FTN Associates Ltd.
water resources / environmental consultants

HYDRAULIC AND WATER QUALITY MODELING OF PROPOSED RIVER REINTRODUCTION INTO MAUREPAS SWAMP (PO-0029)

FEBRUARY 18, 2022

HYDRAULIC AND WATER QUALITY MODELING OF PROPOSED RIVER REINTRODUCTION INTO MAUREPAS SWAMP (PO-0029)

Prepared for

Louisiana Coastal Protection and Restoration Authority
150 Terrace Avenue
Baton Rouge, LA 70802

Prepared by

FTN Associates, Ltd.
7648 Picardy Avenue, Suite 100
Baton Rouge, LA 70808

FTN No. R04835-2176-001

February 18, 2022

EXECUTIVE SUMMARY

A two-dimensional Delft3D hydrodynamic and water quality model was developed, calibrated and validated for the Maurepas swamp study area. The model was applied to simulate water surface elevations, velocity, total nitrogen, and total phosphorous under 20-day continuous diversion flows of 250, 1,000 and 2,000 cfs. The scenarios were applied for current (Year 0) conditions as well future (Year 50) conditions taking into account projected sea level rise, accretion and land subsidence.

The model geometry was based on a combination of channel cross-section field surveys (collected in 2004 and confirmed in 2018 at key locations) and 2012 LIDAR surveys. The model employs a structured grid with cell size varying from about 40 ft (12 m) in streams to over 600 ft (200 m) near the boundary at Lake Maurepas. Cell sizes for the interior swamps range from 40 ft to 160 ft. The model represents the project area using a two-dimensional computational grid composed of 1.3 million points.

The model was calibrated for water surface elevation and velocity using data collected in 2004 to represent normal conditions and Tropical Storm Matthew (2004) data to represent tropical storm conditions. The model was validated using 2020 normal conditions scenario at two Coastwide Reference Monitoring system (CRMS) gages. The final calibration for the normal condition used Manning's n value of 0.035 s/($m^{1/3}$) for roughness for the entire project area. For the tropical storm hydrologic conditions, the final selected values of Manning's n were 0.02, 0.035 and 0.2 s/($m^{1/3}$) for Lake Maurepas, the channels, and the swamp, respectively. The validation used the same roughness as the calibration. For model application to evaluate diversion scenarios, roughness values similar to the storm conditions were used, as they are appropriate for the elevated water levels of the scenarios. The model was not calibrated for nutrients because existing nutrient concentrations (i.e., without the diversion) are assumed to represent background concentrations. Current conditions in the study area do not provide a spatial or temporal gradient of nutrient concentrations that would allow calibration of nutrient parameters. Instead, nutrient input parameters for the model were selected from an extensive literature survey and consultation with the CPRA Technical Advisory Group.

The following are the findings of this study:

- The <u>highest</u> water levels will occur in Hope Canal as it exits I-10 bridge:
    - Year 0: Diversion flow of 250, 1000 and 2000 cfs raises water level by 0.3, 1.3 and 1.9 ft, respectively,
    - Year 50: Diversion flow of 2000 cfs raises water level by 0.6 ft.
- The <u>average</u> water levels in the swamp are affected as follows:
    - Year 0: Diversion flow of 250, 1000 and 2000 cfs raises water level by 0.1, 0.7 and 0.9 ft, respectively,
    - Year 50: Diversion flow of 2000 cfs raises water level by 0.2 ft.
- Water levels <u>near the West Shore Lake Pontchartrain (WSLP)</u> drainage structures:
    - Year 0: Diversion flow of 2000 cfs raises water level by less than 0.3 ft.
    - Year 50: Diversion flow of 2000 cfs raises water level by 0.1 ft.
- Distribution of the diversion flow changes with its magnitude:
    - 250 cfs diversion rate (Year 0):
        - 84% flows through Dutch Bayou to Lake Maurepas.
        - 12% flows towards the Reserve Relief Canal.
        - insignificant flow towards the Blind River.
    - 1,000 cfs diversion rate (Year 0):
        - 46% flows through Dutch Bayou to Lake Maurepas.
        - 25% flows towards the Reserve Relief Canal.
        - 18% flows towards the Blind River.
    - 2,000 cfs diversion rate (Year 0):
        - 32% flows through Dutch Bayou to Lake Maurepas.
        - 26% flows towards the Reserve Relief Canal.
        - 29% flows towards the Blind River.
    - 2,000 cfs diversion rate (Year 50):
        - Due to significant inundation, the diversion flow has more opportunity to overtop the stream banks. Therefore, only 6% of the diversion flow is channelized through Dutch Bayou (2,000 cfs diversion).
- The shallow and relatively slow flow through the swamp allows for nutrients to be removed from the water column before the water reaches Lake Maurepas via

> Dutch Bayou and Reserve Relief Canal. By the time the Mississippi River water reaches Lake Maurepas, it has lost about 54% of its TN and 35% of its TP (Year 0 conditions). Predicted concentrations of TN in the southern end of Lake Maurepas correspond to nitrate concentrations that are much lower than observed concentrations in Lake Pontchartrain that led to increased algae concentrations in 2008 and 2011 after opening the Bonnet Carré Spillway.

Based on the model projection simulations, the proposed diversion of Mississippi River water into the Maurepas swamp is expected to provide beneficial freshening and nutrients to a large area of swamp, without causing large increases in nutrient concentrations in Lake Maurepas.

A version of this report documenting the modeling efforts above was submitted to CPRA on January 26, 2021. Subsequently, CPRA requested preliminary evaluation of drainage of the polders created by the intercepting diversion canal alignment. The modeling and analysis pertaining to polder drainage evaluation is added as an Appendix G to this report.

The model results showed that the construction of the diversion canal isolates region to its west reducing drainage potential of the region. The impact is greater on the area east of LA-641 than the west area. The presence of elevated water levels north of I-10, reduces capacity of the highway culverts to drain the polders. Under the existing conditions, the difference in water levels due to the 2- and the 25-yr rainfall is apparent for about 4 days. Under the with-project conditions, the difference in water levels due to the 2- and the 25-yr rainfall is apparent for over 15 days.

To improve drainage of these polders, especially the west polder, the effect of installing additional (32, 8 and 20) Lateral Release Valves (LRVs) along the banks of the proposed diversion canal was evaluated. The analysis showed that the combined flow through 32 LRVs is about 4 times that through the 8 LRVs at the peak. The culverts flow partially under the water levels predicted for the corresponding scenarios. Generally, a lot of flow from the rainfall drainage comes into Hope Canal via LRVs on the west bank. Most of it exits north through Hope Canal and only some exits through the LRVs on the east bank. The east bank culverts are of no significant benefit to drain water out to east. The model scenarios with 32 LRVs (16 west + 16 east) and 20 LRVs (16 west + 4 east) have similar drainage benefit to the west polder. In general, introduction of LRVs improves drainage and reduces inundation of the polders.

TABLE OF CONTENTS

TABLE OF CONTENTS (CONTINUED)

LIST OF APPENDICES

LIST OF TABLES

LIST OF FIGURES

1.0 INTRODUCTION

The proposed River Reintroduction into Maurepas Swamp (PO-0029) project (the Project) located near Garyville, Louisiana, will divert flow from the Mississippi River (MR) to the Maurepas Swamp wetlands (Figure 1.1). In 2014, URS provided 95% level design of the proposed PO-0029 project to the Coastal Protection and Restoration Authority (CPRA) of Louisiana (URS 2014). The project consists of a gated intake structure at the river capable of diverting 2,000 cfs of river water, a large sand settling basin, and a long, banked diversion conveyance channel. Approximately halfway along the conveyance channel, just north of US Highway 61, the channel follows the existing Hope Canal alignment to distribute the diverted water into the wetlands on the north side of Interstate 10. The proposed diversion channel extends from the Mississippi River to its end approximately 1,000 ft north of its crossing with Interstate Highway I-10. The diversion channel has a variable cross-section along its way. The longest segment between Highway 61 and I-10 has a 60 ft wide bottom and 1V:5H side slope. The channel invert is -7 ft and -8 ft, NAVD88 at Highway 61 and I-10, respectively. The proposed project also includes closing the existing culvert crossings under I-10 between LA 641 and Mississippi Bayou, to prohibit backflow from the diversion into the swamp between I-10 and Highway 61. The design also proposes adding gaps in the railroad embankment along the west bank of Hope Canal. For details, the reader is referred to the 95% Level Design Report (URS 2014).

To support the hydraulic design of the proposed diversion and to evaluate its effect on swamp hydrology, URS developed a two-dimensional (2D) ADvanced CIRCulation (ADCIRC) Model. URS also developed a one-dimensional (1D) Storm Water Management Model (SWMM) of the Garyville-Reserve drainage system to evaluate effects of the projected water levels in the swamp due to the project, on the interior drainage.

Figure 1.1 Maurepas swamp hydraulic modeling study area.

The hydrodynamic modeling performed by URS for the 95% level design did not include modeling the transport of nutrients introduced from the Mississippi River diversion water throughout the swamp. The purpose of the modeling efforts outlined in this document is to develop a water quality model (two-dimensional Delft3D) for the proposed project to simulate fate and transport of nutrients carried by the diverted water. FTN Associates (FTN) completed this modeling study as a sub-contractor to AECOM Technical Services and then as a sub-contractor to Volkert, Inc.

2.0 STUDY OBJECTIVES

The objectives of the modeling study were as follows:

1. Develop a numerical model capable of simulating water surface elevations, velocities, discharge, salinity, total nitrogen (TN) and total phosphorous (TP) throughout the receiving swamp when the diversion flow is introduced in the system.
2. Apply model to predict above parameters for the 250, 1,000 and 2,000 cfs diversion inflow throughout the Maurepas swamp.

3.0 MODELING PROGRAM SELECTION AND DESCRIPTION

The study area is an extensive forested wetland surrounding Lake Maurepas in the upper reaches of the Pontchartrain estuary. The area is influenced by diurnal tides entering from Pass Manchac connecting Lake Maurepas to Lake Pontchartrain. The study area includes several natural and man-made channels that carry flow into and out of the swamp while distributing it in the swamp wherever low banks are present. For the purpose of the study, it is appropriate to assume that the dominant velocities in the system are in the longitudinal and transverse direction (two dimensions). Due to the relatively shallow water depths, the velocities and accelerations in the vertical direction (the third dimension) are negligible and the flow can be assumed to be vertically well-mixed. This assumption allows us to apply a two-dimensional (2D) model instead of a three-dimensional (3D) model. A 3D model for the study area would be extremely computationally intensive resulting in prohibitively long simulation times and would add little to the accuracy of the results. On the other hand, an over-simplified one-dimensional (1D) model would not be adequate for the study purpose. Therefore, a two-dimensional depth-averaged (2D) model is appropriate for this study.

Various public domain and commercial/proprietary computer software are available for 2D, vertically averaged hydrodynamic transport modeling. These models solve the hydrodynamic and constituent transport equations using either a structured or an unstructured computational mesh.

The structured-grid models use rectangular or square elements. These models are simpler in parallel programming implementation because they employ finite-difference schemes to solve governing equations and different portions of the grid can be distributed to multiple processors for optimal balancing of the computational load. Additionally, finite difference schemes do not suffer from mass conservation problems often inherent in the finite element schemes of unstructured grids. However, the accuracy in the complex edge-of-the-water geometry in structured-grid models may not be as good as in unstructured-grid models. The unstructured models (finite element or finite volume-based), on the other hand, allow elements of various shapes (line, triangle, or quadrilateral), which makes it possible to fit elements more closely to the topographic features. Further, the unstructured mesh allows variation of element size in a single mesh enabling creation of a denser mesh where more details are necessary. However, implementation of finite-element models is not as straightforward as finite-difference models. This is mainly due to approximation of the fields within each element with a simple linear, quadratic or polynomial function with finite number of degrees of freedom.

The following are some of the modeling programs commonly used to model 2D, vertically averaged hydrodynamics:

1. RMA-2 model (unstructured mesh) by Resource Modelling Associates, Inc;
2. ADCIRC from the University of North Carolina at Chapel Hill (unstructured mesh);
3. MIKE-21 from the Danish Hydraulic Institute (unstructured mesh); and
4. Delft3D from Deltares (structured mesh).

Although the first two options can better represent the study area consisting of broken swamp, lake, channels and bayous, the Delft3D option was selected for this study because it has been widely applied in south Louisiana and is used for the Louisiana Coastal Master Plan. Delft3D is highly scalable on High Performance Computing (HPC) infrastructures. Equally important is the fact that Delft3D with its DELWAQ module can model a wide variety of water quality parameters including secondary processes. DELWAQ can model 18 independent principal substances with over 20 different sub-substances. It has been applied in studies involving eutrophication, dissolved oxygen depletion, contaminated sediment, and temperature impacts of point sources. A particularly useful feature of DELWAQ is its ability to apply user-defined spatially variable, depth dependent decay rate constants for the constituents of interest.

4.0 METHODOLOGY

FTN developed and applied Delft3D model version 4.02.03 (Deltares, 2018) to predict the tidal circulation and the transport of suspended nutrients. Delft3D FLOW module simulates water levels and velocity driven by boundary conditions of tides and currents. The output from DELFT3D FLOW is used in DELWAQ to simulate the advection and dispersion of nutrients.

The Delft3D FLOW module utilizes a robust numerical finite-difference scheme where model results are computed for a horizontal staggered grid. The water level are determined in the center of a continuity cell and the velocity components are computed perpendicular to the grid cell faces. Delft3D can be operated in a 2D (vertically averaged) or a 3D mode. In the present application, Delft3D is used in 2D mode.

To begin with, a hydrodynamic model of the study area was developed and calibrated. The simulated hydrodynamics (water surface elevations and velocities throughout the study area) were then used to drive the transport of nutrients introduced by the MR inflow. Nutrients were simulated as total nitrogen and total phosphorus rather than individual species of nutrients (e.g., ammonia nitrogen, nitrate nitrogen, etc.).

5.0 DATA COLLECTION TO SUPPORT MODELING

The following topographic survey data and hydraulic monitoring data were used in this modeling study.

5.1 Topographic Data

The topographic field data are used to develop the model geometry which is a digital representation of the terrain. Specifically, topographic data were required to develop model geometry for Lake Maurepas, major streams and the swamp area.

Lake Maurepas bathymetry was obtained from surveys performed by USGS in 2002. Channel cross-section data were available at 29 locations on streams in the swamp north of I-10 (URS, 2005). To evaluate whether the cross-sections have changed significantly over the years, new topographic surveys were collected in April 2018 at six of the 29 locations with cross-sections (MPH 2018). The original 29 and the new six survey locations are shown in Figure 5.1. Figures 5.2 through 5.4 compare the old and the new cross-sections. The comparison shows that the previously surveyed cross-sections have not changed significantly in terms of cross-sectional area and can be used for this study.

It would have been prohibitively expensive to collect topographic field survey data in the forested swamp. Therefore, the LIDAR data from 2012 were used. The LIDAR elevations in the main swamp north of Interstate 10 were much higher than those generally found in this region. Therefore, upon the recommendation of the Technical Advisory Group[1], the marsh floor elevation was capped at 1.0 ft, NAVD88. The revised topographic contours are show in Figure 5.5.

[1] Prof. Gary Shaffer, Southeastern Louisiana University; Prof. Richard Keim and Prof. Jim Chambers, Louisiana State University; and Dr. Ken Krauss, USGS.

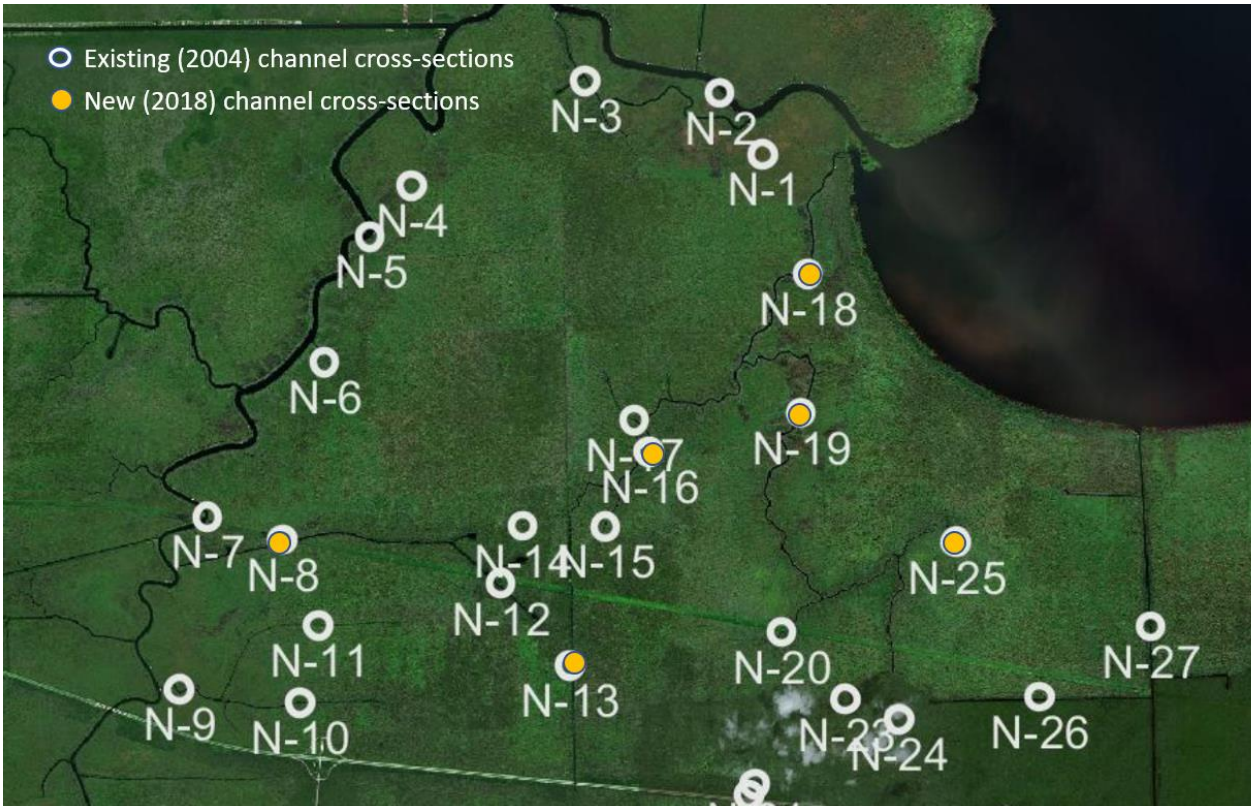
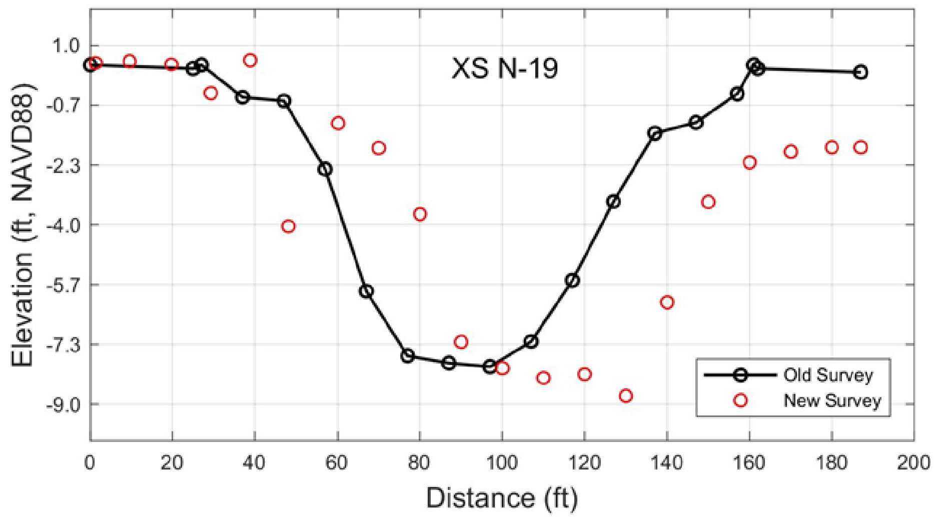


Figure 5.1. Locations of existing (2004) and new (2018) channel cross-section field surveys.



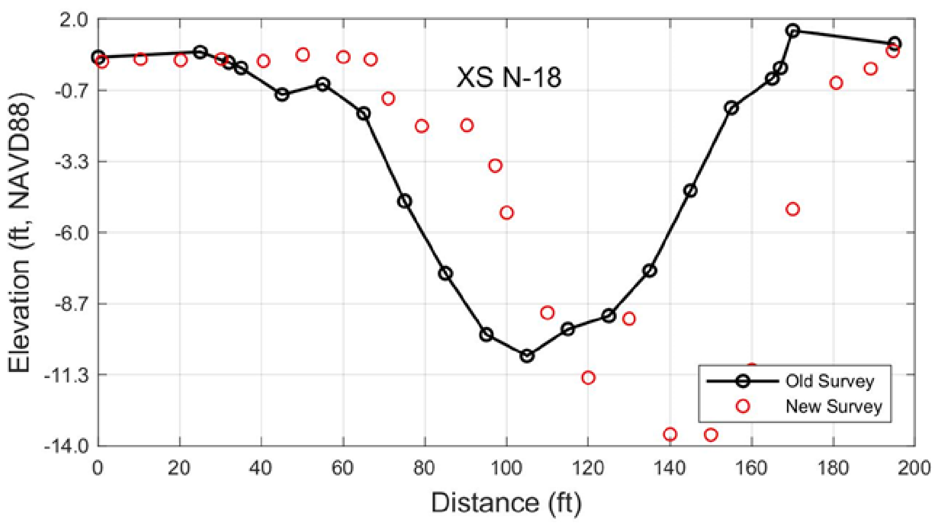
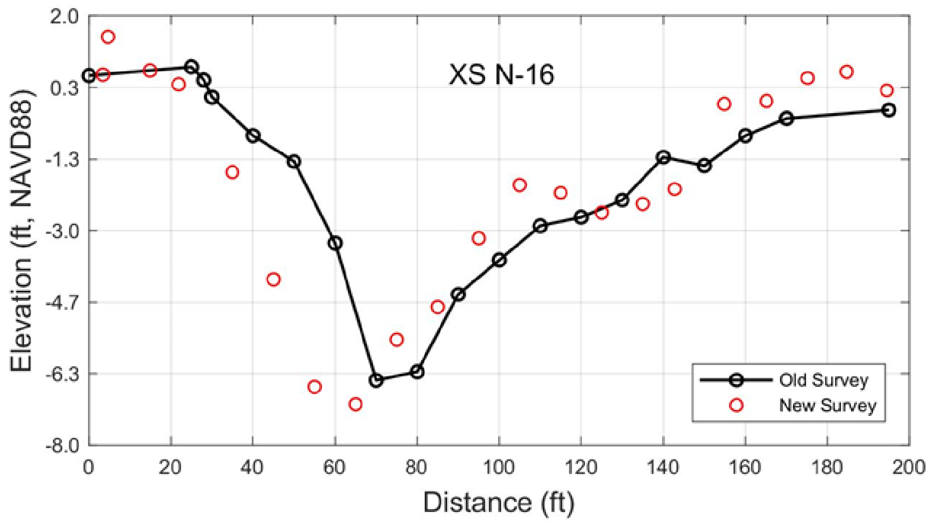


Figure 5.2. Comparison of old (2004) and new (2018) channel cross-sections at N-19 and N-18.



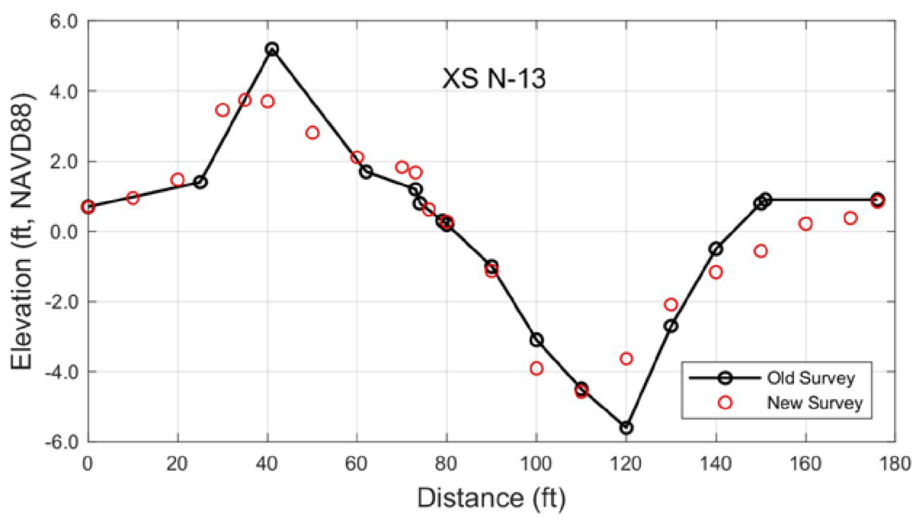


Figure 5.3. Comparison of old (2004) and new (2018) channel cross-sections at N-16 and N-13.

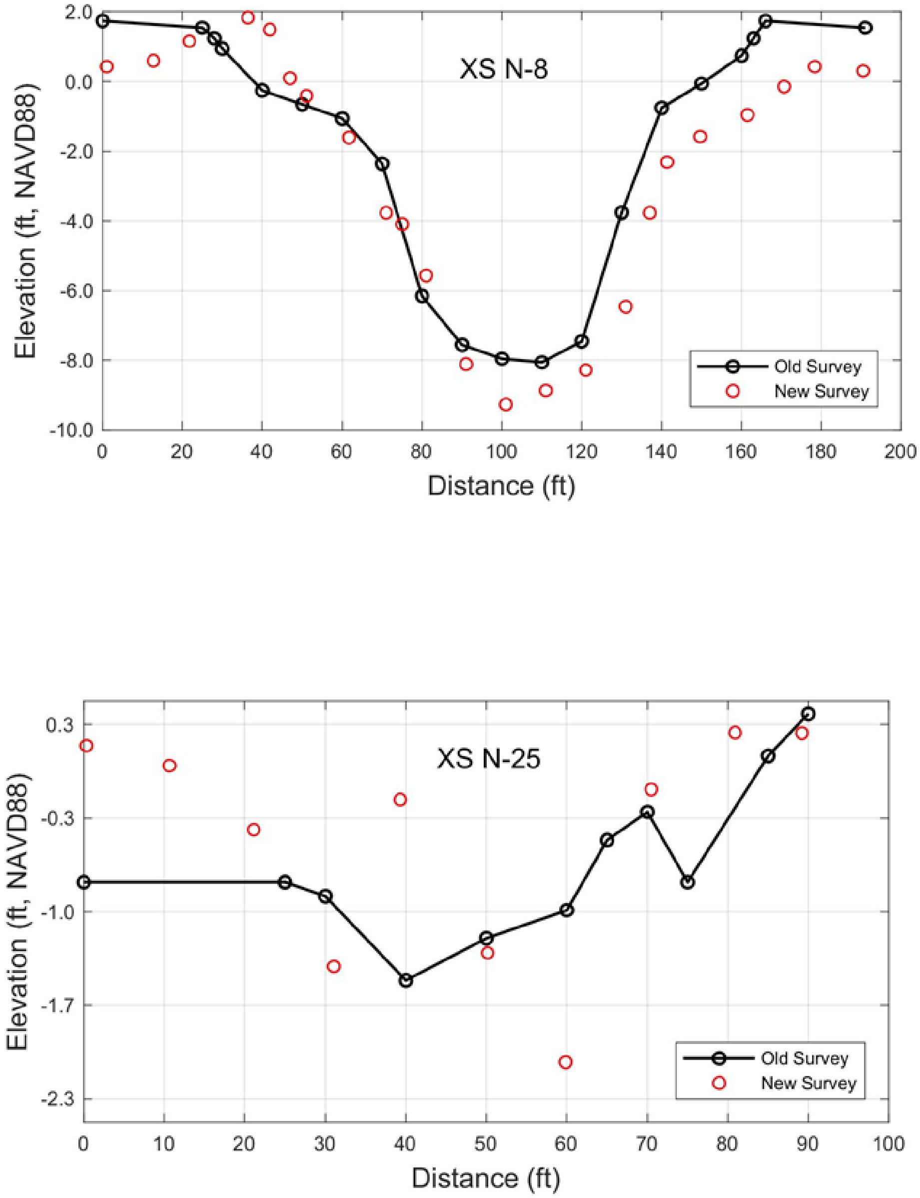


Figure 5.4. Comparison of old (2004) and new (2018) channel cross-sections at N-8 and N-25.

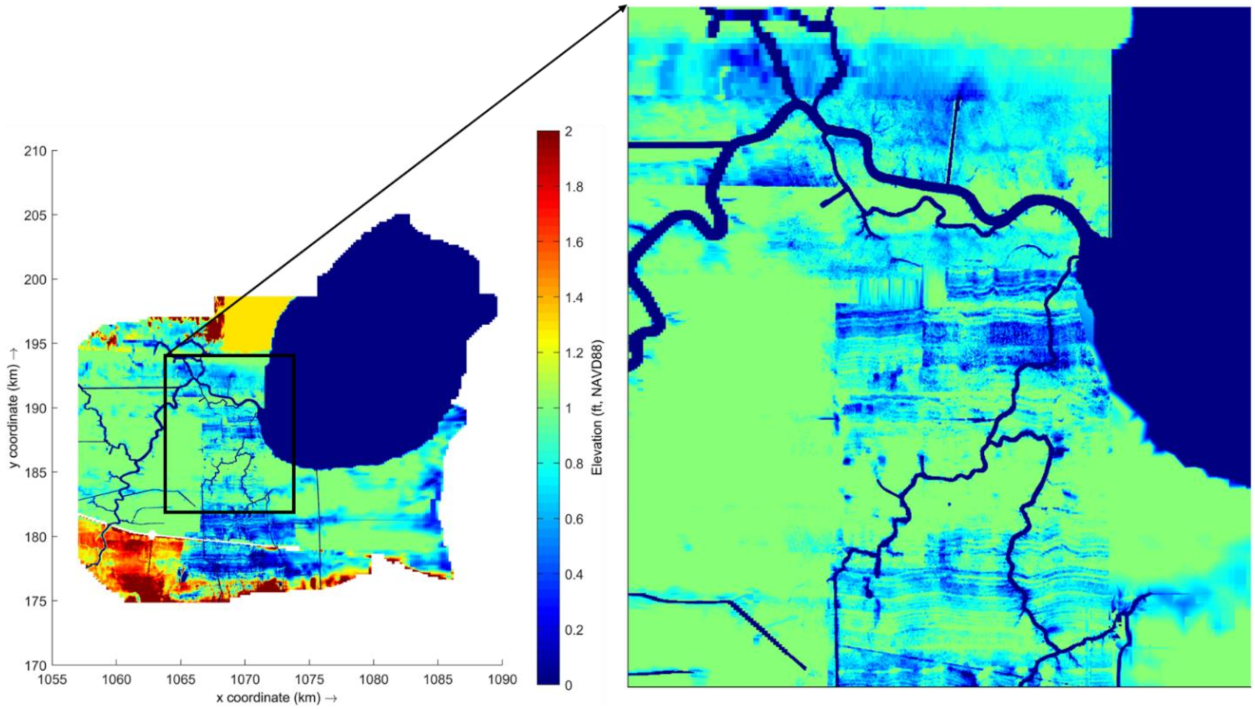


Figure 5.5. Delft3D model bathymetry using topographic contours from 2012 LIDAR data. Swamp floor elevation capped at 1.0 ft in the region shown by the inset.

5.2 Hydraulic Monitoring Data

Hydraulic monitoring data needed for modeling typically consists of time series of water surface elevations, velocity or discharge. These data are used to specify boundary conditions and for calibration/validation of the model. Since the re-surveyed primary channels were found to have no major change in the cross-sectional area, the previously collected hydraulic monitoring data (URS 2006) were judged to be appropriate for use in this study. The hydraulic monitoring gage locations are shown in Figure 5.6. Water surface elevations were collected at all locations. Velocity was collected only at location S-9.

Figure 5.6. Locations of hydraulic monitoring gages.

6.0 MODEL GEOMETRY DEVELOPMENT

The model geometry is a mathematical representation of the study area topography. The model domain size was selected such that the model boundary conditions are specified far away from the area of interest. The domain is represented by a two-dimensional computational grid composed of 1.3 million points. The model grid is most refined (cell size 12 m) at Hope Canal, Mississippi Bayou, Relief Canal, Dutch Bayou, and the interior channels connecting them, where detailed hydrodynamic and nutrient dynamics are expected, and becomes coarser (cell size 200 m) towards the domain boundary at Lake Maurepas. Cell sizes of the model grid for the interior swamps range from 12 to 50 m of depending upon location and importance for nutrient dispersal. Figure 6.1 shows the model grid for existing conditions.

The bathymetry of the primary channels was assigned using previously collected channel cross-sections. The bathymetry of the swamp areas was assigned using the LIDAR data. Figure 6.2 shows the model bathymetry. It should be noted that the model grid bathymetry does not capture numerous rivulets and small open water areas that are widespread in the swamp; rather, it represents the overall relief in the terrain. This is a limitation of the LIDAR data that were used for the bathymetry.

To apply the model for the alternatives analysis, the geometry was modified to include the proposed West Shore Lake Pontchartrain (WSLP) project and the diversion channel as described in Section 8.

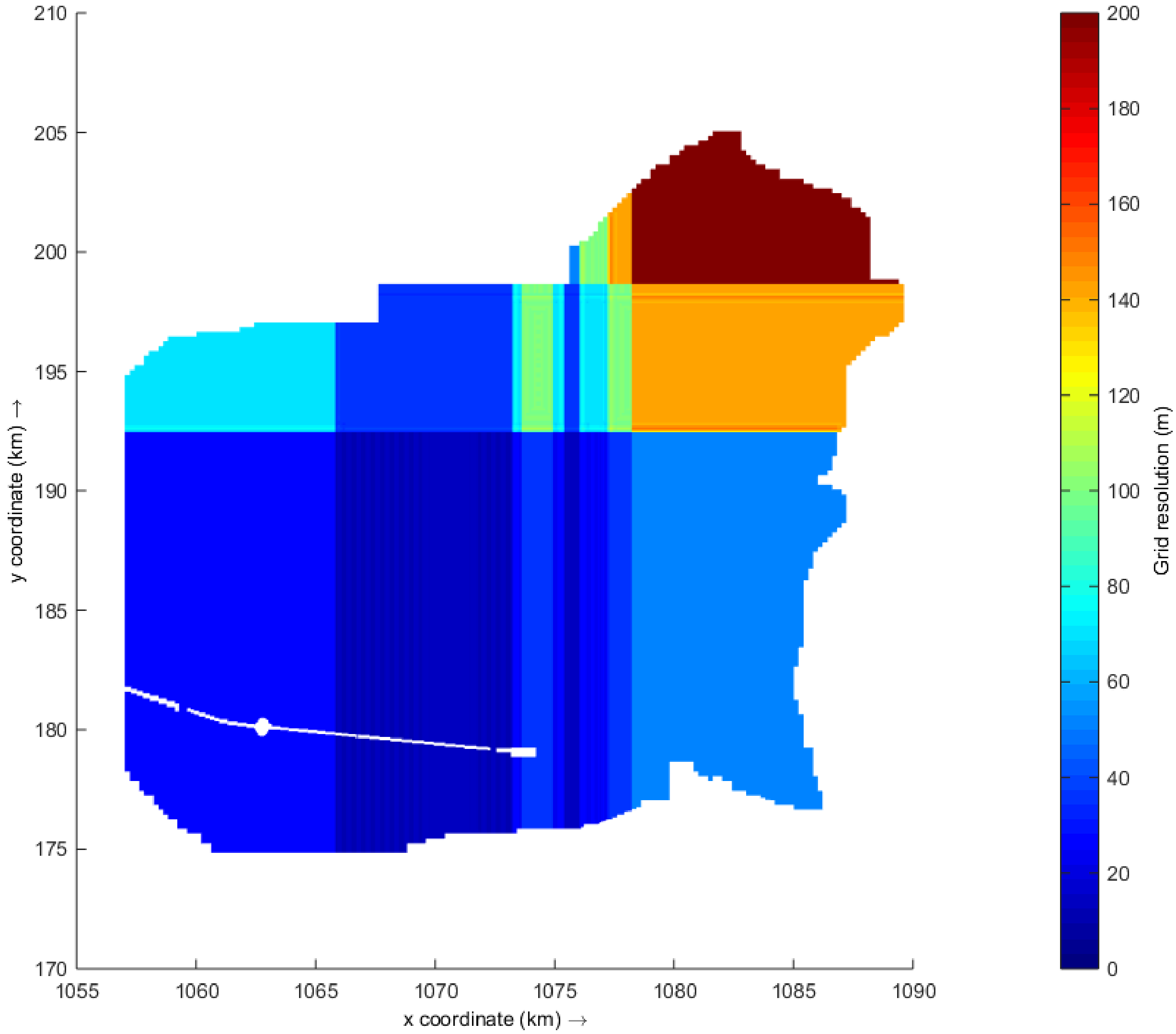


Figure 6.1. Maurepas swamp Delft3D model grid resolution.

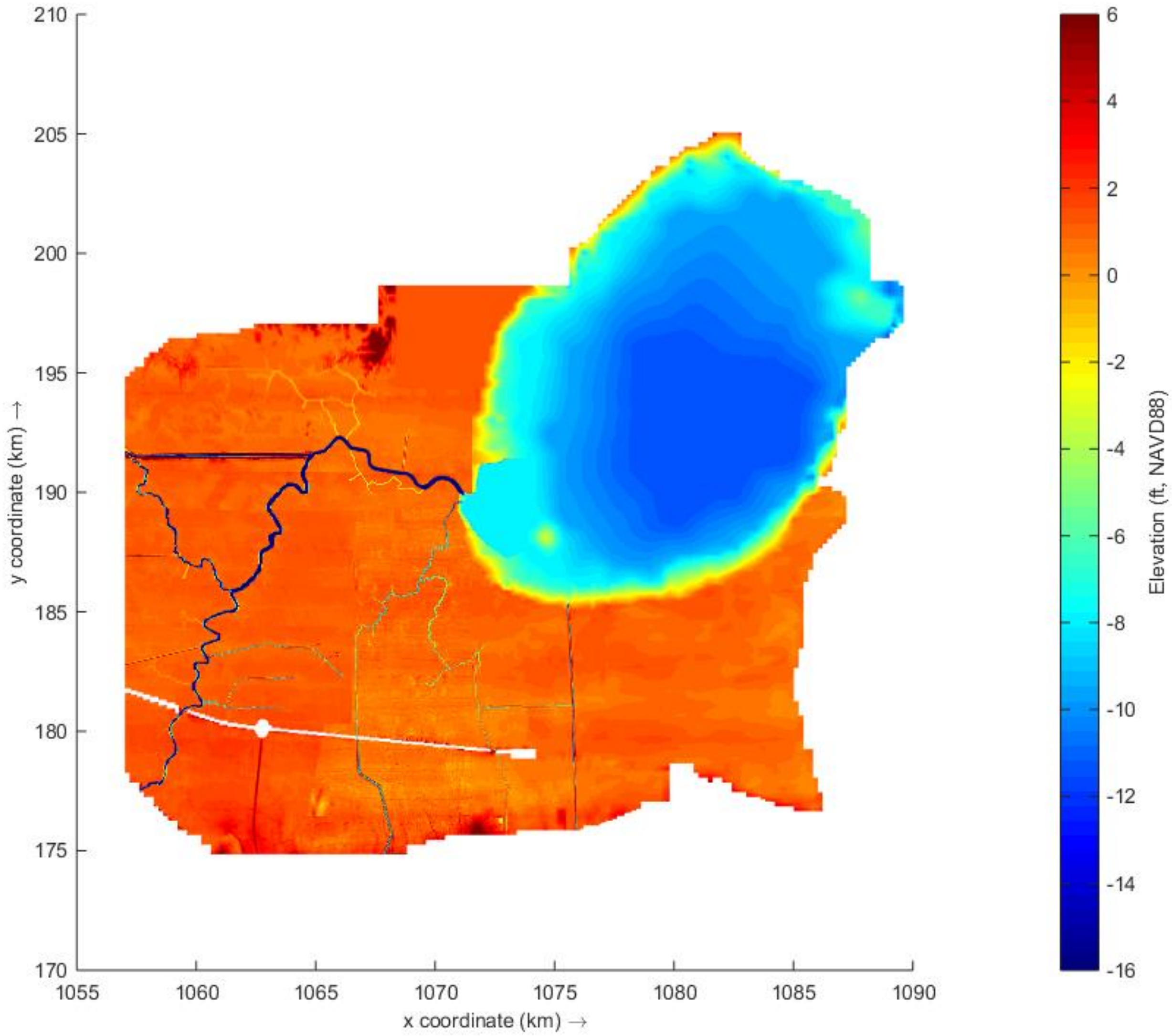


Figure 6.2. Maurepas swamp Delft3D model bathymetry.

7.0 MODEL CALIBRATION AND VALIDATION

Model calibration is an iterative process where model coefficients are systematically varied or "tuned" through a series of simulations to improve model's reproduction of observed data. The range of values used when varying model coefficients should be limited to that which reasonably reflects the physical conditions and processes during the simulation periods. If unreasonable values are required to calibrate a model, it should serve as a warning that there is a process or feature that is not adequately represented in the model.

Model validation involves simulating one or more independent sets of conditions, using model coefficients determined in the calibration process, to assess how well the calibrated model can reproduce observed data for those independent conditions. The hydrologic conditions represented by the calibration and validation periods should be similar. For example, a model calibrated for average conditions should not be validated with hurricane conditions. The primary purpose of the model calibration and validation exercise is to provide greater confidence in the model when it is used to predict the system response to project scenarios.

For the present study, independent observed data were available for two periods. The first period was from December 26, 2003, through January 1, 2004, and represents normal hydrologic conditions. The second period was from October 4, 2004, through October 18, 2004, and represents tropical storm conditions (Tropical Storm Matthew). The two periods represent two distinct hydrologic conditions. Therefore, instead of using them as a calibration and a validation data set, both data sets for calibration. The water movement in a forested swamp under high water level conditions during a tropical storm can be quite different from the water movement under normal conditions, due to the additional frictional drag presented by the tree trunks at high water levels.

7.1 Model Calibration

The model parameters involved in calibration are typically coefficients related to the simulation of physical processes in the model (e.g., friction coefficients in flow simulation).

However, model calibration may also involve variation of other parameters that have uncertainty associated with them, such as model geometry or boundary conditions (driving forces).

The model for this study was calibrated and validated for water surface elevation and velocity through a series of Delft3D FLOW simulations. The calibration was accomplished mainly through improvement in geometry of the channels and tuning the roughness coefficient to improve the match of the model predictions to observed values.

The calibration simulations were performed by using measured tidal water surface elevations at the Pass Manchac boundary. For the normal and tropical storm conditions, Pass Manchac is the most important boundary condition that drives the water movement in the study area. The inflows at the other major boundaries such as Blind River, Amite River, Hope Canal, and Reserve Relief Canal were not measured during the data collection period. However, they have much less influence on the swamp water levels under the conditions used for calibration and validation. Therefore, these inflows were not included as boundary conditions during calibration. These inflows affect local water levels where they enter the study area. The calibration charts comparing predicted to observed water surface elevations and velocity under the normal conditions are shown in Appendix A. The tidal elevations at Pass Manchac are included in the figures for reference as they are the most important boundary conditions driving water movement in the system. After a series of trial runs, a uniform Manning's roughness of 0.035 $s/(m^{1/3})$ was applied for the entire model domain. In general, the figures indicate a good model performance. For the normal conditions modeling period, the statistical measures of correlation coefficient (R^2) and root-mean-square error (RMSE) shown on the figures indicate a good model performance. The model performance is better at the gages in the middle of the swamp. At the gages near I-10 and south, the measured water surface elevations are more affected by local runoff from areas outside the model domain. Rainfall contribution was not modeled in this simulation as it was not a significant driving force for hydraulics in the area of interest (mid-swamp region). In the primary area of interest – the mid-swamp region where nutrient assimilation is expected – the model performance is excellent.

The calibration charts for the tropical storm hydrologic conditions are also shown in Appendix A. The final selected values of roughness (Manning's n) were 0.02, 0.035 and

0.2 s/($m^{1/3}$) for Lake Maurepas, the channels, and the swamp, respectively. The swamp region was assigned a high roughness to account for additional vegetation drag from flooded vegetation. The open water body of the lake was assigned a low roughness. The channels are assigned a typical roughness value used for natural streams. The statistical measures of correlation coefficient and root-mean-square error provided for each gage indicate the model predictions are a satisfactory reproduction of measured conditions. In general, the rising limb and peak of the storm hydrograph is matched well by the model. During the falling limb of the hydrograph, the model underpredicts the water levels indicating faster predicted outgoing flow than observed.

7.2 Model Validation

The model was validated using water surface elevation measurements from the Coastwide Reference Monitoring System (CRMS) available for the year 2020 data. Only water surface elevations were available at the CRMS gages in the study area for this time period. CRMS gage sites are located across the Louisiana coast in a range of ecological conditions, including swamp habitats and fresh, intermediate, brackish, and salt marshes. The CRMS allows for comparisons of changing conditions at both within and outside of restoration and protection projects. CRMS gages are predominantly located in marsh that is only tidally inundated. In the study area, only two gages, CRMS-0092 and CRMS-5522 were found that remain wet over a longer duration without drying out. Measurements from these two gages were used for validation. Charts showing the results of model validation are included as Appendix B.

8.0 APPROACH FOR SIMULATING NUTRIENTS

8.1 Overview of Approach

The nutrients simulated were total nitrogen (TN) and total phosphorus (TP) rather than individual species of nutrients (e.g., ammonia nitrogen, nitrate nitrogen, etc.). Although nutrients in organic and particulate forms are not immediately available for uptake by algae or vegetation, they can be transformed later into inorganic, dissolved forms that have the potential to cause eutrophication. Therefore, predictions for TN and TP are considered appropriate for addressing the modeling objectives.

TN and TP are simulated using a “black box” approach that characterizes the overall loss of nutrients from the water column as the water moves through the swamp. With this approach, the model does not simulate individual processes (mineralization, nitrification, denitrification, sorption of phosphorus, uptake by algae and plants, etc.), but the rates of nutrient loss from the water column are based on published measurements that account for the combined overall effect of all processes. This “black box” approach is being used instead of a more detailed approach of simulating individual processes due to a lack of site-specific data for calibrating numerous coefficients for the processes. The importance of calibration data in applications of complex models is noted in the following statement: “Highly detailed representations of system structures may not be useful to simulate TP dynamics in treatment wetlands if comprehensive data sets are not available to constrain each pathway” (Paudel and Jawitz 2012). Other studies have modeled losses of nutrients from water moving through wetlands without detailed simulations of individual processes (Day et al. 2004; Kadlec et al. 2011; CH2M Hill 2012; CH2M Hill 2013; Kadlec 2016; Merriman et al. 2017).

TN and TP are being simulated with generic user-defined constituents in the model. The nutrient state variables are designated to represent actual concentrations minus background concentrations (i.e., a concentration of zero in the model represents an actual concentration equal to background). With this configuration, the model simulates conditions that represent actual concentrations asymptotically approaching background concentrations without dropping below background concentrations. The assumption that actual concentrations cannot drop below background concentrations has been successfully used in various other studies that estimate losses of nutrients from water moving through wetlands (Kadlec et al. 2011; CH2M Hill 2012; CH2M Hill 2013; Kadlec 2016; Merriman et al. 2017).

The DELWAQ module has been set up to simulate losses of TN and TP from the water column with first order decay rates. For the generic user-defined constituents, the DELWAQ model does not provide any kinetics that are more complex than first order decay. First order decay is not a perfect representation of nutrient loss kinetics in wetlands (Kadlec 2000), but it forms the basis of equations that have been used in recent studies to calculate nutrient loss in wetlands receiving diverted river water and in wetlands receiving municipal wastewater. One of these equations is the “relaxed tanks-in-series” model, also known as the PkC* model (Kadlec and Wallace 2009):

$$\frac{(C_{OUT} - C^*)}{(C_{IN} - C^*)} = \left[1 + \frac{k}{P\,q}\right]^{-P}$$

where: C_{OUT} = Concentration at outlet of wetland (mg/L)
C_{IN} = Concentration at inlet of wetland (mg/L)
C^* = Background concentration (mg/L)
k = First order areal rate constant (m/yr)
q = Hydraulic loading rate per unit area (m/yr)
P = Apparent number of tanks in series (dimensionless)

The parameter “P” in the equation above accounts for: 1) hydraulic inefficiencies of flow through the wetland (i.e., it represents flow through multiple well-mixed tanks in series as opposed to uniform plug flow), and 2) “weathering”, which is a term that describes the effect of different loss rates for different fractions of the component (e.g., loss rates for nitrate and ammonia are individually different than an overall loss rate for TN).

For small areas with short residence times, the value of "P" in the equation above approaches 1.0 and the results become similar to a first order decay equation (with a background concentration incorporated):

$$\frac{(C_{OUT} - C^*)}{(C_{IN} - C^*)} = \exp (-k/h \times t)$$

where: h = depth of water (m)
t = residence time (yr)

For example, for k = 0.05 m/day (18.25 m/yr) and h = 0.5 m, the results from the two equations above differ by only 0.5% for a residence time of 1 day.

The DELWAQ model allows the user to vary the first order decay rates spatially or temporally, but not both. For this project, the decay rates are being varied spatially based on predicted depths. The model cells that represent shallow water moving through the swamp have been assigned higher decay rates and model cells that represent deeper, channelized flow have been assigned lower decay rates. Nutrient loss (from the water column) is expected to be greater in shallow vegetated areas due to vegetative uptake, settling and burial of particulates, and transformations by biological organisms that are either on the bottom or attached to vegetation and/or debris.

8.2 Nutrient Loss Rates

Tables D.1 and D.2 in Appendix D summarize information from published literature that was considered in selection of nutrient loss rates for the project Delft3D model. These tables include values for first order decay rates that were calculated based on hydraulic residence time and percent reduction of TN or TP (except where noted). These tables also include "k" values for the PkC* model that were either reported by the author or were calculated as the first order decay rate multiplied by the reported depth of water.

These studies represent a range of situations with different source water (river water or treated municipal wastewater), different types of wetlands (forested swamp, estuarine marsh, and constructed wetlands), and different climates (southern Louisiana as well as several other states).

The studies based on municipal wastewater are presented for comparison but were not directly used for estimating nutrient loss rates for this project.

The lowest values of first order decay rate and “k” value occurred for the systems with the longest residence times (77 – 512 days for Mandeville, Thibodaux, Luling, and Breaux Bridge). These first order decay rates and “k” values for these systems were not considered useful for developing inputs to the project Delft3D model because the residence times for those systems are much longer than the residence time for individual cells in the Delft3D model. Also, the TN and TP concentrations entering those four wetlands are much higher than the concentrations in the Mississippi River water that will be diverted into the Maurepas swamp.

In addition to the studies with field data summarized in Tables D.1 and D.2, a modeling study was conducted by CH2M Hill (2013) in which nutrient retention was simulated in various wetlands (including Maurepas swamp) with existing or proposed diversions of water from the Mississippi River. The CH2M Hill study used the PkC* model with the following “k” values:

- 27.8 m/yr for nitrate in vegetated habitat,
- 8.2 m/yr for nitrate in shallow lake habitat,
- 14.2 m/yr for ammonium,
- 17.3 m/yr for organic nitrogen, and
- 10.0 m/yr for TP.

The published literature that was reviewed for this project demonstrates variability in first order decay rates and “k” values not only among different sites, but also among different seasons. Much of the loss of nutrients from the water column is due to biological processes whose rates vary based on temperature. Therefore, nutrient loss rates are expected to be generally higher during summer and lower during winter.

Because it is anticipated that the diversion will be operated mostly in the warmer months, nutrient loss rates were selected accordingly. Based on the CH2M Hill (2013) study, as well as the information in Tables D.1 and D.2, the following "k" values were selected for use in the Delft3D model:

- TN: 30 m/yr in swamp, 10 m/yr in Lake Maurepas; and
- TP: 15 m/yr.

A script file was used to divide these "k" values by the predicted water depth in each cell in the model (after previously running the model for hydraulics) to obtain the first order decay rate that the Delft3D model needs for each cell in the model.

8.3 Background Concentrations

For this project, the background concentrations are based on existing concentrations in the Maurepas swamp and in Lake Maurepas. Table 8.1 provides summaries of TN and TP data measured in the Maurepas swamp (Hope Canal, Mississippi Bayou, and Dutch Bayou) and in Lake Maurepas. Table 8.1 includes data collected by Rob Lane during 2002-2003 and routine monitoring data collected by the Louisiana Department of Environmental Quality (LDEQ). Locations of the sampling sites are shown on Figure 8.1.

Table 8.1 Summary statistics for TN and TP data in Maurepas swamp and in Lake Maurepas.

Sampling location [A]	Period of record for nutrient data	TN data			TP data		
		No. of values	Median (mg/L)	Range (mg/L)	No. of values	Median (mg/L)	Range (mg/L)
Sites within the Maurepas swamp simulation area:							
Site 1 (Hope Canal)	4/04/02 – 5/13/03	11	0.79	0.51 – 1.32	11	0.75	0.04 – 1.21
Site 2 (Hope Canal)	4/04/02 – 5/13/03	11	0.78	0.61 – 1.52	11	0.15	0.07 – 0.66
Site 3 (Hope Canal)	4/04/02 – 5/13/03	11	0.82	0.57 – 1.75	11	0.13	0.05 – 1.00
Site 4 (Dutch Bayou)	4/04/02 – 5/13/03	11	0.65	0.49 – 1.58	11	0.11	0.05 – 0.20
Site 5 (Mississippi Bayou)	4/04/02 – 5/13/03	11	0.76	0.45 – 3.89	11	0.11	0.04 – 0.85
Site 0155 (Mississippi Bayou)	5/20/86 – 4/14/98	45	1.00	0.56 – 3.01	45	0.20	0.06 – 0.51
Site 4870 (Dutch Bayou)	10/03/17 – 4/03/18	7	0.94	0.37 – 4.15	7	0.15	0.09 – 0.19
Sites in Lake Maurepas:							
Site 16 (Lake Maurepas – SW)	4/04/02 – 5/13/03	12	0.64	0.44 – 2.42	12	0.11	0.01 – 0.20
Site 17 (Lake Maurepas – S)	4/04/02 – 5/13/03	12	0.59	0.39 – 0.99	12	0.12	0.08 – 0.17
Site 18 (Lake Maurepas – E)	4/04/02 – 5/13/03	11	0.58	0.43 – 0.91	11	0.10	0.03 – 0.16
Site 19 (Lake Maurepas – NE)	4/04/02 – 5/13/03	12	0.53	0.40 – 0.90	12	0.11	0.06 – 0.35
Site 1105 (Lake Maurepas – N)	1/09/01 – 9/25/07	24	0.67	0.30 – 1.82	24	0.09	0.05 – 0.19
Site 4471 (Lake Maurepas – SW)	10/01/13 – 4/03/18	19	0.85	0.35 – 1.39	19	0.15	0.05 – 0.29
Sites representing inflow entering the simulation area:							
Site 11 (Blind River)	4/04/02 – 5/13/03	12	0.60	0.46 – 0.82	12	0.10	0.05 – 0.69
Site 0036 (Pass Manchac)	3/06/78 – 9/08/16	290	0.90	0.09 – 5.54	291	0.10	< 0.05 – 0.51
Site 0228 (Amite River)	1/16/01 – 4/10/18	54	0.86	0.34 – 2.83	56	0.12	0.05 – 0.38
Site 0243 (Blind River)	1/16/01 – 4/03/18	62	0.82	0.24 – 1.42	64	0.15	0.05 – 0.44
Site 0268 (Amite R. Diversion Canal)	1/16/01 – 4/03/18	55	0.86	0.39 – 1.74	58	0.13	0.05 – 0.30
Site 1102 (Blind River near mouth)	1/16/01 – 4/03/18	62	0.80	0.20 – 4.40	64	0.15	0.05 – 0.29
Site 1106 (Tickfaw River)	1/09/01 – 9/03/15	48	0.98	0.21 – 2.57	56	0.13	0.05 – 0.39

Notes:

A. Site numbers between 1 and 19 are Rob Lane's monitoring sites. Site numbers between 0036 and 4870 are LDEQ monitoring sites.

Figure 8.1. Locations of LDEQ and Rob Lane water quality monitoring stations.

In general, the nutrient concentrations in the swamp were slightly higher than in Lake Maurepas. Median TN values in the swamp were mostly between 0.65 and 0.94 mg/L, while median TN values in Lake Maurepas were between 0.53 and 0.85 mg/L. For TP, median values were mostly between 0.11 and 0.15 mg/L in the swamp, while median values in Lake Maurepas were mostly between 0.09 and 0.11 mg/L. Although measured background concentrations of nutrients vary by location, the background concentrations used in the model need to be spatially constant in order to preserve the calculated mass of nutrients being transported in the model. The following values were selected for use as background concentrations for the DELWAQ model:

- Background TN = 0.60 mg/L, and
- Background TP = 0.10 mg/L.

These two proposed background concentrations are more representative of Lake Maurepas than the Maurepas swamp, but it is better to select values towards the low end of the range because the model is able to simulate concentrations above these values, but it cannot simulate concentrations below these values (i.e., the model is not allowed to simulate negative concentrations).

9.0 MODEL APPLICATION TO EVALUATE ALTERNATIVES

9.1 Proposed Model Scenarios

The calibrated and validated model was used to simulate a set of diversion scenarios under current (Year 0) and future (Year 50) conditions. Future conditions accounted for expected future Sea Level Rise (SLR), subsidence and accretion in the study area. Inputs for the Year 0 and Year 50 scenarios are summarized in Tables 9.1 and 9.2, respectively. All scenarios assume presence of the proposed WSLP levee and drainage structures. The Future-Without-Project (FWOP) scenarios establish the base conditions for comparison with the Future-With-Project (FWP), i.e., with the diversion inflow, conditions.

FWP runs for storm or high rainfall events are not needed as the diversion is not expected to be operated under such conditions. Given that intense rainfall events are infrequent and of short-duration (1-2 day), they are expected to contribute significantly less to long-term changes in the conditions of the swamp than the diversion project over a multi-year time scale as in this study.

A 20-day normal conditions period was simulated for each proposed scenario. A constant diversion flow was prescribed over the entire 20-day period. The model results show that the parameters of interest approach steady state within this period. No diversion shutdown period is simulated. A shutdown period can be added to any of these scenarios in the future, if needed without having to rerun the 20-day operation period.

9.2 Sea Level Rise, Subsidence and Accretion

Inputs for the future (Year 50) conditions are based on Eustatic Sea Level Rise, subsidence and accretion information provided by CPRA which is presented in Appendix E. The future conditions model is based on estimated conditions for year 2075, assuming 50 years of project operation after 5 years of engineering, design and construction). Subsidence and accretion were incorporated in the future conditions model by adjusting the swamp floor elevation in the model geometry (Section 9.3.3). The SLR effects were implemented in the future conditions model runs by increasing the boundary water surface elevations specified at Pass Manchac as described in Section 9.4.

Table 9.1. Year 0 scenarios (All with WSLP).

Run ID	Diversion Flow	Diversion Channel and project features	Tidal Boundary Conditions	Rainfall	Nutrients	Comments
Year 0 FWOP without diversion – With WSLP						
10	N/A	(No div. canal) Existing Hope Canal	Normal	None	None	Without diversion - hydraulics Simulate water level, velocity
10a	N/A	(No div. canal) Existing Hope Canal	Normal	None	TN, TP, Salinity	Without diversion – water quality Simulate TN, TP, Salinity
Year 0 FWP with diversion – With WSLP						
11	250 cfs	95% Design	Normal	None	None	With diversion- hydraulics Simulate water level, velocity
11a	250 cfs	95% Design	Normal	None	Salinity	With diversion- Salinity
12	1,000 cfs	95% Design	Normal	None	None	With diversion- hydraulics Simulate water level, velocity
12a	1,000 cfs	95% Design	Normal	None	Salinity	With diversion- Salinity
13	2,000 cfs	95% Design	Normal	None	None	With diversion- hydraulics Simulate water level, velocity
13a	2,000 cfs	95% Design	Normal	None	TN, TP, Salinity	With diversion- water quality Simulate TN, TP, Salinity

Table 9.2. Year 50 scenarios- moderate SLR (All with WSLP).

Run ID	Diversion Flow	Diversion Channel and project features	Tidal Boundary Conditions	Rainfall	Nutrients	Comments
Year 50 FWOP without diversion – With WSLP – Moderate SLR						
50	N/A	(No div. canal) Existing Hope Canal	Normal	None	None	Without diversion - hydraulics Simulate water level, velocity
50a	N/A	(No div. canal) Existing Hope Canal	Normal	None	TN, TP, Salinity	Without diversion – water quality Simulate TN, TP, Salinity
Year 50 FWP with diversion – With WSLP – Moderate SLR						
53	2,000	95% Design	Normal	None	None	With diversion- hydraulics Simulate water level, velocity
53a	2,000	95% Design	Normal	None	TN, TP, Salinity	With diversion- water quality Simulate TN, TP, Salinity

9.3 Model Geometry with Project

To simulate the diversion alternative scenarios, the existing conditions model geometry used in calibration / validation needed to be modified to include features of the proposed diversion project, WSLP project features, and future subsidence and accretion. The model geometries used in the production runs are shown in Figure 9.1 and the modifications to the existing conditions geometry are described below.

9.3.1 Addition of the proposed diversion project

The model geometry was modified to represent the diversion channel and outfall management features proposed in the 95% design report (URS 2014). The following model geometry modifications were performed:

- Added the proposed diversion channel from the Mississippi River to its end approximately 1000 ft north of its crossing with I-10 highway. The channel has a variable cross-section. The longest segment between the Highway 61 and I-10 has a 60 ft wide bottom and 1V:5H side slope. The invert is -7 ft and -8 ft, NAVD88 at Highway 61 and I-10, respectively.
- Removed culvert crossings under I-10 between LA 641 and Mississippi Bayou to prohibit backflow from the diversion into the swamp between I-10 and Highway 61.
- Added gaps in the railroad embankment along the west bank of Hope Canal.

The Mississippi River, the details of diversion complex, and the sediment settling basin were not represented in the model as they were not necessary to simulate the hydraulics in the swamp, which is the purpose of this modeling effort.

9.3.2 Addition of the WSLP project features

The WSLP project was represented by addition of its proposed levee. In the model, this levee prevented any diversion water discharged north of I-10 from flowing south into the protected area. Of the many proposed drainage structures under the WSLP levee, only those that are within the study area / model domain needed to be added to allow two-way flow. Discharges from the proposed WSLP drainage pumps were not added to the model because they represent intermittent rainfall runoff inflows that are not evaluated in this study. Table 9.3 describes the WSLP structures added to the model.

Table 9.3. WSLP drainage structures included in the Maurepas Delft3D model.

Station Name	Number of gate drainage structures	Invert (ft, NAVD88)	Location
Mississippi Bayou	4 each 14 ft x 14 ft gates	-8	30.101215, -90.575144
Reserve Relief Canal	4 each 16 ft x 16 ft gates with one 16 ft wide navigation gate	-10	30.106680, -90.546472
Perriloux	2 each 14 ft x 14 ft gates	-8	30.113405, -90.502920
Ridgefield	2 each 14 ft x 14 ft gates	-8	30.113194, -90.489849

9.3.3 Addition of subsidence and accretion

Based on the guidance provided by CPRA (Appendix E), the study area bathymetry was adjusted to represent subsidence and accretion as follows:

<u>FWOP conditions: Year 50 *without* the diversion project</u>

Subsidence in 55 years = -7.1 mm/yr x 55 yr = -0.391 m or -1.282 ft

Accretion in 55 years = +5.0 mm/yr x 55 yr = +0.275 m or +0.902 ft

Net in 55 years = -0.38 ft subsidence

Assumed only swamp floor at or above 0 ft, NAVD88, is affected by accretion, so bed elevations at or above 0 ft, NAVD88 were lowered by 0.38 ft. For all bed elevations, below 0 ft, NAVD88 (e.g., streams, lakes), only subsidence (no accretion) was applied and the bed elevations were lowered by 1.282 ft.

<u>FWP conditions: Year 50 *with* the diversion project</u>

Subsidence in 55 years = -7.1 mm/yr x 55 yr = -0.391 m or -1.282 ft

Accretion in 55 years = +10.0 mm/yr x 55 yr = +0.55 m or +1.804 ft

Net in 55 years = +0.522 ft accretion

Assumed only the swamp floor at or above elevation 0 ft, NAVD88 is affected by accretion, so bed elevations at or above 0 ft, NAVD88 were raised by 0.522 ft. For all bed elevations, below 0 ft, NAVD88 (e.g., streams, lakes), only subsidence (no accretion) was applied and the bed elevations were lowered by 1.282 ft.

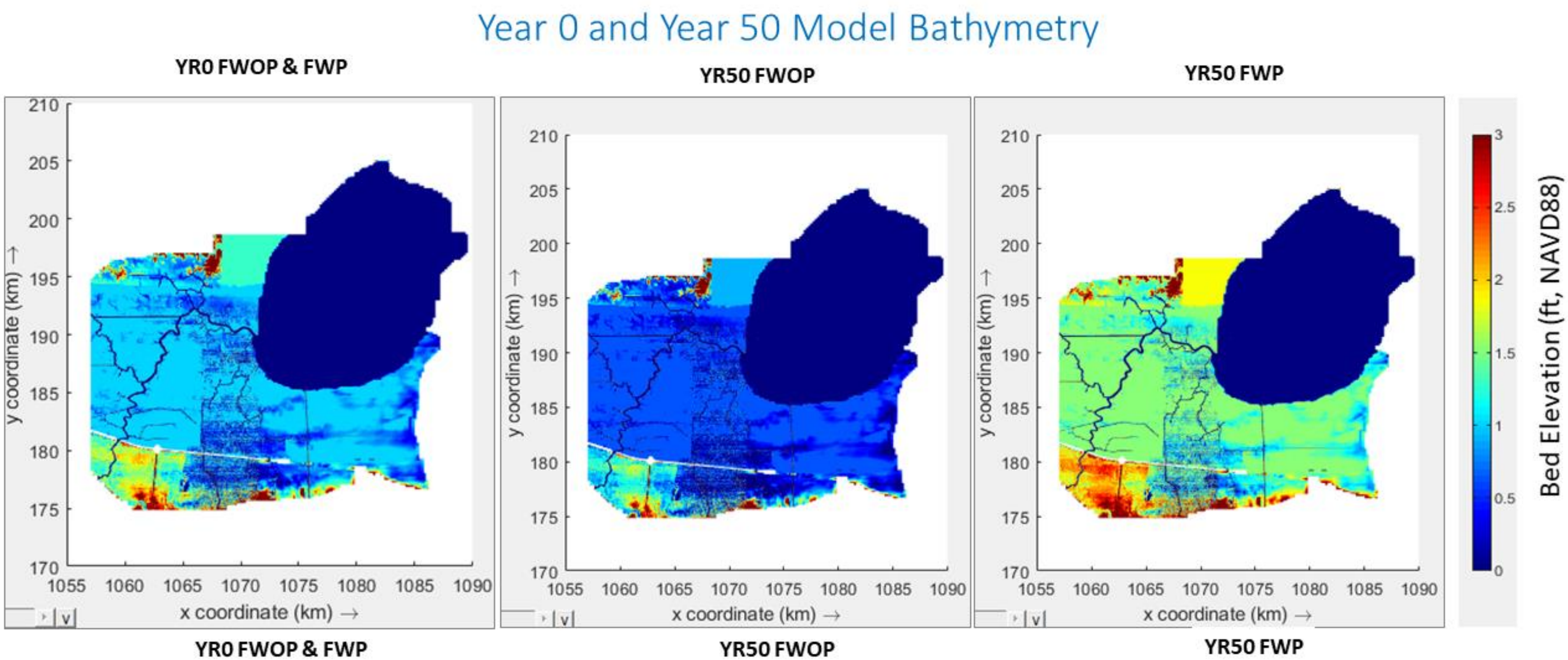


Figure 9.1 Delft3D model geometries used to the production runs.

9.4 Model Boundary Conditions

The boundary conditions are specified by the user at the edges of the model boundary. These are the hydraulic (water levels, flows) and water quality (nutrient, salinity concentrations) conditions that drive and influence the study area. The locations of these boundaries are shown on Figure 9.2.

9.4.1 Hydraulics – Water Levels and Flows

Pass Manchac is simulated as a tidal water level boundary (water can flow in or out of the simulated area based on head differences); all of the other boundaries are simulated as flow boundaries. For each flow boundary (except the diversion), the flow was set to a constant value to represent median (i.e., typical) flow conditions (see Table 9.4). The diversion of Mississippi River water into Hope Canal was set to a constant value of 250, 1,000 or 2,000 cfs depending on the simulated scenario.

The stage boundary at Pass Manchac was specified with hourly values to represent typical tidal fluctuations about the historical median water level. For the future (Year 50) conditions a mean water surface elevation rise of 2.1 ft (0.64008 m) was applied based on the guidance in Appendix E.

The Pass Manchac tidal water level is the most important boundary condition that drives the water movement in the Maurepas Swamp. The high Amite River flood conditions can affect water levels in the study area, but such condition is not evaluated in the present analysis.

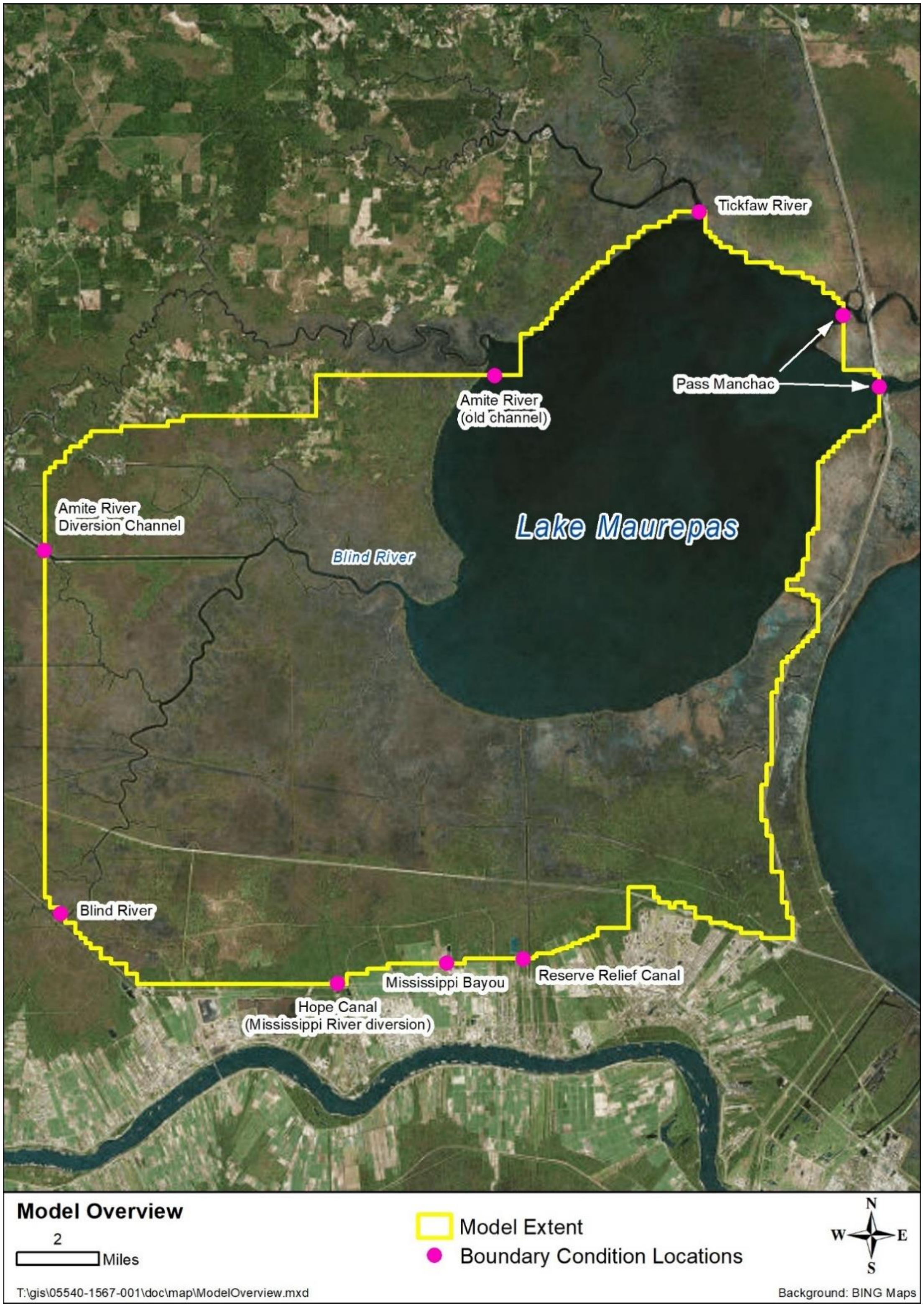


Figure 9.2. Locations where boundary conditions were specified in the model.

9.4.2 Water Quality – Nutrient and Salinity Concentration

Concentrations of TN, TP, and salinity must be specified in the model for each boundary where water can flow into the simulated area. TN and TP data for the Mississippi River are summarized in Table 9.5 for US Geological Survey (USGS) monitoring stations at Baton Rouge and Belle Chasse. Although these two stations are located 86 miles upstream and 68 miles downstream, respectively, of the proposed diversion location near Garyville, the TN and TP concentrations are similar between the two stations, which suggests that these data are representative of concentrations at Garyville.

Concentrations of TN, TP, and salinity that are being used in the model at each boundary location are summarized in Tables 9.6 and 9.7. Initial conditions for TN, TP and salinity are specified in Table 9.8.

Table 9.4. Input values for flows and stages at model boundaries.

Location of boundary	Model input value	Comment
Hope Canal (diversion from Mississippi River)	250, 1,000 or 2,000 cfs	Assumed operational flow rate
Tickfaw River	412 cfs	Sum of median flows for Oct. 1989 – Sep. 2017 for Tickfaw River at Holden (158 cfs) and Natalbany River at Baptist (27 cfs) multiplied times ratio of published drainage area at the mouth (727 mi^2; USGS 1971) to combined drainage area at the two gages (247 mi^2 + 79.5 mi^2).
Amite River (old channel)	173 cfs	Median flow for Amite River at Port Vincent (USGS 07380120) for entire period of record (Oct 1987 – Sep 2015) is 1,090 cfs. Assumed flow split is 16% into old channel and 84% into Diversion Canal based on 5/09/2007 flow measurements published by Amite River Basin Drainage and Water Conservation District (2007).
Amite River Diversion Canal	917 cfs	
Blind River	40 cfs	Approximate median flow per unit area of 0.6 cfs/mi^2 (based on USGS gages on Amite, Tickfaw, and Natalbany rivers) multiplied times estimated drainage areas (outside the model grid) of about 60-70 mi^2 for Blind River and < 10 mi^2 for Mississippi Bayou and Reserve Relief Canal
Mississippi Bayou	5 cfs	
Reserve Relief Canal	5 cfs	
Pass Manchac (Year 0 conditions)	0.71 – 1.21 ft NAVD88	Synthetic stage hydrograph based on tidal cycle of 24.7 hours, typical tidal fluctuation of 0.5 ft, and median water level of 0.96 ft over entire period of record (Feb. 2002 – Aug. 2018) at Corps station 85420 (Pass Manchac near Ponchatoula)
Pass Manchac (Year 50 conditions)	2.81 – 3.31 ft NAVD88	Year 0 water levels raised by 2.1 ft (Appendix E)

Table 9.5 Monthly statistics for TN and TP in the Mississippi River.

Month	TN Data			TP Data		
	Number of values	**Median (mg/L)**	**Range (mg/L)**	**Number of values**	**Median (mg/L)**	**Range (mg/L)**
USGS 07374000 Mississippi River at Baton Rouge (5/18/04 – 2/13/17):						
January	14	1.88	1.49 – 2.77	13	0.23	0.13 – 0.34
February	13	2.11	1.63 – 3.00	12	0.27	0.15 – 0.33
March	21	2.07	1.56 – 3.48	20	0.24	0.15 – 0.51
April	26	2.15	1.41 – 3.23	26	0.22	0.14 – 0.33
May	23	2.15	1.43 – 3.75	23	0.21	0.14 – 0.37
June	25	2.54	1.62 – 3.38	26	0.25	0.14 – 0.68
July	10	2.63	1.86 – 3.68	10	0.24	0.10 – 0.32
August	14	1.67	1.10 – 2.38	14	0.23	0.13 – 0.35
September	3	1.30	1.21 – 1.57	3	0.22	0.18 – 0.25
October	11	1.39	0.94 – 2.52	10	0.19	0.16 – 0.33
November	6	1.69	1.15 – 2.69	6	0.24	0.14 – 0.29
December	11	1.79	1.30 – 2.41	10	0.22	0.12 – 0.36
All Months	177	2.06	0.94 – 3.75	173	0.22	0.10 – 0.68
USGS 07374525 Mississippi River at Belle Chase (5/11/06 – 5/08/18):						
January	12	1.95	1.50 – 2.79	11	0.28	0.17 – 0.39
February	11	1.97	1.69 – 2.80	10	0.25	0.17 – 0.51
March	23	2.02	1.51 – 3.34	21	0.29	0.17 – 0.62
April	24	2.15	1.50 – 3.80	22	0.25	0.18 – 0.39
May	26	1.99	1.33 – 3.78	25	0.24	0.16 – 0.39
June	24	2.48	1.61 – 3.51	24	0.24	0.14 – 0.35
July	9	2.59	1.99 – 3.86	9	0.27	0.14 – 0.43
August	12	1.83	1.00 – 2.37	12	0.26	0.11 – 0.40
September	2	1.18	1.15 – 1.21	2	0.17	0.17 – 0.17
October	10	1.37	0.81 – 2.54	9	0.22	0.09 – 0.38
November	4	1.48	1.03 – 2.57	4	0.20	0.16 – 0.29
December	10	1.74	1.19 – 2.65	9	0.23	0.14 – 0.37
All Months	167	2.00	0.81 – 3.86	158	0.25	0.09 – 0.62

Table 9.6. Input values for nutrient concentrations at model boundaries.

Location of boundary	Actual concentrations	Model input concentrations*	Comment
Hope Canal (diversion from Mississippi River)	2.21 mg/L TN 0.25 mg/L TP	1.61 mg/L TN 0.15 mg/L TP	Averages for January 1 – August 31 using USGS data for Mississippi River at Baton Rouge (07374000) and Mississippi River at Belle Chasse (07374525) during 2004 – 2018.
Tickfaw River	0.98 mg/L TN 0.13 mg/L TP	0.38 mg/L TN 0.03 mg/L TP	Median values for LDEQ station 1106 (Tickfaw River near Lake Maurepas) for |2001 – 2015
Amite River (old channel)	0.86 mg/L TN 0.12 mg/L TP	0.26 mg/L TN 0.02 mg/L TP	Median values for LDEQ station 0228 (Amite River at mile 6.5, at Clio) for 2001 – 2018
Amite River Diversion Canal	0.86 mg/L TN 0.13 mg/L TP	0.26 mg/L TN 0.03 mg/L TP	Median values for LDEQ station 0268 (Amite River Diversion Canal north of Gramercy) for 2001 – 2018
Blind River	1.33 mg/L TN 0.24 mg/L TP	0.73 mg/L TN 0.14 mg/L TP	Median values for LDEQ station 0117 (Blind River near Gramercy) for 1978 – 1998
Mississippi Bayou	0.76 mg/L TN 0.11 mg/L TP	0.16 mg/L TN 0.01 mg/L TP	Median values for Station 5 (Mississippi Bayou) from Rob Lane's 2002 – 2003 data
Reserve Relief Canal	0.79 mg/L TN 0.13 mg/L TP	0.19 mg/L TN 0.03 mg/L TP	Median values for Stations 1 and 2 (Hope Canal) and station 5 (Miss. Bayou) from Rob Lane's 2002 – 2003 data
Pass Manchac	0.90 mg/L TN 0.10 mg/L TP	0.30 mg/L TN 0 mg/L TP	Median values for LDEQ station 0036 (Pass Manchac at Manchac) for 1978 – 2016

* Model input concentrations are actual concentrations minus background concentrations.

Table 9.7. Input values for salinity at model boundaries.

Location of boundary	Model input values	Comment
Hope Canal (diversion from Mississippi River)	0.20 ppt	Median value for LDEQ stations 0047 (Mississippi River at Luling) and 0048 (Mississippi River near Luling) for 1978 – 1989
Tickfaw River	0.11 ppt	Median values for LDEQ station 1106 (Tickfaw River near Lake Maurepas) for 2001 – 2015
Amite River (old channel)	0.05 ppt	Median value for LDEQ station 0228 (Amite River at mile 6.5, at Clio) for 2001 – 2018
Amite River Diversion Canal	0.05 ppt	Median value for LDEQ station 0268 (Amite River Diversion Canal north of Gramercy) for 2001 – 2018
Blind River	0.30 ppt	Median value for LDEQ station 0117 (Blind River near Gramercy) for 1978 – 1998
Mississippi Bayou	0.25 ppt	Median value for station 5 (Mississippi Bayou) from Rob Lane's 2002 – 2003 data
Reserve Relief Canal	0.30 ppt	Median values for stations 1 and 2 (Hope Canal) and station 5 (Miss. Bayou) from Rob Lane's 2002 – 2003 data
Pass Manchac	5.0 ppt	Assumed to be the same as the initial concentration (see Table 2.6 below). Because the source of the initial salinity in Lake Maurepas and the Maurepas swamp is exchange with Lake Pontchartrain (via Pass Manchac), then the salinity in Pass Manchac should be similar to the initial value for Lake Maurepas and the Maurepas swamp.

Table 9.8. Input values for initial conditions for water quality.

Constituent	Model input value	Comment
Total nitrogen (TN)	0 mg/L	Zero in the model represents background concentrations for TN and TP. Nutrient concentrations throughout the modeled area are assumed to be at background levels at the beginning of each simulation.
Total phosphorus (TP)	0 mg/L	
Salinity	5.0 ppt	Assumed value for conditions following a tropical storm surge or possibly an extreme drought. Hypothetical scenario.

9.5 Model Coefficients and Settings

For alternative scenarios simulations, roughness was specified using Manning's n values of 0.02, 0.035 and 0.2 s/($m^{1/3}$) for Lake Maurepas, the channels, and the swamp, respectively.

For the nutrient simulations, the diffusion coefficient was set to be 1 m^2/s. The suggested range of this parameter in the Delft3D manual is 0.1 m^2/s to 1.0 m^2/s. Additional two simulations of one-week duration were performed by setting diffusion to 0.1 m^2/s and 0.5 m^2/s. Figure 9.3 shows the TN and TP contours at the end of the simulation. The shift in the contours is insignificant, indicating that the selected value is reasonable.

A computational time step of 2 seconds was used for simulations. The model output was saved at hourly intervals at key locations and at daily intervals at all nodes of the model grid.

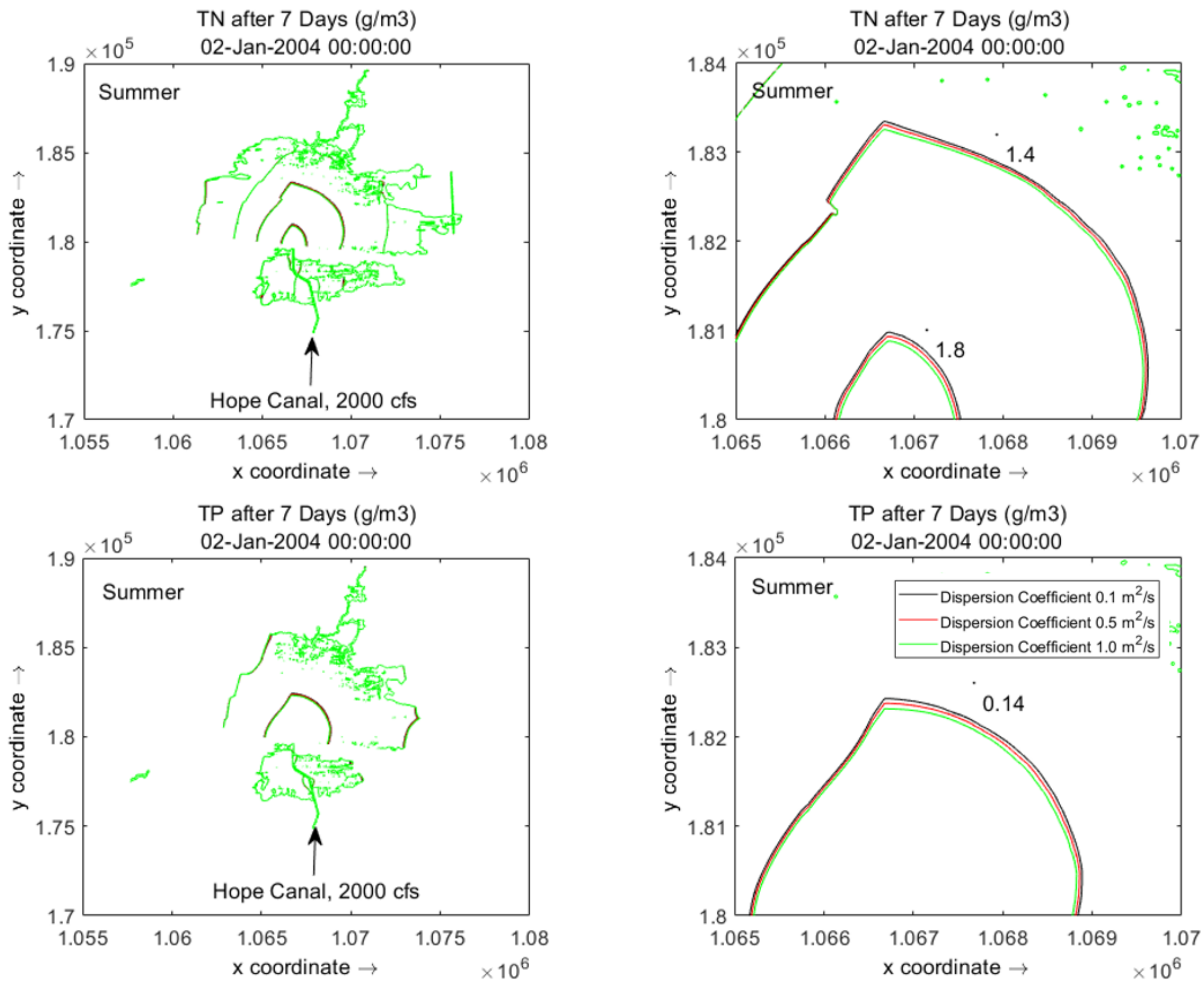


Figure 9.3. Contours of TN and TP from the diffusion coefficient sensitivity simulations.

10.0 MODEL RESULTS

To evaluate project alternatives, model scenarios described in Tables 9.1 and 9.2 were performed. The model output consists of predicted water surface elevations, velocity and concentrations across the model domain at each node. To limit the output file size, the water surface, velocity and concentrations are saved at key locations at an hourly interval. At the remaining nodes they are saved at a daily interval. Several channel transects were specified at key locations where the model saved discharge values at daily intervals. All model results are discussed in the following sections and are shown in maps and time series charts in Appendix C. Note that, even though the model results are available for 20 days, the time series charts show results of 15 days, excluding the first 5 days where minor instabilities exist as the model starts computations from the specified initial conditions.

10.1 Predicted Water Surface Elevation and Velocity

Figures C1 through C4 show the variation of water surface elevation and velocity (time-series charts) at selected locations over the simulation period. These locations are selected to coincide with some of the gages shown in Figure 5.6. The maximum water surface elevation in the swamp is predicted to be about 3 ft, NAVD88 and it occurs where the diversion enters the swamp (i.e., in Hope Canal immediately north of Interstate 10). The velocities peak up to 2.4 ft/s at this location. However, in the adjoining swamp, the highest velocities are around 0.1 to 0.2 ft/s just outside the Hope Canal and lesser in the swamp away from the canal. Under the continuous diversion inflow of 2,000 cfs, the water surface elevation in the swamp reaches a steady state in about two weeks, setting a constant water surface gradient across the swamp from high at Hope Canal to low near Lake Maurepas. Note that the oscillation in the water surface elevations is due to the influence of tides specified at Pass Manchac. It is seen that the diversion water spreads throughout the most of the system within a week. A steady water surface elevation and gradient is established in the system in about two weeks.

The highest water level increases due to the diversion flows occur in Hope Canal where the diversion enters the swamp north of I-10. These are represented by profiles at location Gage

S-7 (Figure C1). The location S-23 (Figure C3) is a good indicator of average water levels over the swamp. To assess the effects of the diversion flow near the proposed WSLP drainage structure, results can be examined in Reserve Relief Canal near WSLP shown in Figure C4. Table 10.1 summarizes rise in water surface elevations at these key locations.

Table 10.1. Water level rise due to diversion inflows.

Diversion Inflow (cfs)	**Water Level Rise Above Normal Water Surface Elevation of 1.0 ft for Year 0 and 3.0 ft for Year 50**		
	Hope Canal at I-10	**Swamp overall**	**Reserve Relief Canal near WSLP**
250 (Year 0)	0.3	0.1	0.0
1,000 (Year 0)	1.3	0.7	0.2
2,000 (Year 0)	1.9	0.9	0.3
2,000 (Year 50)	0.6	0.2	0.1

The contours of water surface elevation at the end of the 20-day simulation are shown in Figure C5. For the Year 0 conditions, the scenario of 250 cfs diversion shows that the majority of the introduced MR water tends to stay within Hope Canal and Dutch Bayou as it flows to Lake Maurepas. The spread of the diversion water into the swamp increases as the diversion flow rate increases to 1,000 cfs and then to 2,000 cfs. For the Year 50 conditions, due to the extensive inundation from the normal tidal levels, the excess inundation due to the diversion is relatively small.

The general distribution of diversion flow through the main streams in the swamp is shown in Figures C7 (Year 0 condition) and C8 (Year 50 conditions). The flow distribution is summarized in Table 10.2.

Table 10.2. Distribution of diversion flow through major streams.

Diversion Inflow (cfs)	Diversion flow exiting through Dutch Bayou (cfs)	Diversion flow exiting through the Blind River (cfs)	Diversion flow exiting through Reserve Canal (cfs)
250 (Year 0)	210 (84%)	0 (0%)	29 (12%)
1000 (Year 0)	462 (46%)	176 (18%)	251 (25%)
2000 (Year 0)	648 (32%)	570 (29%)	513 (26%)
2000 (Year 50)	119 (6%)	816 (41%)	150 (8%)

For Year 50 conditions, due to the significant swamp inundation resulting from the SLR, the diversion flow has more opportunity to overtop the stream banks. Therefore, only 6% of the diversion flow is channelized through Dutch Bayou (2,000 cfs diversion).

Model results show that the diversion water spreading east is intercepted by the Reserve Relief Canal hindering distribution to the wetlands east of this canal in spite of the artificial gapping implemented in the model. This suggests that limited gapping on the east bank of the Reserve Relief Canal may not distribute commensurate quantities of diversion water to the east side. No gapping on the west bank of this canal was evaluated.

10.2 Predicted Percent Mississippi River Water in the Swamp

One of the Delft3D model parameters allows accounting of the percentage of water in each model grid cell that originated from the Mississippi River diversion. The purpose of simulating this variable (percent Mississippi River water) was to show where the Mississippi River water travels once introduced into the swamp. The boundary "concentrations" for this variable were set to 100 for the inflow from the Mississippi River (via Hope Canal) and zero for all other boundaries. The initial concentration was set to zero for the entire model grid.

Figure C9 shows the predicted values of percent Mississippi River water at the end of 20 days. For the 2,000 cfs diversion inflow, the model predicts that most of the swamp water is displaced by the river water in 20 days under the Year 0 conditions. For the future, Year 50, conditions, slightly less but still extensive freshening is projected.

10.3 Predicted Total Nitrogen and Total Phosphorous Transport

The TN and TP results are shown in Figures C10 and C11 for Year 0 and Year 50 conditions, respectively.

As expected, the highest predicted concentrations of TN are in Hope Canal and its immediately surrounding areas north of Interstate 10. As the Mississippi River water spreads into the swamp and even along channels (e.g., Hope Canal to Tent Bayou to Dutch Bayou), the TN concentrations decrease due to losses from the water column that are simulated with the first order decay rates.

Based on the spatial patterns of predicted TN concentrations in Lake Maurepas, it appears that Dutch Bayou and Reserve Relief Canal are contributing similar loadings of TN to Lake Maurepas. The predicted TN concentrations in the southwest corner of Lake Maurepas (excluding the small areas right at the mouth of Dutch Bayou and the mouth of Reserve Relief Canal) were between 0.8 and 1.0 mg/L at the end of day 20. This represents a small increase over the assumed background concentration of 0.6 mg/L.

The TN in the Mississippi River water consists of approximately 71% nitrate, 2% ammonium, and 27% organic nitrogen (based on long term averages of USGS data at Baton Rouge and Belle Chasse). Among these three forms of nitrogen, nitrate is the form that is expected to undergo the greatest losses from the water column because it can be removed from the water column through denitrification (which is one of the most significant removal mechanisms in wetlands) or uptake by algae or plants. By the time the Mississippi River water reaches Lake Maurepas, the remaining TN is expected to consist mostly of organic nitrogen, which is not available for algal uptake unless it is first converted back to inorganic nitrogen through the process of mineralization, which is a relatively slow process.

As with TN, the highest predicted concentrations of TP are in Hope Canal and the immediately surrounding areas north of Interstate 10. Dutch Bayou and Reserve Relief Canal appear to be contributing similar loadings of TP to Lake Maurepas.

10.4 Salinity Flushing Results

The purpose of this simulation was to demonstrate the freshening effect of the diversion on a swamp that has experienced a high salinity event due to a tropical storm. Figures C12 and C13 show contours of salinity after 10 and 20 days of diversion inflow. The initial salinity was set to 5 ppt throughout the entire model domain. Additionally, the constant salinity value of 5 ppt specified at Pass Manchac (Lake Maurepas) boundary may not be realistic. However, this does not affect results in our primary area of interest which is the swamp north of Interstate 10. Therefore, the evaluation of results was focused on this region.

The results show that salinity is rapidly flushed out of the swamp by the diversion flow. As expected, the flushing process is slower in the areas where little diversion flow reaches.

10.5 Comparison with Previous Modeling Studies

The TN predictions discussed in Section 10.3 can be compared with two previous modeling studies for the Maurepas swamp. Comparisons must be done with caution because each study used different modeling approaches based on project objectives and available data.

Day et al. (2004) used output from a two-dimensional hydraulic model to calculate nitrate transport and loss in the Maurepas swamp. The model simulated water being diverted from the Mississippi River into Hope Canal and then moving through the swamp towards the Blind River, Reserve Relief Canal, or Lake Maurepas. The swamp was divided into cells and the equation used to estimate nitrate loss in each cell was:

$$\text{Percent removal} = -14.13 * \text{LN}(X) + 25$$

where X = nitrate loading entering that cell ($g/m^2/day$)

The predicted losses of nitrate for water reaching Lake Maurepas were 87% and 81% for diversion flow rates of 1,500 cfs and 2,500 cfs, respectively (Table 4.4 in Day et al. [2004]). It should be noted that this modeling study did not utilize a background concentration for nitrate because existing concentrations of nitrate in the Maurepas swamp are low.

CH2M Hill (2013) conducted modeling to estimate total nutrient removal for multiple planned and existing diversions along the Mississippi River. Based on objectives of this project and the large area that it encompassed, this modeling was developed at spatial and temporal resolutions that were much coarser than the DELWAQ modeling presented in this report. The CH2M Hill modeling used the pKC* model (described in Section 8.1) with background concentrations of zero for nitrate and ammonium, 0.6 mg/L for organic nitrogen, and 0.042 mg/L for total phosphorus. The model predicted a 57% loss of TN and 46% loss of TP in the Maurepas swamp for "average operations" (Table 14 of CH2M Hill [2013]).

In order to compare the DELWAQ results with these two studies, percentage losses of TN and TP were calculated. For Year 0 simulations, percentage losses were calculated for TN and TP based on concentrations in Mississippi River water that was introduced into the swamp and simulated concentrations in Lake Maurepas at the mouth of Dutch Bayou at the end of day 20. These calculations resulted in percentage losses of 54% for TN and 35% for TP. These percentage losses are similar to the results from CH2M Hill (2013). The percentage loss for TN is lower than the nitrate losses calculated by Day et al. (2004), but nitrate losses are expected to be greater than TN losses because nitrate can be removed from the water column through denitrification and uptake by algae or plants, whereas organic nitrogen (the other primary component of TN in Mississippi River water) can be removed from the water column only by settling of the particulate fraction.

10.6 Comparison with Nutrient Concentrations in Lake Pontchartrain

The predictions of TN in the southern end of Lake Maurepas can be compared with TN concentrations that were observed in Lake Pontchartrain after the Bonnet Carré Spillway was opened in 2008 and in 2011. When the Bonnet Carré Spillway is opened, large volumes of Mississippi River water are diverted into Lake Pontchartrain during a short time. This water reaches Lake Pontchartrain quickly with minimal nutrient loss. In both 2008 and 2011, increased algae concentrations were observed in the lake (including cyanobacteria that were presumably caused by the nutrient loading from the diverted Mississippi River water).

In 2008, the spillway was opened for about a month, with a total volume of diverted water that exceeded the volume of Lake Pontchartrain (Bargu et al. 2011). The average concentration of nitrate nitrogen that was measured within the plume during the spillway opening was 1.3 mg/L (Bargu et al. 2011). The modeling for Lake Maurepas does not specify what portions of the TN are nitrate, ammonium, and organic nitrogen, but the TN in the water that reaches Lake Maurepas is expected to be mostly organic nitrogen (see Section 8.2). If the predicted TN in the southern end of Lake Maurepas is assumed to include about 0.5 mg/L of organic nitrogen (most of the background concentration of TN is expected to consist of organic nitrogen), then the predicted TN values of 0.8 to 1.0 mg/L in the southern end of Lake Maurepas would correspond to nitrate concentrations of about 0.3 to 0.5 mg/L. These are much lower than the average nitrate concentration measured within the plume in Lake Pontchartrain during the spillway opening (1.3 mg/L).

In 2011, the spillway was opened from May 9 to June 20, with a total volume of diverted water that was approximately 330% of the combined volume of Lake Pontchartrain and the downstream estuary (Smith 2014). The average concentration of nitrate nitrogen that was measured along a transect extending from the Bonnet Carré Spillway to the approximate center of the lake was 0.6 mg/L (individual values ranged from below the reporting limit up to 1.4 mg/L; Smith 2014). It is apparent that some dilution or other nutrient loss mechanisms affected some of these values because the nitrate concentrations measured by the USGS in the Mississippi River during the spillway opening ranged from 1.1 to 1.4 mg/L (3 samples at Baton and 6 samples at Belle Chasse). Nitrate concentrations in Lake Pontchartrain near the spillway were probably more similar to the Mississippi River values than the average concentrations reported by Smith (2014) for an entire transect. As discussed above, the TN values predicted for the southern end of Lake Maurepas correspond to estimated nitrate concentrations of about 0.3 to 0.5 mg/L, which are lower than estimated nitrate concentrations in Lake Pontchartrain near the spillway.

11.0 SUMMARY AND CONCLUSIONS

A two-dimensional Delft3D hydrodynamic and water quality model was developed, calibrated and validated for the study area. The model was applied to simulate water surface elevations, velocity, total nitrogen, and total phosphorous under 20-day continuous diversion flows of 250, 1,000 and 2,000 cfs. Below are the findings based on the model results.

- The highest water levels will occur in Hope Canal as it exits I-10 bridge:
  - Year 0: Diversion flow of 250, 1000 and 2000 cfs raises water level by 0.3, 1.3 and 1.9 ft, respectively.
  - Year 50: Diversion flow of 2000 cfs raises water level by 0.6 ft.
- The average water levels in the swamp:
  - Year 0: Diversion flow of 250, 1000 and 2000 cfs raises water level by 0.1, 0.7 and 0.9 ft, respectively.
  - Year 50: Diversion flow of 2000 cfs raises water level by 0.2 ft.
- Water levels near the WSLP drainage structures:
  - Year 0: Diversion flow of 2000 cfs raises water by less than 0.3 ft.
  - Year 50: Diversion flow of 2000 cfs raises water level by 0.1 ft.
- Distribution of the diversion flow changes with its magnitude. For the Year 0 conditions, about 84%, 46% and 32% of diversion inflow 250-, 1000- and 2000 cfs flows through Dutch Bayou to Lake Maurepas. Of the remaining discharge:
  - 12% flows towards the Reserve Canal and insignificant towards the Blind River (250 cfs diversion).
  - 25% flows towards the Reserve Canal and 18% towards the Blind River (1,000 cfs diversion).
  - 26% flows towards the Reserve Canal and 29% towards the Blind River (2,000 cfs diversion).
  - For the Year 50 conditions, due to significant inundation, only 6% of the diversion flow is channelized through Dutch Bayou (2,000 cfs diversion).

- Distribution of the diversion flow changes with its magnitude:
  - 250 cfs diversion rate (Year 0):
    - 84% flows through Dutch Bayou to Lake Maurepas.
    - 12% flows towards the Reserve Relief Canal.
    - insignificant flow towards the Blind River.
  - 1,000 cfs diversion rate (Year 0):
    - 46% flows through Dutch Bayou to Lake Maurepas.
    - 25% flows towards the Reserve Relief Canal.
    - 18% flows towards the Blind River.
  - 2,000 cfs diversion rate (Year 0):
    - 32% flows through Dutch Bayou to Lake Maurepas.
    - 26% flows towards the Reserve Relief Canal.
    - 29% flows towards the Blind River.
  - 2,000 cfs diversion rate (Year 50):
    - Due to significant inundation, the diversion flow has more opportunity to overtop the stream banks. Therefore, only 6% of the diversion flow is channelized through Dutch Bayou (2,000 cfs diversion).

The shallow and relatively slow flow through the swamp allows for nutrients to be removed from the water column before the water reaches Lake Maurepas via Dutch Bayou and Reserve Relief Canal. By the time the Mississippi River water reaches Lake Maurepas, it has lost about 54% of its TN and 35% of its TP. Predicted concentrations of TN in the southern end of Lake Maurepas correspond to nitrate concentrations that are much lower than observed concentrations in Lake Pontchartrain that led to increased algae concentrations in 2008 and 2011 after opening the Bonnet Carré Spillway.

Based on these projection simulations, the proposed diversion of Mississippi River water into the Maurepas swamp is expected to provide beneficial freshening and nutrients to a large area of swamp without causing large increases in nutrient concentrations in Lake Maurepas.

12.0 REFERENCES

Bargu, S., J.R. White, C. Li, J. Czubakowski, and R. Fulweiler. 2011. *Effects of freshwater input on nutrient loading, phytoplankton biomass, and cyanotoxin production in an oligohaline estuarine lake*. Hydrobiologia 661:377–389.

Brantley, C.G., J.W. Day, Jr., R.R. Lane, E. Hyfield, J.N. Day, and J.-Y.Ko. 2008. *Primary production, nutrient dynamics, and accretion of a coastal freshwater forested wetland assimilation system in Louisiana*. Ecological Engineering 34:7-22.

CH2M Hill. 2012. A*pproaches to Water Quality Treatment by Wetlands in the Upper Klamath Basin*. Prepared for PacifiCorp Energy, Portland, OR. August 2012.

CH2M Hill. 2013. *Nitrogen and Phosphorus Removal Estimates for Existing and Planned Mississippi River Diversions*. Technical Memorandum prepared for Louisiana Coastal Protection and Restoration Authority. November 2013.

Day, J.W., Jr., G.P. Kemp, H.S. Mashriqui, R.R. Lane, D. Dartez, and R. Cunningham. 2004. *Development Plan for a Diversion into the Maurepas Swamp - Water Quality and Hydrologic Modeling Components*. Final report prepared for U.S. Environmental Protection Agency Region 6. September 2004.

Deltares. 2018. *Delft3D FLOW, User Manual: Simulation of multi-dimensional hydrodynamic flows and transport phenomena, including sediments*. Delft, The Netherlands.

Hunter, R., R. Lane, John Day, J. Lindsey, Jason Day, M. Hunter. 2009. *Nutrient Removal and Loading Rate Analysis of Louisiana Forested Wetlands Assimilating Treated Municipal Effluent*. Environmental Management 44:865-873.

Hunter, R.G., J.W. Day, R. R. Lane, G.P. Shaffer, J.N. Day, W.H. Conner, J.M. Rybczyk, J.A. Mitsch, and J.-Y. Ko. 2018. *Using Natural Wetlands for Municipal Effluent Assimilation: A Half-Century of Experience for the Mississippi River Delta and Surrounding Environs.* Chapter 2 in Multifunctional Wetlands, edited by N. Nagabhatla and C.D. Metcalfe. Springer Publishing.

Kadlec, R.H. 2000. *The inadequacy of first order treatment wetland models*. Ecological Engineering 15:105-120.

Kadlec, R.H., J.S. Bays, L.E. Mokry, D. Andrews, and M.R. Ernst. 2011. *Performance analysis of the Richland-Chambers treatment wetlands*. Ecological Engineering 37:176-190.

Kadlec, R.H., and S.D. Wallace. 2009. *Treatment Wetland*s, 2nd edition. CRC Press.

Kadlec, R.H. 2016. *Large Constructed Wetlands for Phosphorus Control: A Review*. Journal of Water volume 8, article 243. 36. pp.

Lane, R.R., J.W. Day, D. Justic, E. Reyes, B. Marx, J.N. Day, and E. Hyfield. 2004. *Changes in stoichiometric Si, N and P ratios of Mississippi River water diverted through coastal wetlands to the Gulf of Mexico*. Estuarine, Coastal and Shelf Science 60:1-10.

Merriman, L.S., J.M. Hathaway, M.R. Burchell, and W.F. Hunt. 2017. *Adapting the Relaxed Tanks-in-Series Model for Stormwater Wetland Water Quality Performance*. Journal of Water volume 9, article 691. 18 pp.

Mitsch, W.J., L. Zhang, E. Waletzko, and B. Bernal. 2014. *Validation of the ecosystem services of created wetlands: Two decades of plant succession, nutrient retention, and carbon sequestration in experimental riverine marshes*. Ecological Engineering 72:11-24.

MPH. 2018. *Topographic Survey in the outfall management area of the Mississippi River Reintroduction into the Maurepas Swamp (PO-29)*, Prepared by Morris P. Hebert, Inc for the Coastal Protection and Restoration Authority, Louisiana. July 18, 2018.

Paudel, R., and J.W. Jawitz. *Does increased model complexity improve description of phosphorus dynamics in a large treatment wetland?* Ecological Engineering 42:283-294.

Perez, B.C., J.W. Day, Jr., D. Justic, R.R. Lane, and R.R. Twilley. 2011. *Nutrient stoichiometry, freshwater residence time, and nutrient retention in a river-dominated estuary in the Mississippi Delta*. Hydrobiologia 658:41-54.

Phipps, R.G., and W.G. Crumpton. 1994. *Factors affecting nitrogen loss in experimental wetlands with different hydrologic loads*. Ecological Engineering 3:399-408.

Smith, E.A. 2014. *Cyanobacteria harmful algal blooms in South Louisiana estuaries: a synthesis of field research, management implications, and outreach*. PhD dissertation, Department of Oceanography and Coastal Sciences, Louisiana State University. May 2014.

URS. 2005. *Mississippi River Diversion into Maurepas Swamp Project PO-29, Volume III of VII, Topographic and Bathymetric Survey*, Prepared for the Coastal Protection and Restoration Authority, Louisiana. October, 2005.

URS. 2006. *Mississippi River Diversion into Maurepas Swamp Project PO-29, Volume IV of VII, Hydrologic Data*, Prepared for the Coastal Protection and Restoration Authority, Louisiana. June 29, 2006.

URS. 2014. *Mississippi River Diversion into Maurepas Swamp* (PO-29, Contract No. 2503-11-63), *95% Design Report*, Prepared for the Coastal Protection and Restoration Agency, Louisiana. May 2014.

Zhang, X., S.E. Feagley, J.W. Day, W.H. Conner, I.D. Hesse, J.M. Rybczyk, and W.H. Hudnall. 2000. *A Water Chemistry Assessment of Wastewater Remediation in a Natural Swamp*. Journal of Environmental Quality 29:1960-1968.

APPENDIX A

Model Calibration Results

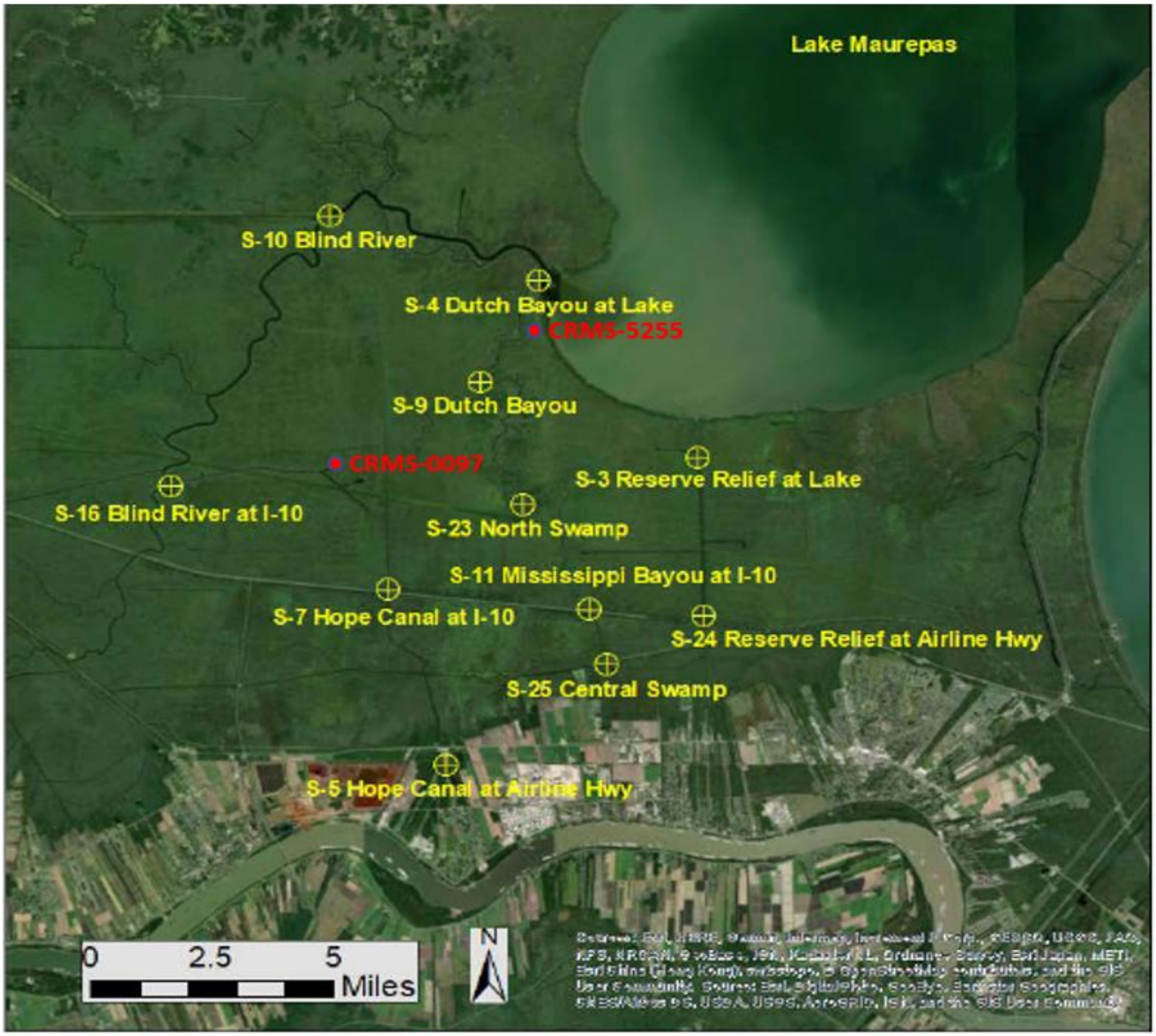


Figure A1. Locations of gages used for calibration (yellow symbols) and validation (red symbols).

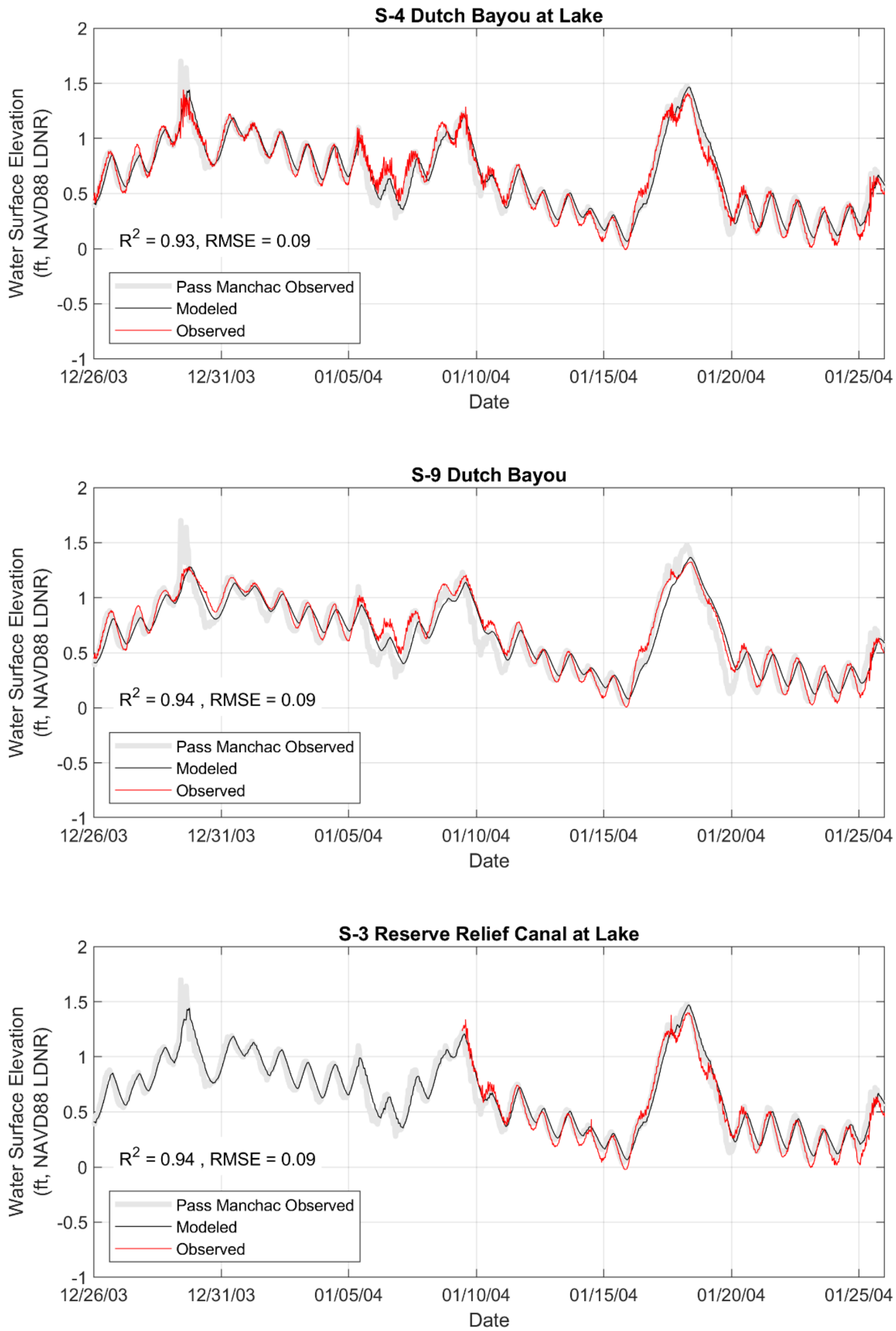


Figure A2. Observed and predicted water surface elevations at gages S-4, S-9 and S-3 under normal conditions.

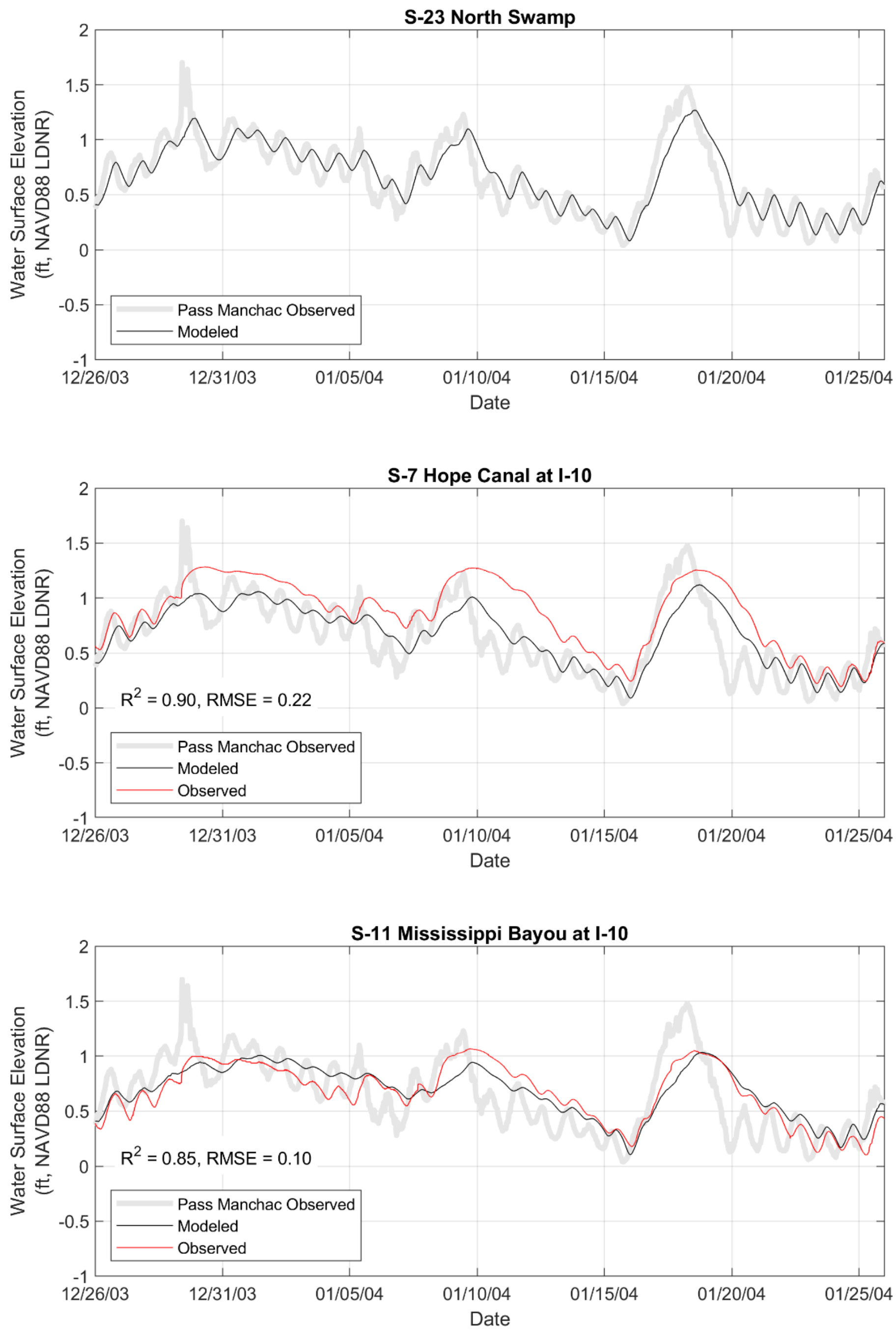


Figure A3. Observed and predicted water surface elevations at gages S-23, S-7 and S-11 under normal conditions.

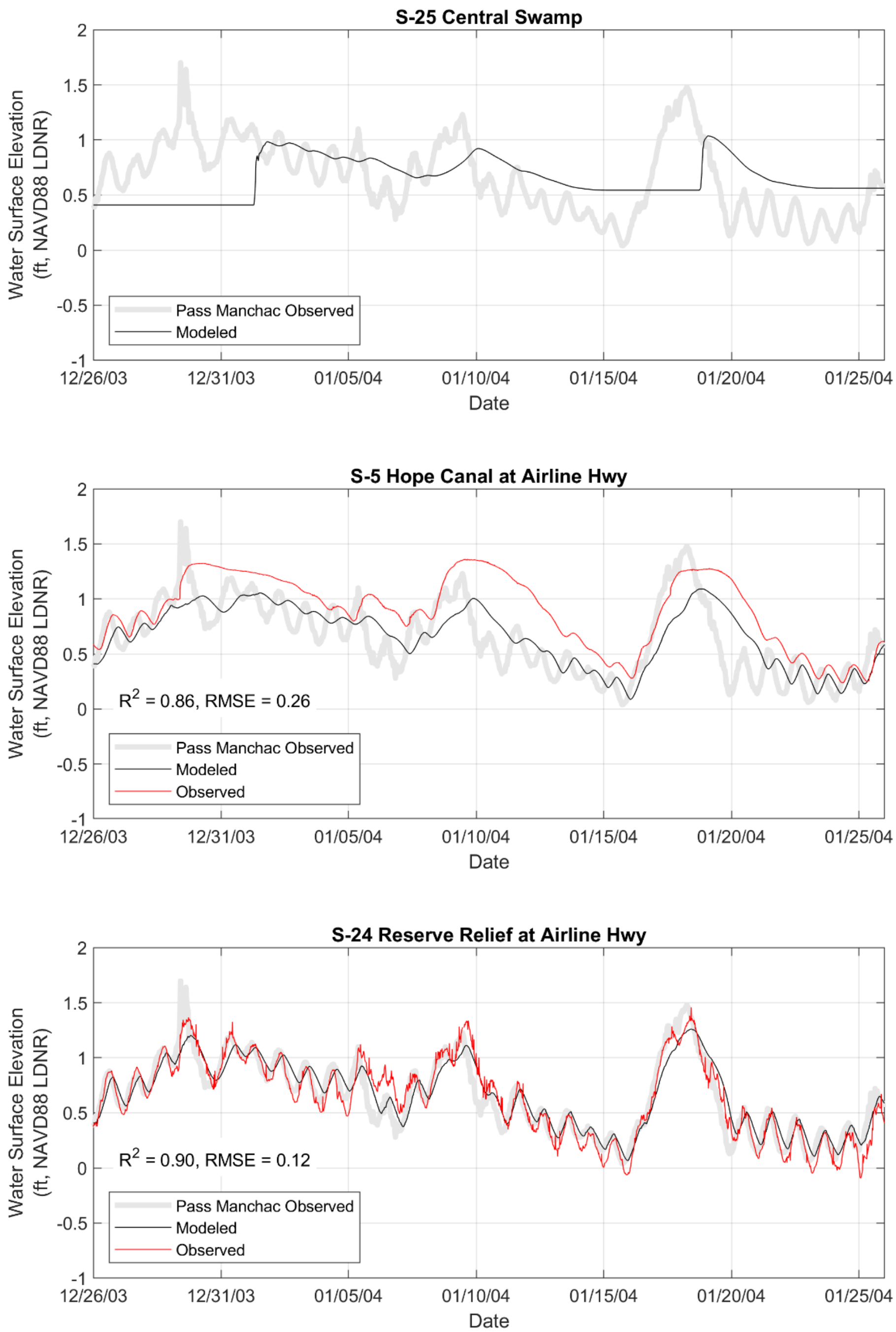


Figure A4. Observed and predicted water surface elevations at gages S-25, S-5 and S-24 under normal conditions.

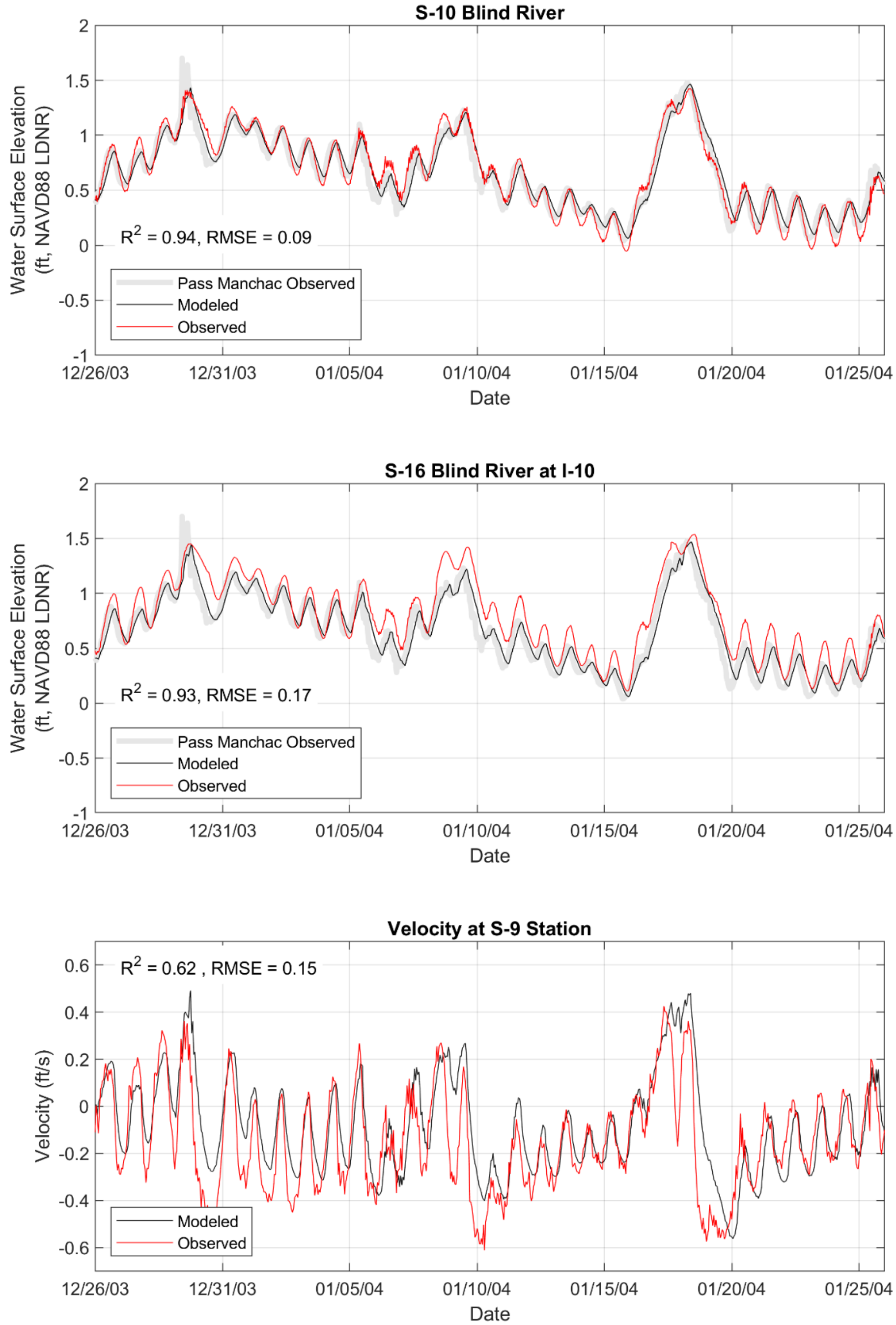


Figure A5. Observed and predicted water surface elevations at gages S-10, S-16 and velocity at S-9 under normal conditions.

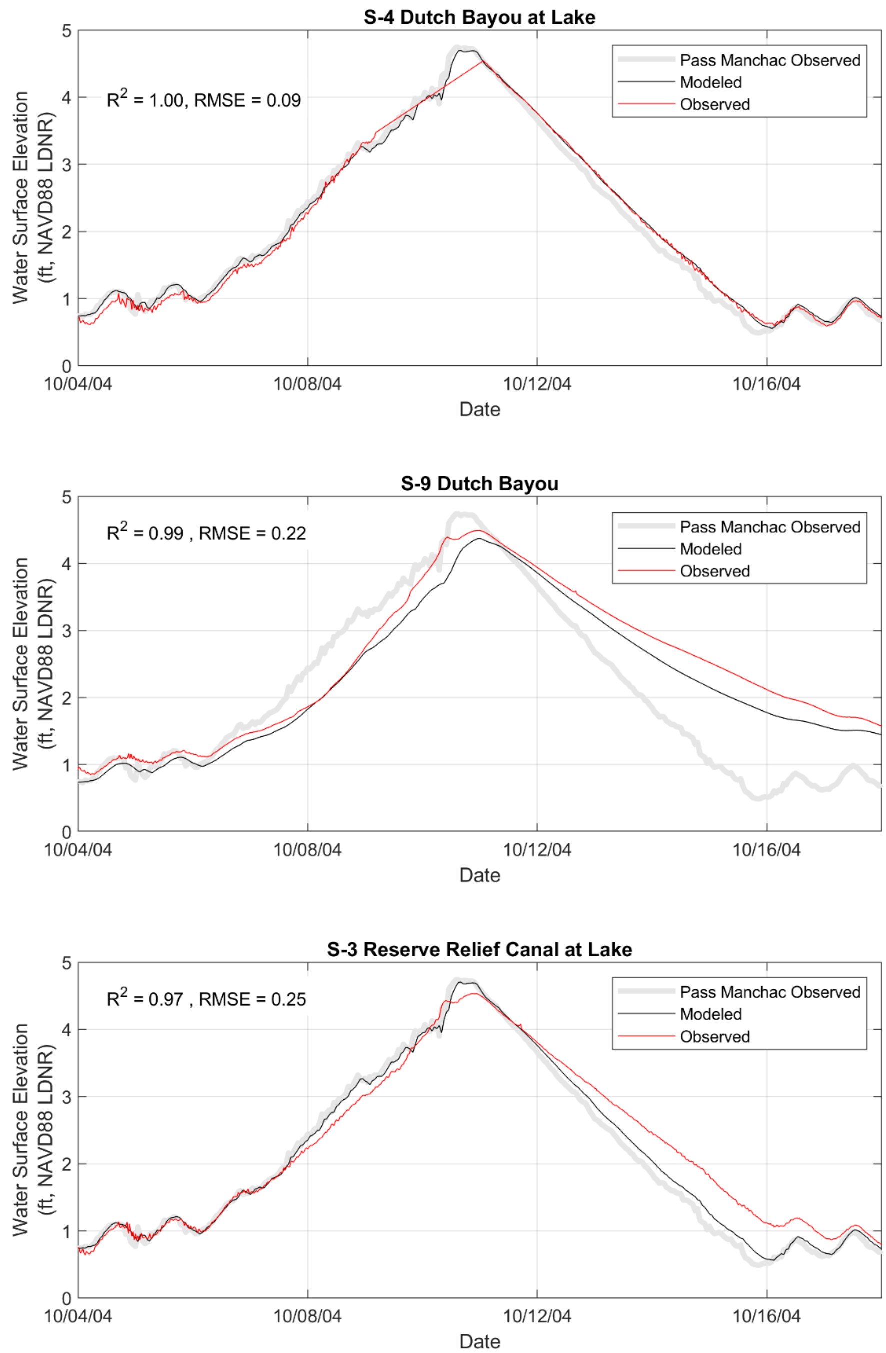


Figure A6. Observed and predicted water surface elevations at gages S-4, S-9 and S-3 under tropical storm conditions.

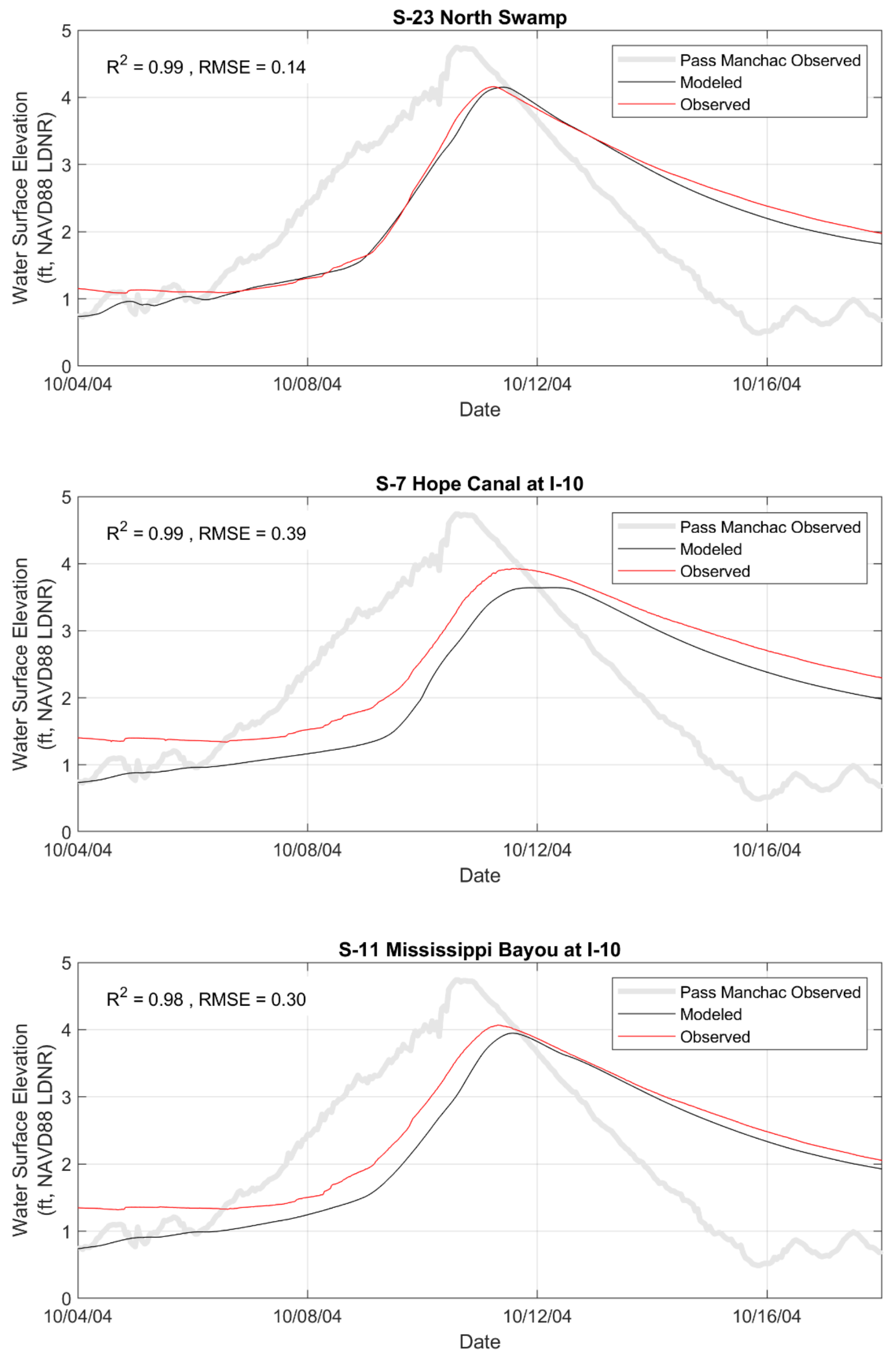


Figure A7. Observed and predicted water surface elevations at gages S-23, S-7 and S-11 under tropical storm conditions.

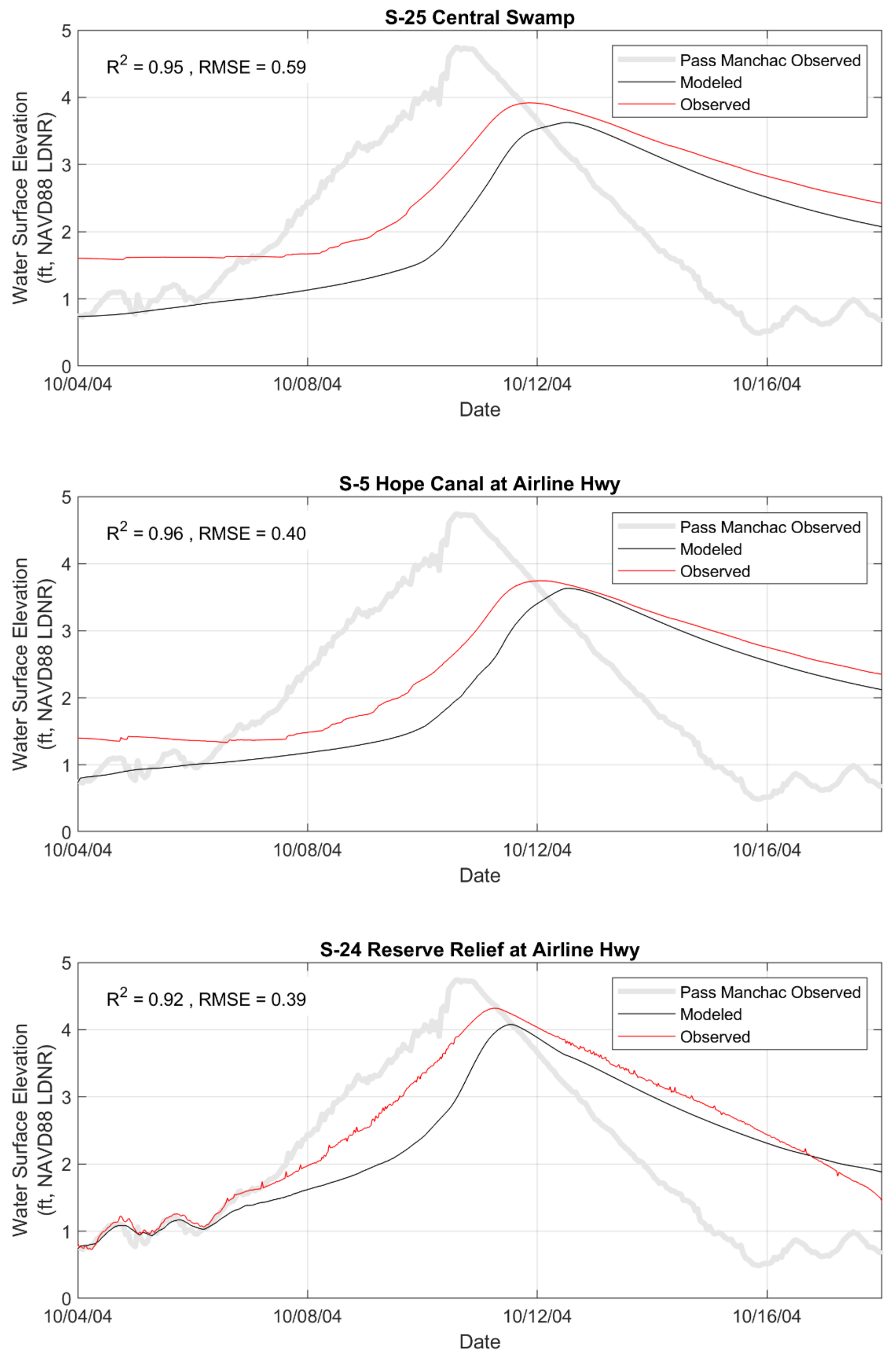


Figure A8. Observed and predicted water surface elevations at gages S-25, S-5 and S-24 under tropical storm conditions.

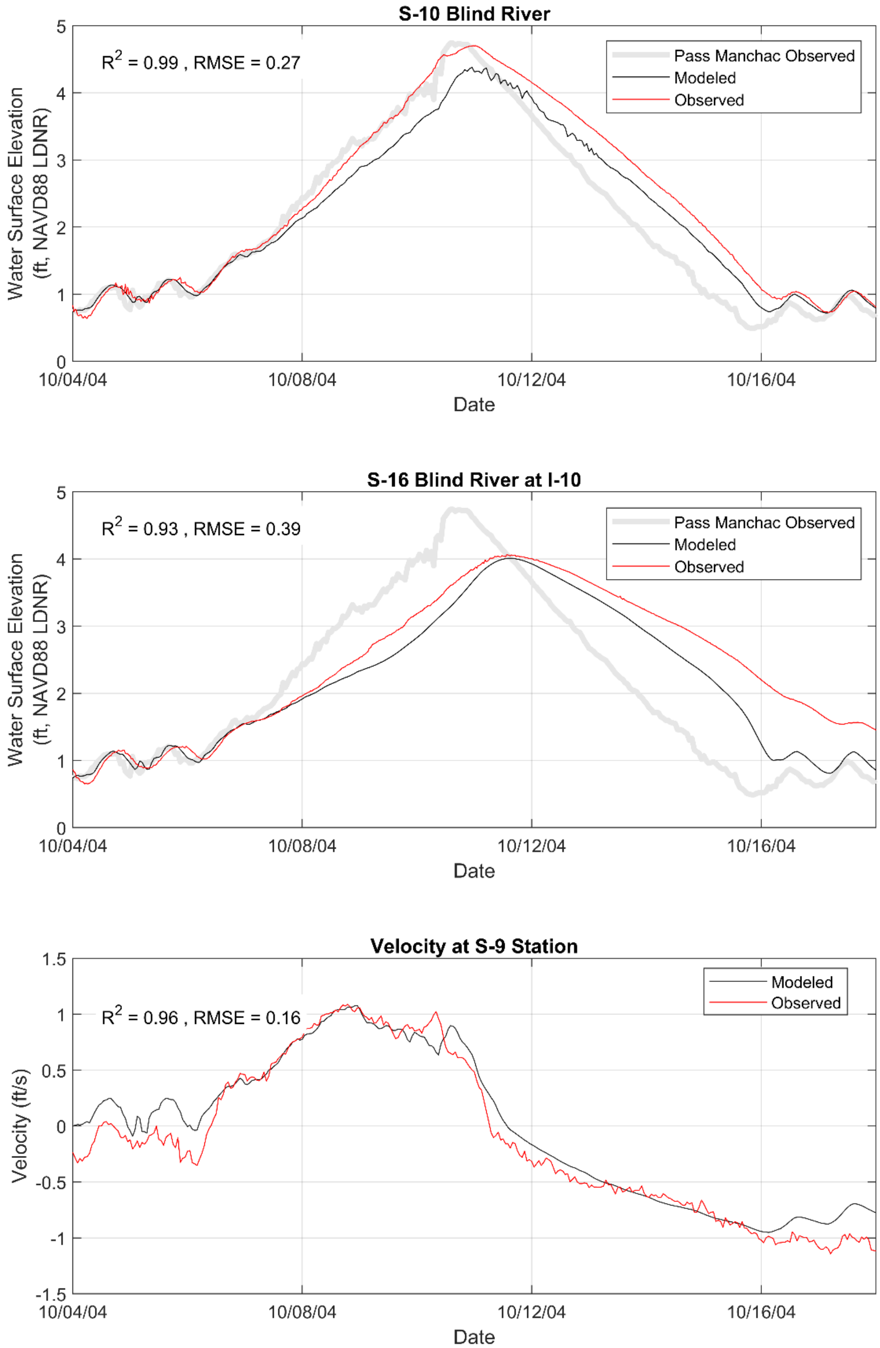


Figure A9. Observed and predicted water surface elevations at gages S-10, S-16 and velocity at S-9 under tropical storm conditions.

APPENDIX B

Model Validation Results

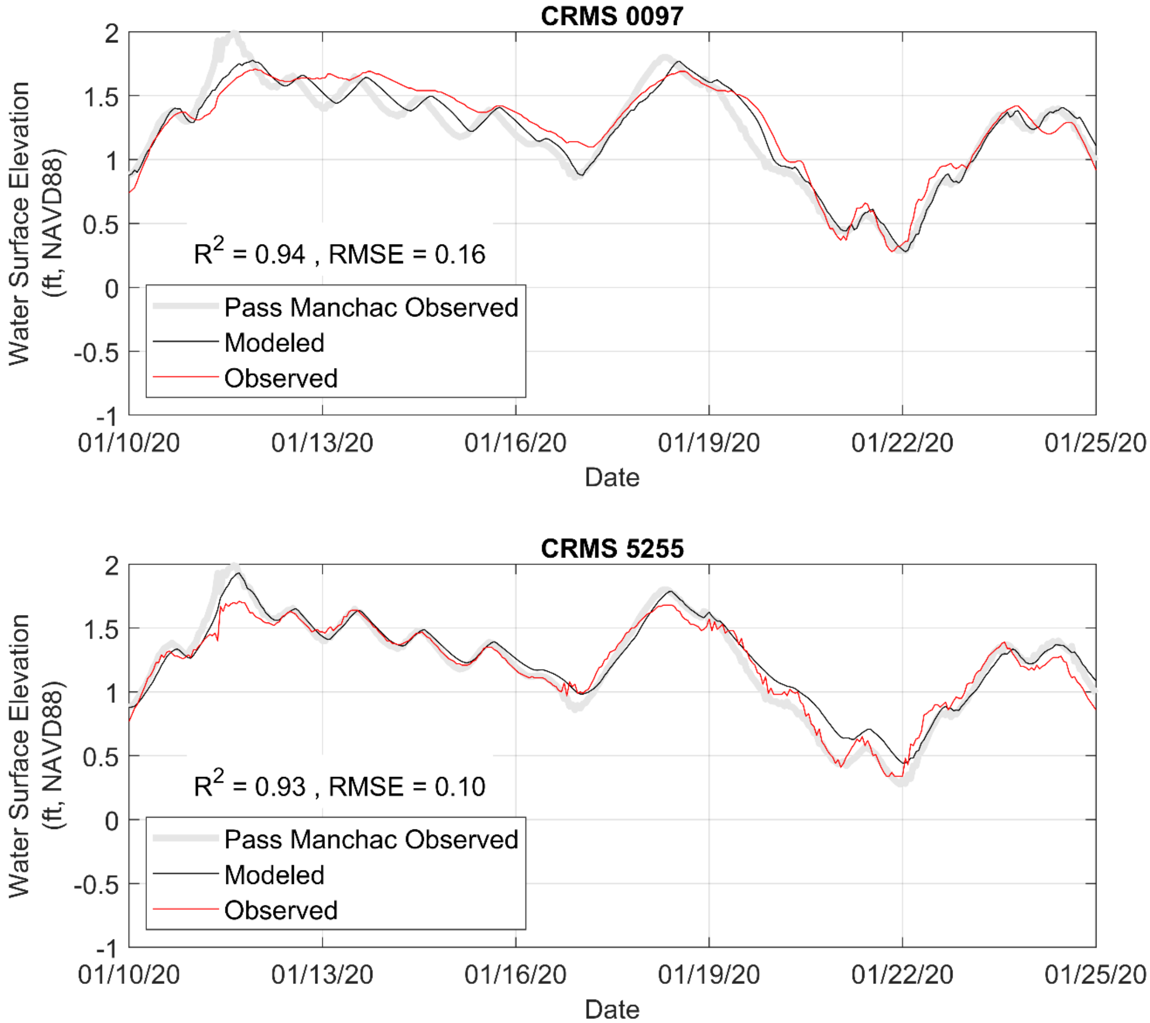


Figure B1. Observed and predicted water surface elevations at CRMS gages 0097 and 5255.

APPENDIX C

Model Alternative Scenarios Results

Delft3D Model Results – Location S-7 (Outfall at I-10)

Highest water levels and velocity magnitude are in Hope Canal as it exits I-10 bridge

<u>YR 0</u>
Normal is 1 ft, NAVD88
250 cfs adds 0.3 ft
1000 cfs adds 1.3 ft
2000 cfs adds 1.9 ft
<u>YR 50</u>
Normal is 3 ft, NAVD88
2000 cfs adds 0.6 ft

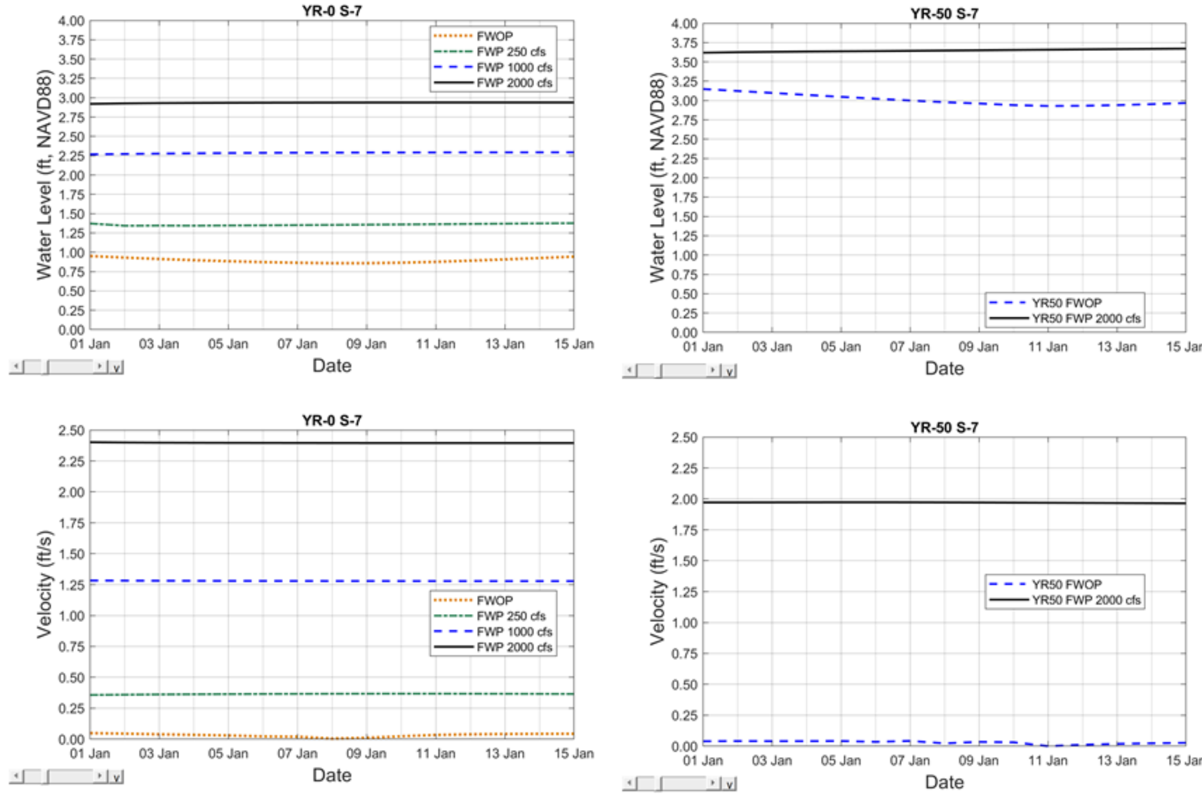


Figure C1. Predicted water surface elevation (upper panel) and velocity (lower panel) profiles over the model simulation period at location S-7 (Hope Canal north of I-10).

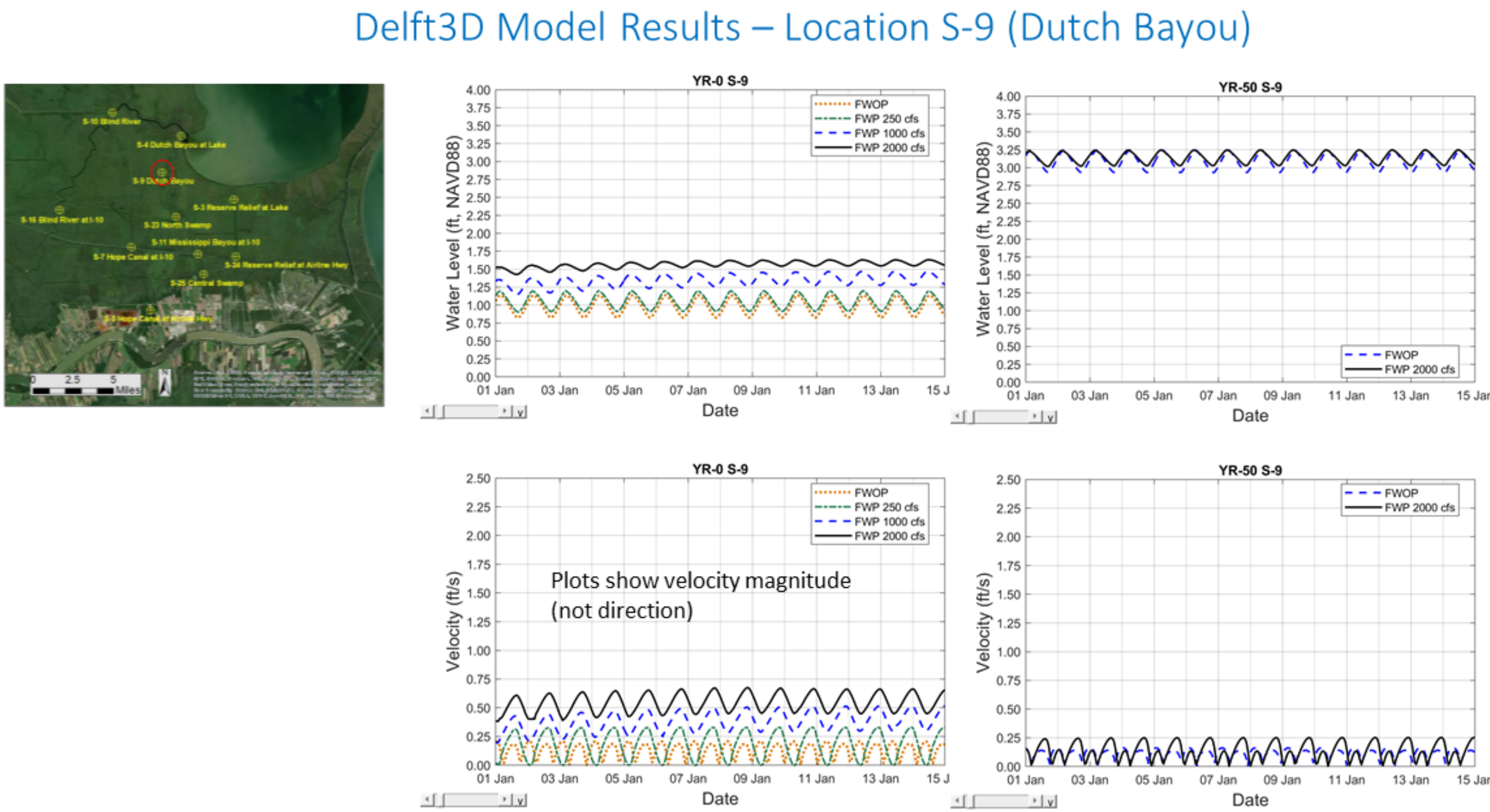


Figure C2. Predicted water surface elevation (upper panel) and velocity (lower panel) profiles over the model simulation period at location S-9 (Dutch Bayou).

Delft3D Model Results – Location S-23 (Mississippi Bayou)

This location can be used for representative mid-swamp water levels and velocity magnitude

<u>YR 0</u>
Normal is 1 ft, NAVD88
250 cfs adds 0.1 ft
1000 cfs adds 0.7 ft
2000 cfs adds 0.9 ft

<u>YR 50</u>
Normal is 3 ft, NAVD88
2000 cfs adds 0.2 ft

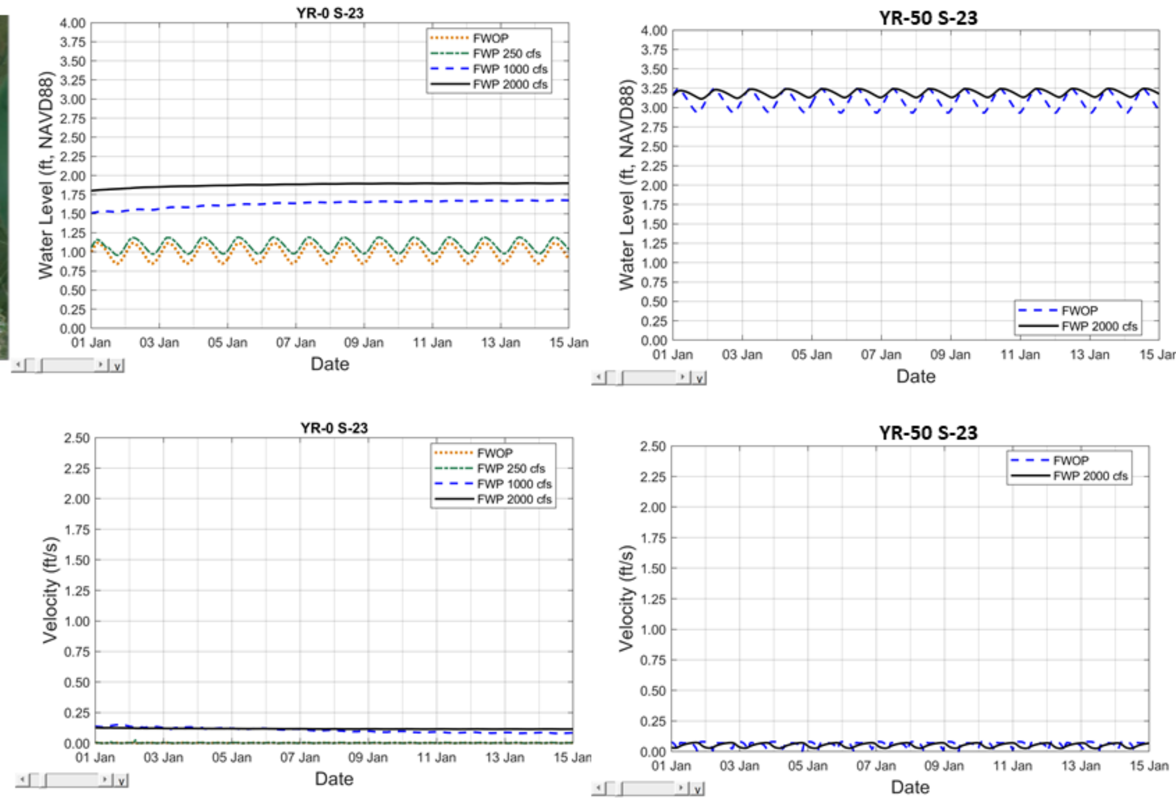


Figure C3. Predicted water surface elevation (upper panel) and velocity (lower panel) profiles over the model simulation period at location S-23 (North Swamp).

Delft3D Model Results – Location Relief Canal at WSLP

Indication of effects near the WSLP drainage structures

YR 0
Normal is 1 ft, NAVD88
250 cfs has no effect
1000 cfs adds 0.2 ft
2000 cfs adds 0.3 ft

YR 50
Normal is 3 ft, NAVD88
2000 cfs adds 0.1 ft

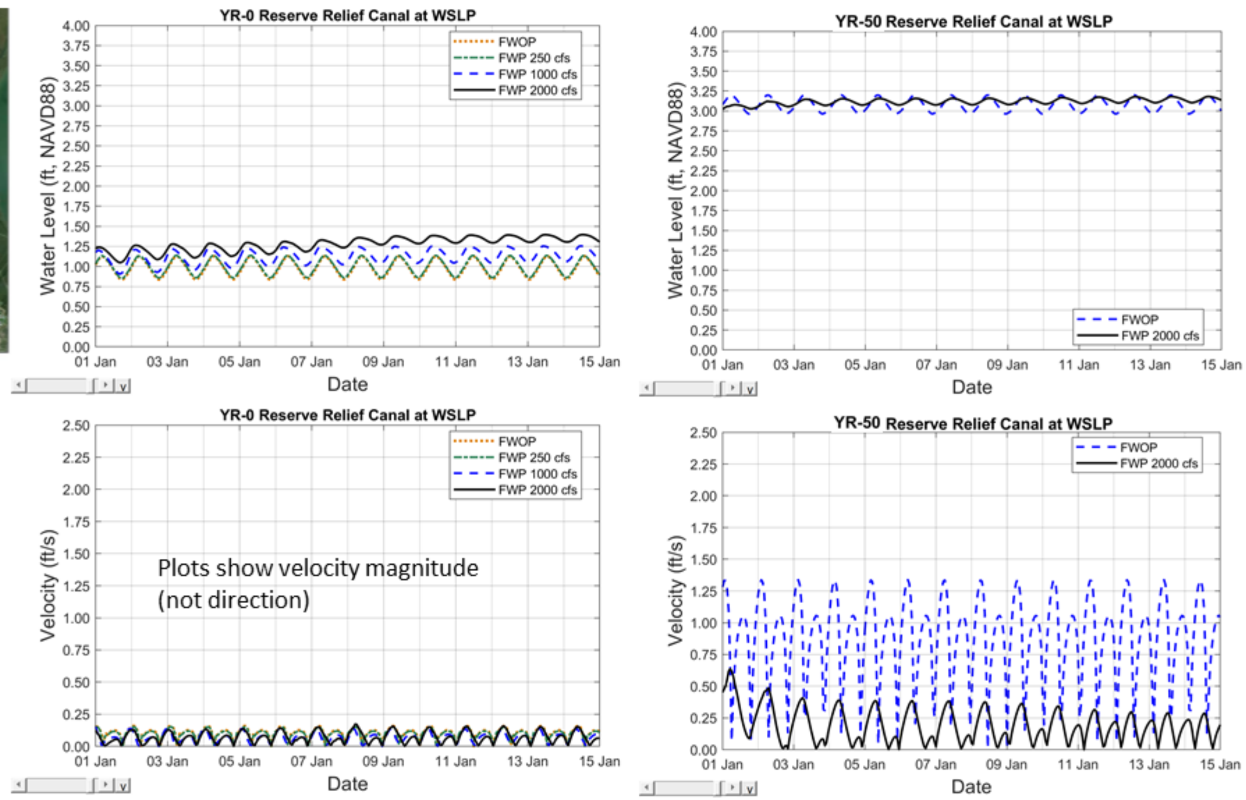


Figure C4. Predicted water surface elevation (upper panel) and velocity (lower panel) profiles over the model simulation period in Relief Canal near WSLP levee.

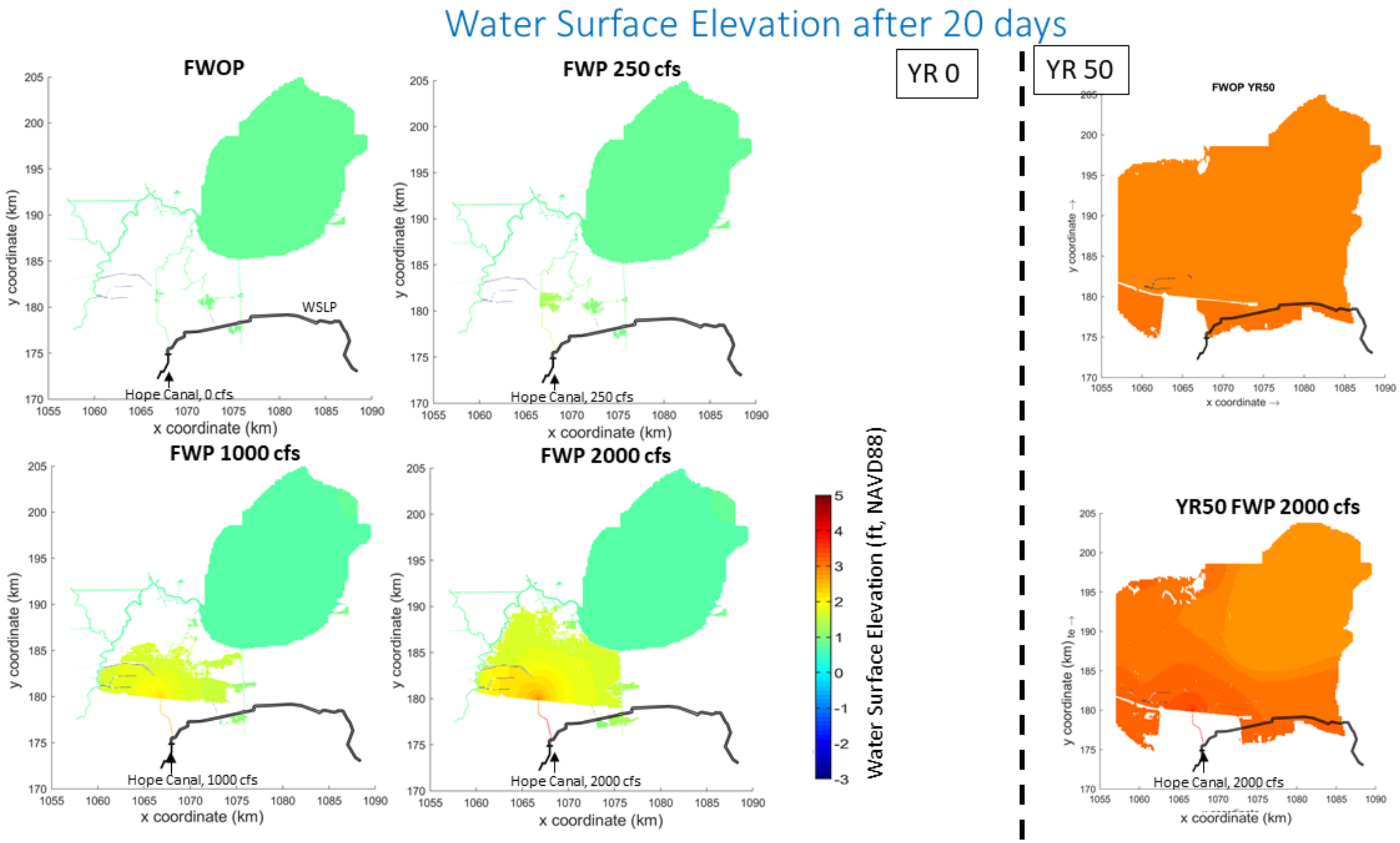


Figure C5. Predicted water surface elevation contours at the end 20 days.

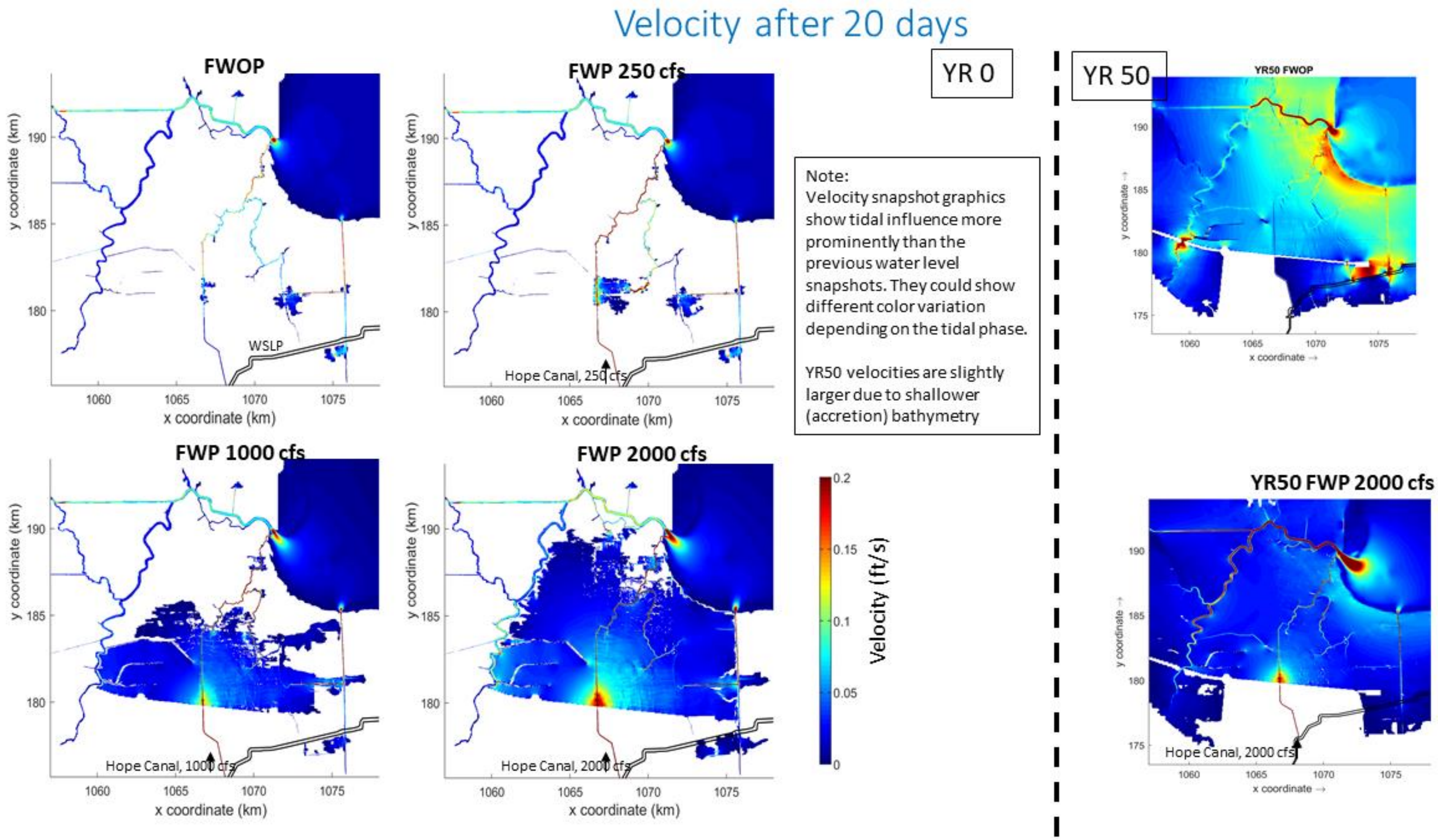
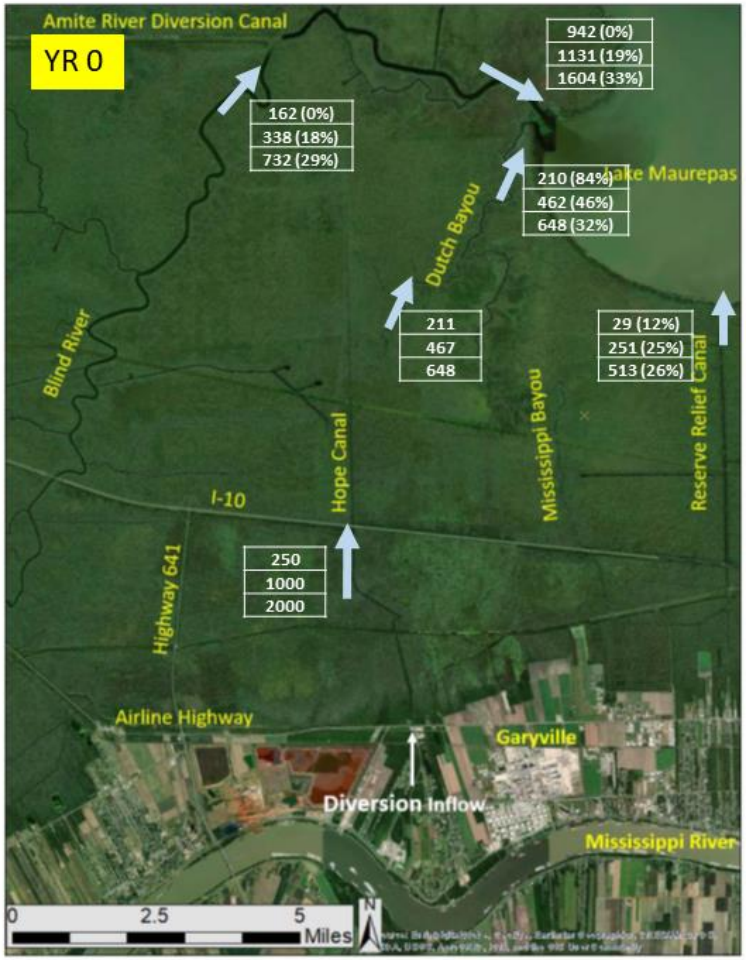
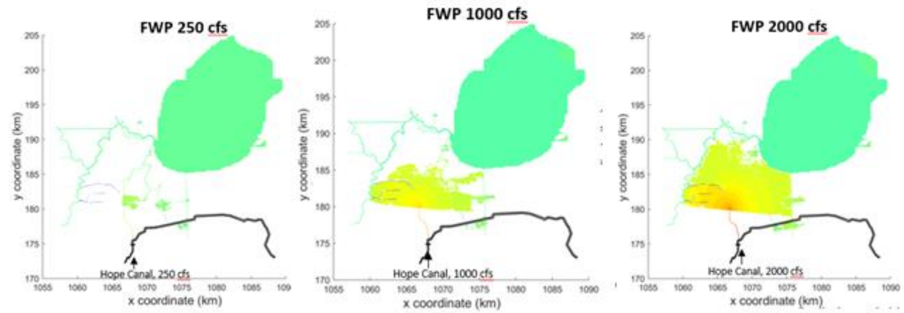


Figure C6. Predicted velocity contours at the end of 20 days.



General Flow Distribution – YR 0 FWP

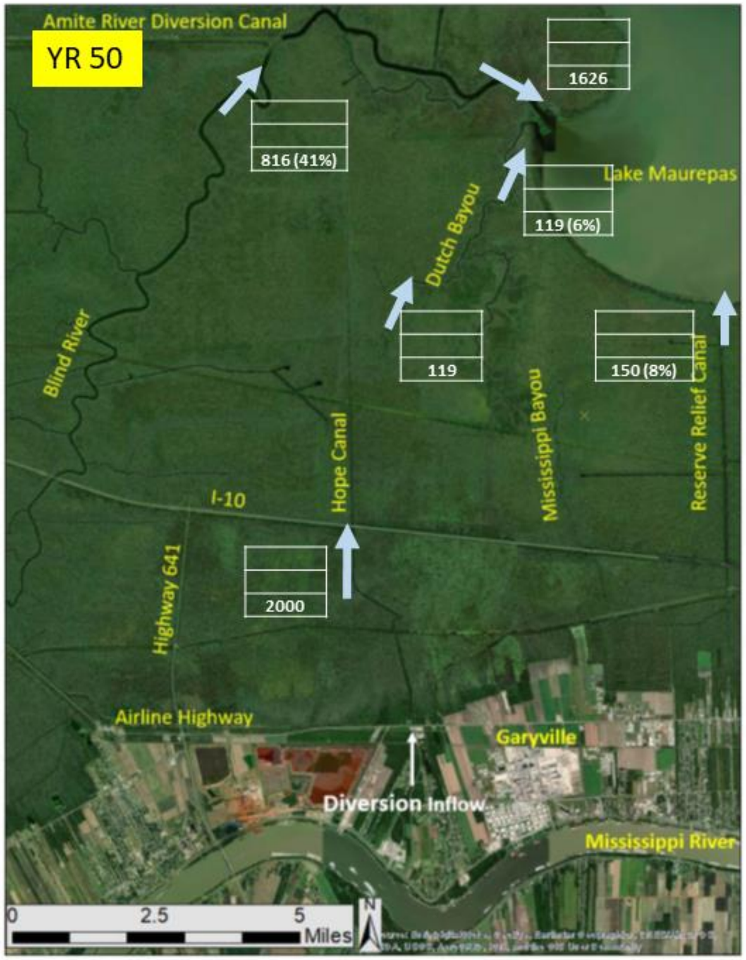


Diversion Inflow (cfs)	Div. flow existing through Dutch Bayou (cfs)	Div. flow existing through the Blind River (cfs)	Div. flow exiting through Reserve Canal (cfs)
250	210 (84%)	0%	29 (12%)
1000	462 (46%)	176 (18%)	251 (25%)
2000	648 (32%)	570 (29%)	513 (26%)

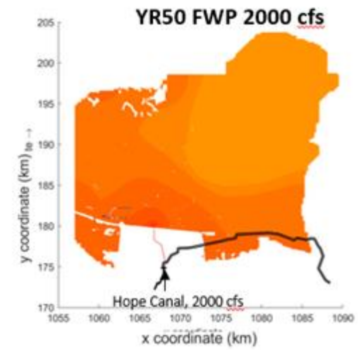
Notes:

1. The columns will not add up to 100% because some flow enters the lake from its banks
2. FWOP/Base flow (not shown) is subtracted for each number

Figure C7. Predicted flow distribution (Year 0 conditions).



General Flow Distribution – YR 50 FWP



Diversion Inflow (cfs)	Div. flow existing through Dutch Bayou (cfs)	Div. flow existing through the Blind River (cfs)	Div. flow exiting through Reserve Canal (cfs)
2000	119 (6%)	816 (41%)	150 (8%)

Note:

1. The columns will not add up to 100% because some flow enters the lake from its banks. This especially true for YR 50 because the entire swamp is under water.

Figure C8. Predicted flow distribution (Year 50 conditions).

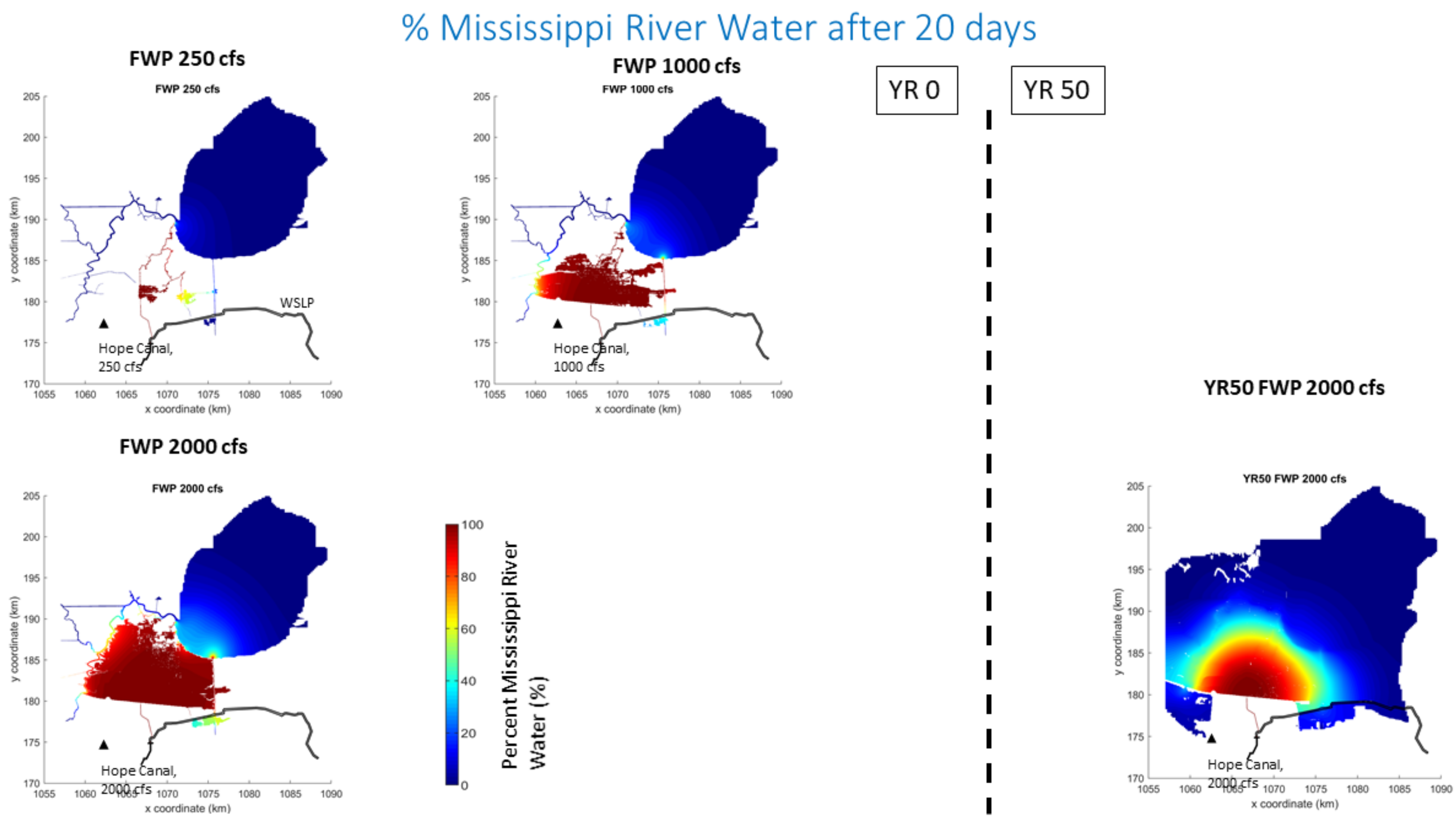


Figure C9. Predicted percent Mississippi River water contours at the end of 20 days.

TN, TP conc. after 10 & 20 days FWP-2000 cfs

YR 0

	TN (mg/L)	TP (mg/L)
Average of 2 stations*	2.21	0.250
Background conc.	0.60	0.10
Values to use in model	1.61	0.150

The average TN and TP concentrations for the Mississippi River for January 1 – August 31 based on USGS data from Baton Rouge and Belle Chasse during 2004 – 2018.

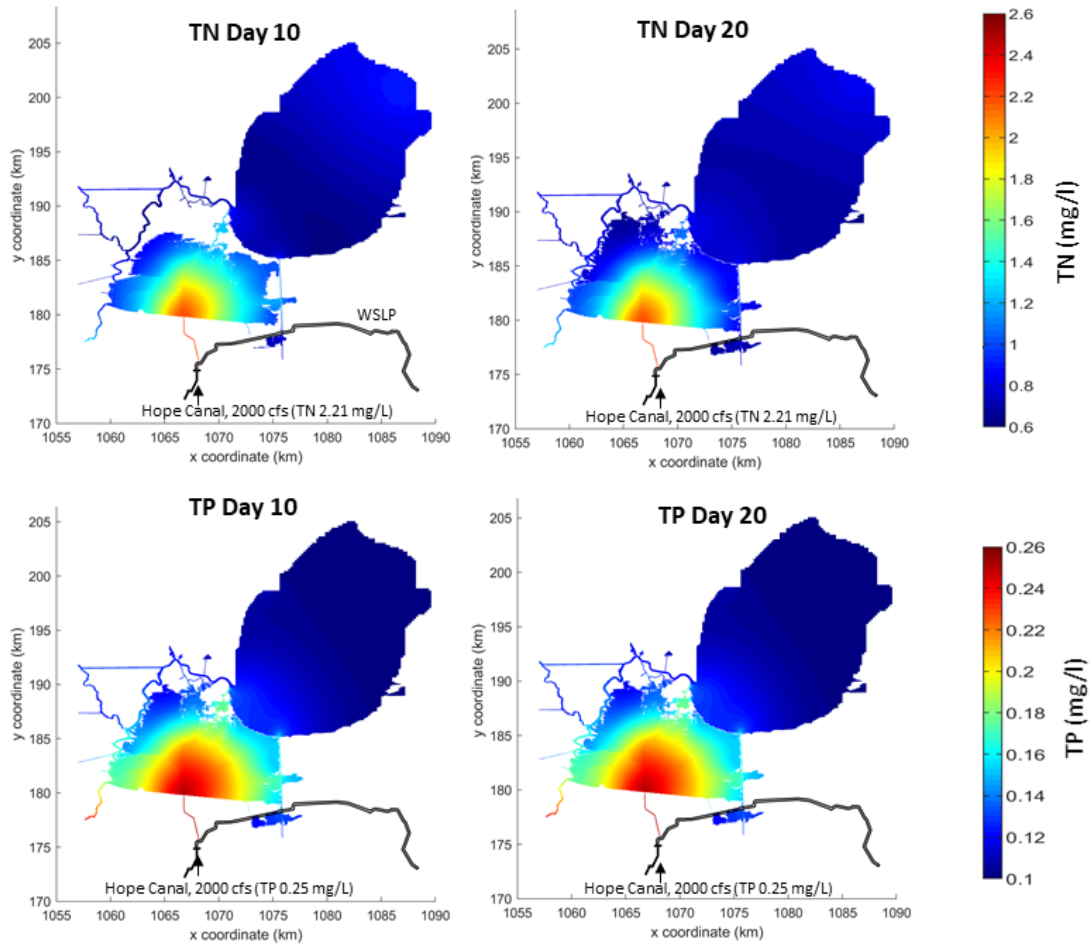


Figure C10. Predicted TN and TP concentrations at the end of 10 and 20 days (Year 0 conditions).

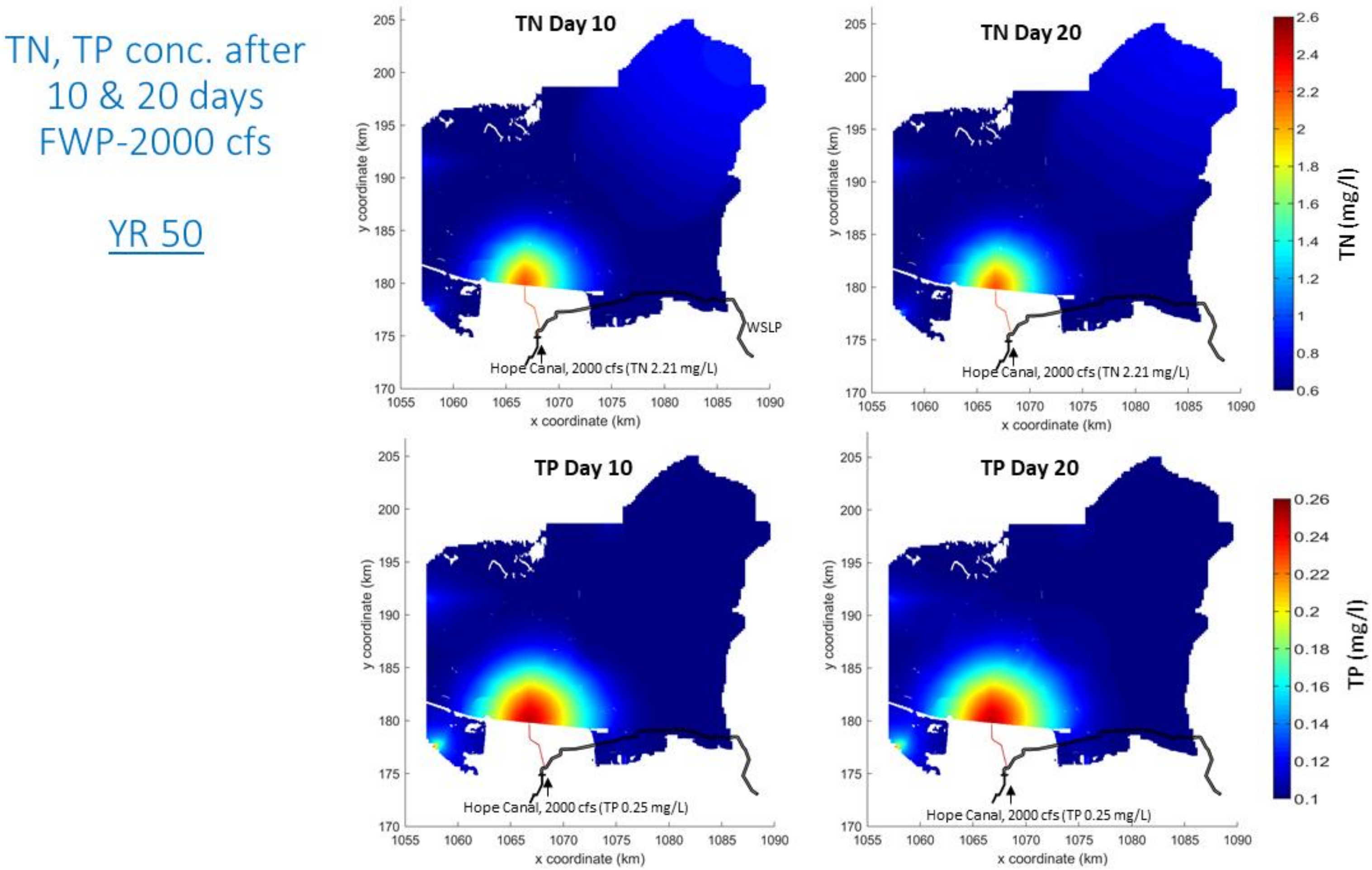


Figure C11. Predicted TN and TP concentrations at the end of 10 and 20 days (Year 50 conditions).

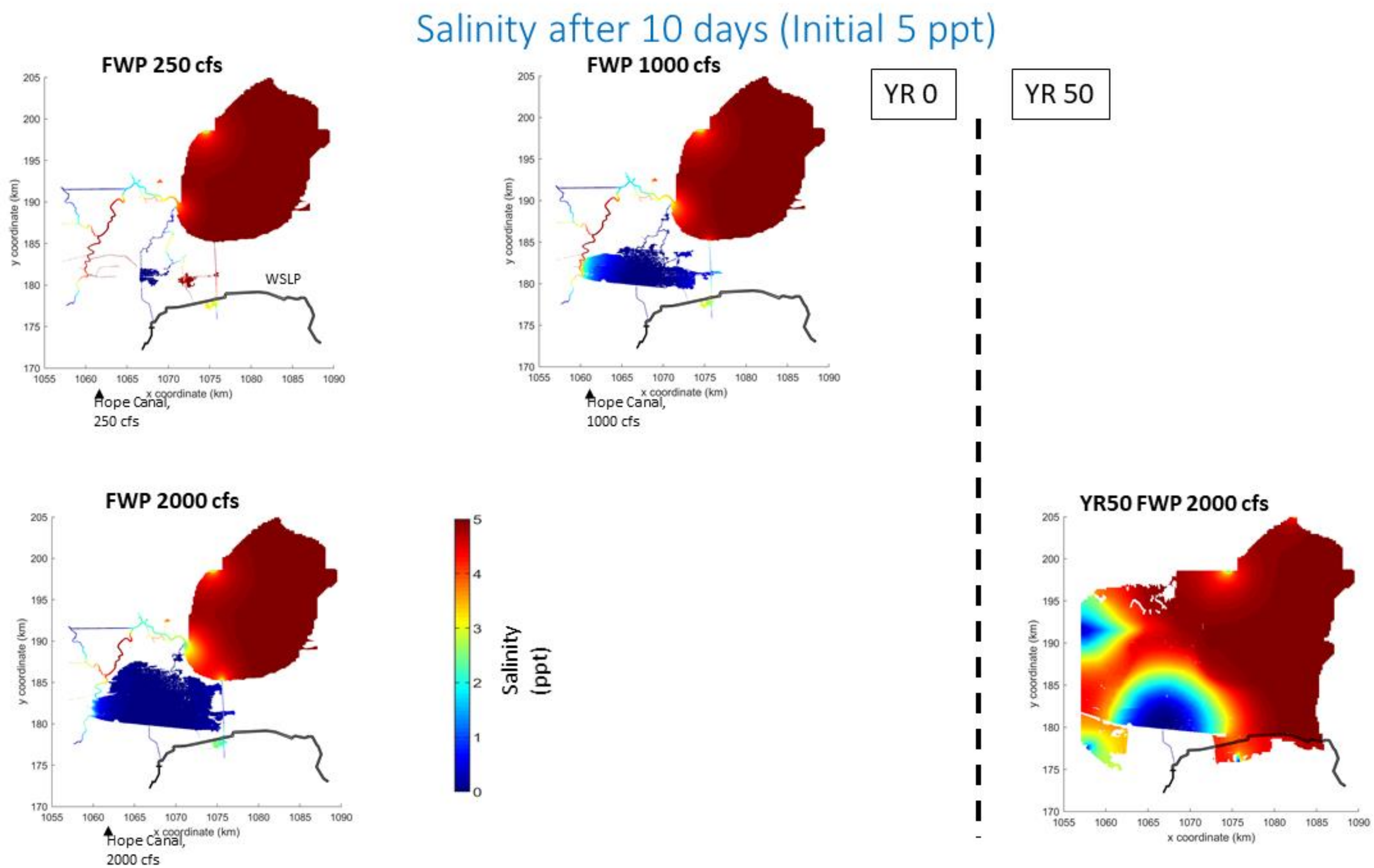


Figure C12. Predicted salinity concentrations at the end of 10 days.

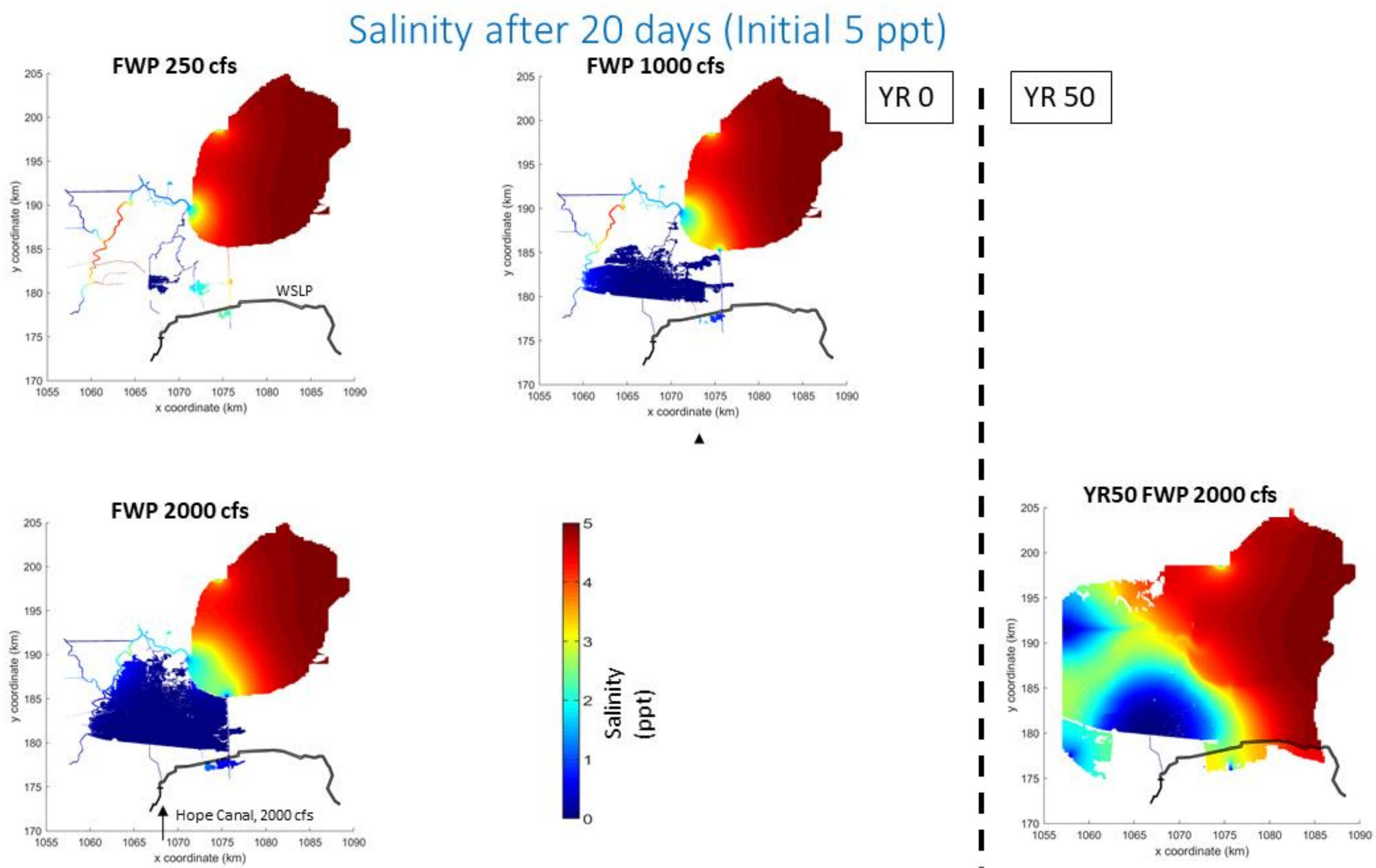


Figure C13. Predicted salinity concentrations at the end of 20 days.

APPENDIX D

Information from Published Literature
Used to Develop Loss Rates

Table D.1. Information from published literature used to develop loss rates for TN.

Description or name of wetlands	TN conc. entering wetland (mg/L)	TN conc. leaving wetland (mg/L)	TN percent reduction (%)	Hydraulic residence time (days)	First order decay rate for TN (1/day)	Average depth (m)	"k" value for PkC* model (m/yr)	Comments
Wetlands below Caernarvon Diversion [1]	1.94	0.51 – 0.89 [A]	38% [B]	"about two weeks"	0.034	not reported	--	Data were collected during a March 2001 pulse; reductions measured over a distance of about 33 – 39 km. Receives water from Mississippi River.
Fourleague Bay [2]	1.2 – 1.6	0.4 – 0.6	Feb: 42% [C] Mar: 38% [C] Apr: 37% [C]	Feb: 5.3 Mar: 5.0 Apr: 18.7	Feb: 0.103 Mar: 0.096 Apr: 0.025	~ 1	Feb: 37.6 Mar: 34.9 Apr: 9.0	Data collected during Feb. – April 1994. This is an open waterbody. Primary source of nutrients is Atchafalaya River.
City of Mandeville – Bayou Chinchuba wetland [3]	7.5	--	65%	77 [D]	0.014	approx. 0.3	1.5	Data collected during Sep. 1998 – Oct. 2000. This is a forested wetland receiving treated municipal wastewater.
City of Thibodaux treatment wetland [4]	12.6	1.08	91%	120	0.021	0.33	2.4	Data were collected during Mar. 1992 – Mar. 1994. This is forested wetland receiving treated municipal wastewater.
City of Luling treatment wetland [5]	7.06	1.18	83%	512 [D]	0.003	not reported	--	Data were collected during 2006 – 2013. This is forested wetland receiving treated municipal wastewater.
City of Breaux Bridge treatment wetland [5]	8.44	1.38	84%	410 [D]	0.004	not reported	--	Data were collected during 2001 – 2013. This is forested wetland receiving treated municipal wastewater.
Richland-Chambers treatment wetlands in Texas [6] [E]	PS1: 4.95 PS2: 4.43 PS3: 4.43 FSS: 3.53	PS1: 1.32 PS2: 1.14 PS3: 1.36 FSS: 1.44	PS1: 73% PS2: 74% PS3: 69% FSS: 59%	PS1: 9.2 PS2: 7.8 PS3: 11.2 FSS: 8.2	PS1: 0.144 PS2: 0.174 PS3: 0.105 FSS: 0.110	PS1: 0.29 PS2: 0.25 PS3: 0.28 FSS: 0.40	PS1: 33.0 PS2: 55.4 PS3: 29.0 FSS: 32.8	Data were collected during Nov. 1993 – Jul. 2000 for pilot systems and Jun. 2003 – May 2008 for field scale system. Inflow is from Trinity River.

Table D.1 Information from published literature used to develop loss rates for TN. (continued)

Description or name of wetlands	**TN conc. entering wetland (mg/L)**	**TN conc. leaving wetland (mg/L)**	**TN percent reduction (%)**	**Hydraulic residence time (days)**	**First order decay rate for TN (1/day)**	**Average depth (m)**	**"k" value for PkC* model (m/yr)**	**Comments**
Stormwater treatment wetlands in North Carolina [7]	0.74 – 2.69	0.56 – 2.06	not calculated	0.1 – 3.0	0.056 – 1.26 [F]	0.1 – 0.3	5.1 – 63.1 (median = 46.1)	Ranges are for 10 constructed wetlands receiving stormwater in different regions of North Carolina.
Olentangy River Wetland Research Park [8]	2.90 [G]	1.97 [G]	31.9%	3.7 [G]	0.104	approx. 0.4 [G]	16.1	Data were collected during 2004 – 2010. Inflow is from Olentangy River. Located in Ohio.
Des Plaines River Experimental Wetlands [9] [H]	< 0.5 to ~ 7.5 [I]	0.5 to 1.5 [I]	EW3: 54% EW4: 75% EW5: 59%	EW3: 12 EW4: 95 EW5: 13	EW3: 0.065 EW4: 0.015 EW5: 0.069	0.6 – 0.7 [G]	EW3: 14.6 EW4: 3.6 EW5: 16.7	Data were collected during Apr. – Nov. 1991. Inflow is from Des Plaines River. Located in Illinois.

Notes:

A. Concentrations leaving the wetland are affected by dilution as well as other (e.g., biological and chemical) processes.
B. The effects of dilution were excluded in the calculations for this reduction percentage.
C. Percent reduction was calculated as 100% minus the percent exported from the bay into the Gulf of Mexico.
D. Estimated value obtained from Table 1 in Hunter et. al. (2009).
E. PS1 = Pilot system #1, PS2 = Pilot system #2, PS3 = Pilot system #3, FSS = Fields scale system.
F. Calculated as "k" value for PkC* model divided by average depth. "k" values were calculated by the author.
G. Calculated using other information in the article.
H. EW3 = Experimental wetland #3, EW4 = Experimental wetland #4, EW5 = Experimental wetland #5.
I. Estimated from Figure 4 (time series plot) in article.

References:

[1] Lane et. al. (2004)
[2] Perez et. al. (2011)
[3] Brantley et. al. (2008)
[4] Zhang et. al. (2000)
[5] Hunter et. al. (2018)
[6] Kadlec et. al. (2011)
[7] Merriman et. al. (2017)
[8] Mitsch et. al. (2014)
[9] Phipps and Crumpton (1994)

Table D.2. Information from published literature used to develop loss rates for TP.

Description or name of wetlands	**TP conc. entering wetland (mg/L)**	**TP conc. leaving wetland (mg/L)**	**TP percent reduction (%)**	**Hydraulic residence time (days)**	**First order decay rate for TP (1/day)**	**Average depth (m)**	**"k" value for PkC* model (m/yr)**	**Comments**
Wetlands below Caernarvon Diversion [1]	0.16	0.059 – 0.065 [A]	35% [B]	"about two weeks"	0.031	not reported	--	Data were collected during a March 2001 pulse; reductions measured over a distance of about 33 – 39 km. Receives water from Mississippi River.
Fourleague Bay [2]	0.11 – 0.15	0.06 – 0. 10	Feb: 0% [C] Mar: 12% [C] Apr: 58% [C]	Feb: 5.3 Mar: 5.0 Apr: 18.7	Feb: 0 Mar: 0.025 Apr: 0.046	~ 1	Feb: 0 Mar: 9.1 Apr: 16.9	Data collected during Feb. – April 1994. This is an open waterbody. Primary source of nutrients is Atchafalaya River.
City of Mandeville – Bayou Chinchuba wetland [3]	2.0	--	50%	77 [D]	0.009	approx. 0.3	1.0	Data collected during Sep. 1998 – Oct. 2000. This is a forested wetland receiving treated municipal wastewater.
City of Thibodaux treatment wetland [4]	2.46	0.85	65%	120	0.009	0.33	1.1	Data were collected during Mar. 1992 – Mar. 1994. This is forested wetland receiving treated municipal wastewater.
City of Luling treatment wetland [5]	2.34	0.51	78%	512 [D]	0.003	not reported	--	Data were collected during 2006 – 2013. This is forested wetland receiving treated municipal wastewater.
City of Breaux Bridge treatment wetland [5]	2.42	0.47	81%	410 [D]	0.004	not reported	--	Data were collected during 2001 – 2013. This is forested wetland receiving treated municipal wastewater.
Richland-Chambers treatment wetlands in Texas [6] [E]	PS1: 0.727 PS2: 0.719 PS3: 0.724 FSS: 0.888	PS1: 0.457 PS2: 0.342 PS3: 0.347 FSS: 0.539	PS1: 37% PS2: 52% PS3: 52% FSS: 39%	PS1: 9.2 PS2: 7.8 PS3: 11.2 FSS: 8.2	PS1: 0.050 PS2: 0.095 PS3: 0.066 FSS: 0.061	PS1: 0.29 PS2: 0.25 PS3: 0.28 FSS: 0.40	PS1: 6.2 PS2: 10.9 PS3: 5.7 FSS: 10.7	Data were collected during Nov. 1993 – Jul. 2000 for pilot systems and Jun. 2003 – May 2008 for field scale system. Inflow is from Trinity River.

Table D.2 Information from published literature used to develop loss rates for TP. (continued)

Description or name of wetlands	TP conc. entering wetland (mg/L)	TP conc. leaving wetland (mg/L)	TP percent reduction (%)	Hydraulic residence time (days)	First order decay rate for TP (1/day)	Average depth (m)	"k" value for PkC* model (m/yr)	Comments
Stormwater treatment wetlands in North Carolina [7]	0.17 – 0.38	0.05 – 0.48	not calculated	0.1 – 3.0	0.048 – 1.01 F	0.1 – 0.3	4.4 – 84.2 (median = 37.0)	Ranges are for 10 constructed wetlands receiving stormwater in different regions of North Carolina.
Olentangy River Wetland Research Park [8]	0.148 G	0.085 G	42.7%	4.1 G	0.136	approx. 0.4 G	21.2	Data were collected during 1994 – 2001 and 2003 – 2010. Inflow is from Olentangy River. Located in Ohio.
37 large constructed wetlands [9]	median = 0.114	median = 0.038	variable	variable	--	variable	median = 12.5	This is literature review of wetlands with measured data; the PkC* model was calibrated for each system.

Notes:

A. Concentrations leaving the wetland are affected by dilution as well as other (e.g., biological and chemical) processes.
B. The effects of dilution were excluded in the calculations for this reduction percentage.
C. Percent reduction was calculated as 100% minus the percent exported from the bay into the Gulf of Mexico.
D. Estimated value obtained from Table 1 in Hunter et. al. (2009).
E. PS1 = Pilot system #1, PS2 = Pilot system #2, PS3 = Pilot system #3, FSS = Fields scale system.
F. Calculated as "k" value for PkC* model divided by average depth. "k" values were calculated by the author.
G. Calculated using other information in the article.

References:

[1] Lane et. al. (2004)
[2] Perez et. al. (2011)
[3] Brantley et. al. (2008)
[4] Zhang et. al. (2000)
[5] Hunter et. al. (2018)
[6] Kadlec et. al. (2011)
[7] Merriman et. al. (2017)
[8] Mitsch et. al. (2014)
[9] Kadlec (2016)

APPENDIX E

Guidance of Sea Level Rise, Subsidence, and Accretion

Annual Rates

Table 1. Annual subsidence, accretion, and eustatic seal level rise rates for Future Without Project (FWOP) and Future with Project (FWP) for use in PO-29 Mitigation Wetland Value Assessment (WVA) spreadsheet models and 50-year assumptions in Delft3D modeling effort.

	Subsidence (mm/yr)	Accretion (mm/yr)	Eustatic SLR (mm/yr)
FWOP	7.1	5.0	11.64
FWP	7.1	10.0	11.64

- Subsidence from USACE Gauge 85625: Lake Pontchartrain West End Gauge.
- Eustatic Sea Level Rise averaged over 55 years (2020-2075) from USACE Gauge 85625: Lake Pontchartrain West End Gauge.
- Accretion from Leigh Anne Sharp document using CRMS data and discussed with TAG.

Year 50 Surface Elevations

Table 2. Calendar year water surface elevation and net change of water surface elevations for application in the PO-29 Mitigation Wetland Value Assessment and 50-year assumptions Delft3D modeling effort.

Water Surface Elevation Change at TY50	
Calendar Year	WSE (m)
2020	0.27432
2025	0.33528
2070	0.85344
2075	0.9144
2020-2070 net change	0.57912
2025-2075 net change	0.57912
2020-2075 net change	0.64008

- Value in green would be added to current Water Surface Elevations in WVA and Delft3D assumptions to account for 55 years of eustatic sea level rise (50 years of project plus 5 years of engineering and design and construction).
- CPRA proposes using value and net change from 2020- 2075 to account for final E&D and construction but is open to discussing other options.

Year 50 Surface Elevations cont.

Table 3. Calendar year swamp surface elevation and net change of swamp surface elevations for application in the PO-29 Mitigation Wetland Value Assessment and 50-year assumptions Delft3D modeling effort for Future Without Project.

FWOP Swamp Surface Elevation Change			
	Subsidence (m)	Accretion (m)	Net Change (m)
2020	0.000	0.000	0.000
2070	-0.355	0.250	-0.105
2075	-0.391	0.275	-0.116

- Values in green would be added to current swamp surface elevation in WVA and Delft3dassumptions to account for subsidence and accretion (50 years of project plus 5 years of engineering and design and construction).
- CPRA proposes using value and net change from 2020- 2075 to account for final E&D and construction but is open to discussing other options.

Table 4. Calendar year swamp surface elevation and net change of swamp surface elevations for application in the PO-29 Mitigation Wetland Value Assessment and 50-year assumptions Delft3D modeling effort for Future with Project.

FWP Swamp Surface Elevation Change			
	Subsidence (m)	Accretion (m)	Net Change (m)
2020	0.000	0.000	0.000
2070	-0.355	0.475	0.120
2075	-0.391	0.525	0.135

- Values in green would be added to current swamp surface elevation in WVA and Delft3dassumptions to account for subsidence and accretion (50 years of project plus 5 years of engineering and design and construction).
- Years 2020-2025 applied FWOP accretion rate to account for the five years of E&D and construction.
- CPRA proposes using value and net change from 2020- 2075 to account for final E&D and construction but is open to discussing other options.

Graphical, Conceptual Depiction of Surface Elevations in FWOP and FWP Conditions

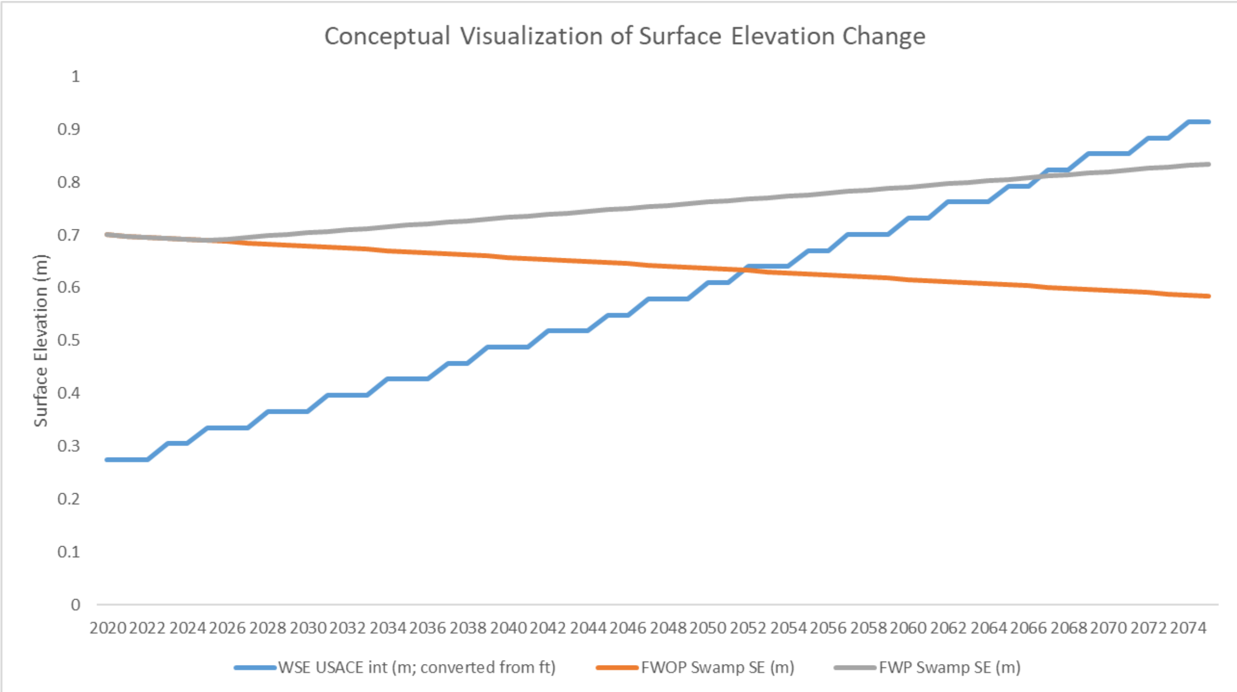


Figure 1. Graphical depiction of water and swamp surface elevations from 2020-2075 in Maurepas Swamp for FWOP and FWP assumptions using arbitrary starting swamp surface elevation of 0.5m for demonstrative purposes only.

I only provide this to demonstrate to the Habitat Evaluation Team how the assumptions chosen will be depicted over time and as a ground trothing effort to see if the assumptions made sense. Of particular note here is that the initial swamp surface elevation is arbitrary and not necessarily reflective of current conditions. The values we have decided on will be inserted into the Delft3D model and WVA model, which will include an actual initial swamp surface elevation. As a final note the first 5 years are assumed to be for E&D and construction so the swamp surface elevations are the same for those years.

APPENDIX F

Adjustment of Velocity Measured at Gage S-9 in Dutch Bayou

ADJUSTMENT OF VELOCITY MEASURED AT GAGE S-9
IN DUTCH BAYOU

Presented to the reviewers of the US Army Corps of Engineers on September 16, 2020

Executive Summary

To understand and explain the difference in the observed and modeled velocity at gage S-9, FTN Associates, Ltd. (FTN) obtained the original S-9 record file from the 2004 data collection effort. The review of the original data file revealed that to accurately compare the model velocity to the measured velocity, the model velocity should be converted to what the observed velocity data represents. The observed velocity represents velocity of a layer of water which is 3 ft below the normal water surface. The model velocity represents average/bulk velocity of the entire water column. Therefore, for a proper comparison, the model velocity was converted in the following two ways.

1. The model outputs X direction (east) and Y direction (north) velocity separately. The two velocities were projected and added along the main direction of Dutch Bayou to obtain the total velocity for a true comparison. The velocity reported in the S-9 data file are reported as measurements along this main channel direction.
2. The gage measured velocity of a water layer 3 ft below the surface (channel is approximately 11 ft deep) while the model produces the bulk velocity averaged over the entire water column. In natural channels, the velocities are highest in the upper layers and gradually decrease toward the channel bottom. Therefore, the velocities measured by the gage are greater than the bulk average velocities of the entire water column. For an accurate comparison, the model velocity is converted to the velocity in the upper layer where the gage was placed.

After adjusting for the above two effects, the modeled velocities (magenta lines) are in good agreement with the observed velocities for the normal conditions (Figure F4) and tropical storm conditions (Figure F5).

Background

CPRA contracted FTN to develop a 2D Delft3D model to simulate water level, velocity, and water quality throughout the Maurepas swamp (bounded by Interstate 10, Blind River, and Lake Maurepas) under the proposed diversion scenarios. The model was calibrated for normal conditions and tropical storm conditions using previously collected data in 2004 at 11 gages. All gages recorded continuous hourly water levels and one gage, S-9, in Dutch Bayou recorded velocity in the direction of the longitudinal axis of the channel. The accuracy of model velocity calibration at S-9 has been discussed with the USACE reviewers over the past few months. The previous discussion is in a comments/response document dated May 14, 2020.

Subsequently, FTN revisited the original LSU file of the S-9 gage velocity data and the modeling results. The present document summarizes findings of the additional review in these sections:

- Section 1: Observations on the original S-9 velocity records.
- Section 2: Observations on the velocity predicted by the model at S-9.
- Section 3: Supplementary calculations.
- Section 4: Updated derivations without assumption of shear velocity equal to canopy velocity and additional velocity calculations.

The purpose of this review and analysis is to provide additional information and guidance to help in the evaluation of the model velocity calibration, sources of uncertainty in the observed data, and interpretation of the model data.

Section 1: Observations on the original S-9 velocity records

The continuous velocity records were measured with an Acoustic Doppler Current Profiler (ADCP) instrument. From the file extensions (.arg), it appears to be a SonTek-Argonaut ADV instrument. It was mounted on the side of the channel at a depth of about 3 ft below the water surface looking across the channel. It recorded velocity using a cross-channel beam at a single depth of 3 ft. The velocity recorded is the average velocity of the water passing through the beam (i.e., a lateral average of velocities at that depth). It does not represent the velocity of the entire water column passing through the channel. In comparison, the model predicts velocities as averaged over the entire depth of the water column. The velocity measured by the gage at a single-depth near the water surface is expected to be higher than the depth-averaged (water column) velocities.

The Dutch Bayou cross-section at this location is about 11 ft deep.

Section 2: Observations on the velocity predicted by the model at S-9

A closer review of the plotted model data revealed that, for a consistent comparison with the velocity records, the model output should be further processed to represent what the observed velocity represents and that is (a) velocity along the channel, and (b) velocity at a specific depth and not the entire water column.

To this end, the model output velocities were transformed into the main (primary) velocity along the channel direction and to the specific depth of the instrument as below.

a. <u>Transforming model velocities to the main channel direction</u>

The Maurepas Delft3D model produces velocity output in terms of X (east) and Y (north) components at each node of the model grid. In the velocity charts presented in the report, only the X-component was plotted inadvertently. In reality, the resultant primary velocity component along the channel direction should have been calculated and plotted. This is the velocity recorded by the ADCP side-looker instrument.

Figure F1 shows an example instantaneous vector plot from the model (an outgoing tide) around the S-9 gage location. The model outputs the Vx (Velocity in the X-direction) and Vy (Velocity in the Y-direction) velocities at the S-9 node. To obtain the true along-channel velocity (that the ADCP measures), the Vx and Vy velocities should be projected along the main channel direction to obtain the total projected velocity, Vp. The angle for projections is obtained from Vx and Vy magnitudes as shown in Figure F1. The values of theta from the model were mostly between 52 to 55 degrees (measured clockwise from positive x axis) during the outgoing normal tide and 180+52 to 180+55 degrees (measured clockwise from positive x axis) during incoming normal tide. These are consistent with the geometric orientation of the channel with respect to the cartesian co-ordinate system.

Therefore, the correct velocity in the direction of the longitudinal axis of the channel is calculated as:

$$Vp=Vx*\cos(\theta)+Vy*\sin(\theta)$$

Figures F2 and F3 show comparison of the Vx, Vy and Vp time-series with the recorded velocities. Note the first 3-5 days of the model runs probably suffer from initial conditions (which start from zero velocities) in the domain.

b. Transforming model depth-averaged velocities to the depth of the instrument

The USGS technical field guidance (USGS, 2010) requires that side-looker ADCPs intended to estimate depth-averaged velocities, be placed at 0.6 of the depth below the water surface (approximately 6.6 ft below the water level and 4.4 ft above the stream bed in this case where the depth of the channel is about 11 ft) in order to reliably estimate the depth-averaged velocities in streams. In this case, the ADCP side-looker was placed at about 3 ft below the water surface. It is, therefore, expected to over-predict the depth-averaged velocity in the stream. This overprediction can become particularly important for vegetated streams like Dutch Bayou which have larger roughness heights than conventional sand or silty bed streams.

Considering the velocity variation in the vertical direction, it can be shown that the depth-averaged velocity predicted by the model should be increased by a conversion factor of about 1.4 to represent velocity measured at a depth about 3 ft below the water surface. The estimation of this conversion factor is described in Section 3.

Figures F4 and F5 show velocity time series adjusted by the conversion factor and their comparison with the observed velocity.

Summary

The velocities recorded at gage S-9 were at a specific depth and expected to be larger than the average velocity in the channel because of the vertical variation in the channel velocity that exists in reality. The velocities produced by the model are depth-averaged values for the entire water column. When adjusted for vertical variation and projected correctly along the main channel direction, the model velocities agree well with the recorded velocities.

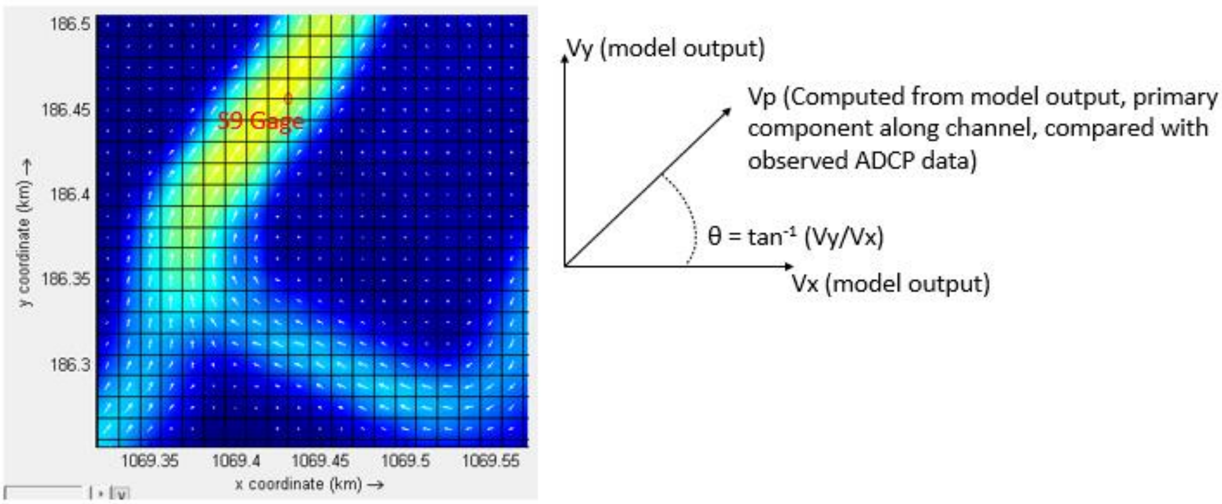


Figure F1. Left panel: Vector plot of velocities in the Dutch Bayou in vicinity of the S-9 Gage during a typical outgoing tide. Contours are colored by the velocity magnitude. The legend is not shown because the exact values are not relevant for this discussion. Right panel: Definition of resultant angle for calculation of the primary velocity (along channel) from the Vx and Vy velocities.

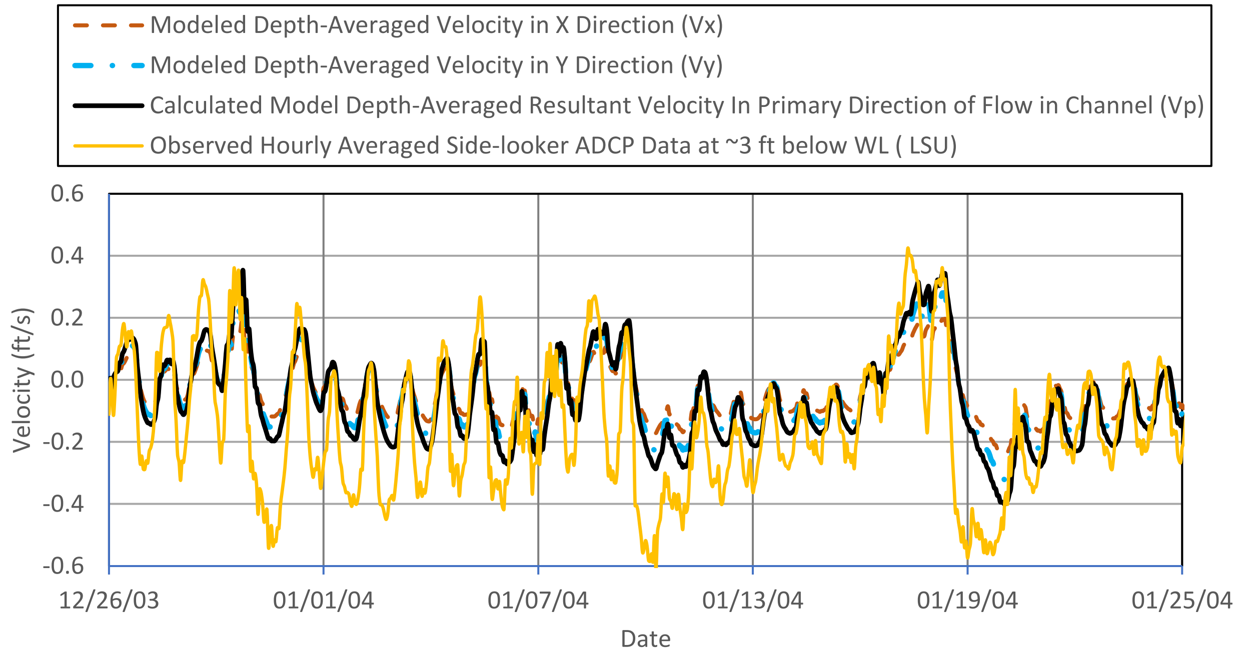


Figure F2. Normal Conditions Time Series of Velocities (S-9 Gage) Comparison. Previously in the report, only the X Direction (Vx) velocity from the model was compared against the observed data.

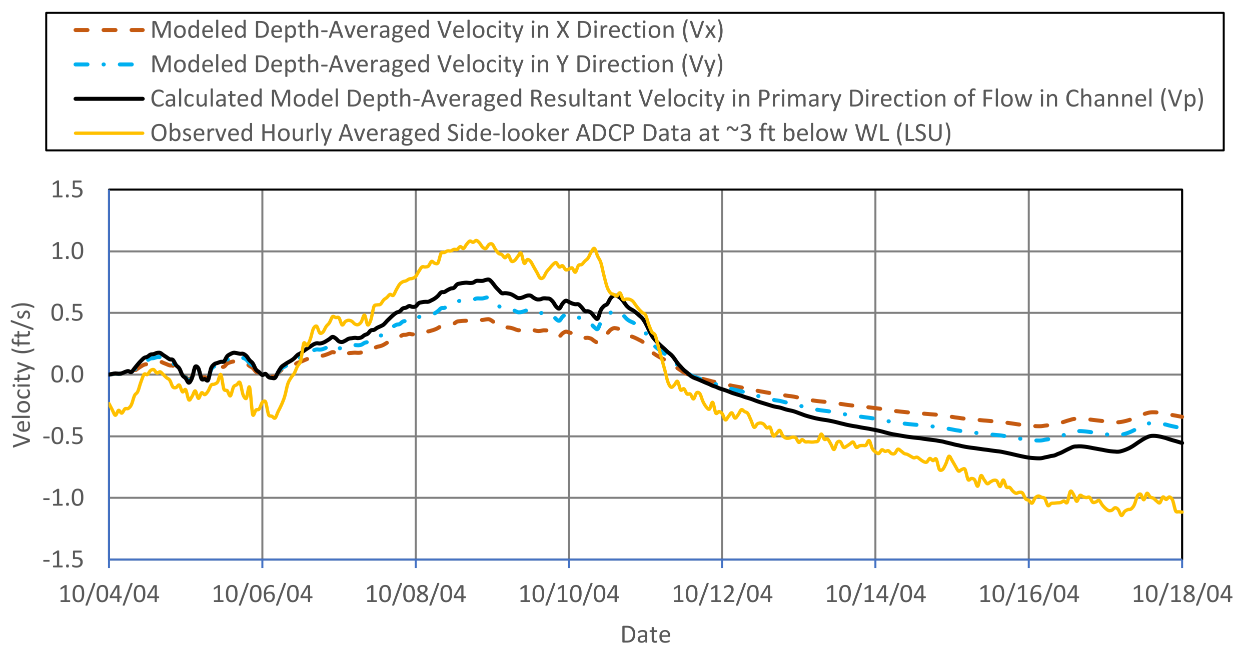


Figure F3. Tropical Storm Conditions Time Series of Velocities (S-9 Gage) Comparison. Previously in the report, only the X Direction (Vx) velocity from the model was compared against the observed data.

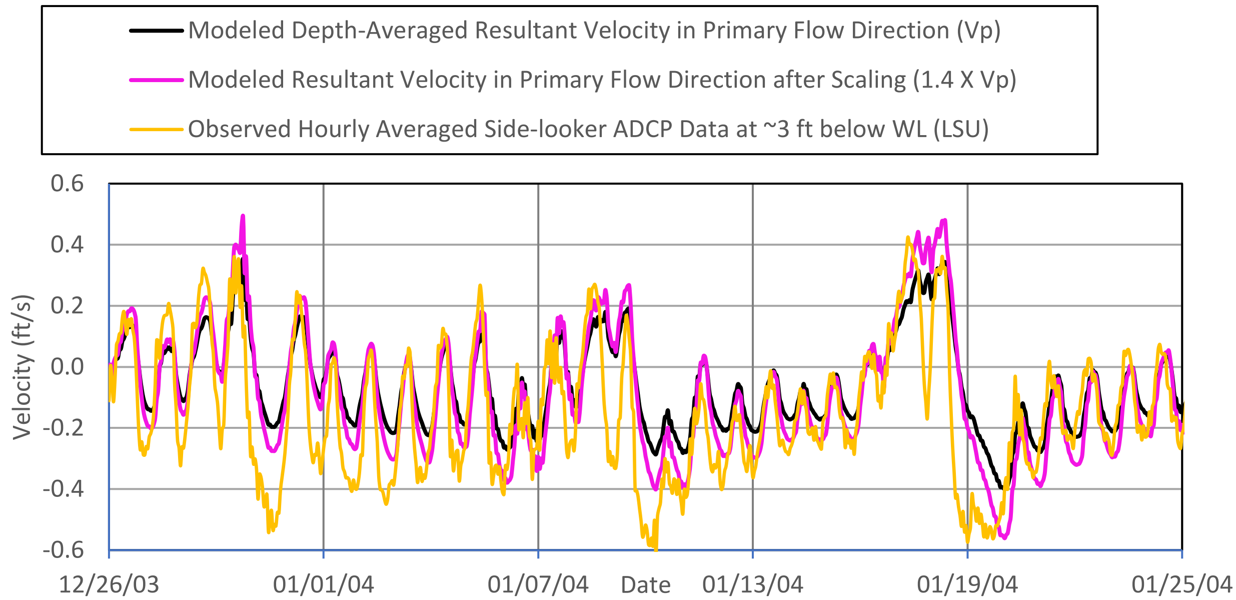


Figure F4. Normal Conditions Time Series of Velocities (S-9 Gage) Comparison. A scale factor of 1.4 is applied to the projected depth-averaged velocity to convert to velocity at a depth of 3 ft measured by the gage.

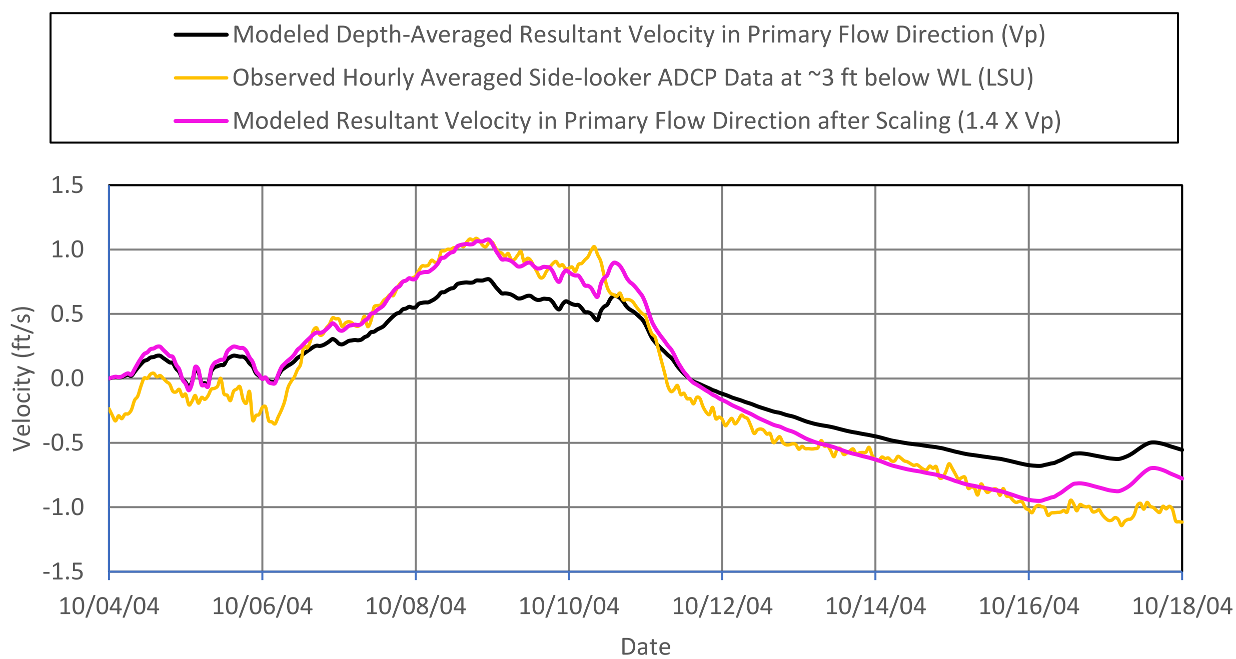


Figure F5. Tropical Storm Conditions Time Series of Velocities (S-9 Gage) Comparison. A scale factor of 1.4 is applied to the projected depth-averaged velocity to convert to velocity at a depth of 3 ft measured by the gage.

Section 3: Supplementary calculations

This section describes the calculations required to transform the depth-averaged velocity produced by the model to the velocity at a specific depth in a vegetated channel.

The USGS technical field guidance (USGS, 2010) requires the that side-looker ADCPs meant to estimate depth-averaged velocities, be placed at 0.6 of the depth below the water surface (approximately 6.6 ft below the water level and 4.4 ft above the stream bed considering the water depth of 11 ft at S-9 gage) in order to reliably measure depth averaged velocities from streams. In this case, the ADCP side-looker which was placed at about 3 ft below the water surface is therefore expected to over-predict the depth-averaged velocity in the stream. This overprediction can become particularly important for vegetated streams such as Dutch Bayou which have larger roughness heights than conventional sand or silty bed streams. For a strict comparison, the vertical flow structure should be considered when interpreting the comparison between the model data and observed data. The modeled depth-averaged velocities here can be considered representative of velocities occurring within the uncertainty of typical roughness lengths associated with Submerged Aquatic Vegetation (SAV) lined channels and are consistent with near surface velocities measured by the ADCP gage.

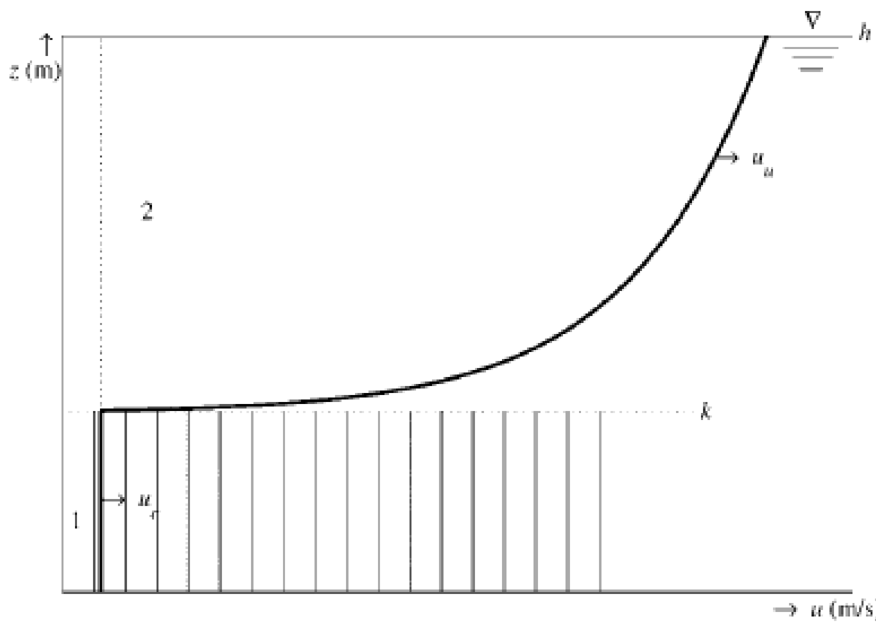


Figure 2 Representation of the vertical velocity profile in two zones for the method of effective water depth, h = water depth (m), k = vegetation height (m), u_c = uniform flow velocity profile (m/s), u_u = logarithmic flow velocity profile (m/s).

Figure F6. Typical logarithmic flow profile over a Submerged Aquatic Vegetation (SAV) canopy (Figure from Baptist et al., 2007). Note u_c is the *uniform velocity within the in-canopy layer* and u_u *logarithmic velocity in the above* canopy *layer*.

Assuming a uniform flow within the canopy and a logarithmic flow above, Baptist et al. (2007) defines the velocity in the above canopy layer, at a given elevation z (positive above the bed) as follows, where u* is the shear velocity, κ = 0.41 the Von Karman constant, z0 the roughness height:

$$u_u(z) = \frac{u_*}{\kappa} \ln\left(\frac{z-k}{z_0}\right) + u_c \quad \text{.. (1)}$$

The depth-averaged velocity in the water column above the in-canopy layer can thus be defined as:

$$\begin{aligned} \bar{u}_u &= \frac{1}{h-k} \int_k^h u_u(z)\mathrm{d}z \\ &= \frac{u_*}{\kappa} \ln\left(\frac{h-k}{z_0} - 1\right) + u_c = \frac{u_*}{\kappa} \ln\left(\frac{h-k}{e z_0}\right) + u_c \end{aligned} \quad \text{.............................. (2)}$$

Where, Baptist et al. (2007) provides analytical relations for the roughness length z0 as,

$$z_0 = (k-d) \exp\left(-\kappa \sqrt{\frac{2L}{c_p \ell}\left(1 + \frac{L}{h-k}\right)}\right)$$

Here d is the zero-plane displacement and is given as,

$$d = k - \int_0^k \frac{\exp(z/L)}{\exp(k/L)} \mathrm{d}z = k - L\left(1 - \exp\left(-\frac{k}{L}\right)\right)$$

Baptist et al. (2007) determines a best fit Cp (turbulence intensity) based on experimental data of Nepf and Vivoni (2000) on submerged flexible plastic canopies representative of SAV with "ℓ" the mixing length assumed as equal to the available length scale of eddies between the vegetation canopy. A characteristic turbulent length scale (L) associated with Cp can be therefore also defined as follows,

$$c_p = \frac{1}{20} \frac{h-k}{\ell}$$

$$L = \sqrt{\frac{c_p \ell}{C_D m D}}$$

D is the diameter of each stem and m the stem density (number of stems per unit area). C_D is the stem drag coefficient typically taken as 1 for high Reynolds number flow.

The depth averaged velocity ($\bar{u}$) in the entire water column, can be thus written as the weighted average of the velocities from the above canopy and in-canopy layers as,

$$\bar{u} = \frac{(h-k)*\bar{u}_u + k*u_c}{h} \quad \ldots\ldots\ldots\ldots\ldots (3)$$

Dividing (1) by (3) yields a simple expression for the scale factor (S(z)) linking the depth-averaged velocity to the velocity at any given elevation (z) above the bed in the above canopy layer (z>k), for SAV dominated stream flows can be obtained,

$$S(z) = \frac{u_u(z)}{\bar{u}} = \left[\frac{h * u_u(z)}{(h-k)\bar{u}_u \ + \ ku_c}\right] = \frac{h * \frac{\mathrm{u}*}{\kappa}\ln\left(\frac{z-k}{z_0}\right) + \mathrm{h}u_c}{(h-k) * \frac{u_*}{\kappa}\ln\left(\frac{h-k}{ez_0}\right) + hu_c}$$

$$S(z) = \frac{h*[A*\ln\left(\frac{z-k}{z_0}\right)+1]}{(h-k)*A*\ln\left(\frac{h-k}{ez_0}\right)+h} \quad \ldots..(4)$$

Where the factor A is written as below and shows that the scale factor is independent of the hydrulic gradient (i),

$$A = \frac{\mathrm{u}*}{\kappa u_c}$$

Where the shear velocity and in-canopy velocities can be calculated as (Baptiste et al., 2007),

$$u_* = \sqrt{g(h-k)i} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (5)$$

$$u_c = \sqrt{\frac{hi}{1/C_b^2 + (C_D m D k)/(2g)}} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Note that bed Chezy coefficient (C_b) above is considered to be without vegetation and can be taken as corresponding to an assumed Mannings of n_{NoVeg}=0.025 (unvegetated channels) as,

$$C_b = \frac{h^{1/6}}{n_{NoVeg}}$$

Wetland SAV density 'mD' (Vegetation density x Stem diameter) typically falls in the range of 0.1 to 1.0 m^{-1} (Baptist et al., 2007; Visser et al., 2013). Therefore, assuming a typical canopy height of 3 ft (approximately lower 1/3rd of the water column), D=5mm, and two extreme ranges of m=20 stems/m^2 and m=200 stems/ m^2 in equation (3), we can find (Table 1) the general range of the scale factor (S(z=8ft)) using Eqn. (4) derived above, connecting the velocity at 3 ft below the water surface (as measured by the ADCP) to the depth-averaged velocity in the water column.

Table 1. Estimation of scale factor (S(z=8ft from stream bed)) at 3 ft below the water surface to the actual depth-averaged velocity in the channel.

D (mm)	m (stms/ m^2)	h (ft)	k (ft)	Cp*l (ft)	L (ft)	d (ft)	z0 (ft)	n_{NoVeg} (s/ $m^{1/3}$)	C_b ($m^{1/2}$/s)	A	S(z=8ft)	Mean S(z=8ft)
5	20	11	3	0.4	3.6	1.0	0.25	0.025	60	0.47	1.30	**1.40**
5	200	11	3	0.4	1.1	2.0	0.37	0.025	60	1.44	1.50	

The above table and calculations indicate a scale factor of 1.40 is needed to convert the depth-averaged velocities from the model to velocity at 8 ft above the bottom which the ADCP records. As shown in Figures F4 and F5, a scaling of 1.4 also makes the modeled depth-averaged velocity match very well with the observed velocities, therefore providing a validation of the modeled velocity results.

Section 4: Additional velocity calculations

A hand calculation example where velocity at each depth is calculated using the previously stated equations (Eqns. 1, 5, and 6) and depth-averaged velocity (DAV) calculated using the analytical expressions (Eqns. 2 and 3) as well as direct computation from direct integration are compared and shown in Table 2 below. The table provides a simple check of the calculations and derivations behind those in Table 1.

For the two vegetation densities (mD=0.1 and 1.0 m^{-1}) discussed in Table 1, Table 2 shows the detailed calculations of the vertical flow profile and computation of the depth-averaged velocity both directly from numerical integration of discrete data points as well as using the analytical expression (Eqn. 3). This provides a hands-on calculation check and a verification for the derived Eqn. (4) as well.

Table 2. Detailed vertical velocity profile computations for the two cases (mD=0.1 and 1.0 m^{-1}) shown below. Assumed a hydraulic gradient of 10^{-5} as an example, S(z) is independent of the choice of the gradient as shown in Eqn. (4). DAV = depth-averaged velocity.

mD (m^{-1})	Hydraulic Gradient (i)	u* (Eqn. 5) (ft/s)	u_c (Eqn. 6) (ft/s)	Elevation from Bed (ft)	U(z) (ft/s)	DAV Calculated (ft/s)	DAV Analytically Computed (Eqn. 3) (ft/s)	S(z)= U(z)/DAV Computed
0.1	10^{-5}	0.05	0.26	0.0	0.26	0.4850	0.4851	0.54
--	--	--	--	1.0	0.26			0.54
--	--	--	--	2.0	0.26			0.54
--	--	--	--	3.0	0.26			0.54
--	--	--	--	4.0	0.43			0.89
--	--	--	--	5.0	0.52			1.07
--	--	--	--	6.0	0.57			1.18
--	--	--	--	7.0	0.61			1.26
--	--	--	--	**8.0**	0.63			**1.30**
--	--	--	--	9.0	0.66			1.36
--	--	--	--	10.0	0.67			1.38
--	--	--	--	11.0	0.69			1.42
--	--	--	--	--	--	--	--	--
1.0	10^{-5}	0.05	0.09	0.0	0.09	0.2783	0.2724	0.33
--	--	--	--	1.0	0.09		--	0.33
--	--	--	--	2.0	0.09		--	0.33
--	--	--	--	3.0	0.09		--	0.33
--	--	--	--	4.0	0.21		--	0.77
--	--	--	--	5.0	0.29		--	1.08
--	--	--	--	6.0	0.34		--	1.27
--	--	--	--	7.0	0.38		--	1.40
--	--	--	--	**8.0**	0.41		--	**1.50**
--	--	--	--	9.0	0.43		--	1.58
--	--	--	--	10.0	0.45		--	1.65
--	--	--	--	11.0	0.47		--	1.71

References

1. USGS (2010). "Discharge measurements at gaging stations: Techniques and Methods 3A-8". US Department of Interior. US Geological Survey.
2. Baptist, M. J., V. Babovic, J. R. Uthurburu, M. Keijzer, R. E. Uittenbo-gaard, A. Mynett, and A. Verwey (2007). On inducing equations for vegetation resistance, J. Hydraul. Res., 45(4), 435–450.
3. Nepf, H.M. and Vivoni, E.R. (2000). "Flow Structure in Depth-limited, Vegetated Flow". *J. Geophys. Res.* 105(C12), 28, 547–28, 557.
4. Visser, J. M., Duke-Sylvester, S. M., Carter, J., Broussard III, W.P. (2013). A computer model to forecast wetland vegetation changes resulting from restoration and protection in coastal Louisiana, 51–59.

APPENDIX G

Preliminary Polder Drainage Evaluation

1.0 INTRODUCTION

The proposed diversion conveyance channel has a high bank on either side to contain diverted river water within its banks until it reaches north of Interstate 10. The conveyance channel intercepts the existing eastward drainage from local rainfall into and through Hope Canal. This results in creation of a polder on each side of the diversion canal (Figure G1). The west polder is bounded by the diversion canal to the east, Interstate 10 to the north, Louisiana State Highway LA-641 to the west and the Airline Highway to the south. The east polder is bounded by the diversion canal to the west, Interstate 10 to the north and the proposed Westshore Lake Pontchartrain (WSLP) levee on the south. This polder can still flow east in case of the local rainfall events. However, the only drainage outlets for the west polder are the culverts under Interstate 10 and Highway LA-641.

To evaluate the changes and improvements of drainage of these polders, the Project Management Team (PMT) decided to examine drainage under 2-year (50% Annual Exceedance Probability) and 25-year (4% Annual Exceedance Probability) rainfall events under TY0 conditions.

2.0 ASSUMPTIONS AND LIMITATIONS

The Delft3d model was developed for this project is primarily to simulate the overall distribution of the diverted river water and the associated nutrient transport in Maurepas swamp. It was not constructed with the goal of guiding the design of drainage structures. The Delft3D models culverts as rectangular openings of equivalent cross-section of the actual culvert shape. The model bathymetry in the polder areas is based on the LIDAR data. This data and the model grid resolution does not capture small drainage pathways leading to the highway culverts. The vicinity of the culverts has been lowered to allow culvert to stay wet during simulations.

In spite of above limitations, the model results can provide a useful comparative analysis of drainage impacts on the polder under specified project and rainfall conditions. For the engineering design purposes, a model such as 2D HEC-RAS is recommended which has the ability to represent a variety of culvert shapes and hydraulic conditions.

3.0 METHODOLOGY

Following steps were followed to conduct the drainage analysis:

- Modify Delft3D model geometry:
    - Add culverts under I-10 and LA-641 based on data provided by LA DOTD.
    - Modify bathymetry along the two highways to enable connectivity to the lower regions.
    - Revise WSLP levee alignment per new information and add the Hope Canal drainage structure.
- Obtain the 2-year (50% Annual Exceedence Probability) and the 25-year (4% AEP), 24-hour duration rainfall estimates from the NOAA server:
    - 2- and 25-yr, 24-hour total rainfall estimates are 5.1 And 9.5 inches, respectively. The total rainfall was applied on the first day using SCS distribution.
- Develop downstream boundary conditions:
    - The normal tidal signal was added to the expected elevated water surface elevations at Pass Manchac boundary based on historical rainfall and water level data analysis.
- Develop model simulation plan consisting of several combinations of with- and without-project conditions, with- and without-diversion flow, and 2- and 25-year rainfall event.
- Simulate a 16-day period for all runs.
- Process model output to develop time-series charts and spatial contours of predicted water surface elevations.

The model runs are listed in Table G1. Note that all Future-With-Project (FWP) scenarios included the proposed West Shore Lake Pontchartrain (WSLP) levee.

Table G1. Delft3D Model Runs to Evaluate Drainage Under Rainfall Events.

Run ID	Diversion Flow (cfs)	Diversion Channel	Lateral Release Valves	Rainfall Frequency	Tidal Boundary at Lake Maurepas	Comments
20a	50	Existing Hope Canal	N/A	2-year	Rain-elevated then normal	FWOP No Div. 2-yr
20b	50	Existing	N/A	25-year	Rain-elevated then normal	FWOP No Div. 25-yr
21a	0	95% Design	0	2-year	Rain-elevated then normal	FWP No flow 2-yr
21b	0	95% Design	0	25-year	Rain-elevated then normal	FWP No flow 25-yr
22a	2000	95% Design	0	2-year	Rain-elevated then normal	FWP 2,000 cfs 2-yr
22b	2000	95% Design	0	25-year	Rain-elevated then normal	FWP 2,000 cfs 25-yr
23	2000	95% Design	140+140 cfs (for first 7 days)	None	Normal	Lateral release valves

4.0 MODEL RESULTS – EFFECT OF RAINFALL ON POLDER WATER LEVELS

The Delft3D model output contains water surface elevations and velocities at every model node for the 16-day simulation period. The output was processed to develop spatial contours of water surface elevations at the end of 5- and 14 days. The time series charts of water surface elevations were also prepared for the locations on the west and east side of the diversion canal. The results are shown in various combination of runs for convenient comparison in Figures G2 through G11. The predicted water surface elevations are summarized in Table G2 for a comparative review.

Table G2. Summary of Predicted Water Surface Elevations (ft, NAVD88).

Run ID	Conditions	West of LA-641				East of LA-641/West of Div. Canal				East of Div. Canal			
		Peak	Day 5	Day 10	Day 15	Peak	Day 5	Day 10	Day 15	Peak	Day 5	Day 10	Day 15
20a	Existing, 2-yr rainfall	2.4	1.5	1.5	1.5	2.4	1.4	1.3	1.3	2.4	1.4	1.3	1.3
20b	Existing, 25-yr rainfall	2.7	1.5	1.5	1.5	2.7	1.4	1.3	1.3	2.7	1.4	1.3	1.3
21a	With Div, 0 cfs, 2-yr rainfall	2.4	1.7	1.6	1.5	2.4	1.9	1.7	1.4	2.4	1.4	1.3	1.3
21b	With Div, 0 cfs, 25-yr rainfall	2.7	1.7	1.6	1.6	2.8	2.1	1.8	1.5	2.7	1.4	1.3	1.3
22a	With Div, 2000 cfs, 2-yr rainfall	2.4	1.9	1.8	1.8	2.4	2.2	2.1	2.0	2.4	1.9	1.7	1.6
22b	With Div, 2000 cfs, 25-yr rainfall	2.8	2.0	1.9	1.8	2.9	2.4	2.2	2.1	2.8	2.0	1.7	1.6
23	With Div, 2000 cfs, Lateral release	N/A	N/A	N/A	N/A	2.2	2.0	2.1	2.2	1.8	1.6	1.7	1.7

The model results show that the construction of the diversion canal isolates region to its west reducing drainage potential of the region. The impact is greater on the area east of LA-641 than the west area. The presence of elevated water levels north of I-10, reduces capacity of the highway culverts to drain the polders. Under the existing conditions, the difference in water levels due to the 2- and the 25-yr rainfall is apparent for about 4 days. Under the with-project conditions, the difference in water levels due to the 2- and the 25-yr rainfall is apparent for over 15 days.

To improve drainage of these polders, especially the west polder, the PMT evaluated effect of installing additional Lateral Release Valves (LRVs) along the banks of the proposed diversion canal which is described in the following section.

5.0 MODEL RESULTS – EFFECT OF LRVS ON POLDER DRAINAGE

To facilitate polder drainage, the currently proposed LRVs were made bi-directional so that they can flow either from the swamp to the diversion channel or from the channel to the swamp depending on the head difference. Simulations of 2-week were performed where rainfall event occurred on the first day like in the previous simulations. The following 3 LRV configurations were simulated. For all 3 configurations the LRV pipe invert was set at 0.0 ft, NAVD88.

- Configuration 1 (32 LRVs): Much larger capacity; 16-24” steel pipes on each side of the canal.
- Configuration 2 (8 LRVs): As in the 95% design report; 4-24” steel pipes of unspecified invert elevation on each side of the diversion canal.
- Configuration 3 (20 LRVs): 16-24” steel pipes on the west and 4 on the east side of the canal.

Three additional runs (24, 25 and 26) were simulated. They are listed in Table G3 which also contains previously completed runs in Table G2. Note that all FWP scenarios included the proposed West Shore Lake Pontchartrain (WSLP) levee.

Table G3. Delft3D Model Runs to Evaluate Later Release Valves for Drainage.

Run ID	Diversion Flow (cfs)	Diversion Channel	Lateral Release Valves	Rainfall Frequency	Tidal Boundary at Lake Maurepas	Comments
20a	50	Existing Hope Canal	N/A	2-year	Rain-elevated then normal	FWOP No Div. 2-yr
20b	50	Existing	N/A	25-year	Rain-elevated then normal	FWOP No Div. 25-yr
21a	0	95% Design	0	2-year	Rain-elevated then normal	FWP No flow 2-yr
21b	0	95% Design	0	25-year	Rain-elevated then normal	FWP No flow 25-yr
22a	2000	95% Design	0	2-year	Rain-elevated then normal	FWP 2,000 cfs 2-yr
22b	2000	95% Design	0	25-year	Rain-elevated then normal	FWP 2,000 cfs 25-yr
23	2000	95% Design	140+140 cfs (for first 7 days)	None	Normal	Lateral release valves
24	0	95% Design	Config 1: 32 LRVs 16 on each side	2-yr	Rain-elevated then normal	Large capacity; 16-24” pipes on each side. Invert 0.0 ft
25	0	95% Design	Config 2: 8 LRVs 4 on each side	2-yr	Rain-elevated then normal	As in the 95% design; 4-24” pipes on each side. Invert 0.0 ft
26	0	95% Design	Config 3: 20 LRVs West 16 & East 4	2-yr	Rain-elevated then normal	4-24” pipes on the east and 16 on the west. Invert 0.0 ft

Similar to the previous runs, the output was processed to develop spatial contours of water surface elevations at the end of 5- and 14 days. The time series charts of water surface elevations were also prepared for the locations on the west and east side of the diversion canal. The results are shown in various combination of runs for convenient comparison in Figures G12 through G17.

The insights from the simulations are listed on each figure. The combined flow through 32 LRVs is about 4 times that through the 8 LRVs at the peak. Note that the culverts are flowing partially under the water levels predicted for the corresponding scenarios. Generally, a lot of flow from the rainfall drainage comes into Hope Canal via LRVs on the west bank. Most of it exits north through Hope Canal and only some exits through the LRVs on the east bank. The east bank culverts are of no significant benefit to drain water out to east. The model scenarios with 32 LRVs (16 west + 16 east) and 20 LRVs (16 west + 4 east) have similar drainage benefit to the west polder. In general, introduction of LRVs improves drainage and reduces inundation of the polders.

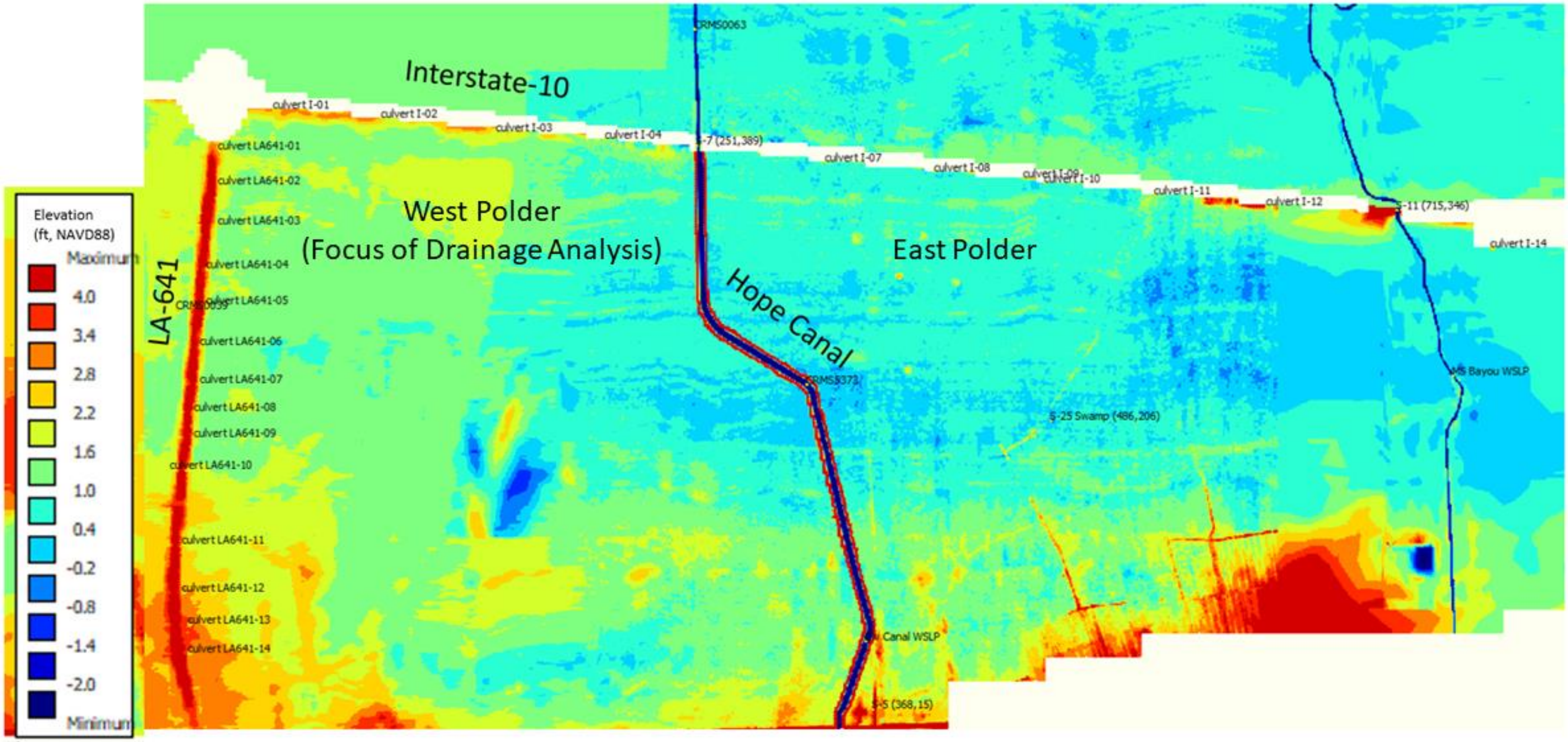
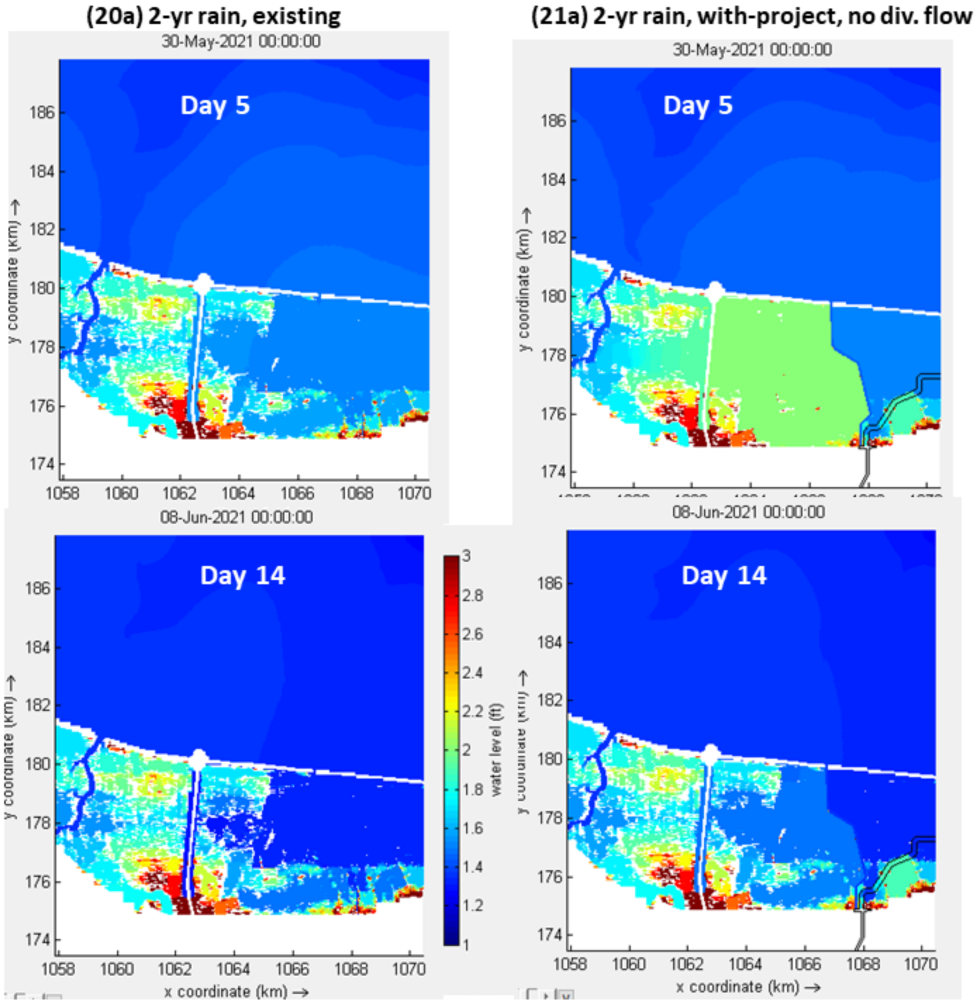
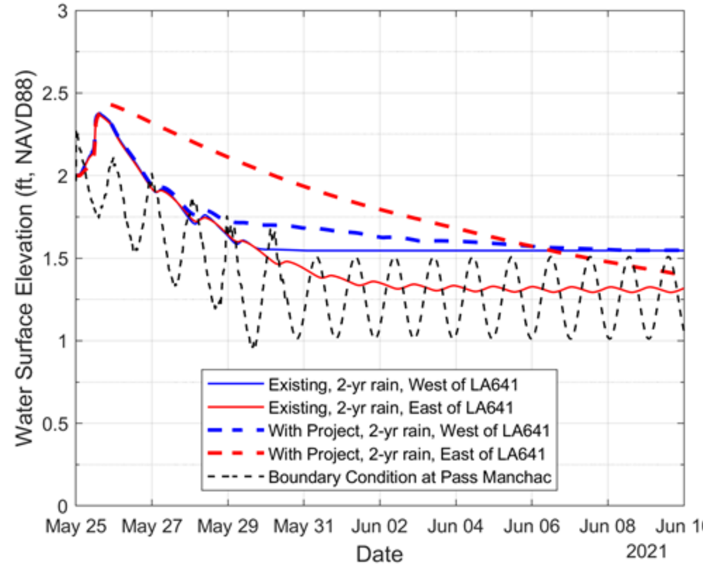


Figure G1. West and East Polders Created by the Proposed Hope Canal Alignment. Locations of highway culverts are shown with labels.





- Results are qualitatively similar to the 25-yr rainfall case

Figure G2. Comparison of existing and with-project, no diversion flow conditions (2-year rainfall).

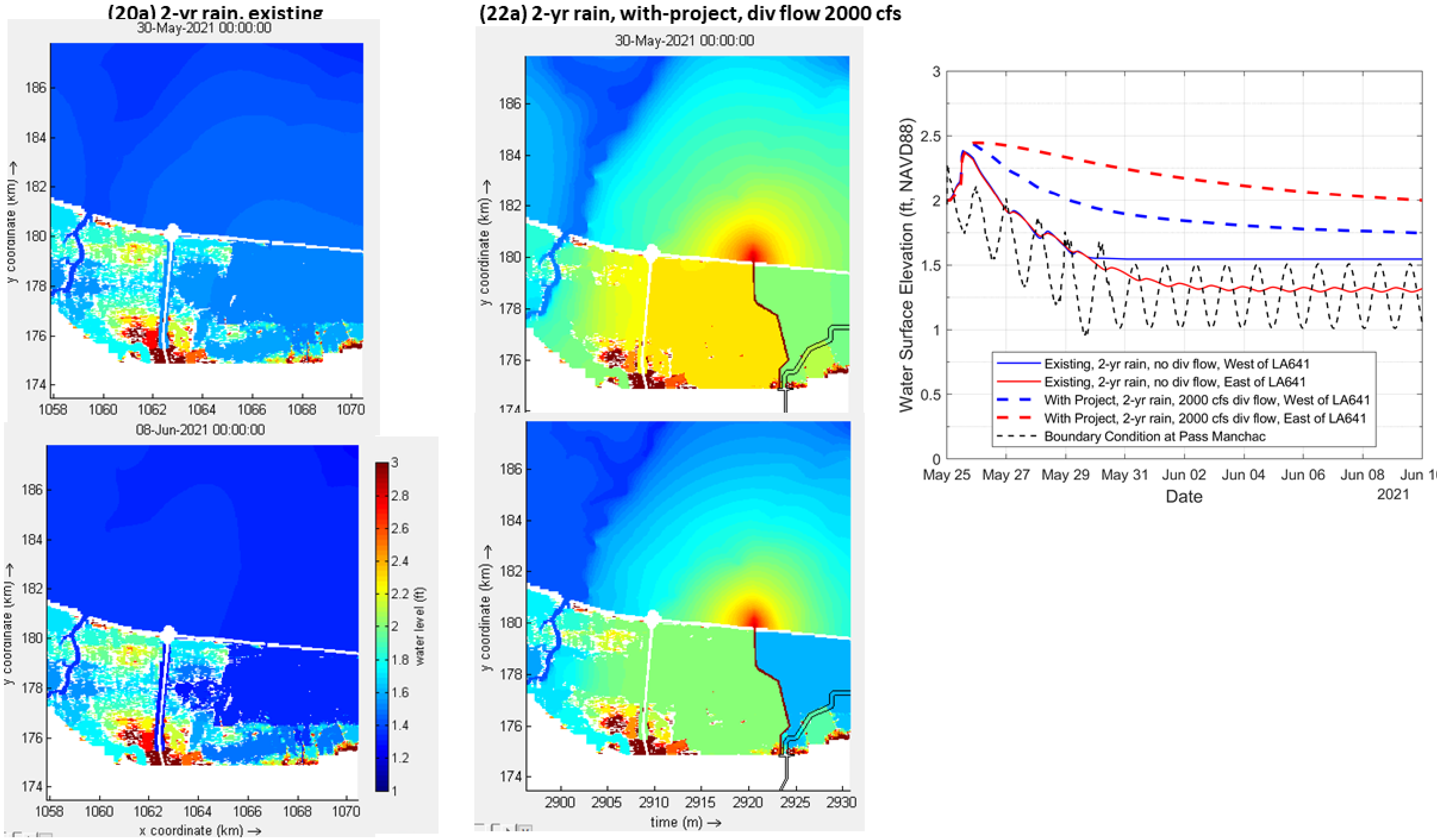
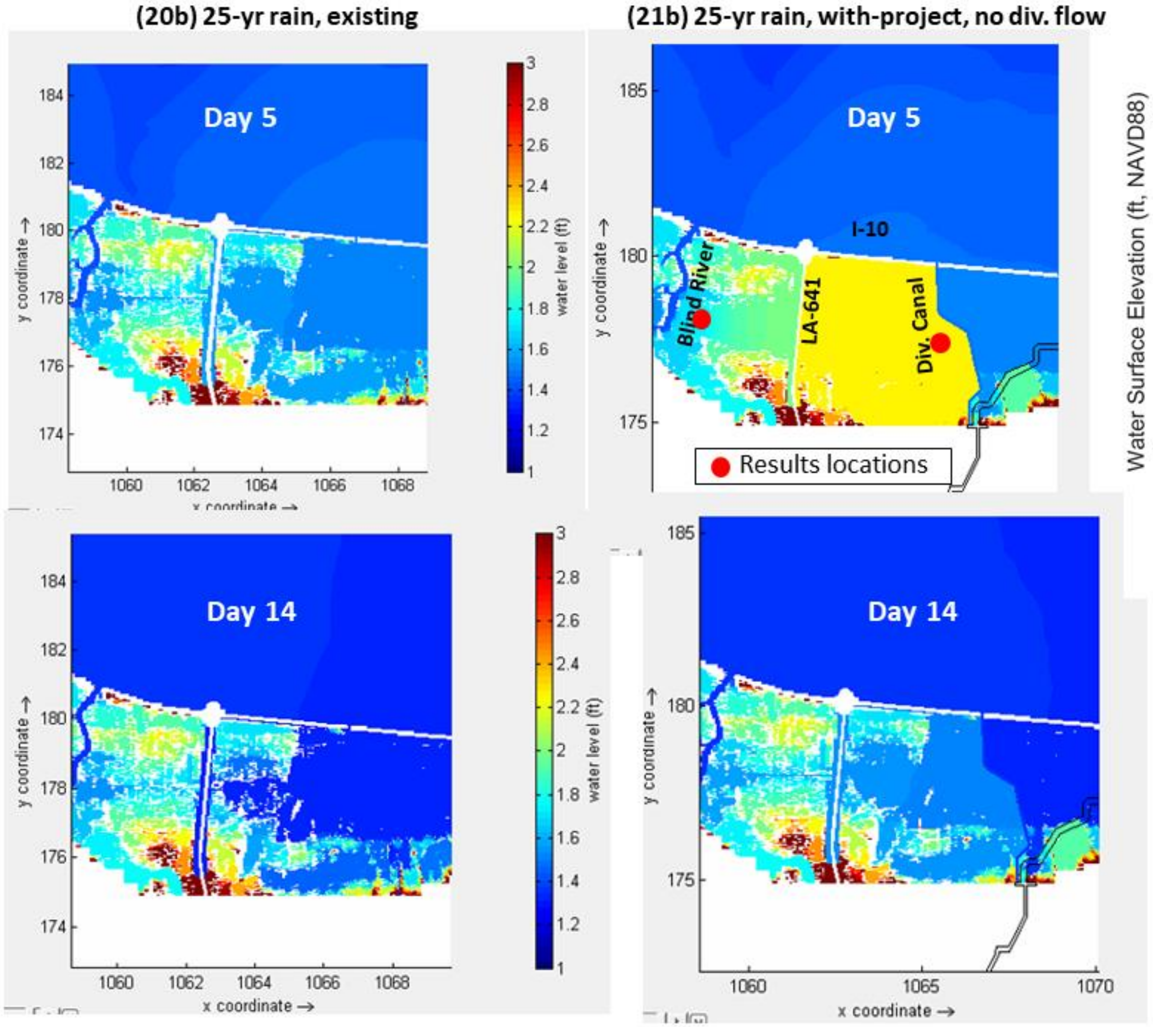
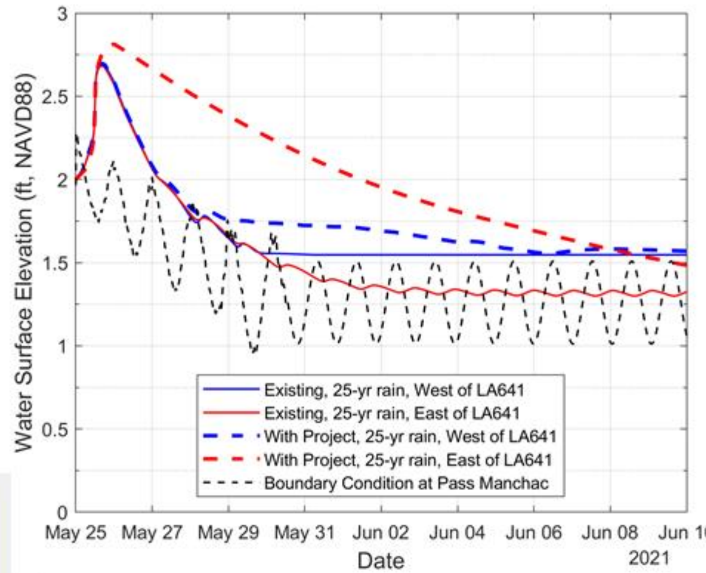


Figure G3. Comparison of existing and with-project, 2,000 cfs diversion flow conditions (2-year rainfall).





- Construction of the diversion canal isolates region to its west reducing drainage potential of the region.
- The impact is greater on the area east of LA-641 than the west area
- The location west of LA641 is showing flat water level possibly because of local cut off. In reality, this region will continue draining.

Figure G4. Comparison of existing and with-project, no diversion flow conditions (25-year rainfall).

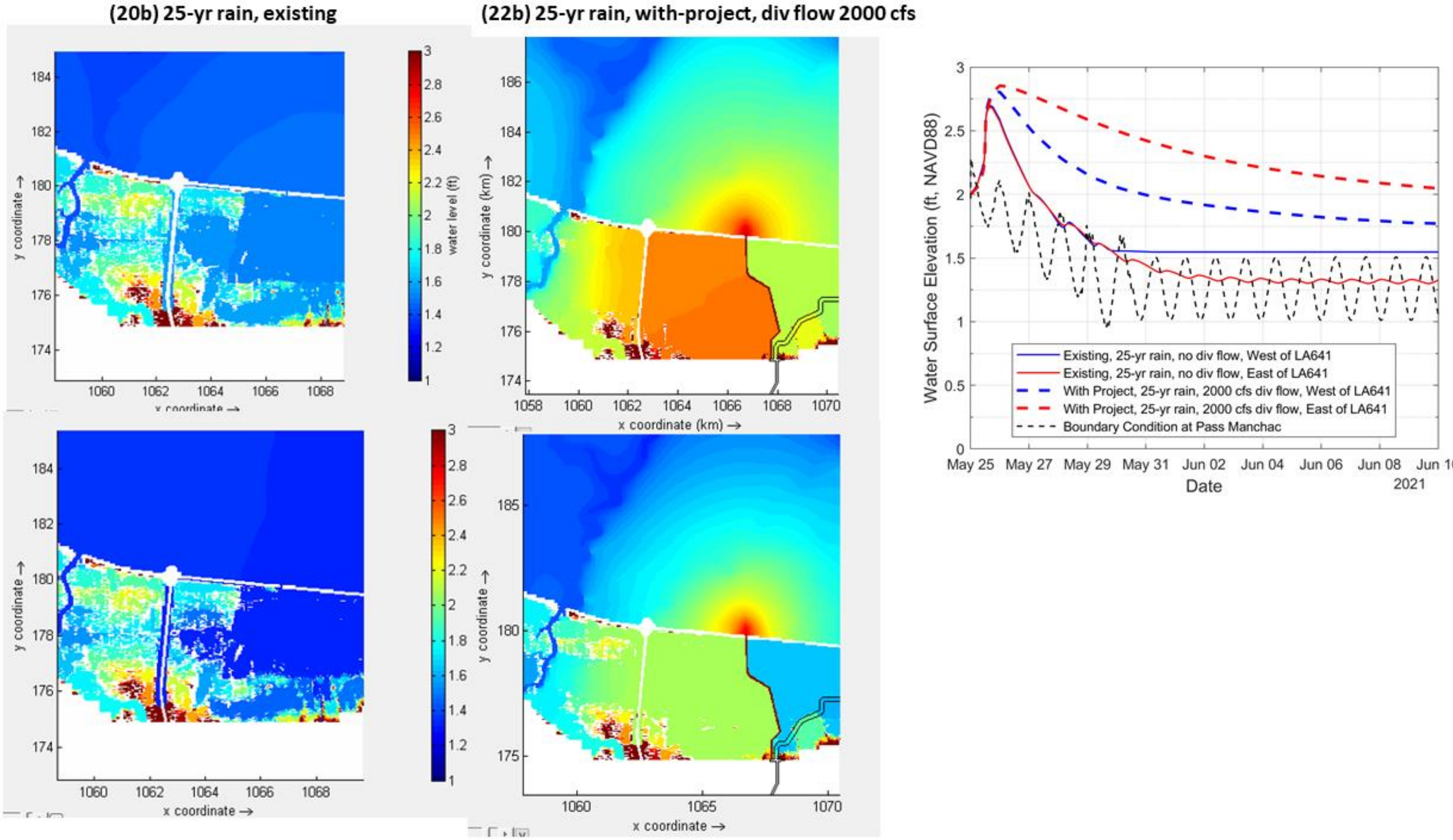
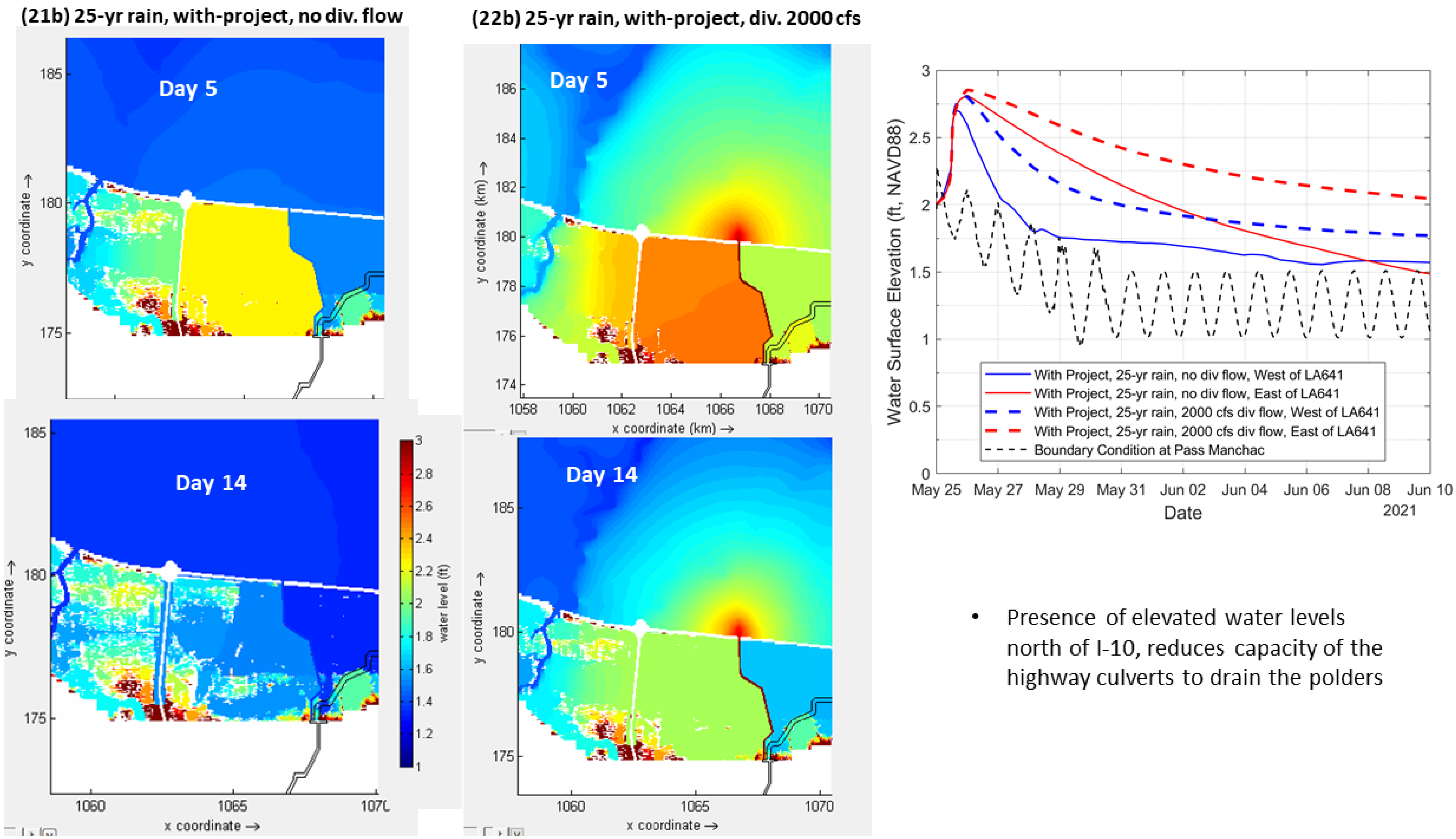


Figure G5. Comparison of existing and with-project, 2,000 cfs diversion flow conditions (25-year rainfall).



- Presence of elevated water levels north of I-10, reduces capacity of the highway culverts to drain the polders

Figure G6. Comparison of with-project, no diversion flow and with-project, 2,000 cfs diversion flow conditions (25-year rainfall).

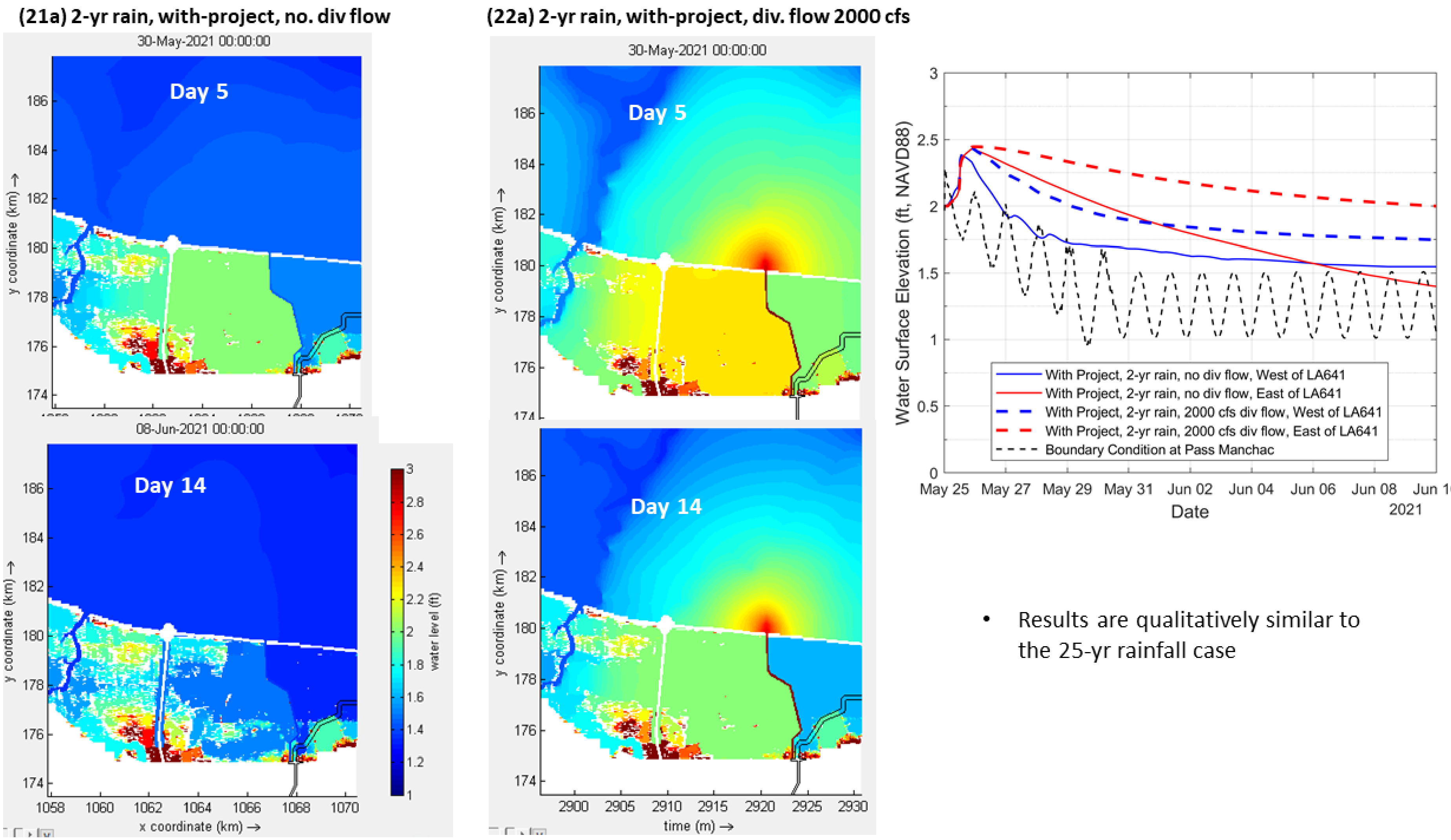
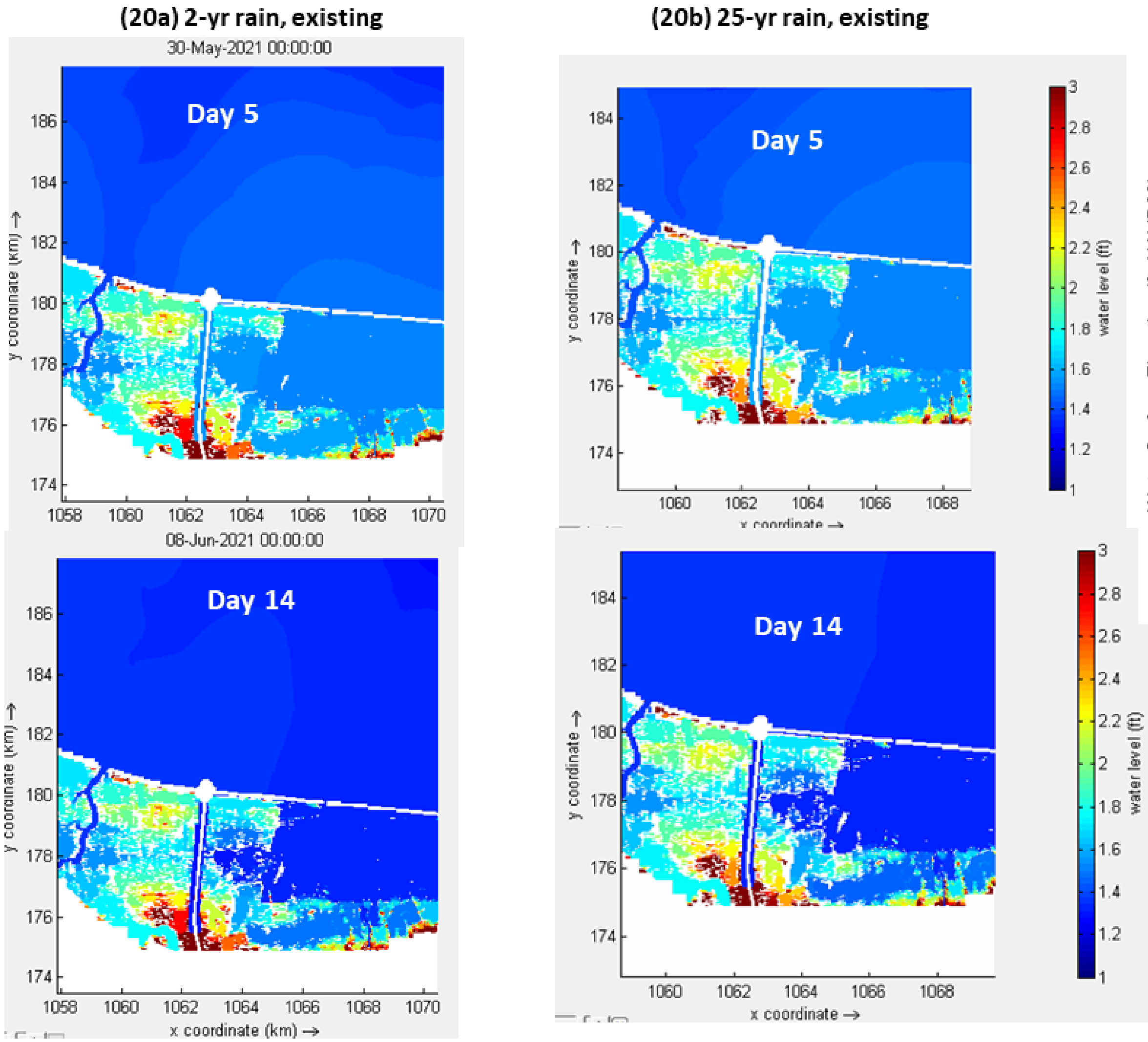
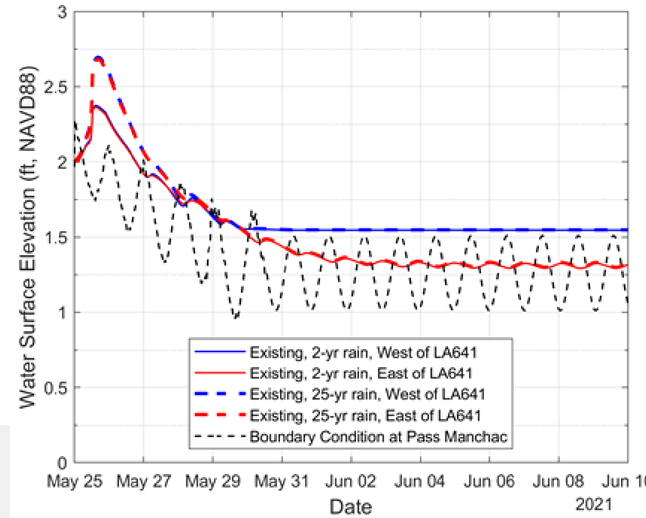


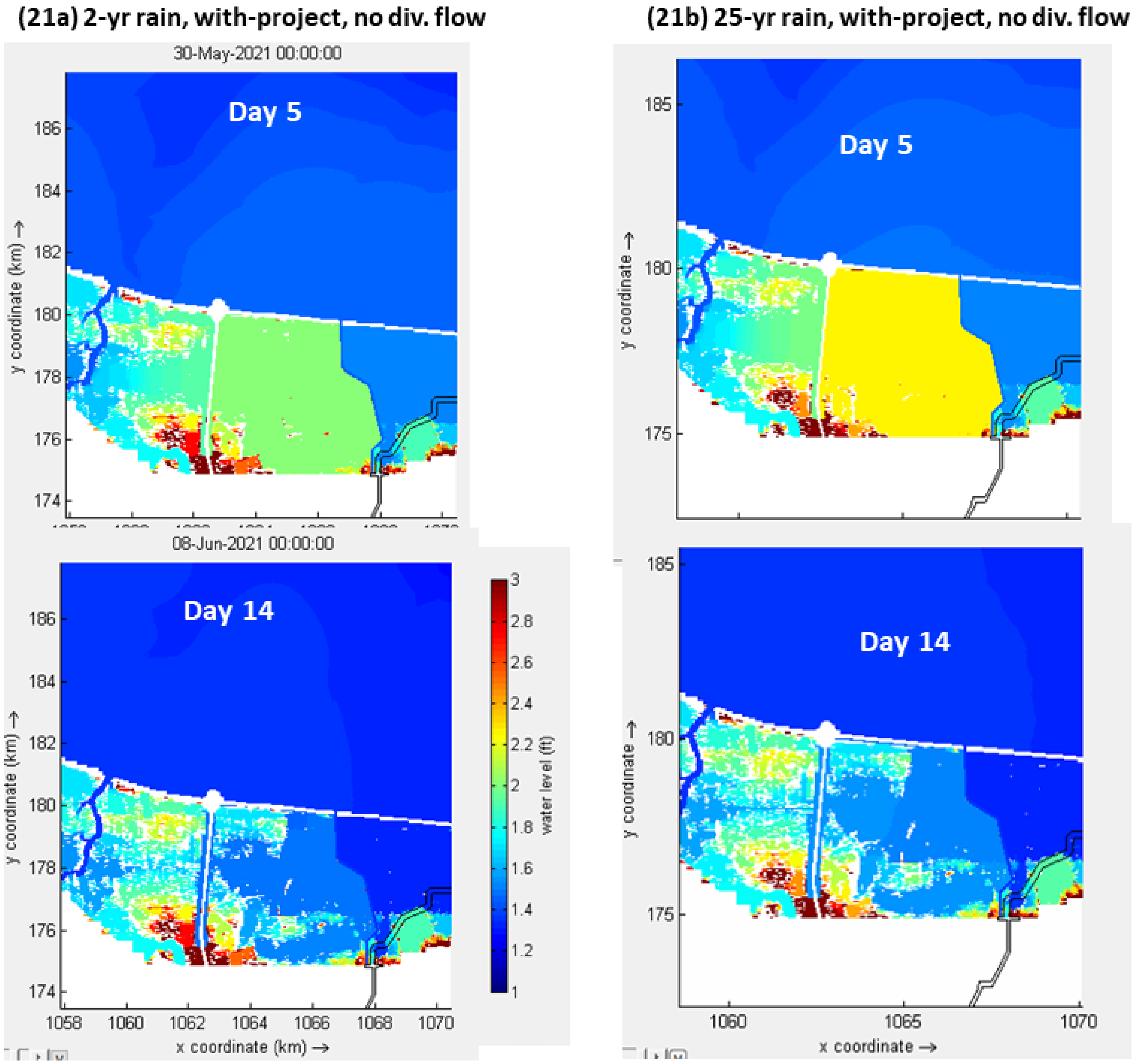
Figure G7. Comparison of with-project, no diversion flow and with-project, 2,000 cfs diversion flow conditions (2-year rainfall).

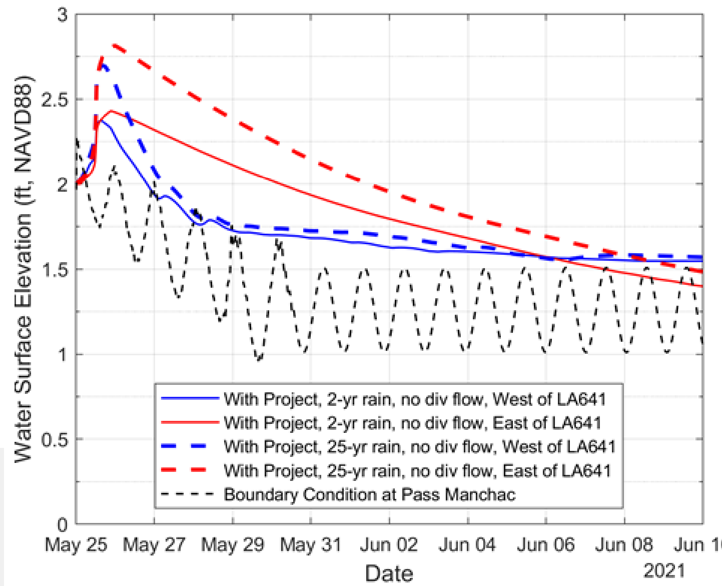




- Under the existing conditions, the difference in water levels due to the 2- and the 25-yr rainfall is apparent for about 4 days

Figure G8. Comparison of existing conditions under 2-year and 25-year rainfall.





- Under the with-project conditions, the difference in water levels due to the 2- and the 25-yr rainfall is apparent for over 3 weeks

Figure G9. Comparison of with-project, no diversion flow conditions under 2-year and 25-year rainfall.

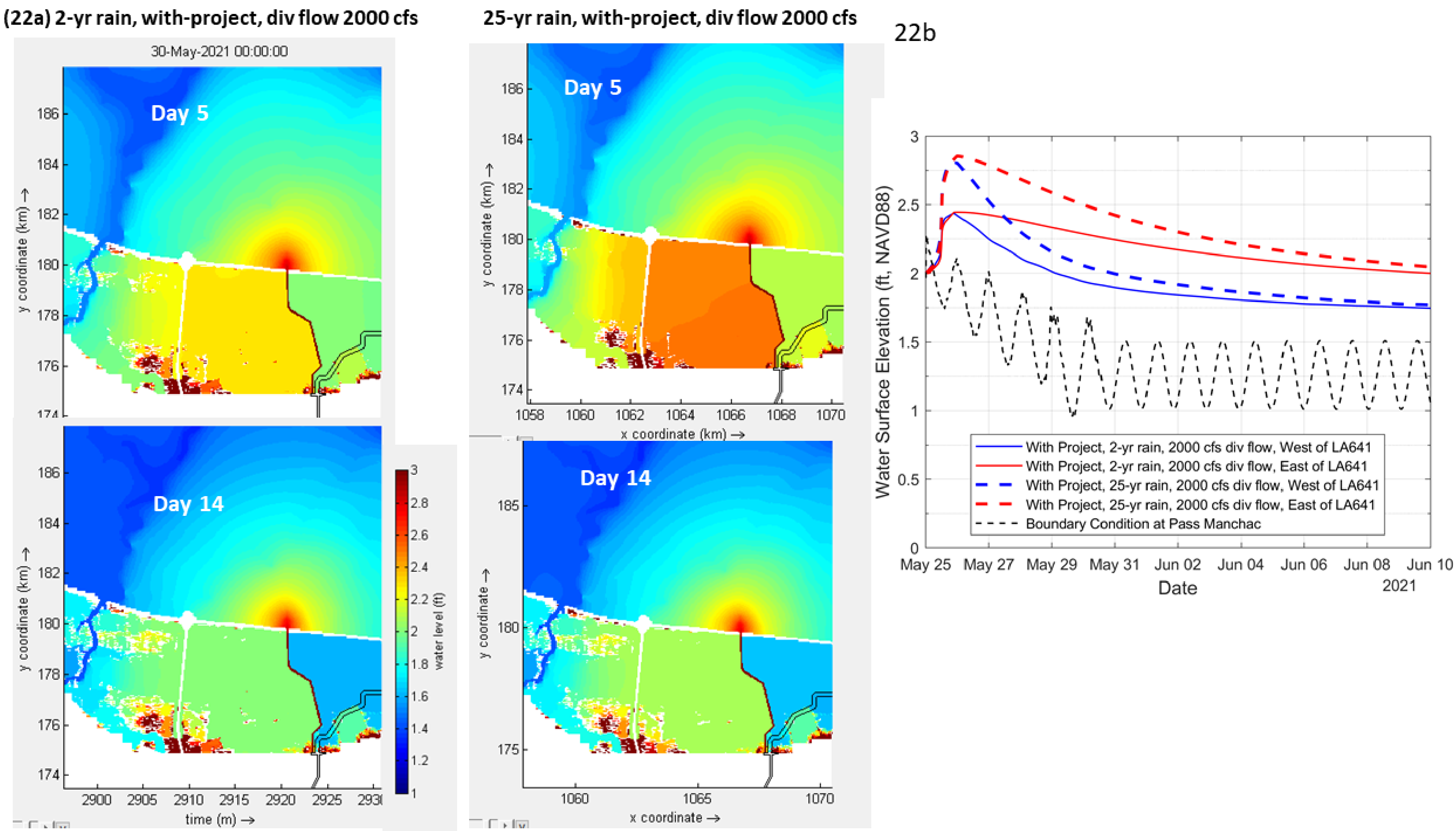


Figure G10. Comparison of with-project, 2,000 cfs diversion flow conditions under 2-year and 25-year rainfall.

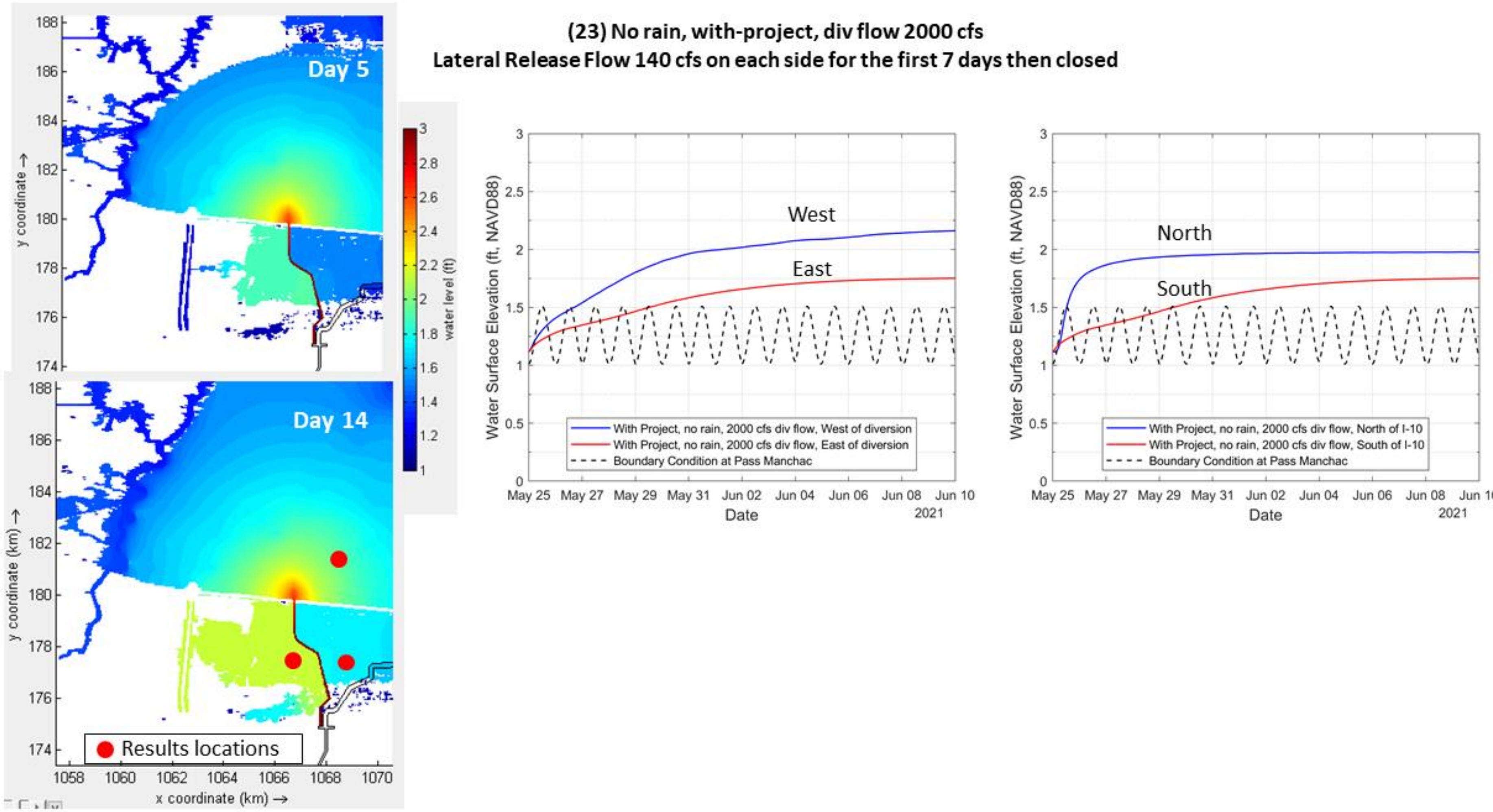
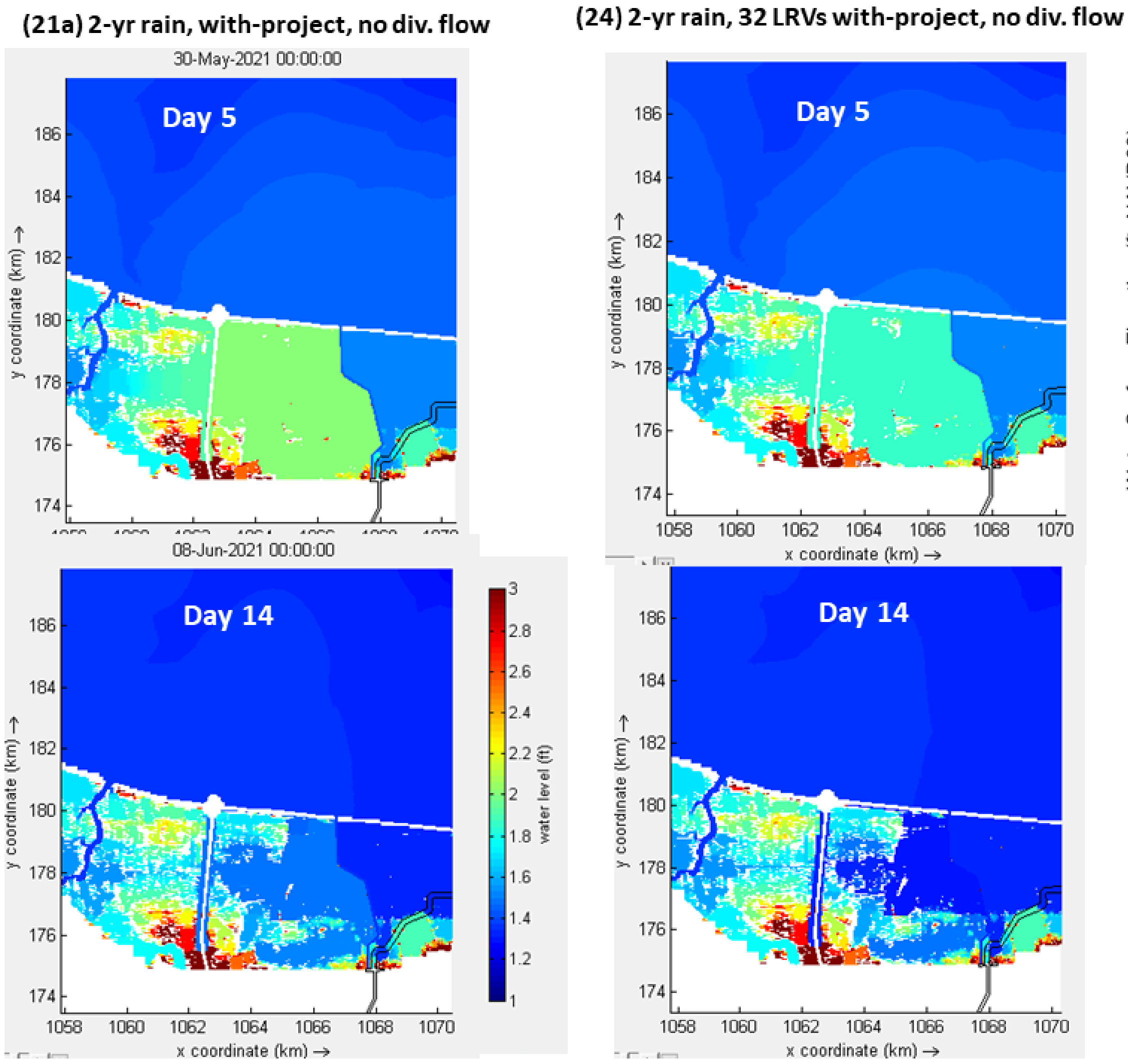
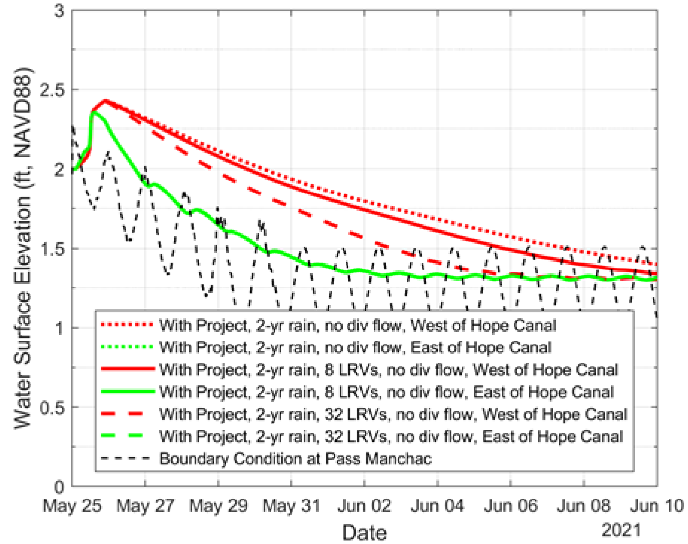


Figure G11. Polder water levels due to Lateral Release Flow 140 cfs on each side for the first 7 days (No rain, with-project, diversion flow of 2,000 cfs).





WSE ft, NAVD88	Days to reach a specific WSE		
	Run 21a No LRVs	Run 25 8 LRVs	Run 24 32 LRVs
2.25	2.6	2.4	2.0
2	5.1	4.7	3.8
1.75	8.6	7.7	6.0
1.5	13.3	11.6	8.6

Figure G12. Effect of 32 lateral release valves on the polder water levels (2-year rainfall, with-project, no diversion flow).

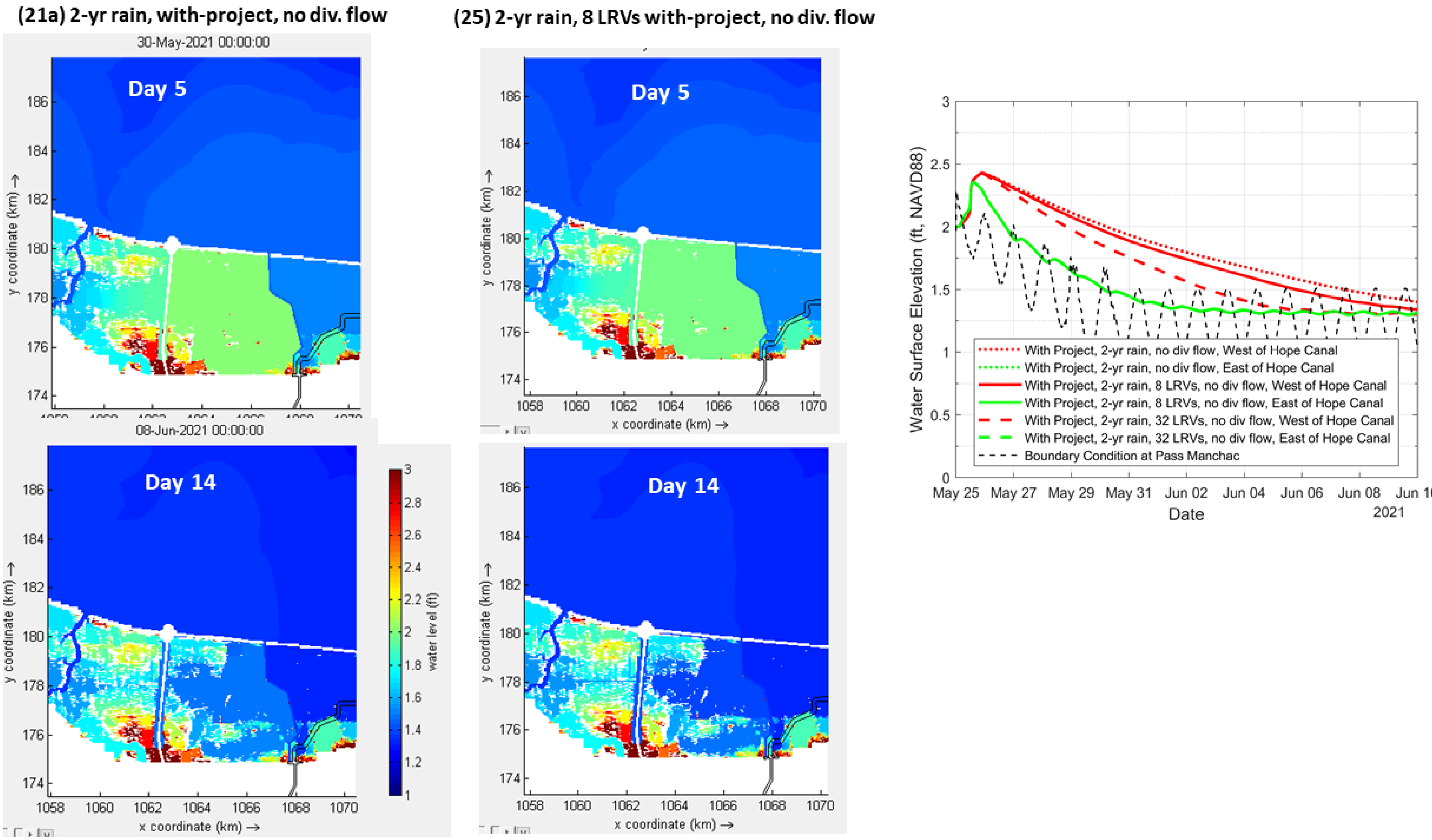
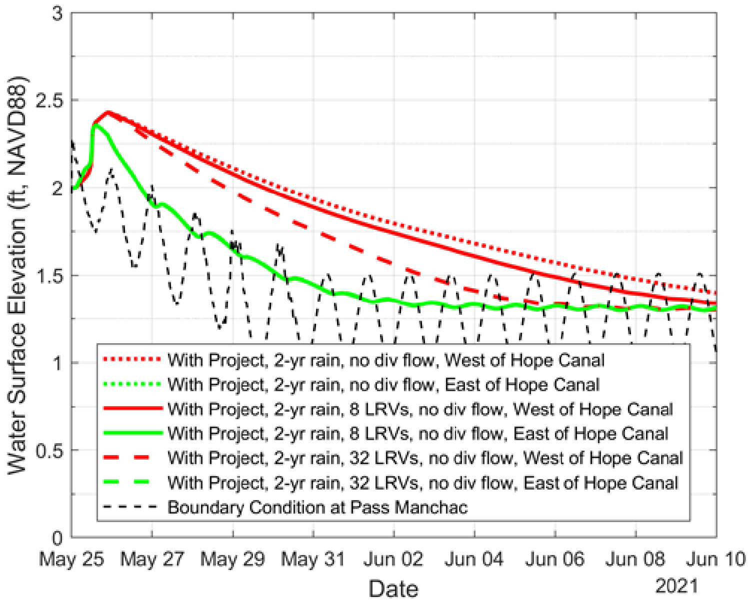
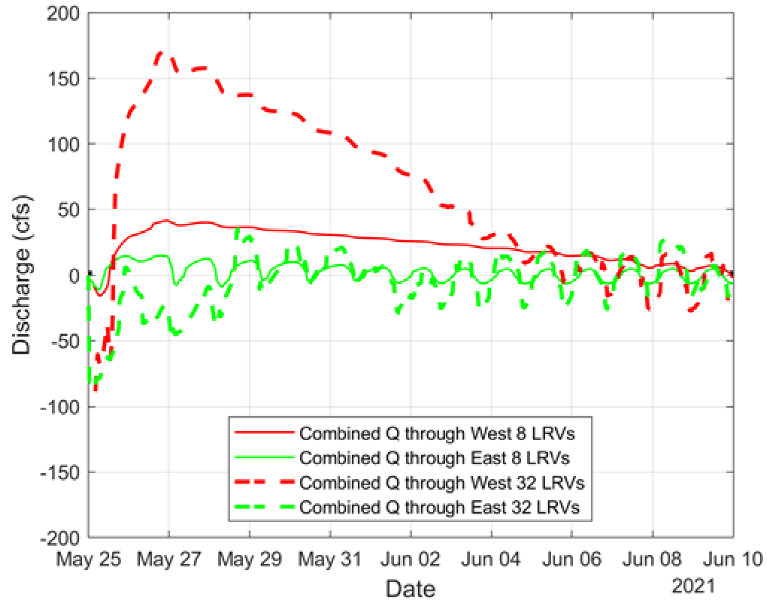


Figure G13. Effect of 8 lateral release valves on the polder water levels (25-year rainfall, with-project, no diversion flow).

Flow through 8 (Run 25) and 32 (Run 24) LRVs



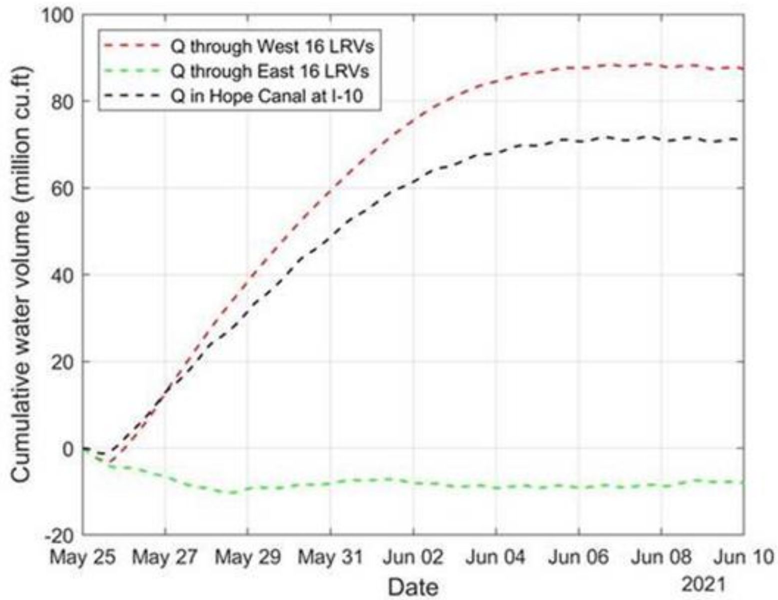


- Red lines are the water levels in the west polder
- Green line is close to the water levels in Hope Canal
- Difference between the two, shows head across the LRVs

- The combined flow through 32 LRVs is about 4 times that through the 8 LRVs at the peak.
- Note the culverts are flowing partially under the prevailing water levels
- The east bank culverts are of no significant benefit to drain water out to east

Figure G14. Comparison of flow through 8 (Run 25) and 32 (Run 24) lateral release valves.

Run 24: 32 LRVs- Cumulative volume of water draining through LRVs and Hope Canal



Generally a lot of flow from the rainfall drainage comes into Hope Canal via LRVs on the west bank. Most of it exits north through HC and only some exits through the LRVs on the east bank.

Note that the final volumes add up approximately to provide a mass balance. They will not add up exactly because the Hope Canal tracking is slight north of the exact outfall location and we loose some accounting due to this.

Figure G15. Estimate of cumulative volume of water draining through 32 lateral release valves and Hope Canal.

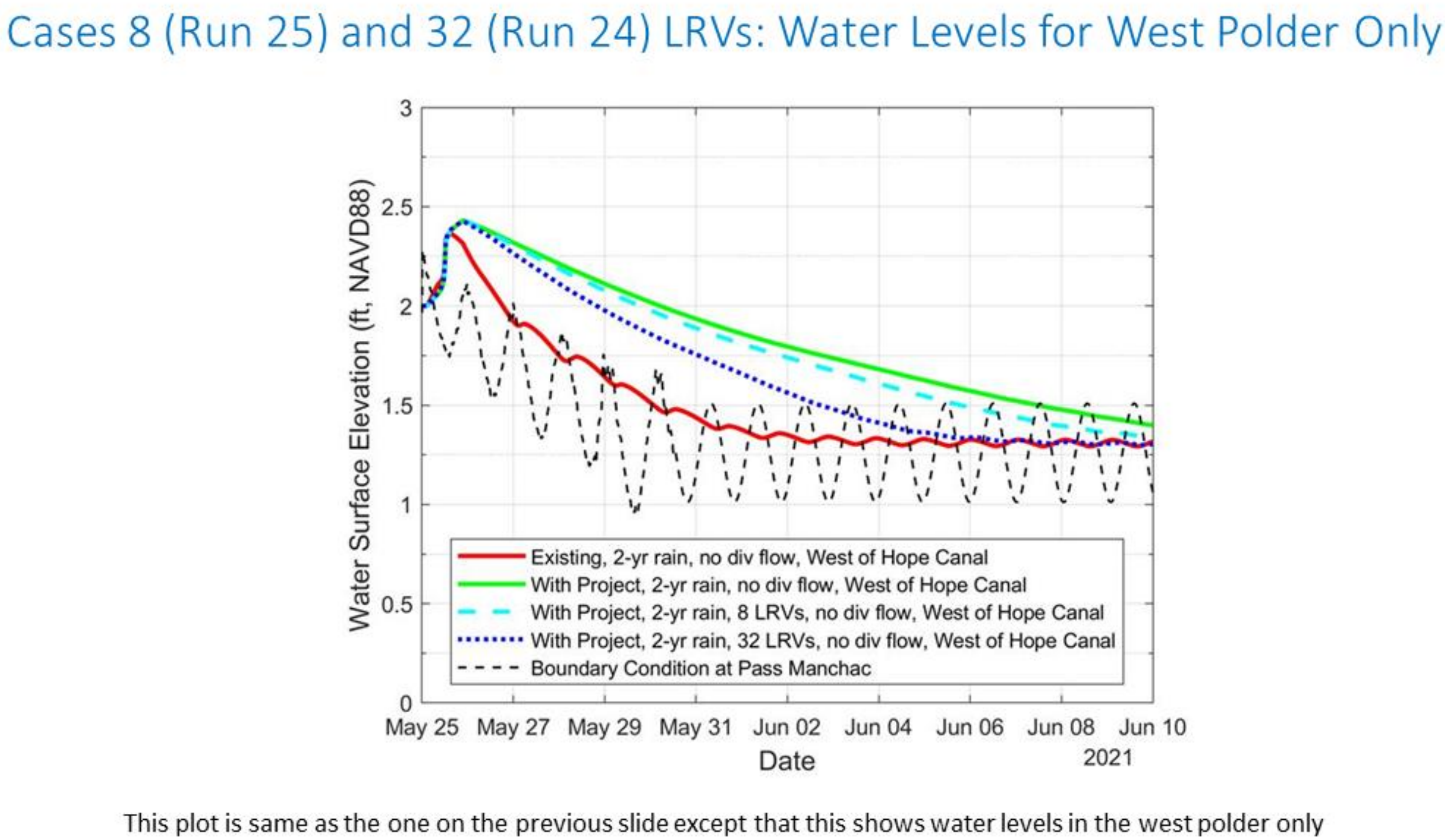
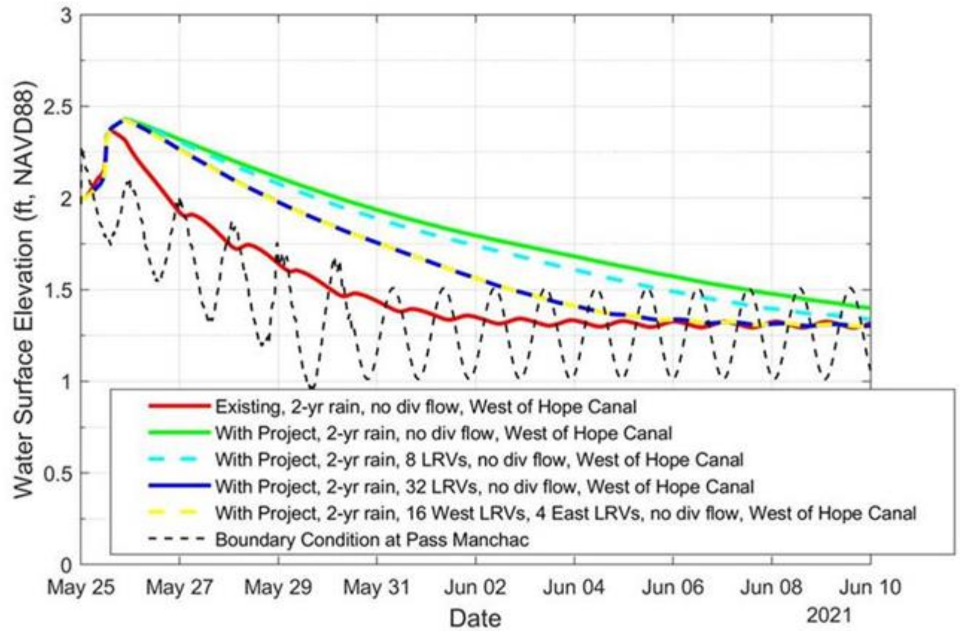


Figure G16. Comparison of water levels in west polder for the 8 and 32 later release valves scenarios (2-year rainfall).

Run 26: 16 LRVs on the West and 4 LRVs on the East Bank – West Polder Only



The plot is same as the previous slide except that this has a yellow line showing results of 16LRVs on the west bank and 4 LRVs on the east bank run.
This run demonstrates that the east bank may not need 16 LRVs for drainage as they do not drain much water out of Hope Canal as indicated by the overlapping blue and yellow lines.

Figure G17. Water levels in the west polder with 16 lateral release valves on the west bank and 4 on the east bank.