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Technical Assessment of the Old, Mississippi, Atchafalaya, and Red (OMAR) Rivers: Mississippi River HEC-6T Model

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Technical Assessment of the Old, Mississippi, Atchafalaya, and Red (OMAR) Rivers

Mississippi River HEC-6T Model

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Abstract

The HEC-6T 1D numerical sedimentation model was used to evaluate the long-term and system-wide sedimentation effects of modifying the operation schedule at the Old River Control Complex (ORCC). The changes evaluated were the increase and decrease of the percentages of flow diverted from the Mississippi River at the ORCC and the modification of the distributions through the four ORCC structures. Also evaluated were dredging scenarios that removed sediment from the bed of the Mississippi River at the ORCC. Sedimentation effects for several operation and dredging scenarios were compared to a Base Condition that represented the existing operation protocols. The predictive simulations extended for 50 years. The model was used to calculate and compare sand transport past various gages along the Mississippi River and in the Outflow Channel. Dredging volumes at the deep-draft crossings and in Southwest Pass and above Head of Passes were compared. Differences in water-surface profiles in the vicinity of ORCC were determined.

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Preface

The investigation documented in this report was conducted for the US Army Corps of Engineers (USACE), Mississippi Valley Division (MVD), as part of the Mississippi River and Tributaries Project, under Project No. 478534, “Old, Mississippi, Atchafalaya and Red Rivers Assessment,” and published through the Mississippi River Geomorphology and Potamology (MRG&P) Program. At the time of publication of this report, the MRG&P program director was Dr. James W. Lewis. The MVD commander was MG Diana M. Holland, and the MVD director of programs was Mr. Edward E. Belk.

The work was performed by the River and Estuarine Engineering Branch of the Flood and Storm Protection Division, US Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory (ERDC-CHL). At the time of publication of this report, Mr. David May was chief of the River and Estuarine Engineering Branch, and Dr. Cary A. Talbot was chief of the Flood and Storm Protection Division. The deputy director of ERDC-CHL was Mr. Keith Flowers, and Dr. Ty V. Wamsley was the director.

Mr. William A. Thomas, Mobile Boundary Hydraulics, and Mr. Ronald E. Heath, USACE, CHL/ERDC, are gratefully acknowledged for their insightful review of this document. Their comments and suggestions significantly enhanced the accuracy and utility of this report.

The commander of ERDC was COL Christian Patterson, and the director was Dr. David W. Pittman.

1 Introduction

The US Army Corps of Engineers (USACE) conducted the Old, Mississippi, Atchafalaya, and Red Rivers (OMAR) Assessment, under the authority of the Mississippi River and Tributaries (MR&T) Program (Investigations) to gain a more comprehensive technical understanding of the Old River Control Complex (ORCC) and to collect data relevant to its operation and management. The goal of the OMAR Assessment was to attain a protocol that would maintain optimum sediment distribution for the ORCC, which is key to maintaining the best possible MR&T system now and into the future.

The ORCC consists of the Low Sill, Overbank, and Auxiliary Structures, which were constructed and are operated by the USACE to control the flow of water and sediment from the Mississippi River to the Atchafalaya River, and a separate commercial hydropower structure that is not part of the federal project. Although the Hydropower Structure is not an authorized component of the regulation protocol, the Hydropower Structure has a significant influence on the transfer of water and sediment from the Mississippi River and therefore was included in this modeling assessment. The three federal structures and the Old River Lock and Dam, which provides navigation between the Mississippi and Atchafalaya Rivers, are referred to as the Old River Control Structure (ORCS).

A primary driver of this investigation was the observation of changing water surface elevations at several locations along the Mississippi River. In addition, the Flowline Assessment (Lewis 2018) revealed that the water surfaces of the Mississippi River are projected to rise in the future over a 663 mi^{*†} length of the Mississippi River from Head of Passes (River Mile [RM] 0) to Helena (RM 663).

The OMAR Assessment (1) evaluated operations at ORCS, with a focus on the Mississippi and Atchafalaya Rivers; (2) determined the current volume of sediment and water passing through ORCC and considered potential

^{*} For a full list of the spelled-out forms of the units of measure used in this document, please refer to *US Government Publishing Office Style Manual*, 31st ed. (Washington, DC: US Government Publishing Office 2016), 248-52. <https://www.govinfo.gov/content/pkg/GPO-STYLEMANUAL-2016/pdf/GPO-STYLEMANUAL-2016.pdf>.

[†] For a full list of the unit conversions used in this document, please refer to *US Government Publishing Office Style Manual*, 31st ed. (Washington, DC: US Government Publishing Office 2016), 345-7, <https://www.govinfo.gov/content/pkg/GPO-STYLEMANUAL-2016/pdf/GPO-STYLEMANUAL-2016.pdf>.

adjustments to those volumes; and (3) assessed if the operation of the structures at ORCS required current or future adjustments in flow and/or sediment to optimize the ORCS achievement of its authorized purpose. Along with the ORCS structures, these considerations included adjustment in the allocation of flow to the Sidney A. Murray Jr. Hydroelectric Plant and the plant's impact on, and capability for, sediment distribution. The OMAR Assessment provided an opportunity to gain a better understanding of the transport of water and sediment throughout the ORCC, which can subsequently inform management options to optimize levels of sediment deposition and flood risk on the Mississippi River, support water control operations, and help evaluate how various operational changes at ORCC could impact the entire system. The Assessment provided a wealth of valuable engineering and scientific data to inform future adjustment of the ORCS.

Several charge questions were identified by the Executive Steering Council for the OMAR Assessment. The charge questions that the HEC-6T one-dimensional (1D) model study (Task 5 of the OMAR Assessment) was to aid in answering were the following:

1. How much sediment could be diverted by ORCS operations if the hydroelectric station was not operated?
2. How can water control operations be optimized to improve sediment transfer based on improved understanding of water flow and sediment transport in the system?
3. How much sediment must be diverted to bring the Mississippi River at ORCC into dynamic equilibrium?
4. What is the sediment transport capacity of the ORCC combined outflow channel under a variety of potential operational schemes?
5. What are the long-term impacts (i.e., change in flowline) above and below ORCC on the Mississippi and Atchafalaya Rivers for the various operational and dredging management options evaluated?
 - a. Operational management options to be evaluated were based on technical operational constraints of the various structures and include scenarios that maintain the present 70/30 flow split as well as scenarios that modify the flow split.
 - b. Dredging management options to be evaluated will include discharge downstream in the Mississippi River as well as discharge to bypass sediment to the Atchafalaya River through

the ORCC outflow channel and will consider continuous versus episodic dredging.

1.1 Background

The Mississippi River is divided into three sections. The Upper Mississippi River extends from the river's headwaters to the Missouri River. The Middle Mississippi River extends from the Missouri River to the Ohio River. The Lower Mississippi River extends from the Ohio River to the Gulf of Mexico. The 1D numerical sedimentation model used in Task 5 of OMAR Assessment was developed over a period of approximately 10 yr and extends from Chester, IL, on the Middle Mississippi River and Olmsted Lock and Dam on the Ohio River, to the Gulf of Mexico. The model has been used in a number of sedimentation studies and has been enhanced at each step along the way.

A Regional Model of the Lower Mississippi River between Pilots Station, which is located at the entrance to Southwest Pass, and Cairo, IL, was completed in 2011 (Copeland et al. 2020). The model included several tributary reaches that served to provide water and sediment supply at the model boundaries. These included 109 mi of the Middle Mississippi River and 36 mi of the Ohio River. Model development was coordinated by the Mississippi Valley Division and was cooperatively developed by USACE engineers from the New Orleans, Vicksburg, Memphis, and St. Louis Districts. Each district developed the Regional Model in its respective reaches of responsibility along the Mississippi River. The model covered over 1000 river miles. The initial cross-section geometry was taken from 1988–1992 hydrographic surveys and was calibrated to 1991–2002 measured data. The objective of the Regional Model was to provide a tool to evaluate the long-term and system-wide effects of specific MR&T Project features. These included sediment diversions, dredging, and dikes. The model was also used to test the effects of changes in upstream sediment inflow and to simulate the effects of natural geomorphic changes, such as erosion of hard points.

The Regional Model was used as a starting point for several other HEC-6T numerical model studies. Mobile Boundary Hydraulics (MBH) (Thomas 2012) investigated the effects of a large-scale diversion structure at Myrtle Grove for the State of Louisiana. At the US Army Engineer Research and Development Center (ERDC), Sharp et al. (2013)

investigated the effects of the West Bay Diversion on maintenance dredging. As part of the Mississippi River Hydrodynamic and Delta Management Study, Heath et al. (2017) and Thomas et al. (2018) evaluated the potential impacts of deepening the Mississippi River Ship Channel on maintenance dredging requirements.

The Regional Model was also used as the starting point for the Flowline Model. The Flowline Model was developed as part of the Flowline Assessment Study, which determined new Project Design Flood (PDF) water-surface elevations for the Lower Mississippi River between Cairo, IL, and Pilots Station, LA. The Flowline Model was used to determine how long-term sedimentation would affect PDF peak water-surface elevations (Copeland 2018). In the Flowline Model, 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 activation of the upper crevasse 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. Updates from recent ERDC and MBH HEC-6T numerical model studies of the Mississippi River were also incorporated. The Regional Model had been calibrated to 2002 conditions (11 yr). The Flowline Model was verified by simulating an additional 11 yr through 2013.

The Flowline Model was in turn used to develop a model focused on evaluating dredging at deep draft crossings in the Lower Mississippi River (Copeland et al. 2019). This study was able to simulate annual dredging and then was used to evaluate the effects of removing the dredged material from the river rather than returning the dredged material to the river. This model served as the starting point for the modeling effort described in this report.

1.2 Objective

The objective of this numerical model investigation was to determine the long-term and system-wide sedimentation effects that would occur in the Mississippi River if the operation protocols at the ORCS and Hydropower Structure were adjusted. The adjustments included increasing and decreasing the percentages of flow that are diverted from the Mississippi River at the ORCC and modifying the distributions of flow through the ORCS and Hydropower Structure. In addition, three alternatives that

included dredging the Mississippi River at the ORCC were evaluated. Specifically, the study determined the response of the Mississippi River in terms of water-surface elevation changes, bed elevation changes, downstream dredging requirements, and sediment transport.

This effort is a part of the overall Old, Mississippi, Atchafalaya, and Red Rivers Assessment. Table 1 lists the series of reports associated with the overall project, with this report listed in bold font.

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

Volume	Report Name	Description
1	Main Report	Summarizes the entire project assessment
2	Geomorphic Assessment	Analyzes the historic trends in hydrology, sedimentation, and channel geometry of the river reaches of interest
3	Channel Geometry Analysis	Analyzes the hydrographic surveys over the past 6 to 7 decades
4	Mississippi River HEC-6T Model	Evaluate the long-term and system-wide sedimentation effects on the Mississippi River
5	Atchafalaya River HEC-6T Model	Evaluate the long-term and system-wide sedimentation effects on the Old, Atchafalaya, and Red Rivers
6	Mississippi River Multi-Dimensional Model	Evaluate the short-term effects on the Mississippi River
7	Red and Atchafalaya Rivers AdH Model	Evaluate the short-term effects on the Old, Atchafalaya and Red Rivers
8	HEC-RAS Model	Investigate how water is stored in the Lower Red River floodplain
9	HEC-RAS BSTEM Analysis of the Atchafalaya River	Compare the relative impact of various scenarios on bank retreat in the upper portion of the Atchafalaya River

1.3 Approach

The objectives of the OMAR Assessment were accomplished using an updated version of the previously developed Flowline Model. Existing conditions for the OMAR Model were established using historical data between 1990 and 2019. The HEC-6T model simulated river responses that occurred during these 29 yr and wrote a new geometry file that was used as a starting point for evaluations in this study. A 50 yr hydrograph determined from historical discharges between 1969 and 2019 was developed to represent the next 50 yr (October 2019–September 2069). This historical time period included major floods in 1973 (2023), 1993

(2043), 2011 (2061), and 2019 (2069). A 50 yr simulation that used existing operation protocols at ORCC was run to establish a Base Condition. River responses from the Base Condition were then compared to river responses from 50 yr simulations of various operation scenarios.

1.4 Scope

This Assessment evaluates the long-term effects of changing operation protocols at ORCC. To accomplish this objective in a timely manner, average diversions of water and sediment through the structures are used in the OMAR Model. These averages were determined from measured data taken over a period of several years. This approach limits the applicability of the model in that it should not be expected to predict or duplicate daily responses to operation changes.

2 Numerical Model Description

2.1 HEC-6T description

The HEC-6T numerical sedimentation model used in the OMAR Assessment was developed by William A. Thomas from Mobile Boundary Hydraulics (Thomas 2021). Version H6TV51325_J18amR_210602 was used. The model is proprietary and can be obtained from MBH.

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.

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 links events together to extend this calculation to long periods of time (e.g., 50 or 100 yr). It does not simulate bank erosion or natural adjustments in channel widths. However, HEC-6T is capable of approximating channel widening that occurs as a result of bank erosion induced by degradation. The program was not designed to compute local scour and deposition around hydraulic structures. When applied by experts using sound engineering judgment, the HEC-6T program will provide valuable 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 2021), Chapter 14–“Computational Modeling of Sedimentation Processes” (Thomas and Chang 2008), and *Guidelines for the Calibration and Application of Computer Program HEC-6* (USACE-HEC 1992).

2.2 Model network

The OMAR model geometry is broken into 25 segments as shown in the network schematics in Figure 1 and Figure 2. The number of cross sections in each segment and the description of the segment boundaries are given in Table 2.

Table 2. Description of HEC-6T segments.

Segment	Number of Cross Sections	Description	1962 River Miles (RM)
1	174	Mississippi River - Gulf of Mexico to Profit Island	-22.0 to 246.3
2	6	Mississippi River - Main Channel (W) Profit Island	246.9 to 251.8
3	6	Mississippi River - Chute Channel (E) 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
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 Saint Francis River	600.1 to 672.6
12	19	Saint Francis River	0.5 to 17.7
13	39	Mississippi River - Saint Francis River to Hatchie River	674.7 to 772.8
14	3	Hatchie River	0.1 to 2.0
15	19	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	10	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	Middle Mississippi River - Cairo to Chester	0.8 to 109.9

Figure 1. Schematic of HEC-6T stream network in New Orleans and Vicksburg Districts (not to scale).

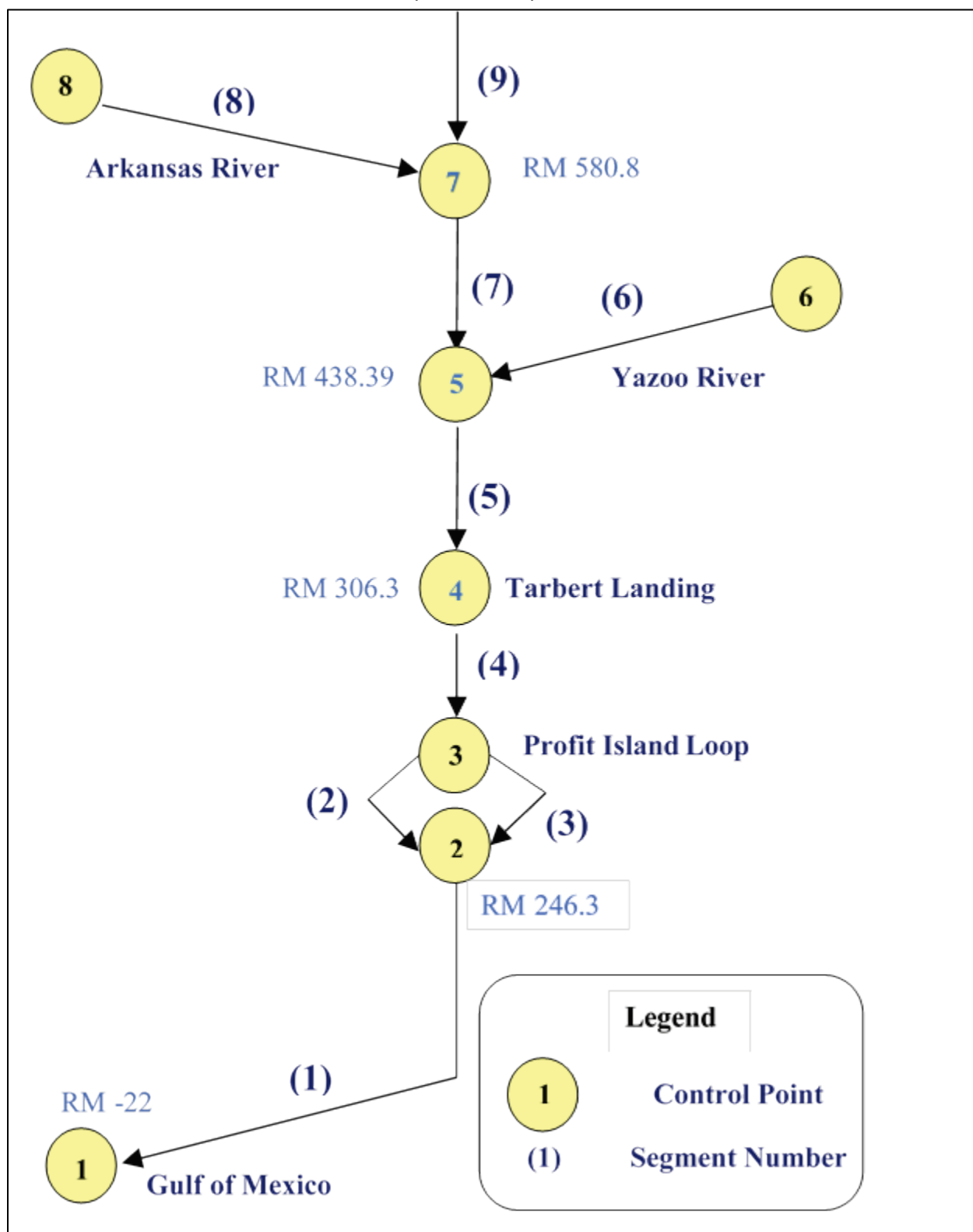
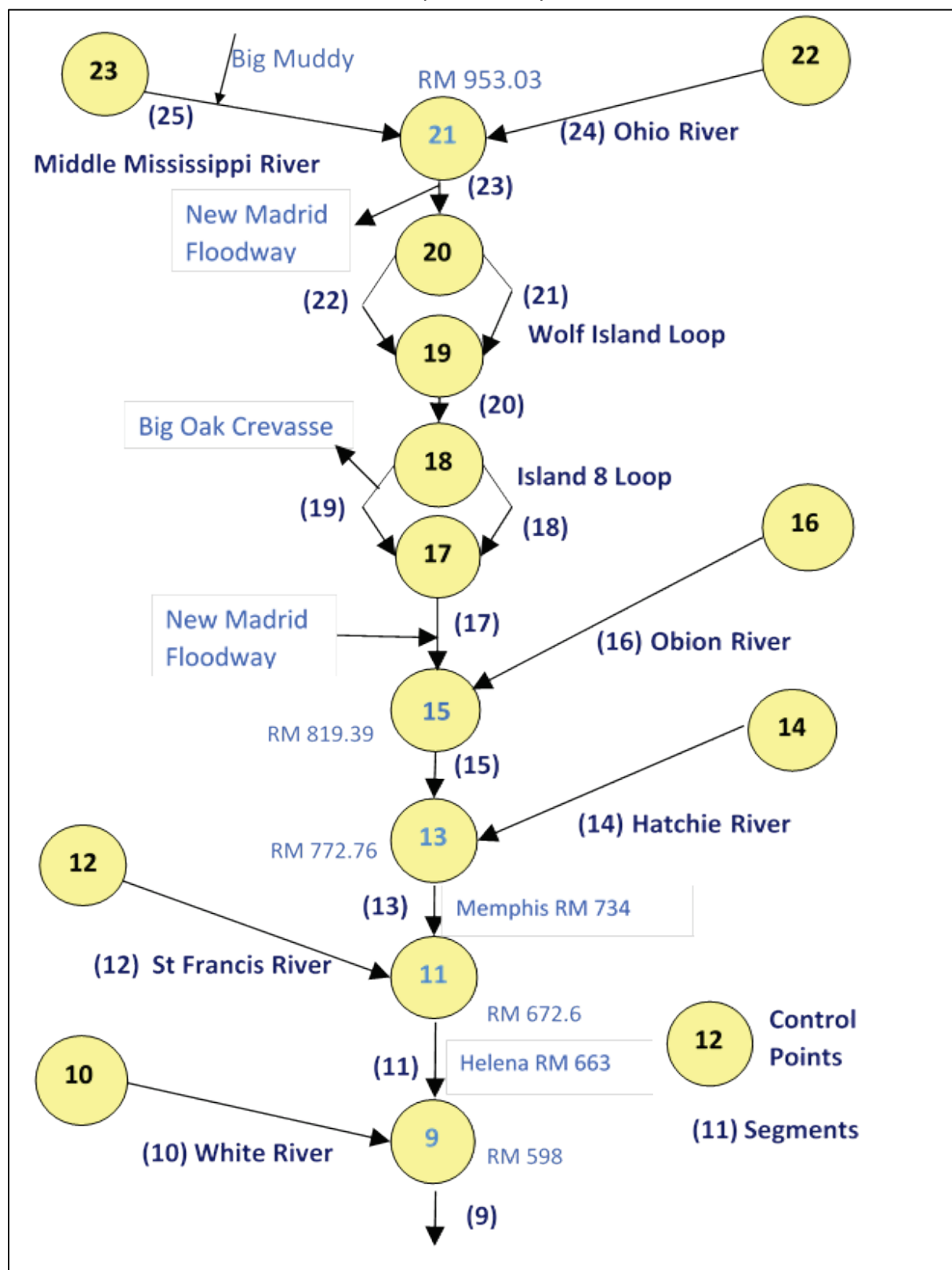


Figure 2. Schematic of HEC-6T stream network in Memphis and St Louis Districts (not to scale).



2.3 Channel geometry

The cross-section geometry in the model was developed from hydrographic survey data taken between 1988 and 1992. Survey datum was NGVD 1929. The hydrographic survey dates are not directly comparable because surveys were conducted at different times in the different districts. Even in the same district, surveys were taken in a sporadic pattern. The initial 1988–1992 cross-section geometry was retained in the OMAR model. This is deemed reasonable because the calibration and verification runs begin using the October 1990 discharges. In this report, elevations are referenced to NGVD 29 unless otherwise specified.

Dike fields were included in the model by modifying hydrographic survey cross sections so that the projection of dike crest elevations obstructed conveyance in the channel. Dike fields were added as the historical simulation progressed corresponding to construction dates. Dike fields constructed up to 2009 are included in the model.

The cross sections are coded from left to right looking downstream and generally have reach lengths of 2 to 4 mi. 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 mi in reaches where it was difficult to find a representative cross section for 1D flow.

More detailed information about the survey data used to develop the cross sections in the Regional and Flowline Models is provided in Copeland et al. (2020) and Copeland (2018).

2.4 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 HEC-6T is successful in predicting long-term sedimentation processes.

Reported daily discharges are available at several gages within the study reach. Uncertainty is associated with the choice of discharge data for the inflow hydrograph because there are discrepancies in the reported data. Continuity of the discharge record is inconsistent with both time and distance. The Regional Model report (Copeland et al. 2020) has a more detailed discussion about the issues related to inconsistent gage data.

The HEC-6T hydrographs were based on data from upstream boundary gages and did not include adjustments to tributary inflows to match downstream gages. Lag was accounted for by slipping daily discharge dates between the boundaries. The true reported data were assigned to the inflow boundary farthest downstream on the Yazoo River, which is approximately at Vicksburg, MS. A 2-day lag was estimated for the Arkansas River, 3 days for the White and Saint Francis Rivers, 4 days for the Hatchie and Obion Rivers, 7 days for the Ohio River, and 8 days for the Big Muddy and Middle Mississippi Rivers. Consequently, in the HEC-6T hydrograph, the recorded discharges for September 23 at Thebes would correspond to the October 1 recorded discharges for the Yazoo River. This also means that the date labels in the HEC-6T files do not correspond to the true dates, except near Vicksburg. 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. Accounting for lag in this approximate fashion was deemed a reasonable simplification because tributary inflow downstream from Cairo is a minor fraction of the total inflow from the Ohio and Middle Mississippi Rivers.

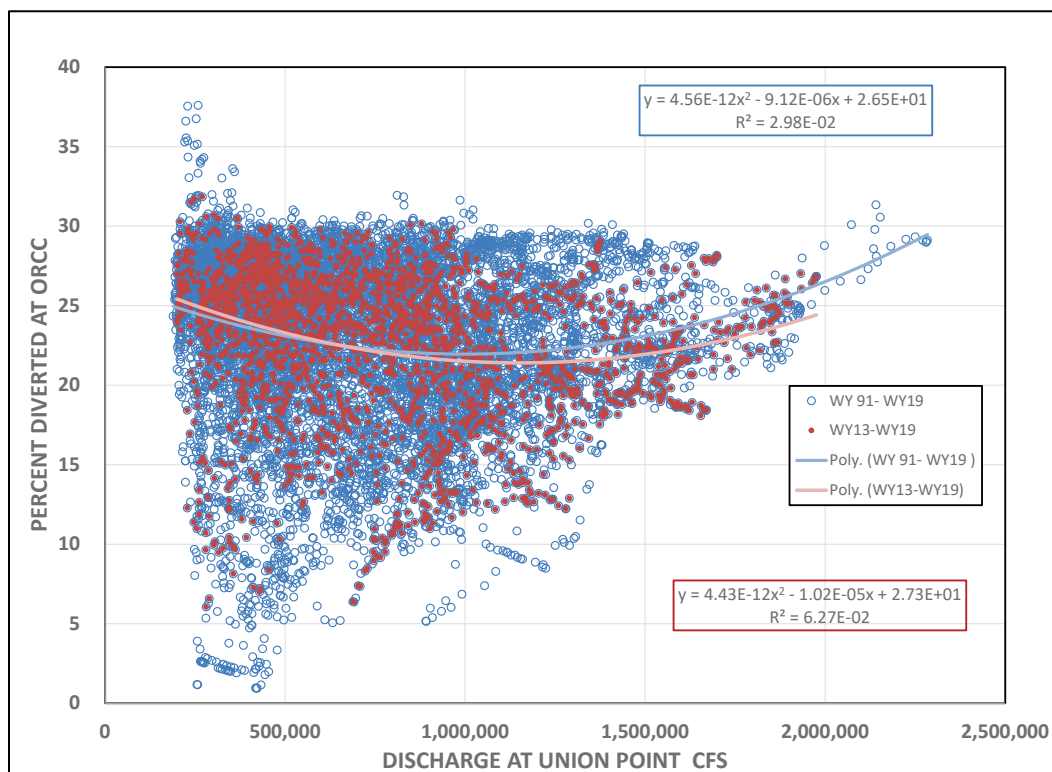
More detailed information about the hydrograph data and sources, including the significance of systematic and random errors, is provided in Copeland et al. (2020) and Copeland (2018). The historical hydrograph from the Flowline Model was extended 6 yr (30 September 2013 to 30 September 2019) in the OMAR model.

2.5 Old River Control Complex (ORCC)

The ORCC consists of four structures. The three USACE structures were constructed to control the distribution of flow and sediment from the Mississippi River to the Atchafalaya River and are referred to as the ORCS. ORCS consist of the Low Sill and Overbank Structures that were constructed in 1963 and the Auxiliary Structure that was constructed in 1986. The fourth structure is the Hydropower Structure that was constructed in 1990 and is not part of the federal project. Its purpose is to generate electricity for commercial use.

The OMAR Assessment was conducted to evaluate the long-term sedimentation effects that would occur with adoption of the various ORCC operation scenarios selected for study. To achieve this objective, it was not necessary to assign actual daily flow diversion percentages at each structure in the model. Rather, long-term average regression curves that varied flow diversion percentages with upstream discharges were developed from the historical data collected between 1991 and 2019. The total daily flow diverted at the ORCC was not affected by operation changes that occurred during those 29 yr (Figure 3).

Figure 3. Percentage of Union Point discharge diverted at ORCC.



However, the distribution of diverted flow through the various ORCC structures changed significantly in 2013. The most significant change was the return of the Overbank Structure into service. The Overbank Structure was not operated between 1991 and 2012. The long-term average allocation of flow between the Low Sill Structure and the Auxiliary Structure also changed dramatically after 2012. Before 2013, approximately 80% of the combined Low Sill and Auxiliary Structure flow went through the Auxiliary Structure. Since 2013, each structure carries approximately half of the long-term average flow. The daily discharge data at the Hydropower Structure indicated some modification in operations. Consequently, the HEC-6T flow distributions at the ORCC were adjusted during the historical numerical simulations to account for changes in the flow distribution. Long-term average flow diversion regression curves were developed from the daily data at each structure.

The historical simulations of the 29 yr between 1991 and 2019 used two different flow diversion regression curves for each ORCC structure. Between 1991 and 2012, the flow distribution was based on 1991–2012 data. This included a special modification for the 2011 flood that was developed using just the data between May and June of 2011. A flow diversion regression curve developed from the 2013–2019 data was used for the historical simulations between 2013 and 2019.

The Base Condition simulations that projected 50 yr sedimentation effects used 2013–2019 regression curves for all the ORCC structures, except the Overbank Structure. Using the 2013–2019 data was deemed the most appropriate representation of existing and future conditions. The diversion percentages for the Overbank Structure were supplemented with historical data between 1979 and 1990 to capture data from the highest historical diversions through the structure.

The index station for the Hydropower and the Overbank Structures was Union Point. The index station for the Low Sill and Auxiliary Structures was the cross section just upstream from the Low Sill Structure. The distribution of flow through the Low Sill and Auxiliary Structures did not correlate well with upstream discharge because discharge is not the only factor taken into consideration when determining operations of those two structures. In the numerical model, the distribution of flow through the Low Sill and Auxiliary Structures was taken to be the long-term average distribution determined from 1991–2012 or 2013–2019 daily flow data.

Regression equations were developed for Hydropower Structure diversion percentages using data between 1991–2012 and 2013–2019. The 2013–2019 regression equation was used to determine Hydropower Structure diversion percentages for future simulations and for 2013–2019 historical simulations. An optical extrapolation was used for Union Point discharges greater than 2,000,000 cfs and a minimum diversion percentage was set based on the minimum measured Union Point discharge (189,000 cfs). The data and flow diversion percentages regression curves are shown in Figure 4 and Table 3.

Figure 4. Flow diverted at ORCC Hydropower Structure.

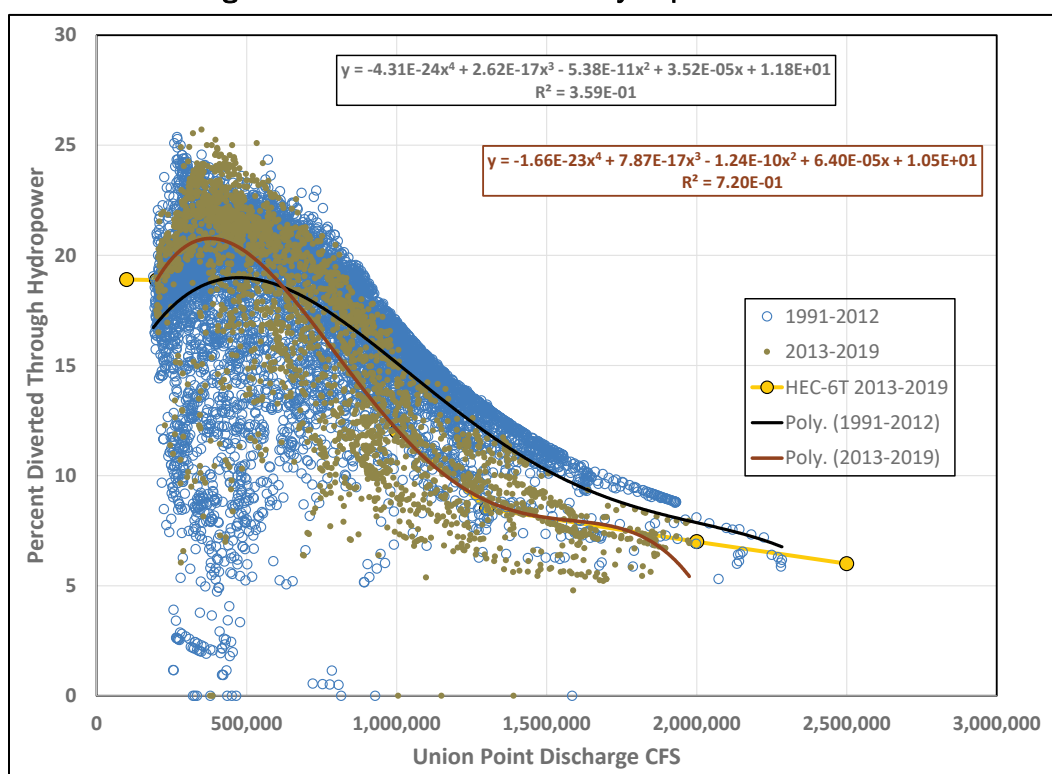


Table 3. Hydropower flow diversion percentages.

Discharge Union Point (cfs)	Diversion Percentages 1991–2012	Diversion Percentages 2013–2019
100,000	17.0	18.9
190,000	17.0	18.9
300,000		20.4
450,000	19.0	20.5
550,000	19.0	19.6
900,000	16.2	
1,000,000		12.1
1,300,000		8.9
1,350,000	11.5	
1,700,000	9.0	
2,000,000		7.0
2,300,000	6.7	
2,500,000	6.0	6.0
3,000,000	5.0	5.0

The Overbank Structure was not used between 1991 and 2012. In addition, it is only used during flood events. To increase the data base, measurements at the Overbank Structure between 1979 and 1990 were incorporated into the regression analysis for the Overbank Structure diversion percentages. The results are shown in Figure 5 and Table 4. There is considerable scatter about the polynomial regression lines. This reflects the operational alternatives available at the structure. Maximum allowable diversion percentage was set at 9.71%. This flow diversion percentages regression curve was used after October 2012 and in the future simulations (between October 1991 and October 2012, diversion percentages were zero).

Figure 5. Flow diverted at ORCC Overbank Structure 1977–2019.

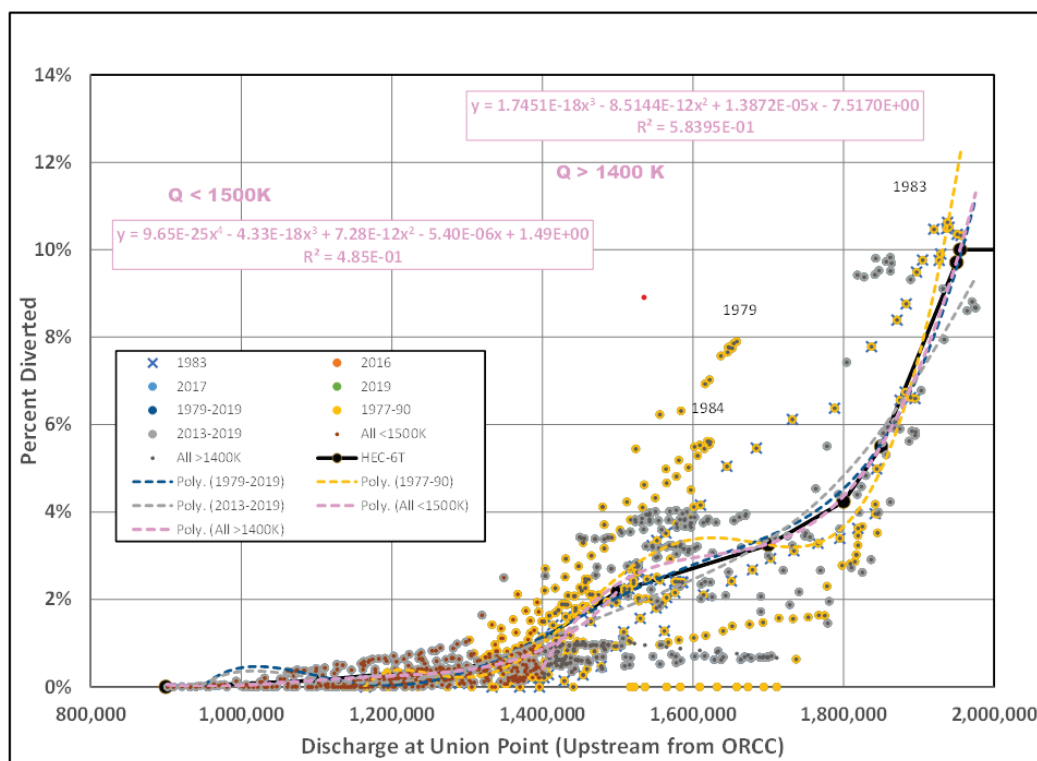


Table 4. Overbank flow diversion percentages.

Discharge Union Point (cfs)	Diversion Percentages
900,000	0
1,100,000	0.15
1,350,000	0.55
1,500,000	2.18
1,700,000	3.25
1,800,000	4.24
1,850,000	5.50
1,950,000	9.71
2,500,000	9.71

The calculated HEC-6T flow diversion percentages regression curves for the combined Low Sill plus Auxiliary were determined using the daily difference in the total ORCC diversion percentages minus the Hydropower and Overbank Structure percentages. There is an extensive scatter about the regression lines (Figure 6). Clearly, there is no distinctive relationship

between discharge and the combined flow through the two structures. The most that can be said is that a representative long-term average is obtained.

When the long-term straight averages of flow distribution through the Low Sill and Auxiliary Structures are applied to the combined regression curves a clear difference is observed between the 1991–2012 and 2013–2019 regression curves (Figure 7) Based on the straight average of 2013–2019 diversions through the Low Sill and Auxiliary Structures, 47% of this difference was assigned to the Low Sill, and 53% was assigned to the Auxiliary. This compared to 80% through the Auxiliary and 20% through the Low Sill used for the 1991–2012 historical simulations. The HEC-6T regression curves are tabulated in Table 5.

Figure 6. Combined Low Sill and Auxiliary flow diverted at ORCC.

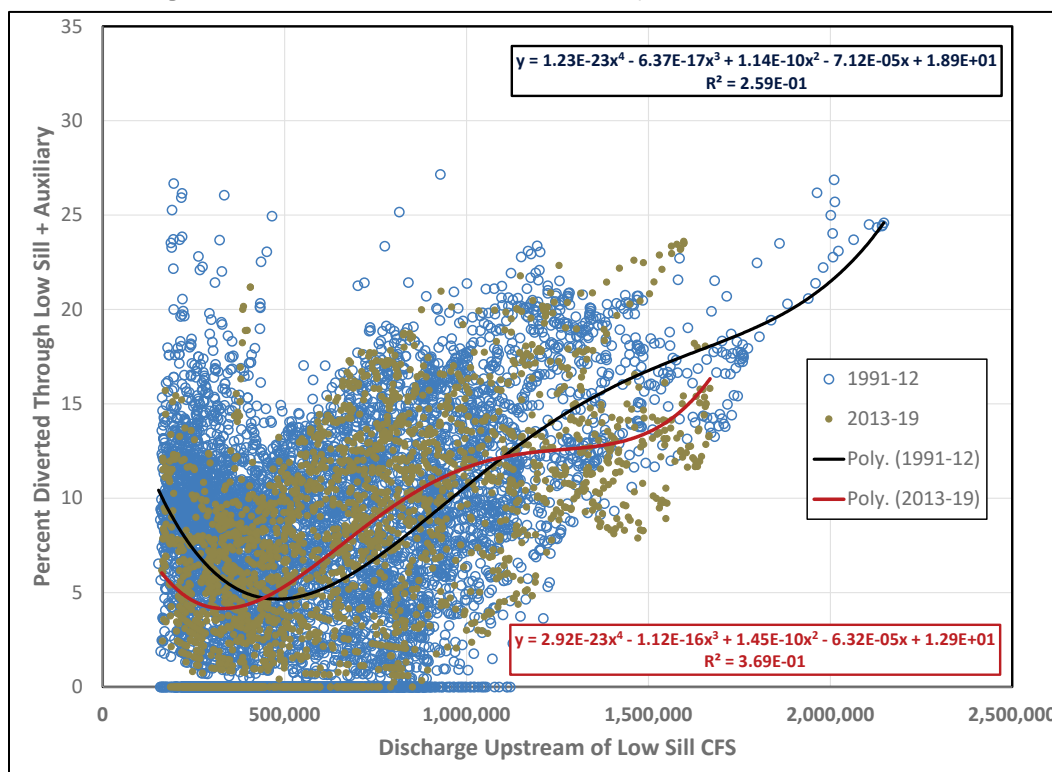


Figure 7. Diversion percentages for Low Sill and Auxiliary Structures.

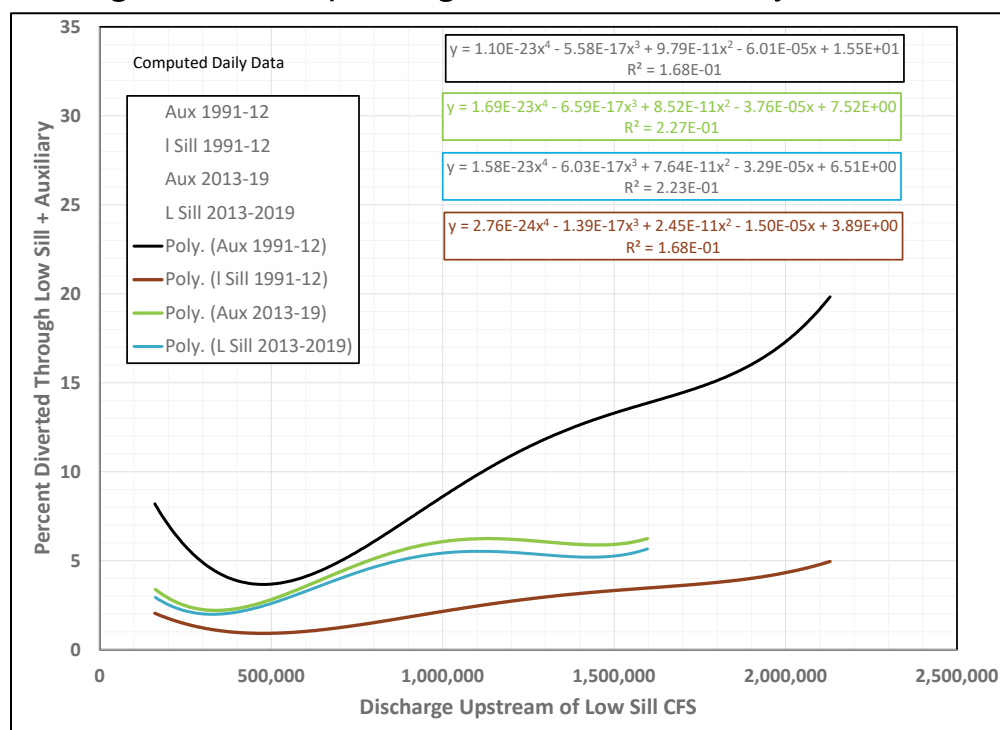


Table 5. Low Sill and Auxiliary diversion percentages.

Discharge CFS Upstream Low Sill	Low Sill Diversion % 1991-2012	Low Sill Diversion % 2013-2019	Auxiliary Diversion % 1991-2012	Auxiliary Diversion % 2013-2019
100,000	1.78	2.10	7.22	2.40
200,000	1.78	2.10	7.22	2.40
350,000	0.99	2.10	4.01	2.40
450,000		2.10		2.40
625,000	0.99		4.01	
900,000		5.13		5.87
1,200,000	2.77	5.83	11.23	6.67
1,350,000	3.12		12.63	
1,450,000		6.06		6.94
1,500,000	3.37		13.63	
1,600,000		6.99		8.01
1,800,000	3.82	8.98	15.43	10.27
2,000,000	4.26	10.03	17.24	11.47
2,500,000	4.26	10.03	17.24	11.47

2.6 Distributaries incorporated from Flowline Model

There are several distributaries from the Lower Mississippi River between Cairo, IL, and the Gulf of Mexico that are included in the HEC-6T model. Many of the distributaries were incorporated into the OMAR Model unchanged from the Flowline Model. Distributaries taken from the Flowline Model are the following:

- New Madrid Floodway, upstream of RM 949
- 2011 Crevasse, upstream of RM 915.6
- Yazoo River Backwater, upstream of RM 444.8
- Morganza Spillway, upstream of RM 277.5
- Devil's Swamp, upstream of RM 244.2
- Manchac Point, upstream of RM 220.9
- Bonnet Carré Spillway, upstream of RM 127.1
- Davis Pond, upstream of RM 117.2
- Caernarvon, upstream of RM 80.3
- Bohemia Spillway upstream of RM 43.2.

Descriptions of the analytical development of flow diversion percentages for these distributaries are contained in the Flowline Assessment report (Copeland 2018).

2.7 Distributaries updated from Flowline Model

2.7.1 Fort Saint Philip and Ostrica

The diversion percentages at Fort Saint Philip, which is between RM 20 and RM 14, and below Ostrica Lock, which is located between RM 25 and RM 22, were included in the OMAR Model. Fort Saint Philip diversion percentages were taken from the Mississippi River Hydrodynamic and Delta Management Study Model (Thomas et al. 2018) using HEC-6T input file, *LMR1_FWOP_NRC1_Final_R19*, for Mississippi River discharges up to 800,000 cfs. These diversion amounts were based on recent measurements including comparisons of United States Geological Survey (USGS) data at Belle Chasse and USACE measurements at Venice. Measurements taken in 2016 by the Water Institute of the Gulf were used for a Mississippi River discharge of 1,250,000 cfs. Without any other available data, the diversion percentage was held constant for discharges exceeding 1,250,000 cfs. The only data available for Ostrica were the 2016

Water Institute measurements. The 2016 measured percentage at Ostrica was used in the model for a discharge of 1,250,000 cfs. The regression curve for lower discharges was determined using a ratio of maximum discharges at Fort Saint Philip and Ostrica. Diversion percentages for Fort Saint Philip and Ostrica are shown in Table 6.

Table 6. Distributary outflow at Fort Saint Philip and Ostrica percentages of discharge at Myrtle Grove (RM 57).

Distributary	River Mile	Discharge (cfs)				
		320,000	400,000	600,000	800,000	1,250,000
Fort Saint Philip	20 to 14	0%	5.48%	8.04%	9.60%	10.84%
Ostrica	25 to 22	0%	1.57%	2.30%	2.75%	3.10%

2.7.2 Flow distributaries between Above Venice and Head of Passes

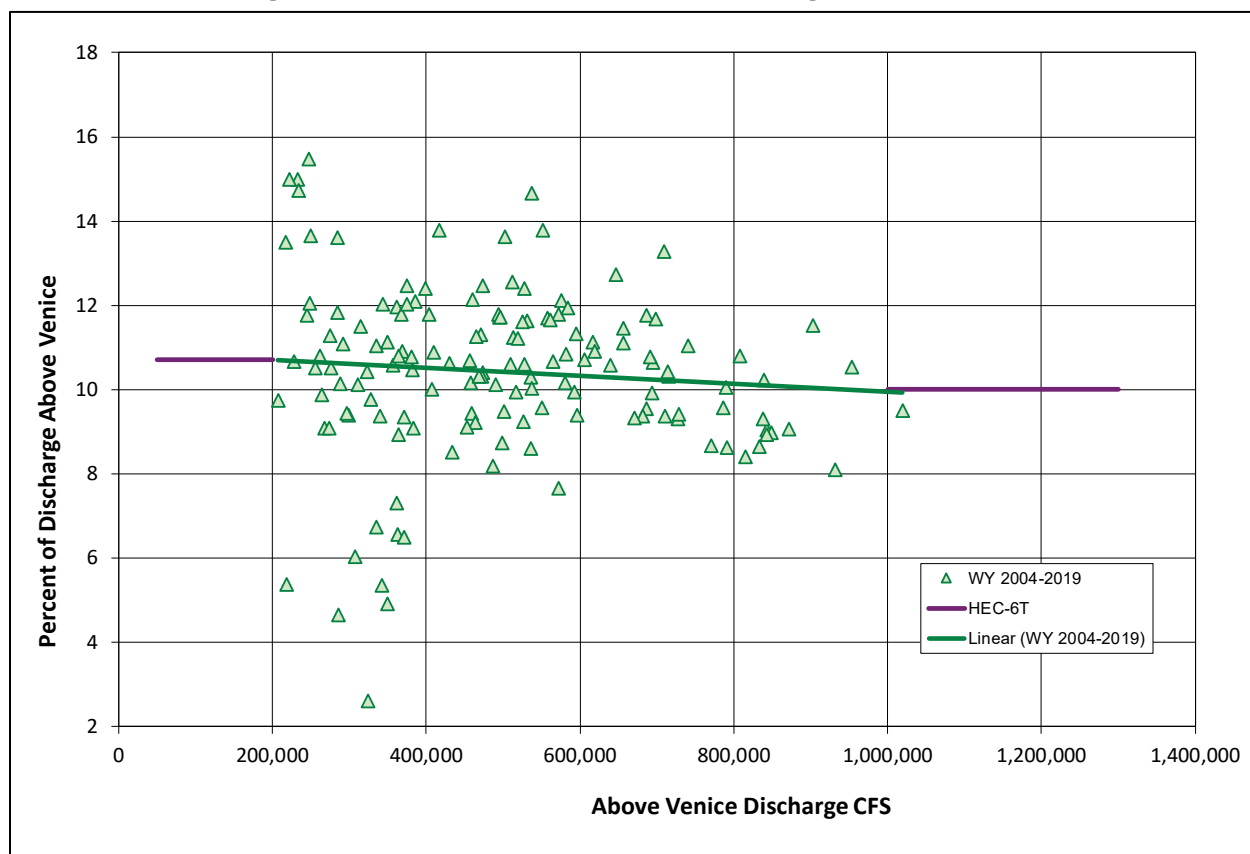
The percentages of Mississippi River flow leaving the river through distributaries between Head of Passes at RM 0.0 and the Above Venice discharge range at RM 11.5 were based on measurements taken by New Orleans District (MVN) between 2004 and 2019.

2.7.2.1 Baptiste Collette

The Baptiste Collette (RM 11.4) flow diversion percentages were modified from those used in the Flowline Model based on additional data collected after 2012. The index station is the Above Venice discharge range. A linear regression of all data collected between water year 2004 and 2019 was used to determine the new flow diversion percentages. A water year begins on October 1 of the previous calendar year and continues through September 30 of the nominal water year. A water year is intended to capture the typical rise and fall of the annual hydrograph in perennial streams and rivers into a single time frame. Data prior to 2004 were not considered due to the infrequency of measurements. The 2004–2019 measurements were checked for continuity using measurements at Above Venice, Above West Bay, Baptiste Collette, and Grand Pass. Only measurements where the continuity error was less than 10% were considered in the analysis. Measurements where the discharge at Above Venice was less than 200,000 cfs were also excluded due to the erratic nature of the reported data at low discharge and the desire to avoid bias in the regression analysis. A total of 147 measurements were used to develop

the final flow diversion percentages. At Baptiste Collette, there were significant variations about the average flow diversion percentages with time. The new 2013–2019 measurements indicated a slightly lower diversion percentages than the 2004–2012 measurements. In addition, a linear regression of the 2004–2019 measurements indicated a slight decrease in diversion percentages with increasing discharge, whereas a linear regression of the 2004–2012 measurements had suggested a slight increase in diversion percentages with increasing discharge. It was determined that the best estimate for long-term average diversion percentages at Baptiste Collette would be achieved using the entire 2004–2019 record. Diversion percentages through Baptiste Collette are plotted against the Above Venice discharge in Figure 8. This figure also shows the diversion percentages used in the HEC-6T model. When Above Venice discharges are less than 200,000 cfs, 10.7% of the flow is diverted through Baptiste Collette. At discharges greater than 1,000,000 cfs, 10% is diverted. Percentages are linearly interpolated between 200,000 and 1,000,000 cfs. This new rating curve was used for both the calibration run (1990–2019) and the 50 yr simulation.

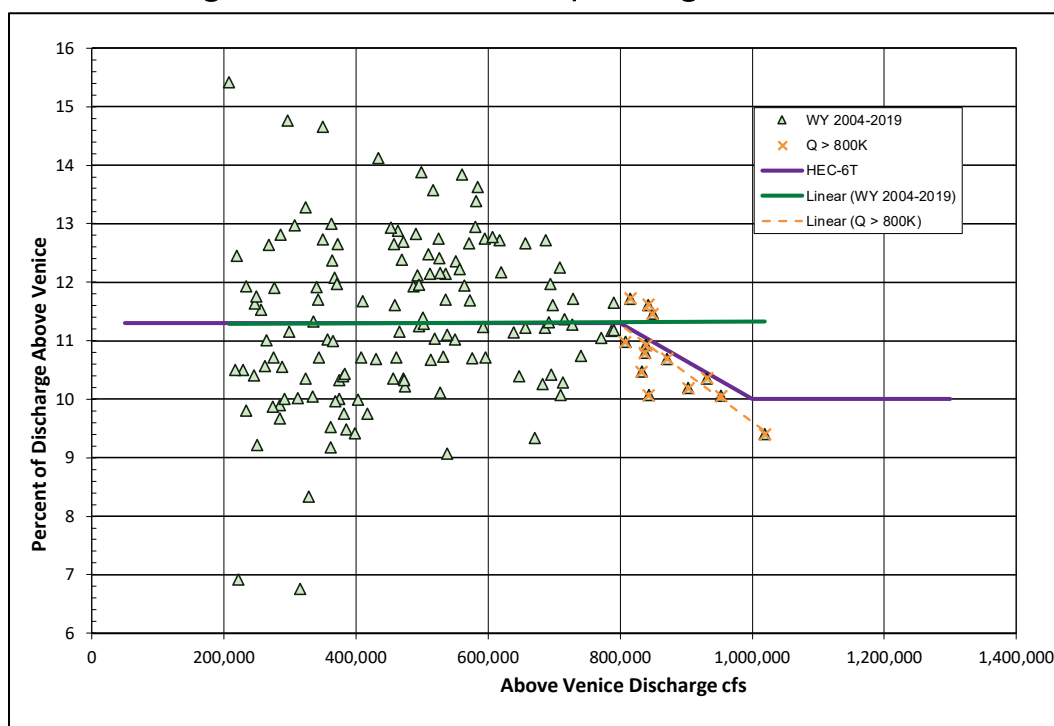
Figure 8. Baptiste Collette Diversion percentages 2004–2019.



2.7.2.2 *Grand Pass*

The Grand Pass (RM 10.4) flow diversion percentages were modified from those used in the Flowline Model based on additional data collected after 2012. The index station is the Above Venice discharge range. A linear regression of all data collected between water year 2004 and 2019 was used to determine the new flow diversion percentages. Data prior to 2004 were not considered due to the infrequency of measurements (3 between 1979 and 1984, 3 between 1989 and 1991, and 10 between 1996 and 2003). The 2004–2019 measurements were checked for continuity using measurements at Above Venice, Above West Bay, Baptiste Collette, and Grand Pass. Only measurements where the continuity error was less than 10% were considered in the analysis. Measurements where the discharge at Above Venice was less than 200,000 cfs were also excluded due to the erratic nature of the reported data and the desire to avoid bias in the regression analysis. A total of 147 measurements were used to develop the final flow diversion percentages. Although there was significant variation about the average flow diversion percentages with time, no significant trend with time was identified using the 2004–2019 measurements. Similarly, there was no significant variation about the average flow diversion percentages with discharge when the discharge was less than 800,000 cfs. At discharges greater than 800,000 cfs there appeared to be a decrease in the flow diversion percentages as discharge increases. This conclusion is based on only a small number of measurements. Diversion percentages are plotted against discharge in Figure 9. This figure also shows the diversion percentages used in the HEC-6T model. At discharges less than 800,000 cfs, 11.3% of the Above Venice discharge is diverted through Grand Pass. At discharges greater than 1,000,000 cfs, 10% of the Above Venice discharge is diverted. Percentages are linearly interpolated between 800,000 and 1,000,000 cfs.

Figure 9. Grand Pass Diversion percentages 2004–2019.



2.7.2.3 West Bay

MVN has taken discharge measurements in the Mississippi River above West Bay Diversion and at Pilottown (below Cubits Gap) since December 2003. On average, approximately 11 measurements are made each year. Diverted discharges through West Bay and Cubits Gap were measured on the same day as the Mississippi River discharges. The first reported measured discharge at West Bay occurred 16 January 2004. Between October 1979 and December 2003, an additional 22 measurements are available at Cubits Gap. The index station in the HEC-6T model is the cross section directly upstream from the West Bay Diversion.

Continuity checks of the measurements were made to ensure that the measurements maintained continuity to within 10%. The continuity check compared the sum of measured discharges through West Bay Diversion, Cubits Gap and at Pilottown to the measured discharge from Above West Bay. The difference was divided by the Above West Bay discharge to determine the percentage error. If measurements were not taken at all four stations, continuity could not be determined and measurements for that day were not included in the analysis. Four measurements with negative discharge through Cubits Gap and two outliers were also

excluded from the analysis. Approximately 14% of the measurements were deleted due to continuity issues or negative flow in the distributaries.

The West Bay Diversion was open 26 Oct 2003, and the first available measurement was 16 Jan 2004. Diverted discharges through West Bay, as a percentage of Mississippi River discharges immediately upstream from West Bay, between water year 2004 and 2019, are shown in Figure 10. This figure shows that for the first 5 yr, diversion percentages steadily increased. Measured discharge percentages varied significantly for the next 5 yr and then seem to have stabilized after water year 2012. Data were insufficient to define a relationship between Mississippi River discharge and the diversion percentages through West Bay prior to water year 2008. Consequently, straight averages were calculated for four time periods and used in the historical simulation. Subsequent to 2008, the measured data as shown in Figure 11 suggest that diversion percentages are slightly less at high discharges than at the lower discharges. 2008–2019 West Bay discharge percentages were varied as a function of Mississippi River approach discharges in HEC-6T in the historical simulation after October 2007 and in the 50 yr future simulation. The Diversion percentages at West Bay used in HEC-6T are shown in Table 7.

Figure 10. Flow Diversion percentages at West Bay 2004–2019.

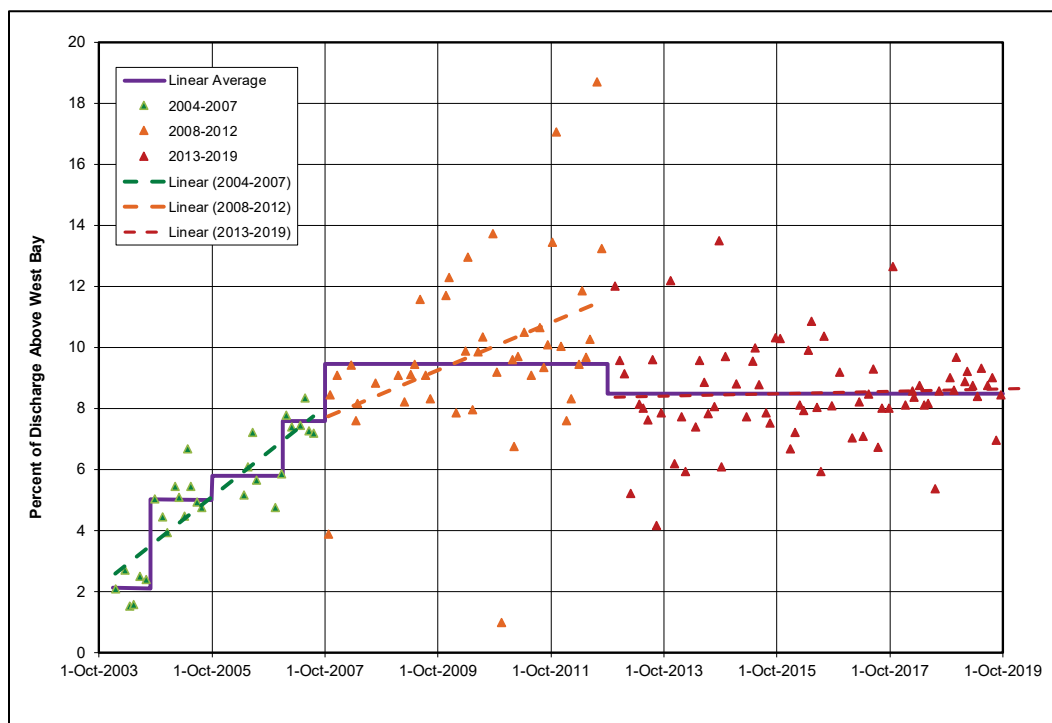


Figure 11. West Bay Diversion percentages – Variation with discharge 2008--2019

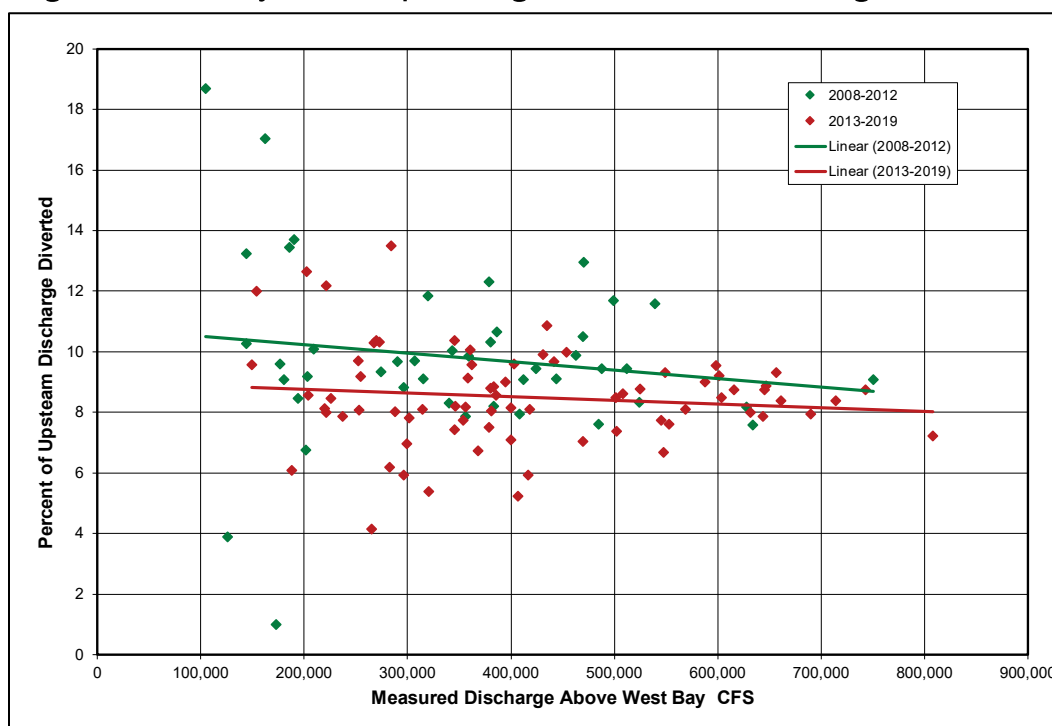


Table 7. Diversion percentages used in HEC-6T at West Bay.

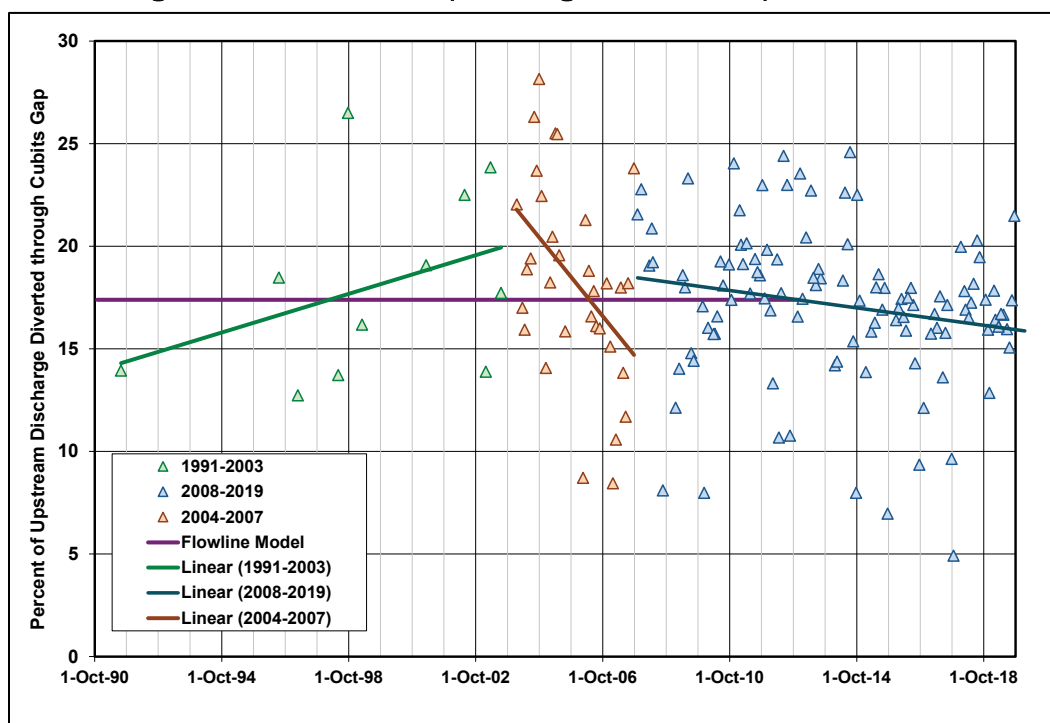
Discharge, cfs	50,000	150,000	800,000	1,500,000
Before 26 Oct 04	0	0	0	0
26 Oct 04 – 31 Aug 04	2.1	2.1	2.1	2.1
1 Sep 04 – 30 Sep 05	5.0	5.0	5.0	5.0
1 Oct 05 – 31 Dec 06	5.8	5.8	5.8	5.8
1 Jan 07 – 30 Sep 07	7.6	7.6	7.6	7.6
1 Oct 07 – 30 Sep 12	10.5	10.5	8.5	8.5
After 1 Oct 12	9.0	9.0	8.0	8.0

2.7.2.4 Cubits Gap

The discharge diversion percentages through Cubits Gap may have varied somewhat with time (Figure 12). Between 1991 and 2003, an average linear regression line suggests that diversion percentages were increasing. However, there were only 11 measurements during these 13 yr, and data scatter was significant. West Bay Diversion, which is located on the opposite bank just upstream from Cubits Gap, was open in water year 2004, and an average linear regression line of the measured diversion percentages indicated a decrease in the percentage diverted at Cubits Gap.

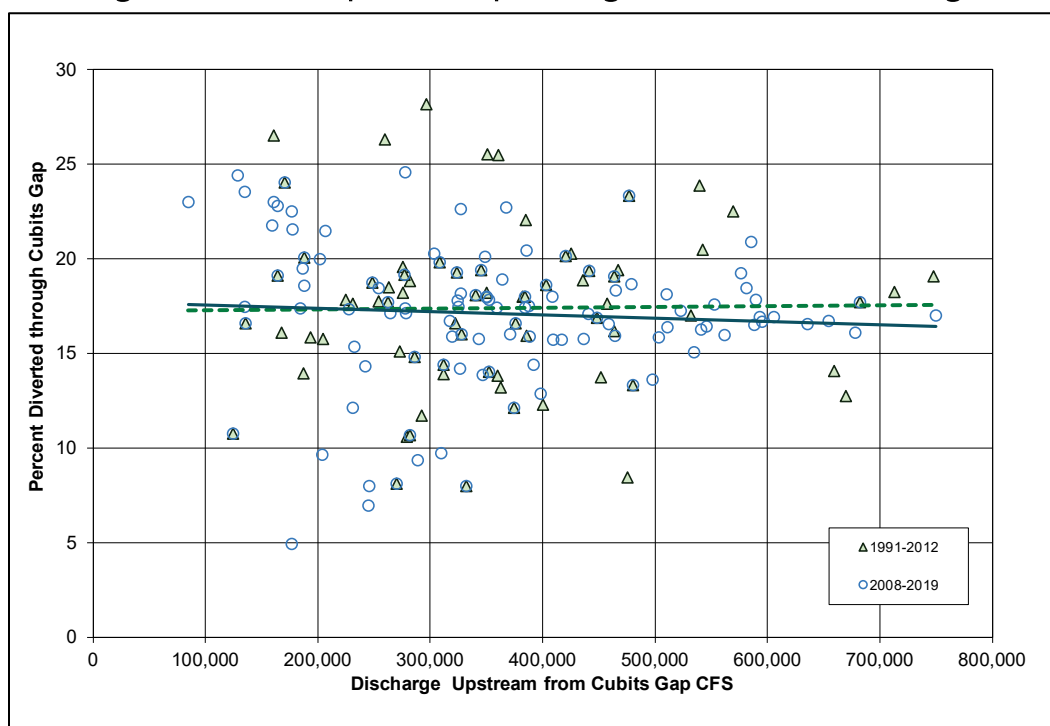
After West Bay diversions stabilized in water year 2008, the diversion percentages at Cubits Gap tended to be more stable with a slight downward trend. The Flowline Model, which utilized data through 2012, concluded that the data scatter about the linear regression lines was too great to define a reliable trend with time. The additional 9 yr of data available for the OMAR Assessment did not change that conclusion.

Figure 12. Flow Diversion percentages at Cubits Gap 1991–2019.



The measured diversion percentages at Cubits Gap, as a function of discharge approaching Cubits Gap, are plotted in Figure 13. The index station in the HEC-6T model is the cross section directly upstream from Cubits Gap. In the Flowline Model, using data through 2012, a constant diversion percentage of 17.4 was assigned at Cubits Gap. This diversion percentage was used in this study for the historical simulations through 2008. After 2008, a diversion percentage of 17.4 was assigned for approaching discharges less than 100,000 cfs and 16.4% was assigned for discharges greater than 750,000 cfs. Linear interpolation was used between these two values. The 50 yr future simulations used the post-2008 rating curve.

Figure 13. Cubits Gap Diversion percentages – Variation with discharge.



2.7.2.5 Overbank flow and minor outlets

At high discharges, flow leaves the Mississippi River over the natural levees and through minor outlets. Previous HEC-6T studies have identified these outflows as Leaks and assigned the outflow at RM 2.46. The Leaks diversion percentages were developed from a mass continuity analysis of measurements in the Mississippi River Above Venice and at Pilottown and in the major diversions in-between. The outflow at Leaks was assumed to be the missing variable that balanced the continuity equation. The regression curve used in the most recent HEC-6T studies considered zero outflow through Leaks until the upstream discharge reached 600,000 cfs. The diversion percentages increased linearly to 3.3% at 800,000 cfs and 10.9% at 1,000,000 and above

Data between 2004 and 2019 were used to develop a new Leaks regression curve for the OMAR Model. The 10% continuity checks applied to the West Bay and Cubits Gap measurements were also applied to the Leaks analysis. Negative percentages were included in the continuity analysis to account for field measurement errors. The analysis showed some discharge through Leaks even at low Mississippi River discharges. This is attributed to the minor outlets. The new rating curve has 2.4% outflow for discharges less than 350,000 cfs, increasing linearly to 6.5% at a discharge of

650,000 cfs and beyond (Figure 14). The cross section immediately upstream from the Leaks distributary is the index station.

Figure 14. Leaks Diversion percentages 2004–2019.



2.7.3 Head of Passes

MVN has been collecting sets of discharge measurements between Head of Passes and Above Venice (RM 11.5) since 1979. Since 2004, these measurements have been made monthly. The data sets typically include measured discharges in the Mississippi River at Above Venice, Above West Bay, and Pilottown. Measurements are also taken in Pass a Loutre, South Pass, Southwest Pass, Cubits Gap, West Bay, Grand Pass, and Baptiste Collette.

OMAR Model runs that included the historical hydrograph (1991–2013), used the Flowline Model distribution percentages at Head of Passes. These account for the increasing flow through the West Bay diversion with time. For future runs in the OMAR Model, flow distributions at Head of Passes were based on recent MVN measurements from water years 2008–2019. Water year 2008 was chosen as the starting point because by that time the West Bay Diversion appeared to have stabilized in terms of diversion

percentages (see Figure 9 in Flowline Model report [Copeland 2018]). Measurements were taken at Pilottown (RM 2) and at the head of the three passes. Only measurements where continuity error was less than 10% were considered. There were 108 such measurements. At flows above 200,000 cfs the average percentages through each of the three distributaries were essentially constant (within 3%). At lower flows, the discharge through Southwest Pass decreases dramatically, and the diversion percentages increase in Pass a Loutre and South Pass. The Pilottown discharge measurements ranged between 80,000 and 600,000 cfs. Distributary diversion percentages are shown in Table 8. At discharges beyond the range of discharges shown in the table, distribution percentages were held constant (i.e., data were not extrapolated).

Table 8. 2008–2019 HEC-6T Distributary distribution percentages at Head of Passes.

Discharge at Pilottown RM 2.0 (cfs)	20,000	80,000	200,000	600,000	1,000,000
Pass a Loutre	32.9	32.9	18.1	16.9	16.9
South Pass	32.3	32.3	16.0	14.6	14.6
Southwest Pass	34.8	34.8	65.9	68.5	68.5

2.7.4 Southwest Pass

The flow diversion percentages in Southwest Pass reach remain unchanged from the original Regional Model. The flow percentages in Burrwood Bayou were based on nine measurements taken in 2003–2005. The data showed no apparent trend between the diversion percentages and the Southwest Pass discharge, so a constant value was used. The flow diversion percentages at the other distributaries were assumed to vary directly with the ratio of the width to the width at Burrwood Bayou (continuity principle). Southwest Pass diversion percentages used in the Regional Model and this model are shown in Table 9.

Table 9. Outflow from Southwest Pass distributaries.

Distributary	RM	Discharge Upstream from Distributary (cfs)			
		10,000	300,000	400,000	500,000
Burrwood Bayou	-14.4	12%	12%	12%	12%
Outlet W-2 and Overbank Flows	-9.8	7%	7%	10%	19%
Joseph Bayou	-4.5	12%	12%	12%	12%
Outlet at RM -3.0	-3.0	4%	4%	4%	4%

2.8 Temperature

Water temperature used in the OMAR Model was taken from average monthly temperatures used in the Flowline Model. These were based on 1965–2014 reported data from the USGS. Monthly averages were calculated from the observed data and assigned to the fifteenth of each month in the computational hydrograph. Mean monthly temperatures assigned to fifteenth of each month are shown in Table 10. In the HEC-6T hydrograph, daily values were assigned based on linear interpolation. Water temperatures were assigned at each upstream boundary. The computer code calculates a discharge weighted water temperature at confluences. The following gage data were used to determine the average monthly temperatures:

- Mississippi River at Thebes, IL (USGS 07022000) 1973–2013
- Ohio River at Dam 53 near Grand Chain, IL (USGS 03612500) 1973–2013
- Obion River at Highway 51 near Obion, TN (USGS 07026040) 1990–2005
- Obion River at Obion, TN (USGS 07026000) 1975–1990
- Hatchie River at Rialto, TN (USGS 07030050) 1977–2008
- Hatchie River at Bolivar, TN (USGS 07029500) 1975–2006
- Saint Francis River at Madison, AR (USGS 07047907) 1977–2009
- White River at Clarendon, AR (USGS 07077800) 1965–1986
- White River at DeValls Bluff, AR (USGS 07077000) 1967–2014
- Arkansas River at Murray Dam near Little Rock, AR (USGS 07263450) 1975–1994
- Arkansas River at David D Terry Lock and Dam below Little Rock, AR (USGS 07263620) 1969–2014
- Arkansas River at Dam No. 2 near Gillett, AR (USGS 07265283) 1972–1994
- Yazoo River at Redwood, MS (USGS 07288800) 1972–1993
- Yazoo River below Steele Bayou near Long Lake, MS (USGS 07288955) 1994–2014.

Table 10. Average monthly water temperature 1965–2014 degrees Fahrenheit.

	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
Middle 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
Saint 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

2.9 Downstream water-surface elevations

The National Oceanic and Atmospheric Administration (NOAA) tide gage at Grand Isle, LA (8761724), was used to set the downstream water-surface elevation for the OMAR Model. 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 sea-level rise and/or subsidence effects, adjusted mean monthly water-surface elevations were calculated. This was accomplished by increasing the magnitude of the older measurements so that a linear regression line through the adjusted data had a zero slope. The resultant adjusted mean monthly water-surface elevations represent 2014 conditions. The reported and adjusted tide elevations are shown in Figure 15. 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 fifteenth of each month. Downstream water-surface elevations assigned between 1991 and 2019 are shown in Table 11. NOAA water-surface elevations relate to mean sea level, which was translated to NGVD29 by adding 0.8 ft.

Figure 15. Mean monthly tide elevations at Grand Isle, LA, 1978–2014.

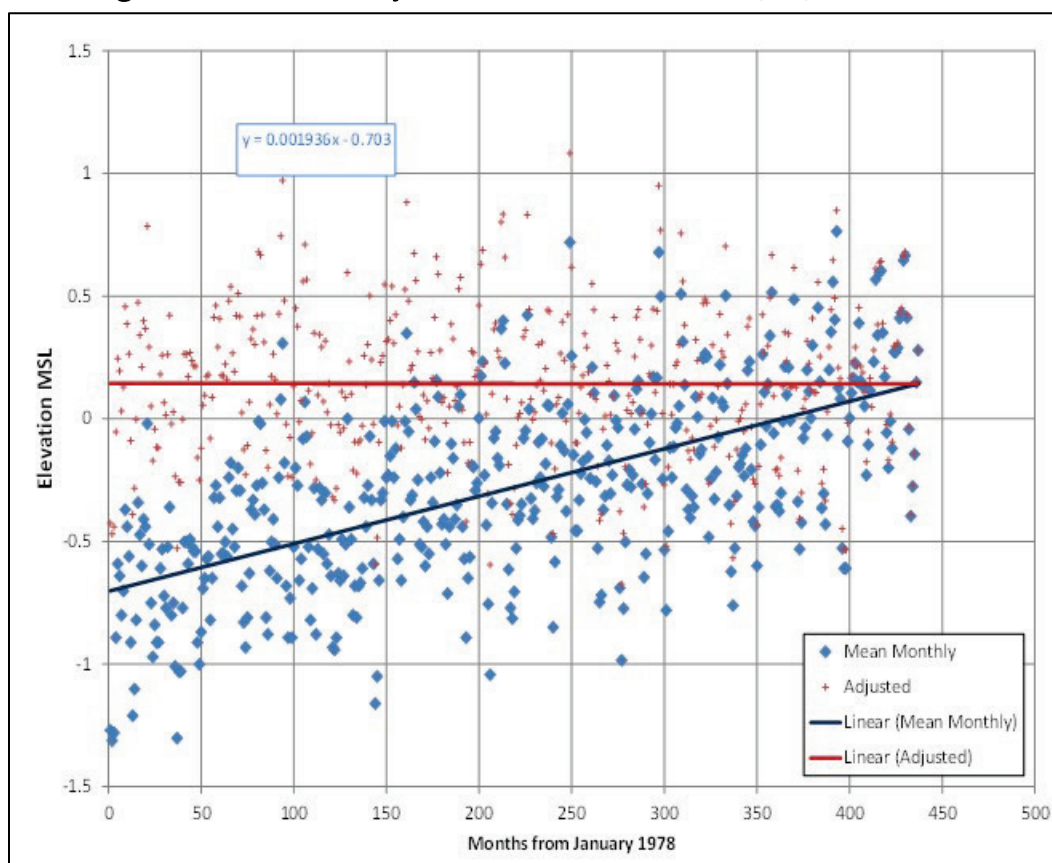


Table 11. Downstream water-surface elevations.

Month	Feet NGVD	Month	Feet NGVD
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

Sea level rise and subsidence were not included in the 1991–2019 historical simulations in the OMAR Model. For the 50 yr future simulations, which begin 1 October 2019 and continue for 50 yr, the downstream water-surface boundary condition used the intermediate sea level rise scenario from ER 1100-2-8162 (USACE 2019). The equation in the appendix of ER 1100-2-8162 is applied to each day after 1 October 2019 to calculate the downstream water surface using the recommended base year 1992. Calculated increase due to sea-level rise was added to the average monthly values from

Table 11. After 50 yr, the predicted sea level rise was 1.36 ft. Subsidence was not included in the 50 yr future simulations.

2.10 Roughness coefficients

Calibration of roughness coefficients assigned in the Flowline Model included stage-discharge gage data from the 2011 flood. However, in MVN, 2011 flood elevations were generally not higher than historical floods due to operation of the Morganza and Bonnet Carré Spillways, and no additional calibration was performed. As part of the deep-draft crossings investigation (Copeland et al. 2019), recent data were used to re-calibrate roughness coefficients in the MVN. The differences from previous assignments, which had used 1991–2002 data, were small. Comparisons of MVN calculated and measured water-surface elevations at several gages with the revised roughness coefficients are shown in Figure 16 through Figure 23. The revised roughness coefficients for MVN used in the OMAR Model are shown in Table 12. Additional discussion related to the roughness coefficient calibration, including comparisons of measured and computed water-surface elevations, is contained in the Flowline Model report (Copeland 2018).

Figure 16. Measured and calculated stage at Red River Landing 1991–2019.

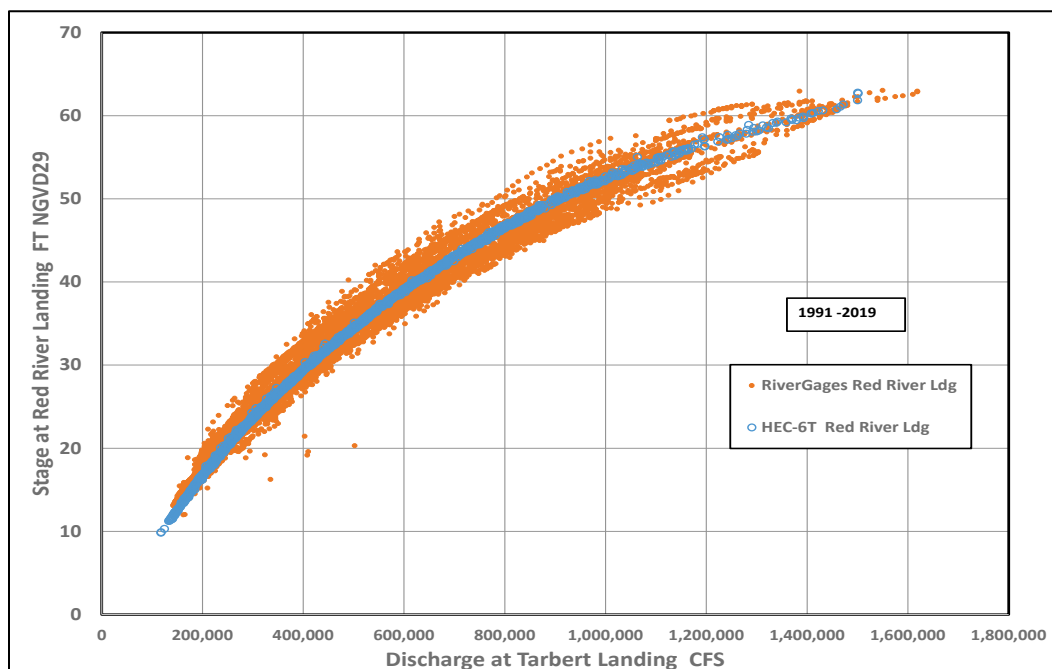


Figure 17. Measured and calculated stage at Bayou Sara 1991–2014.

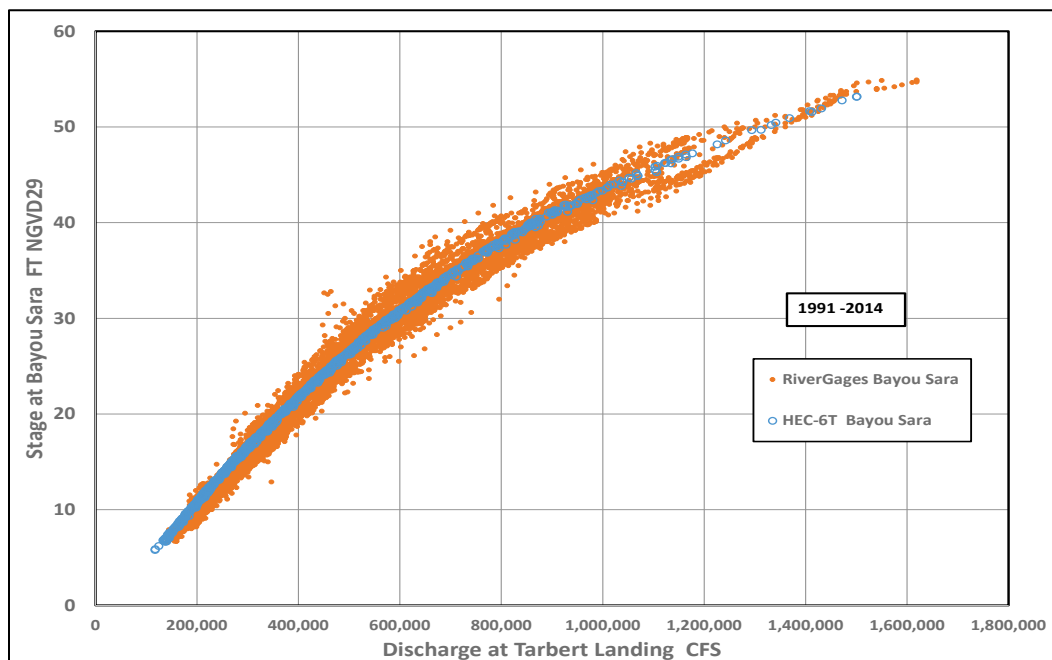


Figure 18. Measured and calculated stage at Baton Rouge 1991–2019.

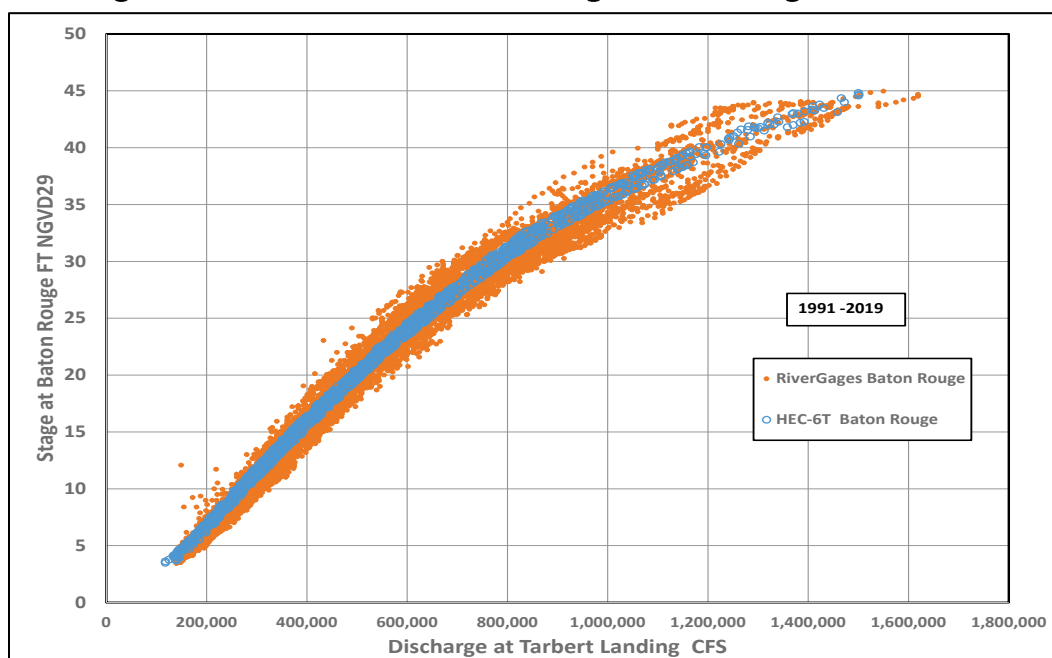


Figure 19. Measured and calculated stage at Donaldsonville 1991–2019.

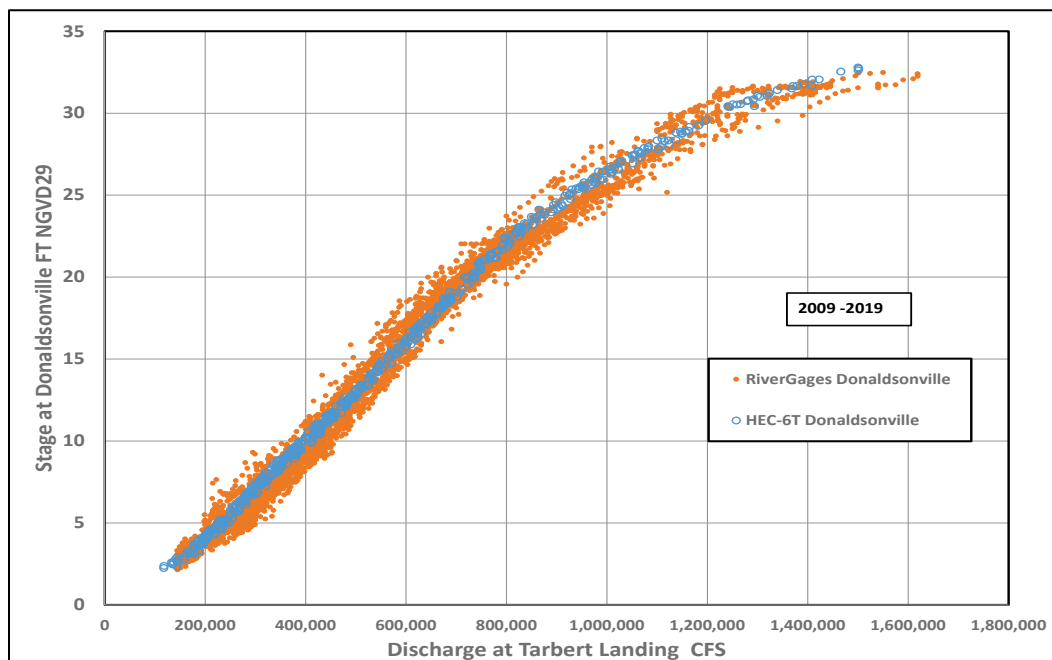


Figure 20. Measured and calculated stage at Reserve 1991–2019.

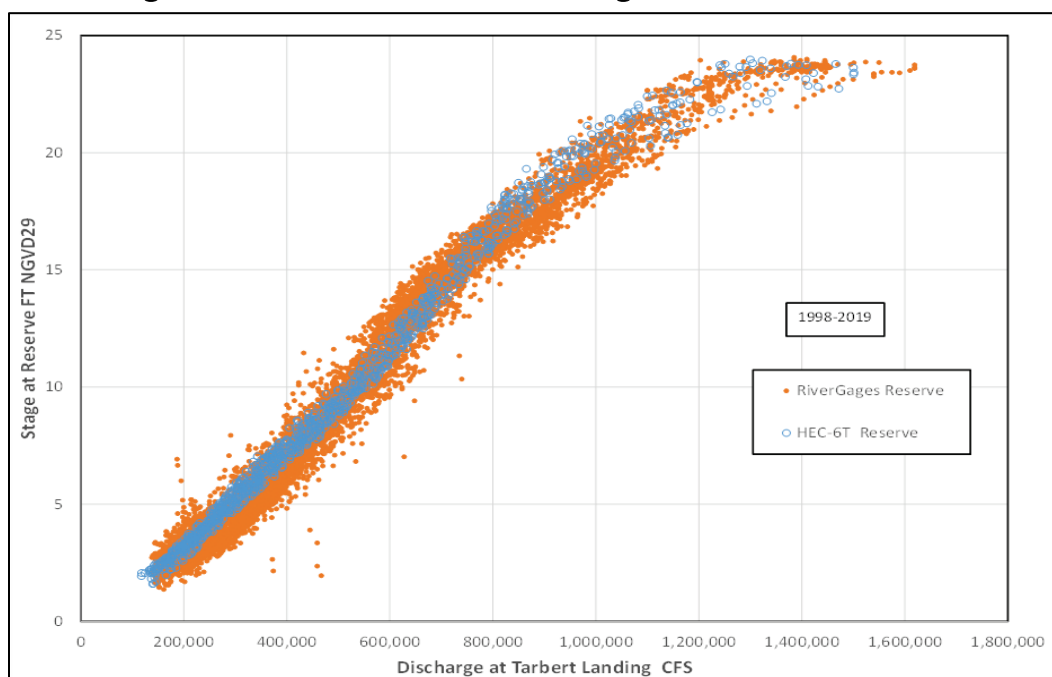


Figure 21. Measured and calculated stage at New Orleans 1991–2019.

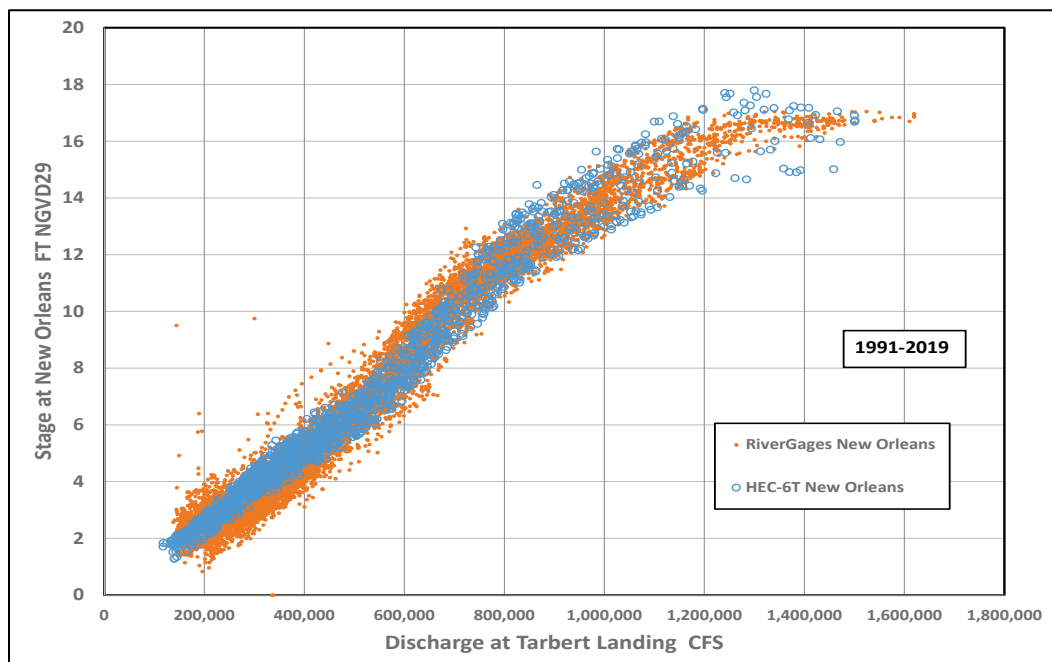


Figure 22. Measured and calculated stage at West Pointe a la Hache 1991–2019.

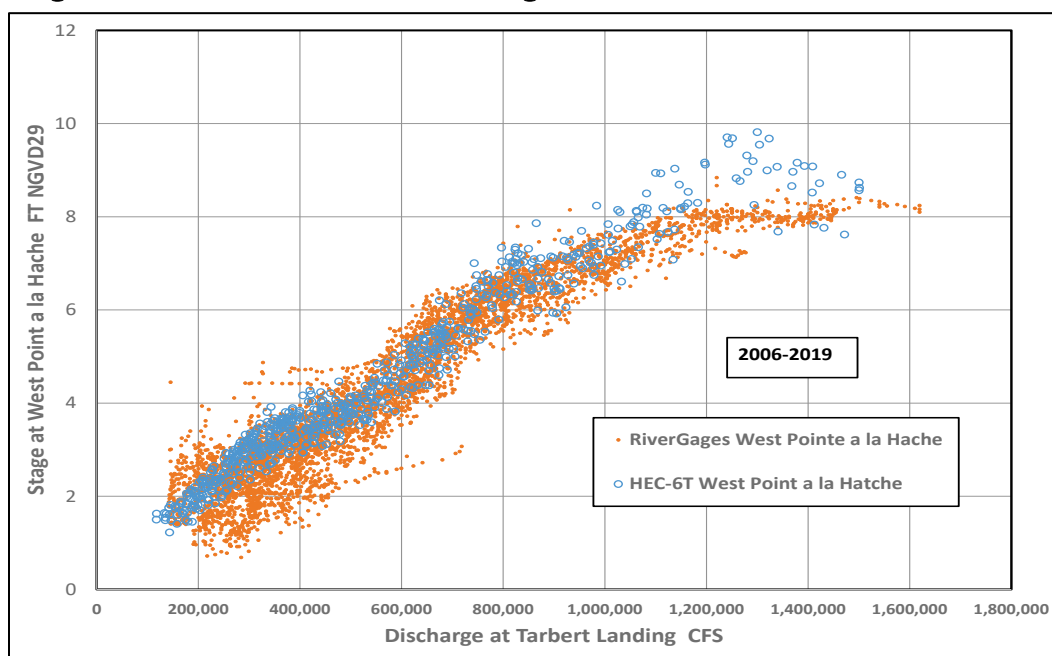


Figure 23. Measured and calculated stage at Venice 1991–2019.

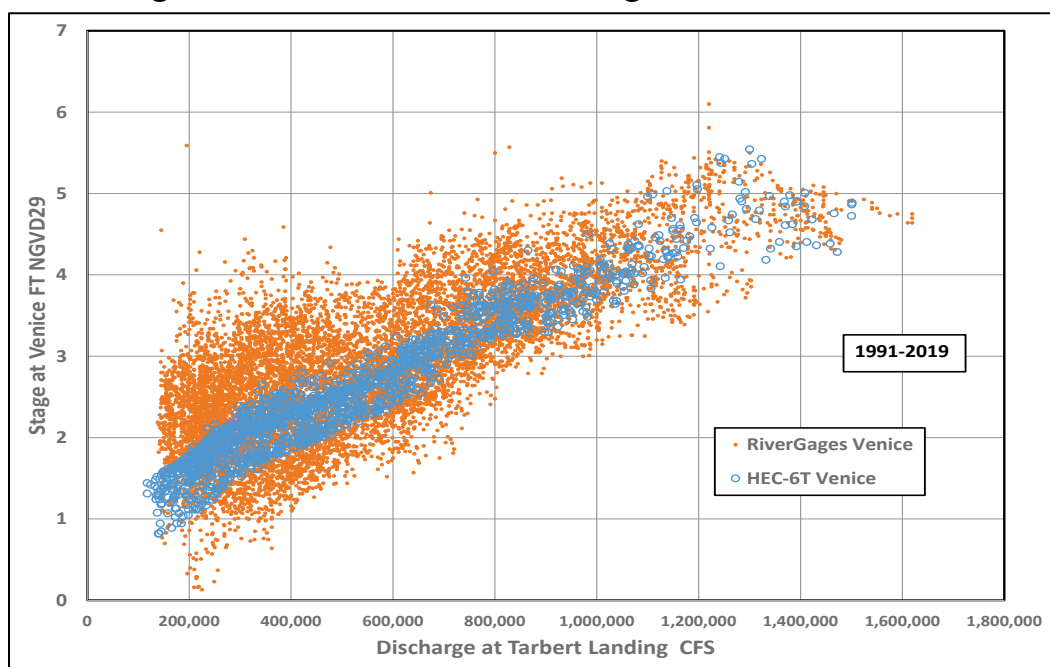


Table 12. HEC-6T roughness coefficients in MVN.

River Mile	Overbanks	Channel - 1000 cfs				
		70	120	180	240	300
-22.0	0.03	0.038	0.028	0.022	0.022	0.018
-18.0	0.05	70	120	180	240	300
		0.038	0.028	0.022	0.022	0.018
		190	320	450	580	710
0.0	0.15	0.038	0.028	0.024	0.024	0.018
		250	500	800	900	1,200
11.05	0.20	0.024	0.016	0.016	0.014	0.012
toffaleti	0.20	200	650	850	950	1,300
		0.026	0.024	0.024	0.023	0.0185
105.0	0.20	275	650	950	1,200	1,500
		0.029	0.026	0.025	0.0215	0.021
141.6	0.20	275	550	950	1,200	1,500
		0.027	0.027	0.024	0.023	0.022
177.3	0.20	0.028	0.028	0.025	0.024	0.022

Table 12. (cont.) HEC-6T roughness coefficients in MVN.

River Mile	Overbanks	Channel - 1000 cfs				
229.7	0.20	0.027	0.027	0.025	0.024	0.024
267.0	0.20	0.031	0.030	0.028	0.026	0.023
		275	550	950	1,200	2,000
306.3	0.20	0.031	0.030	0.028	0.026	0.023
		200	400	900	1,400	2,000
317.3	0.20	0.032	0.028	0.028	0.026	0.024

2.11 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 with Mississippi River data. The equation includes the suspended bed material load and the bed load. However, its formulation did not include gravel bed data. The Meyer-Peter Muller equation was developed for gravel transport. The equation provides for the transport of gravel size classes known to be in the riverbed in the Memphis District (MVM) reach.

2.12 Silt and clay transport functions

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). 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 the following:

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

where

- M_1 = erosion rate for particle scour
[τ_b *ERME/(STME-STCD)] (STCD < τ_b < STME)
- τ_b = bed shear stress
- 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 (1962) equation for mass erosion is as follows:

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

where

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

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 as follows:

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

$$k' = \frac{\omega \left(1 - \frac{\tau_b}{\tau_d} \right)}{2.3 D}$$

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.

Silt and clay deposition coefficients were varied in the model. One set of coefficients was used in Southwest Pass, another in the reach between Head of Passes and RM 11.05, and another upstream from RM 11.05. Varying these coefficients was deemed reasonable to account for the effects of salinity on sediment deposition. Silt and clay coefficients were determined during model calibration and are shown in Table 13.

The OMAR Model deposition coefficients for Southwest Pass and above Head of Passes to RM 11.05 are different from those used in two early numerical sedimentation studies of the Lower Mississippi River (Copeland 1991; Copeland and Thomas 1992). Those earlier coefficients had also been used in the Regional and Flowline Models. The use of the new fine sediment coefficients was assigned as part of the calibration process to better simulate both the volume and location of shoaling above Head of Passes.

Table 13. Silt and clay coefficients for Parthenaides and Krone (1962) equations.

Coefficient	Variable Name in HEC-6T	Southwest Pass	River Miles 0.0 to 11.05	Fresh Water	Units
Shear threshold for clay deposition	DTCL	.012	.008	.001	lbs/ft ²
Shear threshold for silt deposition	DTSL	.020	.018	.002	lbs/ft ²
Shear threshold for erosion of silt and clay particles	STCD	.030	.025	.025	lbs/ft ²

Table 13. (cont.) Silt and clay coefficients for Parthenaides and Krone (1962) equations.

Coefficient	Variable Name in HEC-6T	Southwest Pass	River Miles 0.0 to 11.05	Fresh Water	Units
Shear threshold for mass erosion	STME	.04	.04	.04	lbs/ft ²
Erosion rate at STME	ERME	.001	.001	.001	lbs/ft ² /hr
Slope of the erosion rate curve for mass erosion	ER2	.03	.03	.03	1/hr

2.13 Sediment inflow

A combination of measured and calculated data was 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 suspended size class sediment data are available in the study reach, at the upstream boundary on the Middle Mississippi River at Thebes, located 43.7 mi upstream from the Ohio River confluence, and Chester, located 109.9 mi upstream from the Ohio River confluence. Long-term suspended size class data are also available at Union Point (RM 326.6) and Coochie (RM 317.3), which are located upstream from the ORCC and at Tarbert Landing (RM 306.3), which is located downstream from the ORCC. Data from both of these gages were used to estimate suspended sediment size class distributions for tributary boundaries where size class data were unavailable. Data for the major tributaries, including the Ohio River, were generally limited to the suspended sediment concentrations of sediment with mean diameters greater and less than 0.062 mm, but lacked individual size class definitions. 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 tributary contributory percentages to the Mississippi River water and sediment yields above ORCC for the years 1991–2019, calculated by HEC-6T are shown in Table 14. 83.2% of the water yield and 89.5% of the total sediment from the tributaries upstream from ORCC come from the Ohio and Middle Mississippi Rivers. Interestingly, 81.2% of

the sand but only 38.1% of the water came from the Middle Mississippi River. Boundary conditions for the Ohio and Middle Mississippi Rivers are by far the most important in the model study.

Table 14. 1991–2019 percentages contribution from tributaries upstream from ORCC.

	Percentage of Water Yield	Percentage of Total Sediment Yield	Percentage of Sand Yield
Ohio River	45.1	23.4	11.9
Middle Mississippi River	38.1	66.1	81.2
Arkansas River	7.6	3.4	4.8
White River	4.6	1.6	0.8
Yazoo River	2.5	2.6	0.2
Saint Francis River	1.1	0.7	0.2
Hatchie River	0.5	0.4	0.1
Obion River	0.5	1.8	0.8

New tributary sediment-inflow rating curves were developed for the OMAR Model, incorporating new data between 2013 and 2019. These rating curves are discussed in the following paragraphs.

2.13.1 Middle Mississippi River

Over 80% of the sand yield at the ORCC comes from the Middle Mississippi River, making this boundary the most significant in the numerical model. A combination of measured and calculated sediment data was used to develop the inflowing sediment rating curves.

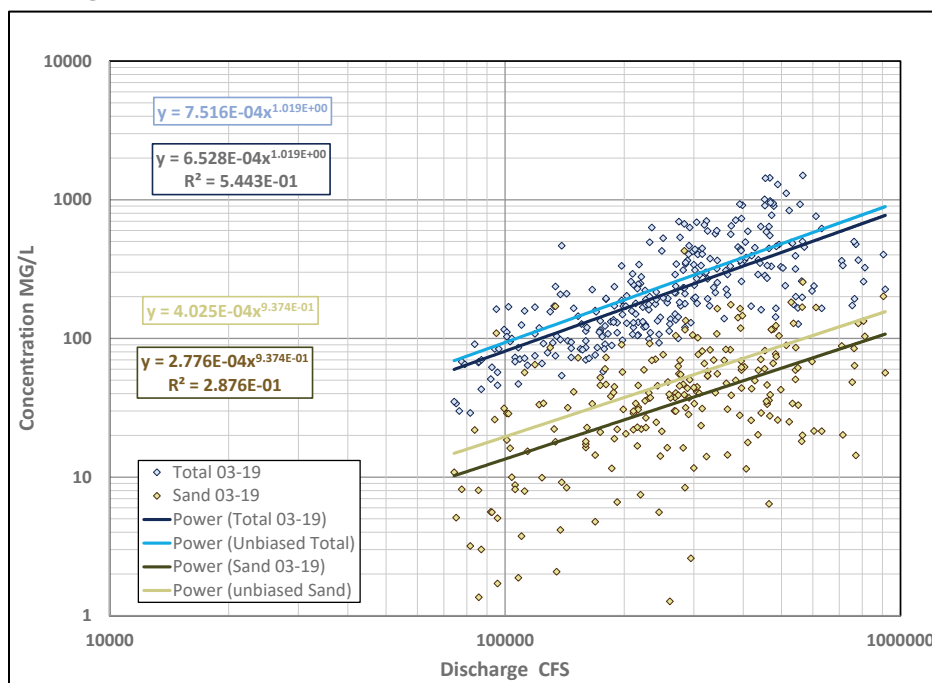
Suspended sediment data were available for the Middle Mississippi River at Thebes, IL (07022000), which is located 43.7 mi upstream from the Ohio River, and at Chester, IL (07020500), which is located 109.9 mi upstream from the Ohio River. The number of measurements at Chester is much smaller than at Thebes. Total measured suspended sediment and measured suspended sand data were collected at Thebes between 1973 and 2019 and at Chester between 1982 and 1991. Size class data were collected at Thebes and at Chester between 1980 and 1991.

Previous studies (Copeland 2018) indicated a significant decrease in sand concentrations at Thebes after 2002, and the sediment inflow rating curves were modified in the HEC-6T model historical simulation after 1 October 2002. New data through August 2019 were incorporated into a new sediment 2003–2019 rating curve for the OMAR Assessment (Figure 24). Sand size class data were collected at Thebes (35 samples) and Chester (36 samples) between 1980 and 1991. Since these are the only available size class data, they continued to be used in the 2003–2019 sediment input file (Figure 25 and Table 15). Sediment inflow calculated from the regression curves, for the fine sand through coarse sand size classes, was increased by 15% to account for unmeasured load.

Very coarse sand through very fine gravel size class inflows were calculated using the recirculation option in HEC-6T. This methodology theoretically accounts for the unmeasured load, including the bed load. However, calculated results are dependent on the accuracy of the bed gradation and the sediment transport equation. Calculated results should be validated by bed load measurements.

Sediment inflow data trends need to be continually monitored to see if the decreased concentration is due to washout from the 2011 flood and if sand concentrations increase again with time.

Figure 24. Measured suspended sediment at Thebes 2003–2019.



2.13.2 Ohio River

Total suspended sediment and sand concentrations from the Ohio River were determined from the USGS gage at Dam 53 near Grand Chain, IL (03612500) and from the USGS gage at Olmsted, Illinois (03612600), which replaced the Dam 53 gage in December 2014. Sediment concentration data were available at Dam 53 between 1974 and 2014 and at Olmsted through May 2019. These gages are located approximately 17 mi upstream from Cairo (Ohio River RM 962.6).

Size class fractions were obtained from a third USGS gage, at Louisville, KY (03294500). This gage is located approximately 372 mi upstream from Cairo (Ohio River RM 607.3). Size class percentages data for clay through medium sand were collected at Louisville between 1979 and 1982. The Louisville gage is located a considerable distance upstream from the Mississippi River confluence at Cairo, and the available data were collected decades ago. In addition, there are downstream tributaries such as the Wabash River, which is likely a significant source of sand, and the Cumberland and Tennessee Rivers, which have significant reservoirs that trap sediment near their confluence with the Ohio River. However, the Louisville data are the only suspended size class data available. The Ohio River is primarily a threshold river in the study reach, and therefore calculation of suspended bed material load from available bed material gradations is not reliable. The overall picture is also complicated by sand and gravel mining between Olmsted and Smithland Lock and Dams. Smithland is 63 mi upstream from the Mississippi River confluence.

For the OMAR Model, measured total suspended sediment load and measured suspended sand load data collected at Dam 53 and Olmsted between 2003 and 2019 were used to calculate power regression curves for the HEC-6T sediment inflow rating curve (Figure 26). Since no apparent trend in the Ohio River data was demonstrated in previous studies, 2003 was chosen as the starting year for development of the sediment rating curves. This made the time frame consistent with Thebes.

The OMAR Model sediment inflow rating curves for the Ohio River are shown in Table 16. Sand loads for very-fine, fine, and medium sand were determined from the sand regression curve, and fine load was determined from the difference between the total and sand load regression curves. Sediment inflow for the fine sand to medium sand size classes determined

from regression curves was increased by 15% to account for unmeasured load. The coarse sand to medium gravel size classes were transported in the unmeasured zone, so inflow fractions for these sediment sizes were determined using the recirculation option in HEC-6T. With this option, a series of steady state discharges are run for a period of time (in this case 25 days) where the sediment load calculated at the downstream end of the segment is reintroduced at the upstream end of the segment for the next time-step until the calculated inflow and outflow of sediment is approximately equal during a time-step. This is equivalent to assuming that the reach is in equilibrium with respect to sediment continuity. Uncertainty exists with this approach because these calculations are directly dependent upon the designated bed gradation. In this case, the bed gradation came from an average of multiple samples collected at Olmsted. Medium sand load was determined using the larger of the fractions calculated from the regression equations or the recirculation option.

Figure 26. Measured suspended sediment concentrations Dam 53 and Olmsted 2003–2019.

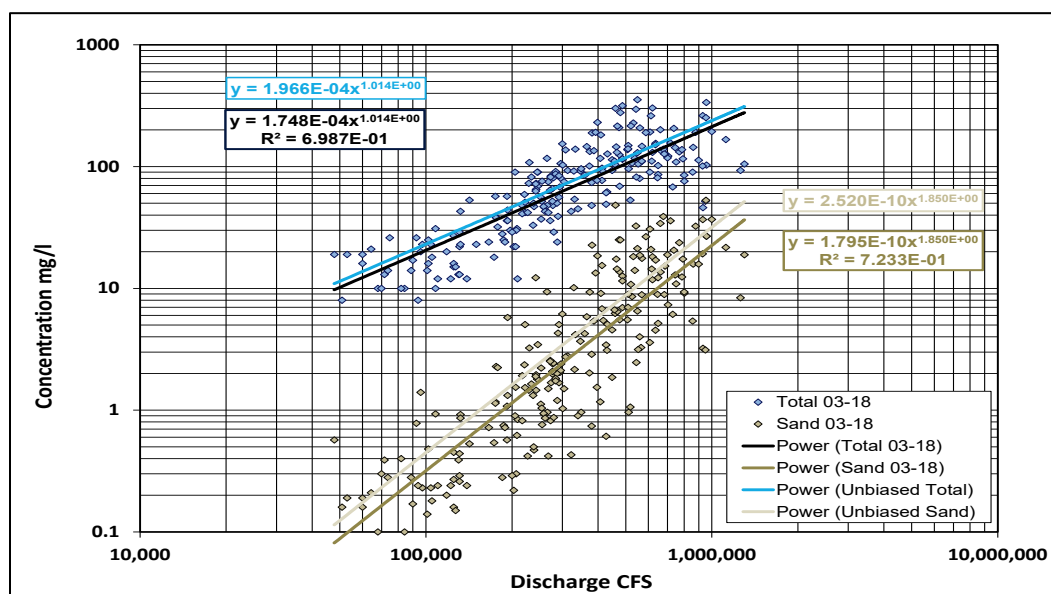


Table 16. Sediment inflow for Ohio River – Segment 24.

Discharge 1000 cfs	10	50	100	200	450	800	1000	1300	3000
Concentration mg/L	11.44	11.44	23.10	46.65	106.16	190.25	238.55	311.26	311.26
Fractions									
Clay	0.6082	0.6082	0.6030	0.5936	0.4337	0.4140	0.4035	0.3883	0.3883
Very Fine Silt	0.1360	0.1360	0.1348	0.1327	0.1272	0.1215	0.1184	0.1139	0.1139
Fine Silt	0.1388	0.1388	0.1376	0.1355	0.1577	0.1506	0.1468	0.1412	0.1412
Medium Silt	0.0825	0.0825	0.0818	0.0805	0.1304	0.1245	0.1214	0.1168	0.1168
Coarse Silt	0.0237	0.0237	0.0235	0.0231	0.0827	0.0790	0.0770	0.0741	0.0741
Very Fine Sand	0.0057	0.0057	0.0102	0.0182	0.0389	0.0630	0.0759	0.0945	0.0945
Fine Sand	0.0054	0.0054	0.0096	0.0172	0.0249	0.0403	0.0485	0.0604	0.0604
Medium Sand	0.0005	0.0005	0.0033	0.0135	0.0177	0.0375	0.0342	0.0271	0.0271
Coarse Sand	0.0	0.0	0.0011	0.0061	0.0115	0.0142	0.0144	0.0117	0.0117
Very Coarse Sand	0.0	0.0	0.0	0.0030	0.0068	0.0093	0.0091	0.0078	0.0078
Very Fine Gravel	0.0	0.0	0.0	0.0	0.0007	0.0024	0.0024	0.0020	0.0020
Fine Gravel	0.0	0.0	0.0	0.0	0.0	0.0005	0.0010	0.0012	0.0012
Medium Gravel	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0001	0.0001
Coarse Gravel	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Very Coarse Gravel	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

2.13.3 Obion and Hatchie Rivers

There were no additional data available for the Obion and Hatchie Rivers since completion of the Regional Model study. Therefore, the same sediment inflow rating curves used in the Regional and Flowline Models were used in the OMAR Model. The Obion River rating curves were based on data from 1975–1995. The Hatchie River rating curves were based on data from 1977–1995 and 2004–2008.

The purpose of the Obion and Hatchie River Segments in the HEC-6T model was to provide sediment inflow to the Mississippi River. Measured suspended sediment data from both rivers are available from USGS gages located considerable distances upstream from the HEC-6T model boundaries. Total suspended and sand inflow concentrations were available from these gages for a wide range of discharges. However,

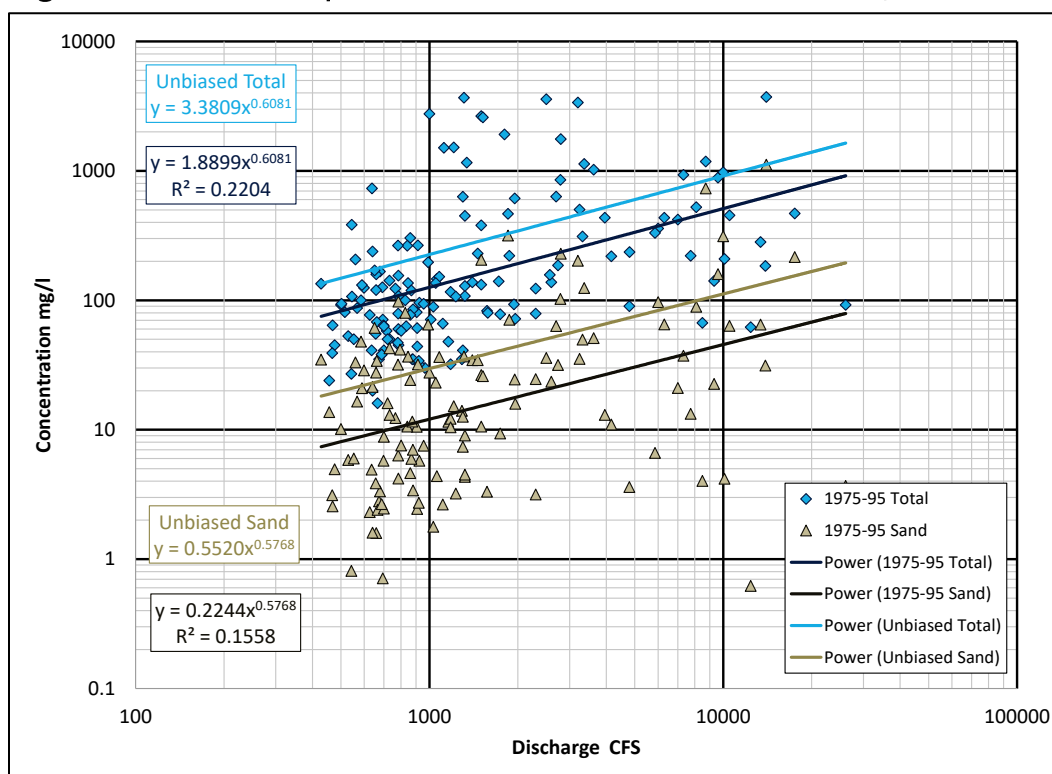
individual size class percentages of the suspended load were lacking. There were no bed load data available.

There were no geometric data available between the Mississippi River and upstream sediment gages. A 2 mi study reach was included in the model based on estimated cross-section geometry. No calibration was attempted for these segments, and no conclusions can be made with respect to calculated morphologic changes in these segments.

In the HEC-6T model, the beds of the Obion and Hatchie Rivers were not allowed to scour below the initial cross section elevations. Therefore, the riverbed was not allowed to contribute any sediment to the Mississippi River. However, deposition was allowed in the model so that sediment inflow at the model boundary could be computationally reduced if hydraulic conditions were favorable.

Measured suspended sediment concentrations from two gages on the Obion River were used to determine sediment inflow concentrations for the HEC-6T model. The USGS reported sediment data at gage number 07026000, Obion River at Obion, TN, from samples collected between 1975 and 1990, and from USGS gage number 07026040, Obion River at Hwy 51 near Obion, TN, from samples collected between 1990 and 1995. These gages are approximately at the same location at about RM 62, which is approximately 60 mi upstream from the HEC-6T model boundary. Unbiased power regression curves were calculated from the combined measurements from these two gages. There were 149 measurements of the total suspended concentration and 122 measurements of the sand concentration. The data are plotted in Figure 27

Figure 27. Measured suspended sediment concentrations Obion River, 1975–1995.



The measured Obion River data included a sand-silt break, and these data were used to determine the inflow concentrations for fines and sand. However, no additional size class percentages data were available. The sand size class percentages were taken to be the same as the average sand size class percentages at Thebes and Chester. Likewise, the clay and silt size class percentages were taken to be the same as the average clay and silt size class percentages at Thebes and Chester.

The sediment inflow rating curves used in the HEC-6T model, for the Obion River, are shown in Table 17. The discharge range used to develop the sediment inflow regression curves was between 400 and 30,000 cfs. In Table 17, the calculated concentration at 30,000 cfs was used for higher discharges. A flat concentration curve at higher discharges is generally closer to prototype behavior than an extrapolation of the regression curve. In the HEC-6T sediment inflow table, fine sand through coarse sand size class percentages were increased by 15% to account for unmeasured load.

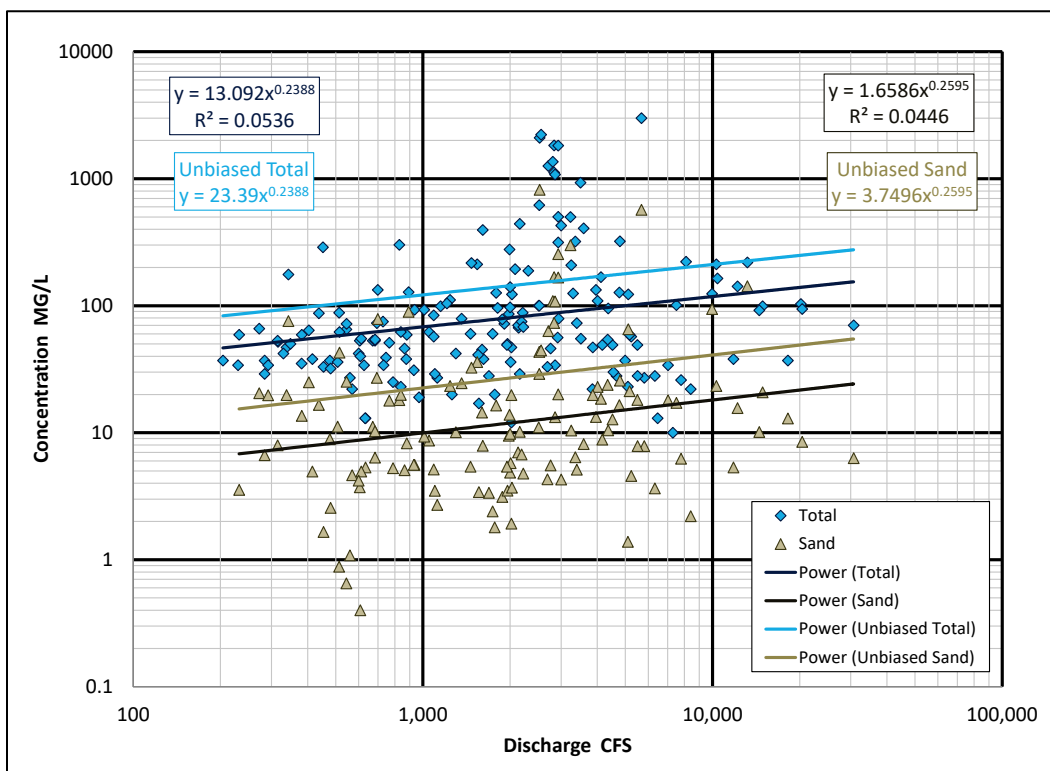
Table 17. Sediment Inflow for Obion River - Segment 16.

Discharge	1	50	500	2,000	5,000	30,000	200,000
Concentration (mg/L)	3.4	36.5	148.0	343.9	600.3	1784.7	1784.7
Fractions							
Clay	0.4735	0.4842	0.4898	0.4931	0.4951	0.4990	0.4990
Very Fine Silt	0.0733	0.0750	0.0758	0.0763	0.0767	0.0773	0.0773
Fine Silt	0.0751	0.0768	0.0777	0.0782	0.0786	0.0792	0.0792
Medium Silt	0.1074	0.1098	0.1111	0.1118	0.1123	0.1132	0.1132
Coarse Silt	0.1074	0.1098	0.1111	0.1118	0.1123	0.1132	0.1132
Very Fine Sand	0.0237	0.0209	0.0195	0.0187	0.0181	0.0171	0.0171
Fine Sand	0.1150	0.1018	0.0947	0.0907	0.0881	0.0833	0.0833
Medium Sand	0.0379	0.0335	0.0312	0.0299	0.0290	0.0274	0.0274
Coarse Sand	0.0072	0.0063	0.0059	0.0056	0.0055	0.0052	0.0052
Very Coarse Sand	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Very Fine Gravel	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Fine Gravel	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Medium Gravel	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Coarse Gravel	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Very Coarse Gravel	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Measured suspended sediment concentrations from two gages on the Hatchie River were used to determine sediment inflow concentrations for the HEC-6T model. The USGS reported sediment data at gage number 07029500, Hatchie River at Bolivar, TN, from samples collected between 1977 and 1995. There were 134 measurements of the total sediment concentration and 110 measurements of the sand concentration at Bolivar. This gage is at RM 135, considerably upstream from the model boundary. Downstream, at RM 34, a smaller data set was collected at USGS gage number 07030050, Hatchie River at Rialto, TN. Data at this gage extend between 1977 and 1978 and between 2004 and 2008. At Rialto, there were only 42 measurements of total sediment concentration and 19 measurements of sand concentration; 4 of which were after 1978. Considering the paucity of data, it was deemed appropriate to use the measurements from the entire 1977–2008 record and from both gages to develop the best estimate of average sediment inflow rating curves.

Unbiased power regression curves of the Hatchie River data are shown in Figure 28.

Figure 28. Measured suspended sediment concentrations Hatchie River, 1977–1995.



The measured Hatchie River data included a sand-silt break and these data were used to determine the inflow concentrations for fines and sand. However, there were no additional size class percentage data available. The sand size class percentages were taken to be the same as the average sand size class percentages at Thebes and Chester. Likewise, the clay and silt size class percentages were taken to be the same as the average clay and silt size class percentages at Thebes and Chester.

The sediment inflow rating curves used in the HEC-6T model, for the Hatchie River, are shown in Table 18. The discharge range used to develop the sediment inflow regression curves was between 200 and 30,000 cfs. In Table 18, the calculated concentration at 30,000 cfs was used for higher discharges. A flat concentration curve at higher discharges is generally closer to prototype behavior than an extrapolation of the regression curve. In the HEC-6T sediment inflow table, fine sand through coarse sand size class percentages was increased by 15% to account for unmeasured load.

Table 18. Sediment inflow for Hatchie River - Segment 14.

Discharge	1	50	200	1000	5000	30,000	200,000
Concentration (mg/L)	23.4	59.5	82.9	121.7	178.8	274.3	274.3
Fractions							
Clay	0.4752	0.4675	0.4647	0.4612	0.4577	0.4536	0.4536
Very Fine Silt	0.0736	0.0724	0.0719	0.0714	0.0709	0.0702	0.0702
Fine Silt	0.0754	0.0742	0.0737	0.0732	0.0726	0.0720	0.0720
Medium Silt	0.1078	0.1060	0.1054	0.1046	0.1038	0.1029	0.1029
Coarse Silt	0.1078	0.1060	0.1054	0.1046	0.1038	0.1029	0.1029
Very Fine Sand	0.0232	0.0252	0.0259	0.0268	0.0277	0.0288	0.0288
Fine Sand	0.1130	0.1225	0.1261	0.1303	0.1347	0.1398	0.1398
Medium Sand	0.0372	0.0403	0.0415	0.0429	0.0444	0.0460	0.0460
Coarse Sand	0.0070	0.0076	0.0078	0.0081	0.0084	0.0087	0.0087
Very Coarse Sand	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Very Fine Gravel	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Fine Gravel	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Medium Gravel	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Coarse Gravel	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Very Coarse Gravel	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

Future studies to assess downward trends in sand and fine sediment yields to downstream reaches of the Mississippi River will require the collection of additional sediment data on tributaries, such as the Obion and Hatchie Rivers.

2.13.4 Saint Francis River

No additional data were available for the Saint Francis River since completion of the Regional Model study. Therefore, the same sediment inflow rating curves used in the Regional and Flowline Models were used in the OMAR Model. The Saint Francis River rating curve was based on data from 1977–2009.

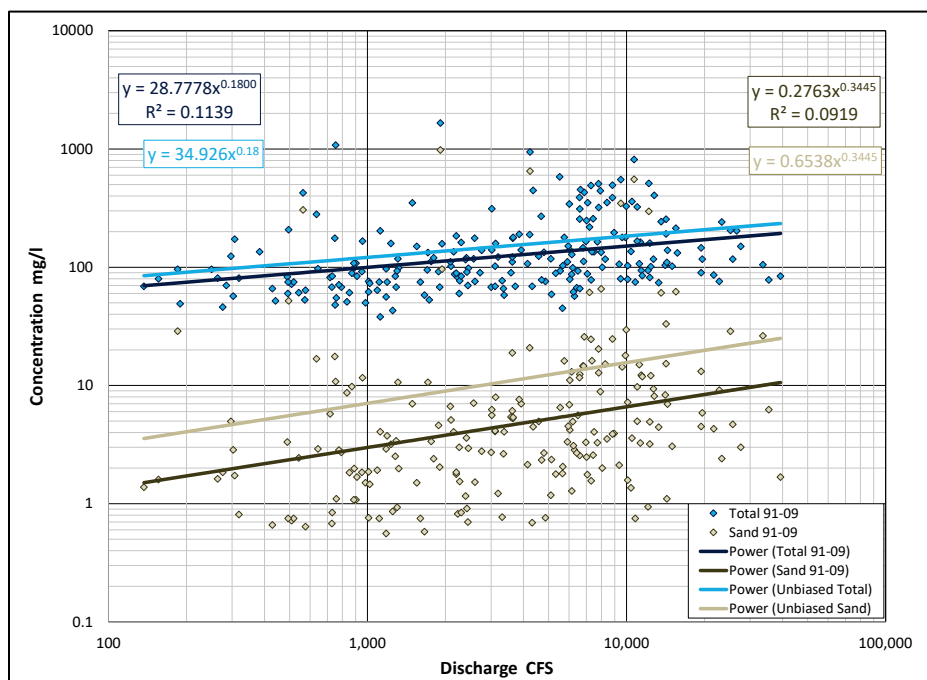
The purpose of the Saint Francis River Segment in the HEC-6T model is to provide sediment inflow to the Mississippi River. Considerable sediment data are available for the Saint Francis River and were used to calibrate a

2000 MVM HEC-6T study. Additional data were collected by the USGS subsequent to the 2000 study and were used to develop sediment inflow curves for the OMAR Model. Total suspended sediment and suspended sand concentrations were available for a wide range of discharges. In almost all cases, bed material gradation data were also collected when the suspended samples were collected. Individual sand size class percentages were determined in the laboratory for almost all of the suspended samples.

Measured suspended sediment concentrations from the USGS gage number 07047907, Saint Francis River at Madison, AR, were used to determine sediment inflow concentrations for the HEC-6T model. The USGS reported sediment data at the Madison gage between 1977 and 2009. This gage is located at RM 37.9, which is approximately 20 mi upstream from the upstream model boundary.

Measured total suspended sediment concentrations and measured suspended sand concentrations collected at Madison between 1991 and 2002 were used to calculate power regression curves for the HEC-6T sediment inflow rating table. There were 139 measurements of the total suspended concentration and 127 measurements of the sand concentration. Unbiased power regression curves of the St Francis River data are shown in Figure 29.

Figure 29. Measured suspended sediment concentrations St Francis River, 1991–2009.



2.13.5 White River

The purpose of the White River Segment in the HEC-6T model was to provide sediment inflow to the Mississippi River. Sediment data for the White River were limited and are insufficient for calibration of the White River segment. Total suspended and sand inflow concentrations were available for a wide range of discharges. However, bed material gradation data and individual size class percentages of the suspended load were lacking. In the HEC-6T model, the White River bed was not allowed to scour below the initial cross-section elevations. Therefore, the riverbed was not allowed to contribute any sediment to the Mississippi River. However, deposition was allowed in the model so that sediment inflow at the model boundary could be computationally reduced if hydraulic conditions were favorable. Calculated morphologic changes in the White River are not reliable.

Measured suspended sediment concentrations from the USGS gage White River at DeValls Bluff, AR (07077000) were used to determine sediment inflow concentrations for the OMAR Model. This gage is located at RM 121.8, which is approximately 22 mi upstream from the model boundary at Clarendon. Data at this gage extend between 2001 and 2019. Previous model studies had used data from other White River gages, but the DeValls Bluff gage is currently active and has the most recent data for the White River.

The measured White River data included a sand-silt break, and these data were used to determine the inflow concentrations for fines and sands. However, there were no size class percentage data available. The size class percentages were taken to be the same as the average size class percentages at Thebes and Chester. The sediment inflow rating curves used in the HEC-6T model, for the White River, are shown in Figure 30 and Table 20. The discharge range used to develop the sediment inflow regression curves was between 7,000 and 90,000 cfs. Concentrations are held constant at the limits of the measured data (i.e., no extrapolation). In the HEC-6T sediment inflow table, fine sand through coarse sand size class percentages were increased by 15% to account for unmeasured load

Figure 30. Measured suspended sediment concentrations - White River at DeValls Bluff 2001–2019.

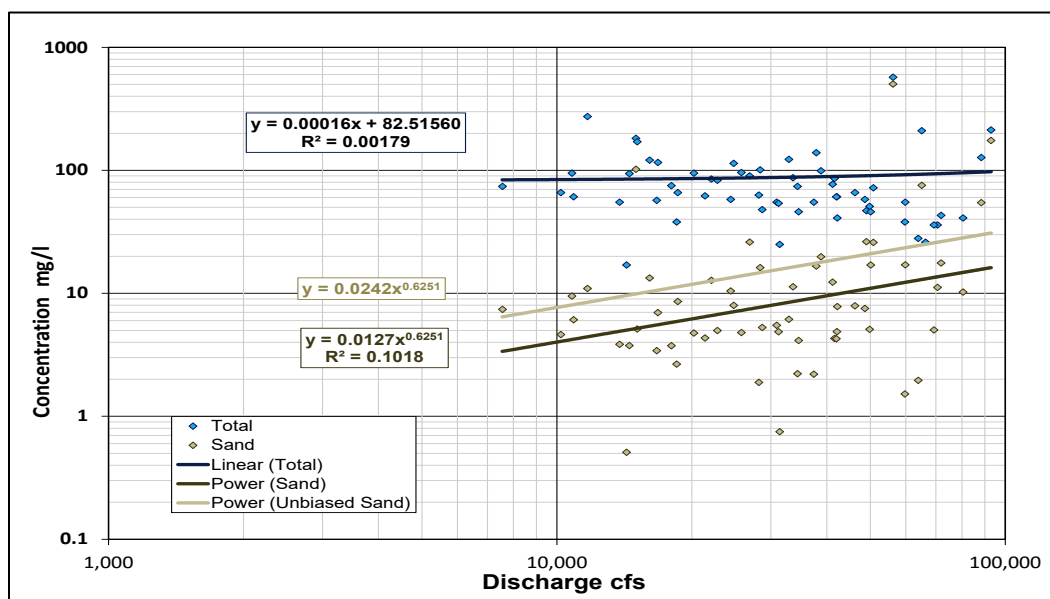


Table 20. Sediment inflow for White River – Segment 10.

Discharge CFS	100	7,000	10,000	50,000	90,000	300,000
Concentration (mg/L)	83.6	83.6	84.1	90.5	96.9	96.9
Fractions						
Clay	0.5244	0.5244	0.5144	0.4349	0.3892	0.3892
Very Fine Silt	0.0812	0.0812	0.0796	0.0673	0.0603	0.0603
Fine Silt	0.0832	0.0832	0.0816	0.0690	0.0618	0.0618
Medium Silt	0.1189	0.1189	0.1167	0.0986	0.0883	0.0883
Coarse Silt	0.1189	0.1189	0.1167	0.0986	0.0883	0.0883
Very Fine Sand	0.0106	0.0106	0.0132	0.0336	0.0453	0.0453
Fine Sand	0.0516	0.0516	0.0642	0.1631	0.2200	0.2200
Medium Sand	0.0170	0.0170	0.0211	0.0537	0.0724	0.0724
Coarse Sand	0.0032	0.0032	0.0040	0.0101	0.0137	0.0137
Very Coarse Sand	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Very Fine Gravel	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Fine Gravel	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Medium Gravel	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Coarse Gravel	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Very Coarse Gravel	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

2.13.6 Arkansas River

The purpose of the Arkansas River Segment in the HEC-6T model is to provide sediment inflow to the Mississippi River. Data are insufficient for calibration of the Arkansas River Segment and calculated morphologic changes should be treated with caution. Sediment data for the Arkansas River were limited. Sand and fine inflow concentrations were available for a wide range of discharges. However, bed material gradation data and individual size class percentages of the suspended load were lacking.

Measured suspended sediment concentrations, taken between 1994 and 2019, from the USGS gage Arkansas River at David D Terry Lock and Dam below Little Rock, AR (07263620), were used to determine sediment inflow concentrations for the HEC-6T Model. This gage is located approximately 95 mi upstream from the model boundary. The model boundary is located at Dam No. 2. Data at the Terry Lock and Dam gage extend between 1974 and 2019, but only data after 1994 specify the type of equipment used to collect sediment samples. Previous model studies had used data from the USGS gage at Dam No. 2 in addition to the 1974–1994 data at Terry Lock and Dam, but only data taken using isokinetic samplers were used to determine the sand concentrations for the Arkansas River in the OMAR Model. This is especially important on the Arkansas River due to the distinctly different flow regimes created by the Arkansas River navigation dams. Fine sediment concentrations taken using non-isokinetic samplers were included in the determination of fine sediment concentrations. This approach is appropriate due to the typically constant vertical distributions of fine sediment and allowed for considerably more information at low discharges.

The measured Arkansas River data included a sand-silt break. These data were used to determine the inflow concentrations for fines and sand. However, there were no additional size class percentage data available. Measured percentages from Mississippi River gage data at Coochie and Union Point (Mississippi RMs 317 and 326, respectively) were assigned for clay and silt sizes on the Arkansas River.

Sediment transport for each sand size class was calculated for a series of steady state discharges at five upstream cross sections (downstream from Dam 2). Due to the small percentages of very fine sand in the bed, very fine sand was considered to be wash load. Very fine sand transport was

assumed to be the difference between the measured sand transport from the unbiased sand regression curve and the calculated sand transport for the fine sand through very coarse sand size classes. Sand size class percentages were then determined from these calculated sand loads. The calculated percentages were then smoothed using regression analysis in EXCEL. Fine sand through very coarse sand size class percentages was increased by 15% to account for unmeasured load.

Arkansas River sediment inflow rating curves, for fines and sand, are shown in Figure 31 and Figure 32. The fine sediment linear regression curve labeled “all” is the one used in the model. These data include dip samplers, weighted bottles, and grab samples as well as isokinetic samplers. Note that even isokinetic samplers are not isokinetic when the velocity is less than 2 ft/sec. The sand measurements used to develop the sand inflow concentrations were taken with isokinetic samplers (including the USGS *frame* bag sampler). The USGS data base does not provide velocity determinations for all the samples, but some of the velocity measurements provided are less than 1.5 ft/sec. Size class model input data are shown in Table 21.

Figure 31. Measured fines 1994–2019 Arkansas River at Terry Lock and Dam.

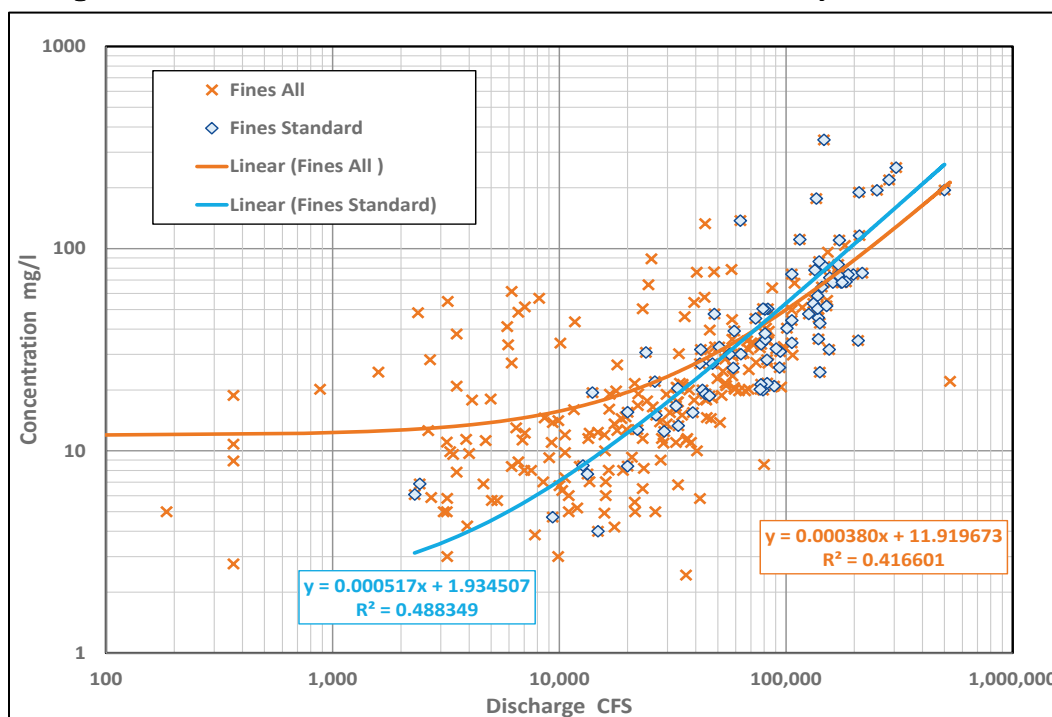


Figure 32. Measured sand 1994–2019 Arkansas River at Terry Lock and Dam.

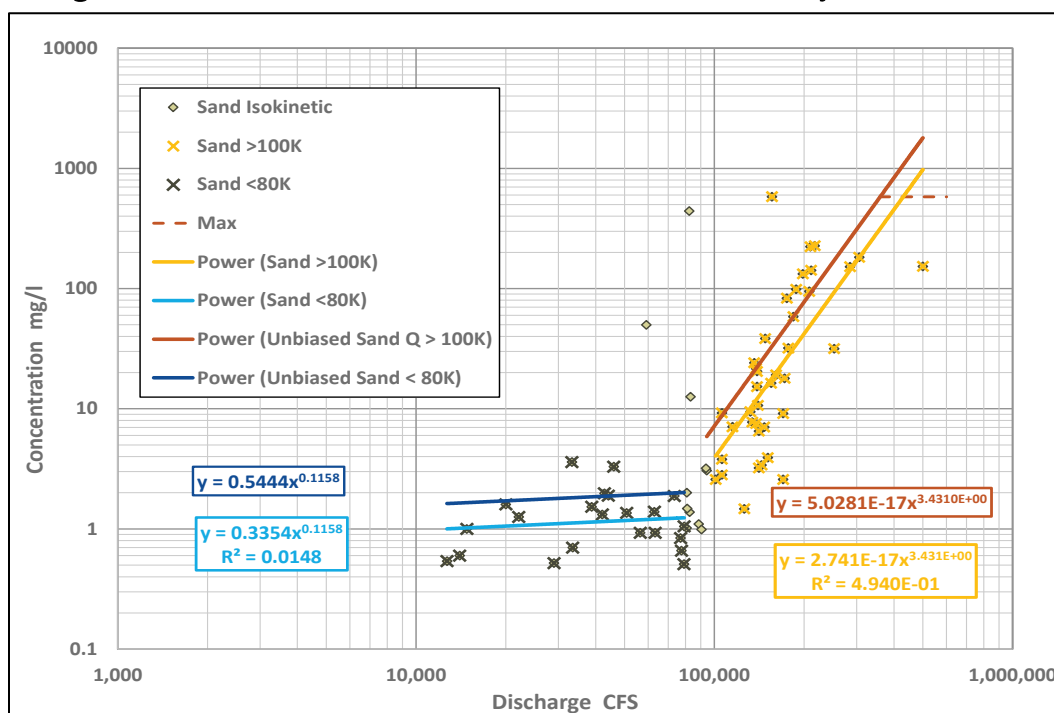


Table 21. Sediment inflow for Arkansas River – Segment 8.

Discharge 1000 cfs	0.01	0.1	0.5	10	80	100	200	360	600
Concentration (mg/L)	13.51	13.51	15.28	17.30	45.31	60.82	182.8	769.0	769.0
Fractions									
Clay	0.3587	0.3587	0.3564	0.3536	0.3141	0.3028	0.2465	0.1564	0.1564
Very Fine Silt	0.1312	0.1312	0.1304	0.1294	0.1149	0.1108	0.0902	0.0572	0.0572
Fine Silt	0.1018	0.1018	0.1012	0.1004	0.0892	0.0860	0.0700	0.0444	0.0444
Medium Silt	0.1663	0.1663	0.1653	0.1640	0.1457	0.1404	0.11143	0.0725	0.0725
Coarse Silt	0.1867	0.1867	0.1855	0.1840	0.1635	0.1576	0.1283	0.0814	0.0814
Very Fine Sand	0.0067	0.0067	0.0075	0.0086	0.0235	0.0278	0.0492	0.0833	0.0833
Fine Sand	0.0127	0.0127	0.0105	0.0178	0.0573	0.0686	0.1250	0.2152	0.2152
Medium Sand	0.0242	0.0242	0.0287	0.0343	0.1126	0.1350	0.2469	0.4258	0.4258
Coarse Sand	0.0	0.0	0.0	0.0007	0.0103	0.0130	0.0267	0.0485	0.0485
Very Coarse Sand	0.0	0.0	0.0	0.0	0.0003	0.0005	0.0014	0.0028	0.0028

Table 21. (cont.) Sediment inflow for Arkansas River – Segment 8.

Very Fine Gravel	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Fine Gravel	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Medium Gravel	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Coarse Gravel	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Very Coarse Gravel	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0

2.13.7 Yazoo River

The purpose of the Yazoo River Segment in the HEC-6T model was to provide sediment inflow to the Mississippi River. Sediment data for the Yazoo River were limited and are insufficient for calibration of the Yazoo River Segment. There are significant suspended sediment measurements of the total and sand load available for a wide range of discharges. However, only two bed samples were available and there were no data defining the individual size class percentages in the suspended load.

Measured suspended sediment concentrations from the USGS gage Yazoo River below Steele Bayou near Long Lake, Mississippi (07288955) were used to determine sediment inflow concentrations for this study. The Steele Bayou gage is located at RM 9.5, above the confluence with the Mississippi River. Previous studies had included data from the USGS gage Yazoo River at Redwood, Mississippi gage (07288800), which is located at RM 16.7. Suspended sediment samples were collected at Redwood between 1978 and 1993 and in 2010. However, the Redwood data do not specify the sampler type. Without knowing the sampler type, it is not possible to determine if the samples were collected isokinetically. After July 1999, sampler types were reported regularly at the Steele Bayou gage. Only samples collected with standardized Federal Interagency Sedimentation Project isokinetic samplers (and the USGS frame bag sampler) were used to develop the linear regression curves for sediment inflow. Samples reported with negative discharge were also excluded. The measured data and regression curves are shown in Figure 33.

The inflowing sediment load size class percentages for fines and sands were assumed to be the same as the measured size class percentages from 1991–2002 data at Coochie and Union Point. These sediment gages are located at Mississippi RMs 317 and 326, respectively, and are the closest

gages with size class data. This methodology results in a significant discontinuity between coarse silt and very fine sand concentrations in the inflowing sediment load rating curves. The sand concentrations were increased by 15% to account for the unmeasured load. The sediment inflow data for the Yazoo River are tabulated in Table 22.

Only two bed material samples were available for the Yazoo River. The USGS collected these samples at the Steele Bayou Gage. Both samples were collected in August, one in 1995 that contained only 8% sand and one in 1997 that contained 5% sand. These data suggest that the bed of the Yazoo River is primarily cohesive. Therefore, the bed sediment reservoir in the HEC-6T model was set to zero. The initial bed gradation in the Yazoo River was determined based on the two measurements, normalizing them to 100% sand. Since the bed sediment reservoir has been given a zero depth, this bed gradation only serves as a starting point in the numerical solution of the sediment continuity equation.

Figure 33. Measured suspended sediment concentrations Yazoo River at Steele Bayou 1999–2019.

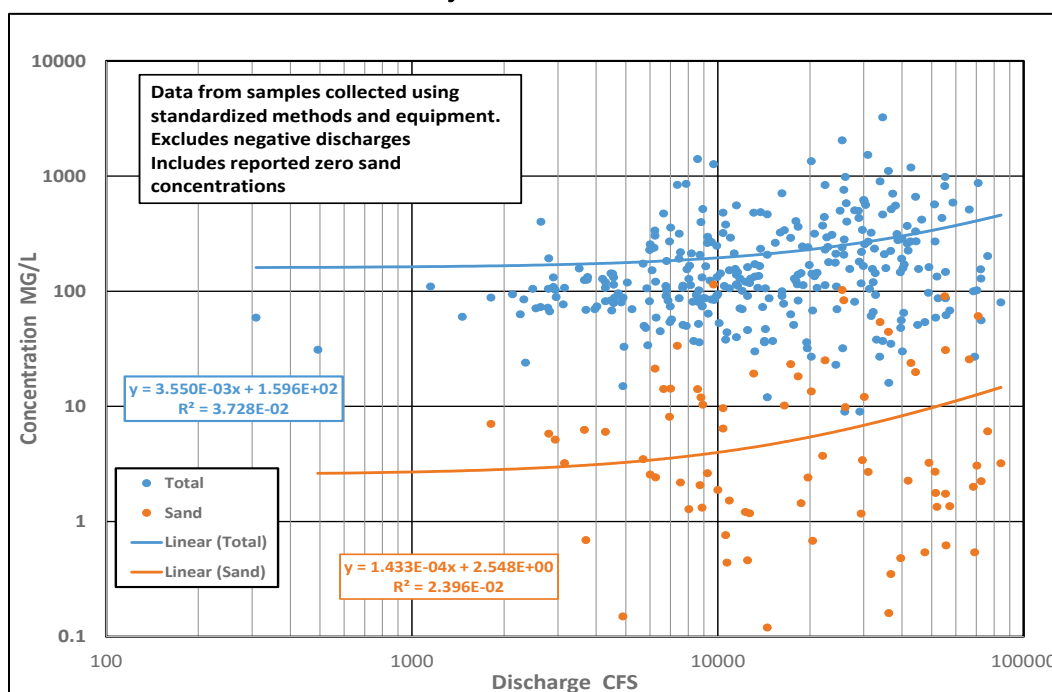


Table 22. Sediment inflow for Yazoo River – Segment 6.

Discharge (cfs)	1	100	1,000	5,000	20,000	50,000	100,000
Concentration (mg/L)	160	160	163	177	231	337	515
Fractions							
Clay	0.3634	0.3634	0.3632	0.3624	0.3603	0.3581	0.3565
Very Fine Silt	0.1277	0.1277	0.1276	0.1273	0.1266	0.1258	0.1252
Fine Silt	0.1080	0.1080	0.1080	0.1077	0.1071	0.1065	0.1060
Medium Silt	0.1768	0.1768	0.1767	0.1763	0.1753	0.1742	0.1734
Coarse Silt	0.2062	0.2062	0.2061	0.2057	0.2045	0.2032	0.2023
Very Fine Sand	0.0032	0.0032	0.0033	0.0037	0.0047	0.0057	0.0065
Fine Sand	0.0068	0.0068	0.0070	0.0078	0.0100	0.0122	0.0139
Medium Sand	0.0079	0.0079	0.0081	0.0091	0.0116	0.0142	0.0162
Coarse Sand	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Very Coarse Sand	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000
Very Coarse Gravel	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000	0.0000

2.14 Depth of bed sediment reservoir

The sorting and armoring algorithm in HEC-6T assumes that the bed sediment reservoir provides an infinite supply unless a bed sediment reservoir depth or bottom elevation that establishes a bed sediment reservoir depth is prescribed. Sediment continuity is maintained in three separate bed layers during the computer simulation, and the bed sediment reservoir only comes into play during the initial time-step and when erosion of the three surface layers occurs. If the supply of bed sediment is limited in the prototype, then that limit must be set in HEC-6T by prescribing a bed sediment reservoir depth.

The bed sediment reservoir depth was limited at several locations in the Mississippi River where erosion is known to be limited by cohesive deposits, bedrock, gravel deposits, or constructed sills. Using a hard bottom or minimal bed sediment reservoir depth essentially ignores the assigned initial bed material gradation, and the bed gradation at each time-step during the simulation is calculated by the program from deposited sediment at each cross section. If there is no deposited sediment, the inflowing sediment is passed on to the next downstream cross section. A bed sediment

reservoir depth of 1 ft was assigned in the model between RM 12.5 and 57 and 5 ft between RM 58.8 and 61.3. This reflects the presence of a cohesive layer underlying the Mississippi River over which sand is transported (Kolb 1962). This assignment has been consistent in all USACE HEC-6T numerical models of this reach of the Mississippi River to date.

The bed sediment reservoir was also limited at several other locations in the Lower Mississippi Rivers upstream from ORCC and in several of the tributaries in the HEC-6T model. Details on these assignments can be found in the Regional Model report (Copeland et al. 2020).

The dredging algorithm in HEC-6T requires that a bed sediment reservoir depth be prescribed. At non-dredging sites, the bed sediment reservoir depth is calculated by the hydraulic sorting and armoring algorithm. A bed sediment reservoir depth of 40 ft was assigned in the HEC-6T model in the MVN between RM -22.0 and 5.5 and at deep-draft crossings where dredging was calculated during the simulation.

2.15 Bed material gradations

Bed material sampling programs of the Mississippi River have indicated significant longitudinal and lateral variability in the bed material gradations. An example of lateral variability is shown in Figure 34, where samples were taken from transects at quarter points at a river bend at Bayou Sara (RM 265). In Figure 34, the point bar is on the right side of the river. Longitudinal variability is primarily due to the differences in bed shear stresses associated with the natural bends and crossings along the course of the river. Variability over short distances of 1 or 2 mi can equal the average variability over the entire 950 mi between Head of Passes and Cairo. This variability makes it difficult to determine if there has been any long-term change in the bed material gradation. Nordin and Queen (1992) compared thalweg bed-material samples that they collected in 1989 to thalweg bed-material samples collected in 1932 (WES 1935) and concluded the following:

1. The 1989 samples were more uniform than the 1932 samples, containing less coarse sand and gravel and less very fine sand than the 1932 samples.
2. Between Cairo and Vicksburg, the median grain diameter was generally smaller in 1989 than it was in 1932. This small difference is shown in Figure 35, which plots the median grain diameters of only the sand

samples. Without the linear regression lines, it would be difficult to determine any difference in the median size between 1932 and 1989. The average difference is contained within the data scatter.

3. Downstream from Vicksburg, the median grain diameter of the bed material was approximately the same in 1989 and 1932.

A comprehensive bed material sampling program of the Mississippi River that extended from Grafton, IL, to Head of Passes, Louisiana, was conducted in November 2013 (Gaines and Priestas 2016). This program was more comprehensive than previous programs in that it extended farther upstream, and five samples were collected laterally across the river at more locations than in previous bed sampling programs. Conclusions from this study were that little change occurred in median grain size between 1989 and 2013 and that the bed material was more uniform with less gravel.

Longitudinal variations in bed-material gradations were accounted for in the numerical model using a polynomial regression curve of the Nordin and Queen (1992) data. Then to account for bed gradation variability associated with cross-section shape and velocity, a bankfull discharge was simulated in the model for 30 days. The new calculated bed gradations were then set as initial conditions in the numerical model. The calculated initial bed gradations are shown in Figure 36.

The initial bed material gradation in the OMAR Model was based on the 1989 Nordin and Queen (1992) data. Reasons for this choice were model simulations started in 1991, which is closer to the 1989 sampling than 2013; *warmup* adjustments had already been made for the 1989 data, and the 2013 and 1989 data were only slightly different.

Downstream from Venice, in the dredging reaches, the initial assigned bed gradation is quickly adjusted in the numerical model as a function of the inflowing sediment load and the average bed shear stress determined by hydrodynamic variables. Deposition is the dominate sedimentation process in the dredging reaches, therefore the gradation of the inflowing sediment load is the driving variable that determines the bed gradation at each time step. Consequently, the initial bed gradation assignment is less important below Venice than in reaches where erosion and deposition processes oscillate. That said, if the initial bed sediment reservoir is assigned a significant silt fraction, the program will quickly erode this

material, creating unrealistic erosion depths. Therefore, initial bed gradations are assigned based only on the sand bed samples. HEC-6T deposition algorithms calculate silt and clay deposition into the active bed layer at low discharges where they can be re-suspended when bed shear stresses increase. Assigning an initial bed gradation consisting only of sand has been consistent in all USACE HEC-6T numerical models of the reach below Venice.

Figure 34. Lateral variation in bed material gradation at RM 265, July 2005.

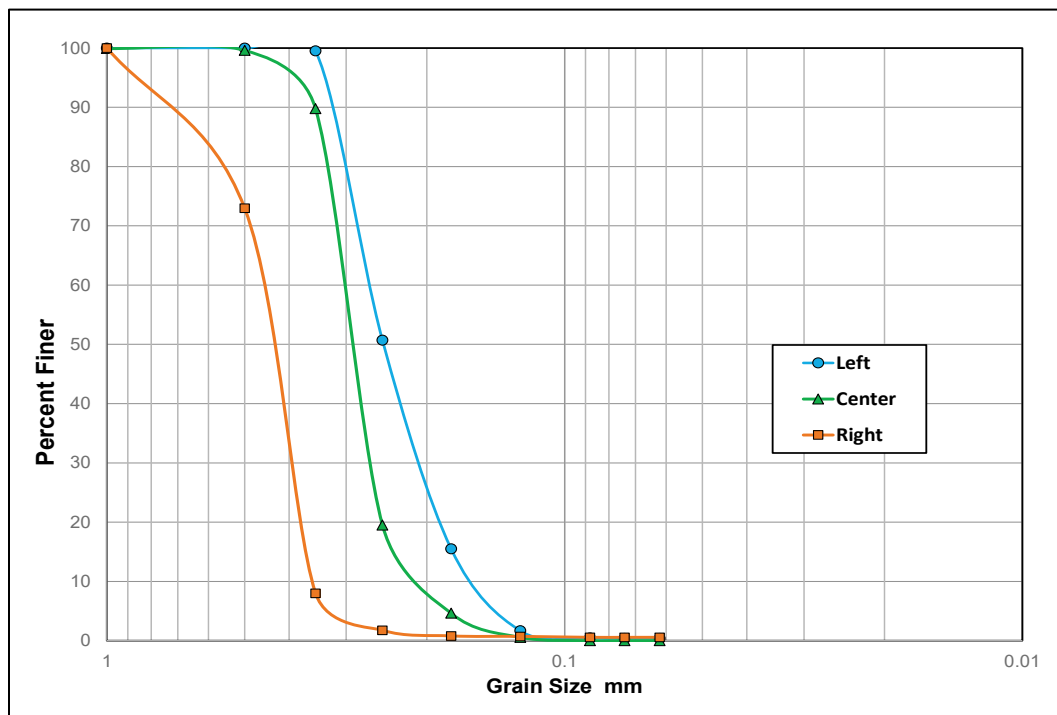


Figure 35. Historic change in median bed grain size (Nordin and Queen 1992).

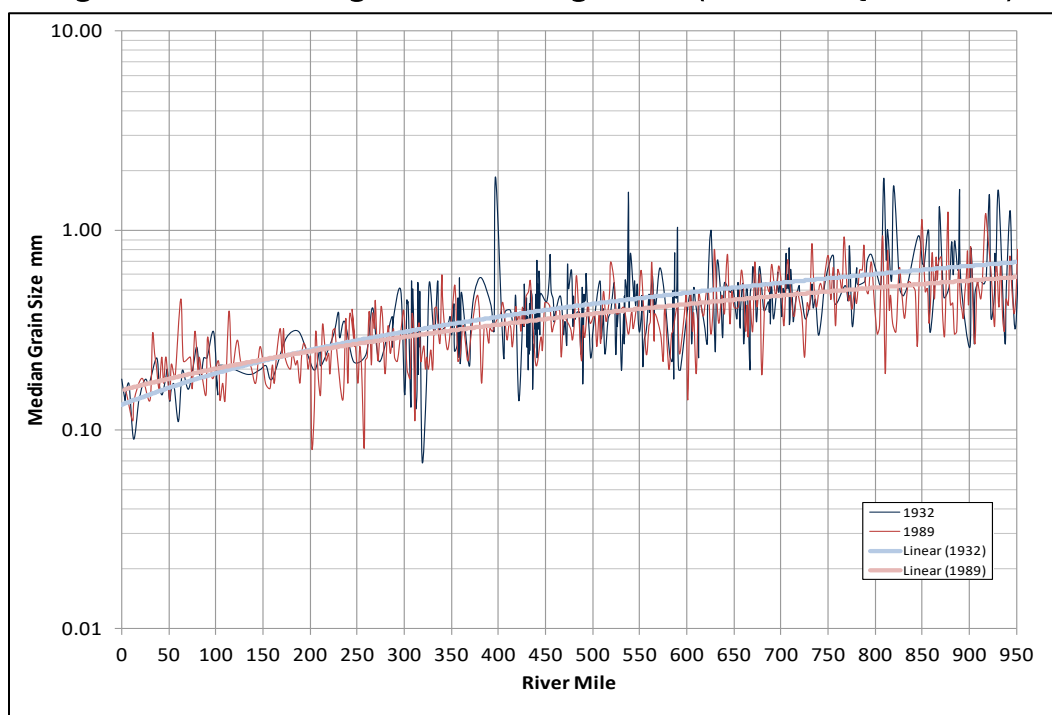
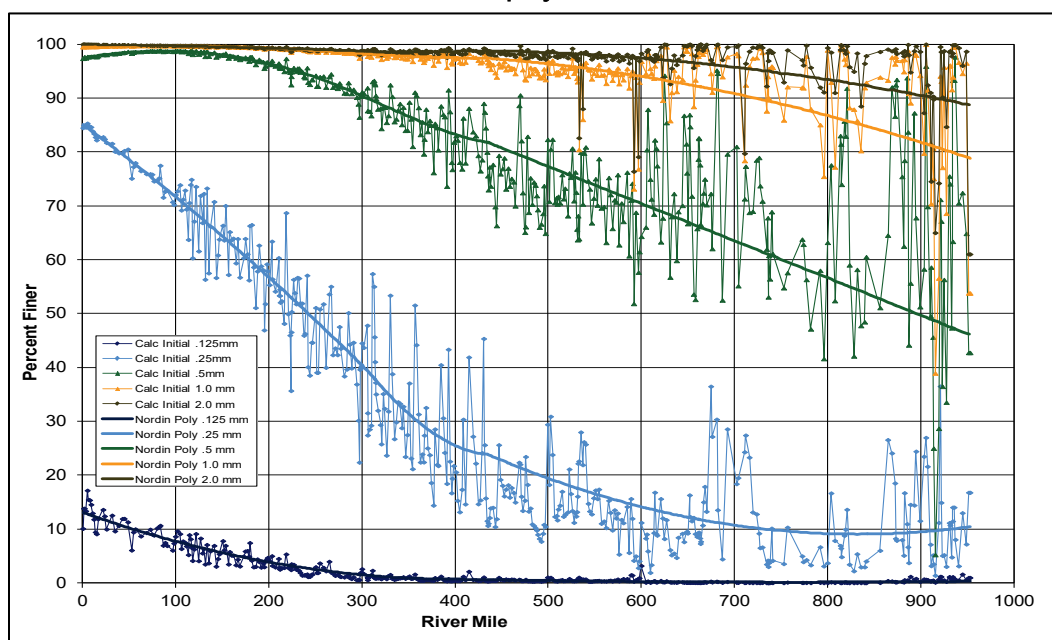


Figure 36. Initial calculated bed gradations — 30-day simulation, starting with Nordin polynomials.

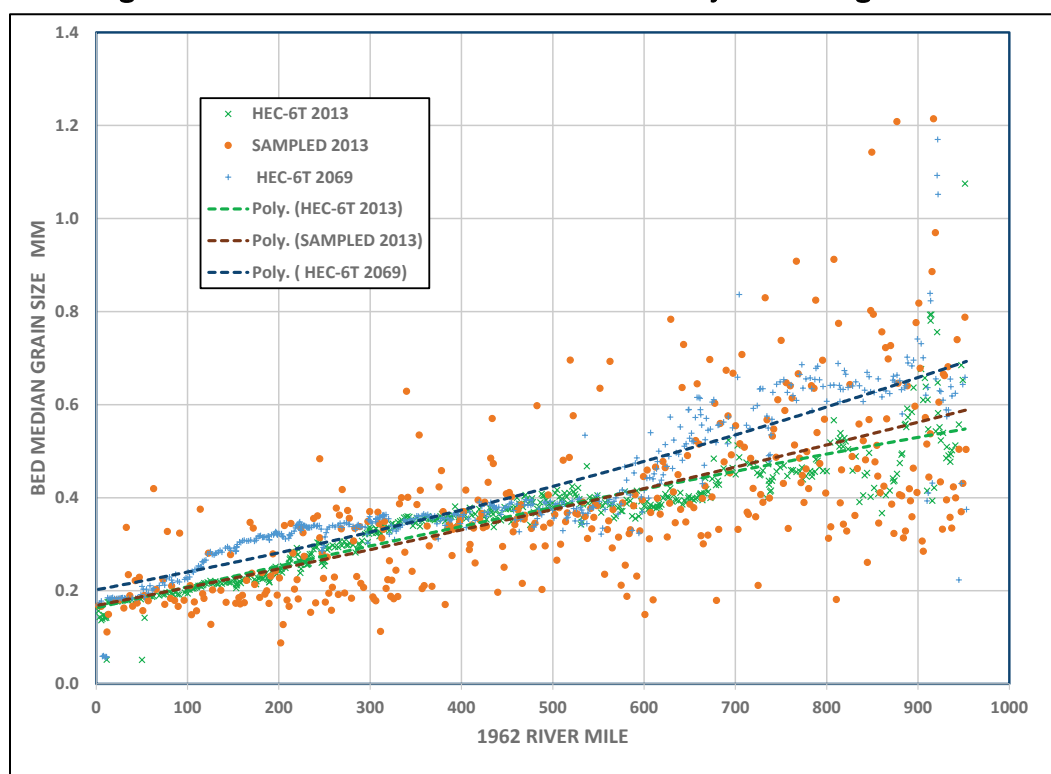


November 2013 calculated median grain sizes from the active bed-layer are compared to measured (Gaines and Priestas 2016) median grain sizes in Figure 37. Of note is the considerable scatter in the measured data. This is attributed to sampling primarily in the thalweg and not accounting for

lateral variations in the cross section. The calculated bed gradations represent the average cross-section data from the 1D model. Second-order polynomial regression lines in the figure show remarkable consistency between the measured and calculated data.

The 2069 calculated median grain sizes from the active bed-layer are also shown in Figure 37. The HEC-6T model predicts coarsening of the active bed layer for the Base (existing) Condition.

Figure 37. Measured and calculated active bed-layer median grain size.



2.16 Sediment concentrations in distributary outflows

The sediment concentrations that are diverted into distributaries are boundary conditions in the 1D model. The ratio of the sediment concentration in the distributary to the sediment concentration in the Mississippi River must be determined externally and coded as input data to the program. The ratio of diverted sediment concentration to the main channel sediment concentration upstream from the diversion is specified for each sediment size class. Typically, inlet channels at diversions are at a higher elevation than the average bed of the river and draw flow from the upper portion of the water column in the river. Coarse sediment

concentrations are higher at the bottom of the water column while fine sediment is more evenly distributed in the water column. Thus, the concentration diversion ratios are closer to 1 for finer sediment than for coarser sediment.

For most of the distributaries, estimates of vertical sediment concentrations were computed using the Rouse equation. Sediment diversion ratios were calculated for each size class by comparing the average Mississippi River bed elevation with the diversion sill or inlet channel elevation. This method accounts for the vertical variation in sediment concentrations, but it does not account for the lateral variation due to river meanders or for variable depths in a river cross section.

If the vertical velocity distribution and the vertical concentration distributions for each size class are known, then the sediment diversion ratios for the sediment load can be calculated from the following equation:

$$Diversion Ratio_i = \frac{\int_{y=h}^{y=D} C_{iy} u_y dy}{\int_{y=a}^{y=D} C_{iy} u_y dy}$$

where

C_i = concentration of sediment size class i

D = depth of flow in the main channel

h = height of diversion invert above average riverbed

a = reference height near the stream bed – taken to be zero for these calculations

u_y = velocity at depth y .

Using the Rouse (1937) equation for calculating sediment concentration, the following equation is derived by substitution.

$$Diversion Ratio_i = \frac{\int_{y=h}^{y=D} C_{ia} \left[\frac{D-y}{y} \frac{a}{D-a} \right]^Z u_y dy}{\int_{y=a}^{y=D} C_{ia} \left[\frac{D-y}{y} \frac{a}{D-a} \right]^Z u_y dy}$$

where

C_{ia} = concentration at the reference elevation

and

$$z = \frac{\omega}{\beta k u_*}$$

$$u_y = u_* 5.75 \log \left(\frac{30.1}{k_s y} \right)$$

where

ω = particle fall velocity

β = ratio of sediment and momentum exchange coefficients –
taken to be 1.0

k = Von Karman constant – taken to be 0.4

u_* = shear velocity

k_s = roughness height.

Removing the constant terms and converting from sediment load to concentration leaves the following equation, which can be solved using numerical integration. Hydraulic terms for this equation were determined at upstream river cross sections from HEC-6T backwater calculations for a range of discharges. Other terms are known.

$$\text{Concentration Diversion Ratio}_i = \frac{\frac{1}{D-h} \int_{y=h}^{y=D} \left[\frac{D-y}{y} \right]^Z u_y dy}{\frac{1}{D} \int_{y=a}^{y=D} \left[\frac{D-y}{y} \right]^Z u_y dy}$$

The Rouse (1937) equation describes sediment concentration due to vertical diffusion. The equation does not account for convective sediment transport or for lateral diffusion. Thus, this analysis does not account for sediment that may ramp up into the diversion inflow channel. Ramping may not be a significant contributor of sediment for diversions located on the outside of meander bends, but ramping may be significant on the inside of bends or at bar locations. Also, three-dimensional flow effects

that tend to turn the deeper flows toward the diversion due their lower velocities are not accounted for. Further, diversion inlet channels with specifically designed angles and locations along the primary flow channel are not accounted for. In the OMAR Model, sediment diversion coefficients were not changed during the simulations due to the effects of sea-level rise, degradation/aggradation, dredging operations, or with different ORCC operation scenarios. This was deemed appropriate due to lack of available data. Actual measurements at diversions and in the upstream main channel are the most reliable means of determining existing sediment diversion coefficients were used in lieu of the Rouse (1937) equation whenever data were available.

2.16.1 Southwest Pass distributaries

The invert elevation in the four Southwest Pass distributaries was estimated to be at elevation -10 ft. This estimate was based on 1992 and 2002 hydrographic survey data. Sills that may have been placed across these structures are accounted for only if they showed up in the hydrographic survey. At Outlet W-2, the sediment concentration ratios represent a combination of flow over the levees at high flow and flow through the distributary channel. The elevation of the natural levee was estimated to be 1.2 ft based on digital data used to develop the cross sections for HEC-6T. Sediment concentration diversion ratios used in Southwest Pass are tabulated in Table 23 through Table 26. Ratios for sediment coarser than fine sand were 0.0.

Table 23. Sediment concentration diversion ratios, Burrwood Bayou.

Discharge (cfs)	24,000	18,000	10,600	7,500	3,000	1
Clay	1.0	1.0	1.0	1.0	1.0	1.0
Very fine silt	1.0	1.0	1.0	1.0	1.0	1.0
Fine Silt	1.0	1.0	1.0	1.0	1.0	1.0
Medium silt	1.0	1.0	0.98	0.92	0.64	0.30
Coarse silt	0.90	0.85	0.65	0.50	0.09	0.0
Very fine sand	0.49	0.38	0.12	0.03	0.0	0.0
Fine sand	0.06	0.02	0.0	0.0	0.0	0.0

Table 24. Sediment concentration diversion ratios, Outlet W-2 and Overbank.

Discharge (cfs)	84,000	36,000	10,600	7,500	3,000	1
Clay	1.0	1.0	1.0	1.0	1.0	1.0
Very fine silt	1.0	1.0	1.0	1.0	1.0	1.0
Fine Silt	1.0	1.0	1.0	1.0	1.0	1.0
Medium silt	1.0	1.0	0.98	0.92	0.64	0.30
Coarse silt	0.88	0.85	0.65	0.50	0.09	0.0
Very fine sand	0.44	0.34	0.12	0.03	0.0	0.0
Fine sand	0.05	0.03	0.0	0.0	0.0	0.0

Table 25. Sediment concentration diversion ratios, Joseph Bayou.

Discharge (cfs)	24,000	18,000	10,600	7,500	3,000	1
Clay	1.0	1.0	1.0	1.0	1.0	1.0
Very fine silt	1.0	1.0	1.0	1.0	1.0	1.0
Fine Silt	1.0	1.0	1.0	1.0	1.0	1.0
Medium silt	1.0	1.0	0.98	0.92	0.64	0.30
Coarse silt	0.90	0.85	0.65	0.50	0.09	0.0
Very fine sand	0.49	0.38	0.12	0.03	0.0	0.0
Fine sand	0.06	0.02	0.0	0.0	0.0	0.0

Table 26. Sediment concentration diversion ratios, Outlet at RM -3.0.

Discharge (cfs)	12,000	9,000	5,300	3,800	1,500	1
Clay	1.0	1.0	1.0	1.0	1.0	1.0
Very fine silt	1.0	1.0	1.0	1.0	1.0	1.0
Fine Silt	1.0	1.0	1.0	1.0	1.0	1.0
Medium silt	1.0	1.0	0.98	0.92	0.64	0.30
Coarse silt	0.90	0.85	0.65	0.50	0.09	0.0
Very fine sand	0.49	0.38	0.12	0.03	0.0	0.0
Fine sand	0.06	0.02	0.0	0.0	0.0	0.0

2.16.2 Head of Passes to Baptiste Collette Distributaries

The invert elevations in South Pass and Pass a Loutre were estimated to be at elevations -31 ft and -17 ft, respectively. This estimate was based on the 2002 hydrographic survey. Sediment diversion ratios were calculated separately for the two major diversions and combined for the HEC-6T model. The combined ratios are tabulated in Table 27. Ratios for sediment coarser than medium sand were 0.0. This treatment is consistent with all previous ERDC, Mississippi Valley Division (MVD), and MBH HEC-6T models.

Table 27. Sediment concentration diversion ratios, South Pass and Pass a Loutre.

Discharge (cfs)	270,000	225,000	160,000	76,000	40,000	1
Clay	1.0	1.0	1.0	1.0	1.0	1.0
Very fine silt	1.0	1.0	1.0	1.0	1.0	1.0
Fine Silt	1.0	1.0	1.0	1.0	1.0	1.0
Medium silt	1.0	1.0	1.0	1.0	0.97	.93
Coarse silt	0.99	0.97	0.96	0.91	0.65	.37
Very fine sand	0.76	0.70	0.65	0.49	0.09	0.0
Fine sand	0.29	0.22	0.16	0.05	0.0	0.0
Medium sand	0.03	0.02	0.01	0.0	0.0	0.0

Flow occurs over the natural levee between Head of Passes and Venice and is labeled “Leaks” in the HEC-6T model. This outflow was assigned between cross sections at RM 2.46, and RM 1.6. Sediment diversion ratios for the overbank flow were taken from the MBH Myrtle Grove Model (Thomas 2012) and ERDC’s Hydrodynamic and Delta Management Model (Thomas et al. 2018) and are shown in Table 28. Ratios for sediment coarser than very fine sand were 0.0.

Table 28. Sediment concentration diversion ratios, “Leaks.”

Discharge (cfs)	130,000	87,000	48,000	28,000	20,000	1
Clay	1.0	1.0	1.0	1.0	1.0	1.0
Very fine silt	1.0	1.0	1.0	1.0	1.0	1.0
Fine Silt	1.0	1.0	1.0	1.0	1.0	1.0
Medium silt	1.0	1.0	1.0	1.0	1.0	1.0
Coarse silt	1.0	1.0	0.98	0.91	0.82	0.60
Very fine sand	0.50	0.50	0.50	0.50	0.50	0.50

The average inlet channel elevation in Cubits Gap was taken to be -13 ft based on 2009 surveyed cross sections. These surveys were conducted by ERDC. The 1992 hydrographic survey shows a sill across Cubits Gap. However, the existence of this sill was not confirmed by the recent surveys. The sill may be buried or it may have washed out. The sediment diversion ratios for Cubits Gap are tabulated in Table 29. Ratios for sediment coarser than fine sand were 0.0.

Table 29. Sediment concentration diversion ratios, Cubits Gap.

Discharge (cfs)	243,700	148,800	74,800	53,400	32,200	16,100	1
Clay	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Very fine silt	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Fine Silt	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Medium silt	1.0	1.0	1.0	1.0	0.97	0.84	0.70
Coarse silt	0.94	0.90	0.85	0.80	0.64	0.32	0.0
Very fine sand	0.57	0.48	0.37	0.27	0.09	0.0	0.0
Fine sand	0.12	0.05	0.02	0.0	0.0	0.0	0.0

Sediment diversion coefficients for West Bay were from the MBH Myrtle Grove Model (Thomas 2012), which were also used in the ERDC Hydrodynamic and Delta Management Model (Thomas et al. 2018). A ratio of 1.0 was assigned for all discharges for size classes between clay and very coarse sand. Ratios for sediment coarser than very coarse sand were 0.0.

The invert elevations in Grand Pass were very close to the average bed elevations in the Mississippi River according to the 2002 hydrographic survey. Sediment concentration diversion ratios were set equal to 1.0 for clay and silt and 0.9 for sands for all discharges. Ratios for sediment coarser than medium sand were 0.0.

The average invert elevation in Baptist Collette was taken from a 2009 survey conducted by MVN. The invert elevation in Baptiste Collette was - 45 ft. The sediment diversion concentration ratios for Baptiste Collette are tabulated in Table 30 Ratios for sediment coarser than medium sand were 0.0.

Table 30. Sediment concentration diversion ratios, Baptiste Collette.

Discharge (cfs)	117,000	75,000	44,000	18,000	1
Clay	1.0	1.0	1.0	1.0	1.0
Very fine silt	1.0	1.0	1.0	1.0	1.0
Fine Silt	1.0	1.0	1.0	1.0	1.0
Medium silt	1.0	1.0	1.0	1.0	1.0
Coarse silt	1.0	1.0	0.98	0.84	0.70
Very fine sand	0.89	0.83	0.68	0.24	0.0
Fine sand	0.49	0.34	0.12	0.0	0.0
Medium sand	0.08	0.02	0.0	0.0	0.0

2.16.3 Bohemia Spillway

The elevation of the natural levee along the Bohemia Spillway was determined by New Orleans District using 2004 Louisiana lidar data. Top of levee elevations were found to vary between 5.3 and 11.8 ft NGVD 29. Sediment concentration diversion ratios for the maximum discharge of 1,250,000 cfs were calculated using the average of the calculated water-surface elevation and the minimum levee elevation in each of the four reaches along the spillway. The sediment concentration diversion ratio for clay through medium silt was 1.0. The coarse silt sediment diversion ratio was 0.96. The very-fine sand diversion ratio was 0.6, and the fine sand diversion ratio was 0.14. The ratio for sediment coarser than fine sand was 0.0.

2.16.4 ORCC

At the ORCC, ratios for each size class were determined from an analysis of 1991–2015 measured suspended data. The following procedure was used to determine size class diversion percentages for each structure.

1. *Determine target sediment-diversion coefficients* – Measured suspended sand data from Union Point (UPT) and the Outflow Channel (ORO) were used to determine composite *target* sediment-diversion coefficients for each size class. The long-term, discharge-weighted, sediment-diversion coefficients for the combined structures should match the long-term target ORO/UPT sediment-diversions coefficients.

2. *Adjust the pump sample concentrations using ORO data* – Pump sample concentrations at each structure were adjusted so that the long-term average composite ORCC size class concentrations matched Outflow Channel-measured suspended load size class concentrations.
3. *Calculate the sediment diversion ratios for each size class* – Initial sediment diversion coefficients for each size class were calculated using the adjusted pump sample concentrations and Union Point suspended load concentrations. These were average values, not a function of discharge.
4. *Final adjustment of sediment diversion coefficients to match ORO/UPT target* – The sediment diversion coefficients were adjusted so that the long-term, discharge-weighted size class concentrations through the structures matched the target size class concentrations.
5. Overbank Structure diversion coefficients were determined using the Rouse (1937) method.

2.16.5 Determine target sediment diversion coefficients

The target diversion coefficients determine the total sediment diverted for each size class into the Outflow Channel from the Mississippi River. The target sediment diversion for each size class from the upstream Mississippi River station at Union Point into the Outflow Channel was determined using USACE/USGS measured data from Norton et al. (2019).

There was a relatively good correlation between size class concentrations and Union Point discharges but not a good correlation with Outflow Channel discharges. HEC-6T calculates the sediment diverted based on the diverted discharge. This suggested that sediment diversion coefficients should be average values rather than a function of discharge. This conclusion is consistent with previous HEC-6T studies.

Sand size class concentrations at UPT and ORO are plotted versus time in Figure 38 and Figure 39. These plots show ORO concentrations significantly lower than UPT for all size classes. Linear regression equations suggest a declining long-term trend in very fine sand (VFS) concentrations at both stations, maybe a very slight declining trend for fine sand (FS) and maybe a slight increasing trend for medium sand (MS). There are not enough data to develop a trend line for coarse sand (CS) as most measurements had no coarse sand. Coarse sand ratios were calculated as a simple average. Target sediment diversion ratios calculated

as a function of time between January 1994 and January 2014 using these linear regression equations are shown in Table 31. The average values in this table are the linear-regression target sediment-diversion coefficients.

Figure 38. UPT and ORO VFS concentrations 1994–2014.

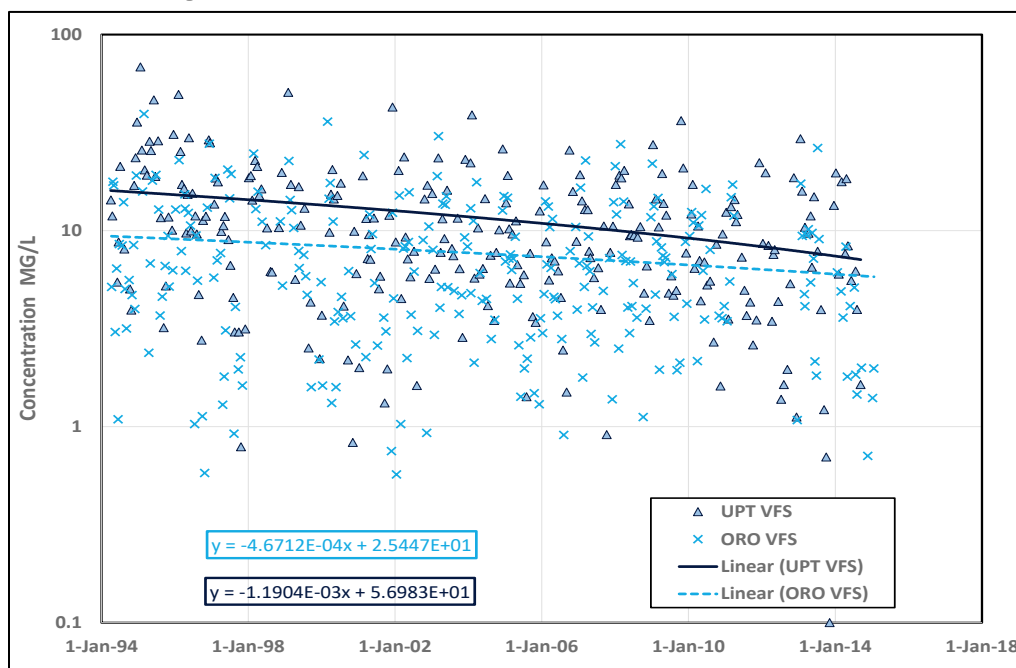


Figure 39. UPT and ORO FS and MS concentrations 1994–2014.

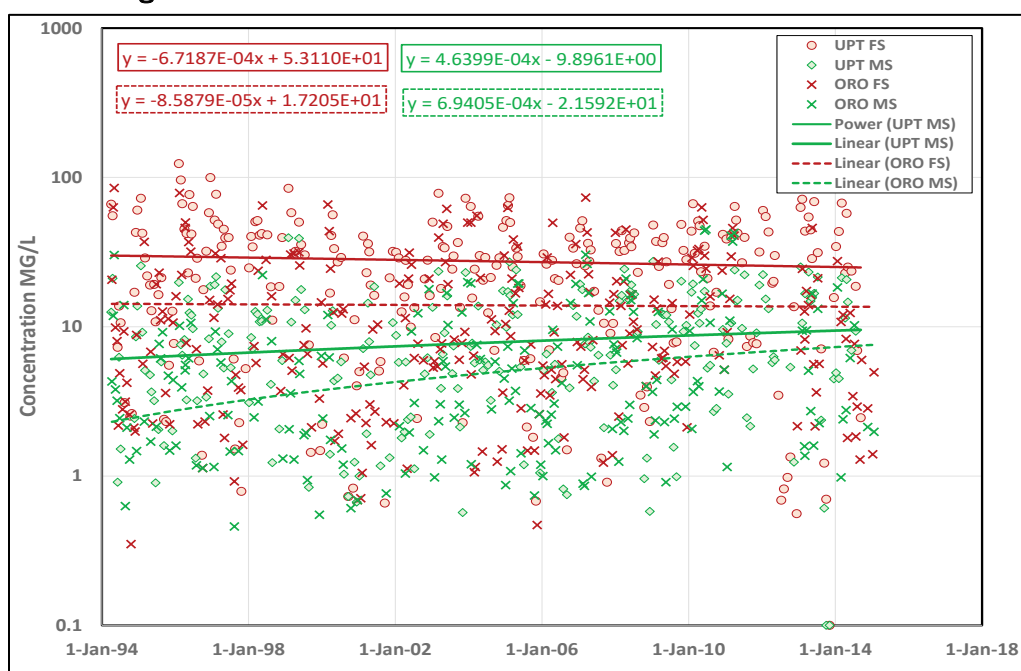


Table 31. Linear-regression sediment diversion ratios for Outflow Channel as a function of time.

	UPT (mg/L)	ORO (mg/L)	SD Ratio	Average
Very-fine sand 1994	16.111	9.408	0.584	0.696
Very-fine sand 2014	7.415	5.996	0.809	
Fine sand 1994	30.041	14.256	0.475	0.508
Fine sand 2014	25.233	13.629	0.542	
Medium sand 1994	6.035	2.238	0.371	0.573
Medium sand 2014	9.424	7.308	0.775	
Coarse sand 1994	0.097	0.034	0.347	0.405
Coarse sand 2014	0.155	0.717	0.463	

Another way to calculate the ORO/UPT target ratios is to use a straight average of ORO and UPT measurements weighted by discharge. This approach provides more weight to the higher discharges and reflects the sediment load rather than sediment concentration. There were 366 measurements at ORO between 25 May 90 and 20 Jan 2015. Twenty had coarse sand. Samples with no sand present were eliminated. UPT had 320 measurements between 26 April 1991 and 8 October 2014. Twenty-four had coarse sand. Samples with no sand present were eliminated. The equation for discharge weighting is the following:

$$\frac{ORO}{UPT} = \frac{\frac{\sum_{i=1}^4 Q(ORO)_i * C(ORO)_i}{\sum_i Q(ORO)_i}}{\frac{\sum_{i=1}^4 Q(UPT)_i * C(UPT)_i}{\sum_i Q(UPT)_i}}$$

where

Q = discharge

C = concentration

i = size class.

Calculated average sediment concentrations are higher with the discharge-weighted methodology. However, the target sediment diversion (SD) ratios from the time linear regression method and the discharge-weighted straight average method were very close. Calculated results using both methods are shown in Table 32.

Table 32. Target ORO/UPT SD ratios calculated two ways.

	Union Point mg/L	ORO mg/L	Target SD Ratio
From Time Regressions 1994-2014 – calculated for 2004			
Very-Fine Sand	11.762	7.702	0.655
Fine Sand	27.587	13.942	0.505
Medium Sand	7.730	4.774	0.618
Coarse Sand	0.126	0.053	0.418
From Discharge Weighted Straight Averages 1990–2014 (included zero concentrations for size classes)			
Very-Fine Sand	15.648	10.410	0.665
Fine Sand	38.010	19.559	0.515
Medium Sand	9.818	6.071	0.618
Coarse Sand	0.224	0.090	0.401

2.16.6 Adjust the pump sample concentrations using ORO data

Pump samplers at the Hydropower, Low Sill and Auxiliary Structures are used to collect sediment samples on a near daily basis. The pump samplers are not isokinetic, and considerable error can be expected with concentrations calculated from pump sampler data. This is especially true when there has been no attempt to calibrate the pump samples with samples collected using conventional sampling techniques. Therefore, concentrations reported at each structure are erratic. Because of this, an adjustment ratio for each size class was developed using a composite pump sample concentration and the ORO USGS measurements. The adjustment ratios were used to adjust pump size class concentrations to the ORO USGS concentrations, which are considered closer to the true values.

Composite pump concentrations were calculated using reported pump sample measurements taken between 1991 and 2015. The daily concentration at each structure was discharge weighted, using the reported discharge for each structure, to obtain the composite value. The overbank structure daily discharges, which were taken from USACE daily estimates, were included in the composite values. The overbank structure sediment concentration was assumed to be zero. Daily pump measurements that contained a reported size class concentration greater than 100 mg/L were eliminated from consideration. This was deemed reasonable because none of the 366 ORO samples collected during this

time period contained a size class fraction in excess of 100 mg/L. In addition to keeping the pump concentrations within a reasonable range, removing the high concentration samples reduced average bias.

The sediment size class adjustment ratios were calculated using two different methodologies. Results from the two techniques were then averaged to obtain the final adjustment ratios.

1. The first technique was to use all of the available data, between 1990 and 2015 (where no size class concentration in the measurement exceeded 100 mg/L) from the pump samples and ORO. There were 366 measurements at ORO between 25 May 1990 and 20 Jan 2015. The maximum size class concentration for any size class in these data was 92 mg/L. A straight average of the 366 concentrations was calculated for each size class. There were 6007 pump measurements at each structure between 15 Jan 1991 and 29 Jan 2015. The maximum size class concentration for any size class in these data was 1220 mg/L. Eliminating days with size class concentrations greater than 100 mg/L left 5861 events. Discharge-weighted straight averages for each size class were calculated from the 5861 events. The ORO/PUMP ratio was then calculated for each size class.
2. Between 25 Jan 1991 and 19 Sep 2014, there were 276 days with same-day measurements (i.e., when measurements were made both with the point samples at ORO and with the pump samplers at the ORCC structures). Days where pump samples that had a size class concentration greater than 100 mg/L were not included. Straight averages of these reported concentrations were calculated and the ORO/PUMP ratio was then calculated.

Calculated results are shown in Table 33. The final adjustment ratios for very-fine, fine, and medium sand were taken to be an average of the ratios calculated using the two methods. The results indicate that the pump samplers oversample the very-fine, while under estimating the fine and medium sand. There is a dearth of coarse sand data, so the ORO/PUMP correction ratio for coarse sand was determined using only the complete data set with no size class concentrations greater than 100 mg/L. A total of 774 of the 5861 pump samples contained coarse sand and 20 of the 366 ORO samples contained coarse sand. The same-day measurements were excluded because there were only 10 days at ORO with coarse sand. From Table 33 the ORO/PUMP ratio for coarse sand is 0.603.

Table 33. Calculated ratios to adjust pump sample concentrations to ORO concentrations.

Concentrations > 100 mg/L removed	Very-Fine Sand	Fine Sand	Medium Sand	Coarse Sand
All Measurements Discharge-Weighted Straight Average				
Composite Pump (5861*) mg/L	11.751	11.308	1.454	0.149
ORO (366*) mg/L	10.410	19.559	6.071	0.090
ORO/PUMP	0.886	1.730	4.174	0.603
Same Day Discharge-Weighted Straight Average				
Composite Pump (276*) mg/L	12.815	13.069	1.395	
ORO (276*) mg/L	10.755	19.808	5.740	
ORO/PUMP	0.839	1.516	4.113	
Average of Averages ORO/PUMP	0.863	1.623	4.144	0.603

*The number in parentheses represents the number of samples in that category.

2.16.7 Calculate the sediment diversion ratios for each size class

The index station for the sediment diversion ratios is Union Point (UPT). Between 26 Apr 1991 and 10 Sep 2014, 323 measurements were available at UPT. The maximum size class concentration at UPT was 148 mg/L. Therefore, for this analysis, the maximum allowable size class concentration for any size class from the pump measurements was taken to be 150 mg/L.

Several methods were tried. Calculating the SD ratio for each measurement produced unreasonable results. Calculating average concentrations for the entire data set (minus measurements with a size class concentration greater than 150 MG/L) and then calculating SD ratios proved to be a better approach. Using data from days when measurements were made at both UPT and at the structures was evaluated, but the number of such days was determined to be insufficient. Between 15 January 1991 and 21 December 2017, there were 250 same-day measurements at Union Point and Hydropower, 157 same-day measurements at Auxiliary, and 60 same-day measurements at Low Sill.

In the final analysis, sediment diversion coefficients were calculated using discharge-weighted concentrations of the entire data set (minus days with a size class concentration greater than 150 mg/L).

Between 15 Jan 1991 and 21 Dec 2017, there were 6,343 pump concentration measurements at Hydropower, 4,195 at the Auxiliary, and 1,145 at the Low Sill with size class concentrations less than 150 mg/L. Discharge-weighted straight averages of the size class concentrations at each structure were calculated and then adjusted using the discharge-weighted ORO/PUMP correction. This correction accounts for some of the error associated with the non-iso-kinetic pump samplers. The corrected average discharge-weighted concentrations were then divided by the discharge-weighted straight averages of the size class concentrations at Union Point to obtain the sediment diversion coefficients. The results of these calculations are shown in Table 34.

Table 34. Initial sediment diversion coefficients at ORCC from pump samplers and UPT measurements.

Concentrations > 150 mg/L removed	VFS	FS	MS	CS
Discharge Weighted Straight Average				
UPT (323*) mg/L	15.615	37.943	9.802	0.223
Hydropower (6343*) mg/L	11.882	8.841	0.503	0.097
Low Sill (1145*) mg/L	8.036	7.473	1.186	0.105
Auxiliary (4195*) mg/L	12.010	17.642	3.977	0.286
ORO/PUMP	0.886	1.730	4.174	0.603
Adjusted Hydropower mg/L	10.526	15.292	2.101	0.058
Adjusted Low Sill mg/L	7.119	12.926	4.950	0.063
Adjusted Auxiliary mg/L	10.639	30.514	16.600	0.164
Adjusted SD Hydropower	0.674	0.403	0.214	0.262
Adjusted SD Low Sill	0.456	0.341	0.505	0.283
Adjusted SD Auxiliary	0.681	0.804	1.694	0.735

*The number in parentheses represents the number of samples in that category.

2.16.8 Final adjustment of sediment diversion coefficients to match ORO/UPT target

A final adjustment to the sediment diversion coefficients was accomplished by checking the initial discharge-weighted sediment-diversion coefficients at the structures against the long-term average sediment diversion ratios between the Outflow Channel and Union Point. This was done by calculating the discharge weighted sediment diversion coefficients through each structure using daily discharge data (1 January 94 through 31 December 2014 from MVN records) and the average of averages from Table 34. An adjustment factor was calculated to reduce the discharge weighted SD ratios to the target ORO/UPT ratios. This adjustment assumes that the initial calculated sediment distribution was correct.

The final adopted sand concentration diversion ratios are tabulated in Table 35 and Table 36. Fine sediments were assigned concentration diversion ratios of 1.0, and sediments coarser than coarse sand were assigned a concentration ratio of 0.0.

Table 35. Sediment diversion ratios at ORCC 1991–2019.

	Hydropower RM 315.9	Low Sill RM 314.6	Auxiliary RM 312.0
Very-fine sand	0.68	0.49	0.73
Fine sand	0.43	0.42	0.98
Medium sand	0.24	0.57	1.90
Coarse sand	0.29	0.18	0.46

Table 36. Sediment diversion ratios for Overbank Structure.

Discharge CFS	250,000	200,000	50,000	4,000	2,500	1
Very-fine sand	0.88	0.84	0.78	0.74	0.0	0.0
Fine sand	0.50	0.43	0.34	0.28	0.0	0.0
Medium sand	0.14	0.09	0.05	0.03	0.0	0.0

3 Model Calibration and Verification

The HEC-6T model used in this study was developed from the Flowline Model that had been calibrated and verified for use on the Mississippi River between Cairo, IL, and the Gulf of Mexico. Additional calibration and verification, focusing specifically on recent dredging data, was conducted for the OMAR Assessment.

3.1 Calibration and verification of HEC-6T Flowline Model

The Flowline Model was a cooperative effort of the USACE New Orleans, Vicksburg, Memphis and Saint Louis Districts and the Mississippi Valley Division. The model extended between the Gulf of Mexico and Chester, IL, on the Middle Mississippi and Metropolis on the Ohio River and simulated over 1000 mi of the river.

Calibration of the Flowline Model is discussed in Copeland (2018). In summary, the calibrated model accomplished the following:

1. Matched 1991 and 2002 water-surface elevation rating curves at gages between Cairo and the Gulf. The calibration included data from the high discharge events during the 2011 flood.
2. Matched measured sand concentrations at Union Point (RM 326), Coochie (RM313), Tarbert Landing (RM 306) and Belle Chasse (RM 76).
3. Matched reported dredging in Southwest Pass and Head of Passes (1975–1984 and 1991–2002).
4. Duplicated measured specific gage trends at gages.
5. Duplicated 1991–2002 degradation caused by the construction of Smithland Dikes (RM 299).
6. Simulated 1991–2002 aggradation of a point bar between the Low Sill and Auxiliary Structures.

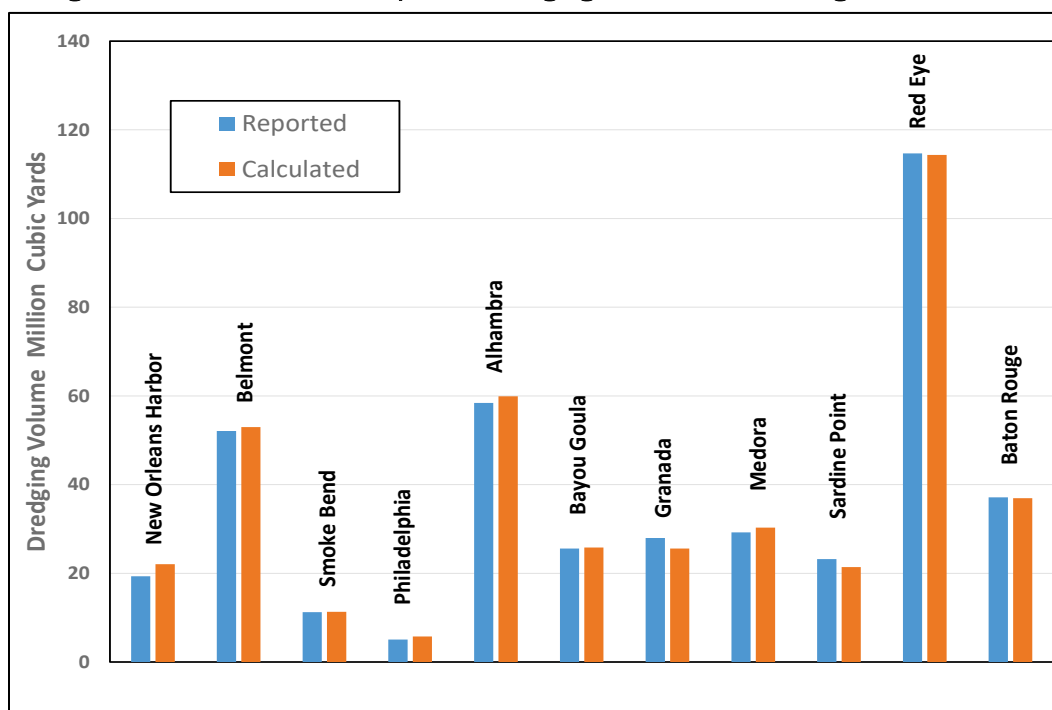
3.2 Dredging at deep-draft crossings downstream from Baton Rouge

The Flowline Model was subsequently modified to evaluate the long-term effects of dredging operations at the Mississippi River deep-draft crossings below Baton Rouge (Copeland et al. 2019). The objective of the deep-draft model study was to investigate the possible advantages of overbank disposal versus the current practice of in-channel disposal. The HEC-6T model used in the Flowline Assessment did not simulate dredging at the

deep-draft crossings because the dredged material was not removed from the river, but just passed on downstream. The deep-draft crossing model provided additional calibration and verification, especially related to dredging simulations which were incorporated into the model used in the OMAR Assessment.

Modifications necessary to properly simulate reported dredging included adding cross sections to better define the limits of dredging reaches, adjustments to the width and depth of dredging templates, adjustments to the movable bed widths and inclusion of more dredging cycles each yr. Dredging operations were simulated for a 23 yr calibration period. Reported dredging quantities were successfully duplicated at dredging sites downstream from Baton Rouge as shown in Figure 40. The 50 yr future simulations included the proposed increased dredging depth to 50 ft below low water reference plane.

Figure 40. Calculated and reported dredging volumes at crossings 1991–2013.



3.3 Dredging downstream from Venice

Calculated average annual dredging quantities were compared to MVN reported contract dredging volumes as part of the model verification for this study. The 29 yr period between October 1991 and October 2019 was used for the verification. Results are shown in Figure 41 and Table 37. Agitation

dredging is not included in the totals. Results from four time periods were compared because 2019 was an exceptionally high runoff year and could bias results. Also, calculated results are based on a simplistic and constant dredging schedule and can deviate significantly from actual operations. In these verification simulations, dredged material was removed instantaneously every day. Calculated and reported volumes compared favorably.

Figure 41. Calculated and reported dredging downstream from Venice 1991–2019.

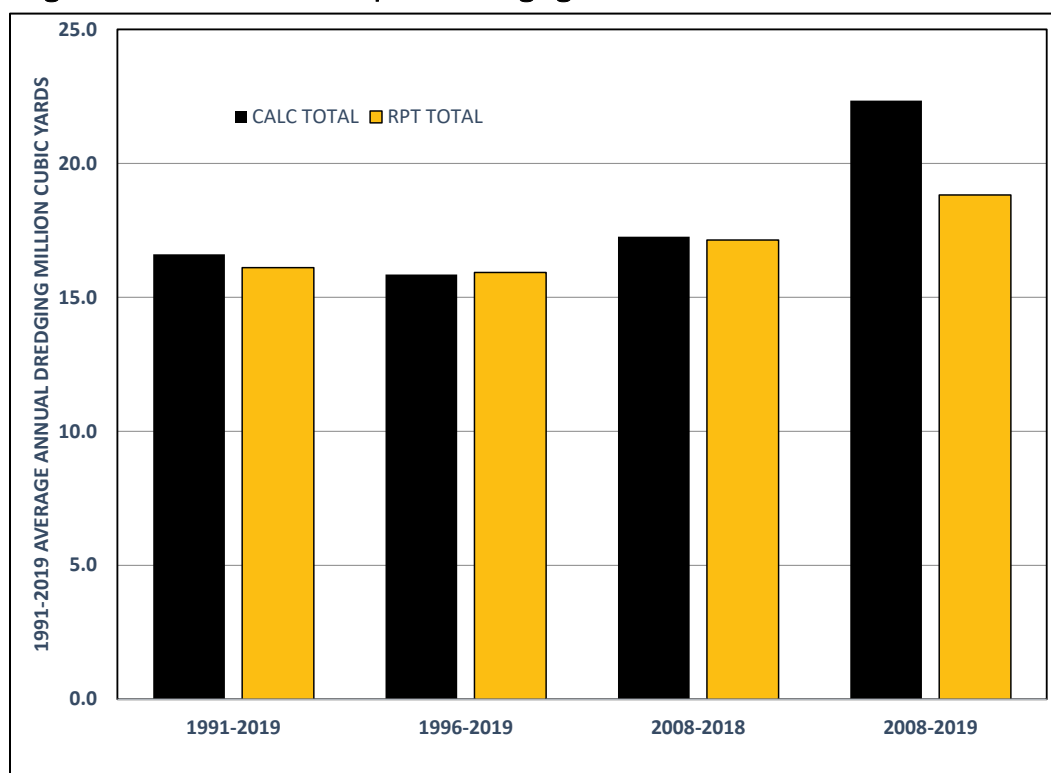
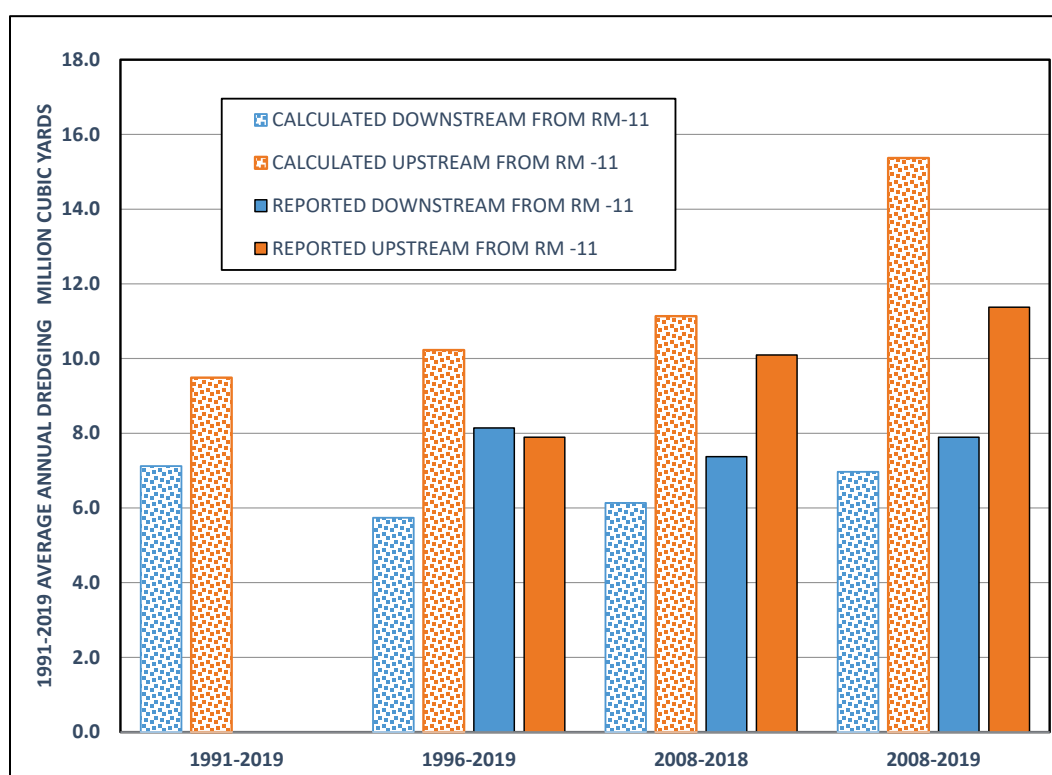


Table 37. Calculated and reported dredging downstream from Venice 1991–2019.

	HEC-6T Calculated MCY			Reported (Minus Agitation) MCY		
	< RM -11	> RM -11	Total	< RM -11	> RM -11	Total
1991–2019	7.12	9.49	16.61			16.11
1996–2019	5.74	10.23	15.85	8.14	7.90	15.93
2008–2018	6.13	11.13	17.27	7.37	10.10	17.14
2008–2019	6.97	15.37	22.34	7.90	11.38	18.82

The distribution of dredging above and below RM -11 can also be compared because that location generally is the line of demarcation between hopper dredged material transported and dumped in the Hopper Dredge Disposal Area (HDDA) at Head of Passes and the Ocean Dredged Material Disposal Area (ODMDS) downstream from the Southwest Pass Jetties. These disposal areas are included with the dredging volumes in the MVN contract reports after 1996. The locations of cutterhead dredging are typically defined in the MVN contract reports. Results are shown in Figure 42. The calculated and reported data suggest that more dredging occurs above RM -11 than below RM -11. Year 2019 was indeed an exceptionally high runoff year, significantly affecting the 11 yr average annual dredging volumes, especially upstream from RM -11.

Figure 42. Reported and calculated dredging upstream and downstream from RM -11.



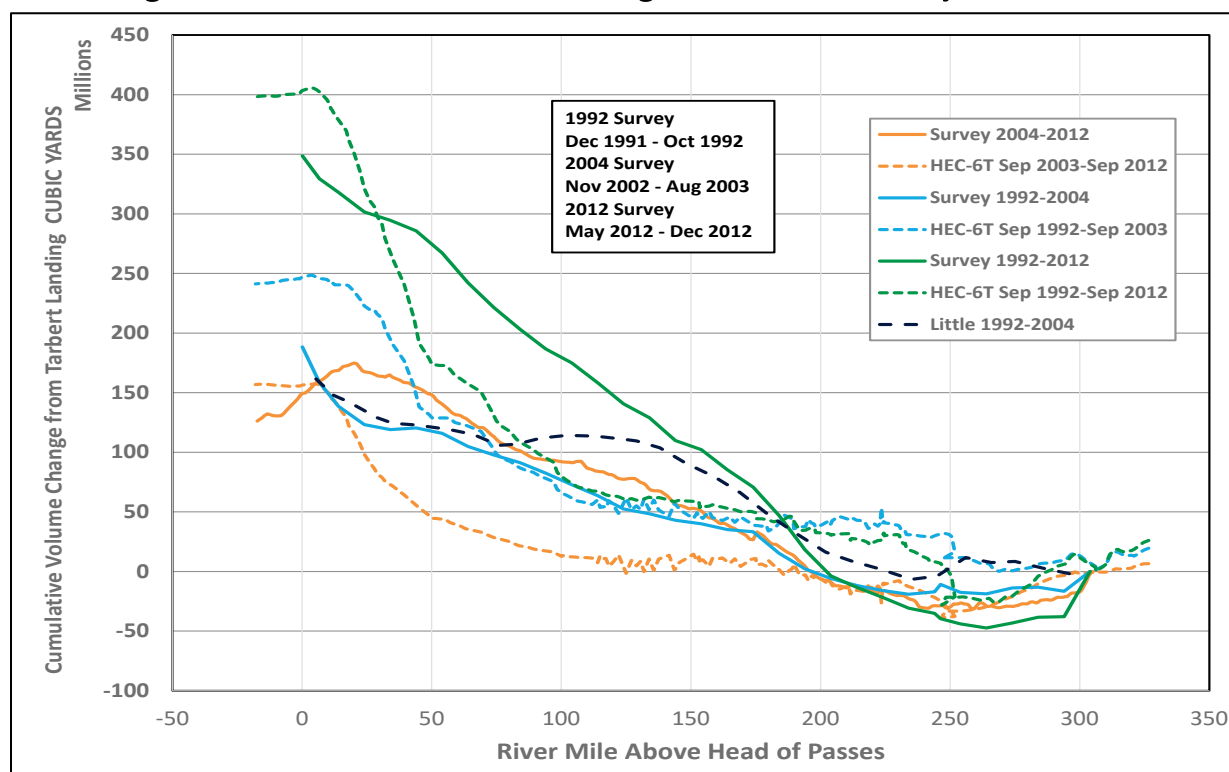
3.4 Volume calibration

Comprehensive hydrographic surveys of the Mississippi River in MVN were taken in 1992, 2004, and 2012. The cross-section surveys are not directly comparable because of time and space differences. The 1992 surveys were taken between December 1991 and October 1992, the 2004

surveys between May and August 2003, and the 2012 surveys between May and December 2012. The time difference between cross-section surveys presents a problem because surveys at a particular river mile may not have been surveyed at the same time of year or at the same location on the annual hydrograph. Differences in surveyed cross-sectional areas may be attributed to normal annual variations caused by the rising and falling of the annual hydrograph. In addition, dredging at specific crossings in relation to the hydrographic survey may influence the comparison. All this to say, annual bed elevation oscillations at cross sections may be more significant than long-term bed volume changes. Therefore, considerable uncertainty is associated with calculating volume changes from hydrographic survey data.

However, uncertainty is the trademark of sediment data, so calculated and measured volume changes are typically conducted to assess numerical model performance even though it makes just as much sense to use the numerical model to test hydrographic survey accuracy. Thomas (2012) calculated accumulated volume changes from Tarbert Landing to the Gulf between the 1992 and 2004 hydrographic surveys. Little and Biedenharn (2014) used the same 1992 and 2004 hydrographic surveys, but a different methodology, to calculate accumulated volume changes between 1992 and 2004. These two studies produced similar results for accumulated bed volume changes between the 1992 and 2004, as shown in Figure 43. Dahl et al. (2018) calculated accumulated volume changes from Tarbert Landing to the Gulf between the 2004 and 2012 hydrographic surveys. They then compared these results with accumulated volume changes calculated with their HEC-RAS numerical model. They reported that their model captured the overall trend of deposition and erosion. They also cautioned that there are some uncertainties in the calculation of the measured volume change. Bed change can vary laterally, longitudinally, and temporally, especially in the presence of moving bed forms. The use of volume change, especially longitudinal cumulative volume change, can help to mitigate these factors. HEC-6T calculated volume accumulation is compared to the Thomas (2012), Little and Biedenharn (2014), and Dahl et al. (2018) survey calculations in Figure 43. The 1992–2004 results are relatively consistent. The 2004-2012 results produce about the same total sediment accumulation, but HEC-6T calculates more deposition downstream from New Orleans (RM 102), while the survey data suggests a more uniform deposition pattern downstream from Baton Rouge (RM 229).

Figure 43. Accumulated bed volume change calculated from surveys and HEC-6T.



4 Flow Diversion Percentages for Scenarios

Flow diversion distributions were varied in eight different scenarios and sedimentation effects were compared to the Base condition. In addition, sedimentation effects were compared to the Base Condition for three dredging scenarios where sediment was dredged from the Mississippi River at the ORCC. These eleven scenarios are briefly described below:

- **Base Condition** - This scenario continues the current (post 2013) operations (no flushing is simulated).
- **Scenario 3** - (R1) defined in the Old River Control O&M Manual, (USACE, MVN 1988): There is no hydropower diversion, and most of the flow goes through the Auxiliary Structure.
- **Scenario 4** - (R1WH) Defined in the Old River Control O&M Manual, (USACE, MVN 1988). Most of the flow goes through the Hydropower and Auxiliary Structures. The Low Sill Structure is not used in this scenario.
- **Scenario 5** - (HF6040) A 60/40 latitude flow is maintained during high flow, but a 70/30 latitude flow is maintained on a long-term basis through the use of a payback/tracking system.
- **Scenario 6** - (HF8020) A 80/20 latitude flow is maintained during high flow, but a 70/30 latitude flow is maintained on a long-term basis through the use of a payback/tracking system.
- **Scenario 7** - (CTAR125) Increase diversion through the ORCC to maintain a maximum discharge of 1,250,000 cfs at Tarbert Landing (up to the capability limit of the Atchafalaya). But a 70/30 latitude flow is maintained on a long-term basis through the use of a payback/tracking system.
- **Scenario 8** - Change the latitude flow to 80/20 on a daily basis.
 - 8A - (D8020) Without any constraints
 - 8B - (D8020WLSC) With Low Sill Head Differential constraint.
- **Scenario 9**- (D6040) Change the latitude flow to 60/40 on a daily basis.
- **Scenario 10** - (MAXAUX) Maintain the latitude flow at 70/30. Increase Auxiliary and decrease Hydropower (to ~ 1/3 ORCC flow), based on conditions.
- **Dredging Scenarios:** These three scenarios have the same hydrograph as the Base Condition. 2.10 million cy/yr of sediment is dredged from the Mississippi River between the Low Sill and Auxiliary

Structures. The scenarios differ in the dredging rates and disposal locations.

4.1 Base Condition

Sedimentation effects of each scenario were compared to those calculated for the Base Condition. All of the scenario simulations and the Base Condition started with 2019 calculated sediment and geometry conditions and simulated sedimentation processes for 50 yr into the future. The 50 yr hydrograph was developed from 1969–2019 hydrologic data at the model boundaries. The diversion percentages through each ORCC structure for each scenario were calculated from the 1991–2019 historical daily data, based on the assigned protocol for that scenario.

The daily discharges through each ORCC structure were based on long-term average diversion percentages that were calculated from the assigned daily allocations for each scenario. It was not necessary to assign actual daily percentages in the model to achieve the objective of comparing long-term sedimentation effects. For the Base Condition, the distribution of diverted flow through the various ORCC structures was based on data collected between 2013 and 2019, which represents the existing operation protocol.

4.2 Ratio 1 Scenarios (R1 and R1WH)

The Ratio 1 Scenario (R1) includes flow through the Overbank, Low Sill and Auxiliary Structures. There is no flow assigned to the Hydropower Structure. Diversion percentages were taken from the ORCC Operation and Maintenance Manual (Revised Table 3, USACE MVN 1988), which provides diversion percentages through each structure as a function of total ORCC discharge up to 620,000 cfs. Using the total ORCC discharge as the independent variable to determine the discharge allocation to each structure is the logical approach to facilitate structure operations, which strives to maintain the designated 70-30 total latitude flow distribution. However, the HEC-6T numerical model diverts flow out of the Mississippi River as a function of the discharge upstream from the diversion. Therefore, a regression relationship between ORCC discharge and Mississippi River discharge was developed. Using the diversion percentages equations for each ORCC structure from the Operation

Manual and the long-term average regression equation, diversion percentages as a function of the Union Point discharge were calculated.

The Ratio 1 with Hydropower (R1WH) scenario uses the same Union Point versus ORCC rating curve used for R1. R1 Low Sill discharges are assigned to the Hydropower Structure up to a maximum of 150,000 cfs, with the remainder assigned to the Auxiliary Structure. There is no diversion at the Low Sill Structure. Overbank diversions are the same as R1.

4.3 Scenarios 5–10

Assigned daily discharges through each of the Old River Control Complex (ORCC) structures for each scenario are located in Excel file, *OMAR_AllScenarios_20210106.xlsx*. These discharges were calculated based on specified scenario allocations using 1 January 1990 through 31 December 2019 reported discharge data as a reference. Daily discharges at Union Point for each scenario were calculated as the sum of Tarbert Landing and (ORCC) daily discharges from the Excel file.

HEC-6T requires that diversion percentages be assigned as a function of an upstream discharge. Therefore, a relationship between daily ORCC and Union Point discharges was required. Regression equations were calculated in Excel using both 1990–2019 and 2013–2019 daily data. The 1990–2019 regressions were adopted to determine the ORCC versus Union Point relationship because the flow distributions through the individual structures do not affect the overall distribution and the data base includes a larger discharge range.

The relationship between Hydropower and Union Point discharge was based on 26 May 1990 to 31 December 2019 data. Hydropower was opened 26 May 1990. Hydropower flow distribution is generally independent from the flow distribution through the other structures.

Overbank Spillway diversion percentages, as a function of Union Point discharge, were taken from scenario allocations that had been calculated using historical 1 October 2012 through 30 September 2019 data as a reference. The Overbank Spillway was not used between 1990 and 2012, and the 2013–2019 scenario allocations reflect future operation conditions and were used to develop the 50 yr predictive rating curve. Flow does not occur over the Overbank Spillway until Union Point discharge exceeds

1,100,000 cfs. The Overbank Spillway discharge was capped at 320,000 cfs, which is the structure's capacity.

The Low Sill and Auxiliary diversion percentages for Scenarios 5 through 10 did not correlate well with the upstream Mississippi River discharge. This is different from the two Ratio 1 scenarios (R1 and R1WH) in that the Ratio 1 scenarios were based on a systematic annual relationship between the ORCC diversions so that diversion percentages could be based reliably on upstream discharges. In contrast, the Scenario 5 through 10 relationships were based on daily historical data poorly correlated to upstream discharge. Consequently, the Low Sill and Auxiliary diversion percentages were obtained by determining the total diversion percentages through the two structures and then assigning a straight average of the long-term percentages through each of the two structures.

Low Sill and Auxiliary diversion percentages for the 50 yr future simulations were calculated using daily discharges between 2013 and 2019. The shorter data base was used because diversion percentages calculated in *OMAR_AllScenarios_20210106* were influenced significantly by the non-usage of the Overbank Structure between 1991 and 2012. Using daily discharges between 2013 and 2019, a 7 yr average diversion percentages was calculated for each structure.

The daily discharge upstream from the Low Sill Structure was calculated by subtracting Hydropower and Overbank daily discharges from the Union Point discharge. The Hydropower discharge was calculated from the 1990–2019 regression equation and the Overbank discharge was calculated from the 2013–2019 regression equation. The 2013–2019 discharges through the combined Low Sill and Auxiliary Structures were calculated by subtracting the Hydropower and Overbank daily discharges from the ORCC discharge, which was determined from the 1990–2013 regression curve.

The historical data base limited the reliability of the regression equations to a Union Point discharge range between 200,000 and 2,300,000 cfs. Maximum Union Point discharge during the 50 yr simulation is 2,542,460 cfs. Low Sill discharge capacity was set at 300,000 cfs. Auxiliary discharge capacity was set at 350,000 cfs.

4.4 Details of scenario flow diversion calculations

Detailed information related to flow diversion calculations for each scenario is contained in Appendix A. Included are figures showing calculated regression curves and tables showing the HEC-6T inputs.

5 Model Results

Numerical models are typically classified as computational models or computational analysis models (Thomas and Chang 2008). Computational models have had their parameters calibrated to match a known event, and then verified to match another event without changing any of the assigned internal parameters. Computational analysis models are models where data are insufficient for calibration. Results from computational analysis models can still be useful as they account for the study area response to the systematic application of hydrodynamic and sedimentation theories. A computational analysis can provide sufficient understanding to predict how reliable a plan will be by comparing the calculated results from the plan test with those from an existing or Base Condition. The objective of this numerical model investigation was to determine the long-term and system-wide sedimentation effects that would occur in the Mississippi River if the operation protocols at the ORCC were changed. One could argue that this objective could be met with a computational analysis model since ranking of different scenarios as to their relative effect on sedimentation processes could meet the objective. However, the HEC-6T model used in this study meets the criteria for a computational (calibrated and verified) model.

Model parameters have been set and calibrated in previous studies. Specifically, in Copeland and Thomas (1992), Copeland (2018), and Copeland et al. (2020); and by association in Thomas (2012), Sharp et al. (2013), Heath et al. (2015), Heath et al. (2017), and Thomas et al. (2018). Results have also been verified in those previous studies as discussed in Chapter 3. That said, the model results from this assessment are not free from uncertainties. Much of the available sediment data are not comprehensive, especially bed load data. There is uncertainty associated with the geometric data and possible underlying stratum. Water and sediment diversion measurements are limited. There is uncertainty associated with assuming that the current hydrologic and sediment supply conditions will extend into the future. Despite uncertainties, the model results from this assessment can be reliably used to assess the sedimentation effects of the various scenarios.

The following model results are presented both quantitatively (elevations, volumes, weights) and qualitatively (percent difference). The qualitative results are more certain. However, both are considered

reasonable estimates of future conditions and can be used to demonstrate the sedimentation effects associated with changing operation protocols at ORCC.

5.1 Long-term sedimentation trends in Lower Mississippi River

One advantage in using the HEC-6T model of the entire Lower Mississippi River from Cairo, Illinois to the Gulf of Mexico is that long-term geomorphological changes are accounted for. Previous studies have demonstrated aggradation and degradation trends in various reaches of the Lower Mississippi River. Specific-gage studies (Biedenharn et al. 2017) demonstrated historical trends, and numerical model studies (Copeland 2018) predicted future trends. Trends generated by the HEC-6T model are a consequence of sedimentation processes and reflect channel aggradation or degradation. Trends generated by specific gage studies of historical data may reflect other processes, including variations in flood plain land use, chute and neck channel cutoffs, changes in tributary sediment loads not included in the HEC-6T model. The fact that these specific gage calculation methods produce consistent results suggests that sedimentation processes are the dominant factors driving long-term changes in water-surface elevations. Results from the numerical model and historical trend studies are shown in Figure 44 through Figure 50. Figure 45 shows a consistent degradation trend at Memphis between 1930 and 2065. There was a degradation trend at Vicksburg between 1930 and 1950 followed by an aggradation trend beginning in 1950 through 2065 (Figure 48). A consistent aggradation trend between 1940 and 1965 was calculated at Red River Landing (Figure 50). These trends are expected to affect sediment delivery and dredging in Southwest Pass and above Head of Passes. The numerical model accounts for the long-term change in sediment delivery and its effect on dredging below Venice.

Figure 44. Specific gage at Hickman RM 922.

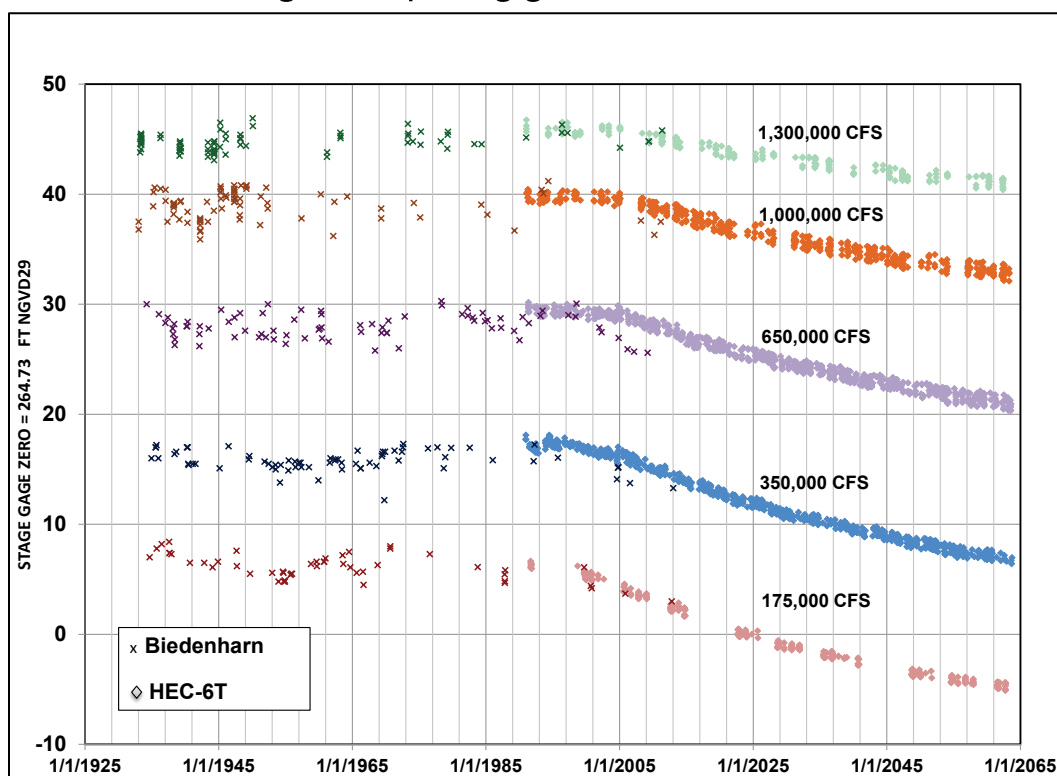


Figure 45. Specific gage at Memphis – RM 734.4.

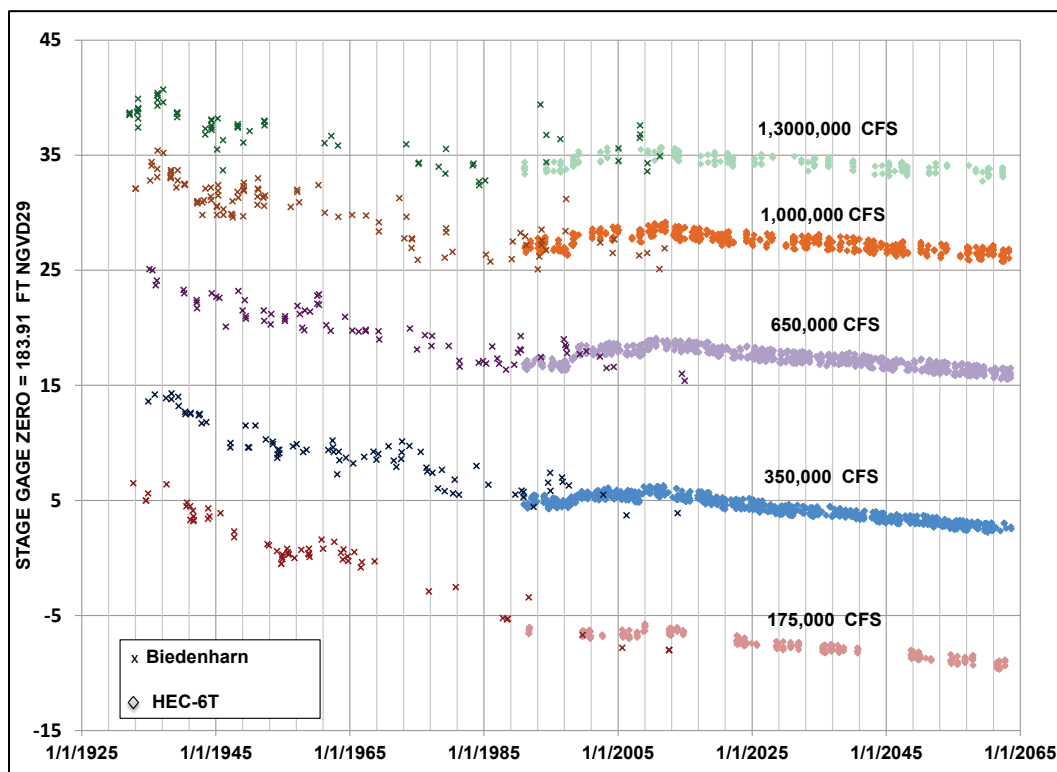


Figure 46. Specific gage at Helena RM 663.

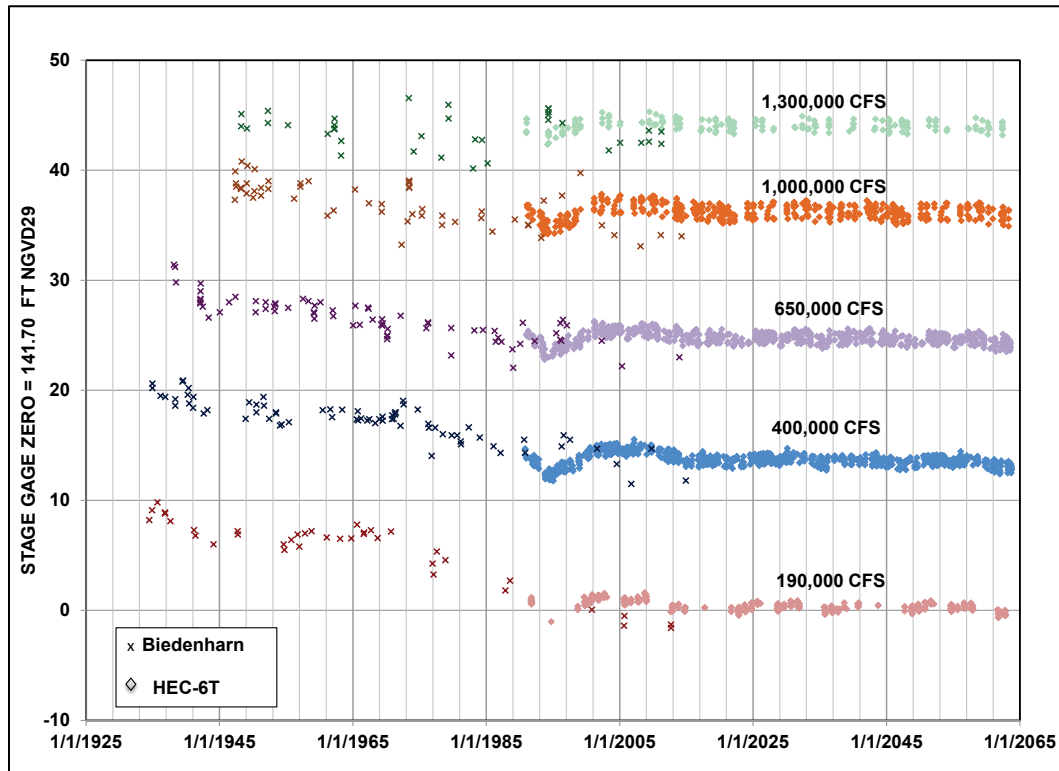


Figure 47. Specific gage at Arkansas City – RM 554.1.

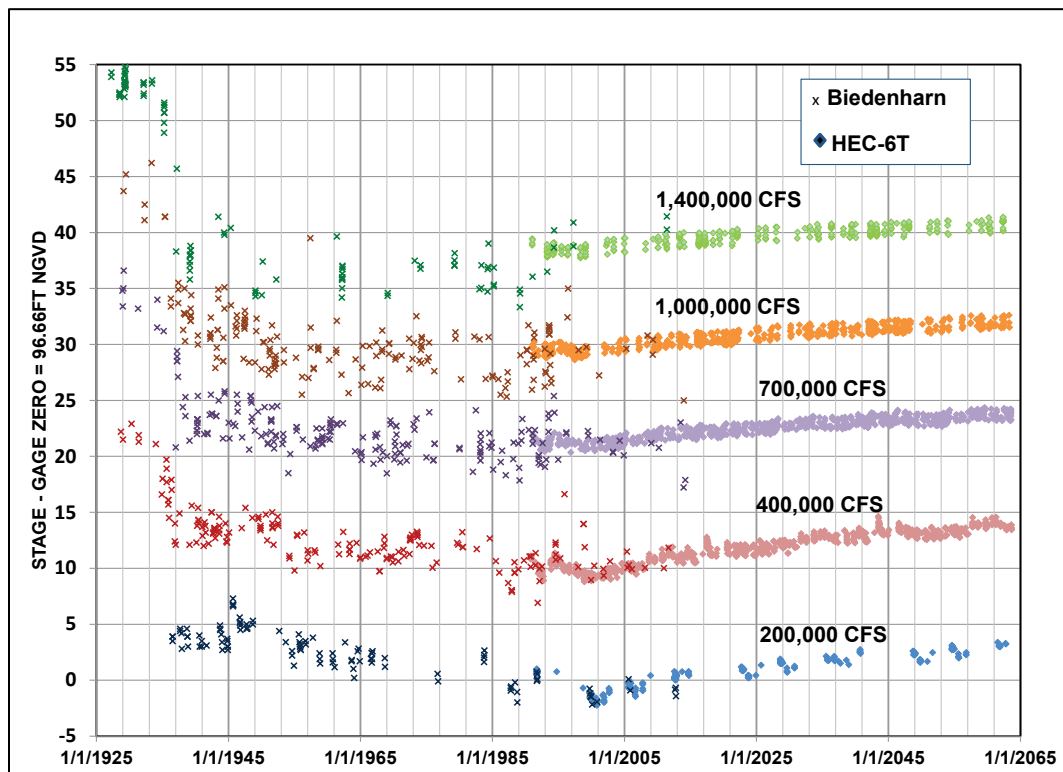


Figure 48. Specific gage at Vicksburg – RM 435.7.

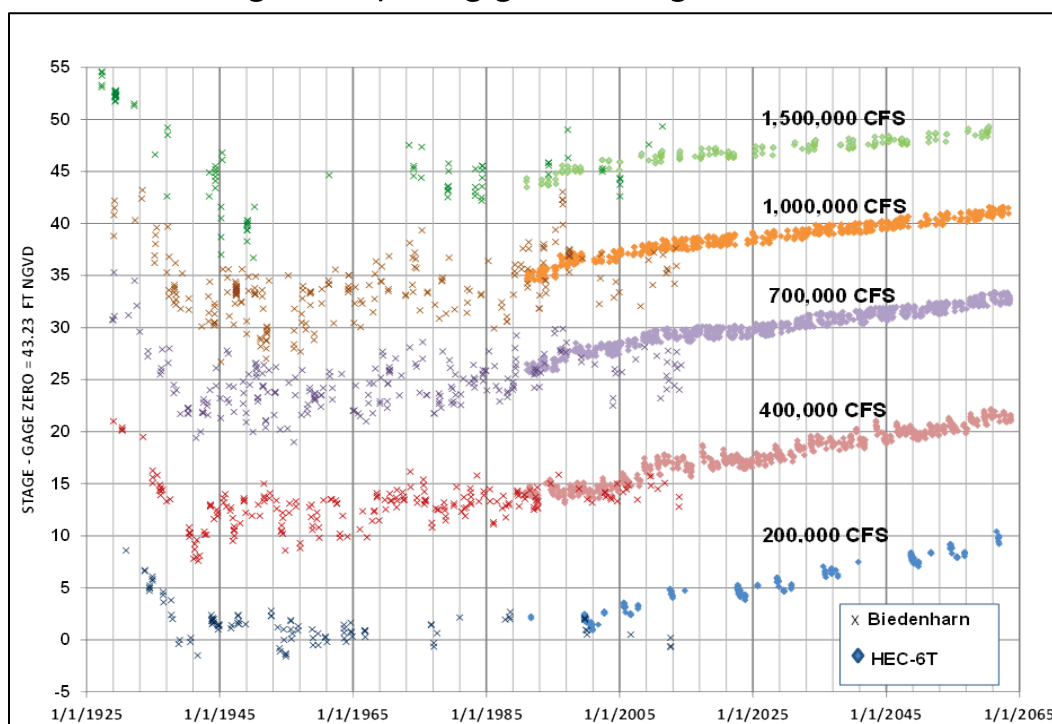


Figure 49. Specific gage at Natchez RM 363.3.

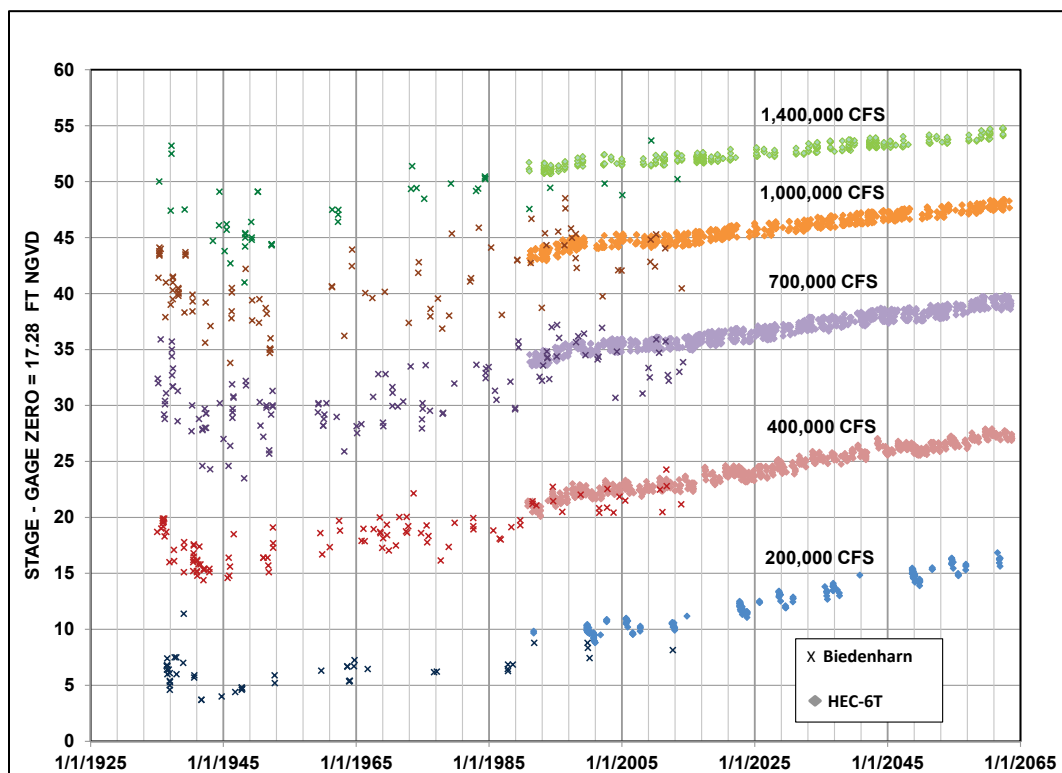
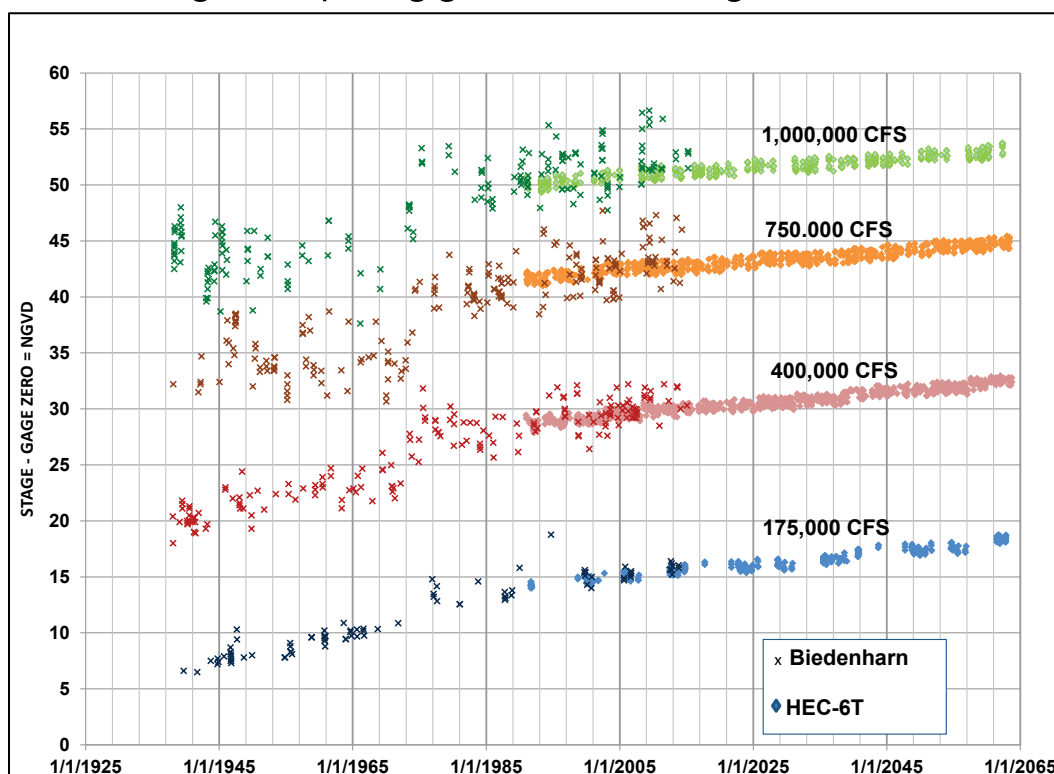


Figure 50. Specific gage at Red River Landing – RM 302.4.



5.2 Projected long-term water-surface profile trends downstream from ORCC

The numerical model was used to predict long-term trends in water-surface elevations with the Base Condition operations at the ORCC. Calculated daily water-surface elevations at Bayou Sara (RM 265.1), Baton Rouge (RM 228.1), Donaldsonville (RM 175.4), and New Orleans (RM 102.2) were used to develop specific gage plots for five discharges varying between 175,000 and 1,400,000 ft³/sec. The direct-step method was used for the specific gage analysis. Results are shown in Figure 51 through Figure 54.

With the direct-step method, water-surface elevations are calculated by HEC-6T every day for the 50 yr simulation. Water-surface elevations calculated for discharges within a specified range (bin) are segregated for plotting. If a bin size of $\pm 2.5\%$ is selected, as was done in this research, a nominal discharge of 10,000 ft³/sec would represent discharges between 9,750 ft³/sec and 10,250 ft³/sec. Calculated water-surface elevations within the selected discharge ranges are plotted against time to produce a specific gage record. The direct step method produces a scatter of data

points about the average trend curve. In the case of computed water-surface elevations from the steady-state numerical model the scatter is due to the discharge range in the selected bin.

Just downstream from the ORCC at Red River Landing, the specific gage analysis (Figure 50) demonstrated an increasing trend in water-surface elevations over the next 50 yr. Downstream at Bayou Sara (Figure 51), the first 25 yr suggest a stable trend in water-surface elevations that eventually changes to a slight increasing trend in the final 25 yr. At Baton Rouge (Figure 52), which is at the upstream end of the deep draft navigation channel, and at Donaldsonville (Figure 53) the specific gage plots show no discernable trends. At New Orleans (Figure 54) there is a slight increasing trend at the lower discharges, likely due to predicted sea level rise.

Figure 51. Bayou Sara RM 265.1 calculated 50 yr specific gage.

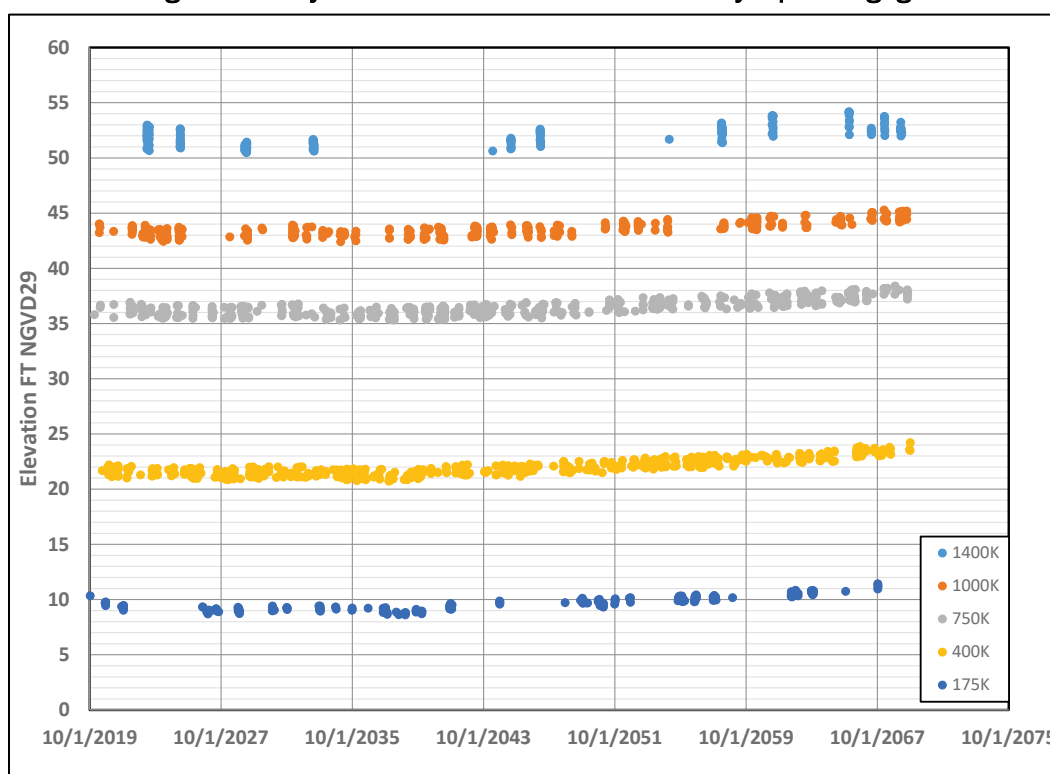


Figure 52. Baton Rouge RM 228.1 calculated 50 yr specific gage.

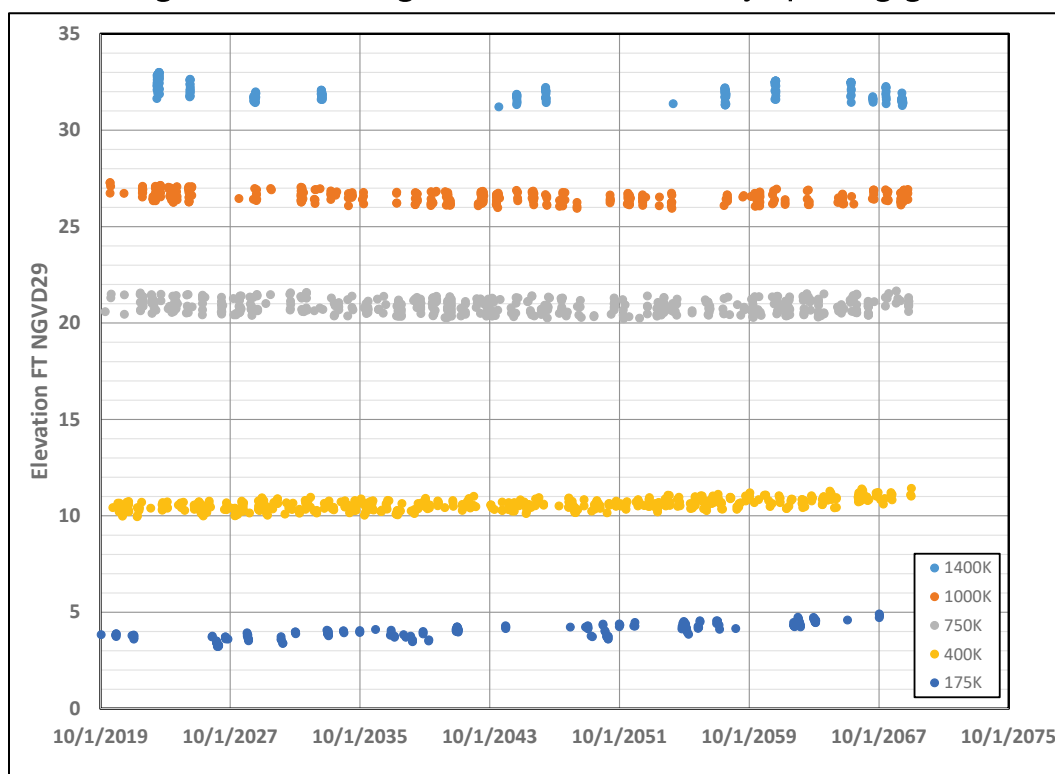


Figure 53. Donaldsonville RM 175.4 calculated 50 yr specific gage.

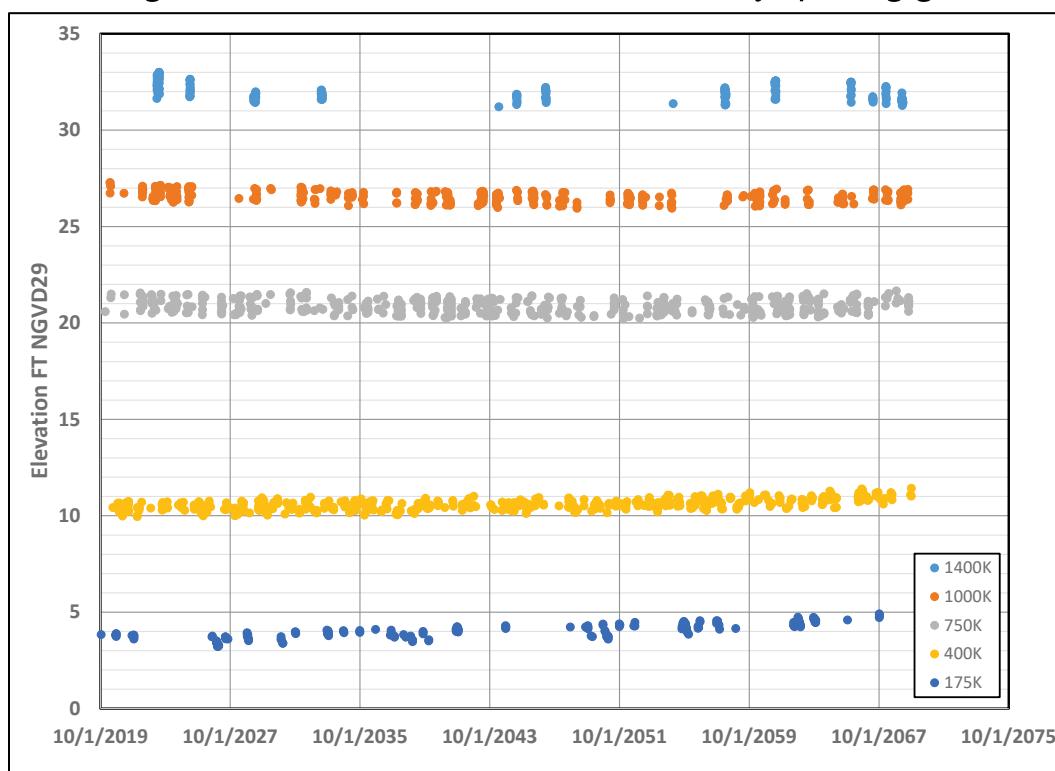
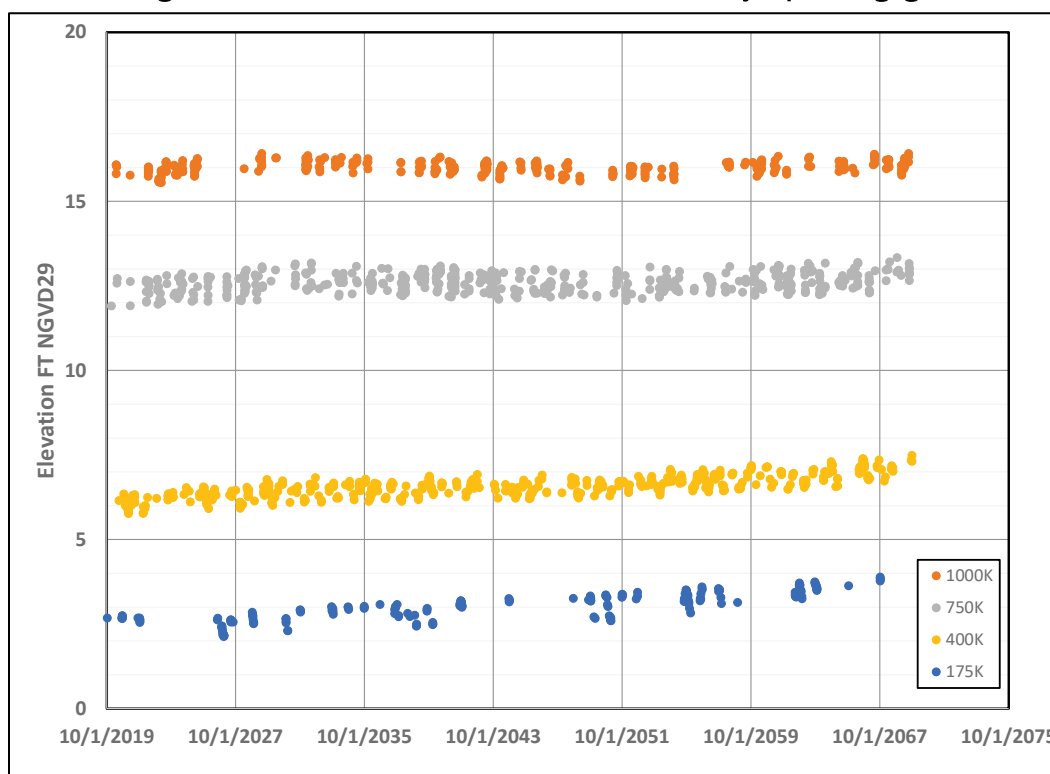


Figure 54. New Orleans RM 102.2 calculated 50 yr specific gage.

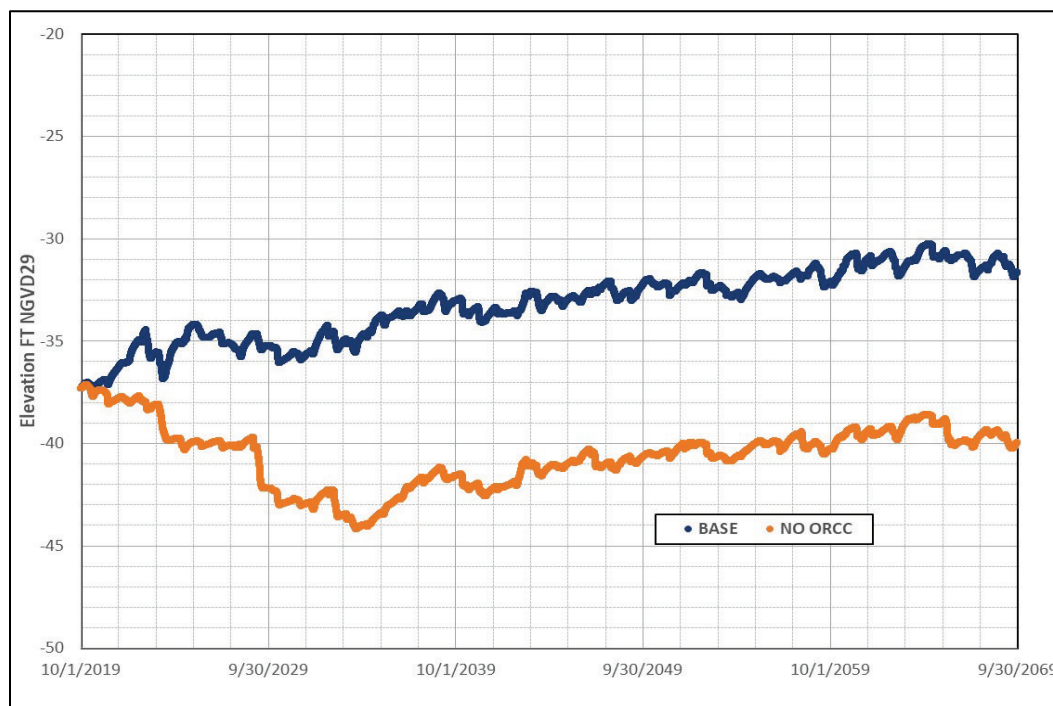


5.3 Analysis of increasing bed elevations at Red River Landing

The specific gage plots at Red River Landing (Figure 50) suggest a bed aggradation trend. One hypothesis is that the diversion of flow at ORCC is responsible for this aggradation trend. The numerical model was used to test this hypothesis by eliminating all diversions at ORCC, Morganza Spillway and Bonnet Carré Spillway, thereby allowing the total Mississippi River discharge at Union Point to move on downstream. This hypothetical test also assumed no levee overtopping. The calculated Base Condition and *No ORCC* bed profiles at Red River Landing for the next 50 yr are shown in Figure 55. With the Base (existing) Condition there is an aggradation trend. The oscillations in the bed profile reflect the bed responding to the annual hydrographs (i.e., short-term deposition/scour [as contrasted to long-term aggradation/degradation]). The *No ORCC* bed profile shows an initial degradation trend that lasts for approximately 15 yr. This degradation is caused by the increased Mississippi River discharge and the associated increase in bed shear stress and sediment transport capacity. It takes 15 yr for the riverbed to adjust to the new hydrologic regime. After that, an aggradation trend takes over and the subsequent bed profile is parallel to the existing condition aggradation trend. This result suggests

that the long-term aggradation trend is not driven by the diversions at ORCC as originally hypothesized.

Figure 55. Calculated 50 yr thalweg elevation at Red River Landing RM 302.1.



5.4 Scenario evaluations

The objective of the 1D model was to determine the long-term (50 yr) sedimentation effects of the various scenarios relative to the Base Condition. Each scenario's diversion percentages, as discussed in Section 4 and in the appendix, were applied to the daily discharges from the same hypothetical 50 yr hydrograph used in the Base Condition. Scenario evaluations included:

1. Comparing the resultant change in sand passage at various gages along the Mississippi River and in the Outflow Channel
2. Comparing changes in the volume of dredging at the deep draft crossings and in Southwest Pass and Head of Passes
3. Comparing water-surface profiles in the vicinity of the ORCC.

5.5 Differences in sediment passing gages

One way to evaluate the effect of changing the operation protocol at the ORCC is to compare the change in the sediment load that would occur

with the adoption of a different operation scenario. It is the sand sediment load that affects channel morphology and the sand loads were compared herein. At the end of 50 yr, the accumulated sand passing several gages for the Base Condition was compared to the 50 yr accumulated sand passing for each scenario (Table 38). The changes in sand load in tons are plotted as the difference between each scenario and the Base Condition in Figure 56 and as the percentage change in Figure 57 and Figure 58.

Two significant factors affect the sediment load downstream from the ORCC. One is the sediment transport potential that changes with changes in flow diversion percentages. This effect carries all the way to the Gulf of Mexico. The other is the sediment supply that changes with the flow diversion percentages. This is a point source/deficit and its effect is limited to reaches just downstream from the ORCC until sediment continuity requirements are satisfied. An increase in downstream discharge increases the sediment transport potential, so that the riverbed itself becomes a sediment source. This added sediment load is in addition to the increased sediment load that accompanies the increased discharge. This response is demonstrated in Figure 56, where sand load continues to increase in a downstream direction with Scenario 8A (D8020). In contrast, decreasing the discharge downstream from ORCC results in decreased sediment transport potential and deposition. This is demonstrated in Figure 56, where the sand load continues to decrease in a downstream direction with Scenario 9 (D6040).

Not surprisingly, the sediment load response in the Outflow Channel is opposite the response in the Mississippi River. In terms of the percentages differences, the Outflow Channel differences are much more significant and the Outflow Channel can be expected to respond much more dramatically than the Mississippi River. Changes in sand supply to the Outflow Channel vary between an increase of 52%, with D6040; to a decrease of 65%, with D8020.

Upstream from the ORCC there are also some differences in sediment load with the different scenarios. Most likely this is due to changes in water-surface elevations that accompany the differences in diversion percentages.

Table 38. 50 yr sand passing gages in million tons.

Scenario	Natchez	Union Point	Outflow Channel	Tarbert	Bayou Sara	Baton Rouge	Donaldsonville	New Orleans	Belle Chasse
HEC-6T XSECTION	363.01	226.6		306.3	265.1	228.1	175.4	102.2	77
BASE	2341	2157	308	1773	1579	1428	1388	1462	1490
3 - R1	2369	2203	440	1703	1525	1389	1347	1409	1428
4 - R1WH	2361	2191	423	1701	1518	1380	1340	1404	1424
5 - HF6949	2329	2142	315	1738	1524	1376	1322	1360	1379
6 - HF8020	2331	2141	239	1827	1643	1487	1455	1546	1586
7 - CTAR125	2326	2135	264	1790	1596	1447	1405	1491	1528
8A - D8020	2283	2078	108	1917	1834	1730	1715	1816	1886
8B - D8020WLSC	2311	2115	211	1834	1676	1533	1498	1591	1632
9 - D6040	2400	2234	469	1663	1355	1191	1099	1056	1036
10 - MAXAUX	2358	2184	381	1732	1558	1418	1383	1464	1494

Figure 56. 50 yr change in sand passing various gages with different scenarios.

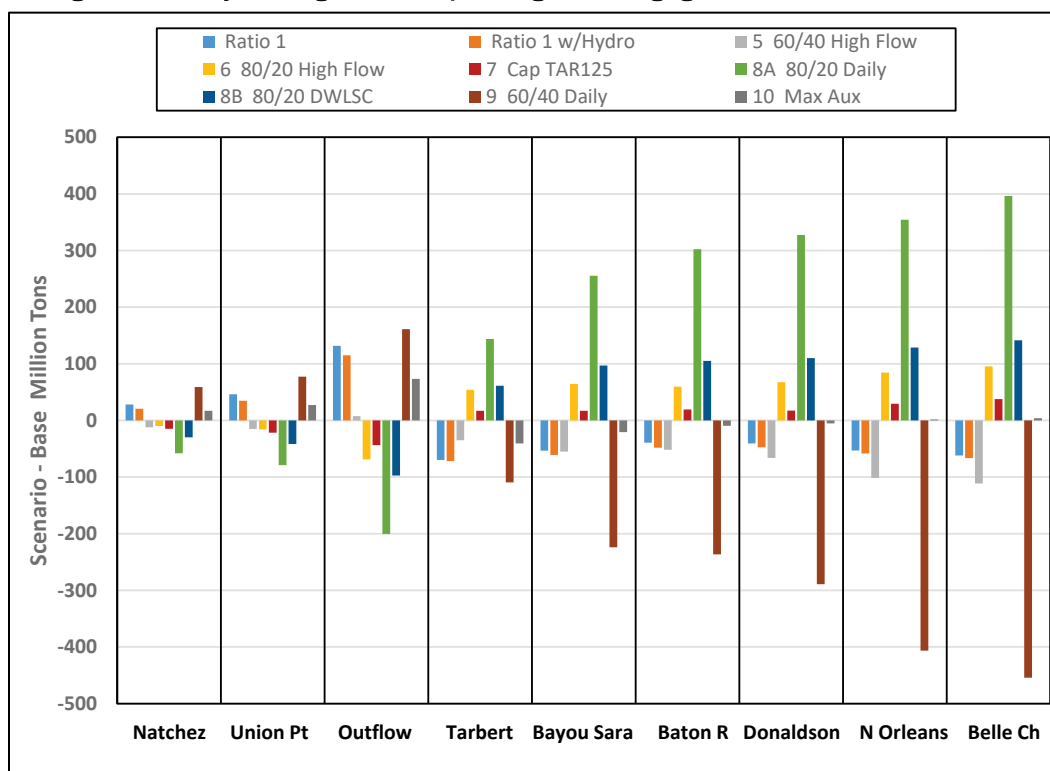


Figure 57. 50 yr change in percentage of sand passing gages with different scenarios.

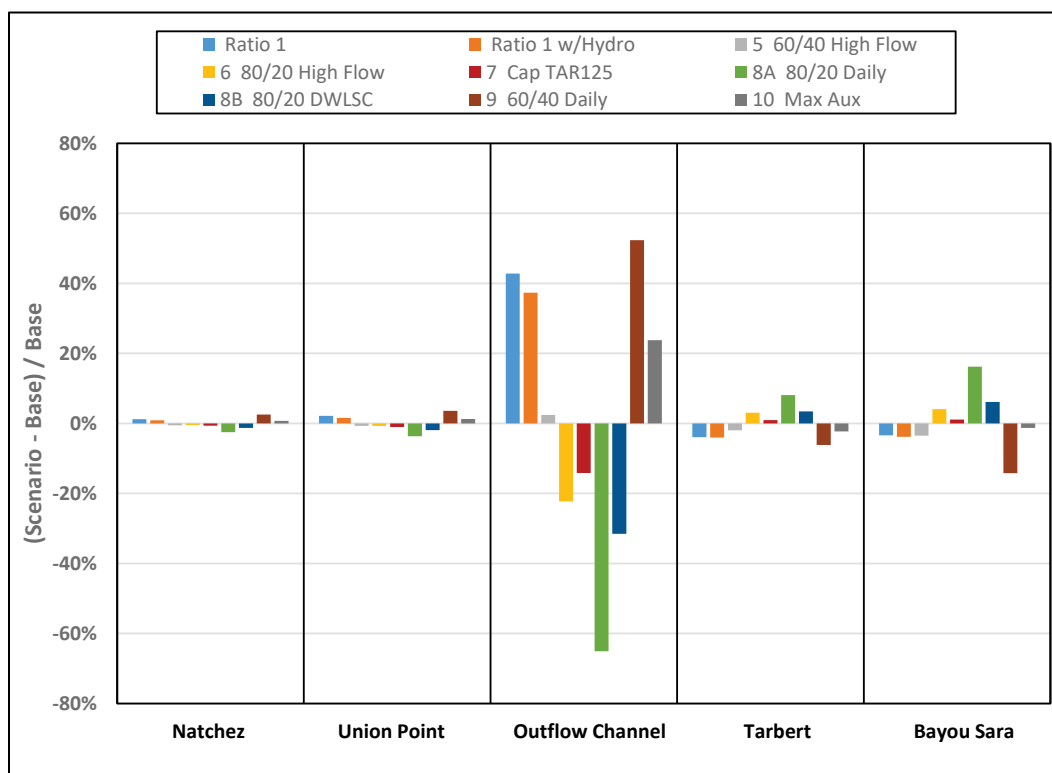
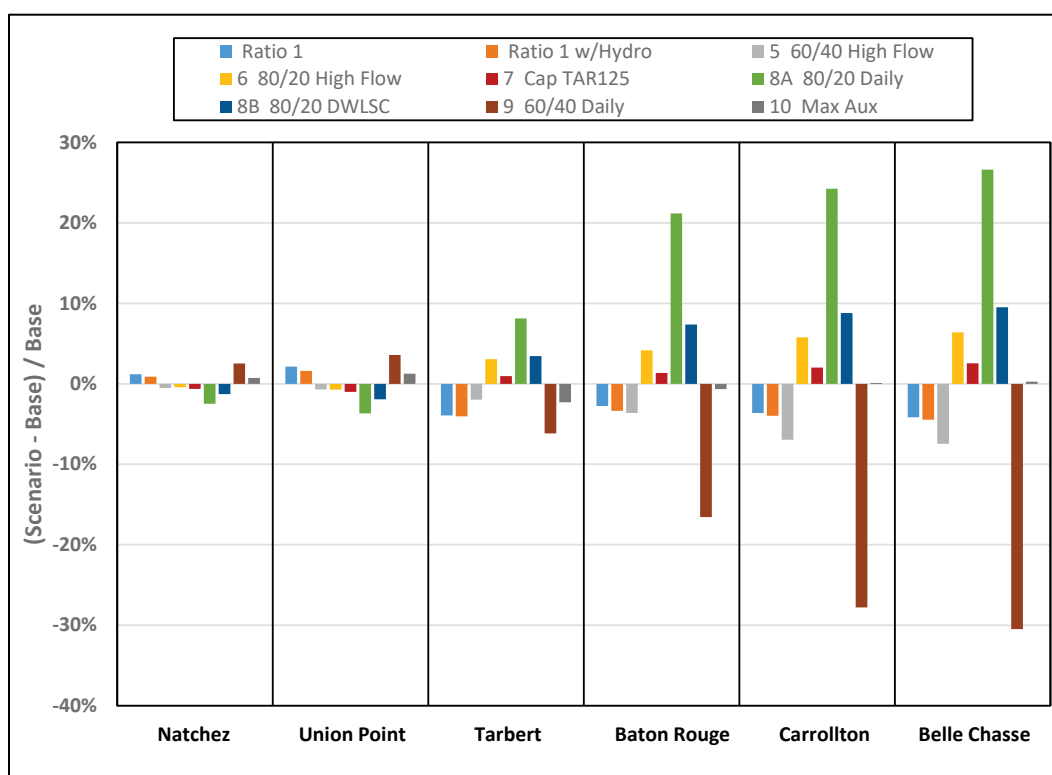


Figure 58. 50 yr change in percentage of sand passing Mississippi River gages with different scenarios.



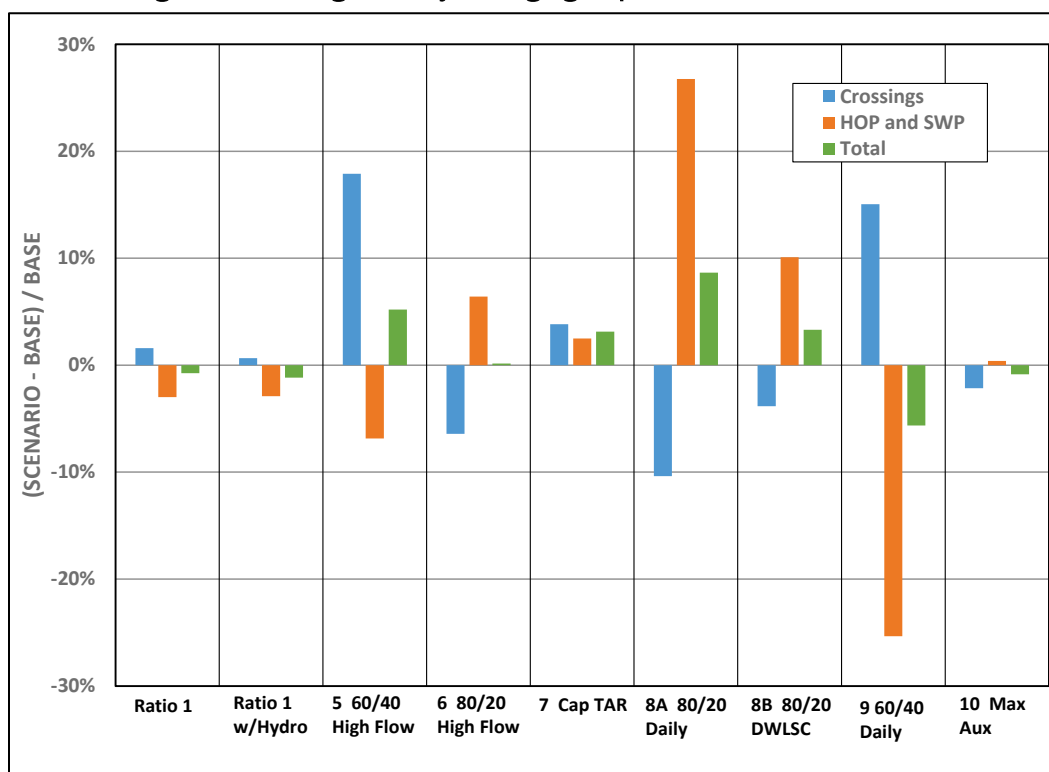
5.6 Differences in required dredging

Material dredged at crossings is returned to the river downstream of the dredging site and is modeled as such in HEC-6T. Material dredged above Head of Passes and in Southwest Pass is typically removed from the river. Agitation dredging occurs above Head of Passes and in Southwest Pass, but it was not included in the HEC-6T modeling or in comparisons between calculated and reported dredging volumes. 50 yr dredging for each scenario was calculated and compared to 50 yr dredging with the Base Condition in Figure 59.

The 60/40 scenarios produce less sediment and less discharge at high flows to the Mississippi River downstream from ORCC. At the crossings, the dominant factor is the decrease in discharge and bed shear stress. More dredging is required to maintain navigation project depths in the crossings. However, above Head of Passes and in Southwest Pass less dredging is required due to the decrease in sediment supply. The daily 60/40 split results in the most significant difference from existing conditions above Head of Passes and in Southwest Pass.

Conversely, the 80/20 scenarios provide more sediment and more discharge at high flow to the Mississippi River downstream from ORCC. At the crossings, the dominant factor is still the discharge, as the increase in bed shear stress reduces dredging requirements. Above Head of Passes and in Southwest Pass the dominant factor is the increase in sediment supply and more dredging is required with the 80/20 scenarios. The daily 80/20 split results in the most significant difference from existing conditions.

Figure 59. Change in 50 yr dredging requirements with scenarios.



5.7 Differences in water-surface elevations

The 50 yr differences in calculated water-surface elevations and bed elevations between the Base Condition and the various scenarios were determined with the numerical model. This was accomplished by inserting a range of discharges at the end of the 50 yr simulation. This methodology provides a means to evaluate relative differences in the various plans when compared to the Base Condition. Bed elevations and water-surface elevations vary not only with the instantaneous discharge but also with the antecedent discharges that determined the bed elevations. Consequently, this methodology does not provide a distinct quantitative value to predict how much of an increase or decrease a particular plan would cause on any specific day. HEC-6T calculates an average water-surface elevation and does not account for unsteady flow effects nor the loop effect associated with rising and falling limbs of flood hydrographs. Calculated water-surface profiles for the various scenarios are compared to the Base Condition in Figure 60 through Figure 68. Calculated bed and water-surface elevation changes at Union Point and Red River Landing are compared to the Base Condition in Table 39 and Table 40. Scenario water-surface elevation changes between initial conditions and conditions at the

end of the 50-yr simulation, for a Union Point discharge of 1,800,000 cfs, are shown in Figure 69 through Figure 71. These three figures demonstrate the effects of aggradation/degradation on water-surface elevations for each scenario.

The Ratio 1 (R1), and Ratio 1 with Hydropower (R1WH) scenarios retained the existing 70/30 split at ORCC. R1 has no flow diverted through the Hydropower Structure. Since the sediment diversion efficiency of the Hydropower Structure is much less than the Low Sill and Auxiliary Structures, more sediment is diverted into the Outflow Channel and Mississippi River sediment concentrations are decreased with R1. The R1WH scenario took R1 Low Sill assigned flow and allocated it to the Hydropower Structure up to a maximum of 150,000 cfs. This left the Auxiliary Structure, which has the highest diversion efficiency, to carry most of the diverted flow to the Outflow Channel. The Ratio 1 scenarios both resulted in reduced sediment concentrations in the Mississippi River when compared to the Base Condition. Consequently, degradation occurred downstream of the ORCC and water-surface elevations decreased at Union Point and Red River Landing after 50 yr compared to the Base Condition. Calculated water surface elevations were generally lower for the Ratio 1 scenarios at both Red River Landing and Union Point, with differences from the Base Condition varying between +0.2 and -1.1 ft. Scenario differences were minor compared to the long-term water-surface increases of between 2.5 and 3.0 ft.

Scenario 5 (HF6040) attempts to retain the annual 70% latitude flow at Tarbert Landing, while increasing the ORCC diversion percentages at high flow to achieve daily 60% latitude flow at Tarbert Landing. Compared to the Base Condition, this results in lower Mississippi River discharges at high flows due to the increased diversion percentages; and higher discharges at low flows, where discharge diversions are increased to balance the higher diversion percentages at high discharges. The overall effect is deposition downstream from the ORCC. At Red River Landing, the 50 yr HF6040 deposition is about 1.4 ft. higher than the Base Condition bed. HF6040 low flow water-surface elevations are about 1.0 ft higher than the Base Condition until the diversion percentages at ORCC are increased. At that point, HF6040 water-surface elevations begin to decrease to about 2.3 ft lower than the Base Condition. The same trend occurs at Union Point.

Scenario 6 (HF8020) attempts to retain the annual 70% latitude flow at Tarbert Landing, while decreasing the diversion percentages at high flow to achieve 80% daily latitude flow at Tarbert Landing. Compared to the Base Condition, this results in higher Mississippi River discharges at high flow due to the decreased diversion percentages; and lower Mississippi River discharges at low flows where discharge diversions are increased to balance the lower diversion percentages at high discharges. The overall effect is scour downstream from the ORCC. At Red River Landing, the 50 yr HF8020 bed is about 0.6 ft. lower than the Base Condition bed. HF8020 low flow water-surface elevations are about 0.9 ft lower than the Base Condition until the diversion percentages at ORCC are decreased. At that point, HF8020 water-surface elevations begin to increase to about 1.0 ft higher than the Base Condition. The same trend occurs at Union Point.

Scenario 7 (CTAR125) caps the flow at Tarbert Landing at 1,250,000 cfs to increase the flow diverted to the Atchafalaya River at high Mississippi River discharges. This scenario retains the annual 70% latitude discharge. Similar to HF6040, compared to the Base Condition, this results in lower Mississippi River discharges at high flows due to the increased diversion percentages; and higher discharges at low flows, where discharge diversions are increased to balance the higher diversion percentages at high discharges. The overall effect is deposition downstream from the ORCC. At Red River Landing, the 50 yr bed and water-surface comparisons to the Base Condition are similar to HF6040, however the magnitude of the differences are less.

Scenario 8A (D8020) increases the daily flow at Tarbert Landing by about 14%; from 70% to 80% of the latitude flow. At Red River Landing this results in a 50 yr increase in scour of about 3.6 ft compared to the Base Condition. The scour is attributed to the increase in sediment transport capacity that accompanies the increase in discharge and bed shear stress. At Union Point, where the discharge is unchanged, there was an increase in bed elevation of about 0.8 ft. Even with the decrease in bed elevation at Red River Landing, 50 yr water-surface elevations generally increased up to 1.1 ft due to the increases in discharge. Water-surface elevations increased up to 0.7 ft at Union Point.

Scenario 8B (D8020WLSC) also increases the daily flow at Tarbert Landing by about 14%, except when a structure safety constraint is exceeded. The flow diversion is restricted by the head differential at the

Low Sill Structure. Total annual diversions at the ORCC are less with D8020WLSC than with D8020. Consequently, the bed elevation and water-surface trends are similar to D8020, but the magnitudes are less. At Red River Landing, the 50 yr increase in scour was about 1.2 ft compared to the Base Condition. At Union Point, there was an increase in bed elevation of about 0.2 ft. Water-surfaces at Red River Landing increased up to 0.6 ft and at Union Point up to 0.4 ft compared to the Base Condition.

Scenario 9 (D6040) decreases the daily flow at Tarbert Landing by about 14%; from 70% to 60% of latitude flow. At Red River Landing this results in a 50 yr increase in deposition of about 4.1 ft compared to the Base Condition. The deposition is attributed to the decrease in sediment transport capacity that accompanies the decrease in discharge and bed shear stress. Even with the increase in bed elevation at Red River Landing, 50 yr water-surface elevations generally decreased up to 1.1 ft due to the decreases in discharge. At Union Point, where the discharge is unchanged, there was a decrease in bed elevation of about 1.2 ft and water-surface elevations decreased up to 1.1 ft. This effect at Union Point is attributed to the lower tailwater elevations downstream from ORCC.

Scenario 10 (MAXAUX) maximizes use of the Auxiliary Structure, which has a higher sediment diversion efficiency. This scenario diverts more sediment into the Outflow Channel without changing the Mississippi River discharges used for the Base Condition. The reduction in sediment supply produces scour downstream of ORCC. At Red River Landing, the 50 yr scour is about 0.6 ft compared to the Base Condition. 50 yr Red River Landing water-surface elevations generally decreased up to 0.7 ft due to the effects of downstream degradation. Union Point bed elevation and water-surface changes were consistent with the changes at Red River Landing.

Figure 60. Calculated 50 yr WSEL profiles Base and Ratio 1.

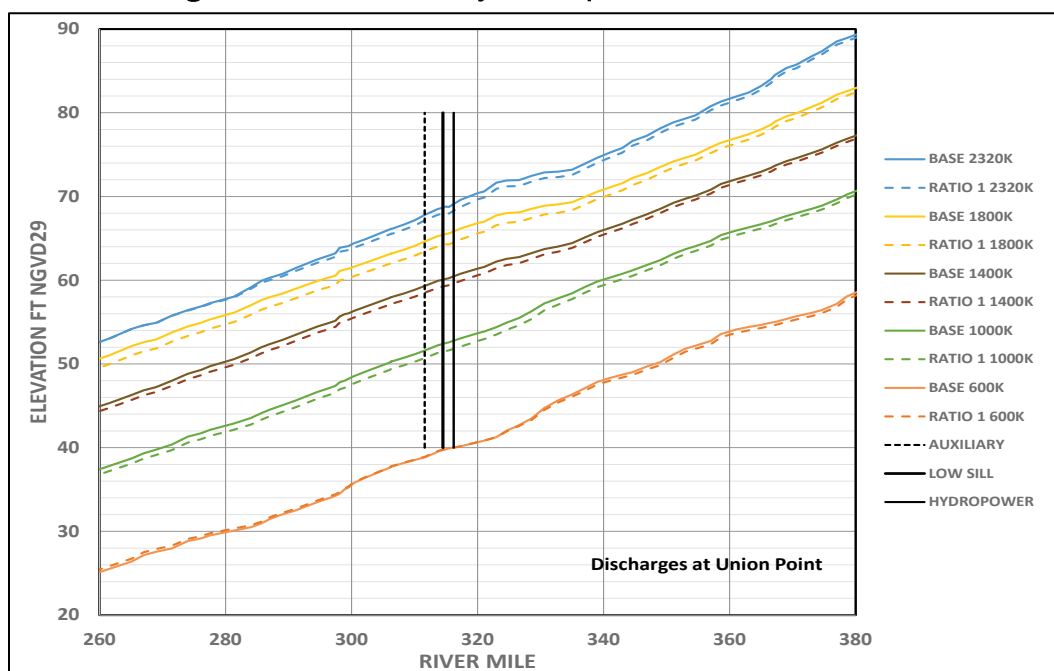


Figure 61. Calculated 50 yr WSEL profiles Base and Ratio 1 w/hydro (R1WH).

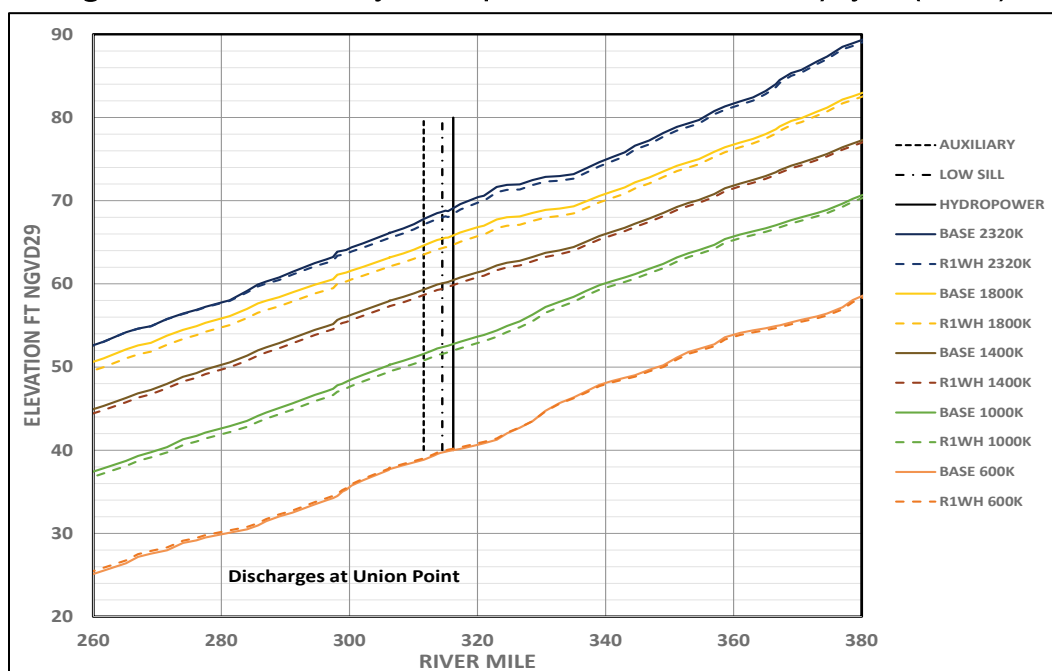


Figure 62. Calculated 50 yr WSEL profiles Base and Scenario 5 (HF6040).

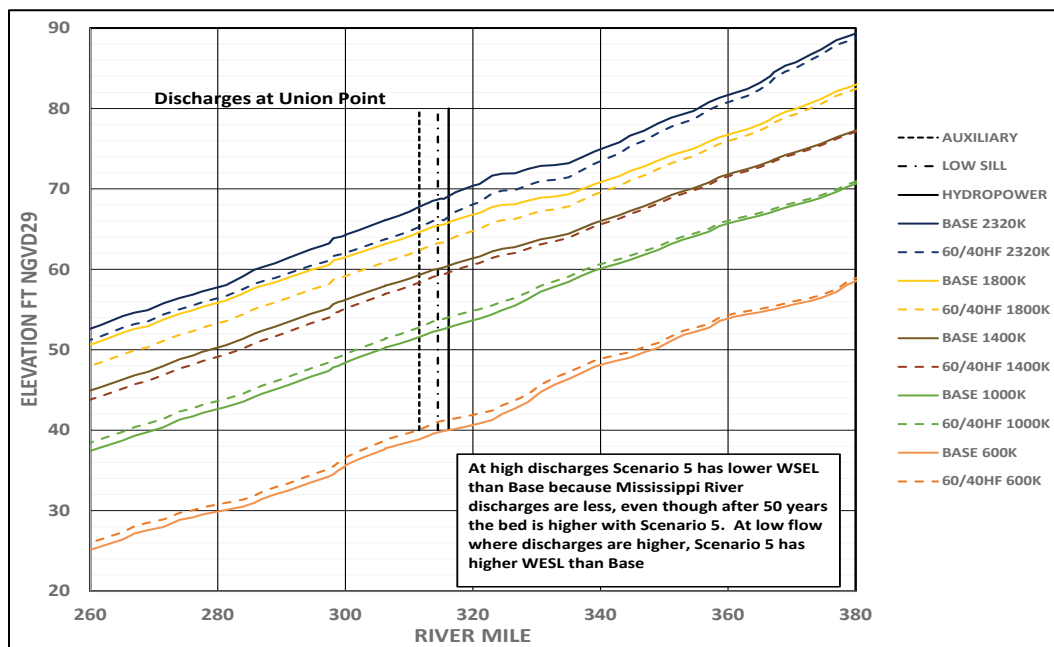


Figure 63. Calculated 50 yr WSEL profiles Base and Scenario 6 (HF8020).

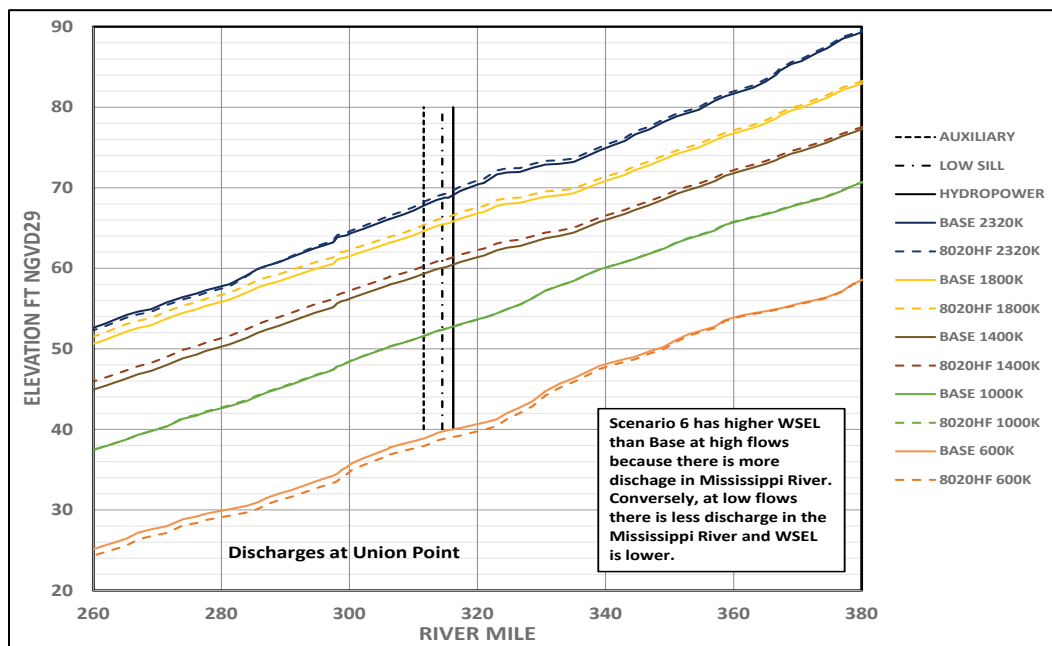


Figure 64. Calculated 50 yr WSEL profiles Base and Scenario 7 (CTAR125).

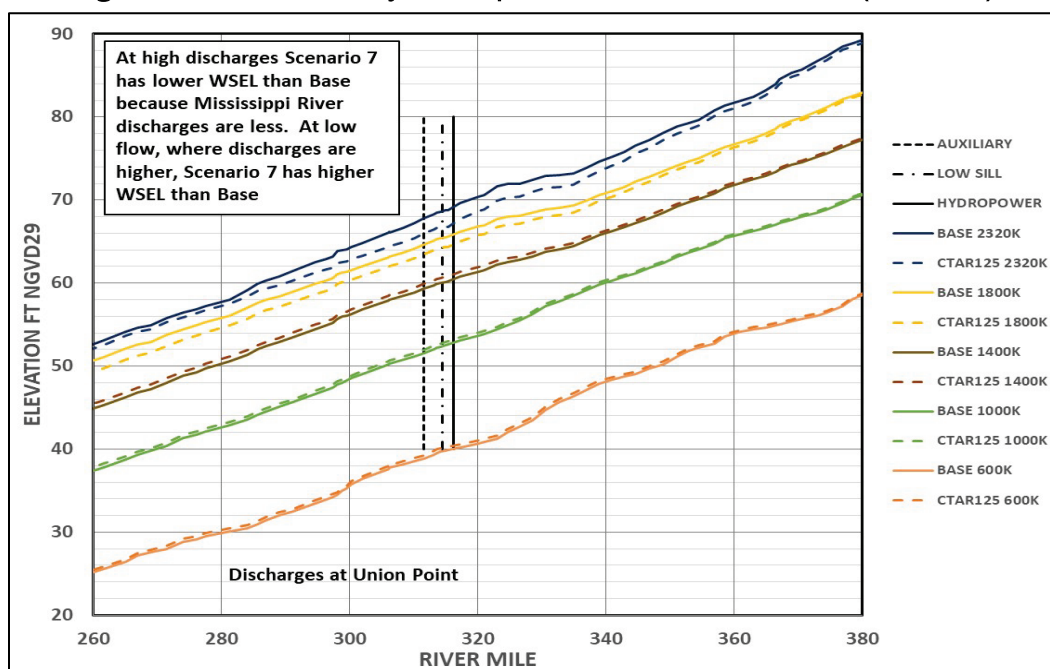


Figure 65. Calculated 50 yr WSEL profiles Base and Scenario 8A (D8020).

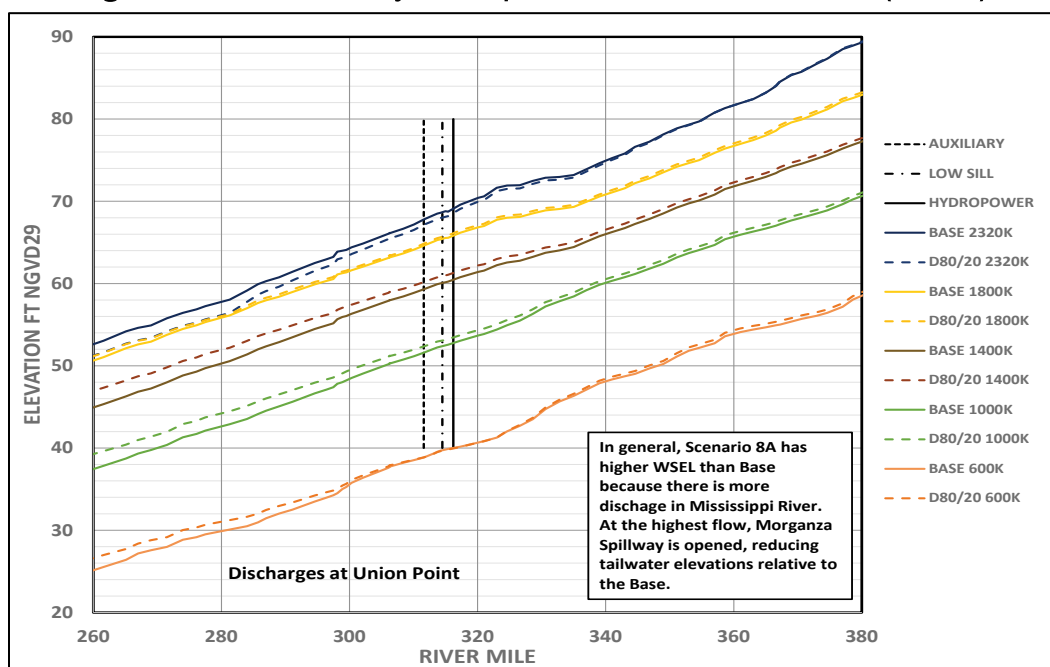


Figure 66. Calculated 50 yr WSEL profiles Base and Scenario 8B (D8020WLSC).

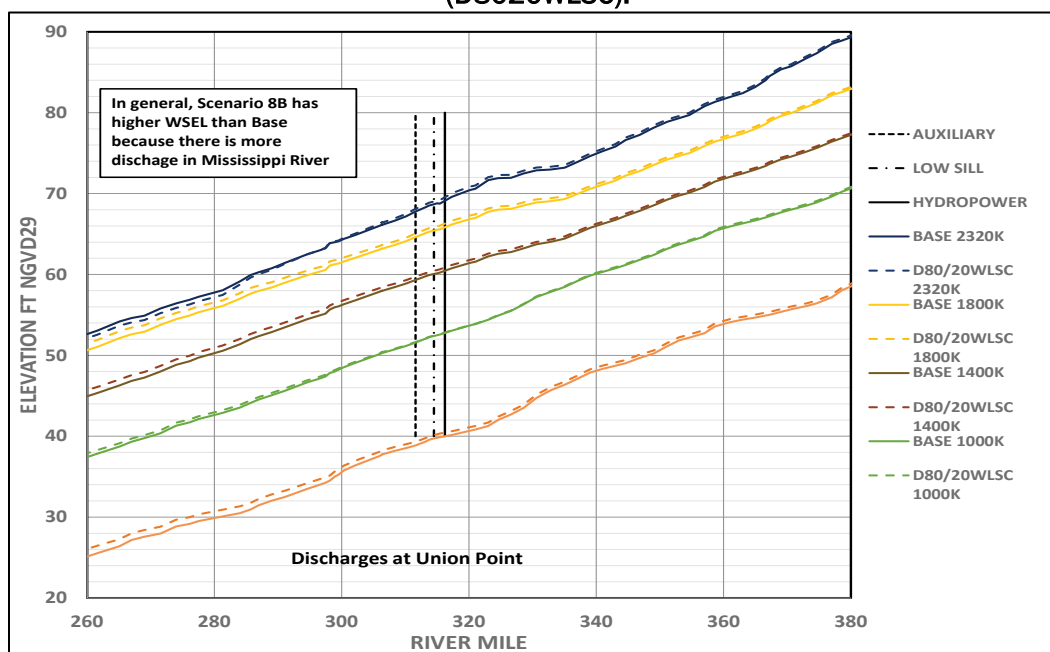


Figure 67. Calculated 50 yr WSEL profiles Base and Scenario 9 (D6040).

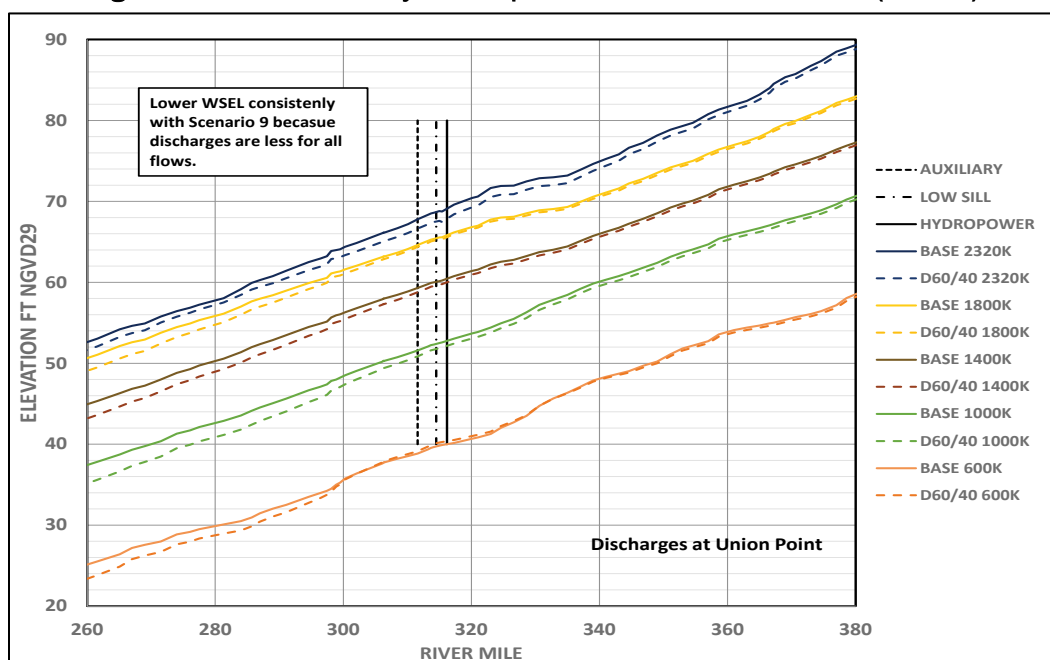


Figure 68. Calculated 50 yr WSEL profiles Base and Scenario 10 (MAXAUX)).

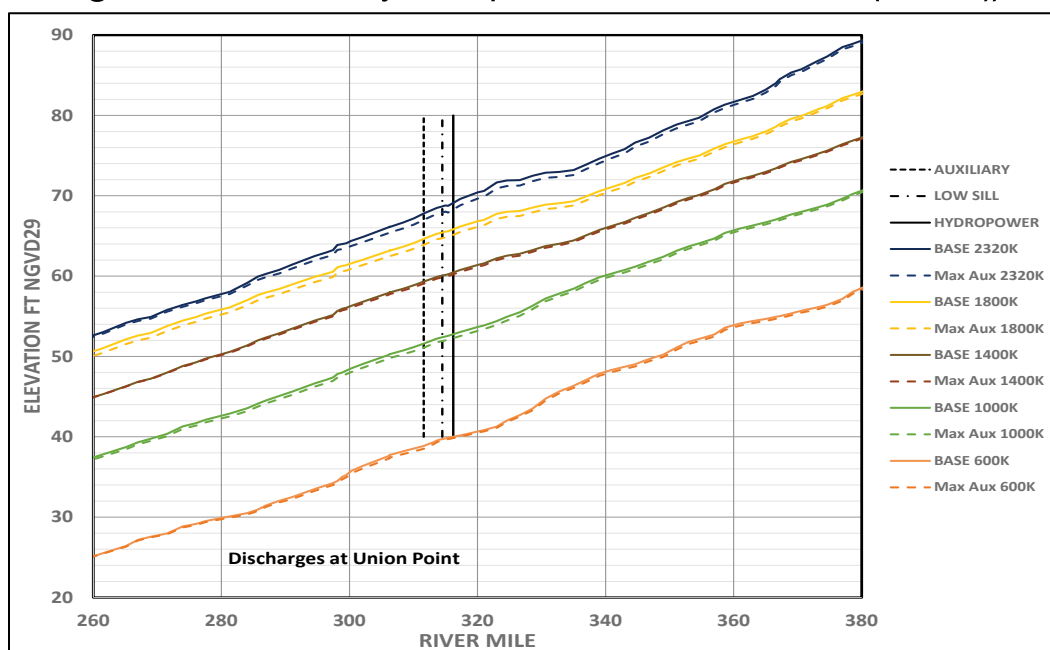


Table 39. Difference in bed and water-surface elevations (Scenario-Base) after 50 yr at Union Point RM 326.6.

Scenario		Bed Feet	Water-Surface Elevation - ft				
			600K	1000K	1400K	1800K	2320K
3	RATIO 1	-1.0	-0.1	-0.9	-0.7	-1.1	-0.7
4	R1WH	-0.8	0.0	-0.7	-0.6	-1.0	-0.6
5	HF6040	0.1	1.0	0.9	-0.7	-1.8	-2.1
6	HF8020	0.3	-0.7	0.0	0.8	0.7	0.5
7	CTAR125	0.4	0.4	0.3	0.5	-1.0	-1.6
8A	D8020	0.8	0.1	0.6	0.7	0.3	-0.4
8B	D8020WLSC	0.2	0.4	0.0	0.3	0.4	0.4
9	D6040	-1.2	0.2	-0.6	-0.4	-0.3	-1.1
10	MAXAUX	-0.5	-0.2	-0.4	-0.2	-0.6	-0.7

Table 40. Difference in bed and water-surface elevations (Scenario-Base) after 50 yr at Red River Landing RM 302.1.

Scenario		Bed feet	Water-Surface Elevation - ft				
			600K	1000K	1400K	1800K	2320K
3	RATIO 1	-0.4	0.1	-0.9	-0.8	-1.1	-0.5
4	R1WH	-0.2	0.2	-0.8	-0.6	-1.1	-0.5
5	HF6040	1.4	1.0	1.1	-1.1	-2.3	-2.3
6	HF8020	-0.6	-0.9	0.0	1.0	0.8	0.3
7	CTAR125	0.2	0.4	0.4	0.6	-1.2	-1.5
8A	D8020	-3.6	0.3	1.0	1.1	0.2	-0.8
8B	D8020WLSC	-1.2	0.6	0.1	0.5	0.5	0.2
9	D6040	4.1	0.0	-1.1	-0.8	-0.4	-1.0
10	MAXAUX	-0.6	-0.3	-0.5	-0.2	-0.7	-0.7

Figure 69. 50 yr difference in WSEL for Ratio 1 scenarios.

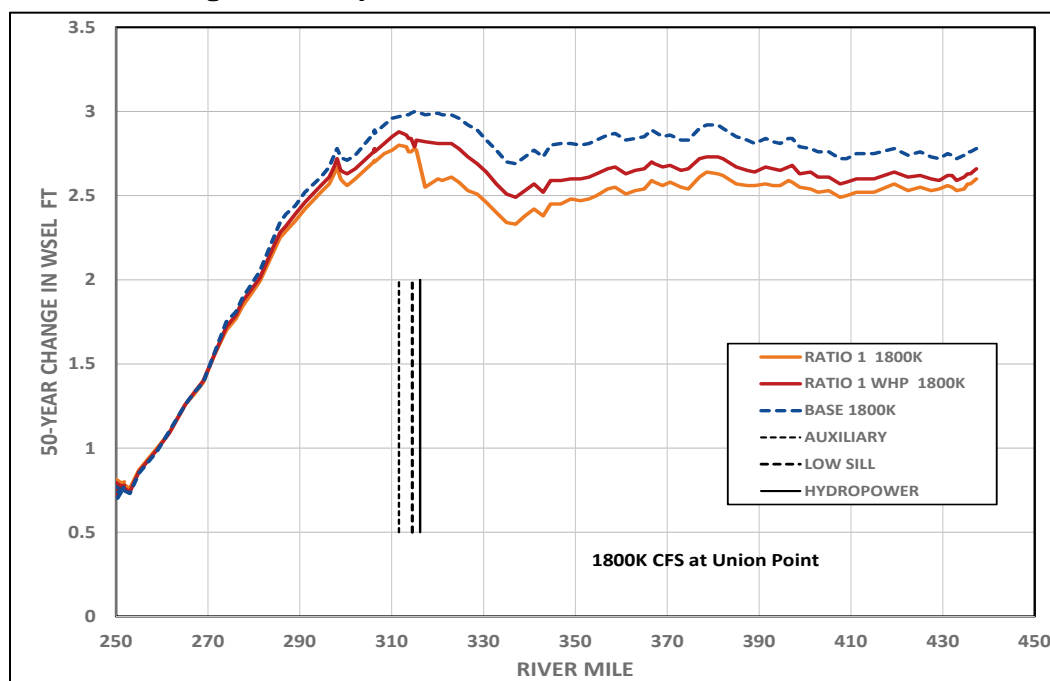


Figure 70. 50 yr difference in WSEL for High Flow and Cap TAR125 scenarios.

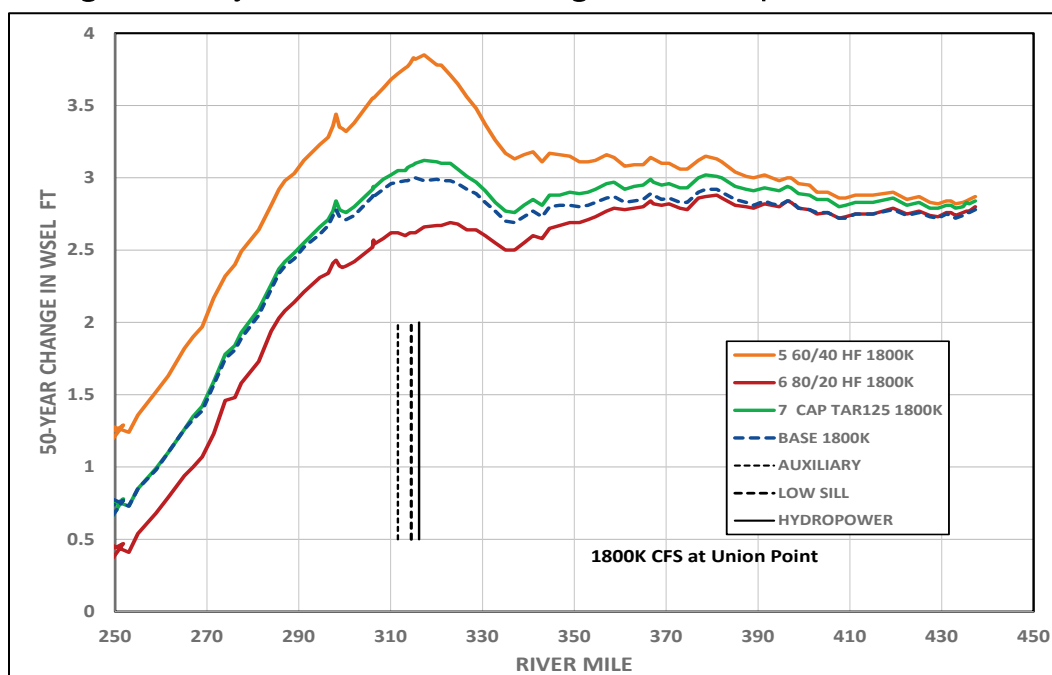
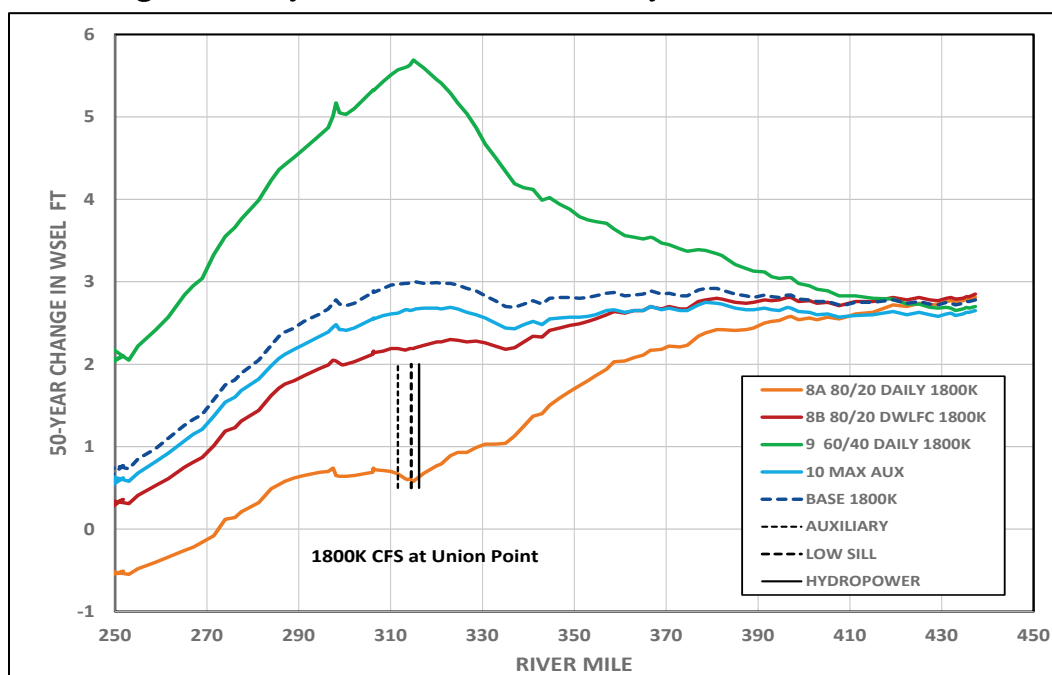


Figure 71. 50 yr difference in WSEL for Daily and MAXAUX scenarios.



5.8 Water-surface comparisons with equal discharge at Tarbert Landing

Water-surface elevations in the previous section were compared to events where the discharges at Union Point were consistent. Another way to compare the differences between the Base Condition and Scenario water-surface elevations is to compare water-surface profiles for cases where the discharges downstream from the ORCC are the same. In this case, the discharge will not be the same at Union Point for the Base Condition and Scenario and the comparison will not be for similar hydrologic events.

To make these comparisons, water-surface profiles were calculated at the end of the 50 yr future simulations for a range of discharges at Tarbert Landing. The five selected discharges were taken from the 50 yr hydrograph. The Base Condition and Scenario discharge for a specific Tarbert Landing discharge was not for the same day. Calculations were made for the most extreme scenarios; D6040 and D8020 and are shown in Figure 72 and Figure 73. Even though the water surface profiles are for different events, the bed elevations consistently represent conditions on 30 September 2069 and are shown in Figure 74.

After 50 yr of operation with Scenario D8020, the bed was significantly lower than the Base Condition due to the higher discharges. Consequently, with the same downstream discharge, D8020 water-surface elevations are lower than the Base Condition.

After 50 yr of operation with Scenario D6040, the bed has aggraded significantly compared to the Base Condition due to the lower discharges. Consequently, with the same downstream discharge, D6040 water-surface elevations are higher than the Base Condition.

Figure 72. 50 yr WSEL profiles Base and D8020 with same discharge at Tarbert.

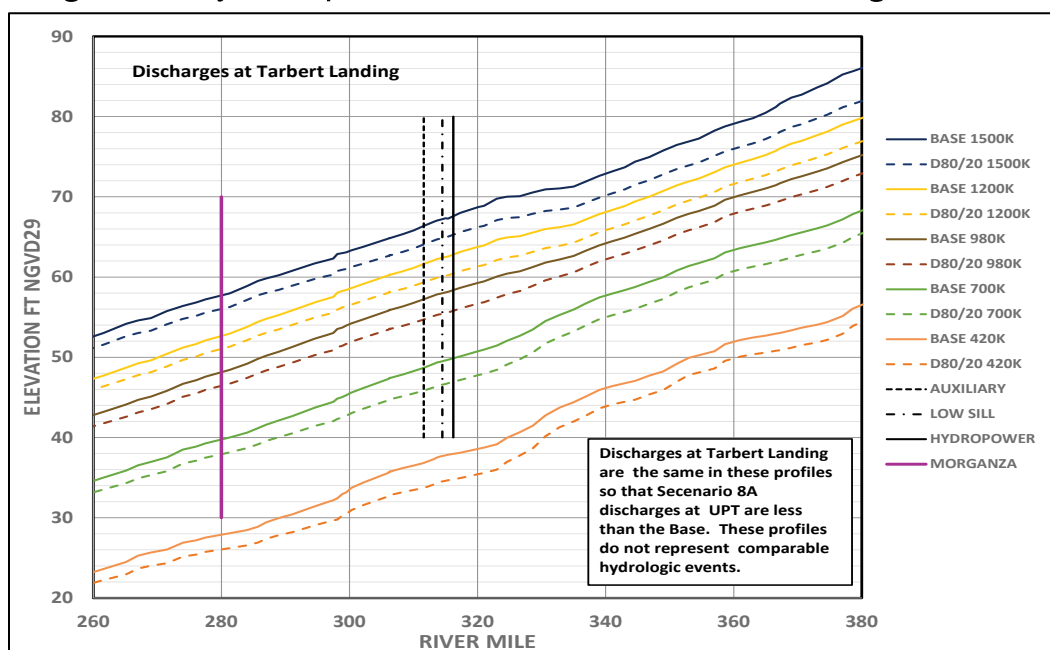


Figure 73. 50 yr WSEL profiles Base and D6040 with the same discharge at Tarbert.

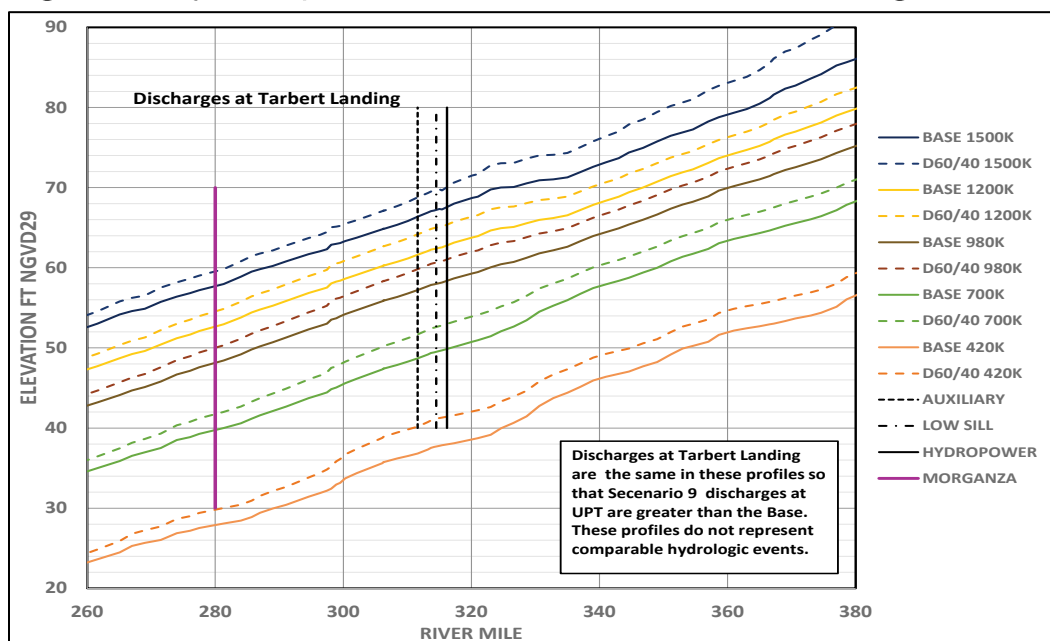
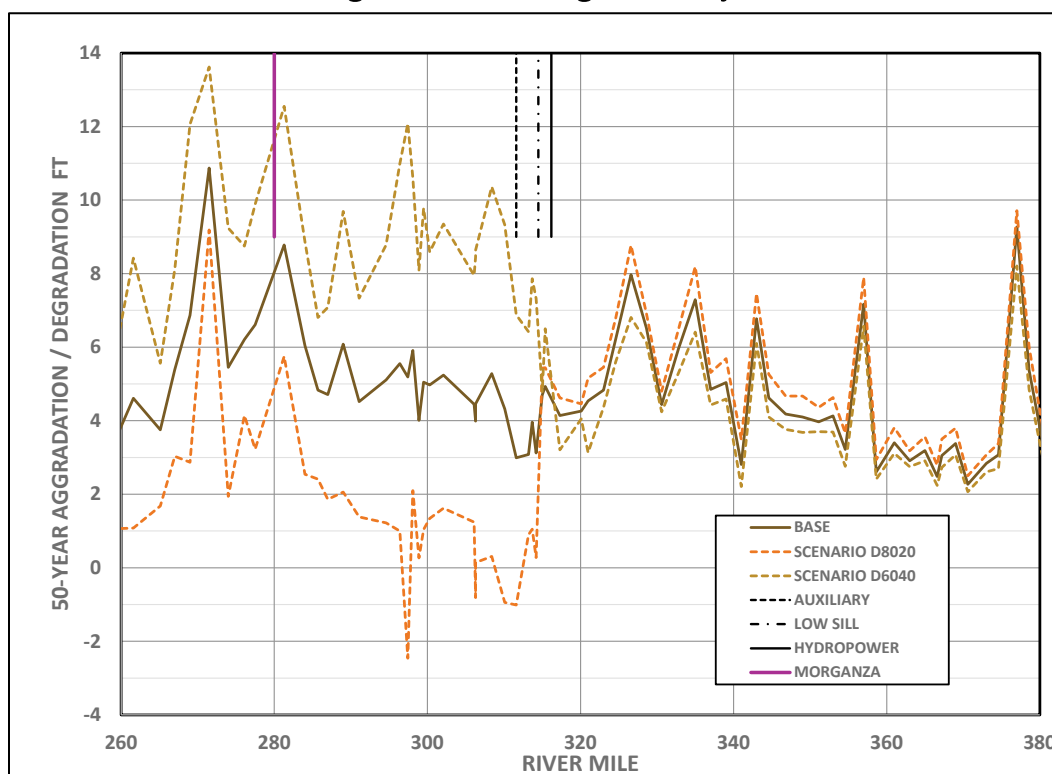


Figure 74. Bed change after 50 yr.



5.9 Dredging scenarios

The long-term effect of dredging the Mississippi River in the vicinity of ORCC was evaluated with the 1D model. The objective of the dredging was to balance the sand load differences between Ratio 1 and Base Condition operating conditions. The target volume was taken to be the calculated 50 yr average annual difference in ORCC total sand load between the Ratio 1 and Base Condition. This volume was 2.64 million tons per yr (2.10 million cubic yards/yr). This quantity is about six% of the sand load passing Tarbert Landing with the Base Condition. Bed material was dredged from the Mississippi River cross sections at RMs 314.2, 313.7, and 313.2. These cross sections are located between the Low Sill and the Auxiliary Structures and are in a crossing with relatively high thalweg elevations.

Three scenarios were evaluated.

1. Dredge 2.10 MCY annually at a daily rate of 50,000 yd³ per day starting January 1 and continuing for 42 days. The dredged material was removed from the Mississippi River. This is labeled the “Waste” scenario.

2. Dredge 2.10 MCY annually at a daily rate of 50,000 yd³ per day starting January 1 and continuing for 42 days. The dredged material was returned to the Mississippi River downstream from the dredged cross sections. This is labeled the “Agitation” scenario.
3. Dredge 6,000 yd³ every day for 350 days and remove the dredged material from the Mississippi River. This volume is equivalent to 2.10 MCY annually. This is labeled the “Daily Waste” scenario.

Calculated 50 yr sand passing several gages in million tons for the Base Condition, Ratio 1 and the three dredging scenarios are shown in Table 41.

Table 41. Calculated 50 yr sand passing gages in million tons for dredging scenarios.

Gage	River Mile	Base	Ratio 1	Agitation	Waste	Daily Waste
Natchez	363.0	2341	2369	2339	2376	2373
Union Point	226.6	2157	2203	2156	2214	2210
Outflow Channel		308	440	307	305	305
Tarbert	306.3	1773	1703	1775	1711	1710
Bayou Sara	265.1	1579	1525	1580	1552	1550
Baton Rouge	228.1	1428	1389	1428	1420	1417
Donaldsonville	175.4	1388	1347	1388	1382	1382
Carrollton	102.2	1462	1409	1462	1459	1464
Belle Chasse	77.0	1490	1428	1490	1488	1492

The Waste option, with a dredging capacity of 50,000 yd³ per day for 42 days, results in more sand transport upstream from ORCC – approximately a 2.6% increase at Union Point and a 1.5% increase at Natchez. Possible causes could be lower water-surface elevations and increased shear stresses that result from the lower bed elevations in the dredged crossing. This increases both bed load movement and turbulence, which keeps sediment in suspension longer. Sand passage downstream from ORCC is about 3.5% less at Tarbert, quickly decreasing to 1.7% less at Bayou Sara and 0.1% less at Belle Chasse. The wasted dredged material is replenished by the river bed downstream.

Agitation dredging removes sediment that has deposited on the bed at a given location and discharges it back into the river at a downstream location where turbulence in the water column keeps the sediment

suspended for a time as it moves downstream with the current. Agitation dredging also lowers the bed elevation at the dredging site, which can be important at crossings and sites with rapid deposition. In this test case, agitation dredging started on January 1 of each yr, at a dredging rate of 50,000 yd³ per day for 42 days. Agitation dredging downstream from the Low Sill Structure results in negligible differences in sand transport at Union Point and Natchez. Differences are also negligible at Tarbert Landing and farther downstream. This result suggests that the re-suspended sand is quickly re-deposited.

The Daily Waste scenario, with a dredging capacity of 6000 yd³ per day for 350 days, has an effect on sand transport, upstream and downstream from ORCC, that is similar to the 42-day 50,000 yd³ per day dredging scenario.

The percentage and volume increase in 50 yr average-annual dredging at the crossings and downstream from Venice for the ORCC dredging scenarios are shown in Table 42. These percentages and volumes are compared to those calculated for the Base Condition. The Daily Waste scenario is not shown due to required differences in assigned dredging commencement dates in the HEC-6T code that make comparisons incompatible. However, considering the similarity in the 42-day and 365-day waste dredging sediment passing calculations demonstrated in Table 41, it can be assumed that the long-term dredging is similar for both waste dredging scenarios. The table shows that Agitation dredging at ORCC has insignificant long-term effects on downstream dredging requirements. Less dredging is required at downstream dredging sites with the 42-day Waste scenario than the Agitation scenario – overall approximately 1% or 400,000 yd³/yr less dredging with the Waste scenario.

Table 42. Calculated increase in 50 yr dredging with dredging at ORCC.

Dredging Site	Agitation%	Waste Percentage	Agitation Thousand yds ³ /yr	Waste Thousand yds ³ /yr
SW Pass and HOP	0.32	-0.07	68.9	- 14.6
NO Harbor	0.36	-0.13	2.4	- 0.9
Belmont	0.10	-1.03	3.6	- 35.8
Smoke Bend	-0.09	-1.19	- 0.6	- 7.0
Philadelphia	0.83	-2.40	1.0	- 2.8
Alhambra	-0.04	-0.46	- 1.3	- 15.1
Bayou Goula	-0.11	-0.90	- 1.3	- 10.4
Granada	-0.05	-1.44	- 0.4	- 11.1
Medora	-0.02	-0.35	- 0.2	- 5.1
Sardine	0.02	-0.41	0.2	- 3.7
Red Eye	0.18	-2.92	9.1	- 152.1
Baton Rouge	0.30	-4.72	7.9	- 124.2
XINGS	0.10	-1.82	20.4	- 368.1
Total	0.21	-0.92	89.3	- 382.7

Water surface profiles were calculated for the Base Condition and the three dredging scenarios after a 50 yr simulation. These calculations reflect the calculated bed elevations that occurred after 50-yr in conjunction with the annual dredging specification of each of the three scenarios. The five water-surface profiles are for steady-state discharges at Union Point. Calculated water-surface profiles, in the vicinity of the ORCC, are shown in Figure 75 through Figure 77. Annual waste dredging provides lower water-surface elevations in the vicinity of ORCC when compared to the Base Condition. Agitation dredging provides no significant change in water surface elevations. Shown in Figure 78. are the 50 yr differences in water-surface elevation for a Union Point discharge of 1,800,000 cfs for each of the dredging scenarios and the Base Condition. These plots indicate that after 50 yr, water-surface elevations for a discharge of 1,800,000 cfs would be between 0.6 ft and 0.3 ft lower between ORCC and Vicksburg for the waste dredging scenarios. The two waste options have essentially the same effects with respect to water-surface elevations. The agitation dredging scenario would not affect water-surface elevations. The

effects of waste dredging extend downstream because the removal of bed material results in a reduction in the sediment supply and degradation occurs to meet the sediment transport capacity, which has not been altered. Also noted is that there is a long-term aggradation trend active in this reach, so that after 50 yr water-surface elevations are increased for the Base Condition and all dredging scenarios.

Calculated bed change profiles after the 50 yr simulation are shown in Figure 79.. The Agitation dredging scenario is essentially the same as the Base Condition. The Daily Waste and 50K Waste scenarios are essentially the same. The Waste scenarios induce approximately a 1 ft decrease in bed elevation at the dredging site, which continues downstream. There is also a reduction of approximately 0.5 ft in bed elevations upstream from the dredging site, relative to the Base Condition, which decrease in magnitude to essentially zero at Natchez. The dredging site itself has only an approximate one foot bed change after 50 yr because the dredged material continues to be re-supplied from upstream.

Figure 75. Calculated 50 yr water-surface profiles with Waste dredging at ORCC.

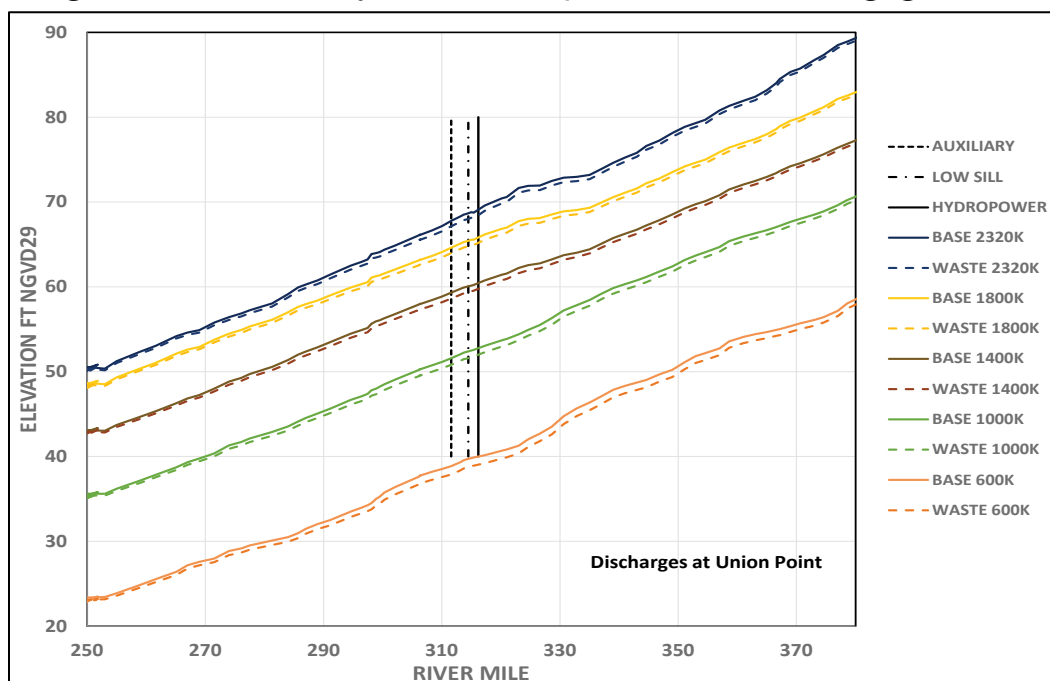


Figure 76. Calculated 50 yr water-surface profiles with Daily Waste dredging at ORCC.

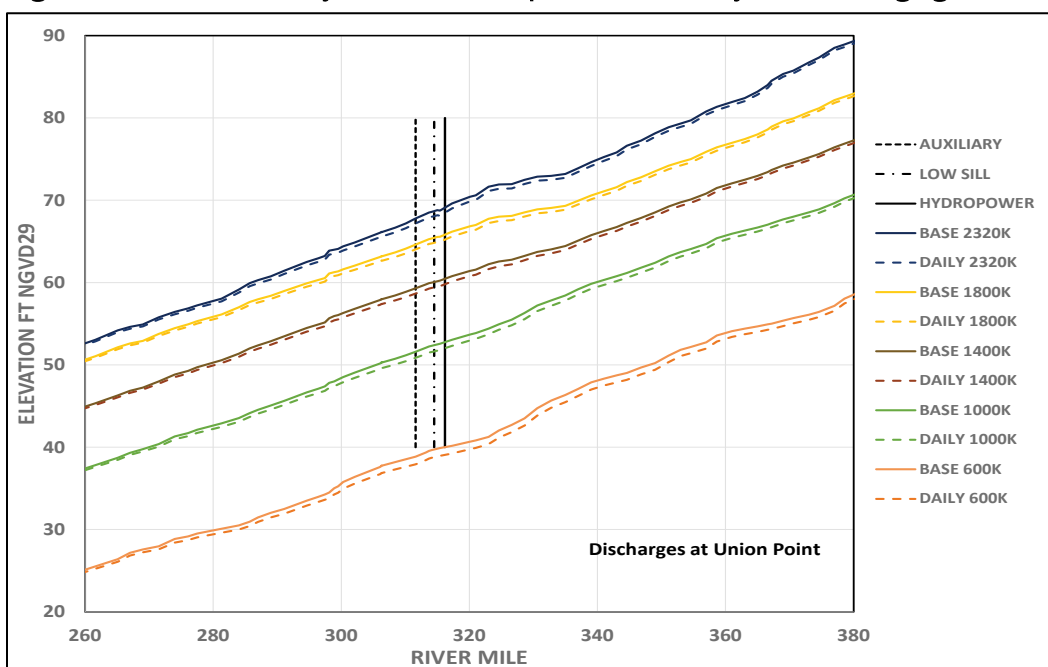


Figure 77. Calculated 50 yr water-surface profiles with Agitation dredging at ORCC.

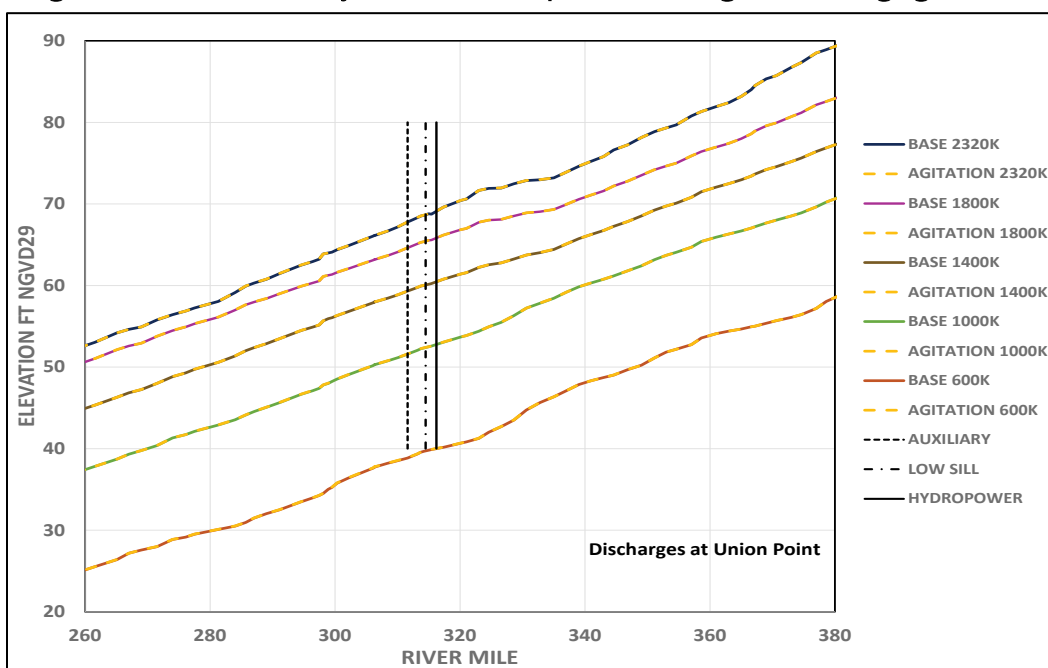


Figure 78. Calculated 50 yr water-surface change with Dredging Scenarios.

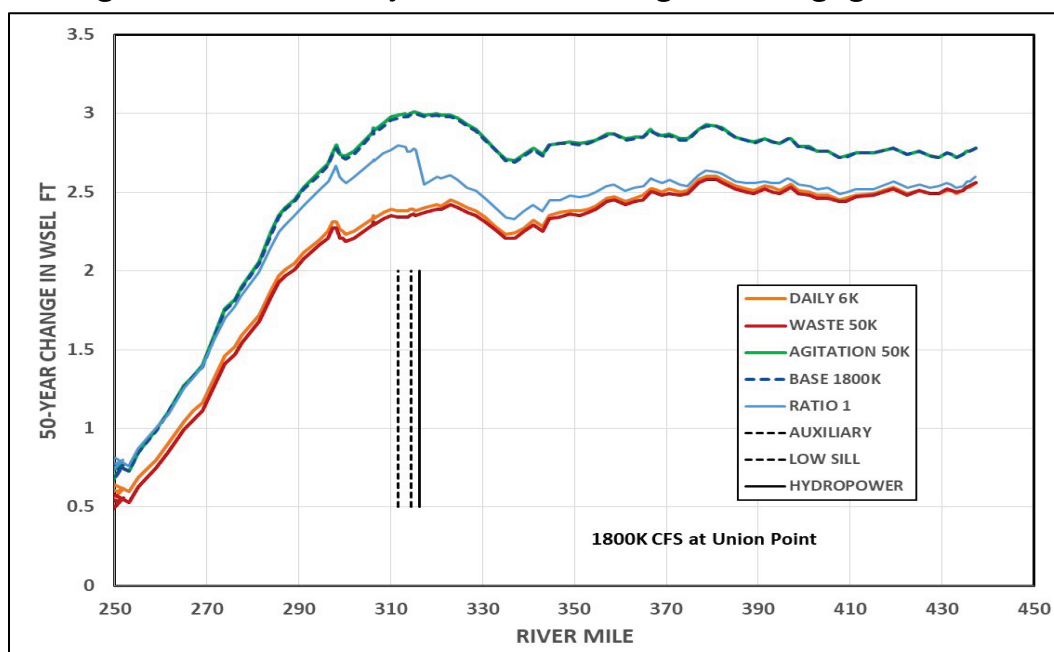
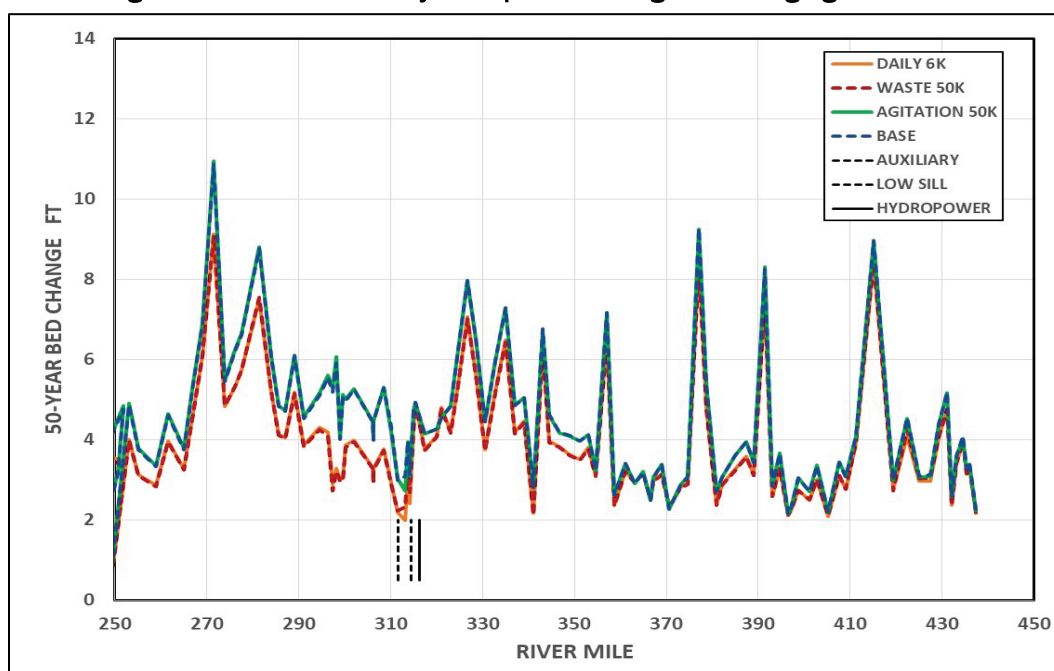


Figure 79. Calculated 50 yr bed profile change for dredging scenarios.



6 Conclusions and Recommendations

6.1 Conclusions

The numerical model study reported herein evaluated the long-term sedimentation effects of modifying the operation schedule at ORCC. Sedimentation effects for each operation scenario was compared to a Base Condition that represented the existing operation protocols. The predictive simulations extended for 50 yr, using the same historical hydrograph that occurred from October 1969 to September 2019. The model was used to calculate and compare sand transport past various gages along the Mississippi River and in the Outflow Channel. Dredging volumes at the deep-draft crossings and in Southwest Pass and above Head of Passes were compared. Differences in water-surface profiles in the vicinity of ORCC were determined.

The numerical model used in this study accounts for both long-term and system-wide effects of alternative operation protocols at the ORCC. River response to modifications can last for decades and affect river reaches hundreds of miles away. Specific gage analyses demonstrated both historical and predicted long-term changes in the river system. These changes will affect sedimentation trends in the vicinity of ORCC. There is a long-term aggradation trend at both Vicksburg and Red River Landing, which suggests an increase in sediment supply to the ORCC reach. Model tests demonstrated that after an initial adjustment period the long-term aggradation trend would continue even if all flow diversions through the ORCC, Morganza Spillway and Bonnet Carré Spillway were eliminated.

Downstream from Bayou Sara (RM 265), the future specific gage analyses suggest no significant increases in water-surface elevations with existing conditions for the next 50 yr.

One way to evaluate the effect of changing the operation protocol at the ORCC is to compare the change in the sediment load that would occur with the adoption of a different operation scenario. The sand size classes are the significant portion of the sediment load that affects channel morphology and the 50 yr accumulated sand loads were compared in this report. Three scenarios (HF8020, D8020WLSC and D8020) increased high flow discharges down the Mississippi River. Compared to the Base Condition, sand load continued to increase in a downstream direction,

accompanied by channel degradation, with these scenarios. In contrast, decreasing the high flow discharge downstream from ORCC resulted in decreased sediment transport potential and **increased deposition**. This was demonstrated by HF6040, D6040 and CTAR125, where sand load continues to decrease in a downstream direction.

Not surprisingly, the sediment load response in the Outflow Channel is opposite the response in the Mississippi River. In terms of the percentages differences, the Outflow Channel differences are much more significant and the Outflow Channel can be expected to respond much more dramatically than the Mississippi River. Changes in sand supply to the Outflow Channel vary between a 52% increase with D6040, to a 65% decrease with D8020.

The changes in sand transport will affect the dredging requirements to maintain the Mississippi River navigation channel downstream from ORCC. Material dredged at the deep-draft crossings is returned to the river downstream of the dredging site and is modeled as such in HEC-6T. Material dredged above Head of Passes and in Southwest Pass is typically removed from the river. Agitation dredging occurs above Head of Passes and in Southwest Pass, but it was not included in the numerical modeling or in comparisons between calculated and reported dredging volumes. 50 yr accumulated dredging for each scenario was calculated and compared to 50 yr accumulated dredging with the Base Condition.

The 60/40 scenarios provide less sediment and fewer high discharges to the Mississippi River downstream from ORCC. **At the crossings, the dominant factor is the decrease in discharge and bed shear stress**. More dredging is required to maintain navigation project depths in the crossings. Approximately 18% more dredging was required at the crossings with HF6040, and approximately 15% more with D6040. However, above Head of Passes and in Southwest Pass less dredging is required due to the decrease in sediment supply. D6040 required approximately 25% less dredging and HF6040 approximately 7% less dredging.

Conversely, the 80/20 scenarios provide more sediment and more high discharges to the Mississippi River downstream from ORCC. **At the crossings, the dominant factor is still the discharge, as the increase in bed shear stress reduces dredging requirements**

between 4% and 10%. Above Head of Passes and in Southwest Pass the dominant factor is the increase in sediment supply and more dredging is required with the 80/20 scenarios: 6% with HF8020, 10% with D8020WLSC, and 27% with D8020.

The 50 yr change in water-surface elevation and bed elevation affected by the various scenarios was calculated with the numerical model. This was accomplished by inserting a range of discharges at the end of the 50-yr simulation.

The Ratio 1 plans and Scenario 10 (MAXAUX) retained the existing 70/30 split at ORCC and resulted in a decrease in bed elevation at Union Point and Red River Landing after 50 yr compared to the Base Condition. **This is attributed to the more efficient diversion of sediment through ORCC with these scenarios.** The most scour occurred at Union Point with Ratio 1, and was about 1.0 ft. Calculated water surface elevations after 50 yr were generally less than the Base Condition at both Union Point and Red River Landing, varying between +0.2 and -1.1 ft.

The plans that decreased high discharges in the Mississippi River were Scenario 5 (HF6040), Scenario 9 (D6040) and Scenario 7 (CTAR125). **Since diverted sand concentrations are less than the upstream sand concentrations in the Mississippi River, these scenarios produce higher sand concentrations downstream in the Mississippi River with less discharge and bed shear stress to move it.** Consequently, at Red River Landing, bed elevations after 50 yr increased for all three scenarios, with the most significant being 4.1 ft with D6040. Scenarios 5 and 7 attempt to retain the annual 70% latitude flow at Tarbert Landing, while increasing the ORCC diversion percentages at high flow. Compared to the Base Condition, this results in lower Mississippi River discharges at high flows due to the increased diversion percentages; and higher discharges at low flows, where discharge diversions are increased to balance the higher diversion percentages at high discharges. The overall effect is deposition downstream from the ORCC. At Red River Landing, the 50 yr HF6040 deposition is about 1.4 ft. higher than the Base Condition bed. The CTAR125 deposition is approximately 0.2 ft higher than the Base Condition bed. HF6040 low flow water-surface elevations are approximately 1.0 ft higher than the Base Condition until the diversion percentages at ORCC are increased. At that point, HF6040 water-surface elevations begin to decrease to about 2.3 ft

lower than the Base Condition. The same trend occurs with CTAR125, with less magnitude. Trends are the same at Union Point. D6040 maintains a lower Mississippi river discharge for the entire yr. Water-surface elevations were generally lower for D6040 at both Union Point and Red River Landing. **This is the result of a decrease in discharge and tailwater elevations in the Mississippi River downstream.**

The plans that increased the high discharges and sediment concentrations in the Mississippi River were Scenario 6 (HF8020), Scenario 8A (D8020) and Scenario 8B (D8020WLSC). **These scenarios all resulted in increased deposition at Union Point and increased scour at Red River Landing.** The most significant decrease in bed elevation at Red River Landing was 3.6 ft and occurred with the D8020 plan. In most cases, the 80/20 scenario water-surface elevations at Red River Landing and Union Point were higher than the Base Condition. **The backwater created by increased discharges downstream were more influential than the downstream scour.** The exceptions were for HF8020 at 600,000 cfs where more flow was being diverted as payback to achieve the annual 70/30 split; and for D8020 at 2,320,000cfs where the downstream scour and the influence of the Morganza Spillway resulted in reduced water surface elevations at both Union Point and Red River Landing.

The long-term effect of dredging the Mississippi River in the vicinity of ORCC was evaluated with the 1D model. The objective of the dredging was to balance the sand load differences between Ratio 1 and Base Condition operating conditions. The target volume was taken to be the calculated 50 yr average annual difference in ORCC sand load between the Ratio 1 and Base Condition. This volume was 2.64M tons/yr (2.10 MCY). This quantity is approximately 6% of the sand load passing Tarbert Landing with the Base Condition. Bed material was dredged from the Mississippi River cross sections at RMs 314.2, 313.7, and 313.2. These cross sections are located between the Low Sill and the Auxiliary Structures and are in a crossing with relatively high thalweg elevations.

Three scenarios were evaluated.

1. Dredge 2.10MCY annually at a daily rate of 50,000 yd³ per day starting January 1 and remove the dredged material from the Mississippi River. This is labeled the “Waste” scenario.

2. Dredge 2.10 MCY annually at a daily rate of 50,000 cubic yards per day starting January 1 and return the dredged material to the Mississippi River downstream from the dredging site. This is labeled the “Agitation” scenario.
3. Dredge 6000 yd³ every day for 350 days and remove the dredged material from the Mississippi River. This volume is equivalent to 2.10 MCY annually. This is labeled the “Daily Waste” scenario.

The Waste option, with a dredging capacity of 50,000 yd³ per day for 42 days, results in more sand transport upstream from ORCC — approximately a 2.6% increase at Union Point and a 1.5% increase at Natchez. Possible causes could be lower water-surface elevations and increased bed shear stresses that result from the lower bed elevations in the dredged crossing. This increases both bed load movement and turbulence, which keeps sediment in suspension longer. Sand passage downstream from ORCC is approximately 3.5% less at Tarbert, quickly decreasing to 1.7% less at Bayou Sara and 0.1% less at Belle Chasse. The wasted dredged material is replenished by the bed downstream.

Agitation dredging removes sediment that has deposited on the bed at a given location and discharges it back into the river at a downstream location where turbulence in the water column keeps the sediment suspended for a time as it moves downstream with the current. Agitation dredging also lowers the bed elevation at the dredging site, which can be important at crossings and sites with rapid deposition. In this test case, agitation dredging started on January 1 of each year, at a dredging rate of 50,000 cubic yard per day for 42 days. Agitation dredging downstream from the Low Sill Structure results in negligible differences in sand transport at Union Point and Natchez. Differences are also negligible at Tarbert Landing and farther downstream. This result suggests that the re-suspended sand is quickly re-deposited.

The Daily Waste scenario, with a dredging capacity of 6000 yd³/day for 350 days, has an effect on sand transport, upstream and downstream from ORCC, that is similar to the 42 day 50,000 yd³/day dredging scenario.

Agitation dredging at ORCC has insignificant long-term effects on downstream dredging requirements. The re-suspended bed material is quickly redeposited downstream. Less dredging is required at downstream dredging sites with the 42-day Waste scenario than with the Agitation

scenario. The total reduction in dredging, including both the crossings and downstream from Venice was about 1% or 400,000 yd³/yr with the Waste scenario.

Water surface profiles were calculated for the Base Condition and the three dredging scenarios after a 50 yr simulation. These calculations reflect the calculated bed elevations that occurred after 50 yr, in conjunction with the annual dredging specification of each of the three scenarios. Annual waste dredging provides lower water-surface elevations in the vicinity of ORCC when compared to the Base Condition. Agitation dredging provides no significant change in water surface elevations.

6.2 Recommendations

The MRT&T Project is critical to nation's economy and the USACE mission. The project provides essential flood damage reduction, navigation, and environmental needs. Proper operation, maintenance, and improvement of the Project requires continuous comprehensive engineering analyses to ensure beneficial river functions by reliably predicting river responses to both past and future engineering and geomorphic changes.

The OMAR Assessment has demonstrated the unique capabilities of the HEC-6T numerical model to predict system-wide and long-term sediment responses to geomorphological and design changes throughout the MR&T Project. It is recommended that the numerical model be enhanced by providing additional detail and by collection of more robust sediment and hydraulic data.

As described in this report, some approximations were necessary to construct this comprehensive model. One notable approximation was the sediment data required to establish boundary conditions. Sediment data collections on the Mississippi River should be improved in the following ways:

1. Sampling protocols in terms of equipment, sampling methods, laboratory methods and analytical treatment of data at existing stations should be more consistent.
2. Optimal sediment sampling sites for determining sediment transport processes should be selected. Sites where there is significant lateral

variation in velocity (bends), where there is reverse flow (eddies), where there is significant overbank flow, or where the channel is subject to artificial change (dredging) are not ideal.

3. Ensure adequate duplicate sampling necessary to identify the effects of protocol changes.
4. As Einstein (1950) demonstrated, sediment size class distribution should be included in laboratory protocols in to determine accurate sediment transport volumes. This is especially significant over 1000 mi of the Mississippi River where the forces that determine the quantity of sediment in transport change along the course of the river.
5. A readily available comprehensive sediment data base should be considered. Frequently absent from existing data are sampler type (recently, the USGS has reported sampler types at most sites but historic samplers are undocumented), point-sample vertical and horizontal distributions (typically, available on raw data sheets but not on readily available web sites), analytical methodology for calculating average sediment concentration, information on the maximum sample depth for depth integrated samples
6. The contribution of the bed load to the total sediment load is relatively unknown. However, recent technological advancements allow the bed load to be measured by implementing the ISSDOTv2 technology to measure bed form transport (Abraham et al. 2011, 2017, 2018; Heath et al. 2015; McAlpin et al. 2022). Previous work at various sites indicates that a constant 15% increase in suspended load transport understates the bed load contribution to total sand load at low to moderate flows. A future consideration should be analytical studies to determine if it is possible to correlate bed load to suspended load measurements (e.g., modified Einstein method (Colby and Hembree 1956), or to calibrated transport functions). Bed-load measurements are required to make these advances.
7. The sediment data collection program will not only benefit the HEC-6T model but is equally essential to all engineering efforts on the Mississippi River.

It is recommended that the sediment sampling and analysis program for the MR&T Project be consolidated under the leadership of a dedicated person in the MVD Mississippi River Science and Technology Office. This organization is appropriate because the Mississippi River acts as a system and does not change characteristics at district boundaries, and this office already coordinates sediment research for the MR&T Project and has the expertise to understand the data requirements for sedimentation research.

To achieve the necessary organizational impetus to achieve system-wide coordination of the sediment data collection program, it is recommended that all USACE funding for sediment sampling disseminate through the MVD Science and Technology Office. This includes all funding provided by the USACE to the USGS related to the MR&T Project.

In addition to sediment data, the collection of more consistent hydraulic data would allow for monitoring continuing geomorphic changes and provide helpful data for engineering analyses to predict future trends. MVN has been taking monthly discharge measurements in the distributaries between Head of Passes and Venice. These measurements should continue. Additional measurements should be considered for the distributaries in Southwest Pass and above Venice.

6.2.1 Recommendations to enhance model capabilities

It is recommended that the existing HEC-6T model (not the existing code itself) be enhanced to provide more detailed capabilities to model river responses at specific locations along the river. Some of these are listed below:

1. Create a segment representing the Auxiliary Inflow Channel to investigate the long-term impacts of operations scenarios on channel trap efficiency and flushing.
2. Modify the model using reported or proposed daily diversion flows at each of the ORCC structures. This would allow non-concurrent operation of individual structures and potentially increase the accuracy of bed change estimates at sections between the individual structures. This would not require modification of the existing code but would require considerably more detailed model input.
3. Conduct a sensitivity analysis of sediment diversion coefficient estimates. Modify sediment diversion coefficients used in operational scenarios based on changes computed by multi-dimensional models.
4. Extend the upstream model boundary on the Middle Mississippi/Missouri Rivers to a Missouri River sediment gage. This would provide for a more reliable upstream boundary.

6.2.2 Specific recommendations related to sediment inflow

Additional recommendations were provided by William A. Thomas as part of his review of this document and are contained in Appendix B.

Mr. Thomas concluded that it does not seem likely that improvements in the inflowing sediment load that are recommended in Appendix B will affect the rankings of scenarios in the OMAR Assessment. However, these recommendations should be considered when the model is used for overall management of the Mississippi River.

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Appendix A: Scenario Flow Diversions

A.1 Base Condition

Sedimentation effects of each scenario were compared to those calculated for the Base Condition. All of the scenario stimulations and the Base Condition started with 2019 calculated sediment and geometry conditions and simulated sedimentation processes for 50 yr into the future. The 50 yr hydrograph was developed from 1969–2019 hydrologic data at the model boundaries. The diversion percentages through each ORCC structure for each scenario was calculated from the 1991–2019 historical daily data, based on the assigned protocol for that scenario.

The daily discharges through each ORCC structure were based on long-term average diversion percentages that were calculated from the assigned daily allocations for each scenario. It was not necessary to assign actual daily percentages in the model to achieve the objective of comparing long-term sedimentation effects. For the Base Condition, the distribution of diverted flow through the various ORCC structures was based on data collected between 2013 and 2019, which represents the existing operation protocol.

A.2 Scenario 3 - Ratio 1 (R1)

The Ratio 1 scenario includes flow through the Overbank, Low Sill, and Auxiliary Structures. There is no flow assigned to the Hydropower Structure. Diversion percentages were taken from the ORCC Operation and Maintenance Manual (revised Table 3, USACE MVN 1988), which provides diversion percentages through each structure as a function of total ORCC discharge up to 620,000 cfs. Using the total ORCC discharge as the independent variable to determine the discharge allocation to each structure is the logical approach to facilitate structure operations, which strives to maintain the designated 70-30 total latitude flow distribution. However, the HEC-6T numerical model diverts flow out of the Mississippi River as a function of the discharge upstream from the diversion. Therefore, a regression relationship between ORCC discharge and Mississippi River discharge was developed. Scatter about this relationship is expected because the regulated ORCC discharge is also a function of Red River discharge. The average Ratio 1, water year 1991–water year 2019, relationship between Union Point discharges, as a function of ORCC discharges, is shown in Figure A-80. A third-order polynomial regression

defines the relationship for ORCC discharges less than 500,000 cfs, and a linear regression defines the relationship for ORCC discharges greater than 500,000 cfs. Use of this relationship is appropriate for determining long-term effects of ORCC operations.

Using the diversion percentages equations for each ORCC structure from the Operation Manual and the regression equation from Figure A-80, diversion percentages of the Union Point discharge were calculated. The Ratio 1 calculated discharges for each structure, as a function of ORCC discharge, are provided in Table A-43. Maximum flow through the ORCC was constrained at 970,000 cfs. Maximum discharge through the Auxiliary, Low Sill, and Overbank Structures were set at 350,000; 300,000; and 320,000 cfs, respectively.

The 1991–2019 daily Ratio 1 discharge distributions are plotted with the HEC-6T rating curves on Figure A-81 through Figure A-83. These figures show the scatter that occurs in the daily discharges due to neglecting coincident Red River discharges and to inconsistent weighting of discharge between the Auxiliary and Low Sill Structures. The “HEC-6T R1 equations” curve was used in the model and is based on the Operation Manual equations. The 1991–2019 data and regression curves are based on actual historical data and are provided for reference. The final diversion percentages, as a function of Union Point discharge, used in the model for Ratio 1 are shown in Table A-44 through Table A-46.

Figure A-80. Ratio 1 discharge relationship ORCC and Union Point 1991-2019.

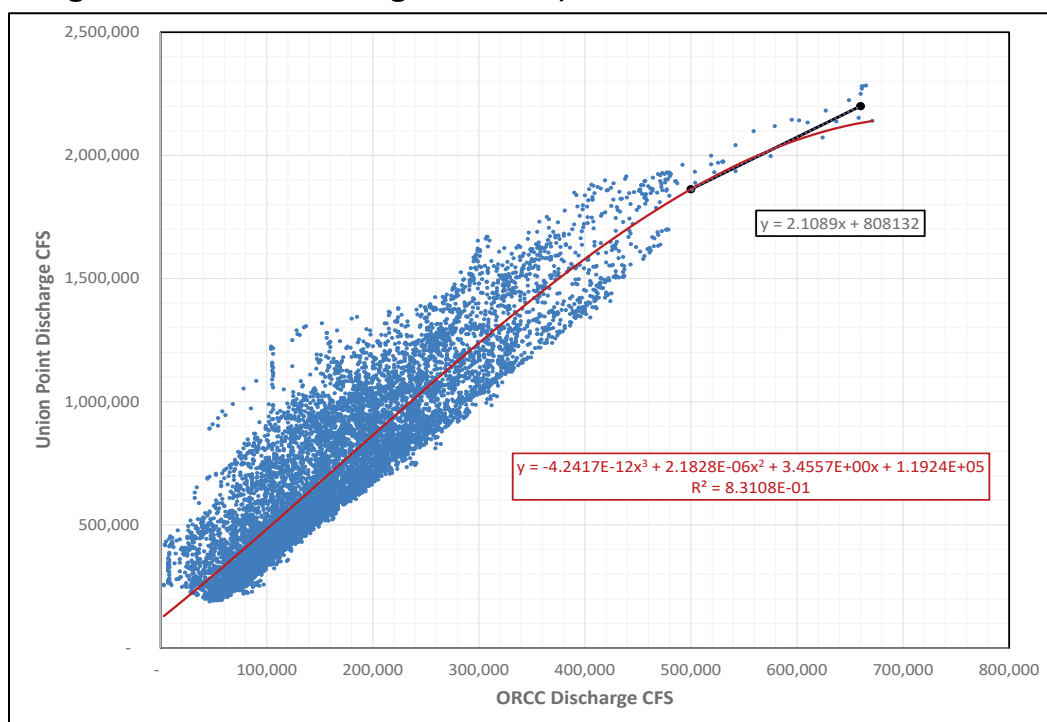


Figure A-81. Ratio 1 discharge rating curve for Overbank Structure.

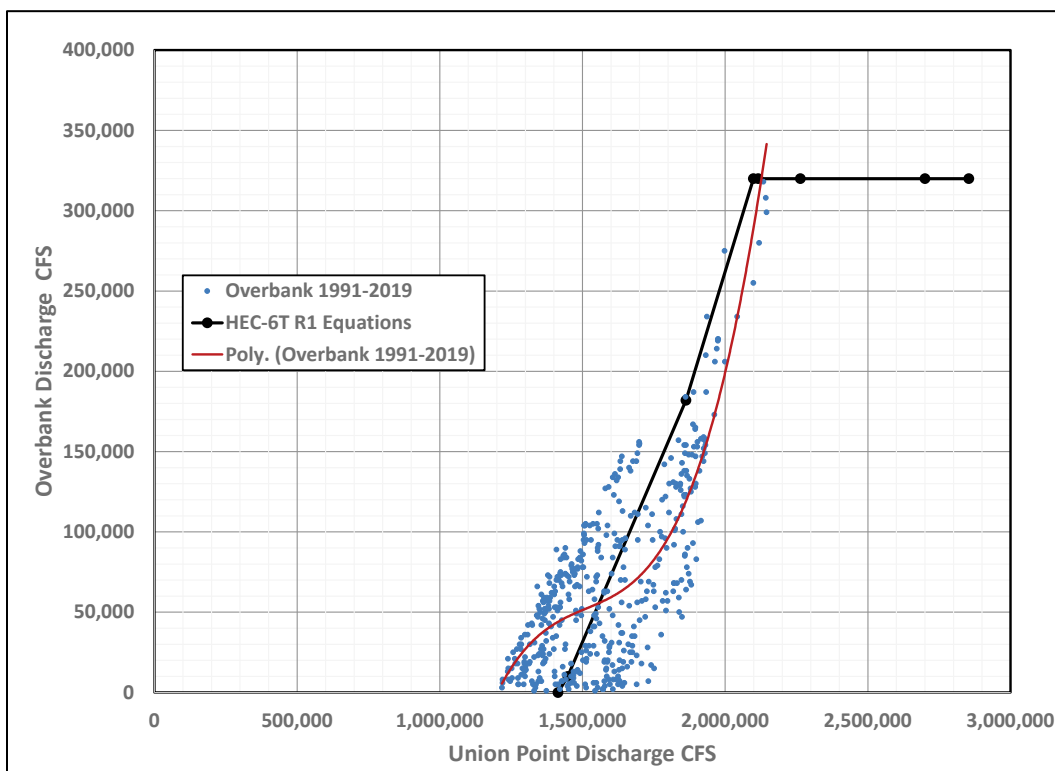


Figure A-82. Ratio 1 discharge rating curve for Low Sill Structure.

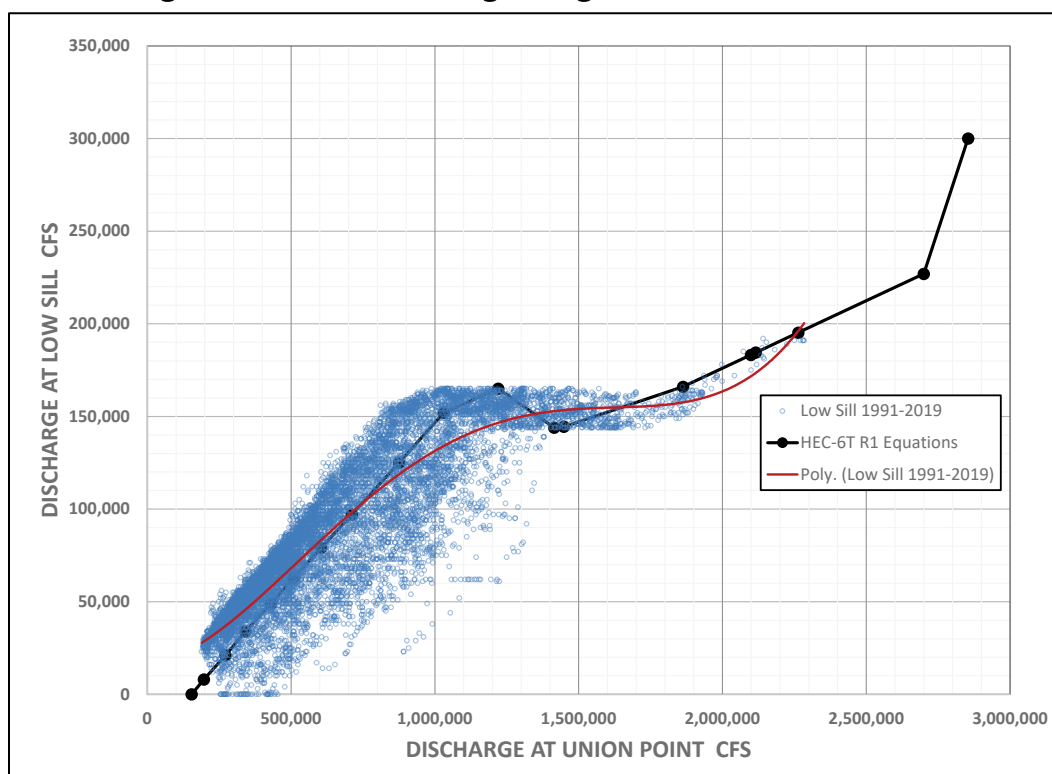


Figure A-83. Ratio 1 discharge rating curve for Auxiliary Structure.

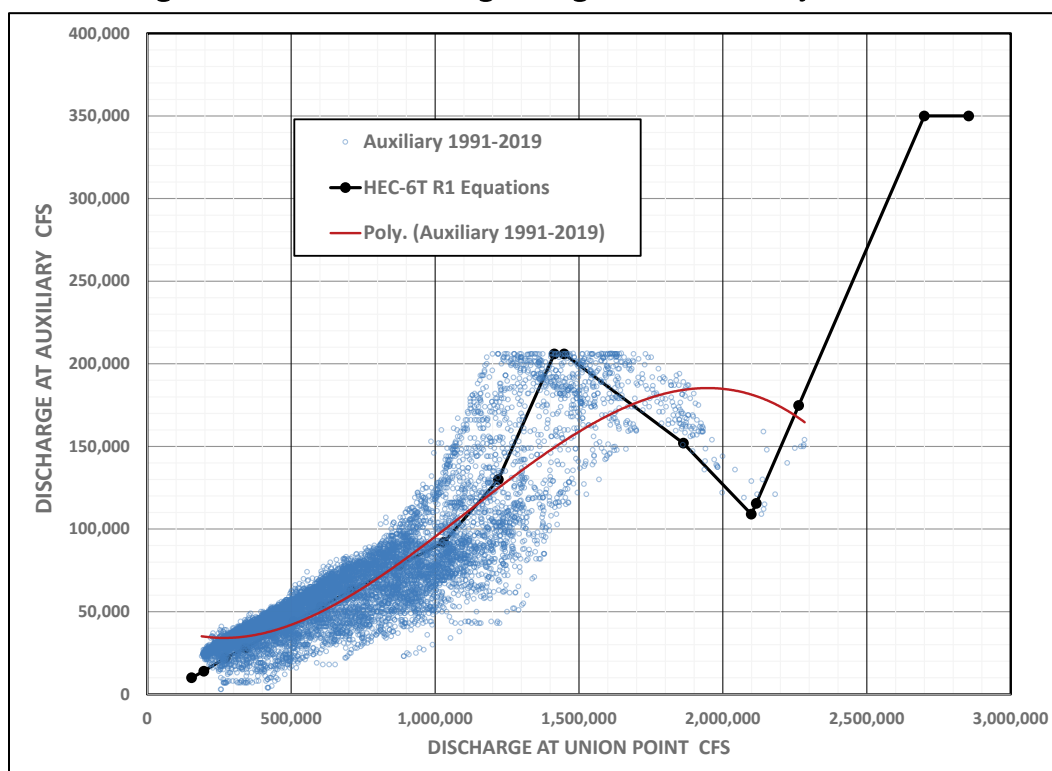


Table A-43. Ratio 1 upstream and structure discharges.

ORCC ft ³ / sec	Union Point Regression ft ³ / sec	Overbank Ratio 1 ft ³ / sec	Low Sill Ratio 1 ft ³ / sec	Auxiliary Ratio 1 ft ³ / sec
10,000	154,000	0	0	10,000
22,000	196,000	0	8,000	14,000
43,000	272,000	0	21,000	22,000
62,000	341,000	0	34,000	28,000
86,000	430,000	0	49,000	37,000
107,000	509,000	0	63,000	44,000
132,000	604,000	0	79,000	53,000
160,000	711,000	0	97,000	63,000
203,000	875,000	0	125,000	78,000
244,000	1,031,000	0	152,000	92,000
295,000	1,220,000	0	165,000	130,000
350,000	1,414,000	0	144,000	206,000
360,000	1,448,000	10,000	144,000	206,000
500,000	1,863,000	182,000	166,000	152,000
612,000	2,099,000	320,000	183,000	109,000
620,000	2,116,000	320,000	184,000	116,000
690,000	2,263,000	320,000	195,000	175,000
897,000	2,700,000	320,000	227,000	350,000
970,000	2,854,000	320,000	300,000	350,000

Table A-44 HEC-6T Ratio 1 Diversion percentages at Overbank Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
0	1,414,000
0.69	1,448,000
15.25	2,099,000
15.13	2,116,000
14.14	2,263,000
11.85	2,700,000
11.21	2,854,000

Table A-45. HEC-6T Ratio 1 Diversion percentages at Low Sill Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
0	154,000
3.92	196,000
9.89	341,000
12.37	509,000
14.75	1,031,000
13.55	1,220,000
9.94	1,448,000
8.91	1,863,000
8.41	2,700,000
10.51	2,854,000

Table A-46. HEC-6T Ratio 1 Diversion percentages at Auxiliary Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
6.49	10,000
6.49	154,000
8.30	341,000
8.93	1,031,000
10.64	1,220,000
14.54	1,414,000
14.22	1,448,000
5.21	2,099,000
12.96	2,700,000
12.26	2,854,000

A.3 Scenario 4 - Ratio 1 with Hydropower (R1WHP)

This scenario uses the same Union Point versus ORCC rating curve used for the Ratio 1 (Figure A-80). Ratio 1 Low Sill discharges are assigned to the Hydropower Structure up to a maximum of 150,000 cfs, with the remainder assigned to the Auxiliary Structure. There is no diversion at the Low Sill Structure until Union Point discharge exceeds 2,263,000 ft³/sec. Overbank diversions are the same as Ratio 1. HEC-6T Hydropower, Low

Sill, and Auxiliary rating curves for Scenario 4 are shown in Table A-47 through Table A-49.

Table A-47. HEC-6T R1WH Diversion percentages at Hydropower Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
0.0	154,000
3.92	196,000
9.89	341,000
12.37	509,000
14.55	1,031,000
12.30	1,220,000
10.36	1,448,000
8.05	1,863,000
6.63	2,263,000
5.26	2,854,000

Table A-48. HEC-6T R1WH Diversion percentages at Low Sill Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
0	10,000
0	2,263,000
2.85	2,700,000
5.26	2,854,000

Table A-49. HEC-6T R1WH Diversion percentages at Auxiliary Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
6.49	10,000
6.49	154,000
8.30	341,000
9.12	1,031,000
11.89	1,220,000
14.14	1,414,000
6.77	2,099,000
9.72	2,263,000
12.96	2,700,000
12.26	2,854,000

A.4 Scenarios 5–10

Assigned daily discharges through each of the ORCC structures for each scenario are located in Excel file *OMAR_AllScenarios_20210106.xlsx*. These discharges were calculated based on specified scenario allocations, using reported 1 January 1990 through 31 December 2019 discharge data as a reference. Daily discharges at Union Point for each scenario were calculated as the sum of Tarbert Landing and ORCC daily discharges from the Excel file.

HEC-6T requires that diversion percentages be assigned as a function of an upstream discharge. Therefore, a relationship between daily ORCC and Union Point discharges was required. Regression equations were calculated in Excel using both 1990–2019 and 2013–2019 daily data. The 1990–2019 regressions were adopted to determine the ORCC versus Union Point relationship because the flow distributions through the individual structures do not affect the overall distribution, and the data base includes a larger discharge range.

The relationship between Hydropower and Union Point discharge was based on 26 May 1990 to 31 December 2019 data. Hydropower was opened 26 May 1990. Hydropower flow distribution is generally independent from the flow distribution through the other structures.

Overbank Spillway diversion percentages, as a function of Union Point discharge, were taken from scenario allocations that had been calculated using historical 1 October 2012 through 30 September 2019, data as a reference. The 2013–2019 scenario allocations were used to develop the rating curve used in the 50 yr predictive simulations because the Overbank Spillway was not used between 1990 and 2012, and therefore those years were deemed unrepresentative of future operations. Flow does not occur over the Overbank Spillway until Union Point discharge exceeds 1,100,000 cfs. The Overbank Spillway discharge was capped at 320,000 cfs, which is the structure's capacity.

The Low Sill and Auxiliary diversion percentages for Scenarios 5 through 10 did not correlate well with the upstream Mississippi River discharge. This is different from the two Ratio 1 scenarios (R1 and R1WH) in that the Ratio 1 scenarios were based on a systematic annual relationship between the ORCC diversions so that diversion percentages could be based reliably

on upstream discharges. In contrast, the Scenario 5 through 10 relationships were based on daily historical data that are poorly correlated to upstream discharge. Consequently, the Low Sill and Auxiliary diversion percentages were obtained by determining the total diversion percentages through the two structures and then assigning a straight average of the long-term percentages through each of the two structures.

Low Sill and Auxiliary diversion percentages for the 50-yr future simulations were calculated using daily discharges between 2013 and 2019. The shorter data base was used because diversion percentages calculated in *OMAR_AllScenarios_20210106* were influenced significantly by the non-usage of the Overbank Structure between 1991 and 2012. Using daily discharges between 2013 and 2019, 7 yr average diversion percentages were calculated for each structure.

The daily discharge upstream from the Low Sill Structure was calculated by subtracting Hydropower and Overbank daily discharges from the Union Point discharge. The Hydropower discharge was calculated from the 1990–2019 regression equation, and the Overbank discharge was calculated from the 2013–2019 regression equation. The 2013–2019 discharges through the combined Low Sill and Auxiliary Structures were calculated by subtracting the Hydropower and Overbank daily discharges from the ORCC discharge, which was determined from the 1990–2013 regression curve.

The historical data base limited the reliability of the regression equations to a Union Point discharge range between 200,000 and 2,300,000 cfs. Maximum Union Point discharge during the 50 yr simulation is 2,542,460 cfs. Low Sill discharge capacity was set at 300,000 cfs. Auxiliary discharge capacity was set at 350,000 cfs.

A.5 Scenario 5 – 60/40 High Flow (HF6040)

Scenario 5 increases the latitude target discharge in the Atchafalaya River from 30% to 40%, but only at high flow. The annual 70/30 latitude flow is maintained by decreasing diversion percentages at low flows. The assigned Scenario 5 relationship between ORCC discharges as a function of Union Point discharges is shown in Figure A-84. Three regression curves were used to define the relationship.

The average Scenario 5 relationship between the Hydropower Structure daily discharges, as a function of Union Point discharges, is shown in Figure A-85. A fifth-order polynomial regression defined the relationship when Union Point discharges are less than 1,400,000 cfs. When the Union Point discharge exceeded 1,400,000 cfs, Hydropower discharge was capped at 170,000 cfs, which is the maximum Hydropower discharge for Scenario 5.

HEC-6T Scenario 5 diversion percentages for the Overbank Spillway were determined from 2013–2019 data when the discharge at Union Point was less than 2,000,000 cfs. Of note, these discharges are higher than those designated in the Scenario 5 hindcasting spreadsheet. When the Union Point discharge exceeded 2,200,000 cfs, the HEC-6T Overbank Spillway was operated at capacity, which is 320,000 cfs. A fourth-order polynomial relationship between Overbank Spillway diversion discharge and Union Point discharge is shown in Figure A-86.

Long-term average diversion percentages through the Low Sill and Auxiliary Structures were calculated using the assigned daily discharges between 2013 and 2019. The discharges through the combined Low Sill and Auxiliary Structures were calculated by subtracting Hydropower and Overbank Spillway regression-equation discharges from ORCC regression-equation discharges. Using the Scenario 5 data for Union Point discharges above 2,355,000 cfs, the combined Low Sill and Auxiliary discharge was held constant at 480,000 cfs. This assignment is consistent with assignments in *OMAR_AllScenarios_20210106*. Years 2013–2019 average daily discharges for the Low Sill and Auxiliary Structures were summed, and it was determined that 52.13% of the flow went through the Low Sill and 47.87% went through the Auxiliary. These fractions were used to determine each structures diversion percentages.

Diversion percentages for Scenario 5 (HF6040) are shown in Table-8 through Table A-10.

Figure A-84. Scenario 5 discharge relationship ORCC and Union Point 1991–2019.

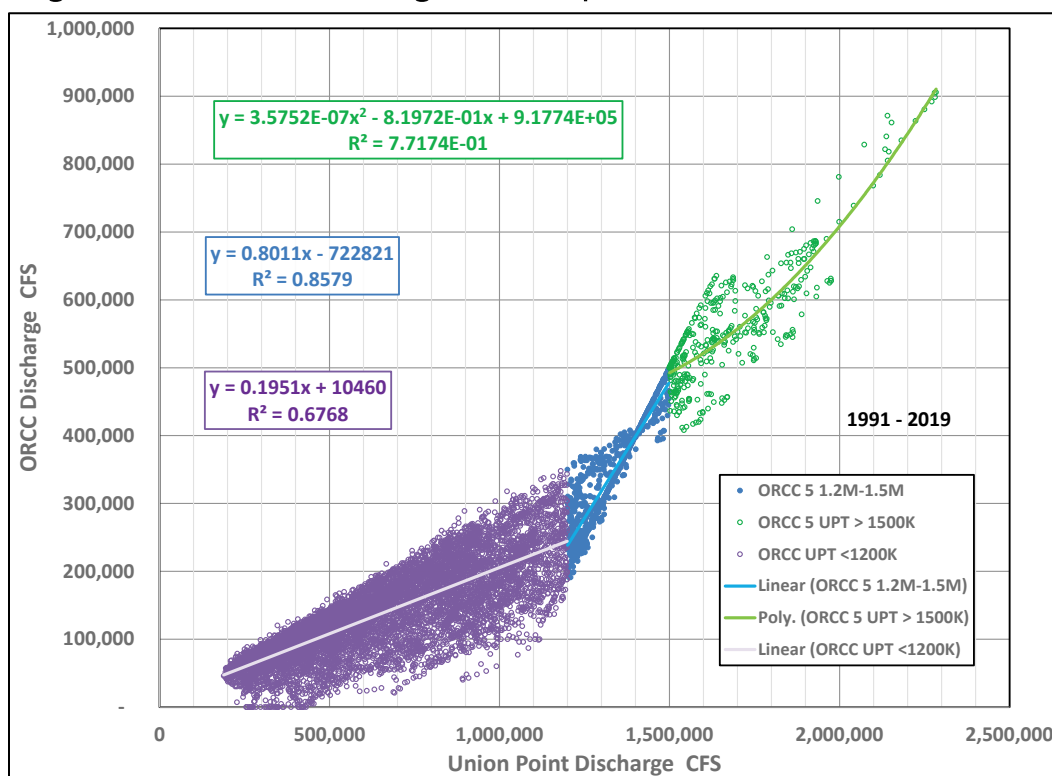


Figure A-85. Scenario 5 discharge relationship Hydropower and Union Point 1990–2019.

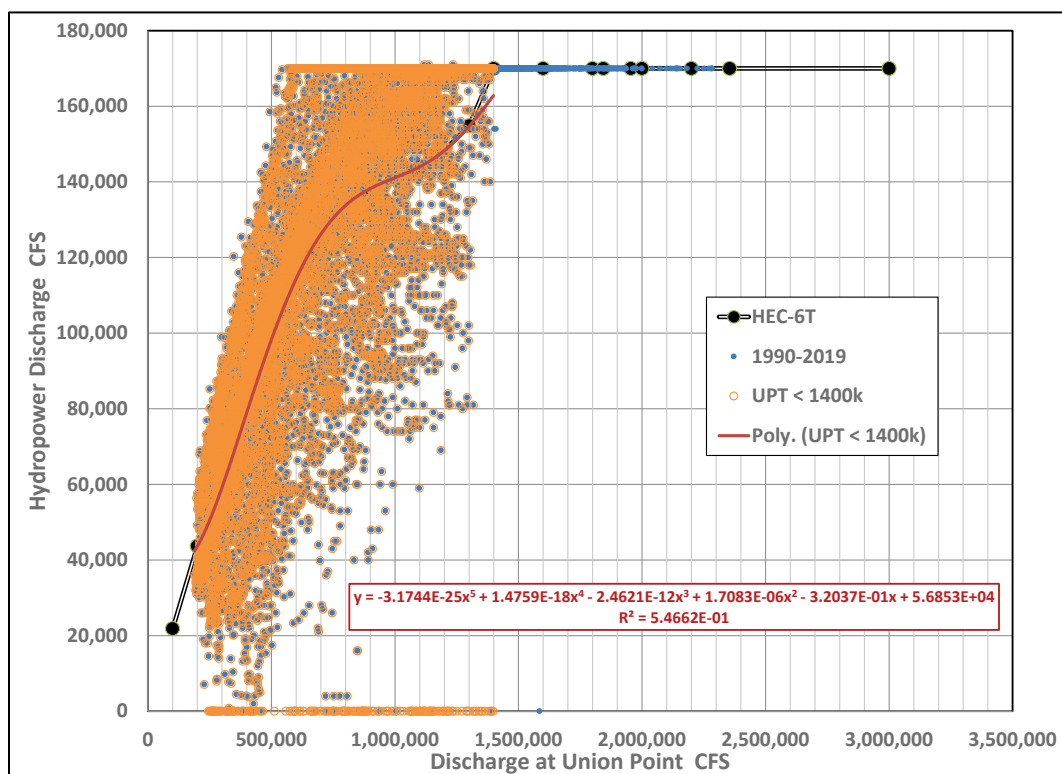


Figure A-86. Scenario 5 discharge relationship Overbank and Union Point 2013–2019.

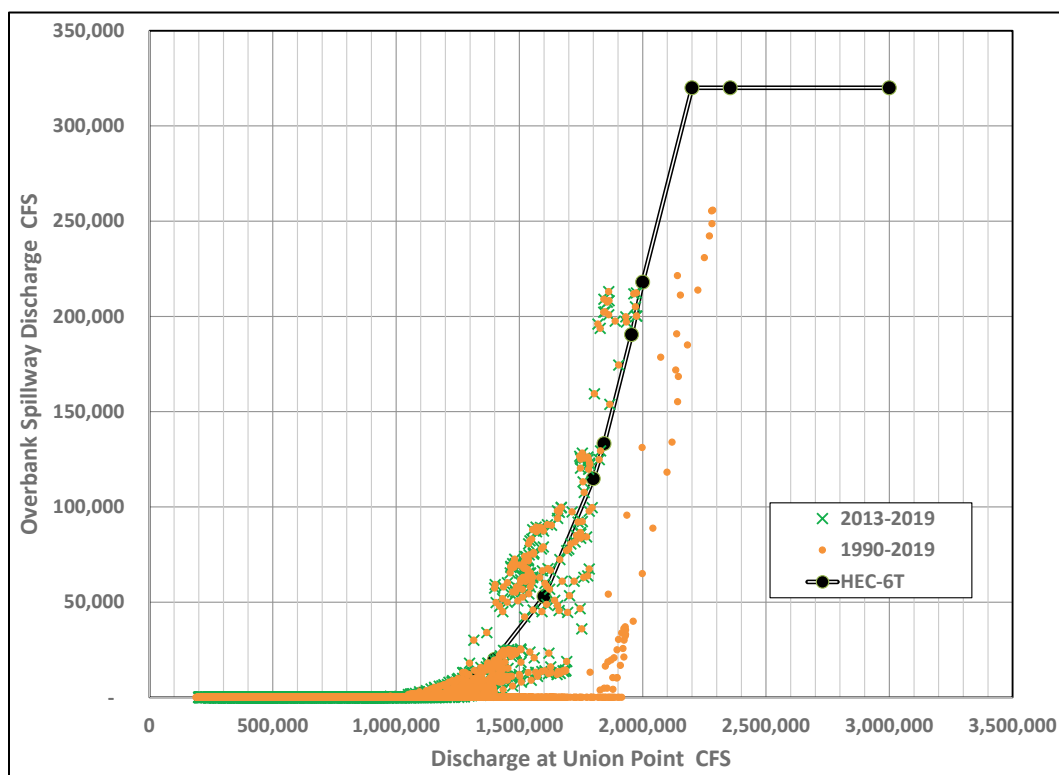


Table A-50. Scenario 5 (HF6040) diversion percentages at Hydropower Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
21.84	100,000
21.84	200,000
18.59	650,000
15.32	900,000
13.10	1,100,000
12.14	1,400,000
10.63	1,600,000
9.44	1,800,000
7.22	2,355,000
5.67	3,000,000

**Table A-51. Scenario 5 (HF6040) diversion percentages
at Overbank Structure.**

Percentage Diverted	Union Point Discharge ft ³ /sec
0	900,000
0.17	1,100,000
1.42	1,400,000
3.31	1,600,000
6.38	1,800,000
10.90	2,000,000
14.55	2,200,000
13.59	2,355,000
10.67	3,000,000

**Table A-52. Scenario 5 (HF6040) diversion percentages
at Low Sill Structure.**

Percentage Diverted	Above Low Sill Discharge ft ³ /sec
0	78,000
1.94	156,000
1.26	402,000
4.54	1,047,000
9.00	1,210,000
11.30	1,377,000
10.36	1,612,000
10.81	1,710,000
16.09	1,865,000
11.95	2,510,000

Table A-53. Scenario 5 (HF6040) diversion percentages at Auxiliary Structure.

Percentage Diverted	Above Low Sill Discharge ft ³ /sec
0	78,000
1.78	156,000
1.16	402,000
4.17	1,047,000
8.26	1,210,000
10.38	1,377,000
9.51	1,612,000
9.93	1,710,000
9.65	1,865,000
7.17	2,510,000

A.6 Scenario 6 – 80/20 High Flow (HF8020)

Scenario 6 decreases the latitude target discharge in the Atchafalaya River from 30% to 20%, but only at high flow. The annual 70/30 latitude flow is maintained by increasing diversion percentages at low flows.

The assigned Scenario 6 daily data were used to develop regression curves to define the average ORCC and Union Point relationship and are shown in Figure A-87. A fourth-order polynomial regression was calculated in Excel from the 1990–2019 data. For reference purposes, a third-order polynomial regression was calculated from the 2013–2019 data. The 1990–2019 regression curve was adopted to take advantage of the larger data base and the inclusion of data at higher discharges. The total diversion percentages at ORCC should not be affected by the changes in operations at the Overbank Structure. The adopted regression curve was modified slightly when Union Point discharges exceeded 1,900,000 cfs. A linear rating curve was found to fit the high flow calculated data better than the polynomial equation. The range of data used to develop the regression curve was between 200,000 and 2,300,000 cfs. In developing the HEC-6T rating curve, the percentage diverted through the ORCC from the Mississippi River was held constant at the high and low limits of data (no extrapolation).

The average Scenario 6 relationship between the Hydropower Structure daily discharges, as a function of Union Point discharges, is shown in Figure A-88. A fourth-order polynomial regression was calculated in Excel from the 26 May 1990–31 December 2019 data. This regression curve was used for the HEC-6T rating up to Union Point discharges of 2,300,000 cfs. A linear regression was also calculated for Union Point discharges greater than 1,600,000 cfs. The linear regression was used for reference in developing the HEC-6T rating curve at high Union Point discharges. The range of data used to develop the regression curves was between 200,000 and 2,300,000 cfs. In developing the HEC-6T rating curve, the discharge diverted through the Hydropower Structure from the Mississippi River was capped at a maximum of 170,000 cfs at a Union Point discharge of 2,600,000 cfs. At discharges less than 200,000 cfs, the diversion percentage was held constant at the data limit (no extrapolation).

Overbank Spillway diversion percentages, as a function of Union Point discharge, were determined from scenario allocations that had been calculated using historical 1 October 2012 through 30 September 2019 data as a reference. Flow does not occur over the Overbank Spillway until Union Point discharge exceeds 1,100,000 cfs. A fourth-order polynomial regression equation defines the Overbank Spillway's discharge relationship to Union Point discharge when the Union Point discharge does not exceed 2,000,000 cfs. The calculated Overbank Spillway discharge was 78,000 cfs at this limit. In developing the HEC-6T rating curve, the percentage diverted through the Overbank Spillway from the Mississippi River was held constant when upstream flow exceeded 2,000,000 cfs (no extrapolation). The Overbank Spillway versus Union Point relationship is shown in Figure A-89.

Long-term average diversion percentages through the Low Sill and Auxiliary Structures were calculated using the assigned daily discharges between 2013 and 2019. The flow distribution between the Low Sill and Auxiliary Structures was determined by calculating the long-term (2013–2019) average distributions. This calculation determined that the flow distribution was essentially evenly divided between the two structures, with 50.1% allocated to the Low Sill and 49.9% to the Auxiliary. There was no need to cap any of the flows through the structures because of the reduced diversion through ORCC for Scenario 6.

Diversion percentages for Scenario 6 (HF8020) are shown in Table A-54 through Table A-57.

Figure A-87. Scenario 6 discharge relationship ORCC and Union Point 1990–2019.

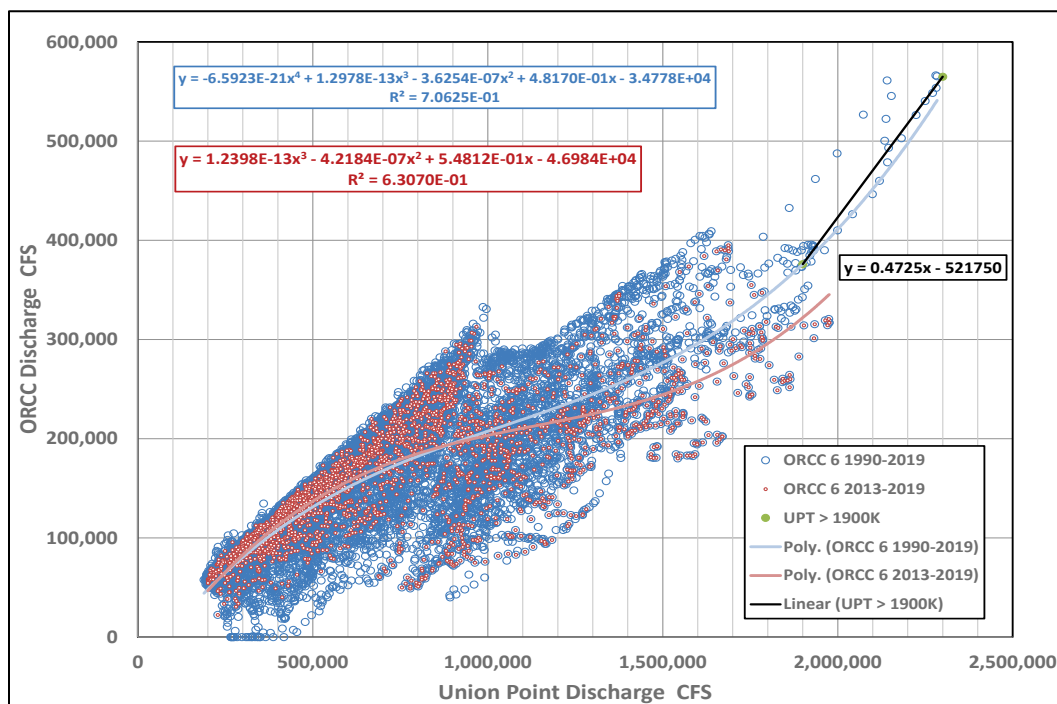


Figure A-88. Scenario 6 discharge relationship Hydropower and Union Point 1990–2019.

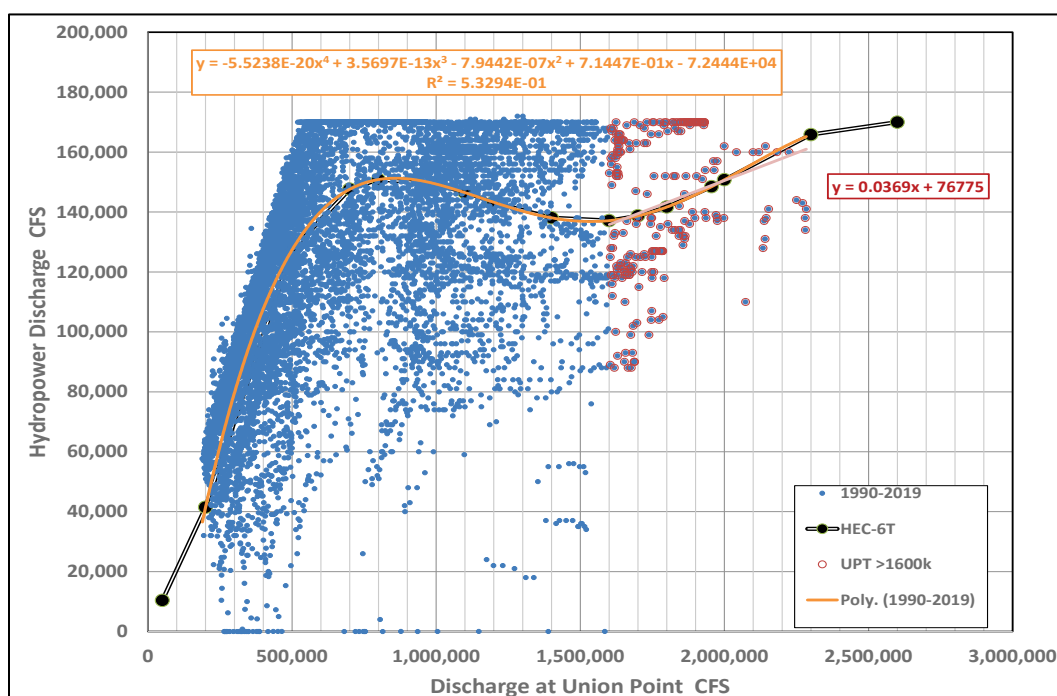


Figure A-89. Scenario 6 discharge relationship Overbank and Union Point 2013–2019.

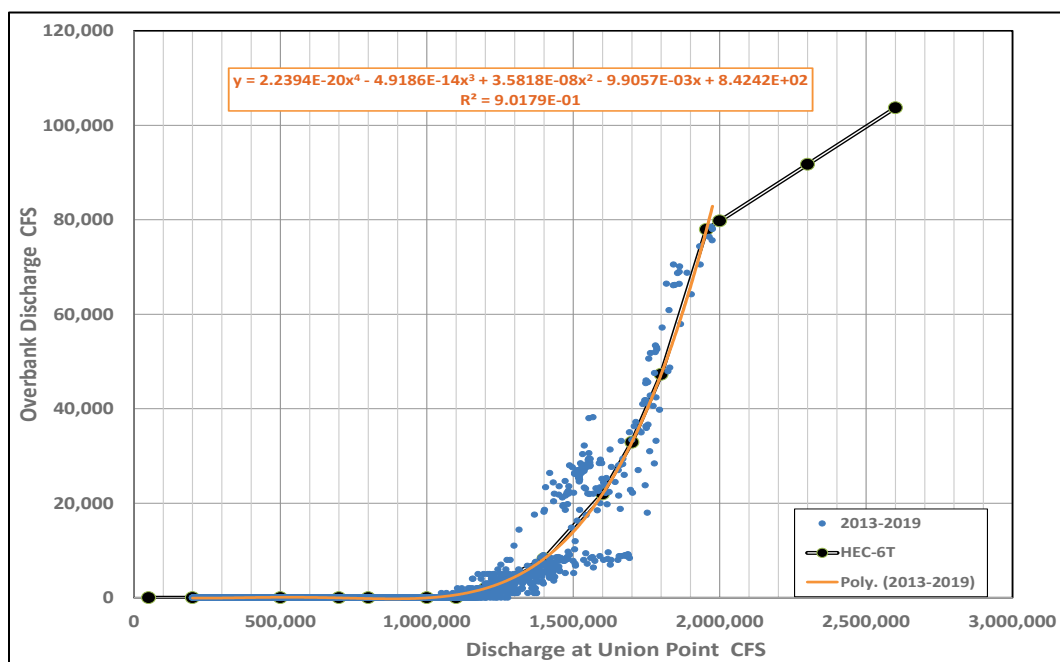


Table A-54. Scenario 6 (HF8020) diversion percentages at Hydropower Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
20.72	100,000
20.72	200,000
25.47	500,000
14.93	1,000,000
9.86	1,400,000
8.57	1,600,000
7.87	1,800,000
7.21	2,300,000
6.54	2,600,000

Table A-55. Scenario 6 (HF8020) diversion percentages at Overbank Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
0	1,100,000
0.59	1,400,000
1.37	1,600,000
2.63	1,800,000
3.99	2,000,000
3.99	2,600,000

Table A-56. Scenario 6 (HF8020) diversion percentages at Low Sill Structure.

Percentage Diverted	Above Low Sill Discharge ft ³ /sec
0	159,000
0.52	373,000
3.43	851,000
4.54	1,254,000
5.09	1,729,000
7.54	2,042,000
7.86	2,326,000

Table A-57. Scenario 6 (HF8020) diversion percentages at Auxiliary Structure.

Percentage Diverted	Above Low Sill Discharge ft ³ /sec
0	159,000
0.52	373,000
3.42	851,000
4.52	1,254,000
5.07	1,729,000
7.51	2,042,000
7.83	2,326,000

A.7 Scenario 7 –Cap Tarbert Landing at 1,250,000 cfs (CTAR125)

Scenario 7 retains the current 70/30 latitude target discharge on an annual basis, but not on a daily basis. Mississippi River discharge is limited to 1,250,000 cfs downstream from ORCC (up to the ORCC capacity). A payback/tracking protocol ensures the 70/30 distribution annually.

The assigned Scenario 7 daily data were used to develop regression curves to define the average ORCC and Union Point relationship and are shown in Figure A-90. A fourth-order polynomial regression defines the relationship for Union Point discharges between 1990 and 2019, and a third-order polynomial regression defines the relationship between 1990 and 2019. The regression curves are similar. The 1990–2019 regression curve was used to determine ORCC discharges from Union Point discharges. Maximum discharge through the ORCC was capped at 970,000 cfs.

The average Scenario 7 relationship between the Hydropower Structure daily discharges, as a function of Union Point discharges, is shown in in Figure A-91. A fourth-order polynomial regression defines the relationship between 1990 and 2019 Hydropower and Union Point discharges when Union Point discharges are less than 1,700,000 cfs. When the Union Point discharge exceeded 1,800,000 cfs, Hydropower discharge was capped at 170,000 cfs, which is the maximum Hydropower discharge for Scenario 7.

Overbank Spillway diversion percentages, as a function of Union Point discharge, were determined from scenario allocations that had been calculated using historical 1 October 2012 through 30 September 2019 data as a reference. Flow does not occur over the Overbank Spillway until Union Point discharge exceeds 1,100,000 cfs. A fourth-order polynomial regression equation defines the Overbank Spillway's discharge relationship to Union Point discharge when the Union Point discharge does not exceed 2,000,000 cfs. The calculated Overbank Spillway discharge was 256,000 cfs at this limit. The Overbank Spillway discharge was capped at 320,000 cfs when the Union Point discharge exceeded 2,300,000 cfs. The Overbank Spillway versus Union Point relationship is shown in Figure A-92.

Long-term average diversion percentages through the Low Sill and Auxiliary Structures were calculated using the assigned daily discharges

between 2013 and 2019. The flow distribution between the Low Sill and Auxiliary Structures was determined by calculating the long-term (2013–2019) average distributions. This calculation determined that the flow distribution was essentially evenly divided between the two structures, with 50.1% allocated to the Low Sill and 49.9% to the Auxiliary.

Long-term average diversion percentages through the Low Sill and Auxiliary Structures were calculated using the assigned daily discharges between 2013 and 2019. The flow distribution between the Low Sill and Auxiliary Structures was determined by calculating the long-term (2013–2019) average distributions. This calculation determined that the flow distribution was essentially evenly divided between the two structures, with 50.1% allocated to the Low Sill and 49.9% to the Auxiliary. The combined flow through the Low Sill and Auxiliary was capped at 480,000 cfs when the Mississippi River discharge upstream from the Low Sill exceeded 2,110,000 cfs. The maximum Auxiliary Structure discharge of 180,000 was reached when the Mississippi River discharge upstream from Low Sill exceeded 1,574,000 cfs. When this occurred, the excess combined flow was added to the Low Sill. The Low Sill reached its capacity when the upstream discharge reached 2,110,000 cfs.

Diversion percentages for Scenario 7 (CTAR125) are shown in Table A-58 through Table A-61.

Figure A-90. Scenario 7 discharge relationship ORCC and Union Point 1990–2019.

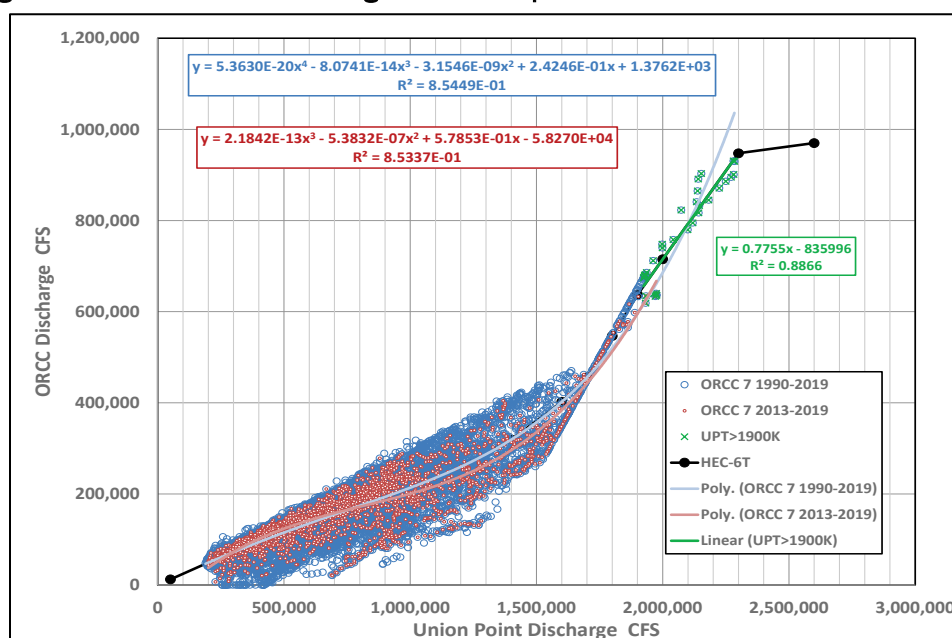


Figure A-91. Scenario 7 discharge relationship Hydropower and Union Point
1990-2019.

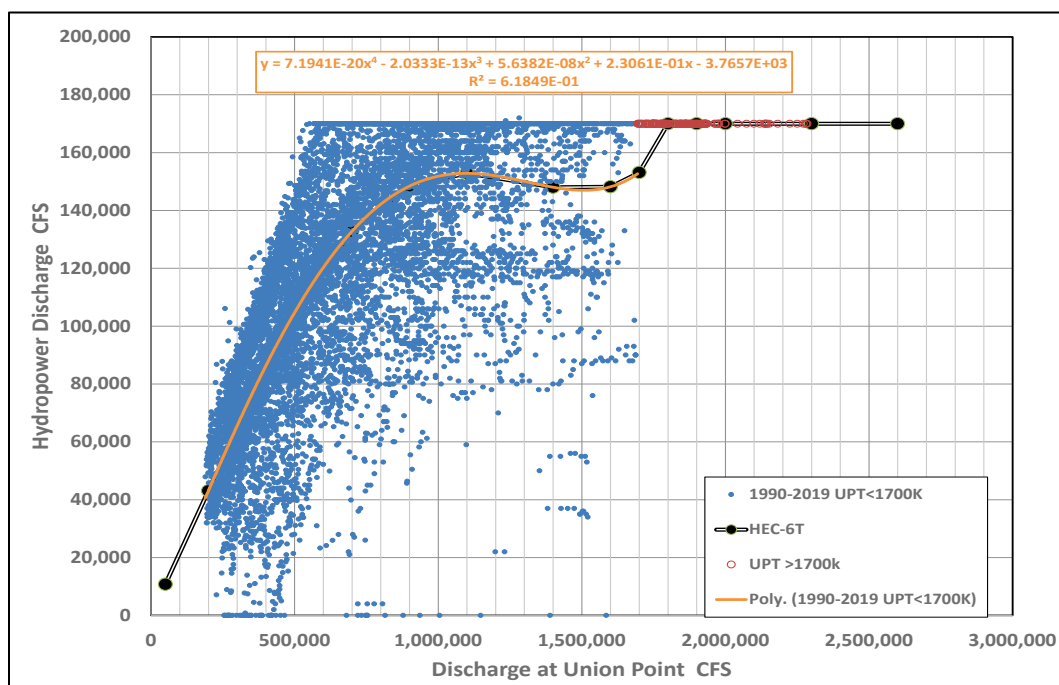


Figure A-92. Scenario 7 discharge relationship Overbank and Union Point
2013-2019.

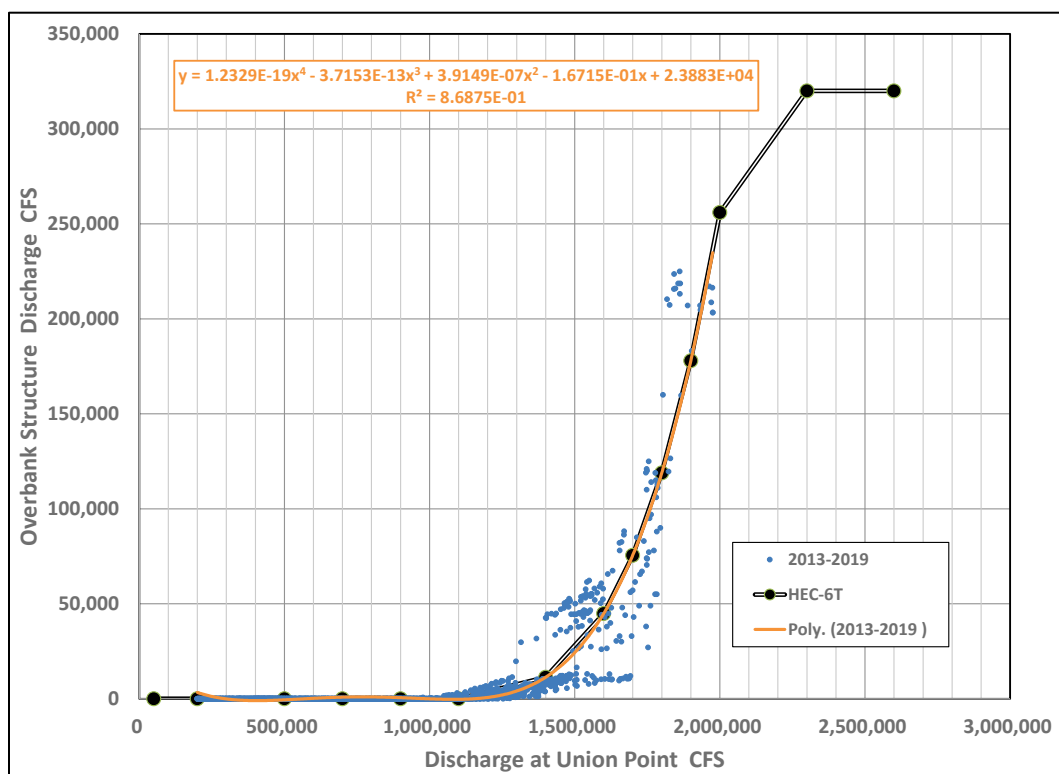


Table A-58. Scenario 7 (CTAR125) diversion percentages at Hydropower Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
21.55	100,000
21.55	200,000
20.94	500,000
10.57	1,400,000
9.01	1,700,000
9.44	1,800,000
8.50	2,000,000
7.39	2,300,000
6.54	2,600,000

Table A-59. Scenario 7 (CTAR125) diversion percentages at Overbank Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
0	1,100,000
0.81	1,400,000
2.80	1,600,000
6.61	1,800,000
9.37	1,900,000
12.80	2,000,000
13.91	2,300,000
12.31	2,600,000

Table A-60. Scenario 7 (CTAR125) diversion percentages at Low Sill Structure.

Percentage Diverted	Above Low Sill Discharge ft ³ /sec
1.53	39,000
1.53	157,000
1.04	395,000
1.57	567,000
6.56	1,407,000
7.26	1,574,000
12.07	1,810,000
11.54	2,110,000

Table A-61. Scenario 7 (CTAR125) diversion percentages at Auxiliary Structure.

Percentage Diverted	Above Low Sill Discharge ft ³ /sec
1.51	39,000
1.51	157,000
1.03	395,000
1.56	567,000
6.49	1,407,000
7.19	1,574,000
7.83	1,810,000
6.92	2,110,000

A.8 Scenario 8A – 80 -20 No Constraint on Low Sill (D8020)

Scenario 8A decreases the flow diverted from the Mississippi River on a daily basis. The daily latitude target discharge in the Atchafalaya River is decreased from 30% to 20%.

The assigned Scenario 8A daily data were used to develop regression curves to define the average ORCC and Union Point relationship and are shown in Figure A-93. A fifth-order polynomial regression was calculated in Excel from the 1990–2019 data. For reference purposes, a third-order polynomial regression was calculated from the 2013–2018 data. The 1990–2019 regression curve was adopted to take advantage of the larger data base and the inclusion of data at higher discharges. The adopted

regression curve was modified slightly when Union Point discharges exceeded 1,900,000 cfs. A linear rating curve was found to fit the calculated data better. The range of data used to develop the regression curves was 200,000–2,300,000 cfs.

The 1990–2019 Scenario 8A calculated daily discharges for the Hydropower Structure and Union Point were used to develop regression curves to define the average diversion from the Mississippi River and are shown in Figure A-94. A fourth-order polynomial regression was calculated in Excel using data from 26 May 1990 to 31 December 2019. This regression curve was used for the HEC-6T rating up to Union Point discharges of 1,800,000 cfs. A linear regression of data greater than 1,600,000 cfs was also developed and used for Union Point discharges between 1,800,000 and 2,300,000 cfs. In developing the HEC-6T rating curve, the discharge diverted through the Hydropower Structure from the Mississippi River was capped at a maximum of 170,000 cfs at a Union Point discharge of 2,600,000 cfs.

Overbank Spillway diversion percentages, as a function of Union Point discharge, were determined from scenario allocations that had been calculated using historical 1 October 2012 through 30 September 2019 data as a reference. Overbank Structure diversion percentages for Scenario 8A were significantly less than those determined from the historical 1979–2019 relationships with Union Point discharges. Flow does not occur over the Overbank Spillway until Union Point discharge exceeds 1,200,000 cfs. A fourth-order polynomial regression equation defines the Overbank Spillway discharge relationship to Union Point discharge when the Union Point discharge does not exceed 2,000,000 cfs. The calculated Overbank Spillway discharge was 13,000 cfs at this limit. In developing the HEC-6T rating curve, the percentage diverted through the Overbank Spillway from the Mississippi River was held constant when upstream flow exceeded 2,000,000 cfs (no extrapolation). The Overbank Spillway versus Union Point relationship is shown in Figure A-95.

Diversion percentages through the Low Sill and Auxiliary Structures were calculated using daily discharges between 2013 and 2019. The discharge through the combined Low Sill and Auxiliary Structures was calculated by subtracting Hydropower and Overbank Spillway regression-equation discharges from ORCC regression-equation discharges. The flow distribution between the Low Sill and Auxiliary Structures was determined

by calculating the long-term (2013–2019) average distributions. This calculation determined that the flow distribution was essentially evenly divided between the two structures, with 53.66% allocated to the Low Sill and 46.34% to the Auxiliary. There was no need to cap any of the flows through the structures because of the reduced diversion through ORCC for Scenario 8A.

Diversion percentages for Scenario 8A (D8020) are shown in Table A-62 through Table A-65.

Figure A-93. Scenario 8A discharge relationship - ORCC and Union Point 1990–2019.

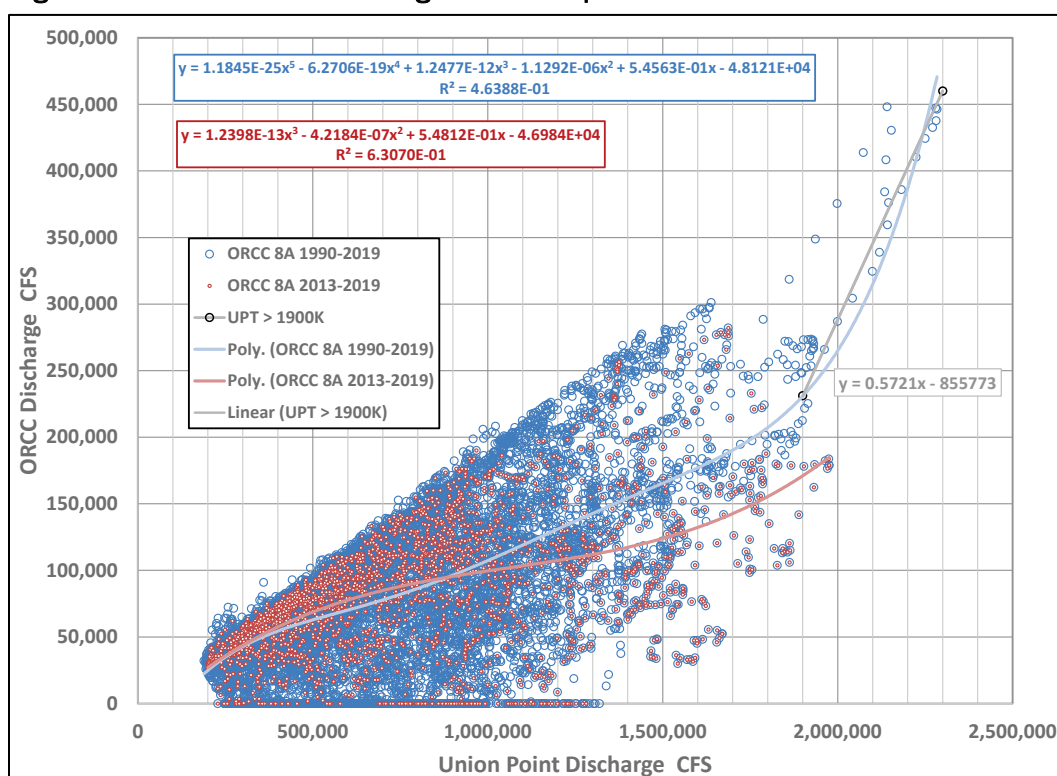


Figure A-94. Scenario 8A discharge relationship–Hydropower and Union Point 1990–2019.

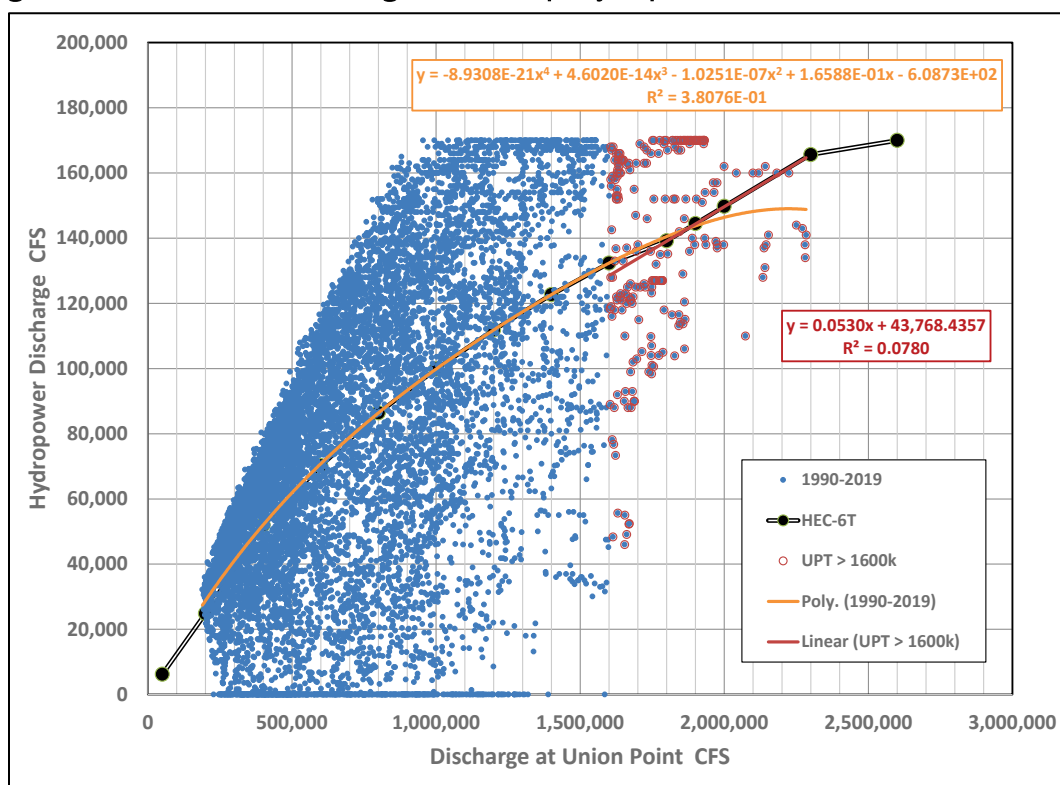


Figure A-95. Scenario 8A discharge relationship - Overbank and Union Point 2013–2019.

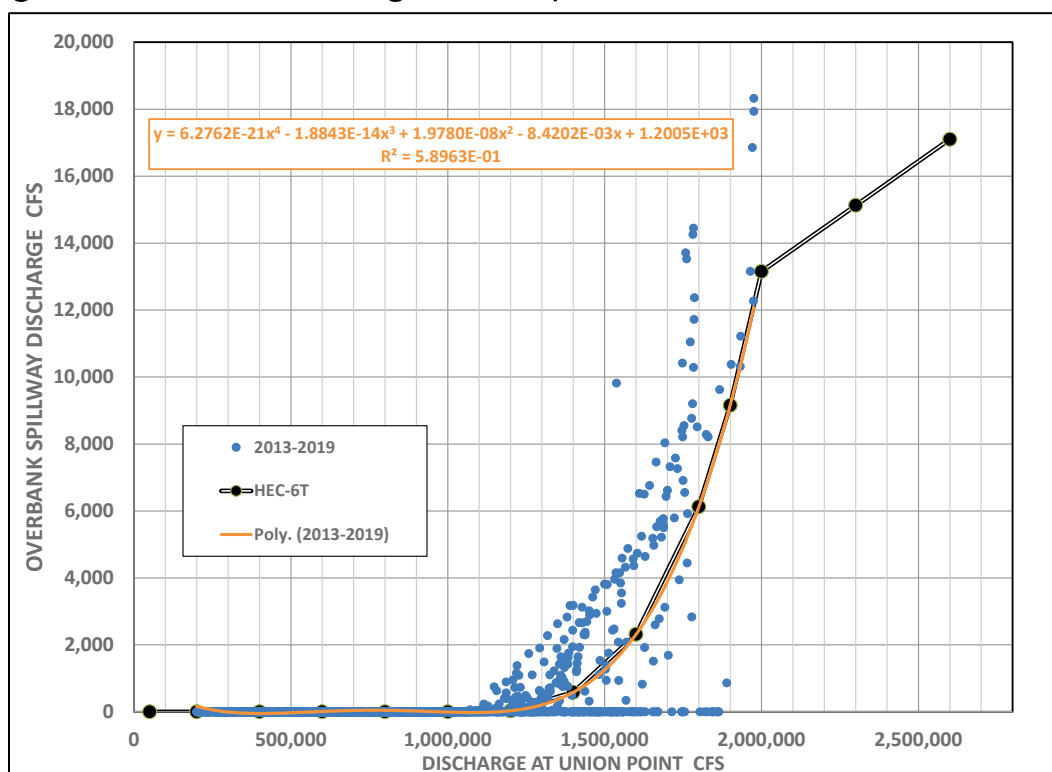


Table A-62. Scenario 8A (D8020) diversion percentages at Hydropower Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
12.43	100,000
12.43	200,000
13.62	400,000
10.80	800,000
9.32	1,200,000
7.73	1,800,000
7.49	2,000,000
7.20	2,300,000
6.54	2,600,000

Table A-63. Scenario 8A (D8020) diversion percentages at Overbank Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
0	1,200,000
0.04	1,400,000
0.14	1,600,000
0.34	1,800,000
0.66	2,000,000
0.66	2,300,000
0.66	2,600,000

Table A-64. Scenario 8A (D8020) diversion percentages at Low Sill Structure.

Percentage Diverted	Above Low Sill Discharge ft ³ /sec
0.0	530,000
0.01	714,000
1.31	1,277,000
2.02	1,655,000
3.67	1,837,000
7.07	2,119,000
7.40	2,413,000

Table A-65. Scenario 8A (D8020) diversion percentages at Auxiliary Structure.

Percentage Diverted	Above Low Sill Discharge ft ³ /sec
0.0	530,000
0.01	714,000
1.13	1,277,000
1.74	1,655,000
3.17	1,837,000
6.11	2,119,000
6.39	2,413,000

A.9 Scenario 8B - 80/20 Daily (Low Sill constraint) (D8020WLSC)

Scenario 8B decreases the daily latitude target discharge in the Atchafalaya River from 30% to 20% but is constrained if the head differential across the Low Sill Structure exceeds safety limits.

The assigned Scenario 8B daily data were used to develop regression curves to define the average ORCC and Union Point relationship and are shown in Figure A-96. A fifth-order polynomial regression was calculated in Excel from the 1990–2019 data. For reference purposes, a third-order polynomial regression was calculated from the 2013–2019 data. The 1990–2019 regression curve was adopted to take advantage of the larger data base and the inclusion of data at higher discharges. The adopted regression curve was modified slightly when Union Point discharges exceeded 1,900,000 cfs. A linear rating curve was found to fit the calculated data better. The range of data used to develop the regression curve was 200,000–2,300,000 cfs. In developing the HEC-6T rating curve, the percentage diverted through the ORCC from the Mississippi River was held at the high and low limits of data (no extrapolation).

The 1990–2019 Scenario 8B calculated daily discharges for the Hydropower and Union Point were used to develop regression curves to define the average diversion from the Mississippi River (Figure A-97). A fifth-order polynomial regression was calculated in Excel from the 26 May 1990–31 December 2019 data. This regression curve was used for the HEC-6T rating up to Union Point discharges of 2,000,000 cfs. A linear regression was also calculated for Union Point discharges greater than

1,600,000 cfs. The linear regression was used for reference in developing the HEC-6T rating curve at high Union Point discharges. The range of data used to develop the regression curve was between 200,000 and 2,300,000 cfs. In developing the HEC-6T rating curve, the discharge diverted through the Hydropower Structure from the Mississippi River was held to a maximum 170,000 cfs. At discharges less than 200,000 cfs, the diversion percentages was held constant at the data limit (no extrapolation).

Overbank Spillway diversion percentages, as a function of Union Point discharge, were determined from scenario allocations that had been calculated using historical 1 October 2012 through 30 September 2019 data as a reference. Overbank Structure diversion percentages for Scenario 8B were significantly less than those determined from the historical 1979–2019 relationships with Union Point discharges. Flow does not occur over the Overbank Spillway until Union Point discharge exceeds 1,200,000 cfs. A fourth-order polynomial regression equation defines the Overbank Spillway's discharge relationship to Union Point discharge when the Union Point discharge does not exceed 2,000,000 cfs. The calculated Overbank Spillway discharge was 89,000 cfs at this limit. In developing the HEC-6T rating curve, the percentage diverted through the Overbank Spillway from the Mississippi River was held constant when upstream flow exceeded 2,000,000 cfs (no extrapolation). The Overbank Spillway versus Union Point relationship is shown in Figure A-98.

Diversion percentages through the Low Sill and Auxiliary Structures were calculated using daily discharges between 2013 and 2019. The discharge through the combined Low Sill and Auxiliary Structures was calculated by subtracting Hydropower and Overbank Spillway regression-equation discharges from ORCC regression-equation discharges. The flow distribution between the Low Sill and Auxiliary Structures was determined by calculating the long-term (2013–2019) average distributions. This calculation determined that the flow distribution was essentially evenly divided between the two structures, with 50.40% allocated to the Low Sill and 49.60% to the Auxiliary. There was no need to cap any of the flows through the structures because of the reduced diversion through ORCC for Scenario 8B.

Diversion percentages for Scenario 8B (D8029WLSC) are shown in Table A-66 through Table A-69.

Figure A-96. Scenario 8B discharge relationship ORCC and Union Point 1990-2019.

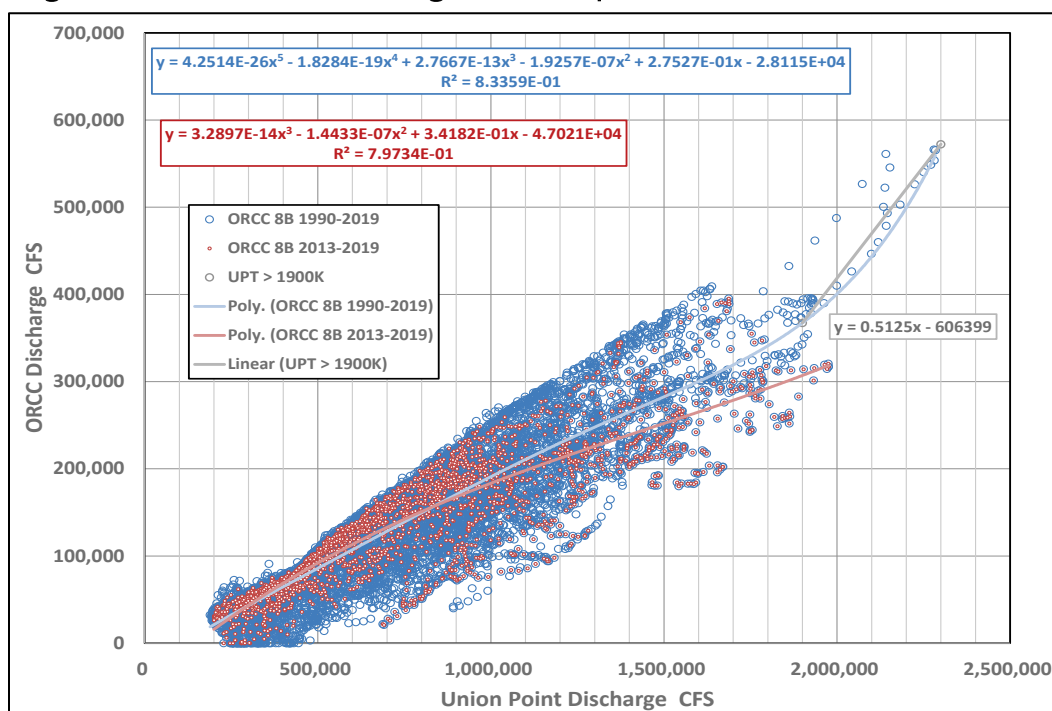


Figure A-97. Scenario 8B discharge relationship Hydropower and Union Point 1990-2019.

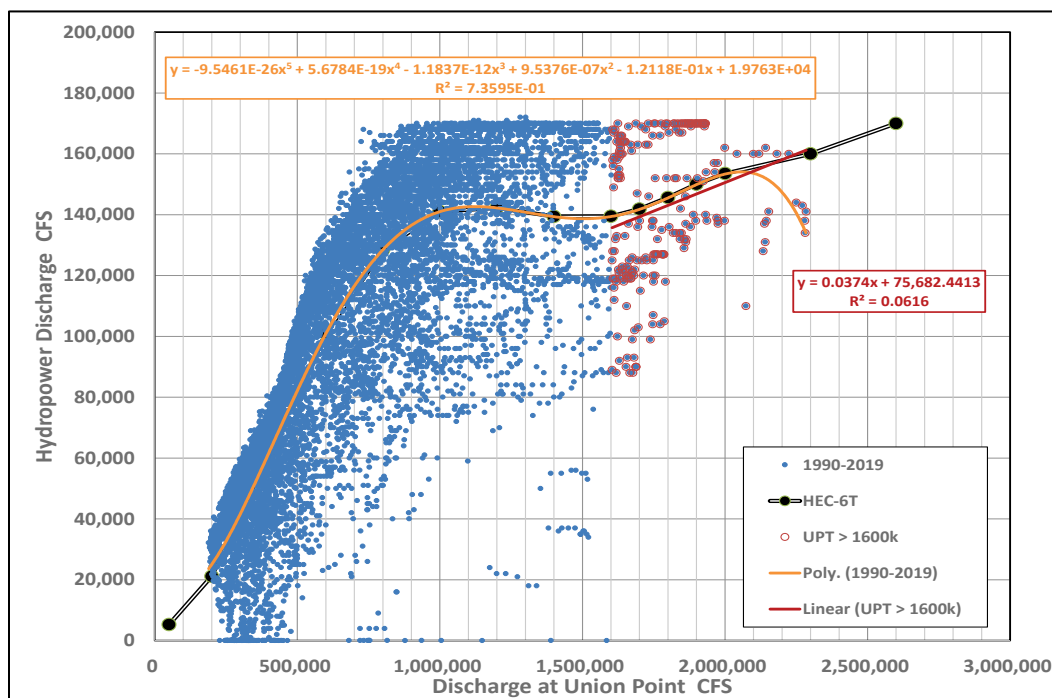


Figure A-98. Scenario 8B discharge relationship Overbank and Union Point 2013–2019.

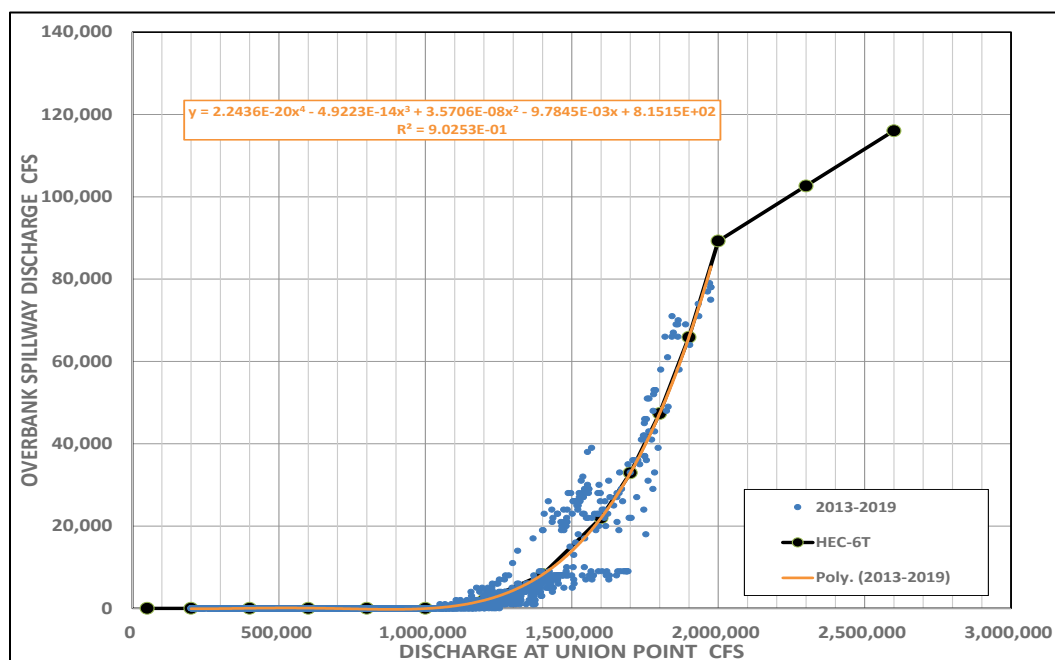


Table A-66. Scenario 8B (D8020WLSC) diversion percentages at Hydropower Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
10.59	100,000
10.59	200,000
15.42	400,000
16.82	600,000
16.06	800,000
9.96	1,400,000
8.72	1,600,000
7.68	2,000,000
6.54	2,600,000

Table A-67. Scenario 8B (D8020WLSC) diversion percentages at Overbank Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
0	1,100,000
0.59	1,400,000
1.37	1,600,000
2.63	1,800,000
4.46	2,000,000
4.46	2,600,000

Table A-68. Scenario 8B (D8020WLSC) diversion percentages at Low Sill Structure.

Percentage Diverted	Above Low Sill Discharge ft ³ /sec
0.0	179,000
0.44	338,000
0.63	499,000
4.07	1,058,000
4.70	1,261,000
4.78	1,461,000
4.36	1,750,000
4.80	1,846,000
7.29	2,140,000
7.49	2,430,000

Table A-69. Scenario 8B (D8020WLSC) diversion percentages at Auxiliary Structure.

Percentage Diverted	Above Low Sill Discharge ft ³ /sec
0.0	179,000
0.43	338,000
0.62	499,000
4.01	1,058,000
4.63	1,261,000
4.71	1,461,000
4.29	1,750,000
4.72	1,846,000
7.18	2,140,000
7.37	2,430,000

A.10 Scenario 9 – 60–40 Daily (D6040).

Scenario 9 increases the daily latitude target discharge in the Atchafalaya River from 30% to 40% on a daily basis. The assigned Scenario 9 daily data were used to develop regression curves to define the average ORCC and Union Point relationship and are shown in Figure A-99. A fifth-order polynomial regression equation was used to calculate diversions from the Mississippi River into the Outflow Channel for the HEC-6T simulation. The 1990–2019, fifth-order polynomial regression curve shown in Figure A-99 was adopted for use in this study. The maximum diversion discharge through the ORCC was set at 970,000 cfs; this occurs at a Union Point discharge of 2,300,000 cfs.

Scenario 9 gives the Hydropower Structure priority usage of the diverted discharge. Hydropower was assigned a maximum discharge of 170,000 cfs. This generally occurs when Union Point discharge exceeds 650,000 cfs. If the discharge at Tarbert Landing is greater than 425,000 cfs, and the Scenario 9 allocation for the ORCC is less than 170,000 cfs, then all flow is diverted through the Hydropower Structure. When the Tarbert Landing discharge is less than 425,000 cfs, the Hydropower allocation is capped at 40% of the Tarbert Landing discharge. The average 1990–2019 relationship between Scenario 9 Hydropower Structure discharges and historical Union Point discharges,

is shown in Figure A-100. A linear regression defines the relationship when Union Point discharges are less than 650,000 cfs.

HEC-6T Scenario 9 diversion percentages for the Overbank Spillway were determined from 2013–2019 data when the discharge at Union Point was less than 2,000,000 cfs. The 2,000,000 cfs upper limit was employed because that is within the limits of measured data. Of note, these discharges are higher than those designated in the Scenario 9 hindcasting spreadsheet. When the Union Point discharge exceeded 2,200,000 cfs, the HEC-6T Overbank Spillway was operated at capacity, which is 320,000 cfs. A fourth-order polynomial relationship between Overbank Spillway diversion discharge and Union Point discharge is shown in Figure A-101. A polynomial regression of the 1990–2019 overbank structure data is shown for reference.

Diversion percentages through the Low Sill and Auxiliary Structures were calculated using daily discharges between 2013 and 2019. The discharge through the combined Low Sill and Auxiliary Structures was calculated by subtracting Hydropower and Overbank Spillway regression-equation discharges from ORCC regression-equation discharges. Using the Scenario 9 data for Union Point discharges above 2,300,000 cfs, the combined Low Sill and Auxiliary discharge was held constant at 480,000 cfs. This assignment is consistent with assignments in *OMAR_AllScenarios_20210106*. Years 2013–2019 average daily discharges for the Low Sill and Auxiliary Structures were summed, and it was determined that 57.14% of the flow went through the Low Sill and 42.86% went through the Auxiliary. These fractions were used to determine each structures diversion percentages.

Diversion percentages for Scenario 9 (D6040) are shown in Table A-70 through Table A-73.

Figure A-99. Scenario 9 discharge relationship ORCC and Union Point 1990–2019.

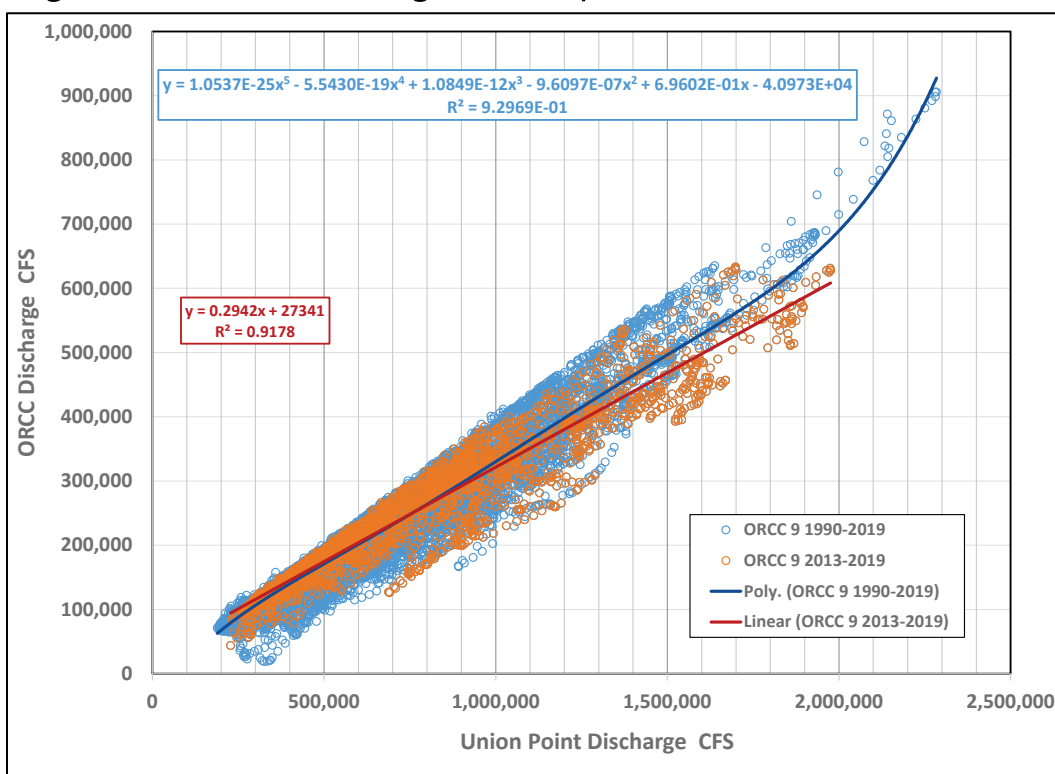


Figure A-100. Scenario 9 discharge relationship Hydropower and Union Point 1990–2019.

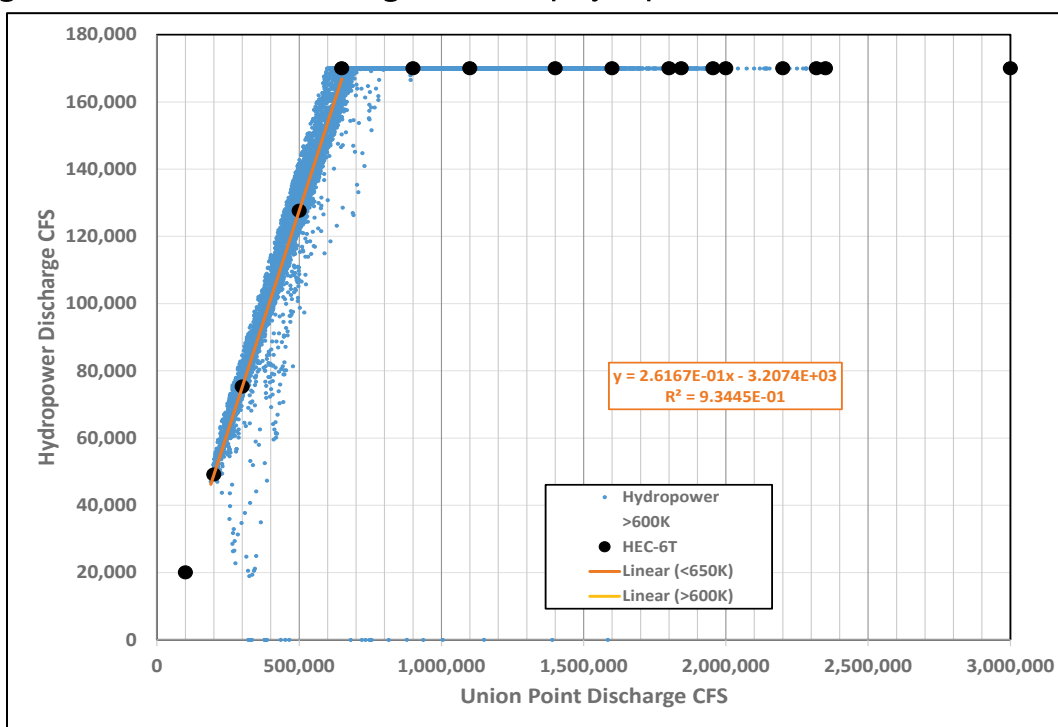


Figure A-101. Scenario 9 discharge relationship Overbank and Union Point.

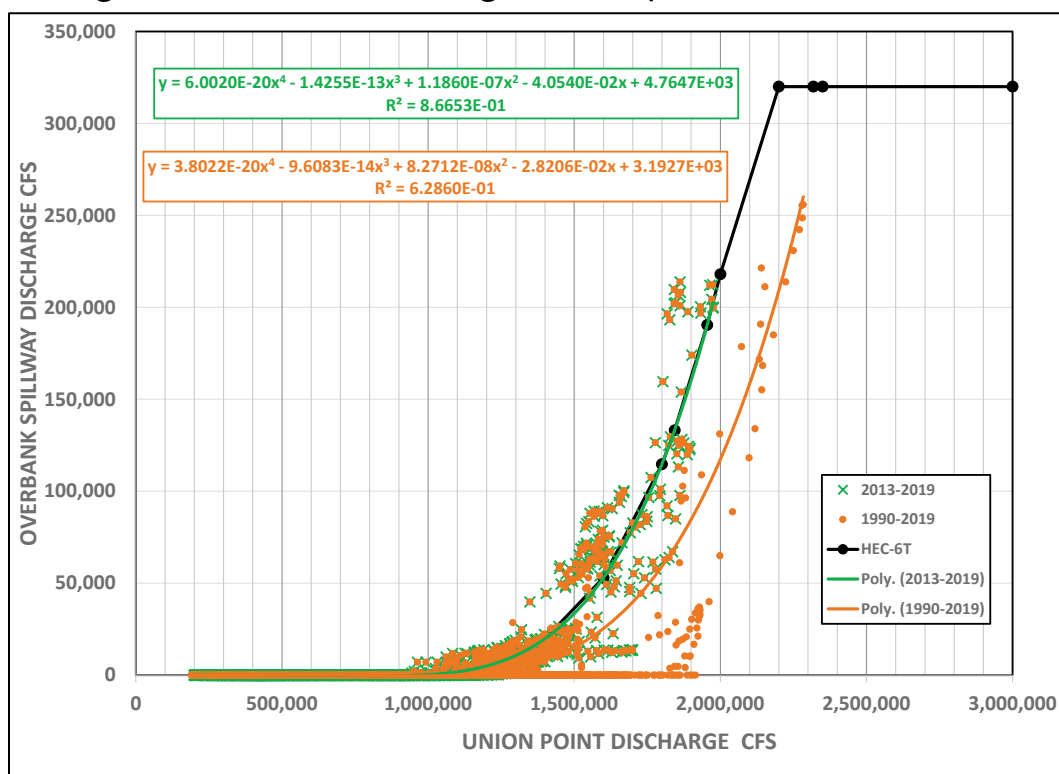


Table A-70. Scenario 9 (D60/40) diversion percentages at Hydropower Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
20.05	100,000
24.56	200,000
26.15	650,000
18.89	900,000
15.45	1,100,000
12.14	1,400,000
10.63	1,600,000
9.44	1,800,000
7.33	2,318,000
5.67	3,000,000

**Table A-71. Scenario 9 (D60/40) diversion percentages
at Overbank Structure.**

Percentage Diverted	Union Point Discharge ft ³ /sec
0	900,000
0.17	1,100,000
1.42	1,400,000
3.31	1,600,000
6.38	1,800,000
10.90	2,000,000
14.55	2,200,000
13.81	2,318,000
10.67	3,000,000

**Table A-72. Scenario 9 (D60/40) diversion percentages
at Low Sill Structure.**

Percentage Diverted	Above Low Sill Discharge ft ³ /sec
0.0	80,000
7.00	151,000
7.91	225,000
5.55	480,000
11.83	928,000
12.95	1,210,000
12.68	1,377,000
10.68	1,612,000
16.41	1,828,000
11.95	2,510,000

Table A-73. Scenario 9 (D60/40) diversion percentages at Auxiliary Structure.

Percentage Diverted	Above Low Sill Discharge ft ³ /sec
0.0	80,000
5.25	151,000
5.93	225,000
4.17	480,000
8.88	928,000
9.72	1,210,000
9.51	1,377,000
8.01	1,612,000
9.85	1,828,000
7.17	2,510,000

A.11 Scenario 10 – Maximize Auxiliary (MAXAUX)

The objective of this scenario is to flush as much as possible through the Auxiliary Structure while still maintaining the overall 70/30 latitude flow. Changes from the base condition occur only at low flow (Knox Landing below 40 ft or Tarbert Landing less than 500,000 cfs). This limit is required to prevent bank erosion in the Auxiliary channel. Further, there are no adjustments to base flow percentages during the months of April, May, or June because of fish spawning. If ORCC flow is less than 20,000 cfs then all the flow goes to the Hydropower Structure. Therefore, if the flow at Tarbert Landing is less than 500,000 cfs and it is not April, May, or June, then hydropower gets one-third of the ORCC allocation and the Auxiliary Structure gets two thirds. At high flows, Scenario 10 is the same as the base.

The total ORCC diverted discharge, as a function of Union Point discharge, was determined using 1990–2019 regression equations. A fourth-order polynomial regression equation, shown in Figure A-102, was used when the discharge at Union Point was less than 2,000,000 cfs, and a linear equation was used when the Union Point discharge exceeded 2,000,000 cfs.

Hydropower versus Union Point is defined by a fifth-order polynomial regression and is shown in Figure A-103. The Scenario 10 hydropower allocation is set at one-third of the ORCC flow when Tarbert Landing

discharge is less than 500,000 cfs. When Union Point discharge exceeds 2,000,000 cfs, Hydropower discharge is set at a maximum of 160,000 cfs.

Overbank Spillway diversion percentages, as a function of Union Point discharge, were determined from scenario allocations that had been calculated using historical 1 October 2012 through 30 September 2019 data as a reference. Flow does not occur over the Overbank Spillway until Union Point discharge exceeds 1,100,000 cfs. A fourth-order polynomial regression equation defines the Overbank Spillway's discharge relationship to Union Point discharge when the Union Point discharge does not exceed 2,000,000 cfs. The calculated Overbank Spillway discharge was 204,000 cfs at this limit. The Overbank Spillway discharge was capped at 320,000 cfs when the Union Point discharge exceeded 2,300,000 cfs. The Overbank Spillway versus Union Point relationship is shown in Figure A-104.

Diversion percentages through the Low Sill and Auxiliary Structures were calculated using assigned daily discharges between 2013 and 2019. The discharge through the combined Low Sill and Auxiliary Structures was calculated by subtracting Hydropower and Overbank Spillway regression-equation discharges from ORCC regression-equation discharges. When the discharge at Tarbert Landing was less than 500,000 cfs, the Scenario 10 allocation called for two-thirds of the ORCC flow to be assigned to the Auxiliary Structure. At higher discharges, the flow distribution between the Low Sill and Auxiliary Structures was determined by calculating the long-term (2013–2019) average distributions using only days when the flow at Tarbert Landing exceeded 500,000 cfs. This calculation determined that the flow distribution was evenly divided between the two structures. There was no need to apply capacity limits to the Low Sill nor the Auxiliary Structure with Scenario 10.

Diversion percentages for Scenario 10 are shown in Table A-74 through Table A-77.

Figure A-102. Scenario 10 discharge relationship ORCC and Union Point
1990–1991.

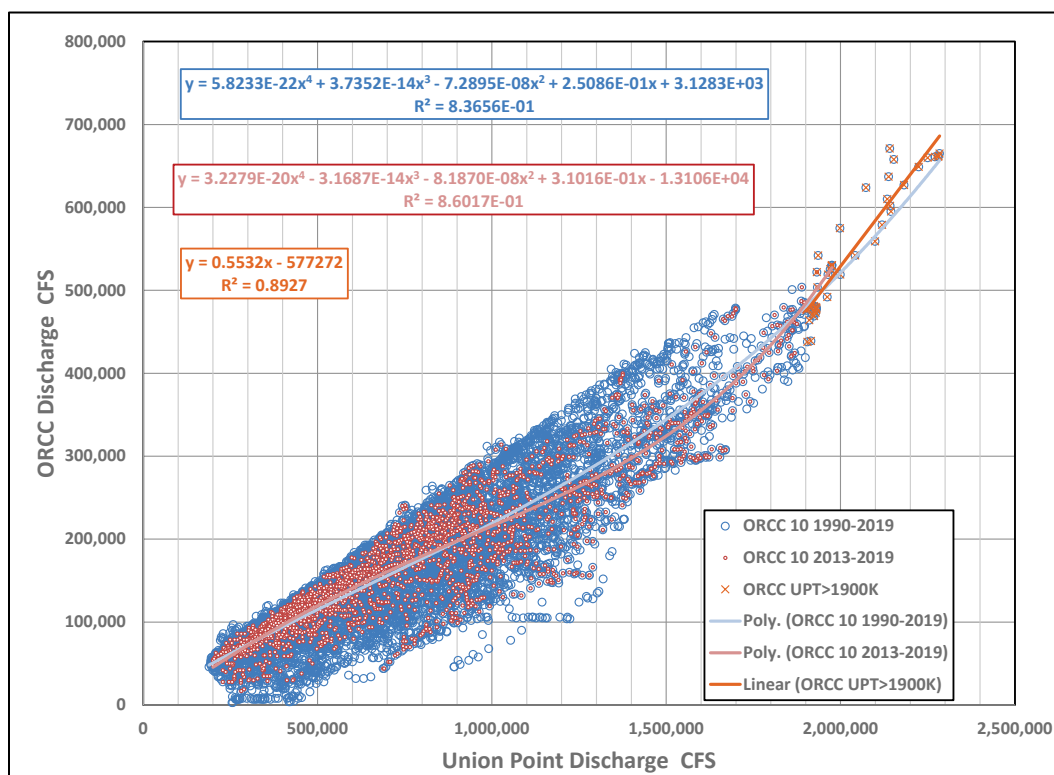


Figure A-103. Scenario 10 discharge relationship Hydropower and Union Point.

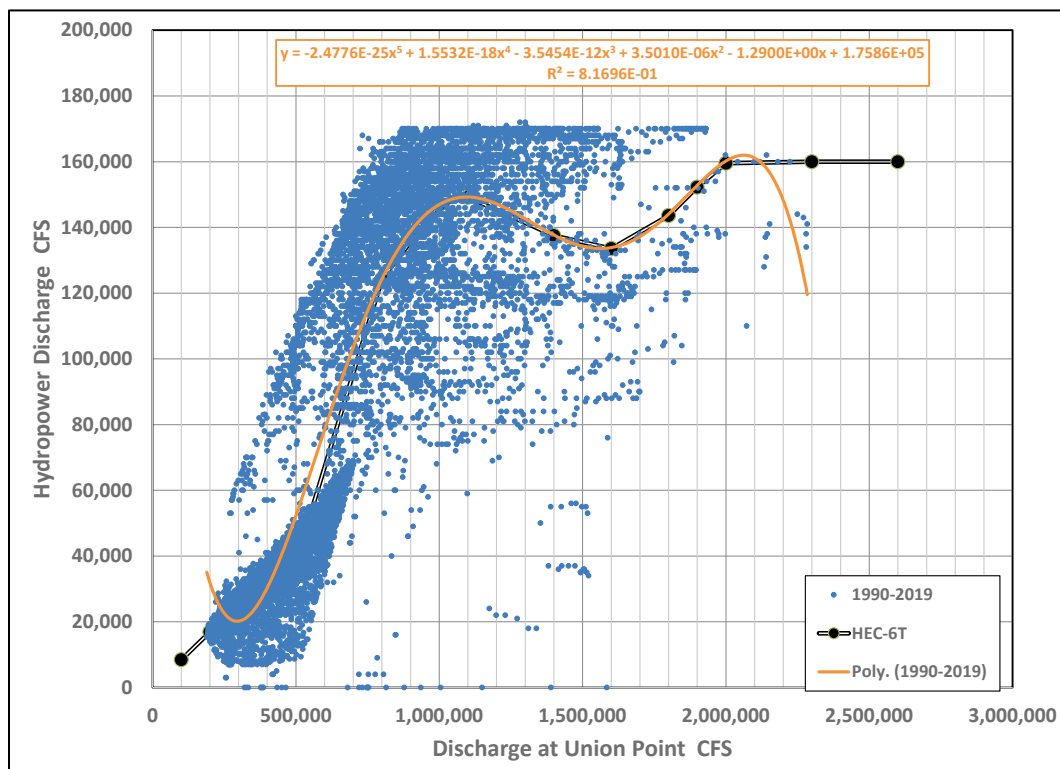


Figure A-104. Scenario 10 discharge relationship Overbank Spillway and Union Point 2013–2019.

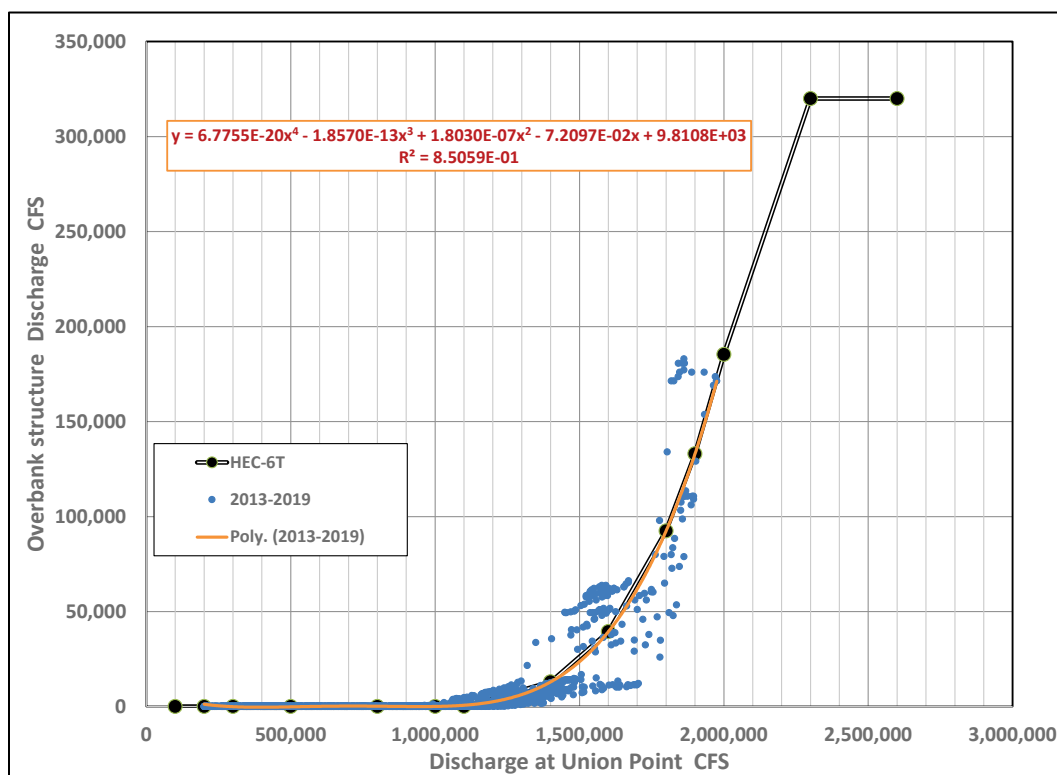


Table A-74. Scenario 10 (MAXAUX) diversion percentages at Hydropower Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
8.45	100,000
8.45	200,000
7.67	500,000
15.53	800,000
14.69	1,000,000
13.56	1,100,000
9.82	1,400,000
8.35	1,600,000
7.98	2,000,000
6.15	2,600,000

Table A-75. Scenario 10 (MAXAUX) diversion percentages at Overbank Structure.

Percentage Diverted	Union Point Discharge ft ³ /sec
0	1,100,000
0.93	1,400,000
2.46	1,600,000
5.14	1,800,000
7.00	1,900,000
9.26	2,000,000
13.91	2,300,000
12.31	2,600,000

Table A-76. Scenario 10 (MAXAUX) diversion percentages at Low Sill Structure.

Percentage Diverted	Above Low Sill Discharge ft ³ /sec
0.0	92,000
0.0	183,000
0.0	462,000
3.87	676,000
4.23	853,000
6.63	1,250,000
7.07	1,427,000
6.60	1,564,000
5.33	1,655,000
7.21	2,120,000

Table A-77. Scenario 10 (MAXAUX) diversion percentages
at Auxiliary Structure.

Percentage Diverted	Above Low Sill Discharge ft ³ /sec
18.45	92,000
18.45	183,000
16.61	462,000
3.87	676,000
4.23	853,000
6.63	1,250,000
7.07	1,427,000
6.60	1,564,000
5.33	1,655,000
7.21	2,120,000

Appendix B: Recommendations

The following recommendations were provided by William A. Thomas as part of his peer review of this document. Mr. Thomas is a world-renowned expert in river engineering, sediment engineering, hydraulics, and computer programming. He conceived, designed, and coded the HEC-6T sediment transport model. He served for 32 yr with the USACE and currently is owner and chief engineer of Mobile Boundary Hydraulics engineering firm. Mr. Thomas's vast experience and renowned reputation in the field of sedimentation engineering make his comments especially significant and useful to engineers conducting future studies.

Mr. Thomas concluded that it does not seem likely that improvements in the inflowing sediment load that are recommended below will affect the rankings of scenarios in this assessment. However, these recommendations should be considered when the model is used for overall management of the Mississippi River.

B.1 Specific recommendations related to sediment inflow

B.1.1 Middle Mississippi River

Analysis. The boundary condition for the Middle Mississippi River is located at Chester, IL (RM 109.9). Suspended sediment was measured at Thebes, IL (RM 43.7) during the time period 1973 to 2019 and at Chester between 1980 and 1991. The concentration of sand in the sample, called the fines/sand break, is available. A regression curve was fitted through each population, and the unbiased estimator was computed. No bed load measurements were available. The unmeasured load, including bed load was calculated using the re-circulation option in HEC-6T. These calculations should be checked using currently available bed load measurement technologies.

The next step in the inflowing sediment load analyses is the particle size gradations. Gradation refers to the fraction of particle sizes in the samples by weight. The only available gradation measurements for the suspended sand and fine load from the Mississippi River were made at Thebes and Chester between 1980 and 1991. Those data should be updated for future studies.

Collecting no data for gradation analysis means the user assumes historical gradations continue into the future without change. Such is true only for the equilibrium condition, and the Mississippi River is not in equilibrium. NOAA recorded five major floods between 1983 and 2019. They occurred in 1983, 1997, 2002, 2008, and 2011. In addition, there was the 1993 flood, which came largely from the sediment rich waters of the Missouri River. Major floods are episodic events. They interrupt the continuum of the normal hydrology and change the sedimentation processes along the river.

Even though the available sediment data for the Middle Mississippi River contains uncertainties that need to be eliminated, Mr. Thomas concurred that the sediment inflow is adequate for this OMAR Assessment because the purpose of this OMAR Assessment is to rank different scenarios for operating the ORCC. Improving the sediment data as recommended below will probably change the magnitudes of the metrics that were used in this study, but I do not believe it will change the ranking of scenarios.

Recommendations for this gage. Mr. Thomas proposed six recommendations for modifying the sediment data collection:

1. Select either Chester or Thebes to be the inflow gage/gages for the MR&T HEC-6T sediment model. Make all sediment measurements from the Middle Mississippi at the selected gage/gages. If Chester is selected, allowance for the Big Muddy River will be required.
2. Measure the bed load with the ISSDOTv2 procedure and assume the fraction of the sediment load that moves between the bed load and the suspended sediment sampler. For depth-integrated samples, the lowest half foot of the water column is negligible.
3. The unmeasured load includes two types of sediment transportation: (a) sediment particles in suspension, which pass beneath the nozzle of a suspended sediment sampler, and (b) sand, gravel and larger particles that actually move as bed load. At Thebes, the bed load should be measured.
4. Do not assume that water discharge is the best parameter for presenting the measured sediment concentrations. Experiment with other hydraulic parameters like water velocity or stream power. Choose the one that minimizes the scatter in the measured bed-material load. Plot the bed load measurements on the same relationship as the suspended bed material Load. Plot the wash load on the same relationship, but wash load

does not correlate well with hydraulic parameters. Add measurements to these relationships as one would add discharges to a stream flow gage.

Use the same parameters to relate sediment inflow from the Ohio and the other tributaries in the MR&T model.

5. Compute the total inflowing sediment load by adding the bed load to the suspended load.
6. Modify the sediment data collection program to include the gradation of the suspended sediment load and the gradation of the bed load in the analysis of samples. Use these fractions to develop the inflowing sediment load table for sediments entering the MR&T from the Middle Mississippi River.
7. Add the necessary cross sections and collect samples of the riverbed sediment to determine the gradation profile of the bed but continue to use the Middle Mississippi River just to deliver the inflowing sediment load to the Lower Mississippi River.

B.1.2 Ohio River

Analysis. Water and sediment from the Ohio River enter the model at Olmsted Dam. Suspended Sediment Concentrations, and the sand components, were measured at Dam 53 from 1974 to 2014. The gage was moved to Olmsted Lock and Dam in December 2014. However, gradation measurements of the suspended load came from Louisville, Kentucky (Ohio RM 607.3). Louisville is 372 mi upriver from the end of the OMAR Model.

Recommendations for the Ohio River at Olmsted Dam. For future studies, Mr. Thomas recommended that the sediment data collection program for the Ohio River be modified as discussed above for the Middle Mississippi River. Using the suspended sand size class data from Louisville is highly speculative. Suspended sand class data should be determined at Olmstead.

B.1.3 Obion River and Hatchie River

Recommendations for Obion River and Hatchie River. For future studies, Mr. Thomas recommended the following:

1. Establish permanent gages for the water-sediment mixtures from these rivers. These sites should be upstream from any Mississippi River backwater influence. If feasible, the historical gage locations would be

best. Replace any hypothetical cross sections with actual cross sections of these rivers and extend the HEC-6T model limits to the gage sites. Collect bed material samples between the Mississippi River and the upstream limit of the tributary segment to establish a reliable bed gradation along the Segments.

2. Establish a sediment data collection program on each river to measure the concentration of the suspended load. The sand/fines break and the gradation of the suspended sediment load, including silts and clays, should be determined by laboratory analysis
3. Use the re-circulation option in HEC-6T to compute the unmeasured load.
4. Continue using The Obion and Hatchie Rivers to deliver the sediment loads to the Mississippi River. Do not refine the tributary models to study the morphology of the tributary. The tributary models should not control the computational time steps in the Mississippi River models.

B.1.4 Saint Francis River

Analysis. Water and sediment from the Saint Francis River enter the OMAR Model at Madison, Arkansas (Saint Francis RM 37.9). Suspended sediment concentrations and the gradations of those concentrations were measured for the time period 1977 to 2009. The bed load of the Saint Francis River was not measured. To account for bed load, the suspended sediment concentrations were increased by 15%.

Recommendations for the Saint Francis River. Mr. Thomas made the following recommendations for future studies:

1. Reinstate the suspended sediment measurement program at Madison. Determine the fines/sands break as in the past, but also analyze the suspended sediment samples to determine the gradation of the suspended sediment load including the fines.
2. Extend the HEC-6T geometry to Madison using measured cross sections of the Saint Francis River.
3. Collect samples of the bed sediment so a reliable gradation of the bed sediment mixture can be coded into the model.
4. Continue using this segment to deliver the sediment loads to the Mississippi River. Do not refine it to study the morphology of the tributary.
5. Use the re-circulation option in HEC-6T to compute the unmeasured load.

B.1.5 White River

Analysis. Water and sediment from the White River enter the OMAR Model at Clarendon, Arkansas (RM 100). The inflowing sediment load was based on sediment measurements that were made between 2001 and 2018 at DeValls Bluff, AR (RM 121.8). The samples were not analyzed for gradation of the suspended load. Therefore, the gradations at Thebes, IL, and at Chester, IL, were used for the White River. This approach does not provide a reliable estimate. Rather, it was an option for filling a data gap.

Recommendations for the White River. Mr. Thomas made the following recommendations for future studies. Select a permanent gage site for upstream boundary of the White River. Continue measuring the suspended sediment concentrations at that gage. Continue to determine the fines/sands break as in the past. In addition, add the following measurements:

1. Determine the gradation of the suspended sediment load including silts and clays.
2. Add new cross sections and replace the hypothetical cross sections with actual White River cross section spaced to avoid short reach lengths. Continue using this segment to deliver the sediment loads to the Mississippi River.
3. Take sufficient samples of the bed material along the White River segment to establish the gradation of the bed sediment reservoir, which was assumed to be zero in the current HEC-6T model.
4. Use the re-circulation option in HEC-6T to compute the unmeasured load on the White River as recommended above for the Saint Francis River.

B.1.6 Arkansas River

Analysis. Water and sediment from the Arkansas River enter the OMAR Model at Lock & Dam Number 2 (Arkansas RM ~ 47). However, in this assessment, sediment data 100 mi upstream at David D. Terry Lock and Dam were used to determine the sediment inflow. The suspended sediment concentrations have been measured there since 1994. The sand component is available, but gradation data are missing. The bed load is not measured.

Mr. Thomas concurred that the sediment load curves are probably adequate for the OMAR Assessment. However, he made one suggestion. Consider making a sensitivity test. Double the concentrations in the inflowing sediment load from the Arkansas River. Run the OMAR Model Base Condition. If the Base Condition values do not change significantly at Union Point, we can be assured the refinement to the sediment load that is recommended below will not affect the ranking of scenarios.

Recommendations for the Arkansas River. For future studies, Mr. Thomas made the following recommendations. Select a permanent gage site for the upstream boundary. Continue measuring the suspended sediment concentrations at that gage. Continue to determine the fines/sands break as in the past. In addition, add the following measurements:

1. Determine the gradation of the suspended samples including the fines.
2. If David D. Terry is adopted for the sediment gage, the geometry of the Arkansas River from L&D 2 to David D. Terry L&D should be added to the segment.
3. Note: Flow of the water-sediment mixture down the Arkansas River is complex. It passes through four navigation dams. In addition, sediment must pass through the backwater curve to enter the Mississippi River. The phasing of delivery will be improved if the geometry of the segment is used in the delivery.
4. Take sufficient samples of the bed material along the segment to establish the gradation of the bed sediment reservoir. However, continue using this segment to deliver the sediment load from the Arkansas River to the Mississippi River. Do not make refinements that will control the computational time step for the Mississippi River model.
5. Use the re-circulation option in HEC-6T to compute the unmeasured load as recommended above for the Saint Francis River.
6. Whether the gage remains at L&D 2 or is moved to David D. Terry, compute the unmeasured load with the re-circulation option in HEC-6T as recommended above for the other tributaries.

B.1.7 Yazoo River

Analysis. Water and sediment from the Yazoo River enter the model at Redwood, MS (RM 16.7). The suspended sediment concentrations and the sand component have been measured at the USGS gage Yazoo River below

Steele Bayou near Long Lake, MS (07288955) (RM 9.5) since 1999. The unmeasured load was estimated by adding 15% to the sand concentration. The gradations of the suspended sand concentrations are missing. The gradation of the bed material for the 16.7 mi length of the Yazoo River segment is inadequate. It is based on only two samples taken at the Steel Bayou Gage. Consequently, the gradation of the suspended sediment load was computed with data from the Mississippi River at the Coochie gage and the Union Point gage. Mr. Thomas concurred with the assumptions that were used to fill in the missing data as recorded in the OMAR report.

Recommendations for the Yazoo River. The only way to eliminate the uncertainties in the present inflowing sediment load is to improve the sediment data collection program. For future studies, Mr. Thomas recommended that the upstream boundary be located at the inflowing gage or an upstream gage not influenced by the Mississippi River backwater.

1. Continue to measure the total suspended sediment concentrations and continue to determine the sand/fines break in the samples.
2. Add a gradation analysis of the suspended sediment samples including the fines as described above for the other tributaries.
3. Take sufficient samples of the bed material along the segment to establish the gradation of the bed sediment reservoir. However, continue using this segment to deliver the sediment load from the Yazoo River to the Mississippi River. Do not make refinements that will control the computational time-step for the Mississippi River model.
4. Use the re-circulation option in HEC-6T to compute the unmeasured load.

B.2 Recommendations for the main-stem gages for future studies

The main-stem gages refer to Memphis, Greenville, Vicksburg, and Natchez. The sediment data collection programs for the main-stem gages should be standardized for the MR&T. The same equipment and procedures should be used at all sites. The laboratory analysis should provide the same information at all gages. The suspended sediment measurements should include the total concentration, the sands/fines break and the gradation of the samples. Mr. Thomas recommended that enough bed load measurements be made over time to plot the relationship

between flow, suspended bed-material load, bed load, and wash-load discharges as described above for the gage at Thebes.

Enough measurements should be made over time to give the engineer confidence that the sediment load and the fraction of each particle size class is known. Also, enough samples of the wash load should be analyzed to give confidence in the gradation of the silt and clay sizes. Those components of the sediment load become bed load in the diversions and even in the main river at Venice.

B.3 Recommendations for diversions

Continue collecting field data on the fraction of water that is diverted. Maintain records at both controlled and uncontrolled diversion sites. Collect the sediment concentrations that are diverted with the water and compute the diverted coefficients as a fraction of the approaching concentrations in the river. Compute the diversion coefficients by particle size class.

B.4 Recommendation for the geometry for future studies

The HEC-6T model created for the lower 1000 mi of the Mississippi River is remarkable. It will provide good insight into the questions being asked. Mr. Thomas believes its use could even be expanded for future studies if it were upgraded from 3-strips to 5-strips. The upgrade would allow the sediment computations to follow the procedures in ASCE Manuals more closely by separating the channel bed from the channel banks. Such is common practice in sediment engineering. Bed roughness predictors can then be applied, and the change in water surface elevation that results from changing the bed gradation over time can be predicted. Figure 35, which compares the 1932 and 1989 median bed material size along the Mississippi River, shows that the bed gradation did change over the 57 yr between surveys. HEC-6T will use the new gradations to compute new n -Values for the bed of the channel. A 5-strip model will also allow the bank roughness to be changed from natural bank values to a bank revetment values or other constructed features during the computations.

B.5 Recommendations for the hydrographs in future studies

Mr. Thomas suggested that the channel forming water discharge (CFD) be developed along the entire Mississippi River. Embed the event as one of the standard tests for trends in future studies. Include it with the specific gage plots. If the CDF discharges were run for an infinitely long period of the time, the model would predict the water surface profile, the bed surface profile, and the delivery of sediment in the prototype trending toward the equilibrium condition.

Abbreviations

1D	One-dimensional
CFD	Channel forming water discharge
CS	Coarse sand
ERDC	US Army Engineer Research and Development Center
FS	Fine sand
MBH	Mobile Boundary Hydraulics
MR&T	Mississippi River and Tributaries
MS	Medium sand
MVD	Mississippi Valley Division
MVM	Memphis District
MVN	New Orleans District
NOAA	National Oceanic and Atmospheric Administration
OMAR	Old, Mississippi, Atchafalaya, and Red Rivers
ORCC	Old River Control Complex
ORCS	Old River Control Structure
ORO	Outflow Channel Gaging Station
PDF	Project Design Flood
RM	River Mile
UPT	Union Point
USACE	US Army Corps of Engineers
USGS	United States Geological Survey
VFS	Very fine sand

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