

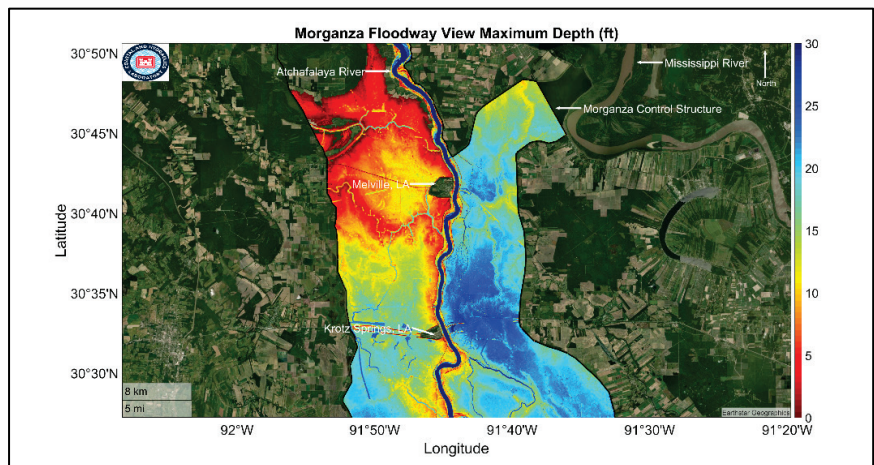


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# Hydrodynamics in the Morganza Floodway and Atchafalaya Basin, Report 3: Phase 3

A Report for the US Army Corps of  
Engineers, MRG&P

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MRG&P

Mississippi River  
Geomorphology &  
Potamology Program





# **Hydrodynamics in the Morganza Floodway and Atchafalaya Basin, Report 3: Phase 3**

A Report for the US Army Corps of Engineers, MRG&P

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Final report

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## Abstract

The Morganza Floodway and the Atchafalaya Basin, located in Louisiana west of the Mississippi River, were evaluated using a two-dimensional Adaptive Hydraulics model. Prior to this study, Phase 1 and 2 model studies were performed that indicated that the existing floodway may not be able to pass the Project Design Flood discharge of 600,000 cubic feet per second due to levee overtopping. In this study, all elevations of exterior and interior levees were updated with current crest elevations. In addition, the Phase 3 effort evaluated the sensitivity of the floodway's flow capacity to variations in tree/vegetation density conditions. These adjustments in roughness will improve the understanding of the role of land cover characteristics in the simulated water surfaces. This study also provides a number of inundation maps corresponding to certain flows through the Morganza Control Structure.

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## Preface

This study was conducted for the Mississippi River Geomorphology and Potamology (MRG&P) Program, under Project No. 490391, US Army Corps of Engineers, Mississippi Valley Division (MVD). The MRG&P Technical Director was Dr. James W. Lewis. The MVD Commander was MG Diana Holland. The MVD Director of Programs was Mr. Edward E. Belk, Jr.

The work was performed by the River and Estuarine Engineering Branch of the Flood and Storm Protection Division, US Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory (ERDC CHL). At the time of publication of this report, Mr. David P. May was Branch Chief; Dr. Cary A. Talbot was Division Chief; and Dr. Julie D. Rosati was the Technical Director for the Flood and Coastal Risk Management Research and Development. The Deputy Director of ERDC CHL was Mr. Keith Flowers, and the Director was Dr. Ty V. Wamsley.

The Commander of ERDC was COL Teresa A. Schlosser, and the Director was Dr. David W. Pittman.

# 1 Introduction

This US Army Engineer Research and Development Center, Coastal and Hydraulics Laboratory, report presents a description of the Phase 3 process to modify, simulate, and analyze a two-dimensional (2D) Adaptive Hydraulics (AdH) model of the Morganza and Atchafalaya Floodways. It is an extended effort of the Phase 1 and 2 studies of the two floodways (Bell et al. 2017, 2018). Four tasks were completed in the Phase 3 effort:

- Task 1: Modify the existing mesh with any new information obtained for levee elevations and/or bathymetry.
- Task 2: Verify that the updated model still produces valid output hydrographs at the locations throughout the model domain for which measured data are available.
- Task 3: Explore hypothetical variations in vegetation density at strategic locations throughout the floodways to evaluate the sensitivity of flow capacity and levee overtopping.
- Task 4: Provide inundation maps at strategic locations for varying flow conditions through the Morganza Control Structure (MCS) and Atchafalaya River.

## 1.1 Background

The entire model domain (Figure 1, outlined in white) extends from the village of Hamburg, LA, at its northwest corner, to the Gulf of Mexico at its southern-most extremity. The length of the model domain is approximately 180 mi<sup>1,2</sup> along its central longitudinal axis, and its average width is approximately 13 mi (from the East Atchafalaya Basin Project Levee [EABPL] to the West Atchafalaya Basin Project Levee [WABPL]), with a minimum of approximately 7 mi at the northern end and a maximum of approximately 18 mi at the southern end. Farther

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<sup>1</sup> 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>.

<sup>2</sup> 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>.

downstream, the southern portion of the model (downstream of the Wax Lake Outlet by Calumet, LA, and the Lower Atchafalaya River at Morgan City, LA) is approximately 120 – 190 mi wide and includes the marshes and a part of the Gulf of Mexico. The northern part of the basin is divided by the Atchafalaya River and its levees, with the West Atchafalaya Basin (WAB) to the west of the river levees and the Morganza Floodway to the east of the river levees. South of Interstate 10 at a location near Butte La Rose, the Atchafalaya River levees end, and if the water level is high enough to be above the natural ground elevations, the flows from the west and east Atchafalaya basins merge. The combined flow then moves between the WABPL and the EABPL south through the Lower Atchafalaya River and the Wax Lake Outlet to the Gulf of Mexico. Figure 2 shows the model location from the upstream boundary to Morgan City and Wax Lake Outlet. Figure 2 also displays the extents of the levees that run along both sides of the Atchafalaya River (extending from the upstream model boundary to approximately 9 mi north of Butte La Rose and 1.75 mi south of Butte La Rose for the east and west Atchafalaya River levees, respectively). If flow in the Mississippi and Atchafalaya Rivers is high, then it is possible that Mississippi River floodwaters diverted into the Morganza Floodway can also move upstream (north) into the Atchafalaya River or the WAB, connected near Butte La Rose where the upstream river levees end.

Figure 1. Model domain.

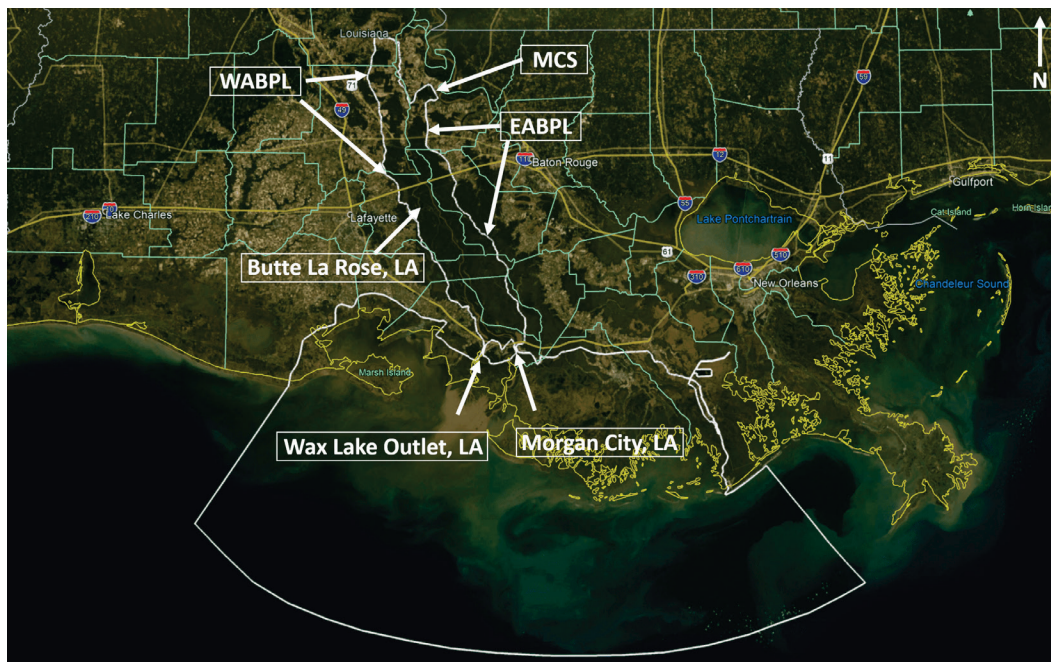


Figure 2. Morganza/Atchafalaya floodway location. The brown lines indicate the position of levees.



## 1.2 Objectives

One of the findings from the Phase 2 study was that the simulated Project Design Flood (PDF) flow of 600,000 cfs through the Morganza Floodway raised concerns about the existing levee heights in some locations. Possible causes of this could be inaccurate levee elevations in the model and extensive vegetative growth in the floodway since its construction. The objective in this phase was to update all levee elevations with current data and incorporate hypothetical levels of vegetation to analyze the sensitivity in the simulated water surfaces to these roughness adjustments. A final objective was to produce inundation maps for selected flows through the MCS and Atchafalaya River using the updated model.

### 1.3 Approach

The hydraulic model investigation was conducted using the SW2D AdH-v4.6 numerical code to solve the depth-averaged, shallow water equations (ERDC USACE 2017). The list of tasks that utilize the AdH model to address the objectives of this study are the following:

1. Examine the current existing model for necessary modifications.
2. Run the 2011 flood event simulation for verification of the model with updated levee elevations and any newly acquired bathymetry.
3. Conduct simulations of alternative conditions to evaluate hydrodynamics throughout the system.
4. Develop flood inundation maps.

## **2 Model Details**

### **2.1 Numerical code**

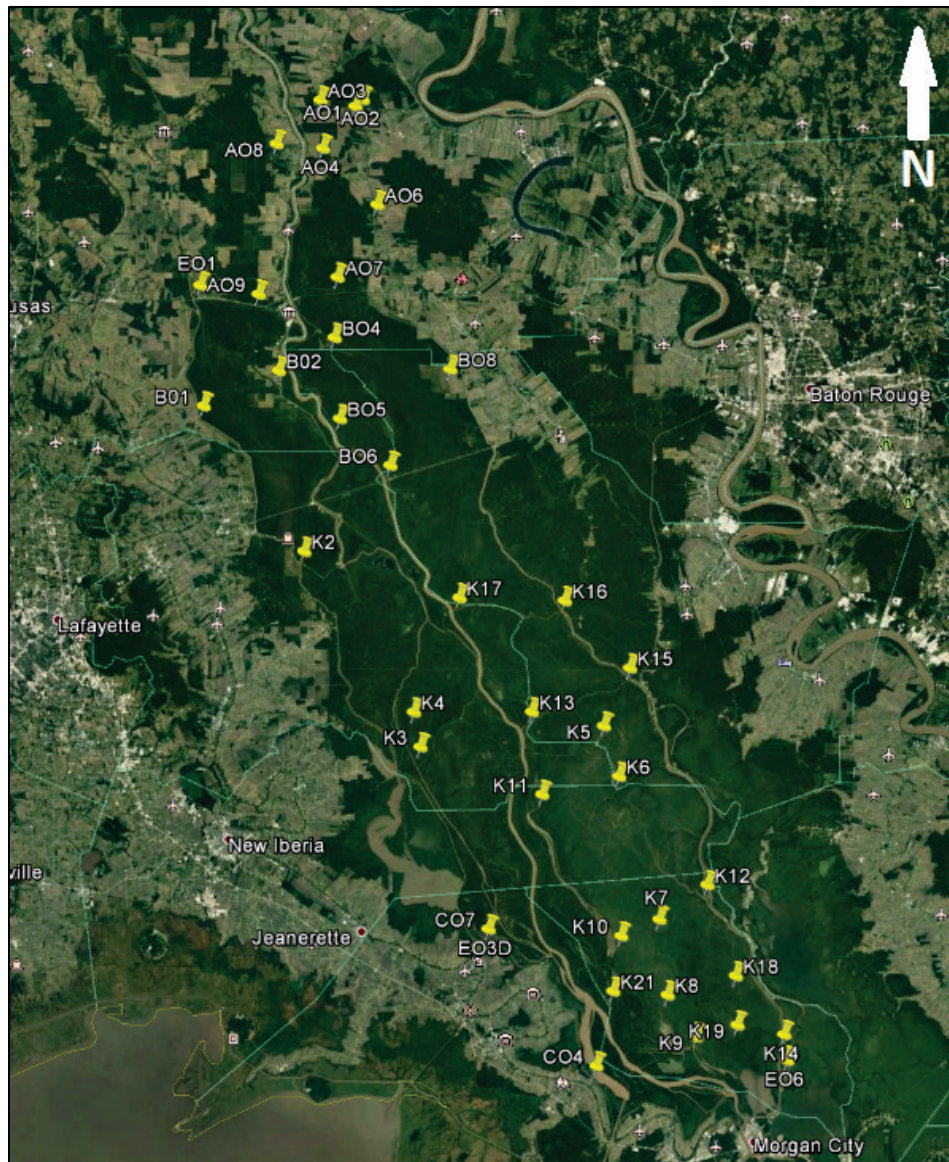
AdH is a multi-dimensional, finite element code capable of automatically refining the unstructured computational mesh and time-steps, when necessary, to resolve gradients in the flow field. It is the numerical model code applied for the simulations in this study. It is a finite element code that is capable of simulating three-dimensional (3D) Navier Stokes equations, 2D and 3D shallow water equations, and groundwater equations. It can be used in a serial or multiprocessor mode on personal computers and high-performance computing systems. AdH will refine the domain mesh in areas where more resolution is needed at certain times due to changes in the flow conditions and then remove the added resolution when it is no longer needed to minimize computational burden. The code also includes automatic time-step adaption, as needed. For spatial resolution, the model simulations in this report were set to a second level of adaption, meaning that each element in the mesh had the capability of splitting twice throughout a simulation. The ability of AdH to allow the domain to wet and dry within the floodplain areas as the hydrograph changes is suitable for shallow floodplain environments and varying flow conditions. For this study, the 2D shallow water hydraulic module of AdH was applied for all simulations. The code solves for water depth and depth-averaged velocity throughout the model domain. More details of the 2D shallow water module of AdH and its computational philosophy and equations are available in Savant et al. (2014). Note that there were significant efforts made to run sensitivity analyses in regard to the mesh resolution as well as time-step size in the Phase 2 study (Bell et al. 2018). The findings of these analyses gave every indication that the model was converging properly.

### **2.2 Interior gage data and boundary conditions**

Similar to Phases 1 and 2 (Bell et al. 2017, 2018), the model was validated after making the changes described later in section 3.1 of this report by comparing the water surface elevations (WSE) to the measured data during the 2011 flood event. US Geological Survey (USGS) pressure transducer gages were deployed throughout the basin just prior to and during the 2011 opening of the MCS and recorded time histories of rising and falling water levels at the locations where they were installed. These data were then

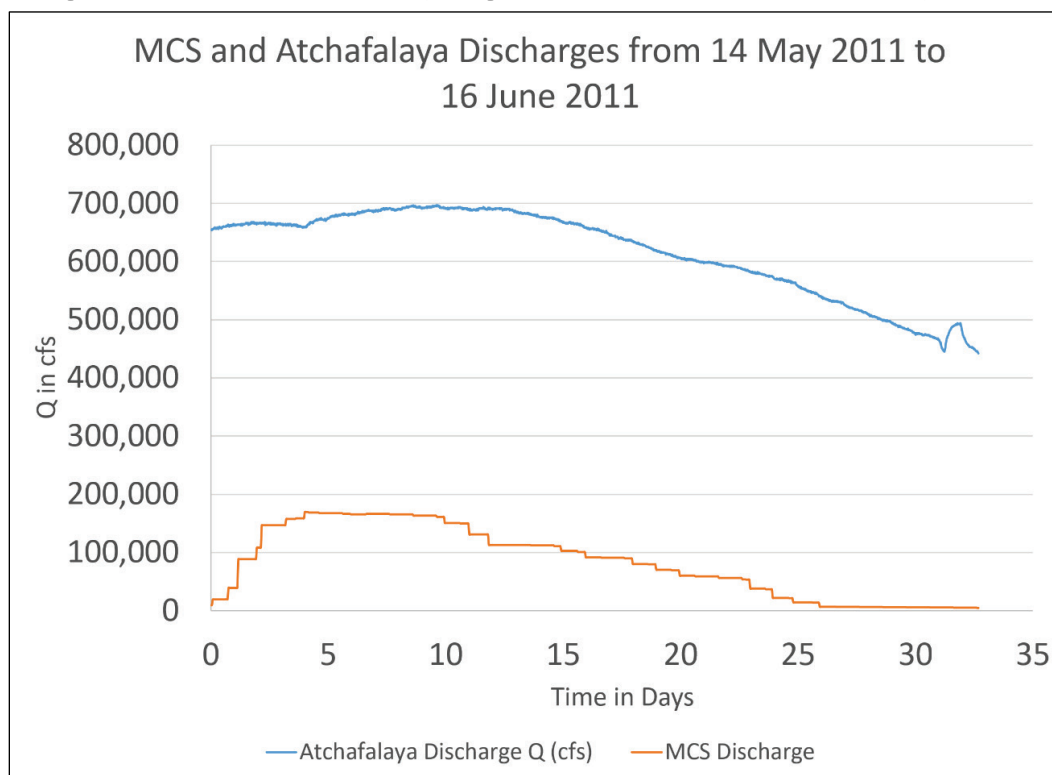
compared to model output at the same locations to assess how the model performed. Each gage was surveyed to NAVD88 Geoid 09. The gages were Hobo water level loggers (0 – 30 ft) corrected for air pressure via additional pressure transducers that were set out of water in the surrounding area. The location of the water level loggers can be seen in Figure 3. The “K” and “CO” sensors were deployed in water. The “B” sensors were deployed on dry ground and later became inundated. The “AO” sensors are the same ones used in the Phase 1 study. The order the gages were placed in the figures/tables of the gages is not specific in any way.

Figure 3. USGS gage locations.



For the inflow boundary conditions, the values provided in Table 10 “Revised discharge for 2011 flood based on physical model discharge rating” from Maynard (2014) were used for the flow through the MCS. The peak flow through the MCS for the 2011 flood was approximately 170,000 cfs. The flow through the Atchafalaya River at the time of peak MCS discharge was 664,000 cfs (maximum flow through the Atchafalaya River throughout the entire simulation was 697,000 cfs). Figure 4 displays the hydrographs for the MCS and the Atchafalaya River. Inflow values for the Atchafalaya River were obtained from the USGS gage located at Simmesport, LA. The tailwater boundary conditions at Calumet, LA, and Morgan City, LA, used USGS gages that collected data through the 2011 flood event. The model simulation begins on 14 May 2011 at 1600 (opening of the MCS) and runs to 16 June 2011 at 0800.

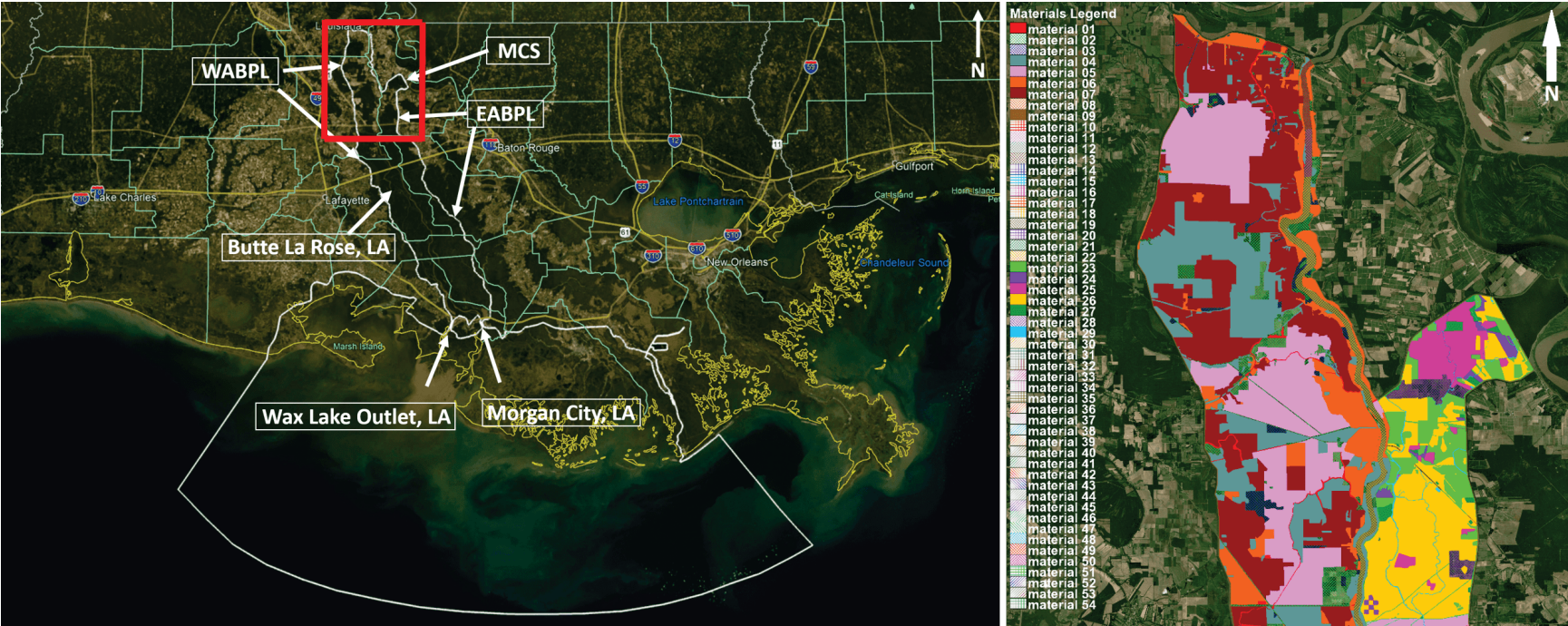
**Figure 4. 2011 flood event hydrograph for the MCS and Atchafalaya River.**



## 2.3 Existing model description

The model used for this study contains bathymetry/topography that was established with multibeam and airborne LiDAR data. The horizontal datum for the model is the State Plane Coordinate System (Louisiana South, North American Datum 83), US Survey Feet. The vertical datum is referenced to NAVD88, US Survey Feet. The mesh is comprised of 1,758,219 nodes and 3,503,688 elements that encompass an area of approximately 393,999,558,766 sq ft (14,131 sq mi). A total of 48 different material types were assigned to the model. Figure 5 displays the model boundaries on the left of the image; the right side depicts a zoomed-in subset of the model to show an example of the elaborate material specification. Differing surface attributes (channels, trees, shrubs, open land, etc.) were denoted as separate material types in the mesh and in the boundary condition input file. Each material type is assigned attributes of surface roughness, eddy viscosity, and adaption. In this model, all material types were assigned a roughness based on the Manning's Roughness Coefficient ( $n$ ) except where Un-submerged Rigid Vegetation (URV) cards were used for forested areas to more accurately define friction effects of larger vegetation such as shrubs and trees. For areas that use the URV cards, the parameters are characterized as average roughness height, average stem diameter, and average stem density (in that order) in the boundary condition file. Table 1 displays the range of values of Manning's  $n$  (or URV card) used for all simulations. Note that "FR URV" stands for *un-submerged rigid* vegetation and is used to simulate the flow through trees and other vegetated areas throughout the domain. For areas that do not contain a material type description, the associated footprint in the model was small or considered to have a minimal impact to the model results compared to the rest of the model.

Figure 5. Assigned model materials (subset image on the right outlined by red box in the overview image on the left).



**Table 1. List of simulated hydraulic roughness parameters.**

<b>Material Type</b>	<b>Value</b>
Water	0.0162
Open land	0.028
Shrubs to Small Trees	0.02, 0.7, 0.1
Select Cut Timber	0.02, 0.9, 0.1
Large Timber	0.02, 0.7, 0.1
Open Land/Vegetation	0.03
Open Land with Different Crops	0.032
Berm	0.03
Levee	0.026
Atchafalaya River (Lower)	0.02
East Levee Channel	0.0162
West Channel	0.0162
Channel Near Gage K17	0.0162
Channel near Gage K13	0.0162
Side Channel east of Levee Channel	0.0162
Channel to K9 and K19	0.0162
Channel from K19 to Lake	0.0162
Channel to K18	0.0162
Upper Basin West Channel	0.0162
Channels South of K9	0.0162
Atchafalaya River (Middle)	0.027
Atchafalaya River (Upper)	0.03
Phase 1 Open Land	0.036
Phase 1 Shrubs to Small Trees	0.066
Phase 1 Select Cut Timber	0.02, 1.7, 0.1
Phase 1 Large Timber	0.02, 2.05, 0.1
Phase 1 Open Land/Vegetation	0.038
Phase 1 Open Land with Different Crops	0.046
Phase 1 Water	0.034
Marsh Area at Ocean Boundary	0.04
Custom	0.03
Custom	0.04

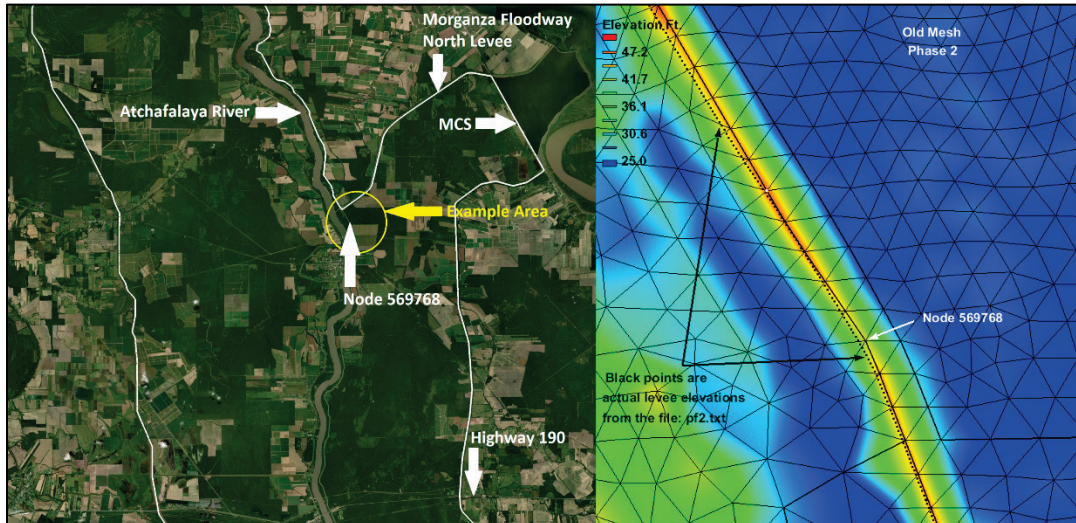
<b>Material Type</b>	<b>Value</b>
Custom	0.03
Custom	0.03
Custom	0.04
Birds Foot Paths	0.03
Custom	0.03
Custom	0.03
Custom	0.03
Custom	0.03
Custom	0.03
Custom	0.03
Deep Ocean	0.03
Custom	0.03
Custom	0.03
Custom	0.03
Custom	0.03
Custom	0.03
Open Land/Vegetation	0.03

## 3 Results

### 3.1 Task 1: Existing model modifications

The Phase 3 model used for this effort is a modified version of the Phase 2 model. In analyzing the surveyed levee elevations provided by the Mississippi Valley Division (MVD), it was discovered that the Phase 2 model did not capture the crest elevation of the levees in every location. The model grid elevations were assigned by LiDAR data that were interpolated linearly to the AdH model. The resolution of the grid defined how elevations were assigned to individual nodes throughout the grid. The interior levees in the numerical model need a flat top if flow is passing over from one side to the other. The existing model from Phase 2 has many locations where only a single row of nodes defines the levee crest (Figure 6). Areas that display this condition can cause mass conservation issues in the model. Note that the black dots in Figure 6 represent the levee survey of updated levee elevations.

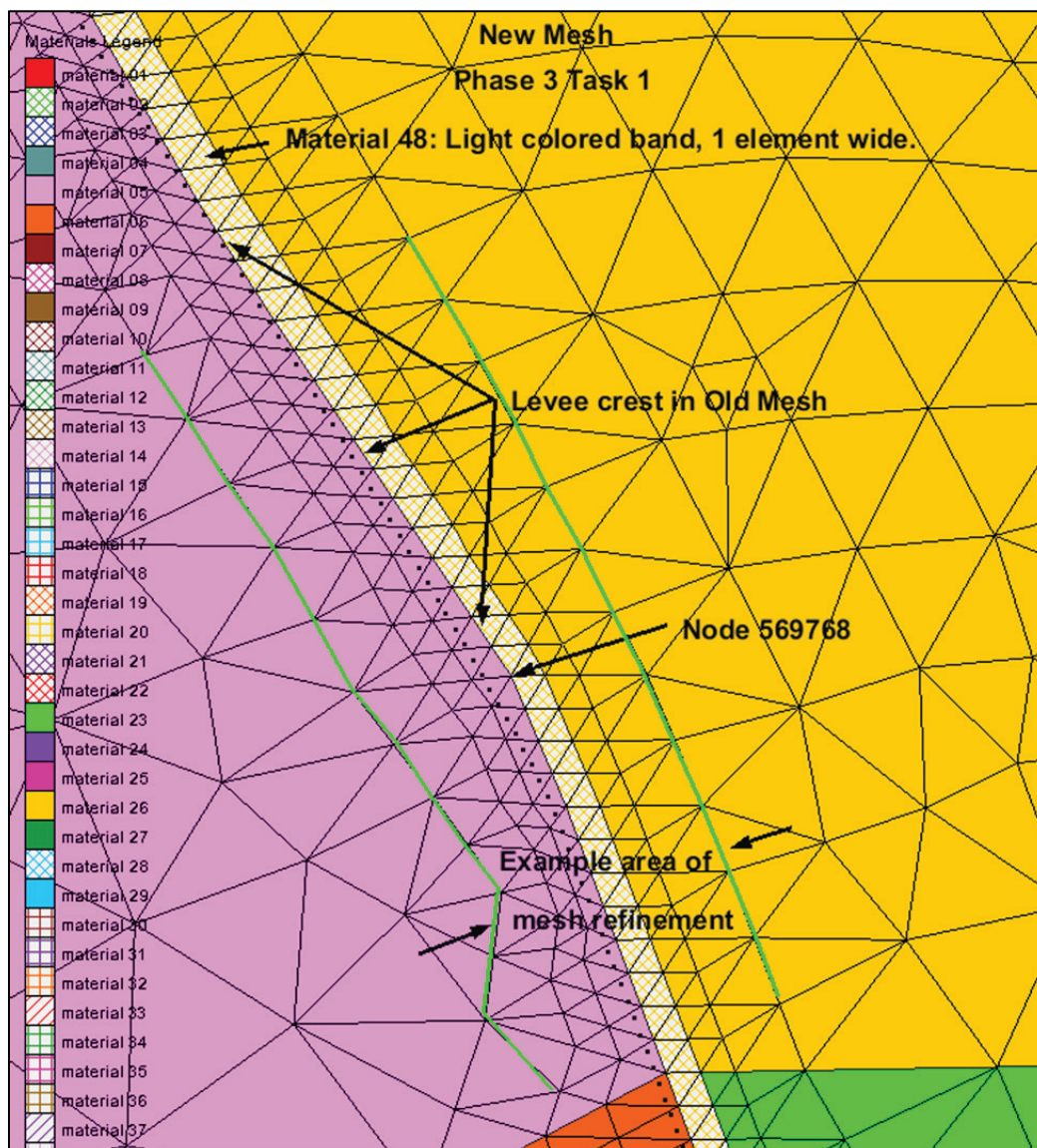
Figure 6. Phase 2 model example levee.



To address this problem, a flat surface assigned to the levee crest can help to prevent the mass conservation issue and can be modeled by making two rows of nodes with the same elevations as the top of the levee. Thus, the levee cross section is essentially turned into a trapezoid with a very short (compared to the base of the trapezoid) but flat top. To change the single row of nodes into two, several mesh refinement steps were required. The area of refinement in the example location is between the green lines as indicated in Figure 7. Node 569768 is shown only as a reference point

between the different Figures and in the discussion. The elements created between the two rows of nodes that were selected as the levee top were assigned material type 48, as shown in the figure. The new levee elevation data points are shown as the black dots and do not necessarily fall within the new levee-top (material 48) elements. This is not critical as a program was written to identify the nodes associated with material 48, three nodes per element. Then, the elevation of the nearest new levee data point was interpolated to each node in the element by a distance weighted scheme.

Figure 7. Area of refinement in the example area.



This effort provided two rows of nodes with nearly identical elevations as one moves forward or backward along the crest of the levee, and the crest elevations are essentially those of the survey data. The result was a flat levee crest with the corrected elevations, as shown in Figure 8.

Figure 8. Modified levee results.

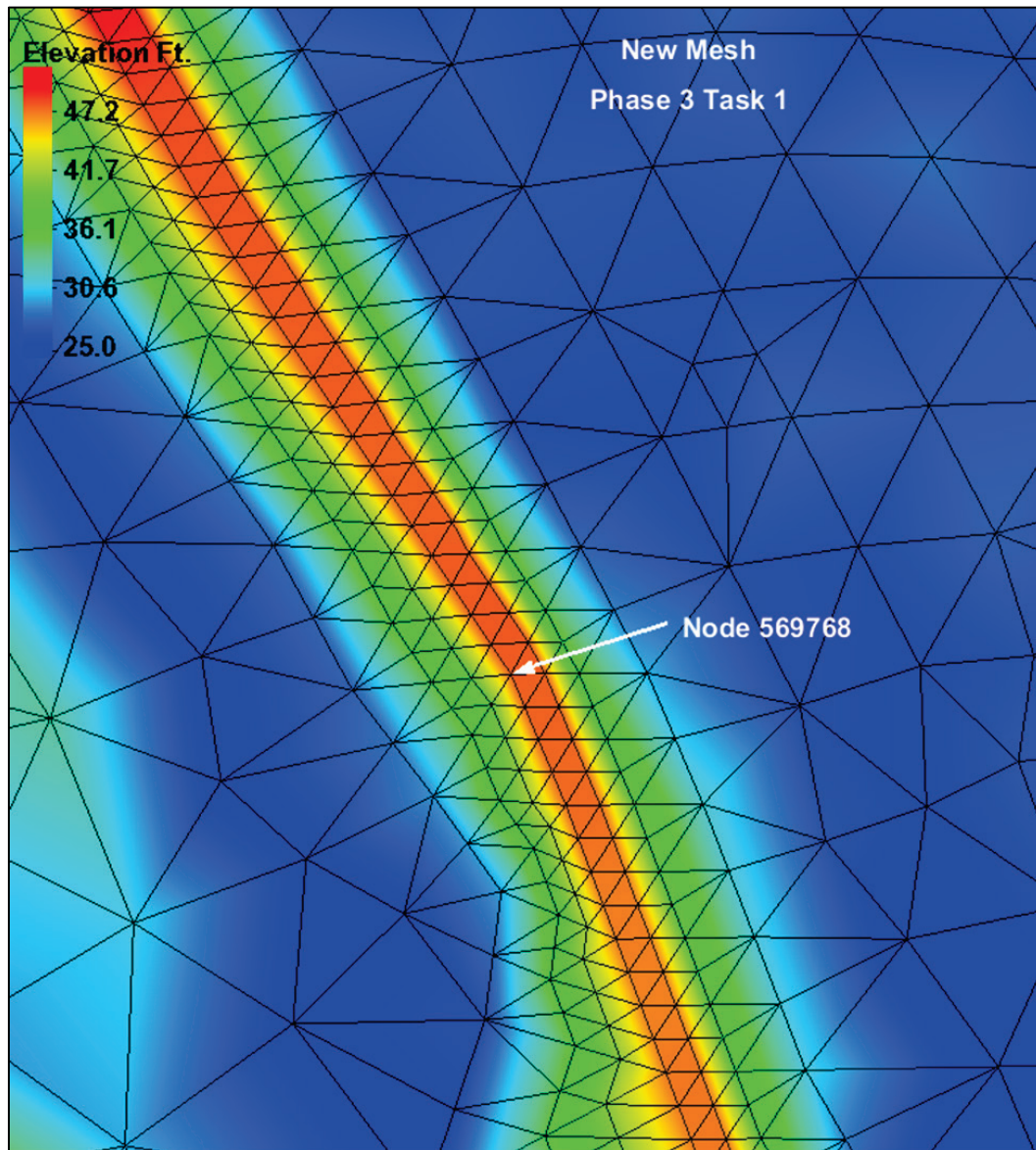
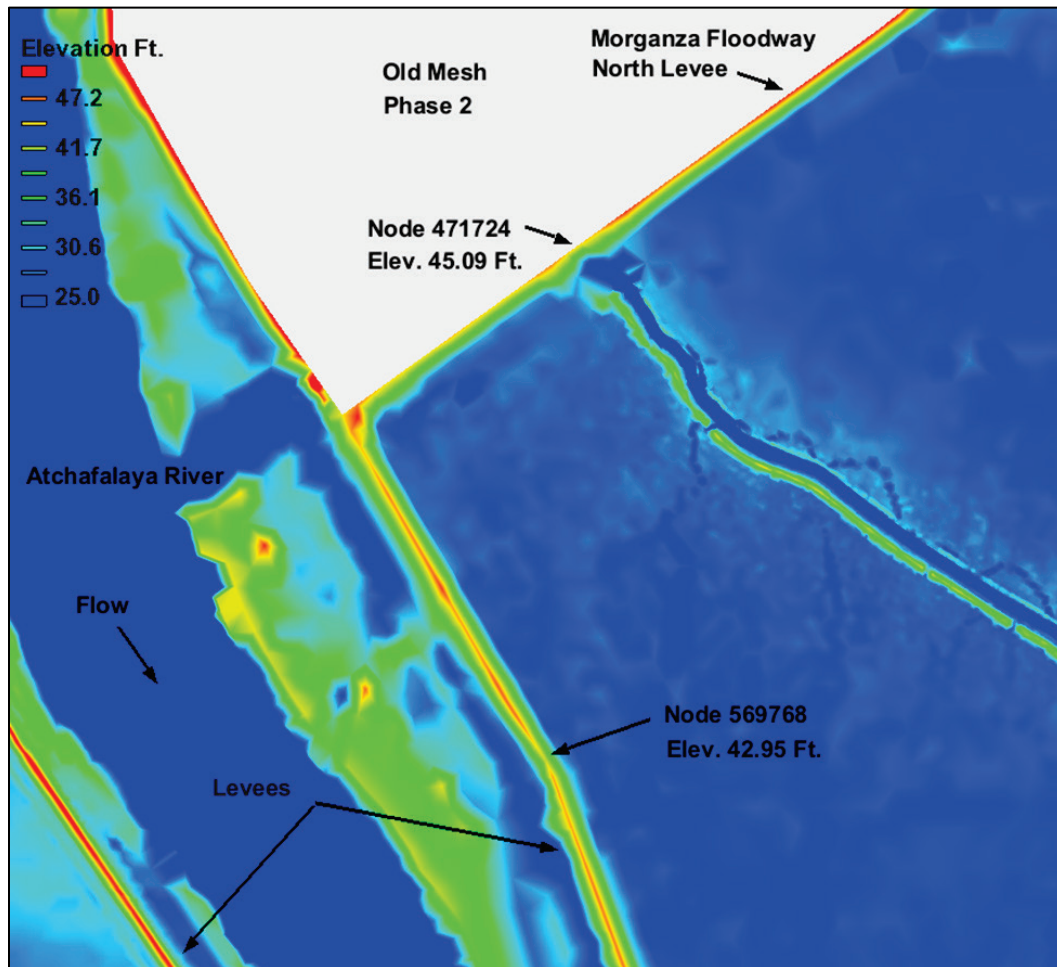


Figure 9 shows a wider view of the example area. It is taken from the old mesh. A second node, number 471724, was also selected for comparison, being on an exterior levee, and thus an exterior boundary of the model. Since the boundary is treated as an infinite vertical wall (even though this exterior levee has an elevation assigned to it), there is no need for a flat levee top. This is the same for all external boundaries throughout the

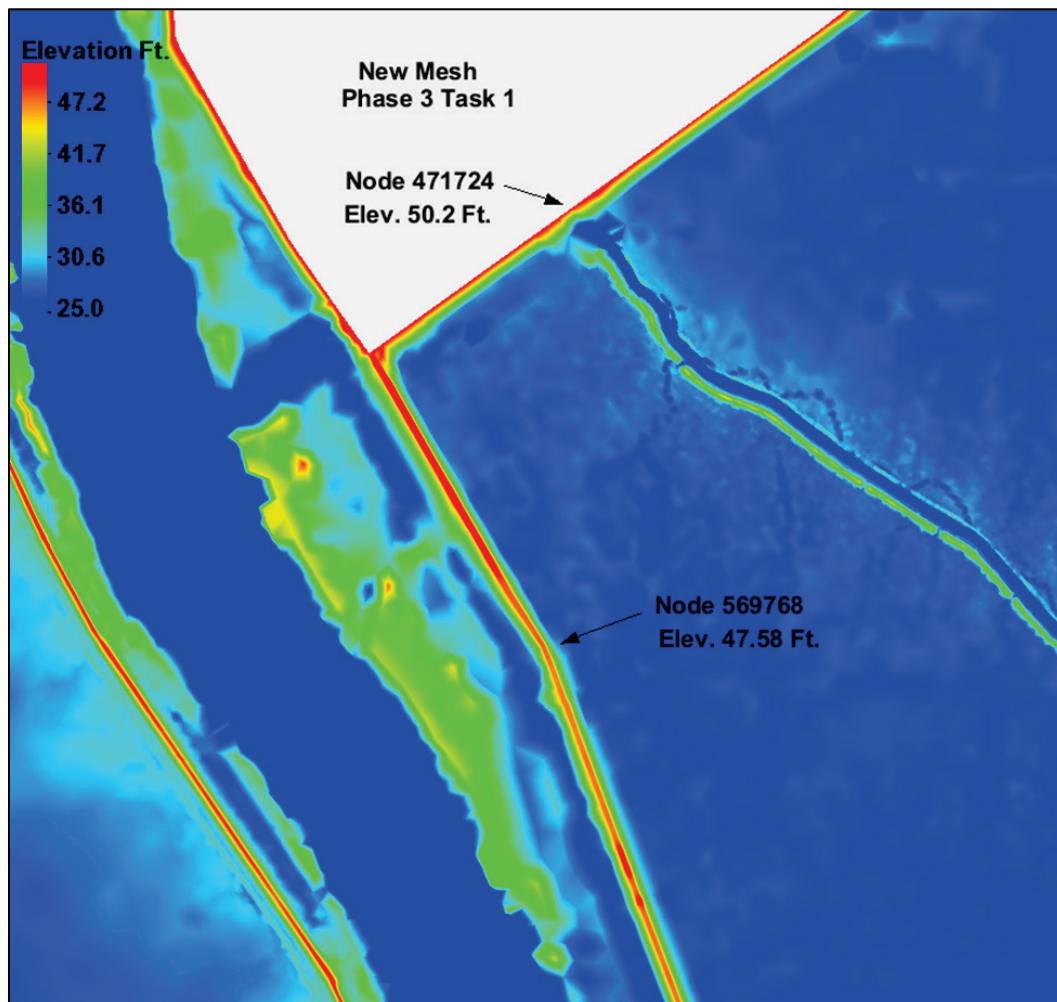
model domain. Nevertheless, it is still necessary to assign proper elevations to the boundary to determine when overtopping might occur. In the figure it is clear that there are many low (yellow) points along this boundary levee, and also along the interior levee containing node 569768.

Figure 9. Levee elevation correction.



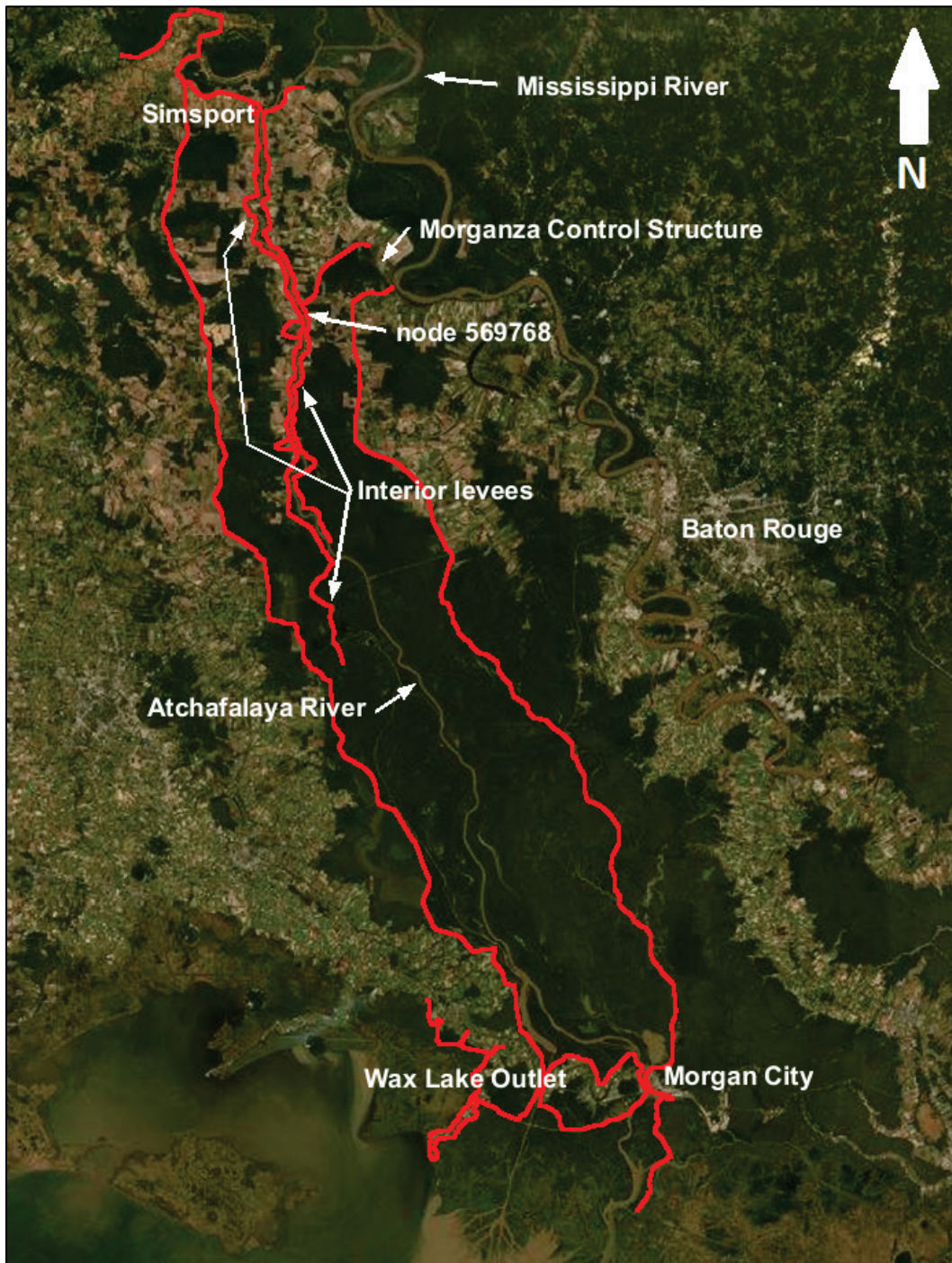
For node 569768, on the interior levee, the difference between the Phase 2 and Phase 3 models is 4.63 ft. For node 471724, on the exterior levee, the difference between the two models is 5.11 ft. Figure 10 shows the improved and updated levees.

Figure 10. Levee elevation correction.



The examples given for the levee elevation updates show only a tiny portion of the model. The work was extensive and included all the new levee data. To put it in perspective, Figure 11 shows all the new levee data in red points. The approximate location of node 569768 in the example area is noted. It is only a very small portion of the entire model domain. Thus, the size of the modeled area and sheer number of nodes, elements, and new levee data points necessitated an automated procedure. Automatic mesh refinement and custom utility codes were used whenever possible. The results produced a more accurate model for the Phase 3 modeling effort, to include the possibility of overtopping the interior levees in scenarios having floodwaters spilling laterally over the interior levees from one portion of the basin to the other. This updated version of the model will be used for the efforts in this report.

Figure 11. Model modification overview.



### 3.2 Task 2: 2011 flood event simulation

To validate the model, it was simulated with the 2011 flood event and compared to the results from the measured data as well as the Phase 2 model data. The flow in the Atchafalaya River was the same as it was during the 2011 event, and the tailwater boundary remained as the constant ocean boundary. In Figure 12 – Figure 18, the measured data are shown in blue, Phase 2 model results in red, and Phase 3 model results in yellow for the USGS gage measurement comparisons. Note the agreeable results between the values produced by the models compared to the measured values. Also note that the adjustments made in section 3.1 of this report have had minimal or no effect on the model's results compared to the Phase 2 model. The remaining gage comparisons can be found in Appendix A.

Figure 12. Gage A01 model results.

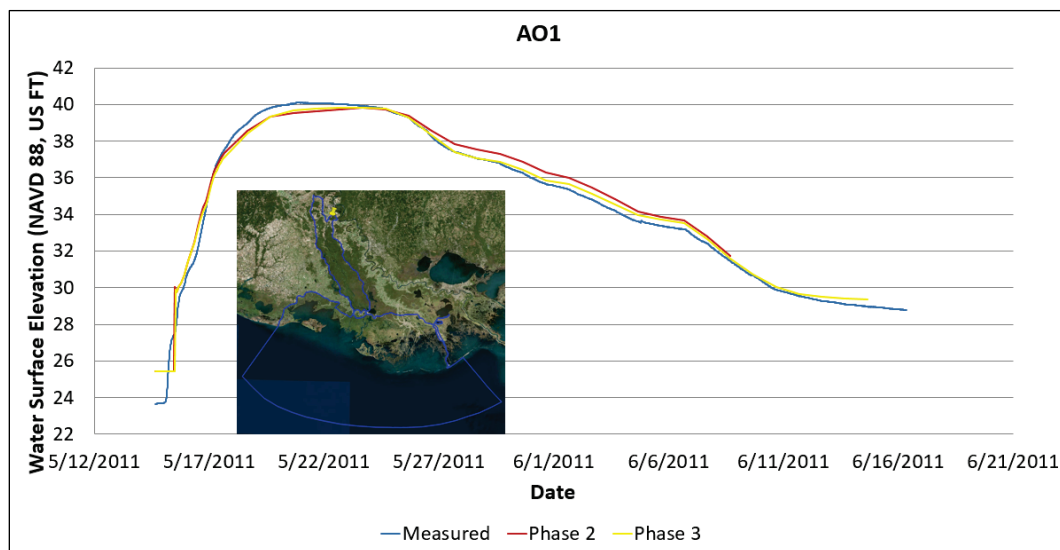


Figure 13. Gage K3 model results.

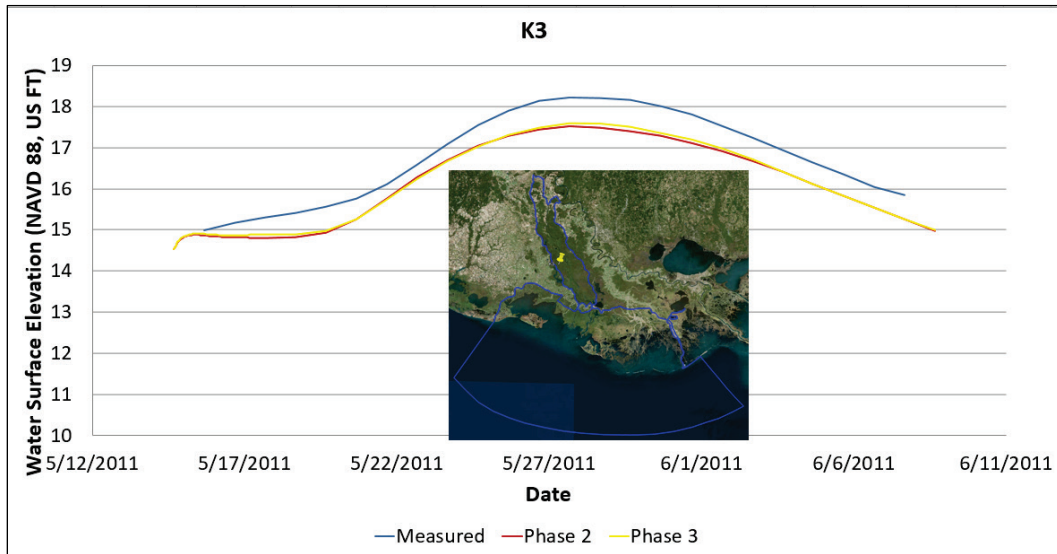


Figure 14. Gage K13 model results.

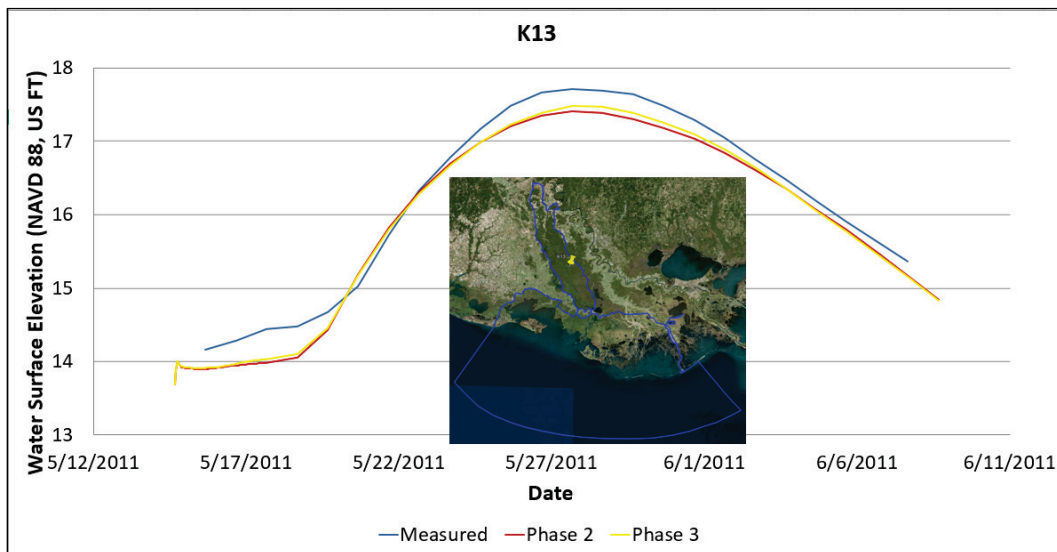


Figure 15. WSE at Calumet in Wax Lake Outlet Channel.

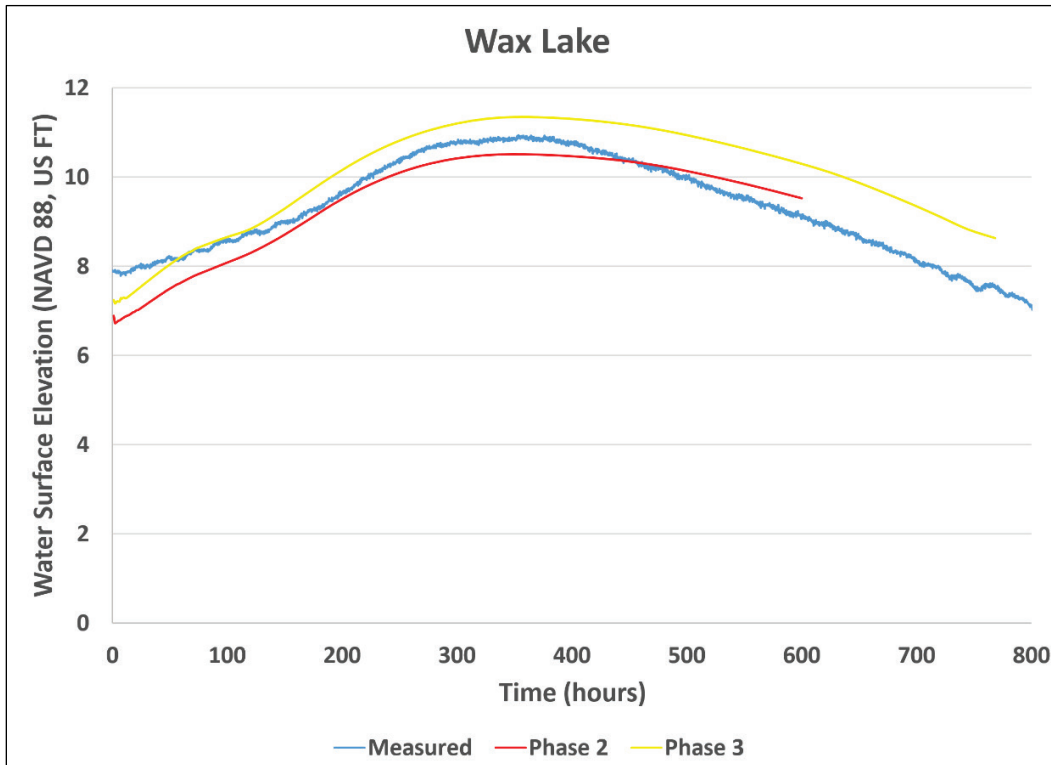


Figure 16. Discharge at Calumet in Wax Lake Outlet Channel.

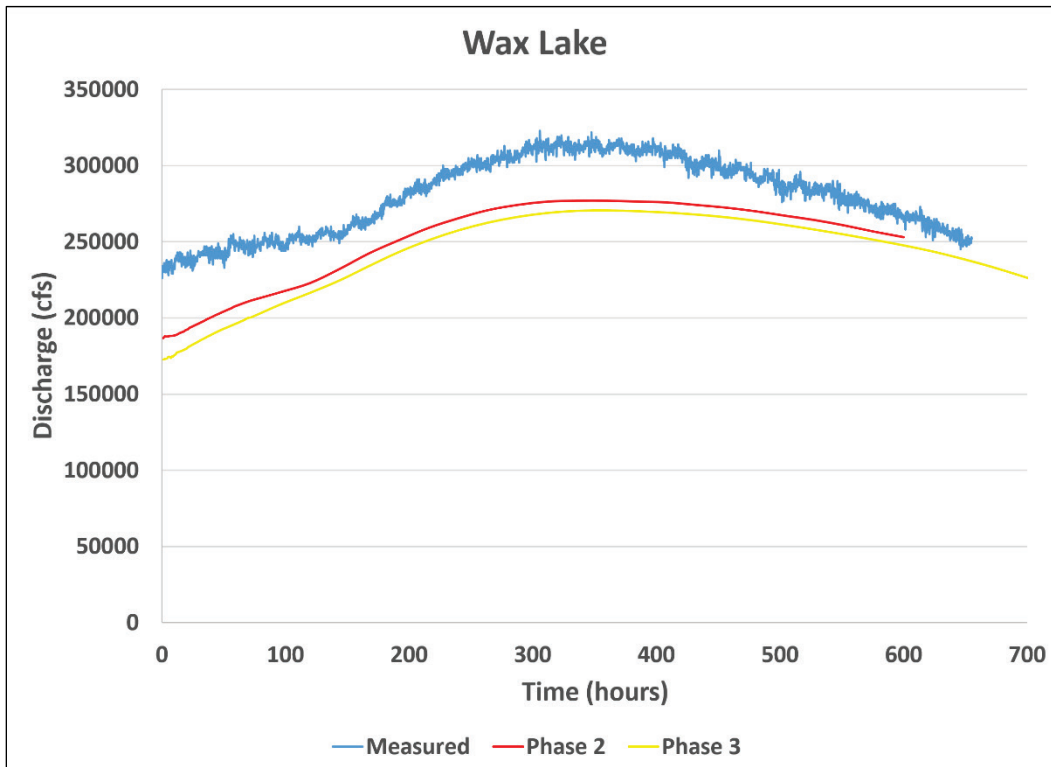


Figure 17. WSE at Morgan City.

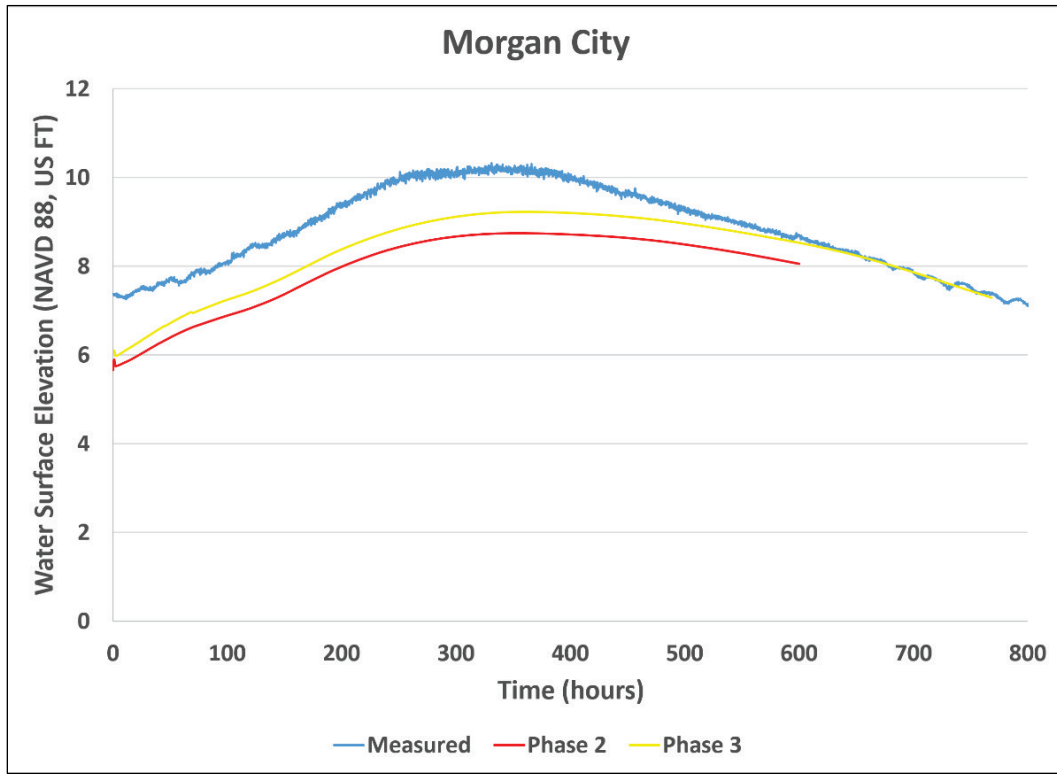
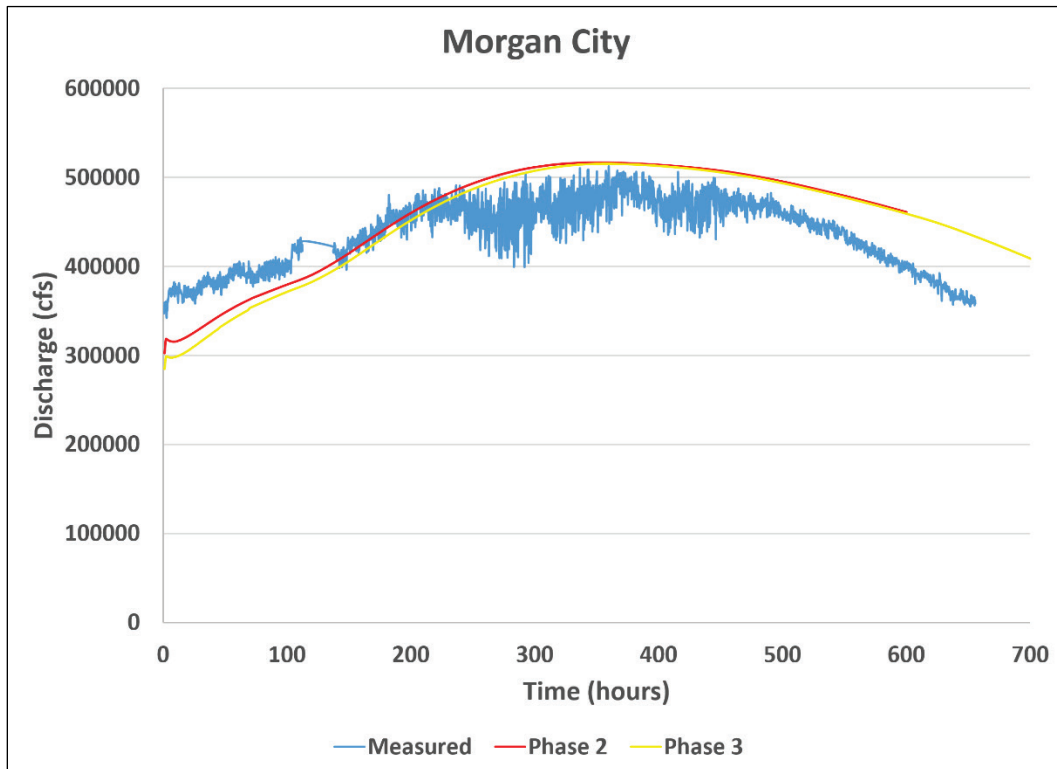


Figure 18. Discharge at Morgan City.



Model performance metrics were performed on the Phase 2 and Phase 3 models and can be seen in Table 2. The models were analyzed for correlation, Willmott, and Nash-Sutcliffe efficiency (NSE). Note that the values of NSE at gage K2 are discussed in Appendix A.

**Table 2. Model performance metrics for the Phase 2 and Phase 3 models.**

Gage	Model	Correlation	Willmott	NSE
A01	Phase 2	0.99	0.99	0.96
	Phase 3	0.99	0.99	0.96
A02	Phase 2	1.00	0.99	0.98
	Phase 3	1.00	0.99	0.98
A03	Phase 2	1.00	0.95	0.86
	Phase 3	0.99	0.95	0.86
A04	Phase 2	0.98	0.98	0.93
	Phase 3	0.98	0.98	0.93
A06	Phase 2	0.99	0.96	0.89
	Phase 3	0.99	0.96	0.89
A07	Phase 2	0.98	0.99	0.95
	Phase 3	0.98	0.98	0.93
K5	Phase 2	0.99	0.97	0.92
	Phase 3	0.99	0.97	0.92
K7	Phase 2	0.99	0.97	0.90
	Phase 3	0.99	0.97	0.90
K4	Phase 2	0.99	0.92	0.68
	Phase 3	0.99	0.92	0.71
K2	Phase 2	0.94	0.50	-6.24
	Phase 3	0.95	0.51	-6.12
B04	Phase 2	0.95	0.83	0.17
	Phase 3	0.90	0.76	-0.71
C04	Phase 2	0.99	0.96	0.85
	Phase 3	1.00	0.95	0.84
K9	Phase 2	0.99	0.77	0.07
	Phase 3	0.99	0.76	0.03
K14	Phase 2	0.99	0.97	0.90

Gage	Model	Correlation	Willmott	NSE
	Phase 3	0.99	0.97	0.90
K13	Phase 2	0.99	0.99	0.96
	Phase 3	1.00	0.99	0.97
K17	Phase 2	0.99	0.75	-0.46
	Phase 3	0.99	0.76	-0.46
K16	Phase 2	1.00	0.85	0.36
	Phase 3	0.99	0.85	0.36
K15	Phase 2	1.00	0.98	0.94
	Phase 3	1.00	0.98	0.94
K3	Phase 2	0.99	0.93	0.73
	Phase 3	1.00	0.94	0.76
K18	Phase 2	0.99	0.96	0.88
	Phase 3	0.99	0.96	0.87
K19	Phase 2	0.99	0.97	0.91
	Phase 3	0.99	0.97	0.91
K21	Phase 2	0.99	0.96	0.88
	Phase 3	0.99	0.96	0.88
K10	Phase 2	0.99	0.96	0.87
	Phase 3	0.99	0.96	0.87
K11	Phase 2	0.99	0.95	0.86
	Phase 3	0.99	0.95	0.86
K12	Phase 2	0.99	0.97	0.90
	Phase 3	0.99	0.97	0.90
K6	Phase 2	0.99	0.97	0.92
	Phase 3	0.99	0.97	0.92
K8	Phase 2	0.99	0.93	0.77
	Phase 3	0.99	0.92	0.76

### 3.3 Task 3: PDF vegetation adjustment simulations

As found in Bell et al. (2018), the Morganza Floodway in its current condition overtops its boundary levees before reaching the PDF discharge of 600,000 cfs through the MCS. The “Base” model featured in the following alternative simulations is the Phase 3 model resulting from the modifications described in sections 3.1 and 3.2. The “AdjustVeg” simulations stand for each of the model runs that simulated some amount of hypothetical adjustment to the vegetation roughness. Vegetation was adjusted in the model by changing the desired material types of vegetated areas to the “open land” material type, which has a much smaller  $n$ -value. A smaller  $n$ -value results in reduced flow resistance.

To analyze the sensitivity in the simulated WSEs throughout the Morganza Floodway, vegetation roughness was hypothetically adjusted. The primary flow path immediately downstream from the MCS can be seen in Figure 19. Three separate simulations were performed: AdjustVeg 1, AdjustVeg 2, and AdjustVeg 3 (Figure 20 – Figure 25). AdjustVeg 1 features changing the vegetation roughness to open land in the areas that are in the immediate path of the water exiting the MCS just south of the gates. In Figure 20 and Figure 21, the areas that are green in the picture on the right but not on the left are where the vegetation roughness was hypothetically adjusted to open land (circled in red). This was done by changing the element material type in the actual mesh, so the forested areas went from URV parameters to a Manning’s roughness value equal to that of “bare ground.” In Figure 22 and Figure 23, the AdjustVeg 2 simulation maintained the changes made in AdjustVeg 1 while hypothetically adjusting more vegetation roughnesses farther south and west of the areas in AdjustVeg 1 (circled in red). In Figure 24 and Figure 25, the AdjustVeg 3 simulation maintained the changes made in AdjustVeg 1 and AdjustVeg 2, and in addition, all areas from the entire Morganza Floodway down to Highway 190 were hypothetically switched to open land. This was done by changing the resistances values to that of bare ground. These areas are represented by the purple (24), mauve (25), and gold (26) material types shown in Figure 24.

Figure 19. Flow path graphic.

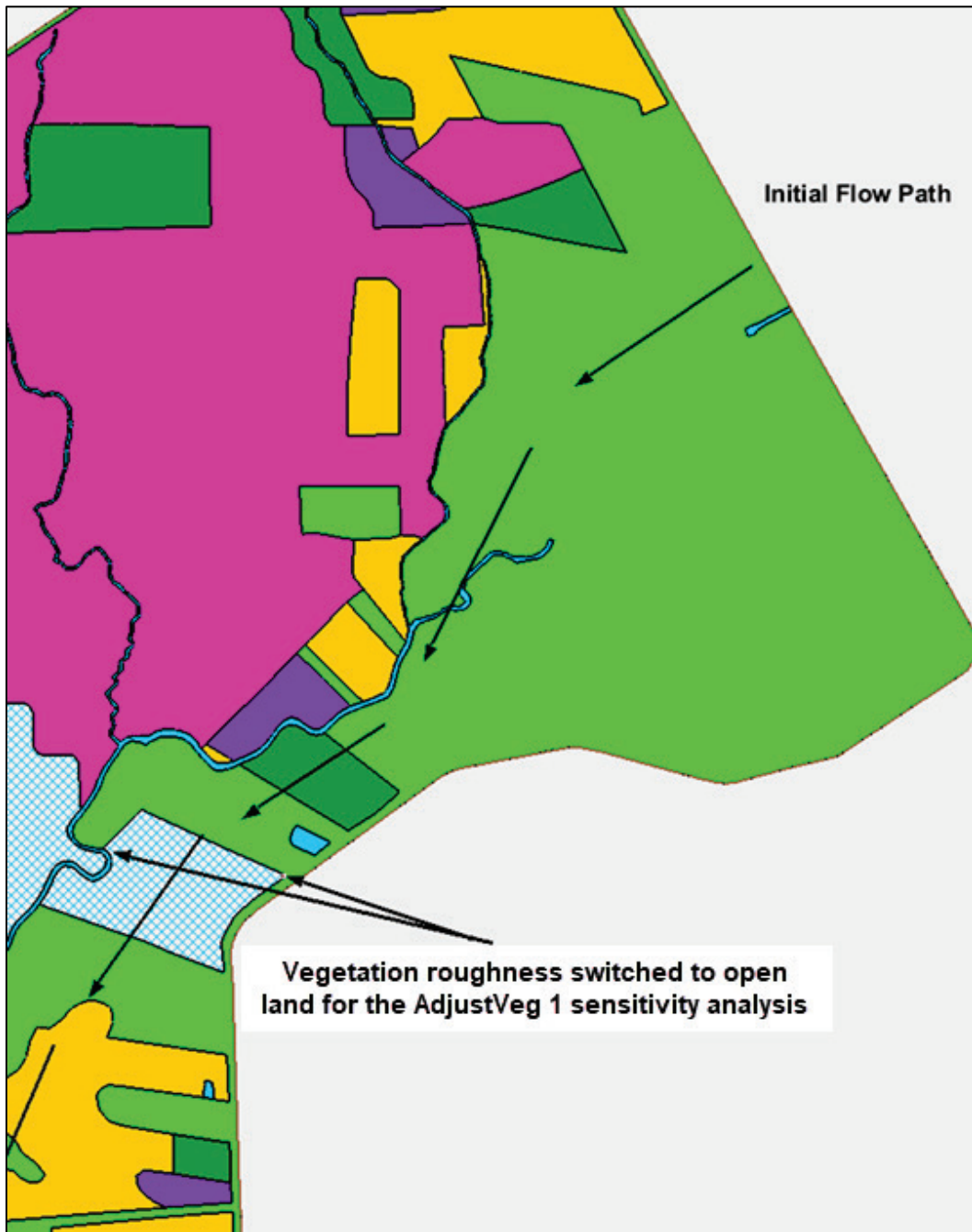


Figure 20. Base model (left) and AdjustVeg 1 model (right) material comparison.



Figure 21. AdjustVeg 1 hypothetical vegetation adjustments for sensitivity analysis.

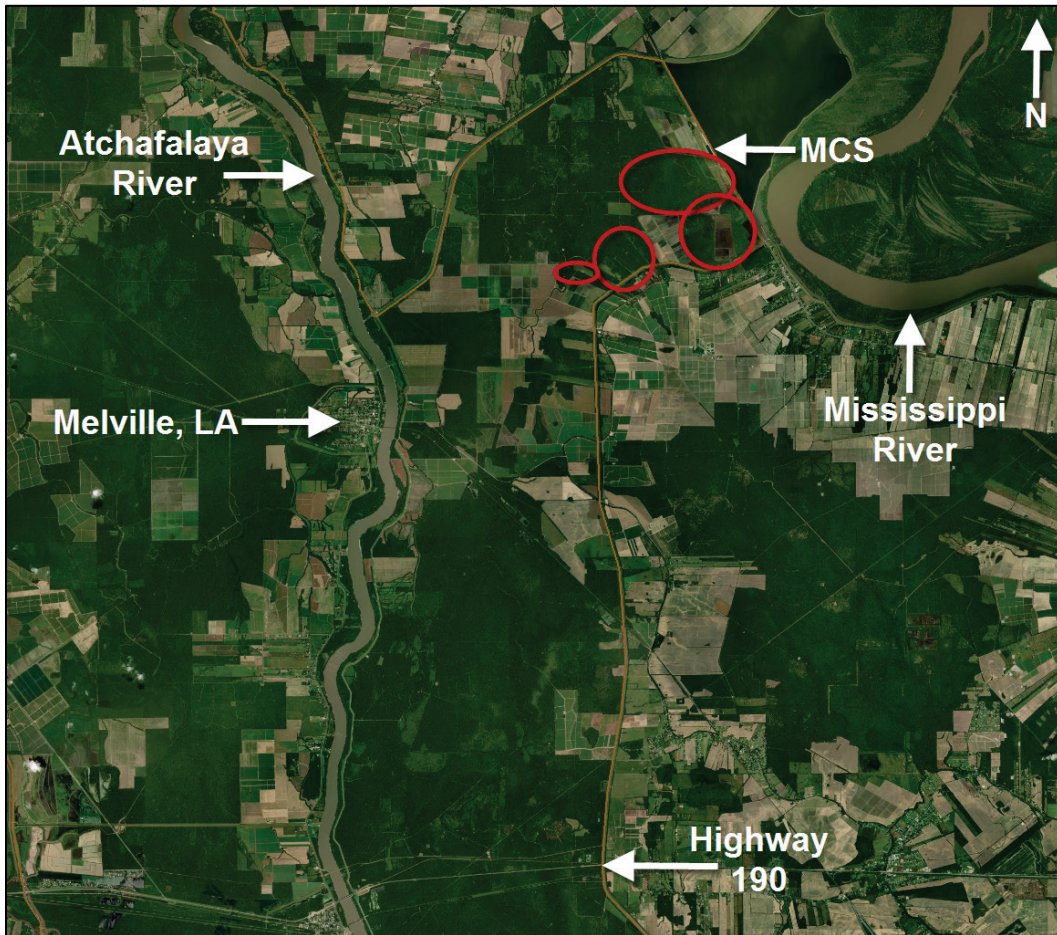


Figure 22. Base model (left) and AdjustVeg 2 model (right) material comparison.

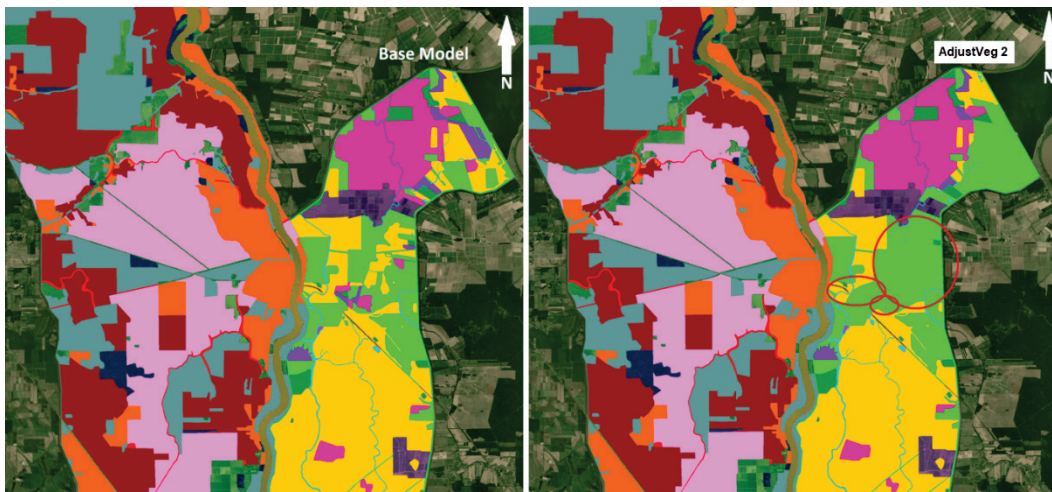


Figure 23. AdjustVeg 2 hypothetical vegetation adjustments for sensitivity analysis.

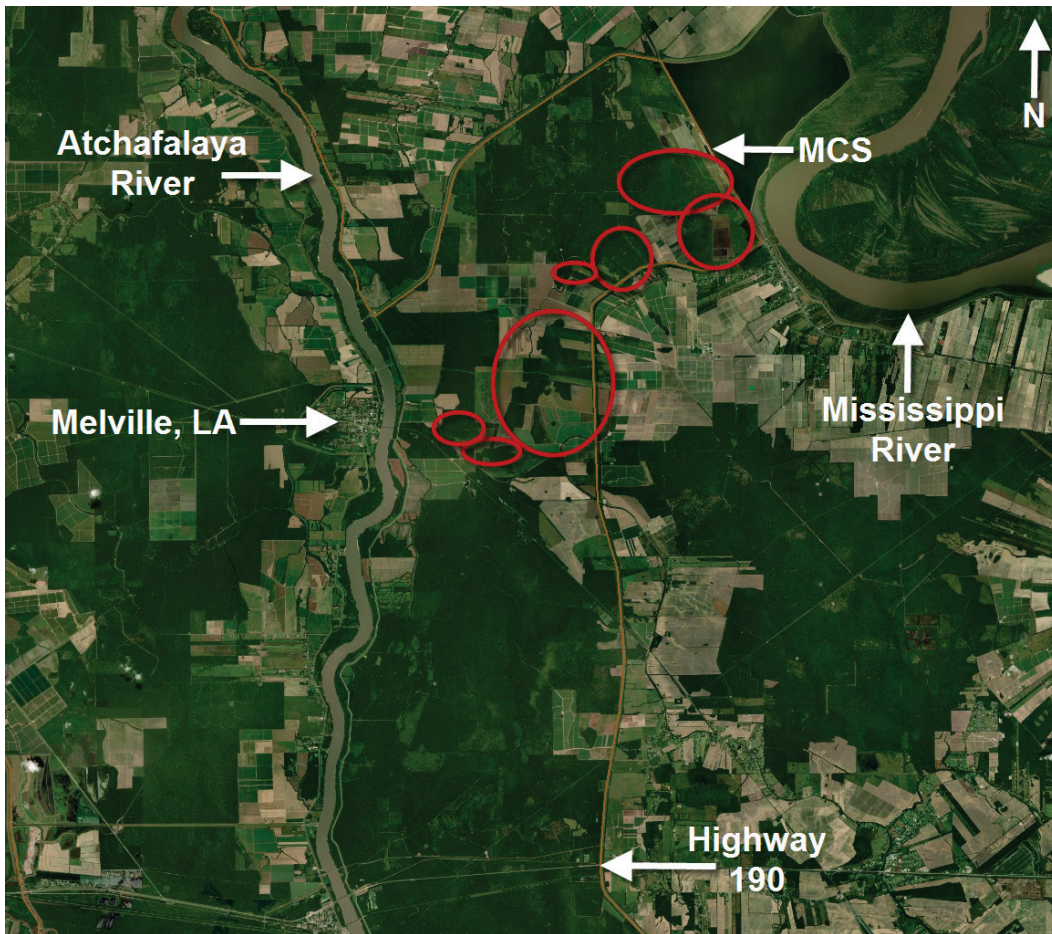


Figure 24. AdjustVeg 3 model description.

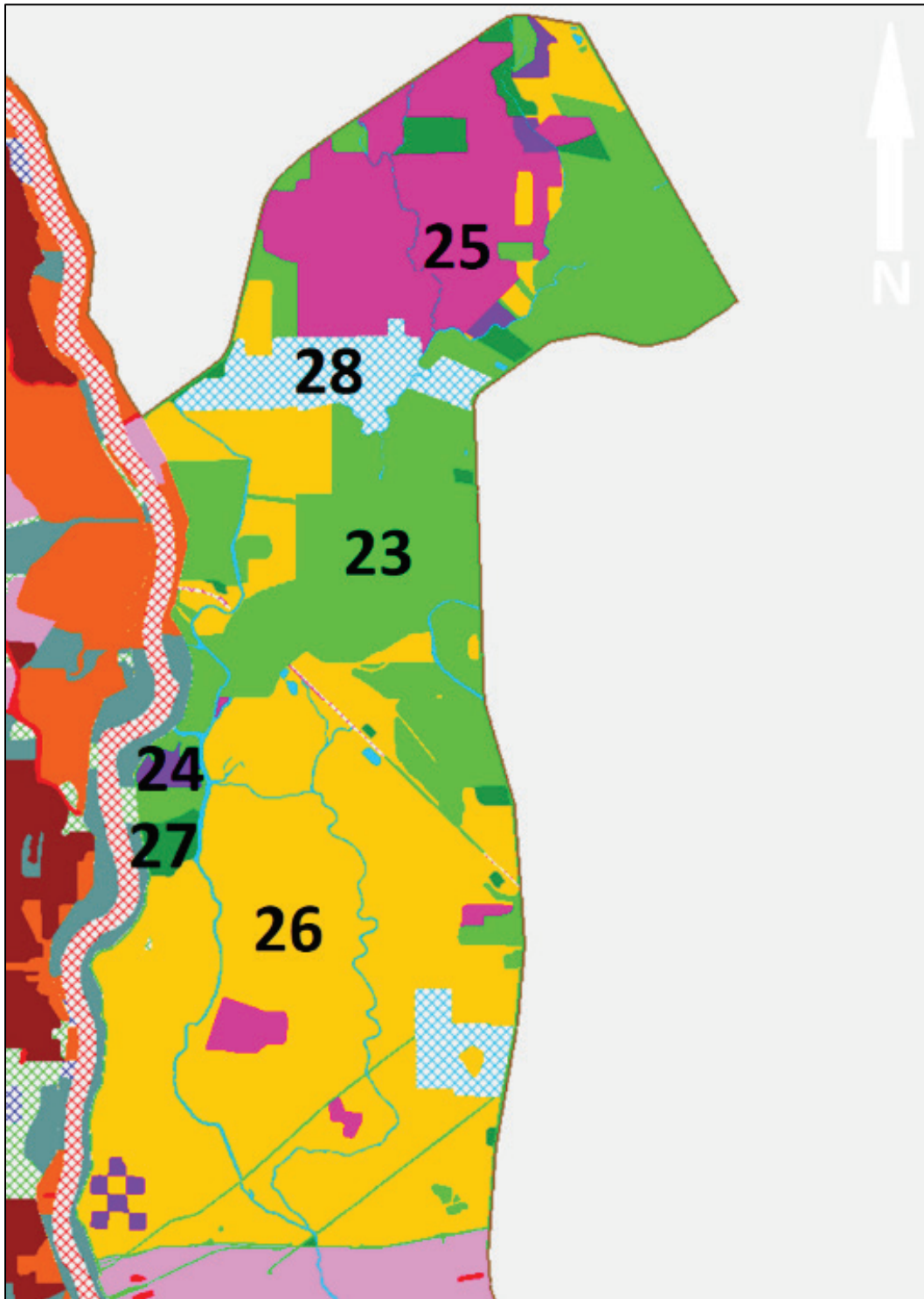
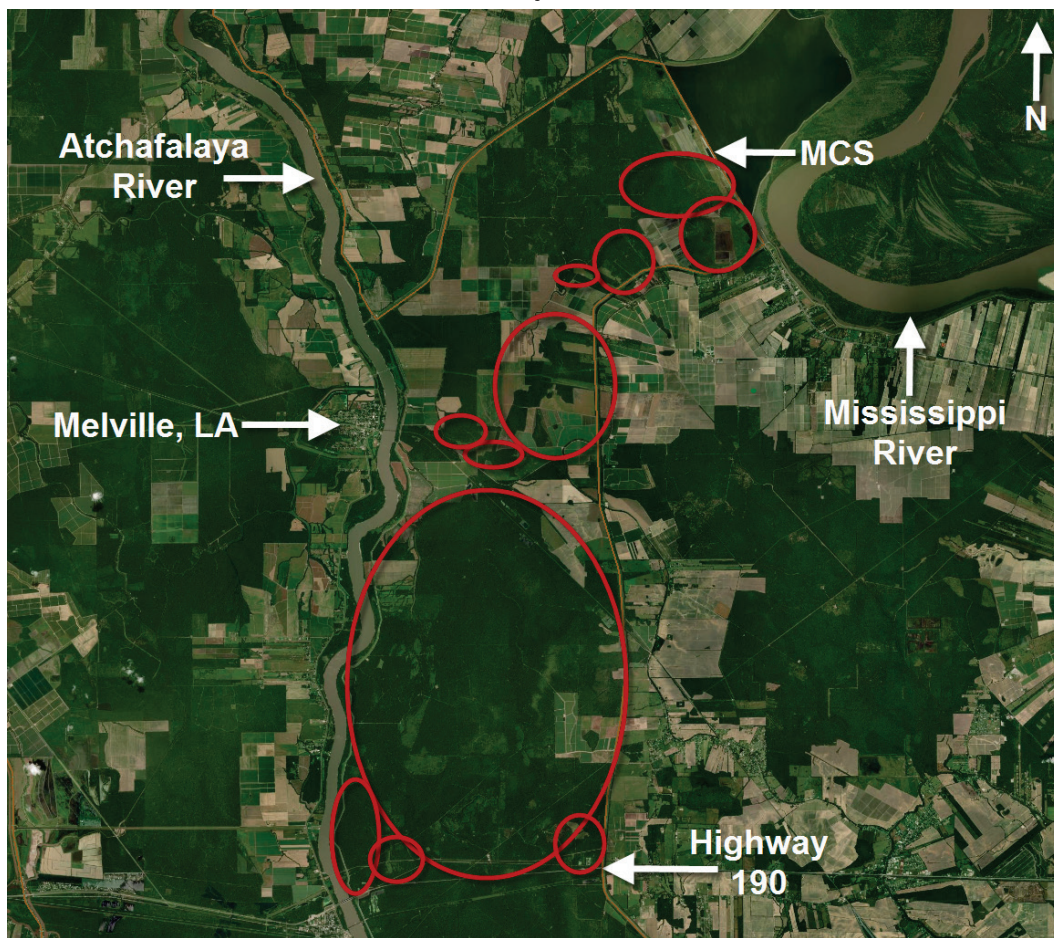
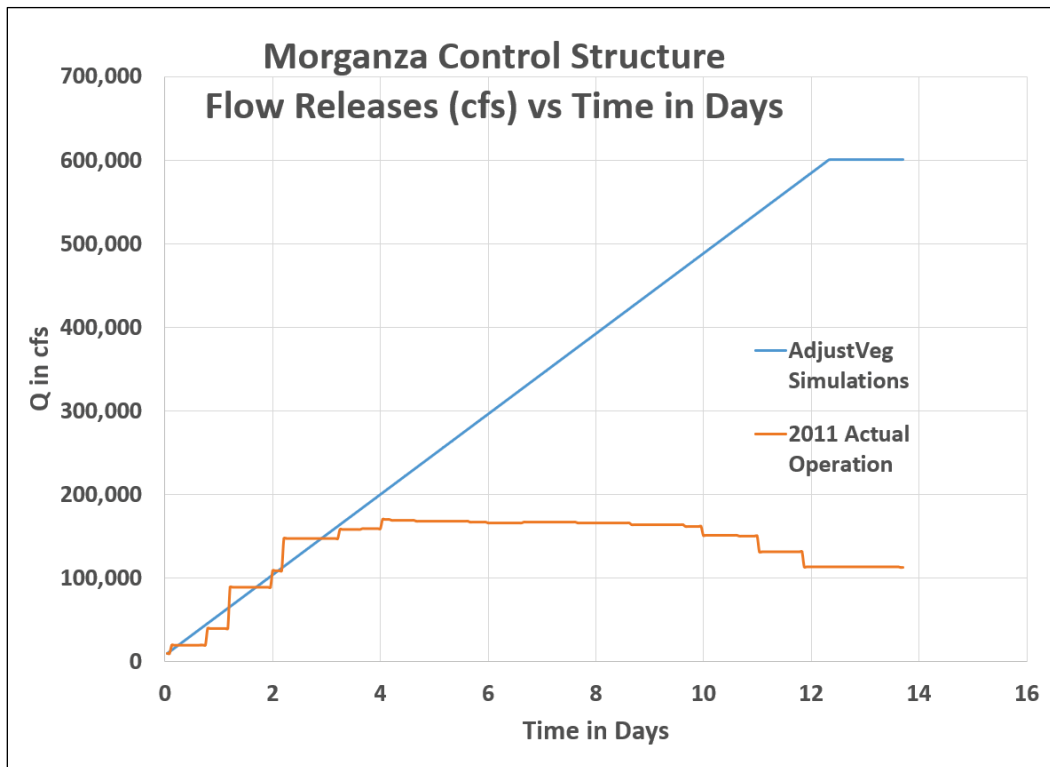


Figure 25. AdjustVeg 3 hypothetical vegetation adjustments for sensitivity analysis.



The boundary conditions used for these hypothetical vegetation adjustment simulations feature the MCS discharge increasing at a rate of 3,600 cfs per hour until 12 days, 8 hr into the simulation when the PDF of 600,000 cfs is reached and held constant for the duration of the run. The flow in the Atchafalaya River was kept identical to the 2011 event. When the river reaches the peak flow for that event (~ 694,000 cfs), it is kept steady at that flow for the duration of the simulation. The tailwater remains constant as the ocean boundary. The input flow rates for the MCS can be seen in Figure 26. Note the similarity in the slope of the rising limbs of the two hydrographs indicating realistic operation of the MCS for the sensitivity analysis simulations.

Figure 26. MCS flow rates for hypothetical vegetation adjustment simulations.



The maximum depth for each of the hypothetical vegetation adjustment simulations can be seen in Figure 27 – Figure 36. These figures show model results of the sensitivity in water depth compared to the base condition (top of each figure). The first pair of figures show the entire modeling domain; the second and third pair show zoomed-in plots of the Morganza Floodway and Morgan City/Calumet in Wax Lake Outlet Channel, respectively. The maximum depth plots display the highest water depth at each node throughout the mesh during any point in time throughout the simulation; they do not show variation of water depth with respect to time. In viewing these plots, it is important to remember that nothing in the plot indicates *when* the water reached that maximum depth. AdjustVeg 1 and 2 simulations ran approximately 8 days after reaching and maintaining the peak flow of 600,000 cfs through the MCS. AdjustVeg 3 simulation ran approximately 18 days after reaching and maintaining the peak flow. The sensitivity analysis demonstrates that for the lowest roughness simulations, the water depth decreases throughout the Morganza Floodway and in fact throughout the entire Atchafalaya Basin, indicating a lower risk of levee overtopping.

Figure 27. Base (top) and AdjustVeg 1 (bottom) maximum depth for the entire model domain.

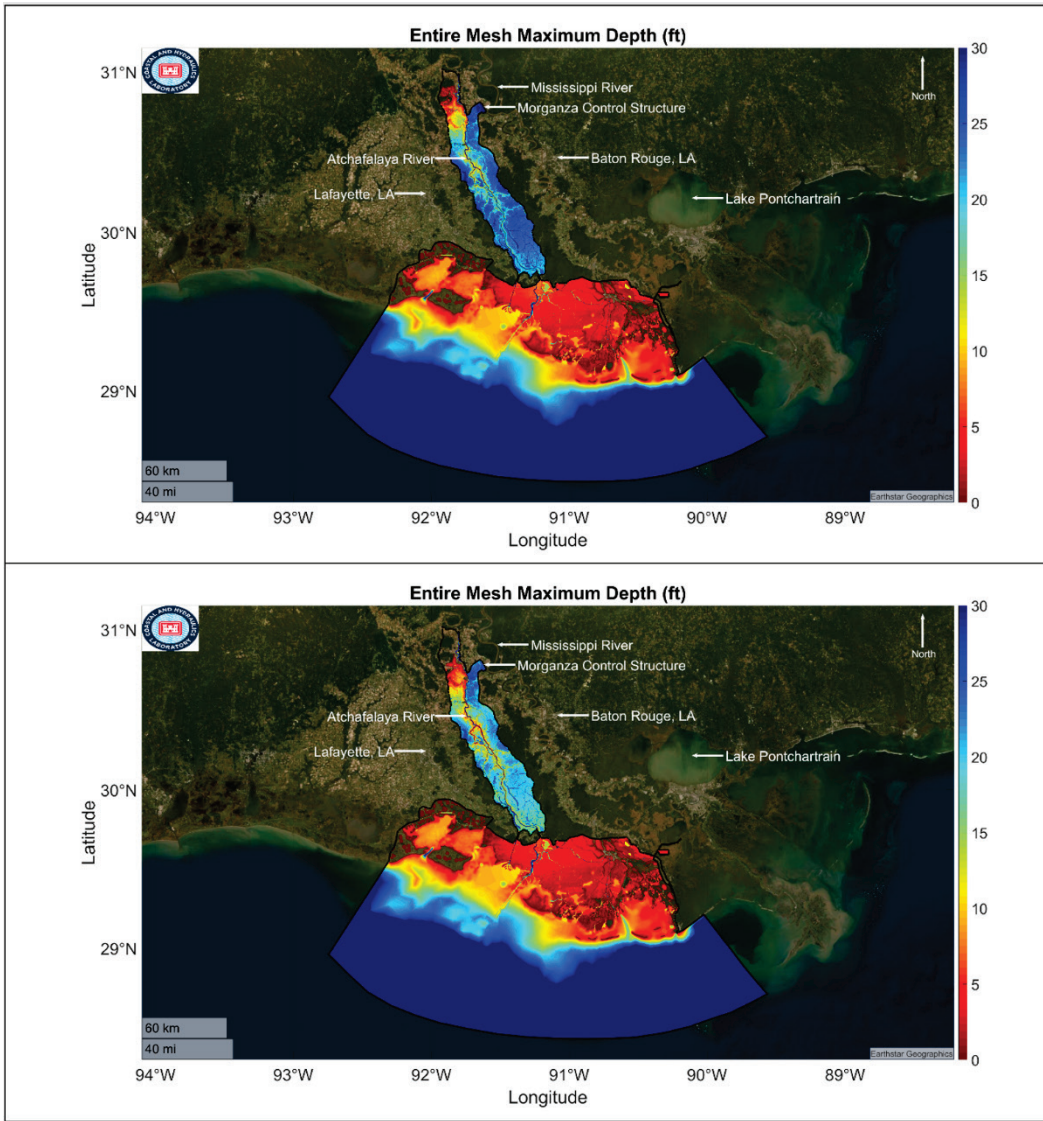


Figure 28. Base (top) and AdjustVeg 1 (bottom) maximum depth for the Morganza Floodway.

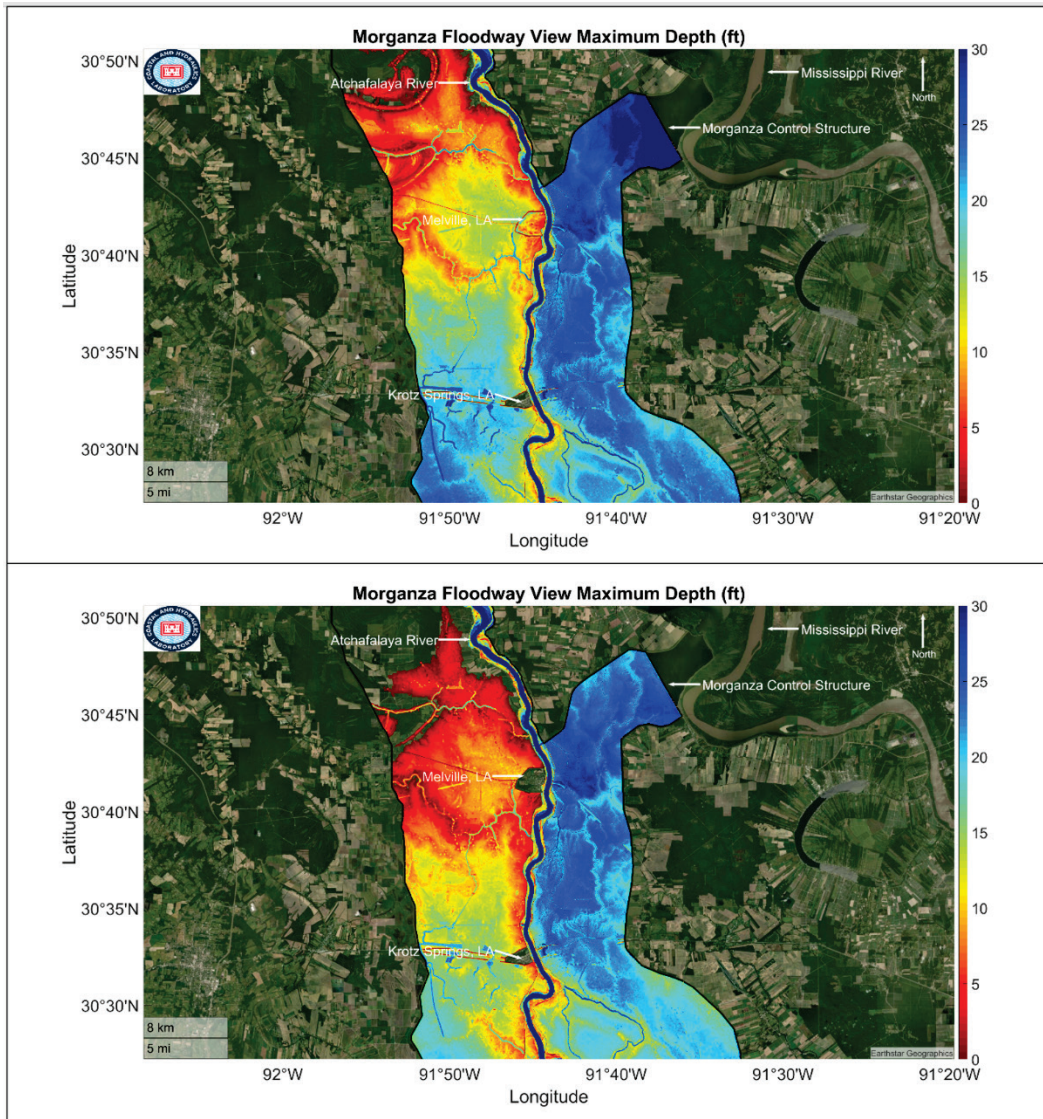


Figure 29. Base (top) and AdjustVeg 1 (bottom) maximum depth for the Calumet in Wax Lake Outlet Channel.

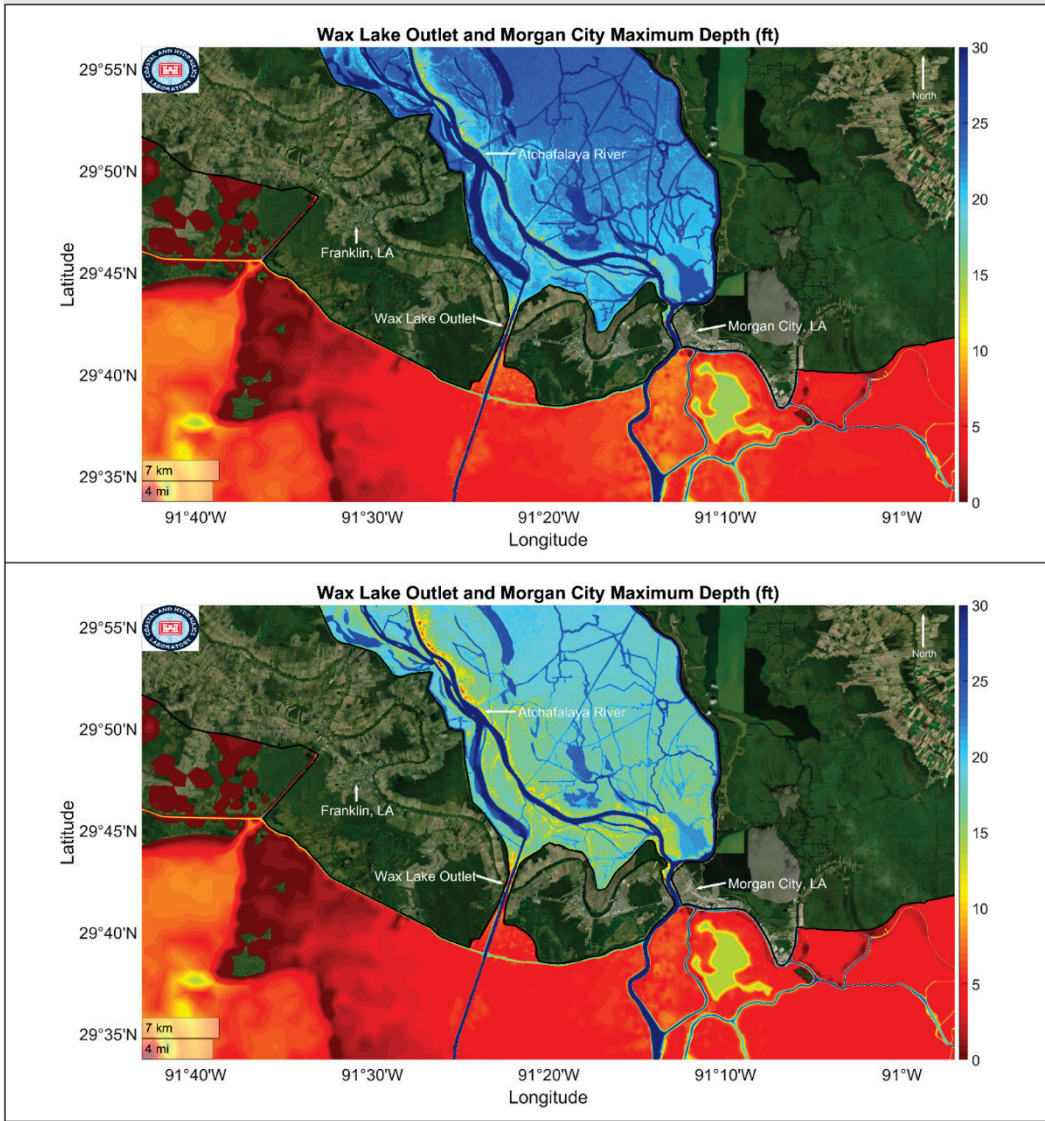


Figure 30. Base (top) and AdjustVeg 2 (bottom) maximum depth for the entire model domain.

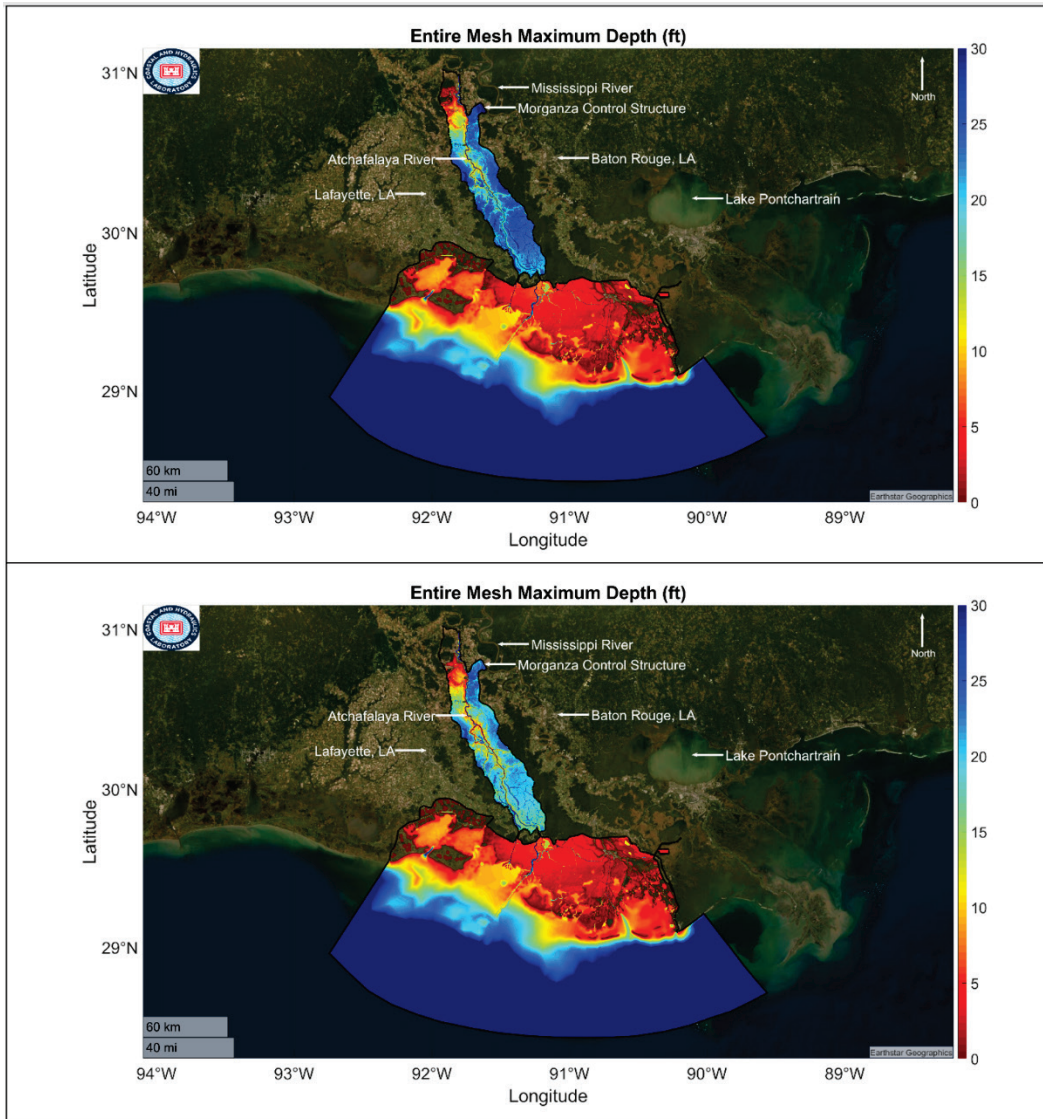


Figure 31. Base (top) and AdjustVeg 2 (bottom) maximum depth for the Morganza Floodway.

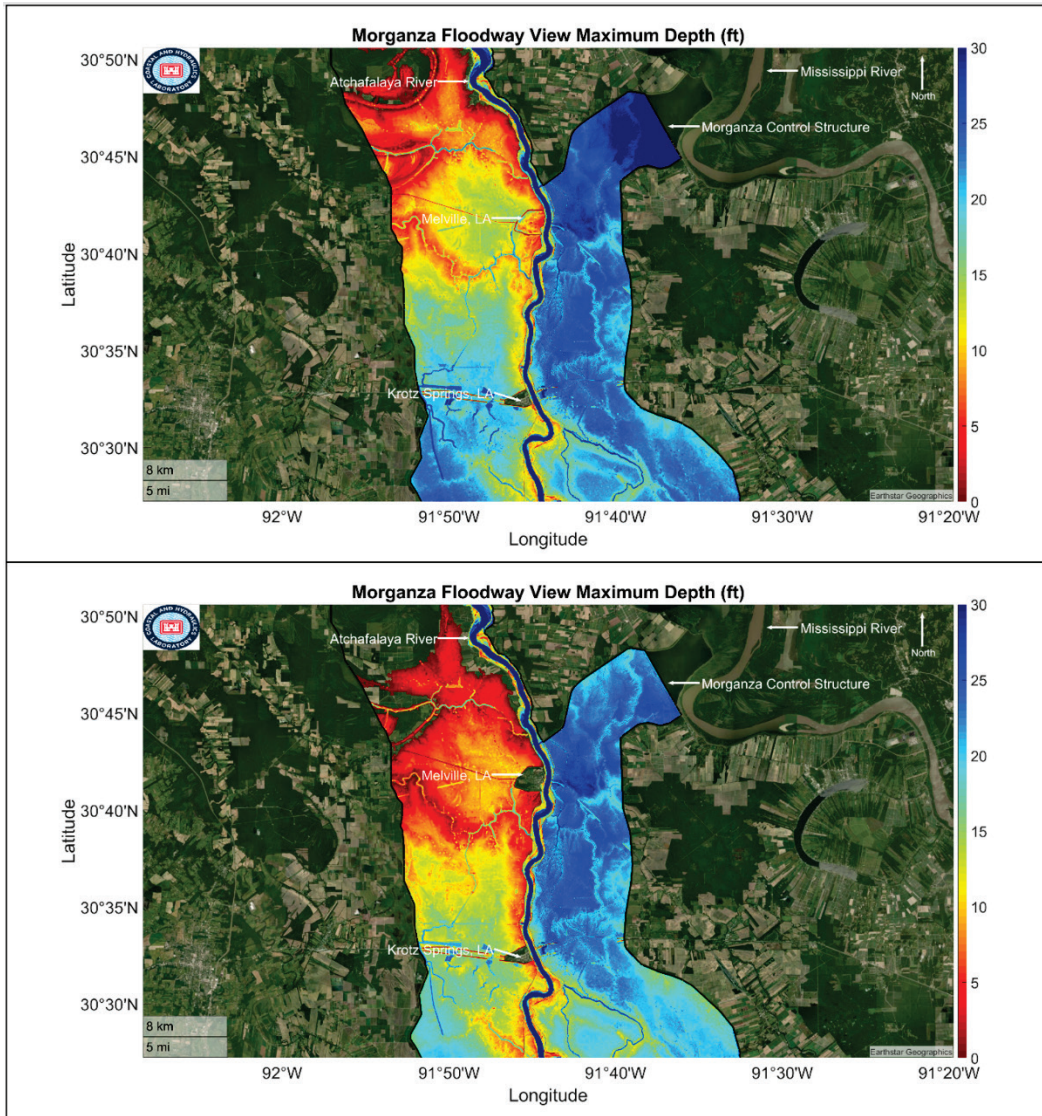


Figure 32. Base (top) and AdjustVeg 2 (bottom) maximum depth for the Calumet in Wax Lake Outlet Channel.

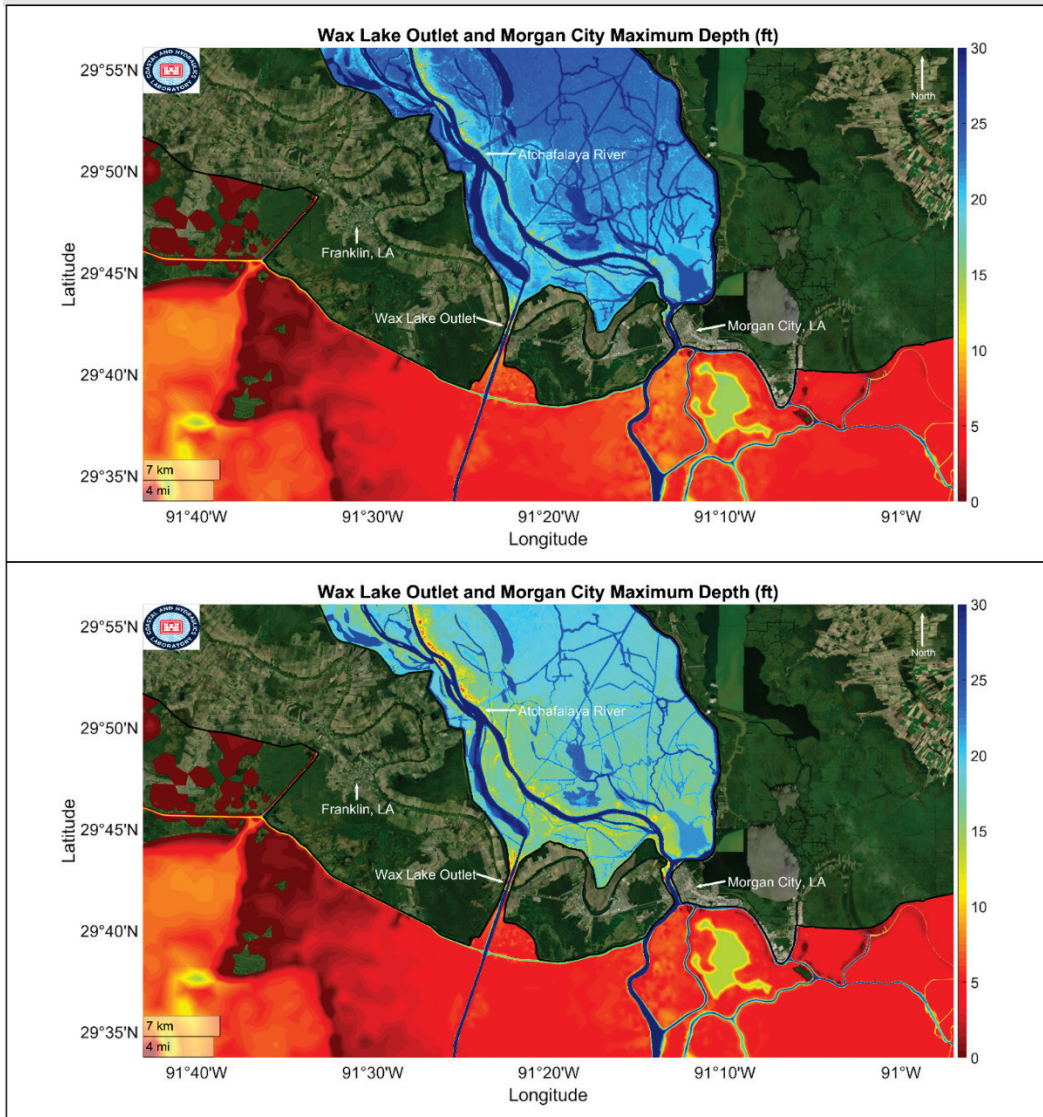


Figure 33. Base (top) and AdjustVeg 3 (bottom) maximum depth for the entire model domain.

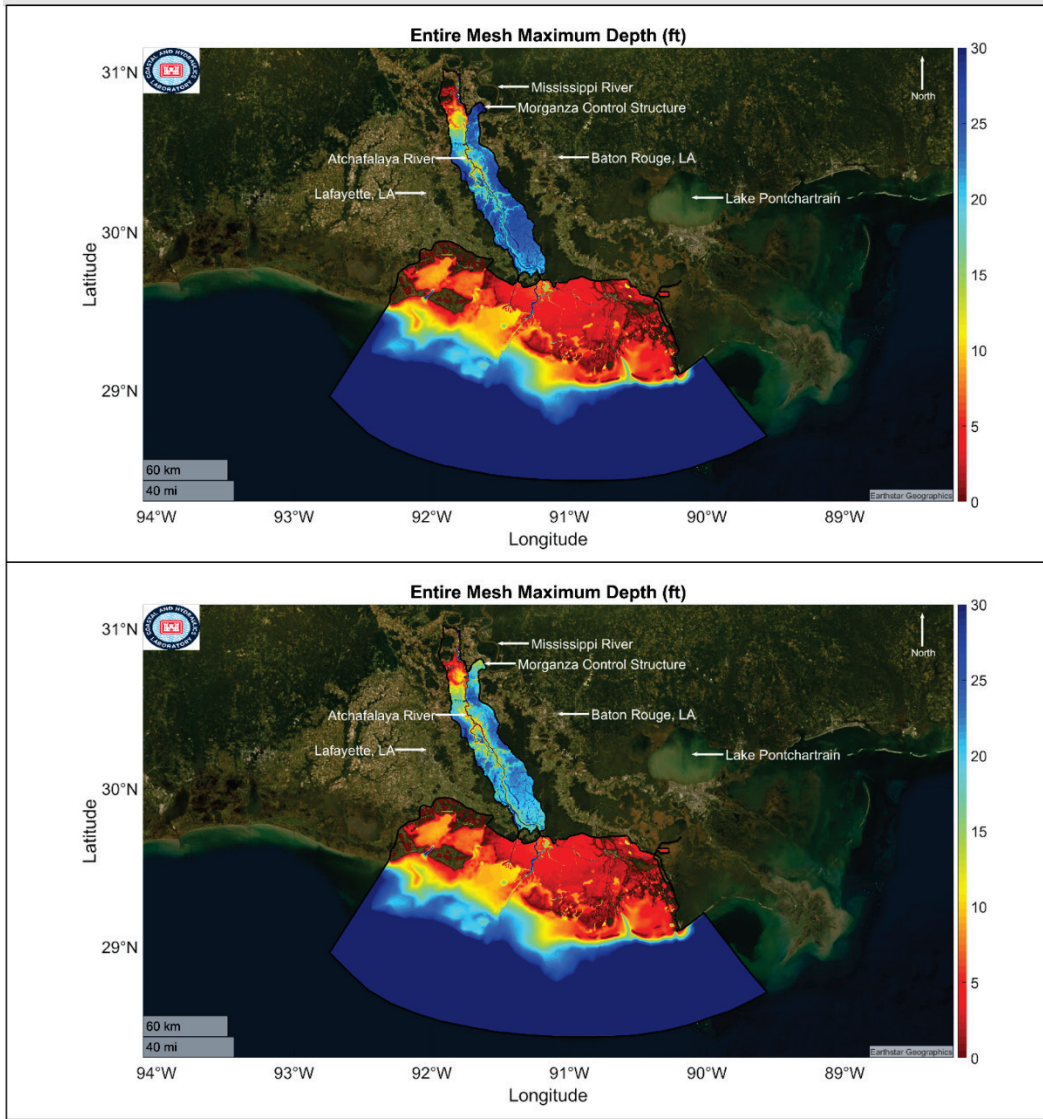


Figure 34. Base (top) and AdjustVeg 3 (bottom) maximum depth for the Morganza Floodway.

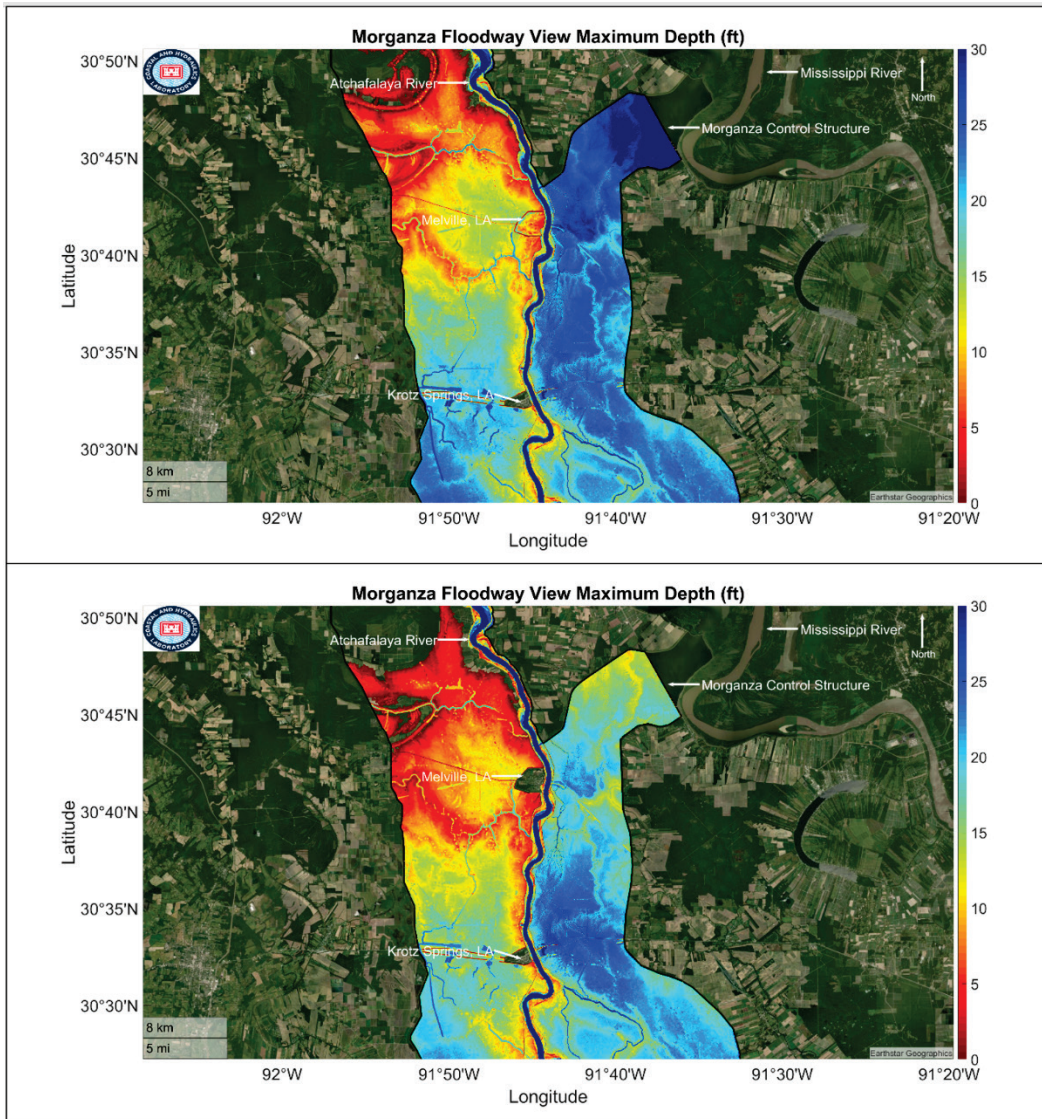


Figure 35. Base (top) and AdjustVeg 3 (bottom) maximum depth for the Calumet in Wax Lake Outlet Channel.

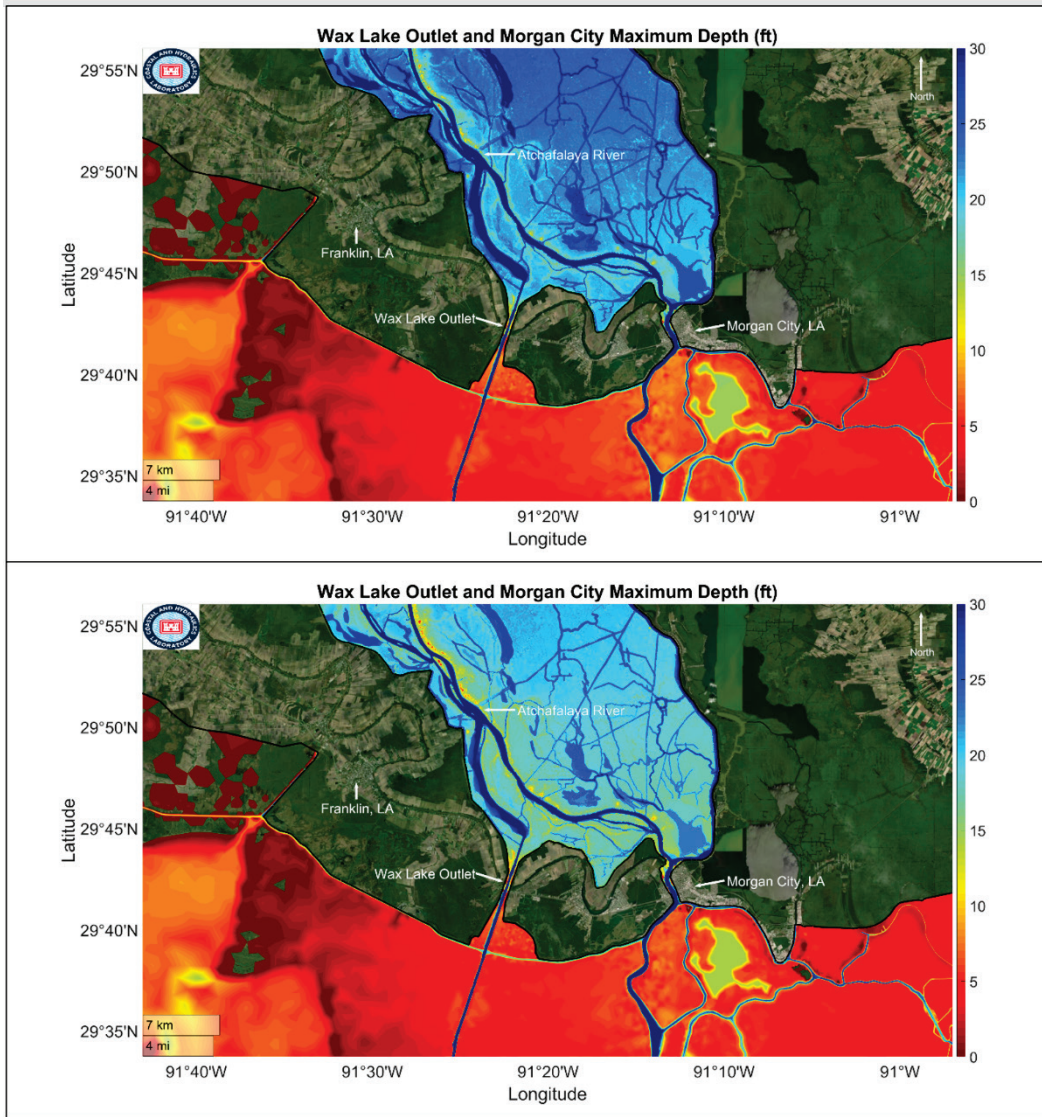
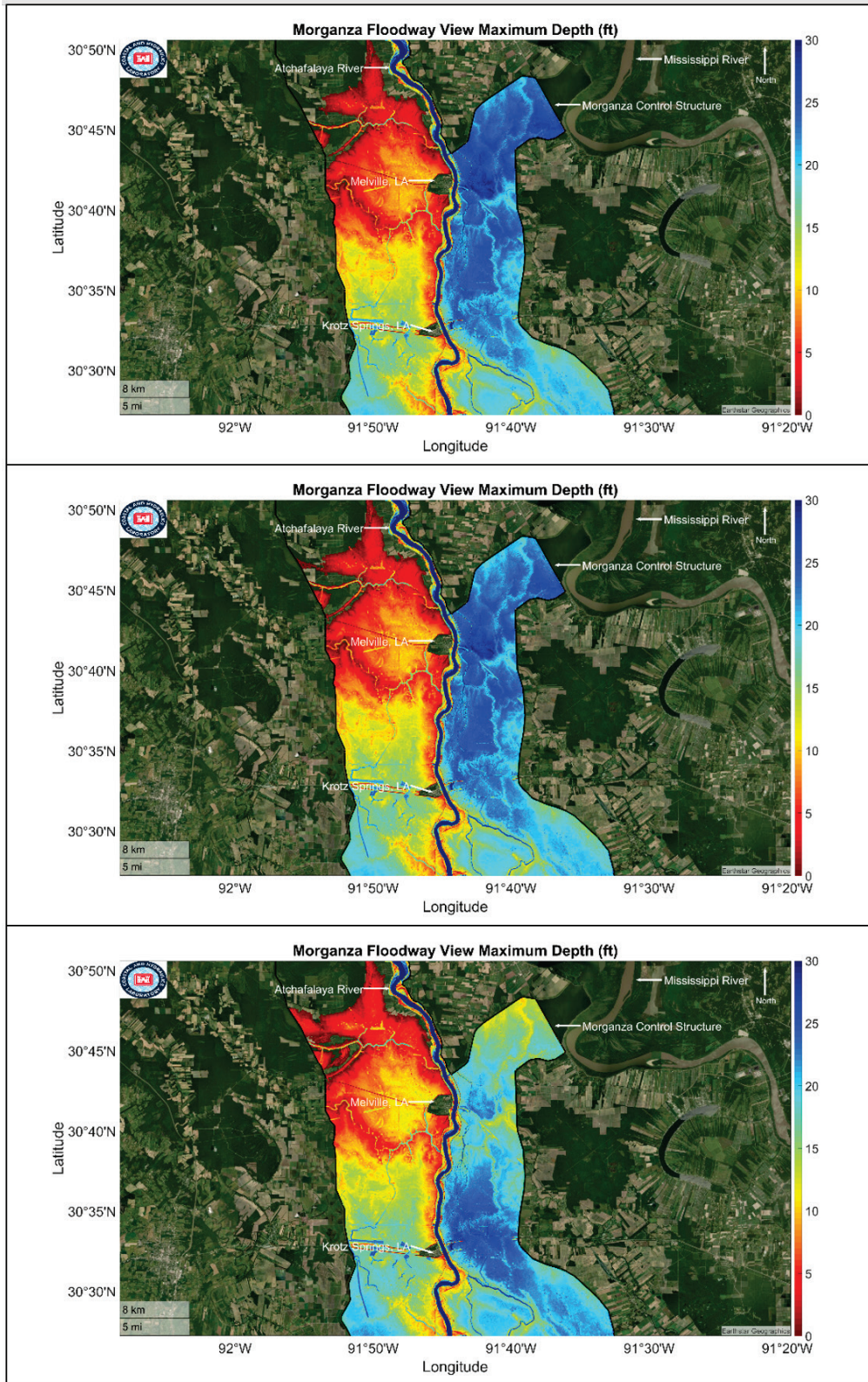


Figure 36. AdjustVeg 1, 2 and 3 comparison (top to bottom, respectively) for the Morganza Floodway.



For each of the hypothetical vegetation adjustment simulations, a total of 15 cross sections (depicted as arcs in the following figures) were analyzed to monitor the WSE at the levees (Figure 37). As more of the area of the Morganza Floodway was adjusted to have a roughness corresponding to open land, the water depth decreased. The temporal change (day 7, 9, and 11) for each arc throughout the simulation can be seen in Appendix B. WSE profiles at each arc at day 13 can be seen in Figure 38 – Figure 52. For the *x*-axis scale in the figures, the distance = 0 ft starts on the west Morganza Floodway levee. Note that at day 13 (Figure 26), the MCS has reached and held constant at 600,000 cfs for approximately 16 hr. Also note that the model is overtopping in numerous locations for the base model simulation. The model results indicate that the AdjustVeg 1 simulations also have multiple levee overtopping locations but are reduced in comparison with the base model. Similarly, the AdjustVeg 2 simulation results in a further decrease in WSE throughout the Morganza Floodway. AdjustVeg 3, which is the most extreme of the sensitivity analysis runs, results in no levees being overtopped on day 13 at any of the selected arcs. In general, the sensitivity analysis showed that the hypothetical changes in roughness impact the WSE upstream of the adjusted area. Note that the base model WSE profile varies slightly more along the cross-sectional plots than some of the AdjustVeg simulations due to increased roughness in the areas where vegetation was not adjusted (i.e., higher WSEs).

Figure 37. Arc locations for the vegetation adjustment simulations.

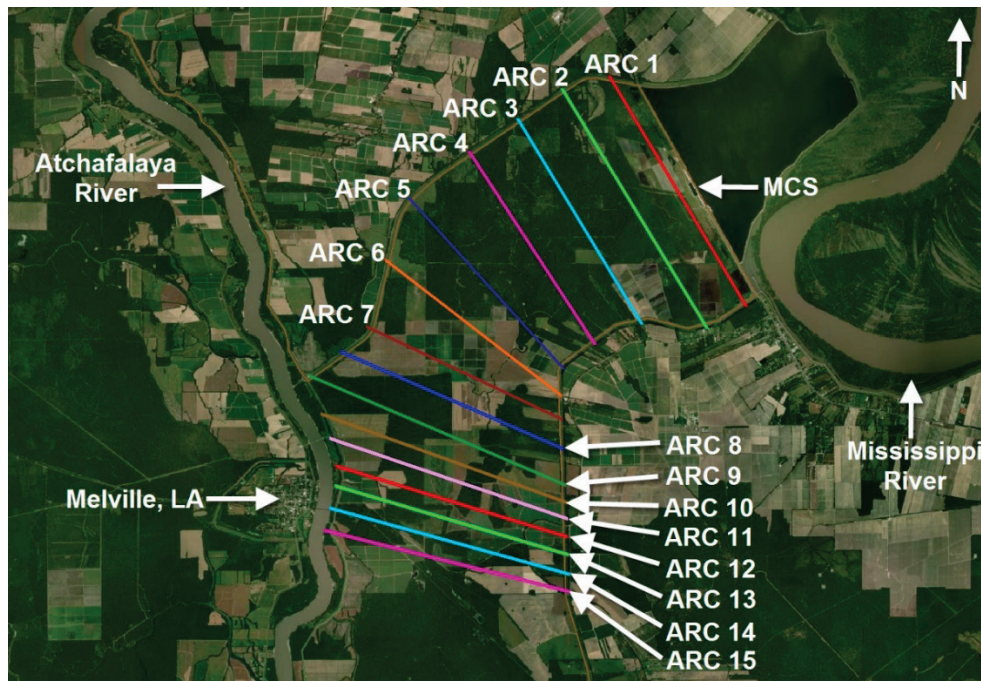


Figure 38. Arc 1 day 13 WSE profile.

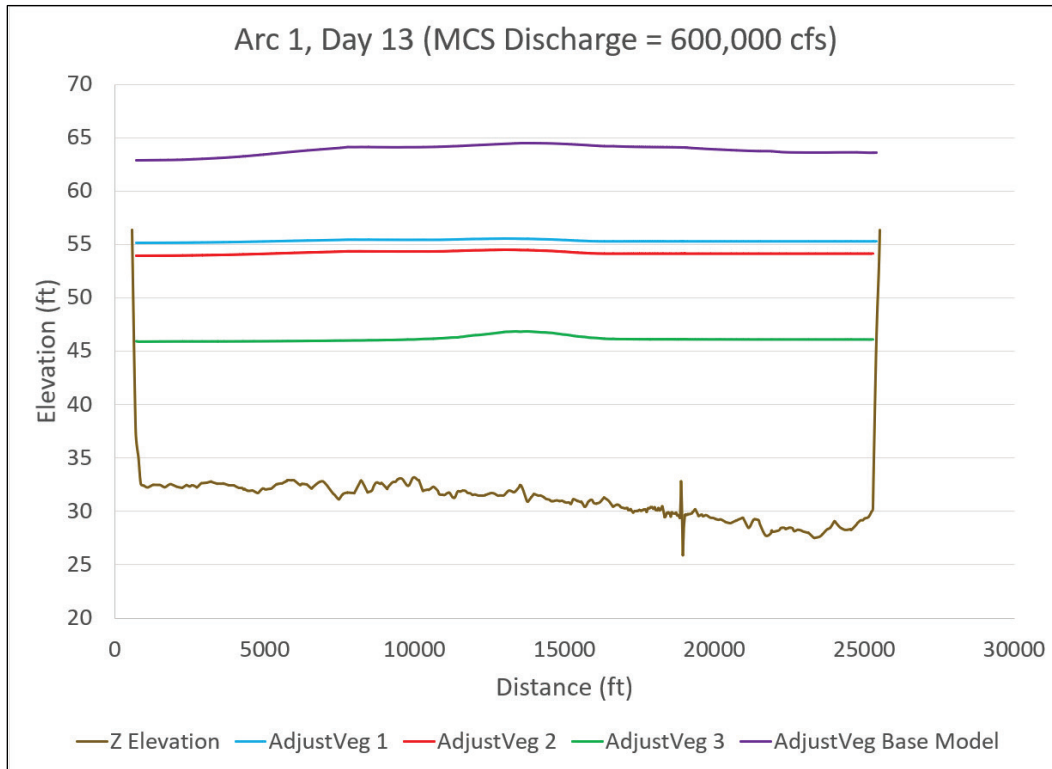


Figure 39. Arc 2 day 13 WSE profile.

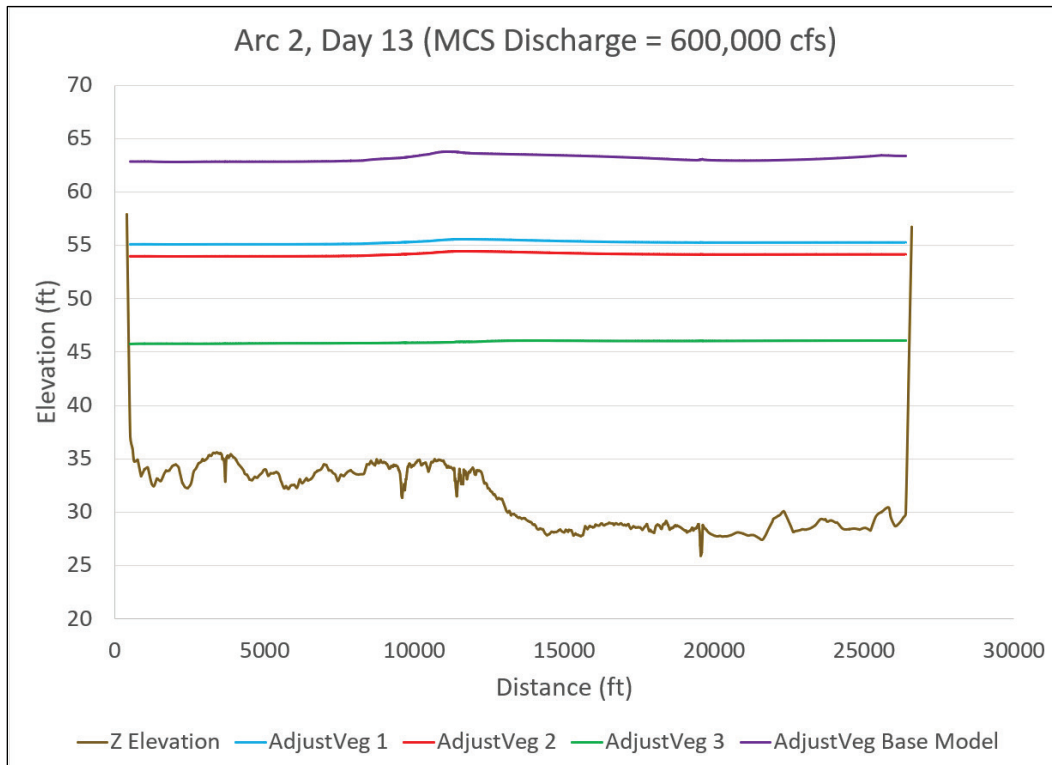


Figure 40. Arc 3 day 13 WSE profile.

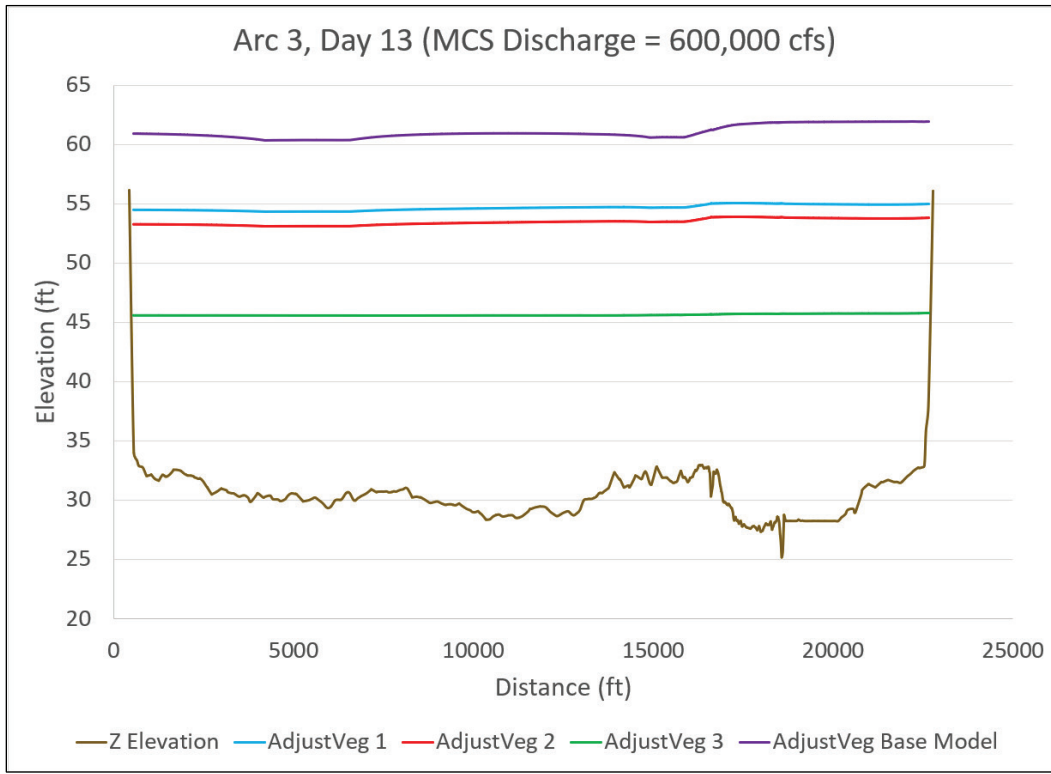


Figure 41. Arc 4 day 13 WSE profile.

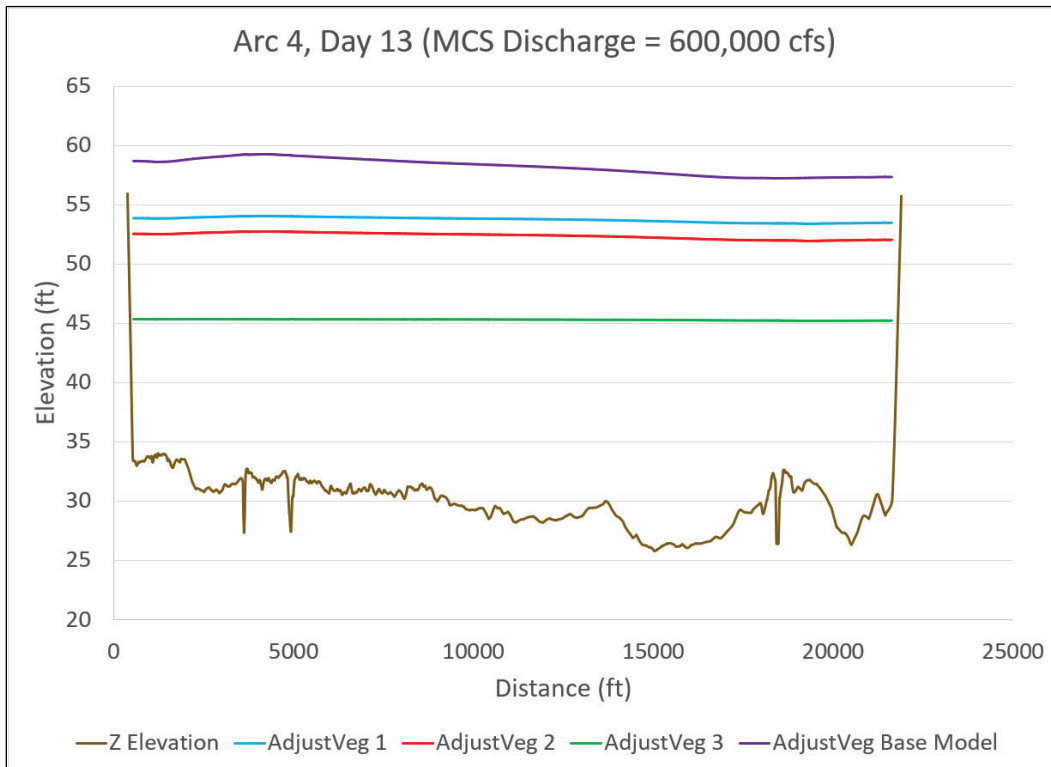


Figure 42. Arc 5 day 13 WSE profile.

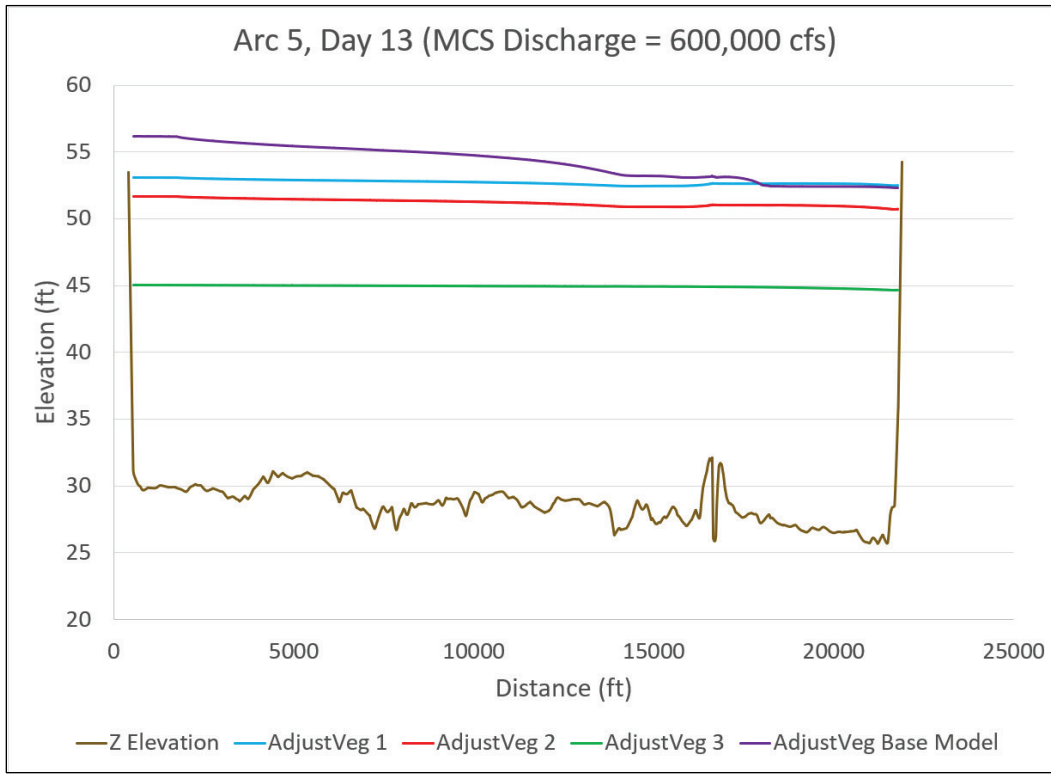


Figure 43. Arc 6 day 13 WSE profile.

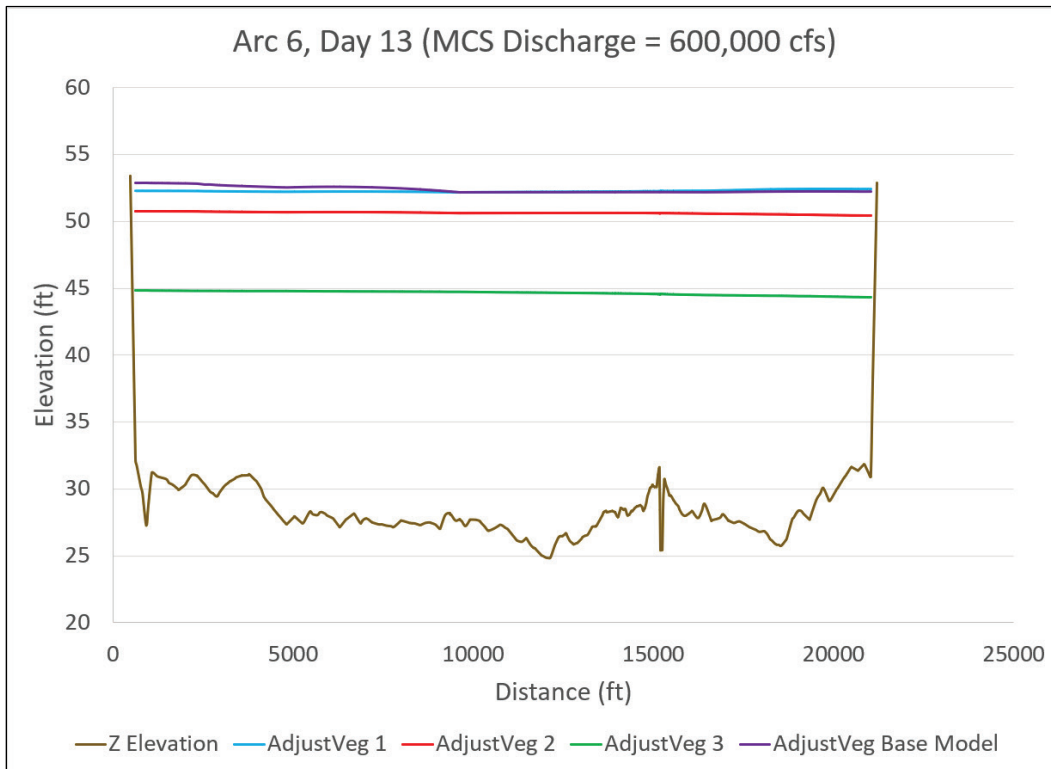


Figure 44. Arc 7 day 13 WSE profile.

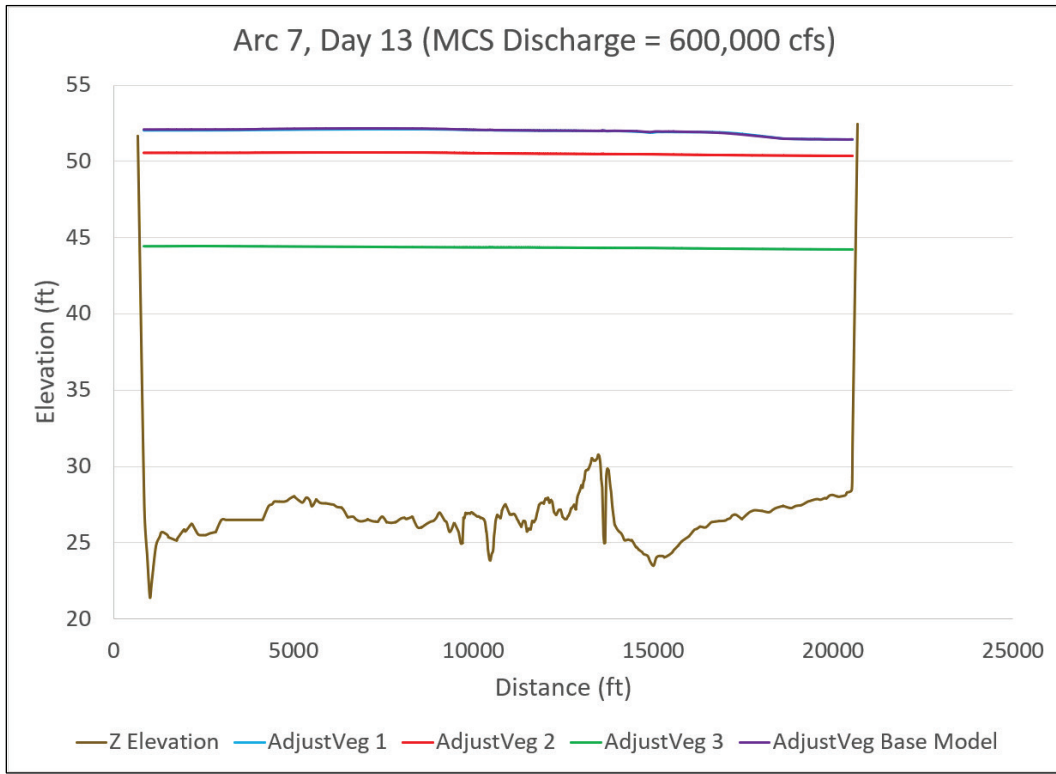


Figure 45. Arc 8 day 13 WSE profile.

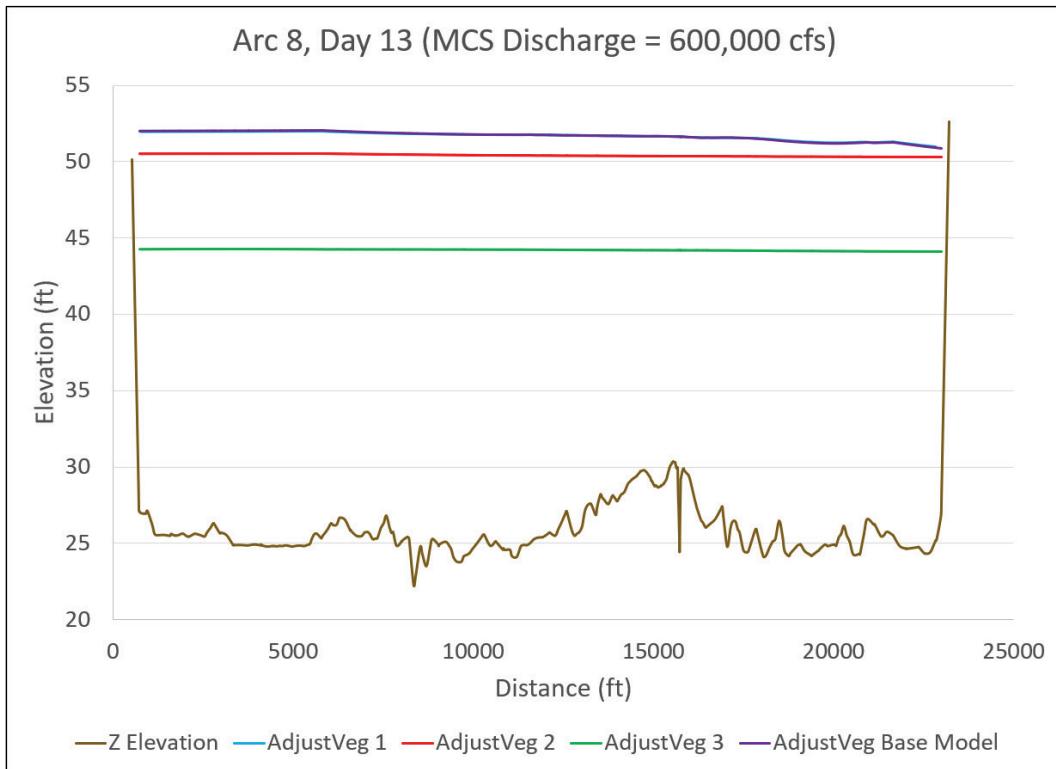


Figure 46. Arc 9 day 13 WSE profile.

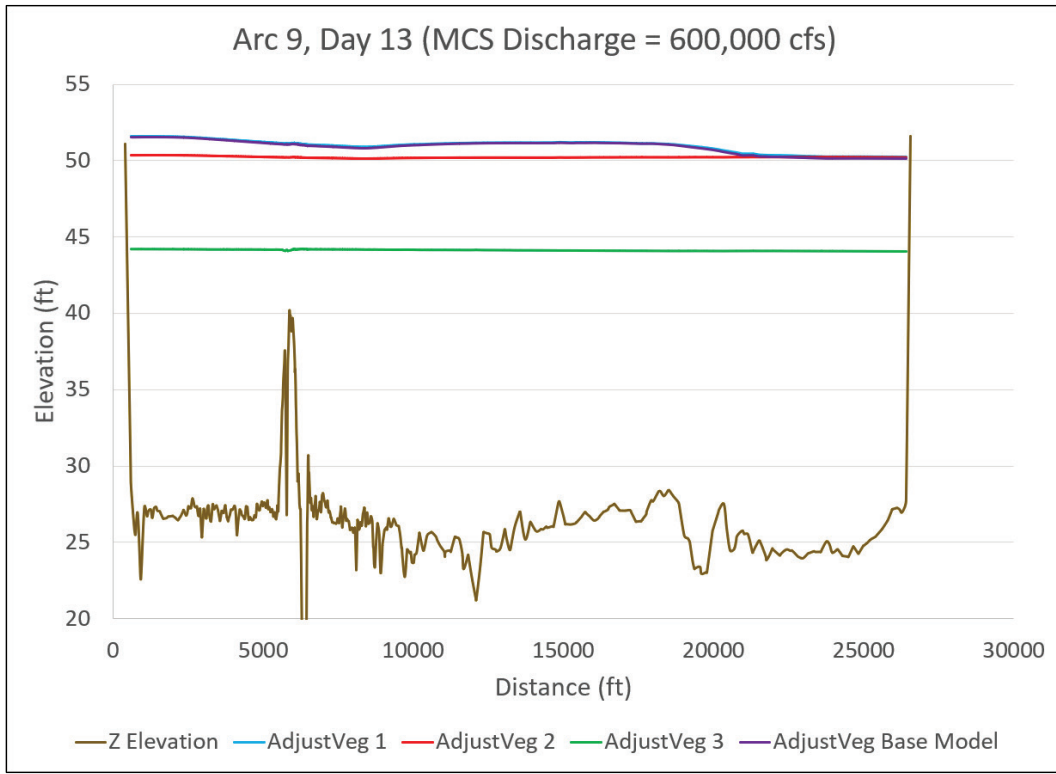


Figure 47. Arc 10 day 13 WSE profile.

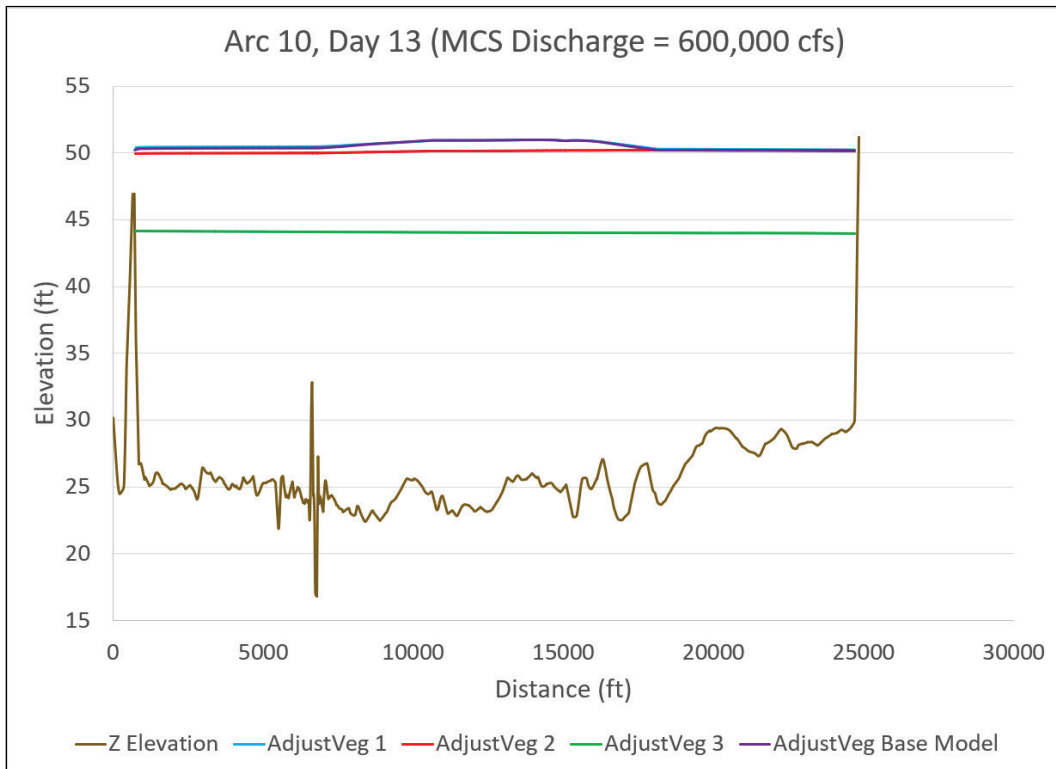


Figure 48. Arc 11 day 13 WSE profile.

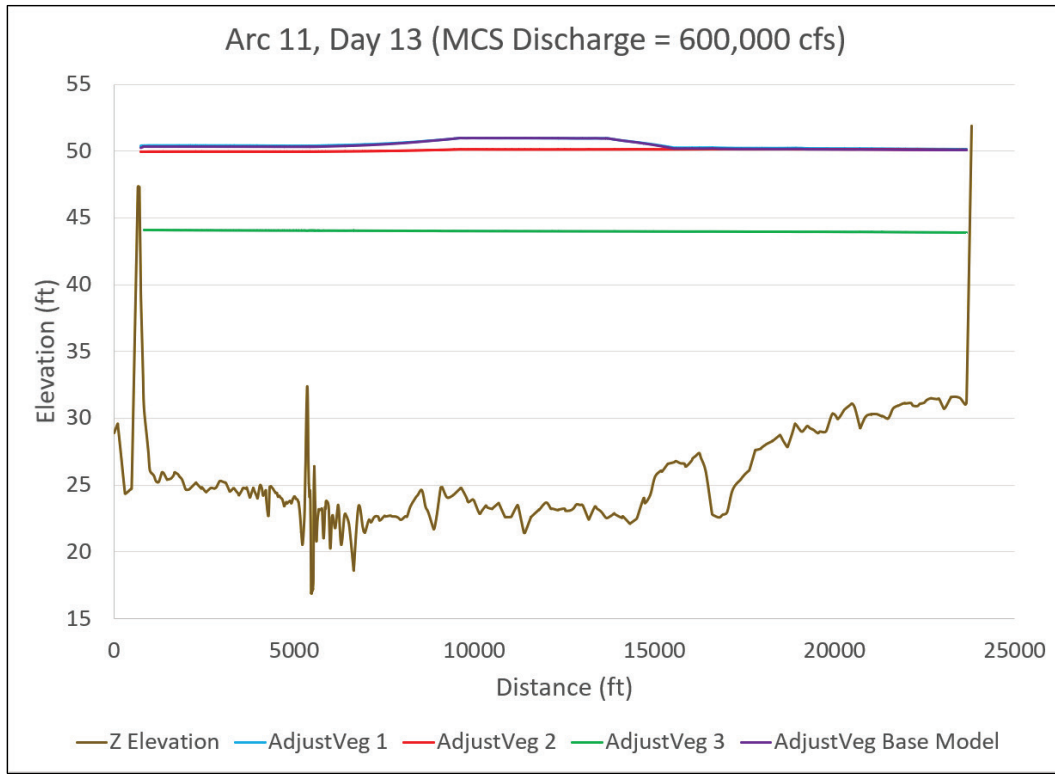


Figure 49. Arc 12 day 13 WSE profile.

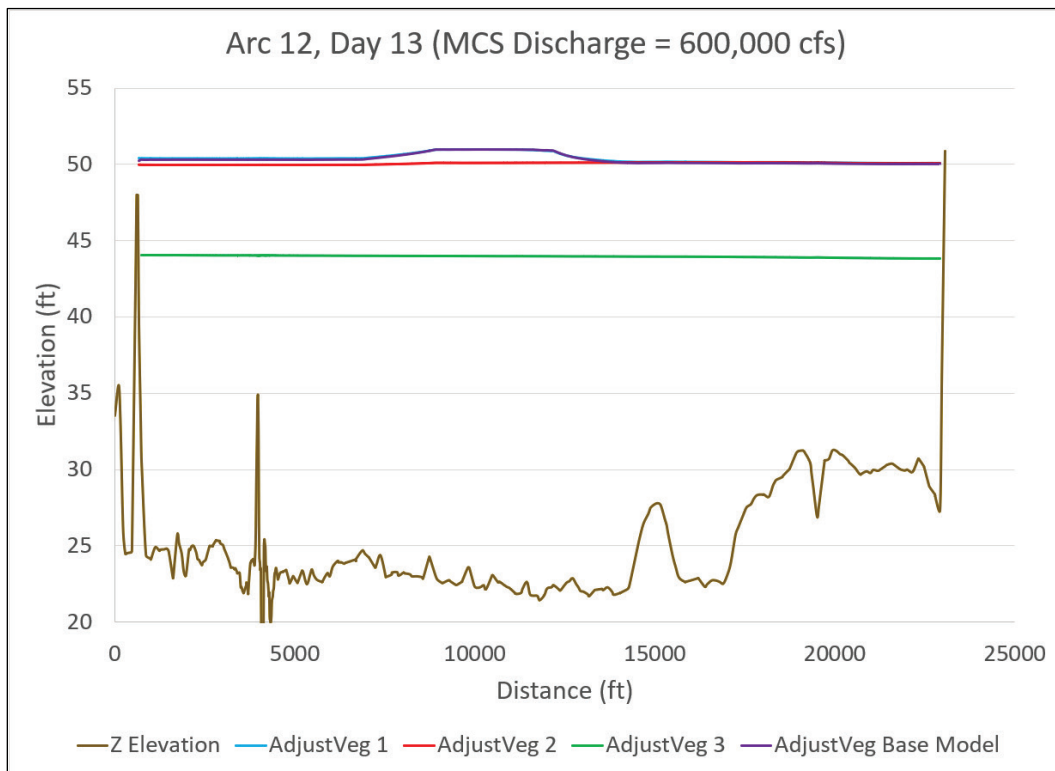


Figure 50. Arc 13 day 13 WSE profile.

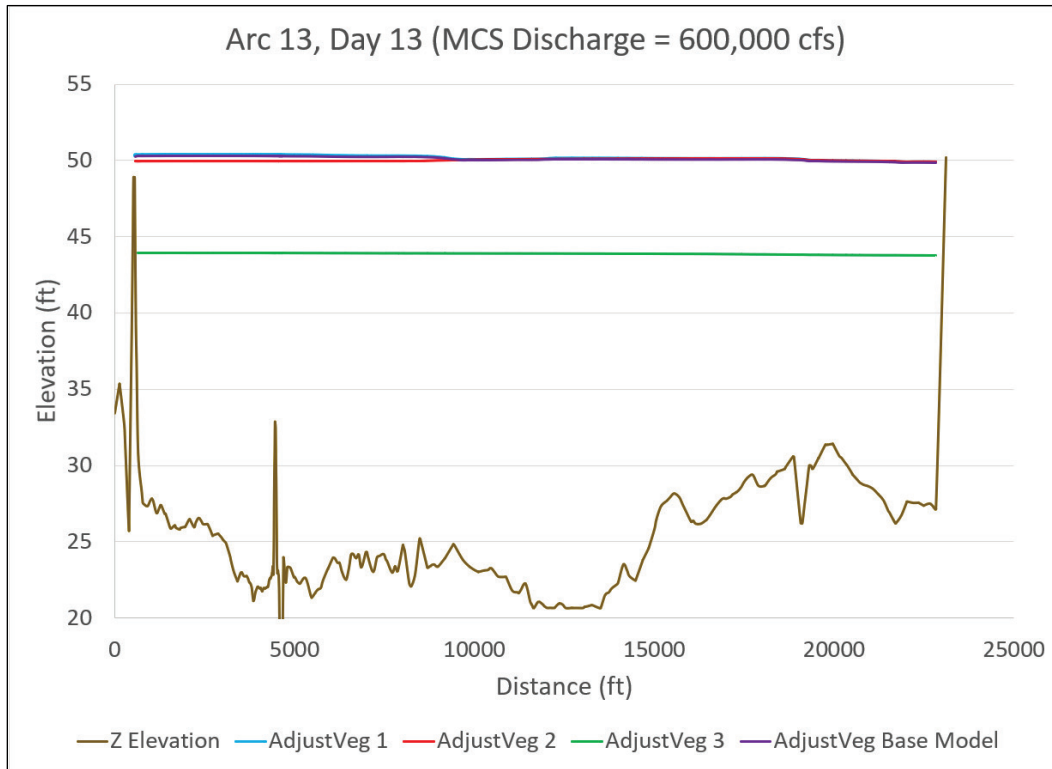


Figure 51. Arc 14 day 13 WSE profile.

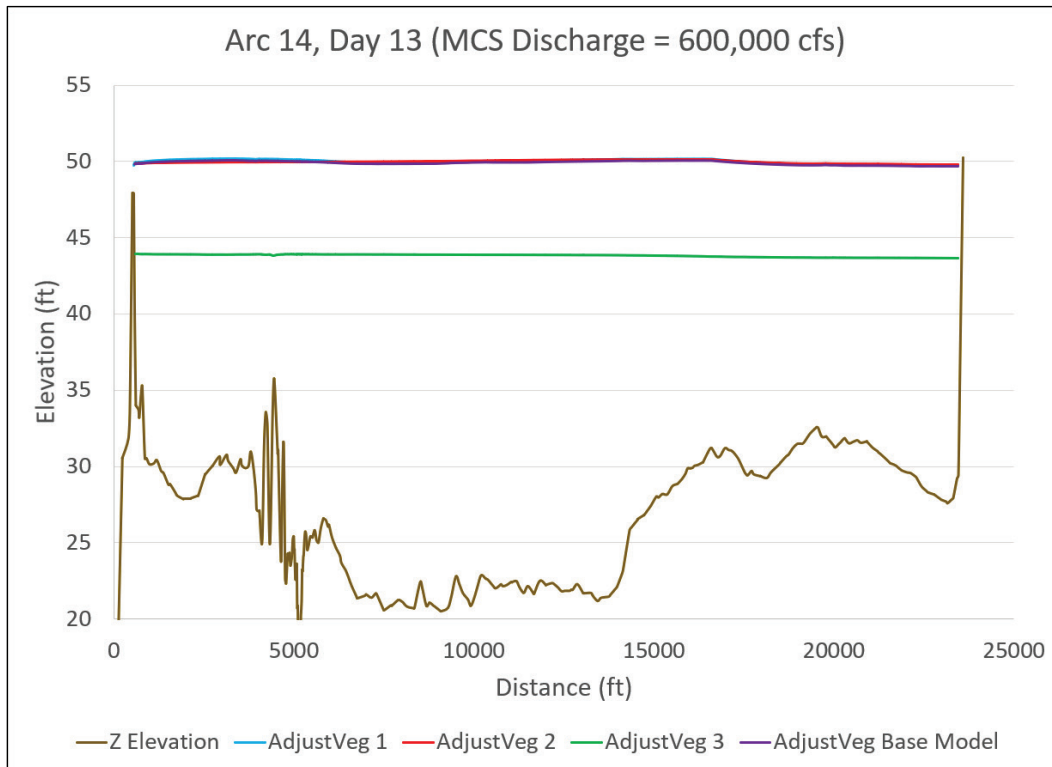
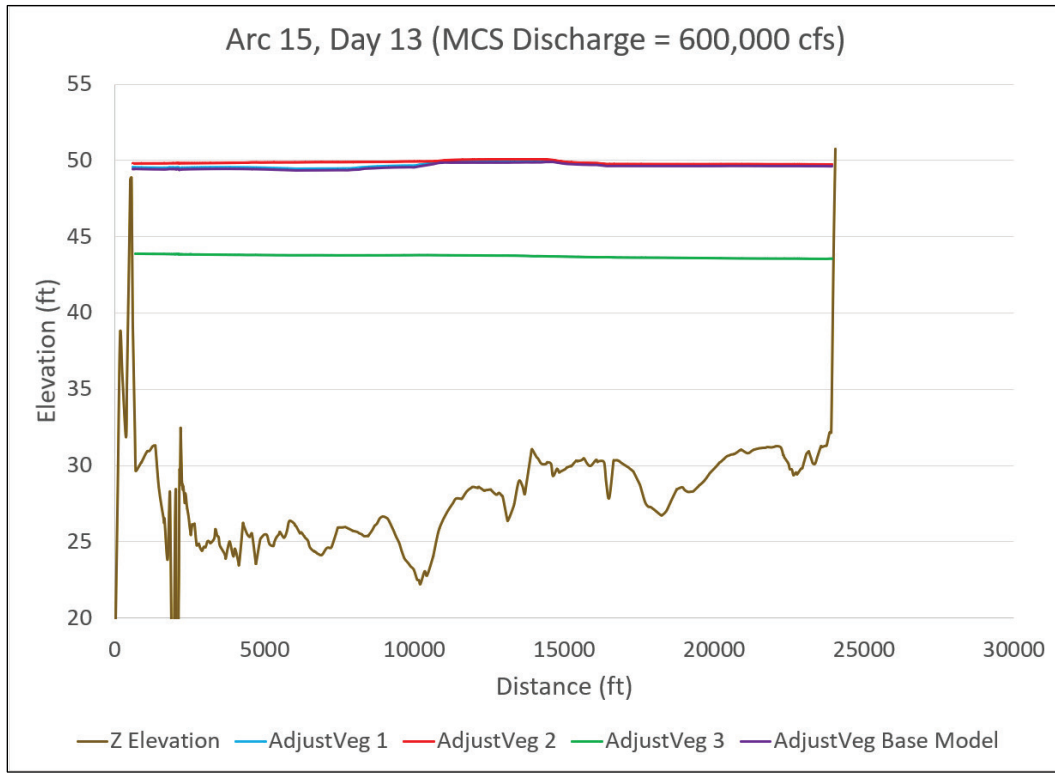


Figure 52. Arc 15 day 13 WSE profile.



For each of the AdjustVeg simulations, inundation maps were created for days 7, 9, 11, and 13 to capture the differences in WSE for each simulation. These maps can be seen in Figure 53 – Figure 56. These inundation maps show the change in water depth with respect to time throughout the floodway. These maps are different from those shown in Figure 27 – Figure 35 above in that they do not necessarily show the maximum elevation during the simulation. Instead, they show the actual change of elevation with time for each incremental amount of roughness adjustment.

Figure 53. AdjustVeg inundation maps for day 7 (MCS discharge = 345,745 cfs).

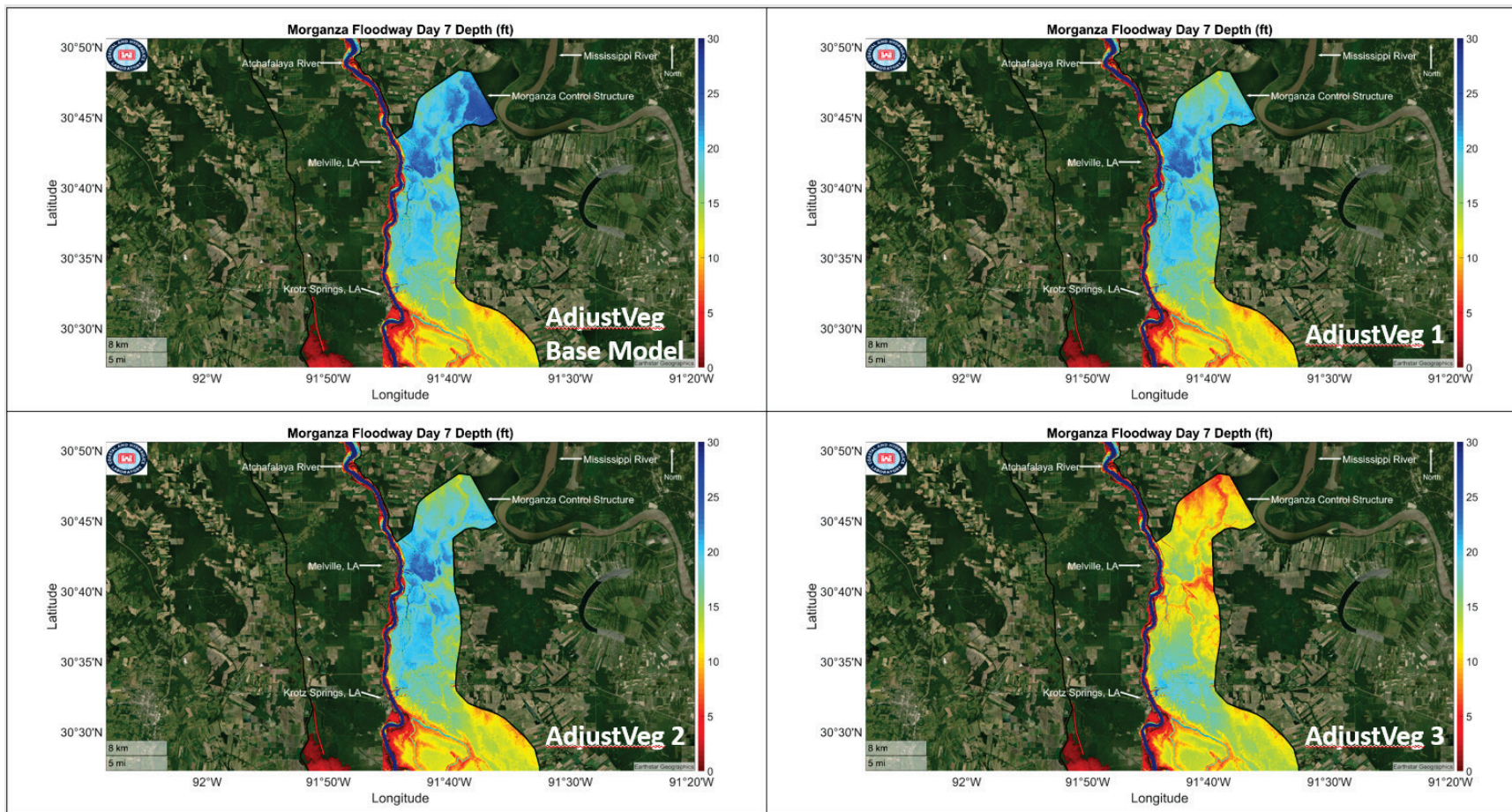


Figure 54. AdjustVeg inundation maps for day 9 (MCS discharge = 441,745 cfs).

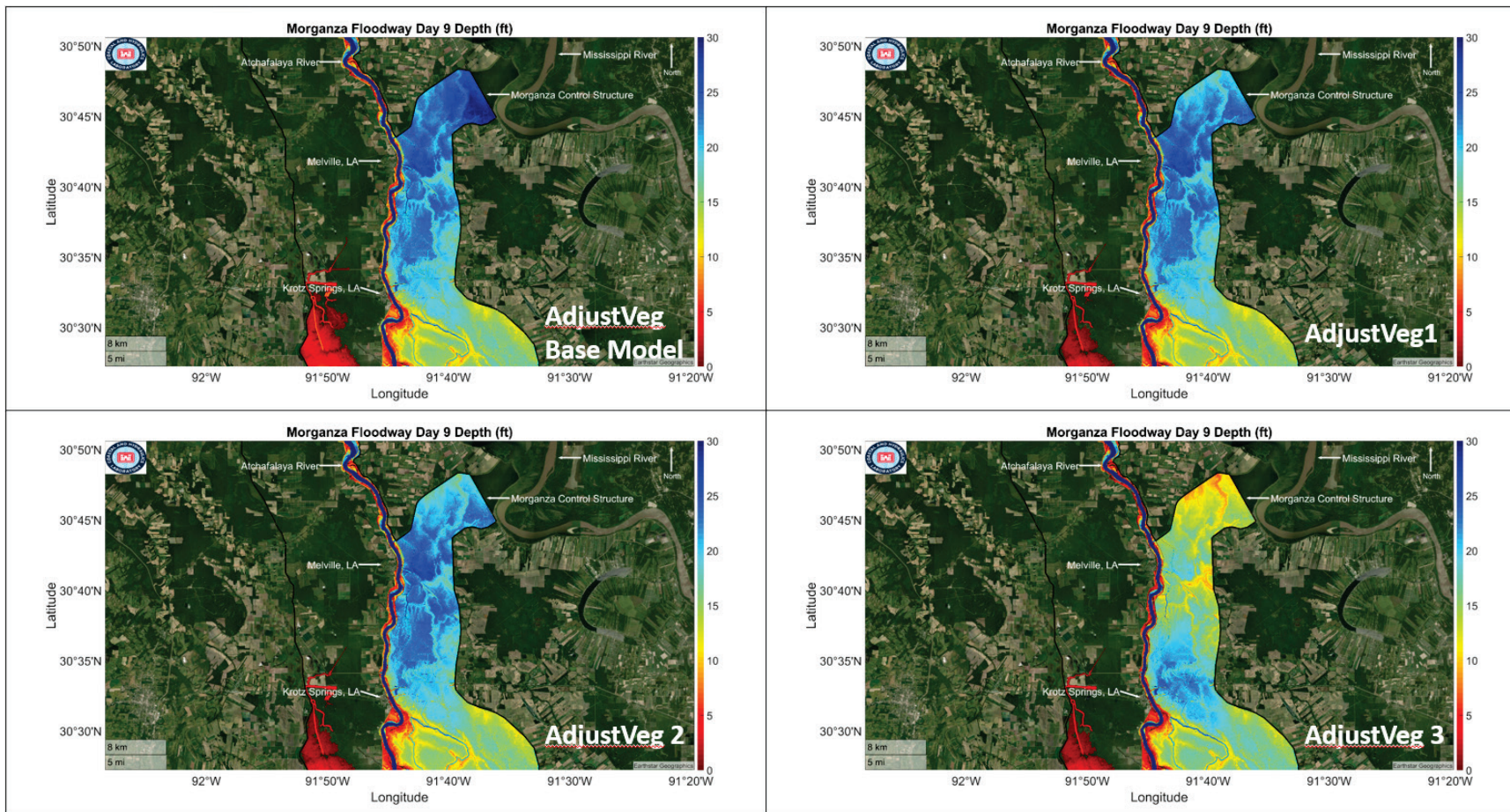


Figure 55. AdjustVeg inundation maps for day 11 (MCS discharge = 537,745 cfs).

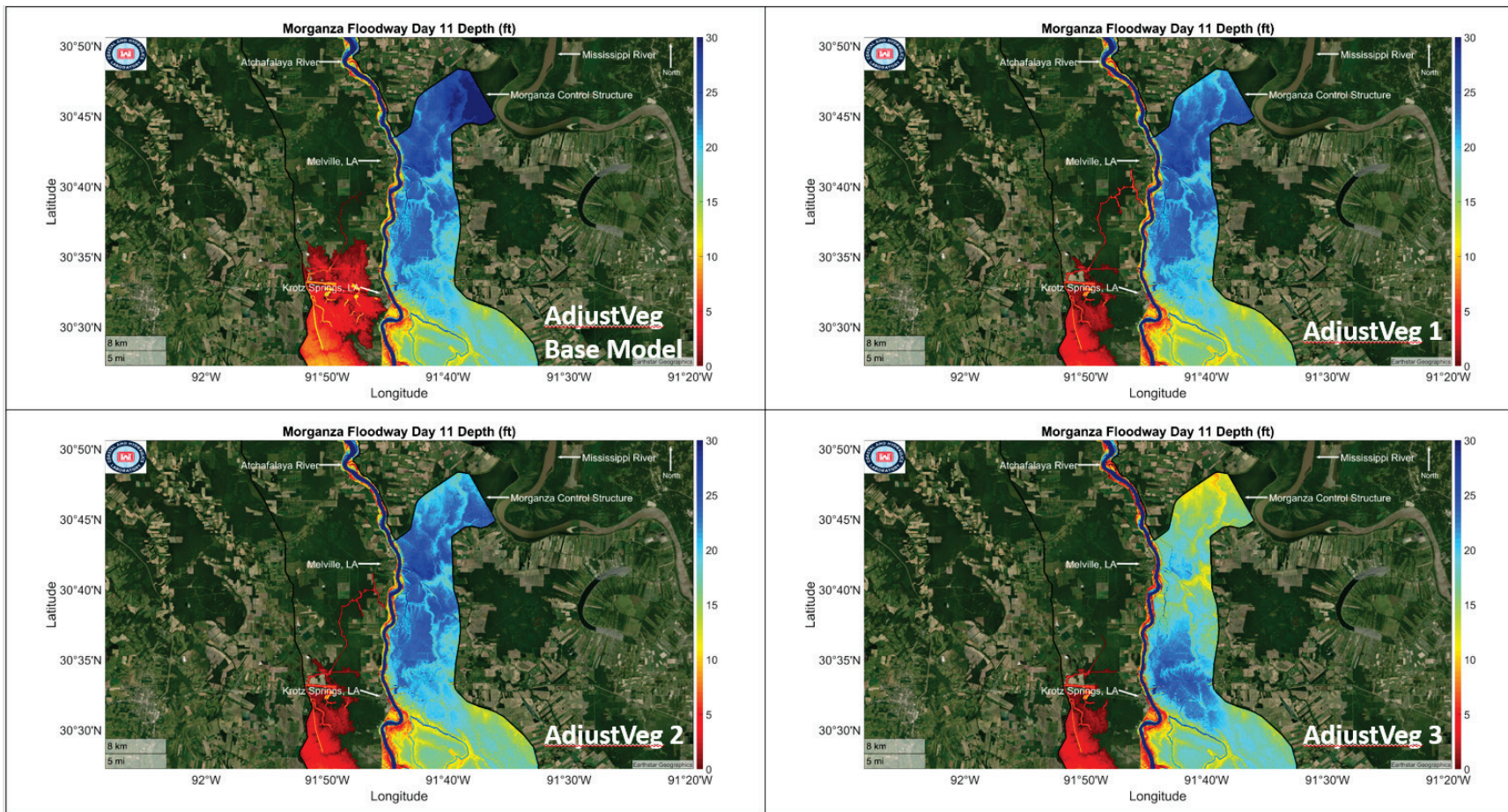
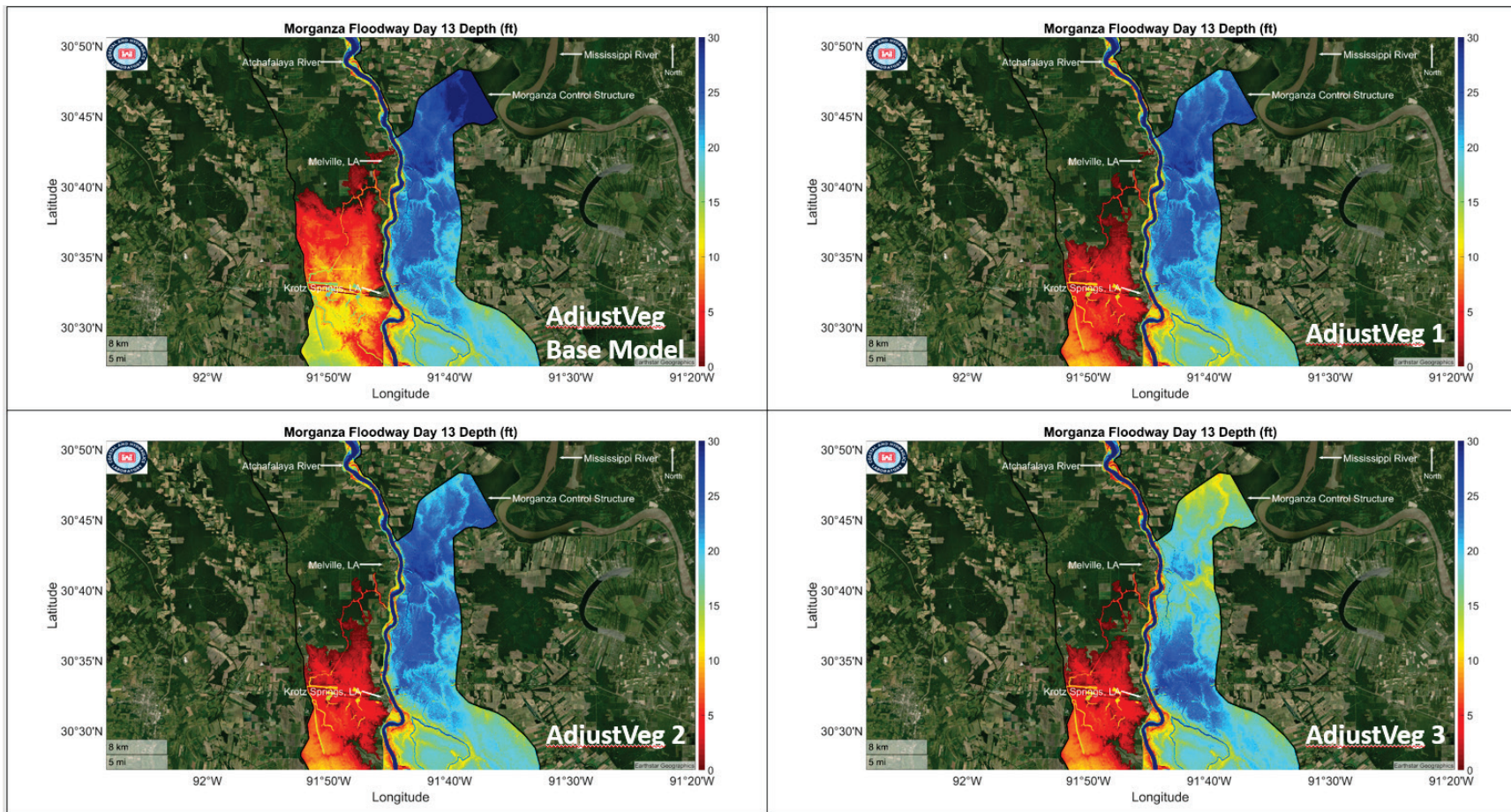


Figure 56. AdjustVeg inundation maps for day 13 (MCS discharge = 600,000 cfs).



An important observation based on the model results is that it appears the floodway levees, in their present condition, begin to overtop before day 9 or a discharge of 441,745 cfs. In Figure 57 – Figure 59, the WSE profile for Arc 1 can be seen for days 7, 9, and 11 to show an approximation of the levee overtopping in the base conditions well before reaching the PDF of 600,000 cfs. The rest of these temporal arcs can be seen in Appendix B.

Figure 57. Arc 1 day 7 WSE profile.

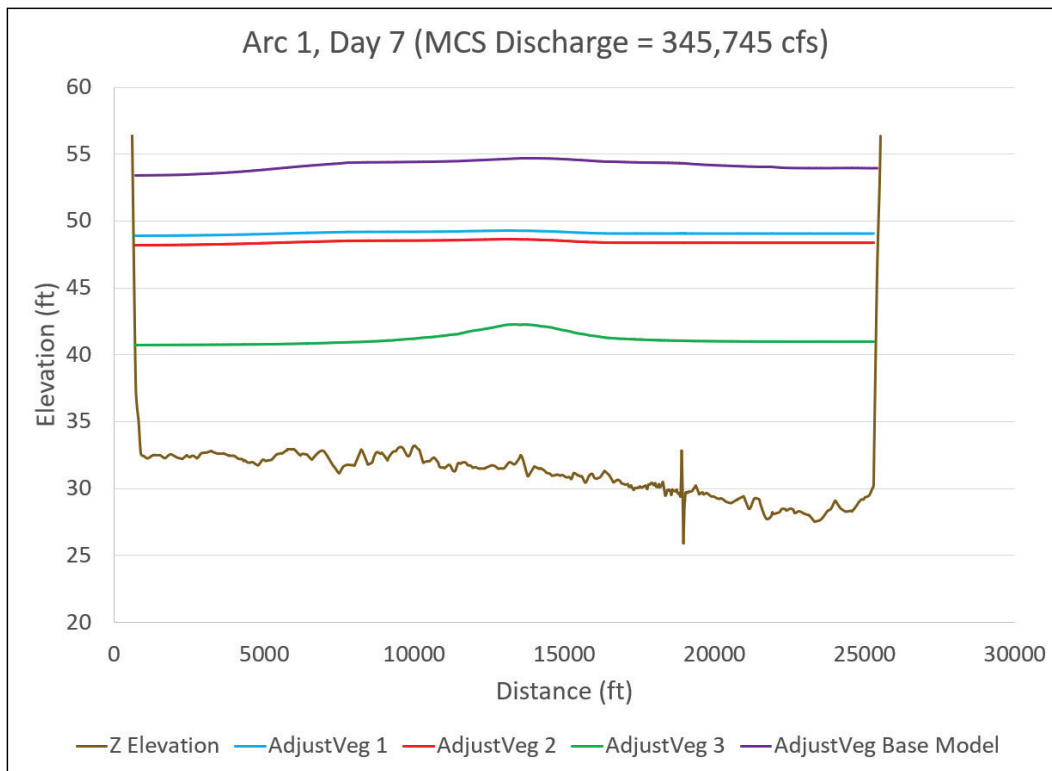


Figure 58. Arc 1 day 9 WSE profile.

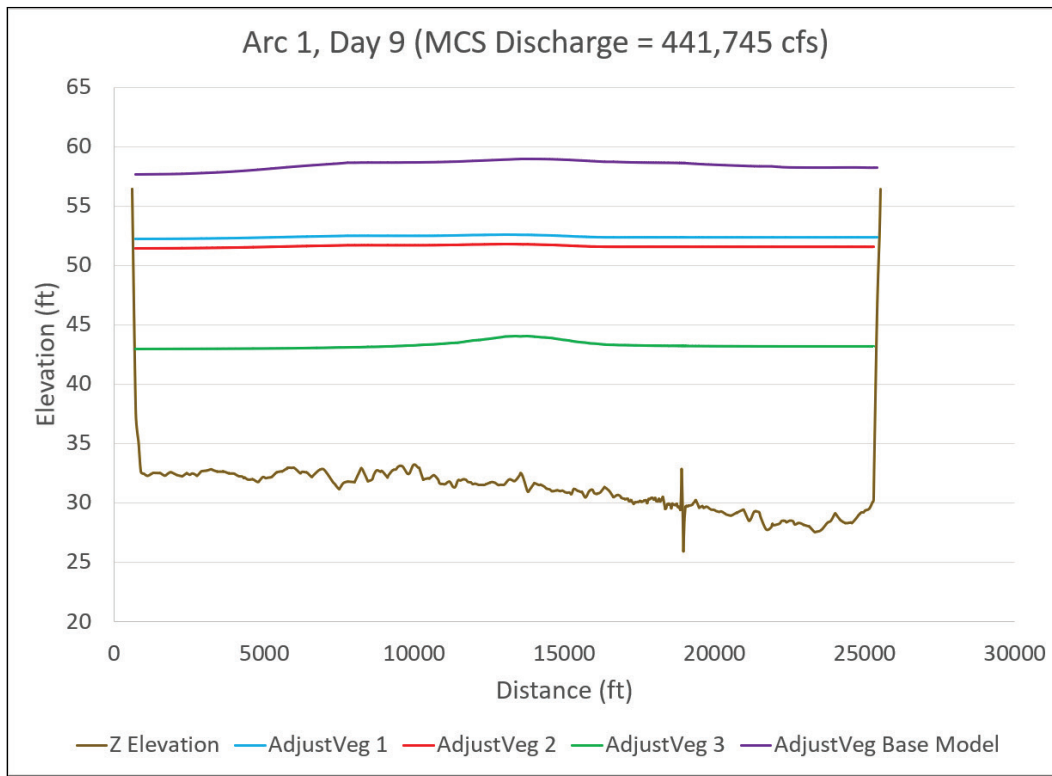
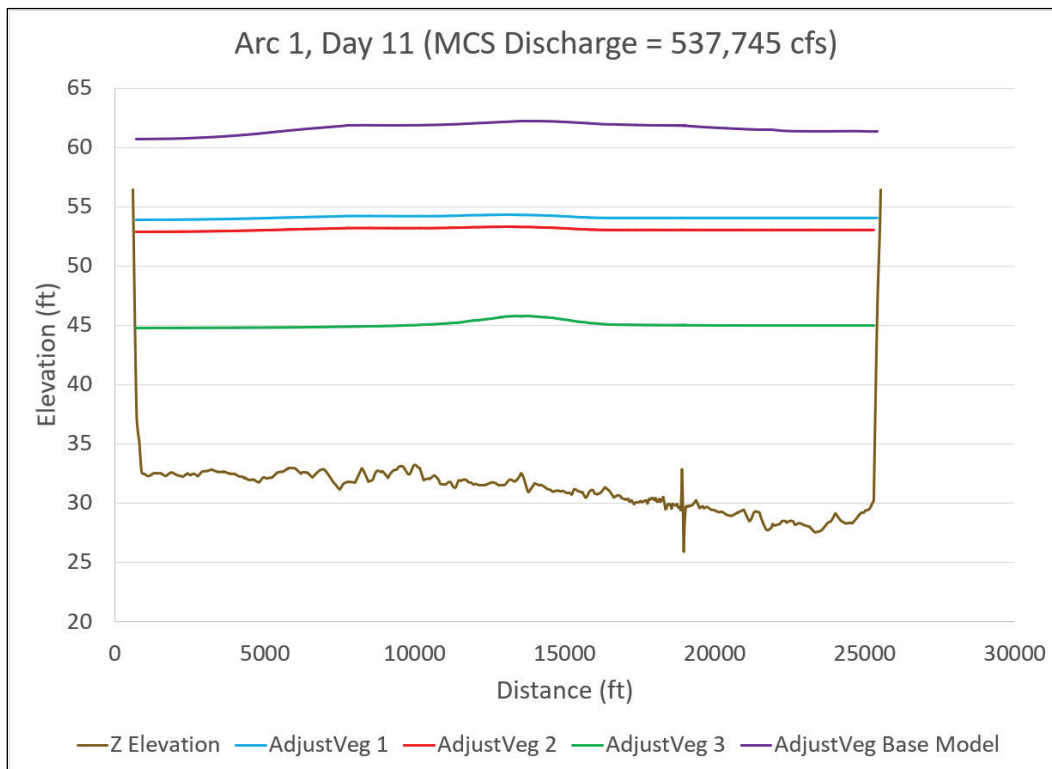


Figure 59. Arc 1 day 11 WSE profile.



### **3.4 Task 4: Inundation mapping for specified Morganza Control Structure (MCS) and Atchafalaya River inflows**

For many flood control projects, like the Morganza Floodway and Atchafalaya Basin, levee elevations were determined through the application of various hypothetical hydrologic events, such as the Standard Project Flood (SPF)<sup>1</sup>. The inflow values used in Task 4 were suggested from collaboration efforts among MVD and the New Orleans District (MVN). These hypothetical scenarios of interest were used for the inundation maps found in this section (labeled as P1 – P4). The modeled boundary inflow rates for the Atchafalaya River as well as the MCS can be found in Figure 60 – Figure 63. The model used for all the Task 4 simulations was the base-condition model (i.e., as is with no vegetative adjustments). Scenario 1 uses flows corresponding to the project design flood. Scenarios 2 through 4 utilize the 2019 hydrograph. Scenario 2 keeps the 2019 hydrograph as it is and releases a flow of 30 kcfs through MCS. Scenario 3 includes an increase in the peak Atchafalaya flow of the 2019 event and includes a flow of 100 kcfs through MCS. Scenario 4 has an additional increase in the Atchafalaya River peak flow and a flow of 200 kcfs through MCS.

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<sup>1</sup> USACE (US Army Corps of Engineers). 2012. Unpublished. *Mississippi River and Tributaries System, Draft 2011 Post-Flood Report*. See page II-5.

Figure 60. P1 inflow rates.

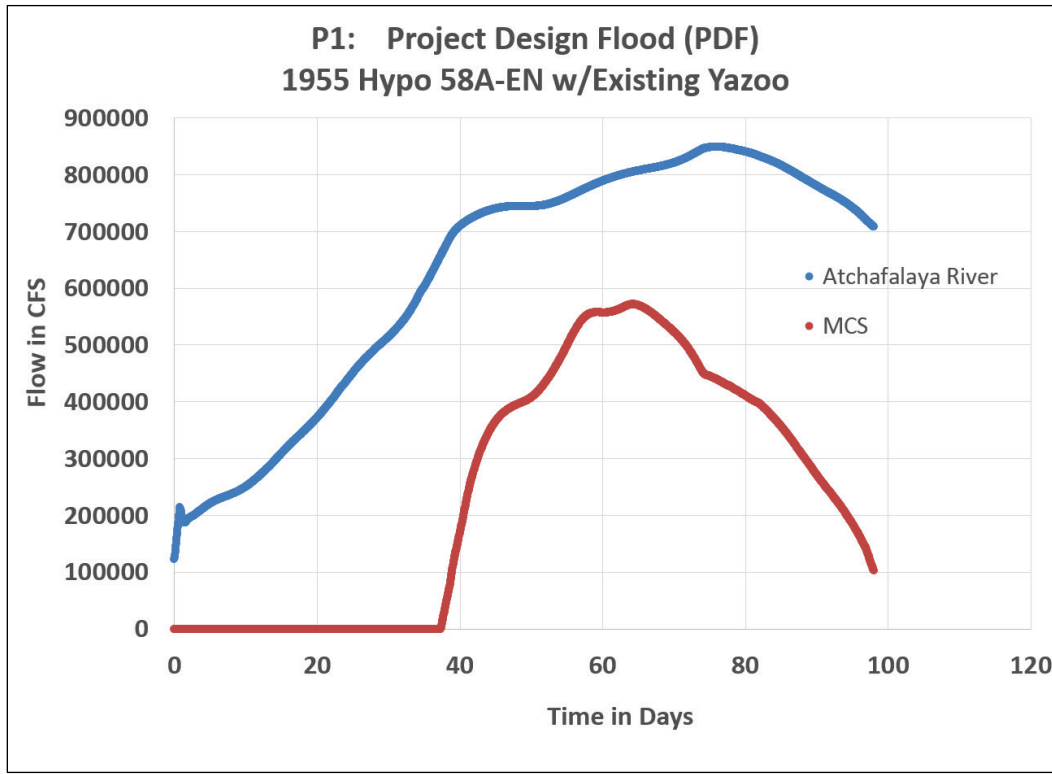


Figure 61. P2 inflow rates.

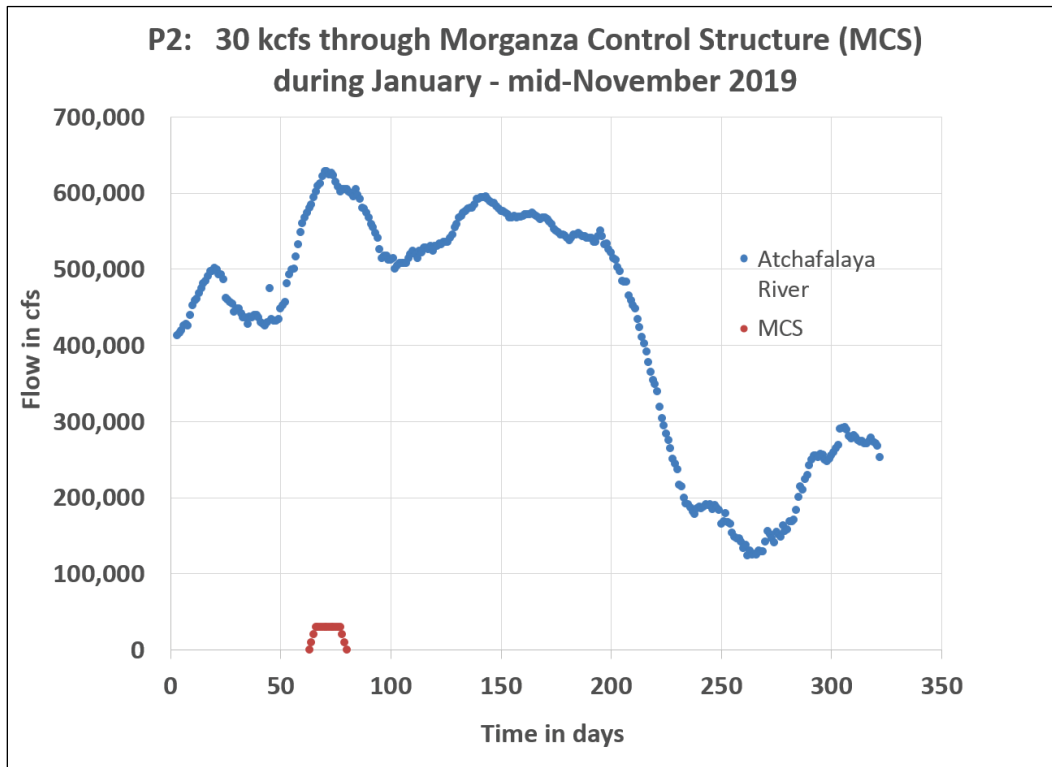


Figure 62. P3 inflow rates.

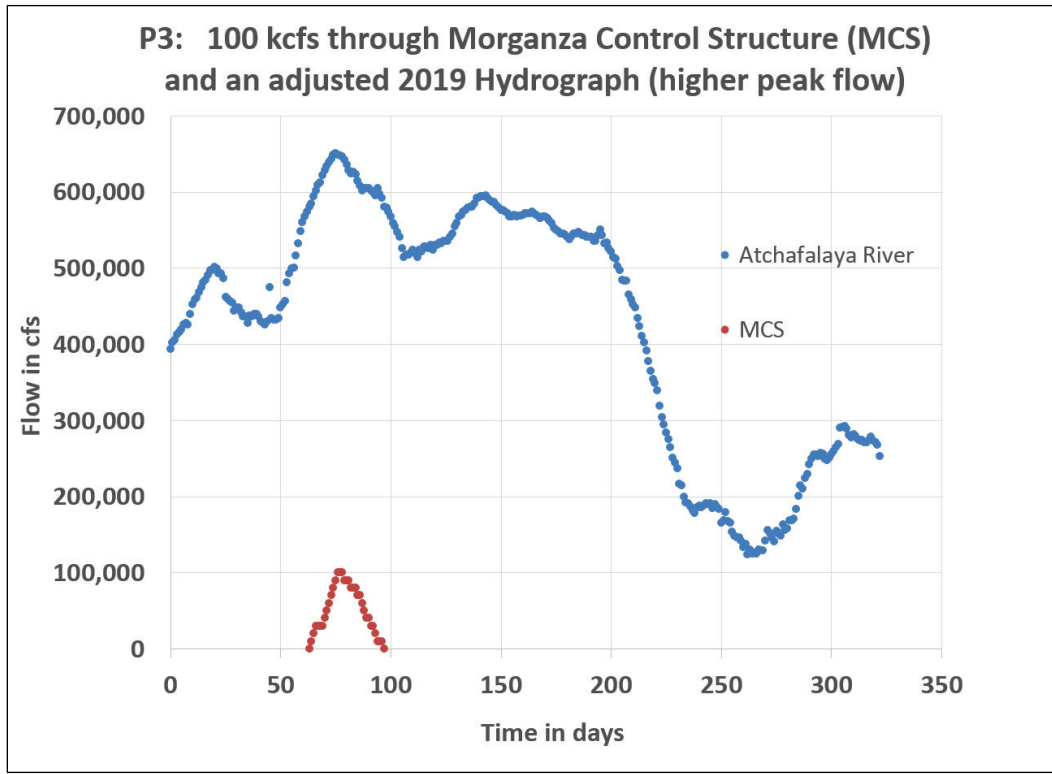
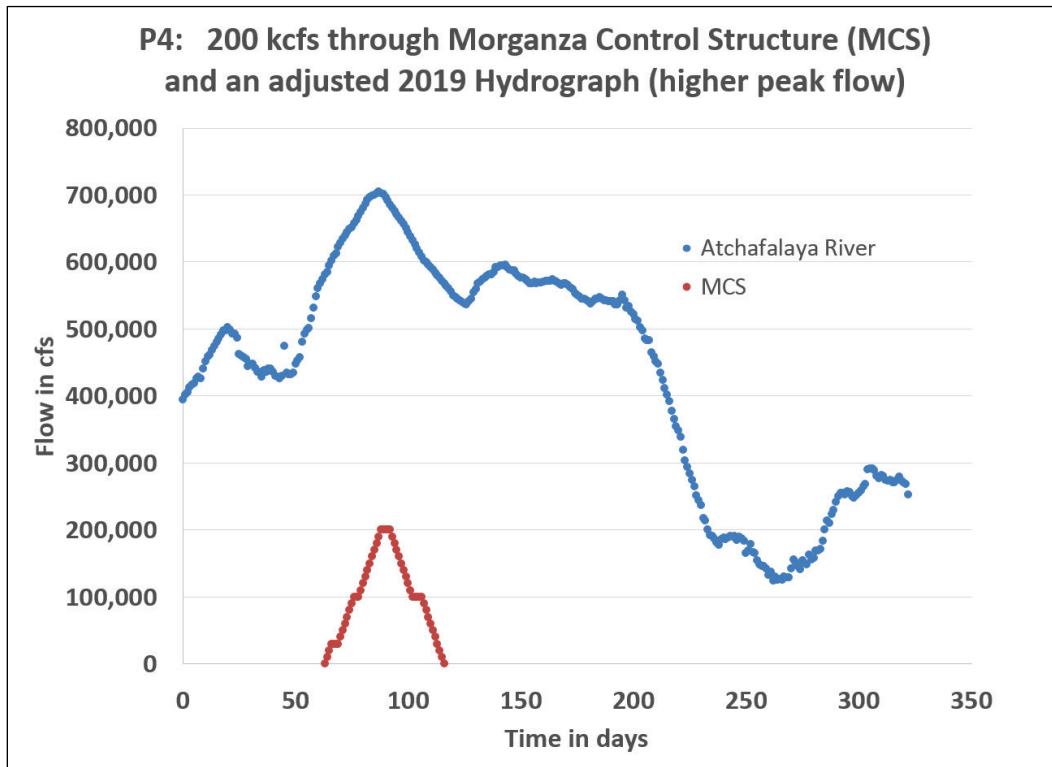


Figure 63. P4 inflow rates.



Maximum depth plots for each of the simulations can be seen in Figure 64 – Figure 67. As in the AdjustVeg plots in the previous task, it is important to remember that nothing in these figures indicates when the water reached that maximum depth.

Figure 64. Maximum depth for P1 simulation.

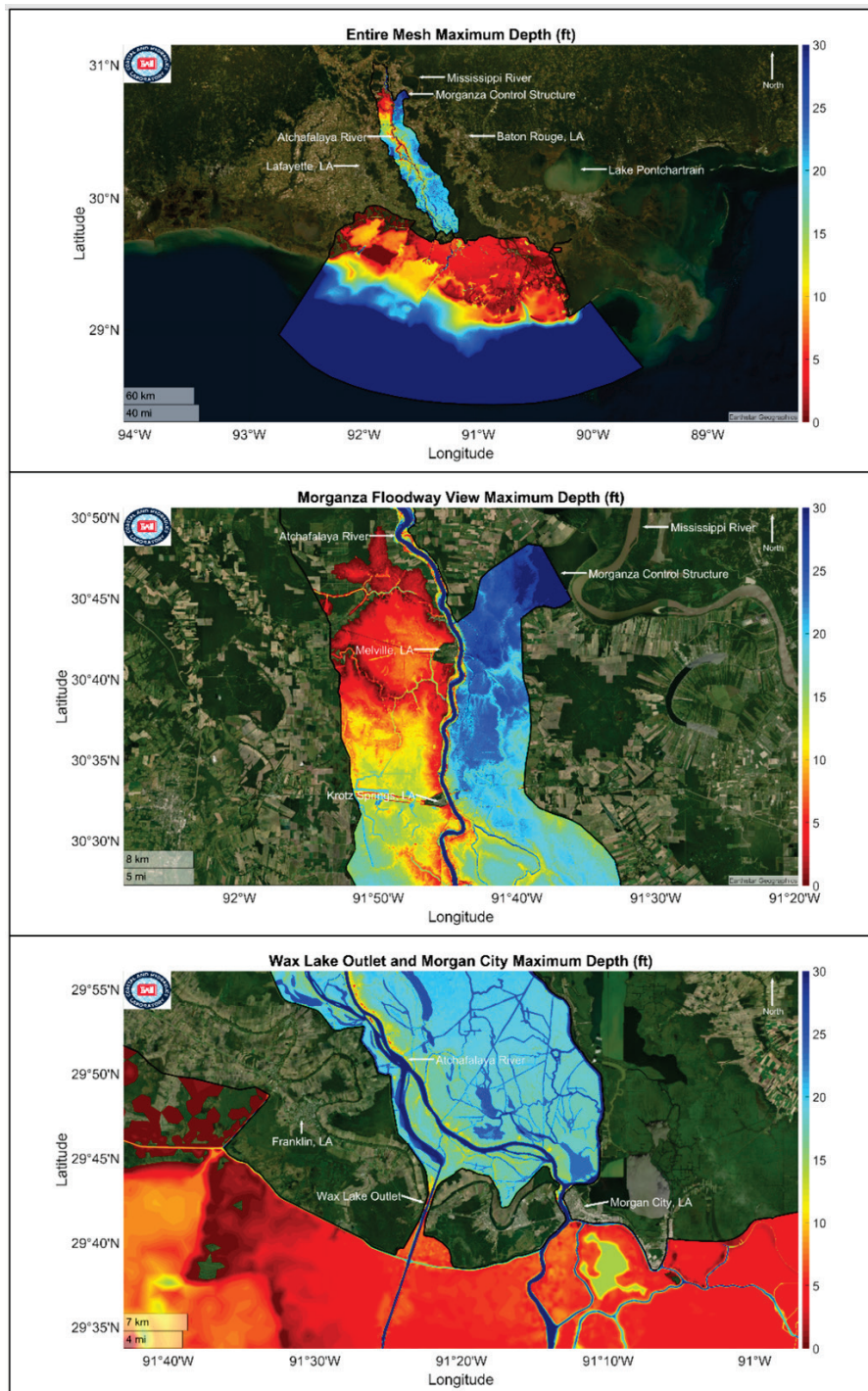


Figure 65. Maximum depth for P2 simulation.

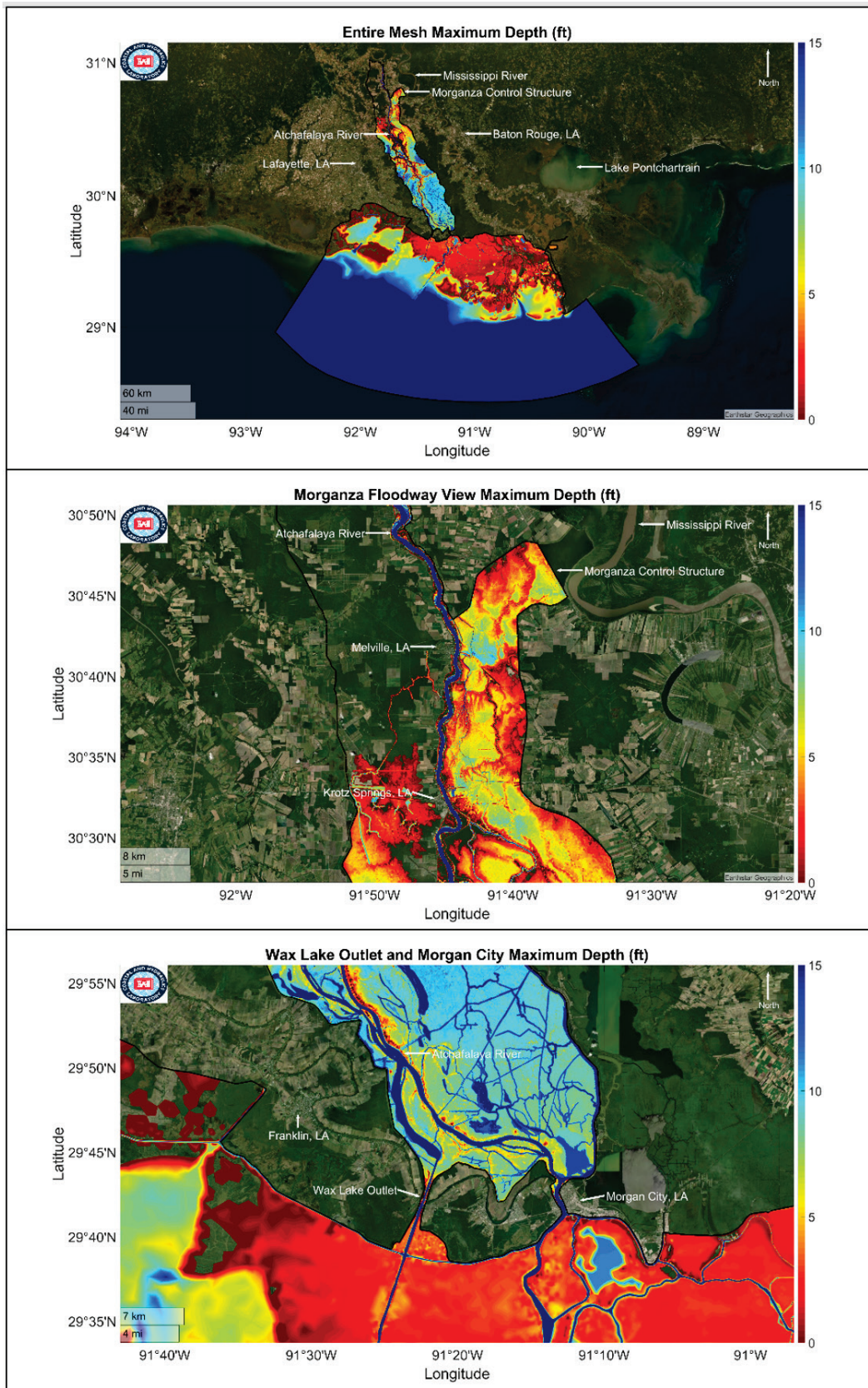


Figure 66. Maximum depth for P3 simulation.

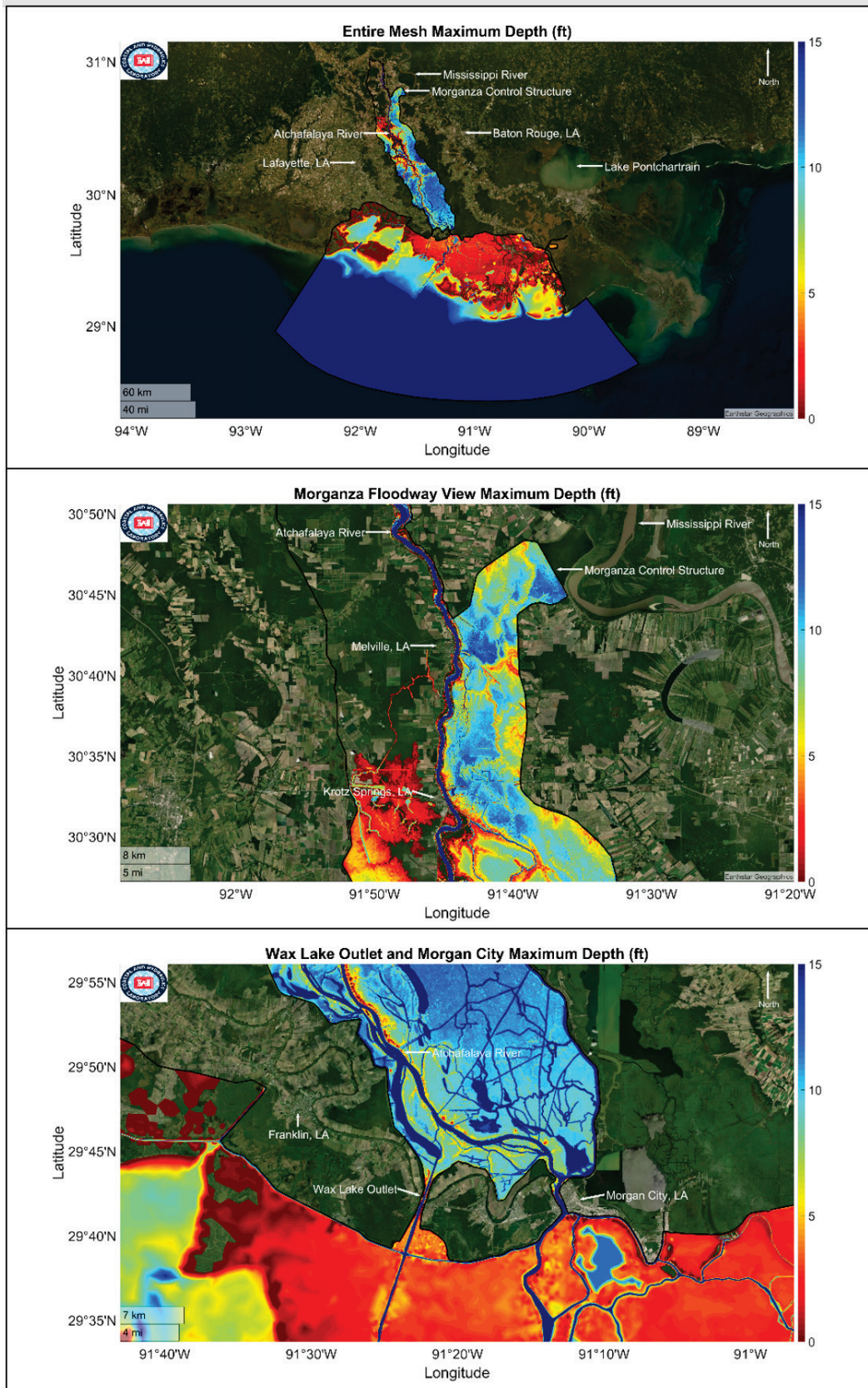
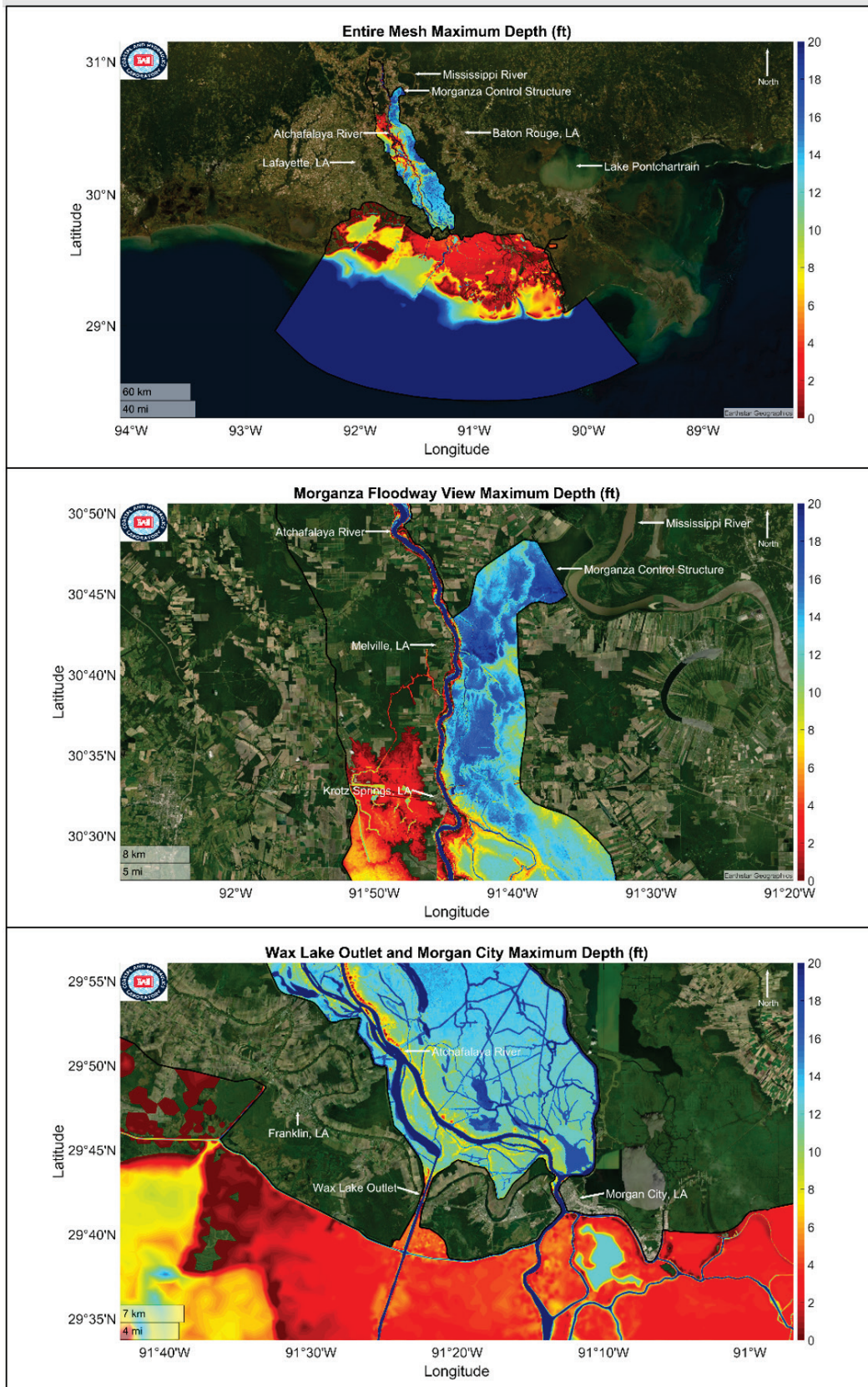


Figure 67. Maximum depth for P4 simulation.



For each of the simulations, inundation maps were created that capture the start of the rising limb of the hydrograph for the MCS as well as the end of the falling limb of the MCS hydrograph. These maps differ from the “Maximum Depth” maps above in that they do indicate the time corresponding to the inundation levels shown in the map, which can be correlated to the flowrate in the model at that time. They can be seen in Figure 68 – Figure 75. The series of plots for each run differ slightly in what days are shown based on the input hydrographs. In general, the goal was to capture both the rising and falling limb of the MCS hydrographs in the inundation maps.

Figure 68. P1 inundation maps for the Morganza Floodway (day 40 upper left, day 50 upper middle, day 60 upper right, day 70 lower left, day 80 lower middle, and day 90 lower right).

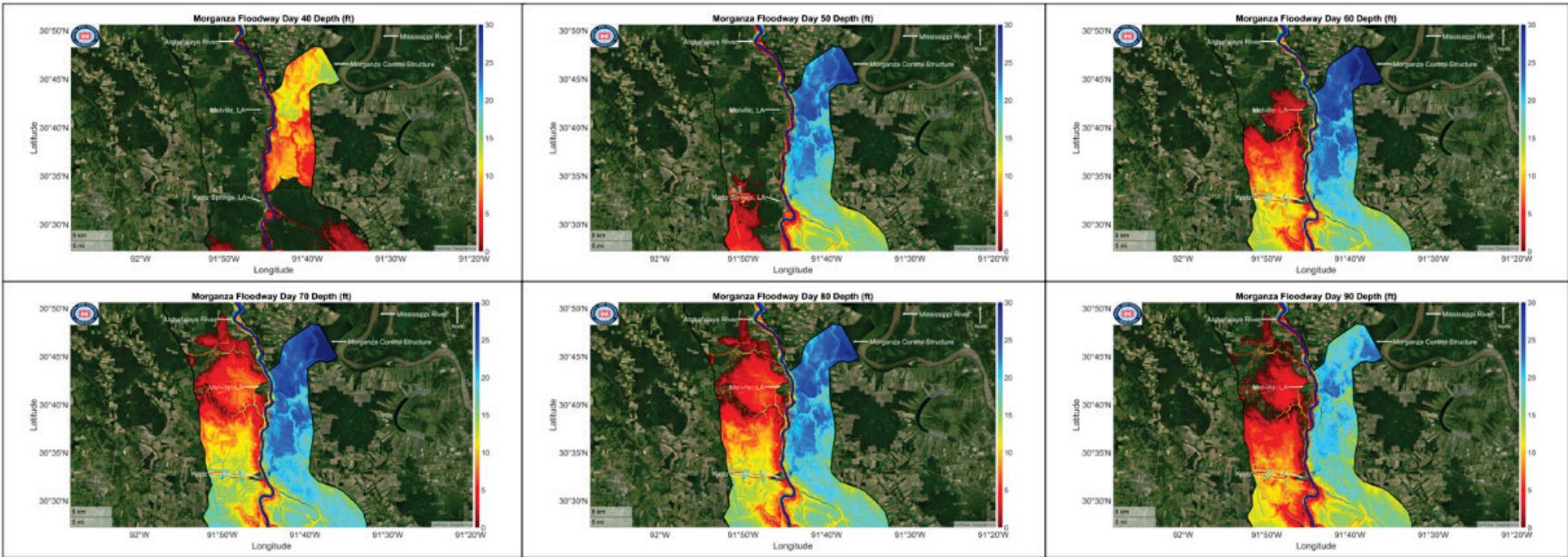


Figure 69. P1 inundation maps for Morgan City and Calumet in Wax Lake Outlet Channel (day 40 upper left, day 50 upper middle, day 60 upper right, day 70 lower left, day 80 lower middle, and day 90 lower right).

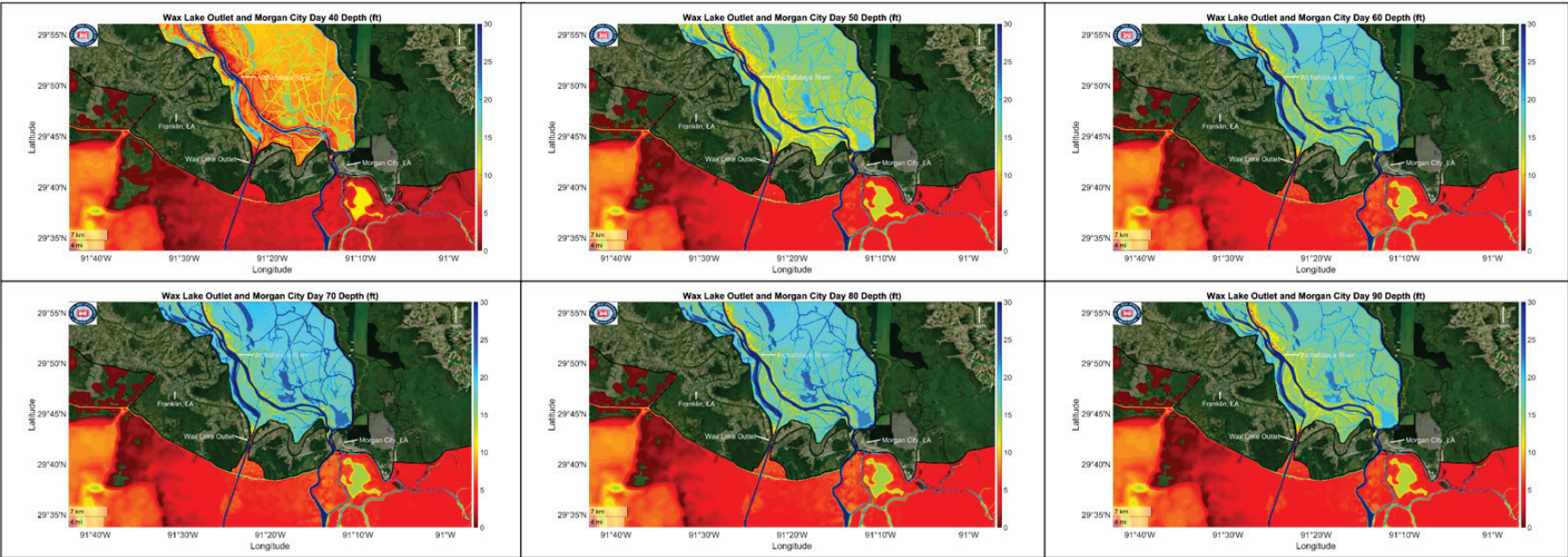


Figure 70. P2 inundation maps for the Morganza Floodway (day 67 top, day 80 middle, and day 90 bottom).

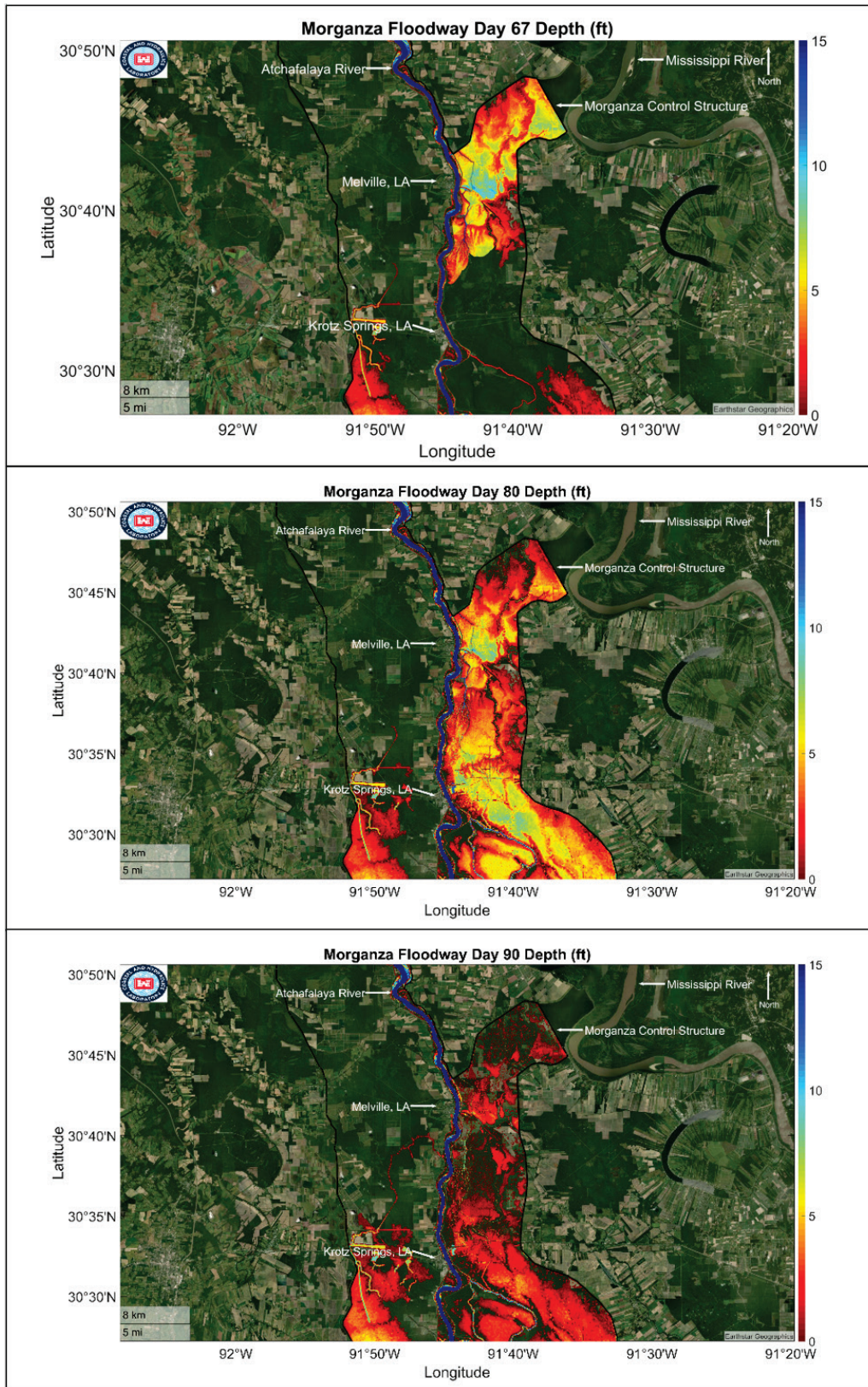


Figure 71. P2 inundation maps for Morgan City and Calumet in Wax Lake Outlet Channel (day 60 upper left, day 70 upper middle, day 80 upper right, day 90 lower left, day 100 lower middle, and day 110 lower right).

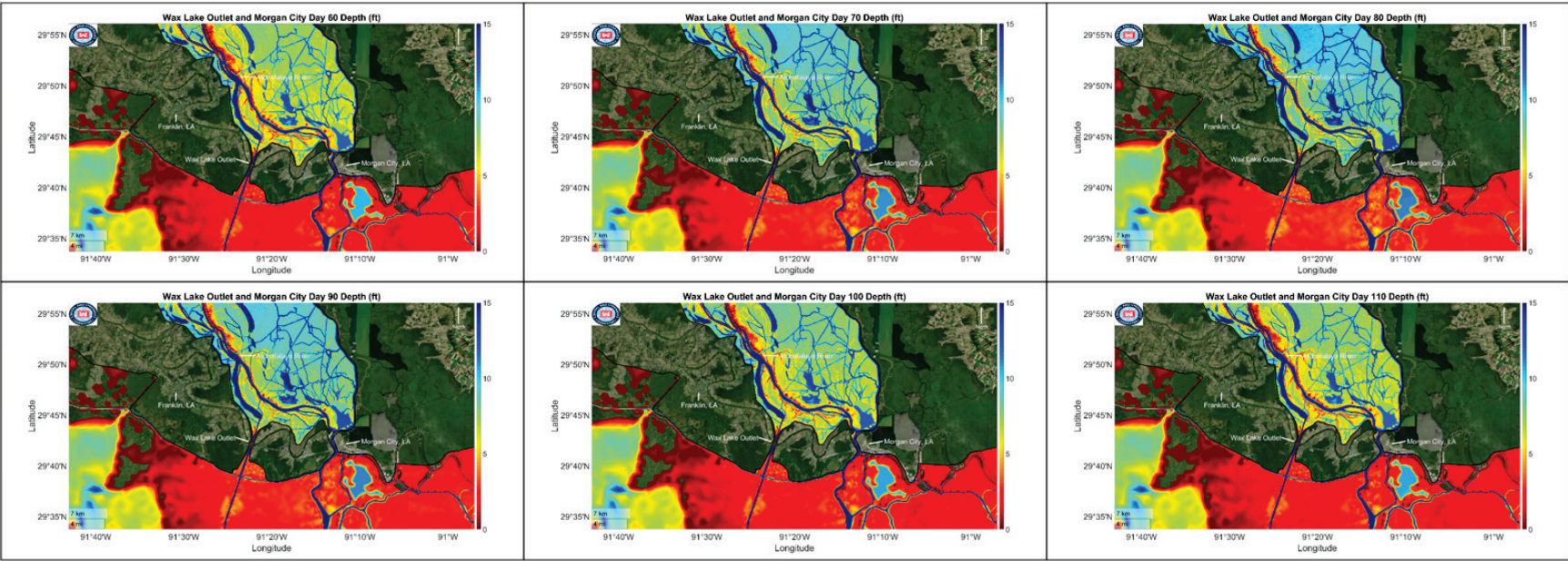


Figure 72. P3 inundation maps for the Morganza Floodway (day 60 top left, day 70 top middle, day 80 top right, day 90 lower left, day 100 lower middle, and day 110 lower right).

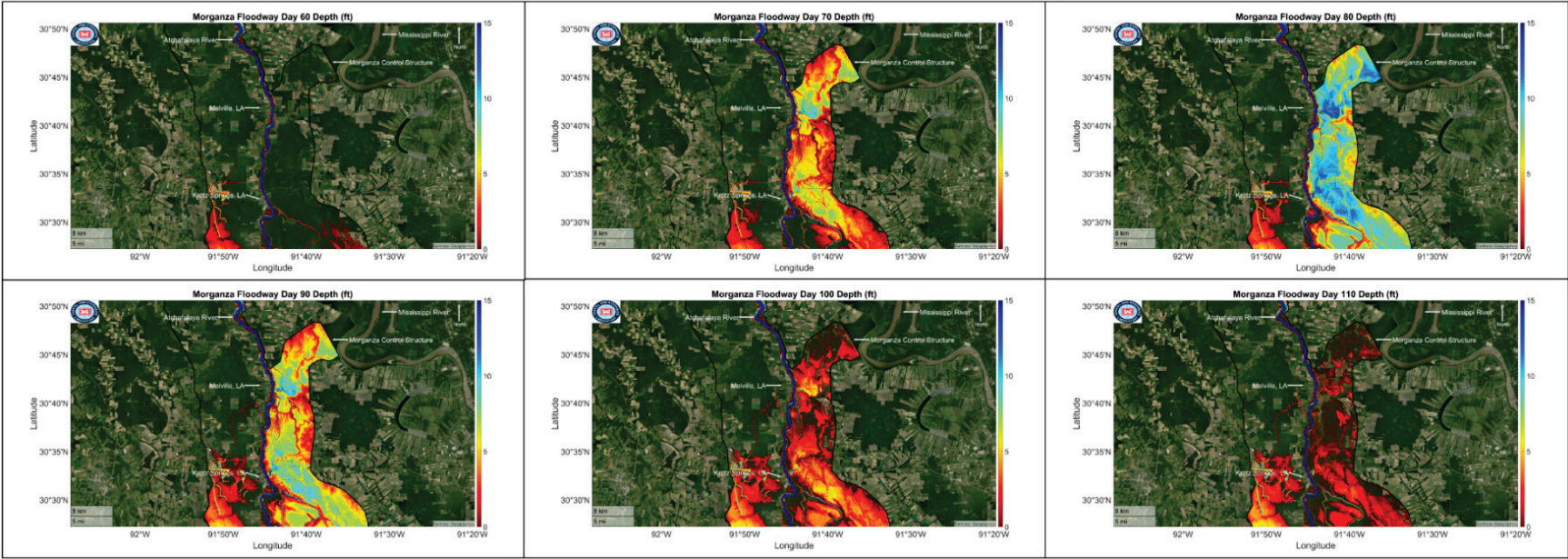


Figure 73. P3 inundation maps for Morgan City and Calumet in Wax Lake Outlet Channel (day 70 upper left, day 80 upper middle, day 90 upper right, day 100 lower left, day 110 lower middle, and day 120 lower right).

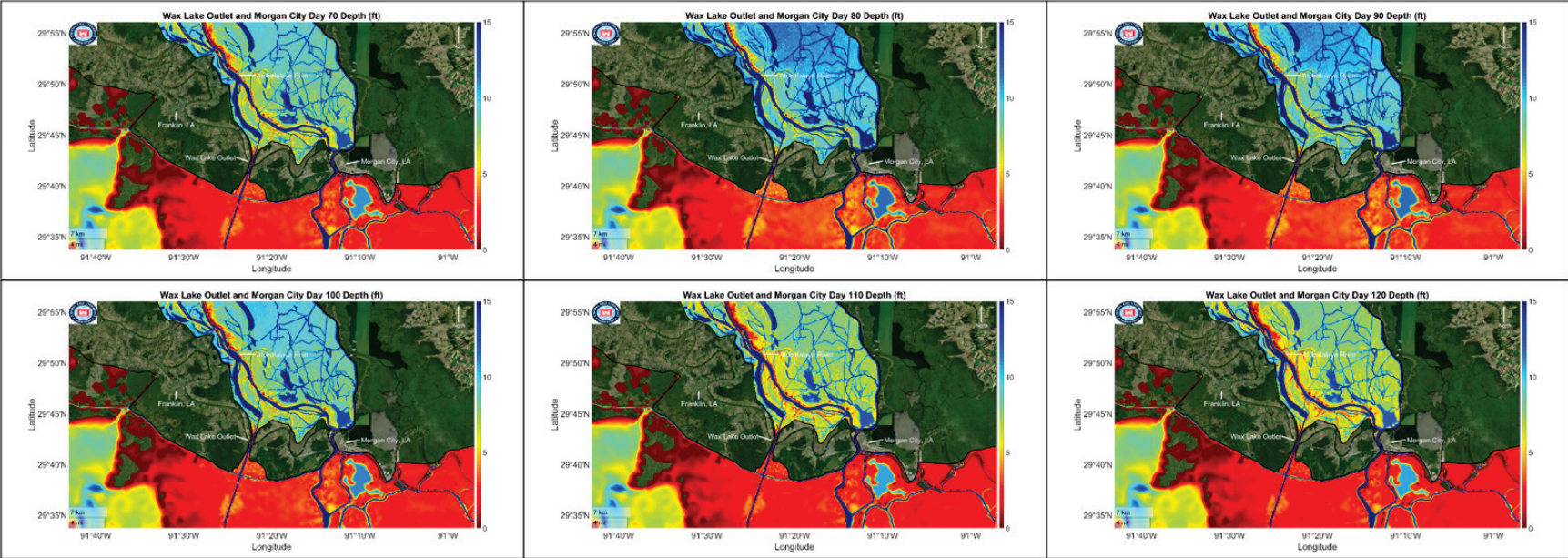


Figure 74. P4 inundation maps for the Morganza Floodway (day 60 upper left, day 70 upper middle, day 80 upper right, day 90 lower left, day 100 lower middle, and day 110 lower right).

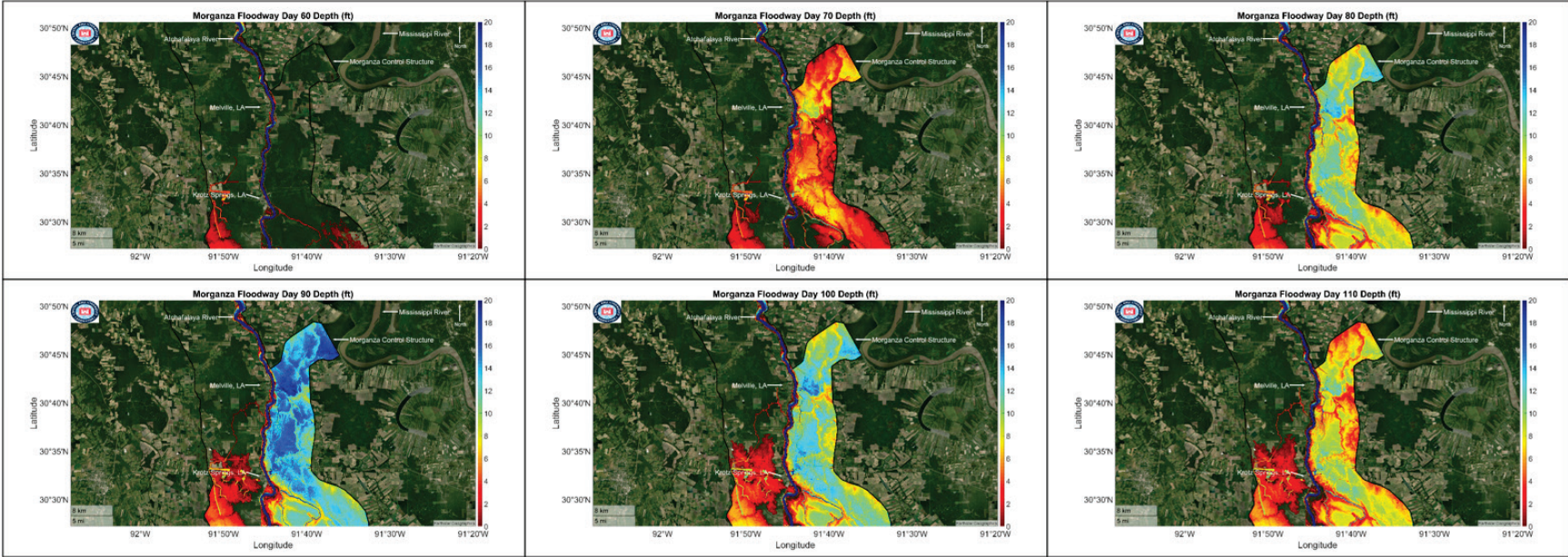
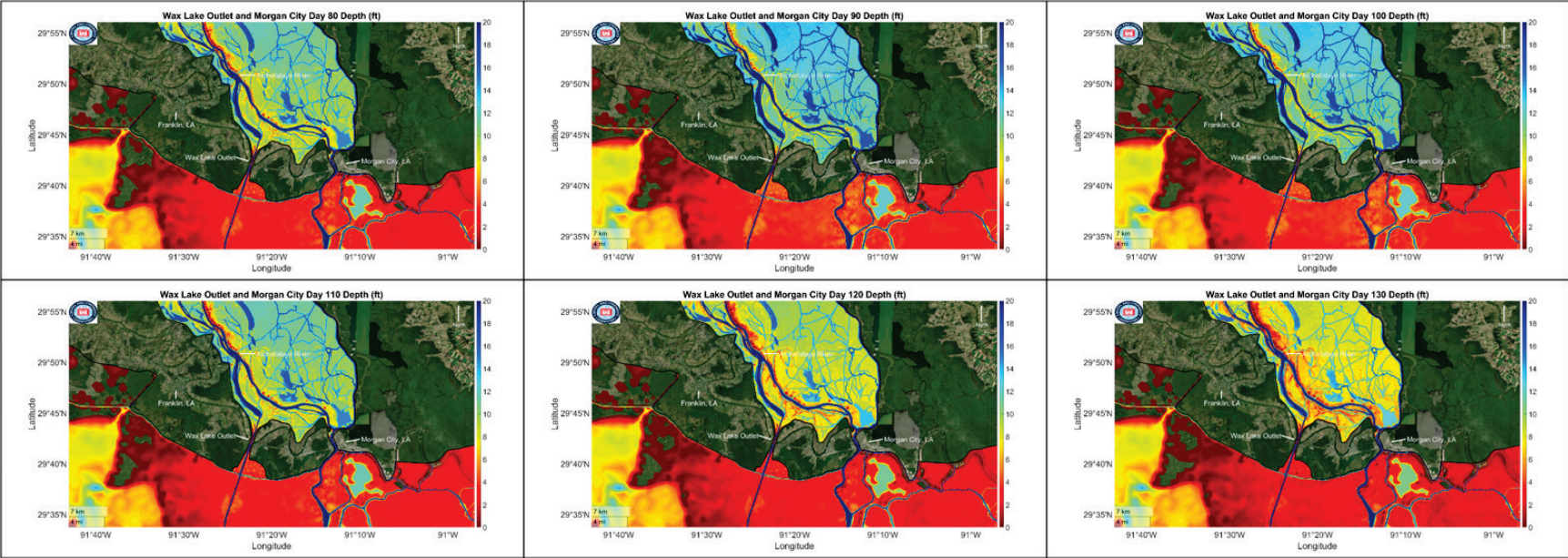
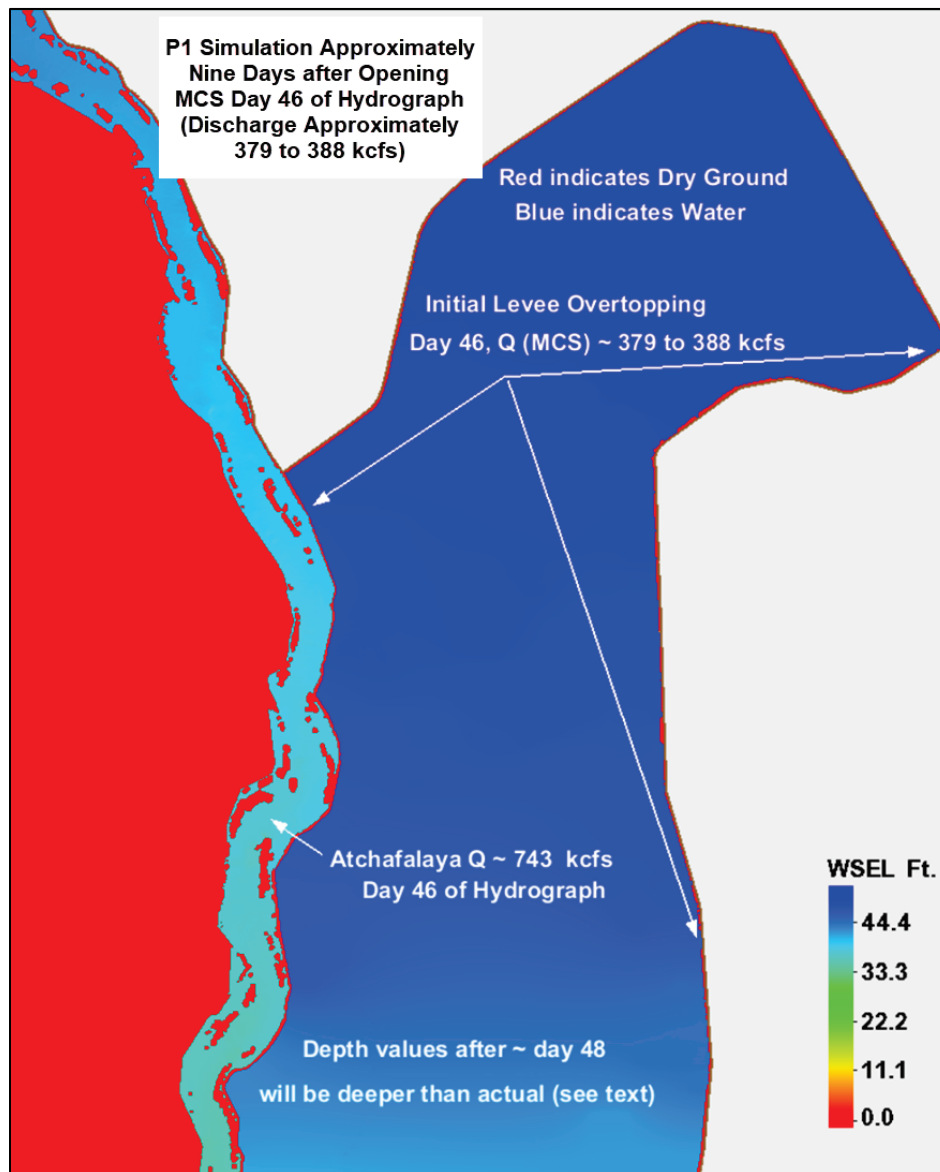


Figure 75. P4 inundation maps for Morgan City and Calumet in Wax Lake Outlet Channel (day 80 upper left, day 90 upper middle, day 100 upper right, day 110 lower left, day 120 lower middle, and day 130 lower right).



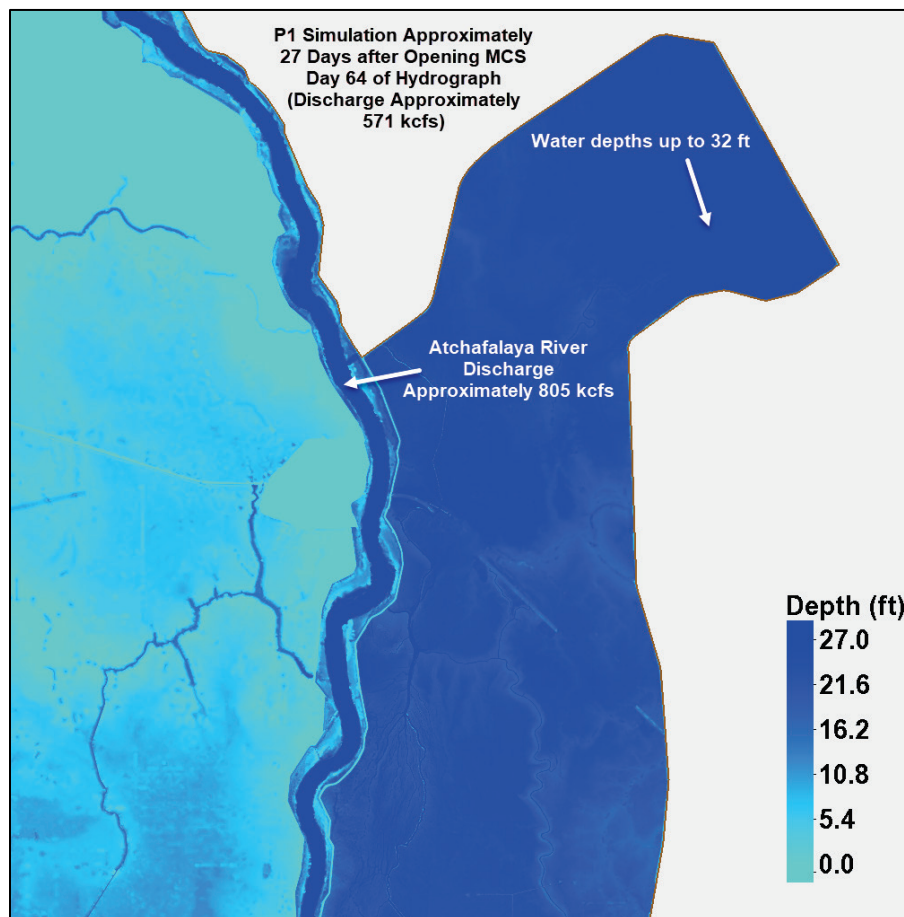
The following figures show snapshots of the applied hydrologic conditions (hydrograph) for each simulation, as well as the time, date, and location of inundation for the Morganza Floodway. In scenario P1, the MCS was opened on day 37 and increased in flow as shown in Figure 60. Overtopping of the levees appeared to begin on approximately day 46, as shown in Figure 76, which was 9 days after opening the MCS gates. The flow through the MCS at this time varied from 379,000 cfs to 388,000 cfs. This is very similar to the magnitude of flow showing overtopping at Arc 1 in the results of the previous section.

Figure 76. Day 46 of P1 inundation plot (WSE values shown in feet).



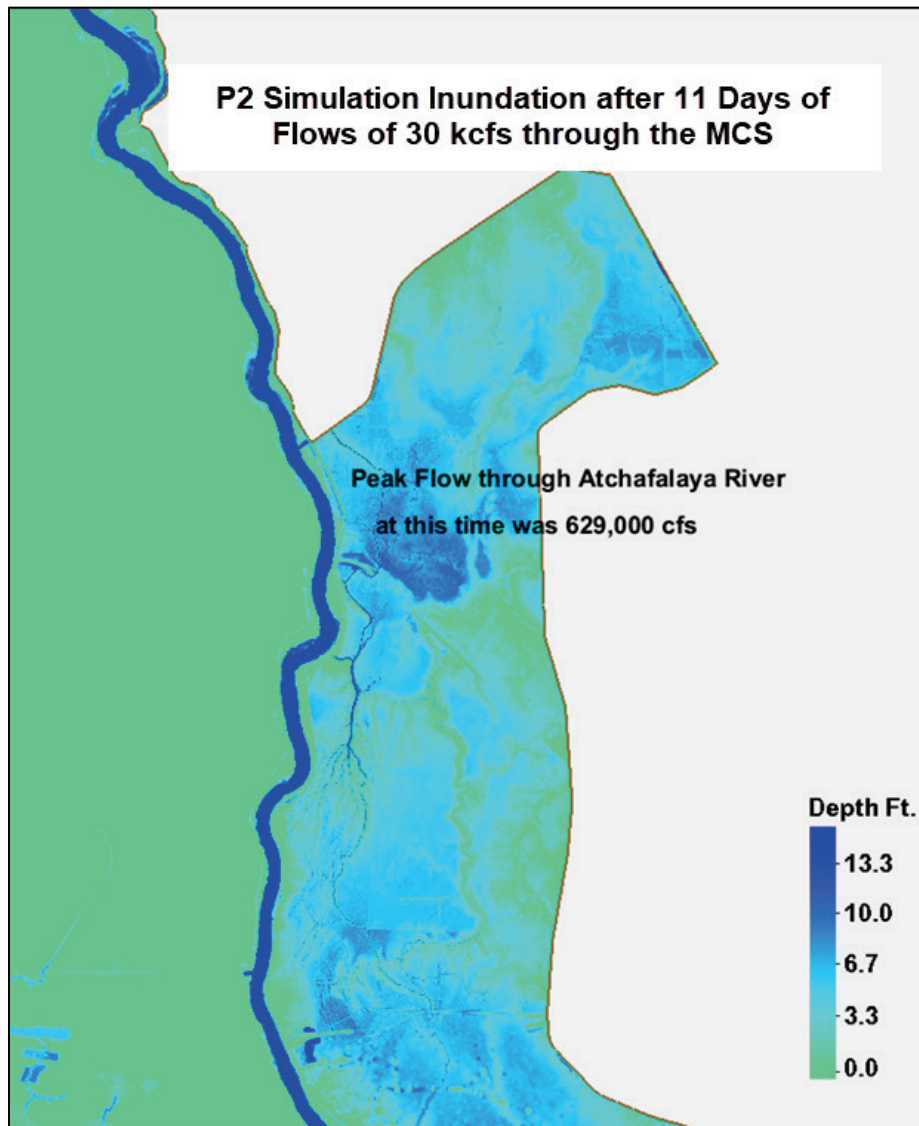
Review of the AdH simulation of P1 showed reasonable results throughout the domain, as shown below in Figure 77. The peak flow through the MCS was 571,000 cfs and occurred on day 64. The simulated flow in the Atchafalaya River was 805,000 cfs on day 64. This was approximately 27 days after the MCS began operation. At that time, all interior surface features of the Morganza Floodway, including railroad berms, were inundated by at least 9 to 12 ft of water. In most locations, the water was much deeper. Note that WSE values shown after approximately day 48 of this hypothetical simulation would be higher than actual values because the model does not spill water over the exterior levee crests. Therefore, more water is retained in the floodway thus increasing flow and depth values. At very low overtopping depths this is probably not a problem, but as flow depths over the levee crests get larger, the differences would also become larger. A modification to the model would be required to account for that fact.

Figure 77. Day 64 of P1 inundation plot.



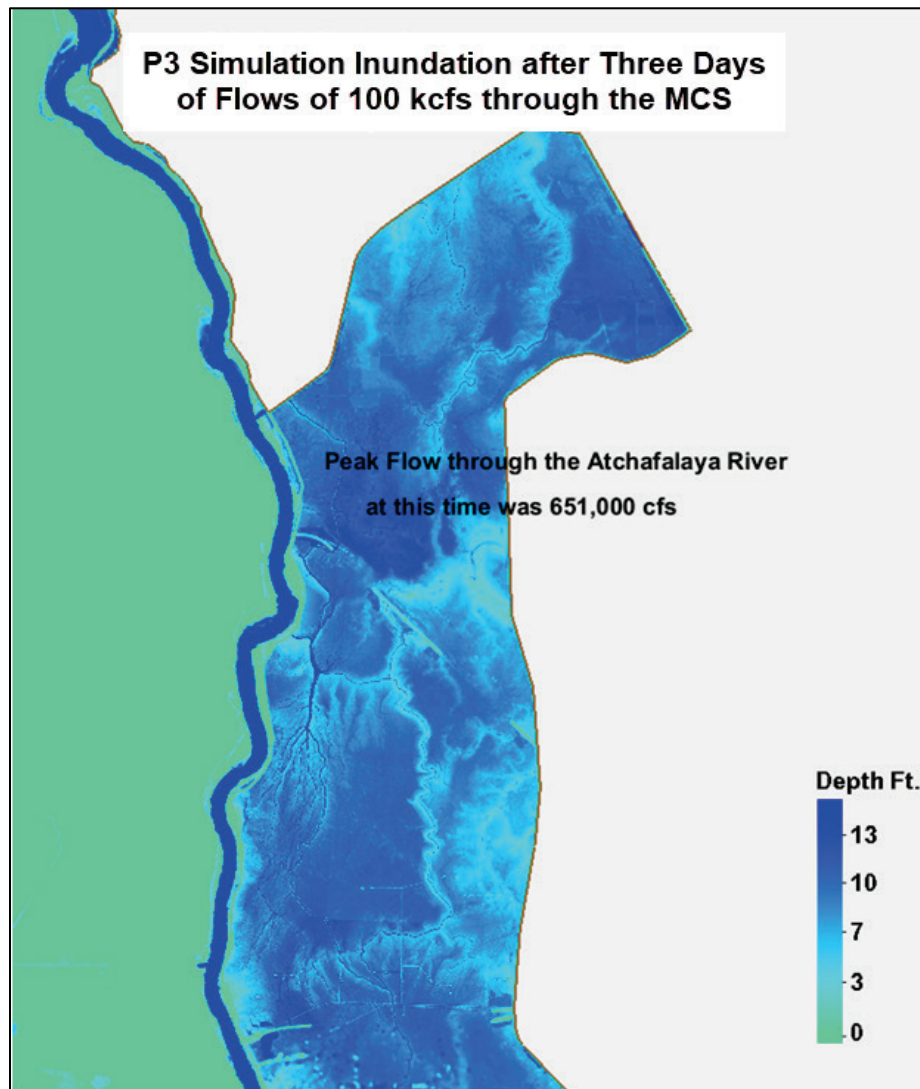
The P2 simulation used a hydrograph through the Atchafalaya River from January to mid-November of 2019, a period of 322 days, as shown in Figure 61. During this time the MCS was opened on day 64 and increased in flow as shown in Figure 61 to 30,000 cfs by day 66. It was maintained at that flow until day 77, a period of 11 days of continuous flow at 30,000 cfs. It was closed on day 80 resulting in a total time of MCS operation of 16 days. Figure 78 below shows that, in general, water depths throughout the Morganza Floodway were not greater than 15 ft in the channels and between 5 to 10 ft in other low-lying areas. There was no levee overtopping in this simulation.

Figure 78. Day 77 of P2 inundation map.



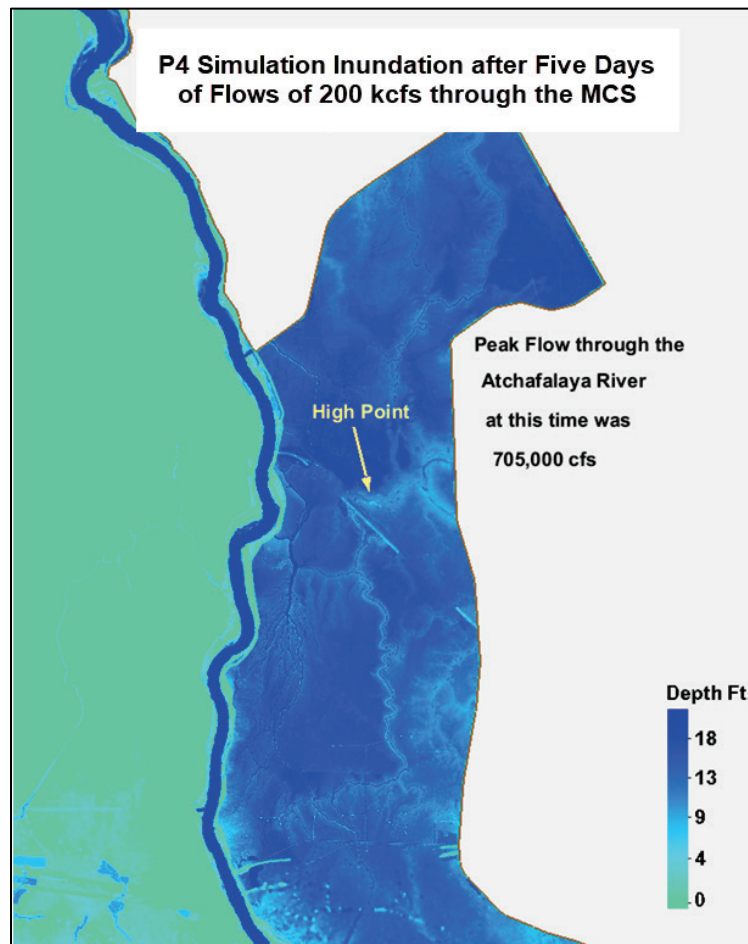
The P3 simulation (hydrograph shown in Figure 62) modeled the Atchafalaya River from January to mid-November of 2019, a period of 322 days. During this time, the MCS was opened on day 64 and increased to a peak flow of 100,000 cfs for a period of 3 days (day 76 to 78) and then reduced in flow to 0 cfs by day 97. The total time of operation for the MCS was 33 days. The peak flow in the Atchafalaya River was adjusted to 651,000 cfs and occurred on day 75, which was the day before the peak flows were sent through the MCS. In Figure 79 the water depths throughout the Morganza Floodway, in general, were not greater than 19 ft in the channels and between 9 to 14 ft in other low-lying areas. There was no levee overtopping in this simulation.

Figure 79. Day 78 of P3 inundation map.



The P4 simulation deployed the hydrograph shown in Figure 63. The Atchafalaya River flowed from January to mid-November of 2019 for a period of 322 days. During this time, the MCS was opened on day 64 and increased to a peak flow of 200,000 cfs for a period of 5 days (day 88 to 92) and then decreased in flow to 0 cfs by day 116. Total time of MCS operation was 53 days. The peak flow in the Atchafalaya River was adjusted to 705,000 cfs and occurred on day 87, which was the day before the peak flow was sent through the MCS. In Figure 80 below, water depths generally were not greater than approximately 26 ft in the channels and approximately 15 – 21 ft in other low-lying areas. The highest topographic elevation in the floodway other than the levees, transportation berms (for roads and rails), drilling platforms, and other manmade features, was a high point in the middle of the floodway just north of the railroad berm. It has an elevation of approximately 35 ft and an inundation depth of slightly more than 5 ft on day 92 of the simulation. There was no levee overtopping in this simulation.

Figure 80. Day 92 of P4 inundation map.



## **4 Conclusions and Recommendations**

### **4.1 Conclusions**

Following the completion of Tasks 1 and 2, it was found that the model still validated well to the 2011 flood event. A sensitivity analysis was conducted of the flow capacity of the Morganza Floodway to hypothetical variations in vegetation roughness. These adjustments in roughness will improve the understanding of the role of land cover characteristics in the simulated water surfaces. It was found that with decreased roughness (more open land in the Morganza Floodway), the flow capacity increased, and inundation water levels decreased. The most extreme simulation, which effectively converted roughness to that of open land throughout the Morganza Floodway from the MCS to Highway 190, resulted in no Morganza Floodway levees in danger of overtopping during the PDF flow of 600,000 cfs through the MCS.

The simulations of interest (Task 4) were conducted as suggested by MVD and MVN. The base model resulting from the efforts of Tasks 1 and 2 was used for all simulations (i.e., no roughnesses were adjusted within the Morganza Floodway), so the maps show inundation levels for the present condition of the floodway. Input for the simulation boundary inflow consisted of the application of various hypothetical hydrologic events, such as the SPF. Each of the simulations were selected to represent some variation of a flooding scenario. Output from the various model runs produced a series of inundation maps that can be used to tell where and how deep the maximum flood waters are at various locations throughout the Morganza Floodway and Atchafalaya Basin. Additionally, in this report, the inundation depth maps were also produced for selected time periods to show how the depth changed with variations in the inflow hydrographs.

### **4.2 Recommendations**

Task 3 showed that adjustments to roughness within the Morganza Floodway have a significant impact on simulated WSEs. It is recommended that a Phase 4 of this study be conducted to analyze potential scenarios that could involve channel dredging or excavation, or some combination of channel dredging and roughness adjustments. These

efforts would be conducted to continue investigating the sensitivity of flood event WSEs and the conditions within the Morganza Floodway.

This study has produced a small sample of the type of inundation maps that can be produced. The model simulations stated in this report can be used to produce many more maps (on the order of thousands) if so desired. The location, view (zoomed-in or out), time, etc., characteristics to each map are virtually limitless.

## References

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- Bell, G. L., N. D. Clifton, and D. D. Abraham. 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.
- ERDC USACE (US Army Engineer Research and Development Center, US Army Corps of Engineers). 2017. *Adaptive Hydraulics 2D Shallow Water (AdH-SW2D) User Manual* (Version 4.6). Vicksburg, MS: US Army Engineer Research and Development Center.
- Maynard, 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.
- Savant, G., R. C. Berger, T. O. McAlpin, and C. J. Trahan. 2014. *Three-Dimensional Shallow-Water Adaptive Hydraulics (AdH-Sw3): Hydrodynamic Verification and Validation*. ERDC TR-14-7. Vicksburg, MS: US Army Engineer Research and Development Center.

# Appendix A: Task 2 2011 Flood Event Simulation Model Results

In Figure 81 – Figure 104, the resulting WSE at each gage for the measured Phase 2 and Phase 3 data is shown for the 2011 flood event simulation.

Figure 81. Gage A02 model results.

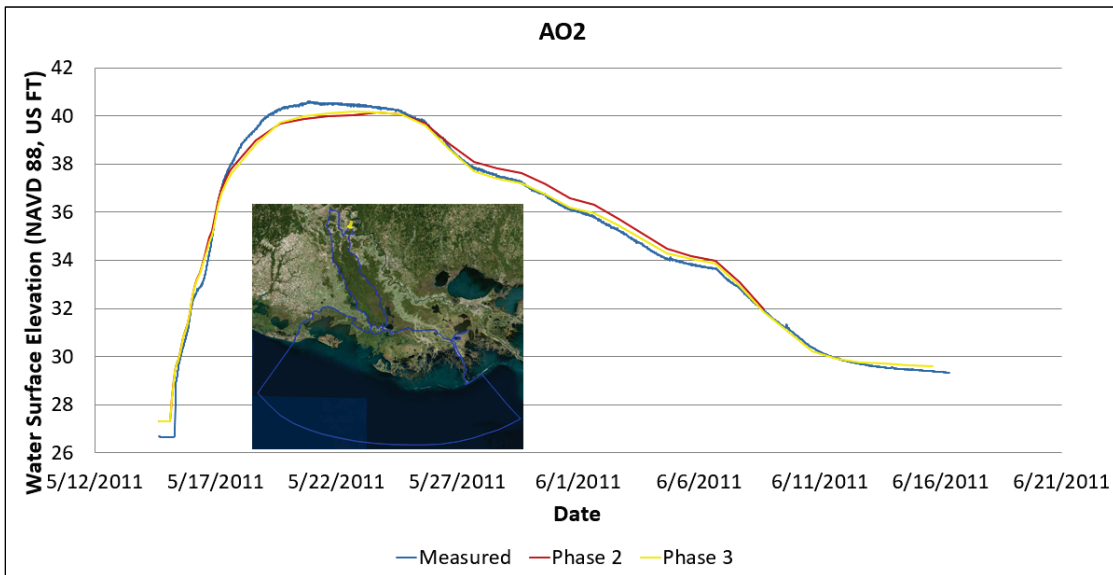


Figure 82. Gage A03 model results.

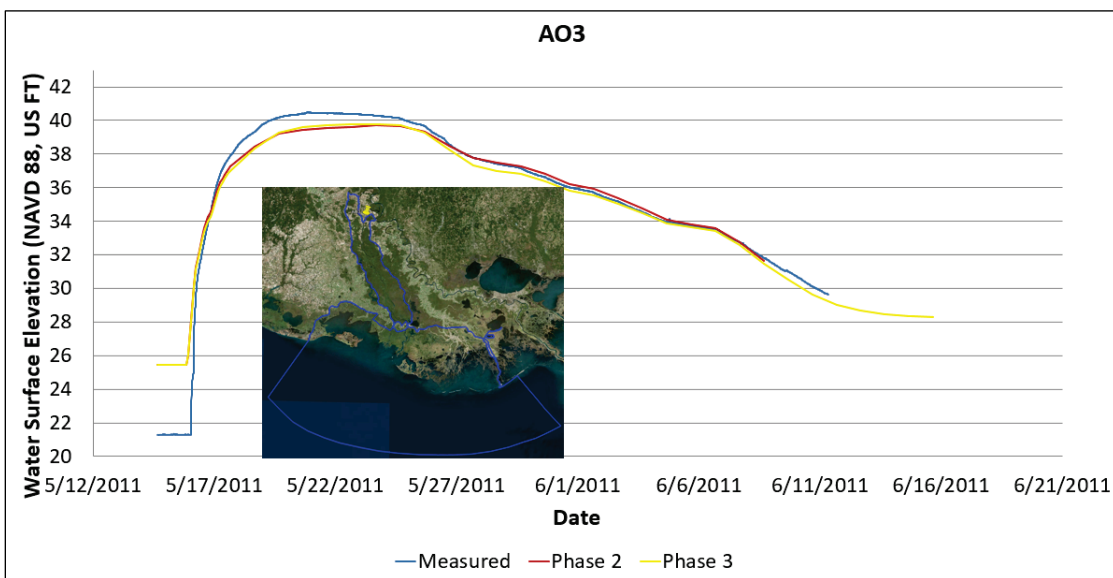


Figure 83. Gage A04 model results.

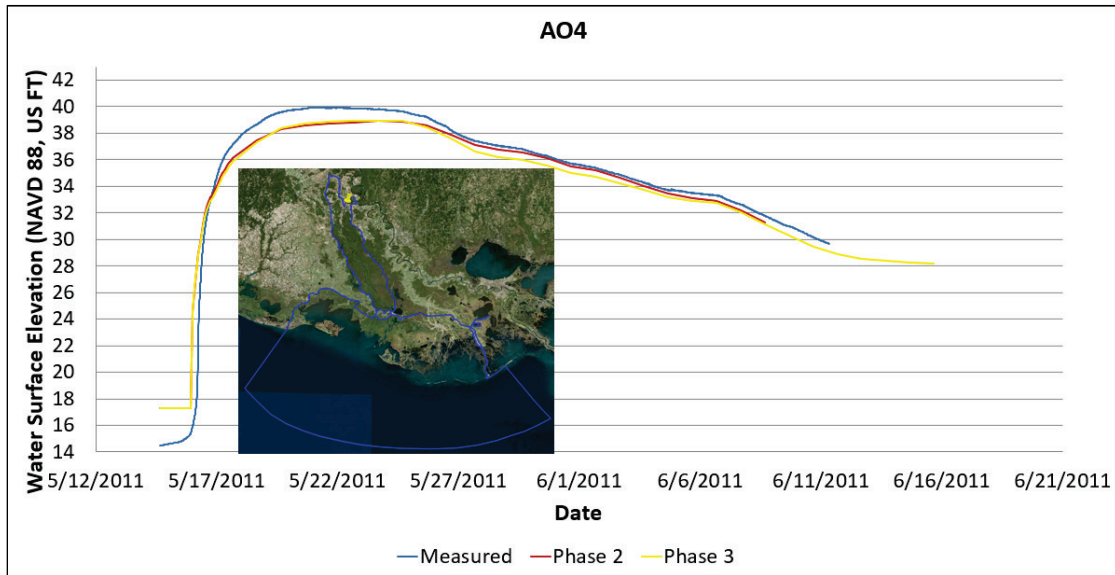


Figure 84. Gage A06 model results.

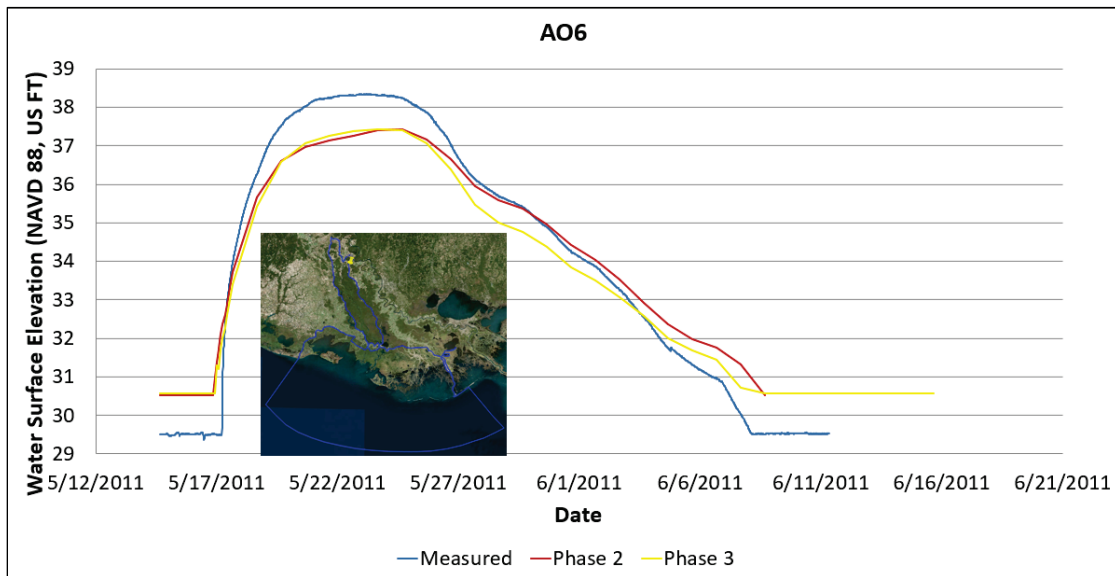


Figure 85. Gage A07 model results.

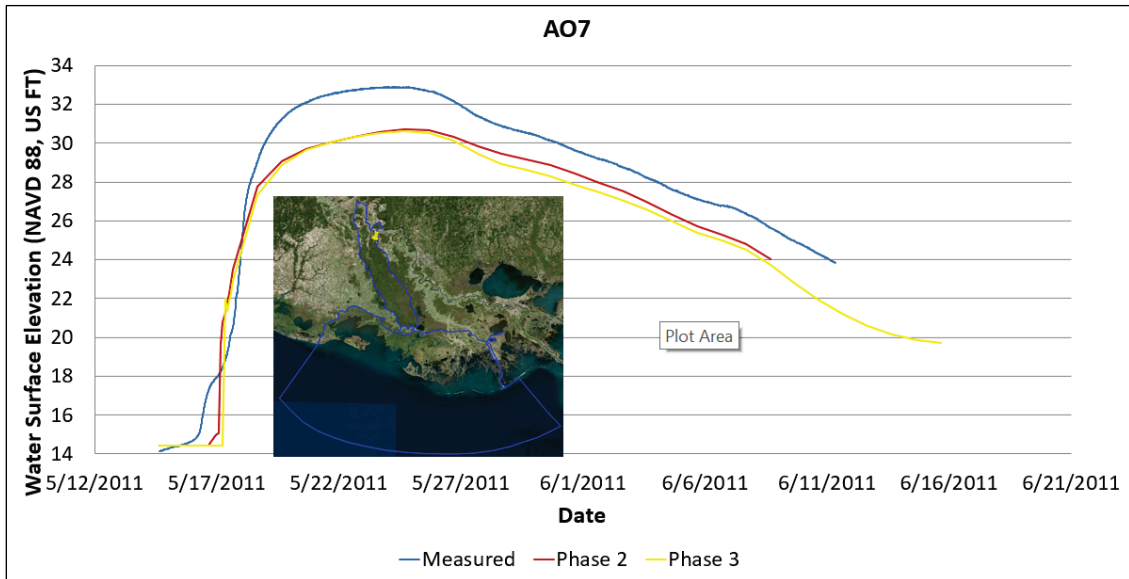


Figure 86. Gage B04 model results.

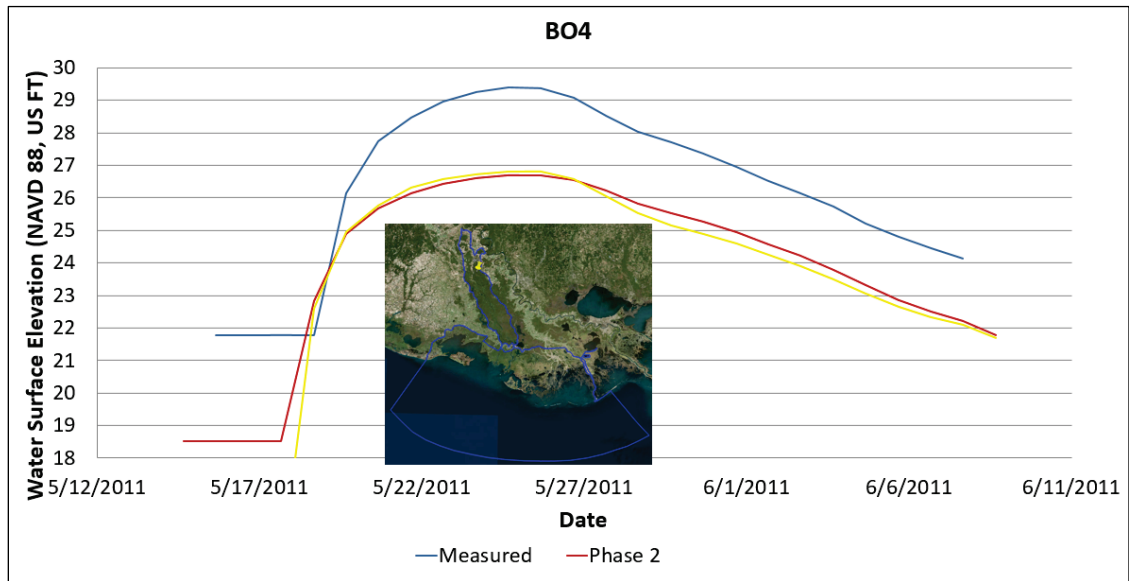


Figure 87. Gage C04 model results.

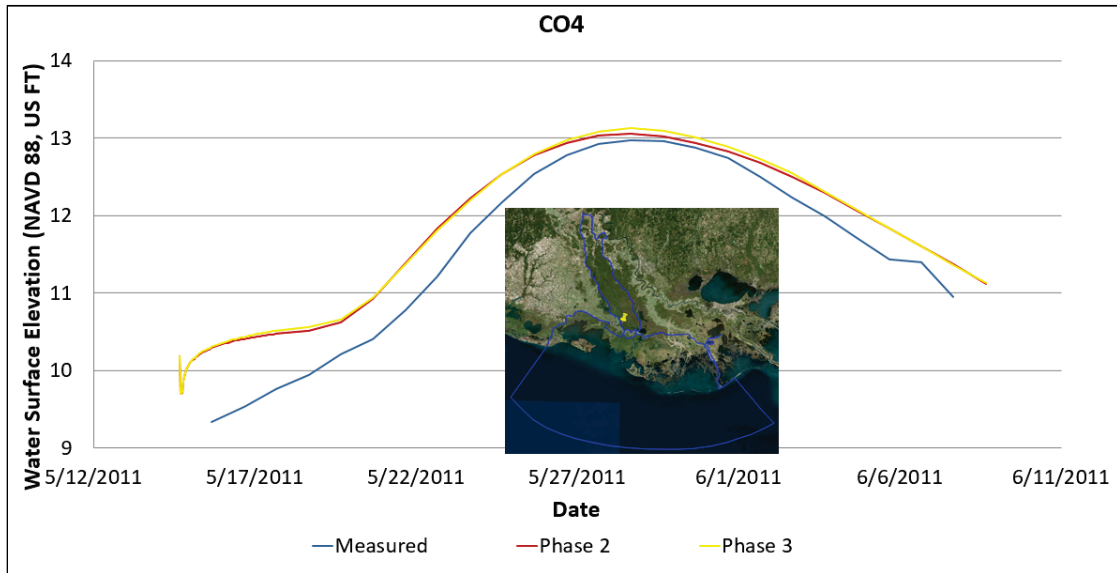


Figure 88. Gage K2 model results.

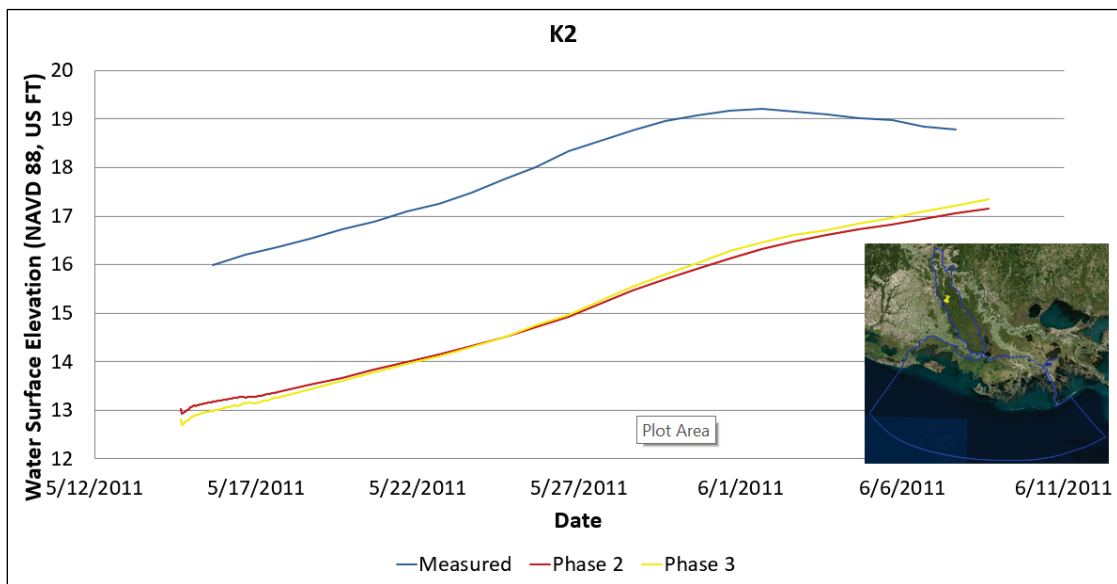


Figure 89. Gage K4 model results.

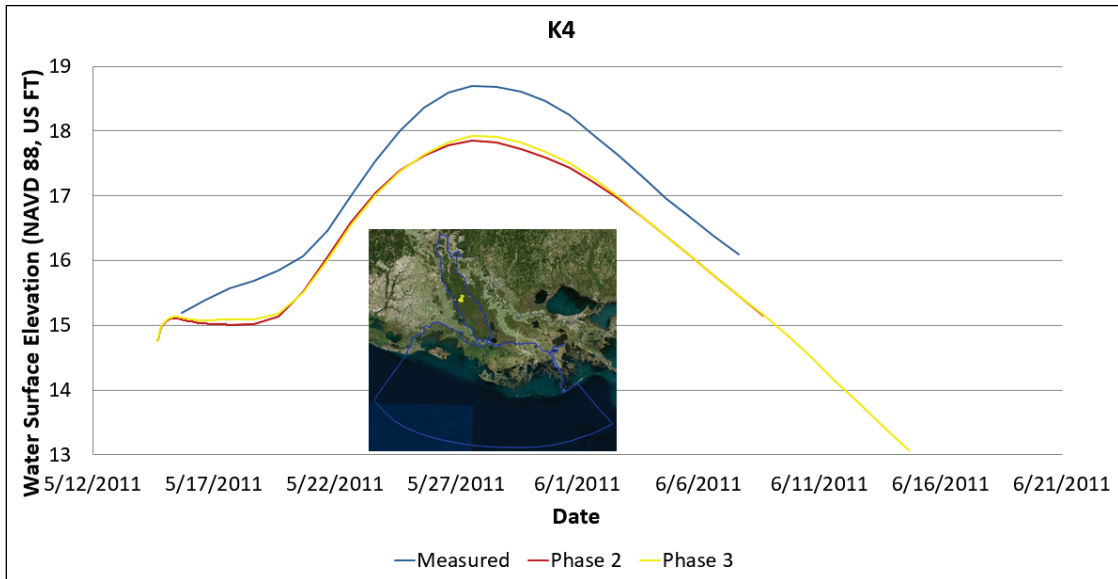


Figure 90. Gage K5 model results.

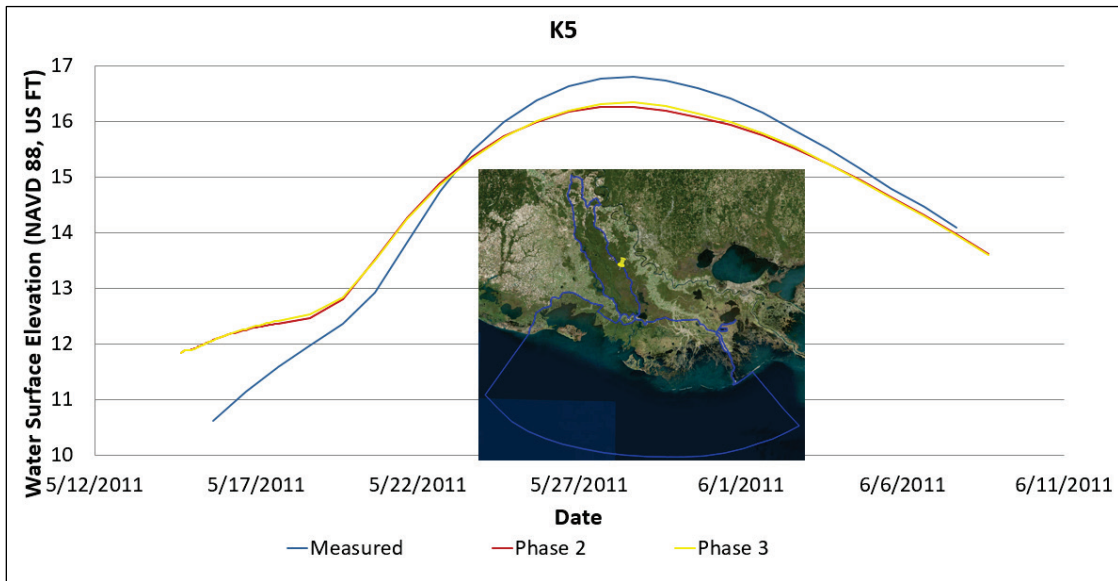


Figure 91. Gage K6 model results.

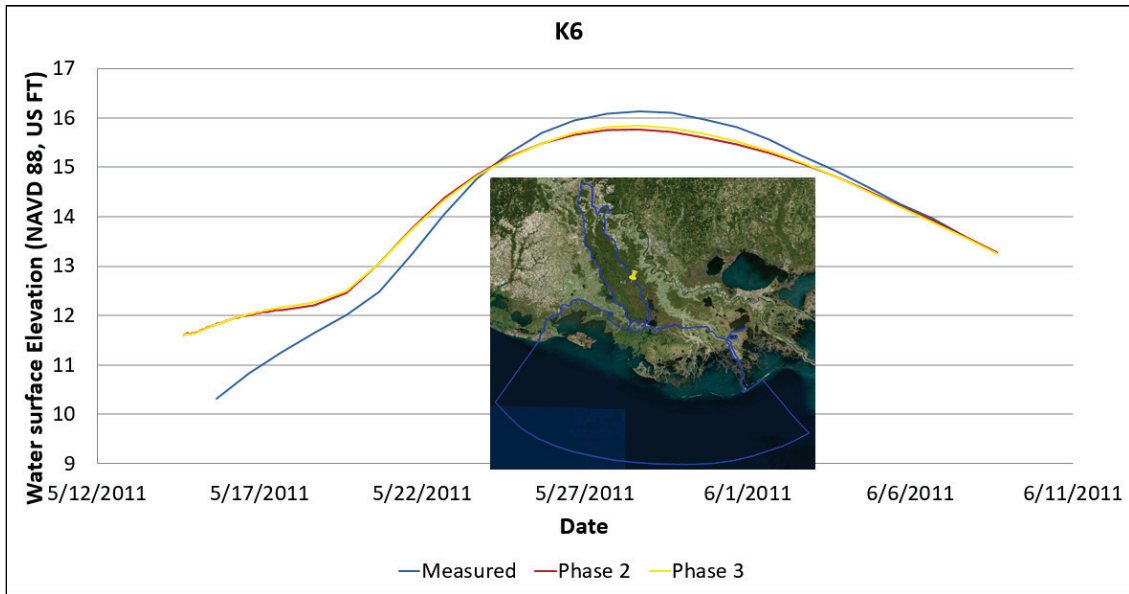


Figure 92. Gage K7 model results.

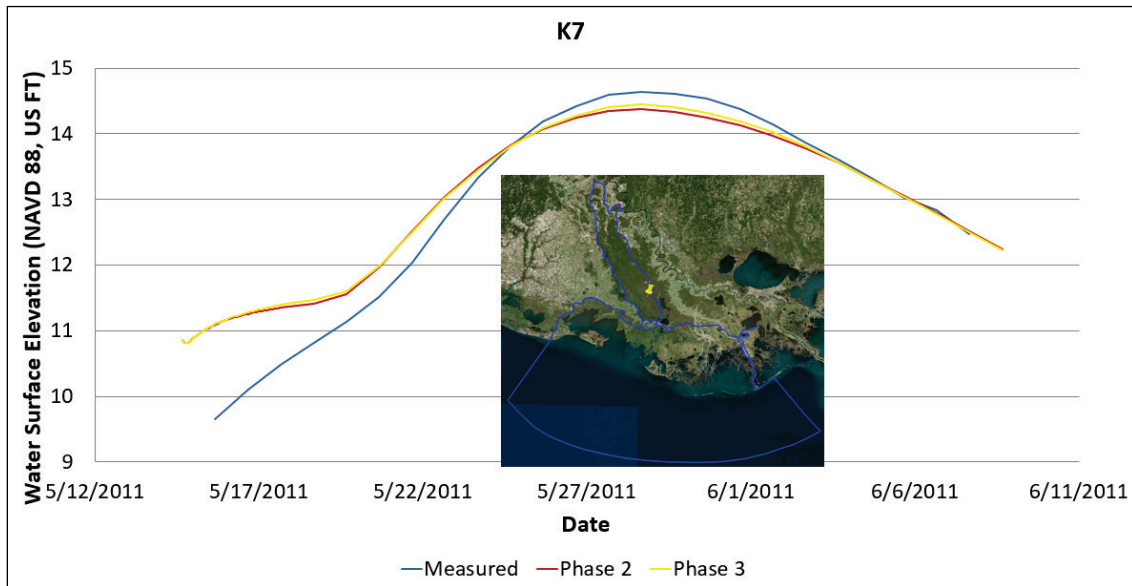


Figure 93. Gage K8 model results.

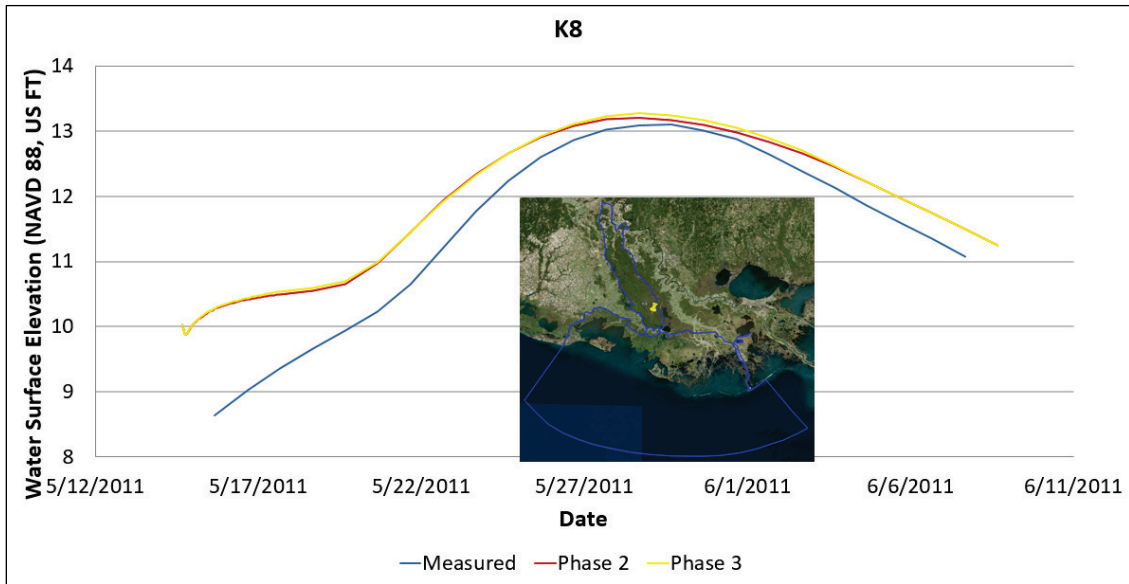


Figure 94. Gage K9 model results.

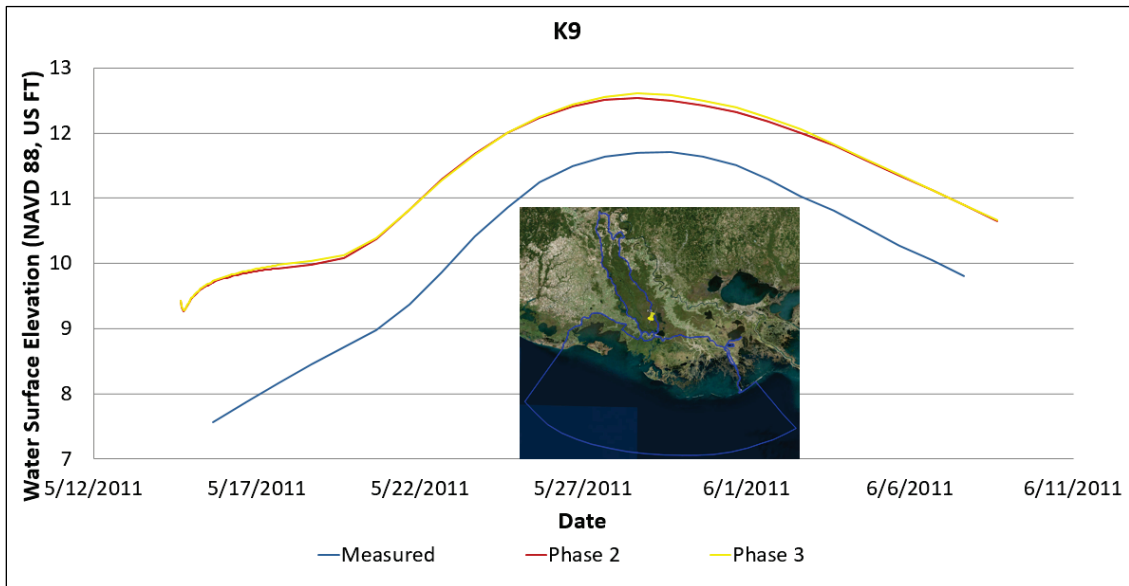


Figure 95. Gage K10 model results.

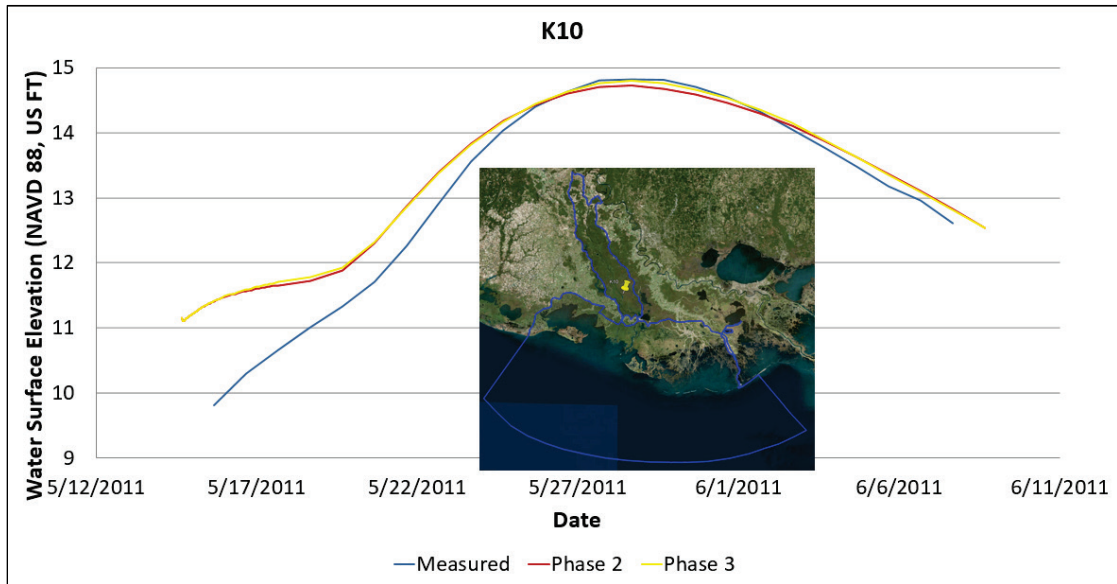


Figure 96. Gage K11 model results.

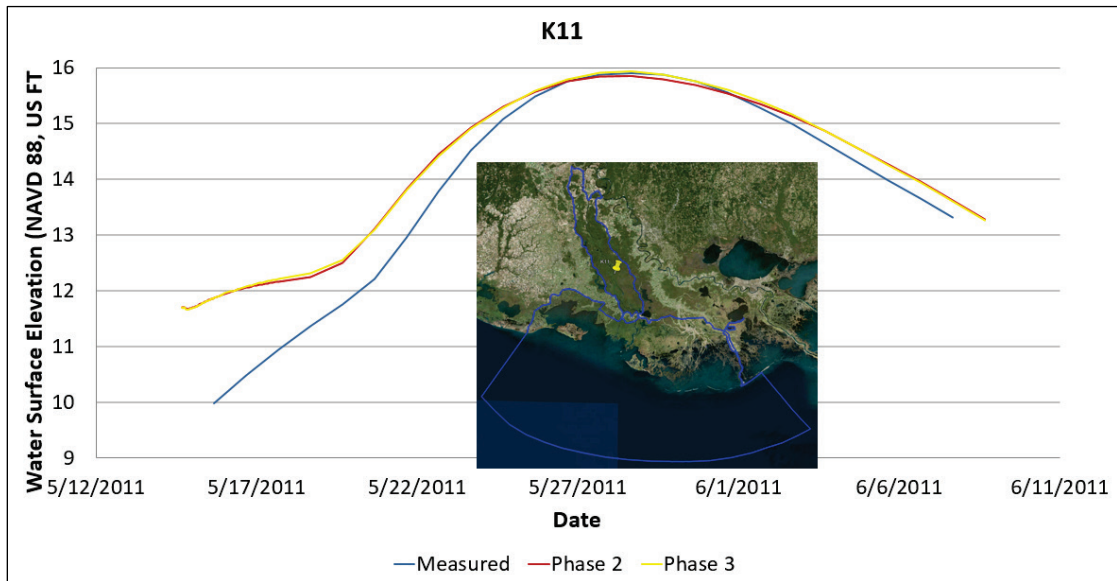


Figure 97. Gage K12 model results.

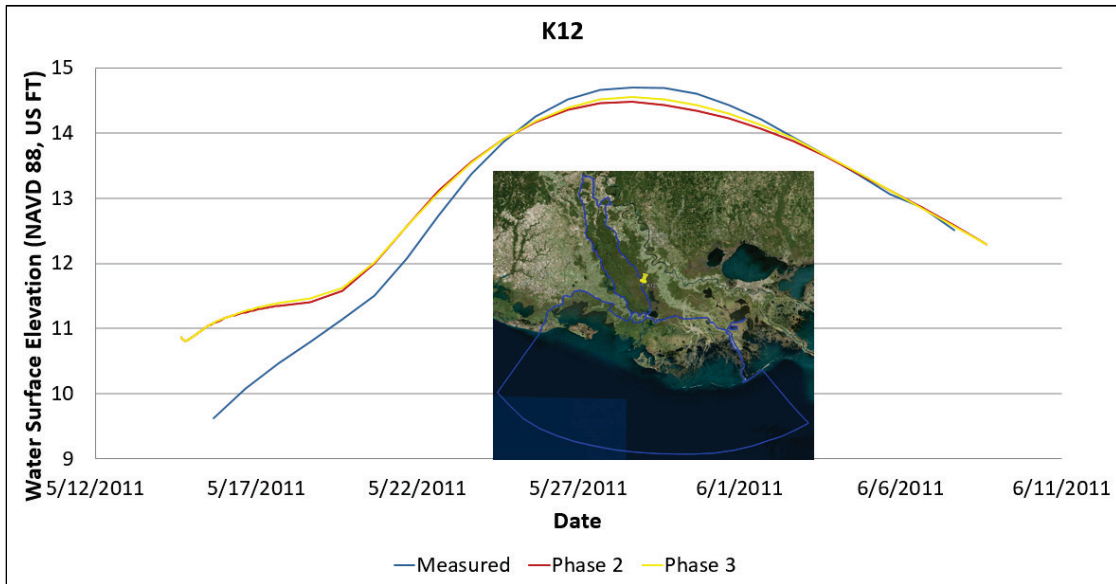


Figure 98. Gage K14 model results.

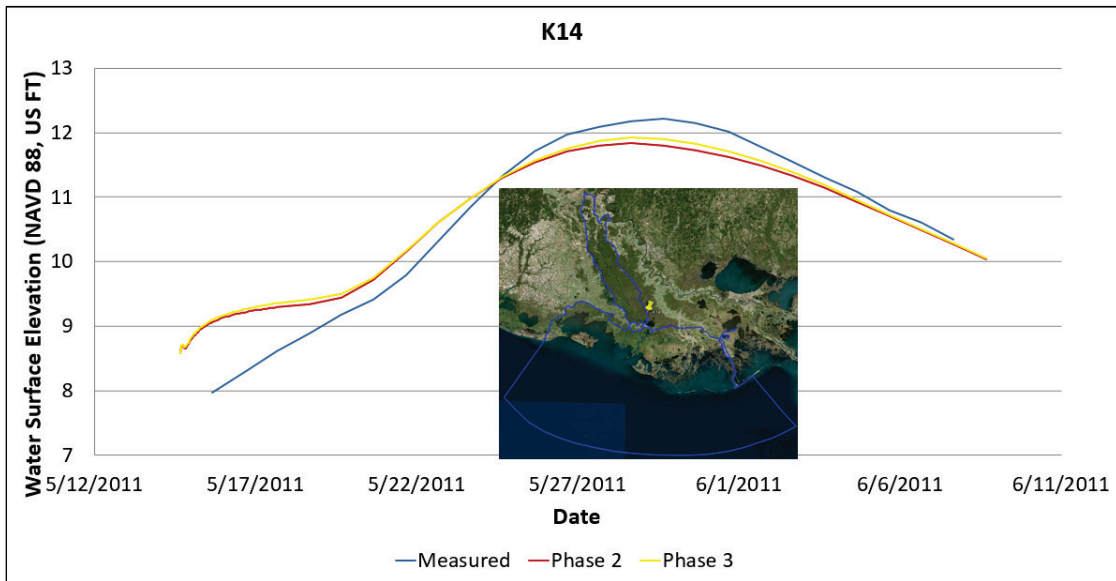


Figure 99. Gage K15 model results.

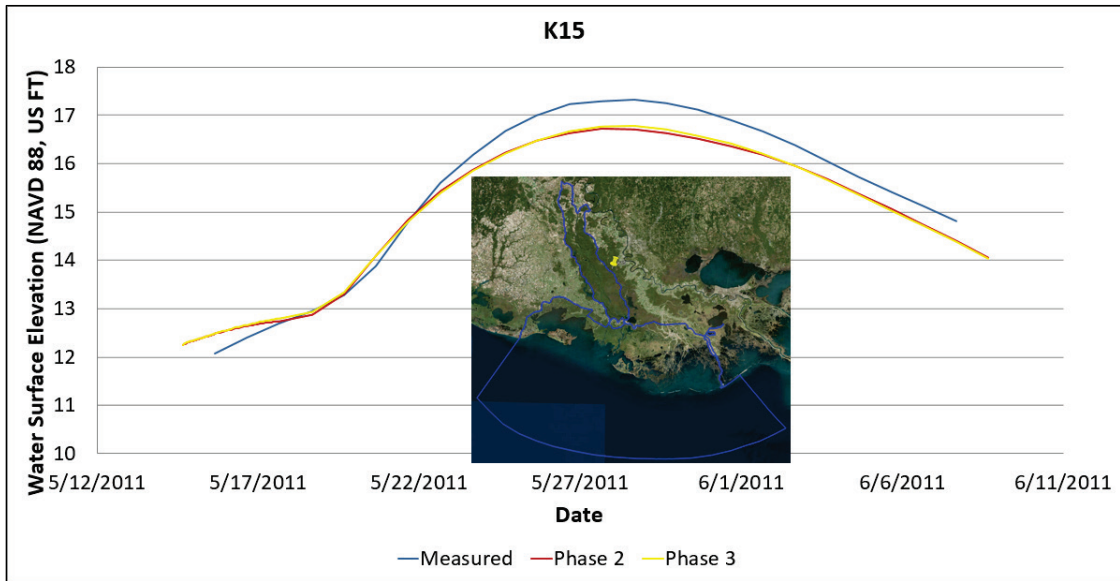


Figure 100. Gage K16 model results.

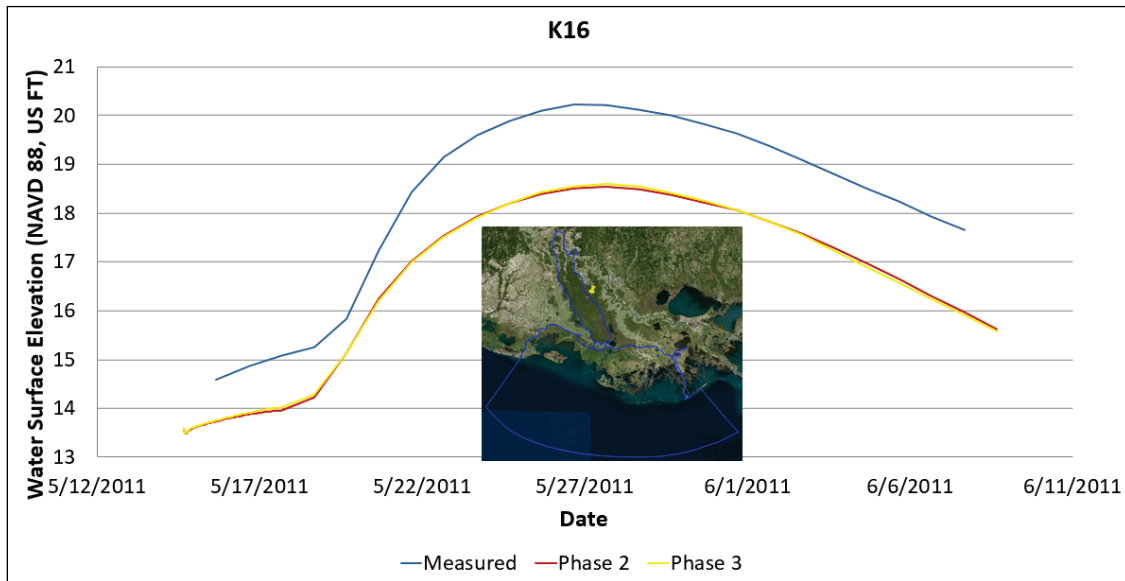


Figure 101. Gage K17 model results.

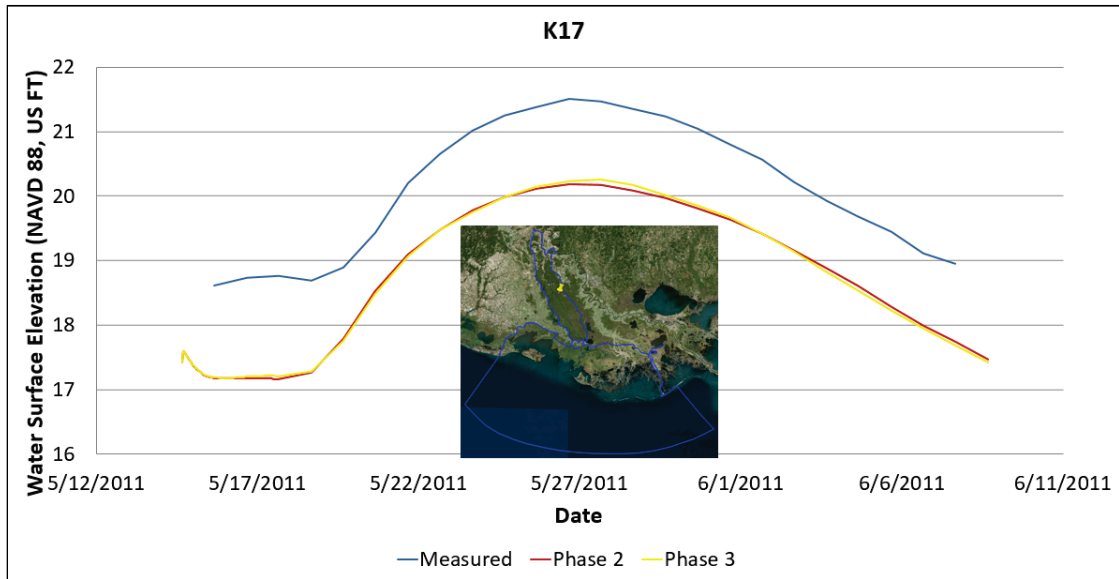


Figure 102. Gage K18 model results.

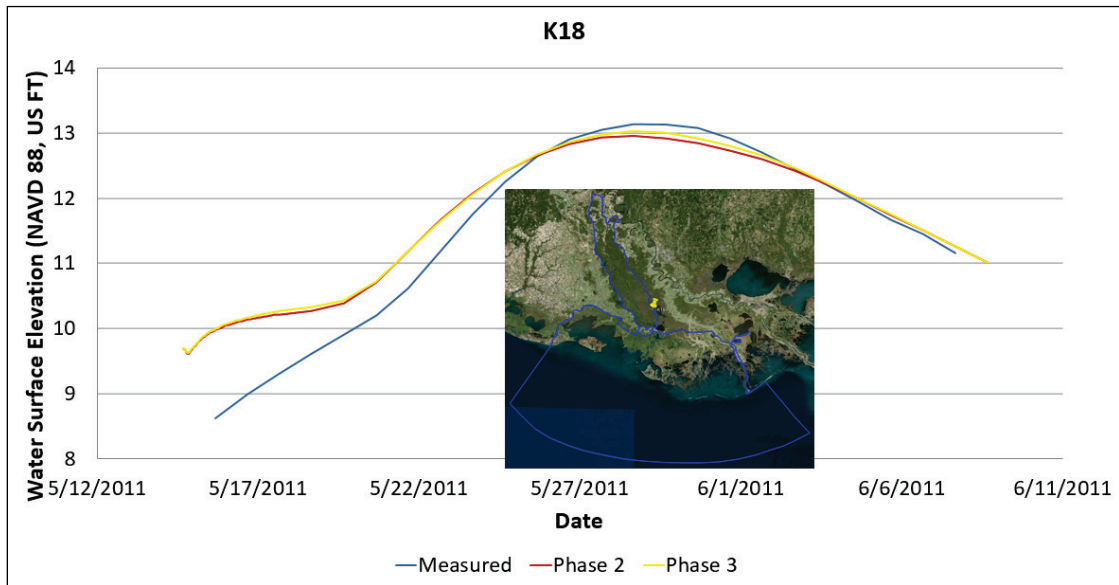


Figure 103. Gage K19 model results.

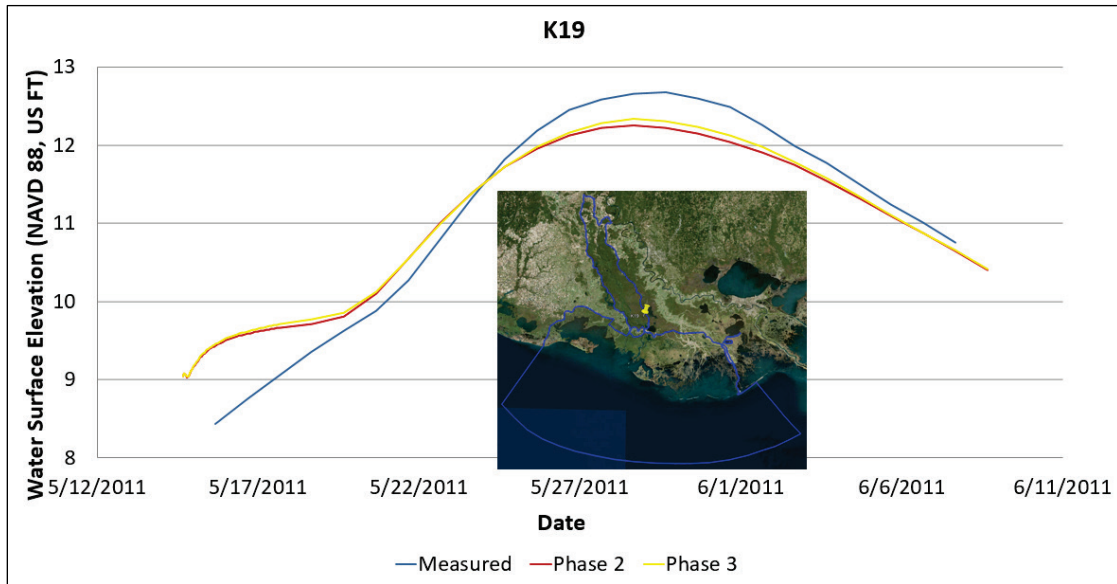
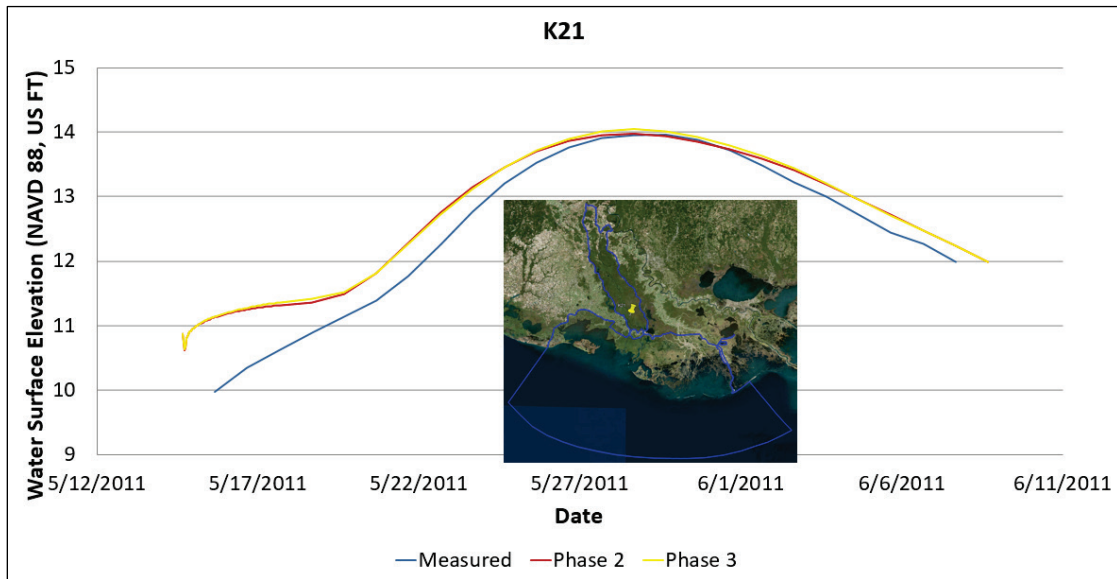


Figure 104. Gage K21 model results.



There are two gages (BO4 and K2) that have much less accurate results over the entire simulation period than the other comparison sites. Gage BO4 has some differences for the initial period of the simulation. This gage was also placed on dry land near a roadway before the flood waters reached this location (Figure 105). Gage K2 was located on the western border of the model in a channel that runs along the WABPL (Figure 106). Several sources for these discrepancies are possible. The model may not properly capture hydraulic connectivity throughout the entire domain (likely due to resolution deficiencies in some areas) for extreme flood events. There could be some associated error with the gage placement (Figure 107) due to possible flow accelerations and/or constrictions through the bridge piers and other unknown submerged bridge features. It is also possible that the discrepancies at these two sites are associated with a local instrumentation error given that the rest of the gages throughout the entire domain compare more favorably to the model results.

Figure 105. Gage B04 location.



Figure 106. Aerial view of gage K2 location.



Figure 107. Gage K2 location.



## Appendix B: Task 3 AdjustVeg Results Continued

For reference to Arc locations, please see Figure 37. Figure 108 – Figure 149 show, for each location, how the water levels change with increasing flow through the MCS on days 7, 9, and 11 (Figure 26 presents discharge information on each day).

Figure 108. Arc 2 day 7 WSE profile.

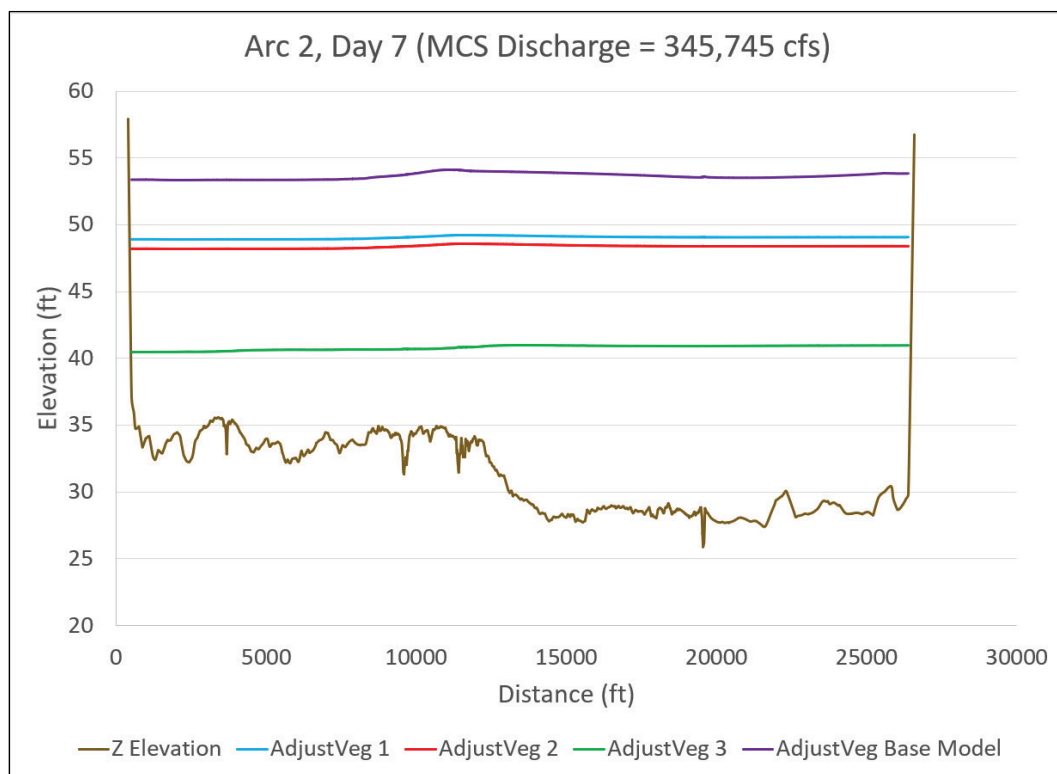


Figure 109. Arc 2 day 9 WSE profile.

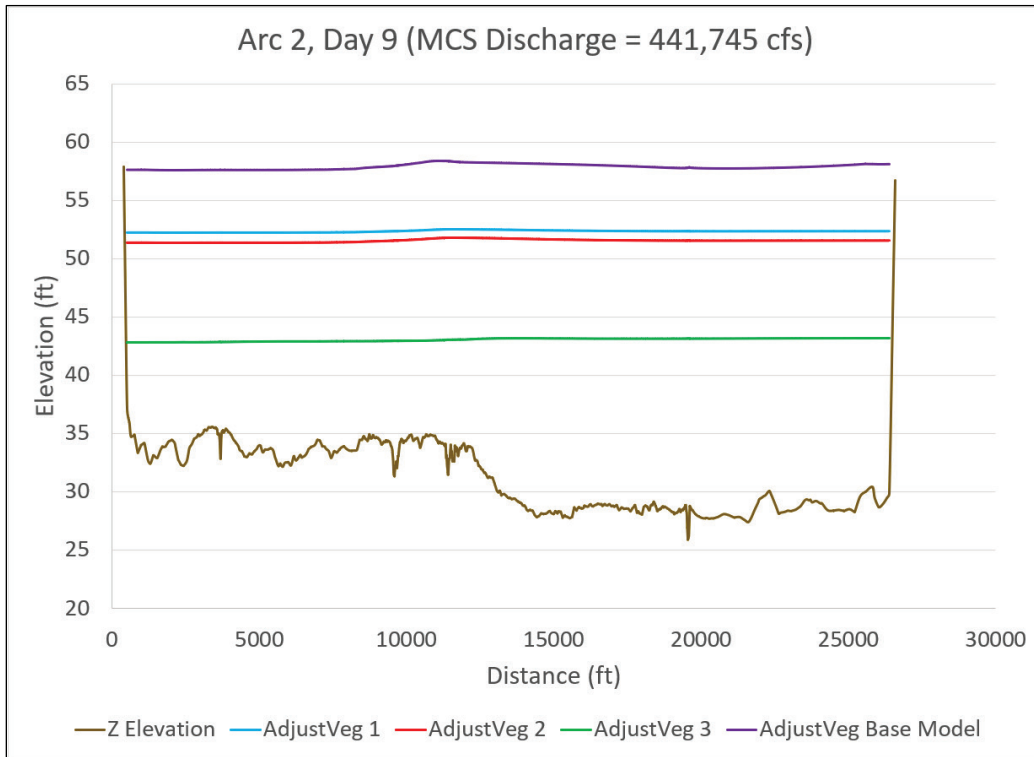


Figure 110. Arc 2 day 11 WSE profile.

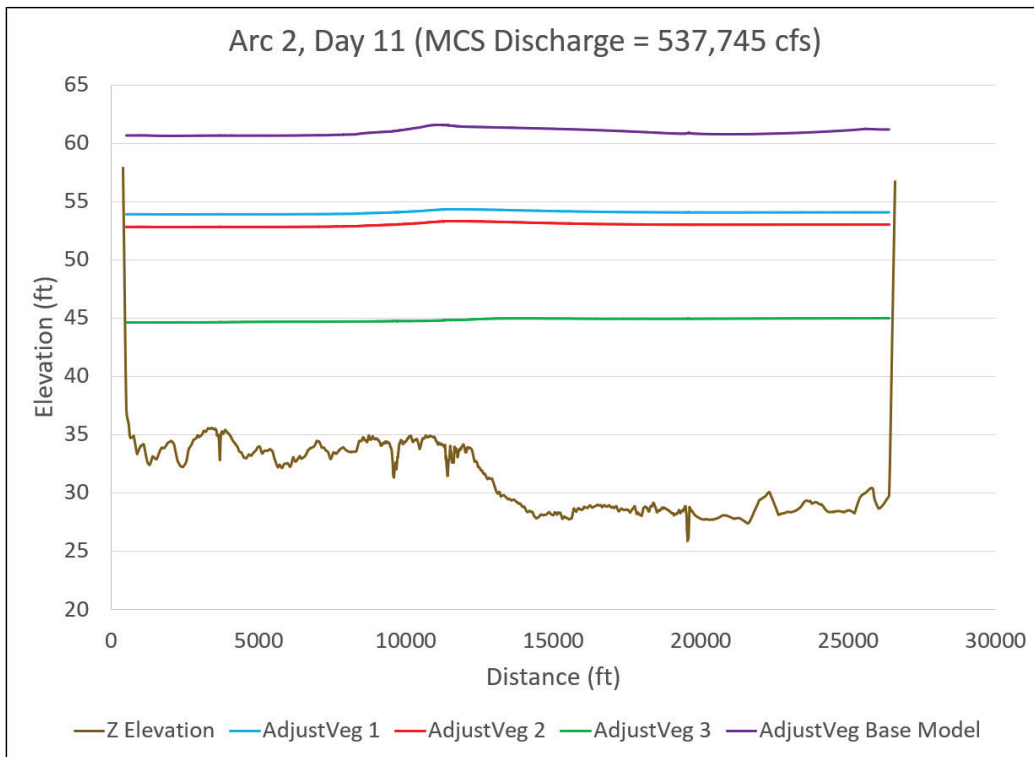


Figure 111. Arc 3 day 7 WSE profile.

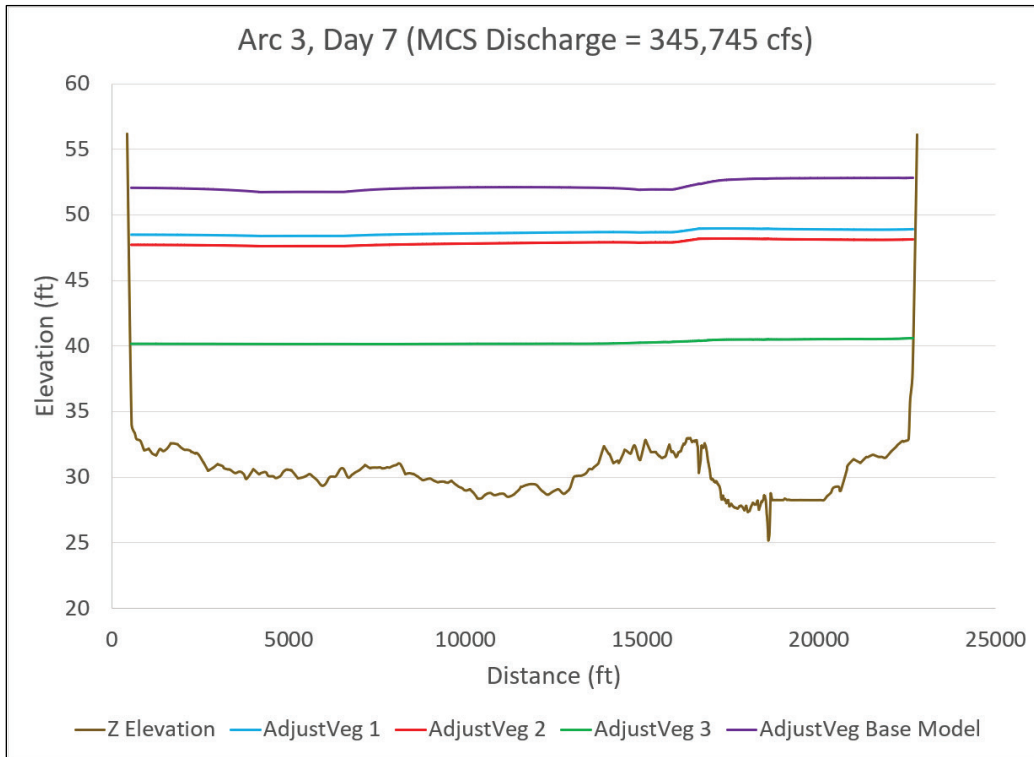


Figure 112. Arc 3 day 9 WSE profile.

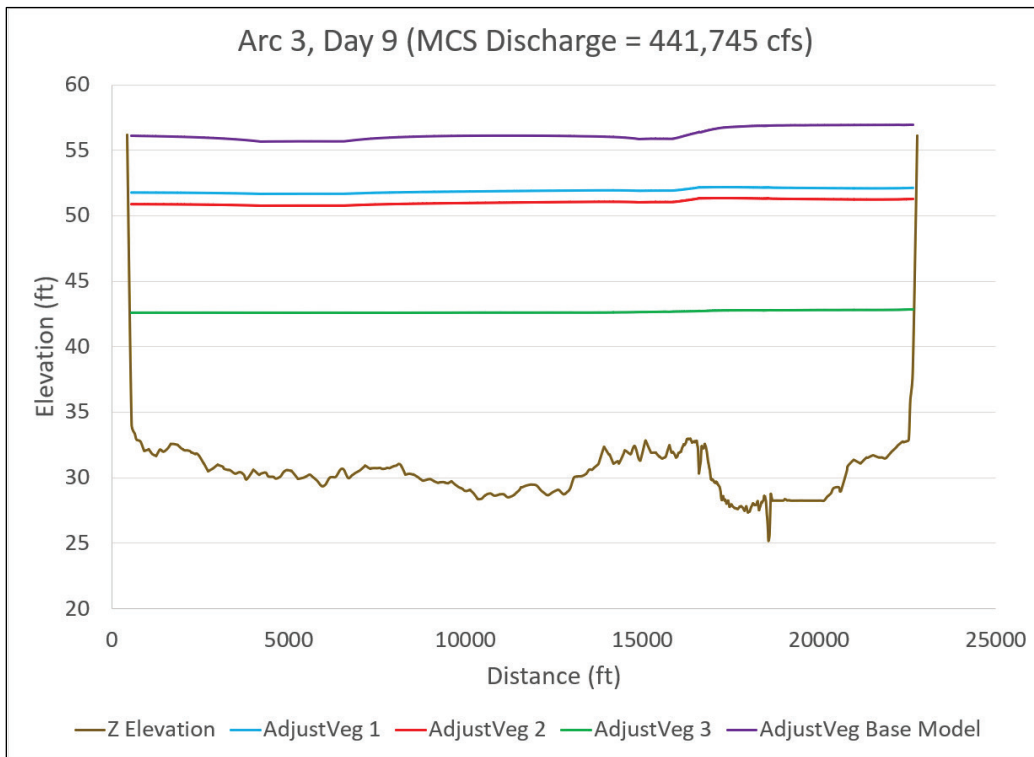


Figure 113. Arc 3 day 11 WSE profile.

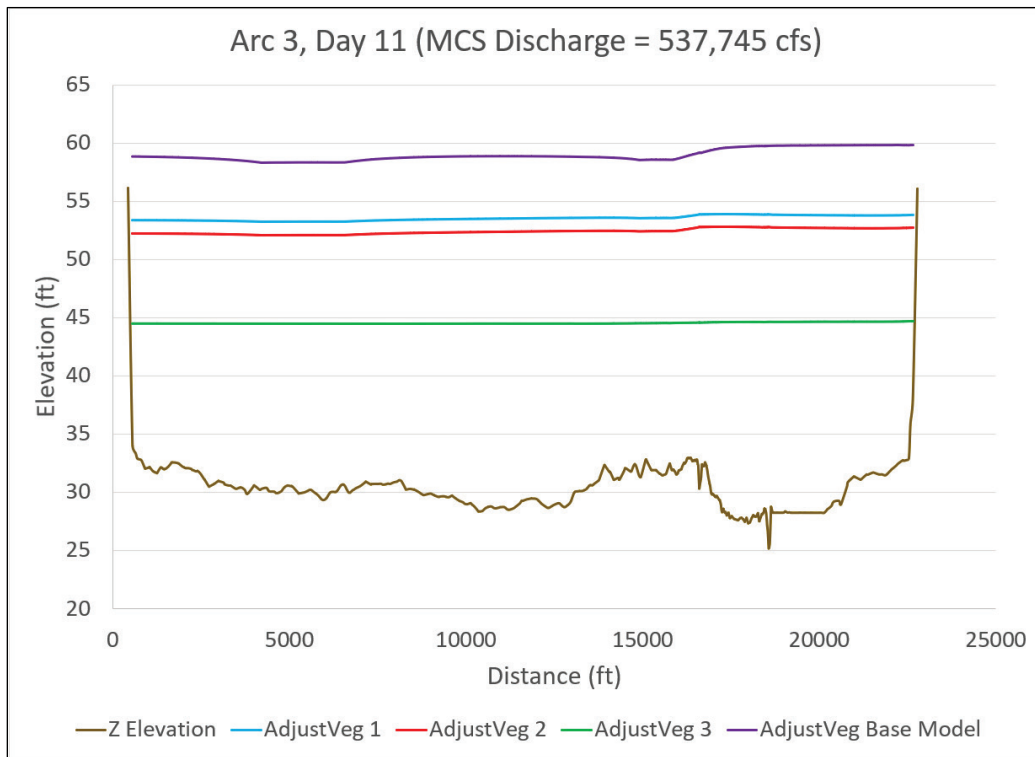


Figure 114. Arc 4 day 7 WSE profile.

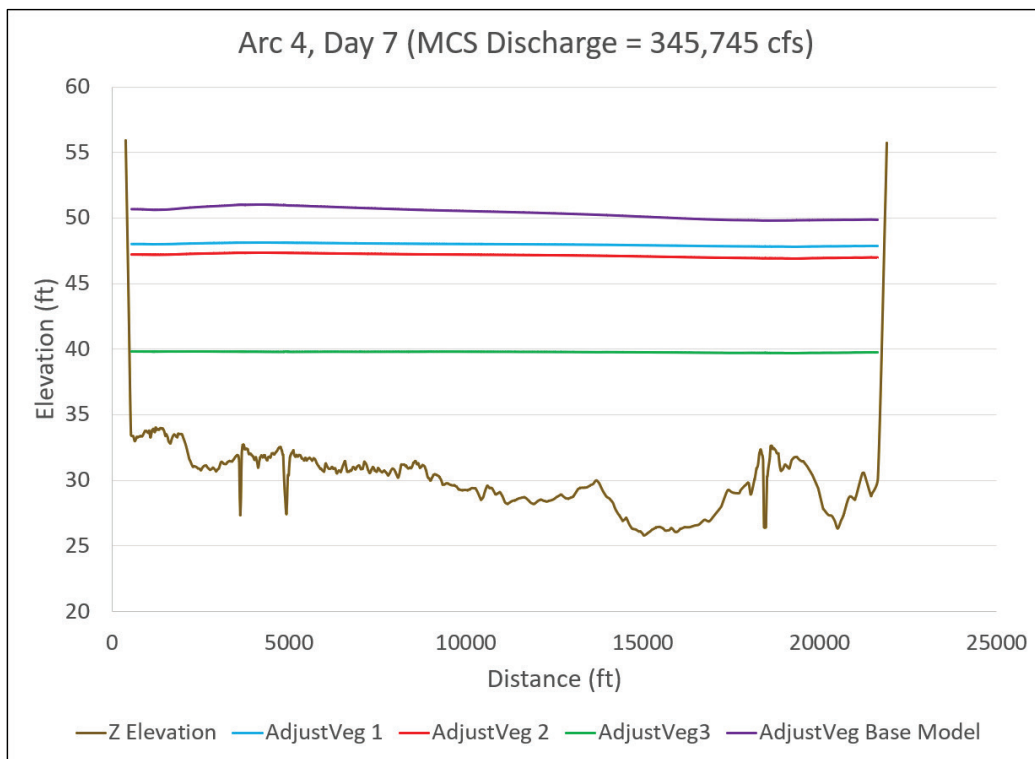


Figure 115. Arc 4 day 9 WSE profile.

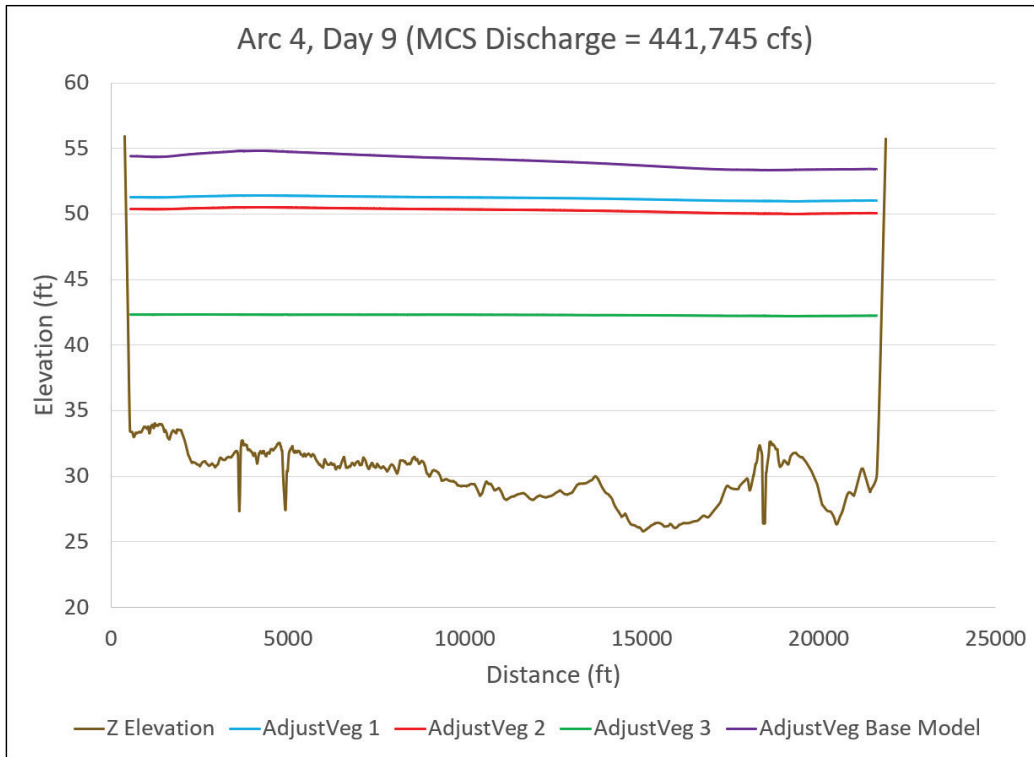


Figure 116. Arc 4 day 11 WSE profile.

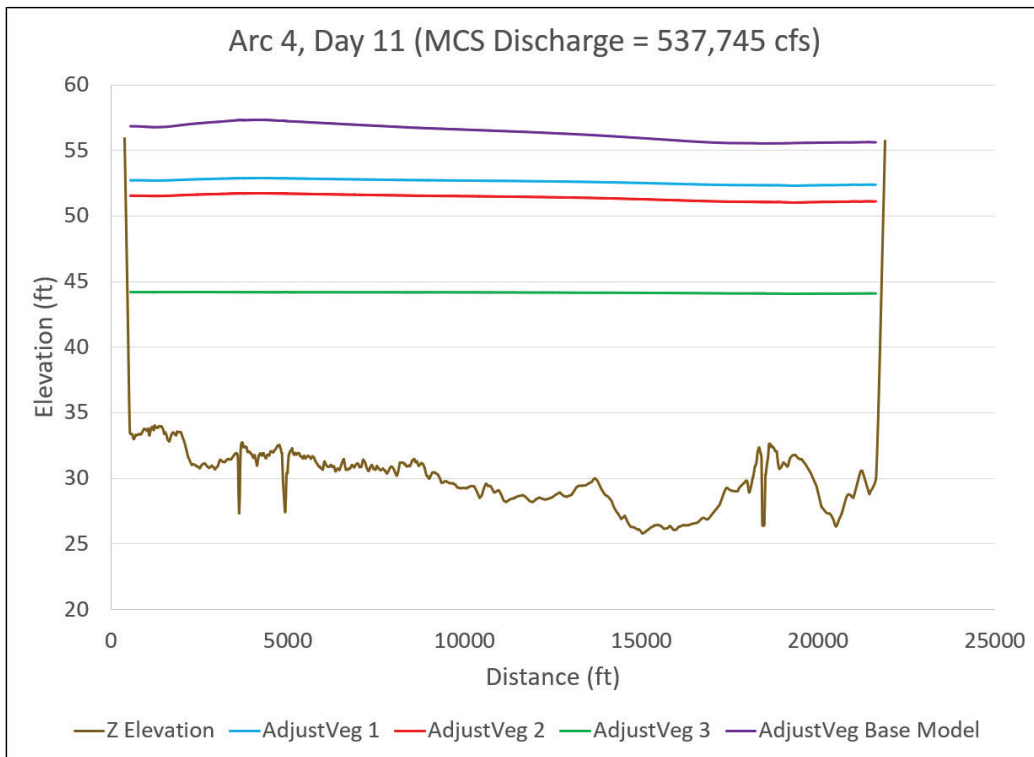


Figure 117. Arc 5 day 7 WSE profile.

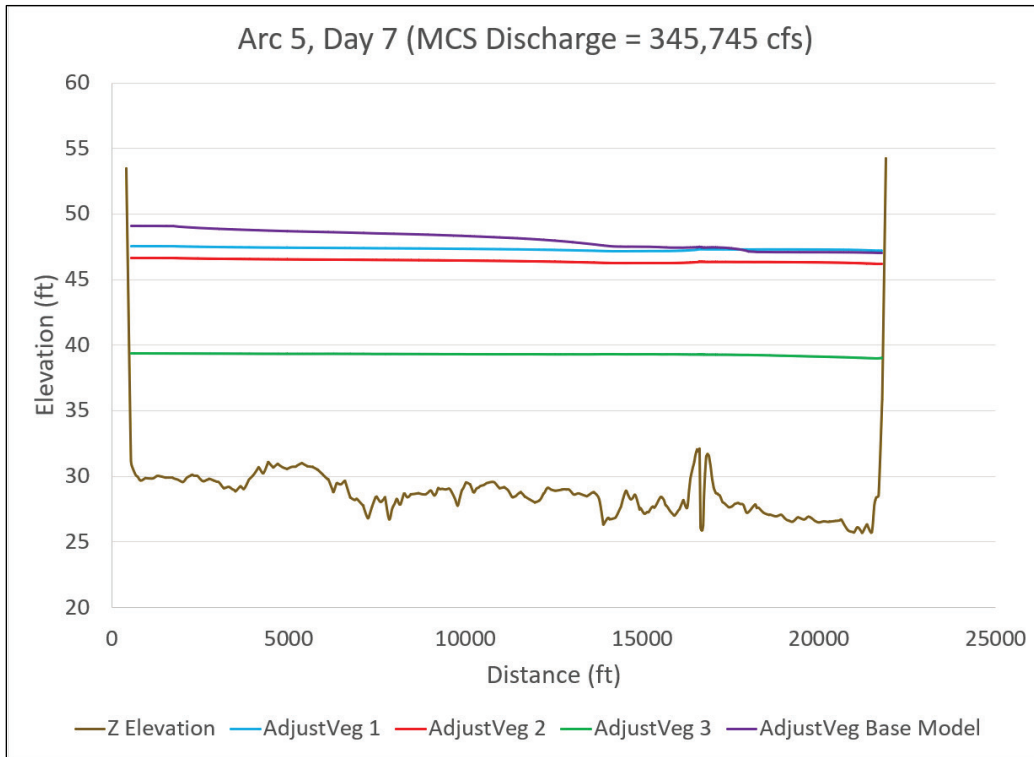


Figure 118. Arc 5 day 9 WSE profile.

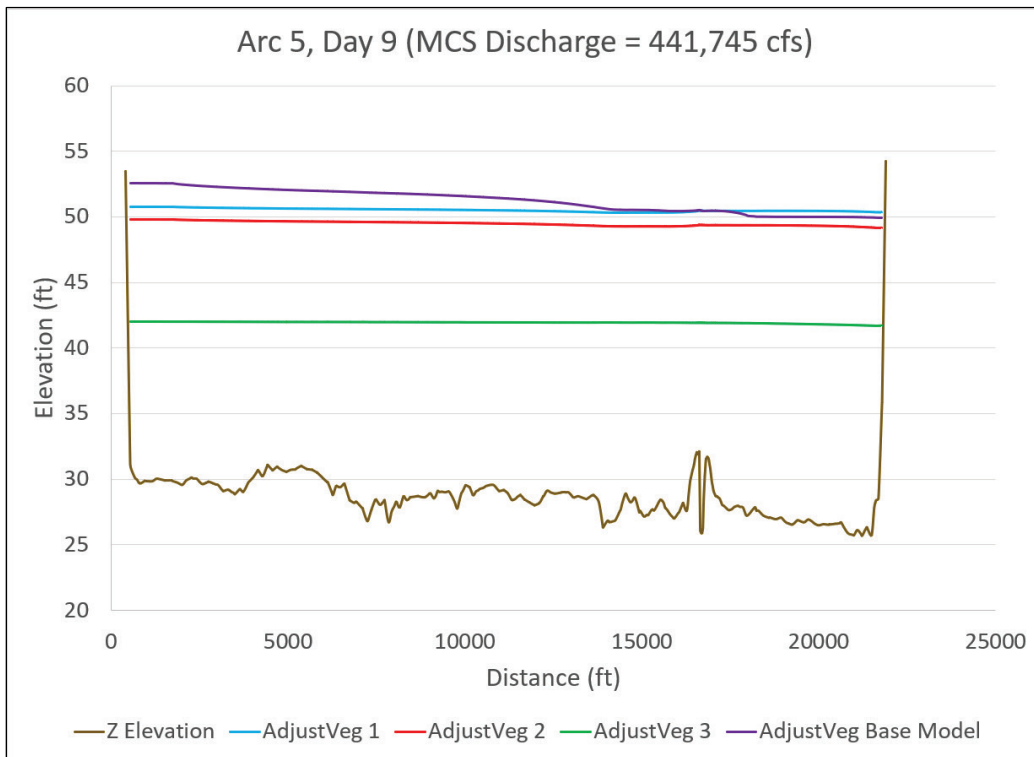


Figure 119. Arc 5 day 11 WSE profile.

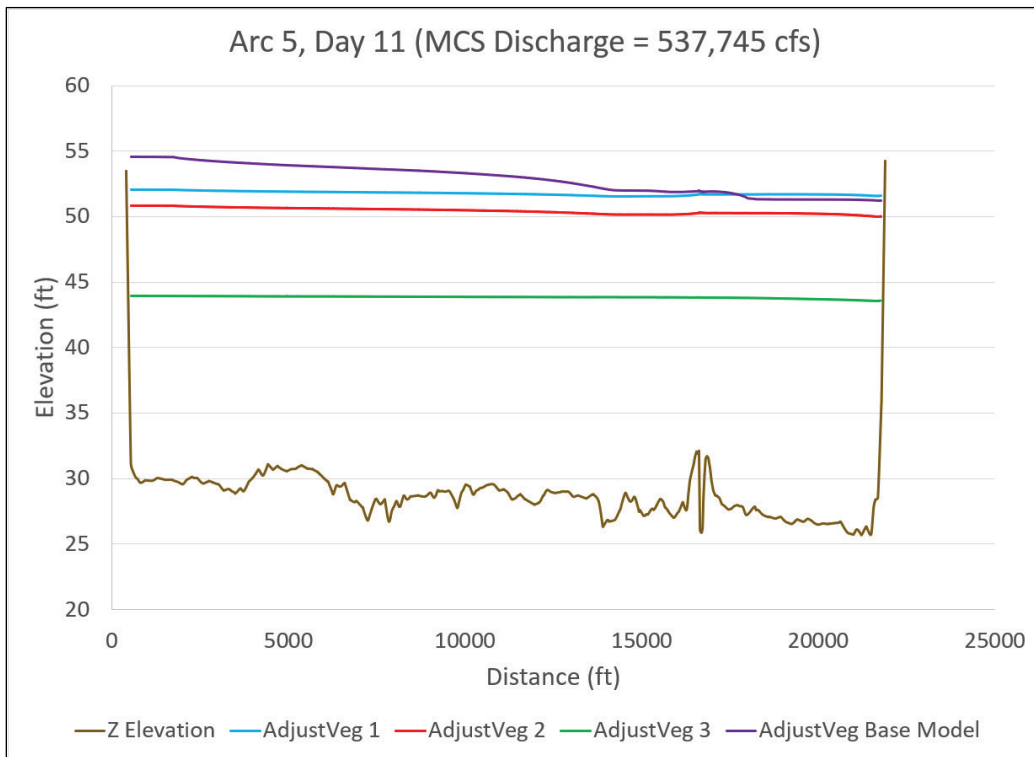


Figure 120. Arc 6 day 7 WSE profile.

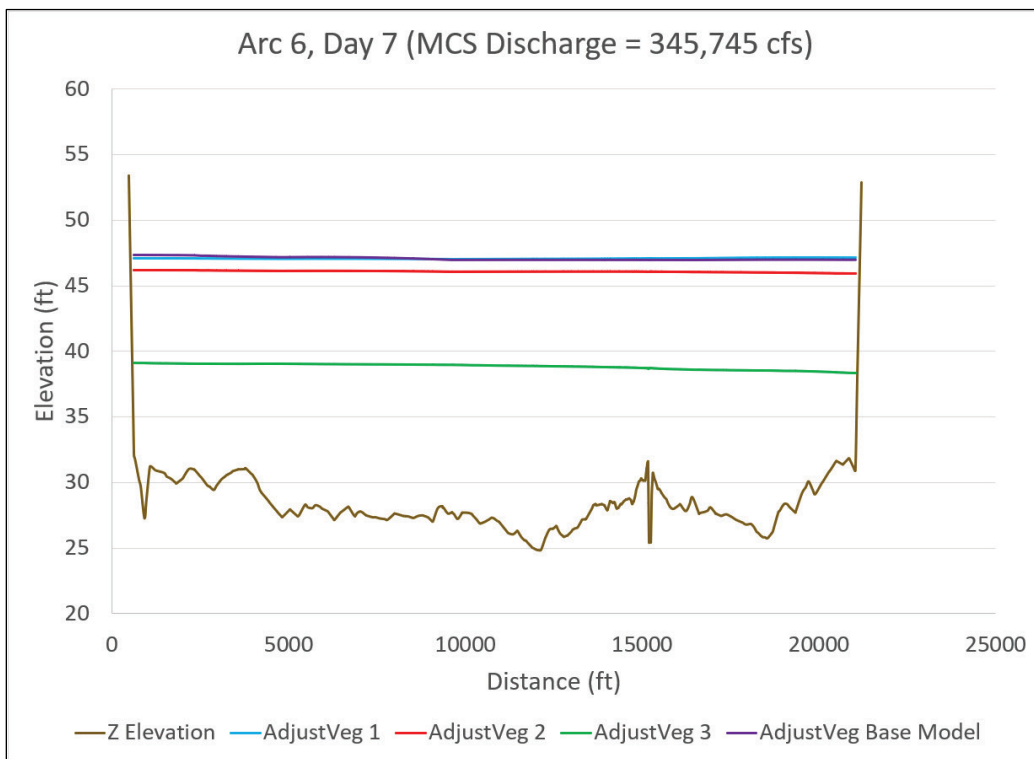


Figure 121. Arc 6 day 9 WSE profile.

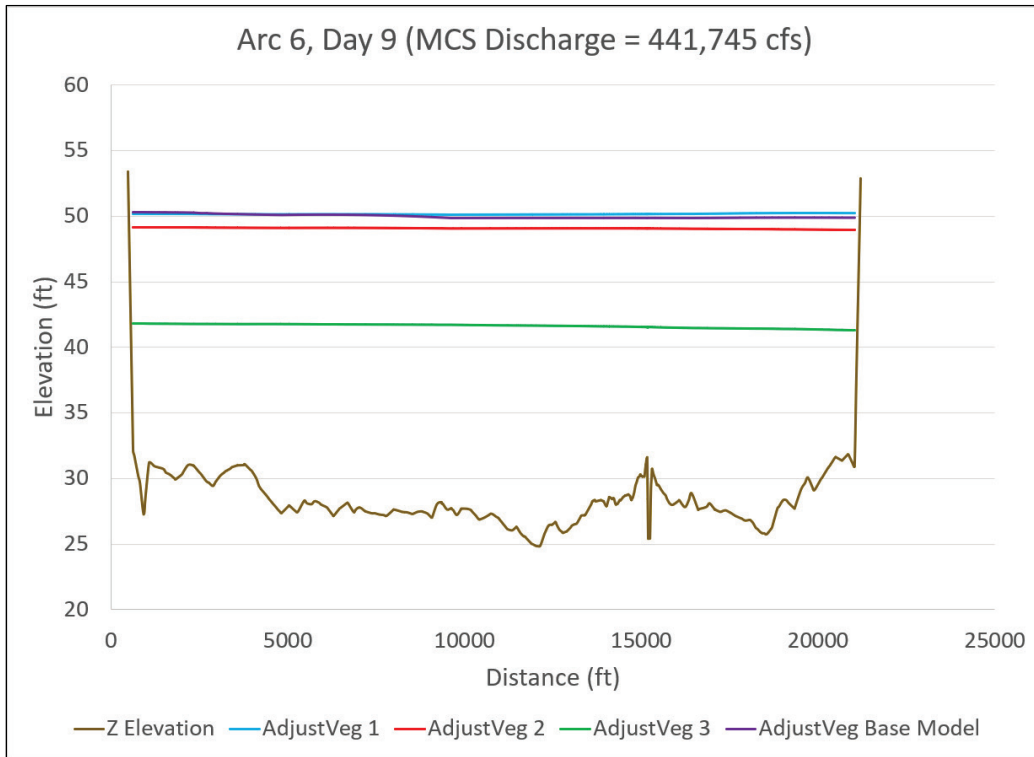


Figure 122. Arc 6 day 11 WSE profile.

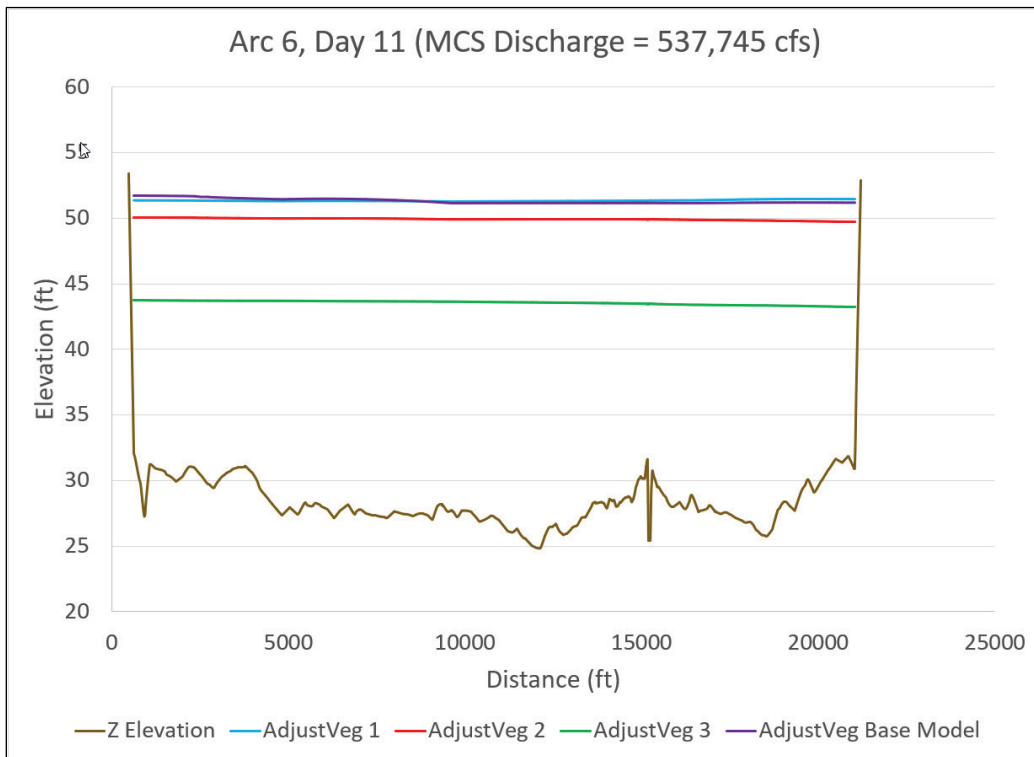


Figure 123. Arc 7 day 7 WSE profile.

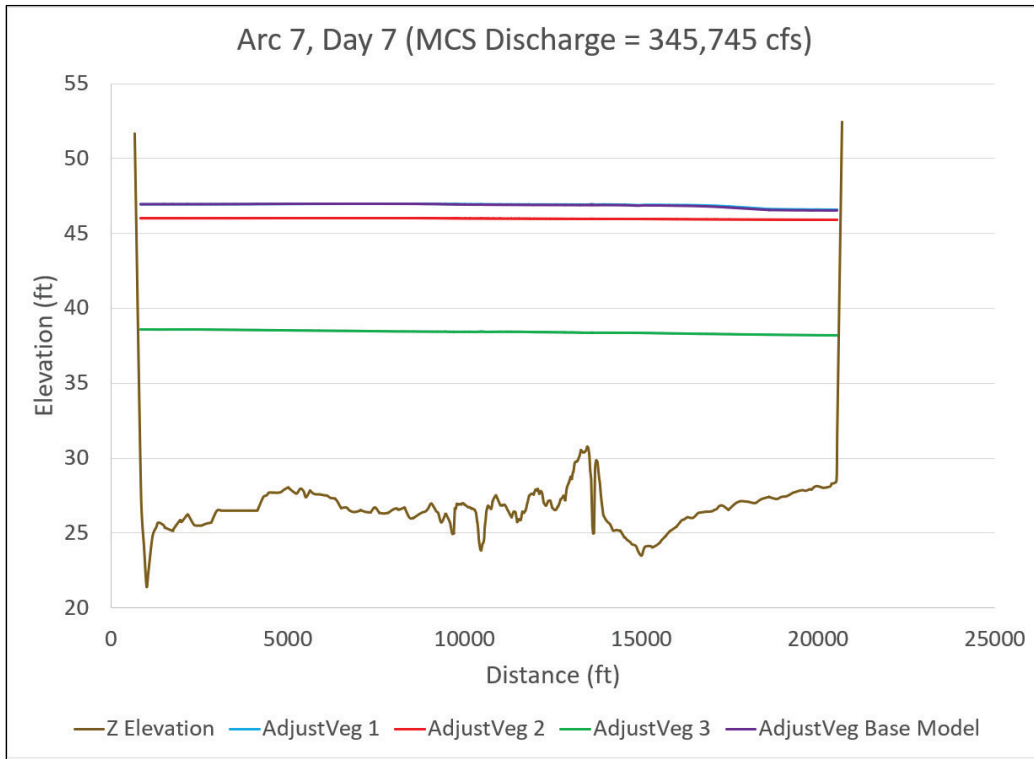


Figure 124. Arc 7 day 9 WSE profile.

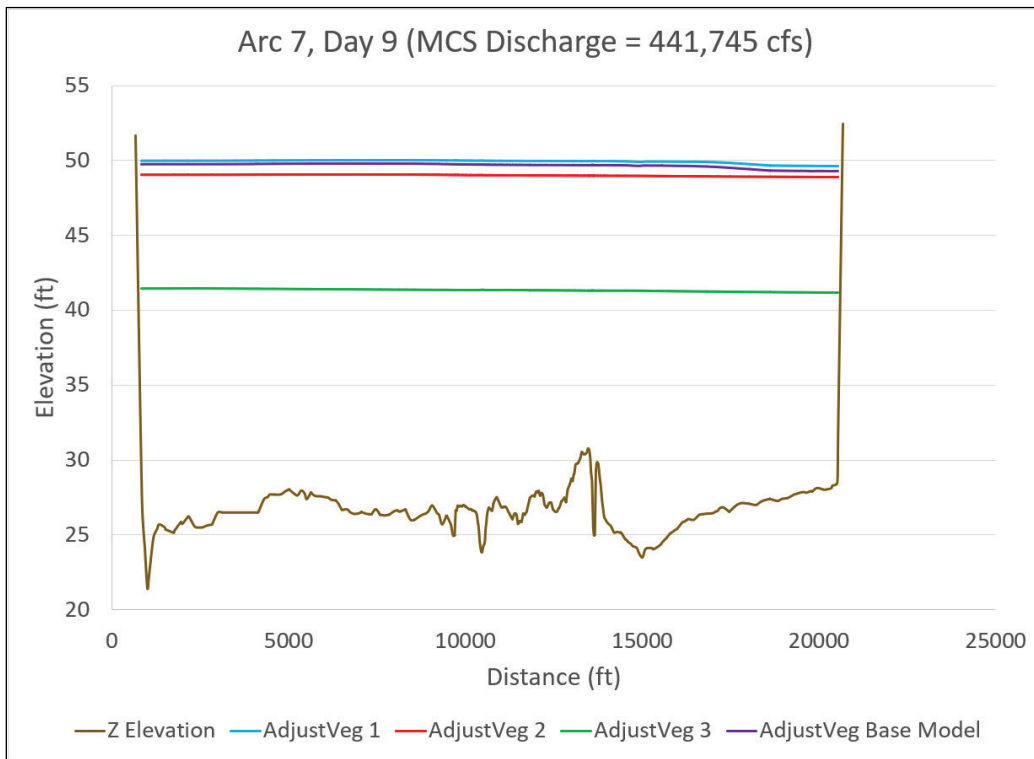


Figure 125. Arc 7 day 11 WSE profile.

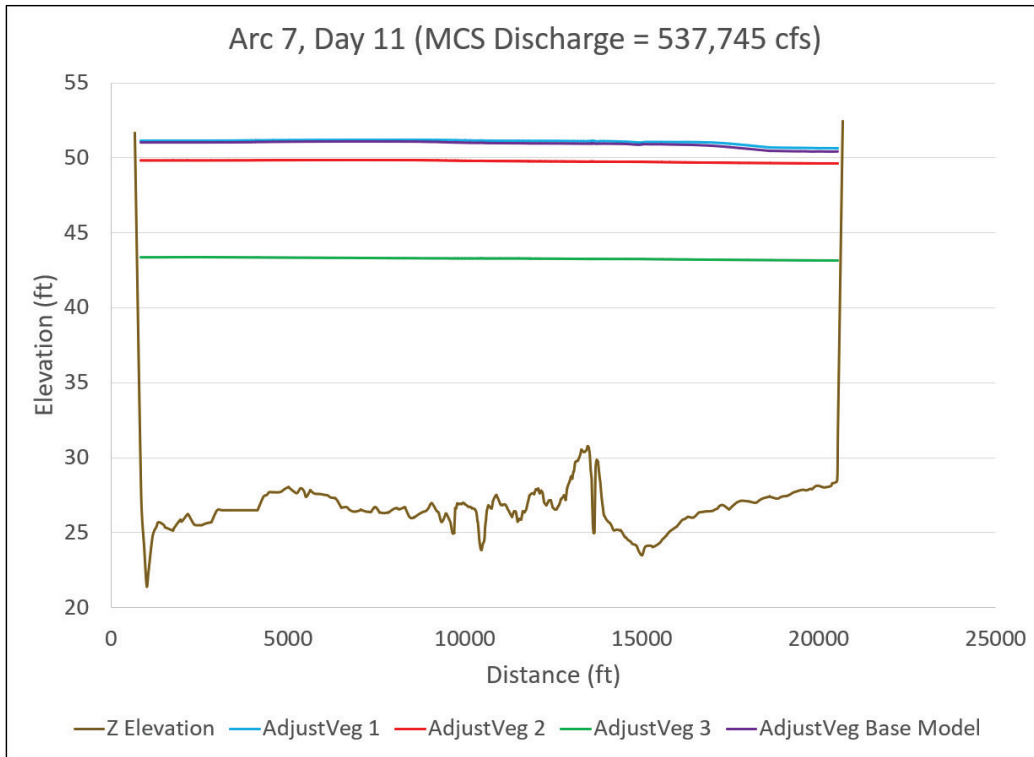


Figure 126. Arc 8 day 7 WSE profile.

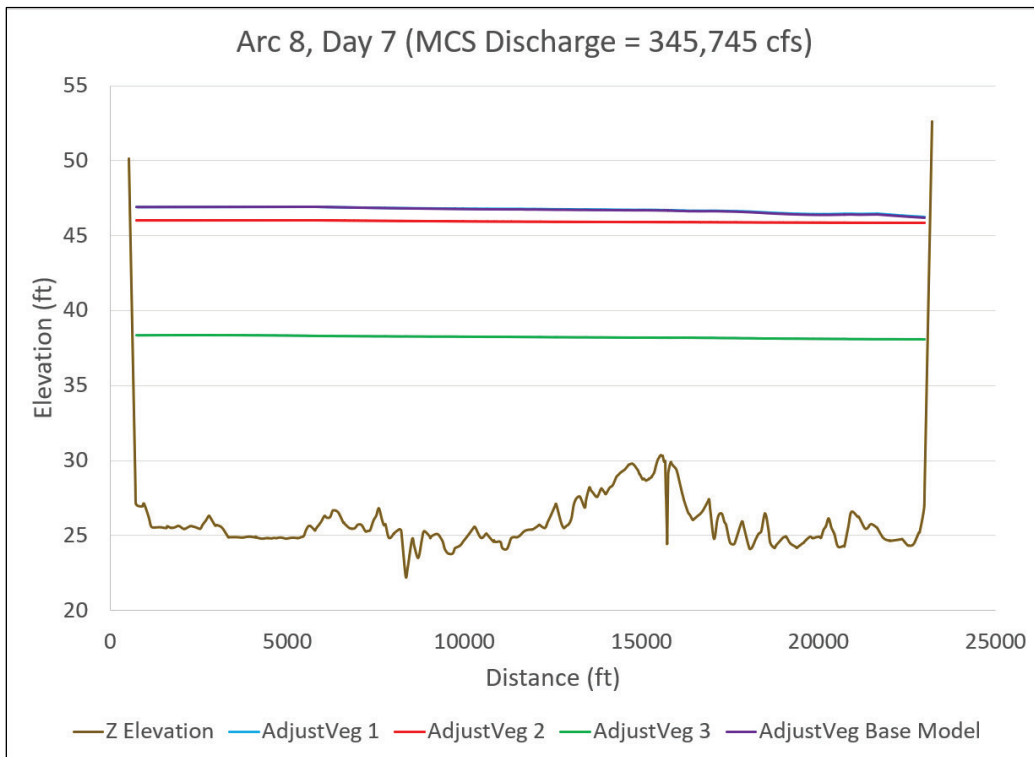


Figure 127. Arc 8 day 9 WSE profile.

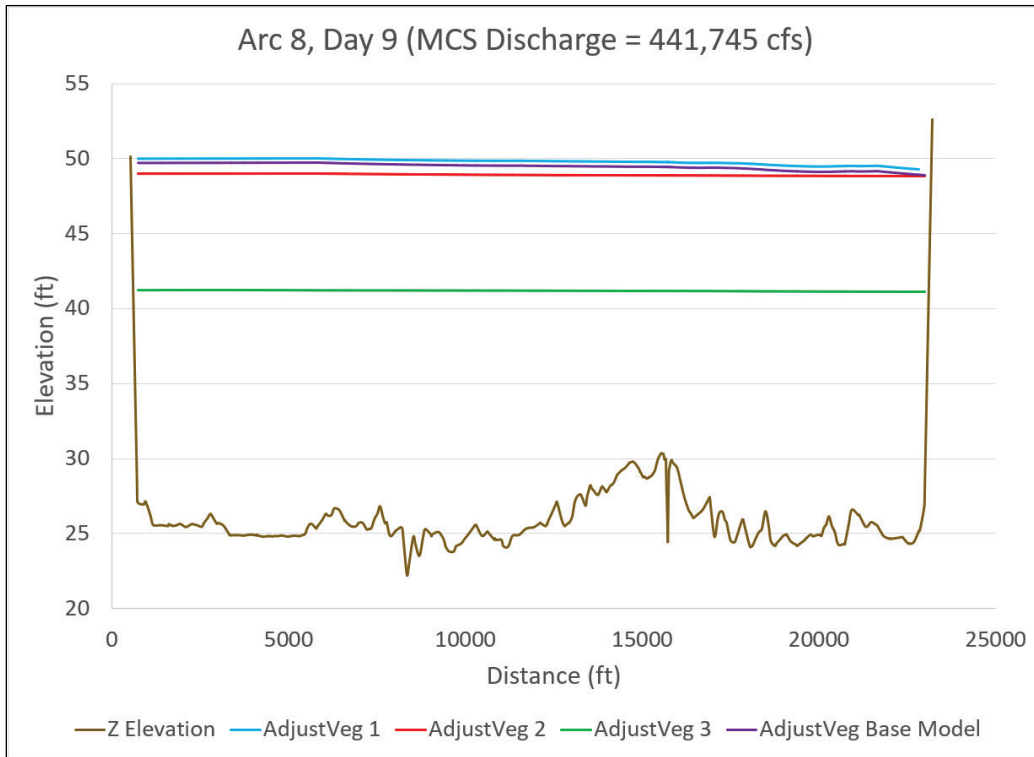


Figure 128. Arc 8 day 11 WSE profile.

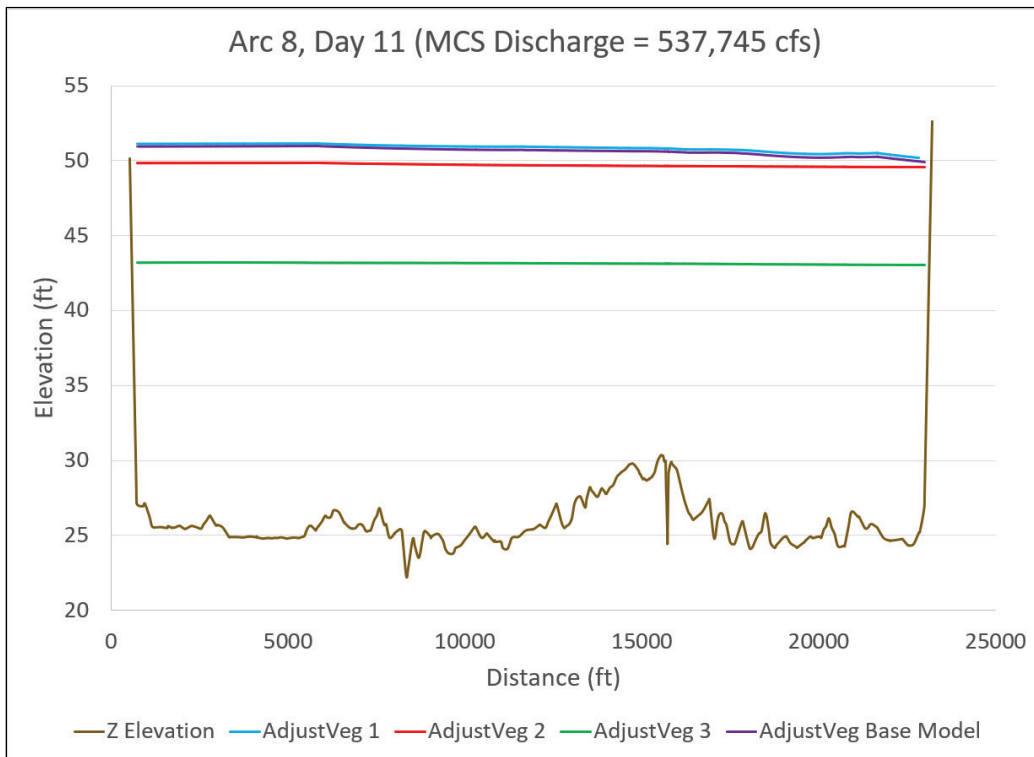


Figure 129. Arc 9 day 7 WSE profile.

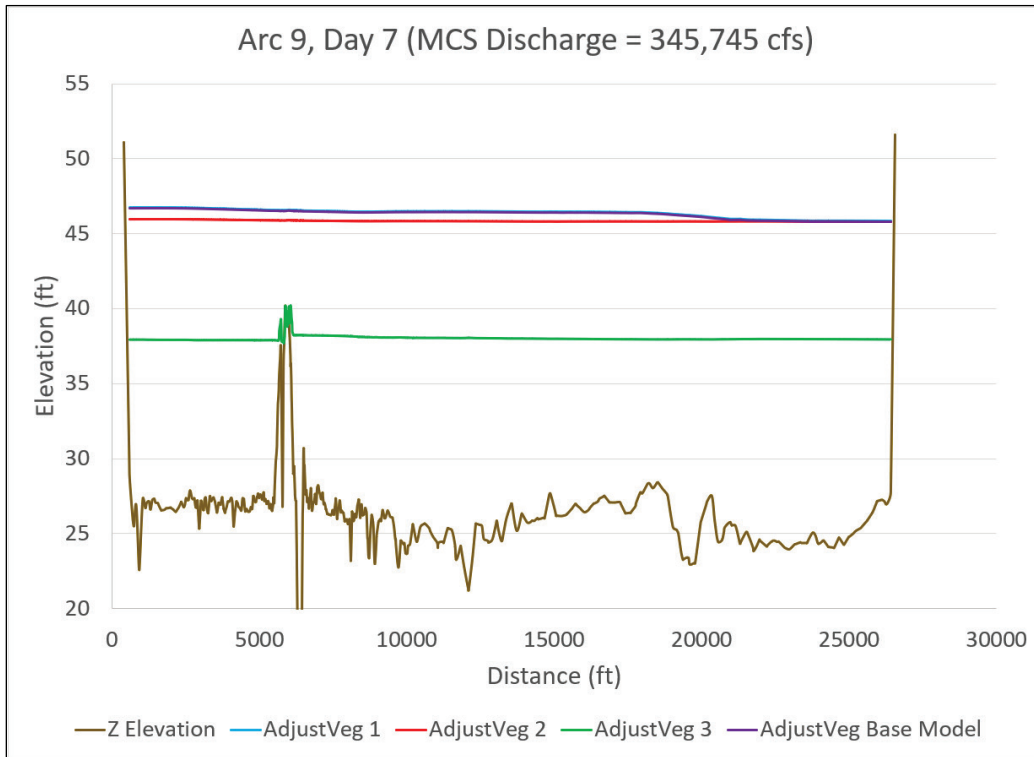


Figure 130. Arc 9 day 9 WSE profile.

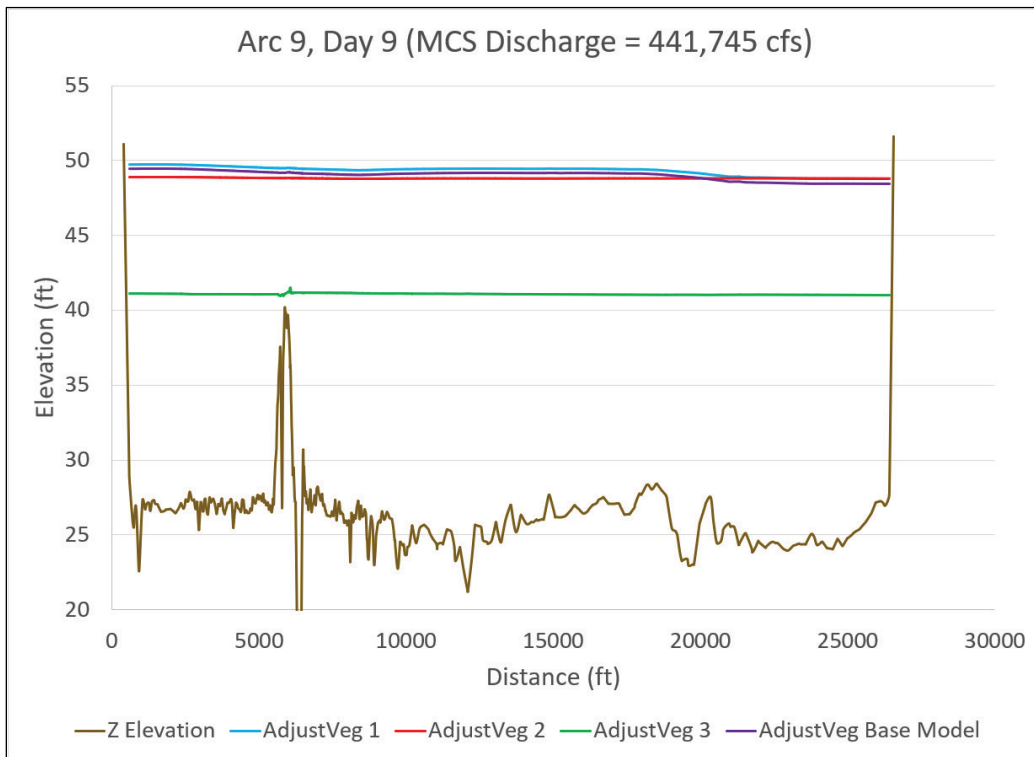


Figure 131. Arc 9 day 11 WSE profile.

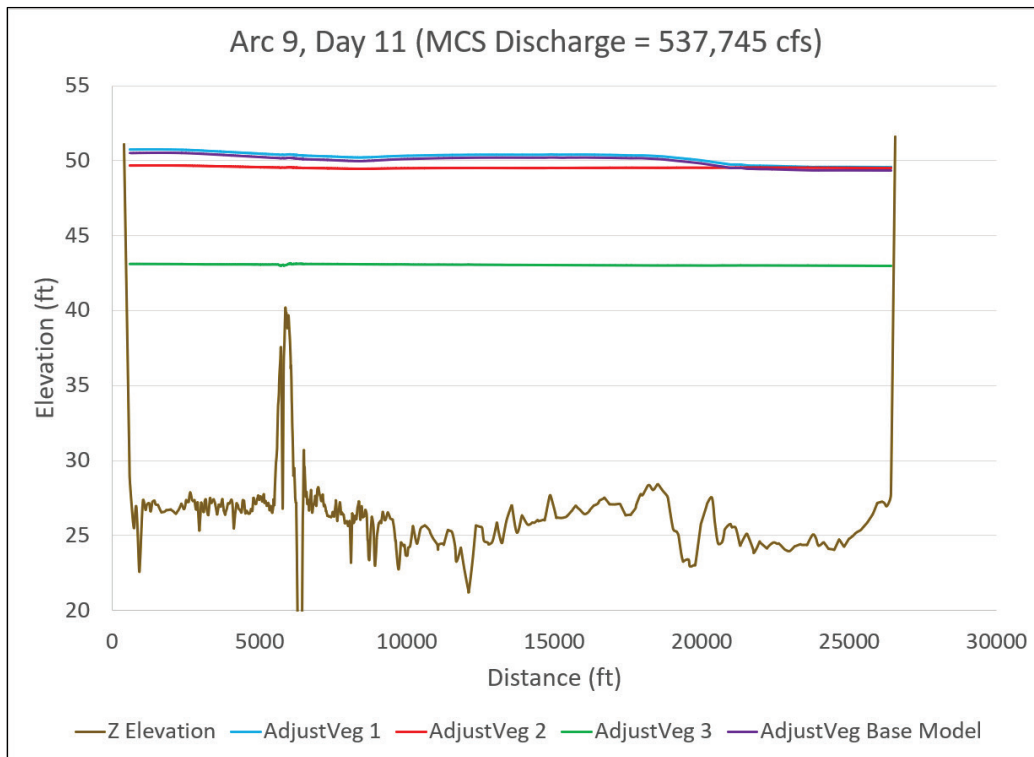


Figure 132. Arc 10 day 7 WSE profile.

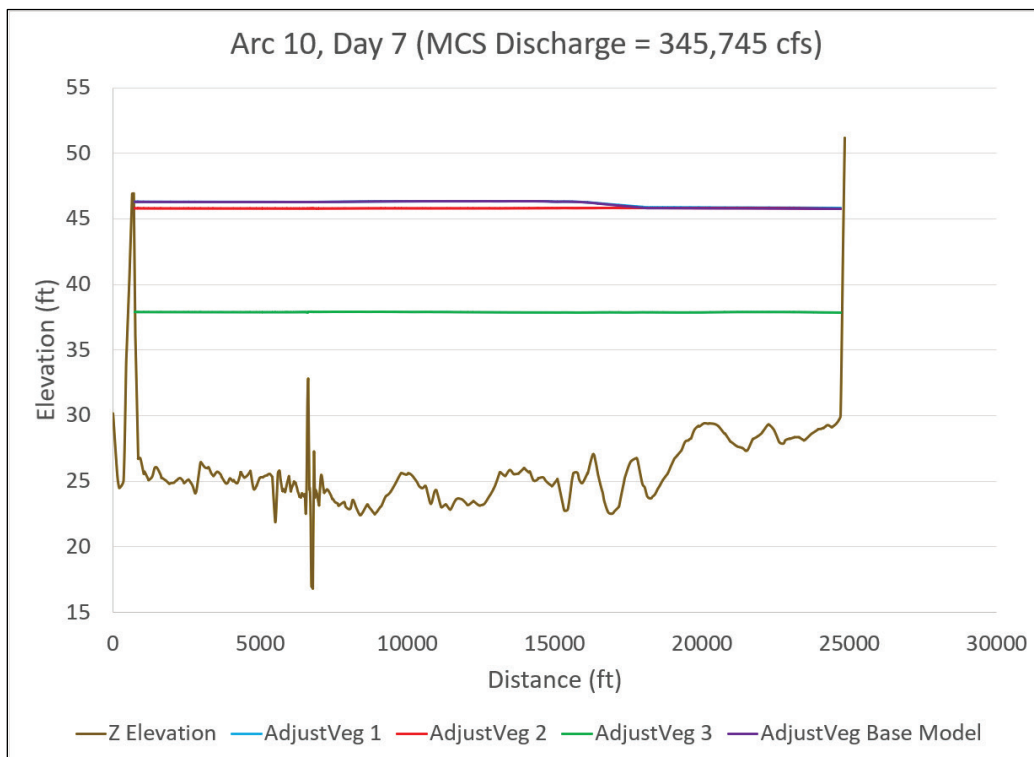


Figure 133. Arc 10 day 9 WSE profile.

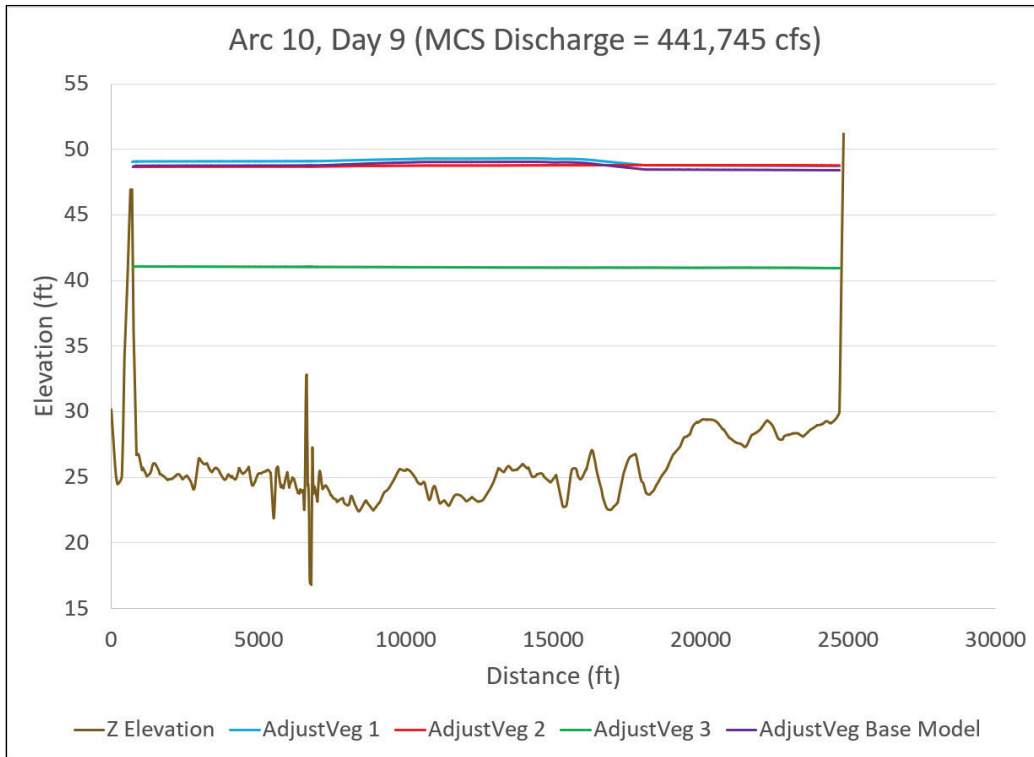


Figure 134. Arc 10 day 11 WSE profile.

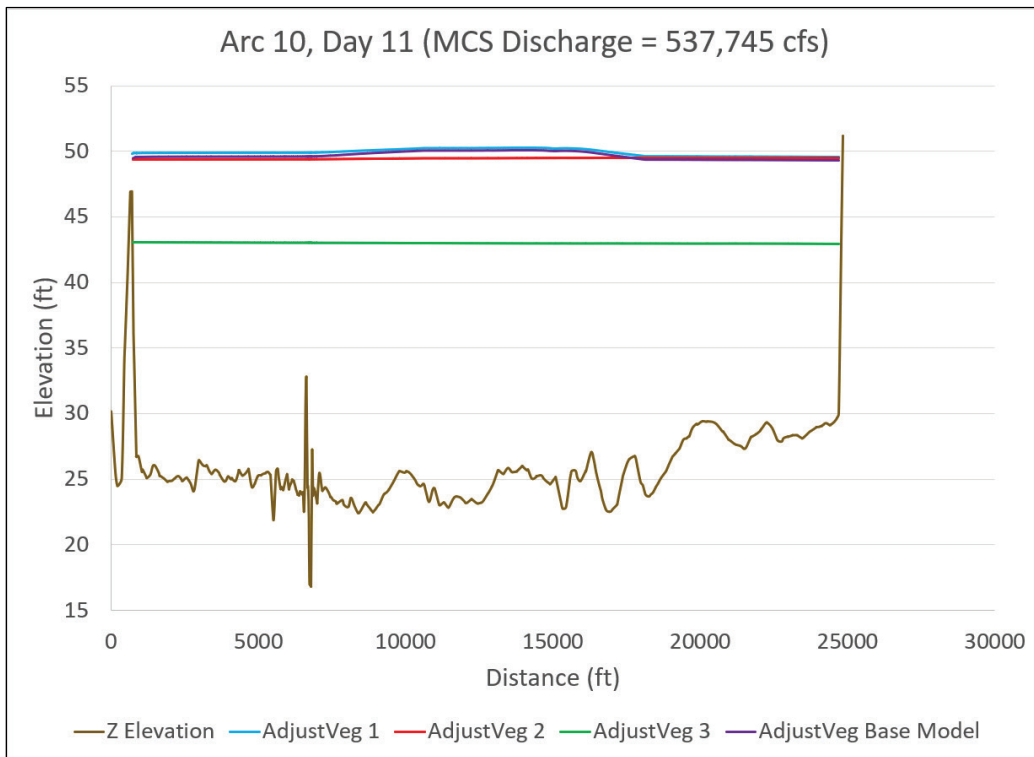


Figure 135. Arc 11 day 7 WSE profile.

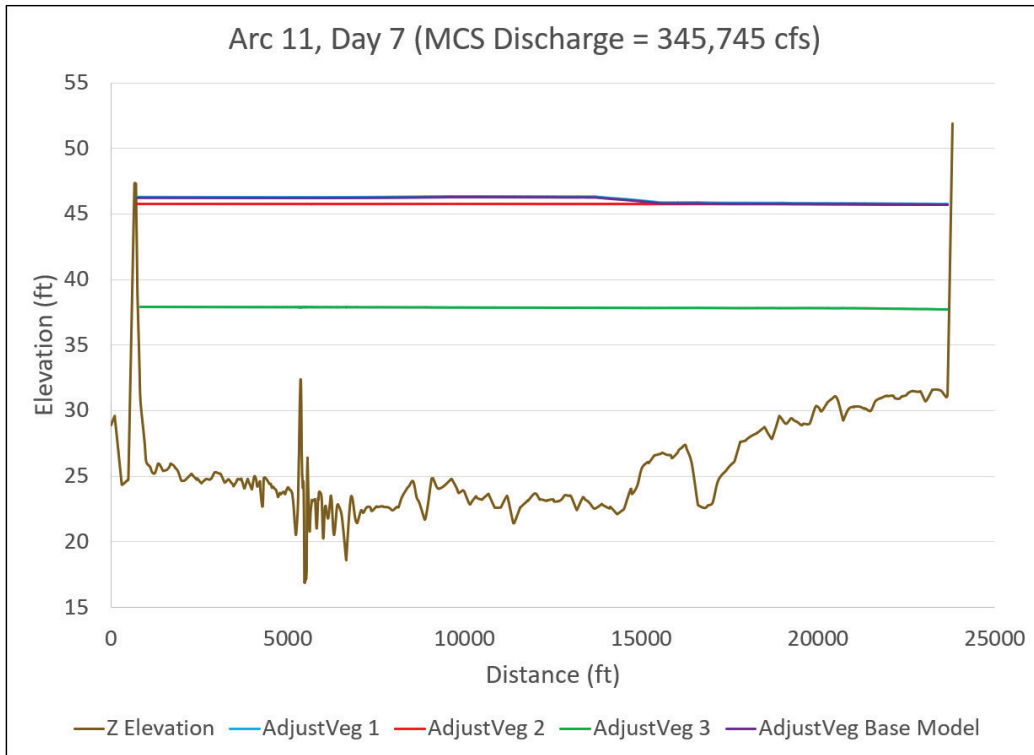


Figure 136. Arc 11 day 9 WSE profile.

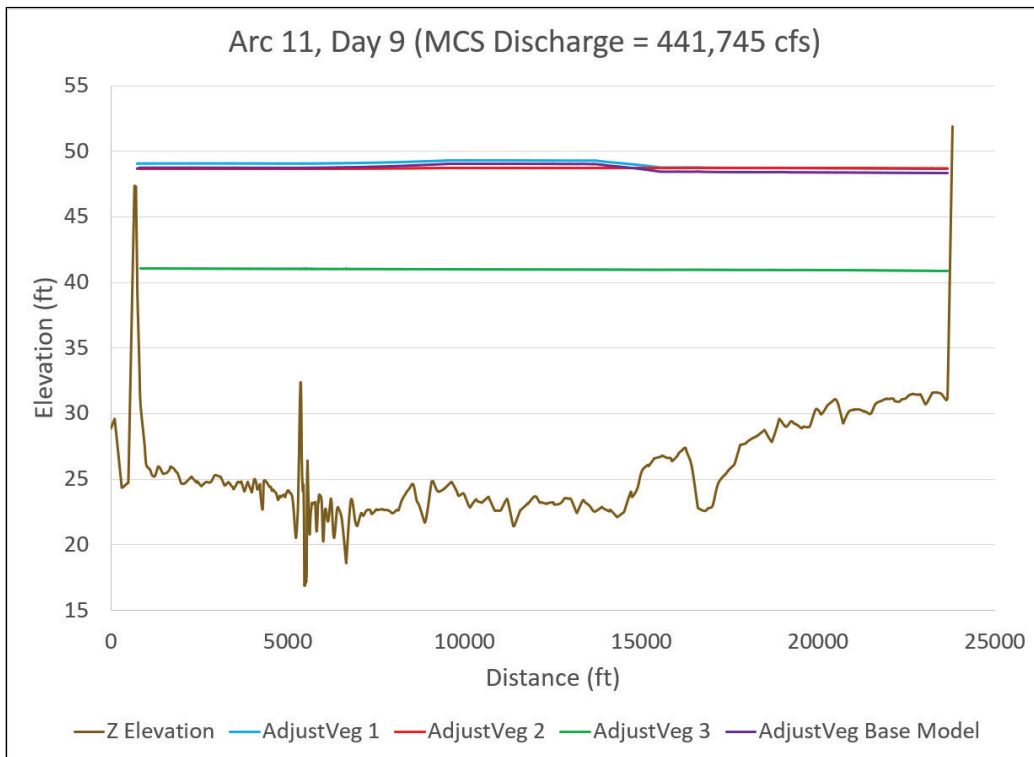


Figure 137. Arc 11 day 11 WSE profile.

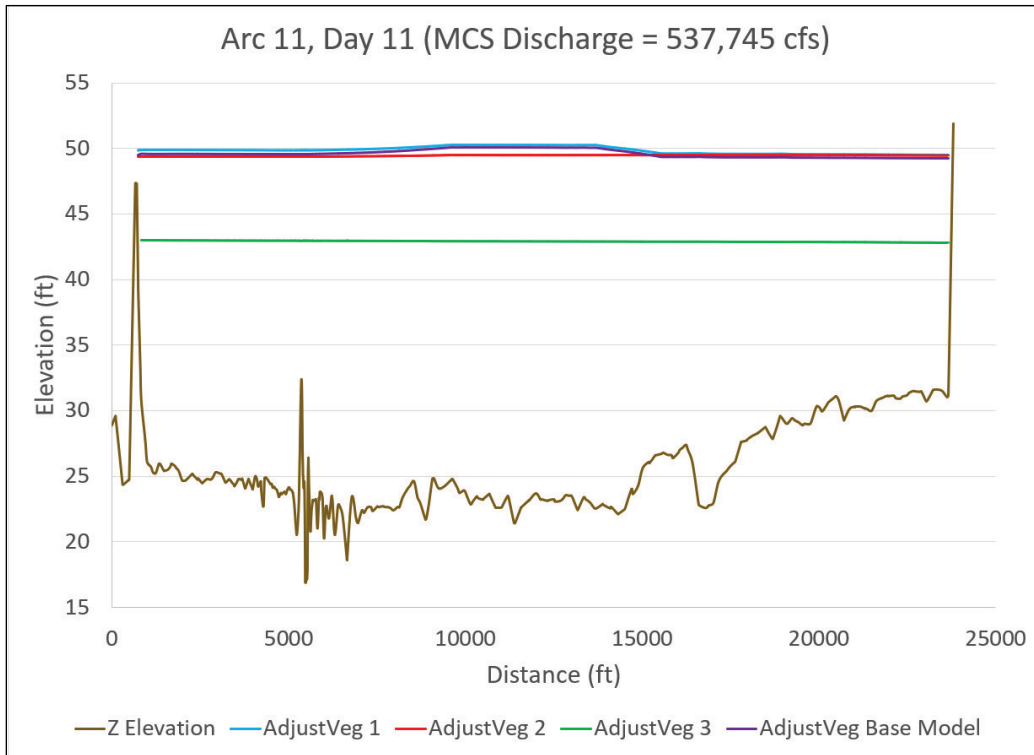


Figure 138. Arc 12 day 7 WSE profile.

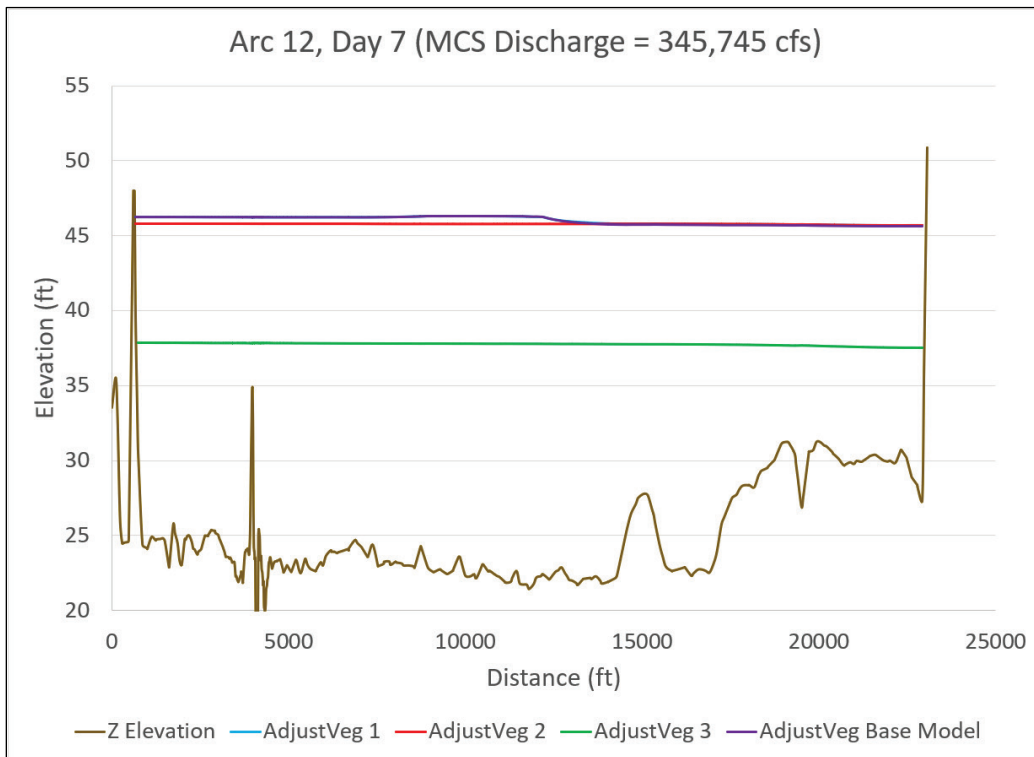


Figure 139. Arc 12 day 9 WSE profile.

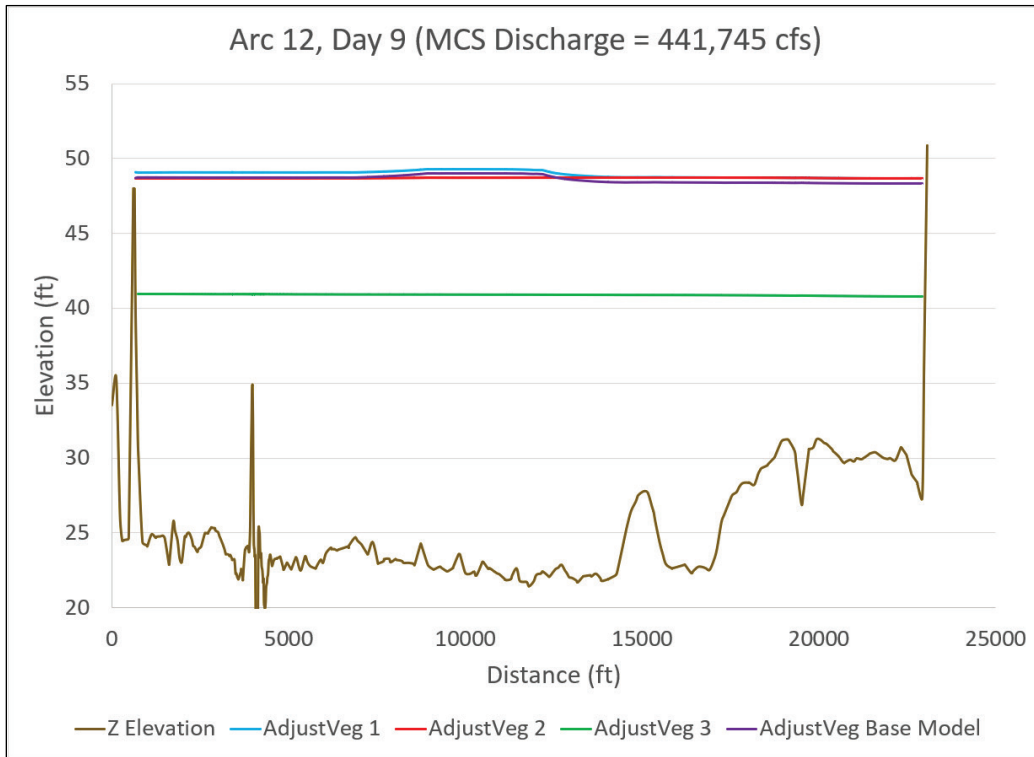


Figure 140. Arc 12 day 11 WSE profile.

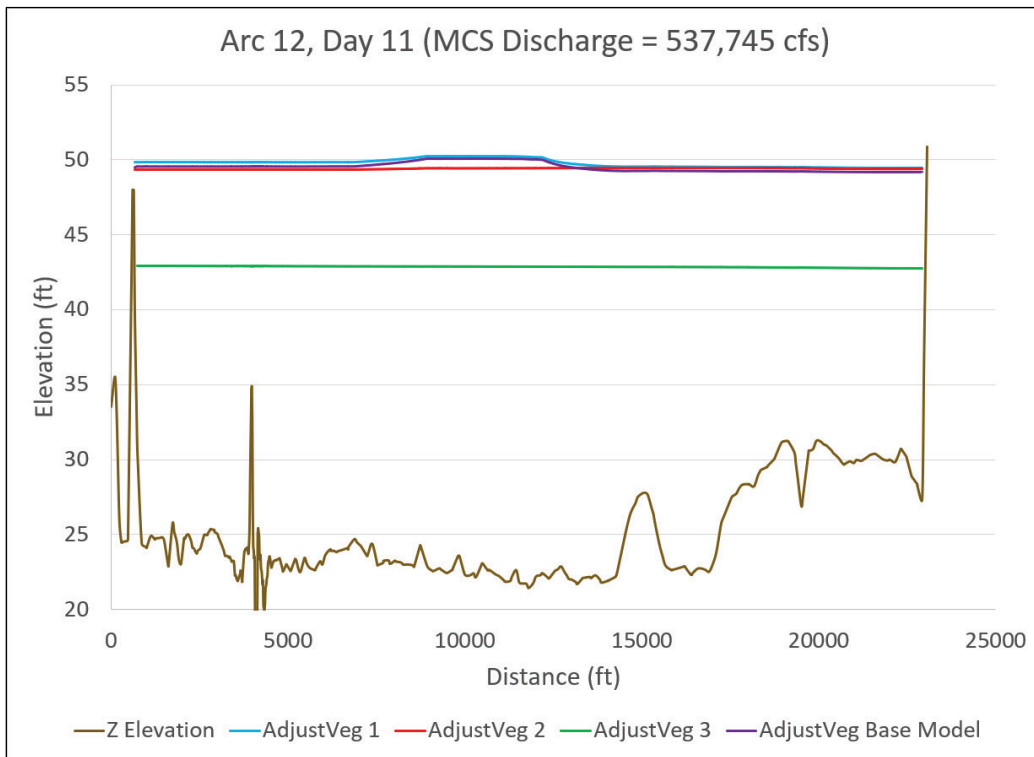


Figure 141. Arc 13 day 7 WSE profile.

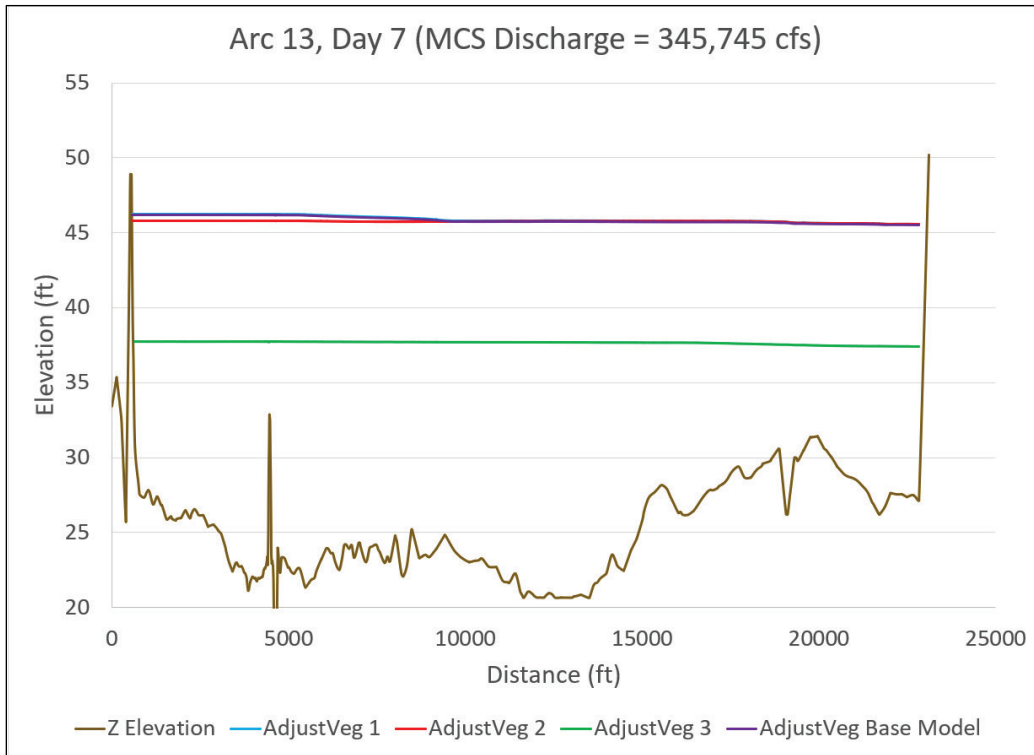


Figure 142. Arc 13 day 9 WSE profile.

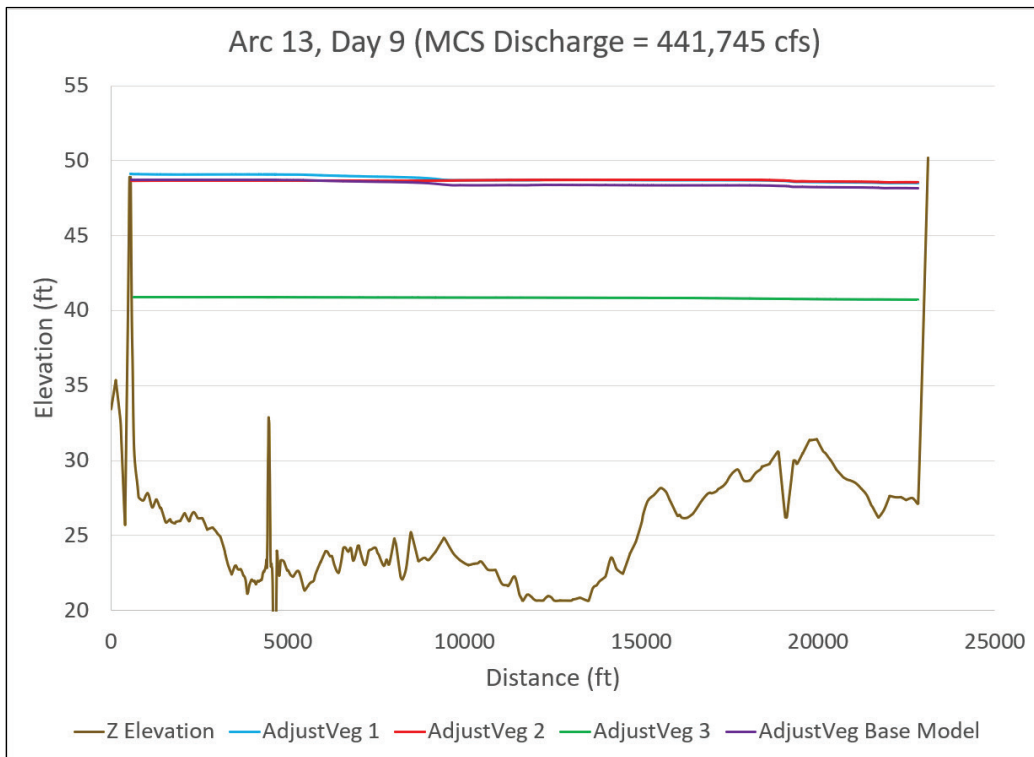


Figure 143. Arc 13 day 11 WSE profile.

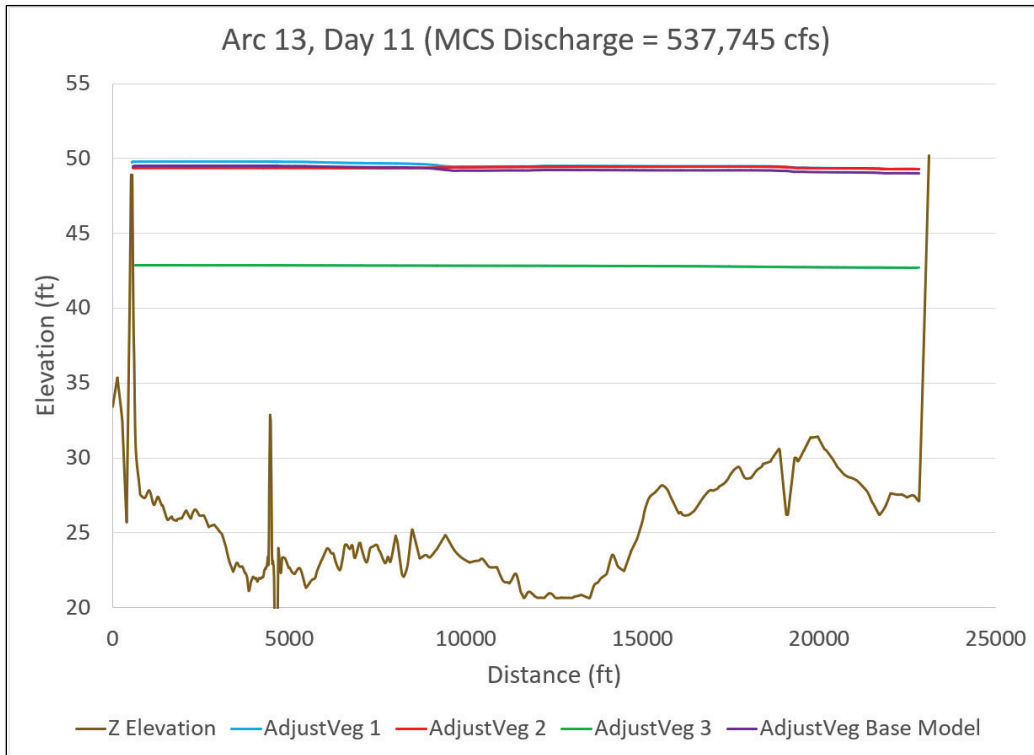


Figure 144. Arc 14 day 7 WSE profile.

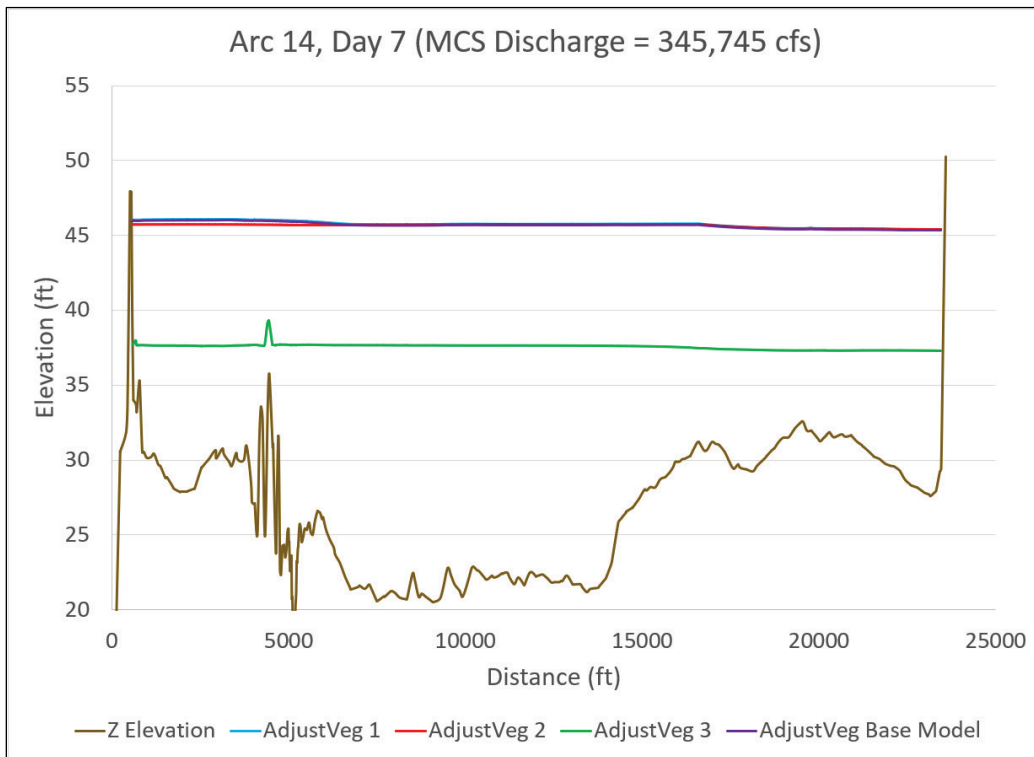


Figure 145. Arc 14 day 9 WSE profile.

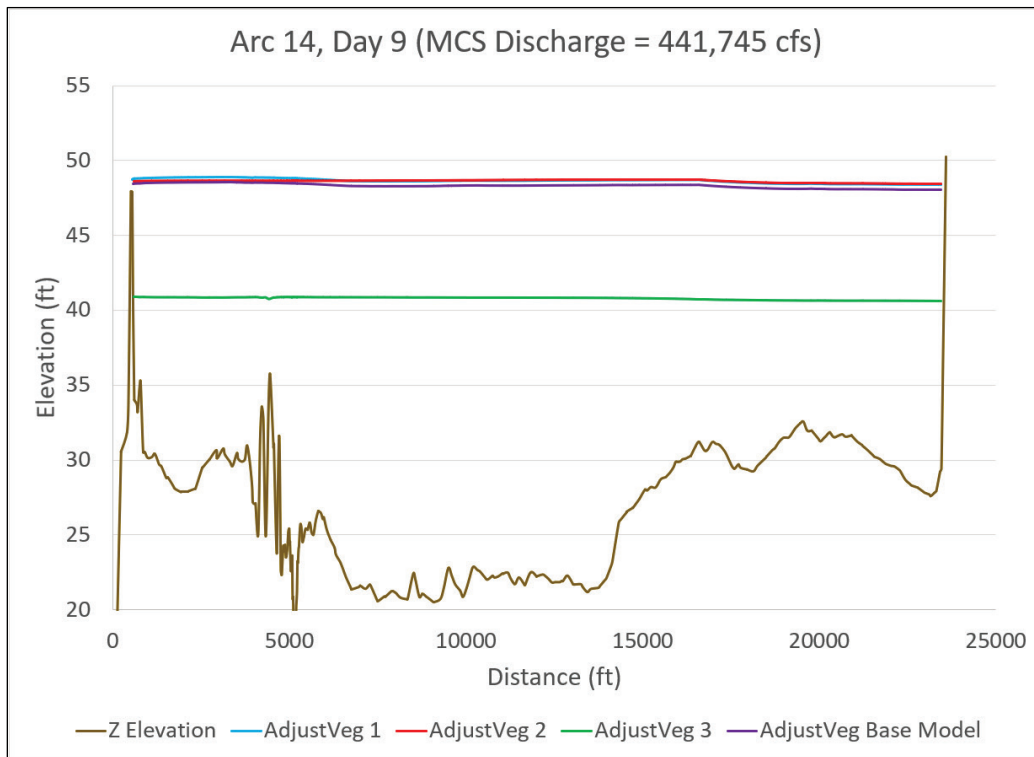


Figure 146. Arc 14 day 11 WSE profile.

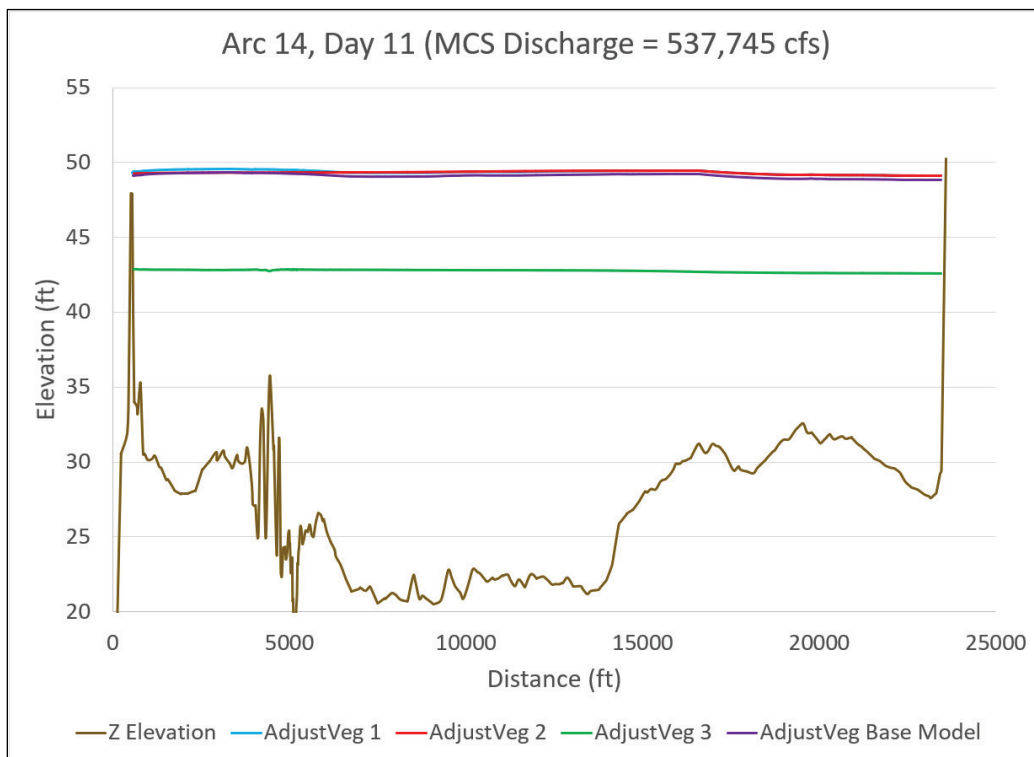


Figure 147. Arc 15 day 7 WSE profile.

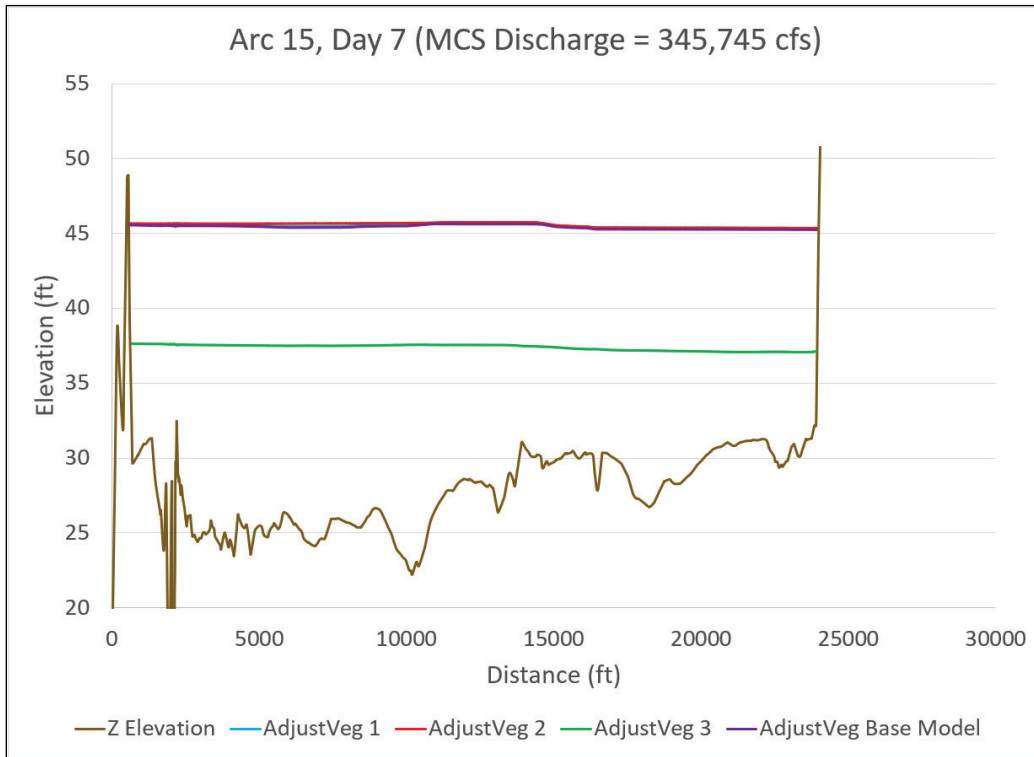


Figure 148. Arc 15 day 9 WSE profile.

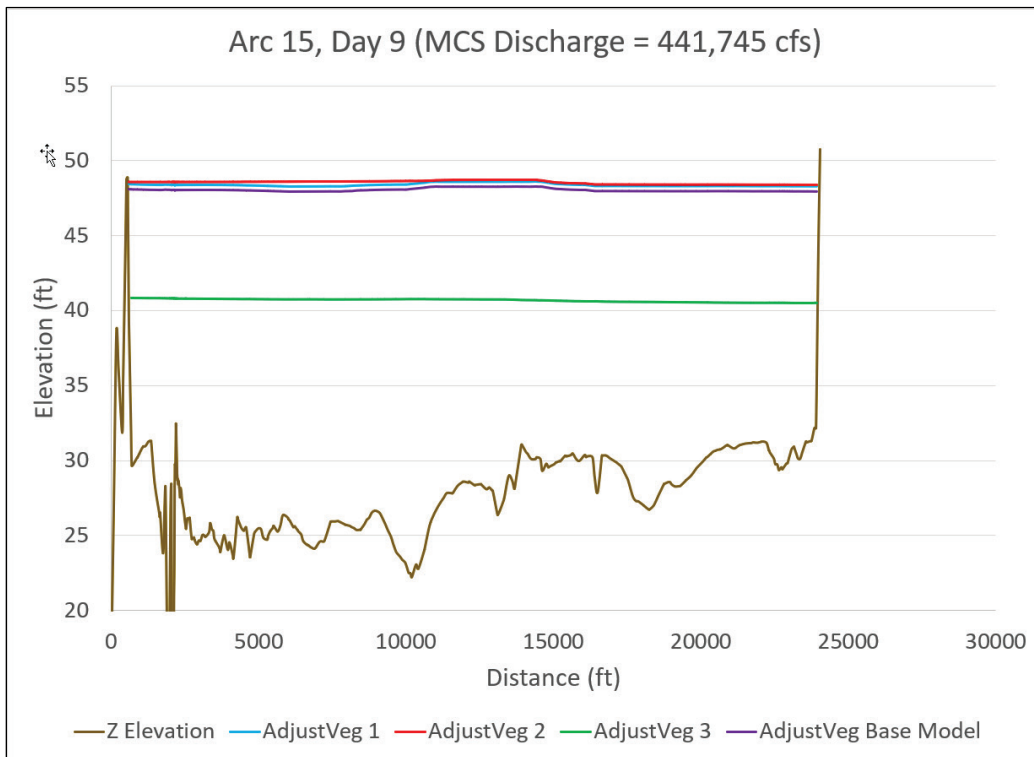
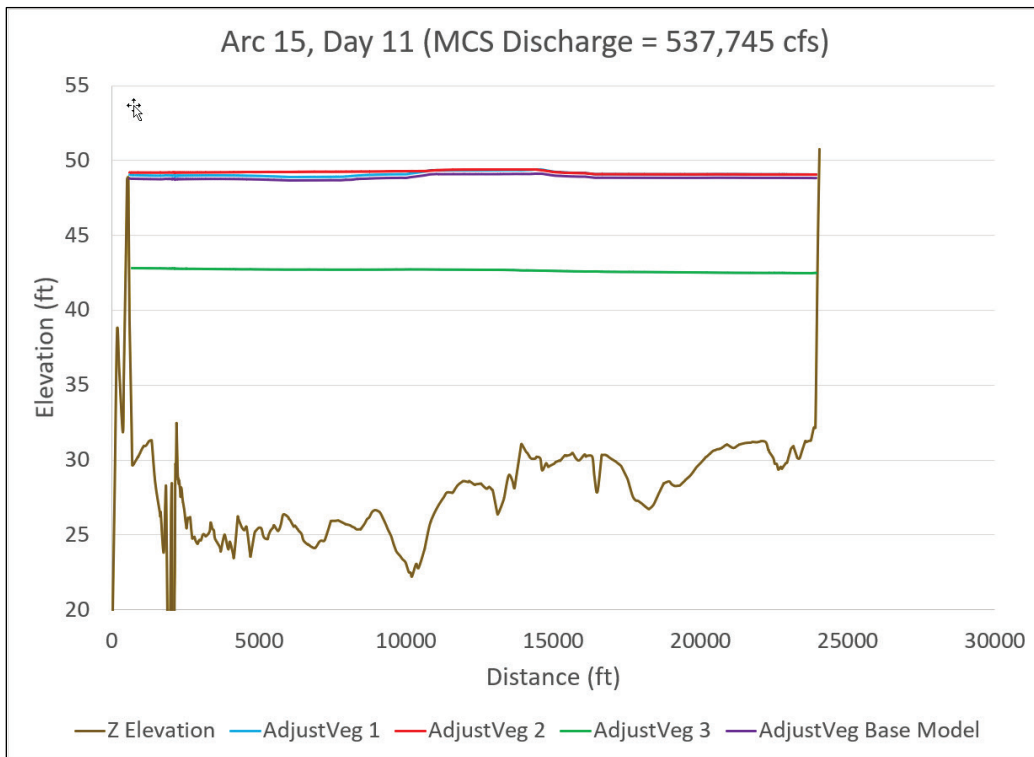


Figure 149. Arc 15 day 11 WSE profile.



## Acronyms and Abbreviations

2D	Two-dimensional
3D	Three-dimensional
AdH	Adaptive Hydraulics
EABPL	East Atchafalaya Basin Project
MVD	Mississippi Valley Division
MCS	Morganza Control Structure
NSE	Nash-Sutcliffe efficiency
MVN	New Orleans District
PDF	Project Design Flood
SPF	Standard Project Flood
URV	Un-submerged Rigid Vegetation
USGS	US Geological Survey
WSE	Water surface elevations
WAB	West Atchafalaya Basin
WABPL	West Atchafalaya Basin Project Levee

## Unit Conversion Factors

Multiply	By	To Obtain
acres	4,046.873	square meters
feet	0.3048	meters
miles (US statute)	1,609.347	meters
square feet	0.09290304	square meters

## REPORT DOCUMENTATION PAGE

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<b>14. ABSTRACT</b> The Morganza Floodway and the Atchafalaya Basin, located in Louisiana west of the Mississippi River, were evaluated using a two-dimensional Adaptive Hydraulics model. Prior to this study, Phase 1 and 2 model studies were performed that indicated that the existing floodway may not be able to pass the Project Design Flood discharge of 600,000 cubic feet per second due to levee overtopping. In this study, all elevations of exterior and interior levees were updated with current crest elevations. In addition, the Phase 3 effort evaluated the sensitivity of the floodway's flow capacity to variations in tree/vegetation density conditions. These adjustments in roughness will improve the understanding of the role of land cover characteristics in the simulated water surfaces. This study also provides a number of inundation maps corresponding to certain flows through the Morganza Control Structure.					
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