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

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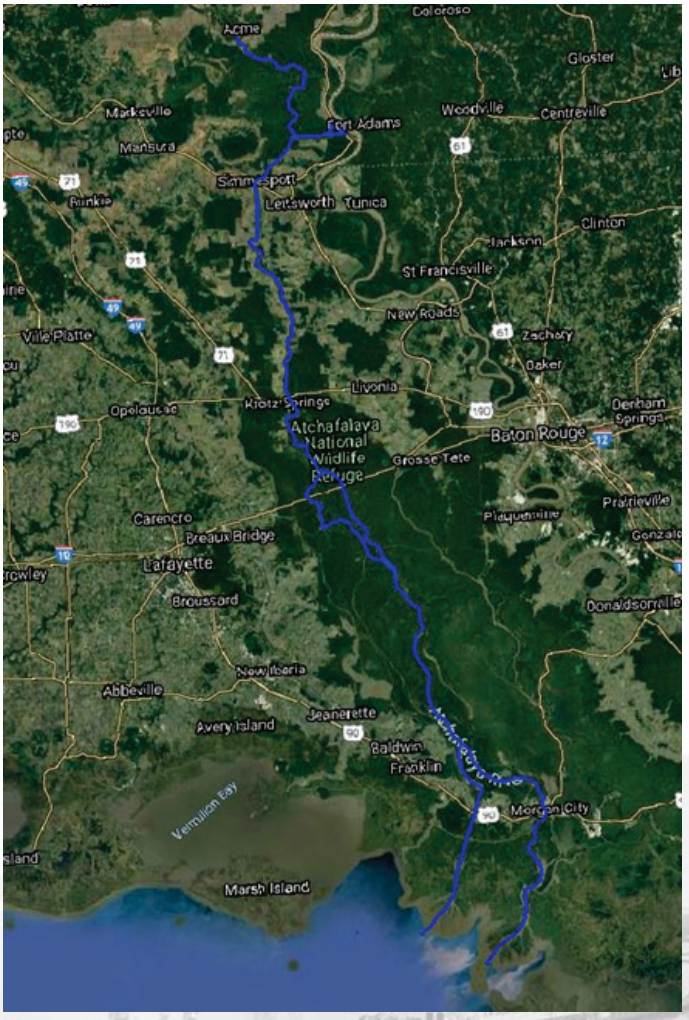

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Technical Assessment of the Old, Mississippi, Atchafalaya, and Red (OMAR) Rivers

Atchafalaya River HEC-6T Model

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Abstract

The HEC-6T one-dimensional numerical sedimentation model was used to evaluate the long-term and system-wide sedimentation effects of modifying the operation schedule at the Old River Control Complex (ORCC). The changes evaluated were increasing and decreasing the percentage of flow that is diverted to the Atchafalaya River from the Mississippi River at the ORCC and modifying the distributions through the four ORCC structures.

Sedimentation effects for several operation scenarios were compared to a Base Condition that represented the existing operation protocols. Additionally, a dredging scenario was developed and analyzed. This scenario featured dredging material from the Mississippi River and depositing it into the Outflow Channel, downstream of the ORCC. The predictive simulations extended for 50 years. The model was used to calculate and compare sand transport past various gages along the Atchafalaya River. Also, the model compared differences in water surface elevations and bed elevations using a specific gage analysis at several gages along the Atchafalaya River. Last, the effects of the various scenarios on annual dredging requirements in Berwick Bay were also determined.

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Preface

The investigation documented in this report was conducted for the US Army Corps of Engineers, Mississippi Valley Division (MVD), as part of the Mississippi River and Tributaries Project, under Project Number 478534, and published through the Mississippi River Geomorphology and Potamology (MRG&P) Program. At the time of publication of this report, the MRG&P program director was Dr. James W. Lewis. The MVD commander was MG Diana M. Holland, and the MVD director of programs was Mr. Edward E. Belk.

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

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

1 Introduction

1.1 Background

The Atchafalaya River is intended to provide a large flood risk reduction to the Mississippi River. The Atchafalaya River receives up to one-half of the peak flow passing through Natchez, MS. A major diversion of water from the Mississippi River to the Atchafalaya River occurs at the Old River Control Complex (ORCC) — the focus of this project. The ORCC consists of three structures constructed and operated by United States Army Corps of Engineers (USACE) to control the flow of water and sediment from the Mississippi River to the Atchafalaya River and a separate commercial hydropower structure that is not part of the federal project. Although the Hydropower Structure is not an authorized component of the regulation protocol, the Hydropower Structure has a significant influence on the transfer of water and sediment from the Mississippi River and therefore was included in this modeling assessment. The three federal structures are referred to as the Old River Control Structure (ORCS). Additionally, during the Project Design Flood (PDF), the Morganza Control Structure (MCS) would pass up to 600,000 cfs[1],[2] through the Morganza Floodway into the Atchafalaya River. During the PDF event, the Atchafalaya River carries 1.5 million cfs to the Gulf of Mexico. For these reasons, the Atchafalaya River is a vital part of the Mississippi River and Tributaries (MR&T) strategy for passing the flood flows through the system. This assessment focuses on proposed operational changes at the ORCC. These include flow distribution changes through the four ORCC structures (three USACE structures and one commercial hydropower structure) that divert water from the Mississippi River into the Atchafalaya River. The assessment determines the sedimentation effects of increasing and decreasing the flow volume from the Mississippi River to the Atchafalaya

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

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

River and the effects of depositing dredged material from the Mississippi River into the Old River Outflow Channel.

Rivers are very dynamic and can exhibit major changes over periods of time. These changes are caused by a multitude of natural and anthropogenic factors, but one of the most important drivers of river channel changes is the transport, deposition, and erosion of sediment particles within the river. If a reach of river is aggrading, that means that sediment is generally being deposited in the reach, and this aggradation can increase the water-surface elevation (WSEL) for a particular flow. Conversely, a degrading reach of river generally experiences sediment erosion and would likely have a decreasing water-surface elevation for a particular flow. In some cases, a river can take decades to reach a new equilibrium. A numerical sedimentation model, calculating sediment balances and transport through the river, provides a quantitative estimate of channel changes expected in the future.

Major changes have occurred in the Atchafalaya River over the course of the last several decades due to system adjustments and changes in water and sediment inflows. Notable changes to the system include the following:

- The Capture of the Red River by the Mississippi River in the fifteenth century, making the Red River a tributary of the Mississippi River and making the Atchafalaya a distributary.
- Formation of the Old River by Shreve's Cutoff on the Mississippi River in 1831.
- Removal of Great Raft on the Red River 1833–1838. Removal of the massive log jams increased peak flows on the Red River, induced degradation, and increased the rate of capture of the Mississippi River by the Atchafalaya River.
- The construction of the Whiskey Bay Pilot Channel between 1934 and 1937.
- The construction of the ORCS in 1963.
- The J. Bennett Johnston Waterway, a system of locks and dams between Simmesport and Shreveport along the Red River, constructed between 1984 and 1994.
- The Auxiliary Structure and the Sidney A. Murry, Jr., Hydroelectric Station were added to the ORCC in 1986 and 1990, respectively. The

hydroelectric station is commercially owned and is not part of the federal project.

Over the last several decades, measured data at the Simmesport gage shows decreased sediment inflows to the Atchafalaya River. Between the 1930s and 1970s, the upper end of the Atchafalaya River experienced large declines in water-surface elevations irrespective of particular flow, as seen by a large drop in the specific gage behavior at Simmesport, LA. Since the mid-1980s, the Simmesport specific gage behavior has been more stable. In the lower end of the Atchafalaya River, an increase in water-surface elevations for a given flow has been observed in recent decades.

A HEC-6T numerical sedimentation model of the Atchafalaya River was developed in 2010 as part of the US Army Corps of Engineers Refined Project Flood Flow Line Report for the Atchafalaya Basin (USACE MVN 2010). The HEC-6T model developed for that study is referred to hereinafter as the "2010 Model." That study was conducted for the USACE New Orleans District (MVN) by the Rock Island District. The 2010 Model was based on 1997 surveys and extended from the Gulf of Mexico to the confluence of the Red and Black Rivers. The model included both the Wax Lake Outlet and the Lower Atchafalaya River.

The 2010 Model served as the starting point for a subsequent sediment study of the Atchafalaya River, which was conducted to determine water surfaces associated with a revised PDF flood. The HEC-6T model developed for that study is referred to hereinafter as the "Flowline Model" (Lewis and Copeland 2018). Changes to the 2010 Model, required to adequately calibrate and validate the Flowline Model to recent data, were described in the Flowline Model Report.

The Flowline Model served as the starting point for this assessment. The Flowline Model upstream boundary was extended up the Red River to the Black River and up the Outflow Channel to the ORCC. The HEC-6T model developed for this assessment is referred to hereinafter as the "OMAR Model." The OMAR Model was updated using the latest data. However, the Flowline Model calibration and verification were considered adequate for the reaches downstream from Simmesport.

1.2 Objective

The objective of this numerical model investigation was to determine the long-term and system-wide sedimentation effects that would occur in the Atchafalaya River if the operation protocols at the ORCC were changed. The changes evaluated included increasing and decreasing the percentage of flow that is diverted from the Mississippi River at the ORCC and modifying the distributions through the four ORCC structures (refer to Figure 1 to see the ORCC system depicted). Another objective was to develop and analyze the long-term and system-wide sedimentation effects due to a dredging scenario wherein material dredged from the Mississippi River was deposited into the Outflow Channel, downstream of the ORCC. Specifically, the assessment evaluated the response of the Atchafalaya River in terms of specific gage changes, bed elevation changes, sediment transport, and long-term dredging requirements at Berwick Bay.

Figure 1. The Old River Control Complex (ORCC) and its various systems.

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

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

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

1.3 Approach

The objectives of this assessment were accomplished using an updated version of a previously developed HEC-6T Flowline Model. The updates included extending the upstream end of the model to the ORCC and to the confluence of the Red and Black Rivers. Initial conditions were based on a 1997 survey. Existing conditions (1 October 2019) for the OMAR Model were established using a 22.4 yr historical simulation. The model continued to simulate an additional 50 yr to 30 September 2069. The 50 yr future hydrograph was taken to be the historical discharges between 1969 and 2019. A 50 yr simulation that used existing operation protocols at ORCC was run to establish a Base Condition. Daily discharge and sediment concentrations from the ORCC for both the Base Condition and the scenarios were supplied from the OMAR HEC-6T model of the Mississippi River that covered the same time period (Volume 4) (Copeland and Lewis 2022). Calculated river responses from the Base Condition were then compared to river responses from 50 yr simulations of various operation scenarios.

1.4 Scope

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

2 Numerical Model Description

2.1 HEC-6T description

The HEC-6T one-dimensional (1D) numerical sedimentation program, Version H6TV51325_J18amR_210602, was used in this assessment (Thomas 2021). The program is proprietary and can be obtained from Mobile Boundary Hydraulics (MBH). The HEC-6T program produces a 1D model that simulates the response of the riverbed profile to sediment inflow, bed-material gradation, and hydraulic parameters. Additionally, 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 Atchafalaya River. The model calculates aggradation and degradation of the streambed profile over the course of a hydrologic event. It links events together to extend this calculation to long periods of time (i.e., 50 or 100 yr). It does not simulate bank erosion or natural adjustments in channel widths. However, HEC-6T is capable of approximating channel widening that occurs as a result of bank erosion induced by degradation. The program was not designed to compute local scour and deposition around hydraulic structures. When applied by experts using sound engineering judgment, the HEC-6T program will provide accurate insight into the behavior of mobile bed rivers. Additional guidance related to model calibration, application, and interpretation can be found in the following references:

- *Sedimentation in Stream Networks* (Thomas 2021), Chapter 14
- "Computational Modeling of Sedimentation Processes" (Thomas and Chang 2008)
- *Guidelines for the Calibration and Application of Computer Program HEC-6* (USACE-HEC 1992).

2.2 Model network

The model's geometry is broken into eight segments as shown in the network schematic (Figure 2). The number of cross sections in each segment and the description of the segment boundaries are given in Table 2. The model network below Simmesport was taken from the Flowline Model. Additional details on how that network was created are contained in Lewis and Copeland (2018). Segments 7 (Red River) and Segment 8 (Outflow Channel) were added in this study based on cross sections from the 2010 Model.

Historically, the river miles on the Atchafalaya River were assigned each time a new Hydrographic Survey was made. The 1997–1998 Hydrographic Survey resulted in the 1998 river mile convention and was used in the HEC-6T model. However, the most recent set of river miles in the Wax Lake Outlet and Old Atchafalaya River Channel portions of the study were from the 1963 Hydrographic Survey and hence were used in this study. Positive river mile values were used in the Red River and ORCC portions of the model. Essentially the *zero point* of the model was the confluence of the Red River, Old River Navigation Channel, and Atchafalaya River and is shown in Figure 3 (refer to Figure 3 and Figure 4 for a depiction of the river miles used in the HEC-6T model).

Figure 2. Schematic view.

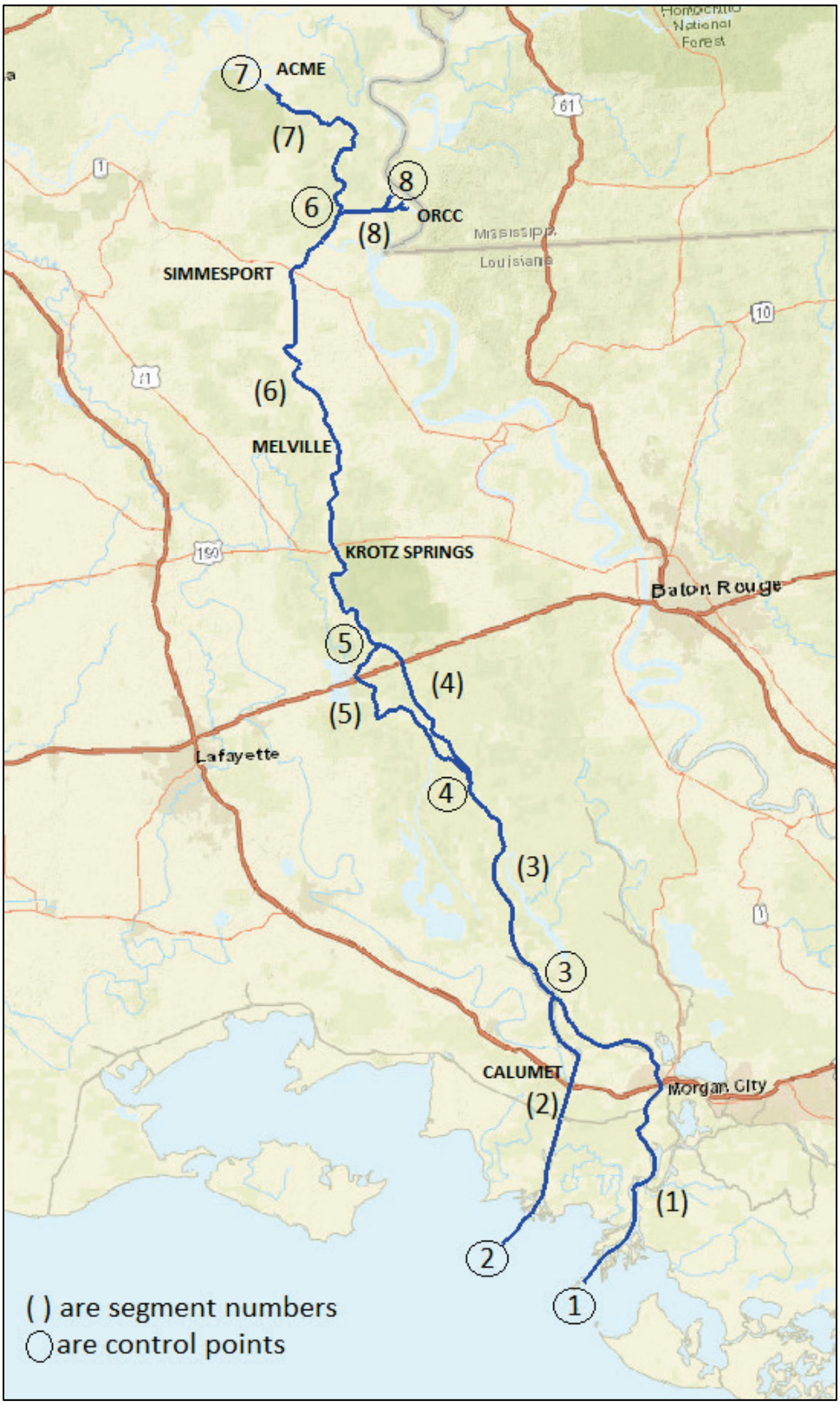


Figure 3. Atchafalaya River–river miles (1 of 2).

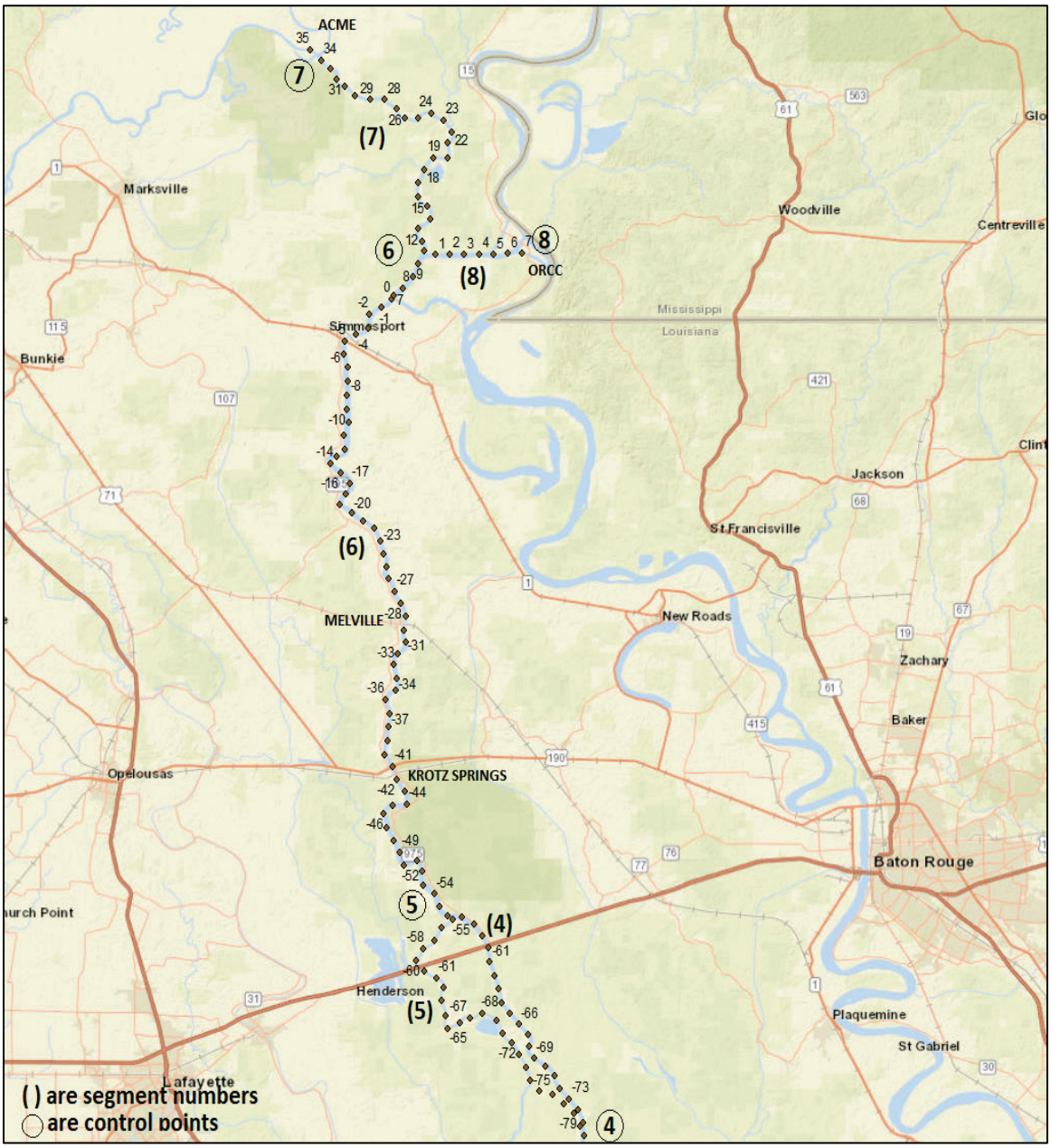


Figure 4. Atchafalaya River–river miles (2 of 2).

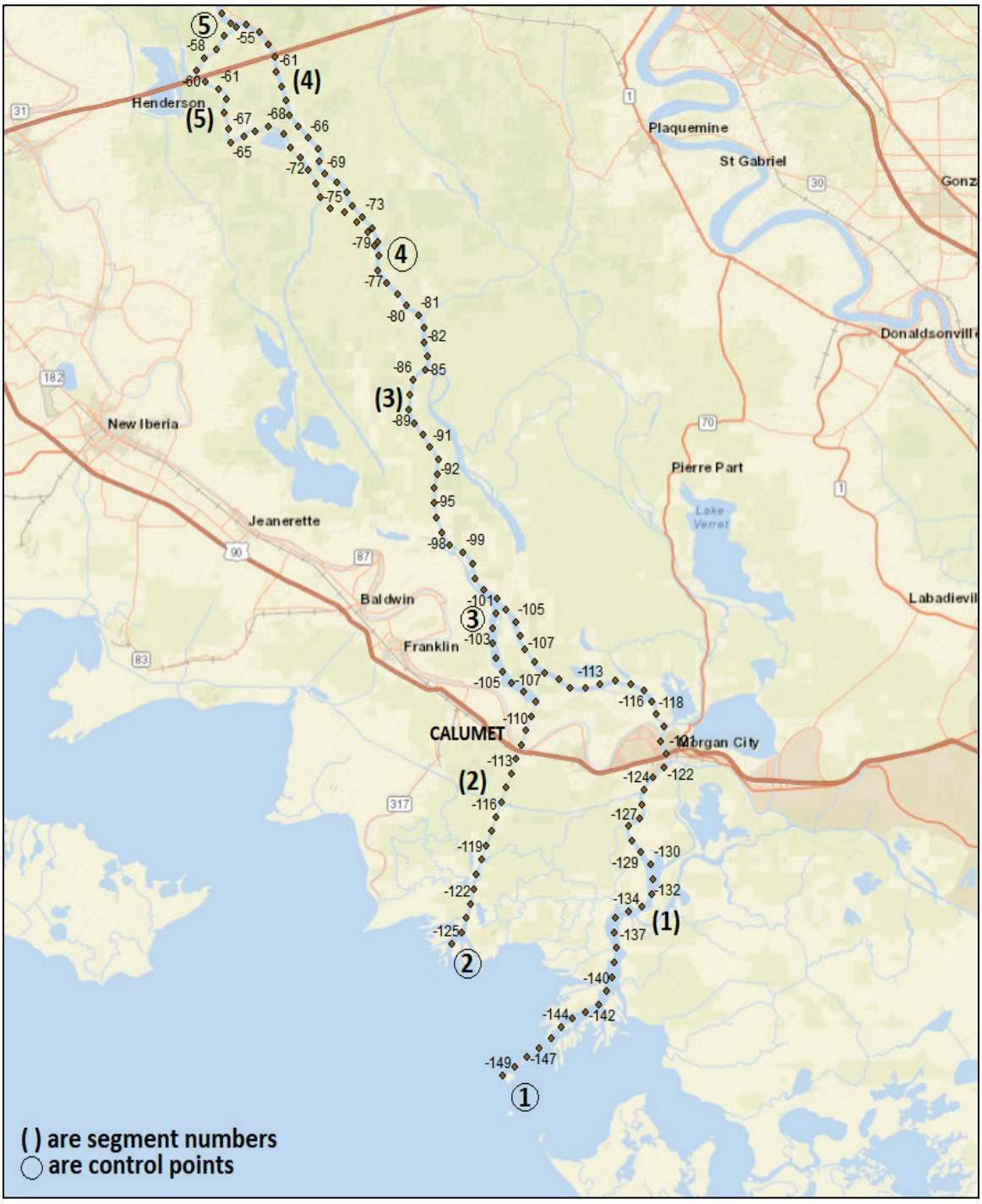


Table 2. River segment descriptions.

River Segment	Segment Number	Location of Upstream End and River Mile (RM)	Location of Downstream End and RM	Number of Cross Sections
Outflow Channel	8	The Old River Complex RM: 7.0 (Outflow Channel)	Confluence of Red River and Outflow Channel RM: 0.1 (Outflow Channel)	11
Red River	7	Confluence of the Red River and the Black River. RM: 33.0 (Red River)	Confluence of the Red River and Outflow Channel RM: 10.60 (Red River)	16
Red/Atchafalaya Rivers	6	Confluence of the Outflow Channel and the Red River RM: 10.12 (Red River)	Old Atchafalaya River Diversion RM: -56.5 (Atchafalaya River)	91
Old Atchafalaya River	5	Diversion from Atchafalaya River RM: -56.5 (Atchafalaya River)	Confluence of the Atchafalaya River and the Old Atchafalaya River RM: -70.3 (Atchafalaya River)	31
Atchafalaya River (Whiskey Bay Pilot Channel)	4	Old Atchafalaya River Diversion RM: -56.5 (Atchafalaya River)	Confluence of the Atchafalaya River and the Old Atchafalaya River RM: -70.31 (Atchafalaya River)	25
Atchafalaya River	3	Confluence of the Atchafalaya River and the Old Atchafalaya River RM: -70.30 (Atchafalaya River)	Wax Lake Outlet RM: -102.5 (Atchafalaya River)	40
Wax Lake Outlet	2	Wax Lake Outlet RM: -102 (Wax Lake Outlet)	Gulf of Mexico RM: -125 (Wax Lake Outlet)	33
Lower Atchafalaya River	1	Wax Lake Outlet RM: -104 (Atchafalaya River)	Gulf of Mexico RM: -149.1 (Lower Atchafalaya River)	57

2.3 Flowline Model

The HEC-6T numerical model for the Old, Mississippi, Atchafalaya, and Red Rivers (OMAR) assessment was developed primarily from the Flowline Model, which was completed in 2018. The objective of the Flowline Model was to determine the effect of long-term sedimentation processes in the Atchafalaya River on the PDF event for the system encompassed by the MR&T. The Flowline Model was calibrated and validated to the time period 1997 to 2016. Then, the model simulated the years 1997 through 2066 using a hypothetical hydrograph.

The following major conclusions were made from the results of the Flowline Model study:

- The Wax Lake Outlet is capturing an increasing amount of flow from the Lower Atchafalaya River.
- The PDF event scours the bed elevations by a large amount during the rising limb of the hydrograph, especially near Morgan City. This scour decreases the PDF water surfaces that would occur without scour by as much as 5 ft.
- Although the Upper Atchafalaya River has exhibited large amounts of scour in recent decades, as shown by MVN surveys, the Flowline Model projected a dynamic equilibrium to a slightly increasing trend in water-surfaces elevations looking forward. Future water-surface elevations in the upper reach could be lower than model projections due to channel widening, a process which should be included in future sediment modeling efforts of the Atchafalaya River.
- Over the next 50 yr, the Flowline Model projected trends of aggradation, or higher water surfaces into the future, for the Lower Atchafalaya River. Water surfaces during a PDF event occurring in 2067 would be up to 3 ft higher than a PDF event occurring in 2011.

2.4 Channel geometry

The cross-section geometry in the 2010 Model and the Flowline Model was developed from 1997 hydrographic survey data. The survey datum was the North American Vertical Datum of 1988 (NAVD88). The OMAR Model cross sections came from the 2010 Model and the Flowline Model, and thus, elevations refer to NAVD88.

The upstream extent of the Flowline Model was the confluence of the Red River and the ORCC Outflow Channel, which is located approximately 8.5 mi upstream of Simmesport, LA. There were two downstream ends of the model: one for the Wax Lake Outlet and one for the Lower Atchafalaya River. Many of the Flowline Model features were taken from the 2010 Model. In the OMAR Model, cross-section geometry for the Outflow Channel and the Red River above the Outflow Channel were taken from the 2010 Model. Red River overbank elevations were adjusted in the OMAR Model to ensure consistent overbank-channel discharge ratios.

2.5 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.

HEC-6T uses a quasi-unsteady flow approach, which approximates the discharge hydrograph as 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.

It was determined during the Flowline assessment that the 1997 hydrograph contained a large flow event, so the start time of the model simulation was important. The HEC-6T model was run with start times before and after the large flow event in the spring of 1997 to see the impacts, and there were significant differences in the bed elevations for the later part of the year depending on whether the 1997 event was, or was not, included. The 1997 bathymetric survey, which was the basis for the model geometry, showed that the survey data were collected as late as April of 1997. For this reason, the simulation start time was adjusted to 30 April 1997 for all runs. The April 1997 start time was retained in the OMAR Model.

The 50 yr hydrograph used to calculate future sedimentation effects for the Base condition and each scenario in the OMAR assessment was developed from 1969–2019 hydrologic data. This 50 yr period was deemed adequate to represent future conditions.

2.5.1 Outflow channel

William A. Thomas, of MBH, developed the HEC-6T enhancement that allowed transfer of calculated daily discharges and sediment concentrations from one HEC-6T model to another. This work was accomplished under contract to USACE as a part of this OMAR assessment. The enhanced code writes an output file (called a "TAPE90" file) from the Mississippi River simulation that is used as an input file to the Atchafalaya River model. (Thomas 2021)

Daily discharges at the upstream end of the Outflow Channel were calculated for each of the four ORCC structures by the companion HEC-6T model of the Mississippi River (Copeland and Lewis 2022). The calculations were based on long-term average diversion percentages developed from the historical record or from the assigned diversion percentages for each specific scenario. The historical simulation extended from April 1997 to September 2019. The 1 October 1969 to 30 September 2019 historical hydrograph was assumed to represent the next 50 yr for the future simulations.

2.5.2 Red River

The Red River upstream model boundary is influenced significantly by backwater conditions from the Outflow Channel. In addition, long-term daily discharge records are unavailable for the reach of the Red River between the Outflow Channel and the Black River. However, long-term daily discharge data were available at Simmesport and the Outflow Channel. Since the HEC-6T model is focused on the long-term average sedimentation effects, it was decided to use the difference between the reported daily discharges at Simmesport and the Outflow Channel as inflow from the Red River. This methodology provides for appropriate long-term sediment supply from the Red River but ignores the effects of backwater storage of both water and sediment on the Red River morphology. Thus, the Red River segment cannot be used to evaluate daily or even annual sedimentation

effects in the Red River. The Red River segment should be considered only as a supply reach for the Atchafalaya River.

2.5.3 Morganza Control Structure (MCS)

Inflow from the MCS during the 2011 flood was determined for the Flowline Model, and these boundary conditions were retained in the OMAR Model. Based on a review of lidar surface elevations and AdH 2D hydraulic model computations (Bell et al. 2018), three primary locations were identified where the Morganza flow entered the Atchafalaya River main channel. Accordingly, it was decided to distribute the MCS flow to the Lower Atchafalaya River into three different cross sections between the junction with Wax Lake Outlet Channel and Morgan City, namely into cross sections -104.8, -112.5, and -120.1. The timing of flow through MCS is different than the timing of flow entering into the Lower Atchafalaya River. The total volume of flow through MCS, based on Table 10 of Maynord (2014), was used to simulate the proper volume of water discharge during the 2011 event. To determine the proper attenuation of the MCS flow arriving to the Atchafalaya River channel, the 2011 observed data for Morgan City and Calumet were used. First, the HEC-6T model was simulated without any MCS inflow for the 2011 event. These results showed a clear deficit of water volume at both Morgan City and Calumet beginning on 21 May 2011, which is lagged from the reported flow through MCS beginning on 14 May 2011. Then, the difference in volume between simulated (Morgan City plus Calumet) and observed (Morgan City plus Calumet) was calculated. This calculated difference in discharge was used as the HEC-6T inflow until its accumulated volume matched the total volume reported in Table 10 of Maynord (2014). The HEC-6T model was rerun with the new MCS flows to check the simulation with observed flows at Morgan City and Calumet.

2.6 Distributary flow

The amount of flow remaining within the channel, as opposed to flow moving across the overbank areas or into a distributary, has a large influence on the sediment dynamics, and any flows leaving or entering the channel must be defined clearly. This is difficult with the Atchafalaya River Basin because there is a large amount of flow that is being exchanged between the river channels and the floodplain areas. Refer to Figure 5, Figure 6, and Table 3, which identify distributary and return flow

locations. Distributary and return flow quantities in the HEC-6T Flowline Model were determined using calculated results from a companion HEC-RAS model of the Atchafalaya Basin that simulated the channel reaches with 1D simulation and the floodplain areas with two-dimensional (2D) simulation. In the HEC-RAS model, adjacent boundaries between the 1D and 2D were calculated using a lateral weir representation. Distributary designations from the Flowline Model were retained in the OMAR Model.

Figure 5. Inflow and outflow locations in segments 3, 4, and 5 of the HEC-6T model.

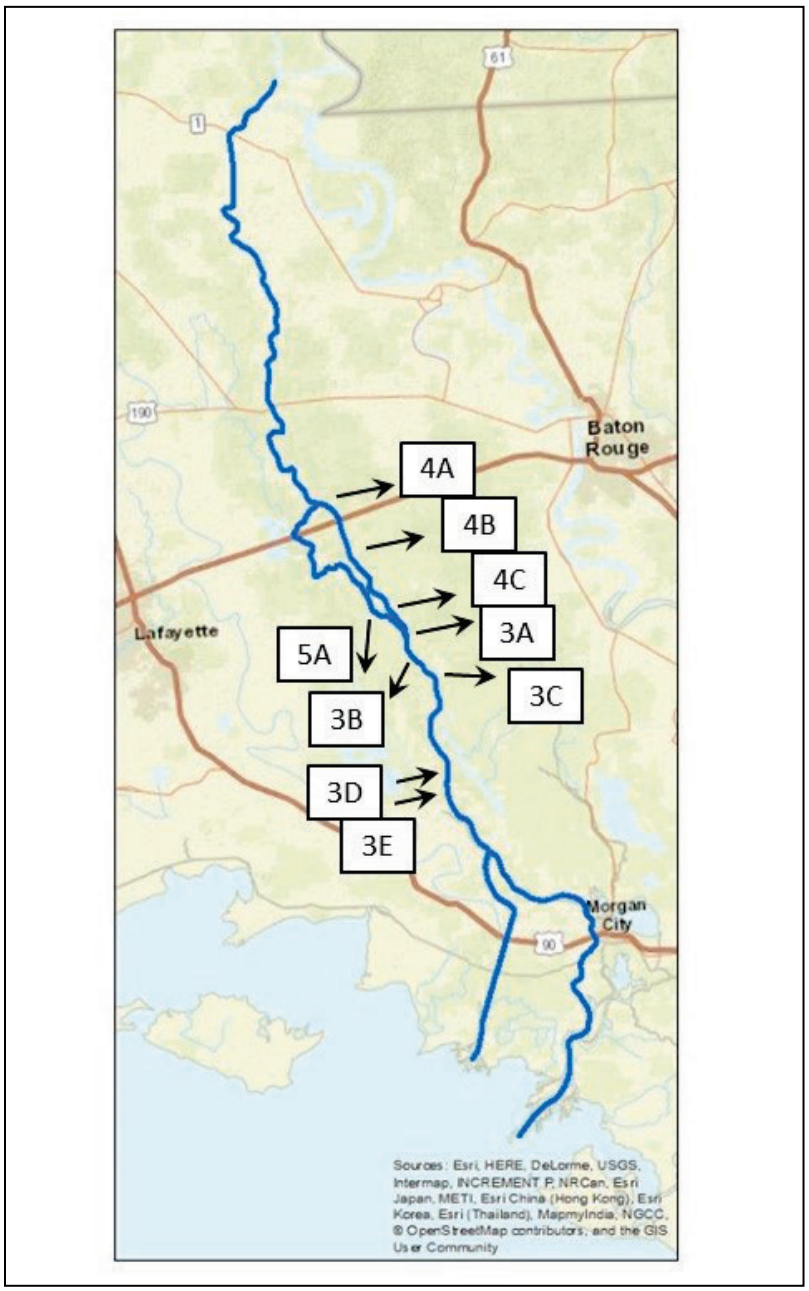


Figure 6. Inflow and outflow locations in segments 1 and 2 of the HEC-6T model.

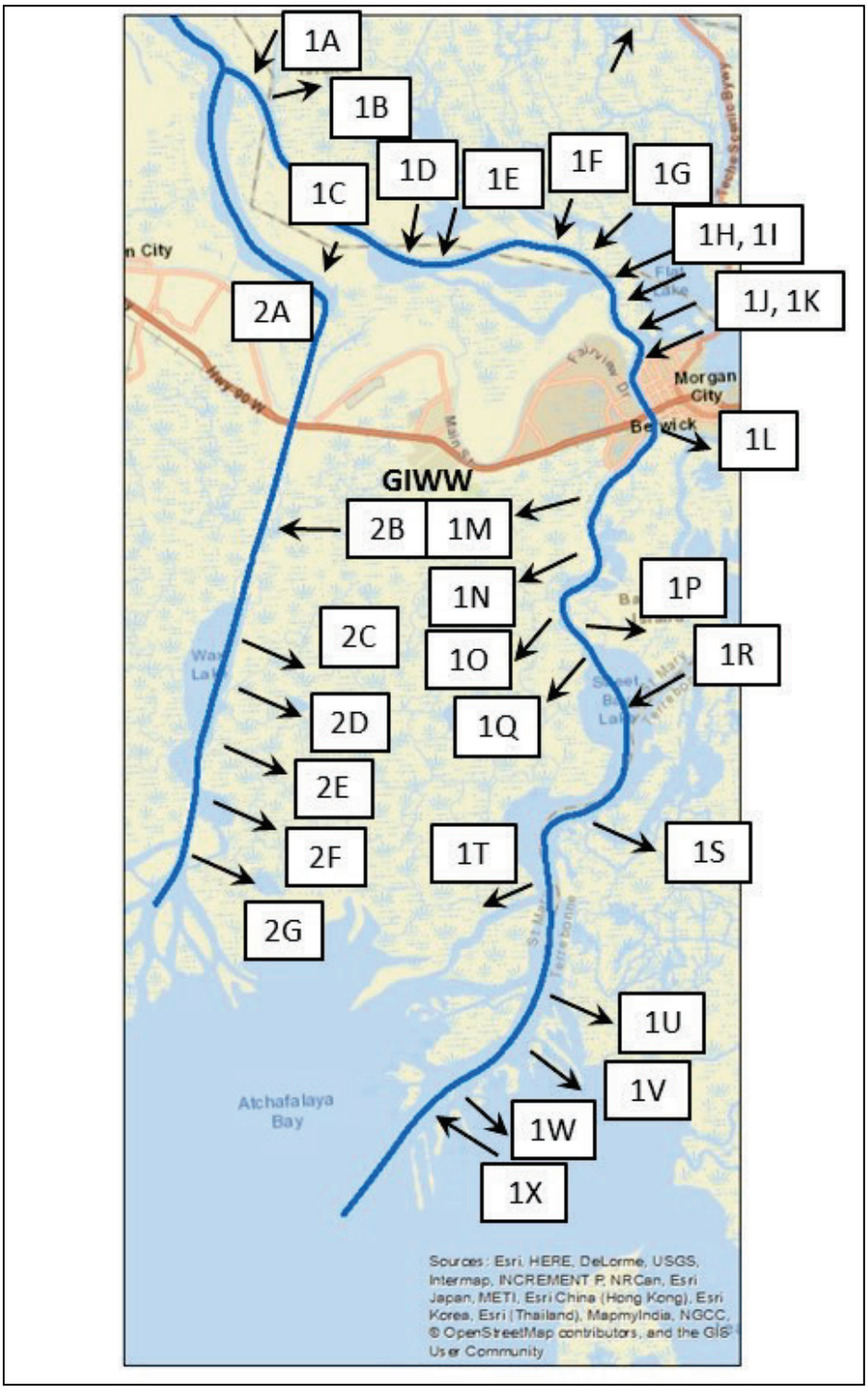


Table 3. Location and details of tributaries and distributaries.

Label from Figures 5 and 6	Type	Entry Point/ Cross Section	Segment #	Name
5A	Outflow	-74.3	5	WEST FRESHWATER DIVERSION
4A	Outflow	-57.3	4	FLOODPLAIN FLOW
4B	Outflow	-62.6	4	FLOODPLAIN FLOW
4C	Outflow	-69.4	4	FLOODPLAIN FLOW
3A	Outflow	-70.9	3	EAST FRESHWATER DIVERSION
3B	Outflow	-78.95	3	WEST ACCESS CHANNEL
3C	Outflow	-81.2	3	EAST ACCESS CHANNEL
3D	Return Flow	-94.4	3	WEST ACCESS CHANNEL RETURN FLOW
3E	Return Flow	-95.2	3	WEST FRESHWATER DIVERSION RETURN FLOW
2A	Return Flow	-102.5	2	1/3 OF MORGANZA INFLOW
2B	Return Flow	-108	2	DIVERTED FROM ATCHAFALAYA
2C	Outflow	-115.2	2	GIWW FROM ATCHAFALAYA
2D	Outflow	-119.8	2	GULF OUTLETS
2E	Outflow	-120.8	2	GULF OUTLETS
2F	Outflow	-122.7	2	GULF OUTLETS
2G	Outflow	-123.3	2	NEW PASS (CREWBOAT)
1A	Inflow	-125.	1	GULF OUTLETS
1B	Outflow	-105.6	1	WILLOW COVE OUTFLOW
1C	Outflow	-109.4	1	LAR TO WAX LAKE
1D	Return Flow	-111.2	1	RETURN OVERBANK RM
1E	Inflow	-112.5	1	1/3 OF MORGANZA INFLOW
1F	Return Flow	-116.7	1	RETURN FROM WILLOW COVE
1G	Return Flow	-117.8	1	RETURN OVERBANK RM
1H	Return Flow	-118.5	1	RETURN OVERBANK RM
1I	Return Flow	-118.8	1	RETURN EAST ACCESS
1J	Return Flow	-119.2	1	RETURN EAST FRSHWTR DVRSN
1K	Inflow	-120.1	1	1/3 OF MORGANZA INFLOW
1L	Outflow	-121.6	1	GIWW EAST - BAYOU SHAFFER
1M	Outflow	-124.7	1	GIWW WEST
1N	Outflow	-127.5	1	OVERBANK FLOW
1O	Outflow	-128.8	1	OVERBANK FLOW
1P	Outflow	-129.6	1	OVERBANK FLOW

Table 3. (cont.) Location and details of tributaries and distributaries.

Label from Figures 5 and 6	Type	Entry Point/ Cross Section	Segment #	Name
1Q	Outflow	-130.5	1	OVERBANK FLOW
1R	Return Flow	-131.3	1	RETURN FROM BAYOU SHAFFER
1S	Outflow	-134.5	1	AVOCA CUTOFF
1T	Outflow	-139.5	1	SHELL ISLAND PASS
1U	Outflow	-142.6	1	BIG, DEER AND EAST PASSES
1V	Outflow	-144.4	1	AMEREDA PASS
1W	Outflow	-146.2	1	RATCLIFF AND LOG ISLAND PASSES
1×	Return Flow	-147.2	1	RETURN FROM RATCLIFF PASS

2.7 Water temperature

Water temperature can have an important influence on both sediment erosion and deposition dynamics through changes in water viscosity and density, which impact shear stresses, particle settling velocities, etc. Seasonal water temperature variations were used in the Flowline Model as well as in the OMAR Model. The monthly average water temperatures from the Flowline Model were updated in this study to include additional data collected since the completion of the Flowline Model.

Water temperatures in the OMAR Model were based on 1969–2019 USACE and US Geological Survey (USGS) reported data. Temperatures for 1,880 days between 1969 and 2019 were available from the gages at Simmesport, Melville, and the Outlet Channel. Gaps between reported temperature measurements varied between zero days to approximately 3 months. There were no data available between 1969 and 1972. Less data has been reported since 2000, where the only available data were from the USGS gage at Melville. Initially, an attempt was made to use linear interpolation between reported data to obtain a historical temperature hydrograph. However, when reported temperatures at the three gages sometimes varied by as much as 10°F within 1 or 2 days, this effort was abandoned. Instead, 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 the fifteenth of each month are shown in Table 4. In the HEC-6T hydrograph, daily values were assigned based on linear interpolation

between the monthly averages. The following gages were used to determine the average monthly temperatures:

- USACE Simmesport 1974–1999
- USACE Old River Outflow 1974–1999
- Atchafalaya River at Simmesport, LA, 1972–1977 (USGS 07381490)
- Atchafalaya River at Melville, LA, 1977–2019 (USGS 07381495).

Table 4. Average monthly water temperature 1969–2019 (°F).

January	44.8	July	84.2
February	47.4	August	84.4
March	53.3	September	80.8
April	61.4	October	69.2
May	70.1	November	59.4
June	78.9	December	50.2

2.8 Downstream water-surface elevations

The downstream water-surface elevations used in the OMAR Model were updated from those used in the Flowline Model to take advantage of more recent data. The downstream water-surface elevations for the historical 30 April 1997 to 30 September 2019 hydrograph were taken from reported average monthly data. The 50 yr future downstream water-surface elevations were calculated using average monthly water-surface elevations adjusted for sea level rise.

The NOAA tide gage at the Lower Atchafalaya Wildlife Management Area (LAWMA), Amerada Pass, LA (8764227), was used to set the downstream water-surface elevations for the OMAR Model between 2006 and 2019. Mean monthly tide elevations were obtained from the NOAA web site (http://tidesandcurrents.noaa.gov). The Flowline Model had used the NOAA tide gage at Eugene Island, North of, LA (8764314), and a discontinued NOAA gage at Eugene Island, LA (8764311), to determine the downstream water-surface elevations. Reported data for both the LAWMA and Eugene Island, North of, gages were based on the 1983–2001 epoch. The LAWMA gage has a longer period of record (2006–2019) and was chosen to obtain a better representation of long-term average stages. The Eugene Island, North of, gage record extends between 2015 and 2019, and the Eugene Island gage record extends between 1934 and 1971. The

LAWMA and Eugene Island, North of, average monthly stage at mean sea level (MSL) were compared for the years where data were reported for both gages (2015–2019) and found to be consistent (Figure 7). Downstream water-surface elevations from the Flowline Model were used between 1997 and 2006.

In the numerical model, mean daily values were determined by interpolation between the mean monthly values. At the LAWMA gage, NOAA reports that the datum adjustment from MSL to NAVD88 is -0.11 ft (NAVD88 = MSL – 0.11).

Consistent with the Flowline Model, the downstream water-surface elevation for the Wax Lake Outlet was set 0.3 ft higher than the downstream water-surface elevation for the Atchafalaya River. This accounts for the Wax Lake Outlet downstream boundary being farther from the Gulf of Mexico than the Atchafalaya River downstream boundary. This difference occurred because the downstream end of the Wax Lake Outlet was moved upstream from the Gulf of Mexico to obtain a reasonable representation of 1D flow in the HEC-6T model for sediment transport computations. The offset was determined by evaluating the water-surface differences in the HEC-6T model before eliminating the initial downstream cross sections that had extended to the Gulf of Mexico. The downstream water-surface boundary conditions were 0.3 ft higher for the Wax Lake Outlet than for the Atchafalaya River for each time-step.

The 50 yr future hydrograph (2019–2069) was constructed to account for both seasonal variations in the mean tide elevation and sea level rise. The data indicated a long-term increase in the mean tide elevation. To differentiate the seasonal variation from sea-level rise and/or subsidence effects, adjusted mean monthly water-surface elevations were calculated. This was accomplished by increasing the magnitude of the older measurements so that a linear regression line through the adjusted data had a zero slope. The resultant adjusted mean monthly water surface elevations represent September 2019 conditions. The reported and adjusted tide elevations are shown in Figure 8. Base-level (September 2019) seasonal downstream water-surface elevations in the HEC-6T model are shown in Table 5. Linear interpolation of these monthly values was used to determine base-level daily values. These values were then increased to account for sea level rise according to Level II as prescribed in USACE (2013).

Figure 7. Average monthly stage in Atchafalaya Bay, 2006–2019.

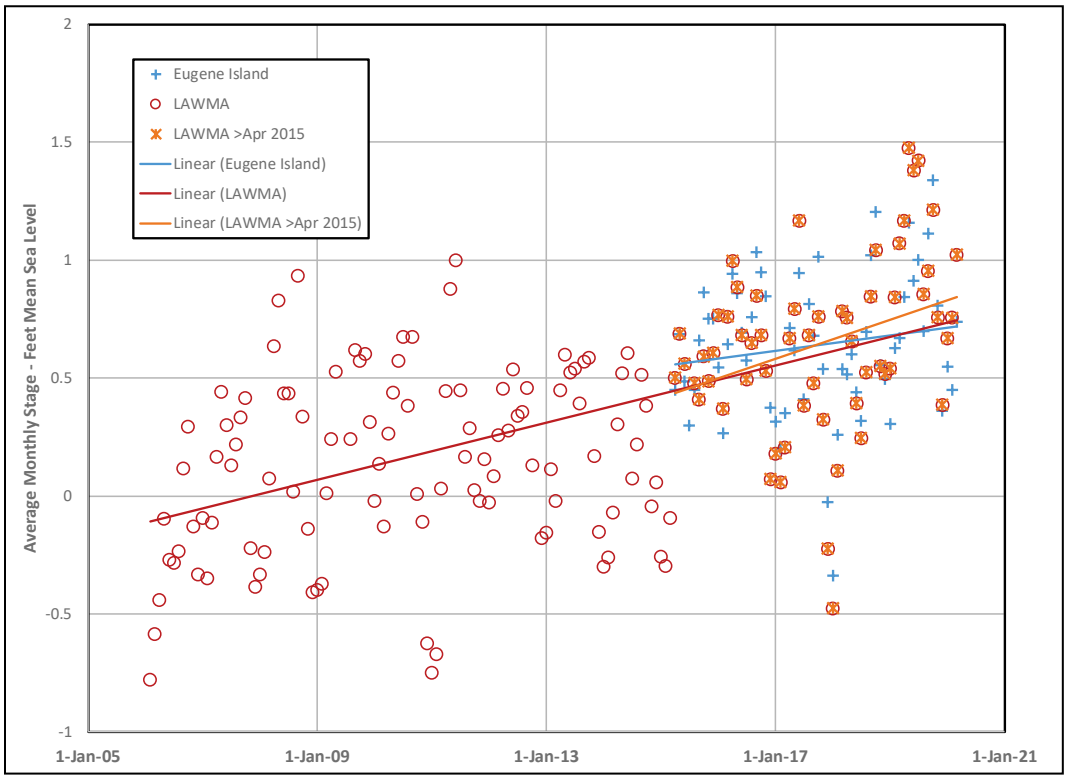


Figure 8. Mean monthly tide elevations at LAWMA, Amerada Pass, 2006–2019.

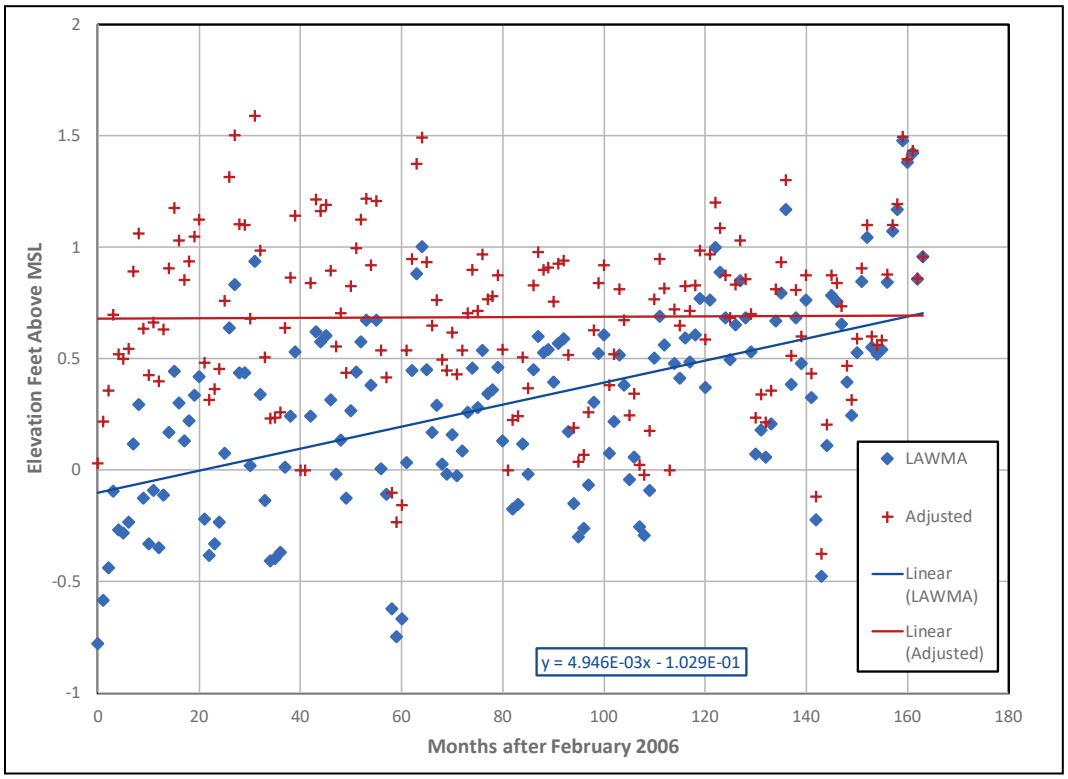


Table 5. Average base-level (2019) monthly downstream water-surface elevations for Atchafalaya River and Wax Lake Outlet.*

	Atchafalaya (ft) NAVD88	Wax Lake (ft) NAVD88		Atchafalaya (ft) NAVD88	Wax Lake (ft) NAVD88
January	0.18	0.48	July	0.69	0.99
February	0.22	0.52	August	0.63	0.93
March	0.46	0.76	September	0.85	1.15
April	0.77	1.07	October	0.75	1.05
May	0.93	1.23	November	0.46	0.76
June	0.88	1.18	December	0.25	0.55

*Based on NOAA data from LAWMA, Amerada Pass. LA 2006–2019.

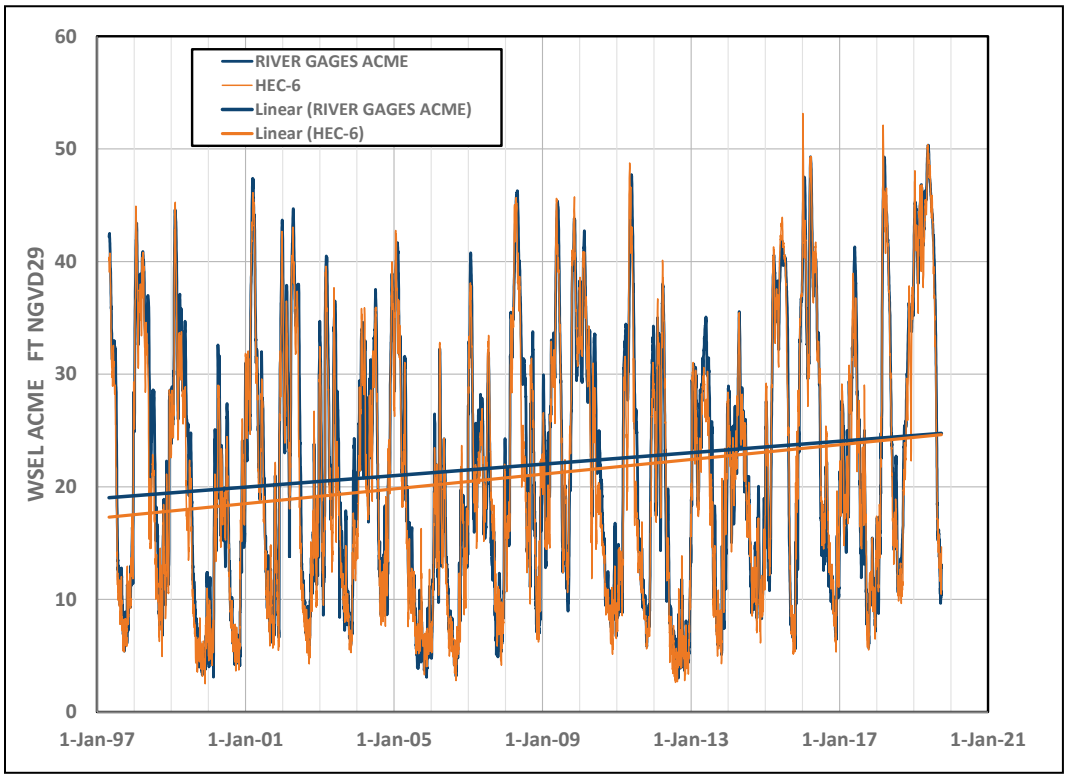
2.9 Roughness coefficients

The roughness coefficients from the Flowline Model downstream from the confluence of the Red River and the Outflow Channel were retained in the OMAR Model. Roughness coefficients from the 2010 Model were used for the Outflow Channel and the Red River above the Outflow Channel.

The Flowline Model hydraulic calibration was achieved by adjusting roughness, (Manning's n) values such that the simulation of 1997 to 2010 represented the observed data at several gages, as described in Lewis and Copeland (2018). These included gage data from Simmesport, Melville, Butte la Rose, Myette Point, Morgan City, and Avoca Island Cutoff on the Atchafalaya River and Calumet on the Wax Lake Outlet. At the downstream end, the roughness values in the Lower Atchafalaya River and Wax Lake Outlet Channel influence not only the water surfaces in those channels but also the flow distribution between the two channels. Manning's n values, varying as a function of discharge, in the downstream segments of the Flowline Model were adjusted during calibration of that model.

Roughness coefficients for the Red River retained from the 2010 Model were verified using gage data at Acme (RM 33). Measured and calculated water-surface elevations at Acme are shown in Figure 9. Both the measured and calculated water-surface elevations show a long-term, increasing stage trend.

Figure 9. Measured and calculated water surface elevations at Acme.



2.10 Sand transport function

The Flowline Model used the Laursen-Copeland function (USACE-HEC 2020) for sand transport. This function was first developed in 1989, for the Corte Madera Creek Sedimentation Study (Copeland and Thomas 1989), and has been used successfully in gravel-bed and sand-bed streams and rivers. Other transport functions were evaluated during the calibration phase of the Flowline Model study. All the tested transport functions gave very similar results in terms of sand concentrations at gage locations and bed elevation changes. The evaluation of the various transport functions is discussed in the Flowline Model report. The Laursen-Copeland function was retained for use in the OMAR Model.

2.11 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 equations were taken from previous numerical model work by Thomas et al. (1988). These coefficients were adjusted for an unpublished 2005 study of the Lower Atchafalaya River and Bayous Chene-Boeuf-Black conducted for

USACE MVN in 2005 (USACE MVN 2005) by USACE, Vicksburg District (MVK), and are shown in Table 6. These same coefficients were adopted for use in the Flowline Model and for use in the OMAR Model. It is acknowledged that there is considerable uncertainty associated with these coefficients. However, lacking data for calibration, it was deemed appropriate to retain the values used in previous studies. Note that different values were used in the Mississippi River OMAR HEC-6T model where dredging data were used for calibration.

It is the cohesive properties of the clay that determine the erosion thresholds. For this reason, the same erosion coefficients are used for silt and clay in HEC-6T. The Ariathurai and Krone equation for particle erosion is

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

where

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

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

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

where

M_2 = erosion rate for mass erosion (ERME + ER2 {τ_b – STME})
$\Delta t / T_c$
T_c = characteristic time of erosion (1 hr)
T = 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 *SLDTSL* for silt. The Krone equation is

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

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

where

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

Table 6. Silt and clay erosion and deposition coefficients.

Coefficient	Variable Name in HEC-6T	Value	Units
Shear threshold for clay deposition	DTCL	0.0002	lb/ft^2
Shear threshold for silt deposition	SLDTSL	0.0005	lb/ft^2
Shear threshold for erosion of silt and clay particles	STCD	0.0024	lb/ft^2
Shear threshold for mass erosion	STME	0.015	lb/ft^2
Erosion rate at STME	ERME	0.001	lb/ft^2/hr
Slope of the erosion rate curve for mass erosion	ER2	0.3	1/hr

2.12 Sediment inflow

The OMAR Model sediment inflow boundaries included the Outflow Channel just below the ORCC, the Red River at the confluence with the Black River, and the Morganza Spillway. The daily sediment inflows for

each size class through each of the ORCC structures was calculated from a companion HEC-6T model of the Mississippi River and imported into the Atchafalaya River HEC-6T model as a boundary condition. Red River sediment inflow rating curves were developed for each size class by taking the difference between measured daily sediment discharge at Simmesport and at the Old River Outflow Channel gage. Sediment concentrations from the Morganza Spillway were taken from the Flowline Model and only included fines. No sand is included in the Morganza inflows to the Atchafalaya River.

2.12.1 Outflow channel

William A. Thomas, of MBH, developed the HEC-6T enhancement that allowed transfer of calculated daily discharges and sediment concentrations from one HEC-6T model to another. This work was accomplished under contract to USACE as a part of this OMAR assessment. The enhanced code writes an output file (called a "TAPE90" file) with calculated daily discharges and sediment concentrations from the main river HEC-6T model that is used as an input file to the distributary HEC-6T model (Thomas 2021).

Daily discharges and sediment concentrations for each of the four ORCC structures were calculated by the OMAR HEC-6T model of the Mississippi River (Copeland and Lewis 2022). The discharge calculations were based on long-term average diversion percentages developed from the historical record or from the assigned diversion percentages for each specific scenario. The sediment diversion coefficients were based on long-term average rating curves developed from historical sediment measurements and are shown in Table 7. The sediment diversion ratio is defined as the ratio of diverted sediment concentration to the upstream sediment concentration. (A detailed discussion of how these coefficients were determined is contained in Chapter 2 of Volume 4 [Copeland and Lewis 2022].)

Note that sediment diversion coefficients calculated using the vertically averaged 2D AdH model, described in Volume 6 (Savant et al. 2022) are different than those used in the HEC-6T model. Sensitivity analyses should be a part of future studies that require a more quantitative result.

Table 7. HEC-6T sediment diversion ratios at ORCC from the Mississippi River 1991–2019.

	Hydropower RM 315.4	Overbank Structure – RM 315					Low Sill RM 314.5	Auxiliary RM 311.6
		cfs ≥ 250K	cfs = 200K	cfs = 50K	cfs = 4k	cfs ≤ 2.5K		
Very-fine sand	0.68	0.88	0.84	0.78	0.74	0.00	0.49	0.73
Fine sand	0.43	0.50	0.43	0.34	0.28	0.00	0.42	0.98
Medium sand	0.24	0.14	0.09	0.05	0.03	0.00	0.57	1.90
Coarse sand	0.29	0.01	0.00	0.00	0.00	0.00	0.18	0.46

2.12.2 Red River

2.12.2.1 Red River sediment gages

The upstream boundary of the model at Madam Lee on the Red River is located just downstream from the Black River confluence at RM 33. This boundary is not ideal because it is not in equilibrium and does not exhibit uniform sediment transport characteristics. It is frequently influenced by backwater effects from Mississippi River diversions, which induce aggradation upstream from the Outflow Channel. When the backwater conditions recede and higher Red River flows occur, the excessive deposition will scour. Sediment transported by the Red River is further complicated by operation of navigation dams. The dams maintain pools at low flow and operate *open river* when flow exceeds approximately 95,000 cfs (LD2). This means that sediment from the Red River watershed is essentially shut off at low flow and then re-suspended at high flow.

Average annual water yield from the Red River is 32% of the Atchafalaya River average annual water yield at Simmesport. Peak discharges on the Red River do not coincide with Mississippi River peak discharges. Flow through the ORCC is typically reduced during peak Red River flows to maintain a constant 30/70 latitude flow distribution at Simmesport and Tarbert Landing.

USACE MVN collected suspended sediment and bed material samples at *Madam Lee Revetment* between 1992 and 1996 and at *Red River above Outflow Channel* between January 1973 and January 2005. The Madam

Lee gaging station was located just below the Black River confluence at Red River RM 33. The Above Outflow Channel gaging station was located one-half mile upstream from the Outflow Channel at Red River RM 11. A depth-integrated sampler was used at Madam Lee, and after 1974 a point sampler was used at RM 11. During the years when samples were collected at both sites, higher total sediment concentrations were reported for the samples collected downstream with the point sampler. The RM 11 data collected between 1989 and 2005 were corrected for dissolved solids in the USGS sediment data review study (Norton et al. 2019).

Of note is the construction of the J. Bennett Johnston Red River Waterway Project, which occurred between 1984 and 1994. This project included five lock and dams as well as numerous channel straightening reaches where trench-fill revetment was constructed to improve the alignment of the navigation channel. Sediment concentrations were enriched as banks eroded back to the new riprapped bank line. Extensive bank and bed erosion occurred downstream from the Lindy C. Boggs Lock and Dam (Lock and Dam No. 1) after construction. For these reasons, the construction years were not considered representative of future conditions, and samples collected between 1984 and 1994 were not used in this study.

Suspended sediment samples collected at *Red River above Old River Outflow Channel* after 1994 were collected at 0.5 and 0.7 depth at each of two verticals. The vertical stations were at a fixed distance from a monument on the bank. Depending on the flow velocity, either a US P-61 or US P-63 was used to collect samples. Sediment data reported on MVN 3pg files include size class distributions of the suspended sediment, bed material gradations, sampler type, number and location of verticals, sample depths, water temperature, and gage height. A simple average of the four bottle concentrations was used to determine average concentrations.

USACE MVK collected suspended sediment and bed material samples at *Madam Lee Revetment* and at *Black River at Acme* between 2004 and 2016. Suspended sediment samples were collected at both gages using a US D-43 depth integrated sampler. The Madam Lee gaging station was located just below the Black River confluence at Red River RM 33. The Acme gaging station was located just above the Red River and Black River confluence at Black River RM 0.1.

The USACE MVK data from Madam Lee and Acme were not used to determine suspended sediment inflow for the OMAR assessment due to issues relating to the reliability.

Two factors led to exclusion of the Red River sediment data from consideration in this study. The first was the lack of reliable recent data at the Red River sediment gages. The second was that this reach of the Red River between the Black River and the Outflow Channel is subject to significant and frequent backwater effects so that a reliable daily-discharge rating curve does not exist at these Red River gaging stations.

2.12.2.2 Red River sediment inflow for HEC-6T

Sediment inflow for the Red River was taken to be the difference between calculated daily suspended sediment transport at Simmesport (SIM) and the Outflow Channel (ORO) based on measured suspended data collected between November 1998 and September 2016; and April 1997 to September 2019 reported discharge data. The approach uses polynomial regression curves to obtain a long-term average sediment rating curve for the Red River. It does not account for backwater effects or daily sediment transport potential and cannot be used to evaluate daily geomorphic river response on the Red River. To evaluate Red River geomorphic responses, the upstream boundary would need to be moved upstream beyond the reach of the backwater effects to a point where there are sediment data. The OMAR Model Red River segment is intended to be a supply reach and relies on the 20 mi between the Outflow Channel and the Black River to store/supply sediment to maintain long-term sediment continuity.

Sediment rating curves for Simmesport were based on USGS/USACE data collected between November 1998 and January 2016. The data included dissolved solid corrections from Norton et al. (2019). A third-order polynomial regression equation was developed for both sand and total sediment concentrations. The start date was chosen based on the commencement of a sampling protocol with equally spaced collection depths for the point samples. The USGS averaged all bottles to determine average concentration in their corrected analysis. Therefore, since the point sample concentrations are not discharge weighted, the reported average concentrations are not representative of the vertical until November 1998. The end date was selected based on available data at the time of the study and the absence of collection equipment and sampling

protocol data. Size class fractions were also available from the Norton study. Simmesport regression curves for total sediment and sand are shown in Figure 10. Measured size class fractions are shown in Figure 11.

Power regression curves frequently used to represent sediment data were not used here because the relationship between sediment concentration and discharge is better represented by a polynomial, especially at high discharges. However, Figure 12 demonstrates the power regression curve for sand concentrations at Simmesport.

ORO sediment data were also taken from the Norton report. Data between November 1998 and January 2016 were used to develop a third-order polynomial regression equation for sand and total sediment concentrations at the ORO gage at RM 4.33. The start and end dates were chosen to be consistent with the Simmesport data. The ORO regression curves for total sediment and sand are shown in Figure 13. At ORO, between 1998 and January 2010, point samples were collected at depth fractions of 0.1, 0.3, 0.5, and 0.95. In February 2010, equally spaced collection depths of 0.2, 0.4, 0.6, and 0.8 were adopted. The USGS averaged all bottles to determine average concentration in their corrected analysis. Therefore, since the point sample concentrations are not discharge weighted, the reported average concentrations may not be representative of the vertical before February 2010. A comparison of data collected between 1998 and 2010 and between 2010 and 2016 was made and is shown in Figure 14. When considering the data scatter, there are minimal differences between the two time periods. Size class fractions were also available at ORO from the Norton study. Measured sand size class fractions are shown in Figure 15.

Figure 10. Measured sediment concentrations at Simmesport, 1998–2016.

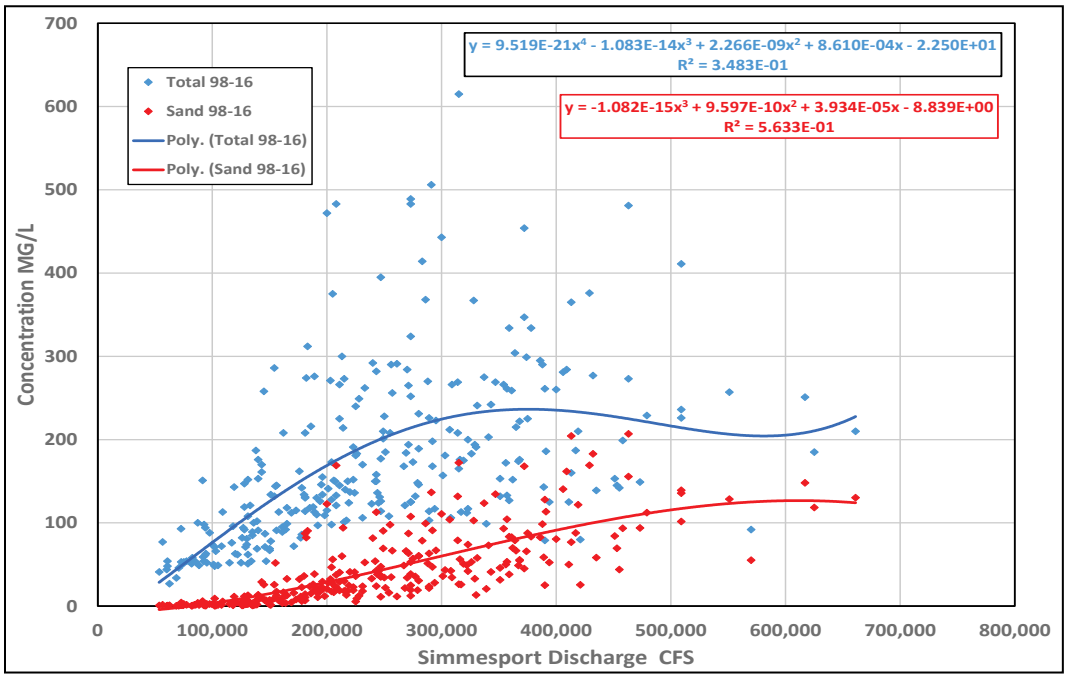


Figure 11. Measured sand size fractions at Simmesport, 1998–2016.

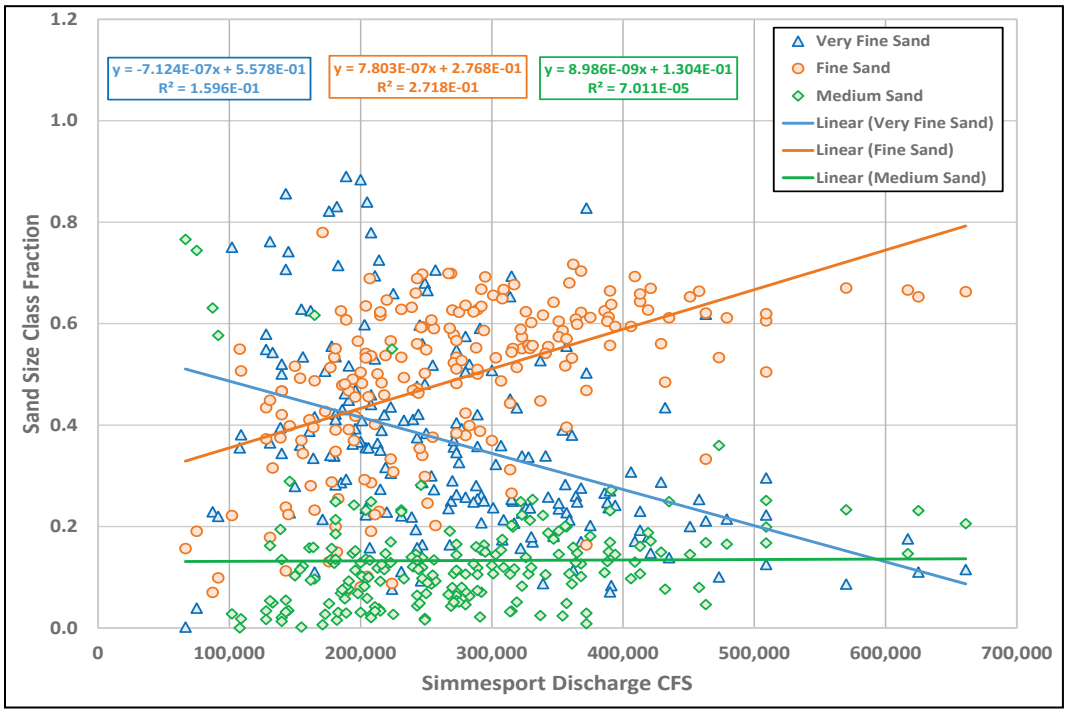


Figure 12. Measured sand concentrations at Simmesport, 1998–2016, with power regressions.

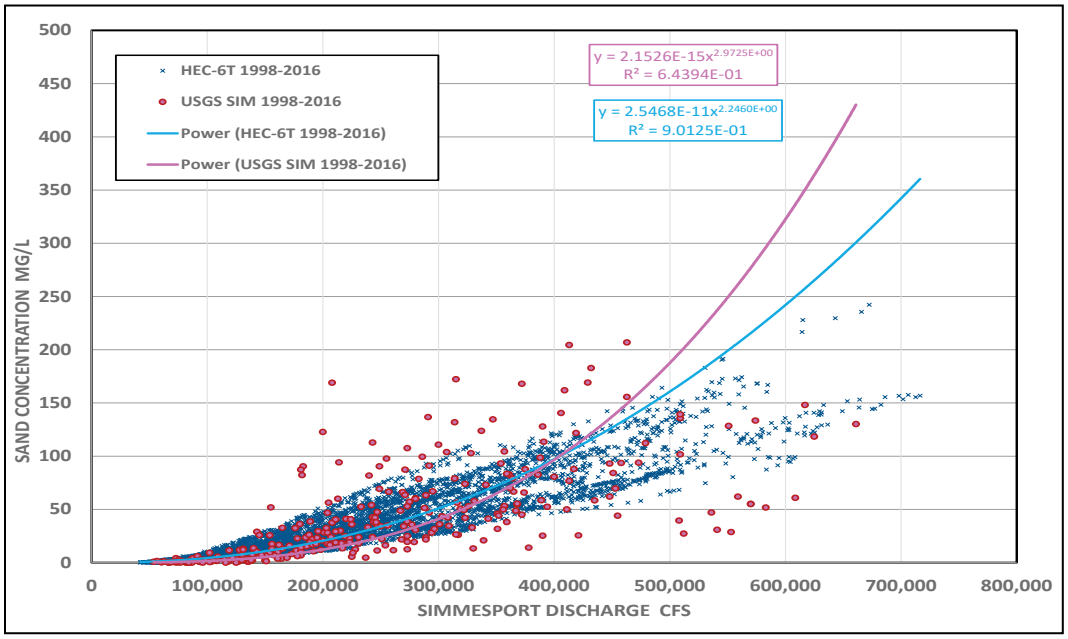


Figure 13. Measured sediment concentrations at Old River Outflow Channel, 1998–2016.

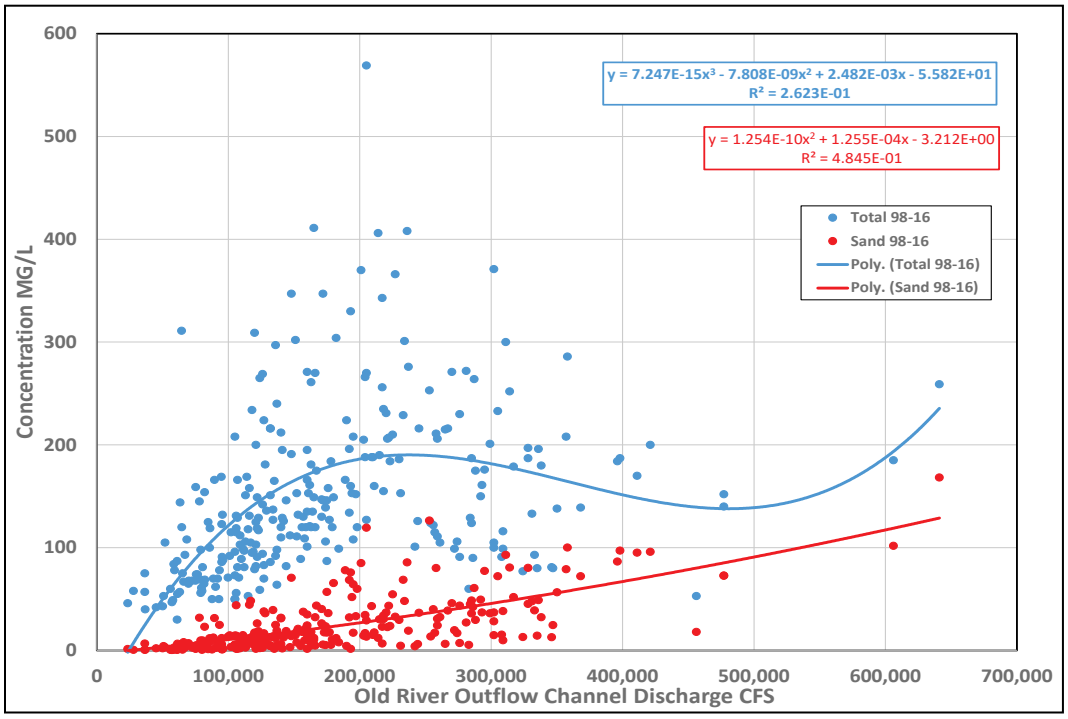


Figure 14. Measured sediment concentrations at Old River Outflow Channel, 1998–2010 and 2010–2016.

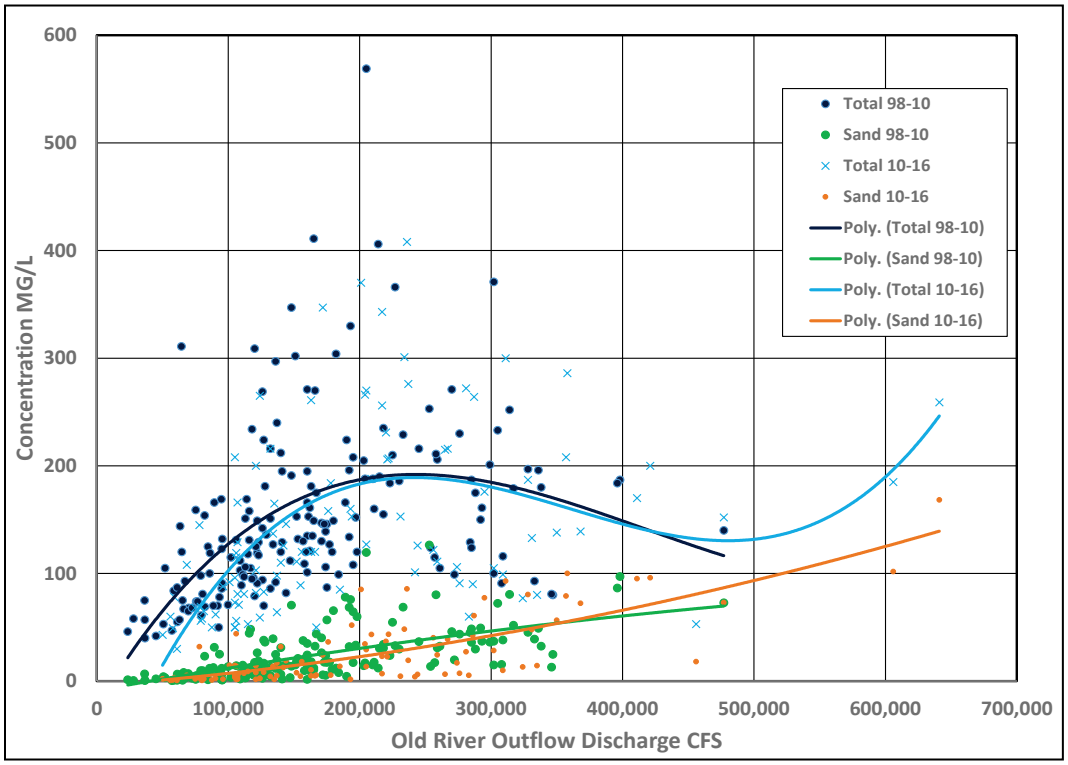
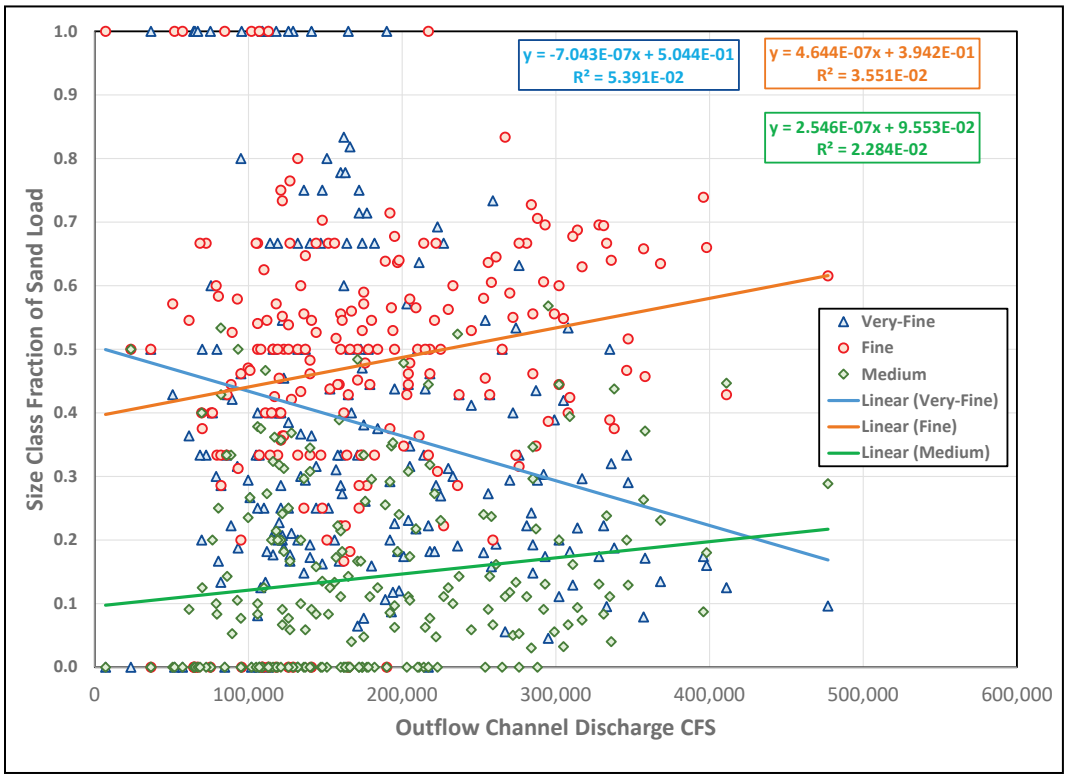


Figure 15. Measured sand size fractions at Old River Outflow Channel, 1998–2016.



In summary, the analytical procedure for calculating Red River sediment inflow was the following:

1. Red River discharges between 30 April 1997 and 30 September 2019 were calculated as the difference between Simmesport and Outflow Channel daily discharge data from Rivergages.com.
2. Daily Simmesport and Outflow Channel sediment loads (tons) were calculated from the 1998–2016 polynomial regression curves in Figure 10 and Figure 13.
3. Sand size class fractions were taken from Figure 11 and Figure 15 and used to calculate sediment loads (tons) for the sand size classes.
4. Fine sediment size class fractions were taken from Simmesport data used in the Atchafalaya Flowline Model (Lewis and Copeland 2018, Figure 11).
5. The daily Red River size class sediment loads (tons) were calculated as the difference in the Simmesport and Outflow Channel sediment size class loads and new regression curves for the Red River were calculated.
6. Red River rating curve concentrations were then calculated using the corresponding daily discharge.
7. The fine-sand and medium-sand fractions were increased by 15% to account for unmeasured load.

In the calibration phase of the study, the total sediment inflow for the Red River was reduced by 20% to achieve reasonable bed changes in the upstream reaches of the Red River segment and to achieve a better match between measured and calculated sediment transport at Simmesport. The final Red River sediment inflow curves used in HEC-6T are shown in Table 8.

Table 8. Red River sediment inflow.

Discharge 1000 cfs	.001	1	10	20	50	100	200	300	500
Concentration mg/L	0	92.32	117.2	141.1	192.8	228.7	220.2	203.1	197.8
Fraction									
Clay	0.283	0.283	0.302	0.310	0.311	0.278	0.182	0.153	0.153
Very Fine Silt	0.065	0.065	0.070	0.073	0.075	0.070	0.053	0.048	0.048
Fine Silt	0.049	0.049	0.052	0.053	0.051	0.043	0.020	0.014	0.014
Medium Silt	0.096	0.096	0.102	0.104	0.103	0.089	0.051	0.039	0.039
Coarse Silt	0.042	0.042	0.060	0.073	0.089	0.109	0.146	0.154	0.154
Very Fine Sand	0.310	0.310	0.243	0.201	0.146	0.121	0.121	0.127	0.127
Fine Sand	0.171	0.171	0.180	0.189	0.220	0.279	0.408	0.459	0.459
Medium Sand	0.032	0.032	0.034	0.036	0.042	0.053	0.079	0.092	0.092
Coarse Sand	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

2.13 Sediment diversion coefficients

Sediment diversion coefficients within the HEC-6T model specify how much of the sediment leaves when there is a split or outflow of water. These values were entered as the fraction, by particle size class, of the approaching sediment concentration that will be diverted relative to the upstream sediment concentration. Sediment diversion coefficients in the OMAR Model are unchanged from those used in the Flowline Model.

Flowline Model distributaries used sediment diversion coefficients from the 2010 Model or a previous 2005 MVN unpublished study (USACE MVN 2005). If newly added distributaries did not have available coefficients from the 2005 Study, the flow leaving was set to carry 100% of the upstream concentration within the clay and silt size classes, 50% of the upstream concentration of very fine sands, and 0% of the coarser size classes. This was deemed reasonable because the distributaries had bed elevations significantly higher than the bed elevation of the main channel.

In the Flowline Model, the major splits within the model, the upstream junction of the Whiskey Bay Pilot Channel and the upstream junction of the Wax Lake Outlet Channel, sediment diversion coefficients were adjusted during calibration. To reproduce measured data, the sediment

coefficients passing sand into the Wax Lake Outlet (relative to the concentration upstream of the junction) were increased to 100% to achieve calibration. At the Whiskey Bay junction, sediment diversion coefficients were adjusted to duplicate measured sediment accumulation in the Old Atchafalaya River between 1997 and 2010. More information on calibration of sediment diversion coefficients can be found in the Flowline Model Report.

2.14 Bed gradations

The initial bed gradations downstream from Simmesport were taken from the Flowline Model. In that study, bed samples collected in 1991 by Nordin et al. (1995) were numerically adjusted by simulating a bankfull flow for 173 days to adjust the bed to equilibrium conditions. The artificial warm-up period was used to calculate an initial bed gradation, which was then input into the numerical model with the original cross-section geometries.

In the OMAR Model, two segments were added to the Flowline Model upstream from Simmesport. A segment on the Red River extended upstream to the confluence of the Black River at RM 33. Another new segment included the Outflow Channel. Initial bed gradations for these new segments came from USACE and USGS measurements.

In the OMAR Model, Red River bed gradations were determined from samples collected at three sites in the reach between the Outflow Channel and the Black River. A single sample, taken in 2005, by the USGS was used at RM 17.5. USACE MVN collected 38 bed samples between 1999 and 2004 at RM 11. USACE MVK collected 37 bed samples between 2007 and 2016 at RM 33. Averages of the USACE data collection efforts were input into the HEC-6T model at their respective river mile and are shown in Figure 16. The average bed material gradations were remarkably consistent over several years at both high and low flows so no warm-up runs were deemed necessary to establish an equilibrium bed condition.

The OMAR Model initial Outflow Channel bed gradation was determined from the average of 184 bed samples collected by the USACE MVN between 1991 and 1999 at the Old River Outflow Channel near Knox Landing gage. The gage was located approximately 1.7 mi upstream from the confluence with the Red River. The 1991–1999 average bed gradation was consistent with data collected between 1974 and 1990, as shown in Figure 17. The bed material samples were collected over a 9 yr period at

both high and low flows so no warm-up runs were deemed necessary to establish an equilibrium bed condition.

Figure 16. Measured bed gradations Red River, 1999–2016.

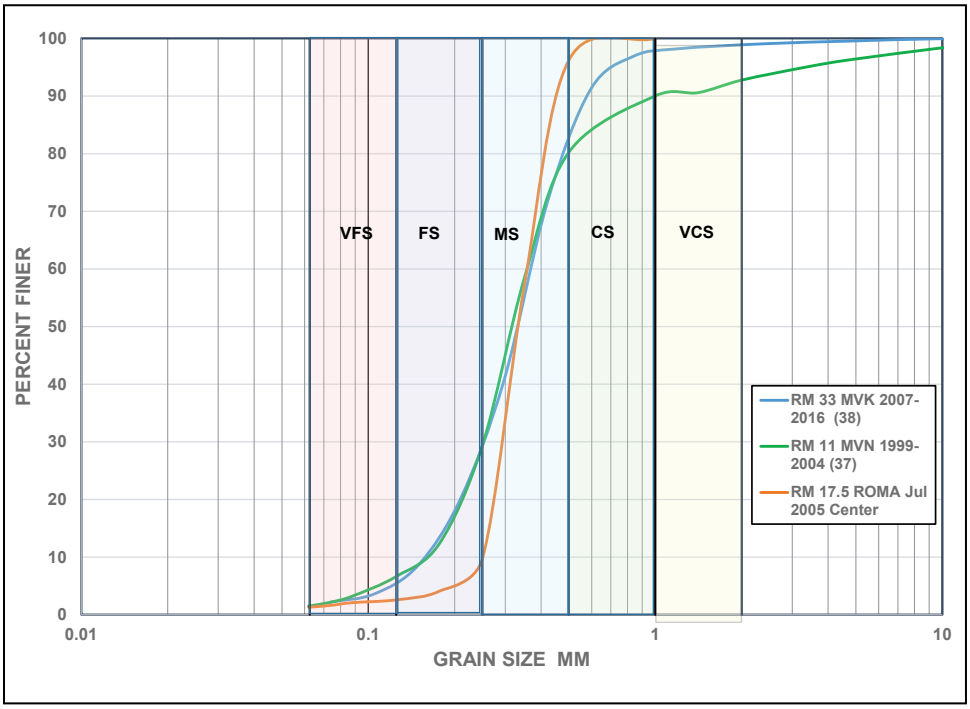
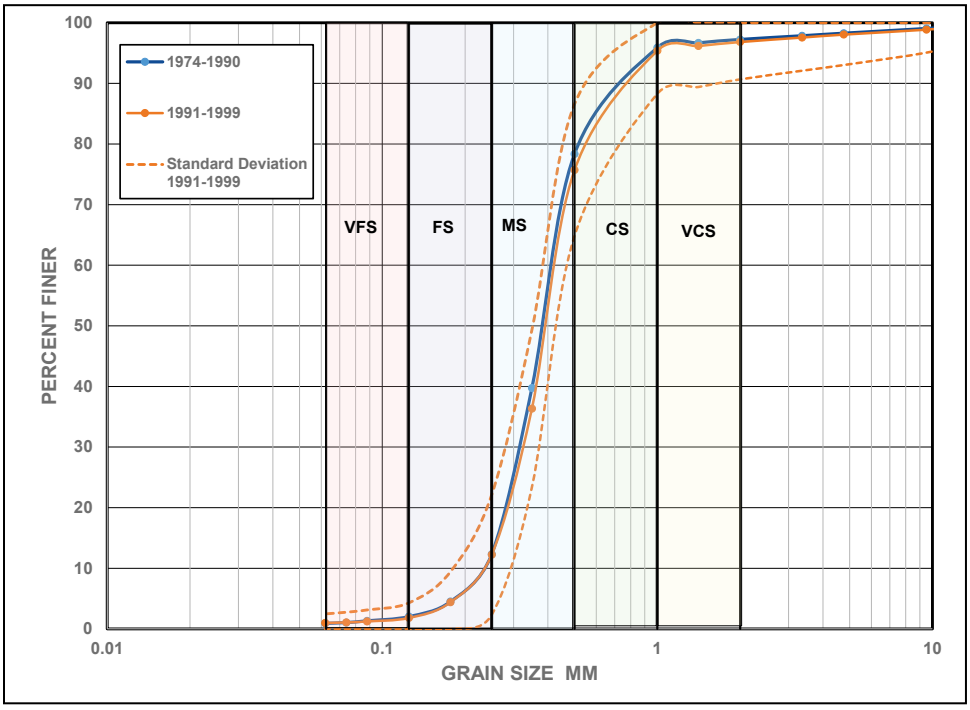


Figure 17. USACE Measured bed gradations Outflow Channel, 1974–1999.



3 Model Calibration and Verification

3.1 Flowline Model calibration and verification results

The calibrated and verified Flowline Model was deemed appropriate for use in the OMAR Model without any additional adjustment in the reaches downstream from Simmesport.

The Flowline Model calibration and verification results are discussed in Lewis and Copeland (2018). Calculated stages, discharges, and sediment concentrations agreed with observed data at gage locations. The root mean square error and the Nash-Sutcliffe Efficiency coefficient were calculated to quantify errors between observed and computed data. Many of the computed sand concentrations were higher than the observed sand concentrations, especially at lower discharges. This can be attributed to the fact that the HEC-6T results include both sand moving along the bed and suspended sand whereas the observed data include only the measured suspended sand. The survey data from 2010 were compared with the HEC-6T calculated cross-section geometries for 2010. Overall, the trends of deposition or scour within the channel agreed with the survey results. Recent survey efforts from 2012 to 2016 by MVN have revealed some locations in the upper reach of the Atchafalaya River where bank failure has occurred, which is not a process simulated in the HEC-6T model.

3.2 Adjustment of Outflow Channel sediment inflow

Initial runs with the HEC-6T Atchafalaya River model, using calculated daily discharges and sediment concentrations from the HEC-6T Mississippi River model, indicated excessive deposition in the scour hole downstream from the Low Sill Structure and in the Outflow Channel upstream from the confluence with the Hydropower Outflow Channel. This behavior does not match the actual sediment transport response downstream from the ORCC. One reason is that the deep scour hole downstream from the Low Sill Structure is not representative of average hydraulic conditions required in a 1D model. In addition, the OMAR model was designed to replicate long-term average flow and sediment conditions and does not account for the operational manipulations of the ORCC structures that occur frequently to flush or move accumulated sediment downstream.

The objective of the HEC-6T assessment was to evaluate long-term system-wide sedimentation effects rather than the local operational effects at the ORCC. Therefore, artificial dredging was incorporated into the model to move sediment downstream and maintain sediment continuity. The dredged material was reintroduced downstream from the Hydropower Outflow Channel. This adjustment maintained sediment continuity but renders the model simulations upstream from the Hydropower Outflow Channel artificial. Response to flushing operations is not accounted for in the model.

Outflow Channel suspended sand concentrations, calculated by HEC-6T, and USACE/USGS reported sand concentrations are shown in Figure 18. Data in the figure are downstream from the Hydropower Outlet Channel. Data collected between April 1997 and February 2018 were included in the analysis. April 1997 was chosen because it is the beginning of the OMAR historical simulation. In February 2018, the sampling protocol at the Outflow Channel gage changed from point sampling to depth integrated sampling. There was no comparative sampling reported in the USGS water quality web site, and it is assumed that none were done. The calculated HEC-6T sand concentrations are generally greater than the sand concentrations determined from sampling. Some of this difference can be attributed to the unmeasured sand load and bed load. The HEC-6T calculations include bed load whereas the USACE/USGS measurements do not.

The reported sand concentrations from the depth integrated sampling are considerably different than those obtained from the point samplers as shown in Figure 19. The USACE collected point samples at five equidistant points from three verticals between 1997 and 2010. The USGS collected point samples at four equidistant points from three verticals between 2010 and 2018. Depth integrated sampling commenced in 2018 and continued through 2021. Polynomial regression lines for the two point sampler data sets compare favorably (shown are third-order polynomials to be consistent with the depth integrated regression, but a linear and second-order polynomial provided similar results for the point sampler data). The depth integrated samples appear to be distinctly different from the point samples.

Calculated bed change in the Outflow Channel and at Simmesport for the 1997 through 2019 historical simulation period are shown in Figure 20.

Although there is significant variability in the calculated bed changes over the 2 yr simulation, the overall change is negligible.

The specific gage and survey data analyses from the OMAR geomorphic assessment (Task 2) (Lauth et al. 2022) can provide a measure of confidence to the HEC-6T simulated results. The geomorphic study and the HEC-6T historical simulation are in relative agreement that the Outflow Channel downstream from the Hydropower Outflow Channel and the Atchafalaya River at Simmesport are in dynamic equilibrium. The geomorphic report specific gage analysis at the Outflow Channel gage determined that true trend detection was impossible after 1980 due to the lack of data. However, the plotted data in the report certainly suggest relatively consistent water-surface elevations between 1992 and 2018. This result is consistent with HEC-6T calculated bed changes, which show no significant long-term aggradation or degradation. The specific gage analysis at Simmesport from the geomorphic study concluded that dynamic equilibrium has existed at the Simmesport gage since 1980. This is consistent with the HEC-6T long-term bed stability trend at Simmesport. The geomorphic study also used 1992, 2004, and 2013 Mississippi River hydrographic surveys to determine volume changes in the Outflow Channel downstream from the Hydropower Outflow Channel. They found degradation between 1992 and 2004 and then no change after 2004.

There are significant uncertainties associated with the sediment and survey data. The least uncertainty is associated with the relative long-term stability of the Outflow Channel between 1997 and 2019. Based on this conclusion, it was decided that the calculated sediment inflow from the Mississippi River, although higher than the reported data, was appropriate for use in the HEC-6T model. The calculated long-term stability also served to confirm the choice of sediment diversion coefficients for the ORCC structures and to verify the artificial dredging methodology used to maintain sediment continuity.

Figure 18. Calculated and measured sand concentrations in ORO, 1997–2018.

Figure 19. Sand concentrations at ORO from different sampling protocols.

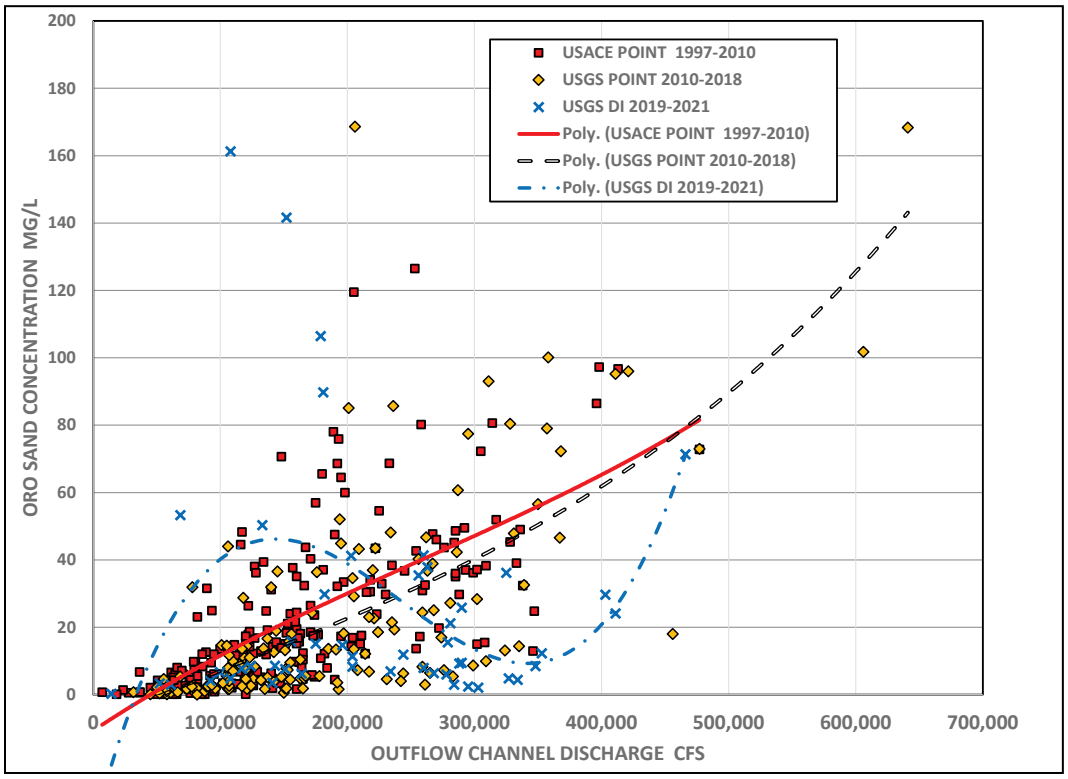
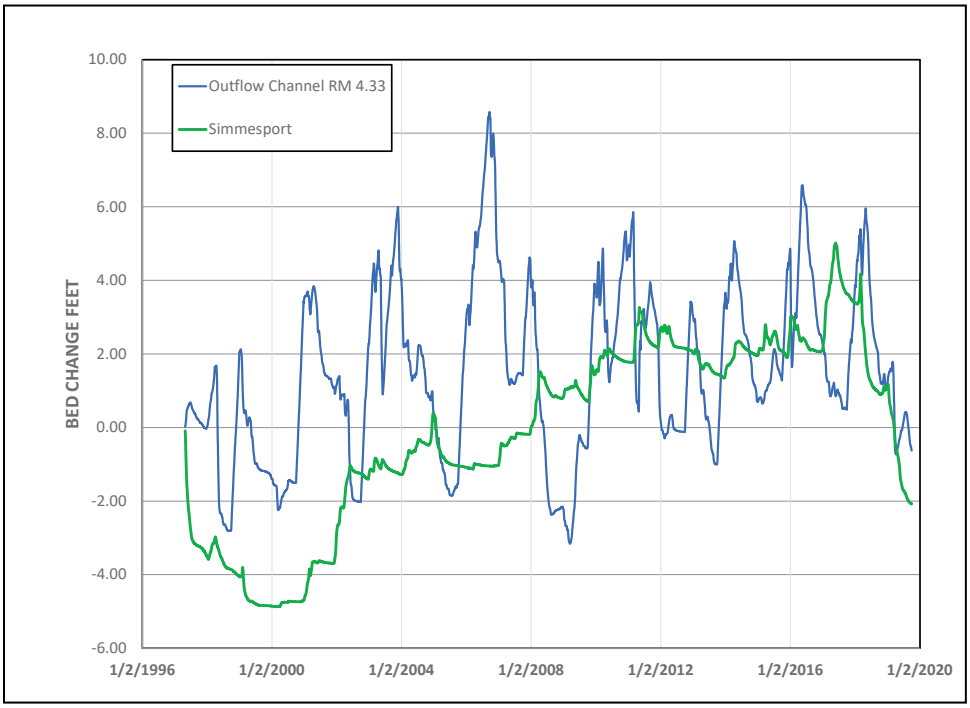


Figure 20. 1997–2019 calculated bed change in Outflow Channel and at Simmesport.



3.3 Calibration/Verification of Red River sediment inflow

The initial total sediment inflow rating curve developed from the difference in Simmesport and Outflow Channel data was reduced by 20% to bring calculated deposition at the upstream boundary into a more stable condition and to achieve a better match between measured and calculated sediment transport at Simmesport.

The Simmesport (RM -4.34) measured and calculated sand concentrations as a function of discharge, between 4 November 1998 and 30 September 2019, are compared in Figure 21 and the total sediment concentrations in Figure 22. The measured data are USGS/USACE data from the Norton report and the USGS web page. The HEC-6T sand concentrations were calculated using the adjusted sediment inflow at the Red River upstream boundary. Linear regression lines are shown in the figures for reference. In general, the calculated data are representative of the measured data in a long-term average sense. The USGS reported data are poorly correlated to discharge. The HEC-6T daily data indicate that there are probably several rating curves representing different sediment inflow and backwater conditions. The plots do give the reader some confidence that the sediment concentrations and particle size fractions that were determined

analytically are reasonable. The data scatter suggests that sediment data should be collected more frequently so that sediment rating curves can be adjusted with time.

Figure 21. Measured and calculated sand concentration at Simmesport, 1998–2019.

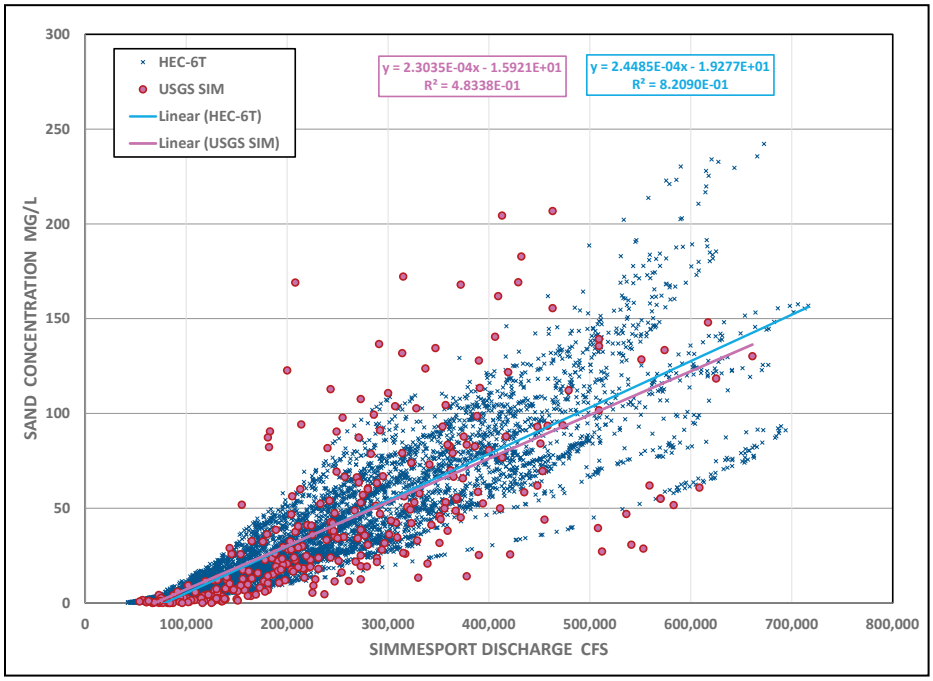
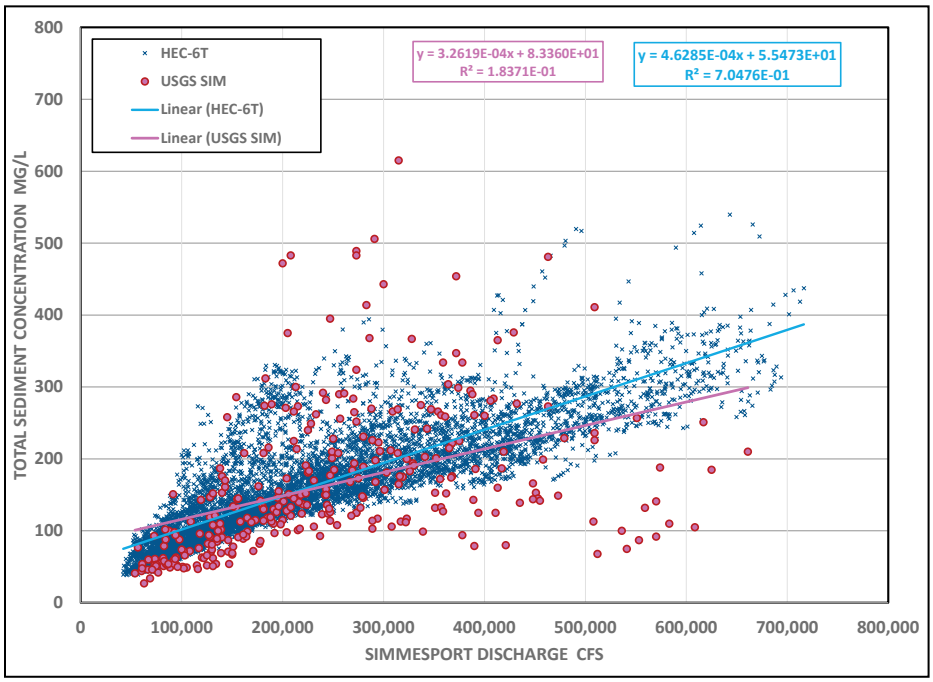
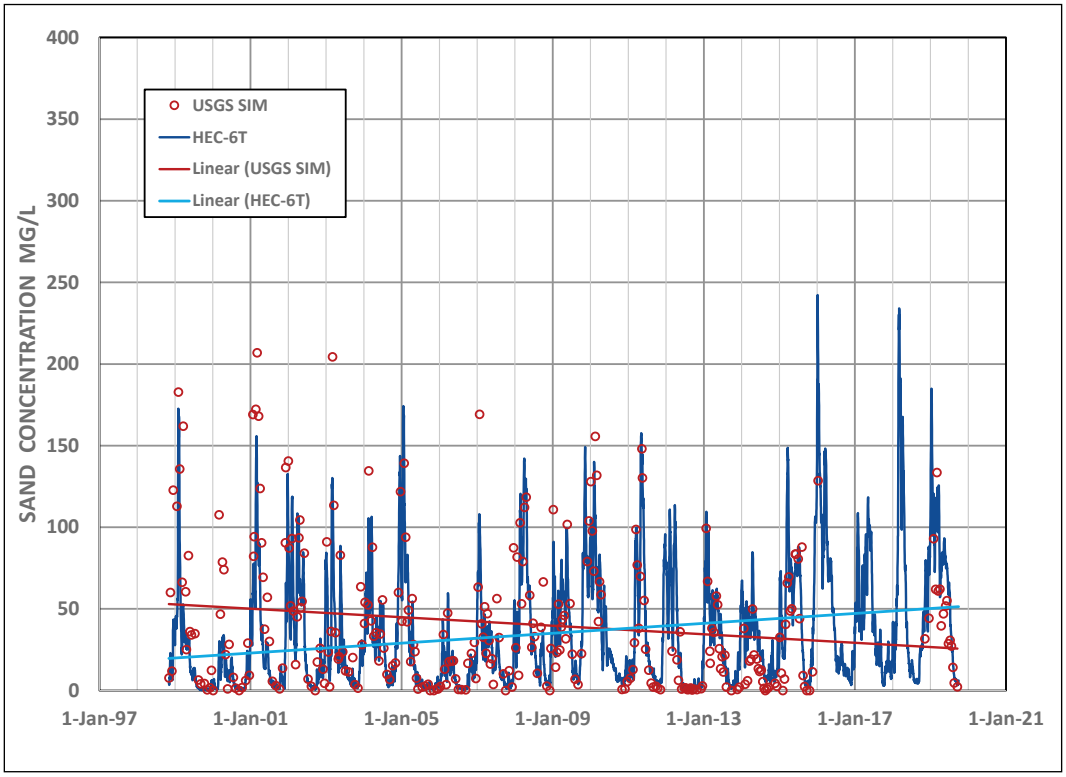


Figure 22. Measured and calculated total sediment concentration Simmesport, 1998–2019.



Simmesport measured and calculated sand concentration as a function of time, between 4 November 98 and 30 September 19, is shown in Figure 23. This plot indicates that the measured sand concentrations are higher than HEC-6T calculated sand concentrations at peak discharges through 2007 and that sand concentrations are decreasing with time. The daily HEC-6T calculated sand concentrations indicate that there is an increasing trend in the sand concentration. The paucity of data collection may be responsible for contradictory conclusions between the daily calculated data and the intermittent measurements.

Figure 23. Measured and calculated sand concentration at Simmesport, November 1998–September 2019.



The calculated sand yield at Simmesport between 1 October 1998 and 30 September 2019 was 12,330,000 tons/yr. Total sediment yield was 43,580,000 tons/yr.

Using the Simmesport linear regression curves developed from 4 November 1988 to 30 September 2019 USACE/USGS data (Figure 21 and Figure 22) and daily discharges from river gages.com, the calculated sand yield was 12,098,000 tons/yr, and total sediment yield was 40,448,000 tons/yr.

The HEC-6T calculated sand yield was 102% of the linear regression of measured yield. The HEC-6T calculated total sediment yield was 108% of the linear regression of measured yield. These results are well within acceptable limits considering the extreme scatter in measured data.

4 Model Results

4.1 Base Condition — Bed changes at select stations

HEC-6T calculates the average bed change at every cross section of the model for every time-step. Here, the bed change results at some representative locations are highlighted. Calculated bed change in the Red River is shown for Madam Lee (RM 33) and RM 11. RM 11 is just upstream from the Outflow Channel. Results for the historical years 1997–2019 are shown in Figure 24 and for the next 50 yr in Figure 25. The 50 yr future projection of deposition was calculated using the September 1969–September 2019 historical hydrograph, with current operation protocols at the ORCC (2013–2019) and the most recent available sediment inflow data. Sea-level rise was also included. Figure 24 shows significant annual variation in deposition depths at Madam Lee with accumulation at low flow and scour at high flow. Although the model assigns this deposition at the upstream boundary, in reality, this deposition is distributed throughout the backwater reach. The model also shows that the calculated annual variations have smoothed out at RM 11. The model is calculating a long-term aggradation trend in the Red River between Madam Lee and RM 11. Figure 25 shows a continued significant annual variation at Madam Lee for the next 50 yr with deposition at low flow and scour at high flow. The aggradation trend at RM 11 also continues but at a lower rate than for the historical period.

Figure 24. Calculated 1997–2019 bed change Red River at Madam Lee and RM 11.

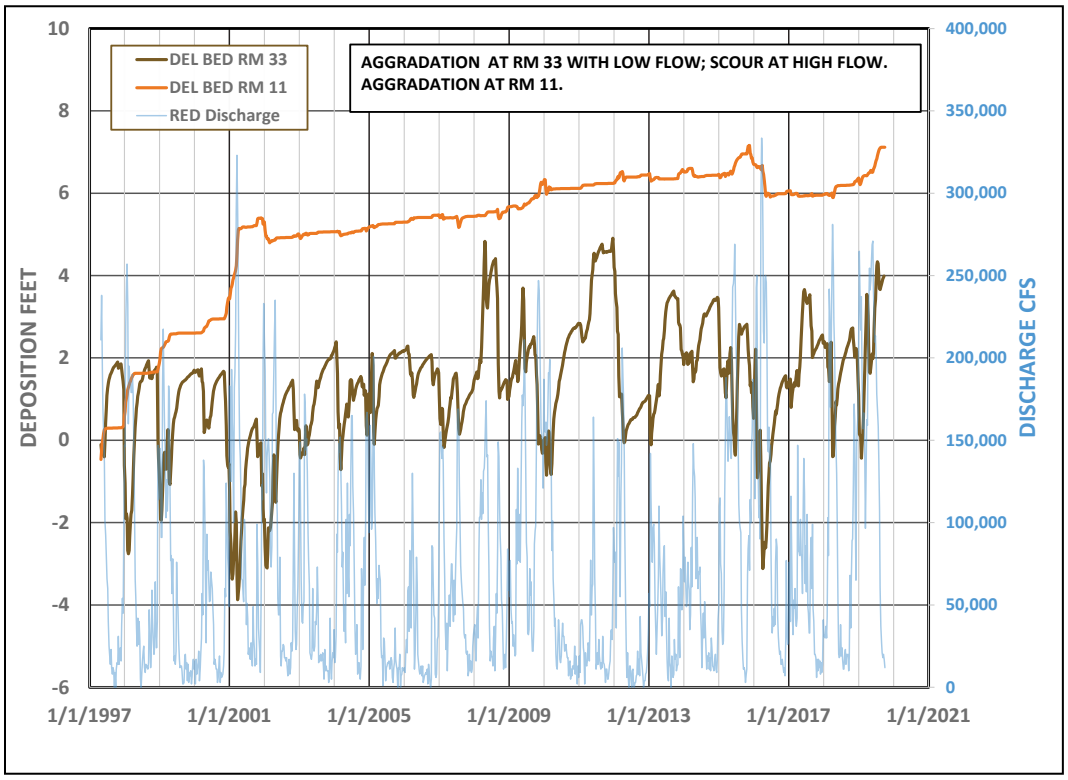
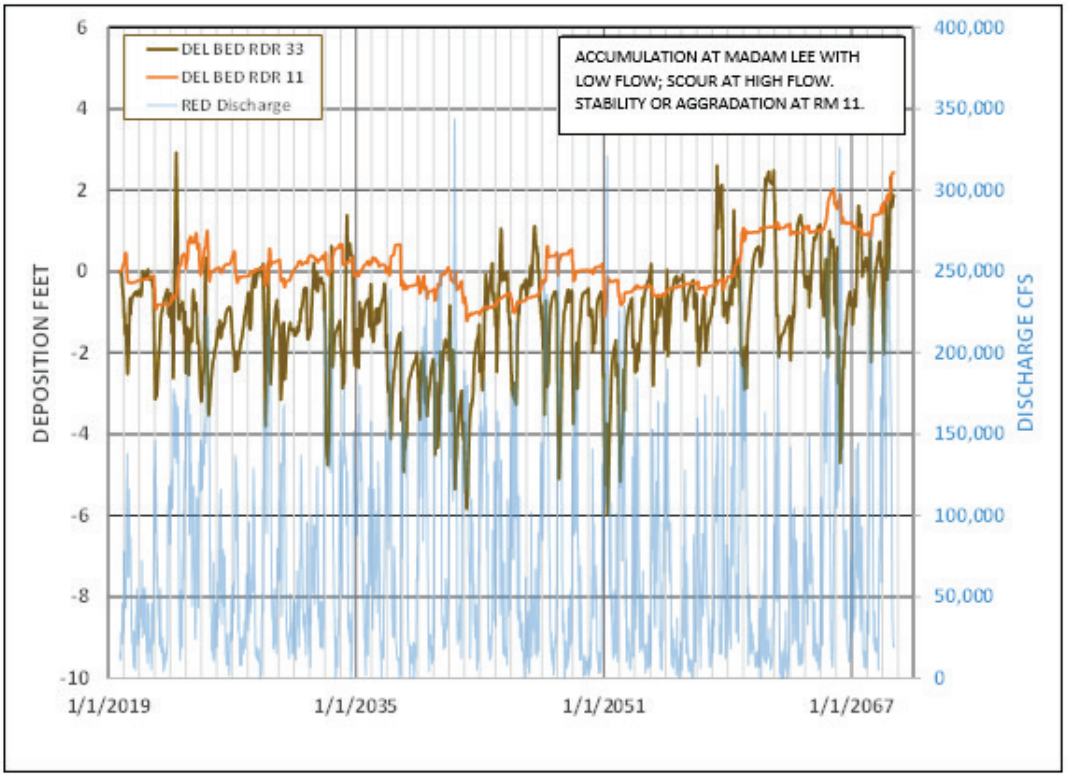


Figure 25. Calculated 50 yr bed change Red River at Madam Lee and RM 11.



Calculated bed change is shown for the Outflow Channel at RM 4.33, which is located just below the hydropower outflow channel at the USGS gage. Results for the historical years 1997–2019 are shown in Figure 26 and for the projected 50 yr in Figure 27. The historic hydrograph assumes current operations (2013–2019) at ORCC. Similar to Madam Lee, Figure 26 shows significant annual variation in deposition depths with accumulation at low flow and scour at high flow. It is difficult to determine if there is a long-term trend in deposition or scour. Figure 27 shows a continued annual variation in bed change for the next 50 yr. There may be a long-term aggradation trend in the Outflow Channel, but determination of such a trend would depend on the beginning and ending limits of the analysis. Any long-term trend is hidden in annual variations.

Figure 26. Calculated 1997–2019 bed change at Outflow Channel RM 4.33.

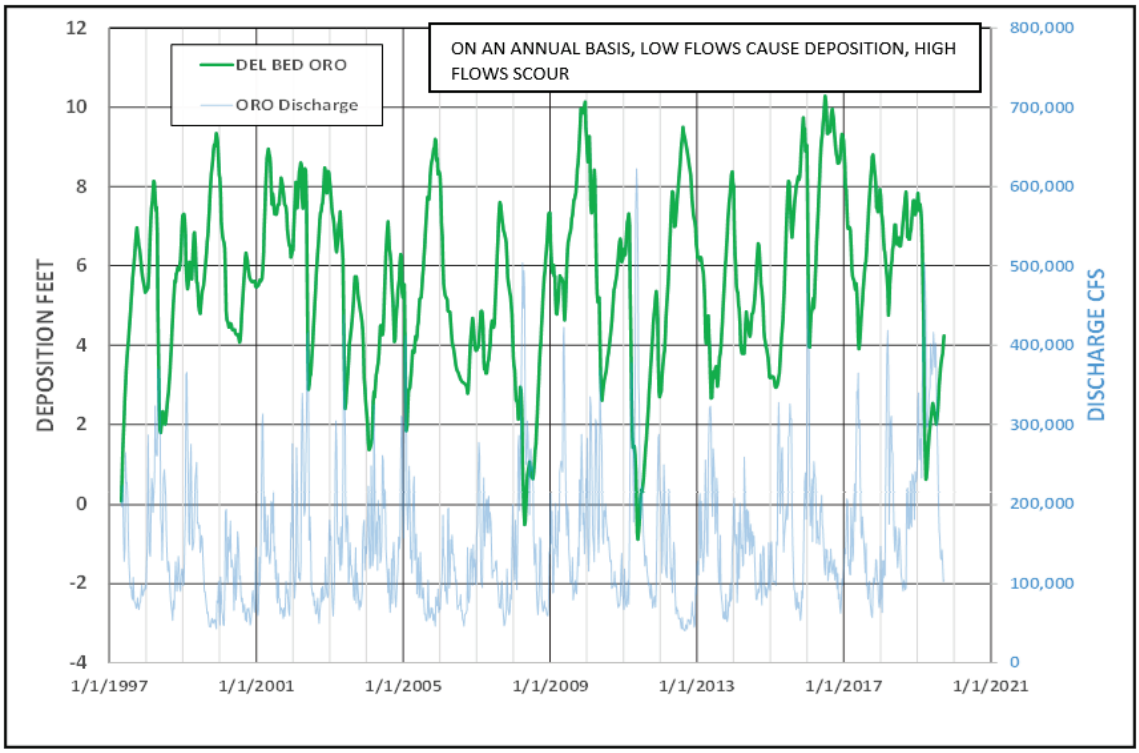
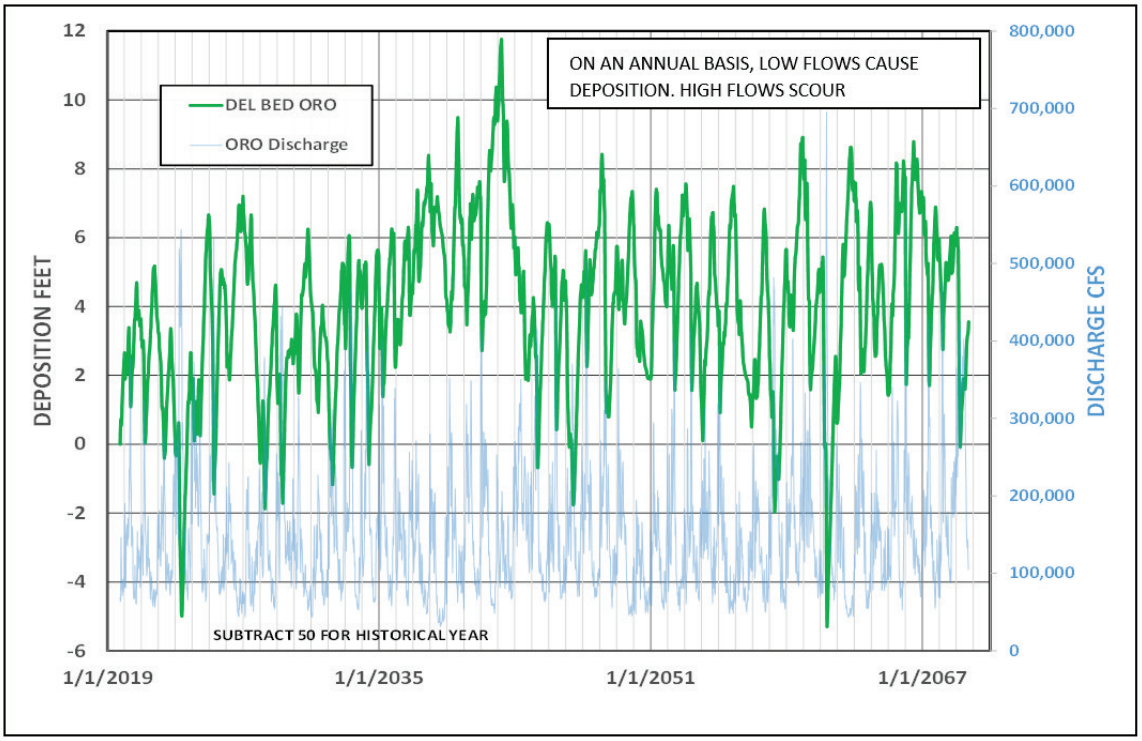
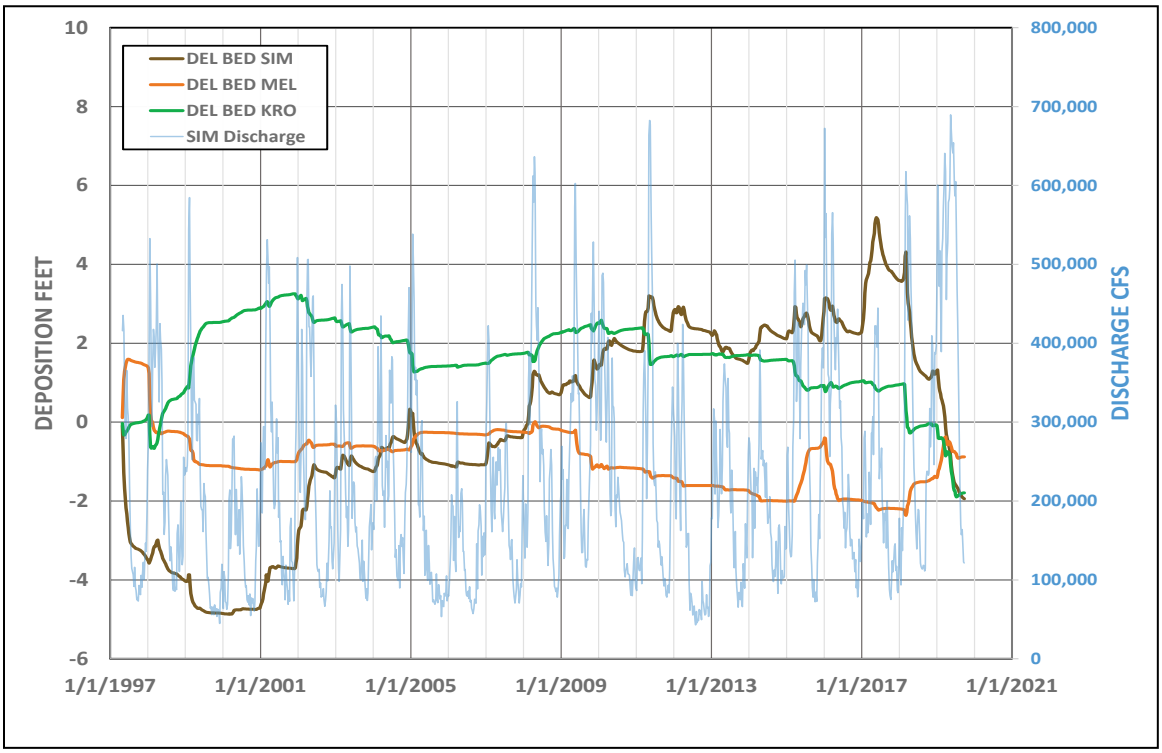


Figure 27. Calculated 50 yr bed change at Outflow Channel RM 4.33.



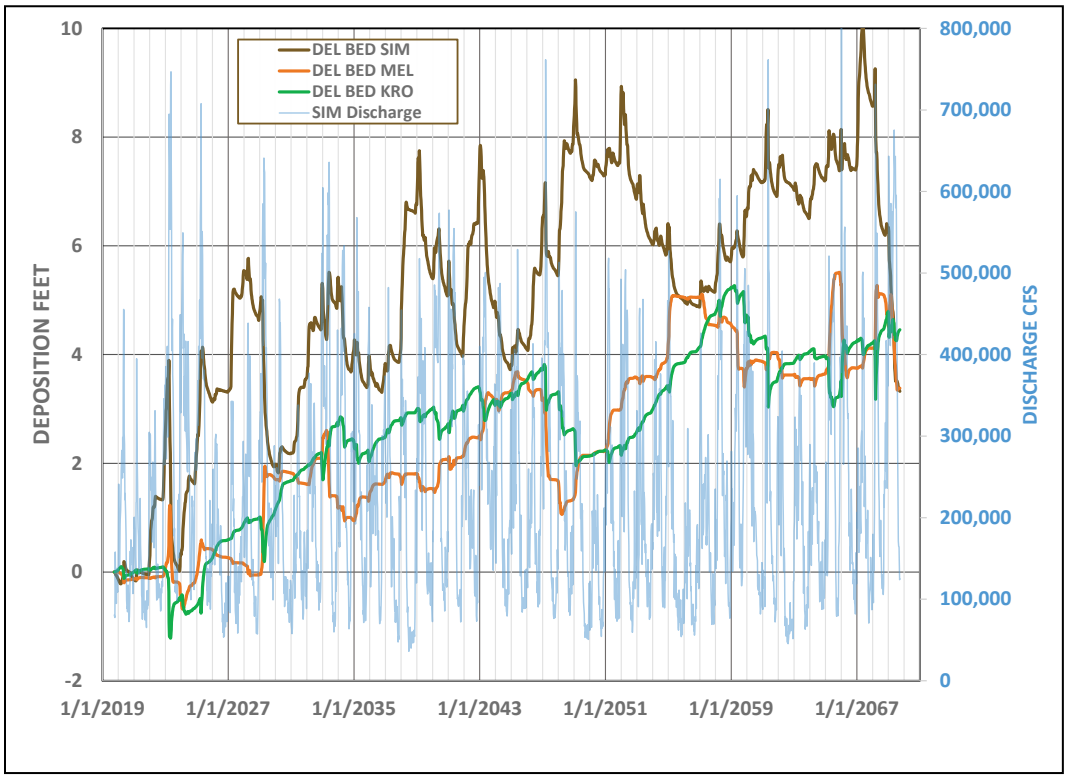
Similar plots were developed for the Atchafalaya River at Simmesport (RM -4.34), Melville (RM -29.6), and Krotz Springs (RM -39.95). Figure 28 shows the bed change between 1997 and 2019. Annual variations are less distinct than at the boundary stations in the Outlet Channel and the Red River. There appears to have been a deposition trend at Simmesport from 2001 through 2017. However, the last 2 yr indicated a degradation trend; comparing the starting and ending data points over the 22.4 yr simulation, there was a total scour of approximately 2 ft. Bed change at Melville appeared to be relatively stable until approximately 2009 when a slight degradation trend commenced. However, an abrupt increase in bed elevation of approximately 2 ft occurred in 2019, bringing the bed elevation back near its original 1997 elevation. At Krotz Springs, there was approximately 2 ft of deposition during the first 2 yr; then, the bed was relatively stable between 1999 and 2014. After 2014, a slight degradation trend began, and the bed fell back to its initial condition by 1 January 2019 and then another 2 ft by the end of 2019.

Figure 28. Calculated 1997–2019 bed change Atchafalaya River at Simmesport.



Results for the projected 50 yr are shown in Figure 29. The calculated bed change trend at Simmesport is similar to the 1997–2019 simulation with significant oscillations but not necessarily annually. There is an apparent deposition trend. However, when the starting and ending days of the 50 yr simulation are compared, there is only approximately 3.5 ft of deposition at Simmesport, which is considerably less than expected observing the plot. Long-term deposition is also apparent at Melville and Krotz Springs, though not as much as at Simmesport; however, the 50 yr difference is approximately the same at all three gages.

Figure 29. Calculated 50 yr bed change Atchafalaya River at Simmesport.



4.2 Base Condition — bed profiles

Accumulated bed volume changes between the Intracoastal Waterway at RM -124 to RM 0 were calculated and are presented for several time snapshots during the simulation in Figure 30 and Figure 31. Volume change during the first 22 yr, which extended between 30 September 1997 and 30 September 2019, is shown in Figure 30. Each line represents the total volume of sediment accumulation in the Atchafalaya River between River Mile 0 and the point on the *x*-axis between 30 September 1997, and 30 September of the specified year in the legend. After approximately 5 yr, deposition was calculated upstream from Simmesport (RM -4.34), then degradation between Simmesport and Melville (RM -29.6). The river was relatively stable between RM -30 and RM -85. Another degradation trend occurred between RM -85 and RM -100, which is upstream from the Wax Lake Diversion. Downstream from the Wax Lake Diversion aggradation occurs for approximately 10 mi. The deposition spike at Morgan City (RM -120) is related to the dredging in Berwick Bay.

After 2002, aggradation was the dominant process until 2019. The 2007 accumulated volume change profile followed the same general trend as

2002 but with less accumulated degradation. There was a significant flood in 2008 that may be responsible for the change in distribution of the accumulated volume change between 1997 and 2010. Some locations between 2007 and 2010 show aggradation and others degradation, with aggradation being dominant. There was another significant flood in 2011 with inflow from the Morganza Spillway. The general trend continued to be aggradation between 2010 and 2013.

Between 2013 and 2019, deposition continued to be the dominant process. However, considerable degradation occurred between RM 0 and RM -47. The period of 2013–2019 marked a time where annual flow volumes were significant and included several high peak flows. In addition, ORCC operations were changed in 2013.

Figure 31 shows little change between 2019 and 2023. The 1973 hydrograph (a high flow year) was used for 2023. After 2023, there is generally increasing accumulation volumes through 2063, especially downstream of RM -60. Degradation returned between 2063 and 2069 for the area upstream of RM -80.

Figure 30. Calculated 1997–2019 accumulated bed-deposition volume change–Atchafalaya River RM 0 to RM -130.

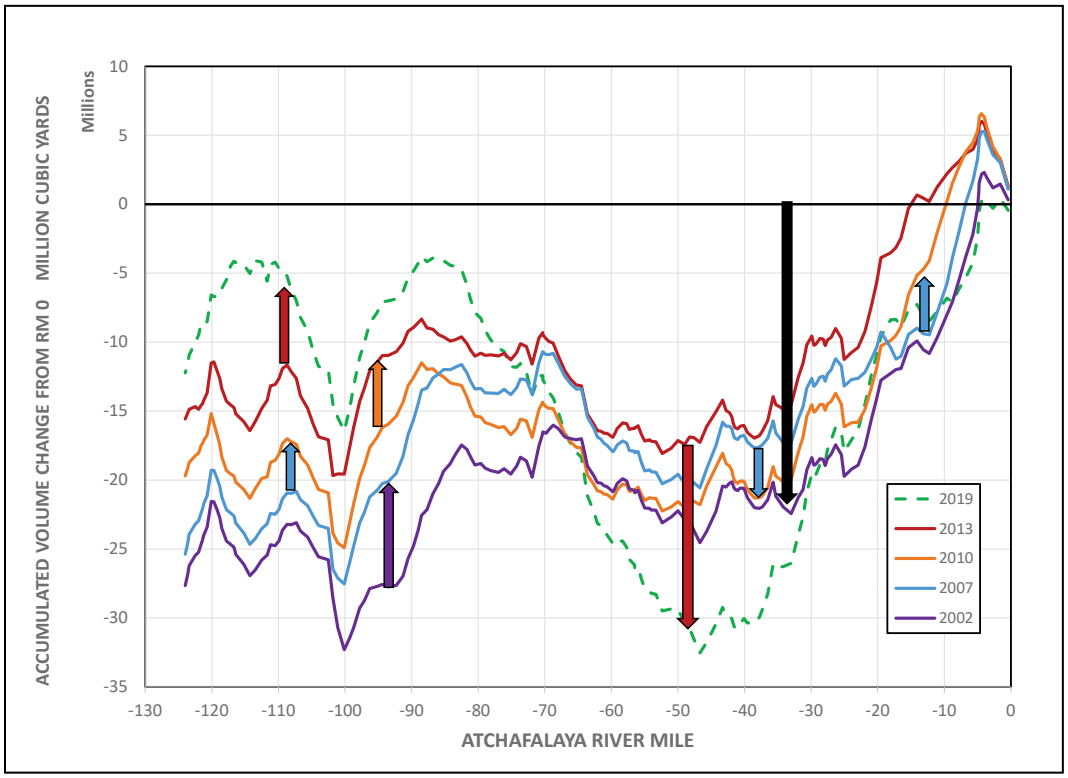
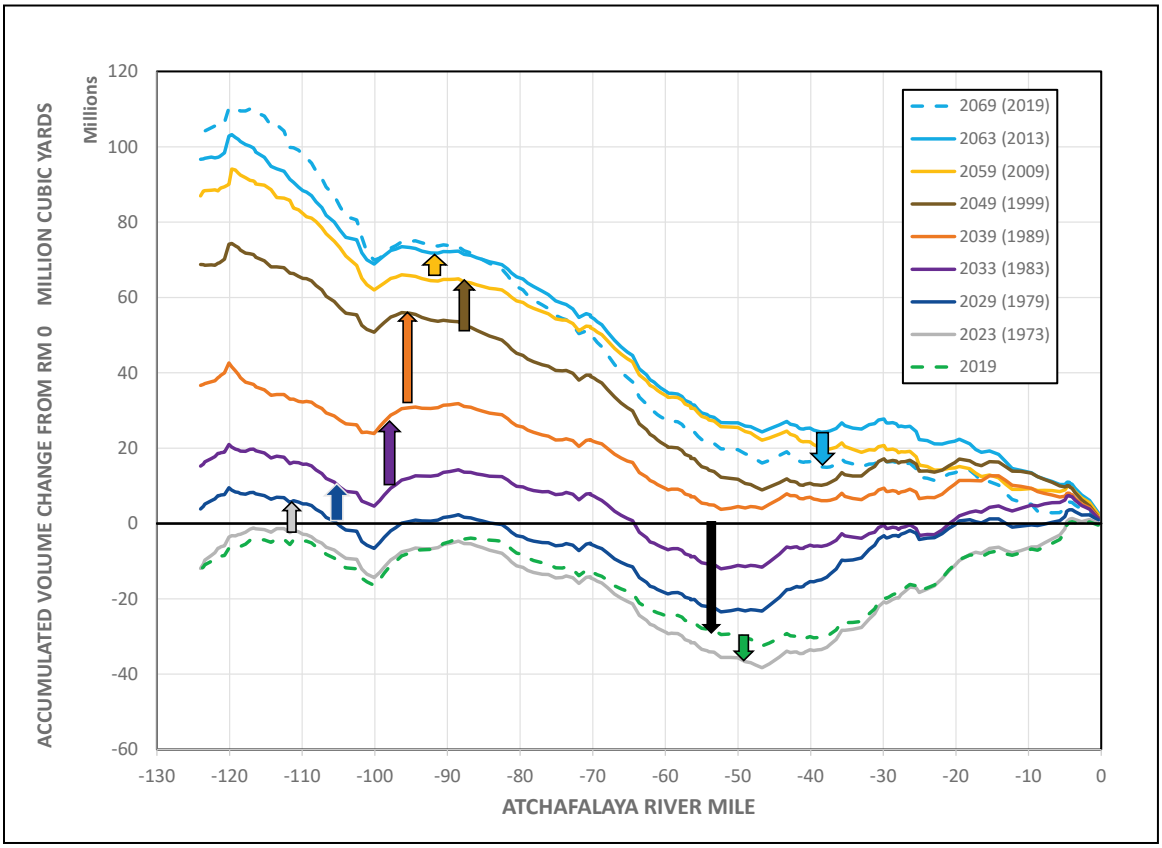


Figure 31. Calculated 2019–2069 accumulated bed volume change–Atchafalaya River RM 0 to RM -130.



4.3 Scenario evaluations

The objective of the OMAR Model was to determine the long-term (50 yr) and system-wide sedimentation effects that would occur in the Atchafalaya River if the operation protocols at the ORCC were changed. The changes evaluated included increasing and decreasing the percentage of flow that is diverted from the Mississippi River at the ORCC and modifying the distributions through the four ORCC structures. Another scenario was to dredge material from the Mississippi River and dispose of it in the Outflow Channel. Daily sediment and water discharges through the ORCC structures for the Base Condition and the various scenarios were calculated by the OMAR Mississippi River HEC-6T model. Scenarios were evaluated by

- Comparing future specific gage trends at several gages in the Atchafalaya and Red River and in the Outflow Channel
- Comparing 50 yr changes in water-surface elevations
- Comparing the 50 yr changes in water yields

- Comparing the resultant change in sand passage and sediment yield at various gages along the Atchafalaya River and in the Outflow Channel
- Comparing changes in the volume of dredging at Berwick Bay in Morgan City
- There were nine different scenarios compared to the Base condition in this assessment. These scenarios are described in the following.

Base Condition - Base projection of continuing current (post 2013) operations (no flushing).

Scenario 3: (R1) — RATIO 1, as defined in Design Memorandum 17 (USACE MVN 1980). This is a scenario where there is no hydropower, and a large amount of flow goes through the Auxiliary Structure.

Scenario 4: (R1WH) — RATIO 1 WITH HYDROPOWER, (as defined in Old River Control O&M Manual [USACE MVN 1988]). Essentially, hydropower shares with Auxiliary. Low Sill will not be used in this scenario. Anything that would go to Low Sill in the table of the O&M Manual will add to Auxiliary instead.

Scenario 5: (6040HF) — 60/40 during high flow, but 70/30 on long-term basis through the use of a payback/tracking system.

Scenario 6: **(8020HF) —** 80/20 during high flow, but 70/30 on long-term basis through the use of a payback/tracking system.

Scenario 7: **(CTAR125) —** Push all flow over 1.25 Mcfs at Tarbert through ORCC into the Atchafalaya River and 70/30 on long-term basis through the use of a payback/tracking system.

Scenario 8: Daily 80/20.

8a: (D8020) — Without any constraints.

8b: (D8020WLSC) — With Low Sill Head Differential constraint.

Scenario 9: **(D6040)** — Daily 60/40.

Scenario 10: **(MAX AUX) —** Maintain 70/30. Increase Auxiliary and decrease Hydropower (to ~ 1/3 ORCC flow), based on conditions.

Dredge Scenario: The base condition except that sediment material is dredged from the Mississippi River and deposited into the outflow channel on a daily basis.

4.4 Study locations of interest

For this study, several different locations were selected to highlight the model results. For all these locations, a specific gage graph was generated that compared every scenario to the base condition. The study locations of interest and their associated river miles can be found below (Figure 32–Figure 37).

Figure 32. Simmesport, LA — RM: -4.34; Melville, LA — RM: -29.6; Krotz Springs, LA — RM: -39.59.

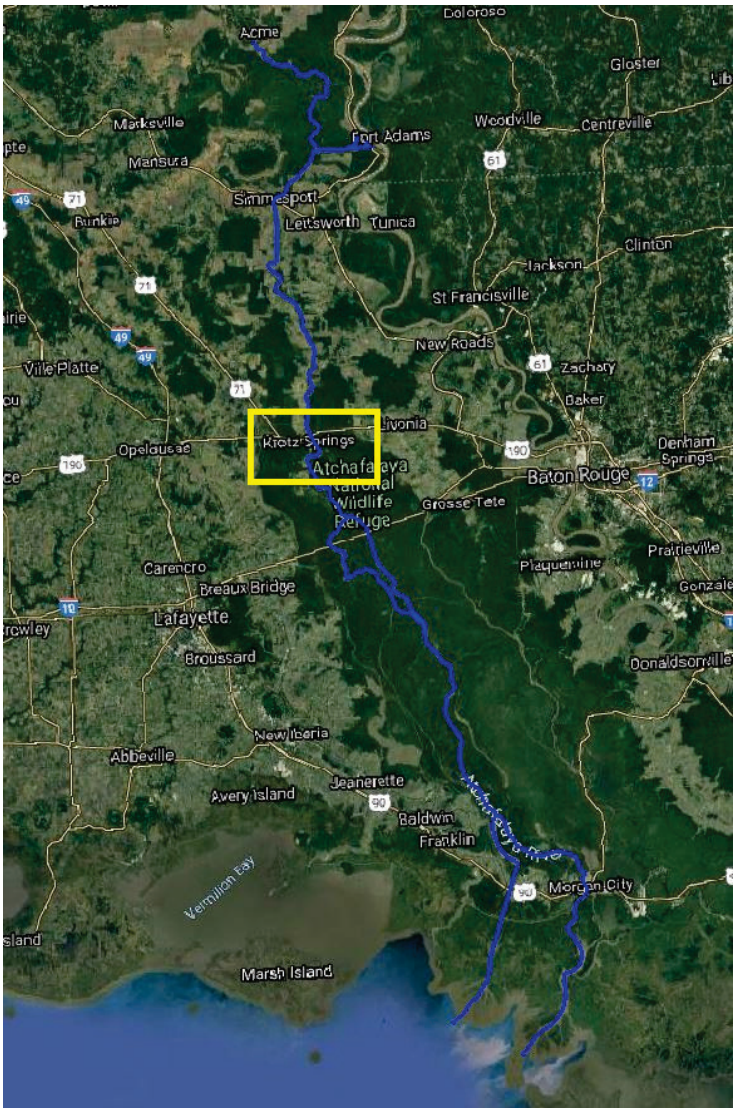


Figure 33. Calumet, LA – RM: -111.2.

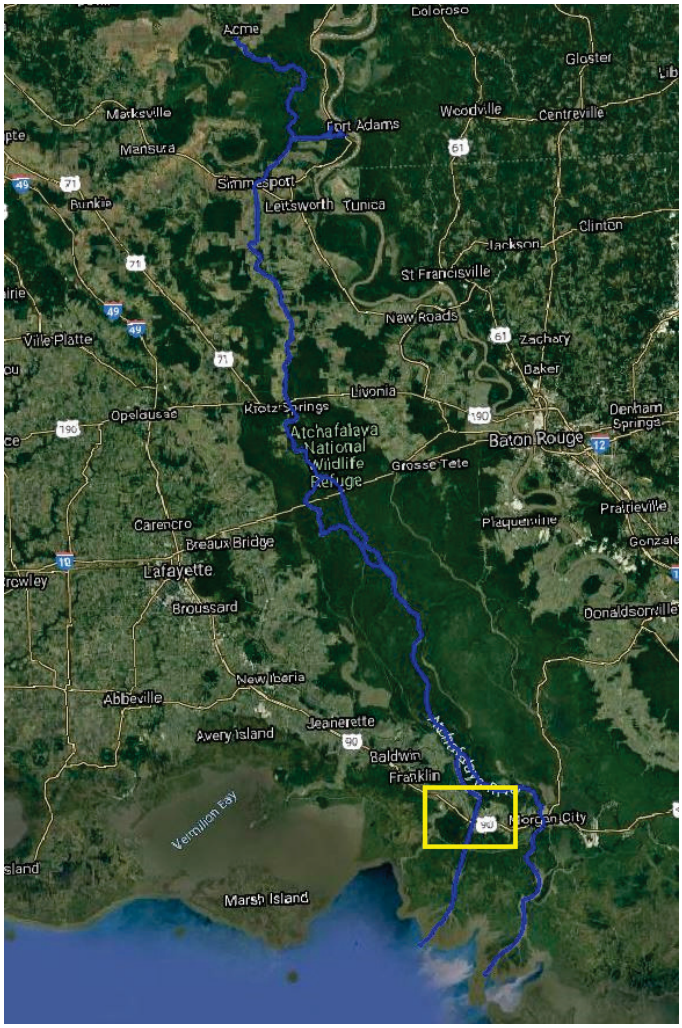
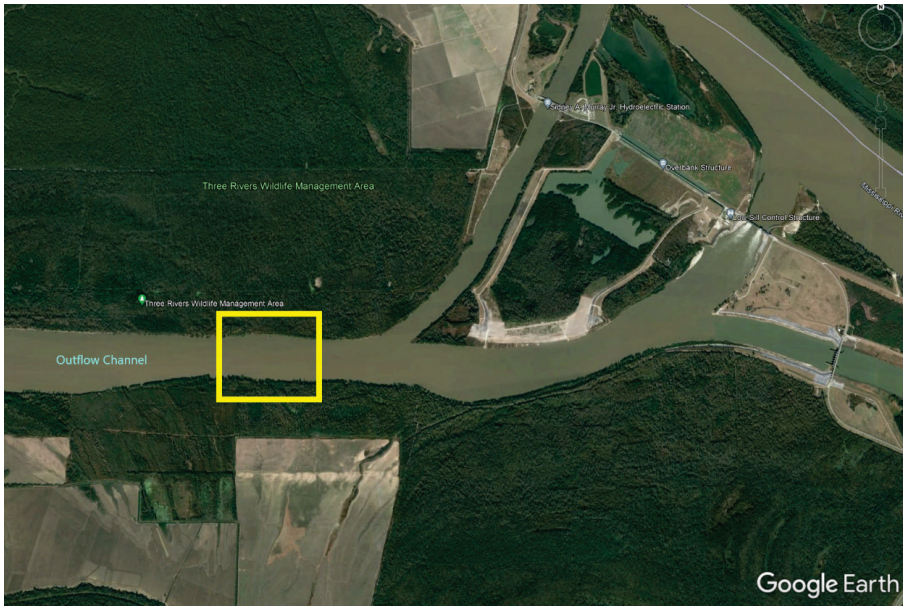


Figure 34. Outflow Channel — RM 4.33. The study location is a point just downstream of the ORCC structures in Louisiana.



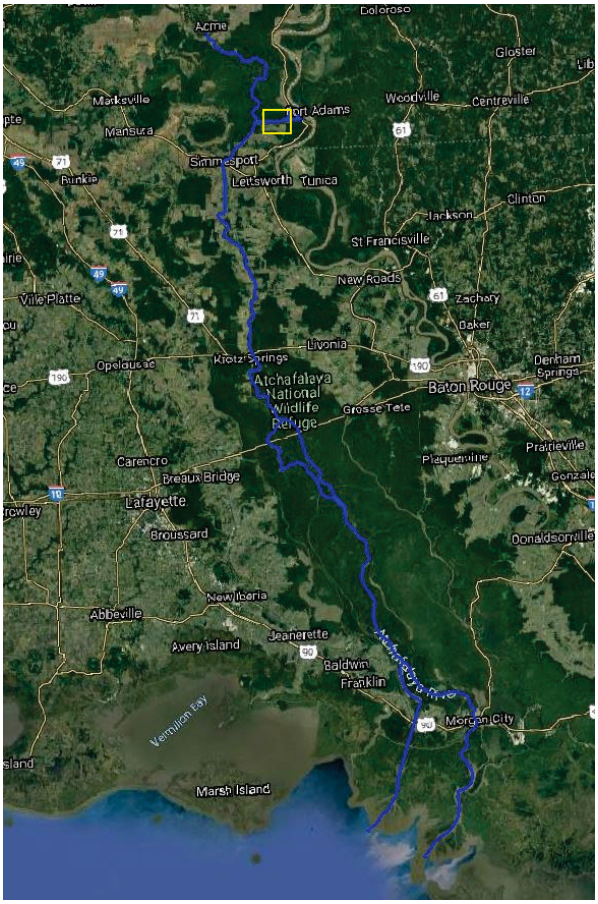
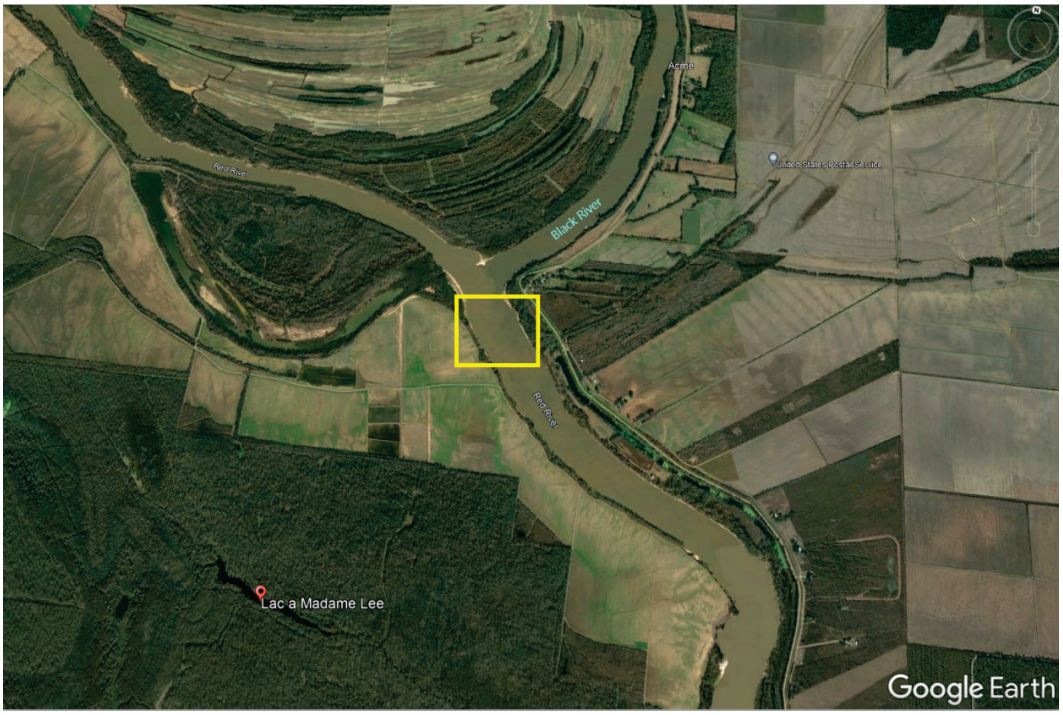


Figure 35. Red River Upstream — RM 33. The study location is the upstream boundary of the Red River.



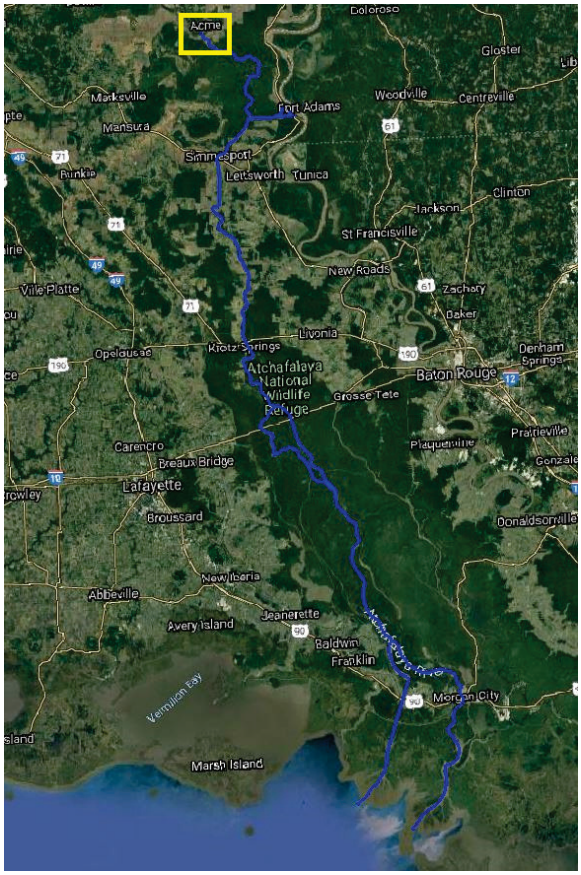


Figure 36. Red River Downstream — RM 10.6. The study location is the downstream boundary of the Red River just before it confluences with the Outflow Channel.

Figure 37. Morgan City, LA — RM: -121.2.

4.5 Difference in specific gage

As part of this study, specific gage plots were developed for every scenario at each study location of interest. The Base condition and scenario simulations started with the 1997 geometry and historical inputs between 30 April 1997 and 30 September 2019. On 1 October 2019, the model input *switches* to the scenario that is being studied. For example, in Scenario 3 (R1), historical conditions are modeled from 30 April 1997 to 30 September 2019. When the model run reaches 1 October 2019, the model switches and begins running the flow distribution conditions described by R1. This change is marked by a vertical line in the specific gage plots by displaying “Scenario Begins”. The specific gage plots compare specific discharge water-surface elevations for each scenario to the base condition water-surface elevations. Note that for every study location, water surfaces were calculated for four specific discharges unless there were an insufficient number of days within the specific discharge range to produce a meaningful plot. Table 9 shows the specified discharges at each location. All points that were not within plus or minus 1% of the specified discharge were removed. For the highest specified discharge, the range was extended to plus or minus 1.5% to capture additional points. Note that in certain scenarios such as Scenario 8A (80/20 split), no points are captured in the

highest specified flow since the Atchafalaya River would receive only 20% of the flow distribution.

In comparing specific gage plots for scenarios to the Base condition, it must be remembered that the plots compare calculated water-surface elevations for specific discharges and not for comparable events. That is to say, on a given date in the simulation hydrographs, scenario discharges will be different from the Base condition discharges. Therefore, calculated water-surface elevations for a specific discharge does not represent the same hydrologic frequency.

Additionally, the scenario thalweg versus the Base condition thalweg was calculated for every scenario at each location. The thalweg plots represent the true simulated bed elevations for each day, and the Base condition and scenario are directly comparable.

The geomorphological response of the Atchafalaya River and its effect on water-surface elevations can be inferred from the specific gage plots. The scenarios that provided the most extreme response in the Atchafalaya River is Scenario 8A (D8020) and Scenario 9 (D6040). Specific gage plots from these scenarios along with the dredged disposal scenario are discussed in the following paragraphs. Specific gage plots from the other scenarios are found in the Appendix.

Table 9. Specific gage — specified discharges at each location.

Location	Specified Discharges
Simmesport	100,000 cfs 200,000 cfs 400,000 cfs 600,000 cfs
Melville	100,000 cfs 200,000 cfs 400,000 cfs 600,000 cfs
Krotz Springs	100,000 cfs 200,000 cfs 400,000 cfs 600,000 cfs

Table 9. (cont.) Specific gage — specified discharges at each location.

Location	Specified Discharges
Calumet	50,000 cfs 100,000 cfs 200,000 cfs 300,000 cfs
Morgan City	50,000 cfs 100,000 cfs 200,000 cfs 300,000 cfs
Outflow Channel	70,000 cfs 140,000 cfs 280,000 cfs 420,000 cfs
Red River Upstream (RM 33)	30,000 cfs 60,000 cfs 120,000 cfs 180,000 cfs
Red River Downstream (RM 10.6)	30,000 cfs 60,000 cfs 120,000 cfs 180,000 cfs

4.6 Scenario 8A (D8020)

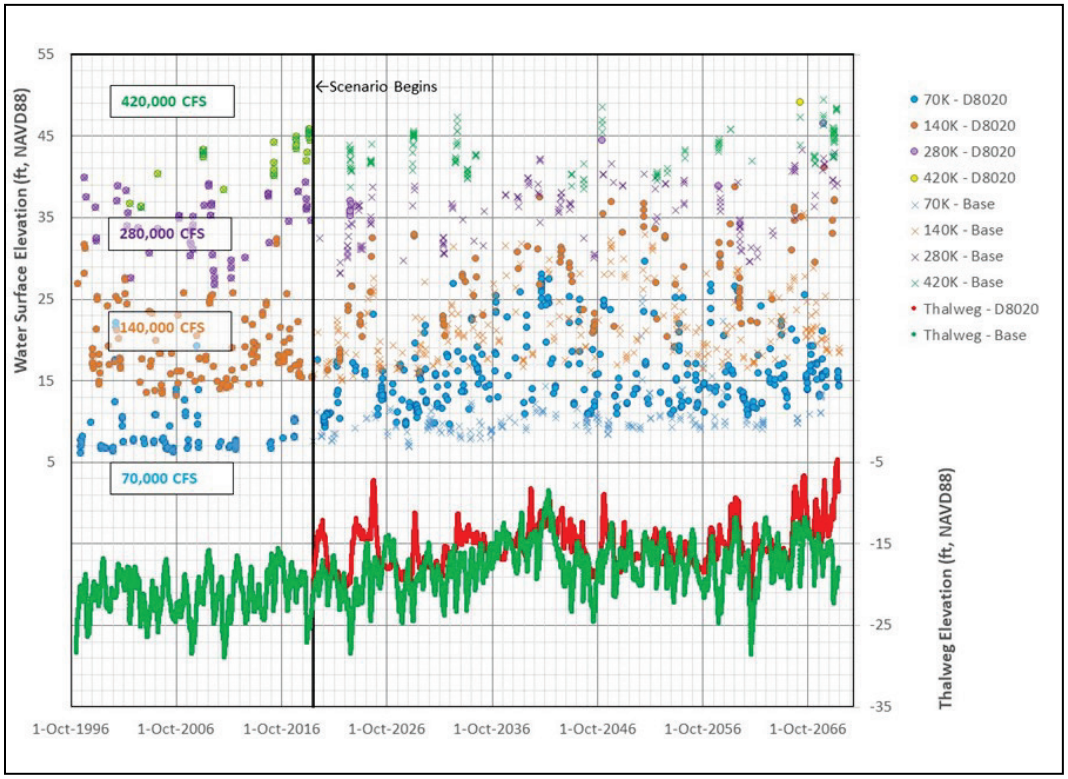
This scenario changes the flow distribution at the ORCC so that 80% of the latitude flow at Tarbert Landing and Simmesport goes down the Mississippi River for every day. The effect on the Atchafalaya River is significant because the discharge is reduced by a third and produces an even larger reduction in sediment transport capacity.

4.6.1 Outlet channel (RM 4.33)

As presented in Figure 38, annual bed oscillations occur at this location in the Old River Outlet Channel with the Base and with the other hypothetical scenarios. On average, the Scenario D8020 bed elevations were 2.4 ft higher relative to the Base across the whole 50 yr. The maximum daily difference was 14.7 ft toward the end of the 50 yr simulation, but this magnitude of deposition was uncharacteristic of the simulation period. The final year of the 50 yr simulation used the 2019

hydrograph, which was a very high flood year. There was only 1 day with a discharge of 420,000 cfs and only a few at a discharge of 280,000 cfs due to the diversion reduction with this scenario. The two lower specific gage discharges produced significant scatter in the calculated water-surface elevations. Scenario water-surface elevations were generally higher than Base water-surface elevations. The higher water-surface elevations are attributed to the increased deposition with Scenario D8020.

Figure 38. Difference calculated in specific gage at Outflow Channel RM 4.33 for Scenario D8020.

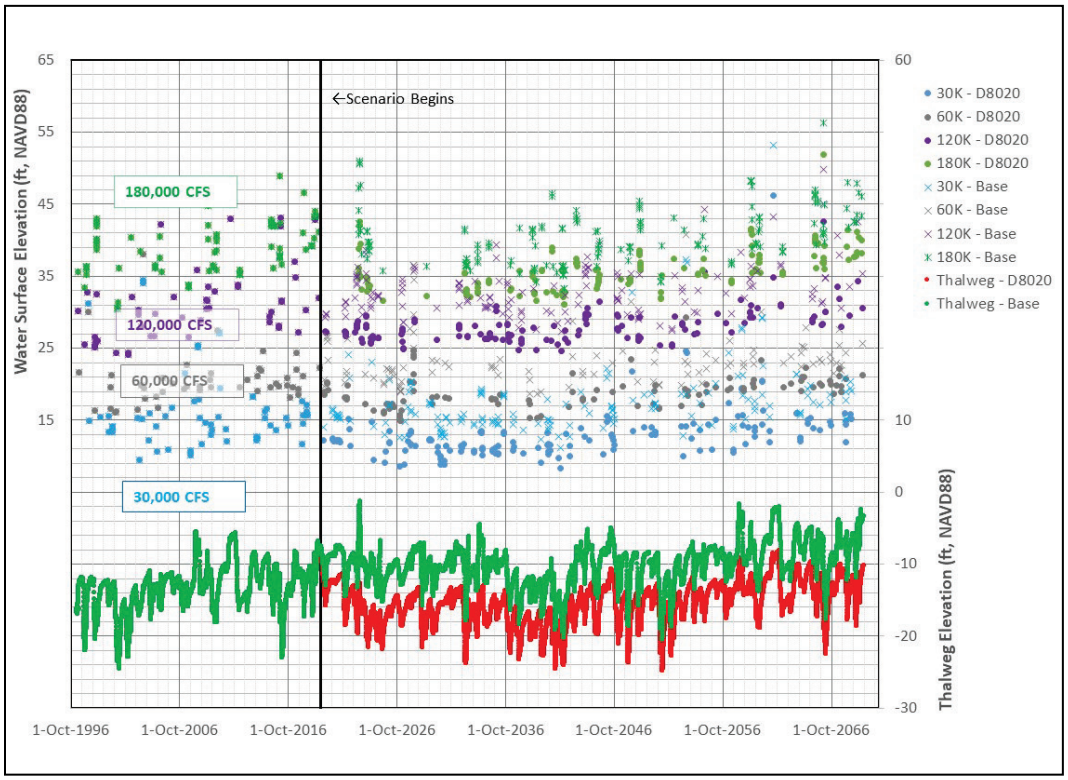


4.6.2 Red River at Madam Lee (RM 33)

Changing the latitude flow to 80/20 results in lower discharges and lower water-surface elevations in the Atchafalaya River at Simmesport. The decrease in downstream water-surface elevations results in degradation, relative to the Base, in the Red River where discharges are unchanged. Degradation of the bed elevation, relative to the Base, commences in the first year of the 50 yr simulation, averages 5.4 ft and varies between 0.0 and 11.5 ft. Scenario and Base water-surface elevations are characterized by significant scatter as a consequence of both oscillating bed elevations and variable backwater conditions from the Outflow Channel. On average,

scenario water-surface elevations are 4.5 ft lower than Base water-surface elevations due to Scenario D8020's lower tailwater elevations. Daily differences vary between 0.6 and 12.9 ft. Red River specific gage plots for scenarios and the Base can be used to compare water-surface elevations for specific discharges because they are from comparable events. That is to say, on a given date in the simulation hydrographs, scenario discharges are the same as the Base discharges at this location. Refer to Figure 39. The D8020 Scenario has lowered the base level in the Red and Black Rivers. The consequence is bed erosion up both rivers. How much erosion and whether it reaches the first hydraulic structure are beyond the scope of this model. Lindy Boggs Lock and Dam Number 1 is located on the Red River approximately 10 mi upstream from the Black River, and Jonesville Lock and Dam is located on the Black River approximately 25 mi upstream from the Red River. If Scenario D8020 is to receive more consideration, the following questions would need to be addressed: Are the foundations and the stilling basin designed for a lowering of the water surface? Can tows enter the lock with a lower water surface? Are there bridges or pipelines or other hydraulic structures crossing these rivers that might be jeopardized by erosion?

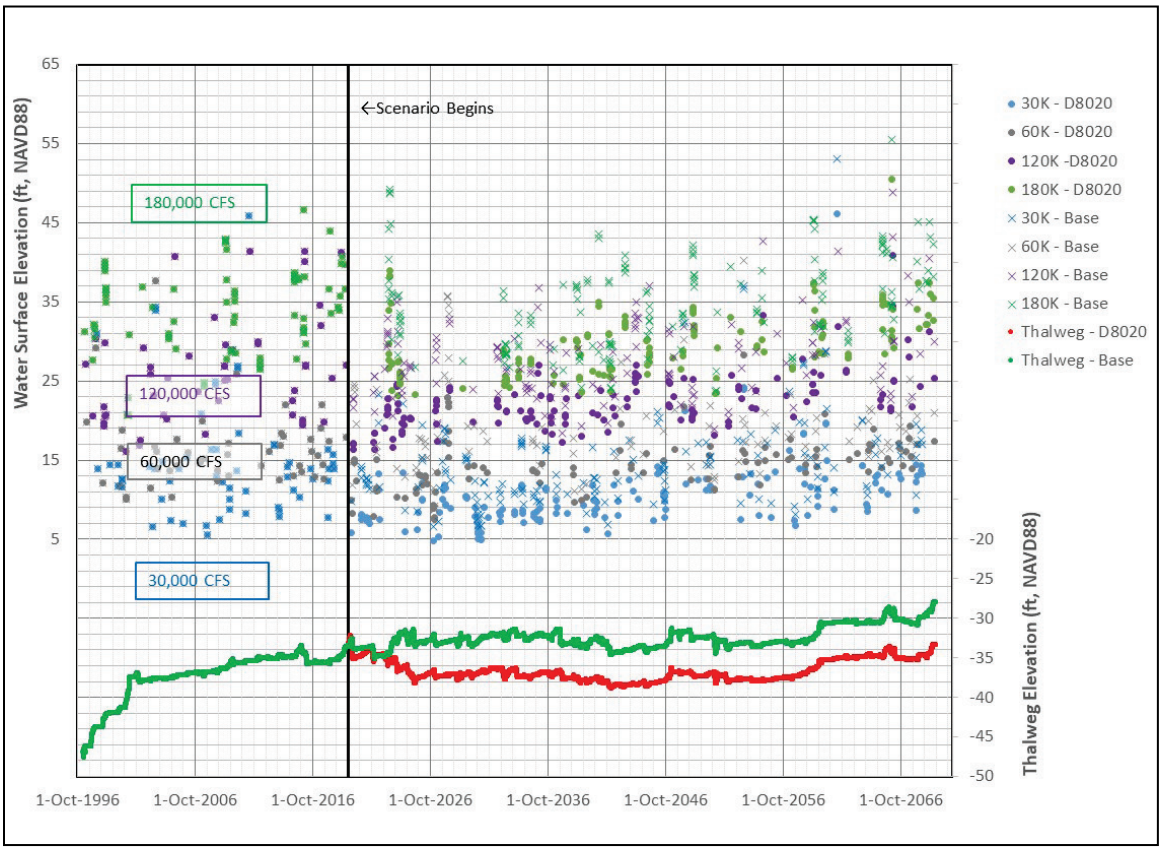
Figure 39. Difference calculated in specific gage at Red River RM 33 for Scenario D8020.



4.6.3 Red River above Outflow Channel (RM 10.6)

Consistent with the Red River at Madam Lee, degradation, relative to the Base, occurs with Scenario D8020. At RM 10.6 the degradation does not start until after year three of the 50 yr simulation. For the rest of the simulation, the scenario bed elevations are an average 4.4 ft lower than the Base. Daily differences vary between 2.5 and 6.1 ft. Bed elevations at RM 10.6 do not have the annual oscillations calculated at the model's upstream boundary at Madam Lee. However, scenario water-surface elevations at RM 10.6 show the same characteristics as the Madam Lee water-surface elevations due to variable backwater effects (Figure 40).

Figure 40. Difference calculated in specific gage at Red River RM 10.6 for Scenario D8020.

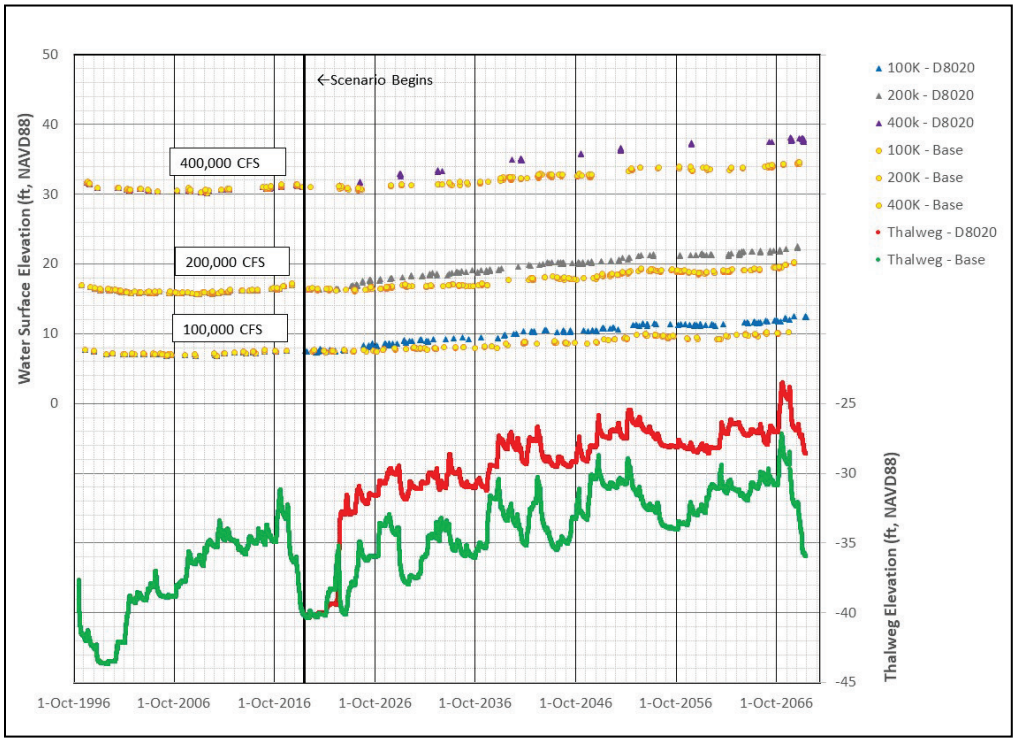


4.6.4 Simmesport (RM -4.34)

After approximately 3 yr, the scenario channel-bed elevation rapidly aggrades by approximately 4 ft, relative to the Base, and maintains an average 4.6 ft higher bed elevation throughout the rest of the 50 yr simulation. Rapid aggradation is calculated at the beginning of the 50 yr

simulation for both the Base Condition and Scenario D80/20, as the bed rebounds from the significant degradation that occurred during the high flow years 2018 and 2019. The daily bed elevation difference varies between 2.1 and 8.1 ft. This aggradation is a consequence of the oversized channel at Simmesport created by the drastic reduction in discharge and the increase is sediment supply from the Red River associated with the degradation induced by the long-term decrease in downstream water-surface elevation. Scenario water-surface elevations also increase relative to the Base water-surface elevations. After 50 yr, scenario water-surface elevations are approximately 2 ft higher than the base for 200,000 cfs and 100,000 cfs. Scenario water-surface elevations increased approximately 4 ft for a discharge of 400,000 cfs. No scenario points were calculated for the higher 600,000 cfs specific discharge (Figure 41). Remember that the scenario and Base condition specific discharges at Simmesport do not correspond to the same hydrologic event. For the same event, Scenario D8020 discharges and water-surface elevations are less than those for the Base.

Figure 41. Difference calculated in specific gage at Simmesport RM -4.34 for Scenario D8020.

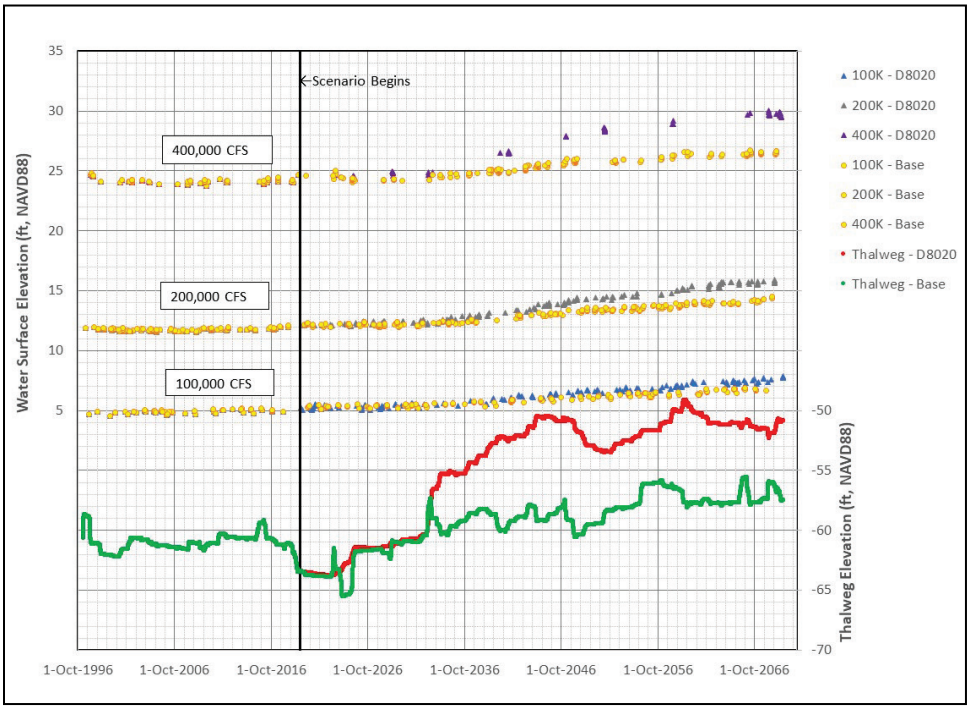


4.6.5 Melville (RM -29.6)

After approximately 10 yr, aggradation commences at Melville. Although the decline in discharge with the D8020 Scenario commences at the beginning of the 50 yr simulation, the deposition that accompanies the reduction in transport capacity is not immediate at Melville. Reducing sediment transport capacity lengthens the time required to move the sediment input from the Red River downstream. After another 10 yr, the aggradation trend, relative to the Base, stabilizes between 3.6 and 9.3 ft, with an average of 6.5 ft, for the remainder of the 50 yr simulation. Scenario water-surface elevations begin to increase coincident with the commencement of aggradation. At the end of 50 yr, scenario water-surface elevations are approximately 1 ft higher at 100,000 cfs, 2 ft higher at 200,000 cfs, and 3 ft higher at 400,000 cfs. No scenario points were captured for the 600,000 cfs specific discharge (Figure 42).

Consistent with Simmesport, the scenario and Base specific discharges at Melville do not correspond to the same hydrologic event. For the same event, Scenario D8020 discharges and water-surface elevations are less than those for the Base.

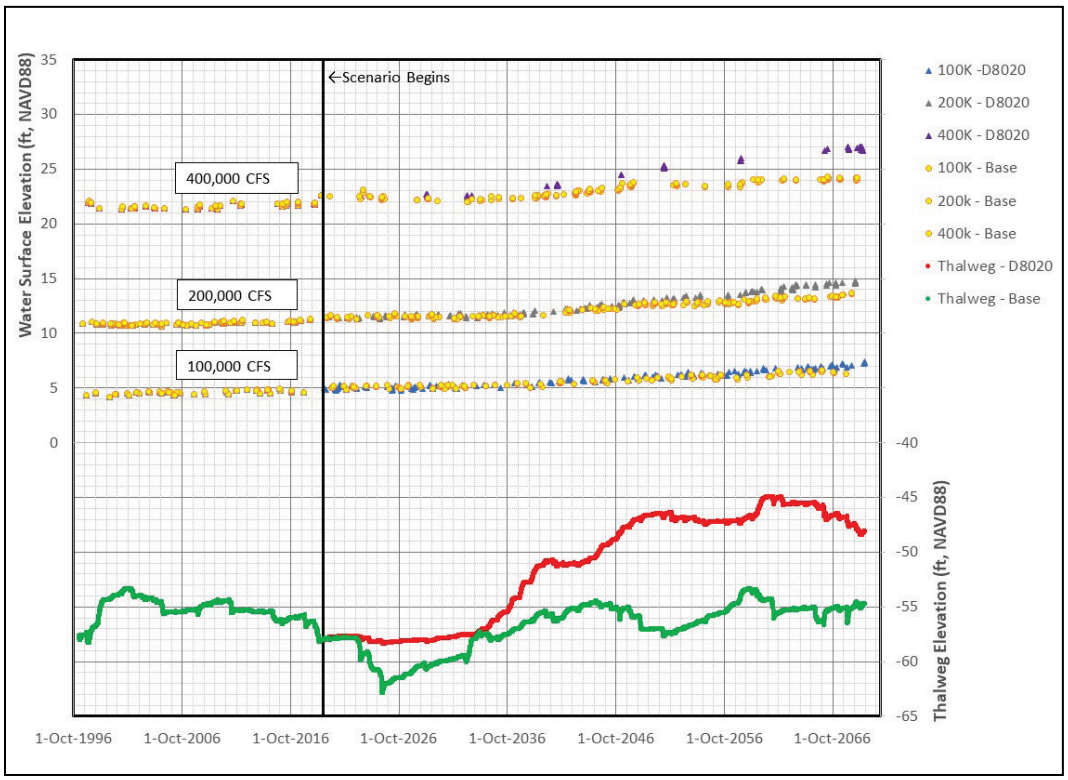
Figure 42. Difference calculated in specific gage at Melville RM -29.6 for Scenario D8020.



4.6.6 Krotz Springs (RM -39.59)

Thalweg and water-surface elevation comparisons between Scenario D8020 and the Base are similar to those at Melville. At Krotz Springs, after 20 yr the aggradation trend, relative to the Base, stabilizes between 3.9 and 11 ft, with an average of 7.9 ft, for the remainder of the 50 yr simulation (Figure 43).

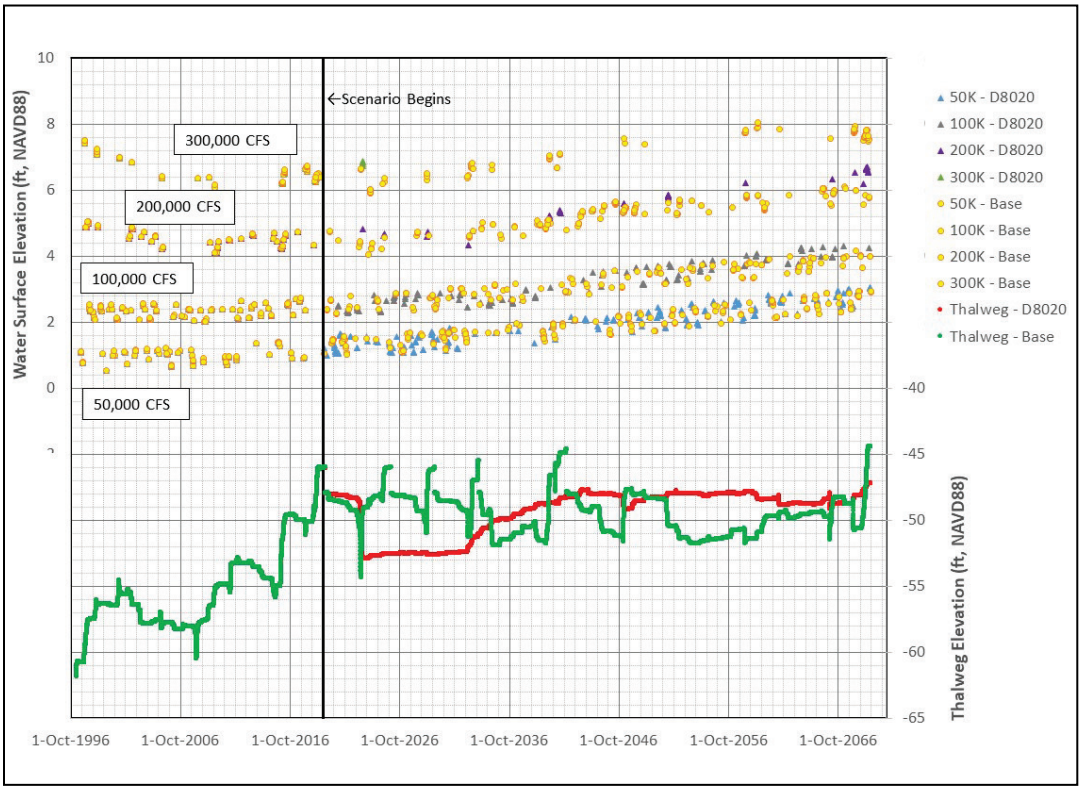
Figure 43. Difference calculated in specific gage at Krotz Springs RM -39.59 for Scenario D8020.



4.6.7 Morgan City (RM -121.2)

Differences between Scenario D8020 and the Base condition are negligible at Morgan City. The bed changes are more stable with the scenario because dredging is reduced with the lower discharges associated with Scenario D8020. Scenario water-surface elevations are approximately 0.5 ft higher than the Base condition for a specific discharge of 200,000 cfs. Lower discharges exhibit no discernable differences between scenario and Base condition (Figure 44).

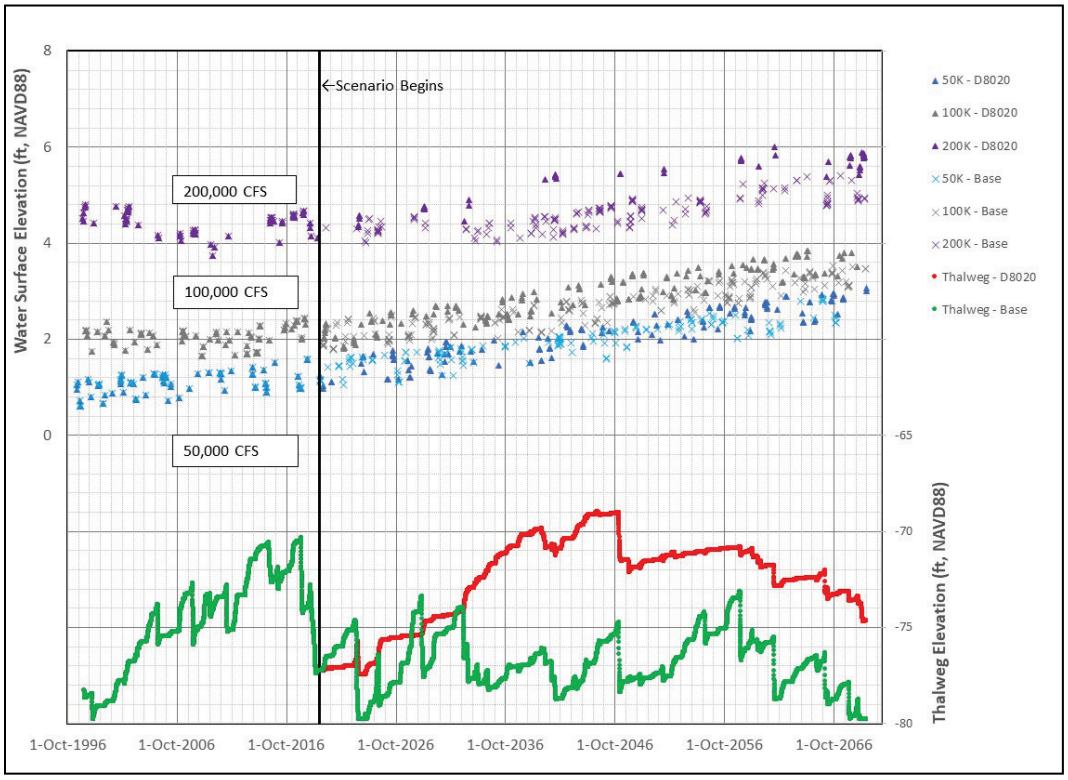
Figure 44. Difference calculated in specific gage at Morgan City RM -121.2 for Scenario D8020.



4.6.8 Calumet (RM -115.7)

After approximately 13 yr, a significant aggradation trend commences at Calumet relative to the Base. Scenario bed elevations are between 2 and 6 ft higher than the Base. Scenario water-surface elevations are generally higher than Base water-surface elevations with a specific discharge of 50,000 cfs, approximately 1 ft higher at 100,000 cfs, and 1 to 2 ft higher at 200,000 cfs. No scenario points were captured for the 300,000 cfs specific discharge (Figure 45).

Figure 45. Difference calculated in specific gage at Calumet RM -115.7 for Scenario D8020.



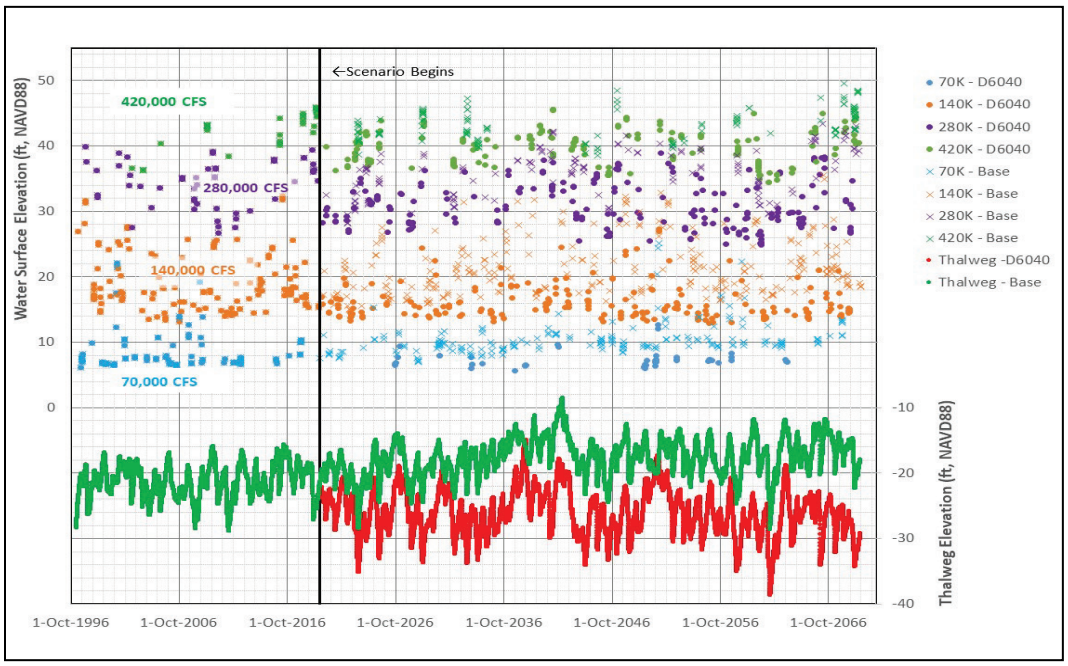
4.7 Scenario 9 (D6040)

This scenario changes the flow distribution at the ORCC so that 60% of the latitude flow at Tarbert Landing and Simmesport goes down the Mississippi River. The effect on the Atchafalaya River is significant as the flow is increased by a third, which causes an even larger increase in sediment transport potential. The increase in the Atchafalaya River discharge will also increase the channel-forming discharge, which will increase the potential for channel widening. There are already some reaches of the Atchafalaya River that are experiencing bank erosion issues. These can be expected to worsen with Scenario D6040. Other reaches, without bank protection, would be subject to increased potential for bank failure. The OMAR HEC-6T model does not account for change in cross-section area due to bank erosion. If consideration of Scenario D6040 is advanced further, the bank erosion issue needs to be addressed in an additional study. Bank erosion issues were addressed in Volume 9 of the OMAR Assessment (Harris and Dahl 2022).

4.7.1 Outflow Channel (RM 4.33)

Annual bed oscillations occur with both the Base and Scenario D6040. These oscillations tend to mask long-term aggradation and degradation trends in the Outflow Channel. However, it is apparent that significant degradation of the Outflow Channel commences with the initiation of scenario operation and increases during the 50 yr simulation. The maximum daily scenario degradation, relative to the Base, was 15.3 ft, and the average degradation over the 50 yr was 8.5 ft. Due to the oscillating bed and variable backwater conditions created by the Red River, there is significant scatter in the calculated water-surface elevations. On average, the daily scenario water-surface elevations increased approximately 2.2 ft relative to the Base when calculated on a coincident daily basis. However, the specific gage plots show an approximate average decrease of approximately 3 ft in scenario water-surface elevation relative to the base when calculated on a specific discharge basis (Figure 46).

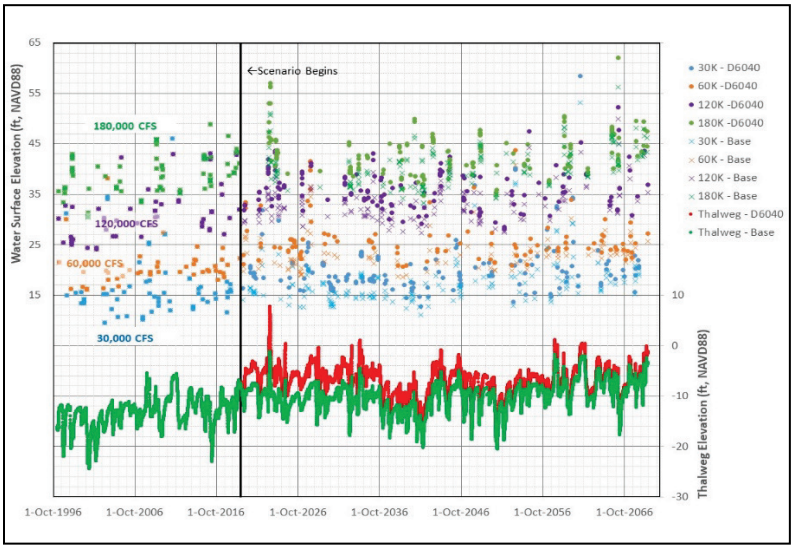
Figure 46. Calculated difference in specific gage at Outflow Channel RM 4.33 for Scenario D6040.



4.7.2 Red River at Madam Lee (RM 33)

Changing the latitude flow to 60/40 results in higher discharges and higher water-surface elevations in the Atchafalaya River at Simmesport. This increase in water-surface elevations result in aggradation, relative to the Base, in the Red River, where discharges are unchanged. Aggradation, relative to the Base, commences in the first year of the 50 yr simulation. This aggradation is significant initially, up to 9.0 ft in the fourth year. However, the scenario aggradation, relative to the Base, decreases to 2.1 ft at the end of the 50 yr simulation. The 50 yr average aggradation, relative to the Base, was 3.0 ft. On average, scenario water-surface elevations were 2.6 ft higher than Base water-surface elevations. The higher scenario water-surface elevation was a consequence of both higher bed elevations and higher backwater elevations. Significant scatter in water-surface elevation calculations occur due to the oscillating bed and variable backwater conditions from the Outflow Channel. Red River specific gage plots for scenarios and the Base can be used to compare water-surface elevations for specific discharges because they are from comparable events. That is to say, on a given date in the simulation hydrographs, scenario discharges are the same as the Base discharges here in the Red River (Figure 47). The aggradation effects that would occur upstream from the OMAR HEC-6T model boundary into the Black River and up to Lindy Boggs Lock and Dam on the Red River should be addressed further with additional study should this scenario receive additional consideration.

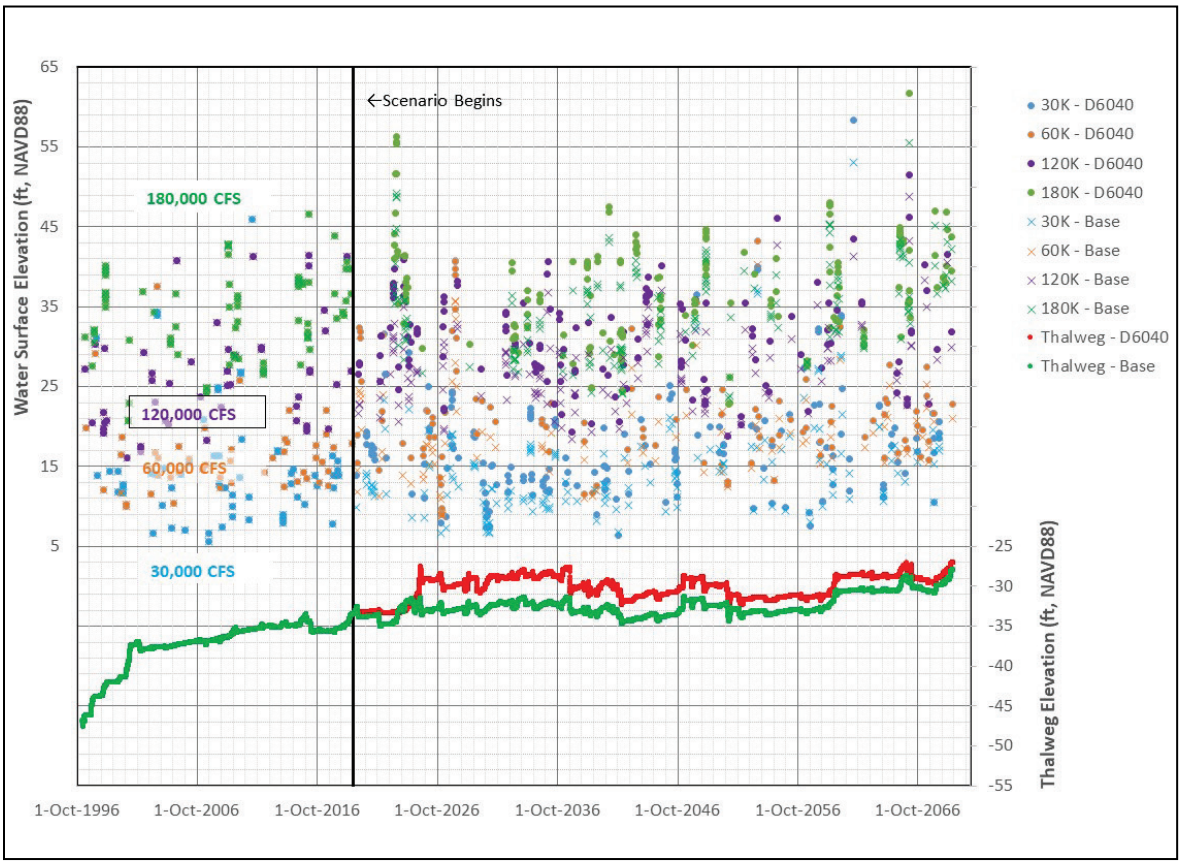
Figure 47. Calculated difference in specific gage at Red River RM 33 for Scenario D6040.



4.7.3 Red River above Outflow Channel (RM 10.6)

Consistent with the Red River at Madam Lee, aggradation relative to the Base occurs with Scenario D6040. At RM 10.6, the aggradation does not start until after year five of the 50 yr simulation. The 50 yr average aggradation, relative to the Base, was 2.4 ft. On average, scenario water-surface elevations were 2.6 ft higher than Base water-surface elevations. Bed elevations at RM 10.6 do not have the annual oscillations calculated upstream at Madam Lee. However, scenario water-surface elevations at RM 10.6 show the same scatter as the Madam Lee water-surface elevations due to variable backwater effects (Figure 48).

Figure 48. Calculated difference in specific gage at Red River RM 10.6 for Scenario D6040.



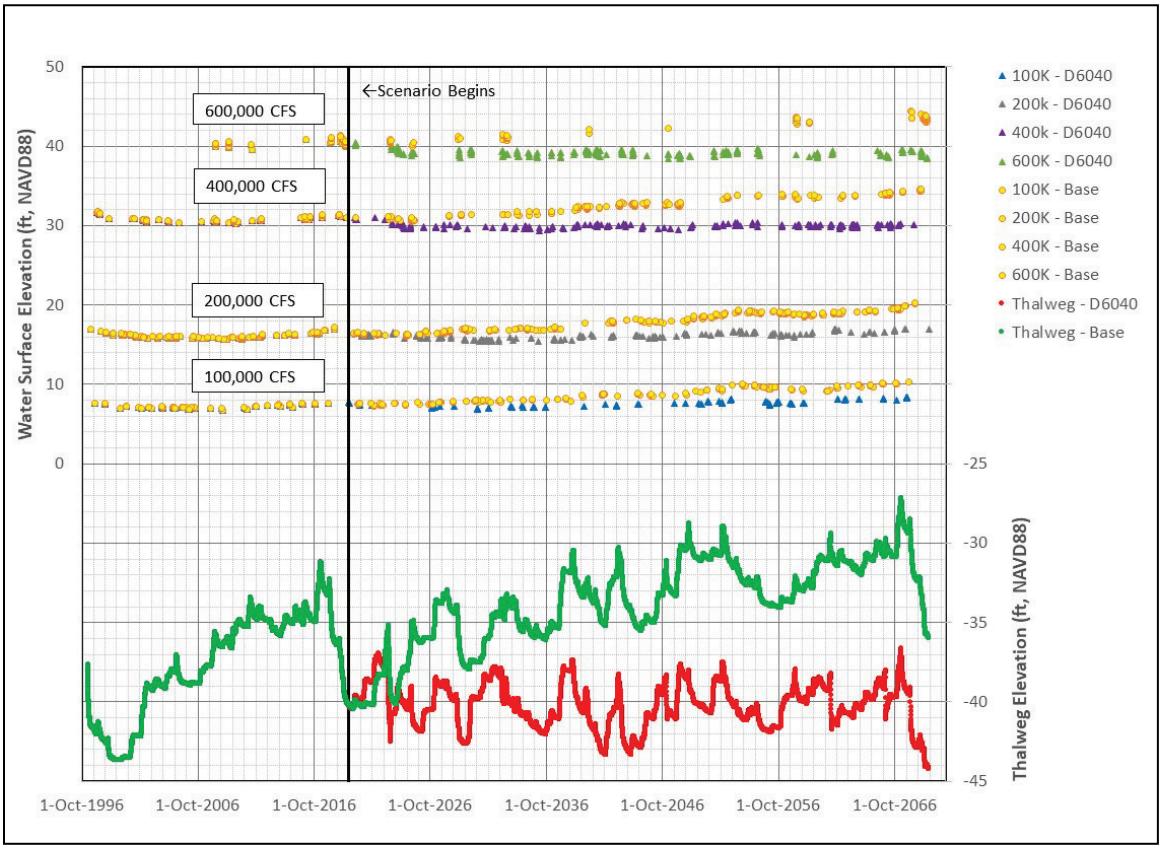
4.7.4 Simmesport (RM -4.34)

After approximately 3 yr, the Scenario D6040 channel bed begins to degrade, relative to the Base. The degradation below the Base increases as the simulation progresses to a maximum of 11.5 ft toward the end of the

50 yr. The average 50 yr degradation was 6.5 ft. This degradation is a consequence of increased discharge from the Outflow Channel. Scenario water-surface elevations for the specific discharges progressively decrease, relative to the Base, during the 50 yr simulation. After 50 yr, scenario-specific discharge water-surface elevations are approximately 2 ft lower than the base for a specific discharge of 200,000 cfs, 3 ft lower for 100,000 cfs, 4 ft lower for 400,000 cfs, and 5 ft lower for 600,000 cfs (Figure 49).

The scenario and Base specific discharges at Simmesport do not correspond to the same hydrologic event. On average, the daily scenario water-surface elevations increased approximately 2.8 ft relative to the Base when calculated on a coincident daily basis. Consistent with the Outflow Channel, the overall response to the increased flow with Scenario D6040 at Simmesport is bed degradation, with lower water-surface elevations for specific discharges but higher water-surface elevations at any given time.

Figure 49. Calculated difference in specific gage at Simmesport RM -4.34 for D6040.



4.7.5 Melville (RM -29.6) and Krotz Springs (RM -39.59)

Bed and water-surface responses for Scenario D6040 at Melville and Krotz Springs are similar to those at Simmesport. The 50 yr average scenario bed degradation, relative to the Base, was 5.8 ft at both Melville and Krotz Springs. On average, the daily scenario water-surface elevations increased approximately 3.1 ft relative to the Base when calculated on a coincident daily basis at Melville and 3.0 ft at Krotz Springs. The overall response to the increased flow with Scenario D6040 at Melville and Krotz Springs is bed degradation, with lower water-surface elevations for specific discharges, but higher water-surface elevations over time (Figure 50 and Figure 51).

Figure 50. Calculated difference in specific gage at Melville RM -29.6 for Scenario D6040.

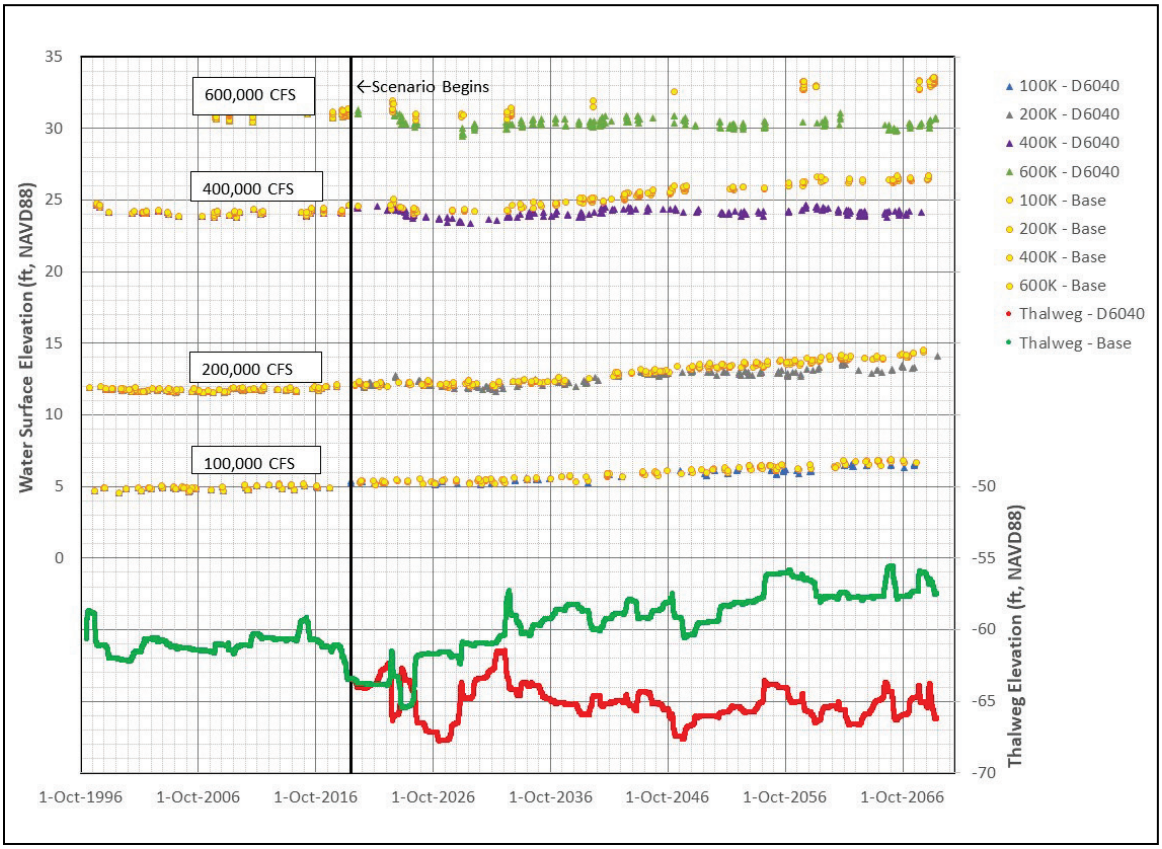
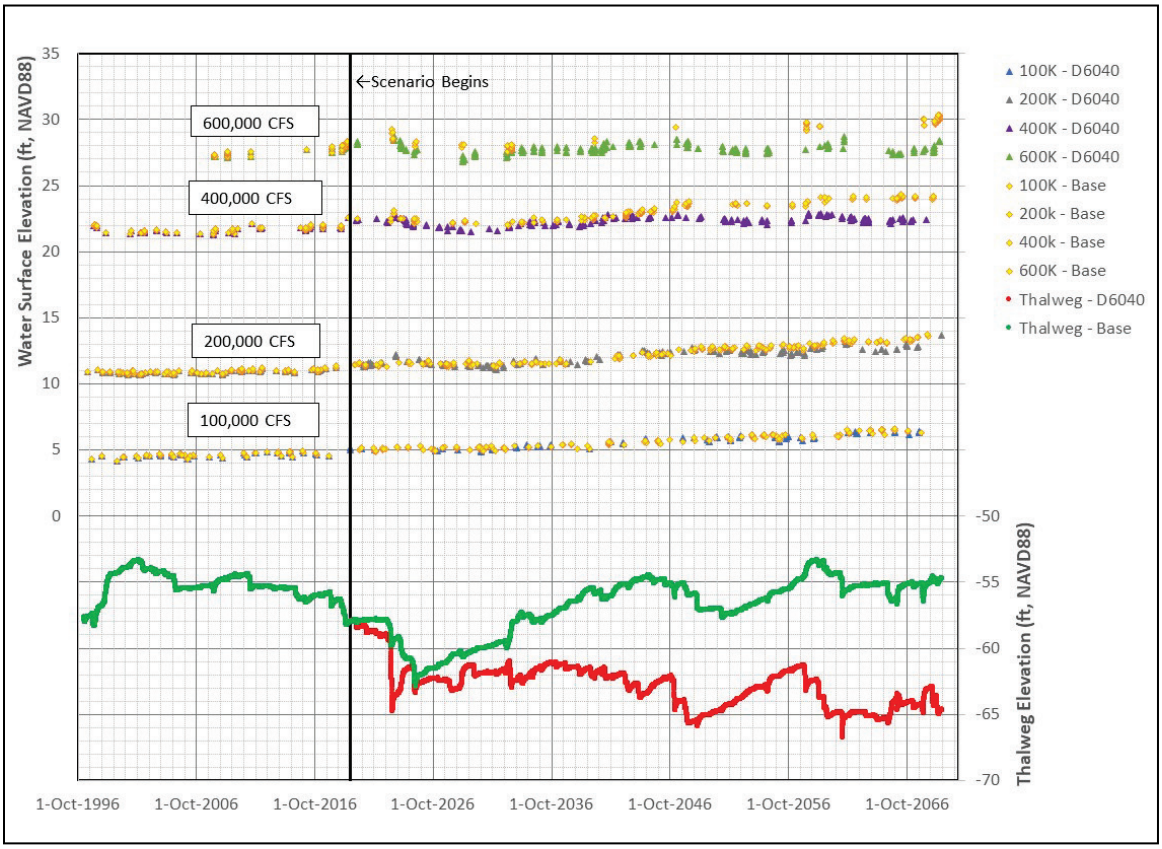


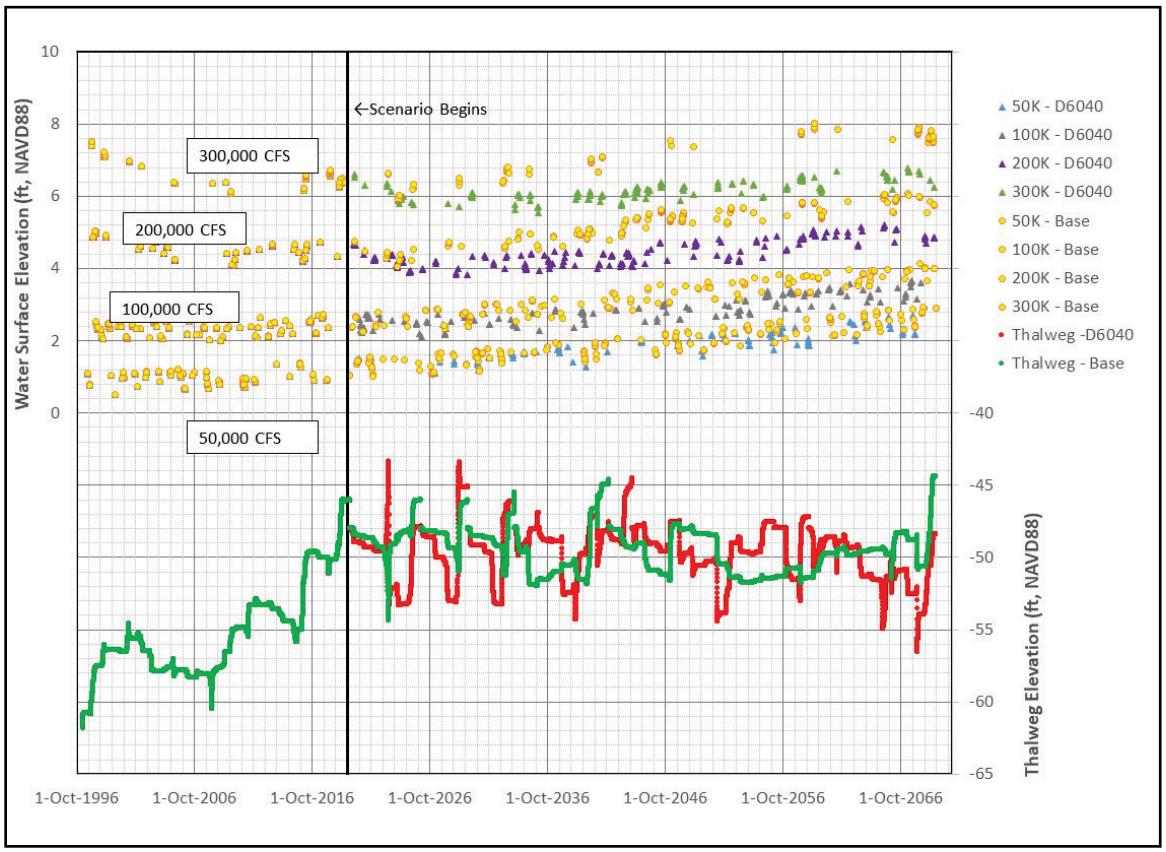
Figure 51. Calculated difference in specific gage at Krotz Springs RM -39.59 for Scenario D6040.



4.7.6 Morgan City (RM -121.2)

Overall bed differences between Scenario D6040 and the Base appear to be negligible at Morgan City. The average 50 yr scenario degradation, with respect to the Base, was 0.2 ft. Dredging appears to occur more frequently with Scenario D6040. Water-surface elevations for the scenario-specific discharges decrease, with respect to the Base, during the 50 yr simulation. At 50,000 cfs, the difference is less than 0.5 ft, at 100,000 cfs the difference is between 0.5 and 1 ft, at 200,000 cfs approximately 1 ft, and at 300,000 cfs, approximately 1.5 ft (Figure 52). However, 50 yr coincident scenario water-surface elevations were 0.4 ft higher than the Base.

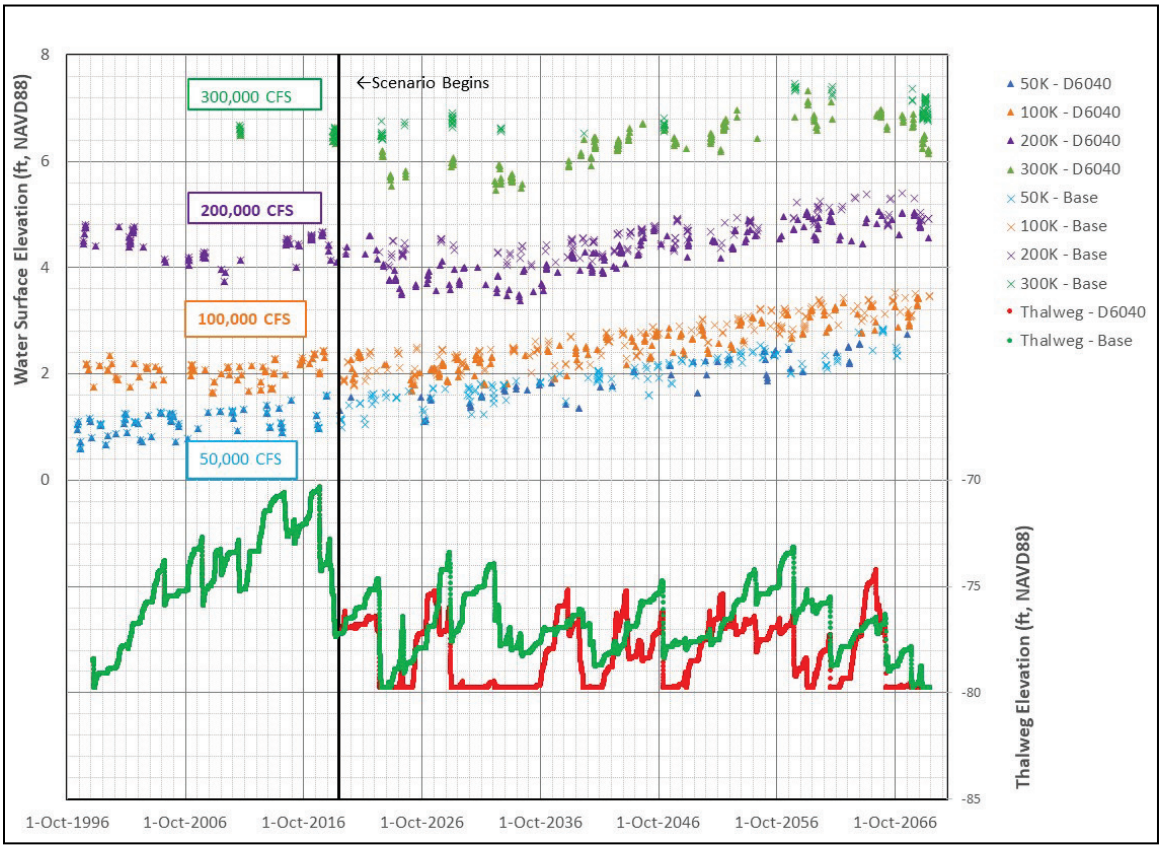
Figure 52. Calculated difference in specific gage at Morgan City RM -121.2 for Scenario D6040.



4.7.7 Calumet (RM -115.7)

There is a non-erosive bed at elevation -80 in the numerical model. This erosion limit is reached more frequently during the 50 yr simulation with Scenario D6040 than with the Base. Discharge increases with Scenario D6040 induce more degradation than occurs with the Base. Scenario water-surface elevations are generally approximately 0.5 ft lower than Base water-surface elevations for specific discharges of 200,000 cfs and higher (Figure 53). However, 50 yr coincident scenario water-surface elevations were 1.2 ft higher than the Base.

Figure 53. Calculated difference in specific gage at Calumet RM -115.7 for Scenario D6040.



4.8 Dredge Disposal Scenario

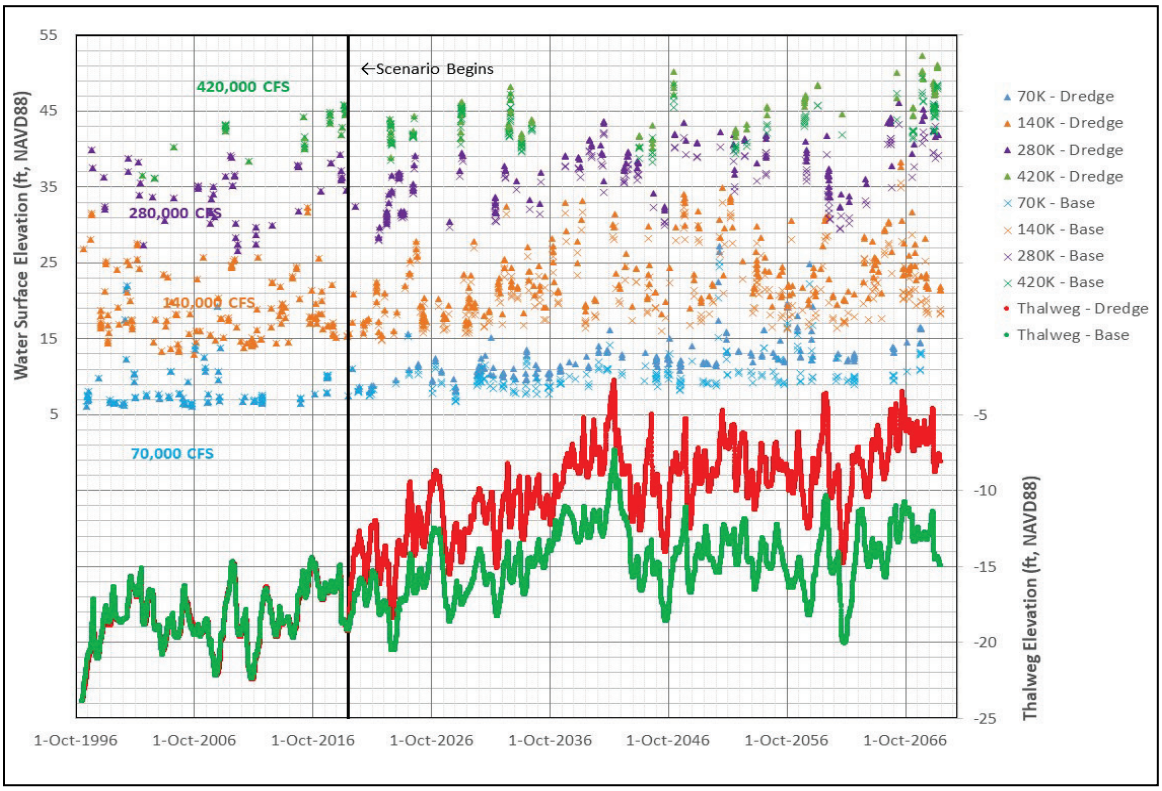
The Dredge Disposal Scenario has the same hydrograph as the Base Condition. Sediment dredged from the Mississippi River was deposited into the Outflow Channel, downstream from RM 4.33, daily at a rate of 5,750 cy per day (2,100,000 cy/yr). Since the Dredge Disposal Scenario used the same daily discharges as the Base, specific gage plots are useful to compare both specific discharges and event discharges. That is to say, on a given date in the simulation hydrographs, scenario discharges are the same as the Base discharges.

4.8.1 Outflow Channel (RM 3.76)

The Outflow Channel specific gage calculations were made at RM 3.76, which is downstream from the dredge material disposal area, rather than at RM 4.33 where the rest of the scenario specific gage plots were calculated. The scenario bed aggrades 3.3 ft, relative to the Base, during the first year of dredge disposal, increasing to 6.8 ft after 50 yr. The

average bed increase over the 50 yr simulation was 4.8 ft, and the maximum was 8.7 ft. The dredge scenario exhibits the same annual bed oscillations as the base condition. Water-surface elevations also show considerable scatter in the specific gage plot. The average water-surface increase, relative to the Base, was 1.9 ft, and the maximum increase was 3.9 ft (Figure 54).

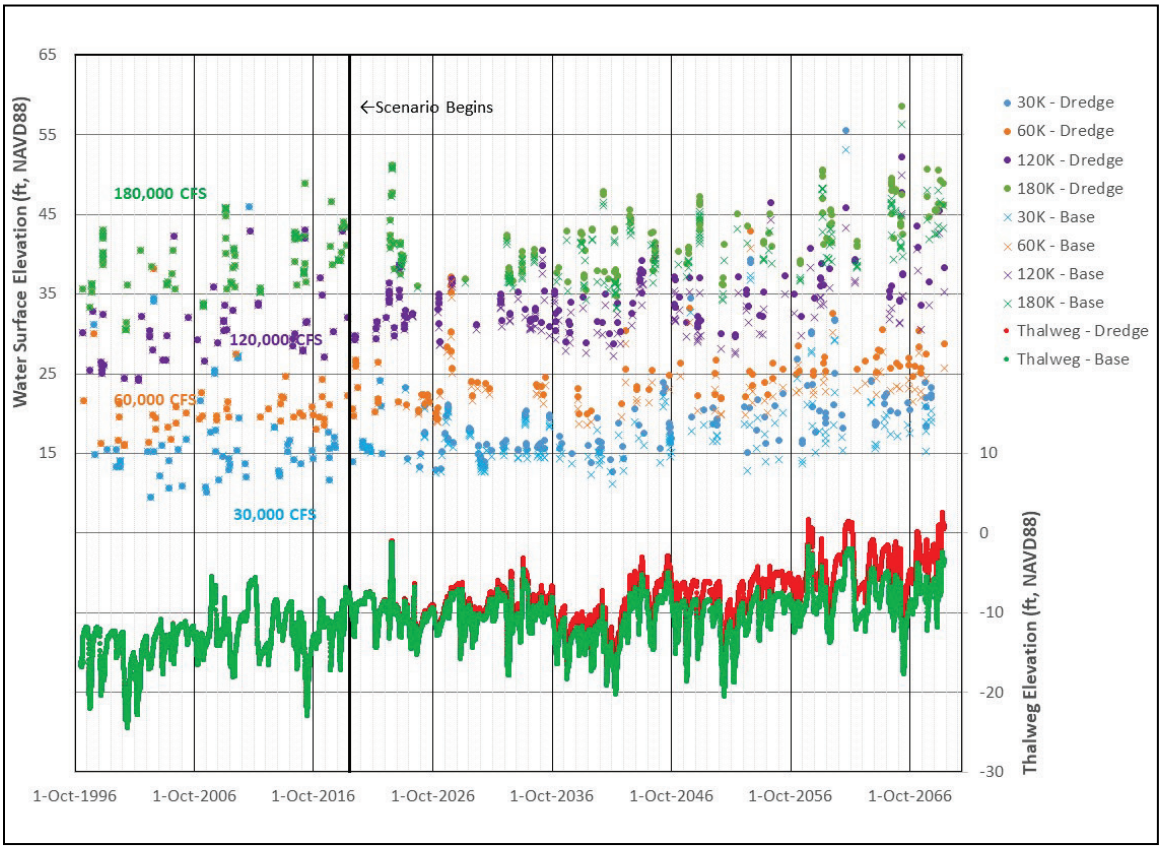
Figure 54. Calculated difference in specific gage at Outflow Channel RM 3.76 for Dredge Disposal Scenario.



4.8.2 Red River at Madam Lee (RM 33)

After approximately 10 yr, the scenario channel bed at the model's upstream boundary begins to aggrade, relative to the Base. After 50 yr, the scenario channel bed aggraded 4.3 ft, relative to the Base. The average increase in bed elevation with the Dredge Disposal Scenario was 2.1 ft and the maximum 5.1 ft. The average increase in water-surface elevations for the scenario was 1.6 ft, and the maximum increase was 3.3 ft (Figure 55).

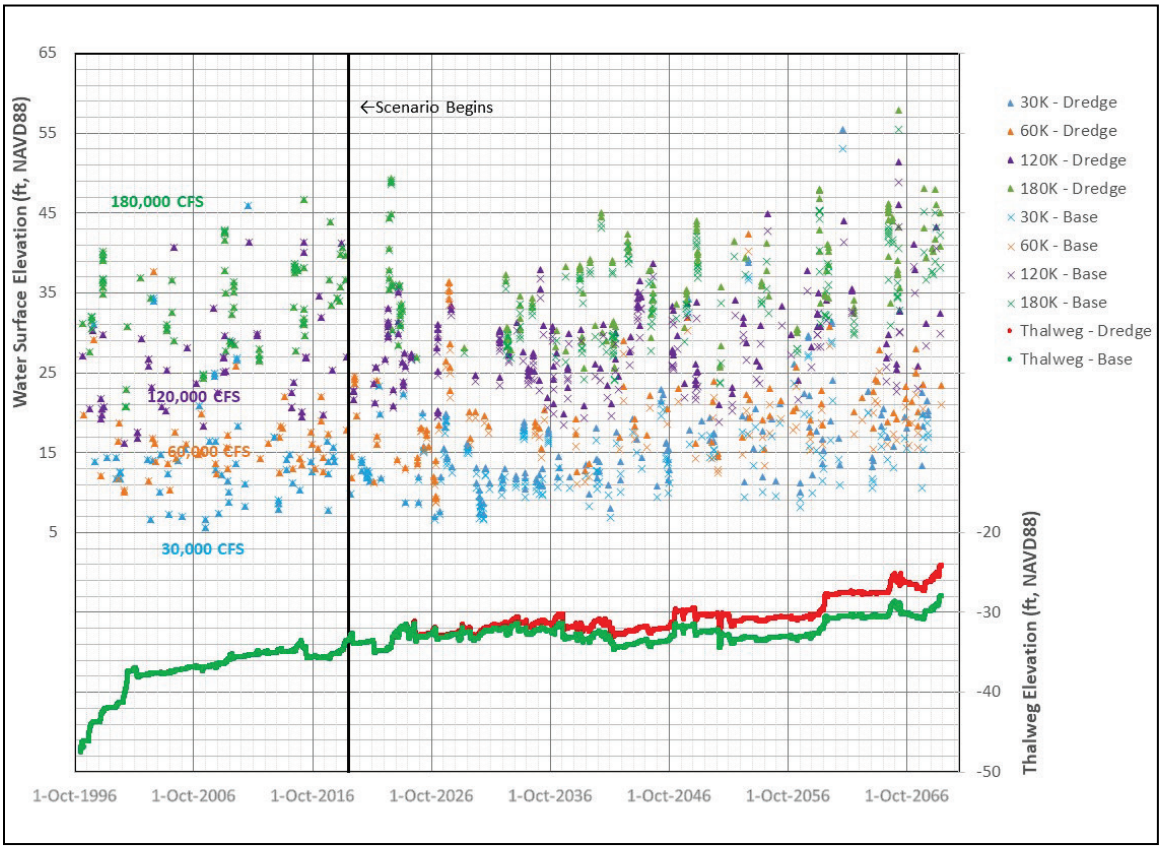
Figure 55. Calculated difference in specific gage at Red River RM 33 for Dredge Disposal Scenario.



4.8.3 Red River above Outlet Channel (RM 10.6)

Consistent with Madam Lee upstream, after approximately 10 yr, the Red River channel bed just upstream from the Outflow Channel begins to aggrade, relative to the Base. The aggradation increases to 3.8 ft after 50 yr. Water-surface elevations at the end of 50 yr increase by 2.3 ft, relative to the Base. The downstream Red River station does not exhibit the annual bed oscillations that occur at the upstream station, but it does exhibit the same water-surface scatter (Figure 56).

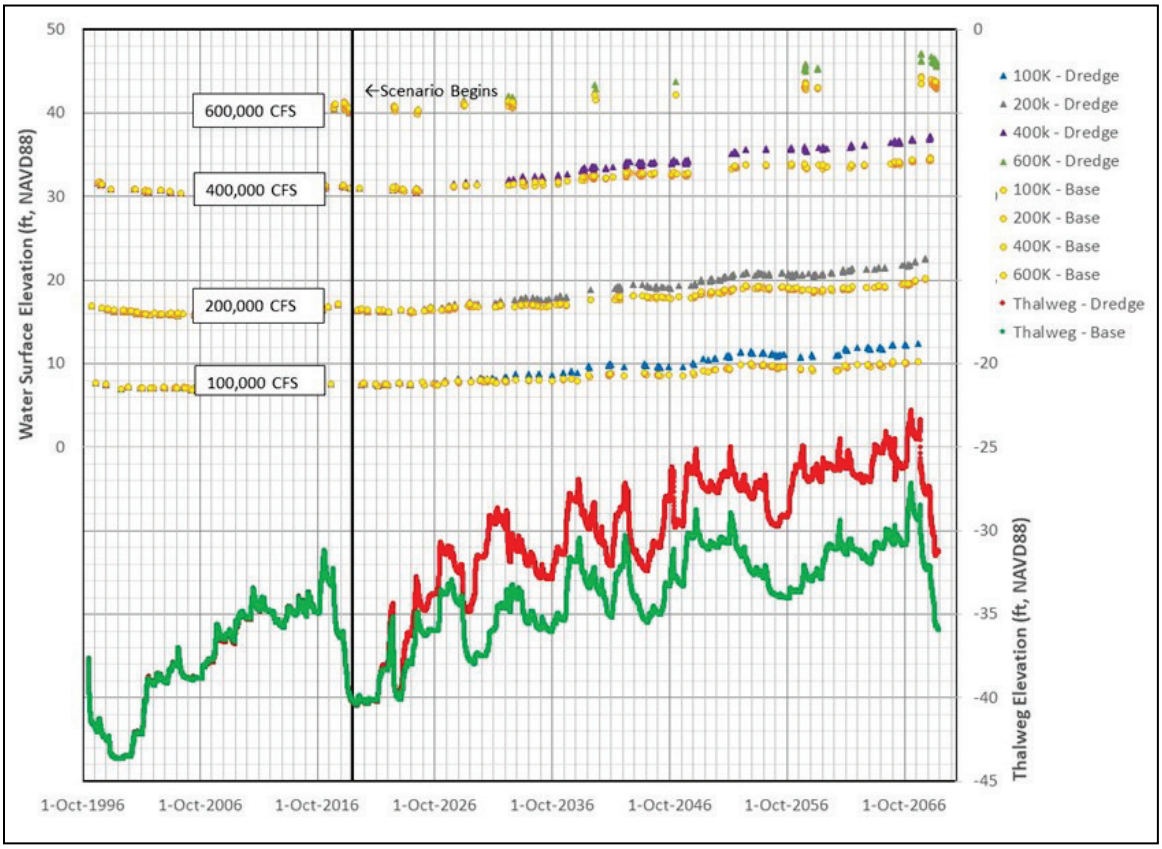
Figure 56. Calculated difference in specific gage at Red River RM 10.6 for Dredge Disposal Scenario.



4.8.4 Simmesport (RM -4.34)

After approximately 5 yr, the channel bed at Simmesport begins to aggrade. The model shows a steady increase in bed elevation to 4.7 ft after 50 yr. There was a corresponding increase in water-surface elevations so that after 50 yrs water-surface elevations were about 2.0 ft higher than the Base condition for all discharges (Figure 57).

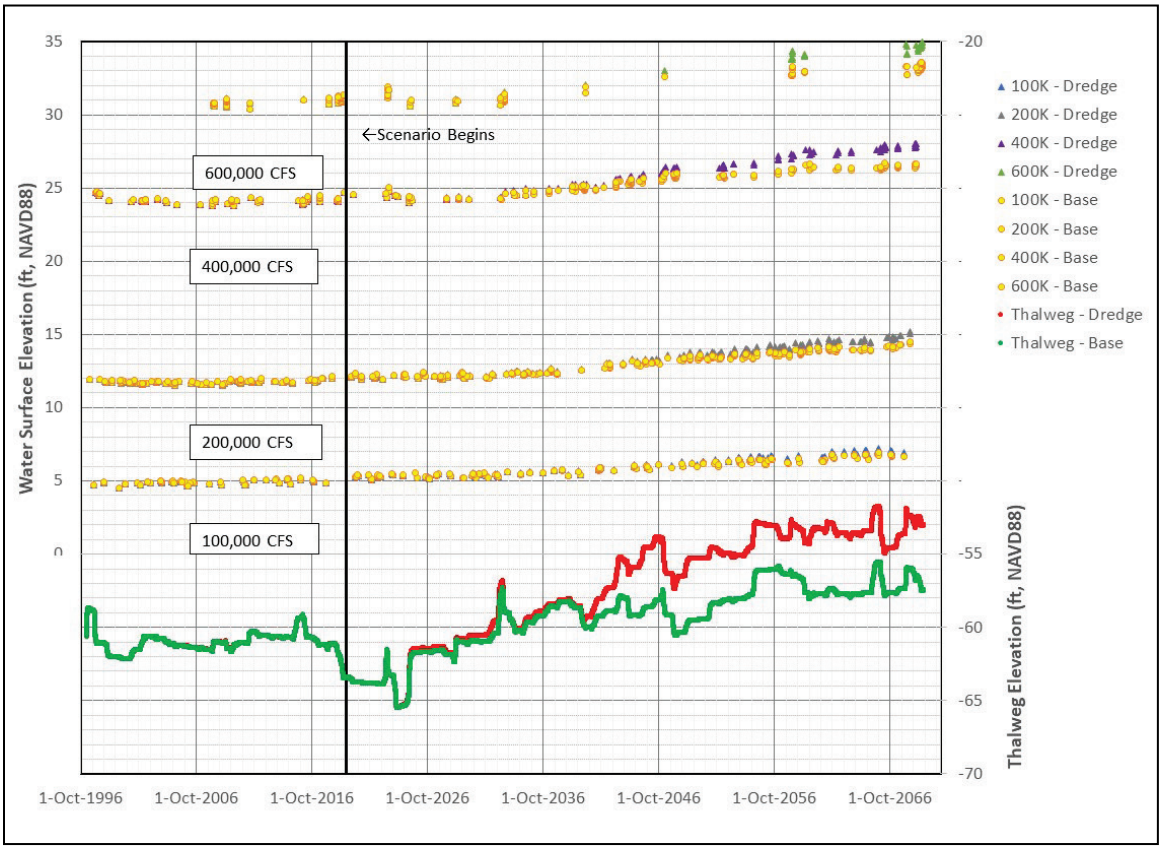
Figure 57. Calculated difference in specific gage at Simmesport RM -4.34 for Dredge Disposal Scenario.



4.8.5 Melville (RM-29.6)

It takes approximately 21 yr for effects of dredge disposal in the Outflow Channel to induce aggradation at Melville. Then, within 6 yr, the bed aggrades 4.3 ft, relative to the Base, where it essentially stabilizes. At the end of the 50 yr simulation, the scenario bed elevation is 4.5 ft higher than the Base. Water-surface elevations at Melville begin to increase at the same time as aggradation commences. At the end of 50 yr, water-surface elevations are approximately 0.5 ft higher for a discharge of 100,000 cfs, increasing to approximately 1.5 ft higher for the maximum specific discharge of 600,000 cfs. The average increase for all discharges was 0.3 ft (Figure 58).

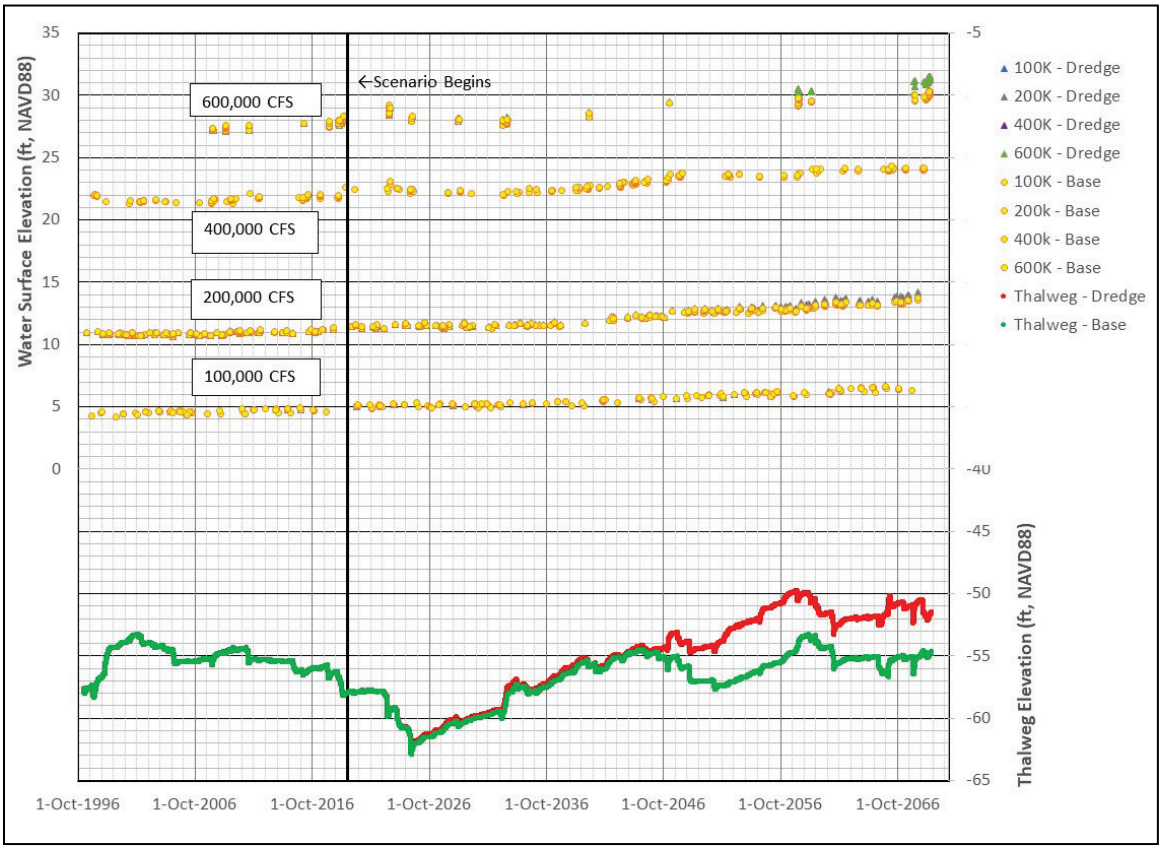
Figure 58. Calculated difference in specific gage at Melville RM -29.6 for Dredge Disposal Scenario.



4.8.6 Krotz Springs (RM -39.59)

It takes approximately 27 yr for effects of dredge disposal in the Outflow Channel to induce additional aggradation above the Base at Krotz Springs. Then, within 6 yr, the bed aggrades 3.2 ft more than the Base, and thereafter the bed aggradation oscillates between 2 and 5 ft. Average water-surface elevation increases are between 0 and 1.7 ft (Figure 59).

Figure 59. Calculated difference in specific gage at Krotz Springs RM -39.59 for Dredge Disposal Scenario.



4.8.7 Calumet and Morgan City

The model shows neither aggradation nor degradation in the channel bed, and the water-surface elevations very closely match the base conditions. The results appear consistent for all discharges (Figure 60 and Figure 61).

Figure 60. Calculated difference in specific gage at Morgan City RM -121.2 for Dredge Disposal Scenario.

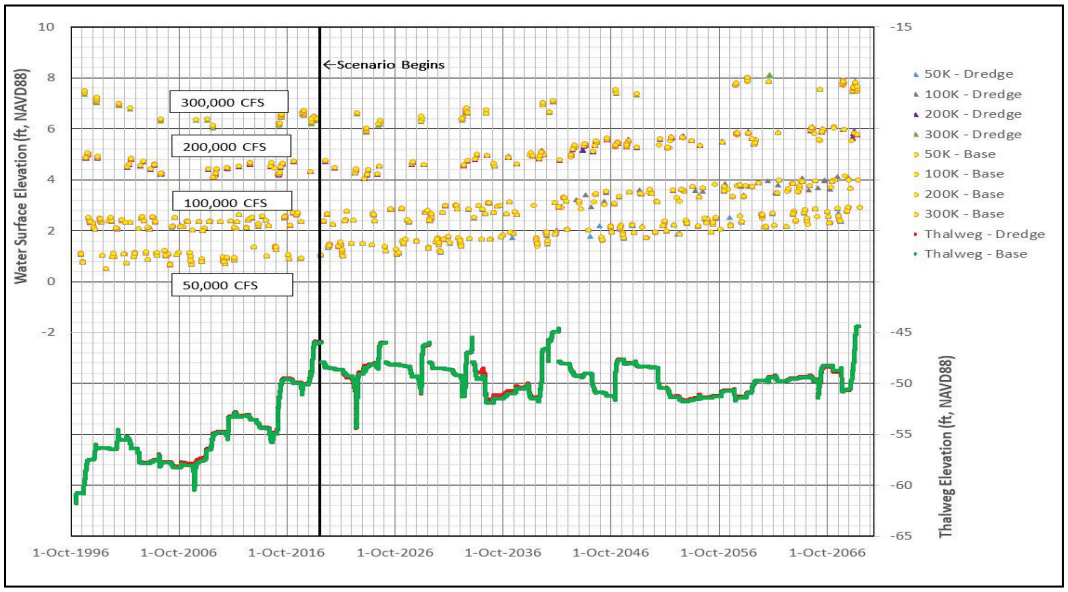
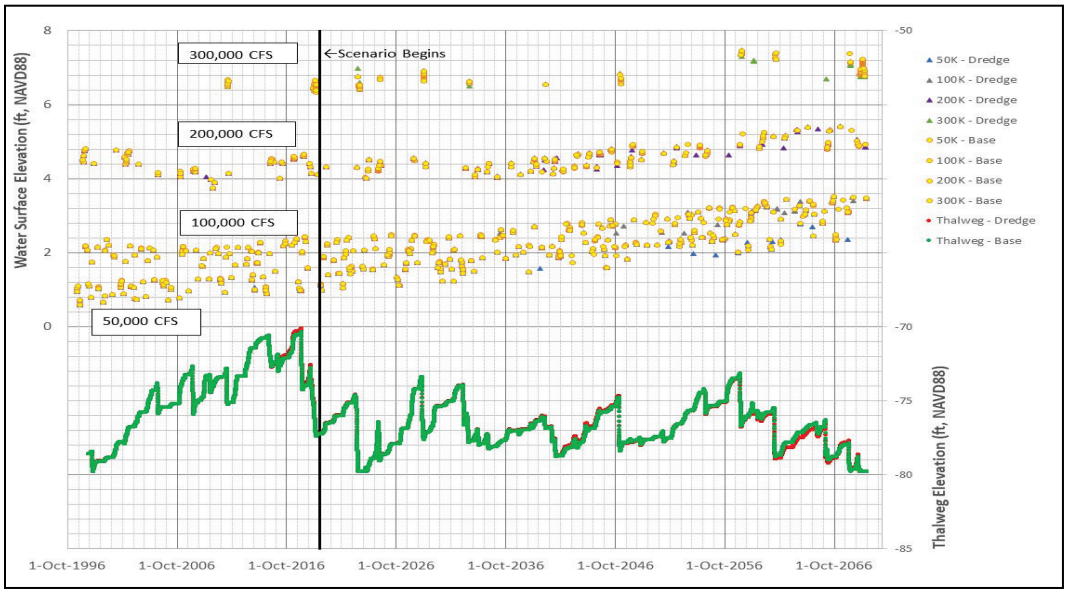


Figure 61. Calculated difference in specific gage at Calumet RM -115.7 for Dredge Disposal Scenario.



4.9 Differences in water-surface elevations

The specific gage analyses provided useful information with respect to the sedimentation effects of the various scenarios to water-surface elevations for specific discharges. However, to effectively assess both the effects of sedimentation and the changes in flow distribution at ORCC on water-surface elevations, it is necessary to compare calculated water-surface

elevations for the same date (event) on the hydrograph. To this end, the differences between scenario and base condition water-surface elevations at Simmesport and Morgan City for each day during the last year of the 50 yr future hydrograph were calculated and are shown in Figure 62 through Figure 67. Using the final year captures the sedimentation effects over the preceding 49 yr as well as the 50th year. The figures also account for the flow distribution effects because the *x*-axis is the base discharge associated with a specific day. The figures also demonstrate the variability of the discharge versus stage relationship due to the shape of the hydrograph. The final year of the 50 yr future hydrograph corresponds to the historical 2019 hydrograph as depicted in Figure 68.

Figure 62. 2069 calculated water-surface elevation (WSEL) differences for Ratio 1 and Dredging Scenarios at Simmesport.

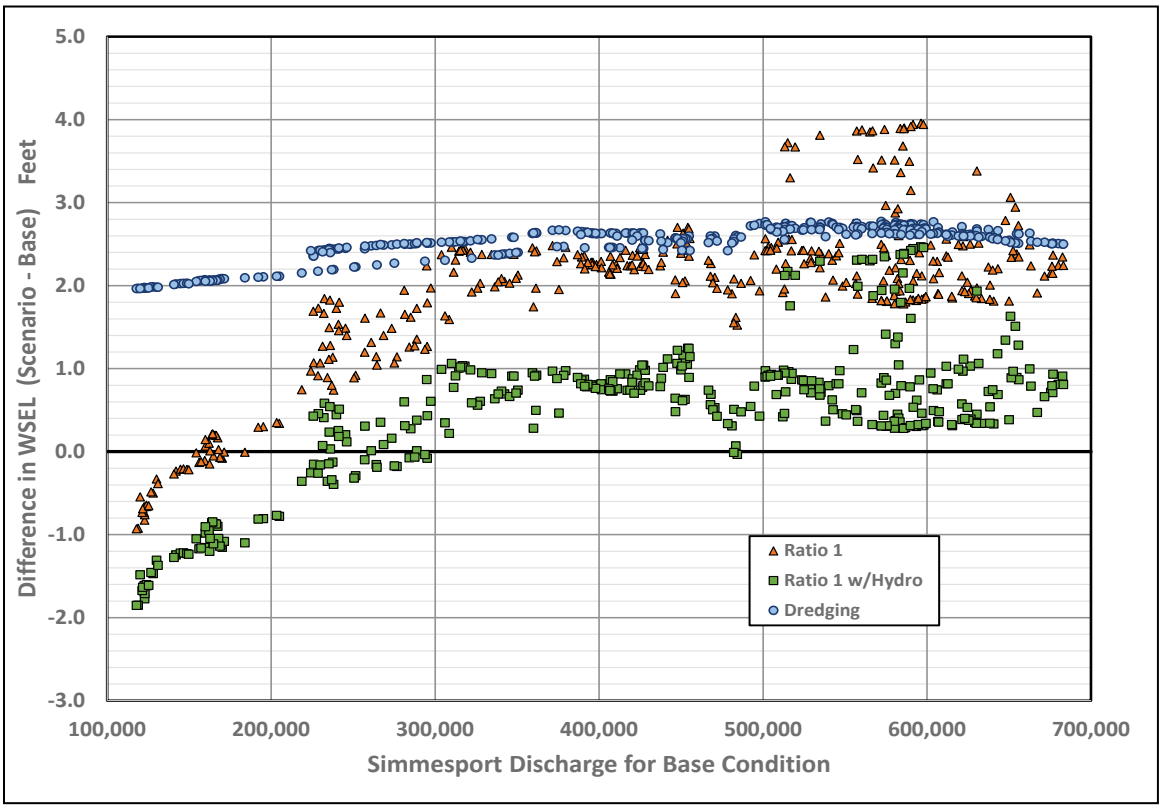


Figure 63. 2069 calculated WSEL differences for High Flow Scenarios and CTAR125 at Simmesport.

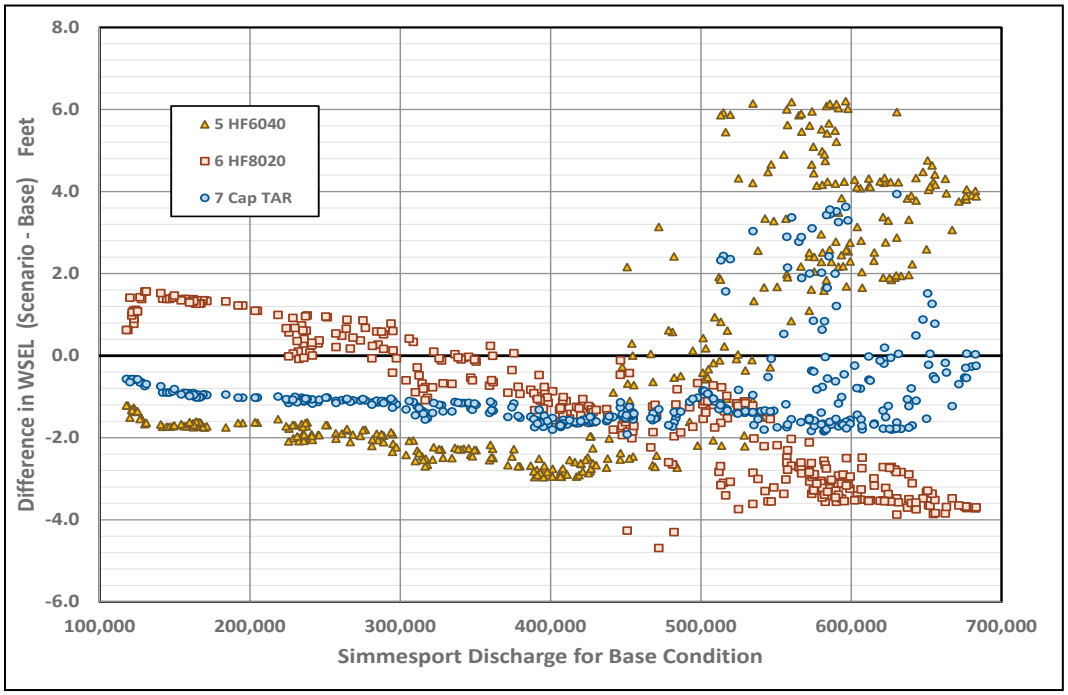


Figure 64. 2069 calculated WSEL differences for Daily and Max Aux Scenarios at Simmesport.

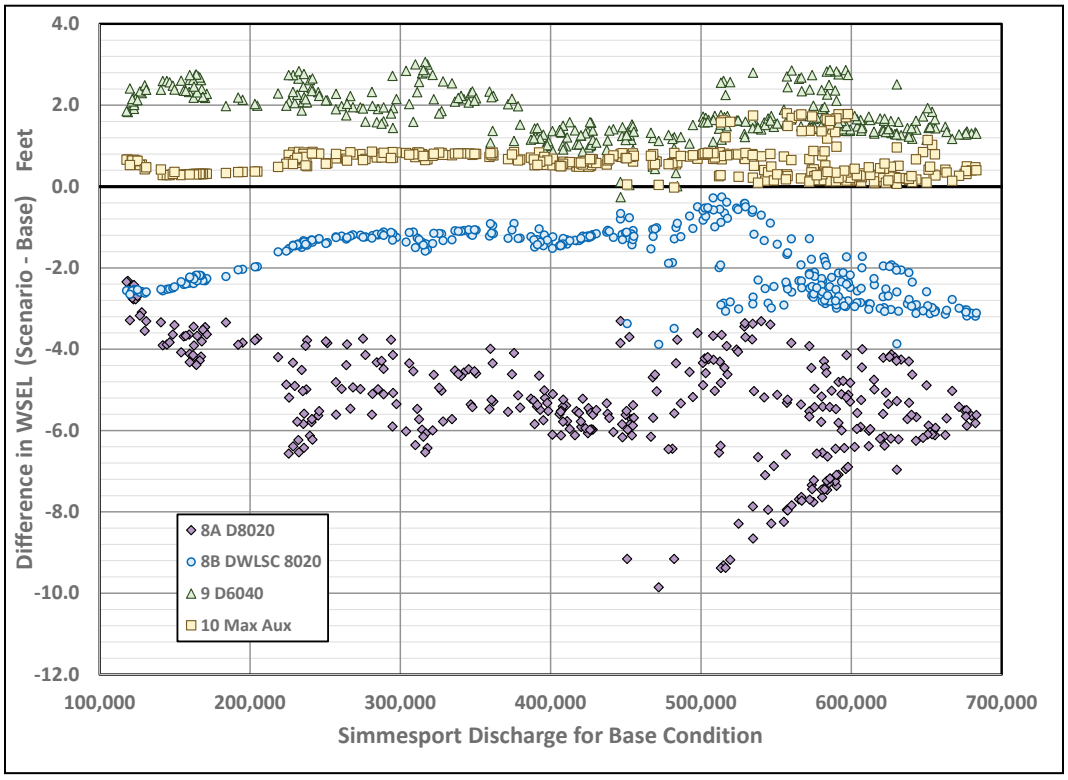


Figure 65. 2069 calculated WSEL differences for Ratio 1 and Dredging Scenarios at Morgan City.

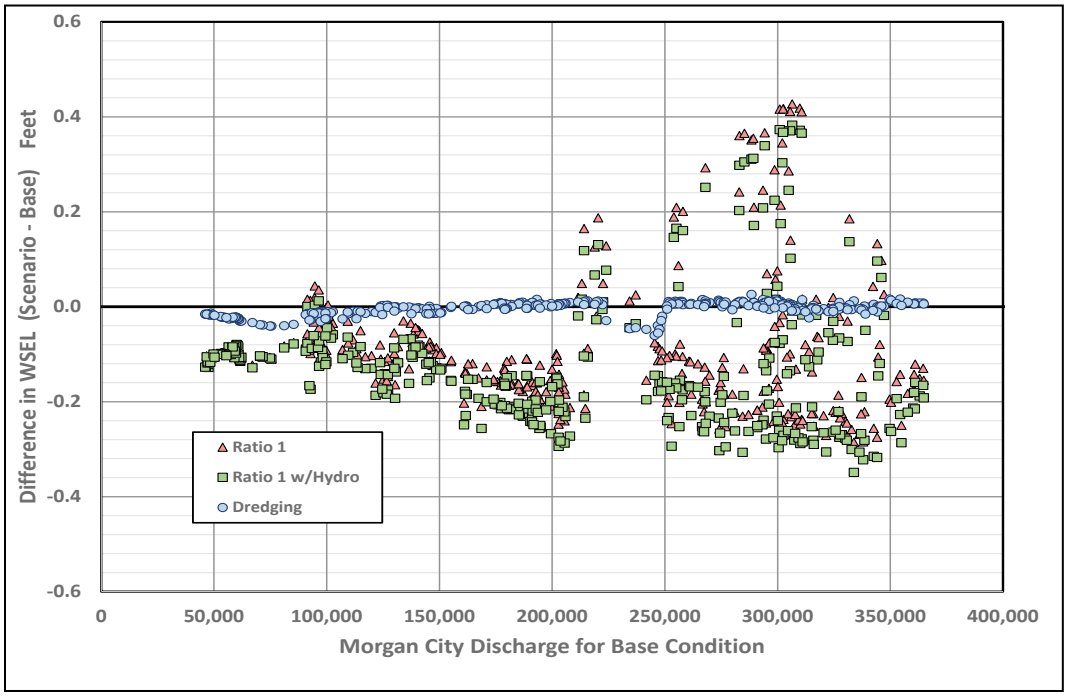


Figure 66. 2069 calculated WSEL differences for High Flow Scenarios and CTAR125 at Morgan City.

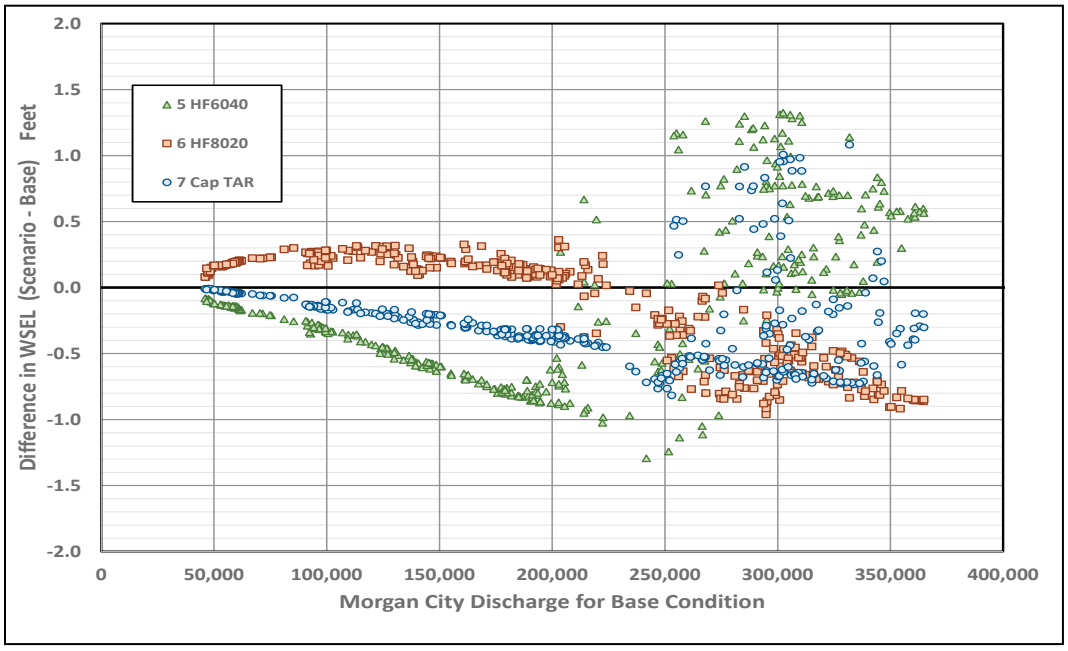


Figure 67. 2069 calculated WSEL differences for Daily and Max Aux Scenarios at Morgan City.

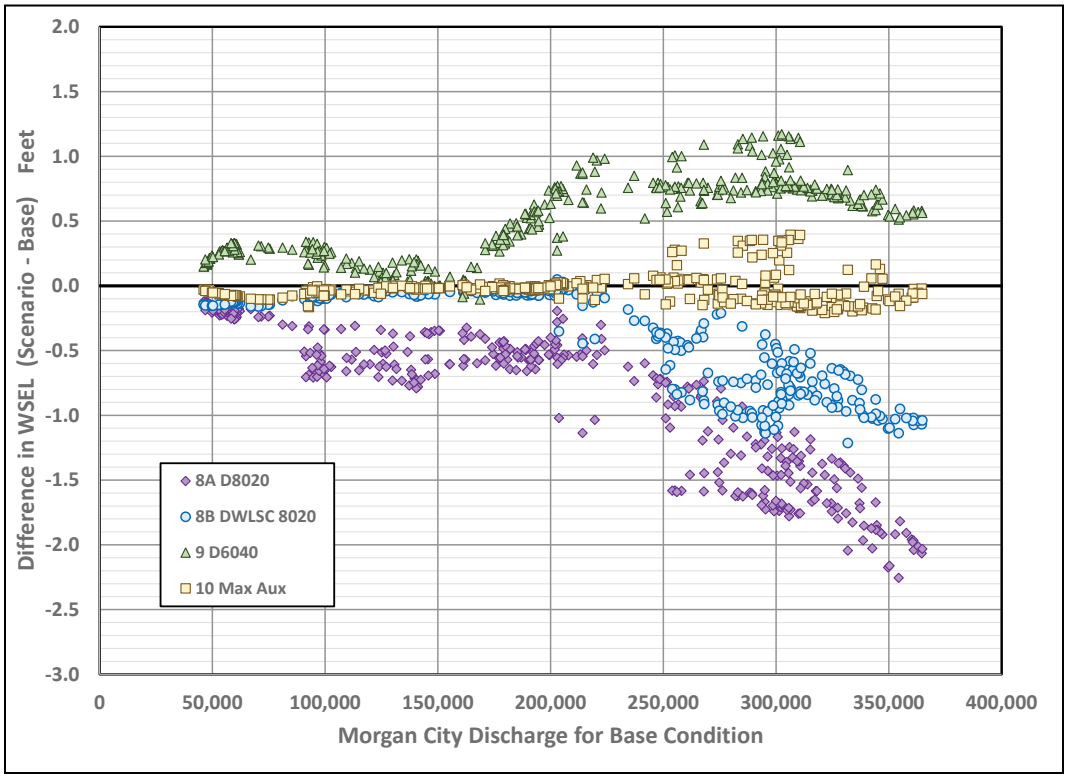
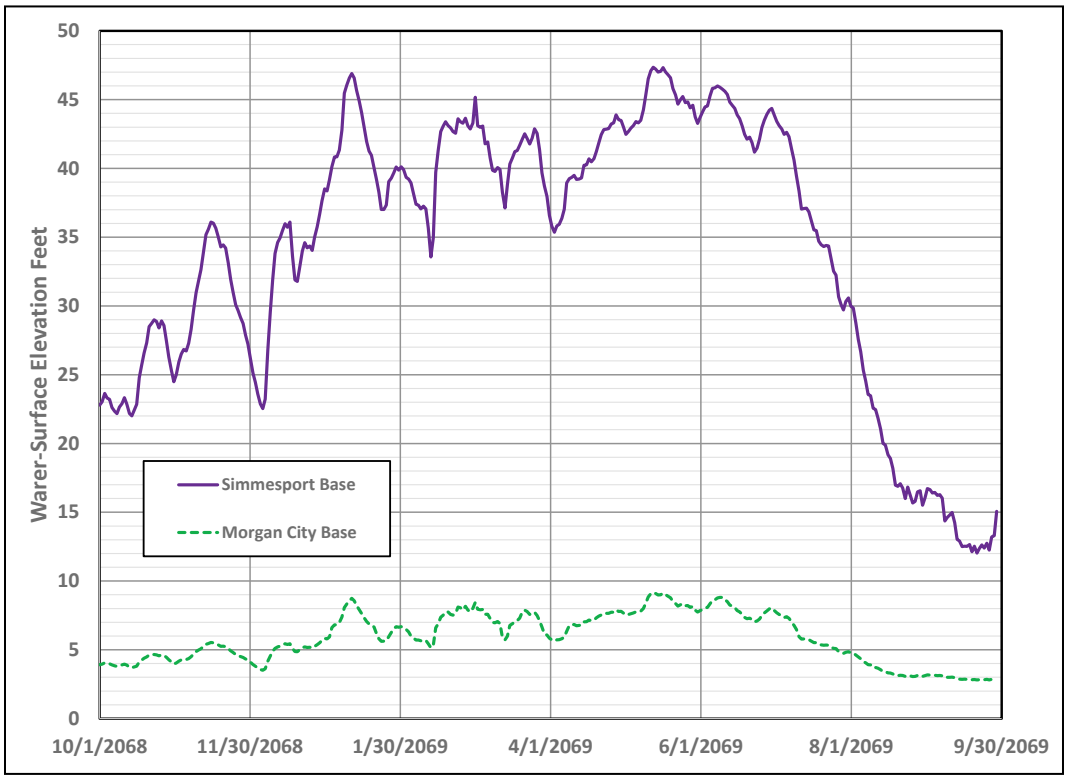


Figure 68. 2069 calculated Base WSEL at Simmesport and Morgan City.



General conclusions can be drawn from these figures. Remember that the scenario discharges are different from the base condition discharge and that the plotted points represent water-surface differences for a specific day during the 2069 annual hydrograph.

The Dredging Scenario has the same hydrograph as the Base Condition. Sediment dredged from the Mississippi River was deposited into the Outflow Channel daily at a rate of 5,750 cy/day (2,100,000 cy/yr). At Simmesport, after 50 yr, this resulted in increased water-surface elevations of about 2.5 ft. This increase can be attributed to an increase in aggradation for the dredging scenario. At Simmesport, there is also a long-term aggradation trend for the Base condition, which results in an additional 2–4 ft increase in water-surface elevations over 50 yr. At Morgan City, the 50 yr difference in water-surface elevations between the Base and Dredging Scenario was negligible, and there is no apparent long-term aggradation trend. The long-term specific gage at Morgan City shows increasing water-surface elevations due to predicted sea level rise.

The Ratio 1 Scenario increased sediment delivered to the Outflow Channel because hydropower was not included. The sediment diversion efficiency at the Hydropower Structure is lower than the Low Sill and Auxiliary Structures. At Simmesport, the increase in water-surface elevations at the highest discharges were between 2 and 4 ft with Ratio 1. When hydropower was included in Ratio 1, the increase in water-surface elevations was generally less than 1 ft at Simmesport. The Ratio 1 and Ratio 1 with hydropower scenarios had little effect on water-surface elevations at Morgan City. Differences ranged between plus and minus 0.4 ft.

Scenario 5 attempts to retain the annual 30% latitude flow at Simmesport while increasing the diversion percentages at high flow to achieve 40% latitude flow. This results in lower water-surface elevations at low flows where discharge diversions are reduced to balance the higher diversion percentages at high discharges. At Simmesport, water-surface elevations are reduced by approximately 3 ft until the diversion percentages are increased. At that point, the range in water-surface change varies dramatically between approximately plus 6 ft and minus 2 ft. This apparent inconsistency is related to the multi-peak hydrograph. At Morgan City, water-surface elevations are reduced by approximately 1 ft until the diversion percentages are increased. At that point, the range in water-surface change varies dramatically from approximately plus or

minus 1.5 ft. Calculated 2069, discharge and stage hydrographs at Simmesport are as shown in Figure 69 and Figure 70.

Figure 69. 2069 discharge hydrograph at Simmesport for high flow scenarios and Scenario 7.

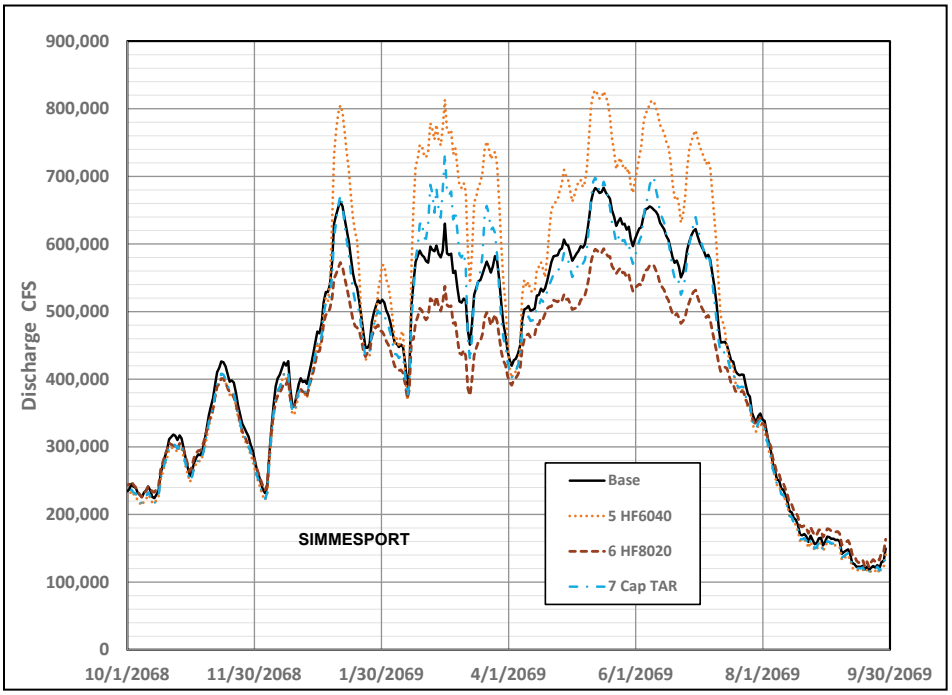
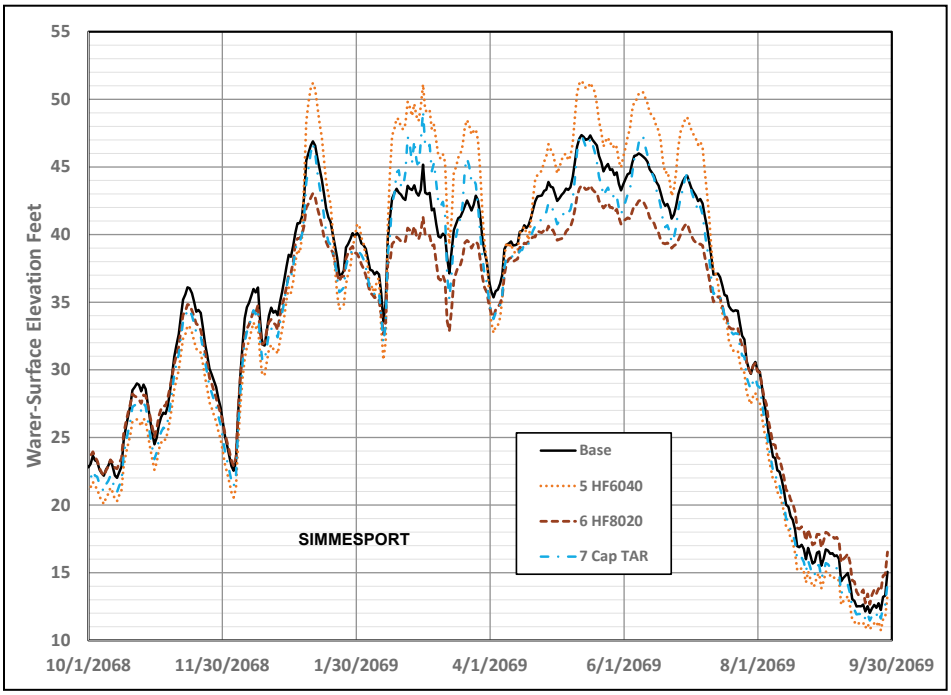


Figure 70. 2069 stage hydrograph at Simmesport for high flow scenarios and Scenario 7.



Scenario 6 attempts to retain the annual 30% latitude flow at Simmesport while decreasing the diversion percentages at high flow to achieve 20% latitude flow. This results in higher water-surface elevations at low flows, where discharge diversions are increased to balance the lower diversion percentages at high discharges. At Simmesport, water-surface elevations are increased by approximately 2 ft at the lowest discharges, and then, as discharge increases, water-surface differences decline to a maximum difference of approximately minus 4 ft at the highest discharge. At Morgan City, water-surface elevations are increased by less than 0.5 ft; then, as discharge increases, water-surface differences decline to a maximum difference of about minus 1 ft at the highest discharge.

Scenario 7 caps the flow at Tarbert Landing at 1,250,000 cfs to increase the flow diverted to the Atchafalaya River at high Mississippi River discharges. This scenario retains the annual 30% latitude discharge. Similar to Scenario 5, this results in lower water-surface elevations at low annual flows where discharge diversions are reduced to balance the higher diversion percentages at high annual discharges. At Simmesport, water-surface elevations are reduced by approximately 2 ft until the diversion percentages are increased. At that point, the range in water-surface change varies dramatically from approximately plus 4 ft to minus 2 ft. At Morgan City, water-surface elevations are reduced by approximately 0.5 ft until the diversion percentages are increased. At that point, the range in water-surface change varies from approximately plus or minus 1 ft.

Scenario 8A reduces the daily flow at Simmesport by a third, from 30% to 20% of the latitude flow. At Simmesport this results in water-surface decreases of approximately 4 to 6 ft at low and medium flows to approximately 4 to 10 ft at the highest annual discharges. At Morgan City this results in water-surface decreases of approximately 0.5 ft at low and medium flows to approximately 2 ft at the highest annual discharges.

Scenario 8B also reduces the daily flow at Simmesport by a third except when a structure safety constraint is exceeded. The reduced flow diversion is restricted by the head differential at the Low Sill Structure. At Simmesport, with low to medium flows, decreases in water-surface elevation range between 3 and 1 ft. At the highest annual discharges, the water-surface decrease is between zero and 4 ft. At Morgan City, with low to medium flows, there is no significant difference from the base

condition. At the highest annual discharges, the water-surface decrease is approximately 1 ft.

Scenario 9 increases the daily flow at Simmesport by a third, from 30% to 40%. At Simmesport, this results in water-surface increases between 1 and 3 ft. At Morgan City, this results in water-surface increases up to approximately 1 ft.

Scenario 10 maximizes use of the Auxiliary Structure, as this structure has a higher sediment diversion capability. At Simmesport, the increase in water-surface elevations varies between zero and 2 ft. At Morgan City, there is no significant difference in water-surface elevations at low to medium flows and between plus or minus less than a 0.5 ft difference at higher flows.

4.10 Water yields

The total water yields were evaluated for the base condition and each scenario. The values presented represent the predictive simulation for 50 yr into the future or the year 2069. Values for the total water yields are given at the following locations for each scenario and the base condition:

- The Red River
- Simmesport, Louisiana
- Morgan City, Louisiana
- Calumet, Louisiana
- The Outlet Channel and its ORCC subsections
    - Hydropower Structure
    - Old River Overbank Structure
    - Low Sill Structure
    - Auxiliary Structure.

The daily discharge values used in HEC-6T were calculated from rating curves that were based on the daily assigned discharges for each structure. Using rating curves introduced some error into the total average annual discharges. The maximum error was 3.5%.

Refer to Figure 71 and Table 10 for the values of the total calculated water yield in million acre-feet for the predictive 50 yr simulation. Additionally, Figure 72 and Table 11 represent the total change of water volume in

percentage passing through the ORCC for each scenario versus the base scenario as well as the total water volume difference passing through Simmesport, LA.

Table 10. 50 yr water yields in million acre-ft.

Scenario Name and Number	Red River	Simmesport	Outlet Channel	Hydropower Structure	Old River Overbank	Low Sill	Auxiliary Structure	Calumet	Morgan City
Base Condition	2,722	8,247	5,526	3,544	140	859	983	4,266	3,995
R1 Scenario 3	2,722	8,078	5,357	0.00	174	2,920	2,262	4,021	4,068
R1WH Scenario 4	2,722	8,078	5,356	2,867	174	0.49	2,314	4,021	4,067
60/40HF Scenario 5	2,722	8,320	5,598	3,887	187	796	729	4,079	4,254
80/20HF Scenario 6	2,722	8,056	5,334	4,272	71	497	495	4,193	3,876
CTAR125 Scenario 7	2,722	8,068	5,346	3,985	165	602	593	4,093	3,988
D80/20 Scenario 8a	2,722	5,560	2,838	2,625	8	110	95	2,792	2,774
D80/20WLSC Scenario 8b	2,722	7,034	4,312	3,325	73	461	454	3,569	3,473
D60/40 Scenario 9	2,722	10,763	8,042	4,782	187	1,757	1,317	5.327	5,457
Max Auxiliary Scenario 10	2,722	8,229	5,507	2,698	140	685	1,984	4,302	3,941
Dredging Scenario	2,722	8,248	5,526	3,544	140	859	983	4,284	3,977

Table 11. ORCC Flow and Simmesport 50 yr difference.

Scenario Name and Number	ORCC Diversion Percentage vs. the Base Condition	Change of Volume through Simmesport, LA (Million acre-ft)
Base	100%	0
R1 Scenario 3	96.9%	-169
R1WH Scenario 4	96.9%	-170
60/40HF Scenario 5	101.3%	73
80/20HF Scenario 6	96.5%	-191
CTAR125 Scenario 7	96.8%	-180
D80/20 Scenario 8a	51.4%	-2,688
D80/20WLSC Scenario 8b	78%	-1,213
D60/40 Scenario 9	145.5%	2,516
Max Auxiliary Scenario 10	99.7%	-19
Dredging Scenario	100%	0.36

Figure 71. 50 yr water yields.

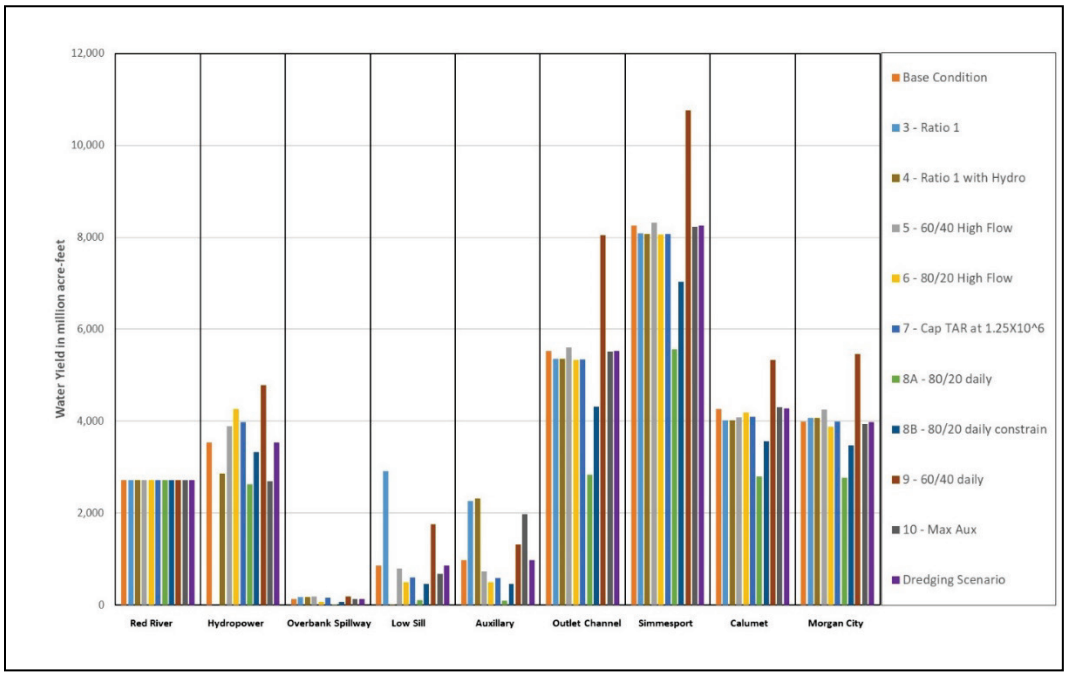
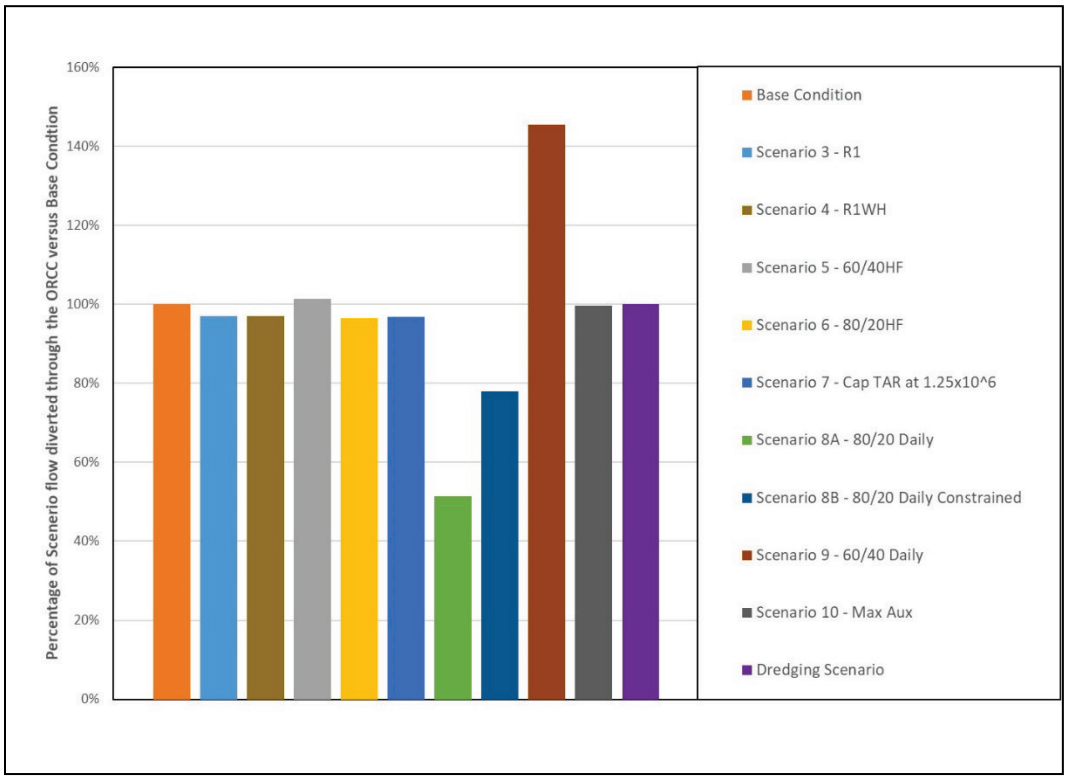


Figure 72. 50 yr ORCC diversion (%).



As depicted in Table 10, 60/40 Daily (Scenario 9) shows the largest increase of water volume down the Outflow Channel and, in turn, passing through Simmesport. Additionally, 80/20 Daily (Scenario 8A) showed the largest decrease of water volume passing down the Outflow channel and through Simmesport towards the Gulf. There was no change in water volume flowing down the Red River between the scenarios. The dredging scenario runs the same hydrograph as the base condition, and the water volume passing through the system between the two scenarios are identical.

Table 11 shows the difference in flow through the ORCC versus the base condition. The results presented show that 60/40 Daily (Scenario 9) has an increase of 46% of water volume through the ORCC while 80/20 Daily (Scenario 8A) has a decrease of 49% in water volume. Additionally, D80/20WLSC (Scenario 8B) also shows a large 22% decrease in water volume through the ORCC.

4.11 Differences in sediment passing gages

One way to evaluate the effect of changing the operation protocol at the ORCC is to compare the change in the sediment load that would occur with the adoption of a different operation scenario. At the end of 50 yr, the total sediment passing several gages for the base condition was compared to the 50 yr accumulated total sediment passing for each scenario. The changes in total sediment load in tons are plotted as the difference between each scenario and the base condition in Figure 73 and as the percentage change in Figure 74. However, the sand size classes are the significant portion of the sediment load that affects channel morphology, and the sand loads were also compared herein. The sand loads are plotted as the difference between each scenario and the base condition in Figure 75 and as the percentage change in Figure 76. Calculated 50 yr sand passing volumes and the percentage differences between base and scenario sand passing gages are shown in Table 12 and Table 13, respectively.

Figure 73. 50 yr difference in total sediment passing gages.

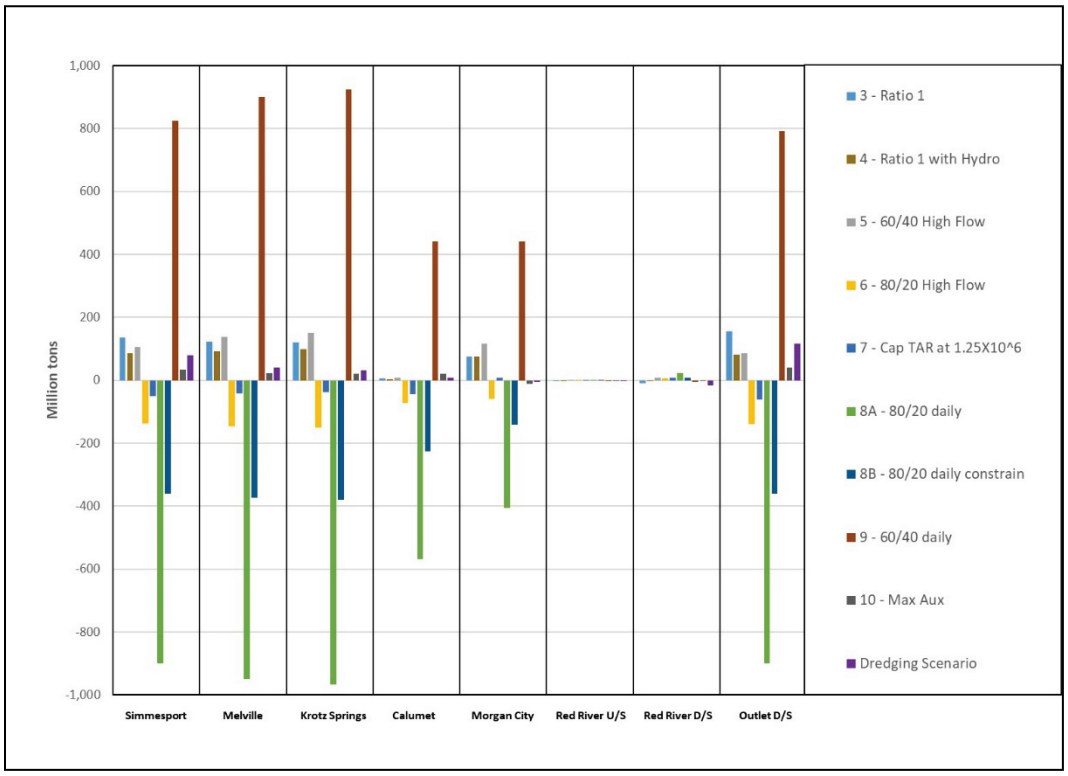


Figure 74. 50 yr difference in total sediment passing gages (%).

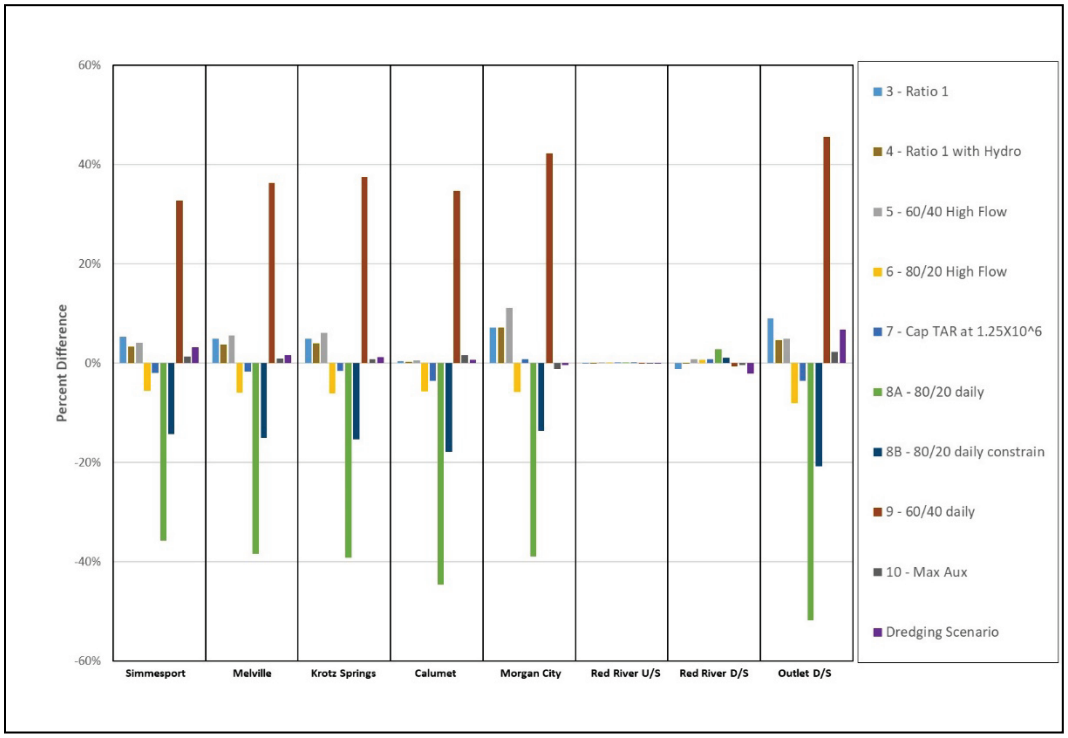


Figure 75. 50 yr difference in sand passing gages.

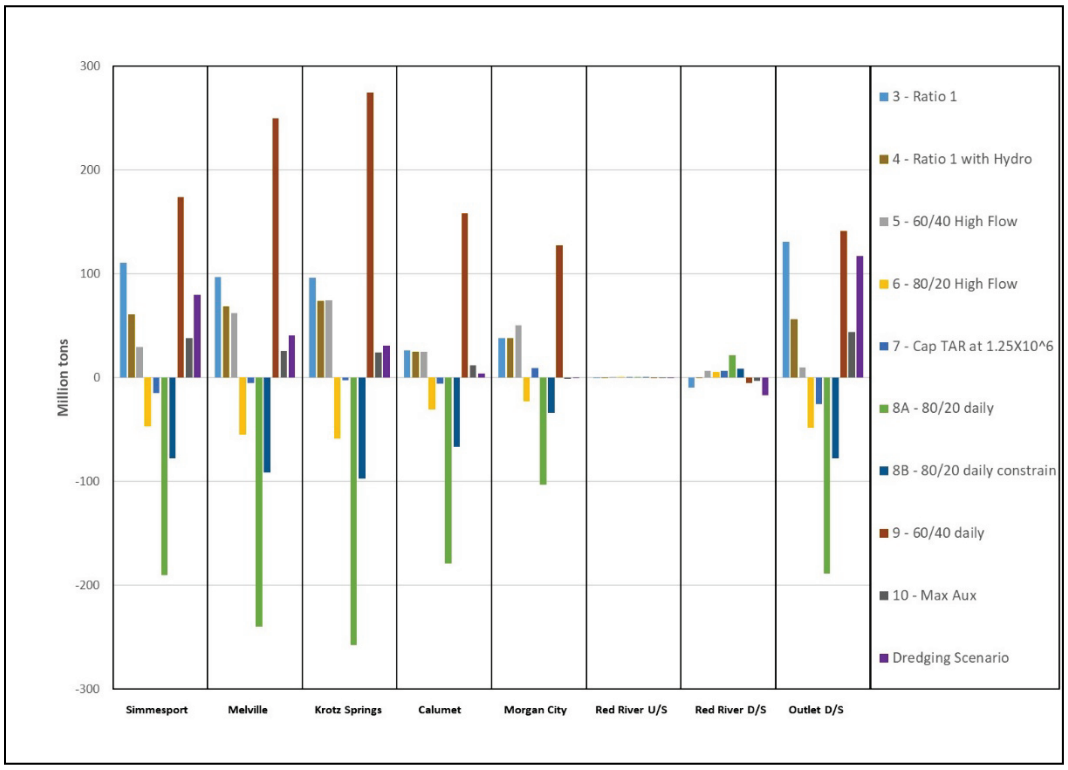


Figure 76. 50 yr difference in sand passing gages (%).

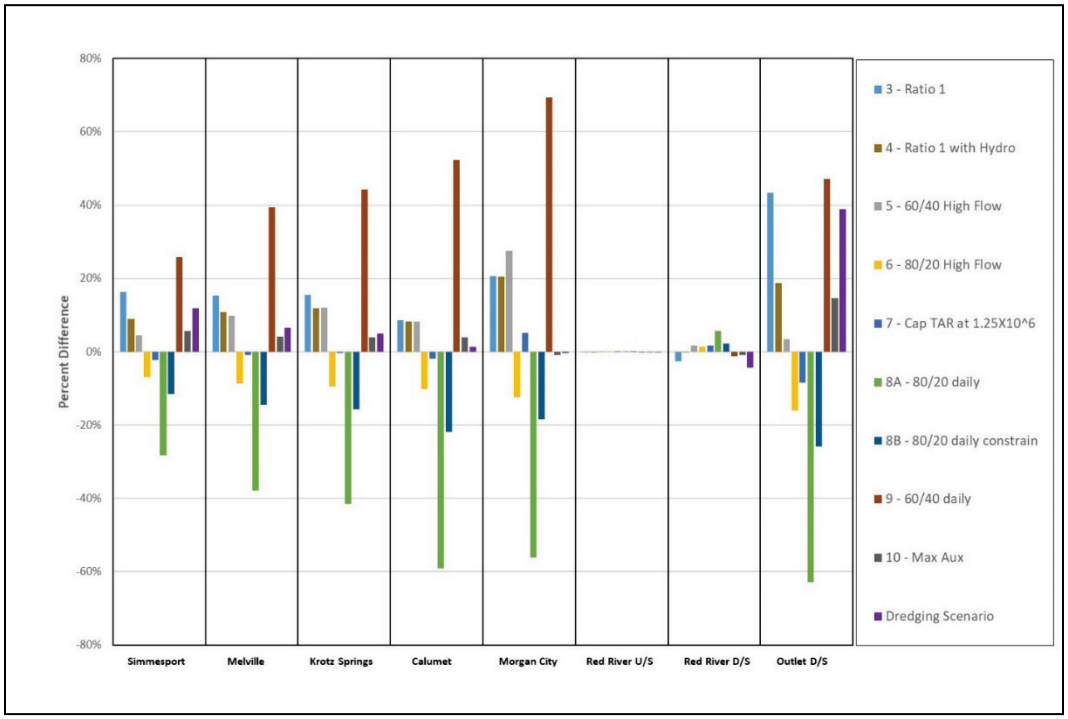


Table 12. 50 yr calculated sand passing — million tons.

Scenario	Red River RM 10.6	Outflow Channel RM 0.1	Simmesport RM -4.34	Melville RM -29.6	Krotz Springs RM 39.59	Calumet RM -111.2	Upstream Morgan City RM -119.7
Base Condition	386	301	672	633	621	303	184
R1 Scenario 3	376	431	782	730	717	329	222
R1WH Scenario 4	386	357	732	701	695	328	221
60/40HF Scenario 5	393	311	701	695	695	328	234
80/20HF Scenario 6	391	252	625	578	562	272	161
CTAR125 Scenario 7	393	275	657	627	618	297	193
D80/20 Scenario 8a	408	112	482	393	363	124	81
D80/20WLSC Scenario 8b	395	223	594	541	523	236	150
D60/40 Scenario 9	381	442	846	882	895	461	311
Max Auxiliary Scenario 10	383	344	709	658	645	315	182
Dredging Scenario	369	418	751	674	651	307	183

Table 13. 50 yr calculated sand passing — percentage increase from Base Condition.

Scenario	Red River RM 10.6	Outflow Channel RM 0.1	Simmesport RM -4.34	Melville RM -29.6	Krotz Springs RM 39.59	Calumet RM -111.2	Upstream Morgan City RM -119.7
R1 Scenario 3	-2.56%	43.40%	16.41%	15.33%	15.47%	8.63%	20.67%
R1WH Scenario 4	-0.12%	18.73%	9.02%	10.83%	11.95%	8.20%	20.54%
60/40HF Scenario 5	1.75%	3.36%	4.42%	9.78%	12.07%	8.27%	27.48%
80/20HF Scenario 6	1.34%	-16.11%	-6.97%	-8.66%	-9.42%	-10.12%	-12.50%
CTAR125 Scenario 7	1.70%	-8.45%	-2.19%	-0.85%	-0.39%	-1.91%	5.08%
D80/20 Scenario 8a	5.65%	-62.83%	-28.30%	-37.93%	-41.50%	-59.04%	-56.10%
D80/20WLSC Scenario 8b	2.19%	-25.83%	-11.60%	-14.43%	-15.68%	-21.97%	-18.47%
D60/40 Scenario 9	-1.30%	47.08%	25.89%	39.45%	44.19%	52.30%	69.45%
Max Auxiliary Scenario 10	-0.91%	14.54%	5.64%	4.05%	3.92%	3.96%	-0.85%
Dredging Scenario	-4.34%	38.91%	11.86%	6.45%	4.97%	1.36%	-0.43%

Two significant factors affect the sediment load downstream from the ORCC. One is that sediment transport potential changes with changes in discharge diversion percentages. This effect carries all the way to the Gulf of Mexico. The other is that sediment supply changes with the ORCC diversion percentages.

A decrease in Atchafalaya River discharge, due to decreased diversion percentages, will decrease the sediment transport potential so that deposition is likely in the Atchafalaya River. This potential deposition trend is countered somewhat by a decreased sediment supply that accompanies the decreased diversion discharge. The Atchafalaya River response is demonstrated in Figure 75 where sand load continues to decrease in the downstream direction for the 80/20 daily scenario.

In contrast, increasing the Atchafalaya River discharge results in increased sediment transport potential, and the bed itself becomes a sediment source. This is shown in Figure 75 where the sand load continues to increase in a downstream direction with the 60/40 Daily scenario.

The sediment load response in the Outflow Channel follows the same trends as the downstream reaches of the Atchafalaya River.

As expected, there is very little difference in the sand transport in all scenarios versus the base condition in the upstream portion of the Red River. The downstream portion of the Red River experiences minor differences caused by backwater in high flow events.

4.12 Differences in dredging requirements at Berwick Bay

Dredging in HEC-6T at Berwick Bay is modeled according to protocols established in the Flowline Model (Lewis and Copeland 2018). The dredging reach extends from RM -121.5 to RM -119.9 and includes four cross sections. The dredging width varies between 1,380 and 1,120 ft. During the numerical simulation, on January 1 of each year, if the calculated bed elevation at any cross section in the dredging reach is above elevation -48, then dredging commences at a rate of 50,000 cy/day. Bed material is removed from the downstream control volume, which is associated with the downstream cross section, until the designated dredging elevation of -48 ft is reached. The dredging calculations move upstream until the dredging elevation is reached at the upstream control volume. This calculation sweep can start later in the year if the target elevation has not been reached on January 1. However, only one sweep is calculated per year. The assigned movable-bed width and the dredging-template width are the same in the OMAR Model so that all the sediment deposited in the dredging reach is dredged when the trigger elevation is exceeded. The dredged material is deposited downstream from the dredging reach (RM -122.0).

The numerical model reproduced reported average dredging quantities over a 20 yr period within approximately 1%. Reported dredging in Berwick Bay between 1997 and 2016 was 23,226,567 cy or 1,161,328 cy/yr. Between 1997 and 2016, HEC-6T calculated 22,944,501 cy or 1,147,225 cy/yr.

The numerical model was used to predict the 50 yr average annual dredging requirements at Berwick Bay for each of the scenarios. The accumulated dredging calculated between 30 September 2019 and 30 September 2069 was divided by 50 to get the average annual dredging. Calculated dredging volumes are shown in Figure 77. The percentage increase/decrease for each scenario was compared to the base condition in Figure 78. Calculated values are shown in Table 14.

Dredging requirements in Berwick Bay are a function of both the sediment delivered to the dredging reach and the velocity of the discharge available to transport the sediment past the site. The daily 60/40 scenario (D6040) results in a higher annual water yield than the Base (existing) condition for the Atchafalaya River. The high flow 60/40 scenario (HF6040) results in higher Atchafalaya River discharges at high flow and lower discharges at low flow, retaining the annual 70/30 latitude flow split. Higher discharges result in higher velocities and greater sediment transport potential upstream from Morgan City and through the Berwick Bay dredging reach. More sediment is input into the Atchafalaya from the Mississippi with the 60/40 scenarios than with the Base Condition. In addition, more sediment is supplied from the river bed to meet the increase in sediment transport capacity with the 60/40 scenarios. However, even with this increase in sediment supply, calculated results indicate that both 60/40 scenarios result in significantly less dredging in Berwick Bay. This result demonstrates that the increased velocities move more sediment through Berwick Bay and are more significant than the increased sediment supply. Overall, the daily 60/40 scenario (D6040) has a larger sediment supply, so its decrease in dredging is less than the decrease in dredging for the High Flow 60/40 scenario (HF6040).

In contrast, the daily 80/20 scenario (D8020) provides less annual water yield than the Base (existing) condition for the Atchafalaya River. The high flow 80/20 scenario (HF8020) results in lower Atchafalaya River discharges at high flow and higher discharges at low flow, retaining the annual 70/30 latitude flow split. Lower discharges result in lower velocities and less sediment transport potential upstream from Morgan City and through the Berwick Bay dredging reach. Less sediment is input into the Atchafalaya from the Mississippi with the 80/20 scenarios. In addition, less sediment is supplied from the river bed upstream from Morgan City. The HEC-6T results indicate a more complicated response to the 80/20 scenarios in terms of dredging in Berwick Bay. The daily

scenario (D8020) results in significantly less dredging in Berwick Bay, even less than the daily 60/40 scenario. This result suggests that lower water yields are not transporting as much sediment to Berwick Bay, and, therefore, less dredging is required even though the transport potential through Berwick Bay is less. This conclusion is supported by the scenario that constrains flow through the Low Sill at high ORCC upstream and downstream water-surface differentials (D8020WLSC). This scenario diverts more flow into the Atchafalaya at low flow than D8020, increasing sediment transport potential and supplying more sediment to Berwick Bay. Calculated D8020WLSC dredging is still less than the Base condition but not as much as with the D8020 scenario. For the high flow 80/20 scenario (HF8020), high flow discharges are reduced, and low flow discharges are increased to retain the annual 70/30 latitude discharge. The overall effect is more dredging required at Berwick Bay. Results with the 80/20 scenarios demonstrate that both sediment supply and transport potential through Berwick Bay are significant factors in determining dredging requirements (Figure 79).

Although the average annual dredging at Berwick Bay is primarily a function of the annual water yield, the distribution of the daily discharges can also be important. This fact is demonstrated by comparing the calculated dredging volumes for the Base Condition and the two Ratio 1 alternatives. The average annual water yields for these three runs were within 2% of each other. However, the Ratio 1 alternatives had sand passing volumes that were approximately 21% higher than the Base Condition and dredging volumes that were approximately 22% lower than the Base Condition. This result is expected if there are more high discharges with the Ratio 1 alternatives. Higher discharge results in increased sediment inflow into the dredging reach but also results in increased sediment transport potential that carries sediment through the dredging reach without depositing. Flow distribution at the ORCC for the Ratio 1 alternatives was based on a prescribed operation protocol whereas the Base Condition flow distribution at the ORCC was based on 2013–2019 historical operations.

Figure 77. Projected 50 yr average annual dredging at Berwick Bay.

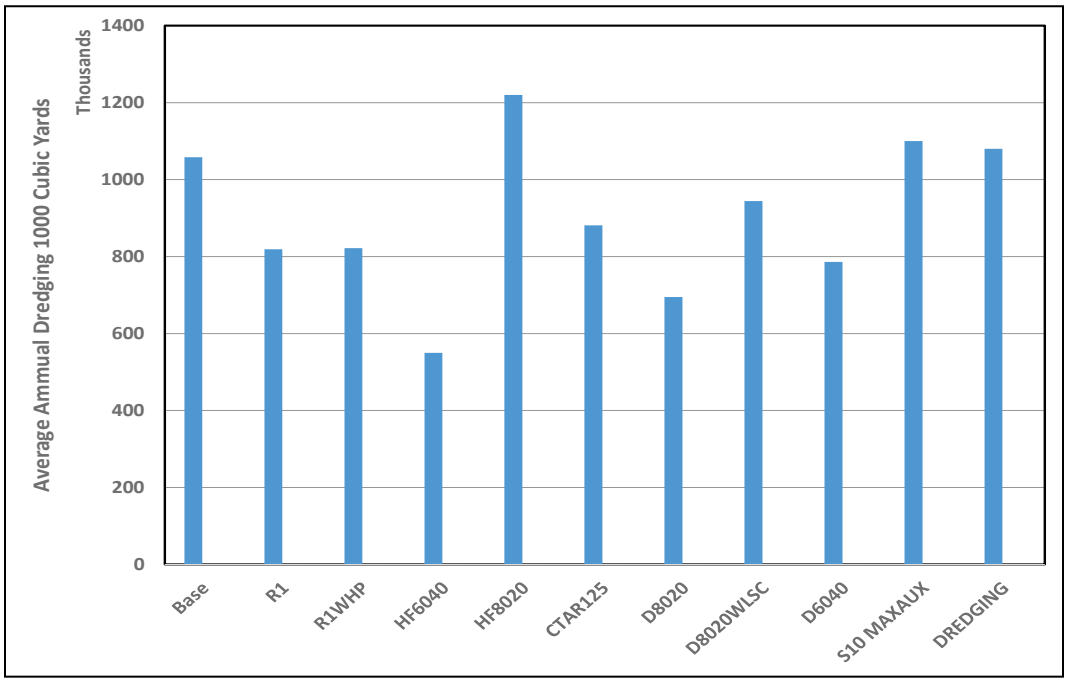


Figure 78. Projected percentage change in 50 yr average annual dredging at Berwick Bay.

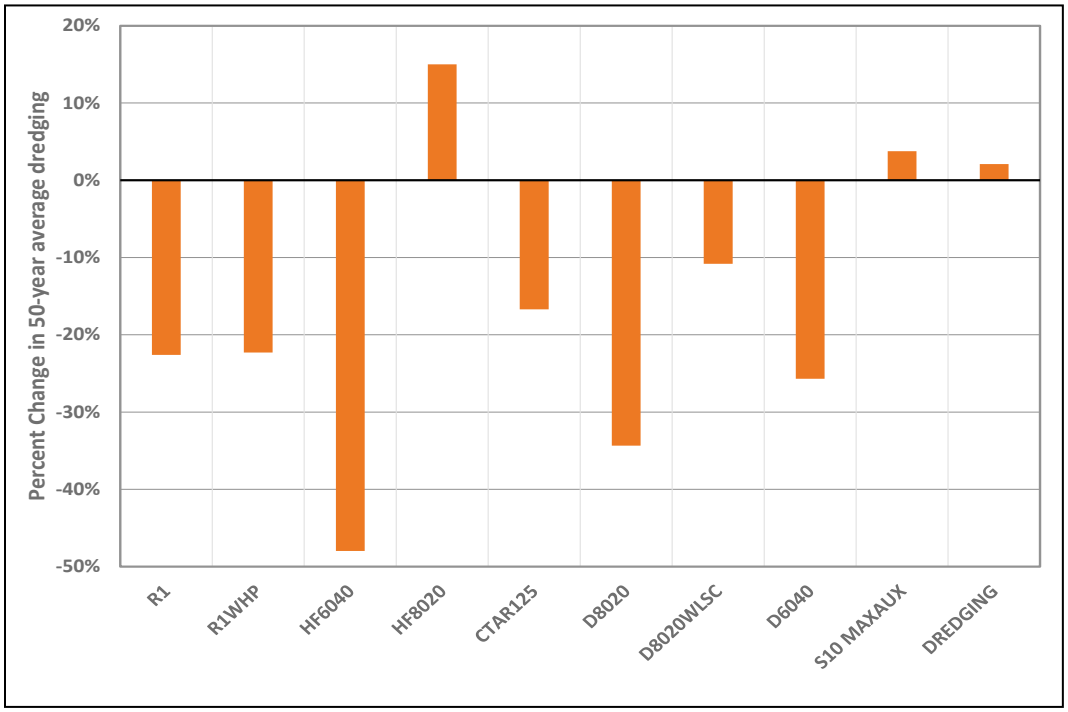


Figure 79. Projected accumulated dredging at Berwick Bay for the 80/20 scenarios.

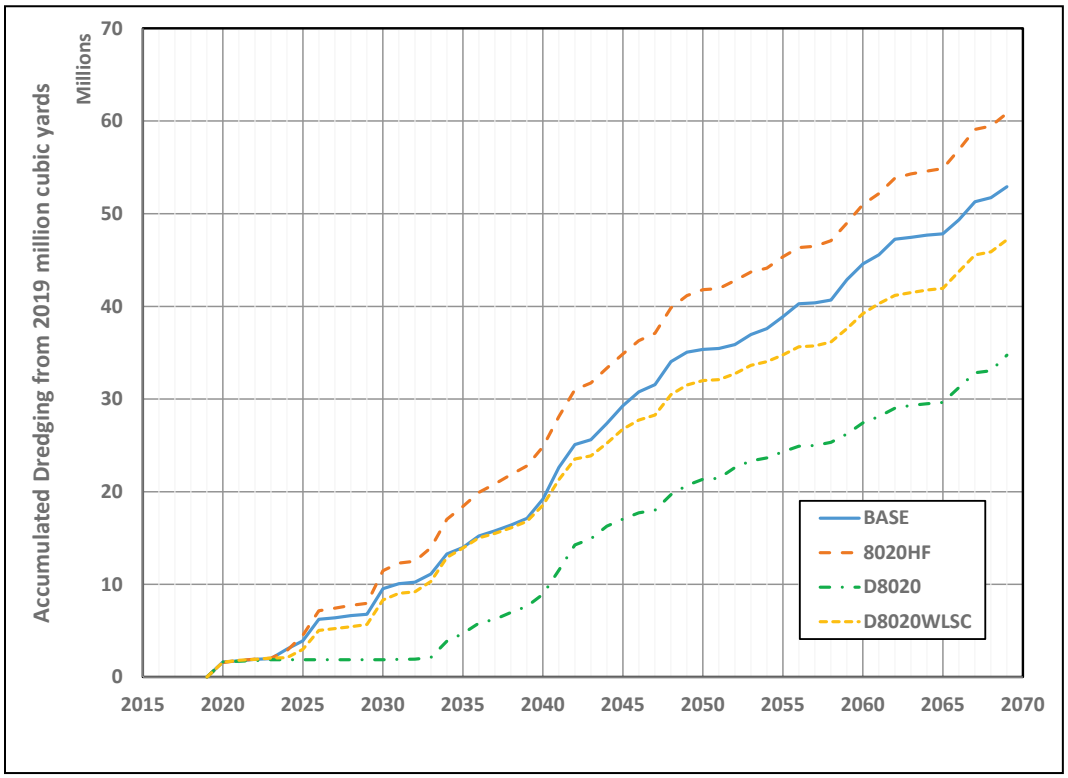


Table 14. Projected 50 yr average annual dredging in Berwick Bay.

Scenario		50 yr Average Annual Dredging 1000 cy	50 yr Average Percentage Change in Dredging
	Base Condition	1,058	
3	R1	819	-22.6
4	R1WHP	822	-22.3
5	6040HF	550	-48.0
6	8020HF	1,220	15.0
7	CTAR125	881	-16.7
8A	D8020	695	-34.3
8B	D8020WLSC	944	-10.8
9	D6040	786	-25.7
10	MAXAUX	1,100	3.8
	MISS R DREDGING	1,080	2.1

5 Conclusions and Recommendations

5.1 Conclusions

The objectives of this study were to determine the long-term and system-wide sedimentation effects that would occur in the Atchafalaya River if the operation protocols at the ORCC were changed as well as to develop and analyze the long-term sedimentation effects due to a dredging scenario wherein material dredged from the Mississippi River is deposited into the outflow channel downstream of the ORCC. The changes evaluated included increasing and decreasing the percentage of flow that is diverted from the Mississippi River at the ORCC and modifying the distributions through the four ORCC structures. Specifically, the study evaluated the differences in the following:

- specific gage heights
- bed elevations
- water-surface elevations
- sediment passing gages and sediments yields
- water yields
- dredging requirements at Berwick Bay.

Two significant factors affect the sediment load downstream from the ORCC. One is that sediment transport potential changes with changes in discharge diversion percentages. This effect carries all the way to the Gulf of Mexico. The other is that sediment supply changes with the ORCC diversion percentages.

A decrease in Atchafalaya River discharge, due to decreased diversion percentages, will decrease the sediment transport potential so that deposition is likely in the Atchafalaya River. This potential deposition trend is countered somewhat by a decreased sediment supply that accompanies the decreased diversion discharge. An example is the Atchafalaya River response to the 80/20 daily scenario (D8020) where sand load continues to decrease in the downstream direction.

In contrast, increasing the Atchafalaya River discharge results in increased sediment transport potential, and the bed itself becomes a sediment source. An example is the Atchafalaya River response to the 60/40 daily

scenario (D6040) where the sand load continues to increase in a downstream direction.

The results of this study successfully evaluated the response of the Atchafalaya River in terms of water-surface elevation changes, bed elevation changes, and sediment transport. The results of this study were provided by a robust and intricate HEC-6T model of the Atchafalaya River that is ready for application in future studies.

5.2 Comparison of scenarios and Base Condition — river response

- The Ratio 1 Scenario eliminated diversions through the hydropower structure, thereby increasing sediment delivered to the Outflow Channel because sediment diversion efficiency at the Hydropower Structure is much lower than the Low Sill and Auxiliary Structures. The average annual flow diverted through the ORCC was essentially equal to the Base condition. The average annual sand load at Simmesport increased by 16.4% relative to the Base condition and at Morgan City by 20.7% relative to the Base condition. At Simmesport, after 50 yr, the bed elevation increased approximately 3 ft, and the maximum increase in water-surface elevation was approximately 4 ft, relative to the Base condition. At Morgan City, after 50 yr, the annual variations in dredging operations made long-term aggradation/degradation indistinguishable. At the end of the 50 yr simulations, the scenario bed elevation was approximately 1 ft lower than the Base condition. Water-surface elevation differences, relative to the Base condition, ranged between plus and minus 0.4 ft. Adoption of this scenario would result in a 22.6% decrease in Berwick Bay dredging.
- When hydropower was included in Ratio 1 (R1WH), the average annual flow diverted through the ORCC was the same as R1. The average annual sand load at Simmesport increased by 9.0% relative to the Base condition and at Morgan City by 20.5%. The change in bed elevation at Simmesport, relative to the Base condition, was negligible after the 50 yr simulation. The increase in water-surface elevations was generally less than 1 ft at Simmesport. At Morgan City, similar to R1, neither scenario aggradation nor scenario degradation, relative to the Base condition, were distinguishable due to annual variability in dredging operations. At the end of the 50 yr simulations, the scenario bed elevation was approximately 1 ft lower than the Base condition. The R1WH scenario had little effect on water-surface elevations at

Morgan City. Differences ranged between plus and minus 0.4 ft. Adoption of this scenario would result in a 22.3% decrease in dredging in Berwick Bay.

- Scenario 5 (HF6040) attempts to retain the annual 30% latitude flow at Simmesport while increasing the diversion percentages at high flow to achieve 40% latitude flow. The average annual flow diverted through the ORCC was essentially the same as the Base condition. The average annual sand load at Simmesport increased by 4.1% relative to the Base condition and at Morgan City by 27.5%. This results in lower water-surface elevations at low flows, where discharge diversions are reduced to balance the higher diversion percentages at high discharges. After 50 yr at Simmesport, the bed elevation degraded approximately 4 ft, relative to the Base condition. Water-surface elevations were reduced approximately 3 ft, relative to the Base condition, until the diversion percentages were increased at the higher latitude discharges. At that point, the range in water-surface change varied dramatically between approximately plus 6 ft and minus 2 ft. This apparent inconsistency is related to the multi-peak hydrograph. At Morgan City, after 50 yr, neither scenario aggradation nor scenario degradation, relative to the Base condition, were distinguishable due to annual variability in dredging operations. However, the scenario bed elevation was lower than the Base condition's bed elevation during most of the 50 yr simulation, and at the end of the 50 yr, the scenario bed was approximately 6 ft lower than the Base condition bed. At Morgan City, water-surface elevations were reduced by approximately 1 ft until the diversion percentages are increased. At that point, the range in water-surface change varies from approximately plus or minus 1.5 ft. Adoption of this scenario would result in a 48% decrease in Berwick Bay dredging.
- Scenario 6 (HF8020) attempts to retain the annual 30% latitude flow at Simmesport while decreasing the diversion percentages at high flow to achieve 20% latitude flow. The average annual flow diverted through the ORCC was essentially the same as the Base condition. The average annual sand load at Simmesport decreased by 7% relative to the Base condition and at Morgan City by 12.5%. This results in higher water-surface elevations at low flows, where discharge diversions are increased to balance the lower diversion percentages at high discharges. After 50 yr, there was negligible difference between scenario and Base condition bed elevations at Simmesport. Water-surface elevations are increased by approximately 2 ft at the lowest

discharges, and then, as discharge increases, water-surface differences decline to a maximum difference of approximately minus 4 ft at the highest discharge. At Morgan City, after 50 yr, neither scenario aggradation nor scenario degradation, relative to the Base condition, were distinguishable due to annual variability in dredging operations. The average scenario and Base condition bed elevation appear to be approximately equal during most of the 50 yr simulation, but at the end of the 50 yr, the scenario bed was approximately 2.5 ft lower than the Base condition. At Morgan City, water-surface elevations are increased by less than 0.5 ft; then, as discharge increases, water-surface differences decline to a maximum difference of approximately minus 1 ft at the highest discharge. Adoption of this scenario would result in a 15% increase in Berwick Bay dredging. This was the highest increase in dredging for all scenarios relative to the Base condition.

- Scenario 7 (CTAR125) caps the flow at Tarbert Landing at 1,250,000 cfs to increase the flow diverted to the Atchafalaya River at high Mississippi River discharges. This scenario retains the annual 30% latitude discharge. The average annual water diverted through the ORCC was essentially the same as the Base condition. The average annual sand load at Simmesport decreased by 2% relative to the Base condition and increased at Morgan City by 5.1%. Similar to Scenario 5, this results in lower water-surface elevations at low annual flows, where discharge diversions are reduced to balance the higher diversion percentages at high annual discharges. After 50 yr, there was approximately 1 ft of degradation, relative to the Base condition, at Simmesport. At Simmesport, water-surface elevations are reduced by about two ft until the diversion percentages are increased. At that point, the range in water-surface change varies dramatically from approximately plus four ft to minus two ft. At Morgan City there were negligible differences in the scenario and Base condition bed elevations during the 50 yr simulation. At Morgan City, water-surface elevations are reduced by approximately 0.5 ft until the diversion percentages are increased. At that point, the range in water-surface change varies dramatically from approximately plus or minus 1 ft. Adoption of this scenario would result in a 16.7% decrease in Berwick Bay dredging.
- Scenario 8A (D8020) reduces the daily flow at Simmesport by a third, from 30% to 20% of the latitude flow. The average annual flow diverted through the ORCC was 51% of the Base condition. The average annual sand load at Simmesport decreased by 28% relative to the Base condition and at Morgan City by 56.1%. At Simmesport, this scenario

results in channel aggradation of approximately 7 ft after 50 yr, relative to the Base condition. At Simmesport this results in water-surface decreases of approximately 4 to 6 ft at low and medium flows to approximately 4 to 10 ft at the highest annual discharges. At Morgan City, the average difference between scenario and Base condition bed elevations was negligible. Annual variations in bed elevation were less with the scenario because dredging operations were reduced with the lower discharges associated with Scenario D8020. At Morgan City this results in water-surface decreases of approximately 0.5 ft at low and medium flows to approximately 2 ft at the highest annual discharges. Adoption of this scenario would result in a 34.3% decrease in Berwick Bay dredging.

- Scenario 8B (D8020WLSC) also reduces the daily flow at Simmesport by a third except when a structure safety constraint is exceeded. The reduced flow diversion is restricted by the head differential at the Low Sill Structure. The average annual water diverted through the ORCC was 78% of the Base condition. The average annual sand load at Simmesport decreased by 12% relative to the Base condition and at Morgan City by 18.5%. At Simmesport, this results in channel aggradation of approximately 3 ft after 50 yr, relative to the Base condition. At Simmesport, with low to medium flows, decreases in water-surface elevation range between 3 and 1 ft. At the highest annual discharges, the water-surface decrease is between zero and 4 ft. At Morgan City, the average difference between scenario and Base condition's bed elevations was negligible. At Morgan City, with low to medium flows, there is no significant difference from the base condition. At the highest annual discharges, the water-surface decrease is approximately 1 ft. Adoption of this scenario would result in a 10.8% decrease in Berwick Bay dredging.
- Scenario 9 (D6040) increases the daily flow at Simmesport by a third, from 30% to 40%. The average annual flow diverted through the ORCC was 146% of the Base condition. The average annual sand load at Simmesport increased by 26% relative to the Base condition and at Morgan City by 69.5%. At Simmesport, this results in channel degradation of approximately 8 ft after 50 yr, relative to the Base condition. At Simmesport this results in water-surface increases between 1 and 3 ft. At Morgan City this results in water-surface increases up to approximately 1 ft. Average bed differences between Scenario D6040 and the Base condition were negligible at Morgan City. However, there were significant annual differences due to

dredging operations. The average 50 yr scenario degradation, with respect to the Base condition was 0.2 ft. Adoption of this scenario would result in a 25.7% decrease in Berwick Bay dredging.

- Scenario 10 (MAXAUX) maximizes use of the Auxiliary Structure, as this structure has a higher sediment diversion capability. The average annual water diverted through the ORCC was essentially the same as the Base condition. The average annual sand load at Simmesport increased by 6% relative to the Base condition and decreased at Morgan City by 0.9%. After 50 yr, there was negligible difference between scenario and Base condition bed elevations at Simmesport. At Simmesport, the increase in water-surface elevations varies between zero and two ft. At Morgan City, the average difference between scenario and Base condition bed elevations was negligible At Morgan City there is no significant difference in water-surface elevations at low to medium flows and between plus or minus less than a 0.5 ft difference at higher flows. Adoption of this scenario would result in a 3.8% increase in Berwick Bay dredging.
- The Dredging Scenario has the same hydrograph and average annual flow diversion as the Base condition. Sediment dredged from the Mississippi River was deposited into the Outflow Channel daily at a rate of 5,750 cy/day (2,100,000 cy/yr). At Simmesport, after 50 yr, this resulted in increased bed elevations of 4.7 ft and increased water-surface elevations of approximately 2.5 ft relative to the Base condition. At Morgan City, the average differences in bed elevation and water-surface elevations after 50 yr, relative to the Base condition were negligible. Adoption of this scenario would result in a 2.1% increase in Berwick Bay dredging.

5.3 Recommendations

The engineers who authored this report recommend continued use of this numerical model to perform additional analysis. The results of this study showed general aggradation to the filling of side channels and lakes with sediment and also that aggradation is projected throughout the Atchafalaya River to varying degrees. Additional research is recommended, building from the OMAR HEC-6T model, to explore whether the expected aggradation can be reduced. Additional research tasks are suggested in the following:

- It is recommended that additional research is conducted to study and project the long-term sediment transport effects in the Atchafalaya

River, specifically in the area of the confluence of the Whiskey Bay Pilot Channel and the Old Atchafalaya River.

- It is recommended that investigations be made to develop methods to further quantify the unmeasured bed load transport so that sediment loads and bed load gradations entering the Atchafalaya River can be more reliably defined.
- It is recommended that consistent sediment sampling protocols be established in regard to equipment, sampling methods, laboratory methods, and analytical treatment of data at existing stations. Implement duplicate sampling protocols so that historic data can be correlated to current data.
- Additionally, it is recommended that additional research be performed to study the effects of a *mock* multi-purpose water-control structure north of Butte La Rose and to analyze the sediment transport and water-surface effects of raising the upstream water-surface elevation. It is expected that this structure will benefit the interests of local industry such as crawfish and fisheries.
- Next, it is recommended that additional research be conducted with the HEC-6T model to explore changing the flow distribution between the Atchafalaya River and Wax Lake Outlet to analyze the long-term upstream and downstream sediment transport effects.
- Last, it is recommended that additional research studies closing and/or reopening the many distributaries from the Atchafalaya River to compare and analyze the long-term sedimentation effects and water-surface elevation trends.

By performing this additional research, a greater understanding of how various factors would impact the sedimentation dynamics in the Atchafalaya River Basin.

References

Ariathurai, R., and R. B. Krone. 1976. "Finite Element Model for Cohesive Sediment Transport." *Journal of the Hydraulics Division, ASCE* 2(3): 323–33.

Bell, G. L., D. D. Abraham, and N. D. Clifton. 2018. *Hydrodynamics in the Morganza Floodway and Atchafalaya Basin Report 2: Phase 2*. MRG&P Report No. 26. Vicksburg, MS: US Army Engineer Research and Development Center.

Copeland, R. R., and J. W. Lewis. 2022. *Technical Assessment of the Old, Mississippi, Atchafalaya, and Red (OMAR) Rivers: Mississippi River HEC-6T Model*. MRG&P Report No. 41; Vol. 4. Vicksburg, MS: US Army Engineer Research and Development Center.

Copeland, Ronald R., and William A. Thomas 1989. *Corte Madera Creek Sedimentation Study, Numerical Model Investigation*. Technical Report HL-89-6. Vicksburg MS: Hydraulics Laboratory, US Army Engineer Waterways Experiment Station.

Harris, K. E., and T. A. Dahl. 2022. *Technical Assessment of the Old, Mississippi, Atchafalaya, and Red (OMAR) Rivers: HEC-RAS BSTEM Analysis of the Atchafalaya River*. MRG&P Report No. 41; Vol. 9. Vicksburg, MS: US Army Engineer Research and Development Center.

Krone, R. 1962. *Flume Studies of the Transport of Sediment in Estuarial Shoaling Process*. Berkeley, CA: Hydraulic Engineering Laboratory and Sanitary Engineering Research Laboratory, University of California.

Lauth, T. J., D. S. Biedenharn, T. A. Dahl, C. M. Mayne, K. E. Jones, C. D. Little, Jr., J. B. Dunbar, S. L. Lucker, and N. Torres. 2022. *Old River, Mississippi River, Atchafalaya River, and Red River (OMAR) System Technical Assessment: Geomorphic Assessment*. MRG&P Report No. 41; Vol. 2. Vicksburg, MS: US Army Engineer Research and Development Center.

Lewis, James W., and Ronald R. Copeland. 2018. *Mississippi River and Tributaries Flowline Assessment: Atchafalaya River Sedimentation Model Report*. MRG&P Report No. 24, Vol. 5. Vicksburg, MS:US Army Engineer Research and Development Center.

Maynord, S. T. 2014. *Scour Protection Downstream of Morganza Control Structure, Morganza, Louisiana*. ERDC/CHL TR-14-1. Vicksburg, MS: US Army Engineer Research and Development Center. https://erdc-library.erdc.dren.mil/xmlui/handle/11681/73.

Nordin, Carl F., R. E. Rentschler, and F. L. Ogden, 1995. *Particle Size Distribution of Bed Sediments along the Thalweg of the Atchafalaya River, Old River to the Gulf, November 1991*. Ft. Collins, CO: Engineering Research Center, Colorado State University.

Norton, K. K., L. D. Olsen, T. E. Baumann, L. B. Simmons, A. P. Clark, D. K. Demcheck, and M. Johnson. 2019. *Revisions to Suspended-Sediment Concentration, Percent Smaller than 0.063 Millimeter, and Instantaneous Suspended-Sediment Discharge Reported for a Cooperative Program between the US Geological Survey and the US Army Corps of Engineers in the Lower Mississippi-Atchafalaya River Basin, October 1989 to February 2015*. US Geological Survey Scientific Investigations Report 2018–5147. Reston, VA: USGS. https://doi.org/10.3133/sir201.

Parthenaides, E. 1965. "Erosion and Deposition of Cohesive Soils." *Journal of the Hydraulics Division, ASCE* 91(1): 105–139. http://cedb.asce.org/CEDBsearch/record.jsp?dockey=0013.

Savant, G., G. L. Brown, and S. Ayres. 2022. *Technical Assessment of the Old, Mississippi, Atchafalaya, and Red (OMAR) Rivers: Mississippi River Multi-Dimensional Model*. MRG&P Report No. 41; Vol. 6. Vicksburg, MS: US Army Engineer Research and Development Center.

Thomas, William A. 2021. *Sedimentation in Stream Networks (HEC-6T)*. Hot Springs, AR: Mobile Boundary Hydraulics. mbh20@aol.com.

Thomas, William A., and Howard Chang. 2008. "Computational Modeling of Sedimentation Processes. Chapter 14. *Sedimentation Engineering*. ASCE Manuals and Reports on Engineering Practice No. 110, edited by Marcelo H. Garcia. Reston, VA: ASCE.

Thomas, William A., Ronald E. Heath, Douglas Clarke, and Phillip Stewart. 1988. *The Atchafalaya River Delta, Quasi Two-Dimensional Model of Delta Growth and Impacts on River Stages*. Technical Report HL-82-1, Report 5 of a Series. Vicksburg, MS: US Army Engineer Waterway Experiment Station, Hydraulics Laboratory.

USACE (US Army Corps of Engineers). 2013. *Incorporating Sea Level Change in Civil Works Programs*. ER 1100-2-8162. Washington, DC: US Army Corps of Engineers.

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: US Army Corps of Engineers, Hydrologic Engineering Center.

USACE-HEC. 2020. *HEC-RAS Sediment Transport, Users Manual, Version 6*. Davis CA: US Army Corps of Engineers, Hydrologic Engineering Center. https://www.hec.usace.army.mil/software/hec-ras/documentation/HEC-RAS_6.0_1D_Sediment_User_Manual.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, November. https://erdc-library.erdc.dren.mil/xmlui/handle/ 11681/3188.\.

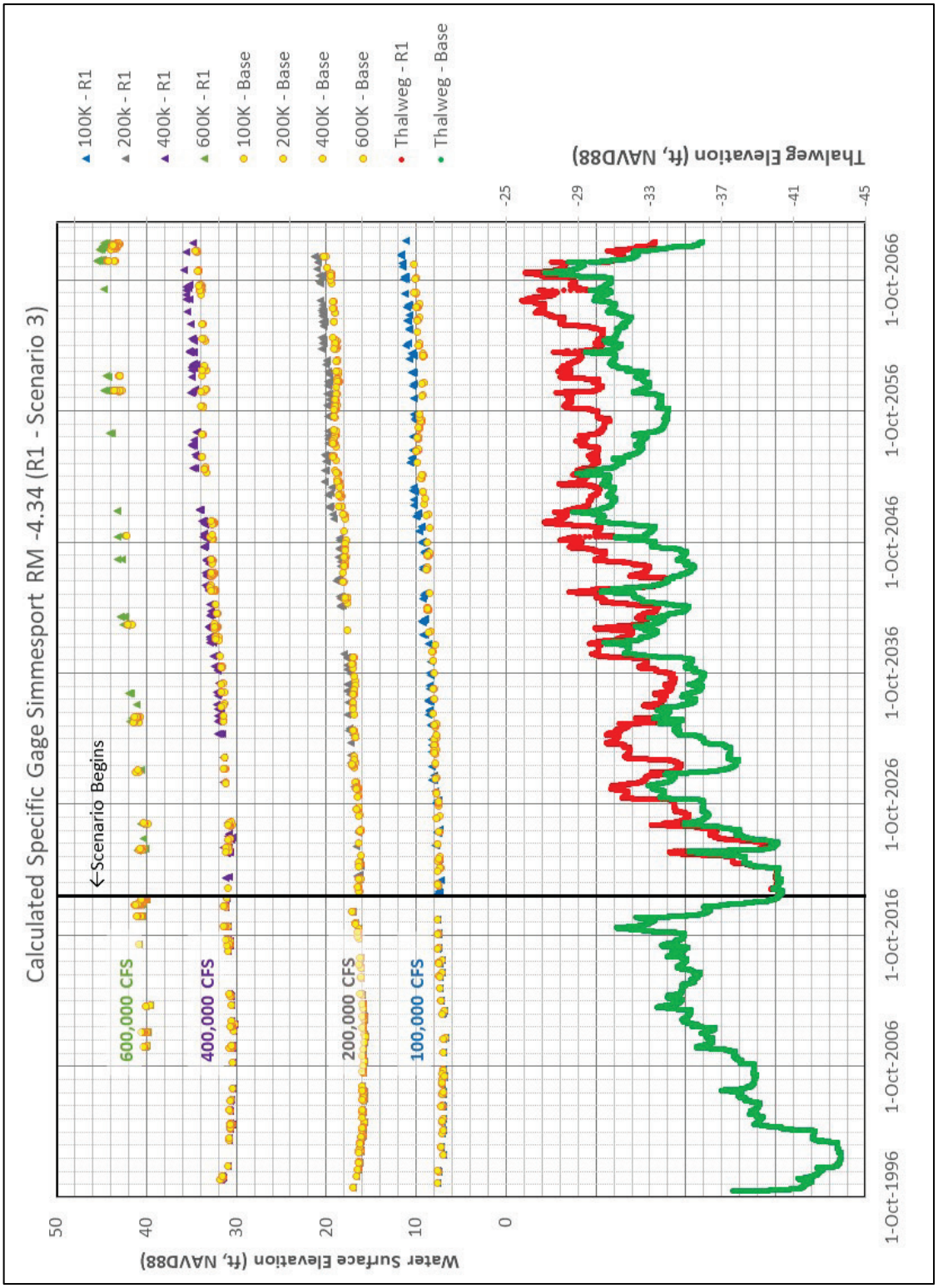
USACE MVN. 1988. *Flood Control, Mississippi River and Tributaries, Old River Complex*. Operation and Maintenance Manual. New Orleans, LA: US Army Engineer District.

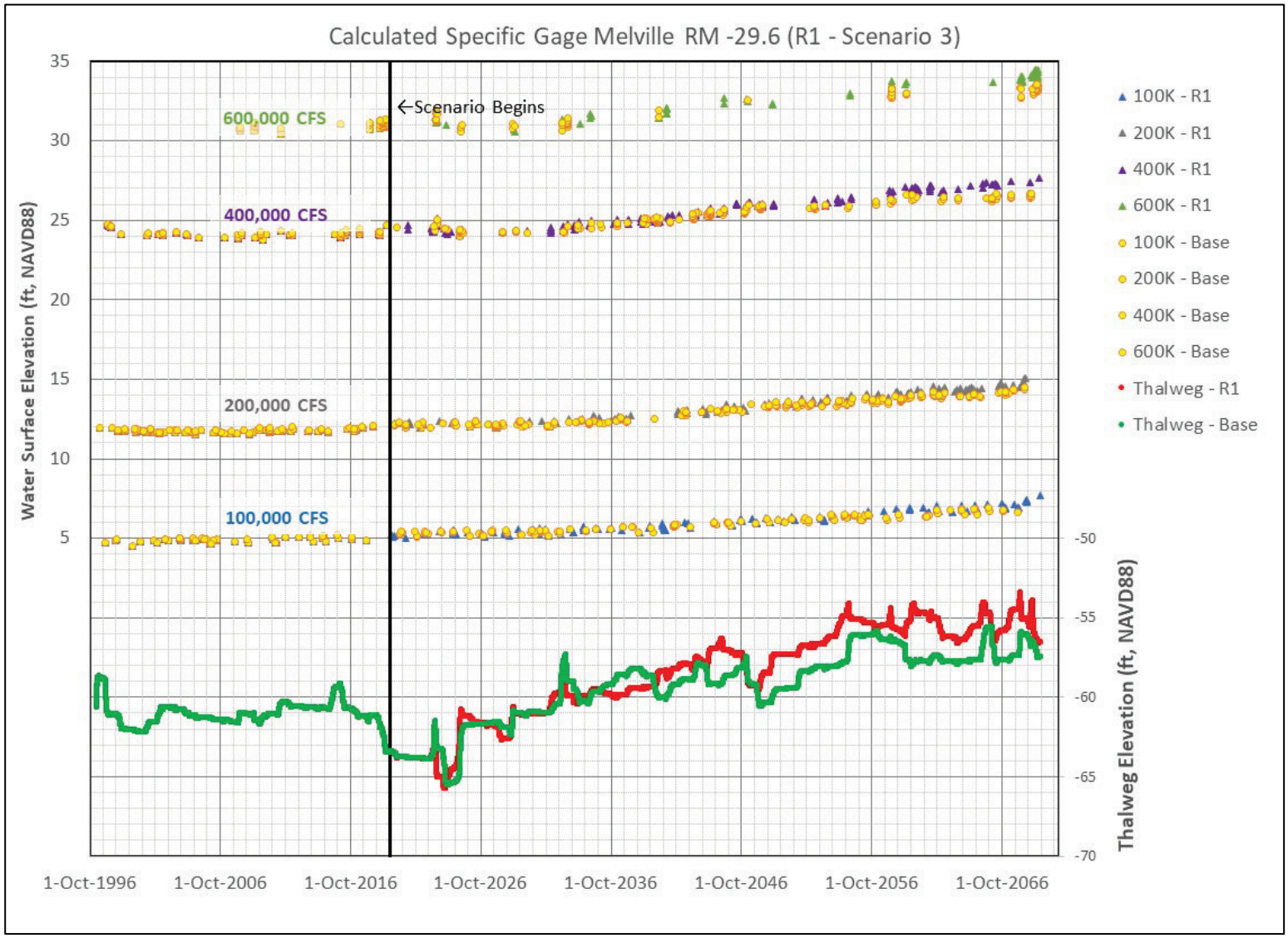
USACE MVN. 2005. *Lower Atchafalaya River and Bayous Chene-Boeuf-Black Feasibility Study Sediment Analysis*. Prepared for US Army Engineer District, New Orleans by US Army Engineer District, Vicksburg.

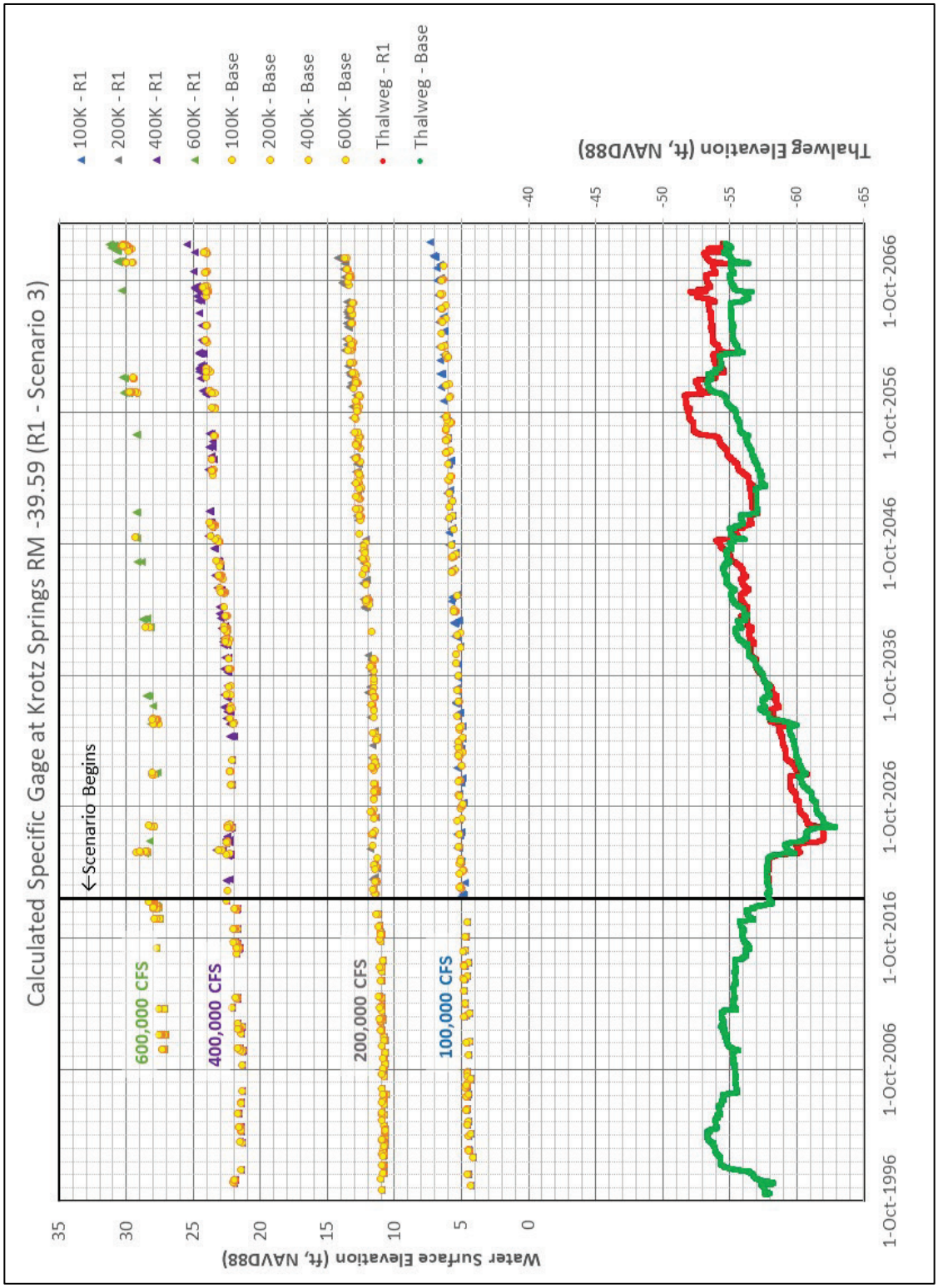
USACE MVN. 2010. *Mississippi River and Tributaries, Project Design Flood Flowline - 2010 HEC-6T Sediment Modeling Atchafalaya Basin, Louisiana*. Technical Appendix Volume 3. US Army Engineer District, New Orleans, LA. Prepared by US Army Engineer District, Rock Island, IL.

Appendix: Scenario-Specific Gage Plates

R1 Flow Regime – Scenario 3




Calculated Specific Gage Melville RM -29.6 (R1 - Scenario 3)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
600,000 CFS
400,000 CFS
200,000 CFS
100,000 CFS
35
30
25
20
15
10
5
-50
-55
-60
-65
-70
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066
100K - R1
200K - R1
400K - R1
600K - R1
100K - Base
200K - Base
400K - Base
600K - Base
Thalweg - R1
Thalweg - Base


Calculated Specific Gage at Krotz Springs RM -39.59 (R1 - Scenario 3)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
600,000 CFS
400,000 CFS
200,000 CFS
100,000 CFS
100K - R1
200K - R1
400K - R1
600K - R1
100K - Base
200k - Base
400k - Base
600K - Base
Thalweg - R1
Thalweg - Base
35
30
25
20
15
10
5
0
-40
-45
-50
-55
-60
-65
1-Oct-1996
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1-Oct-2016
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1-Oct-2036
1-Oct-2046
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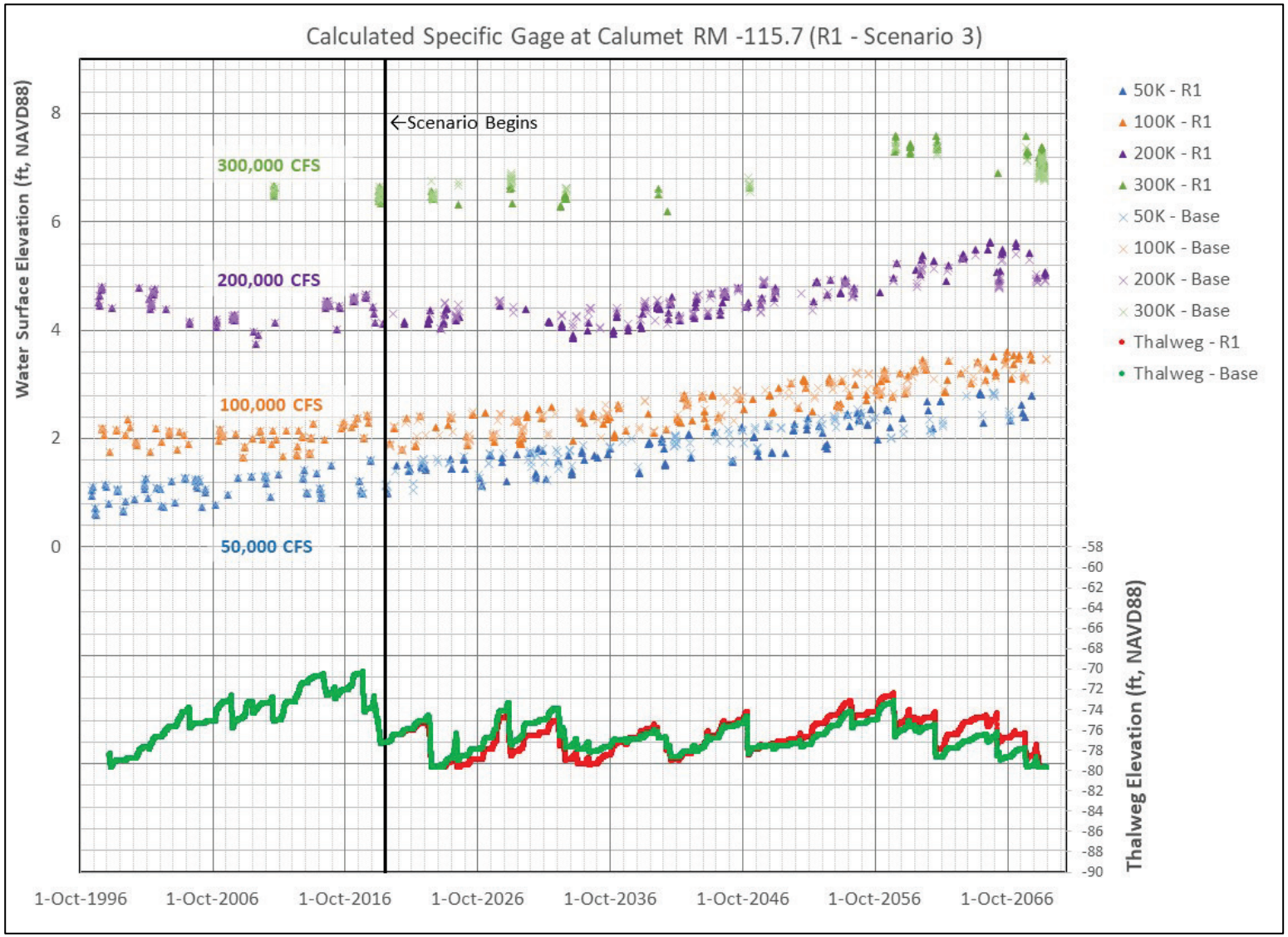
1-Oct-2066


Calculated Specific Gage at Calumet RM -115.7 (R1 - Scenario 3)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
300,000 CFS
200,000 CFS
100,000 CFS
50,000 CFS
50K - R1
100K - R1
200K - R1
300K - R1
50K - Base
100K - Base
200K - Base
300K - Base
Thalweg - R1
Thalweg - Base
8
6
4
2
0
-58
-60
-62
-64
-66
-68
-70
-72
-74
-76
-78
-80
-82
-84
-86
-88
-90
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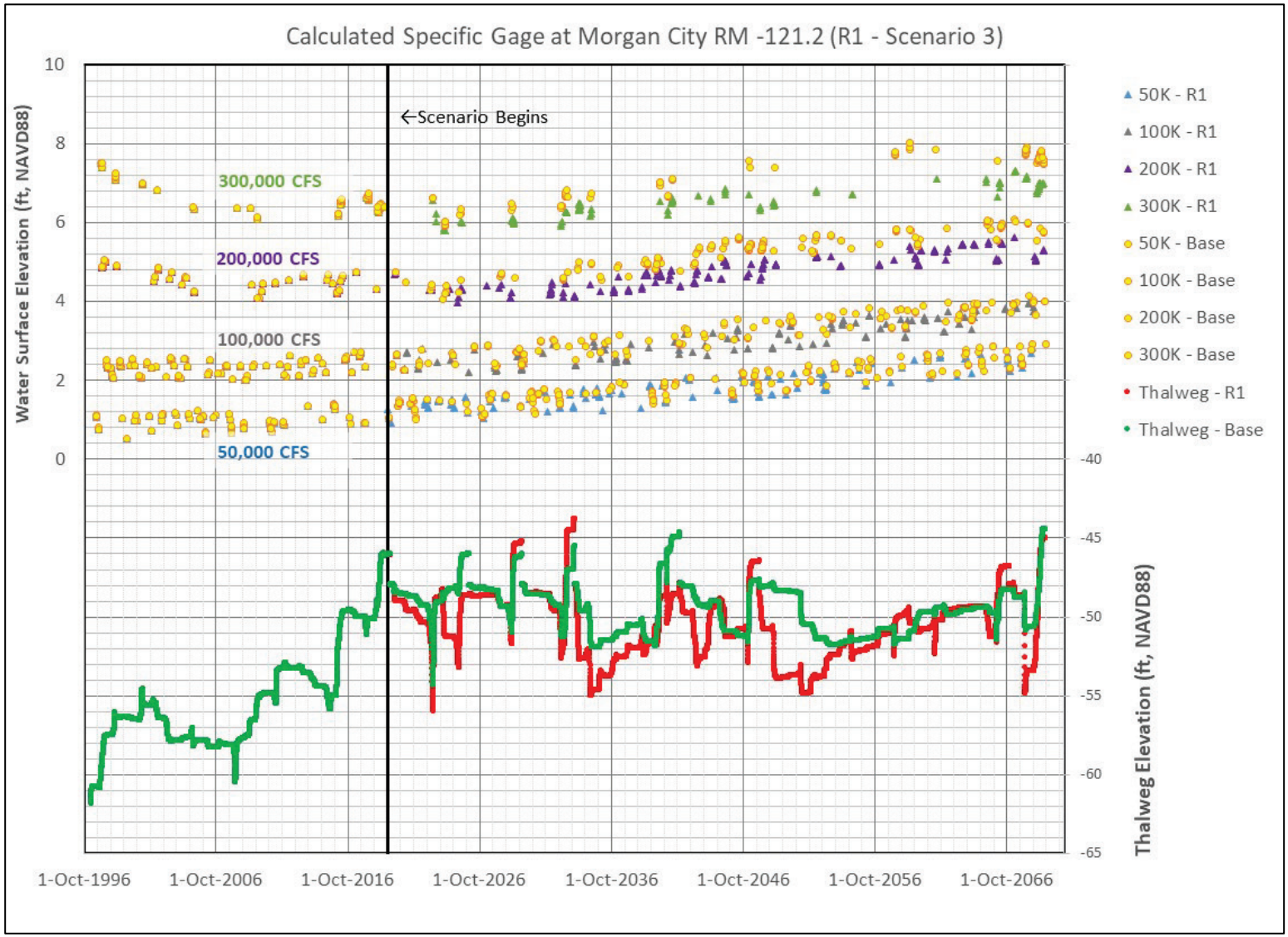
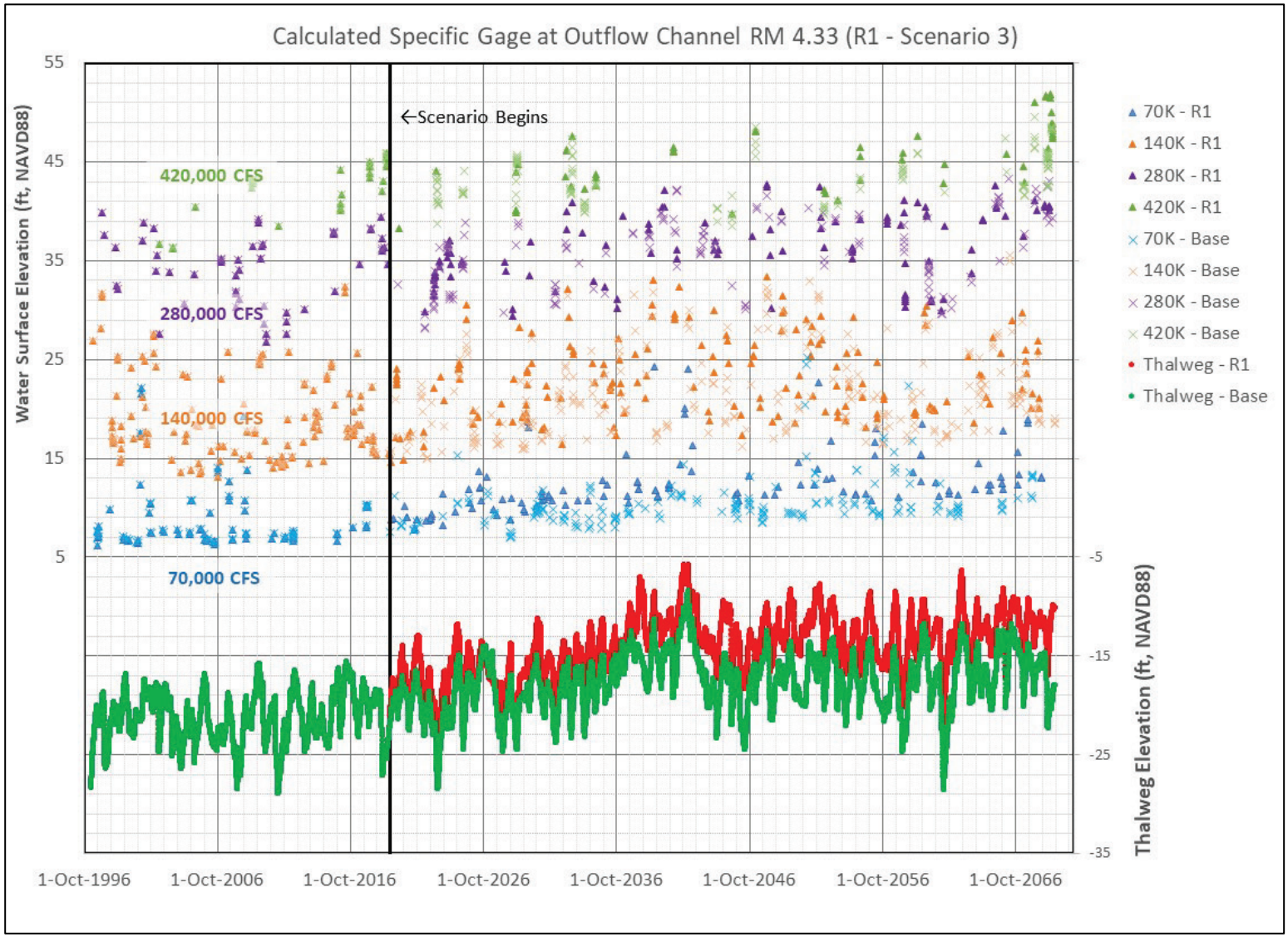
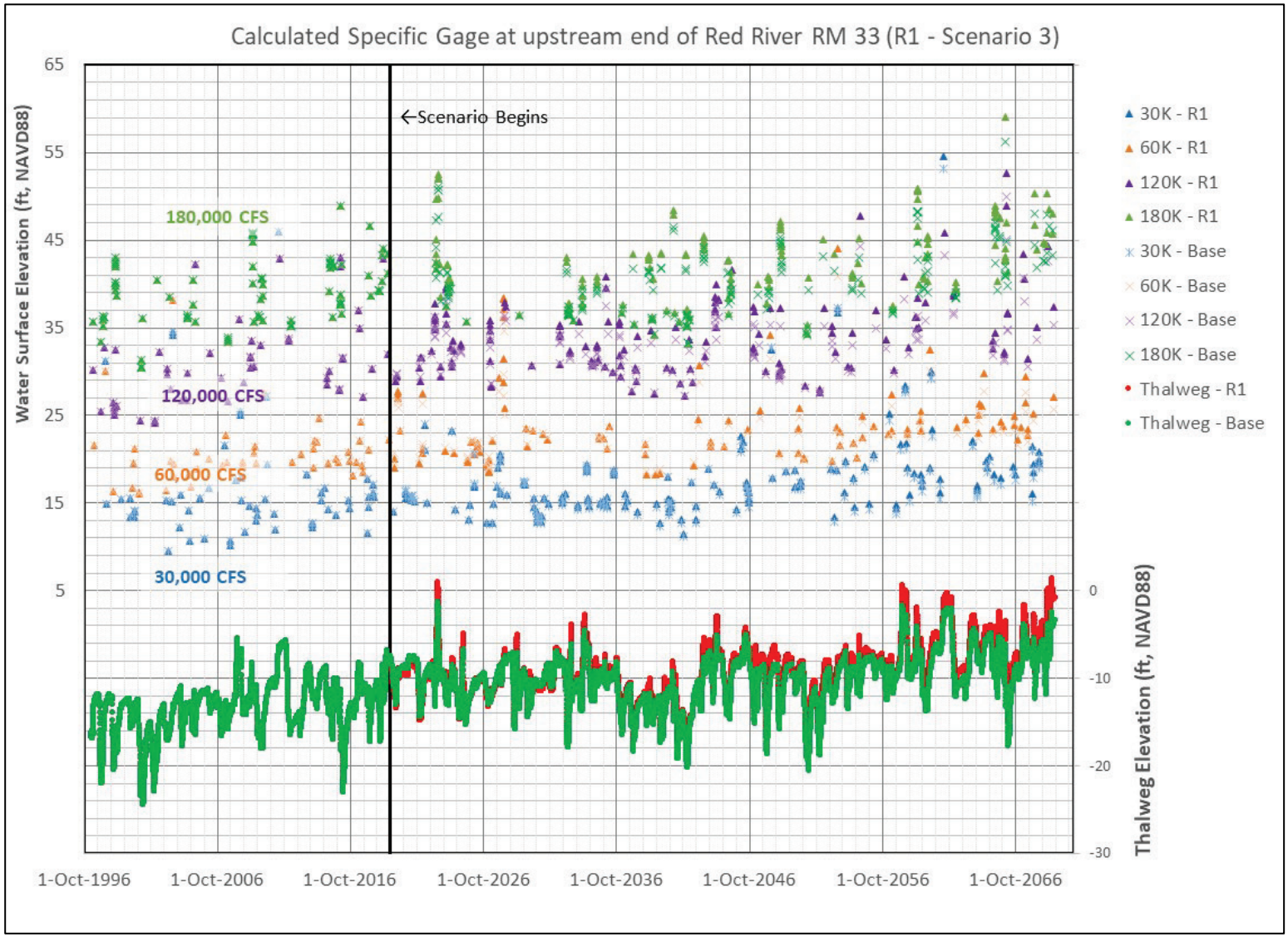
1-Oct-2066


Calculated Specific Gage at Morgan City RM -121.2 (R1 - Scenario 3)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
300,000 CFS
200,000 CFS
100,000 CFS
50,000 CFS
50K - R1
100K - R1
200K - R1
300K - R1
50K - Base
100K - Base
200K - Base
300K - Base
Thalweg - R1
Thalweg - Base
10
8
6
4
2
0
-40
-45
-50
-55
-60
-65
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at Outflow Channel RM 4.33 (R1 - Scenario 3)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
420,000 CFS
280,000 CFS
140,000 CFS
70,000 CFS
70K - R1
140K - R1
280K - R1
420K - R1
70K - Base
140K - Base
280K - Base
420K - Base
Thalweg - R1
Thalweg - Base
55
45
35
25
15
5
-5
-15
-25
-35
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at upstream end of Red River RM 33 (R1 - Scenario 3)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
180,000 CFS
120,000 CFS
60,000 CFS
30,000 CFS
30K - R1
60K - R1
120K - R1
180K - R1
30K - Base
60K - Base
120K - Base
180K - Base
Thalweg - R1
Thalweg - Base
65
55
45
35
25
15
5
0
-10
-20
-30
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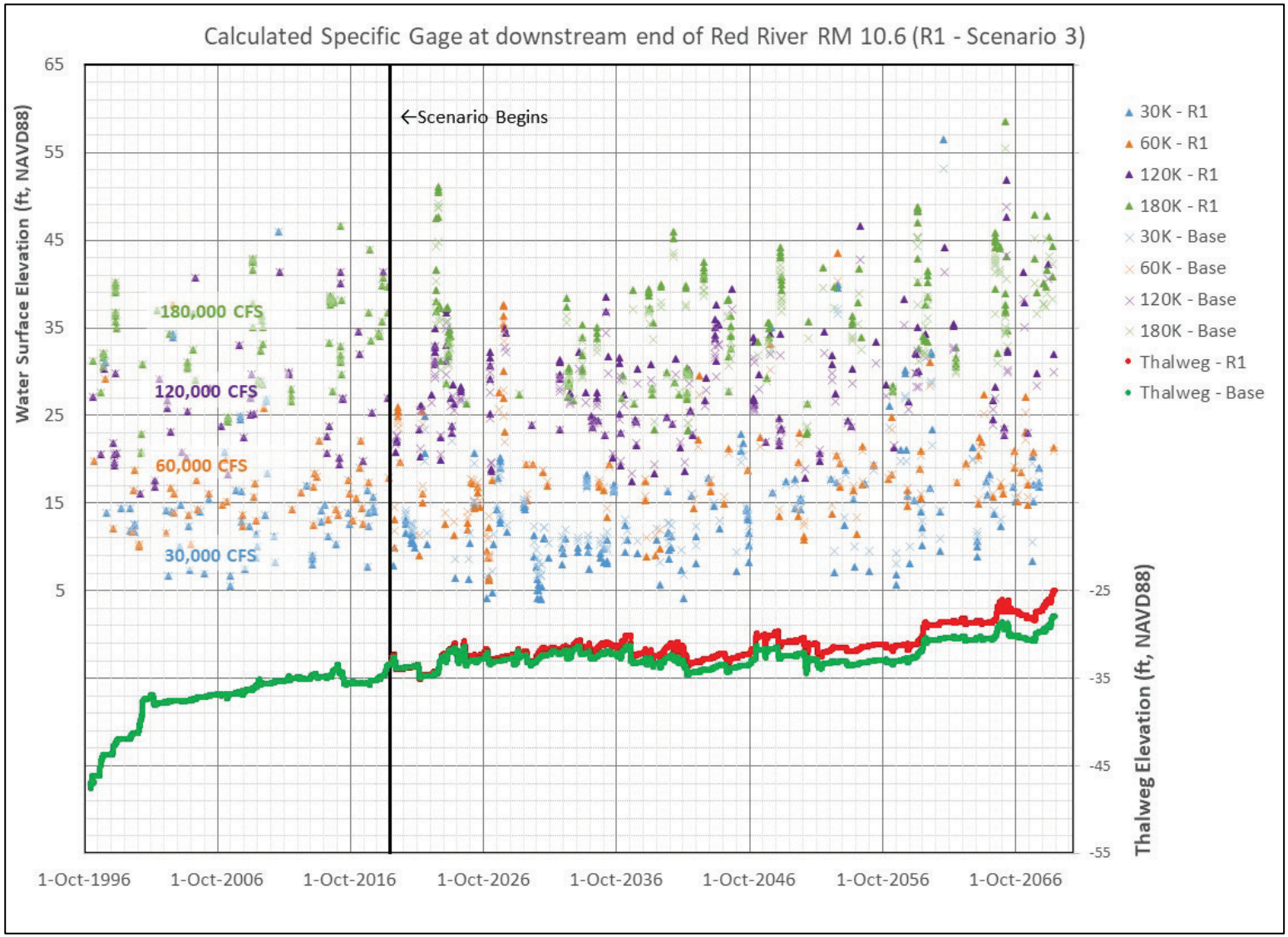
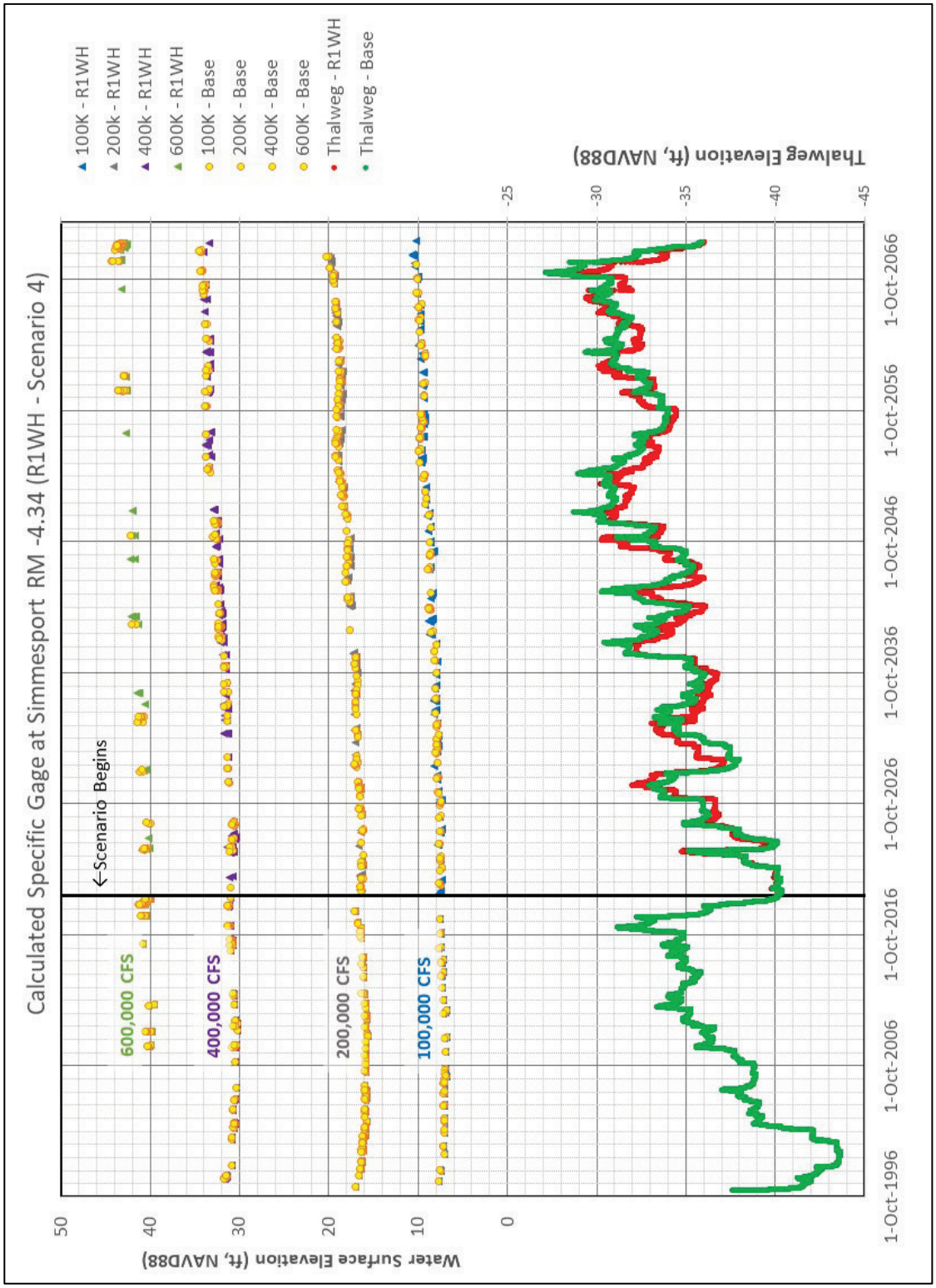
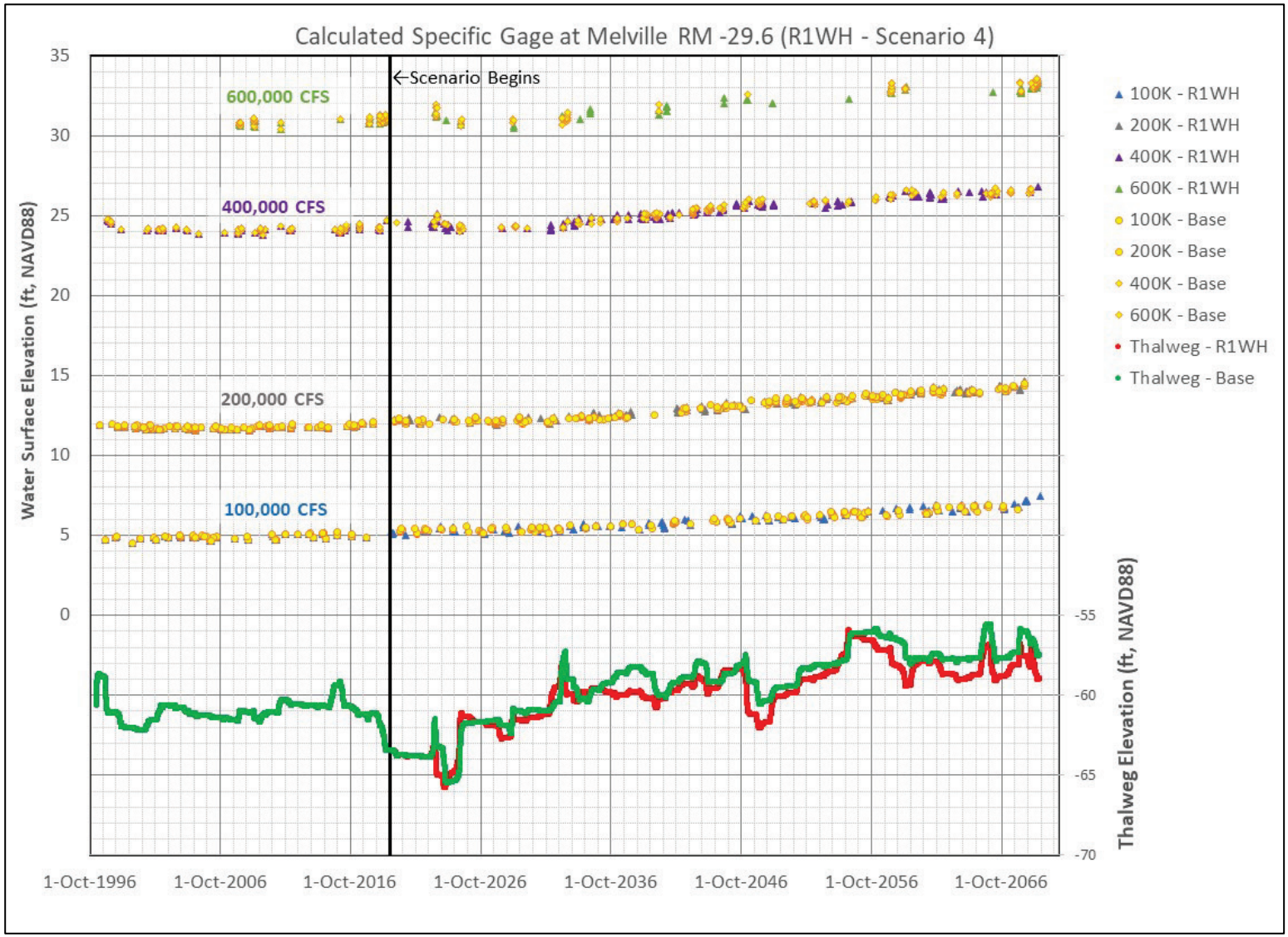
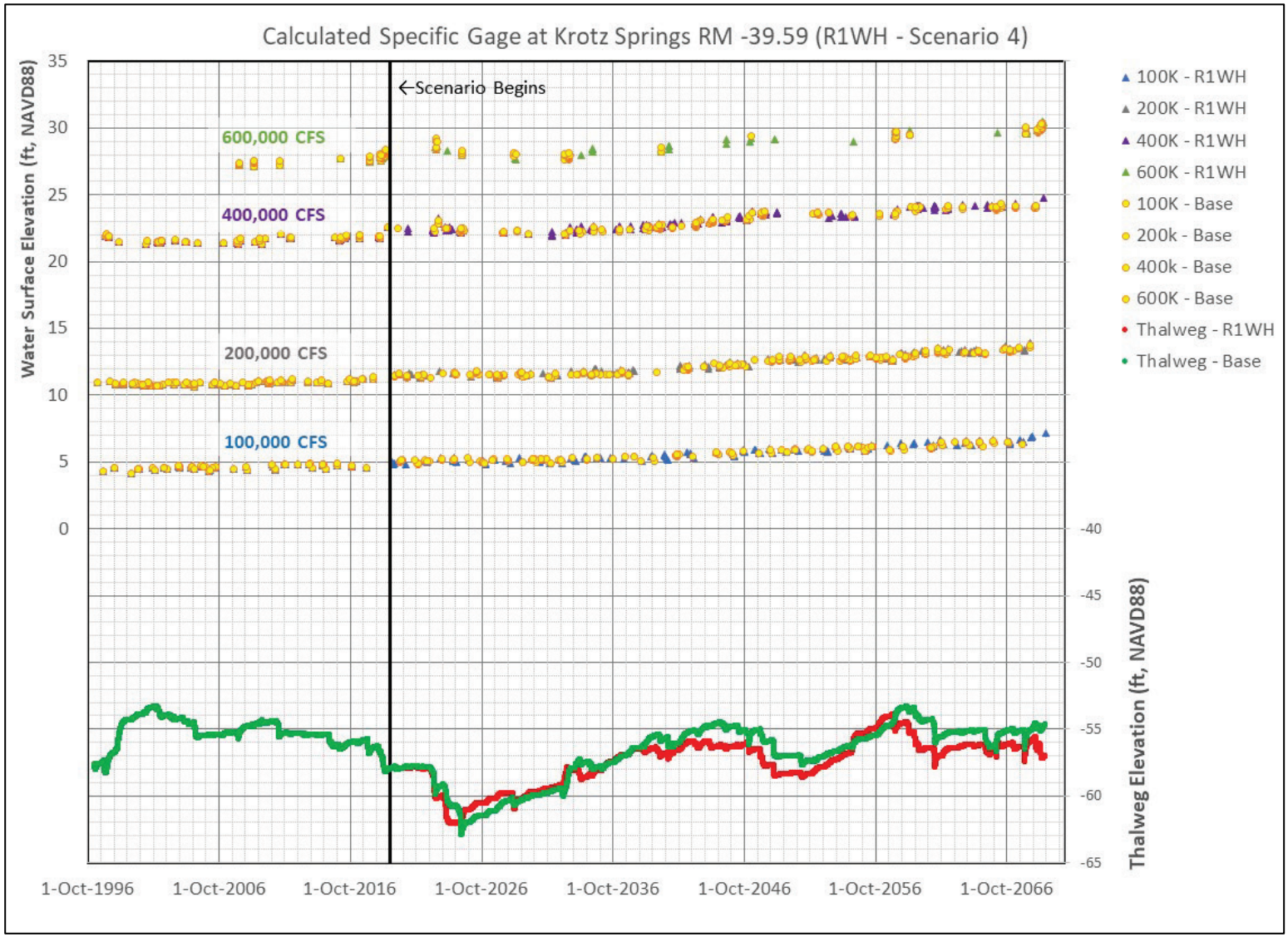
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at downstream end of Red River RM 10.6 (R1 - Scenario 3)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
180,000 CFS
120,000 CFS
60,000 CFS
30,000 CFS
30K - R1
60K - R1
120K - R1
180K - R1
30K - Base
60K - Base
120K - Base
180K - Base
Thalweg - R1
Thalweg - Base
65
55
45
35
25
15
5
-25
-35
-45
-55
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066

R1WH Flow Regime – Scenario 4




Calculated Specific Gage at Melville RM -29.6 (R1WH - Scenario 4)
←Scenario Begins
600,000 CFS
400,000 CFS
200,000 CFS
100,000 CFS
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
35
30
25
20
15
10
5
0
-55
-60
-65
-70
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066
100K - R1WH
200K - R1WH
400K - R1WH
600K - R1WH
100K - Base
200K - Base
400K - Base
600K - Base
Thalweg - R1WH
Thalweg - Base


Calculated Specific Gage at Krotz Springs RM -39.59 (R1WH - Scenario 4)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
600,000 CFS
400,000 CFS
200,000 CFS
100,000 CFS
100K - R1WH
200K - R1WH
400K - R1WH
600K - R1WH
100K - Base
200k - Base
400k - Base
600K - Base
Thalweg - R1WH
Thalweg - Base
35
30
25
20
15
10
5
0
-40
-45
-50
-55
-60
-65
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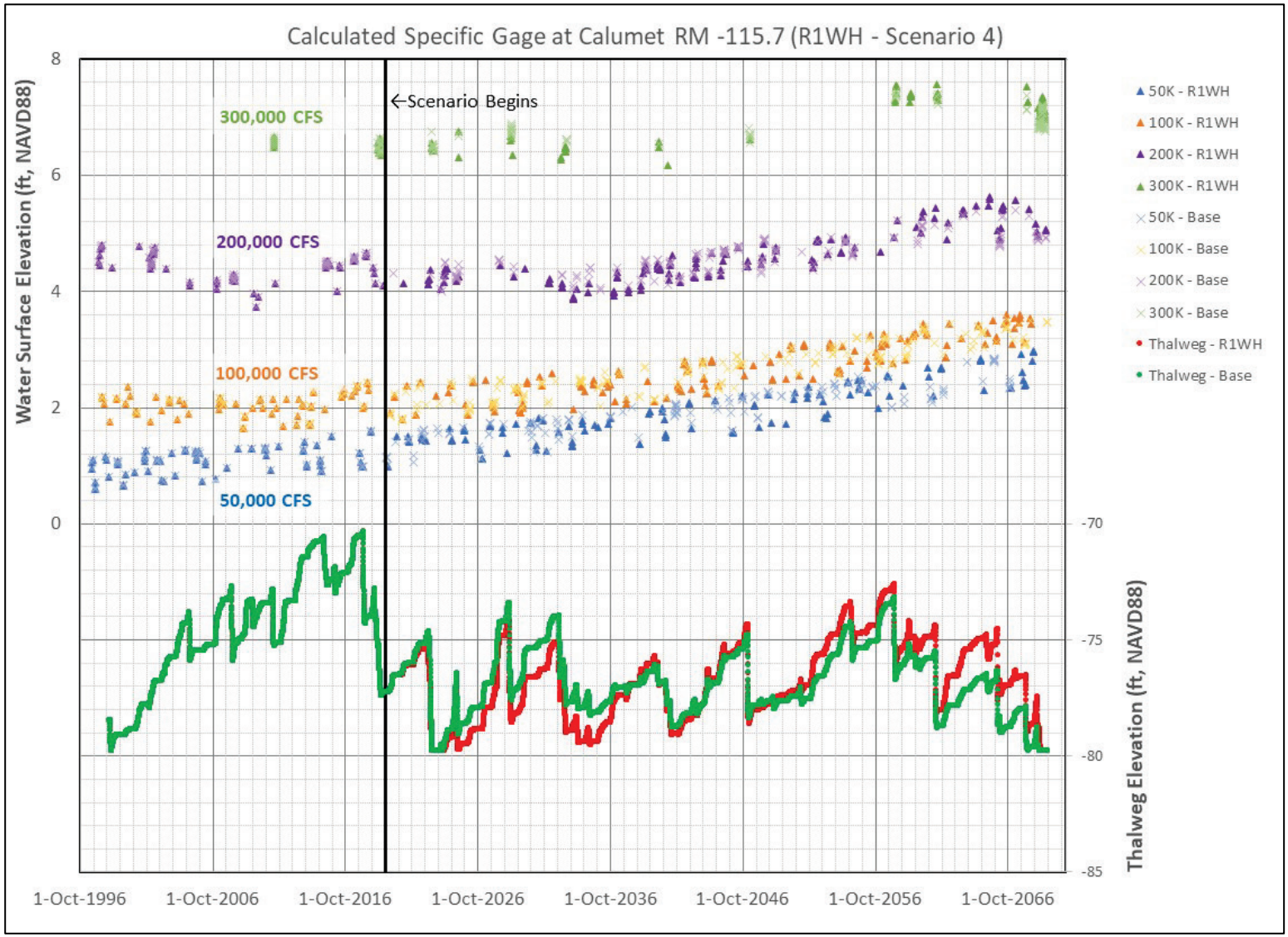
1-Oct-2066


Calculated Specific Gage at Calumet RM -115.7 (R1WH - Scenario 4)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
300,000 CFS
200,000 CFS
100,000 CFS
50,000 CFS
50K - R1WH
100K - R1WH
200K - R1WH
300K - R1WH
50K - Base
100K - Base
200K - Base
300K - Base
Thalweg - R1WH
Thalweg - Base
8
6
4
2
0
-70
-75
-80
-85
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
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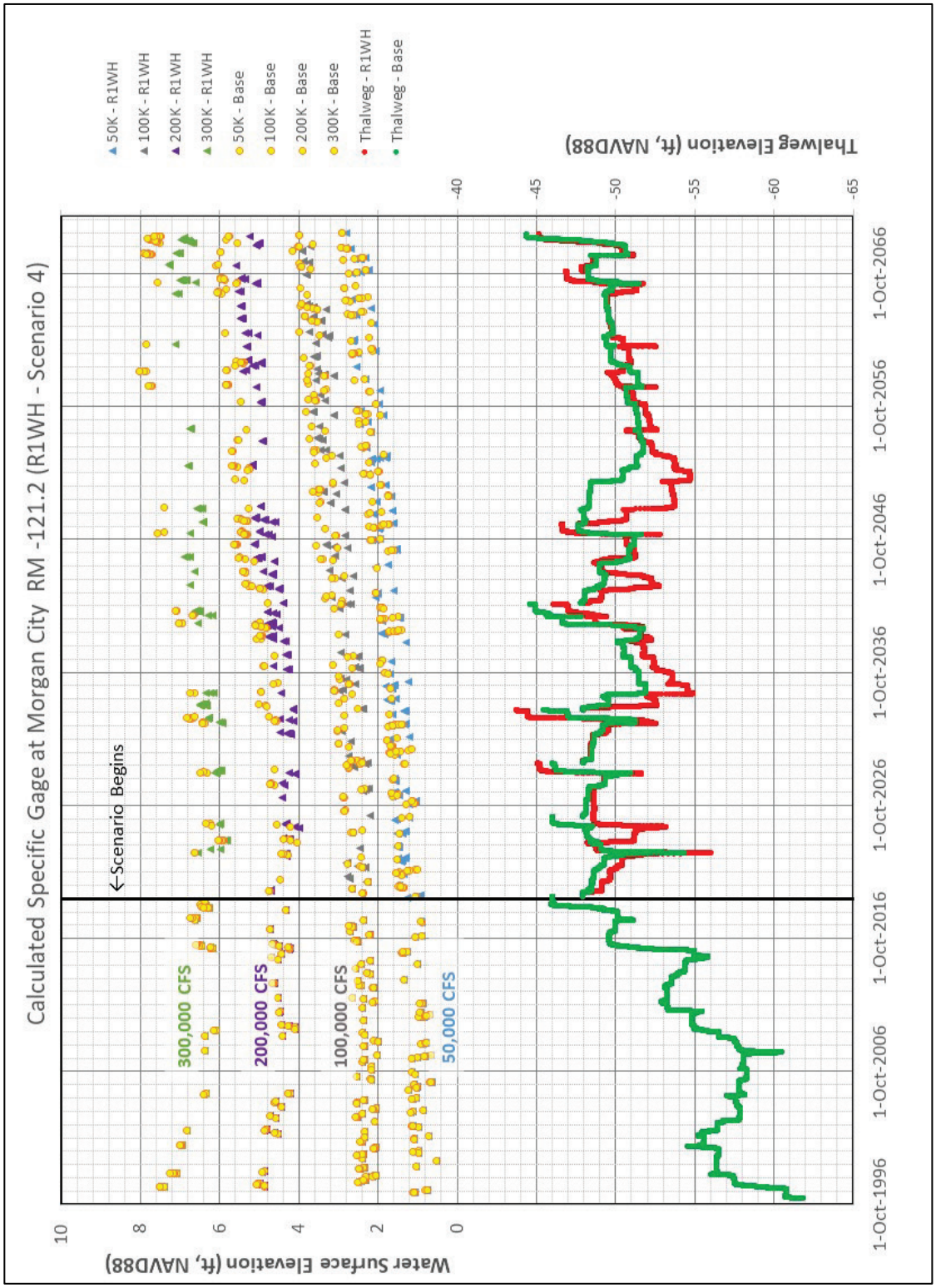
1-Oct-2066


Calculated Specific Gage at Morgan City RM -121.2 (R1WH - Scenario 4)
50K - R1WH
100K - R1WH
200K - R1WH
300K - R1WH
50K - Base
100K - Base
200K - Base
300K - Base
Thalweg - R1WH
Thalweg - Base
←Scenario Begins
300,000 CFS
200,000 CFS
100,000 CFS
50,000 CFS
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
10
8
6
4
2
0
-40
-45
-50
-55
-60
-65
1-Oct-1996
1-Oct-2006
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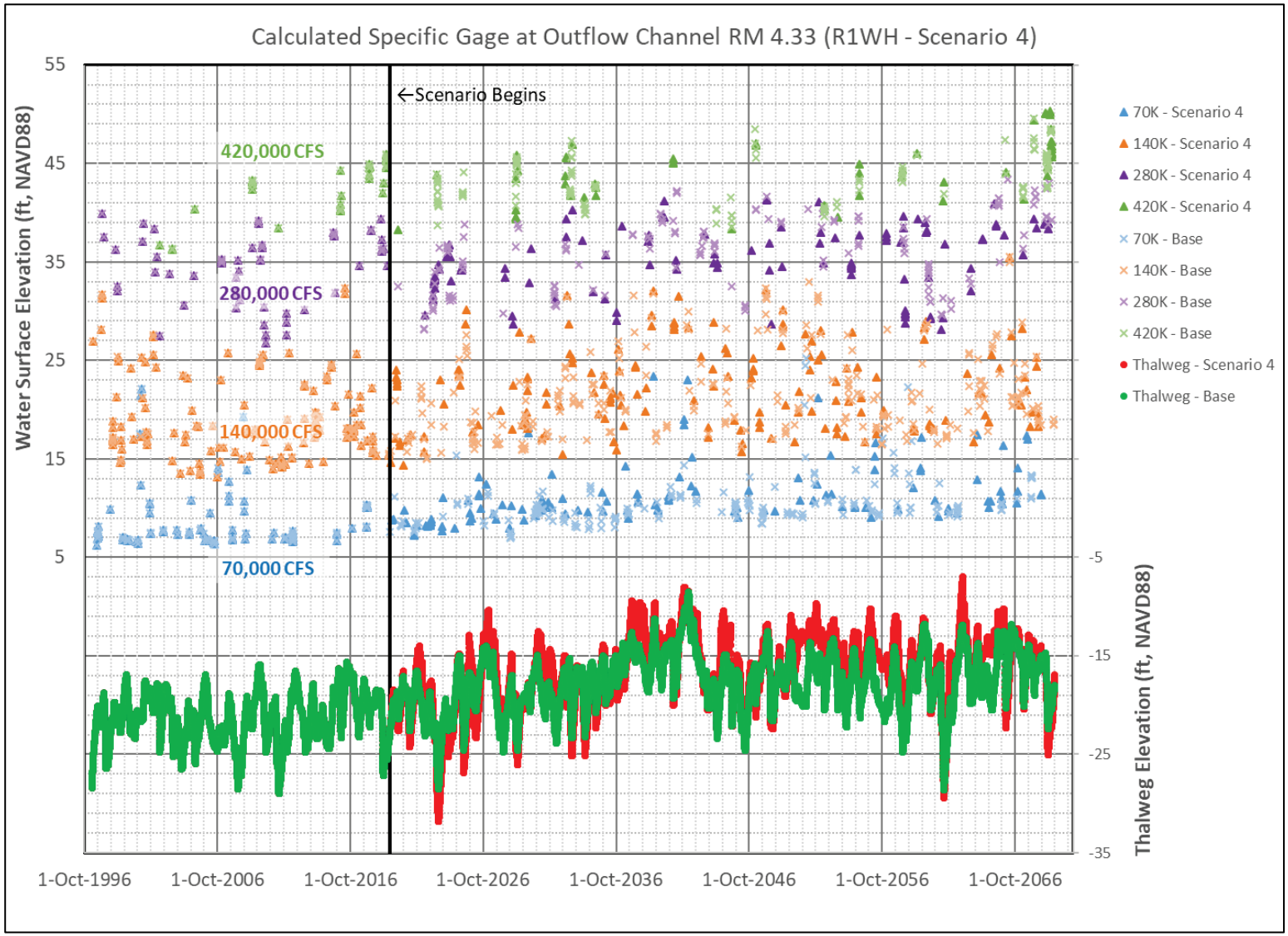
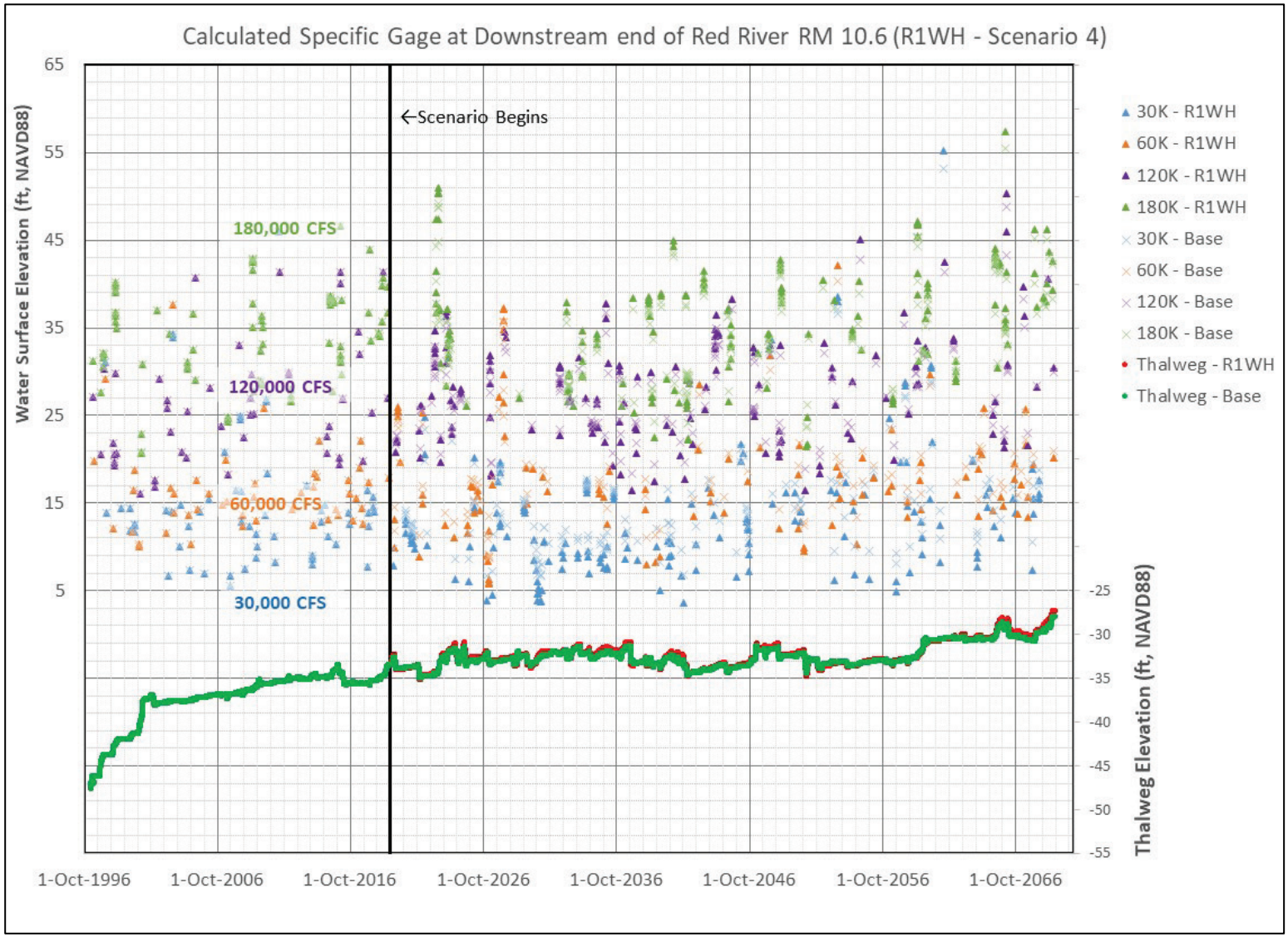
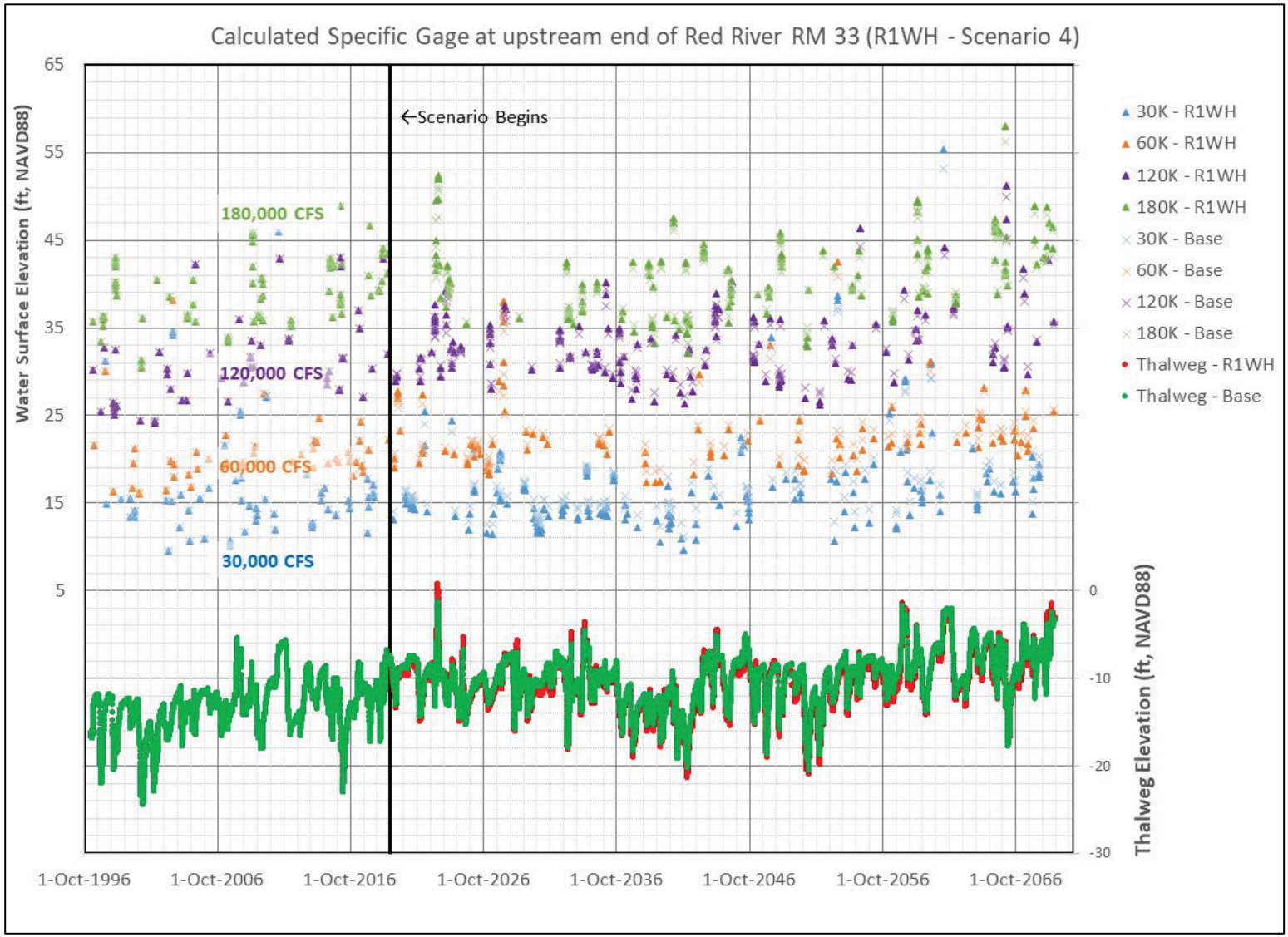
1-Oct-2066


Calculated Specific Gage at Outflow Channel RM 4.33 (R1WH - Scenario 4)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
420,000 CFS
280,000 CFS
140,000 CFS
70,000 CFS
70K - Scenario 4
140K - Scenario 4
280K - Scenario 4
420K - Scenario 4
70K - Base
140K - Base
280K - Base
420K - Base
Thalweg - Scenario 4
Thalweg - Base
55
45
35
25
15
5
-5
-15
-25
-35
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at Downstream end of Red River RM 10.6 (R1WH - Scenario 4)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
180,000 CFS
120,000 CFS
60,000 CFS
30,000 CFS
30K - R1WH
60K - R1WH
120K - R1WH
180K - R1WH
30K - Base
60K - Base
120K - Base
180K - Base
Thalweg - R1WH
Thalweg - Base
65
55
45
35
25
15
5
-25
-30
-35
-40
-45
-50
-55
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at upstream end of Red River RM 33 (R1WH - Scenario 4)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
180,000 CFS
120,000 CFS
60,000 CFS
30,000 CFS
30K - R1WH
60K - R1WH
120K - R1WH
180K - R1WH
30K - Base
60K - Base
120K - Base
180K - Base
Thalweg - R1WH
Thalweg - Base
65
55
45
35
25
15
5
0
-10
-20
-30
1-Oct-1996
1-Oct-2006
1-Oct-2016
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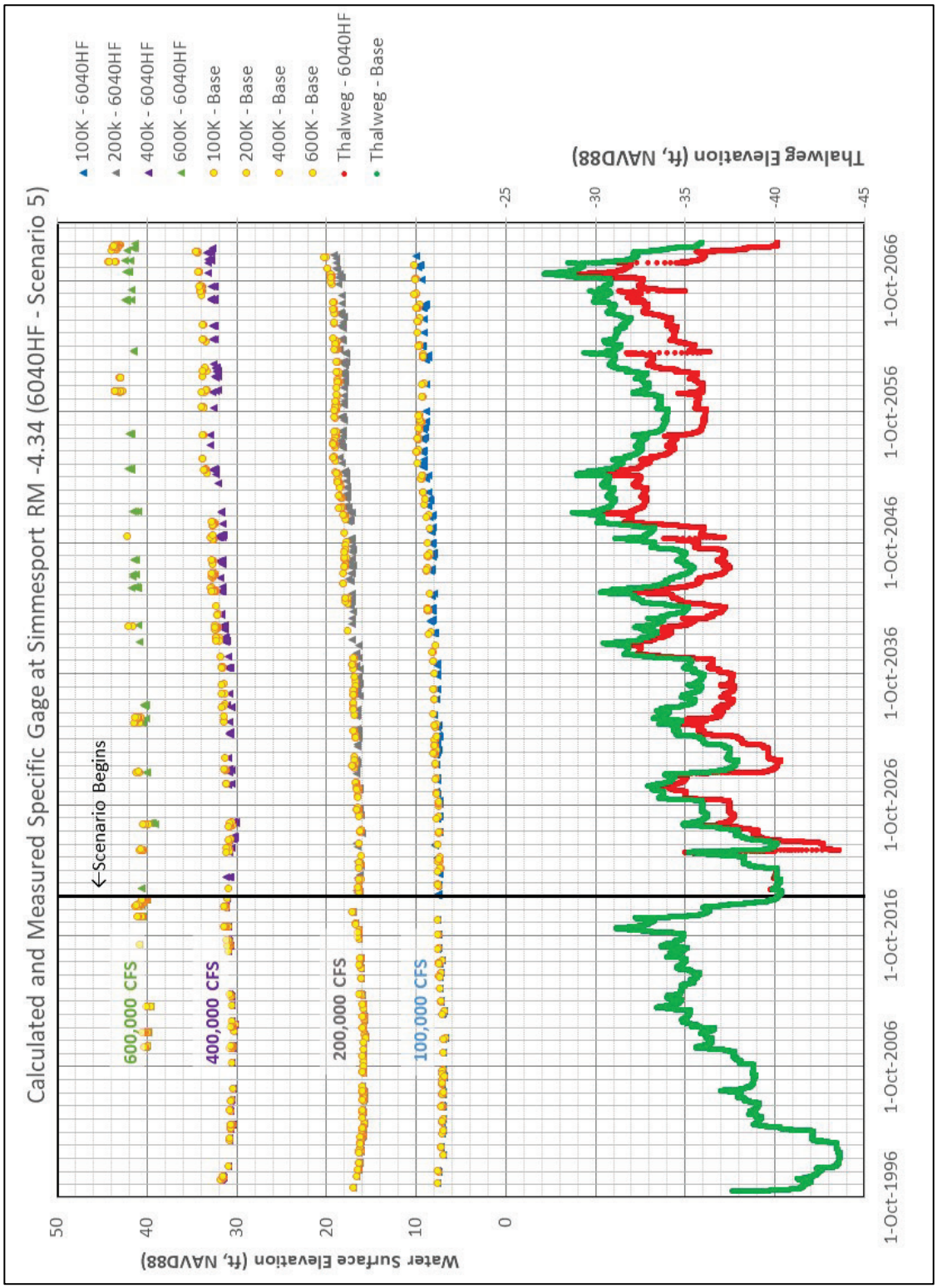
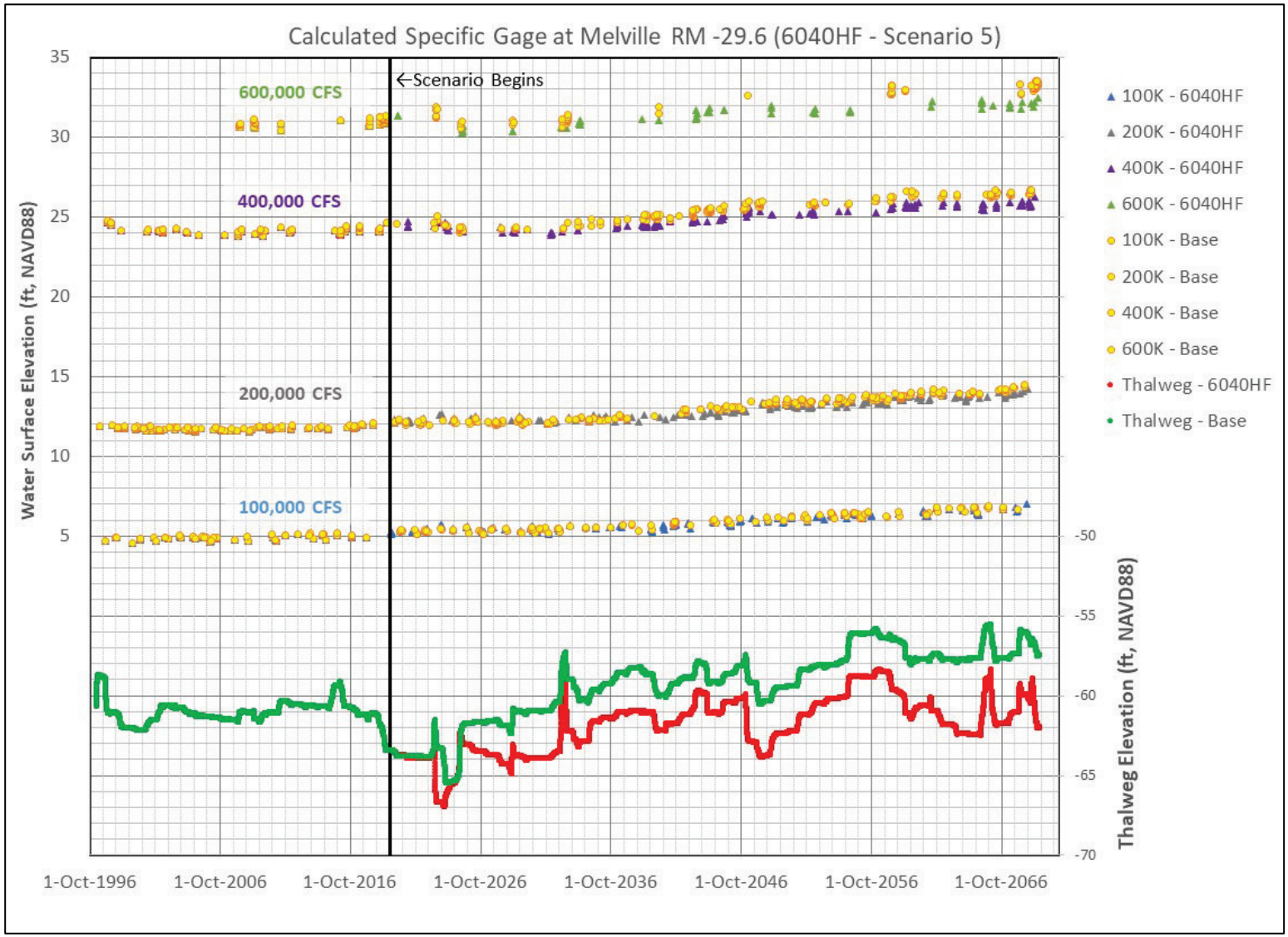
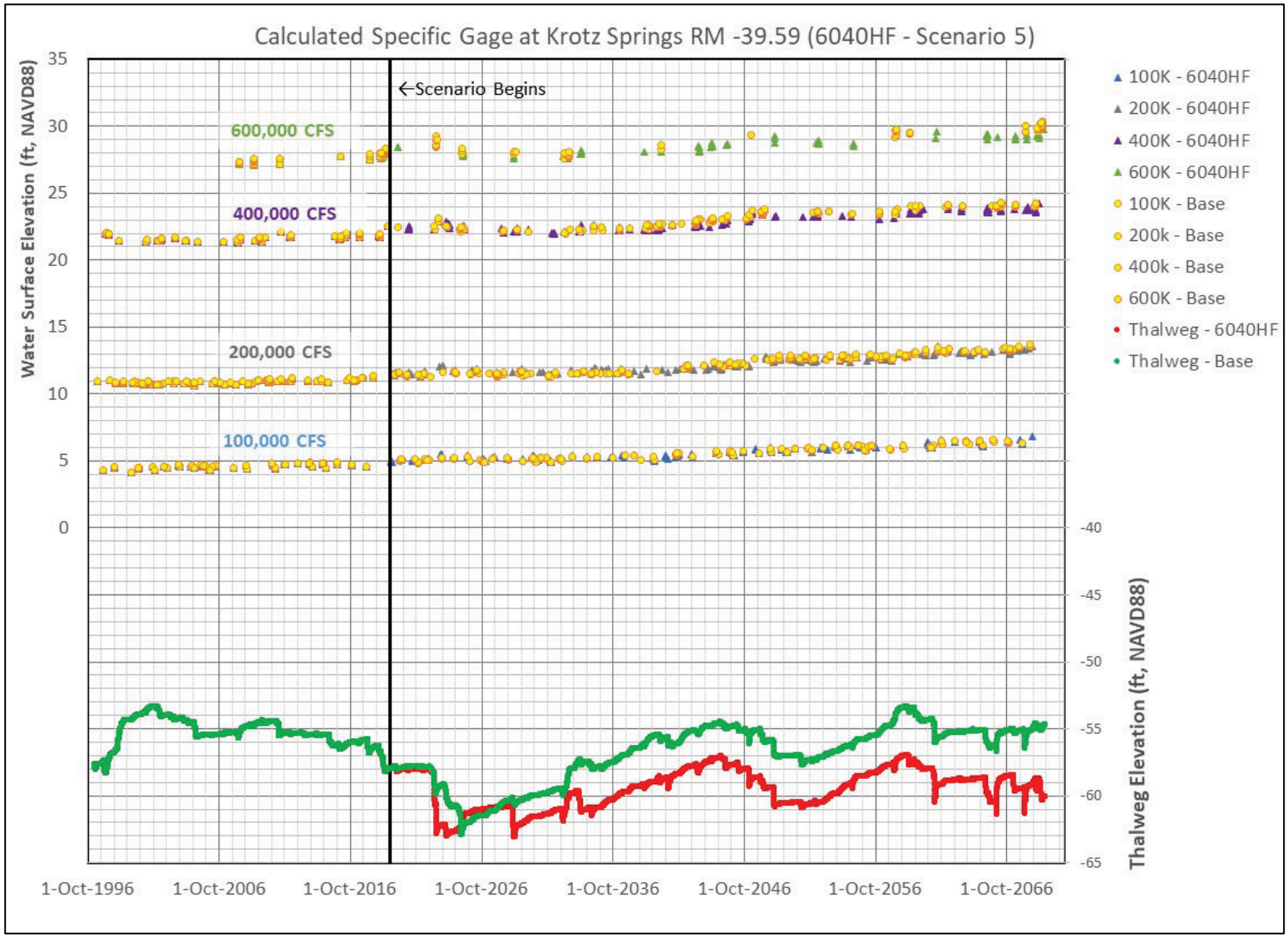
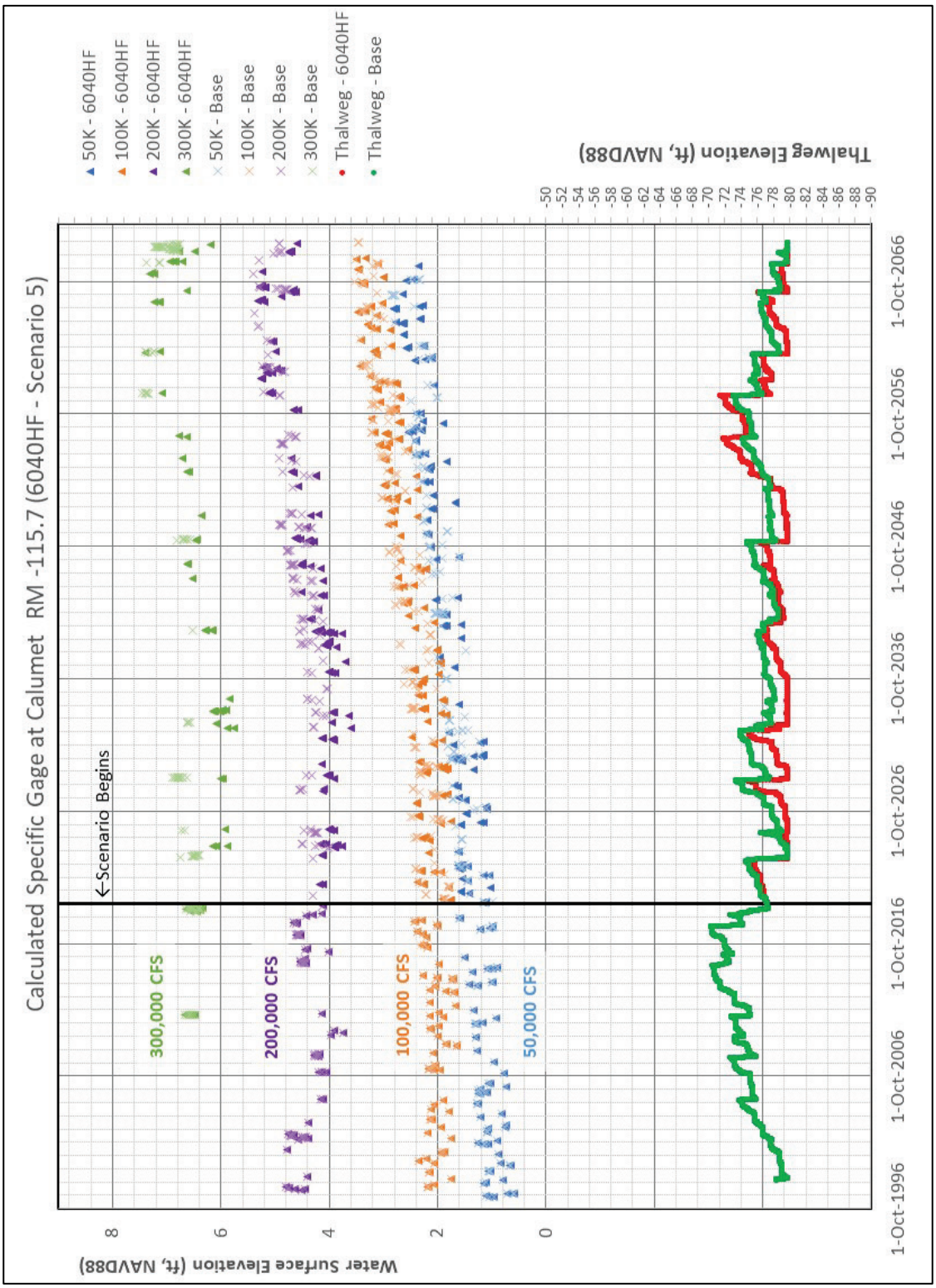
1-Oct-2056
1-Oct-2066

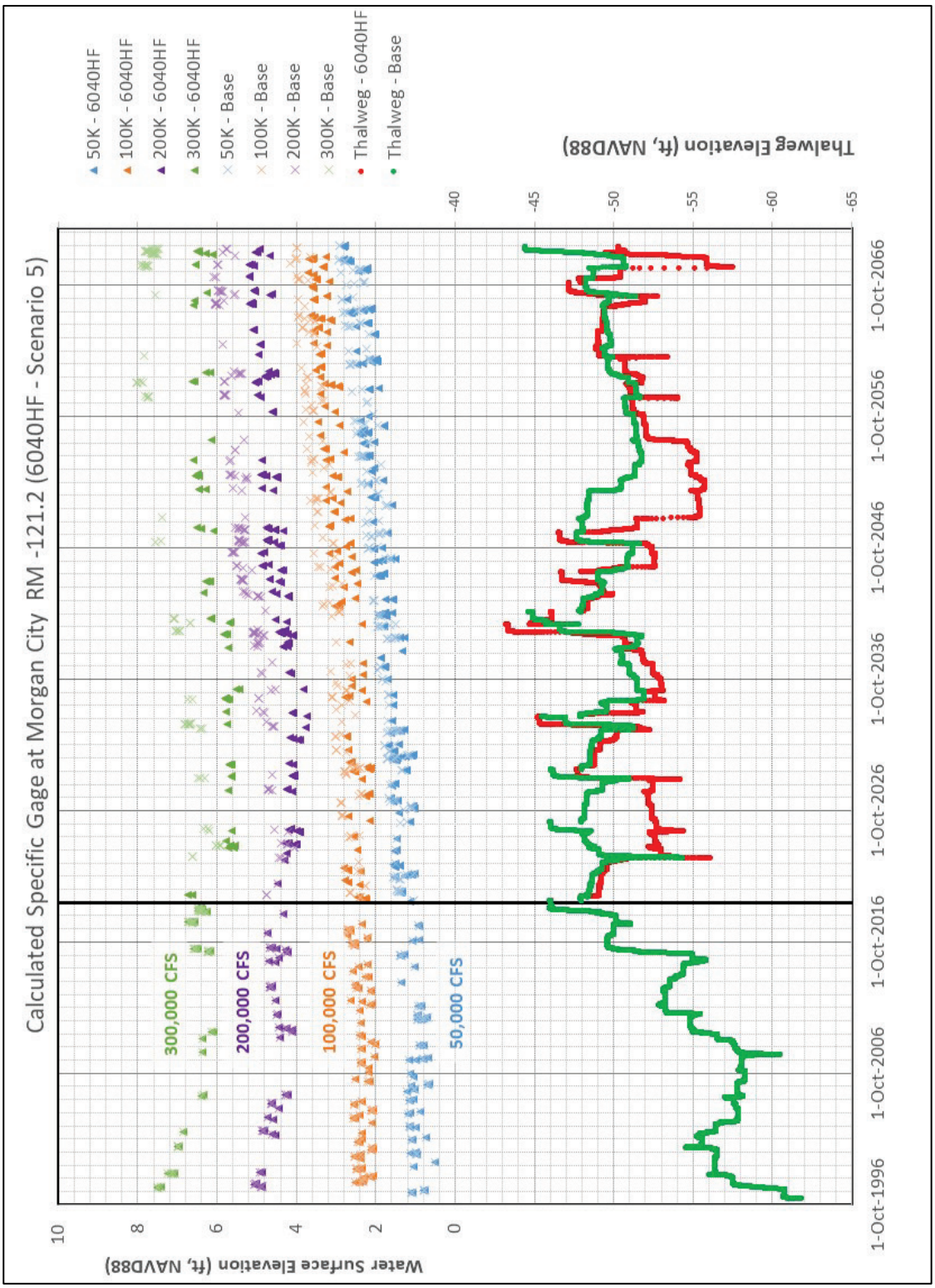
6040HF Flow Regime – Scenario 5

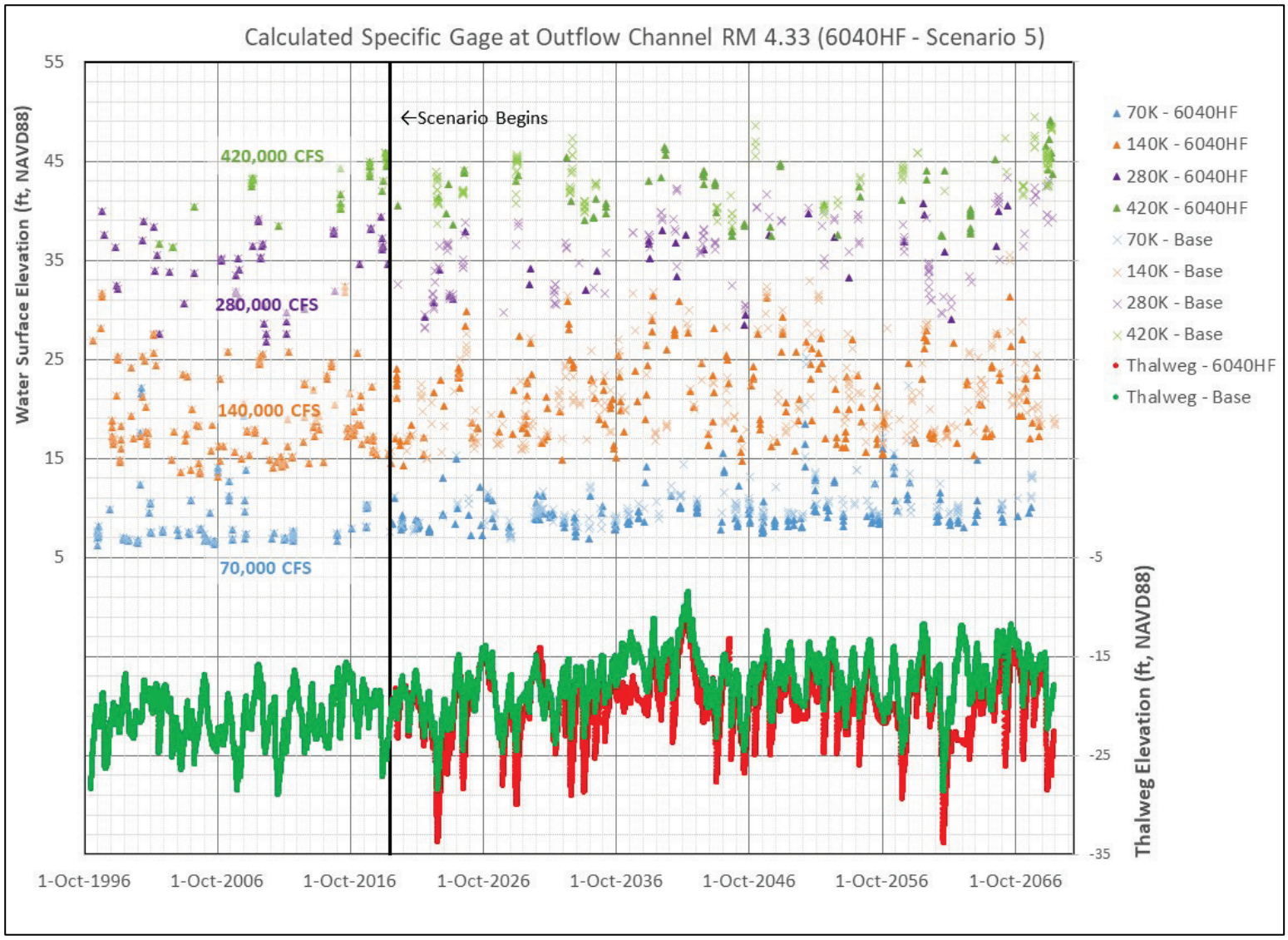


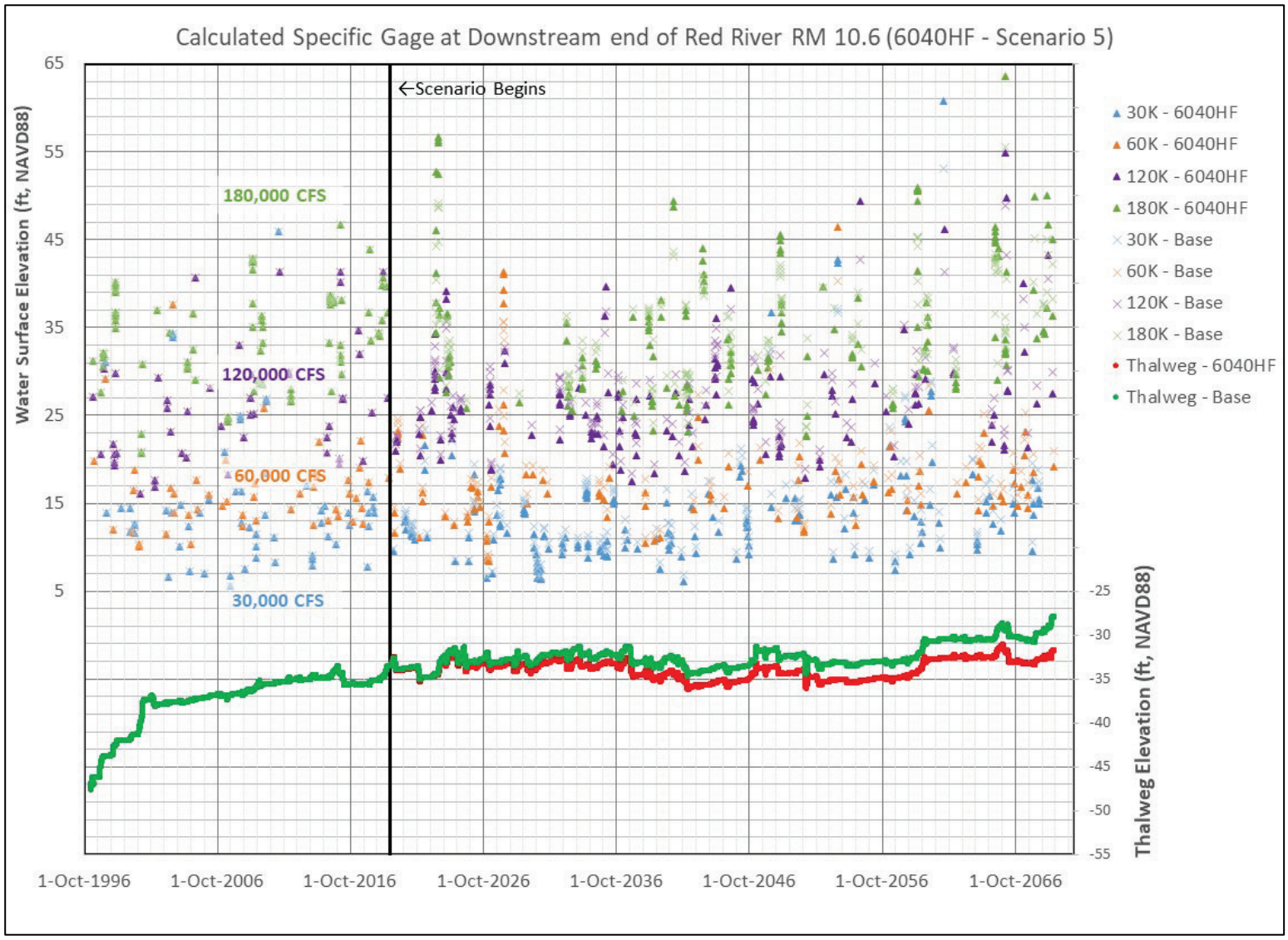

Calculated Specific Gage at Melville RM -29.6 (6040HF - Scenario 5)
←Scenario Begins
600,000 CFS
400,000 CFS
200,000 CFS
100,000 CFS
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
35
30
25
20
15
10
5
-50
-55
-60
-65
-70
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066
100K - 6040HF
200K - 6040HF
400K - 6040HF
600K - 6040HF
100K - Base
200K - Base
400K - Base
600K - Base
Thalweg - 6040HF
Thalweg - Base

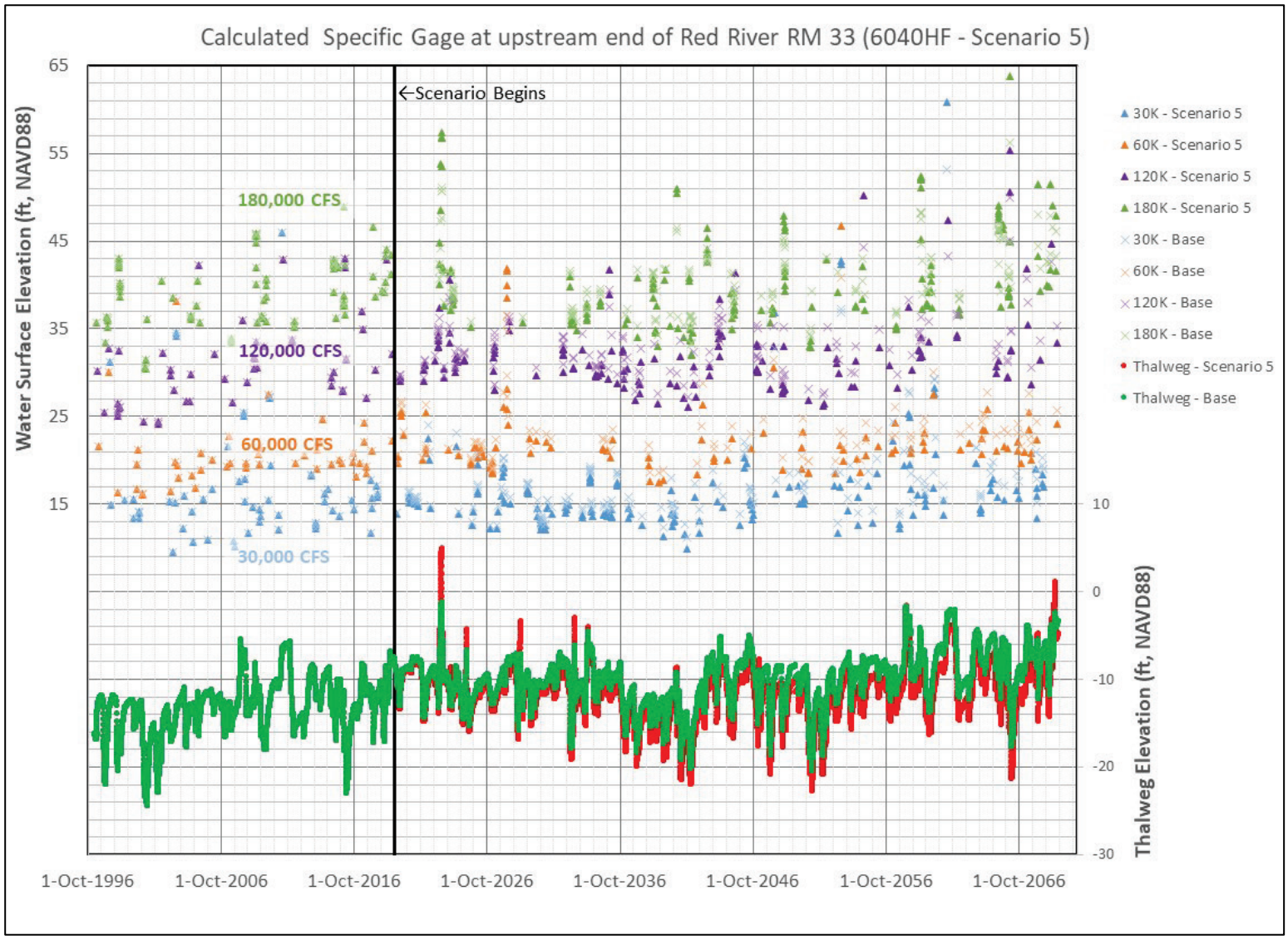

Calculated Specific Gage at Krotz Springs RM -39.59 (6040HF - Scenario 5)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
600,000 CFS
400,000 CFS
200,000 CFS
100,000 CFS
100K - 6040HF
200K - 6040HF
400K - 6040HF
600K - 6040HF
100K - Base
200k - Base
400k - Base
600K - Base
Thalweg - 6040HF
Thalweg - Base
35
30
25
20
15
10
5
0
-40
-45
-50
-55
-60
-65
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at Calumet RM -115.7 (6040HF - Scenario 5)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
300,000 CFS
200,000 CFS
100,000 CFS
50,000 CFS
50K - 6040HF
100K - 6040HF
200K - 6040HF
300K - 6040HF
50K - Base
100K - Base
200K - Base
300K - Base
Thalweg - 6040HF
Thalweg - Base
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at Morgan City RM -121.2 (6040HF - Scenario 5)
50K - 6040HF
100K - 6040HF
200K - 6040HF
300K - 6040HF
50K - Base
100K - Base
200K - Base
300K - Base
Thalweg - 6040HF
Thalweg - Base
300,000 CFS
200,000 CFS
100,000 CFS
50,000 CFS
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
10
8
6
4
2
0
-40
-45
-50
-55
-60
-65
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at Outflow Channel RM 4.33 (6040HF - Scenario 5)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
420,000 CFS
280,000 CFS
140,000 CFS
70,000 CFS
70K - 6040HF
140K - 6040HF
280K - 6040HF
420K - 6040HF
70K - Base
140K - Base
280K - Base
420K - Base
Thalweg - 6040HF
Thalweg - Base
55
45
35
25
15
5
-5
-15
-25
-35
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at Downstream end of Red River RM 10.6 (6040HF - Scenario 5)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
180,000 CFS
120,000 CFS
60,000 CFS
30,000 CFS
30K - 6040HF
60K - 6040HF
120K - 6040HF
180K - 6040HF
30K - Base
60K - Base
120K - Base
180K - Base
Thalweg - 6040HF
Thalweg - Base
65
55
45
35
25
15
5
-25
-30
-35
-40
-45
-50
-55
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at upstream end of Red River RM 33 (6040HF - Scenario 5)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
180,000 CFS
120,000 CFS
60,000 CFS
30,000 CFS
30K - Scenario 5
60K - Scenario 5
120K - Scenario 5
180K - Scenario 5
30K - Base
60K - Base
120K - Base
180K - Base
Thalweg - Scenario 5
Thalweg - Base
65
55
45
35
25
15
10
0
-10
-20
-30
1-Oct-1996
1-Oct-2006
1-Oct-2016
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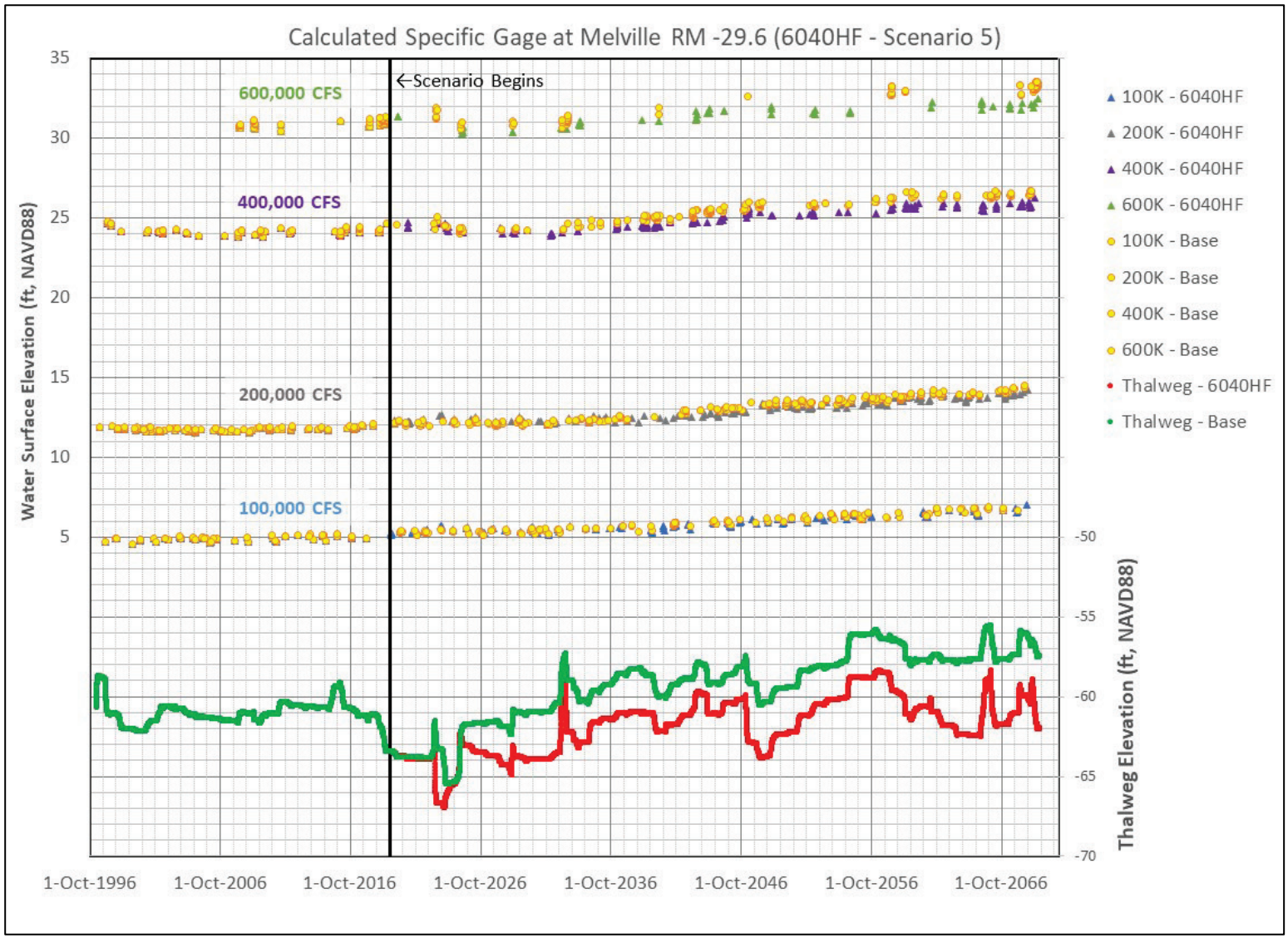
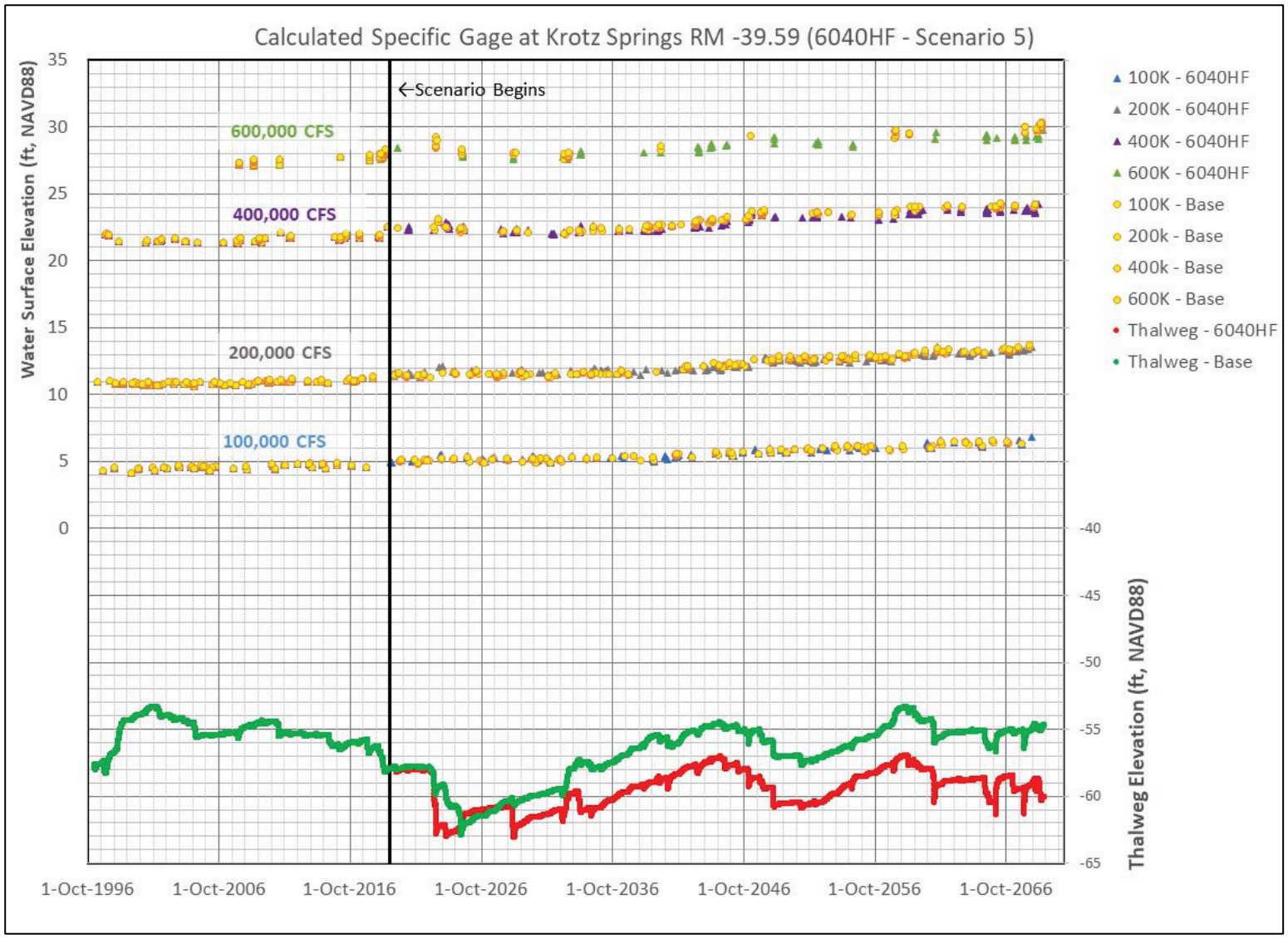
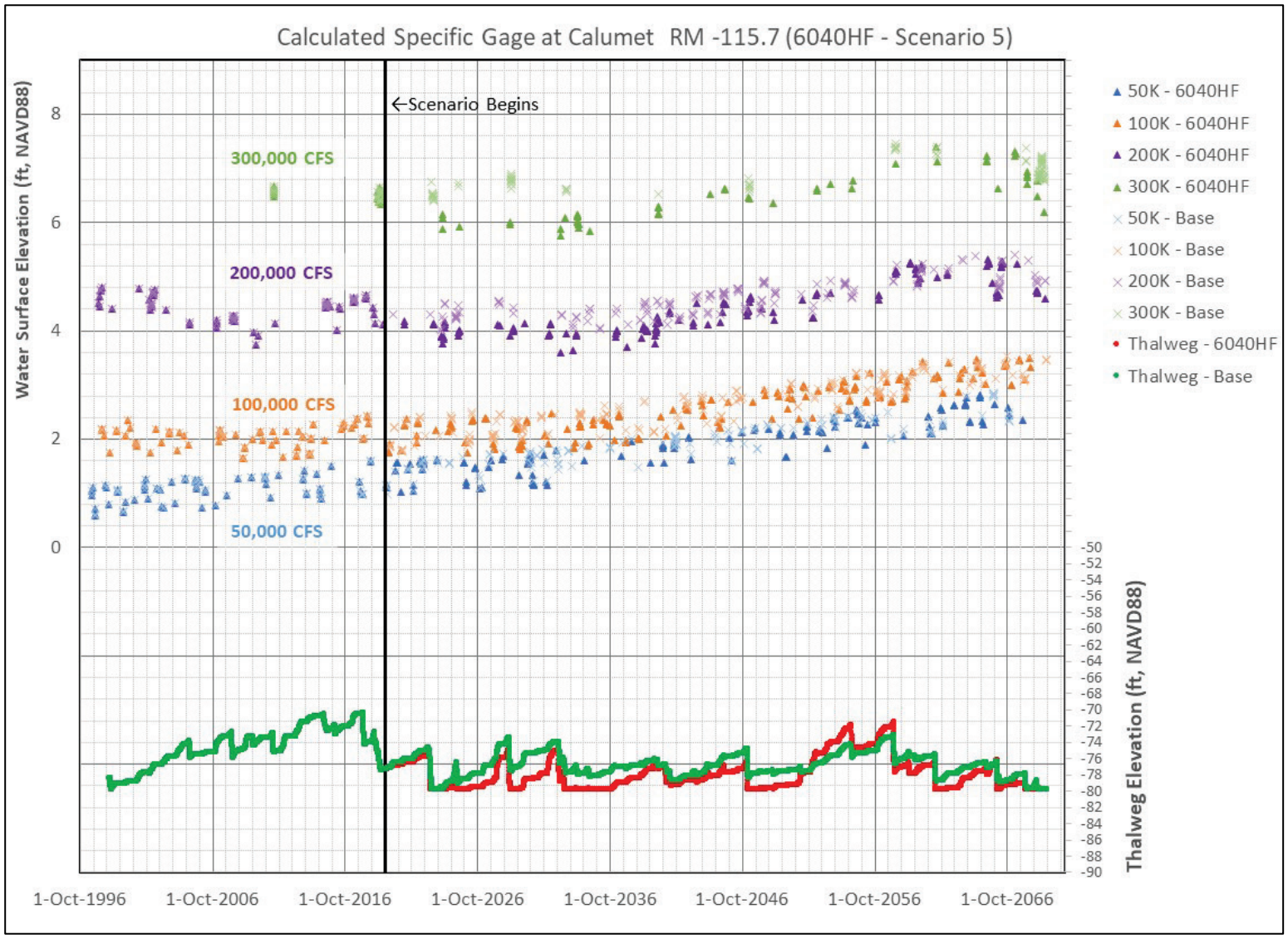
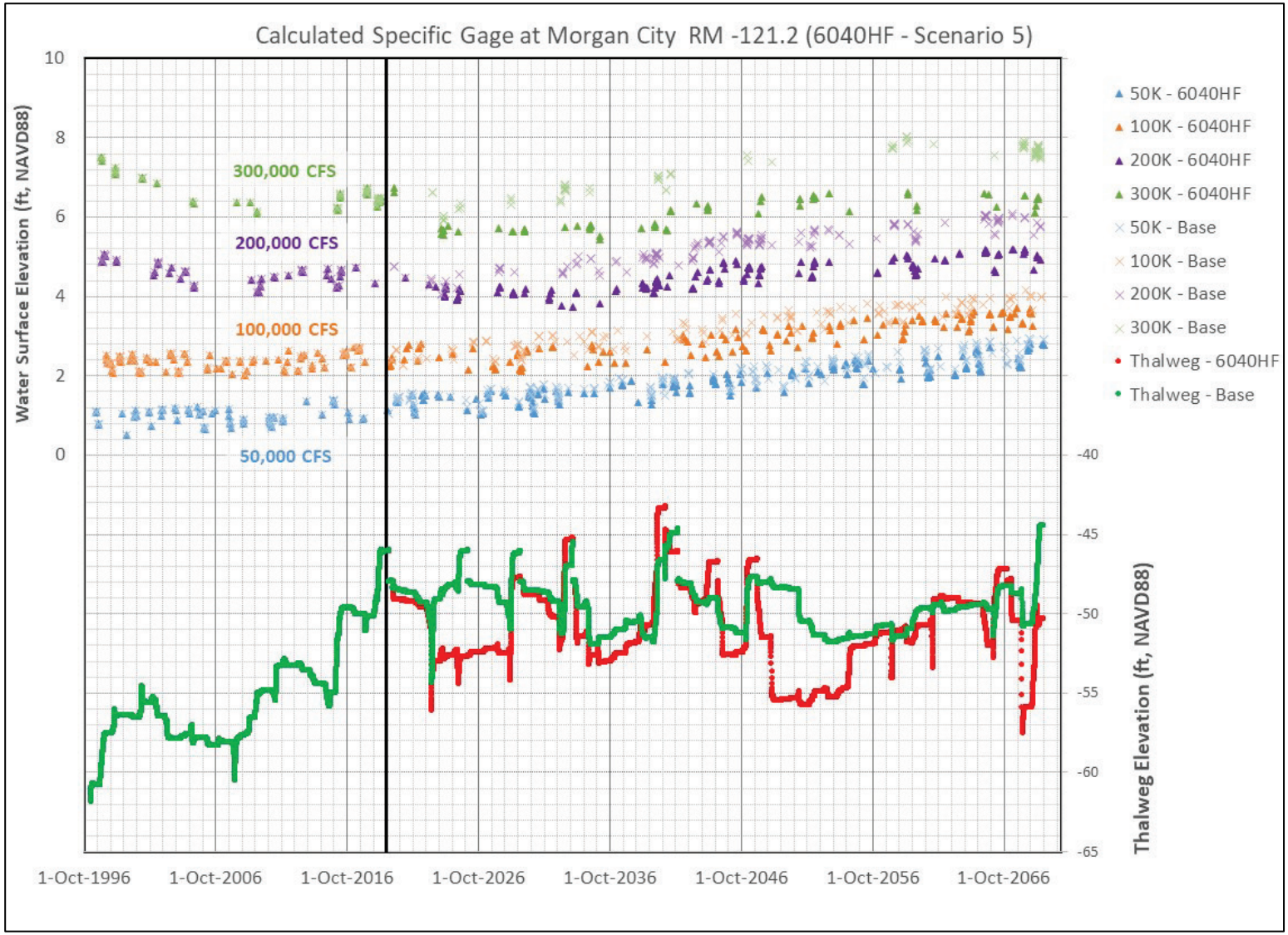
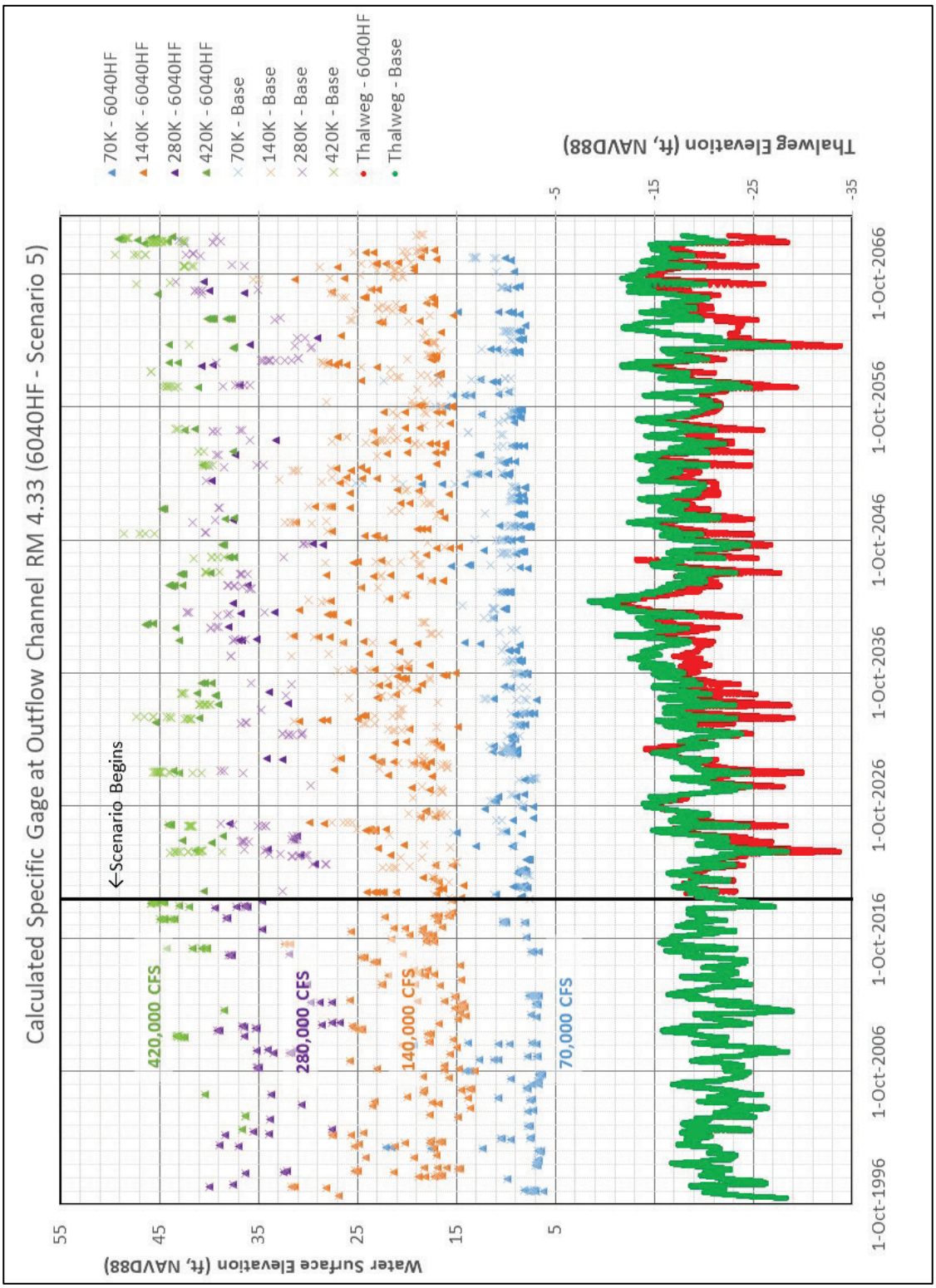
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066

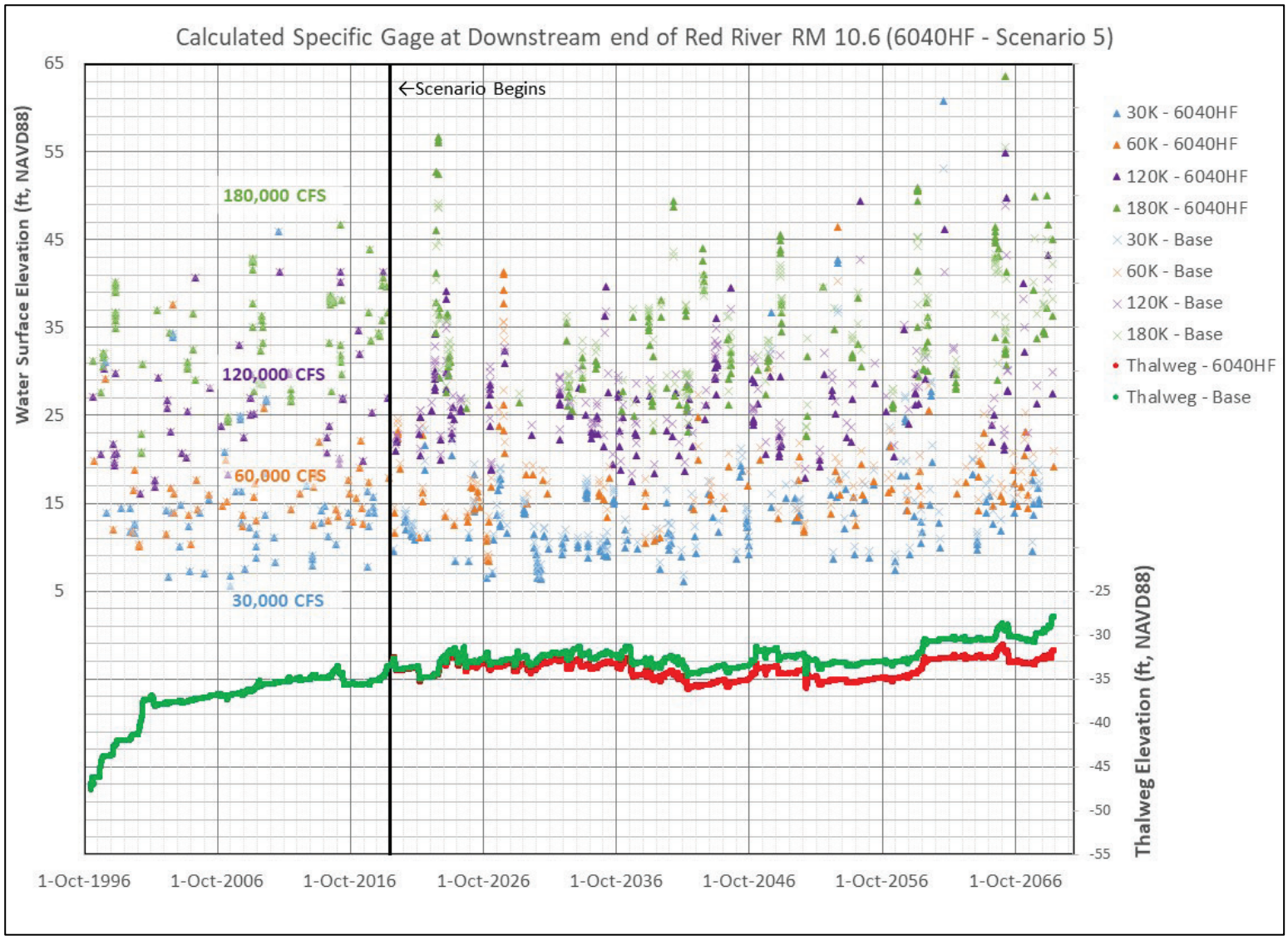

Calculated Specific Gage at Melville RM -29.6 (6040HF - Scenario 5)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
600,000 CFS
400,000 CFS
200,000 CFS
100,000 CFS
100K - 6040HF
200K - 6040HF
400K - 6040HF
600K - 6040HF
100K - Base
200K - Base
400K - Base
600K - Base
Thalweg - 6040HF
Thalweg - Base
35
30
25
20
15
10
5
-50
-55
-60
-65
-70
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066

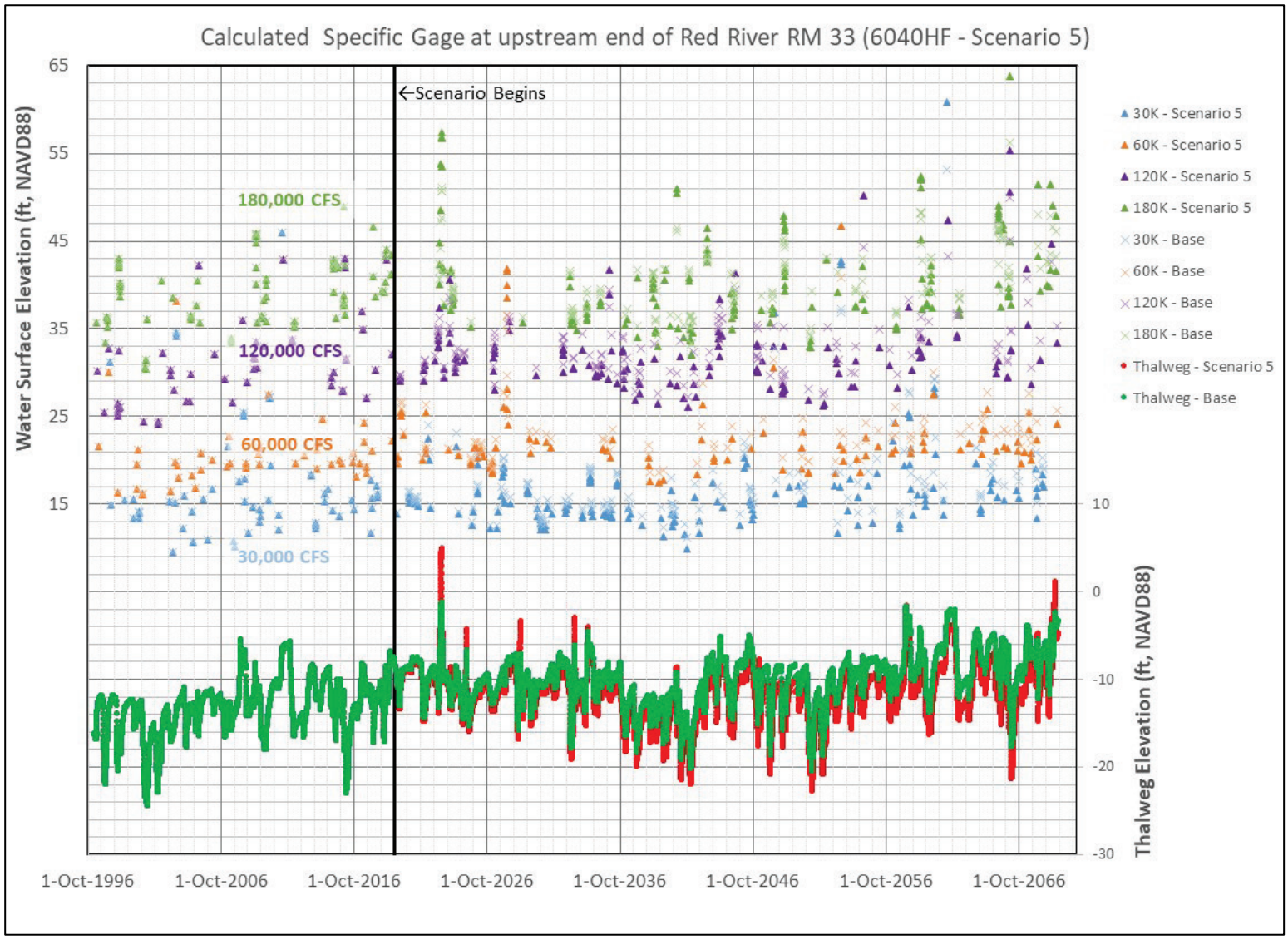

Calculated Specific Gage at Krotz Springs RM -39.59 (6040HF - Scenario 5)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
600,000 CFS
400,000 CFS
200,000 CFS
100,000 CFS
100K - 6040HF
200K - 6040HF
400K - 6040HF
600K - 6040HF
100K - Base
200k - Base
400k - Base
600K - Base
Thalweg - 6040HF
Thalweg - Base
35
30
25
20
15
10
5
0
-40
-45
-50
-55
-60
-65
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at Calumet RM -115.7 (6040HF - Scenario 5)
←Scenario Begins
300,000 CFS
200,000 CFS
100,000 CFS
50,000 CFS
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
50K - 6040HF
100K - 6040HF
200K - 6040HF
300K - 6040HF
50K - Base
100K - Base
200K - Base
300K - Base
Thalweg - 6040HF
Thalweg - Base
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at Morgan City RM -121.2 (6040HF - Scenario 5)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
300,000 CFS
200,000 CFS
100,000 CFS
50,000 CFS
50K - 6040HF
100K - 6040HF
200K - 6040HF
300K - 6040HF
50K - Base
100K - Base
200K - Base
300K - Base
Thalweg - 6040HF
Thalweg - Base
10
8
6
4
2
0
-40
-45
-50
-55
-60
-65
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at Outflow Channel RM 4.33 (6040HF - Scenario 5)
70K - 6040HF
140K - 6040HF
280K - 6040HF
420K - 6040HF
70K - Base
140K - Base
280K - Base
420K - Base
Thalweg - 6040HF
Thalweg - Base
←Scenario Begins
420,000 CFS
280,000 CFS
140,000 CFS
70,000 CFS
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
55
45
35
25
15
5
-5
-15
-25
-35
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at Downstream end of Red River RM 10.6 (6040HF - Scenario 5)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
180,000 CFS
120,000 CFS
60,000 CFS
30,000 CFS
30K - 6040HF
60K - 6040HF
120K - 6040HF
180K - 6040HF
30K - Base
60K - Base
120K - Base
180K - Base
Thalweg - 6040HF
Thalweg - Base
65
55
45
35
25
15
5
-25
-30
-35
-40
-45
-50
-55
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at upstream end of Red River RM 33 (6040HF - Scenario 5)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
180,000 CFS
120,000 CFS
60,000 CFS
30,000 CFS
30K - Scenario 5
60K - Scenario 5
120K - Scenario 5
180K - Scenario 5
30K - Base
60K - Base
120K - Base
180K - Base
Thalweg - Scenario 5
Thalweg - Base
65
55
45
35
25
15
10
0
-10
-20
-30
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
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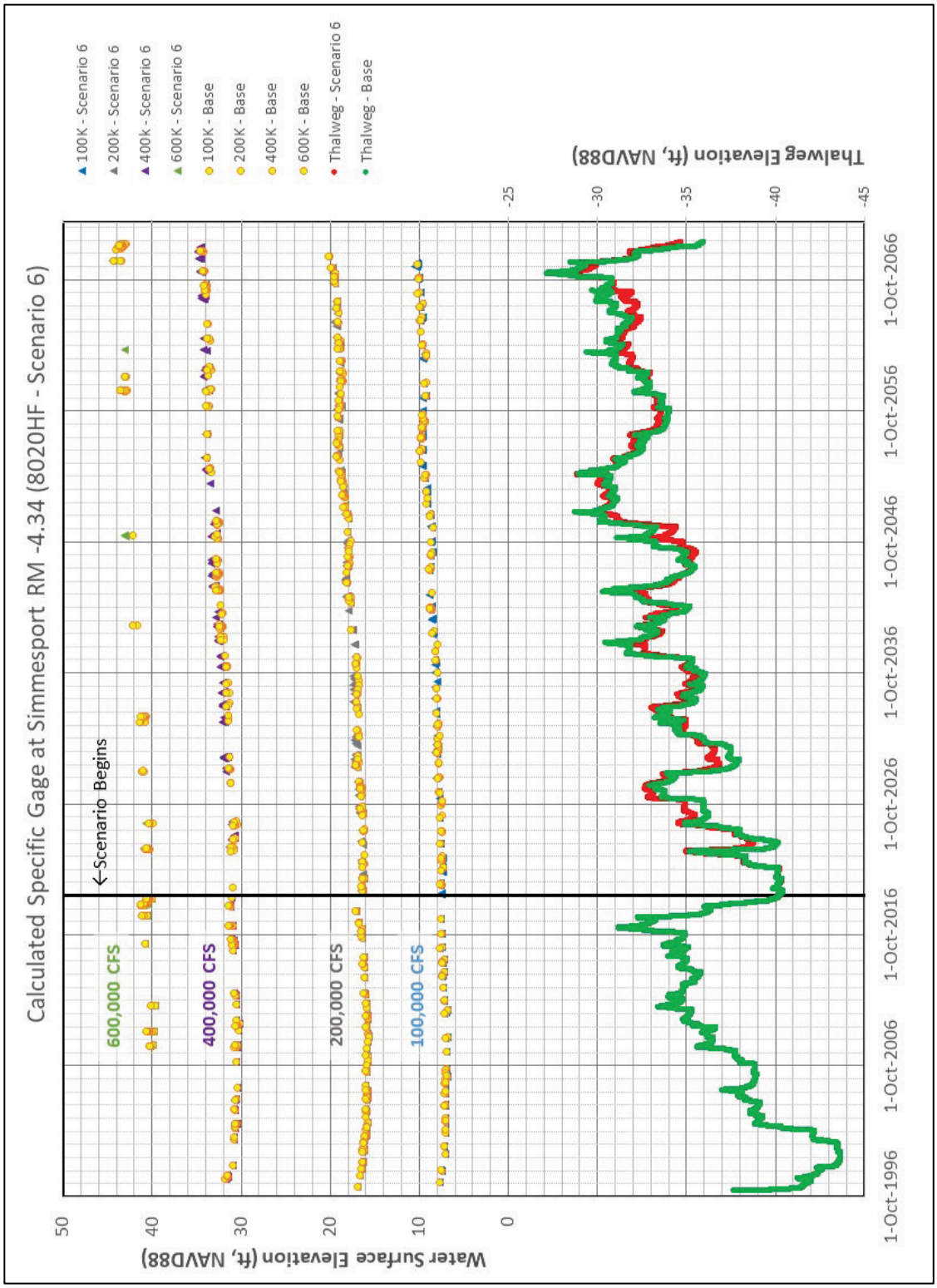
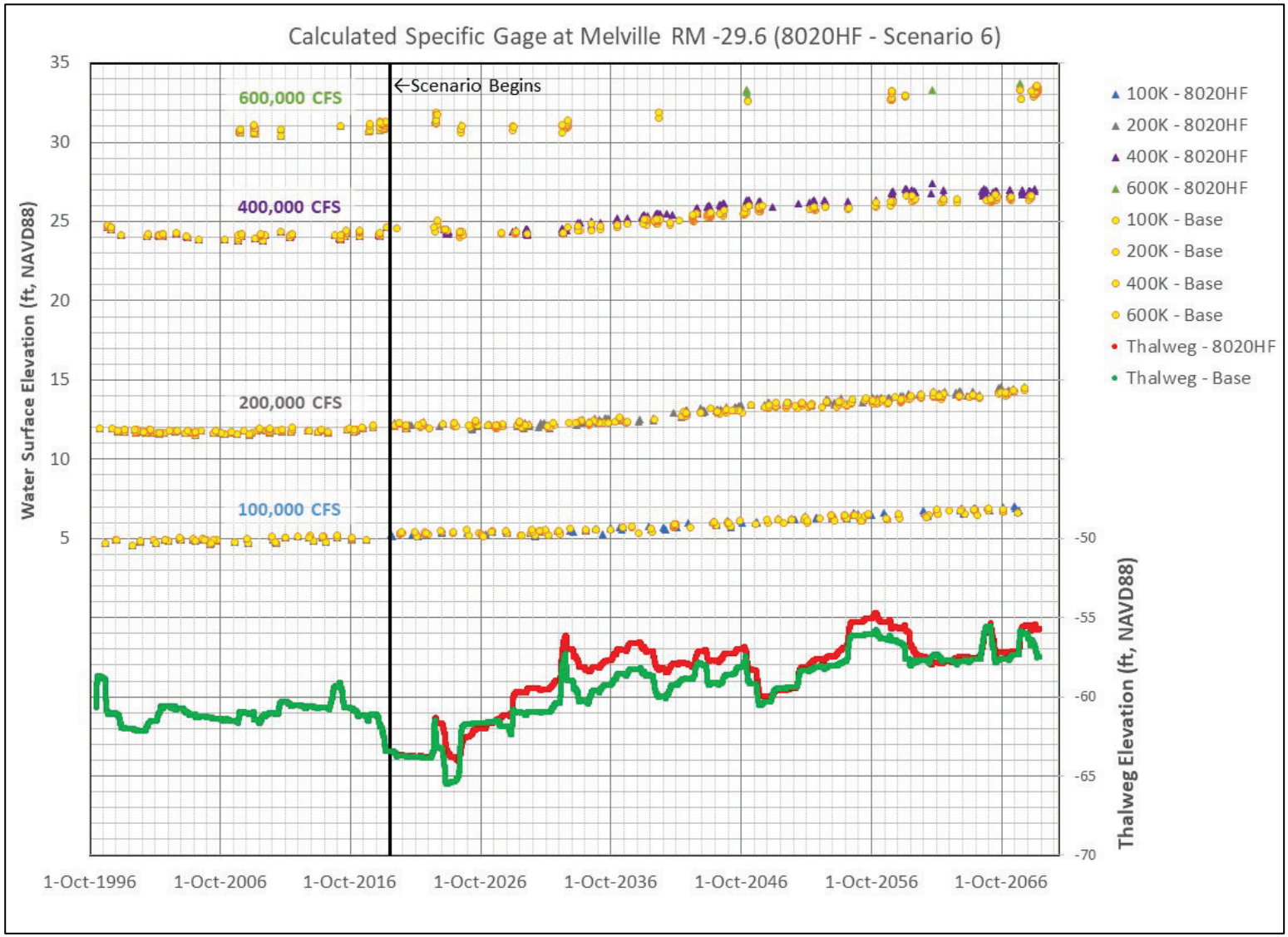
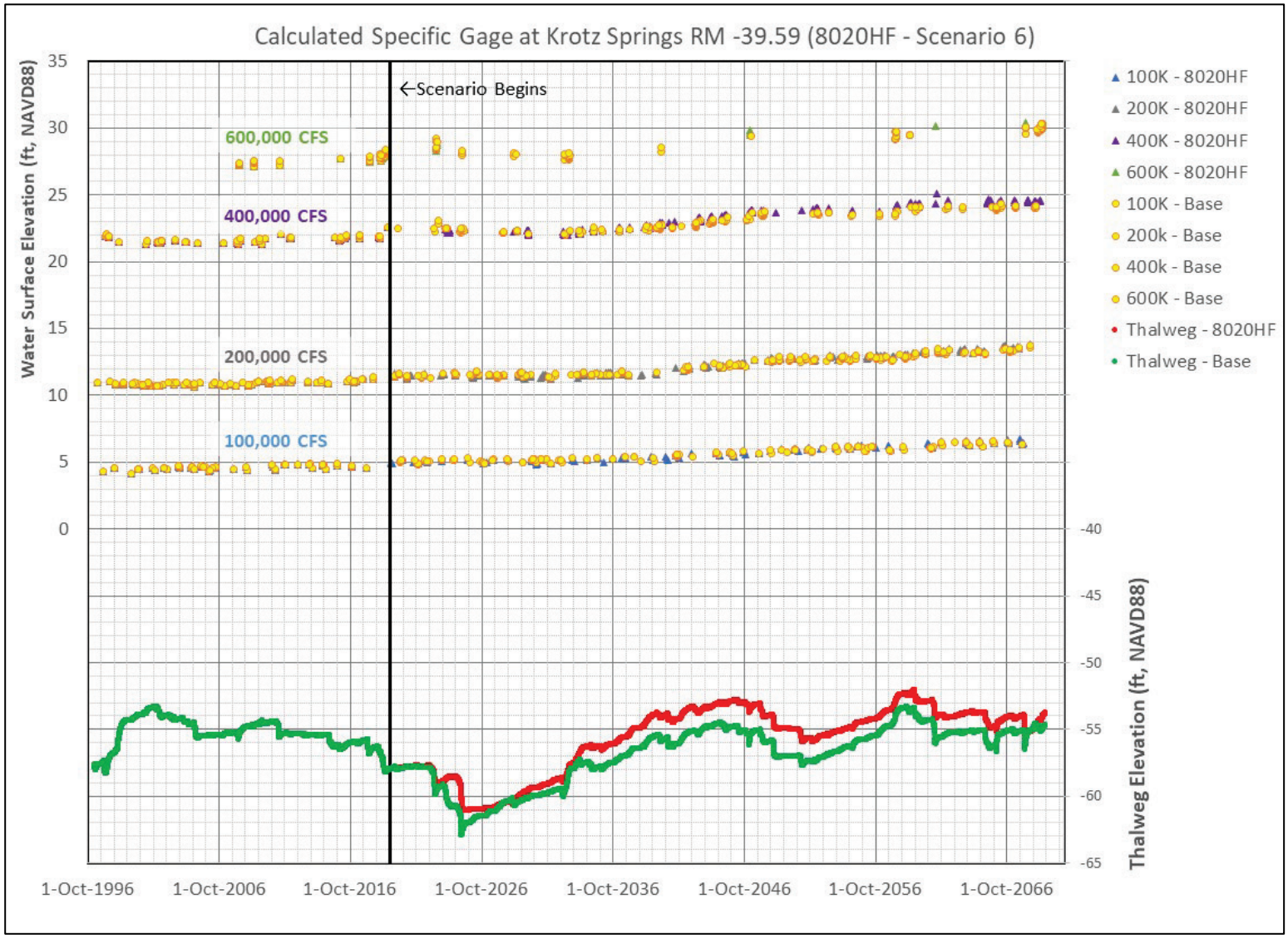
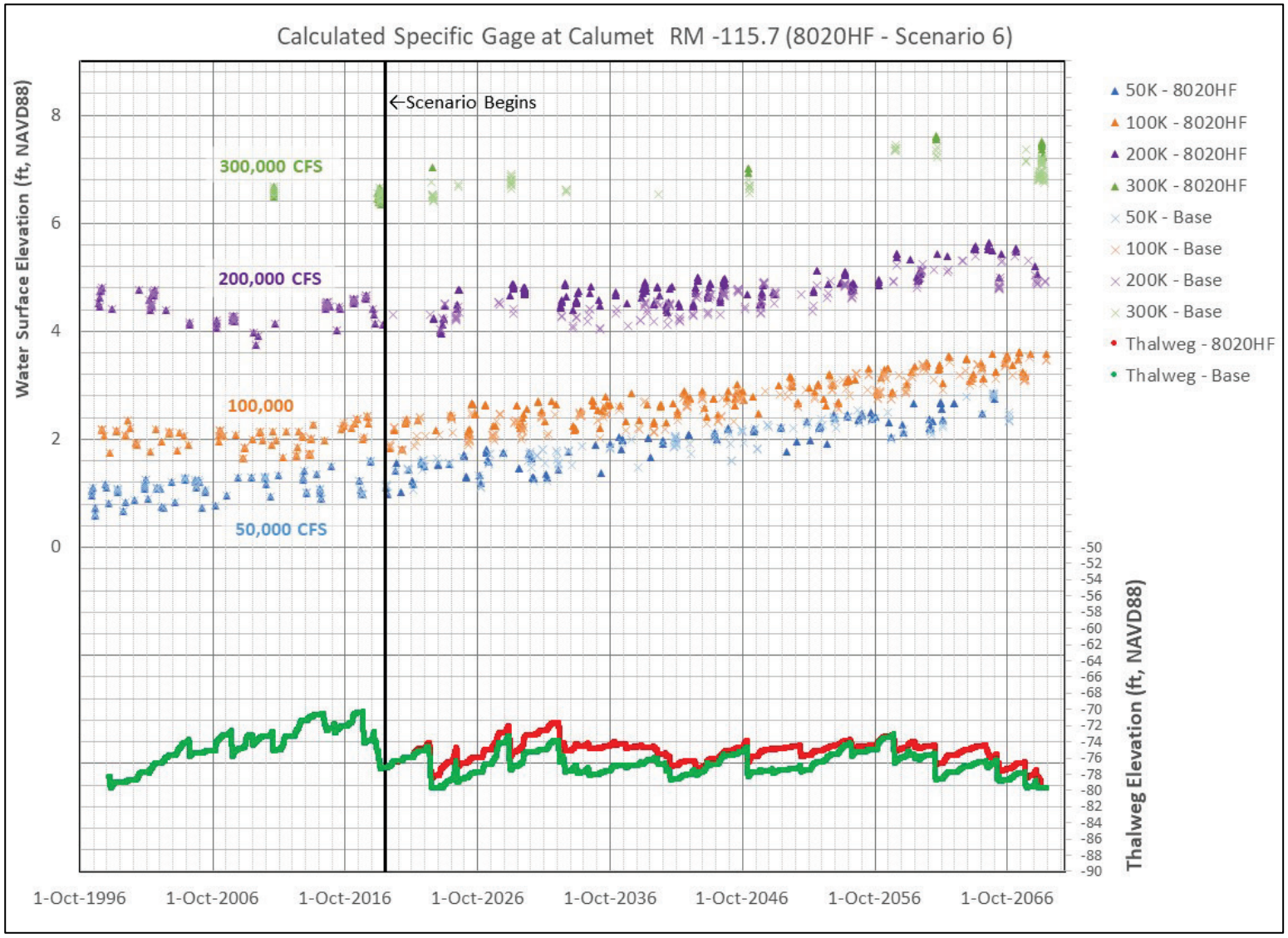
1-Oct-2046
1-Oct-2056
1-Oct-2066

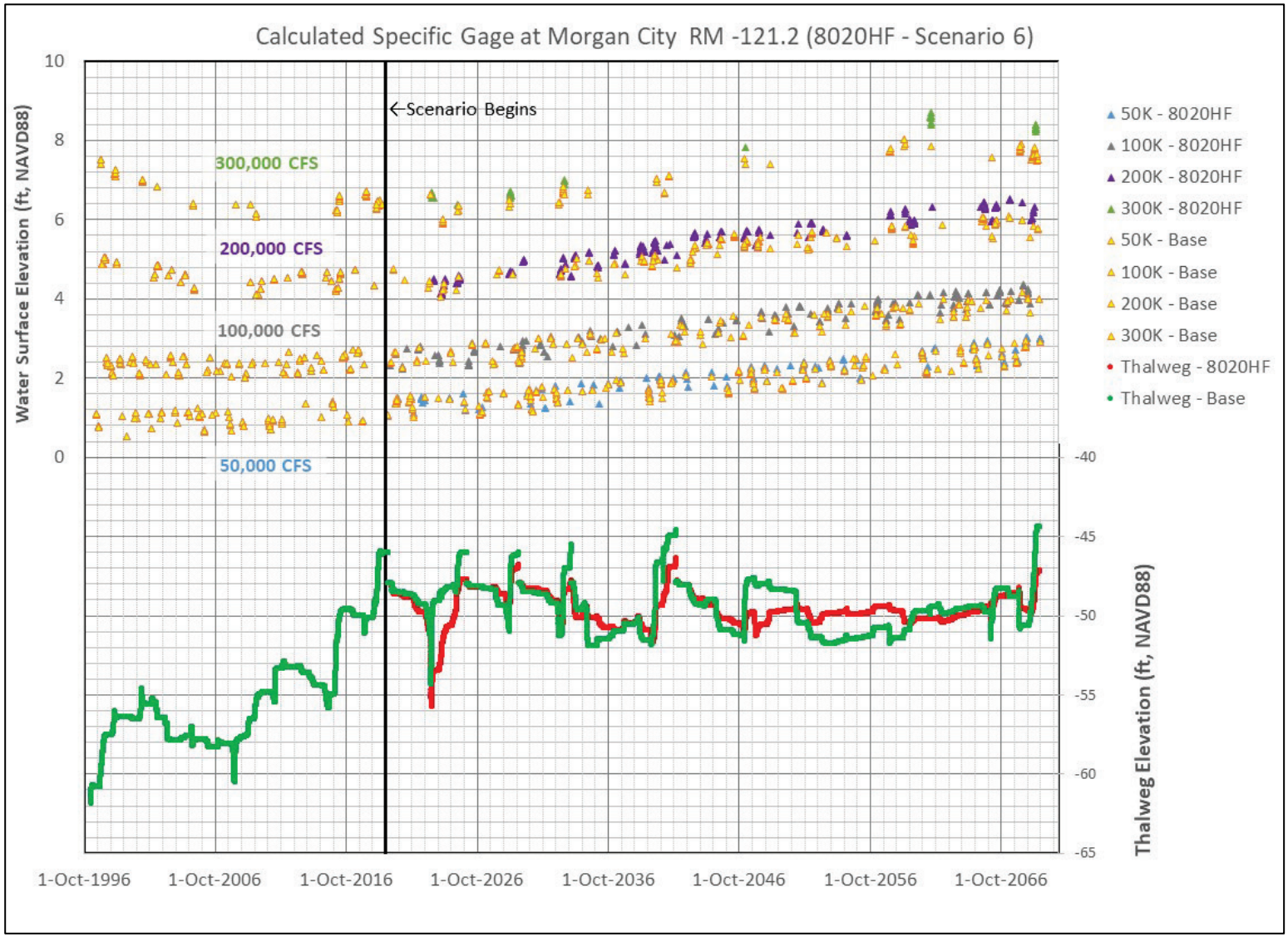
8020HF Flow Regime – Scenario 6

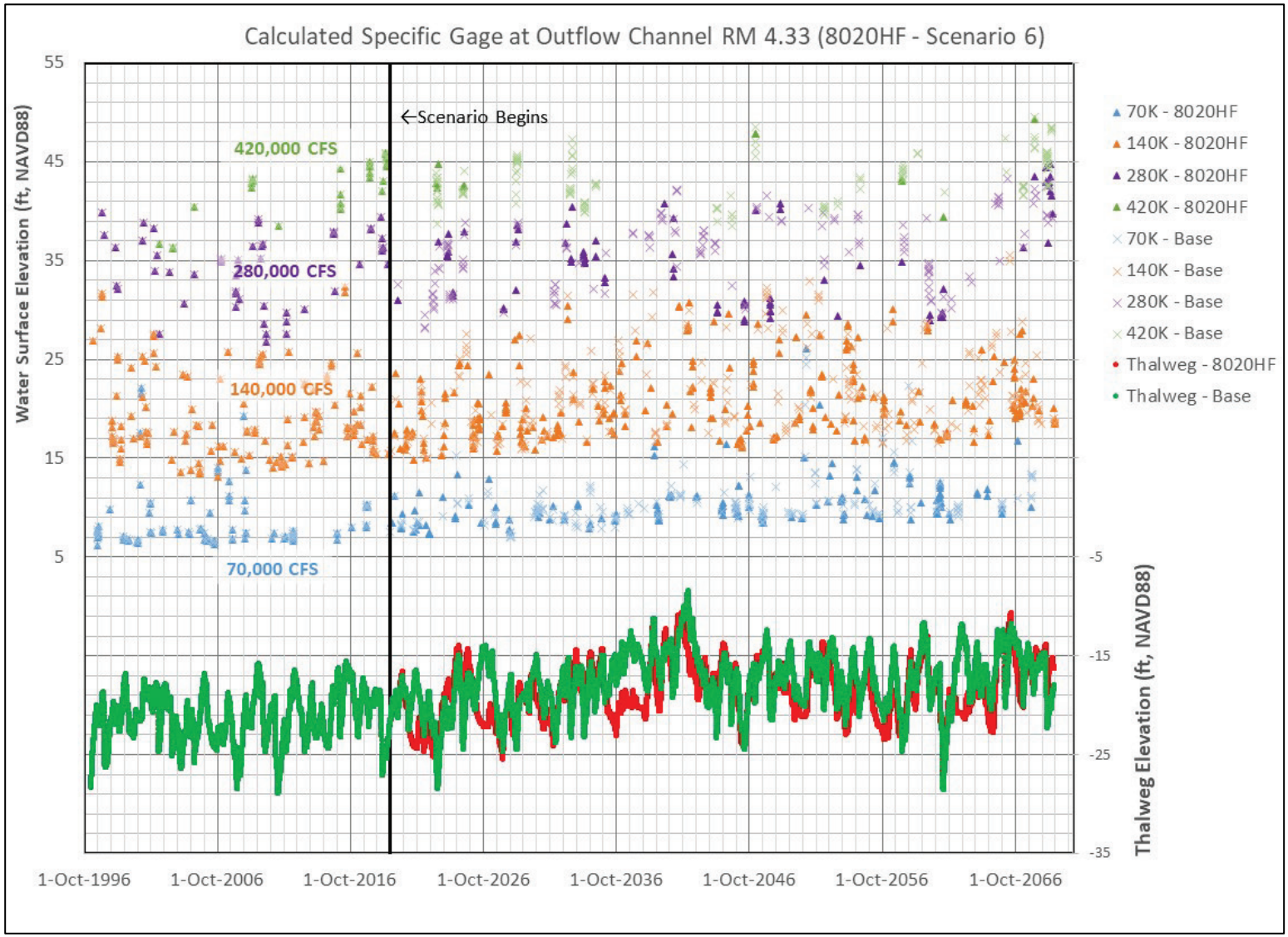


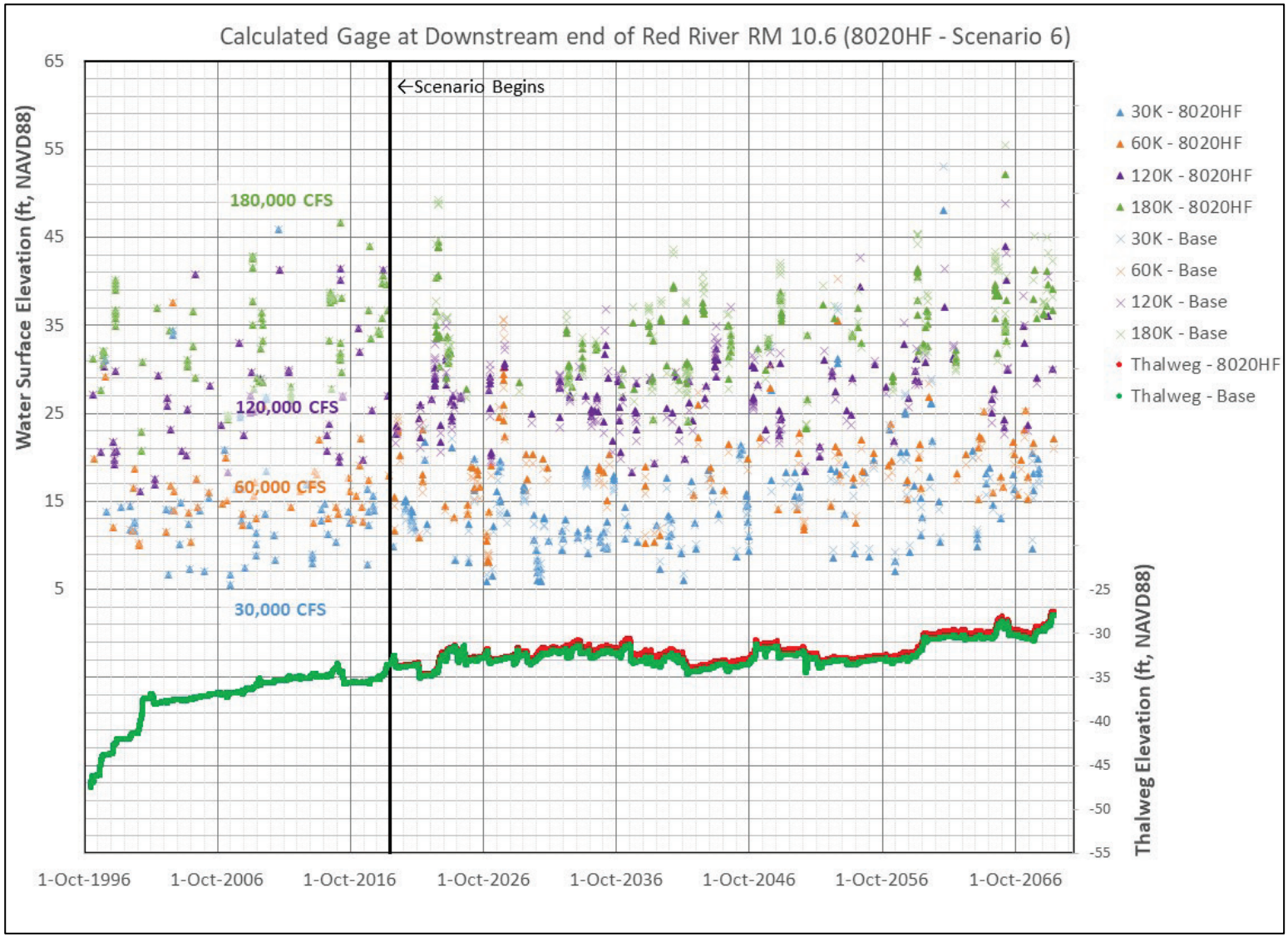

Calculated Specific Gage at Melville RM -29.6 (8020HF - Scenario 6)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
600,000 CFS
400,000 CFS
200,000 CFS
100,000 CFS
100K - 8020HF
200K - 8020HF
400K - 8020HF
600K - 8020HF
100K - Base
200K - Base
400K - Base
600K - Base
Thalweg - 8020HF
Thalweg - Base
35
30
25
20
15
10
5
-50
-55
-60
-65
-70
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066

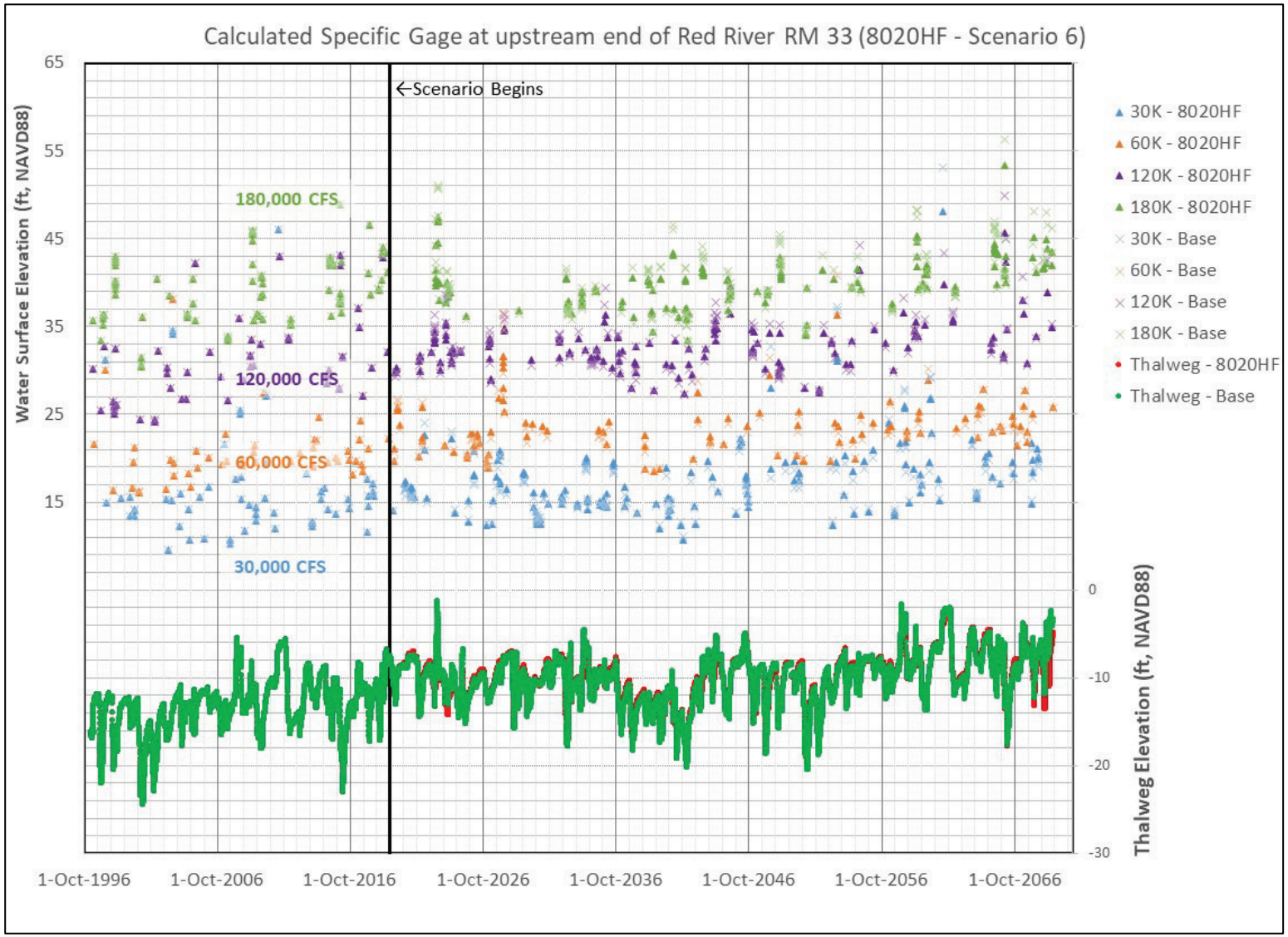

Calculated Specific Gage at Krotz Springs RM -39.59 (8020HF - Scenario 6)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
600,000 CFS
400,000 CFS
200,000 CFS
100,000 CFS
100K - 8020HF
200K - 8020HF
400K - 8020HF
600K - 8020HF
100K - Base
200k - Base
400k - Base
600K - Base
Thalweg - 8020HF
Thalweg - Base
35
30
25
20
15
10
5
0
-40
-45
-50
-55
-60
-65
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at Calumet RM -115.7 (8020HF - Scenario 6)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
300,000 CFS
200,000 CFS
100,000
50,000 CFS
50K - 8020HF
100K - 8020HF
200K - 8020HF
300K - 8020HF
50K - Base
100K - Base
200K - Base
300K - Base
Thalweg - 8020HF
Thalweg - Base
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at Morgan City RM -121.2 (8020HF - Scenario 6)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
300,000 CFS
200,000 CFS
100,000 CFS
50,000 CFS
50K - 8020HF
100K - 8020HF
200K - 8020HF
300K - 8020HF
50K - Base
100K - Base
200K - Base
300K - Base
Thalweg - 8020HF
Thalweg - Base
10
8
6
4
2
0
-40
-45
-50
-55
-60
-65
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at Outflow Channel RM 4.33 (8020HF - Scenario 6)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
420,000 CFS
280,000 CFS
140,000 CFS
70,000 CFS
70K - 8020HF
140K - 8020HF
280K - 8020HF
420K - 8020HF
70K - Base
140K - Base
280K - Base
420K - Base
Thalweg - 8020HF
Thalweg - Base
55
45
35
25
15
5
-5
-15
-25
-35
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Gage at Downstream end of Red River RM 10.6 (8020HF - Scenario 6)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
180,000 CFS
120,000 CFS
60,000 CFS
30,000 CFS
30K - 8020HF
60K - 8020HF
120K - 8020HF
180K - 8020HF
30K - Base
60K - Base
120K - Base
180K - Base
Thalweg - 8020HF
Thalweg - Base
65
55
45
35
25
15
5
-25
-30
-35
-40
-45
-50
-55
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at upstream end of Red River RM 33 (8020HF - Scenario 6)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
180,000 CFS
120,000 CFS
60,000 CFS
30,000 CFS
30K - 8020HF
60K - 8020HF
120K - 8020HF
180K - 8020HF
30K - Base
60K - Base
120K - Base
180K - Base
Thalweg - 8020HF
Thalweg - Base
65
55
45
35
25
15
0
-10
-20
-30
1-Oct-1996
1-Oct-2006
1-Oct-2016
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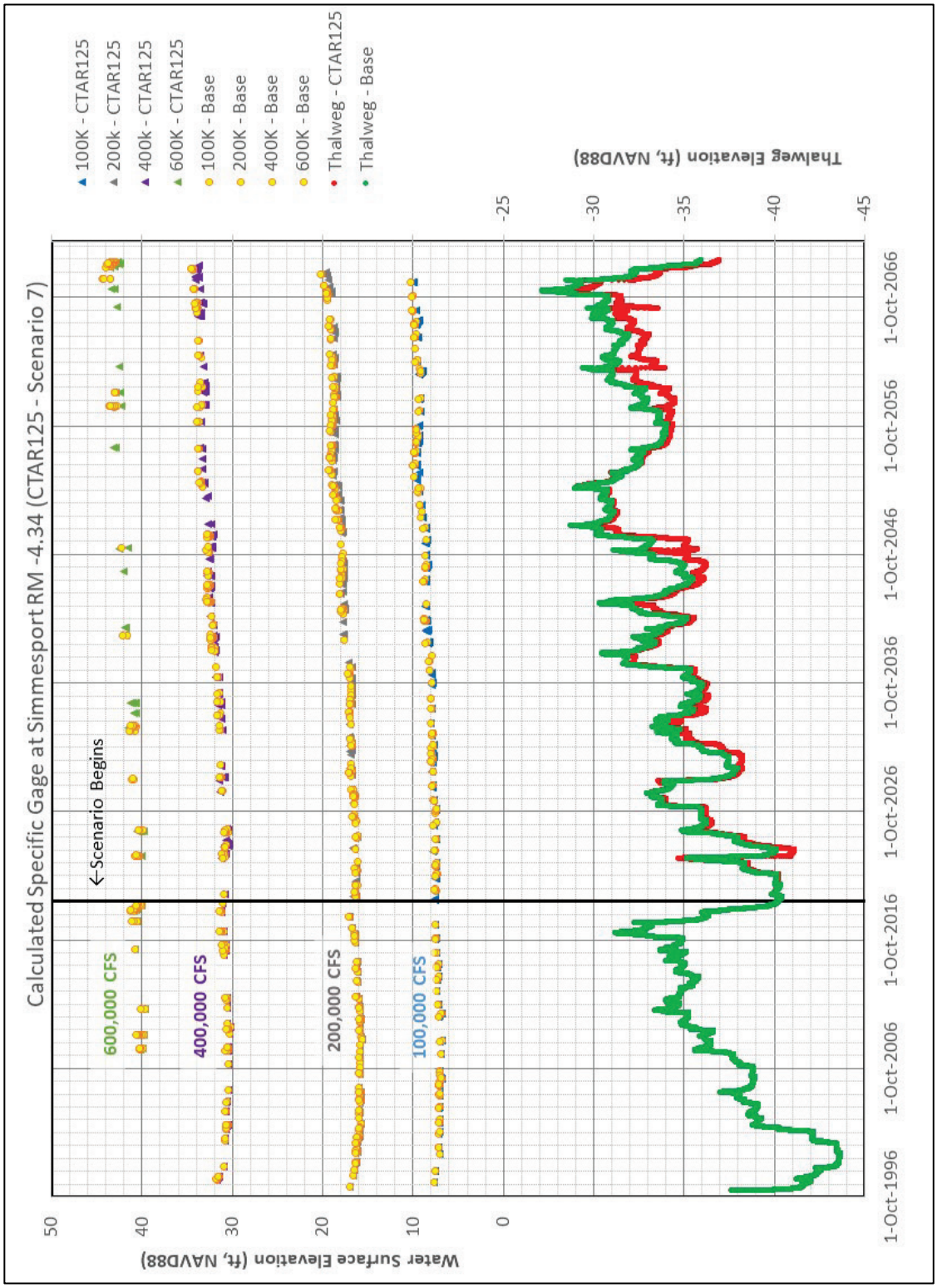
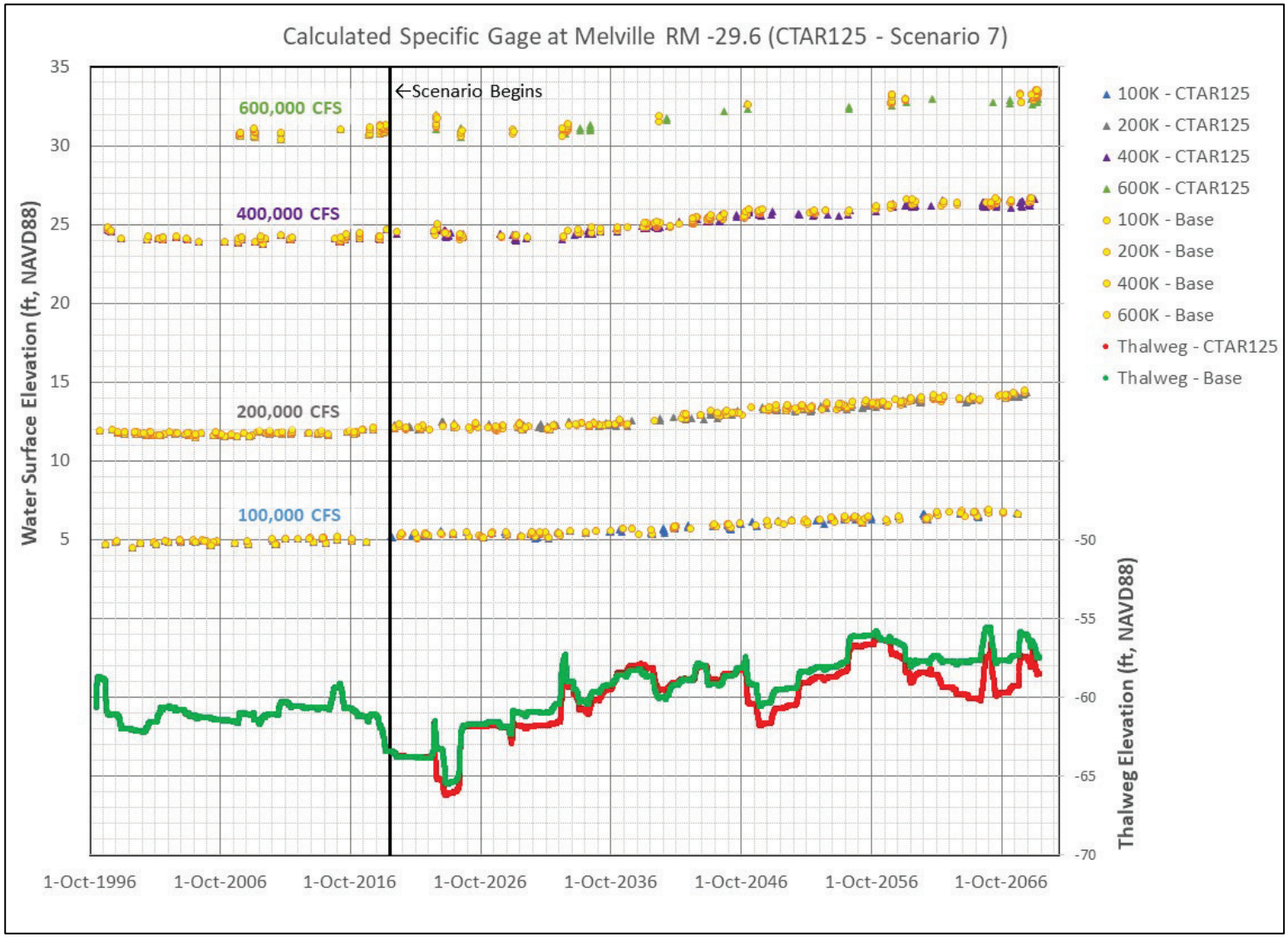
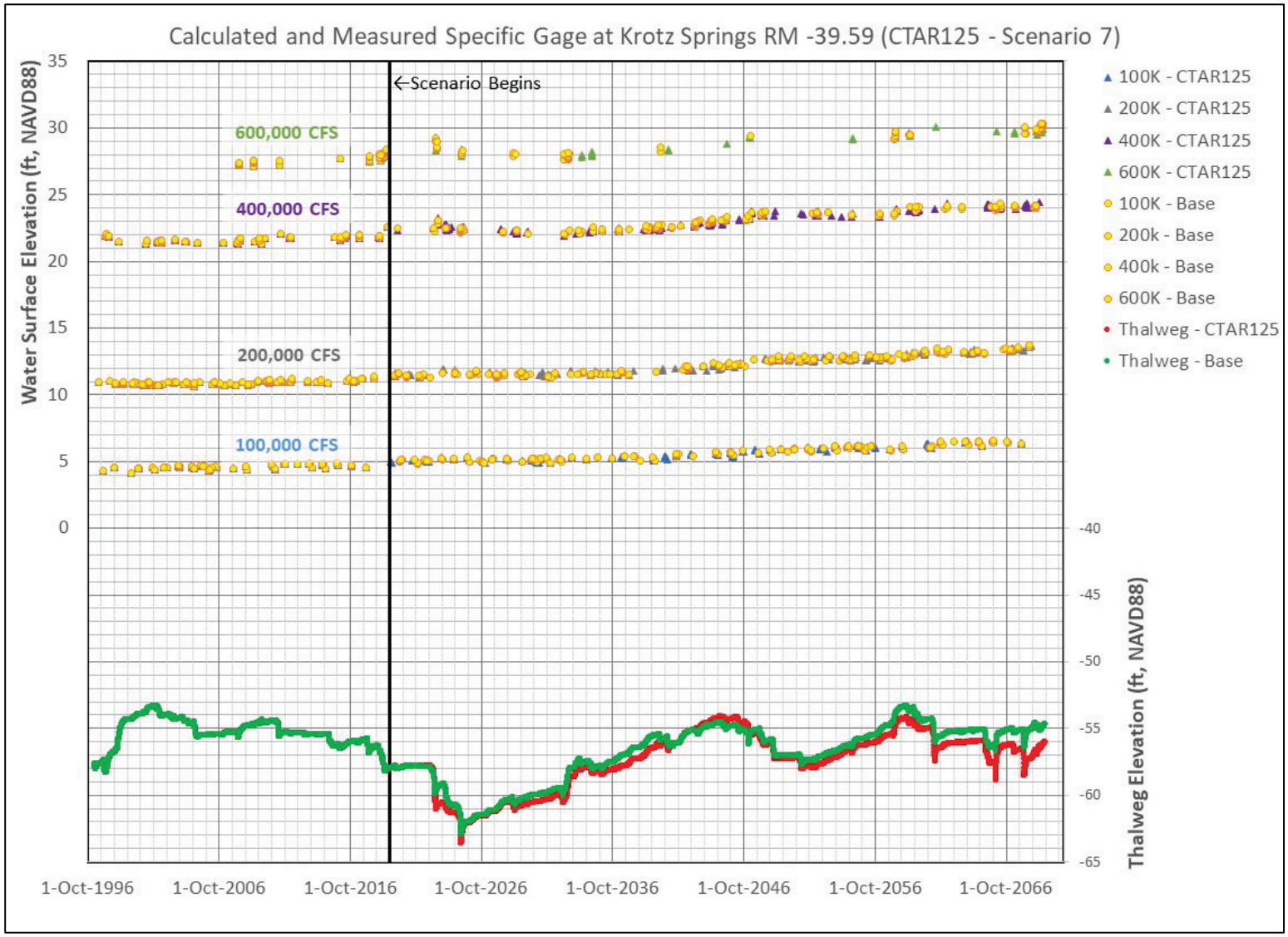
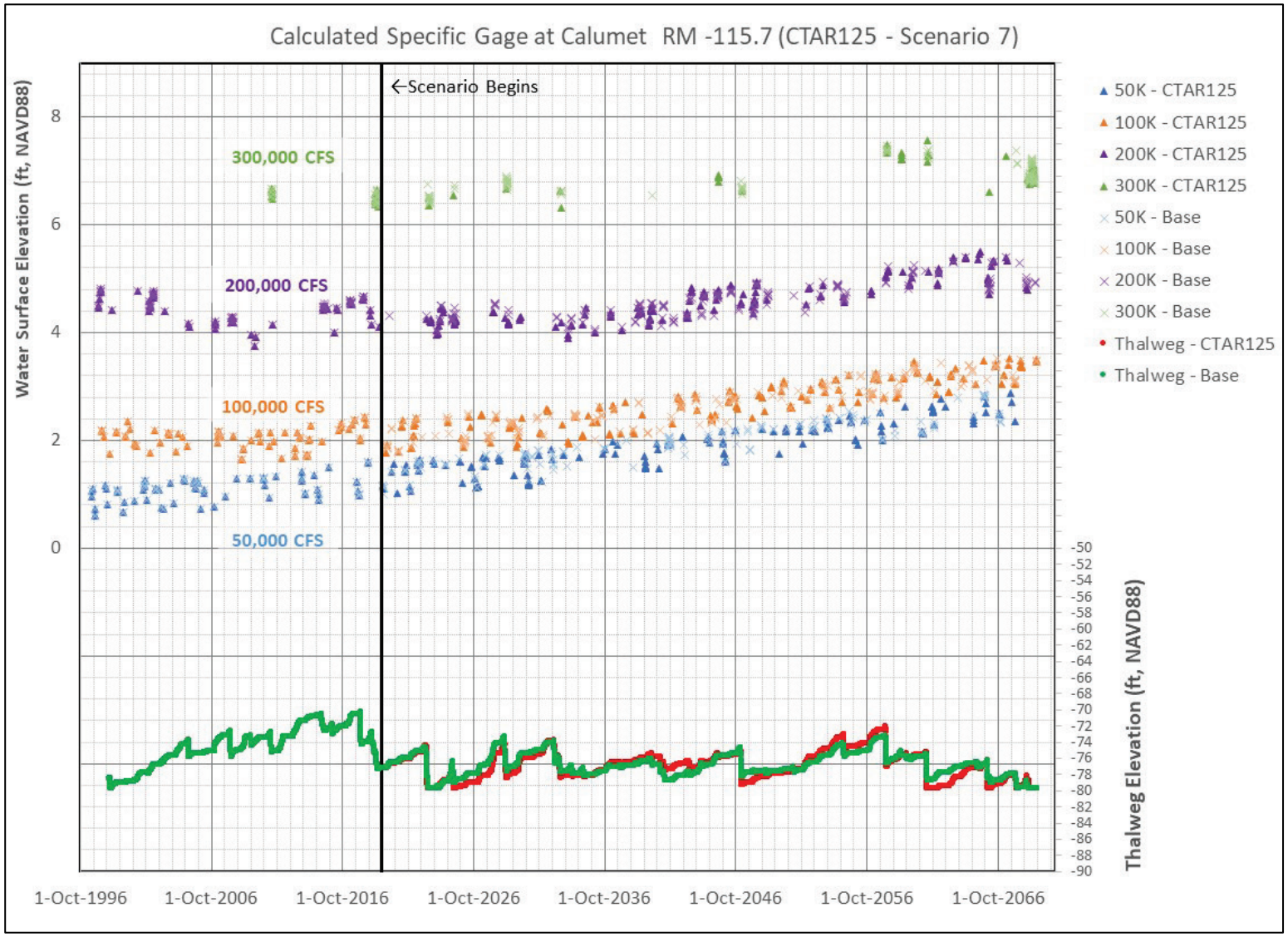
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066

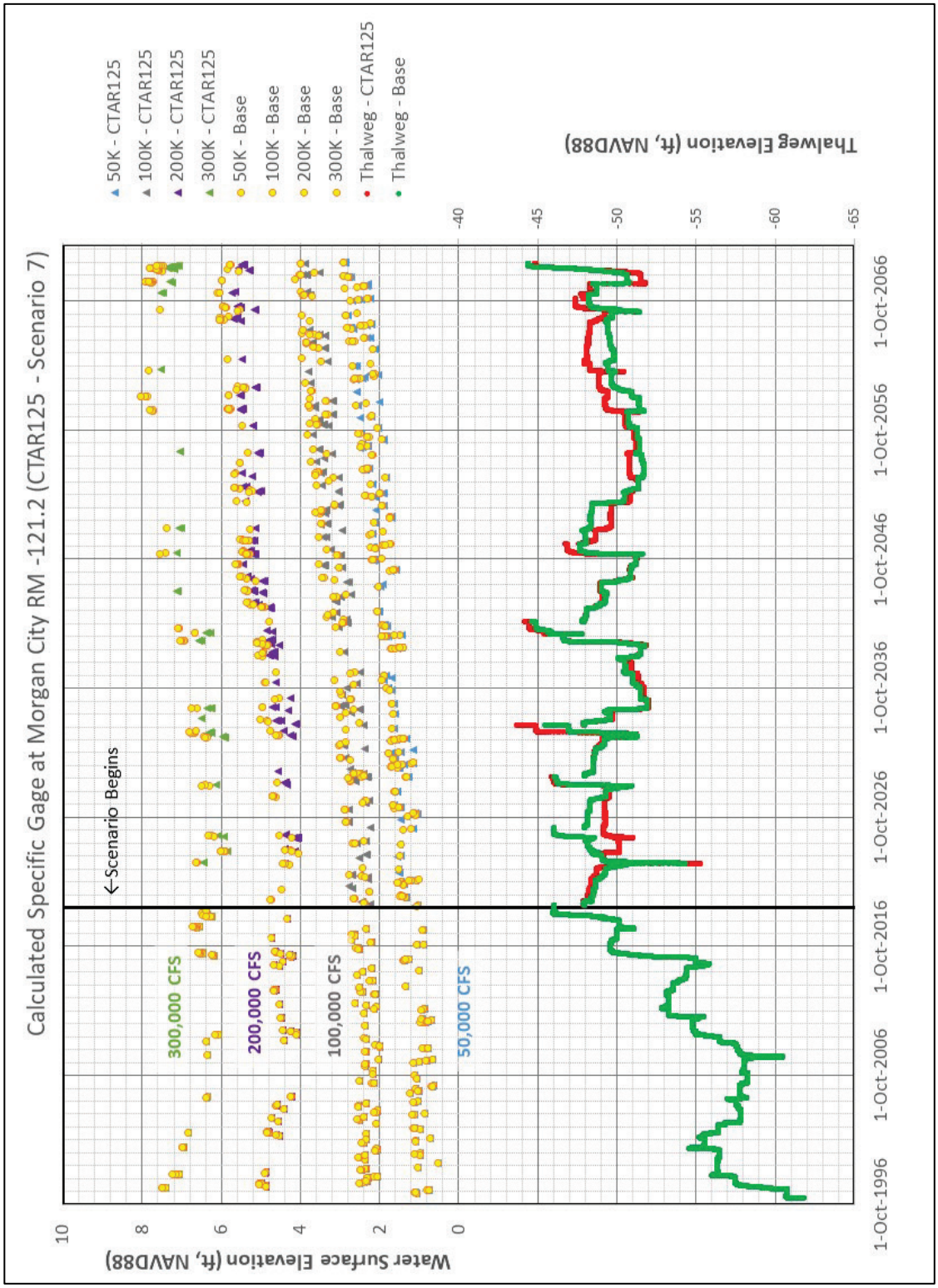
CTAR125 Flow Regime – Scenario 7

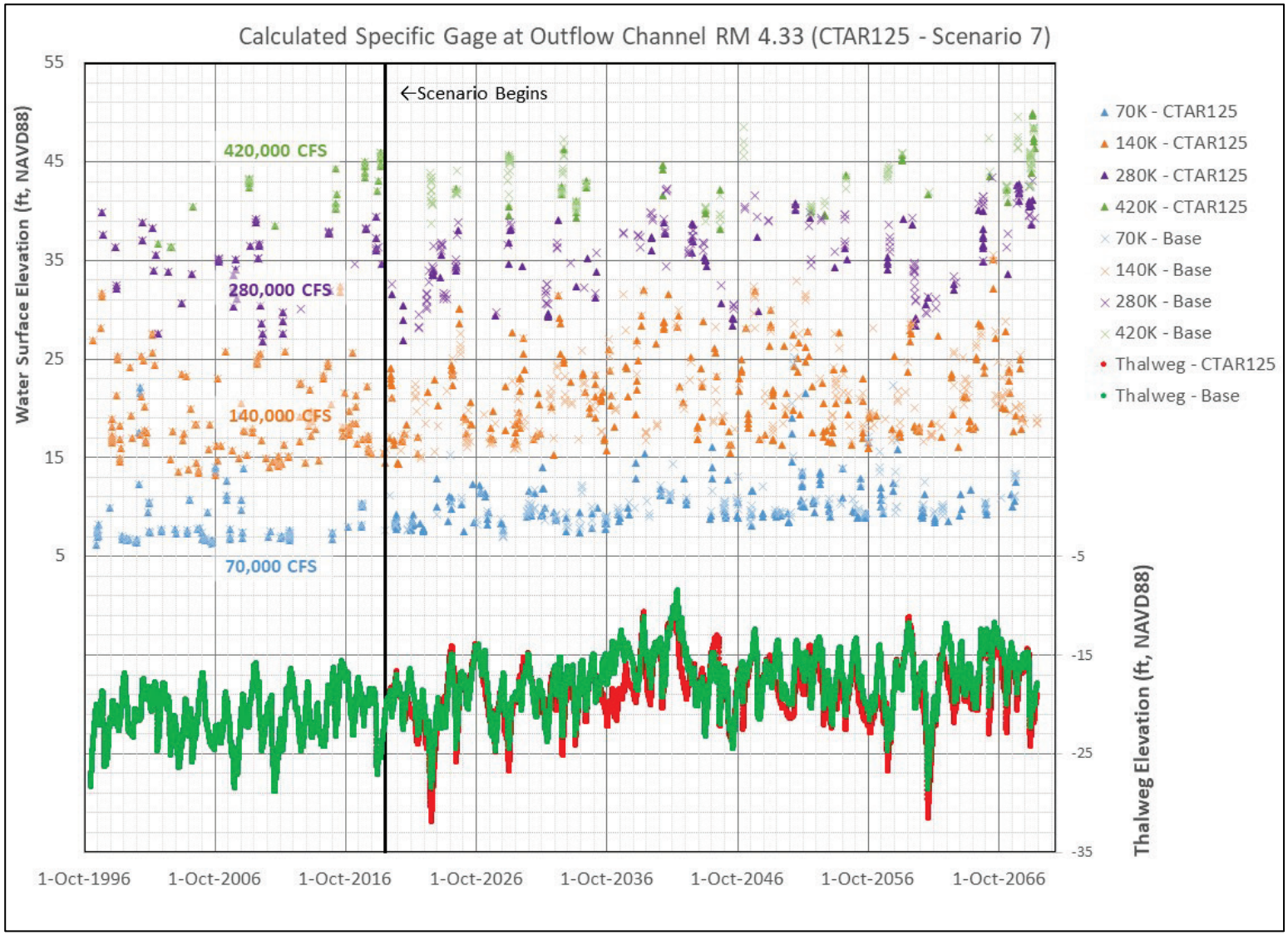


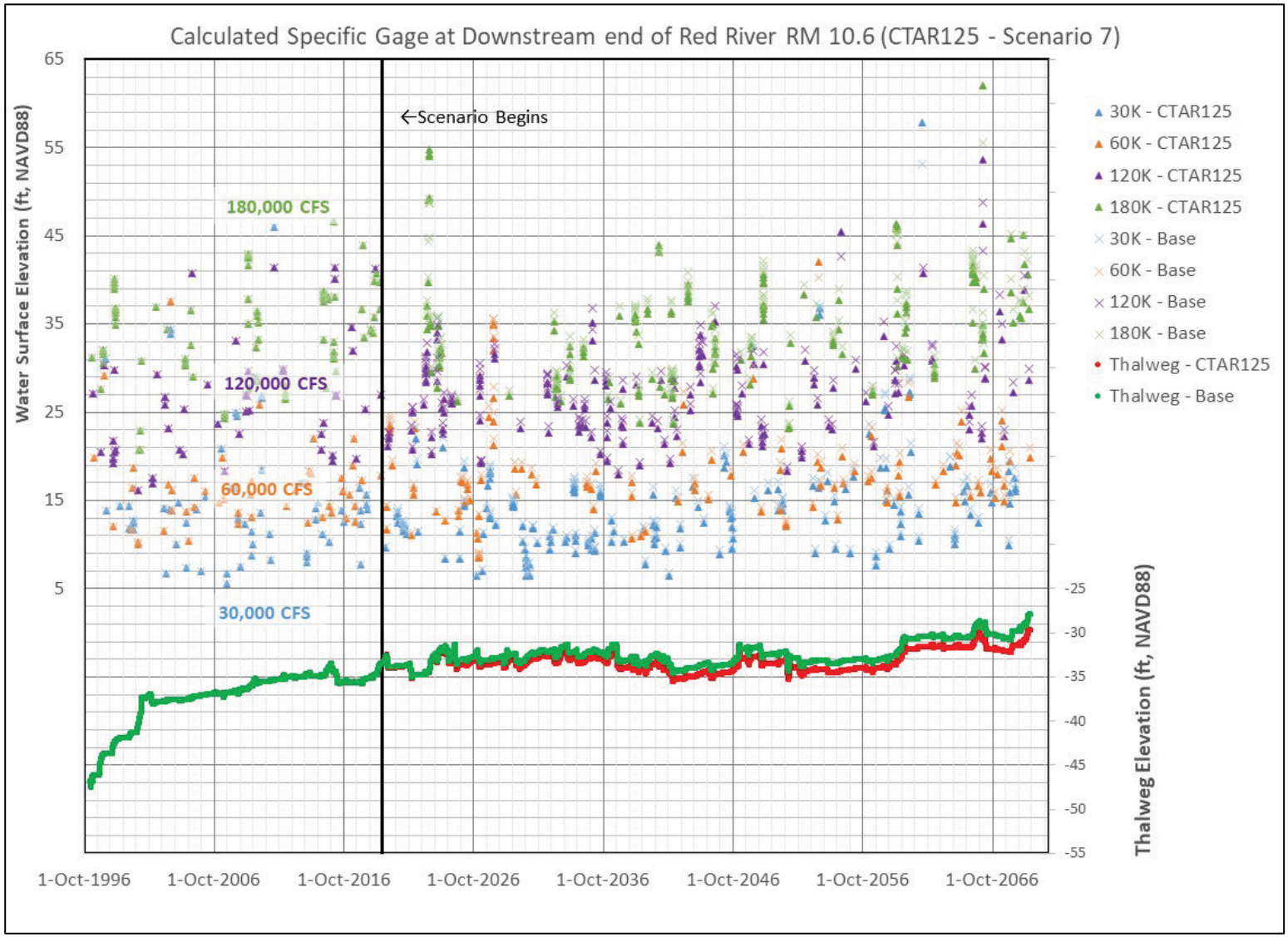

Calculated Specific Gage at Melville RM -29.6 (CTAR125 - Scenario 7)
←Scenario Begins
600,000 CFS
400,000 CFS
200,000 CFS
100,000 CFS
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
35
30
25
20
15
10
5
-50
-55
-60
-65
-70
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066
100K - CTAR125
200K - CTAR125
400K - CTAR125
600K - CTAR125
100K - Base
200K - Base
400K - Base
600K - Base
Thalweg - CTAR125
Thalweg - Base

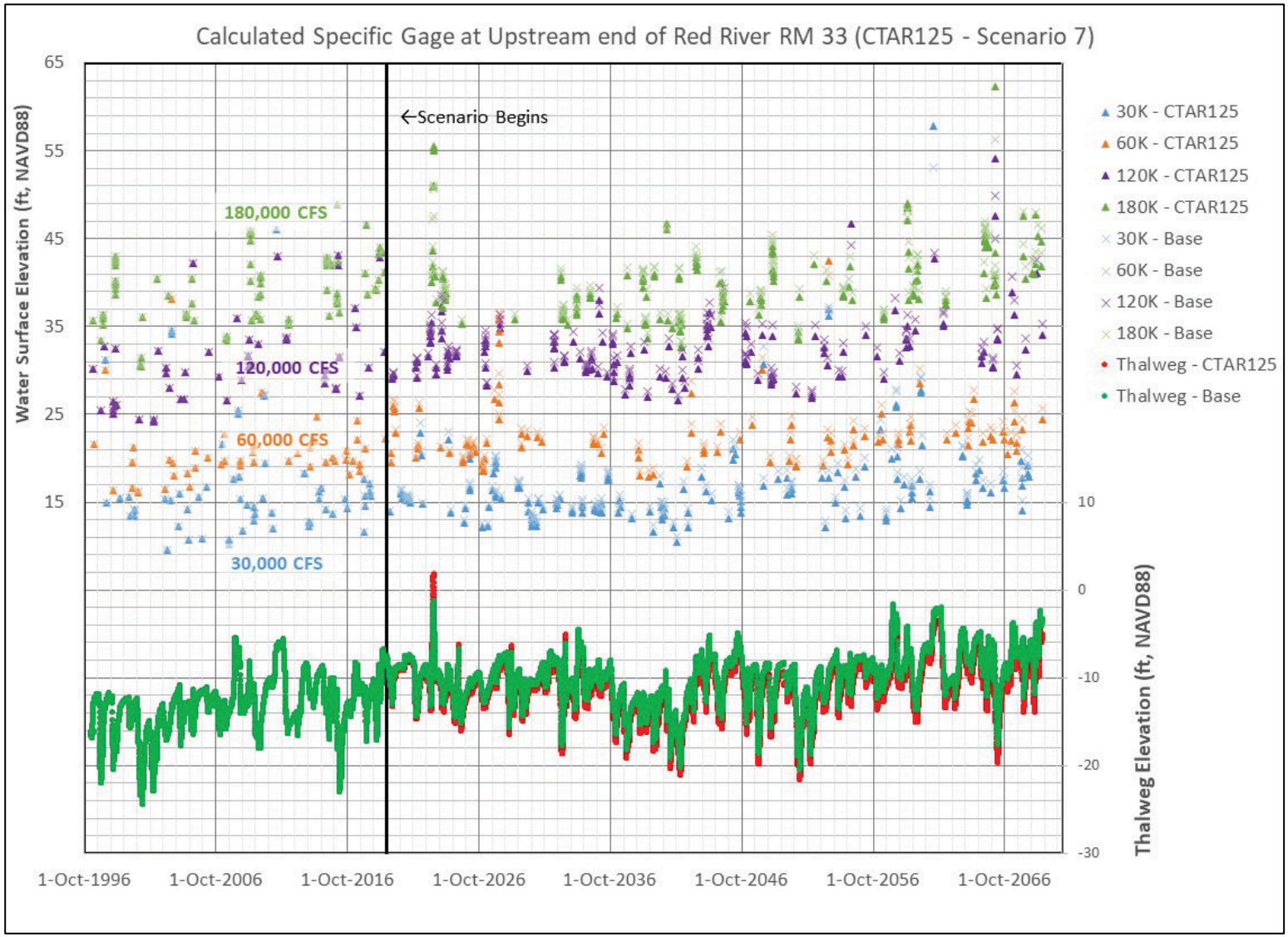

Calculated and Measured Specific Gage at Krotz Springs RM -39.59 (CTAR125 - Scenario 7)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
600,000 CFS
400,000 CFS
200,000 CFS
100,000 CFS
100K - CTAR125
200K - CTAR125
400K - CTAR125
600K - CTAR125
100K - Base
200k - Base
400k - Base
600K - Base
Thalweg - CTAR125
Thalweg - Base
35
30
25
20
15
10
5
0
-40
-45
-50
-55
-60
-65
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at Calumet RM -115.7 (CTAR125 - Scenario 7)
←Scenario Begins
300,000 CFS
200,000 CFS
100,000 CFS
50,000 CFS
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
50K - CTAR125
100K - CTAR125
200K - CTAR125
300K - CTAR125
50K - Base
100K - Base
200K - Base
300K - Base
Thalweg - CTAR125
Thalweg - Base
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066

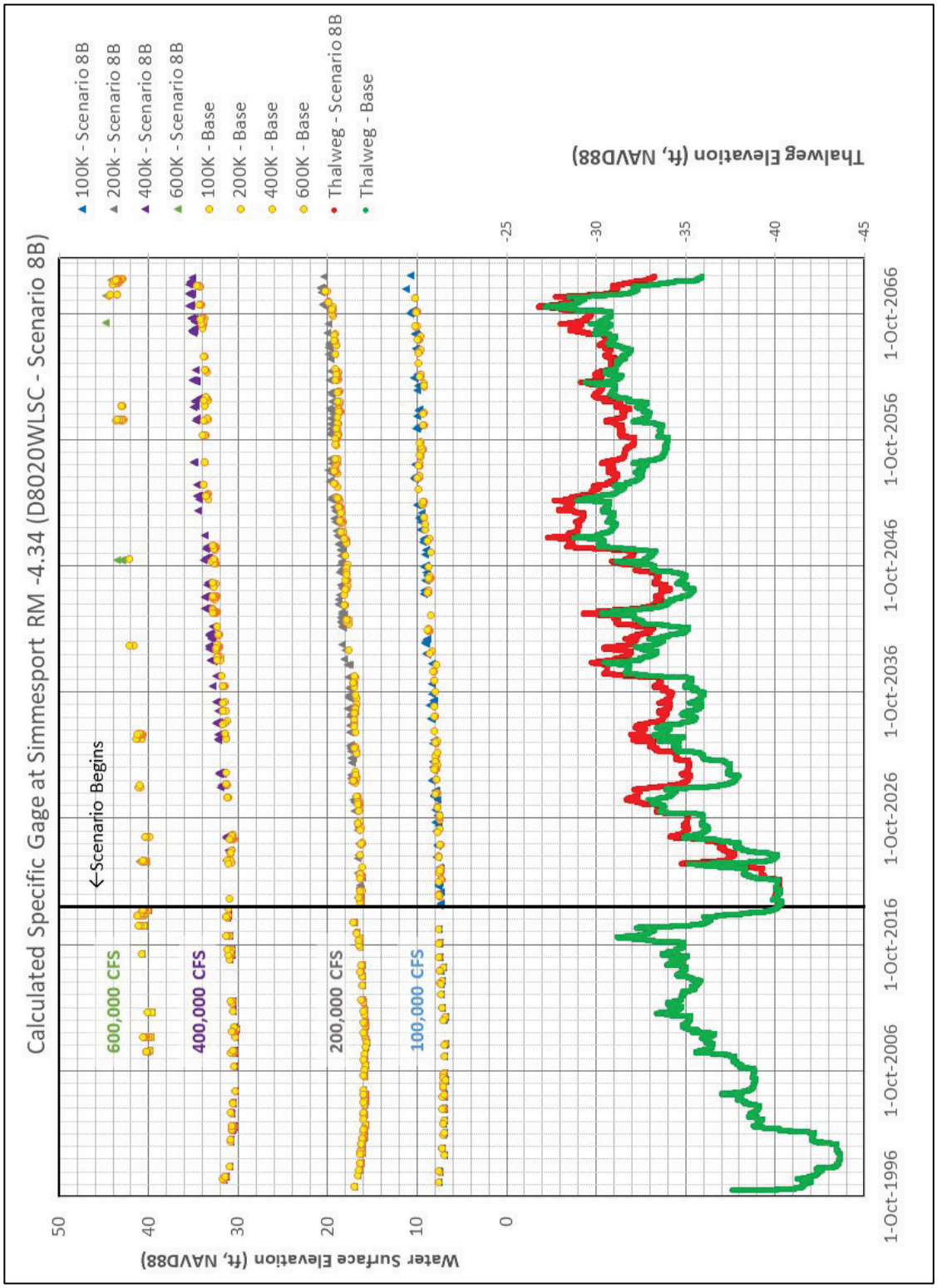

Calculated Specific Gage at Morgan City RM -121.2 (CTAR125 - Scenario 7)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
300,000 CFS
200,000 CFS
100,000 CFS
50,000 CFS
50K - CTAR125
100K - CTAR125
200K - CTAR125
300K - CTAR125
50K - Base
100K - Base
200K - Base
300K - Base
Thalweg - CTAR125
Thalweg - Base
10
8
6
4
2
0
-40
-45
-50
-55
-60
-65
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066

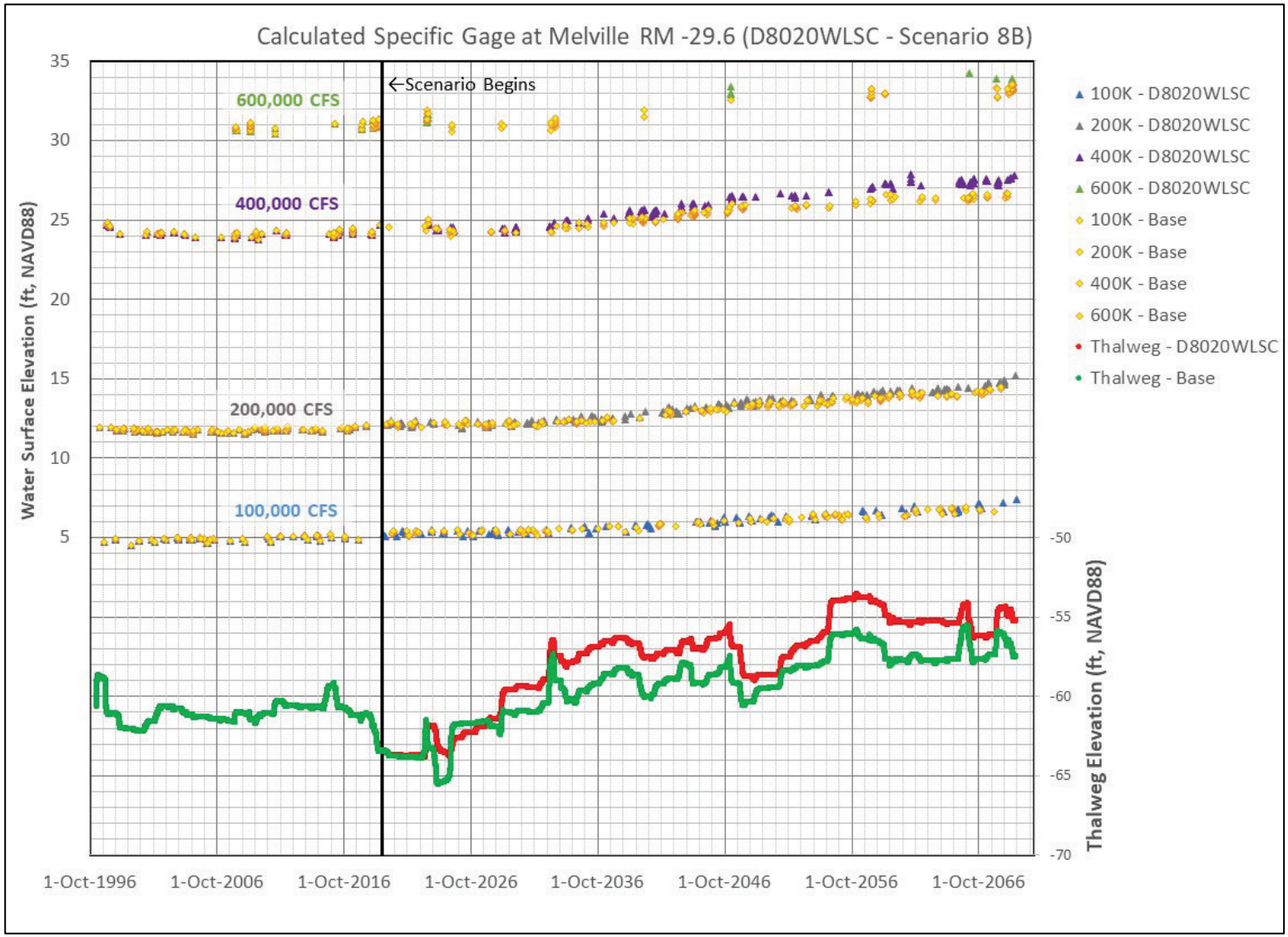

Calculated Specific Gage at Outflow Channel RM 4.33 (CTAR125 - Scenario 7)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
420,000 CFS
280,000 CFS
140,000 CFS
70,000 CFS
70K - CTAR125
140K - CTAR125
280K - CTAR125
420K - CTAR125
70K - Base
140K - Base
280K - Base
420K - Base
Thalweg - CTAR125
Thalweg - Base
55
45
35
25
15
5
-5
-15
-25
-35
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066

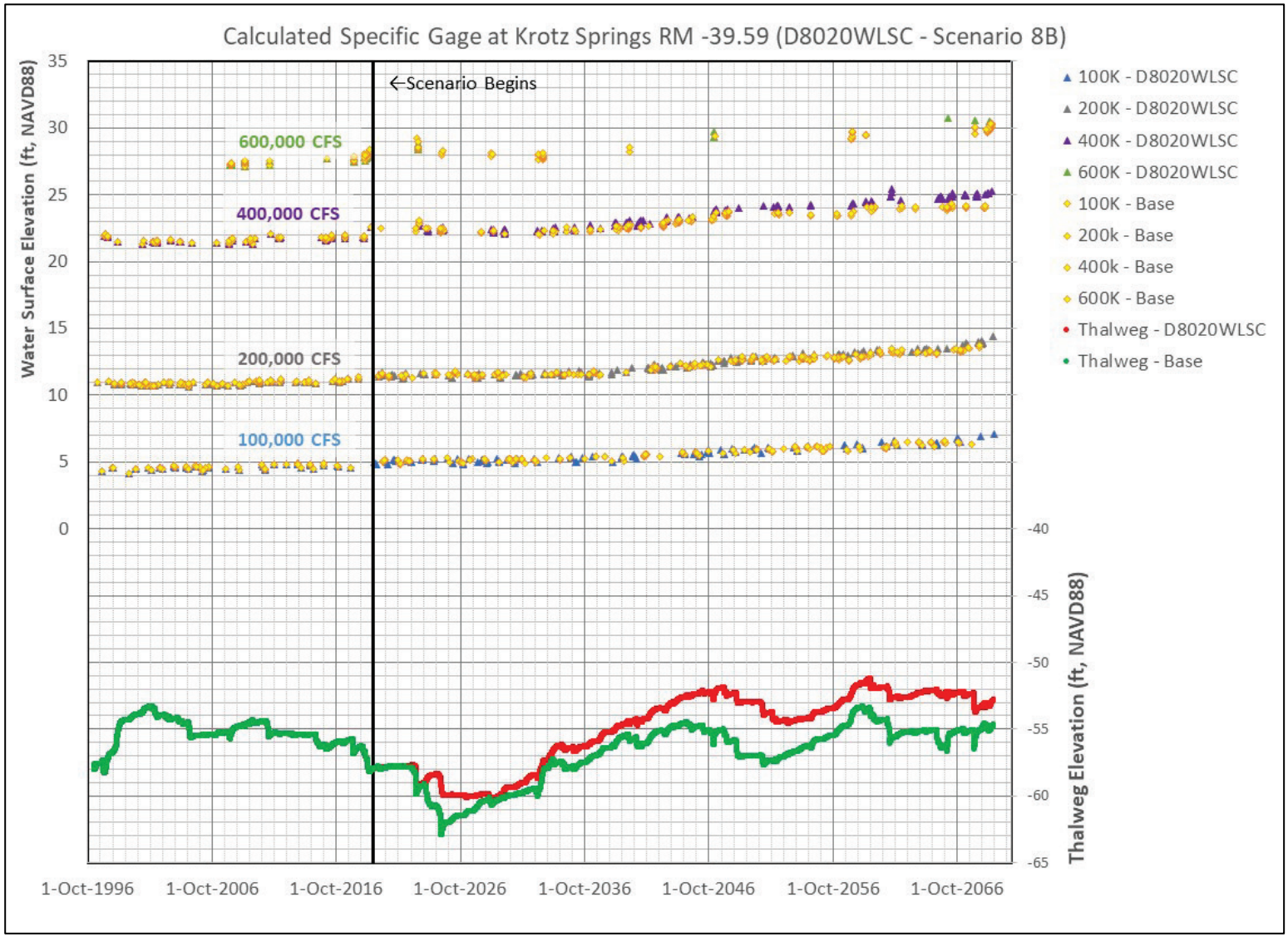

Calculated Specific Gage at Downstream end of Red River RM 10.6 (CTAR125 - Scenario 7)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
180,000 CFS
120,000 CFS
60,000 CFS
30,000 CFS
30K - CTAR125
60K - CTAR125
120K - CTAR125
180K - CTAR125
30K - Base
60K - Base
120K - Base
180K - Base
Thalweg - CTAR125
Thalweg - Base
65
55
45
35
25
15
5
-25
-30
-35
-40
-45
-50
-55
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066

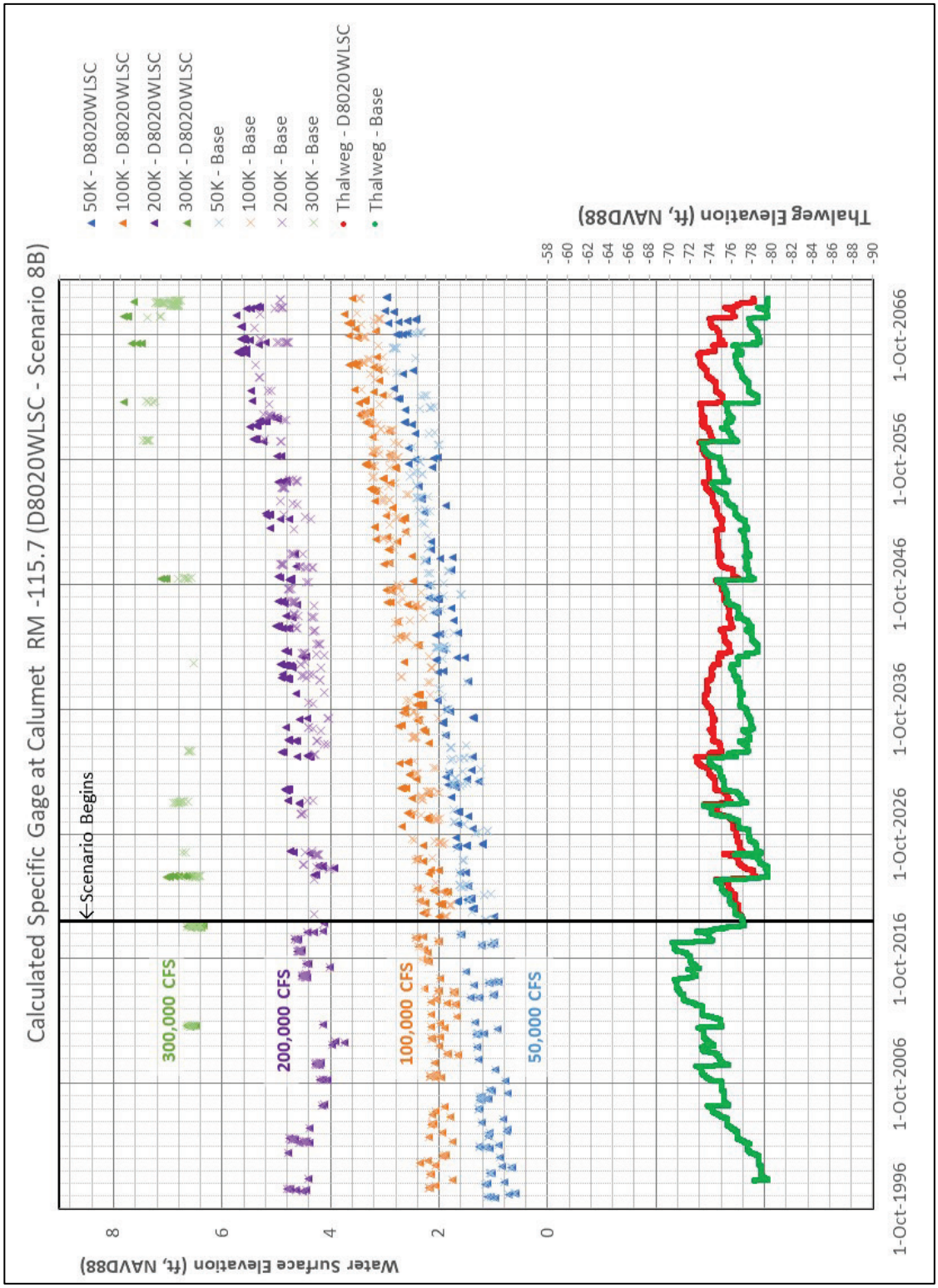

Calculated Specific Gage at Upstream end of Red River RM 33 (CTAR125 - Scenario 7)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
180,000 CFS
120,000 CFS
60,000 CFS
30,000 CFS
30K - CTAR125
60K - CTAR125
120K - CTAR125
180K - CTAR125
30K - Base
60K - Base
120K - Base
180K - Base
Thalweg - CTAR125
Thalweg - Base
65
55
45
35
25
15
10
0
-10
-20
-30
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066

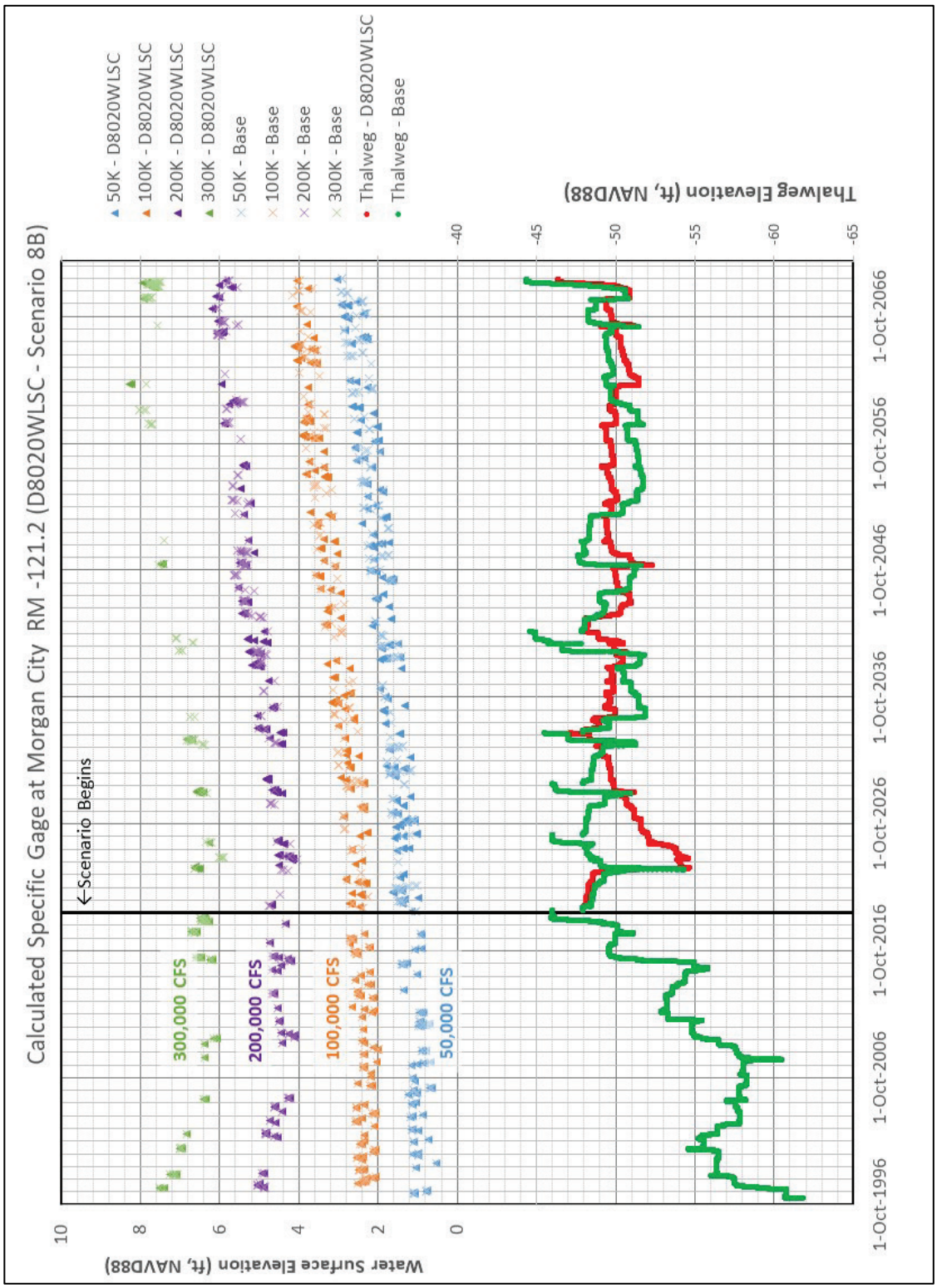
D8020WLSC Flow Regime – Scenario 8B

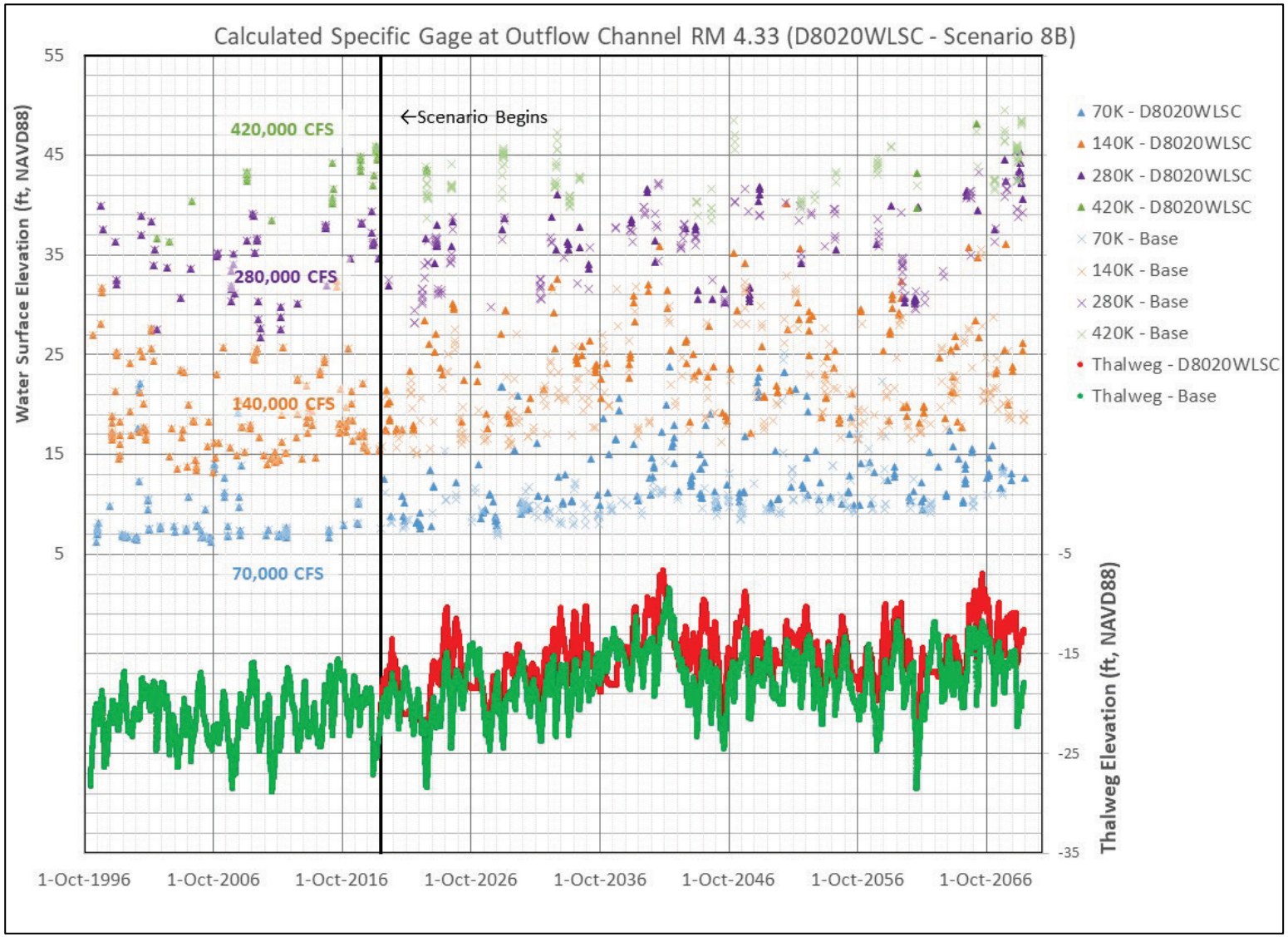


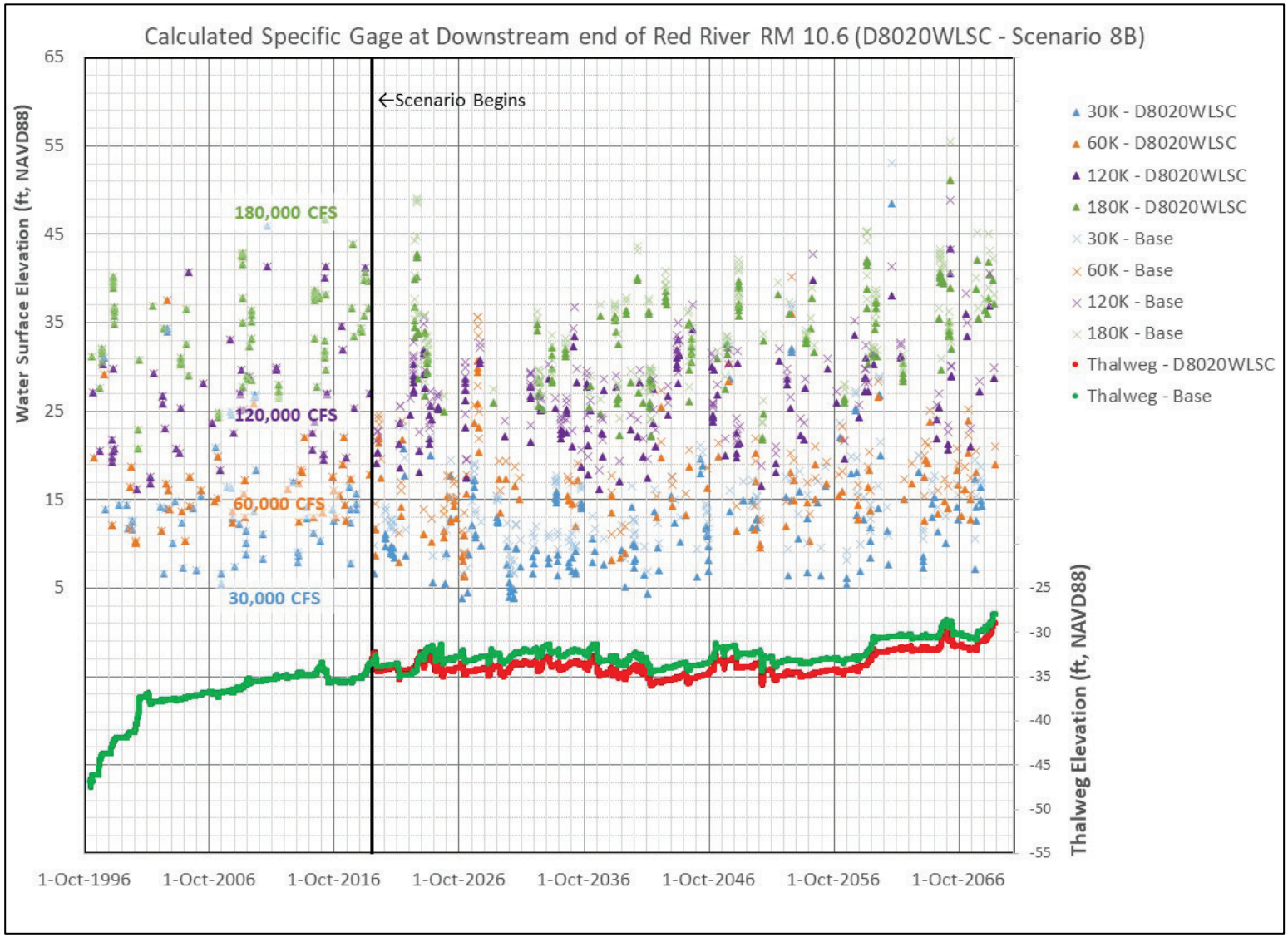

Calculated Specific Gage at Melville RM -29.6 (D8020WLSC - Scenario 8B)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
600,000 CFS
400,000 CFS
200,000 CFS
100,000 CFS
35
30
25
20
15
10
5
-50
-55
-60
-65
-70
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066
100K - D8020WLSC
200K - D8020WLSC
400K - D8020WLSC
600K - D8020WLSC
100K - Base
200K - Base
400K - Base
600K - Base
Thalweg - D8020WLSC
Thalweg - Base

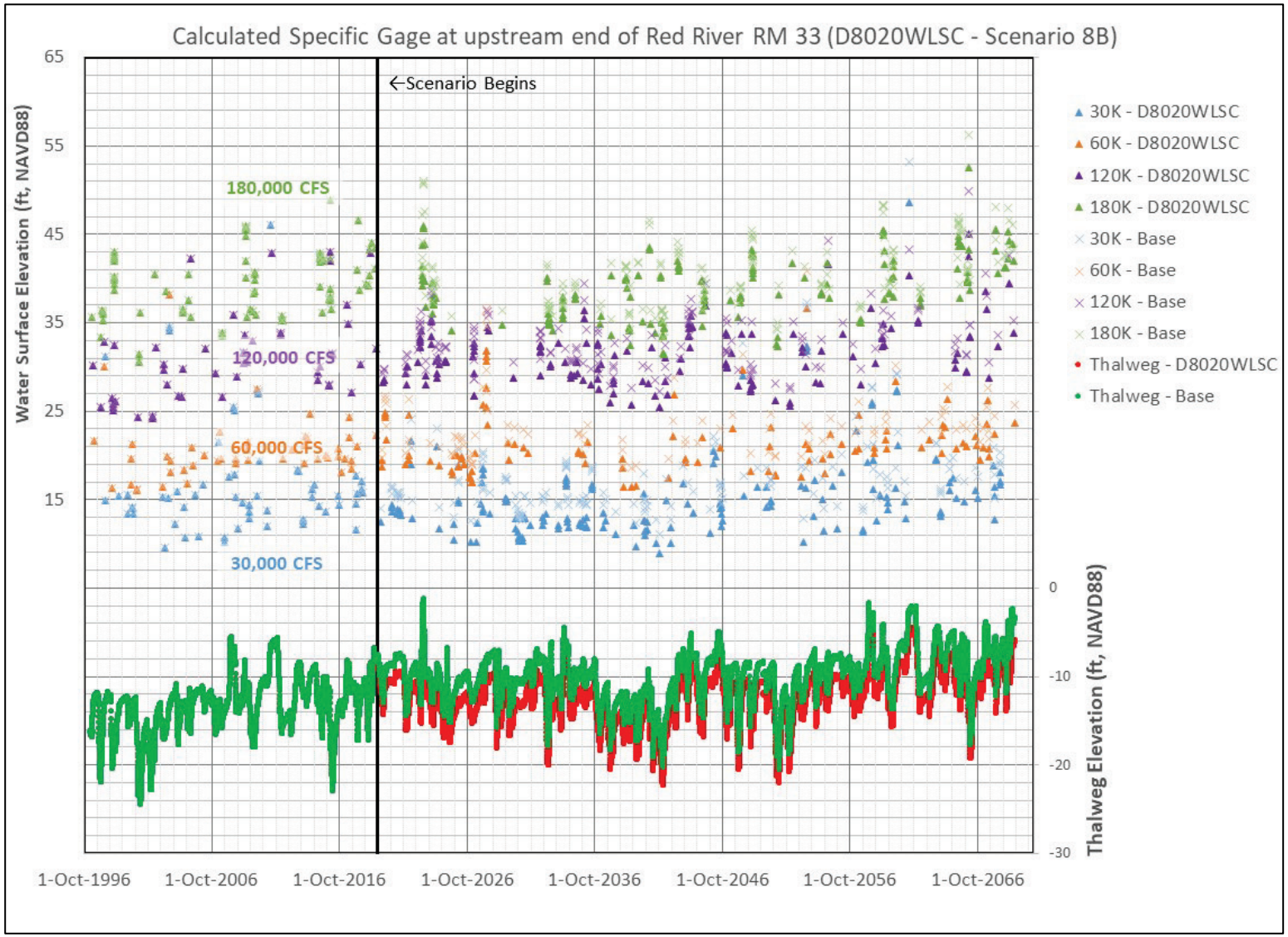

Calculated Specific Gage at Krotz Springs RM -39.59 (D8020WLSC - Scenario 8B)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
600,000 CFS
400,000 CFS
200,000 CFS
100,000 CFS
100K - D8020WLSC
200K - D8020WLSC
400K - D8020WLSC
600K - D8020WLSC
100K - Base
200k - Base
400k - Base
600K - Base
Thalweg - D8020WLSC
Thalweg - Base
35
30
25
20
15
10
5
0
-40
-45
-50
-55
-60
-65
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at Calumet RM -115.7 (D8020WLSC - Scenario 8B)
50K - D8020WLSC
100K - D8020WLSC
200K - D8020WLSC
300K - D8020WLSC
50K - Base
100K - Base
200K - Base
300K - Base
Thalweg - D8020WLSC
Thalweg - Base
←Scenario Begins
300,000 CFS
200,000 CFS
100,000 CFS
50,000 CFS
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066

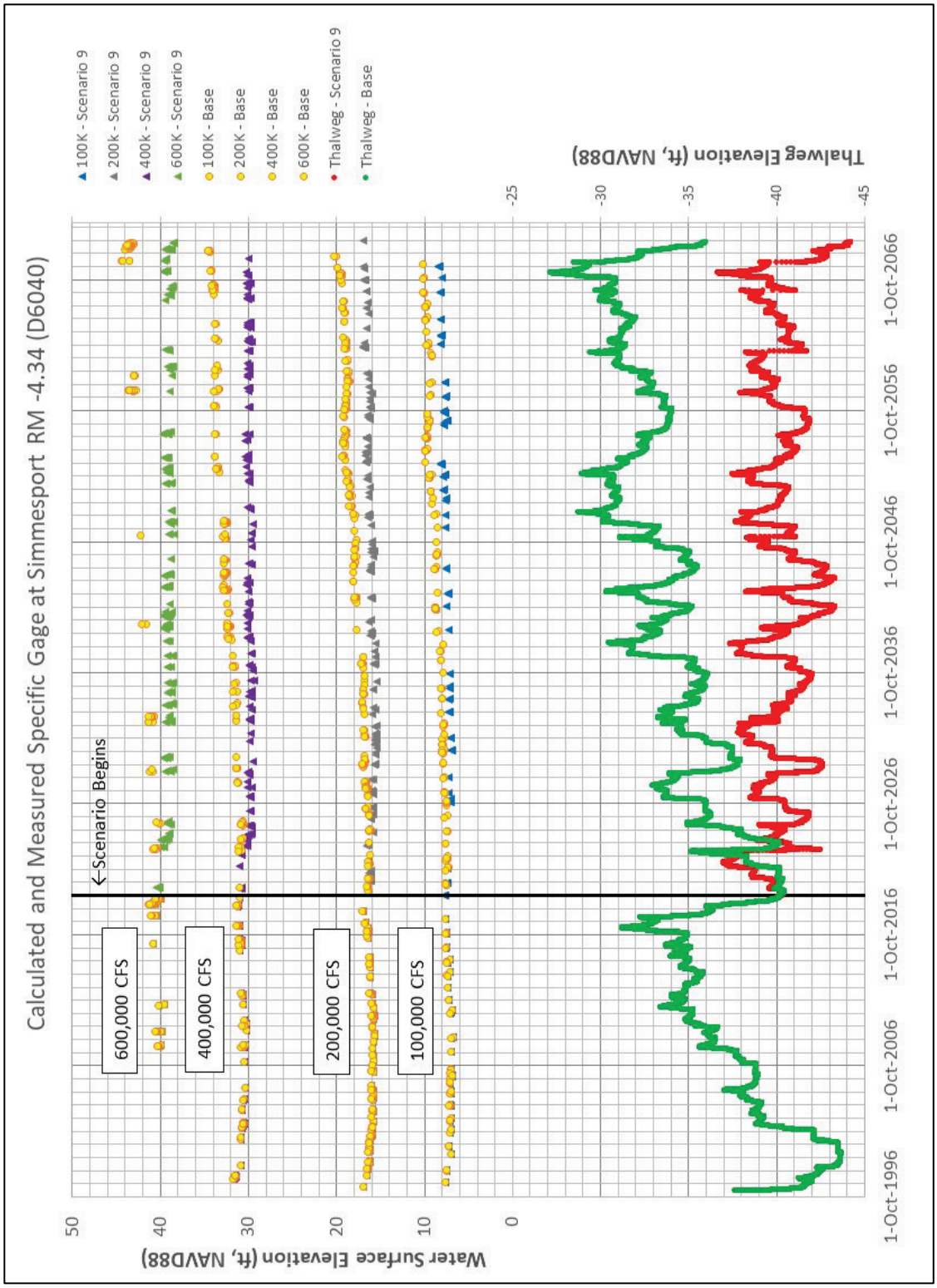

Calculated Specific Gage at Morgan City RM -121.2 (D8020WLSC - Scenario 8B)
50K - D8020WLSC
100K - D8020WLSC
200K - D8020WLSC
300K - D8020WLSC
50K - Base
100K - Base
200K - Base
300K - Base
Thalweg - D8020WLSC
Thalweg - Base
←Scenario Begins
300,000 CFS
200,000 CFS
100,000 CFS
50,000 CFS
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
10
8
6
4
2
0
-40
-45
-50
-55
-60
-65
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066

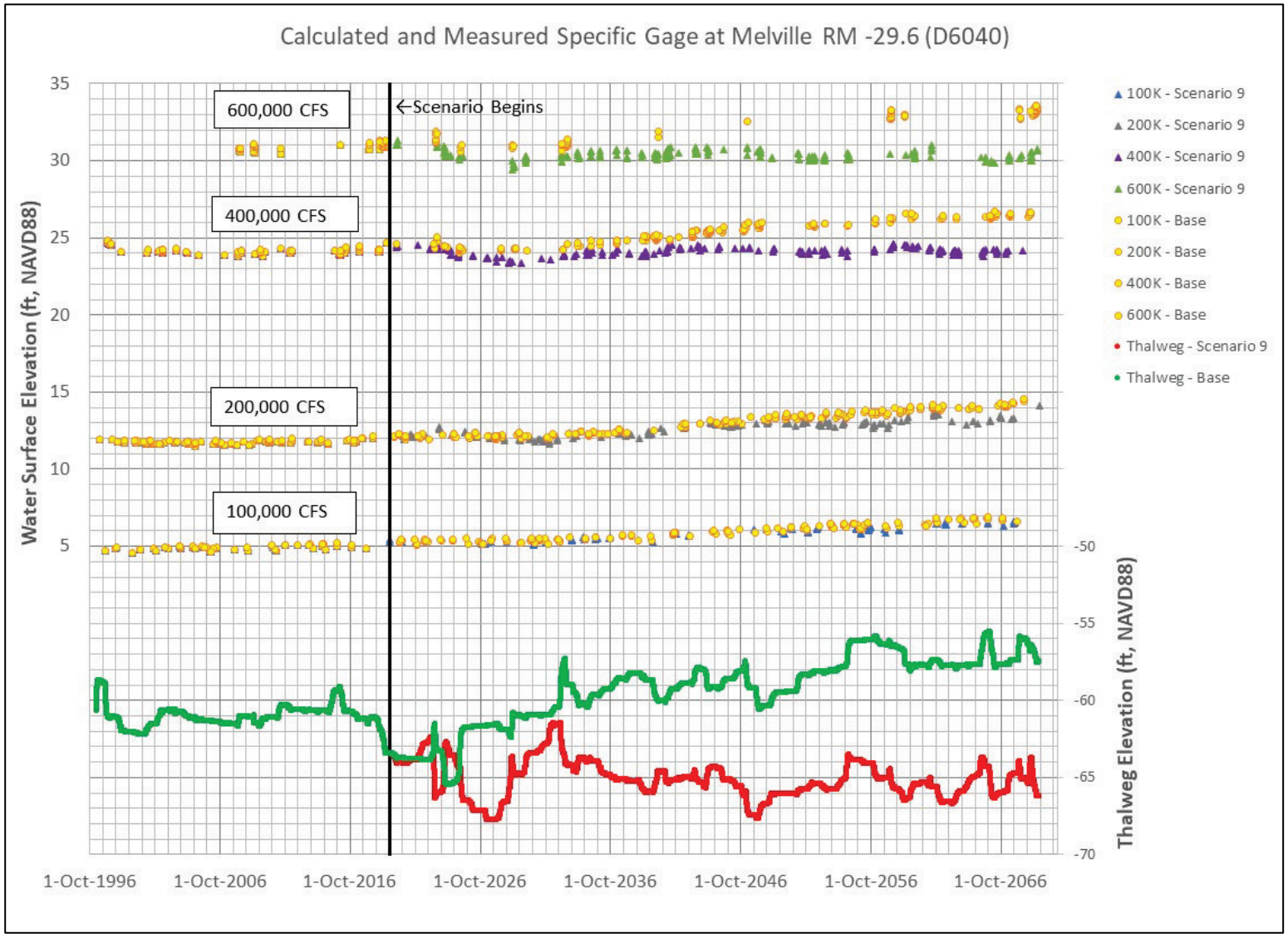

Calculated Specific Gage at Outflow Channel RM 4.33 (D8020WLSC - Scenario 8B)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
420,000 CFS
280,000 CFS
140,000 CFS
70,000 CFS
70K - D8020WLSC
140K - D8020WLSC
280K - D8020WLSC
420K - D8020WLSC
70K - Base
140K - Base
280K - Base
420K - Base
Thalweg - D8020WLSC
Thalweg - Base
55
45
35
25
15
5
-5
-15
-25
-35
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066

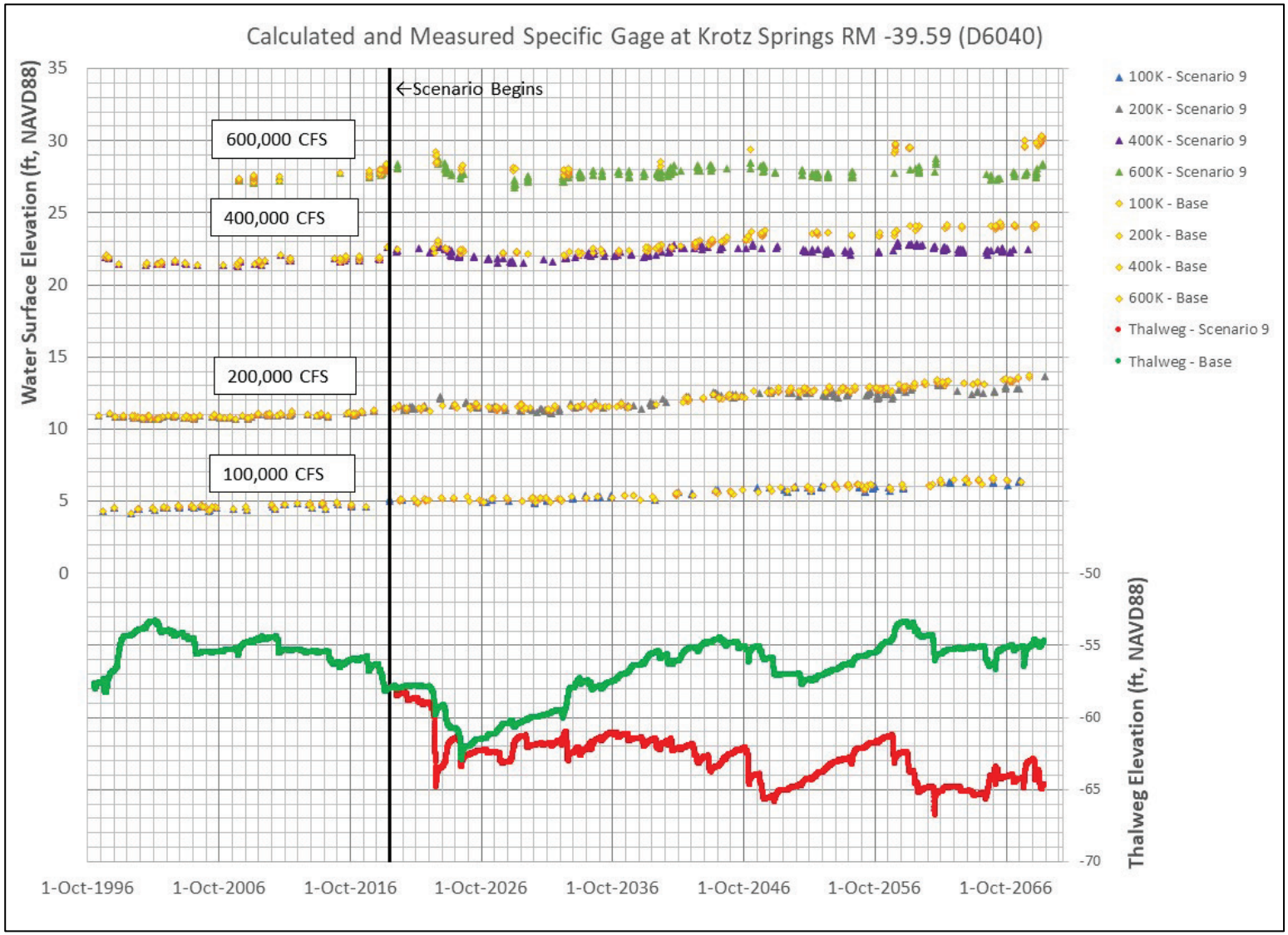

Calculated Specific Gage at Downstream end of Red River RM 10.6 (D8020WLSC - Scenario 8B)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
180,000 CFS
120,000 CFS
60,000 CFS
30,000 CFS
30K - D8020WLSC
60K - D8020WLSC
120K - D8020WLSC
180K - D8020WLSC
30K - Base
60K - Base
120K - Base
180K - Base
Thalweg - D8020WLSC
Thalweg - Base
65
55
45
35
25
15
5
-25
-30
-35
-40
-45
-50
-55
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated Specific Gage at upstream end of Red River RM 33 (D8020WLSC - Scenario 8B)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
180,000 CFS
120,000 CFS
60,000 CFS
30,000 CFS
30K - D8020WLSC
60K - D8020WLSC
120K - D8020WLSC
180K - D8020WLSC
30K - Base
60K - Base
120K - Base
180K - Base
Thalweg - D8020WLSC
Thalweg - Base
65
55
45
35
25
15
0
-10
-20
-30
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066

D6040 Flow Regime – Scenario 9




Calculated and Measured Specific Gage at Melville RM -29.6 (D6040)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
600,000 CFS
400,000 CFS
200,000 CFS
100,000 CFS
←Scenario Begins
100K - Scenario 9
200K - Scenario 9
400K - Scenario 9
600K - Scenario 9
100K - Base
200K - Base
400K - Base
600K - Base
Thalweg - Scenario 9
Thalweg - Base
35
30
25
20
15
10
5
-50
-55
-60
-65
-70
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
1-Oct-2036
1-Oct-2046
1-Oct-2056
1-Oct-2066


Calculated and Measured Specific Gage at Krotz Springs RM -39.59 (D6040)
Water Surface Elevation (ft, NAVD88)
Thalweg Elevation (ft, NAVD88)
←Scenario Begins
600,000 CFS
400,000 CFS
200,000 CFS
100,000 CFS
100K - Scenario 9
200K - Scenario 9
400K - Scenario 9
600K - Scenario 9
100K - Base
200k - Base
400k - Base
600K - Base
Thalweg - Scenario 9
Thalweg - Base
35
30
25
20
15
10
5
0
-50
-55
-60
-65
-70
1-Oct-1996
1-Oct-2006
1-Oct-2016
1-Oct-2026
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1-Oct-2046
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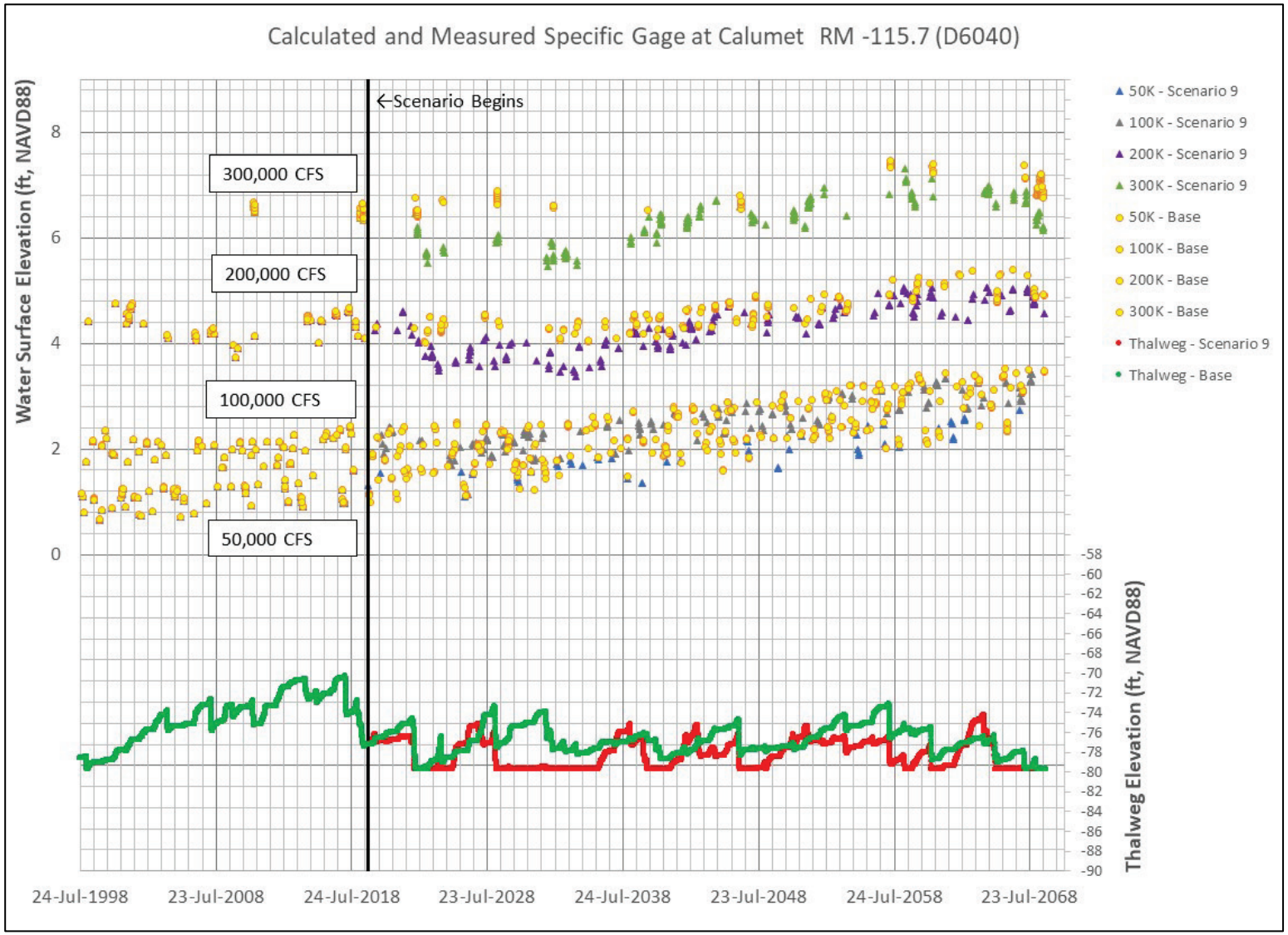
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Thalweg Elevation (ft, NAVD88)
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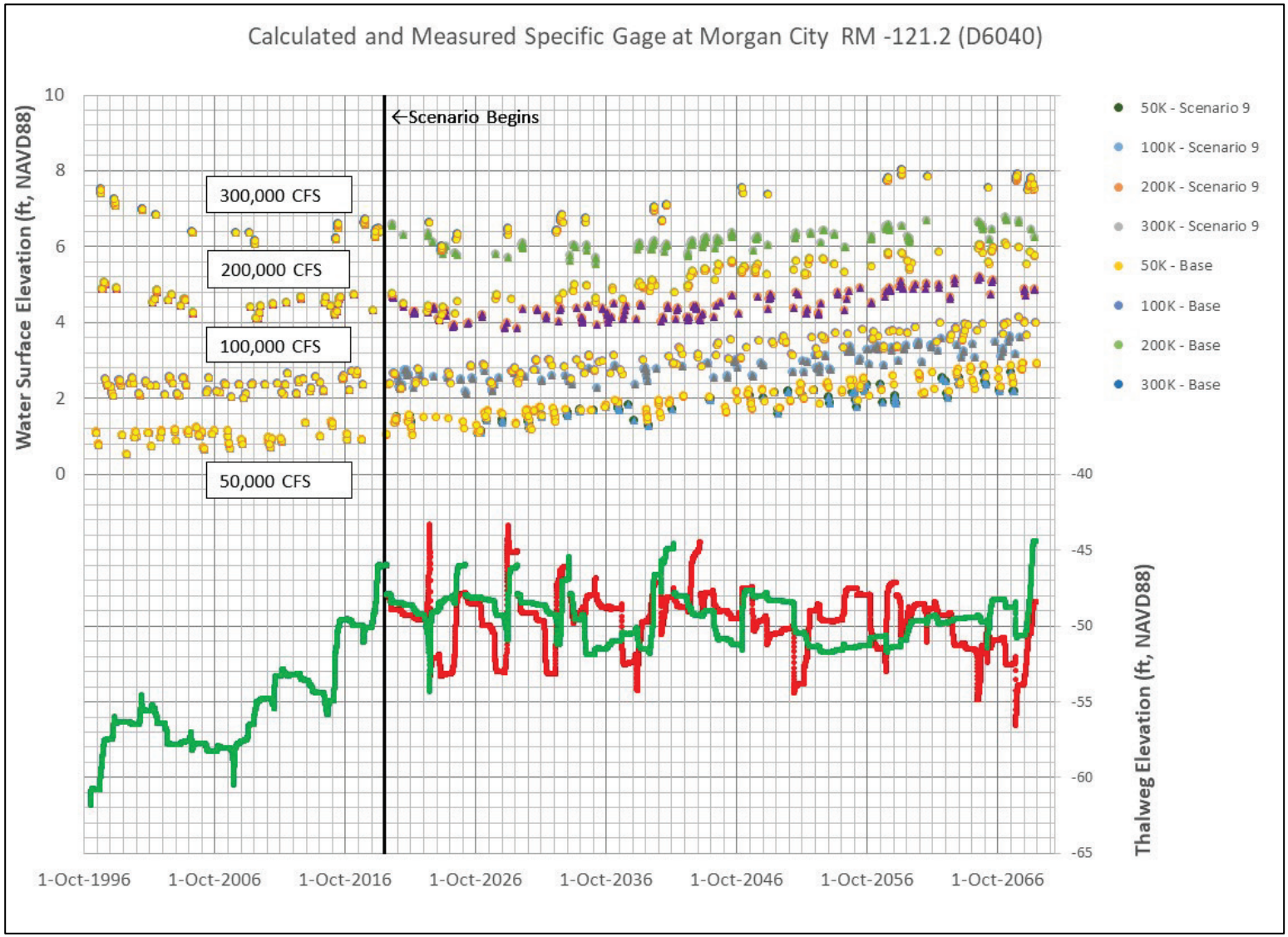
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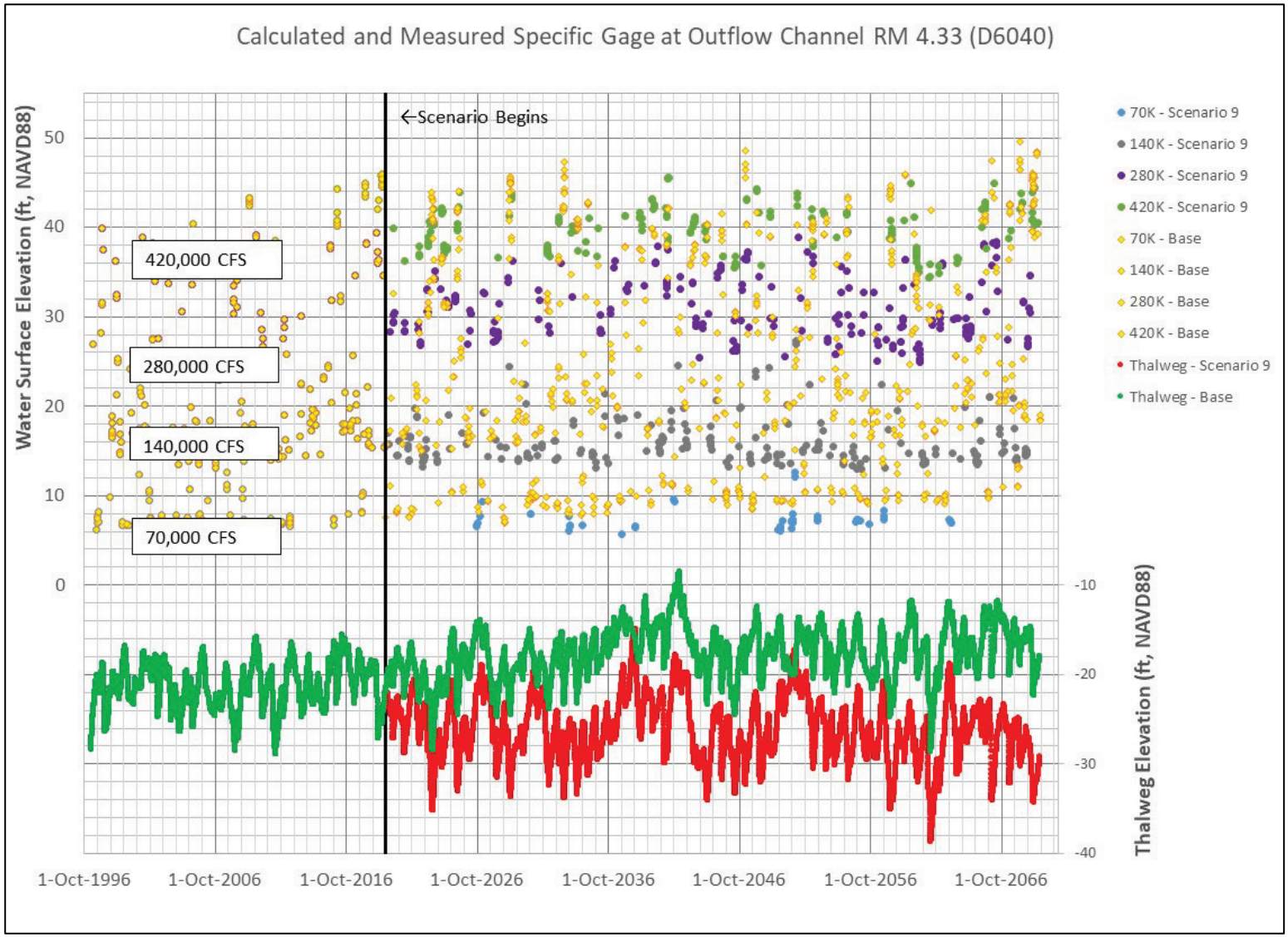
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280,000 CFS
140,000 CFS
70,000 CFS
70K - Scenario 9
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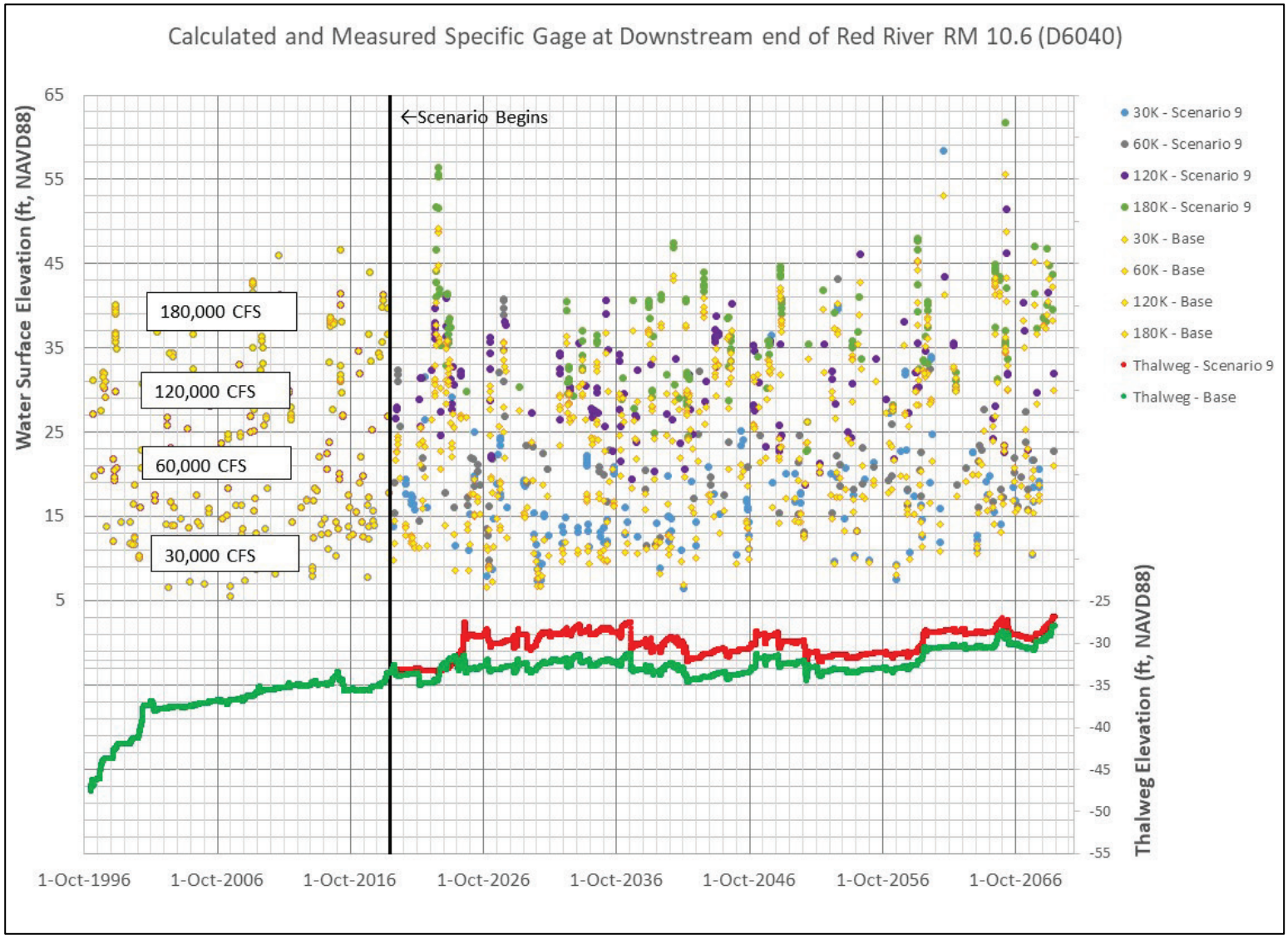
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Thalweg - Base
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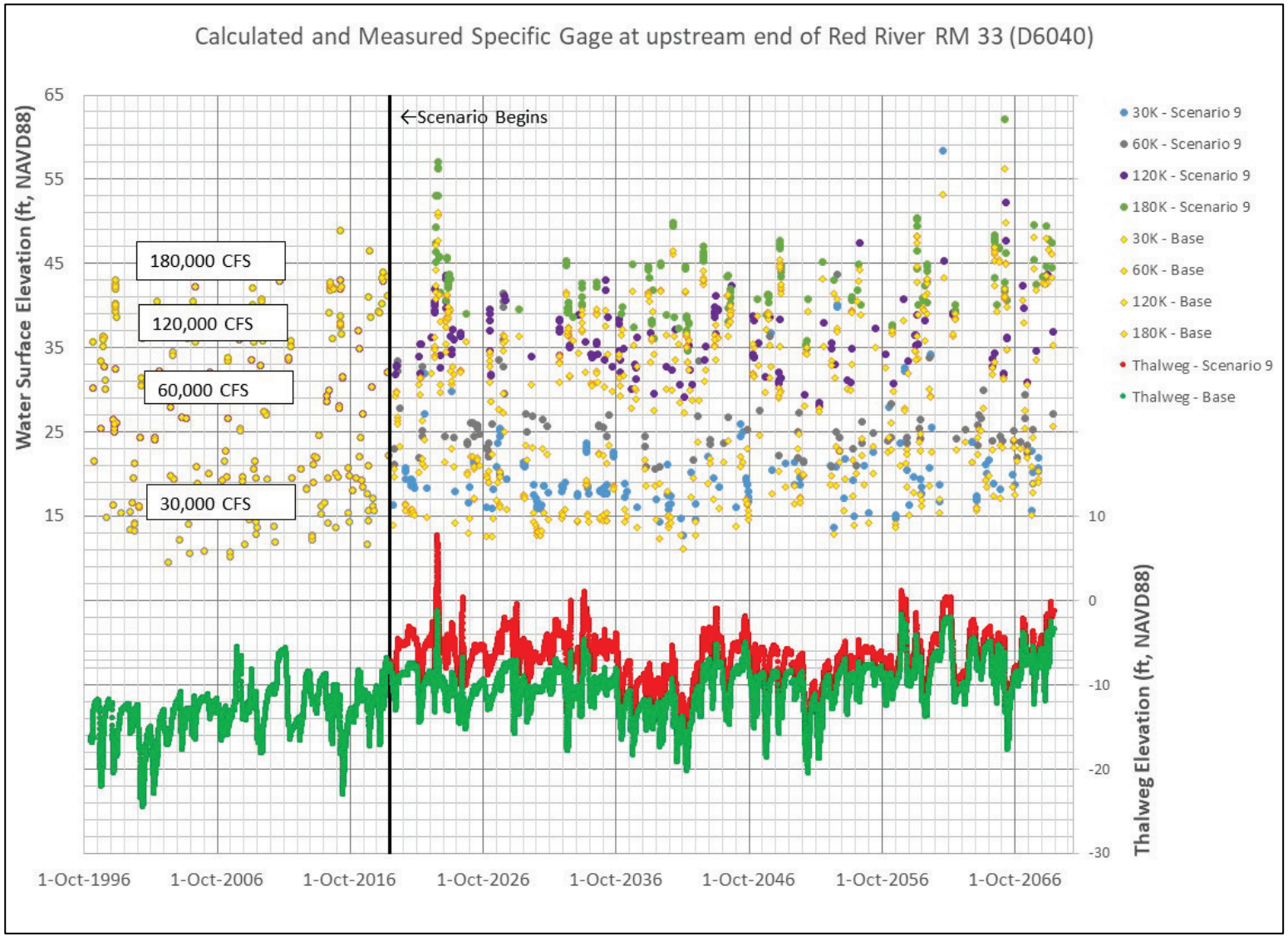
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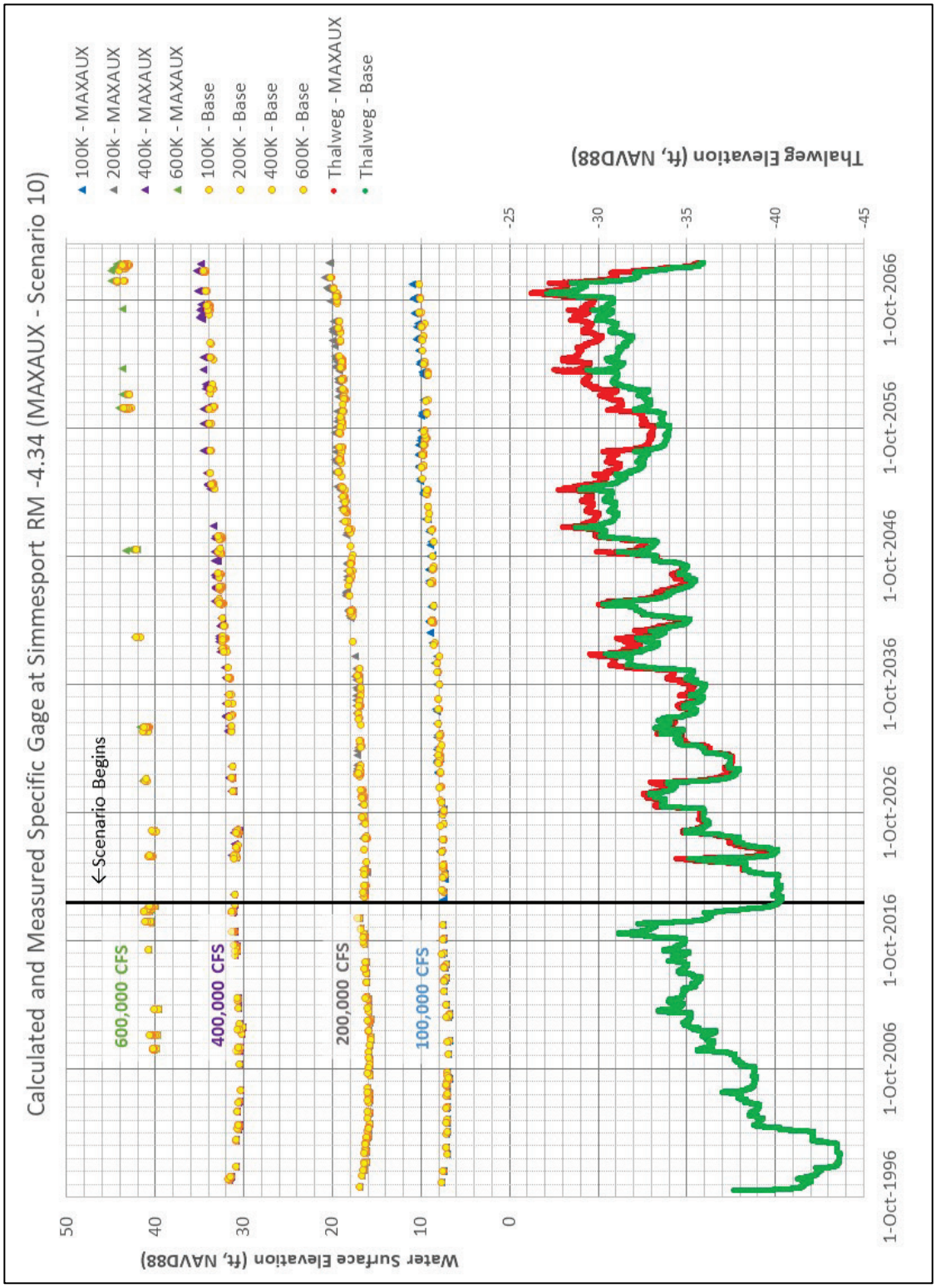
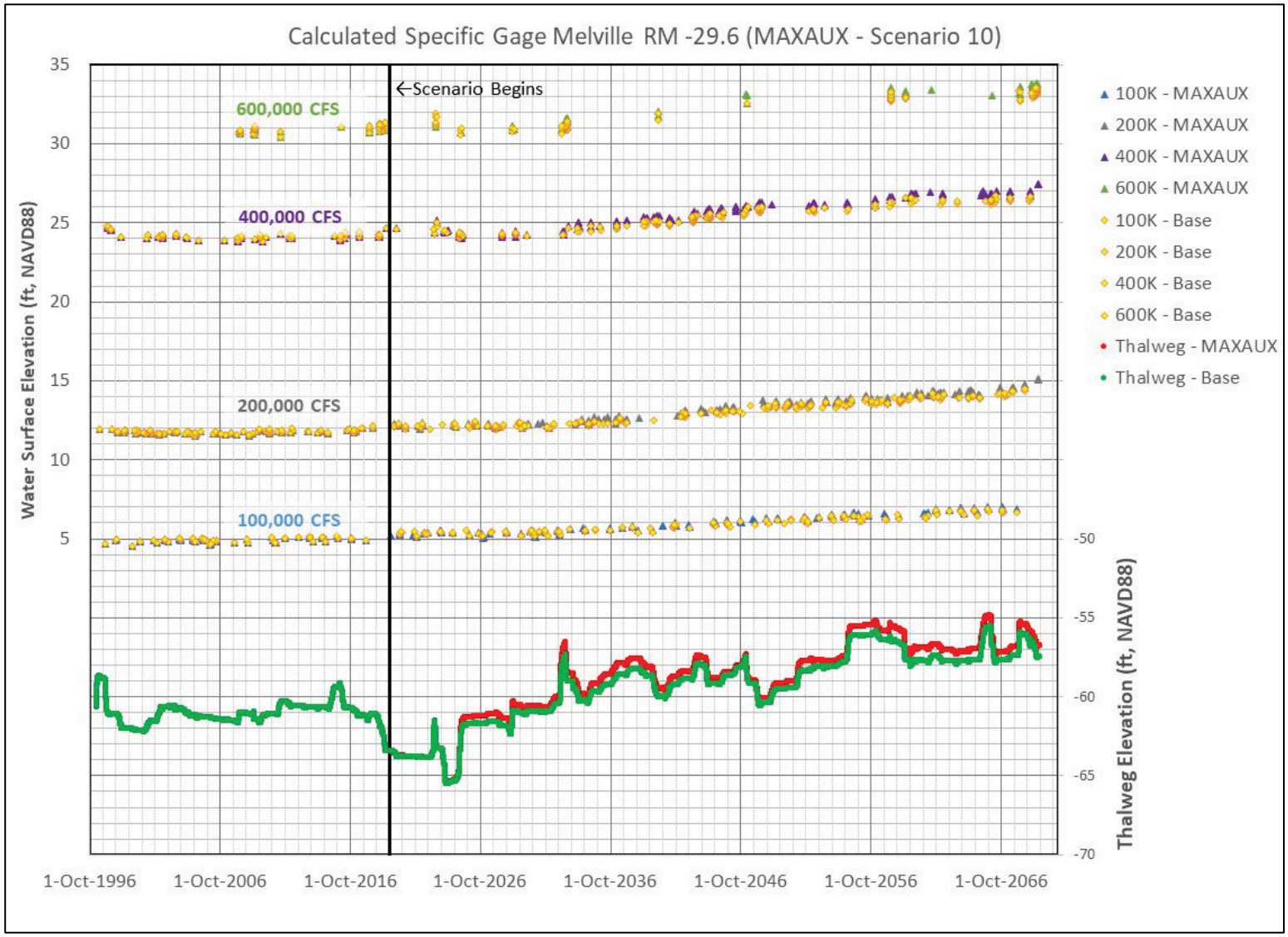
MaxAux Flow Regime – Scenario 10

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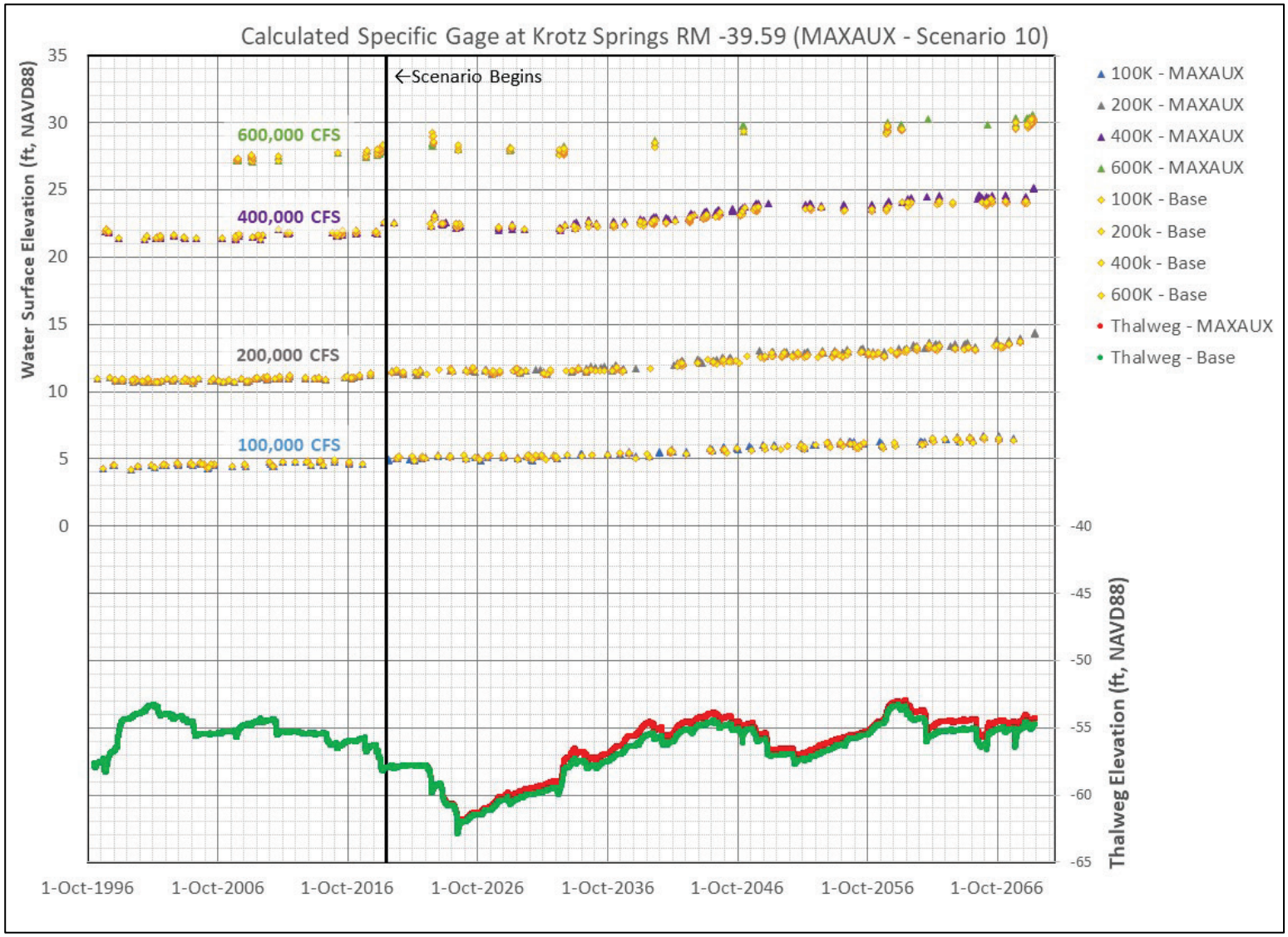
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Thalweg Elevation (ft, NAVD88)
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100K - MAXAUX
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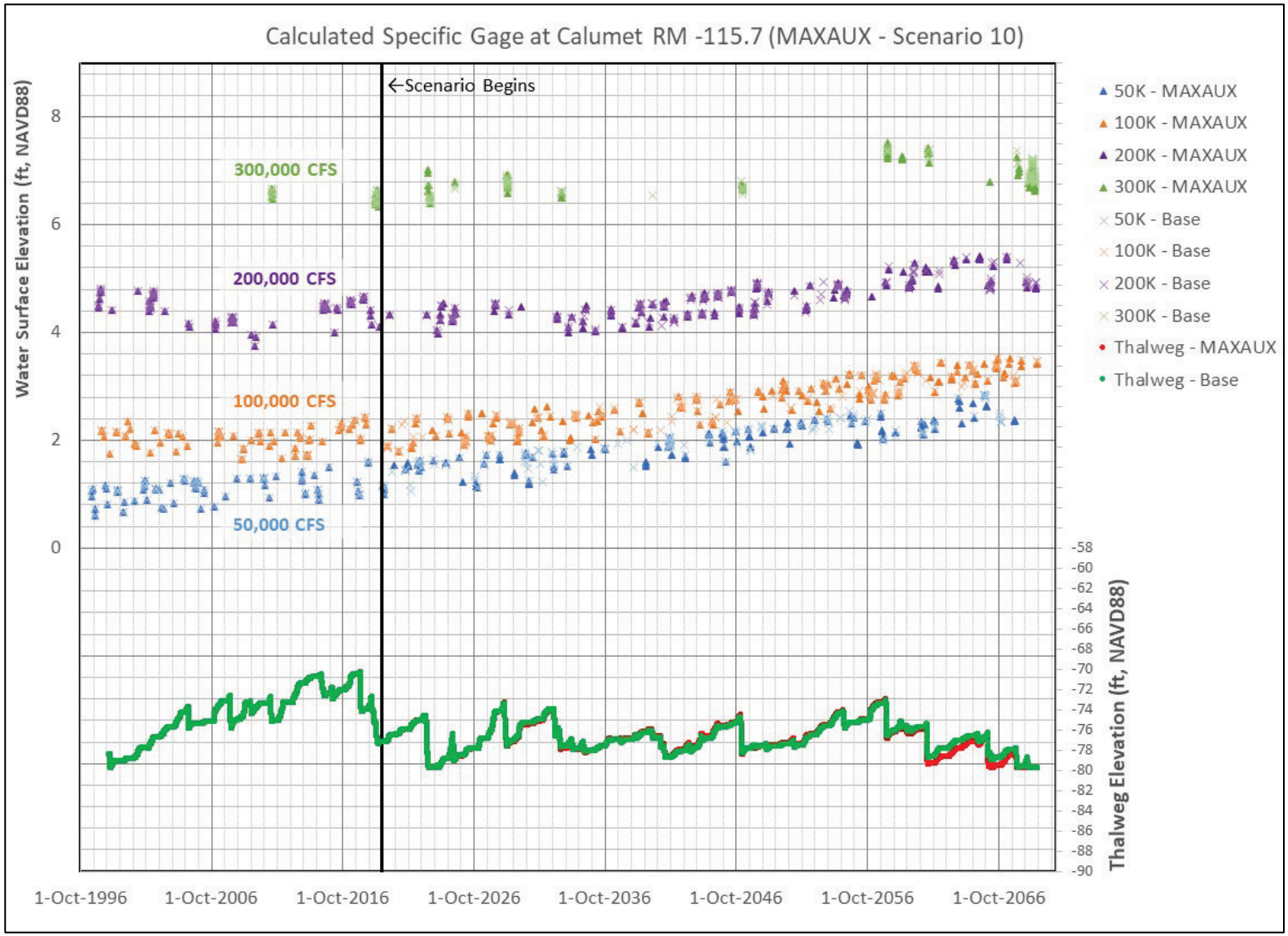
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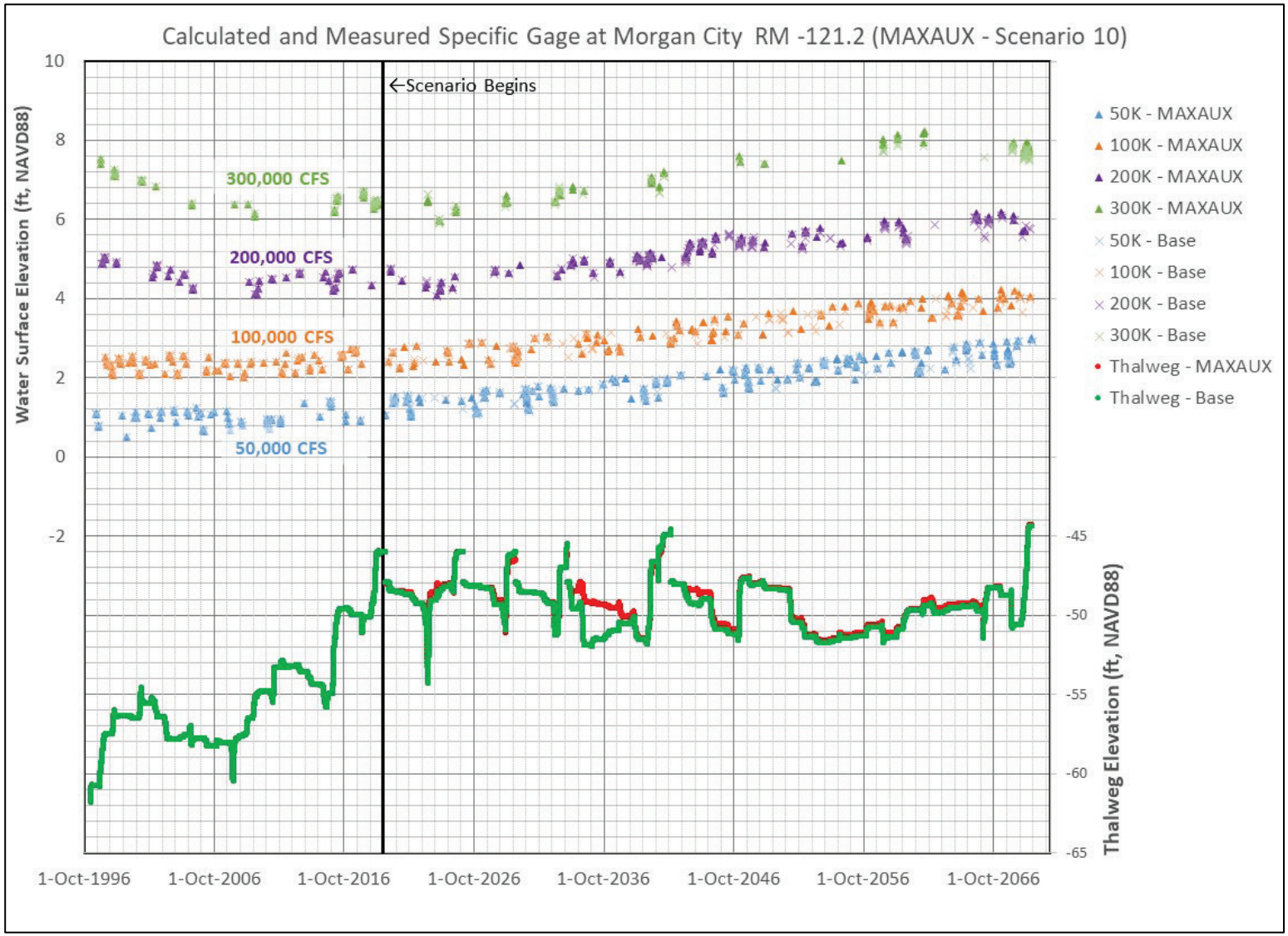
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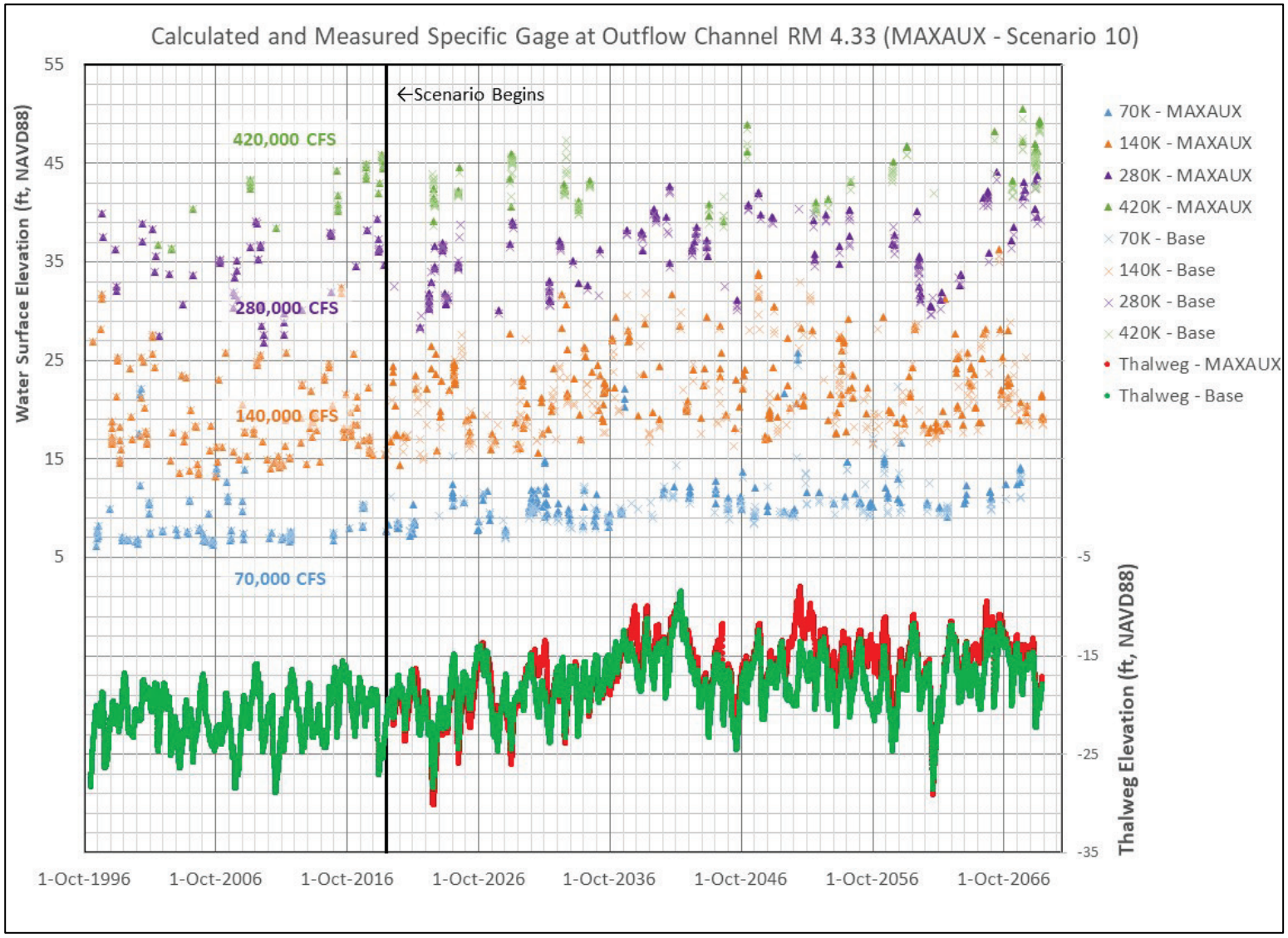
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Thalweg - MAXAUX
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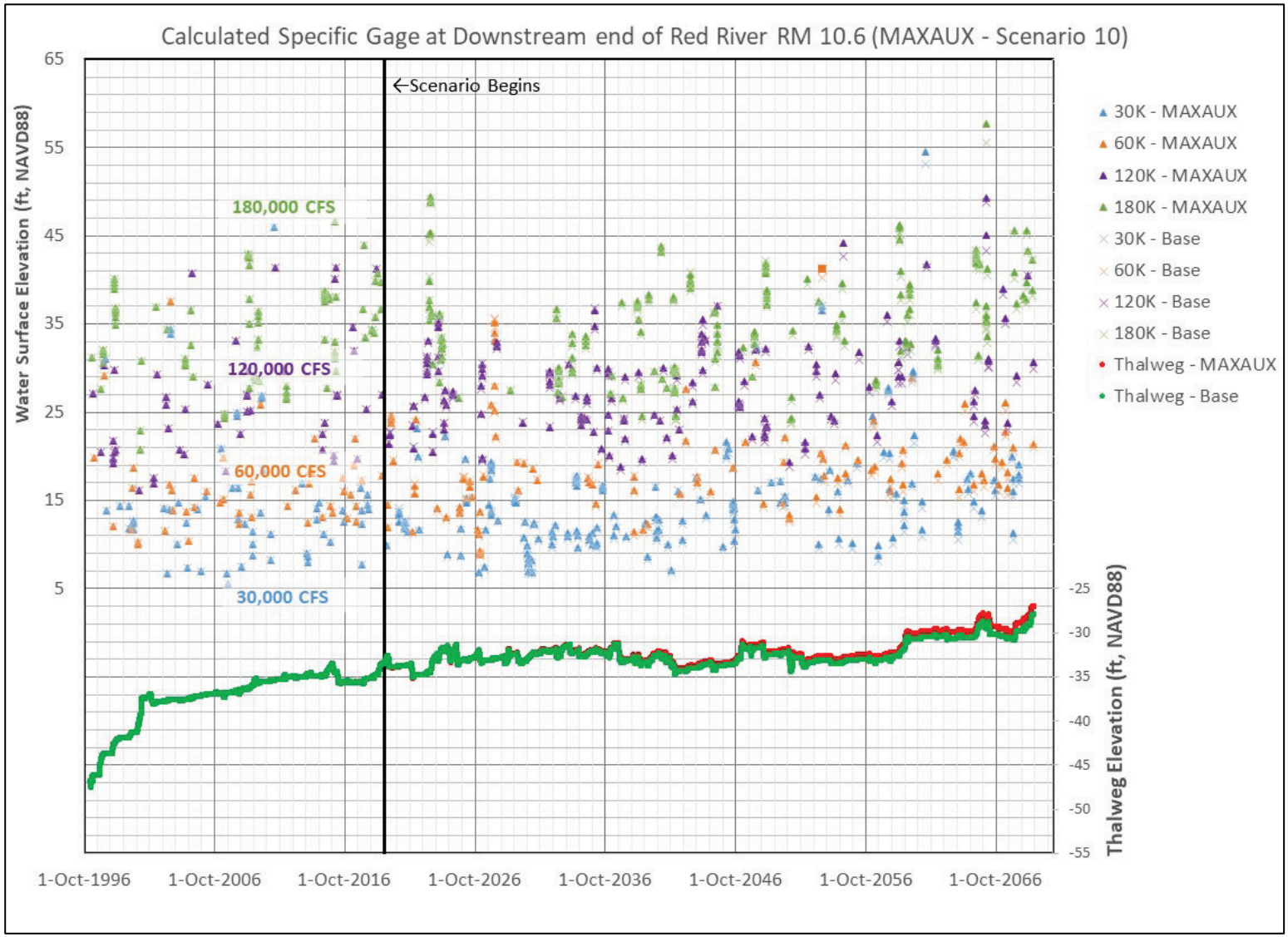
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Thalweg Elevation (ft, NAVD88)
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Thalweg - Base
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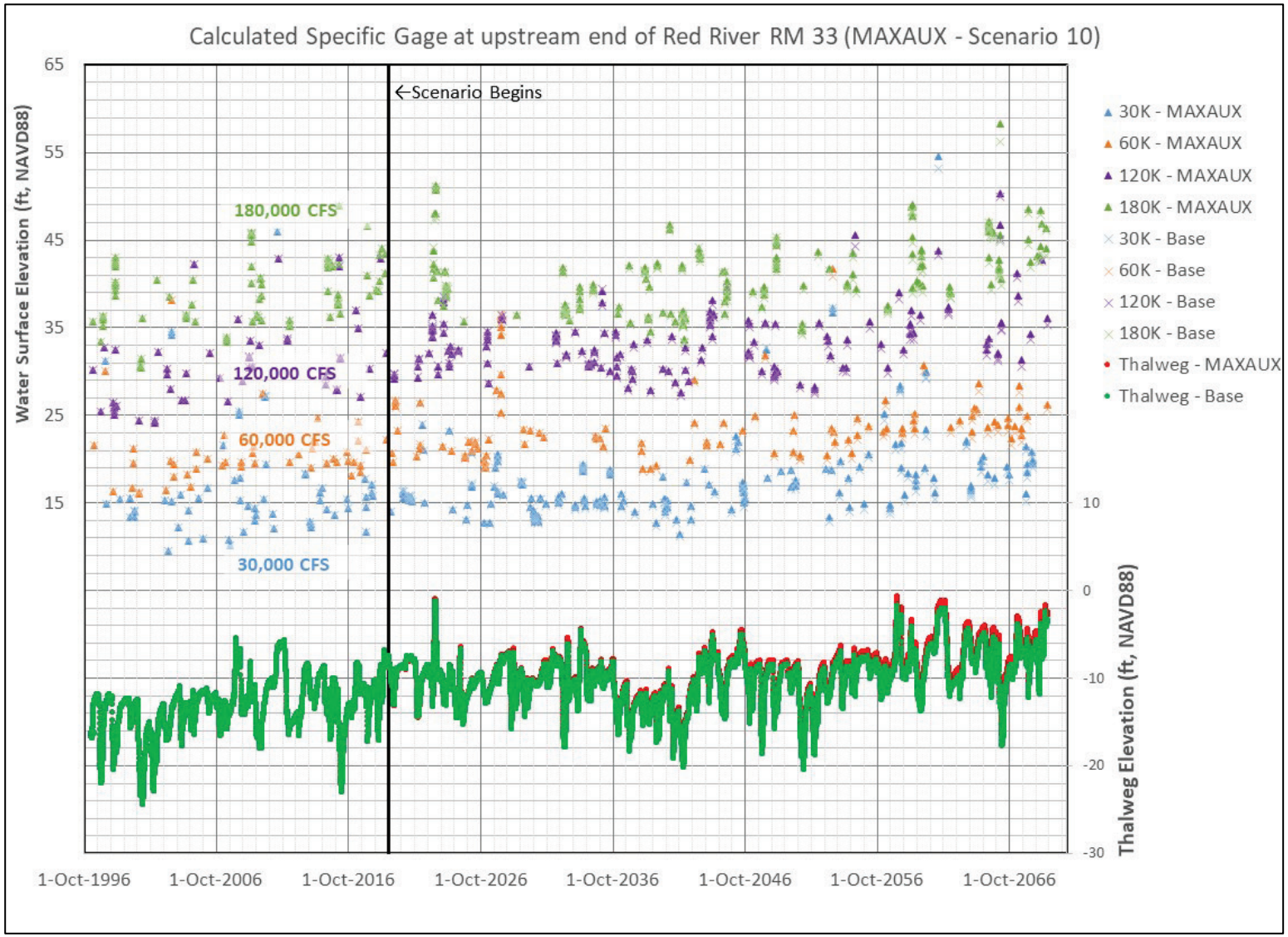
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Abbreviations

1D	One-dimensional
2D	Two-dimensional
LAWMA	Lower Atchafalaya Wildlife Management Area
MBH	Mobile Boundary Hydraulics
MCS	Morganza Control Structure
MR&T	Mississippi River and Tributaries
MSL	Mean sea level
MVK	Vicksburg District
MVN	New Orleans District
NAVD88	North American Vertical Datum of 1988
OMAR	Old, Mississippi, Atchafalaya, and Red Rivers
ORCC	Old River Control Complex
ORCS	Old River Control Structure
ORO	Sediment gage on the Outflow Channel
PDF	Project Design Flood
RM	River Mile
USACE	United States Army Corps of Engineers
USGS	US Geological Survey
WSEL	Water-surface elevation

REPORT DOCUMENTATION PAGE

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

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5c. PROGRAM ELEMENT NUMBER

6. AUTHOR(S)
Patrick I. Donohue, Ronald R. Copeland, and James W. Lewis

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14. ABSTRACT
The HEC-6T one-dimensional numerical sedimentation model was used to evaluate the long-term and system-wide sedimentation effects of modifying the operation schedule at the Old River Control Complex (ORCC). The changes evaluated were increasing and decreasing the percentage of flow that is diverted to the Atchafalaya River from the Mississippi River at the ORCC and modifying the distributions through the four ORCC structures.

Sedimentation effects for several operation scenarios were compared to a Base Condition that represented the existing operation protocols. Additionally, a dredging scenario was developed and analyzed. This scenario featured dredging material from the Mississippi River and depositing it into the Outflow Channel, downstream of the ORCC. The predictive simulations extended for 50 years. The model was used to calculate and compare sand transport past various gages along the Atchafalaya River. Also, the model compared differences in water surface elevations and bed elevations using a specific gage analysis at several gages along the Atchafalaya River. Last, the effects of the various scenarios on annual dredging requirements in Berwick Bay were also determined.

15. SUBJECT TERMS
Atchafalaya River (La.), Dredging, Hydraulic models, Hydraulic structures, Old River (La.), Sedimentation and deposition, Sediment transport

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