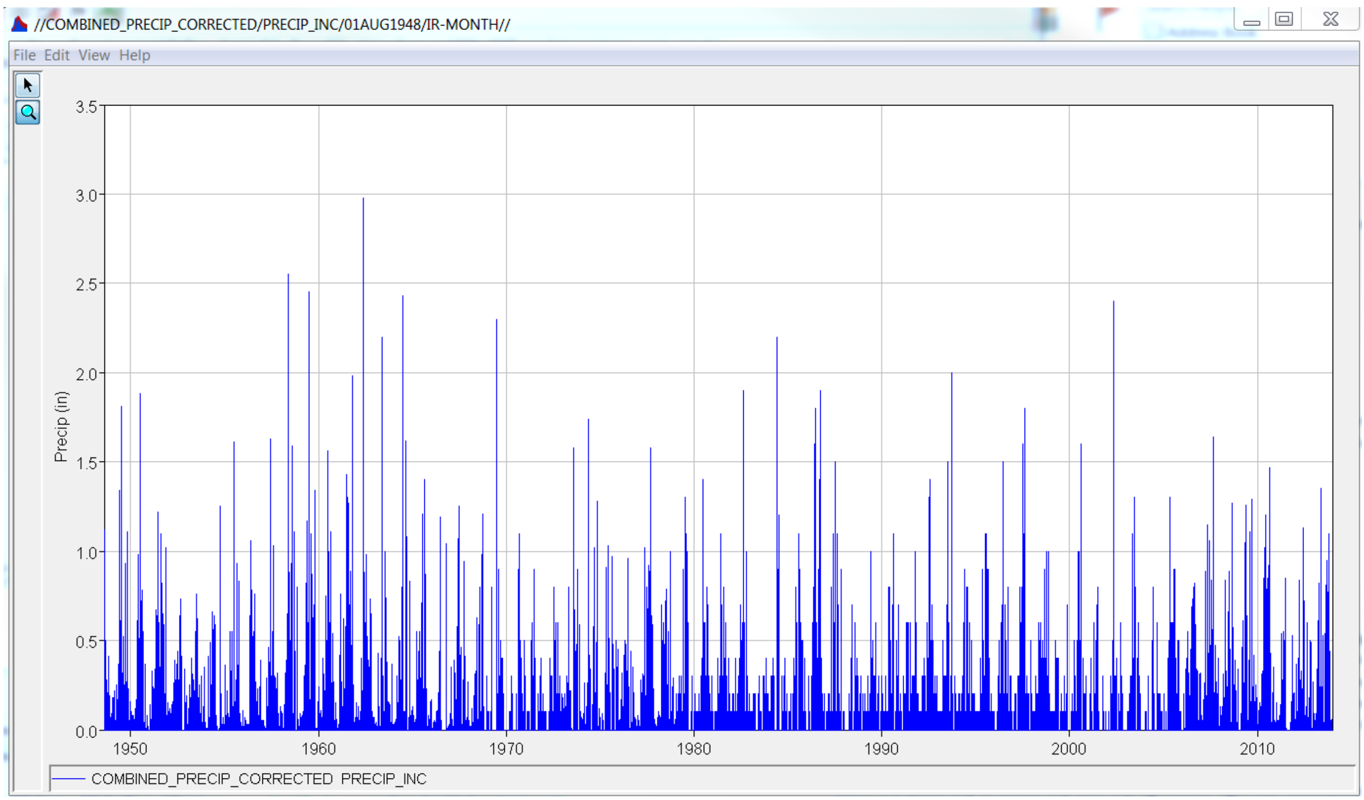

Figure 5: Adopted L488 Hourly Rainfall Period of Record

2.4 MODEL CALIBRATION AND VALIDATION

Model calibration involved comparing computed interior drainage pool elevations produced with modeled inflows within the hydraulic analysis to observations. This is described in detail in Section 5.2 in Appendix E of the Missouri River Unsteady HEC-RAS Models Alternative Analysis. Other calibration analysis involved the Mill Creek pilot study that included calibration to a stream gage within the basin. Several basin parameters from the Mill Creek pilot study were used as a starting point for the L488 analysis.

Additional site specific analysis was conducted to validate that the model was generating reasonable long-term runoff volumes and peak flows. Annual computed runoff in the L488 basin was accumulated and plotted against 25 years of available records at the USGS Mill Creek streamgage as shown in Figure 6. A USGS (1987) analysis that developed runoff estimates between 1951-1980 was compared to the modeled annual runoff for validation. The 1987 USGS publication estimated an average annual runoff of 7.5 to 8 inches of runoff during these years, whereas the HEC-HMS analysis produced a slightly higher estimate with an average of 8.6 inches. Canopy and ET losses affected these results by reducing the average modeled annual runoff to about 2.4 inches. This unrealistically low estimate of computed average annual runoff resulted in not including canopy or ET losses into the adopted modeling.

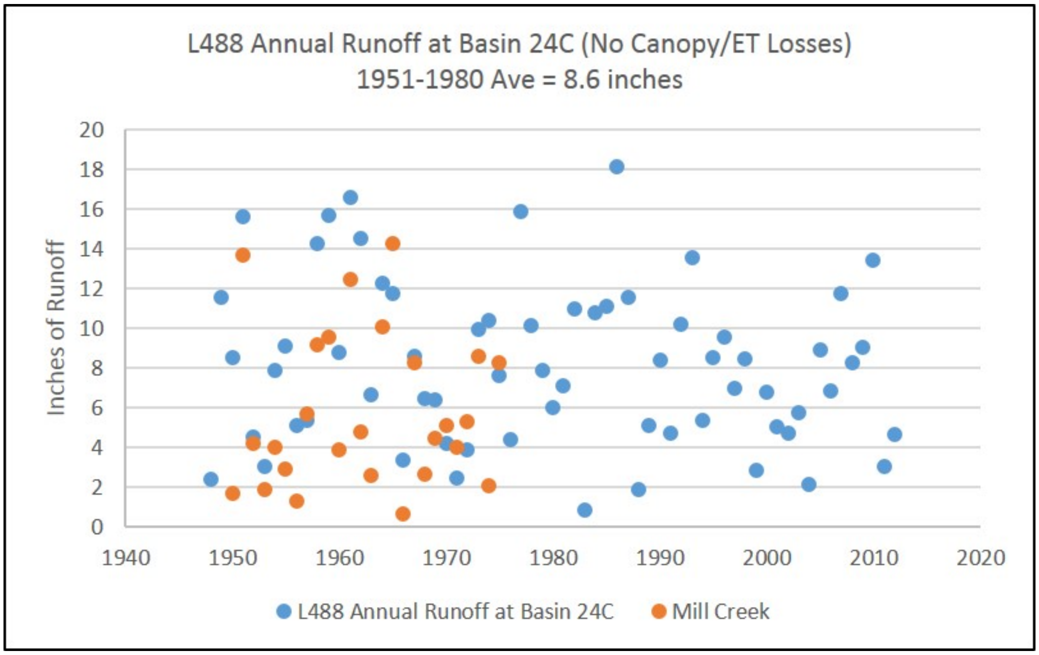

Figure 6: Annual Computed Runoff Volume

Additional analysis was conducted to validate the reasonableness of the results by comparing the frequency of annual peak flows from the HEC-HMS model to USGS regional streamflow statistic equations. Annual peak hourly inflows from HEC-HMS were used in the analysis, which would be expected to somewhat underestimate frequency event values derived from instantaneous flow records. Regression analysis inputs included a 4.8 mile longest flow path, 5.49 square mile basin, and a B-shape of 4.188. The Weibull positioned computed annual peak streamflow from Basin 9B are plotted, both with and without canopy and ET losses, along with the results of USGS regional streamflow regression equations in Figure 7. The annual peak streamflow with canopy and ET losses more closely matches the USGS regression equations than the annual peak streamflow without canopy and ET losses. However, it was determined to not include canopy and ET in the adopted modeling since neglecting these losses more reasonably matched long term runoff volumes. Additionally, the impact of the modeling producing large peak flows more often than expected would likely produce conservative results in the alternative analysis.

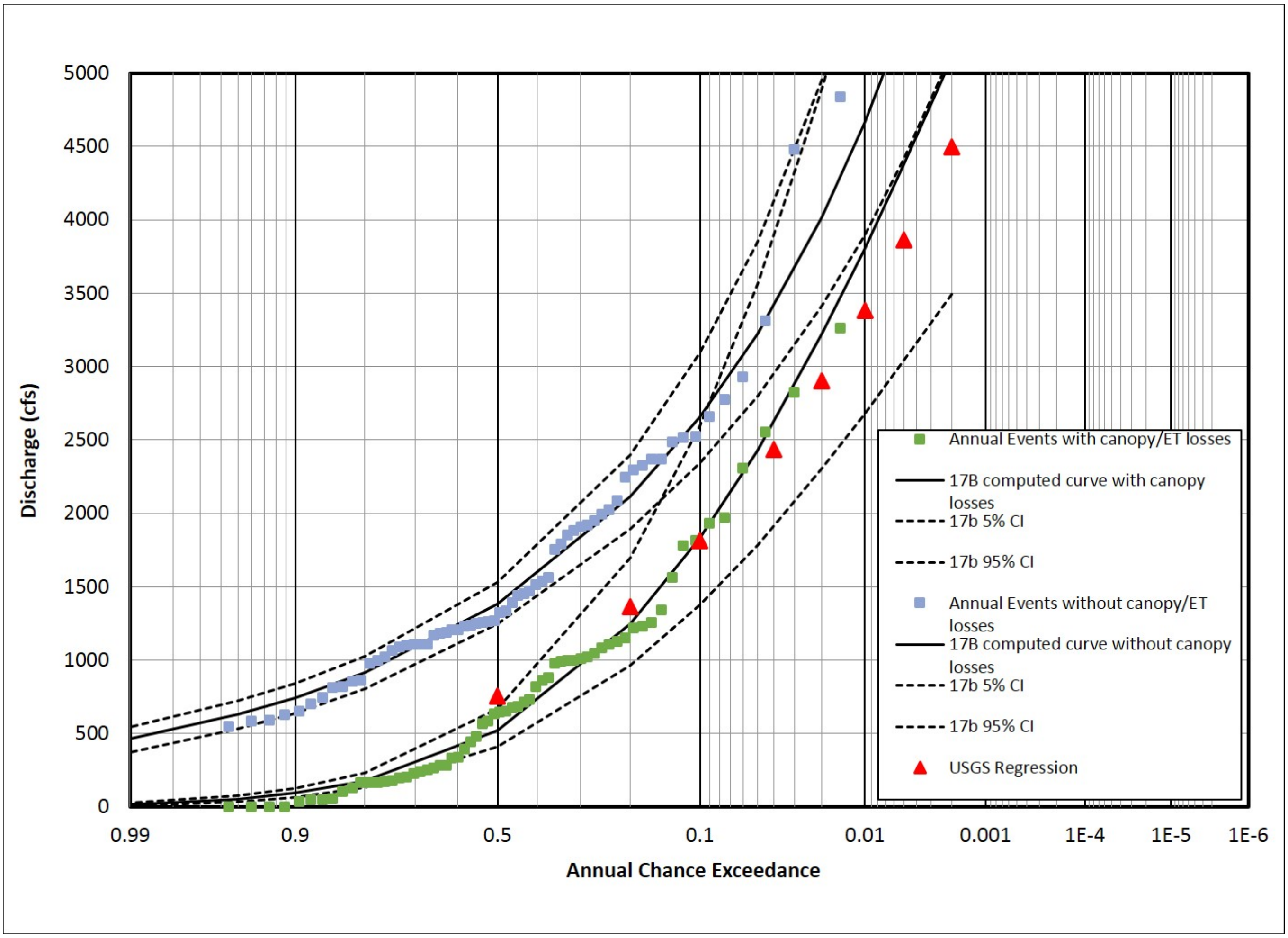

Figure 7: L488 Flow Frequency Comparison on Normal Probability Plot

2.5 RESULTS

Hourly discharge data was input to the unsteady hydraulic analysis to simulate interior drainage ponding and interactions with the Missouri River system. Typical results from HEC-HMS that were used as flow input to the HEC-RAS model is shown for the L488 levee at the station 442+43 outlet in Figure 8.

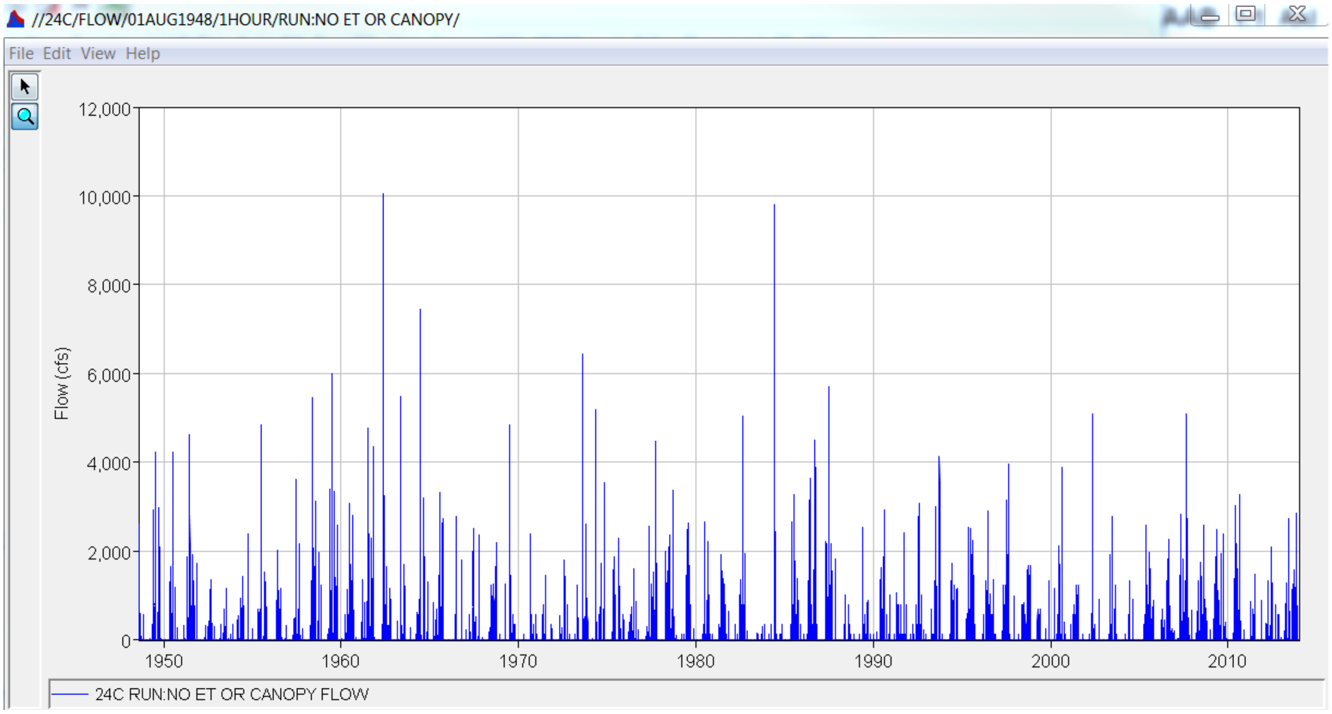

Figure 8: Hourly Computed Discharge at the 442+43 Outlet Structure at the L488 Levee

3 MRLS L246 INTERIOR DRAINAGE

The MRLS L246 levee, noted throughout the report as L246, interior drainage analysis has been developed to provide period of record runoff estimates that are produced by the contributing drainage basin of the levee system. The leveed area is generally east of Brunswick, Missouri and protects several agricultural lands along the Missouri River floodplain. The contributing drainage area consists of about 46 square miles of flood plain and about 44 square miles of area above the bluff line. Figure 9 shows a general layout of the interior drainage area.

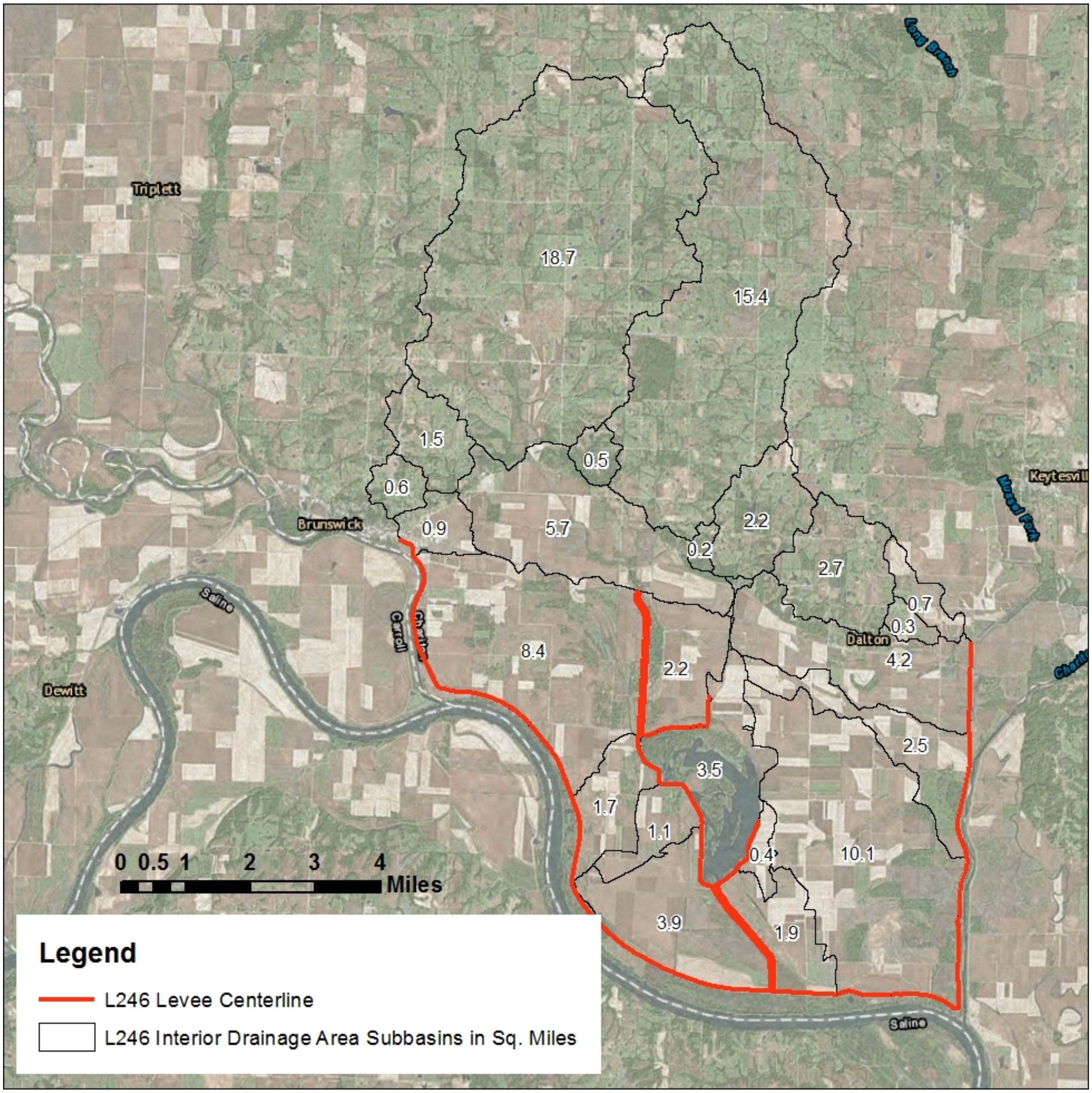

Figure 9: L246 Interior Drainage Layout and Subbasin Boundaries

3.1 HYDROLOGIC MODEL DEVELOPMENT

A hydrologic analysis was developed with parameters shown in Table 6. The basin model development involved estimating terrain, soil, and land use properties as well as calibrating the basin to meet the objectives of the study.

Table 6: L246 Hydrologic Model Parameters

Model Parameter	**Adopted Method**
Hydrologic Modeling Software	HEC-HMS v 4.2
Basin delineation and physical terrain analysis	A 10-meter terrain grid obtained from the USGS National Elevation Database (NED) was processed with Aquaveo WMS software.
Precipitation Data	Hourly point precipitation data at several gages
Loss Methods	Deficit and Constant (initial deficit of 1-inch, maximum storage of 3.6 inches, and constant rate of 0.2 inches/hour)
Routing Methods	Muskingum Cunge (0.024 manning's n)
Transform methods	Snyder Unit Hydrograph (0.6 peaking coefficient, computed lag time for each subbasin)
Canopy Methods	none
Hydrologic model calibration approach	Adapted from the Mill Creek pilot study basin parameters
HEC-HMS working File Location	K:\MissionProjects\sec\ed-h\MoRiver_Models\500_Rivers\592_Interior-Drainage\HMS

The hydrologic model was developed so that much of the basin could estimate runoff that were routed to Cutoff Lake. These Cutoff Lake inflows were then modeled as storage area inflows within the HEC-RAS analysis and routed to the mainstem Missouri River through a lateral connection. Flows were also modeled to three outlet structures on the Chariton River tieback, which were inflows to the Chariton River reach of the hydraulic model. One outlet structure on the Grand River was modeled which provided inflow to the Grand River reach of the hydraulic model. Two outflows from the leveed area to the Missouri River were modeled and connected as inflows to the Missouri River reach of the hydraulic model. The hydrologic basin schematic is shown in Figure 10.

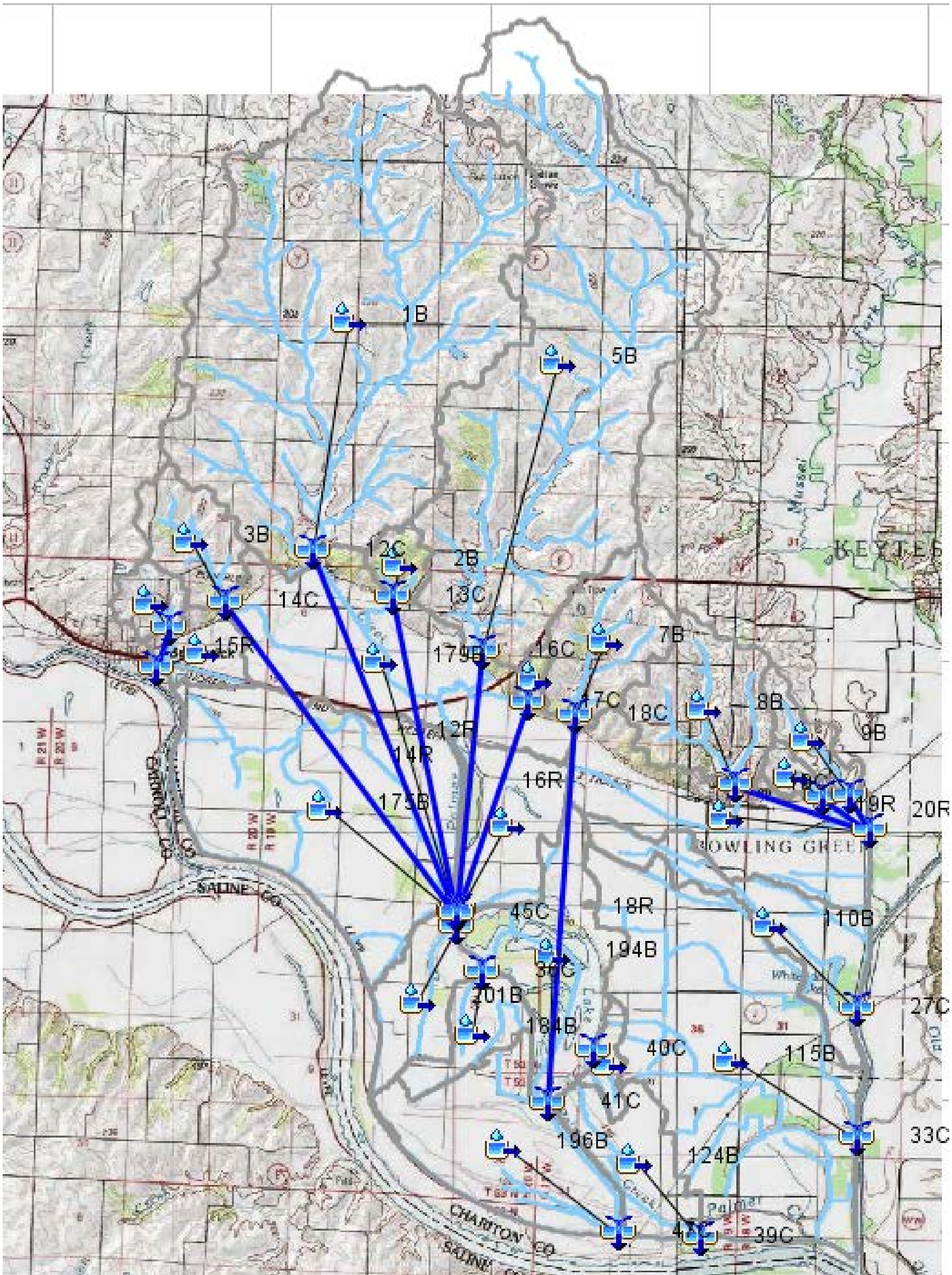

Figure 10: L246 Hydrologic Basin Schematic

3.2 BASIN MODEL COMPONENTS

The basin model consisted of estimating several parameters that were required to route flow to each outlet structure in the levee. Outlet structures and Cutoff Lake inflows are shown in Figure 10 with contributing subbasins summarized in Table 7.

Table 7: L246 Hydrologic Index for each Outlet

Outlet Levee Station	HEC-HMS Junction	Cumulative Drainage Area (mi²)
241+03	29C	8.37
498+00	47C	3.89
240+50	32C	2.2
288+68	36C	1.11
400+05	41C	5.71
327+27	40C	0.42
17+75	34C	1.54
24+90	25C	7.96
142+00	27C	2.49
228+00	33C	10.1
627+00	39C	1.91
Palmer Creek inflows to Cutoff Lake	45C	41.95
288+68	49C	1.65

Hydrologic parameters of each subbasin are shown in Table 8 that include hydrologic connections, outlets through the levee, basin area, and lag time. Uniform parameters for all basins included the Snyder's transform method with a peaking coefficient of 0.6, the recession baseflow initial discharge of 0.5 cfs/square mile, and initial/constant loss methods with an initial deficit of 1-inch. Basins located in the floodplain do not have downstream reaches because they are directly connected to the outlet structures. Variation of parameters such as loss rates and recession constants is discussed in the subsequent sections.

Table 8: L246 Subbasin Parameters

Basin Name	Downstream Reach	Outlet Structure	Basin Location	Basin Area (mi^2)	Lag Time (hours)	Recession Constant	Threshold Flow (cfs)	Maximum Storage (in)	Constant Rate (in/hr)
201B	--	49C	floodplain	1.65	6.47	0.45	1.65	4.0	0.33
196B	--	47C	floodplain	3.89	8.85	0.45	3.89	3.6	0.12
191B	--	40C	floodplain	0.42	1.75	0.4	0.42	3.8	0.22
4B	15R	34C	above bluffs	0.63	0.58	0.4	0.63	3.9	0.24
181B	--	34C	floodplain	0.91	1.06	0.4	0.91	3.7	0.23
180B	--	32C	floodplain	2.20	4.52	0.45	2.2	3.7	0.15
175B	--	29C	floodplain	8.37	11.25	0.45	8.37	3.3	0.27
6B	17R	45C	above bluffs	0.23	0.42	0.4	0.23	4.1	0.32
5B	16R	45C	above bluffs	15.38	3.68	0.5	15.38	3.5	0.18
3B	14R	45C	above bluffs	1.51	1.00	0.45	1.51	3.8	0.21
2B	13R	45C	above bluffs	0.52	0.70	0.4	0.52	4.1	0.25
1B	12R	45C	above bluffs	18.65	2.61	0.5	18.65	3.5	0.17
179B	--	45C	floodplain	5.66	3.40	0.45	5.66	3.7	0.19
184B	--	36C	floodplain	1.11	8.82	0.45	1.11	3.2	0.11
7B	18R	41C	above bluffs	2.21	1.22	0.45	2.21	4.2	0.24
194B	--	41C	floodplain	3.50	16.14	0.45	3.5	3.2	0.21
124B	--	39C	floodplain	1.91	6.07	0.45	1.91	4.3	0.34
110B	--	27C	floodplain	2.49	7.87	0.45	2.49	3.7	0.22
115B	--	33C	floodplain	10.10	9.41	0.5	10.1	3.6	0.38
10B	21R	25C	above bluffs	0.34	0.82	0.4	0.34	4.0	0.29
9B	20R	25C	above bluffs	0.71	1.56	0.4	0.71	4.1	0.29
8B	19R	25C	above bluffs	2.69	1.26	0.45	2.69	4.0	0.26
143B	--	25C	floodplain	4.22	2.34	0.45	4.22	3.6	0.24

3.2.1 Loss Methods

Saturated hydraulic conductivity (Ksat) and hydrologic soil groups (HSGs) were evaluated at each subbasin of the L246 hydrologic model and compared to the Mill Creek pilot study parameters to identify reasons why possible differences in runoff might occur between each basin. The comparison of soil properties for the two basins was used to improve the site investigations for the L246 basin, which has no gaged information with which could be used for calibration. Aquaveo's Watershed Modeling System (WMS) v 10.0 software was used to develop geospatial layers of both soil parameters that are shown in Figure 11 and Figure 12. The analysis indicates the Mill Creek basins have Ksat values of 2-4 um/sec and mostly soil types B. For comparison purposes, it is noted that 1um/sec = 0.142inches/hour. The L246 basins have Ksat values that range between 0-3 um/sec with soil types D&C.

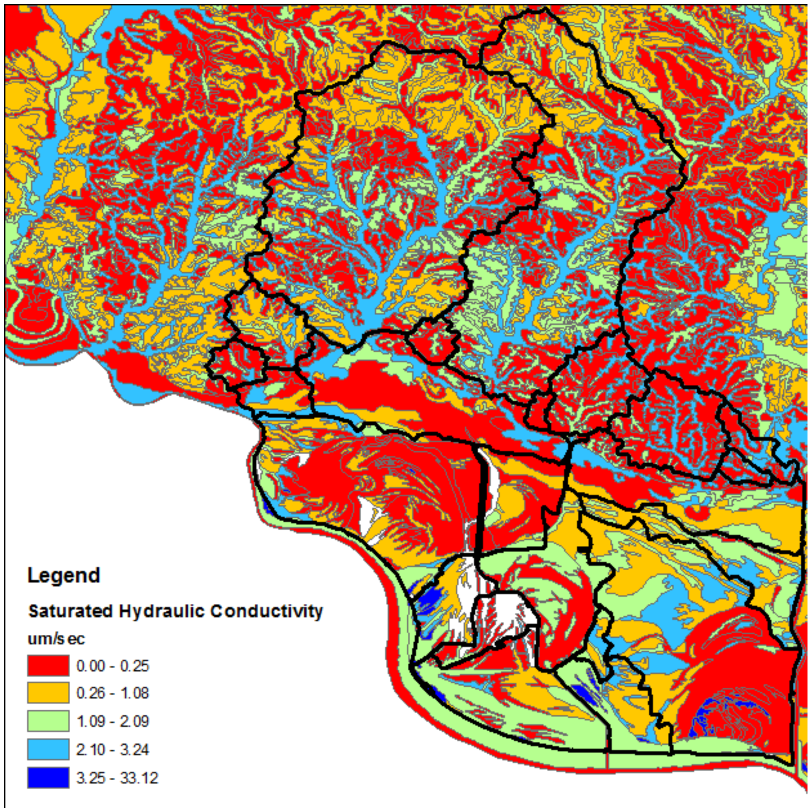

Figure 11: SSURGO Saturated Ksat in each sub-basin of L246

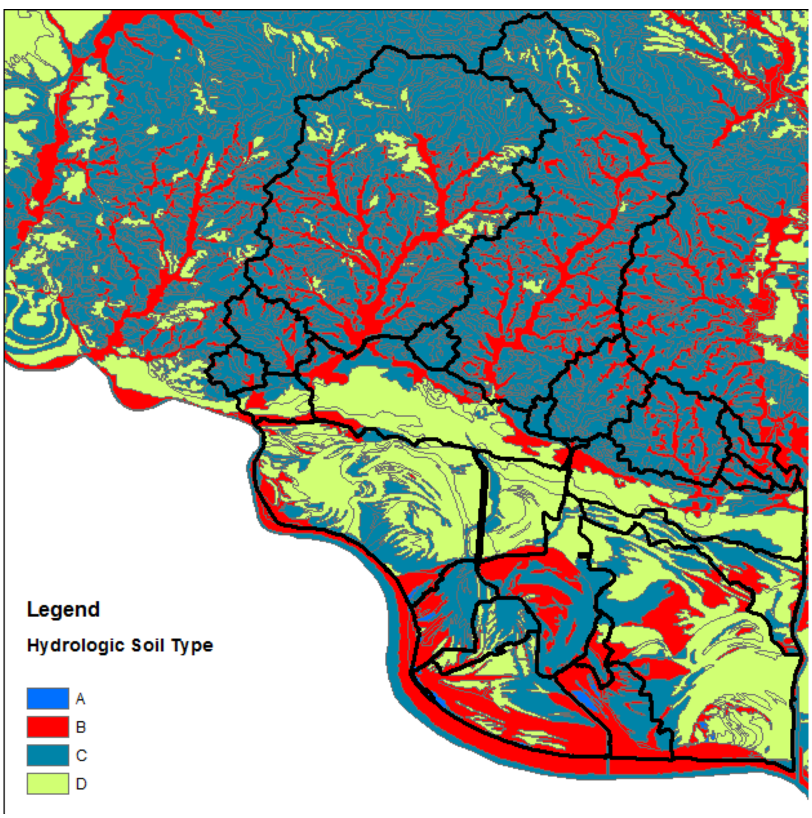

Figure 12: SSURGO Hydrologic Soils in each sub-basin of L246

Rainfall losses were modeled using the deficit and constant loss method. This method uses the following parameters: initial deficit, maximum storage, constant rate, and percent impervious. The initial deficit is an estimate of the moisture required to saturate the soil at the start of the simulation. The maximum storage (or maximum deficit) is the amount of moisture needed to saturate the soil. The constant loss rate is similar to the hydraulic conductivity of the soil and is an important calibration parameter. Soil properties were determined based on the SSURGO soils data and associated properties from Rawls, et al (1983). L246 is composed of mostly silt loam, clay, clay loam, and silty clay soils. The percent impervious was left at zero as it was considered negligible due to this being mostly agriculture land.

3.2.1 Baseflow

Baseflow was modeled by scaling the baseflow as a function of the basin area for each subbasin. A baseflow of 0.5 cfs per square mile was used. This compares to the Mill Creek pilot study, where the median flow from the gage was 0.53 cfs per square mile. The recession constant was also estimated as a function of basin area. Recession constants of 0.4, 0.45, and 0.5 were applied to basins with less than 1 square mile, 1-10 square miles, and 10-20 square miles, respectively.

3.2.2 Routing Reaches

Routing reaches were modeled using the Muskingum-Cunge method with an automatic fixed interval, a manning's n-value of 0.035, and a trapezoidal channel shape. Computed values of slope and length and adopted channel widths and side slopes for each modeled reach are shown in Table 9.

Table 9: L246 Routing Reach Parameters

Routing Reach	Final Outlet	Length (ft)	Slope (ft/ft)	Channel Width (ft)	Side Slope
15R	34C	3149.1	0.003176	10	2
17R	45C	22991.2	0.001156	80	3
16R	45C	21450.5	0.000382	80	3
14R	45C	31964.3	0.00085	50	2
13R	45C	25647.7	0.001011	80	3
12R	45C	28368.2	0.000647	80	3
18R	41C	31526.7	0.000614	30	2
21R	25C	4873.0	0.002565	20	2
20R	25C	3606.7	0.001802	20	2
19R	25C	12054.6	0.00083	30	2

3.3 METEOROLOGICAL MODEL COMPONENTS

3.3.1 Evapotranspiration and Canopy Loss Considerations

The meteorological model considered the Mill Creek study's evapotranspiration (ET) with a monthly average rate and coefficient as shown in Table 5. Efforts to model ponding area elevations during the calibration process resulted in not modeling ET or canopy losses.

3.3.2 Precipitation

Hourly precipitation since 1948 was assembled and reviewed at the nearest 8 weather stations to L246 from the NCDC – CDO database to ensure the best quality precipitation dataset could be adopted for the study. Since continuous coverage was not available at any of the stations, the measurement coverage in each year at each gage was evaluated. Precipitation data input to the HEC-HMS model was assembled into a single time series by prioritizing measurement coverage and minimizing station distance from L246 and shown in Figure 13. Gage data sources are shown in Exhibit 2**Error! Reference source not found.**.

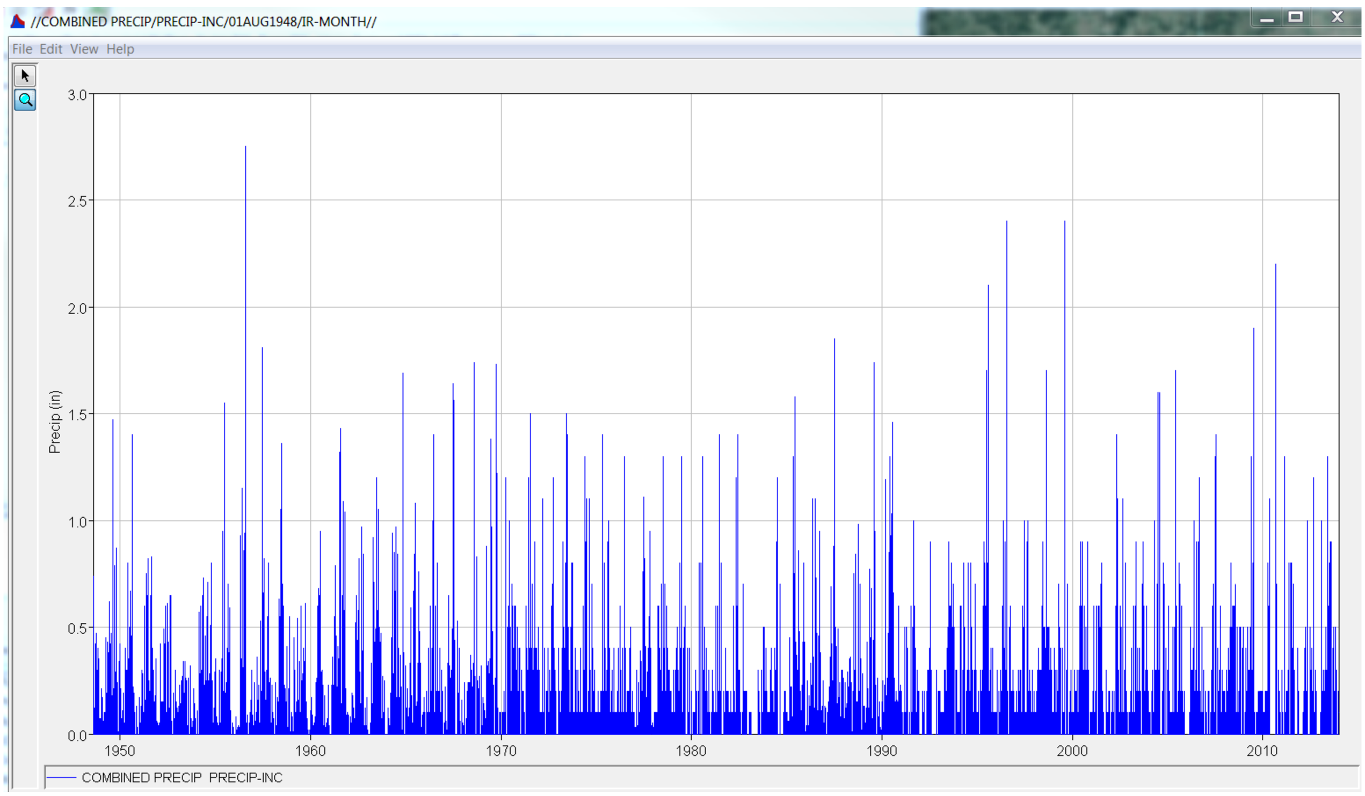

Figure 13: Adopted L246 Hourly Rainfall Period of Record

3.4 MODEL CALIBRATION AND VALIDATION

Model calibration involved comparing computed interior drainage pool elevations produced with modeled inflows within the hydraulic analysis to observations. This is described in detail in

Section 5.2 in Appendix E of the Missouri River Unsteady HEC-RAS Models Alternative Analysis. Other calibration analysis involved the Mill Creek pilot study that included calibration to a stream gage within the basin. Several basin parameters from the Mill Creek pilot study were used for the L246 analysis.

Additional site specific analysis was conducted to validate that the model was generating reasonable long-term runoff volumes and peak flows. Annual computed runoff in the L246 basin was accumulated and shown in Figure 14. A USGS (1987) analysis that developed runoff estimates between 1951-1980 was compared to the modeled annual runoff for validation. The 1987 publication estimated an average annual runoff of 8 to 9 inches of runoff during these years, whereas the HEC-HMS analysis produced a slightly higher estimate with an average of 10.1 inches. Canopy and ET losses affected these results by reducing the average modeled annual runoff to about 2.5 inches. This unrealistically low estimate of computed average annual runoff resulted in not including canopy or ET losses into the adopted modeling.

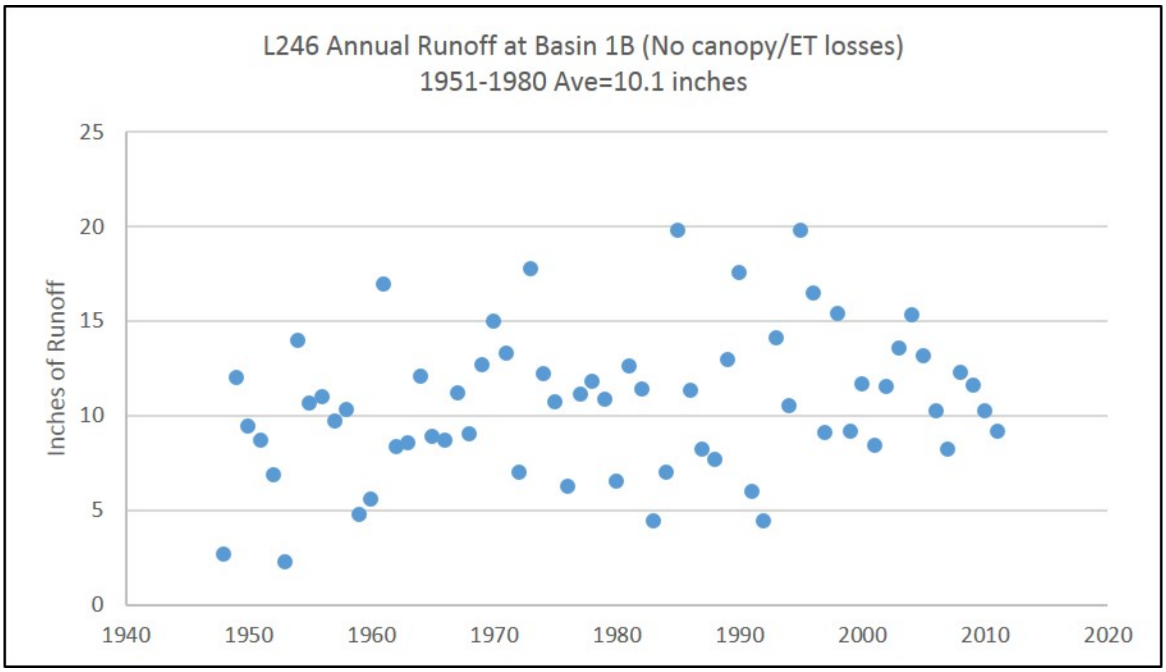

Figure 14: Annual Computed Runoff Volume

Additional analysis was conducted to validate the reasonableness of the results by comparing the frequency of annual peak flows from the HEC-HMS model to USGS regional streamflow statistic equations. Annual peak hourly inflows from HEC-HMS were used in the analysis, which would be expected to somewhat underestimate frequency event values derived from instantaneous flow records. Regression analysis inputs included a 7.8 square mile longest flow path, 18.65 square mile basin, and a B-shape of 3.266. The Weibull positioned computed annual peak streamflow from Basin 1B are plotted, both with and without canopy and ET losses, along with the results of USGS regional streamflow regression equations in Figure 15. The annual peak streamflow with canopy and ET losses more closely matches the USGS regression equations than the annual peak streamflow without canopy and ET losses. However, it was determined to not include canopy and ET in the adopted modeling since neglecting these losses more reasonably matched long term runoff volumes. Additionally, the impact of the modeling producing large peak flows more often than expected would likely produce conservative results in the alternative analysis.

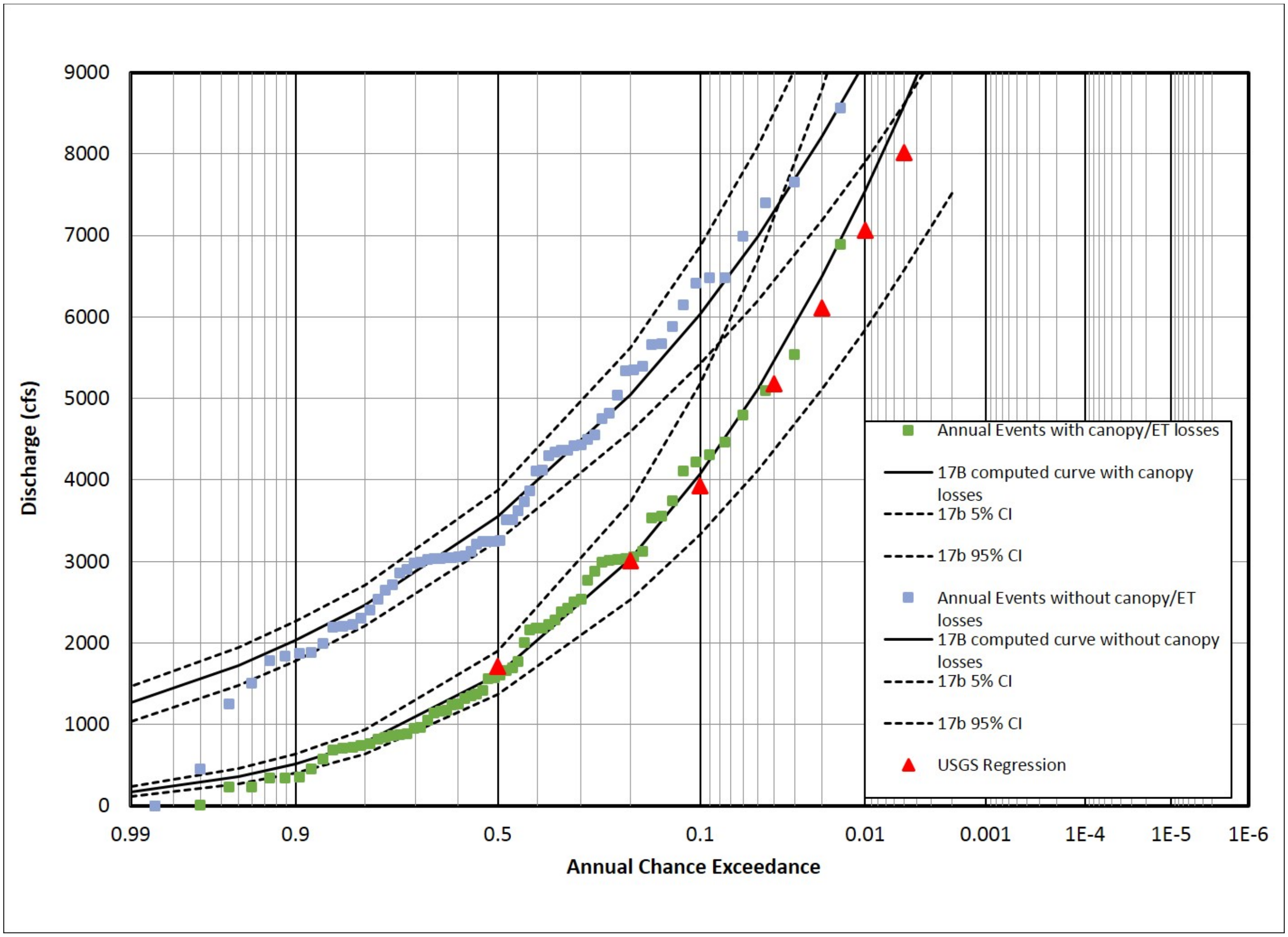

Figure 15: L246 Flow Frequency Comparison on Normal Probability Chart

3.5 RESULTS

Hourly discharge data was input to the unsteady hydraulic analysis to simulate interior drainage ponding and interactions with the Missouri River system. Typical output from the analysis is shown for the L488 levee at the station 442+43 outlet in Figure 16.

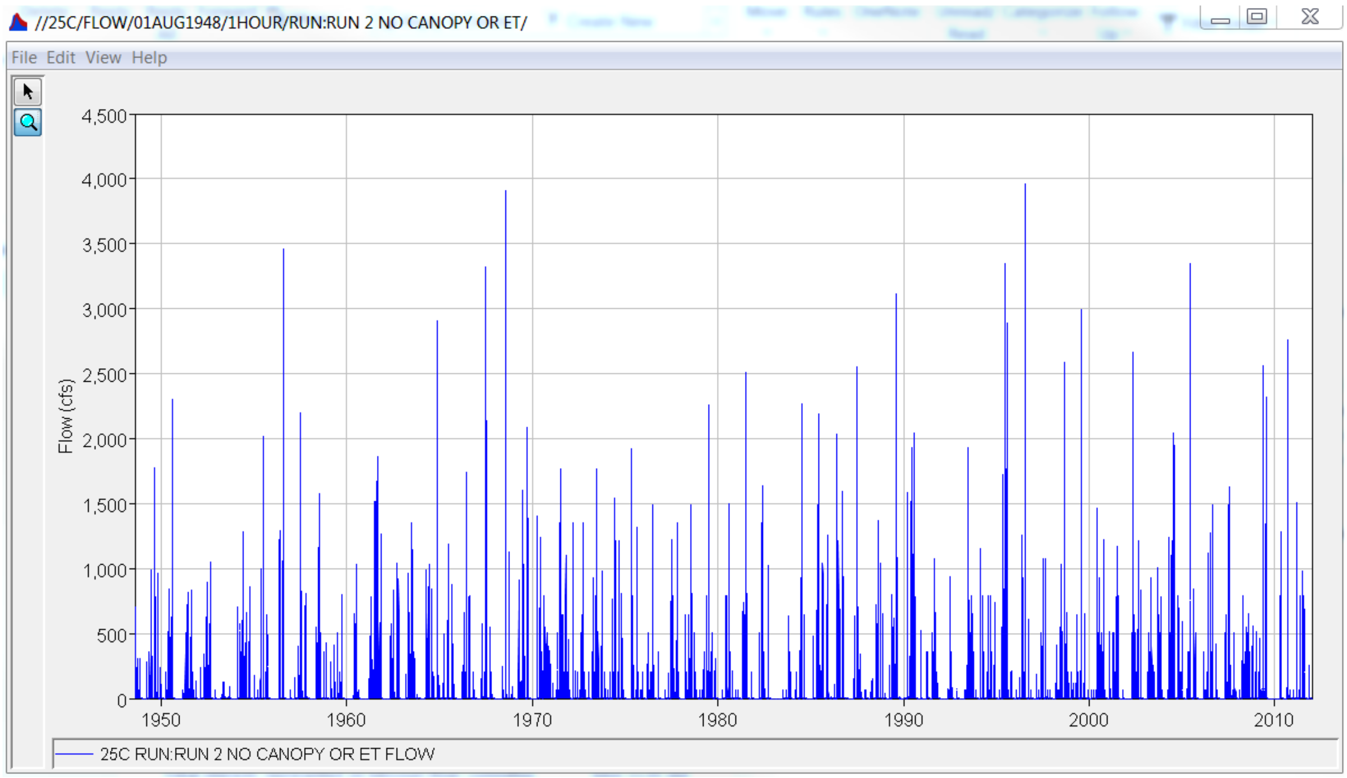

Figure 16: Hourly Computed Discharge at the 24+90 Outlet Structure at the L246 Levee

4 MODEL NOTES AND LIMITATIONS

Several model limitations were identified for this study that are listed below:

1. Routings in the floodplain are approximate and would be improved with a 2-dimensional hydraulic model because many channels and overland flow areas have very mild slopes and extensive overbanks.

2. The Mill Creek Pilot study's model calibration focused on matching daily volumes. Since these models adopt several hydrologic parameters from that study, they are not intended to be used to estimate peak discharge during flood events.

3. Historic precipitation data used in this study was selected based on the best nearby gage vs trying to Thiessen weight hourly data that is not continuous and applied this precipitation evenly across each drainage sub basin. Actual precipitation would have likely varied across the sub basins and would not have exactly matched the records of nearby precipitation gages. Additionally, some of the years had to use gages that were 46 miles away for L488 and 58 miles away for L246, as all the other nearby gages had little to no precipitation data. This distance creates some uncertainty in the precipitation that would have occurred at the study areas.

4. Many assumptions were made to develop each hydrologic analysis that are documented throughout this report. These assumptions were required to address issues associated with uncertain basin and meteorological conditions because of a general lack of observed hydrologic data at each site. However, these assumptions are considered reasonable for the purposes and scope of this study.

5 REFERENCES

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6 EXHIBITS

Exhibit 1: L488 Precipitation Station Summary

Gage Location	Troy3	Troy2	Oregon	Oregon1	Rosecrans Airport	StJoseph	KCI	
Gage Name	COOP:148250	COOP:148252	COOP:236357	COOP:236359	COOP:237440, COOP:237435	COOP:237445	COOP:234358	
distance from center of L488	5 miles S	9 miles S	10 miles NW	10 miles NNW	15 miles SE	20 miles SE	46 miles SE	
latitude (degrees)	39.8283	39.78333	39.9807	39.98333	39.7736	39.7533	39.2972	
longitude (degrees)	-95.088	-95.1	-95.1465	-95.13333	-94.9233	-94.8577	-94.7306	
Period of record	03/01/1988-12/25-2013	01/01/1971-04/01/1988	06/01/1979-12/22-2013	01/01/1951-01/01/1978	COOP:237440: 08/01/1948-01/01/1965 COOP:237435: 09/01/1974-12/29/2013	01/01/1965-1/1/1993	11/1/1972-12/29/2013	Adopted Gage
Year	Measurement coverage in each year							for analysis
1948	--	--	--	--	42%	--	--	Rosecrans Airport
1949	--	--	--	--	100%	--	--	Rosecrans Airport
1950	--	--	--	--	100%	--	--	Rosecrans Airport
1951	--	--	--	99%	100%	--	--	Rosecrans Airport
1952	--	--	--	99%	100%	--	--	Rosecrans Airport
1953	--	--	--	100%	100%	--	--	Rosecrans Airport
1954	--	--	--	100%	97%	--	--	Rosecrans Airport
1955	--	--	--	100%	100%	--	--	Rosecrans Airport
1956	--	--	--	100%	100%	--	--	Rosecrans Airport
1957	--	--	--	100%	100%	--	--	Rosecrans Airport
1958	--	--	--	99%	100%	--	--	Rosecrans Airport
1959	--	--	--	99%	100%	--	--	Rosecrans Airport
1960	--	--	--	98%	100%	--	--	Rosecrans Airport
1961	--	--	--	98%	100%	--	--	Rosecrans Airport
1962	--	--	--	99%	100%	--	--	Rosecrans Airport
1963	--	--	--	100%	100%	--	--	Rosecrans Airport
1964	--	--	--	98%	100%	--	--	Rosecrans Airport
1965	--	--	--	98%	100%	97%	--	St. Joseph
1966	--	--	--	99%	--	83%	--	St. Joseph
1967	--	--	--	99%	--	100%	--	St. Joseph
1968	--	--	--	91%	--	90%	--	St. Joseph
1969	--	--	--	95%	--	87%	--	St. Joseph
1970	--	--	--	100%	--	84%	--	St. Joseph
1971	--	56%	--	78%	--	84%	--	St. Joseph
1972	--	38%	--	73%	--	89%	17%	St. Joseph
1973	--	10%	--	87%	--	36%	100%	Oregon 1
1974	--	0%	--	61%	100%	42%	100%	KCI
1975	--	5%	--	84%	61%	100%	100%	Oregon 1
1976	--	11%	--	44%	17%	100%	100%	KCI
1977	--	0%	--	0%	0%	100%	100%	KCI
1978	--	36%	--	0%	68%	100%	100%	KCI
1979	--	64%	25%	--	100%	88%	100%	StJoseph
1980	--	100%	70%	--	100%	76%	100%	Troy 2
1981	--	100%	75%	--	100%	17%	100%	Troy 2
1982	--	98%	14%	--	100%	38%	100%	Troy 2
1983	--	67%	42%	--	100%	8%	100%	Troy 2
1984	--	79%	13%	--	100%	25%	100%	Troy 2
1985	--	92%	8%	--	100%	55%	100%	Troy 2
1986	--	92%	100%	--	100%	52%	100%	Troy 2
1987	--	89%	100%	--	100%	84%	100%	Troy 2
1988	78%	3%	100%	--	100%	35%	100%	Troy 3
1989	100%	--	100%	--	100%	0%	100%	Troy3
1990	100%	--	100%	--	100%	0%	100%	Troy3
1991	100%	--	77%	--	100%	0%	100%	Troy3
1992	100%	--	0%	--	100%	0%	100%	Troy3
1993	99%	--	24%	--	100%	0%	100%	Troy3
1994	100%	--	0%	--	100%	--	100%	Troy3
1995	100%	--	19%	--	100%	--	100%	Troy3
1996	91%	--	50%	--	100%	--	100%	Troy3
1997	100%	--	5%	--	100%	--	100%	Troy3
1998	99%	--	58%	--	100%	--	100%	Troy3
1999	82%	--	57%	--	100%	--	100%	Troy3
2000	81%	--	57%	--	100%	--	100%	Troy3
2001	50%	--	78%	--	100%	--	100%	Troy3
2002	99%	--	72%	--	100%	--	100%	Troy3
2003	68%	--	68%	--	16%	--	100%	Troy3
2004	43%	--	75%	--	0%	--	100%	Oregon
2005	79%	--	51%	--	0%	--	100%	Troy3
2006	28%	--	48%	--	100%	--	100%	Rosecrans Airport
2007	50%	--	57%	--	100%	--	100%	Rosecrans Airport
2008	100%	--	68%	--	100%	--	100%	Rosecrans Airport
2009	94%	--	94%	--	100%	--	100%	Rosecrans Airport
2010	74%	--	99%	--	100%	--	100%	Rosecrans Airport
2011	100%	--	80%	--	100%	--	100%	Rosecrans Airport
2012	99%	--	90%	--	100%	--	100%	Rosecrans Airport
2013	77%	--	92%	--	99%	--	99%	Rosecrans Airport

Exhibit 2: L246 Precipitation Station Summary

MO Gage Location	Marshall	Bynumville	Hamden	New Franklin	Higbee	Bedford	Sweet Springs	Columbia Regional Airport	
Gage Name	COOP:235298	COOP:231156	COOP:233565	COOP:236012	COOP:233835	COOP:230503	COOP:238223	COOP:231791	
Distance from Cutoff Lake	19 miles SSW	19 miles NE	21 miles NE	28 miles SSE	29 miles SEE	29 miles NW	35 miles SSW	58 miles SE	
Latitude (degrees)	39.1341	39.58333	39.6	39.0172	39.2415	39.68333	38.9663	38.81667	
Longitude (degrees)	-93.2225	-92.81667	-92.78333	-92.7558	-92.5068	-93.38333	-93.4195	-92.21667	
Period of record	07/31/1948-12/26/2013	05/31/1980-02/28/1990	07/31/1948-04/30/1980	07/31/1957-12/25/2013	4/30/1949-08/31/1992	07/31/1948-01/31/1965	06/30/1992-12/31/2013	10/1/1969-12/23/2013	Adopted Gage
Year	Measurement coverage in each year								for analysis
1948	42%	--	42%	--	--	42%	--	--	Marshall
1949	99%	--	97%	--	56%	99%	--	--	Marshall
1950	99%	--	100%	--	100%	99%	--	--	Marshall
1951	52%	--	95%	--	97%	97%	--	--	Hamden
1952	64%	--	99%	--	99%	98%	--	--	Hamden
1953	39%	--	100%	--	99%	99%	--	--	Hamden
1954	18%	--	100%	--	100%	100%	--	--	Hamden
1955	50%	--	99%	--	99%	100%	--	--	Hamden
1956	83%	--	99%	--	99%	99%	--	--	Hamden
1957	100%	--	99%	42%	100%	100%	--	--	Marshall
1958	100%	--	100%	99%	99%	100%	--	--	Marshall
1959	99%	--	99%	98%	100%	99%	--	--	Marshall
1960	89%	--	99%	98%	100%	98%	--	--	Hamden
1961	99%	--	100%	100%	100%	99%	--	--	Marshall
1962	99%	--	100%	99%	99%	98%	--	--	Marshall
1963	98%	--	99%	99%	99%	100%	--	--	Marshall
1964	98%	--	99%	98%	99%	100%	--	--	Marshall
1965	91%	--	100%	91%	100%	2%	--	--	Hamden
1966	100%	--	95%	67%	97%	--	--	--	Marshall
1967	83%	--	99%	58%	91%	--	--	--	Hamden
1968	71%	--	99%	100%	97%	--	--	--	Hamden
1969	86%	--	100%	82%	99%	--	--	25%	Hamden
1970	81%	--	92%	42%	91%	--	--	100%	Hamden
1971	100%	--	93%	67%	100%	--	--	100%	Marshall
1972	94%	--	92%	82%	76%	--	--	100%	Marshall
1973	88%	--	99%	13%	96%	--	--	100%	Hamden
1974	89%	--	99%	72%	82%	--	--	100%	Hamden
1975	56%	--	99%	100%	92%	--	--	100%	Hamden
1976	72%	--	78%	100%	97%	--	--	100%	New Franklin
1977	84%	--	64%	86%	95%	--	--	100%	Higbee
1978	54%	--	92%	100%	82%	--	--	100%	New Franklin
1979	8%	--	79%	100%	53%	--	--	100%	New Franklin
1980	0%	39%	26%	83%	0%	--	--	100%	New Franklin
1981	0%	5%	--	88%	13%	--	--	100%	New Franklin
1982	0%	21%	--	83%	59%	--	--	100%	New Franklin
1983	100%	17%	--	66%	36%	--	--	100%	New Franklin
1984	8%	0%	--	76%	13%	--	--	100%	New Franklin
1985	3%	0%	--	41%	5%	--	--	100%	Columbia Regional Airport
1986	9%	0%	--	17%	0%	--	--	100%	Columbia Regional Airport
1987	0%	0%	--	3%	0%	--	--	100%	Columbia Regional Airport
1988	0%	0%	--	0%	0%	--	--	100%	Columbia Regional Airport
1989	2%	8%	--	0%	0%	--	--	100%	Columbia Regional Airport
1990	31%	0%	--	14%	0%	--	--	100%	Columbia Regional Airport
1991	28%	--	--	76%	0%	--	--	100%	New Franklin
1992	8%	--	--	61%	0%	--	50%	100%	New Franklin
1993	69%	--	--	67%	--	--	100%	100%	Sweet Springs
1994	17%	--	--	80%	--	--	100%	100%	Sweet Springs
1995	1%	--	--	95%	--	--	100%	100%	Sweet Springs
1996	3%	--	--	91%	--	--	100%	100%	Sweet Springs
1997	52%	--	--	24%	--	--	100%	100%	Sweet Springs
1998	85%	--	--	49%	--	--	100%	100%	Sweet Springs
1999	75%	--	--	87%	--	--	94%	100%	Sweet Springs
2000	64%	--	--	98%	--	--	82%	100%	New Franklin
2001	23%	--	--	88%	--	--	80%	100%	New Franklin
2002	40%	--	--	55%	--	--	97%	100%	Sweet Springs
2003	75%	--	--	100%	--	--	96%	100%	New Franklin
2004	88%	--	--	83%	--	--	74%	100%	Marshall
2005	70%	--	--	88%	--	--	52%	100%	New Franklin
2006	80%	--	--	83%	--	--	93%	100%	Sweet Springs
2007	59%	--	--	95%	--	--	94%	100%	New Franklin
2008	65%	--	--	79%	--	--	91%	100%	Sweet Springs
2009	80%	--	--	88%	--	--	90%	100%	Sweet Springs
2010	69%	--	--	35%	--	--	46%	100%	Marshall
2011	71%	--	--	33%	--	--	41%	100%	Marshall
2012	8%	--	--	24%	--	--	63%	100%	Sweet Springs
2013	0%	--	--	97%	--	--	57%	98%	New Franklin

Appendix E

Attachment 7 – L-488 Interior Drainage Alternative Statistics

448A Stage

Min, Max, Percentile Statistics on the period of record (March 1930 - Dec 2012) hydrographs

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	825.4	825.4	825.4	825.4	825.4	825.4
10%	825.4	825.4	825.4	825.4	825.4	825.4
25%	825.4	825.4	825.4	825.4	825.4	825.4
50%	825.4	825.4	825.4	825.4	825.4	825.4
75%	825.9	825.8	825.9	825.9	825.9	825.9
90%	827.0	826.9	827.0	827.1	827.0	827.1
Max	834.3	834.2	834.3	834.4	834.3	834.3

Min, max, percentile change from No Action

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.1	0.0	0.1
90%	-	-0.1	0.0	0.1	0.0	0.1
Max	-	-0.2	0.0	0.0	0.0	0.0

Min, max, percentile on the daily change from No Action

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-3.9	-1.5	-3.5	-5.6	-2.9
10%	-	-0.2	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	5.6	2.2	5.8	3.9	5.9

448B Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	820.4	820.4	820.4	820.4	820.4	820.4
10%	821.3	821.3	821.3	821.3	821.3	821.3
25%	821.3	821.3	821.3	821.3	821.3	821.3
50%	821.3	821.3	821.3	821.3	821.3	821.3
75%	821.8	821.7	821.8	821.8	821.8	821.8
90%	823.5	823.2	823.5	823.6	823.5	823.6
Max	834.3	834.2	834.3	834.4	834.3	834.3

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	-0.2	0.0	0.1	0.0	0.1
Max	-	-0.2	0.0	0.0	0.0	0.0

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-3.9	-1.5	-3.5	-5.6	-2.9
10%	-	-0.2	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	5.6	2.2	5.8	3.9	5.9

448C
Stage

448D
Stage

Min, Max, Percentile Statistics on the period of record (March 1930 - Dec 2012) hydrographs

448C

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	821.3	821.3	821.3	821.3	821.3	821.3
10%	821.3	821.3	821.3	821.3	821.3	821.3
25%	821.3	821.3	821.3	821.3	821.3	821.3
50%	821.3	821.3	821.3	821.3	821.3	821.3
75%	821.4	821.4	821.4	821.4	821.4	821.4
90%	821.5	821.5	821.6	821.6	821.6	821.6
Max	833.4	832.5	833.3	833.8	833.4	833.4

448D

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	819.8	821.4	821.4	821.4	821.4	821.4
10%	821.3	821.3	821.3	821.3	821.3	821.3
25%	821.3	821.3	821.3	821.3	821.3	821.3
50%	821.3	821.3	821.3	821.3	821.3	821.3
75%	821.3	821.3	821.3	821.3	821.3	821.3
90%	821.3	821.3	821.3	821.3	821.3	821.3
Max	821.3	821.3	821.3	821.3	821.3	821.3

Min, max, percentile change from No Action

448C

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.1	0.1	0.0	0.1
Max	-	-0.9	-0.1	0.4	0.0	0.0

448D

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	-0.1	0.1	0.1	0.1	0.1
Max	-	-0.9	-0.1	0.4	0.0	0.0

Min, max, percentile on the daily change from No Action

448C

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-4.3	-1.1	-3.1	-5.7	-2.6
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	5.1	2.8	6.1	7.1	6.1

448D

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-4.8	-1.4	-3.6	-6.3	-3.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	5.3	2.8	6.3	7.6	6.4

448E
Stage

Min, Max, Percentile Statistics on the period of record (March 1930 - Dec 2012) hydrographs

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	819.1	819.1	819.1	819.1	819.1	819.1
10%	820.8	820.8	820.8	820.8	820.8	820.8
25%	820.8	820.8	820.8	820.8	820.8	820.8
50%	820.8	820.8	820.8	820.8	820.8	820.8
75%	820.9	820.9	820.9	820.9	820.9	820.9
90%	821.1	821.1	821.1	821.1	821.1	821.1
Max	833.4	832.6	833.3	833.8	833.4	833.4

Min, max, percentile change from No Action

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	-0.1	0.1	0.1	0.1	0.1
Max	-	-0.9	-0.1	0.4	0.0	0.0

Min, max, percentile on the daily change from No Action

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-4.8	-1.4	-3.6	-6.3	-3.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	5.3	2.8	6.3	7.6	6.4

Appendix E

Attachment 8 – L-246 Interior Drainage Alternative Statistics

246B Stage	246C Stage

Min, Max, Percentile Statistics on the period of record (March 1930 - Dec 2012) hydrographs

246B Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	617.5	617.5	617.5	617.5	617.5	617.5
10%	621.2	621.2	621.2	621.2	621.2	621.2
25%	621.2	621.2	621.2	621.2	621.2	621.2
50%	621.2	621.2	621.2	621.2	621.2	621.2
75%	622.3	622.2	622.3	622.3	622.3	622.3
90%	623.3	623.3	623.3	623.3	623.3	623.3
Max	642.1	641.5	642.2	642.2	642.2	642.2

246C Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	615.4	615.4	615.4	615.4	615.4	615.4
10%	616.1	616.1	616.1	616.1	616.1	616.1
25%	616.1	616.1	616.1	616.1	616.1	616.1
50%	616.4	616.4	616.4	616.5	616.5	616.5
75%	619.7	619.5	619.7	619.8	619.7	619.8
90%	622.6	622.5	622.6	622.7	622.6	622.6
Max	642.1	641.5	642.1	642.1	642.1	642.1

Min, max, percentile change from No Action

246B Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	-0.1	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	-0.6	0.1	0.1	0.1	0.1

246C Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.1
75%	-	-0.2	0.0	0.1	0.0	0.0
90%	-	-0.1	0.0	0.1	0.0	0.0
Max	-	-0.6	0.1	0.1	0.1	0.1

Min, max, percentile on the daily change from No Action

246B Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-2.6	-0.9	-1.7	-1.5	-1.7
10%	-	-0.1	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	1.3	0.5	2.0	3.1	1.4

246C Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-5.1	-1.3	-4.1	-4.1	-2.7
10%	-	-0.3	0.0	0.0	0.0	0.0
25%	-	-0.1	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	4.4	2.1	4.5	2.6	4.0

246F Stage

Min, Max, Percentile Statistics on the period of record (March 1930 - Dec 2012) hydrographs

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	619.0	619.0	619.0	619.0	619.0	619.0
10%	619.1	619.1	619.1	619.1	619.1	619.1
25%	619.1	619.1	619.1	619.1	619.1	619.1
50%	619.2	619.2	619.2	619.2	619.2	619.2
75%	620.4	620.4	620.4	620.5	620.4	620.4
90%	622.1	622.1	622.1	622.1	622.1	622.1
Max	641.0	640.0	641.0	641.0	641.0	641.0

Min, max, percentile change from No Action

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	-1.0	0.0	0.0	0.0	0.0

Min, max, percentile on the daily change from No Action

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-3.9	-0.7	-1.5	-2.6	-1.5
10%	-	-0.1	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	2.9	0.3	2.8	2.7	2.8

246G Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	619.4	619.4	619.4	619.4	619.4	619.4
10%	620.8	620.8	620.8	620.8	620.8	620.8
25%	620.8	620.8	620.8	620.8	620.8	620.8
50%	620.8	620.8	620.8	620.8	620.8	620.8
75%	621.5	621.5	621.5	621.5	621.5	621.5
90%	622.3	622.3	622.3	622.3	622.3	622.3
Max	642.1	641.5	642.1	642.1	642.1	642.1

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	-0.6	0.1	0.1	0.1	0.1

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-9.1	-3.4	-3.5	-3.5	-3.4
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	2.2	0.1	2.1	2.5	2.1

246H Stage

Min, Max, Percentile Statistics on the period of record (March 1930 - Dec 2012) hydrographs

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	616.9	616.9	616.9	616.9	616.9	616.9
10%	619.1	619.1	619.1	619.1	619.1	619.1
25%	619.1	619.1	619.1	619.1	619.1	619.1
50%	619.2	619.2	619.2	619.2	619.2	619.2
75%	620.4	620.4	620.4	620.5	620.4	620.5
90%	622.1	622.1	622.1	622.1	622.1	622.1
Max	642.1	641.5	642.1	642.1	642.1	642.1

Min, max, percentile change from No Action

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	-0.6	0.1	0.1	0.1	0.1

Min, max, percentile on the daily change from No Action

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-4.9	-0.7	-1.5	-2.6	-1.5
10%	-	-0.1	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	2.9	0.3	2.8	2.7	2.8

246I Stage

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	611.8	611.8	611.8	611.8	611.8	611.8
10%	611.8	611.8	611.8	611.8	611.8	611.8
25%	611.9	611.9	611.9	611.9	611.9	611.9
50%	612.8	612.8	612.9	612.9	612.9	612.9
75%	617.3	617.2	617.3	617.3	617.3	617.3
90%	620.1	620.1	620.2	620.2	620.1	620.2
Max	642.0	641.4	642.1	642.1	642.1	642.1

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.1	0.1	0.1
75%	-	-0.1	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.1	0.0	0.1
Max	-	-0.6	0.1	0.1	0.1	0.1

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-4.1	-2.2	-3.0	-4.2	-2.8
10%	-	-0.2	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.1	0.1	0.1	0.1
Max	-	4.5	1.6	5.5	5.2	5.1

246J
Stage

246K
Stage

Min, Max, Percentile Statistics on the period of record (March 1930 - Dec 2012) hydrographs

246J

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	614.0	614.0	614.0	614.0	614.0	614.0
10%	614.1	614.1	614.1	614.1	614.1	614.1
25%	614.1	614.1	614.1	614.1	614.1	614.1
50%	614.2	614.1	614.2	614.2	614.2	614.2
75%	615.1	614.9	615.1	615.1	615.1	615.1
90%	618.9	618.9	618.9	618.9	618.8	618.9
Max	641.0	640.7	641.0	641.0	641.0	641.0

246K

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	612.8	612.8	612.8	612.8	612.8	612.8
10%	613.8	613.8	613.8	613.8	613.8	613.8
25%	613.8	613.8	613.8	613.8	613.8	613.8
50%	613.8	613.8	613.8	613.8	613.8	613.8
75%	614.1	614.0	614.1	614.1	614.1	614.1
90%	617.7	617.7	617.7	617.8	617.7	617.7
Max	641.0	640.7	641.0	641.0	641.0	641.0

Min, max, percentile change from No Action

246J

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	-0.2	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	-0.3	0.0	0.0	0.0	0.0

246K

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.1	0.0	0.0
Max	-	-0.3	0.0	0.0	0.0	0.0

Min, max, percentile on the daily change from No Action

246J

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-4.8	-2.0	-2.3	-4.3	-2.3
10%	-	-0.4	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	4.2	1.9	4.0	5.8	4.1

246K

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-8.9	-1.6	-3.2	-4.0	-2.7
10%	-	-0.1	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	4.9	2.4	4.5	6.9	3.8

246L
Stage

246M
Stage

Min, Max, Percentile Statistics on the period of record (March 1930 - Dec 2012) hydrographs

246L

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	619.7	619.7	619.7	619.7	619.7	619.7
10%	620.5	620.5	620.5	620.5	620.5	620.5
25%	620.5	620.5	620.5	620.5	620.5	620.5
50%	620.5	620.5	620.5	620.5	620.5	620.5
75%	620.5	620.5	620.5	620.5	620.5	620.5
90%	621.9	621.9	621.9	621.9	621.9	621.9
Max	642.0	641.4	642.0	642.0	642.0	642.0

246M

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	613.0	613.0	613.0	613.0	613.0	613.0
10%	613.9	613.9	613.9	613.9	613.9	613.9
25%	613.9	613.9	613.9	613.9	613.9	613.9
50%	613.9	613.9	613.9	613.9	613.9	613.9
75%	614.7	614.7	614.8	614.8	614.7	614.8
90%	619.1	619.2	619.2	619.3	619.2	619.2
Max	642.0	641.4	642.1	642.1	642.1	642.1

Min, max, percentile change from No Action

246L

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	-0.6	0.1	0.1	0.1	0.1

246M

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	0.0	0.0	0.0	0.0	0.0
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.1	0.1	0.1	0.0	0.1
Max	-	-0.6	0.1	0.1	0.1	0.1

Min, max, percentile on the daily change from No Action

246L

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-5.0	-0.7	-1.1	-0.7	-1.1
10%	-	0.0	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.0	0.0	0.0	0.0
Max	-	1.6	0.2	1.1	2.1	1.2

246M

	Alt 1	Alt 2	Alt 3	Alt 4	Alt 5	Alt 6
Min	-	-4.3	-1.6	-3.3	-4.1	-4.9
10%	-	-0.1	0.0	0.0	0.0	0.0
25%	-	0.0	0.0	0.0	0.0	0.0
50%	-	0.0	0.0	0.0	0.0	0.0
75%	-	0.0	0.0	0.0	0.0	0.0
90%	-	0.0	0.1	0.0	0.0	0.0
Max	-	5.8	2.5	6.3	6.2	3.9

Attachment 9a

US Army Corps of Engineers
Kansas City District

Lower Missouri River Sediment Model Calibration Report

31 JUL 2018

1.0 Introduction

The bed of the Missouri River experiences both short-term fluctuations, episodic responses to major floods, and long-term trends (USACE 2017a). Effective long-term management of the river should acknowledge and incorporate the effects of these bed elevation changes. This document serves as an initial orientation to development of a mobile-bed sediment model for the lower 500 miles of the Missouri River. Hereafter this model is referred to as the Lower Missouri River Sediment Model (LMRSM). The model creation and calibration followed the basic principles outlined in the HEC-6 Calibration Guide (HEC 1982) and EM 1110-2-4000, with some adaptation. This document summarizes river behavior and provides information on the model set up and calibration.

The purpose of the LMRSM is to compare future bed elevations under alternative management scenarios. The immediate purpose is to provide a 2033 bed elevation projection for use in the Missouri River Recovery Program under the various proposed alternatives. This is a planning level model to assess the reach and river scale effects of proposed alternatives. Final design of habitat projects will require additional calculations or modeling beyond the output of this mobile-bed model.

This report covers model development and calibration. A subsequent report will document alternatives testing.

2.0 River Behavior

2.1 River History and Modifications

The Missouri River in its current form is a highly regulated, highly stabilized river that drains approximately 529,350 square miles (USACE 2006). Major tributaries include the Yellowstone River, the Platte River, and the Kansas River, each of which drains more than 60,000 square miles. Prior to channel modification, the Missouri River was a wide, braided channel which occupied approximately 300,000 acres downstream of Sioux City, IA. Through the construction of a series of river training structures (dikes and revetments), the river was transformed into a single-thread channel with projected surface area of 112,000 acres (USACE 1981) downstream of Sioux City, IA. The current river is significantly narrower than the original, pre-modified river.

The system of river training structures is called the Bank Stabilization and Navigation Project (BSNP). Structures that run parallel to the river are called revetments, while those that protrude into the river are commonly called dikes. The dikes are generally several hundred feet to over a half mile long, but many of the dikes have most of their length buried in accreted land so that only a small portion of each dike is actually exposed to flow. Most dike structures downstream of Rulo, Nebraska are extended by a low sill which protrudes further into the channel. In common usage, the entire structure, both dike and sill, is referred to as a dike. Figure 1 illustrates a dike structure with a low sill with typical dimensions in reference to the CRP (Construction Reference Plane). The CRP is a sloping datum mirroring the water surface profile exceeded 75% of the time during navigation season, and is used to set structure heights to overtop at consistent frequencies based on design criteria from December 1973. A more detailed history of BSNP structure design standards, modifications, and current condition is included in the Missouri River Bed Degradation Feasibility Study report.

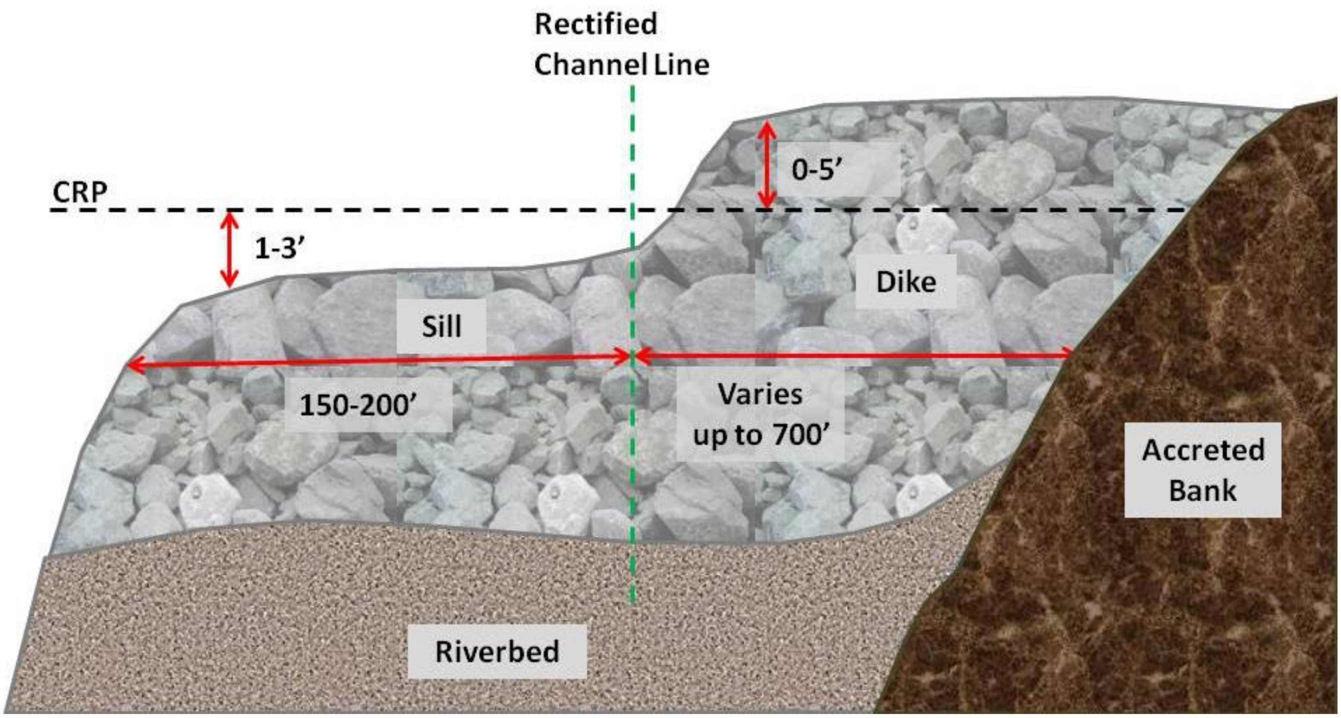


Figure 1. BSNP Dike and Sill with Typical Dimensions, not to scale

In conjunction with the BSNP, a series of six mainstem dams were constructed. These dams store a significant volume of water, reducing downstream flooding and supplying water to support navigation on the lower Missouri River eight months out of the year. The authorized purposes for the Missouri River Mainstem Reservoir System as outlined in the Section 9 of the 1944 Flood Control Act include: flood control, navigation, irrigation, hydropower, water supply, water quality control, recreation, and fish and wildlife (USACE 2006).

The channel stabilization accomplished through the BSNP has allowed the construction of miles of federally operated and maintained levees and floodwalls and additional privately owned levees generally located on both banks of the Missouri River downstream of Omaha, Nebraska. Portions of these levees, particularly in the more urbanized areas, as well as smaller privately owned agricultural levees, are located immediately adjacent to the river bank. At many locations, these small, privately-owned levees contain flows with a 5 to 10-year return interval at top width only slightly larger than the channel. Larger floods exceeding a 50-year return period are generally only contained by the federally constructed levees and only a few private levee systems. Due to the varying levels of protection of these levees, channel widths for major floods vary considerably from location to location (see Figure 2.)

Figure 2 graphically depicts the level of confinement imposed on the river corridor by levees vs. the natural floodplain width. The widths shown in Figure 2 were developed using existing models not the ManPlan study model. The "Valley Width" line in Figure 2 indicates the floodplain width for the 1% AEP profile if there were no levees and is provided as a reasonable approximation for the valley width. Figure 2 is not used in the sediment modeling, but is provided for context to differentiate geologic constrictions from levee-induced constrictions. The LMSM includes the levees at the actual locations.

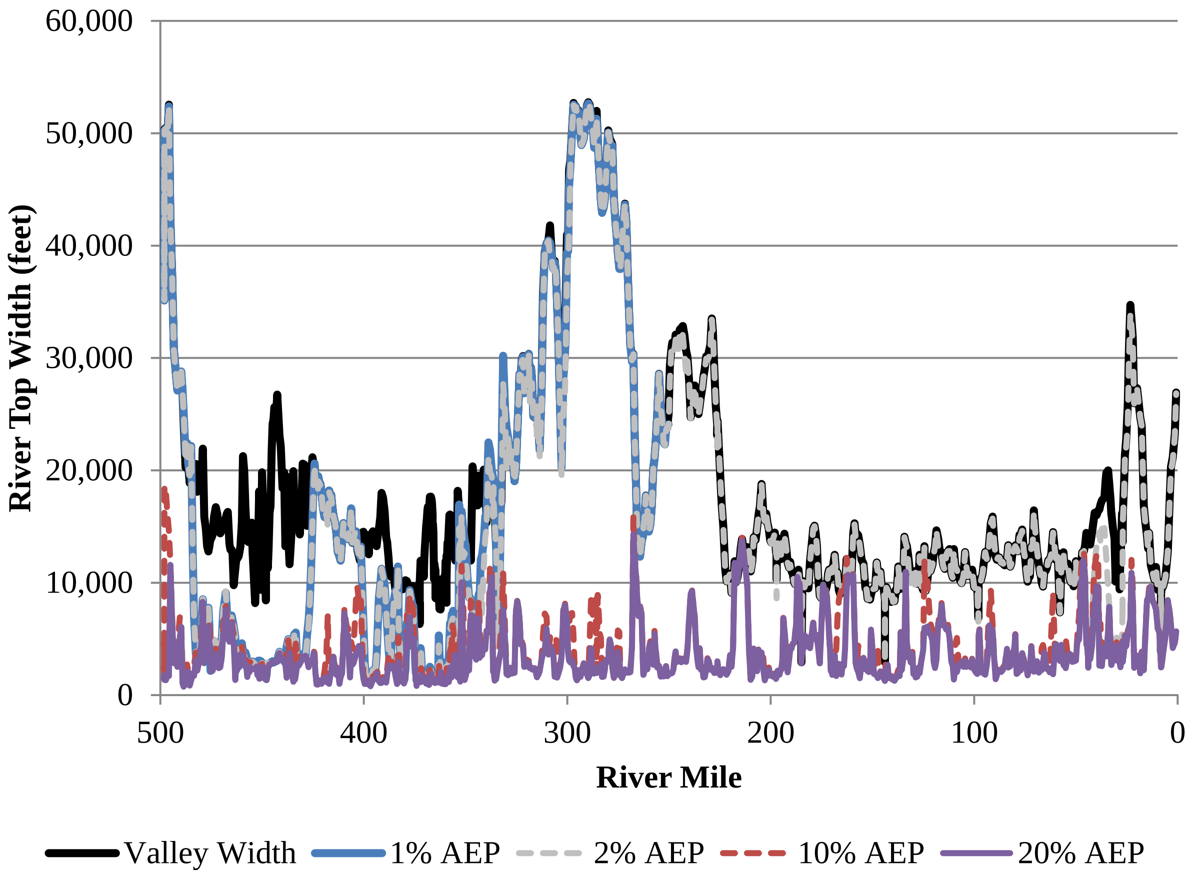


Figure 2. Valley Width and River Top Width at Different Flood Levels

There are no man-made grade controls on the lower Missouri River downstream of Gavin's Point Dam which is located at RM 811. Riprap placed at bridge piers does not armor the entire bed and has not stabilized the river. There are no known natural grade controls in the active channel, though this has not been the subject of a thorough investigation. There are some natural bedrock outcroppings on the river banks (Laustrup et al. 2007). The stability of the river near Waverly may be due to natural rock outcropping at that location, but may be caused by other factors. Borings at bridge locations indicate 40 to 100 ft of sand to bedrock in the Kansas City Reach (MoDOT 2008).

2.2 Sediment Loads

The sediment load in the Missouri River is quite variable, with high flows typically carrying exponentially more sediment than low flows. USACE (2017a) notes that the historic flood of 1993 brought about a downward shift in the flow-sediment relationship at the St. Joseph gage. The exact cause of this phenomenon is not known, but may be due to the deposition of bed material on the floodplains leading to less in-channel sediments available for transport (Horowitz 2006). This was a temporary phenomenon, however, and not a trend. As seen in Figure 3, sediment loads after 1993 were within the scatter of the sediment loads from 1952 – 1992. During the 2011 flood, the flow-load relationship temporarily decreased, which can be explained by the high volumes of clear water which came from the upstream reservoirs. It appears that for low flows, the post-2011 sediment levels are lower than pre-flood (but within the pre-1993 scatter), but for moderate flows, the flow-load relationship has returned to pre-2011 levels.

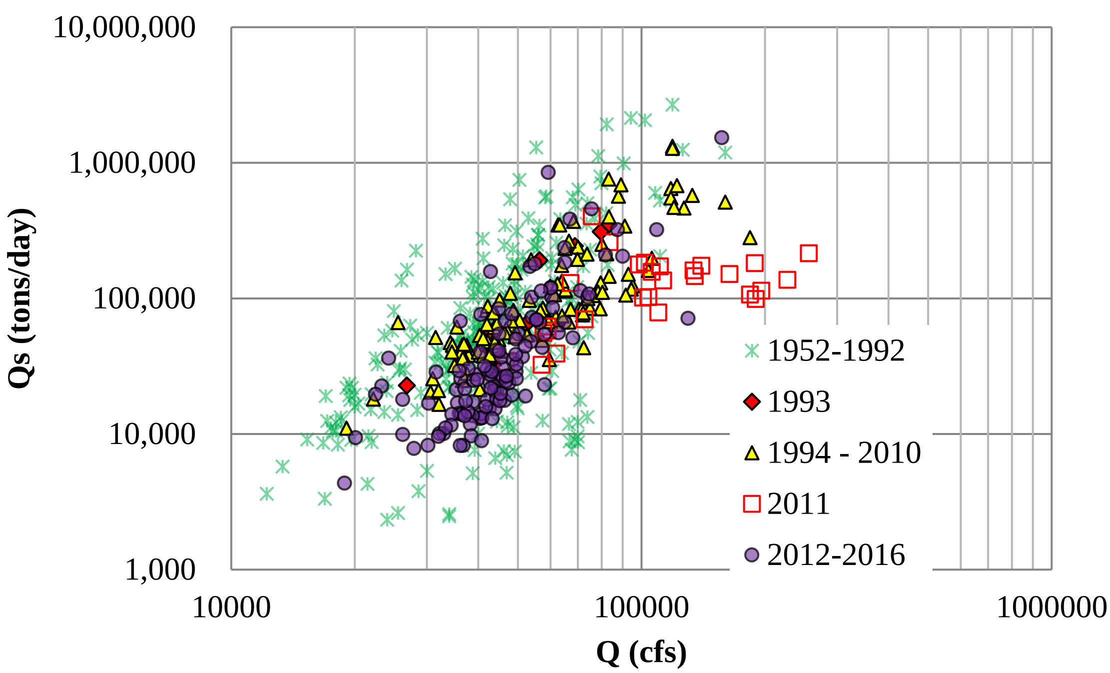


Figure 3. Sediment load at the St. Joseph, MO gage.

2.3 Commercial Dredging

The bed of the Missouri River is dredged (mined) downstream of Rulo, Nebraska as a source of sand and gravel. Commercial dredging on the Missouri River has taken place for many years to varying amounts. Historically, most of the extraction from the bed of the Missouri River took place in the Kansas City metro area, with additional operations in St. Joseph, Waverly, Jefferson City, and St.

Charles. Figure 4 shows the extracted quantities for the Missouri River from 1935 to 2016. As seen, the total dredging take began increasing in the 1950s. It increased sharply in the early 90s and remained high through the 2000s. This sharp increase was a result of regulatory restrictions on dredging on the Kansas River and increased local demand for construction materials. The level of dredging in 2002 includes USACE dredging for the construction of the L-385 unit of the Federal Missouri River Levee System. The annual extraction began falling in the late 2000s and is now around 4 million tons per year. Commercial sand and gravel dredging removed a total of approximately 247 million tons (181 million yd^3) from the bed of the lower 500 miles of Missouri River from 1935 to 2016. USACE (2017a) concludes that commercial sand and gravel dredging was the dominant cause of bed degradation from St. Joseph to Waverly, MO from 1994 to 2014.

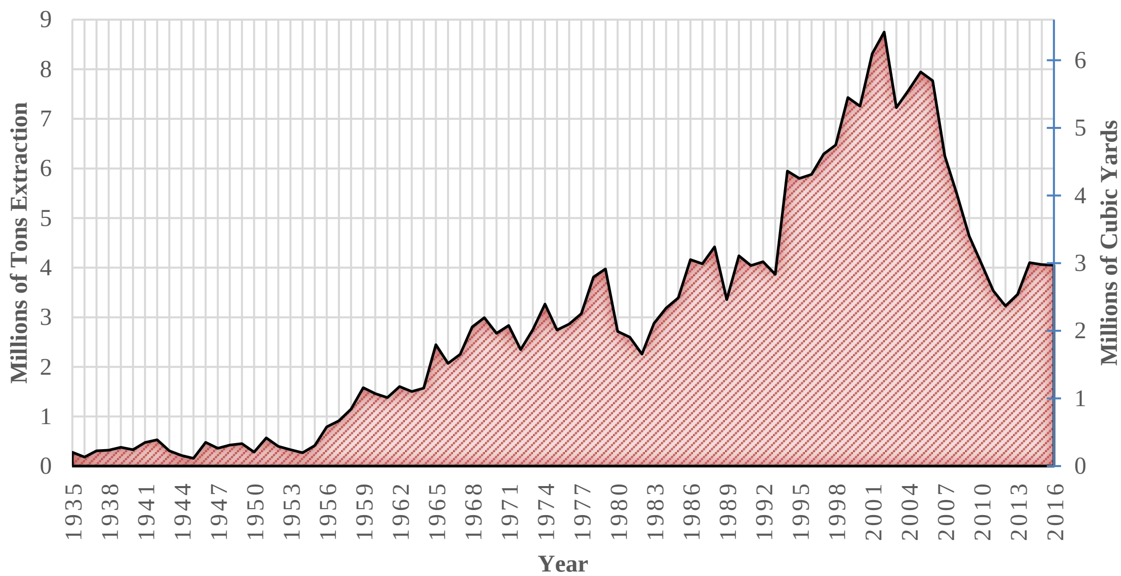


Figure 4. Extracted Dredging Quantities for the Missouri River in Kansas City

2.4 Stage and Low Water Surface Profile Degradation

The lower Missouri River has experienced significant bed degradation – persistent lowering of the river bed (USACE 2017a, 2011, 2009). Bed degradation of the Missouri River has caused a corresponding though not necessarily equivalent drop in the water surface elevations for low discharges. Figures 5, 6, 8, and 9 demonstrate that the stage of low discharges has been dropping at St. Joseph, Kansas City, Boonville, and Hermann gages, respectively (USACE 2017b). At the Waverly gage (Figure 7) the low stages have been relatively stable.

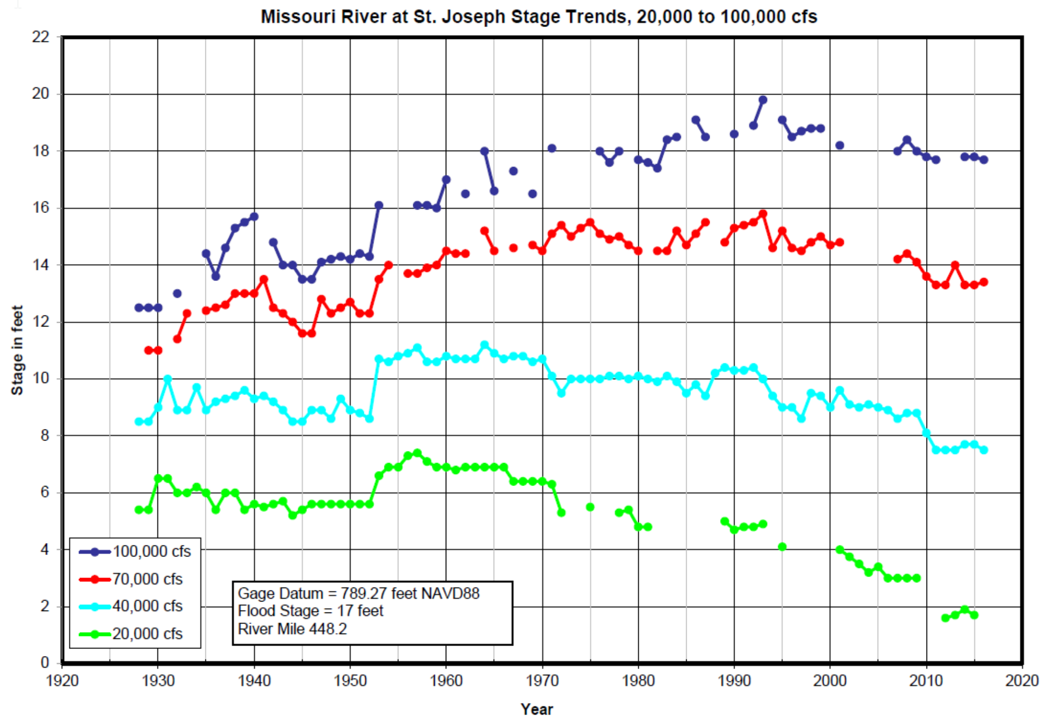


Figure 5. Missouri River Stage Trends- Missouri River at St. Joseph, MO

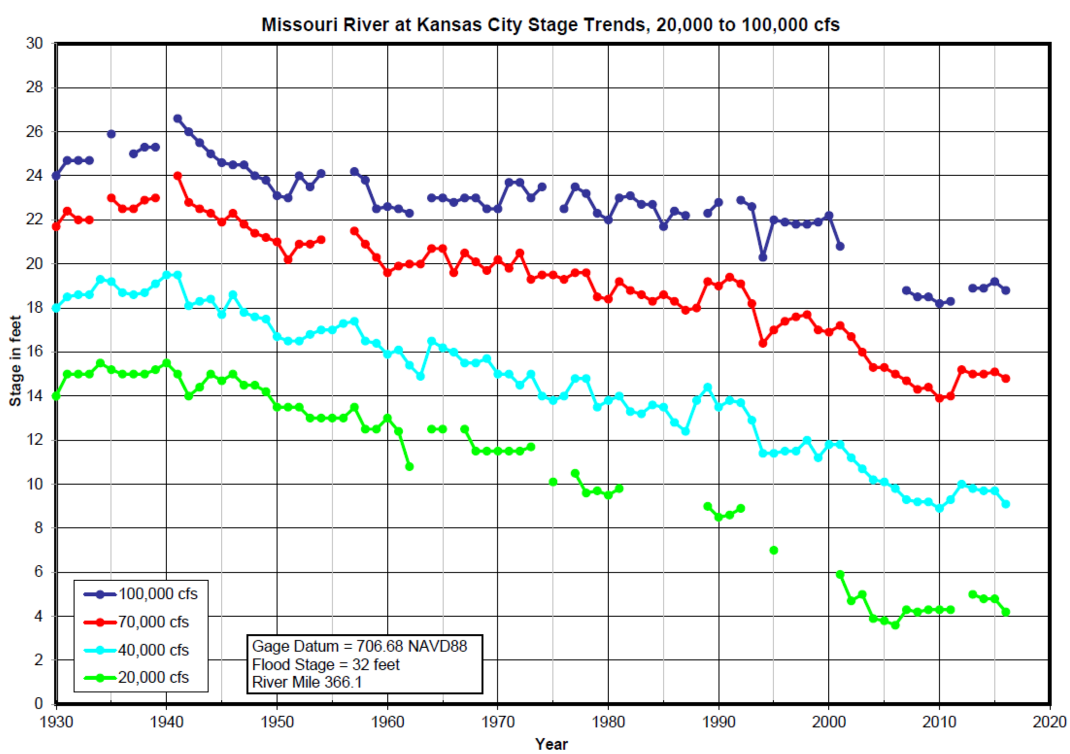


Figure 6. Missouri River Stage Trends- Missouri River at Kansas City, MO

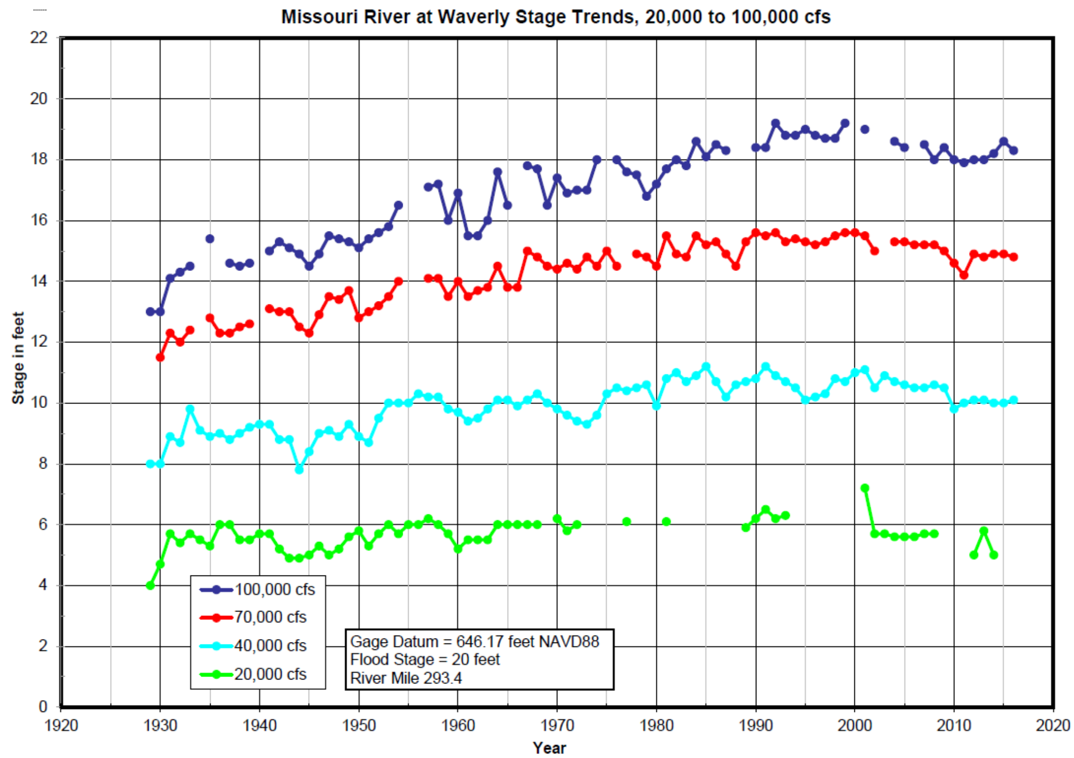


Figure 7. Missouri River Stage Trends- Missouri River at Waverly, MO

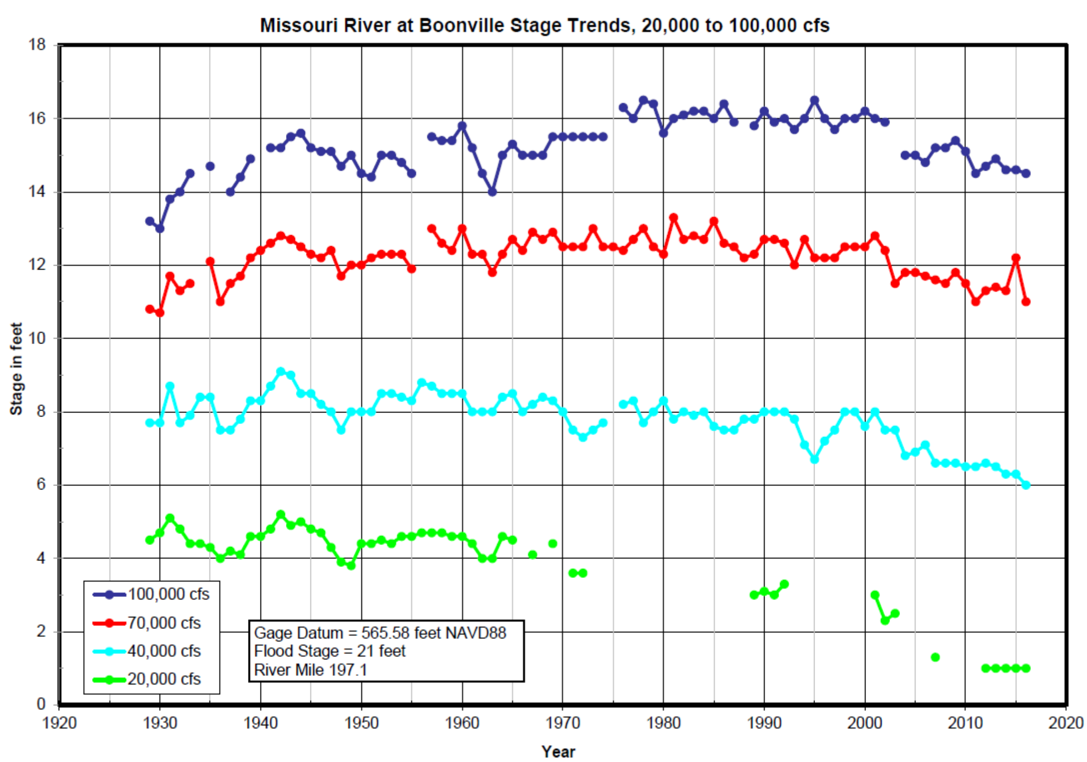


Figure 8. Missouri River Stage Trends- Missouri River at Boonville, MO

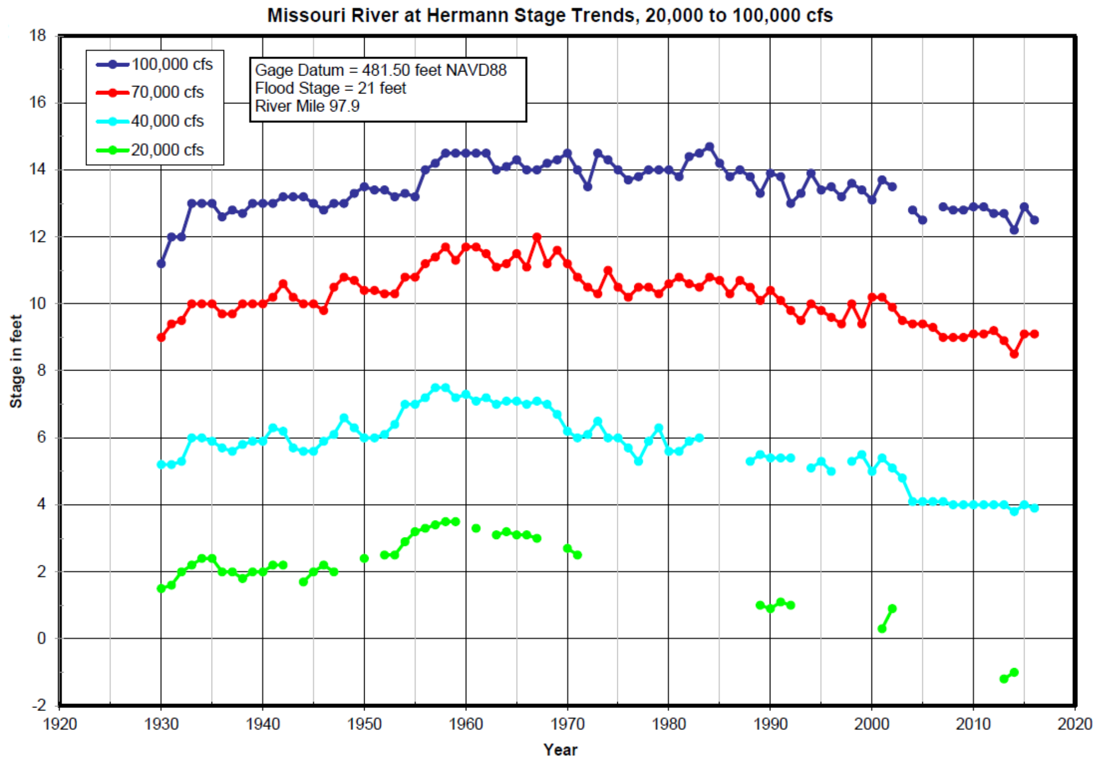


Figure 9. Missouri River Stage Trends- Missouri River at Hermann, MO

On the lower Missouri River, water surface elevations at dozens of locations have been measured on an annual or biannual basis for decades, which provided a way to track stage trends over time for the full lower 500 miles. These low water profiles were measured when flow rates were within a tight range, and then were adjusted to a consistent discharge based on rating curves at nearby gages to allow valid comparison (USACE 2010). Figure 10 plots selected profiles from 1974 to 2017 as a change compared to the average slope of the river. The average slope is defined by the starting and ending elevations of the 2010 Construction Reference Plane.

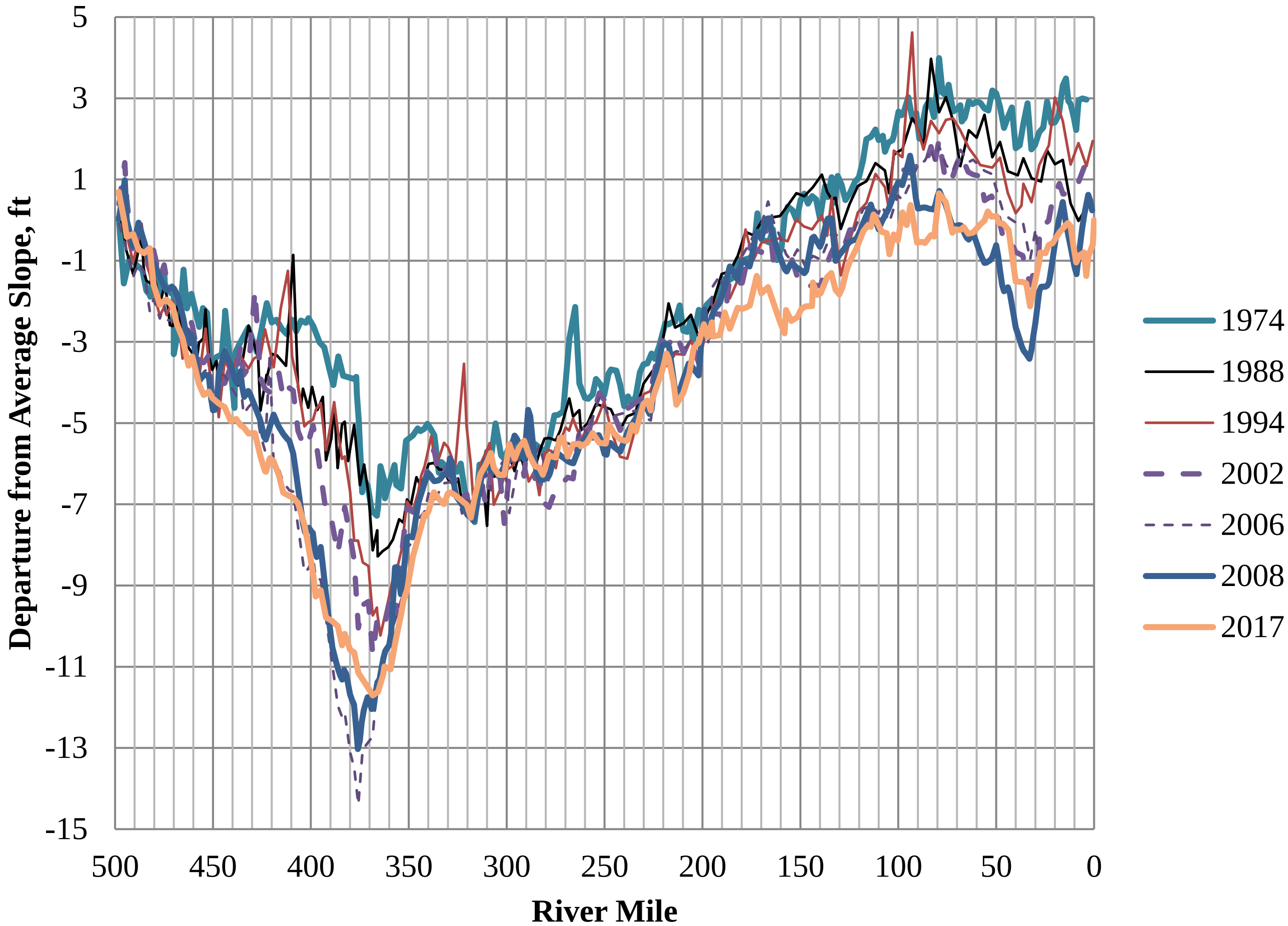


Figure 10. Low Water Surface Profiles—Departure from Average Slope

The rates of low water surface degradation vary considerably over the 500 miles. In some stretches of the river, degradation is insignificant, while in other areas, degradation has already induced damage to the BSNP and federal levee system, and necessitated expensive repairs and retrofits of other public and private infrastructure (USACE 2017a). As seen in Figure 10, areas of localized depression existed as of 1974 in the Kansas City metro area (RM 350 to 380). Very significant degradation has occurred since 1974, especially RM 450 to 320 and 200 to 0.

2.5 Bed Elevation and Volume Changes

Figure 11 presents the average bed elevations for key survey years, averaged over 5-mile reaches. As in Figure 10, the elevations are spatially de-trended (i.e. what is plotted is the departure from the average river slope) to allow easier visual comparison among years. The average river slope is computed by drawing a linear trend line from the average elevation in the most upstream 5 miles to the average elevation in the most downstream 5 miles.

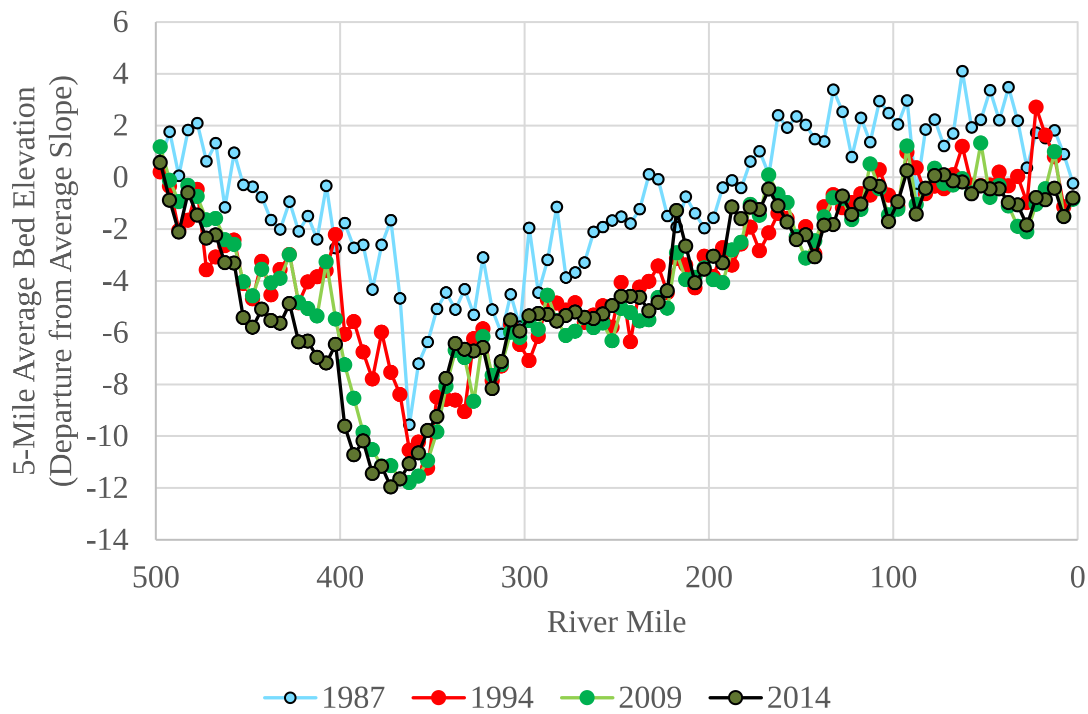


Figure 11. Spatially De-trended Average Bed Elevations (5-Mile-Average Departure from Average Slope)

As seen in Figure 11, very significant bed lowering occurred between the 1987 and 1994 surveys, which can be attributed predominantly to the effects off the 1993 flood. After 1994, the bed continued to degrade from aprox. RM 360 to 450. Numerical modeling in USACE (2017a) indicates that further degradation after 1994 from RM 360 to 390 was induced by commercial dredging and that this degradation migrated upstream as a result of the 2011 flood. The scatter precludes visual identification of trends downstream of RM 360 using Figure 11. These trends can be seen in the longitudinal cumulative volume change curves between each consecutive set of surveys, as presented in Figure 12. On longitudinal cumulative volume change curves, a downward slope indicates degradation, while an upward slope indicates recovery.

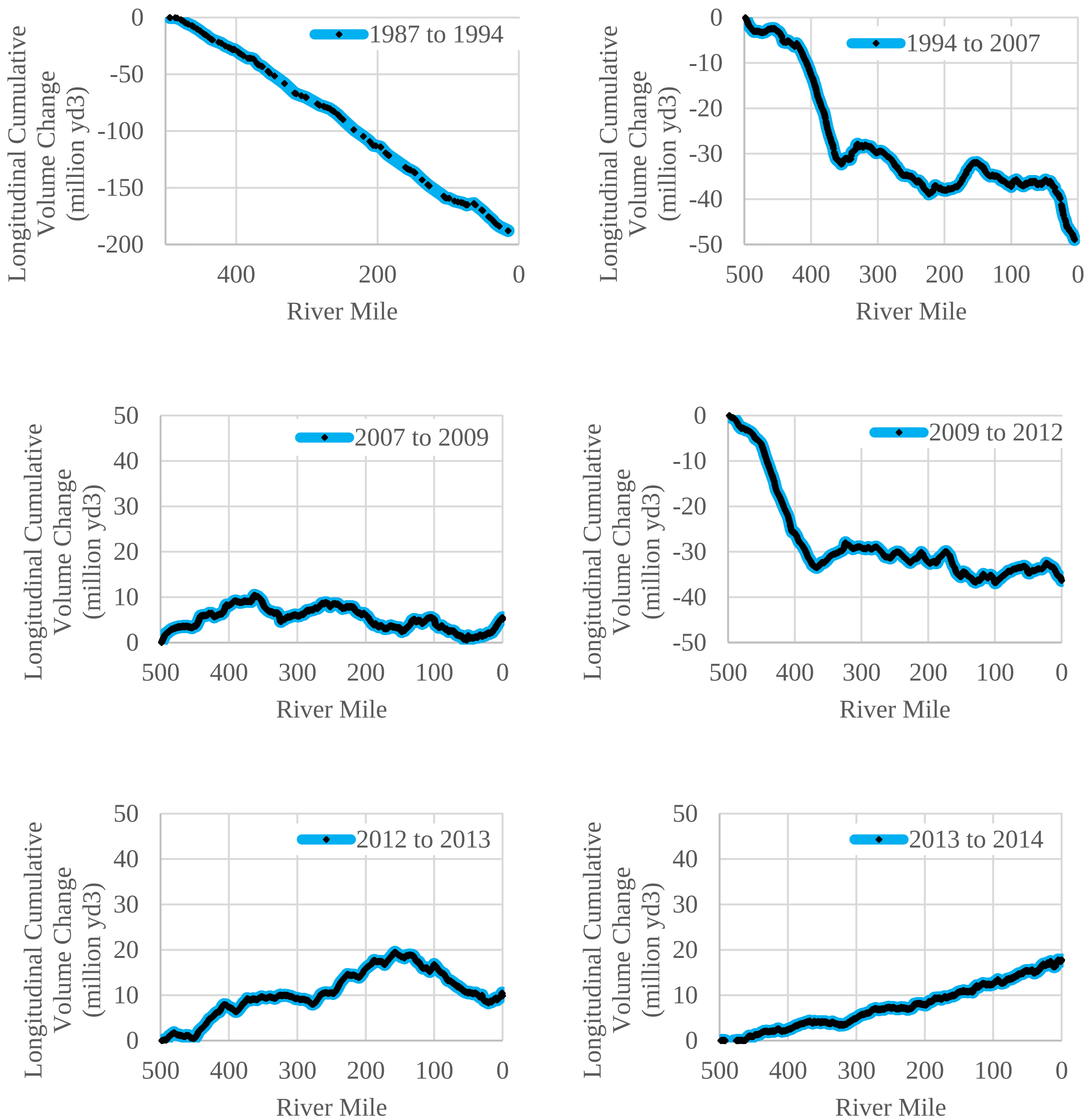


Figure 12. Longitudinal Cumulative Volume Change Between Successive River-Wide Surveys. Note the different scale for the 1987 to 1994 change.

The 1993 flood, which produced discharges of over 540,000 cfs in Kansas City (RM 366.2) caused tremendous degradation on the lower Missouri River. As seen in Figure 12, the Missouri River degraded 187.9 M yd^3 from 1987 to 1994. Commercial dredging over the same time period totaled 22.8 M yd^3, indicating that 165.1 M yd^3 was caused by an imbalance in sedimentation processes (i.e. the sediment leaving the lower Missouri River exceeded the sediment entering). As discussed later in this report, this sediment most likely deposited on the floodplain.

Following the 1993 flood, from 1994 to 2007 the river degraded an additional 48.9 million yd^3. Channel mining during this time period totaled 69.5 million yd^3, indicating that the channel mining prevented what would have been a bed recovery trend of 1.6 million yd^3 / year. The 2007 to 2009 analysis indicates localized degradation and aggradation with overall aggradation of 5.1 million yd^3.

From 2009 to 2012, the river degraded an additional 36.4 million yd^3, principally as a result of the 2011 flood. In Kansas City during the 2011 flood, the flow remained above 142,000 cfs (a 2-year flow) for over 100 days, which was approximately 40 days longer than the record flood of 1993. Upstream of RM 367, the degradation profile from the 2011 flood closely matches that of the 1993 flood. Downstream of RM 367, the bed degraded during the 2011 flood, but not nearly as much as in 1993.

Following the 2011 flood, from 2012 to 2013 the river recovered from RM 500 to 160 but continued to degrade from RKM 160 to 0, with a river-wide net recovery of 9.8 million yd^3. From 2013 to 2014 the river responded much more uniformly, recovering 17.8 million yd^3.

Overall, from 1987 to 2014, the bed of the river for the lower 500 miles of Missouri River degraded approximately 240 M yd^3, computed as the sum of the volume change between each successive set of surveys. Figure 13 indicates the components that sum to this quantity of total bed degradation. The "1993 Flood" component is the bed change seen from 1987 to 1994 minus the dredging over the same time period. The "2011 Flood" component is the bed change from 2009 to 2012 minus the dredging over the same time period. The dredging component is the sum of reported dredging tonnages from 1987 to 2014, converted to a volume. The "Natural Recovery Rate" component was found by subtraction and represents the level of rebound that could have occurred over this time period without direct removal of bed sediment via dredging. These volumes are specific to the 1987 to 2014 time period, which are not necessarily reflective of future bed change or average natural sediment recovery rates.

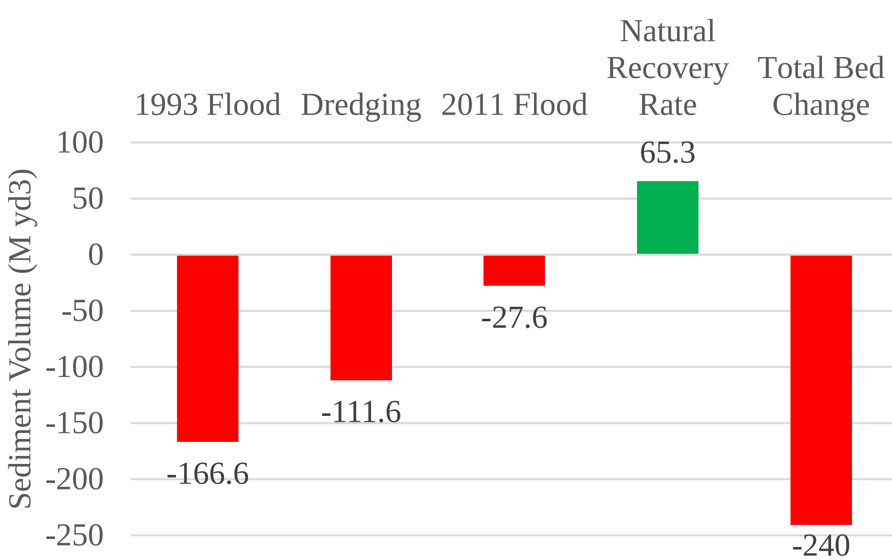


Figure 13. Components of Bed Change from 1987 to 2014

3.0 Model

3.1 Model Introduction

A HEC-RAS 5.0.3 sediment model was developed to predict differences in future bed elevation trends among alternatives. The model runs from RM 498.1, near Rulo, NE to RM 0.74 near the confluence with the Mississippi River. There are 801 cross sections with median spacing of 3354 ft (ranging from 671 ft to 6921 ft.) This resolution allows testing of reach-scale effects.

3.2 Model Schematic

Figure 14 provides a schematic of the model network with river miles, major tributaries, channel cross-sections, and USGS gages located. Due to the length of the river modeled, a detailed mapping of all pertinent features including levees, floodwalls, and river training structures for the full 500 river miles is not provided here. The reader is advised to review the Missouri River Hydrographic Survey Mapbook (USACE 2004).

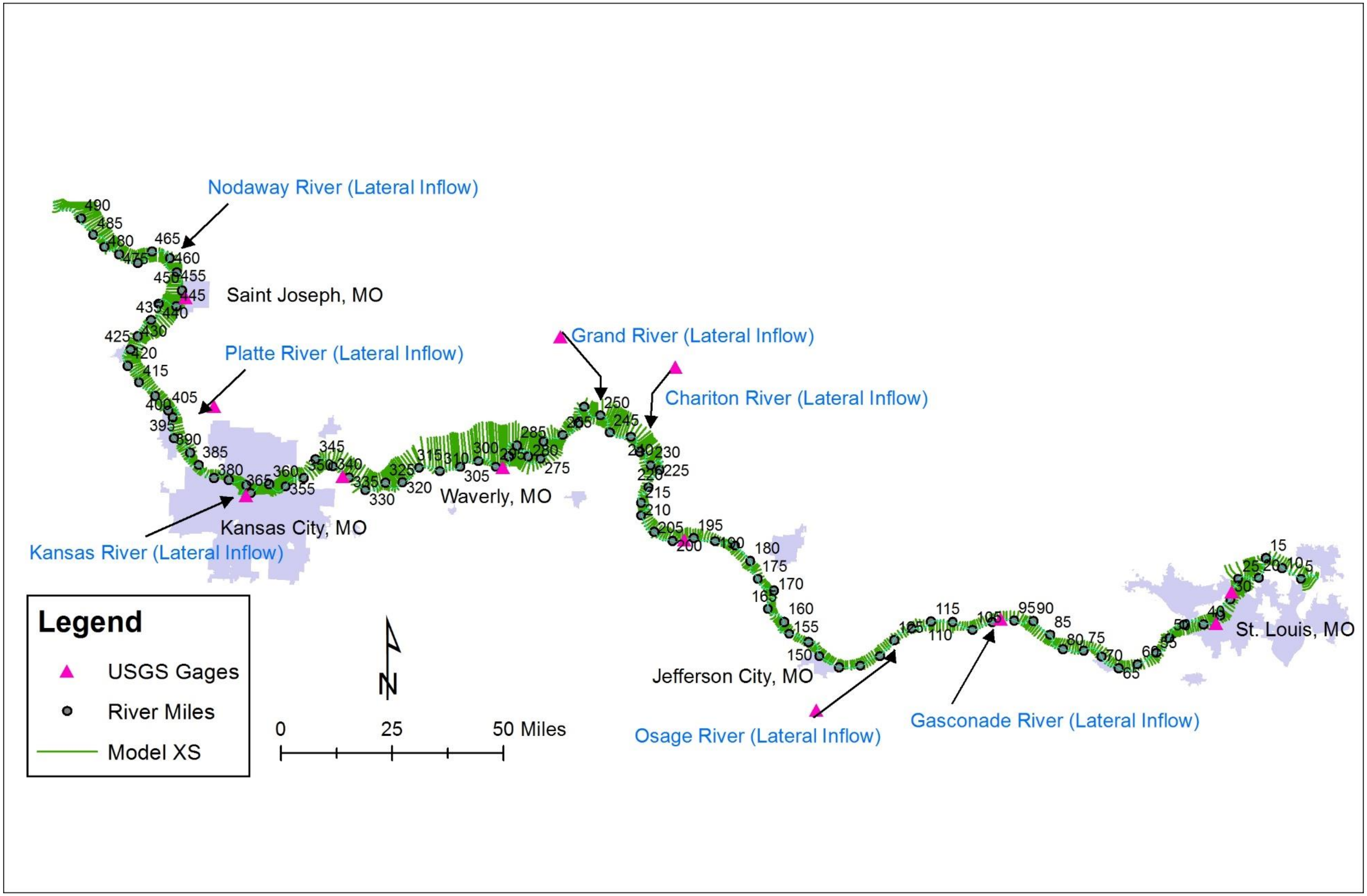


Figure 14. Model Schematic

3.3 Initial Conditions

Initial channel conditions consist of cross-sections, roughness values, and lateral extent, depth, and gradation of the bed. These initial conditions are described in the following paragraphs.

Cross-Sections

The starting geometry was synthesized using bed bathymetry from the 1994 hydrographic survey data (USACE 1994) and bank and overbank data from 2013 LIDAR. The model cross sections were chosen to generally match the locations of the floodway model, with consideration for the locations of the 1994 data. The cross-sections utilize the Missouri River 1960 river mile nomenclature. However, reach lengths are based on the actual channel distance between cross-sections along the sailing line, which varies slightly from using a difference in river miles to compute lengths. The starting year of 1994 was chosen due to data availability; a full hydrographic survey was conducted in 1994 which documented bed elevations as well as dike and revetment geometry. Not all cross-sections present in the 2007 floodway model were retained in the degradation model. Cross-sections with unrealistically small or large cross-sectional areas, cross-sections that were too tightly spaced and bridge cross-sections were removed to achieve model stability.

Dike, sill, and revetment structures were entered into the cross-sections as station/elevation points to account for the blocked flow area between dike structures. The methodology is similar, but more robust, to that used on the Missouri River by Teal and Remus (2001) and in USACE (2017a). Conceptually, the ineffective flow areas within each control volume are summed, then divided by the control volume length in order to compute the blocked area to be entered into each cross section. Figure 15 conceptually displays the process. Figure 16 displays actual computed data for the same river location.

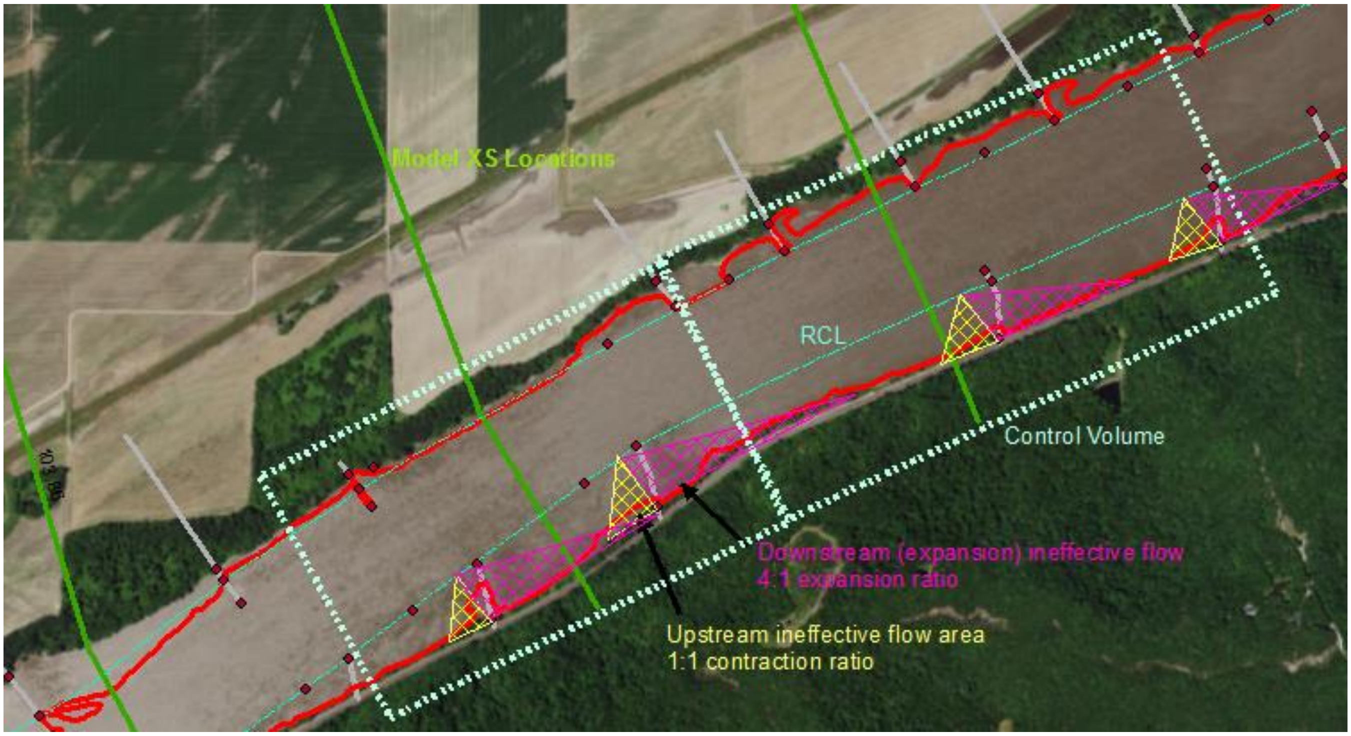


Figure 15. Conceptual depiction of ineffective flow areas from dike structures

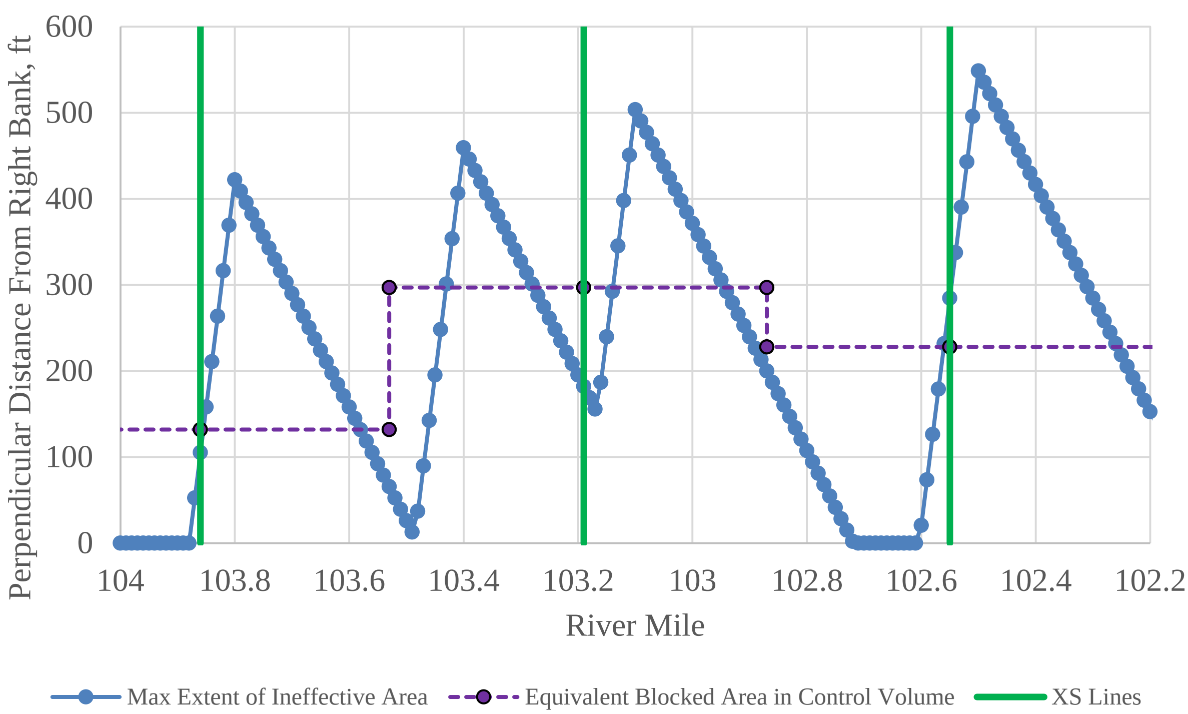


Figure 16. Computation for effective dike length

The following steps summarize the process:

1. Structures in the GIS layer representing dikes and sills were assigned to either the right or left bank.
2. The perpendicular length of each dike structure was found using GIS. This was accomplished by computing the intersection point of each dike structure with a GIS layer representing the low bank, then finding the shortest distance between that intersection point and the Rectified Channel Line. Distances for locations where a dike intersected a bank more than once and other anomalies were manually measured in ArcMap or GoogleEarth.
3. The perpendicular length of each sill structure was assumed equal to the GIS line distance for the portion of structure riverward of the Rectified Channel Lines.
4. The remaining steps were executed four times. Once for the dike structures (structure length as described in step 2) and once for the sill structure (structure length = dike length as in step 2 + the sill length as in step 3). Then repeated twice more for the dikes and sills on the opposite bank.
5. For each hundredth-mile increment from RM 497.7 to RM 0.49, the maximum riverward extent of the ineffective flow area of the bounding dikes was calculated. The zone of expansion was assumed to be 4:1 on the downstream end of each dike. The zone of contraction was assumed to be 1:1 on the upstream and of each dike. The closest bounding dikes did not necessarily generated the most riverward extent of ineffective flow; closer, shorter dikes were at times in the shadow of slightly more distant, longer dikes. To remedy, the ineffective flow expansion/contraction line for four dikes upstream and two dikes downstream of each increment were considered and the maximum length of ineffective flow selected. See Figure 16.
6. The total ineffective flow area was computed for each control volume, then divided by the length of the control volume to yield an effective dike length.
7. This effective dike length was entered into the HEC-RAS model as a blocked obstruction then converted to sta/elev points.

Structures extending from the channel bank to the Rectified Channel Line were assigned the elevation criteria for a dike (the average of the concave and convex design elevations). Structures from the rectified channel line and further into the channel were assigned the design criteria for a sill. These criteria are provided in Table 1. The 1982 CRP, which was the official CRP in use in 1994, was used to set the structure elevations. L-head revetment heights were read individually from the 1994 Missouri River Hydrographic Survey Mapbook (USACE 1994).

Table 1. Dike and Sill Elevations in Model

River Mile Range	Offset from 1982 CRP (ft)			
	Concave Dike Criteria	Convex Dike Criteria	Model Dike Elevation	Sill (Criteria and Model)
498 to 367	+3	+1	+2	-2
367 to 250	+3	+1	+2	-2
250 to 130	+4	+2	+3	-2
130 to 0	+5	+3	+4	-1

As this is a quasi-unsteady, not truly unsteady flow model, levee breaches were not modeled. As RAS allows only one levee point in each overbank, smaller levees were typically included as ineffective flow areas and larger levees as levee points. The levees were placed to maintain an accurate distance between the river bank and the levee, as measured in GIS from the National Levee Database shapefile. The simplified treatment of levees inherent to a quasi-unsteady flow model limits the ability of this model to predict flood heights or floodplain deposition during extreme (levee overtopping) events.

In quasi-unsteady flow modeling, a levee that is overtopped instantly and fully contributes to the flow (i.e. there is no time factor for filling and draining). Where this was problematic, additional permanent ineffective flows were added. Ineffective flows were also included to bridge over chutes present in the LIDAR that were not constructed until later in the calibration period and to fill in the area behind L-head revetments.

Roughness

Channel roughness was assigned as Manning ‘n’ values in four horizontally-varied regions: the active channel (n = 0.028), the channel with sill influence (n = 0.041), the channel with dike influence (n = 0.0413), and the floodplain (n = 0.07). These regions are delineated in Figure 17. Variations in roughness among large reaches were included in the flow-roughness change factors, listed in Table 2, rather than in the base level ‘n’ values. Lisbon chute began flowing in 1996 and was assigned an ‘n’ value of 0.05. Cranberry bend was present prior to 1994 and was assigned an n value of 0.029. The remainder of the chutes were not constructed until late in the calibration period and were assigned the floodplain ‘n’ value of 0.07.

Measured water surfaces at the Kansas City gage indicate that at very high flows, the roughness for the active bed decreases as the bed transitions from dunes to plane bed. This was physically verified using multi-beam bathymetric surveys, as documented in USACE (2017a). Table 2 presents the flow-roughness values used in the model.

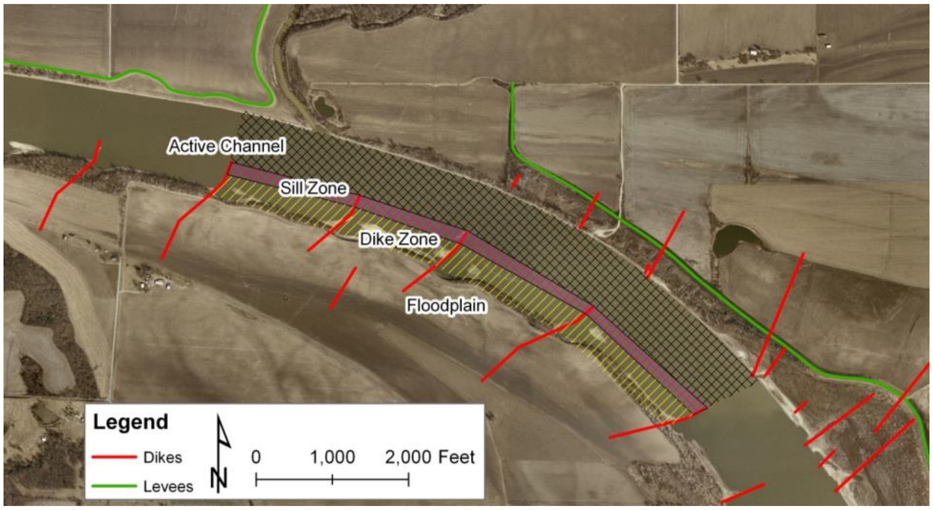


Figure 17. Active Channel and Inter-dike Regions

Table 2. Flow-Roughness Change Factors

RM 498.1 to 463.97	
Q (cfs)	Roughness Factor
0	1.05
70000	1.05
120000	1
150000	0.95
200000	0.92

RM 463.17 to 393.18	
Q (cfs)	Roughness Factor
0	1.09
50000	1.09
70000	1.05
120000	1
150000	0.95
200000	0.9

RM 392.59 to 367.89	
Q (cfs)	Roughness Factor
0	0.95
70000	0.95
120000	0.85
200000	0.8

RM 367.57 to 321.11	
Q (cfs)	Roughness Factor
0	1
50000	1
70000	0.98
120000	0.98
150000	0.95
200000	0.9

RM 320.42 to 250.85	
Q (cfs)	Roughness Factor
0	1.02
50000	1.03
120000	1.03
150000	1.02

RM 250.23 to 169.13	
Q (cfs)	Roughness Factor
0	0.9
70000	0.9
120000	0.9
150000	0.85

RM 62.92 to 0.774	
Q (cfs)	Roughness Factor
0	1
70000	1
120000	0.95
150000	0.95

Tributaries

Seven major tributaries enter the Missouri River in the model reach: the Nodaway River (RM 463.17), Platte River (RM 391.29), Kansas River (RM 367.57), Grand River (RM 250.23), Chariton River (239.32), Osage River (RM 130.37), and Gasconade River (104.49). Over the calibration period (1994 – 2014) the combined flow inputs from these tributaries totals aprx. 31% of the flow in the Missouri River at St. Charles, MO. These tributaries were included as flow and sediment boundary conditions. As explained later, the differences in flow and sediment at the mainstem gages beyond those explicitly specified are included as uniform lateral flows. Thus the flow and sediment inputs from other tributaries such as the Big Nemaha, Lamine / Blackwater, etc. are included in these uniform lateral flow and sediment inputs.

Bed Sediment Extent

The moveable bed limits were initially set at the toe of the most riverward structure (dike or sill). These limits were adjusted as needed for model stability or calibration. The depth of the erodible bed was set to 40 ft, which is beyond the limits of degradation expected to occur over the next 50 years. A river-wide sub-surface investigation has not been performed to accurately locate bedrock in the active channel, but specific borings near bridges indicate 40 to 100 ft of sand in Kansas City.

Bed Sediment Gradation

Bed sediments were sampled at 5-mile increments in 1994 over the entire model reach. The model bed sediment was a weighted average of 50% of the individual sediment sample and 50% of the reach-average sediment sample. The average reaches were defined as follows: RM 500 to 391 (Rulo to Platte River), RM 391 to 250 (Platte River to Grand River), RM 250 to 130 (Grand River to Osage River), and RM 130 to 0 (Osage River to the mouth). Table 3 presents the original, 1994 data (interpolated to standard sizes) compared to the model bed gradations.

Table 3. Initial and Final Model Gradations, Percent Finer (Rounded to Whole Numbers)

RM	Original Data									Model Data								
	Diameter (mm)									Diameter (mm)								
	0.125	**0.25**	**0.5**	**1**	**2**	**4**	**8**	**16**	**32**	**0.125**	**0.25**	**0.5**	**1**	**2**	**4**	**8**	**16**	**32**
1	1	12	49	77	89	94	98	100	100	1	12	49	77	89	94	98	100	100
5	0	25	59	79	90	96	99	100	100	0	25	58	79	90	96	99	100	100
10	1	9	46	76	90	97	99	100	100	1	10	46	76	90	96	99	100	100
15	1	9	38	71	88	95	98	100	100	1	9	39	71	88	95	98	100	100
20	1	22	57	78	90	96	99	100	100	1	22	56	78	90	96	99	100	100
25	2	31	74	84	91	96	99	100	100	2	30	72	84	91	96	99	100	100
30	1	20	61	83	94	98	99	100	100	1	20	61	83	93	98	99	100	100
35	1	24	54	74	89	97	99	100	100	1	23	54	74	89	97	99	100	100
40	0	9	37	62	82	94	98	100	100	0	9	37	63	82	94	98	100	100
45	1	10	35	60	80	93	98	100	100	1	10	36	61	81	93	98	100	100

RM	Original Data									Model Data								
	Diameter (mm)									Diameter (mm)								
	0.125	**0.25**	**0.5**	**1**	**2**	**4**	**8**	**16**	**32**	**0.125**	**0.25**	**0.5**	**1**	**2**	**4**	**8**	**16**	**32**
50	0	8	39	72	90	97	99	100	100	0	8	40	72	90	97	99	100	100
55	1	9	36	63	80	91	97	100	100	1	9	37	63	81	92	97	100	100
60	0	17	55	74	89	97	99	100	100	0	17	54	74	89	97	99	100	100
65	1	23	53	75	89	94	96	99	100	1	23	53	75	88	94	97	99	100
70	0	8	36	65	84	93	97	99	100	0	9	36	65	84	93	97	100	100
75	1	12	40	61	75	86	95	99	100	1	12	40	61	76	87	95	99	100
80	1	15	42	71	87	92	96	98	100	1	15	42	71	87	93	96	98	100
85	1	18	52	77	92	98	100	100	100	1	17	52	77	91	98	100	100	100
90	1	14	49	66	81	92	98	100	100	1	14	49	66	81	92	98	100	100
95	1	9	44	72	89	97	100	100	100	1	9	44	72	89	97	100	100	100
100	1	14	45	79	94	98	100	100	100	1	14	45	78	94	98	100	100	100
105	1	29	66	86	95	99	100	100	100	1	29	66	85	95	99	100	100	100
110	3	22	64	84	92	97	99	100	100	3	22	64	83	92	97	99	100	100
115	0	5	36	65	81	93	98	100	100	0	6	37	65	81	93	98	100	100
120	0	12	50	77	89	95	98	100	100	1	12	50	76	89	95	98	100	100
125	1	24	54	72	85	94	99	100	100	1	24	54	72	85	94	99	100	100
130	1	16	62	86	95	98	99	100	100	1	16	62	86	94	98	99	100	100
135	1	12	48	67	79	91	98	100	100	1	12	49	68	80	92	98	100	100
140	1	15	54	85	96	99	100	100	100	1	15	55	85	96	99	100	100	100
145	3	26	62	77	87	94	98	100	100	3	26	62	78	87	94	98	100	100
150	1	28	67	84	94	97	98	100	100	1	28	67	84	94	97	98	100	100
155	6	47	70	82	92	98	100	100	100	5	46	70	83	93	98	100	100	100
160	1	31	74	85	91	95	98	100	100	1	30	73	85	91	95	98	100	100
165	2	31	70	86	93	97	99	100	100	2	31	69	86	93	97	99	100	100
170	1	9	62	88	96	99	100	100	100	1	10	62	88	96	99	100	100	100
175	1	12	54	79	91	97	99	100	100	1	13	55	79	91	97	99	100	100
180	1	23	74	91	97	99	100	100	100	1	23	73	91	97	99	100	100	100
185	2	27	62	91	99	100	100	100	100	2	27	62	91	99	100	100	100	100
190	1	24	66	85	93	98	100	100	100	1	24	66	85	93	98	100	100	100
195	1	14	51	75	88	95	99	100	100	1	15	52	76	89	96	99	100	100
196.6	0	13	43	76	92	97	99	100	100	0	14	44	76	92	97	99	100	100
200	1	25	67	85	95	99	100	100	100	1	25	67	85	95	99	100	100	100
205	0	14	55	81	93	97	99	100	100	0	15	55	81	93	97	99	100	100
210	1	15	61	85	96	99	100	100	100	1	16	62	85	96	99	100	100	100
215	2	35	75	93	99	100	100	100	100	2	34	74	93	98	100	100	100	100
220	1	17	70	93	98	99	100	100	100	1	17	70	93	98	99	100	100	100
225	1	25	69	93	99	99	99	100	100	1	25	69	93	98	99	99	100	100
230	2	36	75	90	97	99	100	100	100	2	35	74	90	96	99	100	100	100
235	1	33	76	95	100	100	100	100	100	1	33	76	95	99	100	100	100	100
240	1	28	59	76	89	96	99	100	100	1	28	59	76	89	97	99	100	100
245	1	29	72	87	94	98	99	100	100	1	29	72	87	94	98	99	100	100
250	1	22	70	92	98	99	100	100	100	1	22	70	91	97	99	100	100	100
255	1	35	80	96	99	100	100	100	100	1	34	79	95	99	100	100	100	100
260	1	29	71	88	94	97	99	100	100	1	29	71	88	94	97	99	100	100

RM	Original Data									Model Data								
	Diameter (mm)									Diameter (mm)								
	0.125	**0.25**	**0.5**	**1**	**2**	**4**	**8**	**16**	**32**	**0.125**	**0.25**	**0.5**	**1**	**2**	**4**	**8**	**16**	**32**
265	2	39	83	95	98	100	100	100	100	2	38	83	94	98	99	100	100	100
270	1	37	87	96	98	98	99	100	100	1	36	86	95	98	98	99	100	100
275	2	38	69	85	93	97	98	100	100	2	37	69	85	93	97	98	100	100
280	2	47	85	97	99	100	100	100	100	2	46	85	97	99	100	100	100	100
285	1	30	80	95	98	99	100	100	100	1	30	79	94	98	99	100	100	100
290	1	28	62	81	95	99	100	100	100	1	28	62	82	95	99	100	100	100
295	1	25	69	90	98	100	100	100	100	1	25	69	90	98	100	100	100	100
300	1	19	59	83	94	98	99	100	100	1	19	60	84	94	98	99	100	100
305	1	21	52	76	91	97	99	100	100	1	21	53	77	92	97	99	100	100
310	1	36	78	95	99	100	100	100	100	1	36	77	95	99	100	100	100	100
315	9	28	76	92	97	99	100	100	100	8	28	76	92	97	99	100	100	100
320	1	12	57	83	91	93	95	95	99	1	12	58	83	91	94	95	95	100
325	1	19	68	91	97	99	100	100	100	1	19	68	91	97	99	100	100	100
330	1	20	49	79	93	97	98	100	100	1	20	50	79	93	97	98	100	100
335	1	20	53	76	90	97	100	100	100	1	20	53	77	90	97	100	100	100
340	2	27	81	96	99	100	100	100	100	2	27	81	95	99	100	100	100	100
345	1	13	63	89	96	98	99	100	100	1	13	63	89	96	98	99	100	100
355	2	36	71	89	94	96	98	100	100	2	36	71	89	95	96	98	100	100
360	1	39	82	93	97	99	100	100	100	1	38	81	93	97	99	100	100	100
365	2	26	53	75	89	96	99	100	100	2	26	54	76	90	96	99	100	100
370	0	17	68	89	96	98	99	100	100	1	17	68	89	96	98	99	100	100
375	1	36	78	95	99	100	100	100	100	1	35	78	95	99	100	100	100	100
380	1	21	60	82	93	98	99	100	100	1	22	60	82	93	98	99	100	100
385	1	22	68	89	95	97	99	100	100	1	22	68	89	95	97	99	100	100
390	1	28	86	98	99	99	99	100	100	1	28	85	97	99	99	99	100	100
395	1	32	74	83	89	95	98	99	100	1	32	74	84	90	95	98	99	100
400	1	42	84	95	98	99	99	100	100	1	41	83	94	98	99	100	100	100
405	0	20	72	94	99	99	100	100	100	0	20	72	94	98	99	100	100	100
410	1	31	85	94	98	99	100	100	100	1	31	85	94	98	99	100	100	100
415	3	45	89	98	100	100	100	100	100	3	44	88	98	100	100	100	100	100
420	1	18	70	92	97	99	100	100	100	1	18	70	92	97	99	100	100	100
425	1	36	80	93	98	99	100	100	100	1	36	79	93	98	99	100	100	100
430	2	37	82	94	98	99	100	100	100	2	36	82	94	98	99	100	100	100
435	1	23	80	97	100	100	100	100	100	1	24	80	97	99	100	100	100	100
440	0	14	66	89	95	97	99	100	100	0	15	67	89	95	97	99	100	100
450	1	26	80	94	98	99	100	100	100	1	26	79	94	98	99	100	100	100
460	0	29	72	89	96	99	100	100	100	1	29	72	89	97	99	100	100	100
465	1	15	61	80	91	97	99	100	100	1	16	62	80	91	97	99	100	100
470	1	33	75	91	97	99	100	100	100	1	33	75	91	97	99	100	100	100
475	1	24	77	96	99	100	100	100	100	1	25	77	96	99	100	100	100	100
480	0	21	72	94	99	100	100	100	100	0	21	72	94	99	100	100	100	100
485	1	29	77	96	99	100	100	100	100	1	28	77	95	99	100	100	100	100
490	1	14	57	87	96	98	100	100	100	1	15	58	88	96	98	100	100	100
495	1	17	50	80	93	98	99	100	100	1	18	51	81	93	98	99	100	100

3.4 Boundary Conditions

Boundary conditions include flow, water temperature, downstream water surface elevation, incoming sediment load and gradation, floodplain deposition amounts, and dredging amounts, locations, and timing. These boundary conditions are described in the following paragraphs.

Flows

Daily flow values from Aug 1, 1994 – 29 July 2014, computed from seven mainstem Missouri River and seven tributary USGS gaging stations, were used as the flow inputs to the model.

Daily flow values were compiled for the following USGS gage stations listed in Table 4. The model upstream boundary was set to the daily flow reported by USGS for the Missouri River at Rulo, Nebraska. Each tributary was entered as a lateral flow. The difference between the mainstem Missouri River gages that could not be explained by the tributary flows were entered as uniform lateral flows. These uniform lateral flows account for ungagged inflows, totaling 19% of the total flow volume at St. Charles, MO. They also approximate the longitudinal change in the flow profile due to timing effects which are not modeled in quasi-unsteady flow modeling. This same approximation was utilized in USACE (2017a). Additional details regarding drainage area delineations for the watershed are included in the Unsteady HEC-RAS Model Calibration Report, Appendix E.

Scaled down versions (1/10,000) of these uniform lateral flows were entered in order to trigger the floodplain deposition rating curves at the appropriate times. The scaled-down uniform flows are very small and have negligible effect on actual model flows.

The flows from the downstream gage are used in the model as lateral flows and as input to the rating curves. At two gages, insufficient sediment data exists at the downstream gage, so the flow/load relationship was developed from a more upstream gage. These relationships were then used with flows from the downstream gages.

Table 4. USGS Gage Stations Used in the Model

USGS Gage #	Name	USGS Drainage Area (sq mi)
06813500	Missouri River at Rulo, NE	414,900
06818000	Missouri River at St. Joseph, MO	426,500
06893000	Missouri River at Kansas City, MO	484,100
06895500	Missouri River at Waverly, MO	485,900
06909000	Missouri River at Boonville, MO	500,700
06934500	Missouri River at Hermann, MO	522,500
06935965	Missouri River at St. Charles, MO	542,000
06817700	Nodaway River Near Graham, MO	1,520
06817000	Nodaway River at Clarinda, IA*	762

USGS Gage #	Name	USGS Drainage Area (sq mi)
06821190	Platte River at Sharps Station, MO	2,380
06892350	Kansas River at Desoto, KS	59,756
06902000	Grand River Near Sumner, MO	6,880
06905500	Chariton River Near Praire Hill, MO	1,870
06926510	Osage River below St. Thomas, MO	14,584
06934000	Gasconade River Near Rich Fountain, MO	3,180
06933500	Gasconade River Near Jerome, MO*	2,840

* Denotes a gage used for development of the flow/sediment rating curve but not used as the flow input.

Water Temperature

The HEC-RAS sediment model requires a water temperature for each day of the simulation to calculate fall velocity. The annual time series of water temperatures based on measurements at the Kansas City gage, as developed in USACE (2017a), was used in this model.

Downstream Water Surface Boundary Condition

The downstream water surface elevation was originally set to the water surface elevation computed by the Missouri River Recovery Program (MRRP) unsteady flow model which includes the backwater effect from the Mississippi River. However, on inspection, the model water surface output at this location would occasionally drop below the normal depth solution, which produced unreasonably high velocities and excessive scour. To decrease this unrealistic effect, a floor of 1 ft below the normal depth solution (computed from the initial geometry) was imposed. This maintains the backwater effects from the Mississippi River included in the MRRP unsteady flow model. Figure 18 indicates the downstream boundary as a function of the flow in the MRRP unsteady flow model at RS 0.73.

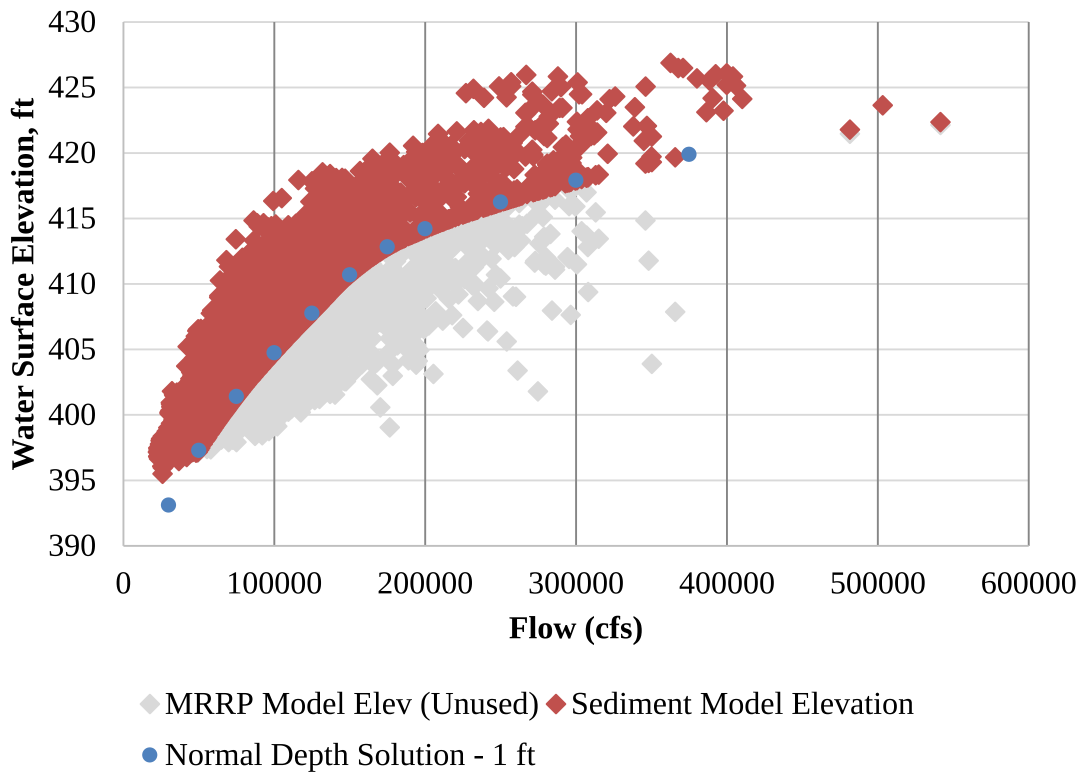


Figure 18. Downstream Boundary Condition

Sediment Load

The incoming sediment load and gradation was computed and entered into the model as a DSS time series for each grain class. The sediment load for a given grain class was computed as the total suspended sediment load multiplied by the percent of the suspended load corresponding to the grain class plus the total bed load multiplied by the percent of the bed load corresponding to that grain class.

Only sands and gravels were included in the model. Finer sediments are wash load in this system; they are not found in appreciably quantities on the bed and do not play a significant role in the physical bed change processes. Wash load causes a numerical artifact in HEC-RAS 5.0.3 and so was not included.

USGS Water Quality data at Saint Joseph, MO were used to develop the flow/load relationship and gradational breakdown of the suspended load. Overall, the suspended sediment load fines considerably with increasing discharge. Figure 17 demonstrates the average, calibrated total load relationships at Saint Joseph compared to USGS measurements for suspended load. (Note that the USGS report many suspended sediment samples that had no coarse sand, which could not be plotted in log space on Figure 19.) These loads computed for St. Joseph were reduced by the Nodaway sediment load in order to create the model upstream boundary condition.

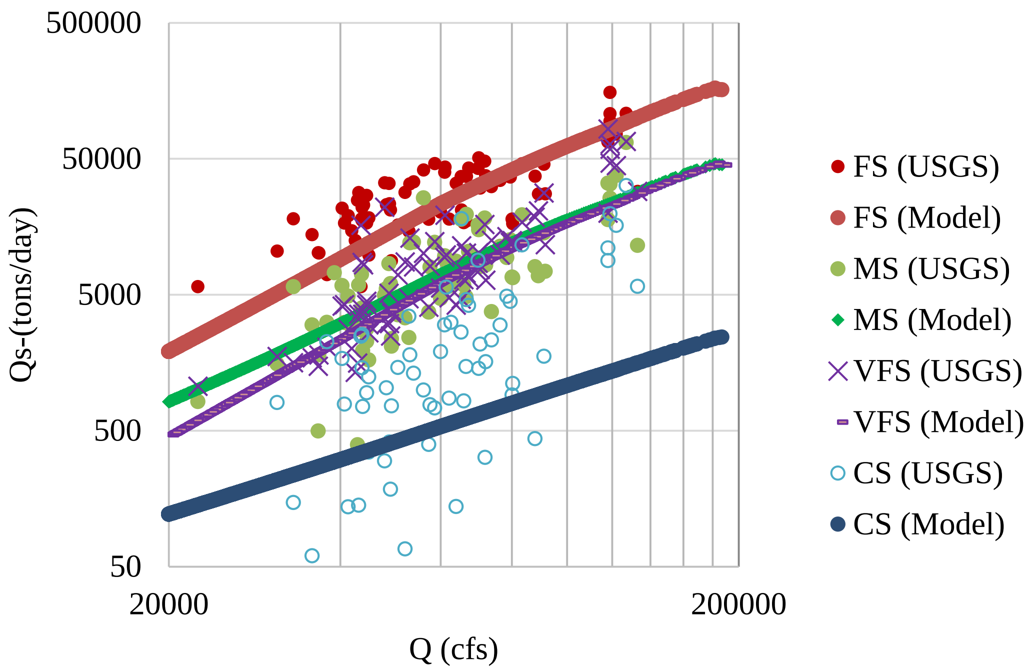


Figure 19. Average relationships at Saint Joseph used to develop upstream rating curve vs. measured suspended sand by grain class

During the 2011 flood, the high volumes of relatively clear water released from the dams resulted in a markedly different suspended sediment relationship. The suspended loads during the 2011 event was based on USGS measurements during the event rather than the long-term relationship shown above.

The bedload portion of the sediment load was computed from the bedload rating curve for St. Joseph presented in Abraham et al. (2017). This bedload rating curve was computed by using successive multi-beam bathymetric surveys (Abraham et al. 2011) with the time correction suggested in Shelley et al. (2013). This rating curve is provided in Figure 20.

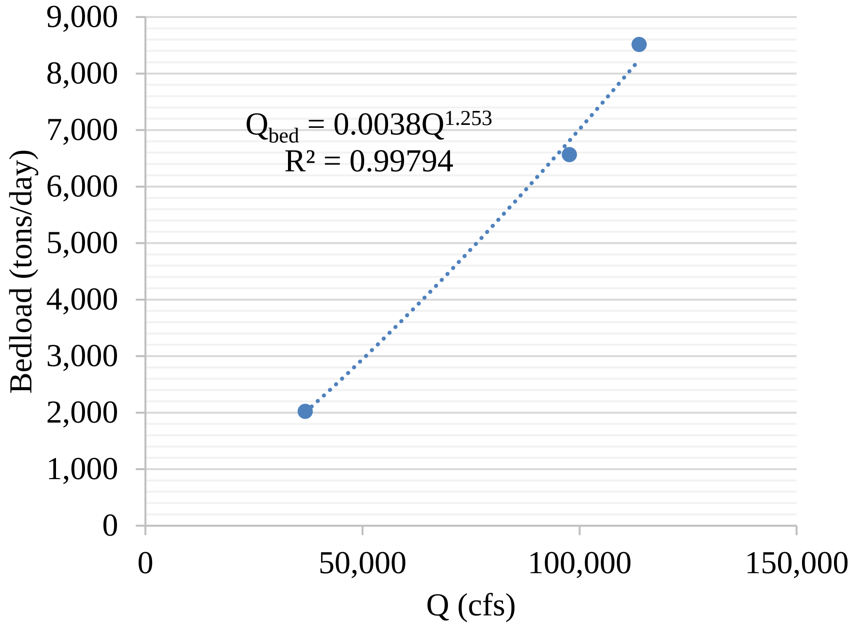


Figure 20. Missouri River Bedload Rating Curve at St. Joseph (from Abraham et al. 2017)

The gradation of the incoming total sediment load is a function of the relative contributions from bed load and suspended load, which varies by flow and by whether the 1994 – 2010 or 2011 flow-load curve is used for the suspended sediment contribution.

Sediment rating curves at seven major tributaries were included in the model as flow-load boundary conditions. This report presents the calculated rating curves—the model itself includes these rating cures with the loads divided by two to compensate for a RAS 5.0.3 bug that doubles tributary loads.

The flow-load curve for the Nodaway River was based on USGS gage data for the Nodaway River at Clarinda, MO. USGS data yielded the gradational breakout of the suspended fines and sands, with over 93% of the suspended sediment load composed of silts and clays. Bed load data was assumed to be 1% of suspended with predominantly fine and medium sand. Table 5 and Figure 21 provide the rating curve for the Nodaway River.

Table 5. Nodaway River Model Bed Material Rating Curve

Q (cfs)	100	2,000	10,000	150,000
Qs (tons/day)	2	1,333	31,200	67,002
VFS	1	436	10,200	21,905
FS	1	556	13,200	28,347
MS	0	205	4,800	10,308
CS	0	77	1,600	3,436
VCS	0	60	1,400	3,007

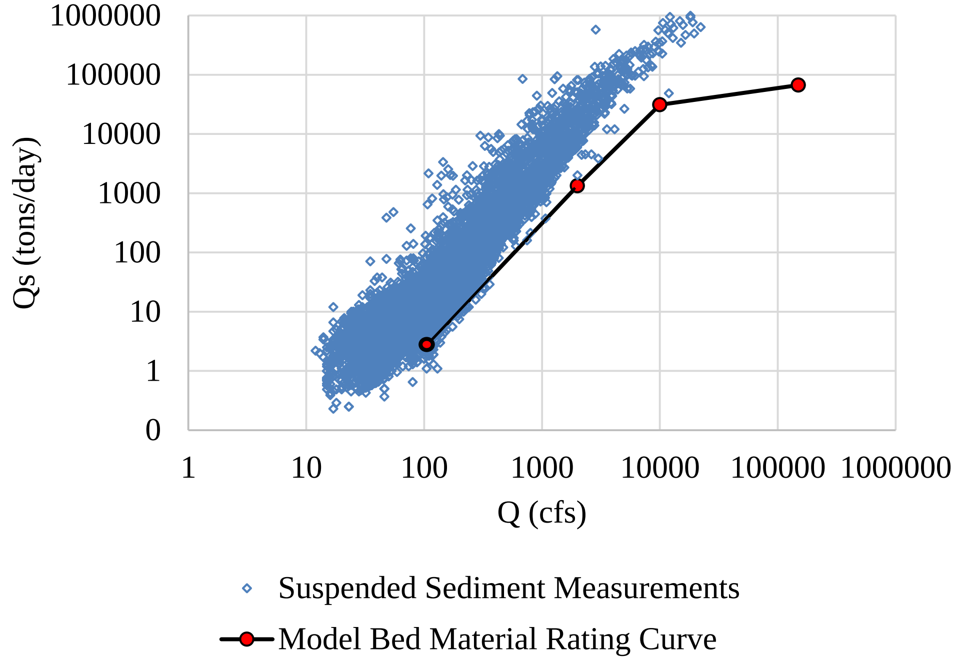


Figure 21. Nodaway River Sediment Loads

The flow-load curve for the Platte River was based on USGS gage data for the Platte River at Sharps Station. A bed material load was developed by subtracting the percent fines recorded in USGS measurements (which increases with increasing flow) and adding 5% as an estimate for bed load. In the absence of measurements, the bed load was assumed composed of very fine, fine, and medium sand. Table 6 and Figure 22 provide the loads and gradations used in the model.

Table 6. Platte River Bed Material Load Rating Curve and Gradation

Q (cfs)	1	1000	5000	10000	50000
Qs (tons/day)	0.004	426	5337	8454	10305
VFS	0.002	265	3149	4763	5504
FS	0.001	151	2004	3322	4247
MS	0.0000	10	184	369	554

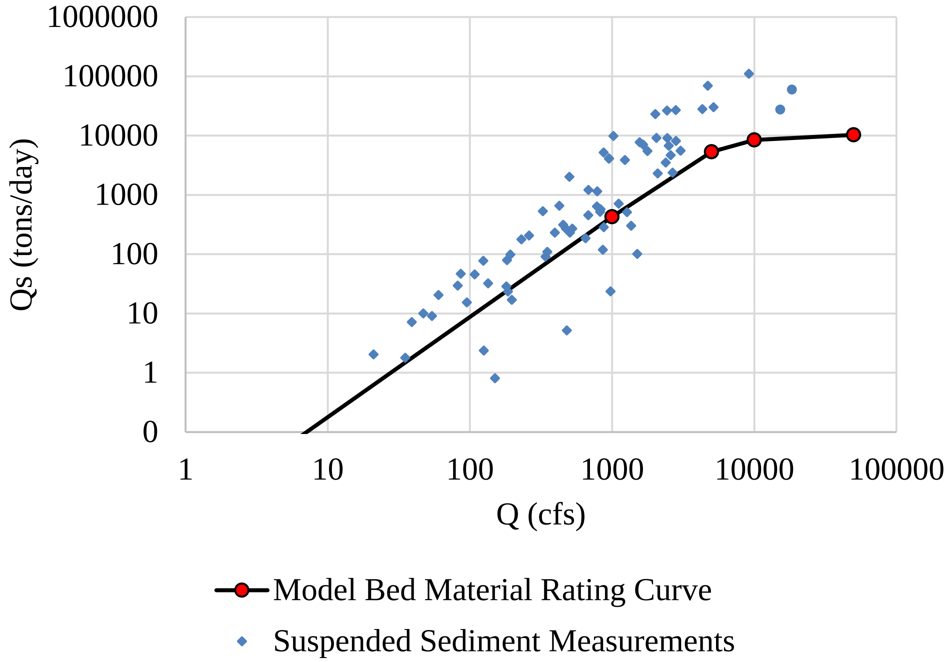


Figure 22. Platte River Sediment Loads

The sediment load and gradation for the Kansas River was based on USGS gage data for the Kansas River at Desoto, Kansas. Bed load data was assumed to be 5% of suspended with predominantly fine, medium, and coarse sand. The Kansas River experiences multiple anthropogenic influences on the sediment load in between the sediment gaging station and the confluence with the Missouri River, including multiple channel mining operations and multiple weirs. Table 7 and Figure 23 provide the rating curve for the Kansas River.

Table 7. Kansas River Bed Material Load Rating Curve and Gradation

Q (cfs)	250	6000	40000	100000	150000
Qs (tons/day)	4	1544	51736	240000	404168
VFS	0.4	160	5278	53418	53418
FS	0.8	568	25922	337534	337534
MS	2.4	738	18236	163036	163036
CS	0	56	1532	20120	20120
VCS	0	20	768	10060	10060

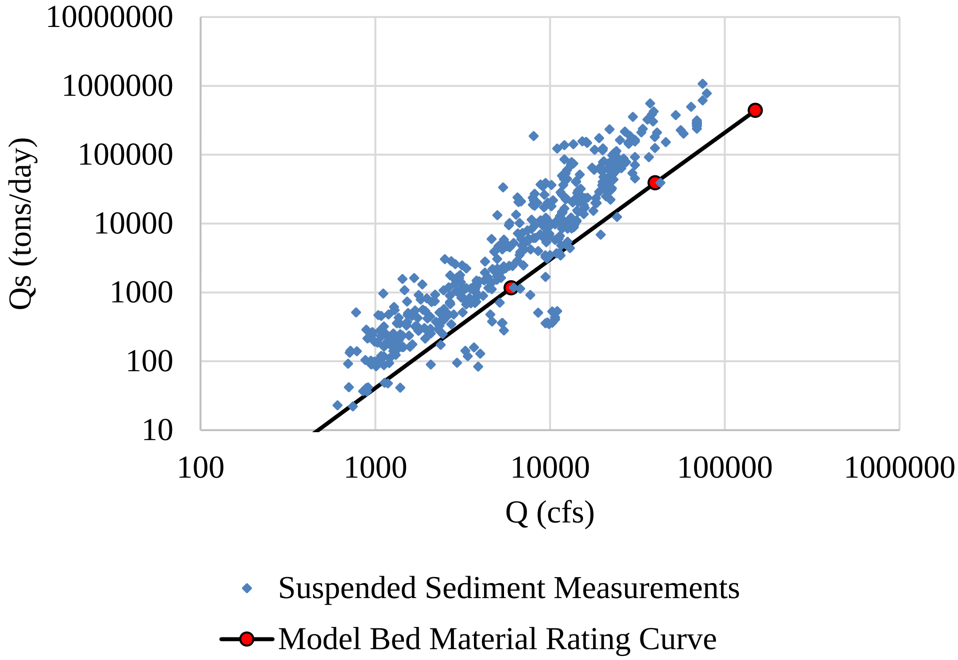


Figure 23. Kansas River Sediment Loads

The sediment load and gradation for the Grand River was based on USGS gage data for the Grand River at Sumner, Missouri. USGS data yielded overall concentrations and % fines. The gradational breakout of the suspended sands was assumed to be predominantly very fine and fine sand. Bed load data was assumed to be 5% of suspended with predominantly fine and medium sand. Table 8 and Figure 24 provide the rating curve for the Grand River.

Table 8. Grand River Bed Material Load Rating Curve and Gradation

Q (cfs)	100	1000	33000	100000
Qs (tons/day)	9	337	61540	70028
VFS	3	128	19361	21181
FS	4	139	25258	28701
MS	2	74	14648	16914
CS	0	10	4279	5387

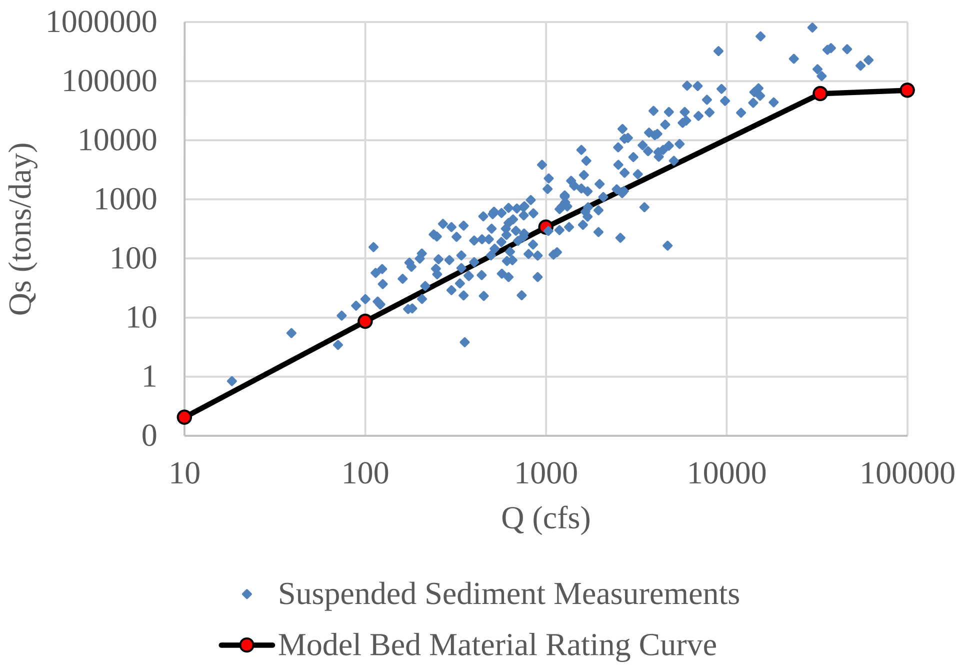


Figure 24. Grand River Sediment Loads

The sediment load and gradation for the Chariton River was based on USGS gage data for the Chariton River near Prairie Hill, MO. USGS data yielded the gradational breakout of the suspended fines and sands. Bed load data was assumed to be 5% of suspended with predominantly fine and medium sand. Table 9 and Figure 25 provide the rating curve for the Chariton River.

Table 9. Chariton River Bed Material Load Rating Curve and Gradation

Q (cfs)	20	10000	150000
Qs (tons/day)	0.1	3927	22305
VFS	0.00	222	1188
FS	0.05	2536	14505
MS	0.01	588	3311
CS	0.01	390	2201
VCS	0.00	191	1100

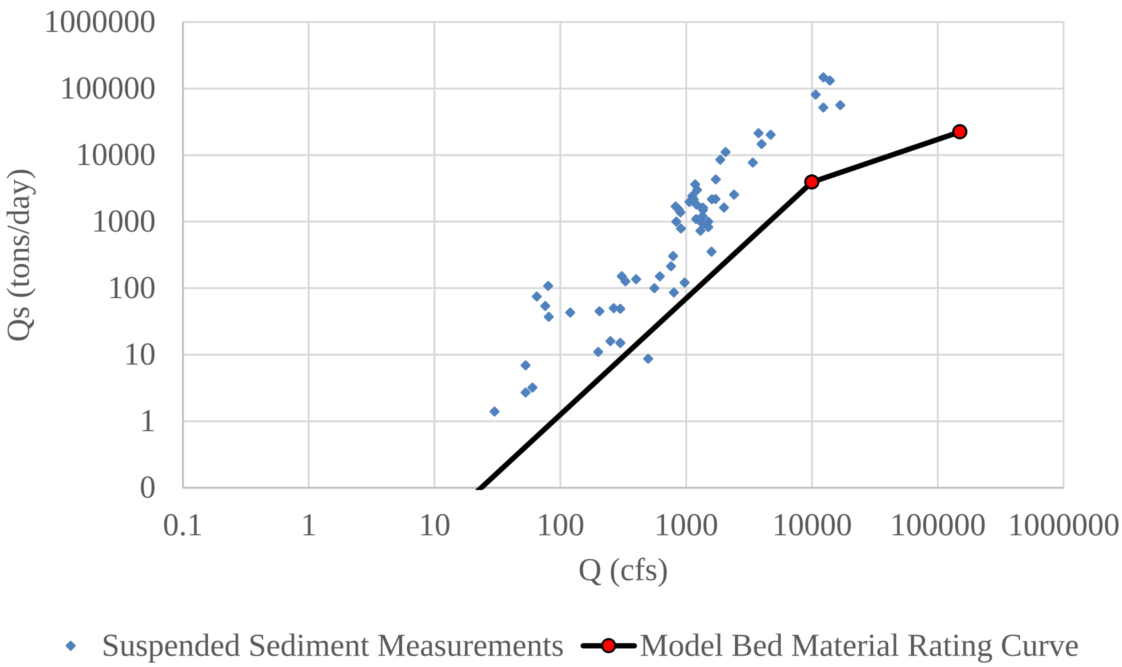


Figure 25. Chariton River Sediment Loads

The sediment load and gradation for the Osage River was based on USGS gage data for the Osage River below St. Thomas, MO. USGS data yielded overall concentrations and % fines. The gradational breakout of the suspended sands was assumed to be predominantly fine and medium sand. Bed load data was assumed to be 5% of suspended with predominantly fine and medium sand. Table 10 and Figure 26 provide the rating curve for the Osage River.

Table 10. Osage River Bed Material Load Rating Curve and Gradation

Q (cfs)	100	1000	5000	40000	100000
Qs (tons/day)	6	66	350	2993	4581
VFS	0.5	6	31	257	388
FS	1.7	19	101	856	1304
MS	2.9	33	175	1497	2291
CS	0.6	7	39	342	528
VCS	0.1	1	4	42	70

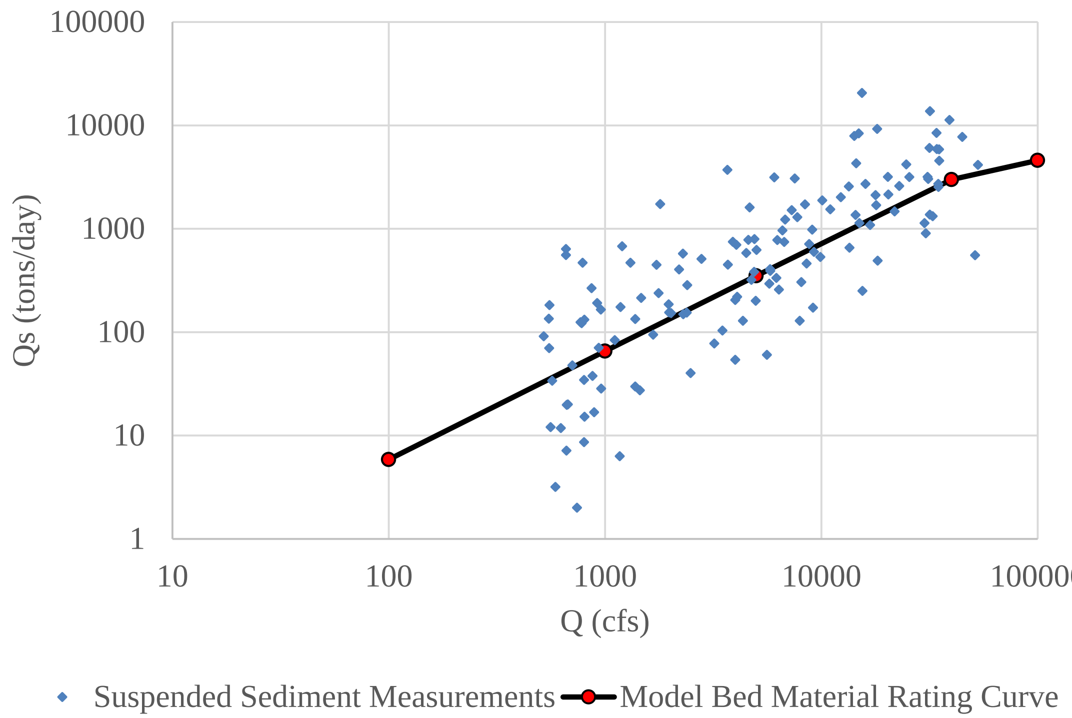


Figure 26. Osage River Sediment Loads

The sediment load and gradation for the Gasconade River was based on USGS gage data for the Gasconade River at Jerome, MO. USGS data yielded overall concentrations. The % fines was taken from the Osage River and the same assumptions were made for the gradational breakout of sands and bed load as the Osage River.

Table 11. Gasconade River Bed Material Load Rating Curve

Q (cfs)	100	1000	5000	25000	100000
Qs (tons/day)	1	59	1085	19839	95736
VFS	0.1	5	95	1714	8109
FS	0.3	17	312	5681	27256
MS	0.4	29	542	9919	47868
CS	0.1	6	122	2254	11038
VCS	0.0	1	13	270	1465

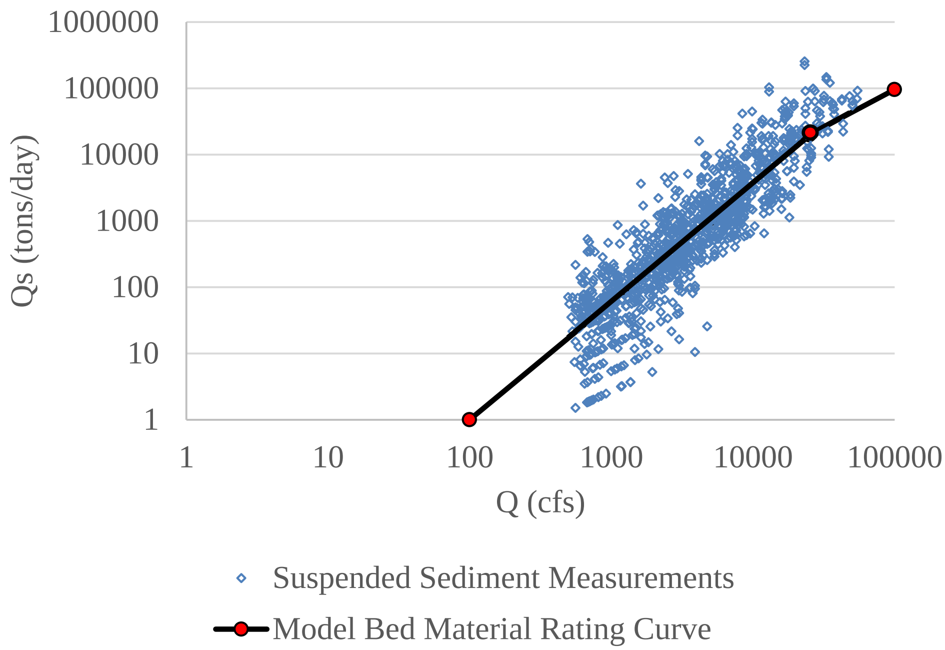


Figure 27. Gasconade River Sediment Loads

2011 Flood Boundary Conditions

As shown in Figure 3, the sediment concentrations were dramatically lower during the 2011 flood than typical. For March – November, 2011, the suspended sediment load was computed by the formula Qs = 0.7641 Q + 6910.7, where Qs = suspended sediment in tons/day and Q = daily flow in cfs. Bedload, computed with the rating curve depicted in Figure 20, was added to this value to yield the total bed material load for the day at St. Joseph. This load was transferred upstream by subtracting the load for the Nodaway. This flow/load/gradation curve is unique to the 2011 event.

Dredging

Commercial dredging on the Missouri River was a significant driver of bed degradation during the calibration period (USACE 2011). The resolution of dredging data varies over time. From 1994 – 1996, the annual tons dredged were reported to USACE's regulatory branch on a reach basis. Since 1997, daily tons dredged and river miles were reported.

Dredging was included in the degradation model as monthly totals at each cross-section with the start date the first day of the month and the end date the last day of the month. For 1997 and later, the reported monthly tonnages of dredging were assigned to the appropriate cross-sections and to the appropriate month. Annual, reach-scale tonnages for 1994 – 1996, were apportioned according to the temporal and spatial distribution of dredging from 1997 – 2009. Figure 28 provides the spatial distribution for dredging for each year.

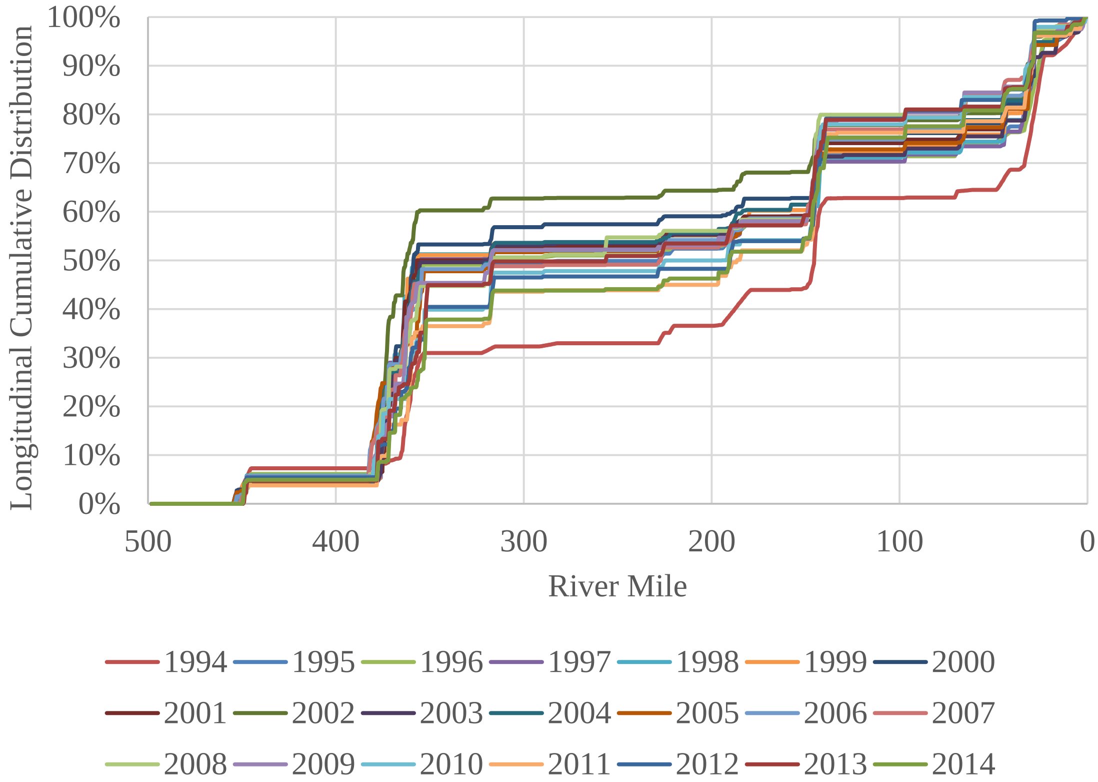


Figure 28. Longitudinal Cumulative Dredging Distribution by River Mile

The impact of dredging was restricted to the actual dredging tonnage, i.e. one ton of extracted material lowers the bed by a volume equivalent to one ton. Potential dredging effects due to material sorting, re-discharge, and bed disturbance were not included. Dredging volumes or locations were not adjusted during calibration.

Unmeasured Sediment Inflows

Sediment budgeting was used to estimate unmeasured sediment inflows for inclusion in the model. A sediment budget quantifies the terms in the continuity equation: Sediment_In – Sediment_Out= Δ Storage. The following equation provides the sediment budget from the St. Joseph to the Kansas City gages, from 1994 to 2005:

$$SJ + PR + KR + UnM - D - FP - KC = \Delta Bed \qquad \text{(Equation 1)}$$

Where SJ = the sand load passing the Saint Joseph gage (tons)

PR = the sand input from the Platte River (tons)

KR = the sand input from the Kansas River (tons)

UnM = the unmeasured sediment inflows (tons)

D = the dredging volume (tons)

KC = the sand load passing the Kansas City gage (tons)

ΔBed = the total mass of bed change (tons)

PR, KR, and KC were taken from USGS (Heimann et al. 2010), using model rating curves to fill in the gaps as needed. As a full 2005 bathymetric survey is not available, the 1994 – 2007 bed change was divided by 13 years then multiplied by 11 years to approximate the 1994 – 2005 bed change. Floodplain deposition (FP) was assumed negligible for 1994 to 2009. The unmeasured sediment (UnM) was solved for arithmetically. Table 12 presents the numerical values for each variable. (Note: Significant digits are retained to the ton in Tables 12 – 14 to make the math reproducible, not to imply that any of these quantities are known to the ton.)

Table 12. Sediment Budget 1994 – 2005 for St. Joseph to Kansas City

Budget Term	Mass (tons)
SJ	116,179,350
PR	3,345,417
KR	10,091,852
UnM	10,778,755
D	20,981,695
FP	0
KC	147,369,350
ΔBed	-27,955,671

A similar analysis was performed for the reach from Kansas City, MO to Herman, MO. Table 13 presents the numerical values for each variable.

$$KC + GR + CH + OS + GS + UnM - D - FP - HR = \Delta Bed \qquad \text{(Equation 2)}$$

Where KC = the sand load passing the Kansas City gage (tons)

GR = the sand input from the Grand River (tons)

CH = the sand input from the Chariton River (tons)

OS = the sand input from the Osage River (tons)

GS = the sand input from the Gasconade River (tons)

UnM = the unmeasured sediment inflows (tons)

D = the dredging volume (tons)

HR = the sand load passing the Hermann gage (tons)

ΔBed = the total mass of bed change (tons)

Table 13. Sediment Budget 1994 – 2005 for Kansas City to Hermann

Budget Term	Mass (tons)
KC	147,369,350
GR	13,796,832
CH	492,949
OS	2,792,519
GS	3,136,888
UnM	60,718,243
D	36,042,315
FP	0
HR	199,524,347
ΔBed	-7,259,881

The unmeasured sediment load incorporates all non-specific sediment sources needed for the sediment budget to balance, including bank erosion, gullies, tributaries, and shallow water habitat construction activities. In addition, the unmeasured term incorporates error in the upstream, downstream, and tributary rating curves. The procedure used to input this sediment to the model causes the needed additional sediment to be more or less uniformly distributed between the gages.

The UnM values listed in Tables 12 and 13 were entered into the model using rating curves tied to the flow boundary conditions representing ungaged inflows. Rating curves of the type Qs = aQ, were created, with *a* set so that the sum of Qs from 1994 to 2005 equals the UnM values presented in Tables 12 and 13.

The flows that drive the unmeasured sediment rating curves are scaled-down versions of the ungaged water inflows. These inflows were computed as the difference between flows at gages that are not accounted for by tributary inflows. On any given day, differences in flows at gages may be negative due to unsteady hydrograph effects. To avoid problems with the rating curves during negative uniform lateral flows, separate flow boundary conditions that are always positive were created for use with the unmeasured sediment rating curves. These separate flow boundary conditions were scaled down by 10,000 so that only negligible additional flow is added.

Floodplain Deposition

As depicted in Figure 2, levees which line most of the Missouri River confine flows to a narrow corridor. These levees reduce floodplain deposition of sediments during moderately high flows, but very high flows which overtop levees can deposit tremendous volumes of sediment on the floodplain. While the precise volumes of floodplain deposition cannot be quantified with existing data, available data do allow approximations based on sediment budgeting (1993 flood) and aerial photographic analysis (2011 flood).

1993 Flood

A sediment budget analysis similar to that presented in the previous section was performed from 1987 to 1994. This time period includes the historic flood of 1993 which deposited tremendous quantities of sediment on the floodplain. The Soil Conservation Service (SCS 1993) reports that the 1993 flood deposited 546 million cubic yards of sediment on the floodplain, though this value includes sediments sourced from significant scour holes in the floodplain as well as the from the channel. Sediment budget analyses Equations 1 and 2 were used to determine the floodplain deposition amount (FP) to be sourced from the channel during the 1993 flood. The ungaged sediment contribution was computed using the *a* and *b* values computed from the analysis described in the previous section. Tables 14 and 15 provide the computed values.

Table 14. Sediment Budget 1987 – 1994 for St. Joseph to Kansas City. Used to solve for the 1993 floodplain deposition.

Budget Term	Mass (tons)
SJ	72,440,000
PR	1,647,389
KR	13,151,706
UnG	8,978,963
D	1,600,786
FP	**51,071,910**
KC	84,240,000
ΔBed	-40,694,638

Table 15. Sediment Budget 1987 – 1994 for Kansas City to Hermann. Used to solve for the 1993 floodplain deposition.

Budget Term	Mass (tons)
KC	84,240,000
GR	7,039,551
CH	234,738
OS	1,475,300
GS	1,678,128
UnG	14,977,292
D	20,980,378
FP	**98,220,908**

HR	84,240,000
ΔBed	-152,526,276

This analysis computes 149 million tons of channel sediments deposited on the floodplain during the 1993 flood from St. Joseph, MO to Hermann, MO. Extending the analysis to the entire model space with the same overall tons/mile rate of deposition = 212,422,000 total tons of floodplain deposition during the 1993 flood.

2011 Flood

The floodplain deposition amount for the 2011 flood was computing following the same analysis used in USACE (2017), which is the aerial extend of sand deposition from Alexander et. al, (2013) times the suggested minimum depth of 2 ft. See Figure 29.

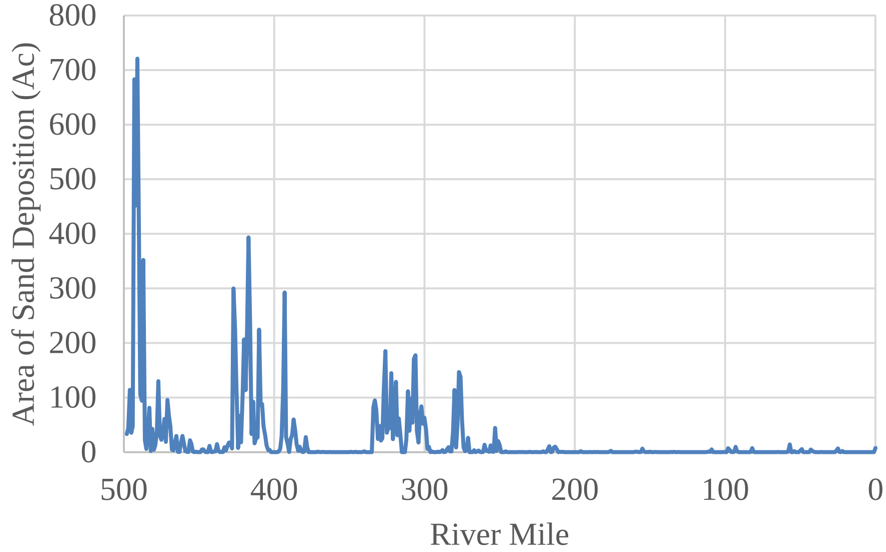


Figure 29. Acreage of Floodplain Sand Deposition from the 2011 flood. Data from Alexander et. al (2013).

The 1993 and 2011 flood events offer two data points from which to interpolate and extrapolate floodplain deposition to other floodplain deposition events. This was accomplished by creating rating curves of the type $FP = aQ^b$, where Q = the daily flow in the mainstem Missouri River and *a* and *b* were chosen such that the total floodplain deposition over the flood event was correct for both the 1993 and the 2011 floods. For flows below a threshold, FP was assumed zero. Table 16 provides the parameters.

Table 16. Parameters Used for Floodplain Deposition

Floodplain Equation	**Gage(s)**	**Flow Threshold (cfs)**	**a**	**b**
#1	Rulo	160,000	5.84E-08	2.43
#2	St. Joseph	200,000	3.65E-06	2.43
#3	St. Joseph + Platte River	200,000	3.65E-06	2.43
#4	Waverly	200,000	1.22	1.21
#5	Boonville	260,000	1.22	1.21
#6	Boonville	260,000	2.53	1.21

The sediment loads so calculated were entered into the model as lateral loads at discrete locations which correspond to the locations of the 2011 floodplain deposition. Several of the computed loads were split to two cross section locations to better distribute the effect of floodplain deposition. Table 17 indicates the lateral loads entered into the HEC-RAS model. In future projects, the same locations for floodplain deposition seen in the 2011 flood are used for any flood that exceeds the flow thresholds.

Table 17. Source of Lateral Loads for Floodplain Deposition

RAS RS	Entered into RAS
492.5	Eq#1 * 75%
478.4	Eq#1 * 25%
427.13	Eq#2
410.01	Eq#3
325.2	Eq#4 * 50%
301.97	Eq#4 * 25%
279.13	Eq#4 * 25%
187.55	Eq#5 * 50%
133.66	Eq#5 * 50%
88.92	Eq#6 * 50%
20.66	Eq#6 * 50%

Table 18 indicates that the model inputs closely match the best computed values for the 1993 and 2011 flood. Figure 30 presents the computed floodplain deposition and the location/magnitude entered into the model for the 2011 flood event. Values are negative because they draw sediment from the channel.

Table 18. Computed Floodplain Deposition vs. Included Model Floodplain Deposition

Reach	1993		2011	
	Computed (tons)	Model (tons)	Computed (tons)	Model (tons)
SJ to KC	-56,847,829	-56,751,019	-14,864,605	-14,832,221
KC to HR	-102,116,528	-101,748,539	-13,454,765	-13,406,075
Full Model	-225,539,173	-225,621,716	-45,695,231	-45,915,488

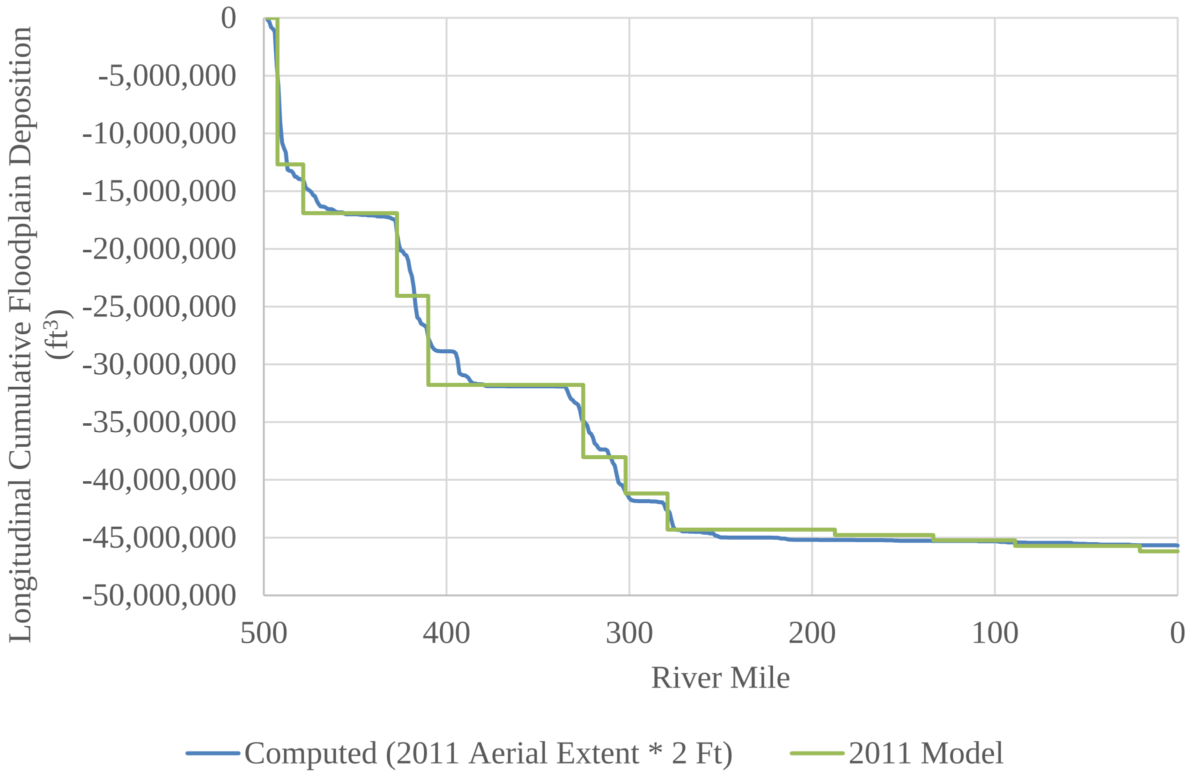


Figure 30. Floodplain Deposition in the Model During the 2011 Flood

3.5 Model Parameters

Model parameters include sediment transport formula, bed mixing algorithm, and computational time steps. These model parameters are described in the following paragraphs.

Sediment Transport Formula

Multiple sediment transport formula were tested, including Laursen-Copeland, Meyer-Peter and Muller, Toffaleti, and Yang. The Toffaleti (1968) sediment transport formula was selected to compute the sediment transport capacity due to its applicability and history of use on large, sand-bed rivers including the Missouri River (USACE 2017) and because it yielded reasonable initial results. During calibration, it was found that Toffaleti produced insufficient transport during the flood of 2011. To increase the transport capacity at high flows, the combined Toffaleti/MPM function was used with a coefficient of 1 and a power of 1.5. This provides slightly more transport capacity, particularly at high flows. The Toffaleti fall velocity method was also used. HEC (2010) details the Toffaleti computational procedure.

Bed Mixing Algorithm

Two bed mixing/armoring algorithms were tested: Exner 5 and Exner 7. Exner 7 produced excessive degradation compared to the prototype and was not selected. Exner 5 yielded reasonable results for total bed degradation and was selected for use in the model.

Computational Increment

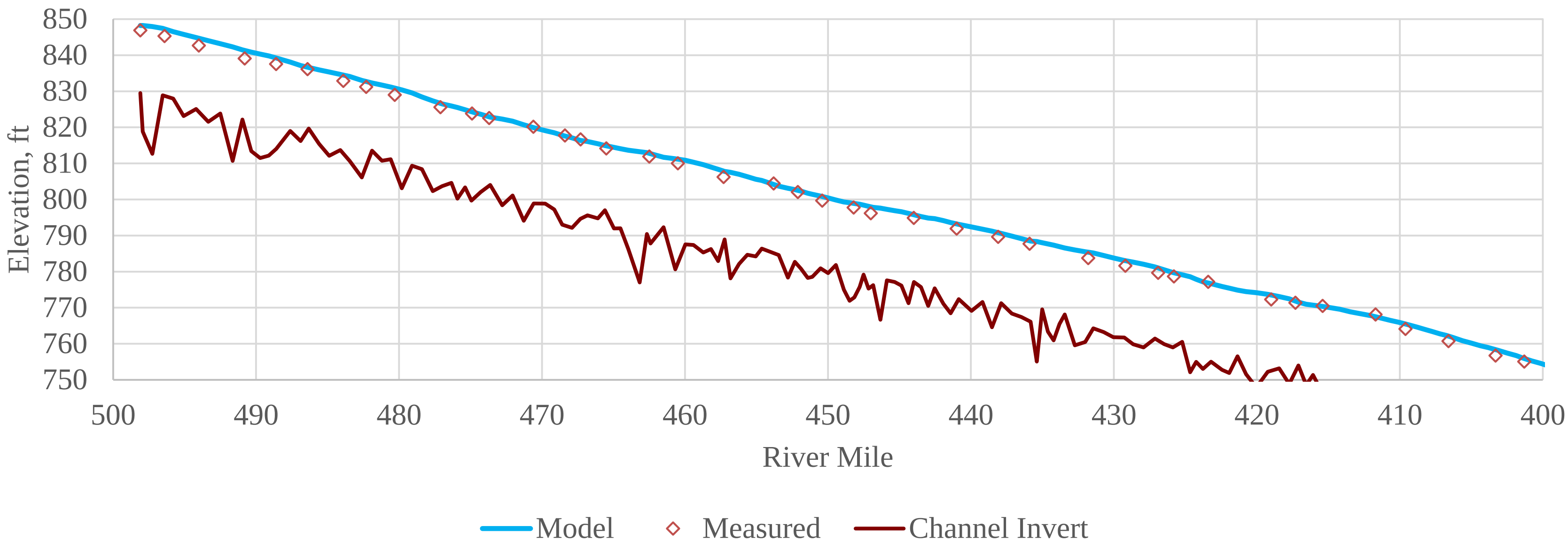
The computational increment was set based on the flow rate. It ranges from 24 hr when flow is less than 60,000 cfs to 30 min when flow exceeds 200,000 cfs. Bed exchange iterations per time step was set to 10, the HEC-RAS default.

4.0 Calibration/Verification

The principle calibration period runs from Aug 1, 1994 to Oct 1, 2009. This time period includes a range of high and low flows and is most representative for future prediction. Water surface elevations at multiple gages, sediment loads, and repeat cross sections in 2007 and 2009 offer robust calibration data over this period. A second time period from Oct 1, 2009 to July 29, 2014 was also used in calibration. However, because this time period includes the historic Missouri River Flood of 2011 which exhibited unique boundary conditions, this time period serves more as a verification of reasonableness than a second calibration point. The principal parameters which were varied to achieve calibration were the Manning 'n' values for the active channel, inter-sill region, and inter-dike region, the flow-based 'n' adjustment factors, bed gradation data, the sediment loading from Kansas River and Grand River, and the moveable bed extents. As described in the previous paragraphs, these calibrated initial conditions and boundary conditions have physical basis in measured data.

Early Hydraulic Calibrations

The model bathymetry is from 1994. The calibration period starts in Aug 1, 1994. On Aug 16 and 17, 1994, the water surface was measured at multiple points along the river. These measured water surface elevations are subject to greater error than USGS gage measurements but are still useful to verify the hydraulic model. Figures 31, 32, and 33 illustrate the model agreement to the low water surface elevations collected on August 16 and 17, 1994. The average absolute difference between modeled and measured water surfaces for August 16 and 17 is 0.8 ft. This analysis is similar to a "fixed-discharge, fixed-bed" analysis for a low discharge because it occurs so soon after the model start.



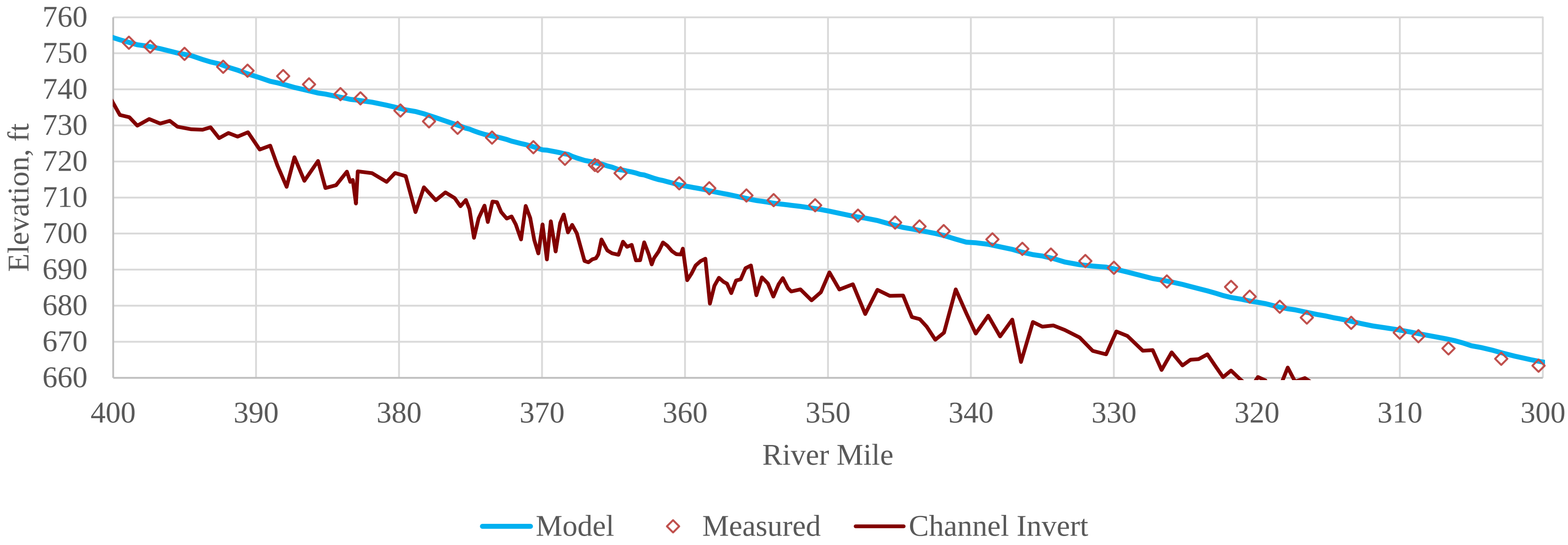


Figure 31. Model hydraulic comparison at low flow: River Miles 500 - 300

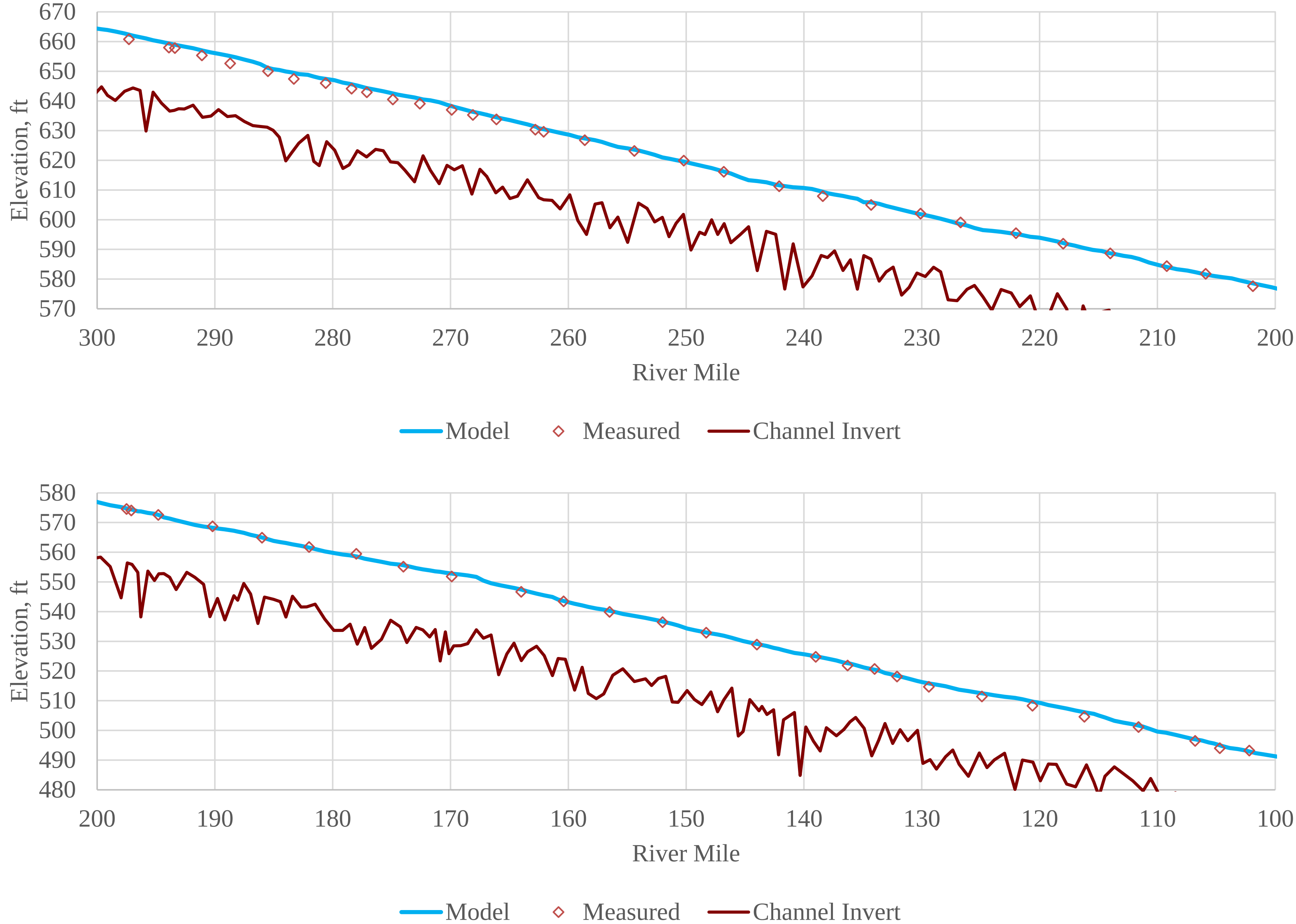


Figure 32. Model hydraulic comparison at low flow: River Miles 300 - 100

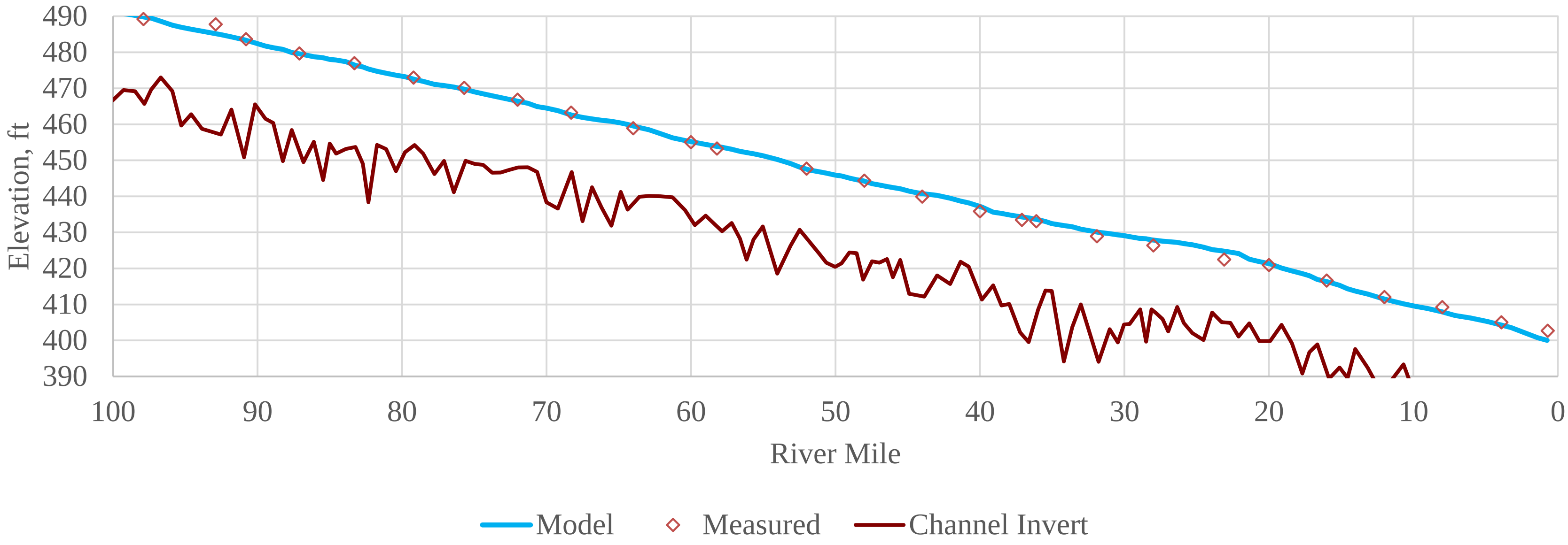


Figure 33. Model hydraulic comparison at low flow: River Miles 100 - 0

A moderately high flow event occurred within a year of the model start. Figures 34 – 39 compare model results to the water surface elevation at the USGS Missouri River gages at St. Joseph, Kansas City, Waverly, Boonville, Herman, and St. Charles, respectively.

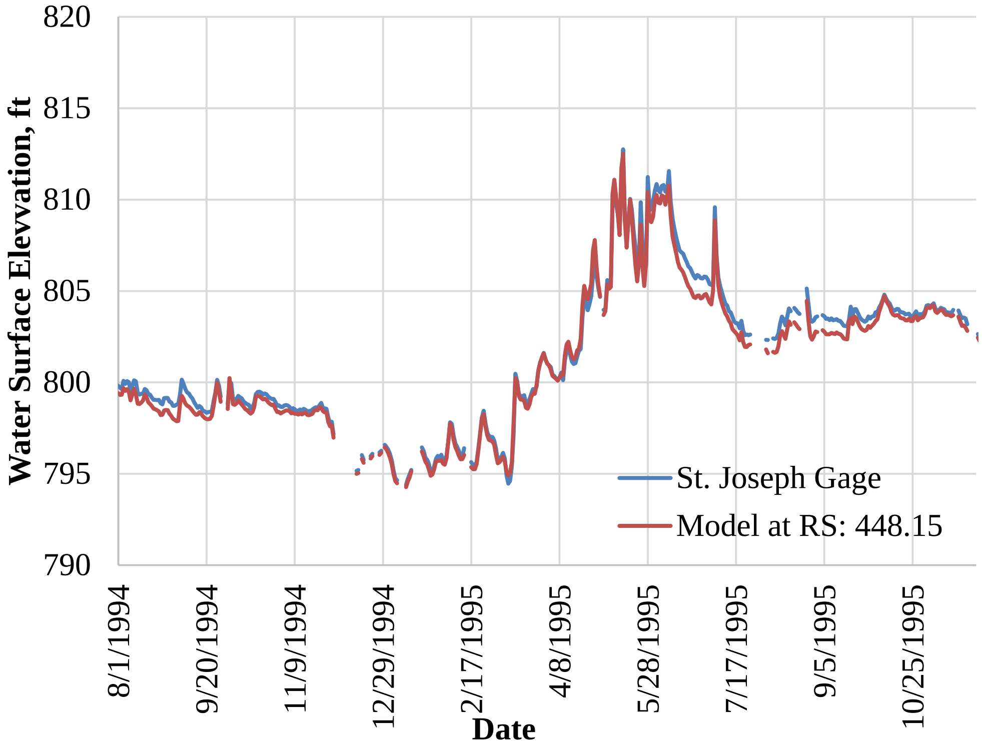


Figure 34. Water surface at the St. Joseph gage during first year of calibration period

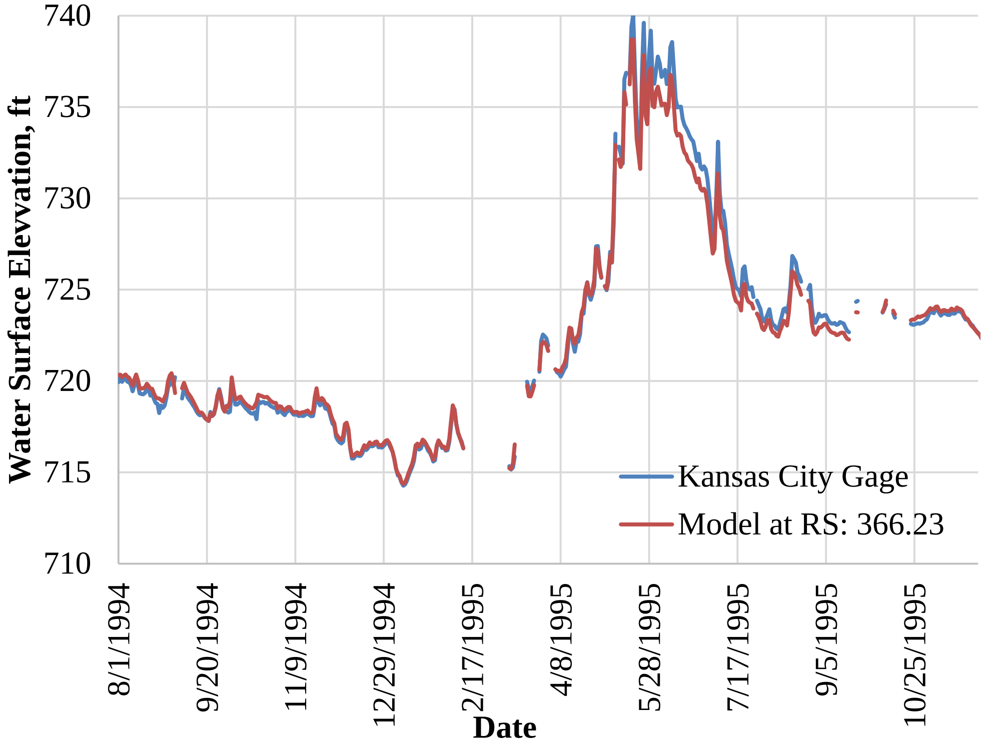


Figure 35. Water surface at the Kansas City gage during first year of calibration period

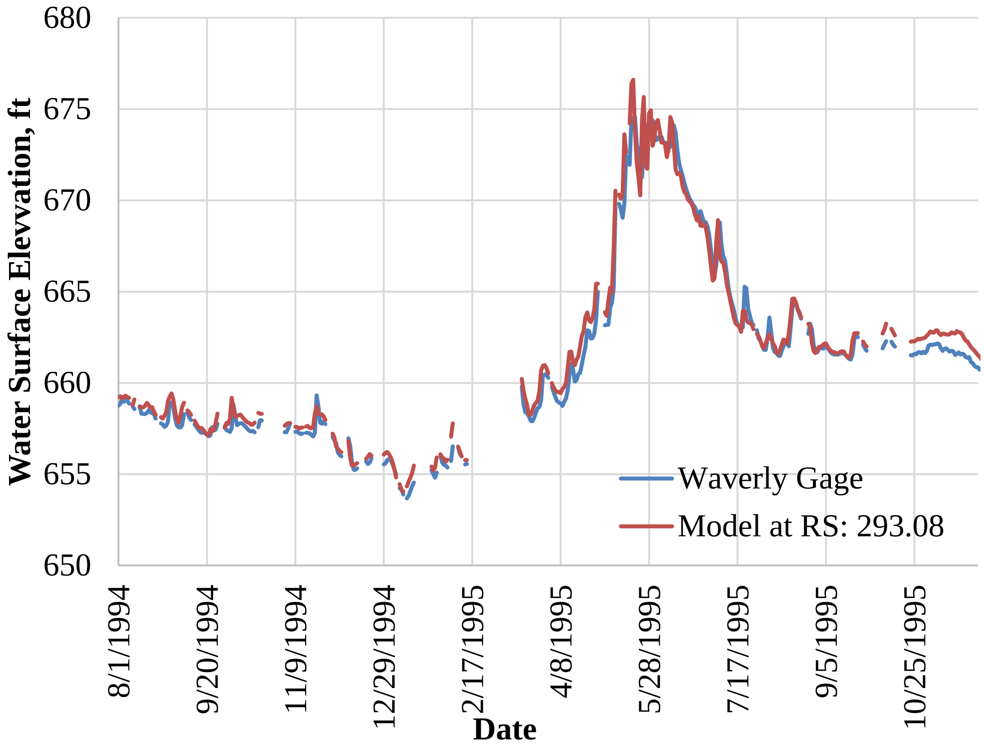


Figure 36. Water surface at the Waverly gage during first year of calibration period

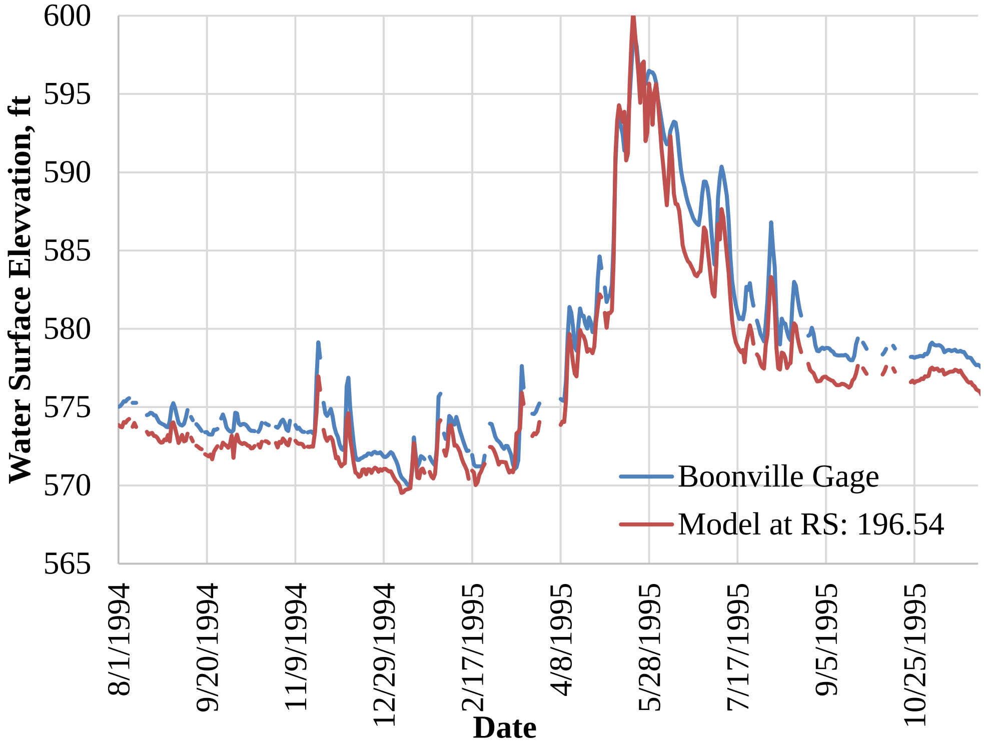


Figure 37. Water surface at the Boonville gage during first year of calibration period

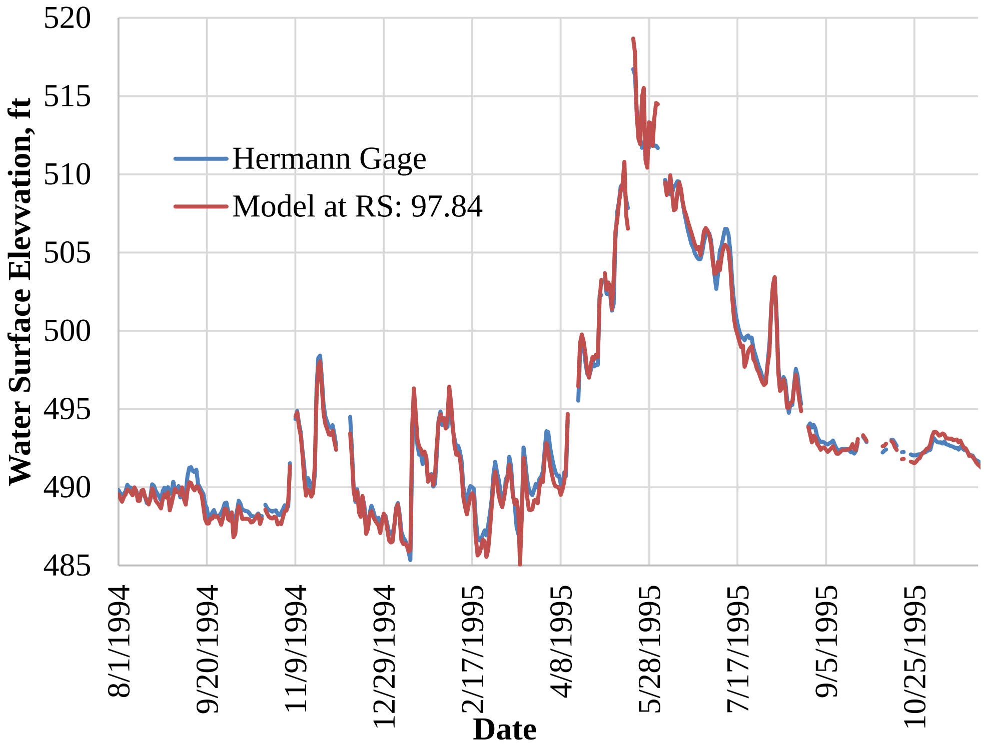


Figure 38. Water surface at the Hermann gage during first year of calibration period

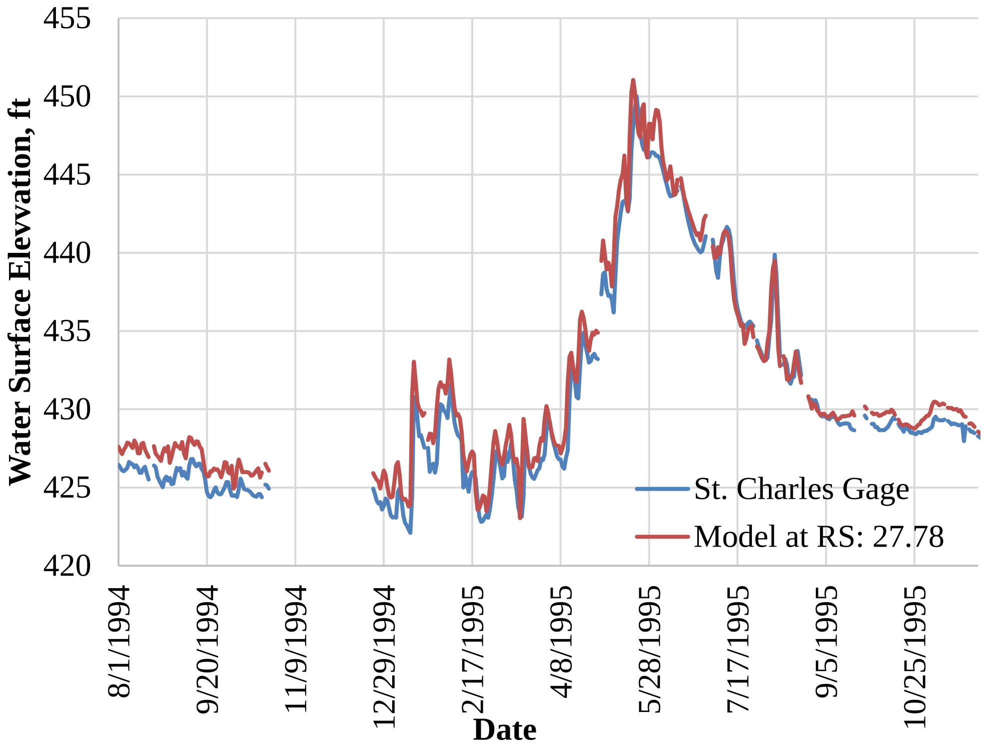


Figure 39. Water surface at the St. Charles gage during first year of calibration period

Output shown in Figures 34 to and 39 is from the mobile-bed model and therefore includes slight bed changes over the course of the first year. Table 19 provides the average absolute difference between the model and measured water surfaces. This is reasonable agreement, given the 498 mile length of this quasi-unsteady model. Attempts to reduce the discrepancy at Boonville were found to cause unreasonable departures in other calibration metrics.

Table 19. Average Absolute Departure from Daily Gage Measurements in the First Year of Simulation (ft)

SJ	KC	WV	BV	HR	SC
0.30	0.46	0.51	1.58	0.53	1.19

Hydraulic Calibration- Long Term

The agreement of the model water surface elevation over the full calibration period (1994 to 2014) is a verification of the temporal fidelity of the sediment modeling. Table 20 indicates small departures over the course of the 20-year simulation.

Table 20. Average Absolute Departure from Daily Gage Measurements over Full Calibration Period -- Aug 1994 to July 2014 (ft)

SJ	KC	WV	BV	HR	SC
-0.03	-0.70	0.15	-0.81	-0.18	0.12

Velocity Calibration

Channel velocities were measured during, soon after, and one year after the 2011 flood via ADCP. As seen in Figure 40, model velocities are in reasonable agreement with measured velocities. The measurements in July of 2012 purposefully measured locations with the greatest dike constriction, which explains some of the higher velocities.

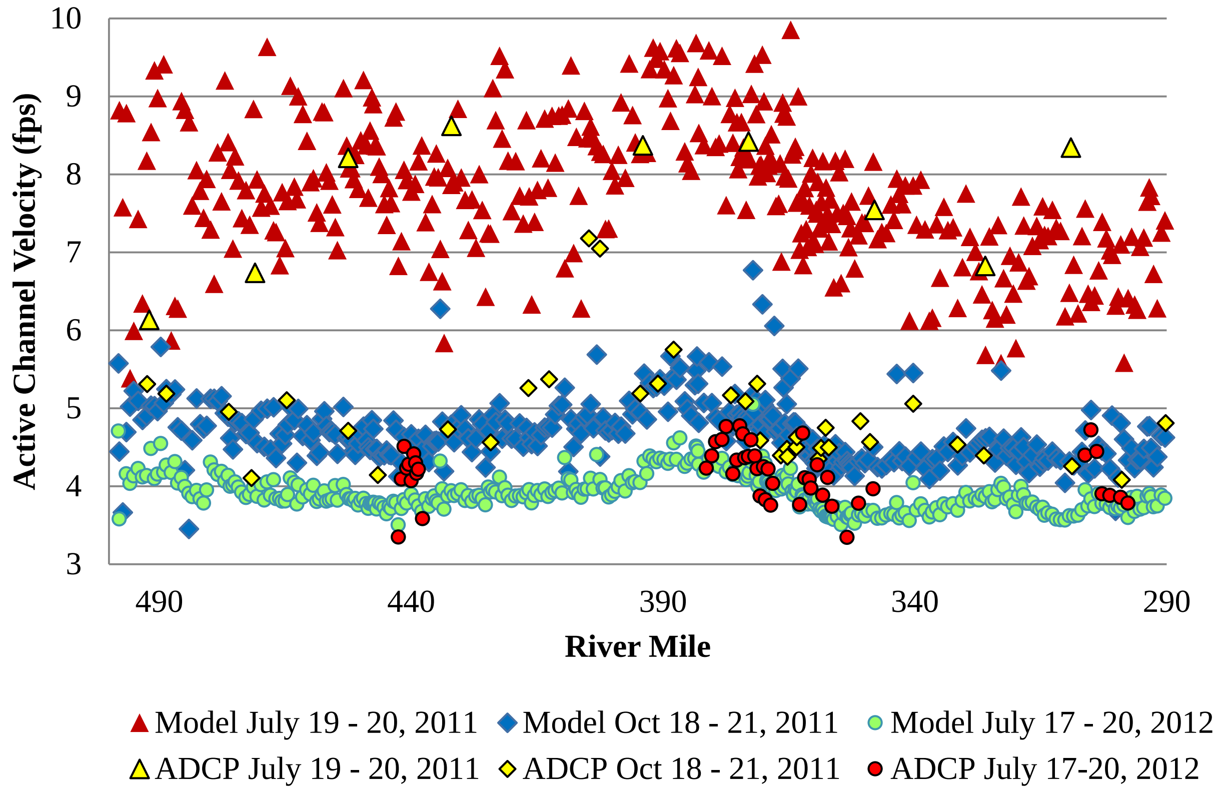


Figure 40. Velocity Comparison

Sediment Load

USGS (Heimann et al., 2010) provides an estimate for annual suspended sediment sand loads through water year 2005 at multiple gages on the Missouri River. Table 21 compares these suspended sediment values plus bedload values from rating curves developed from Abraham et al. (2017) against model values for sediment transport at the gages. The model output agrees quite well with the measured values and is well within the uncertainty estimates presented in (Heimann et al., 2010).

Table 21. Sediment Load Comparison from 01 Oct 1994 to 30 Sep 2005 at Mainstem Gages

Gage	Model	USGS+Bedload	Model/Measured
SJ	113,817,726	116,179,350	0.98
KC	140,280,159	141,362,547	0.99
HR	175,773,026	189,286,266	0.93

Bed Elevation and Mass Calibration

Figure 41 presents bed elevation change at each model cross-section and each measured location. As seen, the model accurately reproduces degradation trends, though both measured data and model output exhibit significant variability and scatter. The nature of the active bedforms on the Missouri River causes individual cross sectional measurements to vary by several feet even without persistent geomorphic change. USACE (2015) finds that 75% of cross sections varied 0.25 ft to 3 ft from 2008 to 2009, but some temporarily rose or fell by as much as 11 ft in a single year.

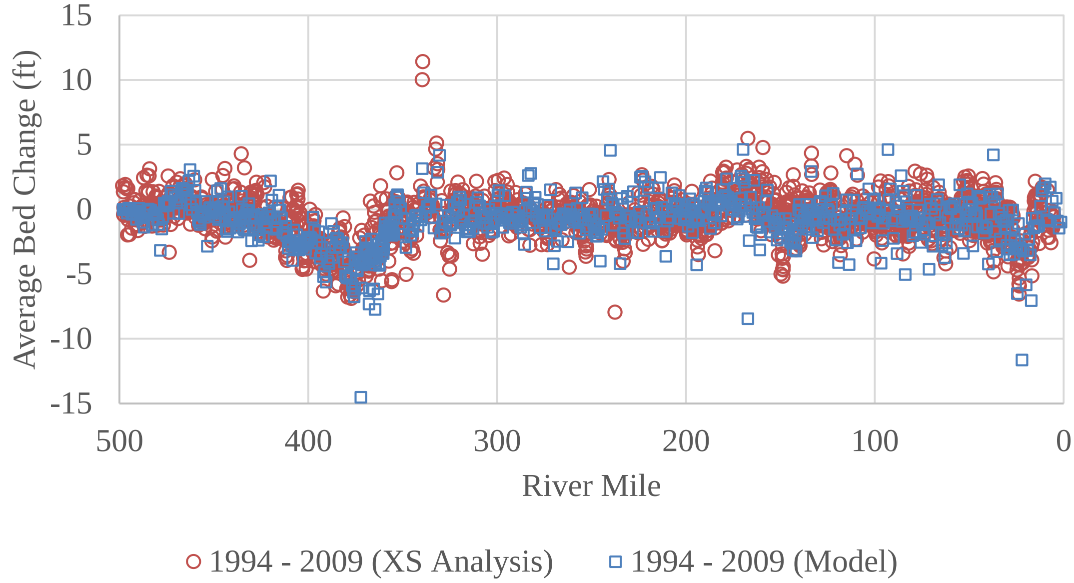


Figure 41. Model vs. Measured Bed Elevation Change 1994 to 2009

With a sufficient number of cross sections, these random fluctuations average out, which makes volume or mass change over reaches especially useful for comparing model to measured output rather than bed change at an individual cross section. Figure 42 plots the longitudinal cumulative mass change for both model and measured cross sections from 1994 to 2009. As seen in Figure 42, the calibrated model closely approximates the magnitude and location of mass change from 1994 to 2009. This time period includes both high flows and low flows and a range of channel mining rates and indicates the strength of the calibration for long-term modeling.

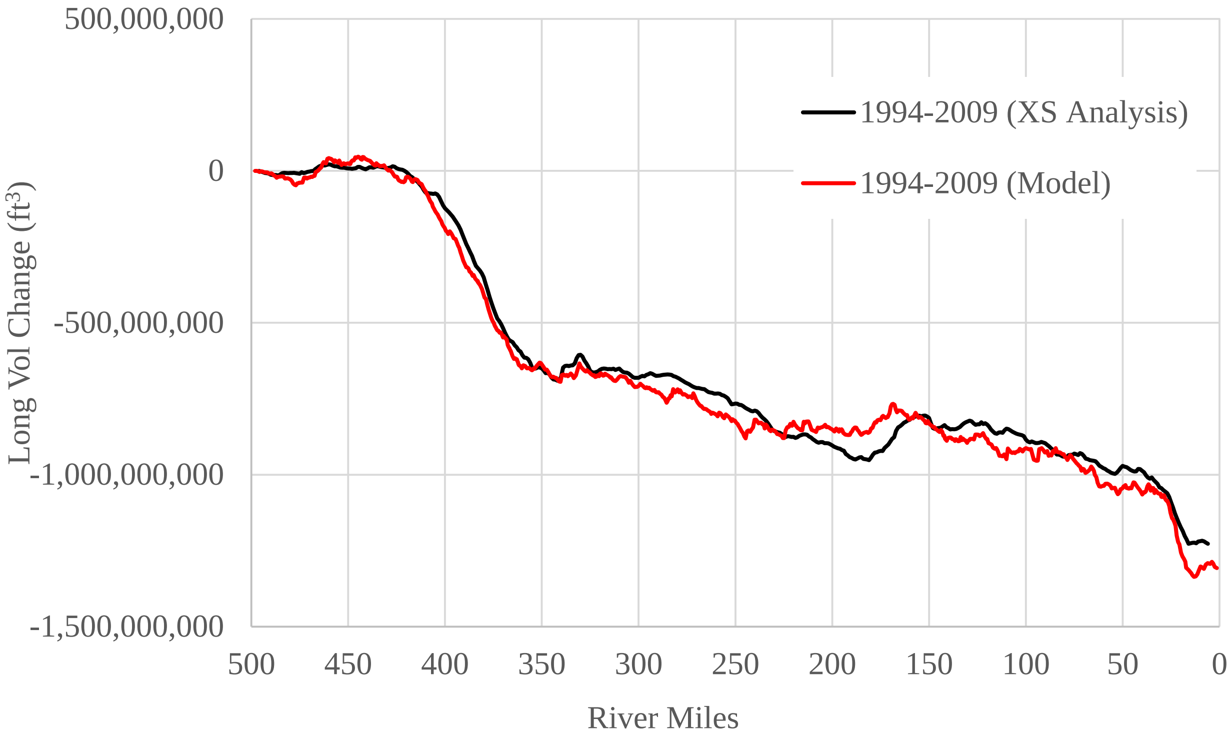


Figure 42. Longitudinal Cumulative Mass Calibration: 1994 to 2009

Figure 43 presents an initial longitudinal cumulative mass change from 2009 to 2014. The 2011 flood and post-flood rebound dominates the bed change over this time period. As seen, the model reasonably reproduced upstream headcut migration visible from RM 500 to aprx. RM 388, as well as the general degradation trend from RM 181 to RM 0. The model did not reproduce the rebound observed from RM 350 to 181. The cross section analysis (depicted in Figure 12) indicates that the sediment eroding from RM 500 to 388 did not simply redeposit downstream; from year 2009 to 2012 the headcut progressed upstream while the downstream channel was also erosional. The rebound occurred after the flood--from 2012 to 2013 and from 2013 to 2014. The mainstem and tributary rating curves developed from USGS data as used in this model do not bring in sufficient sediment to account for the post-flood rebound. Anecdotal evidence suggests that the source of the sediment may be eroding banks or headcutting up tributaries—but these sources have not been quantified.

Including an additional 23 million tons in sediment load from unknown sources following the 2011 event yields Figures 44 and 45. As seen, this provides a better estimate for the post-flood rebound following the 2011 event. To avoid negative bias in future projections, the 23M will be added in after a repeat of the 2011 event in the period of record. As Figure 12 does not indicate a similar rebound following the 1993 flood, the extra tonnage is not added following the 1993 flood or other floods in the projection period.

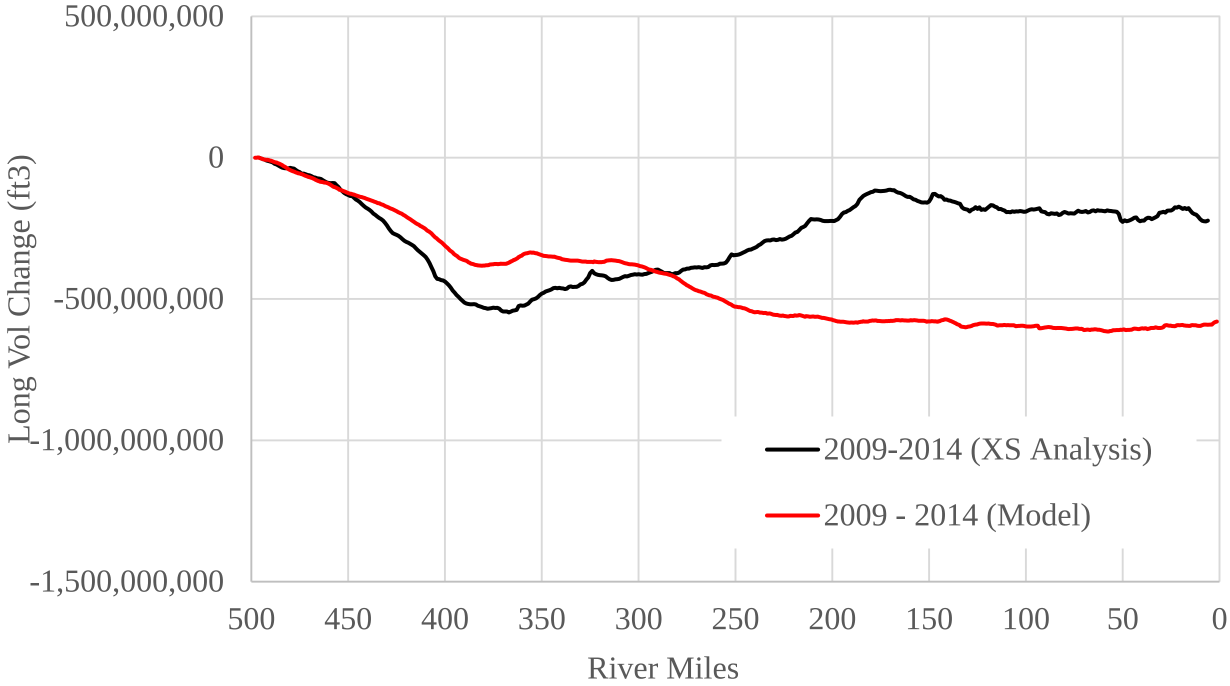


Figure 43. Longitudinal cumulative mass calibration: 2009 to 2014 with no additional post-2011 sediment

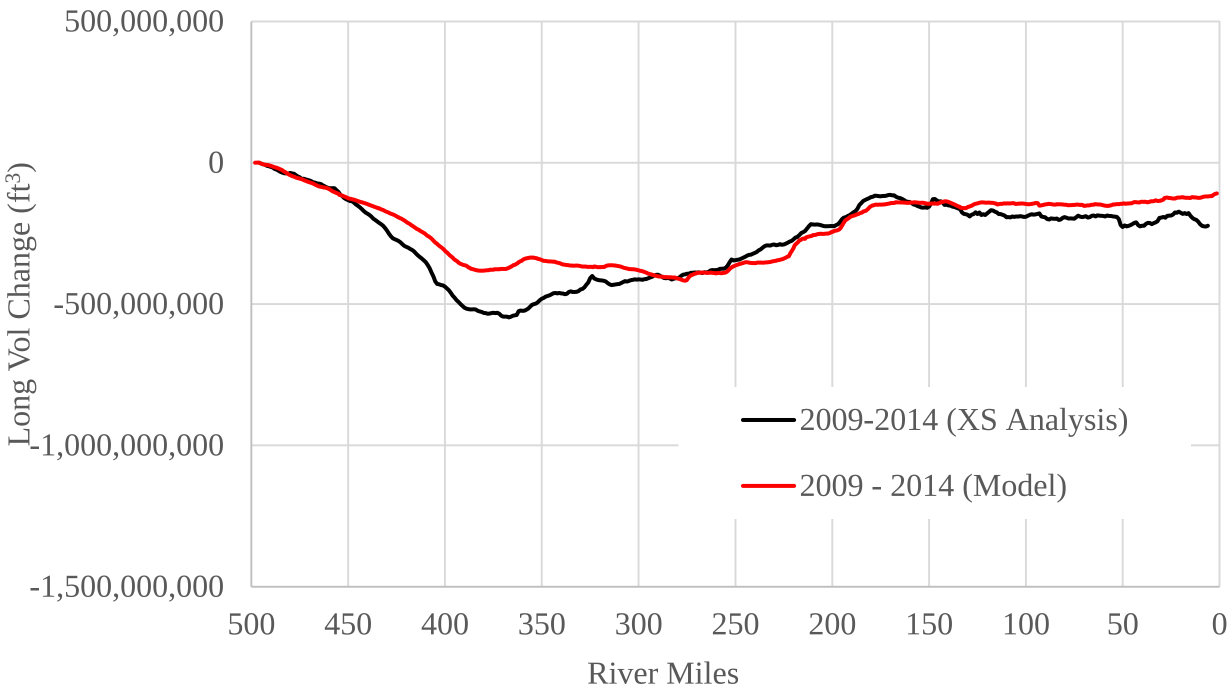


Figure 44. Longitudinal Cumulative Mass Calibration: 2009 to 2014 with additional 23M tons of post-2011 flood sediment

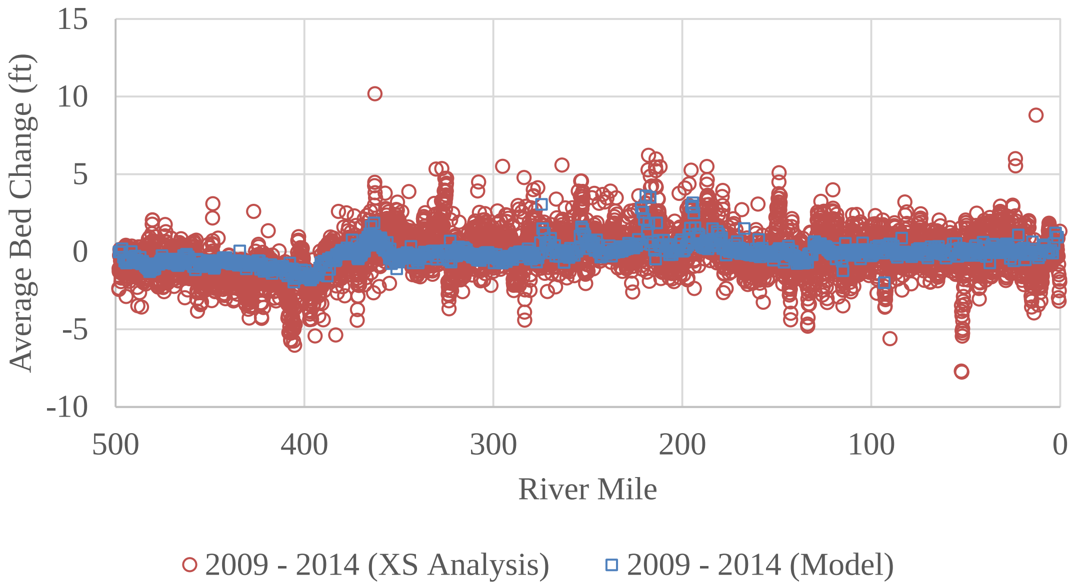


Figure 45. Model vs. Measured Bed Elevation Change 2009 to 2014. Model output includes 23M tons of post-2011 flood sediment.

Conclusion

This report described the mobile-bed model developed for modeling bed change on the lower 500 miles off the Missouri River. It served as an orientation to the inputs, assumptions, and modeling choices that have occurred. The model outputs for water surface, velocity, sediment transport, bed elevation change, and bed volume change over the calibration period reasonably match the prototype using realistic initial conditions and boundary conditions and appropriate model parameters. The model has been calibrated to the Missouri River and is deemed suitable use in MRRP planning.

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Attachment 9b

US Army Corps
of Engineers
Kansas City District

Lower Missouri River Sediment Model
MRRP Alternatives

Aug 2018

1. Introduction

The purpose of this appendix is to document the 2033 projected future bed elevations produced for the Missouri River Recovery Program. The model creation and calibration process are described in the *Lower Missouri River Sediment Model Calibration Report*, referenced herein as the *Model Calibration Appendix*. This appendix describes updates to model geometry, explains the development of the hydrologic boundary condition for the projection period and provides bed elevation results testing six MRRP alternatives.

This analysis produces an adjustment factor to apply to the MRRP Unsteady RAS model, described in the Unsteady HEC-RAS Report, to update the bed from 2009 to 2033 conditions. This is accomplished by dividing the results at the end of 82 years of sedimentation modeling by 82 to create an average annual bed change. The average annual change is multiplied by 24 to compute the average offset from 2009 to 2033. The actual bed elevations in year 2033 will depend on the flows and dredging levels that actually occur between 2014 and 2033, whereas the analysis presented in this appendix represents an average condition.

2. Geometry Updates

The model was built with 1994 bathymetric data and calibrated to water surface, velocity, and bed change from 1994 to July 2014 as described in the *Model Calibration Appendix*. Following model calibration, model bathymetric data was updated to a summer 2014 hydrographic survey. Incorporating the recent survey information provides a more accurate starting geometry than starting with the end-of-simulation geometry from the calibration model. The 2014 bathymetric survey included measured bed transects at 500 ft spacing. The nearest 2014 bathymetric transect was used to update the model bathymetry. Transects were merged into the HEC-RAS cross-sections using geo-referenced stationing. The structure heights were maintained at the same elevations as described in the Model Calibration Appendix.

3. Alternatives

Table 1 summarizes the six MRRP alternatives (from the Unsteady RAS Alternatives Analysis Report). As seen, the six alternatives include one of three different geometries paired with one of six different projection flow scenarios.

Table 1. Alternatives (Table 1-1 from Appendix E)

Alternative	River Geometry (RAS)	Reservoir Flow (Res-Sim)	Adopted Alternative Short Name
Alternative 1	No Action	No Action	No Action
Alternative 2	BiOp	BiOp	BiOp
Alternative 3	IRC	All Mechanical	Mech
Alternative 4	IRC	Spring Bird Flow 2, 42 MAF	Spring 2
Alternative 5	IRC	Fall Bird Flow 5, 35kcfs service level	Fall 5
Alternative 6 *	IRC	Pallid Spawning Cue	Spawn Cue

* Former name was Alternative 7, which may corresponds to some HEC-RAS model runs and file names

4. Geometries

4.1 Overview

The following paragraphs explain the three geometries:

1. **No Action**
   Assumes habitat construction activities follow current practices to achieve 20 acres/mile of SWH, the minimum target specified within the 2003 Amendments to the 2000 Biological Opinion (2003 BiOp) of 20-30 ac/mi (USFWS, Dec 2003). Habitat distributed by 2003 BiOp reaches and the 2014 *Shallow Water Habitat Accounting Report* (USACE, Sept 2014) was used to determine the acreage deficit within each reach to attain the 20 ac/mi goal. Habitat was placed to provide 0 – 5 ft of depth at August 50% exceedance flows. Most of the SWH added to the geometry was in the form of top width widening, the remainder accomplished with chutes.

2. **BiOp As Written/ As Projected (BiOp)**
   Guidance from the US Fish & Wildlife (USFW) documented in *Planning Aid Letter Regarding the Missouri River Recovery Management Plan-EIS: USFWS 2003 BiOp*

Projected Alternative was provided to create a geometry which represents an ideal implementation of the 2003 BiOp. It assumes habitat construction accomplishes 30 ac/mi of SWH, and performs at a wider range of flows. Similar to the No Action geometry, habitat was distributed by 2003 BiOp reaches, and the 2014 *Shallow Water Habitat Accounting Report* was used to determine the acreage deficit within each reach to attain the 30 ac/mi goal. One third of the habitat was set to provide shallow water at summer low flows, one third at median August, and one third at a spring pulse flows. Most of the SWH added to the geometry was in the form of top width widening, the remainder accomplished with chutes. Part of the BiOp requirement was maximizing floodplain connectivity, and a separate analysis was conducted to ensure the requirements were met, although no changes to the HEC-RAS model were necessary.

3. **Interception Rearing Complex (IRC)**
   Assumes construction activities proceed based on findings made by the Effects Analysis (Jacobson, et al., 2016). Total amount of habitat was based on a current annual SWH implementation rate of about 130 ac/year per district for a total of 260 ac/year for 13 years. Distribution was based on conversations with the Effects Analysis team, who specified upper and lower boundaries based on their knowledge of larval pallid spawning locations, drift rates, and timing of interception. Sioux City is the upstream threshold for IRC placement, with the area between the Nebraska Platte River and the Osage River more likely to be successful. Chutes and existing habitat may be modified to meet IRC habitat criteria, but for purposes of HEC-RAS modeling these were not counted toward the target acreage. Habitat was placed to provide 0 – 6 ft of depth at median June flows. All of the SWH goal acreage was accomplished by top width widening.

Table 2 summarizes these geometries. For additional details regarding alternative geometry characteristics, habitat selection criteria, and flow regimes, please see the Missouri River Unsteady HEC-RAS Models Alternative Analysis Draft Report.

Table 2. Summary of Geometries

Characteristic	Geometry		
	No Action	**BiOp**	**IRC**
Target Acres of SWH	20 ac/mi	30 ac/mi	260 ac/yr for 13 years
Basis for SWH Target	Minimum 2003 BiOp target	Full 2003 BiOp target	Current annual SWH implementation rate
Existing Habitat	Counted, used 2014 SWH Report	Counted, used 2014 SWH Report	Not counted
Chutes	Modeled and counted	Modeled and counted	Modeled but not counted
Distribution	20 ac/mi per BiOp reach	30 ac/mi per BiOp reach	Located for optimal interception/retention
Reference Flow	August 50% Exceedance	1/3 Summer Low, 1/3 Median August, and 1/3 Spring Pulse	Median June
SWH Depth Criteria	0-5 ft	0-5 ft	0-6 ft
Additional Requirements	---	Floodplain connectivity	---

The following subsections detail the process of implementing the alternatives into the sediment model.

4.2 Differences Between Unsteady RAS and LMRSM Geometries

Care was taken to maintain consistency with the Unsteady RAS model in the baseline level of shallow water habitat and in how the above alternatives were coded. The LMRSM used the equivalent or nearby cross sections and the same final invert elevations of SWH and chutes as the unsteady RAS model. However, some changes were necessary due to differences in how the models were set up, including differences in LIDAR dates (which influenced the presence or absence of chute projects), cross section locations, and dike representations.

Most cross sections in the LMRSM line up with those in the Unsteady RAS model, but not every cross section included in the Unsteady RAS model has a corresponding LMRSM cross section. Furthermore, the LMRSM sections were occasionally offset in order to line up with locations of bathymetric data. Habitat locations from the Unsteady RAS model were applied to the nearest cross section in the LMRSM, provided the difference in locations was within 0.02 miles.

The two models include river structures such as dikes and revetments differently. The Unsteady RAS model uses permanent ineffective flow areas, whereas the LMRSM uses station elevation points to define the river structures. These differences resulted in a slight alteration to the methodology used to import habitat locations for the development of the three alternative geometries.

4.3 2012 Existing Conditions

For consistency, the chutes were imported from the Unsteady RAS model, which represents the 2012 existing condition, and merged into the LMRSM existing condition geometry. This was implemented to maintain consistency between models and to prevent changes in water surface elevations used to determine habitat locations. The chutes were imported in a manner that maintained the same distance between the main river channel and the center of the chute along with similar elevations.

Additionally, five chute projects (Benedictine Bottoms, Dalby, Cranberry Bend, Jameson Extension, and Cora Island) constructed after the calibration period of the Unsteady RAS model were imported into the sediment model using a standard chute template displayed in Figure 1. A triangular shaped channel bottom 300-ft wide ranging in depth from 0 to 5 feet at the August 50% reference flow was used as the chute template. Chutes were added as overbank conveyance in cross sections that intersect the chute alignment. All chutes were assigned a Manning's n value of 0.03, which is rougher than the main channel, but smoother than the 0.0412 selected for habitat widenings. Both chutes and channel widenings have lower roughness than the floodplain areas they replace. Updating the chutes also required moving ineffective flows and levees, as depicted in Figure 2.

Modifications for the three alternative geometries were made using this 2012 existing conditions geometry as the base.

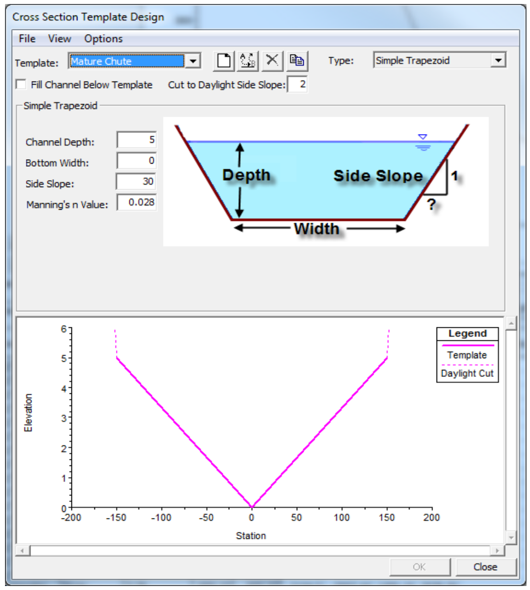

Figure 1. Mature chute bottom template

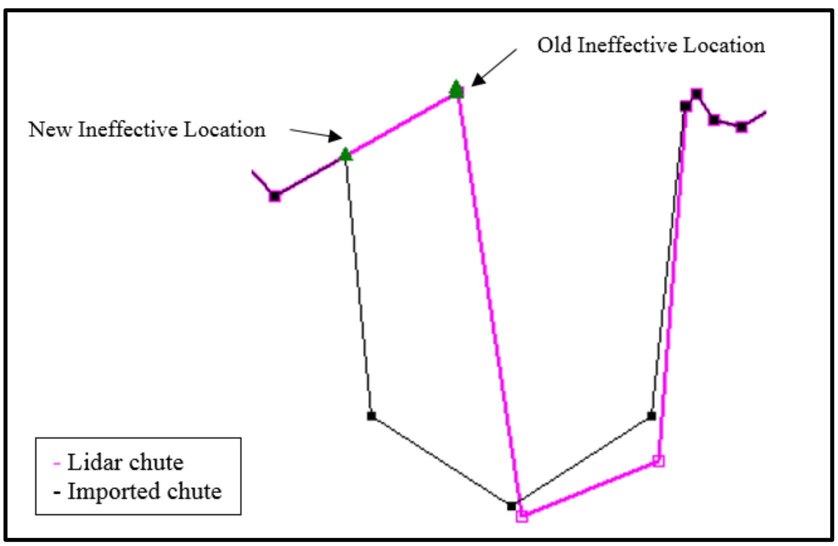

Figure 2. Adjustment to ineffective flow with imported chute

4.4 Top Width Widening

Three alternative geometries were made by applying varying levels of top width widening to the 2012 existing conditions geometry. The elevation for each widening alternative was the required depth below the specified reference water surface as computed by the Unsteady RAS model (see the Unsteady RAS Model Appendix). Ineffective flow and levee points representing levees were set back to the edge of the widened section. Their elevations were left unaltered with the assumption that existing flood protection would be set back to provide the same height of protection rather than removed. Figure 3 provides an example of ineffective flow area adjustments. Bank stations were readjusted to the lowest point of the river structure or to the habitat invert location on a widening.

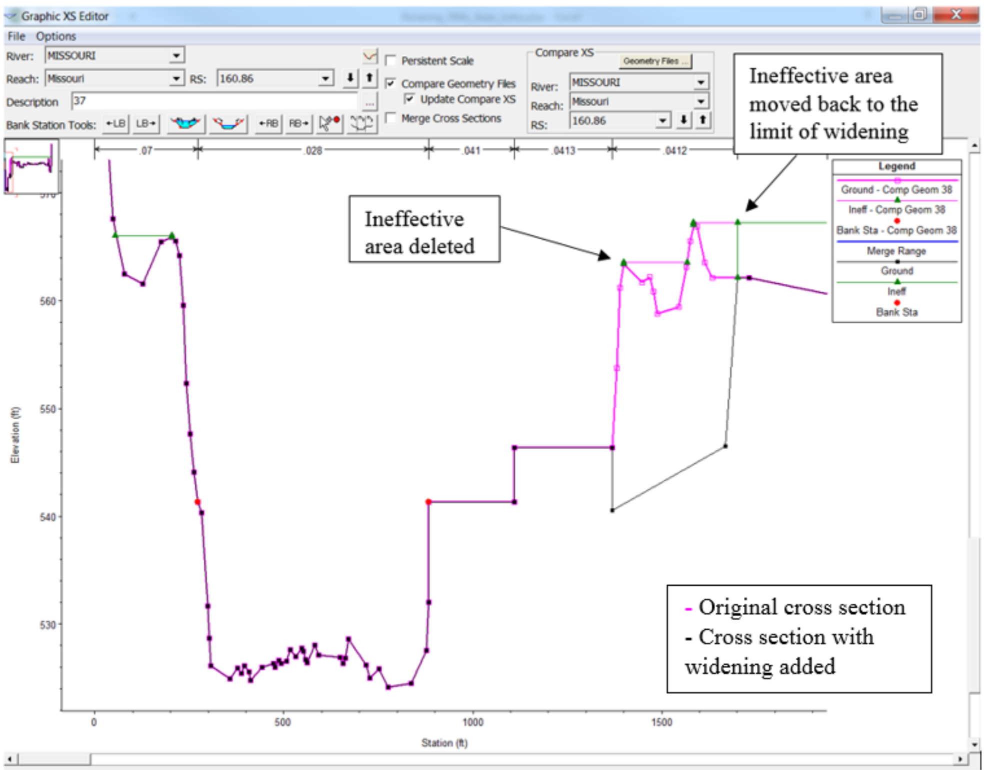

Figure 3. Example ineffective flow adjustment following top width widening

Widened locations in all geometries were assigned a Manning's n-value of 0.0412, which is rougher than the main channel and smoother than the overbanks. As with the Unsteady RAS model, roughness larger than the main channel was selected to account for structure modifications necessary to maintain the navigation channel and habitat at the desired dimensions.

5. Hydrologic Inputs

The sediment modeling uses daily flows developed for each alternative as explained in the Unsteady RAS appendix. Uniform lateral flows were developed from these flows as explained in the Model Calibration Report.

6. Floodplain Deposition

Some floodplain deposition occurs during 22 of the 82 years, with 72% occurring during just the largest three floods (1993, 1951, and 2011). These volumes of sediment removed were computed and entered into the model using the same methodology as described in the Model Calibration Report. As seen in Table 3, the total floodplain deposition differs only slightly between alternatives.

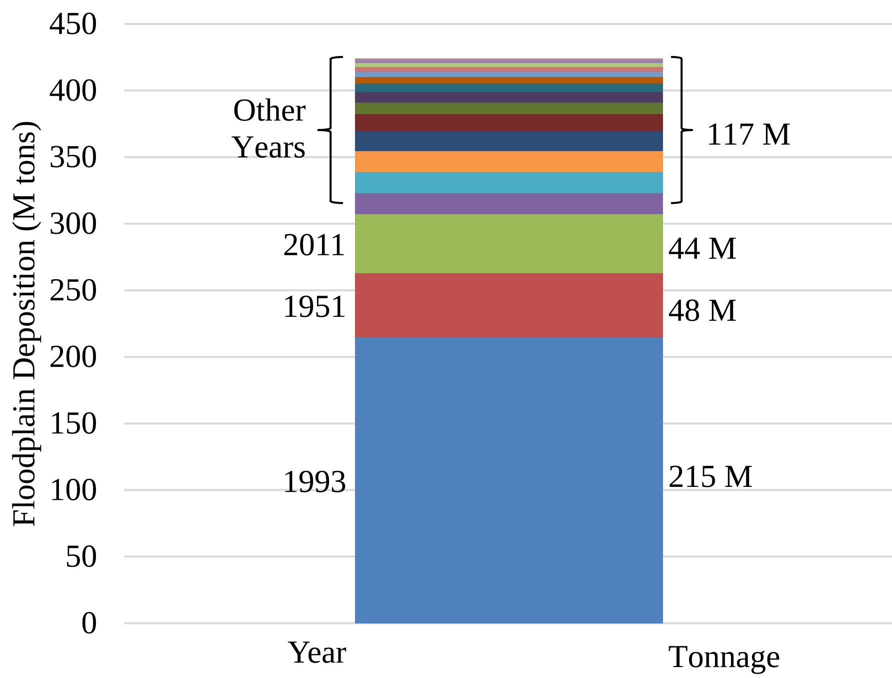

Figure 4. Floodplain Deposition (Alternative 1)

Table 3. Full Simulation (82 Years) Floodplain Deposition

Alternative	Floodplain Deposition (M tons)	Compared to Alternative 1 (%)
1	424	100
2	433	102
3	419	97
4	416	99
5	414	100
6	420	101

7. Dredging Condition

Two model simulation runs allow the modeling of the future commercial dredging. Both include the temporal and spatial distribution within each dredging segment as described in the Model Calibration Report, but differ in the dredging tonnage in each authorized segment. The first simulation includes the currently permitted level of commercial dredging, as defined by the 2015 permit record of decision (USACE 2016). This results in more than 2 ft of degradation (compared to the 2009 regulatory baseline) in much of the St. Joseph dredging segment by year 2033 which, according to the commercial dredging record of decision should trigger some adaptive management. The second simulation includes the tonnage shifted from the St. Joseph segment to the Waverly segment, which is meant to simulate one possible adaptive management strategy. Results from both simulations were annualized. Figure 4 graphs the 2009 – 2033 bed change (i.e. 19 annualized years of the "current dredging" and "shifted dredging" scenarios plus the observed change from 2009 – 2014) in the St. Joseph dredging segment. The MRRP alternatives comparisons uses an "adaptively shifted dredging" which includes 6 years of the "current dredging" and 13 years of the "shifted dredging" simulations. Please note that while some assumption was necessary for MRRP modeling purposes, this is not an official recommendation or prediction for adaptive management decisions by NWK Engineering, Planning, or Regulatory.

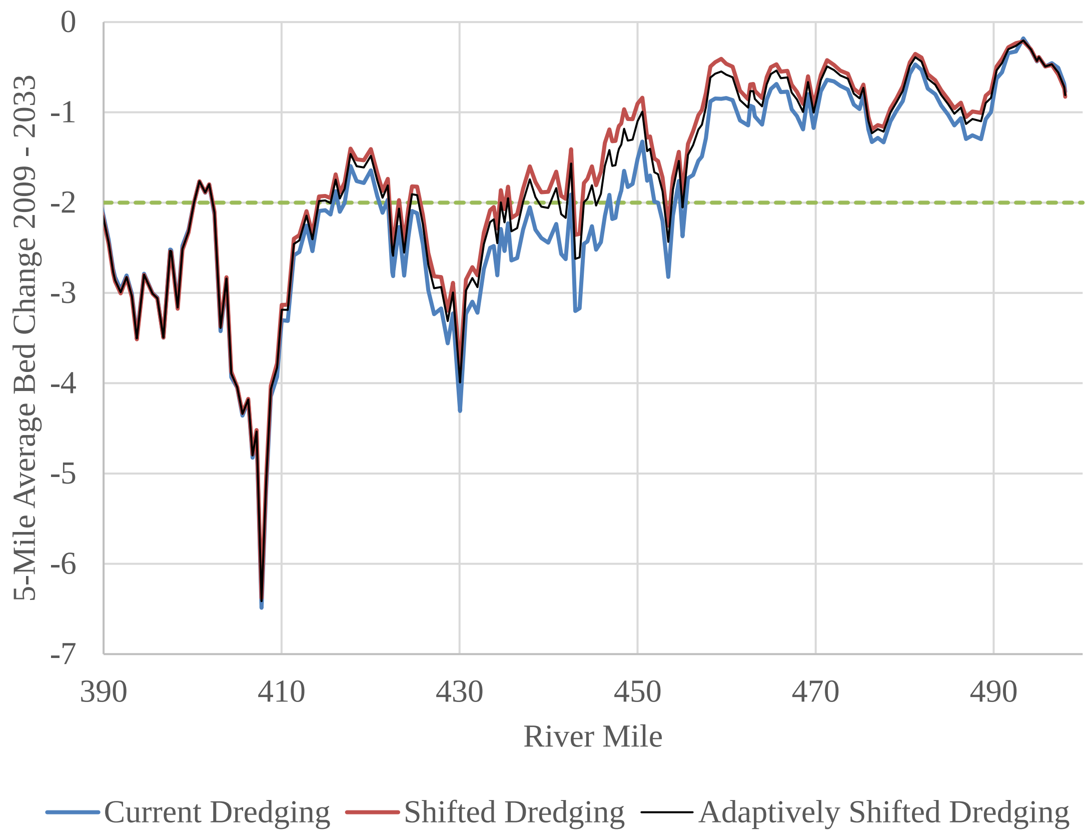

Figure 4. 5-Mile Average Bed Change Projection (2009 – 2033) under Three Dredging Scenarios. Alternative 1.

8. Alternatives Results

Figure 5 plots the 5-mile average bed change under the six alternatives. Each alternative includes measured bed change from 2009 to 2014 plus 6 years of average bed change under the "current dredging" condition plus 13 years of average bed change under the "shifted dredging" condition. Attachment 1 provides the bed changes at the MRRP Unsteady RAS model locations for each alternative in tabular form. Note that while the changes from 2009 to 2033 are fairly large, the differences amongst MRRP alternatives are fairly small.

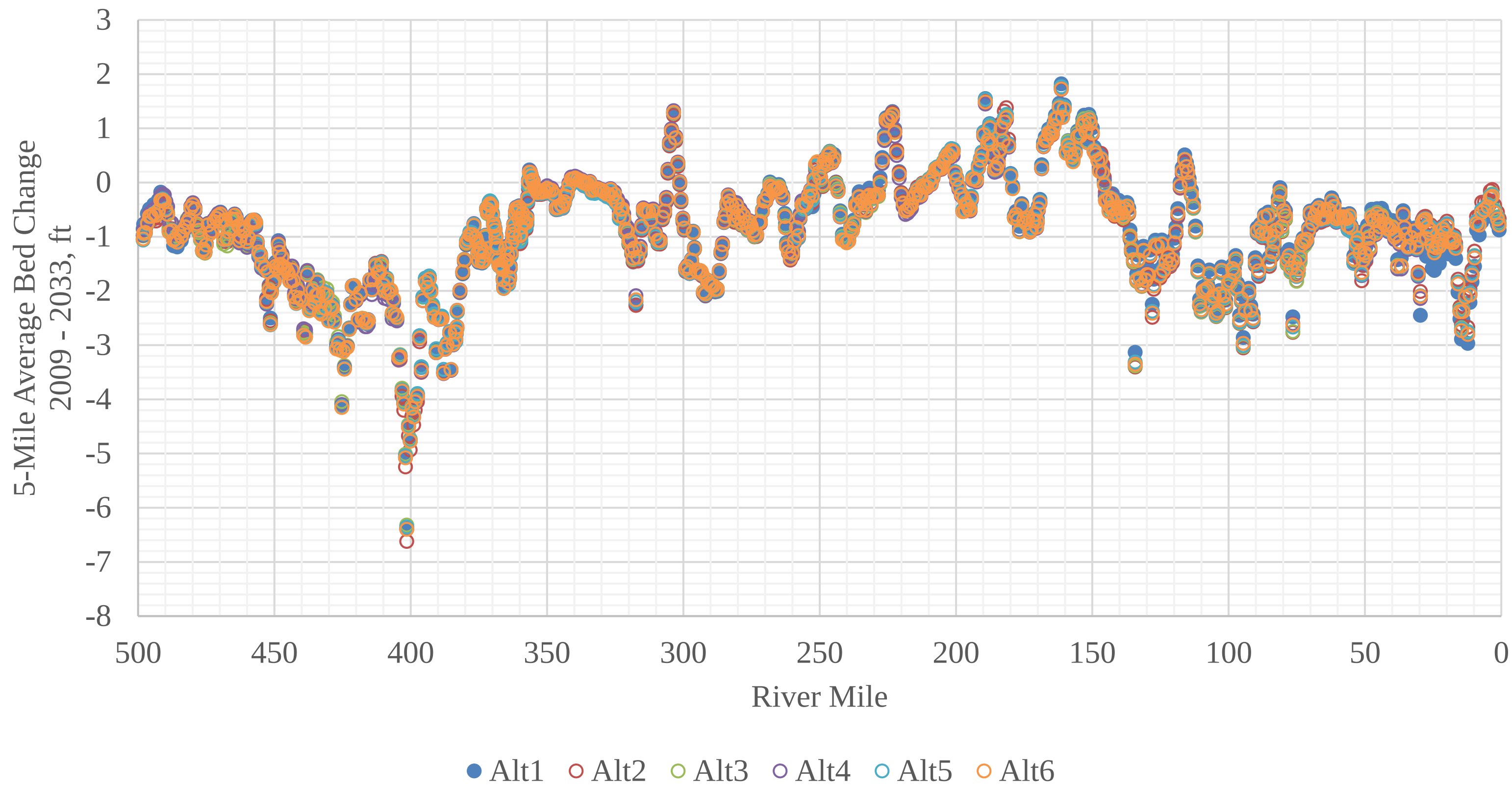

Figure 5. Alternatives Comparison

9. Comparison to Unsteady RAS 2033 Adjustment

The Unsteady RAS model team incorporated an earlier version (Feb 2018) of the sediment model output into their 2033 assessments. The updated results presented in this Appendix (July 2018) follow the same basic trends as the earlier output, as seen in Figure 6. Because the trends are similar, the project delivery team opted to use the earlier output for the 2033 Unsteady RAS model runs.

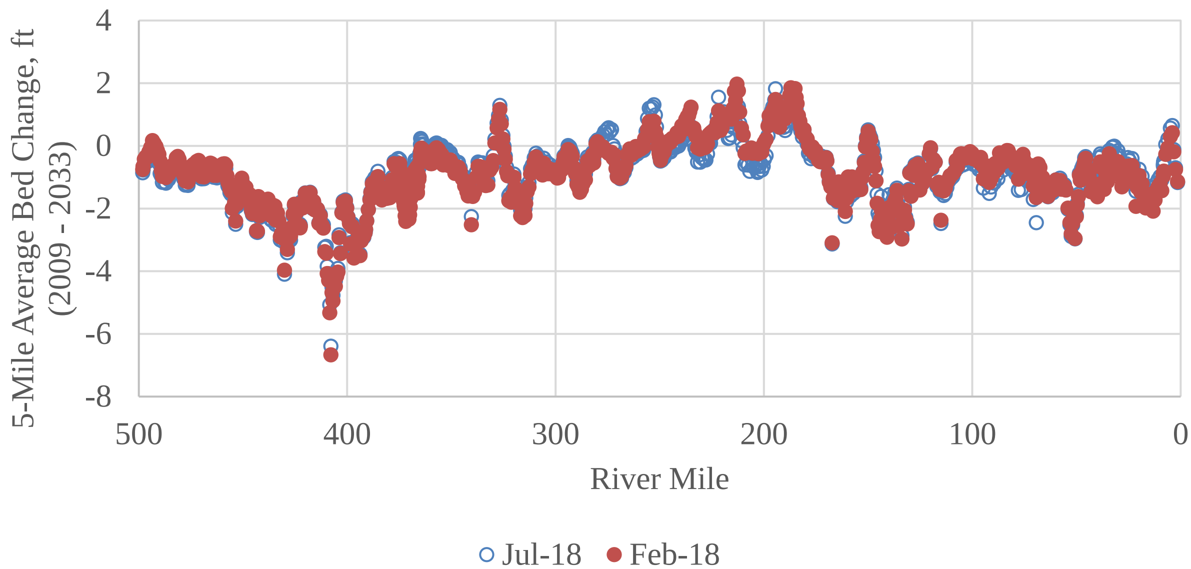

Figure 6. Comparison of Updated Sediment Model Output with Output Used in Unsteady RAS Appendix

10. Conclusions

This analysis indicates that none of the proposed MRRP alternatives will appreciably affect the long-term, reach-scale sediment condition of the river. The bed of the Missouri River has changed and will continue to change significantly since 2009 regardless of the alternatives.

11. References

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USACE (2015b). A Sediment Budget Approach to Stable Dredging Levels on the Lower Missouri River, 1994 to 2014. Memorandum for CENWK-OR-R. October 2, 2015. U.S. Army Corps of Engineers, Kansas City District.

Attachment 1- Five-mile Average Bed Change from 2009 to 2033 (ft)

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
498.1	-0.86	-1.05	-0.99	-0.97	-1.06	-1.04
497.93	-0.77	-0.94	-0.89	-0.88	-0.96	-0.94
497.25	-0.76	-0.93	-0.87	-0.86	-0.94	-0.92
496.53	-0.57	-0.76	-0.71	-0.70	-0.77	-0.75
495.79	-0.49	-0.67	-0.58	-0.57	-0.64	-0.63
495.07	-0.51	-0.69	-0.58	-0.57	-0.62	-0.63
494.19	-0.41	-0.67	-0.51	-0.50	-0.57	-0.58
493.34	-0.45	-0.71	-0.53	-0.51	-0.60	-0.61
492.5	-0.32	-0.65	-0.43	-0.41	-0.51	-0.52
491.64	-0.18	-0.42	-0.32	-0.25	-0.33	-0.33
490.95	-0.26	-0.42	-0.31	-0.21	-0.31	-0.32
490.33	-0.30	-0.39	-0.32	-0.23	-0.34	-0.35
489.71	-0.47	-0.52	-0.45	-0.35	-0.49	-0.51
489.11	-0.56	-0.58	-0.55	-0.45	-0.59	-0.59
488.59	-0.89	-0.85	-0.87	-0.72	-0.92	-0.92
487.61	-0.96	-0.86	-0.79	-0.74	-0.87	-0.89
486.89	-1.17	-1.05	-0.99	-0.92	-1.05	-1.07
486.32	-1.15	-1.03	-1.02	-0.95	-1.03	-1.05
485.59	-1.19	-1.09	-1.09	-1.04	-1.09	-1.10
484.89	-1.00	-0.94	-0.94	-0.92	-0.92	-0.93
484.11	-1.07	-1.00	-1.00	-0.98	-0.99	-1.00
483.45	-0.96	-0.90	-0.89	-0.87	-0.89	-0.90
482.6	-0.86	-0.82	-0.78	-0.78	-0.82	-0.82
481.89	-0.74	-0.72	-0.68	-0.69	-0.73	-0.74
481.2	-0.69	-0.67	-0.63	-0.63	-0.67	-0.68
480.59	-0.48	-0.47	-0.45	-0.44	-0.47	-0.47

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
479.81	-0.43	-0.42	-0.39	-0.38	-0.41	-0.41
479.09	-0.53	-0.52	-0.50	-0.46	-0.50	-0.49
478.4	-0.82	-0.81	-0.79	-0.74	-0.77	-0.76
477.64	-0.93	-0.94	-0.93	-0.87	-0.89	-0.88
476.99	-1.06	-1.06	-1.06	-1.03	-1.03	-1.03
476.34	-1.25	-1.25	-1.28	-1.26	-1.26	-1.26
475.92	-1.22	-1.23	-1.25	-1.24	-1.24	-1.23
475.38	-1.26	-1.28	-1.31	-1.29	-1.29	-1.29
474.94	-1.12	-1.14	-1.15	-1.15	-1.15	-1.15
474.29	-0.76	-0.77	-0.78	-0.78	-0.79	-0.79
473.62	-0.89	-0.91	-0.94	-0.94	-0.95	-0.95
472.78	-0.85	-0.86	-0.88	-0.87	-0.90	-0.90
472.06	-0.68	-0.69	-0.71	-0.70	-0.73	-0.72
471.28	-0.64	-0.66	-0.64	-0.62	-0.67	-0.67
470.58	-0.58	-0.60	-0.59	-0.57	-0.62	-0.61
469.77	-0.55	-0.58	-0.59	-0.56	-0.59	-0.58
469.14	-0.69	-0.72	-0.74	-0.71	-0.72	-0.72
468.58	-1.06	-1.09	-1.14	-1.05	-1.06	-1.05
467.9	-0.70	-0.73	-0.79	-0.71	-0.71	-0.70
467.31	-1.05	-1.10	-1.17	-1.05	-1.06	-1.04
466.82	-0.91	-0.96	-1.02	-0.91	-0.93	-0.91
466.09	-0.84	-0.89	-0.93	-0.83	-0.86	-0.85
465.6	-0.66	-0.70	-0.72	-0.65	-0.67	-0.67
464.97	-0.67	-0.70	-0.72	-0.66	-0.69	-0.69
464.51	-0.58	-0.62	-0.64	-0.59	-0.61	-0.61
463.97	-0.63	-0.68	-0.69	-0.68	-0.68	-0.68

Attachment 1- Five-mile Average Bed Change from 2009 to 2033 (ft)

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6	RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
463.17	-0.74	-0.79	-0.83	-0.82	-0.80	-0.80	448.49	-1.08	-1.11	-1.11	-1.11	-1.15	-1.14
462.66	-1.00	-1.06	-1.09	-1.12	-1.08	-1.08	448.15	-1.22	-1.26	-1.24	-1.25	-1.30	-1.29
462.41	-0.92	-0.97	-1.03	-1.05	-1.00	-0.99	447.78	-1.45	-1.47	-1.45	-1.47	-1.52	-1.51
461.5	-0.83	-0.88	-0.93	-0.96	-0.90	-0.89	447.51	-1.48	-1.49	-1.47	-1.47	-1.53	-1.53
460.68	-0.83	-0.87	-0.92	-0.94	-0.89	-0.88	447.16	-1.34	-1.35	-1.32	-1.32	-1.38	-1.38
459.97	-1.02	-1.07	-1.17	-1.20	-1.11	-1.09	446.83	-1.52	-1.53	-1.51	-1.50	-1.56	-1.56
459.4	-0.95	-1.01	-1.14	-1.15	-1.04	-1.02	446.33	-1.53	-1.54	-1.51	-1.51	-1.57	-1.57
458.72	-0.73	-0.79	-0.88	-0.88	-0.79	-0.77	445.88	-1.54	-1.54	-1.52	-1.52	-1.58	-1.58
458.18	-0.70	-0.75	-0.83	-0.82	-0.75	-0.72	445.33	-1.59	-1.59	-1.56	-1.56	-1.62	-1.63
457.68	-0.70	-0.76	-0.82	-0.81	-0.74	-0.69	444.86	-1.64	-1.64	-1.61	-1.60	-1.68	-1.68
457.23	-0.69	-0.74	-0.80	-0.79	-0.74	-0.71	444.36	-1.78	-1.78	-1.75	-1.75	-1.82	-1.82
456.82	-0.73	-0.78	-0.81	-0.82	-0.77	-0.74	443.98	-1.78	-1.79	-1.76	-1.74	-1.83	-1.83
456.22	-1.09	-1.15	-1.20	-1.21	-1.15	-1.10	443.49	-1.58	-1.58	-1.56	-1.54	-1.62	-1.61
455.64	-1.27	-1.33	-1.36	-1.37	-1.32	-1.28	442.99	-1.77	-1.77	-1.76	-1.74	-1.82	-1.81
455.05	-1.32	-1.37	-1.41	-1.42	-1.37	-1.34	442.53	-2.08	-2.07	-2.06	-2.03	-2.12	-2.11
454.63	-1.49	-1.54	-1.57	-1.58	-1.54	-1.50	441.92	-2.19	-2.19	-2.18	-2.15	-2.23	-2.23
453.44	-1.54	-1.56	-1.60	-1.62	-1.59	-1.58	441.42	-1.96	-1.94	-1.95	-1.90	-1.99	-1.99
452.8	-2.13	-2.16	-2.21	-2.25	-2.21	-2.20	440.85	-2.09	-2.08	-2.09	-2.04	-2.13	-2.13
452.31	-1.58	-1.60	-1.64	-1.65	-1.65	-1.65	439.96	-2.12	-2.11	-2.13	-2.07	-2.17	-2.17
451.88	-1.90	-1.93	-1.97	-1.99	-1.98	-1.99	439.19	-2.75	-2.75	-2.77	-2.70	-2.82	-2.82
451.41	-2.51	-2.56	-2.59	-2.61	-2.63	-2.62	438.53	-2.77	-2.78	-2.80	-2.72	-2.84	-2.86
451.09	-1.94	-1.98	-2.01	-2.03	-2.04	-2.04	437.88	-1.65	-1.66	-1.67	-1.62	-1.69	-1.71
450.52	-1.76	-1.80	-1.82	-1.83	-1.86	-1.85	437.14	-2.28	-2.29	-2.31	-2.25	-2.34	-2.37
449.99	-1.74	-1.78	-1.78	-1.80	-1.84	-1.82	436.46	-2.23	-2.25	-2.26	-2.21	-2.28	-2.31
449.44	-1.47	-1.51	-1.51	-1.52	-1.56	-1.55	435.82	-1.93	-1.96	-1.96	-1.92	-1.99	-1.99
448.89	-1.51	-1.54	-1.54	-1.55	-1.60	-1.59	435.395	-2.15	-2.17	-2.19	-2.15	-2.20	-2.21

Attachment 1- Five-mile Average Bed Change from 2009 to 2033 (ft)

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6	RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
435.01	-2.17	-2.20	-2.19	-2.17	-2.21	-2.23	419.24	-2.52	-2.57	-2.53	-2.58	-2.54	-2.54
434.61	-2.04	-2.05	-2.04	-2.03	-2.06	-2.09	418.44	-2.04	-2.06	-2.03	-2.08	-2.05	-2.04
434.21	-1.82	-1.81	-1.80	-1.82	-1.82	-1.84	417.73	-2.51	-2.54	-2.51	-2.57	-2.53	-2.51
433.805	-2.06	-2.06	-2.03	-2.06	-2.05	-2.08	417.09	-2.52	-2.55	-2.51	-2.58	-2.53	-2.52
433.44	-2.36	-2.37	-2.31	-2.36	-2.37	-2.39	416.55	-2.57	-2.60	-2.58	-2.66	-2.60	-2.58
432.73	-2.39	-2.39	-2.31	-2.38	-2.38	-2.43	416.07	-2.57	-2.60	-2.58	-2.66	-2.60	-2.58
432.01	-2.01	-2.02	-1.94	-2.01	-2.01	-2.05	415.5	-2.53	-2.56	-2.55	-2.62	-2.56	-2.55
431.44	-2.28	-2.30	-2.20	-2.27	-2.28	-2.32	414.9	-1.79	-1.81	-1.80	-1.86	-1.81	-1.80
430.71	-2.05	-2.06	-1.97	-2.04	-2.05	-2.08	414.2	-1.97	-1.98	-1.98	-2.07	-1.99	-1.98
430.04	-2.51	-2.54	-2.42	-2.51	-2.52	-2.56	413.47	-1.78	-1.79	-1.80	-1.89	-1.80	-1.80
429.27	-2.25	-2.28	-2.18	-2.25	-2.27	-2.30	412.8	-1.50	-1.49	-1.54	-1.63	-1.51	-1.53
428.65	-2.27	-2.31	-2.21	-2.28	-2.29	-2.33	412.01	-1.63	-1.60	-1.70	-1.80	-1.63	-1.67
427.92	-2.52	-2.56	-2.46	-2.52	-2.55	-2.58	411.39	-1.62	-1.58	-1.70	-1.80	-1.62	-1.65
427.13	-3.00	-3.05	-2.95	-3.01	-3.04	-3.07	410.68	-1.48	-1.46	-1.55	-1.63	-1.48	-1.51
426.48	-2.90	-2.94	-2.85	-2.91	-2.93	-2.97	410.01	-1.89	-1.86	-1.95	-2.05	-1.86	-1.90
425.87	-3.05	-3.08	-3.00	-3.05	-3.08	-3.09	409.49	-2.00	-1.98	-2.06	-2.14	-1.97	-2.00
425.22	-4.10	-4.12	-4.04	-4.10	-4.13	-4.15	408.79	-1.78	-1.77	-1.83	-1.91	-1.76	-1.79
424.66	-3.08	-3.09	-3.04	-3.08	-3.11	-3.12	408.26	-2.04	-2.03	-2.08	-2.16	-2.01	-2.04
424.24	-3.41	-3.42	-3.38	-3.41	-3.43	-3.45	407.74	-2.00	-2.01	-2.04	-2.10	-1.98	-1.99
423.77	-3.03	-3.05	-3.02	-3.04	-3.06	-3.06	407.19	-2.02	-2.02	-2.04	-2.09	-2.00	-1.99
423.2	-3.01	-3.03	-3.00	-3.02	-3.04	-3.05	406.75	-2.46	-2.46	-2.46	-2.52	-2.43	-2.41
422.439	-2.69	-2.70	-2.69	-2.71	-2.72	-2.71	406.25	-2.19	-2.18	-2.19	-2.24	-2.17	-2.15
421.93	-2.24	-2.25	-2.25	-2.26	-2.25	-2.26	405.6	-2.48	-2.47	-2.48	-2.52	-2.46	-2.44
421.35	-1.91	-1.93	-1.92	-1.93	-1.93	-1.92	405.01	-2.52	-2.52	-2.52	-2.55	-2.50	-2.49
420.75	-1.90	-1.92	-1.91	-1.93	-1.92	-1.91	404.36	-3.24	-3.28	-3.23	-3.27	-3.22	-3.23
420.04	-2.15	-2.18	-2.16	-2.19	-2.16	-2.17	403.81	-3.21	-3.27	-3.18	-3.22	-3.19	-3.21

Attachment 1- Five-mile Average Bed Change from 2009 to 2033 (ft)

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6	RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
403.14	-3.85	-3.95	-3.80	-3.84	-3.83	-3.85	385.66	-2.77	-2.77	-2.79	-2.79	-2.75	-2.78
402.47	-4.08	-4.21	-4.03	-4.07	-4.06	-4.09	385.15	-3.46	-3.46	-3.47	-3.47	-3.45	-3.45
401.86	-5.08	-5.25	-5.02	-5.07	-5.04	-5.08	384.4	-2.99	-2.98	-3.00	-3.00	-2.99	-2.99
401.41	-6.40	-6.62	-6.32	-6.38	-6.36	-6.40	383.65	-2.77	-2.76	-2.77	-2.76	-2.78	-2.76
400.76	-4.51	-4.68	-4.47	-4.51	-4.50	-4.52	383.41	-2.92	-2.91	-2.92	-2.90	-2.94	-2.90
400.15	-4.77	-4.94	-4.73	-4.77	-4.75	-4.77	383.24	-2.79	-2.78	-2.78	-2.76	-2.80	-2.76
399.53	-4.16	-4.31	-4.13	-4.17	-4.15	-4.17	383.01	-2.69	-2.69	-2.69	-2.66	-2.70	-2.67
398.86	-4.32	-4.47	-4.29	-4.33	-4.30	-4.33	382.89	-2.39	-2.39	-2.39	-2.36	-2.38	-2.36
398.31	-4.04	-4.20	-4.02	-4.07	-4.03	-4.05	381.89	-2.04	-2.02	-2.00	-1.99	-2.01	-1.99
397.477	-3.91	-4.05	-3.90	-3.96	-3.90	-3.95	380.87	-1.69	-1.67	-1.66	-1.64	-1.65	-1.65
396.71	-2.84	-2.94	-2.84	-2.89	-2.83	-2.88	380.28	-1.47	-1.45	-1.44	-1.42	-1.42	-1.43
396.04	-3.42	-3.51	-3.40	-3.49	-3.40	-3.47	379.53	-1.15	-1.13	-1.13	-1.10	-1.08	-1.11
395.5	-2.11	-2.18	-2.12	-2.18	-2.11	-2.18	378.85	-1.10	-1.07	-1.06	-1.05	-1.02	-1.05
394.54	-1.77	-1.83	-1.78	-1.83	-1.77	-1.84	378.26	-1.02	-0.98	-0.97	-0.96	-0.92	-0.96
393.74	-1.86	-1.92	-1.87	-1.92	-1.86	-1.93	377.43	-1.00	-0.96	-0.95	-0.93	-0.93	-0.96
393.18	-1.73	-1.77	-1.74	-1.79	-1.73	-1.79	376.76	-0.82	-0.78	-0.79	-0.76	-0.77	-0.79
392.59	-1.96	-2.00	-1.97	-2.02	-1.96	-2.02	376.11	-1.06	-1.01	-1.04	-0.98	-1.00	-1.03
391.93	-2.27	-2.31	-2.28	-2.32	-2.26	-2.32	375.71	-1.12	-1.07	-1.09	-1.05	-1.06	-1.10
391.29	-2.44	-2.49	-2.47	-2.49	-2.43	-2.49	375.33	-1.32	-1.27	-1.29	-1.24	-1.26	-1.30
390.57	-3.09	-3.14	-3.12	-3.14	-3.07	-3.14	375.07	-1.47	-1.41	-1.44	-1.38	-1.41	-1.45
389.75	-2.50	-2.52	-2.52	-2.53	-2.48	-2.53	374.76	-1.27	-1.21	-1.24	-1.19	-1.22	-1.25
389.01	-2.49	-2.51	-2.51	-2.52	-2.47	-2.52	374.43	-1.23	-1.17	-1.20	-1.16	-1.18	-1.21
388.51	-2.49	-2.51	-2.51	-2.52	-2.47	-2.52	374.02	-1.24	-1.19	-1.22	-1.17	-1.20	-1.23
387.86	-3.49	-3.51	-3.52	-3.53	-3.45	-3.52	373.79	-1.49	-1.43	-1.46	-1.42	-1.45	-1.47
387.32	-3.06	-3.07	-3.08	-3.08	-3.03	-3.08	373.46	-1.38	-1.34	-1.37	-1.34	-1.33	-1.36
386.62	-2.98	-2.99	-3.01	-3.01	-2.96	-3.00	373.15	-1.46	-1.42	-1.45	-1.42	-1.40	-1.44

Attachment 1- Five-mile Average Bed Change from 2009 to 2033 (ft)

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6	RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
372.86	-1.30	-1.26	-1.30	-1.28	-1.28	-1.29	364.65	-1.44	-1.43	-1.45	-1.41	-1.45	-1.44
372.47	-1.01	-0.97	-1.01	-0.99	-0.98	-0.99	364.34	-1.33	-1.33	-1.34	-1.31	-1.35	-1.33
372.13	-0.50	-0.47	-0.50	-0.48	-0.47	-0.49	364.05	-1.85	-1.86	-1.87	-1.82	-1.89	-1.85
371.83	-0.57	-0.54	-0.58	-0.55	-0.53	-0.56	363.73	-1.75	-1.77	-1.77	-1.73	-1.79	-1.76
371.47	-0.53	-0.49	-0.53	-0.51	-0.48	-0.52	363.43	-1.56	-1.57	-1.58	-1.54	-1.59	-1.56
371.14	-0.42	-0.36	-0.42	-0.39	-0.36	-0.41	363.14	-1.30	-1.34	-1.33	-1.29	-1.35	-1.31
370.83	-0.40	-0.35	-0.40	-0.38	-0.34	-0.39	362.85	-1.10	-1.15	-1.14	-1.09	-1.16	-1.11
370.54	-0.43	-0.37	-0.43	-0.40	-0.37	-0.42	362.54	-0.94	-1.00	-0.99	-0.93	-1.01	-0.96
370.26	-0.44	-0.40	-0.44	-0.42	-0.39	-0.43	362.33	-0.87	-0.95	-0.93	-0.88	-0.94	-0.89
369.96	-0.61	-0.56	-0.61	-0.59	-0.56	-0.60	362.17	-0.77	-0.85	-0.83	-0.77	-0.84	-0.78
369.66	-0.77	-0.73	-0.77	-0.75	-0.72	-0.76	361.84	-0.98	-1.11	-1.08	-1.00	-1.10	-1.01
369.38	-0.82	-0.78	-0.82	-0.80	-0.77	-0.81	361.54	-0.54	-0.62	-0.60	-0.55	-0.61	-0.56
369.05	-0.89	-0.85	-0.89	-0.87	-0.84	-0.88	361.23	-0.46	-0.54	-0.52	-0.47	-0.53	-0.48
368.73	-1.02	-0.98	-1.02	-1.00	-0.97	-1.01	360.91	-0.46	-0.54	-0.52	-0.47	-0.53	-0.48
368.48	-1.17	-1.13	-1.16	-1.15	-1.12	-1.15	360.6	-0.50	-0.60	-0.57	-0.52	-0.58	-0.52
368.19	-1.22	-1.18	-1.21	-1.20	-1.18	-1.21	360.29	-0.63	-0.75	-0.71	-0.66	-0.73	-0.66
367.89	-1.45	-1.41	-1.43	-1.42	-1.40	-1.43	360.15	-0.43	-0.52	-0.49	-0.46	-0.50	-0.45
367.57	-1.54	-1.50	-1.54	-1.52	-1.50	-1.53	359.84	-0.94	-1.13	-1.06	-1.00	-1.09	-0.99
367.03	-1.54	-1.50	-1.53	-1.52	-1.50	-1.53	359.55	-0.92	-1.11	-1.04	-0.98	-1.07	-0.96
366.75	-1.55	-1.52	-1.55	-1.54	-1.52	-1.54	359.39	-0.79	-0.96	-0.90	-0.84	-0.92	-0.83
366.48	-1.58	-1.55	-1.58	-1.56	-1.54	-1.57	359.26	-0.62	-0.76	-0.71	-0.66	-0.73	-0.65
366.23	-1.61	-1.58	-1.61	-1.59	-1.58	-1.61	358.91	-0.54	-0.66	-0.61	-0.57	-0.64	-0.57
366.06	-1.80	-1.76	-1.80	-1.77	-1.77	-1.80	358.57	-0.71	-0.85	-0.79	-0.75	-0.81	-0.74
365.84	-1.96	-1.92	-1.95	-1.92	-1.93	-1.95	358.26	-0.70	-0.84	-0.78	-0.74	-0.80	-0.73
365.43	-1.45	-1.43	-1.45	-1.42	-1.44	-1.45	357.94	-0.71	-0.85	-0.79	-0.75	-0.81	-0.74
365.1	-1.21	-1.20	-1.22	-1.18	-1.21	-1.21	357.63	-0.71	-0.86	-0.80	-0.75	-0.82	-0.75

Attachment 1- Five-mile Average Bed Change from 2009 to 2033 (ft)

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6	RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
357.3	-0.62	-0.77	-0.71	-0.67	-0.73	-0.66	343.57	-0.35	-0.39	-0.36	-0.32	-0.38	-0.35
357.06	-0.26	-0.39	-0.32	-0.29	-0.35	-0.29	343.09	-0.25	-0.29	-0.26	-0.22	-0.28	-0.25
356.76	0.05	-0.12	-0.03	0.02	-0.06	0.01	342.49	-0.17	-0.22	-0.18	-0.13	-0.21	-0.17
356.43	0.23	0.05	0.14	0.20	0.11	0.19	341.88	-0.04	-0.09	-0.06	-0.01	-0.08	-0.05
356.1	0.20	0.10	0.17	0.20	0.14	0.19	341.06	0.07	0.01	0.05	0.10	0.02	0.06
355.76	0.14	0.04	0.11	0.14	0.08	0.12	340.32	0.07	0.02	0.05	0.09	0.02	0.06
355.39	0.06	-0.02	0.04	0.07	0.01	0.05	339.66	0.09	0.04	0.08	0.12	0.04	0.08
355.01	0.03	-0.03	0.02	0.04	-0.01	0.02	338.79	0.07	0.02	0.06	0.09	0.02	0.06
354.62	-0.07	-0.14	-0.08	-0.06	-0.11	-0.09	337.96	0.04	-0.01	0.04	0.07	0.00	0.03
354.2	-0.11	-0.17	-0.12	-0.10	-0.14	-0.12	337.11	0.03	-0.01	0.03	0.05	0.00	0.03
353.82	-0.12	-0.17	-0.13	-0.10	-0.15	-0.13	336.5	0.00	-0.06	-0.01	0.02	-0.06	-0.01
353.45	-0.18	-0.22	-0.18	-0.16	-0.20	-0.19	335.67	-0.01	-0.07	-0.03	0.00	-0.07	-0.02
353.16	-0.17	-0.21	-0.18	-0.15	-0.20	-0.18	335	-0.02	-0.09	-0.04	-0.01	-0.09	-0.03
352.8	-0.17	-0.21	-0.17	-0.15	-0.19	-0.18	334.24	0.01	-0.03	0.00	0.02	-0.03	0.00
352.57	-0.17	-0.22	-0.18	-0.14	-0.20	-0.18	333.44	-0.13	-0.19	-0.16	-0.13	-0.20	-0.15
351.93	-0.18	-0.22	-0.19	-0.15	-0.20	-0.18	332.39	-0.12	-0.19	-0.15	-0.12	-0.21	-0.14
351.15	-0.14	-0.18	-0.15	-0.12	-0.16	-0.15	331.48	-0.10	-0.15	-0.12	-0.10	-0.16	-0.11
350.5	-0.16	-0.20	-0.17	-0.13	-0.19	-0.17	330.54	-0.12	-0.17	-0.14	-0.12	-0.18	-0.13
349.9	-0.10	-0.14	-0.11	-0.06	-0.13	-0.11	329.83	-0.14	-0.18	-0.15	-0.13	-0.19	-0.15
349.2	-0.14	-0.18	-0.15	-0.11	-0.18	-0.15	329.05	-0.17	-0.21	-0.19	-0.16	-0.23	-0.18
348.25	-0.14	-0.18	-0.15	-0.11	-0.18	-0.15	327.98	-0.21	-0.25	-0.23	-0.20	-0.27	-0.22
347.39	-0.21	-0.25	-0.22	-0.18	-0.25	-0.22	327.28	-0.21	-0.26	-0.23	-0.20	-0.27	-0.22
346.54	-0.47	-0.50	-0.48	-0.44	-0.50	-0.47	326.66	-0.13	-0.16	-0.14	-0.12	-0.17	-0.13
345.68	-0.47	-0.50	-0.47	-0.43	-0.49	-0.46	325.96	-0.27	-0.29	-0.28	-0.24	-0.32	-0.27
344.74	-0.41	-0.44	-0.41	-0.37	-0.44	-0.41	325.2	-0.22	-0.23	-0.23	-0.18	-0.26	-0.22
344.13	-0.44	-0.47	-0.44	-0.40	-0.47	-0.44	324.64	-0.36	-0.35	-0.36	-0.30	-0.39	-0.35

Attachment 1- Five-mile Average Bed Change from 2009 to 2033 (ft)

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
324.08	-0.40	-0.38	-0.39	-0.31	-0.42	-0.38
323.46	-0.65	-0.62	-0.64	-0.53	-0.67	-0.61
322.87	-0.54	-0.52	-0.53	-0.43	-0.55	-0.50
322.36	-0.53	-0.52	-0.52	-0.42	-0.53	-0.49
321.8	-0.70	-0.70	-0.69	-0.58	-0.68	-0.65
321.11	-0.93	-0.92	-0.90	-0.79	-0.88	-0.86
320.42	-0.96	-0.95	-0.94	-0.83	-0.92	-0.90
319.92	-1.15	-1.13	-1.11	-0.99	-1.08	-1.07
319.4	-1.14	-1.13	-1.10	-0.98	-1.07	-1.07
318.9	-1.32	-1.32	-1.29	-1.18	-1.27	-1.26
318.38	-1.47	-1.46	-1.43	-1.32	-1.41	-1.40
317.84	-1.27	-1.27	-1.24	-1.17	-1.23	-1.21
317.322	-2.26	-2.27	-2.20	-2.09	-2.20	-2.16
316.64	-1.44	-1.45	-1.40	-1.33	-1.40	-1.38
315.79	-1.27	-1.31	-1.25	-1.18	-1.27	-1.23
315.18	-0.85	-0.88	-0.85	-0.78	-0.85	-0.81
314.61	-0.52	-0.56	-0.52	-0.47	-0.53	-0.50
314.24	-0.54	-0.57	-0.55	-0.50	-0.55	-0.52
313.55	-0.56	-0.60	-0.57	-0.52	-0.58	-0.54
312.68	-0.55	-0.59	-0.57	-0.51	-0.57	-0.53
311.91	-0.72	-0.76	-0.73	-0.65	-0.73	-0.69
311.06	-0.55	-0.58	-0.56	-0.49	-0.56	-0.52
310.17	-0.99	-1.03	-1.00	-0.91	-1.00	-0.95
309.3	-1.11	-1.14	-1.11	-1.04	-1.10	-1.07
308.45	-1.12	-1.14	-1.11	-1.04	-1.10	-1.07
307.6	-0.67	-0.69	-0.66	-0.62	-0.66	-0.64

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
306.81	-0.55	-0.58	-0.54	-0.50	-0.54	-0.53
306.16	-0.32	-0.36	-0.31	-0.28	-0.32	-0.30
305.6	0.21	0.17	0.21	0.24	0.20	0.22
305	0.72	0.67	0.72	0.75	0.70	0.72
304.3	0.97	0.93	0.97	0.99	0.95	0.96
303.5	1.29	1.24	1.28	1.32	1.29	1.29
302.8	0.82	0.76	0.81	0.85	0.81	0.82
301.97	0.33	0.29	0.33	0.37	0.34	0.36
301.25	-0.01	-0.04	-0.01	0.02	-0.01	0.00
300.83	-0.33	-0.36	-0.33	-0.31	-0.33	-0.32
300.13	-0.68	-0.73	-0.69	-0.66	-0.69	-0.67
299.62	-0.84	-0.88	-0.85	-0.83	-0.86	-0.84
299.12	-1.59	-1.64	-1.61	-1.59	-1.62	-1.59
298.45	-1.53	-1.58	-1.55	-1.54	-1.57	-1.55
297.66	-1.64	-1.68	-1.66	-1.65	-1.68	-1.66
296.95	-1.42	-1.46	-1.43	-1.44	-1.45	-1.44
296.35	-0.90	-0.93	-0.91	-0.92	-0.93	-0.92
295.85	-1.20	-1.24	-1.21	-1.22	-1.23	-1.22
295.25	-1.60	-1.64	-1.61	-1.62	-1.63	-1.62
294.53	-1.64	-1.67	-1.65	-1.65	-1.66	-1.66
293.84	-1.63	-1.65	-1.63	-1.63	-1.63	-1.62
293.45	-1.66	-1.67	-1.65	-1.64	-1.65	-1.64
293.08	-1.71	-1.72	-1.70	-1.70	-1.70	-1.69
292.6	-2.05	-2.05	-2.01	-2.03	-2.03	-2.02
291.85	-2.10	-2.08	-2.05	-2.06	-2.07	-2.06
291.05	-1.85	-1.83	-1.80	-1.81	-1.83	-1.81

Attachment 1- Five-mile Average Bed Change from 2009 to 2033 (ft)

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6	RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
290.33	-1.91	-1.90	-1.86	-1.87	-1.89	-1.87	273.87	-0.96	-1.00	-1.00	-0.97	-0.98	-0.97
289.69	-1.94	-1.93	-1.89	-1.90	-1.92	-1.90	273.06	-0.95	-1.00	-0.98	-0.96	-0.97	-0.96
288.94	-1.95	-1.93	-1.90	-1.90	-1.92	-1.90	272.33	-0.71	-0.76	-0.73	-0.75	-0.74	-0.74
288.25	-2.02	-2.00	-1.97	-1.97	-2.00	-1.98	271.71	-0.70	-0.76	-0.72	-0.75	-0.73	-0.74
287.5	-2.01	-1.99	-1.96	-1.97	-1.99	-1.97	270.96	-0.50	-0.54	-0.51	-0.54	-0.53	-0.53
286.78	-1.67	-1.66	-1.63	-1.63	-1.65	-1.63	270.3	-0.32	-0.36	-0.34	-0.36	-0.34	-0.35
286.15	-1.32	-1.31	-1.28	-1.27	-1.30	-1.28	269.68	-0.30	-0.35	-0.32	-0.35	-0.33	-0.34
285.56	-1.17	-1.16	-1.14	-1.12	-1.15	-1.14	268.99	-0.20	-0.25	-0.23	-0.26	-0.23	-0.24
285.06	-0.74	-0.74	-0.72	-0.70	-0.74	-0.73	268.19	0.01	-0.05	-0.03	-0.08	-0.05	-0.07
284.53	-0.62	-0.65	-0.63	-0.59	-0.62	-0.61	267.5	-0.02	-0.09	-0.06	-0.11	-0.08	-0.09
283.98	-0.39	-0.45	-0.42	-0.38	-0.41	-0.39	266.92	-0.09	-0.15	-0.12	-0.16	-0.14	-0.14
283.5	-0.24	-0.31	-0.27	-0.23	-0.26	-0.25	266.16	-0.07	-0.13	-0.10	-0.14	-0.12	-0.12
282.88	-0.32	-0.38	-0.36	-0.31	-0.35	-0.34	265.58	-0.07	-0.13	-0.10	-0.14	-0.12	-0.12
282.11	-0.41	-0.47	-0.44	-0.36	-0.43	-0.43	264.96	-0.04	-0.10	-0.07	-0.11	-0.10	-0.10
281.61	-0.53	-0.59	-0.56	-0.49	-0.55	-0.55	264.31	-0.13	-0.19	-0.17	-0.20	-0.18	-0.18
281.14	-0.64	-0.74	-0.70	-0.60	-0.68	-0.68	263.47	-0.24	-0.29	-0.27	-0.30	-0.28	-0.28
280.51	-0.40	-0.44	-0.43	-0.37	-0.42	-0.42	262.7	-0.57	-0.63	-0.61	-0.64	-0.63	-0.63
279.83	-0.61	-0.66	-0.65	-0.57	-0.64	-0.64	262.515	-0.76	-0.84	-0.80	-0.83	-0.83	-0.83
279.13	-0.52	-0.56	-0.56	-0.49	-0.55	-0.54	262.1	-1.11	-1.19	-1.15	-1.17	-1.18	-1.19
278.6	-0.73	-0.79	-0.77	-0.70	-0.76	-0.75	261.38	-1.22	-1.30	-1.26	-1.26	-1.29	-1.28
277.9	-0.60	-0.65	-0.65	-0.58	-0.62	-0.62	260.7	-1.35	-1.43	-1.39	-1.37	-1.40	-1.40
277.12	-0.72	-0.77	-0.77	-0.71	-0.75	-0.73	259.88	-1.25	-1.36	-1.32	-1.25	-1.30	-1.30
276.36	-0.83	-0.88	-0.88	-0.81	-0.86	-0.85	259.19	-1.03	-1.12	-1.08	-1.05	-1.11	-1.08
275.7	-0.72	-0.76	-0.75	-0.71	-0.75	-0.73	258.46	-0.75	-0.88	-0.84	-0.78	-0.86	-0.83
275.1	-0.79	-0.83	-0.82	-0.78	-0.82	-0.80	257.73	-0.90	-1.03	-0.99	-0.92	-1.02	-0.98
274.46	-0.88	-0.93	-0.91	-0.87	-0.91	-0.89	257.14	-0.58	-0.67	-0.65	-0.59	-0.66	-0.64

Attachment 1- Five-mile Average Bed Change from 2009 to 2033 (ft)

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6	RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
256.47	-0.35	-0.43	-0.40	-0.34	-0.43	-0.41	238.54	-0.81	-0.85	-0.81	-0.84	-0.84	-0.85
255.8	-0.41	-0.51	-0.47	-0.39	-0.50	-0.43	238	-0.85	-0.90	-0.87	-0.89	-0.89	-0.89
254.96	-0.49	-0.47	-0.41	-0.35	-0.47	-0.39	237.4	-0.70	-0.76	-0.75	-0.78	-0.77	-0.77
254.04	-0.35	-0.33	-0.27	-0.22	-0.34	-0.26	236.69	-0.36	-0.41	-0.41	-0.43	-0.42	-0.43
253.31	-0.22	-0.19	-0.15	-0.14	-0.20	-0.15	236.05	-0.34	-0.41	-0.42	-0.42	-0.41	-0.42
252.67	-0.45	-0.38	-0.27	-0.19	-0.33	-0.25	235.47	-0.17	-0.25	-0.27	-0.26	-0.26	-0.27
252.02	-0.21	-0.08	0.02	0.08	-0.03	0.05	234.92	-0.40	-0.49	-0.54	-0.52	-0.51	-0.51
251.45	0.13	0.23	0.28	0.33	0.28	0.34	234.32	-0.24	-0.32	-0.36	-0.35	-0.34	-0.34
250.85	0.18	0.27	0.32	0.36	0.31	0.38	233.62	-0.27	-0.37	-0.42	-0.41	-0.40	-0.39
250.23	0.00	0.06	0.10	0.13	0.12	0.18	233.03	-0.35	-0.49	-0.56	-0.54	-0.52	-0.52
249.59	0.05	0.03	0.05	0.09	0.06	0.08	232.43	-0.18	-0.26	-0.29	-0.28	-0.27	-0.27
248.85	-0.06	-0.08	-0.05	-0.02	-0.03	-0.02	231.71	-0.11	-0.22	-0.26	-0.22	-0.24	-0.24
248.4	0.33	0.29	0.31	0.36	0.36	0.37	231.07	-0.27	-0.39	-0.44	-0.38	-0.42	-0.40
247.83	0.45	0.39	0.39	0.41	0.42	0.42	230.41	-0.14	-0.26	-0.30	-0.25	-0.28	-0.27
247.32	0.36	0.32	0.35	0.35	0.34	0.34	229.7	-0.18	-0.25	-0.28	-0.25	-0.26	-0.25
246.77	0.53	0.50	0.52	0.50	0.51	0.50	229	-0.13	-0.21	-0.24	-0.21	-0.23	-0.22
246.2	0.57	0.54	0.56	0.52	0.54	0.53	228.4	-0.15	-0.22	-0.26	-0.23	-0.24	-0.23
245.39	0.44	0.40	0.40	0.36	0.38	0.38	227.77	0.09	0.01	-0.03	0.00	-0.01	0.00
244.7	0.52	0.48	0.46	0.43	0.45	0.44	227	0.46	0.41	0.36	0.35	0.40	0.40
243.96	0.01	0.00	-0.02	-0.07	-0.04	-0.06	226.16	0.87	0.83	0.78	0.77	0.83	0.82
243.19	-0.10	-0.11	-0.12	-0.17	-0.15	-0.16	225.53	1.19	1.16	1.12	1.10	1.16	1.15
242.4	-0.52	-0.56	-0.56	-0.64	-0.59	-0.63	224.81	1.18	1.15	1.12	1.09	1.14	1.14
241.63	-0.96	-1.01	-1.00	-1.05	-1.02	-1.06	224.06	1.24	1.22	1.17	1.13	1.20	1.19
240.91	-0.97	-1.02	-1.01	-1.06	-1.04	-1.06	223.27	1.31	1.28	1.24	1.18	1.25	1.26
240.08	-1.05	-1.10	-1.06	-1.10	-1.09	-1.11	222.4	0.98	0.96	0.92	0.85	0.93	0.92
239.32	-1.03	-1.08	-1.03	-1.06	-1.07	-1.08	221.69	0.60	0.59	0.55	0.50	0.55	0.55

Attachment 1- Five-mile Average Bed Change from 2009 to 2033 (ft)

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6	RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
220.78	0.20	0.18	0.15	0.08	0.15	0.15	201.6	0.62	0.59	0.62	0.53	0.62	0.59
220.03	-0.19	-0.19	-0.22	-0.28	-0.22	-0.22	200.99	0.61	0.57	0.60	0.49	0.61	0.57
219.31	-0.35	-0.35	-0.38	-0.45	-0.39	-0.38	200.29	0.20	0.18	0.19	0.13	0.20	0.17
218.49	-0.49	-0.49	-0.52	-0.59	-0.53	-0.53	199.71	0.09	0.06	0.09	0.00	0.09	0.05
217.69	-0.46	-0.47	-0.50	-0.57	-0.50	-0.50	198.89	-0.08	-0.10	-0.08	-0.13	-0.07	-0.09
217	-0.40	-0.40	-0.45	-0.53	-0.45	-0.45	197.96	-0.20	-0.22	-0.22	-0.25	-0.19	-0.23
216.3	-0.38	-0.38	-0.42	-0.48	-0.41	-0.42	197.43	-0.52	-0.52	-0.53	-0.54	-0.50	-0.54
215.48	-0.24	-0.23	-0.27	-0.34	-0.27	-0.28	197.035	-0.30	-0.30	-0.30	-0.31	-0.28	-0.30
214.72	-0.19	-0.17	-0.20	-0.27	-0.21	-0.22	196.54	-0.52	-0.52	-0.52	-0.53	-0.50	-0.53
214.09	-0.11	-0.08	-0.12	-0.19	-0.13	-0.14	196.29	-0.39	-0.39	-0.40	-0.40	-0.36	-0.39
213.53	-0.19	-0.15	-0.20	-0.28	-0.21	-0.24	195.69	-0.42	-0.43	-0.44	-0.42	-0.39	-0.43
212.86	-0.20	-0.17	-0.20	-0.26	-0.21	-0.24	195.13	-0.46	-0.48	-0.49	-0.52	-0.46	-0.50
212.2	-0.03	-0.01	-0.03	-0.08	-0.04	-0.06	194.76	-0.45	-0.47	-0.49	-0.53	-0.44	-0.50
211.55	-0.06	-0.06	-0.05	-0.11	-0.06	-0.09	194.33	-0.26	-0.28	-0.30	-0.33	-0.25	-0.30
210.71	-0.06	-0.08	-0.05	-0.11	-0.06	-0.09	193.84	0.10	0.07	0.05	0.03	0.09	0.06
209.84	0.06	0.03	0.06	0.02	0.06	0.03	193.29	0.11	0.08	0.06	0.04	0.10	0.07
209.06	-0.01	-0.04	0.01	-0.03	0.00	-0.02	192.39	0.08	0.05	0.02	0.01	0.08	0.03
208.34	0.10	0.08	0.13	0.10	0.12	0.10	191.7	0.32	0.29	0.27	0.26	0.32	0.27
207.45	0.23	0.20	0.26	0.22	0.26	0.24	190.95	0.45	0.41	0.39	0.39	0.44	0.41
206.72	0.27	0.24	0.29	0.26	0.29	0.26	190.42	0.56	0.52	0.50	0.50	0.56	0.52
205.98	0.26	0.24	0.28	0.25	0.28	0.26	189.78	0.93	0.89	0.86	0.85	0.92	0.88
205.3	0.26	0.24	0.28	0.25	0.28	0.26	189.16	1.55	1.50	1.46	1.45	1.53	1.48
204.5	0.36	0.35	0.39	0.35	0.38	0.37	188.39	0.76	0.75	0.72	0.71	0.77	0.72
203.68	0.47	0.44	0.49	0.44	0.49	0.46	188.06	0.83	0.81	0.79	0.78	0.82	0.78
202.98	0.52	0.50	0.54	0.49	0.54	0.51	187.55	1.08	1.06	1.03	1.01	1.06	1.00
202.24	0.49	0.47	0.51	0.45	0.51	0.48	186.97	0.77	0.77	0.75	0.72	0.76	0.71

Attachment 1- Five-mile Average Bed Change from 2009 to 2033 (ft)

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6	RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
186.35	0.49	0.52	0.48	0.44	0.49	0.51	170.13	-0.65	-0.70	-0.72	-0.74	-0.69	-0.72
185.79	0.23	0.25	0.21	0.19	0.23	0.24	169.73	-0.44	-0.49	-0.50	-0.52	-0.47	-0.49
185.024	0.25	0.27	0.23	0.20	0.25	0.27	169.13	-0.31	-0.37	-0.36	-0.39	-0.35	-0.37
184.44	0.35	0.42	0.32	0.30	0.35	0.35	168.54	0.32	0.26	0.27	0.25	0.28	0.26
183.97	0.58	0.68	0.56	0.53	0.59	0.60	167.81	0.74	0.66	0.68	0.66	0.68	0.66
183.41	0.97	1.09	0.95	0.91	0.98	1.00	167.21	0.84	0.76	0.78	0.77	0.79	0.77
182.69	0.74	0.84	0.71	0.68	0.75	0.77	166.55	0.87	0.79	0.81	0.80	0.82	0.80
182.2	1.18	1.31	1.15	1.09	1.19	1.11	165.91	0.98	0.90	0.93	0.92	0.94	0.92
181.49	1.25	1.38	1.22	1.15	1.25	1.19	165.22	0.97	0.88	0.91	0.90	0.92	0.90
180.68	0.72	0.80	0.69	0.65	0.70	0.67	164.61	0.96	0.88	0.90	0.89	0.91	0.89
179.9	0.16	0.13	0.13	0.13	0.14	0.12	163.99	1.11	1.04	1.06	1.05	1.06	1.04
179.15	-0.07	-0.11	-0.11	-0.10	-0.10	-0.12	163.45	1.24	1.16	1.19	1.17	1.19	1.16
178.52	-0.61	-0.65	-0.66	-0.65	-0.65	-0.66	162.7	1.26	1.18	1.21	1.20	1.21	1.19
177.91	-0.53	-0.58	-0.60	-0.58	-0.59	-0.59	162.04	1.46	1.37	1.41	1.39	1.41	1.38
177.28	-0.61	-0.67	-0.69	-0.67	-0.68	-0.68	161.35	1.82	1.71	1.76	1.74	1.76	1.72
176.72	-0.81	-0.89	-0.91	-0.88	-0.89	-0.90	160.86	1.27	1.20	1.23	1.22	1.24	1.20
175.86	-0.39	-0.43	-0.45	-0.42	-0.43	-0.44	160.26	1.43	1.35	1.39	1.37	1.39	1.35
175.08	-0.64	-0.69	-0.70	-0.69	-0.68	-0.69	159.47	0.59	0.54	0.58	0.55	0.56	0.54
174.26	-0.69	-0.74	-0.76	-0.74	-0.73	-0.75	158.82	0.77	0.71	0.76	0.72	0.72	0.69
173.71	-0.60	-0.65	-0.67	-0.66	-0.65	-0.66	158.31	0.69	0.62	0.68	0.64	0.64	0.60
172.92	-0.85	-0.90	-0.92	-0.92	-0.87	-0.91	157.63	0.59	0.51	0.56	0.52	0.51	0.48
172.35	-0.82	-0.86	-0.88	-0.88	-0.84	-0.87	156.99	0.48	0.41	0.46	0.42	0.41	0.38
171.77	-0.66	-0.70	-0.72	-0.72	-0.68	-0.71	156.23	0.63	0.56	0.60	0.57	0.55	0.53
171.3	-0.66	-0.70	-0.71	-0.71	-0.68	-0.71	155.37	0.95	0.87	0.91	0.88	0.86	0.85
170.88	-0.55	-0.58	-0.60	-0.60	-0.57	-0.60	154.4	0.95	0.86	0.91	0.87	0.86	0.86
170.44	-0.78	-0.81	-0.84	-0.85	-0.80	-0.83	153.45	1.16	1.05	1.11	1.07	1.06	1.05

Attachment 1- Five-mile Average Bed Change from 2009 to 2033 (ft)

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6	RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
152.93	1.24	1.12	1.18	1.13	1.13	1.12	138.1	-0.39	-0.52	-0.49	-0.47	-0.47	-0.48
152.36	1.02	0.95	0.97	0.94	0.95	0.94	137.24	-0.37	-0.47	-0.45	-0.43	-0.43	-0.44
151.74	0.80	0.74	0.76	0.73	0.74	0.74	136.61	-0.47	-0.58	-0.56	-0.54	-0.54	-0.55
151.18	1.25	1.15	1.19	1.16	1.18	1.16	136.1	-0.88	-1.04	-1.01	-0.99	-0.97	-0.99
150.68	1.17	1.07	1.10	1.08	1.10	1.08	135.61	-1.12	-1.29	-1.26	-1.23	-1.22	-1.24
149.92	1.01	0.89	0.92	0.91	0.92	0.91	134.89	-1.30	-1.48	-1.47	-1.43	-1.42	-1.43
149.3	0.65	0.57	0.59	0.59	0.59	0.58	134.25	-3.13	-3.41	-3.39	-3.34	-3.31	-3.36
148.66	0.59	0.52	0.53	0.53	0.54	0.51	133.66	-1.68	-1.83	-1.82	-1.80	-1.78	-1.80
147.9	0.47	0.43	0.42	0.42	0.42	0.40	133.12	-1.34	-1.45	-1.44	-1.42	-1.42	-1.43
147.33	0.28	0.27	0.22	0.23	0.22	0.20	132.47	-1.66	-1.78	-1.78	-1.76	-1.76	-1.77
146.8	0.51	0.54	0.45	0.47	0.45	0.43	131.84	-1.79	-1.92	-1.91	-1.88	-1.89	-1.90
146.12	0.28	0.33	0.23	0.25	0.19	0.18	131.19	-1.18	-1.27	-1.26	-1.25	-1.25	-1.25
145.57	0.06	0.10	0.03	0.04	-0.02	-0.04	130.37	-1.68	-1.82	-1.81	-1.78	-1.79	-1.80
145.16	-0.24	-0.18	-0.26	-0.25	-0.32	-0.34	129.9	-1.59	-1.73	-1.69	-1.68	-1.70	-1.70
144.6	-0.27	-0.24	-0.30	-0.28	-0.33	-0.34	129.29	-1.59	-1.74	-1.70	-1.69	-1.71	-1.71
143.818	-0.41	-0.41	-0.47	-0.44	-0.52	-0.54	128.76	-1.25	-1.37	-1.34	-1.32	-1.33	-1.35
143.56	-0.21	-0.22	-0.25	-0.23	-0.28	-0.29	127.98	-2.25	-2.49	-2.41	-2.38	-2.38	-2.41
143.14	-0.35	-0.43	-0.41	-0.38	-0.44	-0.46	127.37	-1.79	-1.97	-1.91	-1.89	-1.89	-1.91
142.58	-0.21	-0.32	-0.30	-0.26	-0.30	-0.33	126.83	-1.07	-1.19	-1.14	-1.13	-1.13	-1.16
142.16	-0.39	-0.50	-0.48	-0.45	-0.47	-0.49	126.04	-1.10	-1.22	-1.17	-1.16	-1.16	-1.18
141.73	-0.46	-0.63	-0.59	-0.55	-0.58	-0.60	125.12	-1.60	-1.77	-1.71	-1.70	-1.69	-1.72
140.81	-0.44	-0.61	-0.57	-0.53	-0.54	-0.58	124.47	-1.06	-1.17	-1.13	-1.12	-1.12	-1.14
140.33	-0.33	-0.49	-0.46	-0.41	-0.43	-0.45	123.84	-1.52	-1.66	-1.62	-1.60	-1.60	-1.63
139.84	-0.36	-0.53	-0.50	-0.45	-0.47	-0.49	122.97	-1.37	-1.49	-1.45	-1.45	-1.45	-1.47
139.2	-0.44	-0.61	-0.58	-0.53	-0.54	-0.56	122.09	-1.27	-1.38	-1.35	-1.35	-1.34	-1.36
138.62	-0.50	-0.69	-0.65	-0.62	-0.61	-0.63	121.46	-1.40	-1.55	-1.53	-1.52	-1.50	-1.50

Attachment 1- Five-mile Average Bed Change from 2009 to 2033 (ft)

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6	RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
120.56	-1.33	-1.48	-1.44	-1.45	-1.42	-1.43	102.55	-1.56	-1.67	-1.66	-1.65	-1.64	-1.62
119.93	-1.03	-1.17	-1.13	-1.15	-1.10	-1.12	101.67	-2.02	-2.16	-2.14	-2.12	-2.12	-2.09
119.25	-0.82	-0.96	-0.92	-0.94	-0.90	-0.91	101	-2.17	-2.31	-2.30	-2.27	-2.27	-2.25
118.58	-0.48	-0.64	-0.59	-0.61	-0.57	-0.58	100.04	-1.78	-1.93	-1.92	-1.89	-1.88	-1.86
117.7	0.00	-0.11	-0.07	-0.09	-0.06	-0.07	99.28	-1.73	-1.88	-1.86	-1.83	-1.83	-1.81
116.95	0.27	0.16	0.20	0.19	0.21	0.20	98.48	-1.52	-1.65	-1.64	-1.61	-1.61	-1.59
116.02	0.51	0.37	0.41	0.40	0.43	0.42	97.84	-1.59	-1.73	-1.72	-1.69	-1.70	-1.67
115.39	0.37	0.25	0.28	0.28	0.29	0.29	97.37	-1.35	-1.47	-1.45	-1.43	-1.44	-1.41
114.96	0.26	0.13	0.15	0.16	0.18	0.18	96.7	-1.86	-1.98	-1.97	-1.95	-1.96	-1.93
114.45	0.08	-0.04	-0.03	-0.01	-0.01	0.00	95.9	-2.45	-2.60	-2.59	-2.55	-2.58	-2.54
113.65	-0.14	-0.25	-0.25	-0.23	-0.23	-0.21	95.29	-2.07	-2.20	-2.18	-2.14	-2.17	-2.15
112.88	-0.37	-0.48	-0.48	-0.46	-0.45	-0.44	94.6	-2.86	-3.05	-3.03	-2.97	-3.01	-2.97
112.09	-0.80	-0.91	-0.92	-0.89	-0.89	-0.88	93.85	-2.28	-2.42	-2.41	-2.36	-2.39	-2.36
111.21	-1.54	-1.65	-1.65	-1.63	-1.63	-1.62	92.54	-1.96	-2.07	-2.05	-2.01	-2.05	-2.01
110.57	-2.15	-2.29	-2.29	-2.26	-2.26	-2.24	91.81	-2.28	-2.42	-2.39	-2.36	-2.41	-2.36
110	-2.27	-2.39	-2.39	-2.37	-2.37	-2.35	90.94	-2.43	-2.57	-2.55	-2.51	-2.55	-2.51
109.29	-1.93	-2.05	-2.05	-2.03	-2.03	-2.01	90.18	-1.39	-1.53	-1.50	-1.47	-1.51	-1.47
108.48	-1.91	-2.03	-2.03	-2.01	-2.01	-1.98	89.46	-0.84	-0.94	-0.92	-0.90	-0.93	-0.90
107.65	-1.88	-1.99	-1.99	-1.97	-1.97	-1.95	88.92	-1.53	-1.73	-1.69	-1.67	-1.70	-1.63
107.03	-1.62	-1.72	-1.72	-1.70	-1.70	-1.68	88.25	-0.77	-0.90	-0.87	-0.85	-0.88	-0.83
106.23	-2.14	-2.26	-2.26	-2.24	-2.24	-2.22	87.64	-0.88	-1.02	-0.98	-0.97	-0.99	-0.95
105.63	-2.03	-2.14	-2.14	-2.12	-2.12	-2.10	86.82	-0.59	-0.69	-0.66	-0.65	-0.66	-0.64
105.21	-2.03	-2.14	-2.14	-2.12	-2.12	-2.09	86.11	-0.85	-1.03	-0.98	-0.95	-0.98	-0.93
104.49	-2.34	-2.47	-2.47	-2.45	-2.45	-2.42	85.46	-0.55	-0.66	-0.63	-0.61	-0.63	-0.60
103.86	-2.29	-2.42	-2.41	-2.40	-2.39	-2.37	85	-1.38	-1.55	-1.51	-1.48	-1.50	-1.46
103.19	-1.99	-2.11	-2.10	-2.09	-2.09	-2.06	84.57	-0.90	-1.05	-1.01	-0.98	-1.01	-0.98

Attachment 1- Five-mile Average Bed Change from 2009 to 2033 (ft)

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6	RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
83.89	-1.20	-1.36	-1.33	-1.28	-1.32	-1.28	66.85	-0.42	-0.51	-0.47	-0.49	-0.48	-0.47
83.22	-1.03	-1.17	-1.14	-1.10	-1.13	-1.10	66.2	-0.62	-0.74	-0.68	-0.71	-0.69	-0.67
82.71	-0.80	-0.96	-0.93	-0.88	-0.91	-0.88	65.51	-0.60	-0.72	-0.67	-0.69	-0.68	-0.65
82.33	-0.80	-0.94	-0.92	-0.87	-0.90	-0.88	64.86	-0.46	-0.56	-0.51	-0.53	-0.52	-0.50
81.73	-0.28	-0.44	-0.42	-0.35	-0.38	-0.36	64.38	-0.58	-0.70	-0.65	-0.67	-0.67	-0.65
81.1	-0.10	-0.24	-0.22	-0.16	-0.19	-0.17	63.56	-0.43	-0.54	-0.50	-0.53	-0.53	-0.50
80.42	-0.73	-0.91	-0.89	-0.79	-0.83	-0.81	62.92	-0.44	-0.58	-0.53	-0.56	-0.55	-0.51
79.8	-0.46	-0.57	-0.56	-0.50	-0.52	-0.52	62.12	-0.28	-0.40	-0.35	-0.38	-0.38	-0.34
79.13	-0.52	-0.69	-0.68	-0.59	-0.62	-0.62	61.27	-0.38	-0.49	-0.45	-0.48	-0.48	-0.44
78.54	-1.30	-1.51	-1.50	-1.41	-1.45	-1.43	60.4	-0.59	-0.74	-0.69	-0.73	-0.72	-0.67
77.76	-1.24	-1.42	-1.42	-1.34	-1.38	-1.35	59.73	-0.58	-0.70	-0.66	-0.70	-0.69	-0.64
77.09	-1.47	-1.66	-1.65	-1.57	-1.61	-1.58	58.98	-0.59	-0.68	-0.65	-0.68	-0.67	-0.63
76.42	-2.48	-2.77	-2.75	-2.60	-2.67	-2.63	57.85	-0.59	-0.68	-0.65	-0.68	-0.67	-0.63
75.6	-1.38	-1.56	-1.54	-1.45	-1.49	-1.47	57.18	-0.61	-0.69	-0.66	-0.70	-0.68	-0.64
74.99	-1.58	-1.82	-1.82	-1.70	-1.74	-1.72	56.61	-0.63	-0.73	-0.69	-0.74	-0.72	-0.67
74.39	-1.52	-1.67	-1.69	-1.60	-1.62	-1.60	56.15	-0.76	-0.86	-0.83	-0.87	-0.87	-0.80
73.76	-1.33	-1.46	-1.48	-1.39	-1.41	-1.39	55.67	-0.58	-0.66	-0.64	-0.66	-0.67	-0.61
73.18	-1.12	-1.22	-1.23	-1.16	-1.16	-1.15	55.03	-0.79	-0.89	-0.87	-0.89	-0.90	-0.84
72.64	-1.03	-1.13	-1.13	-1.09	-1.07	-1.07	54.03	-1.35	-1.49	-1.45	-1.47	-1.49	-1.42
71.96	-1.03	-1.16	-1.16	-1.13	-1.11	-1.10	53.11	-1.02	-1.22	-1.15	-1.16	-1.17	-1.11
71.29	-0.98	-1.10	-1.10	-1.09	-1.07	-1.05	52.48	-0.92	-1.15	-1.06	-1.08	-1.08	-1.03
70.65	-0.84	-0.93	-0.91	-0.91	-0.90	-0.88	51.77	-1.17	-1.42	-1.31	-1.34	-1.34	-1.29
70	-0.53	-0.62	-0.59	-0.60	-0.59	-0.56	51.13	-1.52	-1.82	-1.67	-1.72	-1.70	-1.66
69.21	-0.71	-0.80	-0.77	-0.78	-0.77	-0.75	50.64	-1.31	-1.54	-1.43	-1.48	-1.45	-1.43
68.26	-0.57	-0.66	-0.62	-0.64	-0.63	-0.61	50.02	-1.18	-1.39	-1.30	-1.34	-1.31	-1.30
67.51	-0.46	-0.56	-0.52	-0.54	-0.54	-0.51	49.57	-1.19	-1.45	-1.34	-1.41	-1.36	-1.35

Attachment 1- Five-mile Average Bed Change from 2009 to 2033 (ft)

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6	RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
49.03	-0.79	-0.95	-0.88	-0.94	-0.91	-0.90	33.02	-0.96	-1.01	-1.02	-1.06	-1.03	-1.01
48.53	-0.92	-1.05	-1.01	-1.07	-1.03	-1.03	31.79	-0.84	-0.89	-0.91	-0.94	-0.91	-0.89
48.09	-1.05	-1.22	-1.18	-1.27	-1.21	-1.22	31.02	-1.16	-1.19	-1.21	-1.25	-1.22	-1.20
47.47	-0.50	-0.62	-0.60	-0.66	-0.62	-0.64	30.45	-1.71	-1.66	-1.70	-1.73	-1.70	-1.68
46.96	-0.52	-0.67	-0.65	-0.72	-0.67	-0.69	30.03	-1.15	-1.05	-1.07	-1.10	-1.08	-1.06
46.43	-0.59	-0.70	-0.68	-0.73	-0.70	-0.71	29.63	-2.45	-2.01	-2.09	-2.14	-2.09	-2.10
46.03	-0.78	-0.93	-0.90	-0.97	-0.93	-0.94	28.9	-0.99	-0.70	-0.75	-0.76	-0.74	-0.75
45.52	-0.49	-0.57	-0.55	-0.59	-0.57	-0.58	28.5	-0.99	-0.65	-0.70	-0.72	-0.69	-0.71
44.9	-0.58	-0.70	-0.68	-0.73	-0.71	-0.71	28.12	-0.97	-0.63	-0.68	-0.69	-0.67	-0.69
44.45	-0.60	-0.81	-0.77	-0.86	-0.82	-0.82	27.78	-0.96	-0.63	-0.67	-0.69	-0.67	-0.68
43.85	-0.48	-0.63	-0.61	-0.67	-0.64	-0.63	27.35	-1.36	-0.97	-1.02	-1.03	-1.01	-1.04
42.96	-0.59	-0.75	-0.73	-0.80	-0.77	-0.77	26.97	-1.11	-0.76	-0.81	-0.81	-0.79	-0.82
42.06	-0.66	-0.77	-0.76	-0.82	-0.80	-0.79	26.34	-1.36	-0.84	-0.90	-0.91	-0.87	-0.93
41.34	-0.75	-0.86	-0.85	-0.91	-0.88	-0.86	25.89	-1.24	-0.78	-0.85	-0.86	-0.82	-0.88
40.78	-0.72	-0.86	-0.85	-0.91	-0.88	-0.86	25.29	-1.59	-1.06	-1.14	-1.15	-1.10	-1.17
39.86	-0.70	-0.83	-0.82	-0.87	-0.85	-0.83	24.52	-1.62	-1.23	-1.29	-1.29	-1.26	-1.30
39.08	-0.89	-1.00	-0.99	-1.05	-1.01	-1.01	23.93	-1.47	-1.14	-1.19	-1.20	-1.17	-1.21
38.51	-0.86	-0.96	-0.95	-1.01	-0.97	-0.97	23.27	-1.44	-1.06	-1.12	-1.12	-1.10	-1.14
37.97	-1.42	-1.55	-1.53	-1.60	-1.56	-1.55	22.66	-1.50	-1.10	-1.16	-1.16	-1.14	-1.18
37.22	-1.02	-1.11	-1.10	-1.14	-1.10	-1.10	22.1	-1.34	-0.98	-1.04	-1.02	-1.02	-1.06
36.63	-1.40	-1.55	-1.54	-1.60	-1.55	-1.54	21.36	-1.09	-0.81	-0.85	-0.84	-0.83	-0.86
35.97	-0.52	-0.59	-0.59	-0.63	-0.60	-0.59	20.66	-1.17	-0.85	-0.90	-0.90	-0.89	-0.93
35.46	-0.75	-0.83	-0.83	-0.87	-0.84	-0.83	19.91	-1.03	-0.72	-0.77	-0.79	-0.76	-0.81
35.02	-0.89	-0.96	-0.96	-1.00	-0.97	-0.96	19.12	-1.32	-1.00	-1.05	-1.08	-1.05	-1.09
34.19	-1.12	-1.18	-1.20	-1.24	-1.20	-1.19	18.4	-1.25	-0.95	-1.00	-1.03	-0.99	-1.04
33.61	-1.10	-1.17	-1.18	-1.22	-1.19	-1.17	17.68	-1.31	-1.02	-1.07	-1.10	-1.07	-1.11

Attachment 1- Five-mile Average Bed Change from 2009 to 2033 (ft)

RM	Alt1	Alt2	Alt3	Alt4	Alt5	Alt6
17.2	-1.24	-0.98	-1.02	-1.05	-1.03	-1.06
16.64	-1.41	-1.12	-1.17	-1.21	-1.18	-1.22
15.83	-2.03	-1.79	-1.83	-1.87	-1.84	-1.87
15.1	-2.52	-2.30	-2.34	-2.38	-2.36	-2.38
14.55	-2.89	-2.63	-2.68	-2.74	-2.70	-2.74
14.02	-2.53	-2.28	-2.33	-2.38	-2.35	-2.38
13.16	-2.24	-2.02	-2.07	-2.12	-2.08	-2.12
12.24	-2.97	-2.67	-2.75	-2.81	-2.76	-2.80
11.45	-2.21	-1.97	-2.04	-2.08	-2.04	-2.08
10.68	-1.83	-1.61	-1.68	-1.71	-1.68	-1.71
9.78	-1.55	-1.27	-1.35	-1.39	-1.36	-1.39
9.02	-0.88	-0.63	-0.72	-0.75	-0.72	-0.76
8.03	-0.97	-0.60	-0.73	-0.77	-0.72	-0.79
7.1	-0.73	-0.37	-0.51	-0.54	-0.50	-0.55
6.06	-0.64	-0.36	-0.47	-0.50	-0.47	-0.51
4.86	-0.56	-0.32	-0.41	-0.44	-0.42	-0.45
3.93	-0.39	-0.16	-0.24	-0.28	-0.25	-0.28
3.24	-0.34	-0.13	-0.21	-0.25	-0.23	-0.26
2.4	-0.66	-0.42	-0.51	-0.56	-0.53	-0.56
1.39	-0.77	-0.55	-0.63	-0.68	-0.65	-0.68
0.74	-0.88	-0.67	-0.75	-0.79	-0.76	-0.79