



**US Army Corps
of Engineers®**
Engineer Research and
Development Center



*Regional Sediment Management: Integrated Solutions for Sediment
Related Challenges*

Evaluation of Structural and Operational Alternatives to Optimize the Distribution of Water and Sediment in the Passes of the Mississippi River

Ronald R. Copeland, Michael A. Hartman, and James Lewis

August 2023



The US Army Engineer Research and Development Center (ERDC) solves the nation's toughest engineering and environmental challenges. ERDC develops innovative solutions in civil and military engineering, geospatial sciences, water resources, and environmental sciences for the Army, the Department of Defense, civilian agencies, and our nation's public good. Find out more at www.erdclibrary.on.worldcat.org/discovery.

To search for other technical reports published by ERDC, visit the ERDC online library at www.erdclibrary.on.worldcat.org/discovery.

Photo taken 29 August 2019. Patrick M. Quigley, www.gulfcoastairphoto.com. Sidell, Louisiana. 985-788-3458. An SDAV-owned small business.

Evaluation of Structural and Operational Alternatives to Optimize the Distribution of Water and Sediment in the Passes of the Mississippi River

Ronald R. Copeland, Michael A. Hartman, and James Lewis

*US Army Engineer Research and Development Center (ERDC)
Coastal and Hydraulics Laboratory (CHL)
3909 Halls Ferry Road
Vicksburg, MS 39180-6199*

Final report

Distribution Statement A. Approved for public release; distribution is unlimited.

Prepared for US Army Engineer Research and Development Center
3909 Halls Ferry Rd
Vicksburg, MS 39180-6199

Under Funding Account Code 121BG5; AMSCO Code 08303

Abstract

Mississippi River shoaling and dredging processes in the vicinity of Head of Passes and in Southwest Pass were investigated. Existing rates of deposition and dredging were determined using near-daily eHydro bathymetric surveys, National Dredging Quality Management dredge operating data, and geospatial processing steps developed for this study. These surveys provide a means to characterize the highly dynamic and variable sedimentation patterns observed in the navigation channel. The HEC-6T one-dimensional numerical sedimentation model was used to evaluate possible modifications to the distribution of water and sediment in the Mississippi River near Head of Passes in an attempt to reduce shoaling in the navigation channel. The model was used to evaluate the effects of partial closures of several distributaries downstream from Venice and to evaluate the effects of channel widening and channel deepening adjacent to the Hopper Dredge Disposal Area at Head of Passes. In this study, various structural alternatives were compared to a base test that represented existing conditions. Sedimentation and dredging effects were projected 50 years into the future.

DISCLAIMER: The contents of this report are not to be used for advertising, publication, or promotional purposes. Citation of trade names does not constitute an official endorsement or approval of the use of such commercial products. All product names and trademarks cited are the property of their respective owners. The findings of this report are not to be construed as an official Department of the Army position unless so designated by other authorized documents.

DESTROY THIS REPORT WHEN NO LONGER NEEDED. DO NOT RETURN IT TO THE ORIGINATOR.

Contents

Figures and Tables.....	v
Preface.....	ix
1 Introduction.....	1
1.1 Background.....	1
1.2 Objective.....	2
1.3 Approach.....	3
2 Analysis of Survey Changes and Dredging Activity.....	5
2.1 Hydrographic Surveys from eHydro.....	5
2.2 Processing Survey Data.....	6
2.2.1 Sediment Volumes and Bathymetry Profiles from Surveys.....	6
2.2.2 Extracting Bathymetry Profiles.....	7
2.2.3 Calculating Sediment Volumes.....	7
2.3 Dredging Activity.....	9
2.4 Hopper Dredge Disposal Area (HDDA) data.....	9
3 Numerical Model Description.....	12
3.1 HEC-6T Description.....	12
3.2 Model Network.....	13
3.3 Channel Geometry.....	16
3.4 Hydrographs.....	16
3.5 Distributary Flows.....	18
3.5.1 Flowline Model Distributaries Upstream from Belle Chasse.....	18
3.5.2 Old River Control Complex (ORCC).....	18
3.5.3 Bohemia Spillway.....	25
3.5.4 Fort Saint Philip and Ostrica.....	26
3.5.5 Flow Distributaries between Above Venice and Head of Passes (HOP).....	27
3.5.6 Baptiste Collette.....	28
3.5.7 Grand Pass.....	29
3.5.8 West Bay.....	30
3.5.9 Cubits Gap.....	33
3.5.10 Overbank and Minor Outlets between Venice and Pilottown.....	35
3.5.11 South Pass and Pass a Loutre (PAL).....	36
3.5.12 Southwest Pass (SWP) Distributaries.....	37
3.6 Temperature.....	38
3.7 Downstream Water-Surface Elevations.....	39
3.8 Roughness Coefficients.....	41
3.9 Sand Transport Function.....	42
3.10 Silt and Clay Transport Functions.....	42
3.11 Sediment Inflow.....	44
3.11.1 Middle Mississippi River.....	45
3.11.2 Ohio River.....	48
3.11.3 Obion and Hatchie Rivers.....	50

3.11.4 Saint Francis River.....	51
3.11.5 White River.....	51
3.11.6 Arkansas River.....	53
3.11.7 Yazoo River.....	56
3.12 Depth of Bed Sediment Reservoir.....	58
3.13 Bed-Material Gradations.....	59
3.13.1 Lower Mississippi River Bed Sampling Programs.....	59
3.13.2 Bed-Material Sampling Downstream from Venice.....	62
3.14 Sediment Concentrations in Distributary Outflows.....	69
3.14.1 SWP Distributaries.....	71
3.14.2 HOP to Baptiste Collette Distributaries.....	73
3.14.3 Bohemia Spillway.....	75
3.14.4 ORCC.....	75
4 Dredging Specifications.....	77
4.1 Datum.....	77
4.2 Dredging Specifications.....	78
4.3 Dredging Templates.....	78
4.4 Uncertainty Related to Actual Dredging Operations and Numerical Simulations.....	79
5 Model Calibration and Verification.....	80
5.1 Calibration and Verification of HEC-6T Flowline Model.....	80
5.2 Dredging at Deep-Draft Crossings below Baton Rouge.....	81
5.3 Dredging below Venice.....	82
6 Model Results.....	85
6.1 Long-Term Sedimentation Trends in Lower Mississippi River.....	85
6.2 Evaluation of Structural Alternatives.....	87
6.2.1 Reducing Distributary Outflow.....	87
6.2.2 Channel Widening at HDDA.....	91
6.2.3 Reducing Flow into SWP.....	92
6.2.4 Effects of Structural Modifications on Total Dredging.....	92
6.3 Effect of Structural Modifications on Distribution of Dredging.....	94
6.4 Surveyed Bed Elevations in the Navigation Channel.....	100
6.5 Evaluation of Daily Dredging Operations.....	104
6.6 Calculated and Measured Shoaling Rates.....	111
7 Conclusions and Recommendations.....	115
7.1 Conclusions.....	115
7.2 Recommendations.....	116
References.....	118
Abbreviations.....	122
Report Documentation Page (SF 298).....	124

Figures and Tables

Figures

2-1. Vicinity map of Southwest Pass (SWP) navigation channel (<i>left</i>) and a portion of cross-channel transects (<i>right</i>).	7
2-2. A time stack showing volume of sediment within the SWP navigation channel (shades of <i>red</i>) and reported dredge activity (cutterhead dredges— <i>blue circles</i> ; hopper dredges— <i>black circles</i>).	8
2-3. Coverage of Hopper Dredge Disposal Area (HDDA) study area from survey on 8 June 2012.	10
2-4. Dredge placement locations near the HDDA from 2015 to 2019 (a) and relative location density of dredge material placements within the HDDA (b).	11
3-1. Schematic of HEC-6T stream network in New Orleans (MVN) and Vicksburg Districts (not to scale).	14
3-2. Schematic of HEC-6T Stream Network in Memphis and St. Louis Districts (not to scale).	15
3-3. Old River Control Complex (ORCC).	19
3-4. Percentage of Union Point discharge diverted at ORCC.	20
3-5. Percentage of Union Point Flow diverted at Hydropower Structure.	21
3-6. Flow diverted at ORCC Overbank Structure 1977–2019.	22
3-7. Combined Low Sill and Auxiliary flow diverted at ORCC.	23
3-8. Diversion percentages for Low Sill and Auxiliary Structures.	24
3-9. Location of Bohemia, Ostrica, and Fort Saint Philip distributaries (map data: Google Earth, Image Landsat/Copernicus, 12/2017).	26
3-10. Major distributaries downstream from Above Venice (map data: Google Earth, Image Landsat/Copernicus, 12/2017).	27
3-11. Baptiste Collette diversion percentages, 2004–2019.	29
3-12. Grand Pass diversion percentages, 2004–2019.	30
3-13. Flow diversion percentages at West Bay, 2004–2019.	32
3-14. West Bay diversion percentages—variation with discharge, 2008–2019.	32
3-15. Flow diversion percentage at Cubits Gap, 1991–2019.	34
3-16. Cubits Gap diversion percentages—variation with discharge.	35
3-17. Leaks diversion percentages, 2004–2019.	36
3-18. Mean monthly tide elevations at Grand Isle, Louisiana 1978–2014.	40
3-19. Measured suspended sediment at Thebes 2003–2019.	46
3-20. Measured suspended sediment concentrations Dam 53 and Olmsted, 2003–2019.	49
3-21. Measured suspended sediment concentrations—White River at DeValls Bluff, 2001–2019.	52
3-22. Measured Fines 1994–2019 Arkansas River at Terry Lock and Dam.	55
3-23. Measured Sand 1994–2019 Arkansas River at Terry Lock and Dam.	55
3-24. Measured suspended sediment concentrations Yazoo River at Steele Bayou, 1999–2019.	57

3-25. Lateral variation in bed-material gradation at RM 265, August 2005.....	61
3-26. Historic change in median bed grain size (Nordin and Queen 1992).....	61
3-27. Initial calculated bed gradations—30-day simulation, starting with Nordin polynomials.....	62
3-28. Dredge material median grain size, 2004–2005.	63
3-29. 2003–2006 bed gradations above Head of Passes (AHP).	64
3-30. Discharge at Tarbert Landing during 2009 sampling program.	65
3-31. Comparison of median grain sizes from 2003–2006 and 2009 sampling.	65
3-32. Bed gradations, March 2009, on the rising hydrograph below Cubits Gap.....	67
3-33. May and July 2009 bed gradations in SWP above River Mile (RM) 10, below Head of Passes (BHP).	68
3-34. May and July 2009 bed gradations in SWP below RM 10, BHP.....	68
5-1. Calculated and reported dredging volumes at crossings, Water Year (WY) 1991– 2013.....	82
5-2. Calculated and reported dredging, WY 1991–2019.....	83
5-3. Reported and calculated dredging above and below RM –11.	84
6-1. Specific gage at Memphis, RM 734.4.....	85
6-2. Specific gage at Vicksburg, RM 435.7.	86
6-3. Specific gage at Red River Landing, RM 302.4.....	86
6-4. Major distributaries downstream from Belle Chasse (maps data: Google Earth, Image Landsat/Copernicus, 12/2017).....	88
6-5. Conceptual distributary flow reduction structure at West Bay.	88
6-6. The 50 yr average percentage of Belle Chasse water and sediment diverted at downstream distributaries.	90
6-7. Channel widening alternative adjacent to HDDA.....	91
6-8. The 50 yr average annual dredging for diversion alternatives.	93
6-9. Calculated 50 yr average annual accumulated dredging closure alternatives.	95
6-10. Calculated average annual accumulated dredging HDDA and increased flow into Pass a Loutre (PAL) and South Pass.	95
6-11. Calculated 50 yr average annual dredging with 50% diversion at West Bay.	96
6-12. Calculated 50 yr average annual dredging with 50% diversion at Cubits Gap.	96
6-13. Calculated 50 yr average annual dredging with 50% diversion at South Pass.	97
6-14. Calculated 50 yr average annual dredging with 50% diversion at Joseph Bayou.....	97
6-15. Calculated 50 yr average annual dredging with 50% diversion in four SWP distributaries.	98
6-16. Calculated 50 yr average annual dredging with closure of all distributaries except PAL and South Pass.	98
6-17. Calculated 50 yr average annual dredging with channel widening at HDDA.	99
6-18. Calculated 50 yr average annual dredging with 10% more flow diverted into PAL and South Pass.....	99
6-19. Surveyed navigation channel elevations RM 5.5 to RM 2.46.	101
6-20. Surveyed navigation channel elevations RM 1.6 to RM 0.40.	101
6-21. Surveyed navigation channel elevations RM –0.1 to RM –3.1.....	102
6-22. Surveyed navigation channel elevations RM –3.1 to RM –10.7.....	102

6-23. Surveyed navigation channel elevations RM -11.9 to RM -20.2	103
6-24. Assigned dredging reaches for Daily Operations Model (maps data: Google Earth, Image Landsat/Copernicus, 12/2017).....	105
6-25. Calculated and reported annual dredging SWP and HOP 2015–2019.....	106
6-26. Calculated and reported annual dredging plus residual deposition, SWP and HOP 2015–2019.....	108
6-27. Calculated and measured annual dredging plus residual deposition, 2015–2019....	110
6-28. Onset of shoaling—survey and HEC-6T 2015.....	112
6-29. Onset of shoaling—survey and HEC-6T 2016.....	113
6-30. Onset of shoaling—survey and HEC-6T 2017.....	113
6-31. Onset of shoaling—survey and HEC-6T 2018.....	114
6-32. Onset of shoaling—survey and HEC-6T 2019.....	114

Tables

3-1. Description of HEC-6T segments.....	13
3-2. Hydropower flow diversion percentages.....	21
3-3. Overbank flow diversion percentages.....	22
3-4. Percentage of index station flow diverted at Low Sill and Auxiliary.....	24
3-5. Distributary outflow at Fort Saint Philip and Ostrica percentage of discharge at Myrtle Grove (RM 57).....	27
3-6. Diversion percentages used in HEC-6T at West Bay.....	33
3-7. 2008–2019 HEC-6T distributary distribution percentages at Head of Passes (HOP).	37
3-8. Outflow from SWP distributaries percent of discharge immediately upstream from diversion.....	37
3-9. Average monthly water temperature 1965–2014, °F.....	39
3-10. Downstream water-surface elevations.....	40
3-11. HEC-6T roughness coefficients in MVN.....	41
3-12. Silt and clay coefficients for Parthenaides and Krone equations.....	44
3-13. Average annual contributory percentage upstream of ORCC.....	45
3-14. Years 2003–2019 sediment inflow at Thebes–Segment 25.....	48
3-15. Sediment inflow for Ohio River–Segment 24.....	50
3-16. Sediment inflow for White River–Segment 10.....	52
3-17. Sediment inflow for Arkansas River–Segment 8.....	56
3-18. Sediment inflow for Yazoo River–Segment 6.....	58
3-19. Sediment concentration diversion ratios, Burrwood Bayou.....	72
3-20. Sediment concentration diversion ratios, Outlet W-2 and Overbank.....	72
3-21. Sediment concentration diversion ratios, Joseph Bayou.....	72
3-22. Sediment concentration diversion ratios, Outlet W-1.....	72
3-23. Sediment concentration diversion ratios, South Pass and PAL.....	73
3-24. Sediment concentration diversion ratios, “Leaks.”.....	73
3-25. Sediment concentration diversion ratios, Cubits Gap.....	74

3-26. Sediment concentration diversion ratios, Baptiste Collette.	75
3-27. Sediment diversion ratios at ORCC, 1991–2019	76
3-28. Sediment diversion ratios for Overbank Structure.....	76
4-1. Datum offsets for Mississippi River gages from Venice to East Jetty.....	77
5-1. Calculated and reported dredging, Water Year (WY) 1991–2019.....	84
6-1. The 50 yr average percent of the Belle Chasse discharge out SWP distributaries for Base and 50% SWP and 10% PAL and South Pass (SP) alternatives.....	92
6-2. Calculated 50 yr average annual dredging.....	93
6-3. Effect of alternatives on dredging distribution.	94
6-4. Revised fine sediment coefficients.	106
6-5. Designated reaches for dredging analysis.....	108
6-6. Calculated annual dredging and residual deposition in cubic yards.....	109
6-7. Measured annual dredging and residual deposition in cubic yards.....	109

Preface

This study was conducted for the US Army Engineer Research and Development Center (ERDC) under the US Army Corps of Engineers, Regional Sediment Management (RSM) Program, under Funding Account Code 121BG5, AMSCO Code 08303. The technical monitor was Dr. David W. Perkey, RSM program manager.

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 P. May was chief of the River and Estuarine Engineering Branch; Dr. Cary A. Talbot was chief of the Flood and Storm Protection Division; and Mr. Charles E. Wiggins was the technical director for Navigation Systems Research. The deputy director of ERDC-CHL was Mr. Keith W. Flowers, and the director was Dr. Ty V. Wamsley.

Portions of this report have been modified and reprinted from Ronald R. Copeland, Leslie Lombard, and Roger A. Gaines, *Numerical Sedimentation Investigation Mississippi River Cairo to Pilots Station*, MRG&P Report No. 30 (Vicksburg, MS: US Army Engineer Research and Development Center, February 2020), <http://dx.doi.org/10.21079/11681/35573>. Public domain.

Portions of this report have been modified and reprinted from Ronald R. Copeland, *Mississippi River and Tributaries Flowline Assessment: Mississippi River Sedimentation Report*, MRG&P Report No. 24; Volume 4 (Vicksburg, MS: US Army Engineer Research and Development Center, December 2018), <http://dx.doi.org/10.21079/11681/32664>. Public domain.

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

This page intentionally left blank.

1 Introduction

1.1 Background

The 1D numerical sedimentation model used in this study was developed over a period of approximately 10 yr* and extends from Chester, Illinois, on the Middle Mississippi River and Olmstead 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 at the entrance to Southwest Pass (SWP) and Cairo, Illinois, was completed in 2011 (Copeland et al. 2020). Model development was coordinated by the Mississippi Valley Division and was cooperatively developed by US Army Corps of Engineers (USACE) engineers from the New Orleans, Vicksburg, Memphis, and St. Louis Districts. Each district developed the Regional Model in their respective reaches of responsibility along the Mississippi River. The model covers over 1000 river miles. The initial cross-section geometry was taken from 1988 to 1992 hydrographic surveys and was calibrated to 1991 to 2002 measured data. Dikes were added to the channel geometry as they were constructed between 1991 and 2009. The objective of the Regional Model is to provide a tool to evaluate the long-term and system-wide effects of specific Mississippi River and Tributaries (MR&T) Project features. These include 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 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

* For a full list of the spelled-out forms of the units of measure and unit conversions used in this document, please refer to *US Government Publishing Office Style Manual*, 31st ed. (Washington, DC: US Government Publishing Office 2016), 248–52 and 345–7, respectively. <https://www.govinfo.gov/content/pkg/GPO-STYLEMANUAL-2016/pdf/GPO-STYLEMANUAL-2016.pdf>.

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 in developing a Flowline Model for the Lower Mississippi River between Cairo and Pilots Station. The Flowline Model was used to determine how long-term sedimentation would affect Project Design Flood 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 controlled levee breach during the 2011 flood at Bird's Point, which is located at the head of the New Madrid Floodway, and (3) water-surface elevations from the 2011 flood. Enhancements from recent 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 in the model 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

Significant maintenance dredging is required each spring at the entrance of the Mississippi River navigation channel, where an average of 17 million yd³ of dredged material are removed each year from the first 30 mi of the channel. Throughout the life of the project, structural and operational modifications have been made in response to changes in river shoaling patterns and to reduce maintenance requirements. The purpose of this study is to evaluate possible modifications near the river's Head of Passes (HOP) to evaluate the distribution of water and sediment and assess any changes in navigation channel shoaling. This was accomplished using a previously developed HEC-6T 1D numerical sedimentation model. The model had been used successfully to identify and predict the impact of

various planned MR&T Project features and dredging strategies on long-term sediment trends between Cairo, Illinois, and Pilots Station, Louisiana.

Another study objective is to demonstrate the model's capability to predict daily sedimentation processes and dredging operations. The eventual goal is a methodology to optimize the deployment of dredges ahead of anticipated flood hydrographs.

1.3 Approach

The cross-section geometry for the previous HEC-6T numerical sediment models was based on 1988–1992 hydrographic surveys. These initial conditions were brought up to date by simulating the historical daily hydrographs up to 2019, which established the initial channel geometry for this study. This approach takes advantage of calibration and verification analyses from previous HEC-6T studies so that these procedures do not have to be repeated in this study.

In the previous studies, water-surface elevations had been calibrated to gage stage data by adjusting channel roughness coefficients. Roughness coefficients used in the Regional Model were later checked and adjusted as necessary for the Flowline Model. These included adjustments for very high discharges using data from the 2011 flood. The model had been verified by comparing calculated sediment transport at downstream gages with measured data at those same gages. Also, the model had been verified by simulating deposition below the Old River Control Complex (ORCC) and scour that occurred with the construction of the Smithland Dikes downstream from Old River.

Coefficients were recently checked for the reach below Tarbert Landing during the Mississippi River deep-draft crossing study. Model results were verified using specific gage analysis at several gages between the Gulf and Cairo. Additional verification was accomplished using reported dredging data in SWP and at the deep-draft crossings between New Orleans and Baton Rouge.

The model was deemed appropriate for use to meet the objectives of this investigation, related to structural modifications, without further calibration or verification.

Structural modifications in SWP and above Head of Passes (AHP) were evaluated by comparing 50 yr projected average annual dredging requirements for several alternatives to a base condition representing existing conditions. Dredging was simulated using a consistent dredging operation based on a dredging trigger elevation at the authorized depth of 50 ft mean lower low water (MLLW) and an advanced maintenance depth of 6 ft.

Modeling daily shoaling and dredging operations is considered feasible and verifiable due to recent collections of daily survey data in the SWP and HOP navigation channel. Although the HEC-6T model has algorithms to simulate dredging operations, it has not been tested for daily operations. This study evaluated the capabilities of model predictions on a daily basis using the recently available survey data.

2 Analysis of Survey Changes and Dredging Activity

This section summarizes the geospatial processing steps conducted with two USACE enterprise data sets capturing navigation channel bathymetry and operational dredging diagnostics, respectively. The intent of this combined analysis is to characterize the highly dynamic and variable sedimentation patterns observed in the study area as well as to better understand the impact of daily dredging operations on the overall sediment load moving through SWP.

2.1 Hydrographic Surveys from eHydro

Hydrographic surveys capturing navigation channel bathymetric conditions are compiled, standardized, and made available through the USACE eHydro system. Likewise, for dredging operations, the onboard diagnostic sensor arrays for hopper dredges under contract with the USACE provide detailed information to the National Dredging Quality Management (DQM) geospatial data archive. The DQM system monitors critical aspects of dredge activity, including precise dredge locations and the volumes of material dredged, and overall dredge status (e.g., loading, sailing heavy, hull open). Separate records are maintained by the New Orleans District (MVN) pertaining to cutterhead dredge work. Using the same methodology as described in Hartman et al. (2022), historical hydrographic surveys completed by MVN and uploaded to eHydro can be used to quantify shoaling volumes and morphologic patterns across a range of spatial scales. The daily survey frequency in combination with the DQM dredging records provides the necessary data to calculate a navigation channel elevation from River Mile (RM) –22 to RM 13.4 for the 5 yr period between 2015 and 2019.

Most channel condition surveys used in this analysis were completed using single-beam echosounders along cross-channel transects a defined distance apart, typically at 500 ft intervals (Byrnes et al. 2002), a spacing that is adequate for understanding dynamic channel conditions while keeping data processing requirements manageable and timely. If the intent of the survey is to focus on a particular predredge or postdredge area, transect spacing may be varied to provide the desired coverage of the region of interest. Surveys of the Hopper Dredge Disposal Area (HDDA)

used similar single-beam surveying techniques employed in the main navigation channel, albeit with different transect spacing and orientations.

2.2 Processing Survey Data

Each survey consists of a scattered point dataset referenced to the Louisiana South State Plane (feet) Projected Coordinate System. Surveys of SWP collected prior to 26 January 2017 were referenced to Mean Low Gulf (MLG), a local tidal datum used by MVN at projects near the mouth of the Mississippi River, and surveys after that date were referenced to MLLW. For consistency, the surveys originally in MLG were transformed to MLLW using the elevation offset described in (USACE 2017a).

Given the nature of the scattered point data, interpolation between survey data is necessary to describe the bathymetric surface. Once the survey data were in common horizontal and vertical reference systems, they were used to construct Triangulated Irregular Networks (TIN) to represent the bathymetric surface between survey lines. Each TIN was converted to a raster format with consistent raster cell dimensions of 10 ft × 10 ft. The use of a TIN interpolation technique is a form of linear interpolation, and the rasterization process essentially samples the constructed TIN surface at a uniform 10 ft interval, thereby allowing a consistent reference frame to evaluate changes through time. Finally, the rasterized datasets were stored as multidimensional datasets using the NetCDF file format.

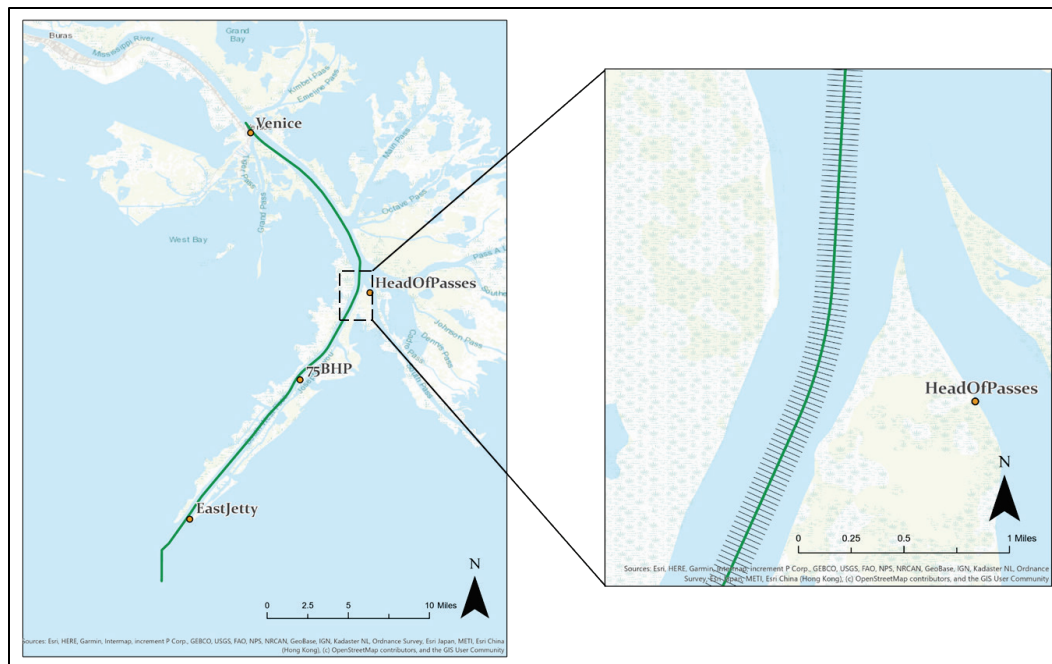
The near-daily frequency of surveying in SWP means that conditions in many parts of the channel are recorded at a temporal sampling rate that enables a detailed analysis of shoaling patterns and trends. For the portions of the main navigation channel that are not sampled as frequently, depths were linearly interpolated temporally to provide a daily time series of bathymetry for the entire channel. For the analysis of the HDDA, the spatial interpolation was conducted as in the navigation channel. However, the temporal interpolation was omitted since the HDDA was surveyed more infrequently than the main navigation channel.

2.2.1 Sediment Volumes and Bathymetry Profiles from Surveys

Daily hydrographic survey data from the navigation channel, between Venice (RM 10.7) and the Bar Channel (RM -22.1), were used to calculate bed elevations for the 5 yr period between 2015 and 2019. A series of transects were generated perpendicular to the navigation channel

centerline at a 100 ft spacing. The features are depicted in Figure 2-1. The channel centerline (*green line*) and locations of US Geological Survey (USGS) gages in the region are shown in both the *left* and *right* parts of the figure. The individual transects (*black lines*) are shown in a zoomed view near HOP (*right*).

Figure 2-1. Vicinity map of Southwest Pass (SWP) navigation channel (*left*) and a portion of cross-channel transects (*right*).



In total, 1,773 transects were created and extended the entire width of the navigation channel. Between RM 11.4 (AHP) and RM -17.5 below Head of Passes (BHP), the transects were 750 ft in length. Between RM -18.0 (BHP) and -22.1 (BHP), the bins were 600 ft in length.

2.2.2 Extracting Bathymetry Profiles

The entire time series of bathymetry raster data was sampled using the cross-channel transects for comparison to modeled results. For each bathymetric profile, depths were extracted at a 10 ft spacing along the transect using bilinear interpolation.

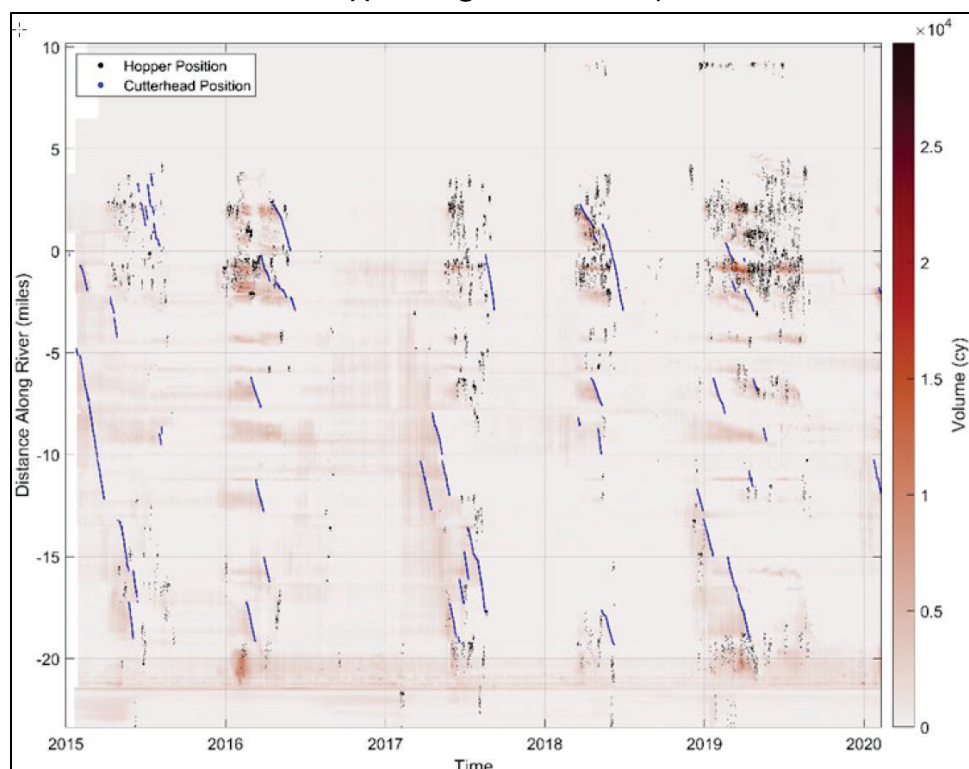
2.2.3 Calculating Sediment Volumes

The cross-channel transects also served as the basis for transect bins spanning the entire channel width and were 100 ft in length in the along-channel direction. The data were divided into 1,772 nonoverlapping bins.

Between RM 11.4 (AHP) and RM -17.5 (BHP), the bins were 750 ft wide and 100 ft long. Between RM -18.0 (BHP) and -22.1 (BHP), the bins were 600 ft wide and 100 ft long. Bins constructed from these transects help summarize bathymetric conditions at a finer resolution than afforded by only using the reach boundaries.

For each daily bathymetry raster in the navigation channel time series, the volume of sediment shallower than certain target depths was calculated. The target depths selected for consideration were 48.5 ft (MLLW), 56 ft (MLLW), and 80 ft (MLLW) and represent the navigation channel's maintained depth, advanced maintenance depth, and a depth sufficiently deeper than any observed depth in the region of interest. Then, the volume of sediment shallower than the target depths were summed per transect bin. This produced a record of sediment volume through time for each transect bin. The volume of sediment within each bin is shown in Figure 2-2 as shades of *red*. The darker gradient represents larger volumes of sediment for a given bin.

Figure 2-2. A time stack showing volume of sediment within the SWP navigation channel (shades of *red*) and reported dredge activity (cutterhead dredges—*blue circles*, hopper dredges—*black circles*).



2.3 Dredging Activity

In addition to summarizing the observed bathymetric conditions and the changes between surveys, these transect bins also help with relating the hopper-dredging activity from DQM within regions of the channel.

DQM has been in place for over a decade and collects performance as measured by sensor arrays outfitted to dredging vessels conducting USACE Civil Works projects and any work requiring Department of the Army permitting (USACE, HQ 2006). Onboard a monitored dredge, multiple sensors continuously record dredge activities and operations. Sensor data are transmitted to the DQM servers for processing, storage, and distribution. As part of the DQM processing phase, the data are enriched with different labels identifying the dredge status. While not an exhaustive list of possible dredging status labels, the labels can indicate whether the dredge was actively dredging sediment, sailing to a placement area, or placing sediment, among other operational states.

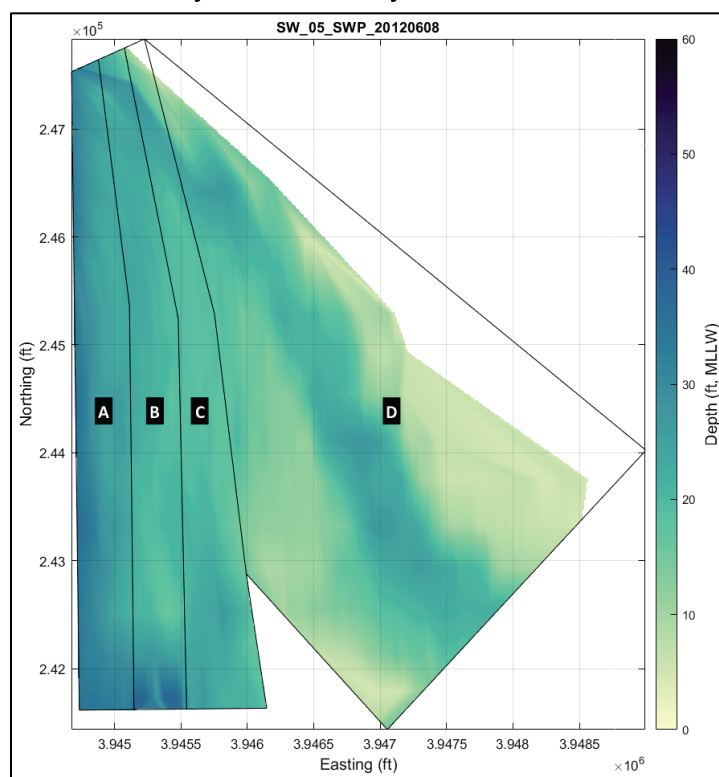
Cutterhead dredging records collected by the USACE MVN for tracking operational mission readiness were used to compile daily production reports from 2010 to 2019. These reports contain the daily starting and stopping location by station and the total volume of sediment dredged for cutterhead vessels working in the SWP. The position of cutterhead dredges from the MVN reports and actively dredging hopper dredges from DQM are shown in Figure 2-2 as *black circles* and *blue circles*, respectively.

2.4 Hopper Dredge Disposal Area (HDDA) data

The HDDA is located at HOP, adjacent to the left side of the navigation channel, at the entrance to Pass a Loutre (PAL), as shown in Figure 2-3 and Figure 2-4. Hydrographic surveys from eHydro and dredging records from DQM were also used to investigate the bathymetry of the HDDA and volumetric changes through time. The eHydro survey data were prepared using the same approach for surveys of the main navigation channel described in Section 2.2. Once the bathymetric survey data were prepared, the HDDA was partitioned into smaller regions to summarize material shallower than the same target depths considered in the main channel. However, instead of using cross-channel transect bins, relatively larger regions were oriented in the along-channel direction were used (Figure 2-3).

The regions of the HDDA are labeled *A*, *B*, *C*, and *D*, in Figure 2-3, with region *A* situated closest to the navigation channel and region *D* closest to PAL. Most of the eHydro surveys offer only partial coverage of the HDDA zones. Typically, surveys of the HDDA were most likely to include region *A*, given its proximity to the navigation channel. Likewise, region *D* was least likely to be surveyed, and surveys that did include some of region *D* rarely provided full coverage.

Figure 2-3. Coverage of Hopper Dredge Disposal Area (HDDA) study area from survey on 8 June 2012.

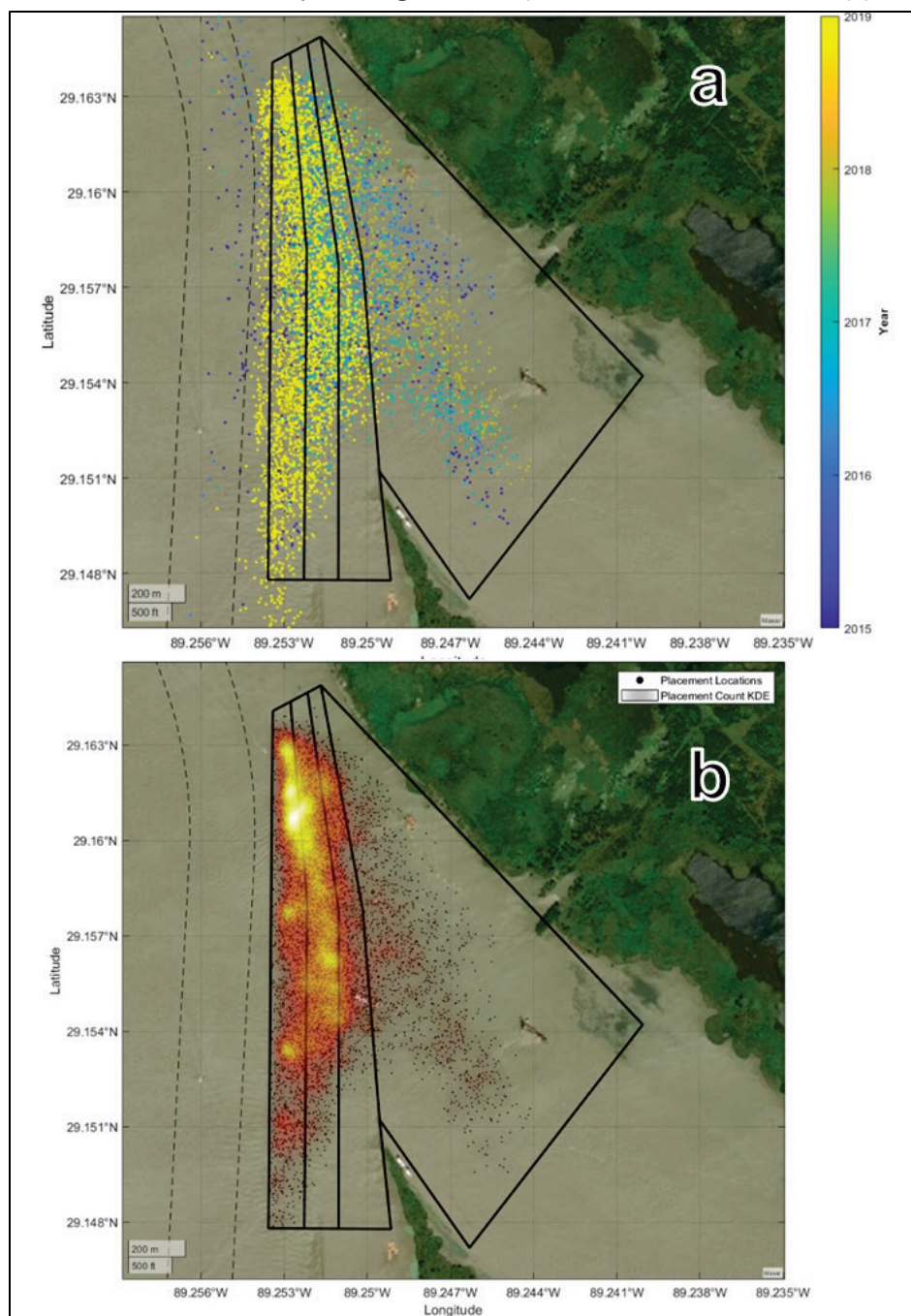


When analyzing the DQM data in the full navigation channel, only records with dredge status labels pertaining to active dredge loading or placement were considered. The DQM data can help inform where sediment is being placed, providing additional insight into how the HDDA has been utilized. Therefore, analysis of DQM data in the HDDA focused on the dredge empty locations. In the fuller SWP work, the active dredging status records were spread over a broad geographic extent. In comparison, the dredge emptying locations were more tightly grouped.

The DQM records near the HDDA are shown in Figure 2-4(a) with the time of placement represented by the color gradient. The density of placement

locations, particularly in regions A, B, and C, leads to overplotting that makes it difficult to see the distribution of placements. In Figure 2-4 (b), a kernel-density estimate of the placement sites highlights regions in the HDDA with relatively higher use (brighter colors like *yellow* and *white*) compared to other portions of the HDDA (*dark red* or *no color*).

Figure 2-4. Dredge placement locations near the HDDA from 2015 to 2019 (a) and relative location density of dredge material placements within the HDDA (b).



3 Numerical Model Description

3.1 HEC-6T Description

The HEC-6T 1D numerical sedimentation model was used in this study (Thomas 2021). Mr. William A. Thomas initiated development of this computer program at the US Army Engineer District, Little Rock, in 1967. Further development at the US Army Engineer Hydrologic Engineering Center by Mr. Thomas produced the HEC-6 generalized computer program for calculating scour and deposition in rivers and reservoirs. Additional modification and enhancement to the basic program by Mr. Thomas and his associates at ERDC led to the HEC-6W program. The HEC-6T program used in this study is the product of additional modification and enhancement conducted by Mr. Thomas at Mobile Boundary Hydraulics PLLC. Version H6TV51325_J18amR_210602 was used to make calculations in this study. The model is proprietary and can be obtained from Mobile Boundary Hydraulics.

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 does not simulate bank erosion or natural adjustments in channel widths. When applied by experts using good engineering judgment, the HEC-6T program will provide good insight into the behavior of mobile-bed rivers. Additional guidance related to model calibration, application, and interpretation can be found in the following references: *Sedimentation in Stream Networks* (Thomas 2021); *Computational Modeling of Sedimentation Processes*—Chapter 14 (Thomas and Chang 2008); and *Guidelines for the Calibration and Application of Computer Program HEC-6* (USACE-HEC 1992).

3.2 Model Network

The model's geometry is broken into 25 segments as shown in the network schematics in Figure 3-1 and Figure 3-2. The number of cross sections in each segment and the description of the segment boundaries are given in Table 3-1.

Table 3-1. Description of HEC-6T segments.

Segment	Number of Cross Sections	Description	1962 River Miles
1	133	Mississippi River–Pilots Station to Profit Island	–18.0 to 246.3
2	6	Mississippi River–Main Channel (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 St Francis River	600.1 to 672.6
12	19	St Francis River	0.5 to 17.7
13	38	Mississippi River–St Francis River to Hatchie River	674.7 to 772.8
14	3	Hatchie River	0.1 to 2.0
15	18	Mississippi River–Hatchie River to Obion River	774.0 to 819.4
16	3	Obion River	0.1 to 2.0
17	35	Mississippi River–Obion River to Island 8	820.0 to 910.0
18	5	Mississippi River–East Loop Island 8	910.5 to 914.6
19	5	Mississippi River–West Loop Island 8	910.5 to 914.6
20	9	Mississippi River–Island 8 to Wolf Island	915.6 to 930.8
21	3	Mississippi River–East Loop Wolf Island	931.5 to 934.4
22	3	Mississippi River–West Loop Wolf Island	931.5 to 934.4
23	8	Mississippi River–Wolf Island to Cairo	934.4 to 953.0
24	37	Ohio River–Cairo to Metropolis	953.0 to 990.0
25	46	Upper Mississippi River–Cairo to Chester	0.8 to 109.9

Figure 3-1. Schematic of HEC-6T stream network in New Orleans (MVN) and Vicksburg Districts (not to scale).

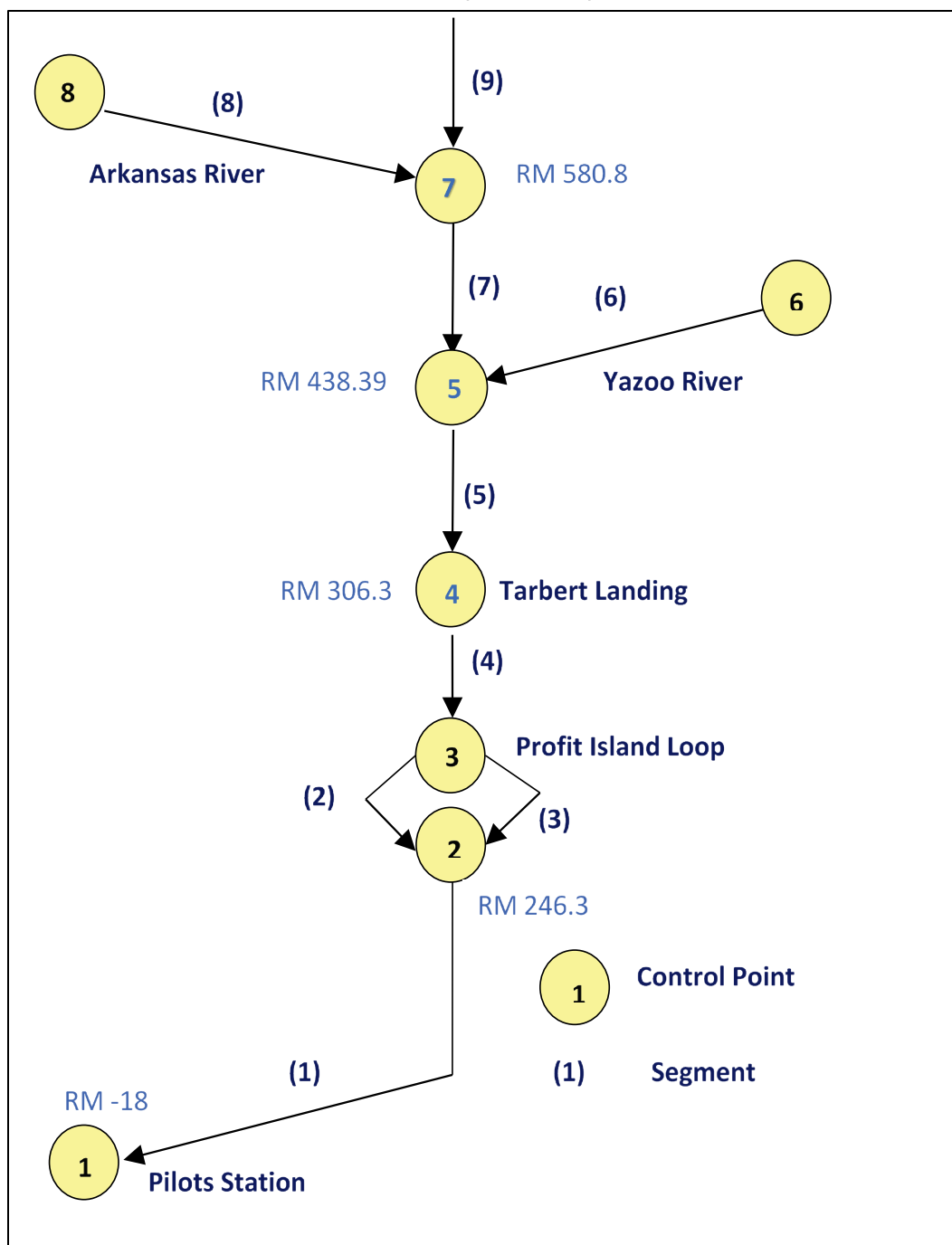
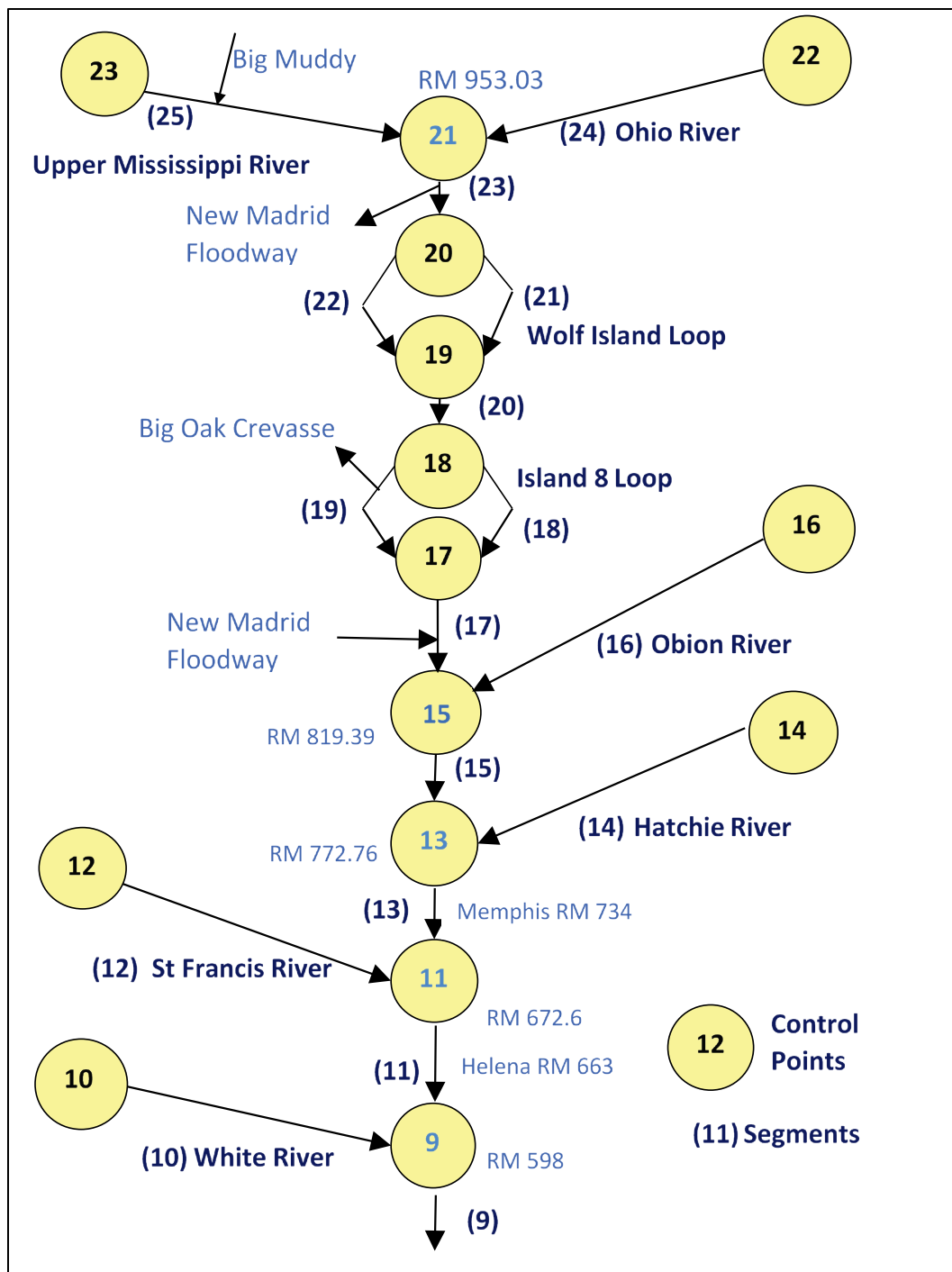


Figure 3-2. Schematic of HEC-6T Stream Network in Memphis and St. Louis Districts
(not to scale).



3.3 Channel Geometry

The cross-section geometry in the Regional HEC-6T model was developed from hydrographic survey data taken between 1988 and 1992. The survey datum was National Geodetic Vertical Datum of 1929 (NGVD 29). The hydrographic survey dates are not homologous 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 is retained in this study. This is deemed reasonable because the calibration and verification runs begin using the October 1990 discharges.

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 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.

Reach lengths for the channel were measured between the thalwegs for each cross section. Reach lengths for the overbanks were measured from two-thirds of the length of the overbank between each cross section.

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).

In this study, 29 yr between October 1990 and September 2019 were simulated starting with the 1988–1992 initial geometry. The resultant channel geometry was then used as the initial conditions for the 50 yr predictive runs.

3.4 Hydrographs

Discharge hydrographs are simulated in the numerical model by a series of steady-state events. The duration of each event is chosen such that the

changes in bed elevation, due to deposition or scour, do not significantly change the hydraulic parameters during that event. In this study, computational time-steps of 1 day were used. A 1-day time-step is not necessary to meet the requirement of insignificant bed change but was chosen for convenience because available data were reported as mean daily flows.

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

Reported daily discharges are available at several gages within the study 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 input 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, Mississippi. 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.

The hydrograph used in this study incorporates additional discharge data that became available after completion of the Regional and Flowline Model studies. Data available through 2019 were used in this study.

More detailed information about the hydrograph data and sources are provided in Copeland et al. (2020) and Copeland (2018).

3.5 Distributary Flows

3.5.1 Flowline Model Distributaries Upstream from Belle Chasse

There are several distributaries from the Lower Mississippi River between Cairo, Illinois, and the Gulf of Mexico that are included in the HEC-6T model. Most of the distributaries upstream from Belle Chase, which is located at RM 76, were incorporated into the current model unchanged from the Flowline Model. The exception is the ORCC. Distributaries taken from the Flowline Study model are the following:

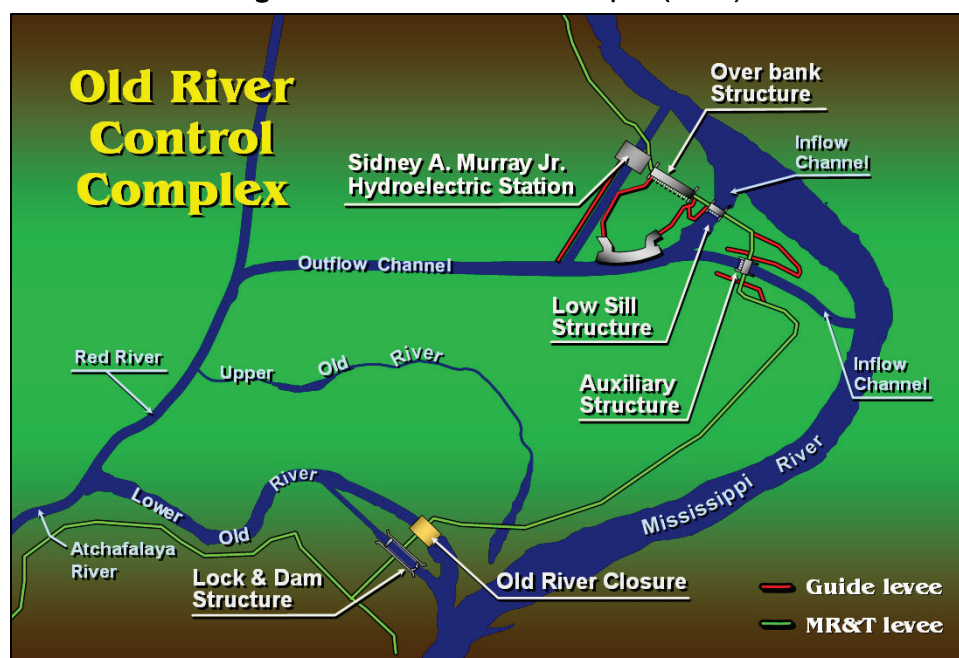
- New Madrid Floodway, upstream of RM 949
- 2011 Crevasse, upstream of RM 915.6
- Yazoo River Backwater, upstream of RM 444.8
- Morganza Spillway, upstream of RM 277.5
- Devil's Swamp, upstream of RM 244.2
- Manchac Point, upstream of RM 220.9
- Bonnet Carré Spillway, upstream of RM 127.1
- Davis Pond,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

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

3.5.2 Old River Control Complex (ORCC)

The total percentage of the Mississippi River discharge diverted at the ORCC was based on historical daily data collected between 1991 and 2019. The features of the ORCC are shown in Figure 3-3.

Figure 3-3. Old River Control Complex (ORCC).



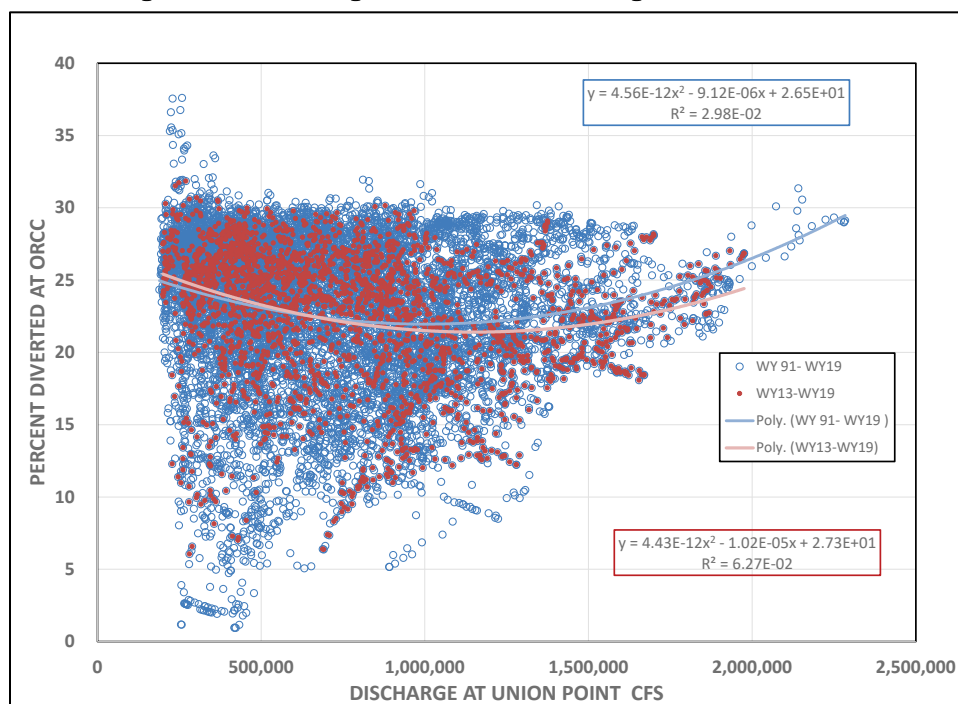
The total flow diverted at the ORCC was not affected by operation changes that occurred during those 29 yr (Figure 3-4). However, the distribution of diverted flow through the various ORCC structures changed significantly in 2013. The most significant change was the return of the Overbank Structure into 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 flow went through the Auxiliary Structure. Since 2013, each structure carries about 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 numerical simulations to account for changes in the flow distribution. Long-term average rating curves were developed from the daily data at each structure.

The historical simulations of the 29 yr between 1991 and 2019 used two different rating 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 rating curve developed from the 2013–2019 data was used for the historical simulations between 2013 and 2019.

The simulations that projected 50 yr sedimentation effects used 2013–2019 rating 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 increase the data base.

The index station for the Hydropower and Overbank Structure 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 to 2012 or 2013 to 2019 daily flow data.

Figure 3-4. Percentage of Union Point discharge diverted at ORCC.



Regression equations were developed for Hydropower diversion percentages using data between 1991 to 2012 and 2013 to 2019. The 2013 to 2019 regression equation was used to determine Hydropower diversion percentages for future simulations and for 2013 to 2019 historical simulations. An optical extrapolation was used for Union Point

discharges greater than 2,000,000 ft³/sec, and a minimum diversion percentage was set based on the minimum measured Union Point discharge (189,000 ft³/sec). The data and rating curves are shown in Figure 3-5 and Table 3-2.

Figure 3-5. Percentage of Union Point Flow diverted at Hydropower Structure.

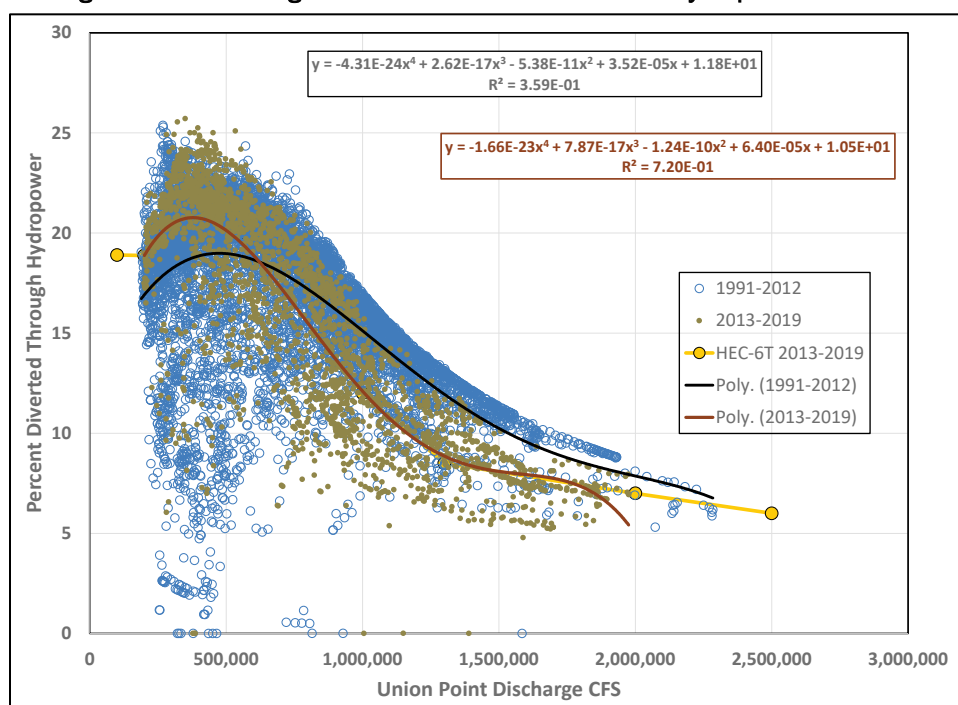


Table 3-2. Hydropower flow diversion percentages.

Discharge, ft ³ /sec Union Point	Diversion Percentage 1991–2012	Diversion Percentage 2013–2019
100,000	17.0	18.9
190,000	17.0	18.9
300,000	—	20.6
450,000	19.0	20.6
550,000	19.0	20.0
750,000	17.6	—
1,000,000	—	12.0
1,300,000	—	8.5
1,400,000	11.0	—
1,700,000	9.0	—
2,000,000	6.7	7.0
2,500,000	6.7	7.0

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 3-6 and Table 3-3. 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 rating curve was used after October 2012 and in the future simulations (between October 1991 and October 2012, diversion percentages were zero).

Figure 3-6. Flow diverted at ORCC Overbank Structure 1977–2019.

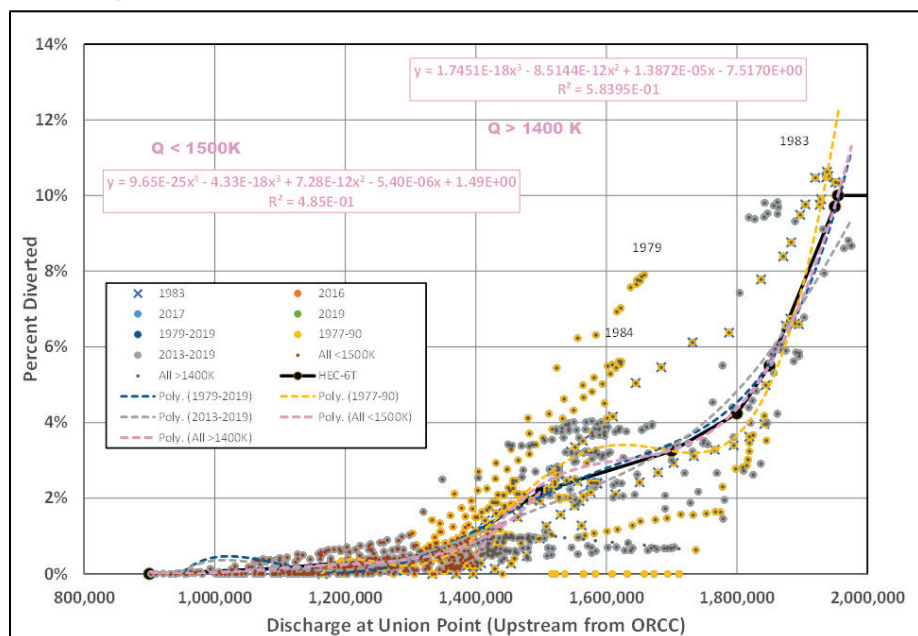


Table 3-3. Overbank flow diversion percentages.

Discharge, ft ³ /sec Union Point	Diversion Percentage
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 rating curves for the combined Low Sill plus Auxiliary were determined using the daily difference in the total ORCC diversion percentage minus the hydropower and overbank spillway percentages. There is an extensive scatter about the regression lines (Figure 3-7). 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 with this methodology.

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 rating curves (Figure 3-8). 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 rating curve is tabulated in Table 3-4.

Figure 3-7. Combined Low Sill and Auxiliary flow diverted at ORCC.

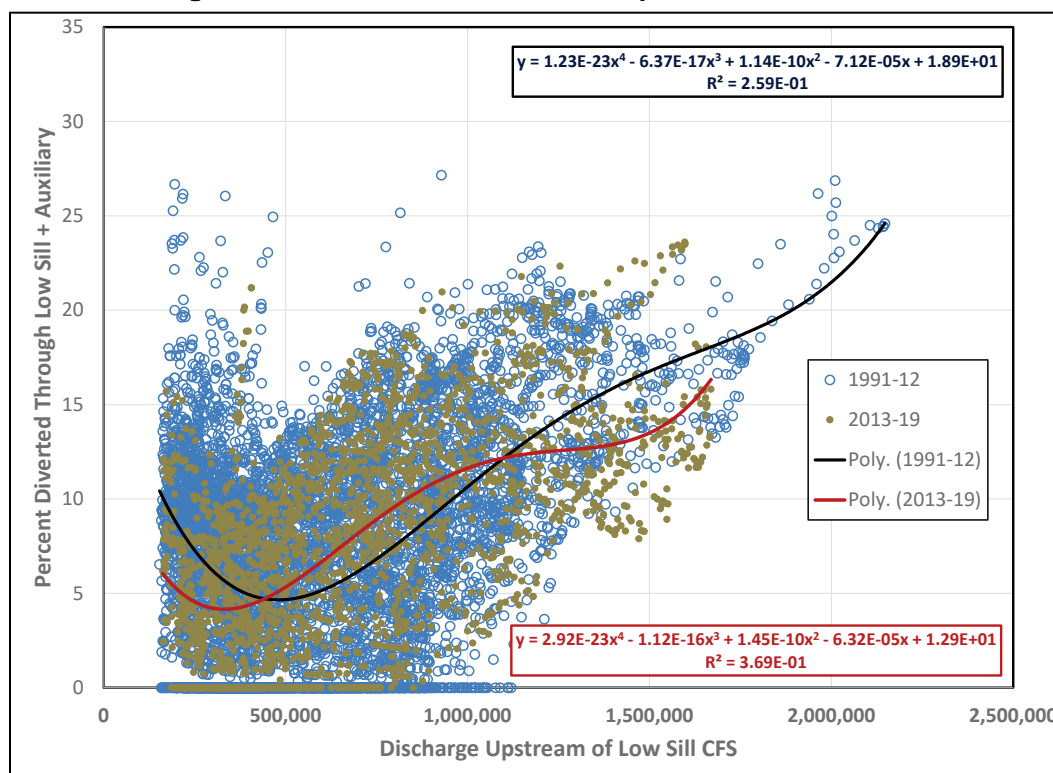


Figure 3-8. Diversion percentages for Low Sill and Auxiliary Structures.

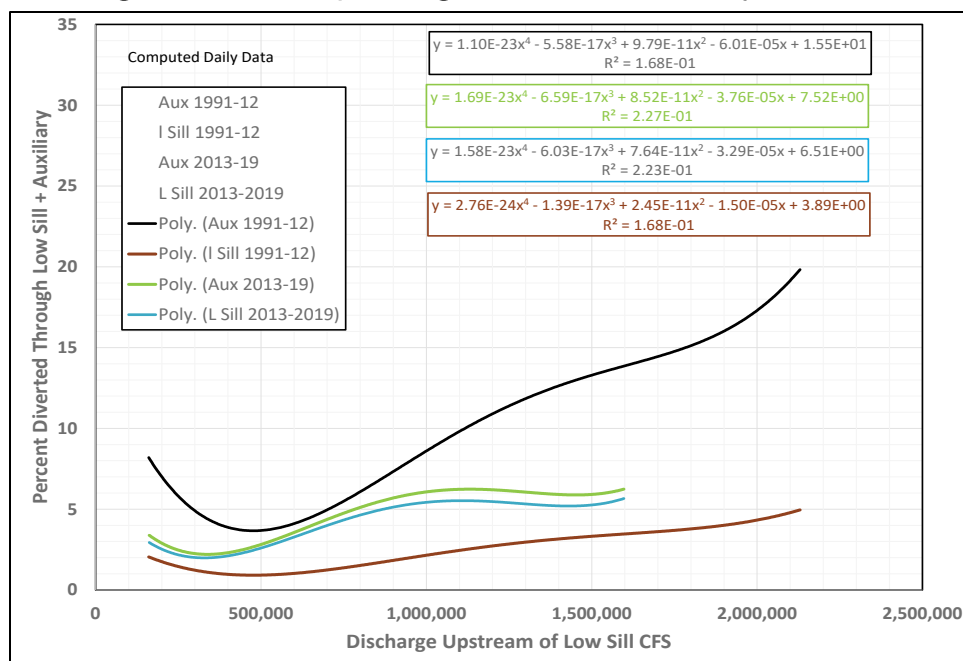


Table 3-4. Percentage of index station flow diverted at Low Sill and Auxiliary.

Discharge, ft ³ /sec Index Station is Upstream from 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

3.5.3 Bohemia Spillway

Diversion percentages at the Bohemia Spillway were taken directly from the Flowline Model. The description of the analytical development of the diversion percentages is included here due to the spillway's proximity to the study area.

The Bohemia Spillway extends for 12 mi along the left descending bank of the Mississippi River between RM 33 and RM 45. It is a constructed feature of the MR&T Project, but it has not been maintained. Flow presently occurs over the spillway during flood flows. The maximum discharge expected in the Mississippi River approaching the Bohemia Spillway is 1,250,000 ft³/sec. The approaching discharge is determined by operations at ORCC, Morganza Spillway, and Bonnet Carré Spillway. Diversion percentages over the Bohemia Spillway were calculated by MVN for the Regional Model Study (Copeland 2020). These percentages were retained in the Flowline Model and in this study.

Diversion percentages were calculated using the weir equation USACE (1994).

$$Q = CLH^{3/2},$$

where

Q = discharge in ft³/sec,

C = weir coefficient taken to be 3.0, and

H = weir height.

The weir heights in these calculations were determined by superimposing the left outer bank elevations over water surface profiles determined from stage gage records. The bank elevations were taken from the 1992 Hydrographic Survey. Water surface profiles were determined from recorded stages at West Pointe a la Hache, Port Sulphur, and Empire. The profiles were taken on dates which correspond to discharges of 800,000 ft³/sec, 1,000,000 ft³/sec, and 1,250,000 ft³/sec at Tarbert Landing. It was determined that flow over the spillway commenced when the Mississippi River flow exceeded 927,000 ft³/sec. The calculated discharge for the project flood, 1,250,000 ft³/sec, was approximately 50,000 ft³/sec or 4% of the river discharge approaching RM 45.

There are four cross-sections in the HEC-6T model for the spillway. These cross sections extend from RM 33 to RM 45. The 50,000 ft³/sec was divided into four 1.0% diversions—one increment for each of the four cross sections. The index station was upstream from the spillway. The locations of the Bohemia Spillway and the distributaries at Ostrica and Fort Saint Philip are shown in Figure 3-9.

Figure 3-9. Location of Bohemia, Ostrica, and Fort Saint Philip distributaries (map data: Google Earth, Image Landsat/Copernicus, 12/2017).



3.5.4 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 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 ft³/sec. These measurements were based on recent measurements, including comparisons of 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 ft³/sec. Without any other available data, the diversion percentage was held constant for discharges exceeding 1,250,000 ft³/sec. 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 ft³/sec. The rating 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 3-5.

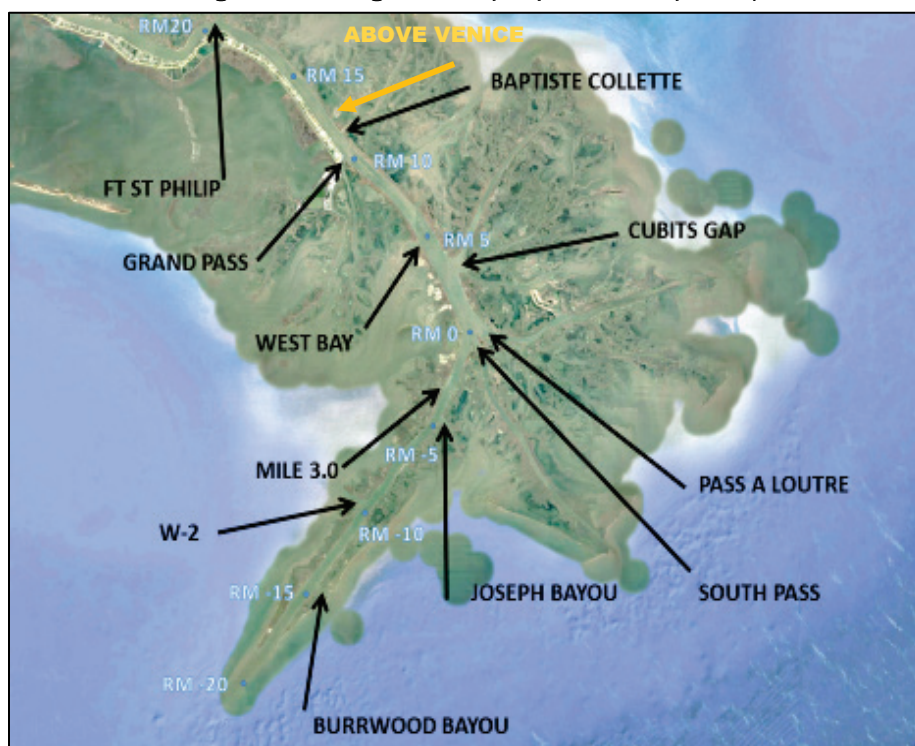
Table 3-5. Distributary outflow at Fort Saint Philip and Ostrica percentage of discharge at Myrtle Grove (RM 57).

Distributary	River Mile	Discharge ft ³ /sec				
		320,000	400,000	600,000	800,000	1,250,000
Fort St. 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%

3.5.5 Flow Distributaries between Above Venice and Head of Passes (HOP)

Distributary flow distributions between HOP at RM 0.0 and the Above Venice discharge range at RM 11.5 are based on measurements taken by MVN between 2004 and 2019. Locations of these distributaries are shown in Figure 3-10.

Figure 3-10. Major distributaries downstream from Above Venice (map data: Google Earth, Image Landsat/Copernicus, 12/2017).

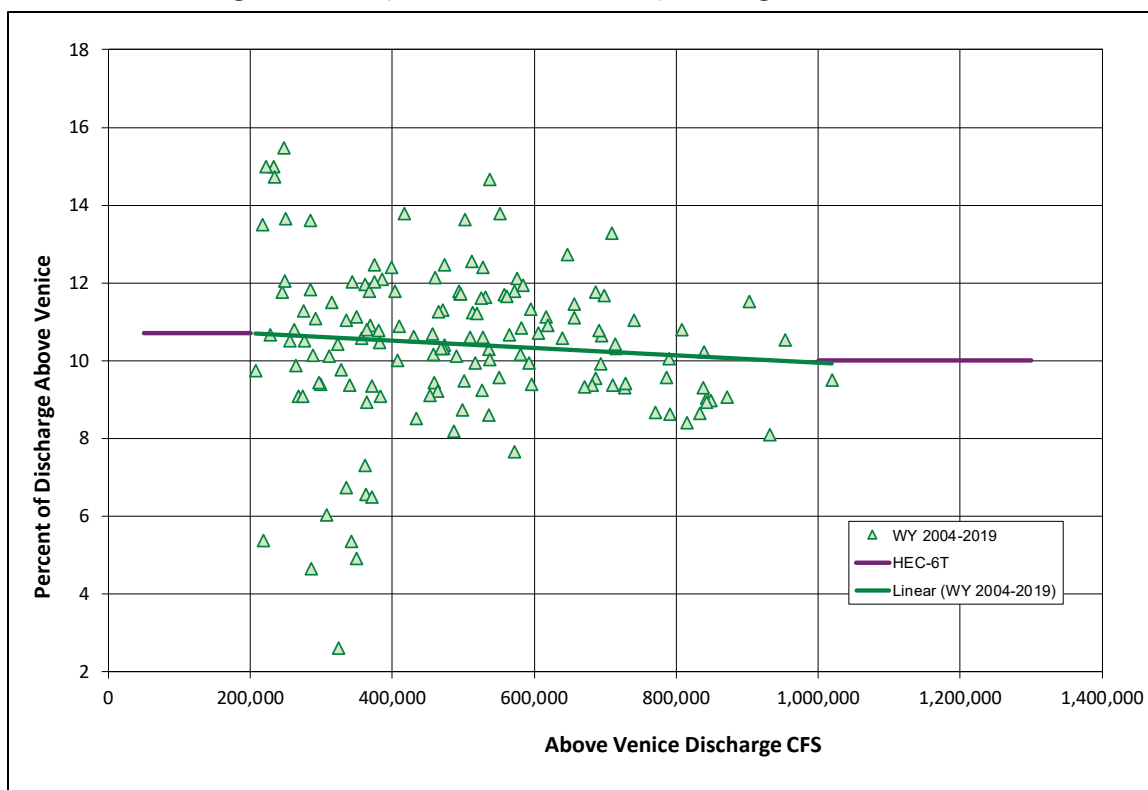


3.5.6 Baptiste Collette

The Baptiste Collette (RM 11.4) flow-diversion percentages were modified from those used in the Flowline Study 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 years (WYs) 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. 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 ft³/sec 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 was used to develop the final flow diversion percentages. Diversion percentages through Baptiste Collette are plotted against the Above Venice discharge in Figure 3-11. This figure also shows the diversion percentages used in the HEC-6T model. At Above Venice discharges less than 200,000 ft³/sec, 10.7% is diverted through Baptiste Collette. At discharges greater than 1,000,000 ft³/sec, 10% is diverted. Percentages are linearly interpolated between 200,000 and 1,000,000 ft³/sec. This new rating curve was used for both the calibration run (1990–2019) and the 50 yr simulation.

Figure 3-11. Baptiste Collette diversion percentages, 2004–2019.

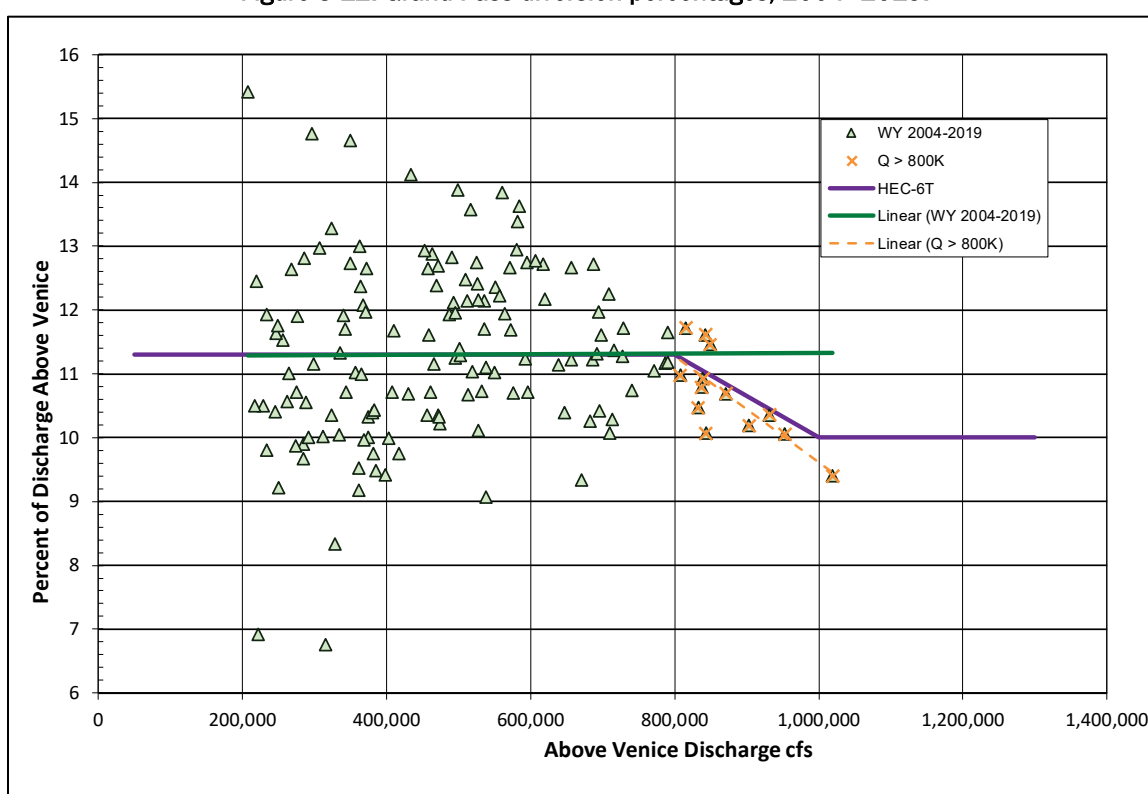


3.5.7 Grand Pass

The Grand Pass (RM 10.4) flow diversion percentages were modified from those used in the Flowline Study based on additional data collected after 2012. The index station is the Above Venice discharge range. A linear regression of all data collected between WYs 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. 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 ft³/sec 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 percentage 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 percentage with discharge when the discharge was less than 800,000 ft³/sec. At discharges

greater than 800,000 ft³/sec, there appeared to be a decrease in the flow diversion percentage as discharge increases. This conclusion is based on only a handful of measurements. Diversion percentages are plotted against discharge in Figure 3-12. This figure also shows the diversion percentages used in the HEC-6T model. At discharges less than 800,000 ft³/sec, 11.3% of the Above Venice discharge is diverted through Grand Pass. At discharges greater than 1,000,000 ft³/sec, 10% of the Above Venice discharge is diverted. Percentages are linearly interpolated between 800,000 and 1,000,000 ft³/sec.

Figure 3-12. Grand Pass diversion percentages, 2004–2019.



3.5.8 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.

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 the West Bay diversion, through Cubits Gap, and passing 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 opened 26 October 2003, and the first available measurement was 16 January 2004. Diverted discharges through West Bay, as a percentage of Mississippi River discharges immediately upstream from West Bay, between WYs 2004 and 2019, are shown in Figure 3-13. 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 WY 2012. Data were insufficient to define a relationship between Mississippi River discharge and the diversion percentage through West Bay prior to WY 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 3-14 suggest that diversion percentages are slightly less at high discharges than at the lower discharges. The 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 3-6.

Figure 3-13. Flow diversion percentages at West Bay, 2004–2019.

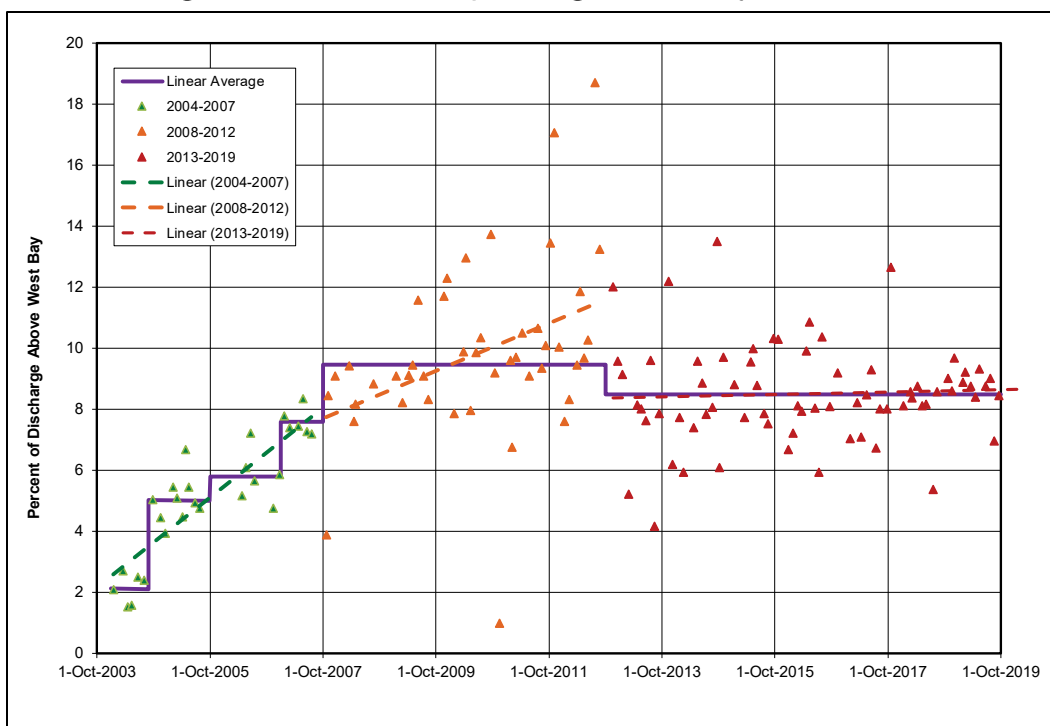


Figure 3-14. West Bay diversion percentages—variation with discharge, 2008–2019.

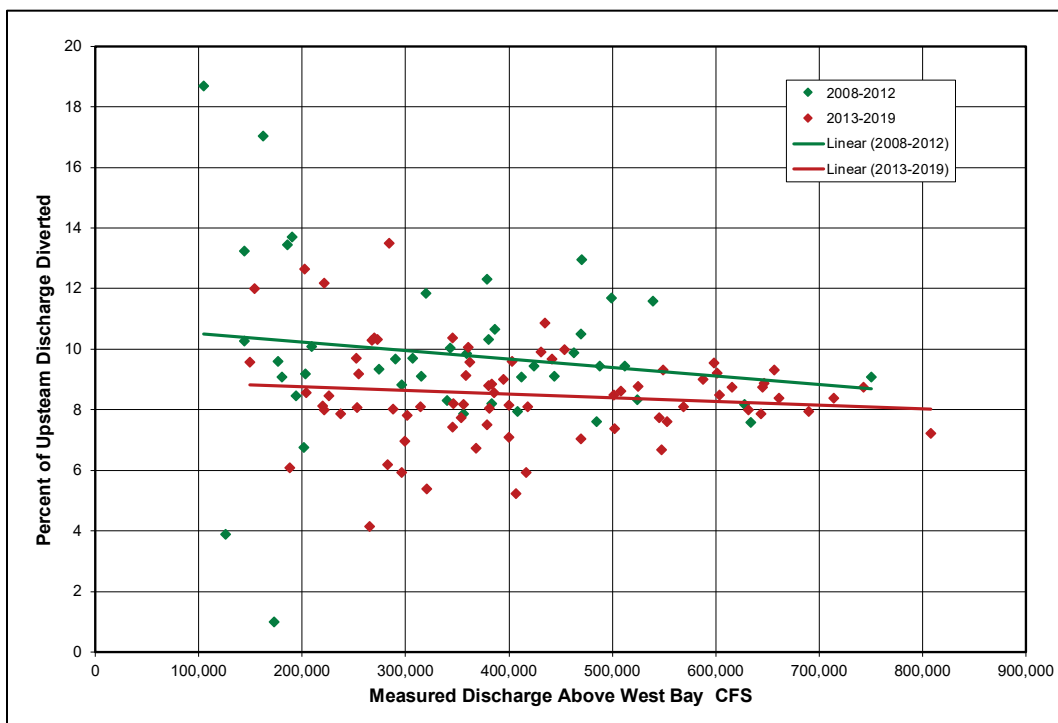


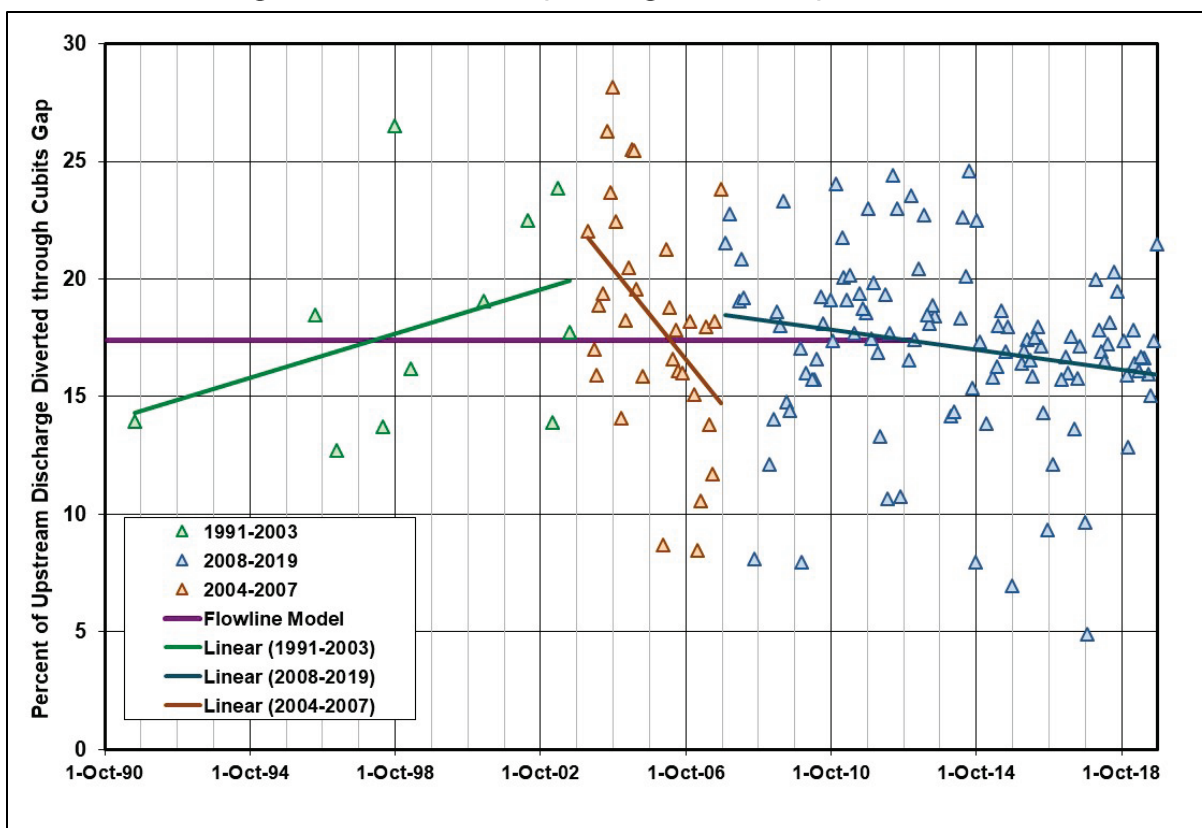
Table 3-6. Diversion percentages used in HEC-6T at West Bay.

Discharge ft ³ /sec	50,000	150,000	800,000	1,500,000
Before 26 October 2003	0	0	0	0
26 October 2003–31 August 2004	2.1	2.1	2.1	2.1
1 September 2004–30 September 2005	5.0	5.0	5.0	5.0
1 October 2005–31 December 2006	5.8	5.8	5.8	5.8
1 January 2007–30 September 2007	7.6	7.6	7.6	7.6
1 October 2007–30 September 2012	10.5	10.5	8.5	8.5
After 1 October 2012	9.0	9.0	8.0	8.0

3.5.9 Cubits Gap

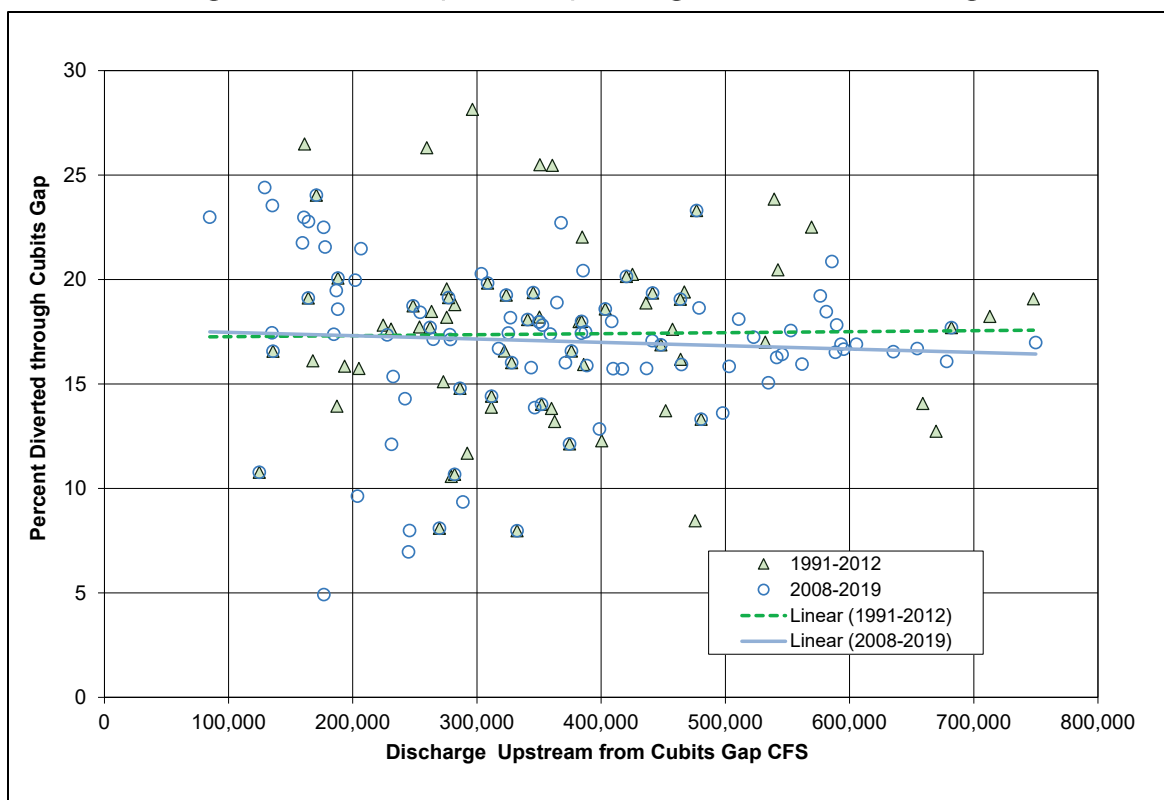
The discharge diversion percentage through Cubits Gap may have varied somewhat with time (Figure 3-15). Between 1991 and 2003, an average linear regression line suggests that diversion percentages were increasing. However, there were only 11 measurements during these 13 yr, and data scatter was significant. West Bay diversion, which is located on the opposite bank just upstream from Cubits Gap, was opened in WY 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 WY 2008, the diversion percentages at Cubits Gap tended to be more stable with a slight downward trend. The Flowline Model (Copeland 2018), 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 this study did not change that conclusion.

Figure 3-15. Flow diversion percentage at Cubits Gap, 1991–2019.



The measured diversion percentages at Cubits Gap, as a function of discharge approaching Cubits Gap, are plotted in Figure 3-16. 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 ft³/sec, and 16.4% was assigned for discharges greater than 750,000. Linear interpolation was used between these two values. The 50 yr future simulations used the post 2008 rating curve.

Figure 3-16. Cubits Gap diversion percentages–variation with discharge.



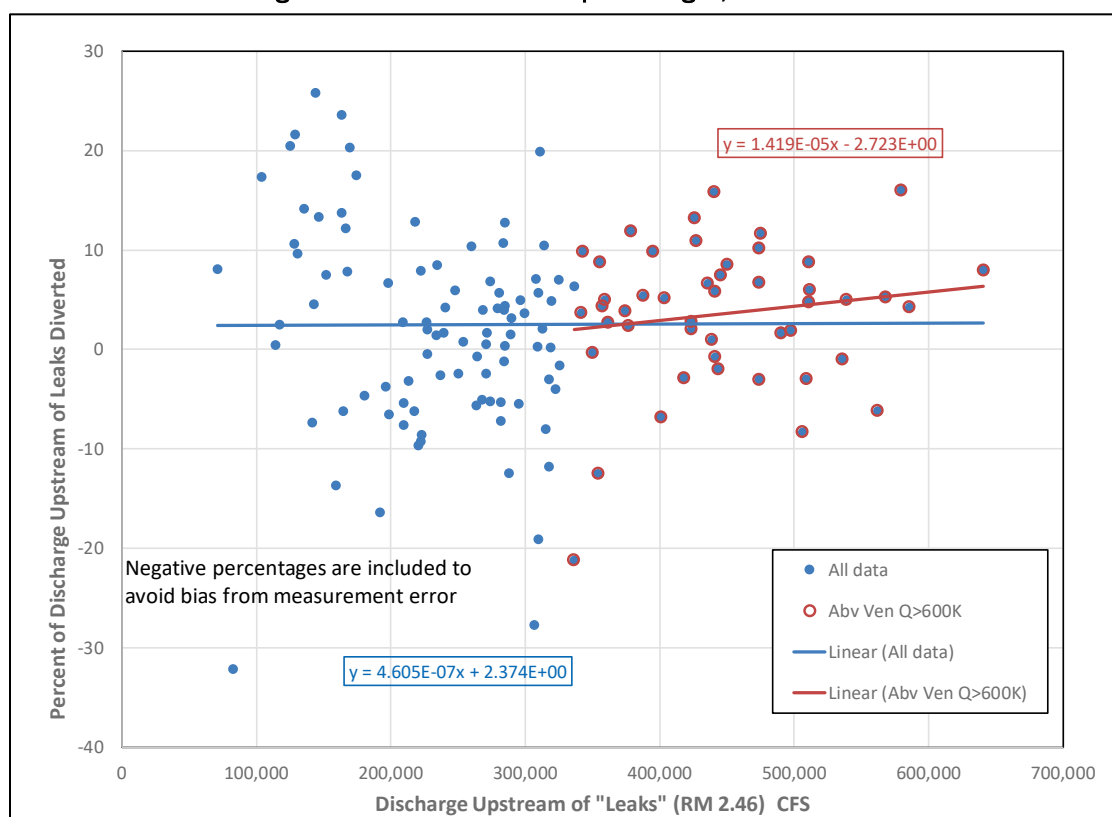
3.5.10 Overbank and Minor Outlets between Venice and Pilottown

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 rating curve used in the most recent HEC-6T studies considered zero outflow through Leaks until the upstream discharge reached 600,000 ft³/sec. The diversion percentage increased linearly to 3.3% at 800,000 ft³/sec and 10.9% at 1,000,000 and above.

In this study, data between 2004 and 2019 were used to develop a new Leaks rating curve. 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 ft³/sec, increasing linearly to 6.5% at a discharge of 650,000 ft³/sec and beyond (Figure 3-17). The upstream cross section is the index station for Leaks.

Figure 3-17. Leaks diversion percentages, 2004–2019.



3.5.11 South Pass and Pass a Loutre (PAL)

MVN has been collecting sets of discharge measurements between HOP 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 PAL, South Pass, SWP, Cubits Gap, West Bay, Grand Pass, and Baptiste Collette.

HEC-6T model runs that include the historical hydrograph (1991–2019), used the Flowline Model distribution percentages at HOP. These account for the increasing flow through the West Bay diversion with time.

For future runs, flow distributions at HOP were based on recent MVN measurements from WY 2008–2019. WY 2008 was chosen as the starting

point because by that time the West Bay diversion appeared to have stabilized in terms of diversion percentage (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 ft³/sec, the average percentages through each of the three distributaries were essentially constant (within 3%). At lower flows the discharge through SWP decreases dramatically and the diversion percentages increase in PAL and South Pass. The Pilottown discharge measurements ranged between 80,000 and 600,000 ft³/sec. Distributary diversion percentages are shown in Table 3-7. At discharges beyond the range of discharges shown in the table distribution percentages were held constant (i.e., data were not extrapolated).

Table 3-7. 2008–2019 HEC-6T distributary distribution percentages at Head of Passes (HOP).

Discharge at Pilottown RM 2.0 (ft ³ /sec)	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

3.5.12 Southwest Pass (SWP) Distributaries

The flow diversion percentages in the SWP distributaries remain unchanged from the original Regional Model. The flow percentage in Burrwood Bayou was based on nine measurements taken in 2003–2005. The data showed no apparent trend between the diversion percentage and the SWP 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). SWP diversion percentages used in the Regional Model and this model are shown in Table 3-8.

Table 3-8. Outflow from SWP distributaries percent of discharge immediately upstream from diversion.

Distributary	River Mile	Discharge Upstream from Distributary (ft ³ /sec)			
		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 W-1	–3.0	4%	4%	4%	4%

3.6 Temperature

Water temperatures in this model were 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 3-9. 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, Illinois (USGS 07022000) 1973–2013
- Ohio River at Dam 53 near Grand Chain, Illinois (USGS 03612500) 1973–2013
- Obion River at Highway 51 near Obion, Tennessee (USGS 07026040) 1990–2005
- Obion River at Obion, Tennessee (USGS 07026000) 1975–1990
- Hatchie River at Rialto, Tennessee (USGS 07030050) 1977–2008
- Hatchie River at Bolivar, Tennessee (USGS 07029500) 1975–2006
- St. Francis River at Madison, Arkansas (USGS 07047907) 1977–2009
- White River at Clarendon, Arkansas (USGS 07077800) 1965–1986
- White River at DeValls Bluff, Arkansas (USGS 07077000) 1967–2014
- Arkansas River at Murray Dam near Little Rock, Arkansas (USGS 07263450) 1975–1994
- Arkansas River at David D Terry Lock and Dam below Little Rock, Arkansas (USGS 07263620) 1969–2014
- Arkansas River at Dam No. 2 near Gillett, Arkansas (USGS 07265283) 1972–1994
- Yazoo River at Redwood, Mississippi (USGS 07288800) 1972–1993
- Yazoo River below Steele Bayou near Long Lake, Mississippi (USGS 07288955) 1994–2014

Table 3-9. Average monthly water temperature 1965–2014, °F.

	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
St. Francis	40.8	43.3	52.4	60.7	68.9	77.5	82.9	82.2	76.9	63.8	54.9	45.7
White	41.8	45.5	53.3	62.8	69.3	75.9	78.4	78.2	74.3	64.3	55.3	46.6
Arkansas	42.5	44.1	52.7	61.1	71.1	79.4	85.2	85.2	79.4	68.7	57.7	48.6
Yazoo	46.5	48.8	56.8	66.0	74.3	81.3	85.3	86.2	81.1	70.4	60.0	51.2

3.7 Downstream Water-Surface Elevations

The NOAA tide gage at Grand Isle, Louisiana (8761724) was used to set the downstream water-surface elevation for the model study. Consistent with previous studies, the downstream water-surface elevation was set to reflect seasonal variation in the mean tide elevation in the Gulf of Mexico. Mean monthly tide elevations were obtained from the National Oceanic and Atmospheric Administration (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 or subsidence effects, or both, 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 3-18. 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 3-10. NOAA water-surface elevations relate to mean sea level, which was translated to NGVD29 by adding 0.8 ft.

Figure 3-18. Mean monthly tide elevations at Grand Isle, Louisiana 1978–2014.

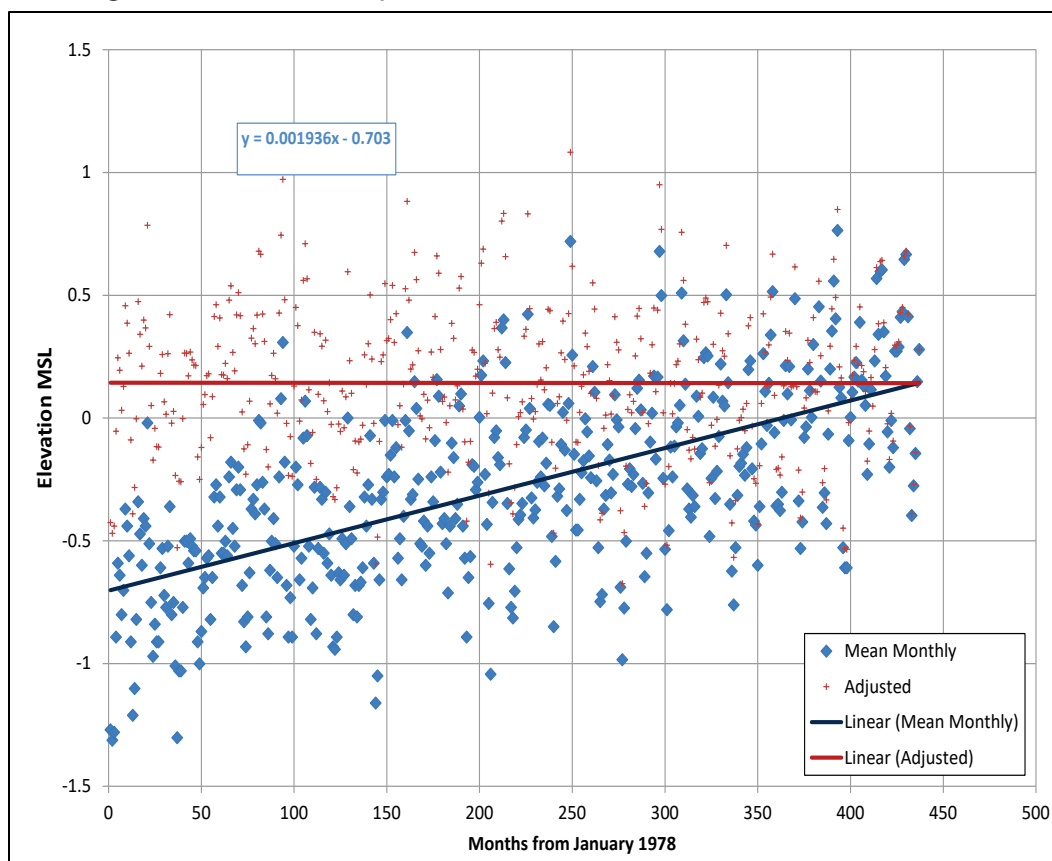


Table 3-10. Downstream water-surface elevations.

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

*National Geodetic Vertical Datum

Sea level rise and subsidence were not included in the 1991–2019 historical simulations in this model study because change in sea level was accounted for in the methodology used to determine average monthly stages. 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 3-10.

3.8 Roughness Coefficients

Calibration of roughness coefficients assigned in the Flowline Model included stage-discharge gage data from the 2011 flood. However, in the 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 recalibrate roughness coefficients. The differences from previous assignments, which had used 1991–2002 data, were small. The new roughness coefficients for the MVN used in this study are shown in Table 3-11. 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).

Table 3-11. HEC-6T roughness coefficients in MVN.

River Mile	Overbanks	Channel–1000 ft ³ /sec				
		70	120	180	240	300
–22.0	0.03	0.038	0.028	0.022	0.022	0.018
		70	120	180	240	300
–18.0	0.05	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
		200	650	850	950	1,300
50.2	0.20	0.026	0.024	0.024	0.023	0.0185
		275	650	950	1,200	1,500
105.0	0.20	0.029	0.026	0.025	0.0215	0.021
		275	550	950	1,200	1,500
141.6	0.20	0.027	0.027	0.024	0.023	0.022
177.3	0.20	0.028	0.028	0.025	0.024	0.022
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

3.9 Sand Transport Function

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

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

3.10 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 silt and clay erosion variable names in HEC-6T are *STCD* (Shear threshold for erosion of silt and clay particles), *STME* (Shear threshold for mass erosion), *ERME* (Erosion rate at *STME*), and *ER2* (Slope of the erosion rate curve for mass erosion). The Ariathurai and Krone equation for particle erosion is

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

where

$$M_1 = \text{erosion rate for particle scour} \left[\tau_b * ERME / (STME - STCD) \right] \\ (STCD < \tau_b < STME),$$

S_a = surface area exposed to scour,
 Q = water discharge,
 τ_s = critical bed shear stress for particle scour (STCD), and
 γ = specific weight of water.

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

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

where

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

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

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

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 $DTSL$), and
 D = water depth.

Silt and clay deposition coefficients were varied in the model. One set of coefficients was used in SWP, another in the reach between HOP and RM 3.0, and another upstream from RM 3.0. 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 3-12.

The deposition coefficients for SWP and upstream from RM 3.0 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 AHP. More recent bed-material measurements have established that shoaling upstream from HOP consists primarily of sand, so silt deposition in that reach was reduced for this study.

Table 3-12. Silt and clay coefficients for Parthenaides and Krone equations.

Coefficient	Variable Name in HEC-6T	Southwest Pass	RM 0.0 to RM 3.0	Fresh Water	Units
Shear threshold for clay deposition	DTCL	0.012	0.008	0.001	lb/ft ²
Shear threshold for silt deposition	DTSL	0.035	0.007	0.002	lb/ft ²
Shear threshold for erosion of silt and clay particles	STCD	0.040	0.025	0.025	lb/ft ²
Shear threshold for mass erosion	STME	0.04	0.04	0.04	lb/ft ²
Erosion rate at STME	ERME	0.001	0.001	0.001	lb/ft ² /hr
Slope of the erosion rate curve for mass erosion	ER2	0.03	0.03	0.03	1/hr

3.11 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 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 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. A shorter record is available at Belle Chasse, located 76 mi AHP. Data for the major tributaries, including the Ohio River, are generally limited to the sediment concentrations greater and less than 0.062 mm. The lack of boundary condition data required that size class percentages be estimated by calculation or judgment, or both.

The relative importance of boundary condition precision at each model boundary is related to the contribution of water and sediment from that tributary. The contributory percentage to the annual Mississippi River discharge above ORCC for the years 1991–2002 is shown in Table 3-13. Over 80% of the annual discharge above ORCC comes from the Ohio and Middle Mississippi Rivers. Boundary conditions for these two rivers are by far the most important in the model study.

Table 3-13. Average annual contributory percentage upstream of ORCC.

	HEC-6T Entry Mile	Percentage
Ohio River	953.0	45.1
Middle Mississippi River	953.0	37.4
Arkansas River	580.8	8.2
White River	598.0	4.4
Yazoo River	437.3	3.0
St. Francis River	672.6	0.8
Hatchie River	772.8	0.6
Obion River	819.4	0.5

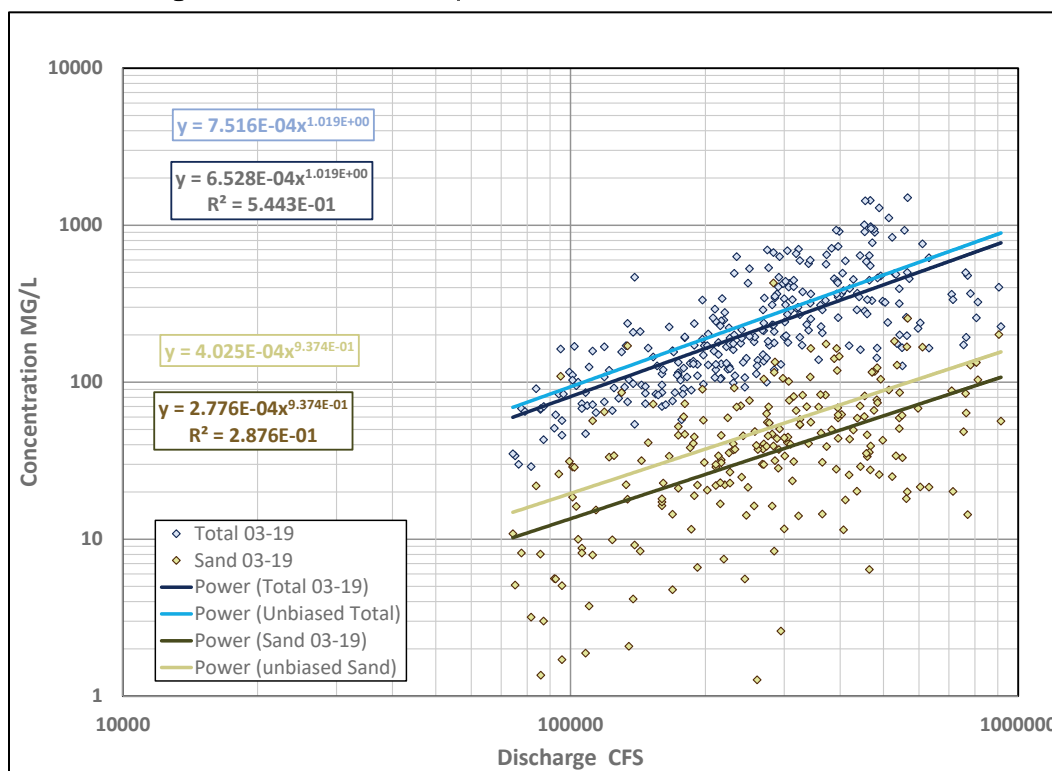
After completion of the Flowline Model, additional data became available for the Middle Mississippi, Ohio, White, Arkansas, and Yazoo Rivers. New sediment-inflow rating curves were developed for these five rivers. (More detailed information on development of the sediment inflow rating curves for the other tributaries can be found in the Flowline Model report [Copeland 2018].)

3.11.1 Middle Mississippi River

Suspended sediment data are available for the Middle Mississippi River at Thebes, Illinois (07022000), which is located 43.7 mi upstream from the Ohio River. A significant decrease in sand concentrations at Thebes

occurred after 2002, and the sediment inflow rating curve was 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 this model study (Figure 3-19). Size class data were collected at Thebes between 1980 and 1991. Since these are the only available size class data at Thebes, these continued to be used in the 2003–2019 sediment input file (Table 3-14).

Figure 3-19. Measured suspended sediment at Thebes 2003–2019.



Sediment inflow trends need to be continually monitored to see if the recent trend of decreased concentration is long term or the result of response to recent hydrologic conditions.

Sediment inflow for the sand size classes determined from regression curves were increased by 15% to account for unmeasured load. The unmeasured load has been historically estimated to be 15% of the total measured load. This approximation was first suggested by Dr. Hans Einstein in a personal communication in the 1960s. Based on subsequent studies and investigations, the New Orleans District concluded that the 15% estimate was appropriate (USACE MVN 1980, 15). Numerical sediment studies conducted on the Lower Mississippi River increased the

measured bed-material load by 10% to 15% to account for unmeasured load at upstream model boundary (Copeland 1991; Copeland and Thomas 1992; Catalyst-Old River Hydroelectric 1999; Thomas 2012; Sharp et al. 2013). Uncertainty related to the unmeasured load decreases as the distance between the area of interest and the upstream boundary increases. This occurs because the numerical model calculates the total bed-material load, including the unmeasured load. As the sediment calculations move in a downstream direction, the sediment continuity equation is solved, extracting material from the streambed as required to meet the total bed-material load transport capacity subject to hydraulic sorting and armoring. However, the uncertainty increases as the simulation time increases.

Recent development of the ISSDOTv2 methodology (Abraham et al. 2011) has made it possible to physically measure the bed-form component of bed load by mapping the movement of dunes using sonar equipment. Measurements taken in the Mississippi River in the vicinity of the ORCC (Heath et al. 2015) demonstrated that the bed load percentage of the bed-material load varied with discharge. The bed load percentage of the bed-material load varied from highs near 45% to low values of approximately 10%. More bed load measurements using this technique would certainly reduce uncertainty associated with the significance of the unmeasured component of the sediment load.

Table 3-14. Years 2003–2019 sediment inflow at Thebes–Segment 25.

Discharge ft ³ /sec	10,000	70,000	100,000	300,000	500,000	900,000	1,200,000
Concentration mg/L	65.03	65.03	93.54	286.53	482.21	877.73	877.73
Fractions							
Clay	0.4011	0.4011	0.4011	0.4051	0.4090	0.4150	0.4150
Very Fine Silt	0.0733	0.0733	0.0733	0.0669	0.0605	0.0510	0.0510
Fine Silt	0.0733	0.0733	0.0733	0.0669	0.0605	0.0510	0.0510
Medium Silt	0.0861	0.0861	0.0861	0.0911	0.0961	0.1035	0.1035
Coarse Silt	0.0861	0.0861	0.0861	0.0911	0.0961	0.1035	0.1035
Very Fine Sand	0.0397	0.0397	0.0397	0.0403	0.0410	0.0419	0.0419
Fine Sand	0.2155	0.2155	0.2155	0.2011	0.1868	0.1653	0.1653
Medium Sand	0.0578	0.0578	0.0578	0.0626	0.0674	0.0746	0.0746
Coarse Sand	0.0031	0.0031	0.0031	0.0106	0.0180	0.0292	0.0292
Very Coarse Sand	0.0000	0.0000	0.0008	0.0016	0.0025	0.0039	0.0039
Very Fine Gravel	0.0000	0.0000	0.0003	0.0006	0.0010	0.0015	0.0015
Fine Gravel	0.0000	0.0000	0.0001	0.0002	0.0003	0.0005	0.0005
Medium Gravel	0.0000	0.0000	0.0000	0.0001	0.0001	0.0002	0.0002
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

3.11.2 Ohio River

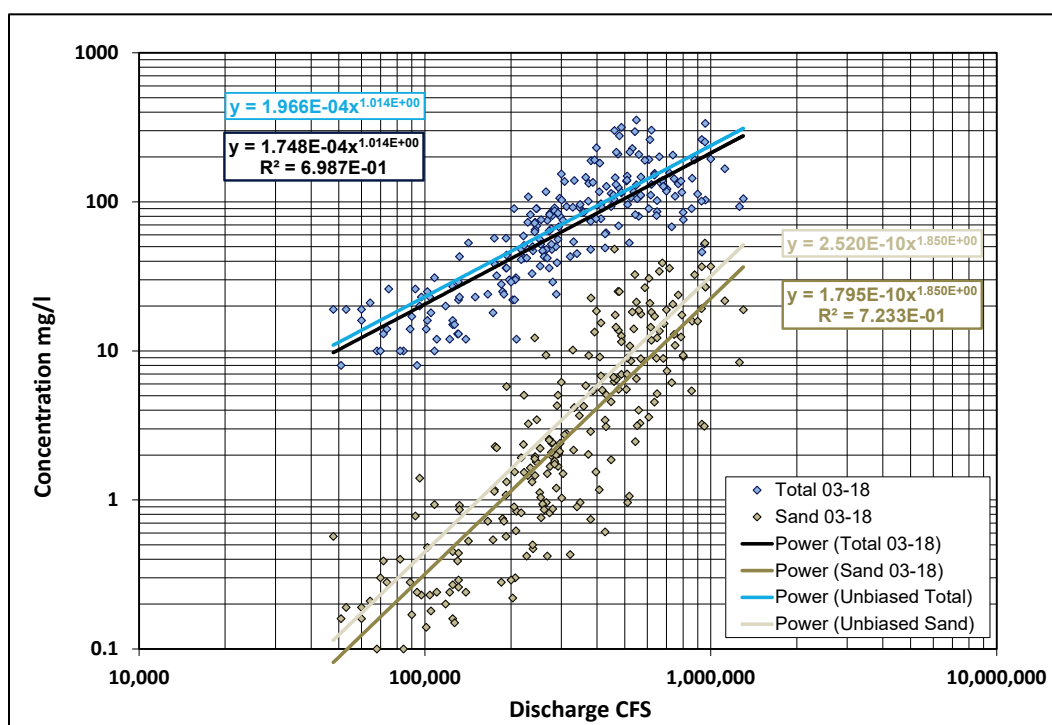
Total sediment and sand concentrations from the Ohio River were determined from the USGS gage at Dam 53 near Grand Chain, Illinois (03612500), and from the USGS gage at Olmsted, Illinois (03612600), which replaced the Dam 53 gage in December 2014. Sediment concentration data were available at Dam 53 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, Kentucky (03294500). This gage is located approximately 372 mi upstream from Cairo (Ohio River RM 607.3). Size class percentage 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. 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.

For this study, 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 3-20). 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.

Figure 3-20. Measured suspended sediment concentrations Dam 53 and Olmsted, 2003–2019.



The sediment inflow rating curves used in the HEC-6T model, for the Ohio River, are shown in Table 3-15. 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 sand size classes determined from regression curves were 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.

Table 3-15. Sediment inflow for Ohio River–Segment 24.

Discharge 1000 ft ³ /sec	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

3.11.3 Obion and Hatchie Rivers

There were no additional data available for the Obion and Hatchie Rivers since completion of the Flowline Model. Sediment inflow rating curves

from the Flowline Model were used in this study. The Obion River rating curve was based on data from 1975 to 1995. The Hatchie River rating curve was based on data from 1977 to 1995 and 2004 to 2008.

3.11.4 Saint Francis River

There were no additional data available for the Saint Francis River since completion of the Flowline Model. Sediment inflow rating curves from the Flowline Model were used in this study. The Saint Francis River rating curve was based on data from 1977 to 2009.

3.11.5 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, Arkansas (07077000), were used to determine sediment inflow concentrations for this study. This gage is located at River Mile 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 sand. However, there were no additional 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 3-21 and Table 3-16. The discharge range used to develop the sediment inflow regression curves was between 7,000 and 90,000 ft³/sec. 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 3-21. Measured suspended sediment concentrations—White River at DeValls Bluff, 2001–2019.

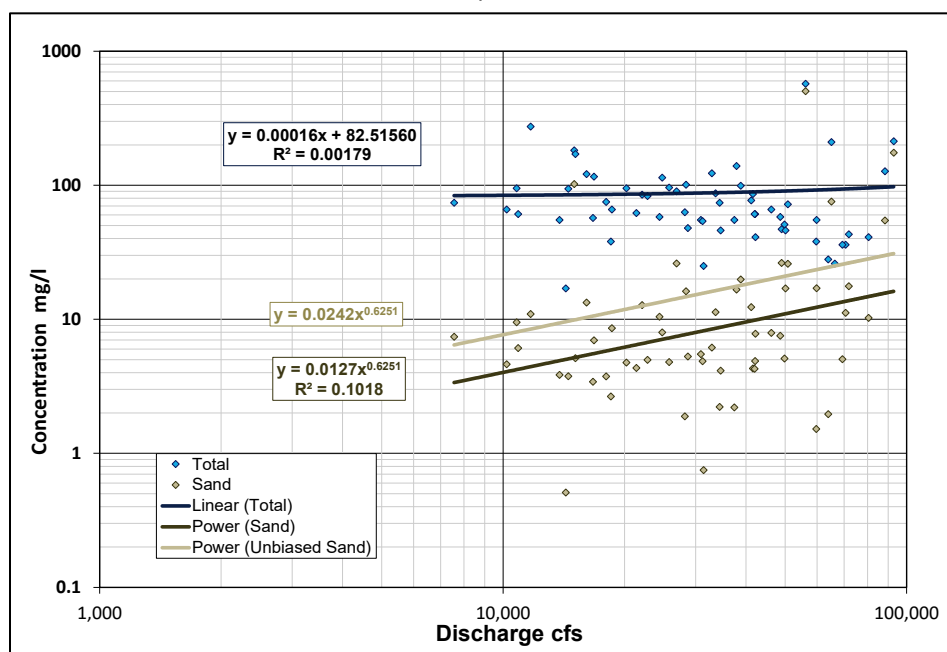


Table 3-16. Sediment inflow for White River—Segment 10.

Discharge ft ³ /sec	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

Table 3-16 (cont.). Sediment inflow for White River–Segment 10.

Discharge ft ³ /sec	100	7,000	10,000	50,000	90,000	300,000
Fractions						
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

3.11.6 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, Arkansas (07263620) were used to determine sediment inflow concentrations for this study. 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 include 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 this study. 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 nonisokinetic 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 percentage 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 Microsoft EXCEL. Fine sand through very coarse sand size class percentages were increased by 15% to account for unmeasured load.

Arkansas River sediment inflow rating curves, for fines and sand, are shown in Figure 3-22 and Figure 3-23. The fine sediment linear regression curve labeled “all” is the one used in the model. The curve labeled “standard” is developed from standardized (isokinetic) samplers. The “all” data include dip samplers, weighted bottles, and grab samples as well as isokinetic samplers. 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 3-17.

Figure 3-22. Measured Fines 1994–2019 Arkansas River at Terry Lock and Dam.

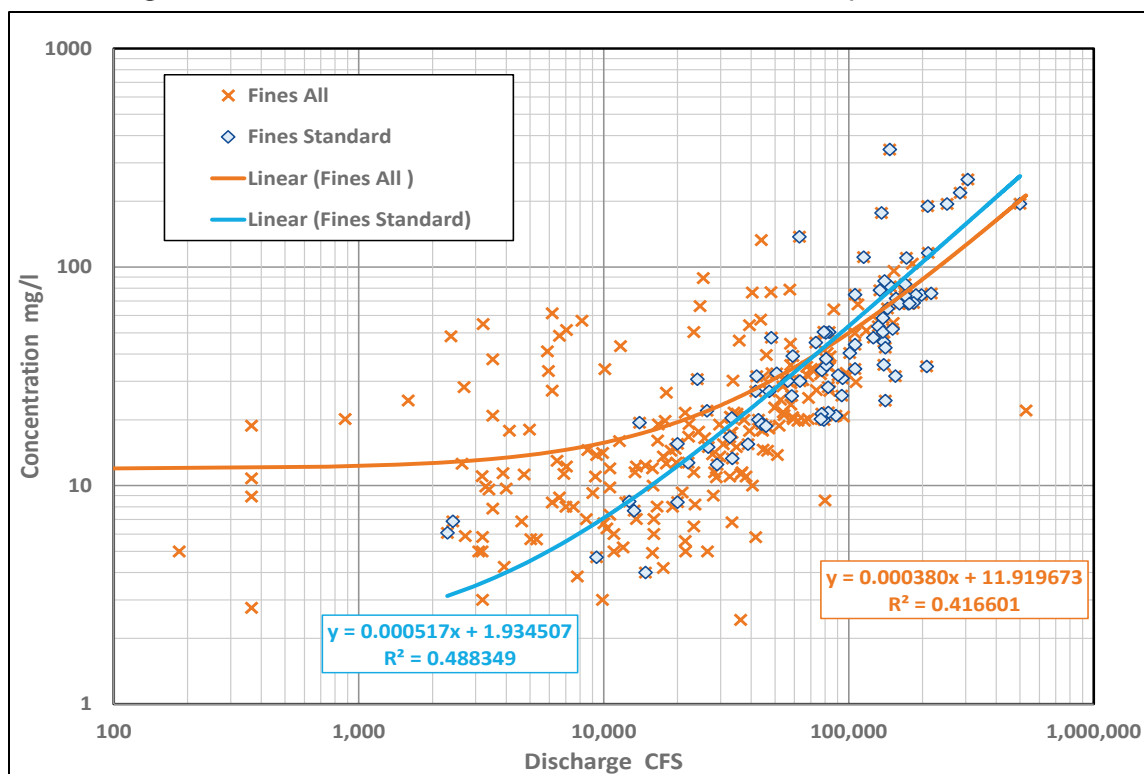


Figure 3-23. Measured Sand 1994–2019 Arkansas River at Terry Lock and Dam.

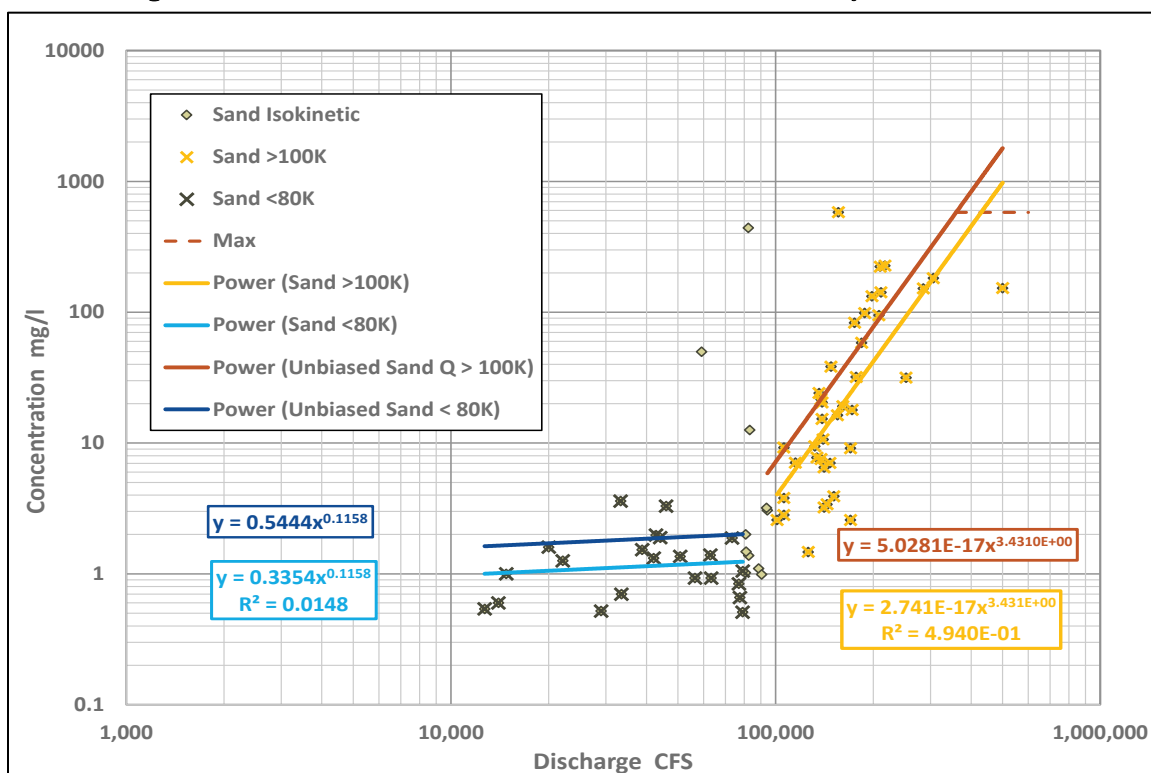


Table 3-17. Sediment inflow for Arkansas River–Segment 8.

Discharge 1000 ft ³ /sec	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
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

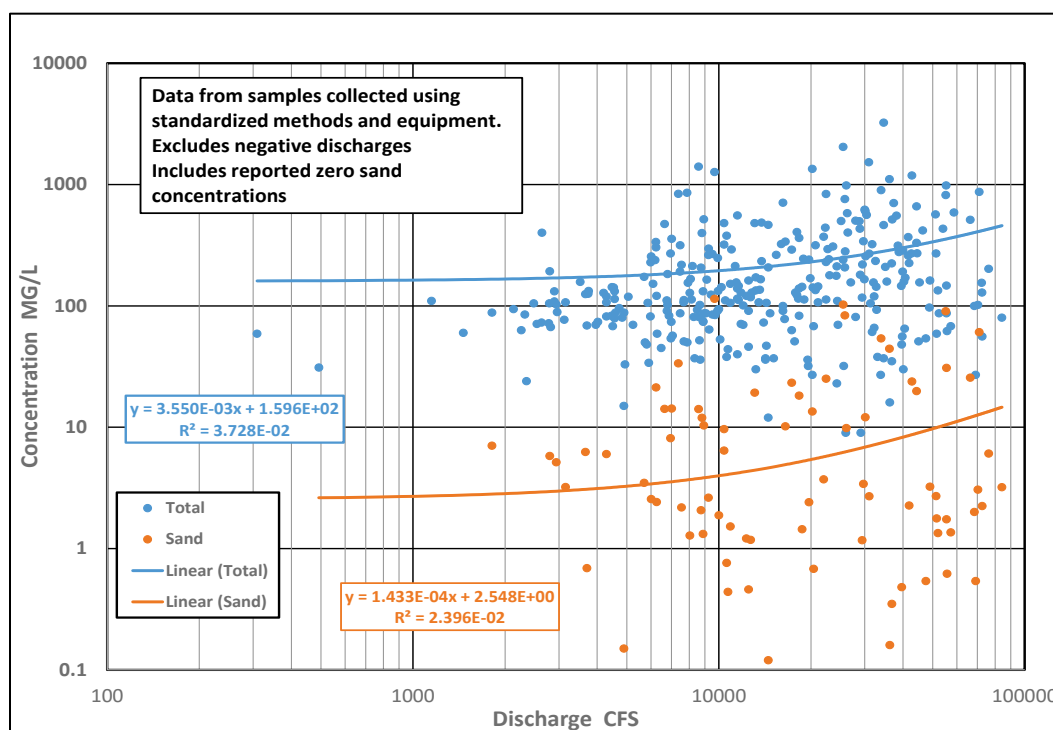
3.11.7 Yazoo River

The purpose of the Yazoo River Segment in the HEC-6T model is to provide sediment inflow to the Mississippi River. Data are insufficient for calibration of the Yazoo River segment and calculated morphologic changes should be treated with caution. 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 include 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 3-24.

Figure 3-24. Measured suspended sediment concentrations Yazoo River at Steele Bayou, 1999–2019.



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 to 2002 data at Coochie and Union Point. These sediment gages are located at Mississippi River 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 3-18.

Table 3-18. Sediment inflow for Yazoo River–Segment 6.

Discharge ft ³ /sec	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

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.

3.12 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 bottom elevation 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.

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 RMs 12.5 and 57 and 5 ft between RMs 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 nondredging 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 New Orleans District between RMs -22.0 and 5.5 and at deep-draft crossings where dredging was calculated during the simulation.

3.13 Bed-Material Gradations

3.13.1 Lower Mississippi River Bed Sampling Programs

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 3-25, where samples were taken from transects at quarter points at a river bend at Bayou Sara (RM 265). In Figure 3-25, 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 HOP 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:

- 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.
- 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 3-26, 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.
- Downstream from Vicksburg the median grain diameter of the bed material was essentially the same in 1989 and 1932.

A comprehensive bed-material sampling program of the Mississippi River that extended from Grafton, Illinois, to HOP, 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 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 3-27.

The initial bed-material gradations in the HEC-6T Model were based on the 1989 Nordin and Queen data rather than the more recent 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.

Figure 3-25. Lateral variation in bed-material gradation at RM 265, August 2005.

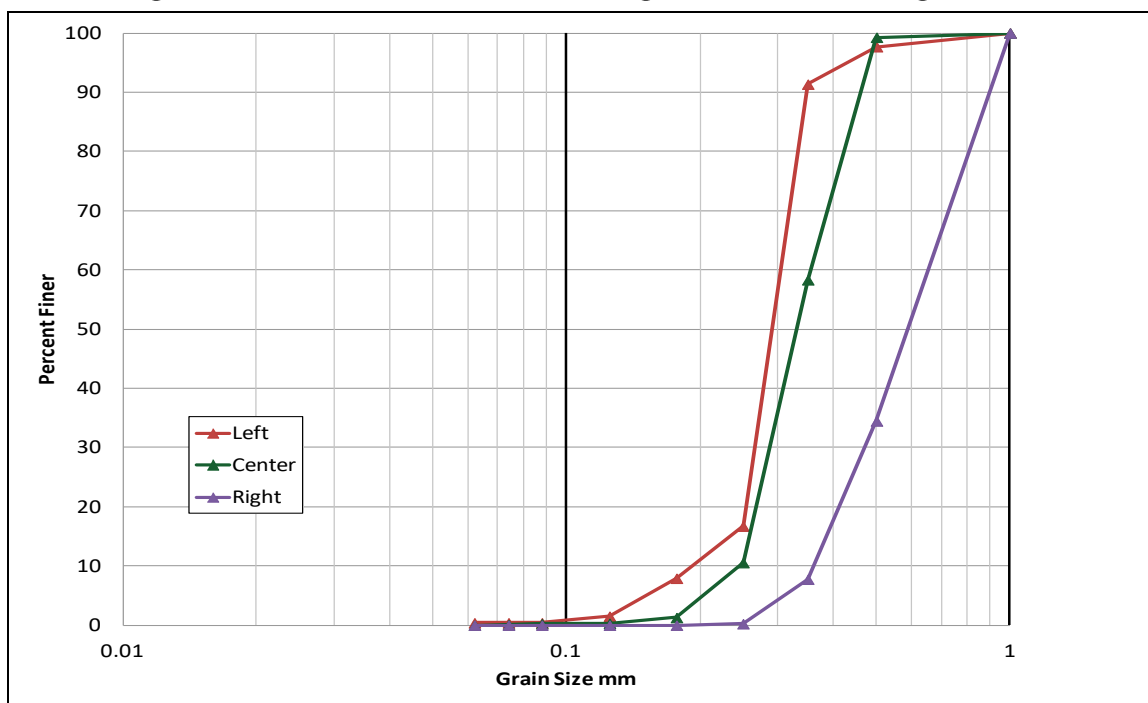


Figure 3-26. Historic change in median bed grain size (Nordin and Queen 1992).

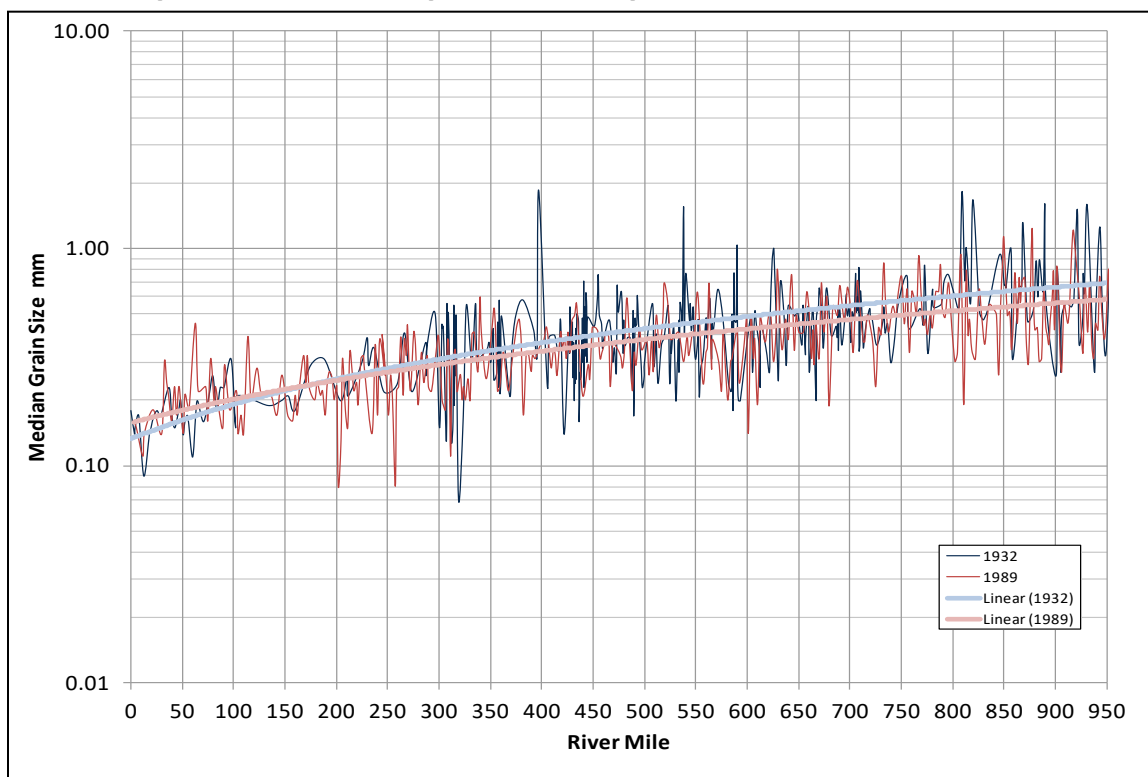
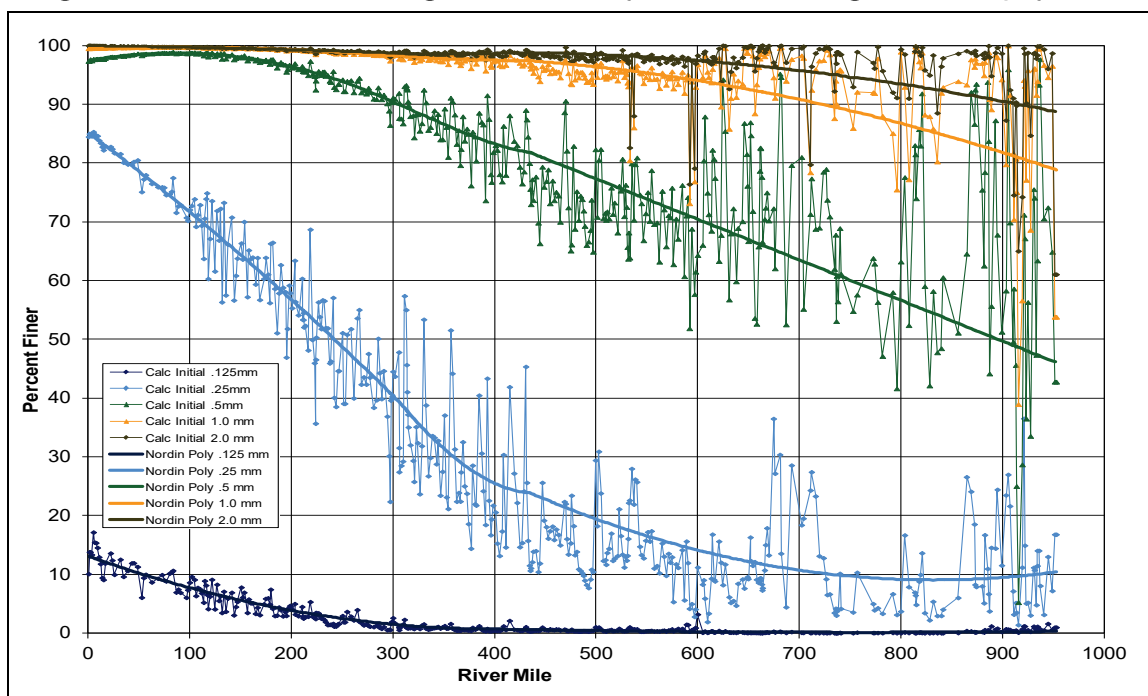


Figure 3-27. Initial calculated bed gradations—30-day simulation, starting with Nordin polynomials.



3.13.2 Bed-Material Sampling Downstream from Venice

Between 2003 and 2006, bed-material samples were collected in SWP, AHP and in the Jetty and Bar Channels downstream from RM -17.8. Samples were collected using a ponar sampler from a dredge. Samples were analyzed using sieve and hydrometer methods. Thirty-four samples were collected between River Miles 0.1 and 3.2 AHP. Of these, 23 had size class analyses performed to at least D15. All but 1 of 34 bed samples had a median grain size greater than 0.0625 mm. Using the Unified Soil Classification System (Sand/Silt break at 0.074 mm), 14 samples were classified as Fine Sand, 2 as Sandy-Silt, 12 as Silty-Sand, 1 as Silt, 1 as Clayey-Sand, 3 as Lean-Clay, and 1 as Fat-Clay.

The median grain size for these samples during WYs 2004 and 2005 are plotted along with the discharge at Tarbert Landing in Figure 3-28. This plot shows the effect of the hydrograph on bed gradation in HOP and SWP. Years 2004 and 2005 were average discharge years in terms of mean daily flow. Year 2004 was below average, and 2005 was above average in terms of annual peak flow. The plot indicates that the dredge material consisted primarily of sand. The dredged material was coarser at higher discharges. Even in the Jetty and Bar Channels, most of the dredged material was sand. Years 2003 through 2006 bed-material gradations

AHP are shown in Figure 3-29. This figure shows most samples consisted mostly of sand. Five had significant silt fractions, but none exhibited the bimodal characteristics of subsequent 2009 sampling.

This result is consistent with Hutchings (1933) who reported that it was the bed load, and not the suspended load, that was the shoaling agent in the SWP bar channel during flood stages (stage >12 ft at Carrollton). He attributed a portion of this bed load deposition to collision with the saltwater wedge. Hutchings also attributed shoaling in the bar channel during the low-water season to littoral drift.

Figure 3-28. Dredge material median grain size, 2004–2005.

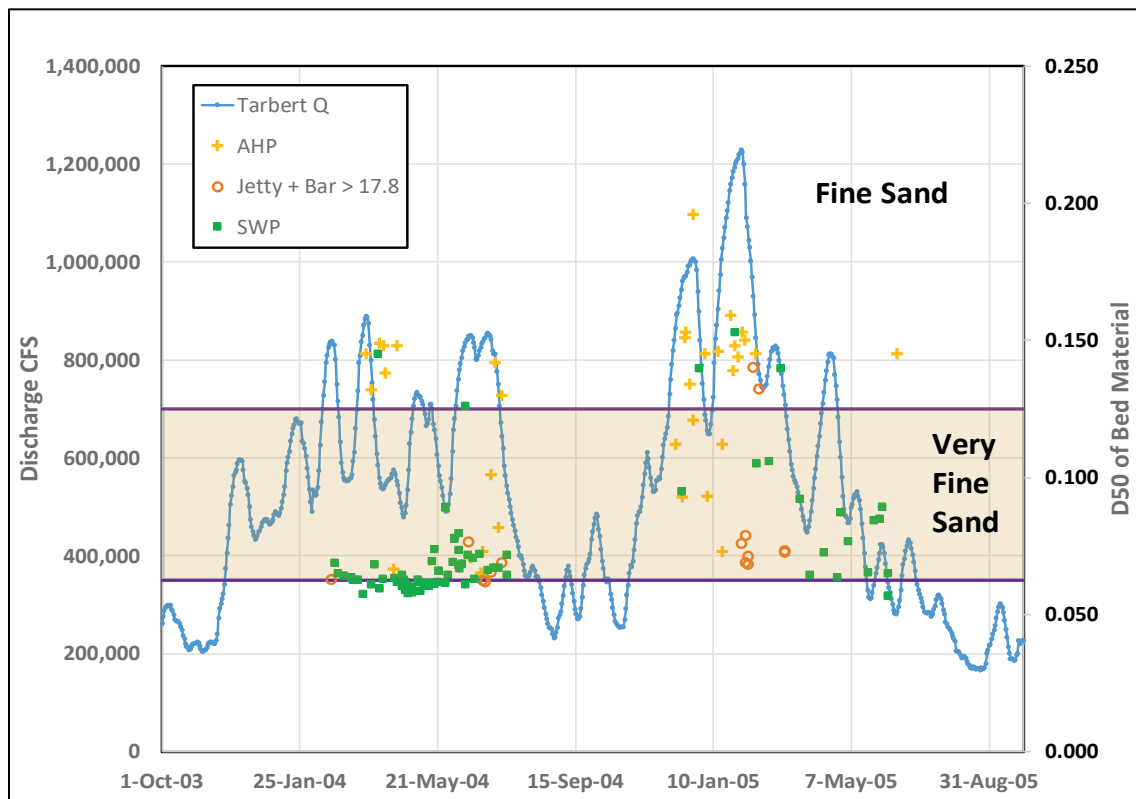
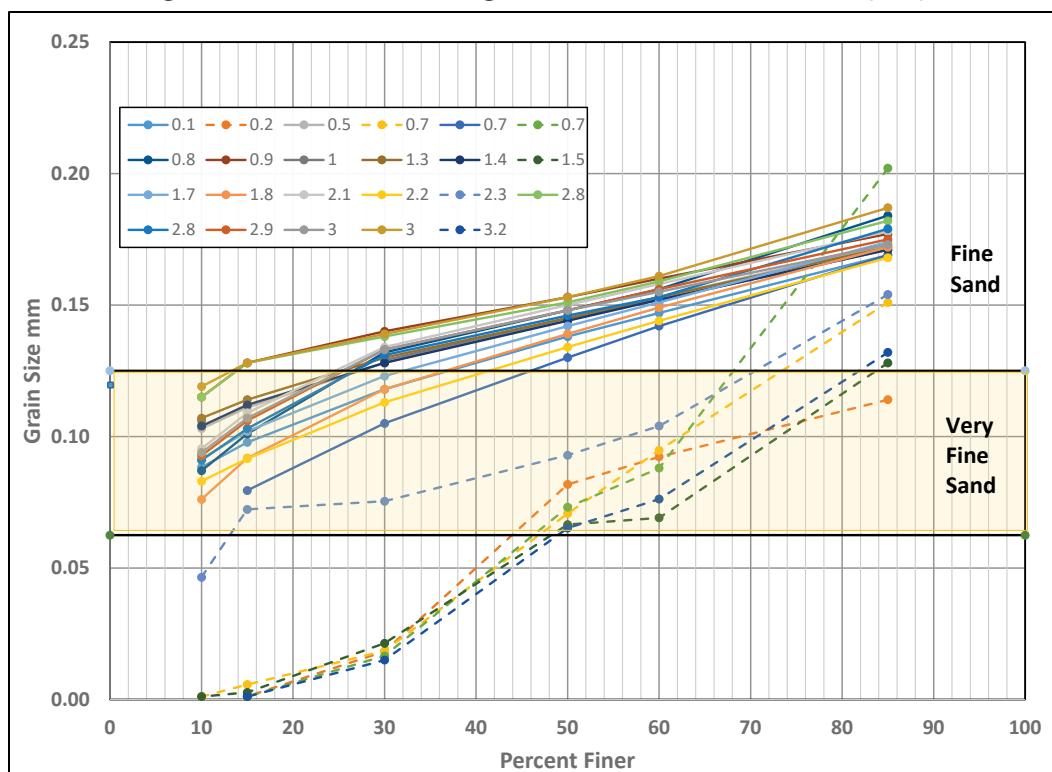


Figure 3-29. 2003–2006 bed gradations above Head of Passes (AHP).



A bed-material sampling program was conducted in 2009, in support of the West Bay Sediment diversion study (Sharp et al. 2013). This sampling program included three trips in March, May, and July 2009. The March data were collected between RM 19.4 and RM 1.6 as the annual hydrograph was rising. The May data were collected at the peak of the annual flood, primarily in SWP between RM 0.9 and RM -19.5. The July data were collected after the annual flood had passed, between RM 12 and RM -17.1. The 2009 discharge hydrograph at Tarbert Landing during the sampling program is shown in Figure 3-30. The hydrograph is capped at 1,250,000 ft³/sec because operation protocols call for a maximum discharge of 1,250,000 ft³/sec below Bonnet Carré Spillway. Samples were collected using a drag bucket. Grain size analyses were conducted using laser diffraction techniques. Median grain sizes from the 2003–2006 and 2009 are compared in Figure 3-31.

Figure 3-30. Discharge at Tarbert Landing during 2009 sampling program.

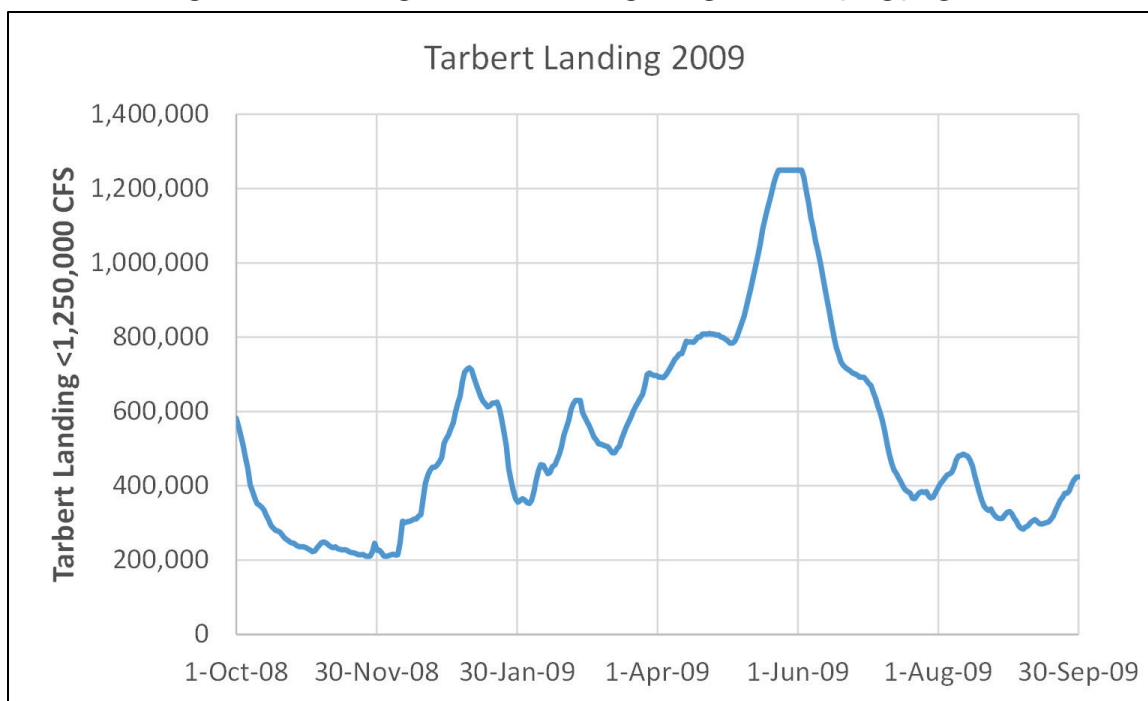
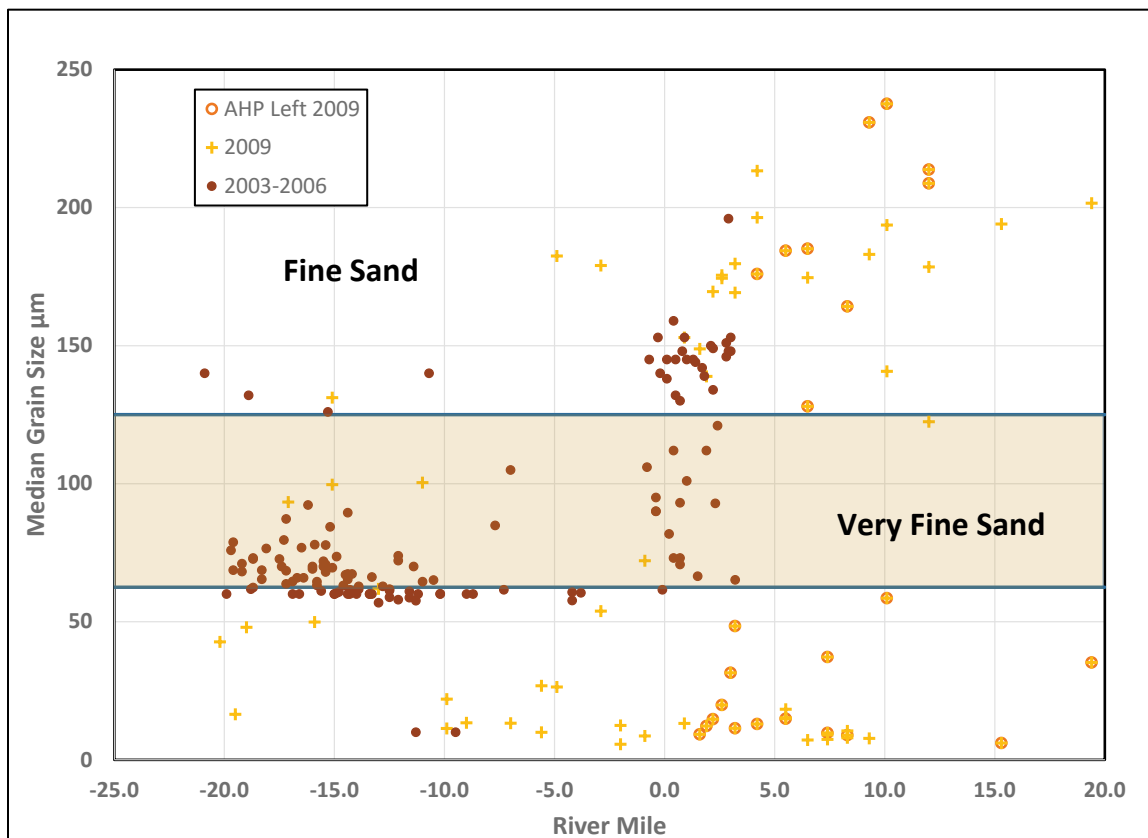


Figure 3-31. Comparison of median grain sizes from 2003–2006 and 2009 sampling.



Bed samples AHP were collected at two verticals in 2009. Interestingly, the samples collected from the alternating bar side of the channel were typically fine sand, and samples from the navigation channel were bimodal or medium silt. This condition was observed in both the March and July samples. This bimodal characteristic of the riverbed makes a simple D50 characterization inadequate for defining sediment transport processes.

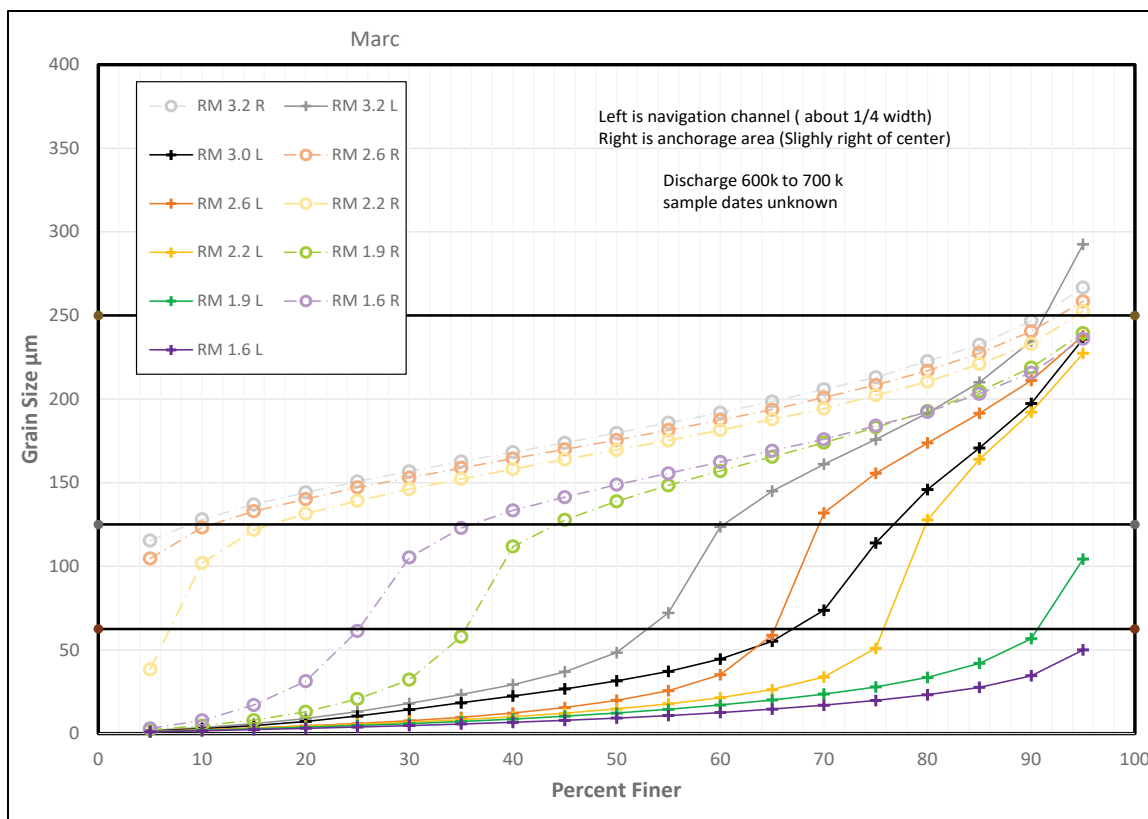
Three bed-material categories were identified in the 2009 data collection program. One was primarily sand, consisting primarily of fine sand. Another category was primarily silt. The third category had a bimodal distribution, where the sample was essentially devoid of very fine sand and coarse silt. The percentage of sand and silt varied widely in the bimodal distributions, with some samples mostly sand and some samples mostly silt. It is interesting that the bimodal characteristic was not identified in the previous 2003–2006 data collection program AHP.

Downstream from Cubits Gap, the right side of the channel is called the anchorage area, which can be characterized as an alternating bar. In March 2009, the anchorage area's first mile downstream from Cubits Gap had a bed composed almost entirely of fine sand. Bimodal distributions occur at RM 1.9 and 1.6, on the alternating bar, with the median grain size in the fine sand range. In the navigation channel, on the left side of the channel, between RM 3.2 and RM 2.2, the bed was characterized by a bimodal distribution. The median grain size in the navigation channel was in the silt range. At RM 1.9 and RM 1.6 the navigation channel bed was composed almost entirely of silt.

The navigation channel, which is the deepest portion of the channel, would be expected to have the highest bed shear stress and therefore the coarsest bed. However, this is a dredged reach and the navigation channel is most likely not in equilibrium and would have a natural tendency toward deposition due to dredging operations that consistently disturb equilibrium by removing bed material. However, the paucity of very fine sand and coarse silt in the bimodal deposits suggests that deposition is not the dominant process. It is possible that the navigation channel has been dredged into an old geologic stratum of consolidated cohesive material and that is what is being sampled by the drag sampler. The existence of a cohesive bed has not been established to date. Most likely, the silt deposition occurs at low flow, and at higher flows bed load moves over the silt deposits before the fine sediments have an opportunity to be

resuspended (maybe due to some salinity-induced cohesion). The coarseness of the drag bucket sample then is a function of how thick the sand layer is and how deep the sampler penetrates into the bed. The sand layer is expected to be thicker upstream. March 2009 bed gradations from the anchorage area, and the navigation channel are shown in Figure 3-32.

Figure 3-32. Bed gradations, March 2009, on the rising hydrograph below Cubits Gap.



The drag bucket samples collected in SWP at the peak of the annual hydrograph in May and the fallen annual hydrograph in July did not exhibit the bimodal characteristics of some of the samples collected AHP. Most of the samples collected in SWP were silty sands. No clear trend was identified due to the fall of the hydrograph. The coarsest samples were collected in May at -2.9 and -4.9 BHP and consisted almost entirely of fine sand. The finest samples were collected at -0.9 and -2.0 BHP, also in May, and consisted almost entirely of silt. Except for the fine sand samples, in general, the July bed gradations were coarser than the May bed gradations. May and July 2009 bed gradations in SWP are shown in Figure 3-33 and Figure 3-34.

Figure 3-33. May and July 2009 bed gradations in SWP above River Mile (RM) 10, below Head of Passes (BHP).

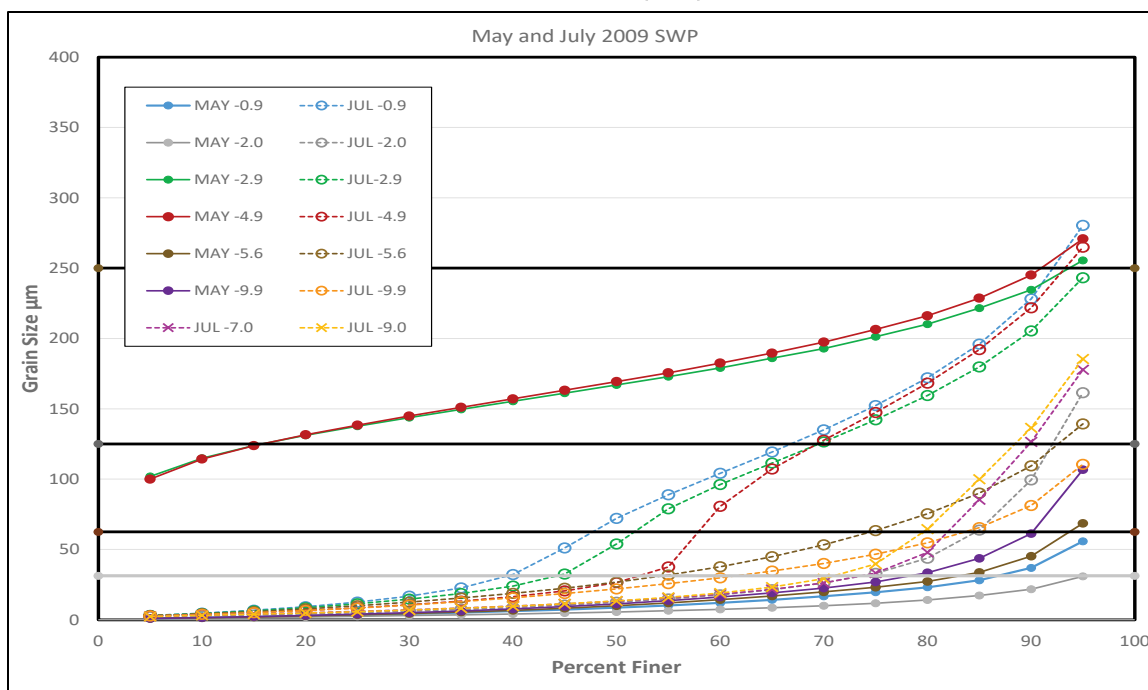
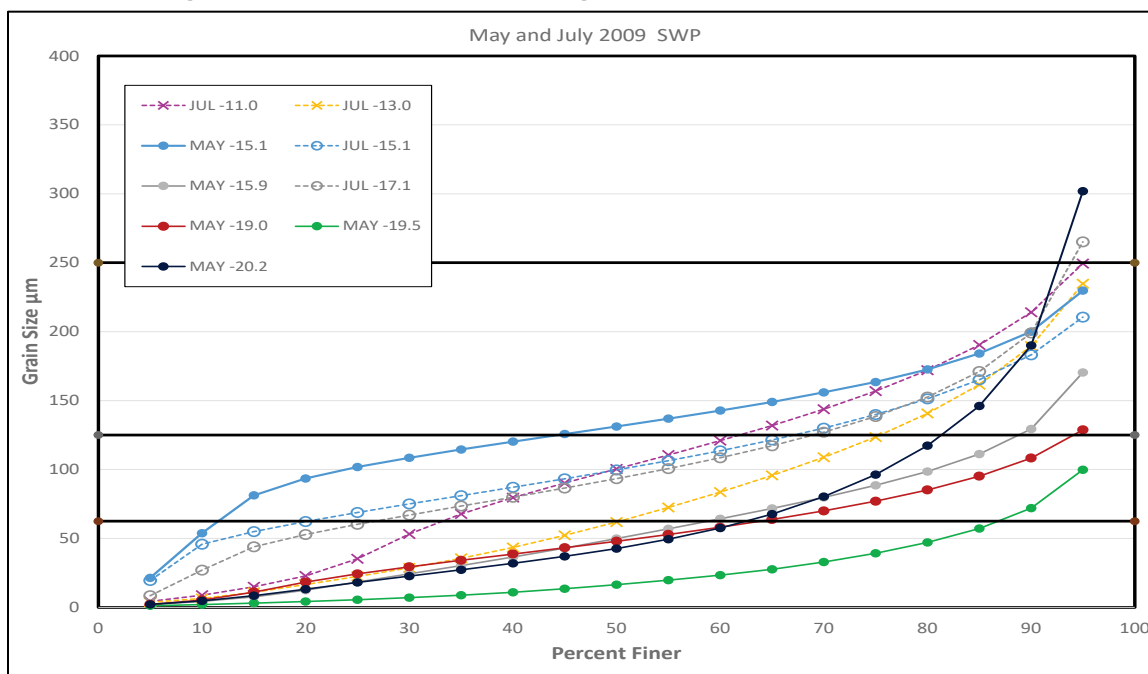


Figure 3-34. May and July 2009 bed gradations in SWP below RM 10, BHP.



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 dominant 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 resuspended 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.

3.14 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 will be higher 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, and
- 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, and
- 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 equation describes sediment concentration due to vertical diffusion. It 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. Actual measurements at diversions and in the upstream main channel are the most reliable means of determining existing sediment diversion coefficients.

3.14.1 SWP Distributaries

The invert elevation in the four SWP 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 SWP are tabulated in Table 3-19 through Table 3-22. Ratios for sediment coarser than fine sand were 0.0.

Table 3-19. Sediment concentration diversion ratios, Burrwood Bayou.

Discharge ft ³ /sec	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 3-20. Sediment concentration diversion ratios, Outlet W-2 and Overbank.

Discharge ft ³ /sec	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 3-21. Sediment concentration diversion ratios, Joseph Bayou.

Discharge ft ³ /sec	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 3-22. Sediment concentration diversion ratios, Outlet W-1.

Discharge ft ³ /sec	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

3.14.2 HOP to Baptiste Collette Distributaries

The invert elevations in South Pass and PAL 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 3-23. Ratios for sediment coarser than medium sand were 0.0. This treatment is consistent with all previous ERDC, Mississippi Valley Division, and MBH HEC-6T models.

Table 3-23. Sediment concentration diversion ratios, South Pass and PAL.

Discharge ft ³ /sec	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	0.93
Coarse Silt	0.99	0.97	0.96	0.91	0.65	0.37
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 HOP 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 the ERDC Hydrodynamic and Delta Management Model (Thomas et al. 2018) and are shown in Table 3-24. Ratios for sediment coarser than very fine sand were 0.0.

Table 3-24. Sediment concentration diversion ratios, “Leaks.”

Discharge ft ³ /sec	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 3-25. Ratios for sediment coarser than fine sand were 0.0.

Table 3-25. Sediment concentration diversion ratios, Cubits Gap.

Discharge ft ³ /sec	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 3-26. Ratios for sediment coarser than medium sand were 0.0.

Table 3-26. Sediment concentration diversion ratios, Baptiste Collette.

Discharge ft ³ /sec	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

3.14.3 Bohemia Spillway

The elevation of the natural levee along the Bohemia Spillway was determined by MVN using 2004 Louisiana lidar data. Top of levee elevations were found to vary between 5.3 and 11.8 ft NVGD 29. Sediment concentration diversion ratios for the maximum discharge of 1,250,000 ft³/sec 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.

3.14.4 ORCC

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

1. Measured suspended sand data from Union Point (UPT) and the Outflow Channel (ORO) were used to determine composite target sand sediment diversion coefficients for each sand size class. The long-term, discharge-weighted sand diversion coefficients for each structure should match the long-term target ORO/UPT sand diversions coefficients.
2. Pump sampler 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. Initial sediment diversion coefficients for each size class were calculated using the adjusted pump sample concentrations and Union Point suspended load concentrations. These are average values, not a function of discharge.
4. 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 method (Rouse 1937).

Due to the wide scatter in the measured data, no attempt was made to vary the concentration ratios with changes in discharge or with variations in operation schedules. The final adopted sand concentration diversion ratios are tabulated in Table 3-27 and Table 3-28. 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 3-27. Sediment diversion ratios at ORCC, 1991–2019

Sand Class Size	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 3-28. Sediment diversion ratios for Overbank Structure.

Discharge ft³/sec	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
Coarse Sand	0.01	0.0	0.0	0.0	0.0	0.0

4 Dredging Specifications

4.1 Datum

The authorized dredging depths for the Baton Rouge to the Gulf navigation channel were originally defined relative to MLG. Downstream from New Orleans, MLG has been replaced as a reference datum by MLLW. The conversion is 0.0 ft MLLW = 3.5 ft MLG^{SWP} (USACE MVN 2017a). Upstream from New Orleans, MLG has been replaced by Low Water Reference Plane (LWRP). MLLW varies periodically based on the years NOAA uses to calculate the average (epoch). Using the 2007–2011 tidal epoch, the conversion to North American Vertical Datum of 1988 (NAVD88) is provided in Table 4-1 (USACE MVN 2017a).

Table 4-1. Datum offsets for Mississippi River gages from Venice to East Jetty.

Gage ID	Gage Name	Datum Offset
01480 RM 10.7	Mississippi River at Venice	0' NAVD88 = -0.30' MLLW = 3.20' MLG ^{SWP}
01525 RM 2.0	Mississippi River at Pilottown	0' NAVD88 = -0.30' MLLW = 3.20' MLG ^{SWP}
01545 RM -0.5	Mississippi River at Head of Passes	0' NAVD88 = -0.18' MLLW = 3.32' MLG ^{SWP}
01575 RM -7.5	Southwest Pass at Mile 7.5	0' NAVD88 = 0.17' MLLW = 3.67' MLG ^{SWP}
01625 RM -12	Southwest Pass at Light 14	0' NAVD88 = 0.39' MLLW = 3.89' MLG ^{SWP}
01670 RM -17.9	Southwest Pass at East Jetty	0' NAVD88 = 0.68' MLLW = 4.18' MLG ^{SWP}

The HEC-6T numerical model was developed using geometry from the 1992 hydrographic survey in the MVN. The datum for this survey was NGVD29. Cross-section geometry was not changed in the HEC-6T model for this study and reflects NGVD29 elevations. This includes the assigned dredging template elevations. However, the dredging template elevations assigned in HEC-6T have been adjusted to reflect the nominal MLLW elevations. This was accomplished using the offset elevations from Table 4-1, which convert MLLW to NAVD88 (2009.55). The conversion from NAVD88 (2009.55) to NGVD29 was accomplished by adding 0.9 ft. The 0.9 ft conversion was determined using data from RiverGages.com at five gages between Empire (RM 29.5) and Inner Harbor Navigation Canal Lock (RM 92.7).

4.2 Dredging Specifications

There are several dredging options in the HEC-6T program. In this study, a trapezoidal dredging template was assigned to each cross section in specified dredging reaches. The earliest commencement date for dredging each year is assigned. In this study, that date is January 1. Also, for dredging calculations to begin in the model, a commencement of dredging (trigger) elevation must be exceeded by calculated deposition in a dredging reach. This trigger elevation is assigned at each cross section that is subject to dredging. The trigger elevation is checked daily during the simulation until it is exceeded at one of the cross sections in a dredging reach. Starting at the downstream cross section of the dredging reach, bed material is removed from the control volumes associated with each cross section, down to an assigned advance maintenance depth. The removal rate can be specified. Once all the sediment is removed down to the advance maintenance elevation, a dredging cycle is complete. More cycles can be assigned to redredge a reach. If more cycles are assigned, dredging continues starting at the downstream end of the dredging reach. When the specified number of dredging cycles have been exhausted, dredging stops for the remainder of the year. Continuous dredging can be achieved by assigning 366 cycles.

The allowable cross-section width for sediment erosion and deposition can be the same as the dredging template width or greater. This assignment can be critical if there is a point bar or dike field where calculated sediment deposition is excessive due to all or most of the sediment removal being assigned to the dredging template. In this study, both movable bed assignments were used depending on comparisons to historical survey data.

Calculated dredged material can be returned to the river as suspended sediment downstream from the dredged cross section or removed from the model. Depositing dredged material in the channel simulates agitation dredging or open-water disposal. Removal of material from the model simulates hopper and cutterhead dredging hauled to disposal areas or pumped over the levees or banks.

4.3 Dredging Templates

The dredging template elevation used in the numerical model between 1991 and 2019 is based on a navigation channel maintained at an elevation

of -48.5 ft in SWP and AHP. A navigation channel maintained at a depth of 45 ft was assigned for the deep-draft crossings.

The future dredging templates for the Baton Rouge to Gulf navigation channel are provided in USACE MVN (2017b). SWP and AHP will be dredged to an elevation of -50 ft MLLW. This amounts to an increase of approximately 1.5 ft from the current maintenance practice. The crossings between New Orleans and Baton Rouge will be dredged to an elevation of -50 ft below LWRP. This amounts to an increase of approximately 5.0 ft from the current maintenance practice. These nominal MLLW elevations are the trigger elevations where dredging commences in the model. The dredging template includes 6 ft of advance maintenance. In this study, for the structural alternative evaluations, the entire 6 ft were dredged at each dredging event. Later in the study when the model was used to simulate actual daily dredging operations, the advance maintenance was reduced to 3 ft.

Actual dredging protocol starts with hopper dredges knocking off shoaling at the worst shoaling areas to maintain a 50 ft deep channel, but not necessarily down to the advance maintenance elevation of -56 ft. If cutterheads are available, they dredge to -56 ft. After the annual crisis passes, all dredges get the channel down to -56 ft. Model results for the structural alternatives focus on total annual dredging. Therefore, the model dredges to elevation -56 ft.

4.4 Uncertainty Related to Actual Dredging Operations and Numerical Simulations

The dredging protocol assigned in the numerical model cannot be expected to match individual annual dredging operations. The objective is to match average long-term dredging quantities rather than specific annual operations. The numerical model protocol is constant and unchanged during the simulation whereas prototype dredging operations are influenced by a variety of constraints, such as availability of dredges, dredging site priorities, and available funding. Uncertainty also arises due to the accuracy of reported dredging data and accuracy of geometric and sediment data used in the numerical model. Nevertheless, when long-term average annual results are consistent in the model and prototype, the model can be used confidently to compare and assess the long-term effects of various channel modifications and changes in geomorphological processes.

5 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, Illinois, and the Gulf of Mexico. Additional calibration and verification, focusing specifically on recent dredging data, were conducted for this study.

The primary purpose of this study is to evaluate the sedimentation effects of several structural alternatives that included partial closure of distributaries below Venice and river widening adjacent to the HDDA. This was accomplished by calibrating the model to reproduce total reported dredging volumes in SWP and AHP between 1991 and 2019. This approach is adequate to evaluate the relative significance of the various structural alternatives.

After completion of the alternative structure evaluations, the hydrographic survey data discussed in Chapter 2 became available, and the model's ability to reproduce the actual shoaling distribution and local dredging volumes measured between 2015 and 2019 was evaluated. This objective required additional calibration, which is discussed in Chapter 6, Model Results.

5.1 Calibration and Verification of HEC-6T Flowline Model

The Flowline Model was a cooperative effort of the USACE MVN, Vicksburg, Memphis, and Saint Louis Districts, and the Mississippi Valley Division. The model extended between the Gulf of Mexico and Chester, Illinois, 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:

- 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.
- Matched measured sand concentrations at Union Point (RM 326), Coochie (RM313), Tarbert Landing (RM 306), and Belle Chasse (RM 76)
- Matched reported dredging in SWP and HOP (1975–1984 and 1991–2002)

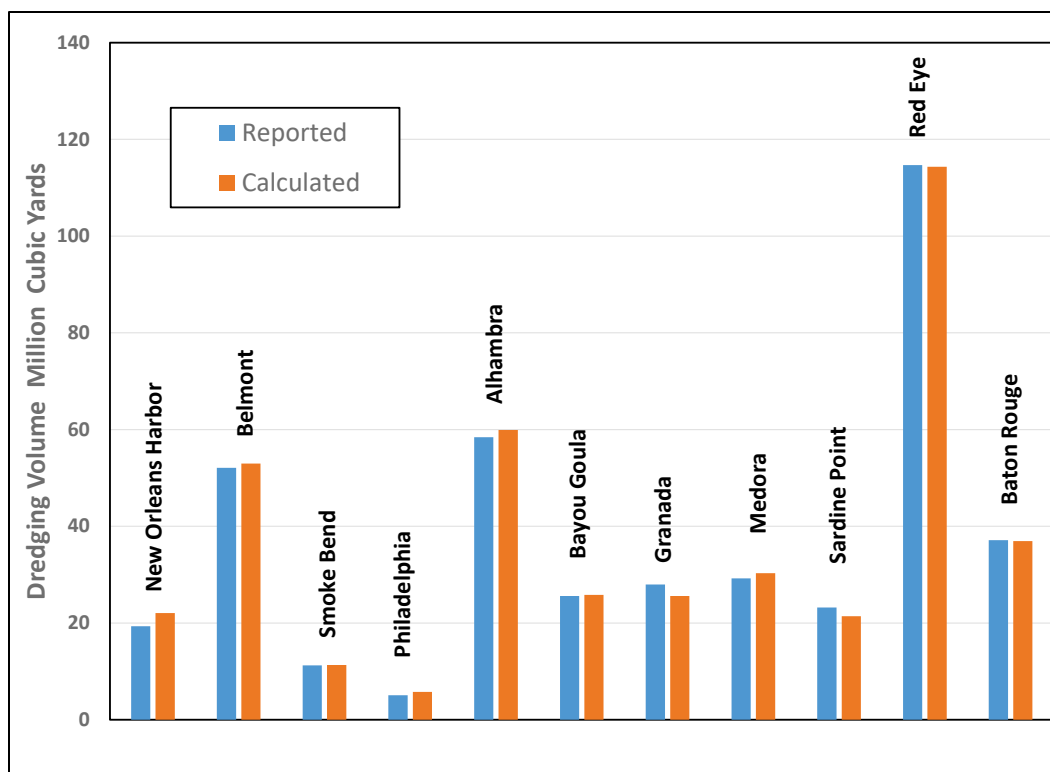
- Duplicated measured specific gage trends at gages
- Duplicated 1991–2002 degradation caused by the construction of Smithland Dikes (RM 299)
- Simulated 1991–2002 aggradation of a point bar between the Low Sill and Auxiliary Structures

5.2 Dredging at Deep-Draft Crossings below Baton Rouge

The Flowline Model was 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 is 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 this study.

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 year. 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 5-1.

Figure 5-1. Calculated and reported dredging volumes at crossings, Water Year (WY) 1991–2013.



5.3 Dredging below Venice

Calculated average annual dredging quantities were compared to MVN reported contract dredging volumes as part of the model calibration for this study. Average annual dredging volumes are adequate for evaluating structural modifications because the objective is to compare long-term effects. The 29 yr period between October 1990 and October 2019 was used for the calibration. Results are shown in Figure 5-2 and Table 5-1. 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 simulations, dredged material was removed instantaneously every day.

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 HDDA and the Ocean Dredged Material Disposal Site (ODMDS). 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 5-3 and Table 5-1. The calculated and reported data suggest that more dredging occurs above RM -11 than below RM -11. 2019 was indeed an exceptionally high runoff year, significantly affecting the 11 yr average annual dredging volumes, especially upstream from RM -11.

Figure 5-2. Calculated and reported dredging, WY 1991–2019.

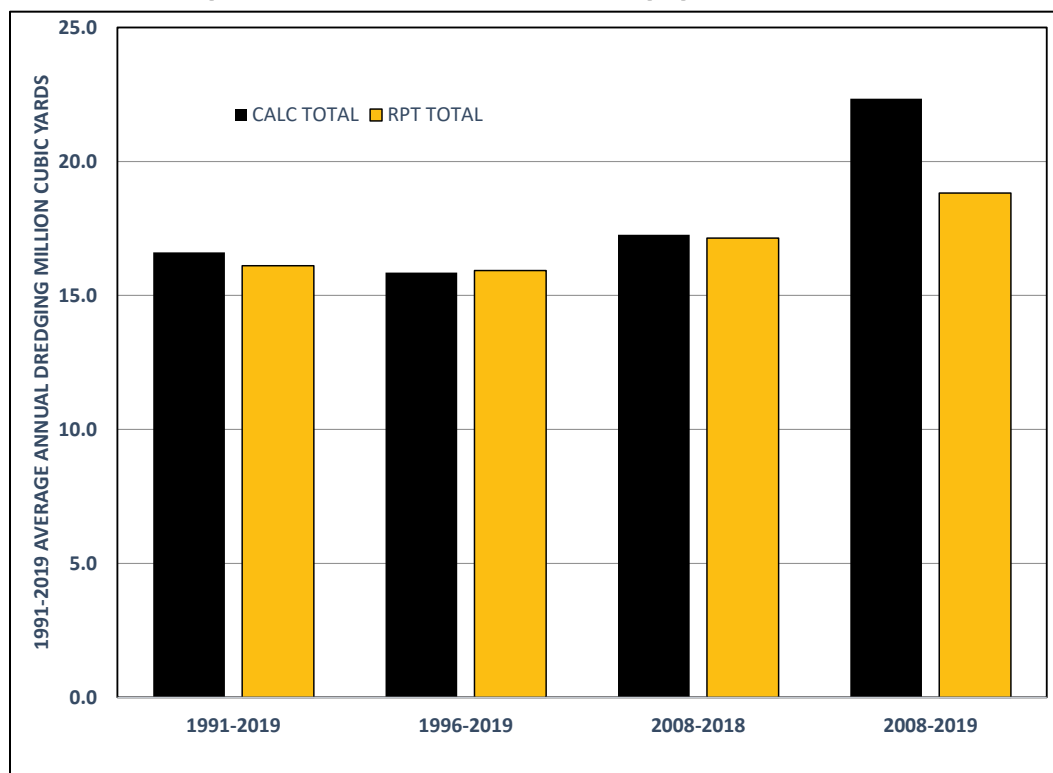


Figure 5-3 Reported and calculated dredging above and below RM -11.

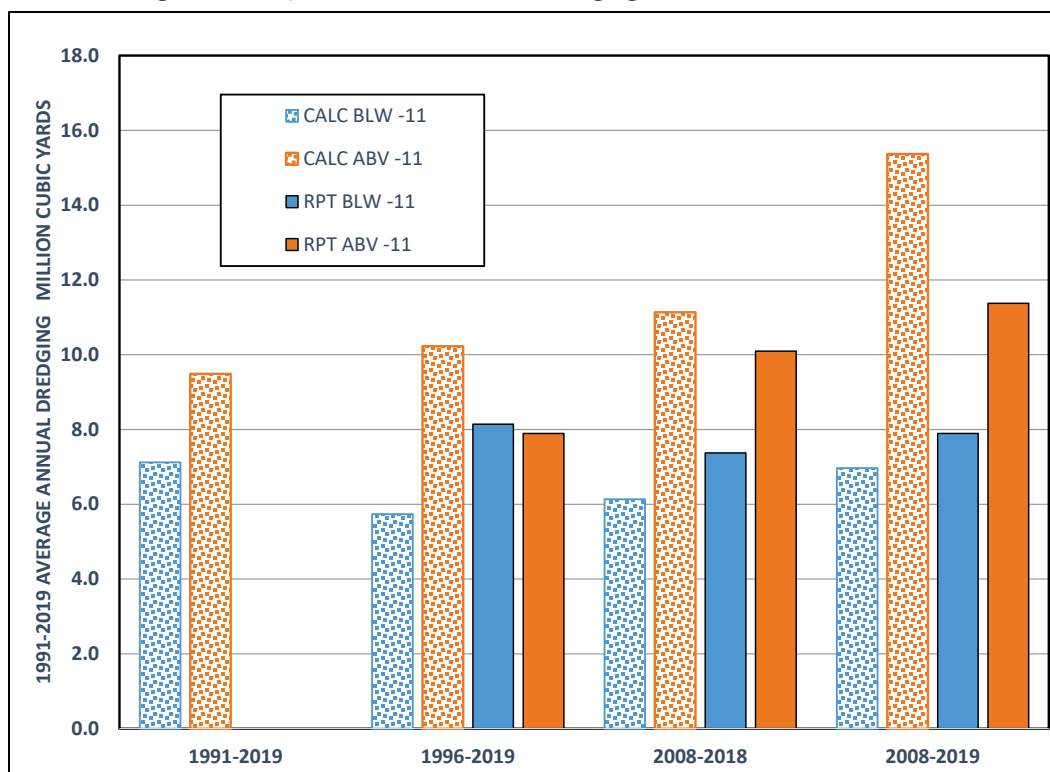


Table 5-1. Calculated and reported dredging, Water Year (WY) 1991–2019.

WY	HEC-6T Calculated $\text{yd}^3 \times 10^6$			Reported (minus agitation) $\text{yd}^3 \times 10^6$		
	<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	15.97	8.14	7.90	16.04
2008–2018	6.13	11.13	17.26	7.37	10.10	17.47
2008–2019	6.97	15.37	22.34	7.90	11.38	19.28
Percentage Difference from Reported						
1991–2019	—	—	+3.1	—	—	—
1996–2019	-29.5	+29.5	-0.4	—	—	—
2008–2018	-16.8	+10.2	-1.2	—	—	—
2008–2019	-11.8	+35.1	+15.9	—	—	—

6 Model Results

6.1 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 using measured data (Biedenharn et al. 2017) demonstrated historical trends, and numerical model studies (Copeland 2018) predicted future trends. These studies showed a consistent degradation trend at Memphis between 1930 and 2065 (Figure 6-1). There was a degradation trend at Vicksburg between 1930 and 1950 followed by an aggradation trend beginning in 1950 and projected through 2065 (Figure 6-2). A consistent aggradation trend between 1940 and 2065 was calculated at Red River Landing (Figure 6-3). These trends are expected to affect sediment delivery and dredging in SWP and AHP. The numerical model accounts for the long-term change in sediment delivery and its effect on dredging below Venice.

Figure 6-1. Specific gage at Memphis, RM 734.4.

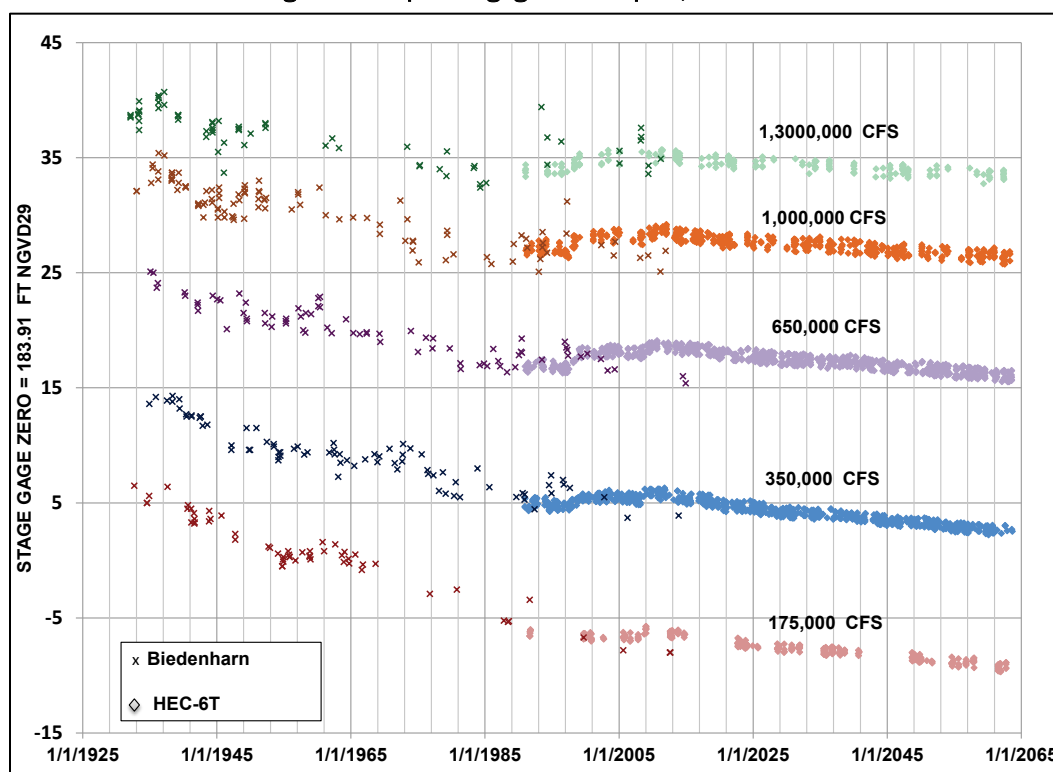


Figure 6-2. Specific gage at Vicksburg, RM 435.7.

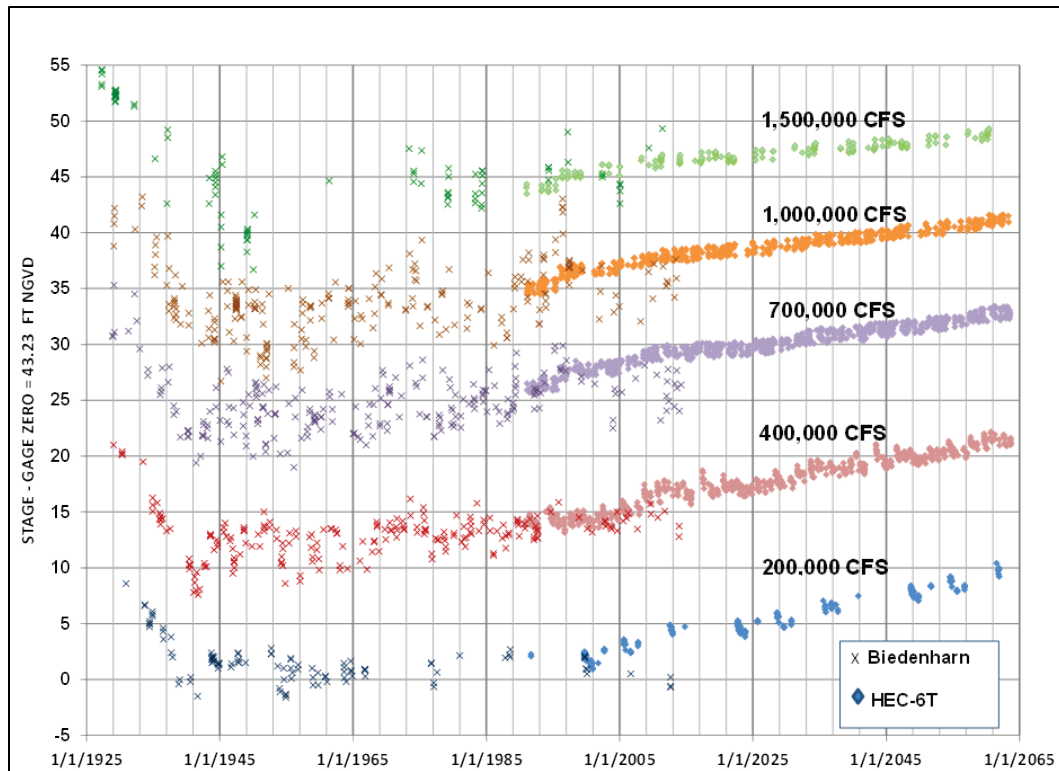
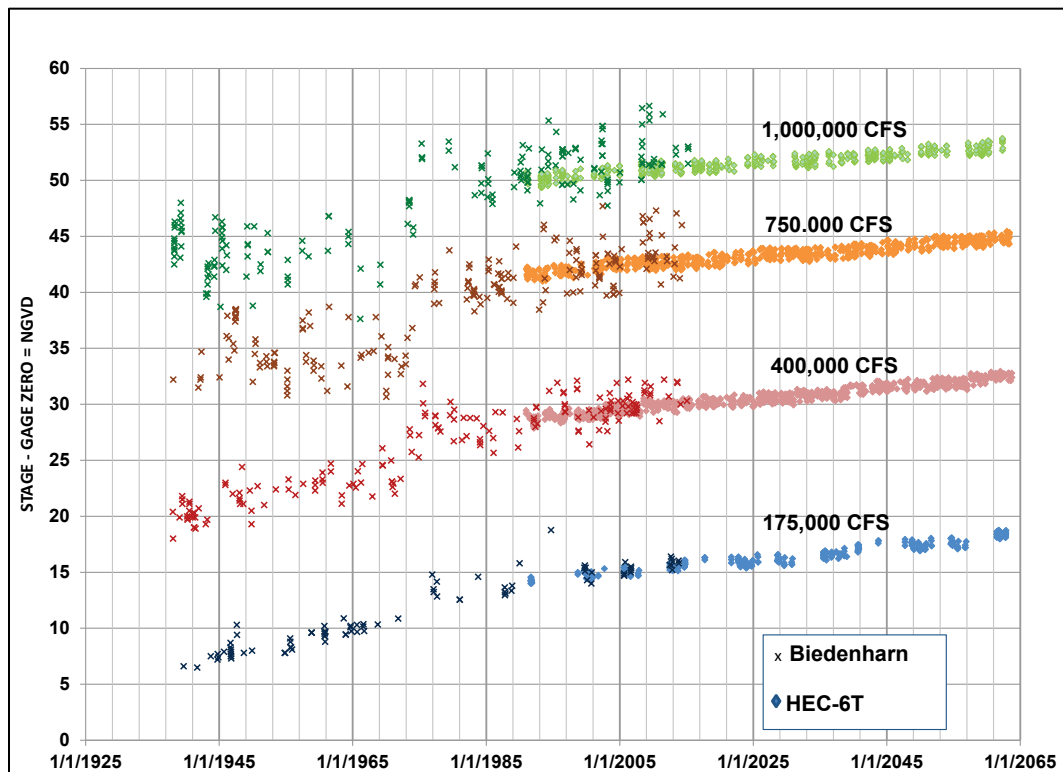


Figure 6-3. Specific gage at Red River Landing, RM 302.4.



6.2 Evaluation of Structural Alternatives

The model was used to determine long-term effects of various engineering changes to the river system, focusing on dredging requirements. The model was calibrated to match reported average annual dredging quantities in SWP and AHP. Then, average annual dredging volumes in SWP and AHP were calculated for a 50 yr hydrograph (1970–2019) and compared to a base test, which contained the existing diversion conditions. Sediment inflow came from the most recent data at upstream gages (Thebes, Olmsted, etc.). Water diversion percentages were based on recent data when available, including at the ORCC. Sediment diversion coefficients were based on the Rouse equation (Rouse 1937) (except at West Bay, Leaks, Fort St. Philip, and Ostrica, where coefficients were based on those used by Thomas et al. (2018)). Sand diversion percentages calculated by the Rouse equation were reduced by 25% downstream from Venice. The model included sea-level rise (II) (USACE 2019) and a 50 ft deep navigation channel with 6 ft of advance maintenance. Two additional feet were added to the dredging depth (trigger elevation) to account for overdredging for the structural modifications study. This adjustment was adopted during the calibration phase of the model study. Dredging operations dredged down to the advance maintenance elevation whenever the target elevation was exceeded by deposition. The dredging occurred instantaneously and continuously throughout the year. This operation is not realistic, but it is systematic, consistent, and appropriate for evaluating alternatives. The model demonstrated deposition patterns upstream and downstream from RM -11 BHP similar to those determined from measured survey data. Alternatives evaluated were closing various outlets and dredging adjacent to the HDDA.

6.2.1 Reducing Distributary Outflow

The numerical model was used to determine the effect of reducing distributary outflow from the Mississippi River below Venice. The major distributaries downstream from Belle Chasse (RM 76) are shown in Figure 6-4. The model tests were intended to demonstrate only the effects of reducing distributary outflows on shoaling and dredging and did not attempt to design the control structure itself. These modifications were intended to be representative of a host of possible alternatives and can be used to determine feasibility and effects of not only the specific modification tested, but of other similar modifications. A conceptual schematic of such a structure is shown in Figure 6-5. In this figure, the

modification could be a stone or timber structure, installed to manipulate and reduce or stop flow exiting West Bay.

Figure 6-4. Major distributaries downstream from Belle Chasse (maps data: Google Earth, Image Landsat/Copernicus, 12/2017).

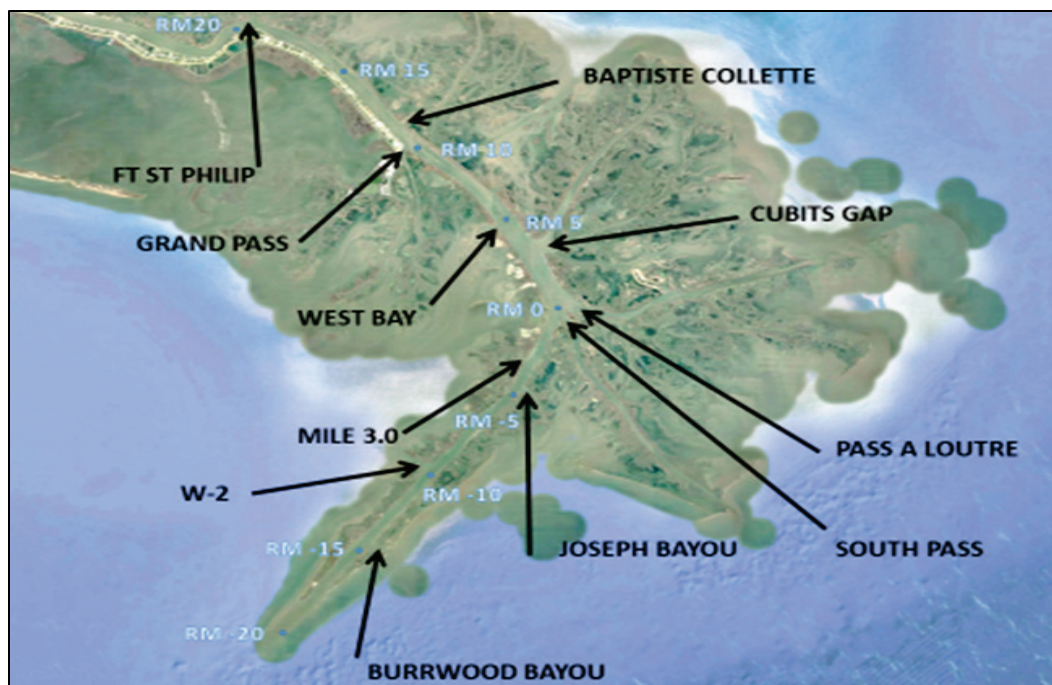


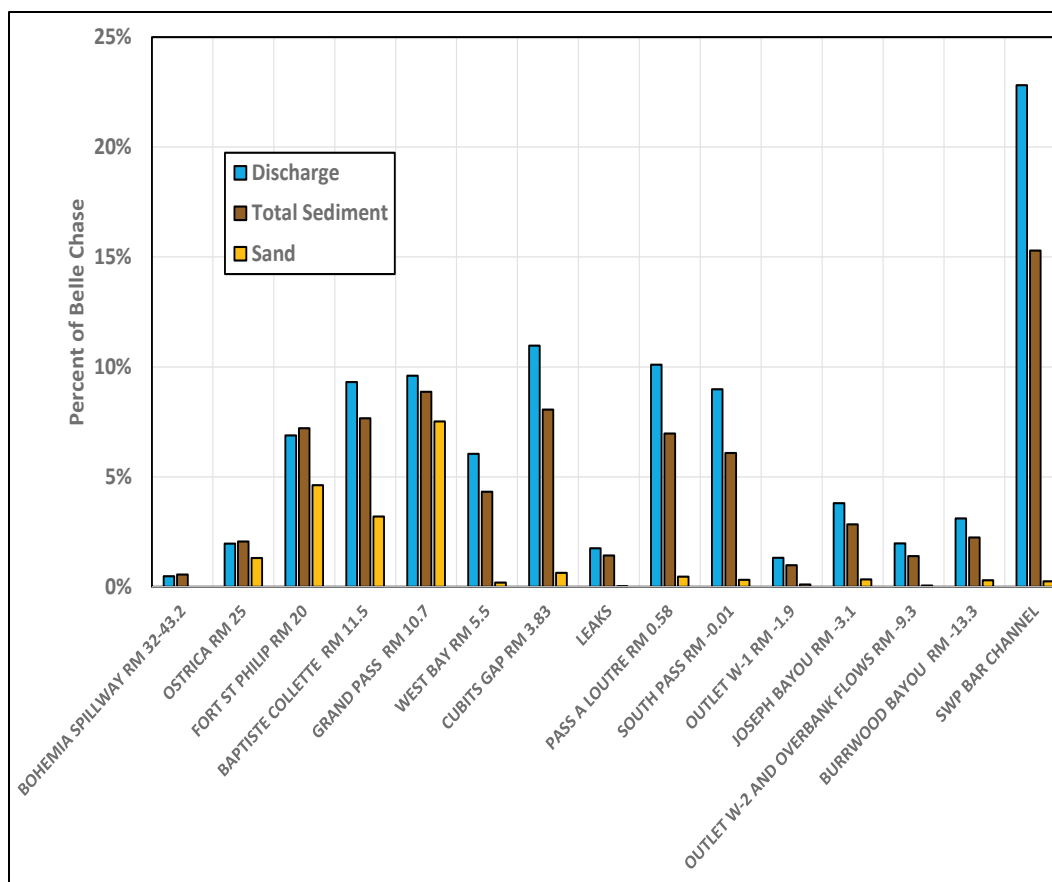
Figure 6-5. Conceptual distributary flow reduction structure at West Bay.



Partial or total closure of a distributary increases the downstream discharge and bed shear stress, which will increase sediment transport capacity and reduce shoaling. The effectiveness of partial closure at any given distributary is not only a function of the water diverted but is also a function of the sand diverted. The percentage of sand diverted is determined by the geometry of the outlet, specifically the base elevation of the outlet channel. Initially, when a crevasse first occurs, a large scour hole typically develops at the breach. Over time, as the distributary matures, a delta forms downstream from the crevasse, which tends to stabilize the outlet elevation. Most of the distributaries in the SWP and HOP reaches are mature. The percentage of each sediment size class diverted at most of the distributary channels was calculated and assigned in the numerical model by considering the vertical distribution of suspended sediment and the elevation of the outlet channel. Except at Grand Pass, where the outlet elevation is approximately the same as the Mississippi River bed elevation, diverted sand concentrations are significantly less than the river concentrations. Fine sediment (silt and clay) typically has a constant vertical distribution, and therefore diversion concentrations of fine sediment are the same as river concentrations. Sediment diversion percentages in the numerical model at Fort St. Philip and Ostrica were treated differently. They were adopted from the ERDC/MBH model for the Mississippi River Hydrodynamic and Delta Management Study (Thomas et al. 2018). Diverted sand concentrations at these distributaries, which are less mature, were higher than at downstream distributaries.

The calculated 50 Yr average percentages of Belle Chasse water and sediment diverted at downstream distributaries are shown in Figure 6-6. Diverted sand concentrations are highest at Ostrica, Fort St. Philip, Baptist Collette, and Grand Pass. Reducing flow out of distributaries with the highest sediment diversion percentages will be less effective in terms of reducing dredging downstream.

Figure 6-6. The 50 yr average percentage of Belle Chasse water and sediment diverted at downstream distributaries.



The distributaries tested were the following:

- West Bay
- Cubit's Gap
- W-1 Outlet (Mile 3.0 W)
- Joseph's Bayou
- W-2 Outlet
- Burrwood Bayou
- All four distributaries in SWP (the overbank flow exiting SWP at high flow was not reduced)

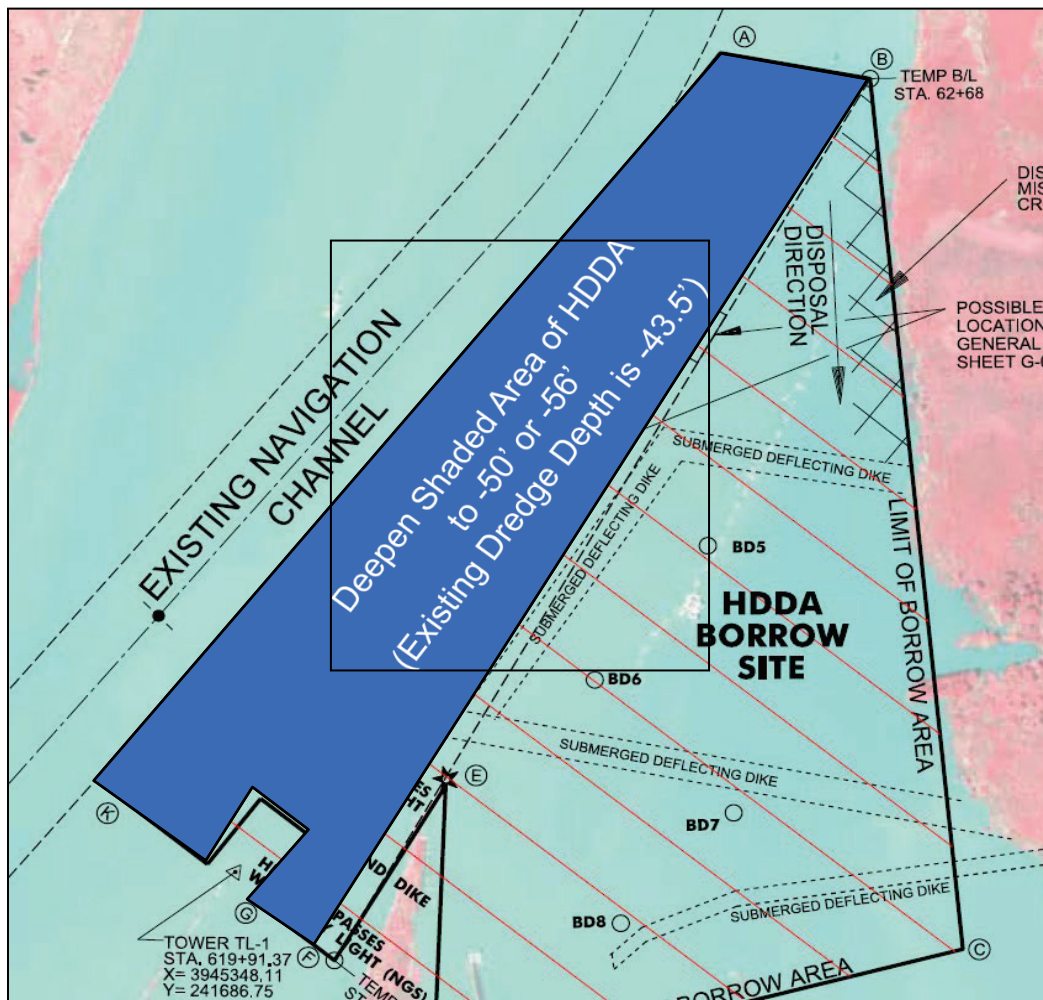
Although numerous reductions are possible, only a 50% flow reduction was tested at this phase of the investigation. An extreme test that

completely closed all diversions between Ostrica and the Jetties, except PAL and South Pass, was also evaluated. This test was labeled “close all.”

6.2.2 Channel Widening at HDDA

A second type of structural modification consisted of widening the navigation channel at HOP. This alternative called for the left-descending boundary of the navigation channel to be extended across the outer lanes of the HDDA and terminate at the Submerged Deflection Dike that parallels the channel. Depth across the deepened section of the HDDA would be -56 ft MLLW. An outline of the tested widening alternative is shown in Figure 6-7.

Figure 6-7. Channel widening alternative adjacent to HDDA.



6.2.3 Reducing Flow into SWP

A third modification was defined by unknown modifications to South Pass or PAL, or both, that would divert an increased percentage of flow into these passes at the expense of flow exiting SWP. In this test, an additional 10% of the Mississippi River discharge at Pilottown (RM 2.0) was diverted into PAL and South Pass (5% into each pass).

The assigned distributions at HOP and in SWP for the alternative that reduced outflow from all four of the SWP distributaries and the alternative that increased outflow into PAL and South Pass by 10% are shown in Table 6-1.

Table 6-1. The 50 yr average percent of the Belle Chasse discharge out SWP distributaries for Base and 50% SWP and 10% PAL and South Pass (SP) alternatives.

Alternative	Base	50% SWP	10% PAL and SP
PAL	10.1	10.19	15.21
SP	8.99	9.07	14.09
W-1	1.32	0.67	0.93
Joseph Bayou	3.80	1.96	2.68
W-2	1.97	1.13	1.38
Burrwood	3.11	1.77	2.19
Pilots Station	22.81	27.78	16.08

6.2.4 Effects of Structural Modifications on Total Dredging

Calculated results for the various modifications and the base test are shown in Figure 6-8 and Table 6-2. The change in annual dredging is small for most alternatives (note that in the figure, the y -axis starts at $20 \text{ yd}^3 \times 10^6$ and not zero). The most extreme case was closing all the openings between Ostrica and the Jetties, except for PAL and South Pass. This resulted in approximately a 19% reduction in average annual dredging. Reducing outflow into individual distributaries resulted in a much smaller decrease in average annual dredging. The calculated reduction in total average annual dredging for the tested alternatives varied between 0.8% for a 50% closure of Joseph Bayou to 2.4% for a 50% closure of South Pass. Increasing the channel width at HDDA resulted in a 0.6% increase in average annual dredging. Decreasing the flow percentage into SWP resulted in a 1.7% increase in average annual dredging.

Figure 6-8. The 50 yr average annual dredging for diversion alternatives.

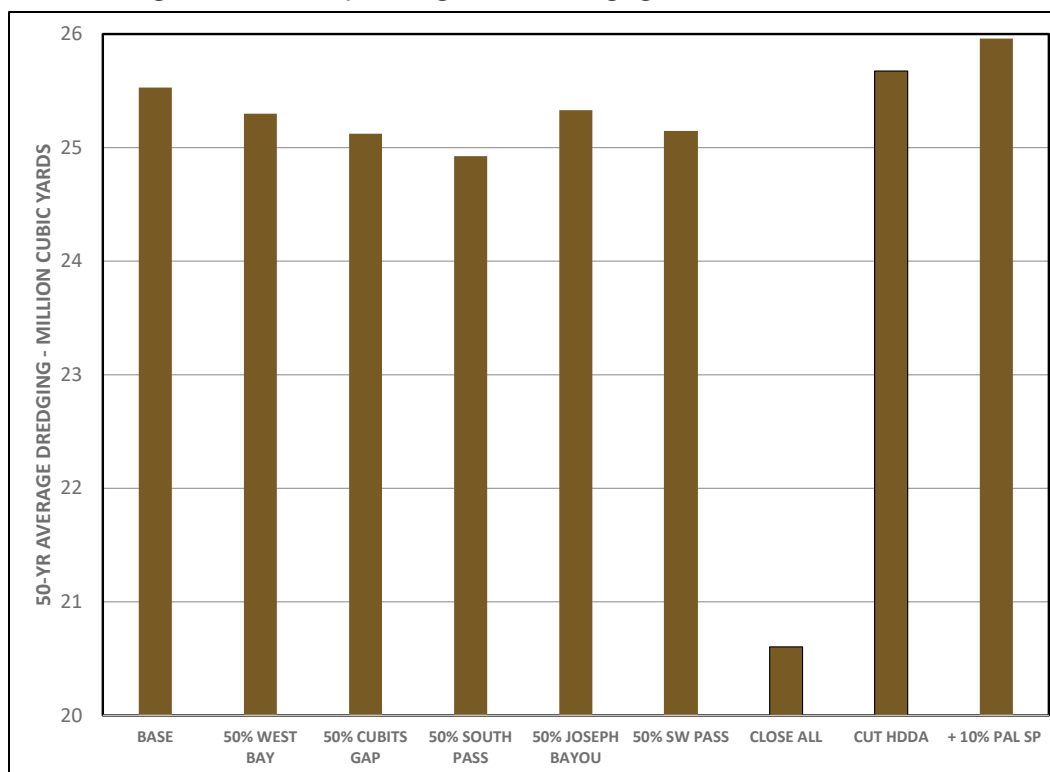


Table 6-2. Calculated 50 yr average annual dredging.

Test	Diversion Percentage Reduction	50 yr Average Annual Dredging— $\text{yd}^3 \times 10^6$	Difference from Base Condition Percentage
Base	—	25.530	—
West Bay	50%	25.299	−0.90%
Cubits Gap	50%	25.122	−1.60%
South Pass	50%	24.925	−2.37%
Joseph Bayou	50%	25.330	−0.78%
SWP *	50%	25.142	−1.52%
Close All **	100%	20.603	−19.30%
HDDA Cut	—	25.675	+0.57%
PAL and SP	Plus 10%	25.961	+1.69%

* Retain overbank flows during high discharges at RM −9.3

** Except PAL and SP (but eliminates overbank flow at RM −9.3 and RM 2.46)

6.3 Effect of Structural Modifications on Distribution of Dredging

Most of the shoaling and dredging occurs just upstream of HOP and in the Jetty and Bar Channel reaches at the end of SWP. Reducing flow and sediment out various distributaries may not affect the total dredging volumes significantly, but it can influence the distribution of shoaling in the study reach. To evaluate the distribution of shoaling, dredging volumes associated with each HEC-6T cross section were calculated and compared to the base test. Results are shown in Figure 6-9 through Figure 6-18 and summarized in Table 6-3. Fifty-year average annual dredging, accumulated from Venice, is compared to the base test in Figure 6-9 and Figure 6-10. Reaches with the steepest slope are the reaches where the most dredging occurs. Most of the dredging occurs upstream from HOP for the Base test and all of the alternatives except the “close all” alternative. Figure 6-9 clearly shows that closing all the distributaries has a significant effect on the distribution of dredging by moving most of the dredging operations down to the Jetty and Bar Channel reaches. These tests demonstrate that the implementation of distributary closures would significantly reduce dredging and move the dredging reach down toward the Gulf, adjacent or at least close to the ODMDS.

Table 6-3. Effect of alternatives on dredging distribution.

Alternative	Diversion Reduction	Distribution of Dredging
Base	—	Most dredging between HOP and West Bay
West Bay	50%	Less dredging downstream (DS) West Bay—more in SWP and Bar Channel
Cubits Gap	50%	Less dredging DS Cubits Gap—more in SWP and Bar Channel
South Pass	50%	Less dredging in SWP—more in the Bar Channel
Joseph Bayou	50%	Less dredging in SWP—more in the Bar Channel
Southwest Pass*	50%	Less dredging in SWP—more in the Bar Channel
Close All**	100%	Almost all dredging in the Bar Channel
Cut HDDA	—	More dredging upstream HOP—less in SWP
PAL and SP	Plus 10%	More dredging upstream and DS of HOP, but less DS from RM 11 BHP

* Retain overbank flows during high discharges

** Except PAL and SP (but eliminates overbank flows at RM -9.3 and RM 2.46)

Figure 6-9. Calculated 50 yr average annual accumulated dredging closure alternatives.

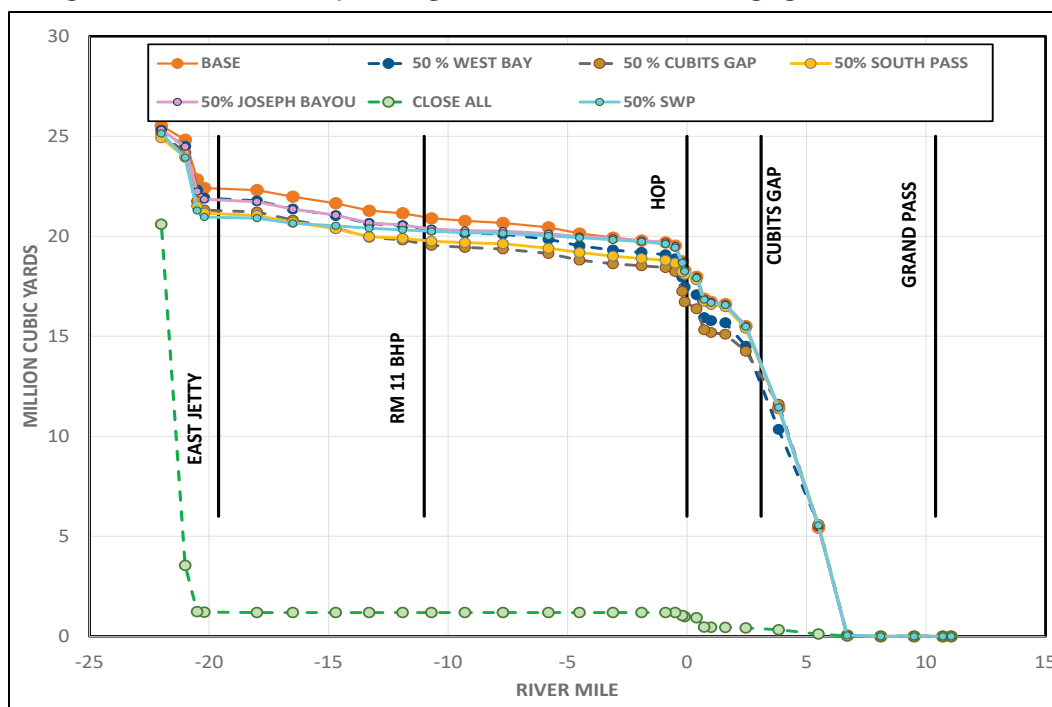


Figure 6-10. Calculated average annual accumulated dredging HDDA and increased flow into Pass a Loutre (PAL) and South Pass.

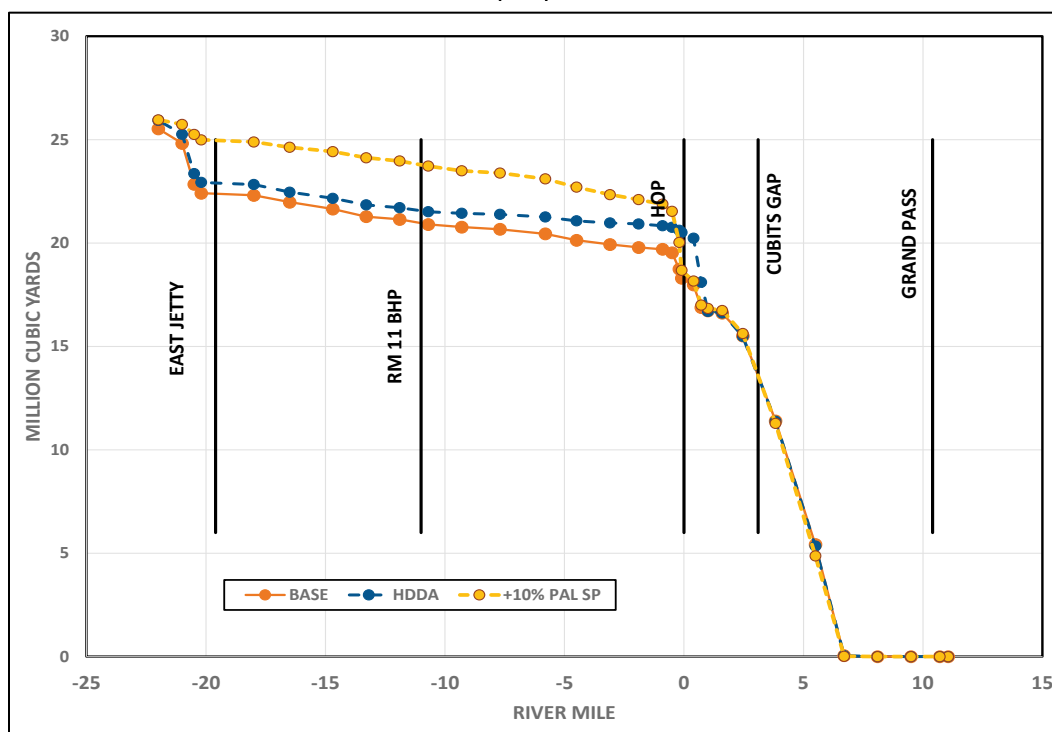


Figure 6-11. Calculated 50 yr average annual dredging with 50% diversion at West Bay.

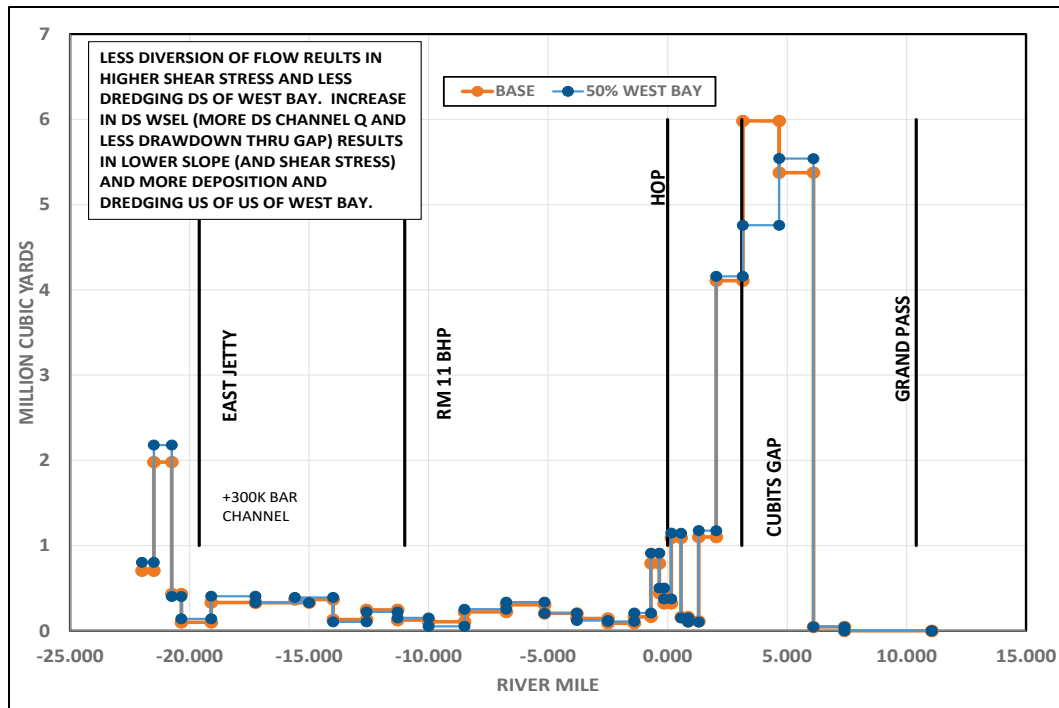


Figure 6-12. Calculated 50 yr average annual dredging with 50% diversion at Cubits Gap.

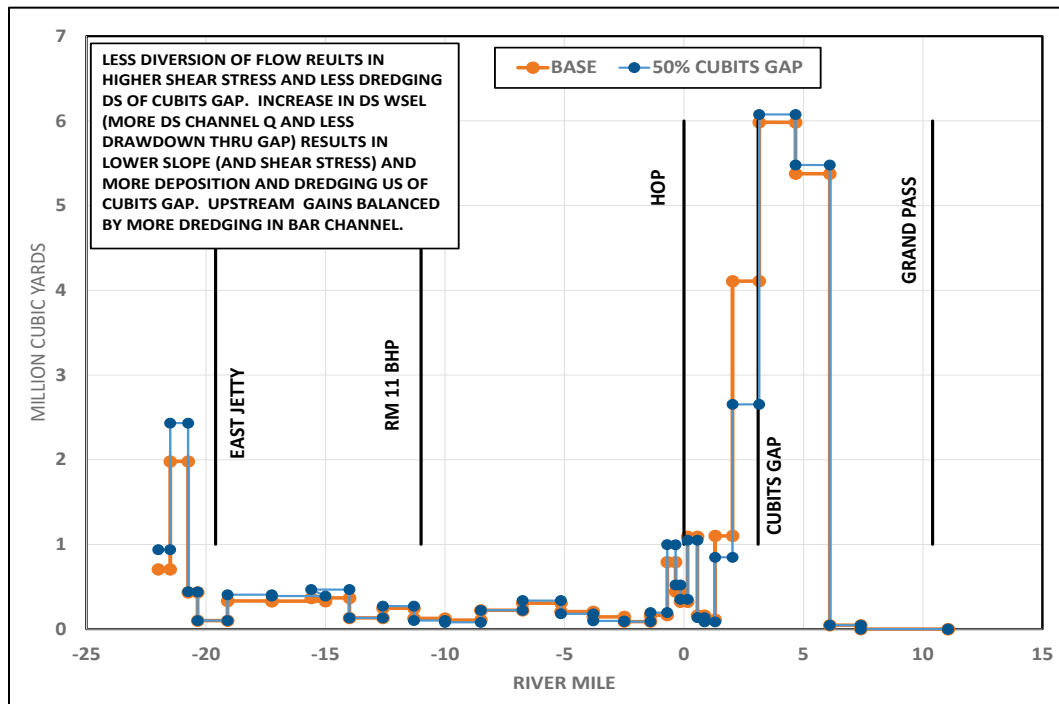


Figure 6-13. Calculated 50 yr average annual dredging with 50% diversion at South Pass.

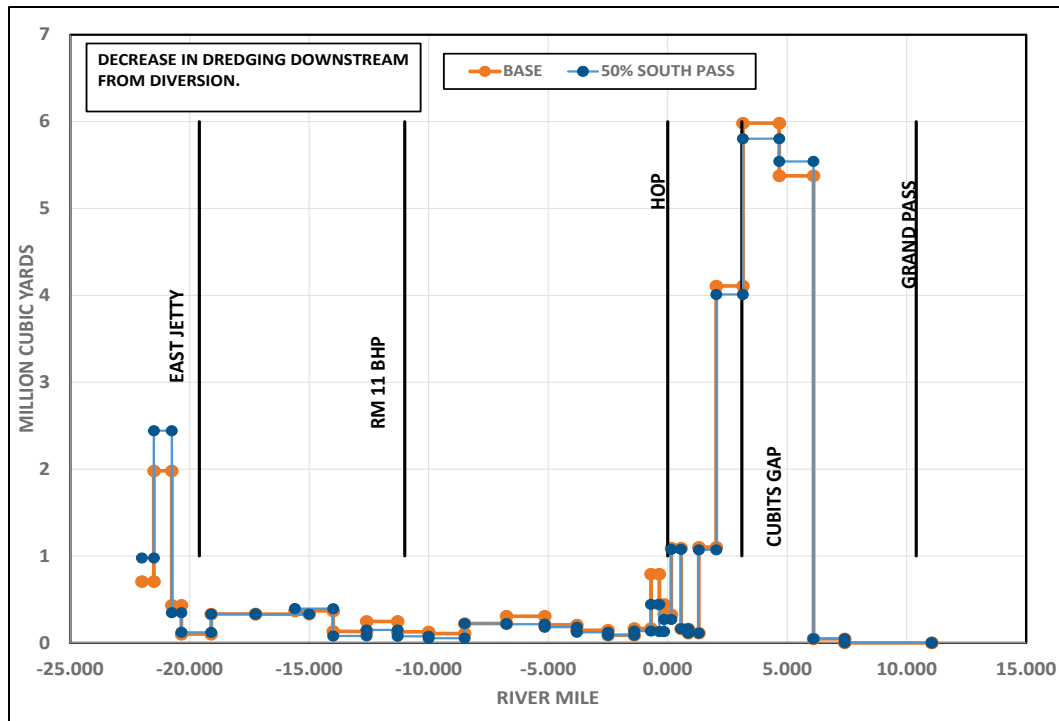


Figure 6-14. Calculated 50 yr average annual dredging with 50% diversion at Joseph Bayou.

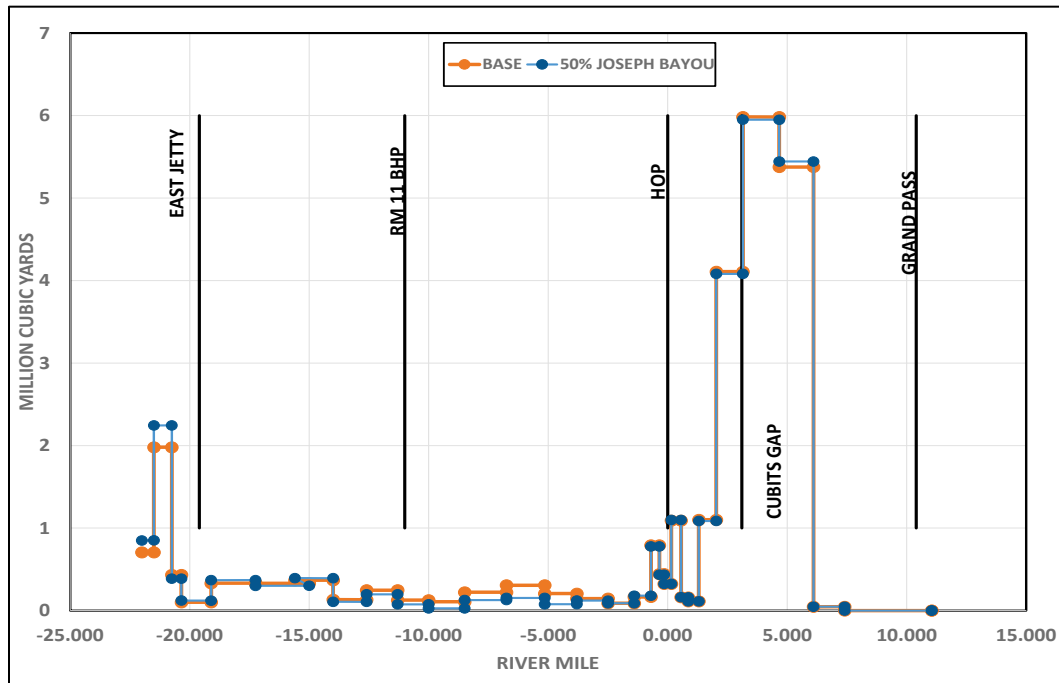


Figure 6-15. Calculated 50 yr average annual dredging with 50% diversion in four SWP distributaries.

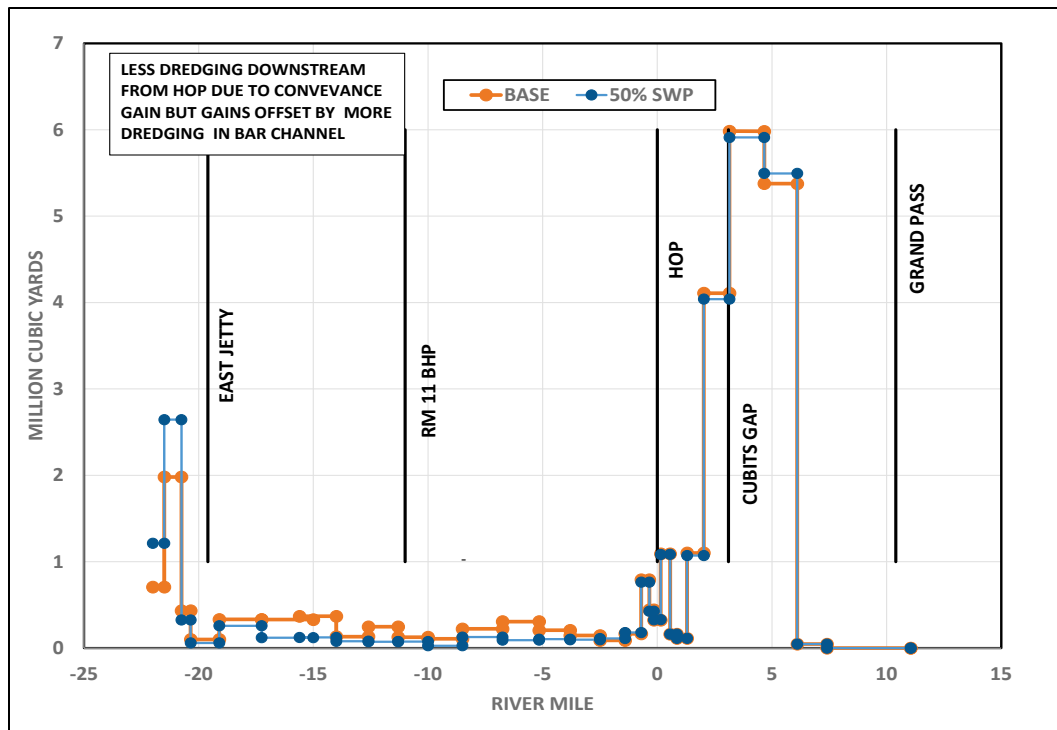


Figure 6-16. Calculated 50 yr average annual dredging with closure of all distributaries except PAL and South Pass.

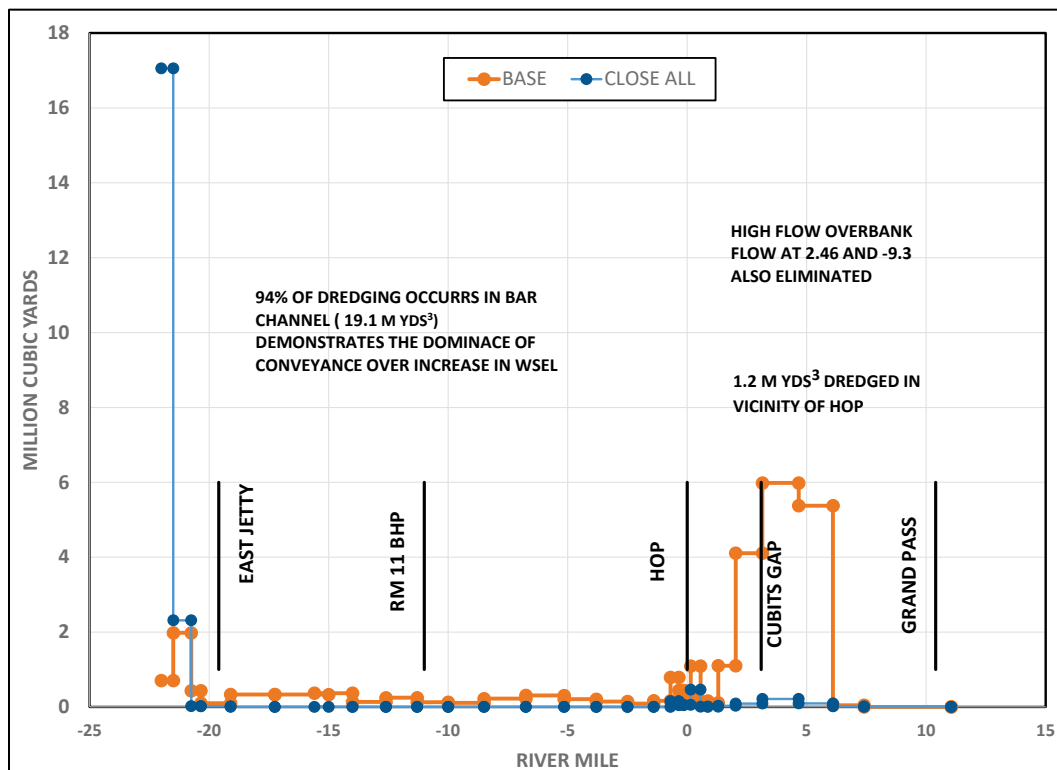


Figure 6-17. Calculated 50 yr average annual dredging with channel widening at HDDA.

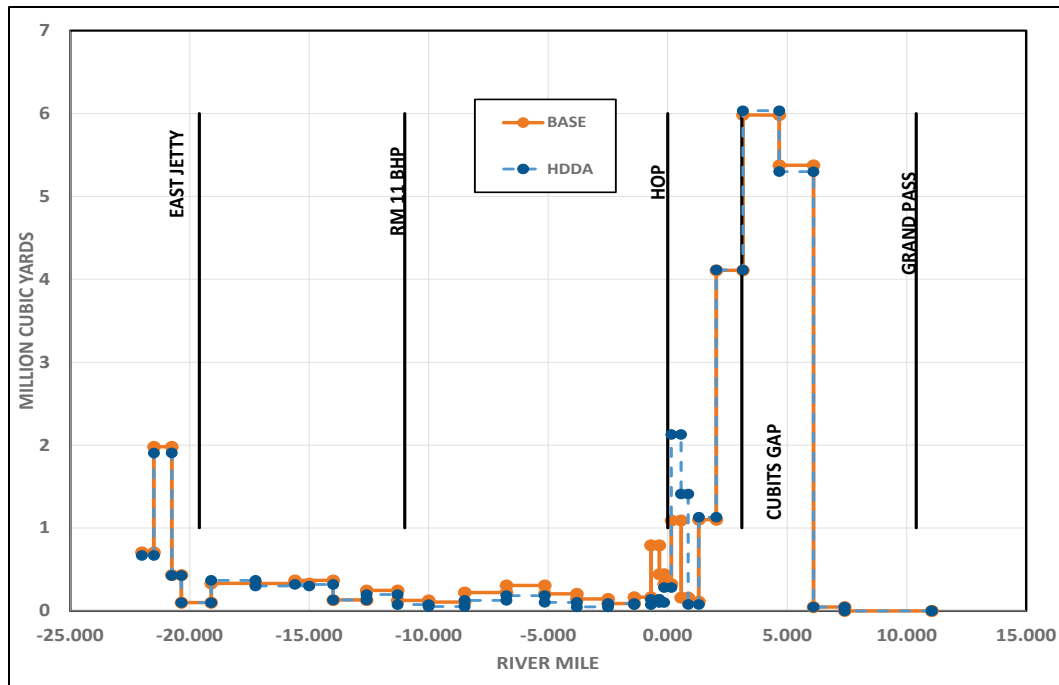
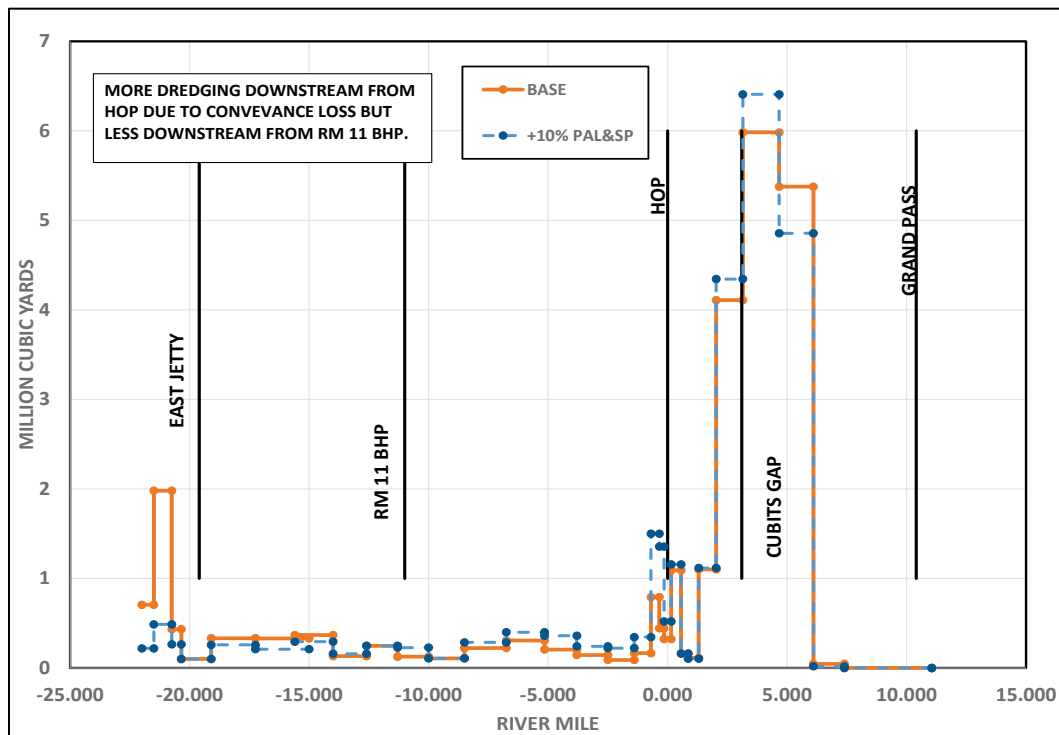


Figure 6-18. Calculated 50 yr average annual dredging with 10% more flow diverted into PAL and South Pass.



6.4 Surveyed Bed Elevations in the Navigation Channel

As discussed in Chapter 2, hydrographic surveys capturing navigation channel bathymetric conditions are compiled, standardized, and made available through the USACE eHydro system. Likewise for dredging operations, the onboard diagnostic sensor arrays for hopper dredges under contract with the USACE provide detailed information to the DQM geospatial data archive. The DQM system monitors critical aspects of dredge activity, including precise dredge locations and the volumes of material removed from depth contours within the channel cross-section. Separate records are maintained by the MVN for cutterhead dredge work. The RSM study conducted at ERDC and discussed in Chapter 2, used historic hydrographic surveys, uploaded through eHydro, to quantify shoaling volumes and morphologic patterns across a range of spatial scales. The daily survey frequency in combination with the DQM dredging records provided the necessary data to calculate a navigation channel elevation for the 5 yr period between 2015 and 2019.

Daily hydrographic survey data from the navigation channel, between Venice and the Bar Channel, were used to calculate bed elevations for the 5 yr period between 2015 and 2019. The data were divided into 1,772 bins. Between RM 11.4 (AHP) and RM -17.5 (BHP), the bins were 750 ft wide and 100 ft long. Between RM -18.0 (BHP) and -22.1 (BHP), the bins were 600 ft wide and 100 ft long. Using these bin areas and volumes above elevation -80 ft (MLLW), navigation channel depths for bins were calculated. Cross sections in the HEC-6T model represent control volumes extending between the half-way points of the upstream and downstream cross sections. There are several bins located in each HEC-6T control volume. An average depth was calculated for each HEC-6T control volume from the bin data located within. The temporal progression of the average depths at each HEC -6T cross section between 2015 and 2019 is shown in Figure 6-19 through Figure 6-23.

The data used to develop these figures contain noise that is expected considering the uncertainty associated with the measurement equipment and the movement of sediment bed forms along the bottom of the channel. Using average elevations from each HEC-6T control volume somewhat reduces this noise. A general trend of increasing bed elevation indicates deposition in the navigation channel. A general trend of decreasing bed elevation could be due to dredging or erosion, or both. However, it can be

assumed that dredging is the most significant process affecting declining bed elevations.

Figure 6-19. Surveyed navigation channel elevations RM 5.5 to RM 2.46.

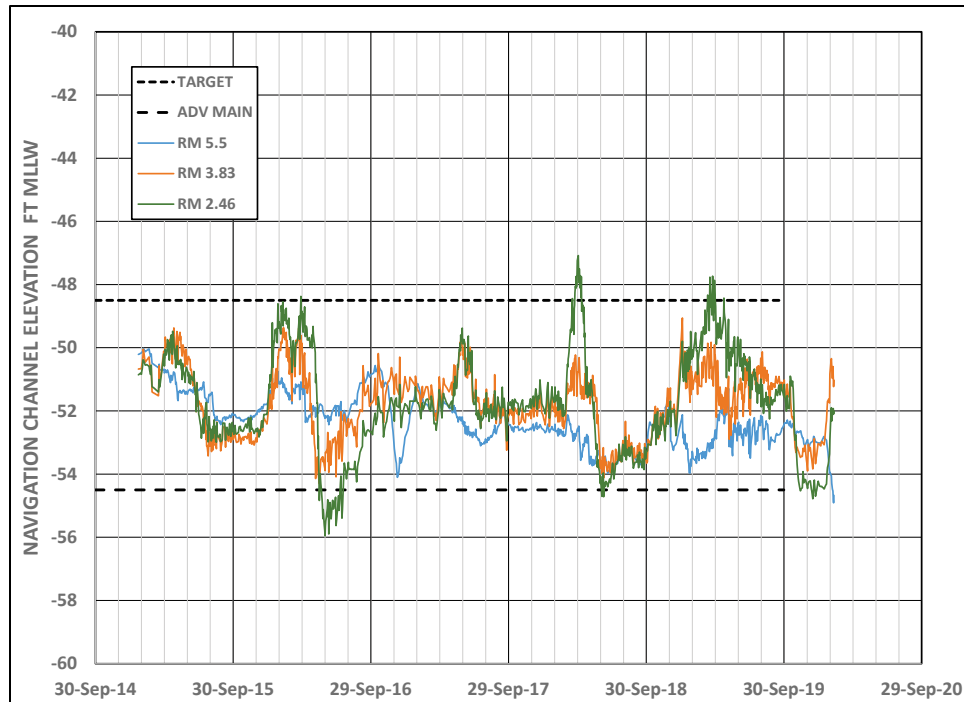


Figure 6-20. Surveyed navigation channel elevations RM 1.6 to RM 0.40.

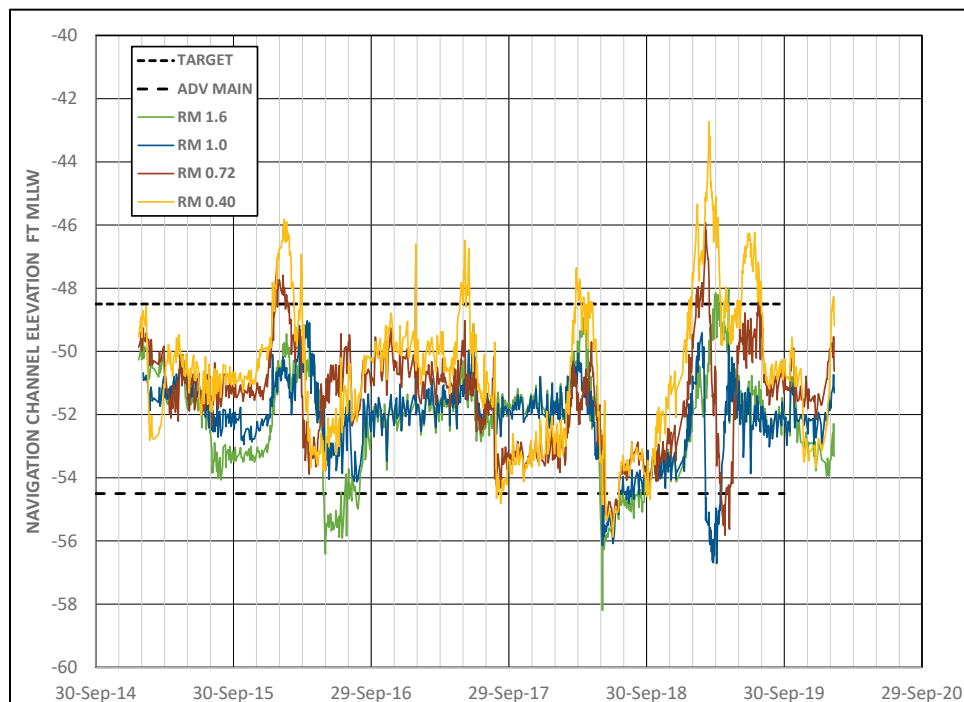


Figure 6-21. Surveyed navigation channel elevations RM -0.1 to RM -3.1.

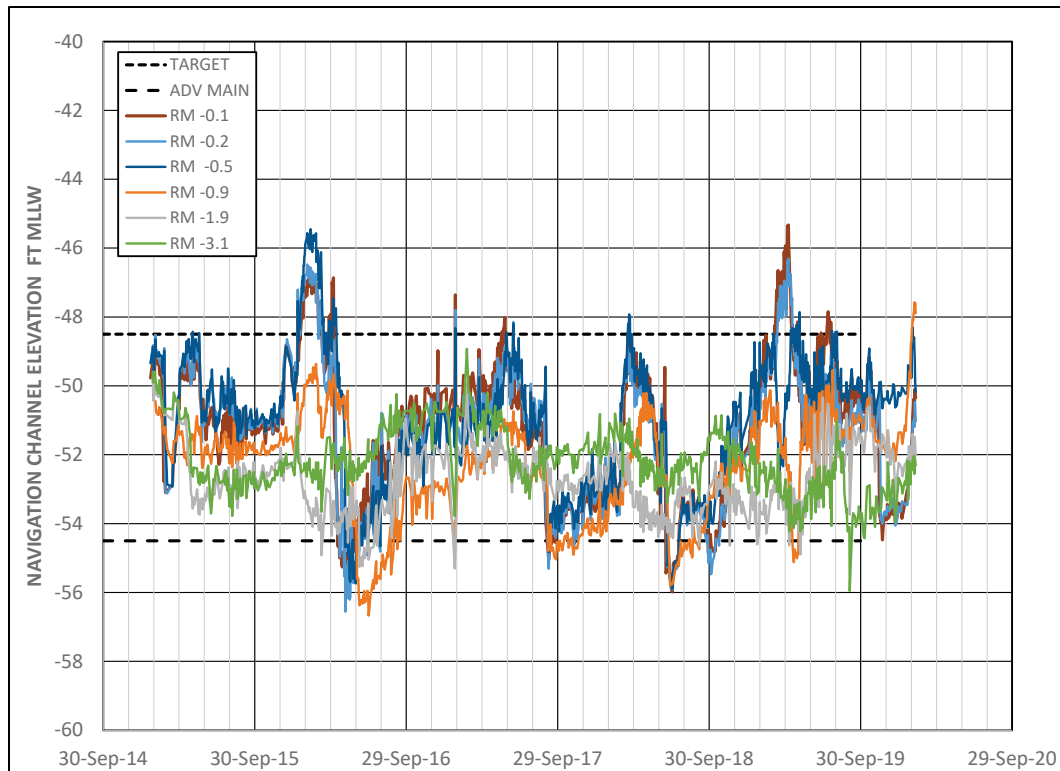


Figure 6-22. Surveyed navigation channel elevations RM -3.1 to RM -10.7.

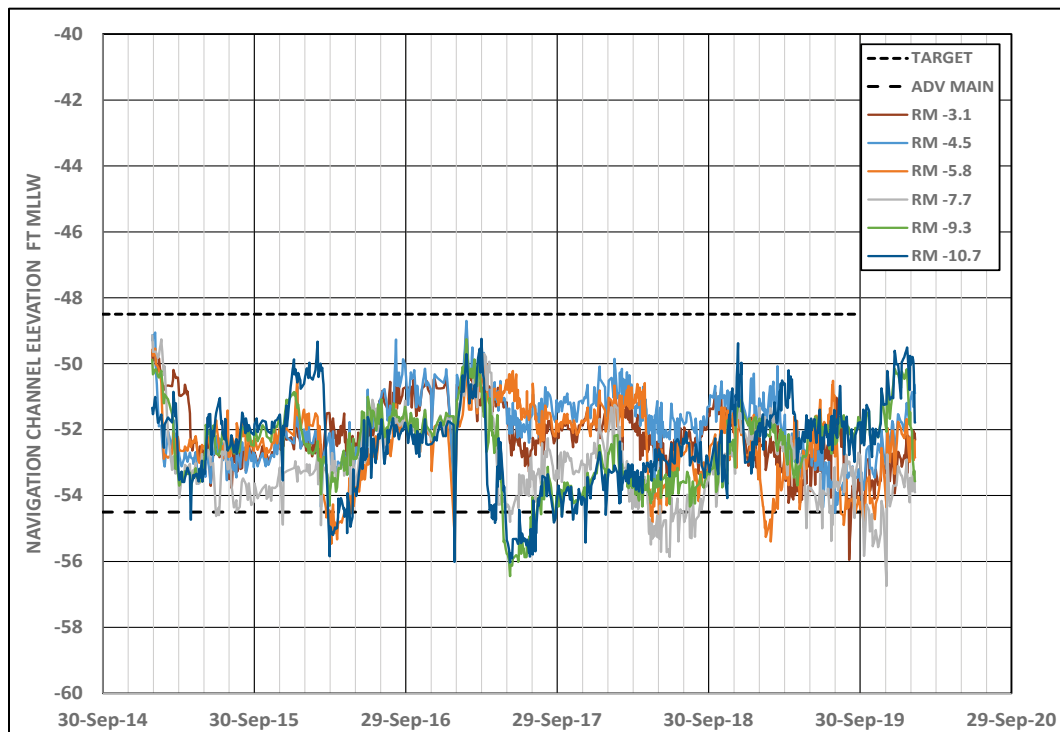
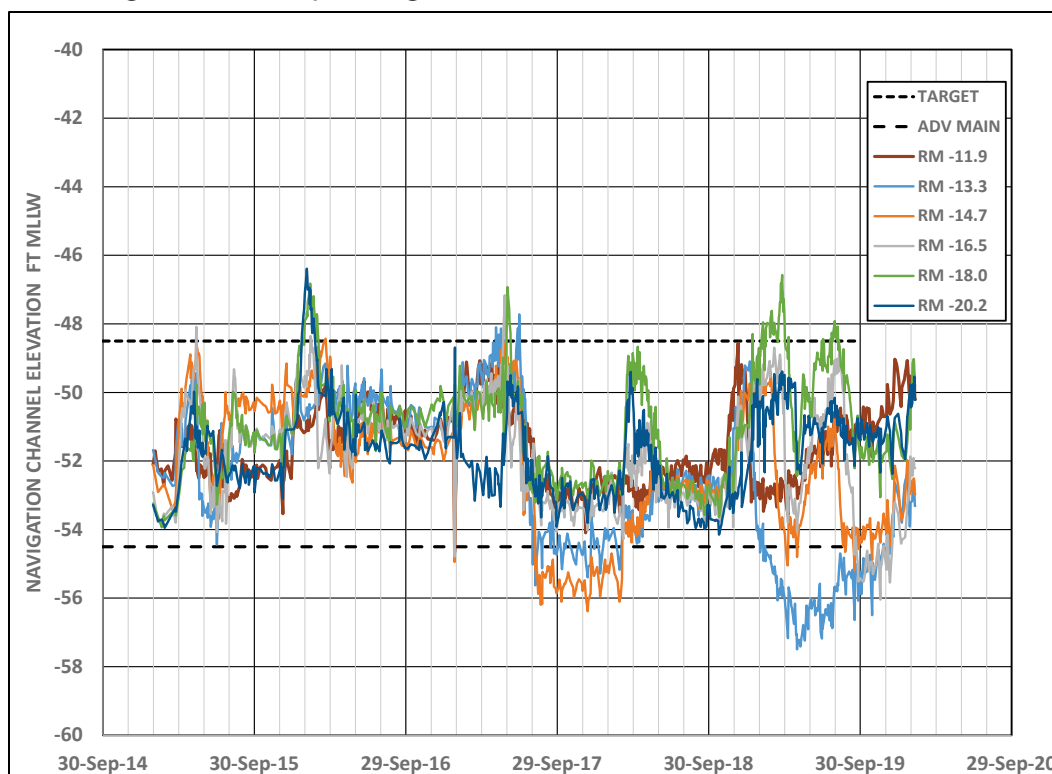


Figure 6-23. Surveyed navigation channel elevations RM -11.9 to RM -20.2.



The HEC-6T Structural Modifications Model, discussed in previous sections of this report had been calibrated and verified to reproduce average annual dredging over a period of years. It used a dredging trigger elevation of -48.5 ft (MLLW) minus an additional 2 ft to account for overdredging, with 6 ft of advanced maintenance. The 2015–2019 survey data demonstrate how the simplistic dredging protocol assigned in the Structural Modifications Model does not mimic the actual daily dredging operations.

Typical dredging protocol is for hopper dredges to prioritize the worst shoaling areas to maintain the authorized 48.5 ft (MLLW) channel depth (50 ft MLLW after 2020). This initial dredging effort does not necessarily include advanced maintenance. If cutterhead dredges are available, they typically dredge down to include advanced maintenance. After the annual shoaling season passes, the protocol is to dredge the channel down to -54.5 ft MLLW.

Note that this dredging protocol was not always adhered to exactly between 2015 and 2019. Due to the inherent uncertainties on the timing and magnitude of shoaling patterns and MVN's emphasis on maintaining a navigable channel throughout the shoaling season, sometimes dredging

commenced before the -48.5 ft elevation was reached. At other times, dredging did not continue all the way down to elevation -54.5 ft. Due to these variations it is not surprising that there are differences in computed and reported dredging volumes.

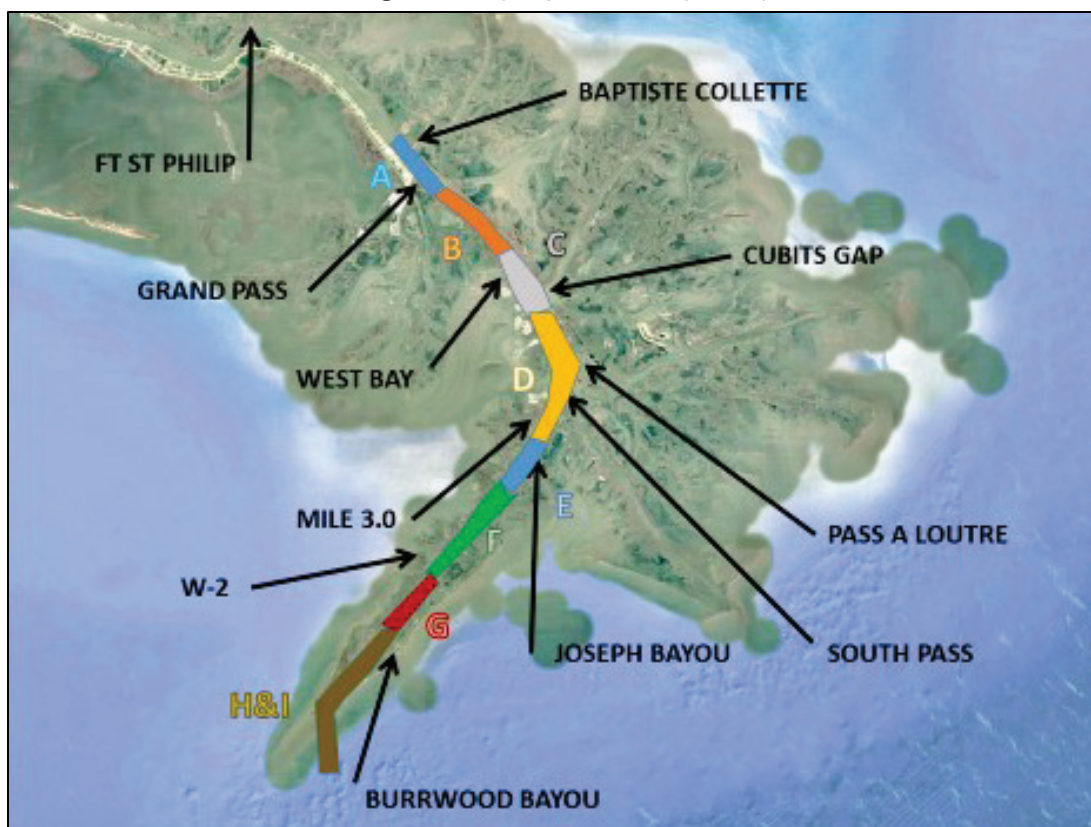
6.5 Evaluation of Daily Dredging Operations

The HEC-6T model was modified to simulate daily measured deposition and dredging during the 2015–2019 water years. The objective was to demonstrate the model's capability to simulate dredging operations with the intent of eventually developing a protocol for assessing operation options prior to annual flood events. The input hydrograph for such a protocol would be the NOAA 28-day predicted discharge at some upstream location, for example, Vicksburg or Tarbert Landing. Variables to determine the dredging protocol would be the timing and capacity of assigned hopper dredges, dredging depths, and dredging locations. The HEC-6T model is capable of doing this already, but verification using the 2019 eHydro data is essential to establish confidence in results.

The HEC-6T model used for the structural modification tests had been calibrated and verified to reproduce reported average annual dredging volumes. Then a 50 yr hydrograph was simulated to predict future average annual dredging volumes. To evaluate the daily dredging operations, only the 1991–2019 historical hydrograph was used, focusing on deposition and dredging for WYs 2015–2018.

Several modifications to the Structural Modifications Model were introduced to provide more detail. The number of dredging reaches was increased to nine from the two used in the original model (Figure 6-24). Dredging capacity was introduced to represent actual operations more closely. Dredging capacity was assigned based on actual operations during 2019. Dredging commenced on January 1 of each year and progressed at the designated rate until the advanced maintenance elevation was reached. Two cycles were assigned so that any additional deposition that occurred during the initial upstream sweep could be accounted for. These modifications were not sufficient to duplicate actual daily conditions.

Figure 6-24. Assigned dredging reaches for Daily Operations Model (maps data: Google Earth, Image Landsat/Copernicus, 12/2017).



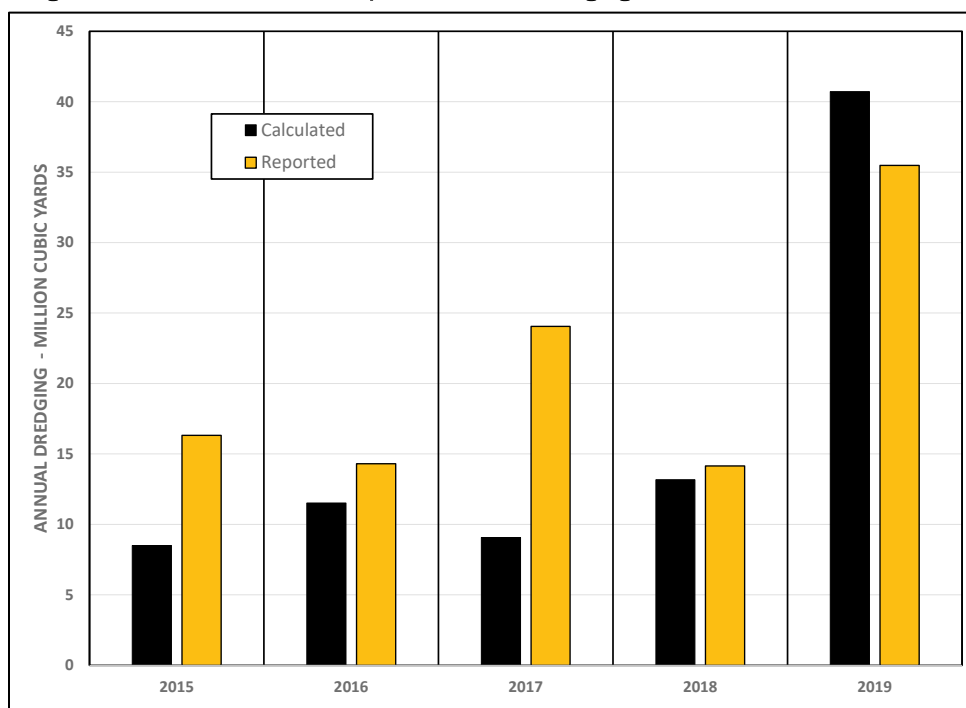
Additional modifications to the model were incorporated. These included allowing deposition on the point bars upstream of HOP, adjusting dredging capacity at different locations above HOP, changing the number of dredging cycles to allow for continuous dredging throughout the year in reaches D, H, and I, removing the 2 ft overdredging from the trigger elevation, and reducing the advance maintenance to 3 ft based on the surveyed navigation channel bed elevations. In addition, the sand diversion percentages that had been reduced 25% in the Structural Modifications Model were increased back to 100% of that calculated using the Rouse equation. The silt deposition coefficients were adjusted to produce calculated bed-material gradations more in line with measured bed-material samples. Revised fine sediment erosion and deposition coefficients, which are shown in Table 6-4, included moving the fresh water limit down to RM 3.

Table 6-4. Revised fine sediment coefficients.

Coefficient	Variable Name in HEC-6T	Southwest Pass	River Miles 0.0 to 3.0	Fresh Water	Units
Shear threshold for clay deposition	DTCL	0.012	0.008	0.001	lb/ft ²
Shear threshold for silt deposition	DTSL	0.035	0.007	0.002	lb/ft ²
Shear threshold for erosion of silt and clay particles	STCD	0.040	0.025	0.025	lb/ft ²
Shear threshold for mass erosion	STME	0.04	0.04	0.04	lb/ft ²
Erosion rate at STME	ERME	0.001	0.001	0.001	lb/ft ² /hr
Slope of the erosion rate curve for mass erosion	ER2	0.03	0.03	0.03	1/hr

Calculated and reported annual dredging for years 2015 and 2019 were compared for the “Daily Operations Model.” Reported dredging AHP and in SWP, between 2015 and 2019, was provided by MVN DOM and cutterhead dredge data and was assigned daily values as described in Chapter 2 of this report. The reported dredging volumes are compared to calculated annual volumes in Figure 6-25. These volumes do not include agitation dredging.

Figure 6-25. Calculated and reported annual dredging SWP and HOP 2015–2019.

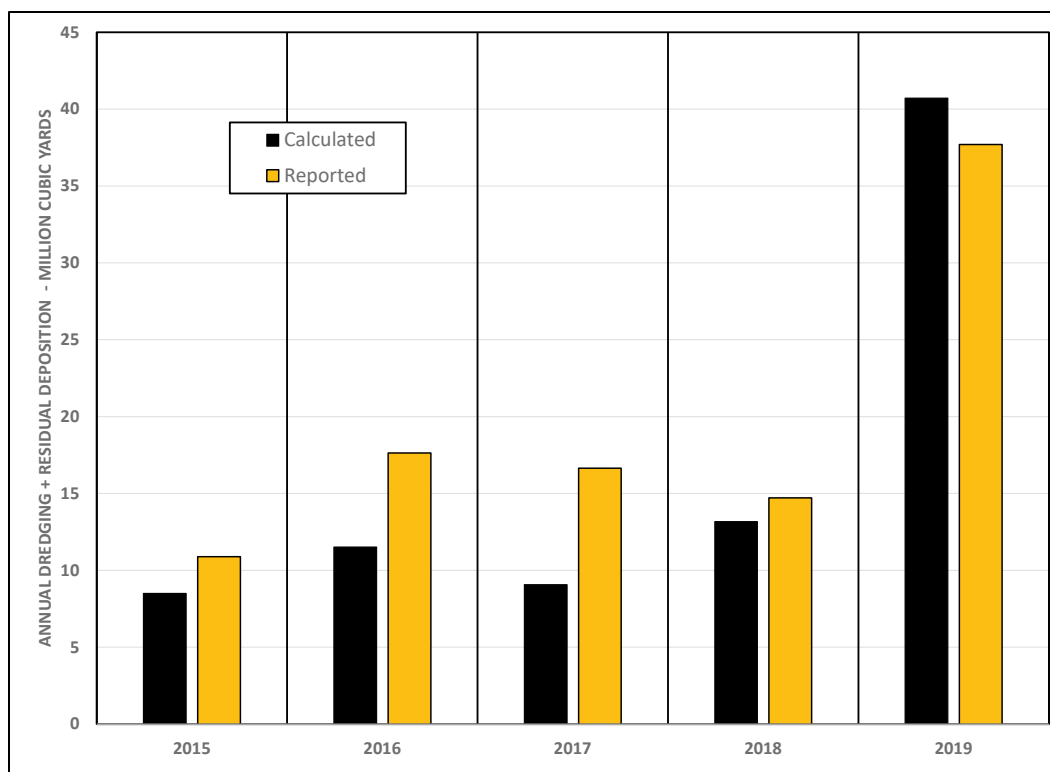


Complete agreement is not expected for several reasons. The HEC-6T operation protocol calls for dredging to commence when deposition exceeds the authorized bed elevation of -48.5 MLLW, and then dredging continues until the advance maintenance elevation is reached. Authorized advance maintenance is 6 ft, but survey data suggest that this elevation is not typically achieved each year in every reach. HEC-6T advance maintenance was set at 3 ft. Actual operation protocol is governed by availability of dredges and location of the most critical condition among other things. The actual dredging operation may commence before deposition reaches elevation -48.5 to get ahead of upcoming deposition, and it may not continue on to the authorized advance maintenance elevation of -54.5 to address critical needs elsewhere.

After the flood peak has passed, dredging may be conducted in areas where the bed elevation is less than the authorized bed elevation down to the advance maintenance elevation as part of a beneficial use project. Another issue with the numerical model is that deposition can occur in the navigation channel where it is subject to dredging, or it can occur outside the navigation channel where it is not.

Calculated and reported annual deposition in the SWP and HOP reaches can also be compared. Deposition in the reported data includes only sediment deposited in the navigation channel while HEC-6T includes deposition outside the navigation channel. HEC-6T total deposition is calculated by combining the annual dredging and residual deposition for the control volume associated with each cross section. The reported total deposition is calculated by combining the reported dredging and measured residual deposition from each bin. This comparison is shown in Figure 6-26. Results are similar to those for annual dredging shown in Figure 6-25, providing additional confidence in the model's ability to reproduce reasonable deposition results.

Figure 6-26. Calculated and reported annual dredging plus residual deposition, SWP and HOP 2015–2019.



The distribution of dredging in SWP and AHP was identified by adopting nine dredging reaches labeled alphabetically, A through I. Reaches H and I were eventually combined due to the poor definition of model cross sections and paucity of survey data in the bar channel (Reach I). The corresponding HEC-6T cross sections in each of these reaches are shown in Table 6-5.

Table 6-5. Designated reaches for dredging analysis.

Reach	HEC-6T Cross Section
A	11.5, 11.05, 10.7
B	9.5, 8.1, 6.7
C	5.5, 3.83
D	2.46, 1.6, 1.0, 0.72, 0.4, -0.1, -0.2, -0.5, -0.9, -1.9, -3.1
E	-4.5 -5.8
F	-7.7, -9.3
G	-10.7, -11.9
H + I	-13.3, -14.7, -16.5, -18.0, -20.2, -20.5, -21, -22

Calculated annual dredging and residual deposition for WYs 2015 through 2019 are shown in Table 6-6. Measured data are shown in Table 6-7. The data are plotted in Figure 6-27. Calculated data do not include agitation dredging.

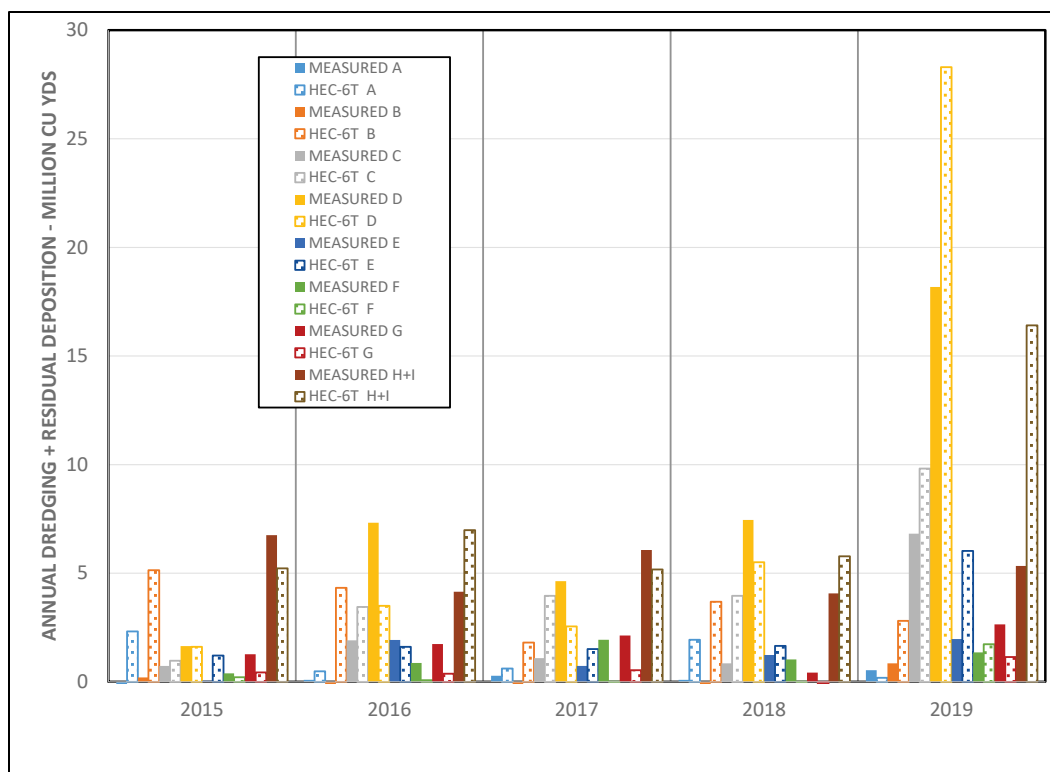
Table 6-6. Calculated annual dredging and residual deposition in cubic yards.

Reach	2015	2016	2017	2018	2019
A	2,318,000	485,000	612,000	1,940,000	186,000
B	5,141,000	4,332,000	1,809,000	3,687,000	2,807,000
C	972,000	3,453,000	3,958,000	3,959,000	9,816,000
D	1,611,000	3,503,000	2,555,000	5,508,000	28,298,000
E	1,213,000	1,611,000	1,515,000	1,660,000	6,031,000
F	215,000	75,000	11,000	10,000	1,735,000
G	439,000	377,000	538,000	-75,000	1,142,000
H + I	5,225,000	6,989,000	5,174,000	5,784,000	16,412,000

Table 6-7. Measured annual dredging and residual deposition in cubic yards.

Reach	2015	2016	2017	2018	2019
A	-154,000	97,000	285,000	93,000	536,000
B	205,000	-392,000	-242,000	-463,000	855,000
C	732,000	1,911,000	1,094,000	853,000	6,825,000
D	1,645,000	7,325,000	4,639,000	7,453,000	18,176,000
E	49,000	1,928,000	733,000	1,237,000	1,973,000
F	390,000	870,000	1,935,000	1,032,000	1,359,000
G	1,272,000	1,741,000	2,135,000	430,000	2,646,000
H+I	6,752,000	4,150,000	6,067,000	4,077,000	5,334,000

Figure 6-27. Calculated and measured annual dredging plus residual deposition, 2015–2019.



Most of the shoaling (dredging plus residual deposition) occurs in Reaches D and H+I. Calculated shoaling in Reach D is greater than the reported shoaling in 2019, but the reported shoaling is greater than calculated shoaling in the other years. Some years have more calculated dredging in Reach H+I than reported shoaling; other years have more reported shoaling in Reach H+I than calculated shoaling. Calculated shoaling is greater than reported shoaling in Reaches A, B, and C. Downstream from Reach D, in SWP, the calculated shoaling is greater than reported shoaling in Reach E (except for 2016); the reported is greater than calculated in Reach F (except for 2019); and the reported is greater than calculated in Reach G.

Baptist Collette and Grand Pass are located in Reach A. It is surprising that there is negligible shoaling related to a 20% diversion through these two distributaries.

Differences in Reach D results could be due to differences between the assumed operations and the actual operations. It is also possible that in 2019, it was not possible to maintain the advanced maintenance depths. The measured data are missing shoaling outside of the navigation channel.

The differences between calculated and reported dredging volumes in SWP may be due to the condition of spur dikes in SWP. The spur dikes in the HEC-6T model were originally modeled at design heights and lengths. Lacking any other data, the model spur dikes in this study still represent design conditions. However, field observations have confirmed that these dikes have degraded in many locations. Thus, the reported data represent degraded (existing) conditions.

The fine sediment deposition coefficients need to be refined. Currently the model assigns the coefficients based on location. They should be based on discharge because at higher discharges the salt wedge is moved downstream. This issue could explain why there is such a discrepancy in 2019 measured and calculated shoaling. The HEC-6T code currently is not able to assign the fine sediment deposition coefficients based on discharge.

6.6 Calculated and Measured Shoaling Rates

The HEC-6T model can be used to predict the onset of significant annual shoaling in the SWP and HOP dredging reaches. One advantage of the HEC-6T model is that it accounts for upstream influences including sediment inflow from the Middle Mississippi, Ohio, Arkansas, and White Rivers, as well as for other smaller tributaries. It also accounts for antecedent conditions in the supply reaches upstream. This is particularly important because sediment supply to the Mississippi River below Venice is a function of sediment aggradation and degradation that occurred in upstream reaches decades ago. To demonstrate this capability, the 2015–2019 survey data were compared to HEC-6T calculated shoaling rates.

The first step was to model annual deposition without dredging. This was done for each year between 2015 and 2018. The calculated results were successful in reproducing the location and timing of the rapid rise in deposition associated with the annual hydrograph. However, the 2019 hydrograph produced too much deposition to analyze prototype conditions without dredging.

The objective was to show that the model can predict both the onset of significant shoaling and the approximate volume of shoaling. The daily survey data are not ideal for this purpose because they only include volume changes in the navigation channel. The HEC-6T model includes deposition outside the navigation channel, which is significant between

Venice and HOP. However, the survey data are suitable for determining the onset of shoaling. HEC-6T calculated deposition is expected to be greater than the survey deposition in Reach D, which extends from RM -3.1 BHP to 2.46 AHP. The survey data include both surveyed bed change and dredging volumes as these represent the total deposition. The HEC-6T model was modified to calculate only deposition during each of the 5 yr (2015–2019). This modification removed the influence of the modeling dredging protocol, which did not conform to the prototype protocol. This approach worked for 2015–2018. Deposition in 2019 was so great that the model river channel filled without dredging.

The 2019 HEC-6T calculations included dredging plus deposition for 2019. Results are shown in Figure 6-28 through Figure 6-32. The model successfully duplicated the commencement of significant shoaling in the primary deposition reach upstream from HOP. Commencement of shoaling in the jetty and bar channel reaches was also duplicated successfully, but total volumes were too dependent on dredging operations in the reach to be consistent.

Figure 6-28. Onset of shoaling—survey and HEC-6T 2015.

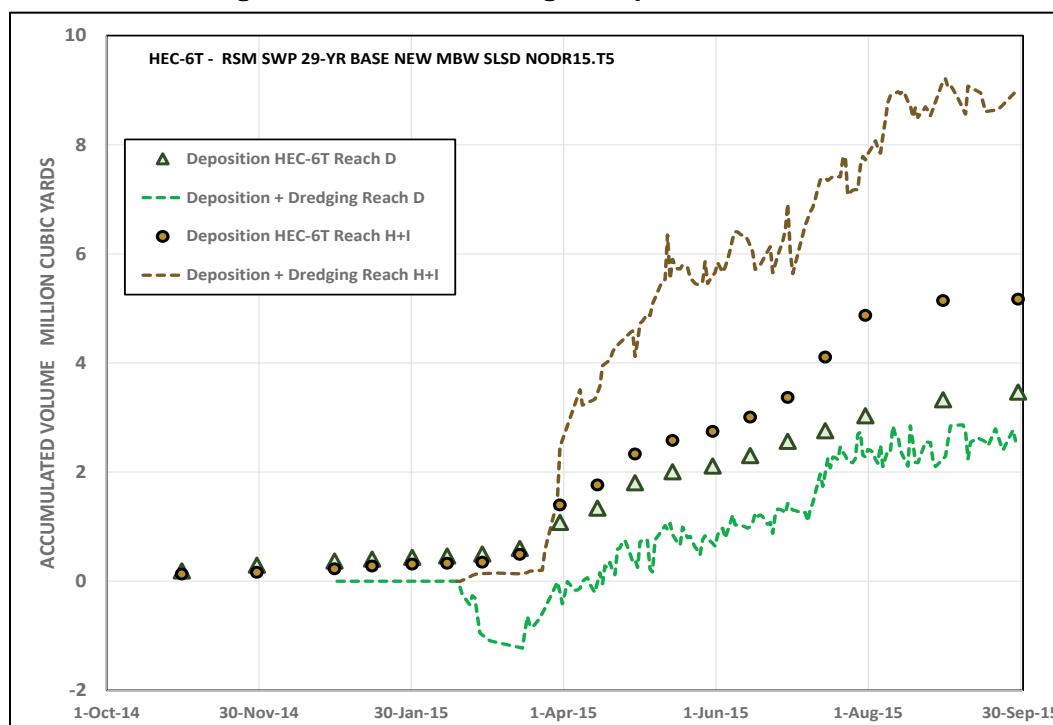


Figure 6-29. Onset of shoaling—survey and HEC-6T 2016.

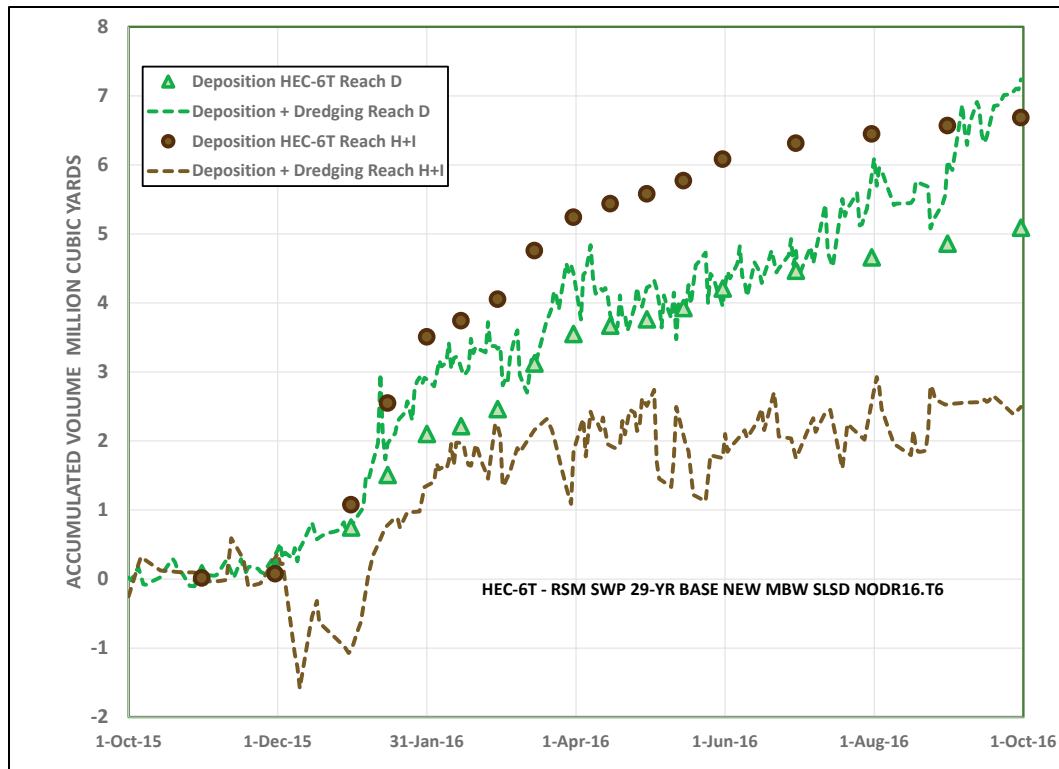


Figure 6-30. Onset of shoaling—survey and HEC-6T 2017.

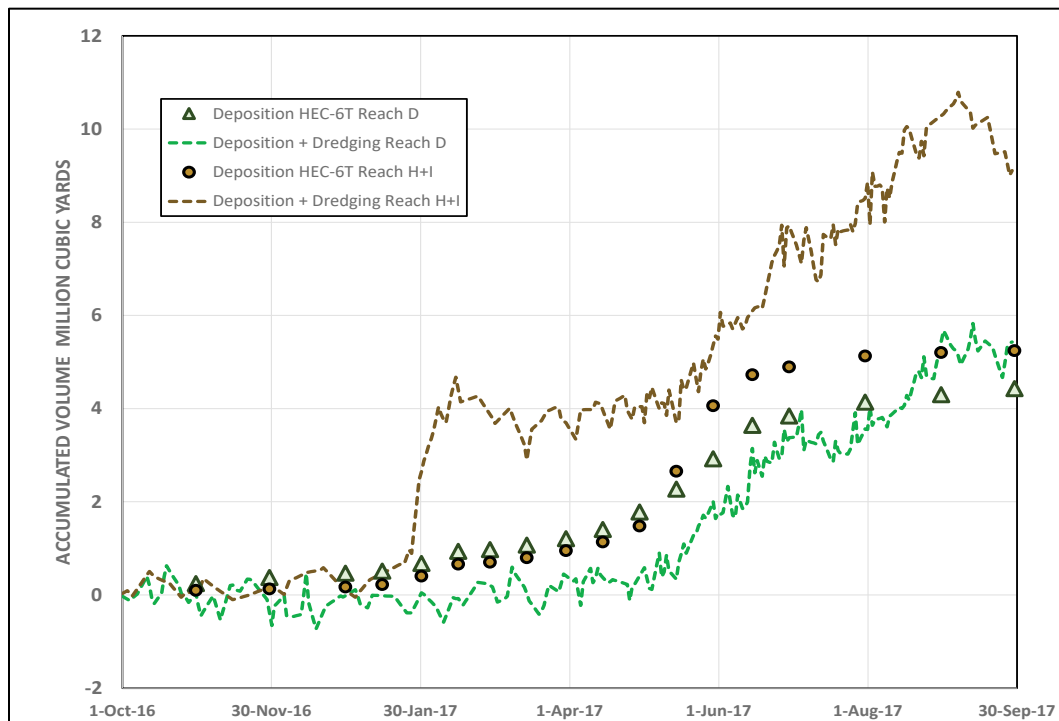


Figure 6-31. Onset of shoaling—survey and HEC-6T 2018.

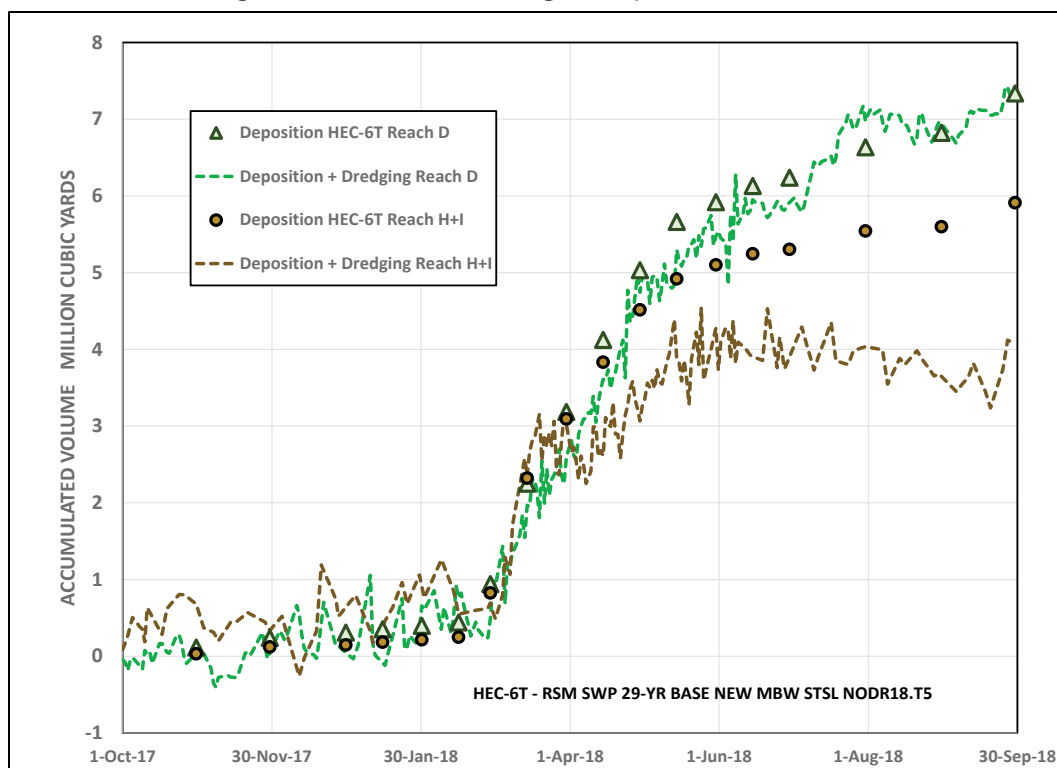
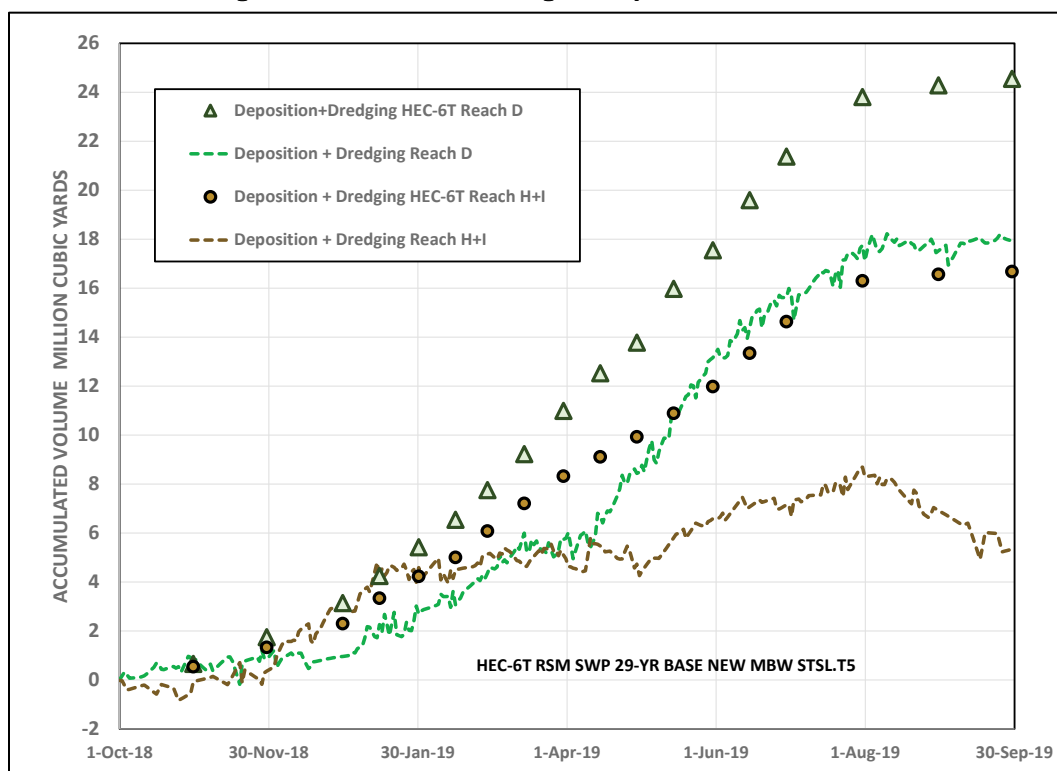


Figure 6-32. Onset of shoaling—survey and HEC-6T 2019.



7 Conclusions and Recommendations

7.1 Conclusions

The purpose of this study is to evaluate possible modifications near the river's HOP to optimize the distribution of water and sediment towards the reduction of shoaling in the navigation channel. This was accomplished using the HEC-6T 1D numerical sedimentation model that has been previously used on several sedimentation studies on the Mississippi River. The model was shown to be appropriate for use in river reaches downstream from Venice (RM 10.7) by reproducing historical dredging records.

Calculations from HEC-6T show a significant aggradation trend between 1991 and 2019 in the Mississippi River upstream from Venice. Currently, this aggradation trend does not affect navigation because the river is deep in this reach. Model results suggest that sediment supply from these cross sections is increasing as sediment transport potential increases with the decrease in water depth and increase in channel velocity. However, there is no indication of a significant antecedent storage component from high flow years before 2016. After 2016, approximately 1 ft of scour occurs on the rise of the annual hydrograph at some cross sections in this reach. This condition will increase the sediment supply to the dredging reaches downstream and annual dredging will be somewhat affected by increasing bed elevations at these upstream cross sections.

The 1D model was shown to be useful for evaluating long-term effects of partial tributary closures on sedimentation processes and dredging AHP and in SWP. It was also shown to be useful to evaluate the effects of channel widening or channel deepening on sedimentation processes and dredging. This was tested in the model using a proposed channel enlargement adjacent to the HDDA. In this study, sedimentation and dredging effects were calculated 50 yr into the future for several structural alternatives.

Calculated results for the various structural modifications were compared to a base test that reflected existing conditions. The change in annual dredging was small for most alternatives tested. The most extreme case was closing all the openings between Ostrica and the Jetties, except for PAL and South Pass. This resulted in approximately a 19% reduction in average

annual dredging. Reducing outflow into individual distributaries resulted in a much smaller decrease in average annual dredging. The calculated reduction in total average annual dredging for the tested alternatives varied between 0.8% for a 50% closure of Joseph Bayou, to 2.4% for a 50% closure of South Pass. Increasing the channel width at HDDA resulted in a 0.6% increase in average annual dredging. Decreasing the flow percentage into SWP resulted in a 1.7% increase in average annual dredging.

Currently, most of the shoaling and dredging occurs just upstream of HOP and in the Jetty and Bar Channel reaches at the end of SWP. Reducing flow and sediment out various distributaries was shown to influence the distribution of shoaling in the study reach. The most significant change in the distribution of shoaling and dredging occurred with the close-all alternative. Closing all the distributaries moved most of the dredging operations down to the Jetty and Bar Channel reaches. In general, test results demonstrated that the implementation of distributary closures would move the dredging operations down toward the Gulf, adjacent or at least close to the ODMDS.

The model can be used to predict the onset of significant shoaling problems AHP. The model accounts for system-wide sedimentation influences that contribute to the shoaling in SWP and AHP. These influences include sediment inflow from the Middle Mississippi and Ohio Rivers as well as other tributaries, deposition patterns from recent annual floods, and long-term sedimentation effects in the river.

7.2 Recommendations

Proposed structural alternatives to modify distributary outflows from the Mississippi River below Belle Chasse can be evaluated with the HEC-6T model to determine sedimentation effects.

The effectiveness of SWP pile dikes can be evaluated with the HEC-6T model. This can be accomplished by collecting survey data to determine the condition and geometry of existing pile dikes in SWP. These data can then be incorporated into the HEC-6T model to determine how much the deteriorated dikes are affecting sedimentation and dredging in SWP. In addition, the effectiveness of various alternative structural alternative fixes can be determined. The current model assumes design pile dike crest elevations and lengths.

Uncertainty related to diversion sediment diversion coefficients could be reduced by collecting bed-material samples in the distributary channels near the point of diversion. The most significant outlets are PAL, South Pass, Cubits Gap, Grand Pass, Baptist Collette, and Fort St. Philip.

Discharge measurements in SWP above and below the four major distributaries over time and for a range of discharges would reduce uncertainty related to the volume of flow lost through these outlets. A program to make these measurements has recently been initiated and should be continued.

The dredging algorithms in HEC-6T should be enhanced to better simulate the unique SWP and AHP dredging operations. These code modifications should be focused on inserting variable trigger and cessation elevations during the annual hydrograph and maintaining the proper bed elevation as a function of assigned dredging capacity.

The HEC-6T code should further be advanced by allowing fine sediment deposition coefficients to vary with discharge to better approximate the influence of salt-water intrusion on deposition in SWP and AHP.

References

- Abraham D., R. Kuhnle, and A. J. Odgaard. 2011. "Validation of Bed Load Transport Measurements with Time Sequenced Bathymetric Data." *ASCE Journal of Hydraulic Engineering* 137 (7) July 2011: 723–728.
- Ariathurai, R., and R. B. Krone. 1976. "Finite Element Model for Cohesive Sediment Transport." *Journal of the Hydraulics Division, ASCE* 2 (3) : 323–33.
- Biedenharn, D. S., M. A. Allison, C. D. Little, C. R. Thorne, and C.C. Watson. 2017. *Large-Scale Geomorphic Change in the Mississippi River from St Louis, MO, to Donaldsonville, LA, as Revealed by Specific Gage Records*. MRG&P Report No. 10. Vicksburg, MS: US Army Engineer Research and Development Center.
- Byrnes, M. R., J. L. Baker, and F. Li. 2002. *Quantifying Potential Measurement Errors and Uncertainties Associated with Bathymetric Change Analysis*. ERDC/CHL CHETN-IV-50. Vicksburg, MS: US Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory.
- Catalyst-Old River Hydroelectric. 1999. *Lower Mississippi River Sediment Study, Vol. 4—HEC-6W Mississippi River Numerical Sedimentation Model Investigation, Vicksburg to Donaldsonville*. Vicksburg MS: US Army Corps of Engineers Waterways Experiment Station, Coastal and Hydraulics Laboratory.
- Copeland, Ronald R. 1991. *Dredging Alternatives Study Cubits Gap, Lower Mississippi River, Report 1, TABS-1 Numerical Model Investigation*. TR HL-90-20. Vicksburg, MS: US Army Corps of Engineers, Waterways Experiment Station.
- Copeland, Ronald R. 2018. *Mississippi River and Tributaries Flowline Assessment: Mississippi River Sedimentation Report*. MRG&P Report No. 24, Vol. 4. Vicksburg, MS: US Army Engineer Research and Development Center.
- Copeland, Ronald R., James Lewis, and Justin Giles 2019. "Long-Term Effects of Dredging Operations in the Lower Mississippi River." In *Proceedings of SEDHYD 2019, Federal Interagency Sedimentation Conference and Federal Interagency Hydrological Modeling Conference*, Reno, NV, June 24–28, Section 7E.
- Copeland, Ronald R., Leslie Lombard, and Roger A. Gaines. 2020. *Numerical Sedimentation Investigation, Mississippi River Cairo to Pilots Station*. MRG&P Report No. 30. Vicksburg, MS: US Army Engineer Research and Development Center.
- Copeland, Ronald R., and William. A. Thomas. 1992. *Lower Mississippi River, Tarbert Landing to East Jetty, Sedimentation Study*. TR HL-92-6. Vicksburg, MS: US Army Corps of Engineers, Waterways Experiment Station.

- Gaines, Roger A., and Anthony M. Priestas. 2016. *Particle Size Distributions of Bed Sediments along the Mississippi River, Grafton, Illinois, to Head of Passes, Louisiana, November 2013*. MRG&P Report No. 7. Vicksburg, MS: US Army Engineer Research and Development Center.
- Hartman, Michael A., Kenneth N. Mitchell, Lauren M. Dunkin, James Lewis, Benjamin Emery, Nicholas F. Lenssen, and Ronald R. Copeland. 2022. "Sedimentation and Dredging Data Analysis." *Journal of Waterway, Port, Coastal, and Ocean Engineering*, ASCE 148(2).
- Heath, Ronald E., Gary L. Brown, Charles D. Little, Thad C. Pratt, Jay J. Ratcliff, David D. Abraham, David Perkey, Naveen B. Ganesh, Keith Martin, and David P. May. 2015. *Old River Control Complex Sedimentation*. MRG&P Report No. 6. Vicksburg, MS: US Army Engineer Research and Development Center.
- Heath, Ronald E., Marielys Ramos-Villanueva, Gary L. Brown, and Ian E. Floyd. 2017. *Mississippi River Ship Channel, Gulf of Mexico to Baton Rouge, LA: Integrated General Reevaluation Report and Supplemental Environmental Impact Statement, April 2018, Appendix H, Hydrology and Hydraulic Reports. Annex 1, Mississippi River Ship Channel Deepening Study, One-Dimensional Numerical Sedimentation Model Investigation*. Vicksburg, MS: US Army Engineer Research and Development Center.
- Hutchings, Henry Jr. 1933. *Report on Hydraulic Study for Eliminating Maintenance Dredging in Southwest Pass*. New Orleans LA: War Department, US Engineer Office, First New Orleans District.
- Kolb, C. R. 1962. *Distribution of Soils Bordering the Mississippi River from Donaldsonville to Head of Passes*. TR No. 3-601. Vicksburg, MS: US Army Waterways Experiment Station.
- Krone, R. B. 1962. *Flume Studies of the Transport of Sediment in Estuarial Shoaling Processes*. Berkeley, CA: Hydraulic Engineering Laboratory, University of California.
- Meyer-Peter, E., and R. Muller. 1948. "Formulas for Bed-Load Transport." In *Second Meeting of the International Association for Hydraulics Research*, Appendix 2, 39–64. Stockholm, Sweden.
- Nordin, Carl F., and Bryan S. Queen. 1992. *Particle Size Distributions of Bed Sediments along the Thalweg of the Mississippi River, Cairo, Illinois to Head of Passes, September 1989*. Potamology Program (P-1) Report 7. Vicksburg, MS: US Army Corps of Engineers, Lower Mississippi Valley Division.
- Parthenaides, E. 1965. "Erosion and Deposition of Cohesive Soils." *Journal of the Hydraulics Division, ASCE* 91 (1) : 755–771. Reston, VA.
- Rouse, H. 1937. "Modern Conceptions of the Mechanics of Fluids Turbulence." *Transactions, ASCE* 102 (Paper No. 1965) : 463–454. ASCE.

- Sharp, Jeremy A., Charles D. Little, Gary L. Brown, Thad C. Pratt, Ronnie E. Heath, Lisa C. Hubbard, C. Freddie Pinkard, Martin S. Keith, Nathan D. Clifton, David W. Perky, and Naveen B. Ganesh. 2013. *West Bay Sediment Diversion Effects*. ERDC/CHL TR-13-15. Vicksburg, MS: US Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory.
- Thomas, William A. 2012. *Allocation of Water and Sediment Resources—Myrtle Grove Diversion for Land Building*. Prepared for Coastal Protection and Restoration Authority, Office of Coastal Protection and Restoration, State of Louisiana, by Mobile Boundary Hydraulics, PLLC, Clinton, MS.
- Thomas, William A. 2021. *Sedimentation in Stream Networks (HEC-6T)*. User Manual. Clinton, MS: Mobile Boundary Hydraulics, PLLC.
- Thomas, William A., and Howard Chang. 2008. "Computational Modeling of Sedimentation Processes (Chapter 14)." In *Sedimentation Engineering*, edited by Marcelo H. Garcia. ASCE Manuals and Reports on Engineering Practice No. 110. ASCE: Reston, VA.
- Thomas, William A., Ronald E. Heath, J. P. Stewart, and D. G. Clark. 1989. *The Atchafalaya River Delta, Report 5: The Atchafalaya River Delta Quasi-Two-Dimensional Model of Delta Growth and Impacts on River Stages*. Technical Report HL-82-15, Report 5. Vicksburg, MS: US Army Waterways Experiment Station, Coastal and Hydraulics Laboratory.
- Thomas, William A., Michael J. Trawle, Ronald E. Heath, Carol P. Richards, and Cherie P. Price. 2018. *HEC-6T One-Dimensional Model Study*. Louisiana Coastal Area Program, Mississippi River Hydrodynamic and Delta Management Study, Prepared for Coastal Protection and Restoration Authority, Office of Coastal Protection and Restoration, State of Louisiana, by USACE, ERDC, Vicksburg MS.
- Toffaleti, Fred B. 1968. *A Procedure for Computation of the Total River Sand Discharge and Detailed Distribution, Bed to Surface*. Technical Report No. 5. Vicksburg, MS: Committee on Channel Stabilization, US Army Corps of Engineers. <http://libweb.wes.army.mil/uhtbin/hyperion/TR-5-1968.pdf>.
- USACE HEC (US Army Corps of Engineers, Hydrologic Engineering Center). 1992. *Guidelines for the Calibration and Application of Computer Program HEC-6*. Training Document No. 13. Davis, CA: Hydrologic Engineering Center.
- USACE, HQ (US Army Corps of Engineers, Headquarters). 1994. *Hydraulic Design of Flood Control Channels*. EM 1110-2-1601, Washington, DC: US Army Corps of Engineers.
- USACE, HQ. 2006. *Silent Inspector Implementation Guidance*. MG Riley April 2006 Memorandum. Regulatory Guidance Letter 06-04. Washington, DC: US Army Corps of Engineers.

- USACE, HQ. 2019. *Incorporating Sea Level Change in Civil Works Programs*. ER 1100-2-8162. Washington, DC: US Army Corps of Engineers.
https://www.flseagrant.org/wp-content/uploads/USACE_SLR_guidance_ER_1100-2-8162.pdf.
- USACE MVN (US Army Corps of Engineers, New Orleans District). 1980. *Mississippi River and Tributaries, Old River Control, LA, Auxiliary Structure, Design Memorandum No. 17, Hydraulic Design*. New Orleans, LA: US Army Corps of Engineers, New Orleans District.
- USACE MVN. 2017a. *MLG to MLLW Vertical Datum Conversion. Mississippi River, Venice, Louisiana to the Gulf of Mexico (Vicinity of Southwest Pass)*. Engineering Document Report ERD-OD-01, Revision 1. New Orleans, LA: US Army Corps of Engineers, New Orleans District.
- USACE MVN. 2017b. *Mississippi River Ship Channel Gulf of Mexico to Baton Rouge, LA, Integrated General Reevaluation Report and Supplemental Environmental Impact Statement, April 2018, Appendix K. Dredged Material Management Plan Preliminary Assessment Summary of Findings and Recommendations November 2017*. New Orleans, LA: US Army Corps of Engineers, New Orleans District.
- WES (Waterways Experiment Station). 1935. *Studies of River Bed Materials and Their Movement with Special Reference to the Lower Mississippi River*. Paper No. 17. Vicksburg, MS: US Army Engineer Waterways Experiment Station.

Abbreviations

AHP	Above Head of Passes
BHP	Below Head of Passes
DQM	Dredging Quality Management
DS	Downstream
DTCL	Clay
DTSL	Silt
ERDC	US Army Engineer Research and Development Center
ERME	Erosion rate at STME
ER2	Slope of the erosion rate curve for mass erosion
HDDA	Hopper Dredge Disposal Area
HOP	Head of Passes
LWRP	Low Water Reference Plane
MBH	Mobile Boundary Hydraulics
MLG	Mean Low Gulf
MLLW	Mean lower low water
MR&T	Mississippi River and Tributaries
MVN	New Orleans District
NAVD88	North American Vertical Datum of 1988
NGVD29	National Geodetic Vertical Datum of 1929
NOAA	National Oceanic and Atmospheric Administration
ODMDS	Ocean Dredged Material Disposal Site

1D	One-dimensional
ORCC	Old River Control Complex
ORO	Outflow Channel
PAL	Pass a Loutre
RM	River Mile
STCD	Shear threshold for erosion of silt and clay particles
STME	Shear threshold for mass erosion
SP	South Pass
SWP	Southwest Pass
TIN	Triangulated Irregular Networks
UPT	Union Point
USACE	US Army Corps of Engineers
USGS	US Geological Survey
WY	Water Year

REPORT DOCUMENTATION PAGE

1. REPORT DATE August 2023		2. REPORT TYPE Final report		3. DATES COVERED <table border="1" style="width: 100%; border-collapse: collapse;"> <tr> <td style="width: 50%; vertical-align: top;"> START DATE FY21 </td> <td style="width: 50%; vertical-align: top;"> END DATE FY22 </td> </tr> </table>		START DATE FY21	END DATE FY22	
START DATE FY21	END DATE FY22							
4. TITLE AND SUBTITLE Evaluation of Structural and Operational Alternatives to Optimize the Distribution of Water and Sediment in the Passes of the Mississippi River								
5a. CONTRACT NUMBER		5b. GRANT NUMBER		5c. PROGRAM ELEMENT				
5d. PROJECT NUMBER		5e. TASK NUMBER		5f. WORK UNIT NUMBER				
6. AUTHOR(S) Ronald R. Copeland, Michael A. Hartman, and James Lewis								
7. PERFORMING ORGANIZATION NAME(S) AND ADDRESS(ES) US Army Engineer Research and Development Center (ERDC) Coastal and Hydraulics Laboratory (CHL) 3909 Halls Ferry Road Vicksburg, MS 39180-6199				8. PERFORMING ORGANIZATION REPORT NUMBER ERDC/CHL TR-23-8				
9. SPONSORING/MONITORING AGENCY NAME(S) AND ADDRESS(ES) US Army Engineer Research and Development Center (ERDC) 3909 Halls Ferry Road Vicksburg, MS 39180-6199				10. SPONSOR/MONITOR'S ACRONYM(S) ERDC				
11. SPONSOR/MONITOR'S REPORT NUMBER(S)								
12. DISTRIBUTION/AVAILABILITY STATEMENT Distribution Statement A. Approved for public release; distribution is unlimited.								
13. SUPPLEMENTARY NOTES Funding Account Code 121BG5; AMSCO Code 08303								
14. ABSTRACT Mississippi River shoaling and dredging processes in the vicinity of Head of Passes and in Southwest Pass were investigated. Existing rates of deposition and dredging were determined using near-daily eHydro bathymetric surveys, National Dredging Quality Management dredge operating data, and geospatial processing steps developed for this study. These surveys provide a means to characterize the highly dynamic and variable sedimentation patterns observed in the navigation channel. The HEC-6T one-dimensional numerical sedimentation model was used to evaluate possible modifications to the distribution of water and sediment in the Mississippi River near Head of Passes in an attempt to reduce shoaling in the navigation channel. The model was used to evaluate the effects of partial closures of several distributaries downstream from Venice and to evaluate the effects of channel widening and channel deepening adjacent to the Hopper Dredge Disposal Area at Head of Passes. In this study, various structural alternatives were compared to a base test that represented existing conditions. Sedimentation and dredging effects were projected 50 years into the future.								
15. SUBJECT TERMS Dredging; Geomorphology; Hydrodynamics; Mississippi River; River engineering; Sedimentation and deposition; Sediment transport								
16. SECURITY CLASSIFICATION OF: <table border="1" style="width: 100%; border-collapse: collapse;"> <tr> <td style="width: 33%; vertical-align: top;"> a. REPORT Unclassified </td> <td style="width: 33%; vertical-align: top;"> b. ABSTRACT Unclassified </td> <td style="width: 33%; vertical-align: top;"> c. THIS PAGE Unclassified </td> </tr> </table>				a. REPORT Unclassified	b. ABSTRACT Unclassified	c. THIS PAGE Unclassified	17. LIMITATION OF ABSTRACT SAR	
a. REPORT Unclassified	b. ABSTRACT Unclassified	c. THIS PAGE Unclassified						
18. NUMBER OF PAGES 136				19a. NAME OF RESPONSIBLE PERSON David W. Perkey				
19b. TELEPHONE NUMBER (include area code) 601-634-2736								