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Mississippi River and Tributaries Flowline Assessment Main Report

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Mississippi River and Tributaries Flowline Assessment Main Report

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Under Project Number 449963, "Mississippi River and Tributaries Flowline Assessment"

Abstract

The flowline assessment was undertaken to review the current Mississippi River and Tributaries (MR&T) flowline on which the project design levee elevations are based. The assessment was initiated in 2014 to address the issues outlined in the MR&T 2011 Post Flood Report. Evaluations were to not only consider the hydrology and hydraulic conditions of the system today but were to consider any other factors that would impact the flowline over the next 50 years. To assess the flowline, a seven-step process was utilized: (1) develop precipitation amounts; (2) calculate the amount of runoff; (3) calculate the amount of flow in the river from the runoff, both for regulated and unregulated conditions; (4) calculate a water surface profile/elevation with appropriate levees, structures, backwater areas, and floodways assumptions; (5) develop estimates for future degradation/aggradation of sedimentation at various locations, climate change, loop effect, sea level rise, and subsidence, all of which would affect the water surface profile; (6) analyze water surface profile estimates to identify areas of concern; and (7) perform District Quality Control Review, Agency Technical Review, and Independent External Peer Review on all of the above steps to ensure quality.

Contents

Figures and Tables

Figures

Tables

Preface

This assessment was conducted for the Mississippi Valley Division under Project Number 449963, “Mississippi River and Tributaries Flowline Assessment.” The report is published through the Mississippi River Geomorphology and Potamology (MRG&P) Program. The MRG&P Program is part of the Mississippi River and Tributaries (MR&T) Program and is managed by the U.S. Army Corps of Engineers (USACE), Mississippi Valley Division (MVD), and Districts. The MVD Commander was MG Richard G. Kaiser. The MVD Director of Programs was Mr. Jim Bodron. The Vicksburg District (MVK) Commander was COL Michael C. Derosier. The Memphis District (MVM) Commander was COL Michael A. Ellicott. The New Orleans District (MVN) Commander was COL Michael N. Clancy.

The work was performed by the U.S. Army Engineer Research and Development Center (ERDC), Coastal and Hydraulics Laboratory (CHL) – River Engineering Branch; the MVM; the MVK; the MVN; the St. Louis District (MVS); and the MVD.

At the time of publication of this report, Mr. Jeffrey R. Eckstein was Acting Director of CHL, and Mr. John T. Tucker III was the Acting Deputy Director of CHL.

COL Ivan P. Beckman was the Commander of ERDC, and the ERDC Director was Dr. David W. Pittman.

Unit Conversion Factors

Multiply	By	To Obtain
acres	4,046.873	square meters
cubic feet	0.02831685	cubic meters
second-foot-day	86,400	cubic feet
feet	0.3048	meters
hectares	1.0 E+04	square meters
miles (U.S. statute)	1,609.347	meters
square feet	0.09290304	square meters
square miles	2.589998 E+06	square meters

1 Introduction

1.1 Objective

The Mississippi River mainline levees, floodways, and backwater areas are designed to protect the Lower Mississippi River Valley from flooding by confining flows to the leveed channel and floodplain, backwater areas, floodway areas, and tributary basins. The primary purpose of this assessment is to review the current Refined 1973 Mississippi River and Tributaries (MR&T) project design elevations (flowline). This assessment is a joint effort of the U.S. Army Corps of Engineers (USACE), Memphis (MVM), Vicksburg (MVK), New Orleans (MVN), and St. Louis (MVS) Districts, conducted with the oversight of the Mississippi Valley Division (MVD). MVK was designated as the lead District in the conduct of studies. Collaboration and assistance in this effort was also obtained from the Great Lakes and Ohio River Division (LRD), the Southwestern Division, the U.S. Army Engineer Research and Development Center (ERDC), the National Weather Service (NWS), and the Tennessee Valley Authority (TVA). The project flood flowline is the basis for designing the MR&T mainstem levees and the floodways, which are the backbone of the flood risk management program for the lower Mississippi Valley. The report is not intended as a project justification document, nor is it intended to address the environmental aspects of the MR&T Project.

1.2 Approach

To assess the hydrologic and hydraulic conditions of the MR&T flowline, a seven-step process was utilized: (1) develop precipitation amounts; (2) calculate the amount of runoff; (3) calculate the amount of flow in the river from the runoff, both for regulated and unregulated conditions; (4) calculate a water surface profile/elevation with appropriate levees, structures, backwater areas, and floodways assumptions; (5) develop estimates for future degradation/aggradation of sedimentation at various locations, climate change, loop effect, sea level rise, and subsidence, all of which would affect the water surface profile; (6) analyze water surface profile estimates to identify areas of concern; and (7) perform District Quality Control Review, Agency Technical Review, and Independent External Peer Review on all of the above steps to ensure quality.

Many of the sections of this report are summaries of individual efforts explained in separate, more detailed reports. Table 1-1 lists the series of reports associated with the overall project, with this report listed in bold font. The report is organized as follows:

- Section 1 explains background information about the project.
- Section 2 describes the hydrology effort to determine the appropriate time series of flows that represent the Project Design Flood (PDF) events.
- Section 3 is focused on Hydrologic Engineering Center-River Analysis System (HEC-RAS) model simulations to determine the water surface elevations that occur during the PDF events.
- Section 4 explains channel capacity changes in the future due to the next 50 years of expected sedimentation.
- Section 5 is focused on the hysteresis effect where multiple stages can exist for a given discharge due to varying river conditions.
- Section 6 is focused on bringing the results of the individual efforts together and providing comparisons.

Table 1-1. List of reports included in the overall project.

Report Name	Description
Executive Summary	The Executive Summary briefly summarizes the important information from the entire project assessment.
Main Report	The Main Report summarizes the results in each of the aspects of the entire project assessment and shows the combined effects of the PDF event scenarios.
Hydrology Report	The Hydrology Report assesses the flow of water arriving to the MR&T System during the PDF event scenarios.
Hydraulics Report	The Hydraulics Report assesses the water surface elevations in the Mississippi and Atchafalaya rivers during the PDF event scenarios.
Mississippi River Sedimentation Report	The Mississippi River Sedimentation Report assesses how the next 50 years of sedimentation are expected to change the Mississippi River channel; these changes would impact the water surface elevations expected during the PDF event in the future.

Report Name	Description
Atchafalaya River Sedimentation Report	The Atchafalaya River Sedimentation Report assesses how the next 50 years of sedimentation are expected to change the Atchafalaya River channel; these changes would impact the water surface elevations expected during the PDF event in the future.

1.3 Background

1.3.1 Mississippi River and Tributaries (MR&T) System information

The Mississippi River Basin, which is the third largest drainage basin in the world, consists of a total drainage area of approximately 1,245,000 square miles. Approximately 13,000 square miles of this drainage area lies in Canada, and the remainder is within the geographic boundaries of the United States. At its most easterly point, the divide of this watershed is within 250 miles of the Atlantic Ocean while its most western limits are within 500 miles of the Pacific Ocean (see Figure 1-1).

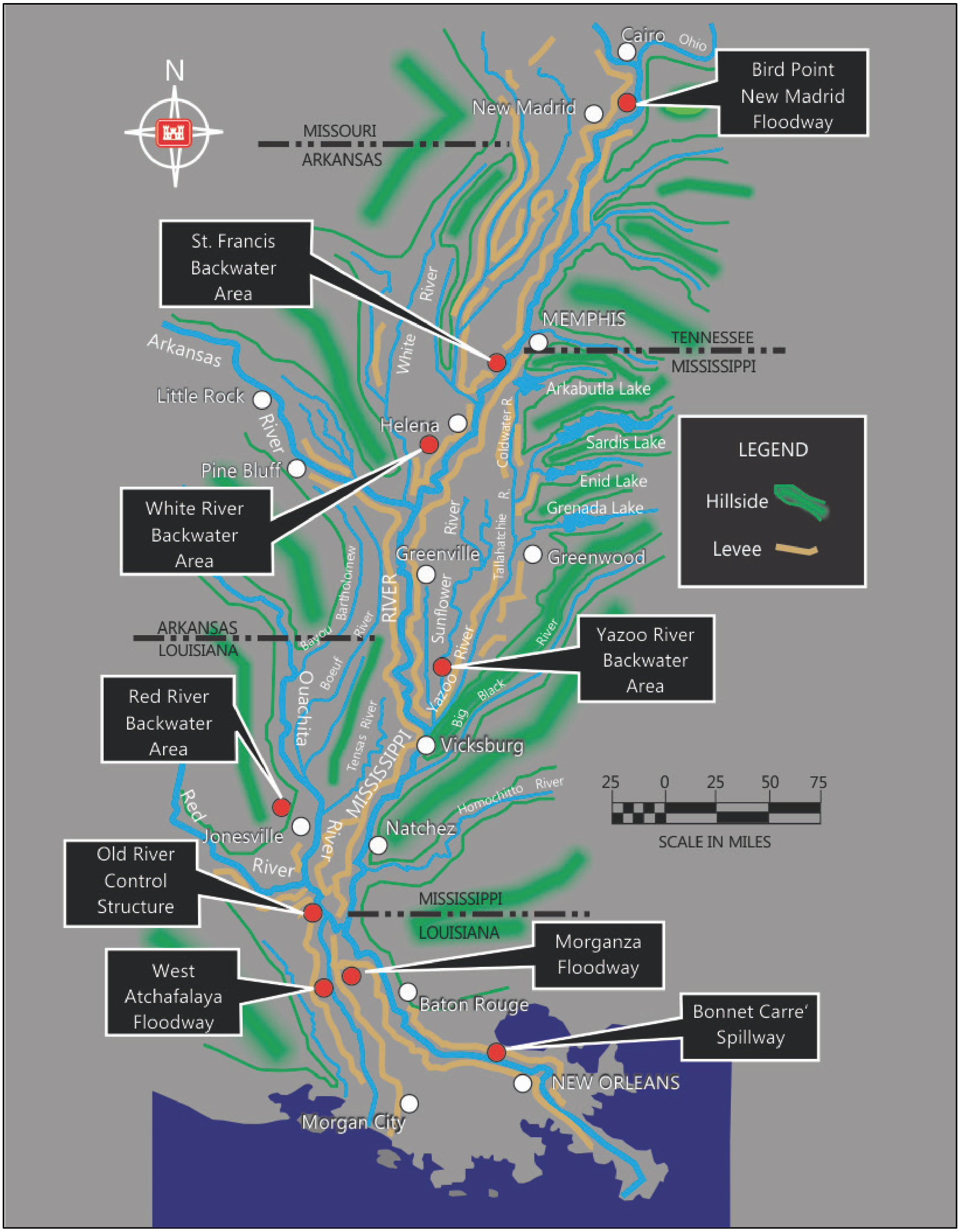
Figure 1-1. Major river systems within the Mississippi River Basin.

The Mississippi River Basin is characterized by great topographic variety. The Basin is bounded on the west by the Rocky Mountains, which exceeds an elevation of 10,000 feet (ft) at many points. Between these mountains and the river itself lie the Great Plains, which vary in elevation from 4,000 ft above mean sea level (MSL) upward. From the Great Plains, the land slopes eastward to the river itself, which rises at an elevation of 1,466 ft above MSL and traverses a channel distance of approximately 2,470 miles to the Gulf of Mexico. The Appalachian Mountain chain forms the eastern divide of the basin. From these mountains, the Appalachian Plateau extends westward at elevations varying from 2,000 to 4,000 ft above MSL. In contrast to the east and west divides, the northern divide is more varied. It consists of an irregular series of elevations varying from less than 1,000 ft to more than 2,000 ft above MSL. Additionally, the Chicago Sanitary and Ship Canal diverts flow from Lake Michigan into the Mississippi River Basin.

The MR&T Project protects the 36,000-square-mile Lower Mississippi River Valley (LMRV) from periodic overflows of the Mississippi River. Figure 1-1 shows the major river systems that comprise the Mississippi River drainage basin. The MR&T System is designed to convey the PDF, represented by the maximum event that has a reasonable chance of occurring from a meteorological viewpoint.

The MR&T System includes an extensive levee system; floodways to divert excess flows past critical reaches; channel improvement and stabilization features to protect the integrity of flood risk management measures and to ensure proper alignment and depth of the navigation channel; and a system of reservoirs to regulate flows and backwater areas to provide storage during extreme events. Additionally, there are tributary basin improvements including levees and pumping stations that expand flood risk management coverage and improve drainage into adjacent areas within the alluvial valley. The MR&T levee system begins at the head of the alluvial valley at Cape Girardeau, MO, and continues to Venice, LA, near the Gulf of Mexico on the right descending bank and to Bohemia, LA, on the left descending bank. Figure 1-2 identifies and provides the general locations of the primary MR&T System components. The MR&T levee system includes 3,787 miles of authorized embankments and floodwalls. Of this, nearly 2,216 miles are along the mainstem Mississippi River, and the remaining levees are backwater, tributary, and floodway levees.

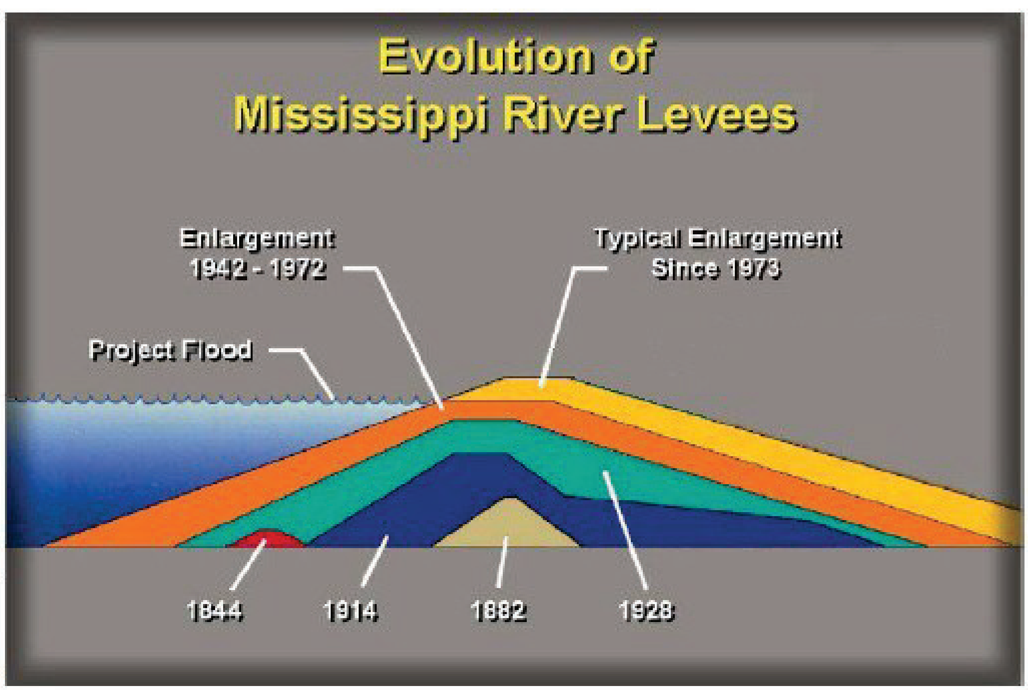
Figure 1-2. General locations of primary MR&T System components.



MR&T flood risk management components currently consist of completed and uncompleted structures and improvements. The individual structures are operated as a system in accordance with the MR&T Flood Risk Management Plan, to reduce overall flood risks throughout the LMRV. The design and operational strategy for the MR&T System does not attempt to entirely exclude the river from its natural floodplain. Instead, it accommodates the natural tendency of the river during extraordinary floods by incorporating floodway and backwater features that are not utilized during small and more frequent flood events.

Levees are the backbone of the MR&T Flood Risk Management Plan. They protect the vast expanse of the developed alluvial valley from periodic overflows of the Mississippi River. The grade and section of the present levee system are much larger than those of the system that was overwhelmed during the 1927 flood (Figure 1-3). In addition to higher and wider levees, the MR&T levee designs incorporate technological advancements that account for the type, condition, and moisture content of material used in the construction of the levees. The design levee grades provide for freeboard—the distance between the PDF flowline and the top of the levee.

Figure 1-3. Evolution of Mississippi River levees (1844 – present).



The integrity of the levee system is also bolstered by advancements in the design, construction, installation, and maintenance of seepage control measures that include landside berms, drainage trenches, drainage blankets, relief wells, and cut-off walls. Additionally, more than 1,000 miles of articulated concrete mattress revetment, over 300 miles of dikes and numerous hard points, chevrons, and bendway weirs associated with channel improvement efforts maintain a stable channel protecting the levees from erosion and assuring the reliability of the navigation channel.

At critical stages or flow rates, other features are activated to control and convey floodwaters and relieve stress on the levees. The first key feature is in the vicinity of Cairo, IL. When the river reaches critical stages on the Cairo gage, the Bird's Point New Madrid Floodway (BPNMF) is operated to divert up to approximately 550,000 cubic feet per second (cfs) and prevent flood stages from exceeding the design elevation of the levees at and near Cairo, IL, the levees along the west bank above Birds Point, and the east bank levee adjacent to the floodway.

There are two major reservoirs, Kentucky and Barkley Lakes, on the Tennessee and Cumberland Rivers that are not features of the MR&T project but are authorized to reduce flood stages on the Mississippi River in the vicinity of and downriver from Cairo. Because of the close proximity of these reservoirs to the confluence of the Mississippi and Ohio Rivers, regulation of the reservoirs has a predictable influence on the operation of the BPNMF. The 1944 Flood Control Act (FCA 1994) directs the TVA to regulate the release of water from the Tennessee River into the Ohio River in accordance with instructions from USACE. Objectives developed by the LRD for the Kentucky and Barkley reservoir outflows have priorities to safeguard the Mississippi River levee system, to reduce the frequency of use of the BPNMF, and to reduce the frequency and magnitude of flooding of lands along the lower Ohio and Mississippi Rivers, which are unprotected by levees. When floods threaten the flood control features along the upper reaches of the MR&T project, the MVD commander and the LRD commander work together to regulate releases from Kentucky and Barkley Lakes with the concurrence of the general manager of the TVA to accomplish these objectives.

Between the lower end of the BPNMF and the Red River, a combination of flood control reservoirs, backwater areas, and comprehensive channel improvement and rectification programs supplement the levee system in passing floods. Backwater areas are located at the mouths of the St. Francis, White, Yazoo, and Red Rivers. Significant portions of the upper sections of these backwater areas receive protection from overflows of the Mississippi River afforded by the mainline levees. The lower portions of these areas serve as natural storage during larger floods. The backwater levees are designed to naturally overtop when flood stages along the mainstem of the Mississippi River reach specified levels. When flood stages subside, floodwaters within the backwater areas drain through floodgates or are pumped. The channel rectification program improves the carrying capacity of the main channel and lowers the flood flowline through the use of historical cutoffs (severed large bends from the river in the past) and corrective dredging.

From the Red River backwater to the Gulf of Mexico, the MR&T flood control plan uses a more elaborate system to manipulate flood waters. The first key component of that reach is the Old River Control Complex (ORCC). The ORCC was authorized in 1954 to prevent the Atchafalaya River from capturing the Mississippi River. The complex is designed to maintain the 1950 latitude flow distribution between the Mississippi River and the Atchafalaya/Red River System of 70% to 30%, respectively.

Approximately 30 miles downstream from the ORCC, the Morganza Floodway provides for additional diversion of floodwaters. Governed by a 3,900 ft long and a 125-bay intake structure, the floodway is designed to divert up to 600,000 cfs from the Mississippi River to the Atchafalaya basin when the Mississippi River flows below Red River Landing are projected to exceed 1,500,000 cfs.

The West Atchafalaya floodway extends along the west side of the Atchafalaya River Basin. The floodway contains an 8-mile long fuseplug section of levee at its head. When the fuseplug section crevasses or when the west bank Atchafalaya River levee overtops, the floodway is designed to divert up to 250,000 cfs. Under the present water control plan, the West Atchafalaya Floodway would be the last feature of the flood control system to be used. The Atchafalaya River, the Morganza floodway, and the West Atchafalaya floodway converge at the lower end of the Atchafalaya River levees to form the Atchafalaya Basin Floodway. The Atchafalaya Basin Floodway receives flow from the Red River and from the Mississippi River via the ORCC and the Morganza Floodway; it is designed to carry 1,500,000 cfs, the combined flow of the West Atchafalaya Floodway, Atchafalaya River, and Morganza Floodway. The Atchafalaya Basin Floodway has two outlets, Lower Atchafalaya River, with a project flood flow of 919,000 cfs, and Wax Lake Outlet, with a project flood flow of 581,000 cfs. The Avoca Island levee and levees west of Berwick provide measures of risk reduction below Morgan City to communities such as Franklin, Calumet, and Patterson.

The MR&T Flood Risk Management Plan provides additional control of the system below the Morganza floodway through the Bonnet Carré spillway, located approximately 30 miles above New Orleans, LA. The 7,200 ft long spillway structure is governed by 350 intake bays and connects to a 6-mile long floodway that empties into Lake Pontchartrain. The floodway is designed to divert up to 250,000 cfs from the Mississippi River, to ensure the peak discharge at New Orleans does not exceed 1,250,000 cfs.

1.3.2 Past MR&T flowline studies

1.3.2.1 Studies prior to 1955

The Mississippi River Valley has experienced a number of great floods in its history. From 1861 to 1927, the design flood for the lower Mississippi River progressively increased and in general was based on the maximum flood of record. The flood of 1927 so far surpassed these prior floods that it necessitated a review of the flood risk management plans and a reconsideration of all levee grades. The estimated peak confined discharge of the 1927 flood at Cairo, IL, was 1,800,000 cfs. In 1927 the Mississippi River Commission (MRC), after studying a number of flood combinations, adopted a design peak flood discharge at Cairo of 2,250,000 cfs. This flow was approximately 25% greater than that of the 1927 flood.

Other studies were made between 1927 and 1950 that resulted in revisions to the original project flood flows. These project flood studies are outlined in detail in the *Mississippi River Project Flood Study, Memorandum Report No. 1*, dated 16 December 1955 (MRC 1955). In a letter to the President, MRC, from the Chief of Engineers dated 9 March 1954, subject: Mississippi River Project Flood Study, the MRC was directed to undertake a review of the Mississippi River project flood to determine if any changes were considered advisable in light of improved techniques and basic data available. The Hydrometeorological Section of the Weather Bureau (now NWS) assisted in these studies. A comparison of the peak discharges for the most recent project flood reviews at key locations is shown in Table 1-2.

Table 1-2. Project flood review (flow in cubic feet per second).

Location	1928	1941	1955*
Cairo, IL	2,400,000	2,450,000	2,360,000
Arkansas City, AR	3,200,000	3,065,000	2,890,000
Latitude of Red River Landing, LA**	3,000,000	3,000,000	3,030,000

* flows after reduction for reservoir storage effects.

** Latitude of Red River Landing includes flow in both the Atchafalaya and Mississippi Rivers.

1.3.2.2 1955 Project design flood (PDF) study

The flood risk management plan for the lower Mississippi River and its tributaries is designed to contain within its levees the project flood. During the project flood review made in the 1950s, some 35 different hypothetical combinations of historical storms were sequentially arranged to conform with frontal movements and synoptic situations consistent with those in nature to determine the meteorologically feasible pattern that would produce the greatest runoff in the lower Mississippi River. Project design storms were developed from combinations of events that were considered plausible from a meteorological viewpoint, with a reasonable probability of occurrence. The studies revealed that the 1937 storm, with excess rainfall increased by 10%, followed 4 days later by the 1950 storm, and 3 days later by the 1938 storm (with rainfall rotated and transposed northward to maximize its effect), was the series that produced the maximum runoff among those investigated. This series was designated as the Hypo-Flood[1] 58A. Runoff from this storm combination was routed through the tributary reservoirs under three conditions: (1)“U” or unregulated (no reservoirs); (2)“E” or reservoirs existing in 1950; and (3)“EN” or existing reservoirs plus those proposed to be constructed by 1970. Runoff was routed down the mainstem under each of these conditions to determine the project flood flow at key locations. The near-future condition, or 1970, was selected as the basis for the project flood flows and is referred to as the 58A-EN PDF. Table 1-3 presents a comparison of project flood flows under each of the three conditions.

[1] Hypo-Flood = hypothetical flood.

Table 1-3. Peak discharges for the PDF (cubic feet per second).

	Reservoir Condition		
Location	Unregulated	Existing (1950)	Near Future (1970)[A]
Cairo, IL	2,850,000[C]	2,500,000	2,360,000
Arkansas City, AR	3,210,000[C]	2,950,000[C]	2,890,000
Latitude of Red River Landing, LA[B]	3,320,000[C]	3,130,000[C]	3,030,000

[A] These are 58A-EN PDF flows.

[B] Latitude of Red River Landing includes flow in both the Atchafalaya and Mississippi Rivers.

[C] Peak values from hydrographs in Annex C (USACE 1959).

It is reasonable to assume that all critical storm combinations have not been observed, and therefore the project flood is smaller than the probable maximum flood (the term generally used to describe the greatest flood that could occur). Comparison of the project flood with the flood of 1927, the maximum observed flood in the lower Mississippi River at the time of the 1955 study, reveals that the project flood regulated by reservoirs is approximately 25% greater in magnitude than the 1927 flood (the 2011 flood exceeded the 1927 flood in some locations). Table 1-1 presents the relation among the project flows adopted in 1928, 1941, and 1955 at some of the key gages on the Mississippi River. As indicated in Table 1-1, the magnitude of the design flows has remained fairly constant throughout these reviews.

1.3.2.3 1973 Adjusted PDF flowline study

In the fall of 1972, the Mississippi River Basin experienced heavy rainfall, often torrential in nature, and the river began to assume a pre-flood pattern, somewhat similar to the flood of 1927. As a result of the incessant rainfall, flood control reservoirs in the tributary basins began to fill, the ground became saturated, ponding areas became filled, and continuing rains could no longer be absorbed. In the spring of 1973, a serious flood developed throughout the lower Mississippi River Basin.

As the 1973 flood developed and preparations for a major flood continued, stage-discharge relationships were observed to be several feet higher than the stage-discharge relationship upon which levee grades were based. As a result, the MR&T Project design flowline on the Mississippi River, last reviewed in the early 1950s, and in the Atchafalaya Basin, last reviewed in the early 1960s, was found to be inadequate. Therefore, based on available data, the flowline was revised in 1973 for use in raising levees and floodwalls to the grade required for authorized protection. The 1973 flowline adjustment required approximately 800 miles of permanent levee raises for the MR&T Project on the Mississippi River, in the Yazoo Basin, in the Red River backwater areas (Tensas Basin), and in the Atchafalaya Basin.

1.3.2.4 Refined 1973 MR&T Project Flood Flowline study

This study was made for the purpose of refining the 1973 project design flowline for the MR&T Project, which is contained entirely within the MVS, MVM, MVK, and MVN. The study was basically an analytical evaluation, using the HEC-2 water-surface profile computer program to determine flowline elevations that would be attained for the flows resulting from the 58A-EN PDF. Channel and overbank cross sections were surveyed for use in analyzing the flood-carrying capacity of the Mississippi River floodway under 1973 conditions.

The refinements to the 1973 MR&T Project Flood Flowline (hereafter referred to as the Refined 1973 Flowline) resulted in flowline elevations at major gaging stations on the Mississippi River essentially the same as those established in the 1973 study. However, between major gaging stations where detailed studies were not made in 1973, there were changes as a result of the refined analyses. Higher flowline elevations in many reaches revealed that additional raises on 561 miles of MR&T Project levees would be required. This included 336 miles on the Mississippi River, 46 miles in the Yazoo Basin, and 179 miles in the Atchafalaya Basin.

1.3.3 MR&T System 2011 Post-Flood Report

The MVD MR&T System 2011 Post-Flood Report (PFR) was prepared in response to the historic flooding within an extensive portion of the Mississippi River basin in 2011. The 2011 Flood tested the MR&T flood risk management system like no flood before. Stage and flow rates broke records at several locations, and for the first time, three floodways—BPNMF, the Morganza Floodway, and the Bonnet Carré Spillway—were operated during a single flood event to relieve the enormous and sustained stress on the levee system. Rare and extreme events such as the 2011 Flood presented a unique opportunity to learn about the function, capability, and reliability of FRM systems and to improve the knowledge and science that guide pre-flood planning, operational decision-making processes, and post-event recovery operations. The PFR effort was designed and conducted to capitalize on that opportunity.

The purposes of the overall PFR effort were to (1) evaluate and document the performance of the MR&T System and how it was managed to reduce flood risks during the 2011 Flood; (2) develop a system-wide approach to sequencing repair efforts to effectively address risks and repair the system to pre-flood conditions; and (3) identify and recommend opportunities to improve the System's future performance. One of the recommendations resulting from the PFR was to evaluate the need to conduct an updated flowline study for the MR&T System using 2011 hydraulic flood data and associated MR&T component performance. An examination of the physical and hydraulic changes in the river system and complex flow patterns at Morganza, Bonnet Carré, and ORCC was recommended to determine if a change in flowline data or water control plans were warranted.

2 Hydrology

2.1 Objective

The MR&T System 2011 PFR concluded that the current 3-storm combination that makes up the PDF is still adequate but that a review of the hydrology of the basin was required to determine if the inflow hydrographs and lateral inflows to the MR&T system (developed in 1955) should be updated to reflect changing hydrologic conditions in the basin from factors such as climate change, land use changes, constructed projects, etc. The PDF is an extreme event and is larger than previous floods of record, including the 2011 event. While it is likely that significant hydrologic changes have occurred in the Mississippi River Basin, the impacts in total runoff for extreme events may not be significantly different today from when the PDF was first developed. The assessment of PDF hydrology became necessary to develop flow inputs required for the HEC-RAS model and was also used to evaluate changes in total runoff.

2.2 Approach

Re-evaluation of the PDF was instigated by the magnitude of the 2011 flood that broke peak stage and flow records along significant portions of the Lower Mississippi River. The current PDF is based on hypothetical storm combinations (from the 1955 study) that produce the greatest storm with a reasonable chance of occurring.

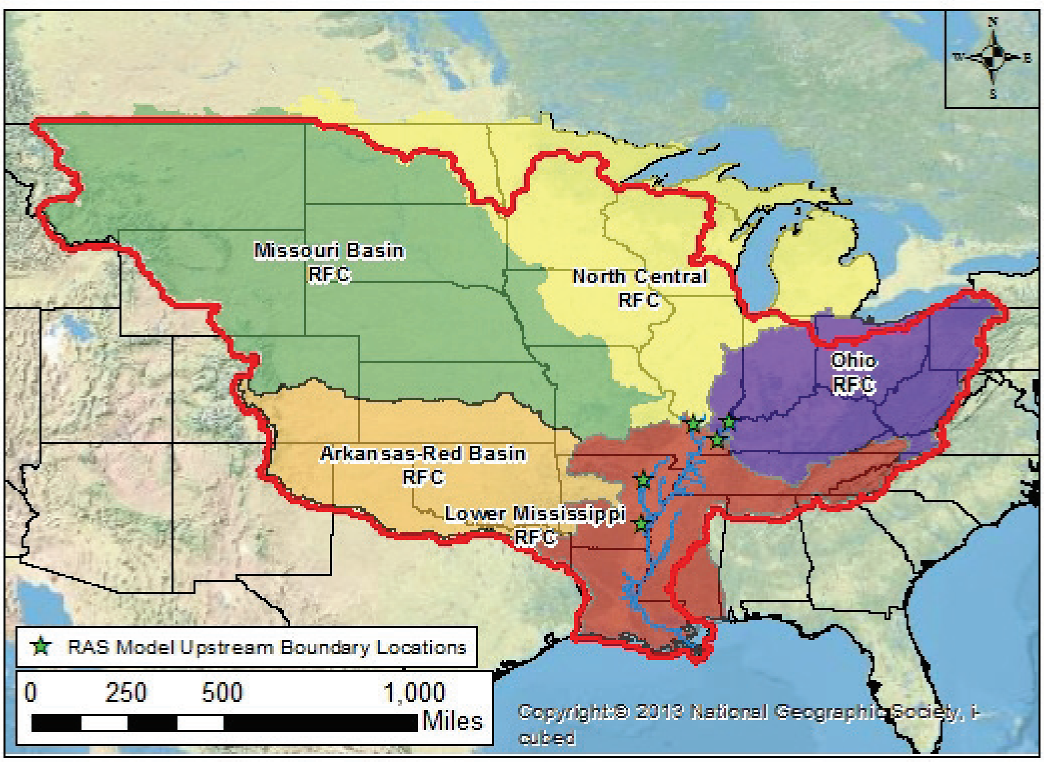
Initially, a HEC-RAS unsteady model was developed following the 2011 flood to evaluate system performance during the flood. This model was expanded for this Mississippi River Flowline Assessment to evaluate the PDF flowline. During HEC-RAS model development for PDF conditions, it became clear that hydrologic data available from the 1955 study were not sufficient to define flow inputs required for model simulations. The need for additional flow inputs precipitated the need to update the hydrologic analysis.

Updating the 1955 vintage hydrology involved developing a strategy for performing the required analysis. The general strategy involved the following steps:

- defining meteorological conditions
- defining required outputs
- developing model of rainfall-runoff processes
- calculating reservoir reduction effects.

Previous studies utilized steady-state hydraulics to calculate water surface elevations using peak discharges developed using unit hydrograph theory and various methods for routing hydrographs. The 2016 analysis utilized an unsteady hydrodynamic model. This unsteady flow model included changes in flow and river dynamics through time and required continuous flow hydrographs at upstream boundaries and for other areas that required internal flow contributions. Figure 2-1 shows the Mississippi River Basin with the limits of the unsteady HEC-RAS model extent shown along with NWS River Forecast Centers (RFC).

Figure 2-1. Mississippi River Basin with HEC-RAS extent shown. (Note: Red line indicates the USACE area of responsibility for the Mississippi River watershed.)



To develop the required hydrologic inputs, the entire watershed was simulated using NWS hydrologic models.

2.2.1 Hydrologic model linkages

The HEC-RAS model was built to include the main-stem Mississippi River as well as significant tributary streams. The model extended from Chester, IL, to the Gulf of Mexico on the Mississippi River with the Ohio River reach extending upstream to Smithland, KY. The model was extended up significant tributaries, those having a contribution that influenced model calibration for the 2011 event, to locations that had NWS model sub-basin computation points or at the end of model routing reaches. Inflow hydrographs were directly input at boundaries where the HEC-RAS model started on river channels. Internal inputs were defined as areas that provide local flow within a reach or a minor tributary that provides a relatively small percentage of overall flow to the total Mississippi River.

The hydrologic model used for the current assessment produced outputs for all modeled sub-areas and computation points in the NWS Community Hydrologic Prediction System-Flood Early Warning System (CHPS-FEWS[1]) configurations. The hydrologic model included 7,066 sub-basins, significantly more sub-basins than the 44 from the 1955 study.

After each RFC model run was completed, results were passed successively to downstream RFCs to use as inflows until simulations for the entire Mississippi River Basin were complete. Once a complete simulation from all CHPS-FEWS model segments was finished, Lower Mississippi River Forecast Center (LMRFC) exported the specific output locations needed for the HEC-RAS model.

2.2.1.1 Replication of 1955 study

The HEC-RAS unsteady model was configured to run using the limited hydrologic data from the 1955 study. This coarser resolution of tributary inflows was not desirable for sufficient unsteady modeling of the tributary connections, but it provided a direct means to compare results with prior analyses. HEC-RAS inputs were developed directly from the Memorandum Report No. 1 (MRC 1955) and related computation tabulations from the MRC archives.

[1] The NWS Community Hydrologic Prediction System (CHPS) utilizes the DELTARES Flood Early Warning System (FEWS) model to simulate rainfall-runoff events and develop river forecasts throughout the United States. This suite of modeling tools is referred to as CHPS-FEWS.

2.2.1.2 Current assessment

The HEC-RAS model was reconfigured to run using the current hydrologic data available from NWS RFC model outputs.

2.2.2 Procedural approach

Two scenarios were considered to determine the effects of different methodology used between the 1955 analysis and the current NWS forecast models. Only Scenario I was carried forward.

Scenario I developed the PDF inflow hydrographs using the 1955 PDF report antecedent climatic conditions. This scenario utilized precipitation and temperature data from the historic period prior to the first storm in each HYPO sequence as initial forcings[1] in the NWS hydrologic models. To warm up the model simulation, a period of 6 to 9 months was simulated to produce flows and runoff characteristics consistent with the period used in the 1955 analysis. The warmup period varied in length to capture a full winter season prior to start of the actual HYPO sequence. This was necessary to develop snow water equivalents for the entire winter season that would be available to contribute to flood hydrographs.

Scenario II would have developed the PDF inflow hydrographs using antecedent conditions reflecting spring (April/May, i.e., 2011 event) snowmelt and precipitation. Study objectives, schedule, and funding prohibited the execution of Scenario II.

Scenario I included analysis of HYPO storms 58A, 56, 52A, and 63. A description of the seasonality and predominant area impacted by each HYPO is given below:

River Forecast Centers
Running Scenario I with the NWS CHPS-FEWS model suite required coordination across five different NWS RFCs:
ABRFC = Arkansas Basin RFC
LMRFC = Lower Mississippi RFC
MBRFC = Missouri Basin RFC
NCRFC = North Central RFC
OHRFC = Ohio RFC

- HYPO 58A (winter season) storm combination for Mouth of Ohio River to Gulf of Mexico
- HYPO 63 (early-spring season) storm combination for Middle Mississippi from mouth of Ohio River to Cape Girardeau, MO

[1] Forcings are state conditions imposed on the model at startup. Model state conditions include antecedent soil moisture, stream flow, temperature, precipitation, and snow pack state at the first model time-step.

- HYPO 52A (late-spring season) storm combination for Arkansas-White Tributaries
- HYPO 56 (early-spring season) storm combination for White and Red River tributaries.

The 1955 study determined that HYPO 58A was the governing combination that defines the PDF for the Lower Mississippi River. This determination was confirmed by the PFR. Therefore, the current assessment used HYPO 58A to develop required inputs for the HEC-RAS unsteady hydrodynamic model. Analysis using HYPO 52A, 56, and 63 was included only to provide a means to evaluate and to check the methodology using current practice against the 1955 study results.

The HYPO storm events that define the 1955 PDF include the effects of Groups E and N reservoirs. The specific HYPO event for the 1955 PDF was labeled as 58A-EN ("EN": existing reservoirs plus those proposed to be constructed by 1970, according to the 1955 study). The CHPS-FEWS model simulations were first run including reservoirs that existed in the model configurations to assess the regulated condition. While reservoir configurations existed in the CHPS-FEWS models, the NWS simulations for this assessment used automated routing through the reservoirs and did not include any operator over-rides; therefore, it was necessary to expand the analysis group to include 10 USACE districts to perform more detailed reservoir routing. The more detailed reservoir routing was done only for projects identified by the USACE districts to have a possible influence on flows at Cairo, IL. The TVA also assisted in detailed regulation calculations for reservoirs on the Tennessee River system. The NWS automated reservoir routing continued to be used for projects that were considered to have no influence on flows at Cairo, IL. Detailed reservoir routing by the USACE districts and TVA followed published operational guide curves (or rule curves) with regulator adjustment for the current assessment.

The designation for the current (labeled "2016") regulated ("R") PDF HYPO was 58A-R. The new designation, 58A-R, was necessary because the EN condition represented a specific list of reservoirs that was different than used in the 2016 analysis. The 2016 analysis used reservoirs that existed at the beginning of the investigation (circa 2014).

Each RFC prepared and executed model simulations for each historic event coordinating with other RFC offices just as in preparing their daily forecast.

LMRFC provided a gateway between USACE data and CHPS-FEWS modelers. LMRFC has developed procedures for exchange of data between the CHPS-FEWS model format and the USACE Hydrologic Engineering Center-Data Storage System Visual Utility Engine (HEC-DSSVue) format. USACE coordinated with the U.S. Geological Survey (USGS), TVA, and various USACE District offices to obtain the necessary reservoir release data for event simulations.

Unregulated and regulated conditions were simulated to evaluate hydrologic model outputs against the previous results. The unregulated results provided a direct way to compare model results for 1955 calculations and 2016 simulations with the only difference being basic hydrologic parameterization and calculations. No additional assumptions or configurations related to reservoir regulation were needed for direct comparison with 1955 unregulated results. Excluding the reservoir routing assumptions with the unregulated simulations provided a means to assess the differences in methodology (parameterization and computation technique) employed in development of the hydrologic models.

2.2.3 Hydrologic model

The USACE collaborated with the NWS to leverage its existing hydrologic forecast CHPS-FEWS models to simulate scenarios that examined how hydrologic changes within a watershed impact the PDF runoff and by extension, the PDF Flowline.

CHPS-FEWS operational models included calibration measures[1] that best reproduced existing 2010–2014 conditions. This does not imply that all sub-basins within each RFC model domain had been calibrated in the same level of detail; rather, the overall model performance at NWS forecast points had been used to adjust model parameters to achieve reliable forecast values for each RFC. These adjustments are continuously made as part of routine forecast operations to refine the model's ability to predict flow and stage throughout each RFC's area of responsibility. Therefore, land use, overland and channel routing, as well as any channel ratings used in the CHPS-FEWS models, likely had significant differences from the conditions that existed at the time of each historic event considered in this analysis

[1] NWS forecast models are calibrated empirically to historical streamflow records, using historical precipitation and temperature forcings, with a tendency to weight more recent periods of history more heavily when basin characteristics show evidence of change. This is done to provide the best parameterization for current event forecasting.

(1913 to 1950). The historic data needed to calibrate the NWS operational model to prior periods of time were not available, which made it impossible to address changes in model parameterization for infiltration, routing, land use, and snow melt over time.

2.2.3.1 Basic CHPS-FEWS model

In 2010, the NWS began full implementation of its CHPS as part of NWS modernization plan.

FEWS is a suite of infrastructure software maintained and supported by Deltares. On its own it does very little; it is primarily a generic mechanism to pass data from one place (e.g., a user interface) to another (e.g., a hydraulic model), and it performs some basic time series data transformations. Only when FEWS is configured for the user's specific domain does it transform into a functioning system. A user would supply the necessary modeling operations or acquire them from a source which shares (open source) or sells FEWS-compatible models.

CHPS is a National Oceanic and Atmospheric Administration (NOAA)-customized application of FEWS. CHPS runs models that are compatible with FEWS—including those migrated from the National Weather Service River Forecast System (NWSRFS). NWSRFS was the NWS legacy modeling framework that provided extra user capabilities not available via FEWS, such as model calibration. In the future, NOAA will make CHPS models available to other FEWS users.

Most hydrological forecast models are initial state type models; that is, they require a known state from which to start. CHPS required initial forcings (data inputs) to define the model start state. This includes climatic and meteorological inputs as well as any reservoir regulation states that should exist at the start of a run.

The NWS CHPS models run entirely on Linux on the NWS Advanced Weather Interactive Processing System. A general description of the primary NWS models that are run in CHPS follows in the next sections[1].

The TVA also uses a version of CHPS-FEWS to simulate hydrology over the Tennessee River basin as part of its daily operations. TVA operations

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[1] http://www.nws.noaa.gov/oh/hrl/general/indexdoc.htm

included hydro power generation, navigation, and flood risk reduction. TVA also uses RiverWare[1] to assess regulation needs and release requirements based on hydropower, navigation, flooding, and environmental requirements.

2.2.3.2 Primary NWS hydrologic operations

Snow Accumulation and Ablation (SNOW-17) Model[2]

The NWS uses the SNOW-17 snow accumulation and ablation model as a component of the CHPS system. SNOW-17 is a conceptual model that uses precipitation and temperature as inputs to maintain an accounting of the water equivalents and depth. All of the RFCs participating in the Flowline assessment use the SNOW-17 model with the exception of the LMRFC, which typically does not have much snow to contend with.

Sacramento Soil Moisture Accounting (SAC-SMA) Model[3]

The SAC-SMA is a conceptually based soil moisture accounting model that computes the runoff derived from rainfall inputs and evaporation losses. The model is characterized by two layers (zones) with tension and free water components. The soil moisture parameters making up the SAC-SMA model are not tied to any physically measured soil characteristics; rather, they are manually calibrated along with the SNOW-17 model (where used) and a unit hydrograph operation to time-distribute the generated runoff to produce simulated flows that reproduce historical flows as closely as possible[4]. All of the RFCs in the Flowline assessment area including the TVA used the SAC-SMA model.

Lag and K Routing (LAG/K) Model[5]

The LAG/K routing is widely used by the NWS as a method of storage routing between flow-points. It is configured to accommodate either constant or variable Lag and K elements that can be used together or

[1] RiverWare™ is a general river basin modeling tool developed by the Center for Advanced Decision Support for Water and Environmental Systems (CADSWES) that includes reservoir operations and optimization components (http://riverware.org/).

[2] http://www.nws.noaa.gov/oh/hrl/general/chps/Models/SNOW-17.pdf

[3] http://www.nws.noaa.gov/oh/hrl/general/chps/Models/Sacramento_Soil_Moisture_Accounting.pdf

[4] http://www.nws.noaa.gov/oh/hrl/general/chps/Models/Unit_Hydrograph.pdf

[5] http://www.nws.noaa.gov/oh/hrl/general/chps/Models/Lag_and_K_Routing.pdf

separately to account for lag with no attenuation or attenuation with negligible lag. LAG/K routing was the predominant lumped routing method used by NWS offices within the Flowline assessment area.

Tatum Routing Model[1]

Tatum coefficient routing is a numerical non-storage method of routing a hydrograph through a channel. Tatum coefficient routing is not closely tied to the physical processes of river hydraulics. Coefficients at specific ordinates are used to route flow. These coefficients are usually empirically derived through examination of inflow and outflow hydrographs. The North Central RFC (NCRFC) uses Tatum routing extensively.

Single Reservoir Regulation (RES-SNGL) Operation[2]

The NWS RES-SNGL operation is designed for the simulation of single independently operated reservoirs. The operation is set up so that it may be calibrated with rules and seasonal guide curves to replicate real-world reservoir operation objectives such as flood control and power generation. The RES-SNGL operation is designed to accommodate simulation of reservoir outputs with or without forecast releases.

Joint Reservoir Regulation (RES-J) Model[3]

The RES-J model can simulate either a single reservoir or a system of reservoirs.

2.2.3.3 Running NWS CHPS for the flowline assessment

The NWS RFC models are operationally run as a continuous model and typically store model states for soil moisture, unit hydrographs, routings, and reservoirs for each model time-step so that the model is prepared for operations using live *warm model states*. These models can also be initiated at other times using a *cold state*, which contains initial conditions to start the model. However, these cold states can represent any variety of

[1]http://www.nws.noaa.gov/oh/hrl/nwsrfs/users_manual/part2/_pdf/24tatum.pdf
[2]http://www.nws.noaa.gov/oh/hrl/general/chps/Models/Single_Reservoir_Regulation.pdf
[3]http://www.nws.noaa.gov/oh/hrl/general/chps/Models/Joint_Reservoir_Regulation.pdf

starting conditions from wet to dry depending upon when and how the original model was developed.

The RFC models had anywhere from 500 to over 1,000 model segments each, and it was not feasible for the RFCs to modify all of the model states to represent a specified initial condition across an RFC-sized domain. It was discussed among the RFCs and USACE that a period of at least 6 months of forcings data (i.e., precipitation and temperature) would be prepared for ingest into the RFC models to *warm* the model up to a state close to equilibrium prior to introducing the historical or hypothetical floods.

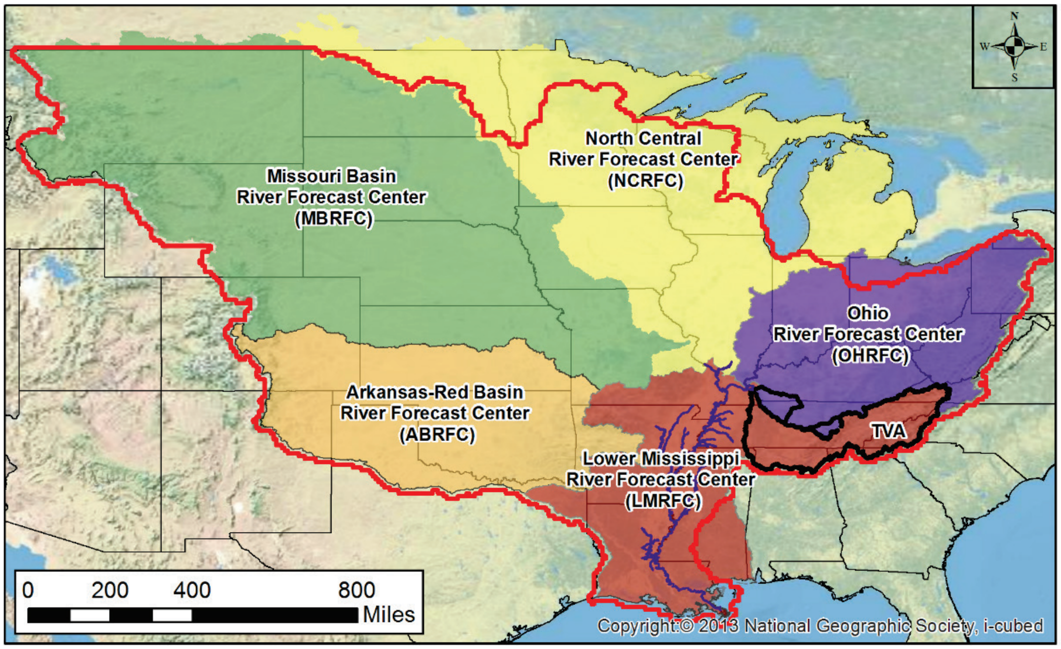
The RFC models ran on a 6-hour time-step and required continuous data during the simulation period for reasonable results. During the preparation of the precipitation and temperature grids for the historical runs, care was taken to ensure that a continuous record of ASCII grid data was available for each 6-hour period. Likewise, offices requiring temperature grids used either the historical grids for the period of record available or in some cases had to revert to climatology to extend the forecast runs.

2.2.3.4 NWS RFCs in the flowline assessment area

Coordination of this project required the collaboration of five NWS RFCs and the TVA to generate inflows for the USACE hydraulic model used in the Flowline assessment (Figure 2-2).

Each of these RFCs utilized the existing CHPS-FEWS models to ingest the HYPO storm precipitation (and where applicable, the corresponding temperatures) to develop flow hydrographs at points needed for the assessment. LMRFC served as the focal point for exchange of data between the RFCs and USACE-MVD.

Figure 2-2. Map of the NWS RFCs and TVA hydrologic service areas participating in the flowline assessment.



2.2.4 Land use considerations

A general land use comparison between 1949 and 2007 was available and indicated minimal land use change as listed in Table 2-1.

Table 2-1. Land use within the Mississippi River Basin obtained from the U.S. Department of Agriculture (USDA).

	By Land Use Category			By Total Area		
Land Use	1949 (1,000 acres)	2007 (1,000 acres)	% Difference	1949, % of Total Acres	2007, % of Total Acres	Change from 1949, %
Crop Land	362,307	317,890	-13	34%	30%	-4%
Grassland Pasture	317,462	308,948	-3	29%	29%	0%
Forest Land	298,476	308,938	3	28%	29%	1%
Special Uses[1]	47,669	74,023	43	4%	7%	3%
Urban[2]	9,546	27,598	97	1%	3%	2%
Other Land[3]	45,995	37,366	-21	4%	3%	-1%

Notes
[1] Special Uses includes rural transportation, parks and wildlife areas, defense installations, and farmsteads.
[2] In 1969, urban areas shifted from areas of 1,000 acres to areas of 2,500 acres or more.
[3] Marshes, swamps, bare rock, deserts, tundra plus other uses not estimated, classified, or inventoried.

While these land use categories failed to depict drainage alterations[1] and improvements within each category, the data still indicated minimal change over the large drainage basin (1,245,000 square miles). Given the relatively small change in general land use depicted above, deviations between model outputs and observed flow were not likely a result of land use change over the past 70 years.

2.2.5 Infiltration and base flow considerations

The Hydrology Report (USACE 2018a) describes how the 1955 analysis utilized simplified loss coefficients across 44 relatively large sub-basins to adjust rainfall depths in runoff calculations (Table 2-2). This method grouped large portions of the watershed into a few loss rates. This grouping limited the spatial resolution applied in the calculations and had very minor allowances for temporal change in infiltration. Base flows used during 1955 calculations were derived from discharge observations using standard separation techniques. Resulting values were given for monthly average flows.

In the 1955 study, seven major drainage basin divisions (1 through 7 in Figure 2-3) were made—one for each of the major tributary streams and one which contains all the minor tributaries and local areas along the main stem. The major divisions were then subdivided into 25 secondary divisions (labeled A through Y in Figure 2-3), and these secondary divisions were further divided into 30 tertiary divisions for convenience in determining runoff (labeled as A-1, etc., in Figure 2-3).

[1] Drainage alterations (i.e., tile drainage and ditching) have changed the hydrologic responses in lower flow regimes and snowmelt; however, this assessment only considers large rain events, and these drainage alterations have minimal impact.

Figure 2-3. Watershed map from 1955 analysis of the Mississippi River and Tributaries PDF.

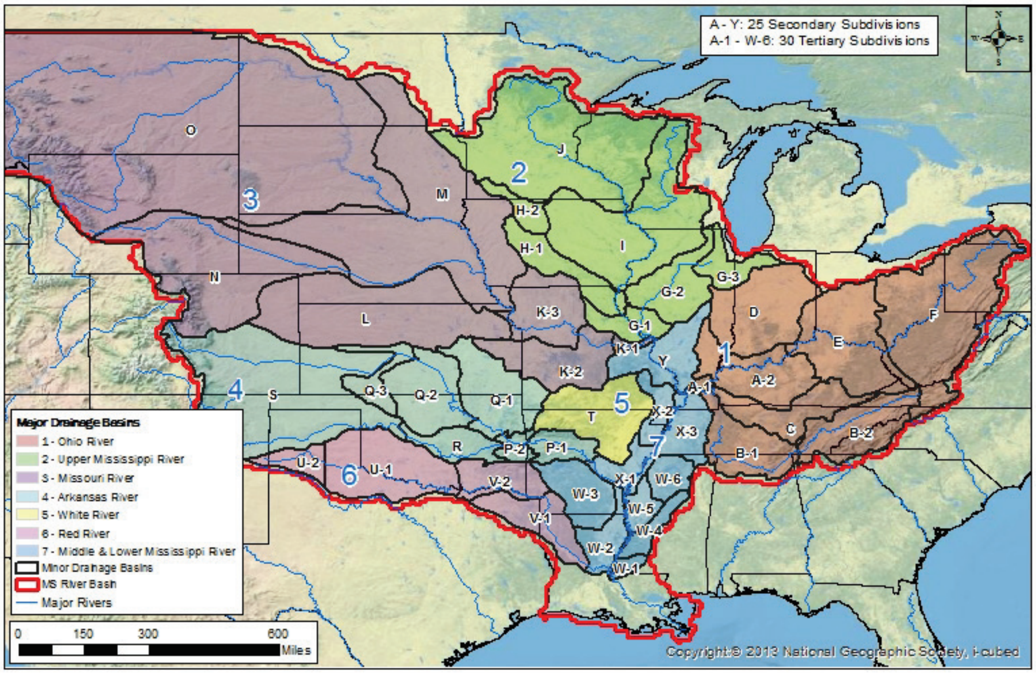


Table 2-2. Infiltration coefficients from 1955 study.*

Area No.	Sub Area	Infiltration Indices (in./hr)
1	A-1	0.04
	A-2	0.04
	B-1	0.05
	B-2	0.04
	C	0.04
	D	0.04
	E	0.02
	F	0.02
2	G-1	0.06
	G-2	0.06
	G-3	0.06
	H-1	0.09
	H-2	0.12
	I	0.1
3	K-1	0.08
	K-2	0.08
	K-3	0.08

Area No.	Sub Area	Infiltration Indices (in./hr)
	L	0.12
	M	0.12
4	P-1	0.05*
	P-2	0.05
	Q-1	0.05
	Q-2	0.1
	R	0.09
5	T	0.05*
6	U-1	0.1
	V-1	0.08
	V-2	0.07
7	W-1	0.04
	W-2	0.04
	W-3	0.07
	W-4	0.03
	W-5	0.03
	W-6	0.03
	R-1	0.05
	X-2	0.03
	X-3	0.05
	Y	0.03

*Note: Storms separated by more than 4 days, use 0.8 in. initial loss in June and 1.0 in. in July, August, and September.

The 2016 analysis utilized the SAC-SMA model that included continuous soil moisture accounting and evaporation losses. The NWS models had a cumulative total of 7,066 sub-basins that were each adjusted to calibrate the SAC-SMA parameters. The more physically based SAC-SMA method and greater spatial resolution in the NWS models would be a significant improvement in accounting for losses over the earlier analysis. Base flow was not inherently required by the NWS models because stream flow at index points was continually computed using routing algorithms within the models. Ideally, the effects of parameter calibration on computed peak flows would be investigated using multiple model simulations to evaluate sensitivity and possibly to estimate uncertainty. The high degree of complexity in generating an individual model simulation and the limited time available for the analysis precluded such sensitivity runs. Future analysis should explore model sensitivity to different infiltration calibration parameters.

Fictional Year Assignment – Clipped Merged
The storms were given fictional future dates that used a century of 26 for Clipped-Merged (CM) Sequences plus the HYPO storm number:
58A CM = 2658
52A CM = 2652
56 CM = 2656
63 CM = 2663
This gave a unique time property for the resulting model outputs that helped to keep track of the different runs.

2.2.6 HYPO storm combinations

This assessment reconstructed the 1955 HYPO 58A storm event. Individual precipitation datasets were rebuilt for each storm by converting tabular precipitation data from the 1955 study or from NOAA national archives into spatial data with continual coverage over the entire Mississippi River Basin. Similarly, the hydrologic models also required temperature data obtained from NOAA national archives for the individual storm periods. The point data for precipitation and temperature were interpolated to develop values over the entire basin. This process is described in Section 2.2.7. In addition to the HYPO 58A storm event, precipitation datasets were reconstructed from NOAA archives for the 1955 HYPO 52A, 56, and 63 storm events. The latter three HYPO events were included to assess the new methodology being used.

Following the 2011 flood, the MVD completed a review of the meteorology of the PDF. This review concluded that the current 3-storm combination that makes up the PDF is still adequate, so storms and storm combinations were not reassessed during this review. Considerable effort was spent during the 1955 study to evaluate severe storms and storm combinations. Sections 2.2 and 2.3 give a basic overview of methods used for selection of the HYPO storm combinations. The Hydrology Report (USACE 2018a) shows the historical peaks determined from the 1955 study. More extensive coverage of the procedures can be found in Memorandum Report No. 1 (MRC 1955) (particularly Appendix J) and Hydrometeorological Report Number 34 (Weather Bureau 1956).

Recent major floods have occurred during early spring while the HYPO 58A storm (current PDF storm) produces a winter flood. This brought concerns about (1) the need to include extreme storms that have occurred since 1950 and (2) if another storm with a new higher peak flood might result from a combination from more recent extreme events. For these reasons, it was determined that one additional combination for the 1973 and 2011 events could be included, as discussed in Section 2.2.6.2.

2.2.6.1 Storm sequencing

Each HYPO storm combination prescribed an order for sequencing the historic events. The sequence order was developed as part of the meteorology study described in the 1955 study (See Hydrology Report [USACE 2018a]). In general terms, the combinations include antecedent rainfall with a core period of intense rainfall (two or three historic storms coupled with one or more blocks of time with no rain) immediately followed by another period of rainfall. To facilitate use of continuous simulation hydrologic models, a warm-up period was included prior to the core period. This allowed initial model forcings at the start of simulations to develop an antecedent state based on period input data that lead up to the core period. The model warm-up period was 6 to 9 months for original HYPO sequences with an exception of 10 months for the new HYPO 11-73. The actual length of warm-up depended on the start date for the core period because the simulation needed to entirely span the preceding winter to fully capture water stored as snow pack. A recession period of 60 days following the core period was included to provide sufficient time for flood peaks to route through the lower Mississippi River to the Gulf of Mexico.

As discussed in more detail in the Hydrology Report (USACE 2018a), the 1955 study divided the Mississippi River Basin into seven sub-basins to allow hydrologic hand computations to be completed at a manageable level. This included selectively applying storm sequences across the watershed. The individual storms comprising each HYPO storm were applied to selected sub-basins within the Mississippi River Basin, which allowed separate rainfall/runoff calculations for different sub-basins. This, however, resulted in discontinuous precipitation across adjacent sub-basin boundaries where different rainfall sequences were applied.

The current hydrologic method applies the individual storms across all sub-basins in the time sequence in which they appeared in the HYPO storm. In other words, a storm would begin to develop and then move across the watershed according to the precipitation patterns defined by the underlying historical events and as combined to make up the HYPO storms (just as a radar animation shows storm tracks). There would be no discontinuities in precipitation amounts across adjacent sub-basin boundaries as was the case for the 1955 study.

Clipped-merged and straight sequence comparison

To create the spatial precipitation grids that match the exact sequence presented in the 1955 study, each time series of storm grids was clipped to match its respective coverage area (e.g., sub-areas 1, 2, 3, and 7Y; and 4, 5, 6, 7W, and 7X for HYPO 58A) as detailed in the 1955 report. Each HYPO storm combination was comprised of two sequences except for HYPO 52A that had three sequences. Once each clipping operation was completed for each region, the regions were merged to compile a full grid input for each time-step. This process is referred to as the Clipped-Merged Sequence.

Point precipitation data were interpolated[1] (USACE 2018a) to produce a continuous spatial representation and then converted into ASCII grids that cover the entire Mississippi River Basin. This process is referred to as the Straight Sequence.

Initial results from HYPO 58A unregulated and regulated simulations showed that clipped-merged results were slightly lower than computed using the straight sequence. The difference between Straight Sequence peak flow and Clipped-Merged Sequence peak flow immediately below the Mississippi/Ohio River confluence was approximately +5% (Straight Sequence being higher) for both unregulated and regulated simulations. This level of difference is within the standard accuracy given for flow discharge measurement on the Mississippi River which is ±5%. A recommendation to adopt the current standard methodology following the Straight Sequence approach was made to the Executive Steering Committee (ESC). The ESC subsequently approved this recommendation, and no further analysis was done using the clipped-merged approach (USACE 2018a, Appendix F).

Both the Clipped-Merged and Straight Sequences were evaluated for HYPO 58A to assess how well the new methodology developed spatially continuous hypothetical storms. Table 2-3 compares the HEC-RAS Straight Sequence (58A-U) and Clipped-Merged Sequence (58A-U-CM) peak flow results at various points along the Mississippi River within the MVM against the peak flow values provided in MRC (1955) and USACE (1957). The 1955 study values are labeled as 58A and 58A-EN in the table.

[1] Interpolation was performed using Inverse Distance Weighted methods with bias correction to the observed point data values (see Section 4.8.3 of the Hydrology Report [USACE 2018a]).

Table 2-3. Straight Sequence and Clipped-Merged Sequence peak flow results compared to historic peak flow values.*

Location	Project Design Flood				
	Unregulated Discharge (U), cfs			Regulated Discharge (Existing and Near-Term Reservoirs, EN), cfs	
	58A[1]	58A-U	58A-U-CM	58A-EN[1]	58A-R
Ohio at Cairo, IL	2,460,000	2,458,000	2,392,678	2,250,000	2,326,000
Mississippi/Ohio Confluence (combined)	2,850,000	2,937,000	2,785,000	2,360,000	2,791,000
New Madrid, MO	NA	2,751,000	2,628,000	NA	2,660,000
Caruthersville, MO	NA	2,391,000	2,353,000	NA	2,342,000
Osceola, AR	NA	2,915,000	2,628,000	NA	2,833,000
Memphis, TN	2,770,000	2,956,000	2,758,000	2,410,000	2,863,000
Helena, AR	2,710,000	2,862,000	2,684,000	2,460,000	2,788,000
Arkansas City, AR	3,210,000	3,366,000	3,065,000	2,890,000	3,263,000

*Note: [1] Values obtained from MRC 1955 and USACE 1957.

Generally, the Clipped-Merged Sequence results had peak discharges that were lower than authorized peak PDF flows from Memorandum Report No. 1 (MRC 1955). The Straight Sequence results had peak discharges that were higher than authorized peak PDF flows. Table 2-4 lists the percent difference between the peak flow values.

Table 2-4. Percent differences from the historic 58A peak flow values.

Location	Percent Differences (for flows in Table 2-3)		
	Unregulated Discharges		
	58A-U to 58A	58A-U-CM to 58A	58A-U to 58A-U-CM
Ohio at Cairo, IL	0%	-3%	3%
Mississippi/Ohio Confluence (combined)	3%	-2%	5%
New Madrid, MO	NA	NA	5%
Caruthersville, MO	NA	NA	2%
Osceola, AR	NA	NA	10%
Memphis, TN	6%	0%	7%
Helena, AR	5%	-1%	6%
Arkansas City, AR	5%	-5%	9%

The differences (Table 2-4) between unregulated peaks for the Clipped-Merged and Straight Sequences were not significantly different than the unregulated peak values[1]. Using the 2016 Straight Sequence methodology generally resulted in higher unregulated peak flows coming from the Upper Mississippi, shown in Table 2-3. However, this represented only a small fraction of the total combined confluence flow of the Mississippi and Ohio Rivers. The combined difference represented 3%–6% increases in unregulated peak flow compared to the 1955 unregulated peak values for the MVM reach. Since the Straight Sequence follows current methodologies and has values within an acceptable range of the authorized peak flows from the 1955 analysis, only the Straight Sequence approach was used for the remaining HYPO storm simulations.

2.2.6.2 Evaluation of new HYPO storm sequence

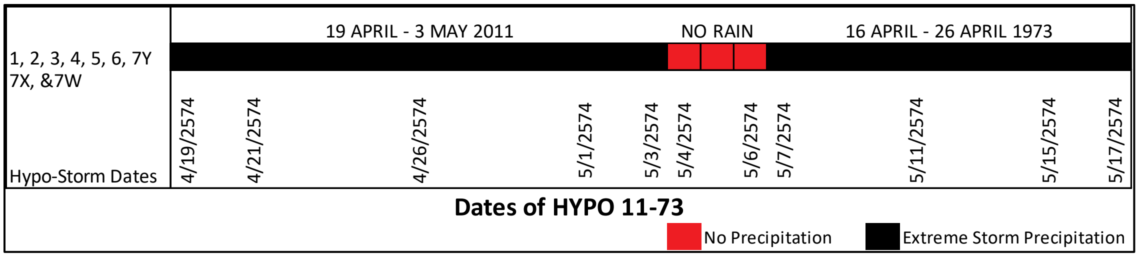
Recent major floods have occurred during early spring while the HYPO 58A storm (current PDF storm) produces a winter flood. For this reason, there were concerns about (1) the need to include extreme storms that have occurred since 1950 and (2) if another storm with a new higher peak flood might result from a combination from more recent extreme events.

The current assessment developed one additional storm combination that was prepared following the methodology outlined in HMR 35 (Weather Bureau 1959) and Memorandum Report No. 1 (MRC 1955) because there were concerns that more recent extreme storms in a different season might produce higher flood peaks than HYPO 58A. The new storm combination, named HYPO 11-73, was based on events that produced the 1973 and 2011 floods on the Lower Mississippi River as detailed in the Hydrology Report (USACE 2018a). The 2011 and 1973 storms could be combined under general meteorological criteria applied for the 1955 HYPO storms.

This new HYPO 11-73 storm combined those events and consisted of three distinct periods of intense rainfall placed in sequence. Individual precipitation datasets were built for each storm for point precipitation and temperature data that covered the entire Mississippi River Basin. The point data were interpolated to develop values over the entire basin following the same methodology used to develop the other HYPO gridded data sets. HYPO 11-73 consists of the April 16–26, 1973, and the April 19 – May 3, 2011, over all Areas 1, 2, 3, 4, 5, 6, 7X, 7W, and 7Y, where the sequence of the event is shown in Figure 2-4.

[1] Significant difference was defined by expert opinion of the ESC as deviations greater than 10%.

Figure 2-4. HYPO 11-73 Straight Sequence.



The HYPO 11-73 storm was simulated in the CHPS-FEWS hydrologic models to produce boundary hydrographs for the HEC-RAS unsteady model. The results show that HYPO 58A is still the governing storm event for the PDF as the resulting peak flows from HYPO 11-73-U were lower than determined for HYPO 58A-U for locations downstream of the Ohio River confluence, as shown in Table 2-5.

Table 2-5. HYPO 11-73 results compared to 1955 HYPO storm results.

River	Location	Peak Flow (kcfs)*	
		HYPO 58A-U	HYPO 11-73-U
Mississippi	St Louis, MO	433	1,024
Missouri	Hermann, MO	220	668
Ohio	Metropolis, IL	2,461	1,734
Mississippi	Cairo, IL	2,937	2,806
Mississippi	Arkansas City, AR	3,367	3,286
White	Clarendon, AR	271	311

*kcfs = 1000 cubic feet per second

2.2.7 Storm development

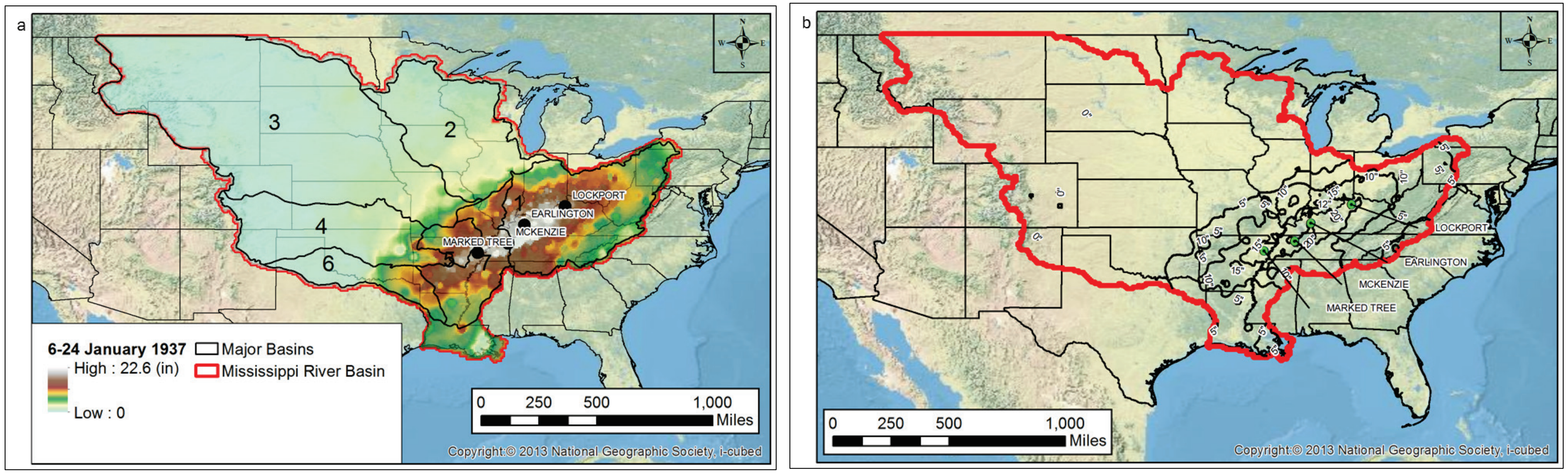
Historic precipitation data to develop the storms needed for inclusion in the HYPO storm events were obtained from the original working files and from old reports like Memorandum Report No. 1 (MRC 1955) dated 1955. Generally, the data were 6-hour point values for the storm duration. To facilitate the conversion of point data to a spatial representation of the data, the point values were entered into Excel to build a database for each separate historic event. Once the database had been completed, the data were interpolated using ESRI Arcmap. The interpolation was first tried using Kriging techniques. Each interpolated 6-hour interval was then summed over the storm period. Isohyetal contours were generated for the interpolated data. The resulting isohyets were then compared to maps of isohyets in the published reports. The comparison was visual by the NWS and USACE meteorologists. As the process developed, a more rigorous comparison method was adopted to check areas within each isohyet.

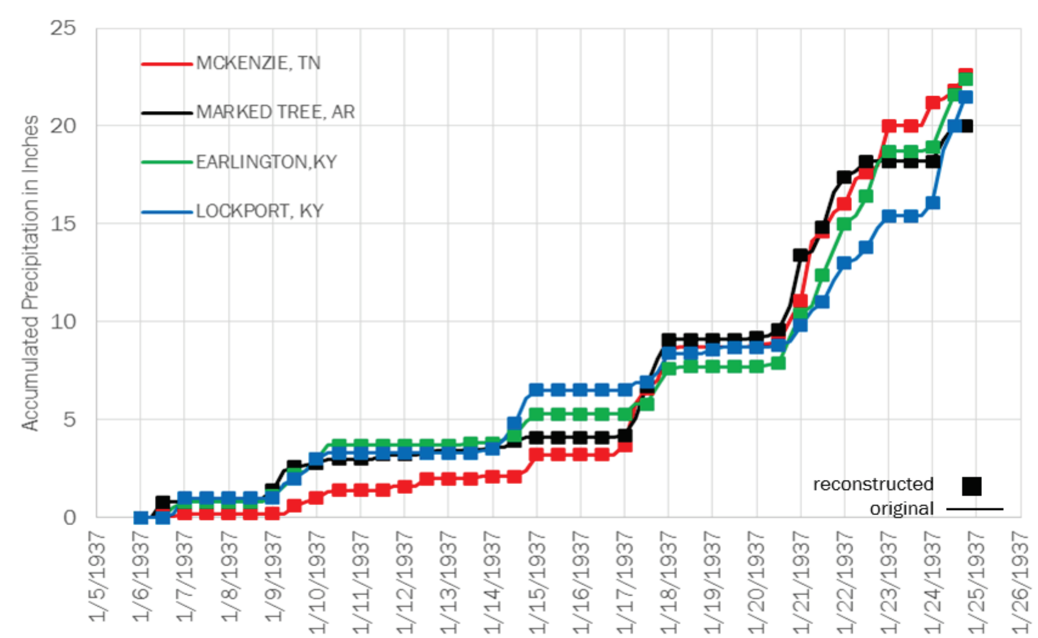
An extended simulation period was necessary to allow the NWS CHPS models to arrive at the appropriate antecedent state at the beginning of the actual storm. The team determined that a 6-month warm-up period would be sufficient and there needed to be a post-event period to permit the unsteady model sufficient time to fully route the event all the way to the Gulf of Mexico. This proved sufficient except for events beginning in April or later; for these events, the warm-up period was extended so that the model could account for snow water equivalents (SWE) from when the first snow fall occurred. The post-event period was first set to 1 month past the last day of the event but was later extended an additional month—2 months total. This required extending the point datasets to cover an approximate 10-month simulation period.

HYPO 58A Storm Precipitation

HYPO Storm 58A is comprised of three historical storm events centered primarily over the Ohio, Red, and White River Basins. The HEC transposed one storm in this HYPO event according to parameters given in the 1955 documentation (MRC 1955) (Chapter 2.2.8). Figure 2-5 through Figure 2-7 depict the total precipitation and coverage of each storm event that compiles HYPO 58A. Figure (a) in each of the figures depicts the storm coverage of the storm event. Figure (b) depicts the isohyetal for the original storm event. Figure (c) looks at several locations and compares the original precipitation inputs against the processed inputs after they were converted to a raster. Table (d) tabulates the 1955 values, the original 2016 inputs, the processed 2016 inputs, and (if applicable) the transposed 2016 processed inputs. Each HYPO storm in the 2016 simulations included a warm-up period to capture the snow pack that would have been lost otherwise. A recession period following the storm sequence was included to provide sufficient time to route the hydrograph peak downstream to the Gulf of Mexico. Figure 2-8 shows the warm-up and recession period for HYPO 58A.

Figure 2-5. HYPO 58A – 6-24 January 1937 (a) storm coverage over the Mississippi River Basin; (b) isohyetal for unadjusted January 1937 storm; (c) comparison of mass rainfall curves for original point precipitation data versus the processed/interpolated precipitation data for the unadjusted storm; and (d) comparison of 1955 precipitation, 2016 precipitation, and 2016 post-processed precipitation at select locations.





Notes:
Reconstructed cumulative precipitation data is represented by square symbols.
Original cumulative precipitation data is represented by a solid line.

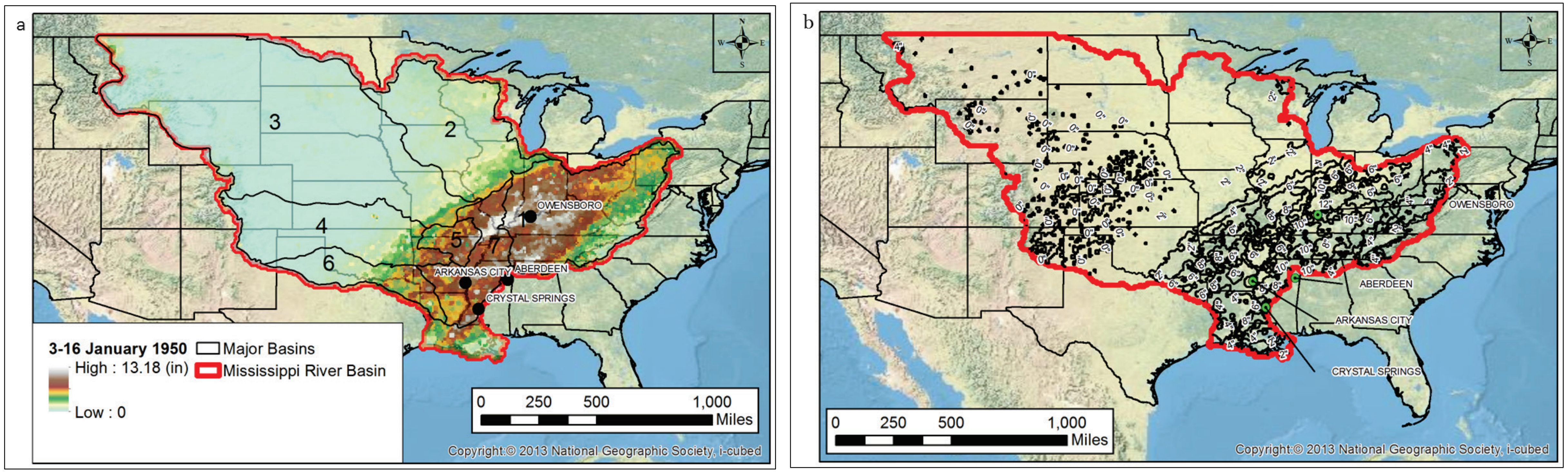
d

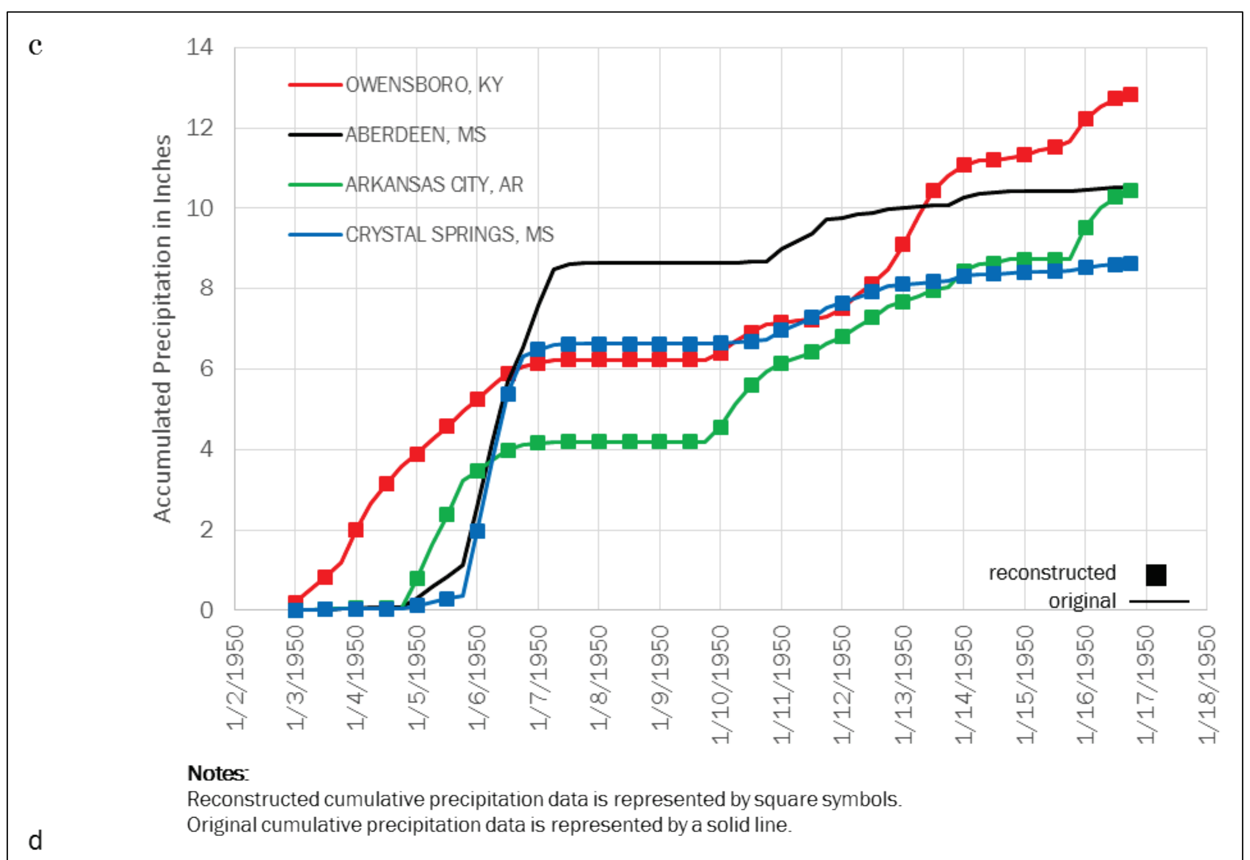
d

Location	6–26 January 1937 Precipitation² (in.)		
	1955¹	2016	2016 Reconstructed
McKenzie, TN	22.60	22.60	22.60
Marked Tree, AR	20.00	20.00	20.00
Earlington, KY	22.40	22.40	22.40
Lockport, KY	21.60	21.50	21.50

Notes
¹ Derived by digitizing the 1955 isohyet map and interpolating to obtain values.
² Values are compared against the original values, not the 10% increase.

Figure 2-6. HYPO 58A – 3-16 January 1950 (a) storm coverage over the Mississippi River Basin; (b) isohyetal for storm; (c) comparison of mass rainfall curves for original point precipitation data versus the processed/ interpolated precipitation data; and (d) comparison of 1955 precipitation, 2016 precipitation, and 2016 post-processed precipitation at select locations.



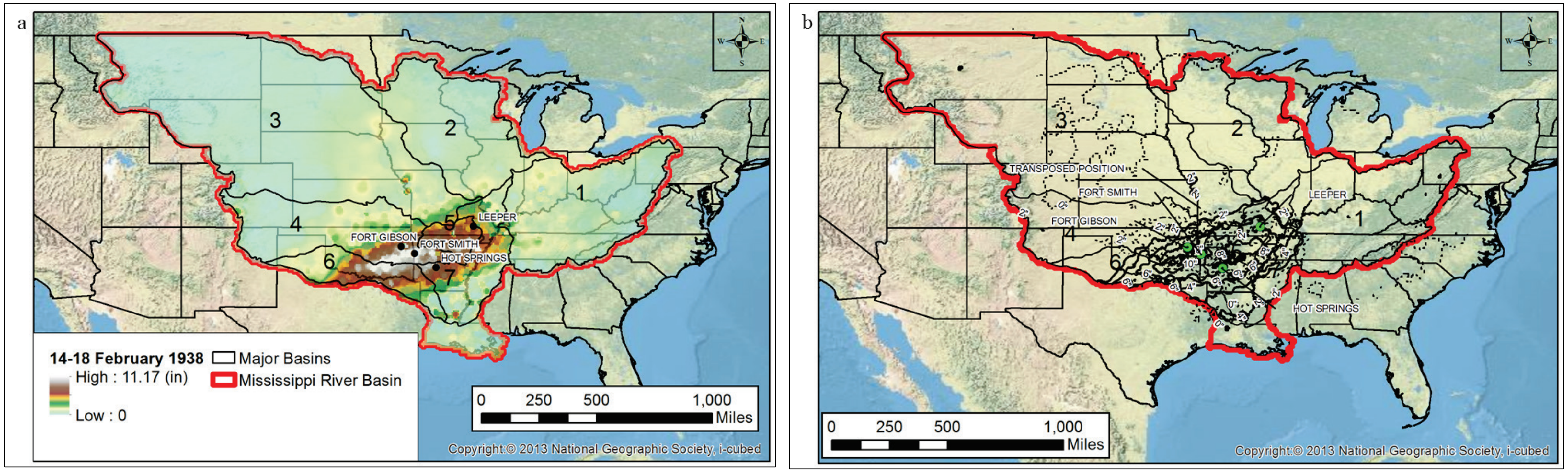


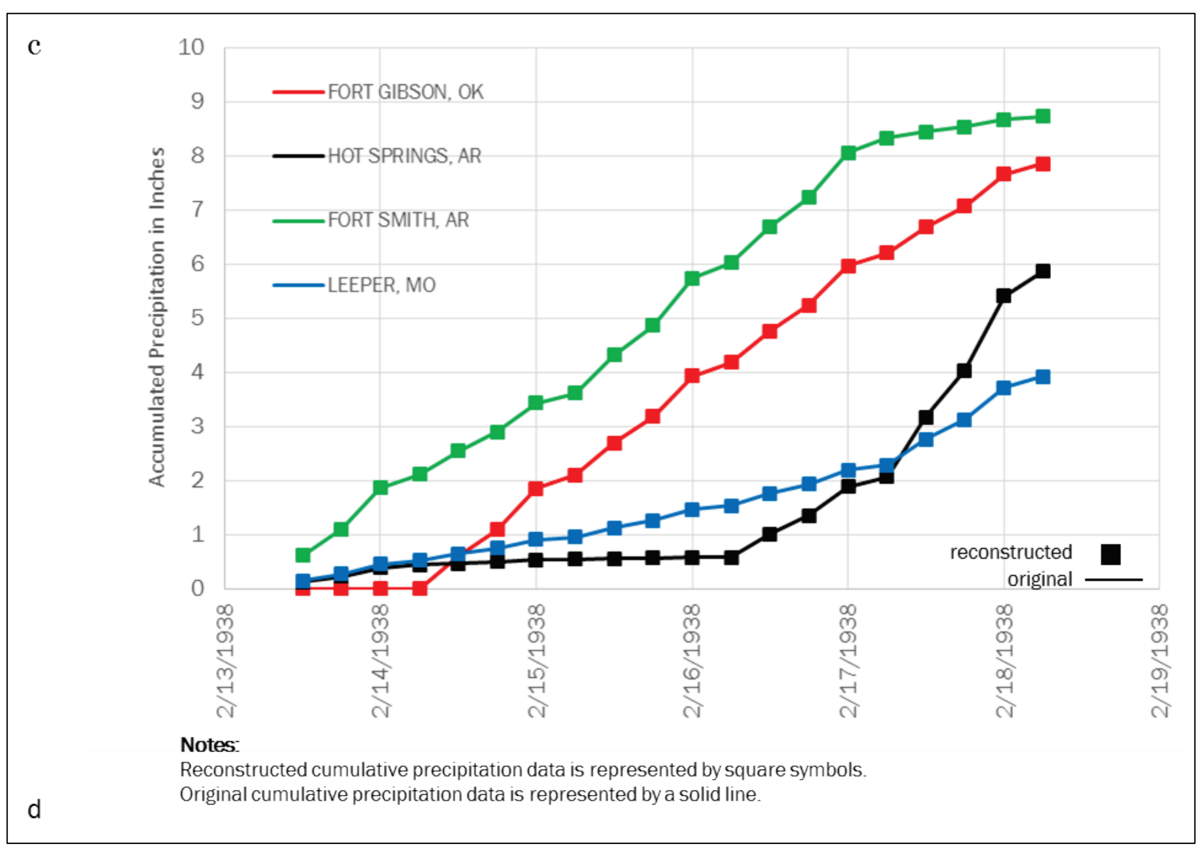
d

Location	3–16 January 1950 Precipitation (in.)		
	1955[1]	2016	2016 Reconstructed
Owensboro, KY	13.20	12.83	12.83
Aberdeen, MS	10.6	10.53	NA[2]
Arkansas City, AR	10.80	10.44	10.44
Crystal Springs, MS	9.89	8.62	8.62

Notes
[1] Derived by digitizing the 1955 isohyet map and interpolating to obtain values.
[2] Location not included in results as Aberdeen, MS falls outside of the MS River Basin.

Figure 2-7. HYPO 58A – 14-18 February 1938 (a) storm coverage over the Mississippi River Basin; (b) isohyetal for storm with and without transposition; (c) comparison of mass rainfall curves for original point precipitation data versus the processed/interpolated precipitation data for the storm with and without transposition; and (d) comparison of 1955 precipitation, 2016 precipitation, 2016 post-processed precipitation, and 2016 transposed precipitation at select locations.



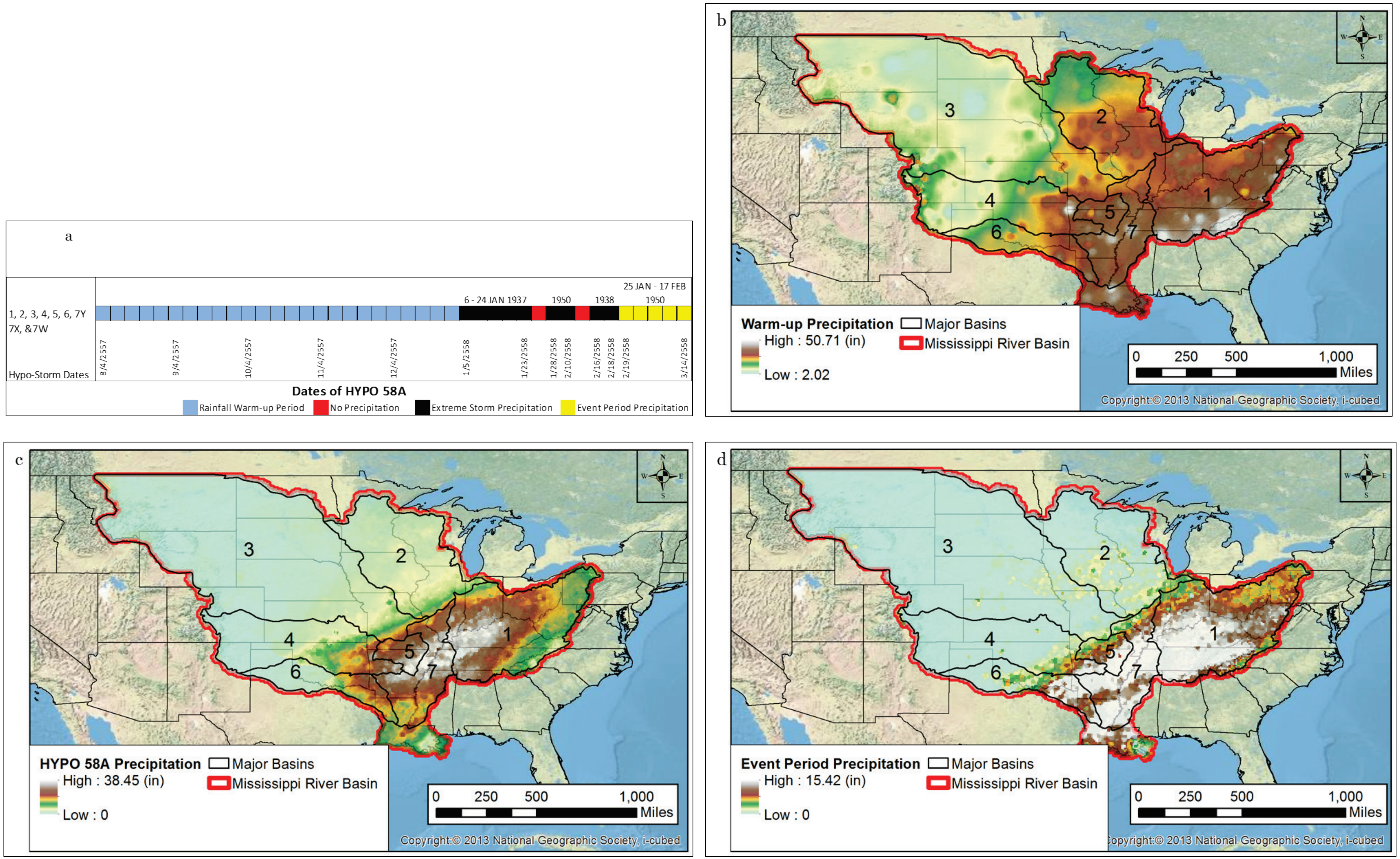


d

Location	14–18 February 1938 Precipitation (in.)			
	1955[1]	2016	2016 Reconstructed	2016 Reconstructed and Transposed
Fort Gibson, OK	8.83	7.86	7.86	7.90
Hot Springs, AR	4.28	5.87	5.87	5.80
Fort Smith, AR	6.70	8.73	8.73	8.70
Leeper, MO	4.92	3.92	3.92	3.90

Note: [1] Derived by digitizing the 1955 isohyet map and interpolating to obtain values.

Figure 2-8. HYPO 58A (a) storm sequencing with warm-up and cool down periods; (b) warm-up precipitation coverage over the Mississippi River Basin; (c) 1937, 1950, and 1938 combined storm coverage over the Mississippi River Basin; and (d) cool down precipitation coverage over the Mississippi River Basin.



The January 1937 storm had the same gage locations available to make a direct comparison of the 2016 results to the gage locations in the 1955 report. The tabular data and cumulative precipitation plot (Figure 2-5) indicate that the original 1955 values are almost an exact match with the precipitation data used to regenerate this storm for the current assessment at the prescribed locations. After the data were processed and converted to an ASCII grid, the values still matched the 2016 input point data.

The January 1950 storm had three of the same gage locations available to make a direct comparison of the 2016 results to the gage locations in the 1955 report. The tabular data (Figure 2-6) shows slight deviations between the original 1955 values and the 2016 values compiled from the archives; however, once the 2016 data were converted to an ASCII grid, each location matched the input value. The 2016 processed value for Aberdeen, MS, was not included because this location is just outside of the Mississippi River Basin.

The February 1938 storm did not have the gage data available to make a direct comparison of the 2016 results with the gage locations used in the 1955 report, so the isohyetal map in the 1955 report was digitized to obtain the values at select locations (Figure 2-7). The deviation between the 1955 data and the 2016 data ranges from 0.97 in. to 2.03 in. Considering the contour lines from the 1955 report are at 2 in., 5 in., and 10 in. intervals, the deviations between the 1955 and 2016 values fell within anticipated accuracy ranges given the coarse precision of the historic data. After the 2016 data were processed, the new grids retained the original input gage value. After the raster was transposed (see Section 2.2.8), the value of that grid cell lost one significant digit and was rounded to the nearest tenth of an inch.

Figure 2-8 shows the complete storm sequence for HYPO 58A that includes the warm-up period, the extreme storm event period, and the event period precipitation.

2.2.7.1 Temperature data

The 1955 analysis did not include temperature effects in any hydrologic computations. Instead, any snow melt that occurred due to temperature effects was embedded in the measured flow records that were used to estimate base flow and other hydrograph characteristics.

The NWS hydrologic models used for the current assessment included temperature effects using the Snow-17 model that maintained accounting of water equivalents and depths throughout the simulations. Required temperature data for each historical storm event were compiled by USACE from NOAA archives. These point data were then interpolated to generate a complete spatial grid of temperatures needed for CHPS-FEWS simulations. The Hydrology Report (USACE 2018a) provides the details for point data interpolation.

2.2.7.2 Temperature effects on peak flow

In the USACE HYPO 58A temperature dataset, temperatures in the upper mid-west and the northern Mississippi River Basin were much warmer than general historic trends for the January/February timeframe. The NCRFC noted the higher temperatures as one possible reason why CHPS model results produced higher flows than in the 1955 analysis. After thorough checks and an extensive review of the data, it was confirmed that the higher-than-expected temperatures during late January and early February agreed with observed temperature data for the 1937 event. Meteorologists at least partially attributed the unusually warm period during January and February 1937 as a causal factor that produced the major flood that occurred in that year.

To assess the influence of temperature on model results, the NCRFC and Missour Basin RFC (MBRFC) simulated the HYPO 58A precipitation inputs with two different temperature datasets in the CHPS model for each region. Figure 2-9 presents a sample map for one time-step based on original USACE temperatures derived from historical 1937 data; Figure 2-10 presents a sample map developed using historical average temperatures for the identical simulation time-step. The original USACE temperatures derived using actual storm event data were the first dataset used. For the second temperature dataset, both NCRFC and MBRFC applied a correction factor to those periods where temperatures were outside the normal expected range of values. The correction factor lowered the grid values up to 40˚F, which was derived from a best fit line over historical average temperature data.

The USACE dataset temperatures for 1937 from the national archives for January 24–27 and for February 11–13 resulted in no SWE going into February for the NCRFC area. With the modified temperatures, SWE accumulated throughout the winter season until March. MBRFC reported similar results with the exception that HYPO 58A precipitation amounts over its region did not produce significant SWE even with the modified temperatures. Table 2-6 compares the original USACE temperature dataset flow results with the RFC modified temperature flow results at various locations within the NCRFC region. Minimal difference exists between the peak flows at St. Louis, MO, and Chester, IL (within approximately ±3%). However, Figure 2-11 shows that the timing of the peak is significantly different. The USACE temperature dataset, which included actual temperature data from the 1937 event, produced a peak much earlier than occurred when using historic normal temperatures.

Figure 2-9. Sample map of temperatures from HYPO 58A: USACE dataset developed from NOAA archives for 1937 period.

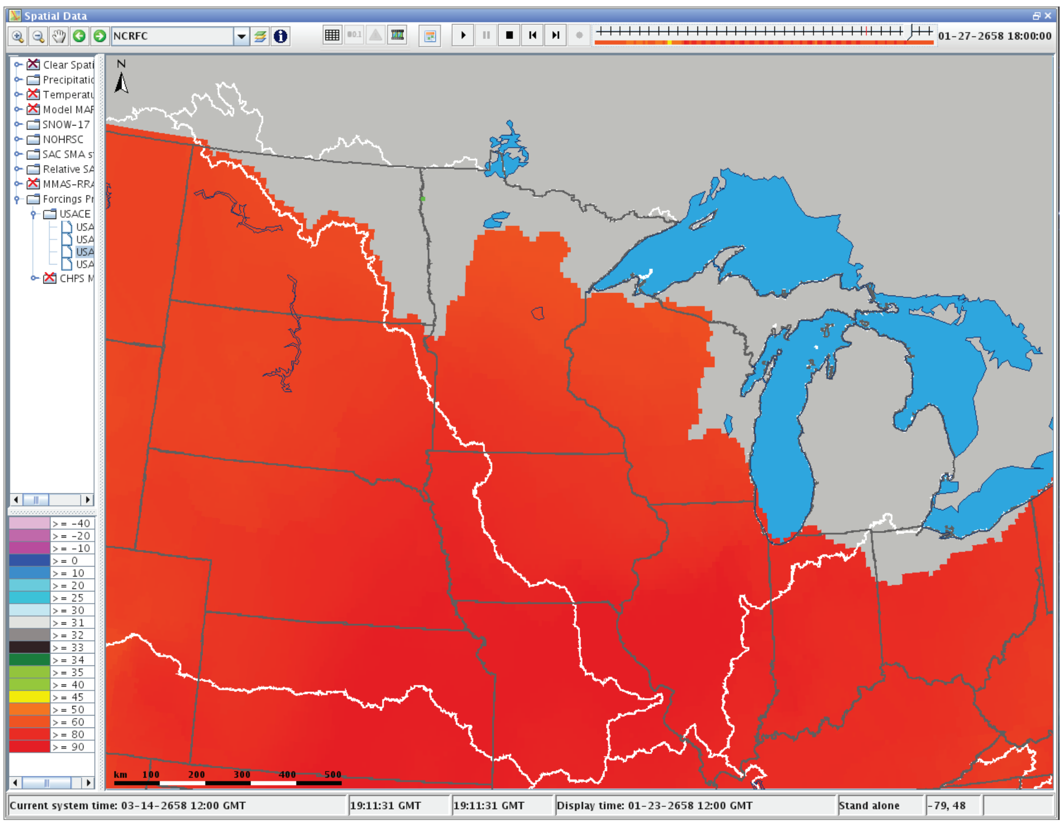


Figure 2-10. Sample map of temperatures from HYPO 58A: NCRFC historical average temperatures.

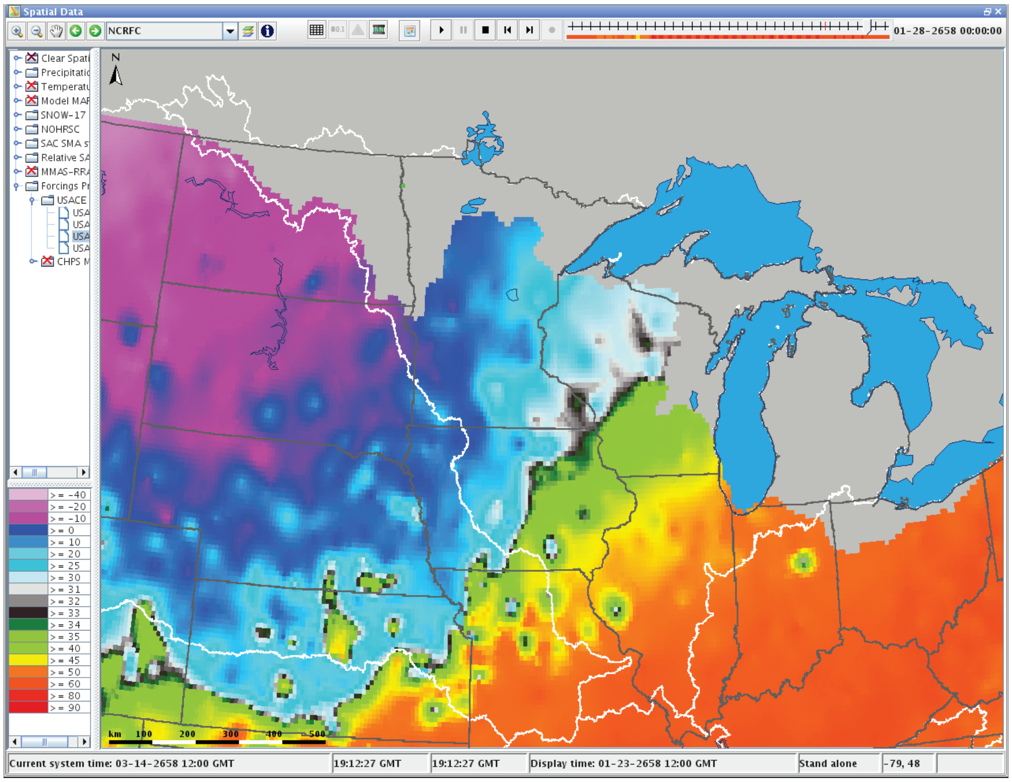


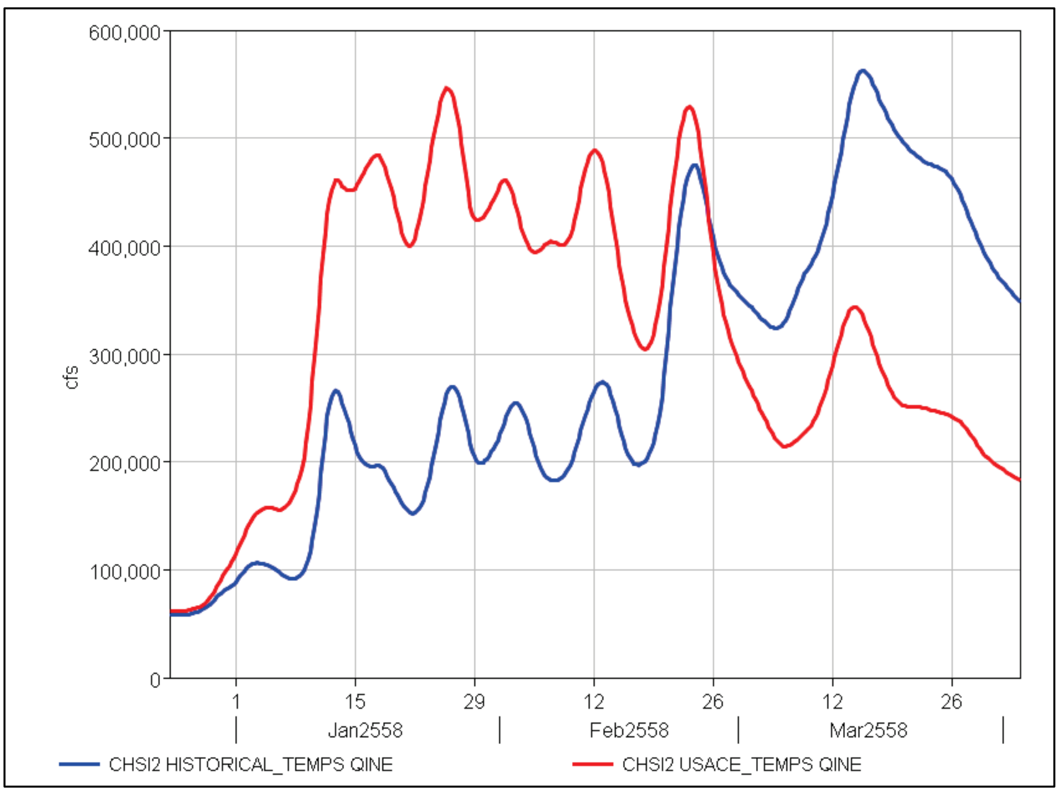
Table 2-6. Temperature effects on computed peak unregulated flows at select locations for HYPO 58A.*

River Forecast Center	Location	Gage Name	River	Peak UNREGULATED Flow in 1,000 cfs (CHPS)		
				HYPO 58A USACE Temperature Dataset	HYPO 58A RFC Modified Temperatures	Difference [1] (percent)
NCRFC	L/D 20		Mississippi	106	63	41%
NCRFC	L/D 21		Mississippi	107	67	37%
NCRFC	L/D 22		Mississippi	109	74	32%
NCRFC	L/D 24		Mississippi	111	80	28%
NCRFC	L/D 25		Mississippi	111	80	28%
NCRFC	GRFI2		Mississippi	195	175	10%
NCRFC	**EADM7**	**St. Louis, MO**	**Mississippi**	**321**	**316**	**2%**
NCRFC	**CHSI2**	**Chester, IL**	**Mississippi**	**546**	**562**	**-3%**
NCRFC	VALI2		Illinois	102	102	0%
NCRFC	AGSI4		Skunk	5	5	0%
NCRFC	MLLI2		Rock	15	16	-7%
NCRFC	WAPI4		Iowa	13	8	38%
NCRFC	KEQI4		Des Moines	2	3	-50%
NCRFC	ERKM7		Meramec	41	47	-15%
NCRFC	MURI2		Big Muddy	31	31	0%
NCRFC	NASI2		Kaskaskia	37	38	-3%

*Note: [1] Percent differences shown are greatest in Iowa and Minnesota or where computed peak values are small.

Table 2-6 shows discharge hydrograph peak unregulated flows for the Mississippi River at Chester, IL, for the NCRFC outputs using the USACE temperature dataset and RFC modified temperatures. As seen in Figure 2-11, the flow from the USACE HYPO storm sequence temperature dataset for 1937 peaks at Chester, IL, on January 25, while Cairo, IL, peaks on January 26. There is a significant increase in flow between January 8 and 12 with a lesser peak on January 17 that leads up to the second larger peak on January 25. The RFC modified temperature simulation for 1937 produced a peak flow on March 15 at Chester, IL, well after the HYPO 58A peak at Cairo, IL, with only relatively minor flows occurring until approximately February 20. There is a 276,000 cfs difference between flows computed at Chester, IL, using the two different temperature inputs for the January to mid-February period.

Figure 2-11. NCRFC computed flows for Mississippi River at Chester, IL, using two different temperature inputs; HYPO 58A unregulated flow.



The 1955 analysis did not directly consider temperature effects on SWE accumulation; rather, it used gage flows for the relevant time periods to develop base flows and unit hydrographs. Using this approach biased outcomes toward normal historic conditions and did not directly consider abnormal effects of individual extreme storms such as evident in the temperatures from the 1937 event. It is therefore likely that flow contributions from the upper Mississippi and Missouri River basins were understated in 1955 results given for HYPO 58A. The current assessment utilized historic temperatures that captured the unique aspects of the actual extreme events thereby providing a better representation.

2.2.8 Storm transpositioning

The original development of the HYPO storm series included meteorological adjustment of historic extreme storms to maximize their effects on parts of the Mississippi River Basin. This included transposing, that is rotating and moving, storms over different parts of the watershed from where they actually occurred. Transpositioning storm events is commonly used to shift the location of precipitation over the watershed. All transpositioning adjustments only applied to location of rainfall and storm extent; there was no impact on the size or extent of the watershed boundaries or their characteristics.

The Hydrologic Engineering Center (HEC) designed and implemented enhancements to the HEC-MetVue program, version 2.2.8.10, to provide a command-line utility for rotation and translation of ESRI ASCII precipitation grids to rotate and translate two historical storm events.

2.2.8.1 HEC-MetVue enhancements

HEC-MetVue transformed the original grid into an equal-area projection to preserve the original precipitation volume, then rotated the grid about the center of the specified coordinates, translated the rotated grid in the specified east-west and/or north-south direction(s), and re-sampled the rotated and translated grid back into the original coordinate system. The HEC-MetVue enhancements allowed the user to specify, from a command line or script, the center, rotation angle, and translation and output new ASCII grids with the same georeferencing and spacing as the source data. The rotation angle was specified in decimal degrees with positive numbers denoting counterclockwise rotation and negative numbers denoting clockwise rotation. The translation distance may be specified in degrees or linear units, such as miles. HEC-MetVue triangulated the input data to a Triangulated Irregular Network (TIN) and, by default, resampled the values using bi-linear interpolation.

To best preserve the original maximum and minimum precipitation values, an option was added to HEC-MetVue to perform nearest neighbor resampling, selecting the value from the nearest grid cell within a specified range, expressed as a multiple of the input grid cell size. The user controlled this interpolation option by specifying a search radius as a multiplier of the cell size or specifying NONE to use bilinear interpolation to set all output values.

2.2.8.2 Post-processing

HEC developed a Python (www.python.org) script to automate the processing of a large number of ASCII grids. The script can process all ASCII grids in a specified folder or a sub-set of these grids using a specified time window. HEC-MetVue determined boundaries for the output grids to include the original data as well as any necessary increase in extents to accommodate the rotated and translated data. To meet model requirements, the transposed grids must have the same bounds as the original grids; HEC developed a Python script to clip or fill the boundary with nulls to match the boundaries of the original grids.

2.2.9 Reservoir regulation effects

PDF hydrology from the 1955 study began using unregulated flows calculated by applying unit hydrograph theory, observed historic flow data, and hydrologic routing. The outputs from this analysis were generally in the form of peak discharges with some hydrographs documented at key locations along the main-stem Mississippi River. Reservoir effects were based on a determination of the percentage reduction that would result from the EN Group (see Hydrology Report, USACE [2018a]). The 1955 report did not document development of regulated outflow values from each reservoir listed under the EN group (151 reservoirs in total). However, each USACE office at the time was asked to provide percentage reductions from each major tributary system, which included any effects from reservoirs in the EN group. Of the original 151 reservoirs listed in the EN group, some were not constructed while others that were in the D group (distant future) have actually been constructed (see the Hydrology Report, USACE [2018a]). For this reason, current analyses considered only reservoirs that existed at the beginning of the investigation (circa 2014).

For the 2016 PDF update, reservoir effects were assessed by coordinating with each USACE region. While using percentage reductions as a way to capture reservoir effects was plausible for adjusting peak flow, this method was not appropriate where continuous flow hydrographs are required for unsteady flow models. Therefore, reservoir routings were performed to provide the full hydrographs needed by the unsteady flow models.

USACE Districts

LRL	= Louisville District
LRN	= Nashville District
MVK	= Vicksburg District
MVR	= Rock Island District
MVS	= St. Louis District
NWD-MR	= Northwestern Division Missouri River
NWK	= Kansas City District
NWO	= Omaha District
SWF	= Fort Worth District
SWL	= Little Rock District
SWT	= Tulsa District
TVA	= Tennessee Valley Authority

2.2.9.1 Reservoir selection

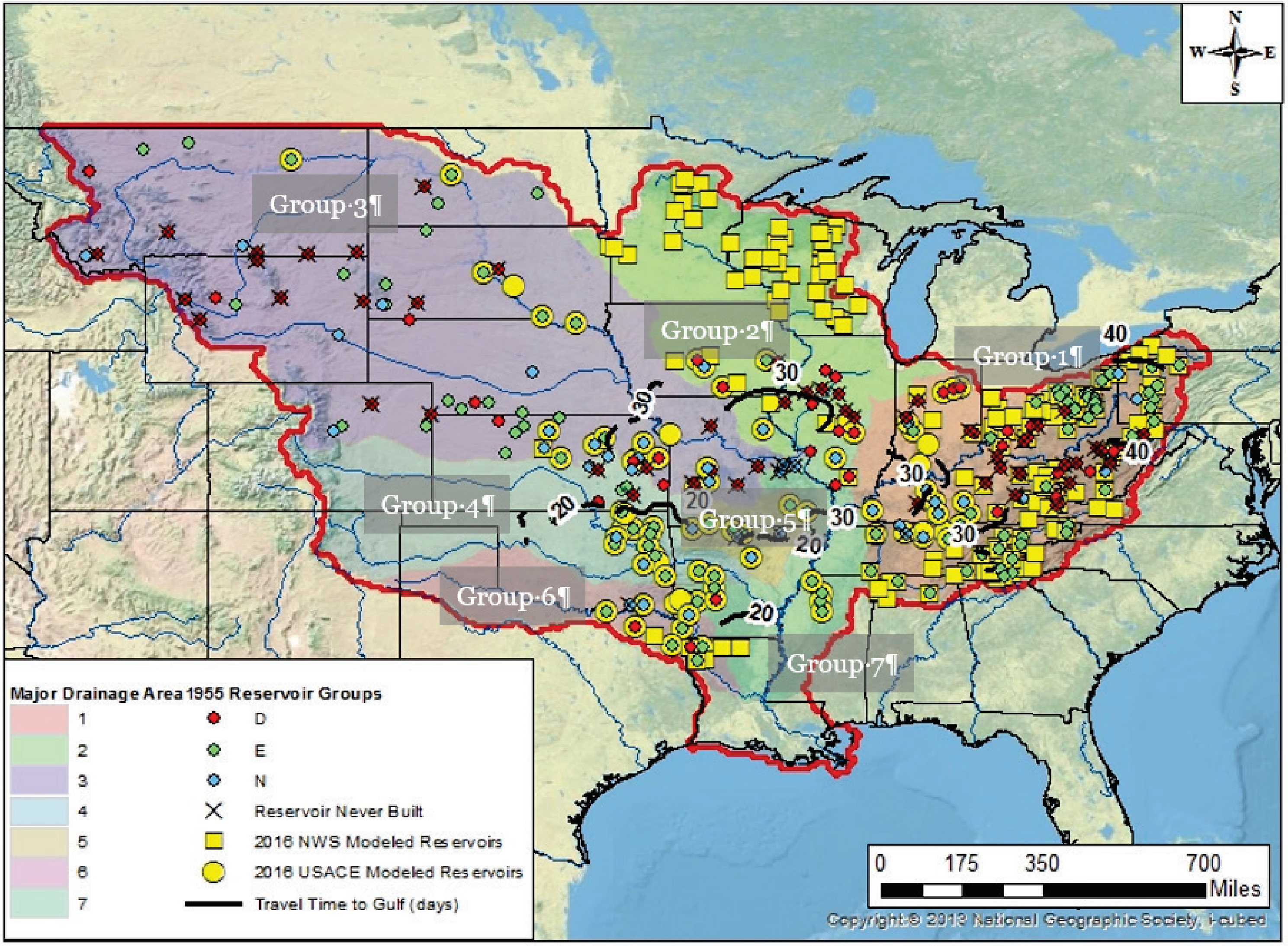
Each USACE office made a qualitative evaluation to determine which reservoirs should be included in routing computations to determine regulated outflows for each HYPO storm combination based on the HYPO storm precipitation maps and known tributary travel time to the Lower Mississippi River/Gulf of Mexico. Reservoirs were selected based on their flood storage capacity, location relative to major rivers and other reservoirs, and release travel time to the Lower Mississippi River. The reservoirs identified in this manner were included in detailed routing computations by each respective USACE district. Reservoirs located outside of a 30-day tributary travel time to the Gulf of Mexico were deemed unlikely to have any reduction effect on a flood occurring on the Lower Mississippi River; however, CHPS-FEWS RES-SNGL and RES-J configurations were utilized to model reservoirs beyond the 30-day travel time to generate hydrograph inputs needed for the hydraulic model of the Lower Mississippi River. Some reservoirs were included in the detailed USACE routing computations even though they were screened out as having no expected impact on main-stem Mississippi River flows. Such is the case for several large reservoirs located on the Upper Missouri River. These reservoirs were included because of their perceived impacts to capture any possible effects from their operation.

Models

CWMS	= Corps Water Management System
HEC-ResSim	= Reservoir System Simulation
HEC-HMS	= Hydrologic Modeling System
MFP	= Meteorological Forecast Processor
CHPS	= Community Hydrologic Prediction System
FEWS	= Deltares Flood Early Warning System

Reservoir effects for the updated hydrology were calculated using available models and techniques. Reservoirs expected to have significant potential to reduce flood peaks were modeled by the respective USACE or TVA offices. More detail is provided in the Hydrology Report (USACE 2018a). USACE and NWS reservoirs modeled in the current analysis are shown in Figure 2-12 along with the travel time to the Gulf of Mexico. State-by-state reservoir maps are available in Appendix J of the Hydrology Report (USACE 2018a). Other reservoirs that were expected to have minor or no impact were modeled by the NWS using their existing RES-SNGL (single reservoirs) or RES-J (single reservoirs and systems of reservoirs) configurations in CHPS-FEWS.

Figure 2-12. 2016 reservoirs modeled in current analysis.*



*Note: Symbols illustrate which agency's modeling efforts addressed regulation effects of reservoirs; they do not indicate the agency responsible for daily operations of those projects.

2.2.9.2 Reservoir routing

Detailed reservoir routing was accomplished using current water management plans to determine gate and pool operations. Each project was evaluated to develop regulated releases by applying the established guide curves in a general sense. While it would have been possible to route each event with the intention of maximizing or optimizing the regulation effect, it was determined that such analysis would not be consistent with how a water manager would operate the project during an actual event. This was because a water manager would effectively know the future for the PDF simulations; that would not be the case should an actual event occur that might have characteristics of the PDF.

There was a multi-step process required to produce the final regulated outputs from the NWS hydrologic models. First, the NWS RFCs produced model outputs at each reservoir for project inflow and local contributions. These outputs were then provided to each USACE office for use as input to its regulation computations. Each USACE office had water management staff members perform the regulation based on their experience with floods and the projects' operating guide curves. For projects in sequence, the regulation included joint operations. Finally, the regulated outflows from each USACE office were then passed to the respective RFC for input to models and simulation of the final regulated run. Each respective office provided the regulation and routing for reservoirs under its control.

The current CHPS-FEWS models include various reservoirs with configurations that vary by RFC. Some of the configurations provided a simplified means to route events through the outlets; others required input of observed reservoir releases. Many of the reservoirs that exist today did not exist when the historic storms occurred. The Hydrology Report (USACE 2018a) gives a list of reservoirs and the dates when regulation began for select reservoirs.

2.2.9.3 Reductions from standard reservoir operations

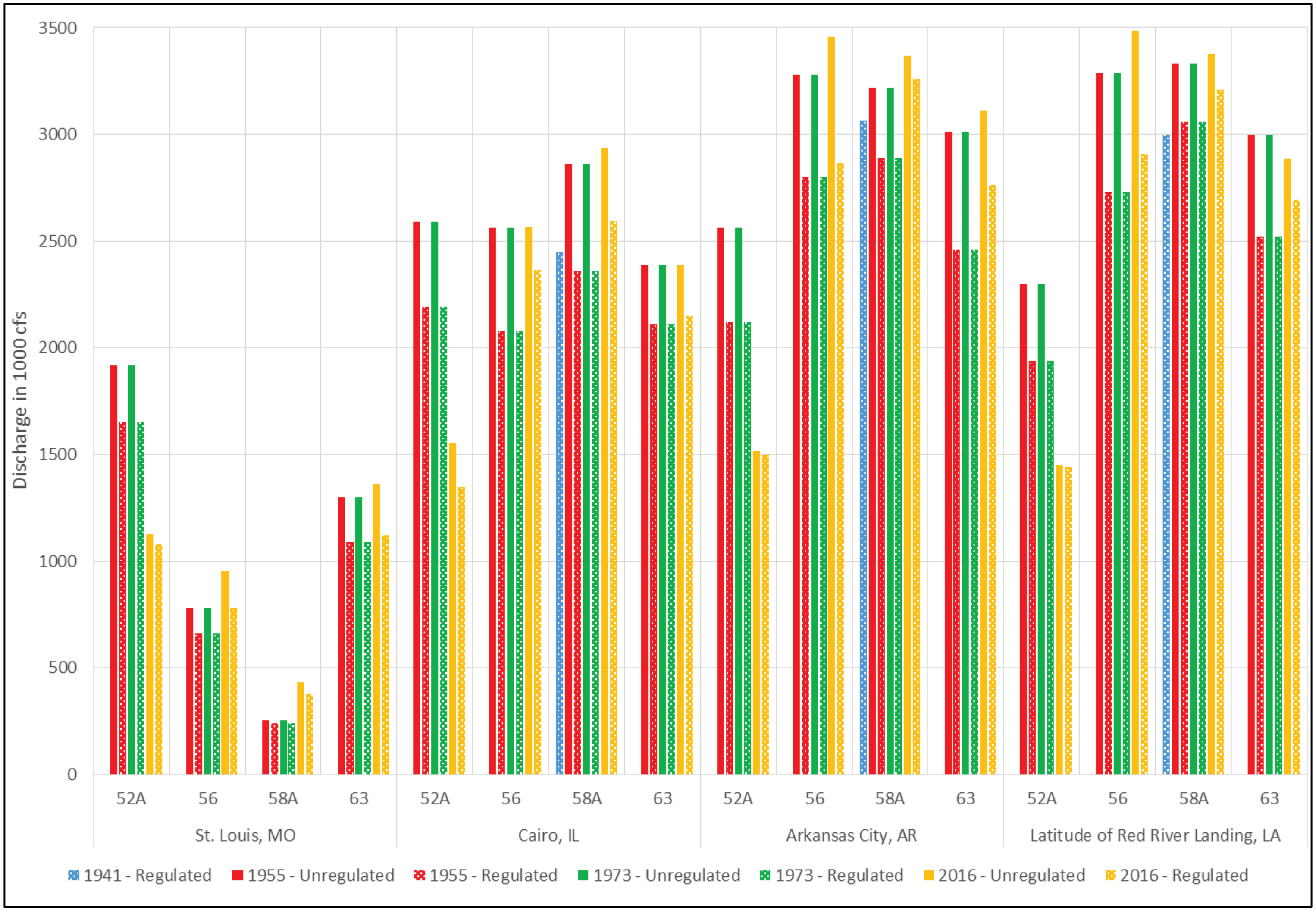
The reductions due to reservoir regulation were evaluated by calculating the difference between peak flows (regardless of times when peaks actually occurred) for the unregulated and regulated discharge hydrographs. This calculation was made for peaks published in MRC (1955) and at those same locations for the current assessment. Table 2-7 lists the calculated percent reductions and differences between the 1955 and 2016 flow reductions at nine locations.

Figure 2-13 shows the differences calculated for each of the four HYPO storms for four location.

Table 2-7. Reservoir peak flow reductions in percent for four HYPO storms.

Reductions from Regulation										
HYPO	Description	Based on Peak Flows (cfs)								
		Alton, IL Mississippi River	Hermann, MO Missouri River	Little Rock (Pine Bluff), AR Arkansas River	Clarendon, AR White River	Alexandria, LA Red River	St. Louis, MO Mississippi River	Metropolis, IL Ohio River	Cairo, IL Combined Ohio and Mississippi River	Arkansas City, AR Mississippi River
1955										
52A	% Reduction	14%	15%	31%	28%	15%	14%	10%	16%	16%
58A	% Reduction	0%	12%	42%	28%	30%	5%	9%	17%	10%
56	% Reduction	5%	21%	44%	26%	37%	15%	19%	18%	14%
63	% Reduction	1%	25%	7%	24%	16%	16%	5%	11%	14%
2016										
52A	% Reduction	3%	10%	6%	19%	20%	3%	*20%*	*11%*	*1%*
58A	% Reduction	7%	12%	7%	17%	19%	13%	*5%*	*5%*	*2%*
56	% Reduction	10%	32%	22%	42%	61%	12%	*5%*	*5%*	*10%*
63	% Reduction	6%	28%	10%	38%	15%	16%	*8%*	*0%*	*8%*
Difference between 1955 reductions and 2016 Reductions										
Change in % Reduction: 1955 versus 2016 [negative = decrease in 2016; plus = increase in 2016]										
52A	% Reduction	-11%	-5%	-25%	-9%	5%	-11%	10%	-5%	-15%
58A	% Reduction	7%	0%	-35%	-11%	-11%	8%	-4%	-12%	-8%
56	% Reduction	5%	11%	-22%	16%	24%	-3%	-14%	-13%	-4%
63	% Reduction	5%	3%	3%	14%	-1%	0%	3%	-11%	-6%

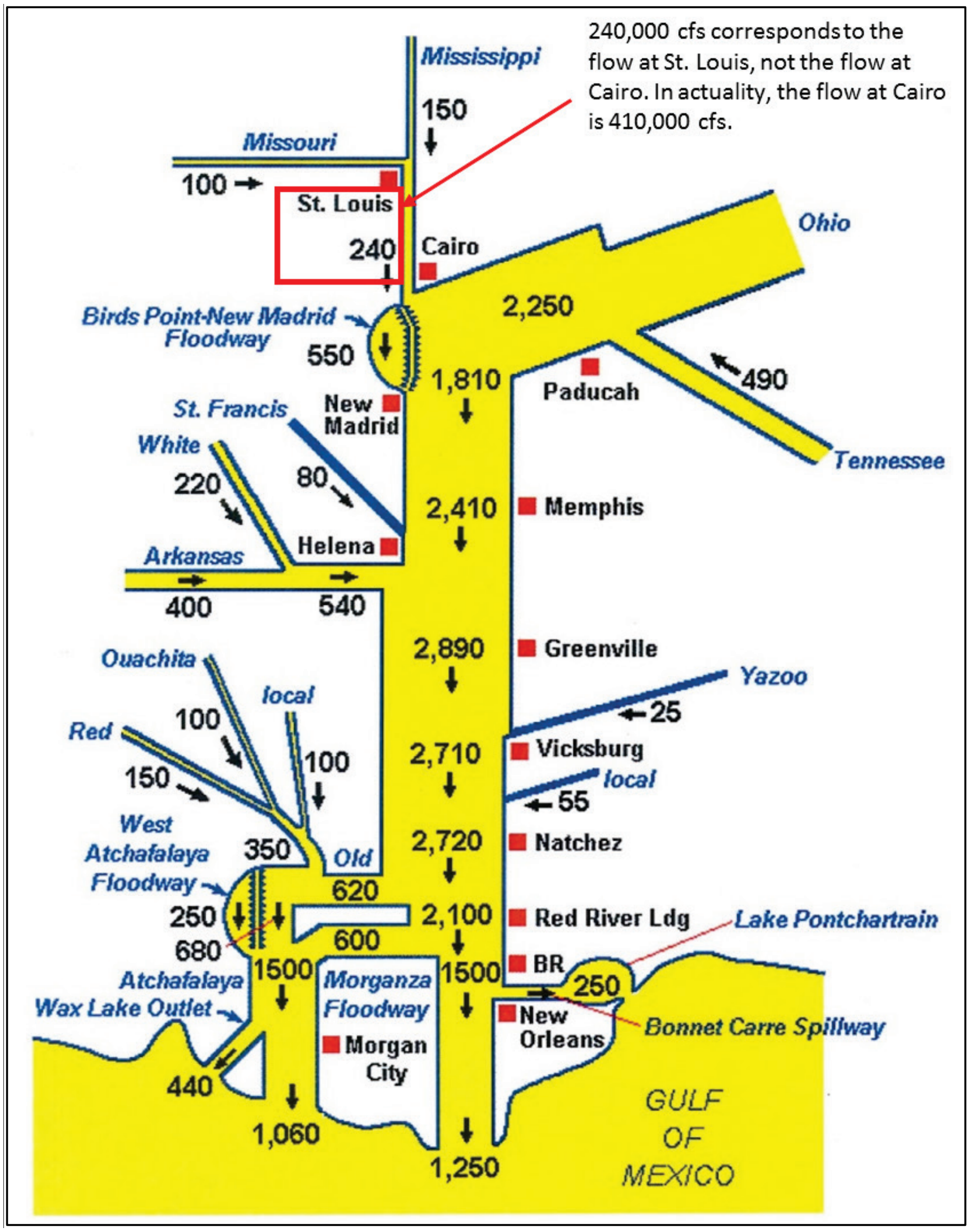
Figure 2-13. Regulation flow reductions for HYPO 58A, 52A, 56, and 63 in the 1955 and 2016 hydrology studies.



2.2.10 Alternative regulation effects

New hydrologic model results appear to show significant increases in peak flows for the Middle Mississippi River when compared to values given in the 1955 HYPO 58A-EN flow diagram as shown in Figure 2-14.

Figure 2-14. Flow diagram developed from 1955 study showing 240 kcfs flow from Middle Mississippi River at Cairo, IL.*



*"The correct flow at St. Louis should be 240 kcfs, and the Mississippi River flow just above the confluence should be 410 kcfs. This is discussed further in Appendix F of the Hydrology Report (USACE 2018a). Also, the Atchafalaya River outflows were adjusted in the 2010 Analysis to 581 kcfs through the Wax Lake Outlet and 919 kcfs through the Lower Atchafalaya River."

A review of the tabulated hydrographs used in the 1955 routing computations revealed that the flow given for the Middle Mississippi River (240,000 cfs) was the flow at St. Louis, MO, and did not include flow contributed by the local area between St. Louis and the Ohio River confluence. This local contributing area was designated sub-basin 7-Y in the 1955 study. When the St. Louis hydrograph was routed and combined with the local sub area, 7-Y, the peak flow was found to be 410,000 cfs.

For example, peak flows for HYPO 58A-U were 3% higher than 1955 HYPO 58A peak unregulated flows at the confluence of the Mississippi and Ohio Rivers while regulated peak flows for HYPO 58A-R were 17% higher at this location than were the 1955 HYPO 58A-EN peaks.

Factors identified as contributing causes of this difference were the following:

1. The total volume of computed unregulated flow utilizing the 2016 methodologies is greater than the 1955 Study, which may have limited the ability of reservoirs to reduce releases since they had to account for additional volumes of water.
2. Operation of reservoir projects to achieve maximum reductions at Cairo, IL, as for the HYPO 58A-EN results is not achievable during regular flood operations because this type of operation requires a priori knowledge of the entire event. It is not possible to know future storage requirements for a major flood like HYPO 58A beforehand in the event such a storm event occurs.
3. The HYPO 58A-EN results included reservoirs that were not constructed, which reduces the amount of storage available to reduce flood peaks routed downstream to Cairo, IL.

To demonstrate how reservoir operations effect downstream flows, Kentucky and Barkley Dam operations were analyzed as detailed in the following section.

2.2.10.1 Kentucky and Barkley dam operations

The 1955 study results were based on maximizing the use of storage in Groups E and N reservoirs to achieve the greatest reduction in peak flows at Cairo, IL. Kentucky and Barkley dams (located on tributaries to the Ohio River which have a significant impact on flow at Cairo) were included in the Groups E and N analysis. Storage utilization was managed to have the greatest reduction in flow at Cairo regardless of local impacts at each project. Specifically, locally induced flooding due to this regulation scheme was not considered in the decision process with respect to conditions on the main-stem Ohio River at Cairo, IL.

The present investigation utilized project operation manuals to determine regulation effects. Guide curves (or rule curves) from each project's Water Control Manual were used to develop regulated releases from each impoundment that had flood storage. There is a significant difference between operating a reservoir project solely for the maximum effect at a single downstream target location and following a set of guide curves developed to balance multiple objectives such as flood risk management, environmental sustainment, recreation, hydropower, and water supply.

The base regulated model simulation (HYPO 58A-R) for the 2016 investigation utilized standard reservoir operation protocols. This represented how both Kentucky and Barkley projects would be operated during a normal flood scenario. This approach did not include any extraordinary measures of joint regulation efforts by USACE, TVA, and NWS that could be employed during critical floods. Kentucky and Barkley projects were operated strictly by the Water Control Manual. Using an event that is known, such as a design event like the PDF, removes the uncertainty and results in decisions that are biased due to the known characteristics of the full event.

To estimate the difference between operating Kentucky and Barkley Dams using the *maximized effect* approach and what would be possible with current guide curve regulation during the HYPO 58A PDF event, several different combinations of project storage utilization, release rates, and pool elevations were back-calculated to determine a new release hydrograph. Five combinations were considered. The operating combinations were labeled Mod 1 through Mod 5 (Mod for Modified Regulation Operations as compared to the base HYPO 58A-R).

Regulated outflow hydrographs from Mod 1 through Mod 4 were used as inputs to the HEC-RAS unsteady model to assess computed hydrographs at Metropolis, IL, and to compare them with the unmodified HYPO 58A-R outputs (Figure 2-15). HYPO 58A was used here to demonstrate the effects of reservoir regulation. Table 2-8 provides tabulated peak flows and the associated reductions from the base regulated model output for HYPO 58A-R.

Figure 2-15. HYPO 58A hydrographs for Ohio River at Metropolis, IL: unregulated, regulated, and modified Kentucky and Barkley reservoir releases.

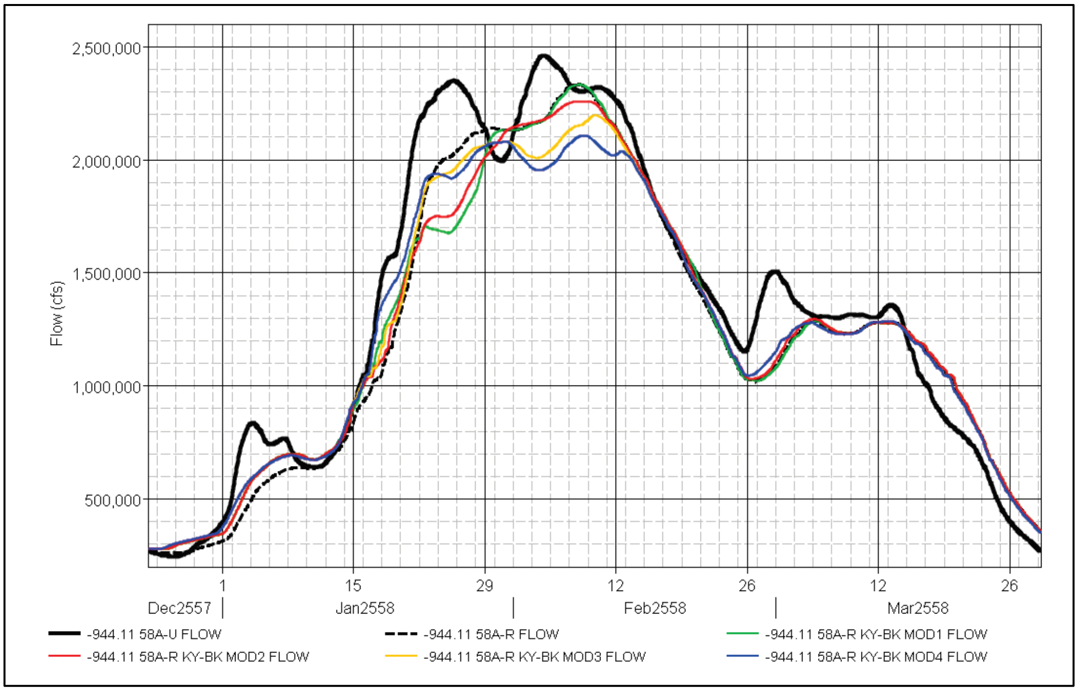


Table 2-8. Tabulated peak discharges for Ohio River at Metropolis, IL, with changes from the base regulated model result.

Run	Res Operation at Kentucky and Barkley Dams	Peak Flow at Metropolis, IL (cfs)	Date of Peak	Change from H58A-R (cfs)
H58A-U	Unregulated	2,461,083	4-Feb-58	-
H58A-R (Base)	Regulated	2,332,557	8-Feb-58	-
H58A-R M1	Mod-1	2,332,770	8-Feb-58	213
H58A-R M2	Mod-2	2,257,680	8-Feb-58	(74,877)
H58A-R M3	Mod-3	2,196,429	9-Feb-58	(136,128)
H58A-R M4	Mod-4	2,107,750	8-Feb-58	(224,806)

2.2.10.2 Description of modified reservoir operations

Mod 1 through Mod 4 regulation adjusted outflow while balancing storage and pool level using project guide curves. Mod 1 made minor changes in outflow, storage, and pool level. Each successive modification—Mod 2, Mod 3 and Mod 4—made more significant changes to alter the project outflows, each modification building on knowledge gained from the previous modification result. Mod 5 attempted to increase peak reductions in reservoir outflows further than achieved with Mod 4; however, no combination could be found to yield improvements without violating storage utilization and pool level guide curves.

2.2.10.3 Base regulation for 58A-R

The base regulation modeling applied basic guide curve constraints with no manual over-rides by the operator. This approach utilized an available HEC-RAS model for Kentucky and Barkley reservoirs. The HEC-RAS model is used by USACE in developing planned releases during flood events. The model configuration follows guide curves when operated in an unsupervised simulation. Note that USACE regulation of these projects during flood events seeks to pre-evacuate storage volume prior to each successive rise thereby making room to store flood waters later in the flood sequence. There are operational constraints on how much and how quickly drawdown can occur for real-time operations. For the modified reservoir calculations for the known HYPO 58A event, it was determined that drawdown to the allowable minimum pool could be accomplished in 1 or 2 days.

All modified regulation calculations took the base regulated model run and altered outflow to maximize storage in a way that maximized peak flow reductions on the Ohio River at Metropolis, IL. Routing calculations were made by simple routing computations based on the following equation:

$$\text{Inflow} - \text{Outflow} = \Delta\text{Storage/time}$$

The inflows were the same as used for the base regulated model run. The outflow from both Kentucky and Barkley Dams directly impacts the Ohio River discharge and became the variable that was adjusted. Storage calculated from the equation above was combined with project storage-elevation curves to determine changes in water level. Computed water levels and storage utilization were evaluated at each time period, 1 day in this analysis, for compliance with project guide curves.

2.2.10.4 Modified regulation 1

The first modification, Mod 1, made to Kentucky and Barkley releases attempted to release a greater quantity of water beginning in late December with more severe cutbacks during the crest. The increased releases were evaluated against storage utilization and pool guide curves at each time-step. If the release resulted in a condition that violated the guide curves, then an adjustment was made to remain within the guide curve constraints. The higher release rates were continued until the pool level dropped to the guide curve; after that point, discharges followed those in the base regulation. Regulated outflows from Mod 1 were used as input to the HEC-RAS model to obtain the flow hydrograph for the Ohio River at Metropolis, IL. Results from the Mod 1 computations are shown in Figure 2-16 and Figure 2-17.

Figure 2-16. Plot of Mod 1 Kentucky and Barkley Reservoir operations for HYPO 58A-R.

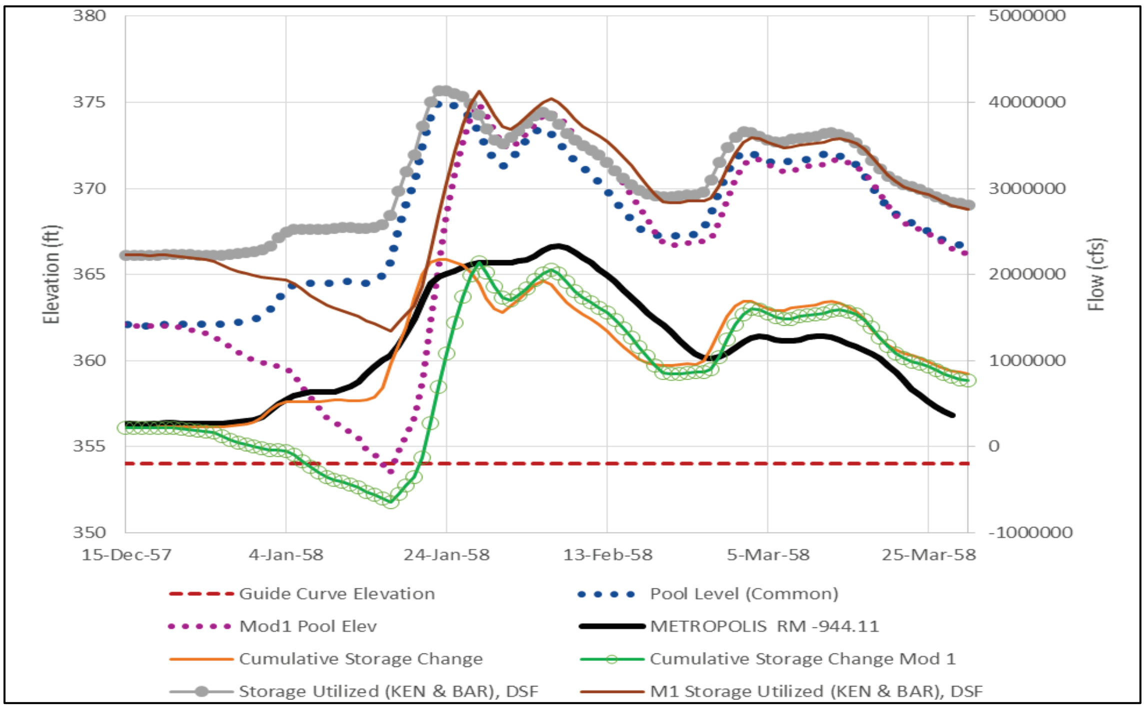
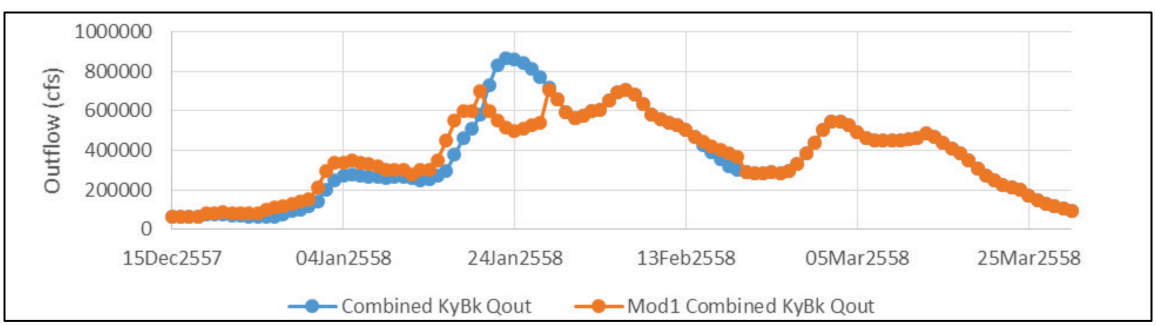


Figure 2-17. Mod 1 regulation outflows compared to the Base regulation outflows.



2.2.10.5 Modified regulation 2

The second modification, Mod 2, made to Kentucky and Barkley releases, attempted to release a greater quantity of water beginning in late December as done for Mod 1. However, combined releases from Kentucky and Barkley were held nearly constant at approximately 600,000 cfs through the peak period. The increased releases were evaluated against storage utilization and pool guide curves at each time-step. If the release resulted in a condition that violated the guide curves, then an adjustment was made to remain within the guide curve constraints. Regulated outflows from Mod 2 were used as input to the HEC-RAS model to obtain the flow hydrograph for the Ohio River at Metropolis, IL. Results from the Mod 2 computations are shown in Figure 2-18 and Figure 2-19.

Figure 2-18. Plot of Mod 2 Kentucky and Barkley reservoir operations for HYPO 58A-R.

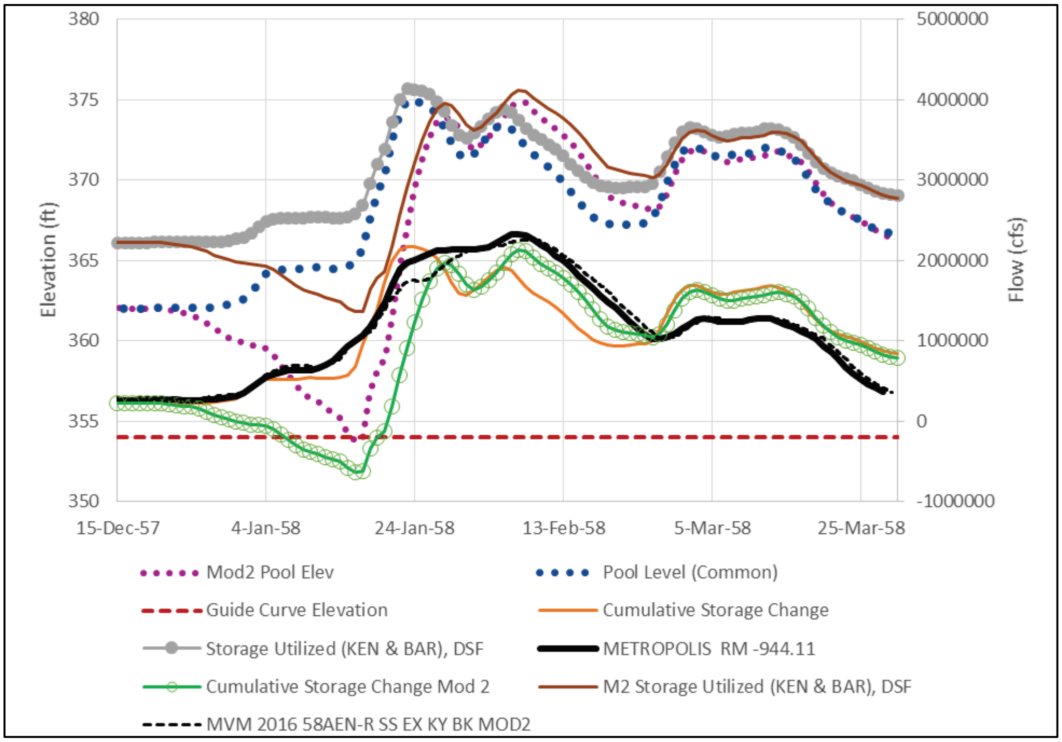
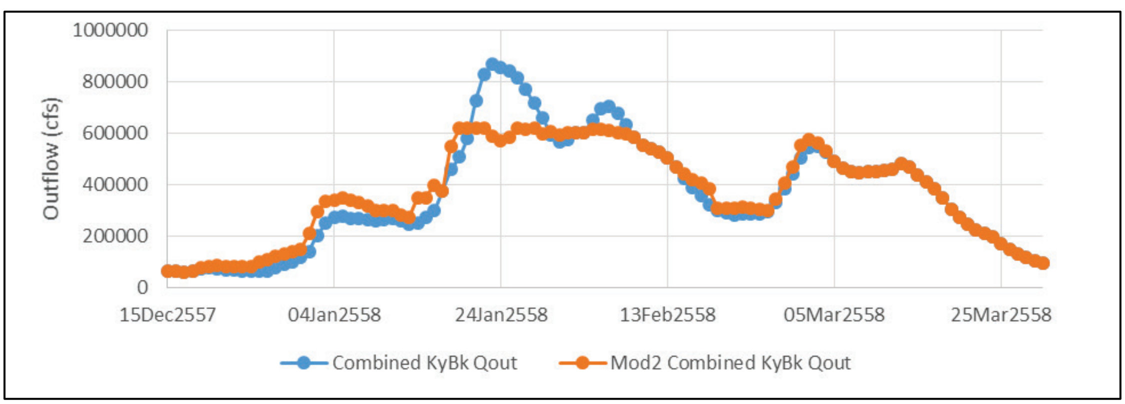


Figure 2-19. Mod 2 regulation outflows compared to the base regulation outflows.



2.2.10.6 Modified regulation 3

The third modification, Mod 3, made to Kentucky and Barkley releases, attempted to release a greater quantity of water beginning in late December as done for Mod 1. Results from the Mod 2 regulation run at Metropolis, IL, indicated that lowering the peak to a constant 600,000 cfs did not yield the desired effect on the Ohio River flows. As a result, Mod 3 regulation closely tracked the base regulated outflows between January 20 and January 22, slightly decreased outflows from the base January 23 to January 31, then had significantly lower outflows from February 1 through February 8. The altered releases were evaluated against storage utilization and pool guide curves at each time-step. If the release resulted in a condition that violated the guide curves, then an adjustment was made to remain within the guide curve constraints. Regulated outflows from Mod 3 were used as input to the HEC-RAS model to obtain the flow hydrograph for the Ohio River at Metropolis, IL. Results from the Mod 3 computations are shown in Figure 2-20 and Figure 2-21.

Figure 2-20. Plot of Mod 3 Kentucky and Barkley reservoir operations for HYPO 58A-R.

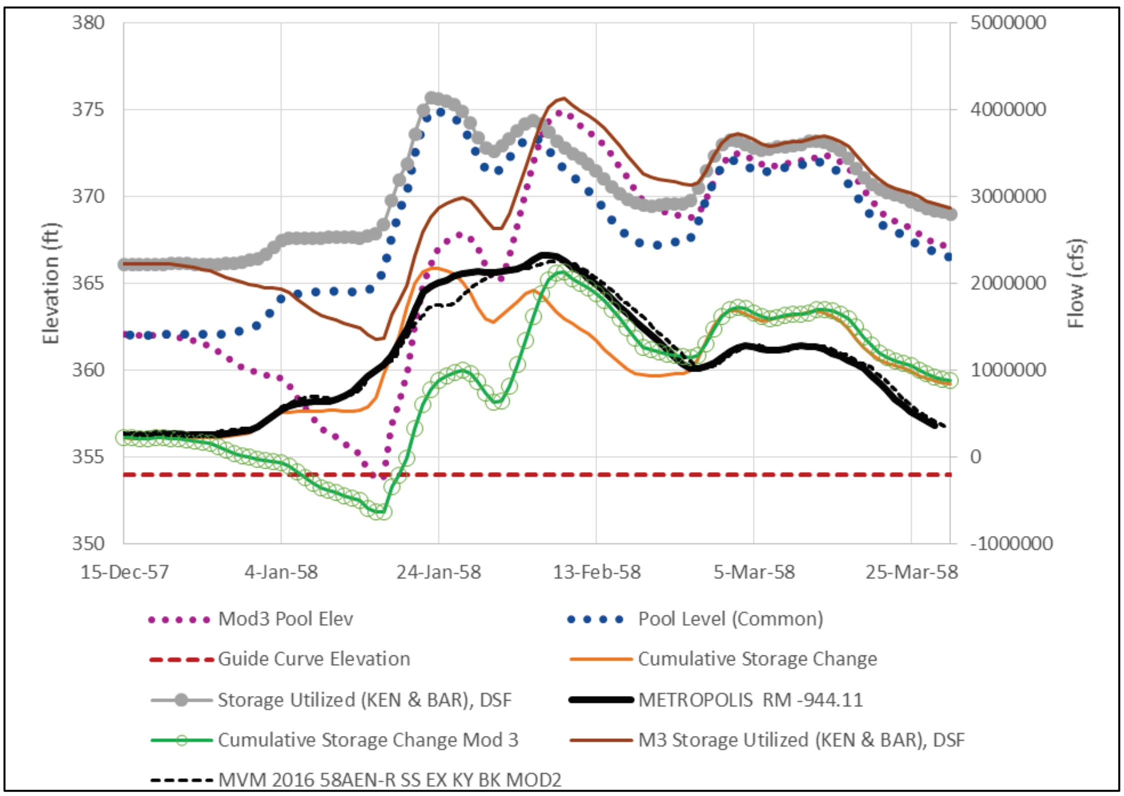
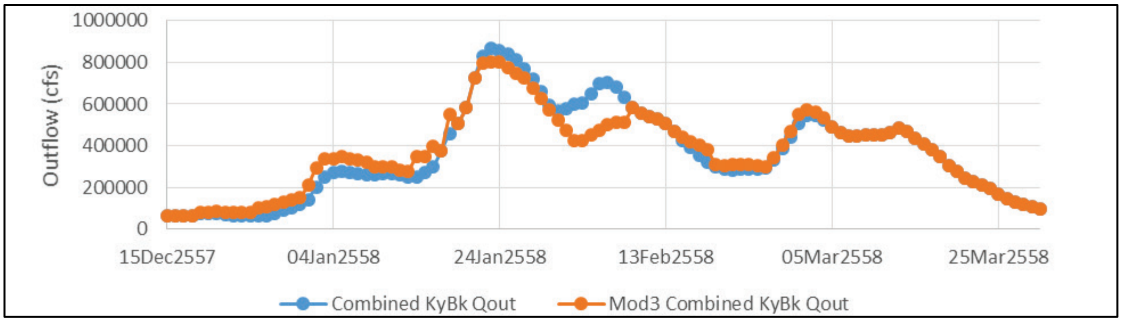


Figure 2-21. Mod 3 regulation outflows compared to the Base regulation outflows.



2.2.10.7 Modified regulation 4

The fourth modification, Mod 4, made to Kentucky and Barkley releases, attempted to build on results obtained from the Mod 3 regulation. The primary objective was to evacuate additional storage volume before January 21 that would increase the storage available that allowed for lower releases between January 30 and February 11. The altered releases were evaluated against storage utilization and pool guide curves at each time-step. If the release resulted in a condition that violated the guide curves, then an adjustment was made to remain within the guide curve constraints. Regulated outflows from Mod 4 were used as input to the HEC-RAS model to obtain the flow hydrograph for the Ohio River at Metropolis, IL. Results from the Mod 4 computations are shown in Figure 2-22 and Figure 2-23.

Figure 2-22. Plot of Mod 4 Kentucky and Barkley reservoir operations; HYPO 58A-R.

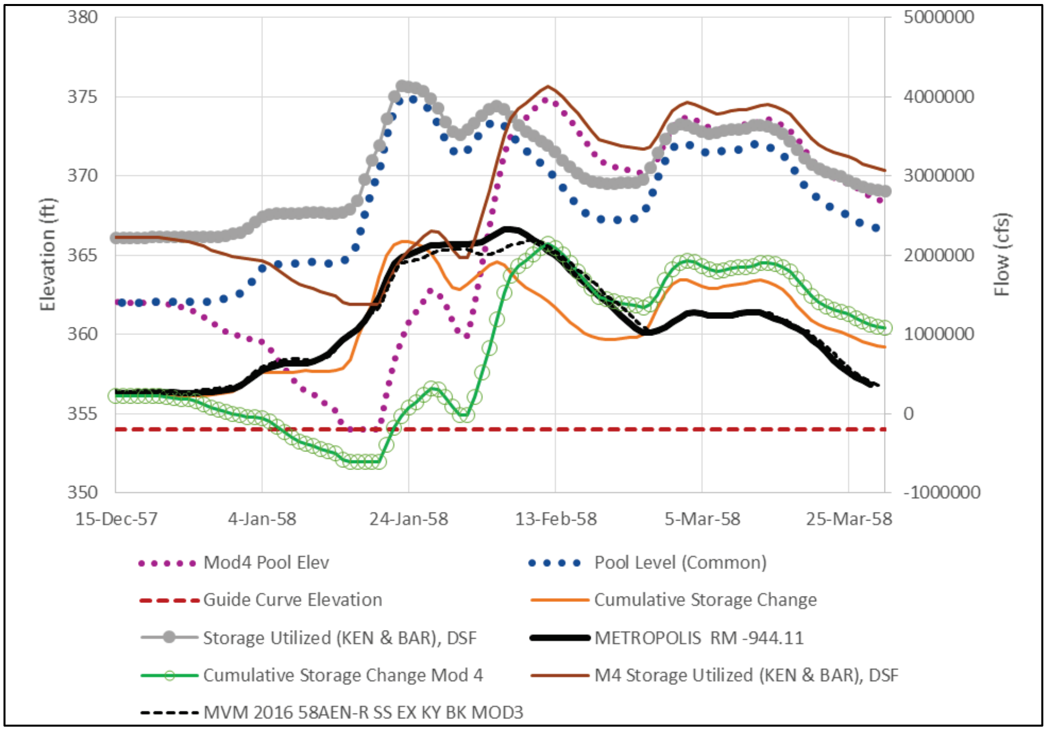
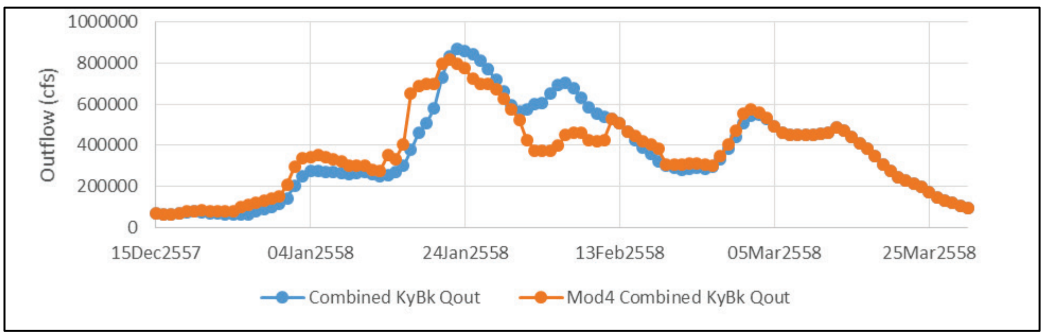


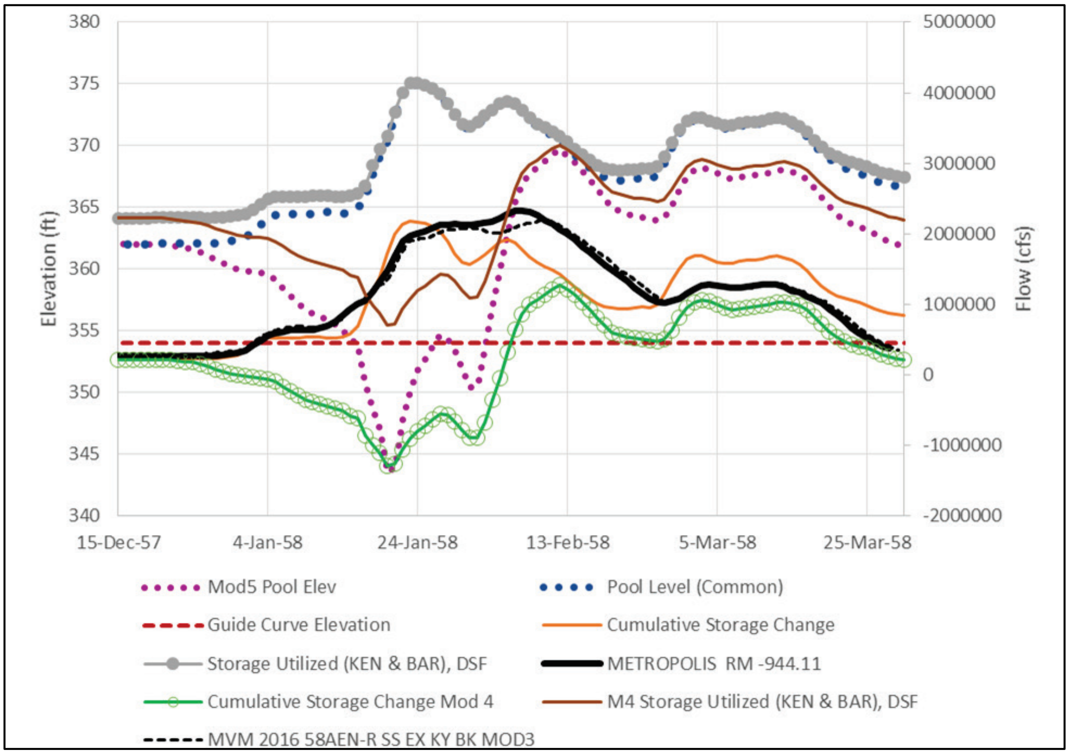
Figure 2-23. Mod 4 regulation outflows compared to the base regulation outflows.



2.2.10.8 Modified regulation 5

The fifth modification, Mod 5, made to Kentucky and Barkley releases, attempted to build on results obtained from the Mod 4 regulation. The primary objective was to evacuate significantly more storage volume before January 21 that would increase the storage available for use later. The altered releases were evaluated against storage utilization and pool guide curves at each time-step. There were no combinations that permitted early evacuation of storage beyond what was achieved by Mod 4 without drawing the pool level below the guide curve. The outflow developed for Mod 5 was not simulated in HEC-RAS to assess flows at Metropolis, IL, because there was no improved outcome from Mod 5 over results from Mod 4 without deviating from the guide curves. The interim calculations from the Mod 5 computations are shown in Figure 2-24 to illustrate how pool levels drop below the guide curve.

Figure 2-24. Plot of Mod 5 Kentucky and Barkley reservoir operations for HYPO 58A-R.



2.2.11 Synopsis of methodology differences, 2016 versus 1955

The combined CHPS and USACE hydrologic modeling was an extensive effort. There was good overall agreement between model results for the HYPO 58A results within the focus reach for the MR&T Project. The agreement for HYPO 52A, HYPO 56, and HYPO 63 varied.

HYPO 58A flows provided the design condition for the main-stem Mississippi River from Cairo, IL, to the Gulf with few exceptions as noted in House Document 308 (H.R. Doc. No. 308). The 2016 re-evaluation of hydrologic conditions resulted in minor changes in the unregulated flow hydrographs needed for developing the PDF water surface profile from HEC-RAS. The unregulated combined peak flow calculated for the Mississippi River at the Ohio/Mississippi confluence was within 3% of the previous peak values published in MRC (1955). Given the significant differences between the 1955 and 2016 methodologies, a 3% difference in peak unregulated flows instills confidence in how the 1955 input parameters and calculations were translated to the more refined current assessment process.

The other three HYPO storms analyzed, 52A, 56, and 63, provided a check on the methodology. The 2016 unregulated peak flows for HYPO 56 and HYPO 63 were in fair agreement with prior 1955 values, which provided further validation of the methodology. However, computed HYPO 52A unregulated peak values were significantly lower than those from 1955. Various elements of the analysis were checked in an effort to determine why this one model run produced lower flow. The precipitation inputs for HYPO 52A had greater variation from the 1955 precipitation depth-area curves indicating that the precipitation inputs were the primary source of difference. Multiple measures were employed to resolve the discrepancy in precipitation inputs without success. The analysis team concluded that the problem with HYPO 52A results arose because the precipitation inputs for the 2016 analysis were different than used in the 1955 analysis; it was not due to the methodology.

The following lists provide a summary of principal differences in the current modeling approach and that used for 1955 analysis.

Meteorology (climatic inputs)

- Same historical extreme events from 1937, 1950, and 1938
- Same HYPO 58A storm sequence: 1937 + 1950 + 1938
- Same storm scaling factors and transpositions
- Combined historic events into HYPO sequence using 2016
- Straight Sequence methodology (different from 1955 methodology replicated by the Clipped-Merge Sequence)

Hydrology (rainfall-runoff)

- 2016 analysis used NWS production forecast models
- 2016 model calibration was based on 2010 to 2014 data and daily operational adjustments
- 2016 model used 7,066 sub-basins versus 44 from 1955 study
- 2016 analysis included temperature effects (not part of 1955 study)
- 2016 model utilized continuous soil moisture and snow pack/snow melt accounting (not part of 1955 study)

- 2016 regulated simulations employed joint NWS/USACE/TVA modeling of reservoir releases
- 2016 regulation included reservoirs that existed as of 2014 (different than Groups E and N list used in 1955 study)
- 2016 regulation was based on standard operating guide curves (different than maximized reduction used in 1955 study)
- 2016 analysis included both unregulated and regulated runs completed for comparison with 1955 study
- 2016 routing included both hydrologic methods (upper watershed) and unsteady HEC-RAS model* (flowline analysis reach).

* The 1955 study did not consider unsteady hydrodynamic routing.

2.2.11.1 Climatic inputs

Meteorology components of the analysis followed the methods used in 1955 except for how the HYPO storm sequences were applied. A comparison was made between the current approach that used the Straight Sequence versus the 1955 approach that used different precipitation depths for portions of the watershed, as represented by the Clipped-Merged Sequence. There were minor differences, approximately +5%, between the Straight Sequence results and those obtained using the Clipped-Merged sequencing.

Some differences in precipitation inputs were inevitable because not all of the original point data from 1955 report archives could be obtained (Section 2.2.7). This required use of other sources of information including the NOAA national archive database. The sensitivity of results to precipitation inputs was not evaluated as this task fell outside of the study's scope and schedule. Future investigations should include an evaluation of model sensitivity to precipitation and temperature inputs.

2.2.11.2 Hydrologic modeling

The current investigation included significantly higher resolution of watershed characteristics than used in 1955 (7,066 versus 44 sub-basins). There were also improved calculations for rainfall/runoff through continuous soil moisture accounting, snow melt, and routing. The improvements in routing included higher resolution through more and shorter reaches and through unsteady hydrodynamic modeling.

Two noteworthy factors that affected model results were calibration period and reservoir regulation. Use of the NWS forecast models meant that calibration was based on modern data, 2010–2014, which may be considerably different than conditions from the 1950s. The impacts of calibration state were not determined because it was not possible to back-calibrate the models to a 1950s state within available timeframes.

Reservoir regulation effects in the current analysis were based on current Water Management Manuals while the 1955 analysis maximized utilization of available flood control storage. The 1955 study maximized use of total flood storage including prior evacuation of water to make storage available during the peak period which would require foreknowledge of the entire event. While desirable to leverage all available storage to reduce peak flows during a PDF level event, this objective is not achievable since the timing and spatial distribution of the event are not known beforehand. Operating reservoirs according to their standard operating plans was considered achievable and was the approach adopted for model simulations. The best representation of reservoir regulation effects lies between the maximized use of storage and use of daily guide curves and procedures. Section 2.2.9 gives details of reservoir regulation for the current analysis. It is recommended that additional model simulations be performed to develop several scenarios that could be used to guide operational decisions for extreme floods. These additional simulations would be similar to the assessment of the effects of upstream reservoirs on the Project Flood performed for the MRC in USACE (1957).

2.2.11.3 Paleoflood considerations

Although not part of the original PDF studies, paleoflood information has been introduced as a means for supplementing available gage data to assess flood probabilities. While paleoflood information can lead to potentially useful hydraulics and hydrologic information, this should be weighed against the underlying uncertainties and assumptions. Citing the Engineer Technical Letter ETL 1100-2-2 (USACE 2014), paleoflood analysis is not suitable for watersheds that have been altered through time, either by geologic or by anthropogenic processes. Paleoflood information is mostly useful when the channel and surrounding watershed have remained stable. Additionally, there is limited evidence to support the application of paleoflood information when using multiple hydrologic events or the use of flood volumes and durations.

Paleoflood analysis is most appropriate in arid to semi-arid regions. The Mississippi Valley is not within a semi-arid or arid region.

With 60 years of additional data since the 1955 study, more defendable peak flow frequency estimates could be derived. However, the available historical dataset is a mixed population of annual peaks created from events with various levels of regulation from upstream reservoirs over the period of record. Use of this dataset in a frequency analysis would require an incorrect assumption that the dataset is part of a naturally occurring population that fits an assumed statistical distribution. Discharge frequency estimates would only be valid with a complete record of unregulated events and a corresponding record of regulated flows that were based on a consistent regulation scheme over time, neither of which is available. An estimated frequency of the PDF peak discharge at given locations could be developed from statistical analysis using only the recorded peaks, but given the inherently inappropriate assumptions in such an analysis, no such estimates have been made as part of the current assessment.

2.2.12 Climate change considerations

Although climate change was not considered in the 1955 study, potential climate change impacts over the Mississippi River Basin were considered in this assessment, as any change in hydrologic input could have adverse impacts to the updated Flowline analysis. USACE (2016) provides guidance, regional documentation on climate change analyses, and a Climate Hydrology Assessment Tool (CHAT) to determine potential climate change trends. The tool analyzes flowrate trends across hydrologic unit code 4 (HUC-4) watersheds, which represent large river basins at the subregional level.

2.2.12.1 Regional documentation on climate change

The National Climate Assessment summarizes changing climate conditions in each of eight regions within the United States. The Lower Mississippi River flows through the Midwest Region and the Southeast and Caribbean Region. The key regional topics of interest in the Southeast and Caribbean Region are outlined below:

- Sea Level Rise (SLR) Threats – Sea level rise poses a widespread and continuing threats to both the natural and built environments and to the regional economy.

- Increasing Temperatures – Increasing temperatures and the associated increase in frequency, intensity, and duration of extreme heat events will affect public health, natural and built environments, energy, agriculture, and forestry.
- Water Availability – Decreased water availability, exacerbated by population growth and land-use change, will continue to increase competition for water and affect the region's economy and unique ecosystems.

The key regional topics of interest in the Midwest Region are outlined below:

- Impacts to Agriculture – In the next few decades, longer growing seasons and rising carbon dioxide levels will increase yields of some crops, though those benefits will be progressively offset by extreme weather events. Though adaptation options can reduce some of the detrimental effects, in the long term, the combined stresses associated with climate change are expected to decrease agricultural productivity.
- Forest Composition – The composition of the region's forests is expected to change as rising temperatures drive habitats for many tree species northward. The role of the region's forests as a net absorber of carbon is at risk from disruptions to forest ecosystems, in part due to climate change.
- Public Health Risks – Increased heat wave intensity and frequency, increased humidity, degraded air quality, and reduced water quality will increase public health risks.
- Fossil-Fuel Dependent Electricity System – The Midwest has a highly energy-intensive economy with per capita emissions of greenhouse gases more than 20% higher than the national average. The region also has a large and increasingly utilized potential to reduce emissions that cause climate change.
- Increased Rainfall and Flooding – Extreme rainfall events and flooding have increased during the last century, and these trends are expected to continue, causing erosion, declining water quality, and negative impacts on transportation, agriculture, human health, and infrastructure.
- Increased Risk to the Great Lakes – Climate change will exacerbate a range of risks to the Great Lakes, including changes in range and distribution of certain fish species, increased invasive species and

harmful blooms of algae, and declining beach health. Ice cover declines will lengthen the commercial navigation section.

The USACE Institute for Water Resources (IWR) summarizes the available climate change literature available for each of the smaller regions within the Mississippi River Basin (USACE 2015a–f). The report series uses available data to determine observed trends at various gages and to project trends. Summaries for each region are detailed in Table 2-9 and Table 2-10.

Table 2-9. Observed climate trends within the six major basins in the Mississippi River Basin as summarized in USACE (2015a-f).

Region	Observed Trend		
	Streamflow	Precipitation	Temperature
Ohio Region 05	Streamflow data were collected to study the trends in the Ohio Region. The general consensus for the Ohio Region is an increase in streamflow. Data from 1948–2004 showed an increase in annual runoff between .2 and 1.8 millimeters (mm) per year.	A clear consensus is lacking about the precipitation in the Ohio Region. Multiple studies conclude an increase in precipitation while others show a decrease or no change at all. Grundstein (2009) has found increasing trends in soil moisture for several climate stations in the Ohio region.	The Ohio Region has been recognized as a transition zone due to a warming trend toward the north and a cooling trend toward the south. Research concludes that there has been a linearly increasing trend between 0°C and 0.4°C per century.
Tennessee Region 06	No trend has been observed in the streamflow data for the region.	Annual precipitation totals have become more variable in recent years compared to earlier in the twentieth century. Evidence has also been presented, but with limited consensus, of mildly increasing trends in the magnitude of annual and seasonal precipitation for parts of the study region.	Evidence has been presented in the recent literature of mild increases in annual temperature in the Tennessee Region over the past century, particularly since the 1970s. Consensus, and the number of available region specific studies, is low, however.
Upper Mississippi Region 07	A mild upward trend in mean streamflow in the study region has been identified by multiple authors, but a clear consensus is lacking.	A mild upward trend in precipitation in the study region has been identified by multiple authors.	Increasing trends in observed air temperature in the study basin, including daily mean and minimum temperatures, were reported by multiple authors.

Region	Observed Trend		
	Streamflow	Precipitation	Temperature
Lower Mississippi River Region 08	A mild upward trend in mean streamflow in the study region has been identified by multiple authors, but a clear consensus is lacking.	A mild upward trend in precipitation in the study region has been identified by multiple authors, but a clear consensus is lacking.	No significant trend in observed mean air temperature in the study region, though extreme minimum daily air temperatures may be increasing.
Missouri River Region 10	A mild upward trend in mean streamflow in the Missouri River Region has been identified by multiple authors, but a clear consensus is lacking in the upper portion of the region.	A mild upward trend in annual and extreme precipitation in the lower portion of the Missouri River Region has been identified by multiple authors while the upper portion has been identified to have a decreasing trend for annual and extreme precipitation.	An increasing trend in observed mean and daily minimum air temperature in the study region was observed; however, a trend in daily maximum air temperature is lacking.
Arkansas, White and Red Rivers Region 11	Streamflow data were collected to study the trends in Region 11. Water runoff increased 140 mm for the lower portion and increased 20 mm for the upper portion of the region in a study done by Qian et al. (2007). A general consensus amongst recent peer reviewed literature indicates an upward trending for average streamflow.	A general consensus amongst recent peer-reviewed literature indicates a mild upward trending for average precipitation and extreme precipitation events.	Studies were conducted on mean temperatures and extreme temperatures in Region 11. Slight warming trends occurred during the winter and spring months while mild cooling trends occurred for the summer and fall months for mean temperatures. One-day extreme minimum temperatures have begun to increase since 1995.

Table 2-10. Projected climate trends within the six major basins in the Mississippi River Basin as summarized in USACE (2015a-f).

Region	Projected Trends		
	Streamflow	Precipitation	Temperature
Ohio Region 05	Projected changes in streamflow in the Ohio Region vary significantly.	Although precipitation is projected to increase in most studies surveyed, there are no clear trends in the literature indicating the magnitude or geographic distribution of future changes to average or extreme precipitation.	Although there is a strong consensus that temperatures will increase, the amount of projected increase varies between studies. Several studies also show considerable variation within the Ohio Region.
Tennessee Region 06	Variability exists with projected streamflow changes in the Tennessee Region.	Strong consensus exists in the literature that the intensity and frequency of extreme storm events will increase in the future for the Tennessee Region. Low consensus exists with respect to projected changes in total annual precipitation for the region.	Strong consensus exists in the literature that projected temperatures in the study region show a sharp increasing trend over the next century.
Upper Mississippi Region 07	Clear consensus in the literature is lacking, with some studies projecting an increase in future streamflow (as a result of increased precipitation) in the study region, while others project a decrease in flows (a result of increased evapotranspiration). Seasonally, multiple studies suggest increased flows in the winter and spring and decreased flows in the summer.	General consensus exists in the literature with respect to projected increasing trends in future annual and extreme precipitation in the study basin.	Strong consensus exists in the literature that projected temperatures in the study region will rise over the next century.
Lower Mississippi River Region 08	Although consensus is lacking, a small number of reviewed studies indicates a mild decreasing trend in streamflow for the study	Little consensus exists in the literature with respect to projected trends in future precipitation in the study region.	Strong consensus exists in the literature that projected temperature in the study region show a sharp increasing trend over the next century.

Region	Projected Trends		
	Streamflow	Precipitation	Temperature
	region through the next century.		
Missouri River Region 10	Consensus amongst recent literature is lacking regarding the direction of projected trends in streamflow and related variable such as runoff and water yield. The trend direction seems to be dependent on selection of General Circulation Model (GCM), emissions scenario, and hydrologic model.	A general consensus exists in the literature with respect to an increasing trend in future precipitation in the study region.	Strong consensus exists in the literature that projected temperature trends in the study region show a steady increase over the next century.
Arkansas, White and Red Rivers Region 11	There is limited consensus that projected streamflow will decrease for portions of Water Resources Region 11; however, projected trends are highly dependent on GCM selection.	A general consensus amongst recent peer-reviewed literature indicates no change in projected annual precipitation levels within the Water Resources Regions 11. However, there is consensus that the projected occurrence of extreme precipitation events will increase as well as the number of consecutive dry days.	A large consensus amongst recent peer-reviewed literature indicates a moderate upward trending for projected mean temperature and a significant upward trend for maximum temperature within the Water Resources Region 11.

The results above do not pinpoint critical sub-basins within each region that are most impacted and those sub-basins that may have a greater effect on the MR&T Flowline.

2.2.12.2 Evaluation of discharge

The present tools available from USACE-IWR evaluate available and projected stream discharge data to evaluate potential changes to river discharge due to climate change or other changes. Two tools are presently available for this purpose: the Nonstationarity Detection Tool (NDT) evaluates observed streamflow data for trends and change-points while the CHAT assesses modeled future streamflows based on downscaled general circulation model outputs and scenarios of future climate. Changes in streamflow and temperature that are projected by the general circulation models are manifested as changes in streamflow through simplified hydrologic and routing models.

2.2.12.3 Nonstationarity Detection Tool (NDT) and Climate Hydrology Assessment Tool (CHAT)

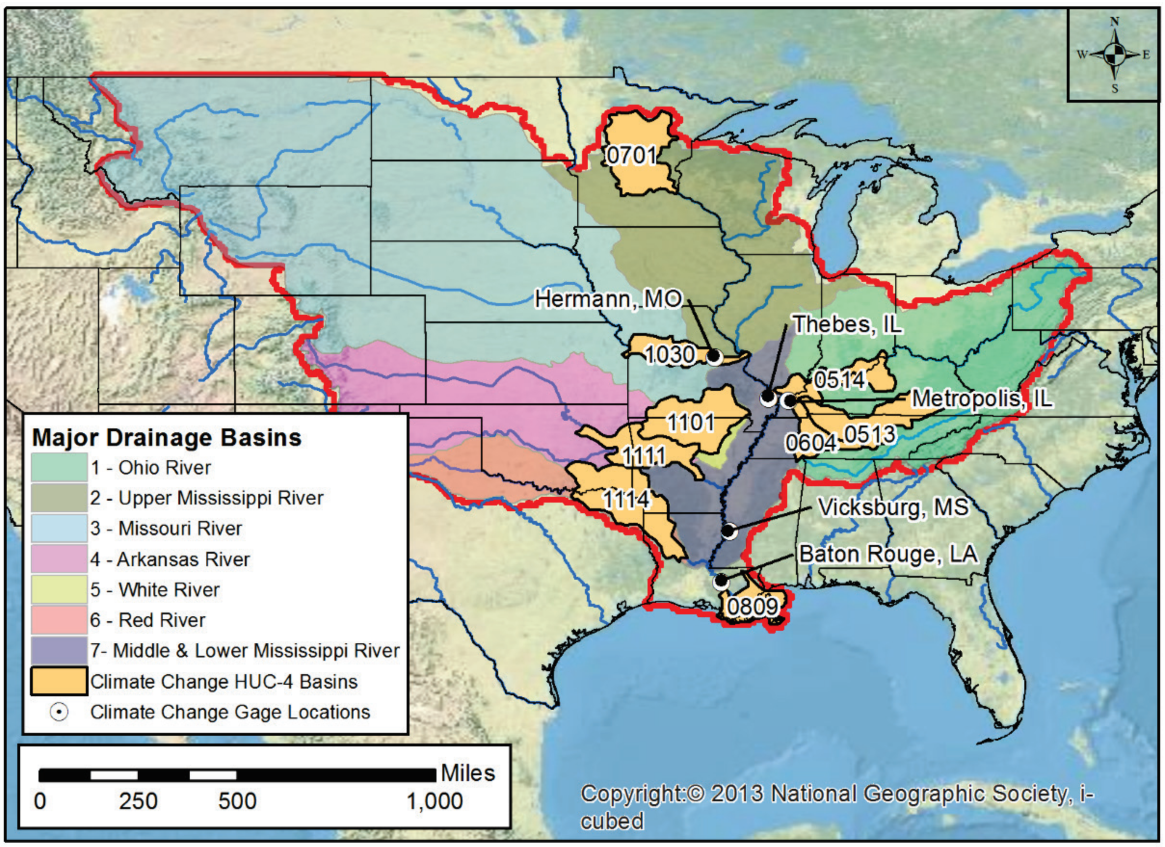
The USACE NDT detects nonstationarities in annual instantaneous peak streamflow data in flow records of 30 years or more for one gage location (Friedman et al. 2016). A change point in the data record is defined by two or more periods within the record that are described by different statistical distributions, mean, and/or variance. Change points can be used to identify those periods of record that violate the stationarity assumption that is implicit in traditional statistical methods in hydrology (i.e., the assumption that the mean and standard deviation do not change over time). Nonstationary is useful in water management planning on a local or regional level when the cause of the nonstationarity is understood and there is a way to describe the shift (Hirsch 2011). Otherwise, the detected change point could lead to erroneous conclusions with little meaning. USACE guidance prescribes running a nonstationarity analysis; however, the scale of this assessment, which spans over 40% of the contiguous United States and two Canadian provinces, is such that a nonstationarity analysis would have little meaning over the entire watershed. On a localized scale, locations at key points with major sub-basins can be analyzed for nonstationarity to obtain a general signal for the watershed.

CHAT is able to analyze the observed streamflow trend at a single gage location by running a simple linear regression. In the future, the user will be able to select multiple gage locations to analyze the observed streamflow trend for the entire HUC-4. At its present state, the current observed trend at a single location within a HUC-4 for this assessment would have little meaning. CHAT is able to generate future climate change trends based on available historic observed data as it incorporates the Representative Concentration Pathways 4.5 (RCP4.5) emissions model and the ACCESS1.0 climate model generated by the Commonwealth Scientific and Industrial Research Organisation's Bureau of Meteorology[1]. The CHAT database includes all available USGS gage flowrate data on record. These flowrate

[1] The RCP4.5 scenario has a mean global surface temperature increase of 1.4 °C between 2046 to 2065 and an increase of 1.8 °C between 2081 to 2100 relative to the reference period of 1986–2005. ACCESS1.0 uses the UK MetOffice UM atmosphere model, the GFDL MOM4p1 ocean model, the LANL CICE4.1 sea-ice model, and the MOSES2 land surface model to simulate the global climate between 1850 to 2006 using historical forcings.

observations then feed into 93 different hydrologic climate models for a period of 2000–2099. Since the Mississippi River Basin encompasses a large number of HUC-4 basins, only HUC-4 basins that represent main areas of concern (those areas along main tributaries to the Mississippi River and directly impacted by HYPO 58A), were selected for the Flowline assessment. Since NDT is primarily useful on a localized scale, consideration was taken to evaluate the appropriate USGS gage locations on tributaries just above the confluence to the Mississippi River and several locations along the Lower Mississippi River. Figure 2-25 displays the HUC-4 basins and gage locations selected for the climate change analysis.

Figure 2-25. HUC-4 drainage basins and gage locations selected for climate change analysis.



Streamflow data prior to 1933 were excluded from the analysis as discharge measurements prior to 1933 were proven to be unrealiable and inconsistent as advances in streamflow measurement tools were becoming standardized (Watson et al. 2013). Significant nonstationarity is defined as one in which a change point has statistical consensus within a 5-year period or a known event (e.g., reservoir construction/completion) caused nonstationarity. Table 2-11 shows the results of the nonstationarity detection analysis and the corresponding trend analysis.

While the analysis is not robust enough to understand stationarity in the entire Mississippi River Basin, the NDT analysis does provide a snapshot of the trends (or lack of trends) along the main reach of concern for the MR&T PDF. There were no significant nonstationarities or significant trends between 1933 to 2014 detected at any of the USGS gage locations examined.

Table 2-11. Stationarity analysis on selected gage locations that impact the Lower Mississipi River.

USGS Gage	Nonstationarity Detected (Y/N)[1]	Significant Trend Detected (Y/N)
3611500 - Ohio River at Metropolis, IL	N	N
6934500 - Missouri River at Hermann, MO	N	N
7022000 - Mississippi River at Thebes, IL	N	N
7289000 - Mississippi River at Vicksburg, MS	N	N
73740002 - Mississippi River at Baton Rouge, LA	N	N

[1] Significant nonstationarity is defined as a change point where statistical consensus is reached within a 5-year period or a known event caused nonstationarity.
[2] The results from this gage location may be affected by the significant amount of missing data.

With numerous climate change models to choose from, CHAT streamlines the selection process and allows USACE to consider climate change in a reproducable and comparable manner for all domestic projects by allowing climate change trends to be analyzed with one model that produces the projected annual maximum monthly flow trend line for years 2000–2099 and the climate-modeled annual maximum monthly flow range from 93 different climate models as shown in the Hydrology Report (USACE 2018a). Each graph includes statistical significance (p) as shown in Table 2-12.

Table 2-12. HUC-4 basins from USACE CHAT with calculated significance, p.

HUC—Basin Name	Projected Annual Maximum Monthly Flow, p (2000–2099)
1111—Lower Arkansas River	0.04 -
1101—Upper White River	0.44 +
0701—Mississippi River Headwaters	0.16 +
0809—Lower Mississippi River	0.76 +
1030—Lower Missouri River	<0.0001 +
0514—Lower Ohio River Basin	<0.0001 +

HUC—Basin Name	Projected Annual Maximum Monthly Flow, *p* (2000–2099)
0604—Lower Tennessee River	0.07 +
0513—Cumberland River	0.002 +
1114—Red Sulfur	0.003 -

– = downward trend
\+ = upward trend
Climate Change Web Analysis Tool accessed on 05/02/2017 with the Demo Tool v.1.0.

The data show that the projected trends in the Lower Missouri River, Lower Ohio River, and Cumberland River Basin have a significant positive trend ($p < 0.05$) in discharge for increasing maximum monthly flows over the next century while the Red Sulfur and Lower Arkansas River show a significant decreasing trend. The Upper White River, Mississippi River Headwaters, Lower Tennessee River, and Lower Mississippi River do not have a statistically significant trend for maximum monthly flows over the next century. The goodness-of-fit for all of the analyses is low indicating that little of the variability in flow can be explained by change over time.

2.2.12.4 Watershed Vulnerability Assessment (VA) tool

The USACE VA tool evaluates climate change vulnerabilities at the HUC-4 level by using a weighted order weighted average that assigns vulnerability scores based on numerous indicators. These indicators assess eight business line vulnerabilities critical to the USACE mission for two scenarios (dry and wet) over two epochs (2050 and 2085). This tool was used to determine which hydrologic unit codes (HUCs) in the MVD have business line vulnerabilities for both the wet and dry 2085 conditions.

Each of the eight business lines—flood risk reduction, navigation, ecosystem restoration, emergency management, hydropower, recreation, regulatory, and water supply—was evaluated. Flood risk reduction, navigation, and emergency management are of particular interest for this assessment.

The VA tool identifies flood risk vulnerabilities determined by the tool based on the following indicators:

- Acres of urban area within the 500-year floodplain
- Coefficient of variation of cumulative annual flow
- Streamflow elasticity, or ratio of streamflow response to precipitation

- Flood magnification: ratio of 10% exceedance flow in the future to the 10% exceedance flow in the base flow period, for cumulative monthly flows
- Flood magnification: ratio of 10% exceedance flow in the future to the 10% exceedance flow in the base flow period, for local monthly flows.

The VA tool identifies navigation risk vulnerabilities determined by the tool based on the following indicators:

- The ratio of the change in the sediment load in the future to the present load
- Land area that is urban or suburban as a percentage of the total U.S. land area
- Measure of the short-term variability in the region's hydrology: 75th percentile of annual ratios of the standard deviation of monthly runoff to the mean of monthly runoff. Includes upstream freshwater inputs (cumulative)
- Median of deviation of runoff from monthly mean times average monthly runoff to the mean of monthly runoff; excludes upstream freshwater inputs (local).
- Acres of urban area within the 500-year floodplain
- Flood magnification: ratio of 10% exceedance flow in the future to the 10% exceedance flow in the base flow period, for cumulative monthly flows
- Low runoff: monthly runoff that is exceeded 90% of the time, including upstream freshwater (cumulative and local)
- Change in low runoff: ratio of indicator exceeded 90% of the time
- Greatest precipitation deficit: The most negative value calculated by subtracting potential evapotranspiration from precipitation over any 1-, 3-, 6-, or 12-month period.

The VA tool identifies emergency management risk vulnerabilities determined by the tool based on the following indicators:

- Population within the 500-year floodplain
- Ratio of the standard deviation of annual runoff to the annual runoff mean; includes upstream freshwater (cumulative)
- Median of deviation of runoff from monthly mean times average monthly runoff divided by deviation of precipitation from monthly mean times average monthly precipitation

- Number of people living below the poverty line
- Percent of people who are disabled
- Experience with declared disasters in the past
- Number of communities enrolled in the National Flood Insurance Program
- Flood magnification: ratio of 10% exceedance flow in the future to the 10% exceedance flow in the base flow period, for cumulative monthly flows
- Change in low runoff: ratio of indicator exceeded 90% of the time.

Indicators for the other business lines are available for review in the tool. Figure 2-26 displays each of the eight USACE business lines for the wet scenario 2085 epoch. There are two HUCs vulnerable for flood risk, six HUCs vulnerable for navigation, and one HUC vulnerable for emergency management.

Figure 2-26. MVD business line vulnerability assessment for the wet scenario 2085 epoch.

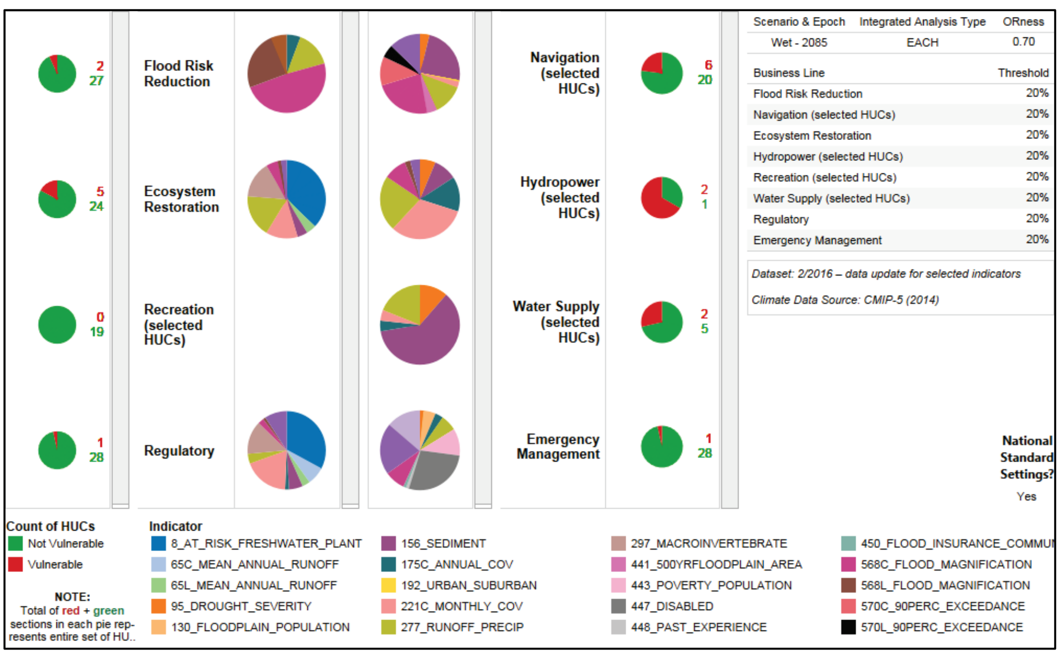
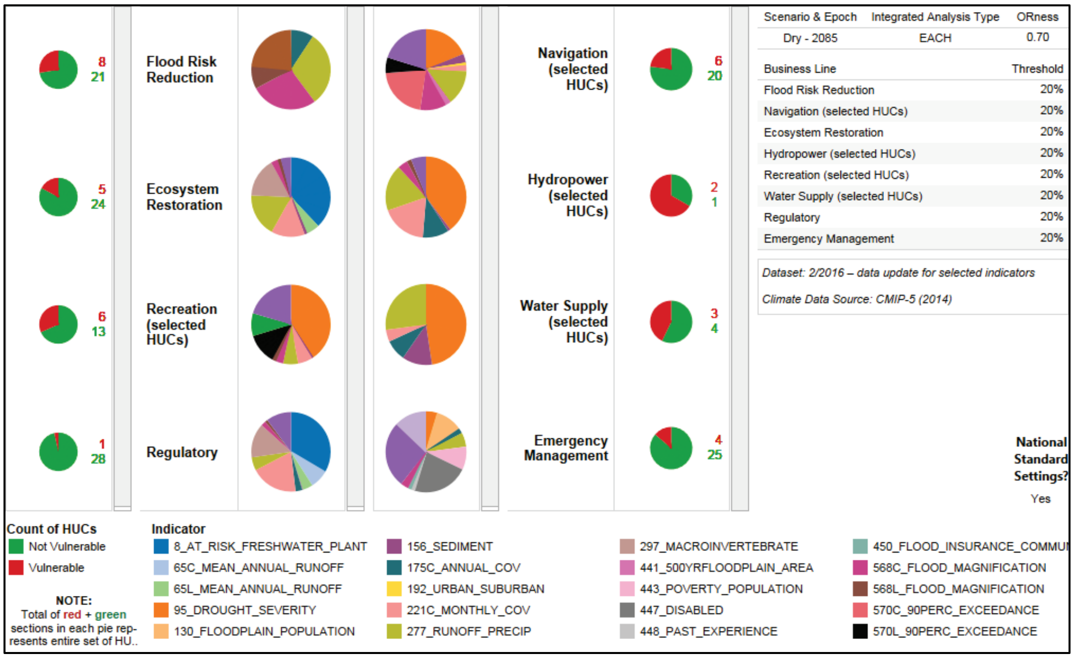


Figure 2-27 displays each of the eight USACE business lines for the dry scenario 2085 epoch. There are eight HUCs vulnerable for flood risk, six HUCs vulnerable for navigation, and four HUC vulnerable for emergency management.

Figure 2-27. MVD business line vulnerability assessment for the dry scenario 2085 epoch.



The USACE-IWR summarizes the available literature to summarize business line vulnerabilities in each region within the Mississippi River Basin (USACE 2015a–f). Summaries for each region are detailed in Figure 2-28 through Figure 2-33.

Figure 2-28. Summary of projected climate trends and impacts on USACE business lines for Region 05 – Ohio (USACE 2015a).

CLIMATE VARIABLE	VULNERABILITY
Increased Ambient Temperatures	Increased ambient air temperatures throughout the century, and over the next century are expected to create the following vulnerabilities on the business lines in the region: • Loss of vegetation from increased periods of drought and reduced streamflows may have impacts on vegetation within the region, which is important for sediment stabilization in the watershed. Loss of non-drought resistant vegetation may result in an increase in sediment loading, potentially causing geomorphic changes in the tributaries to the river system. • Decrease in flows may result from periods of drought and reduced streamflow has implications for maintain water levels in the rivers. **BUSINESS LINES IMPACTED:**
Increased Maximum Temperatures	Air temperatures are expected to increase 2-6°C in the latter half of the century with the number of heat wave days increasing as much as 75 days. This is expected to create the following vulnerabilities on business lines in the region: • Increased water temperatures leading to water quality concerns, particularly for the dissolved oxygen (DO) levels, growth of nuisance algal blooms and influence wildlife and supporting food supplies. • Increased evapotranspiration. • Human health risk increases from extended heat waves, impacting recreational visitors and increasing the need for emergency management. **BUSINESS LINES IMPACTED:**
Increased Annual Precipitation	By the middle of the century, annual precipitation is expected to increase in the region which are expected to influence the following vulnerabilities on business lines in the region: • Increased flows and runoff, which may carry pollutants to receiving water bodies, decreasing water quality. • Increased erosion with subsequent changes in sediment accumulation rates and creating water quality concerns. • Increased flooding, which may have negative consequences for all infrastructure, habitats, and people in the area. **BUSINESS LINES IMPACTED:**
Streamflow Variability	Streamflow will have more extreme variability, which greatly depends on where the area of the region. This may result in: • Increased flows and runoff, which may carry pollutants to receiving water bodies, decreasing water quality. • Increased erosion with subsequent changes in sediment accumulation rates and creating water quality concerns. • Increased flooding, which may have negative consequences for all infrastructure, habitats, and people in the area. • Loss of vegetation from increased periods of drought and reduced streamflows may have impacts on vegetation within the region, which is important for sediment stabilization in the watershed. Loss of non-drought resistant vegetation may result in an increase in sediment loading, potentially causing geomorphic changes in the tributaries to the river system. • Decrease in flows may result from periods of drought and reduced streamflow has implications for maintain water levels in the rivers. **BUSINESS LINES IMPACTED:**

NOTE: The Regulatory and Military Program business lines may be impacted by all climate variables

= Navigation = Flood Risk Management = Ecosystem Restoration = Hydropower = Recreation = Water Supply = Emergency Management

Figure 2-29. Summary of projected climate trends and impacts on USACE business lines for Region 06 – Tennessee (USACE 2015b).

CLIMATE VARIABLE	VULNERABILITY
Increased Ambient Temperatures	Increased ambient air temperatures throughout the century, and over the next century are expected to create the following vulnerabilities on the business lines in the region: • Loss of vegetation from increased periods of drought and reduced streamflows may have impacts on vegetation within the region, which is important for sediment stabilization in the watershed. Loss of non-drought resistant vegetation may result in an increase in sediment loading, potentially causing geomorphic changes in the tributaries to the river system. • Decrease in flows may result from periods of drought and reduced streamflow has implications for maintain water levels in the rivers. • Flora and fauna that are not drought resistant can also be impacted by longer drought conditions, which may reduce opportunities for recreational wildlife viewing. **BUSINESS LINES IMPACTED:**
Increased Maximum Temperatures	Air temperatures are expected to increase, including maximum high and low temperatures in each season. This is expected to create the following vulnerabilities on business lines in the region: • Increased water temperatures leading to water quality concerns, particularly for the dissolved oxygen (DO) levels, growth of nuisance algal blooms and influence wildlife and supporting food supplies. • Increased evapotranspiration. • Human health risk increases from extended heat waves, impacting recreational visitors and increasing the need for emergency management. **BUSINESS LINES IMPACTED:**
Increased Storm Intensity and Frequency	Extreme storm events may become more intense and frequent over the coming century which are expected to influence the following vulnerabilities on business lines in the region: • Increased flows and runoff, which may carry pollutants to receiving water bodies, decreasing water quality. • Increased erosion with subsequent changes in sediment accumulation rates and creating water quality concerns. • Change in engineering design standards to accommodate new extreme storms magnitudes. • Increased groundwater recharge rates, as residence times are shortened within areas where evapotranspiration takes place during high intensity events. • Increased flooding, which may have negative consequences for all infrastructure, habitats, and people in the area. **BUSINESS LINES IMPACTED:**
Streamflow Variability	The changes in streamflow may be positive or negative, depending on local conditions. **BUSINESS LINES IMPACTED:**

NOTE: The Regulatory and Military Program business lines may be impacted by all climate variables

= Navigation = Flood Risk Management = Ecosystem Restoration = Hydropower = Recreation = Water Supply = Emergency Management

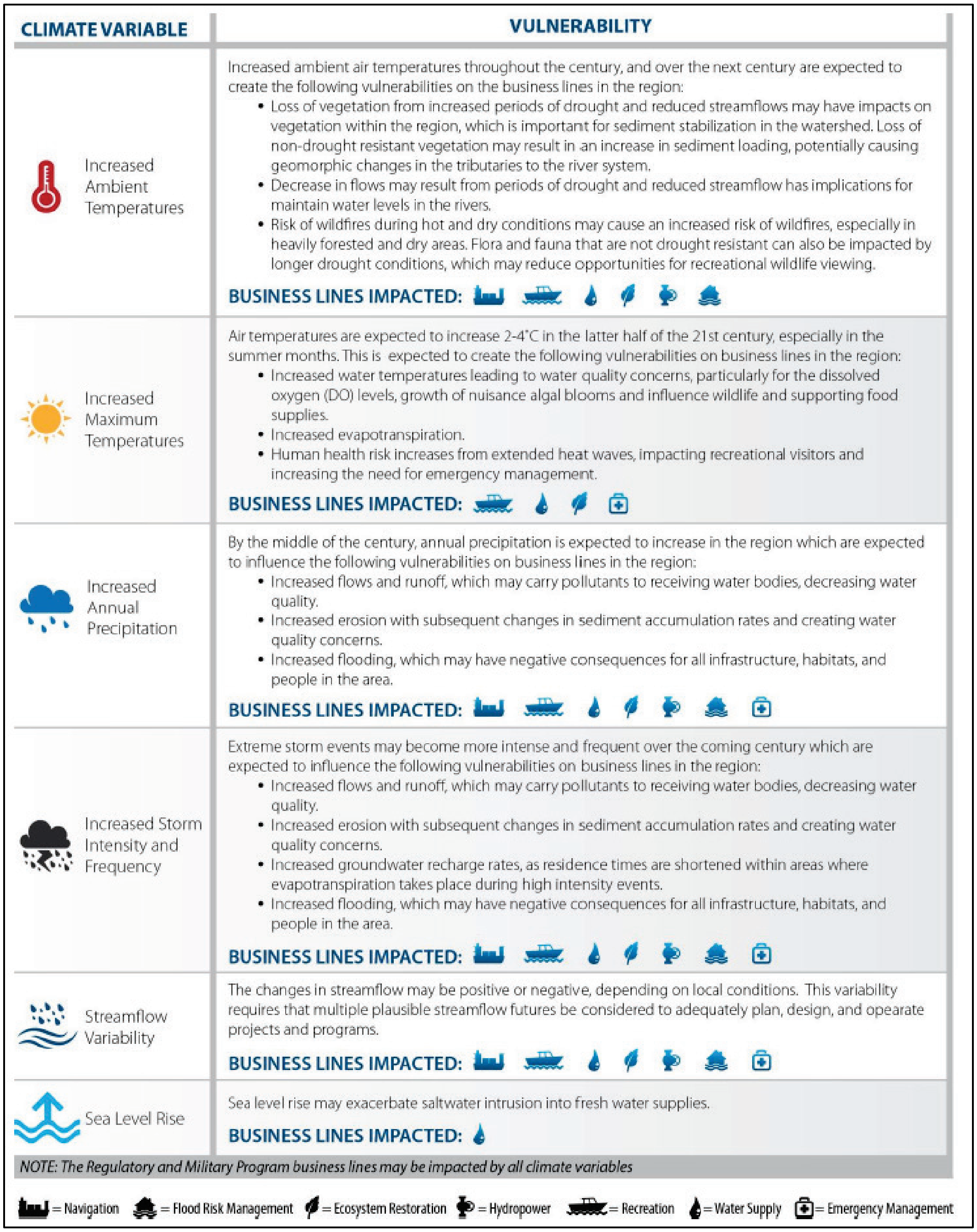
Figure 2-30. Summary of projected climate trends and impacts on USACE business lines for Region 07 – Upper Mississippi (USACE 2015c).

CLIMATE VARIABLE	VULNERABILITY
Increased Ambient Temperatures	By mid-century, increased ambient air temperatures are expected to create the following vulnerabilities on the business lines in the region: • Loss of vegetation from increased periods of heat and variable streamflows may have impacts on vegetation within the region, which is important for sediment stabilization in the watershed. Loss of non-drought resistant vegetation may result in an increase in sediment loading, potentially causing geomorphic changes in the tributaries to the river system. • Variable flows, have implications for maintain water levels in the rivers and lakes. • Risk of wildfires during hot and dry conditions may cause an increased risk of wildfires, especially in heavily forested and dry areas. Flora and fauna that are not drought resistant can also be impacted by longer drought conditions, which may reduce opportunities for recreational wildlife viewing. **BUSINESS LINES IMPACTED:**
Increased Maximum Temperatures	Air temperatures are expected to increase 1-4.5°C by mid century, with the number of heat wave days per year increasing by 15-50 days. This is expected to create the following vulnerabilities on business lines in the region: • Increased water temperatures leading to water quality concerns, particularly for the dissolved oxygen (DO) levels, growth of nuisance algal blooms and influence wildlife and supporting food supplies. • Increased evapotranspiration. • Human health risk increases from extended heat waves, impacting recreational visitors and increasing the need for emergency management. **BUSINESS LINES IMPACTED:**
Increased Annual Precipitation	Annual precipitation is expected to increase in the region which are expected to influence the following vulnerabilities on business lines in the region: • Increased flows and runoff, which may carry pollutants to receiving water bodies, decreasing water quality. • Increased erosion with subsequent changes in sediment accumulation rates and creating water quality concerns. • Increased flooding, which may have negative consequences for all infrastructure, habitats, and people in the area. **BUSINESS LINES IMPACTED:**
Increased Storm Intensity and Frequency	Extreme storm events may become more frequent and intense over the coming century which are expected to influence the following vulnerabilities on business lines in the region: • Increased runoff during an event, which may carry pollutants to receiving water bodies, decreasing water quality. • Increased erosion with subsequent changes in sediment accumulation rates and creating water quality concerns. • Change in engineering design standards to accommodate new extreme storms magnitudes. • Increased flash flooding, which may have negative consequences for all infrastructure, habitats, and people in the area. **BUSINESS LINES IMPACTED:**
Streamflow Variability	Streamflow is expected to increase by the end of the century. This includes an increase in overall flow and an increase of peak flow: • Increased flows and runoff, which may carry pollutants to receiving water bodies, decreasing water quality. • Increased erosion with subsequent changes in sediment accumulation rates and creating water quality concerns. • Increased flooding, which may have negative consequences for all infrastructure, habitats, and people in the area. • Loss of vegetation from increased periods of drought and reduced streamflows may have impacts on vegetation within the region, which is important for sediment stabilization in the watershed. Loss of non-drought resistant vegetation may result in an increase in sediment loading, potentially causing geomorphic changes in the tributaries to the river sytem. • Decrease in flows may result from periods of drought and reduced streamflow has implication for maintaining water levels in the rivers. **BUSINESS LINES IMPACTED:**

NOTE: The Regulatory and Military Program business lines may be impacted by all climate variables

= Navigation = Flood Risk Management = Ecosystem Restoration = Hydropower = Recreation = Water Supply = Emergency Management

Figure 2-31. Summary of projected climate trends and impacts on USACE business lines for Region 08 – Lower Mississippi (USACE 2015d).*



CLIMATE VARIABLE	VULNERABILITY
Increased Ambient Temperatures	Increased ambient air temperatures throughout the century, and over the next century are expected to create the following vulnerabilities on the business lines in the region: • Loss of vegetation from increased periods of drought and reduced streamflows may have impacts on vegetation within the region, which is important for sediment stabilization in the watershed. Loss of non-drought resistant vegetation may result in an increase in sediment loading, potentially causing geomorphic changes in the tributaries to the river system. • Decrease in flows may result from periods of drought and reduced streamflow has implications for maintain water levels in the rivers. • Risk of wildfires during hot and dry conditions may cause an increased risk of wildfires, especially in heavily forested and dry areas. Flora and fauna that are not drought resistant can also be impacted by longer drought conditions, which may reduce opportunities for recreational wildlife viewing. **BUSINESS LINES IMPACTED:**
Increased Maximum Temperatures	Air temperatures are expected to increase 2-4°C in the latter half of the 21st century, especially in the summer months. This is expected to create the following vulnerabilities on business lines in the region: • Increased water temperatures leading to water quality concerns, particularly for the dissolved oxygen (DO) levels, growth of nuisance algal blooms and influence wildlife and supporting food supplies. • Increased evapotranspiration. • Human health risk increases from extended heat waves, impacting recreational visitors and increasing the need for emergency management. **BUSINESS LINES IMPACTED:**
Increased Annual Precipitation	By the middle of the century, annual precipitation is expected to increase in the region which are expected to influence the following vulnerabilities on business lines in the region: • Increased flows and runoff, which may carry pollutants to receiving water bodies, decreasing water quality. • Increased erosion with subsequent changes in sediment accumulation rates and creating water quality concerns. • Increased flooding, which may have negative consequences for all infrastructure, habitats, and people in the area. **BUSINESS LINES IMPACTED:**
Increased Storm Intensity and Frequency	Extreme storm events may become more intense and frequent over the coming century which are expected to influence the following vulnerabilities on business lines in the region: • Increased flows and runoff, which may carry pollutants to receiving water bodies, decreasing water quality. • Increased erosion with subsequent changes in sediment accumulation rates and creating water quality concerns. • Increased groundwater recharge rates, as residence times are shortened within areas where evapotranspiration takes place during high intensity events. • Increased flooding, which may have negative consequences for all infrastructure, habitats, and people in the area. **BUSINESS LINES IMPACTED:**
Streamflow Variability	The changes in streamflow may be positive or negative, depending on local conditions. This variability requires that multiple plausible streamflow futures be considered to adequately plan, design, and opearate projects and programs. **BUSINESS LINES IMPACTED:**
Sea Level Rise	Sea level rise may exacerbate saltwater intrusion into fresh water supplies. **BUSINESS LINES IMPACTED:**

NOTE: The Regulatory and Military Program business lines may be impacted by all climate variables

= Navigation = Flood Risk Management = Ecosystem Restoration = Hydropower = Recreation = Water Supply = Emergency Management

*Note: In addition to MR&T Authority, USACE has been authorized and funded to design and construct the Hurricane and Storm Damage Risk Reduction System.

Figure 2-32. Summary of projected climate trends and impacts on USACE business lines for Region 10 – Missouri River (USACE 2015e).

CLIMATE VARIABLE	VULNERABILITY
Increased Ambient Temperatures	Increased ambient air temperatures throughout the century, and over the next century are expected to create the following vulnerabilities on the business lines in the region: • Loss of vegetation from increased periods of drought and reduced streamflows may have impacts on vegetation within the region, which is important for sediment stabilization in the watershed. Loss of non-drought resistant vegetation may result in an increase in sediment loading, potentially causing geomorphic changes in the tributaries to the river system. • Decrease in flows may result from periods of drought has implications for maintain water levels in the rivers; however hydrological models show little concenseous on streamflow. • Risk of wildfires during hot and dry conditions may cause an increased risk of wildfires, especially in heavily forested and dry areas. Flora and fauna that are not drought resistant can also be impacted by longer drought conditions, which may reduce opportunities for recreational wildlife viewing. **BUSINESS LINES IMPACTED:**
Increased Maximum Temperatures	Air temperatures are expected to increase 4-8°C in the latter half of the 21st century, especially in the summer months. This is expected to create the following vulnerabilities on business lines in the region: • Increased water temperatures leading to water quality concerns, particularly for the dissolved oxygen (DO) levels, growth of nuisance algal blooms and influence wildlife and supporting food supplies. • Increased evapotranspiration. • Human health risk increases from extended heat waves, impacting recreational visitors and increasing the need for emergency management. **BUSINESS LINES IMPACTED:**
Increased Annual Precipitation	By the middle of the century, annual precipitation is expected to increase in the region which are expected to influence the following vulnerabilities on business lines in the region: • Times of increased streamflows and runoff, which may carry pollutants to receiving water bodies, decreasing water quality. • Increased erosion with subsequent changes in sediment accumulation rates and creating water quality concerns. • Increased flooding, which may have negative consequences for all infrastructure, habitats, and people in the area. **BUSINESS LINES IMPACTED:**

NOTE: The Regulatory and Military Program business lines may be impacted by all climate variables

= Navigation = Flood Risk Management = Ecosystem Restoration = Hydropower = Recreation = Water Supply = Emergency Management

Figure 2-33. Summary of projected climate trends and impacts on USACE business lines for Region 11 – Arkansas, White, and Red Rivers (USACE 2015f).

CLIMATE VARIABLE	VULNERABILITY
Increased Ambient Temperatures	Increased ambient air temperatures throughout the century, and over the next century are expected to create the following vulnerabilities on the business lines in the region: • Loss of vegetation from increased periods of drought and reduced streamflows may have impacts on vegetation within the region, which is important for sediment stabilization in the watershed. Loss of non-drought resistant vegetation may result in an increase in sediment loading, potentially causing geomorphic changes in the tributaries to the river system. • Decrease in flows may result from periods of drought and reduced streamflow has implications for maintain water levels in the rivers. • Risk of wildfires during hot and dry conditions may cause an increased risk of wildfires, especially in heavily forested and dry areas. Flora and fauna that are not drought resistant can also be impacted by longer drought conditions, which may reduce opportunities for recreational wildlife viewing. **BUSINESS LINES IMPACTED:**
Increased Maximum Temperatures	Air temperatures are expected to increase 2-4°C in the latter half of the 21st century, especially in the summer months. This is expected to create the following vulnerabilities on business lines in the region: • Increased water temperatures leading to water quality concerns, particularly for the dissolved oxygen (DO) levels, growth of nuisance algal blooms and influence wildlife and supporting food supplies. • Increased evapotranspiration. • Human health risk increases from extended heat waves, impacting recreational visitors and increasing the need for emergency management. **BUSINESS LINES IMPACTED:**
Increased Storm Intensity and Frequency	Extreme storm events may become more intense and frequent over the coming century which are expected to influence the following vulnerabilities on business lines in the region: • Increased flows and runoff, which may carry pollutants to receiving water bodies, decreasing water quality. • Increased erosion with subsequent changes in sediment accumulation rates and creating water quality concerns. • Increased groundwater recharge rates, as residence times are shortened within areas where evapotranspiration takes place during high intensity events. • Increased flooding, which may have negative consequences for all infrastructure, habitats, and people in the area. **BUSINESS LINES IMPACTED:**
Streamflow Variability	The changes in streamflow may be positive or negative, depending on local conditions. This variability requires that multiple plausible streamflow futures be considered to adequately plan, design, and opearate projects and programs. **BUSINESS LINES IMPACTED:**

NOTE: The Regulatory and Military Program business lines may be impacted by all climate variables

= Navigation = Flood Risk Management = Ecosystem Restoration = Hydropower = Recreation = Water Supply = Emergency Management

The results show that the Mississippi River Basin will continue to be vulnerable in business lines related to flood risk management, navigation, emergency management, and other business lines across the twenty-first century, with higher vulnerability under a drier future scenario. This information should be considered during future analysis to increase resiliency of proposed project alternatives and reduce vulnerabilities. Vulnerabilities consist primarily of uncertainty in basic hydrologic parameters such as temperature and rainfall inputs, infiltration, land use, rainfall-runoff characteristics, and snowmelt. Because there is no defined approach for quantitative assessment of climate change or its effects for a large complex watershed such as the Mississippi River Basin, each of the hydrologic parameters should undergo sensitivity testing to determine the significance of each relative to future climate change scenarios.

2.2.12.5 Climate change summary

The regional literature review indicates an overall increase in streamflow over the next century. The Lower Missouri River Basin, Lower Ohio River Basin, Lower Tennessee River Basin, and Cumberland show a significant upward trend that indicates increased streamflow over the next century, but the results are conflicting with other studies that show both an upward and downward trend for the same basins. The Mississippi River 2011 Post Flood Assessment concludes that a projected increase in maximum dew point temperature would require a revision to the rainfall used for the PDF, but current precipitation values are valid for the climate trend data available. Therefore, the meteorological and hydrological underpinnings of the MR&T PDF are found to be adequate for the present climate. As climate change understanding, methodology, and tools develop in the coming years and decades, the effects of climate change on the Mississippi River Basin hydrology should be monitored, and future PDF Flowline computations should reassess the observed and potential changes due to climate change.

HYPO Storm Naming Convention

58A = 1955 Unregulated
58A-EN = 1955 Regulated (Group EN)
58A-U = 2016 Unregulated
58A-R = 2016 Regulated
58A-U-CM= 2016 Clipped-Merged Unregulated
58A-R-CM= 2016 Clipped-Merged Regulated

In the appendices of the Hydrology Report (USACE Hydrology 2018a), HYPO 52A, 56, and 63 results follow the same naming convention for each respective HYPO event.

2.3 Results

This section presents the results of the 2016 hydrologic assessment of the HYPO 58A flowline. To assess the adequacy of the 1955-vintage hydrology for the PDF, this hydrologic assessment provided (1) a means to compare current (Straight Sequence) and original (Clipped-Merged Sequence) methodology by running both methods through the CHPS-FEWS model; (2) upstream inflow and local contributing hydrographs needed for the boundary conditions in the HEC-RAS unsteady model; (3) output hydrographs comparable in magnitude and volume to the 1955 study; and (4) the hydrologic computations for antecedent conditions, rainfall-runoff, and reservoir effects based on current practice.

The 1955 methodology (Clipped-Merged Sequence) was reproduced with current tools/technology and was compared to the 2016 methodology (Straight Sequence). The Straight Sequence represented the standard method to apply storms across the Mississippi River Basin according to current practice. Local USACE Districts and the NWS RFCs were provided with precipitation and temperature grid files from the Straight Sequence to simulate rainfall-runoff processes through their sub-basins and reservoirs. HEC-RAS inflow boundary and local contribution hydrographs are included in this section. Finally, the resulting 2016 peak flow values at several locations along the Lower Mississippi River are presented.

2.3.1 Comparison of peak flows for HYPO storms

Peak flows at select locations are given in Table 2-13. The table includes peak flow values resulting from the hydrologic modeling and from the HEC-RAS unsteady flow routing.

Table 2-13. HYPO peak unregulated flow comparisons at select locations.

River	Location	Peak Flow (kcfs)				
		HYPO 58A-U	HYPO 52A-U1	HYPO 56-U	HYPO 63-U	HYPO 11-73-U
Missouri	Hermann, MO	220	675	575	946	668
Mississippi	St Louis, MO	433	1,125	955	1,362	1,024
Ohio	Metropolis, IL	2,461	585	1,548	1,076	1,734
Mississippi	Cairo, IL	2,937	1,555	2,568	2,387	2,806
White	Clarendon, AR	271	40	316	213	311
Mississippi	Arkansas City, AR	3,367	1,517	3,457	3,111	3,286

[1] Results from HYPO 52A simulations were not comparable to other HYPO simulations. Precipitation and temperature inputs were determined as the cause for the discrepancy, but problems could not be resolved within study schedule.

HYPO 58A remains the governing storm event for the PDF on the lower Mississippi River even considering the new HYPO 11-73. Note that Table 2-13 shows peak unregulated flow. Peak flows from HYPO 11-73 results were lower than determined for HYPO 58A for Cairo, IL; Metropolis, IL; and Arkansas City, AR.

HYPO 11-73 has the second highest magnitude of flow compared to HYPO 58A as shown in Figure 2-34.

Figure 2-34. HYPO storm event comparison for 2016 HYPO 52A, 56, 58A, and 63 plus the 2016 HYPO 11-73 storm for Cairo, IL, for the combined confluence flows.

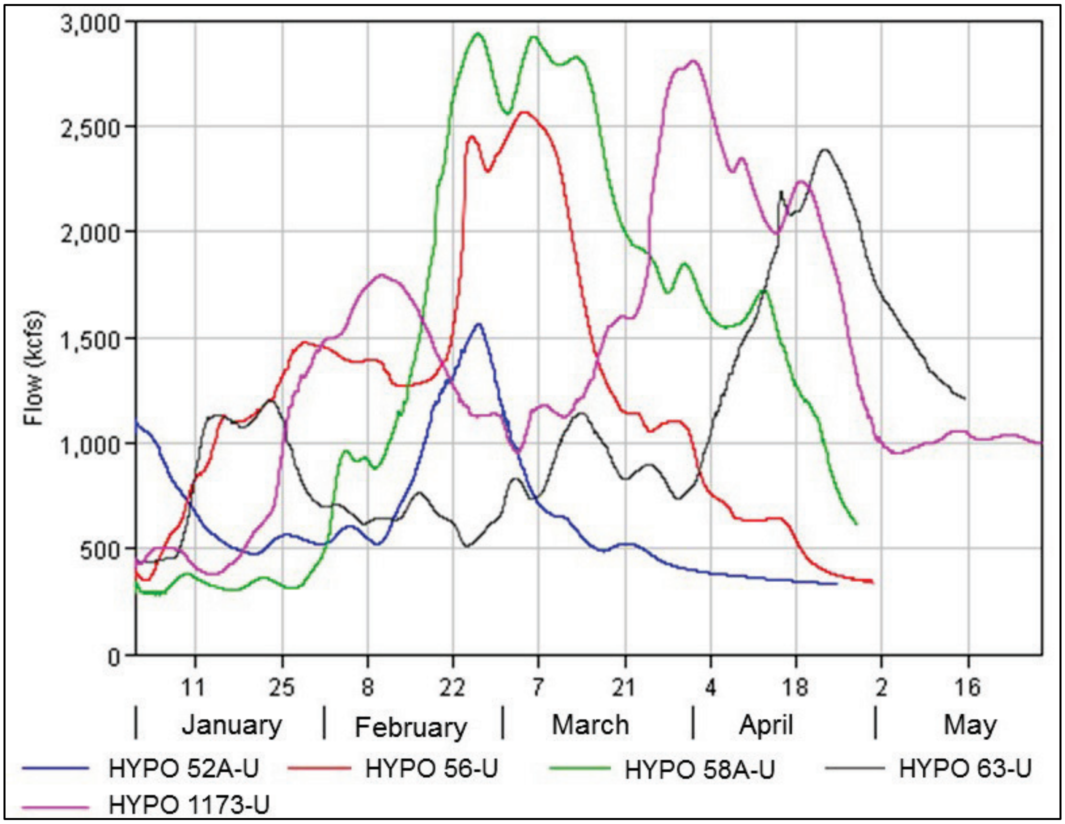


Table 2-14 compares the peak flows for the HYPO 58A storm event.

2.3.1.1 Hydrograph shape and volume

In addition to comparing peak discharges and a visual comparison of hydrograph plots, hydrograph shape was assessed using volume and mean discharge over 3-, 15-, and 30-day periods. Table 2-14 shows that results from the current assessment yielded higher 1-day peaks as well as larger volumes than obtained from the 1955 analysis. For example, at Cairo (combined confluence), 2016 58A-R values of 2,791 kcfs, 2,776 kcfs, 2,687 kcfs, and 2,518 kcfs are shown for the 1-day peak flow, 3-day mean flow, 15-day mean flow, and 30-day mean flow, respectively. Corresponding values for 1955 58A-EN were 2,336 kcfs, 2,336 kcfs, 2,318 kcfs, and 2,303 kcfs, which illustrates that 2016 results produced higher values throughout the 30-day flood crest period. This means a larger volume of flow actually passed through the system for the 2016 simulations than indicated from the 1955 study. This may have limited the ability of reservoirs to reduce releases since they had to account for additional volumes of water.

Note that the table includes an additional simulation, labeled 58A-EN-2016HECRAS, that utilized the 1955 discharge hydrographs (see Section 2.2.1.1). More information about this run, which used the existing elevations of the backwater areas, can be found in the Hydraulics Report (USACE 2018b). The additional run provided a means to directly compare hydrodynamic routing effects from HEC-RAS with the 1955 study results. No 58A-EN-2016HECRAS values were given at St. Louis, MO, because that location was outside the HEC-RAS model domain. Peak and 3-day, 15-day, and 30-day mean flows from the HEC-RAS 58A-EN-2016 HECRAS results were only slightly higher than the corresponding 1955 58A-EN values. This suggested that hydrodynamic effects had no appreciable effect on the computed peak flows; however, the timing of the peak values was different. The 2016 peaks occurred approximately 6 days later than indicated in the 1955 study.

Table 2-14. Hydrograph volume and shape comparisons for HYPO 58A.

Mississippi River HYPO Flood Flows are in 1,000 cfs and second-foot-day (sfd) units															
				3 Days				15 Days				30 days			
	Storm	Peak Flow	Peak Flow Date	Start	End	Volume (sfd)	Mean (cfs)	Start	End	Volume (sfd)	Mean (cfs)	Start	End	Volume (sfd)	Mean (cfs)
St. Louis	58A-R	378	24-Jan	23-Jan	25-Jan	1,112	371	11-Jan	25-Jan	5,015	334	14-Jan	12-Feb	9,684	323
	58A-U	433	22-Feb	20-Feb	22-Feb	1,254	418	10-Jan	24-Jan	5,342	356	10-Jan	8-Feb	9,874	329
	58A-EN	241	11-Feb	10-Feb	12-Feb	695	232	7-Feb	21-Feb	2,514	168	31-Jan	1-Mar	4,747	158
	58A	255	11-Feb	10-Feb	12-Feb	738	246	7-Feb	21-Feb	2,779	185	31-Jan	1-Mar	5,126	171
Combined Confluence	58A-R	2,791	8-Feb	7-Feb	9-Feb	8,327	2,776	29-Jan	12-Feb	40,311	2,687	21-Jan	19-Jan	75,546	2,518
	58A-U	2,937	26-Jan	24-Jan	26-Jan	8,735	2,912	23-Jan	6-Feb	41,932	2,795	19-Jan	17-Feb	80,480	2,683
	58A-EN-2016HECRAS	2,393	30-Jan	17-Feb	19-Feb	7,160	2,387	7-Feb	21-Feb	35,236	2,349	26-Jan	24-Feb	69,683	2,323
	58A-EN	2,336	12-Feb	11-Feb	13-Feb	7,008	2,336	2-Feb	16-Feb	34,800	2,318	27-Jan	25-Feb	69,096	2,303
	58A	2,856	12-Feb	11-Feb	13-Feb	8,520	2,840	2-Feb	16-Feb	39,989	2,666	27-Jan	25-Feb	74,026	2,468
Arkansas City	58A-R	3,263	22-Feb	21-Feb	23-Feb	9,772	3,257	11-Feb	25-Feb	47,930	3,195	2-Feb	3-Mar	92,216	3,074
	58A-U	3,366	22-Feb	20-Feb	22-Feb	10,086	3,362	10-Feb	24-Feb	49,732	3,315	1-Feb	2-Mar	97,101	3,237
	58A-EN-2016HECRAS	2,874	4-Mar	2-Mar	4-Mar	8,607	2,869	22-Feb	8-Mar	42,220	2,815	10-Feb	11-Mar	80,730	2,691
	58A-EN	2,880	2-Mar	1-Mar	3-Mar	8,620	2,873	21-Feb	7-Mar	42,070	2,805	9-Feb	10-Mar	80,195	2,673
	58A	3,220	2-Mar	1-Mar	3-Mar	9,640	3,213	21-Feb	7-Mar	46,560	3,104	9-Feb	10-Mar	88,170	2,939
Latitude of Red River Landing	58A-R	3,211	2-Mar	28-Feb	2-Mar	9,926	3,309	22-Feb	8-Mar	49,029	3,269	13-Feb	14-Mar	95,346	3,178
	58A-U	3,376	1-Mar	27-Feb	1-Mar	10,589	3,530	20-Feb	6-Mar	52,246	3,483	13-Feb	14-Mar	101,983	3,399
	58A-EN-2016HECRAS	2,788	6-Mar	5-Mar	7-Mar	8,359	2,786	27-Feb	13-Mar	41,494	2,766	23-Feb	24-Mar	81,651	2,722
	58A-EN	2,956	13-Mar	12-Mar	14-Mar	8,738	2,913	7-Mar	21-Mar	43,800	2,920	28-Feb	29-Mar	86,100	2,870
	58A	3,325	13-Mar	12-Mar	14-Mar	9,967	3,322	7-Mar	21-Mar	49,030	3,269	28-Feb	29-Mar	93,946	3,132

2.3.1.2 Hydrograph timing

The timing between hydrographs at different locations affected the way peak values could be related at different locations. The travel time of the flood crest as it moves downstream can be clearly seen in Table 2-14. While this phenomena is straight forward between locations separated by some distance, like Cairo and Arkansas City, considering peak values at points near the Mississippi and Ohio River confluence was not as clear. For example, estimating the flow contribution for the Mississippi River immediately upstream of the Ohio River could not be done using the individual peak values for the Ohio River and the Lower Mississippi River (below the mouth of the Ohio River). Figure 2-35 and Table 2-15 show hourly computed flows for four locations that are near the Mississippi/Ohio River confluence: Thebes, IL; Birds Point, MO; Metropolis, IL; and the Mississippi River just below the Ohio River.

Figure 2-35. Comparing peak flows near the Mississippi/Ohio River confluence.

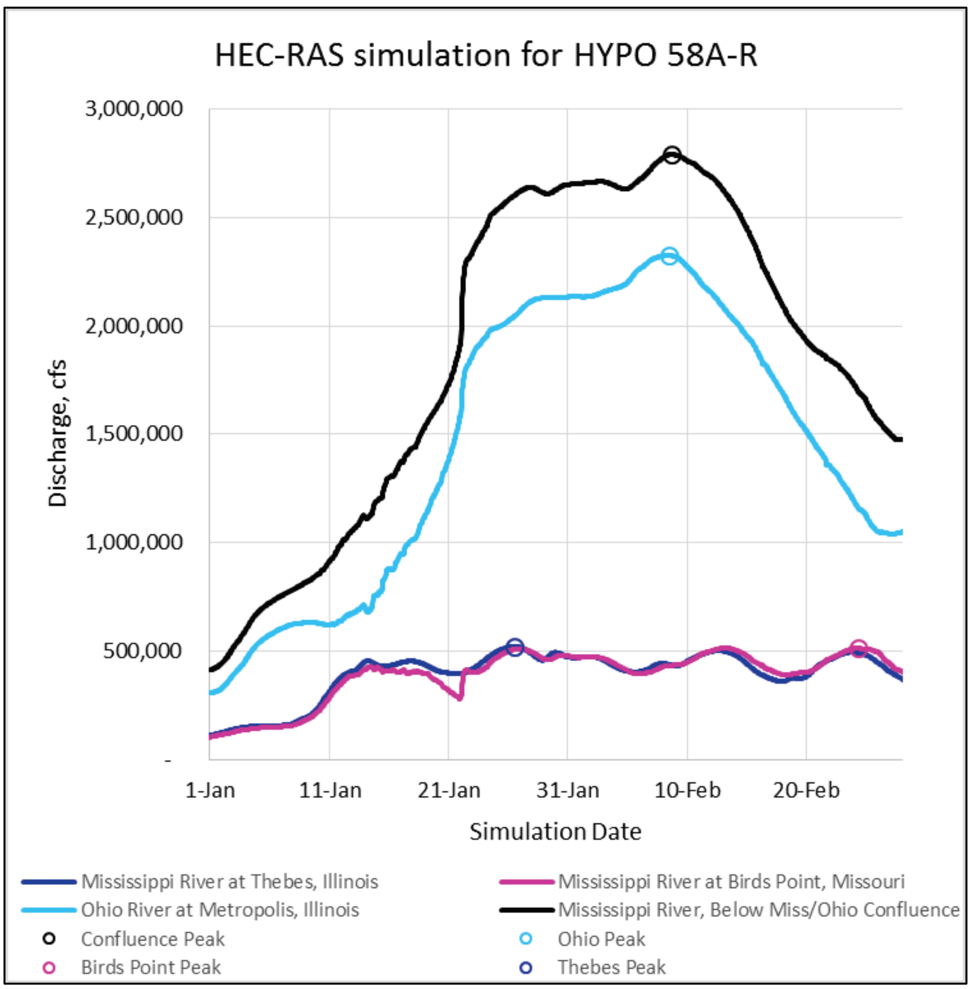


Table 2-15. Comparing peak flows near the Mississippi/Ohio River confluence.

HEC-RAS Simulation HYPO 58A-R	Mississippi River at Thebes, Illinois	Mississippi River at Birds Point, Missouri	Ohio River at Metropolis, Illinois	Mississippi River, Below Miss/Ohio Confluence
	HEC-RAS River Mile 43.7	HEC-RAS River Mile 4.6	HEC-RAS River Mile -979.7	HEC-RAS River Mile 973.8
Simulation date and time	Flow in cfs	Flow in cfs	Flow in cfs	Flow in cfs
1/26/58 6:00	519,000	504,000	2,033,000	2,592,000
1/26/58 7:00	520,000	505,000	2,035,000	2,594,000
1/26/58 8:00	520,000	506,000	2,037,000	2,596,000
1/26/58 9:00	520,000	506,000	2,038,000	2,597,000
1/26/58 10:00	520,000	507,000	2,040,000	2,599,000
1/26/58 11:00	520,000	508,000	2,042,000	2,600,000
1/26/58 12:00	**520,000**	508,000	2,043,000	2,602,000
1/26/58 13:00	520,000	509,000	2,045,000	2,603,000
1/26/58 14:00	520,000	509,000	2,047,000	2,605,000
1/26/58 15:00	520,000	510,000	2,049,000	2,606,000
1/26/58 16:00	520,000	510,000	2,051,000	2,608,000
1/26/58 17:00	520,000	510,000	2,053,000	2,609,000
2/8/58 6:00	440,000	432,000	2,325,000	2,786,000
2/8/58 7:00	439,000	432,000	2,326,000	2,787,000
2/8/58 8:00	439,000	432,000	2,326,000	2,788,000
2/8/58 9:00	438,000	432,000	2,326,000	2,788,000
2/8/58 10:00	438,000	433,000	**2,326,000**	2,789,000
2/8/58 11:00	437,000	433,000	2,326,000	2,790,000
2/8/58 12:00	437,000	433,000	2,326,000	2,790,000
2/8/58 13:00	436,000	433,000	2,326,000	2,791,000
2/8/58 14:00	436,000	433,000	2,325,000	2,791,000
2/8/58 15:00	435,000	434,000	2,325,000	2,791,000
2/8/58 16:00	435,000	***434,000***	***2,324,000***	***2,791,000***
2/8/58 17:00	435,000	434,000	2,324,000	2,791,000
2/24/58 6:00	493,000	515,000	1,166,000	1,705,000
2/24/58 7:00	493,000	**515,000**	1,162,000	1,702,000
2/24/58 8:00	492,000	515,000	1,159,000	1,699,000
2/24/58 9:00	492,000	514,000	1,157,000	1,696,000
2/24/58 10:00	491,000	514,000	1,155,000	1,693,000
2/24/58 11:00	491,000	513,000	1,154,000	1,691,000

For example, the peak computed flow below the Mississippi/Ohio confluence (2,791,000 cfs) and the peak computed flow on the Ohio River at Metropolis, IL (2,326,000 cfs), by basic continuity where volume is preserved (flowrate in – flowrate out = zero) suggests that flow from the upper Mississippi River would be 465,000 cfs (2,791,000 – 2,326,000 = 465,000). However, the table shows that flow from the upper Mississippi River was actually 435,000 cfs at the time when the combined Mississippi/Ohio flow was 2,791,000 cfs. In other words, the hydrodynamic routing effects from HEC-RAS must be considered when analyzing peak flow values, particularly near major tributary junctions. Additionally, the peak flow of 515,000 cfs on the Mississippi River at Birds Point, MO, occurred 16 days after the peak at the confluence due to the hydrodynamics of the reach.

Corresponding values (Table 2-16) from 1955 obtained from the MRC archives[1] were 2,250,000 cfs (Ohio River at Metropolis), 2,384,000 cfs (below Mississippi/Ohio confluence), and in this case, 134,000 cfs (Mississippi upstream of the Ohio River). The estimated peak on the Mississippi River upstream of the Ohio River was 410,000 cfs, which occurred 6 days prior to the peak at the confluence. Routing calculations from the 1955 study were based on a simple lag approach as described in the Hydrology Report (USACE 2018a), where total flows were determined by summing all contributing flows for a time-step.

Peak regulated flow for HYPO 58A on the Ohio River was computed in 2016 as 2,326,000 cfs compared to 2,250,000 cfs from the 1955 study. Peak flow on the Mississippi River near Birds Point, MO, was computed in 2016 as 515,000 cfs compared to 410,000 cfs from the 1955 study. The combined peak flow for the Mississippi and Ohio Rivers was computed in 2016 as 2,791,000 cfs compared to 2,360,000 cfs from the 1955 study. At the time of the main flood crest, computed flows were 435,000 cfs versus 134,000 cfs (Mississippi) and 2,324,000 cfs versus 2,250,000 (Ohio) for 2016 and 1955 results, respectively. Differences between the 2016 and 1955 results were +300,000 cfs (Mississippi) and +71,000 cfs (Ohio). The increase on the Ohio River for the current assessment can be attributed to differences in how regulation effects were calculated. The large increase on the Mississippi River for the current assessment can likewise be attributed to differences in regulation effects.

[1] MRC archives: Files containing original tabulated routing calculations (in support of the MRC 1955 study).

Table 2-16. Tabulated flows from 1955 routing calculations.

Mississippi River Project Flood Study					
Flood Routing of Hypo Flood No. 58-A					
			Condition	EN_Reservoirs	
			Reach	St. Louis-Metroplis to Cairo	
NOTE: New Madrid Floodway Operating (flows in cfs)					
HYPO 58A-EN (1955) Date	Ohio River at Metropolis, Illinois	Mississippi River at St. Louis, Missouri	Local Contribution St. Louis to Ohio River (Area 7-Y)	Mississippi River Upstream of Ohio River 58A-EN St. Louis + 7Y Local	Mississippi and Ohio River Combined (downstream of confluence)
2/8/58	2,130,000	156,000	48,000	204,000	2,318,000
2/9/58	2,050,000	193,000	78,000	271,000	2,318,000
2/10/58	1,970,000	235,000	123,000	358,000	2,318,000
2/11/58	1,940,000	241,000	169,000	410,000	2,336,000
2/12/58	1,950,000	218,000	164,000	382,000	2,336,000
2/13/58	1,990,000	214,000	132,000	346,000	2,336,000
2/14/58	2,050,000	177,000	109,000	286,000	2,336,000
2/15/58	2,120,000	145,000	41,000	186,000	2,318,000
2/16/58	2,200,000	125,000	11,000	136,000	2,327,000
2/17/58	2,250,000	124,000	10,000	134,000	2,353,000
2/18/58	2,240,000	130,000	10,000	140,000	2,362,000
2/19/58	2,220,000	132,000	10,000	142,000	2,362,000
2/20/58	2,180,000	142,000	18,000	160,000	2,353,000

2.3.2 Hydrologic Engineering Center, River Analysis System (HEC-RAS) inflow boundary discharge hydrographs

The hydrologic modeling produced discharge hydrographs at the same key locations used from computations in the 1955 study. The hydrologic modeling also produced discharge hydrographs at upstream stream boundary points required by the HEC-RAS model. Plots of unregulated (U: shown in red) and regulated (EN: shown in blue) model simulations are presented in Figure 2-36 through Figure 2-46. Plots of local inflow hydrographs for minor tributaries and intervening areas are not included in this report.

Figure 2-36. HYPO 58A hydrographs: Missouri River at Hermann, MO.

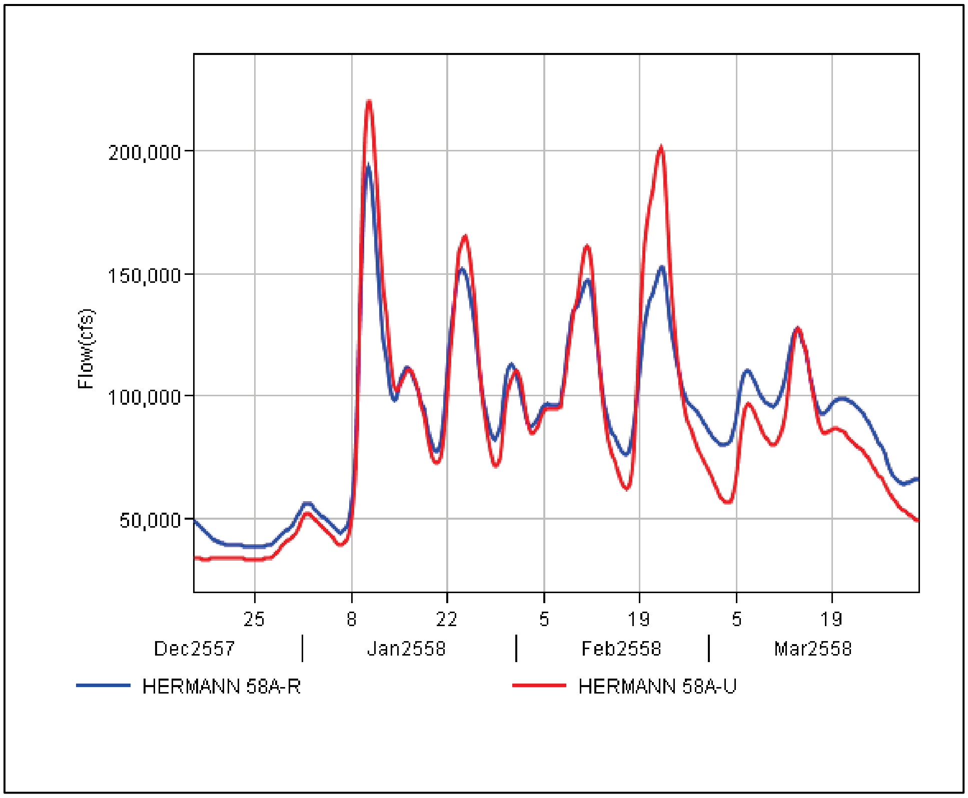


Figure 2-37. HYPO 58A hydrographs: Mississippi River at St. Louis, MO.

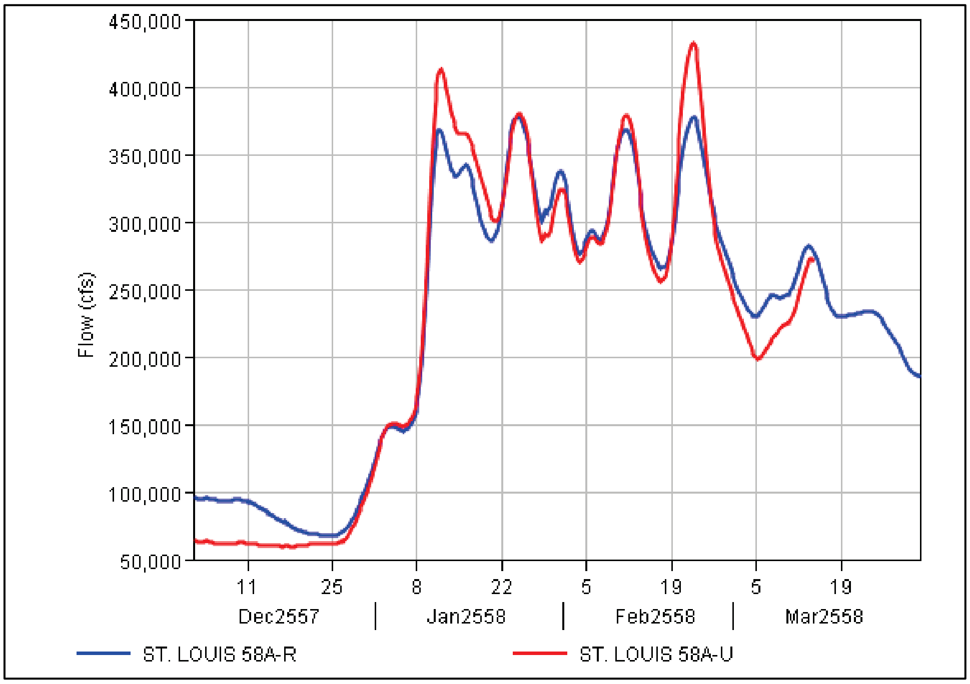


Figure 2-38. HYPO 58A hydrographs: Mississippi River at Chester, IL.[1]

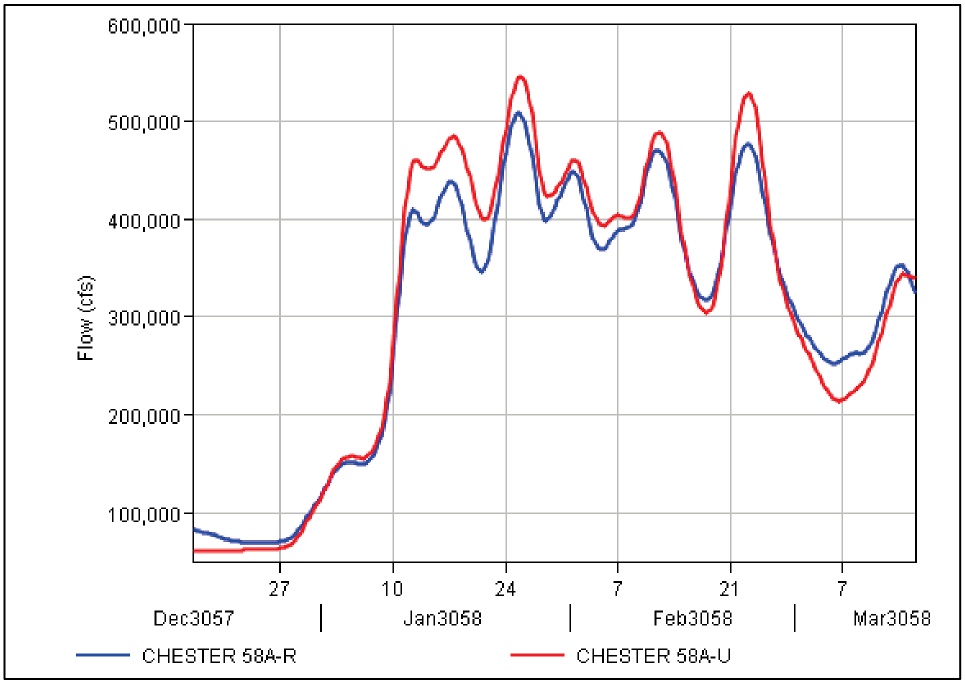
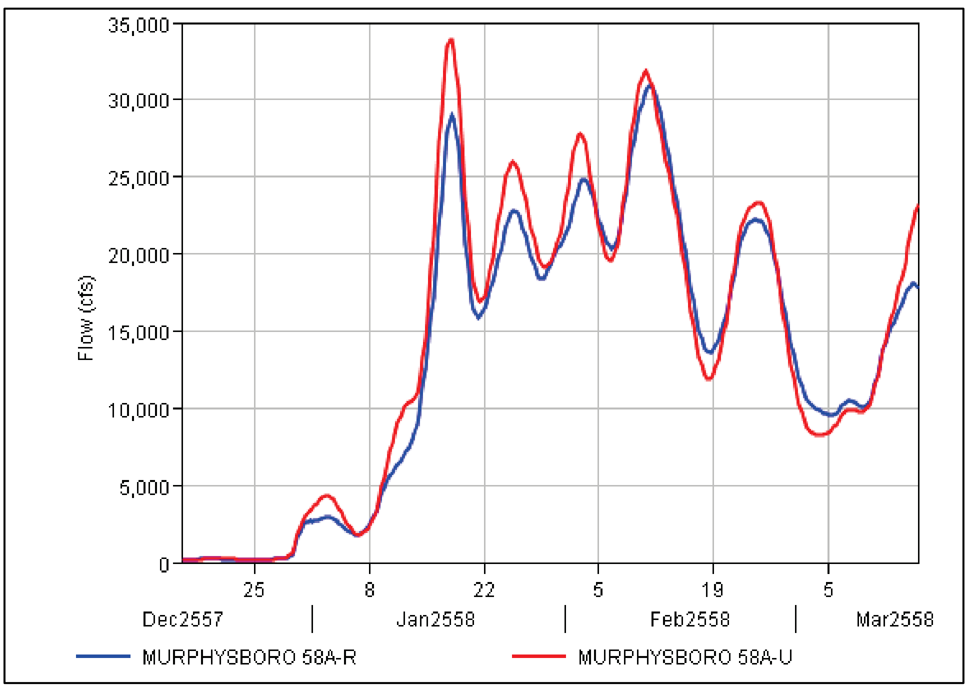


Figure 2-39. HYPO 58A hydrographs: Big Muddy River at Murphysboro, IL.



[1] The NWS model simulation for Mississippi River at Chester, IL, ended on March 14. To extend this hydrograph to March 31, the recession was manually estimated to follow the daily average recession for the additional 17 days required to allow routing the full hydrograph to the Gulf of Mexico in HEC-RAS.

Figure 2-40. HYPO 58A hydrographs: Cumberland River at Barkley Dam.

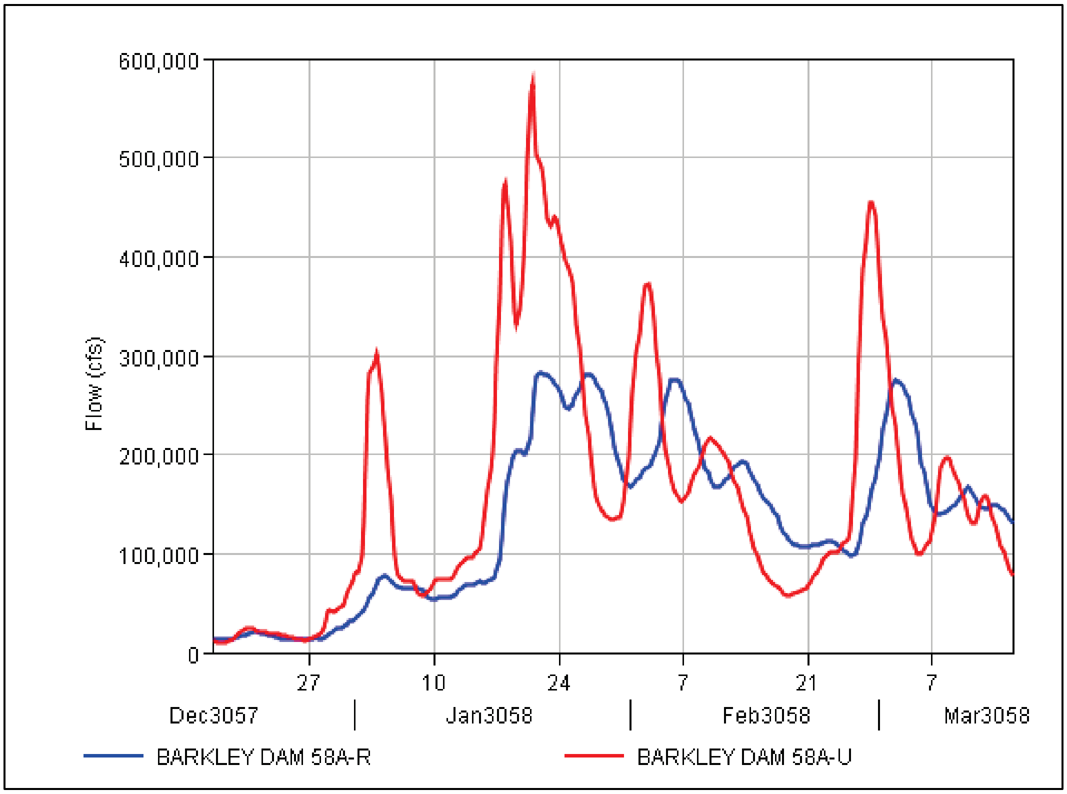


Figure 2-41. HYPO 58A hydrographs: Tennessee River at Kentucky Dam.

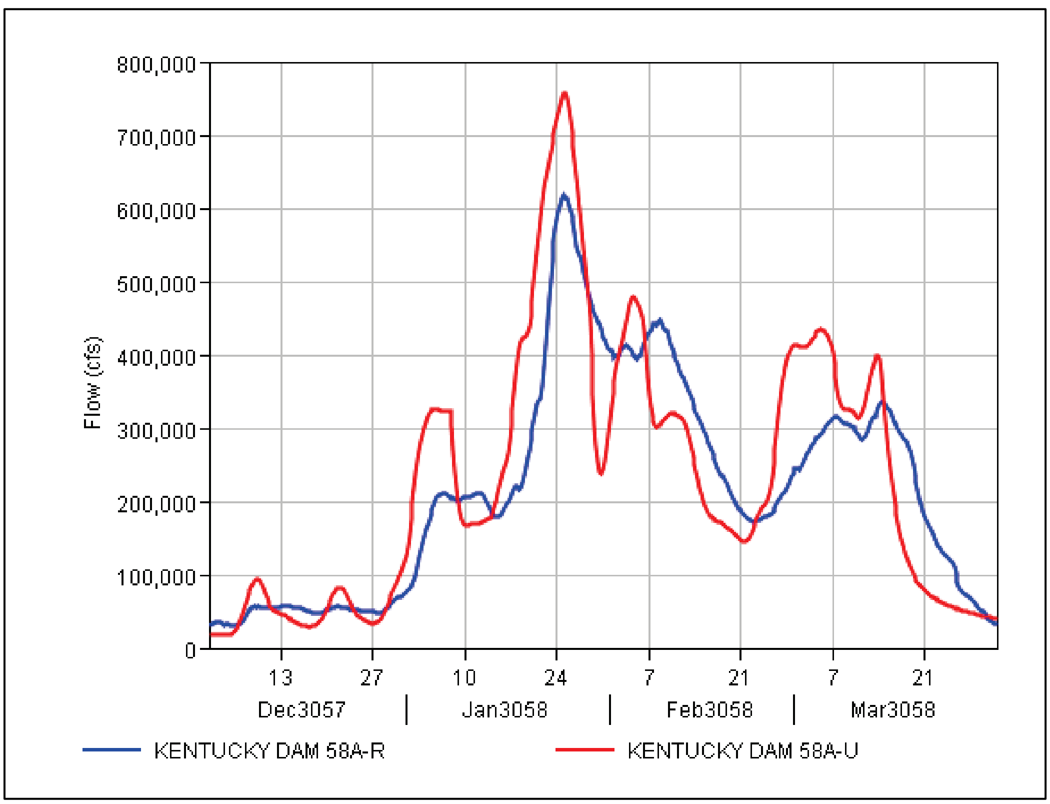


Figure 2-42. HYPO 58A hydrographs: Ohio River at Smithland, IL.

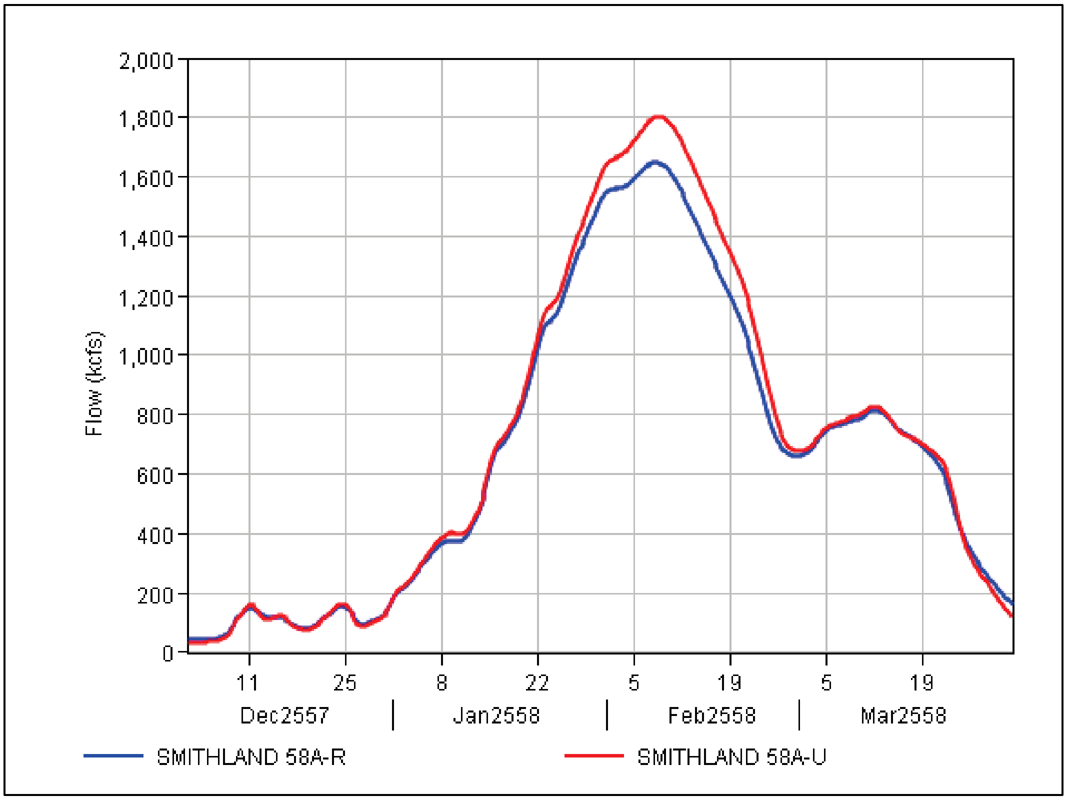


Figure 2-43. HYPO 58A hydrographs: Arkansas River at Pine Bluff, AR.

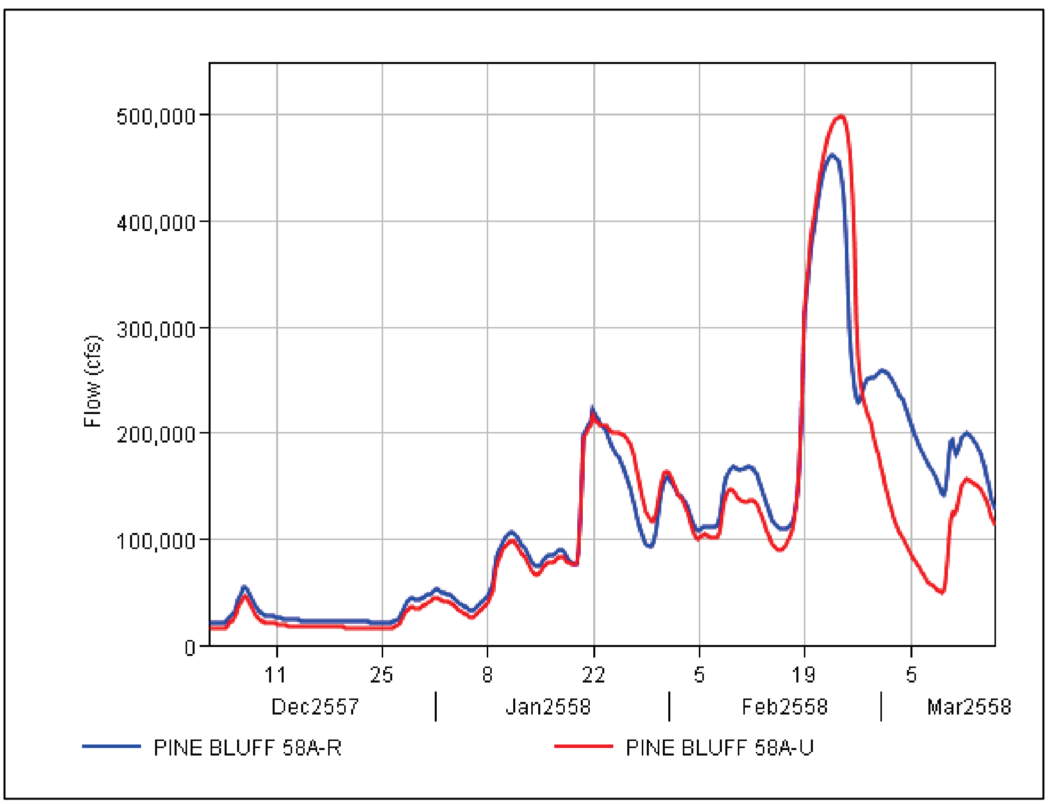


Figure 2-44. HYPO 58A hydrographs: Red River at Shreveport, LA.

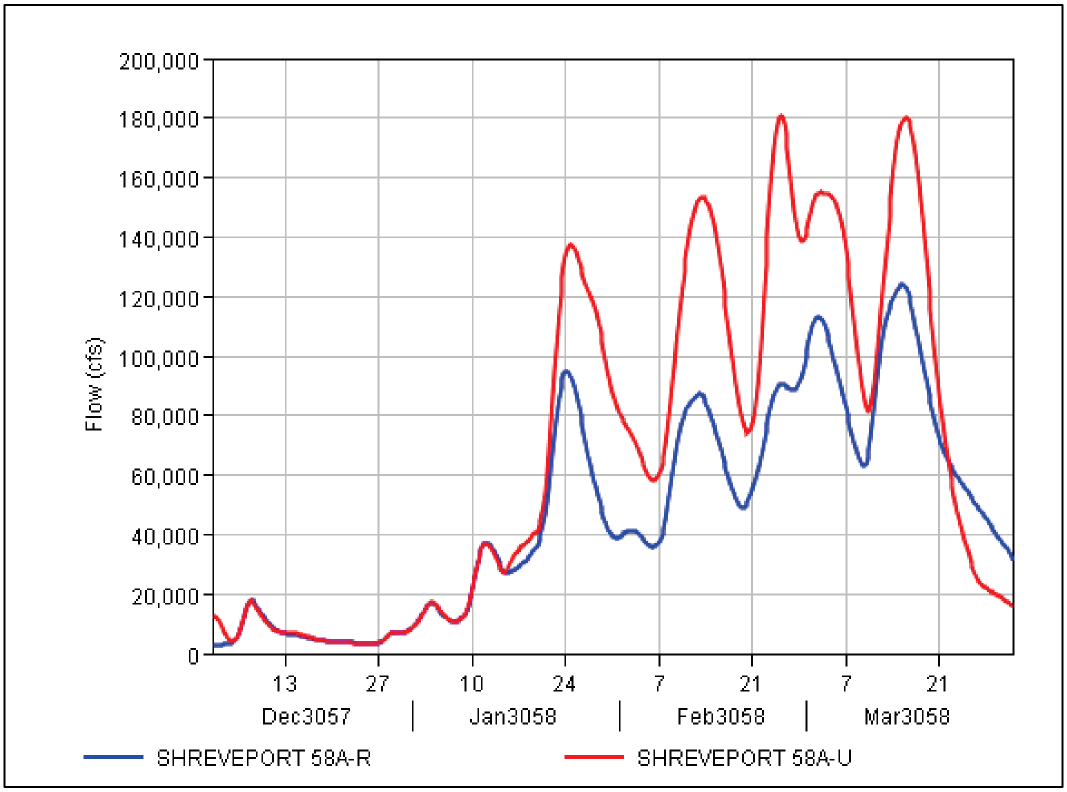


Figure 2-45. HYPO 58A hydrographs: Ouachita River at Monroe, LA.

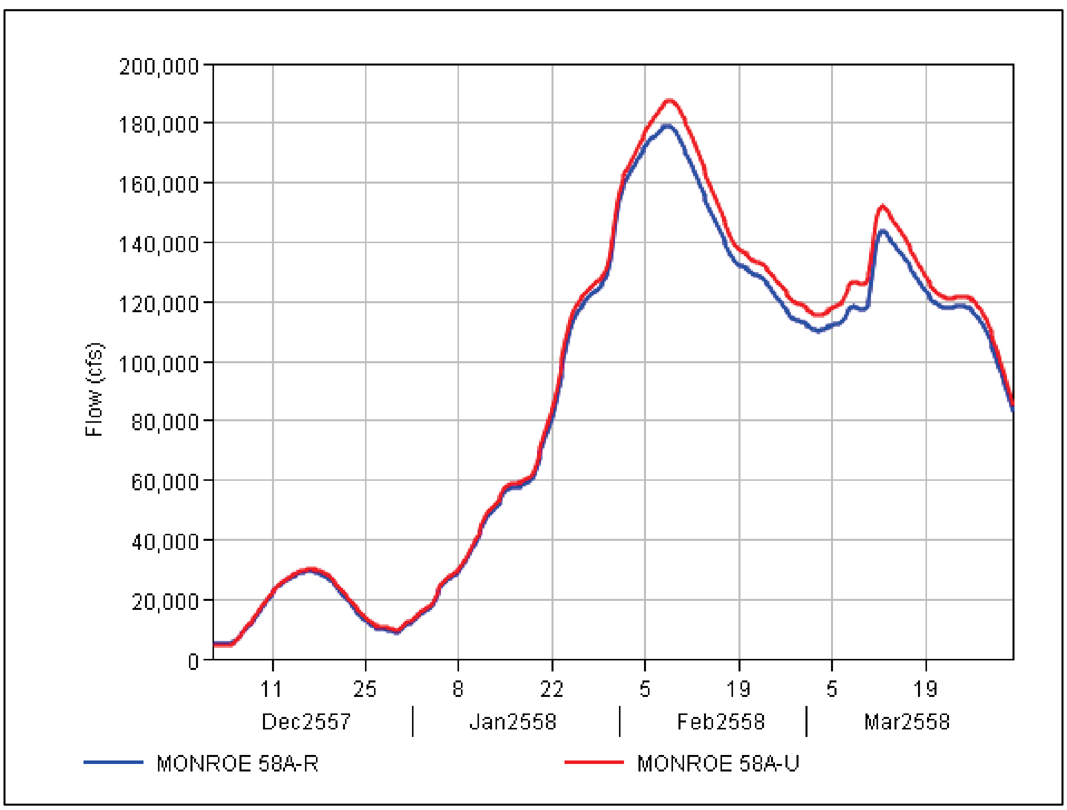
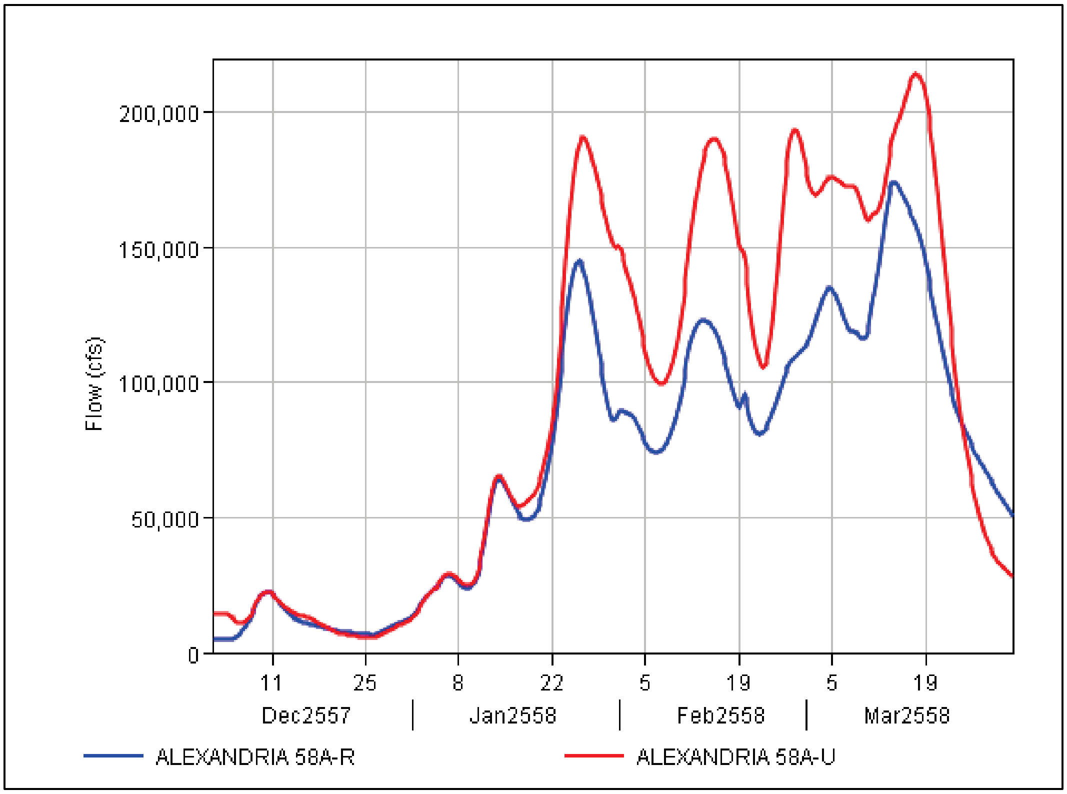


Figure 2-46. HYPO 58A hydrographs: Red River at Alexandria, LA.



2.3.3 Peak flows for the Lower Ohio and Mississippi Rivers

The combined MVD HEC-RAS model developed for the current assessment provided unsteady hydrodynamic flood routing along the main river channels within the assessment area. Peak flow for the Ohio and Lower Mississippi Rivers (points downstream of the Ohio/Mississippi confluence) are direct outputs from the HEC-RAS unsteady calculations. Table 2-17 provides the HEC-RAS results for HYPO 58A under existing conditions of the Yazoo Backwater levee and authorized conditions of the Yazoo Backwater levee[1] (see Section 3.3.2).

[1] The Yazoo Backwater levee was constructed after the 1957 report was published; therefore, the 1957 peak flow values are comparable to the Yazoo Backwater authorized condition. The authorized condition has a design grade of elevation 112.8 while the existing levee height is at elevation 107.0.

Table 2-17. Comparison of peak flow values for HYPO 58A.

Location	Project Design Flood (PDF) (from Table 7, USACE 1957)					
	Unregulated Discharge, cfs			Regulated Discharge, cfs		
	58A	58A-U Existing Yazoo	58A-U Authorized Yazoo	58A-EN	58A-R Existing Yazoo	58A-R Authorized Yazoo
Ohio at Cairo, IL	2,460,000	2,458,000	2,458,000	2,250,000	2,326,000	2,326,000
Miss/Ohio Confluence (Combined)	2,850,000	2,937,000	2,937,000	2,360,000	2,791,000	2,791,000
Hickman, KY	NA	1,988,000	1,988,000	NA	1,973,000	1,973,000
Memphis, TN	2,770,000	2,956,000	2,956,000	2,410,000	2,863,000	2,863,000
Helena, AR	2,710,000	2,862,000	2,862,000	2,460,000	2,788,000	2,788,000
Arkansas City, AR	3,210,000	3,367,000	3,367,000	2,890,000	3,264,000	3,263,000
Greenville, MS	NA	3,364,000	3,364,000	NA	3,260,000	3,260,000
Lake Providence, MS	NA	3,360,000	3,361,000	NA	3,255,000	3,257,000
Vicksburg, MS	2,960,000	3,081,000	3,100,000	2,710,000	2,979,000	3,060,000
Natchez, MS	2,970,000	3,084,000	3,112,000	2,720,000	2,977,000	3,069,000
Red River Landing, LA	2,240,000	2,460,000	2,490,000	2,100,000	2,351,000	2,445,000
Baton Rouge, LA	NA	1,853,000	1,886,000	NA	1,742,000	1,837,000
Donaldsonville, LA	NA	1,852,000	1,886,000	NA	1,741,000	1,837,000
Carrollton, LA	NA	1,596,000	1,630,000	NA	1,486,000	1,581,000
Empire, LA	NA	1,527,000	1,560,000	NA	1,420,000	1,513,000
Venice, LA	NA	1,104,000	1,126,000	NA	1,031,000	1,094,000

The original 1955 hydrographs were digitized and compared to the hydrographs produced from the 2016 RFC models at the boundary locations for the HEC-RAS model. Example hydrographs are shown in Figure 2-47 through Figure 2-51, where the label 58A represents 1955 unregulated values and 58A-U represents the current (2016) unregulated values. Labels of 58A-R represent the current (2016) regulated values, and 58A-EN represent the 1955 regulated values.

The shape and/or magnitude between the hydrographs at Alexandria, Alton, St. Louis, and Hermann are significantly different with the 2016 unregulated hydrographs being much higher than the 1955 study results. Little Rock shows that the 1955 results are much higher than the 2016 results, but the shape of the hydrograph is similar. The source of the differences is not easily identifiable; however, their source most likely lies in the precipitation sequencing (Straight Sequence versus Clipped-Merged Sequence), the inclusion of temperature effect, and differences in hydrologic modeling/routing techniques (see Section 2.2.3).

Figure 2-47. Alexandria, LA, HYPO 58A 1955 unregulated flow compared to 2016 unregulated flow generated by the RFC.

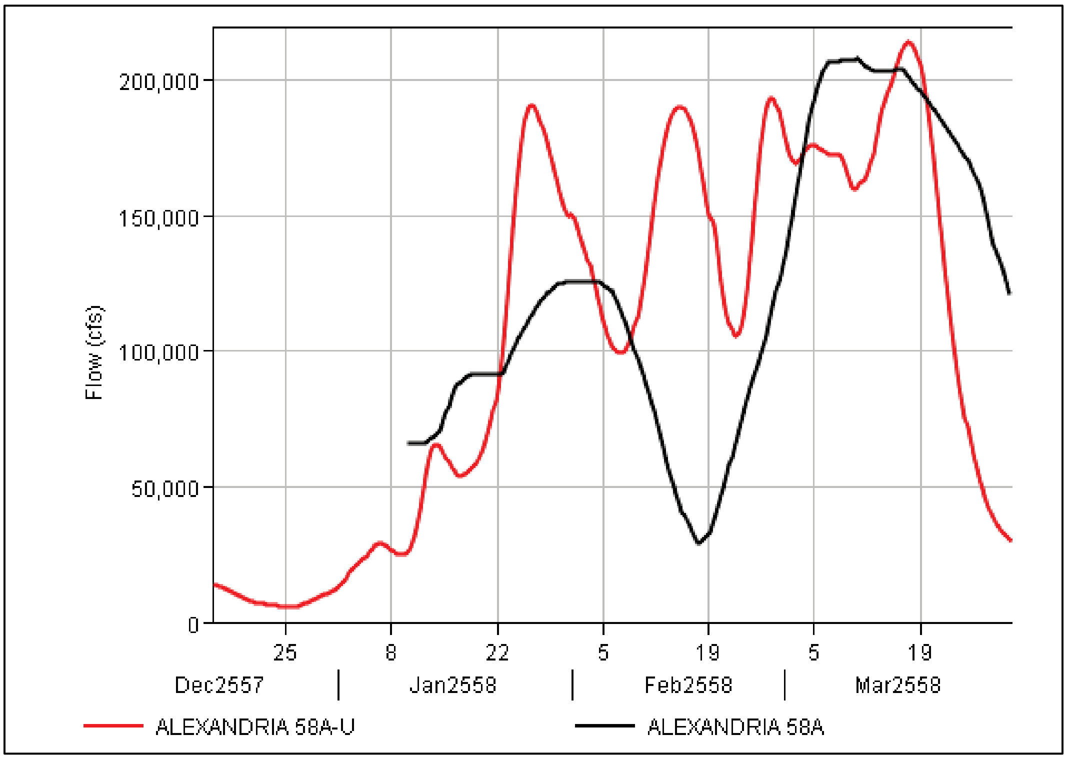


Figure 2-48. Alton, IL, HYPO 58A 1955 unregulated flow compared to 2016 unregulated flow generated by the RFC.

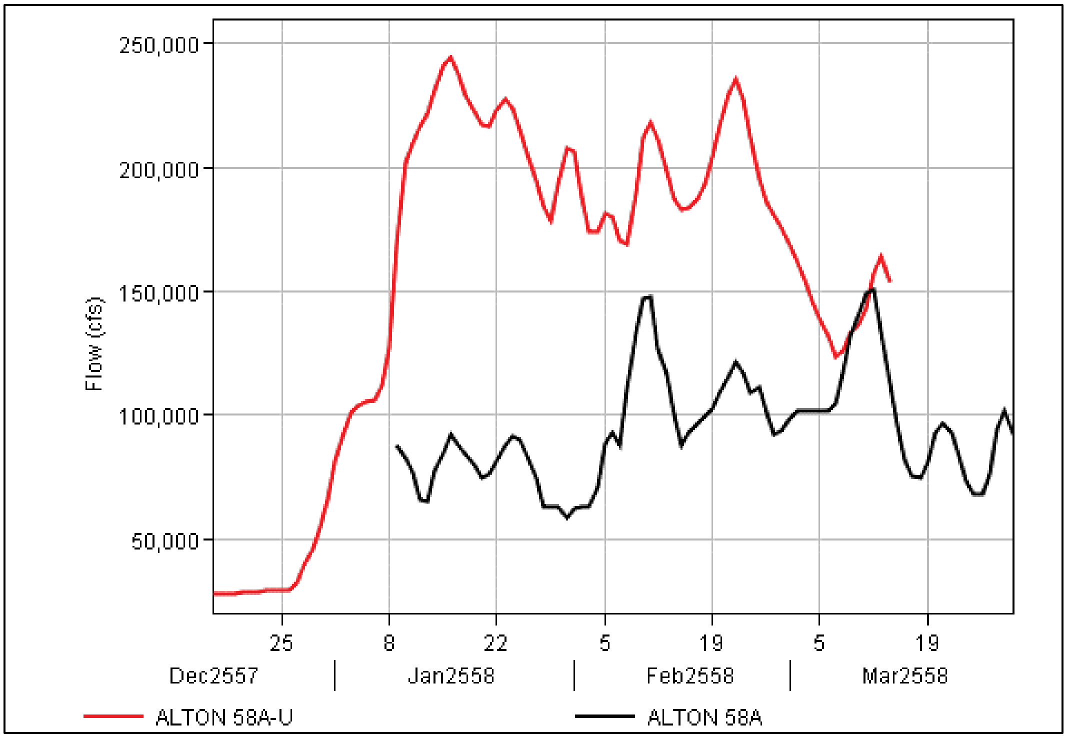


Figure 2-49. Hermann, MO, HYPO 58A 1955 unregulated flow compared to 2016 unregulated flow generated by the RFC.

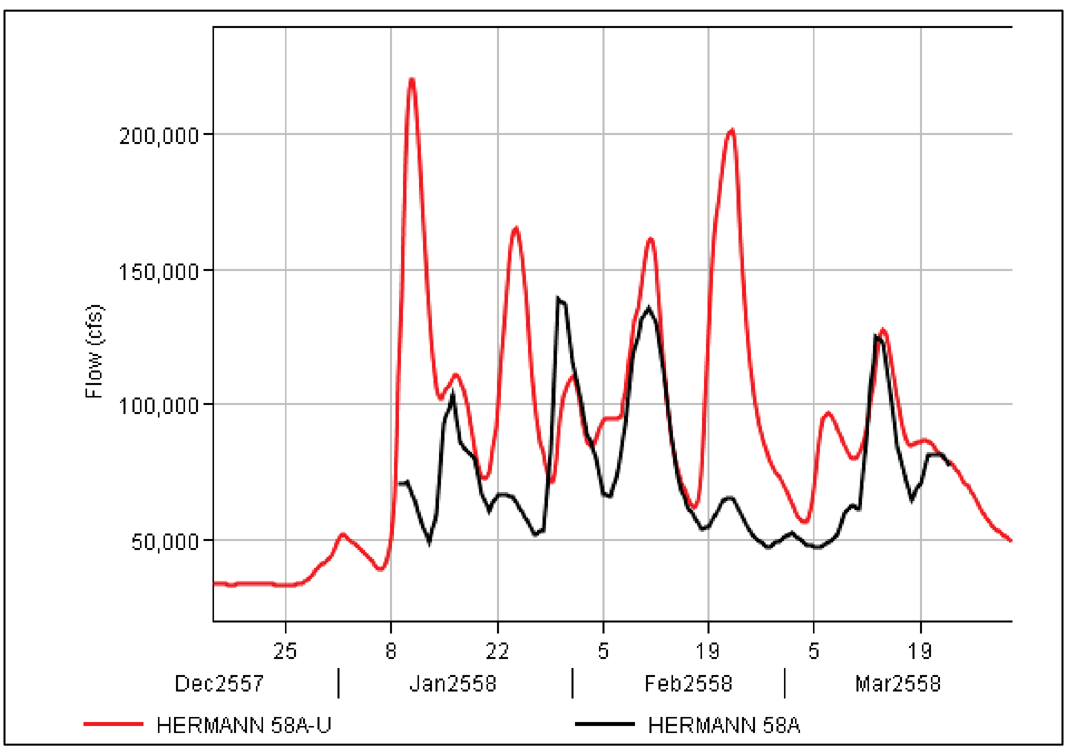


Figure 2-50. Little Rock, AR, HYPO 58A 1955 unregulated flow compared to 2016 unregulated flow generated by the RFC.

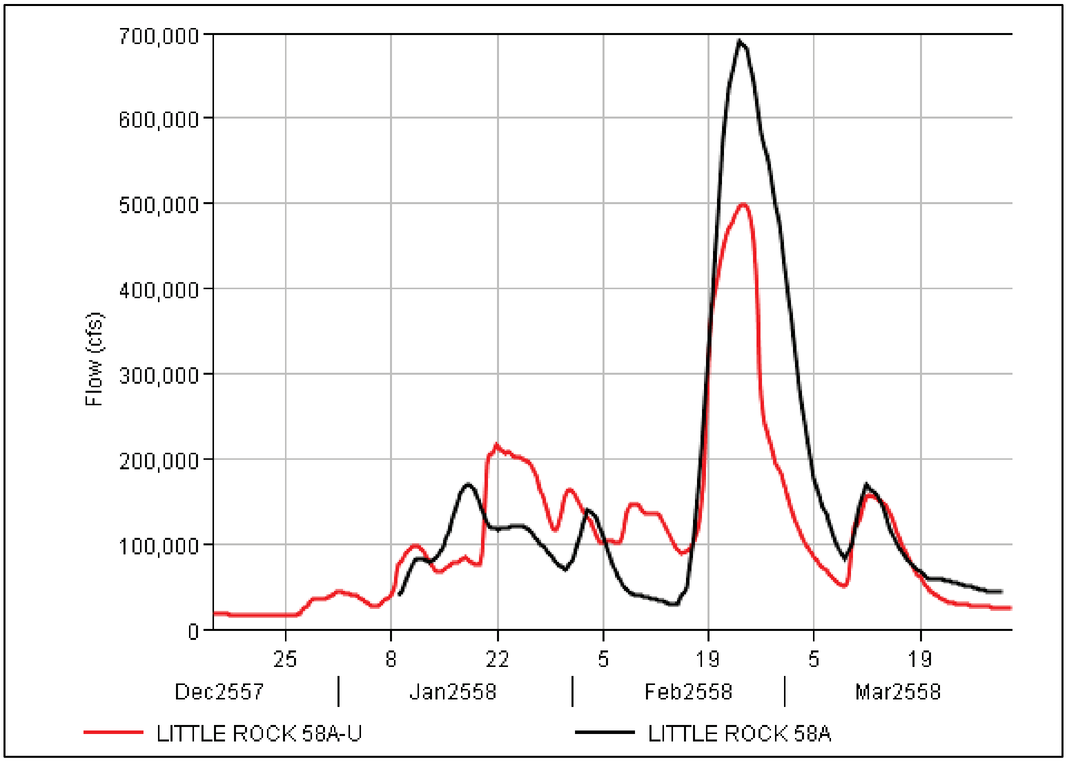
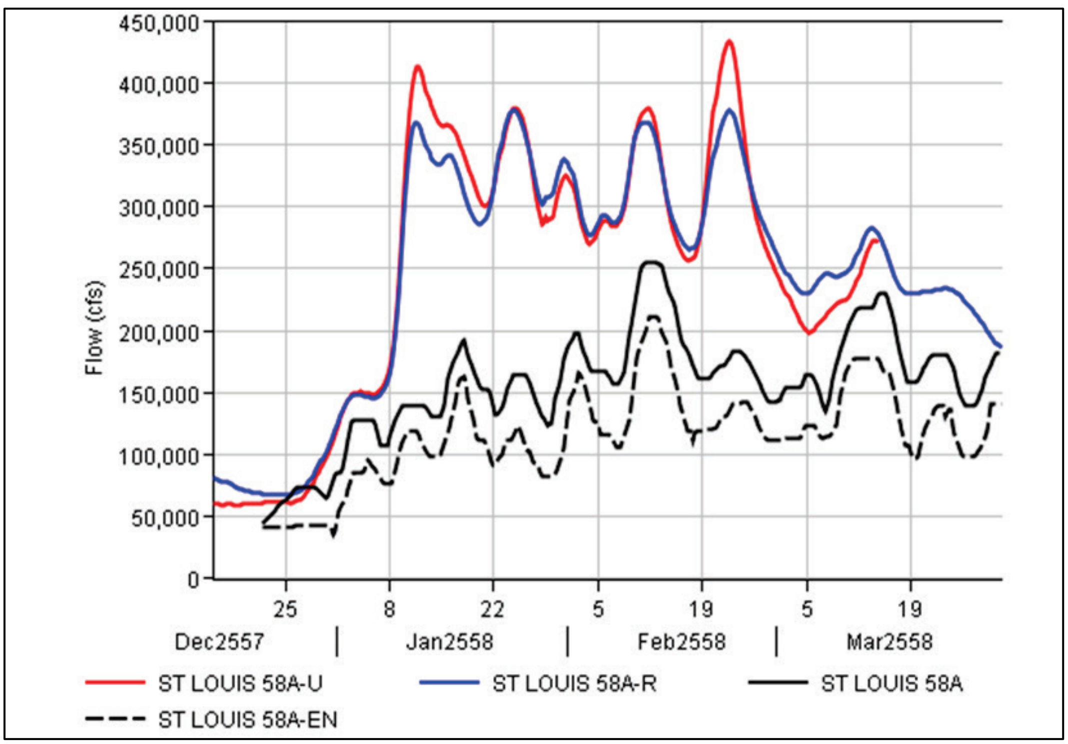


Figure 2-51. St. Louis, MO, HYPO 58A 1955 unregulated and regulated flow compared to 2016 unregulated and regulated flow generated by the RFC.



To compare the magnitude and volume of the hydrographs produced in this assessment, the 1955 hydrographs were digitized and input as the boundary conditions to the 2016 HEC-RAS model. Figure 2-52 through Figure 2-56 show the comparisons between regulated and unregulated flow results. The authorized and existing conditions have nearly the same flow values upstream of Vicksburg, MS. At the latitude of Red River Landing, the authorized and existing condition flow values deviate at the peak. Figure 2-56 shows the entire hydrograph along with a shorter window hydrograph that focuses on the deviation between each condition. The label "Historic 58A-R" represents the 1955 flows used within the current (2016) regulation procedures.

Figure 2-52. Metropolis, IL, HYPO 58A 1955 and 2016 regulated and unregulated flow compared to the 2016 regulated historic HEC-RAS results.

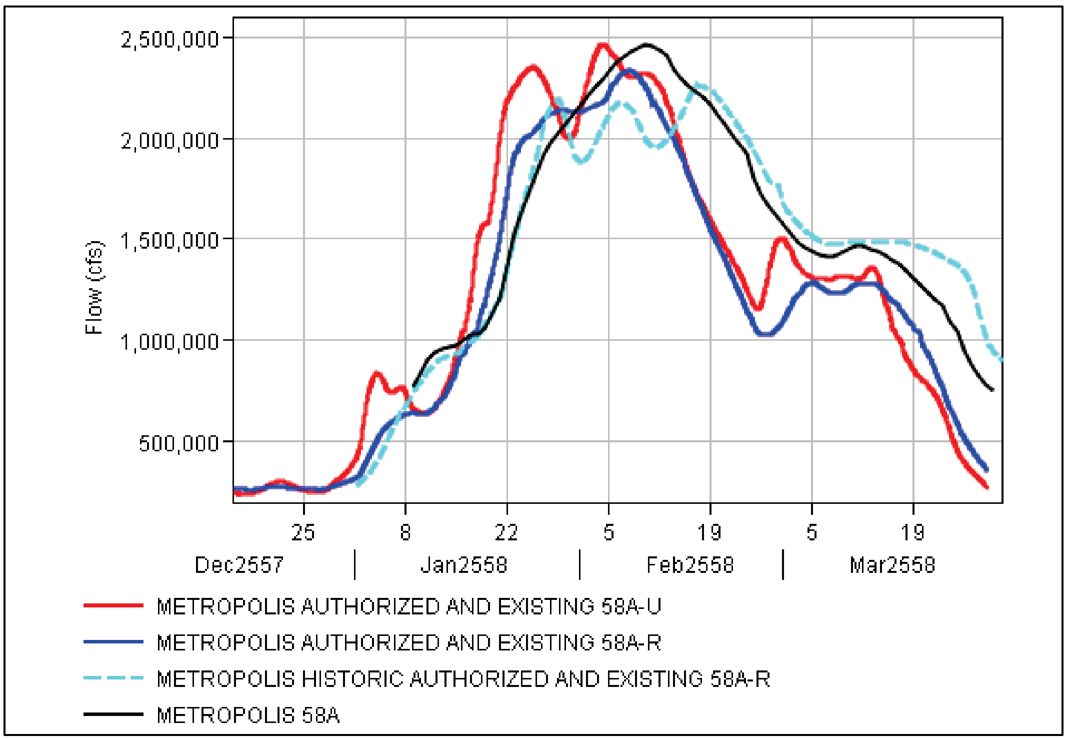


Figure 2-53. Cairo, IL, HYPO 58A 1955 and 2016 regulated and unregulated flow compared to the 2016 regulated historic HEC-RAS results.

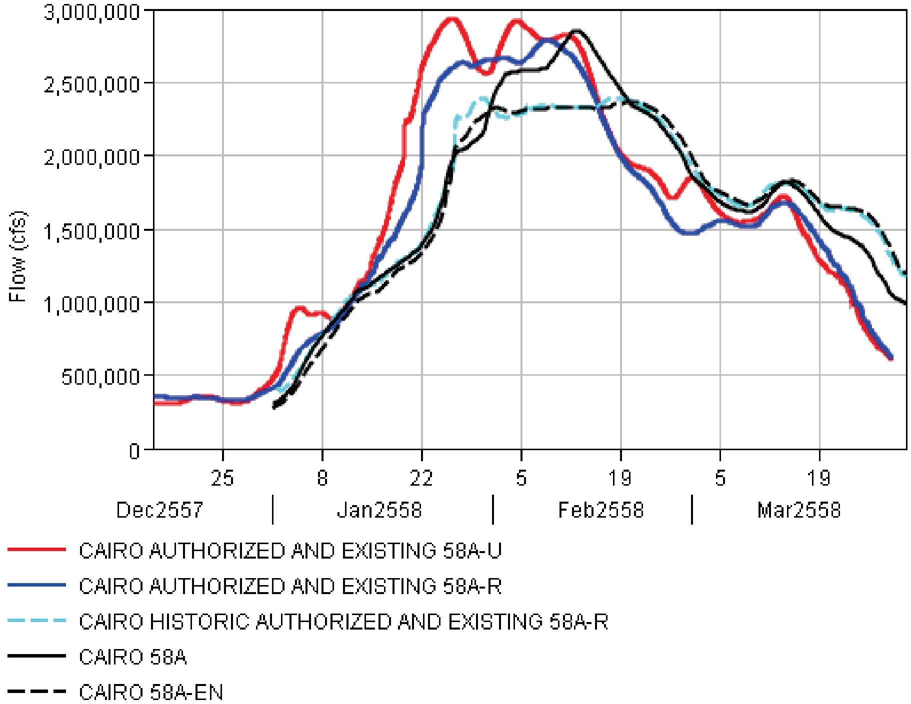
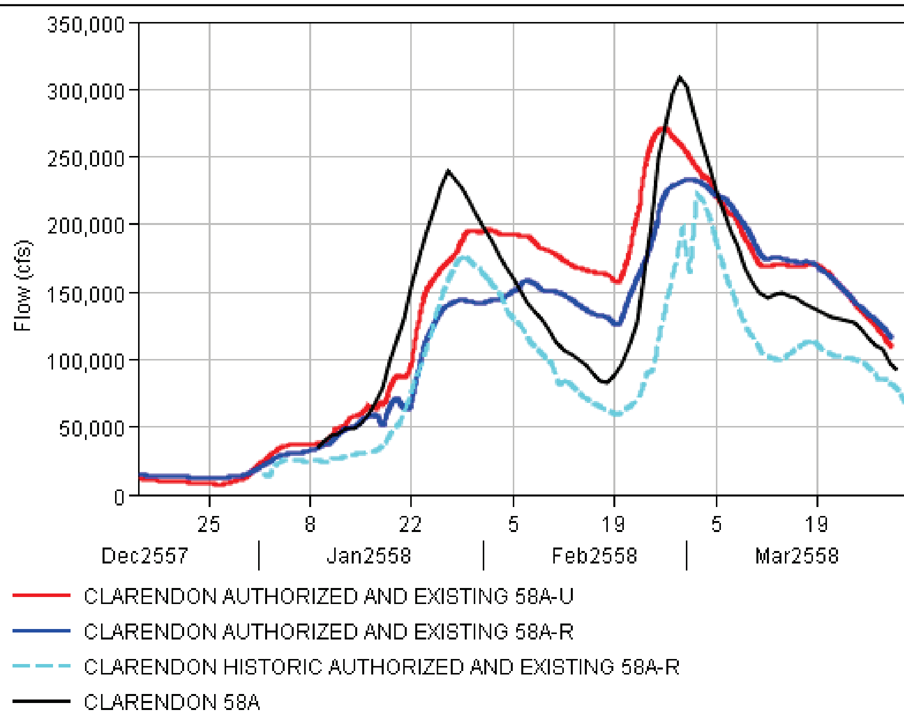
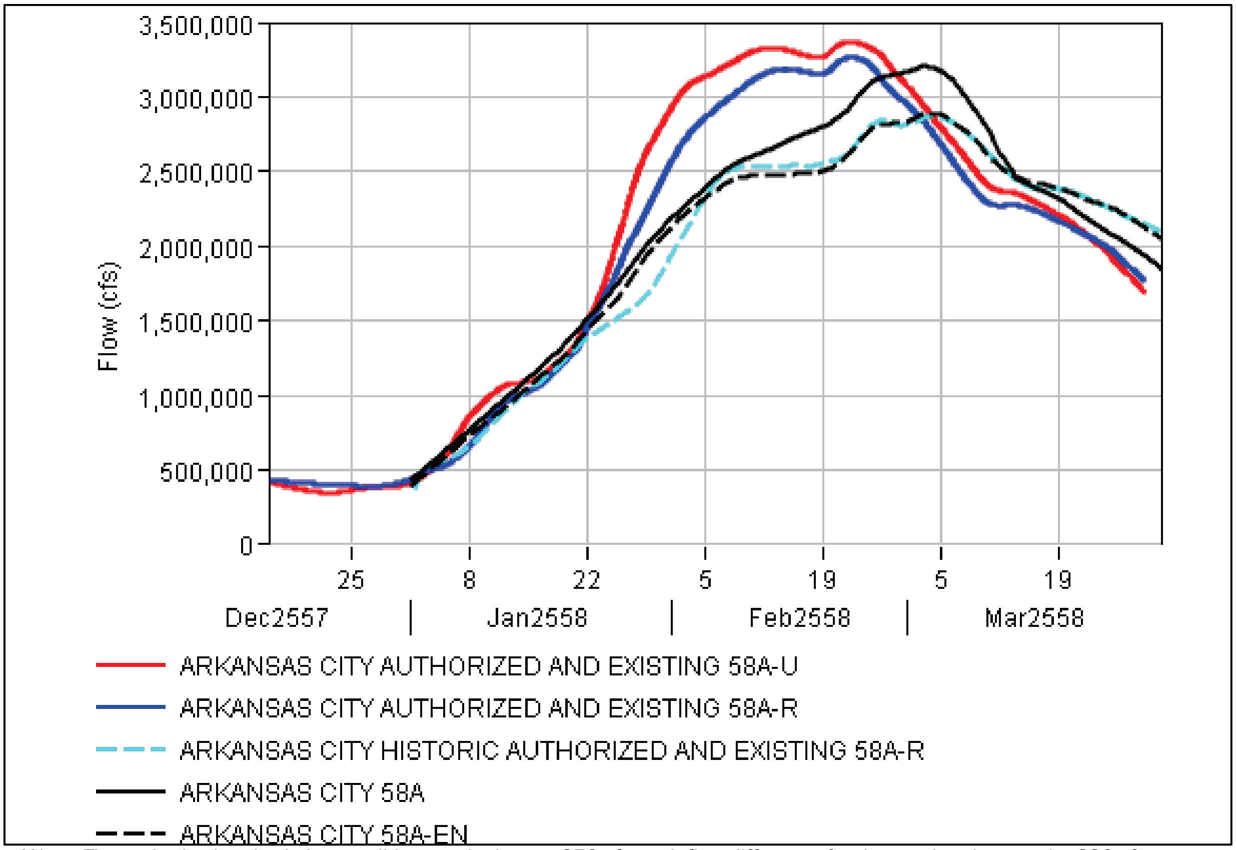


Figure 2-54. Clarendon, AR, HYPO 58A 1955 and 2016 regulated and unregulated flow compared to the 2016 regulated historic HEC-RAS results.*



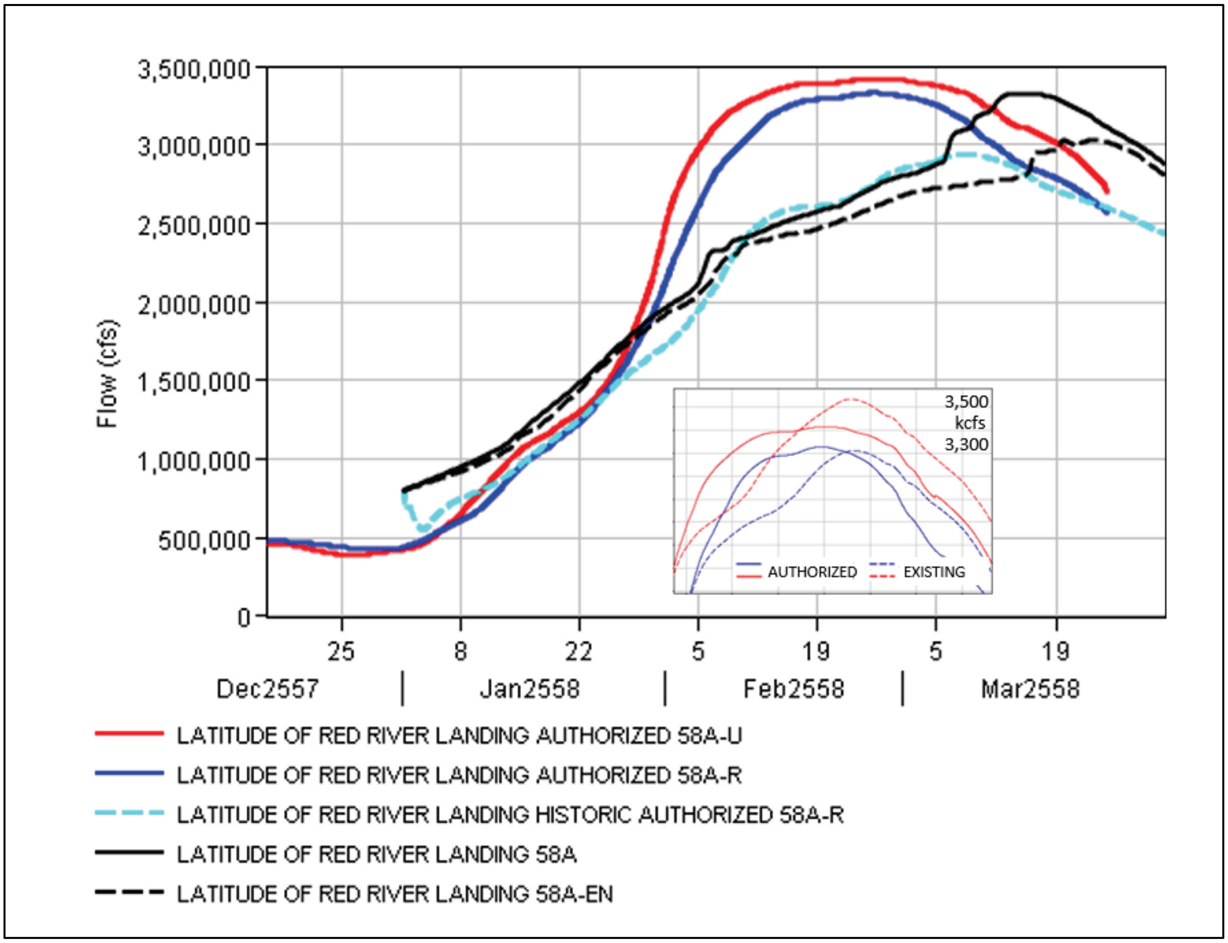
*Note: The authorized and existing condition results have a 60 cfs peak flow difference for the regulated run and a 5 cfs peak flow difference for the unregulated run.

Figure 2-55. Arkansas City, AR, HYPO 58A 1955 and 2016 regulated and unregulated flow compared to the 2016 regulated historic HEC-RAS results.*



*Note: The authorized and existing condition results have a 370 cfs peak flow difference for the regulated run and a 330 cfs peak flow difference for the unregulated run.

Figure 2-56. Latitude of Red River Landing, LA, HYPO 58A 1955 and 2016 regulated and unregulated flow compared to the 2016 regulated historic HEC-RAS results.*



*Note: Figure 2-56 inset shows slight difference around the peak for Authorized and Existing Yazoo Backwater Levee conditions.

2.3.3.1 HYPO 58A HEC-RAS unsteady hydraulic routing

Hydrographs, shown in Figure 2-57 through Figure 2-71, for the Ohio and Lower Mississippi Rivers (points downstream of the Ohio/Mississippi confluence) are direct outputs from the HEC-RAS unsteady calculations. For additional details see the Hydraulics Report (USACE 2018b).

Figure 2-57. HYPO 58A HEC-RAS hydrograph: Ohio River at Cairo, IL.

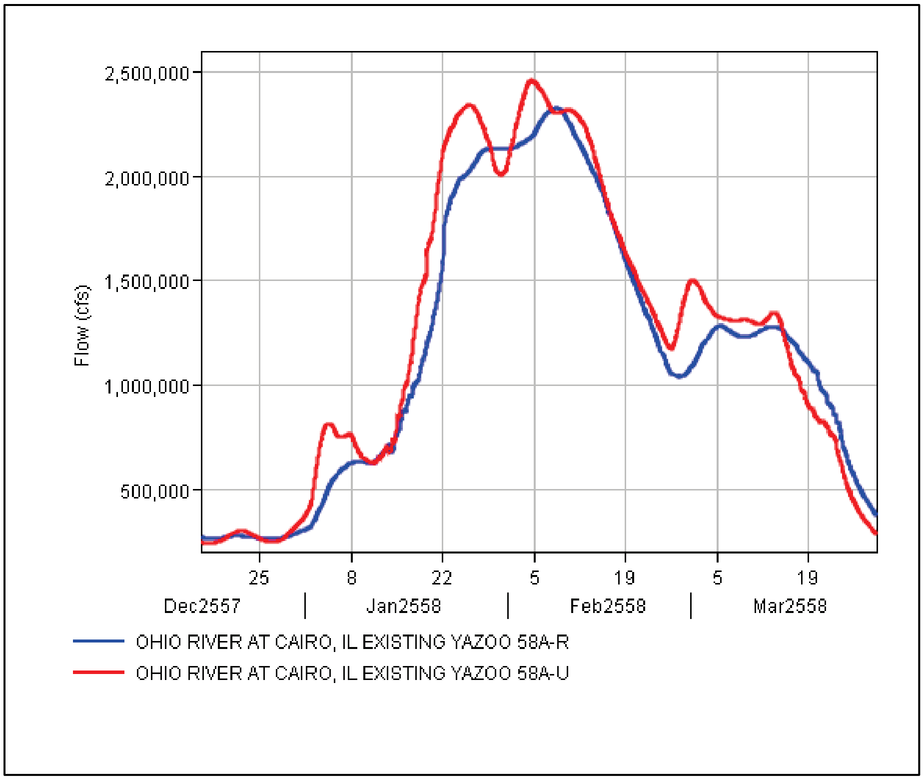


Figure 2-58. HYPO 58A HEC-RAS hydrograph: Combined Ohio and Mississippi River flow near Cairo, IL.

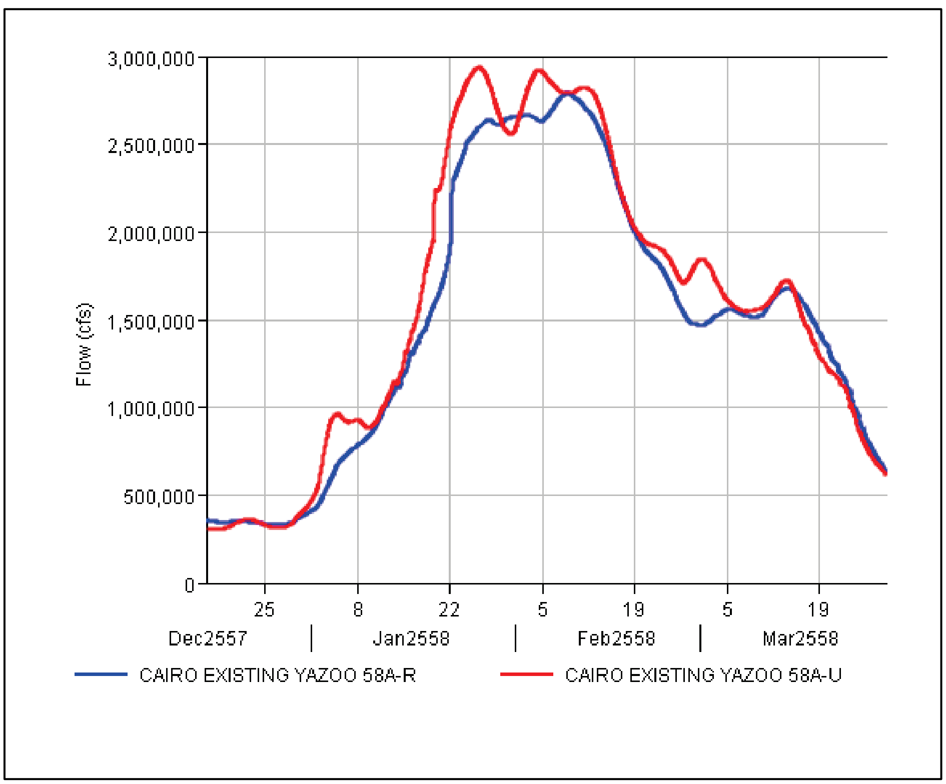
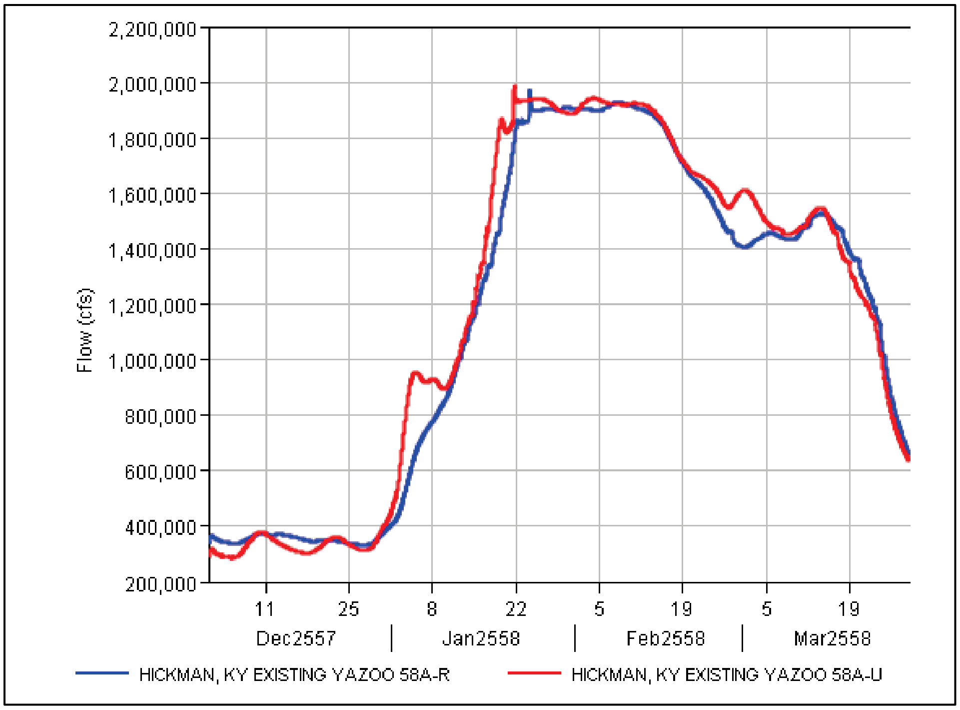


Figure 2-59. HYPO 58A HEC-RAS hydrograph: Mississippi River at Hickman, KY.*



*Note: The peak flow is lower at this location than the upstream and downstream figures, because part of the river flows through the BPNMF.

Figure 2-60. HYPO 58A HEC-RAS hydrograph: Mississippi River at Memphis, TN.

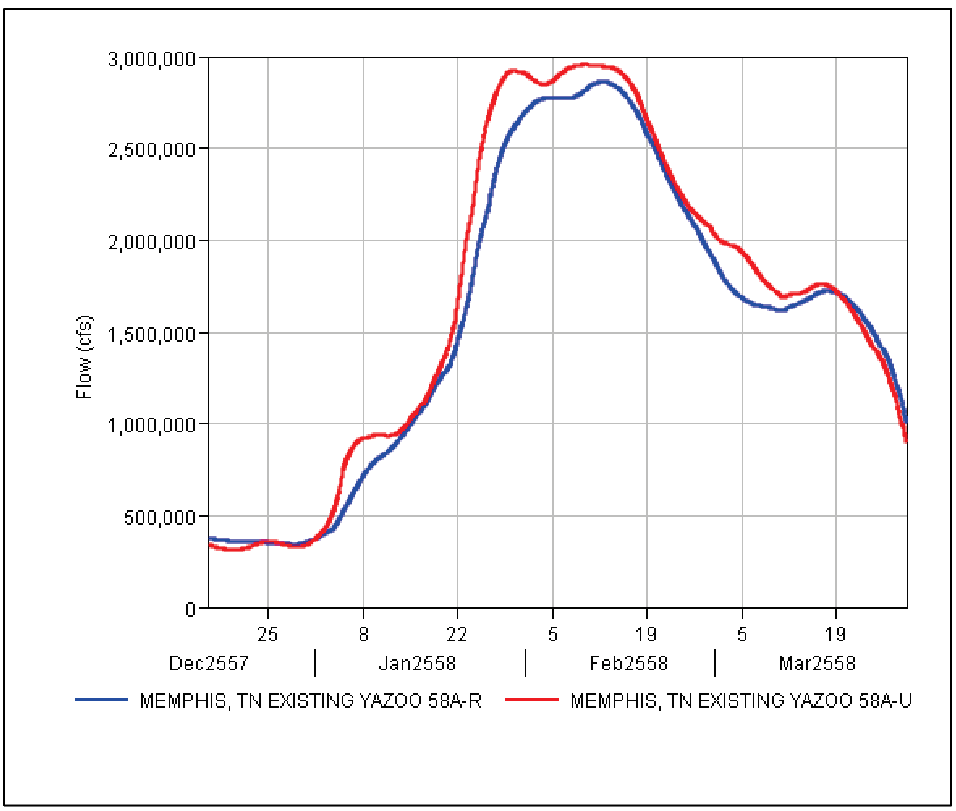


Figure 2-61. HYPO 58A HEC-RAS hydrograph: Mississippi River at Helena, AR.

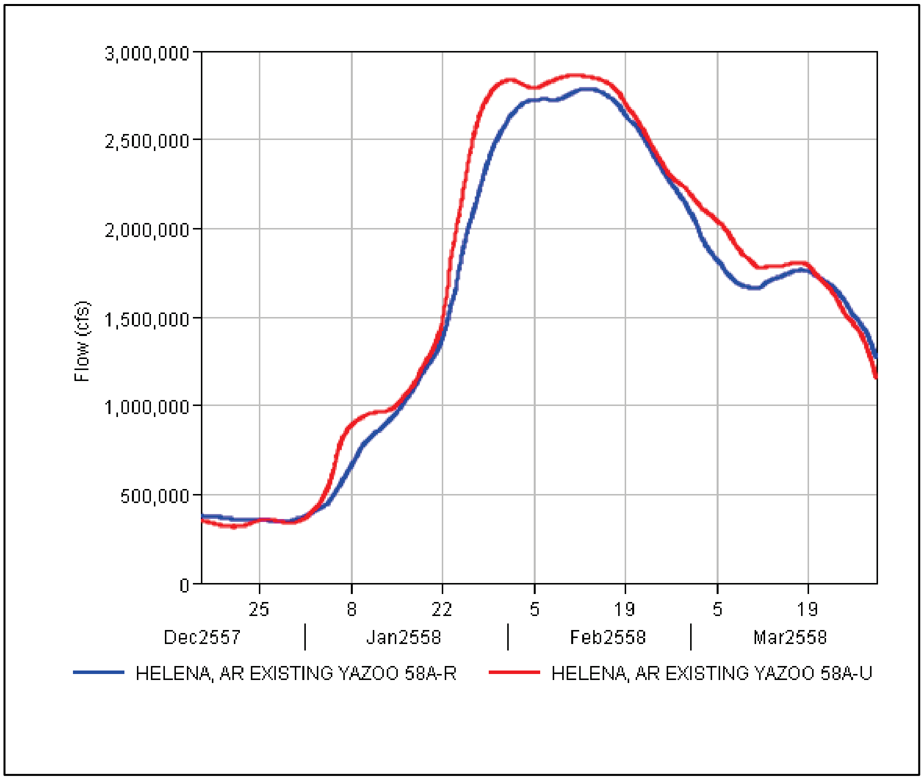


Figure 2-62. HYPO 58A HEC-RAS hydrograph: Mississippi River at Arkansas City, AR.

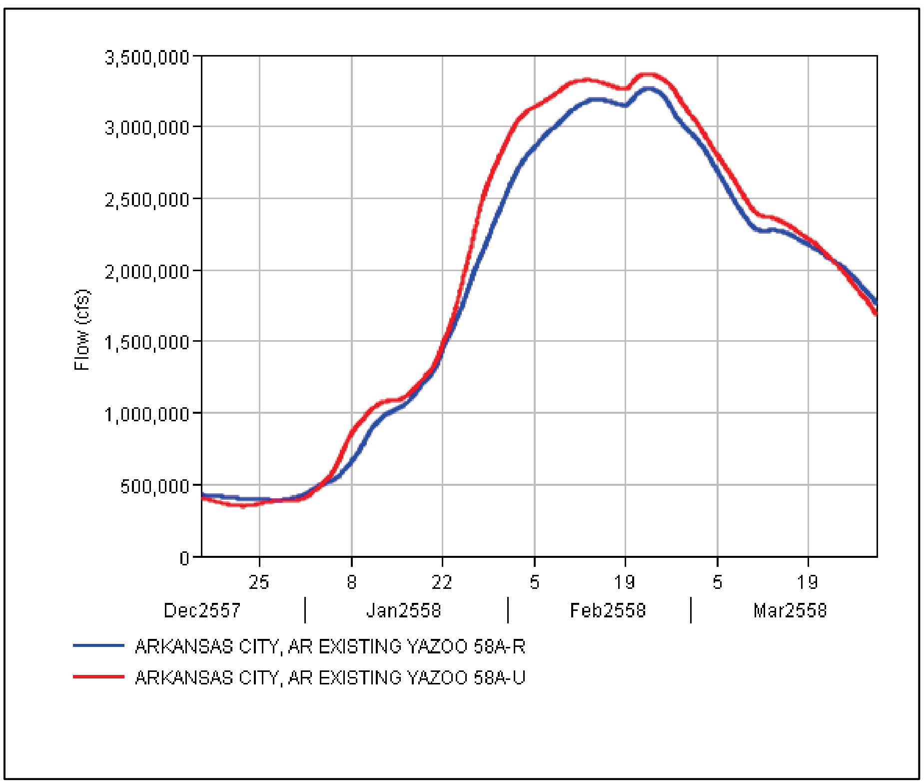


Figure 2-63. HYPO 58A HEC-RAS hydrograph: Mississippi River at Greenville, MS.

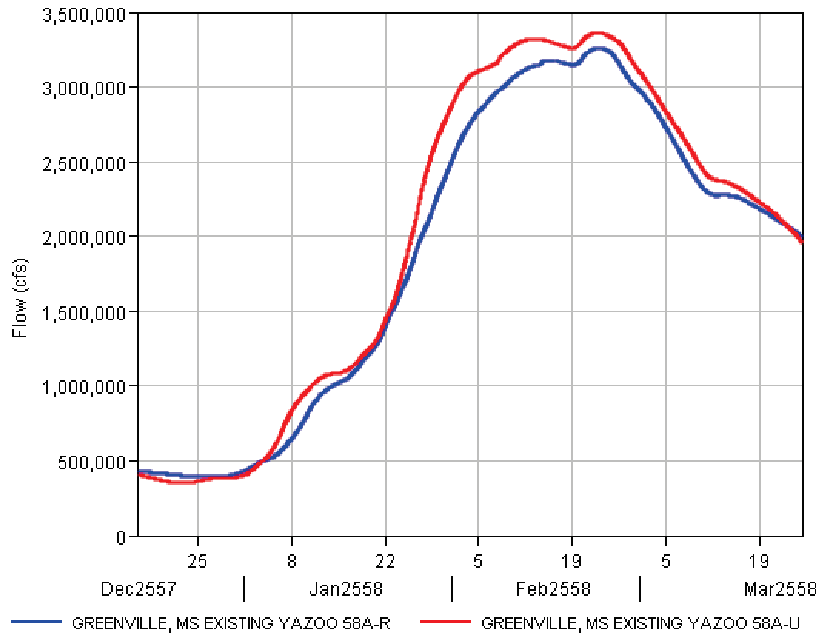


Figure 2-64. HYPO 58A HEC-RAS hydrograph: Mississippi River at Lake Providence, LA.

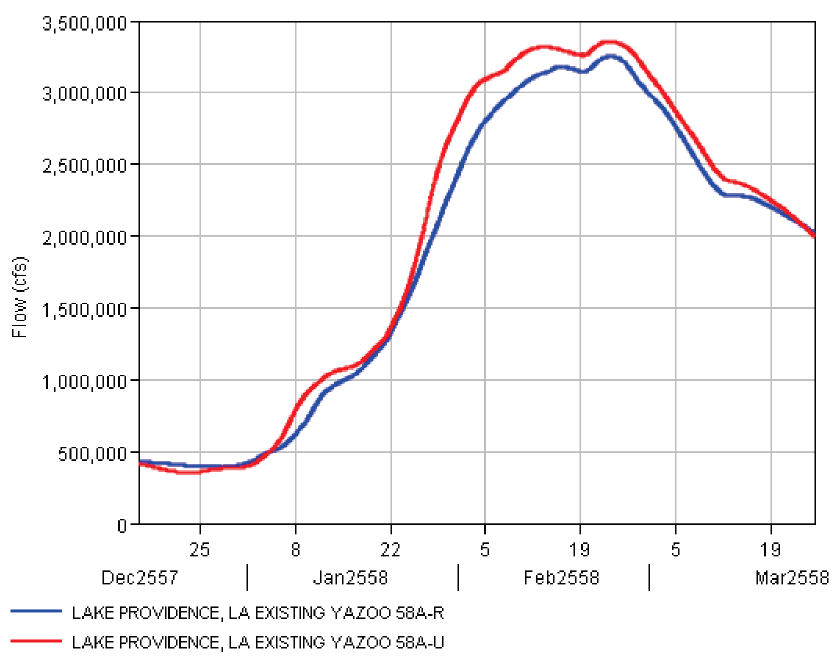


Figure 2-65. HYPO 58A HEC-RAS hydrograph: Mississippi River at Vicksburg, MS.

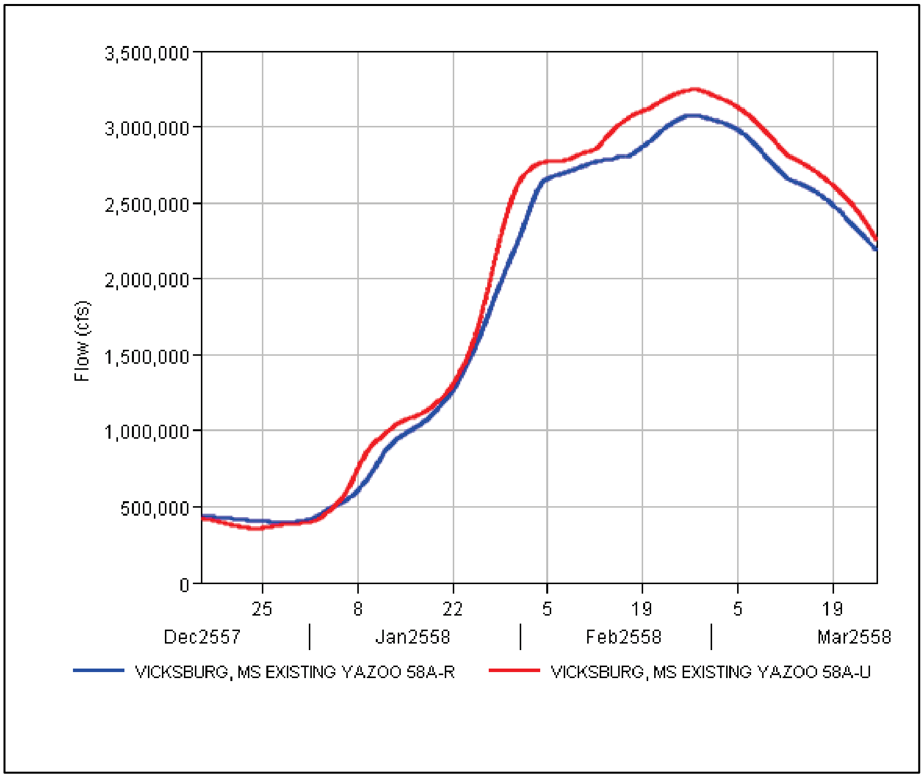


Figure 2-66. HYPO 58A HEC-RAS hydrograph: Mississippi River at Natchez, MS.

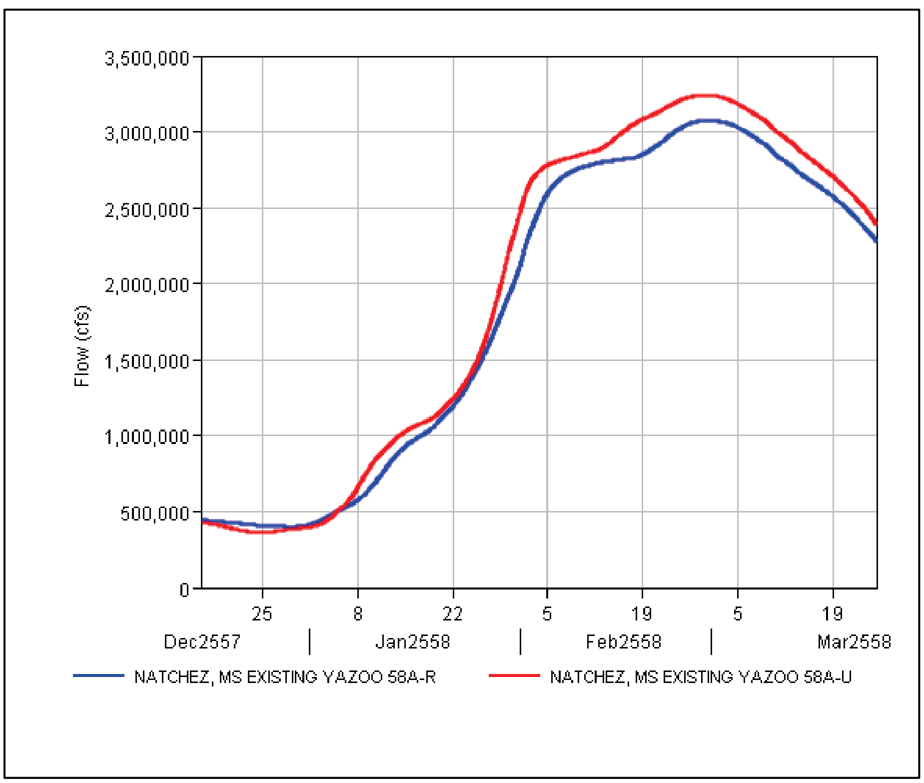


Figure 2-67. HYPO 58A HEC-RAS hydrograph: Mississippi River at Red River Landing, LA.

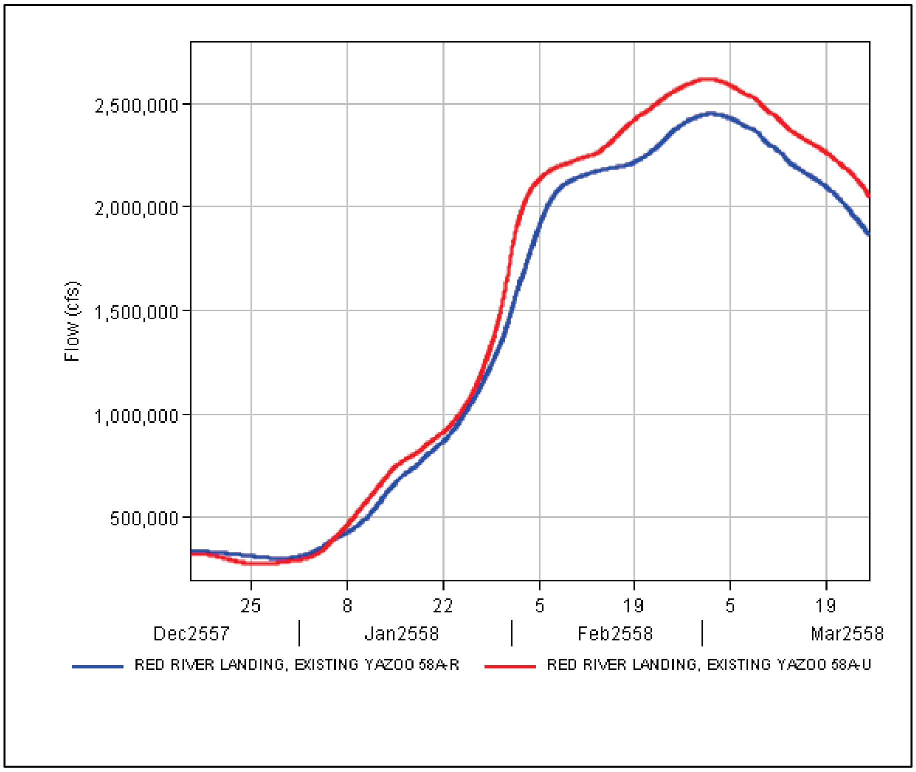


Figure 2-68. HYPO 58A HEC-RAS hydrograph: Mississippi River at Baton Rouge, LA.

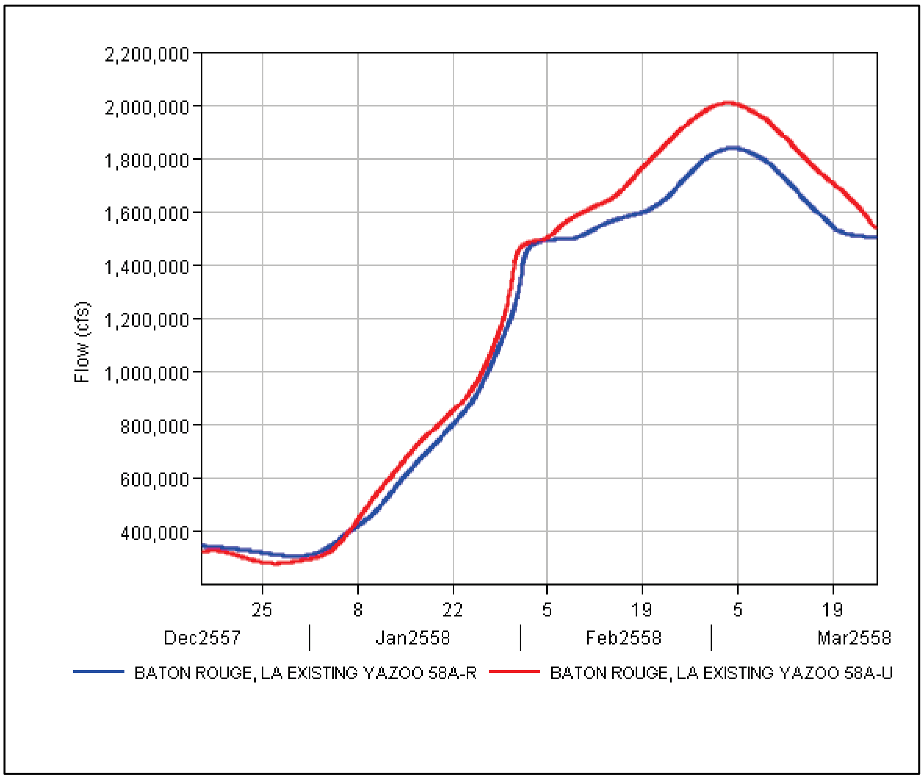


Figure 2-69. HYPO 58A HEC-RAS hydrograph: Mississippi River at Donaldsonville, LA.

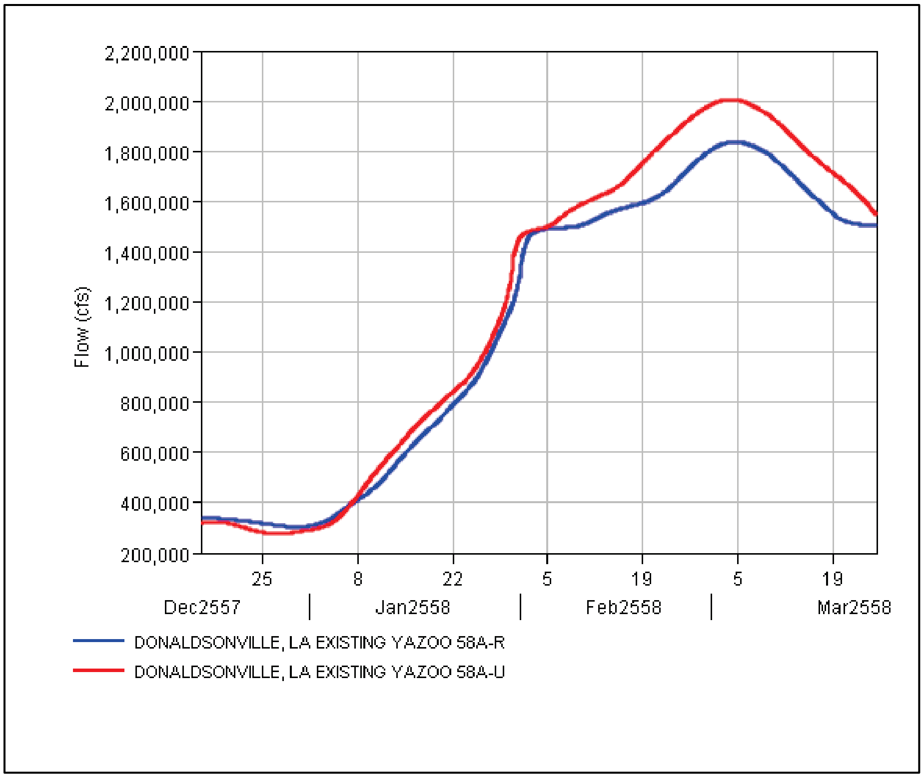


Figure 2-70. Figure 6.1-38 HYPO 58A HEC-RAS hydrograph: Mississippi River at Carrollton, LA.

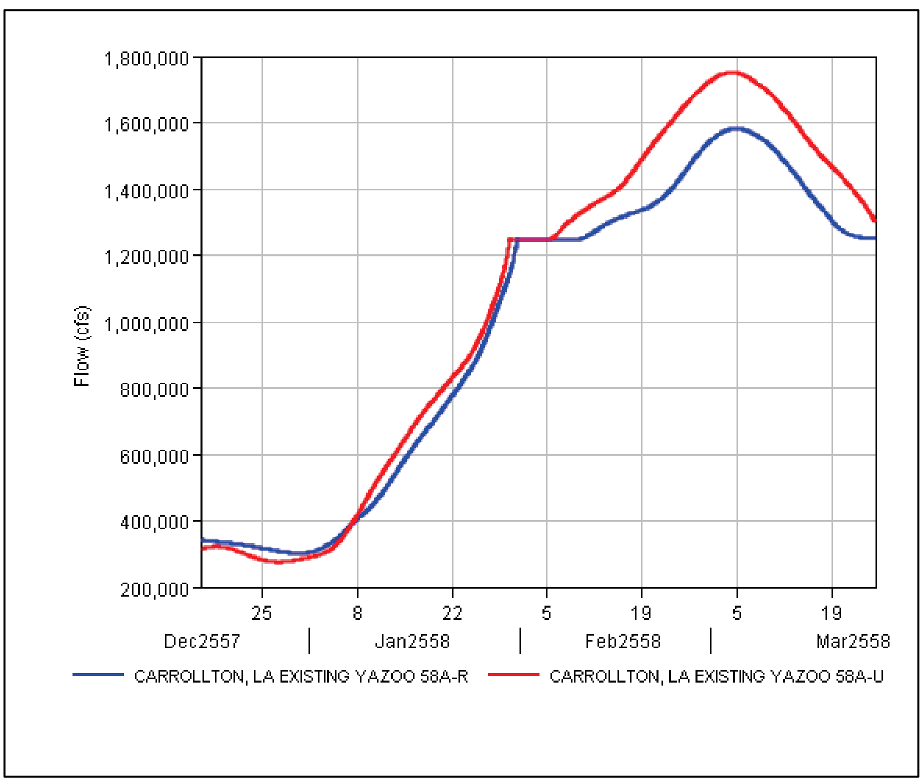
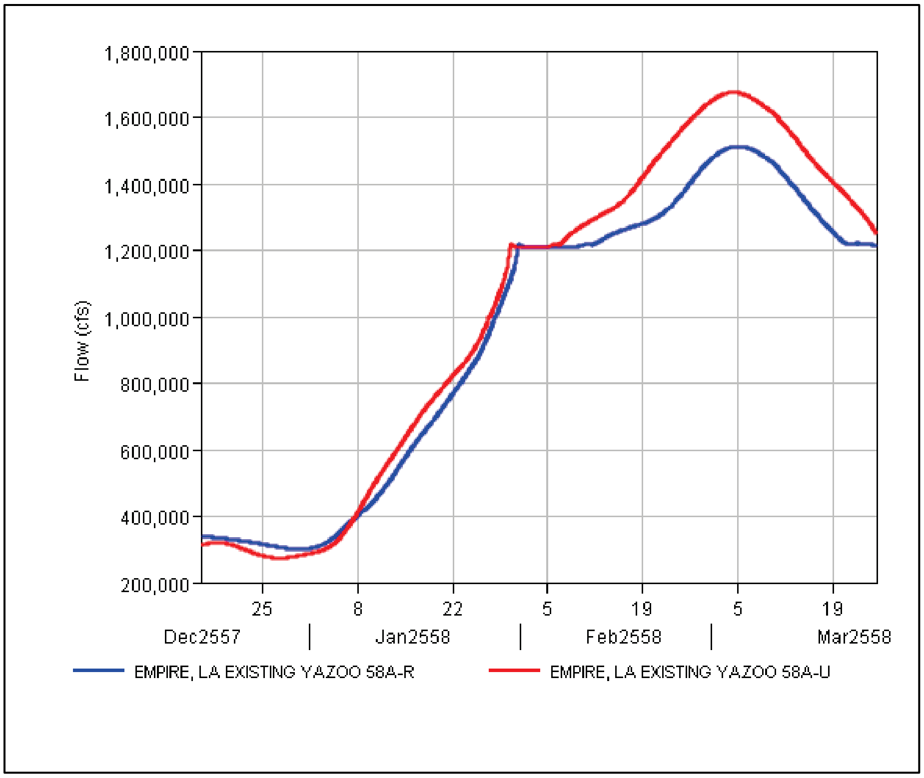


Figure 2-71. HYPO 58A HEC-RAS hydrograph: Mississippi River at Empire, LA.



2.3.4 Summary

The combined NWS, TVA, and USACE hydrologic modeling was an extensive effort. There was good overall agreement between model results for the HYPO 58A results within the focus reach for the MR&T Project. The agreement for HYPO 52A, HYPO 56, and HYPO 63 varied.

2.3.4.1 HYPO storms

HYPO 58A flows provide the design condition for the main-stem Mississippi River from Cairo, IL, to the Gulf of Mexico except with few exceptions as noted in House Document 308 (H.R. Doc. No. 308). The 2016 re-evaluation of hydrologic conditions resulted in minor changes in the unregulated flow hydrographs needed for developing the PDF water surface profile from HEC-RAS. The unregulated combined peak flow calculated for the Mississippi River at the Ohio/Mississippi confluence was within 3% of the previous peak values published in MRC (1955). This close outcome is a significant result given the different methodologies employed.

The other three HYPO storms, 52A, 56, and 63, were included in the analysis to provide a check on the methodology. The unregulated peak flows for HYPO 56 and HYPO 63 were in fair agreement with prior 1955 values. However, computed HYPO 52A unregulated peak values were significantly lower than those from 1955 (see Figure 2-34). Various elements of the analysis were checked in an effort to determine why this one model run produced lower flows. Indications point to differences in precipitation and temperature inputs for HYPO 52A; original data that were used in the 1955 analysis were not available requiring use of NOAA archive data. Multiple checks with NOAA archive data failed to resolve the issue. Further investigation to resolve the issues was not within time and funding constraints of the current assessment.

The effects of reservoir regulation were different with the 2016 reductions being less than indicated by the prior 1955 study (MRC 1955). There is potential to alter reservoir operations to reduce the amount of regulated flow for HYPO 58A from those indicated by 58A-R results (see Section 2.2.10). It is not feasible to achieve reductions based on maximum utilization of flood control storage during real-time operations as applied in the 1955 study. Additional studies using realistic real-time operational considerations should be conducted to determine best operating procedures and their level of reduction for reservoirs during extreme events like HYPO 58A.

2.4 Future recommendations

Hydrologic model uncertainty should be addressed for potential effects on computed inflow hydrographs for the HEC-RAS model. Particularly, infiltration parameters and model sensitivity to parameter values should be evaluated by future model runs. Sensitivity to the effects of climate change could also be evaluated by additional model runs using different rainfall depths, durations, and intensities. Future modeling should assess how sensitive model outputs are to changes in rainfall, infiltration, and temperature.

Uncertainty also exists for the effects of reservoir regulation. Reservoir regulation decisions are dependent upon multiple factors including antecedent conditions, current and forecasted pool elevations, operational constraints, and future estimates of rainfall. Therefore, it is possible to achieve slightly different results for a given event. The model simulations completed to date include one possible regulation outcome. There are also some flexibilities in operational guide curves for each project. Additional model simulations are required to first identify which reservoirs may have a direct and predictable influence on flows at the Ohio/Mississippi River confluence. Additional model simulations should then be undertaken to determine if there are alternate ways to regulate those key projects such that the inflow to the HEC-RAS model can be reduced or altered in such a way to benefit the MR&T system's ability to manage the PDF event.

3 Hydraulics

3.1 Objective

The objective of the hydraulics effort was to use a HEC-RAS unsteady hydraulic model to simulate water surface elevations for historical and new hypothetical PDF events.

3.2 Model validation

The 2011 Mississippi River flood demonstrated the need to re-evaluate the maximum design water surface elevation (flowline) along the mainstem of the Mississippi River from Chester, IL, to Venice, LA. With significant enhancements in numerical modeling tools and additional data availability since the previous flowline study conducted in the 1970s (USACE 1978), and with the 2011 flood providing at or near maximum design water surface elevations at various locations along the Mississippi River, this was an appropriate time to evaluate the Mississippi River flowline to better prepare the nation for future flood events. To this end, USACE developed unsteady flow hydraulic models of the MR&T (Chester, IL/Smithland, KY, to Venice, LA) to evaluate the Mississippi River flowline.

The USACE Districts MVM, MVK, and MVN worked in parallel to create separate HEC-RAS models of their adjoining areas of responsibility. Each district collected additional data and updated one-dimensional (1D) hydraulic models for their respective model reaches within the MR&T domain. The individual district HEC-RAS models were combined through team cooperation, and simulations were performed by the MVK.

3.2.1 Data compilation

The most important hydraulic properties in this application of HEC-RAS were water surface elevation (stage) and cross-sectional discharge (flow). The best, most accurate data reasonably obtainable were used for the model development. Each district developed its own Geographic Information System library for the construction of its models using HEC-GeoRAS 10.2 for ArcGIS 10.2 (USACE 2012). The projection and datum for all of the data related to the terrain were USGS Albers Equal Area and North American Vertical Datum of 1988 (NAVD 88), respectively. However, due to subsidence and SLR in the Southeast Louisiana region, elevations there were referenced to a particular temporal epoch of the particular vertical datum of use. For the southern portions of the MVN segment, the gages were surveyed to the NAVD 88 2004.65 epoch.

The terrain geometry was developed by supplementing hydrographic surveys of channel bathymetries with available lidar or Digital Elevation Model data sets of the overland areas. More detailed descriptions of data sets used within each district's model are included in the Hydraulics Report (USACE 2018b). All data used in the terrain geometry were obtained between 2001 and 2010. Although survey data were used from multiple years, the overall volume for the system has not changed significantly. Weighting the calibration of the model to the recent 2011 event addressed some local differences in geometry.

Observed flow and stage data were obtained from the USGS, USACE, and the NWS. The typical error of the published flow data for calibration was considered to range from +/– 2% to 8%. The USGS published the results of flow measurements taken and assigned an estimated accuracy for each measurement. Multiple gage locations that measure stage also provide flow estimates through the use of stage-discharge rating curves. Rating curves were developed by fitting a curve that relates stage, the typical value measured in near real-time at a gage, to flow through a number of instantaneous flow measurements. Because 2011 included some of the highest water levels in modern times, the historical records of flow measurements at these high stages were limited due to their rarity, and therefore the rating curves used to compute flow from observed stage were subject to significant uncertainty.

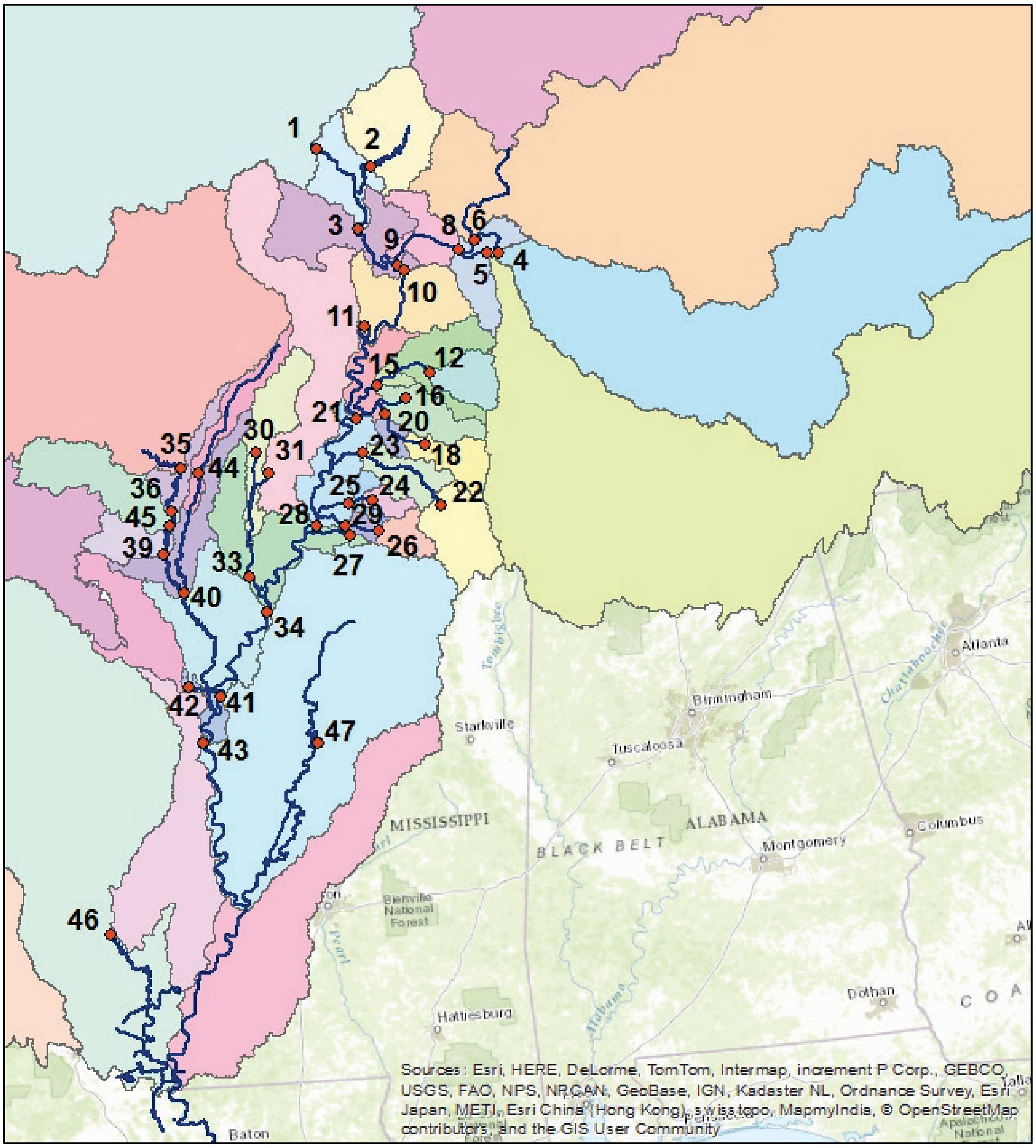
3.2.2 Model setup and calibration

3.2.2.1 Hydrology connections

The upstream boundary conditions of the HEC-RAS model required flow data arriving from the multiple drainage areas within the Mississippi River Basin. For the calibration and validation runs, these upstream boundary condition flows came from either a discharge gage at that specific location or the NWS hydrologic model. Figure 3-1 and Figure 3-2 show the watershed subbasins and points where boundary condition data were required. For more information about the HEC-RAS tributary connections, see the Hydraulics Report (USACE 2018b).

Figure 3-1. Hydrologic connections of the HEC-RAS model.

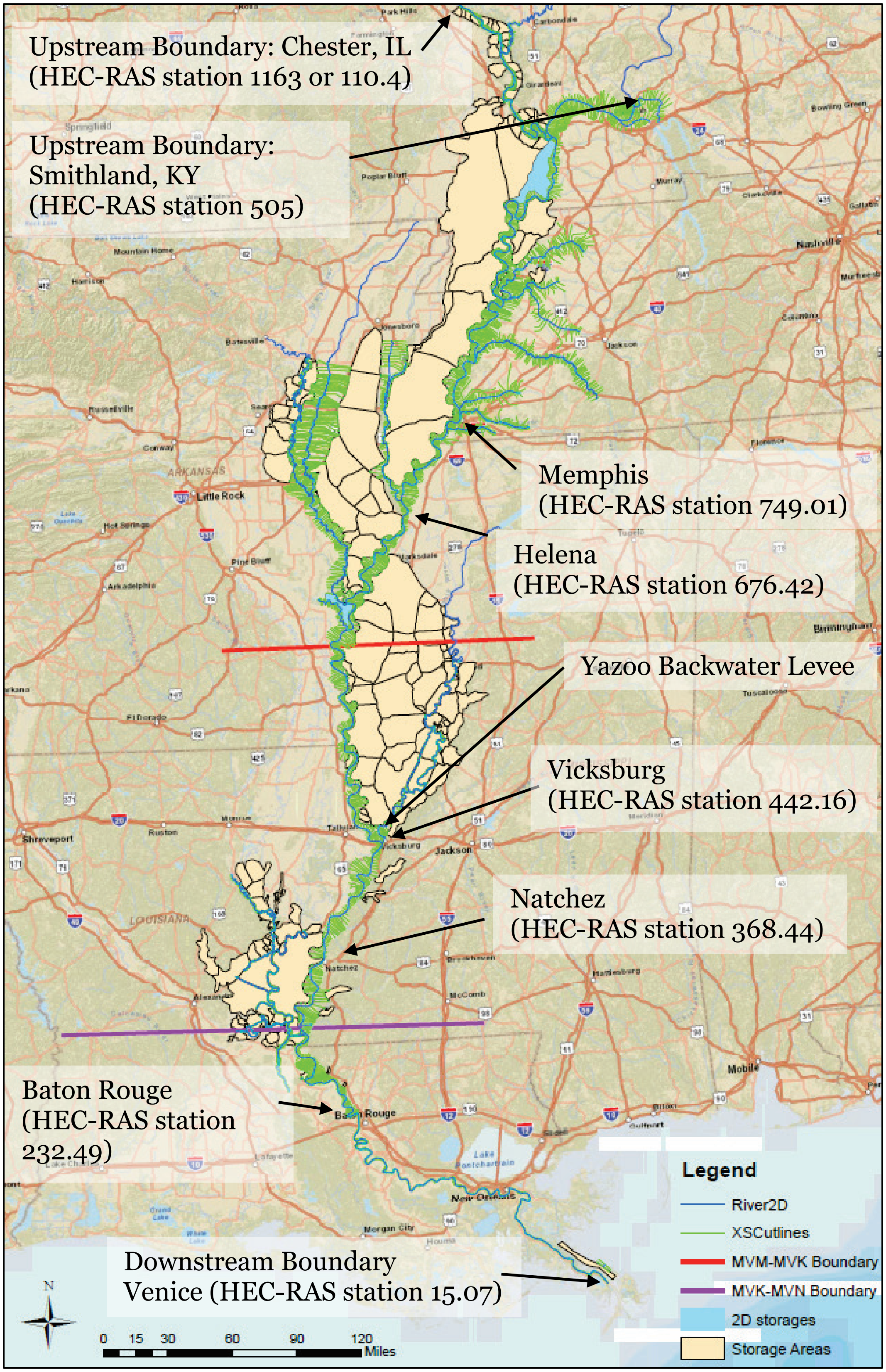
Figure 3-2. Labeled hydrologic connection points for model calibration simulation.



3.2.2.2 Model geometry

Figure 3-3 shows the overall geographic extents of the modeling effort. The individual district HEC-RAS models were developed and calibrated to the 2011 Mississippi River flood, beginning January 1 and continuing through December 31. Adjustments were made to the roughness values (Manning's *n*-values), the ineffective flow areas, and the lateral flow coefficients. The calibration effort aligned the model simulation to daily stage and daily flow records during the 2011 event, with particular attention to matching the timing and magnitude of the peaks. The HEC-RAS models were also run for the 2002 and 2008 events to check the performance of the model, since they were recent events that reached relatively high stages. The river path alignment was recalculated during the development of this model, and the HEC-RAS station labels used throughout this report differ from the previous river alignment labels. The 1962 river mile labels are typically used, so there are references throughout this report that allow for connections between the 1962 river miles and the new HEC-RAS stationing labels. More detail about the model calibration and validation process can be found in the Hydraulics Report (USACE 2018b).

Figure 3-3. MR&T Flowline HEC-RAS model domain.



The final model was developed using HEC-RAS, version 5.0.1. HEC-RAS 5.0 contains a two-dimensional (2D) finite-volume algorithm that solves either the full 2D Saint Venant equations or the 2D diffusive wave equations. The software has the ability to perform computations with 1D river reaches and storage areas combined with 2D flow areas. The combined 1D/2D computations are performed together for each time-step, making the connections from 1D areas to 2D flow areas more accurate than modeling them separately. The grids for the 2D areas are discretized, and an elevation-volume relationship is computed for each cell based on the underlying terrain and bathymetry.

The effort to refine and calibrate the 2D areas was limited, partly because the software was only in a beta version during the time of the initial calibration. The areas modeled in 2D were delineated in such a way to represent overbank flow. The cell size used for BPNMF was 500 ft, and the cell size used for the Arkansas, White, and Mississippi Rivers confluence was 1000 ft. Manning's *n*-values were spatially assigned to the cells using land use datasets for the BPNMF area, and a Manning's *n*-value of 0.15 was used for the Arkansas, White, and Mississippi Rivers confluence area, since it is a densely wooded area.

3.2.3 Model combination

The process of combining the three models was accomplished using a two-phase method, allowing the modeler to look at results of the individual district models and combined model in more detail. First, the two models (MVM and MVK) were combined with only the Mississippi River and attached tributaries. This allowed a faster turnaround in the running time and check on the model results. The Mississippi River flow and stage values were evaluated. The first changes on the MVK model prior to combining were to reanalyze the channel *n*-values, extend cross sections as needed, and make changes in the seasonal roughness values. Once this was accomplished, the entire MVM and MVK models were combined. Minor modifications were then made in the reach near the intersection of the two models and rerun to ensure the models were operating as one.

The MVK model was then joined with the MVN model in a similar manner. This required some changes in the lower reach of the MVK model and the upper reach of the MVN model as well as near the Morganza Floodway and the Bonnet Carré Spillway. Changes at the Morganza Floodway and Bonnet Carré Spillway were coordinated between Vicksburg and New Orleans to ensure correct operation. For more detailed information about the model combining process, see the Hydraulics Report (USACE 2018b).

The HEC-RAS models were then joined and run as a complete unit. Computed stage and flow hydrographs of the combined MVM, MVK, and MVN models were then checked with the individual and paired models to ensure consistency among the three phases of joining the models. The combined model has a combination of 1D and 2D areas and was completed using HEC-RAS 5.0.1. The simulation time to run the 2011 full-year model is approximately 4 hours on a standard desktop computer.

3.2.4 List of model simulations

Table 3-1 lists the HEC-RAS plans contained within the Mississippi River HEC-RAS model, provides a brief description of each, and lists the various sections in the report that describe the results of each plan.

Table 3-1. List of simulations of the HEC-RAS model described in this report.

HEC-RAS Plan Name	Description	Section of This Report
2011_MVM_MVK_MVN	2011 conditions were simulated for calibrating the combined HEC-RAS model.	Section 3.2
2008_MVM_MVK_MVN	2008 conditions were simulated to validate the combined HEC-RAS model.	Section 3.2
2002_MVM_MVK_MVN	2002 conditions were simulated to validate the combined HEC-RAS model.	Section 3.2
1955 Historic 58 AEN Existing Yazoo_9-28	Flows for the 58A-EN event from the 1955 Hydrology Report were run with the Yazoo Backwater Levee at the existing height.	Section 3.3
1955 Historic 58AEN Authorized Yazoo	Flows for the 58A-EN event from the 1955 Hydrology Report were run with the Yazoo Backwater Levee at the authorized height.	Section 3.3
2016_58A-R_Existing_Yazoo	Flows for the 58A-R (regulated) event from the new hydrology were run with the existing Yazoo Backwater Levee height.	Section 3.4
2016_58A-U_Existing_Yazoo	Flows for the 58A-U (unregulated) event from the new hydrology were run with the existing Yazoo Backwater Levee height.	Section 3.4
2016_58A-R_Authorized_Yazoo	Flows for the 58A-R event from the new hydrology were run with the authorized Yazoo Backwater Levee height.	Section 3.4
2016_58A-U_Authorized_Yazoo	Flows for the 58A-U event from the new hydrology were run with the authorized Yazoo Backwater Levee height.	Section 3.4

HEC-RAS Plan Name	Description	Section of This Report
2016_58A-R_Existing_YZ_SLR Adjustment	Flows for the 58A-R event from the new hydrology were run with the existing Yazoo Backwater Levee height and the future downstream boundary condition for SLR.	Section 3.5
2016_58A-R_Authorized_YZ_SLR Adjustment	Flows for the 58A-R event from the new hydrology were run with the authorized Yazoo Backwater Levee height and the future downstream boundary condition for SLR.	Section 3.5
2016 52A-R Existing Yazoo	Flows for the 52A-R event from the new hydrology were run with the existing Yazoo Backwater Levee height.	Section 3.4
2016 52A-U Existing Yazoo	Flows for the 52A-U event from the new hydrology were run with the existing Yazoo Backwater Levee height.	Section 3.4
2016 52A-R Authorized Yazoo	Flows for the 52A-R event from the new hydrology were run with the authorized Yazoo Backwater Levee height.	Section 3.4
2016 52A-U Authorized Yazoo	Flows for the 52A-U event from the new hydrology were run with the authorized Yazoo Backwater Levee height.	Section 3.4
2016 56-R Existing Yazoo	Flows for the 56-R event from the new hydrology were run with the existing Yazoo Backwater Levee height.	Section 3.4
2016 56-U Existing Yazoo	Flows for the 56-U event from the new hydrology were run with the existing Yazoo Backwater Levee height.	Section 3.4
2016 56-R Authorized Yazoo	Flows for the 56-R event from the new hydrology were run with the authorized Yazoo Backwater Levee height.	Section 3.4
2016 56-U Authorized Yazoo	Flows for the 56-U event from the new hydrology were run with the authorized Yazoo Backwater Levee height.	Section 3.4
2016 63-R Existing Yazoo	Flows for the 63-R event from the new hydrology were run with the existing Yazoo Backwater Levee height.	Section 3.4
2016 63-U Existing Yazoo	Flows for the 63-U event from the new hydrology were run with the existing Yazoo Backwater Levee height.	Section 3.4
2016 63-R Authorized Yazoo	Flows for the 63-R event from the new hydrology were run with the authorized Yazoo Backwater Levee height.	Section 3.4
2016 63-U Authorized Yazoo	Flows for the 63-U event from the new hydrology were run with the authorized Yazoo Backwater Levee height.	Section 3.4

3.2.5 Results

The results of the calibration event of 2011 as well as the 2002 and 2008 validation events for the combined model are compared with the observational data. Of the many possible comparisons that can be made, a few important locations along the mainstem of the Mississippi River have been chosen to present visual comparisons for this report. Stage comparisons for Helena, AR; Natchez, MS; and Baton Rouge, LA; as well as flow for Baton Rouge, are shown for the 2011 calibration event in Figure 3-4 through Figure 3-7. The same comparisons are made for the 2002 and 2008 events to check the performance of the combined model. 2002 comparisons are shown in Figure 3-8 through Figure 3-11, and the

2008 comparisons are shown in Figure 3-12 through Figure 3-15. More detailed results from individual district models can be found in the appendices of the Hydraulics Report (USACE 2018b).

Figure 3-4. 2011 simulated and observed stages for Helena, AR (HEC-RAS station 676.42).

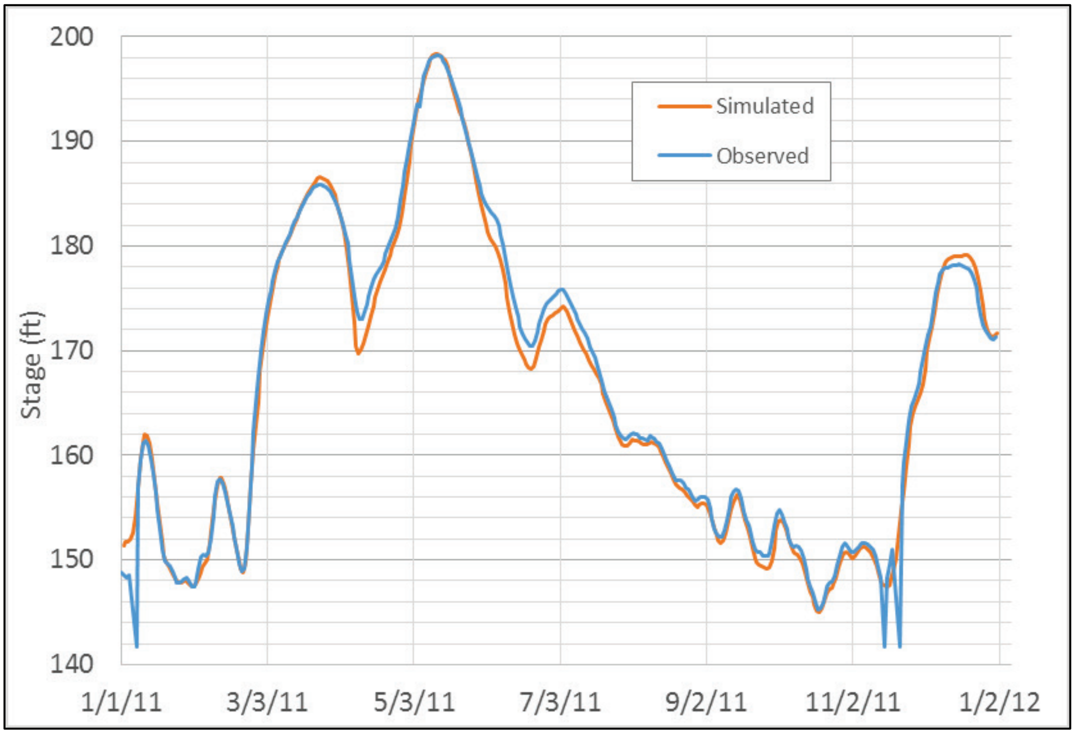


Figure 3-5. 2011 simulated and observed stages for Natchez, MS (HEC-RAS station 368.44).

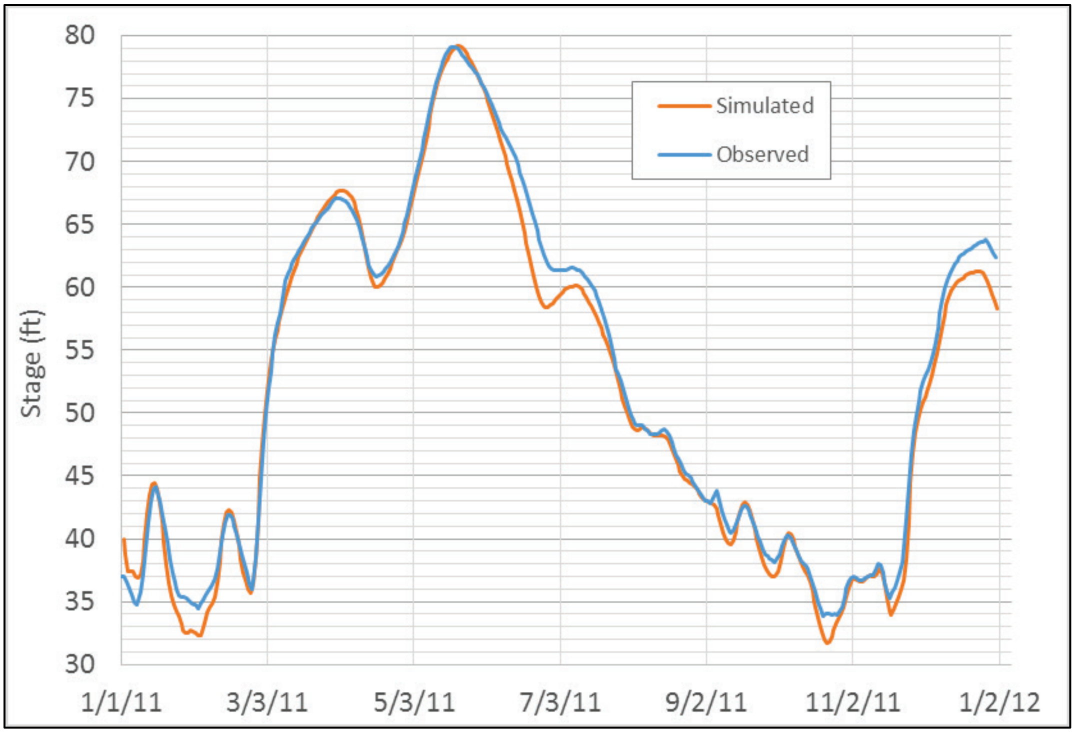


Figure 3-6. 2011 simulated and observed stages for Baton Rouge, LA (HEC-RAS station 232.49).

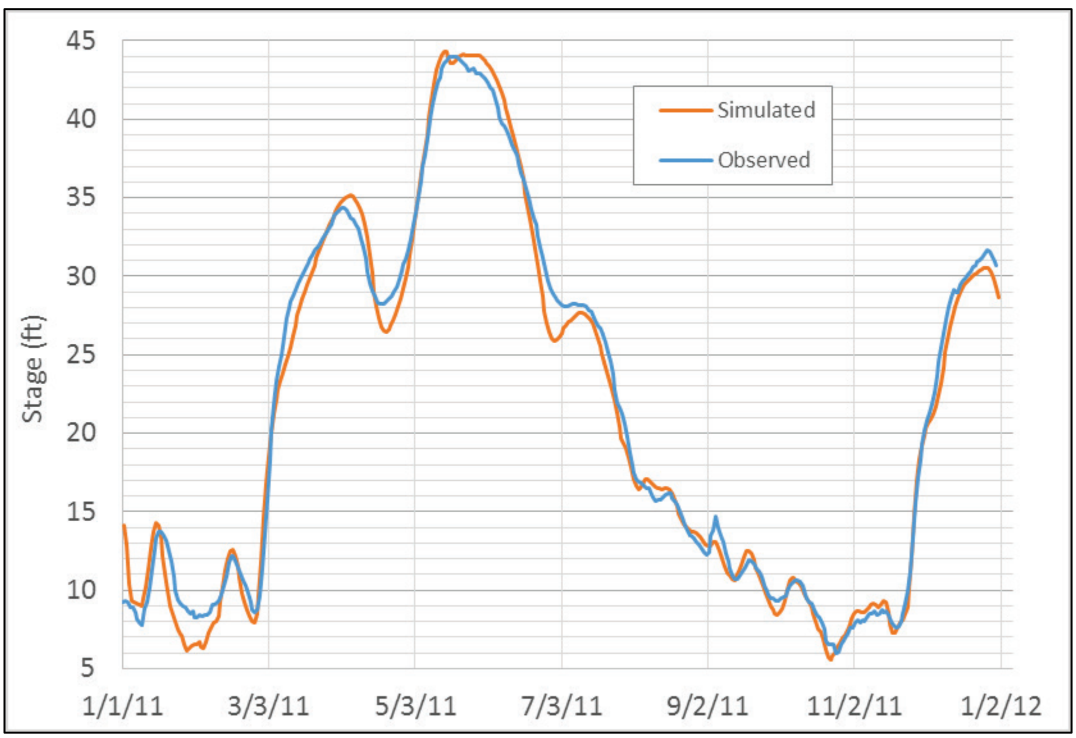


Figure 3-7. 2011 simulated and observed flows for Baton Rouge, LA (HEC-RAS station 232.49).

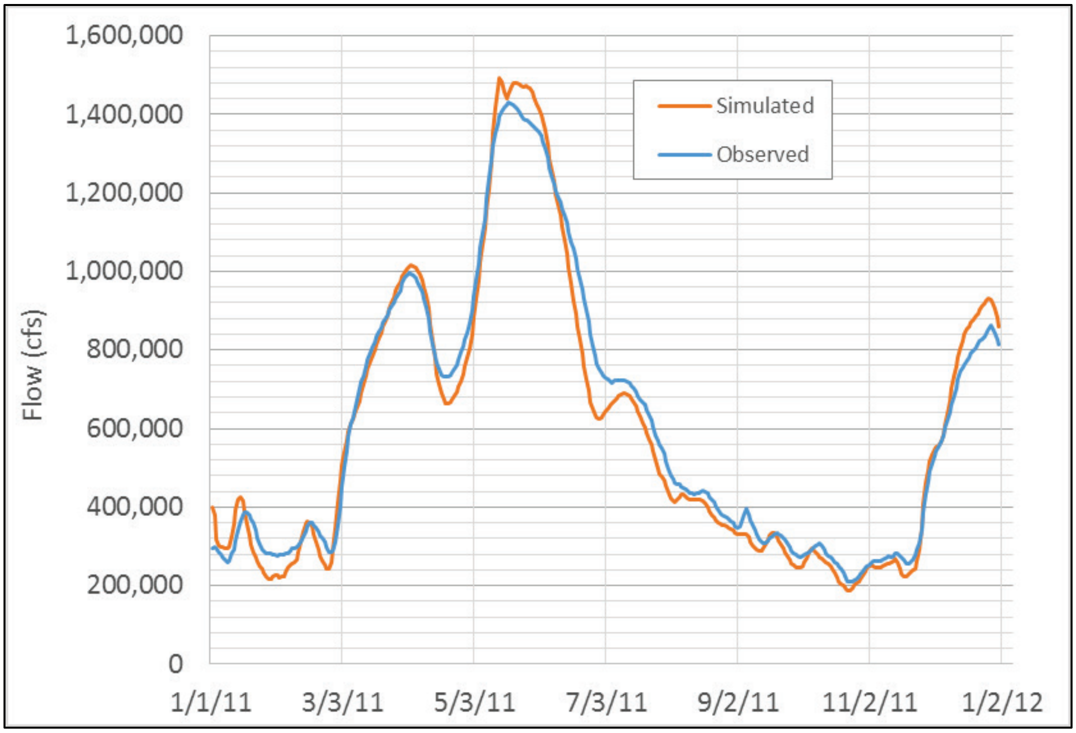


Figure 3-8. 2002 simulated and observed stages for Helena, AR.

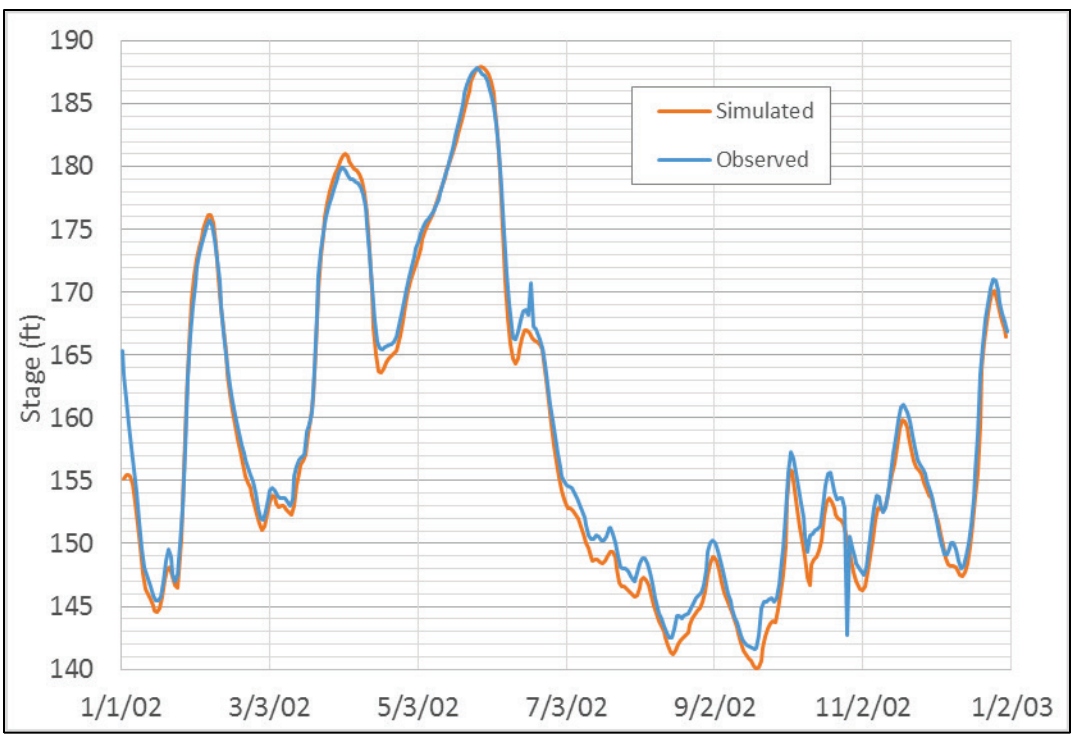


Figure 3-9. 2002 simulated and observed stages for Natchez, MS.

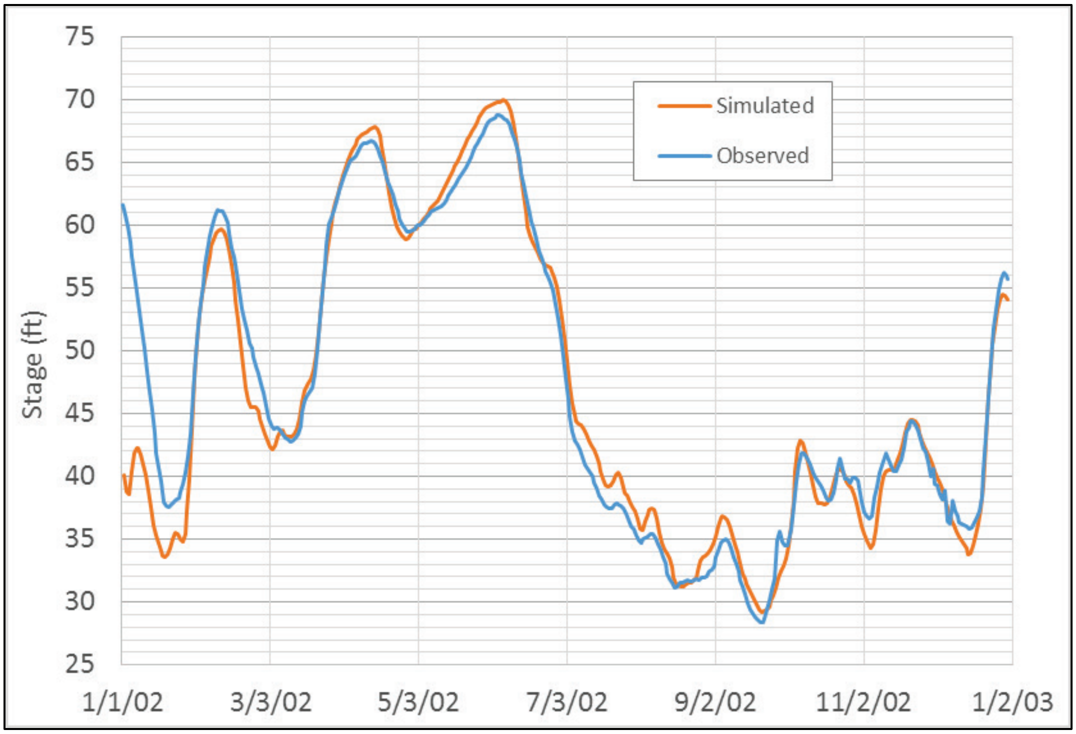


Figure 3-10. 2002 simulated and observed stages for Baton Rouge, LA.

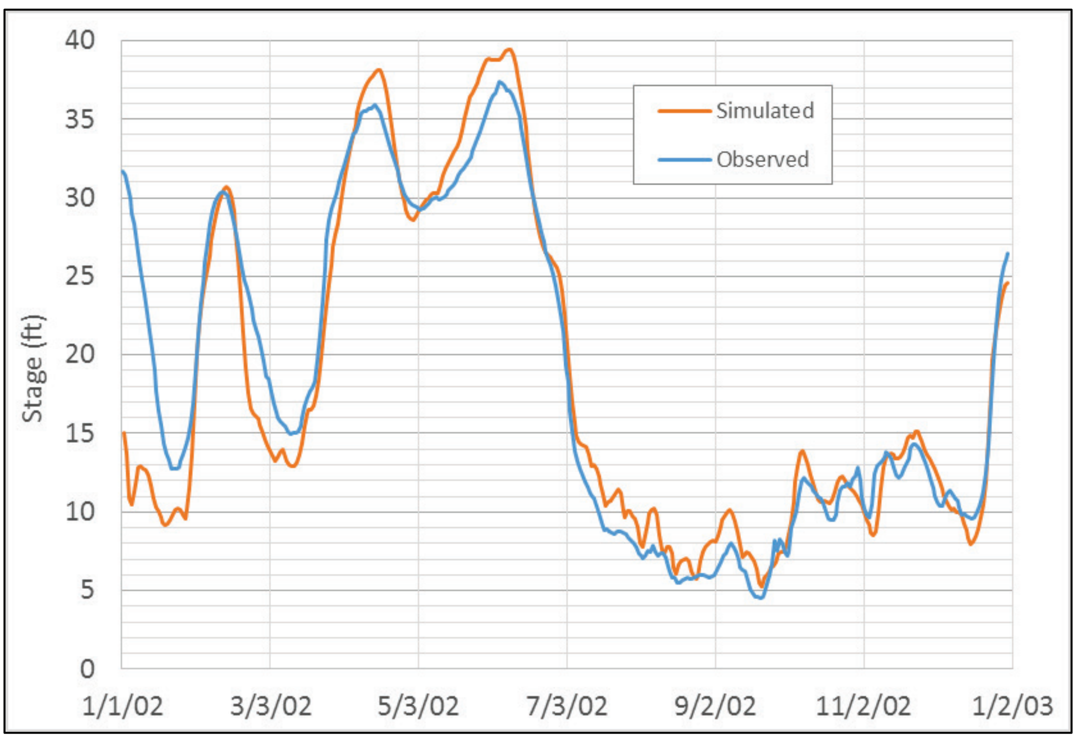


Figure 3-11. 2002 simulated and observed flows for Baton Rouge, LA (gaps indicate data were not received for that time period).

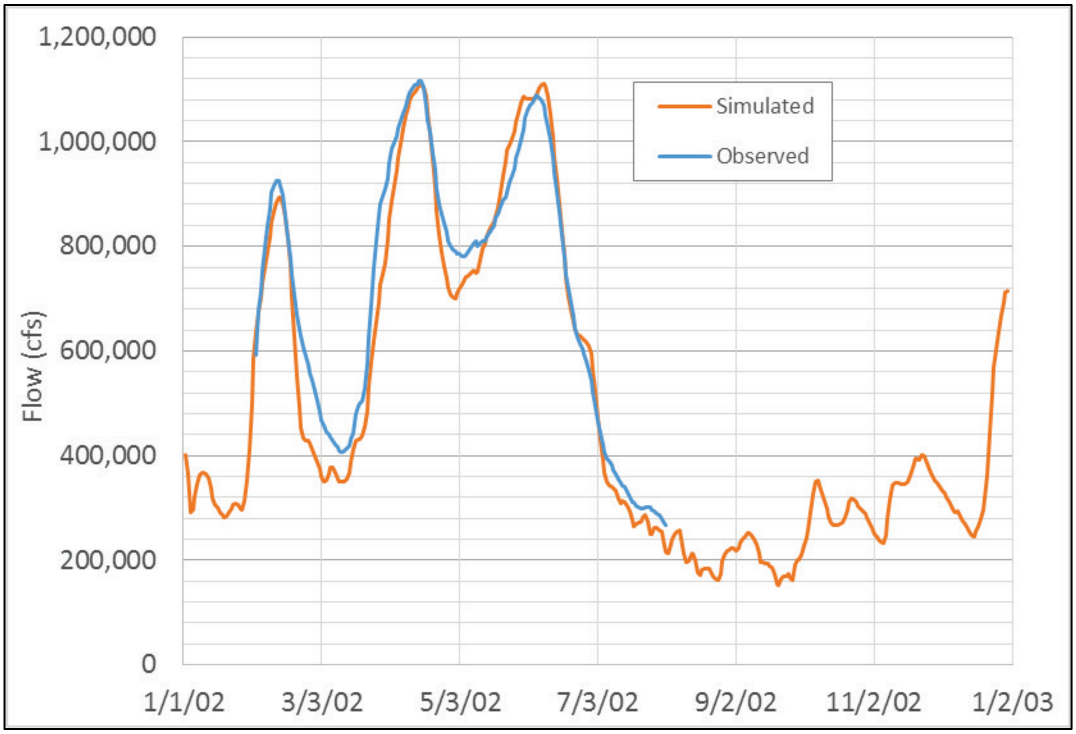


Figure 3-12. 2008 simulated and observed stages for Helena, AR.

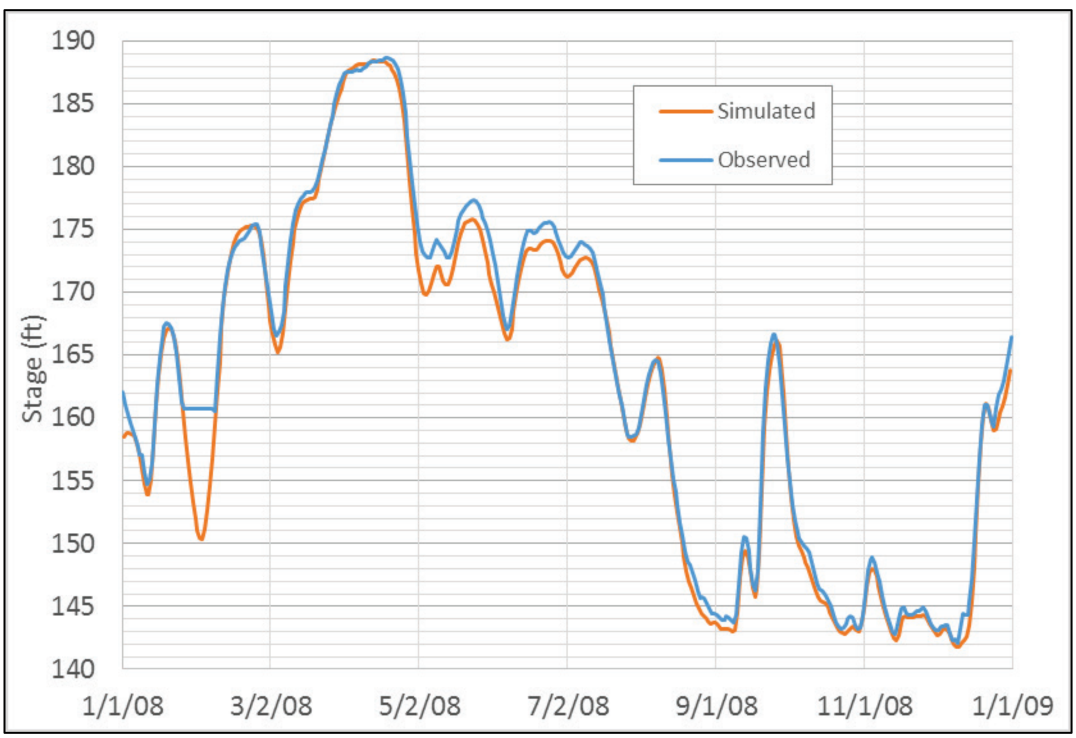


Figure 3-13. 2008 simulated and observed stages for Natchez, MS.

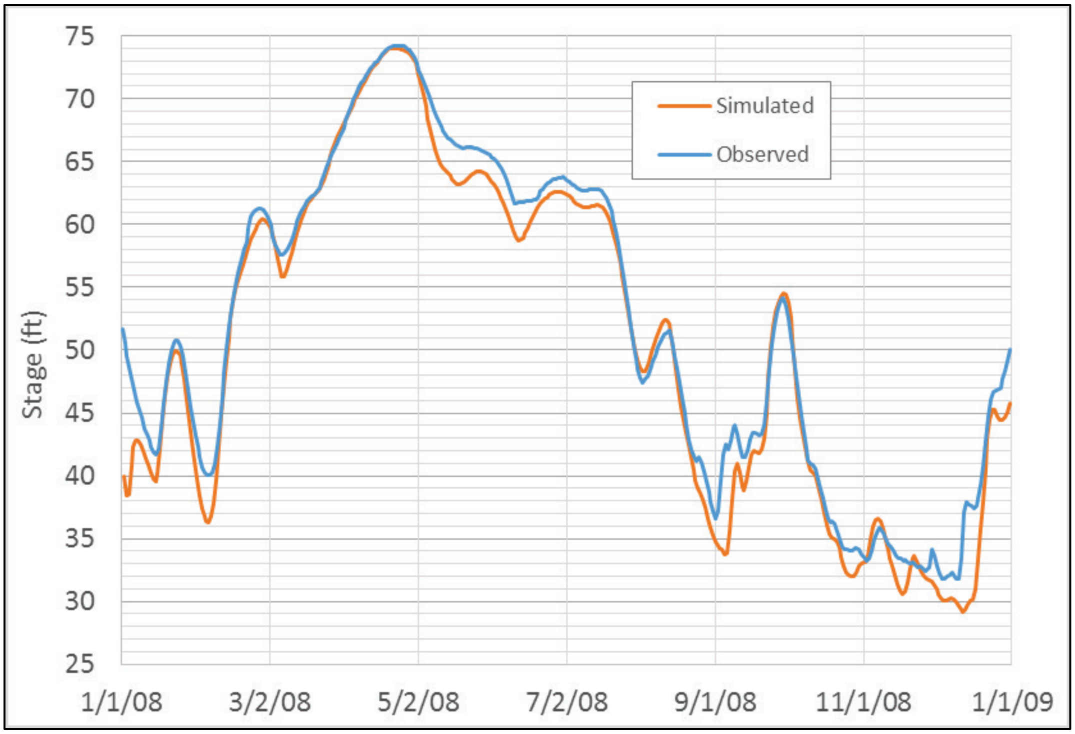


Figure 3-14. 2008 simulated and observed stages for Baton Rouge, LA.

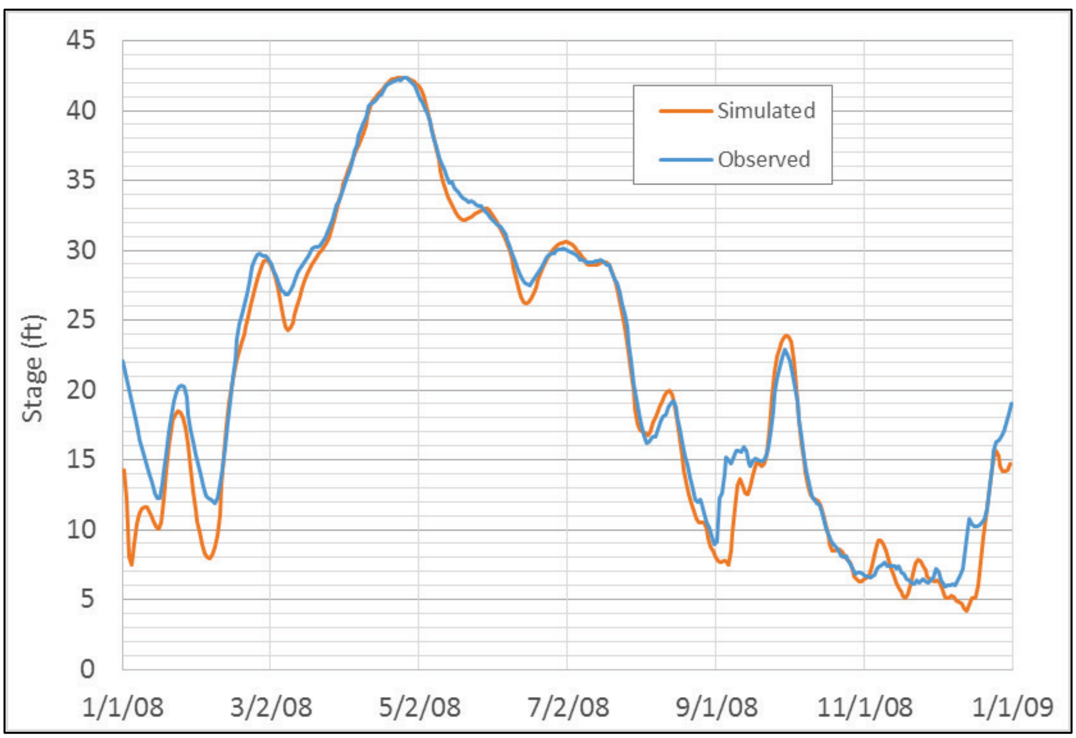
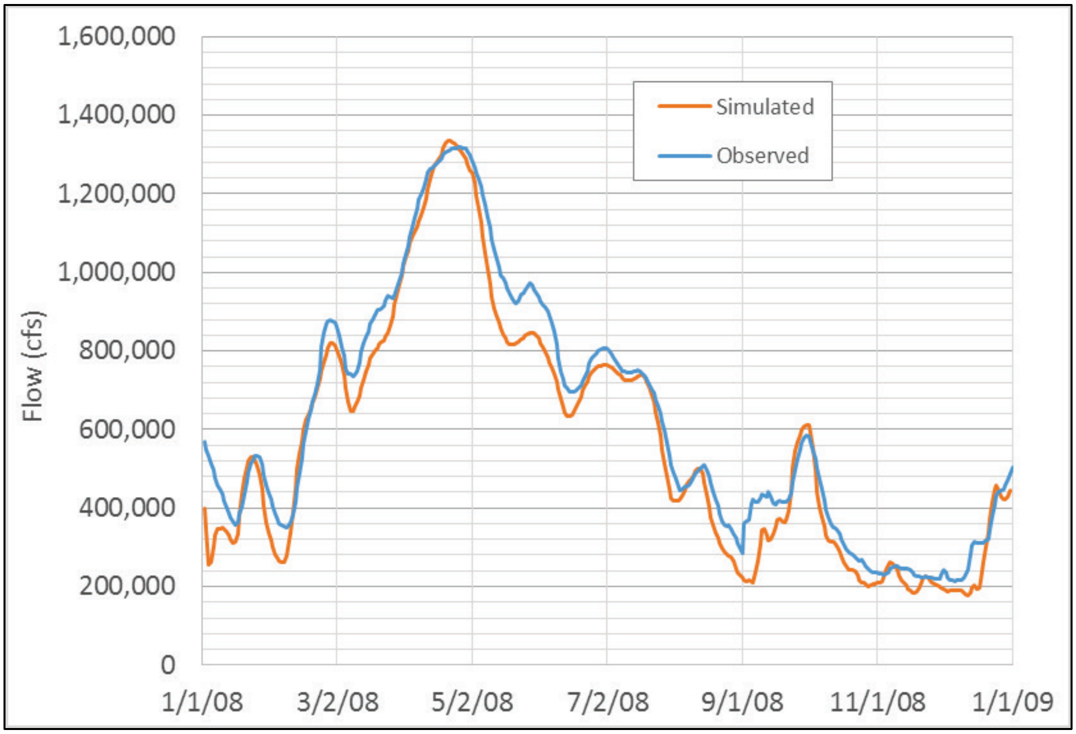


Figure 3-15. 2008 simulated and observed flows for Baton Rouge, LA.

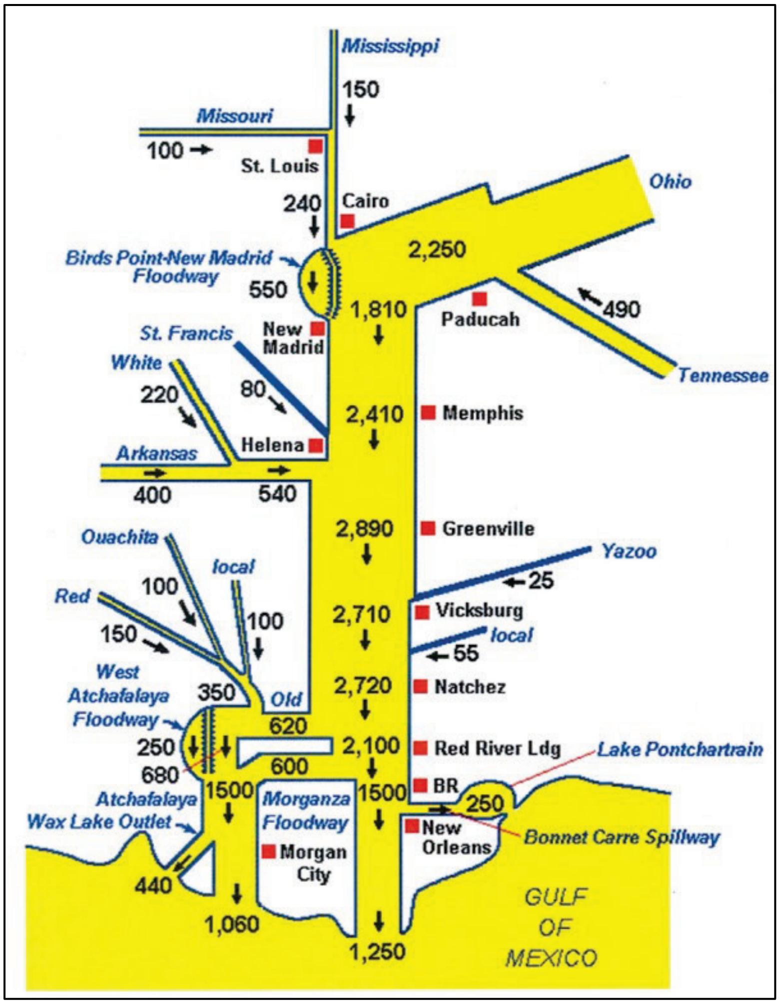


3.3 Simulations using the 1955 historical PDF flows

3.3.1 Methodology

To assess the changes that have occurred since the approval of the 1955 MR&T system, this project's model team ran two model simulations of the 1955 historical 58A-EN PDF using the validated HEC-RAS model, namely the "Historic PDF – Existing Yazoo" and "Historic PDF – Authorized Yazoo" simulations. Figure 3-16 is a schematic of the historical PDF flows at various locations of the MR&T Flowline Assessment. Differences between the historically determined PDF water surface elevations and the current HEC-RAS results (based on routing the historical flows) demonstrated the impact of differences in the numerical modeling framework, changes in geometry, and changes in roughness between the Refined 1973 Flowline Study (USACE 1978), as described in Section 1.3.4, and the current study.

Figure 3-16. Historic PDF flows within the MR&T system (flow in kcfs).*



*Note: the "240" below St. Louis represents the flow farther upstream from the confluence; at the confluence with the Ohio River, the flow from the reach is 410 cfs.

The diversion of the Mississippi River water through the ORCC was accomplished by following the Water Control Plan for the operation of ORCC structures used in the 1955 and 1973 studies. The ORCC operational plan directs the USACE to evaluate the forecasted Mississippi River and Red River discharges. Based on the forecast, the USACE is directed to adjust the ORCC operation so that, on an annual basis, 70% of the Mississippi-Atchafalaya River combined discharge downstream of ORCC passes Red River Landing while 30% of the discharge passes Simmesport. The hydraulic model simulated the ORCC such that the 70/30 distribution of flow was met on a daily basis to not deviate too far from the mandated annual percentages. Additionally, the models were limited to stay below the current PDF peak flow through the

ORCC of 620,000 cfs. This is based on an upstream PDF peak flow of 2.72 million cfs.

The remaining flood flow was routed downstream of ORCC. The resulting peak flow in the updated model at Red River Landing was computed as 2,105,900 cfs, as compared to the 2,100,000 cfs that was routed in the 1955 study. Below Red River Landing, the Morganza Floodway exists to maintain a Mississippi River downstream flow of 1.5 million cfs. In the Refined 1973 Flowline Study (USACE 1978) model, any measured flow above 1.5 million cfs was diverted through the Morganza Floodway and into the Atchafalaya Basin as according to the approved Water Control Plan for the Floodway (USACE 2000, 2014).

The Bonnet Carré Spillway, located approximately 30 miles upstream of New Orleans on the Mississippi River, was modeled to limit the flows in the Mississippi River to 1.25 million cfs downstream of the structure. The design flows through Morganza Floodway and Bonnet Carré Spillway are 600,000 cfs and 250,000 cfs, respectively.

The Red River Backwater area was more difficult to model due to the large amount of flood water storage available. The model was revised to include flood storage in the backwater area by modeling levees as lateral structures and raising lateral structures to divert the maximum authorized flow of 350,000 cfs. The model diverted 364,200 cfs, which was more than the authorized flow and was routed to combine with the authorized flow of 620,000 cfs through the ORCC. In addition, proper timing of these two peaks required an additional amount of floodwater be added from the backwater area near the Simmesport gage, on the Atchafalaya River, to get the appropriate flow at that location. The official peak flow at Simmesport on the Atchafalaya River was 930,000 cfs, and the adjusted HEC-RAS model achieved a peak flow of 964,450 cfs.

The first run, "Historic PDF – Existing Yazoo," routed the 1955 58A-EN hydrology through the updated 2016 model to determine the resulting water surface elevations based on current conditions. The simulation was completed by utilizing the historic flows as the upstream boundaries and allowing the model to route the flow throughout the system. The results of this simulation evaluate the effectiveness of multiple MR&T backwater storage areas (including the Yazoo Backwater area) based on the current system. The "Historic PDF – Existing Yazoo" run was performed with the

Yazoo Backwater Levee at the existing elevations, with levee elevations as low as 107 ft (NAVD 88).

The second run, “Historic PDF – Authorized Yazoo,” was similar to the first run except that the Yazoo Backwater Levee was raised to its authorized elevation of 112.8 ft (NAVD 88), as set by the Refined 1973 Flowline Study (USACE 1978).

3.3.2 Model adjustments

This section briefly lists the changes that were required to complete the two historic runs “Historic PDF – Existing Yazoo,” and “Historic PDF – Authorized Yazoo.” After model validation, the HEC-RAS model was upgraded to newer release versions of the HEC-RAS software. The combined model was converted from a HEC-RAS, beta version 4.2, to a HEC-RAS, beta version 5.0, to HEC-RAS, version 5.0, and then finally to HEC-RAS, version 5.0.1. Each of the calibration and validation simulations were recomputed with HEC-RAS, version 5.0.1. No significant differences were noticed, so no changes were required to upgrade the model to version 5.0.1.

A list of the model adjustments is provided below. See the Hydraulics Report (USACE 2018b) for more information about each of the adjustments.

- Hydrology connections: Adjustments were made to the inflow boundary condition locations based on the available historic hydrograph flow data to provide a comparison between the Refined 1973 Flowline Study (USACE 1978) and the current assessment. The hydrologic inflows were the same in the Refined 1973 Flowline Study as in the 1955 Study.
- Merriwether-Cherokee Levee and Sheep’s Ridge road elevations: Adjustments were made to the model based on a major breach in the revetment and spur levee in the location of the Merriwether-Cherokee bend of the river (in MVM near HEC-RAS station 885) and the current conditions after its repair.
- Main line levees: The validated HEC-RAS model was modified to not allow overtopping for the PDF events by setting the weir coefficients of many lateral structures to zero. This also included the deletion of non-influential storage areas that were outside of non-overtopping levees.

- Bird's Point – New Madrid: An activation sequence of the Bird's Point – New Madrid Floodway Inflow Crevasse, based on river conditions and logistical timing, was assumed for PDF scenarios, as outlined in the activation plan by MVM.
- Seasonal roughness: Typically, the warm season has larger roughness values and results in higher stages for a given flow rate. While the model was calibrated using seasonal variations in the roughness (Manning's n) values, all PDF scenarios used constant roughness values corresponding to the warm season.
- Flow roughness: Since the PDF flows in some cases were higher than the flows which the model was calibrated to, it was necessary to extrapolate the relationship between Manning's n and flow rate.
- Yazoo Backwater Levee: The "Historic PDF – Existing Yazoo" simulation used the existing Yazoo Backwater Levee profile with some levee elevations as low as approximately 107 ft. However, for the "Historic PDF – Authorized Yazoo" simulation, the Yazoo Backwater Levee profile was raised to 112.8 ft.
- Morganza Floodway: The operation of the Morganza Floodway, or Morganza Control Structure (MCS), was modeled to follow the Interim Standing Instructions to the Project Manager for Water Control for the Morganza Control Structure, which was recently revised in March 2014 (USACE 2000, 2014) to allow for openings based on either stage or flow river conditions.
- Bonnet Carré Spillway: Under the MR&T project, the Bonnet Carré Spillway is operated when flow in the river below Morganza exceeds 1,250,000 cfs. The floodway operation is also authorized to lower stages along the river below the spillway to protect the levee system through New Orleans.
- Downstream boundary condition: The downstream boundary of the model is at Venice, LA. To simulate the PDF events, a downstream boundary was needed that would produce a stage, for each time-step, that accounts for the dependence of the flow at Venice on the stage. The 2002, 2008, and 2011 simulations used observed water surface elevations for the downstream boundary condition, but the hypothetical events required a rating curve. The rating curve was developed based on a non-linear regression analysis of observed historical data (discussed further in the Hydraulics Report, USACE [2018b]).
- Gulf of Mexico stage: Since the downstream boundary of the Mississippi River Flowline model is at Venice, LA, which is 10.5 river

miles above Head of Passes, an analysis was performed to check the relationship in stages between Venice and the Gulf of Mexico.

3.3.3 Results

Differences between the historically recorded PDF water surface elevations and the current HEC-RAS results, driven by the historical flows, demonstrated the impact of differences in the numerical modeling framework, changes in geometry, and changes in roughness between the Refined 1973 Flowline Study (USACE 1978) and the current assessment. Table 3-2 compares the previously published flows from the 1955 hydrology with the flows resulting from the new HEC-RAS model developed for this assessment. In general, the flows that have been routed through the HEC-RAS model compare very closely with the flows reported in the Refined 1973 Flowline Study (USACE 1978). Table 3-3 shows the water surface comparisons between the results of this assessment and the previous elevations from the Refined 1973 Flowline Study (USACE 1978) without adjustments for hysteresis, or loop effect, and channel degradation (converted to NAVD 88). The loop effect and channel degradation were also performed for the current assessment and are documented in Sections 6 and 5, respectively.

Table 3-2. Maximum flow comparisons of the historic PDF simulations (cubic feet per second).

Description	HEC-RAS Station	Published 1955 Hydrology	Historic – PDF Existing Yazoo	Historic – PDF Authorized Yazoo
Mississippi at Chester	110.4	240,000	301,000	301,000
Ohio River at Cairo	-979.68	2,250,000	2,255,000	2,255,000
Mississippi at Ohio River	973.85	2,360,000	2,393,000	2,393,000
Mississippi at Hickman	942.45	1,810,000	1,881,000	1,881,000
Mississippi at Memphis	749.01	2,410,000	2,415,000	2,415,000
Mississippi at Helena	676.42	2,460,000	2,445,000	2,445,000
Mississippi at Arkansas City	562.18	2,890,000	2,874,000	2,874,000
Arkansas River at Dam 02	28.07	400,000	403,000	403,000
St. Francis at 82.47	82.47	80,000	74,000	74,000
White River at Clarendon	100.05	220,000	223,000	223,000
Mississippi at Greenville	539.13	*	2,869,000	2,867,000

Description	HEC-RAS Station	Published 1955 Hydrology	Historic – PDF Existing Yazoo	Historic – PDF Authorized Yazoo
Mississippi at Lake Providence	494.47	*	2,866,000	2,863,000
Mississippi at Vicksburg	442.16	2,710,000	2,678,000	2,849,000
Mississippi at Natchez	368.44	2,720,000	2,694,000	2,852,000
Mississippi at Red River Landing	306.43	2,100,000	2,077,000	2,226,000
Mississippi at Baton Rouge Gage	233.1	1,500,000	1,501,000	1,616,000
Mississippi at Donaldsonville gage	179.04	1,500,000	1,501,000	1,615,000
Mississippi at Carrollton Gage	107.2	1,250,000	1,250,000	1,360,000
Mississippi at Empire Gage	33.79	1,250,000	1,216,000	1,303,000
Mississippi at Venice gage	15.07	1,250,000	892,000	951,000

*Blank values indicate the historic study's values were not available at those locations.

Changes between the previously published stages in the Refined 1973 Flowline Study (USACE 1978) and the results of these historic simulations of this assessment highlight differences between the modeling framework, changes in the geometry, and changes in roughness. Geomorphic assessments of the Mississippi River (Biedenharn et al. 2017) provide comparison information for the trends in changes to water levels for varying flow rates over long periods of time. The broad-scale geomorphic results from Biedenharn et al. (2017) show dynamic equilibrium or slightly decreasing stages for reaches of the river between Hickman and Arkansas City for high flows. The simulated water surfaces in Table 3-3 agree with the geomorphic assessment since they show lower water surfaces than the Refined 1973 Flowline Study (USACE 1978) between Hickman and Arkansas City. The reaches of the river from Arkansas City to Red River Landing have experienced increasing stages for high flows, according to geomorphic analysis (Biedenharn et al. 2017). From Table 3-3, peak water surfaces from around Vicksburg and going downstream to around Baton Rouge agree with the geomorphic observation of increasing stage trends as the simulated water surfaces are higher for the current assessment than the Refined 1973 Flowline Study (USACE 1978) for that reach of the river.

Table 3-3. Maximum stage comparisons of the historic PDF runs (feet, NAVD 88).

Description	HEC-RAS Station	Refined 1973 Flowline (58A-EN)*	Historic – PDF Existing Yazoo	Historic – PDF Authorized Yazoo
Mississippi at Chester	110.4		360.9	360.9
Ohio River at Cairo	-979.68		332.5	332.5
Mississippi at Ohio River	973.85	331.7	332.0	332.0
Mississippi at Hickman	942.45	320.7	318.6	318.6
Mississippi at Memphis	749.01	237.2	235.8	235.8
Mississippi at Helena	676.42	202.9	201.9	201.9
Mississippi at Arkansas City	562.18	155.5	154.8	154.8
Arkansas River at Dam 02	28.07		172.2	172.2
St. Francis at 82.47	82.47		216.7	216.7
White River at Clarendon	100.05		179.1	179.1
Mississippi at Greenville	539.13	145.2	144.0	144.2
Mississippi at Lake Providence	494.47	129.4	127.5	128.1
Mississippi at Vicksburg	442.16	106.9	107.0	108.8
Mississippi at Natchez	368.44	82.7	83.6	85.2
Mississippi at Red River Landing	306.43	63.5	63.5	65.5
Mississippi at Baton Rouge Gage	233.1	45.7	44.4	47.1
Mississippi at Donaldsonville gage	179.04	33.8	31.7	34.3
Mississippi at Carrollton Gage	107.2	19.6	16.8	18.6
Mississippi at Empire Gage	33.79	9.8	5.8	6.2
Mississippi at Venice gage	15.07	8.1	3.8	3.9

*From the Refined 1973 Flowline Study (USACE 1978), this column is without loop effect, sedimentation, or freeboard effects.

Simulated stages at Arkansas City, Greenville, and Lake Providence are in slight disagreement with the observed trends in the geomorphic assessment since the current assessment simulated a lower water surface than the Refined 1973 Flowline Study (USACE 1978). This is likely caused by a difference in how the unsteady hydraulics were computed between the two time periods. The current HEC-RAS model is a robust model that computes a fully unsteady simulation including dynamic interactions with storage areas and backwater areas. Differences can also be explained by the fact that the simulated flow during the historic PDF event is much higher than the geomorphic assessments are able to consider, since there are no observations of river stages for that high of a flow. Figures in Section 3.4.3 also show some historic simulated results, labeled as "published."

3.4 Simulations using the new hypothetical flows

3.4.1 Methodology

This section documents the methodology used to simulate the MR&T system hydraulic response to newly generated hypothetical PDF events (labeled "2016" in this assessment). The 2016 PDF simulations are based on replicating the same design storms labeled 58A, 52A, 56, and 63 in the 1955 Report (MRC 1955) using a current NWS hydrology model. For more detailed information on the development of the 2016 PDF events, refer to Section 2.

The storm event labels used in this report are listed below.

- 2016 58A-U: The simulation labeled "2016 58A-U" represents the use of the NWS model to simulate the 58A design storm without reservoir regulation impacts (U: for unregulated). For more discussion about the reservoirs, see the Hydrology Report (USACE 2018a).
- 2016 58A-R: The simulation labeled "2016 58A-R" represents the use of the NWS model to simulate the 58A design storm with the expected reservoir regulation based on current operational procedures (R: for regulated). This is different than the 1955 Report label of 58A-EN, which included regulation from reservoirs which were existing (E) and near future (N) at the time of the 1955 Report.
- 2016 52A-R: The simulation labeled "2016 52A-R" represents the use of the NWS model to simulate the 52A design storm with the expected reservoir regulation based on current operational procedures (R: for regulated).
- 2016 56-R: The simulation labeled "2016 56-R" represents the use of the NWS model to simulate the 56 design storm with the expected reservoir regulation based on current operational procedures (R: for regulated).
- 2016 63-R: The simulation labeled "2016 63-R" represents the use of the NWS model to simulate the 63 design storm with the expected

reservoir regulation based on current operational procedures (R: for regulated).

Ten 2016 PDF simulations were performed with the HEC-RAS model: "2016 58A-U PDF – Existing Yazoo," "2016 58A-U PDF – Authorized Yazoo," "2016 58A-R PDF – Existing Yazoo," "2016 58A-R PDF – Authorized Yazoo," "2016 52A-R PDF – Existing Yazoo," "2016 52A-R PDF – Authorized Yazoo," "2016 56-R PDF – Existing Yazoo," "2016 56-R PDF – Authorized Yazoo," "2016 63-R PDF – Existing Yazoo," and "2016 63-R PDF – Authorized Yazoo." Unregulated simulations of the 52A, 56, and 63 events were not simulated within the HEC-RAS model, though some information about them can be found in the Hydrology Report (USACE 2018a).

3.4.2 Model adjustments

Most of the assumptions and boundary conditions for the 2016 PDF simulations matched those of the historic PDF simulations. The assumptions described in Section 3.3.2 for Merriwether-Cherokee, main line levees, storage areas, BPNMF, seasonal roughnesses, flow roughnesses, Morganza Floodway, Bonnet Carré Spillway, Venice, and Gulf of Mexico were also applied to the 2016 PDF runs.

3.4.2.1 Flow adjustments from NWS model

For the new hypothetical HEC-RAS simulations, all of the upstream boundary conditions of inflow required data from the NWS hydrologic model. Up to this point of the assessment, many of the boundary conditions of the HEC-RAS model were supplied with either available gage data or available historic hydrograph data. For the new hypothetical simulations, neither of these alternative options were available. The inflow connections for the new hypothetical simulations were in the same locations as used in the calibration and validation simulations (see Figure 3-2). The MVK noticed that some localized basins used in the NWS modeling did not factor in manmade structures such as levees, so flows had to be relocated, reduced, or removed from the HEC-RAS model. The Hydraulics Report (USACE 2018b) describes locations where the flows from the NWS basins needed to be manipulated using flow multipliers. The MVM and MVN portions of the NWS model did not require any adjustments to the flows from the hydrology model.

3.4.2.2 ORCC

The flow diversion for the ORCC typically operates at a distribution of 70% Mississippi River and 30% Atchafalaya River (70/30) for the total flow from the Mississippi River and Red River combined at the latitude of Red River Landing. During the 2016 PDF event simulations, the flow was assumed to remain at the 70/30 distribution until the flow through the ORCC reached its authorized capacity of 620,000 cfs. The calculation of the flow diversion is based on a 1-day forecast of the Mississippi and Red Rivers. Since HEC-RAS does not have the functionality to automatically handle this type of flow distribution in the ORCC operations, a separate spreadsheet was used to manually check and iterate the HEC-RAS simulations until the ORCC was properly operated within the HEC-RAS model.

3.4.3 Results

The flows and stages of the 2016 PDF simulations indicate the expected hydraulic behavior using the newly generated hypothetical design storm hydrology. The unsteady HEC-RAS model calculates how the 2016 PDF storm flows (from USACE [2018a]) would be routed through the system. Flow hydrographs are used to see how the flows vary as the events propagate. The peak flows and peak water surface elevations can also be extracted at key locations.

3.4.3.1 2016 58A PDF

Figure 3-17 through Figure 3-21 show the flow hydrographs of the 2016 PDF 58A-R simulations at key locations. The Refined 1973 Flowline study (USACE 1978), Historic PDF Authorized Yazoo, and Historic PDF Existing Yazoo used the previously developed inflow hydrographs from the 1955 Hydrology, hence the “1955” label in the figures. In comparison with those three lines, only the 58A-R Authorized Yazoo results are shown in the figures for visual clarity. Other simulation results are presented in Table 3-4 and Table 3-5. Table 3-4 compares the previously published flows from the 1955 hydrology with the 2016 PDF flows within the new HEC-RAS model developed for this assessment. Table 3-5 shows the water surface comparisons between the previous elevations from the Refined 1973 Flowline Study (USACE 1978) (converted to NAVD 88) and the 2016 PDF results of this assessment. The water surface profiles of the Refined 1973 Flowline Study (USACE 1978), Historic PDF Authorized Yazoo, and

58A-R Authorized Yazoo results are shown in 100-mile increments in Figure 3-22 through Figure 3-31.

Results show that the flows are higher in the current assessment than the Refined 1973 Flowline Study (USACE 1978). This is to be expected since the companion Hydrology Report (USACE 2018a) associated with the current assessment provided higher flows that were used as input boundary conditions to this HEC-RAS model. (For more discussion about the increase in flow, see Section 2.) Figure 3-17 through Figure 3-21 show that the peak flows of the 58A-R Authorized Yazoo simulation were higher and arrived sooner than the previous results.

Table 3-5 and Figure 3-22 through Figure 3-31 show that the computed water surfaces are generally higher for the current assessment than the results using the 1955 hydrology. Many water surface elevations are approximately 4 ft higher in the 58A-R Authorized Yazoo simulation than the Refined 1973 Flowline Study (USACE 1978) water surfaces, converted to NAVD 88. None of the data sets shown here include the hysteresis (i.e., loop effect) or channel degradation impacts.

The water surface results for the downstream end of the river are lower in the current assessment than the Refined 1973 Flowline Study due to differences in downstream boundary condition assumptions. The rating curve boundary condition is a significant difference from the Refined 1973 Flowline Study (USACE 1978). The amount of water that is flowing out of the river through natural and man-made diversions is also much higher now in the lower part of the Mississippi River than it was in the 1970s. There are other dynamics that could occur at the downstream end of the river due to coastal events that were not included within the scope of this assessment, and these would have significant impacts in terms of the expected maximum range of water surfaces for that reach.

A couple of the biggest differences in stage occur just upstream of Memphis (see Figure 3-24) and near Baton Rouge (Figure 3-29). The Refined 1973 Flowline Study profile upstream of Memphis was not as smooth as the current HEC-RAS model. This was noticed during the calibration to the 2011 event. In other words, the current HEC-RAS model has been calibrated to the 2011 conditions such that a high flow event exhibits a smoother profile than shown in the Refined 1973 Flowline Study results. As seen in Figure 3-24, approximately 3 ft of the 8 ft of difference in stage can be attributed to the smoother model simulation here at this specific location. The 58A-R Authorized Yazoo stage at Baton Rouge is 7.1 ft higher than the Refined 1973 Flowline Study (USACE 1978), as seen in Table 3-5. Due to the higher flows and earlier arrival time, the unsteady hydraulic results show a generally increasing difference in water surface from Vicksburg moving downstream until near Baton Rouge where it peaks. The Baton Rouge location is also near where the geomorphic trends (Biedenharn et al. 2017) switch from being aggradational upstream of this location to a state of dynamic equilibrium downstream.

Table 3-4. Maximum flow comparisons of the 2016 58A PDF simulations (cubic feet per second).

Location	HEC-RAS Station	1955 Published 58A-EN	2016 PDF 58A-R Existing Yazoo	2016 PDF 58A-U Existing Yazoo	2016 PDF 58A-R Authorized Yazoo	2016 PDF 58A-U Authorized Yazoo
Mississippi at Chester	110.4	240,000	508,000	546,000	508,000	546,000
Ohio River at Cairo	-979.68	2,250,000	2,326,000	2,458,000	2,326,000	2,458,000
Mississippi at Ohio River	973.85	2,360,000	2,791,000	2,937,000	2,791,000	2,937,000
Mississippi at Hickman	942.45	1,810,000	1,973,000	1,988,000	1,973,000	1,988,000
Mississippi at Memphis	749.01	2,410,000	2,862,000	2,956,000	2,862,000	2,956,000
Mississippi at Helena	676.42	2,460,000	2,787,000	2,861,000	2,787,000	2,861,000
Mississippi at Arkansas City	562.18	2,890,000	3,263,000	3,367,000	3,263,000	3,367,000
Arkansas River at Dam 02	28.07	400,000	487,000	521,000	487,000	521,000
St. Francis at 82.47	82.47	80,000	85,000	85,000	85,000	85,000
White River at Clarendon	100.05	220,000	233,000	271,000	233,000	271,000
Mississippi at Greenville	**539.13**		**3,259,000**	**3,362,000**	**3,260,000**	**3,364,000**
Mississippi at Lake Providence	494.47		3,253,000	3,357,000	3,257,000	3,361,000
Mississippi at Vicksburg	442.16	2,710,000	3,076,000	3,245,000	3,087,000	3,122,000
Mississippi at Natchez	368.44	2,720,000	3,072,000	3,242,000	3,099,000	3,135,000
Mississippi at Red River Landing	306.43	2,100,000	2,449,000	2,618,000	2,475,000	2,514,000

Location	HEC-RAS Station	1955 Published 58A-EN	2016 PDF 58A-R Existing Yazoo	2016 PDF 58A-U Existing Yazoo	2016 PDF 58A-R Authorized Yazoo	2016 PDF 58A-U Authorized Yazoo
Mississippi at Baton Rouge Gage	233.1	1,500,00	1,838,000	2,007,000	1,869,000	1,910,000
Mississippi at Donaldsonville gage	179.04	1,500,00	1,837,000	2,007,000	1,868,000	1,910,000
Mississippi at Carrollton Gage	107.2	1,250,000	1,581,000	1,751,000	1,613,000	1,654,000
Mississippi at Empire Gage	33.79	1,250,000	1,513,000	1,675,000	1,543,000	1,583,000
Mississippi at Venice gage (15.07)	15.07	1,250,000	1,094,000	1,204,000	1,115,000	1,142,000

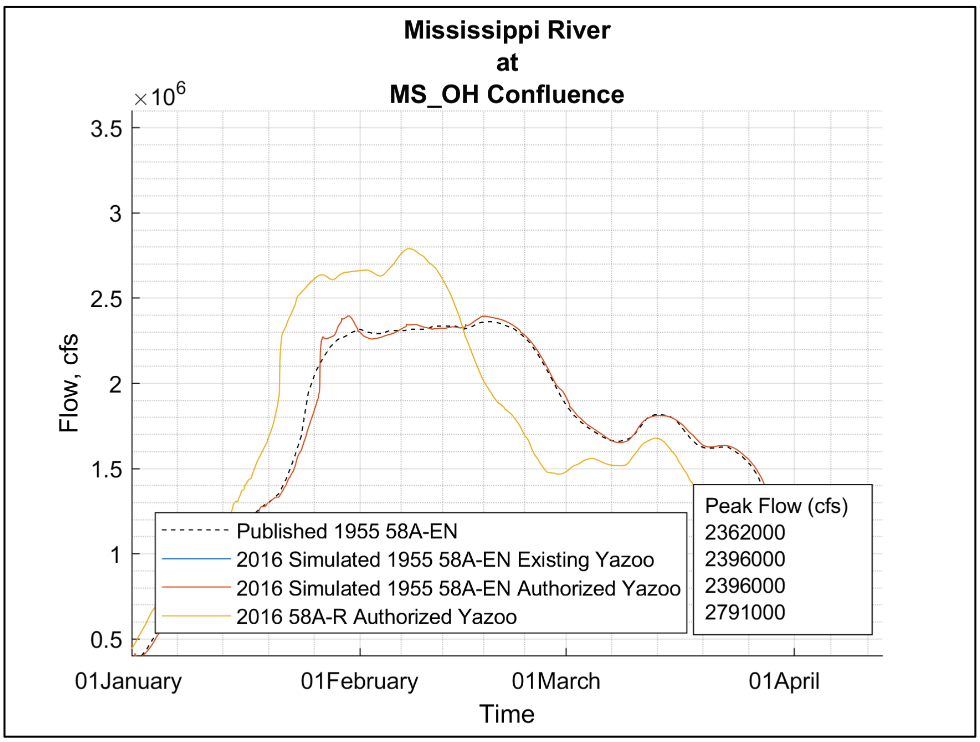
Table 3-5. Maximum stage comparisons of the 2016 58A PDF simulations (feet, NAVD 88).

Location	HEC-RAS Station	Refined 1973 Flowline (58A-EN)*	2016 PDF 58A-R Existing Yazoo	2016 PDF 58A-U Existing Yazoo	2016 PDF 58A-R Authorized Yazoo	2016 PDF 58A-U Authorized Yazoo
Mississippi at Chester	110.4		372.5	374.3	372.5	374.3
Ohio River at Cairo	-979.68		334.8	335.5	334.8	335.5
Mississippi at Ohio River	973.85	331.7	334.3	335.0	334.3	335.0
Mississippi at Hickman	942.45	320.7	321.4	322.2	321.4	322.2
Mississippi at Memphis	749.01	237.2	241.4	242.7	241.4	242.7
Mississippi at Helena	676.42	202.9	206.0	207.1	206.0	207.1
Mississippi at Arkansas City	562.18	155.5	158.9	160.1	159.0	160.1
Arkansas River at Dam 02	28.07		177.1	178.3	177.2	178.3
St. Francis	82.47		217.1	217.1	217.1	217.1
White River at Clarendon	100.05		181.9	183.9	181.9	183.9
Mississippi at Greenville	539.13	145.2	148.0	149.2	148.2	149.1
Mississippi at Lake Providence	494.47	129.4	131.7	133.1	132.0	132.9
Mississippi at Vicksburg	442.16	106.9	111.3	113.3	111.7	112.1
Mississippi at Natchez	368.44	82.7	88	90.1	88.4	88.9

Location	HEC-RAS Station	Refined 1973 Flowline (58A-EN)*	2016 PDF 58A-R Existing Yazoo	2016 PDF 58A-U Existing Yazoo	2016 PDF 58A-R Authorized Yazoo	2016 PDF 58A-U Authorized Yazoo
Mississippi at Red River Landing	306.43	63.5	69.3	72.2	69.8	70.5
Mississippi at Baton Rouge Gage	233.1	45.7	52.1	55.8	52.8	53.7
Mississippi at Donaldsonville gage	179.04	33.8	39.4	43.1	40.1	41.0
Mississippi at Carrollton Gage	107.2	19.6	22.3	25.1	22.8	23.5
Mississippi at Empire Gage	33.79	9.8	7.0	7.8	7.2	7.3
Mississippi at Venice gage	15.07	8.1	4.1	4.3	4.2	4.2

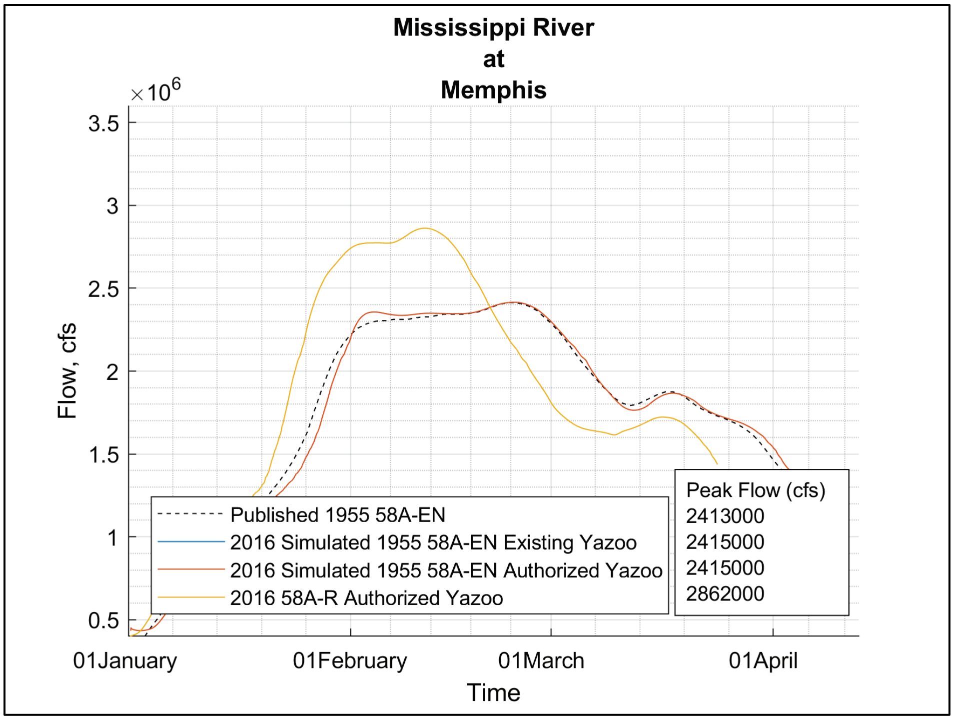
*From the Refined 1973 Flowline Study (USACE 1978), this column is without loop effect, sedimentation, or freeboard effects.

Figure 3-17. 2016 PDF 58A-R flow hydrograph at the Below Cairo location.*



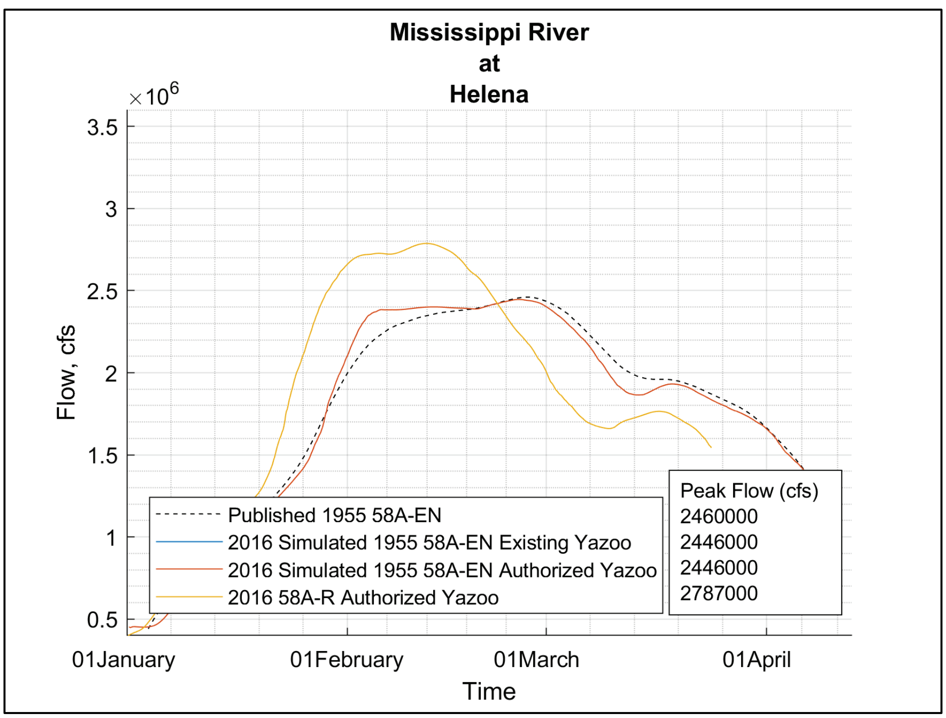
*Note: The flows are the same at this location for the “Existing Yazoo” and “Authorized Yazoo” runs since it is relatively far upstream.

Figure 3-18. 2016 PDF 58A-R flow hydrograph at the Memphis location.*



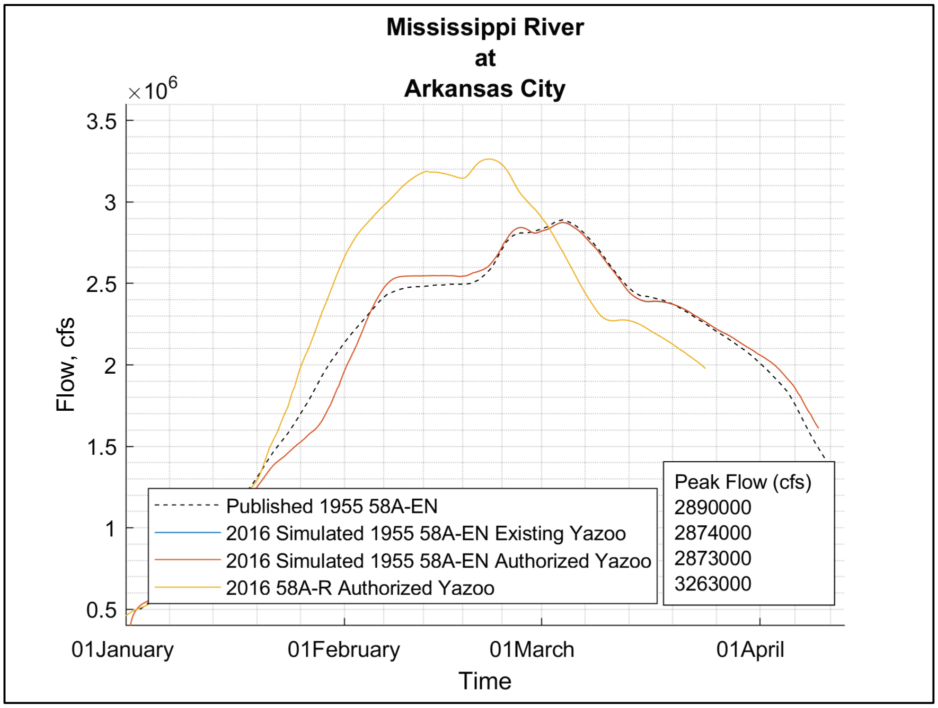
*Note: The flows are the same at this location for the “Existing Yazoo” and “Authorized Yazoo” runs since it is relatively far upstream.

Figure 3-19. 2016 PDF 58A-R flow hydrograph at the Helena location.*



*Note: The flows are the same at this location for the “Existing Yazoo” and “Authorized Yazoo” runs since it is relatively far upstream.

Figure 3-20. 2016 PDF 58A-R flow hydrograph at the Arkansas City location.*



*Note: The flows are the same at this location for the “Existing Yazoo” and “Authorized Yazoo” runs since it is relatively far upstream.

Figure 3-21. 2016 PDF 58A-R flow hydrograph at the Red River Landing location.

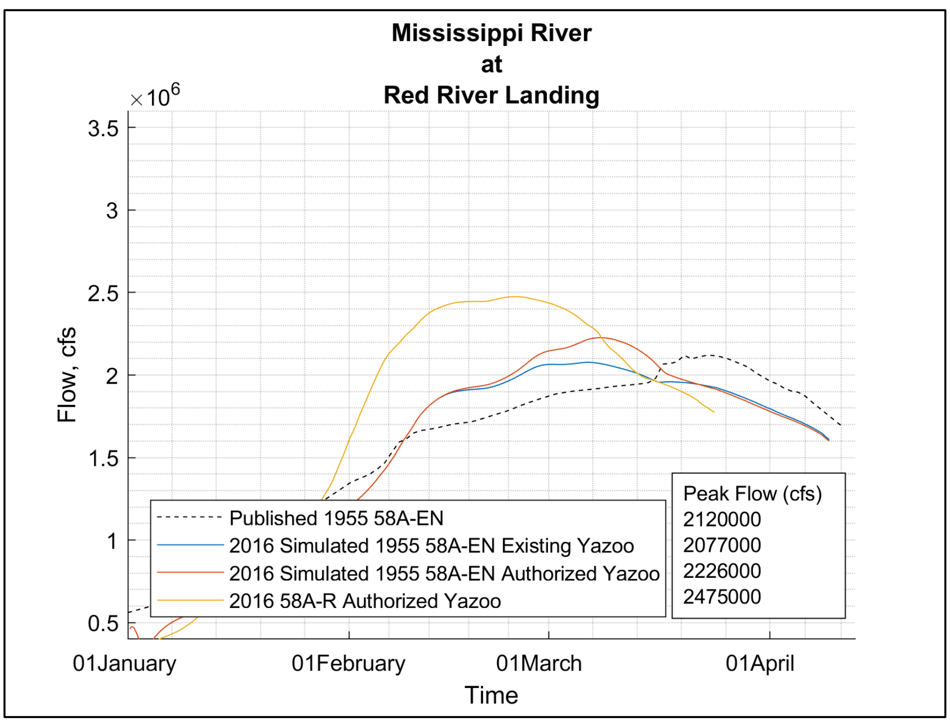
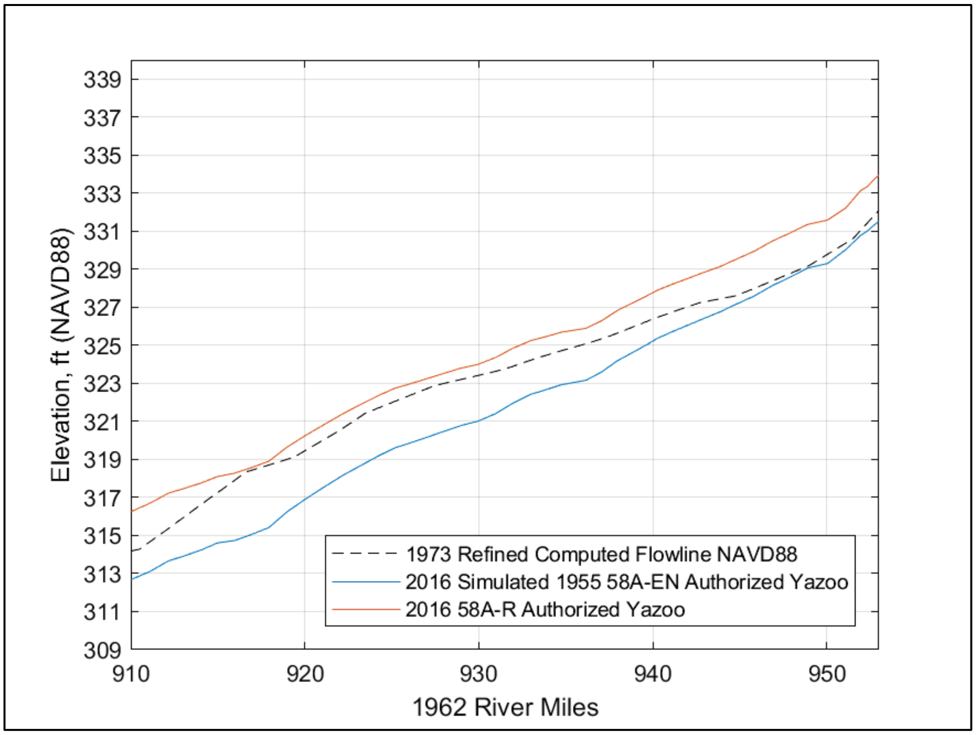
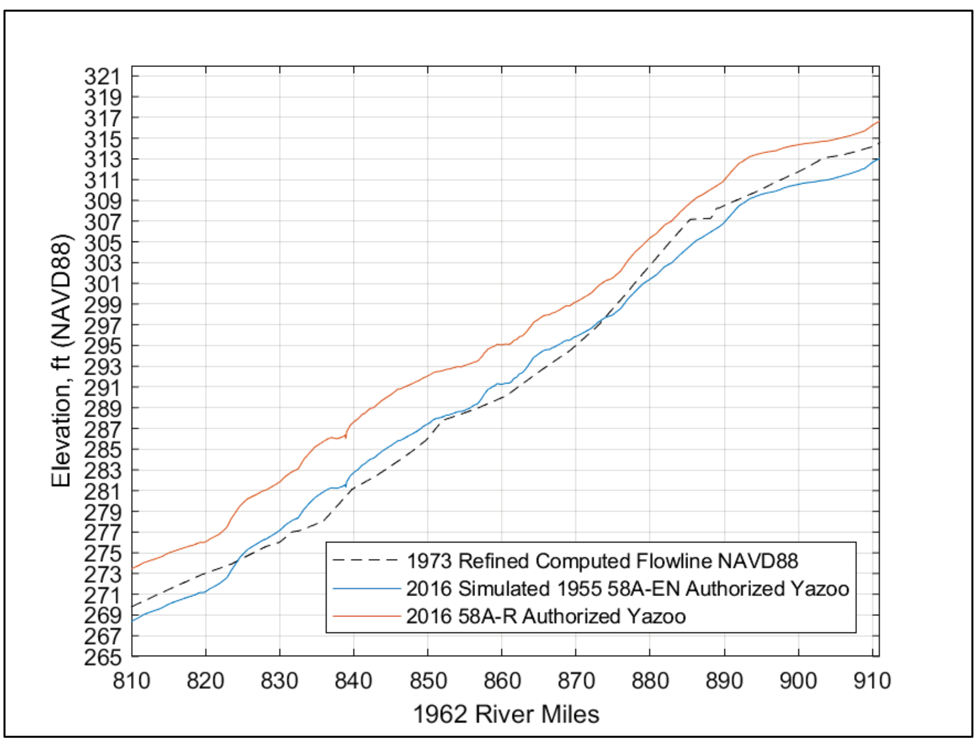


Figure 3-22. 2016 58A-R water surface profile for RMs 910–953.*



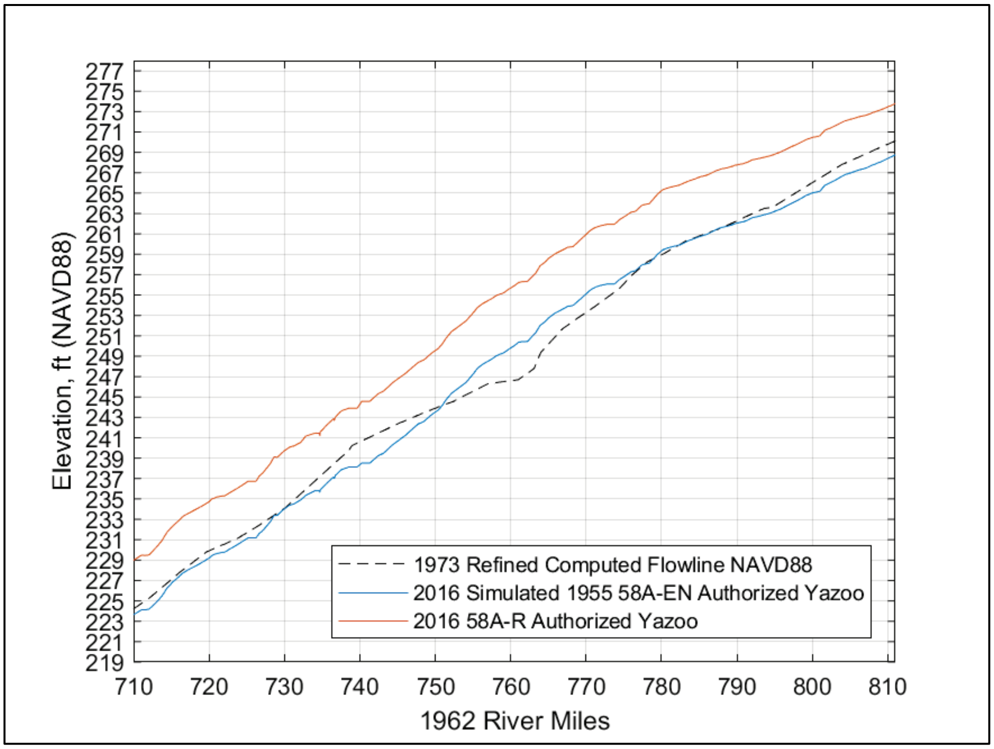
*Note: Refined 1973 Computed Flowline (USACE 1978) is without loop, sediment, and freeboard effects.

Figure 3-23. 2016 58A-R water surface profile for RMs 810–910.*



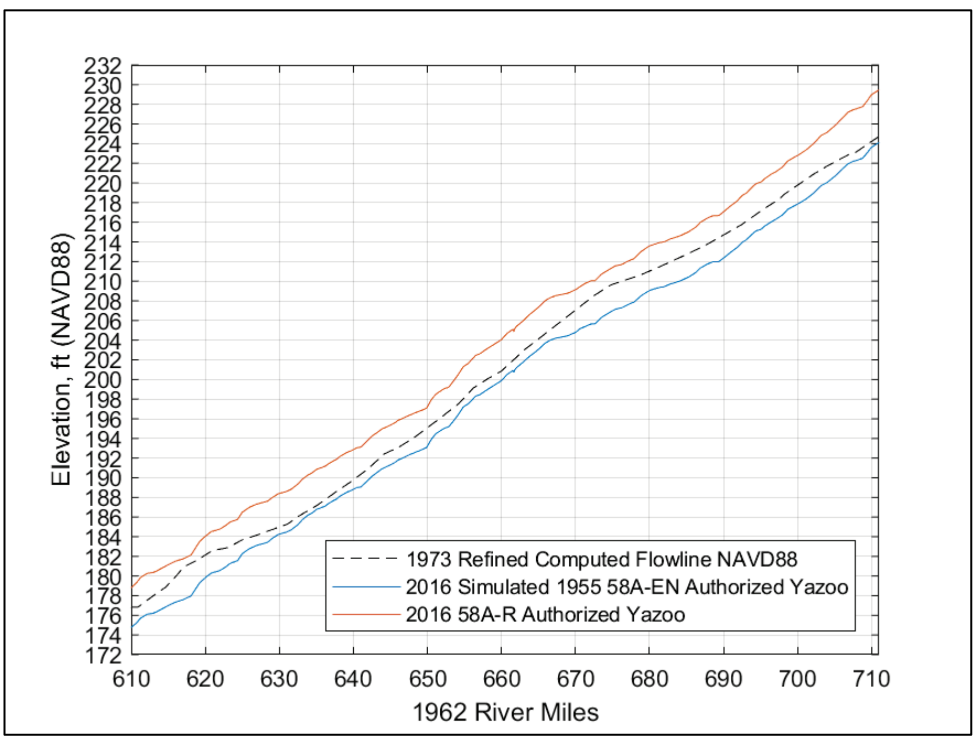
*Note: Refined 1973 Computed Flowline is without loop, sediment, and freeboard effects.

Figure 3-24. 2016 58A-R water surface profile for RMs 710–810.*



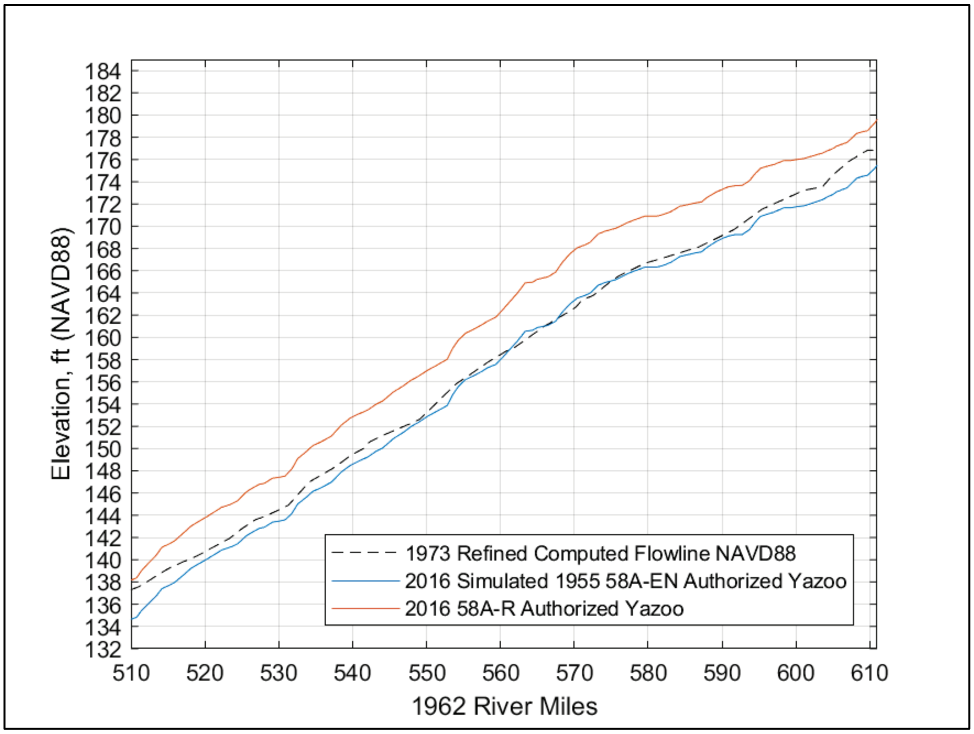
*Note: Refined 1973 Computed Flowline is without loop, sediment, and freeboard effects.

Figure 3-25. 2016 58A-R water surface profile for RMs 610–710.*



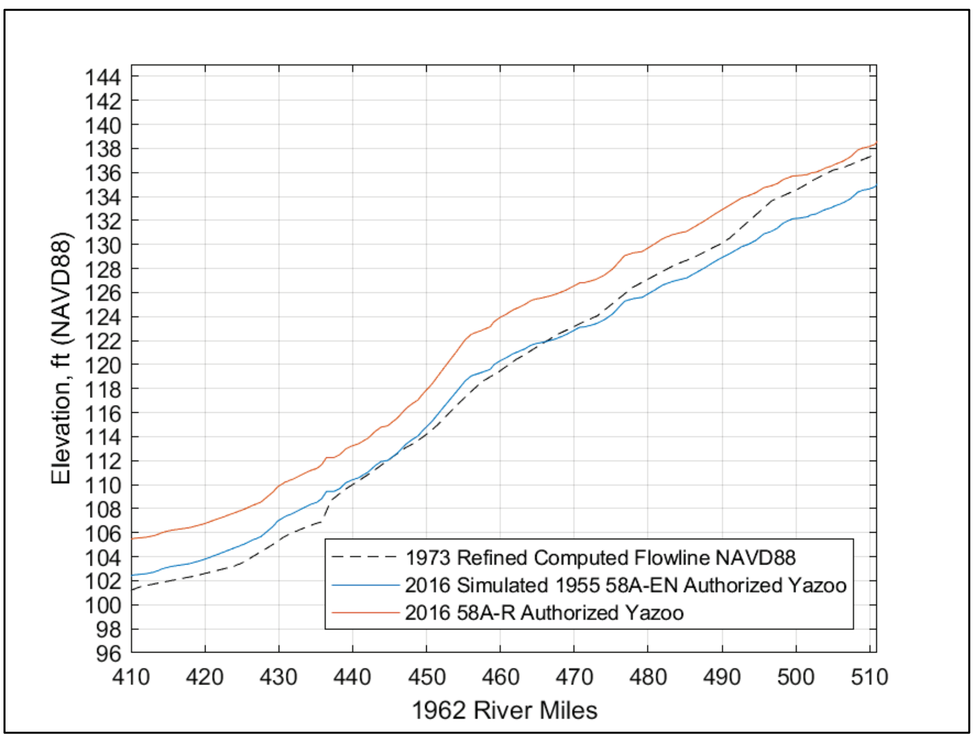
*Note: Refined 1973 Computed Flowline is without loop, sediment, and freeboard effects.

Figure 3-26. 2016 58A-R water surface profile for RMs 510–610.*



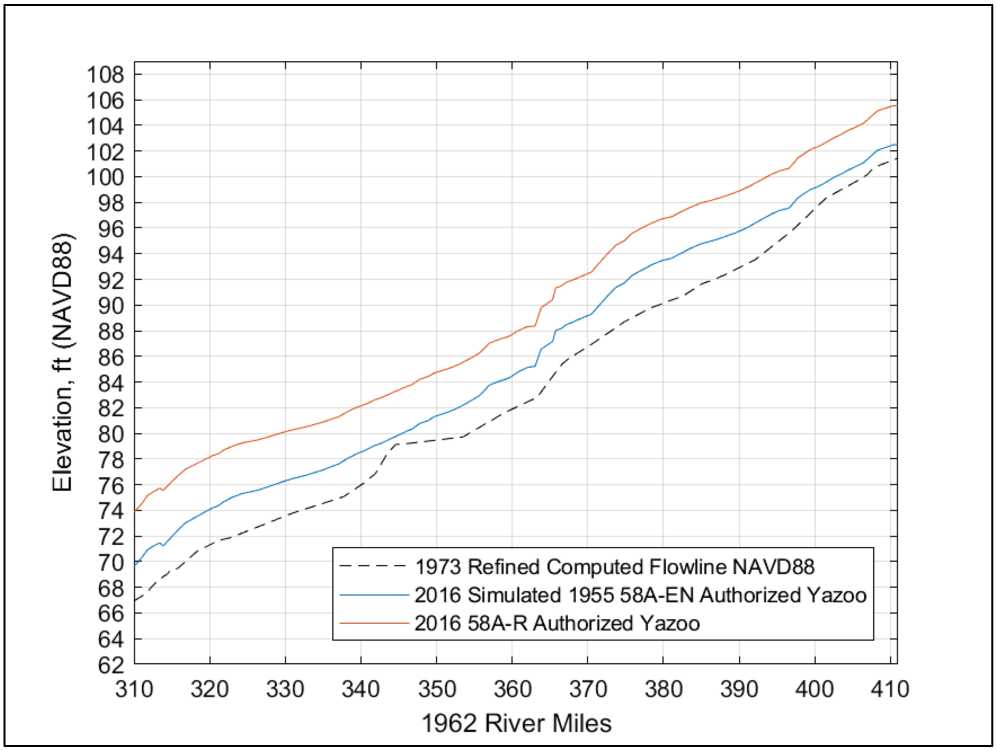
*Note: Refined 1973 Computed Flowline is without loop, sediment, and freeboard effects.

Figure 3-27. 2016 58A-R water surface profile for RMs 410–510.*



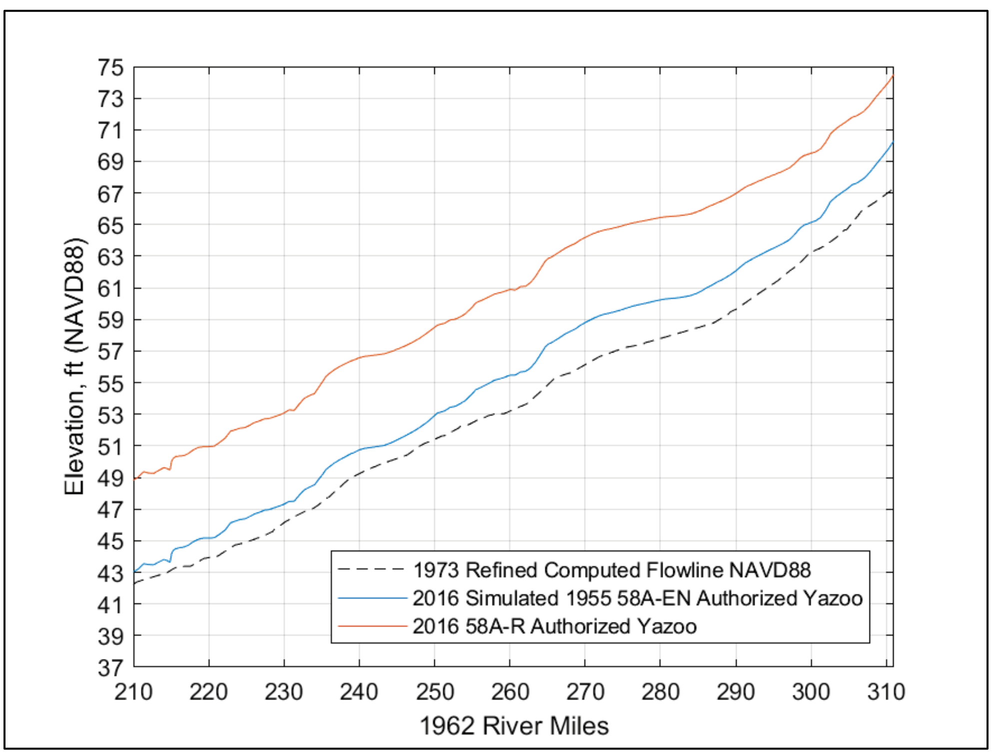
*Note: Refined 1973 Computed Flowline is without loop, sediment, and freeboard effects.

Figure 3-28. 2016 58A-R water surface profile for RMs 310–410.*



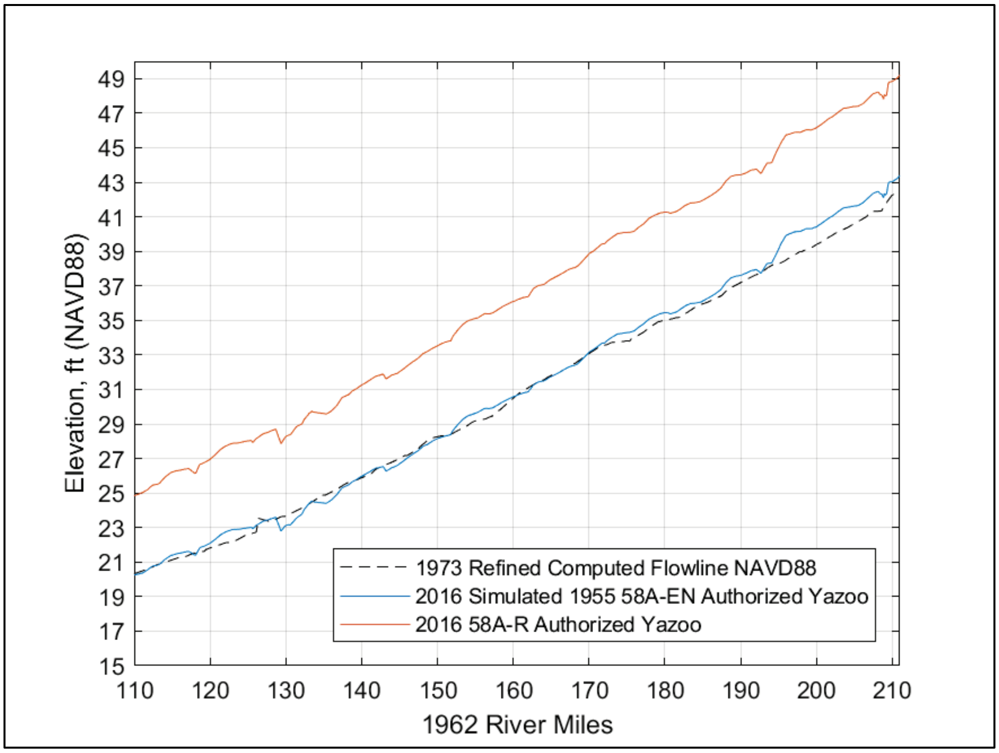
*Note: Refined 1973 Computed Flowline is without loop, sediment, and freeboard effects.

Figure 3-29. 2016 58A-R water surface profile for RMs 210–310.*



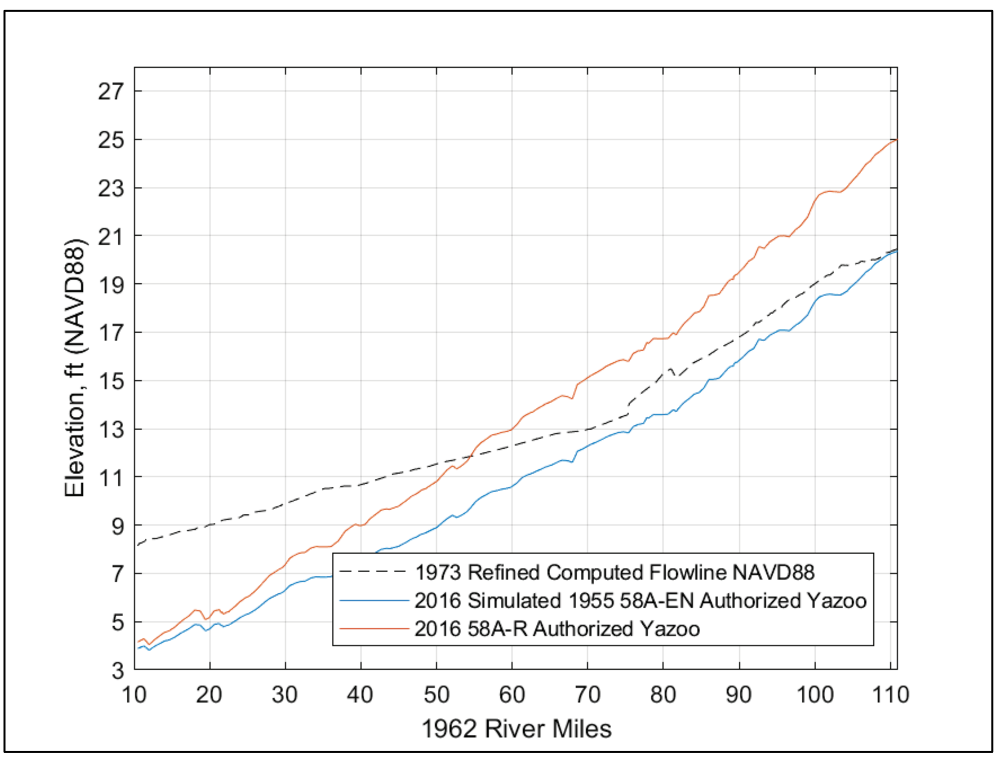
*Note: Refined 1973 Computed Flowline is without loop, sediment, and freeboard effects.

Figure 3-30. 2016 58A-R water surface profile for RMs 110–210.*



*Note: Refined 1973 Computed Flowline is without loop, sediment, and freeboard effects.

Figure 3-31. 2016 58A-R water surface profile for RMs 11–110.*



*Note: Refined 1973 Computed Flowline is without loop, sediment, and freeboard effects.

3.4.3.2 Other new hypo PDF simulations

The other 2016 PDF events were also simulated with the HEC-RAS model, namely “2016 52A-R PDF – Existing Yazoo,” “2016 52A-R PDF – Authorized Yazoo,” “2016 56-R PDF – Existing Yazoo,” “2016 56-R PDF – Authorized Yazoo,” “2016 63-R PDF – Existing Yazoo,” and “2016 63-R PDF – Authorized Yazoo.” The tabulated maximum flow results at key locations are shown in Table 3-6, and maximum stage results are shown in Table 3-7. More information about the flow hydrographs of each other 2016 PDF event can be found in the Hydrology Report (USACE 2018a). Water surface elevation profiles can be found in the Hydraulics Report (USACE 2018b).

Table 3-6. Maximum flow comparisons of the other 2016 PDF simulations (cubic feet per second).

Location	HEC-RAS Station	1955 Published 58A-EN	2016 PDF 58A-R Authorized Yazoo	2016 PDF 52A-R Existing Yazoo	2016 PDF 52A-R Authorized Yazoo	2016 PDF 56-R Existing Yazoo	2016 PDF 56-R Authorized Yazoo	2016 PDF 63-R Existing Yazoo	2016 PDF 63-R Authorized Yazoo
Mississippi at Chester	110.4	240,000	508,000	1,133,000	1,133,000	995,000	995,000	1,245,000	1,245,000
Ohio River at Cairo	-979.68	2,250,000	2,326,000	498,000	498,000	1,468,000	1,468,000	1,078,000	1,078,000
Mississippi at Ohio River	973.85	2,360,000	2,791,000	1,354,000	1,354,000	2,365,000	2,365,000	2,207,000	2,207,000
Mississippi at Hickman	942.45	1,810,000	1,973,000	1,339,000	1,339,000	1,881,000	1,881,000	1,884,000	1,884,000
Mississippi at Memphis	749.01	2,410,000	2,862,000	1,306,000	1,306,000	2,387,000	2,387,000	2,152,000	2,152,000
Mississippi at Helena	676.42	2,460,000	2,787,000	1,293,000	1,293,000	2,377,000	2,376,000	2,151,000	2,151,000
Mississippi at Arkansas City	562.18	2,890,000	3,263,000	1,498,000	1,498,000	2,872,000	2,871,000	2,778,000	2,778,000
Arkansas River at Dam 02	28.07	400,000	487,000	442,000	442,000	536,000	536,000	629,000	629,000
St. Francis at 82.47	82.47	80,000	85,000	11,000	11,000	53,000	53,000	51,000	51,000
White River at Clarendon	100.05	220,000	233,000	65,000	65,000	205,000	205,000	143,000	143,000
Mississippi at Greenville	539.13		3,260,000	1,489,000	1,489,000	2,867,000	2,865,000	2,772,000	2,771,000
Mississippi at Lake Providence	494.47		3,257,000	1,475,000	1,475,000	2,864,000	2,861,000	2,768,000	2,765,000

Location	HEC-RAS Station	1955 Published 58A-EN	2016 PDF 58A-R Authorized Yazoo	2016 PDF 52A-R Existing Yazoo	2016 PDF 52A-R Authorized Yazoo	2016 PDF 56-R Existing Yazoo	2016 PDF 56-R Authorized Yazoo	2016 PDF 63-R Existing Yazoo	2016 PDF 63-R Authorized Yazoo
Mississippi at Vicksburg	442.16	2,710,000	3,087,000	1,452,000	1,452,000	2,683,000	2,805,000	2,642,000	2,702,000
Mississippi at Natchez	368.44	2,720,000	3,099,000	1,443,000	1,443,000	2,692,000	2,808,000	2,628,000	2,685,000
Mississippi at Red River Landing	306.43	2,100,000	2,475,000	1,008,000	1,008,000	2,070,000	2,179,000	2005,000	2,060,000
Mississippi at Baton Rouge Gage	233.1	1,500,00	1,869,000	998,000	998,000	1,502,000	1,568,000	1,503,000	1,503,000
Mississippi at Donaldsonville gage	179.04	1,500,00	1,868,000	997,000	997,000	1,502,000	1,567,000	1,503,000	1,503,000
Mississippi at Carrollton Gage	107.2	1,250,000	1,613,000	993,000	993,000	1,252,000	1,312,000	1,252,000	1,252,000
Mississippi at Empire Gage	33.79	1,250,000	1,543,000	978,000	978,000	1,215,000	1,258,000	1,214,000	1,215,000
Mississippi at Venice gage	15.07	1,250,000	1,115,000	727,000	727,000	892,000	920,000	891,000	891,000

Table 3-7. Maximum stage comparisons of the other 2016 PDF simulations (feet, NAVD 88).*

Location	HEC-RAS Station	Refined 1973 Flowline (58A-EN)*	2016 PDF 58A-R Authorized Yazoo	2016 PDF 52A-R Existing Yazoo	2016 PDF 52A-R Authorized Yazoo	2016 PDF 56-R Existing Yazoo	2016 PDF 56-R Authorized Yazoo	2016 PDF 63-R Existing Yazoo	2016 PDF 63-R Authorized Yazoo
Mississippi at Chester	110.4		372.5	391.8	391.8	388.8	388.8	393.1	393.1
Ohio River at Cairo	-979.68		334.8	322.2	322.2	332.1	332.1	332.1	332.1
Mississippi at Ohio River	973.85	331.7	334.3	322.2	322.2	332.0	332.0	332.0	332.0
Mississippi at Hickman	942.45	320.7	321.4	309.1	309.1	318.0	318.0	317.7	317.7
Mississippi at Memphis	749.01	237.2	241.4	218.4	218.4	235.1	235.1	231.8	231.8
Mississippi at Helena	676.42	202.9	206	183.7	183.7	200.8	200.8	197.8	197.8
Mississippi at Arkansas City	562.18	155.5	159	135.3	135.3	154.7	154.8	153.7	153.7
Arkansas River at Dam 02	28.07		177.2	165.1	165.1	172.8	172.8	174.8	174.8
St. Francis at 82.47	82.47		217.1	209.1	209.1	215.0	215.0	214.8	214.8
White River at Clarendon	100.05		181.9	167.5	167.5	178.8	178.8	175.5	175.5
Mississippi at Greenville	539.13	145.2	148.2	126.1	126.1	144.0	144.1	143.0	143.0
Mississippi at Lake Providence	494.47	129.4	132	109.5	109.5	127.5	127.8	126.6	126.7
Mississippi at Vicksburg	442.16	106.9	111.7	91.4	91.4	107.0	108.3	106.4	107.0

Location	HEC-RAS Station	Refined 1973 Flowline (58A-EN)*	2016 PDF 58A-R Authorized Yazoo	2016 PDF 52A-R Existing Yazoo	2016 PDF 52A-R Authorized Yazoo	2016 PDF 56-R Existing Yazoo	2016 PDF 56-R Authorized Yazoo	2016 PDF 63-R Existing Yazoo	2016 PDF 63-R Authorized Yazoo
Mississippi at Natchez	368.44	82.7	88.4	69.5	69.5	83.5	84.7	82.9	83.4
Mississippi at Red River Landing	306.43	63.5	69.8	53.0	53.0	63.5	64.7	63.3	63.5
Mississippi at Baton Rouge Gage	233.1	45.7	52.8	36.9	36.9	44.4	45.9	44.4	44.4
Mississippi at Donaldsonville gage	179.04	33.8	40.1	25.8	25.8	31.7	33.2	31.7	31.7
Mississippi at Carrollton Gage	107.2	19.6	22.8	13.5	13.5	16.8	17.7	16.8	16.8
Mississippi at Empire Gage	33.79	9.8	7.2	5.0	5.0	5.9	6.0	5.8	5.8
Mississippi at Venice gage	15.07	8.1	4.2	3.5	3.5	3.8	3.8	3.8	3.8

*From the Refined 1973 Flowline Study (USACE 1978), this column is without loop effect, sedimentation, or freeboard effects.

The 2016 58A-R PDF simulation produced the highest peak water surfaces among the 2016 PDF events for all Mississippi River locations between Cairo, IL, and Venice, LA, which was anticipated based on the magnitude of peak flows and is in agreement with the 1955 relative comparisons of the events. The 2016 52A-R PDF event produced significantly lower peak water surfaces than the other 2016 PDF events. The 56-R and 63-R simulations produced similar results to each other for most locations, but the results were still lower than the 58A-R results. Due to the higher peak water surfaces, the 58A-R PDF is the simulation with the greatest focus in this assessment.

3.5 Simulations with future sea level rise (SLR)

Although the Lower Mississippi River experiences a high rate of subsidence, the PDF flowline is computed relative to the geodetic datum NAVD 88, which is not affected by subsidence. As the land sinks around it, the datum remains fixed in space. The flowline is therefore also unaffected by subsidence because without subsidence in the datum, there is no elevation change to alter the relationship between the flowline and the datum. However, the flowline is affected by SLR, through the backwater effect on the river that results from an increase in its downstream boundary condition. This rise is eustatic rise only, not relative change, which does include subsidence. As an estimate of future sea level conditions, an increase in the downstream boundary of 2.4 ft was used, relative to present sea level. This estimate is based on the “High” SLR scenario from ER 1100-2-8162 (USACE 2018, par. B-4), using the year 2066 to represent future conditions. Following the guidance in ER 1100-2-8162 (USACE 2013), the High scenario was chosen in recognition of the great importance of the MR&T project and the potential consequences of project failure. Note that this approach requires several assumptions, including the assumption that the flowline will always be referenced to a maintained, accurate vertical datum and that subsidence will not change the hydraulic characteristics of the river, for example by allowing levees to overtop or channel geometry to change shape. Both of these assumptions were assessed and found justifiable.

Since the downstream boundary of the Mississippi River HEC-RAS model is at Venice, LA (10.5 miles upstream from the Gulf of Mexico), the boundary will not experience the full SLR that will be observed in the ocean until sometime well after the period of analysis, especially considering the high river flow conditions relevant to this assessment. The dampening effect of SLR with distance upriver is demonstrated in Driessen and Van Ledden (2013). An adjustment to the existing Venice rating curve was needed to incorporate the effect of SLR. Similarly, new ratings (outlet flow vs. river flow) were also needed at several lateral outlets (e.g., Fort St. Phillip and Bohemia Diversions) into the Gulf of Mexico to represent the behavior of these outlets under future conditions.

An existing Adaptive Hydraulics (AdH) model (Sharp et al. 2013) was used to simulate a range of flows in the downstream area of the HEC-RAS model domain in two dimensions. The AdH model used the 2D shallow-water module of the USACE unstructured finite element model (for more information, see https://chl.erdc.dren.mil/chladh/). The AdH model applied the elevated sea level corresponding to future conditions at the boundaries of the model domain. A range of river flows were simulated to determine rating curves from stages and flows at various outlets of the HEC-RAS model, which were then applied to the future conditions HEC-RAS model. Note that this approach does not incorporate any subsidence in the channel inverts of the outlets, in addition to any change to river channel or overbank geometry at all. This approach is consistent with the approach used throughout the flowline effort. In reality, the topography and bathymetry of the outlets is likely to change in the future but in such an unpredictable way that it would be extremely difficult to estimate for a future conditions simulation. Therefore, the existing conditions geometry was preserved. This allowance for future condition sea level does not include the effects of tides, tropical storms, or wave effects.

The SLR adjustments were implemented in the HEC-RAS model, and the primary simulations were recomputed: 2016 58A-R Authorized Yazoo and 2016 58A-R Existing Yazoo. Peak flow comparisons for the 2016 58A-R simulations at key locations are shown in Table 3-8 and peak water surface elevations are shown in Table 3-9. Table 3-8 demonstrates that, for future conditions, the flow at Venice is approximately 60,000 cfs less than for current conditions for the 2016 58 A-R simulations. At Empire, the difference is only 3,000 cfs while there is no difference in current vs. future flow from Carrollton to any point upstream. The differences in flow for outlets vs. river for future conditions are caused by the more efficient conveyance in the outlets vs. the river between the diversion point and the Gulf, when considering future vs. current. Whereas the flow differences in future vs. current conditions are relatively minimal, the stage differences between the two scenarios are more significant. Table 3-9 demonstrates that for the future conditions, the stage at Venice is 1.2 ft higher than for current conditions for the 2016 58 A-R simulations while at Carrollton this difference is 0.4 ft. The difference in future vs. current is noted as far upstream as Red River Landing, where there is a 0.1 ft difference.

Figure 3-32 through Figure 3-35 depict the impact of future SLR on the water surface elevations in the lower 410 river miles.

Table 3-8. Maximum flow comparisons of the future SLR simulations (cubic feet per second).

Location	HEC-RAS Station	1955 Published 58A-EN	2016 PDF 58A-R Existing Yazoo	2016 PDF 58A-R Existing Yazoo, Future	2016 PDF 58A-R Authorized Yazoo	2016 PDF 58A-R Authorized Yazoo, Future
Mississippi at Chester	110.4	240,000	508,000	508,000	508,000	508,000
Ohio River at Cairo	-979.68	2,250,000	2,326,000	2,326,000	2,326,000	2,326,000
Mississippi at Ohio River	973.85	2,360,000	2,791,000	2,791,000	2,791,000	2,791,000
Mississippi at Hickman	942.45	1,810,000	1,973,000	1,973,000	1,973,000	1,973,000
Mississippi at Memphis	749.01	2,410,000	2,862,000	2,862,000	2,862,000	2,863,000
Mississippi at Helena	676.42	2,460,000	2,787,000	2,787,000	2,787,000	2,788,000
Mississippi at Arkansas City	562.18	2,890,000	3,263,000	3,263,000	3,263,000	3,263,000
Arkansas River at Dam 02	28.07	400,000	487,000	487,000	487,000	487,000
St. Francis River at 82.47	82.47	80,000	85,000	85,000	85,000	85,000
White River at Clarendon	100.05	220,000	233,000	233,000	233,000	233,000
Mississippi at Greenville	539.13		3,259,000	3,259,000	3,260,000	3,260,000
Mississippi at Lake Providence	494.47		3,253,000	3,253,000	3,257,000	3,257,000
Mississippi at Vicksburg	442.16	2,710,000	3,076,000	3,076,000	3,087,000	3,087,000
Mississippi at Natchez	368.44	2,720,000	3,072,000	3,072,000	3,099,000	3,099,000

Location	HEC-RAS Station	1955 Published 58A-EN	2016 PDF 58A-R Existing Yazoo	2016 PDF 58A-R Existing Yazoo, Future	2016 PDF 58A-R Authorized Yazoo	2016 PDF 58A-R Authorized Yazoo, Future
Mississippi at Red River Landing	306.43	2,100,000	2,449,000	2,449,000	2,475,000	2,475,000
Mississippi at Baton Rouge	233.1	1,500,00	1,838,000	1,838,000	1,869,000	1,869,000
Mississippi at Donaldsonville	179.04	1,500,00	1,837,000	1,837,000	1,868,000	1,868,000
Mississippi at Carrollton	107.2	1,250,000	1,581,000	1,582,000	1,613,000	1,613,000
Mississippi at Empire	33.79	1,250,000	1,513,000	1,510,000	1,543,000	1,539,000
Mississippi at Venice	15.07	1,250,000	1,094,000	1,031,000	1,115,000	1,051,000

Table 3-9. Maximum stage comparisons of the future SLR simulations (cubic feet per second).

Location	HEC-RAS Station	Refined 1973 Flowline (58A-EN)*	2016 PDF 58A-R Existing Yazoo	2016 PDF 58A-R Existing Yazoo, Future	2016 PDF 58A-R Authorized Yazoo	2016 PDF 58A-R Authorized Yazoo, Future
Mississippi at Chester	110.4		372.5	372.5	372.5	372.5
Ohio River at Cairo	-979.68		334.8	334.8	334.8	334.8
Mississippi at Ohio River	973.85	331.7	334.3	334.3	334.3	334.3
Mississippi at Hickman	942.45	320.7	321.4	321.4	321.4	321.4
Mississippi at Memphis	749.01	237.2	241.4	241.4	241.4	241.5
Mississippi at Helena	676.42	202.9	206.0	206.0	206.0	206.0
Mississippi at Arkansas City	562.18	155.5	158.9	158.9	159	159.0
Arkansas River at Dam 02	28.07		177.1	177.1	177.2	177.2
St. Francis River at 82.47	82.47		217.1	217.1	217.1	217.1
White River at Clarendon	100.05		181.9	181.9	181.9	181.9

Location	HEC-RAS Station	Refined 1973 Flowline (58A-EN)*	2016 PDF 58A-R Existing Yazoo	2016 PDF 58A-R Existing Yazoo, Future	2016 PDF 58A-R Authorized Yazoo	2016 PDF 58A-R Authorized Yazoo, Future
Mississippi at Greenville	539.13	145.2	148	148.0	148.2	148.2
Mississippi at Lake Providence	494.47	129.4	131.7	131.7	132	132.0
Mississippi at Vicksburg	442.16	106.9	111.3	111.3	111.7	111.7
Mississippi at Natchez	368.44	82.7	88	88.0	88.4	88.4
Mississippi at Red River Landing	306.43	63.5	69.3	69.3	69.8	69.8
Mississippi at Baton Rouge	233.1	45.7	52.1	52.2	52.8	52.9
Mississippi at Donaldsonville	179.04	33.8	39.4	39.6	40.1	40.3
Mississippi at Carrollton	107.2	19.6	22.3	22.7	22.8	23.2
Mississippi at Empire	33.79	9.8	7	7.8	7.2	7.9
Mississippi at Venice	15.07	8.1	4.1	5.3	4.2	5.3

*From the Refined 1973 Flowline Study (USACE 1978), this column is without loop effect, sedimentation, or freeboard effects.

Figure 3-32. Future SLR water surface profiles for RMs 310–410.

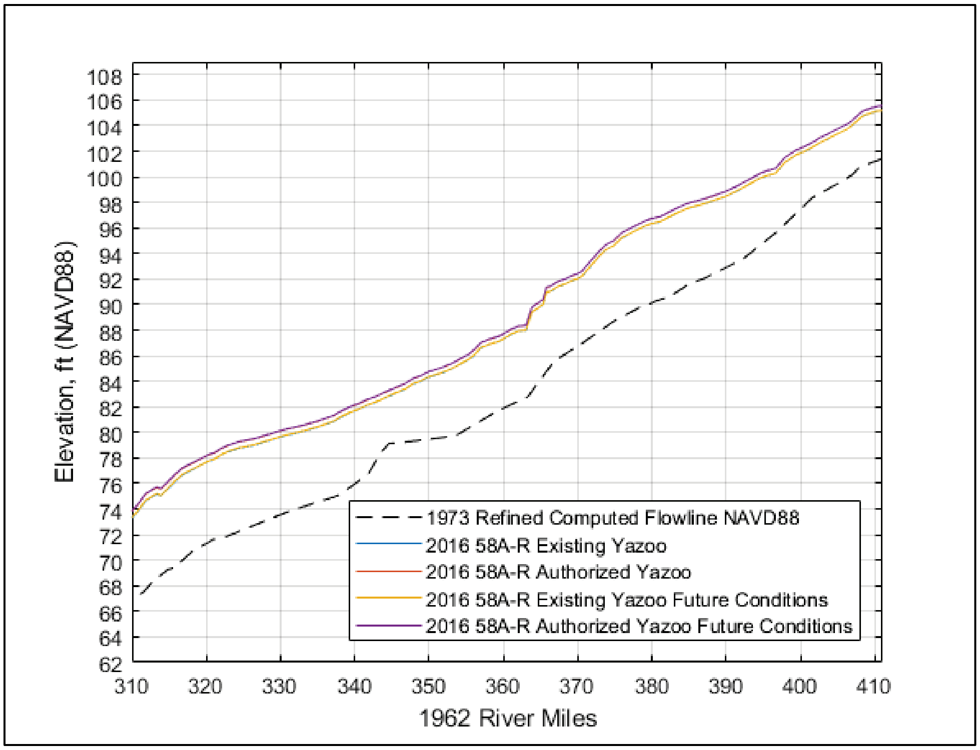


Figure 3-33. Future SLR water surface profiles for RMs 210–310.

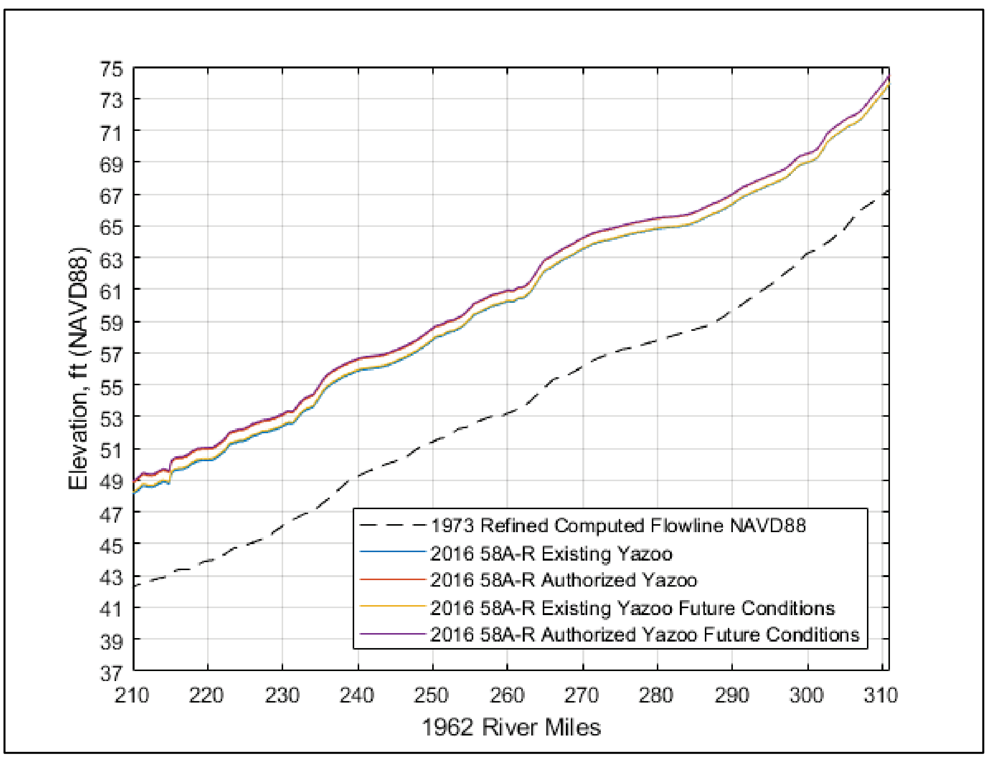


Figure 3-34. Future SLR water surface profiles for RMs 110–210.

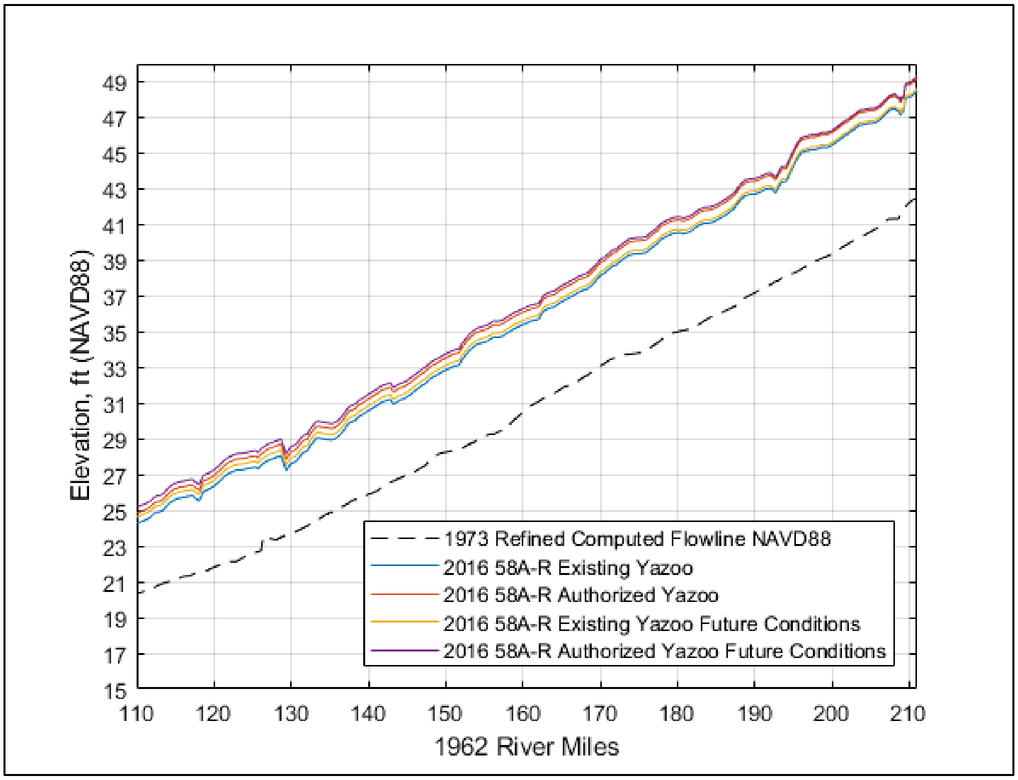
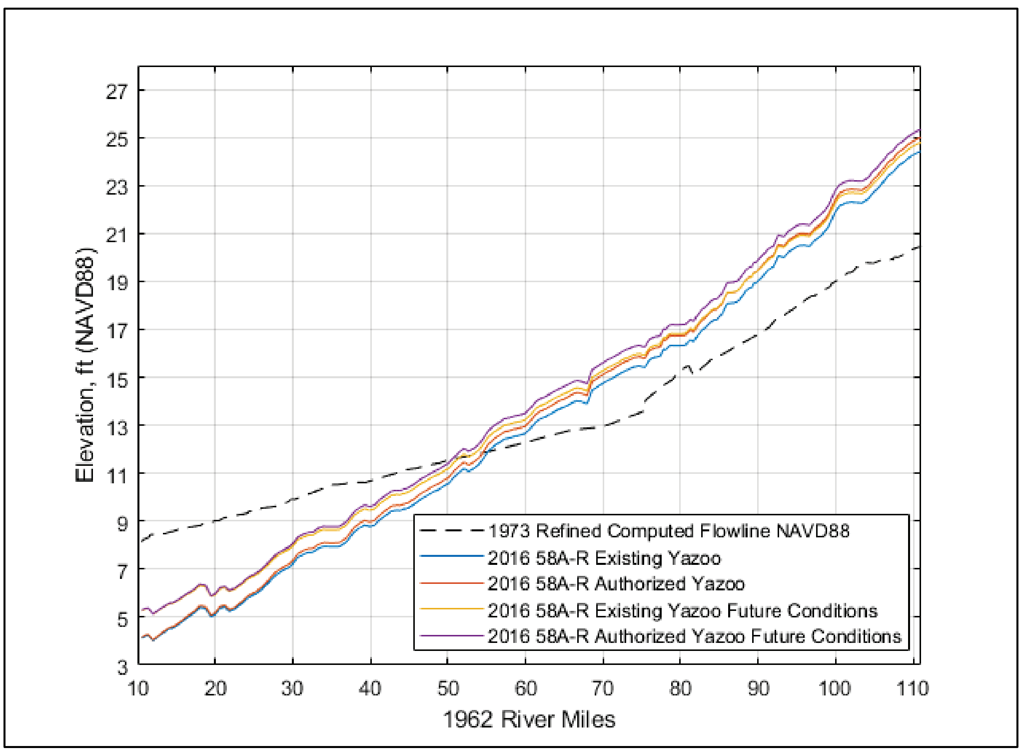


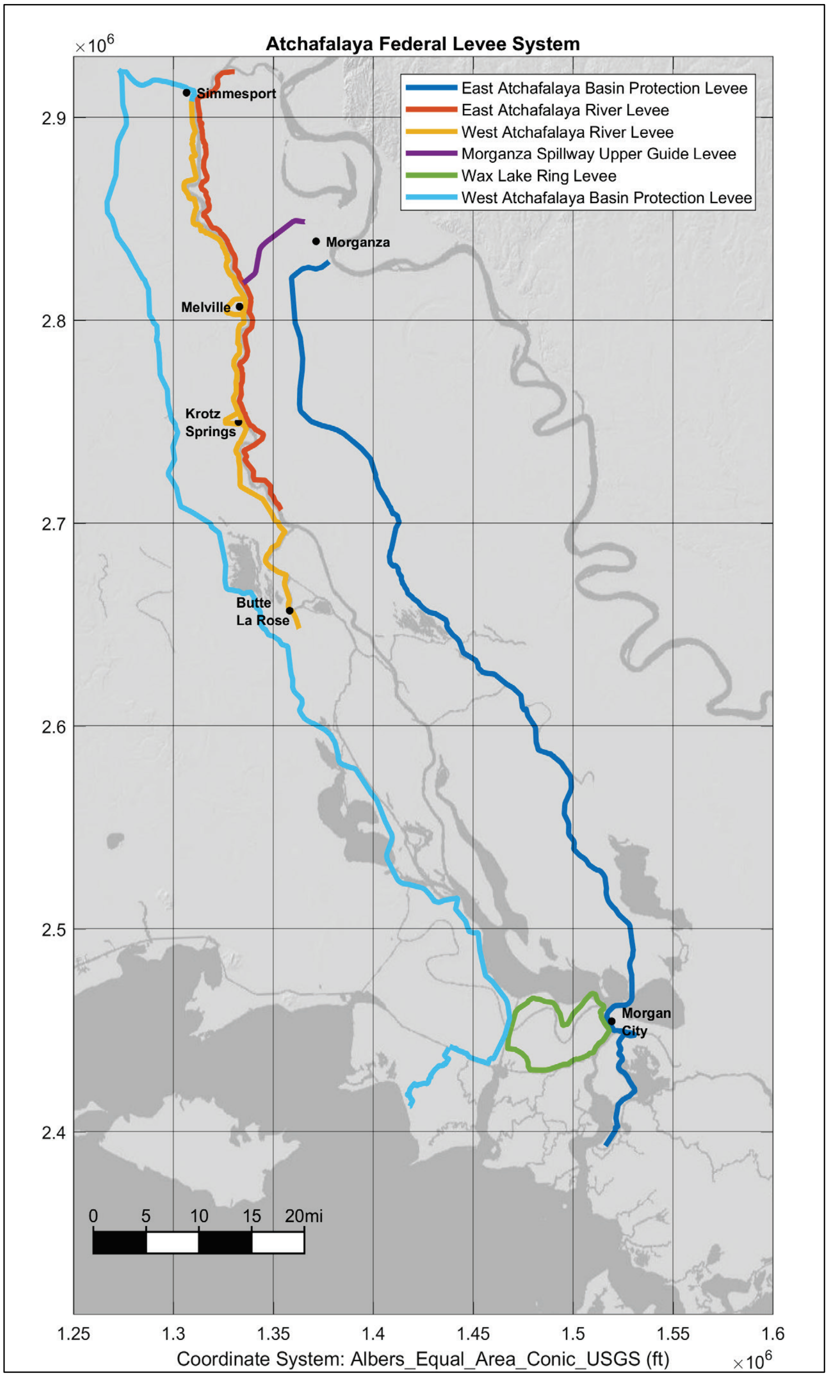
Figure 3-35. Future SLR water surface profiles for RMs 11–110.



3.6 Atchafalaya River

The MVN employed the HEC-RAS (version 5.0.3) (USACE 2016) hydraulic modeling software for the Atchafalaya River from Simmesport, LA, to the Gulf of Mexico. Figure 3-36 displays the Atchafalaya River federal levee system from Simmesport to the Gulf of Mexico.

Figure 3-36. Map of federal levees of the Atchafalaya Basin.



3.6.1 HEC-RAS geometry

The model domain includes the entire Atchafalaya floodway. A schematic of the hydraulic model domain is shown in Figure 3-37. The 2D mesh contains 41,632 cells. The 1D portion contains 286 cross sections, 512 lateral structures, 7 junctions/splits. The 1D cross sections are drawn bank to bank to effectively capture conveyance within the channel. Lateral structures drawn along the natural or artificial levees to link the 1D rivers to the 2D areas. The overbank areas are modeled as 2D flow areas. The vertical datum being used for the model is NAVD 88. The lateral extent of the model was limited by the basin levees. This does not allow flow out of the system should levees overtop. The system was not overtopped in any of the hindcast simulations. The model is specifically set up to not allow overtopping of the levee system to properly calculate design water surface elevations.

Figure 3-38 displays the various sources of bathymetric/topographic data used in the Atchafalaya RAS model. The primary source of topographic information for the model comes from the 2013 USGS LIDAR (NOAA 2013). The primary source of bathymetric data for the Atchafalaya River is the 2010 multibeam sonar dataset. A mix of other terrain sources covers the remaining model domain. In areas where no bathymetric surveys were available, the high resolutions ADCIRC SL17 mesh was used. Some channel surveys for the wax lake outlet were also applied to the model. Table 3-10 contains information on the various terrain data sources applied in the model. The terrain was constructed in such a way to allow the bathymetric surveys to overwrite the water surface that was captured in the lidar. RasMapper was used to mosaic the various terrain sources into one terrain. Figure 3-39 displays a zoomed-in view of the model geometry showing the various 1D and 2D features, as well as the resulting river analysis system (RAS) terrain.

Two different terrains were created for the project. One terrain was developed mainly to map elevations to the lateral structures. Another terrain was developed purely for the 2D modeling. The terrain developed for 2D modeling includes bathymetry at all major channels.

Table 3-10. Terrain data sources.

Terrain Data	Source	Spatial Resolution
Atchafalaya River Multibean SONAR 2010	USACE, New Orleans District	2 ft
Atchafalaya River Levee Lidar 2007	USACE, New Orleans District	1 ft
ADCIRC SL17 Bathymetry	University of Notre Dame Computational Hydraulics Laboratory	20 ft
Atchafalaya LIDAR 2013	Northrop Grumman, Advanced GEOINT Solutions Operating Unit	1 meter (m)
Louisiana LIDAR (FEMA) 1999	FEMA and CEMVN	5 m

3.6.1.1 RAS geometry details

Manning's *n*-values were obtained from a few sources including previous modeling studies and direct calibration of the 2011 event. The 2D area Manning's *n*-values are mapped from the National Land Cover Database (NLCD) 2011 Land Cover Dataset (Homer et al. 2015) using the values contained in Table 3-11.

Table 3-11. Manning's *n*-values by landcover.

Value	Description	*n*-value
11	Open Water	0.022
21	Developed, Open Space	0.12
22	Developed, Low Intensity	0.121
23	Developed, Medium Intensity	0.05
24	Developed, High Intensity	0.05
31	Barren Land	0.05
41	Deciduous Forest	0.16
42	Evergreen Forest	0.18
43	Mixed Forest	0.17
52	Shrub/Scrub	0.07
71	Grassland/Herbaceous	0.035
81	Pasture/Hay	0.033
82	Cultivated Crops	0.04
90	Woody Wetlands	0.14
95	Emergent Herbaceous Wetlands	0.035

No bridges or inline structures were modeled in the 1D geometry; however, the 2D model accounts for major bridge abutments and other raised features. Levees were modeled as lateral structures connecting the 1D model to the overbank areas. Breaklines were drawn at significant raised features that might impede flow in the basin. No culverts under roadways were modeled. The 2D areas were initially created on a 1500 × 1500 ft cell spacing, although further refinements with breaklines increased resolution in certain areas.

3.6.2 Boundary conditions

Flows entering the Atchafalaya Basin originate from the 70/30 split at the ORCC. During extreme floods, flows also enter the basin at the Morganza Spillway. Local rainfall is also another source of flow, although it was not accounted for in this analysis.

The upstream boundary of the Atchafalaya model is located slightly upstream from the railroad bridge at Simmesport, LA. The downstream boundary is in the Gulf of Mexico, south of the Wax Lake and Atchafalaya Deltas. The downstream boundary is set far enough into the Gulf to not be influenced by river conditions. The hydraulic boundary conditions needed to run the model include a stage hydrograph at the Gulf of Mexico, a flow hydrograph at Simmesport, and a flow hydrograph at the Morganza Spillway. For all simulations, the required boundary condition time series are stored in HEC Data Storage System (DSS) files.

For simulation of actual events, measured flows at Simmesport and Morganza were obtained from the USGS and USACE. The downstream water levels were supplied from USACE.

Figure 3-37. ORCC to Morgan City, LA (MVN), hydraulic model schematic.

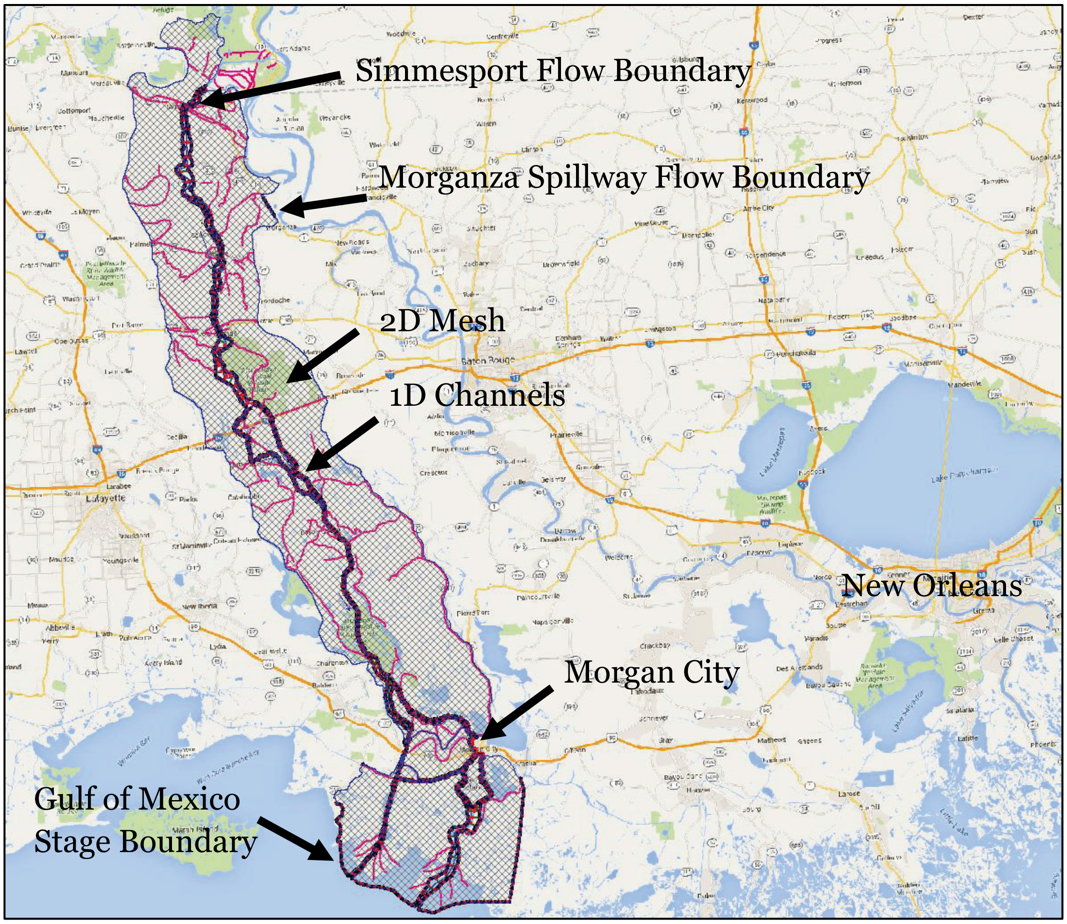


Figure 3-38. Terrain data sources of the Atchafalaya Model.

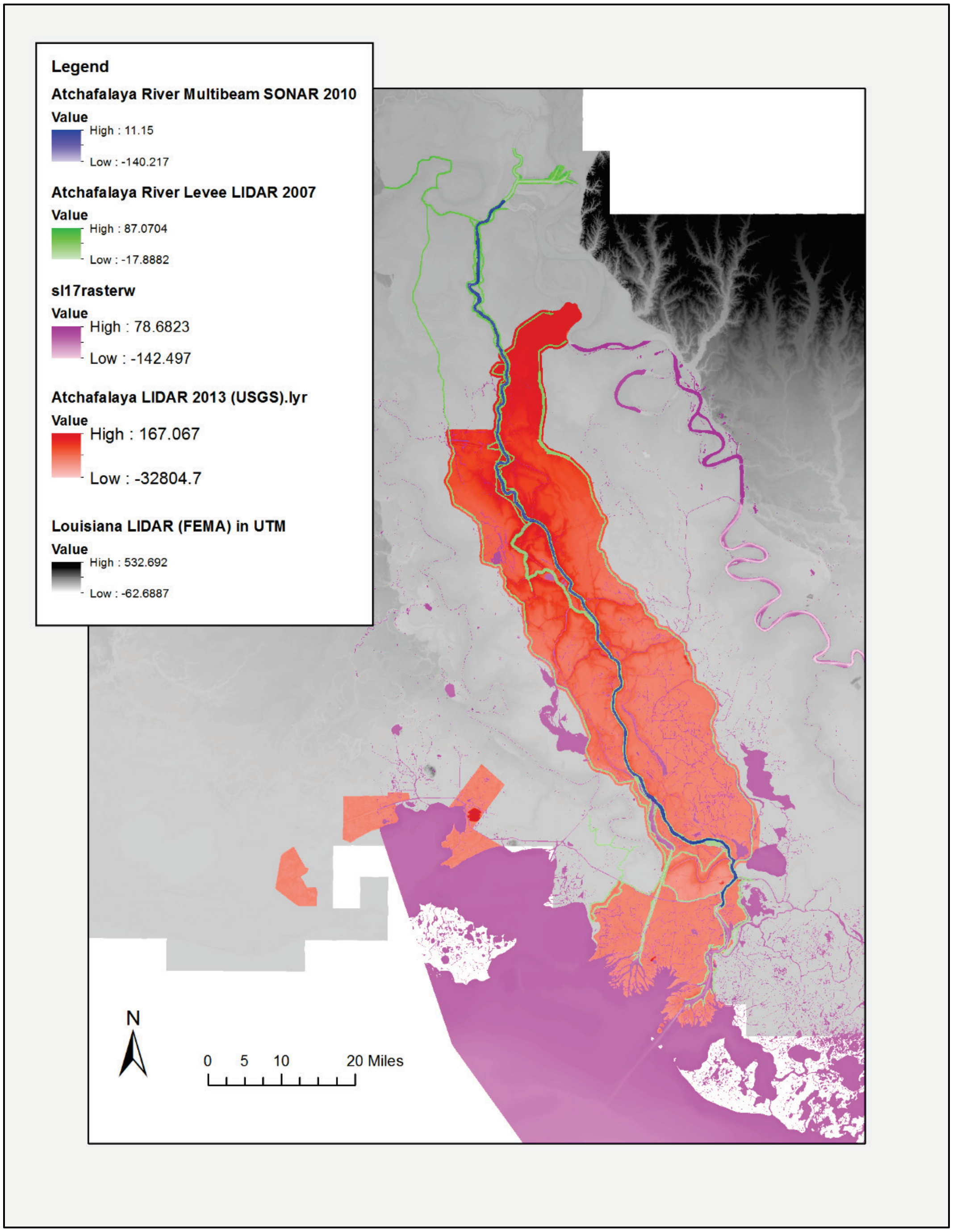
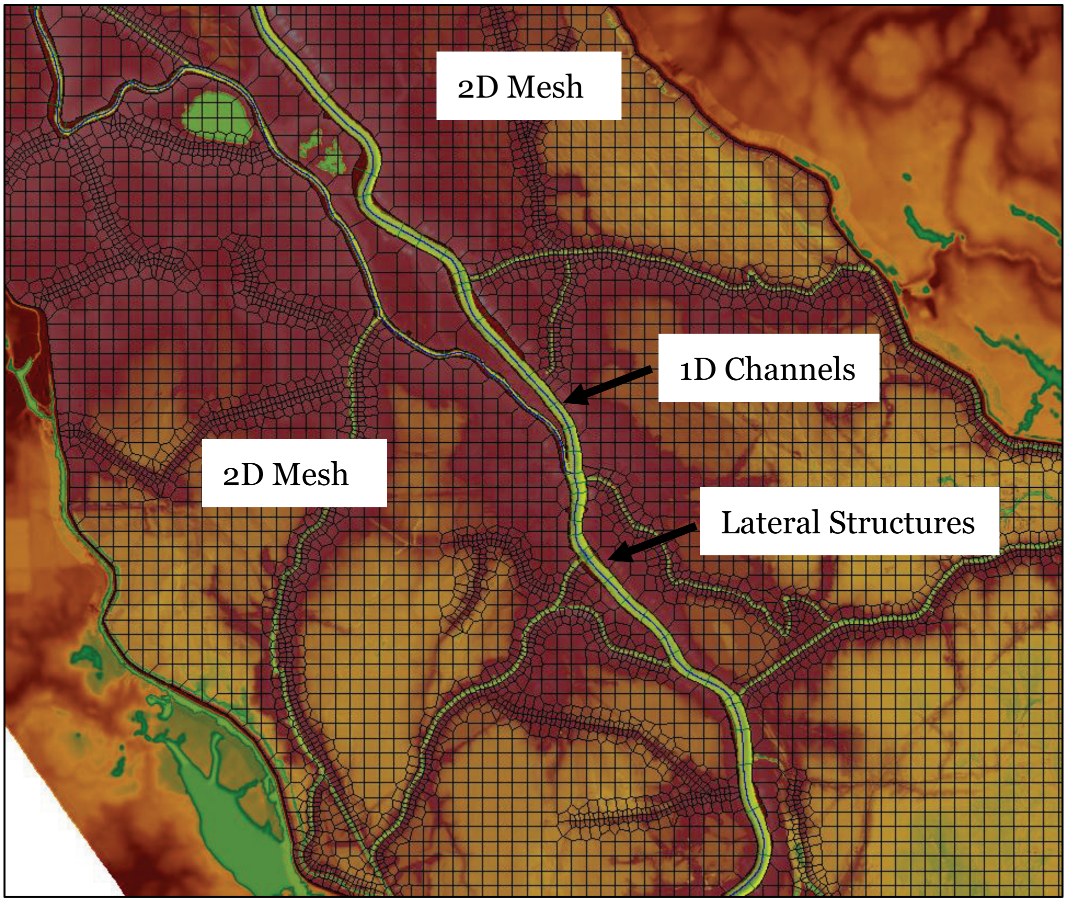


Figure 3-39. Zoomed view of 1D/2D RAS geometry.



3.6.3 Flow and stage calibration and validation

For the external and internal boundaries of the Atchafalaya River unsteady flow HEC-RAS model, flow and stage data were obtained from the USGS and the USACE. The gage measurements were adjusted to the NAVD 88 datum. Table 3-12 contains the gage name, RAS station, and whether the gage reports stage and/or flow data. Figure 3-40 displays the locations of the gages. The Simmesport gage was used as an inflow boundary condition at the upper end of the model. Additionally, for the 2011 calibration event, flows estimated at the Morganza Spillway were forced at the model boundary. The Morganza Spillway was not operated for the 2002 and 2008 events. Stage and flow measurement gages located at Calumet, LA, and Morgan City, LA, were used in calibration of the model within the model domain. The remainder of the USACE gages only measure stage and were used in calibration.

The following section presents modeled vs. observed data for the 2011 calibration run and the 2002 and 2008 validation runs. These events are the same events as used for calibration/validation in the mainstem of the Mississippi River modeling effort. The 2011 event was selected because it was a major event for the basin. The model performs well at the gages in the basin. Modeled peak water levels at all gages are within +/- 1.38 ft of the measurements, although most gages have a smaller difference. The modeled hydrograph shape is also consistent with the observations. For more information about the calibration and validation results, see the Hydraulics Report (USACE 2018b). Figure 3-41, Figure 3-42, and Figure 3-43 display the modeled maximum water surface elevation within the basin for the 2002, 2008, and 2011 simulations.

Table 3-12. Summary of stage and flow gages used in calibration/validation.

River	Station Name	RAS Station	Parameter	Entity
Atchafalaya	Simmesport	Atch 7 732010.6	Stage/Flow	USACE/USGS
Atchafalaya	Morganza Diversion	2D Boundary Condition	Flow	USACE
Atchafalaya	Melville	Atch 7 608311.8	Stage	USACE
Atchafalaya	Krotz Springs	Atch 7 561101.4	Stage	USGS
Atchafalaya	Whiskey Bay Pilot Channel	Atch Split 125437.1	Stage	USACE
Atchafalaya	Butte La Rose	Atch Split 76965.38	Stage	USACE
Atchafalaya	Chicot Pass near Myette Point	Atch 5 256773	Stage	USACE
Atchafalaya	Six Mile Lake	Wax Lake 2 130064.4	Stage	USACE
Atchafalaya	Morgan City	Atch 4 138318.5	Stage/Flow	USACE
Wax Lake Outlet	Calumet	Wax Lake 2 130064.4	Stage/Flow	USACE
Wax Lake Outlet	Crewboat Channel	Wax Lake 1 38069.36	Stage	USACE
Atchafalaya	Avoca Island Cutoff	Atch 1 68558.72	Stage	USACE
Atchafalaya	Eugene Island	Downstream Stage Boundary	Stage	USGS

Figure 3-40. Location of stage and flow gages.

Figure 3-41. Maximum water surface elevation for the 2002 Flood.

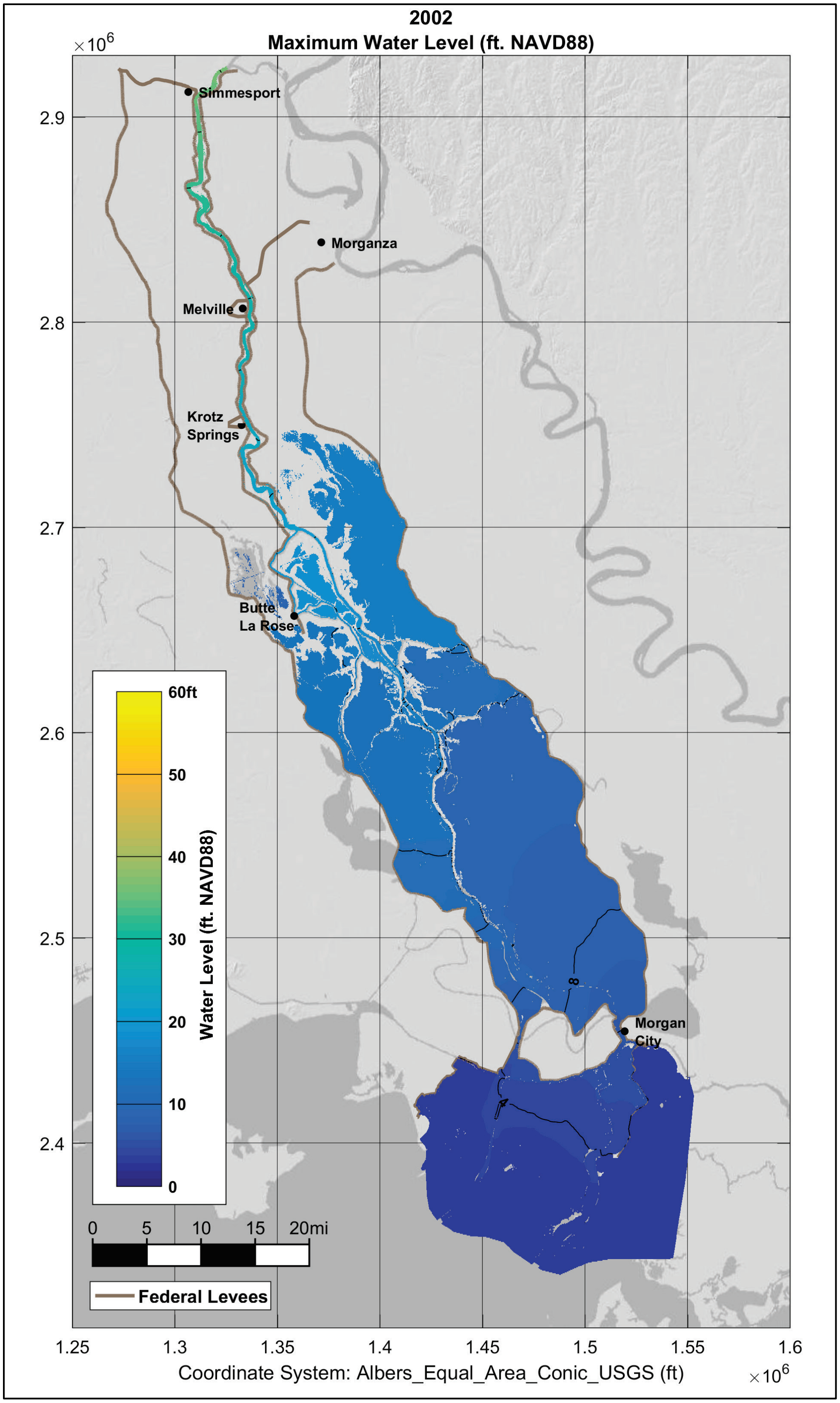


Figure 3-42. Maximum water surface elevation for the 2008 flood.

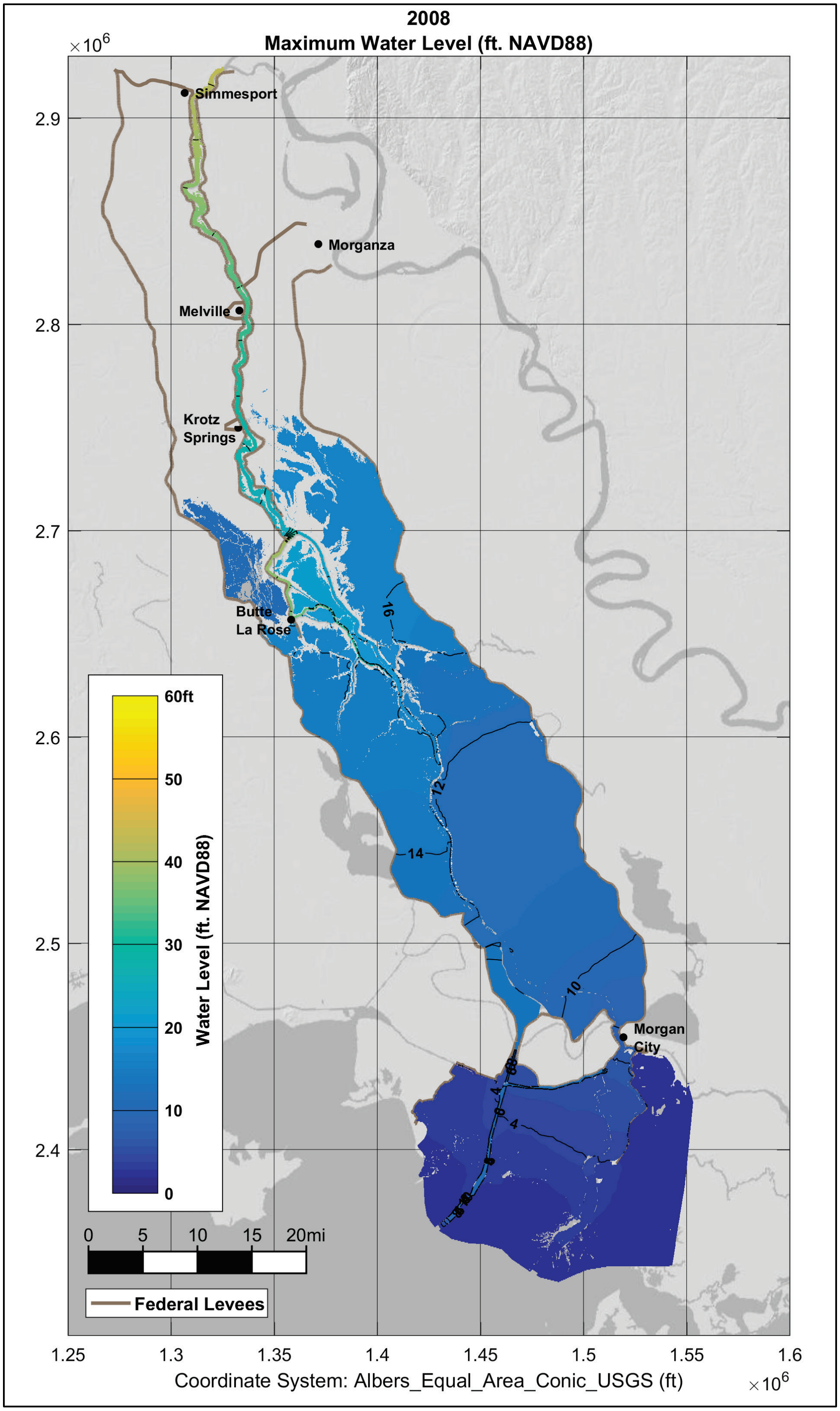
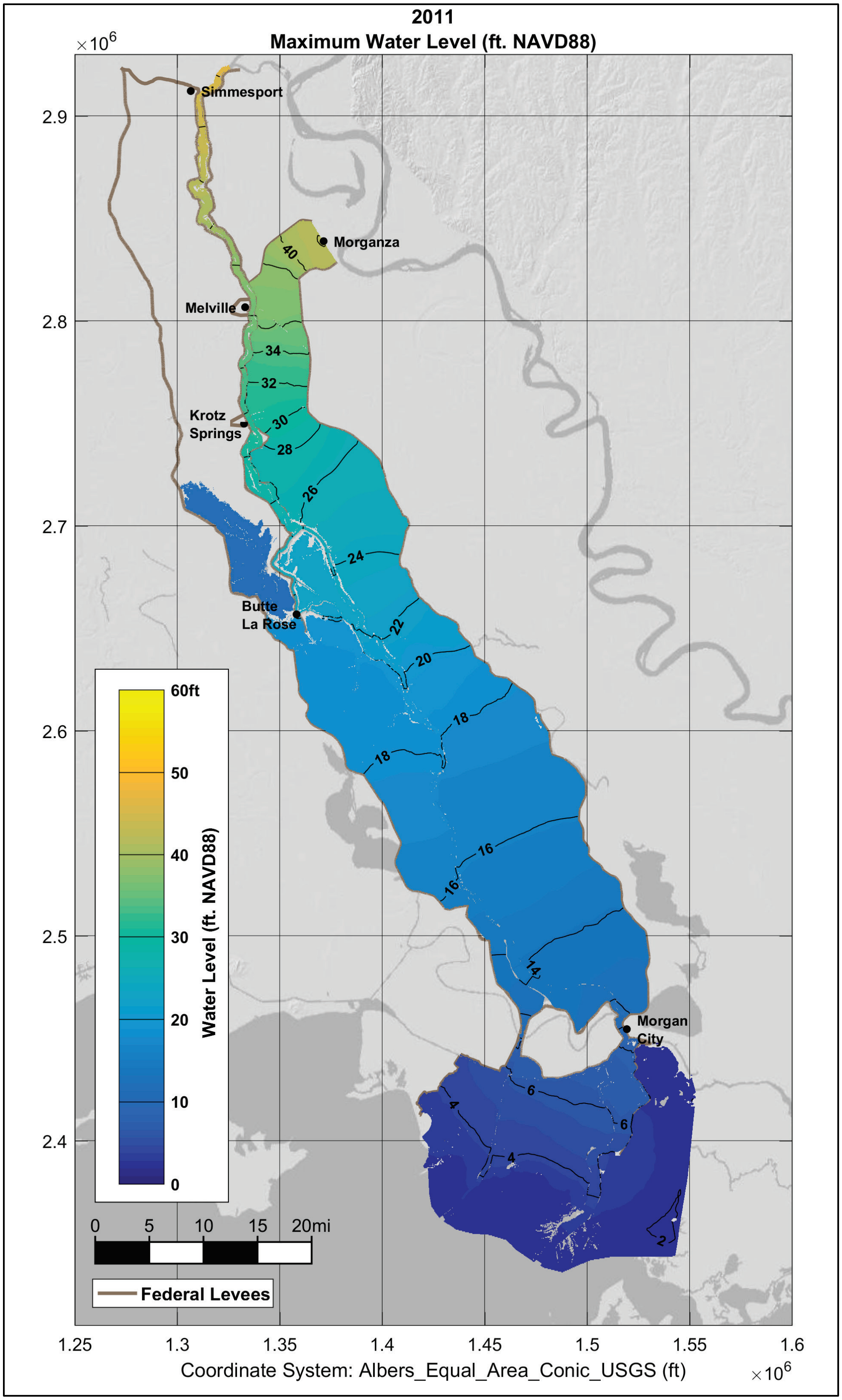


Figure 3-43. Maximum water surface elevation for the 2011 flood.



3.6.4 Simulation of hypothetical design events

Twenty-six hypothetical flood events were simulated with the calibrated Atchafalaya River HEC-RAS model. See the Hydraulics Report (USACE 2018b) for more description of each hypothetical event. A list of all plans simulated is provided below. For the Atchafalaya, a special focus was assigned to the “2016 58A-R Authorized Yazoo” and “2016 58A-R Existing Yazoo” plans. These plans produce the most extreme water levels for the basin.

For the simulation of the hypothetical events, flows were provided from the Mississippi River HEC-RAS model at the Morganza Floodway and the Simmesport flow boundary. For simulation of existing condition hypothetical events, the stage boundary at the Gulf was set to 0.9 ft NAVD 88. For future conditions, the elevation of the Gulf was assumed to be 3.3 ft NAVD 88. These values represent a mean Gulf water surface elevation and are consistent with the mainstem Mississippi River model. The Hydraulics Report (USACE 2018b) also includes a sensitivity analysis on the effects of the Atchafalaya River’s downstream boundary.

The PDF events are much larger than the 2011 calibration event. Note that the HEC-RAS model has more uncertainty for larger events, especially considering the fact that there are no PDF-scale events in the historic record.

3.6.4.1 Existing condition simulations

- Forced 58AEN
- Existing 58AEN
- Authorized 58AEN
- 2016 Unreg YBW at 112.8
- 2016 Unreg Existing YBW
- 2016 Reg YBW at 112.8
- 2016 Reg Existing YBW
- 1955 Historic 58 AEN Authorized Yahoo
- 1955 Historic 58 AEN Existing Yahoo
- 2016 58A-R Authorized Yazoo
- 2016 58A-R Existing Yazoo
- 2016 58A-U Authorized Yazoo
- 2016 58A-U Existing Yazoo
- 2016_52-R_Authorized_Yazoo

- 2016_52-R_Existing_Yazoo
- 2016_52-U_Authorized_Yazoo
- 2016_52-U_Existing_Yazoo
- 2016_56-R_Authorized_Yazoo
- 2016_56-R_Existing_Yazoo
- 2016_56-U_Authorized_Yazoo
- 2016_56-U_Existing_Yazoo
- 2016_63-R_Authorized_Yazoo
- 2016_63-R_Existing_Yazoo
- 2016_63-U_Authorized_Yazoo
- 2016_63-U_Existing_Yazoo
- 2016 Concept Existing (see USACE Hydraulics 2017)

Figure 3-44 displays the resulting maximum water surface for simulation of the 2016 58A-R Authorized Yazoo plan. Figure 3-45 displays the maximum water surface of the 2016 58A-R Existing Yazoo plan. The simulations appear to provide realistic inundation extent and depth for the hypothetical events. Table 3-13 contains the results for the two critical case hypothetical events. The two 58A-R plans produce approximately equivalent water surface elevations.

Figure 3-44. Maximum water surface elevation of the 2016 58A-R Authorized Yazoo plan.

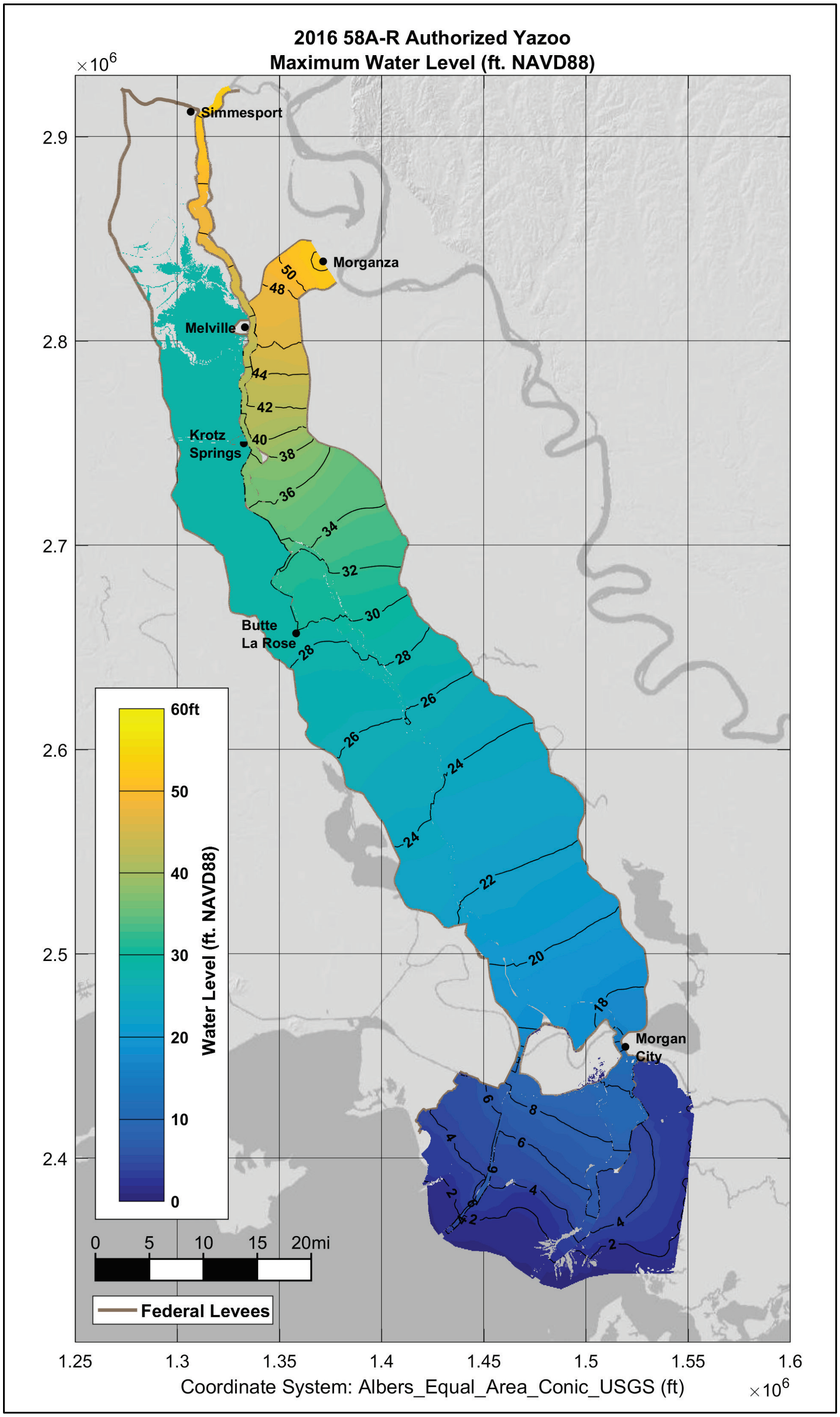


Figure 3-45. Maximum water surface elevation of the 2016 58A-R Existing Yazoo plan.

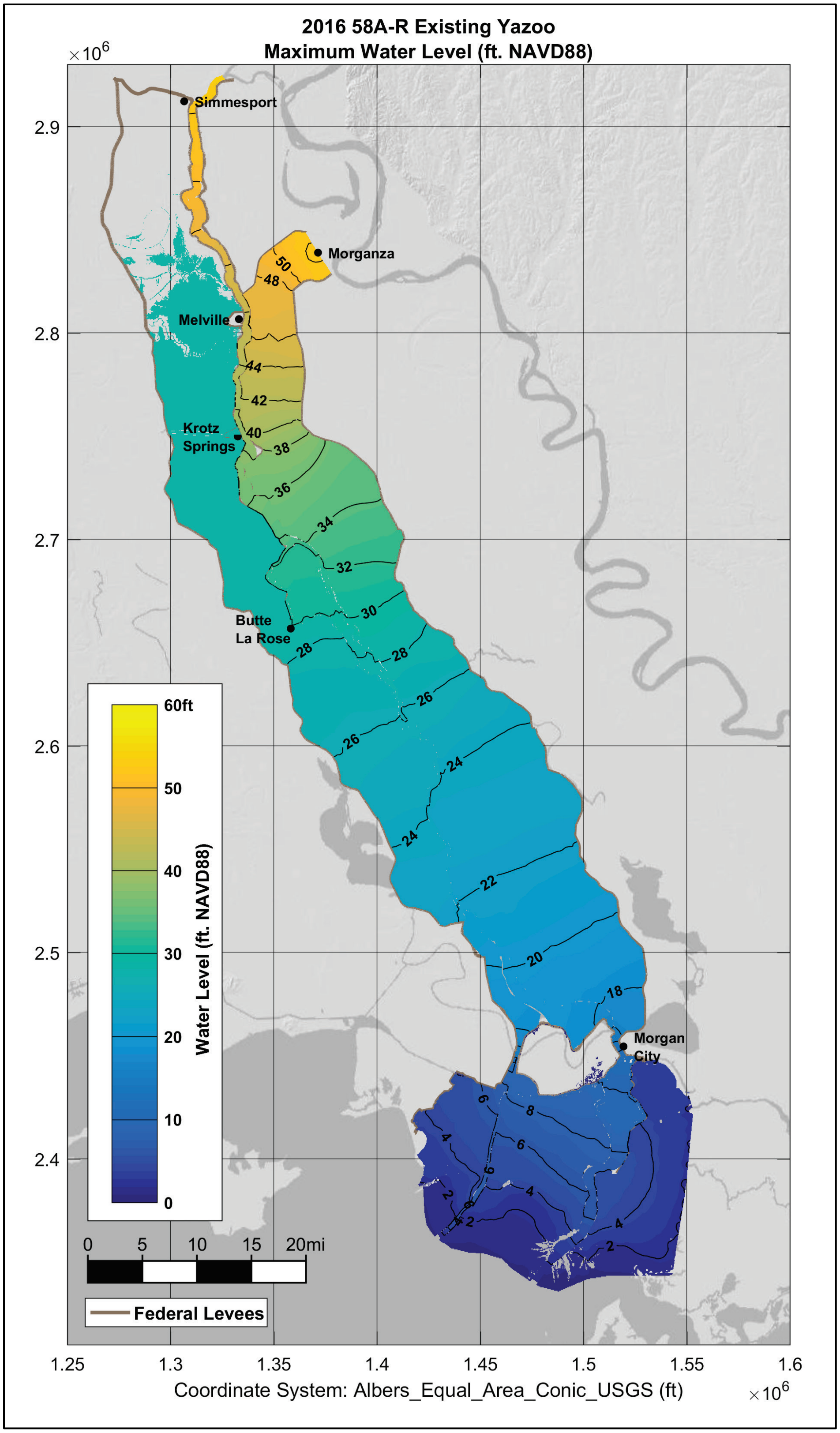


Table 3-13. Summary of hypothetical event maximum water surface elevations for existing conditions.

River	Station Name	RAS Station	2016 58A-R Authorized Yazoo	2016 58A-R Existing Yazoo
Atchafalaya	Simmesport	Atch 7 732010.6	52.18	52.47
Atchafalaya	Melville	Atch 7 608311.8	43.49	43.72
Atchafalaya	Krotz Springs	Atch 7 561101.4	40.05	40.22
Atchafalaya	Whiskey Bay Pilot Channel	Atch Split 125437.1	32.87	32.95
Atchafalaya	Butte La Rose	Atch Split 76965.38	29.83	29.9
Atchafalaya	Chicot Pass near Myette Point	Atch 5 256773	21.6	21.68
Atchafalaya	Six Mile Lake	Wax Lake 2 130064.4	19.06	19.12
Atchafalaya	Morgan City	Atch 4 138318.5	11.56	11.62
Wax Lake Outlet	Calumet	Wax Lake 2 99864.52	14.92	14.97
Wax Lake Outlet	Crewboat Channel	Wax Lake 1 38069.36	6.77	6.78
Atchafalaya	Avoca Island Cutoff	Atch 1 68558.72	6.69	6.73

3.6.4.2 Future condition geometry

To account for future condition bed changes within the basin, a HEC-6T model was developed to simulate scour and deposition over a 50-year time period. Table 3-14 lists the total bed change applied to the various reaches of the HEC-RAS model. The fourth column of Table 3-14 represents the bed change caused by 50 years of sedimentation, approximately 2016 to 2066. The fifth column is the change from that point (2066 numbers) to the peak of the PDF event. The bed change in the fifth column is caused by the hydrograph of the PDF event itself. Since the HEC-RAS model is using a fixed-bed geometry, it was necessary to include an estimate of the bed changes that happen during the flood. The two bed change values are added together to get the expected bed change from 2016 to the peak of a PDF event occurring in the year 2067. Reaches that are not listed (Gulf Intracoastal Waterway [GIWW] and A21-99), were not simulated in the 6T model, so no bed change was applied to the model in these areas. A Matlab script was developed to modify the geometry file and apply the bed change to each individual cross-section. Figure 3-46 displays a future condition cross section in the Atchafalaya River with a +9.5 ft bed adjustment. Figure 3-47 displays a cross section with the -11.5 ft bed adjustment The Matlab script was set to only modify bathymetry between the banks. No changes were made to the elevations of overbank areas, including the terrain applied to the 2D areas.

3.6.4.3 Future condition simulations

Three plans were simulated using the future condition geometry:

- 2016 58A-R Existing Yazoo Future
- 2016 58A-R Authorized Yazoo Future
- 2016 58A-R Concept Future (see USACE [2018b]).

The results of each future condition plan were compared to the corresponding existing condition simulation. Figure 3-48 and Figure 3-49 show the water surface elevation profiles for the existing and future condition scenarios. In general, the trend in water surface elevation change appears to be somewhat counter intuitive. The future condition water level increases near the Gulf due to SLR. Near Morgan City, the water level actually decreases in the future condition, most likely because the cross sections downstream were lowered 11.5 ft. Figure 3-50 through Figure 3-51 display the existing and future condition water level time series at Simmesport and Calumet. Table 3-15 contains the maximum water surface elevations at various locations for the 58A-R Existing Yazoo and 58A-R Authorized Yazoo hypothetical events for future conditions.

Table 3-14. Future condition bed change adjustments.

River	Upstream Cross Section	Downstream Cross Section	Bed Adjustment from 10/1/2010 to 10/1/2066 (ft)	Bed Adjustment from 10/1/2066 to the peak of the PDF (ft)	Total Bed Adjustment Applied to Model (ft)
Atchafalaya (Atch)	18957.01	2719.914	1	-4	-3
Atch	25881.71	25881.71	2.7	0	2.7
Atch	41382.14	33563.18	2.8	0	2.8
Atch	57287.42	47666.67	2.8	-4.5	-1.7
Atch	63075.32	63075.32	8.2	-4.5	3.7
Atch	94485.92	68558.72	4.8	-4.5	0.3
Atch	120513.6	98835.9	5	-12.5	-7.5
Atch	125947	123105.8	5	-25.5	-20.5
Atch	134315.2	128249.9	5.7	-25.5	-19.8
Atch	143623.7	135620.3	0	-9	-9
Atch	156948.7	146057.3	6	-16.5	-10.5
Atch	162338.7	159741.5	6	-0.5	5.5
Atch	227227.7	165558.3	10.5	-0.5	10
Wax Lake	8237.124	1445.184	0	0	0
Wax Lake	32561.78	26764.01	0.5	-6.5	-6
Wax Lake	38069.36	38069.36	-5.3	-2.5	-7.8
Wax Lake	48692.56	41565	-5.3	-1	-6.3
Wax Lake	79657	56100.93	-5.3	0	-5.3
Wax Lake	97192.81	81922.72	0	0	0
Wax Lake	99864.52	99864.52	-4	0	-4
Wax Lake	112305.5	104402.8	-10	0	-10
Wax Lake	155746	113545.4	-0.7	0	-0.7
Atch	243947.4	230375.6	8.2	0	8.2
Atch	306553.2	246498.7	6.8	0	6.8
Atch	398154.3	310726.6	4	0	4
Atch Split	28319.64	2874.749	4	0	4
Atch	470193.8	400592.8	4	4	8
Atch Split	125437.1	33140.38	2	0	2
Atch	751747.4	475518.4	3.5	-1.5	2

Figure 3-46. Future condition cross section with accretion.

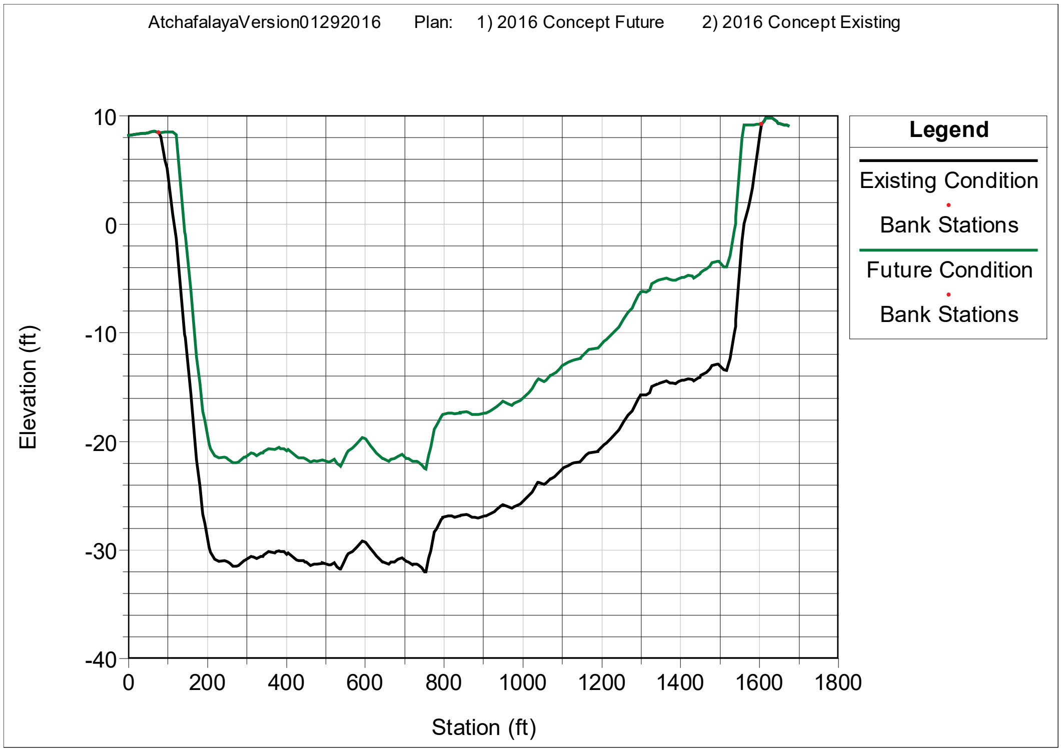


Figure 3-47. Future condition cross section with scour.

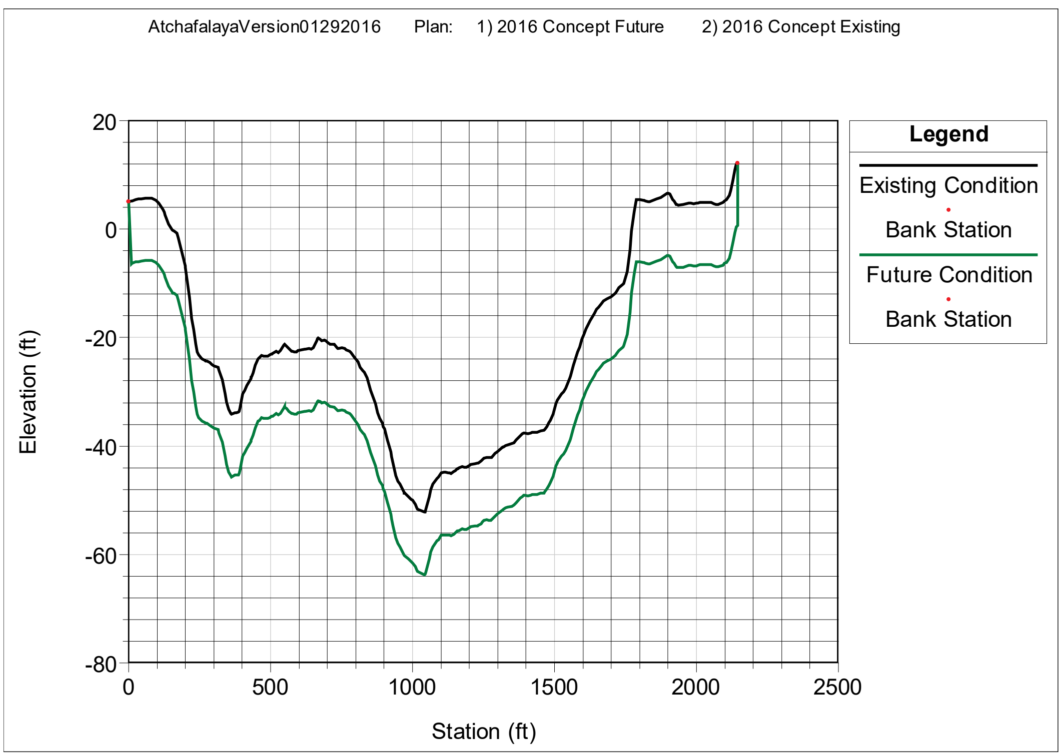


Figure 3-48. Comparison of existing and future condition bed change and peak water surface for the 58 A-R Authorized Yazoo Plan.

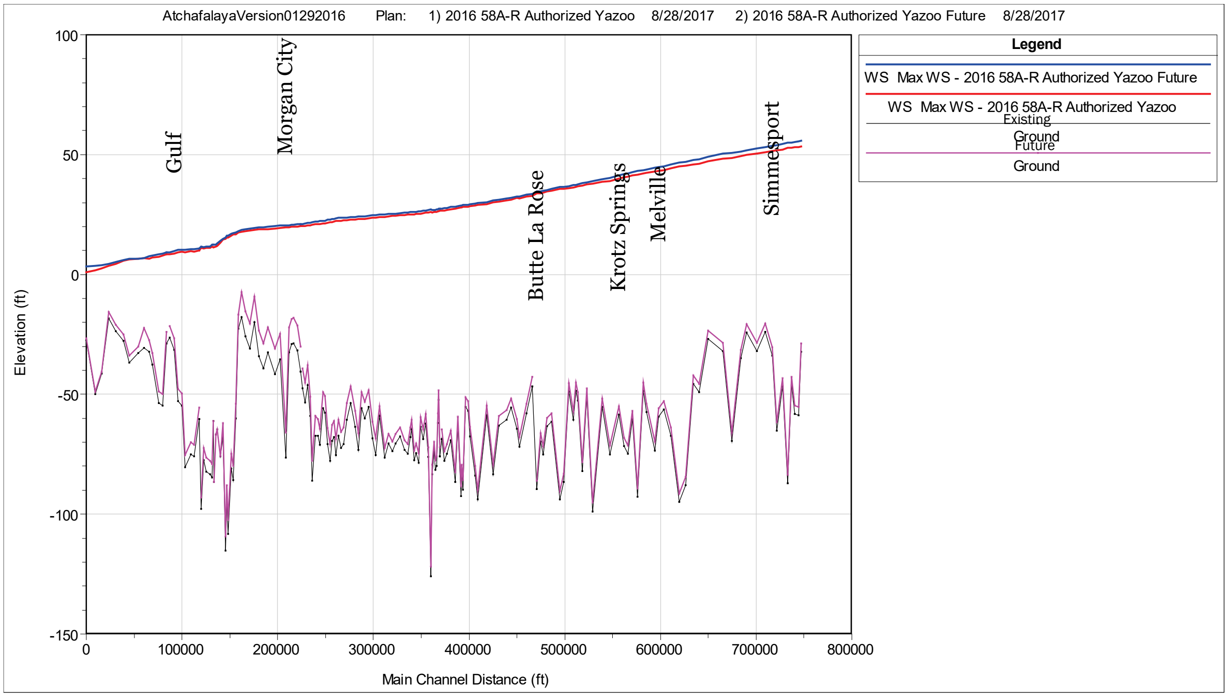


Figure 3-49. Comparison of existing and future condition bed change and peak water surface for the 58 A-R Existing Yazoo Plan.

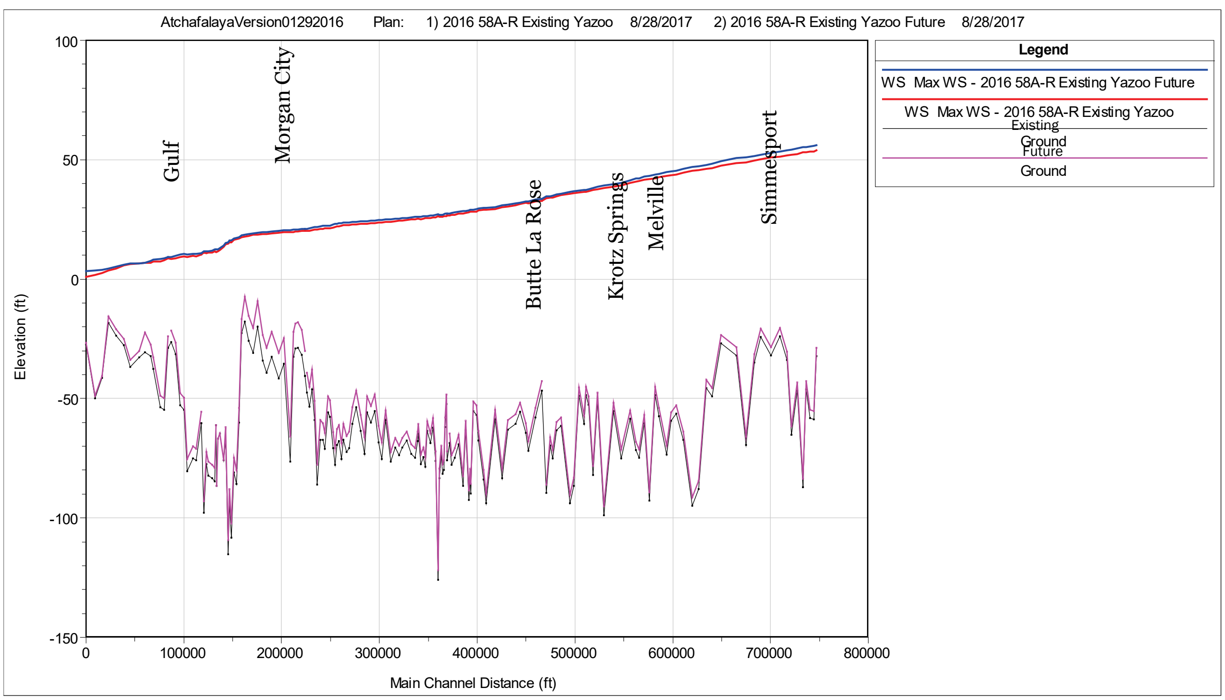


Figure 3-50. 58A-R modeled stages at Simmesport, LA.

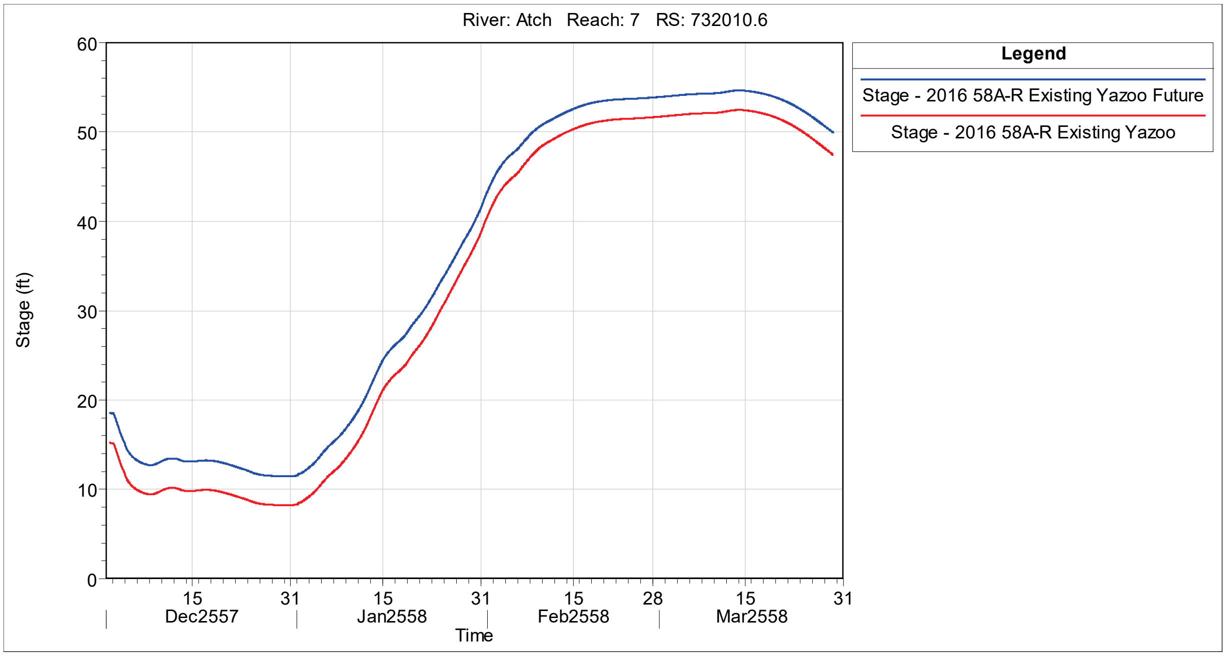


Figure 3-51. 58A-R modeled stages at Calumet.

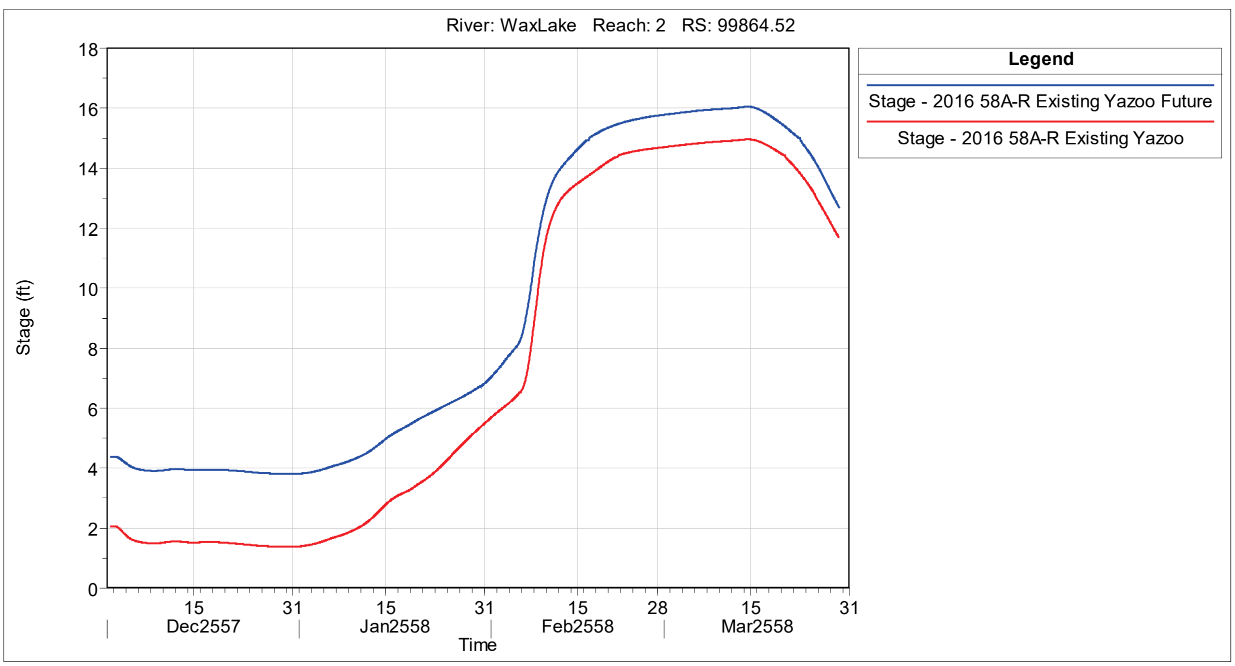


Table 3-15. Summary of hypothetical maximum water surface elevations for future conditions.

River	Station Name	RAS Station	58A-R Authorized Yazoo Future	58A-R Existing Yazoo Future
Atchafalaya	Simmesport	Atch 7 732010.6	54.38	54.66
Atchafalaya	Melville	Atch 7 608311.8	45.06	45.28
Atchafalaya	Krotz Springs	Atch 7 561101.4	41.32	41.47
Atchafalaya	Whiskey Bay Pilot Channel	Atch Split 125437.1	34.38	34.4
Atchafalaya	Butte La Rose	Atch Split 76965.38	30.64	30.66
Atchafalaya	Chicot Pass near Myette Point	Atch 5 256773	22.84	22.91
Atchafalaya	Six Mile Lake	Wax Lake 2 130064.4	19.95	20.03
Atchafalaya	Morgan City	Atch 4 138318.5	12.45	12.49
Wax Lake Outlet	Calumet	Wax Lake 2 99864.52	15.98	16.05
Wax Lake Outlet	Crewboat Channel	Wax Lake 1 38069.36	6.9	6.9
Atchafalaya	Avoca Island Cutoff	Atch 1 68558.72	7.62	7.64

3.6.5 Comparison to 2010 flowline analysis

In 2010, a separate flowline analysis was conducted for the Atchafalaya River. The purpose of this section is to compare the results from the 2010 analysis to the current (2016) analysis. Table 3-16 shows some of the significant differences in methodology between the older and newer studies. A comparison was made between the 2010 flowline water levels and the latest 2016 58 A-R water levels[1]. Figure 3-52 displays the

[1] The comparison analysis between 2010 and 2016 results was performed in response to comments received after the Independent External Peer Review phase of this assessment.

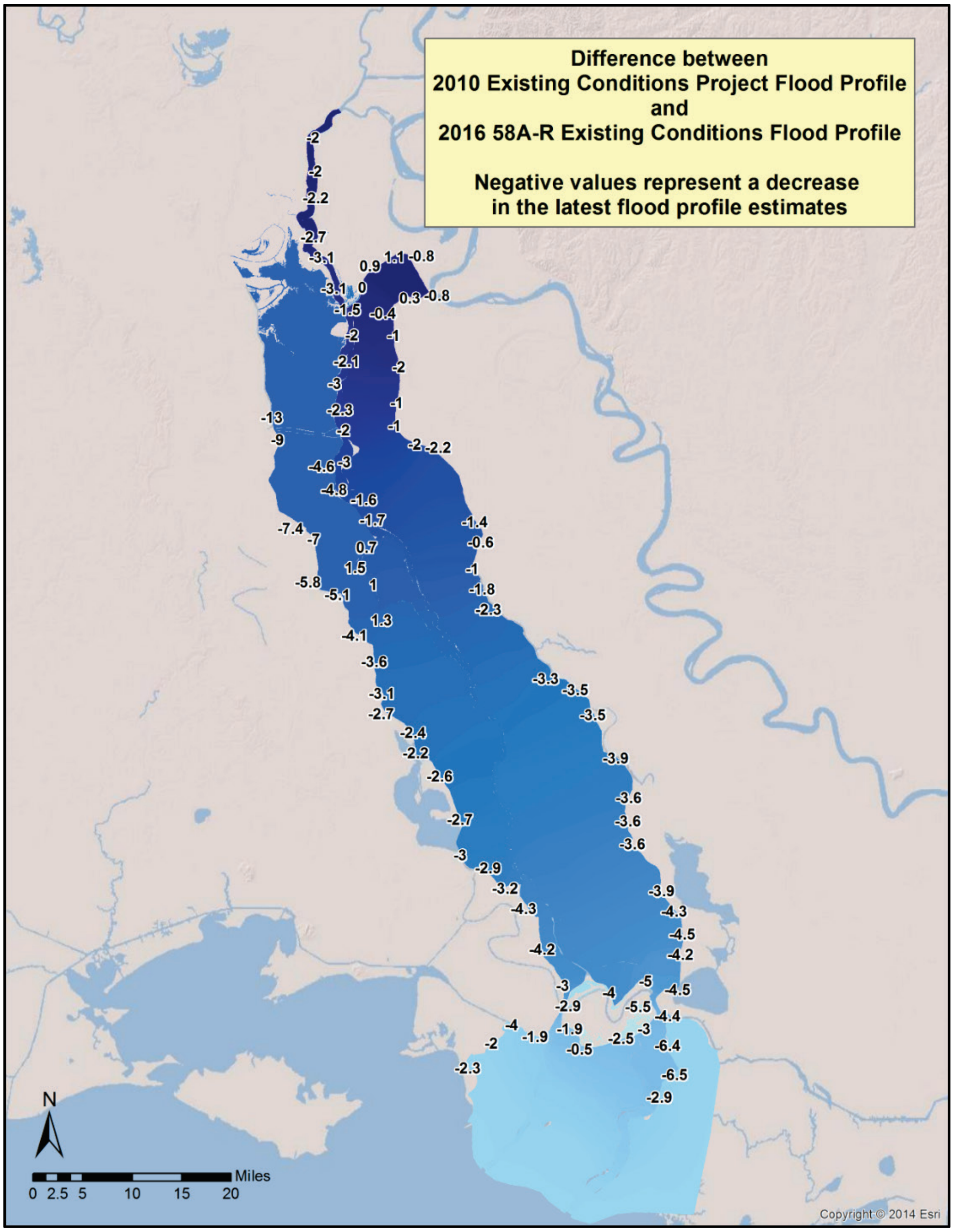
comparison between 2010 and 2016 water levels. In general, the newer values are significantly lower.

Changes to the methodology are one reason for the decrease in values. In addition to modeling differences, channel surveys in recent years have consistently shown wide-spread scouring, or channel lowering, in the Upper Atchafalaya River. In the Lower Atchafalaya Basin there has been a long trend of accretion occurring in the river channel and in the floodplain areas. This filling has caused the stage-flow relationships to increase for all flow regimes. From an uncertainty perspective, both 2010 and 2016 HEC-RAS models calibrated well to large-scale flood events. However, the PDF event is much larger than anything that is observed in the historical record. The uncertainty of the modeling is higher for extreme floods, since there is no way to validate the response of the models for events that have not happened.

Table 3-16. Comparison of 2010 and 2016 Atchafalaya flowline analyses.

	2010 Flowline Analysis	2016 Flowline Analysis (2016 58 A-R)
Geometry	RAS 4.0 1D, AdH model	RAS 5.0.3 1D/2D
Flows	Steady State	Unsteady
	930,000 Simmesport 630,000 Morganza; Includes West Floodway	886,440 peak Simmesport 600,000 peak Morganza; No west floodway.
Downstream Boundary	AdH with 5 ft gulf elevation	0.9 ft gulf elevation
Calibration	1997 (~630,000 cfs at Simmesport)	2011 (~700,000 cfs at Simmesport)
Validation	1998, 2005	2002, 2008

Figure 3-52. Difference between 2010 and 2016 peak flood profiles.



3.6.6 Summary and path forward

The Atchafalaya River HEC-RAS model has been used to evaluate a series of past floods including the 2002, 2008, and 2011 events. The model was also applied to evaluate hypothetical design events. In addition to these tasks, the model was also used during the 2016 flood to forecast stages within the basin. The model will be useful for a variety of future uses.

3.7 Summary

An unsteady flow HEC-RAS model was developed for the Mississippi River from Chester, IL, to Venice, LA (HEC-RAS stations 1163 to 15.07), by combining the individual MVM, MVK, and MVN models. The model was calibrated to the 2011 Mississippi River flood event and validated to the 2002 and 2008 high flow events. The purpose of the modeling effort was to capture the behavior of the Mississippi River during large events. The model developed for this effort performs well in terms of both timing and magnitude of the peak events in each simulation when compared with gage observations.

This model was then used to simulate the PDF event. The historic PDF simulations used the previous hydrographs developed from the 1955 hydrology as inflow boundary conditions to the HEC-RAS model. These simulations resulted in very similar flows occurring throughout the river as was reported previously (USACE 1978), but the stages showed some differences, primarily due to geomorphic channel changes that have occurred since the 1970s. The PDF simulations used the new hydrograph flows developed within this assessment as inflow boundary conditions to the HEC-RAS model (USACE 2018a). 2016 PDF HEC-RAS simulations were performed for the 58A, 52A, 56, and 63 storm events. The 58A storm event produced the highest maximum water surface elevations throughout the river from Cairo, IL, to Venice, LA.

The peak flows of the new hydrology results were higher and occurred sooner than the hydrology results developed in 1955. The 2016 58A-R PDF simulations generally resulted in higher peak stages throughout the Mississippi River HEC-RAS model. Many water surface elevations were approximately 4 ft higher in the 58A-R Authorized Yazoo simulation than the Refined 1973 Flowline Study water surfaces, converted to NAVD 88. Locations with the greatest differences between the Refined 1973 Flowline Study (USACE 1978) and the 58A-R Authorized Yazoo results were just upstream of Memphis and near Baton Rouge.

Future SLR was incorporated into the downstream boundary conditions of the HEC-RAS model to estimate its impact 50 years from now. There was an increase in peak water surface elevations in the lower few hundred miles of the river.

4 Channel Capacity Changes

4.1 Objective

The primary purpose of this study was to determine the long-term effects of sedimentation processes on the PDF flowline for the Mississippi River between Venice, LA, at RM 10.7 and the confluence of the Ohio River at RM 953.8, near Cairo, IL, and for the Atchafalaya River from the confluence of the ORCC outflow channel, upstream of Simmesport, LA, to the Gulf of Mexico through both the Wax Lake Outlet channel and the Lower Atchafalaya River. The HEC-6T numerical sedimentation model was used in this study.

4.2 Sedimentation of the Mississippi River

The primary purpose of this study was to determine the long-term effects of sedimentation processes on the PDF flowline for the Mississippi River between Venice, LA, at RM 10.7 and the confluence of the Ohio River at RM 953.8, near Cairo, IL. In addition, the study determined the effects of scour and deposition, which occur during the passage of the PDF hydrograph, on maximum water surface elevations. The HEC-6T numerical sedimentation model was used in this assessment, as were 1962 river miles.

The numerical model was also used to evaluate specific geomorphic variables to determine which are the most significant in forming the future character of the Mississippi River. The numerical model makes it possible to evaluate the effect of a single independent driving variable by varying that variable in the model and holding all others constant. Historical data mask the effect of individual variables because the driving variables act together at the same time. Effects evaluated included a significant reduction in sediment supply, dikes, cutoffs, SLR, and subsidence.

The Regional Model of the Mississippi River between Pilots Station (RM 8) and Cairo was used as the starting point in developing the Flowline Model. The Regional Model was completed in 2011 and is documented in a report entitled *Numerical Sedimentation Investigation,* *Mississippi River, Cairo to Pilots Station*[1]. The cross-section geometry for the Regional Model was taken from 1988 to 1992 hydrographic surveys. The Regional Model had been calibrated to 2002 conditions (11 years). Modifications were made to the Regional Model to incorporate additional data collected after completion of the original study. Most notably, the new data included (1) flow diversion measurements at several distributaries downstream from New Orleans, (2) flow diversion measurements from the controlled levee breach during the 2011 flood at Bird's Point, which is located at the head of the New Madrid Floodway, and (3) water surface elevations from the 2011 flood. Enhancements from recent HEC-6T numerical model studies of the Mississippi River were also incorporated. The Regional Model was verified by simulating an additional 11 years through 2013.

The Flowline Model study simulated the 1991–2013 hydrograph to obtain an existing condition. These 2013 calculated conditions were then written to a new HEC-6T geometry and sediment file that was used as the initial condition for future predictions. Using the calculated 2013 geometry, the project flood peak water surface elevations were determined using steady state discharges in the numerical model. Then, using the 2013 geometry for the initial condition, a 50-year hydrograph, followed by a steady-state run with the PDF peak discharges was simulated. The difference in these two calculated water surface elevations was taken to be the difference in water surface elevations due to sedimentation in the 50-year period.

4.2.1 Numerical model description

4.2.1.1 HEC-6T description

The HEC-6T program produces a 1D model that simulates the response of the riverbed profile to sediment inflow, bed-material gradation, and hydraulic parameters. The model simulates a series of steady-state discharge events, their effects on the sediment transport capacity at cross sections, and the resulting degradation or aggradation. The program calculates hydraulic parameters using a standard-step backwater method.

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[1] Copeland, R. R. In preparation. *Numerical Sedimentation Investigation: Mississippi River Cairo, IL to Pilots Station, LA*. MRG&P Report. Vicksburg, MS: U.S. Army Engineer Research and Development Center.

HEC-6T is a state-of-the-art program for use in mobile bed channels. The numerical model computations account for all the basic processes of sedimentation: erosion, entrainment, transportation, deposition, and compaction of the bed for the range of particle sizes found in the Mississippi River. The model calculates aggradation and degradation of the streambed profile over the course of a hydrologic event. It does not simulate bank erosion or natural adjustments in channel widths. When applied by experts using good engineering judgment, the HEC-6T program will provide good insight into the behavior of mobile bed rivers. Additional guidance related to model calibration, application, and interpretation can be found in the following references: *Sedimentation in Stream Networks* (Thomas 2016); *Computational Modeling of Sedimentation Processes* (Chapter 14) (Thomas and Chang 2008); and *Guidelines for the Calibration and Application of Computer Program HEC-6* (USACE-HEC 1992).

4.2.1.2 Model network

The MVN, MVK, MVM, and MVS were each responsible for developing the Regional Model in their respective reaches of responsibility along the Mississippi River. The model's geometry is broken into 25 segments as shown in the network schematics in Figure 4-1 and Figure 4-2. The number of cross sections in each segment and the description of the segment boundaries are given in Table 4-1.

Table 4-1. Description of HEC-6T segments.

Segment	Number of Cross Sections	Description	1962 River Miles
1	133	Mississippi River - Pilots Station to Profit Island	-18.0 to 246.3
2	6	Mississippi River - Main Channel (West) Profit Island	246.9 to 251.8
3	6	Mississippi River - Chute Channel (East) Profit Island	246.8 to 248.6
4	27	Mississippi River - Profit Island to Tarbert Landing	253.0 to 306.3
5	72	Mississippi River - Tarbert Landing to Yazoo River	306.3 to 437.3
6	3	Yazoo River	1.5 to 16.7

Segment	Number of Cross Sections	Description	1962 River Miles
7	81	Mississippi River - Yazoo River to Arkansas River	438.4 to 580.8
8	19	Arkansas River	0.6 to 29.0
9	9	Mississippi River - Arkansas River to White River	580.8 to 598.0
10	69	White River	0.0 to 100.0
11	45	Mississippi River - White River to St Francis River	600.1 to 672.6
12	19	St Francis River	0.5 to 17.7
13	38	Mississippi River - St Francis River to Hatchie River	674.7 to 772.8
14	3	Hatchie River	0.1 to 2.0
15	18	Mississippi River - Hatchie River to Obion River	774.0 to 819.4
16	3	Obion River	0.1 to 2.0
17	35	Mississippi River - Obion River to Island 8	820.0 to 910.0
18	5	Mississippi River - East Loop Island 8	910.5 to 914.6
19	5	Mississippi River - West Loop Island 8	910.5 to 914.6
20	9	Mississippi River - Island 8 to Wolf Island	915.6 to 930.8
21	3	Mississippi River - East Loop Wolf Island	931.5 to 934.4
22	3	Mississippi River - West Loop Wolf Island	931.5 to 934.4
23	8	Mississippi River - Wolf Island to Cairo	934.4 to 953.0
24	37	Ohio River – Cairo to Metropolis	953.0 to 990.0
25	46	Upper Mississippi River - Cairo to Chester	0.8 to 109.9

Figure 4-1. Schematic of HEC-6T stream network in MVN and MVK.

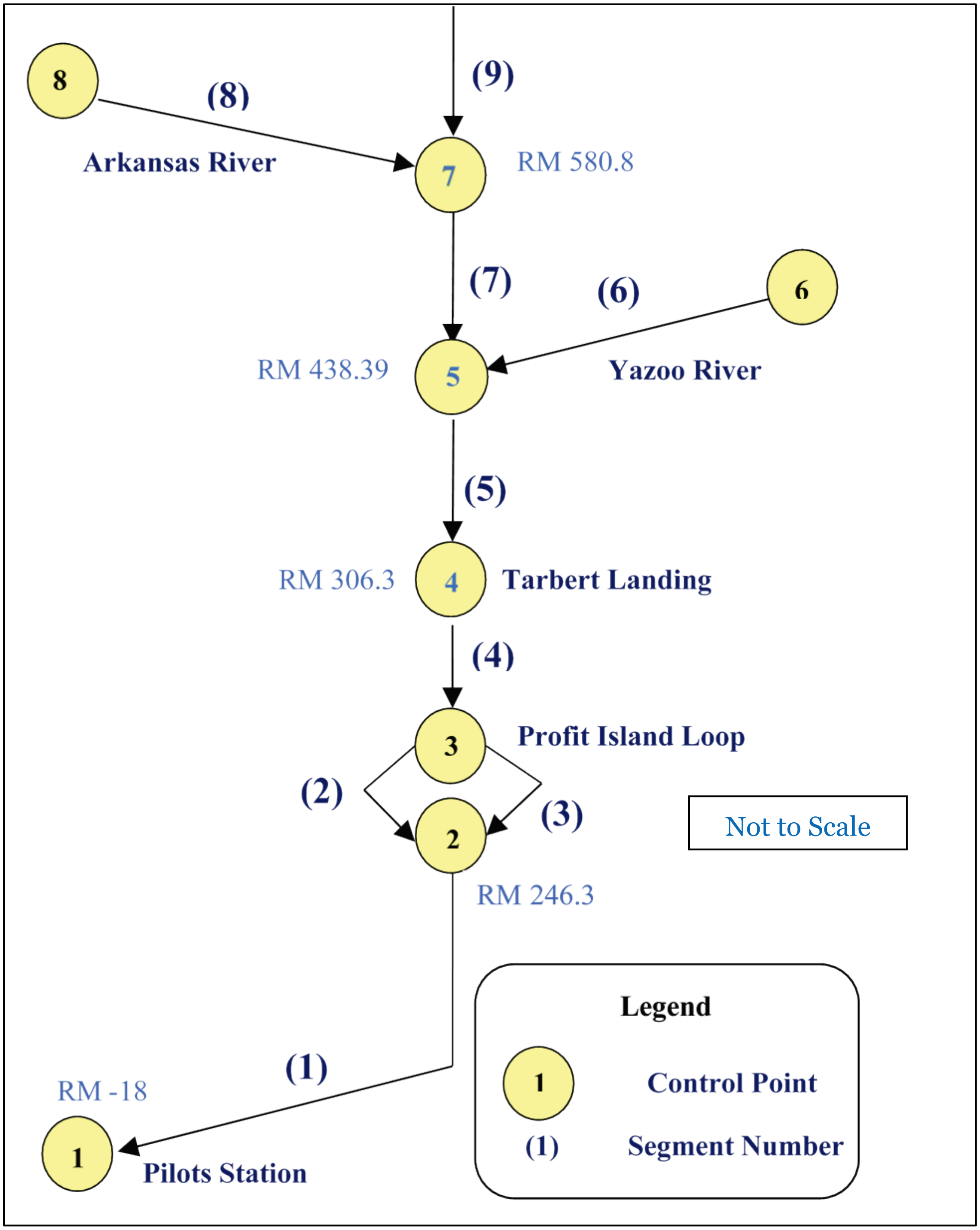
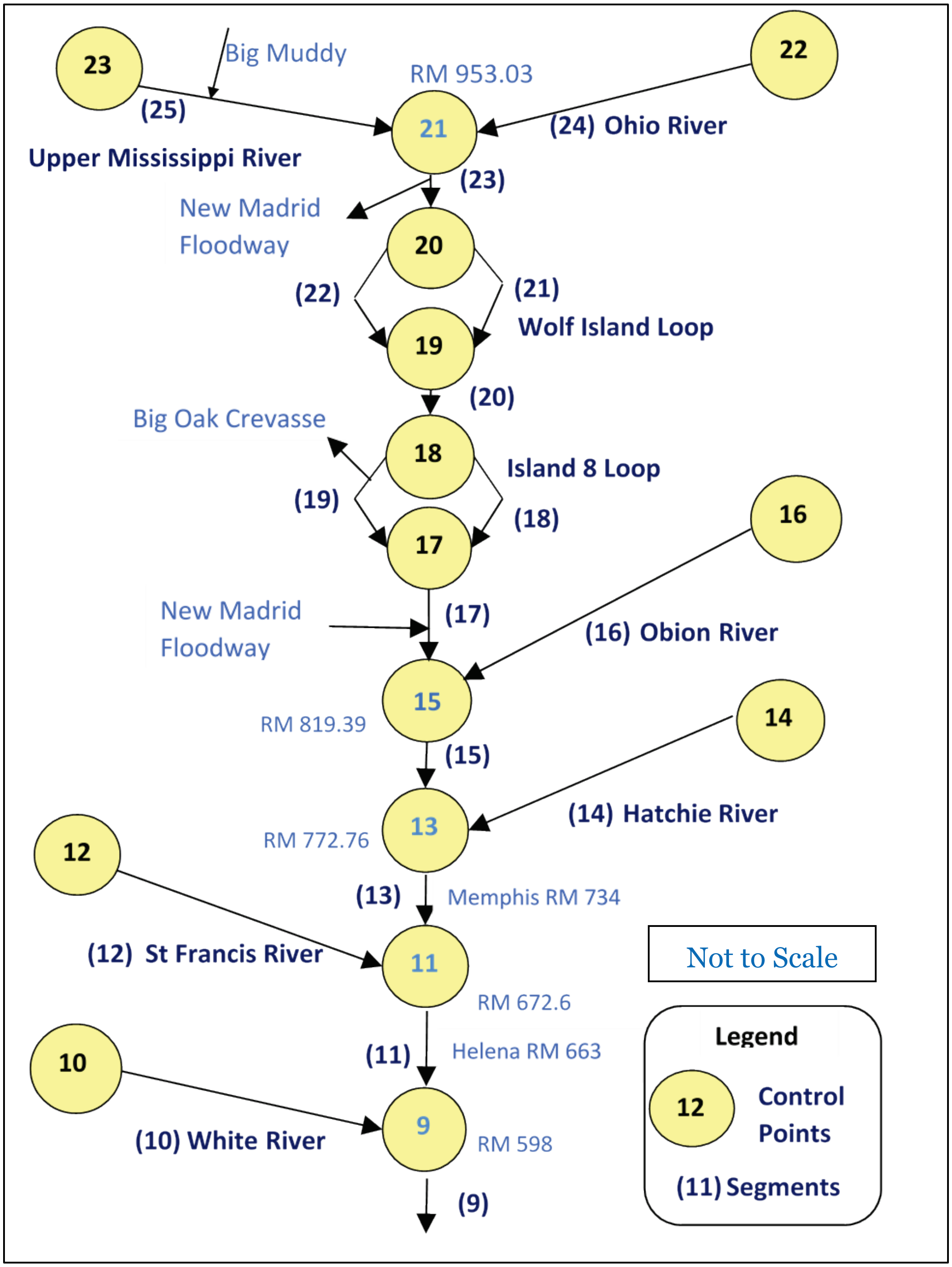


Figure 4-2. Schematic of HEC-6T stream network in MVM and MVS.



4.2.1.3 Channel geometry

The cross section geometry in the Regional HEC-6T model was developed from hydrographic survey data taken between 1988 and 1992. Survey datum was NGVD 1929. The cross sections are coded from left to right looking downstream and generally have reach lengths of 2 to 4 miles. Some cross-section distances are less than a half-mile in more complex areas of the river such as bends, dike fields, and crossings. Some cross-section reach lengths are greater than 4 miles in reaches where it was difficult to find a representative cross section for 1D flow. More detail about the channel geometry can be found in the Mississippi River Sedimentation Report (USACE 2018c)

4.2.1.4 Hydrographs

Discharge hydrographs are simulated in the numerical model by a series of steady-state events. The duration of each event is chosen such that the changes in bed elevation, due to deposition or scour, do not significantly change the hydraulic parameters during that event. In this study, computational time-steps of 1 day were used. A 1-day time-step is not necessary to meet the requirement of insignificant bed change but was chosen for convenience because available data were reported as mean daily flows.

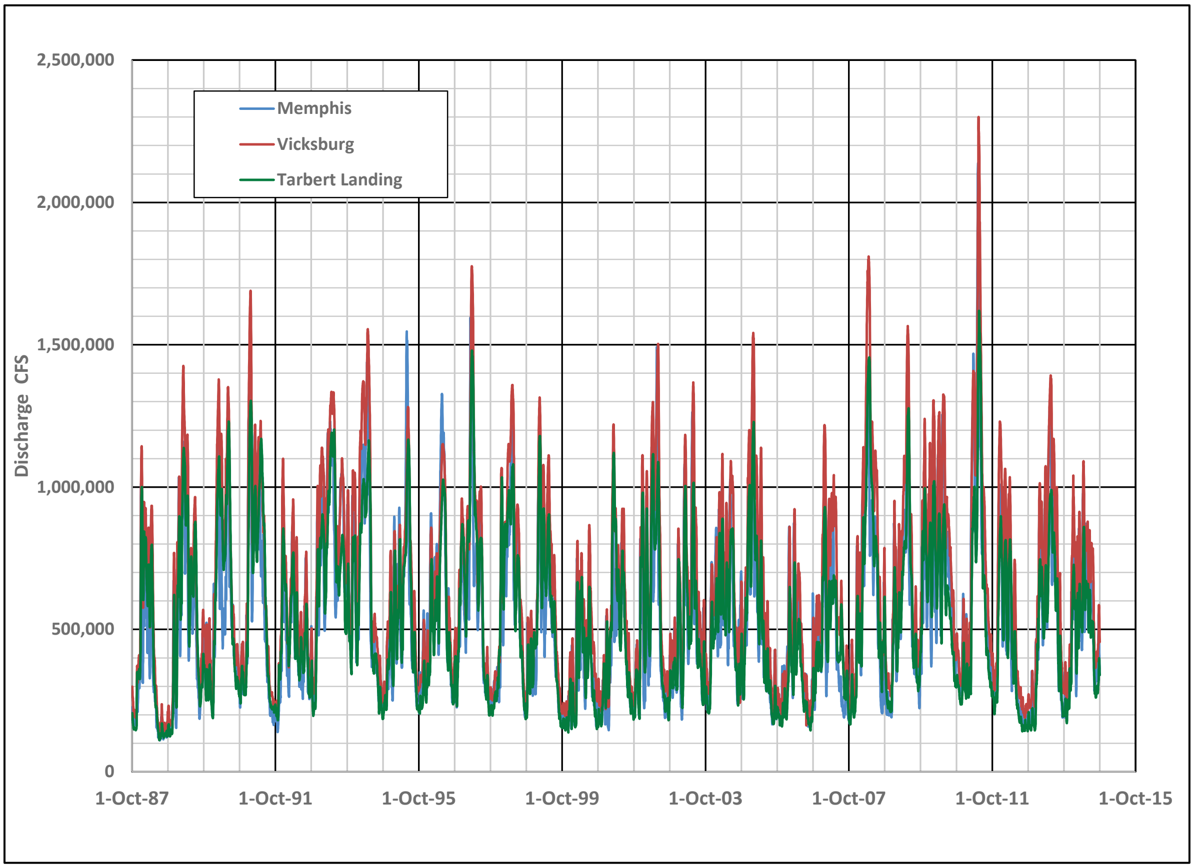
The approach used in HEC-6T approximates unsteady flow by stepping through a time-sequence of steady discharges. This method does not take into consideration flow routing or backwater storage effects, introducing the potential for timing issues in flow propagation through the model. Thus, HEC-6T may not be appropriate for predicting flood hydrographs and stages, but experience has shown that it is successful in predicting long-term sedimentation processes.

Reported daily discharges are available at several gages within the study area. Uncertainty is associated with the choice of discharge data for the inflow hydrograph because there are discrepancies in the reported data. These discrepancies are described in more detail in the Sedimentation Report (USACE 2018c). The HEC-6T hydrographs used in this assessment were based on data from upstream boundary gages and did not include adjustments to tributary inflows to match downstream gages. This approach maintains the beneficial effect of simulating the correct timing of tributary contributions but does not account for differences in measured discharges in upstream and downstream gages. The Flowline Model hydrograph incorporates additional discharge data that became available after completion of the Regional Model study. New data were available for the Yazoo, Arkansas, White, St Francis, Hatchie and Obion Rivers.

4.2.1.5 50-year Hydrograph

The projected 50-year hydrograph was developed from historical data. The 1991–2013 hydrograph was followed by the 1988–2014 hydrograph. The 2011 high water hydrograph occurred two times during the 50-year simulation. The 1988–2014 hydrographs at Memphis, Vicksburg, and Tarbert Landing are shown in Figure 4-3.

Figure 4-3. 1988–2014 hydrographs.



4.2.1.6 Project design hydrograph

Peak discharges for the 1955 58A-EN PDF were used initially to calculate the difference in peak water surface elevations due to 50 years of sedimentation processes in the Mississippi River between Pilots Station and Cairo. The hydrograph from this flood was used to calculate differences in water surface elevations due to sedimentation processes during the passage of a major flood. The 1955 58A-EN PDF peaks were also used to evaluate the influence of various geomorphic driving variables on Mississippi River geomorphic trends.

The PDF hydrograph was in the process of being assessed during the early stages of the sedimentation study. Peak discharges are significantly greater with the estimated 2016 58A-R Authorized Yazoo PDF. (See Section 3 for more information comparing the PDF flows.) The estimated 2016 PDF peaks were used to re-calculate the difference in peak water surface elevations due to 50 years of sedimentation processes. However, the estimated PDF was not used to re-evaluate geomorphic trends.

4.2.1.7 Distributary flows at Fort St. Philip and downstream

Distributary flow distribution percentages used in the Regional Model at Fort St. Philip and downstream were updated in this Flowline Model to incorporate additional data collected after completion of the Regional Model study. Significant data were collected in the distributaries below Venice, including the diversion at West Bay, which was opened in 2004 and was not included in the Regional Model. The distributary at Fort St. Philip (RM 20) was also added to the Flowline Model. The distributary diversion reference stations were updated from the Regional Model to the new HEC-6T usage. The Mississippi River Sedimentation Report (USACE 2018c) explains the distributary flows for this reach of the river in greater detail.

4.2.1.8 Distributary flows upstream from Ostrica

Distributary flow distribution percentages used in the Regional Model upstream from Ostrica were updated in this Flowline Model to incorporate additional data collected after completion of the Regional Model study. Outflows through the Davis Pond diversion structure (RM 118.4) opened in 2002 were added to the Flowline Model. Flow diversion percentages at Bonnet Carré Spillway (RM 128) were modified in the Flowline Model to account for *leaks* identified during the 2011 flood. Overbank flow at Manchac Point (RM 221–RM 210.8) and Devil's Swamp (RM 244.2–RM 236) were included in the Flowline Model. Flood diversions at Birds Point (RM 951) were also added to the Flowline Model. Distributary diversion percentages at Bird's Point were determined from measured data collected by the USGS during the 2011 flood. The Mississippi River Sedimentation Report (USACE 2018c) explains the distributary flows for this reach of the river in greater detail.

4.2.1.9 Morganza Spillway

The Morganza Spillway (RM 280) was designed to operate such that the downstream discharge does not exceed 1,500,000 cfs. Diversion percentages at Morganza for the 1955 58A-EN PDF were assigned in the Flowline Model to match the operation schedule. However, the maximum design flow through the Morganza Spillway is 600,000 cfs. Due to the higher discharges for the estimated 2016 PDF, both operation criterion cannot be achieved. Flowline Model diversion percentages for the estimated 2016 PDF were set to match the structure's maximum design capacity.

4.2.1.10 ORCC

The flow diversion percentages at the ORCC structures for existing conditions are based on historical data between 1991 and 2011. There is not a unique relationship between Mississippi River discharge and diversion percentage though any of the Old River structures. Therefore, average values and linear trend lines were used to estimate a long-term average diversion percentage for each structure. The maximum authorized flow diversion through the Old River complex is 620,000 cfs. Due to the higher discharges for the estimated 2016 PDF, Flowline Model diversion percentages for discharges above 2,220,000 cfs were reduced to maintain a maximum diversion of 620,000 cfs.

Historical flow distribution fractions through the Auxiliary Structure, Low Sill Structure, Old River Spillway, and Hydropower Structure from the Regional Model were updated using 1991–2011 daily discharge data. There was no flow over the Old River Spillway between 1991 and 2011. Historical distribution fractions were determined for three distinct discharge ranges: less than 800,000 cfs; 800,000 to 1,600,000 cfs; and 1,600,000 to 2,220,000 cfs. For discharges less than 800,000 cfs, the Hydropower and the combined Auxiliary and Low Sill diversion fractions were taken to be the mean average determined for all the daily discharges less than 800,000 cfs for the 20-year period. A combined flow for the Auxiliary and Low Sill Structures was used because it is the combined flow that is used to maintain the designated flow split at Old River. For discharges between 800,000 and 1,600,000 cfs average linear trend lines were determined for the Hydropower and the combined Auxiliary and Low Sill structures. Likewise, for discharges between 1,600,000 and 2,220,000 cfs average linear trend lines were determined for the Hydropower and the combined Auxiliary and

Low Sill structures. The individual fractions for the Auxiliary and Low Sill structures used in the HEC-6T model were determined using the combined distribution from the trend lines and the mean average flow fraction for each of the two structures for each of the three discharge ranges.

At the Old River Complex, sediment diversion coefficients for each structure were the same as those used in the Regional Model, which in turn were extracted from a previous study (Catalyst-Old River Hydroelectric 1999). The sediment concentration ratios from the 1999 study had been developed from available measured data (1991–1997) and numerical model calibration. Due to the wide scatter in the measured data, no attempt was made to vary the concentration ratios with changes in discharge or with variations in operation schedules. A more detailed description of the historical measurements is contained in the Regional Model study.

4.2.1.11 Temperature

Water temperature was based on 1965–2014 reported data from the USGS. Monthly averages were calculated from the observed data and assigned to the 15th of each month in the computational hydrograph. Mean monthly temperatures assigned to 15th of each month are shown in Table 4-2. In the HEC-6T hydrograph, daily values were assigned based on linear interpolation.

Table 4-2. Average monthly water temperature, 1965–2014, °F.

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Upper Miss	36.2	37.6	45.2	55.5	66.8	76.5	82.4	82.3	76.9	62.9	48.6	39.8
Ohio	40.8	41.2	47.2	56.6	68.4	76.8	83.8	83.9	79.9	62.7	57.2	46.5
Obion	40.9	44.0	51.5	62.2	71.0	77.2	80.4	78.3	71.2	62.4	53.8	44.6
Hatchie	41.2	45.4	52.5	61.8	68.1	76.6	80.4	78.2	73.1	63.4	56.5	46.4
St. Francis	40.8	43.3	52.4	60.7	68.9	77.5	82.9	82.2	76.9	63.8	54.9	45.7
White	41.8	45.5	53.3	62.8	69.3	75.9	78.4	78.2	74.3	64.3	55.3	46.6
Arkansas	42.5	44.1	52.7	61.1	71.1	79.4	85.2	85.2	79.4	68.7	57.7	48.6
Yazoo	46.5	48.8	56.8	66.0	74.3	81.3	85.3	86.2	81.1	70.4	60.0	51.2

4.2.1.12 Downstream water surface elevation

The NOAA tide gage at Grand Isle, LA (8761724), was used to set the downstream water surface elevation for the model study. Consistent with previous studies, the downstream water surface elevation was set to reflect seasonal variation in the mean tide elevation in the Gulf of Mexico. Mean monthly tide elevations were obtained from the NOAA web site (http://tidesandcurrents.noaa.gov) for the years 1978–2014. The data indicate a long-term increase in the mean tide elevation. To differentiate the seasonal variation from SLR and/or subsidence effects, adjusted mean monthly water surface elevations were calculated. The adjusted water surface elevations varied between 0.2 and 1.2 ft and were approximately 0.3 ft higher than those used in the Regional Model study. In the numerical model, mean daily values were determined by interpolation, assigning the mean monthly magnitude to the 15th of each month. This conversion was also used in the Interagency Performance Evaluation Task force (IPET) study (USACE 2006) for the City of New Orleans. The Grand Isle gage is in an area with significant changes in relative MSL (+9.24 mm/year), and tide elevations have a special epoch designation (2007–2011). There was no correlation to NOAA tide elevations and NGVD 29 or NAVD 88 found on its web site. Downstream water surface elevations assigned in the Flowline Model are listed in Table 4-3.

Table 4-3. Downstream water surface elevations at Pilots Station.

Month	NGVD 29 (feet)	Month	NGVD 29 (feet)
January	0.55	July	0.97
February	0.61	August	1.09
March	0.75	September	1.35
April	0.91	October	1.26
May	1.06	November	1.02
June	1.07	December	0.70

4.2.1.13 Roughness coefficients

Flood discharges on the Mississippi River in 2011 produced stages significantly higher than those used to determine roughness coefficients for the MVM reach in the Regional Model. In the MVN, 2011 flood elevations were generally not higher than historical floods due to operation of the Morganza and Bonnet Carré Spillways. The MVK reach in the Regional Model had already been calibrated to 2011 flood data. In this assessment, 2011 flood data were used to adjust high-flow roughness coefficients for the MVM reach. Roughness coefficients used in the Flowline Model are listed in Table 4-4, Table 4-5, and Table 4-6.

Table 4-4. HEC-6T roughness coefficients (Manning's *n*) in MVN.

RM	Overbanks	Channel - 1000 cfs				
		70	**120**	**180**	**240**	**300**
-18.0	0.05	0.038	0.026	0.022	0.022	0.0165
		190	**320**	**450**	**580**	**710**
0.0	0.15	0.038	0.026	0.022	0.022	0.0165
		250	**600**	**800**	**900**	**1,200**
11.05	0.20	0.024	0.019	0.019	0.015	0.014
		150	**650**	**950**	**1,100**	**1,200**
35.1	0.20	0.024	0.022	0.022	0.018	0.015
		275	**650**	**850**	**950**	**1,300**
53.0	0.20	0.027	0.026	0.024	0.023	0.0205
		275	**650**	**950**	**1,200**	**1,500**
105.0	0.20	0.029	0.026	0.025	0.0215	0.021
		275	**550**	**950**	**1,200**	**1,500**
141.6	0.20	0.028	0.026	0.024	0.021	0.022
		275	**550**	**850**	**950**	**1,500**
177.3	0.20	0.026	0.026	.025	0.022	0.021
195.3	0.20	0.026	0.026	.025	0.022	0.021
229.7	0.20	0.027	0.027	.026	0.025	0.024
		275	**550**	**850**	**1,100**	**1,500**
267.0	0.20	0.029	0.027	.024	0.023	0.020
		200	**400**	**850**	**1,000**	**2,000**
306.3	0.20	0.034	0.031	.031	0.030	0.026

Table 4-5. HEC-6T roughness coefficients (Manning's *n*) in MVK.

RM		Overbanks	Channel -1000 cfs				
			200	**400**	**900**	**1,400**	**1,800**
328.6	Upstream Union Point	0.20	0.034	0.030	0.030	0.027	0.026
365.0	Upstream Natchez	0.20	0.027	0.027	0.0275	0.025	0.024
396.4	Upstream St. Joseph	0.20	0.027	0.027	0.0275	0.025	0.024
			200	**600**	**1,100**	**1,400**	**1,800**
438.4	Upstream Vicksburg	0.20	0.027	0.025	0.025	0.024	0.023
			200	**400**	**700**	**1,400**	**1,800**
487.8	Upstream Lake Providence	0.20	0.033	0.029	0.027	0.027	0.027
			200	**400**	**600**	**1,400**	**1,800**
531.7	Upstream Greenville	0.20	0.032	0.027	0.026	0.027	0.027
			200	**400**	**800**	**1,400**	**1,800**
557.2	Upstream Arkansas City	0.20	0.033	0.030	0.030	0.027	0.023

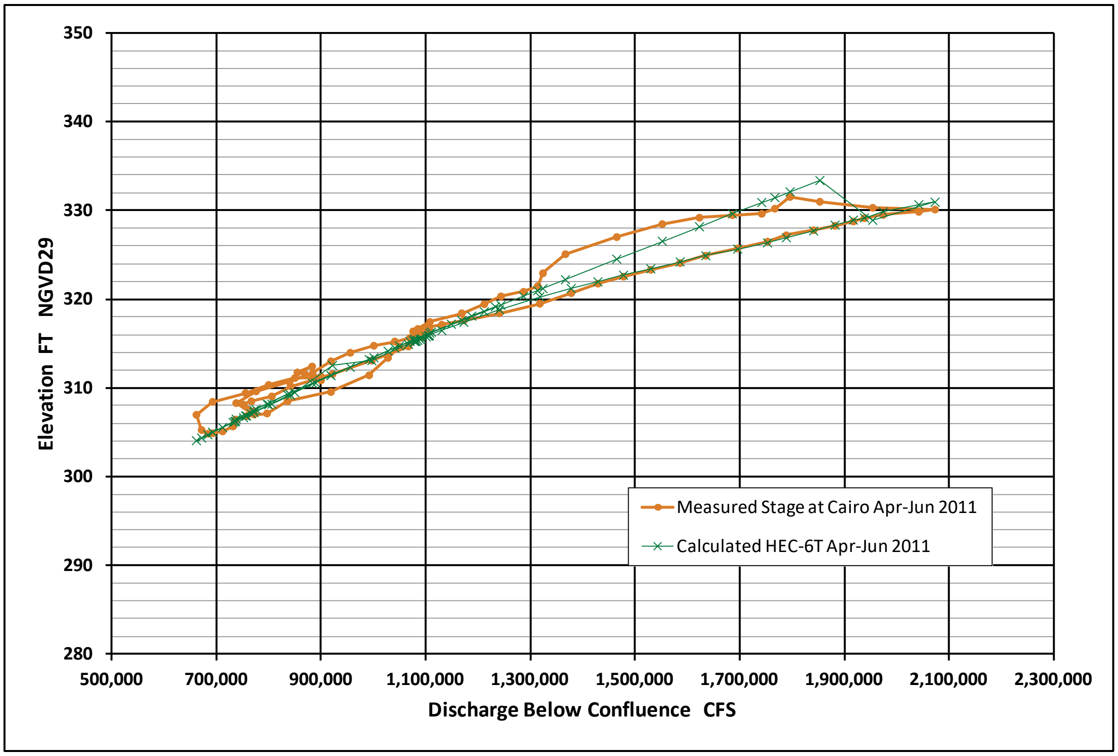
Table 4-6. HEC-6T roughness coefficients (Manning's *n*) in MVM.

RM		Overbanks	Channel -1000 cfs				
			200	**400**	**800**	**1,400**	**1,800**
580.8	Upstream Arkansas River	0.20	0.033	0.030	0.030	0.027	0.023
			900	**1,100**	**1,600**	**1,800**	**2,200**
600.1	Upstream White River	0.15 -0.25	0.029	0.030	0.031	0.030	0.029
			200	**500**	**1,000**	**1,400**	**2,200**
666.01	Upstream Helena	0.15 -0.25	0.029	0.028	0.027	0.027	0.027
			200	**500**	**1,000**	**1,400**	**2,000**

RM		Overbanks	Channel -1000 cfs				
			200	400	800	1,400	1,800
744.98	Upstream Memphis	0.15 -0.25	0.036	0.034	0.033	0.033	0.034
			100	**500**	**1,000**	**1,250**	**2,000**
910.54	Island 8 Main	0.15 -0.25	0.036	0.036	0.036	0.036	0.036
910.54	Island 8 Secondary	0.15 -0.25	0.036	0.024	0.024	0.024	0.024
			200	**500**	**1,000**	**1,400**	**2,000**
915.59	Upstream Island 8	0.20	0.036	0.034	0.033	0.033	0.034
			50	**100**	**250**	**625**	**1,000**
931.53	Wolf Island Secondary	0.20	0.036	0.035	0.031	0.027	0.034
931.53	Wolf Island Main	0.20	0.024	0.023	0.024	0.025	0.032
			200	**500**	**1,000**	**1,400**	**2,000**
936	Upstream Wolf Island	0.15 -0.25	0.036	0.034	0.033	0.033	0.034

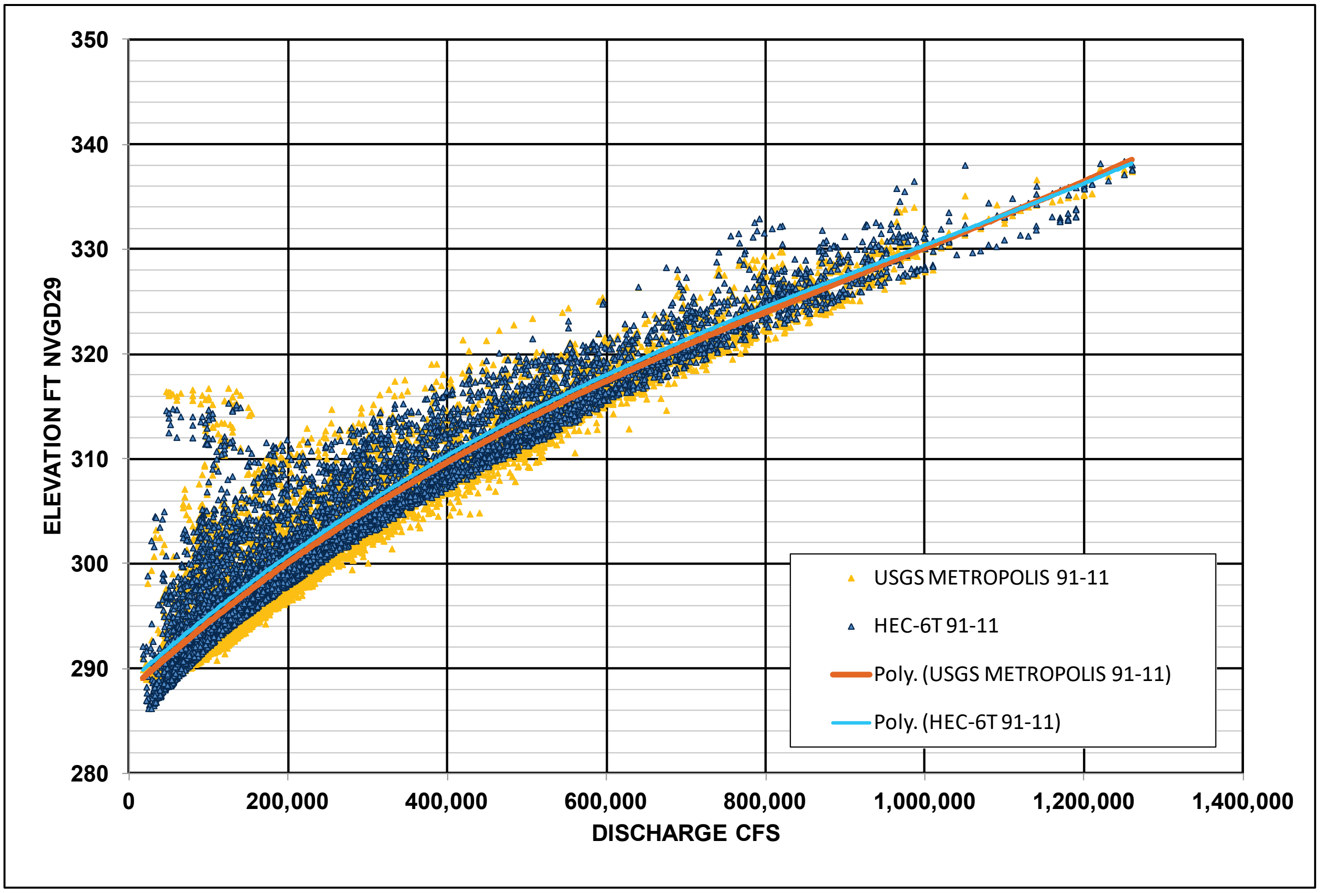
Stages at Cairo are influenced by the combined discharge from the Ohio River and the Mississippi River. This makes roughness coefficient adjustment to a general rating curve of Ohio River discharges difficult. Calibrating roughness coefficients for flood discharges are further complicated by the diversion into the New Madrid Floodway. The diversion significantly reduces backwater downstream. Roughness coefficients for the 2011 flood were therefore determined based on plotting the stage at Cairo versus the combined discharge and accounting for diverted flows in both the HEC-6T calculations and the measured data. Roughness coefficients for discharges greater than 1,250,000 cfs, from the original model, were increased between Hickman and Cairo to achieve the results shown in Figure 4-4.

Figure 4-4. Calculated and measured elevation at Cairo, April 1 to June 30, 2011.



No adjustment to roughness coefficients at high discharges was required to replicate measured water surface elevations at Metropolis. The channel roughness coefficient was 0.026 for all discharges. Metropolis is located at Ohio RM 943, which is approximately 38 miles upstream from the Mississippi River confluence. Measured water surface elevations between 1991 and 2011 are compared to calculated stages in Figure 4-5. A comparison of polynomial regression curves through the two data sets demonstrates that average water surface elevations are similar confirming that roughness coefficients are reasonable. A wide scatter in the stage-discharge relationship is expected at this station because stage is significantly affected by inflow from the Mississippi River above Cairo. This effect is apparent in both the measured and calculated data.

Figure 4-5. 1991–2011 measured and calculated water surface elevations, Ohio River at Metropolis.



4.2.1.14 Sand transport function

The sand sediment transport function in the HEC-6T numerical model calculates transport of the total sand and gravel sediment loads. This includes both the sand and gravel bed-material load and the sand wash load. The bed-material load includes size classes found in substantial quantities in the bed and moves as bed load and suspended load. The armoring and sorting algorithm in the numerical model also allows for transport of sand sizes, not found in the bed, as suspended wash load.

The sediment transport function for sand and gravel chosen for this study was the Toffaleti-Meyer-Peter Muller function. With this function, bed load is calculated using the Toffaleti (1968) and the Meyer-Peter Muller (1948) methods, and the larger of the two is used. Suspended bed-material load is then calculated using the Toffaleti method. The Toffaleti-Meyer/Peter Muller function is capable of calculating both sand and gravel transport rates for the size classes. The Toffaleti equation was developed for large rivers like the Mississippi River. The Meyer-Peter Muller equation was important to facilitate the transport of gravel size classes known to be in the river bed in the MVM.

4.2.1.15 Silt and clay transport functions

The equation for silt and clay deposition used in HEC-6T is the Krone (1962) equation. The required calibration coefficient is the critical bed shear stress below which deposition occurs. In HEC-6T, this coefficient has a variable name *DTCL* for clay and *DTSL* for silt. The Krone equation is

$$\frac{C}{C_o} = e^{-k't}$$

$$k' = \frac{\omega\left(1 - {}^{\tau_b}\!/_{\tau_d}\right)}{2.3\,D}$$

where:

C = concentration at end of time-step
C_o = concentration at beginning of time-step
t = time = reach length/flow velocity
ω = settling velocity of sediment particle
τ_b = bed shear stress
τ_d = critical bed shear stress for deposition (*DTCL* and *DTSL*)
D = water depth.

Erosion in HEC-6T is calculated based on work by Parthenaides (1965) as adapted by Ariathurai and Krone (1976). Initial erosion coefficients for the equation were taken from previous numerical model work by Thomas et al. (1989). These were modified during model calibration. It is the cohesive properties of the clay that determine the erosion thresholds. For this reason, the same erosion coefficients are used for silt and clay in HEC-6T. The Ariathurai and Krone equation for particle erosion is

$$C = \frac{M_1\, S_a}{Q\,\gamma}\left[\frac{\tau_b}{\tau_s} - 1\right] + C_o$$

where:

M_1 = erosion rate for particle scour
$[\tau_b * ERME/(STME-STCD)]\ (STCD < \tau_b < STME)$
S_a = surface area exposed to scour
Q = water discharge
τ_s = critical bed shear stress for particle scour *(STCD)*
γ = specific weight of water.

As the bed shear stress increases, particle erosion gives way to mass erosion, and the erosion rate increases. Because the mass erosion can theoretically be infinite, a characteristic time, T_c, is used. With a computation time interval of Δt, the Ariathurai and Krone equation for mass erosion is

$$C = \frac{M_2 S_a}{Q \gamma} \frac{T_c}{\Delta t} + C_o$$

where:

M_2 = erosion rate for mass erosion $(ERME + ER2\ \{\tau_b - STME\})$
$\Delta t / T_c$
T_c = characteristic time of erosion (1 hour)
Δt = duration of time-step.

Silt and clay deposition coefficients were varied in the model. One set of coefficients were used in Southwest Pass, another in the reach between Head of Passes and RM 11.0, and another upstream from RM 11.0. Varying these coefficients was deemed reasonable to account for the effects of salinity on sediment deposition. Silt and clay coefficients used in the Regional Model were determined during model calibration.

4.2.1.16 Sediment inflow

A combination of measured and calculated data were used to establish sediment inflow boundary conditions. To model sedimentation trends in the Mississippi River it is necessary to account for movement and storage of each sediment size class. HEC-6T allows for this accounting; however, the required input data are generally lacking. Long-term size class sediment data are available in the study reach, at the upstream boundary on the Middle Mississippi River at Thebes, located 43.7 miles upstream from the Ohio River confluence, and Chester, located 109.9 miles upstream from the Ohio River confluence. Long-term size class data are also available at Union Point (RM 326.6) and Coochie (RM 317.3), which are located upstream from the Old River Structure and at Tarbert Landing (RM 306.3), which is located downstream from the Old River Structure. A shorter record is available at Belle Chasse, located 76 miles above Head of Passes. Data for the major tributaries, including the Ohio River, are generally limited to the sediment concentrations greater and less than 0.062 mm. The lack of boundary condition data required that size class percentages be estimated by calculation and/or judgment.

The relative importance of boundary condition precision at each model boundary is related to the contribution of water and sediment from that tributary. The percentage of the annual Mississippi River discharge at Vicksburg for the years 1991–2002 is shown in Table 4-7. Over 80% of the annual discharge at Vicksburg comes from the Ohio and Middle Mississippi Rivers. Boundary conditions for these two rivers are by far the most important in the model study.

Table 4-7. Percentage of the annual Mississippi River discharge at Vicksburg, 1991–2002.

Source	%
Ohio River	45.1
Middle Mississippi River	37.4
Arkansas River	8.2
White River	4.4
Yazoo River	3.0
St. Francis River	0.8
Hatchie River	0.6
Obion River	0.5

After completion of the Regional Model study, additional data became available for the Middle Mississippi, Ohio, White, and Yazoo Rivers. New sediment-inflow rating curves were developed for the Flowline Model for these four rivers.

A statistical correction factor was applied to the power regression equations to account for bias created by using a least-squares regression for the logarithm of concentration. This bias occurs because the power regression produces a geometric mean instead of an arithmetic mean. The geometric mean is necessarily lower than the arithmetic mean, so concentrations are underestimated using the biased equation. The bias increases with the degree of scatter about the regression. A correction factor proposed by Ferguson (1986) was used to produce an unbiased estimator for both the total measured load and the sand load. The correction factor is given in the following equation:

$$\overline{C_i} = \hat{C_i} \exp(2.651\, s^2)$$

$$s^2 = \frac{\sum_{i=1}^{n} (\log C_i - \log \hat{C_i})^2}{n - 2}$$

where:

$\overline{C_i}$ = unbiased concentration at discharge event *i*

$\hat{C_i}$ = concentration from biased regression curve

C_i = measured concentration

N = number of measurements.

For more information about the sediment inflow boundary conditions used in the Mississippi River HEC-6T model, see the Mississippi River Sedimentation Report (USACE 2018c).

4.2.2 Calibration and verification

4.2.2.1 Calibration and verification of the Regional Model

The Regional Model was calibrated and verified according to the methodologies described in USACE, HEC (1992) and Thomas and Chang (2008). Terminology for the study process of preparing a numerical model for predictive use is not consistent. Calibration, circumstantiation, validation, and verification are common terms used in practice. Terminology from the above sources was adopted for use in this study. Calibration is defined herein to be the study process of arriving at roughness coefficients, a sediment transport function, model parameters, and representative data that will allow the model to calculate values that agree with values measured in the prototype. Verification is defined herein as the study process that demonstrates that the calibrated model will match the prototype when specified boundary conditions, such as time period or cross-section geometry, are changed.

Calibration of the Regional Model is described in detail in the Regional Model Report. The calibration process included selection of model parameters, sediment transport function, steady-state fixed-bed tests, steady-state movable-bed tests, and quasi-unsteady movable-bed tests.

Model parameters of computational time-step and sediment exchange increment were chosen based on convenience and experience. The model time-step was 1 day. Previous studies on the Lower Mississippi River had shown that a longer computational time-step could be used, but since discharge data are typically provided as a mean daily value, it is more convenient to use a constant 1-day time-step. The exchange increment used in the sorting and armoring algorithm was 1/20 day. This value was found to reproduce the same results as a much shorter exchange increment calculated by the program.

The sediment transport function chosen for this study was a combination of the Toffaleti (1968) and Meyer-Peter Muller (1948) equations. This function reproduced measured sand size class yields at downstream sediment gages between 1991 and 2002.

Steady-state, fixed-bed tests were conducted to calibrate roughness coefficients and to insure that the model reproduced acceptable hydraulic results. Roughness coefficients were adjusted so that the calculated water-surface profiles matched 1991–2002 average stages at gages in the study reach. Channel velocity and channel discharge profiles were plotted to ascertain if the model was reproducing acceptable hydraulic results. Adjustments to channel geometry were made in cases where the adjacent cross sections had unreasonable differences.

Steady-state, movable-bed tests were conducted with a constant channel-forming discharge run for approximately 5 years. This test was used to adjust the initial bed-material gradation to reflect reach-averaged conditions. The test also demonstrated that cross-section changes over time tended to converge on an equilibrium condition in response to a constant-discharge simulated hydrograph. This test also validated the choice of computational time-step.

The quasi-unsteady flow, movable-bed test was conducted simulating the 1991–2002 hydrograph. In a sense, this test was a verification of the numerical model because no further adjustments were made to achieve the reported results. The model reasonably duplicated 1991–2002 specific gage trends that had been developed from measured data. Measured and calculated bed-material gradations from 1991 and 2005 were compared and found to be within a reasonable range. Measured and calculated sediment concentrations and size class distributions at Tarbert Landing and Belle Chasse were found to be reasonable. Measured and calculated sediment concentrations and total sand yields at Union Point/Coochie, Vicksburg, and Memphis were also compared and found to be consistent.

The quasi-unsteady, movable bed test was also used to adjust the model to measured dredging quantities in Southwest Pass. This was strictly a calibration test, as cohesive sediment parameters were adjusted to match reported dredging between 1991 and 2002.

The Regional Model was verified/validated by simulating a deposition trend in the Mississippi River downstream from the Old River Structures. Measured deposition between RMs 310.1 and 315.4, as determined from the 1992 and 2002 hydrographic surveys, was compared to calculated deposition from the HEC-6T model. The calculated deposition was 90% of the measured deposition. These results provide an indication of the accuracy associated with quantifying deposition. Of course, systematic and random errors are attached to both the measured data and the calculated data and neither can be assumed to be *truth*. All things considered, the comparison suggests that the numerical model is very successful in predicting deposition on a reach scale.

The Regional Model was further verified by simulating erosion that occurred at Smithland Crossing (RM 297.45–300.3) in response to construction of a dike field between 1990 and 1996. The HEC-6T model did not reproduce the changes in channel shape due to the 1D limitations of the model. However, calculated degradation in the dike field was within 10% of the measured change determined from the 1992–2002 hydrographic survey.

As a result of the calibration/verification analysis, the Regional Model was established as a tool to evaluate the effects of various boundary condition changes upon long-term sedimentation processes in the Lower Mississippi River. The Regional Model's comprehensive calibration/verification analysis applies equally to the Flowline Model. Additional calibration of roughness coefficients was conducted for the Flowline Model to account for the record flood discharges in 2011. Additional verification was achieved using specific gage trends for the additional simulation time between 2002 and 2013 as discussed in the following section of this report.

4.2.2.2 Calculated specific gage trends

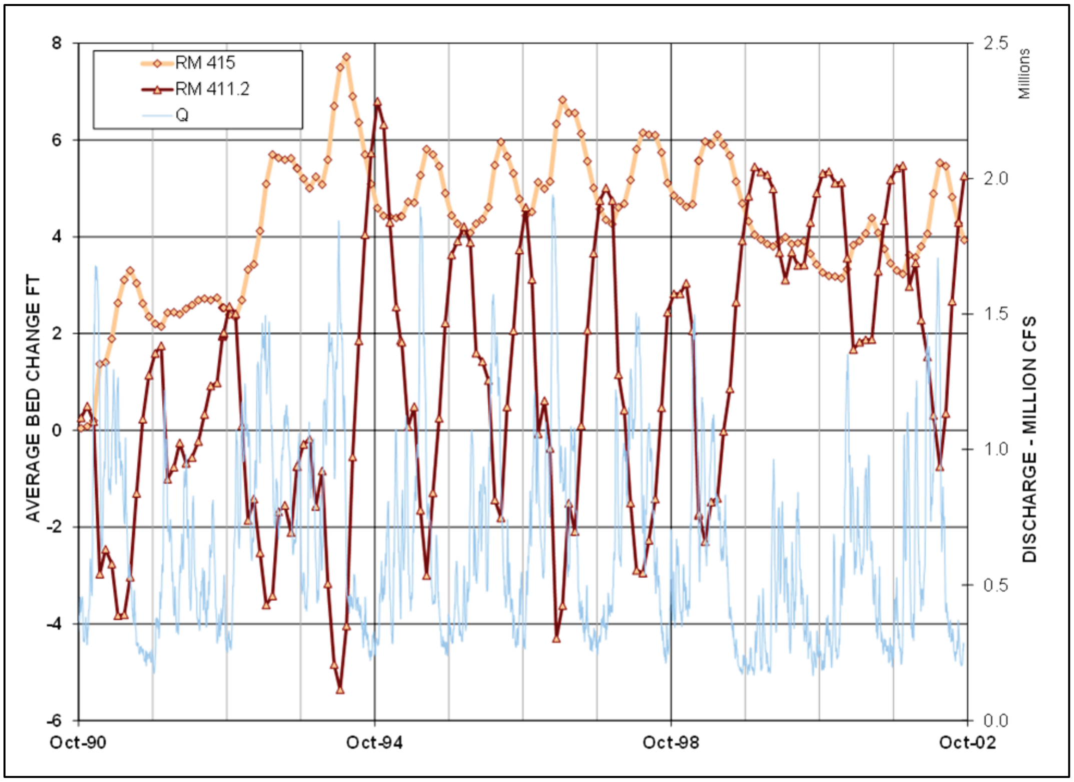
Stage is typically a better indicator of geomorphic change than bed elevation. Specific gage plots, which are based on historical stage and discharge data, have been used extensively to document trends in water surface elevations at gages. Specific gage analyses provide important perspectives relative to past river behavior at a specific location.

There are a number of limitations associated with specific gage analysis. Specific gage depicts conditions at a gaging station, which may or may not be representative of conditions farther upstream or downstream. Specific gage records chart the historical record and cannot be extrapolated to predict future conditions. Specific gage incorporates all the morphological factors influencing stage and cannot be used to identify the effect of an individual variable.

The advantage of a numerical model is that it can be used to predict future trends in water surface elevations and it can be used to predict both past and future trends between gages. The reliability of the model's prediction is predicated upon the model's ability to simulate the significant geomorphic processes that influence long-term change in stage. The HEC-6T numerical model accounts for future aggradation and degradation of the river bed but does not account for possible future changes in bed roughness, floodplain roughness, or change in channel shape due to meandering. There is also uncertainty associated with future changes in the boundary conditions, such as sediment inflow. Confidence in numerical model prediction is enhanced when historical changes in specific gage are correctly reproduced by the model. This success implies that the most significant geomorphic processes have been accounted for in the numerical simulations.

River bed elevations can change significantly with discharge. An example is shown in Figure 4-6 where calculated bed-elevation changes with time at two adjacent cross sections are plotted. At RM 415, average bed elevations tend to increase at high flow and decrease at low flow. The opposite trend occurs at RM 411.2. In addition, the change in bed elevation occurs more slowly than the change in discharge. There is not a direct correlation between discharge and bed elevation because the bed elevation is heavily dependent on antecedent conditions.

Figure 4-6. Calculated bed elevation change with time at a bend and at a crossing.



Specific gage plots at major gages along the Mississippi River using both traditional specific gage techniques and HEC-6T numerical model calculations are presented in Figure 4-7 through Figure 4-13. Historical specific gage data were provided by Biedenharn et al. (2017). Biedenharn et al. presented specific gage plots for 22 gages along the Lower Mississippi River between Cairo and Donaldsonville. Only the gages with discharge and stage measurements are compared herein. The historic and HEC-6T calculated curves were developed using the direct step method (defined by Biedenharn et al.) with a bin range of ± 2.5%. The bin includes all the discharges within a specified percentage of the nominal discharge. The historic data are subject to random errors and natural variability about the stage discharge relationship. The HEC-6T results are subject to systematic errors and represent average conditions, neglecting natural variability about the stage-discharge relationship. The historical data come from actual field measurements. The HEC-6T results come from daily calculations and are thus more numerous.

Results from both methods can be compared for the period between 1991 and 2013. No adjustments were made to the numerical model to match the specific gage data. The favorable correlation of measured and calculated stages between 2002 and 2013 provides verification of the Regional Model.

Figure 4-7. Specific gage at Hickman – RM 922.

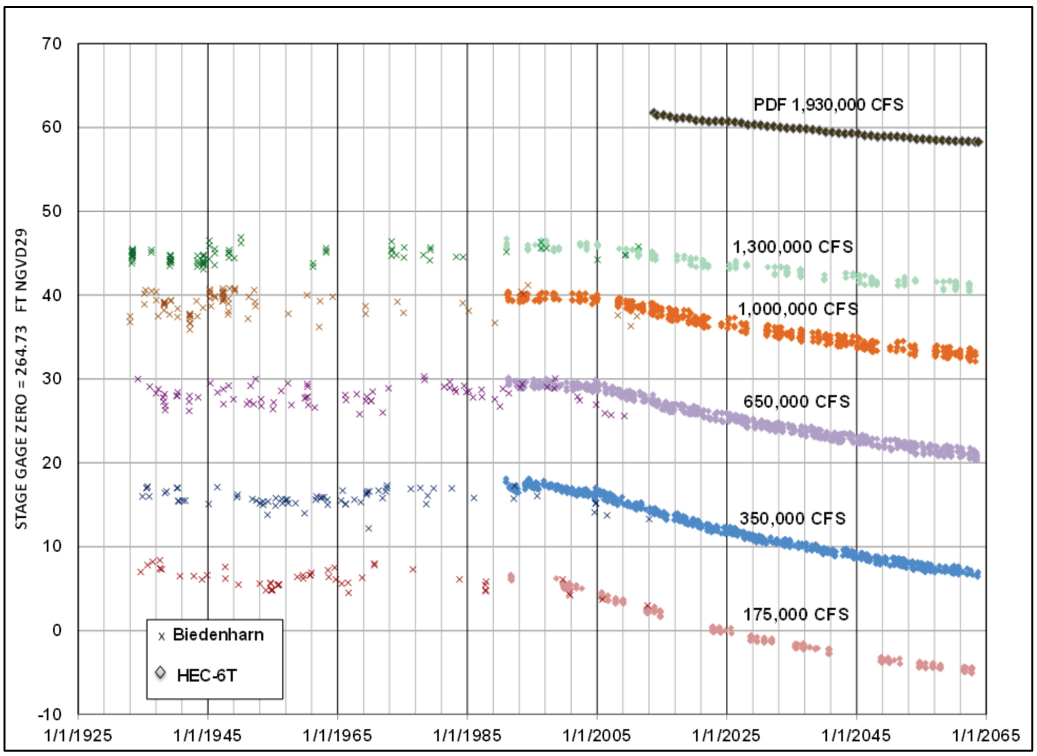


Figure 4-8. Specific gage at Memphis – RM 734.4.

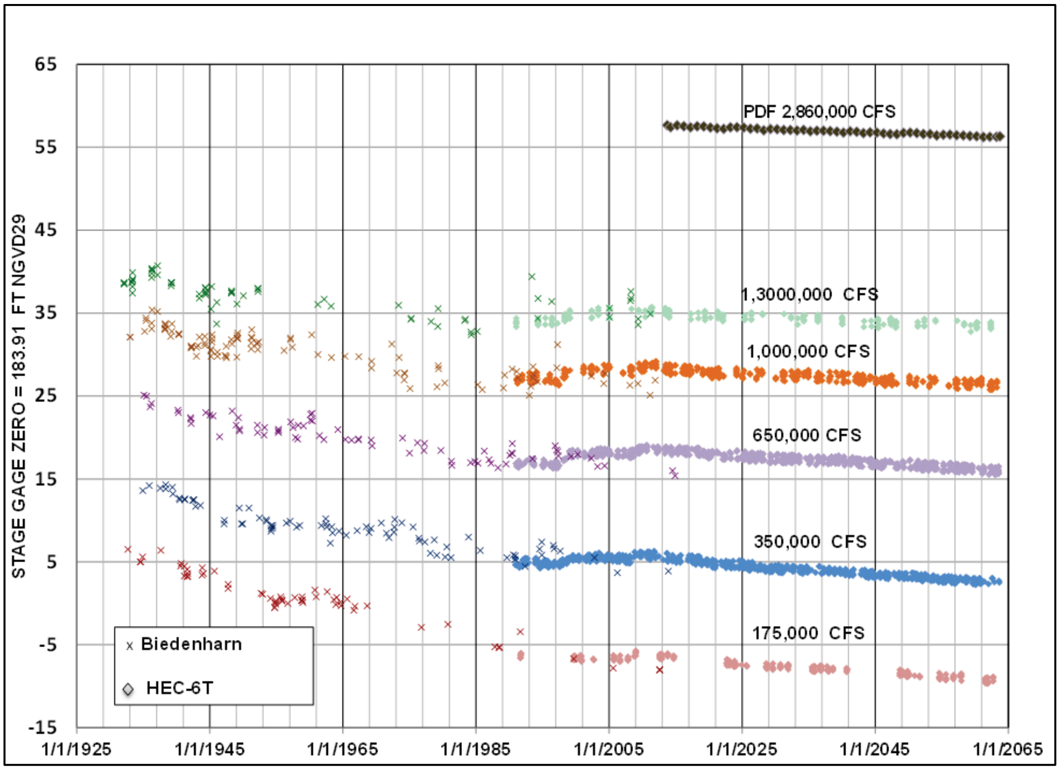


Figure 4-9. Specific gage at Helena – RM 663.1.

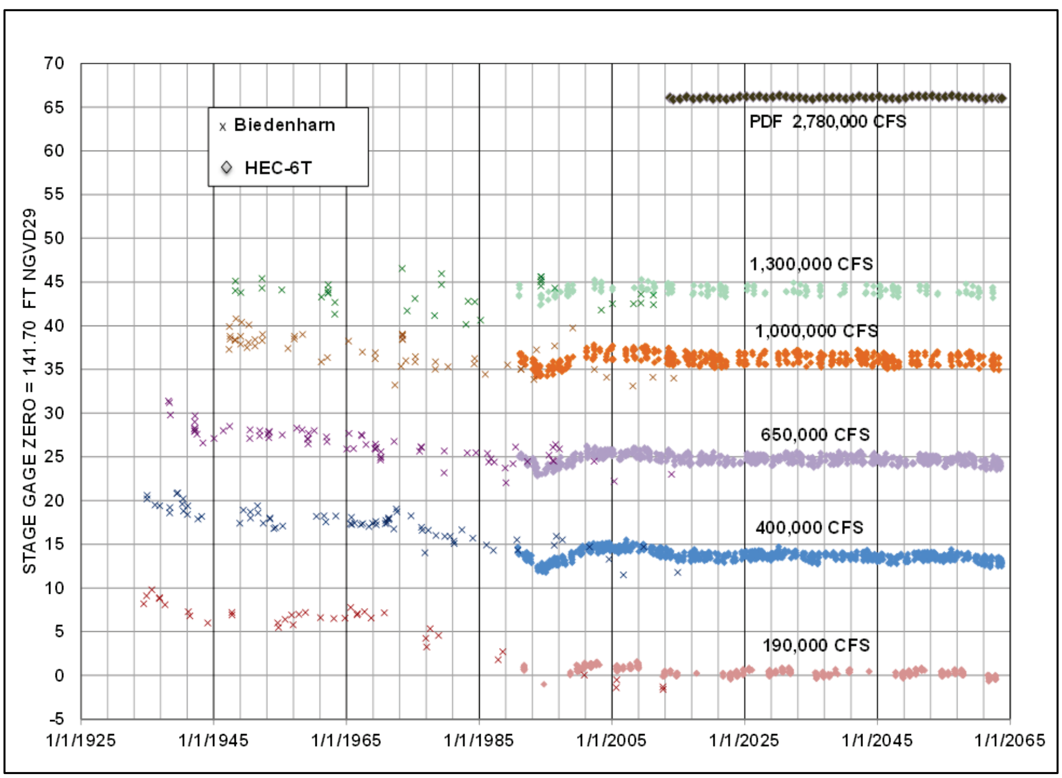


Figure 4-10. Specific gage at Arkansas City – RM 554.1

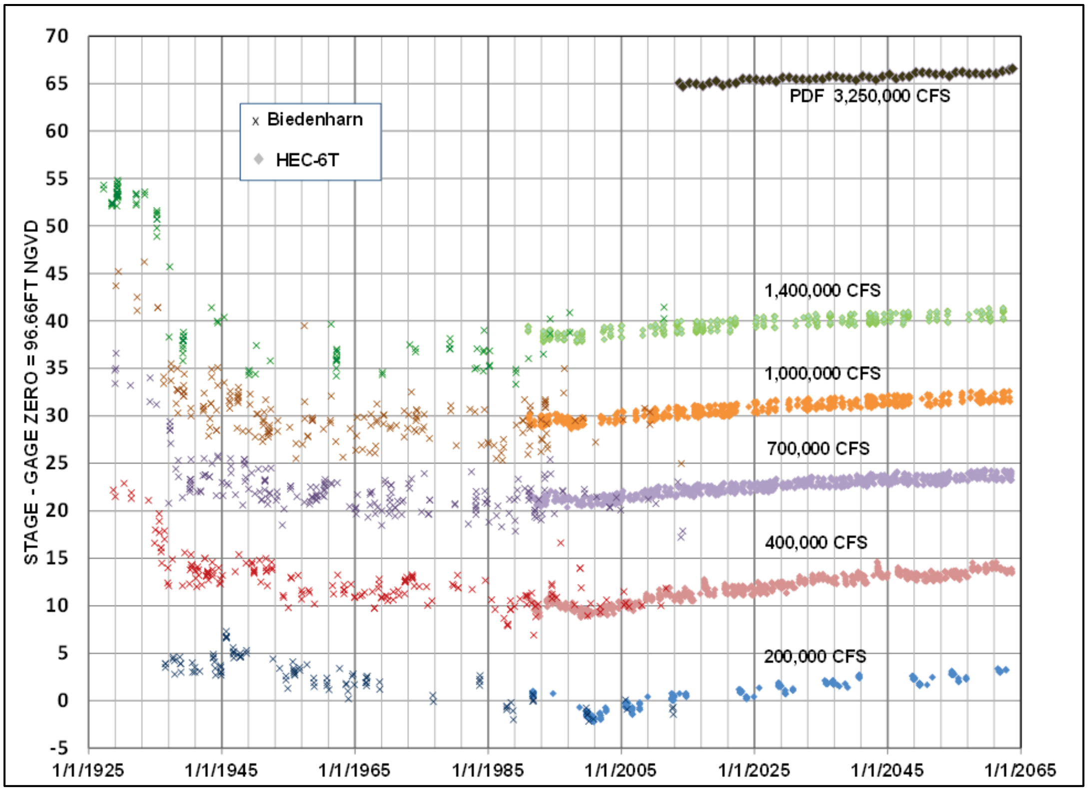


Figure 4-11. Specific gage at Vicksburg – RM 435.7.

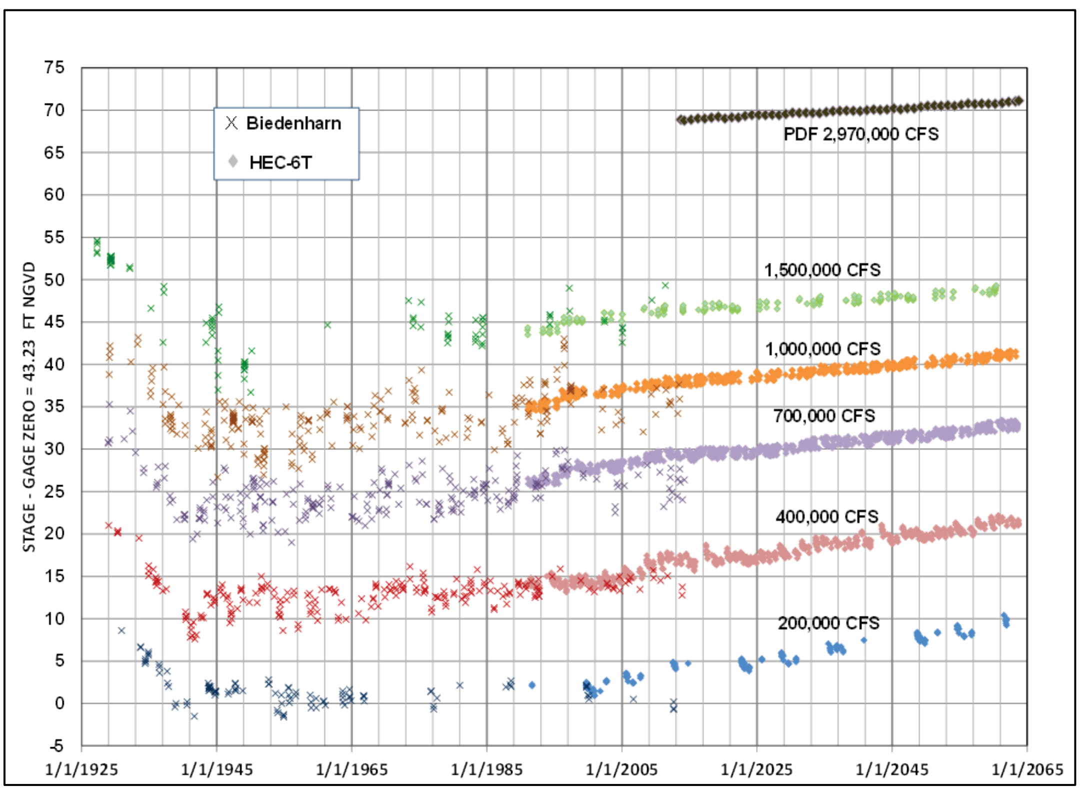


Figure 4-12. Specific gage at Natchez – RM 363.3.

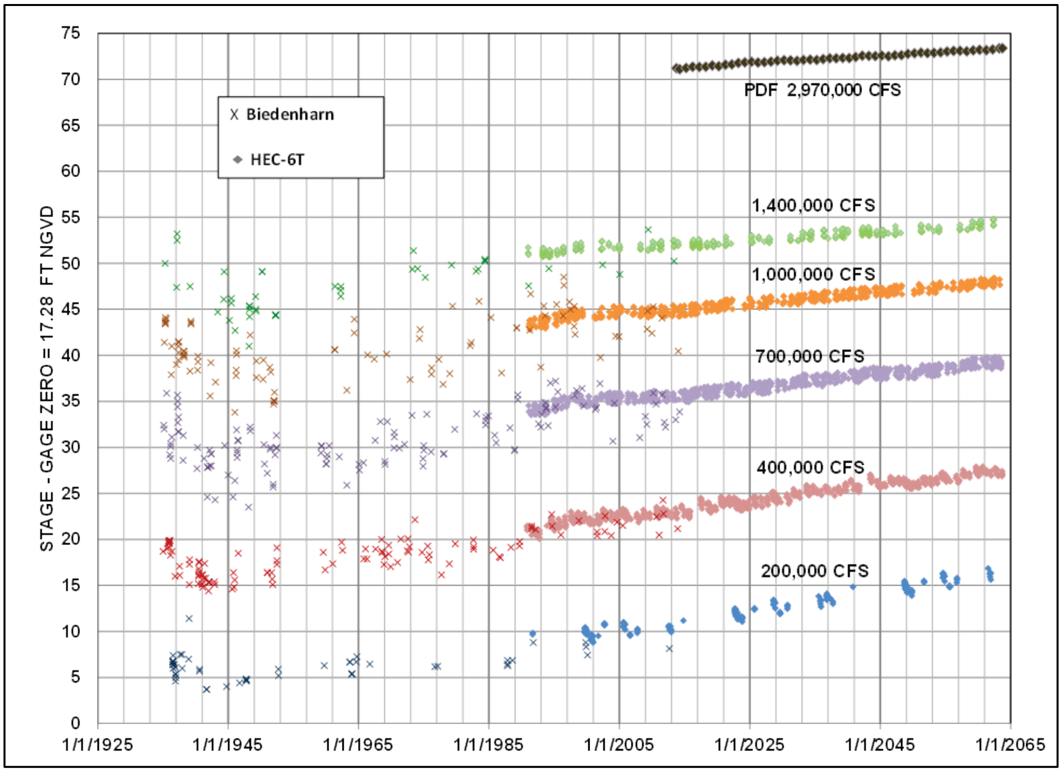
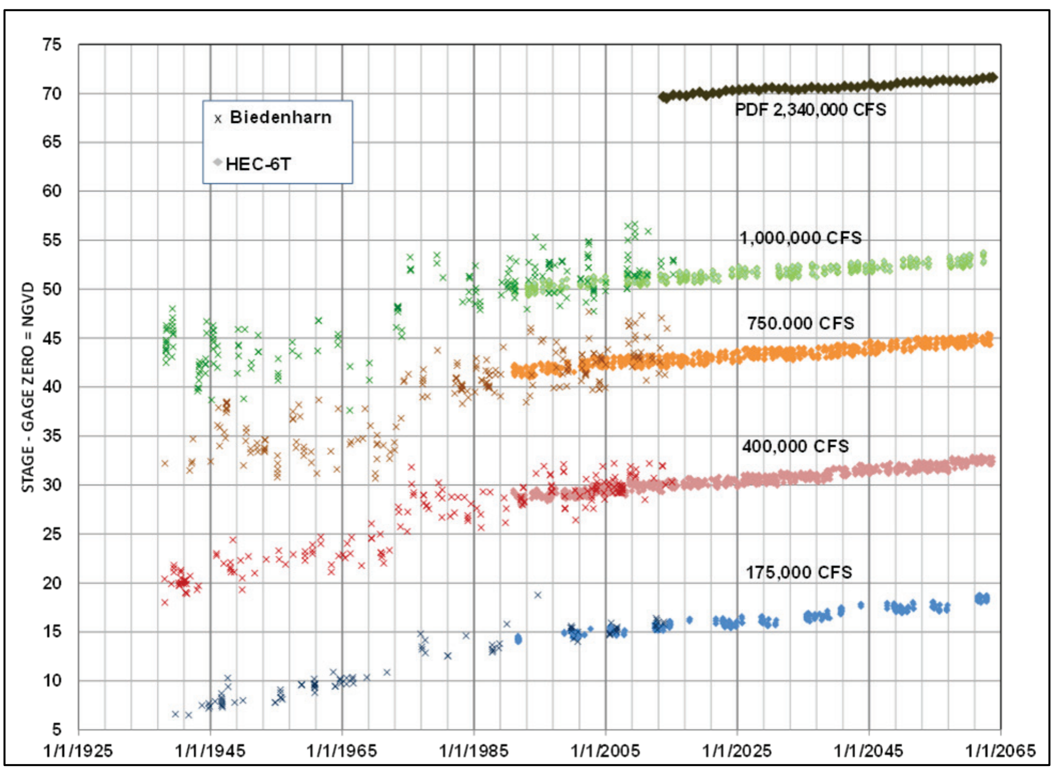
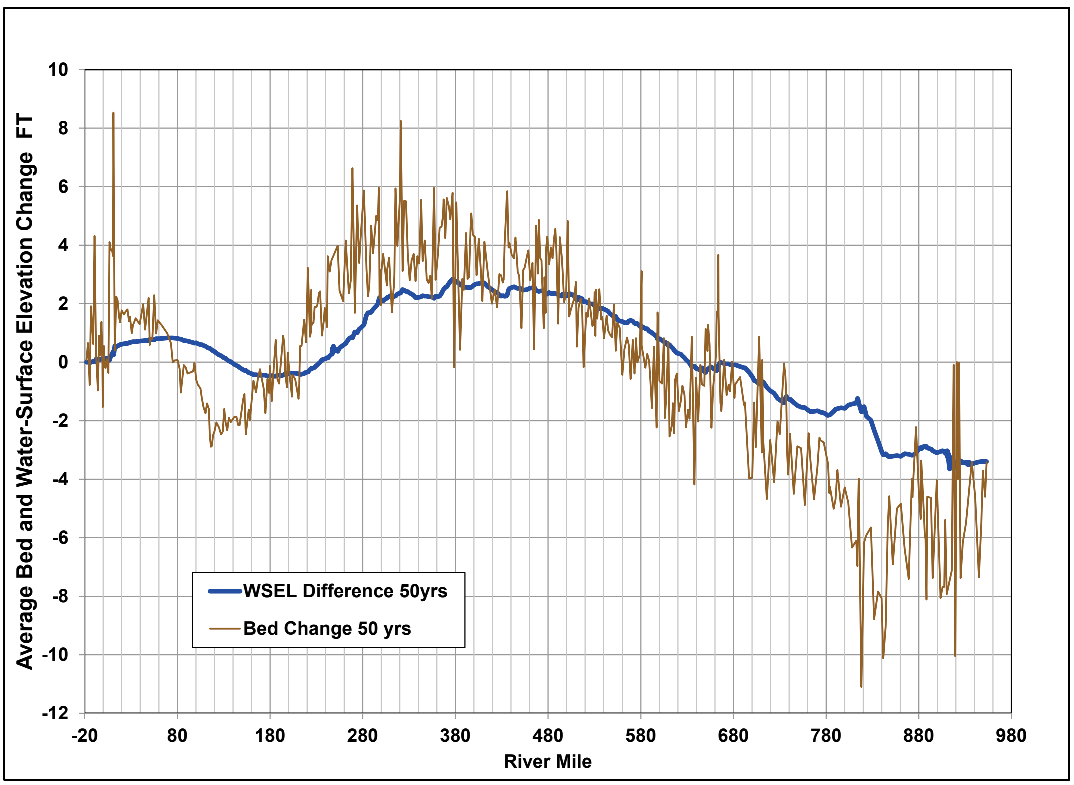


Figure 4-13. Specific gage at Red River Landing – RM 302.4.



Specific gage analyses typically refer to a long-term decline in water surface elevation as degradation and a long-term increase in water surface elevation as aggradation. This terminology conflicts with sediment studies that typically refer to degradation as a long-term decline in bed elevation and aggradation as a long-term increase in bed elevation. Bed elevation change does not necessarily correlate with water surface elevation change. The calculated bed elevation and PDF water surface elevation change after 50 years is shown in Figure 4-14. The figure shows significant fluctuation in bed elevation from station to station. Water-surface fluctuations are less pronounced. However, the overall trends are consistent. Of particular interest is the hard point in the vicinity of Hickman (RM 922), where the bed change is zero, but the water surface elevation is declining. The hard point is submerged by the river and does not act as an effective control on water surface elevation.

Figure 4-14. Calculated water surface elevation and bed changes for the 1955 PDF after 50 years.



4.2.3 Predicted increase in project design water surface elevation due to sedimentation

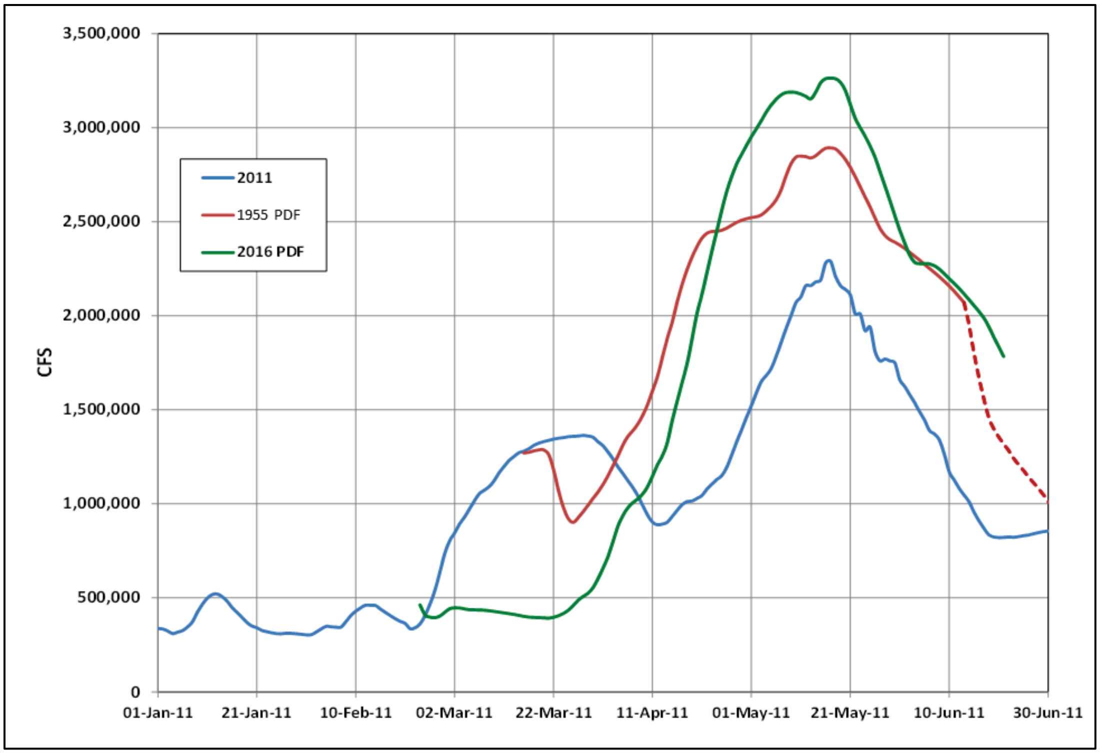
4.2.3.1 PDF

The sediment study was initiated and essentially completed before completion of the hydrologic study that estimated a PDF for 2016. Most of the sedimentation study objectives were achieved during the course of this investigation using the 1955 PDF. The 1955 PDF was used to determine long-term sedimentation effects on the design flowline and to evaluate the effects of various geomorphic influences on the river regime. The PDF estimated in 2016 was used to confirm results calculated using the 1955 PDF hydrograph. The estimated 2016 PDF was also used specifically to determine the effect of the rising flood hydrograph on channel change and maximum stage.

The estimated 2016 PDF discharges were significantly higher than those determined in 1955. Consequently, floodway structures in the MVN would not operate according to existing protocols. The initial assumption by the Flowline Study team is that the structures will be operated at design capacity rather than by changing the authorized diversion percentages and limits. The result of this assumption is a higher percentage of the flood discharge passing by Red River Landing and New Orleans.

Sedimentation study results, using the 1955 PDF, are presented first in this report. Numerical simulation results are typically presented as differences in PDF calculated water surface elevation between a base test and a test with a specific variable change. Conclusions are based on differences in calculated water surface elevation rather than on magnitudes. Therefore, the magnitude of the PDF should have an insignificant effect on results. This assumption was validated toward the end of this study by comparing results from the 1955 PDF and estimated 2016 PDF. The 1955 and estimated 2016 PDFs are compared to the 2011 flood at Arkansas City in Figure 4-15. In the figure, the hydrographs have been positioned so that the flood peaks coincide.

Figure 4-15. 2011 Flood and 1955 and estimated 2016 PDFs at Arkansas City.

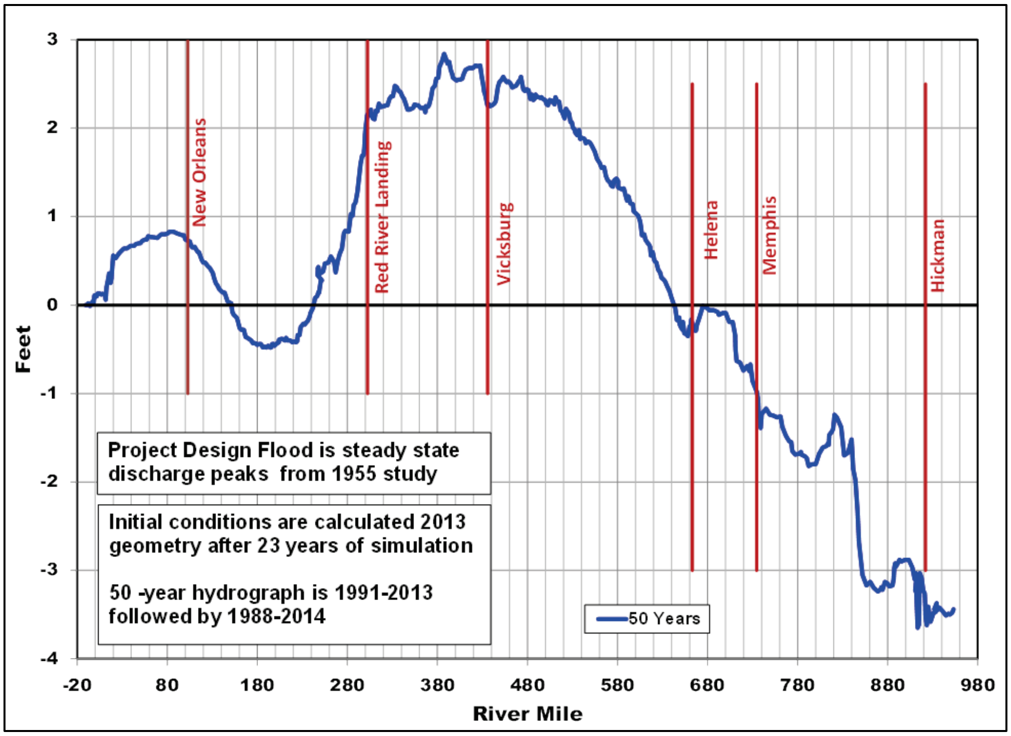


4.2.3.2 1955 Project design hydrograph

Numerical model results can be used to determine the specific gage trends between gages. For purposes of this study, it is more useful to display those results as a profile of the differences in PDF peak water surface elevation for two points in time. Figure 4-16 shows the change in calculated peak water surface elevation between 2013 and 50 years into the future due to sedimentation. This profile can be used to estimate the increased levee height required to contain the PDF, 50 years into the future, to account for sedimentation in the Lower Mississippi River.

The HEC-6T results predict that, after 50 years of aggradation and degradation in the Lower Mississippi River, PDF peak water surface elevations will generally be higher downstream from Helena and lower upstream from Helena. At New Orleans, peak water surface elevations are predicted to be approximately 1 ft higher after 50 years. Peak water surface elevations between Red River Landing and Arkansas City are predicted to be between 2 and 3 ft higher after 50 years. At Hickman, peak water surface elevations are predicted to be approximately 3 to 3.5 ft lower in 50 years.

Figure 4-16. Difference in 1955 PDF peak water surface elevations after 50 years of sedimentation.*



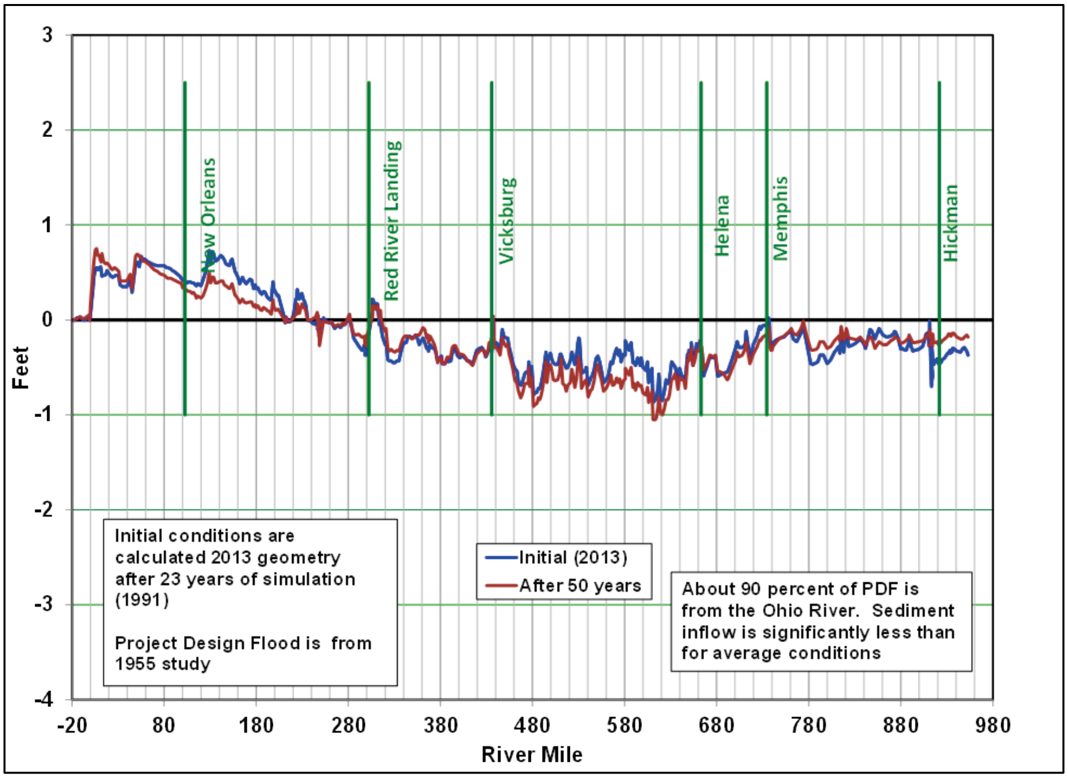
*Note: For a description of the 50-year hydrograph, see Section 4.2.1.5.

Channel geometry may change significantly with the rise and fall of a flood hydrograph. The channel change is not so much affected by the increase in bed sediment transport due to the increased discharge but by the difference in transport capacity between adjacent reaches. Thus, at any given time, the bed elevation of some cross sections may rise while others may decline. The numerical model was used to predict the difference in PDF water surface elevations due to channel changes during the rise of the 1955 hydrograph.

The approach was to first calculate a steady-state, water-surface profile using only the peak discharges with the 2013 geometry. This profile was compared to the peak water-surface profile calculated by simulating the PDF hydrograph starting with the 2013 initial geometry. This methodology was also applied after 50 years of sedimentation was calculated.

As shown in Figure 4-17, channel changes that occur during the flood rise result in water surface elevation differences of less than 1 ft. As the flood rises, the river upstream from RM 250 scours, and the sediment supply increases, and downstream from RM 250 the river is depositing sediment. Trends after 50 years of sedimentation are very similar to those calculated at the beginning of the simulation.

Figure 4-17. Difference in PDF peak water surface elevation due to rising hydrograph.



4.2.4 Factors affecting morphological change

The HEC-6T numerical model allows one to evaluate specific driving variables to determine which agents of change are the most significant in determining the future character of the Mississippi River. With the numerical model, a single variable may be changed to determine its effect. Historical data mask the effect of individual variables because the driving variables act together at the same time.

The historic record of discharge and stage on the Mississippi River has been affected by a variety of factors. There has been a significant change in the hydrologic regime due to the construction of reservoirs in the watershed. Flood peaks have been dramatically reduced on both the Middle Mississippi and Ohio Rivers. There has been a significant reduction in sediment supply due to sediment storage in reservoirs and behind navigation structures, watershed soil conservation measures, bank stabilization, and channel stabilization in tributaries. Engineering structures and channel modifications have also affected the sediment delivery through the river.

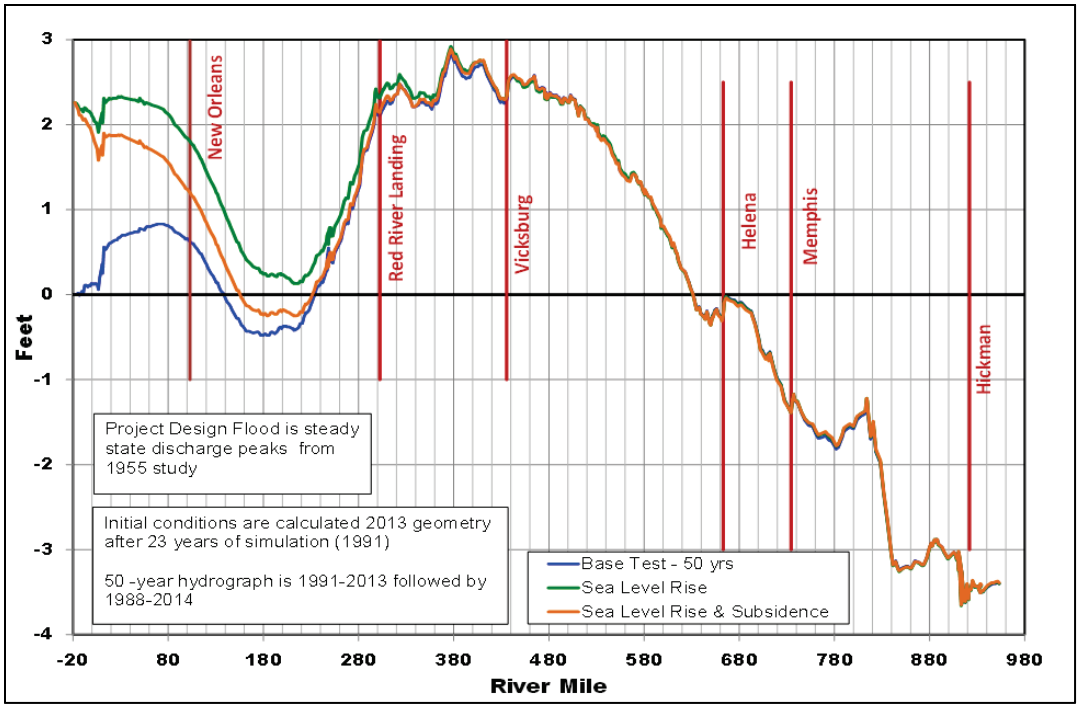
Breakdowns of the various important factors and their impacts to the sedimentation results is discussed within the Mississippi River Sedimentation Report (USACE 2018c).

4.2.5 SLR and subsidence

The effects of SLR and subsidence on sedimentation and calculated water surface elevations were determined using the HEC-6T numerical model. The water surface elevation at the downstream boundary was increased by adding an SLR component determined from Curve III of the National Research Council projection recommended in ER 1100-2-8162 (USACE 2013). Curve III provides a high estimate for SLR. The downstream boundary elevation in HEC-6T was increased for each event during the 50-year simulation using the National Research Council equation as specified on the $SLR record ("$SLR" is a HEC-6T specific type of input statement). Subsidence values at each cross section up to RM 184 were taken from the ERDC MS-Hydro model LMR2_FWOPa_NRC1_R28.t5 (www.mvd.usace.army.mil/Missions/Mississippi-River-Science-Technology/Mississippi-River-Hydro/). Subsidence calculations, in which all elevations in the cross section were lowered at a constant rate, were made each October 1 in the HEC-6T model. Calculated water surface elevations due to SLR and subsidence do not include any changes to the elevation datum that might occur for whatever reason.

The difference in PDF peak water surface elevations after 50 years of aggradation and degradation in the Mississippi River study reach is shown in Figure 4-18. Basically, the effects of SLR and subsidence are negligible upstream from Vicksburg. The 50-year differences shown in Figure 4-18 represent the combined effect of SLR, subsidence, and sedimentation.

Figure 4-18. Difference in PDF peak water surface elevations after 50 years of sediment accumulation with subsidence and NRC Curve III SLR.

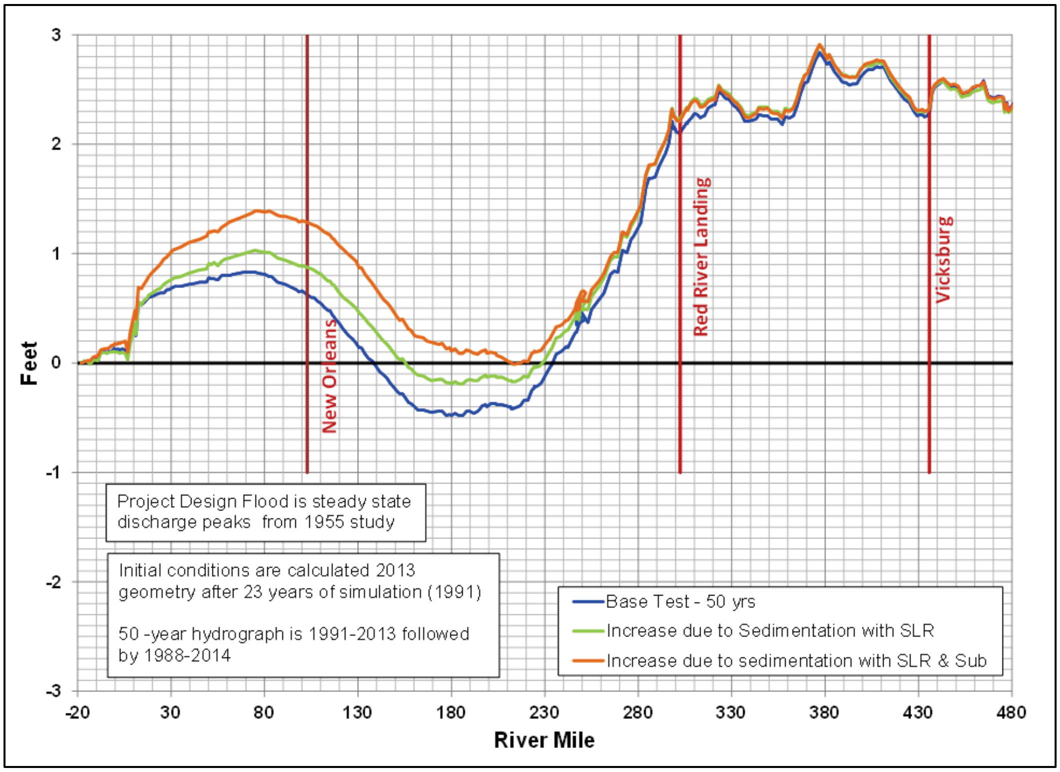


There are several uncertainties associated with the calculations. There was no differentiation between overbank and channel subsidence. The numerical model simulation made no adjustment for flow diversion percentages out the distributaries. There was no adjustment made to the dredging template.

The results of this analysis should not be added to the flowline determined by the HEC-RAS flowline study due to SLR as this would double count the effects of SLR and subsidence.

The increase in water surface elevation differences shown in Figure 4-18 is primarily due to the increase in the downstream boundary elevation and not due to increased sedimentation due to SLR and subsidence. To segregate the effects of sedimentation on the PDF peak water surface elevations with respect to both SLR and subsidence, the PDF peak water surface elevations were calculated with the initial geometry conditions (2013) adjusted for 50-year subsidence and 50-year SLR at the downstream boundary. Subtracting this value from the PDF peak water surface elevations calculated after 50 years of sediment accumulation (with both SLR and subsidence progressing with time) provided the segregated effect of sedimentation due to both SLR and subsidence and is shown in Figure 4-19. The figure shows that the 50-year increase in water surface elevation, due to sedimentation, for the PDF at New Orleans with both SLR and subsidence is approximately 1.3 ft. Note from Figure 4-18 that the total effect of SLR, subsidence, and sedimentation is an increase of only 1.2 ft. Increases in sedimentation due to subsidence is countered by decreases in water surface elevations. If HEC-RAS water surface elevations include the effects of subsidence and SLR, the differences shown in Figure 4-19 can be used to account for 50 years of sediment accumulation.

Figure 4-19. Difference in PDF water surface elevations after 50 years of sediment accumulation with NRC Curve III SLR and subsidence — attributed to sedimentation.



4.2.6 2016 Project design hydrograph

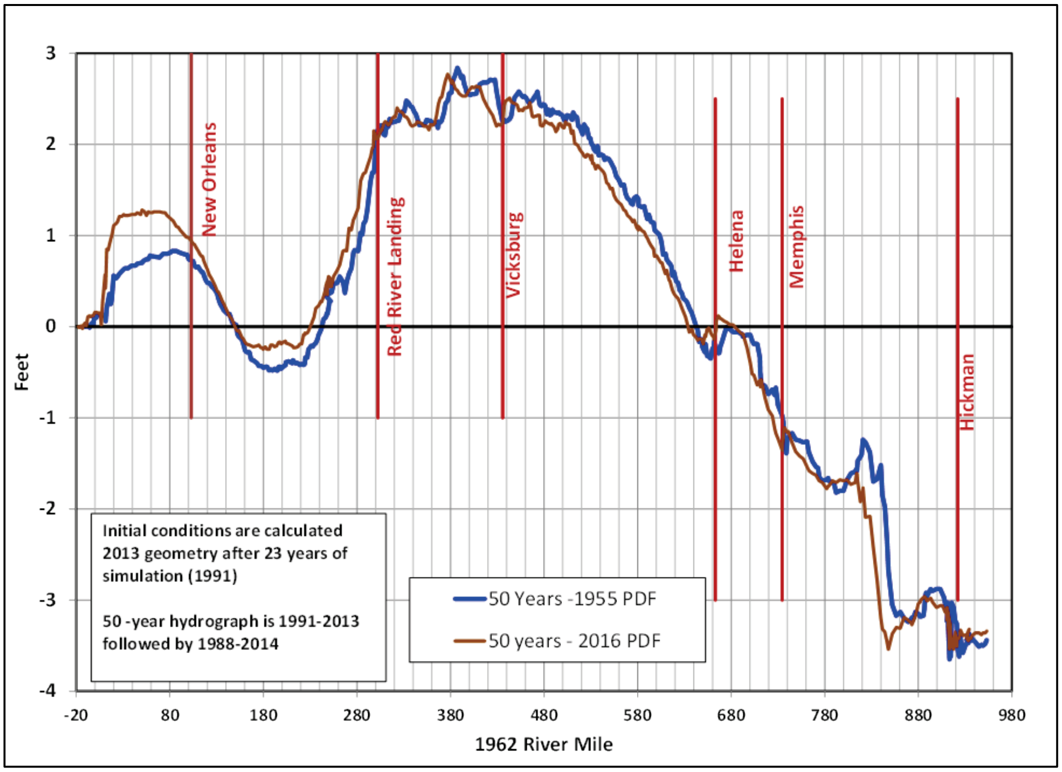
The numerical model was used to determine the difference in maximum water surface elevation due to 50 years of sedimentation for project design peaks of different magnitudes. The 1955 58A-EN PDF and the estimated 2016 PDF (58A-R Existing) peak discharges were compared in this study. The maximum calculated discharges for these two design floods are shown in Table 4-8 and discussed in more detail in the Hydrology Report (USACE 2018a).

Table 4-8. PDF peak discharges.

Mississippi River	1962 RM	2016 RM	1955 PDF 58A-EN Regulated cfs	Estimated 2016 PDF 58A-R Existing Condition From HEC-RAS cfs
Chester	109.9	109.9	240,000	508,000
Thebes	43.7	43.7	410,000	520,000
Ohio Confluence	954.3	973.8	2,360,000	2,791,000
Hickman	922.0	941.2	1,810,000	1,973,000
Memphis	734.4	748.7	2,410,000	2,863,000
Helena	663.1	676.3	2,460,000	2,788,000
Arkansas City	554.1	562.2	2,890,000	3,264,000
Vicksburg	437.0	442.2	2,710,000	2,979,000
Natchez	363.3	368.4	2,720,000	2,977,000
Red River Landing	302.0	307.1	2,100,000	2,351,000
Baton Rouge	228.4	232.5	1,500,000	1,742,000
Donaldsonville	175.4	177.4	1,500,000	1,741,000
New Orleans	102.8	107.0	1,250,000	1,486,000
Venice Discharge Range	12.4	17.85	1,250,000	1,031,000

The approach was to calculate a steady-state, water-surface profile for PDF peak discharges at the beginning of the simulation. The numerical model then calculated sedimentation changes for a 50 year period. Finally, another steady-state, water-surface profile was calculated for PDF peak discharges at the end of the 50 year simulation. The difference in the two PDF water-surface profiles is the 50 year effect of sedimentation. These differences are shown in Figure 4-20. The figure shows that results using the 1955 PDF and estimated 2016 PDF are similar.

Figure 4-20. Difference in 1955 and estimated 2016 PDF peak water surface elevations after 50 years of sedimentation.*



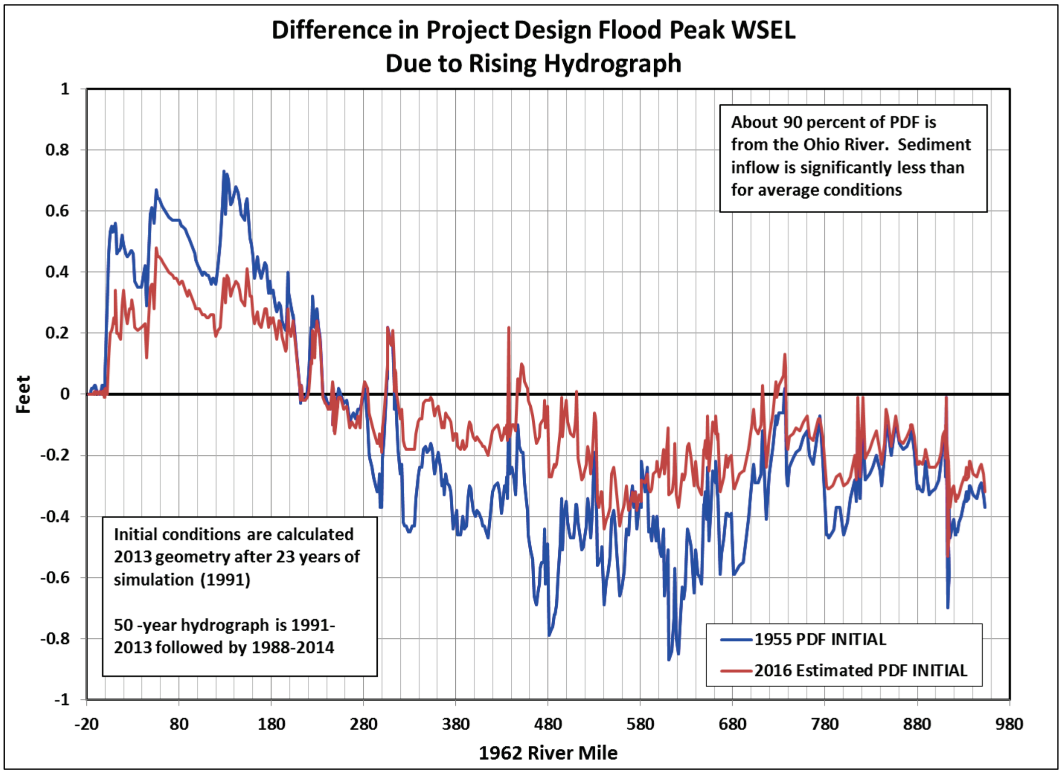
*Note: For a description of the 50-year hydrograph, see Section 4.2.1.5.

Channel geometry may change significantly during the rise of a major flood hydrograph. The channel change is not so much affected by the increase in bed sediment transport due to the increased discharge but by the difference in transport capacity between adjacent reaches. Thus, at any given time, the bed elevation of some cross sections may rise while others may decline. The numerical model was used to predict the difference in maximum PDF water surface elevations due to channel changes during the rise of both the 1955 and estimated 2016 PDF hydrographs.

The approach was to first calculate a steady-state, water-surface profile using only the peak discharges. This profile was compared to a maximum water-surface profile calculated by simulating the PDF hydrograph in the numerical model.

As shown in Figure 4-21, channel changes that occur as the flood rises result in maximum water surface elevation differences of less than 1 ft. The estimated 2016 PDF has less channel change before the flood peak than the 1955 PDF. Less degradation occurs upstream from Old River because a higher percentage of the flood is diverted into floodways and backwaters with the PDF hydrograph used in this assessment. Less aggradation occurs downstream from Old River because a lower percentage of the flood is diverted through Old River, Morganza, and Bonnet Carré with the PDF hydrograph used in this assessment.

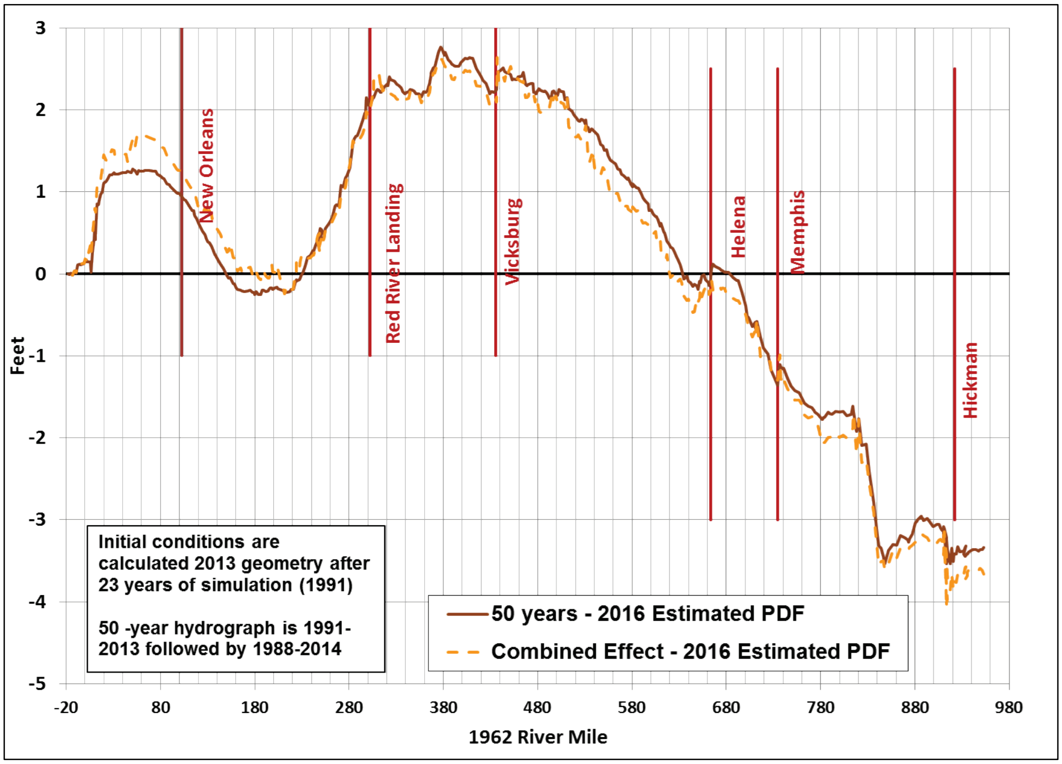
Figure 4-21. Difference in PDF peak water surface elevation due to rising hydrograph.*



*Note: For a description of the 50-year hydrograph, see Section 4.2.1.5.

The combined difference in estimated 2016 PDF water surface elevations accounting for 50 years of sedimentation and the rising of the flood hydrograph is shown in Figure 4-22.

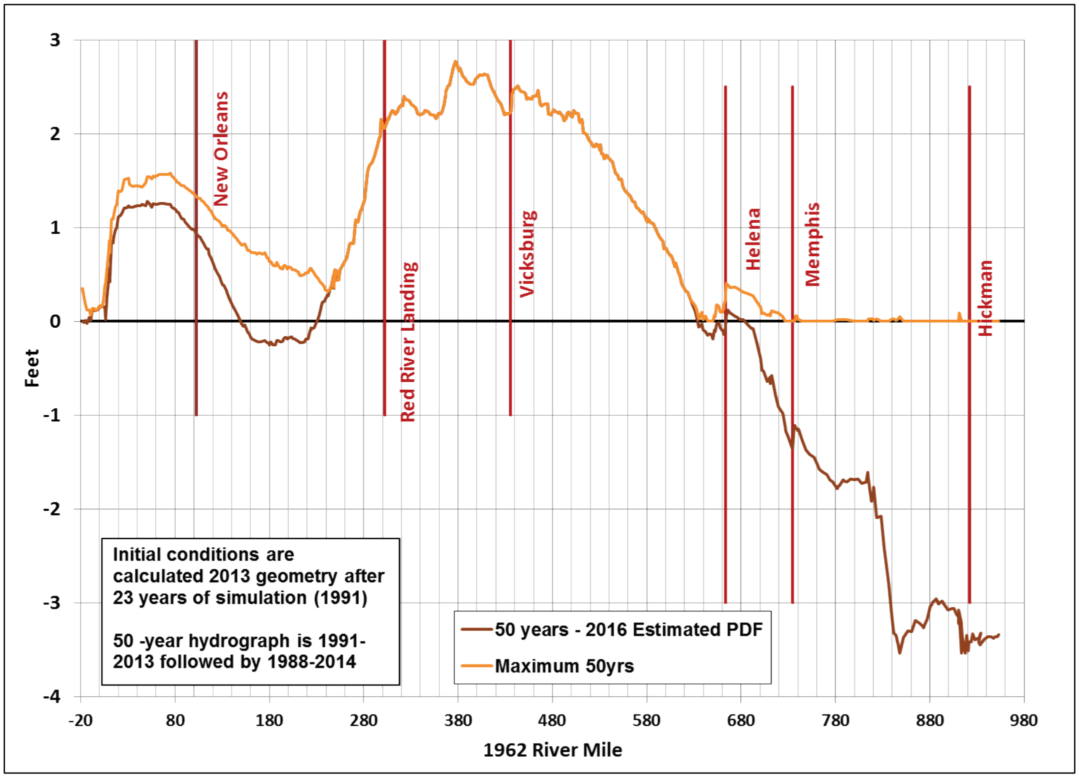
Figure 4-22. Combined effect of 50 years of sedimentation and the rise of the hydrograph.*



*Note: For a description of the 50-year hydrograph, see Section 4.2.1.5.

Since the PDF may occur at any time during the 50-year simulation, the differences in Project Flood water surface elevations at every year during the 50-year simulation were calculated. The maximum difference is plotted in Figure 4-23. This was accomplished by inserting the estimated 2016 PDF peak discharges into the 50-year hydrograph on March 31 of each year. The time-step was set very small so that there was no bed change during the peak flows. As shown in Figure 4-23, the maximum differences in water surface elevations between Head of Passes (RM 0) and RM 240 (upstream from Baton Rouge) occur before the end of the 50-year simulation. Likewise, the maximum water surface elevations upstream from RM 640 (just downstream from Helena) occur before the end of the 50-year simulation. Upstream from Memphis, the maximum water surface elevations occur near the very beginning of the simulation period.

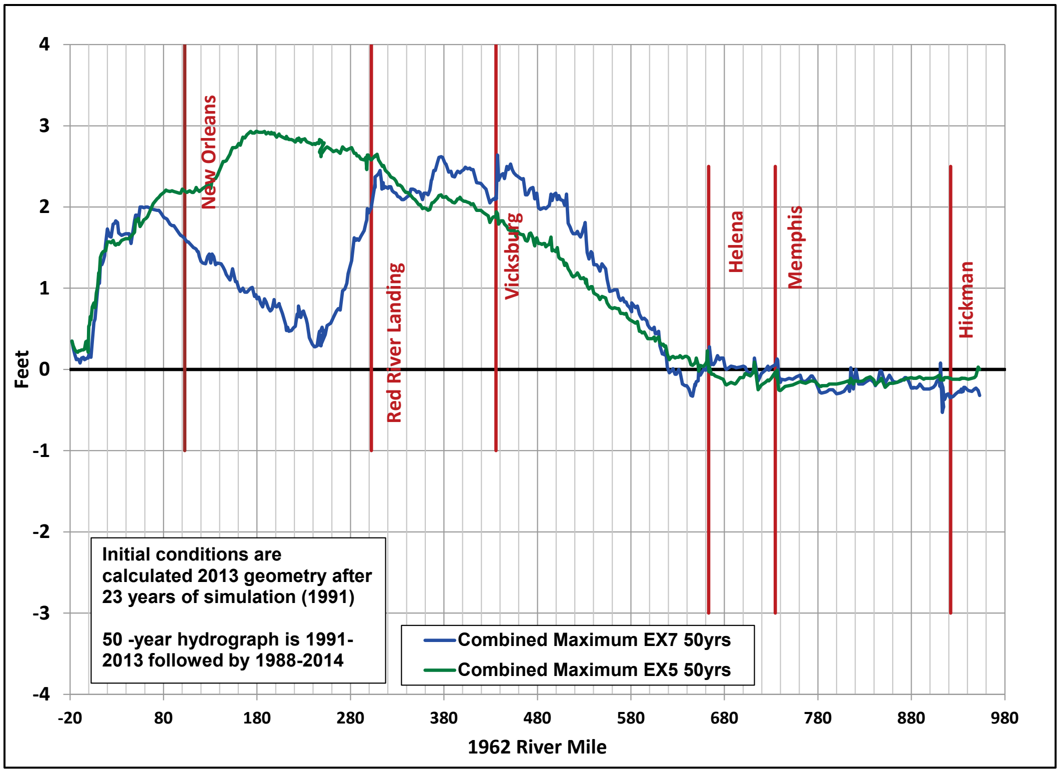
Figure 4-23. Maximum difference in Project Flood peak water surface elevation due to sediment aggradation/degradation over 50 years.*



*Note: For a description of the 50-year hydrograph, see Section 4.2.1.5.

The calculated increase in the estimated 2016 PDF water surface elevations to account for 50 years of sedimentation is shown in Figure 4-24. The preferred armoring algorithm used in the calculations is Exner 7. Figure 4-24 combines the calculated increase in water surface elevation over the 50 years with the difference in water surface elevation at the peak of the hydrograph related to sedimentation during the rise of the flood hydrograph using either Exner 5 or Exner 7. Calculated differences were determined every year during the 50-year period to account for the fact that the PDF could occur at any time during the 50-year simulation. Maximum water surface elevations are less than zero at some stations because degradation occurs during the rise of the PDF hydrograph, and when the hydrograph results are compared with a steady state peak discharge at time zero, the result is negative. Calculated results using Exner 5 are also included. Conservative engineering practice would suggest using the highest calculated results for levee height design. Tabulated results for the sedimentation impacts can be found in the Mississippi River Sedimentation Report (USACE 2018c).

Figure 4-24. Maximum increase in estimated 2016 PDF water surface elevations due to sedimentation, including the rise of the flood hydrograph, over a 50-year period. Calculated with Exner 7 and Exner 5.



4.2.7 Conclusions

Modeling indicates that sedimentation in the Mississippi River over the next 50 years could cause the peak water surface elevations of the project flood to increase to 2.5 ft between Head of Passes (RM 0.0) and Helena (RM 663). During the 50-year period, peak water surface elevations between Helena and Cairo (RM 953) are predicted to decrease. The Project Flood could occur at any time during the 50-year period, so levee design heights need to account for design water surface elevations should the flood occur today as well as 50 years in the future.

Results from the HEC-6T sedimentation study should be combined with results of the unsteady-flow HEC-RAS study to determine maximum PDF water surface elevations for levee design along the Mississippi River between Cairo and Venice. The HEC-6T study predicted the maximum increase in water surface elevation due to 50 years of sedimentation. The HEC-RAS study predicted the maximum increase in water surface elevation due to other hydraulic factors.

Sedimentation effects due to the rise of the estimated 2016 Project Flood hydrograph are predicted to result in an increase in peak water surface elevations of less than 0.5 ft between Head of Passes and RM 280 (upstream from Baton Rouge) and a decrease in peak water surface elevations of less than 0.5 ft between RM 280 and Cairo.

There is a degree of uncertainty related to choice of the Exner equation in the numerical simulation, so a sensitivity analysis was performed. Based on experience gained from previous HEC-6T studies, results calculated using the Exner 7 algorithm are preferred. However, conservative design principles suggest using the higher increases in design water surface elevation using Exner 5. The estimated 2016 PDF peak water surface elevations are projected to increase to 3.0 ft between Head of Passes (RM 0.0) and Helena (RM 663) with the Exner 5 algorithm.

The most significant driving variable causing the degradation trend upstream from Helena and the aggradation trend downstream is the long-term geomorphic response to the USACE cutoff program that shortened the river by approximately 152 miles in the 1930s and 1940s, or 170 miles including natural cutoffs that also occurred in this time period. Reduction in sediment inflow due to land use changes, reservoirs, locks and dams, and bank protection has affected the deposition patterns but not the long-term trend. Likewise constriction of the river by dike fields has affected sedimentation but not the long-term trend. The long-term trend does not appear to be significantly accelerated by individual flood years. Rather, the trend appears to be steady and continuous. The long-term trend continues past the 50-year study period for at least an additional 50 years.

These conclusions apply to engineering time scales. Over longer time scales, sedimentation processes could change due to changes in boundary variables. For example, sediment inflow from the Middle Mississippi River could change or the composition of the bed sediment reservoir could change as the bed continues to degrade. The coarsest bed materials being supplied to the Lower Mississippi River are not transported all the way to the Gulf of Mexico. These coarse materials could eventually form an armor layer. In addition, if the finer sands are eventually exhausted from the bed sediment reservoir to the point where the volume of coarse material deposition exceeds fine material extraction, then the river response will be an increase in slope (and upstream stages). This change most likely would not be uniform in space or time.

Development of the HEC-6T numerical model of the Lower Mississippi River between Cairo and Pilots Station should not be considered complete. As additional data become available and as the river continues to adjust its geomorphic form, the model should be updated. For example, measurements taken during the January 2016 flood near Morganza, LA, indicated that the channel flow was less than two-thirds of the total flow of 1,300,000 cfs. Treatment similar to that conducted for Devils Swamp in this assessment (see Mississippi River Sedimentation Report for more details, USACE 2018c) should be considered in future studies for wide floodplains inside the flood control levees.

4.3 Sedimentation of the Atchafalaya River

4.3.1 Objective

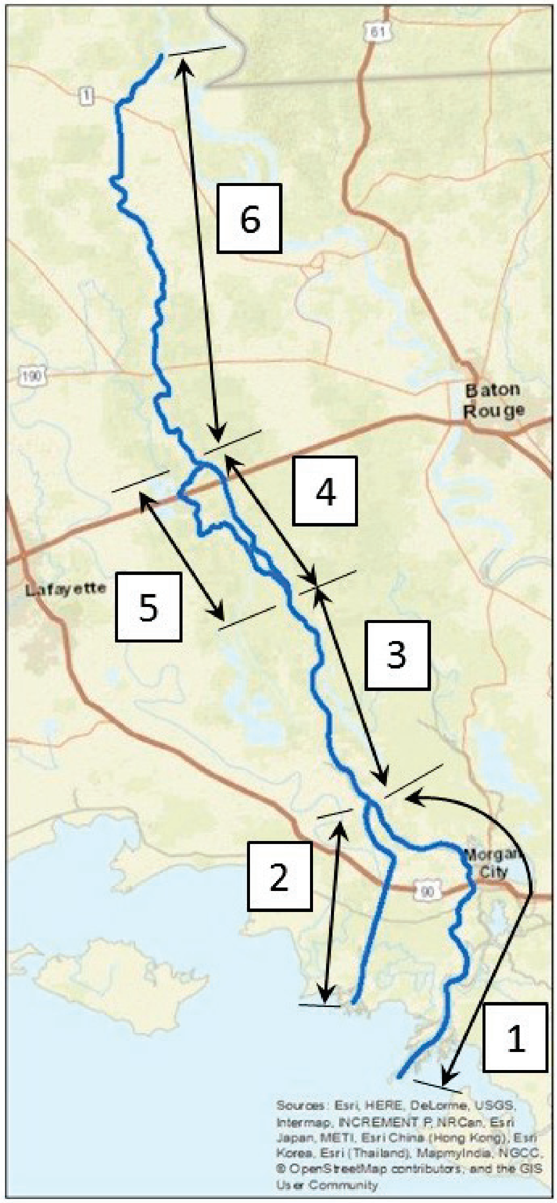
This section summarizes the numerical sedimentation modeling of the Atchafalaya River. Fifty years of hydraulic and sediment conditions were simulated, through the year 2066, to determine channel changes that could impact the water surface elevations of the PDF event for the MR&T System. The resulting channel changes were then provided to the Atchafalaya River HEC-RAS model for simulating water surfaces during the PDF event. More detailed information on this effort can be found in the Atchafalaya River Sedimentation Report (USACE 2018d).

4.3.2 Background

The MR&T System contains several water management strategies that were developed to pass the high flows that could occur during the PDF. These strategies include employment of backwater areas, floodways, historical cutoffs, and diversions. During the PDF event, the Atchafalaya River is intended to provide a large relief to the Mississippi River, receiving one-half of the peak flow passing Natchez, MS. A major diversion of water from the Mississippi River to the Atchafalaya River occurs at the ORCC. The ORCC is presently authorized to divert 620,000 cfs. Second, the Morganza Control Structure is authorized to pass up to 600,000 cfs through the Morganza Floodway into the Atchafalaya River during the PDF. During the PDF event, the Atchafalaya River carries 1.5 million cfs to the Gulf of Mexico. For these reasons, the Atchafalaya River is an important part of the MR&T System strategy for passing the PDF.

Sedimentation changes in the Atchafalaya River were last studied by USACE in the 2010 Refined Project Flood Flow Line Report for the Atchafalaya Basin (USACE 2010). The 2010 Report superseded the 1986 Design Flow Line (as reported in the January 1987 Atchafalaya Basin Project Flood Flow Line Report). The numerical sedimentation model used for the 2010 Report was provided by the MVN to the authors for this assessment. In many ways, the effort summarized in the Atchafalaya River Sediment Report (USACE 2018d) is intended to be an update to the 2010 Report. The changes that were required to adequately calibrate and validate the model to recent data are described therein. The layout of the revised HEC-6T model used in this assessment is shown in Figure 4-25; the reach segment numbers are labeled by the boxes in the figure.

Figure 4-25. Schematic of the HEC-6T model segment numbers.



The 2010 Report (USACE 2010) and the work of Nordin and Posada (1996) document major changes of the Atchafalaya River over the course of the last several decades due to changing inflows and decreased sediment loads. Notable changes to the system include the construction of the Old River Control Structure in 1963. The Auxiliary Structure and Hydropower Structures were added to the ORCC in 1986 and 1990, respectively. Additionally, the J. Bennett Johnston Waterway, a system of locks and dams through the Red River, was constructed from 1984 to 1994. Measured data at the Simmesport gage show increased water inflows and decreased sediment inflows to the Atchafalaya River. The upper end of the Atchafalaya River has experienced degradation, as seen by a large drop in the specific gage behavior at Simmesport, LA, between the 1940s and 1990s. Since the mid-1980s, the Simmesport specific gage behavior has been more stable. However, the 2010 Report (USACE 2010) observed aggradation of the lower end of the Atchafalaya River in recent decades. Atchafalaya River future channel changes are evaluated in the Atchafalaya River Sedimentation Report (USACE 2018d).

4.3.3 Model validation

The previous numerical sediment model provided to the authors was a HEC-6T model based on geometry from a 1997 bathymetric survey. Since the previous HEC-6T model was used in the 2010 Report (USACE 2010), it will be referred to as the 2010 HEC-6T model within this section. The 2010 HEC-6T model was revised and used in this assessment to simulate the calibration and validation time period from 1997 to 2016. A multibeam bathymetric survey and topographic survey were merged to create a new surface for the year 2010 (for more information, see the Atchafalaya River section of the Hydraulics Report, USACE [2018b]). Simulated channel changes from 1997 to 2010 were compared to the 2010 survey data. In general, the model was validated by matching the hydraulic behavior (stages and discharges), the measured sediment concentrations, and randomly selected cross-section locations of the 2010 survey data.

4.3.3.1 Data compilation

There are multiple gage locations with data used for this model. Most of the gages record only the river stage and occasional field discharge measurements. A time series of discharge data was available from the Simmesport (USGS site # 07381490), Calumet (USGS site #07381590 Wax Lake Outlet at Calumet, LA), and Morgan City gages (USGS site #07381600). The 2010 HEC-6T model simulated historical conditions from 1997 to 2007. For this effort, all of the available stage and discharge data for any of the gages through 2016 was retrieved.

All vertical elevations in this section are in NAVD 88. The report for the 2010 HEC-6T model (both the Main Report [USACE 2010] and the Appendix B) states that the previous model vertical elevations were all NAVD 88. All other data retrieved for comparisons within this investigation used, or were converted to, NAVD 88.

Sediment data were also available at some useful locations within the model domain, namely Simmesport, Mellville, Calumet, and Morgan City. The USGS provided total sediment concentrations and the percent finer than 0.0625 mm.

A survey data set was available for model validation, made of a 2007 lidar topographic survey and a 2010 multi-beam bathymetric survey. The topographic and bathymetric surveys were merged together to create a single surface (the horizontal coordinate system was NAD_1983_StatePlane_Louisiana_South_FIPS_1702_Feet). Some areas of the model domain did not have bathymetric data from the 2010 survey, namely the Old Atchafalaya River Channel parallel with the Whiskey Bay Pilot Channel, the Wax Lake Outlet Channel, and the Lower Atchafalaya River below the GIWW. The HEC-6T model was validated to bed changes from 1997 to 2010 where 2010 data were available.

4.3.3.2 Model changes

The 2010 HEC-6T model had been run with a much older version of the HEC-6T software, and it would not run successfully with recent versions. Software improvements, bug fixes, and added capabilities required that the old model be revised to run with a current version of the HEC-6T software (H6TV51425_J01nR_170519). One specific required change was the re-labeling of the dredging sites due to software changes. In general, the approach of the authors was to review all features of the previous model but only change model features which either

- resulted in a better match to available data sets, such as distributary flow data, bed gradation data, etc.
- were necessary to correct mistakes, instabilities, or inconsistencies within the model
- caused a better alignment with the new HEC-RAS hydraulic model (see Section 4.3.5.2 for more information).

The full details of all model changes are described in the Atchafalaya River Sedimentation Report (USACE 2018d).

4.3.3.3 Boundary conditions

The typical boundary conditions required for a HEC-6T model are the flow and sediment arriving from upstream and the water surface elevation downstream. The 2010 HEC-6T model included a small part of the Red River and the ORCC outflow channel. The 2010 model's boundary conditions were based on Simmesport data with the unknown Red River flow determined by subtracting the measured ORCC outflow. Due to some discrepancies in the calculation of water and sediment inflow from the Red River, and the fact that the HEC-RAS model began at Simmesport, the upstream boundary in this study was moved closer to Simmesport. The current model begins just downstream of where the ORCC outflow channel joins the Atchafalaya River. Inflows were entirely based on Simmesport data, which is only a short distance downstream from the model boundary.

Inflow at Simmesport, LA

Water and sediment inflows were required boundary conditions at the upstream end of the model. Water inflow came from daily data available for Simmesport, LA, through river gages (http://rivergages.com, site #03045Q). Sediment inflow relationships were re-developed within this project due to the availability of more recent sediment measurement data at Simmesport, LA. The detailed steps in determining the sediment inflow curve can be found in the Atchafalaya River Sedimentation Report (USACE 2018d).

Morganza Control Structure (MCS)

The MCS, or the Morganza Floodway, is another boundary condition flowing into the model. The 2010 HEC-6T model introduced the entire amount of Morganza flow into the location where the levee ends, just upstream of the Whiskey Bay Pilot Channel. Comparing results of water flow in the channel of the HEC-RAS model, which had modeled the Morganza Floodway with 2D flow elements, showed that the HEC-6T model was carrying too much flow in the channel through the reach beginning at that location. The Morganza flow stays in the floodway to the east of the Atchafalaya River for much longer. The lidar surface elevations were evaluated for potential locations where the Morganza flow could enter the HEC-6T model. An AdH hydraulic model under development for the Atchafalaya River Basin was also evaluated (phase two of the work by Bell et al. [2017]). After evaluating the terrain and the 2D modeling results, it was decided to distribute the MCS flow to the Lower Atchafalaya River into three different cross sections between the junction with Wax Lake Outlet Channel and Morgan City, namely into cross sections -104.8, -

112.5, and -120.1. Other details about the MCS boundary condition can be found in the Atchafalaya River Sedimentation Report (USACE 2018d).

Gulf of Mexico

The 2010 HEC-6T model used daily water level data from the Eugene Island gage from 1997 through 2007. Data were retrieved from the NOAA Tides and Currents website (Station IDs 8764311 and 8764314) where available. Data were also provided by MVN to fill in some of the gaps of available data from the NOAA website. A remaining gap for December 12, 2013, through April 1, 2015, was filled using a value of 0.5 ft, calculated from averaging the available data in the neighboring years 2013 and 2015.

Since the Wax Lake Outlet Channel was significantly shortened in this study to better represent the 1D flow, as discussed in the Atchafalaya River Sedimentation Report (USACE 2018d), an offset was used between the Eugene Island data and stage used at the downstream end of the Wax Lake Outlet Channel. The offset was calculated to be 0.3 ft from evaluating the water surface differences in the HEC-6T model before eliminating the downstream cross sections. The downstream water surface boundary condition was 0.3 ft higher for the Wax Lake Outlet Channel (Segment 2) than the Lower Atchafalaya River (Segment 1) for each time-step.

4.3.3.4 Roughness coefficients

The hydraulic calibration was achieved by adjusting roughness, or Manning's *n*-values such that the simulation of 1997 to 2010 represented the observed data. At the downstream end, the roughness values in the Lower Atchafalaya River and Wax Lake Outlet Channel influence not only the water surfaces in those channels but also the flow distribution between the two channels. Manning's *n*-values, varying as a function of discharge, in the downstream segments of the model were specified and adjusted during calibration. The gages at Calumet (stage and discharge), Morgan City (stage and discharge), and Avoca Island (stage only) were used for calibrating roughness values in the downstream end of the model. Parallel channels exist through the large bend in the Lower Atchafalaya River upstream from Morgan City. Laterally specified roughness values were decreased within the side channel areas of those cross sections, and the island areas between channels remained relatively rough. Roughness values were only slightly adjusted from the 2010 HEC-6T model in other, upstream segments during the calibration process.

4.3.3.5 Results

The calibration and validation results agree with observed stage, discharge, and sediment concentration data at gage locations. The root mean square error (RMSE) and the Nash-Sutcliffe Efficiency (NSE) coefficient were calculated to quantify errors between observed and computed water surface and discharge data, as shown in Table 4-9. The NSE can range from -∞ to 1, with a value of 1 corresponding to a perfect match between modeled and observed data. Figure 4-26 shows an example of validation results for the water surface data at the Melville, LA, location. Stages, discharges, sediment concentrations, and cross-section changes are compared in more depth between computed and observed data for the validation time period within the Atchafalaya River Sedimentation Report (USACE 2018d).

Table 4-9. Metrics for hydraulic differences between computed and observed values.

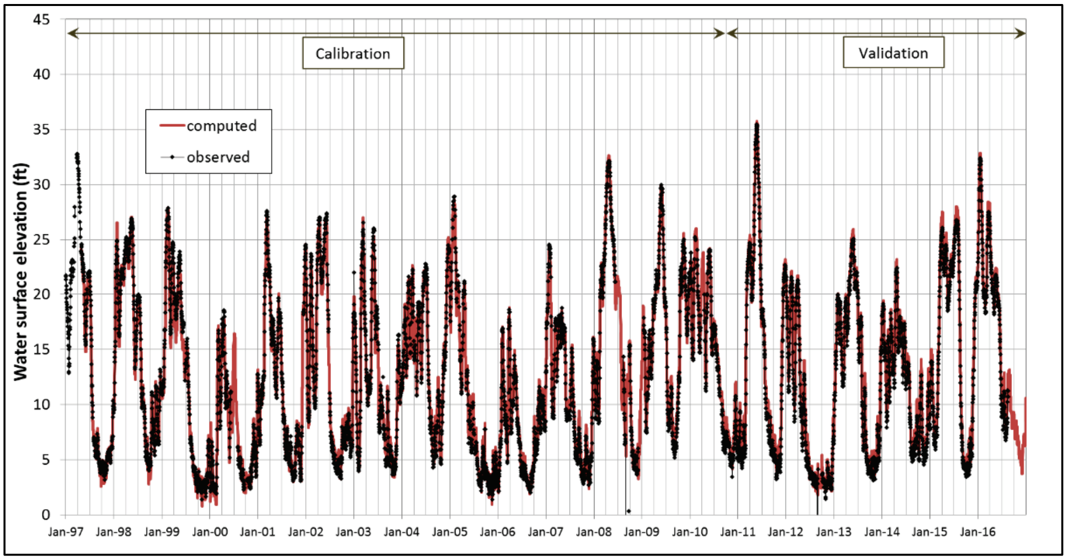
Location	Water Surface Elevation			Discharge		
	Average bias* (ft)	RMSE† (ft)	NSE‡	Average bias* (ft)	RMSE (ft)	NSE
Avoca Island Cutoff south of Morgan City, LA	0.15	0.47	0.481			
Lower Atchafalaya River at Morgan City, LA	-0.47	0.66	0.842	-890	14,270	0.951
Wax Lake Outlet at Calumet, LA	-0.54	0.70	0.842	1840	11,080	0.957
Myette Point near Charenton, LA	0.46	0.64	0.948			
Butte La Rose, LA	1.10	1.30	0.937			
Melville, LA	0.14	0.72	0.990			
Simmesport, LA	-0.19	0.78	0.993		810	1.000**

†RMSE measures the error between the computed and observed values. ‡NSE measures the predictive power of the model. Blank cells indicate that observations are not available.

*The average bias is based on computed minus observed data.

**Agreement is due to using the observed data only a few miles away as the boundary condition.

Figure 4-26. Simulated and observed water surfaces for the Atchafalaya River at Melville, LA.



4.3.4 Simulation of the future

4.3.4.1 Simulation setup

Upstream flow boundary condition

The validated model was used to predict sedimentation changes 50 years into the future, from 2017 through 2066. Upstream flow data for 2017 through 2066 came from the recorded discharges at Simmesport. Since the distribution of flow through ORCC changed significantly after 1976, data prior to 1977 were not used. The time period 1977 to 2016 was used to simulate 2017 to 2056. The remaining time period 2057 through 2066 used data from 2007 through 2016 to represent recent conditions as much as possible. Table 4-10 clarifies the dates of Simmesport recorded discharge used to simulate 50 years into the future. The only time there was flow through MCS during the time period used for inflows was 2011, so the attenuated 2011 MCS inflows, as discussed in the Atchafalaya River Sedimentation Report (USACE 2018d), were used for those particular times.

Table 4-10. Data used for inflow boundary condition.

Simulation Date	Date of Data Source (using Simmesport discharge)
1997 to 2016	1997 to 2016
2017 to 2056	1977 to 2016
2057 to 2066	2007 to 2016

Sea level rise (SLR)

Beginning January 1, 2017, the downstream water surface boundary condition used the intermediate SLR scenario from ER 1100-2-8162. The equation in the appendix of ER 1100-2-8162 (USACE 2013) is applied to each day after December 31, 2016, to calculate the downstream water surface using the recommended base year 1992. The Wax Lake Outlet kept the 0.3 ft offset for a higher water surface at the downstream end of Segment 2, as discussed in the Atchafalaya River Sedimentation Report (USACE 2018d).

Subsidence

Runs were performed with subsidence turned both on and off. Rates of subsidence were already present within the 2010 HEC-6T model. These rates varied spatially such that each cross section had a slightly different rate based on its location. Results showing the impact of subsidence are found in the Atchafalaya River Sedimentation Report (USACE 2018d).

Effects of the PDF

The primary objective of the Atchafalaya River HEC-6T model was to determine the bed changes due to the 50 years of sedimentation. However, the bed is expected to also change during the hydrograph of the PDF event, so the bed could be significantly different at the peak of the event than before the event. To assess this impact, one scenario simulated the peak flows of the PDF instantaneously on December 1, 2066. The other scenario used the full amount of time for the PDF (from the "58A-R Authorized Yazoo" scenario in the Mississippi River HEC-RAS model), which begins in December 1 and reaches a peak inflow to the Atchafalaya River on March 8. For this second scenario, the PDF-event daily flows were implemented in the HEC-6T model from December 1, 2066, through March 8, 2067.

A separate simulation was also performed to assess the changes caused by the PDF event with the 2010 conditions. For this simulation, the PDF peak flows were simulated instantaneously between November 30, 2010, and December 1, 2010, and then the PDF event was simulated from December 1, 2010, through March 8, 2011.

For comparison purposes, the 2011 event was also simulated separately to determine the effects of bed changes during the rise of the 2011 event. For that event, the 2011 peak flows were simulated instantaneously between September 30, 2010, and October 1, 2010, and then the 2011 event was simulated until it reached the peak flows.

4.3.4.2 Results

Wax Lake Outlet

Wax Lake Outlet is an artificial distributary of the Atchafalaya River. The distributary junction is located at Atchafalaya RM -100 (1963) where Grand Lake intersects the river. Shaw et al. (2013) provided the following description of the Wax Lake Outlet/Lower Atchafalaya River bifurcation:

> A 13 to 21m deep channel connects the upstream and downstream Atchafalaya River, and hosts sand dunes. The southwest side of the Atchafalaya River and the entrance to the Grand Lake Reach is shallow (4–7m) and is composed entirely of exposed bedrock. A steep scarp exists between the deep Atchafalaya River and the shallow Grand Lake Reach (4–67% slope). The 200m wide ramp extending toward the Grand Lake Reach was initiated by dredging and hosts some sand dunes. This ramp is the only gradual transition from the sand-rich thalweg and the shallow Grand Lake Reach.

After flowing approximately 8 miles through Grand Lake, diverted flows enter the Wax Lake Outlet, which was dredged in 1941 and connects Grand Lake to Wax Lake in the Atchafalaya Bay. The Wax Lake Outlet was designed to lower flood stages at Morgan City, which is located on the Lower Atchafalaya River approximately 20 miles downstream from the distributary junction.

The beds of both Grand Lake and the dredged outlet channel consist of erosion-resistant clay materials. Shaw et al. (2013) characterized the bed of Grand Lake and the Wax Lake Outlet as bed rock because the bed “is a relict deposit that precedes and is not associated with the sediment transport system, and is a cohesive, exposed surface covered by erosional bed forms and tool marks demonstrating that the material is resistant to erosion.” Although erosion resistant, the bed is still subject to scour. Shaw et al. (2013) determined that between 1942 and 1964 the upstream 5-mile reach of the dredged channel experienced an average erosion rate of 0.92

ft/year. Between 1964 and 2006, the average erosion rate in this reach decreased to 0.31 ft/year. The next 8 miles of the dredged channel had an average erosion rate of 0.15 ft/year between 1942 and 1964, increasing to 0.30 ft/year between 1964 and 2006. Shaw et al. (2013) determined that the Grand Lake reach eroded at a rate of 0.12 ft/year between 1964 and 2006. Data were insufficient to determine a rate for 1942–1964.

Due to the bed erosion in the Wax Lake Outlet, the percentage of the Atchafalaya River flows diverted has increased. Powell (1996) reported that the Wax Lake Outlet was designed in 1941 to divert 20% of the Atchafalaya River discharge, but by 1988 that percentage had increased to 42%. A weir was installed in 1988 to direct more discharge down the Lower Atchafalaya River because it had been aggrading. The weir decreased the Wax Lake Outlet diversion to 30%. However, it was removed in 1994 because stages at Morgan City increased by approximately 1 ft. (Powell 1996). Mossa (2016) reported that the flow down the Wax Lake Outlet is now (2016) equivalent to the flow down the Lower Atchafalaya River.

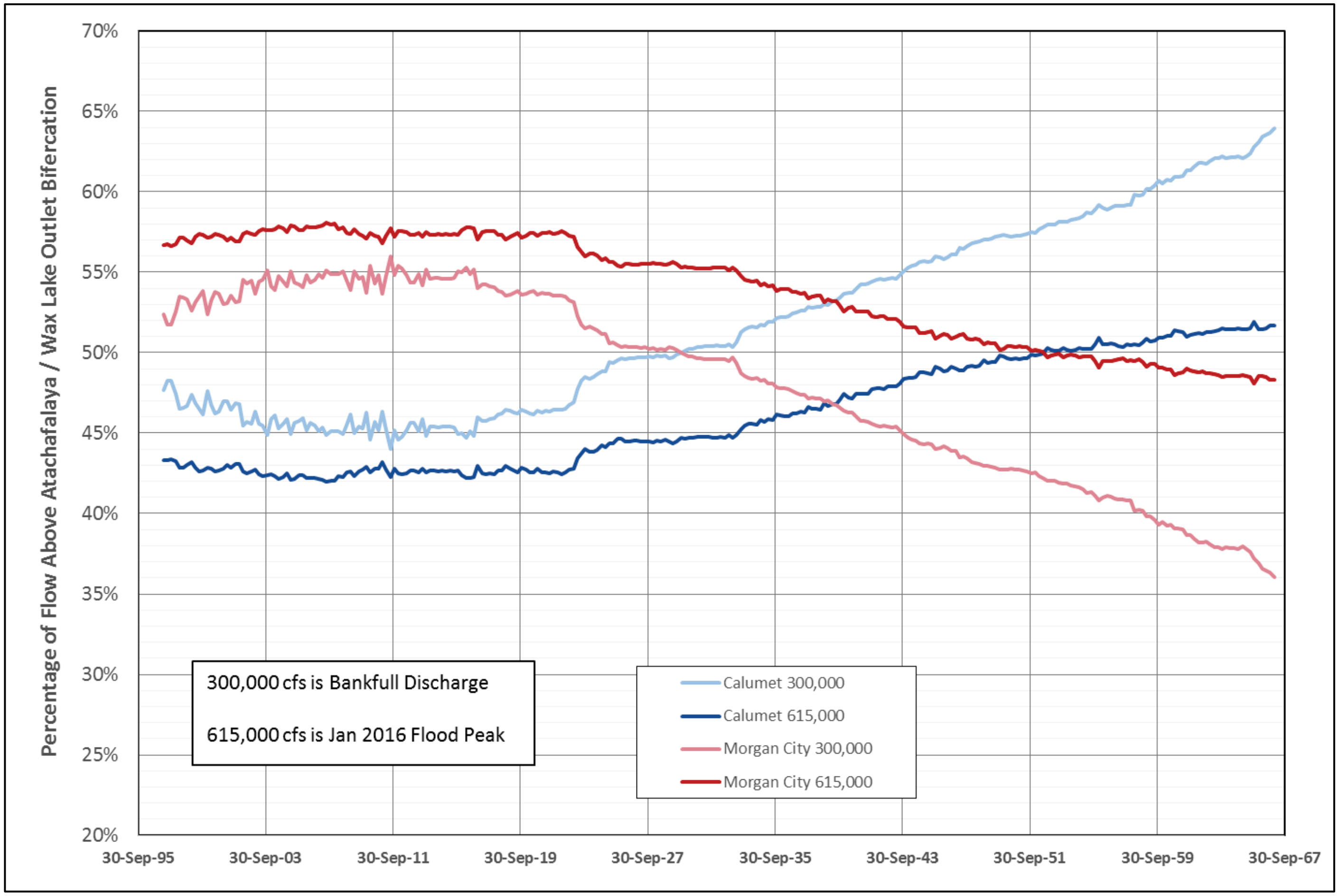
In this study, channel bed elevations in Grand Lake and in the Wax Lake Outlet upstream from the GIWW were treated as non-erodible. Cross sections in these reaches were assigned a bed sediment reservoir depth of 0.1 ft, which effectively prevents erosion below 1997 bed elevations. However, sediment deposited at any cross section could be eroded later in the simulation. Downstream from the GIWW, erosion was allowed. Bed sediment reservoir thicknesses in this reach were set between 4 and 18 ft. This assignment of bed sediment reservoir depths produced the most realistic simulation of channel bed elevation changes during the calibration period between 1997 and 2010. Model results could be improved if erosion rates were determined for the cohesive beds in Wax Lake and Grand Lake and then incorporated into the numerical model. This would require a bed sampling collection and analysis program.

The progression of Wax Lake Outlet diversion percentages with time, calculated by HEC-6T, is shown in Figure 4-27. The percentages of discharge upstream from the bifurcation that flow down the Wax Lake Outlet at Calumet and down the Lower Atchafalaya River at Morgan City at discharges of 300,000 cfs and 615,000 cfs were compared. The flow of 300,000 cfs represents a bankfull discharge at Simmesport, and 615,000 cfs was the peak of the January 2016 flood at Simmesport and was chosen to represent a flood discharge. Bankfull discharge is the maximum discharge that can be contained within the channel without over-topping the banks. The figure shows a decreasing trend in flow percentage down the Lower Atchafalaya River. The percentage of the bankfull flow diverted to Morgan City is predicted to decrease from 54% to 30% over the next 50 years (2016–2066). The percentage of flood flow diverted to Morgan City is predicted to decrease from 57% to 46% over the next 50 years. The numerical simulation also calculated significant aggradation in the Lower Atchafalaya River during this same time period. Note that the simulated results do not account for probable scour in the Wax Lake Outlet. The calculated increase in flow percentage down the Outlet in the numerical model is primarily due to reduced conveyance capacity in the Lower Atchafalaya.

A test was conducted with the numerical model to estimate the effect of potential scour in the Wax Lake Outlet on the flow diversion percentages at the Wax Lake Outlet bifurcation. In the test, restrictions on channel erosion in Grand Lake, the dredged outlet channel, and the Wax Lake delta were removed so that the channel bed was treated as fully alluvial. This assignment of bed sediment reservoir depth is considered unreasonable and was calculated only for comparison purposes. The progression of Wax Lake Outlet diversion percentages with time, calculated by HEC-6T, is shown in Figure 4-28. Figure 4-28 shows a decrease in flow down the Lower Atchafalaya River in both the simulation period between 1997 and 2016 and for the next 50 years. The percentage of the bankfull flow diverted to Morgan City would be predicted to decrease from 47% to 15% over the next 50 years. The percentage of flood flows diverted to Morgan City would be predicted to decrease from 54% to 33% over the next 50 years.

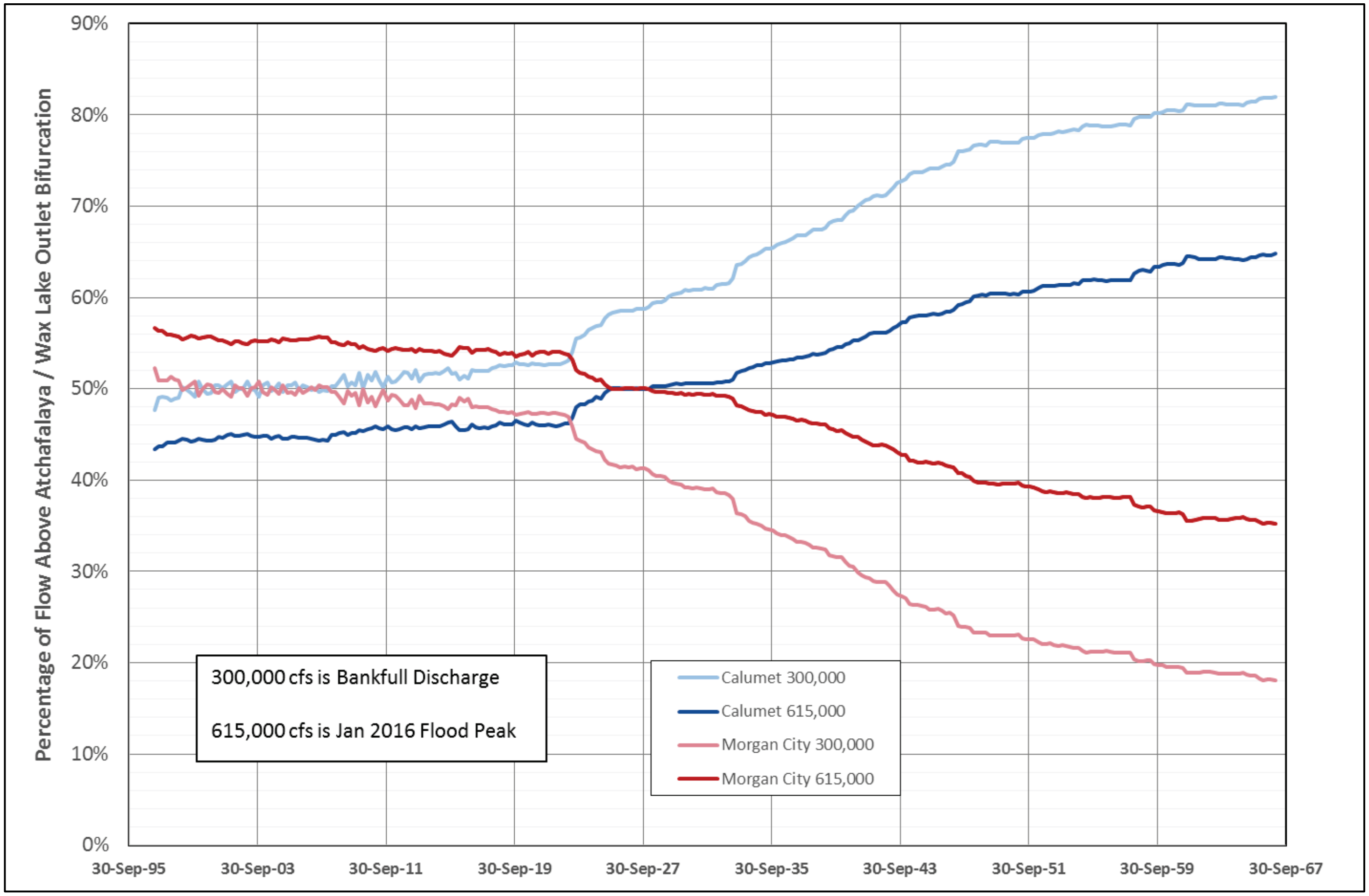
The numerical model also calculated significant aggradation in the Lower Atchafalaya River during this 2010 to 2066 time period for both the adopted bed condition and the fully alluvial bed condition. Calculated average bed changes between 2010 and 2066 for the adopted bed erosion limits and the test with no limit on bed erosion are compared in Figure 4-29. In the Wax Lake Outlet (Segment 2) in the vicinity of RM -110, significant scour is shown in Figure 4-29 for the adopted bed conditions. Even though a fixed bed was assigned in this reach for the 1997 initial geometry, deposition occurred between 1997 and 2010. It is this deposited material that erodes as the simulation progresses. The figure shows a long-term aggradation trend for the Lower Atchafalaya and a long-term degradation trend in Wax Lake Outlet. The existing clay bed in the Wax Lake Outlet is preventing a much more severe degradation trend.

Figure 4-27. Changes in discharge percentages at Calumet and Morgan City.*



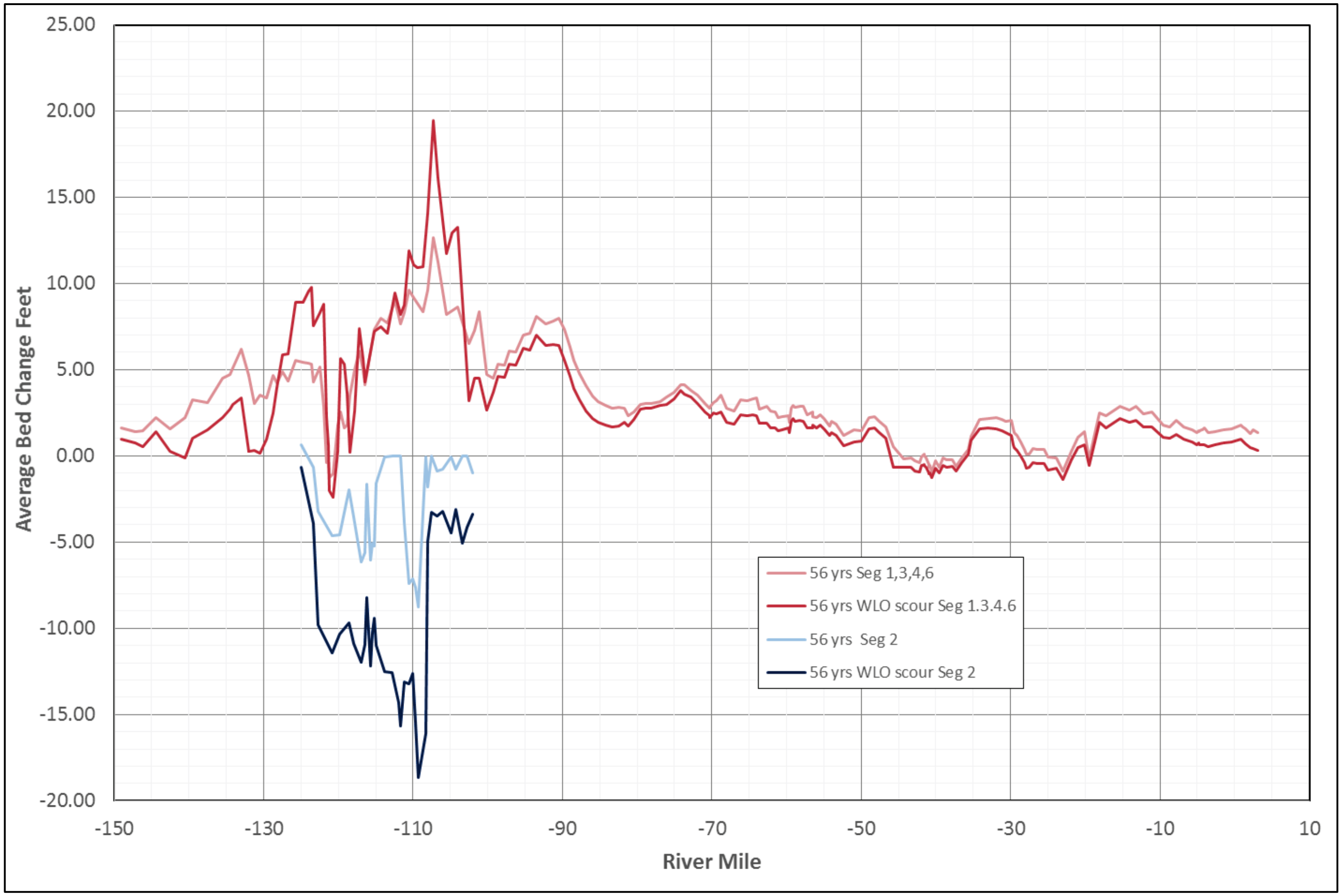
*Calculated between 1997 and 2066 for a bankfull discharge and a flood discharge. Discharge magnitudes are from Simmesport.

Figure 4-28. Changes in discharge percentages at Calumet and Morgan City with no limitation on bed erosion.*



*Note: Calculated between 1997 and 2066 for a bankfull discharge and a flood discharge. Discharge magnitudes are from Simmesport.

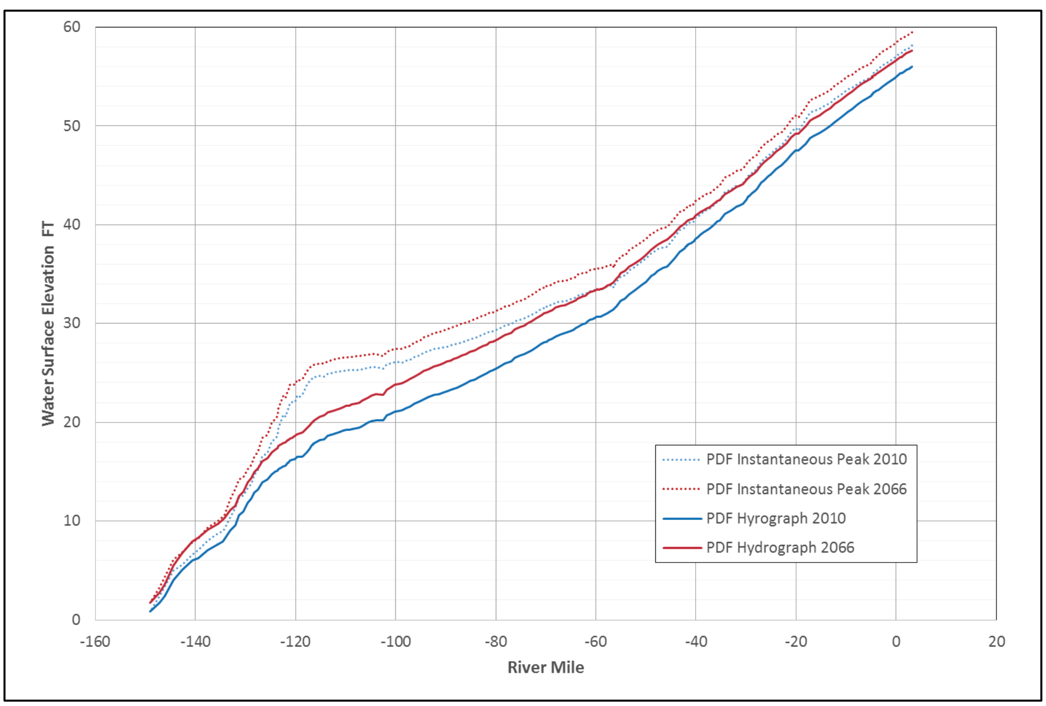
Figure 4-29. Calculated average bed change 2010–2066 showing the effect of limiting erosion potential in the Wax Lake Outlet.



Bed changes during the rise of the PDF

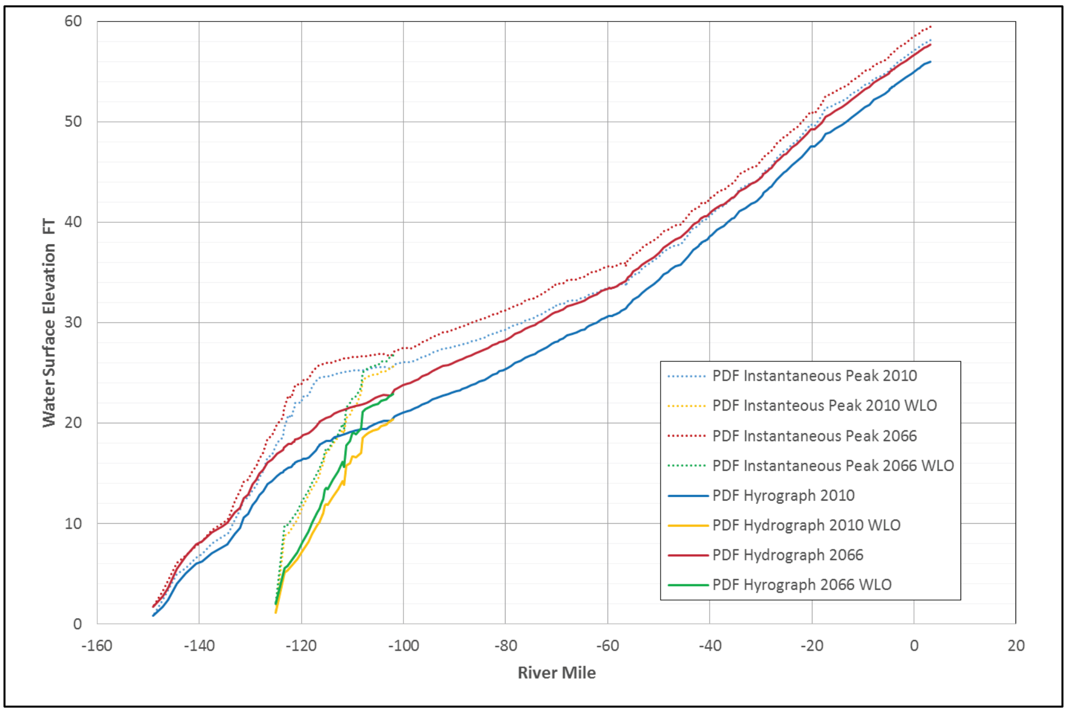
Figure 4-30 shows the effects on peak water surfaces of the bed changes during the rise of the PDF event. The figure includes segments 1, 3, 4, and 6 and does not include the parallel segments 2 and 5, for clarity. The calculated sedimentation effects during the rise of the PDF hydrograph are demonstrated by comparing the calculated PDF instantaneous and PDF hydrograph profiles. This can be done using the 2010 and 2066 profiles (the two red or the two blue lines in Figure 4-30). The change from the dotted line to the solid line shows that during the rise of the PDF hydrograph, sedimentation effects cause a significant decline in the water surface elevations. This is due to channel degradation in the constricted Morgan City reach (vicinity of RM -120). Accounting for sedimentation effects during the rise of the PDF hydrograph results in a water surface difference of approximately 6 ft upstream from RM -120, transitioning to approximately 3 ft at RM -55, and approximately 2 ft between RMs -55 and 0. The average bed change shows scouring of over 20 ft in cross sections immediately downstream of Morgan City. Mossa documented changes of the same magnitude during the 1973 event. From Figure 22 of Mossa (2016), mean bed changes of approximately 6 m, and thalweg changes of approximately 7 m, during the 1973 flood were reported.

Figure 4-30. Effects of the bed changes during the rise of the PDF event on the peak water surfaces.



Calculated water surface elevation differences in Wax Lake Outlet for peak PDF water surface elevations are shown in Figure 4-31. The long-term effects of sedimentation on PDF water surface elevations are similar in the reach downstream from the bifurcation in both the Wax Lake Outlet and the Lower Atchafalaya River. There is only a minor degradation in the Wax Lake Outlet due to the PDF, specifically -2.5 ft at RM -125 and -1 ft at RM -122.7 and -123.3. The large amount of scour near Morgan City causes the flow at Calumet to decrease from 665,000 cfs for the instantaneous peak to 595,000 cfs with the hydrograph, approximately an 11% decrease in the peak flow.

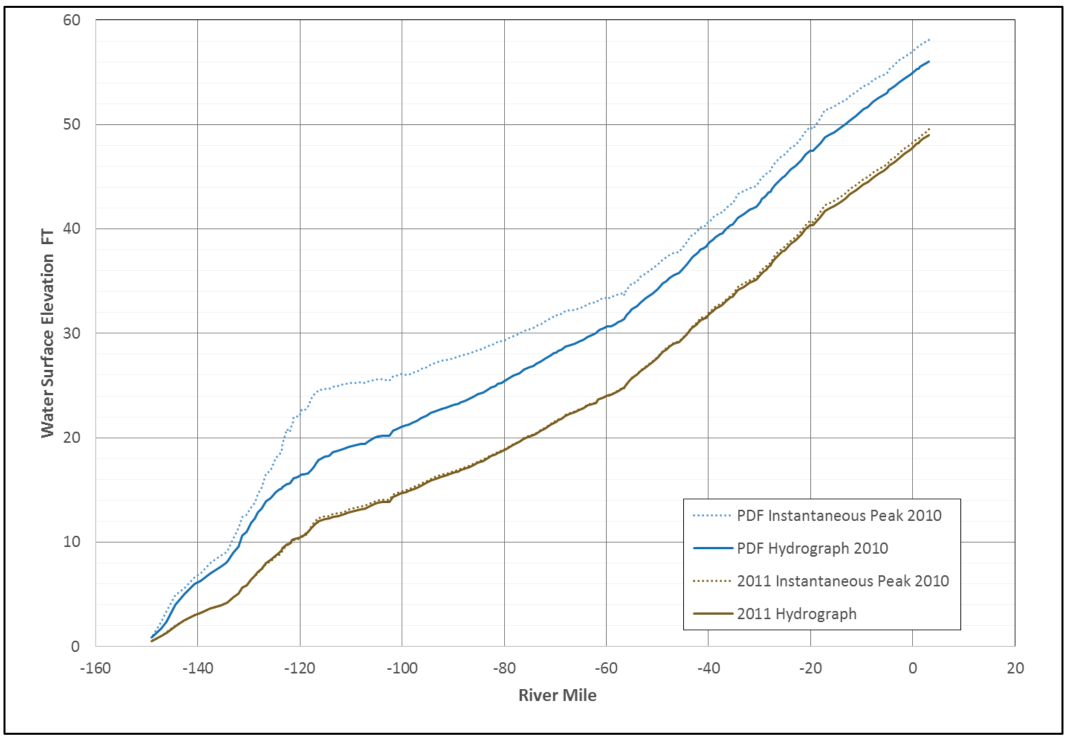
Figure 4-31. PDF water surface elevations of the Atchafalaya River main channel and Wax Lake Outlet.



Bed changes during the 2011 flood had a much smaller impact on the water surfaces, as shown by the difference between the solid brown and dotted brown lines in Figure 4-32. This indicates that a fixed-bed hydraulic model is acceptable for simulating the 2011 event. However, the PDF event (scenario 58A-R Authorized Yazoo) appears to exhibit much different behavior than 2011. The PDF hydrograph imposes an extreme non-equilibrium condition onto the Atchafalaya River system below the Wax Lake Outlet bifurcation. All of the East Atchafalaya Floodway flow rejoins the river at this location. The MCS, accounts for approximately 40% of the combined PDF flow past Calumet and Morgan City, and most of the

MCS flow goes into the Atchafalaya River downstream from the bifurcation. During the 2011 event, the peak flow through MCS was 170,000 cfs, and it was above 160,000 cfs for 11 days (Maynord 2014, Table 10). The flow through MCS during the PDF reaches 600,000 cfs, and it remains at 600,000 cfs for 31 days. Also, the peak flow at Simmesport during 2011 was 692,000 cfs whereas the peak flow during the PDF at Simmesport would be 873,000 cfs. Sand concentrations in the Morganza inflow are negligible. As a result, the bed material concentrations are significantly less than equilibrium in the Atchafalaya River at Morgan City, and degradation potential is extreme during the PDF.

Figure 4-32. Calculated peak water surface profiles for the PDF and the 2011 flood.



Mean bed changes

The average bed changes for the 50-year simulation, from 2010 conditions to 2066 conditions, is shown in Figure 4-29 by the light-blue and light-red lines. The long-term sedimentation effects are demonstrated by comparing the calculated water surface profiles for 2010 and 2066 in Figure 4-31. This comparison can be made using both the "PDF Instantaneous Peak 2010" and "PDF Instantaneous Peak 2066" profiles and the "PDF Hydrograph 2010" and "PDF Hydrograph 2066" profiles (the two dotted lines and the two solid lines in Figure 4-31). The calculations predict that after 56 years, sedimentation effects would result in higher PDF water surface elevations of approximately 2 to 3 ft.

Specific gage comparisons

Specific gage plots based on historical stage and discharge data have been used extensively to document trends in water surface elevations at gage locations. Specific gage analyses provide important perspectives relative to past river behavior at a specific location, but trends cannot be extrapolated to predict future conditions. Specific gage analysis incorporates all the morphological factors influencing stage and cannot be used to identify the effect of an individual variable.

The advantage of a numerical model is that it can be used to predict future trends in water surface elevations, and it can be used to predict past and future trends between gages. The reliability of the model's prediction is predicated upon the model's ability to simulate the significant geomorphic processes that influence long-term change in stage. The HEC-6T numerical model accounts for future aggradation and degradation of the river bed but does not account for possible future changes in bed roughness, floodplain roughness, or change in channel shape due to meandering. There is also uncertainty associated with future changes in the boundary conditions, such as sediment inflow. Confidence in numerical model prediction is enhanced when historical changes in specific gage are correctly reproduced by the model. This success implies that the most significant geomorphic processes have been accounted for in the numerical simulations.

Figure 4-33 shows the calculated and measured specific gage relationships for Simmesport. Daily discharges and stages at Simmesport are available from river gages, where the stages are measured and the discharges are computed using a stage-discharge rating curve. Some discharge measurements are available from USGS, which are also shown in the figure. The calculated and measured specific gage plot for Melville, shown in Figure 4-34, uses the Simmesport discharge and the stage at Melville. Measured stage at Melville is available from river gages. The river gages symbols in Figure 4-34 use discharges from the Simmesport river gages data for the same date. USGS discharge measurements are also shown in the figure.

Figure 4-33. Calculated and measured specific gage at Simmesport (RM -4.34).

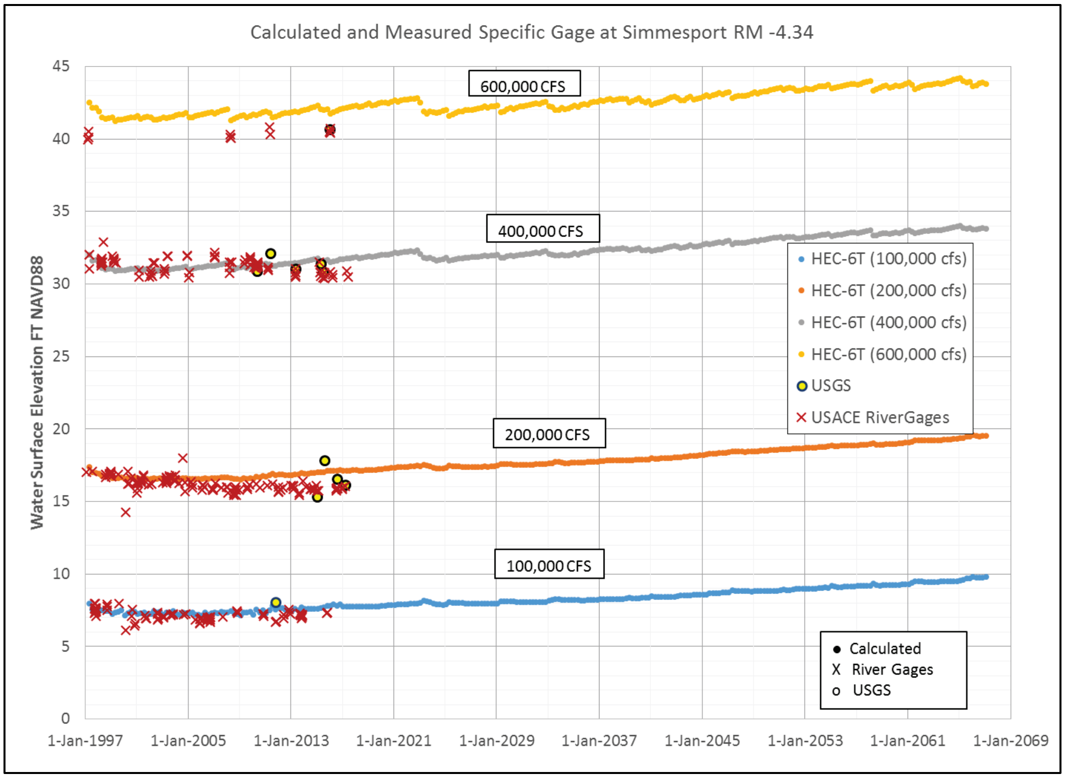


Figure 4-34. Calculated and measured specific gage at Melville (RM -29.6).

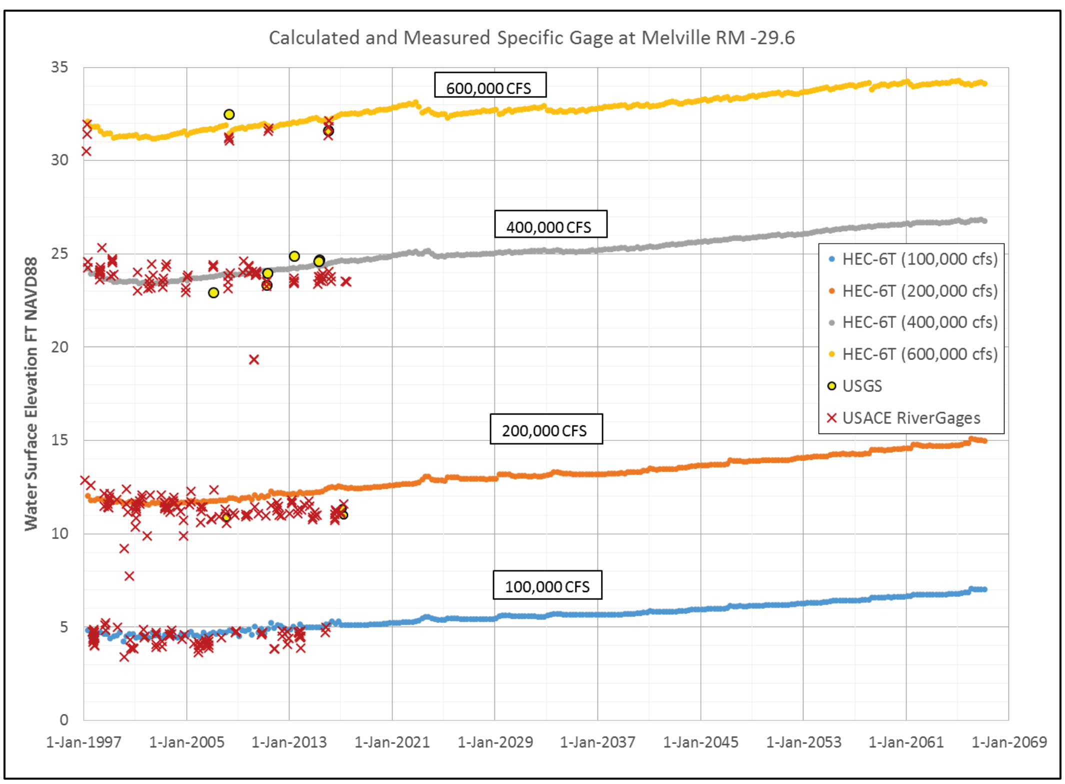


Figure 4-35 and Figure 4-36 show the measured and calculated specific gage relationships for Calumet and Morgan City, respectively. The measured USGS data come from measured stage and computed discharge for the Calumet and Morgan City figures. The large scatter in the measured and calculated data at low flows is caused by a large tidal influence at low flows for Calumet and Morgan City. The clear influence of the Gulf of Mexico water level indicates that these two gages will be influenced by SLR, especially at low discharges. Indeed, both gages show a specific gage trend going upward for low discharges of 50,000 cfs and 100,000 cfs. In addition to SLR, the Morgan City stage trend for all discharge relationships is upward due to deposition in the channel. Calumet specific gage trends are slightly upward for the lower discharges of 100,000 cfs and 200,000 cfs, but the stages trend downward for the 300,000 cfs discharge due to limited scour in the Wax Lake Outlet.

Figure 4-35. Calculated and measured specific gage at Calumet (RM -111.2).

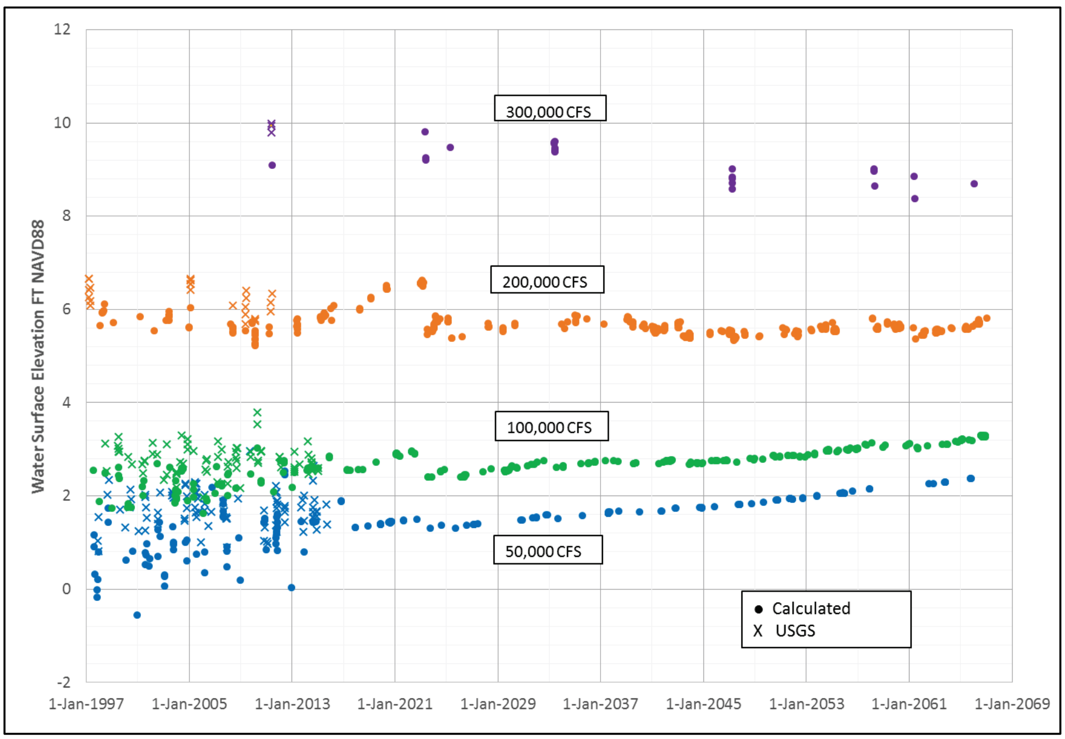
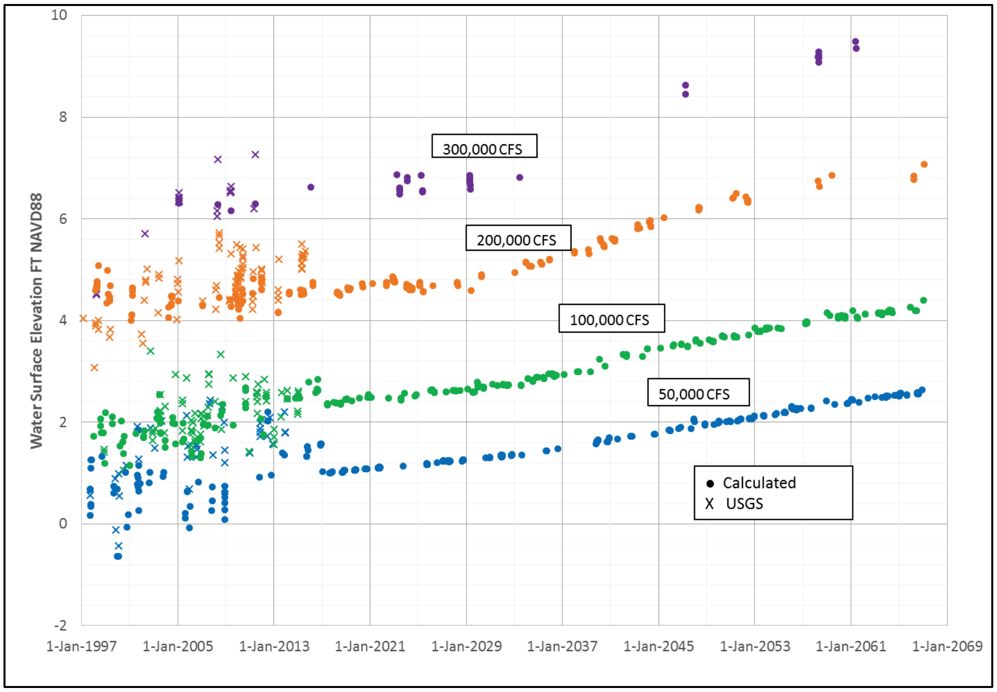


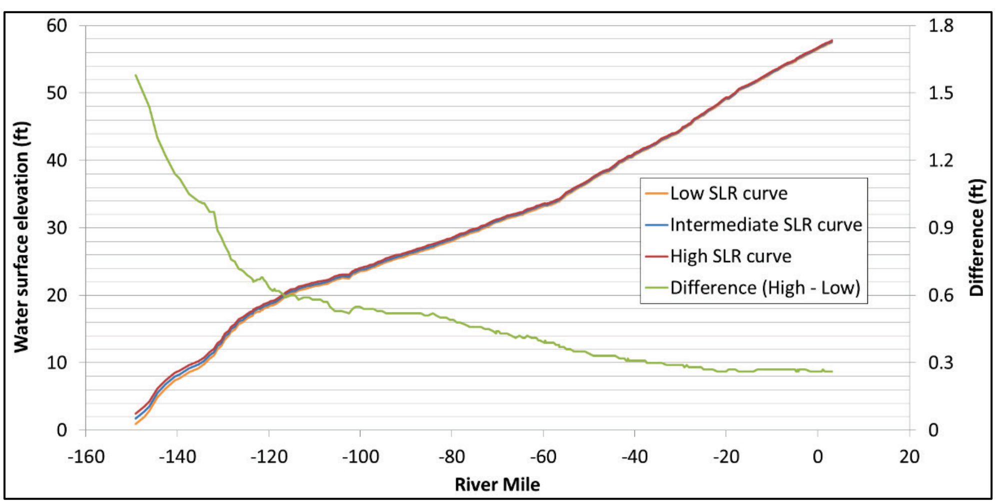
Figure 4-36. Calculated and measured specific gage at Morgan City (RM -121.2).



Effect of SLR

The three different SLR curves recommended in ER 1100-2-8162 (USACE 2013) were simulated within the HEC-6T model to show the sensitivity to SLR. Figure 4-37 shows the water surface elevations of the PDF event hydrograph peak, beginning with 2066 conditions, for each of the SLR curve scenarios. As expected, SLR has a greater influence at the downstream end and decreases in the upstream direction.

Figure 4-37. Water surface elevations for the PDF in 2066 with the three SLR curves.*



*Note: Only segments 1, 3, 4, and 6 are shown for clarity.

4.3.5 Summary

4.3.5.1 Conclusions

The HEC-6T sediment model of the Atchafalaya River simulated historical conditions from 1997 through 2016 and hypothetical conditions from 2017 through 2066. Changes were made to the 2010 HEC-6T model to calibrate and validate the model to the time period 1997 to 2016. The following major conclusions were made from the results of the HEC-6T model.

- The Wax Lake Outlet is capturing an increasing amount of flow from the Lower Atchafalaya River.
- The PDF event scours the channel bed elevations by a large amount during the rising limb of the hydrograph, especially near Morgan City. This scour decreases the PDF water surfaces by up to 5 ft.
- Although the Upper Atchafalaya River has exhibited large amounts of scour in recent decades through MVN surveys, the sediment model projects a dynamic equilibrium to slightly increasing trend in water surfaces going forward. Future water surfaces in the upper reach could be lower than model projections due to channel widening, a process that should be included in future sediment modeling efforts of the Atchafalaya River.
- Over the next 50 years, the sediment model results project trends of aggradation, or higher water surfaces into the future, for the Lower Atchafalaya River. Water surfaces during a PDF event occurring in 2067 will be up to 3 ft higher than a PDF event occurring in 2011.

4.3.5.2 Connection to hydraulic modeling

The objective of the sediment study was to provide changes in channel bed elevations, due to sedimentation, for the fixed-bed HEC-RAS hydraulic model. These bed changes were applied to the HEC-RAS model and used to simulate future water surfaces. The HEC-RAS cross sections were not in the same locations as the HEC-6T cross sections, so the bed changes were averaged by reaches. Reaches varied in lengths and were determined by grouping together adjacent cross sections that exhibited similar bed changes. The HEC-6T software calculates average bed change between the channel stations; these results were output from the model for September 30, 2010; September 30, 2066; and at the peak of the PDF in 2067.
Table 4-11 shows the average bed changes, by reach, that were reported for implementation into the HEC-RAS model. The long-term bed change, from 2010 to 2066, and the bed change during the PDF event (scenario 58A-R Authorized Yazoo) are reported.

Table 4-11. Average bed change results by reach.

Upstream HEC-6T Cross Section	Downstream HEC-6T Cross Section	HEC-RAS River	Upstream HEC-RAS Cross Section	Downstream HEC-RAS Cross Section	Bed Change (ft) from 10/1/2010 to 9/30/2066	Bed Change (ft) within the PDF Event
-146.2	-149.1	Atch	18957.01	2719.914	1	-4
-144.4	-144.4	Atch	25881.71	25881.71	2.7*	0
-140.6	-142.6	Atch	41382.14	33563.18	2.8	0
-137.5	-139.5	Atch	57287.42	47666.67	2.8	-4.5
-135.5	-135.5	Atch	63075.32	63075.32	8.2*	-4.5
-129.6	-134.5	Atch	94485.92	68558.72	4.8	-4.5
-124.7	-128.8	Atch	120513.6	98835.9	5	-12.5
-123.6	-124	Atch	125947	123105.8	5	-25.5
-122	-123.4	Atch	134315.2	128249.9	5.7	-25.5
-120.1	-121.6	Atch	143623.7	135620.3	0	-9
-117.2	-119.7	Atch	156948.7	146057.3	6	-16.5
-116.4	-116.7	Atch	162338.7	159741.5	6	-0.5
-104	-115.2	Atch	227227.7	165558.3	10.5	-0.5
-125	-125	WaxLake	32561.78	26764.01	0.5	-6.5
-123.3	-123.3	WaxLake	38069.36	38069.36	-5.3	-2.5
-120.8	-122.7	WaxLake	48692.56	41565	-5.3	-1
-115.2	-119.8	WaxLake	79657	56100.93	-5.3	0
-111.7	-114.9	WaxLake	97192.81	81922.72	0	0
-111.2	-111.2	WaxLake	99864.52	99864.52	-4	0
-109.3	-110.5	WaxLake	112305.5	104402.8	-10	0
-102	-108.3	WaxLake	155746	113545.4	-0.7	0
-101.1	-102.5	Atch	243947.4	230375.6	8.2	0
-88.5	-100.1	Atch	306553.2	246498.7	6.8	0
-70.3	-87.7	Atch	398154.3	310726.6	4	0
-70.3	-75.1	AtchSplit	28319.64	2874.749	4	0
-56.5	-70.31	Atch	470193.8	400592.8	4	4
-55.4	-78.8	AtchSplit	125437.1	33140.38	2	0
10.12	-56.5	Atch	751747.4	475518.4	3.5	-1.5

Note: * indicates the bed change should only apply to the non-dredged part of the channel.

The error associated with accounting for sedimentation effects by using average reach adjustments to the bed elevations in the numerical model was estimated. The base condition was defined using calculated water surface elevations from the HEC-6T numerical model from the April 30, 1997, to September 30, 2066, simulation followed by the PDF hydrograph. An instantaneous PDF peak discharge was inserted into the base simulation on September 30, 2010, September 30, 2066, and after the rise of the PDF hydrograph to maximum discharge. September 2010 conditions represent the best estimate for the date of the geometry used in the HEC-RAS model.

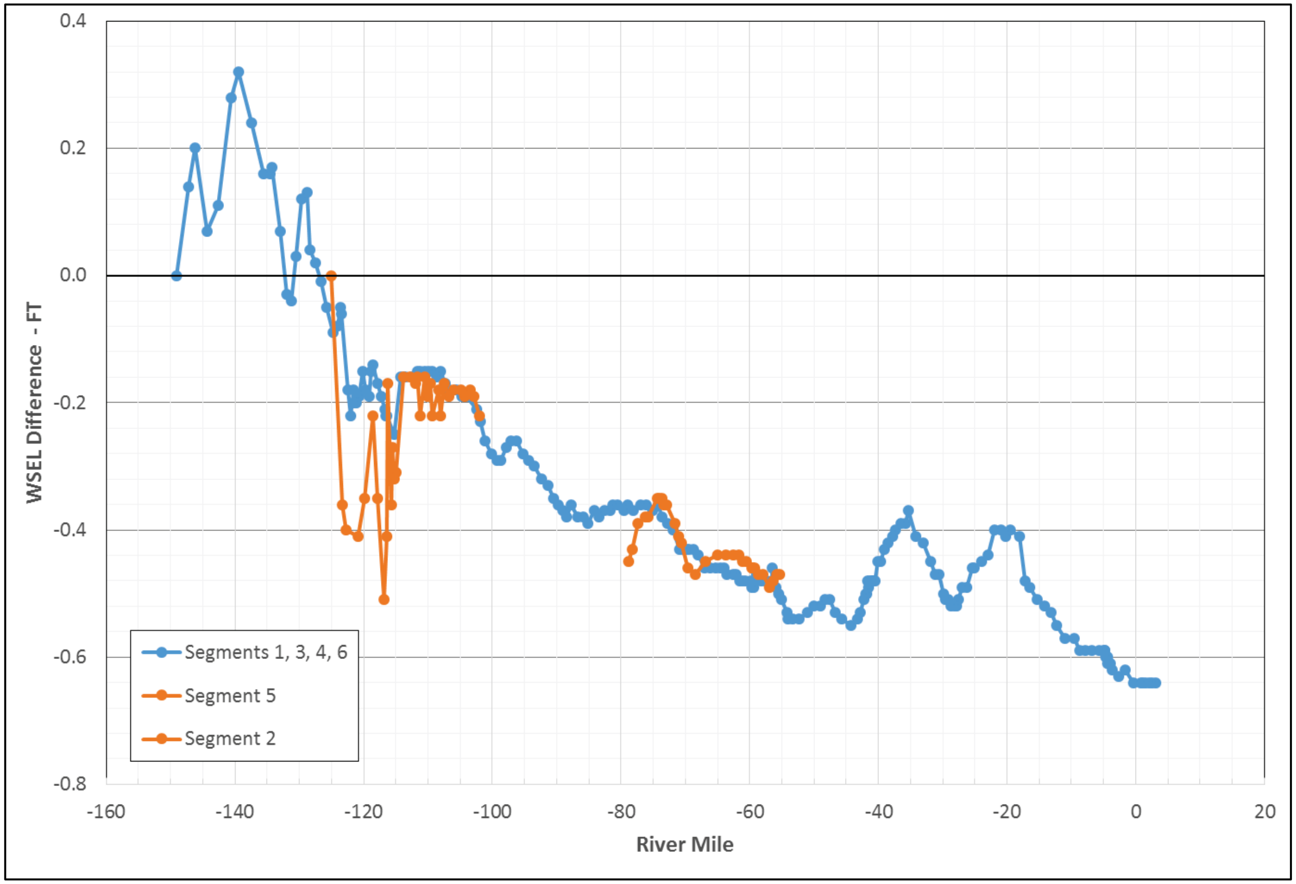
The reach-averaged bed elevation changes for the period between September 30, 2010, and September 30, 2066, (shown in Table 4-11) were determined using both the HEC-6T calculated thalweg and average bed changes at each cross section for the 56-year period. An adjusted HEC-6T input file was created by adding the reach-averaged bed elevation changes to the calculated September 30, 2010, channel bed elevations (the calculated bed elevations for September 2010 came from an HEC-6T output file). A PDF instantaneous peak discharge was then used with the adjusted geometry to calculate PDF peak water surface elevations. The water surface profile calculated with the adjusted geometry was compared to the water surface profile calculated using the actual geometry to estimate the error in this reach-averaged approach.

A comparison of PDF peak water surface elevation differences for the 56-year period is shown in Figure 4-38. Errors range between +0.3 ft and -0.6 ft. Downstream from Morgan City, the PDF water surface elevations calculated using the adjusted bed elevations from Table 4-11 in a fixed bed model would generally be too high. Upstream from Morgan City, the PDF water surface elevations calculated using the adjusted bed elevations from Table 4 in a fixed bed model would generally be too low. The most likely reason is related to the disproportionate deposition and/or scour on crossings and bends that occurs with the actual flood hydrograph.

A comparison of PDF peak water surface elevation differences when using the reach averaged adjusted cross sections to account for the rise of the PDF hydrograph is shown in Figure 4-39. Error associated with the calculated water surface elevations using the reach-averaged adjusted bed elevations to estimate the effects of sedimentation during the hydrograph rise was +/- 0.9 ft.

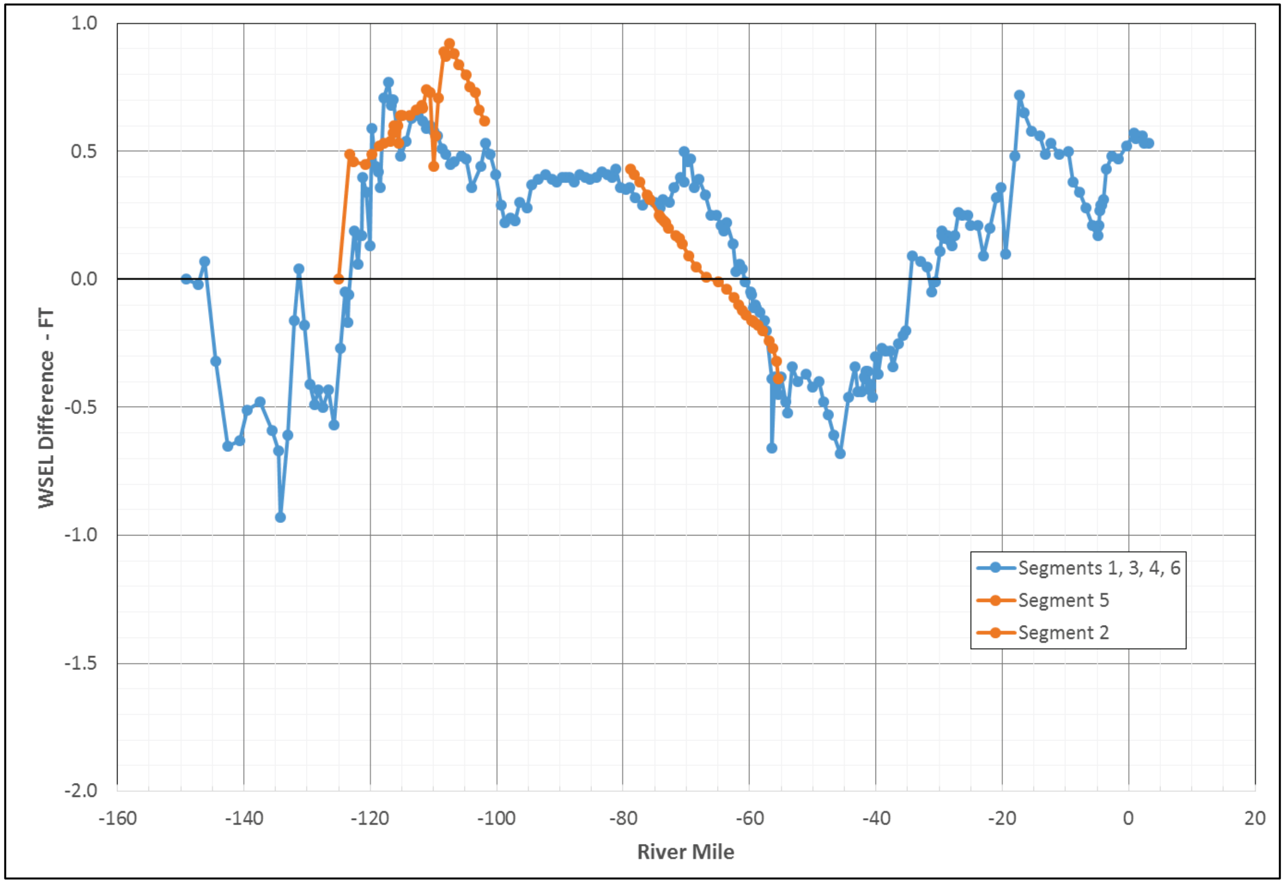
This approach using reach-averaged bed elevation adjustments seemed appropriate at the beginning of the study because there was an existing HEC-6T numerical model that was thought to require only minor adjustments for this study. A more rigorous approach would have been to use the same geometry used in the HEC-RAS model for the HEC-6T model even though this would have required a significant calibration and validation process. Calculated bed elevation changes could then be applied directly to each cross section and transferred to the HEC-RAS model without trying to determine reach-averaged approximations. That is the preferred approach for future work.

Figure 4-38. Difference in calculated PDF peak water surface elevation after 56 years.*



*Note: The difference is determined by subtracting the water surface elevation calculated using the actual geometry and the 2010-2066 hydrograph from the water surface elevation calculated using the “adjusted” channel bed geometry, where “adjusted” is based on Table 4-11.

Figure 4-39. Difference in calculated PDF peak water surface elevation due to the rise of the PDF hydrograph.*



*Note: The difference is determined by subtracting the water surface elevations calculated using the actual geometry and the PDF hydrograph from the water surface elevation calculated using the "adjusted" channel bed geometry and an instantaneous PDF peak discharge, where "adjusted" is based on Table 4-11.

5 Hysteresis

5.1 Objective

This section calculates an expected allowance that should be added to the HEC-RAS simulated water surface elevations to account for the potential hysteresis, or loop effect, that could occur during the PDF design event.

5.2 Introduction

The relationship between stage and water discharge for channels and rivers is one of the most important hydraulic characteristics used by engineers, planners, and designers studying flood events. It is not difficult to obtain a continuous record of the variation of river water level, or stage, with time. From fundamental hydraulics, it is known that if a river were to experience completely steady state, uniform flow conditions, there would be a single water surface elevation corresponding to a particular flow rate. Using discharge measurements at various stages, a stage-discharge rating curve can be developed such that the relationship between the water surface elevation and the flow rate is defined. The rating curve may also be viewed as a snapshot of the channel reach; it reflects the required stage to pass a particular discharge through the channel which implicitly reflects the conveyance of the reach.

A hysteresis phenomenon occurs in alluvial rivers, where the stage does not have a unique value for a particular discharge, because of unsteady, non-uniform conditions (Henderson 1966). Typically, the rising limb of the hydrograph, or as the flow at a location is increasing, exhibits lower stage values than the falling limb of the hydrograph. In events where there are multiple peaks, events after the first peak can result in higher stages for the same flows because the river conditions are starting from a higher level. Figure 5-1 shows how the hysteresis phenomenon deviates from a standard uniform flow relationship. This phenomenon is common to alluvial channels, especially flatter-sloped, large rivers.

Figure 5-1. Conceptual sketch of the hysteresis effect.

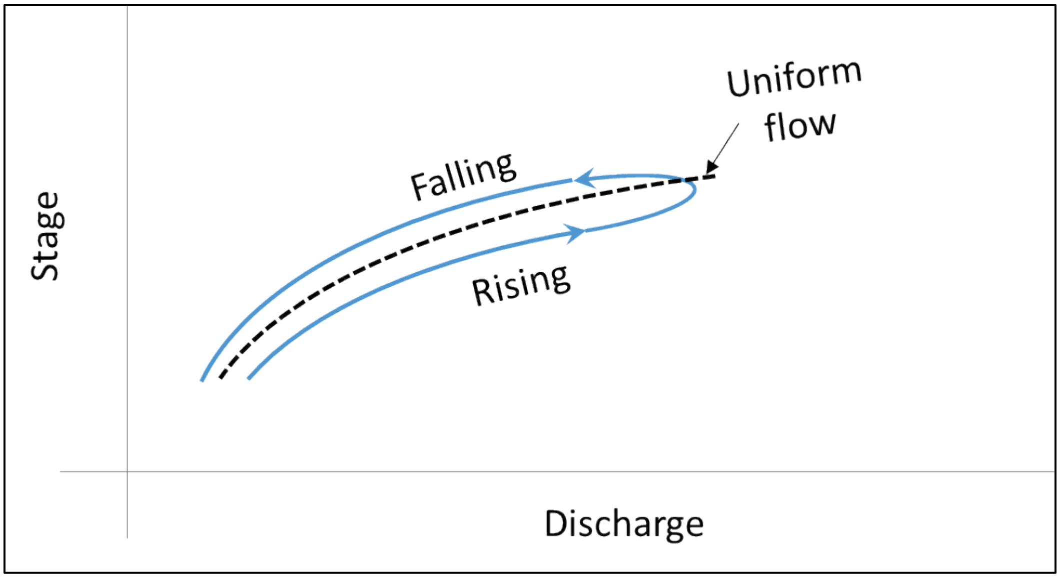
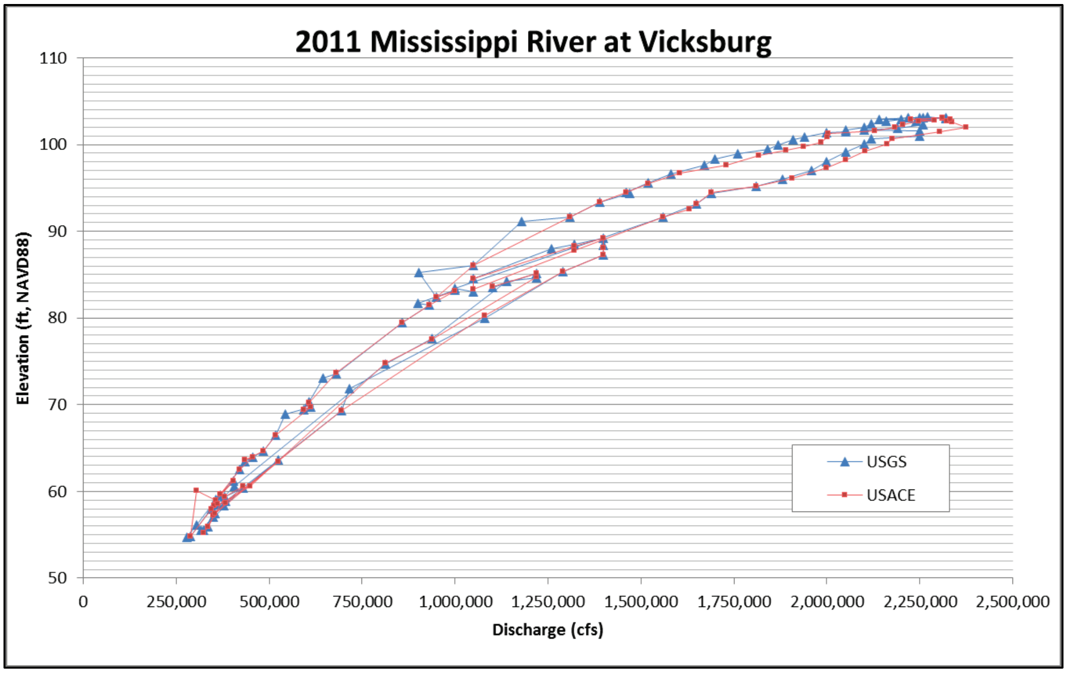


Figure 5-2 is a plot of observed data for the Mississippi River at Vicksburg, MS, during 2011. The plotted data represent observed measurements from the U.S. Geological Survey (site #07289000, http://waterdata.usgs.gov/nwis/uv?site_no=07289000) and MVK (http://rivergages.com). Lines in the figure simply show the chronological connection between the data points. The hysteresis, or loop effect, is clearly seen in Figure 5-2. This type of loop may be small or large and display multiple loops (Combs 1994). For the 2011 data, observed stages on the falling limb of the hydrograph were approximately 4 to 5 ft higher than stages on the rising limb. This behavior falls within the typical range of observations at high flows (> 1.25 million cfs) at this location for other recent years.

Figure 5-2. Stage and discharge measurements at Vicksburg during 2011.



There are four primary factors that influence the hysteresis curve in alluvial rivers. These four factors are (1) dynamic effects, (2) resistance changes, (3) viscous effects, and (4) aggradation/degradation of the river. The dynamic effects are a result of the unsteady flow characteristics of the system. The resistance changes are due to the variation of the roughness of the channel, overbanks, lakes in the flood plain, and bed forms. Viscous effects can change based on the temperature of the water and the magnitude of sediment being transported. Aggradation and degradation can vary during flood events more than during normal river flows and can only be quantified if the bed is surveyed multiple times during the course of a single flood event. These individual contributing factors are very difficult to isolate since there is dynamic feedback among them. For example, water temperature can change viscosity, which changes sediment transport rates, channel resistance, and aggradation/degradation; these change the depth, which in turn changes the sediment transport rates and so on.

The unsteady dynamics during the propagation of a flood wave induce the hysteresis phenomenon, which can be explained hydraulically. The effect of the wave front upstream of a specific location is to increase the velocity into the given cross section of the river. As the wave front passes into the downstream reach, the backwater effect from the higher stage downstream reduces the velocity at the same cross section. This unsteady dynamic causes the stage to be lower on the front side of the flood wave and higher after the flood wave has passed (Peterson-Overleir 2006). Combs (1994) showed that the maximum discharge can be reached before the maximum flow depth (also in Julien [2002]). In fact, the peak flow typically precedes the peak stage because flow accelerates on the rising limb and decelerates on the falling limb (USACE 1993). Other unsteady hydraulic factors that also cause hysteresis would include any water management control, or structure, operational changes, and the impact of floodplain storage areas, which are being filled on the rising side and emptying on the falling side of the flood wave. The 1D unsteady flow models (e.g., HEC-RAS) can predict purely dynamic effects in a fixed-bed channel. Quantifying dynamic feedback in an alluvial (moveable bed) channel is beyond the current state-of-the-art.

Changes in the flow resistance during a flood event can arise from seasonal changes and bed form changes. Floodplain vegetation resistance would generally be smoother in the winter months versus rougher in summer after vegetation sprouts and bushes and trees put on leaves. Bed forms within sand-bed river channels change significantly throughout the hydrograph, and some work has been done (Gessler et al. 1998) with limited data sets to understand and predict multiple depths caused by bed variations. Even with survey data during a flood event, it is difficult to fully explain bed-form dynamics. Multi-beam surveys can quantify the growth and decay of bed forms, but a systematic investigation is needed to correlate bed-form observations to changes in channel resistance.

5.3 Background of the hydraulics used in this project

The HEC-RAS model developed for the current MR&T System flowline assessment simulates the water surface elevations that would be predicted to occur during the PDF design event(s). The model was calibrated to the 2011 Mississippi River Flood and validated with 2002 and 2008 conditions. (For more discussion of the model development and simulation, see the Hydraulics Report of the current study [USACE 2018b]).

The process of calibrating the HEC-RAS model to the 2011 conditions using varying roughness values can indirectly capture many of the factors that influence the hysteresis. The HEC-RAS model used for the PDF simulations is able to perform fully unsteady hydraulic computations. In these ways, the HEC-RAS model is expected to simulate a significant hysteresis, or loop effect, but it is not expected to capture the entirety of the potential hysteresis. To account for some of this, when running the PDF simulations, the HEC-RAS model was run with seasonal roughness values set to their maximum values (usually the *warm,* or summer month, roughness values are the maximum) for the entirety of the simulation. Setting the seasonal roughness values to their maximums accounts for the possibility that a PDF event could occur during various times of the year. However, other influencing factors described in the previous section, such as bed form changes, could potentially cause the water surface elevation of the PDF event to be higher than the numerical model results, even with seasonal roughness values set to their maximum.

One of the approaches used in the Refined 1973 Flowline Study for the Mississippi River (USACE 1978) was to develop two separate stage-discharge curves to quantify the hysteresis, or loop effect, during the 1973 Mississippi River Flood. One stage-discharge curve was developed using resistance values that resulted in the computed surface matching the observations for the rising side of the 1973 flood wave. Separate resistance values were used to develop a second curve that matched the falling side of the 1973 flood wave. This approach allowed for the extrapolation of a predicted hysteresis for the very high PDF discharge. However, the size and the form of the hysteresis is different for each individual flood event, which causes concern about the extrapolation from any single event's conditions to predicted PDF conditions (Peterson-Overleir 2006; Fread 1975).

5.4 Analysis

The focus of this report is on summarizing possible hysteresis for a very high flow event. Since the PDF event is much larger than any observed events for most parts of the river, the direct quantification of an observed hysteresis during a PDF event is not possible. This analysis compared the HEC-RAS simulated hysteresis with the observed hysteresis for recent high flow events to quantify the amount of potential hysteresis, or loop effect, that HEC-RAS was unable to capture. In particular, this analysis focused on the 2008 and 2011 conditions. The 2002 year was also simulated with the HEC-RAS model, but it was not included here since it was not as large of an event, and most locations only slightly exceeded flood stage.

Figure 5-3 shows the observed 2008 discharge measurements and their corresponding stages compared with the HEC-RAS simulated results for the Mississippi River at Vicksburg, MS. Figure 5-4 shows the same data set for 2011 at Vicksburg. Figure A-1 through Figure A-16 in Appendix A: Additional Figures for the Hysteresis Effect Analysis, show the same 2008 and 2011 plots for other locations, namely Hickman, Memphis, Helena, Arkansas City, Greenville, Natchez, Red River Landing, and Baton Rouge. For many locations, the HEC-RAS model captured a larger portion of the loop effect in 2008 than it did in 2011. Since this analysis is focused on the hysteresis for high flows, only the data above the flood stage were included in the calculation of the hysteresis within each data set.

A Visual Basic script within Microsoft Excel was created to systematically calculate the hysteresis within the observed and simulated stage and discharge data. The script searched through the data for the year until it found a data point with an elevation above the flood stage. Then, starting with the data point immediately previous to the one it just found, the stage was calculated for every 50,000 cfs increment of flow using linear interpolation between data points. The script handled the rising and falling directions of any data above the flood stage. In this way, a range of stages was determined for each 50,000 cfs incremental flow, and the difference between the maximum and minimum stages at each incremental flow was calculated. The maximum of these differences was considered the hysteresis for that data set, and these are shown in Table 5-1 (columns 4, 5, 7, and 8). For locations where USACE and USGS observed data were available, the maximum hysteresis of those two observed data sets was included in the table.

The ratio of the hysteresis in the observed data divided by the hysteresis in the HEC-RAS data is also shown in Table 5-1 (columns 6 and 9). The final column of the table uses the higher ratio of the two events for each location to show a recommended ratio of the hysteresis effect between observed data and HEC-RAS model results.

Figure 5-3. Plot of 2008 stage and discharge data at Vicksburg, MS.

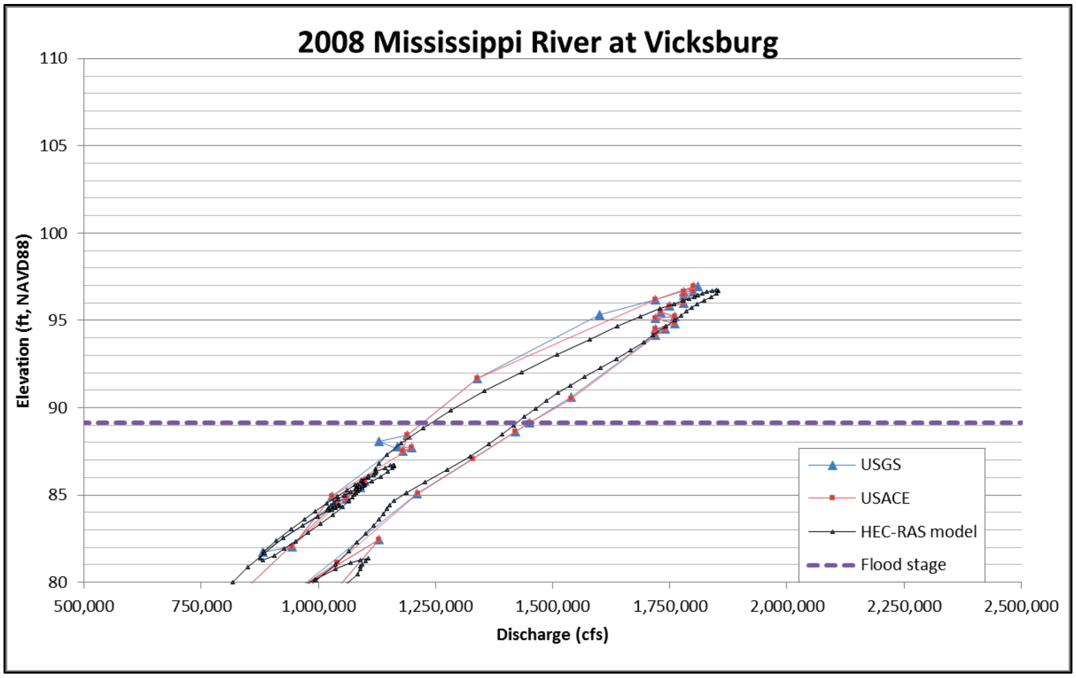


Figure 5-4. Plot of 2011 stage and discharge data at Vicksburg, MS.

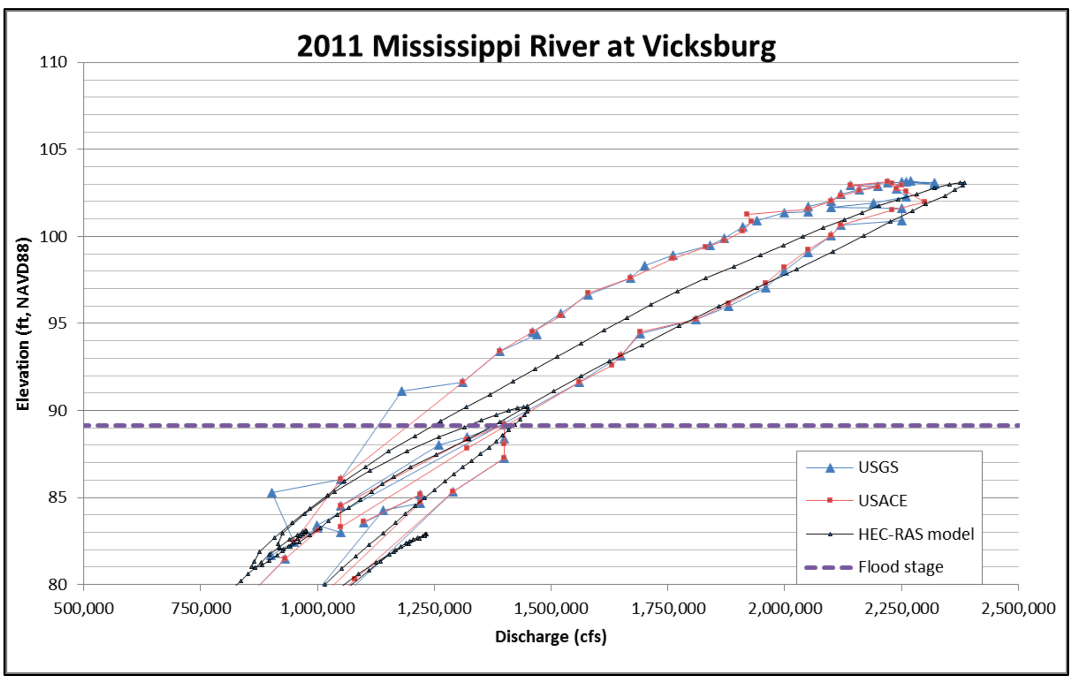


Table 5-1. Amount of hysteresis above the flood stage at each location ("Obs." = Observed).

Location	RM (1962 labels)	HEC-RAS Station	2011 Hysteresis			2008 Hysteresis (ft)			Hysteresis Ratio (Obs./ HEC-RAS)
			Obs. (ft)	HEC-RAS (ft)	Ratio (Obs./ HEC-RAS)	Obs. (ft)	HEC-RAS (ft)	Ratio (Obs./ HEC-RAS)	
Hickman	922	942.45	3.0[a]	1.6	1.9	1.6	1.4	1.1	**1.9**
Memphis	734	749.01	1.7[a]	1.7	1.0	1.2	1.0	1.2	**1.2**
Helena	663	676.42	2.9[a]	1.6	1.8	1.4	1.2	1.2	**1.8**
Arkansas City	554	562.18	1.7[b]	1.7	1.0	4.6	2.3	2.0	**2.0**
Greenville	531	539.13	2.1[c]	1.9	1.1	[c]	2.5	--	**1.1**
Vicksburg	435	442.16	4.8	2.0	2.4	4.0	2.6	1.5	**2.4**
Natchez	363	368.44	4.3	1.7	2.5	3.8	3.1	1.2	**2.5**
Red River Landing	302	306.43	3.1	2.9	1.1	6.9[b]	4.9	1.4	**1.4**
Baton Rouge	228	232.49	1.3	0.5	2.6	2.1	2.6	0.8	**2.6**

[a] An insufficient number of measurements were taken at Hickman, Memphis, and Helena to confidently capture the rising limb of the 2011 hydrograph. Values have been estimated with the best available data.

[b] There is a large amount of fluctuation in the discharge measurements for Arkansas City in 2011 and Red River Landing in 2008. The Arkansas City observations actually show the rising limb stages higher than the falling limb stages near the peak flow in 2011.

[c] There were limited discharge measurements at Greenville for 2011 and no available discharge measurements for 2008.

Figure 5-5 shows the simulated HEC-RAS model results for one of the PDF design events used in the Hydraulics Report labeled 58A-R Authorized Yazoo. See the Hydraulics Report for more information about this simulation (USACE 2018b). Table 5-2 shows the hysteresis ratio from Table 5-1, the simulated loop in HEC-RAS 58A-R Authorized Yazoo results, and the additional allowance recommended for hysteresis. To calculate the fourth column, the third column was multiplied by the second column, and then the value in the third column was subtracted. For the example of Helena, the simulated loop is 2.3 ft, and the ratio of 1.8 indicates that the observed loop could be 4.1 ft (2.3 × 1.8). The HEC-RAS model already captures 2.3 ft of the 4.1 ft, so an additional 1.8 ft would be the recommended allowance. In three locations, Memphis, Greenville, and Red River Landing, the calculation results in an allowance that is less than 1.0 ft. Due to numerous factors that can impact the hysteresis, a minimum allowance of 1.0 ft at all locations is recommended, so the arrows in the right column at those three locations reflect that adjustment. The recommended hysteresis allowance for two locations deviated from the automated script, as noted in the footnotes of Table 5-2. Figure 5-5 shows the simulated hysteresis of the PDF event, the flood stage, the peak HEC-RAS stage of the flood, and the calculated allowance for the hysteresis phenomenon above the HEC-RAS peak stage at Vicksburg. The other locations are shown in Figure A-17 through Figure A-24 of Appendix A.

Figure 5-5. Hysteresis of the simulated HEC-RAS results of the PDF event "58A-R Authorized Yazoo" simulation.

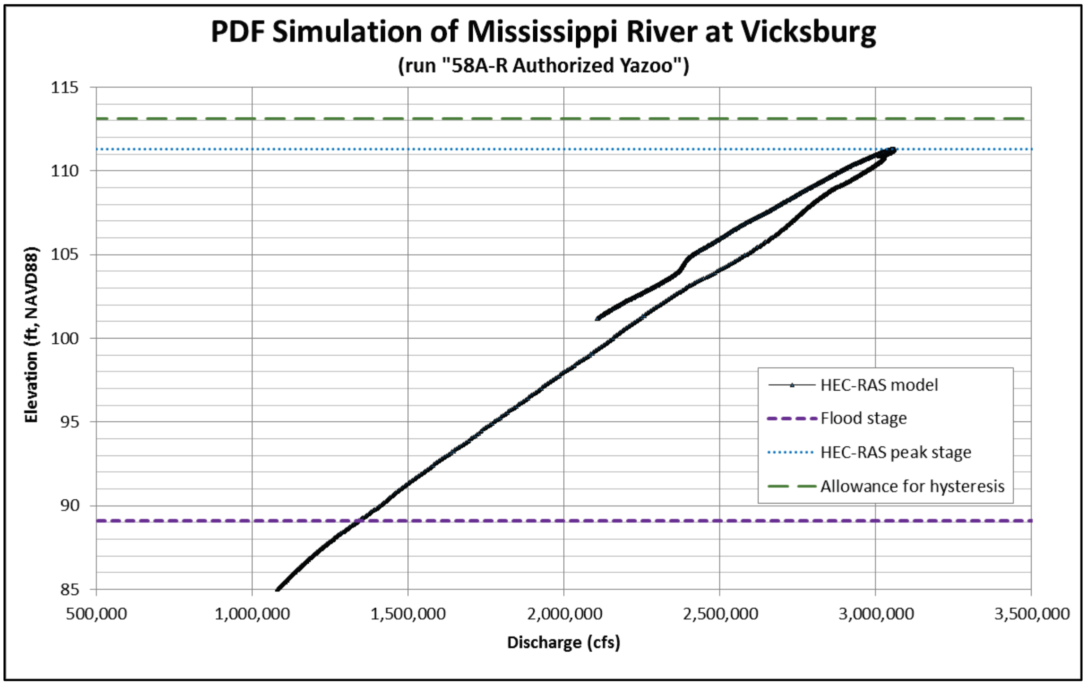


Table 5-2. Additional allowance for hysteresis based on the HEC-RAS PDF simulation.

Location	Hysteresis Ratio (Obs./ HEC-RAS)	HEC-RAS PDF Simulated Loop (ft)	Additional Allowance for Hysteresis (ft)
Hickman	1.9	2.9[a]	2.6 → **1.4**
Memphis	1.2	2.2	0.4 → **1.0**
Helena	1.8	2.3	**1.8**
Arkansas City	2.0	1.7	**1.7**
Greenville	1.1	1.6	0.2 → **1.0**
Vicksburg	2.4	1.3[b]	**1.8**
Natchez	2.5	1.3	**2.0**
Red River Landing	1.4	1.8	0.7 → **1.0**
Baton Rouge	2.6	0.7	**1.1**

[a] The Hickman location is significantly impacted by the initiation of flow into the Bird's Point New Madrid Floodway, as seen by the unique shape in Figure A-17. Instead, the recommended value of 1.4 ft is based on an offset approach using 2011 data (see Table 5-1). In 2011, the observed loop was 1.4 ft greater than the HEC-RAS simulated loop.

[b] Particular attention was paid to the Vicksburg location since it would have resulted in a very large loop allowance using the automated script. The script looked at all elevations above flood stage, but the 1.3 ft came from closer evaluation of data near the peak flow (i.e., >2,700,000 cfs).

5.5 Summary

This report compared observed discharge measurements, taken at specific stages, with HEC-RAS simulated results for recent high flow events. The analysis revealed that the HEC-RAS model captured much of the hysteresis effect but not its entirety. For many locations, the HEC-RAS model captured a larger portion of the hysteresis during the 2008 flood than the 2011 flood. The maximum hysteresis above flood stage for multiple locations was quantified and is shown in Table 5-1. The recommended allowances for additional hysteresis, or loop effect, to be added onto the HEC-RAS simulated PDF results are shown in the right-most column of Table 5-2. It is recommended to extend this analysis in the next phase of the study by simulating more high flow events, such as 2016 and/or 1997 with the HEC-RAS model, and comparing it to observations.

6 Summary

6.1 Levee comparisons

The combination of the results from the individual efforts within this project is compared against the existing Mississippi River & Tributaries (MR&T) levee heights. The computed Hydrologic Engineering Center-River Analysis System (HEC-RAS) water surfaces, the interpolated hysteresis effects, and the sedimentation effects are added together for the combined results. Examples of the combined results along specific levee lengths are shown in Figure 6-1 through Figure 6-4. The hysteresis and sedimentation effects are unchanged for other primary project design flood (PDF) scenarios considered, but the HEC-RAS results shown in the figures represent the 2016 58A-R Authorized Yazoo simulation with future conditions.

Table 6-1 summarizes the combined results compared with levee heights along the Mississippi River. The table compares the existing levees with the HEC-RAS computed water surface for the 58A-R Authorized Yazoo future conditions simulation plus the hysteresis and 50-year sedimentation effects. Values represent the miles of levees and are sorted into columns according to the U.S. Army Corps of Engineers Memphis (MVM), Vicksburg (MVK), and New Orleans (MVN) Districts boundaries.

Figure 6-1. Combined results along right-descending levee near Memphis.

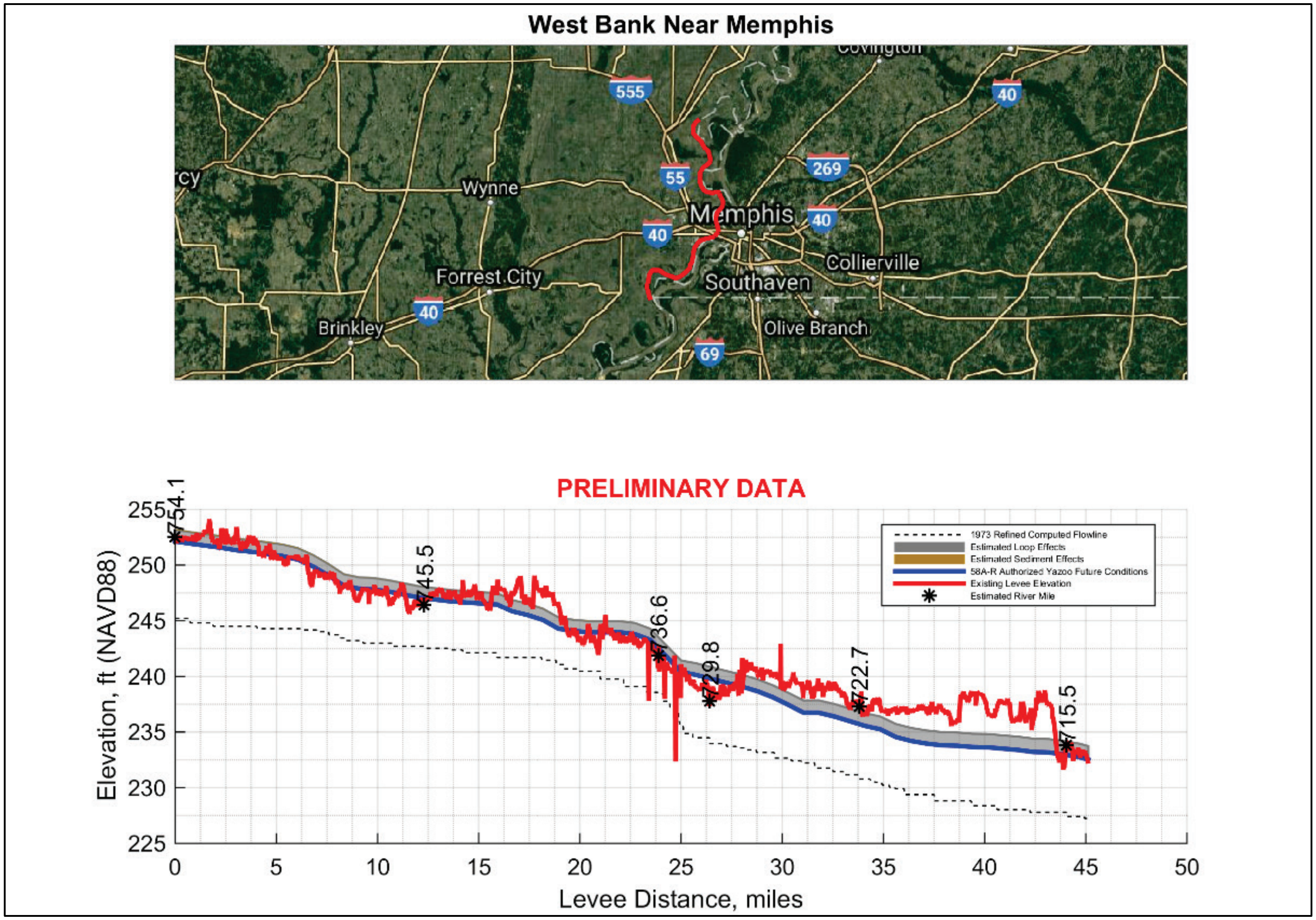


Figure 6-2. Combined results along right-descending levee near Vicksburg.

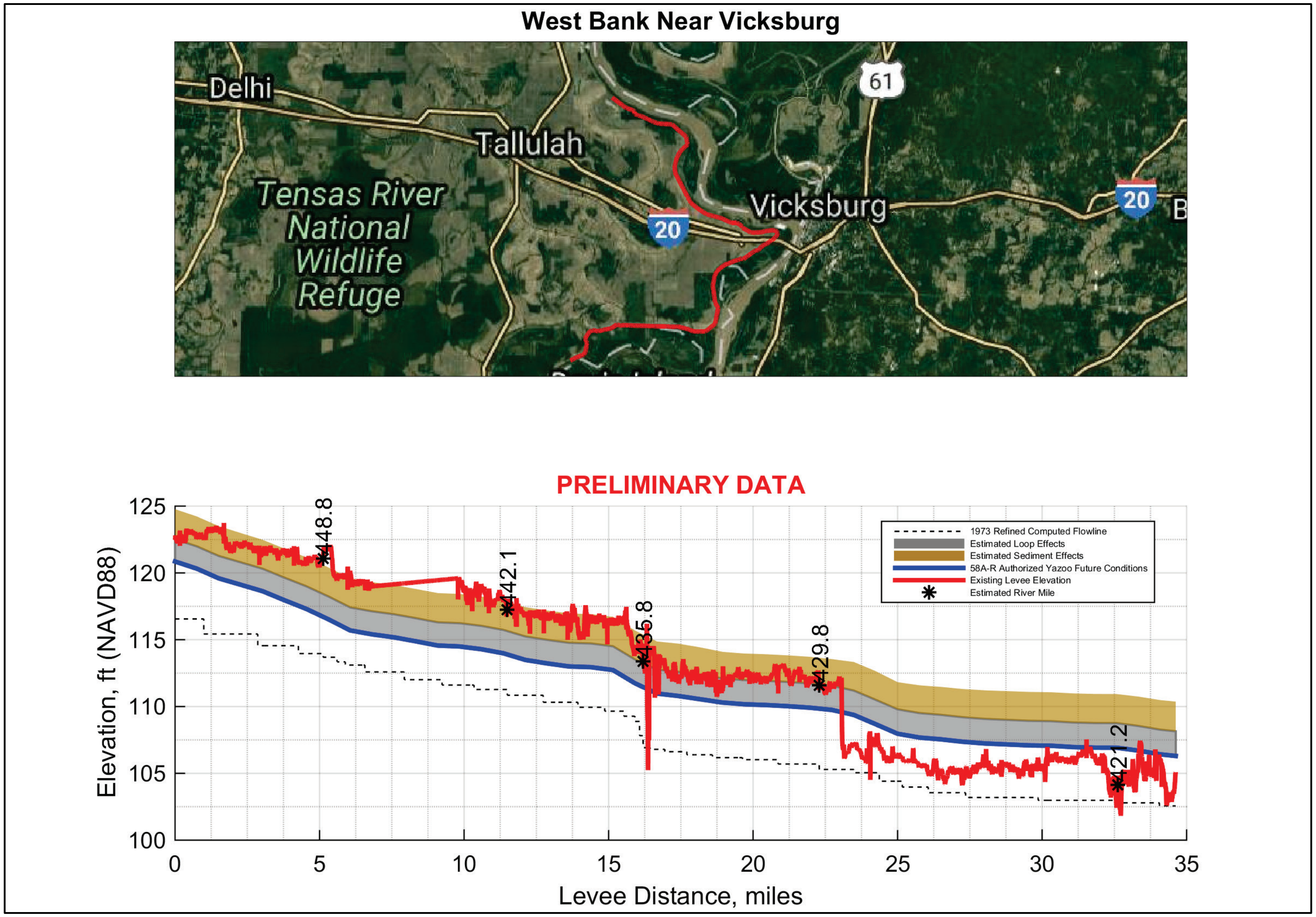


Figure 6-3. Combined results along right-descending levee from ORCC to Baton Rouge.

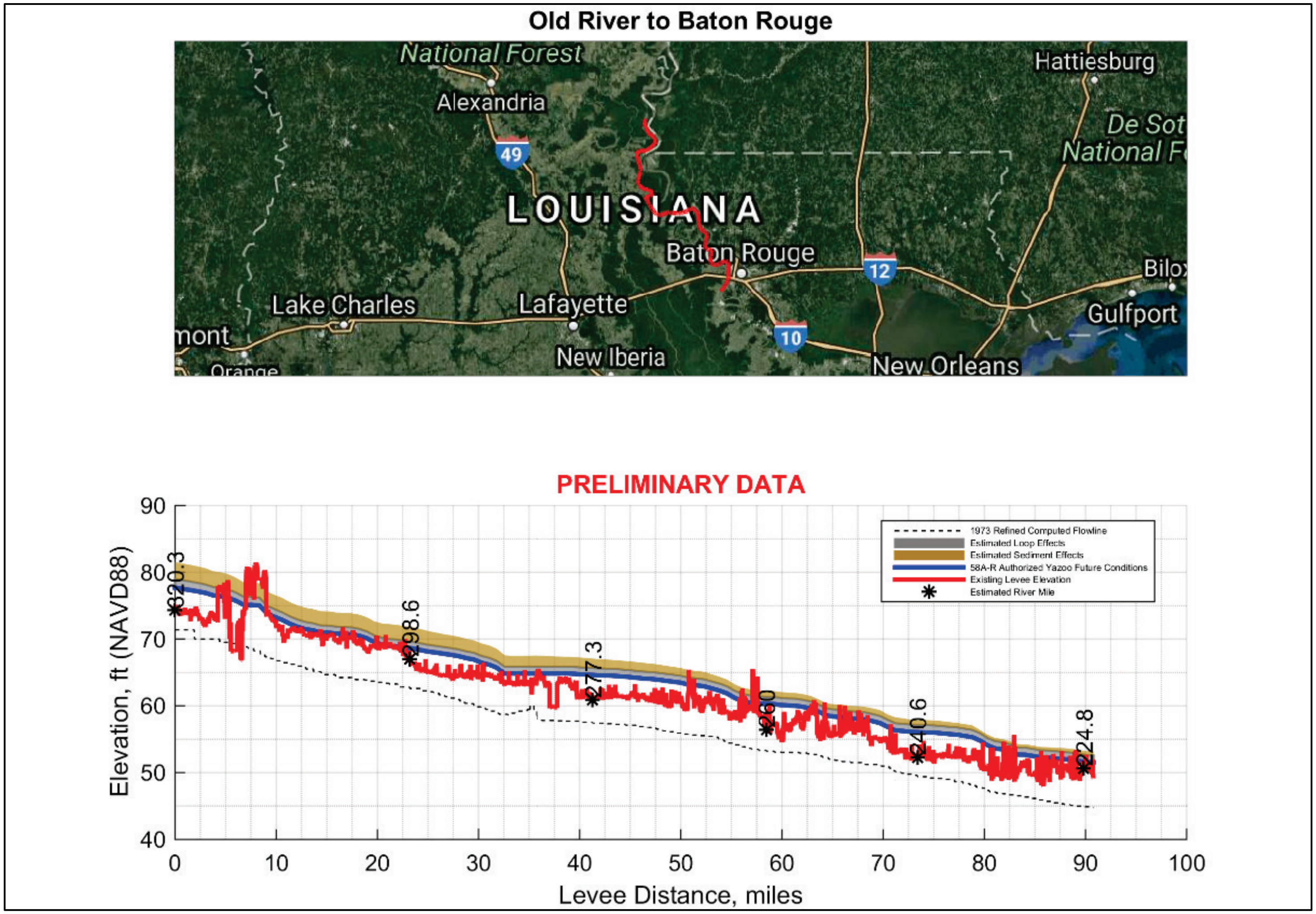


Figure 6-4. Combined results along right-descending levee near New Orleans.

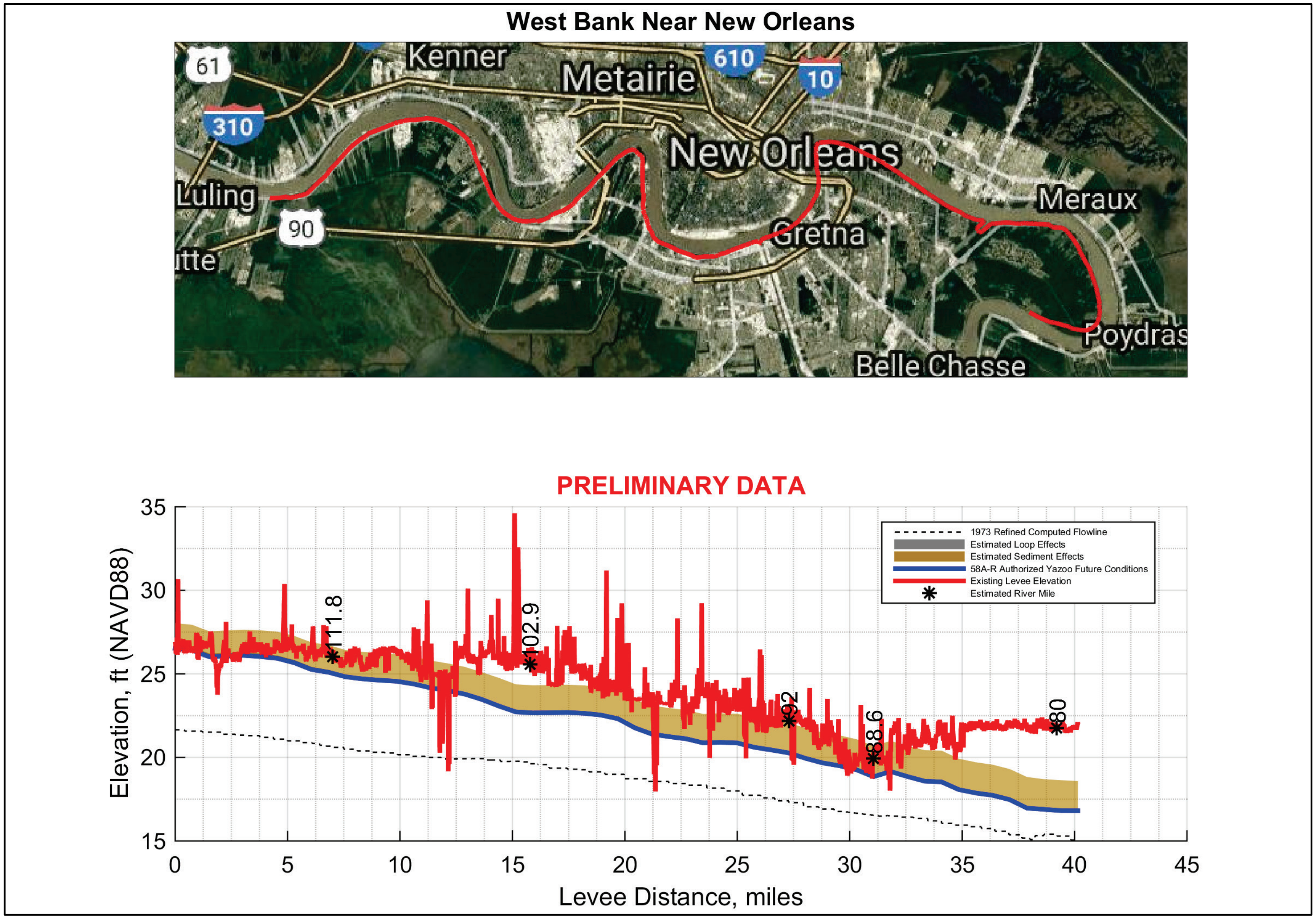


Table 6-1. Summary of levee comparisons by district boundaries.

Existing levee profiles evaluated against the 58A-R Authorized Yazoo Future Conditions (distances are in miles)[c]			
District	MVM[a]	MVK[a]	MVN[a]
No overtopping	369.6	147.6	149.9
Within 1 ft of overtopping	96.4	57.9	55.8
Within 3 ft of overtopping[b]	239.0	127.4	102.9
Overtopping	239.5	336.6	335.9
More than 3 ft of overtopping	61.7	180.1	166.2

[a] Mileage for each district is approximate and may not reflect the exact district boundaries.

[b] This includes the amount that is within 1 ft of overtopping.

[c] Segments not included in the evaluation include The Lower St. Francis, Old Town to Laconia Circle (White River Backwater), Clarendon, The Yazoo Backwater, and The Red River Backwater.

Although MVM has the largest number of levee river miles of the three districts, MVK and MVN have more miles of levee overtopping. In addition to the overall increase in flow through the MR&T System, one of the issues that this comparison also highlights is the important role of sedimentation that has occurred since the Refined 1973 Flowline (USACE 1978) and will continue to occur over the next 50 years. The sediment model projected that increases in water surfaces due to sedimentation were greater for MVK and MVN than are projected for MVM. From Table 6-1, MVK and MVN show 336.6 and 335.9 miles of overtopping, respectively. Of those overtopping distances, a large amount overtops by more than 3 ft: 180.1 miles for MVK and 166.2 miles for MVN. MVM shows 239.5 miles of overtopping, with 61.7 miles overtopping by more than 3 ft.

6.2 Conclusions

In summary, the flowline assessment was undertaken to review the current (Refined 1973 Flowline [USACE 1978]) MR&T flowline on which the project design levee elevations are based. These elevations are the grades at which the levees are being constructed that include some effects from sedimentation, loop effect, and free board to safely pass the PDF while operating the Old River Control Complex (ORCC), the four floodways and four backwater areas at their design capacities. The historic 2011 flood tested the MR&T system along the mainstem, and it passed the event, but physical and hydraulic changes in the system since 1973 revealed issues that needed to be investigated. These issues were detailed in the MR&T 2011 Post-Flood Report.

The flowline assessment was initiated in 2014 to assess the Refined 1973 Flowline. Evaluations considered the hydrology and hydraulic conditions of the system today, as well as other parameters that impact the flowline over the next 50 years. The steps of the assessment procedure were the following:

1. Develop precipitation amounts.
2. Calculate the amount of runoff.
3. Calculate the amount of flow in the river from the runoff for regulated and unregulated conditions.
4. Calculate a water surface profile/elevation with appropriate levees, structures, backwater areas, and floodways assumptions.
5. Develop estimates for future degradation/aggradation of sedimentation at various locations, climate change, loop effect, SLR, and subsidence, all of which would affect the flowline.
6. Compare flowline calculations for various models/runs.
7. Perform District Quality Control (DQC) Review, Agency Technical Review (ATR) and Independent External Peer Review (IEPR) on all of the above steps to ensure quality.

A suite of models was applied to calculate runoff and to simulate the resulting PDF water surface elevation. The results of the modeling are dependent on numerous model input parameters and assumptions. The results presented in the report represent a single set of assumptions and input parameters and do not include an assessment of model sensitivity to model parameters or to varying assumptions regarding reservoir regulation. While the results from this assessment provide a baseline of understanding, future activities will advance the modeling framework and analysis required for a comprehensive approach to operating and maintaining the MR&T project into the future.

A key finding from the assessment is that the meteorological conditions associated with the 1955 “Hypo Flood” still characterize the storm event that generates the PDF. The hydrology had not been updated since the 1955 Study. The 1955 Study evaluated 37 storm events, or HYPO events, that included those reservoirs that had been constructed and those planned for construction. From these evaluations, the storm sequence that provided the largest runoff, with regulation considered, was the 58A-EN. It was a three-storm event from conditions of January 1937, February 1938, and January 1950. The current flowline assessment ascertained that the 58A-EN was still the storm event that generated the maximum PDF. The current assessment also evaluated a new storm event called the HYPO 11-73 that was taken from the floods of 1973 and 2011. The HYPO 11-73 flows did not exceed the flows of 58A-EN. A comparison of the computed 2016 unregulated peak flows to those unregulated peak flows from the 1955 Study were within 3% of each other. However, the total volume of computed unregulated flows is substantially greater applying the 2016 methodology. A comparison of flows under the regulated conditions and assumptions of the 2016 study showed a 17% increase when compared to the 1955 Study 58A-EN (MRC 1955). This analysis revealed reservoir regulation storage assumptions under the existing Flowline are not as large in reality as originally assumed. New technology has allowed for a more accurate and comprehensive representation of precipitation from the Hypo Flood. Both of these updated methods results in greater unregulated runoff volumes and higher regulated flows.

There were differences in reservoir regulation between the 1955 Study and the current assessment. Some of the reservoirs that were planned at the time of the 1955 Study were not constructed, and the assumption of the 1955 Study was that reservoirs would be able to maximize the flood storage capacity. Today, reservoirs are operated according to guide curves that are primarily operated to provide localized flood protection and not for restraining the PDF along the mainstem. The volume of unregulated runoff has increased, and the amount of expected reservoir storage has been reduced from the 1955 Study. The current assessment shows an additional 430,000 cubic feet per second (cfs) flowing into the Mississippi River at the confluence of the Mississippi and Ohio Rivers and over 450,000 cfs at Memphis, TN. Additional analysis must be performed to investigate the sensitivity of runoff routed to the river for various model parameters and alternative regulation approaches to accommodate PDF flows.

The hydraulic assessment evaluated how this additional flow from the PDF would be handled by the levees, ORCC, backwater areas, and floodways. The hydraulically computed water surfaces were updated in 1978. Both the 1955 and Refined 1973 (USACE 1978) studies modeled peak flows using a steady state model. The current (2016) assessment used an unsteady model that would show changes in flow with time. The latest unsteady flow hydraulic model from the HEC-RAS was used in this analysis. It was calibrated from data collected from the 2011 flood and validated from data collected from smaller floods in 2002 and 2008. Once calibration was accomplished by the MVM, MVK, and MVN on their particular reach of the river, the models were combined, and model runs were conducted to make sure the combined model gave the results that each district had observed when modeled on their reach of the Mississippi River. This model could then be utilized to run different assumptions to calculate the expected water surface elevations.

The first model run was undertaken using existing backwater levee conditions. Three of the four backwater levees are at, or above, their authorized grades—with only the Yazoo Backwater levee not at its authorized grade of 112.8 ft but at 107.0 ft (NAVD 88). The ORCC and the four floodways were operated at design capacity while the mainline levees were artificially raised in the model, thereby forcing all the flow to remain between the levees. This allowed the model to calculate water surface elevations to determine an approximate order of magnitude of the miles of levee that would be below the estimated water surface profile of the PDF. Results showed that this forced flow run of estimated water surface profile exceeded the existing levee conditions in all three districts. Three of the four floodways would activate, and only the Yazoo backwater area would be minimally activated.

A second model run was undertaken using the authorized Yazoo backwater levee conditions. The authorized Yazoo backwater levee grade is 112.8 ft. Again, ORCC and the four floodways were operated at design capacity while the mainline levees were artificially raised in the model thereby forcing all the flow to remain between the levees. This forced flow run resulted in slightly higher estimated stages, and existing levee elevations were exceeded in all three districts including the area from Baton Rouge to the Gulf of Mexico. This model change had no effect on stages above Memphis, TN. The Yazoo backwater area operated somewhat efficiently with the previous (existing Yazoo elevation of 107.0 ft) run, although the timing in the removal of the peak flows was not at the optimal time. This run, with the authorized Yazoo elevation of 112.8 ft, showed that the Yazoo backwater area operated even less efficiently. The other three backwater areas only operated minimally, and only three of the four floodways would activate with this model run. This assessment revealed that the backwater areas and the West Atchafalaya floodway fuse plug sections are no longer at optimum levels for activation.

A hydraulic assessment was also done for the Atchafalaya River, incorporating the updated features of unsteady (one-dimensional) 1D and (two-dimensional) 2D HEC-RAS modeling to calculate the water surfaces during the hypothetical PDF events. The upstream boundary conditions for the Atchafalaya River included Old River outflow channel flows, Red River flows, and Morganza Floodway flows from the Mississippi River HEC-RAS model. In general, the calculated Atchafalaya River water surfaces were lower than the currently authorized 2010 flowline. This is understandable due to a number of modeling differences, such as unsteady 2D hydraulic calculations compared to steady-state 1D calculations used in the 2010 study, a lower downstream boundary water surface than the 2010 study, and less flow than the 2010 study. In addition to modeling differences, channel surveys in recent years have consistently shown widespread scouring, or channel lowering, in the Upper Atchafalaya River. In the Lower Atchafalaya Basin, there has been a long trend of accretion occurring in the river channel and in the floodplain areas. This filling has caused the stage-flow relationships to increase for all flow regimes.

Another major part of the flowline process was to develop estimates of future degradation/aggradation due to sedimentation. The sedimentation study of the Mississippi River showed that over the next 50 years, the project flood peak water surface elevations would increase to 2.5 ft between Head of Passes and Helena, AR, with the maximum being in the area immediately upstream of the ORCC. During the same period, the water surface elevations between Helena, AR, and Cairo, IL, would decrease. Since the project flood could occur at any time during the 50-year period, levee design heights would have to account for the higher water surface elevations. These degradation/aggradation trends were shown to be a long-term geomorphic response to the cutoff program that shortened the river by 152 miles in the 1930s and 1940s. The Atchafalaya River sediment model showed that the PDF event itself will scour channel bed elevations by a large amount during the rising flows up to the peak, especially near Morgan City where a large amount of floodplain flow rejoins the river. Over the next 50 years, the sediment model results project trends of aggradation, or higher water surfaces into the future, for the Lower Atchafalaya River. Although the Upper Atchafalaya River has clearly exhibited large amounts of scour in recent decades, the sediment model projects a dynamic equilibrium to slightly increasing trend in water surfaces going forward. Future water surfaces in the upper reach could be lower than model projections due to channel widening, a process which should be included in future sediment modeling efforts of the Atchafalaya River.

The loop effect, or hysteresis, has been studied as part of this assessment. The loop effect is a river flow hydraulic phenomenon that results in a lower river stage for a rising river given the same flow than a falling river. It is more prevalent on large alluvial bed streams such as the Mississippi River and is caused by factors such as variable energy slopes, scour and fill, alluvial bedform changes, backwater areas, temperature, movement of water into and out of storage, etc. While the 1973 Flowline evaluation had some loop effects included, the current assessment evaluated the unsteady hydraulic model relative to measurements taken from past flood events. The hydraulic models used for the current assessment accounted for part of the loop effect but not all of it. The additional loop effect that was calculated for this study varied by location and was determined to be an additional 1 to 2 ft.

The additional components of sedimentation, loop effect, SLR, and subsidence were combined with the two primary hydraulic model runs. This showed how these additional factors would affect the establishment of any water surface profile and in turn the establishment of levee grades. The combined effects showed increases in the flowline at various locations along the mainstem that in turn would cause levee elevations to need to be higher.

All of the studies and model runs described in this report have been thoroughly reviewed by undergoing a DQC review, an ATR, and an intense IEPR. Most of the comments from these reviews showed that the team did a good job in their assumptions and modeling efforts. Comments from these reviews are included in Appendix D of this report.

Efficient management of the system requires the most up-to-date and technically competent data and analyses, and the following future activities are therefore recommended:

- <u>Advance hydrologic modeling capability.</u> Planning and understanding risk for the MR&T requires efficient and accurate hydrologic simulations. Runoff for the assessment was computed by the National

Weather Service (NWS), which applied its operational hydrology model for determining daily river forecasts. The scope of this evaluation limited the team's ability to address sensitivity to model parameters as well as sensitivity to climate change, different rainfall depths, durations, and intensities. Alternative reservoir regulation assumptions could also not be evaluated. Therefore, a USACE hydrology capability should be developed for running the NWS model as well as to investigate alternative hydrology modeling approaches to provide efficient and accurate hydrology results. Efficient hydrology simulation will allow for extensive analysis of hydrology model parameters and inputs, understanding effects of climate change, and simulations for optimizing ways to regulate key features in the basin.

- Develop operational HEC-RAS model of the Mississippi River. An unsteady HEC-RAS model of the MR&T domain was developed for the assessment. The model was calibrated for high flows only. Efficient management of the MR&T requires a model that is kept up to date and is valid over a range of flows such that it can be run operationally. A process, tools, and techniques to obtain, store, and ingest required data for simulation should be developed. The model should be calibrated and verified for low flow events to ensure accurate simulation over a range of flows. The model can then be run operationally and compared to data on a recurring basis. This process will allow for continuous evaluation and update of the model to ensure its accuracy as well as allow for efficient future application as necessary.
- Continue geomorphic assessments of the river. The dominant morphological processes that shape the Mississippi River channel operate over a very large range of spatial and temporal scales. There are many factors, both natural and man-induced, that can contribute to these processes. The effects of large flood events, changing sediment loads and characteristics, channel maintenance activities, dredging practices, diversions (natural and man-made), and relative SLR are just a few such factors. Morphologic changes have important implications related to stage-discharge relationships and operating various flood control features of the MR&T. Geomorphic and sedimentation studies on the Mississippi and Atchafalaya Rivers should continue to be conducted to determine trends and river processes for long-term management of the river.
- Advance risk-based analysis techniques. A comprehensive approach for managing and improving the MR&T project requires risk-informed decision-making. Risk-informed decision-making requires tools and

techniques that consider all the critical vulnerabilities, are done probabilistically, and appropriately incorporate uncertainty. Studies should be conducted and tools developed to support system-wide risk-informed river management. State of the art probabilistic approaches to properly characterize the probability of hazards and consider appropriate non-exceedance confidence limits should be evaluated and incorporated as necessary into the planning process.

- Utilize the results of this assessment and the additional recommended activities above for monitoring and potential future alterations of the MR&T system to ensure safe passage of future floods.

Until the necessary additional analyses are completed, design and construction should continue to utilize the PDF defined by the Refined 1973 PDF profile and existing Congressional authorization.

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Appendix A: Additional Figures for the Hysteresis Effect Analysis

Figure A-1. Plot of 2008 stage and discharge data at Hickman.

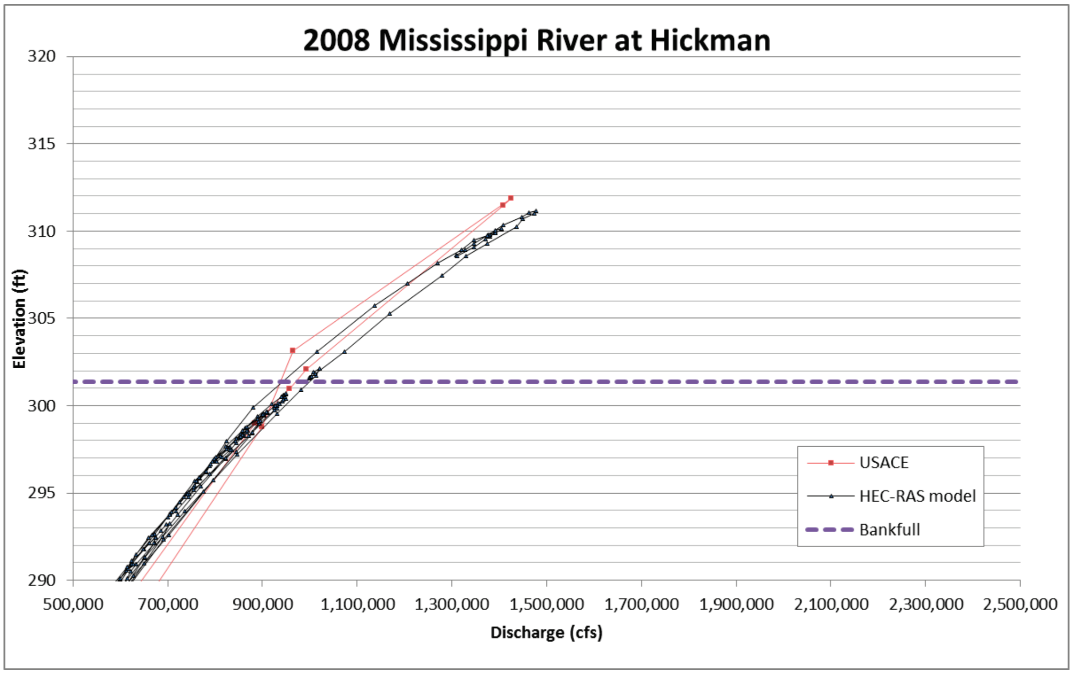


Figure A-2. Plot of 2011 stage and discharge data at Hickman.

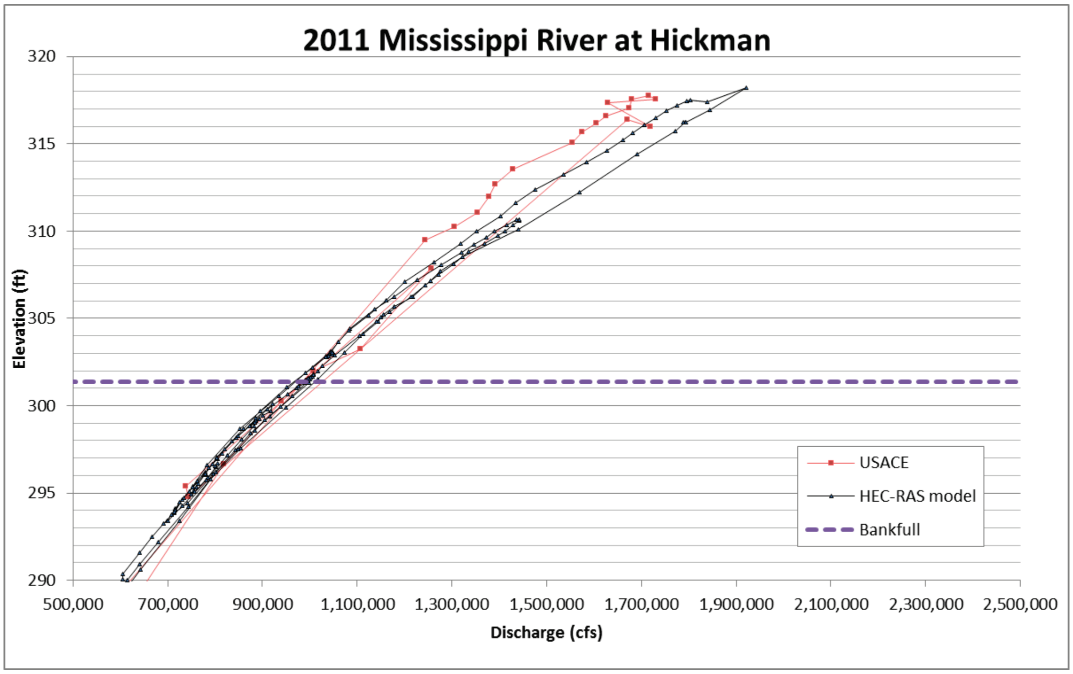


Figure A-3. Plot of 2008 stage and discharge data at Memphis.

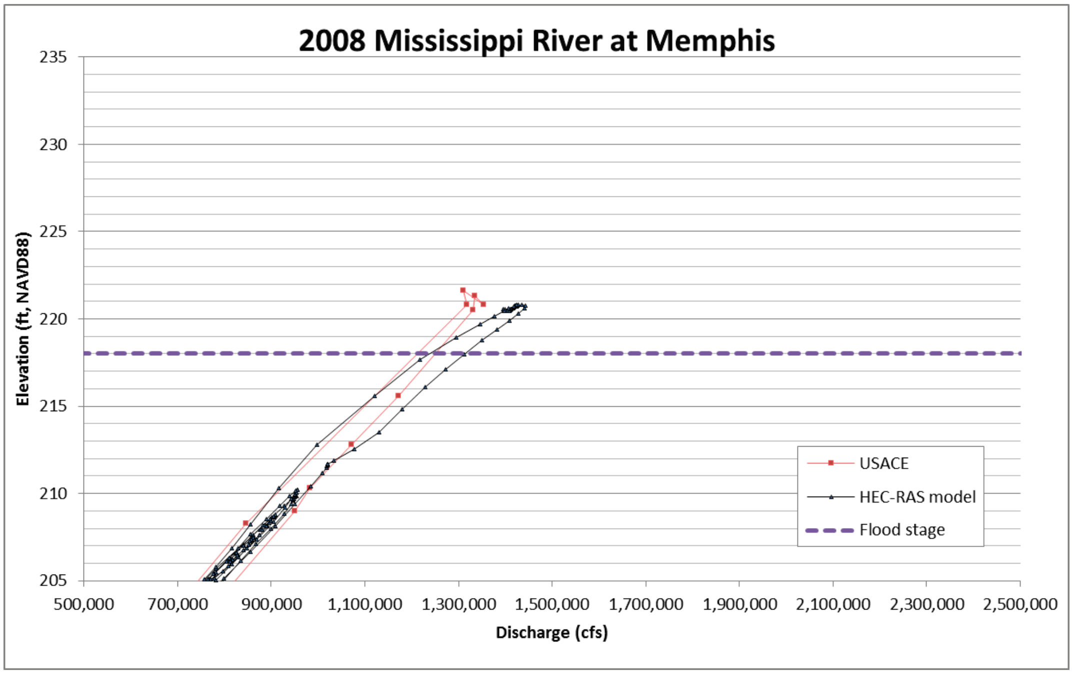


Figure A-4. Plot of 2011 stage and discharge data at Memphis.

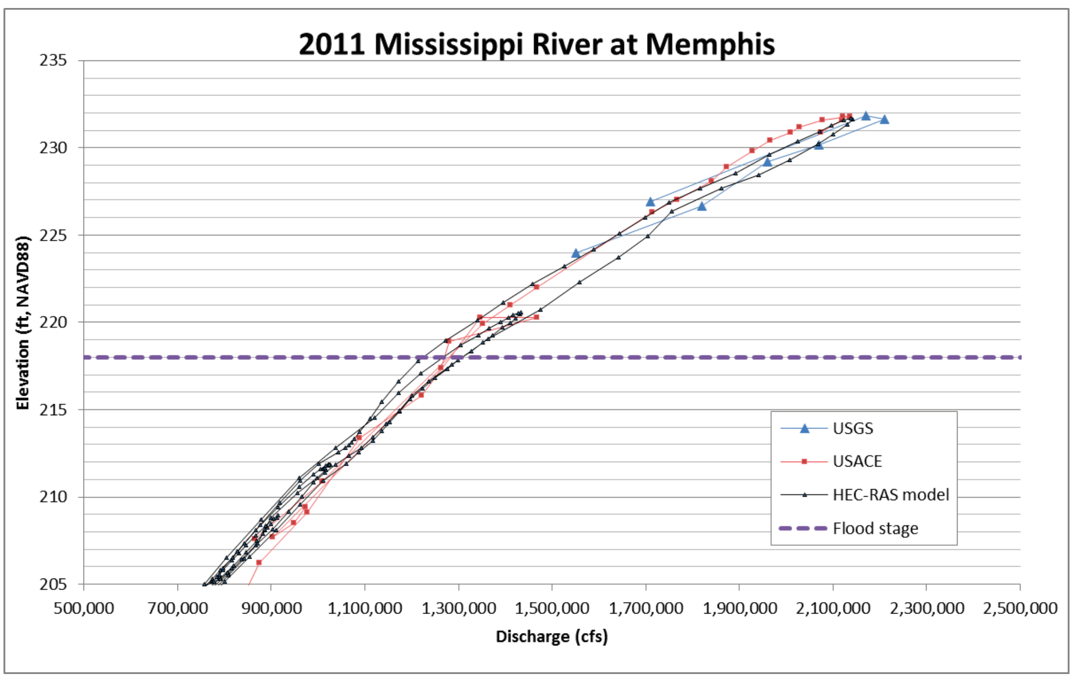


Figure A-5. Plot of 2008 stage and discharge data at Helena.

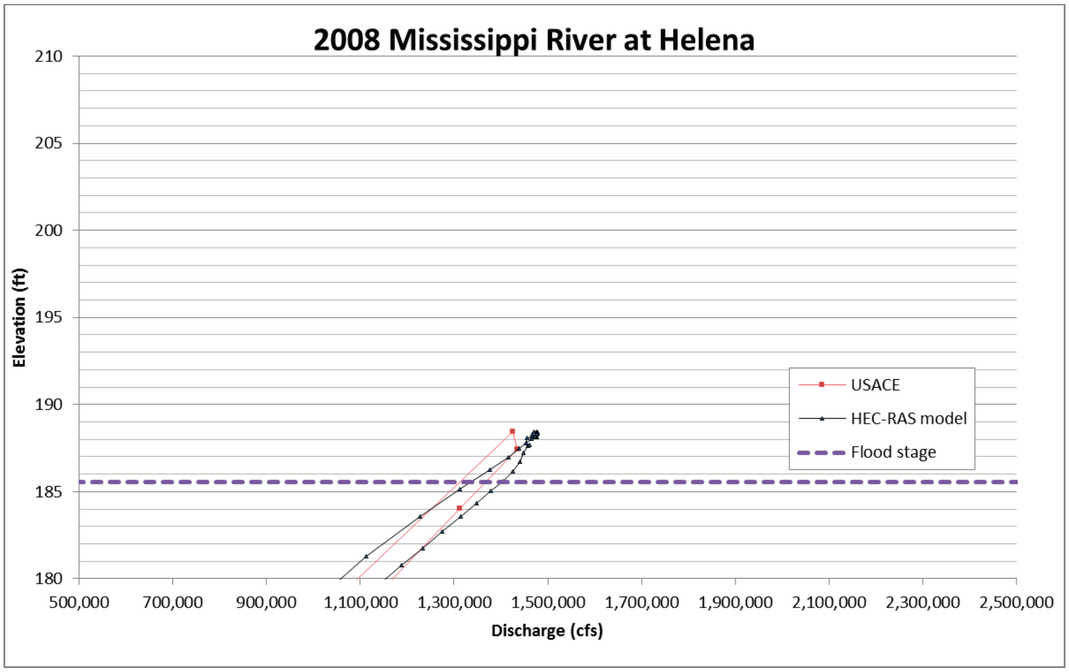


Figure A-6. Plot of 2011 stage and discharge data at Helena.

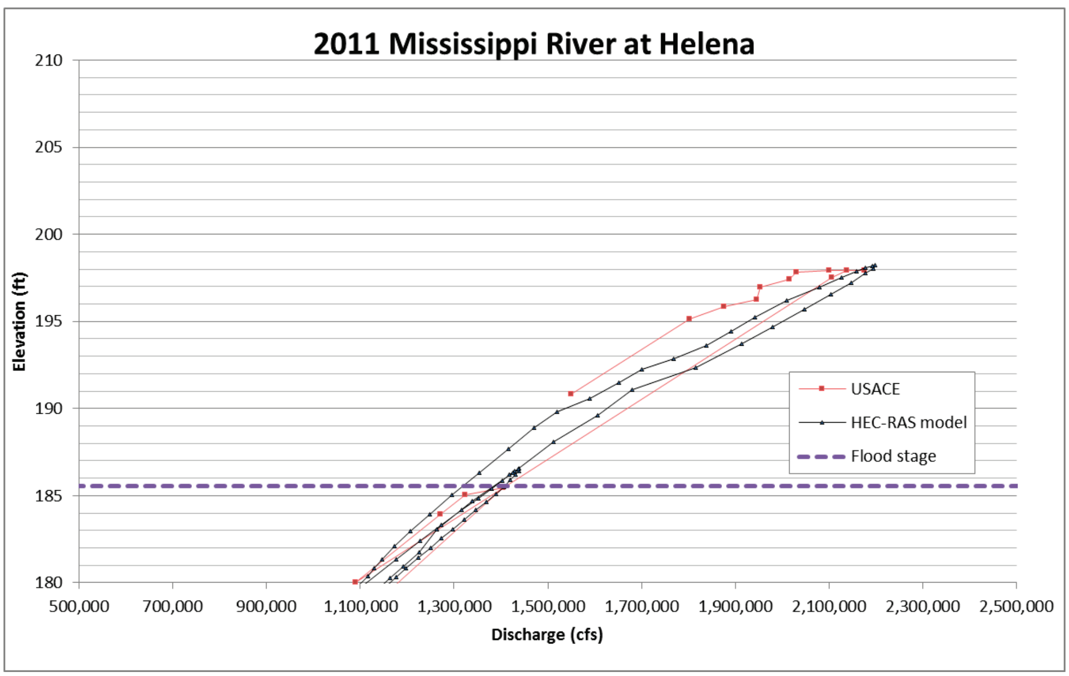


Figure A-7. Plot of 2008 stage and discharge data at Arkansas City.

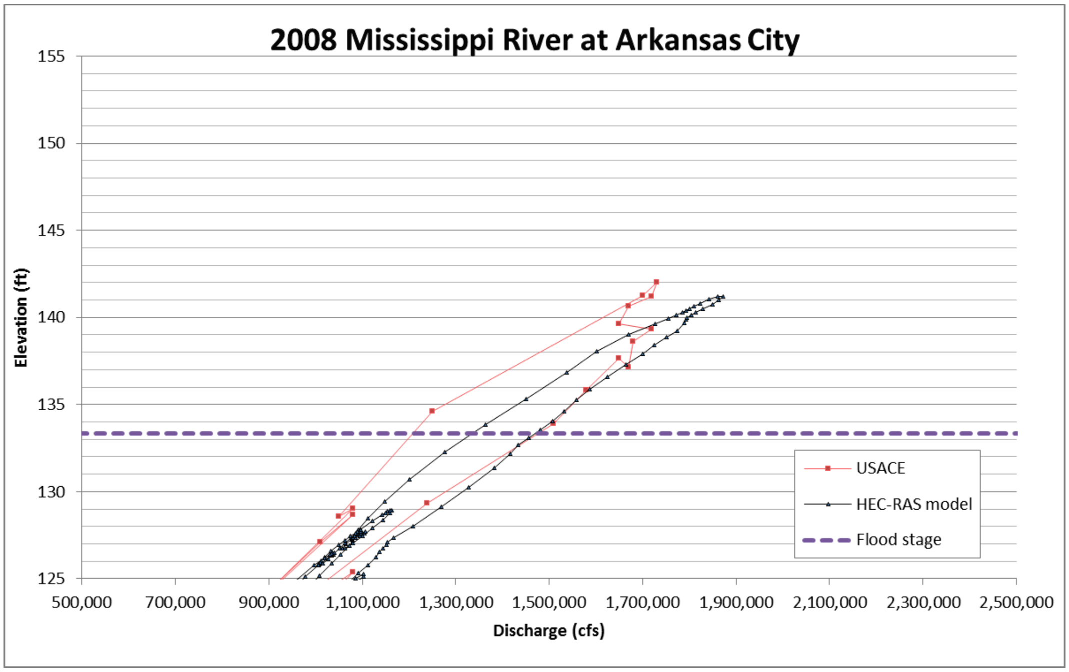


Figure A-8. Plot of 2011 stage and discharge data at Arkansas City.

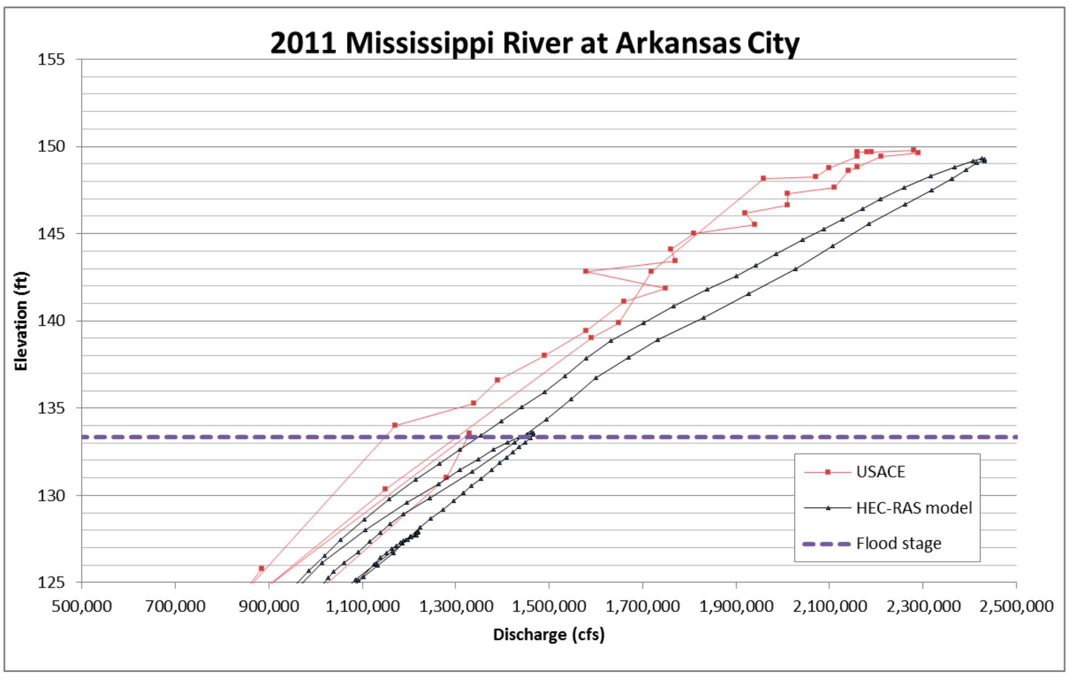


Figure A-9. Plot of 2008 stage and discharge data at Greenville.

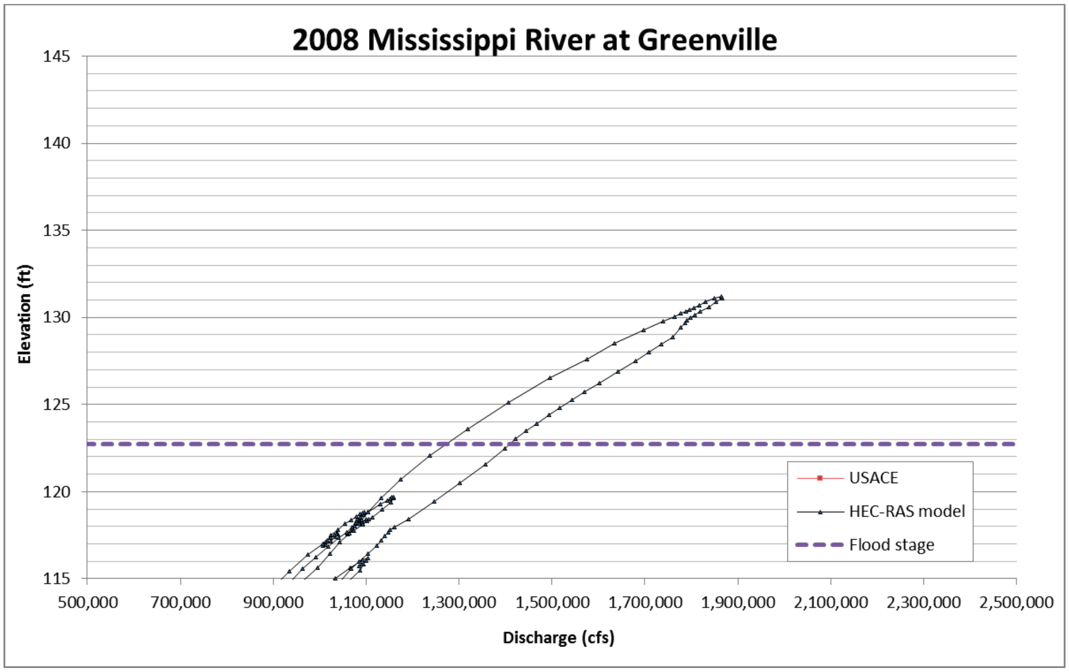


Figure A-10. Plot of 2011 stage and discharge data at Greenville.

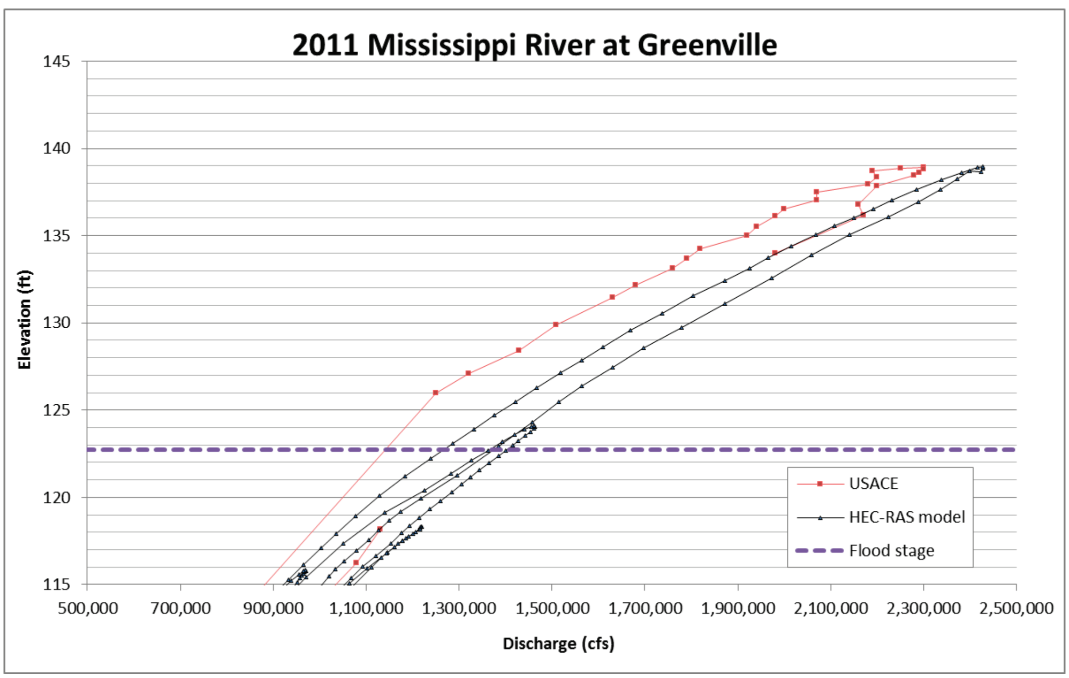


Figure A-11. Plot of 2008 stage and discharge data at Natchez.

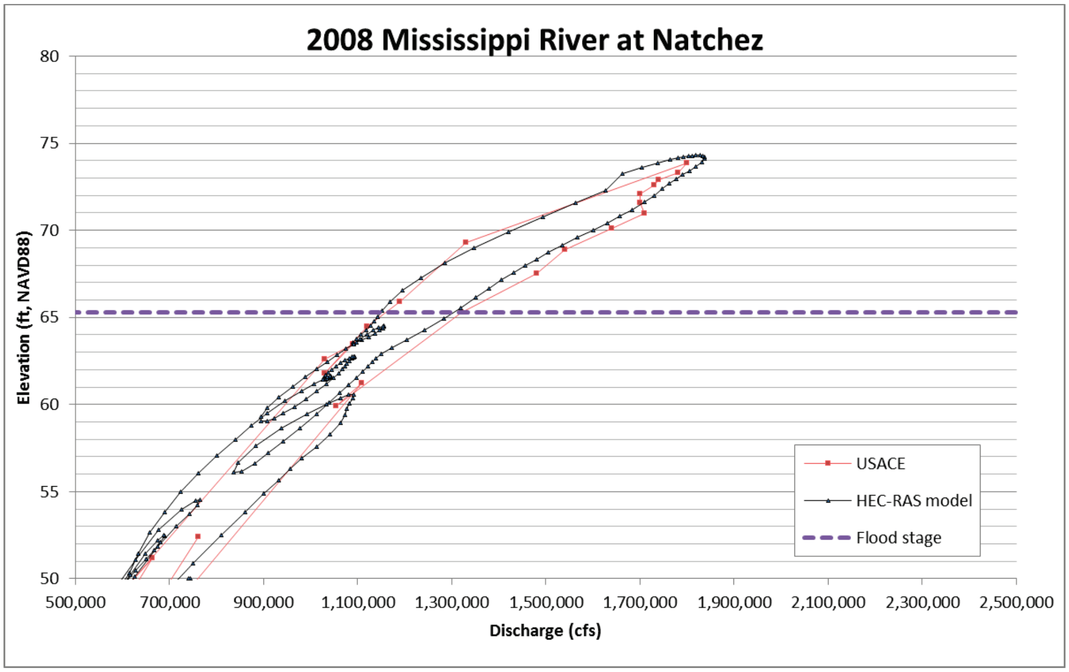


Figure A-12. Plot of 2011 stage and discharge data at Natchez.

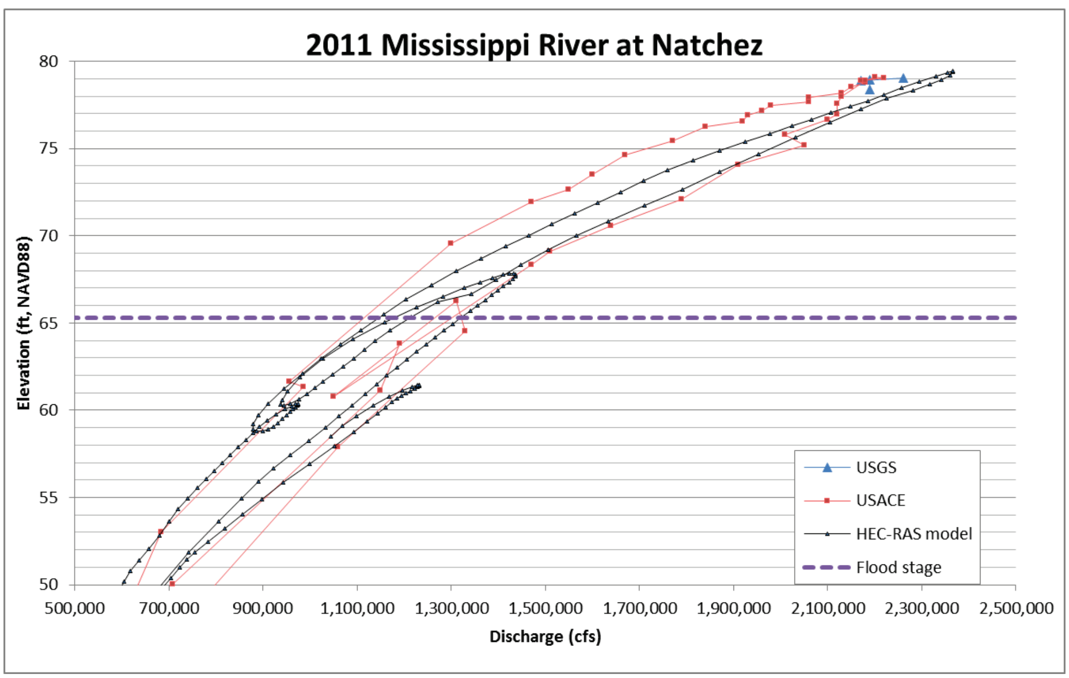


Figure A-13. Plot of 2008 stage and discharge data at Red River Landing.

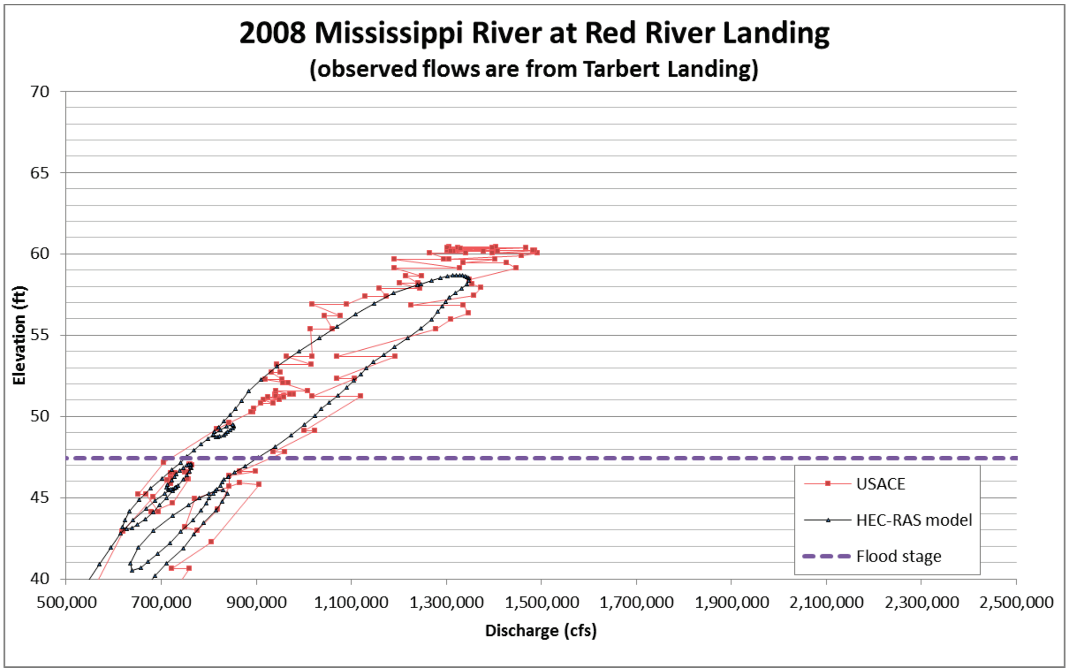


Figure A-14. Plot of 2011 stage and discharge data at Red River Landing.

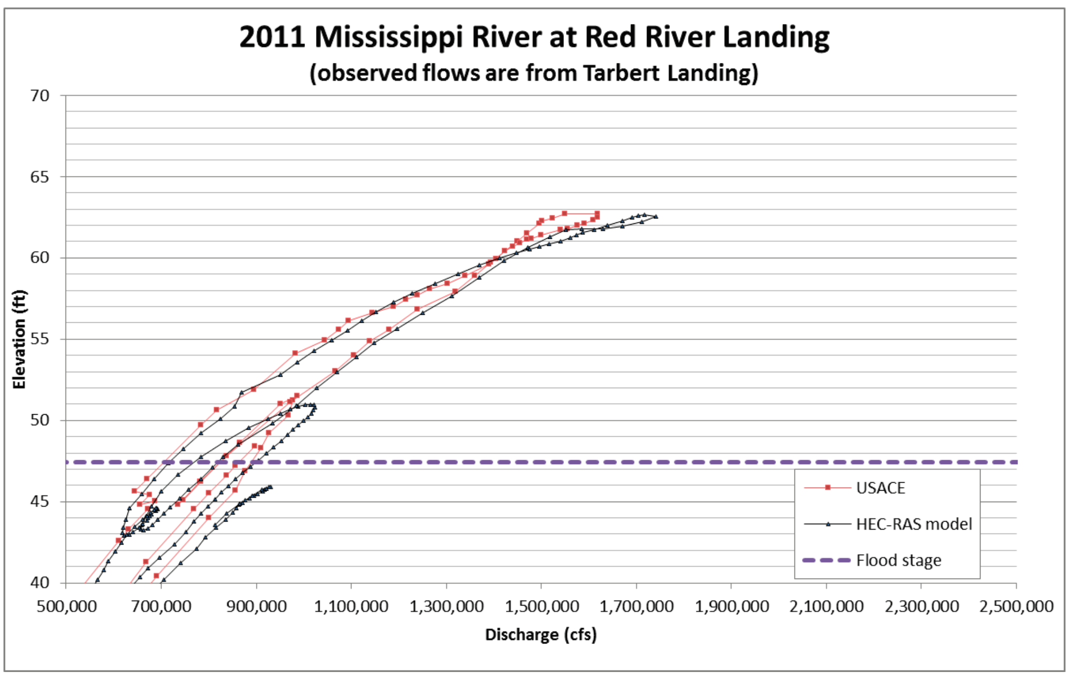


Figure A-15. Plot of 2008 stage and discharge data at Baton Rouge.

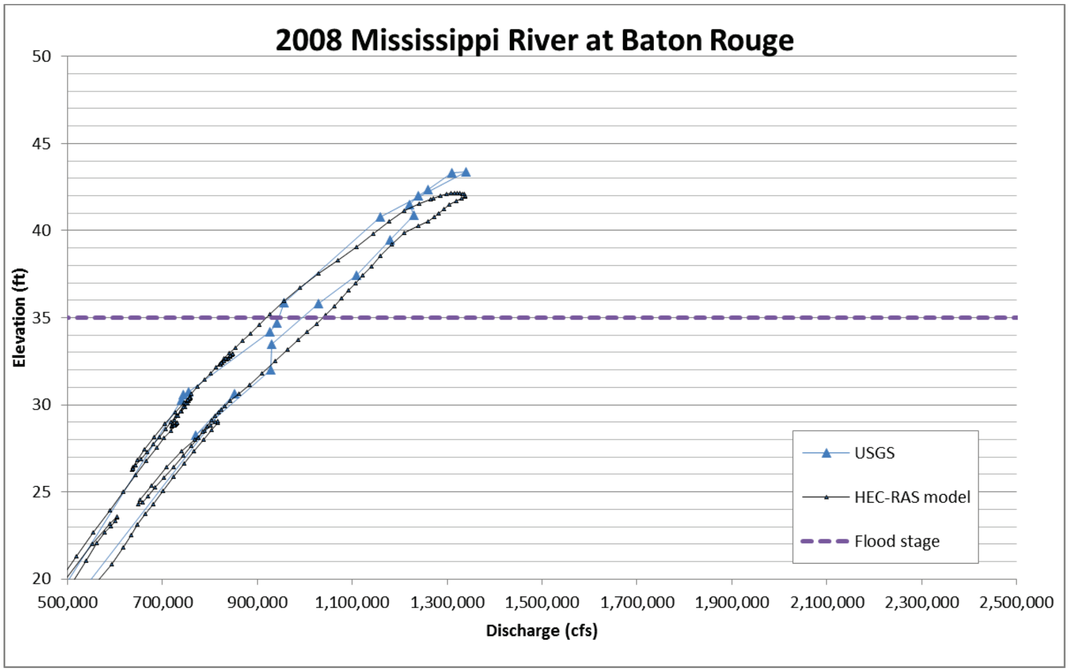


Figure A-16. Plot of 2011 stage and discharge data at Baton Rouge.

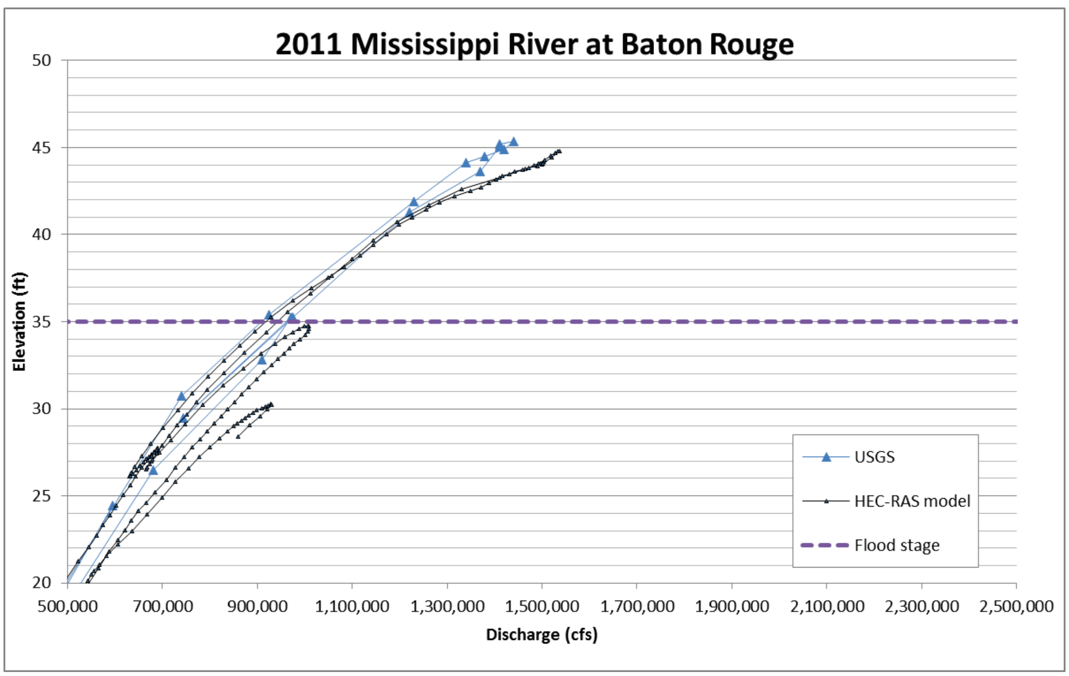


Figure A-17. Hysteresis of the HEC-RAS results of the PDF event “58A-R Authorized Yazoo” simulation at Hickman.

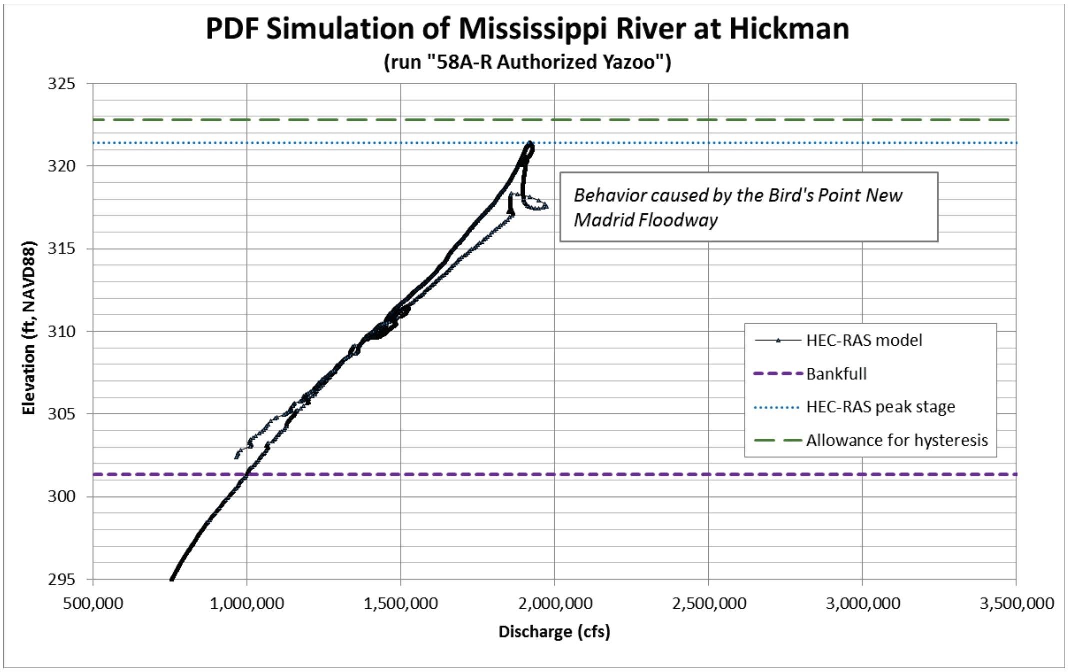


Figure A-18. Hysteresis of the HEC-RAS results of the PDF event at Memphis.

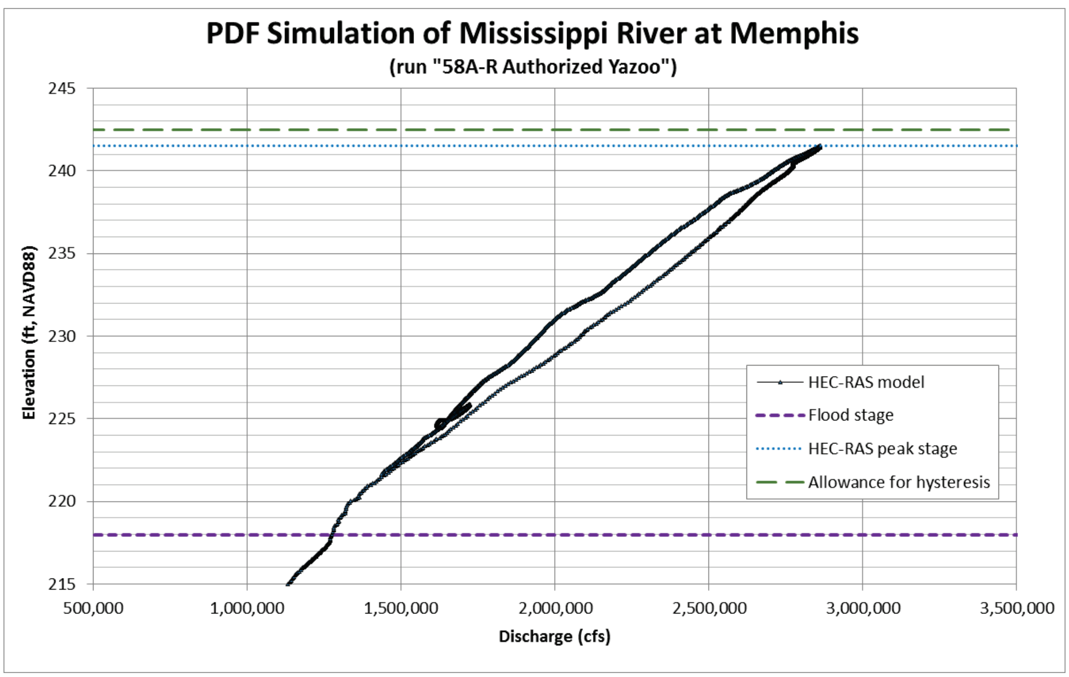


Figure A-19. Hysteresis of the HEC-RAS results of the PDF event at Helena.

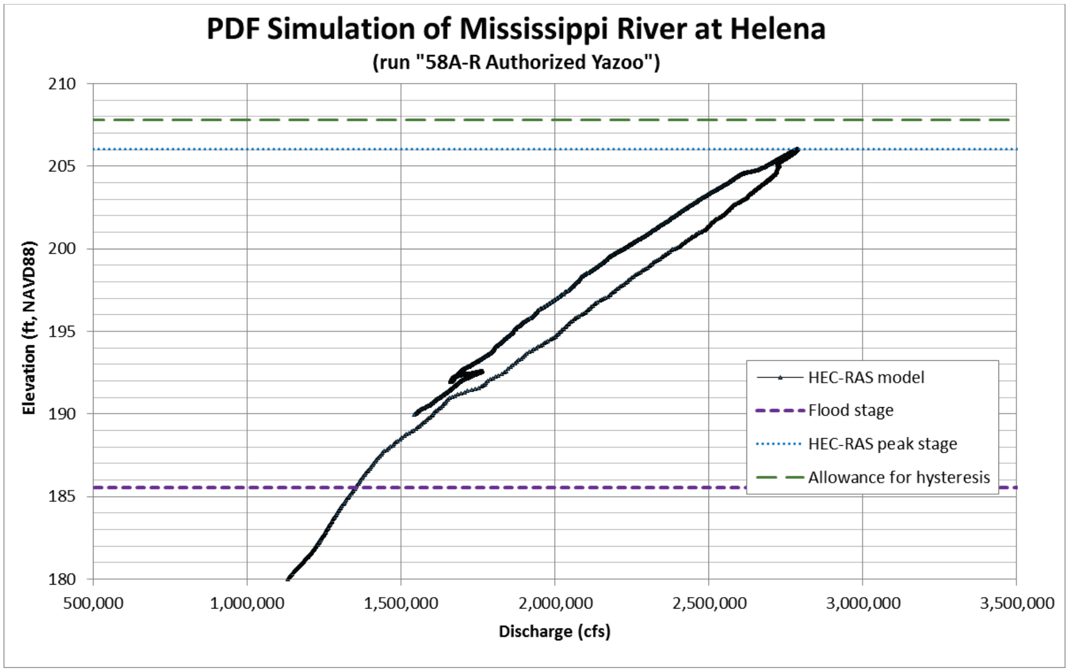


Figure A-20. Hysteresis of the HEC-RAS results of the PDF event at Arkansas City.

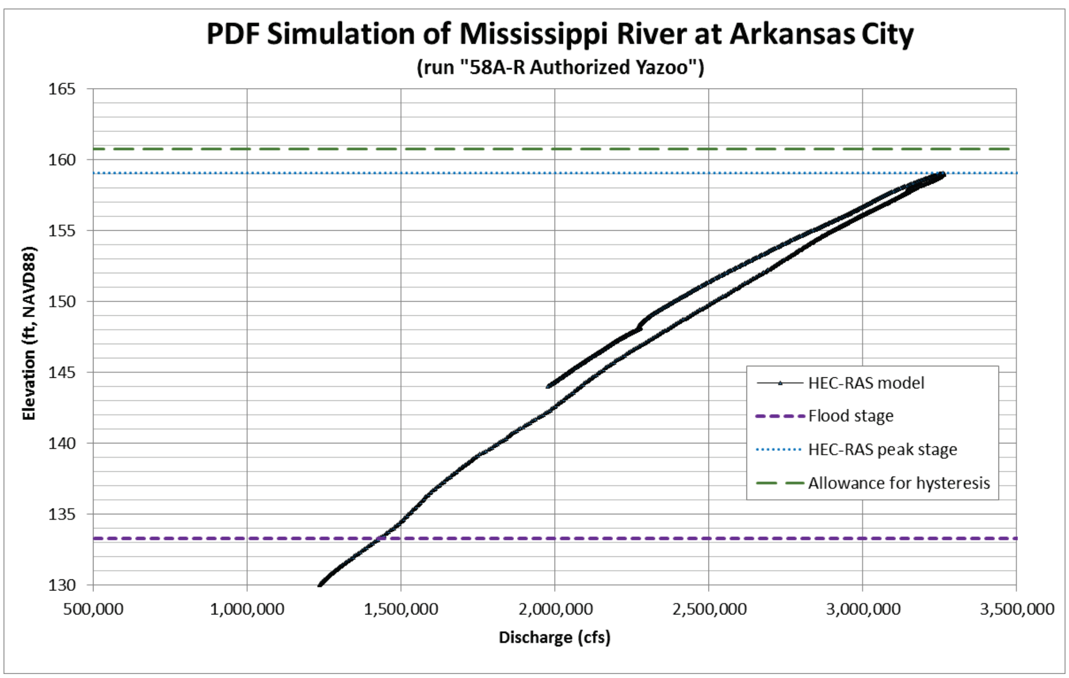


Figure A-21. Hysteresis of the HEC-RAS results of the PDF event at Greenville.

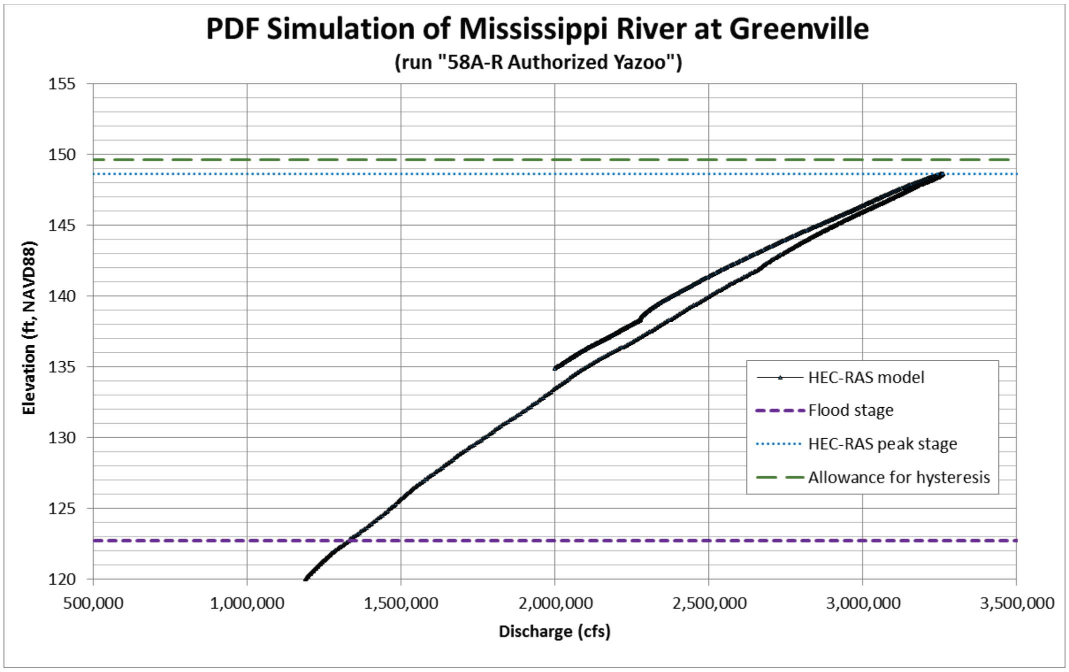


Figure A-22. Hysteresis of the HEC-RAS results of the PDF event at Natchez.

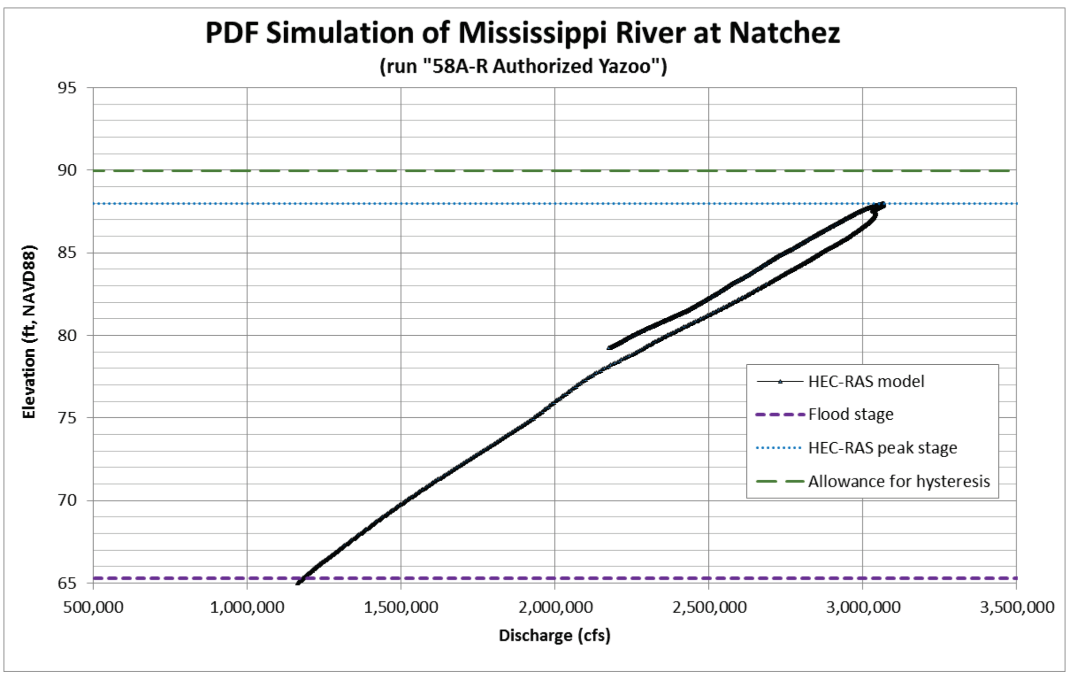


Figure A-23. Hysteresis of the HEC-RAS results of the PDF event at Red River Landing.

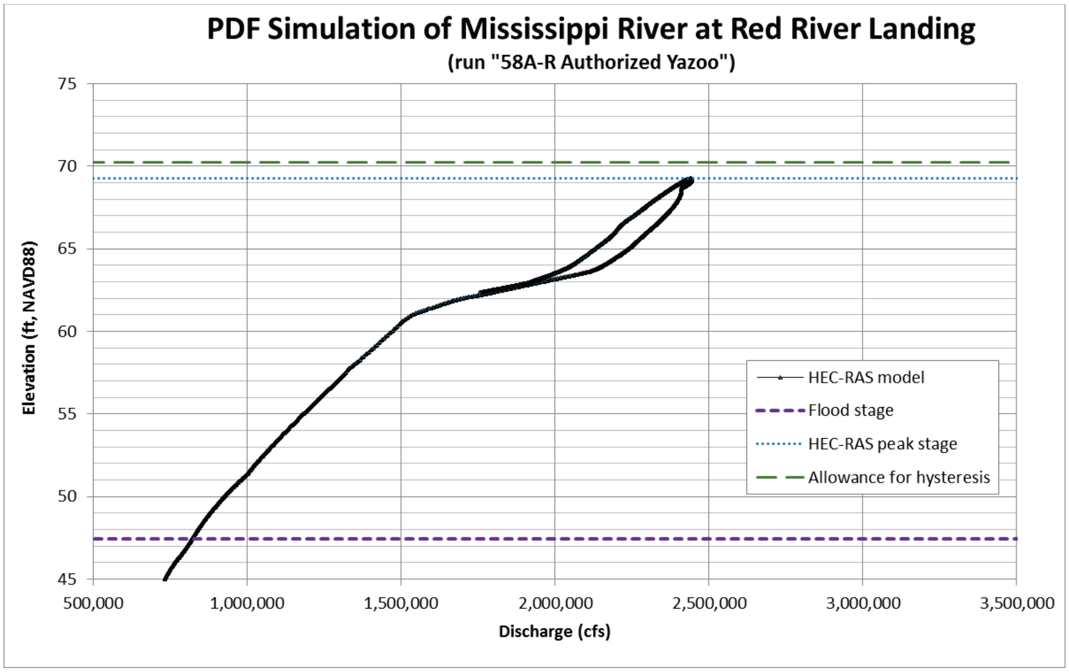
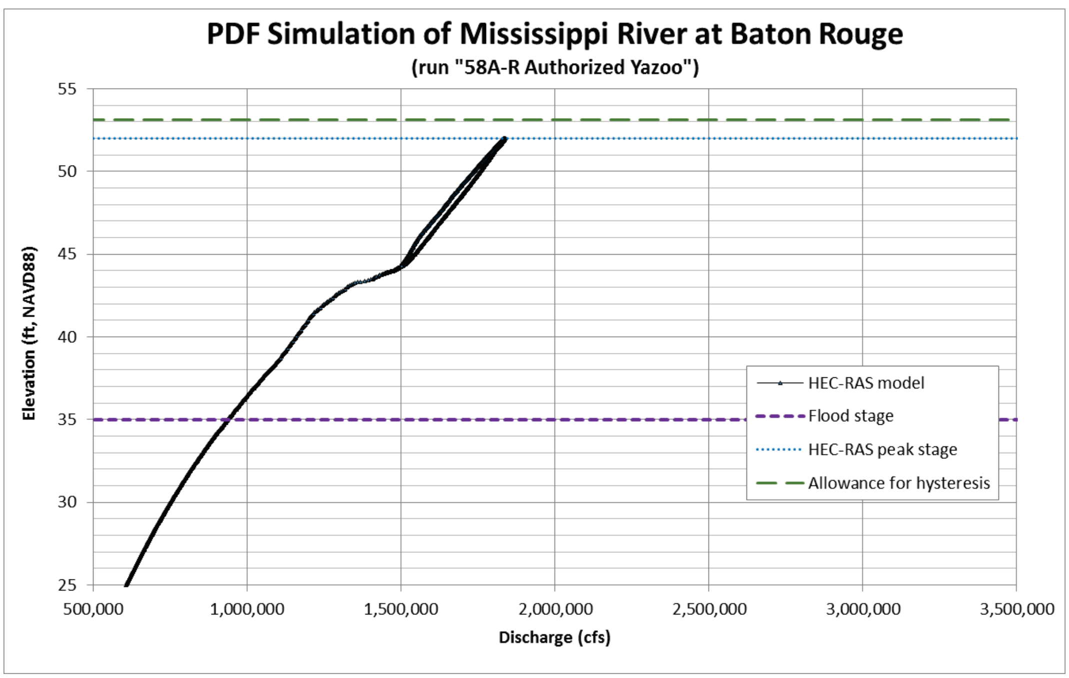


Figure A-24. Hysteresis of the HEC-RAS results of the PDF event at Baton Rouge.



Appendix B: List of Preparers

Name	Office	Discipline	Experience	Role in Preparing Report or Supporting Appendices
James Lewis	ERDC	B.S., M.S., Ph.D. Civil Engineering	4 years Hydraulic Engineer with the USACE Detroit District; 3 years Research Hydraulic Engineer with Coastal and Hydraulics Laboratory	Atchafalaya River HEC-6T sediment model, hysteresis (loop effect) analysis, report writer, hydraulics team support
Charles McKinnie	MVK	B.S., Civil Engineering	3 years Civil Engineer; 26 years Hydraulic Engineer; 9 years Project Manager with the USACE MVK	Project Manager responsible for managing a regional project and coordination of the budget, funding, PDT meetings, schedules, and upward reporting.
Kent Parrish	MVK	B.S., Ag Engineering; M.S., Business Administration	7 years, Asst. Project Engineer, Soil Conservation Service; 12 years, Planning Study Manager and 15 years, Senior Project Manager, USACE; 7 years, Regional MRL Senior Project Manager, USACE	Senior Project Manager/Team Leader
Cassie Love	MVK	A.A.	8 years Admin/Program Analyst, USACE MVK	Program Analyst responsible for funding labor and non-labor on project. Keeping up with actuals for each month and keeping everyone abreast of how much they need to spend each month.
Will Bradley, PE	MVK	B.S. Civil Engineering; M.S., Management Systems Engr.	15 years engineering experience. 11 years engineering at MVK and 4 years at MVD. (7 years structural engineer, 4 years Dam Safety, 2 years assistant chief RB-T at MVD, and 2 years assistant chief E&C at MVK)	Assistant Chief, Engineering and Construction of Vicksburg District, providing guidance and direction to the team
Michael Sorrels	MVK	B.S. Civil Engineering; Masters of Engineering	10 years Hydraulic Engineer with USACE MVK	Steering Committee member
Wesley Crosby	MVK	M.E., Civil Engineering	2 years Civil Engineer, Michael Baker International; 10 years Hydraulic Engineer, USACE	Hydraulic Technical Lead responsible for managing, coordinating and providing technical oversight of the hydraulic modeling on the Mississippi River among the three districts.

Name	Office	Discipline	Experience	Role in Preparing Report or Supporting Appendices
Coral Cruz	MVK	B.S., Civil Engineering	8 years Hydraulic Engineer, USACE MVK	Hydraulic Engineer responsible to calibrate MVK 2011 flood event and combined into a single hydraulic model the MVM, MVK, and MVN models.
Malcolm Dove	MVK	B.S., Civil Engineering; Masters Works in Water Resourses	10 years Hydraulic Design, 20 years Technical Review of Hydrologic design work in MVD	Hydraulic Engineer responsible to calibrate MVK 2011 flood event and combined into a single hydraulic model the MVM, MVK, and MVN models.
Phil Dye	MVK	B.S., Civil Engineering	30 years Hydraulic Engineer with the USACE MVK	District Review HEC-RAS
Ryan Hoben	MVK	M.E., Civil Engineering	10 years Hydraulic Engineer, USACE MVK	Funding for Hydraulic Modelers in MVK
Ann Banitt	MVP	B.S., Civil Engineering	15 years Hydraulic Engineer with the USACE St. Paul District	ATR team member for the Hydrologic and Hydraulic (HEC-RAS) modeling and reports.
John Schmidt	NWS SERFC	B.S., Civil and Environmental Engineering	9 years Development and Operations Hydrologist at NWS SERFC; 16 years Senior Hydrologist and Hydrometeorologist at the Arkansas-Red Basin River Forecast Center	ATR team member for Hydrologic modeling and report
Brian Cornwell	SAJ	B.S., Environmental Engineering M.Eng, Environmental Engineering	15 years Hydraulic Engineer with the USACE Jacksonville District	ATR team member for Hydraulic (HEC-RAS) modeling and report
Thomas Kirkeeng	MVR	B.S., Agricultural Engineering M.Eng, Agricultural Engineering	30 years Hydraulic Engineer with the USACE Rock Island District	ATR team member for Hydraulic (HEC-RAS) modeling and report
John Burant	MVR	B.S., Civil Engineering	25 years Hydraulic Engineer with the USACE Rock Island District	ATR team member for Hydraulic (HEC-RAS) modeling and report
Thoams Gambucci	MVR	B.S., Civil and Environmental Engineering M.Eng, Civil and Environmental Engineering	15 years Hydraulic Engineer with the USACE Rock Island District	ATR team member for Hydraulic (HEC-RAS) modeling and report

Name	Office	Discipline	Experience	Role in Preparing Report or Supporting Appendices
David Busse, PE	MVS	B.S., Civil Engineering; Masters of Engineering	35 years engineering experience. 25 years Hydraulic Engineer with USACE St. Louis District.	Steering Committee member
Don Duncan	MVS	B.S., M.S, Civil Engineering	10 years, Hydraulic Engineer; 5 years, Supervisory Hydraulic Engineer, USACE	Executive Steering Committee member for St. Louis; Consistency reviewer for Summary Report
Andy Lowe	LRL	B.S., Civil Engineering M.Eng, Civil Engineering	17 years Hydraulic Engineer with the USACE Louisville District	ATR Lead and ATR team member for the Hydrologic and Hydraulic (HEC-RAS) modeling and reports.
Dennis O. Norris	MVD	B.S., Engineering	30 years at Vicksburg District (all Operations experience on the Lower Miss); 7 years at MVD HQ (Operations experience on Upper, Middle, and Lower River)	Chief of Regional MR&T Team at MVD HQ providing general guidance and direction
Mike Turner	MVD	B.S., Civil Engineering	30 years, Design/Technical Services, USACE – Vicksburg District; 5 years, USACE MVD	Chief of Engineering and Construction at MVD providing guidance and direction to the study team
Joseph Windham, PE	MVD	B.S., Environmental Science; B.S. and M.S. Civil Engineering; Diplomate of Water Resource Engineering	12 years Hydraulic Engineering with USACE MVK; 2 years MVD	Chief Watershed Division, Mississippi Valley Division, providing guidance and direction to study team
Deanna Prestwood, PE	MVD	B.S., Civil Engineering	11 years, USACE MVK; 4 years, USACE MVD	Regional MR&T Program Manager, strategic communication plans, and POC for Briefs to MVD Sr. Staff and MRC
Jonathan Ashley	MVD	B.S. Civil Engineering; M.S. Civil Engineering	14 years as hydraulic engineer in Jacksonville District, Mobile District, Savannah District, and MVD, primarily in Water Control/Reservoir Regulation	MVD coordination, ATR coordination, Hydrology Report support
Brian Rentfro	MVD	B.A., History	7 years, Historian, USACE; 1 year Historian for Mississippi Valley Division and Mississippi River Commission	Authorization History
Thomas Brown, PE	MVD	B.S., Civil Engineering	23 years as Hydraulics Engineer at MVK, 5 years at MVD	Provided review and guidance as necessary to PDT and others

Name	Office	Discipline	Experience	Role in Preparing Report or Supporting Appendices
Frankie Griggs, PE	MVD	B.S., Civil Engineering	42 years as Hydraulic Engineer at MVK and MVD. Experience in H&H design, analysis, and water management.	Provided review and guidance as necessary to PDT and others
Ron Copeland	MVD	Ph.D., M.S., B.S. Civil Engineering	47 yrs Hydraulic Engineer, SPL, ERDC ,MVK, MVD	Hydraulic Engineer responsible for sedimentation modeling
Edmund Howe, P.E.	MVM (SWL as of Dec. 2016)	B.S., Civil Engineering; M.S. Civil Engineering	7 years, Hydraulic Engineer, USACE	Hydraulic Modeling of Mississippi River, Memphis Reach; assisted with combined MVD HEC-RAS model; developed scripts for post processing of model results
Sarah Girdner, E.I.	MVM	B.S., Civil Engineering; M.S. Civil Engineering	2 years, Hydraulic Engineer, USACE; 2 years, Environmental Engineer, Ensafe	Hydrologic model documentation and review comment resolution; developed climate change appendix for Hydrology report; and HEC-RAS model maintenance and review comment resolution for MVM reach
Robert Gambill, E.I.	MVM	B.S., Civil Engineering	8 years, Hydraulic Engineer, USACE	Hydraulic Modeling of Mississippi River, Memphis Reach, comment resolution for Hydraulics
Roger Gaines, Ph.D., P.E.	MVM	B.S., Civil Engineering, M.S. Civil Engineering, Ph.D. Civil Engineering	32 years, Hydraulic Engineer; 8 years Regional Technical Specialist for Hydraulics, Hydrology and Sedimentation, USACE	Technical lead for MVM; Technical Lead for Hydrologic analysis and NWS coordination for MVD; Assisted in MVM HEC-RAS model development; comment resolution for Hydrology
David Berretta, P.E.	MVM	B.S., Civil Engineering, M.S. Civil Engineering	43 years, Hydraulic Engineer; 14 years as H&H Branch Chief	Executive Steering Committee member for Memphis
Maxwell Agnew, E.I.	MVN	B.S., Civil Engineering; M.S., Civil Engineering	9 years, Hydraulic Engineer, USACE	Hydraulic modeling of Atchafalaya Basin
Matthew Dircksen, P.E.	MVN	B.S., Civil Engineering	8 years, Hydraulic Engineer, USACE	Hydraulic Modeling Support

Name	Office	Discipline	Experience	Role in Preparing Report or Supporting Appendices
David Ramirez, P.E., D.WRE	MVN	B.S., Civil Engineering; M.S., Civil Engineering	2 years, Construction Engineer; 14 years, Hydraulic Engineer; 6 years, Supervisory Hydraulic Engineer; USACE	Technical Lead for MVN flowline team
Ron Taylor, P.E.	MVN	B.S., Civil Engineering	2 Years, Construction Engineer; 13 years, Hydraulic Engineer, USACE	Hydraulic modeling of Mississippi River Basin
William Veatch	MVN	B.A., Environmental Studies; M.S., Hydrology	9 years, Hydrologist; 2 years, Regional Technical Specialist for Climate Change Adaptation, USACE	Climate change appendix and decision documents
Danielle Washington, P.E., CFM	MVN	B.S., Civil Engineering	6 years, Hydraulic Engineer; 4 years Lead Hydraulic Engineer, USACE	District quality control reviews
James Leech	ERDC	B.S., M.S. Civil Engineering	34 years, Research Hydraulic Engineer	Hysteresis analysis support
Ronnie Heath	ERDC	B.S., Civil Engineering	39 years, Research Hydraulic Engineer	Sediment model reviews and general team support
Jeremy Sharp, PE	ERDC	B.S., M.S. Civil and Environmental Engineering	10 years, Hydraulic Engineer at USACE and Mississippi State University	Sediment model review
Marielys Ramos-Villanueva	ERDC	B.S., Civil Engineering	5 years, Research Hydraulic Engineer	Sediment model review
Loren Wehmeyer	Consultant	Ph.D. Civil Engineering	10 years hydraulics and hydrology with USGS and USACE	Hydraulics report support
Dave Welch				

Appendix C: Project Decision Documents

MR&T Flowline Assessment

Decision Documents

An Executive Steering Committee (ESC) was formed at the beginning of the study to serve as technical advisors for this effort. As the study progressed, the technical team wrote up decision documents explaining rationale, assumptions, and forms of analyses for significant tasks that they were performed. There are a total of 12 Decision Documents that have been written and are available in this report. These include:

- Future Conditions, Sea Level Rise Impacts to Downstream Area for Mississippi River Flowline Model
- Assessing Effects of Land Subsidence on Potential Future Levee Deficiencies
- HYPO 58A Decision regarding use of straight sequence or clipped-merged sequence for constructing HYPO storms
- Future Condition Downstream Boundary for Mississippi River Flowline Model
- Decision regarding performing Unsteady HEC-RAS simulations of historic 1950's HYPO events 52A, 56, 58A, 58A-EN and 63
- Using MatLab scripts for processing HEC-RAS output data to aid in visualization and impact analysis
- HYPO 58A-EN Decision regarding use of NWS model results for Pickwick regulated outflows versus use of TVA modeled Pickwick regulated outflows
- Decision regarding loop effect that is not accounted for in the HEC-RAS Unsteady Hydraulic Model
- Additional HEC-RAS Runs – Optimization of Water Control Features of MRT System
- Decision regarding difference in Project Design Flood due to sediment accumulation
- Decision regarding the optimization of the backwater levee heights as related to loop effect and sedimentation
- Flow diagram correction for 1955 HYPO 58A-EN

The Executive Steering Committee for the MR&T Flowline Assessment has reviewed and concurs with the methods, rationale, and underlying assumptions employed as documented in the attached decision documents.

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Joey Windham
Chief, Watershed Division
CEMVD-RB-W

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Kent Parrish
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CEMVK-PP

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Robert Simrall
Director, MMC
CEMVK-EC-HC

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Leonard Hopkins
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CEMVS-EC-H

18 Apr 2017
Date

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Dave Berretta
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11 April 2017
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Mike Sorrels
Chief, H&H Branch
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4/9/2017

Malene Henville
Chief, H&H Branch
CEMVN-ED-H

Date

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Dave Busse
Chief, Engineering & Construction Division
CEMVS-EC

Date

Decision Document for MR&T Flowline Steering Committee
Future Conditions, Sea Level Rise Impacts to Downstream Area for Mississippi River Flowline Model

BLUF: The stage-discharge rating curve at Venice, LA, and the discharge-discharge rating curves for the river outlets at Bohemia, Fort St. Philip, and Baptiste Collette will be replaced with new curves to account for future sea-level rise during the period of analysis. These curves were formulated using the two-dimensional Adaptive Hydraulics (AdH) model of the lower river, adjacent outlets, and marshes using an elevated sea level representing future conditions 50 years in the future. While the exact amount of future sea level rise that will occur in the coming 50 years is uncertain, the AdH model boundary adjustment corresponds to sea level rise under the USACE high sea level rise scenario, in recognition of the value to the nation of MR&T project performance and the consequences of project failure. A sensitivity analysis will be performed to determine the importance of the choice of downstream boundary on the computed flowline.

Background: The MR&T project flood flowline for the Mississippi River is computed using HEC-RAS, a 1-dimensional unsteady hydraulic model. The choice of downstream boundary can exert significant influence on the resulting flowline, especially near the downstream boundary. Therefore, a careful consideration of downstream boundary is essential to the flowline study. The MR&T project has enormous value to the nation and its failure would have global repercussions, so a flowline that is robust to future system changes is a necessity.

The project flood flowline is recomputed after major flood events and accounts for future conditions, but neither “major flood” nor “future conditions” is well defined in USACE or MVD policy. The MR&T flowline steering committee has determined that future conditions refers to 50 years into the future. The decision is based on the goal that the MR&T system, if updated in accordance with this study’s results, should be able to handle a major flood occurring before the 50-year period expiration. In theory, before the expiration of the 50-year cycle, the flowline would be recomputed after the next major flood. As a note, the last flowline study was performed in 1973, 43 years ago.

USACE Engineer Regulation 1100-2-8162 “Incorporating Sea Level Change in Civil Works Programs” states that: “Potential relative sea level change must be considered in every USACE coastal activity as far inland as the extent of estimated tidal influence. Fluvial studies that include backwater profiling should also include potential relative sea level change in the starting water surface elevation for such profiles, where appropriate.” (par. 5b) The portion of the Mississippi River below the Bonnet Carré Spillway is subject to tidal influence, and the flowline study in this reach is a fluvial study that is essentially a study of backwater influence. (Above the Bonnet Carré Spillway, the backwater effect of the Gulf of Mexico is minimal in comparison to the effect of spillway operation. Within the spillway itself, sea level rise will have little effect due to the dropoff between the spillway and Lake Pontchartrain). ER 1100-2-8162 further states that "Decision makers should not presume that the future will follow any one of the SLC scenarios exactly. Instead, analyses should determine how the SLC scenarios affect risk levels and plan performance, and identify the design or operations and maintenance measures that could be implemented to minimize adverse consequences while maximizing beneficial effects." To identify how the range of potential future sea levels can affect risk levels and plan performance, a sensitivity analysis will be performed using all three USACE sea level scenarios. However, a single downstream boundary is required in order to generate a single project flood flowline. The MR&T project is not a modern, risk-based design but a deterministic design based on a single hypothetical event. A single downstream boundary condition is a necessary component of such a design. The USACE high rate sea level rise scenario was selected for this single condition in recognition of the enormous value of MR&T project performance and the potential consequences of its failure.

Method:

Model Structure and Assumptions:

The downstream boundary of the Mississippi River HEC-RAS model is not at the Gulf of Mexico, but rather on the Mississippi River at Venice, LA (Figure 1). Because Venice is upstream of the river/gulf interface, it will not experience the full sea level rise that will be observed in the ocean until sometime well after the period of analysis (this statement and this entire document refer to the high river flow conditions that are relevant to the flowline study). The dampening effect of sea level rise with distance upriver is demonstrated in Figure 2 (from Driessen and Van Ledden 2013).

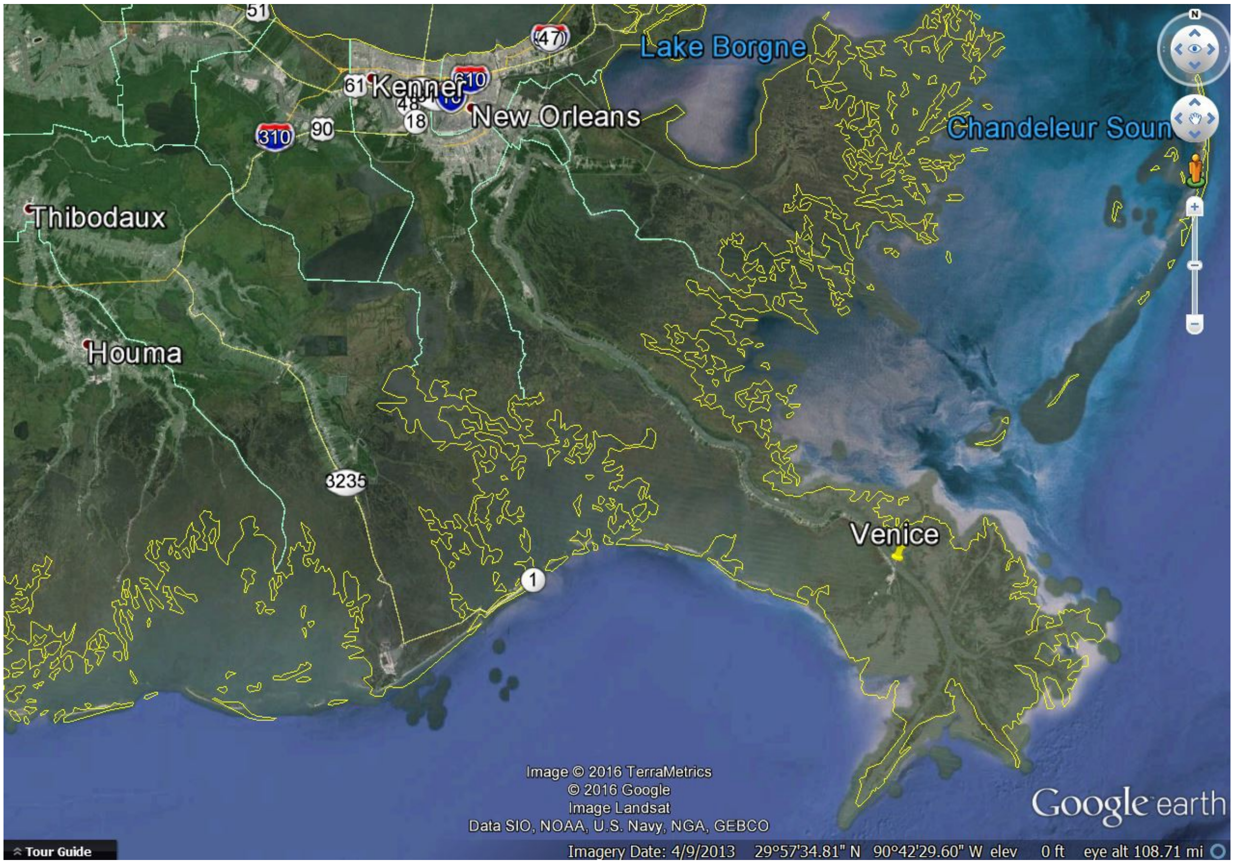


Figure 1: The model downstream boundary at Venice, LA is upstream of the Mississippi River mouth.

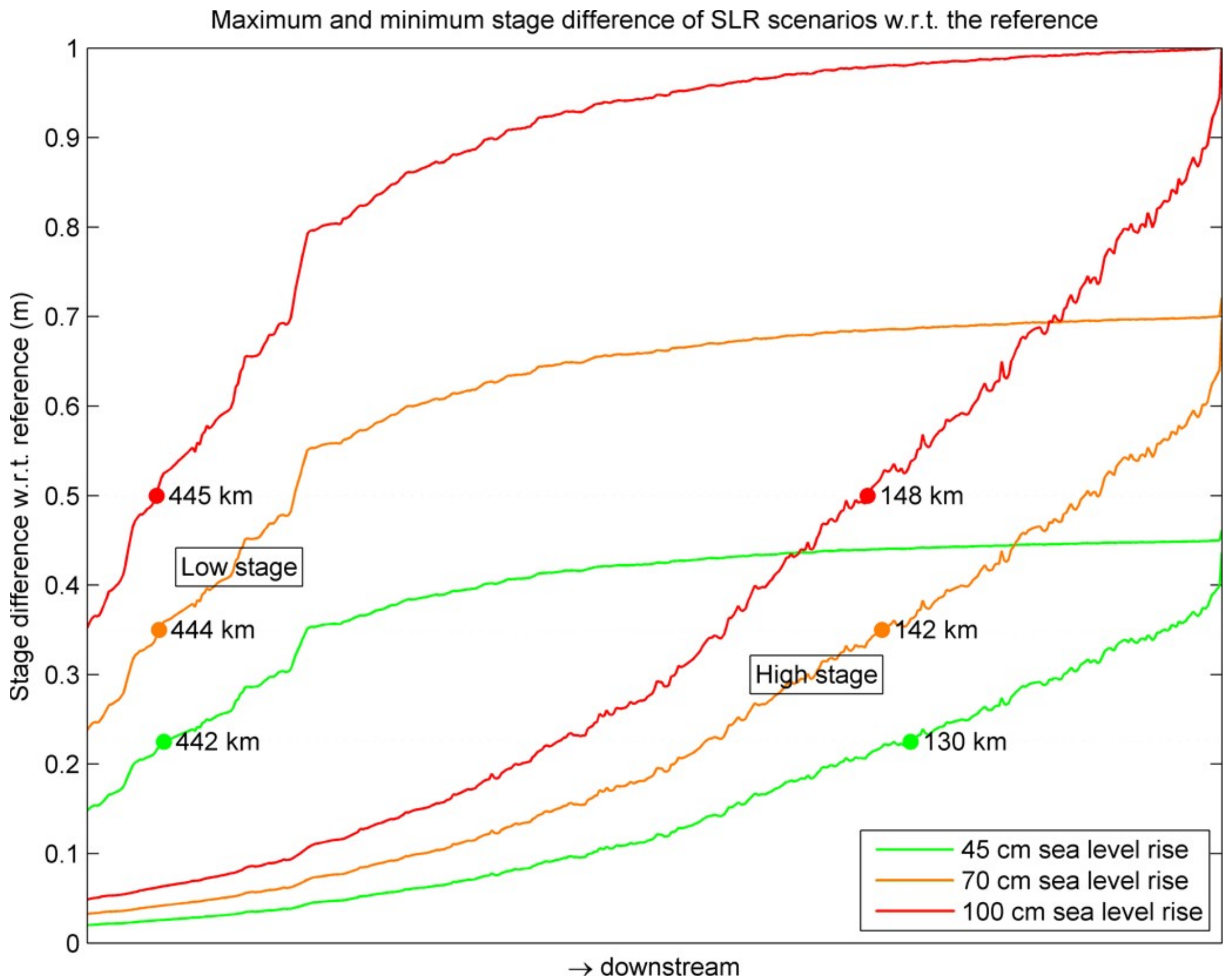


Figure 2: The effect of sea level rise is dampened with distance upstream from the river mouth, particularly when river discharge is high (Driessen and Van Ledden 2013).

While Venice is upstream of the mouth of the Mississippi River, there are three main outlets upstream of Venice that allow water to leave the river. These outlets are the Bohemia Spillway, the Fort St. Phillip outlet, and the Baptiste Collette Diversion, along with several smaller outlets. Fort St. Phillip, Baptiste Collette, and the smaller outlets generally always allow flow to leave the river, but the Bohemia spillway allows water to leave the river only when river discharge exceeds approximately 500k cfs upstream of Bohemia. The efficiency of these passes can be expected to change as the sea rises, due to the complex interaction between changing headwater and tailwater influences at each outlet. In the very long run, the outlets can be expected to become less efficient, as the tailwater stage in the receiving area (the gulf) will rise faster than the headwater stage in the river, which is subject to the dampening effect demonstrated in Figure 2. As a result, the total head across the outlet will eventually reduce to zero. In the short term, however, the outlets may temporarily become more efficient, as the increased headwater depth over the channel invert may be more significant than the increased backwater effect. The future conditions HEC-RAS model simulates the effect of these passes through the incorporation of and flow-flow rating curves, where the amount of flow subtracted from the river at each pass is a function of the flow in the river. These rating curves are generated to match observed discharges through each pass. Both the river and the outlets will be affected by sea level rise, but to different degrees.

The Mississippi River HEC-RAS model is an unsteady hydraulic model, so its downstream boundary is not a single stage, but rather a stage-discharge rating curve, based on observed data collected during the 2011 flood event. The choice of downstream boundary is therefore not a choice of a single stage, but a question of how to adjust this rating curve.

Sea Level Rise Computation and Assumptions:

In most studies, the sea level change that is relevant to the project is relative sea level rise, which is the combination of eustatic sea level rise attributable to increased ocean water volume and local relative land movement (subsidence or uplift). The project flood flowline is not affected by local land movement, because the flowline is computed relative to the terrestrial datum NAVD88, which does not subside (note that this assumption will only hold if the flowline is continually corrected for subsidence through readjustment to the latest NAVD88 epochs as they are issued; we assume that it will be). Therefore, in this case we are only concerned with future eustatic sea level rise (subsidence will be addressed in the impacts analysis, discussed later). In the next 50 years (i.e. by 2066), eustatic sea level rise is expected to be 2.4 ft. higher relative to present sea level (i.e. mean sea level in 1992, corresponding to the midpoint of the most recent National Tidal Datum Epoch, which spans from 1983-2001), using the high sea level rise scenario from ER 1100-2-8162 (par. B-4).

An existing AdH model was used to model a range of flows in this part of the RAS model domain in two dimensions, allowing the elevated sea level corresponding to future conditions to be applied at all edges of the model domain. The resulting stages (at Venice) and flows (at the outlets) corresponding to the range of flow in the river yielded rating curves, which can then be applied to the future conditions RAS model. Note that this approach does not incorporate any subsidence in the channel inverts of the outlets, or for that matter any change to river channel or overbank geometry at all, which is consistent with the approach used throughout the flowline effort. In reality, the topography and bathymetry of the outlets is likely to change in the future, but in such an unpredictable way that it would be extremely difficult to estimate for a future conditions simulation. Therefore, the existing conditions geometry was preserved.

Results:

For several reasons the RAS model and the AdH model do not generate exactly the same rating curves for Venice or the outlets under present conditions (though both tend to match observed data reasonably well, the RAS model since it was actually based on measured data, and the ADH model since it was calibrated to measured data). As a result, it would not be appropriate to use the rating curves directly from the AdH model for future conditions in RAS, since part of the difference between the present and future conditions would be due to model differences rather than differences between present and future conditions. Instead, an adjustment must be applied to the existing conditions RAS curves, informed by the difference between present and future conditions in the AdH model.

For the outlets, there are several reasons for the differences between the existing conditions curves in RAS vs AdH. First, for the RAS model, rating curves to divert flow were developed directly from measured data using an empirical process. Although the ADH model was calibrated to measured data, its development was more conceptual in its usage of geometrical data in determining flow rates. Because of these two different methodologies of development, there are slight inconsistencies in the amount of diverted flow. Inconsistencies also arise from the fact that the two models use different definitions of the delineation of each outlet (the exact edges where one outlet ends and the next begins are not well defined). For instance, in the RAS model, the Fort St. Phillip (FSP) curve included flow for the entire stretch of outlets between the Ostrica Lock and the Venice flow range (since the distribution of flow between FSP and the outlets based on flow measurements between Ostrica Lock and the Venice Range was unknown). For the ADH model, only FSP flow was included in the FSP curve. This is why the FSP flow is significantly higher in the RAS model. There are several other examples of similar inconsistencies between the two models. These differences in delineation among individual outlets do not substantially change the overall volume lost through all outlets.

For the Venice rating curve, another inconsistency explains the difference in the curves when comparing the models at low flows. The AdH model uses a base sea level of zero feet NAVD88 in the gulf, while the RAS model uses a Venice stage of 1.8 ft at very low river discharge, based on observations of past events. In reality, the “zero flow” stage at Venice is likely lower than this (Mean Sea Level at Pilottown, which is near Venice, is approximately 0.8 ft). However, this discrepancy has little impact on the flowline, because the flowline is defined for high river flows, not low flows. Another Venice discrepancy is a distribution that was applied to the RAS model so that the stages would taper off for the high MR flows. A similar distribution was not applied to the Adh model. The adjustments applied to the RAS future conditions factored this inconsistency into the curve.

Lastly, there are likely feedbacks among these effects that lead to nonlinearities, any of which would be hard to pinpoint.

If time allowed, the team could work through these differences and adjust the two models to give more similar results. However, the project schedule does not allow such investigations at this time, so an adjustment had to be devised to allow the RAS rating curves to be adapted based on the change over time in the AdH rating curves. After discussion among the modelers, the method chosen was to first shift the AdH curve to approximately match the RAS curve at very low river discharge. The proportional change in the AdH curve (future divided by present) was then determined. This proportional difference was then applied to the existing conditions

RAS curve to yield the future conditions RAS curve. The results can be seen in Figures 3 through 6.

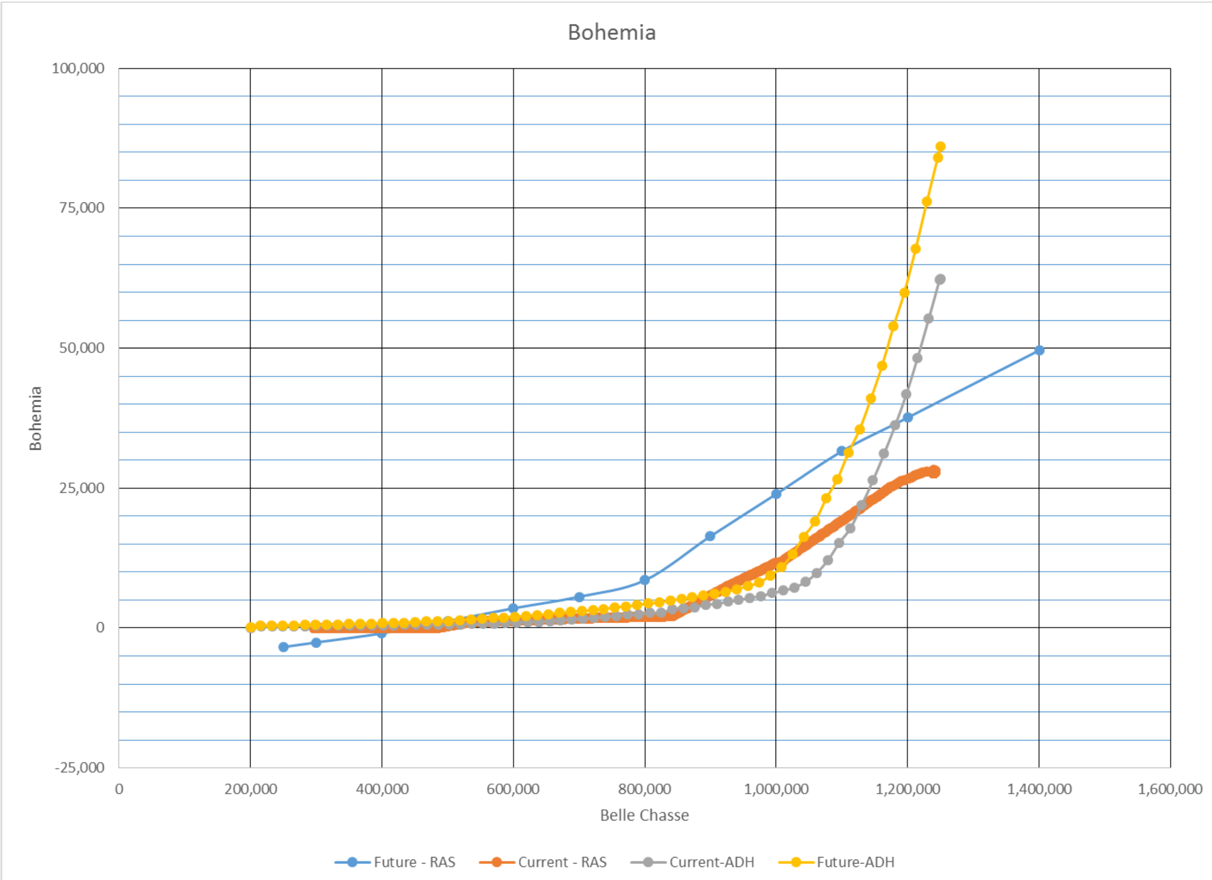


Figure 3: Existing and future conditions rating curves for the Bohemia Spillway. The blue curve is the final curve for use in the future conditions simulation in HEC-RAS.

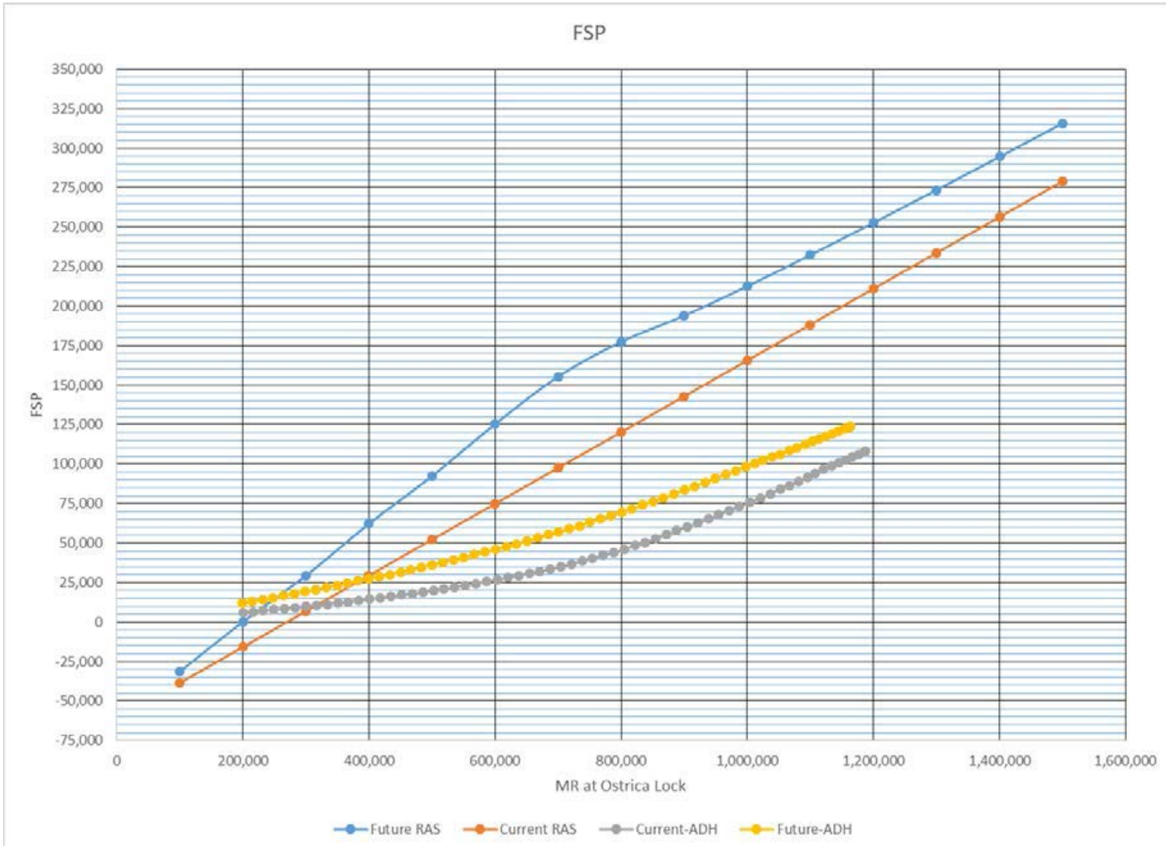


Figure 4: Existing and future conditions rating curves for the Fort St. Philip outlet. The blue curve is the final curve for use in the future conditions simulation in HEC-RAS.

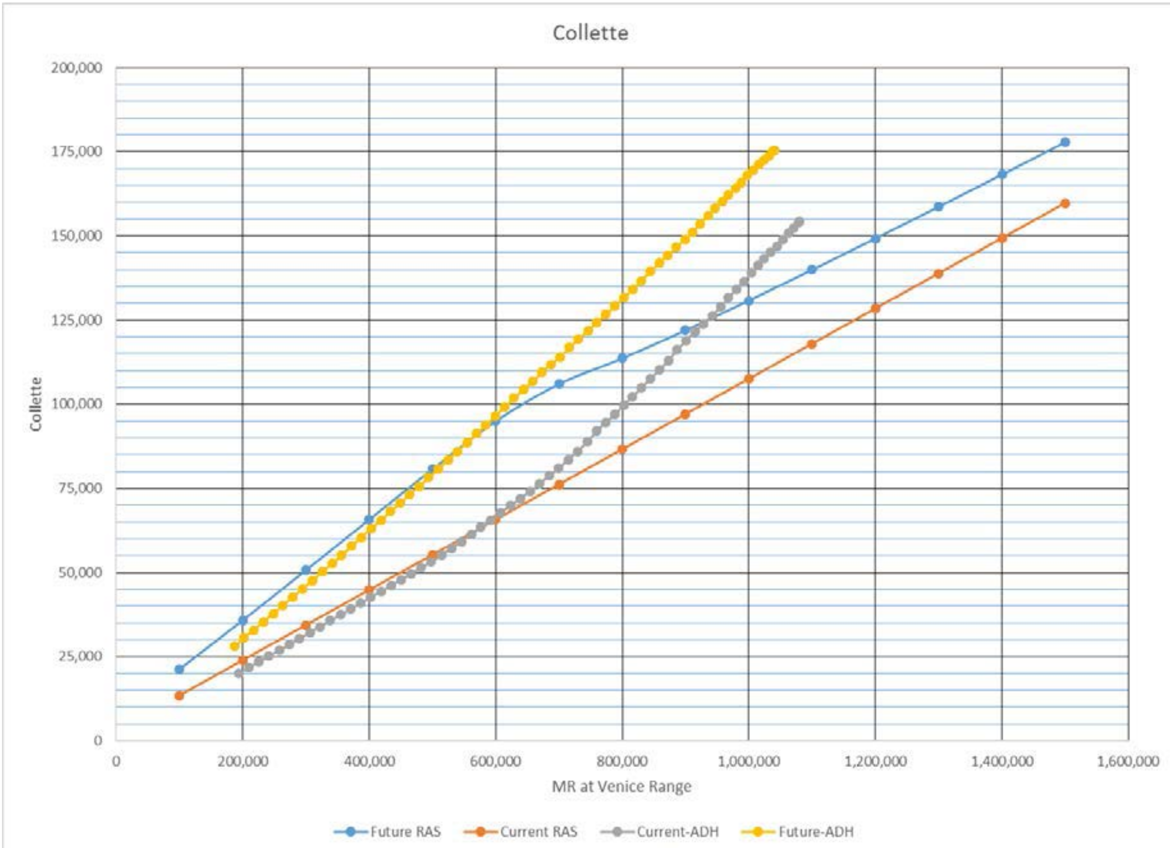


Figure 5: Existing and future conditions rating curves for the Baptiste Collette outlet. The blue curve is the final curve for use in the future conditions simulation in HEC-RAS.

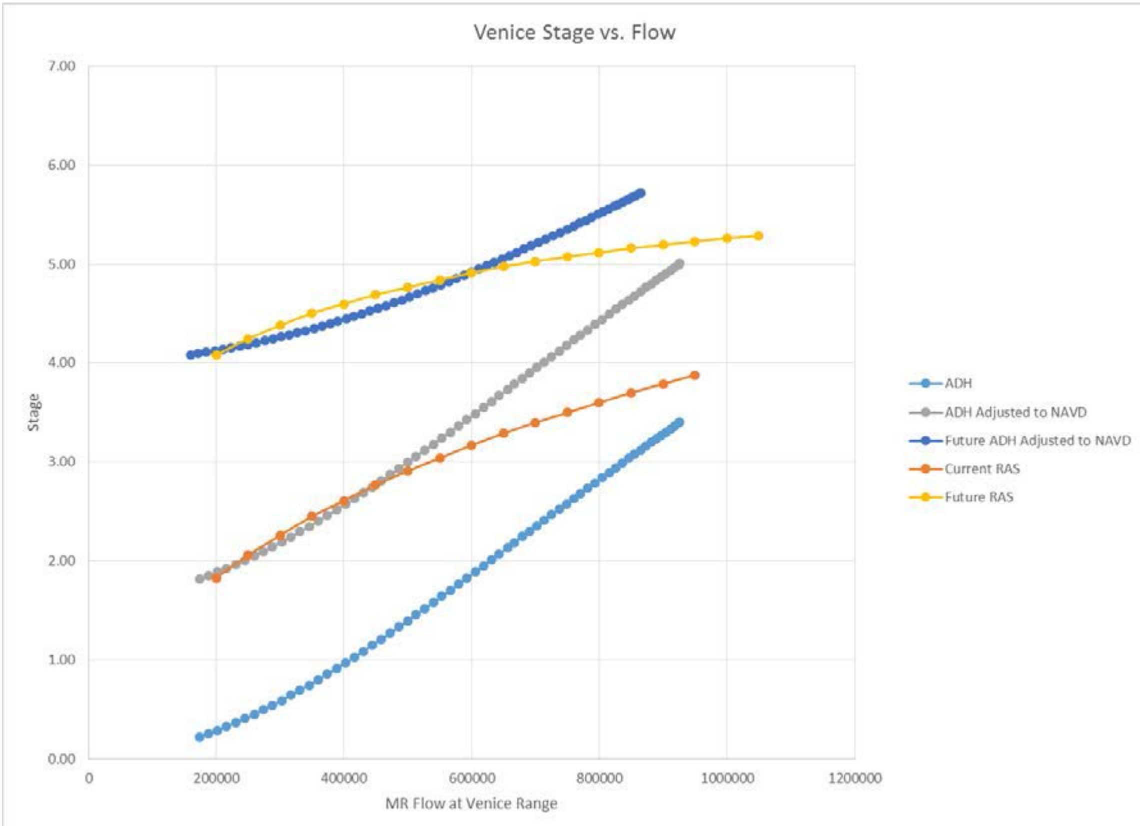


Figure 6: Existing and future conditions rating curves for the model downstream boundary at Venice. The yellow curve is the final curve for use in the future conditions simulation in HEC-RAS.

Conclusions:

All three USACE sea level rise scenarios will be assessed for their impact on the MR&T flowline, to analyze the sensitivity of the flowline to the downstream boundary condition. The flowline itself, being a deterministic rather than a probabilistic design criterion, requires a single downstream boundary. We propose to use the USACE high rate scenario for future sea level rise in recognition of the MR&T project's value and the consequences of its failure, over the timeframe of 50 years in accordance with the decision of the steering committee. Only eustatic sea level change was considered, because the flowline is computed and maintained relative to the geodetic datum NAVD88, which does not subside over time. The USACE high level scenario for eustatic sea level rise yields 2.4 feet of rise over the coming 50 years, so this amount of change will be used for the downstream boundary of the model at Venice and for the receiving areas of the major river outlets. The effects of this elevated sea level were simulated in AdH, allowing new rating curved to be generated for application in HEC-RAS.

This allowance for future condition sea level does not include the effects of tides, tropical storms, or wave effects.

Addendum: Analysis of Impacts of Flowline Update on Levee Deficiencies

As explained above, the MR&T flowline is computed relative to the geodetic datum NAVD88, which does not subside. Therefore, subsidence is not included in future conditions changed to the flowline. However, subsidence is an important process in the coastal area near the mouth of the Mississippi River. To account for the impacts of subsidence on levee deficiencies with respect to the flowline, an analysis will be performed projecting future subsidence along the river and identifying areas of expected future deficiency. This analysis is the subject of a separate decision document.

References:

Driessen, T. L. A., and M. van Ledden. "The large-scale impact of climate change to Mississippi flood hazard in New Orleans." Drinking Water Engineering and Science 6.2 (2013): 81-87.

Engineering Regulation 1100-2-8162, *Incorporating Sea Level Change in Civil Works Programs*, 31 Dec 2013.

Decision Document for MR&T Flowline Steering Committee
Assessing Effects of Land Subsidence on Potential Future Levee Deficiencies

BLUF: A projection of future levee deficiencies will be created using the prorated future conditions flowline and projections of land subsidence based on the rates documented in the Tide Gage Atlas of Coastal Louisiana. Future deficiencies will be presented in table and graphical forms, plotted against river mile for present conditions, 25 years, and 50 years into the future. These projected deficiencies will highlight the impacts of the updated flowline and focus levee construction and maintenance efforts.

Background: The MR&T project flood flowline includes anticipated future conditions, but land subsidence does not affect the flowline directly. This is because the flowline is computed relative to the geodetic datum NAVD88, which does not subside. As the land subsides around and the datum, it remains fixed in space for all practical purposes (the datum in fact does move in response to changes in the gravitational field of the Earth, but these changes are negligible over the temporal and spatial scales relevant to the MR&T project flood flowline). The flowline, being a water surface profile relative to the datum, therefore also remains fixed in space despite the movements of the surrounding landforms. While it is possible for subsidence to affect the flowline indirectly, for example by allowing overtopping of levees, overtopping and related processes are not considered in flowline computations.

Nevertheless, land subsidence is an important process within the MR&T project footprint, particularly in the coastal area near the mouth of the Mississippi River. Therefore, an assessment of the impacts of the flowline update on levee deficiencies should include the effects of subsidence. This paper proposes a method to compute and present those effects.

Method: The present conditions flowline will be used to assess levee deficiencies today (some levees are already deficient now) and to establish a baseline for comparison with future conditions. The future conditions flowline includes the effects of sea level rise and channel morphology changes. This future conditions flowline will be applied directly to the analysis of impacts 50 years into the future. The analysis of impacts 25 years into the future will use a linear proration of the difference between the two flowlines; i.e. the average of the two flowlines. Because sea level is expected to rise (slightly) exponentially with time, this linear interpolation may slightly overestimate deficiencies at 25 years. This uncertainty is small in comparison with the overall uncertainty around future flowline conditions.

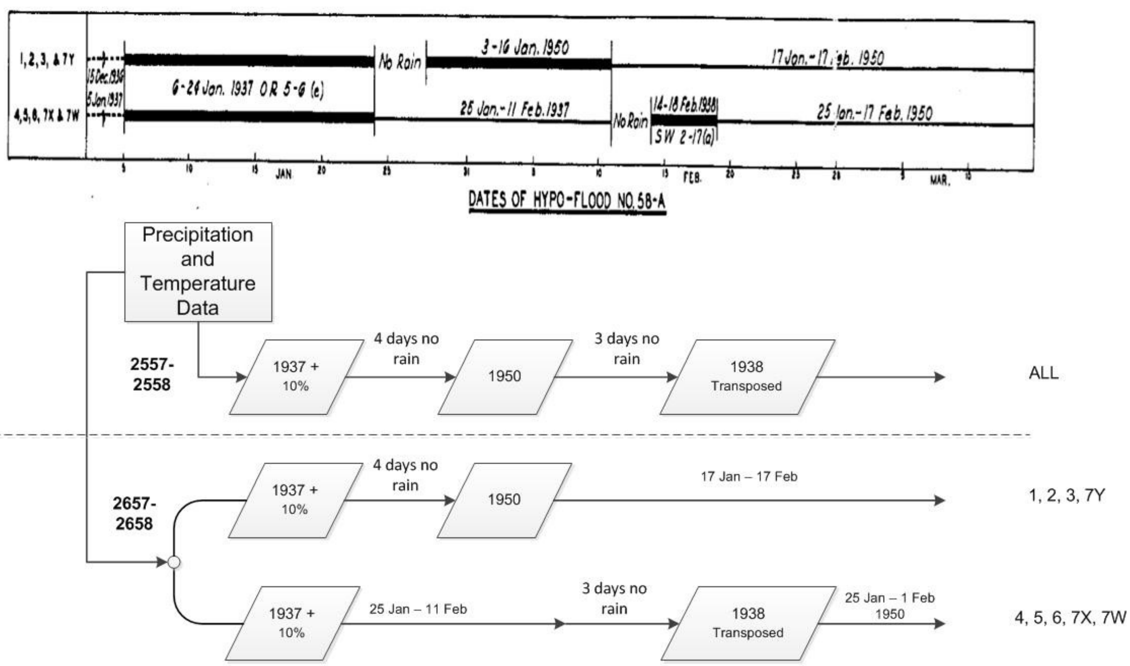
Subsidence will be projected by extending the rates documented in the Tide Gage Atlas of Coastal Louisiana, as updated in 2016. These rates include the entire periods of record for multiple gages along the Mississippi River, so they represent the best understanding of long-term subsidence. While there is some evidence that subsidence is not constant in time, these long-term rates still represent the best estimate of future subsidence, as there is not strong consensus that future subsidence will be either faster or slower than in the past. These rates will be presented in a Keyhole Markup Language (KML) file, modified slightly from one already prepared by ERDC-CHL personnel for the Mississippi River Hydrodynamics modeling effort. This file will be used with a script developed by MVM personnel, lowering levee heights to account for subsidence in the future conditions model. The resulting flowline and levee grades will be differenced to yield deficiencies plotted by river mile, showing where levees can be expected to become deficient in the future.

Schedule: Assuming the future conditions model run is complete by 13 January 2017 as presently scheduled, this analysis will be completed by the following Friday, 20 January 2017.

SUBJECT: Miss River Flowline Study: HYPO 58A Decision regarding use of straight sequence or clipped-merged sequence for constructing HYPO storms.

BLUF: Recommend using current methodology for modeling precipitation events for revised HYPO storm analysis. Current methodology applies continuous spatial event across all parts of the basin. Previous analysis conducted in early 1950's used different storm distributions across Miss River Basin.

Previous 1950's study applied different storm periods over different parts of the Mississippi River watershed to (1) maximize tributary effects and (2) to facilitate the myriad of manual calculations needed to develop peak design discharges. This is illustrated in the following graphic taken from the Memorandum Report No. 1 (1955). The numbers in the left of the bar diagram indicate sub-basins where the storm sequence is applied. The flow charts in the figure attempt to clarify how this is done. This approach results in discontinuities in rainfall amounts for adjacent sub basins.



. Two sequences defined as indicated.

> 2557-2558 dataset compiled for 01Jun2557 through 31Mar2558 with 6-Hour ASCII Grid files listed in order given in graphic above. This includes the pre- and post- event periods. Beginning 15Dec2557, reservoir simulation should be done in a standard forecast mode; other times can utilize observed data (from 1937 OBS period) where data exists but other reservoirs will require estimates of reservoir operations/releases.

> 2657-2658 dataset compiled for 01Jun2657 through 31Mar2658 with 6-Hour ASCII Grid files listed in order given in graphic above. This includes the pre- and post- event periods. Beginning 15Dec2657, reservoir simulation should be done in a standard forecast mode; other times can utilize observed data (from 1937 OBS period) where data exists but other reservoirs will require estimates of reservoir operations/releases.

. Two runs for each sequence given above: Unregulated and Regulated

For HYPO runs regulated simulations will include ALL reservoirs currently in the CHPS-FEWS models. This will be to represent the EN condition as defined for the Project Design Flood studies (EN is the scenario which included reservoirs that existed in early 1950s and those that had a reasonable expectation for being constructed by the 1970s).

Current methodology and modeling utilizes continuous simulation of precipitation as it moves across an entire watershed. Precipitation input for the NWS models is in the form of spatial grids for each time step (6 hour intervals for this study).

In order to compare the NWS model outputs with previous published hydrographs and peak flows a method was developed to attempt capturing the sequence as applied in the 1950's work. This method required clipping portions of rainfall from the different events then combining them into a new spatial grid—referred to as the clipped-merged sequence. The clipped portions were based on the sub-basins delineation used in the 1950 analysis—for example one set of storms was used to build precipitation inputs for sub-basins 1, 2, 3 and 7Y and another was used to build precipitation inputs for sub-basins 4, 5, 6, 7X and 7W.

The original plan was to run both straight and clipped-merged datasets in the NWS models to facilitate comparisons with the original published values. Because there are no historic hydrographs for the 52A, 56 and 63 HYPO results (only peak values) there is no way to compare those results.

The HYPO 58A simulations provide a way to compare the two sequences. Table 1 shows a comparison of HEC-RAS generated peak flows. It is noted that the straight sequence results in higher peak flows than are produced by the clipped-merged sequence. Both straight sequence and clipped-merged sequence peaks are higher than the historic peaks published for the HYPO 58A-EN (regulated). Values for the HYPO 58A (unregulated) aren't too far off published values (Table 2) for either the straight sequence or clipped-merge sequence results although the straight sequence seems to best match the historic values overall.

The entire process for modeling the Regulated (EN) condition involves 5 river forecast centers and 10 USACE Districts; quite an involved and lengthy process which impacts project schedules.

Due to schedule constraints only one sequence can be modeled at present. Additional simulations may be run at a later time.

It is recommended that the remaining HYPO storms (52A, 56 and 63) only use the current methodology which utilizes the straight sequence simulations.

Table 1 Comparison of peak flows from HEC-RAS for HYPO 58 storm simulations

Location	MVM HYPO 58A Unregulated StraightSeq	MVM HYPO 58A Unregulated ClipMerge	Percent Change (CM versus SS)		MVM HYPO 58A Regulated StraightSeq	MVM HYPO 58A Regulated ClipMerge	Percent Change (CM versus SS)
Tennessee River Kentucky Dam Outflow	756,612	745,754	(1.44)		617,490	588,482	(4.70)
Cumberland River, Barkley Dam Outflow	572,887	568,112	(0.83)		282,235	299,436	6.09
Mississippi, Chester	545,796	436,714	(19.99)		501,448	411,915	(17.85)
Mississippi, Cape Girardeau	557,228	462,561	(16.99)		513,010	438,256	(14.57)
Mississippi, Thebes	558,091	467,102	(16.30)		514,854	444,007	(13.76)
Ohio River, Cairo	2,455,702	2,393,775	(2.52)		2,331,371	2,284,077	(2.03)
Mississippi, MS; OH Confluence	2,933,661	2,788,938	(4.93)		2,793,557	2,686,816	(3.82)
Mississippi, Hickman	2,017,766	1,959,520	(2.89)		1,963,755	1,910,016	(2.74)
Mississippi, New Madrid	2,568,013	2,513,852	(2.11)		2,526,756	2,480,869	(1.82)
Mississippi, Tiptonville	2,890,285	2,748,430	(4.91)		2,790,620	2,675,487	(4.13)
Mississippi, Caruthersville	2,386,882	2,338,346	(2.03)		2,332,402	2,314,851	(0.75)
Mississippi, Osceola	2,835,932	2,676,379	(5.63)		2,748,706	2,588,631	(5.82)
Mississippi, Memphis	2,775,515	2,673,812	(3.66)		2,738,423	2,592,646	(5.32)
Mississippi, Tunica	2,787,888	2,675,164	(4.04)		2,744,738	2,592,903	(5.53)
Mississippi, Helena	2,726,695	2,624,352	(3.75)		2,696,977	2,560,172	(5.07)
Arkansas River, Dam 02	521,394	501,312	(3.85)		487,068	434,280	(10.84)
St. Francis, RM82.47	85,091	85,081	(0.01)		85,091	85,081	(0.01)
White River, Newport	445,481	356,029	(20.08)		314,634	209,809	(33.32)
White River, Clarendon	251,824	190,114	(24.51)		202,155	147,691	(26.94)

Table 2 Comparison of peak flows from new hydrology and historic published flows

Location	MVM HYPO 58A Unregulated StraightSeq	MVM HYPO 58A Unregulated ClipMerge	Historic Peak flow (Memorandum Report No. 1, 1955)
Mississippi, Chester	545,796	436,714	255,000
Ohio River, Cairo	2,455,702	2,393,775	2,460,000
Mississippi, MS; OH Confluence	2,933,661	2,788,938	2,860,000
Arkansas River, Dam 02	521,394	501,312	591,000
White River, Clarendon	251,824	190,114	309,000

Decision Document for MR&T Flowline Steering Committee
Future Condition Downstream Boundary for Mississippi River Flowline Model

BLUF: The HEC-RAS Flow Line model has a downstream boundary at Venice, LA. The rating curve at Venice, LA will be shifted upward by 1.1 ft. to account for future sea-level rise during the period of analysis. Additionally, the sea level in the receiving areas that represent the passes of the Mississippi River will be shifted upward by 2.4 ft. These shifts correspond to 50 years of sea level rise under the USACE high sea level rise scenario, in recognition of the value to the nation of MR&T project performance and the consequences of project failure. Additionally, a sensitivity analysis will be performed to determine the importance of the choice of downstream boundary on the computed flowline.

Background: The MR&T project flood flowline for the Mississippi River is computed using HEC-RAS, a 1-dimensional unsteady hydraulic model. The choice of downstream boundary can exert significant influence on the resulting flowline. Therefore, a careful consideration of downstream boundary is essential to the flowline study. The MR&T project has enormous value to the nation and its failure would have global repercussions, so a flowline that is robust to future system changes is a necessity.

The project flood flowline is recomputed after major flood events and accounts for future conditions, but neither major flood nor future conditions is well defined in USACE or MVD policy. The MR&T flowline steering committee has determined that future conditions refers to 50 years into the future, as it can be assumed that within that time another major flood will occur and therefore the flowline will be recomputed. The last flowline study was performed in 1973, 43 years ago.

USACE Engineer Regulation 1100-2-8162 "Incorporating Sea Level Change in Civil Works Programs" states that: "Potential relative sea level change must be considered in every USACE coastal activity as far inland as the extent of estimated tidal influence. Fluvial studies that include backwater profiling should also include potential relative sea level change in the starting water surface elevation for such profiles, where appropriate." (par. 5b) The portion of the Mississippi River below the Bonnet Carré Spillway is subject to tidal influence, and the flowline study in this reach is a fluvial study that is essentially a study of backwater influence (above the Bonnet Carré Spillway, the backwater effect of the Gulf of Mexico is minimal in comparison to the effect of spillway operation). ER 1100-2-8162 further states that "Decision makers should not presume that the future will follow any one of the SLC scenarios exactly. Instead, analyses should determine how the SLC scenarios affect risk levels and plan performance, and identify the design or operations and maintenance measures that could be implemented to minimize adverse consequences while maximizing beneficial effects." To identify how the range of potential future sea levels can affect risk levels and plan performance, and sensitivity analysis will be performed using all three USACE sea level scenarios. However, a single downstream boundary is required in order to generate a single project flood flowline. The MR&T project is not a modern, risk-based design but a deterministic design based on a single hypothetical event. A single downstream boundary condition is a necessary component of such a design. The USACE high rate sea level rise scenario was selected for this single condition in recognition of the enormous value of the MR&T project performance and the consequences of its failure.

Method to determine the Future Boundary Condition:

Model Structure and Assumptions:

The downstream boundary of the Mississippi River HEC-RAS model is not at the Gulf of Mexico, but rather on the Mississippi River at Venice, LA (Figure 1). Because Venice is upstream of the river/ocean interface, it will not experience the full sea level rise that will be observed in the ocean until sometime well after the period of analysis (this statement and this entire document refer to the high river flow conditions that are relevant to the flowline study). The dampening effect of sea level rise with distance upriver is demonstrated in Figure 2 (from Driessen and Van Ledden 2013).

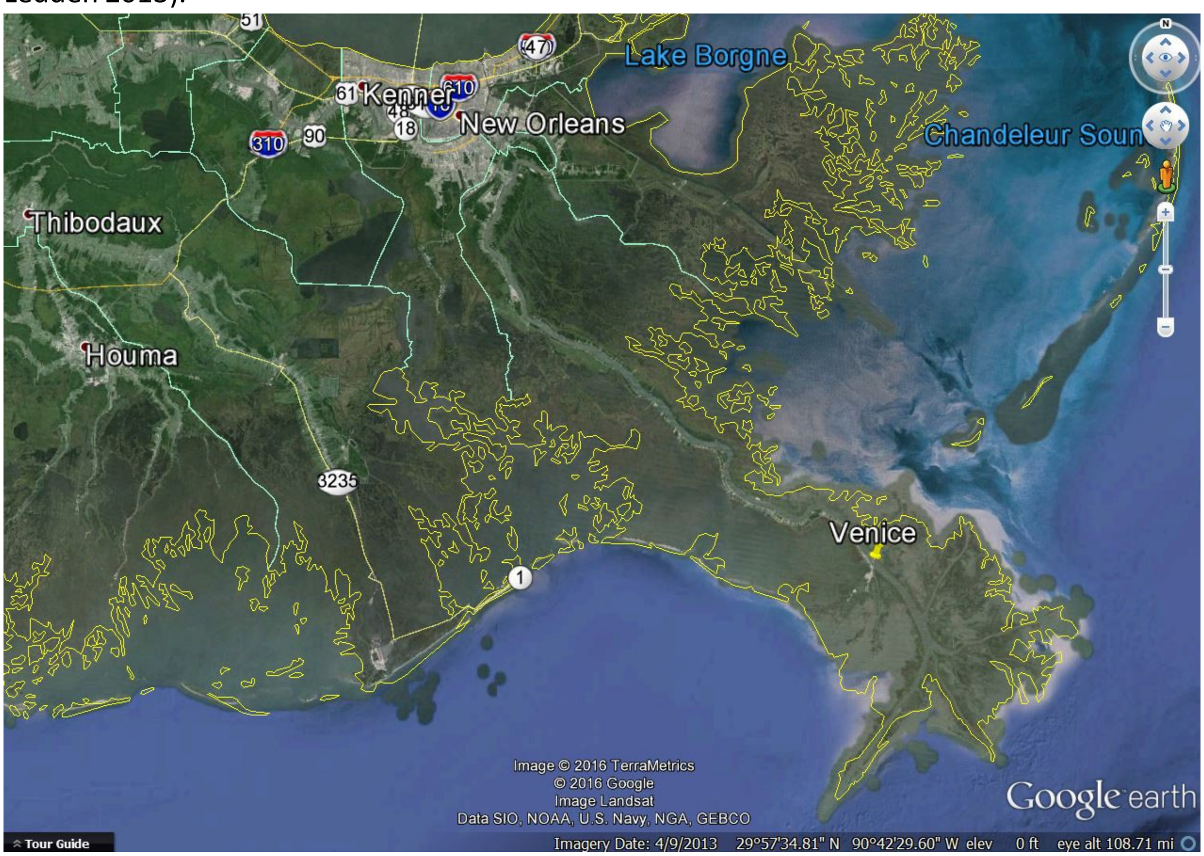


Figure 1: The model downstream boundary at Venice, LA is upstream of the Mississippi River mouth.

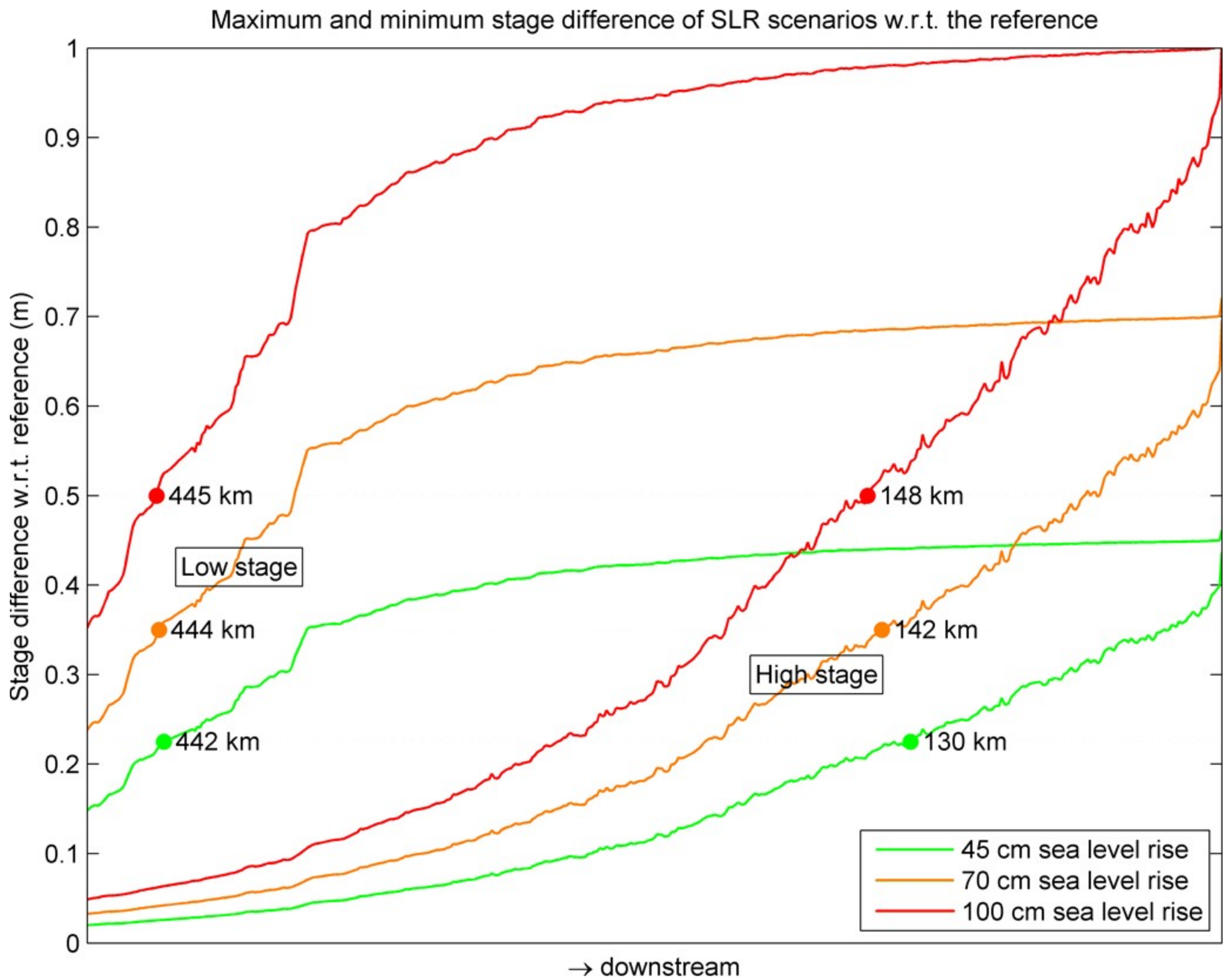


Figure 2: The effect of sea level rise is dampened with distance upstream from the river mouth, particularly when river discharge is high (Driessen and Van Ledden 2013).

While Venice is upstream of the mouth of the Mississippi River, there are passes upstream of Venice that allow water to leave the river during times of high river discharge. These passes can be expected to become less efficient as the sea rises, which will counteract the dampening effect shown in Figure 2 to some degree. The model simulates the effect of these passes through the incorporation of a "dummy reach" alongside the lower course of the river, where the stage in the dummy reach is equal to sea level in the Gulf of Mexico and connecting lateral weirs are used between the river reach and the dummy reach, the weir coefficients for which are calibrated based on the 2011 flood event. Both the river and the dummy reach receiving area will be affected by sea level rise, but to different degrees.

The Mississippi River HEC-RAS model is an unsteady hydraulic model, so its downstream boundary is not a single stage, but rather a stage-discharge rating curve, based on observed data collected during the 2011 flood event. The choice of downstream boundary is therefore not a choice of a single stage, but a question of how much to shift this rating curve. In the dummy reach, a single stage corresponding to future sea level is needed.

Sea Level Rise Computation and Assumptions:

In most studies, the sea level change that is relevant to the project is relative sea level rise, which is the combination of eustatic sea level rise attributable to increased ocean water volume and local relative land movement (subsidence or uplift). The project flood flowline is not affected by local land movement, because the flowline is computed relative to the terrestrial datum NAVD88, which does not subside (note that this assumption will only hold if the flowline is continually corrected for subsidence through readjustment to the latest NAVD88 epochs as they are issued; we assume that it will be). Therefore, in this case we are only concerned with future eustatic sea level rise. In the next 50 years (i.e. by 2066), eustatic sea level rise is expected to be 2.4 ft. relative to present sea level (i.e. mean sea level in 1992, corresponding to the midpoint of the most recent National Tidal Datum Epoch, which spans from 1983-2001), using the high sea level rise scenario from ER 1100-2-8162 (par. B-4).

A stage-stage correlation was used to assess the influence of Gulf of Mexico stage on river stages at Venice. The Southwest Pass at East Jetty (01670) gage was used to represent stages in the Gulf of Mexico near the mouth of the Mississippi River. A stage-discharge plot was created to assess whether this gage is influenced by river discharge (Figure 3). The results show that while there appears to be some influence, approximately 5% of the variance in stage can be attributed to Mississippi River discharge, while an increase in discharge of one million cfs would be expected to raise stages by only 1.5 ft. For an analysis restricted to high flow conditions, this was deemed to be an effectively insignificant influence.

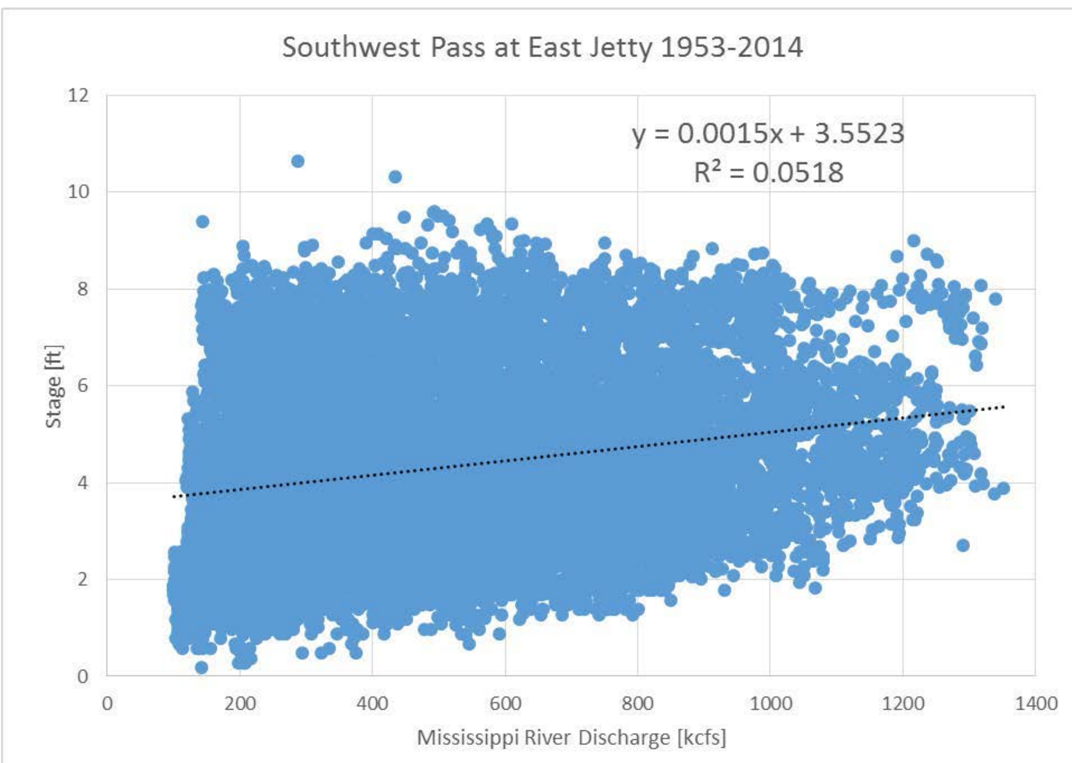


Figure 3: Influence of Mississippi River discharge on stage at Southwest Pass is minimal, particularly at high flows

The stage correlation between Southwest Pass and Venice was restricted to the time period between 1954 (when daily data records began for both gages) and June of 2015, when the gages were shifted to NAVD88 datum. Additionally, the analysis was restricted to times when the Mississippi River discharge (measured at Tarbert Landing) was above 1.25 million cfs, which corresponds roughly to the conditions that create project design flood conditions below the Bonnet Carré Spillway. The analysis was performed using raw stage data from each gage, which includes the effects of gage shifts that were formerly performed in an attempt to counteract the effects of subsidence (this practice has since been discontinued). These shifts were not necessarily performed simultaneously at each gage, however, so another analysis was performed in which the gage shifts were removed to create a continuous record in a consistent datum. This approach has the advantage of consistency in the reference plane but includes historical relative sea level rise at the two gages, which are not equal in rate. The results of both analyses are shown in Figure 4, and indicate that for every increment of stage change at Southwest Pass, stages at Venice change by 43-51% of that increment (during periods of high river flow).

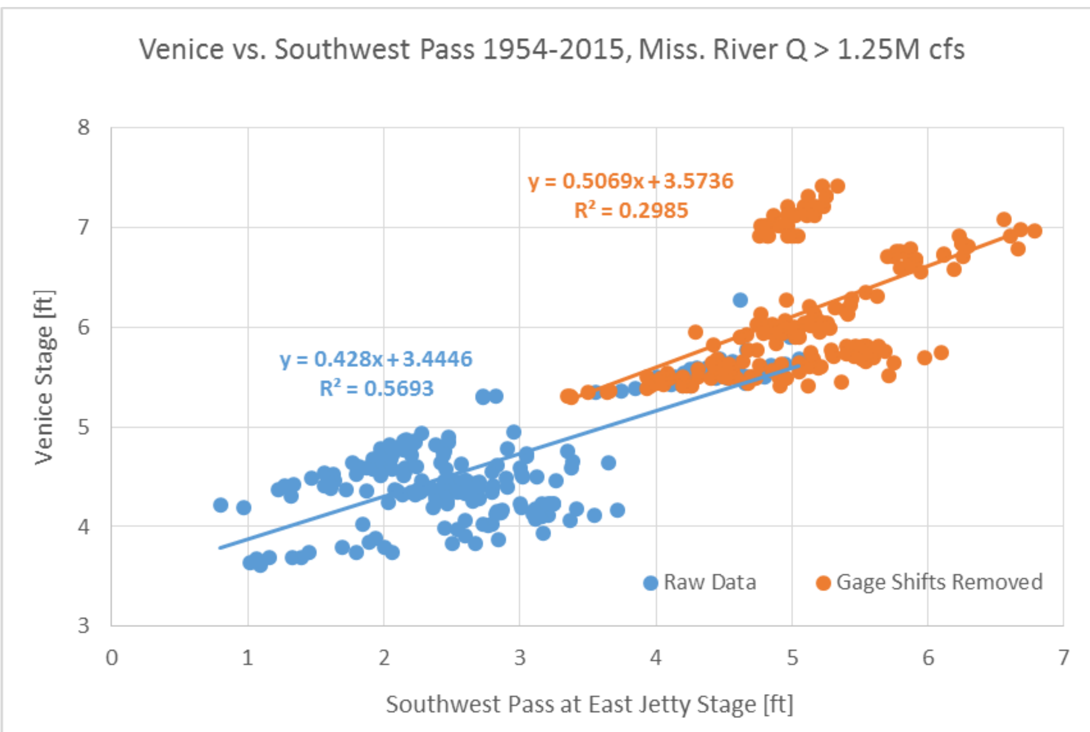


Figure 4: Venice stage vs. Southwest Pass stage during high river flow conditions. The slope of the regression lines show that for every foot of change in stage at Southwest Pass, stages in Venice change between 0.43-0.51 feet.

Using 45% as a middle figure, we can expect stages to rise at Venice approximately 45% of 2.4 feet, or 1.1 feet, over the next 50 years. Therefore we propose to shift the stage-discharge rating at Venice upward 1.1 ft. to account for future conditions. The mean sea level in the dummy reach used to represent the Gulf of Mexico will increase by 2.4 ft. in accordance with the full sea level rise envisioned in the USACE high rate scenario.

Methodology to Apply the Future Condition Downstream Boundary:

The computation of the estimated sea level rise that will likely occur in the Mississippi River at Venice, LA will be used to adjust the existing conditions downstream boundary of the Mississippi River Flow Line model by 1.1 feet. This magnitude of water level increase is to account for the estimated eustatic sea level rise that will occur during the 50 year life of this Flow Line. This adjusted downstream boundary condition will only be used for the future conditions Flow Line model simulations. The downstream model boundary for the existing conditions model simulation will be the current rating curve at Venice, LA, which was developed using historic stage and flow data.

Conclusions:

All three USACE sea level rise scenarios will be assessed for their impact on the MR&T flowline, to analyze the sensitivity of the flowline to the downstream boundary condition. The flowline itself, being a deterministic rather than a probabilistic design criterion, requires a single downstream boundary. We propose to use the USACE high rate scenario for future sea level rise in recognition of the MR&T project's value and the consequences of its failure, over the timeframe of 50 years in accordance with the decision of the steering committee. Only eustatic sea level change was considered, because the flowline is computed and maintained relative to the geodetic datum NAVD88, which does not subside over time. The USACE high level scenario for eustatic sea level rise yields 2.4 feet of rise over the coming 50 years, so this amount of change will be added to the mean sea level in the dummy reach used in the model to represent the receiving area in the Gulf of Mexico. In the river itself under high flow conditions, we found that stages tend to change approximately 45% of the magnitude of change observed in the Gulf, so we propose to use 45% of 2.4 feet, or 1.1 feet, as the future sea level change there. This change will be applied as a shift to the stage-discharge rating curve that serves as the model's downstream boundary condition. This allowance for future condition sea level does not include the effects of tides, tropical storms, or wave effects.

The process to determine the final Flow Line will consist of many model simulations investigating water elevations, channel geomorphology, climate change (sea-level rise and hydrology changes), as well as complex river hydraulics such as loop effects. Some of these parameters will have to be investigated using separate tools and models. The results of these investigations will be combined to complete the composite final Mississippi River Flow Line.

SUBJECT: Miss River Flowline Study: Decision regarding performing Unsteady HEC-RAS simulations of historic 1950's HYPO events 52A, 56, 58A, 58A-EN and 63.

BLUF: There is not sufficient data to run unsteady HEC-RAS for the historic 1950's HYPO events 52A, 56, and 63 and there is no benefit in running the historic 1950's HYPO 58A event.

During a conference call held on January 27, 2016 with the Flow Line RAS team, MVD, Charlie Mckinnie, and Andy Gaines, we discussed whether there was a need to run the Historic HYPO floods 52A, 56, 58A and 63 using our unsteady HEC-RAS model. It was pointed out that some of the Historic floods (i.e. 52A, 56 and 63) would be difficult to run as they did not have daily flow hydrographs like the 58A and 58A-EN floods and would have nothing to check against if the daily flows were now computed; however, we can compare the hydrology between the historic HYPO events and the new hydrology being jointly developed by the NWS and USACE. The method used for the original study for these events consisted of asking each district (LRD, SWL, etc) for a percent reduction based on reservoirs. There are not any "regulated" hydrographs for tributary inputs that we are aware of and the only data published was the unregulated for the tributaries. The Mississippi River flow was just reduced below that particular point by that percentage. As for the 58A HYPO event, the flood was selected to be used as the basis for the Project Design Flood (PDF) for the 1955 study and it does have daily flow hydrographs. Since the approved PDF used the 58A-EN (i.e. 58A event which is regulated with existing reservoirs and those to be constructed by 1970), it was concluded that we should only run the historic 58A-EN flood with the combined Districts HEC-RAS model for comparison of the historic flood routing to the approved 1955 Division model. The comparison of the two models will indicate the degree of confidence we have in using the new HEC-RAS model to route the new flood flows for the 52A, 56, 58A, 58A-EN, and 63 that NWS will furnish for the MR&T Levee Design Project. Specific runs to be made during the current study include the following.

Historic: 1955 HYPO 58A-EN; flow routing with water surface profiles

Updated 2016: 2016 HYPO 58A-EN; flow routing with water surface profiles
2016 HYPO 56-EN; flow routing with water surface profiles
2016 HYPO 52A-EN; flow routing with water surface profiles
2016 HYPO 63-EN; flow routing with water surface profiles
2016 HYPO 58A; flow routing only, no profiles
2016 HYPO 56; flow routing only, no profiles
2016 HYPO 52A; flow routing only, no profiles
2016 HYPO 63; flow routing only, no profiles

<u>It is recommended that only the 1950's historic HYPO 58A-EN be run using the Unsteady HEC-RAS model. The other 1950's historic HYPO events (i.e. 52A, 56, 58A, and 63) should only be compared with the new hydrology being jointly developed by NWS and USACE.</u>

SUBJECT: Miss River Flowline Study: Using MatLab scripts for processing HEC-RAS output data to aid in visualization and impact analysis.

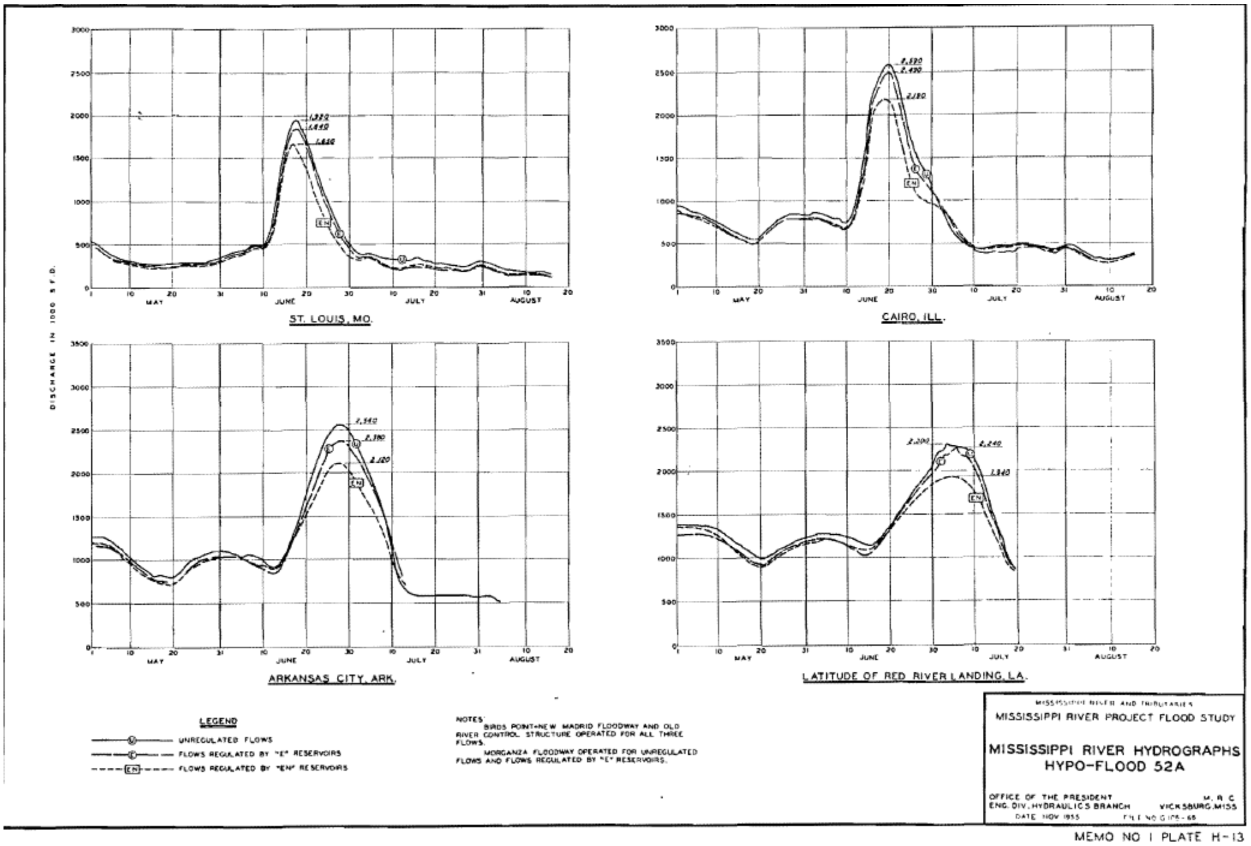
BLUF: Recommend using scripts to do hydrograph plots and tables for key locations on the HEC-RAS model results. Recommend using scripts to develop river profile plots from the HEC-RAS model results. Recommend using scripts to develop levee profile plots and/or tables describing potential deficiencies due to existing levee heights or existing water control structure operations. The scripts will be more expedient and consistent for all three lower MVD districts than traditional plotting techniques provided that all the necessary inputs are provided to the Memphis District within a reasonable time. The examples provided in this document are based on preliminary datasets and are not considered finalized. Additional input from the study team is necessary to refine the scripts to maximize utilization for everyone's needs. A detailed presentation at the Executive Steering Committee Meeting on how the scripts work & their capabilities and limitations will be given.

Hydrograph Script:

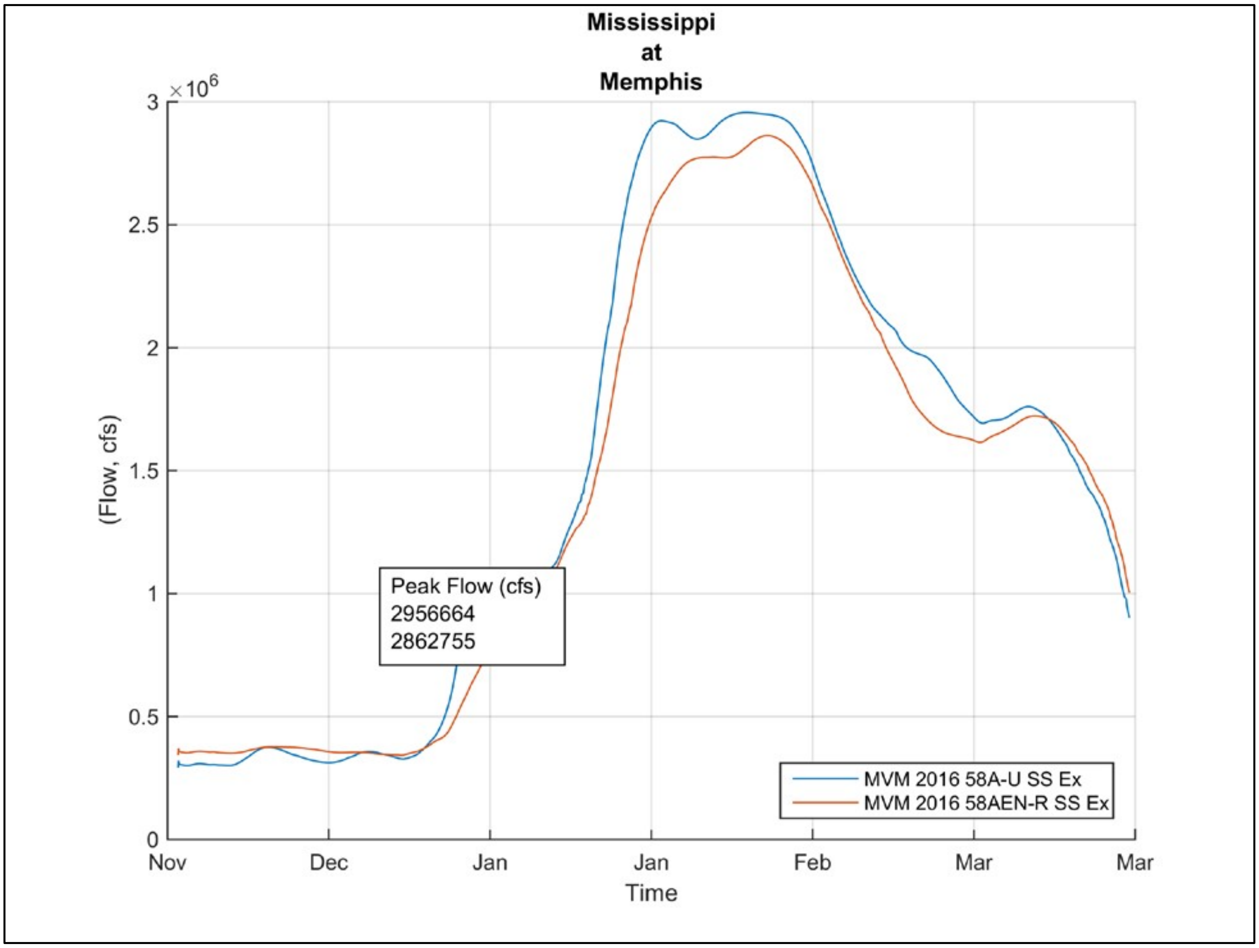
Previous 1950's study had fewer hydrograph locations plotted – especially on the mainstem Mississippi. The intent of this study is to document in as much detail as possible and prudent. A MatLab script is already available that can read the HEC-RAS results and plot comparison hydrographs for as many cross section locations as desired within the HEC-RAS model for all the different HYPO simulations. It is understood that various other plots may be necessary to document more specific features of the study and that those can be generated by each District as needed to fully describe the study results.

The MatLab script was developed by the Memphis District, and it is the recommendation that the Memphis District run the script to produce plots equivalent in nature (with additional locations) to the figure below. Due to the strict timeline of the study and the fact that more complex decisions for the modeling are necessary by the Vicksburg and New Orleans Districts, having the Memphis District run the script for some of the plots at key locations as the simulations are ongoing will help expedite the documentation process due to the large amount of data to be processed.

The team will have input on the scaling, format, and presentation of the plots for the final document. Because the plotting is in a script format, many changes in the presentation can be modified and re-plotted in a much shorter timeframe than other, more manual plotting techniques.



Historic Hydrograph Example



Script Output Hydrograph Example

River	Reach	Label
Tennessee River	Tennessee River	Downstream of Kentucky Dam
Cumberland River	Cumberland River	Downstream of Barkley Dam
Mississippi	Upper Miss	Chester
Mississippi	Below Big Muddy	Cape Girardeau
Mississippi	Below Big Muddy	Thebes
Ohio River	OHS	Cairo
Ohio River	OHS	Metropolis
Mississippi	Below Cairo	MS_OH_Confluence
Mississippi	Below Cairo	Hickman
Mississippi	Below Cairo	New Madrid
Mississippi	Below Cairo	Tiptonville
Mississippi	Below Cairo	Caruthersville
Mississippi	Below Obion	Osce ola
Mississippi	Below Wolf	Memphis
Mississippi	Below Nonconnah	Tunica
Mississippi	Below St. Franc	Helena
Mississippi	Below Arkansas	Arkansas City
Arkansas River	Arkansas River	Dam 02
White River	White River	Newport
White River	White River	Clarendon
White River	White River	Mouth of White River
Black River	R3	Acme
YazooRiver	Reach2	Redwood
Mississippi	Below White	Rosedale
Mississippi	Below Arkansas	Greenville
Mississippi	Below Vicksburg	Lake Providence
Mississippi	Below Vicksburg	Vicksburg
Mississippi	Below Vicksburg	St. Joseph
Mississippi	Below Vicksburg	Natchez
Mississippi	Below Vicksburg	MVN Boundary
Mississippi	Below Vicksburg	Oldr River Complex
Mississippi	Below Vicksburg	Old River Lock
Mississippi	Below Vicksburg	Red River Landing
Mississippi	Below Vicksburg	Morganza
Mississippi	Below Vicksburg	Bayou Sara
Mississippi	Below Vicksburg	Baton Rouge
Mississippi	Below Vicksburg	Donaldsonville
Mississippi	Below Vicksburg	Bonnet Carre
Mississippi	Below Vicksburg	Carrollton
Mississippi	Below Vicksburg	Empire
Mississippi	Below Vicksburg	Venice

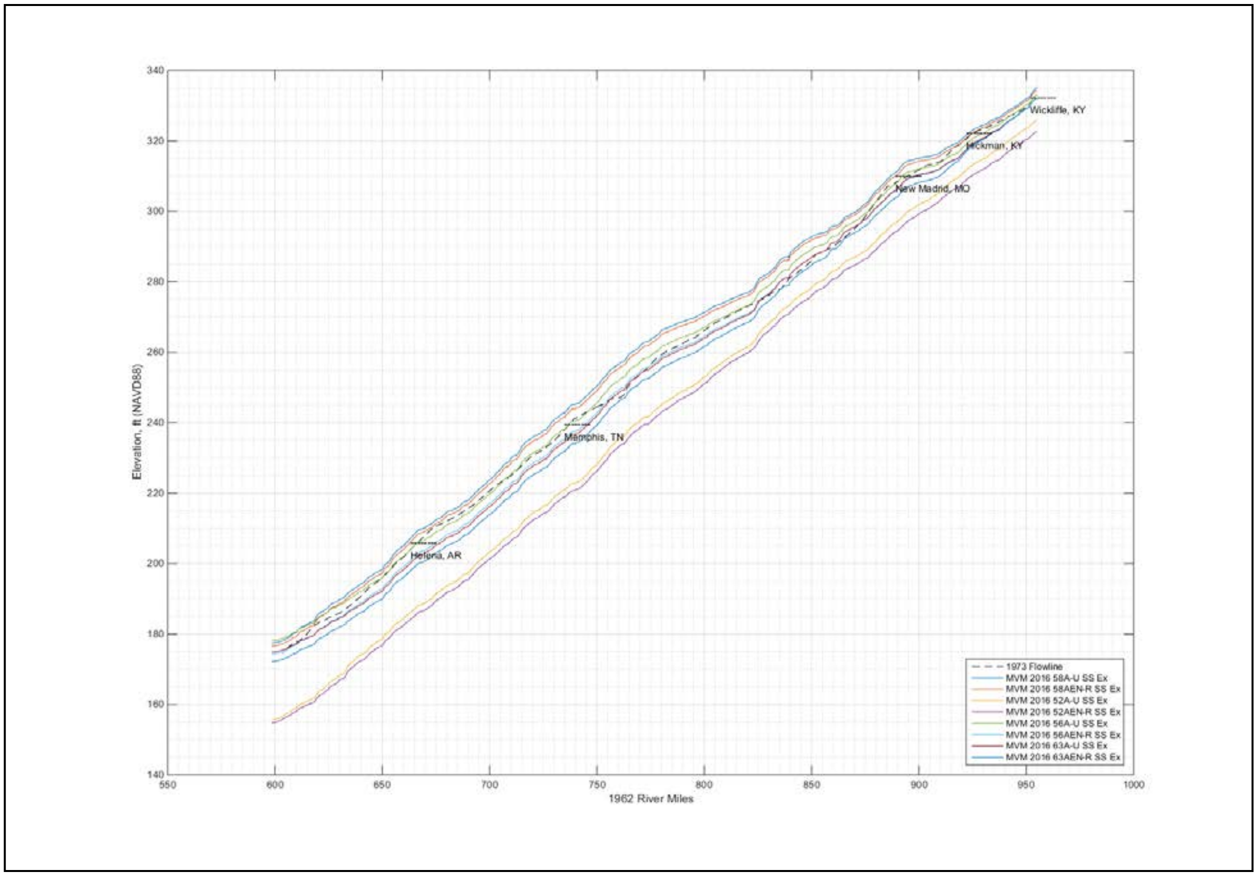
Current Key Locations Identified

<u>It is recommended that a script be used for plotting many key locations in the HEC-RAS model to help expedite documentation and maintain uniformity in all three Districts.</u>

<u>River Profile Script:</u>

Besides a tabulated comparison of the historic hydrology 1955 HYPO flood flows to the 2016 HYPO flood flows, a river stage profile comparison is also necessary. The flowline from the 1970s was based on the same historic hydrology 1955 HYPO flood flows; however, the flowline from the 1970s has different datums, river miles, and multiple components that differ from the newest HEC-RAS model. The flowline from the 1970s was in NGVD29 and the 2016 data is in NAVD88. The flowline from the 1970s used the 1962 Navigation River Miles and the 2016 data uses a 2011 thalweg alignment to compute river miles. The flowline from the 1970s has a raw computational component as well as an additional variable(s) component. The additional variable(s) were added to the computational flowline to take into account various items not fully, or adequately captured in the steady-state modeling. These items included, but are not limited to: general freeboard, loop-effect, wind, and waves. While the river profile script can easily include the additional items, much discussion will still need to take place in determining the magnitude and locations of these various types of additional considerations.

With an adequate shapefile depicting the continuous alignment for the 1962 River Miles, a tabulation of river mile and elevation for the 1970s flowline, and the HEC-RAS output files, the script (combined with CorpsCon6) can convert datums and river miles to be consistent by reporting elevations in NAVD88 and miles in 1962 River Miles. Below is example output of the script showing how it can plot elevations in NAVD88, 1962 River Miles for a tabulated 1970s flowline, and multiple HEC-RAS output plans. The team will have input on the scaling, format, and presentation of the plots for the final document. Because the plotting is in a script format, many changes in the presentation can be modified and re-plotted in a much shorter timeframe than other, more manual plotting techniques.



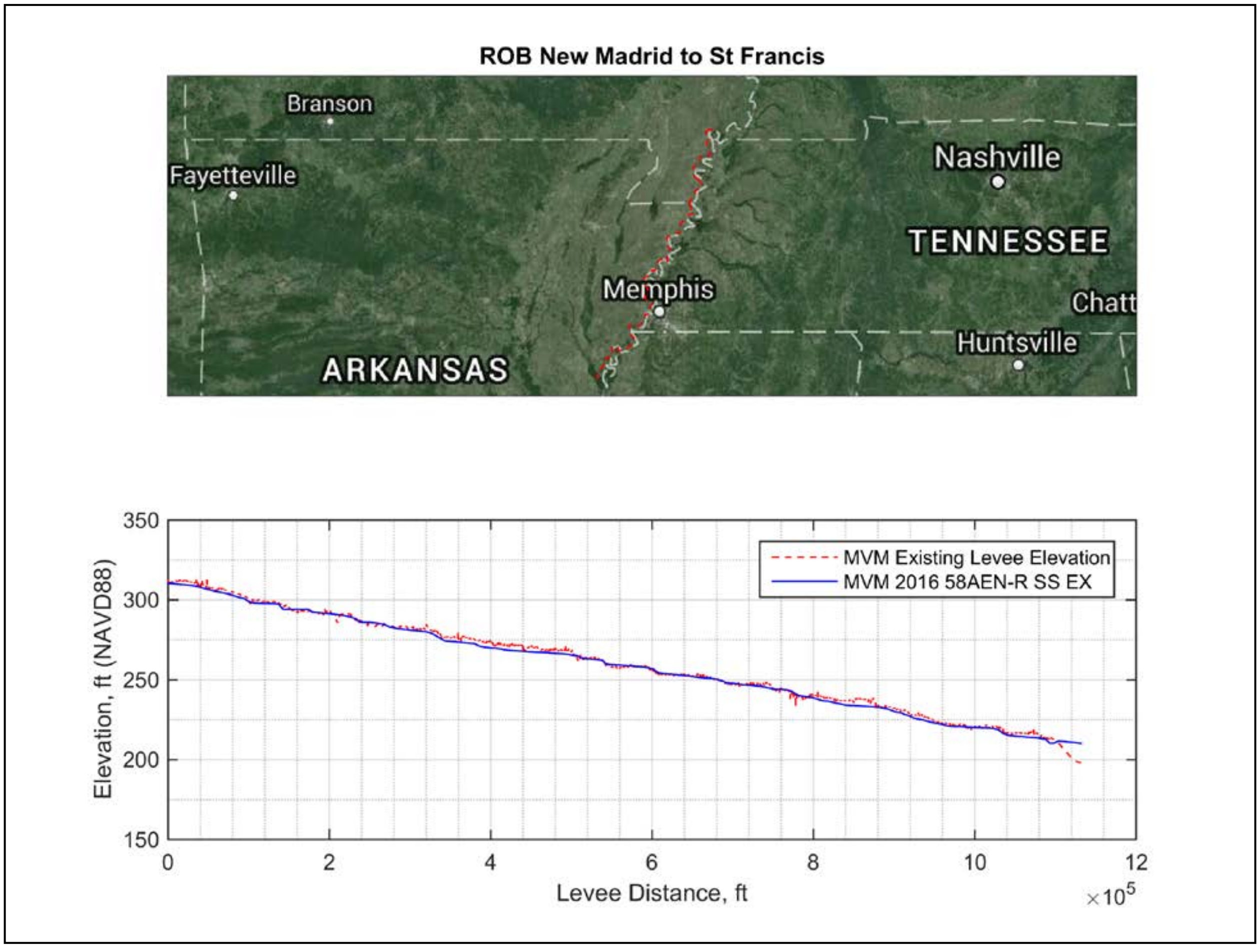
River Profile Example Plot

It is recommended that a script be used for plotting river profile comparisons of the historic flowline to the preliminary 2016 computations for either documentation or informational purposes to help expedite impact analysis and maintain uniformity in all three Districts.

Levee Profile Script:

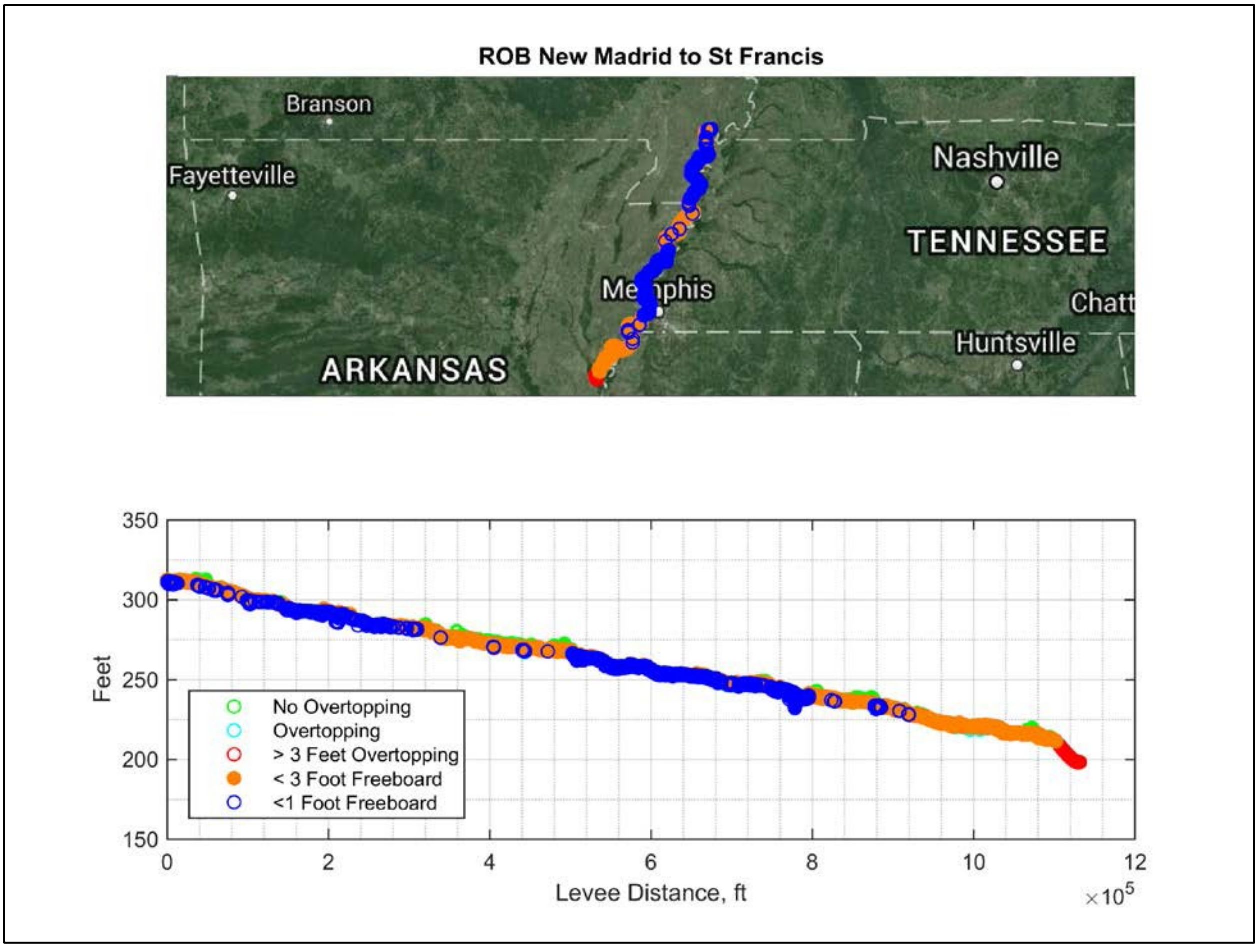
An essential piece to this new flowline study is being able to relate the newest water surface computations to the existing elevations of the levees. Previous flowline studies did not include such relationships directly or as comprehensively as we would today – mostly due to the availability of geographic information system (GIS) technology. This current study is a delicate balance between updating methodology and technology and making a useful comparison to the previous study. Due to such differences in methodology and technology it is not prudent to compare the newest, preliminary flowline to the previous flowline at the levee. Comparisons between the new estimated flowline and the previous flowline should be made near the middle of the river. However, in order to evaluate deficiencies in the system a comparison of the new estimated flowline to the existing levee heights is also made.

The levee profile script utilizes two main inputs, an inundation grid for a scenario simulated in HEC-RAS, and a set of 3D polylines representing the alignment and elevations of the existing grades of levees that the flowline study team intends to evaluate. The script uses all this data and determines a water surface profile at the levee based on user-defined intervals and offsets for a given segment's coordinates. The intervals and offsets are used to overcome challenges such as river mile to levee stationing relationships, inundation on both sides of a levee, and discrepancies between the inundation mapping and the levee alignment. Some example plots are shown below.



Example Levee Profile Plot

The script also uses the elevation information and can color-code specified thresholds of the water surface elevation in relation to the existing levee grade. It can also compute a summation of the horizontal distances at each one of the specified intervals as shown in the corresponding example table below.



Example Levee Profile Plot

ROB New Madrid to St Francis					
Total Levee Length	**1,131,756.64**	**Feet**	**214.35**	**Miles**	
Category	No Overtopping	Overtopping	> 3 Feet Overtopping	< 3 Foot Freeboard	< 1 Foot Freeboard
Horizontal Distance (feet)	854,912.60	276,844.20	28,382.80	756,902.70	233,376.50
Horizontal Distance (miles)	161.92	52.43	5.38	143.35	44.20

*note that < 3 Foot Freeboard includes < 1 Foot Freeboard in computations

Example Deficiency Table

It is recommended that a script be used for determining the preliminary impact analysis of the newest flowline to the existing levee grades to help determine the deficiencies of the current system's design.

SUBJECT: Miss River Flowline Study: HYPO 58A-EN Decision regarding use of NWS model results for Pickwick regulated outflows versus use of TVA modeled Pickwick regulated outflows.

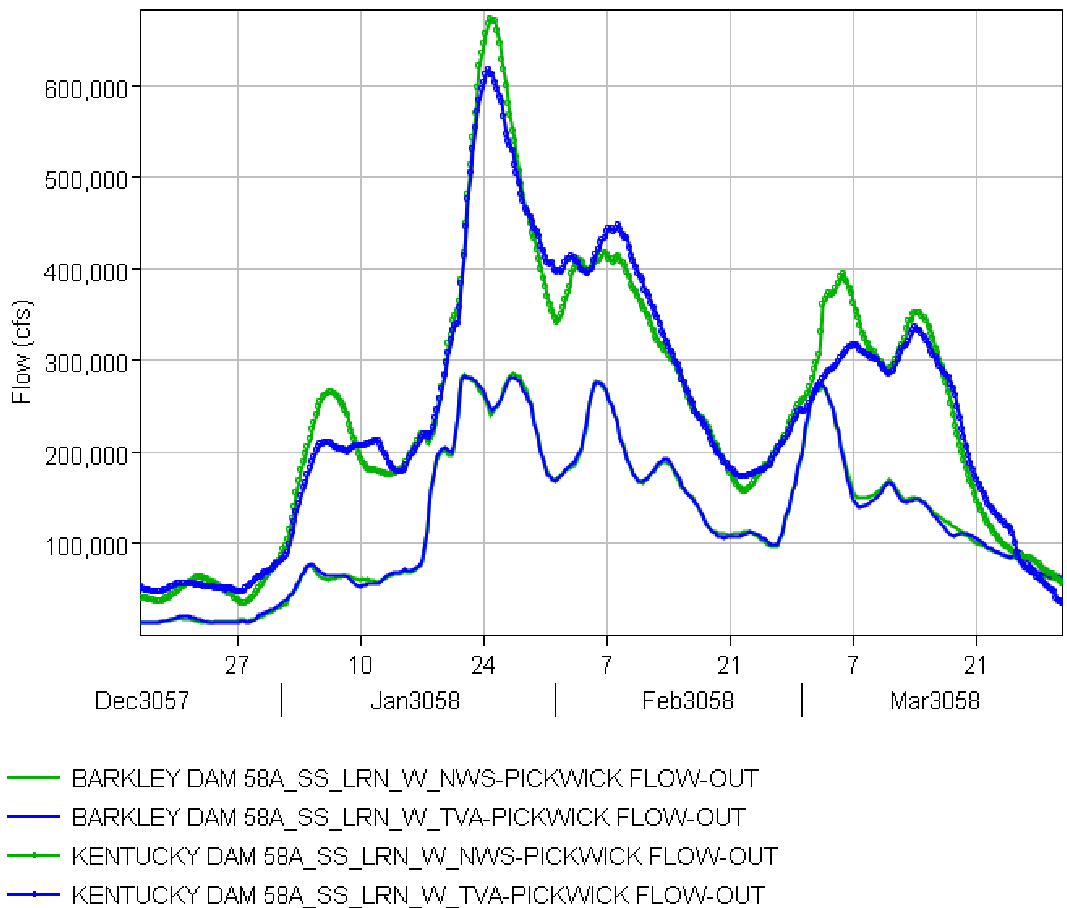
BLUF: TVA support to Flowline study projects slippage of 3-6 months to generate Pickwick outflows; alternate method using NWS-LMRFC models is recommended for primary results with TVA method to be completed to resolve any final ATR/IEPR comments.

Nashville District provided study support to run models of Kentucky and Barkley Dams to develop the required outflows for the HYPO 58A-EN condition. These outflows become inflow boundaries for the Flowline HEC-RAS unsteady model.

Development of regulated outflows from Kentucky and Barkley Dams for HYPO 58A-EN requires regulated outflows from the upper Cumberland River at Cheatam Dam and from the Tennessee River at Pickwick Dam. Nashville also performed system regulation for the Cumberland River. Nashville District has been very responsive and projects no issues in meeting future requirements in a timely manner. The TVA performed system regulation for the Tennessee River down to Pickwick Dam. TVA reported that it took 40 hours for staff to perform system regulation needed to generate Pickwick outflows. TVA stated that they are not able to dedicate this level of effort in producing similar outflows for the three (3) remaining HYPO storms (52A, 56, and 63). TVA projects that they could accomplish this regulation over the next 3 to 6 months.

NWS-LMRFC also models the Tennessee River. The LMRFC model captures basic reservoir operations for the TVA projects but does not capture intricate operational rules or joint operation considerations which are included in the TVA modeling. However, LMRFC indicates that they can include this additional modeling as part of their other modeling. This additional effort by LMRFC is expected to add minimal time to their efforts.

The LMRFC made runs to generate the Pickwick releases for only the straight sequence HYPO 58A. Their results were then passed to Nashville District who made additional runs of the combined KY/BK reservoir model. Resulting KY/BK outflows using the LMRFC model Pickwick flows were approximately 10% higher than when using the TVA generated Pickwick flows. The figure below illustrates the difference in KY/BK outflows for the two methods.



From the figure there is little impact on Barkley Dam releases. There is about a 10% increase in the peak outflow from Kentucky Dam.

The two simulations were run in the HEC-RAS unsteady model to see how changes might result along the main stem MS River. The 10% increase in Kentucky Dam outflow translates into very minor changes at Cairo and below. The changes are given in Table 1.

Recommend adopting the NWS method for generating Pickwick regulated outflows for HYPO 52A, 56 and 63. Also recommend continuing forward with TVA regulation to have available should final ATR or IEPR comments require that information.

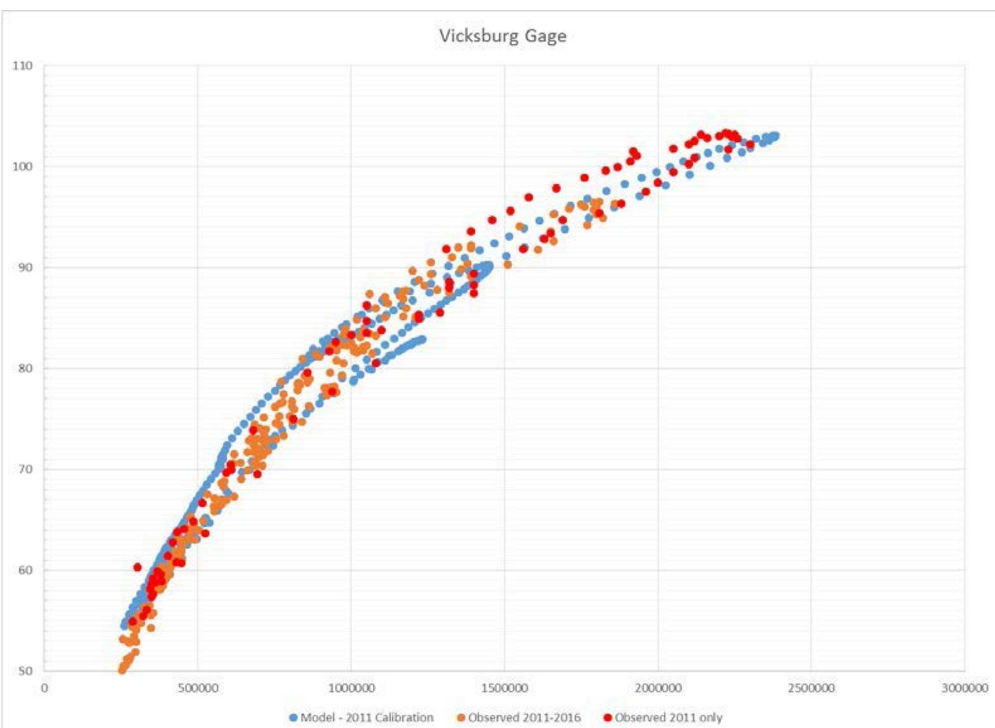
Table 1 Peak flows from HEC-RAS simulation for different regulation methods for Tennessee River, HYPO 58A.

	MVM 58 Regulated StraightSeq A (NWS Pickwick Outflows)	MVM 58 Regulated StraightSeq B (TVA Pickwick Outflows)	Percent Change (NWS versus TVA)
Tennessee River_at_Downstream of Kentucky Dam	672,210	617,490	8.86
Cumberland River_at_Downstream of Barkley Dam	286,371	282,235	1.47
Mississippi_at_Chester	501,451	501,448	0.00
Mississippi_at_Cape Girardeau	512,824	513,010	(0.04)
Mississippi_at_Thebes	514,539	514,854	(0.06)
Ohio River_at_Cairo	2,303,676	2,331,371	(1.19)
Mississippi_at_MS_OH Confluence	2,767,225	2,793,557	(0.94)
Mississippi_at_Hickman	2,007,986	1,963,755	2.25
Mississippi_at_New Madrid	2,543,957	2,526,756	0.68
Mississippi_at_Tiptonville	2,771,887	2,790,620	(0.67)
Mississippi_at_Caruthersville	2,330,311	2,332,402	(0.09)
Mississippi_at_Osceola	2,727,595	2,748,706	(0.77)
Mississippi_at_Memphis	2,727,899	2,738,423	(0.38)

SUBJECT: Miss River Flowline Study: Decision regarding loop effect that is not accounted for in the HEC-RAS Unsteady Hydraulic Model

BLUF: The HEC-RAS Unsteady model is capable of capturing portions of the loop effect on the Mississippi River but it doesn't account for all factors that make up the full loop effect. An analysis of observed data and model data should be completed to determine the amount of loop effect not accounted for in the unsteady HEC-RAS model.

In the previous flowline study that was conducted after the 1973 flood, loop effect was considered and then applied to the design flowline. Loop Effect is a river-flow hydraulics phenomenon which results in a lower stage for a rising river given the same flow than a falling river. This effect is more prevalent on larger alluvial bed streams, such as the Mississippi River and is caused by factors, such as variable energy slopes, scour and fill, alluvial bedform changes, backwater, temperature, movement of water into and out of storage, etc. The unsteady HEC-RAS model is able to account for some of these effects but factors such as bedform changes, scour and fill, etc. are not accounted for and it is unknown how much of the loop effect is attributed to these factors. To determine the impact of the factors that are not accounted for in the unsteady HEC-RAS model, it is proposed that the rating curves from the HEC-RAS model for observed events (i.e. calibration and validation simulations) are compared against observed rating curves for locations throughout the Mississippi River reach in which observed discharge and stages are collected (see figure 1 for an example). Since the unaccounted factors include effects from bedform changes, and scour and fill, it is recommended that only relatively recent events should be used for comparison.



It is recommended that further analysis on the loop effect at several key locations along the Mississippi River should be conducted to determine the amount of loop effect that is not accounted for in the HEC-RAS unsteady model. Recent observed rating curves for multiple locations along the Mississippi River should be compared to the computed rating curves from the HEC-RAS unsteady model to determine the amount of loop effect that is unaccounted for in the hydraulic model and then applied to the design flowline.

Decision Document for MRT Flowline Executive Steering Committee

SUBJECT: Mississippi River Flowline Study: Additional HEC-RAS Runs – Optimization of Water Control Features of MRT System

BLUF: An engineering analysis will be run using the calibrated HEC-RAS model to optimize the MRT system in order to pass the 2016 58A-R flows. Water control features of the MRT system will be adjusted to analyze effects of BW activation elevations being optimized and floodway releases being maximized (if necessary) in order to safely pass the 2016 58A-R flows.

Also, another run needs to be made showing future conditions (50-years) which will incorporate effects of sedimentation and RSLR.

BACKGROUND: The latest computed flowline for the flowline study using the HEC-RAS unsteady model utilizes the updated regulated flows (58A-R) developed in this study. The model was set up to run MR&T authorized conditions (exception being that the mainline levees were not allowed to overtop). The assumptions in making this run include a) setting the BW levees/plugs at currently authorized elevations, and b) releasing flows through ORCC, Bonnet Carre, and Morganza that were equal to their design flows in the current PDF. This run does not include loop- effect, sedimentation, or relative sea level rise (RSLR).

<u>HEC-RAS Runs</u>

The first RAS run will represent the current system with no adjustments for sedimentation or RSLR. The second run will represent the 50-yr projection of the system, which will include projected sedimentation and RSLR. The recommendation is to not include loop-effect in either of these runs in relation to the backwater areas. This is the conservative approach to insure we would remove the required amount should a PDF event occur.

Steps for each of the two RAS runs:

1) MVM runs its portion of the RAS model, adjusting BW activation elevations for the St. Francis and White areas to optimize the amount of flow that can be taken off the 2016 PDF peak. Output from this run will be sent to MVK.
2) MVK runs its portion of the RAS model, adjusting the

Yazoo BW levee elevation to optimize the amount of flow that can be taken off the peak. Output from this run will be sent to MVN.

3) MVN will analyze the options of increasing flows (if necessary after optimization of BW areas upstream) through Bonnet Carre and Morganza. Another option would include increasing flows through the ORCC (surpassing the 70-30 split).

Note: The primary goal of these RAS runs is to see if the 2016 58A-R flows can be reduced down to the current PDF flows at Baton Rouge (1.5M cfs) and New Orleans (1.25M cfs).

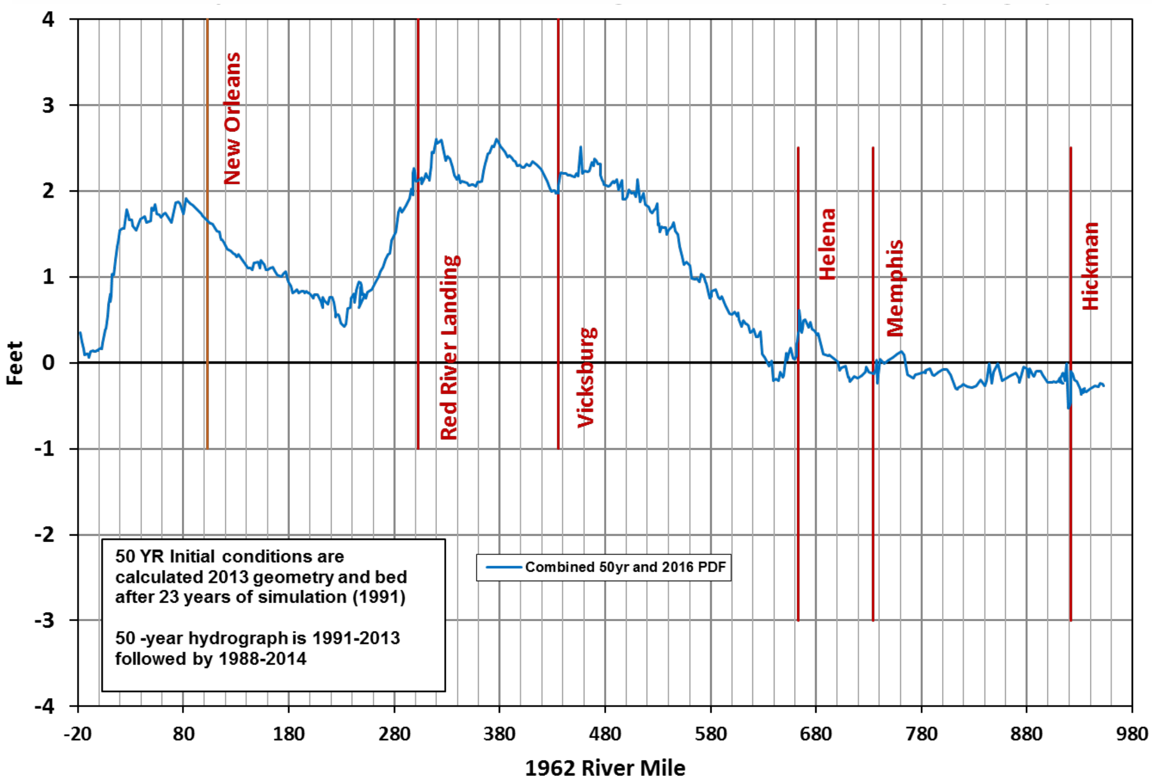
SUBJECT: Miss River Flowline Study: Decision regarding difference in Project Design Flood due to sediment accumulation.

BLUF: During a conference call held on December 6, 2016, with the sediment team members who were involved in the sediment analysis for the Flowline Study, a decision was made on the values to use to account for the effects of sedimentation. It was agreed to use the results from Ron Copeland's HEC-6T model which ran a 50-yr simulation.

Background: The objectives of the sediment analysis were to determine the long-term effect of sedimentation on the project flood water-surface elevations and the effects of bed elevation changes that occur during the rise and fall of the project flood hydrograph on maximum water surface elevations.

The calculated increase in the 2016 Project Design Flood water surface elevations to account for 50 years of sedimentation is shown in the figure below. The figure below combines the calculated increase in water-surface elevation over the 50-years with the difference in water-surface elevation at the peak of the hydrograph related to sedimentation during the rise of the flood. Calculated differences were determined every year during the 50-year period to account for the fact that the project design flood could occur at any time during the 50-year simulation. Maximum water surface elevations are less than zero at some stations because degradation occurs during the rise of the PDF hydrograph and when the hydrograph results are compared with a steady state peak discharge at time zero, the result is negative; however, it will be assumed that any negative numbers will be equal to zero when adjusting the PDF flowline.

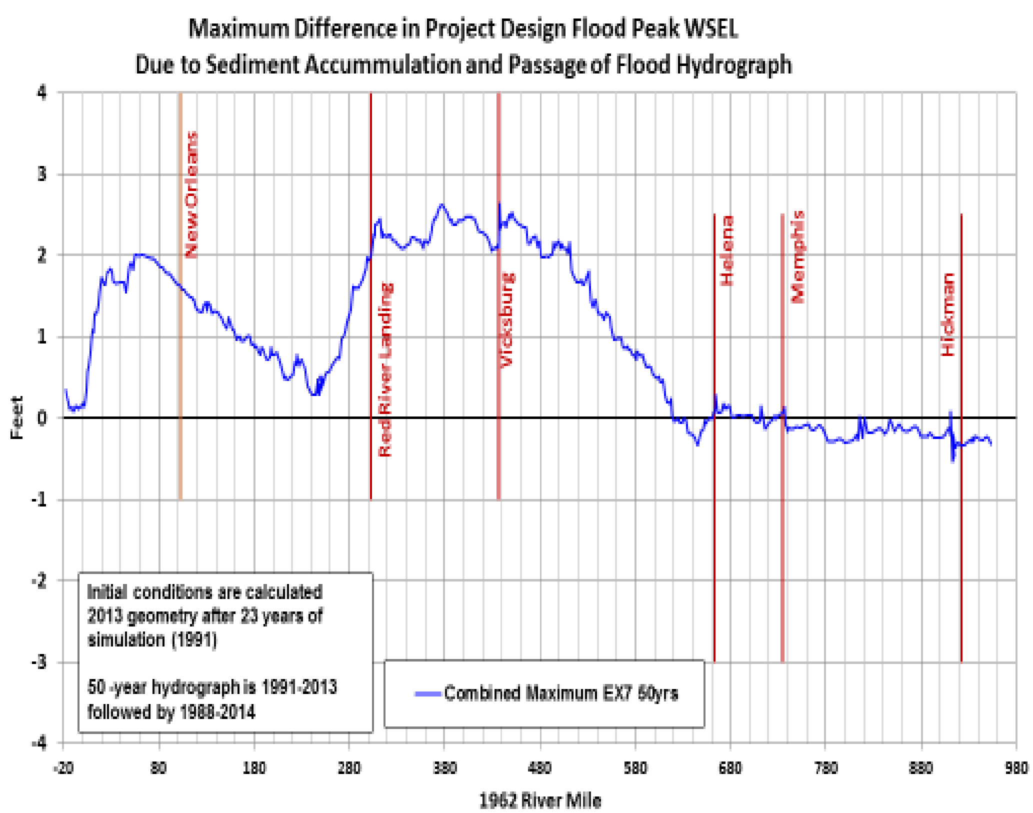
Maximum increase in 2016 estimated project design water-surface elevations due to 50 years of sedimentation, including the rise of the flood hydrograph



SUBJECT: Miss River Flowline Study: Decision regarding the optimization of the backwater levee heights as related to loop effect and sedimentation.

BLUF: During the January 10 weekly conference call with the team members who have been involved in the development of the "Concept Plan" RAS run (optimization features) with-future conditions for the Flowline Study, a decision was made to leave the optimized backwater levee heights the same as the "Concept Plan" run without-future conditions (sedimentation) or loop effect.

Background: The objectives of the sediment analysis were to determine the long-term effect of sedimentation on the project flood water-surface elevations and the effects of bed elevation changes that occur during the rise and fall of the project flood hydrograph on maximum water surface elevations. Calculated differences were determined every year during the 50-year period to account for the fact that the project design flood could occur at any time during the 50-year simulation. The maximum of these annual values are shown in the figure below.



One primary reason for the “Concept Plan” RAS run was to optimize the elevation of the backwater levees (obtain maximum reduction to the PDF flows). In creating the “Concept Plan” flowline for future conditions and the addition of loop effect, the decision was made to leave the optimized backwater levee heights the same as the “Concept Plan” run without-future conditions or loop effect; that is, the same flows would be assumed to be removed from the MS River by the overtopping of the backwater levees. The new flowline, however, for future conditions and loop effect would be adjusted by the amount calculated as a result of the sedimentation analysis plus the estimated amount of loop effect.

The elevation to set the height of the backwater levees due to future expected sedimentation is a complex issue and not within the scope of this work. A more comprehensive analysis would need to be undertaken to further explore this question. It would require revising the RAS model to show the accumulation of sediment and then rerunning the model. Also, it is more conservative to leave the BW levee heights the same, as raising them could lessen or altogether prevent flow from overtopping them.

The phenomenon commonly referred to as “loop effect” occurs in alluvial rivers and creates a situation whereby the stage does not have a unique value for a particular discharge. The process of calibrating the RAS model to the 2011 conditions using varying roughness values can indirectly capture many of the factors which influence the loop effect. The RAS model performs fully unsteady hydraulic computations and is expected to simulate a significant portion of the loop effect, but it is not expected to capture the entire amount. The additional portion of the loop effect at various locations along the river has been estimated based on observed data, and it will be added to the flowline similar to the bed change effects due to sedimentation; however, like the sedimentation effects, it would be very difficult to incorporate these flowline revisions into a recommendation for revising the backwater levee heights.

Decision Document for MR&T Flowline Steering Committee
Flow diagram correction for 1955 HYPO 58A-EN

BLUF: New hydrologic model results appear to show significant increase in peak flows for the Middle Mississippi River when compared to the 1955 flow diagram. It is necessary to confirm that correct values are used in this flow diagram. The flow diagram (plumbing diagram) in current use to represent the 1955 HYPO 58A-EN Project Design Flood (PDF) shows a flow of 240,000 cfs originating from the Mississippi River above the Ohio River confluence. This diagram was also used for developing the refined 1973 flowline. The correct value for flow entering from the Mississippi River at this location should be 410,000 cfs. All future correspondence and communication should reference the correct value of 410,000 cfs for flow entering from the Mississippi River immediately upstream of the Ohio River confluence for HYPO 58A-EN.

Background: The 240,000 cfs flow rate from the Mississippi River above the confluence with the Ohio River is the flow rate at St. Louis and can be misinterpreted as the contributing flow from the Mississippi River above the Ohio River confluence (Figure 1). In the 1955 Project Design Flood study, engineers combined the flow from St. Louis, the local basin flow from sub-basin 7-Y, and the total flow at Metropolis, IL on the Ohio River to determine the combined flow of 2,360,000 cfs below the confluence. An intermediate computation point was not used in the original study between St. Louis and the confluence with the Ohio River. Although not published, the flow from the hydrograph at St. Louis added to the local basin flow from 7-Y would have been computed as 410,000 cfs for 11 February, coincident with the date of the combined flood peak for the Mississippi and Ohio Rivers.

Method: The tabulated hydrographs from the 1955 study were obtained from the MRC archives. The tabulation shows daily flow at St. Louis, MO, sub-basin 7-Y which is the contributing area along the Mississippi River between St. Louis, MO and the Ohio River confluence, and at Metropolis, IL on the Ohio River. These three hydrographs were combined to provide the PDF flow hydrograph for the Mississippi River downstream of its confluence with the Ohio River.

Figure 2 shows the tabulated data plotted for the 1955 study.

Figure 3 shows 1955 unregulated and regulated hydrographs for HYPO 58A for the Mississippi and Ohio Rivers combined flow and for the St. Louis hydrograph added to the flow from sub-basin 7-Y. Figure 2 also shows hydrographs at these locations for the 2016 model results for comparison.

Figure 1. Flow Diagram representing 1955 Project Design Flood discharges (also used for Refined 1973 Flowline)

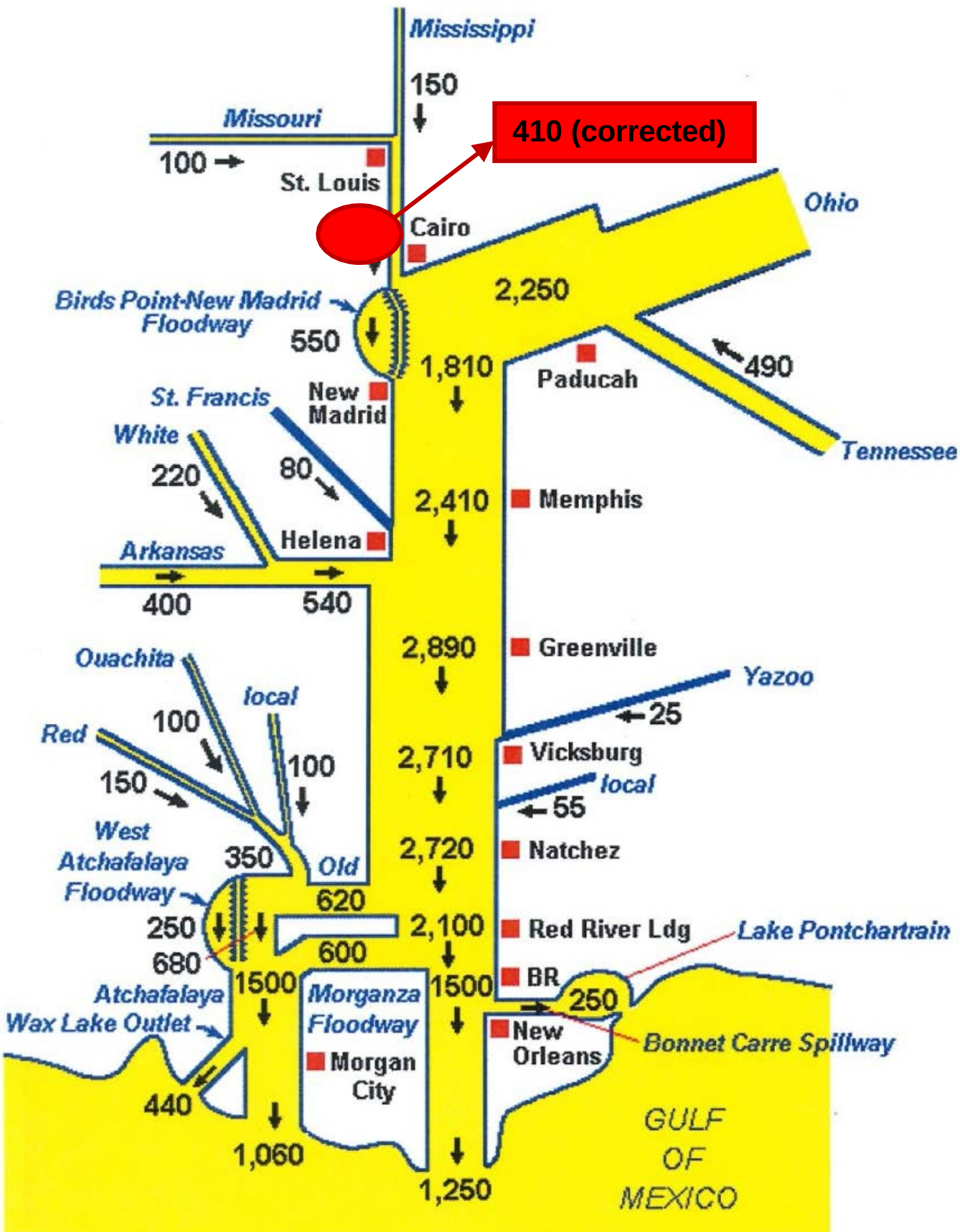


Figure 2 Plotted discharge hydrographs from MRC archive 1955 tabulations

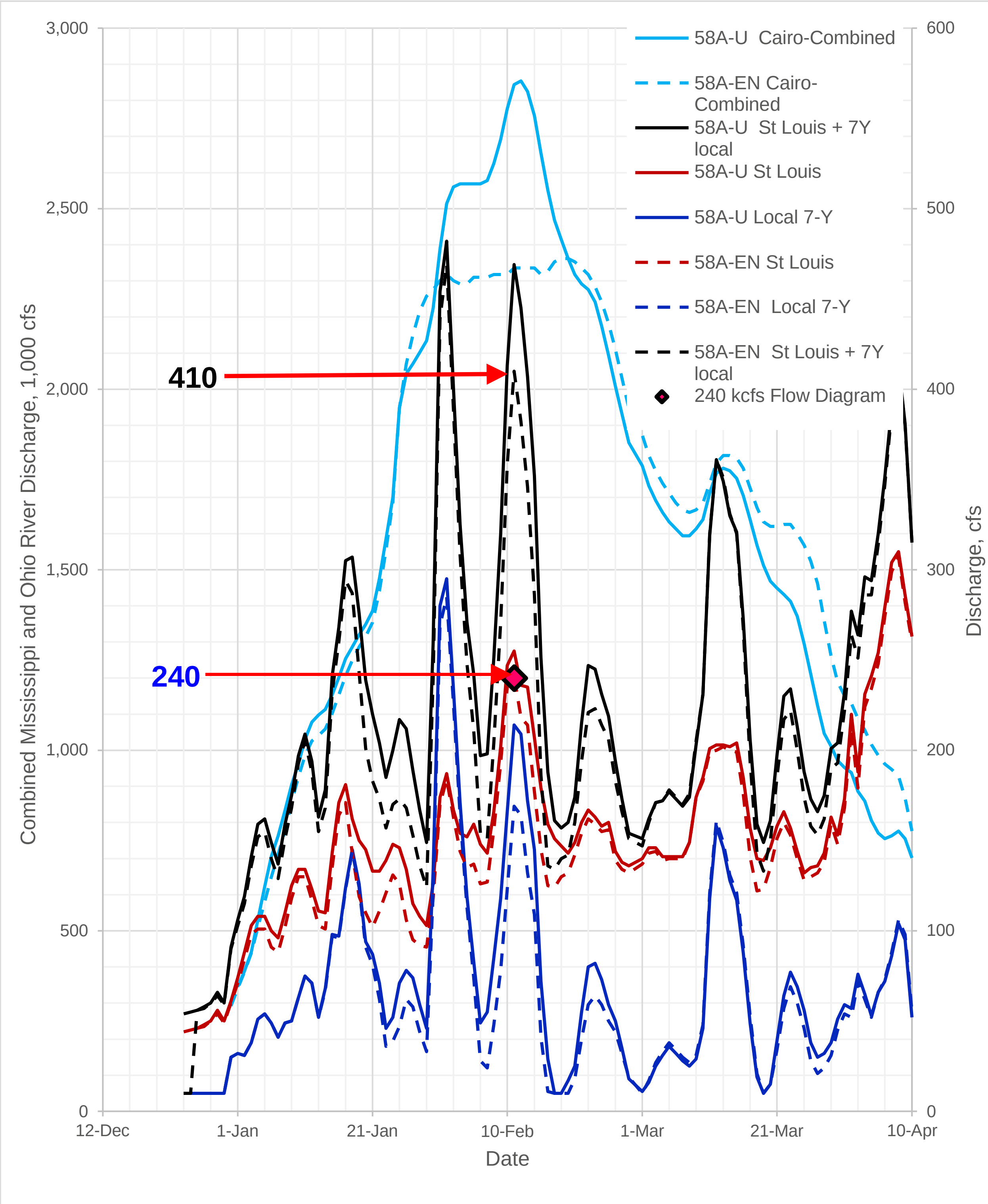
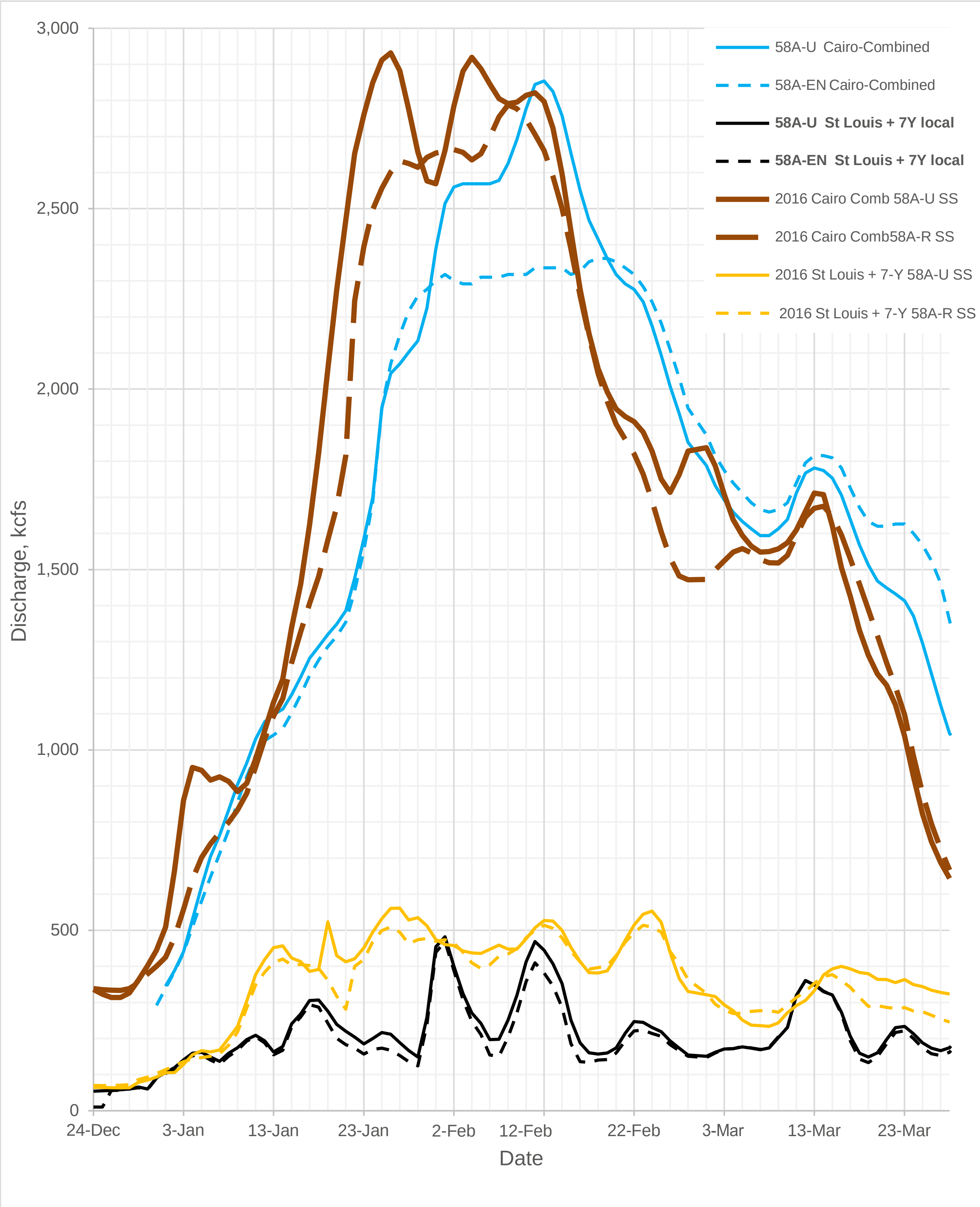


Figure 3 Comparison of 1955 and 2016 hydrographs for HYPO 58A



Appendix D: Reviews of This Project

This appendix can be found at http://dx.doi.org/10.21079/11681/32140.

Appendix E: Internal and External Peer Review Process

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14. ABSTRACT
The flowline assessment was undertaken to review the current Mississippi River and Tributaries (MR&T) flowline on which the project design levee elevations are based. The assessment was initiated in 2014 to address the issues outlined in the MR&T 2011 Post Flood Report. Evaluations were to not only consider the hydrology and hydraulic conditions of the system today but were to consider any other factors that would impact the flowline over the next 50 years. To assess the flowline, a seven step process was utilized: (1) develop precipitation amounts; (2) calculate the amount of runoff; (3) calculate the amount of flow in the river from the runoff, both for regulated and unregulated conditions; (4) calculate a water surface profile/elevation with appropriate levees, structures, backwater areas, and floodways assumptions; (5) develop estimates for future degradation/aggradation of sedimentation at various locations, climate change, loop effect, sea level rise, and subsidence, all of which would affect the water surface profile; (6) analyze water surface profile estimates to identify areas of concern; and (7) perform District Quality Control Review, Agency Technical Review, and Independent External Peer Review on all of the above steps to ensure quality.

15. SUBJECT TERMS
Flood control—Mississippi River, Hydraulic models, Hydrologic models, Levees—Mississippi River, Mississippi River Watershed, Sedimentation and deposition, Water levels

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