# APPENDIX M: ENGINEERING REPORTS

## Volume 1 of 4

**Contents:**



*It should be noted that the Engineering Reports were provided by CPRA as standalone documents and in some cases the terminology within may not match the terminology used in the SEIS (e.g. MSP vs. MSA-2 for the selected alternative).*

# Basis of Design Report - 15% Design (DDR)

# STATE OF LOUISIANA
# COASTAL PROTECTION AND RESTORATION AUTHORITY

## RIVER REINTRODUCTION INTO MAUREPAS SWAMP AND WEST SHORE LAKE PONTCHARTRAIN FLOOD RISK REDUCTION PROJECT PO-0029
## LaGOV NO. 4400019214

# BASIS OF DESIGN REPORT
## 15% DESIGN

**For**

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## December 31, 2020

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# Abbreviations

| | |
|---|---|
| AASHTO | American Association of State Highway and Transportation Officials |
| ACI | American Concrete Institute |
| AHP | Above Head of Passes |
| AISC | American Institute of Steel Construction |
| AREMA | American Railway Engineering and Maintenance-of-Way Association |
| ASCE | American Society of Civil Engineers |
| ASTM | American Society for Testing and Materials |
| BOD | Basis of Design |
| BODR | Basis of Design Report |
| cfs | cubic feet per second |
| CIP | Cast-in-Place |
| CN | Canadian National |
| CORS | Continuously Operating Reference Station |
| CPRA | Coastal Protection and Restoration Authority |
| CPT | cone penetrometer tests |
| CUP | Coastal Use Permit |
| DCD | Design Criteria Document |
| dia | diameter |
| DOTD | Louisiana Department of Transportation and Development |
| DQCP | Design Quality Control Plan |
| DT | Design Team |
| EA | Environmental Assessment |
| EB | East Bank |
| ECB | Environmental Clearance Boundary |
| E&D | Engineering and Design |

| | |
|---|---|
| E-G | Evans-Graves Engineers, Inc. |
| EIS | Environmental Impact Statement |
| EL | Elevation |
| EM | Engineering Manual |
| ft | feet |
| ft/s | feet per second |
| Gr | Grade |
| HEC-RAS | Hydraulic Engineering Center River Analysis System |
| H&H | Hydraulics and Hydrology |
| HSDRRS | Hurricane and Storm Damage Risk Reduction System |
| HSS | Hydraulic Structural Steel |
| Hwy | Highway |
| I-10 | Interstate 10 |
| in | inches |
| KCS | Kansas City Southern |
| k | kips |
| ksi | kips per square inch |
| lb | Pound |
| LDWF | Louisiana Department of Wildlife and Fisheries |
| LiDAR | Light Detection and Ranging |
| LRFD | Load and Resistance Factor Design |
| Maint. | Maintenance |
| MDE | Maximum Design Earthquake |
| mm | millimeter |
| MOA | Memorandum of Agreement |
| MOU | Memorandum of Understanding |
| mph | miles per hour |
| MR | Mississippi River |
| MRL | Mississippi River Levee |
| MSE | Mechanically Stabilized Earth |
| NAVD | North American Vertical Datum |
| NDA | Non-Disclosure Agreement |
| NGS | National Geodetic Survey |
| OBE | Operating Basis Earthquake |
| OLS | Oil Land Services |
| OMRR&R | Operations, Maintenance, Repairs, Replacement and Rehabilitation |
| PI | Point of Intersection |

| | |
|---|---|
| PLD | Pontchartrain Levee District |
| PPC | Prestressed Precast Concrete |
| psf | pounds per square foot |
| psi | pounds per square inch |
| PVC | Polyvinyl chloride |
| RCB | Reinforced Concrete Box |
| ROE | Right of Entry |
| ROM | Rought Order of Magnitude |
| ROW | Right of Way |
| SCADA | Supervisory Control and Data Acquisition |
| SWL | Safe Water Level |
| TBD | To Be Determined |
| T.O. | Top Of |
| TRS | Temporary Retaining Structures |
| TWIG | The Water Institute of the Gulf |
| USACE | United States Army Corps of Engineers |
| WB | West Bank |
| WSE | Water Surface Elevation |
| WSLP | West Shore Lake Pontchartrian |
| WVA | Wetlands Value Assessment |

# 1. Executive Summary

## 1.1 Project Description

The subject project consists of the combination of two projects: 1) the *River Reintroduction into Maurepas Swamp* (Maurepas Diversion) sponsored by the Coastal Protection and Restoration Authority (CPRA), which is intended to restore the dying Maurepas Swamp, and 2) the *West Shore Lake Pontchartrain Flood Risk Reduction Project* (WSLP Project) sponsored by the U.S. Army Corps of Engineers (USACE), which is designed to provide flood protection St. Charles and St. John The Baptist Parishes. The Maurepas Diversion has been combined with the three Western-most reaches of the WSLP project (WSLP-111, WSLP-112, and WSLP-113) to create the subject project, which will be termed the Project.

## 1.2 Engineering and Design

The Engineering and Design (E&D) of the Project will be organized into several phases. The subject Phase 1 (CPRA contract No. 440019214, PO 2000510358, TO 1) authorized the work on Basis of Design (BOD) Development, Data Collection, and 15% Design Plans. The Design Criteria of the major project features were established during the BOD Phase. Subsequent phases will be comprised of detailed E&D and cost estimates of the selected components and appurtenance alternatives, development of the construction contract documents (plans and specifications); and development of an Operations, Maintenance, Repairs, Replacement and Rehabilitation (OMRR&R) Plan.

Conceptual engineering performed during the BOD phase was initiated using existing data, studies, and reports. Additional data collection was begun during the BOD Phase, including additional geotechnical borings and cone penetrometer tests (CPTs) and the accompanying laboratory analyses of the collected samples. Additional topographic surveying, including conventional surveying data collection, acquisition of Light Detection and Ranging (LiDAR) data from an unmanned aerial vehicle (UAV, i.e., a drone), and high-resolution low level photogrammetry were also collected. Additional hydrographic surveying was performed in the Mississippi River (MR) and into the batture area using bathymetric and magnetometer data collection methods.

## 1.3 Basis of Design Report

This Basis of Design Report (BODR) and its appendices summarize and document the major project design criteria; the conceptual-level engineering and design plans; and evaluations of the engineering alternatives considered for each project features. The document also presents the conclusions and recommendations arising from the work completed by the AECOM Design Team (DT) during the BOD Phase. The fundamental goal of the BOD Phase is to develop the basis of design upon which subsequent work will be performed; it does not include detailed engineering and design. These will be performed during succeeding Task Orders comprising the overall E&D effort of the Project.

During the BOD Phase, the major project design criteria are established, and the recommended alternatives are selected. The alternatives studied in the BOD phase are documented herein, along with the pertinent drawings, studies, and reports, which are included as Appendices. In performing the conceptual designs, the DT produced a Design Criteria Document (DCD), which serves to document the design criteria specific to each engineering discipline working on the Project. The DCD was developed with input from CPRA and the USACE, and it is intended to be a living document that will be periodically updated as the design process progresses.

The major components of the Maurepas Diversion are the headworks; conveyance channel; and the road and railway crossings. This BODR presents the decision process for selecting the headworks features, the conveyance channel alignment and geometry, and the approach to each of the road and railroad crossings. The main components of the WSLP Project are levee, floodwalls, floodgates, and the raising of Airline Highway to tie-in to the levee. The decision process for selecting the levee alignment, construction elevation, and footprint; the I-wall and T-wall locations and conceptual design; and the basic features of the floodgates at the various road and railroad crossing are presented in this BODR.

Major decisions and recommendations involving the ancillary features are also documented, including: the modifications to the existing drainage required due to the construction of the diversion and flood protection features; the alignment and basic geometry of the new Kansas City Southern (KCS) railroad bridge; the disposition of the utilities and other infrastructure crossing the diversion and flood protection features; the use of excavated earthen materials for potential levee construction and/or construction fill; and the features comprising the support facilities and associated site work. These aspects will be refined and further developed during the subsequent detailed engineering and design phases of the Project to complete the remainder of the E&D.

During the subject work, there were five major storms that hit the Louisiana coastline, including two hurricanes, which caused major delays in the Survey and Geotechnical data collection efforts. The field crews and equipment had to be demobilized and moved to safety, then re-mobilized to renew the work effort. The Survey crews were even called upon to perform damage assessments and to aid their neighbors in their recovery efforts. None-the-less, all of the Survey and Geotechnical data collection required under the Scope of Work has been conducted within the contract period.

# 2. Project Information

## 2.1 Project Name & Number

The subject project: *River Reintroduction into Maurepas Swamp and West Shore Lake Pontchartrain Flood Risk Reduction Project PO-0029* consists of the combination of two projects: The CPRA is the state sponsor of the *River Reintroduction into Maurepas Swamp* (Maurepas Diversion). The Maurepas Diversion is a freshwater diversion intended to supply freshwater, nutrients, and sediments to restore the health and essential functions of the swamp. The USACE is the federal sponsor of the *West Shore Lake Pontchartrain Flood Risk Reduction Project* (WSLP Project), which is designed to provide hurricane and storm-damage risk reduction in St. Charles and St. John The Baptist Parishes. The three Western-most reaches of the WSLP Project (WSLP-111, WSLP-112, and WSLP-113) are to be constructed parallel to and immediately adjacent to the Maurepas Diversion.

Due to the co-location of the Maurepas Diversion and the three reaches of the WSLP Project in the same alignment corridor, both CPRA and USACE have deemed it prudent to design the two projects together, enabling close coordination between the projects. When referred to individually, the projects will be called the Maurepas Diversion and the WSLP Project. When referred to collectively as a combined project, it will simply be called the Project throughout the remainder of this document.

The Maurepas Diversion design will be executed under State of Louisiana Contract No. 4400010386, “Full General Engineering Services for CPRA” State project number PO-0062.

## 2.2 Project Location

The Project is located in southeastern Louisiana in St. John the Baptist Parish. It begins on the east bank of the MR at approximately River Mile 144. The Maurepas Diversion conveyance channel extends from LA 44 (River Road) northward to approximately 1,000-ft north of Interstate 10 (I-10), discharging into the Maurepas Swamp. The three reaches of the WSLP Project start at the Mississippi River Levee (MRL) and extend approximately 2,500-ft north of US 61 (Airline Highway), where they meet with the USACE Hope Canal Pump Station Complex in reach WSLP-114. **Figure 2.1** shows the location and extent of the Project.

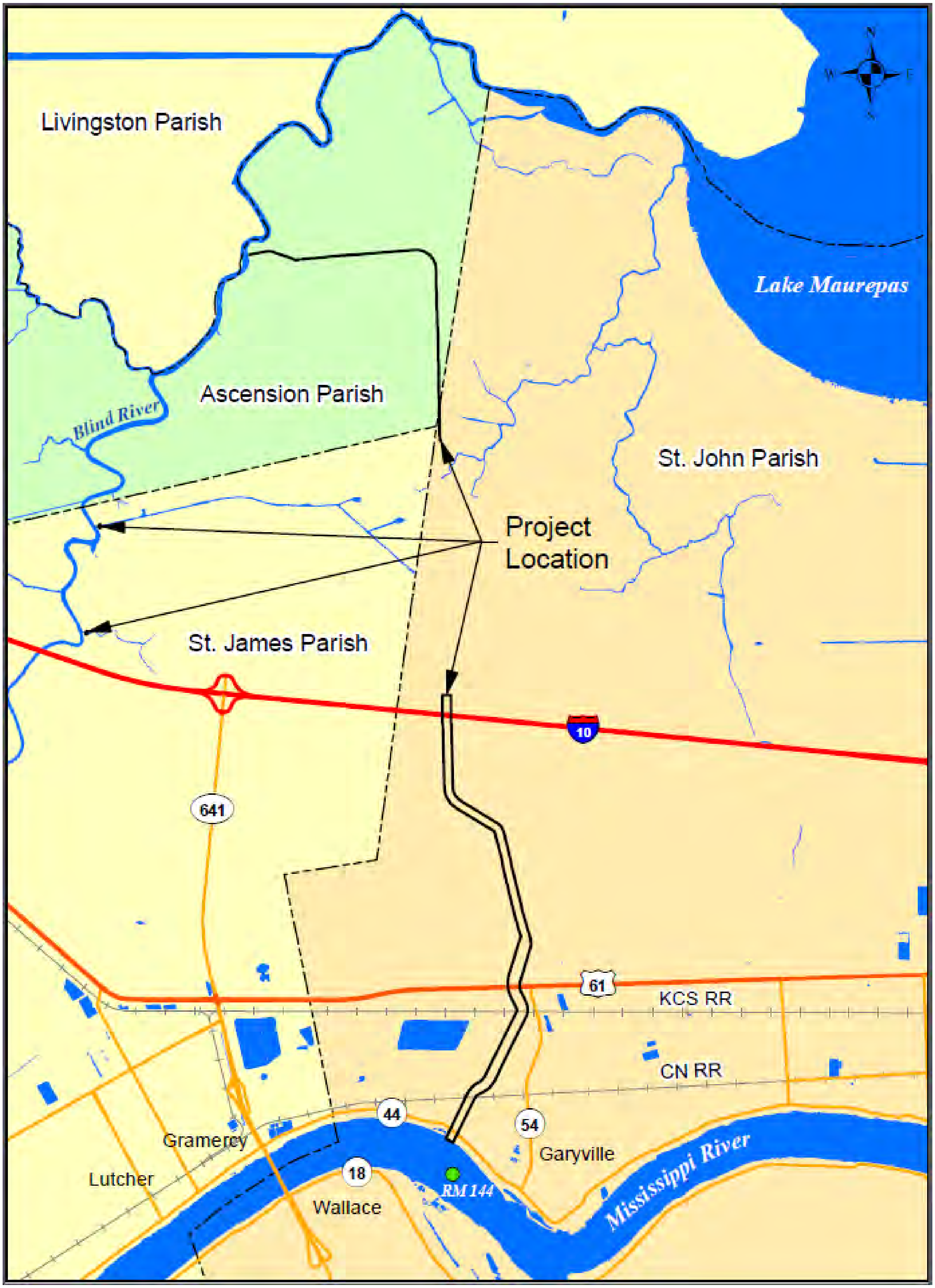


**Figure 2.1 – Project Location Map**

## 2.3 Project Summary

As noted, the subject project: *River Reintroduction into Maurepas Swamp and West Shore Lake Pontchartrain Flood Risk Reduction Project PO-0029* consists of the combination of two projects:1) the Maurepas Diversion, and 2) the WSLP Project reaches WSLP-111, WSLP-112 and WSLP-113. Each of these projects is described below.

### 2.3.1 Maurepas Diversion

The Maurepas Diversion intake is located on the West Bank of the Mississippi River in St. John the Baptist Parish, immediately west of Garyville, Louisiana, at River Mile 144 Above Head of Passes (AHP). It traverses between the Marathon Petroleum Terminal upriver and the Ernest Amann residential subdivision downriver and extends northward for 5½ miles, terminating approximately 1,000-ft north of I-10. The proposed 2,000 cubic feet per second (cfs) freshwater diversion will introduce flowing oxygenated water; ameliorate salinity intrusion; facilitate nutrient uptake and retention; increase forest health and structural integrity; and increase rates of soil surface elevation gain to offset subsidence. The swamp has been severely stressed by saltwater intrusion from Lake Maurepas as well as being cut-off from the historical flooding of the MR that supplied the nutrients and freshwater which built and sustained it.

The diversion project is comprised of the following elements: an intake channel from the MR; an automated gated structure in the MRL; a sedimentation basin; a $28{,}000^{+}$-ft long conveyance channel; submerged weirs in Bayou Secret and Bourgeois Canal; check valves near the I-10 crossing; box culverts under River Road, CN Railroad, and Airline Highway; a bridge over the channel at the KCS Railroad; and reshaping the geometry of the existing Hope Canal channel under I-10 to its original full section.

The plan to reintroduce MR water into the Maurepas Swamp has been investigated for decades by various public and private entities. Most recently, AECOM submitted plans and specifications for the Maurepas Diversion to CPRA in September of 2013 and an accompanying report in May 2014. Since that time, major changes to both the existing conditions and the overall Scope of Work have been made. The primary changes have been the development of the Marathon Petroleum Mt. Airy Terminal facility on land adjacent to the proposed diversion alignment and the construction of two Marathon Petroleum marine docking facilities adjacent to the diversion intake (one is complete at this time, the second is currently under construction).

The proposed construction of the WSLP Project features adjacent to the diversion have necessitated significant changes to the Maurepas Diversion. These changes include, but are not limited to:

- The WSLP floodgate structure at River Road blocks the existing drainage flow, requiring a complete re-design, including the addition of drainage ditches on both sides of the diversion and flood protection features
- The WSLP floodgate structure at CN Railroad similarly blocks drainage flow, necessitating additional drainage culverts and ditches.
- The raising of Airline Highway to the flood protection elevation is a major revision that also entails a complete re-design of the roadway crossing
- The construction of the USACE Pump Station Complex requires the realignment of the diversion Conveyance Channel.
- The Conveyance Channel alignment requires adjustment in other locations to provide room within the Right of Way for WSLP features.

- Geotechnical investigations require updates for the significantly different channel cross-sections.

The development of the Maurepas Diversion to a 95% Design phase has been documented in two previous project reports submitted to CPRA: 1) Preliminary Design Report: *River Reintroduction into Maurepas Swamp (PO-29 LDNR Contract No. 2511-06-10)*, prepared for the State of Louisiana, Department of Natural Resources, with a Federal Sponsorship by the US Environmental Protection Agency, prepared by URS (which was acquired by AECOM on October 20, 2014) in association with Evans-Graves Engineers, Wink Engineering, and 3001, Inc., submitted in June 2008, and 2) 95% Design Report: *Mississippi River Diversion into Maurepas Swamp (PO-29 Contract No. 2503-11-63)*, prepared for the State of Louisiana, Coastal Protection and Restoration Authority, with a Federal Sponsorship of the US Environmental Protection Agency, prepared by URS (nee AECOM), submitted in May 2014.

### 2.3.2 WSLP Project

The WSLP Project is designed to provide hurricane and storm-damage risk reduction in St. Charles and St. John the Baptist Parishes. It consists of a levee system around the communities of Montz, Laplace, Reserve and Garyville, LA. The system will be comprised of approximately 18 miles of earthen levees and floodwalls, six floodgates, a drainage canal running parallel to the levee on the south side, an Environmental Canal running parallel to the levee on the north side, four drainage structures, and four pump stations along the alignment. The flood protection features of the final three reaches of the WSLP Project (WSLP-111, WSLP-112, and WSLP-113) are to be constructed parallel to and immediately adjacent to the Maurepas Diversion.

The WSLP-111 reach is comprised of a section of flood protection levee that ties in to the MRL, a floodgate across River Road, and a floodwall from River Road to the Canadian National Railroad (CN RR). Reach WSLP-112 contains a section of flood protection levee from the CN RR to the Kansas City Southern Railroad (KCS RR) as well as a second levee section from the KCS RR to Airline Highway. The northernmost of the project reaches, WSLP-113 is comprised of a flood protection levee from Airline Highway to the USACE Pump Station Complex in reach WSLP-110.

### 2.3.3 Combined Project

Goals and features of the combined Maurepas Diversion and WSLP Projects include:

- Reconnect the Mississippi River to the Maurepas Swamp
- Provide flood protection for the portions of St. John the Baptist Parish where the two projects share an alignment (reaches WSLP-111, WSLP-112, and WSLP-113),
- Convey up to 2,000 cfs of river-water flow through the conveyance channel from the MRL to the Maurepas Swamp by operating the diversion structure gates. This flow is anticipated to be achievable for at least 6 months per calendar year for the full 50-year design life of the diversion system.
- Maintain the current level of flood risk reduction of the MRL and provide future flood risk reduction levels as determined by the USACE for the WSLP system.
- Design the intake structure, control structure, channel, and appurtenances to guide flow efficiently, and allow for operational adaptability based on monitoring data collected during project operation.
- Meet state and federal design criteria and environmental compliance requirements as required to achieve project regulatory approval
- Develop an operational plan for the diversion structure.

- Develop an Operations, Maintenance, Monitoring and Adaptive Management plan that will enable the Maurepas Diversion to be operated to maximize the rate of recovery of the swamp.
- Ensure positive cut-off of flow at all gated crossings.
- Develop a communication procedure for the USACE to instruct the Pontchartrain Levee District (PLD) to close the floodgates and communicate with CPRA for the closure of the diversion intake gates. This goal has not been addressed in the current design effort; however, it will be in subsequent phases of design.

## 2.4 Scope of Work – Task Order 1

The Scope of Work that CPRA charged AECOM with under Task Order 1 for the combined Project entailed the following.

### 2.4.1 Project Management

This consists of basic project administration, conducting regular conference calls and project design meetings, developing a Safe Work Plan, developing a Design Quality Control Plan (DQCP), providing technical design coordination, and reporting and invoicing.

### 2.4.2 Stakeholder Engagement

This work consists of coordinating with all project stakeholders, including adjacent property owners, private industry (e.g., Marathon Petroleum), CN Railroad, KCS Railroad, St. John the Baptist Parish government, the Pontchartrain Levee District (PLD), Louisiana Department of Transportation and Development (DOTD), and the Louisiana Department of Wildlife and Fisheries (LDWF).

### 2.4.3 Site Surveys & Mapping

This task includes compiling the existing survey data, developing a Survey Data Collection Plan, establishing primary survey control from certified monuments, collecting survey data, and creating maps and drawings for illustration and design purposes.

### 2.4.4 Right-of-Entry & Right-of-Way

This consists of obtaining the required right-of-entry (ROE) permits to enter upon a property (e.g., Marathon Terminal, CN RR and KCS RR, etc.) for surveying, geotechnical, or other data gathering. It also includes identifying the temporary ROE access routes that will be required for the Contractor to construct the project as well as the permanent access routes for maintenance of the project features. AECOM will also assist CPRA in the preparation of right-of-way (ROW) maps, including both permanent and temporary easements required both during and after construction.

### 2.4.5 Geotechnical Exploration, Lab Testing, & Analysis

The scope of this task includes compiling the existing Geotechnical data, developing a Geotechnical Data Collection Plan, collecting the field data (both CPTs and soil borings), conducting the appropriate laboratory analyses, and performing preliminary engineering analyses to progress the conceptual design.

### 2.4.6 Reference Information and Design Criteria

This item requires the compilation of the latest standards and criteria governing all aspects of the design, including the Geotechnical, Hydraulic, Structural, Civil, Mechanical and Electrical disciplines. This includes standards from recognized industry organizations, State design guidance, USACE

Engineering Manuals (EMs), among many others. It also includes development and definition of the Design Criteria documents that will govern future design work for the listed disciplines.

### 2.4.7 Relocations Coordination

This task consists of compiling a list of all existing utilities and pipelines throughout the project corridor. The various utility owners are to be contacted to verify the installation of their facilities, update them on the project, and request their technical requirements and the logistics involved.

### 2.4.8 Temporary Retaining Structures

This consists of identifying all the locations where significant excavation is to be performed, which will require the design of a temporary retaining structure (TRS).

### 2.4.9 Basis of Design Report

The Basis of Design Report (BODR), which is the subject report, includes the overall site development plan, the alignments of the diversion and flood protection features, and notes the design assumptions made. It also includes a description of the alternatives investigated for the various project features and the reasoning for selecting the preferred option. Proposed contract reaches are developed and explained as well.

### 2.4.10 Rough Order-of-Magnitude Cost Estimate

This item includes developing a Rough Order-of-Magnitude (ROM) cost estimate based on the 15% Conceptual Design. The take-off quantities were performed by the various design disciplines and the estimator has applied his expertise to develop unit costs for each item, which then leads to an overall total project cost. No cost estimating software, such as MCACES was used at this conceptual level of design.

### 2.4.11 Hydraulic Modeling Development Plan

This task consists of developing an overall drainage plan for the impacted area, including calculating the hydrologic runoff volumes, sizing the proposed ditches and culverts, and developing the methodology to ensure positive drainage during all phases of construction. The existing Hydraulic Engineering Center's River Analysis System (HEC-RAS) model of the conveyance channel was also reviewed, and a plan developed to revise it as needed.

# 3. Survey Datum & Information

## 3.1 Survey Datum & Control

The survey datums used on the Project are:

- Horizontal coordinates: NAD83, LA State Plane, South, US Survey ft (2011) 2010.00 Epoch
- Vertical control: NAVD88 (2009.55 Epoch) Geoid 12B

For primary control benchmarks the survey subcontractor, Fenstermaker, researched National Geodetic Survey (NGS), CPRA, and USACE information and located recently established survey control monuments existing within the project area. The monuments located include:

- NGS OPUS Shared Benchmarks "MOUNT AIRY", "GARYVILLE", and "48H001 LADH 1979"

- CPRA Secondary Monument “PO29-SM-02”, and
- USACE Levee Monument 4437+49.06=PLMS 435”.

The NGS and CPRA monuments are Deep Rod Permanent Benchmarks and are located along the Diversion Corridor, at the south end, midway, and at the north end of the project. The USACE monument is a brass cap embedded 1-ft south of the levee crown. These monuments will be tied-in to the WSLP benchmarks and alignment during the next phase of design.

During each day that topographic surveys were performed, static GPS was collected at the Permanent Benchmarks. The GPS data was post-processed using a minimum of three Continuously Operating Reference Stations (CORS) adjacent to the project area. A minimally constrained adjustment was performed to determine outliers and cycle slips. Once outliers were removed, a fully constrained adjustment was performed, holding to three CORS stations, and Scaler applied until the GPS network passed the Chi Square Test with 95% Confidence. The final adjusted results for each benchmark were tabulated and are provided in the Survey Data Collection Report in **Appendix A** along with the Survey Monument Data Sheets.

For Quality Control, the raw GPS data was converted to RINEX and submitted to the NGS OPUS Program for an independent check. OPUS Solutions for each benchmark were tabulated and averaged to compare with the fully constrained GPS Network Adjustment as a quality assurance check. All surveys were referenced to the three Permanent Benchmarks to ensure the reliability of the project’s control. The Survey Data Collection Report further explains the QA process and provides details of the OPUS Comparisons to Static GPS Adjustment.

Due to ongoing construction in the river and batture area by Marathon Petroleum, Fenstermaker was not able to perform the bathymetric survey work during this phase; however, such work will be performed during the next phase and when Marathon construction permits. After the field work, Fenstermaker will coordinate the process of converting the bathymetric data to updated datums and to Local Mean Sea Level (LMSL). All geospatial data will contain metadata which defines the relationship between NAVD88 and the LMSL local tidal datum.

## 3.2 Surveying Standards

All surveys have been conducted in accordance with CEMVN-ED-SS-06-01, “USACE New Orleans District Guide for Minimum Survey Standards for Performing Hydrographic, Topographic, and Geodetic Surveys”. GPS static networks shall follow the NGS Publication 58 guidelines for establishing vertical control. All RTK surveying was validated with documented ties to the project control for survey Quality Control\Quality Assurance. **Figure 3.1** shows the GPS network used for survey control.


**Figure 3.1 – GPS Network Utilized for Survey Control**

## 3.3 Alignments and Baselines

An alignment centerline was developed for both the diversion and the flood protection features, centered on the conveyance channel and the flood protection line of each of the projects, respectively. The alignments have been adjusted, as necessary, to minimize the acquisition of private property while maintaining the efficient function of all water diversion and flood protection features. The alignments were created based on the survey data collected during this Task Order and are subject to the approval of CPRA and USACE.

### 3.3.1 Maurepas Diversion

The alignment centerline of the Maurepas Diversion begins in the MR at Station 0+00 and terminates 1,000-ft north of Interstate 10 at Station 302+19 in the current 15% Design. The centerline follows the center of the various diversion features, including the intake channel, the headworks, the River Road culverts, the sedimentation basin, the conveyance channel, the CN RR culverts, the KCS RR culverts, the Airline Highway culverts, and the approximate center of the Hope Canal to the end point. The alignment centerline is a dynamic component of the complete plan set, which will continually shift as the design progresses from 15% Design through Final Design. The CADD set-up of the project uses AutoCAD Civil 3D with a geospatially correct three-dimensional surface, enabling the centerline to be active with a direct connection to the project features. This enables the alignment stationing, various plan and profiles, cross-sections, and other drafting items to be automatically updated each time a shift of the alignment occurs.

#### 3.3.2 West Shore Lake Pontchartrain Flood Protection

The alignment for the WSLP Project was established as the centerline of the levee sections, the wall-line of the various I-wall and T-wall sections, and the centerline of the flood protection gates. The local WSLP Project alignment will be tied into the overall alignment of the West Shore Lake Pontchartrain Flood Risk Reduction Project by station equations and offsets; the correlation will be detailed in the Design Drawings in future submissions, once WSLP alignment information is provided.

# 4. Project Design Criteria

The Project Design Criteria that will govern the development of the subject Project is compiled in **Appendix C**. It includes criteria for each of the various disciplines: Hydraulic Design, Geotechnical Design, Civil\Roadway Design, Structural Design, Mechanical Design, Electrical and Instrumentation Design. The Project Design Criteria will remain a “living document”, subject to addition and revision, until final acceptance by the USACE and CPRA. WSLP design elevations, alignments and other data will be incorporated when available; geotechnical and survey data collected during this Task Order will be added early in the next design phase.

# 5. Data Collection

## 5.1 Survey Data

### 5.1.1 Project Surveys and Imagery

The survey upon which the Maurepas Diversion Preliminary and 2013 Designs were based was performed in 2003. A great deal has changed over the last seventeen years. There has been riverine and landside development, including construction of the Marathon Terminal and its dock and pipe bridge complex. Property development, drainage infrastructure changes, and soil subsidence have also occurred throughout the project area. Over the period, improvements in survey technology have also transpired. The refinement of the Continuously Operating Reference Station (CORS) and the High Accuracy Reference Network (HARN) networks has provided a set of vastly improved positional coordinates. There has also been continued refinement in the reference geoid for projecting the collected data onto the most accurate spherical shape of the earth’s surface enabling more accurate determination of ellipsoid heights. Also, the Maurepas Diversion survey did not extend to the areas in which the WSLP flood protection features are to be constructed.

For the above reasons, the entire project footprint, including both the Maurepas Diversion and the WSLP Project areas were (re)surveyed. Survey data obtained during the BOD phase includes the following:

- Mississippi River Bathymetric and Magnetometer Surveys from the Maurepas Diversion Intake to just downstream of the proposed temporary dock for delivery of construction materials
- Conventional Topographic Survey throughout the entire Project footprint
- High resolution LiDAR data taken via an Aerial Vehicle (UAV), i.e., a drone; using ground control markers for post processing rectification and horizontal/vertical control.
- High-resolution aerial photography taken via a low-flying UAV, from the Mississippi River to 1,000-ft north of Interstate 10

Detailed information is provided in the Survey Data Collection Report in **Appendix A**.

### 5.1.2 Survey Data Collection Plan

A Survey Data Collection Plan for the combined Project's required survey work was submitted to AECOM by their subcontractor, C.H. Fenstermaker, Inc. (Fenstermaker) in May 2020. AECOM edited that plan and submitted it to CPRA for review and comment. After acceptance, Fenstermaker mobilized on July 13, 2020 to begin field work. This plan was revised twice during the course of this Task Order to add new technologies and data collection points to the Plan; details of these modifications are below. The final Survey Data Collection Plan is included in **Appendix A** with the Survey Data Collection Report.

A set of interim combined survey deliverables, including conventional ground surveying, LiDAR data collection, and low level detailed aerial photography of the area was submitted to CPRA on October 16th, 2020. Fenstermaker completed the survey field work (with the exception of the bathymetric and magnetometer work in the MR) on December 18, 2020 and completed the processing of the data, which it submitted to AECOM at the end of December 2020.

### 5.1.3 Revisions to Survey Data Collection Plan

After discussion with CPRA, and under their approval, the Survey Data Collection Plan was revised on May 22, 2020 to include LiDAR survey of the entire project footprint. This enabled the conventional ground survey data collection to be reduced, creating an overall savings to the Project. A second revision to the Survey Data Collection Plan was submitted on August 7, 2020, which added surveying of additional drainage features such as the Sugar Mill Ditch and the Marathon detention ponds, to improve the drainage modeling. This change was also discussed with and approved by CPRA.

Due to the cost savings generated by the use of the LiDAR data, with its concurrent reduction in conventional data collection requirements, the additional work to collect needed drainage information still fit within the proposed budget and therefore is within the umbrella of the original Scope of Work.

Revisions to the Plan were made because additional data was needed for drainage design of the Project area. Two types of additional data were collected: 1) data on the pertinent existing culvert crossings, and 2) data on the key components of the existing drainage ditch network. Annotated images showing the locations where data was definitively recorded are included as **Appendix A**. The surveyors were also charged with recording data on other culverts that might be found within the project area that were not specifically noted but that were encountered during the field work. A brief description of the data to be collected for the culverts and ditches is outlined below.

#### 5.1.3.1 Data on Existing Culverts

Additional existing culvert data collected during Fenstermaker's effort include: 1) the geospatial location of the culverts at their inlet and outlet ends, and 2) the invert elevation of the culverts at their inlet and outlet ends.

Visual inspections of the following were also performed at each location: 1) the number of culverts, 2) the sizes of the culverts, 3) the condition of the culverts (e.g., damaged, crushed, silt build-up), 3) an observation of flow or indications of past flow, and 4) an observation of current blockage or indications of past blockage.

Photographic documentation was to be performed at each location, which required: 1) wide view photographs to orient the viewer, and 2) close-up photographs to document the existing culvert conditions.

Field reports on the locations, invert elevations, visual inspections, and photographs are included within the Survey Data Collection Report.

#### 5.1.3.2 Data on Existing Ditches

The Survey Data Collection Report Appendix also includes annotated images showing the locations where ditch cross-sections were performed. As the images show: 1) four cross-sections were taken along Marathon's discharge ditch (formerly Sugar Mill Ditch), 2) two cross-sections were taken on either side of Grady Lane, and 3) one cross-section was performed across the beginning of Hope Canal.

Visual inspections of the ditches were also required, noting the existing conditions (e.g., whether the ditch section was well maintained or overgrown, evidence of erosion or sedimentation, etc.).

Photographic documentation was also required, including: 1) a wide view photograph to orient the viewer, and 2) close-up photographs to document conditions.

### 5.1.4 Results of Survey Data Collection Effort

The survey work was delayed by five major storm events hitting the Louisiana coastline, including two hurricanes, which caused significant downtime and demobilization. Obtaining ROE approval from the CN RR was also a protracted process. The final survey work consisting of the bathymetric and magnetometer work in the river has not been conducted as of this writing. None-the-less, with this exception, the entire three-dimensional surface has been incorporated into the plan set and the vast majority of the 15% Design plans are based on new survey data. Some individual drawings may not be updated or fully re-annotated, but the overall plan set is using the most current data.

The survey deliverables include:

- AutoCAD Civil 3D electronic files containing the conventional ground survey data,
- AutoCAD Civil 3D electronic files containing a massive data set of the LiDAR data collection,
- Geospatially correct, low level, detailed aerial photography, which was imported into both the AutoCAD Civil 3D drafting and design software and the ESRI ArcMap GIS software,
- Signed and sealed survey drawings, depicting the information collected on the various infrastructure elements and the topographic features of the area.

## 5.2 Geotechnical Data

### 5.2.1 Project Geotechnical Effort

For the 15% Design effort AECOM's Geotechnical subcontractor, Eustis Engineering, LLC, (Eustis) was instructed to concentrate on portions of the Project along the alignment of the WSLP-111, -112, and -113 flood protection reaches, which ranged from River Road to approximately 2,500-ft north of Airline Highway. Sufficient historical data is assumed to have been collected in the MR batture and levee area as well as in northern portions of the Maurepas Diversion in the swamp surrounding Interstate 10.

Eustis' first task was to review the historical Geotechnical data, analyses, and reports supplied to them by AECOM and develop a Geotechnical Data Collection Plan for additional soil borings and Cone Penetrometer Tests (CPTs). As the plan was being developed by AECOM and CPRA, CPRA coordinated closely with the USACE to ensure that the plan met all USACE requirements. After this plan was accepted by all parties, soil samples were collected from boring and CPT operations and sent to the Eustis laboratory for soil classification, consolidation, and other standard Geotechnical tests. Finally, they performed some basic Geotechnical engineering analyses such as determining the lengths of the stability berms: 1) between the top of bank of the conveyance channel to the western toe of the WSLP levee, and 2) between the eastern toe of the levee and the eastern drainage ditch.

### 5.2.2 Geotechnical Data Collection Plan

Eustis developed a Geotechnical Data Collection Plan (see **Appendix B**), which was reviewed by AECOM and submitted to CPRA on July 16, 2020. The plan included the number, depth, type, diameter and location of the proposed borings and CPTs. CPRA recommended modifications to the plan based on locations of historical data sets, and also based on their interpretation of what tests are required to give sufficient support for design and earn USACE acceptance of the geotechnical design parameters. The resulting changes involved eliminating some of the borings\CPTs and shifting some to different locations.

The basic assumptions of the Geotechnical Data Collection Plan were:

- Boring & CPT depths along the proposed levee alignment will be 50-ft,
- Correlation CPT depths will be 100-ft (or to refusal),
- Borings at four structure locations will be 125-ft deep:
  - River Road gates and/or T-wall
  - CN Railroad
  - KCS Railroad
  - Tie-in near Hope Canal

The final proposed Geotechnical exploration scope is summarized below.

***Table 5.1 -— Proposed Geotechnical Exploration Scope***

| Scope Item | Quantity |
|---|---|
| 5-in Diam. Undisturbed Boring | 13 Locations |
| Total 5-in Diam. Sampling | 950-ft |
| Cone Penetration Tests | 14 Locations |
| Total CPT Footage | 950-ft |
| Permit Acquisition & Surveying | To be provided by others |

During the data collection effort, several locations were revised based on the field conditions encountered. For example, six of the test locations would have been in the Angelina Canal. Upon approval of CPRA, those test locations were shifted 5-ft to 10-ft over to avoid drilling within the canal.

Concurrent with the field operations, as sample cores were collected they were sent to the laboratory for testing. The proposed Geotechnical laboratory testing scope is summarized below.

***Table 5.2 – Proposed Geotechnical Laboratory Testing Scope***

| Laboratory Test | Est. Quantity |
|---|---|
| Atterberg Limits | 228 |
| Grain Size Analysis, Sieve & Hydrometer | 26 |
| Unconfined Compression Test | 26 |
| Unconsolidated, Undrained Triaxial Test | 190 |
| Classification of 5-in Diam. Sample | 950-ft |
| Consolidation Test (4-in Diam. Sample) | 12 |

| Laboratory Test | Est. Quantity |
|---|---|
| Specific Gravity | 12 |
| Organic Content | 26 |

Eustis mobilized on August 3, 2020, completed field work on November 6, 2020, and completed their laboratory work on December 18, 2020.

### 5.2.3 Results of Geotechnical Data Collection Effort

As with the surveying work, the Geotechnical data collection was hampered by five major storm events, including two hurricanes, hitting the Louisiana coastline. This caused significant downtime because operations had to be suspended and equipment demobilized to safe locations, then remobilized to resume work. However, the Geotechnical Data Collection Report deliverable was still completed and submitted to CPRA for review in December 2020.

The Geotechnical data from the field collection, the laboratory testing, and the engineering analyses are contained in the Geotechnical Report in **Appendix B**.

## 5.3 Controlling Elevations

**Figure 5.1** shows the hydraulic reaches of the WSLP project as proposed by the USACE.

**Table 5.3** lists the 1% Water Surface Elevations (WSEs) and the 1% Levee design elevations for the years 2020, 2023, and 2070. It also lists the elevations for performing Geotechnical stability analyses, along with the corresponding required factors of safety for the various water levels.

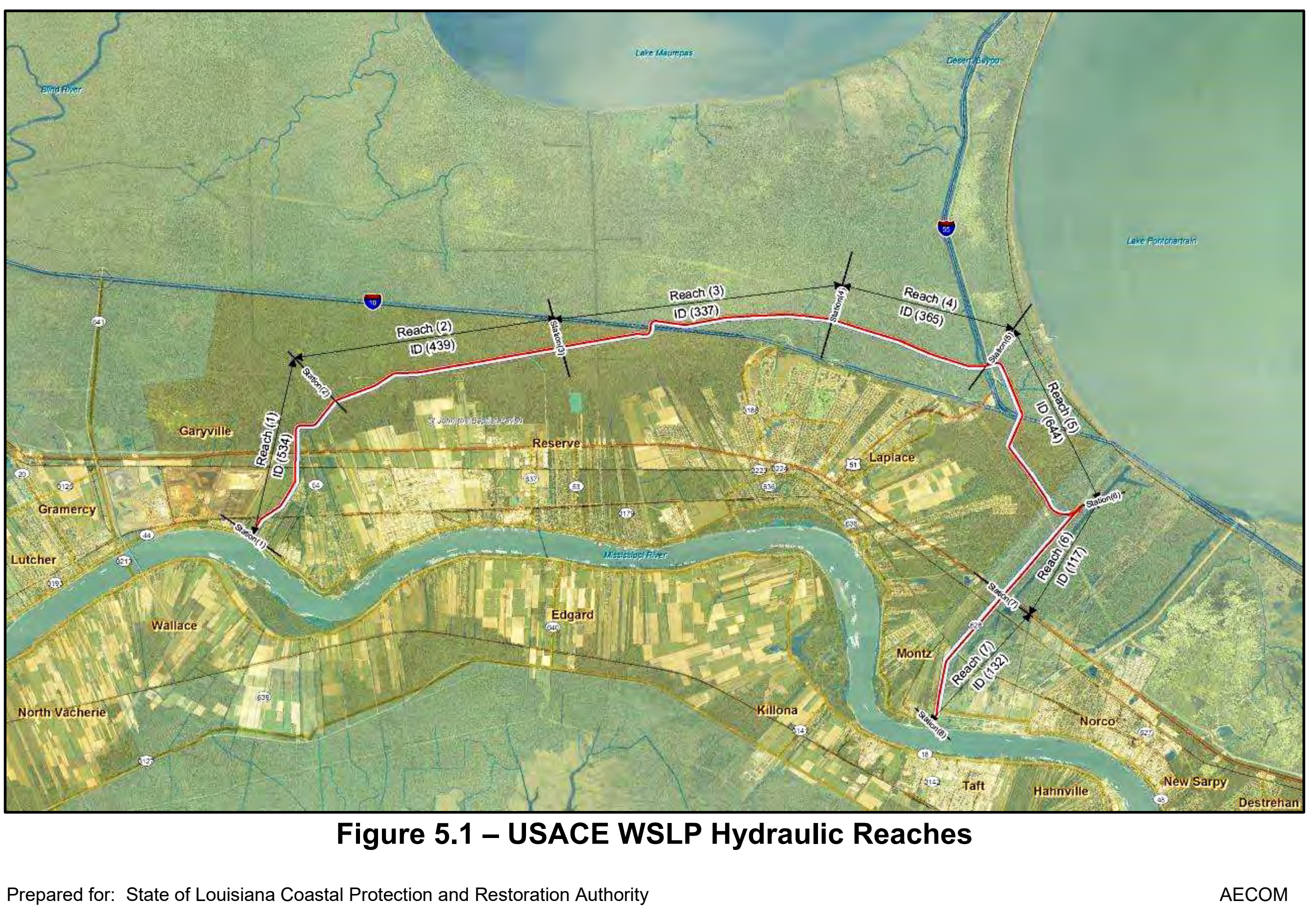


**Figure 5.1 – USACE WSLP Hydraulic Reaches**

***Table 5.3 – USACE WSLP Elevations, Levee Alignment C - Reach 1, Intermediate Sea Level Rise***

| 1% Water Surface Elevations | | 1% Levee Design Elevations | | |
|---|---|---|---|---|
| **Year** | **90% WSE ft NAVD88(2004.65)** | **Year** | **Levee with 1:3 Slope ft NAVD88(2004.65)** | |
| **2023** | 7.3 | **2023** | 9.0 | |
| **2070** | 12.7 | **2070** | 16.0 | |
| **Stability Analyses Elevations** | | | **F.S. requirements** | |
| **Water Grade** | **Flood Side Canal** | **Land Side Canal** | **Spencers Method** | **MOP Method** |
| **Low Water Level** | -2.8 | -2.8 | 1.4 | 1.3 |
| **Still Water Level** | N/A | -1 | 1.5 | 1.3 |
| **Water at Project** | N/A | -1 | 1.4 | 1.2 |
| **Water at Contruction** | N/A | -1 | 1.2 | N/A |

**Table 5.4** lists the 2020 and 2070 design elevations for the various WSLP Project features along with the construction type and the existing grades.

***Table 5.4 – Project Existing and Design Elevations***

| Project Feature | Construction Type | Existing Grade | 2023 Design Elevations | | 2070 Design Elevations | |
|---|---|---|---|---|---|---|
| | | | **SWL** | **Design Grade** | **SWL** | **Design Grade** |
| MRL Tie-In | Levee | +7.0 | +7.3 | +9.0 | +12.7 | +16.0 |
| River Road | Gate | +7.0 | +7.3 | +9.0 | +12.7 | +16.0 |
| I-Wall Reach | Structure | +7.0 | +7.3 | +9.0 | +12.7 | +16.0 |
| T-Wall Reach | Structure | +6.0 | +7.3 | +9.0 | +12.7 | +16.0 |
| CN Railroad | Gate | +11.0 (Top of Rail) | +7.3 | +9.0 | +12.7 | +16.0 |
| KCS Railroad | Gate | +9.85 (Top of Rail) | +7.3 | +9.0 | +12.7 | +16.0 |
| Airline Hwy | Ramp | +6.0 EB, +7.0 WB | +7.3 | +9.0 | +12.7 | +16.0 |

## 5.4 Project Feature Shapefiles for Wetlands Value Assessments

AECOM provided numerous ArcMap GIS shapefiles to CPRA describing the various features of the Maurepas Diversion and WSLP Project Reaches 111, 112 and 113. These files covered the entire project, including proposed staging areas; permanent and temporary access roads; and turn-arounds.

Also included were the Intake structures, proposed temporary staging platform, and board road for material hauling. Representative maps illustrating the various shapefiles are contained in **Appendix H**.

CPRA instructed AECOM to minimize impacts to wetlands to the extent possible, while also covering the acreage potentially needed to construct the project. CPRA and AECOM reviewed the files to assess the underlying assumptions and ensure the correct degree of conservatism was applied. The initial analysis indicated approximately 255 acres of Permanent Impacts and 23 acres of Temporary Impacts.

CPRA sent the shapefiles to the USACE for determination of the wetlands impacts. The USACE Environmental Clearance Boundary (ECB), a compilation of their initial Environmental Impact Statement (EIS) boundary and subsequent Environmental Assessments (EAs) boundaries, was superimposed on the shapefiles to determine areas that had already been cleared for environmental impacts. The USACE performed a habitat analysis to assess the impacts of their proposed WSLP work within the ECB; a similar process was undertaken by CPRA to develop a Wetlands Value Assessment (WVA) for the subject Maurepas Diversion\WSLP Project.

CPRA also forwarded the shapefiles to their contractor, Oil Land Services (OLS), for ownership updates. OLS overlaid the shapefiles onto land ownership maps to prepare plats showing the permanent access features. This will enable CPRA to develop a Real Estate Plan, which will be submitted to the USACE for their review and comment. Once that process is complete, CPRA will develop a land acquisition plan and a cost estimate for delivery to the USACE Mitigation Team.

# 6. Maurepas Diversion Project Features

Maps of the combined Project features are shown in **Figures 6.1, 6.2 and 6.3**. As **Figure 6.1** shows, the Maurepas Diversion features are on the west side of the Project and the WSLP flood protection features are on the east side. The Maurepas Diversion separates from WSLP at the proposed USACE Pump Station and continues north. **Figure 6.2** shows Maurepas Diversion work to be done north of I-10, and **Figure 6.3** highlights the structural features of the combined Project. The function and design process for each of the Maurepas Diversion features are described in the following sections.

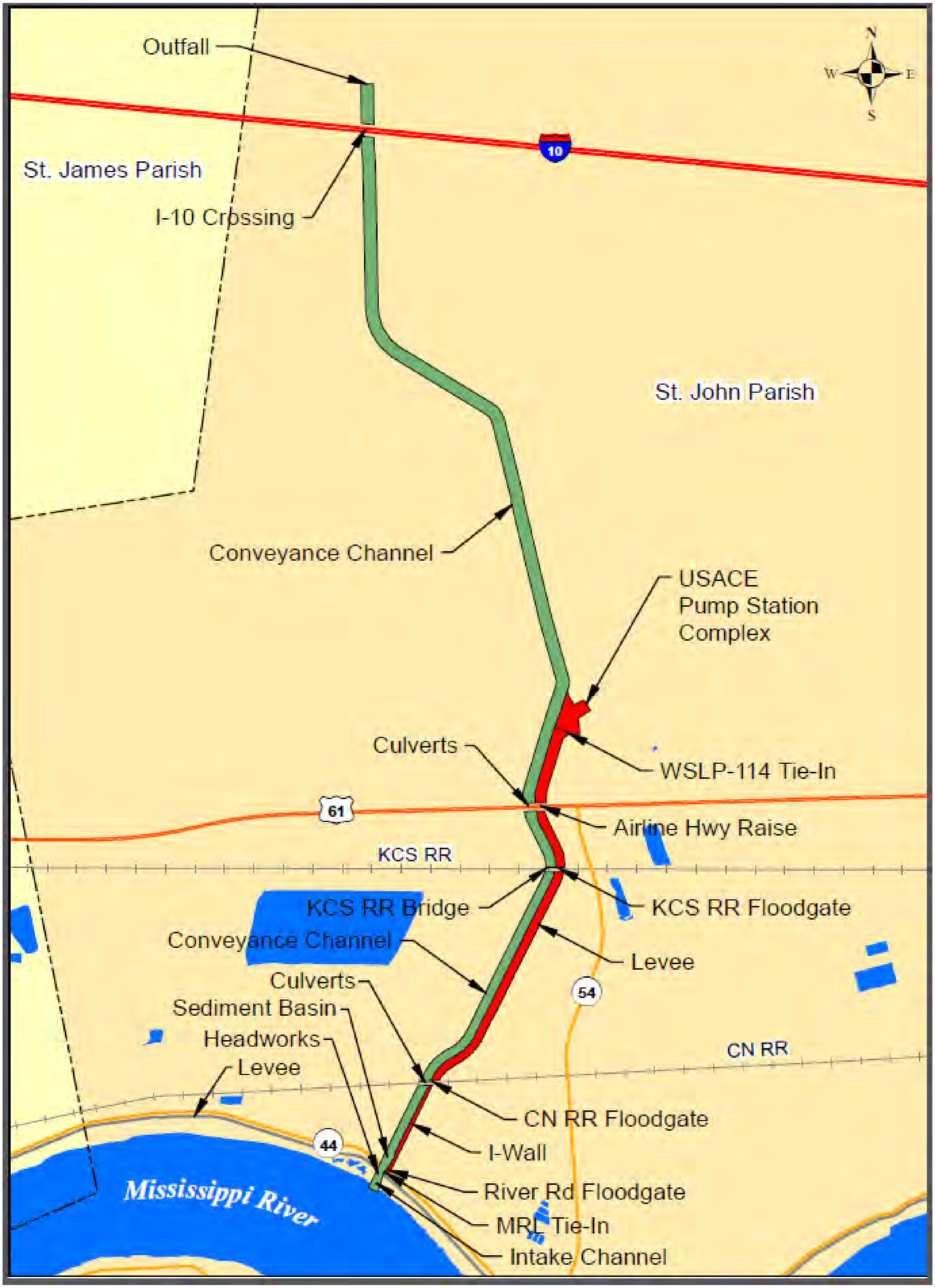


**Figure 6.1 – Project Features from Mississippi River to Interstate-10**

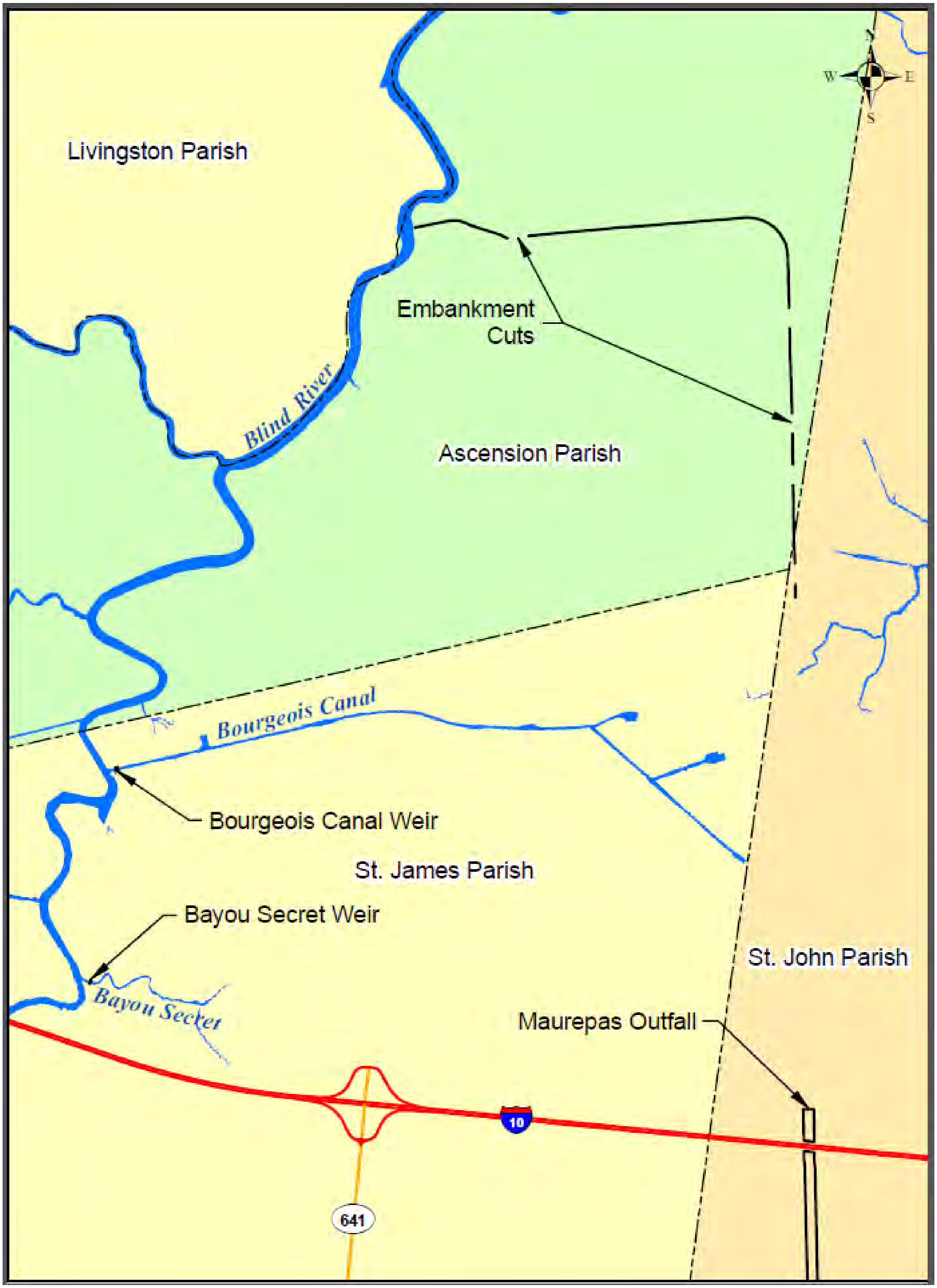


**Figure 6.2 – Project Features North of Interstate-10 (in black)**

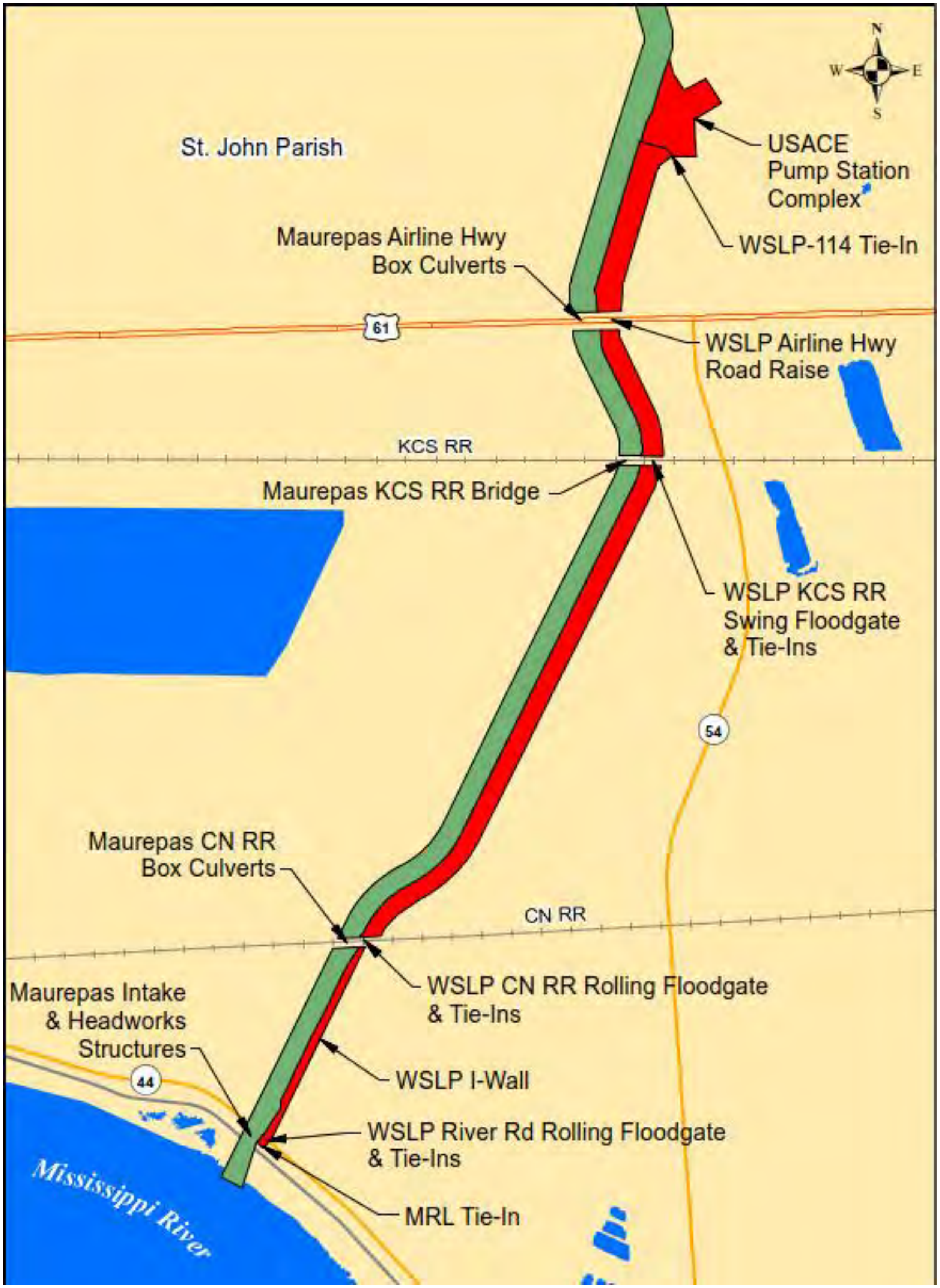


**Figure 6.3 – Structural Project Features**

## 6.1 Intake Channel

### 6.1.1 Previous Design

The layout of the intake channel remains the same as it was for the 2013 Submittal. It is roughly 400-ft long by 200-ft wide, with a bottom depth at EL (-) 4-ft NAVD88 excavated into the batture to route flow from the MR into the diversion Headworks. The channel will be lined with riprap to prevent scour. At the downstream end of the channel, three U-shaped reinforced concrete inflow monoliths are designed to channel river flow into the gated Headworks Structure. The intake U-Channels were re-designed between Preliminary and the 2013 Design to include three sections instead of two to facilitate constructability. The riverside wingwalls were also re-configured to be straight instead of curved to reduce the complexity of construction. The design process is described in detail in the Preliminary and 2013 Design Reports.

### 6.1.2 Marathon Docks and Pipe Bridge

#### 6.1.2.1 Dock and Bridge History and Agreements

A Memorandum of Agreement (MOA) between CPRA and Pin Oak Holdings, LLC (Pin Oak) regarding their Coast Use Permit Application (CUP No. P20131054) for their Marine Facility (Dock 1) was entered into in July 2014. In November 2017, Pin Oak amended that permit to revise the location of Dock 1 and incorporate an additional dock (Dock 2). In March 2018 Pin Oak was granted a Coastal Use Permit (CUP) (LaGov No. C109300406.15 Amendment No.1 “Coastal Use Permit Application for Marine Facility”) to construct Dock 1 upstream of the Maurepas Diversion intake, which would connect to the landside end of its pipe bridge, and Dock 2 downstream of the Maurepas intake. A pipe bridge was proposed to connect the two docks.

In October 2014, a MOA between CPRA and Pin Oak was executed establishing the conditions of operations and development and particularly the ROW line between the Pin Oak land terminal and CPRA’s Maurepas Diversion. Subsequently, Marathon Petroleum, as Mt. Airy Terminal, LLC (Marathon), acquired the land- and marine-based facilities from Pin Oak. In August 2019 Marathon entered into a Memorandum of Understanding (MOU) with CPRA regarding a revision to the alignment of Dock 2 and Marathon’s obligations to construct and maintain the portion of the Maurepas intake from the riverside to 50-ft beyond the landside of the pipe bridge.

Marathon sought and obtained approval of a second CUP to construct a second dock downstream of the Maurepas Diversion intake. This permit included the construction of a pipe bridge along and parallel to the edge of the MR revetment, which crosses over the riverside end of the Maurepas Intake Channel. Based on evaluations of the prosed Marathon structures by the River Boat Pilots Association and others, Marathon sought to re-orient the pipe bridge to facilitate safe docking of the maritime vessels that would be using the facilities. The reorientation moved the pipe bridge toward the landside, causing the bridge to span over more of the Maurepas Diversion Intake Channel.

#### 6.1.2.2 Effects of Marathon Structures on the Project

Modeling conducted by The Water Institute of the Gulf (TWIG) aimed at determining the effects of the pipe bridge across the diversion intake demonstrated that the structures would have no significant impact on the volume of flow entering the diversion. However, TWIG noted that shoaling of sediment against the bridge piers was a potential maintenance issue. To avoid CPRA bearing responsibility for work directly affecting Marathon’s pipe bridge structures, both parties signed a MOA which states that Marathon will excavate the intake from the riverside end of the diversion channel, under the pipe bridge, and to a point 50-ft beyond the pipe bridge on the landside. The MOA also states that Marathon will place riprap throughout this section of the intake area as indicated on the Maurepas

Diversion plans. Further, the MOA designates Marathon as the responsible party to both monitor and remove any sediment accumulation in the areas on either side of as well as underneath the pipe bridge. Criteria for the monitoring effort and trigger points for sediment removal are also established in the MOA.

The construction of the docks and pipe bridge impact the Contractor’s means and methods to excavate the remainder of the landside portion of the Intake Channel as well as to deliver material required to build the Intake Channel and Headworks. Without the Marathon facilities blocking access from the River, portions of the work were anticipated to be conducted from the water. The channel would be dredged from a river borne dredge, the riprap would be delivered and placed from the riverside via a barge with a crane, and the materials to construct the construct the headworks, (e.g., the foundation piling) would also be delivered from the river. The pipe bridge also precludes the dredging of the channel on the landside up to the Headworks, so the Intake Channel will have to be excavated by land-based equipment.

Moving large quantities of materials, such as riprap and piling, is much less expensive by water than by land. The estimated trucking costs of this Project are so extreme that they justify the construction of a temporary dock downriver from the second Marathon dock for the delivery of bulk materials. Materials will be off-loaded by barge-mounted cranes onto the temporary dock, then loaded into trucks from a permanent crane on the dock. This arrangement does require double handling of the material, however an alternatives analysis shows this is still cheaper than overland delivery. A board road is proposed along the batture to enable truck transport of the construction materials from the temporary dock to the project site. The temporary dock is discussed further in Section 11.2.4 (note that the construction of a temporary dock will be at the Contractor’s discretion).

The construction of the Marathon dock and pipe bridge facilities creates added costs for the construction of the Maurepas Diversion. The excavation of the landside portion of the channel by conventional equipment as opposed to dredging, the construction of the temporary dock and board road, and the double handling of materials all add costs that the Project must bear. However, the excavation of the Intake Channel by Marathon as described in the MOA will partially offset additional costs to the Maurepas Diversion project. The Marathon pipe bridge will also serve as a stout protective structure in front of the Headworks, thus obviating the need to construct three protective dolphins originally designed for installation in the Intake Channel. The monitoring and maintenance responsibilities to be conducted by Marathon under the terms of the MOA also transfer some of the Maurepas life-cycle costs to Marathon.

### 6.1.3 Alternatives Analysis

The design of the Intake Channel was fully developed in the 2013 Design submittal. The subsequent MOA with Marathon Petroleum to construct their pipe bridge across the Intake Channel had no significant effect on the design other than eliminating the need for three marine dolphins; however, it did limit the Construction Means and Methods. An Alternatives Analysis was conducted to compare the cost of delivering material to the Intake and Headworks by either: 1) overland trucking, or 2) delivery by river, facilitated by the construction of a temporary dock and a 1973-ft long board road along the batture to the project site.

The construction cost estimate has not been completed as of this writing; it is anticipated to be submitted within the next two weeks. However, in AECOM's estimation it will be significantly cheaper to move piling, riprap and other large materials in bulk quantities by water than by land. The substantial savings of delivering the quantities of material by river is believed to more than offset the cost of constructing the temporary dock and board road. Therefore, the second alternative was selected.

### 6.1.4 Recommended Design and Next Steps

The Recommended Design remains fundamentally the same as the 2013 Submittal. The only changes are the removal of the three protective dolphins planned and the construction of a temporary dock and board road to deliver materials to the project site.

In future design phases the concrete U-frame intake structures will be checked using current USACE and industry codes per the Project Design Criteria.

## 6.2 Headworks Structure

### 6.2.1 Previous Design

This Headworks Structure is designed to control the flow through the diversion system and provide the same level of flood protection as the adjacent MRL. The following sections summarize the state of the 2013 Design and the basic rationale for decisions made.

The primary function of the Headworks Structure is to convey flow from the Intake Channel underneath the MRL. To maximize opportunities to convey the 2,000 cfs design flow, the structure should also be designed to minimize head loss. Beyond these metrics, the structure must also function reliably over the life of the project under a wide range of conditions. A cost-effective solution to these objectives was determined to be a multi-cell box culvert with vertical lift gates (sluice gates). Similar arrangements are used for other diversions along the MR, specifically at Caernarvon (EB of the MR near the St. Bernard/Plaquemines Parish line) and Davis Pond (WB of MR in St. Charles Parish), and their long-term successful operation proves the reliability of the design concept. Other types of structures including pumps, tainter gates, and siphons were evaluated during previous phases of the Maurepas Diversion design and deemed unsuitable because of cost and/or functionality constraints.

#### 6.2.1.1 Headworks Structure Location & Elevation

The Headworks Structure was sited approximately 100-ft south of the existing MRL crown. The proximity to the levee provides a solid foundation for the structure and its adjacent support platform. The platform holds a control house, to be built at approximately EL +31-ft NAVD88 (Top of MRL) for protection against high river stages. Placing the structure close to the levee minimizes the required culvert lengths, which reduces the head loss through them.

The elevations of the Headworks Structure, sluice gates, and culverts are geometrically constrained by several factors. First, the gate was set as high as possible to minimize the excavation costs. Second, the WSE on either side of the culverts was set above their crowns so that they will operate under outlet control. This makes the culvert slope irrelevant to its hydraulic performance; the culverts may be installed level, reducing construction costs. Third, the culverts exiting the intake structure pass under the River Road roadside ditch, which has an invert of EL +7-ft NAVD88. The culverts require at least 1-ft of cover above the bottom of the ditch; subtracting 1-ft of cover yields a top of culvert at EL +6-ft NAVD88. Finally, structural calculations indicate the culvert tops need to be 2.5-ft to 3-ft thick. Subtracting 3-ft for the culvert top thickness results in a top-of-culvert elevation of +3-ft NAVD88.

#### 6.2.1.2 Sluice Gate Design

The primary goal in sluice gate sizing was to maximize the time during which the 2,000 cfs target flow rate could be achieved. The elevation difference between the MR stage and tail-water (outfall) level in the conveyance channel provides the impetus for gravity flow. To maximize the duration of peak flow conditions, head losses must be kept to a minimum. The tail-water elevation was computed through a backwater analysis in HEC-RAS from the Maurepas Swamp to the intake structure. Comparing the tail-water conditions to the seasonal MR stages enabled estimation of the annual duration for delivery of the design flow. (The "Mississippi River Stage Hydrographs at the Reserve

Gage" are presented as Appendix E in the 2014 report.) At 2,000 cfs, the WSE on the outfall side of the diversion structure was calculated to be approximately EL +7-ft NAVD88. Thus, the minimum river stage required would be EL +7-ft NAVD88 plus the head losses through the system at the design rate of flow.

The gates were designed for hydraulic operation. A fluid reservoir accommodates volume changes from cylinder extension and contraction, temperature shifts, and leaks. The reservoir also aids in the removal of air from the fluid and functions as a heat accumulator to cover system losses when peak power is used. Stainless steel reservoirs were selected for corrosion resistance; while initially more expensive, they require less long-term maintenance and are less likely to fail. The hydraulic reservoirs were also designed to be protected from water vapor and contaminant intrusion by a reservoir isolator that provides a closed system into which the reservoir may breathe.

### 6.2.2 Alternatives Considered

AECOM (as URS) performed an analysis of nine alternative sluice gate configurations during the Preliminary Design phase, The geometric parameters for a single 12-ft x 12-ft gate to three 8-ft x 8-ft gates were compared. Cost is proportional to structure size, so the cross-sectional area was used as a reasonable proxy for cost (e.g., three 12-ft x 12-ft gates at 432 $ft^2$ cost significantly more than three 8-ft x 8-ft gates at 192 $ft^2$). Conversely, the larger cross-section of the 12-ft x 12-ft gates provides much less resistance to flow, so a relatively lower river stage would be able to deliver the requisite 2,000 cfs. Based on historical river stage data and the goal of operating at full capacity for at least half of the year, a river stage of +9.38-ft NAVD88 was established as the design point for the intake structure. Three 10-ft x 10-ft gates was selected as the optimum configuration, which minimizes construction costs while also meeting the flow delivery requirements.

The Headworks Structure was complete to a 95% level in the 2013 Submittal to CPRA and requires no adjustments within this new combined Project; no additional work has been performed on any of the components of the structure during the subject Conceptual Design phase.

### 6.2.3 Recommended Design and Next Steps

The recommended Design remains a concrete Headworks Structure with vertical lift sluice gates that convey water under the MRL via three 10-ft by 10-ft box culverts.

The structure will be checked using current USACE and industry codes per the Project Design Criteria. Also, assumed Design WSEs for both the river and tailwater will be verified using the latest Hydrologic and Hydraulic (H&H) data.

In the next phase of design, a Supervisory Control and Data Acquisition (SCADA) system will be incorporated into the design. The system will allow CPRA to monitor and operate the diversion remotely from a location of their choosing. The SCADA system will also be connected to the Marathon Petroleum pressure loss signalization system so that if one of their petroleum pipes loses pressure (e.g., from a leak), both the valves on the pipeline will be closed and a signal will be sent to close the diversion system gates. This will be a protective measure to prevent petroleum-based pollutants from entering the Maurepas Diversion Channel.

## 6.3 Cofferdam

The design process is described in detail in the Preliminary and 2013 Design Reports. The following sections briefly summarize the status of the Cofferdam design as presented in the 2013 Submittal.

### 6.3.1 Previous Design

To construct the intake channel and headworks structure, a cofferdam must be installed to temporarily isolate this area from the river. The Cofferdam must serve as the mainline MRL flood protection throughout construction of the intake facilities, which is estimated to be over a year in length. As such, the Cofferdam design was not to be left to the Means and Methods of the Contractor, rather a fully designed retaining system was presented in the 2013 Design plans. An earthen Cofferdam was designed to conform to all applicable USACE criteria and standards; it will have to be approved by the USACE prior to construction. The Cofferdam design enables construction by the equipment, means, and methods the Contractor is expected to use.

Significant additional Geotechnical investigation was performed in the batture between the Preliminary and 2013 Design phases. This included the collection of additional soil borings and CPTs, as well as the associated laboratory analyses. Numerous Geotechnical engineering analyses were conducted based on the collected data to establish an accurate strength line for the Cofferdam design. The data was submitted to the USACE and comment and review exchanges were conducted until an approved strength line was established as the basis of design. In addition, numerous stability analyses along various potential failure planes were performed to demonstrate stability during each of the multiple construction phases.

Additional topographic surveying was conducted in the batture to define the extent and depth of the pond on the northwest side of the intake channel. The toe of the proposed Cofferdam would extend into the pond, thus additional information was needed to run the requisite geotechnical stability analyses to ensure the USACE factors of safety were met.

#### 6.3.1.1 Proposed Phasing Plan

The proposed phased construction of the Cofferdam, Intake Structure, and Headworks are summarized below:

- Phase I – Construct Access Ramps and Partial Cofferdam

  Construct levee access ramps, remove levee slope paving to nearest joint beyond ramps, fill east end of batture pond with select fill, and provide 10-ft bench to toe of Cofferdam at EL +18-ft NAVD88. Fill remainder of pond with site-supplied material to EL +18-ft NAVD88, construct Cofferdam to full width at EL +22-ft NAVD88 and drive 65-ft ± sheet piling along Cofferdam C/L flush to EL +22-ft NAVD88.

- Phase II – Completion of Cofferdam Construction

  Complete cofferdam construction to EL +32-ft NAVD88.

- Phase III – By-Pass Roadway and Initial Culvert Construction

  Remove section of MRL landside toe, construct by-pass roadway south of existing River Road, remove section of River Road. Install sheet piling for excavation on north side of by-pass, excavate, construct temporary access road. Install culvert sections C-4, C-5, C-6, U-4, U-5 and U-6, and remove sheet piling.

- Phase IV – Reconstruction of River Road and Removal of By-Pass

  Reconstruct removed portion of River Road in its original location, remove roadway by-pass.

- Phase V – Construction of Culvert on South End

  Install sheet piling for excavation both north and south of the culvert, partially excavate, install mechanically stabilized earthen wall on each side at north end of culvert. Complete excavation, construct temporary access roads to bottom of excavation. Install culvert sections C-1, C-2, and U3 and intake structure. Remove sheet piling and backfill.

- Phase VI – MRL Construction and Cofferdam Removal

Reconstruct MRL to EL +33.5-ft NAVD88, providing overbuild for anticipated settlement. Tie east and west ends into original section, install slope paving except for small area adjacent to intake structure. Degrade Cofferdam to batture elevation EL +18-ft NAVD88.

- Phase VII – U-Channel Construction

  Grade area around U-1 and U-2 to EL +12-ft NAVD88. Install sheet pile wall and construct U-1 and U-2.

- Phase VIII – Final Stage

  Excavate intake channel on north side of seepage piling. Drive piling to EL +18-ft NAVD88. Cut seepage piling within channel to match design grade, armor channel with riprap. Excavate channel south of piling to bank of MR, armor channel. Replace slope paving to original condition. Remove west levee access ramp; leave east levee ramp for permanent access.

#### 6.3.1.2 Cofferdam Design Details

The Cofferdam side slopes were designed to a maximum finished grade of 4H:1V. The minimum elevation of the Cofferdam was set at EL +25-ft NAVD88; sheet piling was included to prevent seepage. The construction sequencing precludes use of the excavated material as fill, so imported material was designated for construction of the Cofferdam. Historical MR stage hydrographs show that the river has not surpassed EL +25-ft NAVD88 in the past 50 years. The river data also shows generally predictable seasonal patterns; critical construction operations should be scheduled during low river stages, which is typically from June to November.

### 6.3.2 Alternatives Analysis

During the previous design phases several types of cofferdams were considered, including Cantilever Sheet Piles, Rock-Fill Embankments, Double-Wall Sheet Piles, Cellular Cofferdams, and Earthen Embankments. The Cantilever Sheet Piles and Rock-Fill Embankments were dismissed due to their pervious nature and unacceptable rate of leakage. The Double-Wall Sheet Piles were eliminated due to their need for excessive internal bracing and the requirement of an internal berm to keep the phreatic surface within the berm. The Earthen Embankment and Cellular Cofferdams were left as the remaining viable alternatives.

Both alternatives could effectively hold back the MR and serve as the mainline MRL flood protection; however, both also have requirements that affect the Project and must be weighed in a final decision. The earthen cofferdam requires a proportionally large space on the batture side of the existing MRL for its construction. A sheet pile cut-off wall was also deemed a requirement to minimize potential seepage. The cellular cofferdam is a strong and highly water resistant system; however, it is more difficult to construct and remove than most other cofferdam systems. With two viable cofferdam configurations, cost was the determining factor.

The vertical face of the cellular cofferdam enables it to be constructed within less space than the sloped surfaces of the earthen cofferdam. Thus, the total arc length of the cellular structure would be shorter than the earthen one. However, the construction cost of the cellular cofferdam is significantly more per linear foot. There are two viable options for the Cofferdam that have significantly different costs. Based on previous designs from other projects with similar geometric configurations, a cellular cofferdam for the Maurepas Diversion is estimated to require approximately 9 full cells and 8 half cells. At a cost of approximately $2.88 million per cell, the total cost for a cellular cofferdam would be approximately $37 million. The earthen cofferdam option shown on the proposed plans is estimated to cost approximately $5 million. These are gross costs, less accurate than a ROM cost. They are subject to significant refinement in the next phase of design. However, AECOM believes that the cellular cofferdam will remain by far the more expensive option.

The earthen Cofferdam design was presented in the 2013 Submittal; the design has not been revised during the current Task Order. The USACE submitted a set of comments in December 2014 in response to the AECOM (as URS) September 2013 Submittal, including some which addressed the Cofferdam design. AECOM (as URS) prepared a set of responses to those comments, however the Maurepas Diversion was placed on hold due to funding issues before they were issued. The responses to the comments will be issued to the USACE in the next phase of design.

#### 6.3.3 Recommended Design and Next Steps

The Recommended Design remains an earthen Cofferdam on the batture, with 4H:1V side slopes, including sheet piling for seepage, and a constructed elevation of EL +32-ft NAVD88. In addition to the reasons presented in the previous design, use of a cellular cofferdam may negatively affect the design and construction schedules, as a new cofferdam criteria and design will new require USACE review and acceptance.

In future phases the 2013 Cofferdam design assumptions will be verified, including the design river stages and analytical procedures used, to ensure the design is current. Stability analysis and other detailed geotechnical analyses will be re-assessed with current guidelines and geotechnical data. The strength line and stability analyses will be re-checked by Eustis to validate the original findings. If the design sequence of construction is modified, additional stability analyses will be required. Also, the phased construction process will be updated as needed based on this work.

In October 2019 AECOM, CPRA, and the USACE met to discuss Corps-related Project considerations, which included the Cofferdam design. Discussions with the USACE will continue through meetings and the formal comment\response process as needed to secure their final acceptance of the proposed Cofferdam system.

### 6.4 River Road Crossing

The design process conducted during the Preliminary and 2013 Submittal of the River Road crossing is summarized below.

#### 6.4.1 Previous Design

During the Preliminary Maurepas design, based on communication with Louisiana Department of Transportation and Development (DOTD), the installation of the culverts across River Road were designed as an open cut section. AECOM (as URS) met with DOTD in 2007 to discuss this issue and arrived at a consensus for traffic control.

In early 2013, AECOM (as URS) contacted DOTD to update them on the project status and inquire what was needed for approval of the roadway crossing. During that discussion, DOTD informed AECOM that River Road could only be closed to traffic for 45 days, instead of the year-plus duration required to install the culverts and other components adjacent to the roadway. This change did not affect the design of the Intake and Headworks features. However, maintaining traffic on the roadway during construction dramatically changed the sequence of construction. It restricted the work to one side of the roadway at a time, effectively cutting the work area in half resulting in the need for braced temporary retaining structures (TRS) to excavate to a 30-ft depth below grade (the batture is at EL +18-ft NAVD88, the bottom of the culvert is at EL (-) 12-ft NAVD88). AECOM proposed the phases described in the above Cofferdam section as a workable design using TRS, where required, to facilitate construction while enabling River Road traffic flow to continue throughout the majority of construction.

##### 6.4.1.1 Construction Phasing

The detailed seven phase sequence of construction, discussed in the Cofferdam section above, includes two very significant changes from the open cut approach: 1) the design of a 35 mph temporary by-pass roadway through the construction area to maintain traffic along River Road, and 2) the incorporation of multiple TRSs to provide stability and enable access to the excavation bottom. Geotechnical stability analyses were performed for each construction phase of the revised design to ensure that USACE factors of safety were met.

### 6.4.2 Current Design Effort

During the current effort, the design of the re-routed roadway to enable phased construction was revised slightly from the 2013 Submittal. The revisions include super-elevation of the curves, proper length tangent sections, and revised horizontal curve radii.

### 6.4.3 Alternatives Analysis

While minor revisions were made to the 2013 Submittal, the overall concept has not changed. Therefore, no Alternatives Analysis was performed for the River Road crossing during the subject work.

### 6.4.4 Recommended Design and Next Steps

The Recommended Design of the Maurepas Diversion components at River Road remain fundamentally the same as in the 2013 Submittal. Details of the new survey and geotechnical data will be incorporated into the road and culvert designs along with the refinements to the re-routed roadway geometry.. The concrete culverts will also be checked using current USACE and industry codes per the Project Design Criteria. No other changes to this portion of the design are anticipated. AECOM will assist CPRA in coordinating with DOTD to ensure all planned activities and structures will earn their acceptance.

The reconstruction of River Road and its construction detours will be coordinated with the WSLP Project, as discussed in Section 7.2.

## 6.5 Sedimentation Basin

The design of the Sedimentation Basin is described in detail in the Preliminary and 2013 Design Submittals; the following sections briefly summarize how the design was developed.

### 6.5.1 Previous Design

#### 6.5.1.1 Suspended Solids Capture

There is a high concentration of sand, silt and clay entrained in the MR flow-stream. To re-nourish the Maurepas Swamp, the fine silt and clay particles must be carried through the diversion to its outfall. However, the sand particles must be removed upstream of the conveyance channel to prevent deposition in the downstream reaches. The Sedimentation Basin was designed to remove this unwanted sand from the diversion flow-stream.

The design was based on the Louisiana Department of Natural Resources (LDNR) (from which CPRA was formed) recommendation to remove sand particles ≥ 0.2-mm in diameter; the surface area of the Sedimentation Basin was established based on the settling velocity of a 0.2 mm sand particle. The basin was also designed with adequate storage capacity to accumulate six months of sediment before requiring cleaning. Lastly, the cross-sectional area of the basin was calculated to achieve a flow velocity of approximately 1-ft/s at the design flow rate to prevent re-suspension of the settled solids.

The percent sand in the river water at Maurepas was derived by interpolating from data recorded at St. Francisville and Belle Chasse, which are upstream and downstream of the site, respectively. Data from the Caernarvon project provided a ratio of the percent sand in a diversion with respect to that in the adjacent river water. Applying that ratio to the subject site yielded the percent sand expected in the diversion influent. Using that value, the volumetric accumulation rate of sand expected in the basin was calculated to be approximately 78,000-ft$^3$ over six-months, which defined the additional basin volume required to contain the sediment. The basin was designed with a central section 265-ft long by 66-ft wide with 3:1 side slopes, adding 60-ft of width on each side. To provide the needed volume, the basin was designed to be approximately 4-ft deeper than that of the channel, resulting in an invert of EL (-) 11-ft NAVD88. The sediment storage area will extend up the side slopes to EL (-) 7-ft NAVD88, across a horizontal distance of 12-ft, resulting in an available storage volume 82,680 ft$^3$.

#### 6.5.1.2 Design for Sediment Removal

The top elevation of the Sedimentation Basin was set at EL +9-ft NAVD88, to provide sufficient freeboard during the 2,000 cfs design flow. The design includes a 12-ft wide access road around the basin perimeter for excavation equipment to periodically remove the accumulated sediment. Due to the heavy loads that the road will experience from excavators and haul trucks, the road is provided a 6" crushed stone base and a sub-base created by mixing cementitious materials with the structural fill. The soil mixing will also be performed along the side slopes to insure their stability.

The basin foundation was designed to sustain long term maintenance. It includes a ramp from the guide levee crown to the base of the basin. AECOM assumed that a rubber-tired backhoe front end loader will excavate the sediment and load trucks parked on the ramp. AECOM evaluated the need for soil cement mixing versus a riprap base; it concluded that the riprap base would be the most cost-effective alternative. A soil lining of 12-in of red clay at the base of the basin was incorporated into the design to enable the cleaning contractor to visually observe when he has reached the bottom and thus prevent removal of excess material.

The entire length of the Sedimentation Basin will have a 12-ft wide access road along the crown of both guide levees. The road will have a 6-in crushed stone base without the cement-stabilized sub-base of the sediment basin road. The conveyance channel road is designed only for the light trucks used by monitoring personnel.

#### 6.5.1.3 Function of Weirs

The Sedimentation Basin has been designed with a weir at each end; both will be set at EL +2.5-ft NAVD88. The first weir will reduce the turbulence created at the entrance, establishing more uniform flow. The weir presents a gradual rise in the bottom elevation that will equalize the differential velocities across the channel, creating a more laminar flow regime. This will reduce short-circuiting and allow particles to settle out of suspension and collect on the bottom of the basin.

The second weir serves a separate purpose. The sediment particles contained within the diverted water will not settle out uniformly; the basin will capture progressively greater accumulations of sediment as the flow moves to the downstream end. The second weir will act as a barrier to retain trapped sediment within the storage areaAlternatives Analysis

The design of the Sedimentation Basin has not changed since the 2013 Submittal. Therefore, no Alternatives Analysis was performed under the current scope of work.

### 6.5.2 Recommended Design and Next Steps

The Recommended 15% Design of the Sedimentation Basin remains a 265-ft long x 186-ft wide basin with an invert of EL (-) 11-ft NAVD88, a 12-ft wide top at EL +9-ft NAVD88, and a 12-ft wide geotechnically reinforced access road around its perimeter. The geometry of the Sedimentation Basin will be incorporated into the updated HEC-RAS model of the overall diversion system to confirm that its configuration meets the hydraulic requirements.

## 6.6 Conveyance Channel

### 6.6.1 Previous Design

The 5½-mile long conveyance channel alignment and 300-ft ROW width remained the same between the Preliminary and the 2013 Design. However, for the 2013 Design, the conveyance channel was widened to reduce the frictional headloss in the system, providing additional freeboard between the WSE and the top of the guide levees. The side slopes were also adjusted to reduce headloss, increase freeboard, and minimize potential sloughing.

South of the KCS RR crossing the proposed channel is designed with a typical bottom width of 40-ft, with a side slope of 4-ft horizontal:1-ft vertical (4H:1V) within the wetted portion of the channel and 3H:1V slopes on the outsides. North of the KCS RR the bottom width will be 60-ft, with 5H:1V water-side slopes and 3H:1V land-side slopes to existing grade.

### 6.6.2 Alignment & Geometry

Two revisions to the conveyance channel alignment have been made to accommodate the WSLP Project features:

- At the CN RR crossing, the centerline of the channel alignment was shifted east 60-ft to allow construction of a west drainage ditch within the ROW line agreed to between Pin Oak Terminals (now Marathon Petroleum) and CPRA in their October 8, 2014 Memorandum of Agreement. To match the 60-ft horizontal shift, the reinforced concrete box culverts (RCBs) were skewed eastward 10° from an angle of 29° to angle of 39° with the tracks; the effect of the shift on the hydraulics is discussed in Section 6.7.3.
- The alignment has been shifted to the west around the USACE Hope Canal Pump Station Complex to accommodate the larger footprint of the proposed pump station, bypass gates, wingwall, training berm, and grading and canal backfill areas.
- A privately owned camp is located on the banks of the Hope Canal approximately 1600-ft south of Interstate 10. The camp's dock extends into the canal. Minor local adjustments to the Conveyance Channel alignment will have to be made to accommodate this individual property owner. CPRA is just beginning to perform the Land Rights work;, the detailed requirements affecting the Conveyance Channel design will follow in future submittals.

As shown in **Figures 6.4 and 6.5**, due to the addition of the WSLP features, the cross-section and required ROW of the Project has grown considerably from the original Maurepas Diversion alone. Previously, the Maurepas Diversion was able to easily fit within a 300-ft ROW; the width of the combined Project is much greater. The ROW width varies along the Project alignment, depending on whether there are: new drainage ditches on each side; the WSLP flood protection features are a levee, floodwall, or floodgate; or if just the Conveyance Channel is being constructed in the swamp.

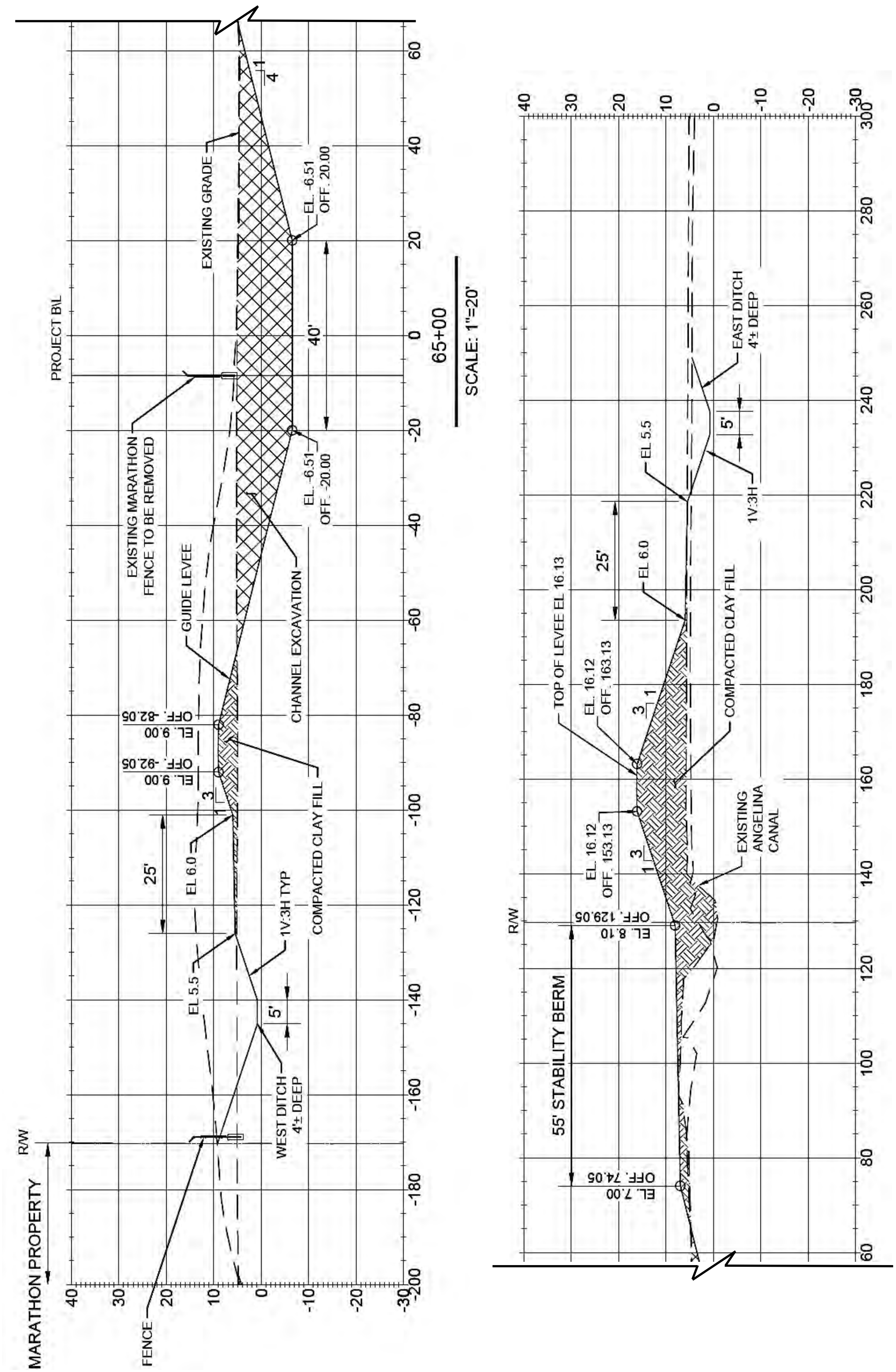


**Figure 6.4 – Cross-Section at Sta. 65+00 Showing Ditches**

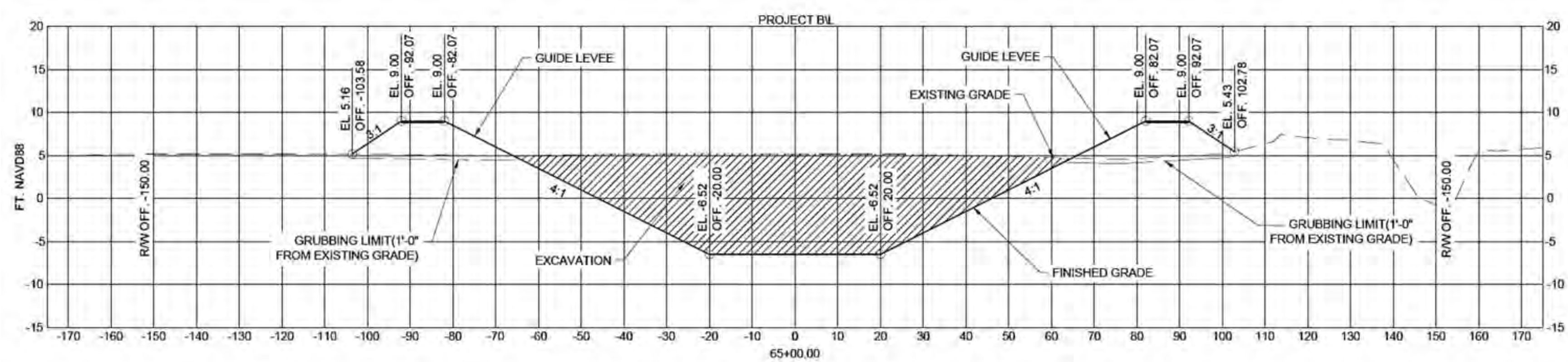


**Figure 6.5 – 2013 Maurepas Cross-Section at Sta. 65+00**

### 6.6.3 Alternatives Analysis

Two alternatives were considered for the alignment on the north side of CN RR adjacent to the Marathon Terminal Facility: 1) Leave the alignment as it is currently laid out, and 2) Shift the alignment eastward to accommodate a new drainage ditch on the west side of the ROW. Based on AECOM's H&H analysis, the ditch that runs along the south side of the CN RR conveys nearly 167 cubic feet per sec (cfs) during the 10-year storm. This represents by far the bulk of the drainage coming from the west side; the ditch on the north side of the railroad conveys a comparatively small volume of flow.

Alternative 1, leaving the alignment as is, requires a utility penetration of the proposed WSLP floodwall that crosses the south railroad ditch, as it is impeding the ditch's eastward flow. AECOM determined that the most viable method of achieving this was the installation of a sluice gated drainage structure through the floodwall. The installation of such a drainage structure would add cost and complexity to the project and would also create an additional operations and maintenance (O&M) burden, requiring periodic inspection, lubrication, and operation. Most importantly, with a storm surge approaching the WSLP flood protection system, the sluice gate would be closed along with all other system elements. The closure could be required days before the actual storm hits, while significant rainfall is occurring. When the floodgate is closed, the flow in the ditch would have no outlet. It would back up, inundating more and more property to the west as the storm progressed. This flooding was deemed unacceptable; therefore this alternative was eliminated from further consideration.

### 6.6.4 Recommended Design and Next Steps

The Recommended Design includes shifting the alignment: 1) eastward 60-ft at the CN RR crossing, and 2) westward around the enlarged USACE Pump Station Complex. These changes have been incorporated into the current design plans. The required adjustment to the alignment at the camp along Hope Canal south of I-10 will be addressed in the next phase of design.

The recommended adjustments to the channel alignment also need to be incorporated into the hydraulic model of the Conveyance Channel. Analyses supporting the changes in alignment at the CN RR (including the increased skew angle), shift at the pump station, and any other required adjustments will be provided in future submissions.

## 6.7 CN Railroad Crossing

### 6.7.1 Previous Design

The design of the CN RR crossing will follow the same concept as that submitted in the 2013 Design: RCBs cross under the tracks, connecting to the conveyance channel on each side. The process of arriving at the 2013 Design is briefly explained below.

AECOM (as URS) contacted Mr. Ed Baswell, CN RR Technical Services Engineer, to explain the proposed project, followed by a meeting in September 2007 at CN RR's office in Jackson, MS to discuss the project. The highlights of CN RR's requirements are summarized below:

- The RCBs must be cast-in-place; pre-cast structures will not be allowed.
- The crossing is close to switch gear and signal equipment. CN RR will relocate these elements 200-ft west to accommodate the culvert installation. After the culvert construction and track replacement, CN will replace the equipment to its original location.
- LDNR (now CPRA) will provide flagmen at all times during construction.
- The minimum distance requirement between tracks shall be 15-ft.

- The minimum distance from the base of rail to the top of the RCB shall be 3-ft.
- A permit from CN RR will be required. CN RR shall review their permit applications in-house. The review process will take 4 – 6 months

The proposed 2013 Design of the CN RR crossing was a 4,000-psi cast-in-place concrete structure 270-ft long, comprised of four 8-ft x 12-ft RCBs. Approximately three-hundred 50-ft long, 50-ton capacity, pre-stressed pre-cast concrete (PPC) piles were included in the foundation design. The invert elevation was set flat at EL (-) 7.25-ft NAVD88, with design WSEs of EL +7-ft NAVD88 and EL +6.3-ft NAVD88 upstream and downstream, respectively. A 100-ft long riprap armored transition was designed for erosion control both upstream and downstream of the culverts.

Earthen guide levees were designed at EL +9-ft NAVD88 and EL +8-ft NAVD88 upstream and downstream, respectively, to contain the water because the WSEs in the channel are higher than the existing grade. The internal wetted slopes of the channel were set at 4H:1V and the exterior guide levee side slopes were designed at 3H:1V to existing grade. The eastern levee toe on the south side of the railroad is set to land approximately 20-ft from the west ROW line, creating a buffer between the channel and the residential areas to the east.

A shoofly was designed to re-route train traffic during culvert construction. The shoofly runs on the north side of the existing tracks, starting approximately 1,025-ft east of the culvert centerline and extending to 1,150-ft west of the centerline, meeting back up with the existing lines just before the elevated section of the Marathon pipe bridge. The degree of curvature, radius, tangent distances, and other shoofly characteristics were designed to meet the American Railway Engineering and Maintenance-of-Way Association (AREMA) standards, 49 CFR Part 214, and those of CN RR.

### 6.7.2 Conceptual Plans for Additional Tracks

In March of 2020, CPRA and AECOM met with John Dinning, CN Manager Public Works, to discuss the proposed CN RR crossing. Mr. Dinning noted that CN RR has plans for a proposed fourth track on the south side of the ROW. This implied that CN RR already has plans for a third track on the north side of ROW, which had been noted on Pin Oak Terminals' plans but had previously not been formally acknowledged by CN RR. Mr. Dinning further indicated that he would supply the plans for the fourth track upon execution of a Non-Disclosure Agreement (NDA); AECOM and CPRA are in the process of coordinating with CNRR in obtaining the plans from Mr. Dinning.

The number of tracks impacts the design of three project elements: 1) the culverts of the Maurepas Diversion, 2) the floodgate of the WSLP Project, and 3) the construction sequencing of the Maurepas Diversion. The addition of tracks and increased skew of the culverts affects both the length of the RCBs and their structural and geotechnical integrity due to the additional loading generated. For each additional track, the RCBs and their pile foundation must be designed to support the load and impact forces induced by multiple trains crossing the boxes simultaneously; additional lines require the design of stronger boxes and a more robust pile foundation. For the purposes of the ROM cost estimate, preliminary calculations have been performed to approximate these increases and the Plans reflect these changes. Effects to the WSLP floodgate are discussed in Section 7.4.

### 6.7.3 Culvert Skew

As discussed in Section 6.6.3, the culverts crossing CN RR will be skewed at a greater angle to accommodate the eastward shift of the diversion alignment centerline. The shift is illustrated in **Figure 6.6**.

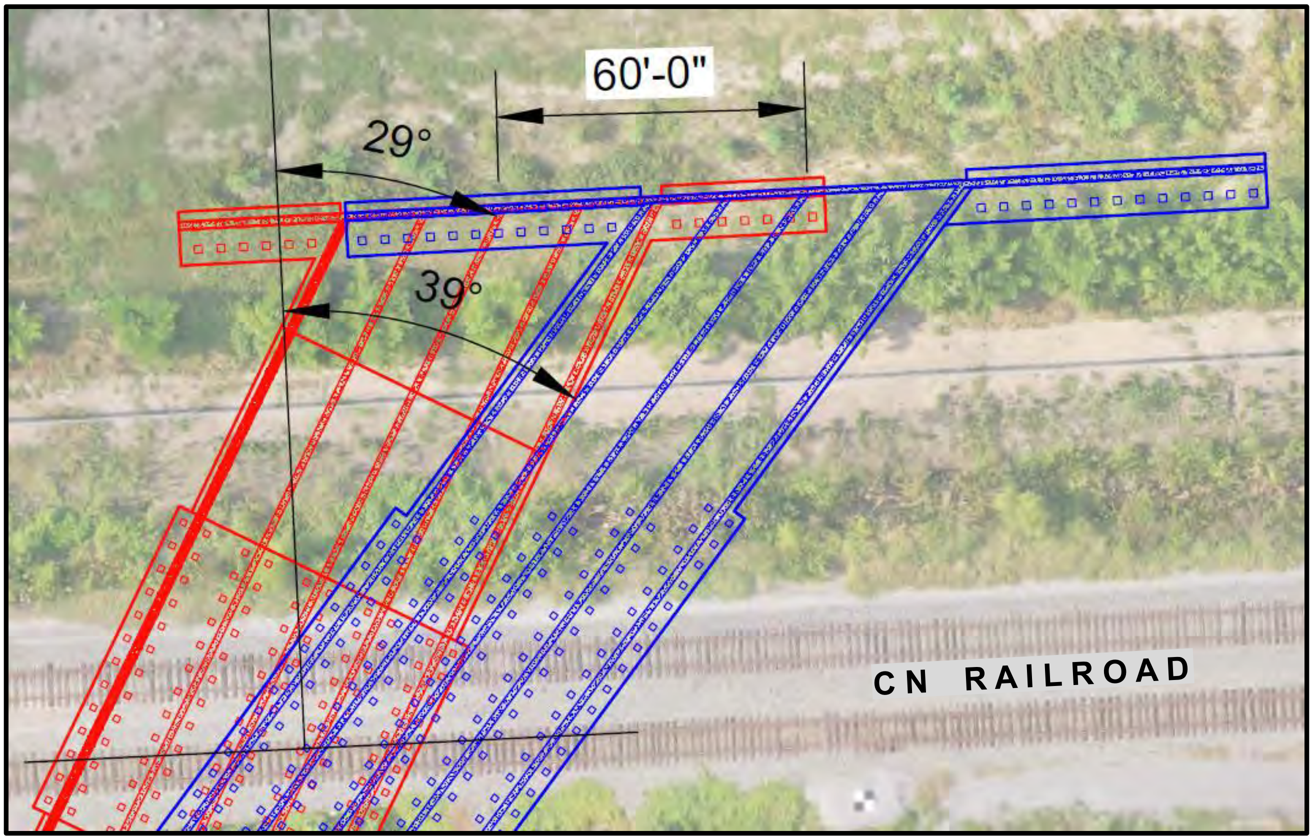


**Figure 6.6 – Culvert Skew Angles v\v 60-ft Shift of Alignment at CN RR**

Increasing the skew angle changes the entrance and exit headloss ($h_L$) coefficient, Ke. The entrance and exit headlosses are calculated by multiplying the coefficient by the velocity head through the culvert as follows:

Velocity head = $v^2/2g$,

where $v$ = velocity (ft/s²), and

$g$ = acceleration due to gravity (32.2-ft/s²)

Thus, $h_L = Ke \cdot v^2/2g$

Since the culverts are multiple boxes, the walls in between the culverts channel the flow, lessening the effect of the skew angle on the headloss. The velocity through the culverts is calculated by dividing the flow through each culvert (Q = ¼ • 2,000-ft³/s = 500-ft³/s per culvert) by the cross-sectional area of the culvert (A = 8-ft • 12-ft = 96-ft²), thus, the velocity, v = 500-ft³/s / 96 ft² = 5.2-ft/s and the Velocity head = (5.2-ft/s)² / (2 • 32.2-ft/s²) = 0.42-ft .

**Table 6.1** presents the resulting $h_L$ for various culvert skew angles, for an 8-ft • 12-ft box culvert with a Velocity head of 0.42-ft.

***Table 6.1 – Headloss in 8-ft • 12-ft box culvert with velocity = 5.2 ft/s***

| Culvert Skew (Angle) | $h_L$ Coeff. Ke (--) | Velocity Head $v^2/2g$ (ft) | $h_L$ (ft) |
|---|---|---|---|
| 0° | 0.35 | 0.42 | 0.147 |
| 15° | 0.36 | 0.42 | 0.152 |
| 30° | 0.44 | 0.42 | 0.185 |
| 45° | 0.46 | 0.42 | 0.194 |

As the table shows, changing the skew angle by as much as 45° only increases the $h_L$ by less than 0.05-ft, a very small difference. The proposed change is much more modest, only a 10° change from 29° to 39°, which results in < 0.005-ft increase in $h_L$, an effect which is negligible.

### 6.7.4 Alternatives Analysis

There are several levels of alternatives to be considered at the CN RR crossing.

The highest level would be whether to have the tracks cross over the conveyance channel (e.g. using a bridge) or to have the channel cross under the tracks (e.g. using culverts). Several factors indicate that a culvert crossing is preferable.

- First, there are currently two tracks in the area of the diversion crossing, which would require the construction of a bridge wide enough and strong enough to carry two rail lines.
- Second, Pin Oak Terminal's site plan showed a third track on the north side. While the current site plan from Marathon does not show any rail lines at the terminal, discussions with Mike Woess, the Marathon Terminal Manager, indicate they are currently focusing on their marine facilities but they do intend on connecting to the CN RR in the future. John Dinning's comments with regard to CN RR activities further support the likelihood this line will be added. This considerably complicates a bridge crossing, since the Marathon Terminal facilities are at grade. Constructing a bridge to convey two tracks over the open diversion channel and culverts to go under the third line would add significant expense.
- Third, John Dinning of CN RR has indicated that they have conceptual plans for a fourth track on the south side. While AECOM has not seen these plans, given Mr. Dinning's declaration, it is probable that CN RR would not approve a concept that did not provide for four operable lines. As another data point, Marathon has raised its pipe-bridge, which carries over a dozen petroleum product lines (and is therefore quite expensive to raise), to clear the entire 100-ft CN RR ROW, as shown in **Figure 6.7**.
- Finally, it is not known whether the southern track would be required at grade similar to the Marathon spur or elevated. If the track were to go on the bridge, the structure carrying three rail lines would be quite substantial. Regardless, culverts would still be required beneath the spur line to the Marathon Terminal.

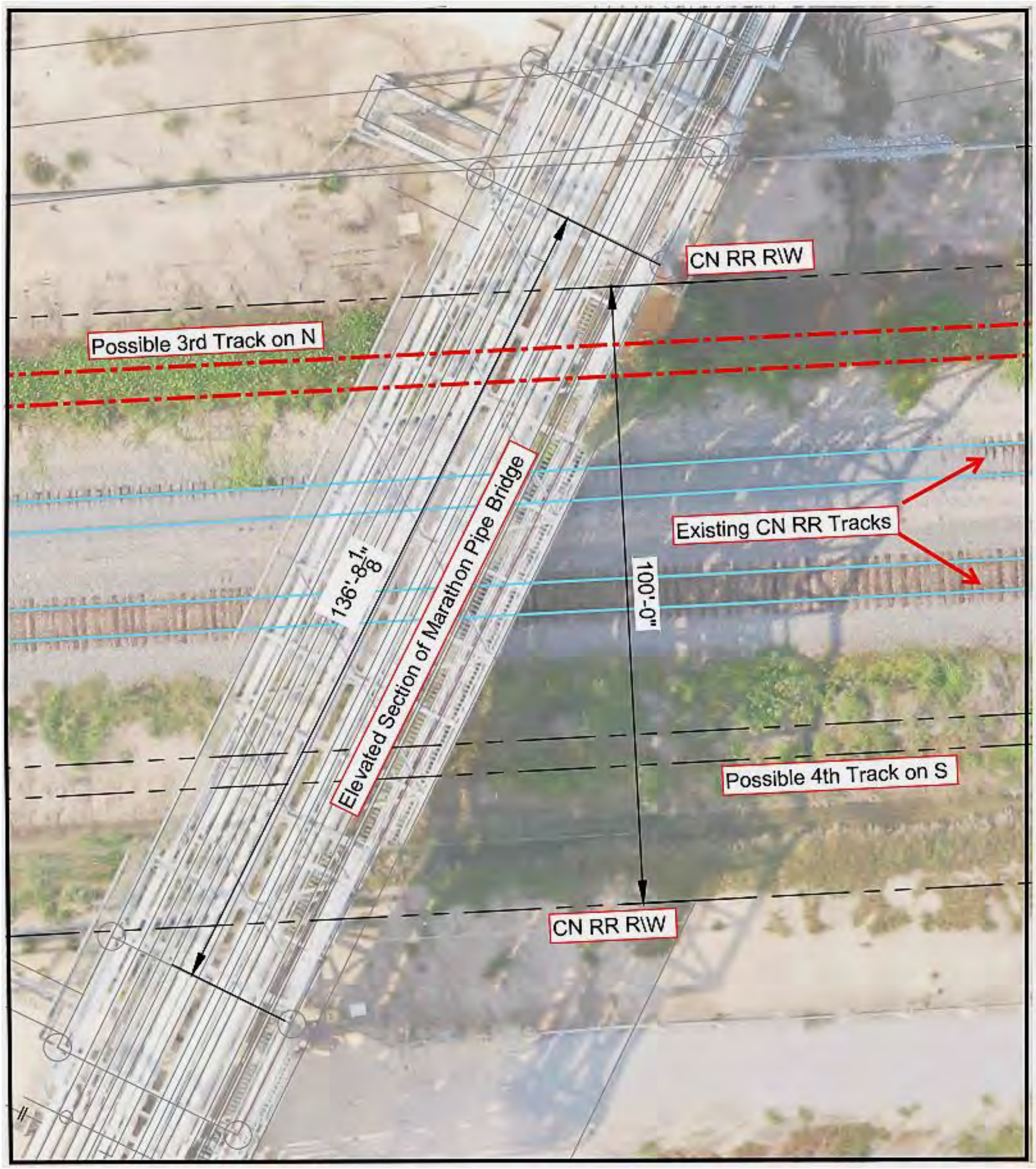


**Figure 6.7 – Marathon Pipe Bridge Crossing of CN RR**

Based on the above analysis, the only feasible alternative is to design the diversion to go underneath the tracks, whether there are two, three or four. Assuming the four track scenario is the most likely to be constructed in the future, it is prudent to design the culverts with sufficient length to span four rail lines.

On a much more detailed level, AECOM has reviewed the shoofly layout that enables construction of the culverts. Fortunately, there is plenty of area to construct the shoofly as designed in the 2013

Submittal. However, the current view is that the proposed arrangement provides more space for construction than necessary. In fact, it may be possible to construct the shoofly entirely within the CN RR ROW if the Maurepas-WSLP construction occurs before the additional railroad track on the north side is installed. This option would reduce the length of track, construction time, and cost; it will be explored in the subsequent phases of design.

### 6.7.5 Recommended Design and Next Steps

The Recommended Design of the CN RR crossing will follow the same concept as that submitted in the 2013 Design: RCBs crossing under the tracks, connecting to the conveyance channel on each side. The culverts will be skewed by leaving the headwall on the south side of the tracks in its current location and shifting the northern side 60-ft to the east. A shoofly north of the current track location will be used to facilitate construction.

AECOM will continue to support CPRA in correspondence and negotiations with Mr. Dinning and CN RR. In addition, AECOM will support USACE engagement to ensure CN's plans for a third and fourth line meet both the USACE's approval and acceptance of the associated cost burden. CPRA has provided guidance that, until more CN documents are provided, designs shall assume four active lines at this crossing. The Plans, calculations, and ROM costs provided with this report follow this assumption; these documents will be updated as needed when all parties have reached a final agreement.

The concrete culverts running beneath all current and future lines will be updated as needed to meet current USACE, CN and industry guidelines. Culverts and their foundations will require new analyses to account for the proper number of lines, their ballast and soil cover, change in skew angle, and any changes in condition presented by the survey information collected during this Task Order.

The shoofly design will largely remain as-is; however, its alignment may be shifted closer to the existing tracks in a subsequent design phase.

## 6.8 KCS Railroad Crossing

### 6.8.1 Previous Design

For the Preliminary Design, AECOM contacted Mr. Sri Honnur, Director of Track and Bridge Construction, for the KCS RR to explain the Maurepas Diversion project and present a proposed railroad crossing consisting of four 8-ft x 12-ft RCBs. Mr. Honnur indicated that KCS RR did not want culverts installed because they required the construction of a shoofly, which would create a prolonged outage and/or slow-down of the rail traffic. He stated that KCS RR would require a bridge to be installed over their tracks.

KCS RR employs specialty third party contractors that regularly construct their standard pre-fabricated bridges within one or two days. Much of the substructure of the bridge is constructed outside of the safe operating zone of the tracks prior to the bridge being placed, allowing rail traffic to continue operation. Next, superstructure components (e.g. pile caps, girders with cross-bracing) are assembled into units which are lifted into place and secured. Excavation of the conveyance channel area underneath the installed bridge would be conducted subsequently, with rail operations resumed.

AECOM designed and submitted a bridge crossing in the 2013 Design. The bridge had five spans with 20-ft span lengths, and 20-in (1.67-ft) beam depths. There was a 1-ft clearance between the maximum WSE in the conveyance channel and the bottom chord of the bridge. This required raising the track from the current elevation of 8.70-ft to 9.85-ft, a raise of 1.15-ft, at the crossing. The following shows the calculations from the 2013 Submittal (note that these values may require updates based on recently collected survey data):

Setting the bottom cord of the beam at EL 6.35-ft NAVD88
+ 1.67-ft for Beam Depth
+ 1.33-ft for Ballast & Timber
+ 0.50-ft for Rail Height

Proposed Top of Rail = EL 9.85-ft NAVD88
– Existing Top of Rail at EL 8.70-ft NAVD88

Rail Raise = 1.15-ft

Bottom Chord Elevation = EL 6.35-ft NAVD88
– Maximum WSE at EL 5.35-ft NAVD88

Clearance = 1.00-ft

AECOM noted that the diversion is designed to convey a maximum of 2,000-cfs with flow is controlled by the operation of the gates. At high river stages and low swamp water levels, the gates would be partially closed so that the maximum WSE in the channel at the KCS crossing would be 5.35-ft NAVD88. Thus, the water surface in the channel would never exceed the 1-ft clearance from the bottom chord of the bridge.

### 6.8.2 KCS RR Design Requirements

All design of railroad components must meet the requirements of AREMA, 49 CFR Part 214, as well as the specific standards of KCS RR. Per KCS, for the vertical curves on a main line track, the maximum allowable change in gradient per 100-ft station (V/L) shall be 0.10 for Summit curves and 0.06 for Sag curves. KCS stated that the minimum tangent distance between vertical curves shall be 100-ft. A

### 6.8.3 Alternatives Considered

Three alternatives were considered for the KCS RR crossing: 1) box culverts with a shoofly, 2) a bridge over the Maurepas Diversion conveyance channel, and 3) a bridge that spans both the diversion channel and the WSLP flood protection levee. Each of these is discussed below.

#### 6.8.3.1 Box Culverts with a Shoofly

Early on, in the Preliminary Design, AECOM proposed a railroad crossing consisting of four 8-ft x 12-ft RCBs. One of the key benefits to this design was that the guide levees on each side of the railroad could be wrapped around the ends of the conveyance channel. In this configuration, the guide levees would not obstruct the railroad drainage ditches. And since the top of the culverts would be below the bottom of the ditches, the existing drainage patterns would remain undisturbed.

To install the RCBs, a shoofly would have to be constructed to maintain rail traffic while the track over the culverts was removed and the culverts installed. KCS' existing speed limit along this section of track is 60-mph. If a shoofly is constructed, its geometry must be such that a minimum of 35-40-mph can be maintained. As described in Section 6.8.1, KCS required a bridge crossing, so the concept of installing the culverts was not pursued further.

#### 6.8.3.2 Bridge over Diversion

As Section 6.8.1 describes, a bridge over the Maurepas Diversion conveyance channel was designed for the 2013 Submittal. With KCS rejecting the culvert and shoofly option, this was the only viable alternative when the project consisted solely of the Maurepas Diversion. The addition of WSLP flood protection features to the project presented a third option, that of a bridge spanning both the diversion channel and the levee, as discussed next.

#### 6.8.3.3 Both Diversion & Levee

For the combined Project, a third alternative was to design a bridge that would span both the conveyance channel and WSLP protection approximately 100-ft away. This alternative would require more than doubling the length of the bridge, and more than double the cost, due to the longer and more robust structural members required. Installing a longer bridge to span the WSLP levee or floodwall as well as the conveyance channel would also require that LA 54 be elevated, a costly and undesirable change.

Based on other project cost estimates, the proposed bridge is projected to cost $15 million. The costs have been estimated by the construction cost on previous projects, but have not been prepared by an estimator. Therefore, the costs are subject to refinement in the next two weeks.

Since the longer bridge was infeasible due to its cost, WSLP flood protection was incorporated into the project via a floodgate crossing the tracks about 100-ft east of the KCS bridge. Additional information on the WSLP KCS crossing is found in Section 7.5.

On March 17, 2020 KCS, CPRA, AECOM, and Larry Marino (CPRA's liaison with the railroads) held a conference call to discuss the KCS RR crossing. AECOM presented a preliminary layout of the Maurepas Diversion conveyance channel, the WSLP flood protection features, and the KCS RR bridge option at the crossing. The layout of the diversion and the bridge crossing had not changed from the 2013 Design. However, the design presented also showed the proposed WSLP flood protection levee, flood wall, and flood gate across the KCS RR about 100-ft east of the bridge.

While the bridge work can be coordinated to create only a one to two day shut-down of operations, per KCS, raising the track the 1.15-ft required would take longer. KCS estimated that raising the length of track shown (using the on-rail machine that installs and compacts additional ballast) would take about 3 days for every 4-inches raised, or about 10 days total. The track would be raised first by coordinating around several rail outage intervals. Next, the substructure of the bridge would be installed, which would not impact rail operations. Finally, the bridge itself would be installed and the track replaced, returning the line to service.

KCS noted that the track bed shown on the proposed cross-section was not wide enough; iIt must be a minimum of 28-ft at the top of the sub-ballast (EL +9.26-ft NAVD88). Thus, more earthwork using cohesive fill material would be required to raise the track. With the relatively flat rail grade requirements, the bridge as designed (without spanning the WSLP flood gate) would meet existing grade on the east side just before the intersection with LA 54.

In December 2020, AECOM discussed the design with Michael Schmidt, KCS Director of Bridge Maintenance and Mark Lindenmeyer, KCS Manager of Public Projects. After a discussion of the proposed 2013 bridge design concept, the KCS personnel indicated that the length of outage required to raise the track the required 1.15-ft is longer than they would prefer if other options are available to them. They indicated that a shoofly to re-route rail traffic during construction is preferable. Subsequent conversation with Garrett Cross, KCS Structures Department Lead, indicated that as long as rail traffic is redirected via a shoofly, either the culvert or bridge option would be viable to KCS.

### 6.8.4 Recommended Design and Next Steps

Additional coordination is required with KCS RR before a formal Recommended Design can be submitted. At this time the bridge concept developed in the 2013 Design is presented, as this is the most recent formal request made by the railroad. AECOM will continue to provide support to CPRA in its negotiation with KCS, and suggests developing concepts for a box culvert crossing and a shoofly bypass during the next design phase for use in future discussions with KCS Railroad. AECOM would then compare the cost of a bridge plus shoofly to that of a box culvert plus shoofly. Full design of the components and development of a construction sequencing plan will follow once the basic arrangement is agreed upon. All new designs will be coordinated with the WSLP KCS Crossing as needed.

## 6.9 Airline Highway (US 61) Crossing

### 6.9.1 Previous Design

In the Preliminary Design of the Maurepas Diversion, AECOM (as URS), in conjunction with Evans-Graves Engr. (E-G), met in August 2007 at the DOTD District 62 office in Hammond to discuss the requirements for crossing Airline Highway. E-G submitted a proposed 375-ft long, five-barrel, 9-ft x 9-ft, DOTD Standard No. CCSM-7-10-S-90°-1 RCB roadway crossing, which was a standard DOTD size at the time (DOTD no longer publishes standard culvert designs; culverts must now be individually designed using LRFD methodology). The RCB was proposed as a precast or cast-in-place 4,000-psi reinforced concrete structure, supported by an approximately 540 - 50-ft long, 50-ton capacity PPC piles. DOTD indicated that a Driveway Permit would be required by the Hydraulics and Right-of-Way Departments and they would provide a new pavement design section for the replacement of the roadway after installation of the RCB.

For the 2013 Submittal, the design was modified to include a sixth box to minimize headloss and maintain upstream freeboard. The channel design WSEs were set at EL +5.3-ft NAVD88 and EL +4.6-ft NAVD88 upstream and downstream, respectively. The guide levees were wrapped around the channel and the tops were set at EL +7-ft NAVD88 and +6.5-ft NAVD88 upstream and downstream, respectively, to prevent flooding. To avoid conflicts with local drainage the culverts were designed to be below the bottom of the roadway drainage ditches; the RCB inverts were set at (-) 9.50-ft NAVD88.

The revised culvert configuration adjoined a 60-ft wide channel bottom with internal side slopes of 4H:1V on the south (upstream) side and 5H:1V on the north (downstream) of Airline Highway. Since the six-box RCB structure is wider and deeper than the diversion channel bottom, a 100-ft long rip-rapped transition section was designed both upstream and downstream, rising on each side to EL (-) 7-ft NAVD88. The levee outside side slopes were designed at 3H:1V, sloping to existing grade.

#### 6.9.1.1 Construction Sequencing

Full traffic flow, with two travel lanes in each direction must be maintained at all times during hurricane season (June – November) per the DOTD. Periodic short term one lane closures will be acceptable outside of hurricane season. Construction of the crossing was proposed in two phases; each phase includes constructing temporary crossover lanes at both ends of the project to detour north-bound (NB) / south-bound (SB) traffic to the opposite side of the existing median.

During Phase 1, the existing shoulder of the EB lanes will be widened by 14-ft creating a total paved section on the EB side of 51-ft. Concrete barriers will be placed on 3-ft of the median side pavement and in the middle 4-ft between two 11-ft lanes traveling in each direction. Sheet piling shall be driven to support the roadway during construction of the RCB. Once the traffic lanes have been constructed and the sheet piling is in place, the first half of the box culvert, including its headwall and containment levee, shall be constructed.

During Phase 2, temporary crossover lanes will be constructed to reroute traffic to the opposite side of the median, and the Phase 1 temporary crossover lanes will be removed. An additional 12-ft of pavement will be installed on the median side of the existing WB lanes, providing another 51-ft wide section of continuous pavement on the opposite side from Phase 1. The same concept will then be applied to route two 11-ft lanes of traffic in each direction, by placing a concrete barrier on the 3-ft of pavement on the median side and a barrier within the middle 4-ft, separating the lanes. Once the traffic lane re-configuration is completed, the second half of the box culvert, headwall, and containment levee shall be constructed.

### 6.9.2 Alternatives Analysis

The Preliminary and 2013 Designs did not involve revision of the Airline Highway roadway geometry because the design was only for the Maurepas Diversion; there were no elements of the WSLP Project involved. For the current design, to incorporate the flood protection elements of the WSLP Project, three alternatives were considered: 1) installing a floodgate across the roadway, 2) raising the road to the 2070 Design elevation of EL +16-ft NAVD88, and 3) constructing a bridge over the WSLP levee or floodwall.

#### 6.9.2.1 Alternative 1 - Floodgate

Installing a floodgate across Airline Highway would be an effective means of extending the line of protection between the levee reaches on either side of the roadway. However, it was removed from consideration because the roadway serves as a hurricane evacuation route and therefore cannot be closed during a major storm event.

#### 6.9.2.2 Alternative 2 – Raising the Roadway

Airline Highway is state roadway classified as an Urban Principal Arterial. It is a 4-lane divided highway with a 30-ft wide median which runs east-west across the project corridor. The Maurepas Diversion elements, the WSLP flood protection features, and the culverts of the re-routed drainage ditches all run from south to north. Thus, they all must cross the roadway as they travel north.

There are numerous aspects to be considered in the raising of Airline Highway: adhering to the roadway design criteria; maintaining access to adjacent residential and commercial properties; selecting the type of retaining system to support the road; minimizing the construction easements needed; and determining the impact on the Maurepas Diversion RCBs; among others.

The first decision required is the selection of the Design Elevation to which the flood protection levee should be built. The levee underneath a roadway is unlike other reaches of levee, which can be built to an initial grade and later raised, perhaps in multiple lifts, to the final Design Elevation. Re-constructing a roadway multiple times is not practical; in that sense it is considered a hardened structure, which should be constructed to the final Design Elevation only once. Thus, the levee underneath Airline Highway has been designed for initial construction to the maximum 2070 Design Elevation of +16-ft NAVD88, which will provide full flood protection throughout the life of the project.

Next, the applicable roadway design criteria must be assigned. While coordination with DOTD is ongoing, for this 15% Design several assumptions had to be made. **Table 6.2** summarizes the applicable geometric criteria used to design the raised roadway of Alternative 2. The assumptions inherent in the table are discussed below.

***Table 6.2 – Basic Design Criteria for Airline Highway***

| | | | |
|---|---|---|---|
| **Posted Speed (mph)** | | 65 | |
| **Lane Width (ft)** | | 12 | |
| **Shoulder Width (ft) (Inside)** | | 4 | |
| **Shoulder Width (ft) (Outside)** | | 8 | |
| **Min. Lateral Offset (ft)** | | 4 | |
| **Max. Longitudinal Grade (%)** | | Preferred | 3 |
| | | Acceptable | 5 |
| **Fore Slopes** | | Preferred | 6:1 |
| | | Acceptable | 4:1 |
| **Back Slopes** | | Preferred | 4:1 |
| | | Acceptable | 3:1 |
| **Vertical Curve Criteria** | **Crest** | $LVC_{MIN}$ (ft) | 1056 |
| | | $SSD_{CREST}$ (ft) | 645 |
| | | $K_{MIN}$ | 193 |
| | **SAG** | $LVC_{MIN}$ (ft) | 477 |
| | | $SSD_{SAG}$ (ft) | 645 |
| | | $K_{MIN}$ | 157 |
| **Lane Width Taper Length (ft)** | | 780 | |
| **ADT (vpd) (2017)** | | 20,755 | |

The first assumption is the speed for which the roadway is to be designed. As listed in **Table 6.2**, the subject stretch of Airline Highway is posted for 65 mph. Typically, a roadway is designed for 10 mph above the Posted Speed, which increases the intrinsic factor of safety in the design. The previous Posted Speed for Airline Highway was 55 mph, which would typically correspond to a Design Speed of 65 mph. With the increase in Average Daily Traffic (ADT) volume over time, the need arose to raise the Posted Speed. The assumption has been made that the raised roadway can be designed based on the 65 mph criteria, which may either be posted at 55 mph or 65 mph, at DOTD's discretion.

As the table shows, the lane and shoulder widths, lateral offset, and lane width taper length are definitively set by the speed. Other factors, such as the maximum grade, fore slopes, and back slopes have preferred and acceptable criteria. As shown, the preferred maximum longitudinal grade is 3%; however, a grade as steep as 5% may be acceptable. The Current Design maintains the longitudinal grade below the 3% maximum criterion which may create a longer roadway raise than the acceptable 5% grade; however, the reduction in length due to a steeper grade is offset by the longer vertical curve required.

The vertical curve criteria are set based on the greater of three times the Design Speed or a function of the $K_{MIN}$ value. The K value is defined as the horizontal distance along which a 1% change in grade occurs on a vertical curve (i.e., it expresses the abruptness of the grade change). For the subject application the $K_{MIN}$ value governs the required length of vertical curve for both the Crest and Sag conditions. Steepening the longitudinal grades (on each side of the vertical curve) will, in turn affect the length of vertical curve (LVC); the trade-off between the two will be optimized during the subsequent design phases, with input from DOTD.

The preferred fore slope, adjacent to the roadway shoulder, is 1V:6H; however, a fore slope of 1V:4H may be acceptable. Similarly, the back slope, the slope beyond the fore slope required to return to existing grade (if needed) is preferred to be 1V:4H, though 1V:3H may be acceptable. In the current design, the acceptable fore slope of 1V:4H has been selected over the preferred 1V:6H slope to minimize the impact to adjacent properties. Another approach, which uses an overlapping geotextile to create a mechanically stabilized earthen (MSE) wall is also under consideration and will be explored during subsequent design efforts. Use of an MSE wall would require the installation of guardrails on the sides of the roadway, but their additional width of only 2-ft would be more than offset by the reduction in width from grading down at a 1V:4H horizontal slope.

The Current Design thus represents a reasonably conservative assumption given the unknowns present. Upon future discussion with DOTD the grades may be revised to minimize the length and height of the raise, especially if the agency determines that the Posted Speed can be reduced to 55 mph.

Another option investigated during the Current Design is the reduction in the median width by using a median barrier and bringing the travel lanes closer together. **Figures 6.8 and 6.9** illustrate the concept. Such an approach could reduce the existing 30-ft median width to only 10-ft, reducing the impact to adjacent properties by 10-ft on both sides of the roadway. This may become the preferred option in the subsequent design, as discussion with DOTD may indicate the reduction in impacts beyond the existing ROW are critically important.

#### 6.9.2.3 Alternative 3 – Constructing a Bridge

The third alternative is to construct a bridge over the WSLP flood protection and keep the Maurepas Diversion crossing roughly the same as the 2013 Design. Depending on the geometric requirements of a bridge, the WSLP flood protection may be either a levee section or a floodwall. In either case, the flood protection feature would be designed to the full 2070 Design Elevation of +16-ft NAVD88 because it would be impractical (not impossible, but certainly more costly) to raise the level of protection underneath a roadway bridge.

It would initially appear that a shorter bridge could be installed over a floodwall than a much wider levee section. However, the same geometric criteria listed in **Table 6.2** would apply to the roadway traversing the bridge, meaning that the LVC for the crest would still be over 1000-ft. The levee footprint becomes irrelevant, as the bridge would extend well beyond the levee bottom width regardless of what flood protection measure is beneath. Since a levee section is more economical that a floodwall, and either one would be constructed to the 2070 Design Elevation, there would be no advantage in constructing a floodwall. Thus, the flood protection adjacent to and under the bridge would be provided by a levee.

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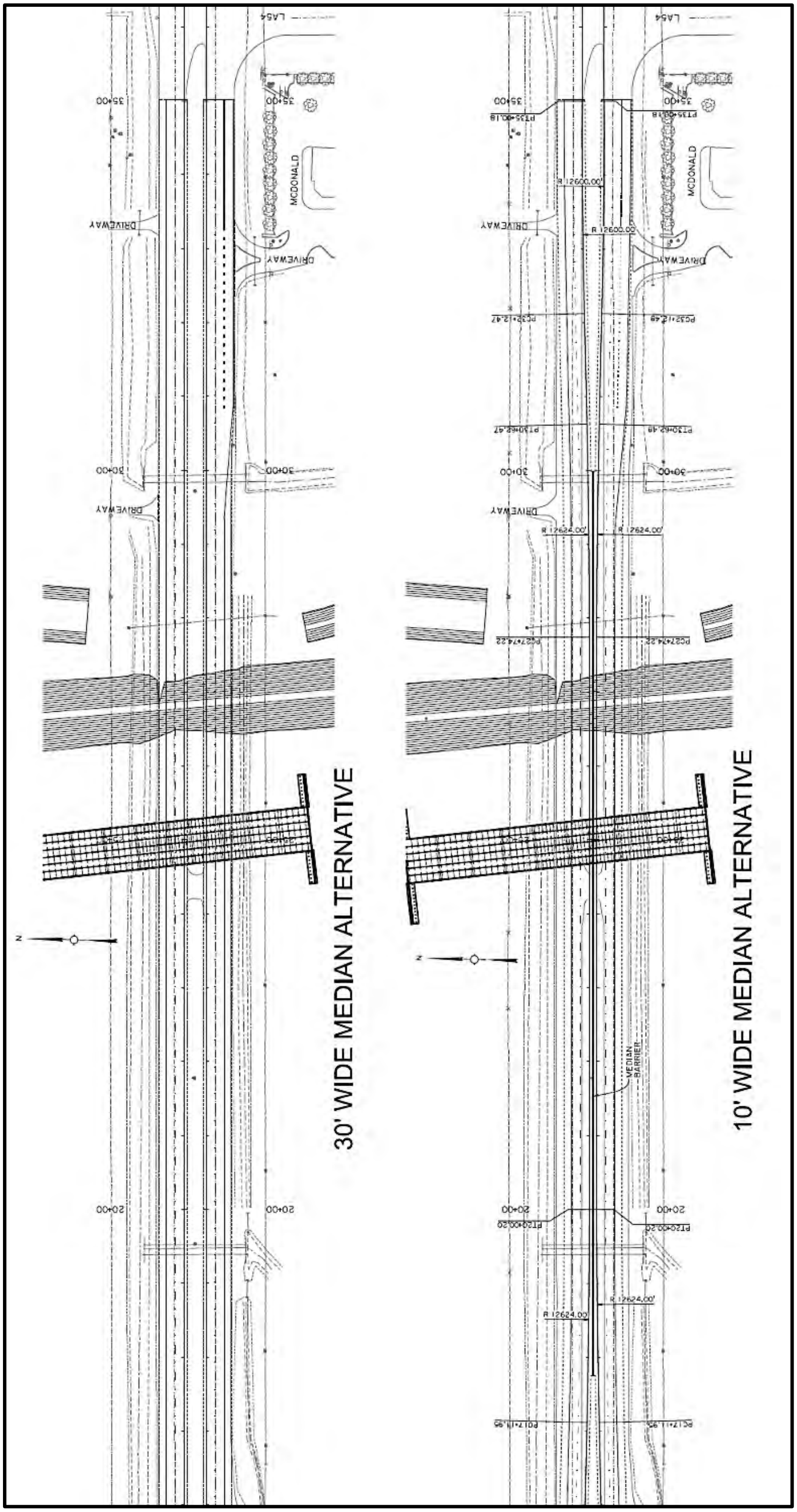


**Figure 6.8 – Plan View of Alternative Median Widths**

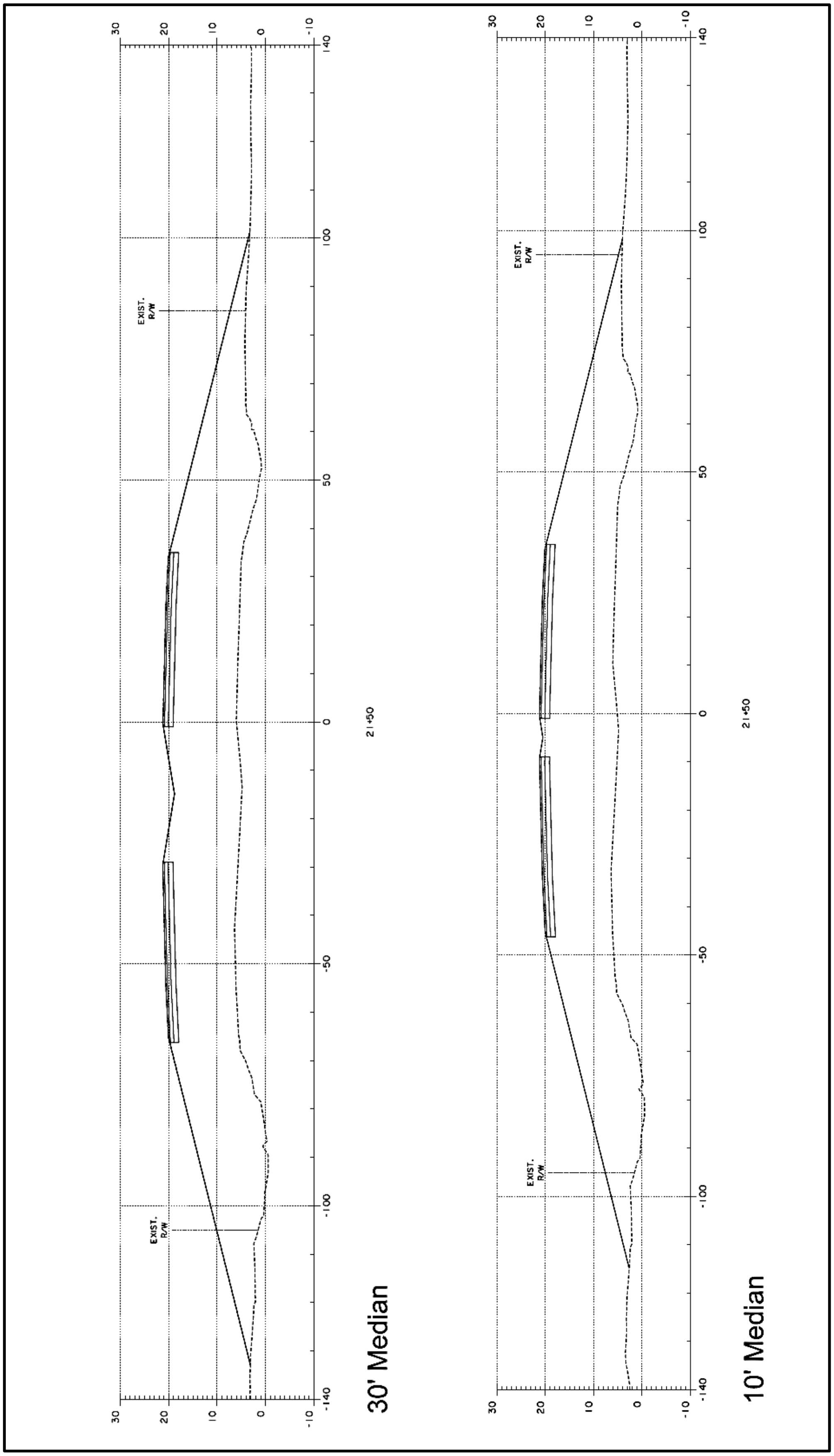


**Figure 6.9 – Section View of Alternative Median Widths**

#### 6.9.2.4 Alternatives Comparison

The design decision is a cost comparison between raising the roadway by placement of an embankment versus that of raising it with a bridge. The cost of raising the roadway by embankment placement includes: the acquisition, placement and compaction of the additional fill; the installation of additional roadway section due to the increased length of the curved pathway; the additional reinforcement in the box culverts to provide sufficient rigidity; and the increase in the pile foundation requirements due to the increased load of the fill. The costs of raising the roadway by constructing a bridge include: the approach slabs on both ends; the bridge foundation on both ends; intermediate bents, as needed; and the superstructure elements required to support the spans.

Based on similar projects such as LPV 109.02b (I-10 crossing of the HSDRRS system in New Orleans east) and the Mid-Barataria LA-23 Highway Bridge estimate, AECOM developed a Rough-Order-of-Magnitude ROM base cost estimate for constructing a bridge. Excluding escalation, contingency, changes and claims, resident inspection, and engineering services during construction, the raw construction cost for a bridge was calculated as approximately $12 million. The comparable ROM construction cost (without the additional items) for raising the roadway is approximately $5½ million. Thus, the bridge alternative is far more expensive than the embankment alternative - a bridge would cost over twice as much to construct as the cost of raising the roadway. The raw construction costs (excluding escalation, contingency, changes and claims, resident inspection, and engineering services during construction) for the floodgate option is estimated to be less than $2½ million.

### 6.9.3 Recommended Design and Next Steps

The Recommended Design is to raise Airline Highway by embankment placement. The Design Speed is set at 65 mph, with longitudinal grades ≤ 3%, fore slopes of 4V:1H, crest vertical curves of 1056-ft with K ≥ 193 and sag vertical curves of 477-ft with K ≥ 157.

As discussed in the Alternatives Analysis, the Airline Highway roadway raise will be further developed in subsequent design efforts. The design of the embankment will be detailed and optimized with the aim of placing as little embankment fill as possible (and thus reducing costs) while providing a robust roadway and flood protection feature. Grades, the use of MSE construction, speed limit and ROW impact requirements, among other considerations will guide the future design efforts, as will continued input from DOTD and CPRA. The final roadway criteria, including the design speed, the optimum grades, the length of vertical curves, and the median widths, among other parameters, will be specified in the next phase of design.

The Conveyance Channel culverts will require complete re-design for the new embankment roadway conditions. For the purposes of the ROM cost estimates included with this Report, simple hand calculations have been performed to estimate likely changes to the PPC pile foundation and the culvert top slab due to the addition of over 10-ft of fill to the current design. This will be replaced with a full design of culvert members and pile foundation in future submissions.

Construction sequencing and structural requirements will also be re-examined in the next design phase. Adjustments will be made to accommodate the embankment alternative, such as accounting for a larger possible construction footprint and analyzing the TRS wall that separates the two construction phase sites for increased soil load.

<u>Update</u>: Depending on CPRA's upcoming discussion with DOTD in March 2021, concerning the possibility of constructing a floodgate, the bridge and embankment raise options may be dropped from further consideration. If the DOTD will allow the construction of a floodgate there would be a significant cost savings. Numerous other issues would be eliminated as well:

- Ground improvements to minimize settlement due to the additional overburden,
- Multiple raises on a levee under a bridge or raising the highway grade to the Design EL,
- The need to stiffen the diversion culverts and provide additional foundation support,
- The effects of expanding the roadway footprint, impacting adjacent properties,
- The provision of temporary drainage during the multi-phase construction process, and
- The extended during of construction.

## 6.10 Interstate 10 Crossing

### 6.10.1 Previous Design

No change has been made to the Maurepas Diversion channel at the Interstate 10 crossing since the 2013 Design. As shown on the plans submitted to CPRA in 2013, the Conveyance Channel will follow the Hope Canal centerline, crossing under the two existing I-10 bridges at stations 290+85 and 291+80. The geometry of the bridge crossings as designed was obtained from DOTD “As-Built” plans. Each bridge is 45-ft wide and they are spaced 49-ft 6-in apart. The original design section of the waterway under the bridge is a trapezoidal channel with revetment on side slopes of 3H:1V down to existing grade, which varies from EL (-) 5-ft to (-) 9-ft NAVD88. The invert along the 25-ft wide flat channel bottom was designed to be at (-) 8-ft NAVD88.

The existing I-10 bridges feature seven spans, five of which will be impacted by the Conveyance Channel: the two 20-ft wide outside spans and three 25-ft wide center spans. The existing 16-in square concrete piles are tipped at EL (-) 54-ft MSL ((-) 48.18-ft NAVD88) except for those supporting the wing walls, which are tipped at EL (-) 29.0-ft MSL ((-) 23.18-ft NAVD88). The overburden material down to EL (-) 12-ft MSL ((-) 6.18-ft NAVD88) is assumed to be muck. The bridge decks have an approximate top of roadway elevation of +15.0-ft NAVD88 and low chords clear elevation of (+)12-ft NAVD88.

### 6.10.2 Restoration of Original Cross-Section

As surveyed, the channels have silted in over time, depositing material on the bottom and side slopes, forming a more V-shaped cross-section. The intent of the Maurepas Diversion work is to restore the cross-section to its original design geometry by excavating the deposited materials. This should not impact the structural integrity of the bridge piers or the overall bridge.

The wider typical Conveyance Channel cross-section will transition to and from the bridge channel over a distance of 85-ft upstream of the eastbound bridge and 85-ft downstream of the westbound bridge. The guide levees of the diversion channel were set at EL +2.6-ft NAVD88. The levees were designed to extend into the roadway embankment to prevent ineffective flow areas.

### 6.10.3 Alternatives Analysis

The proposed plan for the Conveyance Channel crossing under I-10 has not changed since the 2013 submittal. Therefore, no Alternatives Analysis was performed under the current Scope of Work.

### 6.10.4 Recommended Design and Next Steps

The Recommended Design remains unchanged from the 2013 Design. It involves restoring the cross-section of Hope Canal underneath the two I-10 bridges to their original design configuration by the removal of deposited sediment and debris that has accumulated since the crossings were installed. No work will be performed that would undermine the integrity of either structure; removal of accumulated material around the bridge piers would only extend to the original design grade.

Ancillary details such as access points (both permanent and for onstruction) and traffic control will be reviewed in future phases.

## 6.11 Interstate 10 Check Valve Installation

### 6.11.1 Previous Design

The outlet for the conveyance channel is approximately 1,000-ft north of I-10, along the existing centerline of Hope Canal. The guide levee elevations from the I-10 bridges to the termination point gradually transition to existing grade. At that point the diverted water will overflow the canal banks and dissipate into the area above I-10, south of Lake Maurepas.

During the Preliminary Design, hydraulic modeling showed that some of the diverted water could backflow into the area south of I-10 because cross-drains traverse underneath the Interstate. Drainage within this area typically travels northward, so any backflow from the area north of I-10 could impede normal drainage patterns. Also, the design intent of the diversion is to provide much needed water and nutrients to the severely stressed areas of the swamp north of I-10. Short-circuiting of the discharged river water southward via the drainage culverts would reduce the effectiveness of the overall project. The installation of check valves on the culverts underneath the roadway was proposed to mitigate this potential problem.

### 6.11.2 Alternatives Analysis

An Alternatives Analysis was performed to select the most appropriate type of check valve. There are numerous check valves available for various types of service, the two primary choices for this application are metal flap gates and elastomeric valves that uncurl to open and re-curl to close.. The most important properties are low opening pressure, ability to create a positive closure against backflow, overall reliability, long service life, and low operation and maintenance (O&M) requirements.

The metal flap gates are heavy, requiring support to prevent sinking in the soft swampy soils below I-10. They also take significant pressure to open and are subject to clogging, creating an O&M burden. Since O&M is unlikely to be performed on a regular basis, they are not a good selection for the subject application.

Elastomeric check valves are flexible enough to open and allow discharge under very low differential head conditions, yet stiff enough to remain closed and prevent backflow. The pressure created on the valve exterior by reverse flow or submersion seals the lips tightly together, preventing flow into the culvert. Such valves are made to slip-on over the discharge end of an existing culvert and be securely fastened with a simple clamp, resulting in low installation costs.

Along this corridor of I-10 there are both round pipes and arched pipes (the locations are indicated on the 2013 plans). Elastomeric check valves are commercially available for round pipes by numerous manufacturers. In the years since the 2013 Design, the manufacture of such valves for arched pipes has become available from a couple of manufacturers. While relatively expensive, the arched valves are still less expensive than constructing the junction boxes with flap gate valves proposed in the earlier Preliminary design.

### 6.11.3 Recommended Design and Next Steps

The Recommended Design of the check valves for the culverts under I-10 remains unchanged from the 2013 Design. as described above, thus check valves were recommended.

Due to the current availability of elastomeric check valves for arched pipes, AECOM recommends the installation of elastomeric check valves on both the round and arched culverts under I-10.

## 6.12 Weirs at Bayou Secret & Bourgeois Canal

The purpose of the Maurepas Diversion is to restore the health and essential functions of the swamp. To accomplish that goal, the water must be dispersed throughout the swamp so that no areas are left dry whilst others are inundated with more water than is needed. The existing water flow throughout the swamp is conveyed overland as well as through natural and manmade channels. The two most significant channels are Bayou Secret and Bourgeois Canal, which flow from east to west and discharge into Blind River. Blind River, in turn, discharges into Lake Maurepas. **Figure 6.10** shows these waterbodies in relation to Blind River, I-10, and the Maurepas Outfall.

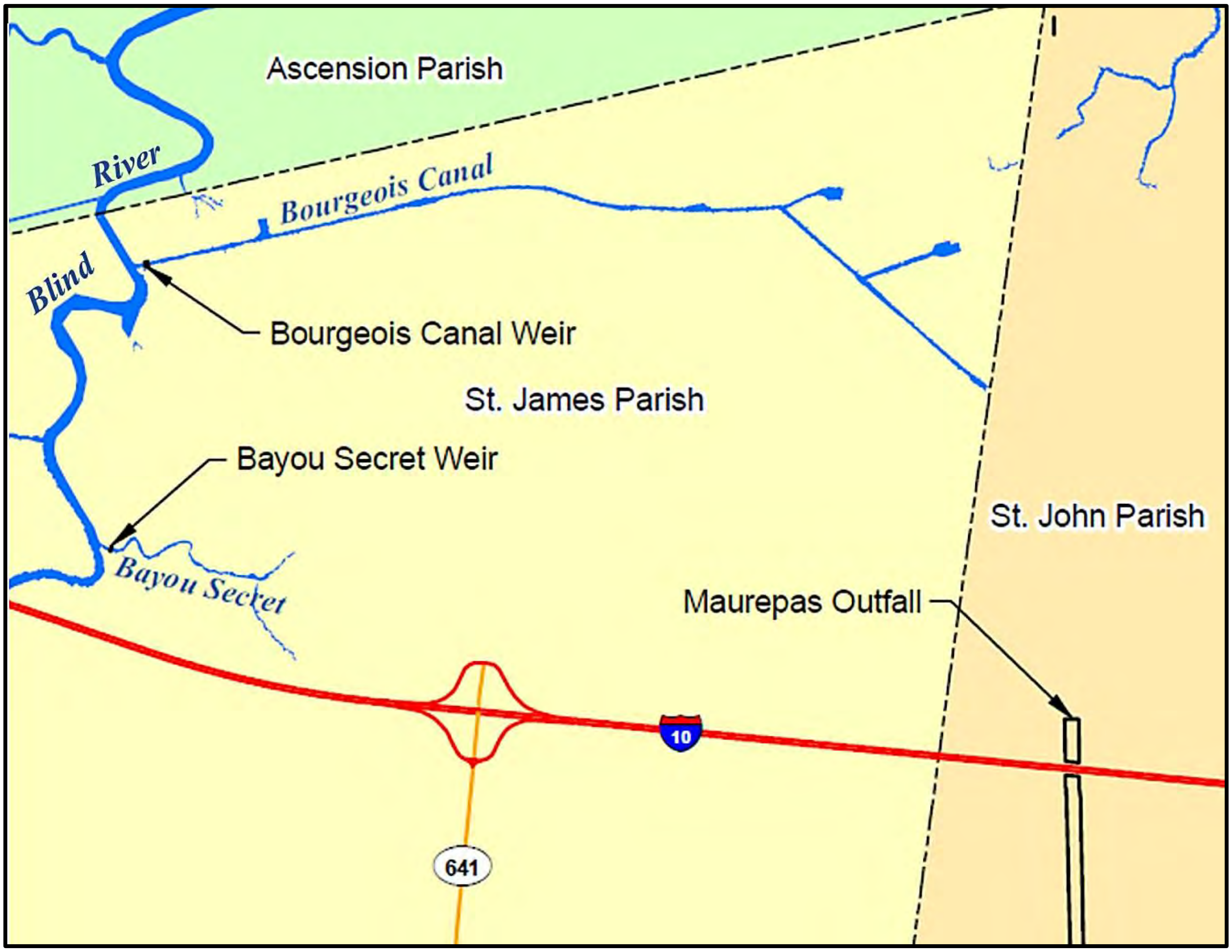


**Figure 6.10 – Bayou Secret & Bourgeois Canal Weir Locations**

Left as is, these two waterbodies would short-circuit the intended water distribution, routing a significant portion of the Maurepas discharge to Blind River and Lake Maurepas, by-passing areas of the swamp which need the freshwater and nutrient input. To prevent the short-circuiting, which would greatly reduce the hydraulic retention time in the swamp, a mechanism to retain a portion of the flow for sufficient time must be implemented to ensure water dispersion occurs throughout the swamp.

### 6.12.1 Previous Design

During the Preliminary and 2013 Designs, the solution selected to prevent the short-circuiting was the installation of riprap weirs in each of the waterbodies near their Blind River end. The geometry of the weirs was based on numerical modeling and their configurations were established to enable the waterbodies to continue functioning as they currently do, but to “hold up” a portion of their flow. The weirs would create what's termed in hydraulic analysis as a back-water effect. This simply means that an impediment downstream causes the WSE upstream to rise. As determined by trial and error in the modeling, the height required to elevate the upstream WSE enough to cause a dispersal effect into the swamp was derived. Based on the upstream WSE target, the design of the proper flow restriction element to place in each channel was developed.

### 6.12.2 Alternatives Analysis

Bayous and canals flow by gravity; the driving force comes from the differential head between the higher upstream WSE and the lower downstream WSE. With a fixed driving head, the basic principal of reducing the flow in any waterbody is to reduce the cross-sectional area of the flowstream. Classic flow restriction devices include orifices, which provide a smaller designated opening (typically circular); channelization, which involves restricting the area along the sides of the flowpath; and weirs, which block a portion of the channel bottom. Other methods applicable for certain situations include, increasing, the sinuosity of the flow path and installing check dams that partially restrict flow in steps along the flowpath, among others.

Bayou Secret and Bourgeois Canal are navigational flowpaths for both natural water denizens as well as for people traversing the waterways by boat. The three classic flow restriction methods are the most viable in the subject situation. Each is listed and discussed briefly below:

- An orifice in the middle of the flowstream would block the bayou\canal except for the hole through which the flow is routed; this would generally be impractical for boat travel. The center of the channel could be left open, with orifices installed in the blocked sides, but this overcomplicates the design.
- Channelization would reduce the cross-sectional area by establishing a relatively narrow lane through which both man and aquatic creatures could travel. However, to make a significant increase in the upstream WSE, the channel would either need to be significantly narrowed or be narrowed for an appreciably long distance. Narrowing the channel considerably at a single location would likely present navigational difficulties, and narrowing the channel for a considerable distance would be an expensive operation.
- Weirs, which are regularly employed in water control for all types of situations, are the most feasible solution. By blocking a portion of the channel bottom, a weir reduces the cross-sectional area of the flowpath. The reduction in cross-section creates the desired back-water effect upstream in the waterbody. The installation of weirs in Bayou Secret and Bourgeois Canal is very straightforward: common riprap is placed in the location and configuration desired. The natural location is relatively close to their convergence with Blind River. The geometric configuration would block a designated portion of the channel bottom up to the banks on each side. The middle section would be left low enough to enable boat travel for the relatively low draft vessels that traverse these waterbodies.

### 6.12.3 Recommended Design and Next Steps

For the reasons discussed above, the installation of riprap weirs near the confluence of Bayou Secret and Bourgeois Canal with Blind River are the Recommended Design. This is the most feasible alternative, which is readily constructible from common materials, and relatively easy to construct. It is also the least costly alternative compared to the construction of either orifices or a long channelized section.

CPRA recently reported that there is a power pole located adjacent to the proposed weir at Bayou Secret. The weir will serve the same purpose located upstream or downstream of its current location, as long as it blocks the same proportion of the channel cross-sectional area. Also, of importance is the location of the weirs in areas of lower wetlands value. CPRA is going to supply AECOM with the optimum location in terms of environmental impact, and AECOM will adjust the weir configuration, as needed.

## 6.13 Embankment Cuts

The Maurepas Diversion is intended to convey fresh water, nutrients, and sediments to restore the health and essential functions of the swamp. Water must be circulated throughout the swamp to accomplish that objective. Water movement into the northwest corner of the swamp is restricted by an embankment that was constructed decades ago to support a defunct logging railroad spur. As **Figure 6.11** shows, the embankment runs approximately two miles both north-south and east-west, acting like a dam, which blocks water movement to the west and north. To restore the original hydrology of the area, the embankment must be degraded in key locations to enable water distribution throughout the swamp.

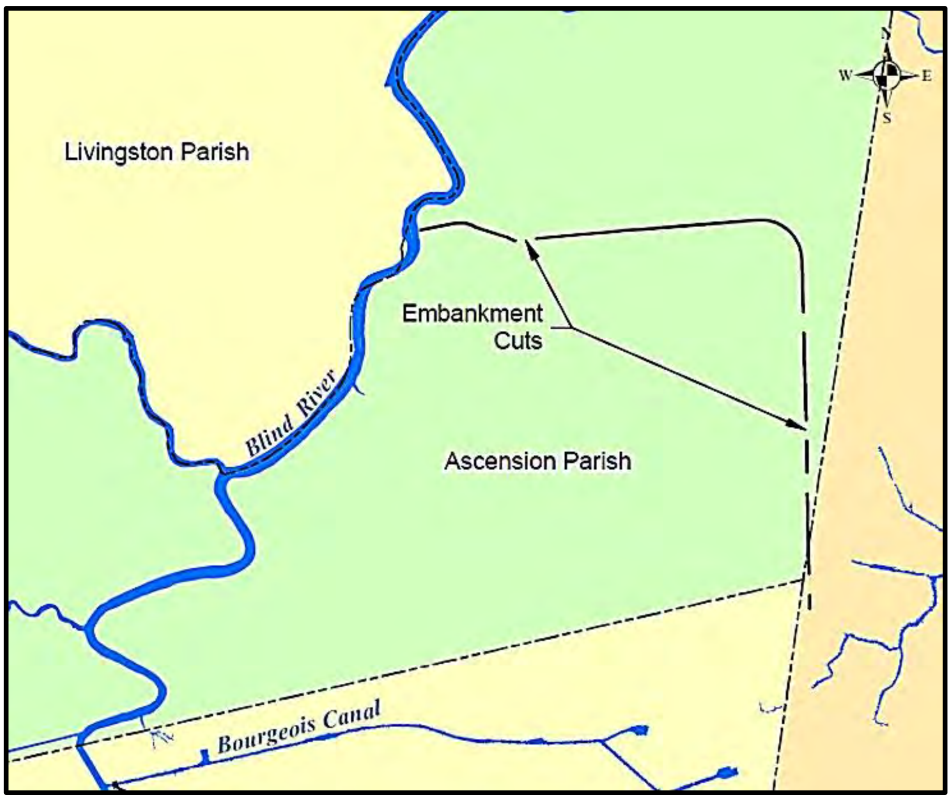


**Figure 6.11 – Bayou Secret & Bourgeois Canal Embankment Cuts**

### 6.13.1 Previous Design

During the Preliminary and 2013 Design phases, the locations and geometry of the cuts to be made to the existing embankment were established. Five cuts were selected, as shown on **Figure 6.11**, each 600-ft long by 30-ft wide, which were to be excavated to grade. The spoil material was designed to be placed on each end of the cuts so that the hydraulic flowpath created by each cut was not impeded by the excavated material.

### 6.13.2 Alternatives Analysis

The alternatives analysis consisted of running the Delft3D hydrodynamic model in 2019 to confirm the analysis performed in 2008 using the AdCirc modeling software. The new model showed that the cuts, as designed would provide adequate water dispersion to the northwest portion of the swamp. This is included with the H&H data in **Appendix D**.

### 6.13.3 Recommended Design and Next Steps

The Recommended Design consists of the five embankment cuts shown on the 2013 Design plans and documented in the Delft3D hydrodynamic modeling conducted in 2019.

Upon field exploration conducted by CPRA, the width of the railroad embankment appears to be narrower than earlier believed. This affects both the design of the embankment cuts as well as the means of access to the five cut locations. The access route was anticipated to be from Blind River then along the top of the embankment. This may not be a feasible route; this will be reviewed in the next design phase.

# 7. West Shore Lake Pontchartrain Project Features

## 7.1 Mississippi River Levee Tie-in

### 7.1.1 Recommended Design and Next Steps

The addition of the WSLP flood protection features in the current Project includes a tie-in of the WSLP levee to the MRL and a floodgate across River Road. These two features connect to one another, as space between the existing River Road and MRL is minimal. Also included in this design component is an emergency access road to allow passage around the River Road floodgate when it is closed.

The tie-in to the MRL south of River Road is designed to the 2070 elevation of +16-ft NAVD88 because it is a short section of the alignment (approximately 55-ft), contains hardened structures, and abuts the new River Road floodgate monolith. The design includes 17-ft of I-wall directly south of the River Road monolith with a 4-ft concrete wall cap; embankment soil begins its upward slope along this wall to meet the design protection height. The remaining length to the MRL is comprised of a 30-ft length of steel sheet pile with a 1-ft concrete cap embedded in the embankment material and concrete slope protection that supports the emergency access road. The sheet pile terminates where the top of the embankment meets the existing MRL grade; the concrete slope protection keys into the embankment just beyond the sheet pile terminus. See **Figure 7.1** for the Profile View of this Feature.

The emergency access road is 12-ft wide and arcs to the south around the River Road structures. The road sits atop the concrete-lined embankment, which slopes up to EL +16-ft NAVD88 on both the flood and protected sides of the sheet pile transition I-wall. Additional details on the design considerations for this item are found in the following Section.

At this time Feasibility-level designs are shown in the Plans and Calculations. Geotechnical, civil and structural design of all components for all WSLP load conditions will be performed in subsequent design phases.

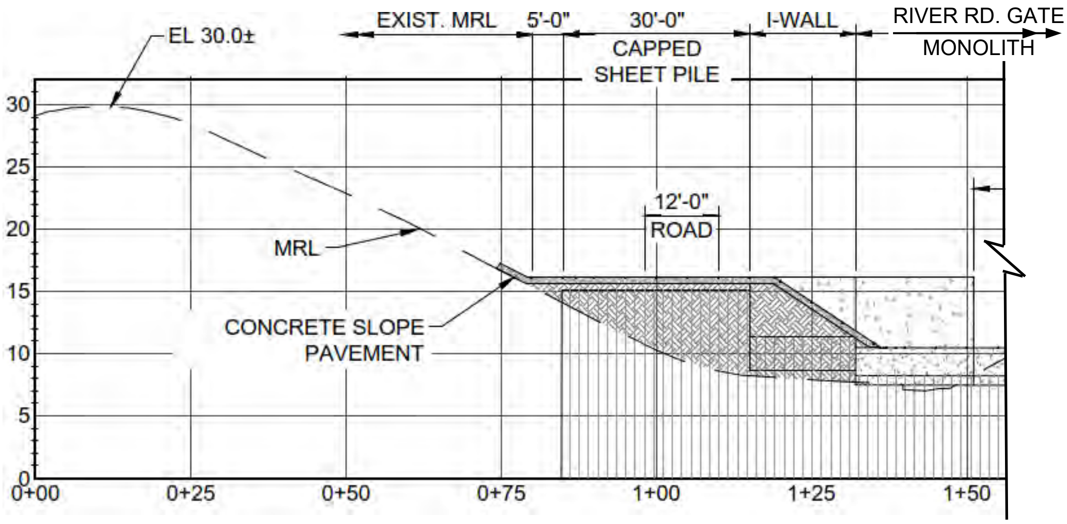


**Figure 7.1 – Mississippi River Levee Tie-In Recommended Design**

## 7.2 River Road Crossing

### 7.2.1 Alternatives Analysis

#### 7.2.1.1 Flood Protection Alternatives

Two flood protection options were considered at River Road: 1) Raising the roadway to the Design Elevation (2020 = EL +8.5-ft NAVD88, 2070 = EL +16-ft NAVD88), and 2) Installing a floodgate across the roadway. The existing elevation of River Road ranges from EL +10.5-ft NAVD88 on the east side of the project corridor to EL +11-ft NAVD88 on the west side. Thus, the existing roadway exceeds the 2020 Design Elevation. However, since the roadway is considered a permanent structure, its elevation would not be continually revised over time to meet the 2070 Design Elevation (multiple road raises would be impractical).

The flood protection is to be raised linearly over the fifty-year interval to meet the Design Elevations required. **Table 7.1** shows the required elevations for each calendar year from 2020 through 2070.

***Table 7.1 – Design Elevations Required per Calendar Year***

| Calendar Year | Design Elevation (ft NAVD88) | Calendar Year | Design Elevation (ft NAVD88) | Calendar Year | Design Elevation (ft NAVD88) | Calendar Year | Design Elevation (ft NAVD88) |
|---|---|---|---|---|---|---|---|
| 2020 | 8.50 | 2033 | 10.45 | 2046 | 12.40 | 2059 | 14.35 |
| 2021 | 8.65 | 2034 | 10.60 | 2047 | 12.55 | 2060 | 14.50 |
| 2022 | 8.80 | 2035 | 10.75 | 2048 | 12.70 | 2061 | 14.65 |
| 2023 | 8.95 | 2036 | 10.90 | 2049 | 12.85 | 2062 | 14.80 |
| 2024 | 9.10 | 2037 | 11.05 | 2050 | 13.00 | 2063 | 14.95 |
| 2025 | 9.25 | 2038 | 11.20 | 2051 | 13.15 | 2064 | 15.10 |
| 2026 | 9.40 | 2039 | 11.35 | 2052 | 13.30 | 2065 | 15.25 |
| 2027 | 9.55 | 2040 | 11.50 | 2053 | 13.45 | 2066 | 15.40 |

| Calendar Year | Design Elevation (ft NAVD88) | Calendar Year | Design Elevation (ft NAVD88) | Calendar Year | Design Elevation (ft NAVD88) | Calendar Year | Design Elevation (ft NAVD88) |
|---|---|---|---|---|---|---|---|
| 2028 | 9.70 | 2041 | 11.65 | 2054 | 13.60 | 2067 | 15.55 |
| 2029 | 9.85 | 2042 | 11.80 | 2055 | 13.75 | 2068 | 15.70 |
| 2030 | 10.00 | 2043 | 11.95 | 2056 | 13.90 | 2069 | 15.85 |
| 2031 | 10.15 | 2044 | 12.10 | 2057 | 14.05 | 2070 | 16.00 |
| 2032 | 10.30 | 2045 | 12.25 | 2058 | 14.20 | N\A | N\A |

As the table shows, the existing roadway will exceed the required Design Elevation through the year 2033 (see highlighted entries). Though the roadway is currently meeting water surface requirements, some seepage-prevention measures would still be required to provide full flood protection. After that year some measure must be employed to provide the required flood protection elevation. One means would be to raise the roadway in 2033 to the 2070 Design Elevation of +16-ft NAVD88; this requires a 5-ft 6-in raise (from EL +10.5-ft NAVD88 to EL +16-ft NAVD88). The raise would require adjustments to enable continued access to the residential streets (Marigold St., Belette/Chestnut St., Marquez St., Periwinkle Ln, Daffodil St, etc.), as well as to the Marathon Terminal Complex. The residential developments are rather close to the roadway, making the installation of a service road difficult.

The second alternative, installing a flood gate across the roadway, is the preferred option because of its relative simplicity. River Road is not considered by the DOTD to be a hurricane evacuation road and can thus be blocked by a floodgate without significant impacts to the community. The residential neighborhoods built around the streets mentioned above can readily access LA 54 for egress to Airline Highway, which is a designated hurricane evacuation route. The industrial facilities to the west of the proposed floodgate have less than 2½ miles to travel along River Road to reach LA 3213, which also connects to Airline Highway.

#### 7.2.1.2 Emergency Bypass Alternatives

An emergency bypass will be installed to provide emergency vehicles a path around a deployed River Road floodgate. For example, Fire Station No. 72 is less than one-third of a mile from the proposed floodgate; the fire company requires access to the industrial sites west of the floodgate regardless of storm conditions. Similarly, EMS ambulances; police cruisers and motorcycles; tow trucks; Parish personnel vehicles; and gas and electric utilities trucks need access in both directions.

There are two options for the by-pass route: 1) using access roads constructed during phased Headworks construction to and from the levee crown, and 2) constructing a shorter by-pass on the levee side of the roadway, close to the floodgate. Alternative one requires no additional construction; however, it is a longer route, the levee crown is only 10-ft wide, and vehicles would pass immediately adjacent to the Headworks Structure. Alternative two requires additional construction, but it is a modest length of 12-ft wide roadway on an embankment required by the MRL tie-in in the same location. The second alternative is recommended because it is a shorter route, it does not require vehicles to drive on the levee crown, and it does not bring emergency and private vehicles past by the Headworks Structure.

### 7.2.2 Recommended Design and Next Steps

This proposed crossing is comprised of a rolling floodgate, gate monolith, and storage monolith. All are designed to the 2070 EL +16-ft NAVD88. To the south the crossing adjoins the MRL Tie-in described above; north of the crossing is the concrete capped I-wall which runs adjacent to the Maurepas Headworks Structure and connects the River Road crossing to the CN railroad crossing.

#### 7.2.2.1 Sizing

The gated opening width of 40-ft assumes the use of guardrails on both sides of the road to protect drivers and gate support pilasters from impact. The clear distance from roadway to pilaster is based on DOTD Standard Plan GR-200, which requires at least 6-ft 6-in from end of road to edge of guardrail for a 50 mph roadway (River Road is posted at 45 mph); added to this is 2-ft 6-in for the guardrail and a small gap around the pilaster. **Figure 7.2** presents a similar arrangement currently installed at the US-90 (Chef Menteur Hwy) Crossing of the USACE LPV109 flood risk reduction system in eastern New Orleans.


**Figure 7.2 – Roller Gate with Removable Guardrail Arrangement**

The 66-ft long by 10-ft wide gate monolith is sized to provide flood protection south of the crossing until grading allows for the beginning of the I-wall transition. The 52-ft 8-in long by 10-ft wide gate storage monolith is designed to provide space for the full 42-ft long steel gate, a winch pedestal, a gate stop, and the Point of Intersection (PI) required just north of the gate storage location. Both monoliths have a Top of Slab Elevation of 10.49-ft NAVD88 to match existing River Road grade.

#### 7.2.2.2 Gate Type Alternatives Analysis

Two primary options were considered for the design of the floodgate: a roller gate and a swing gate. Though either type could be used, the roller gate is more suited to the specifics of this crossing.

First, if a swing gate were to be used, it would be highly advisable to use a one-leaf gate to avoid the requirement of a temporary center support. Such a support would require additional installation time and an anchoring system in the roadway. In this application, a one-leaf gate would also have a large length-to-height ratio (40-ft length x 5-ft 5-inch height). This is not ideal for swing gates because they require support along their free end, usually in the form of a roller on the ground or a tension rod support tying the free end to the hinged support. **Figure 7.3** presents an example of the structural modifications require for these large ratios.

Roller gates are relatively easy to operate in a range of conditions, are adaptable to wide openings, and require only that only a strip of roadway a bit wider than the gate be level. Also, their linear storage arrangement along the wall line is well-suited to this location.

Miter gates were also considered but, when compared to the roller gate option, result in unnecessary complications related to angled loads, latching, and level space for the swing radius.

Taller, larger support pilaster with additional steel framing to support free end of gate

**Figure 7.3 – Swing Gate with Large Length-to-Height Ratio**

7.2.2.3 Design

For this Task Order, both monoliths are sized using two load cases: 1) Water to Top of Wall, and 2) Construction with Surcharge. Simple SAP2000 structural finite element models are used to compile base reactions, which are then used in the Pile Group Analysis Computer Program (CPGA) o develop pile foundations. Stem walls and slab thicknesses are set based on shear and moment calculations (typically performed by hand or specifically developed spreadsheets). At this time, the angle in the storage monolith is neglected in the design; however, this and all WSLP monoliths will reflect unique PI angle and grade details in future submissions.

HP14x73 piles are used for this initial design because the 2013 Maurepas Geotechnical Report provides pile capacities for HP14s at the nearby Headworks Structure and because they are very common in USACE floodwall foundations. Preliminary designs propose two-pile rows, both piles battered outward, with average spacing between rows of 9-ft 6-in and Tip Elevation of EL (-) 35-ft NAVD88. Pile size, type, and layout will be optimized in future design stages. Soil information used in foundation design is also taken from the Headworks 2013 Geotechnical data; this will be updated with site-specific information in future phases.

The primary members of the steel roller gate are sized following ETL 1110-2-584 LRFD procedures for two load cases: 1) Water to Top of Wall, and 2) Construction with Wind. A simple SAP2000 model is used to apply the lateral hydrostatic and wind loading on the gate for each load case. The girders are then sized based on the maximum applied moment and resulting SAP reactions at the top and bottom girder locations. Skin plate thicknesses and intercostal plates are then designed based on hydrostatic loading only. The final roller gate design over River Road has a 40-ft gate opening and 5-ft 7 ½-in approximate gate height, with W16x100 top girder and W24x176 bottom girder. Skin plate thickness used is ¼-in with ¼-in by 4-in intercostal plates spaced at 3-ft 8-in along the gate opening.

**Appendix E** contains all structural calculations developed to size the structures and foundations described in this design. Section 1 of the Appendix presents concrete and pile foundation designs, Section 2 presents steel gate designs.

Geotechnical, civil and structural design of all components for all WSLP load conditions will be performed in subsequent design phases. Geotechnical, survey, and hydraulic data collected and analyzed in this Task Order will be incorporated as it becomes available.

## 7.3 Airline Highway Crossing

### 7.3.1 Alternatives Analysis

The primary alternatives at this major highway crossing are an elevated roadway or a bridge over WSLP flood control features; a gated closure has been removed from consideration. The two projects are closely linked at this location because of the recommended flood control solution; see Maurepas Diversion Airline Highway Crossing Section 6.9.2 for a detailed description of the Airline Hwy Alternatives Analysis process.

### 7.3.2 Recommended Design and Next Steps

The Airline Highway roadway raise will be further developed in subsequent design efforts, once a final decision is made by all involved parties. The design of the embankment will be detailed and optimized with the aim of placing as little embankment fill as possible (and thus reducing costs) while providing a robust roadway and flood protection feature. Grades, the use of MSE construction, speed limit and ROW impact requirements, among other considerations will guide the future design efforts, as will continued input from DOTD and CPRA. The final roadway criteria, including the design speed, the optimum grades, the length of vertical curves, and the median widths, among other parameters, will be specified in the next phase of design.

## 7.4 CN Railroad Crossing

There is a unique complication at the CN RR crossing that will require additional enquiry before a structural arrangement can be finalized: it is still not known whether this crossing will contain three or four rail lines. Currently two CN lines pass through the area, a mainline and a spur. Marathon (legacy Pin Oak) has conceptual site development plans for a rail spur, adding a third line to the north side of the existing tracks. Also, during previous discussion about the Project in March 2020, CN RR stated they plan to add fourth line on the south side in this location. AECOM has repeatedly requested documents from John Dining, our CN Point of Contact, regarding the planned fourth line; however, to date CPRA and AECOM have not received any supporting documents (preliminary plans, written record of an investigation, etc.).

Another complicating factor is that data collection at CN RR has been slower than in other areas due to delays in granting surveyors a Right-of-Entry agreement. Existing structure elevations, sizes, and other pertinent survey data were only recently collected and submitted to the Design Team. All survey work is now complete, however, and this data will be incorporated in future design efforts.

To proceed with preliminary sizing and begin understanding the Rough Order of Magnitude Cost for this project, CPRA directed AECOM to provide a design for four rail lines, as this would be the most costly and technically demanding possibility.

### 7.4.1 Recommended Design and Next Steps

The proposed four-line crossing design is comprised of a rolling floodgate, two-part gate monolith, two-part storage monolith, and one PI T-wall monolith. The crossing arrangement links to sections of I-wall to the north and south. All structures are designed to the 2070 EL +16-ft NAVD88.

#### 7.4.1.1 Sizing

A clear opening size of 89-ft 1-in is required to allow for four adjacent tracks skewed 39° in orientation plus 9-ft clearance from centerline of track to the edge of the nearest obstruction. CN requires a minimum clearance of 8-ft 6-in per *CN Engineering Specifications for Industrial Tracks* (Office of Chief Engineer Structures, Design, and Construction, January 31, 2019); however, to keep RR gate details coordinated during preliminary design, 9-ft was used. The gate and storage monolith structures are bisected with expansion joints because lengths of concrete in excess of 100-ft are difficult to construct monolithically and are prone to temperature and shrinkage problems. Gate monoliths are 59-ft long by 12-ft wide and storage monoliths are approximately 46-ft long by 10-ft wide. **Figure 7.4** is the Plan View layout of the proposed gate monolith showing the two current tracks, two future tracks, distances to hard points, and other pertinent features.

All monoliths at this crossing have an assumed Top of Slab Elevation of EL +11.98-ft NAVD88. This elevation is not confirmed by survey data during preliminary design, as CN data did not become available until very late in the design process. This data point is the Top of Rail Elevation surveyed at the proposed Conveyance Channel crossing during previous Maurepas Diversion design phases.

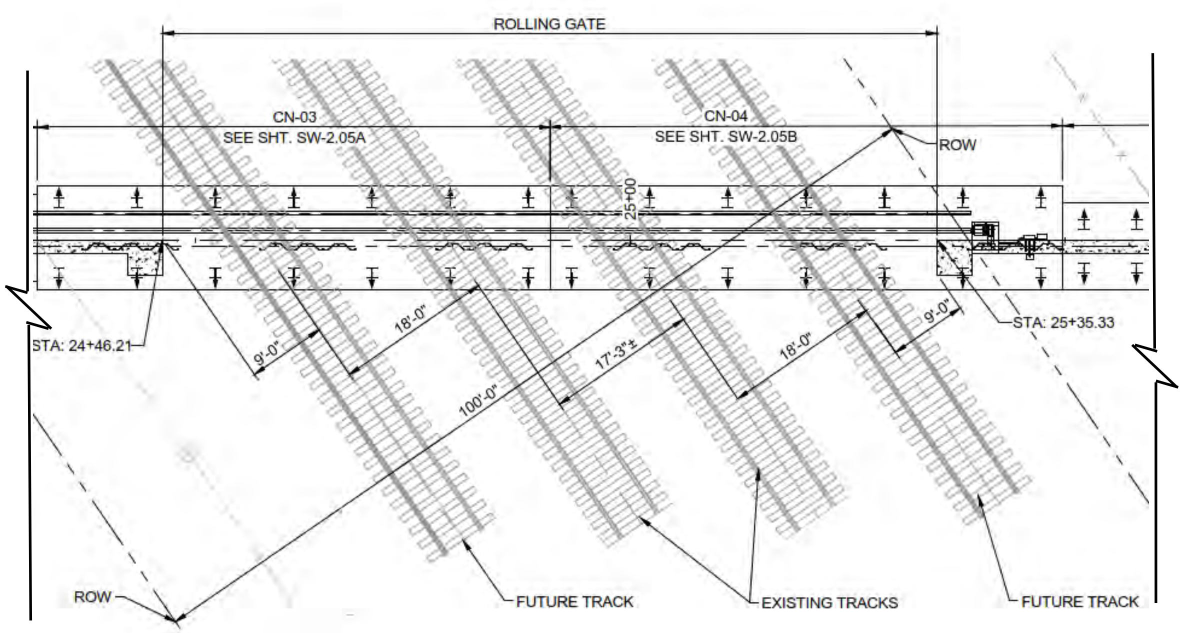


**Figure 7.4 – CN Rolling Gate Plan View**

7.4.1.2 Gate Type Alternatives Analysis

Two primary options are considered for the design of the floodgate: a roller gate and a miter gate. Both are well-suited to large width-to-height ratios, as is the case for this 89-ft by 1-ft gate opening. Primary considerations in this decision are:

- Roller Gate:
  - Roller gate may be designed to span the full opening without use of a center support. This may result in a very heavy, stout gate with large girder members, however at this crossing there is little space for a temporary or permanent center support.
  - Gate stores on the adjacent monolith parallel to the wall stem.
  - Gate is relatively easy to operate without specially trained personnel.
  - Due to the limitations of the 300-ft ROW, the gate requires a skewed alignment relative to the exiting RR tracks. This makes the required gate opening longer.
- Miter Gate:

- Will likely will be less weighty than a roller gate. The miter gate is split into two leaves that create a three-hinged arch when closed (gate shaped like a chevron when closed with center point towards the flood side). This arrangement more efficiently transfers flood loads to the concrete supports and foundations than a flat-faced gate.
- Gate latches to itself at the center; no center support is required even though the gate is comprised of two leaves.
- Gate style allows for a perpendicular crossing of the tracts, creating a smaller clear span between pilasters.
- Hinge and support structure are more complicated and costly than those of other gate types, both in design and fabrication; these items must be adjustable to accommodate the miter angle plus construction tolerances.
- Deflection tolerances are quite low to ensure proper sealing of the miter joint and seals. This effects the gate, the supporting structure, and the foundation.
- Gate is difficult to operate in high wind events. This could be an issue, as railroad gates tend to be closed last-minute to avoid long track outage times.
- Gate storage configuration may interfere with planned access roads

A final decision on the appropriate gate type will be made when the number of rail lines and elevation of the Top of Rail are confirmed.

#### 7.4.1.3 Design

Monoliths are sized in this Task Order using three load cases: 1) Water to Top of Wall, 2) Construction with Surcharge, and 3) Normal Operations with all four rail lines occupied. Simple SAP2000 models are used to compile base reactions, which are input in the CPGA program to develop pile foundations. Stem walls and slab thicknesses are set based on shear and moment hand calculations (often using specifically created spreadsheets). For preliminary design the PI angle in the transition monolith CN-05 is neglected; however, this and all WSLP monoliths will reflect unique PI angle and grade details in future submissions.

HP14x73 piles are used for initial design because the 2013 Maurepas Geotechnical Report provides pile capacities for HP14s at the nearby Headworks Structure and because they are very common in USACE floodwall foundations. Preliminary designs propose two-pile rows, both piles battered outward, for all monolith types. Gate storage monolith piles are spaced at 10-ft and have a Tip Elevation of EL (-) 32-ft NAVD88; PI-monolith piles are spaced at 6-ft with Tip EL (-) 32-ft NAVD88; and gate monolith piles are spaced at 9-ft and have Tip EL (-) 60-ft NAVD88. Preliminary calculations supporting this and the concrete designs are presented in Section 1 of **Appendix E**. Pile size, type, and layout will be optimized in future design stages. Soil information used in foundation design is also taken from the Headworks 95% Geotechnical data; this will be updated with site-specific information in future phases.

The primary members of the steel roller gate are sized following ETL 1110-2-584 LRFD procedures for two load cases: 1) Water to Top of Wall, and 2) Construction with Wind. A simple SAP2000 model is used to apply the lateral hydrostatic and wind loading on the gate for each load case. The girders are then sized based on the maximum applied moment and resulting reactions (calculated by the SAP2000 program) at the top and bottom girder locations. Skin plate thicknesses and intercostal plates are then designed based on hydrostatic loading only. The final roller gate design over the four-line CN Railroad crossing has an 89-ft 1-in gate opening and 4-ft 1 ¾-in approximate gate height, with W24x68 top girder and W36x194 bottom girder. Skin plate thickness used is ¼-in with ¼-in by 4-in intercostal plates spaced at 4-ft 2-in along the gate opening. The preliminary calculations supporting this gate design are presented in Section 2 of **Appendix E**.

## 7.5 KCS Railroad Crossing

### 7.5.1 Recommended Design and Next Steps

A small break in the WSLP levee approximately 120-ft long occurs at the KCS Railroad. The crossing is for one rail line and the proposed design is comprised of a swing floodgate, gate monolith, and four partially embedded T-wall monoliths that tie-in to levees to the north and south. All structures are designed to the 2070 EL +16-ft NAVD88. **Figure 7.5** presents an Elevation View of the proposed WSLP KCS RR Crossing.

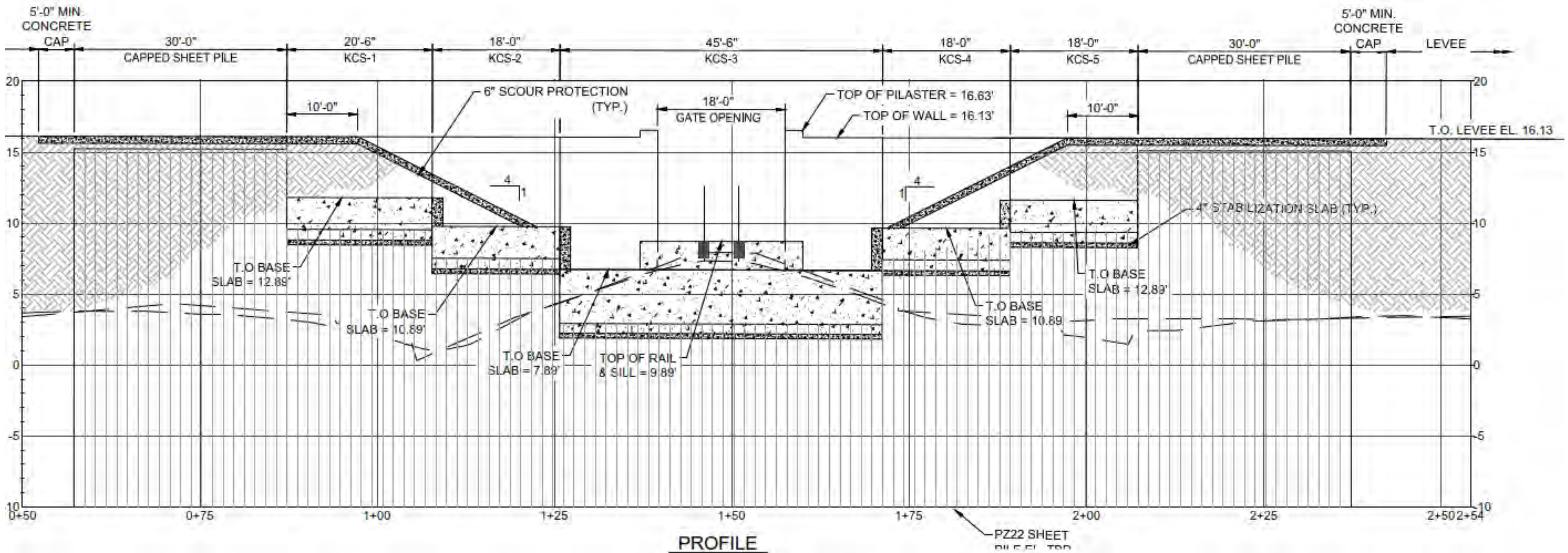


**Figure 7.5 – Elevation View of WSLP KCS RR Crossing**

#### 7.5.1.1 Sizing

The gate's clear opening size is 18-ft. This is based on the KCS requirement of a 9-ft clearance from centerline of track to edge of obstruction (*KCS Guidelines for Design and Construction of Industry Tracks*, August 1, 2017). The 45-ft 6-in long by 10-ft wide gate monolith is extended beyond the gated opening to the north and south until site grading can begin the slope upward to the full levee section. Top of Slab for the gate monolith is EL +7.89-ft NAVD88 and the top of the sill beam, which extends up from the slab and matches existing KCS Top of Rail, is EL +9.89-ft NAVD88.

Two transition T-walls flank the gate monolith on either side. Top of Slab of center monoliths KCS-2 and KCS-4 is EL +10.89-ft NAVD88, and Top of Slab of end monoliths KCS-1 and KCS-5 is EL +12.89-ft NAVD88. All four monoliths are 18-ft long by 10-ft wide, except for KCS-1, which is 20-ft 6-in long. Monoliths are sized and stepped up to minimize excavation and concrete/steel placement; this arrangement will be optimized where possible in future design phases. An example of a similar (but significantly taller) railroad swing floodgate with stepped-up transition T-walls is shown in **Figure 7.6**; the image is of the USACE LPV110 CSX RR Crossing in eastern New Orleans.

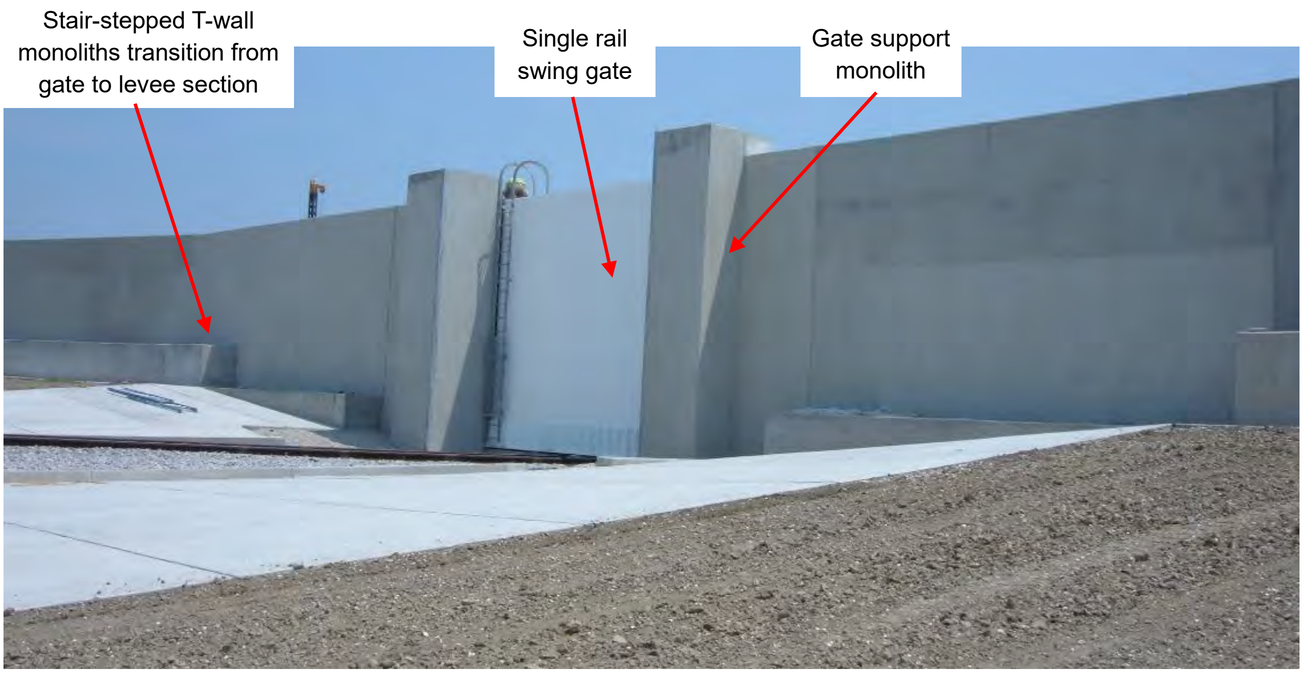


**Figure 7.6 – Swing Railroad Gate with Stepped-up Transition T-walls**

#### 7.5.1.2 Gate Types Alternatives Analysis

As with other WSLP openings, roller and swing gates are both possible solutions. The 18-ft wide by approximately 6-ft 3-in high railroad gate is well-suited to a swing gate: the span is relatively short, there are no obstructions to the gate swing radius, and less length of hardened structure is required because the gate can store parallel to the railroad tracks instead of on an adjacent monolith. Also, swing gates are common features on USACE flood control crossings of railroads in south Louisiana.

#### 7.5.1.3 Design

The design process follows the same methodology as previously described crossings. Three load cases, Water to Top of Wall, Construction with Surcharge, and Normal Operations with Train, are used to size primary members and foundations. Base reactions for CPGA foundation design are developed with SAP200 models; stem walls and slabs are sized with hand and spreadsheet calculations for shear and moment. For preliminary design the PI-angle in monolith KCS-1 and the

3D effects of the levee transition slope are simplified to straight, uniform T-wall designs. Future design phases will develop unique sloped grading and PI-monolith designs.

In keeping with the other WSLP crossing structures, HP14x73 piles are used. HP14 capacity curves and soil properties are taken from the 2013 Maurepas Headworks Geotechnical Report in order to form the Rough Order of Magnitude Cost Estimate, as no data from this area regarding steel piles was available (Maurepas features use PPC piles). Again, two-pile rows with both piles battered outward are employed. The gate monolith structure is founded on piles spaced at 6-ft 6-in to Tip EL (-) 38-ft NAVD88; flanking T-wall monoliths are founded on piles spaced at 6-ft with Tip EL (-) 30-ft NAVD88. Pile size, type, and layout will be optimized in future design stages and soil data will be updated with site-specific information as it becomes available. Preliminary designs propose two-pile rows, both piles battered outward, with average spacing between rows of 6-ft.

The primary members of the steel swing gate are sized following ETL 1110-2-584 LRFD procedures for two load cases: Water to Top of Wall and Construction with Wind. A simple SAP2000 model is used to apply the lateral hydrostatic and wind loading on the gate for each load case. The girders are then sized based on the maximum applied moment and resulting SAP reactions (k/ft) at the top and bottom girder locations. Skin plate thicknesses and intercostal plates are then designed based on hydrostatic loading only. The final swing gate design over KCS Railroad has an 18-ft gate opening and 6-ft 3-in approximate gate height, with W18x106 top and bottom girders. Skin plate thickness used is ¼-in with ¼-in by 4-in intercostal plates spaced at 2-ft 5-in along the gate opening.

**Appendix E** contains all structural calculations developed to size the structures and foundations described in this design. Section 1 of the Appendix presents concrete and pile foundation designs, Section 2 presents steel gate designs.

## 7.6 Connecting Features

### 7.6.1 River Road to CN Railroad

#### 7.6.1.1 Recommended Design and Next Steps

The southern end of this reach is highly congested. Space is limited within the 300-ft project ROW to accommodate the Maurepas Headworks; Sedimentation Basin; and their surrounding berms, drainage ditches, and roads. In lieu of a full levee section, a hybrid levee section with a concrete-capped sheet pile I-wall is proposed for WSLP flood protection between the River Road and CN RR crossings. This approach reduced the required foot-print of the levee. The I-wall runs along the east edge of the Maurepas Project's ROW and has a consistent 4-ft stick-up height throughout. The typical cross-section is shown in **Figure 7.7**; it has a TOW EL +16.13-ft NAVD88 and a Top of Grade EL +12.13-ft NAVD88. There is a 12-ft wide berm on the protected side of the wall before sloping down at 3H:1V; on the flood side (Maurepas Diversion side) the ground slopes downward directly from the wall at 3V:1H to approximate EL +10-ft NAVD88, where the Conveyance Channel Access road is located. Scour protection will be designed and added to the I-wall section in future phases.

Additional examination of this region, especially the portion adjacent to the Maurepas Diversion Headworks, is required before a final decision is made regarding the I-wall solution. It is possible that sections of the area near River Road may need T-wall instead of I-wall structures. I-wall is the preferred alternative between these two floodwall types, as they are cheaper to construct and maintain.

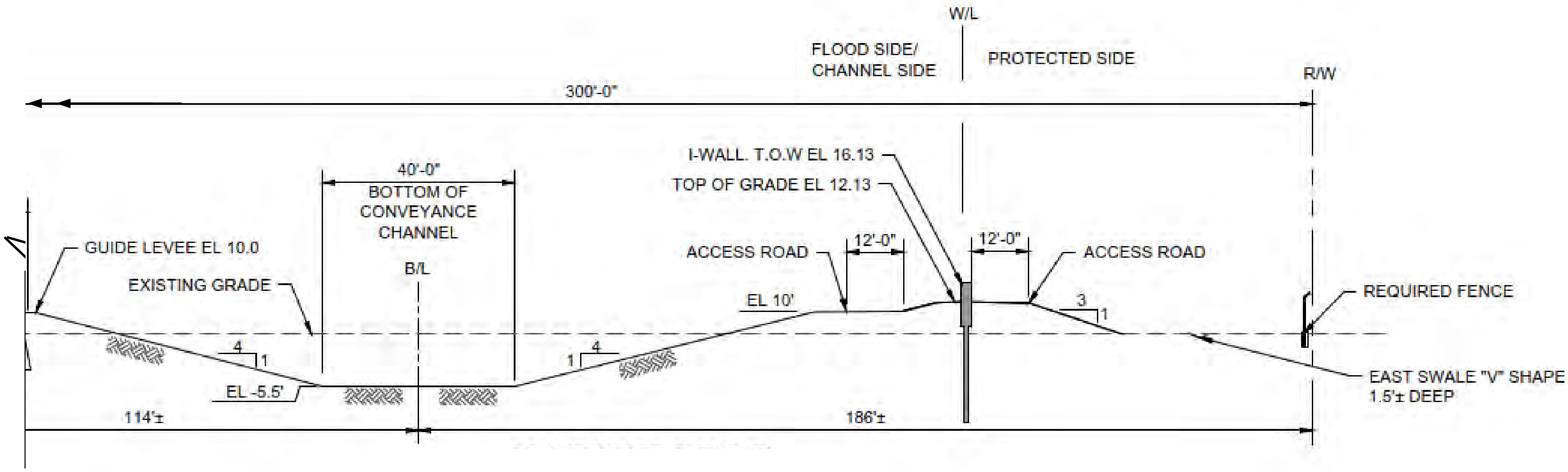


**Figure 7.7 – Typical Cross-Section, River Road to CN Railroad**

### 7.6.2 CN Railroad to WSLP Reach WSLP-114

#### 7.6.2.1 Recommended Design and Next Steps

A levee is proposed for this length of the WSLP alignment, as there is ample space for the full levee footprint. The typical levee section is only adjusted at the rail and road crossings described previously. The typical cross-section has a 10-ft wide levee crown, which also serves as the Conveyance Channel access road, and 3H:1V side slopes. To the east the levee slopes down to approximate EL +6-ft NAVD88 for the CN RR to KCS RR length and EL +2.75-ft NAVD88 for the length from KCS RR to Airline Hwy. After slopes meet these elevations, there is a 25-ft wide mostly level surface (graded only for drainage) followed by the East Drainage Ditch. To the west, a 55-ft wide stability berm is provided between the levee and Maurepas Conveyance Channel slopes. **Figure 7.8** presents the two typical cross-sections for the WSLP levee north of the CN crossing

The majority of the WSLP levee design will be performed in future design phases using the geotechnical and survey data collected during this effort. A high-level stability analysis was performed by Eustis Engineering during this Task Order to develop the typical cross-section described above. Future design phases will include additional stability analyses for multiple sections along the levee\Conveyance Channel alignment along with other USACE-required investigations.

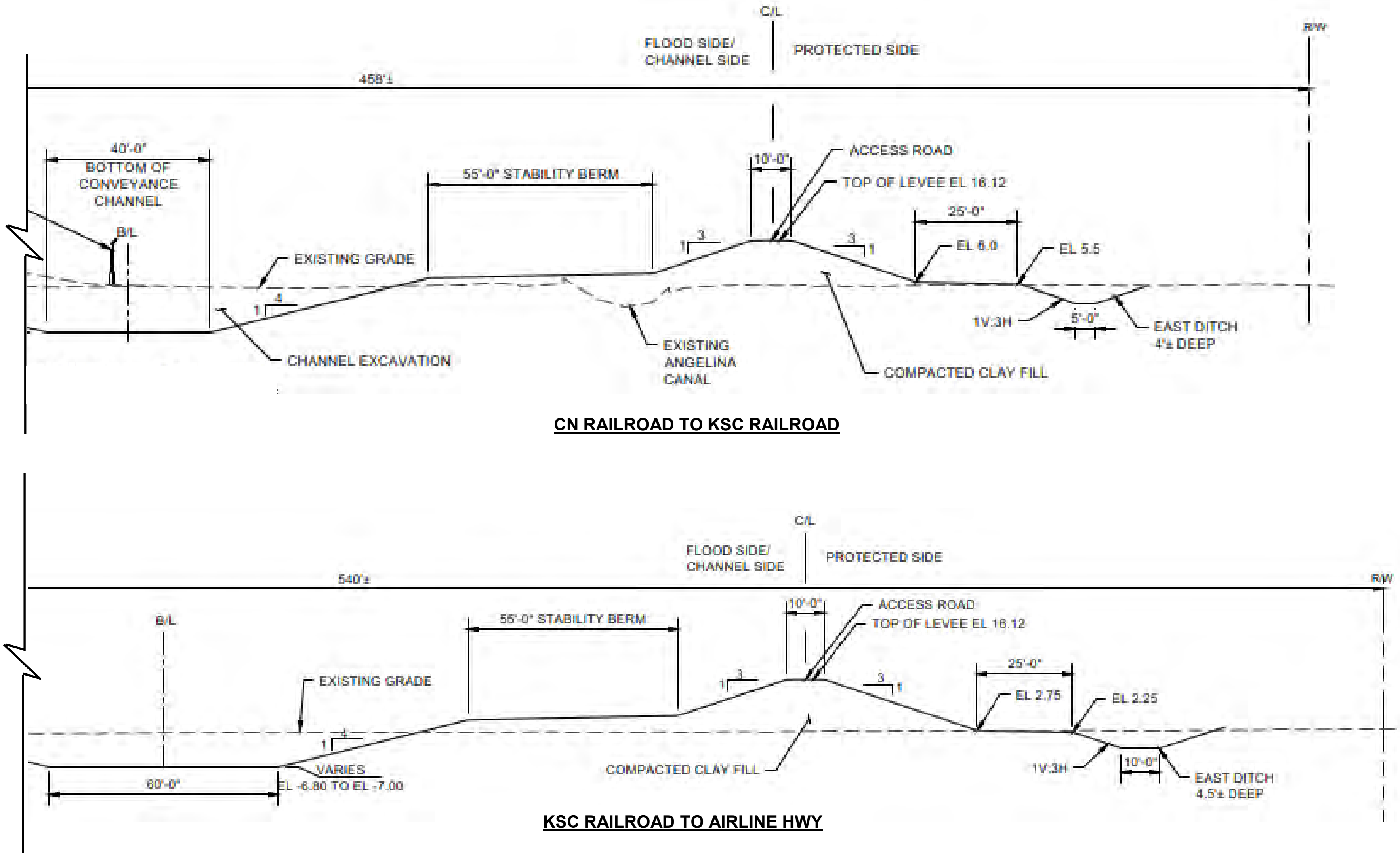


**Figure 7.8 – Typical Cross-Sections, CN Railroad to Airline Hwy**

# 8. Utility & Pipeline Relocations

## 8.1 Need for Relocations

The proposed alignment for the Maurepas Diversion Conveyance Channel stretches 5½ miles from the Mississippi River to deep within the Maurepas Swamp, ending 1,000-ft north of Interstate 10. Due to the length of the proposed construction, the channel intersects numerous utility and industrial product pipeline rights-of-way. The WSLP flood protection features run parallel and adjacent to the Maurepas Diversion features from the MRL to approximately 2,500-ft north of Airline Highway. The levees, I-walls, T-walls, and floodgates that comprise the flood protection features will therefore cross many of the same utilities and pipelines as the Maurepas Diversion, though the precise locations may be a substantial difference from one another.

To construct the channel, these utilities and other infrastructure components must be relocated to positions that will not adversely affect the construction process yet allow continued operation of the utilities\infrastructure elements. While the relocations for Maurepas Diversion and WSLP are technically for the same Project, coordination with utility owners will be conducted for each piece independently to avoid confusion with pipeline\utility owners.

## 8.2 Utility, Pipeline & Infrastructure Conflicts

The utilities crossings were identified from field reconnaissance, historical surveys, maps, records, and information provided directly from the utility and industrial owners. There are six key locations along the proposed Maurepas Diversion\WSLP Project ROW that have numerous utilities and/or product lines which intersect the proposed alignment. These locations include River Road, CN RR, KCS RR, Airline Highway, I-10, and a major pipeline corridor which runs between Airline Highway and I-10. **Figure 8.1** displays the alignment of the Maurepas Diversion and subject reaches of the WSLP Project and highlights the utility/infrastructure crossings, which may have potential conflicts.

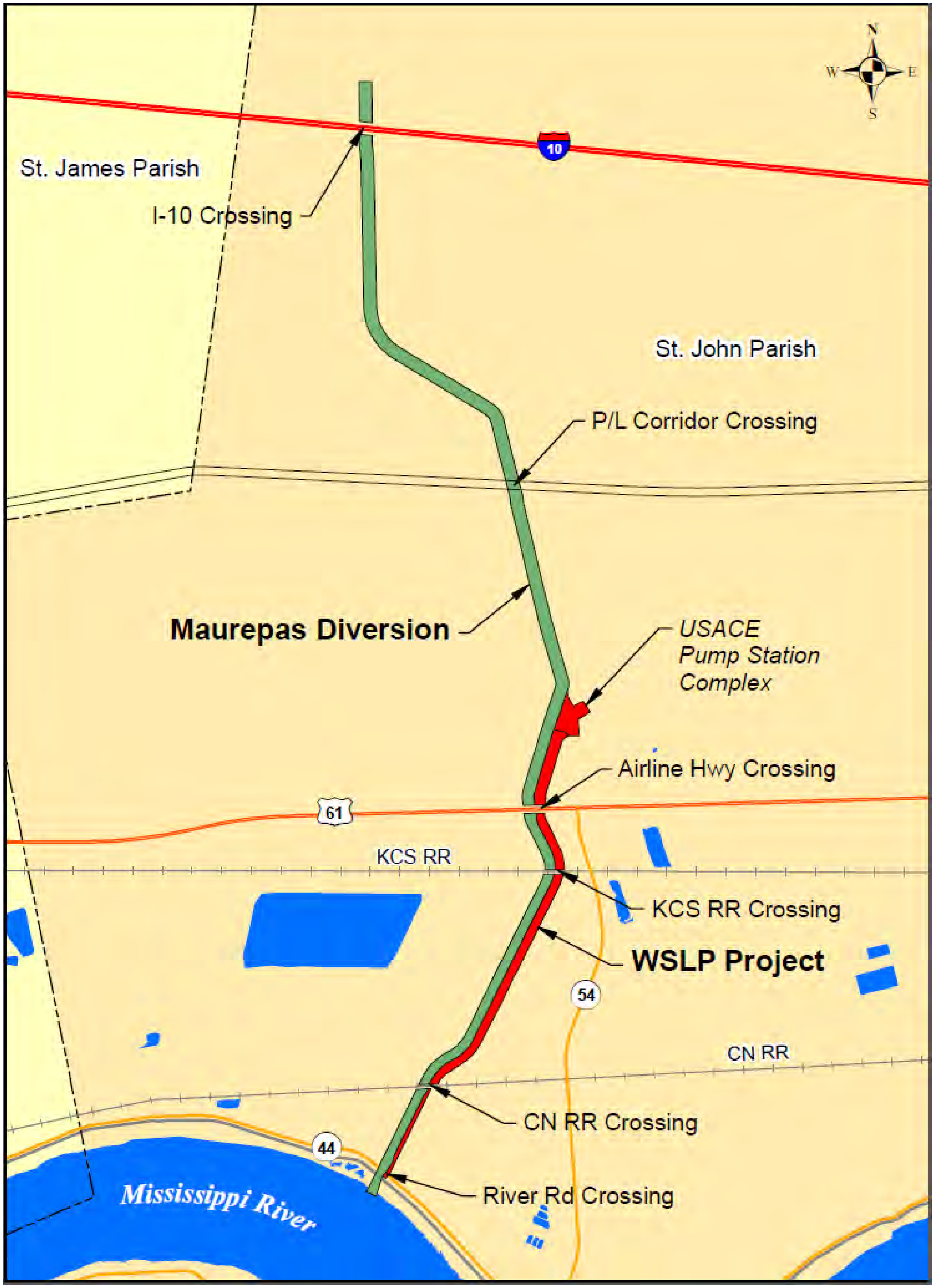


**Figure 8.1 – Major Crossings with Utilities, Pipelines, and Infrastructure**

## 8.3 Utility, Pipeline & Infrastructure Contacts

During Preliminary Design of the Maurepas Diversion, the Design Team contacted all companies with identified utilities or product lines in the proposed alignment right-of-way several times via telephone beginning in November 2007. In those conversations, AECOM (as URS) described the project and established a point of contact within each company for coordination of potential utility/pipeline relocations necessitated by the proposed project. Preliminary design packages containing a letter describing the project, figures displaying the location of the proposed diversion alignment, as well as plan, profile, and cross-section drawings were mailed to each potentially affected company in January 2008. Each company was requested to provide utility/pipeline offsets, depths of cover, and other pertinent requirements, along with any drawings detailing how their utilities and/or product lines are to be relocated.

Each utility and industrial company with services in the potential conflict areas was requested to provide the means by which they preferred their utilities/lines to be relocated. The basic relocation options included: “By Owner” or “By Contractor” and “Prior To” or “Concurrent With” construction. Other options included: “Do Not Disturb”, “Abandon In-Place”, or “Remove without Replacement”. The companies were directed to coordinate with the LDNR (now CPRA) regarding reimbursement for the relocation expenses that they would incur.

In 2013, AECOM (as URS) re-contacted each of the utility companies to re-establish contacts and confirm the dispositions. For most utility companies the points of contact had changed and required updated information. The various firms were all interested to know about the project status and an anticipated construction date. AECOM (as URS) coordinated with the firms and developed the relocations depicted in the September 2013 Design Drawings.

## 8.4 Recommended Design and Next Steps

Under the current Task Order, the list of known entities which have services and product lines crossing the Project ROW was updated; the list is current as of the end of December 2020. All utility information provided by the survey data, field visits, or from the impacted companies is summarized in **Appendix G**. The information has been incorporated into the 15% Design plans, including the anticipated utility/pipeline dispositions based on approximate depths; the exact locations will have to be field verified prior to final design and construction.

The plans note that the Contractor shall be responsible for the relocation of their utilities, pipelines, or other infrastructure elements. They further state that the Contractor shall contact “LA One Call” at 1 (800) 272-3020 at least 72-hours in advance of excavation to have underground utilities located. Utility companies that do not participate in “LA One Call” must be contacted directly for location of their facilities. Aerial utilities are present at the proposed Airline Highway crossing and at the shoofly track adjacent to the CN RR. In both locations, re-location of power poles will be necessary.

AECOM intends to continue to pursue contacts at each utility and pipeline, especially those that have not been previously responsive. CPRA has noted that they will aid in this effort throughout the design process to ensure all utilities\pipelines are relocated in a timely manner that will not negatively effect the Project’s construction.

# 9. Hydraulic Modeling

There are three components of the H&H modeling and the requisite H&H related design elements for the Project: 1) Modeling and design of the local drainage features, which includes the existing and proposed infrastructure, the Maurepas Diversion components, and the WSLP flood protection features. 2) Modeling and design of the Maurepas Diversion network, including the River Intake, Sedimentation Basin, Conveyance Channel, and the road and railroad crossings. And 3) Modeling of the water and nutrient dispersion of the discharge from the Maurepas Diversion channel above I-10 into the northern portion of the Maurepas Swamp. Each of these three components is discussed below; details of the H&H modeling of the local drainage features for the Existing and Proposed drainage Conditions are contained in **Appendix D.1** - "Hydrologic and Hydraulic Modeling Report for the Maurepas Diversion and WSLP Project"; details of the H&H analyses conducted for the dispersion into the Maurepas Swamp are contained in **Appendix D.2** - "Water Quality Modeling of Proposed River Reintroduction Into Maurepas Swamp (PO-0029)".

## 9.1 Local Drainage

The construction of the Maurepas Diversion and the WSLP flood protection features will affect the local drainage network. The primary design objective is to maintain the function of the existing St. John the Baptist Parish drainage system, so that no detrimental impacts are incurred due to construction of the Project. This will require the redesign of some of the hydraulic components that comprise the existing local drainage features and the design of new Project specific drainage features.

### 9.1.1 Existing Hydrologic Analysis

As shown in **Figure 9.1**, The Project watershed is bounded to the south by the MRL, to the north by a point 2,500-ft above Airline Highway , to the west by LA-3213 and the Noranda Alumina plant, and to the east by LA-54. The watershed currently drains into the Angelina and Bourgeois Canals, which flow northward into Hope Canal, and ultimately discharge into the Maurepas Swamp. Marathon on-site retention ponds route the entire site's flow under the CN RR into Sugar Mill Ditch, under Airline Highway, and into Hope Canal. The stormwater for the Noranda Alumina property is also self-contained, flowing northward across Airline Highway in independent channels.

As explained in **Appendix D,** ten sub-basins were delineated using available LiDAR data, USGS Quad maps, aerial photography, the National Hydrographic Dataset, and historical site-specific SWMM modeling. The SCS curve number methodology from NRCS TR-55 was used to determine rainfall volumes and losses, yielding the stormwater runoff from each of the ten sub-basins. Applying the Muskingum-Cunge routing methodology in the USACE HEC-HMS software, the peak flows for both the 10-year and 100-year storms were calculated at various points along the flow-paths. Refer to **Appendix D** for the schematic of the sub-basins, junctions, and reaches for the Existing Conditions and the complete output dataset. The existing peak flows at the key junctions are listed in **Table 9.1.**

***Table 9.1 – Peak Flows under Existing Conditions per HEC-HMS***

| Element | 10-yr Peak Runoff (cfs) | 100-yr Peak Runoff (cfs) |
|---|---|---|
| West Junction 1 | 167 | 303 |
| West Junction 2 | 6.6 | 12.7 |
| Junction at Airline Highway | 114 | 199.1 |
| East Junction 1 | 5.5 | 10.5 |
| East Junction 2 | 252 | 470 |
| East Junction 3 | 372 | 706 |
| East Junction 4 | 442 | 844 |

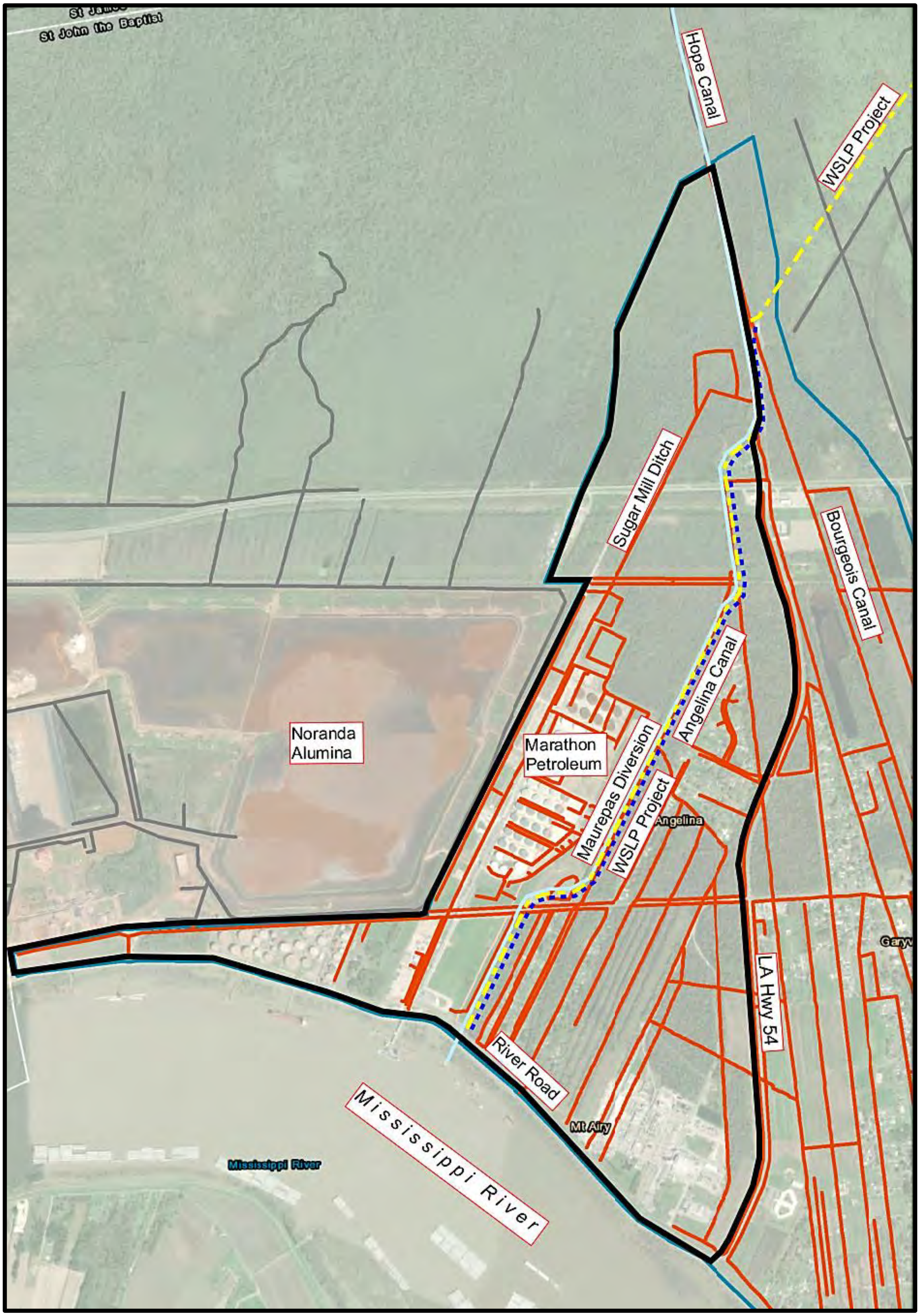


**Figure 9.1 – Maurepas Diversion\WSLP Project Watershed and Significant Features**

### 9.1.2 Existing Hydraulic Analysis

To perform the hydraulic analysis under the Existing Conditions, the LiDAR data and aerial photography collected by C.H. Fenstermaker, LLC for the subject Project was merged with the USACE data from the WSLP Interior Drainage Hydraulic Design Analysis to develop a comprehensive geometric datafile of the existing ground surface and waterways. The HEC-HMS data from the hydrologic analysis was input in a HEC-RAS 2D unsteady state model, assuming a tailwater elevation in the Maurepas Swamp of +1.5-ft NAVD88 as the downstream boundary condition. The model was run to determine the peak water surface elevations at the outfall of the Marathon discharge in the Sugar Mill Ditch into the swamp north of Airline Highway. The output data was plotted as the WSE (Stage in the discharge channel) for the Existing Conditions and the conditions with the Maurepas Diversion flowing at 2,000 cfs (this data is plotted in **Figure 9.2** for both conditions).

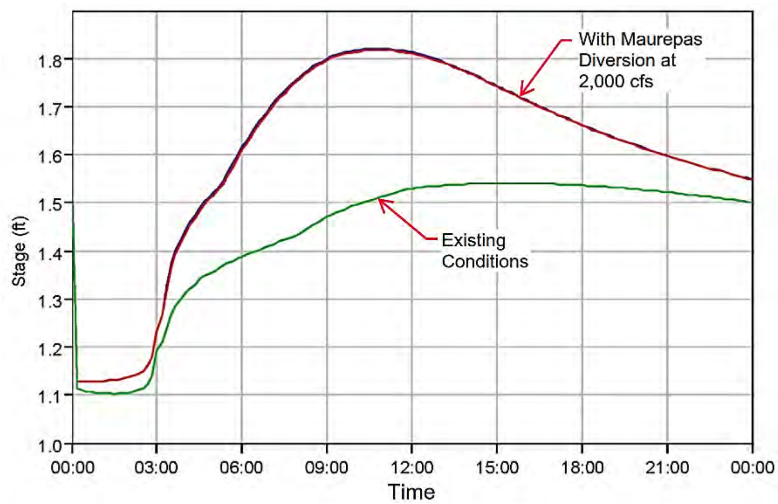


**Figure 9.2 – WSE (Stage) in Sugar Mill Ditch for Existing Conditions and with Maurepas Diversion at 2,000 cfs**

### 9.1.3 Proposed Hydrologic Analysis

The hydrologic analysis for the Proposed Conditions was based on the reconfigured flow patterns due to the blockage of the ditches along the CN RR. This would be caused by the construction of the WSLP Project floodwalls required to support the floodgates (refer to previous Section 7.4). The primary modification was the construction of major drainage ditches on both the east and west sides of the Project from CN RR through Airline Highway. Using the same approach described above, the peak stormwater runoff rates for the Proposed Conditions were calculated; these are compared in **Table 9.2** to the Existing Conditions tabulated above.

***Table 9.2 – Peak Flows under Existing and Proposed Conditions***

| Element | Existing Conditions | | Proposed Conditions | |
|---|---|---|---|---|
| | 10-yr Peak Runoff (cfs) | 100-yr Peak Runoff (cfs) | 10-yr Peak Runoff (cfs) | 100-yr Peak Runoff (cfs) |
| West Junction 1 | 167 | 303 | 167 | 303 |
| West Junction 2 | 6.6 | 12.7 | 173 | 316 |
| East Junction 1 | 5.5 | 10.5 | 5.5 | 10.5 |

| Element | Existing Conditions | | Proposed Conditions | |
|---|---|---|---|---|
| | 10-yr Peak Runoff (cfs) | 100-yr Peak Runoff (cfs) | 10-yr Peak Runoff (cfs) | 100-yr Peak Runoff (cfs) |
| East Junction 2 | 252 | 470 | 177 | 331 |
| East Junction 3 | 372 | 706 | 264 | 495 |
| East Junction 4 | 442 | 844 | 266 | 500 |

### 9.1.4 Proposed Hydraulic Analysis

The sizes of suitable East and West Ditches were estimated using experience and rudimentary calculations based on the 10-year flows. HEC-RAS was again employed to analyze the proposed ditch hydraulics. The final iteration of the ditch geometries, along their various reaches is shown in Table 9.3.

***Table 9.3 – Ditch Geometry by Reach for the 10-yr Flow***

| Ditch Side | Ditch Reach | 10-yr Flow (cfs) | Top Width (ft) | Bottom Width (ft) | Depth (ft) |
|---|---|---|---|---|---|
| East | 1 | 5 | 9 | Swale* | 1.5 |
| | 2 | 167 | 29 | 5 | 4 |
| | 3 | 263 | 37 | 10 | 4.5 |
| | 4 | 266 | 37 | 10 | 4.5 |
| West | 1 | 167 | 29 | 5 | 4 |
| | 2 | 171 | 29 | 5 | 4 |
| | 3 | 247 | 34 | 10 | 4 |
| | 4 | 275 | 37 | 10 | 4.5 |

The proposed culvert sizes underneath the CN RR, KCS RR and Airline Highway were also determined, as shown in **Table 9.4.**

***Table 9.4 – Reinforced Box Culvert Sizes for CN RR, KCS RR, and Airline Highway***

| Ditch Side | Ditch Location | RCB Dimensions |
|---|---|---|
| East | CN RR | 5-ft x 5-ft |
| | KCS RR | 5-ft x 10-ft |
| | Airline Highway | 5-ft x 10-ft |
| West | CN RR | 5-ft x 5-ft |
| | KCS RR | 5-ft x 10-ft |
| | Airline Highway | 5-ft x 10-ft |

Using the above described data for the ditch and RCB geometries, HEC-RAS 2D was again run as an unsteady state model, assuming the same swamp tailwater elevation of EL +1.5-ft NAVD88. The peak WSEs at the Sugar Mill Ditch discharge into the swamp were again calculated. The output peak WSEs for the Proposed Conditions were compared to those of the Existing Conditions, yielding the following results:

| | |
|---|---|
| Peak WSE, Existing Conditions = | +1.47-ft NAVD88 |
| Peak WSE, Proposed Conditions = | +1.80-ft NAVD88 |
| Difference in Peak WSE = | + 0.33-ft |

### 9.1.5 Recommended Design & Next Steps

The small increase in the WSE at the outfall of the Sugar Mill ditch will not cause any significant adverse impacts to the surrounding properties. Thus, the construction of the East and West ditches on each side of the Project alignment is the Recommended Design. However, the design will be revised and optimized during subsequent phases. Due to the concurrence of survey data collection and the modeling process, the latter part of the survey and culvert data collected was not incorporated into the models. Incorporating that additional data will be the first subsequent step to improve the modeling accuracy. Many of the culverts were noted (and documented with reports and photographs) to be partially or nearly completely silted in; recommendations for maintenance to improve the existing local drainage will be made.

## 9.2 Maurepas Diversion Network

### 9.2.1 HEC-RAS Model of Maurepas Diversion

The HEC-RAS 1D model of the Maurepas Diversion Conveyance Channel has not been updated since the 2013 Submittal. The model will be saved and run in the latest version of HEC-RAS 5.0.7, checked for errors, and revised as needed to develop a stable model. The geometry of the Conveyance Channel has changed due to the impact of the WSLP Project on its alignment (refer to Section 6.6 for a detailed discussion of the changes). Revisions to the channel alignment and cross-sections will have to be made in the HEC-RAS model to accurately reflect the revised conditions. Changes to any of the structures at the railroad and\or roadway crossings will also be incorporated. The validity of the assumptions of the boundary conditions used in the previous model will also be checked.

### 9.2.2 Effect of River Stage and Sea Level Rise

Two phenomena have occurred over the recent past that will impact the performance of the Maurepas Diversion: 1) the stage of the Mississippi River has been higher for longer over recent years, and 2) the global sea level is rising. Once the HEC-RAS model of the diversion has been updated, its ability to deliver the target flow rate throughout the year and over the lifetime of the Project will be investigated. The hydrologic cycle, i.e., the amount, distribution, and timing of precipitation events, is expected to intensify with global warming. This is generating debate in the meteorological community over whether the standard storm events, based on decades of historical data, should be revised to account for climate change. The increase in the intensity of extreme precipitation events and the risk of flooding will have hydrologic impacts across the globe.

The observed trend in the river stage data has been attributed to earlier and more significant snowmelt along with the increased intensity of rainfall during storm events. The elevated river stages will increase the head available to drive the flow of water through the diversion, so that effect is anticipated to improve the water delivery capacity. On the other hand, global sea level rise (SLR) will affect the WSE in the Maurepas Swamp during extreme storm events, thus increasing the tailwater elevation at the discharge end of the diversion channel. This will impede the delivery capacity of the diversion. This phenomena and their effects will be incorporated into the diversion model in subsequent design phases.

## 9.3 Dispersion of Diversion Discharge

In the early 2000's, AECOM (as URS) developed a 2D model using the ADvanced CIRCulation (ADCIRC) software to support the hydraulic design of the Maurepas Diversion and evaluate its effect on The Maurepas Swamp hydrology. Taking advantage of the significant advances in H&H modeling software and computing power, in 2018 AECOM (under the auspices of CPRA) contracted FTN Associates, Ltd. (FTN) to model the water and nutrient transport using the latest Delft3D software, currently considered the state-of-the-art. The resulting FTN report, "Water Quality Modeling of Proposed River Reintroduction into Maurepas Swamp (PO-0029)" is included as **Appendix D.2**. The purpose of the modeling efforts was to simulate the dispersion of water and the transport of nutrients into the Maurepas Swamp.

Simulations were run to predict the WSEs, velocities, and nutrient dispersion throughout the swamp with the flow control measures (weirs in Bayou Secret \ Bourgeois Canal and cuts in the embankment) in place. The simulations showed that after the MR water exits the diversion its flow rate greatly exceeds the capacity of Hope Canal, causing the water to flow into the swamp and spread west as far as Blind River, east as far as Reserve Relief Canal, and north into the swamps along Dutch Bayou.

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 river water reaches Lake Maurepas, over half of its nitrogen and a third of its phosphorous have been absorbed by the swamp. Thus, the swamp is up-taking the nutrients as desired, while not delivering such a significant load of nitrogen that major algae blooms would occur in either Lake Maurepas or Lake Pontchartrain. Based on the simulations, the proposed diversion is expected to provide beneficial freshening and nutrient delivery to a large area of the Maurepas Swamp.

## 9.4 Hydrologic and Hydraulic Modeling Plan

The H&H Plan for the subsequent phases of design will consist of three sections; the tasks for each are outlined in the following sections.

### 9.4.1 Local Drainage Design

- Incorporate new survey data, including culverts into local drainage model
- Review alternatives for ditch routing
- Optimize ditch sizing

### 9.4.2 Maurepas Diversion Channel Modeling

- Input new geometry into Conveyance Channel HEC-RAS model
- Optimize Conveyance Channel geometry, as needed
- Evaluate head-losses and determine flow delivery capacity
- Incorporate effects of changes in River Stage and effects of SLR

### 9.4.3 Water and Nutrient Dispersion Modeling

- Re-run model incorporating new Diversion Channel geometry
- Use results of Delft3D model to aid in development of OMMAM Plan

# 10. Construction Considerations

## 10.1 Contract Reaches Plan

The combined Maurepas and WSLP construction project is broken into Contract Reaches (areas) in order to aid the conception of separate construction packages. These possible Contract Reaches can be defined effectively by construction access points.

These reaches will allow bidding the work in smaller sections, if required, allowing more construction work to be done simultaneously, allowing similar tasks to be grouped under one contract, minimizing work overlap, and providing opportunity for small or large contractors to successfully complete for the work. In addition to site access, packages are compiled based on available staging areas, geotechnical reaches, magnitude and type of work required,  and other logistic considerations. The USACE and CPRA may use this recommendation, together with schedule, programmatic, and budget constrains to make a decision on the construction process.

WSLP Hydraulic Reaches are not a deciding factor in this process, as the hydraulic requirements are currently the same for subject WSLP Reaches 111, 112, and 113 and this criteria has little bearing on the construction process.

The work required at the proposed Reaches is similar across the Maurepas Diversion and WSLP features; both are primarily civil\structural projects with similar earthwork, concrete and pile driving requirements.  Therefore, it does not appear necessary to separate the two contractually within each location. In fact, it may be a hindrance to split Maurepas and WSLP items in locations such as the rail crossings because it increases the possible number of contractors engaging with private entities and also complicates construction coordination.

## 10.2 Proposed Contract Reaches

Division of construction into the following Contract Reaches will reduce the chances of contractors overlapping in space, may improve coordination of haul routes and construction access points, and it is possible that phased construction may allow for the re-use of the same staging areas. Construction could be divided into five (5) Reach areas defined by construction access points and are separated as follows:

- Maurepas Intake System and River Road Crossing
- WSLP River Road to CN RR
- North of CN RR to Airline
- Airline Hwy
- Airline Hwy to I-10

### 10.2.1 Maurepas Intake System and River Road Crossing

This Contract Reach includes the full Maurepas Intake system, from the Intake Channel to the final weir of the Sedimentation Basin, and also includes the construction of River Road impacted by the Maurepas diversion project.  It is possible, and may be advantageous to budget and schedule, to build the Maurepas Diversion and WSLP features in this area separately. Major construction items include:

- Mississippi River Cofferdam
- Maurepas Intake, Headworks, Culverts, and Outflow Structures

- Maurepas Channel features such as the non-structural Intake and Outflow Channels, Sedimentation Basin, Sedimentation Basin Weirs, and the portion of the Conveyance.
- Maurepas western Guide Levee and Access Roads.
- All River Road detours, temporary roads, reconstruction, and other required road work
- Significant TRS requirements.

### 10.2.2 WSLP River Road to CN Railroad

Major construction items for this reach include:

- At River Road:
  - WSLP MRL Tie-in, River Road Rolling Gate and T-wall Monoliths (these features may also be included with the Maurepas Headwork Intake and River Road Reach above).
- Between River Road and CN RR:
  - WSLP I-wall and adjacent drainage ditch from the River Road Crossing to the tie-in for the CN Crossing T-walls
  - Maurepas Conveyance channel from the sedimentation basin to the start of CN crossing riprap.

### 10.2.3 CN RR Crossing:

This area contains only the work related to the Maurepas Diversion and WSLP CN Railroad Crossings. Major construction items include:

- Maurepas Concrete Box Culverts and the riprapped Conveyance Channel transition sections beyond
- WSLP T-walls, Rolling Gate Crossing, I-wall and sheet pile transitions.
- Shoofly for temporary relocation of CN service
- Deconstruction and Reconstruction of the rail lines
- Installation of new drainage ditches and their associated culverts

Some portions of the work may be required to be done by CN Railroad personnel. The contractor(s) involved in this phase will need to coordinate all work with CN in addition to their CPRA, USACE, and private clients.

### 10.2.4 North of CN Railroad to Airline Highway

This Contract Reach contains the work related to the Maurepas Diversion and WSLP north of CN RR, including the KCS RR Crossings, through to the south limits of the Airline Hwy Crossing. Major construction items include:

- North of CN RR to South of KCS RR
  - WSLP Levee and adjacent drainage ditch
  - Maurepas Conveyance channel
- At KCS Crossing

If a bridge is retained as the provided Maurepas Diversion feature, the work of constructing the bridge foundation, substructure and superstructure will be performed by KCS Railroad Personnel. Adjustments to the channel, riprap placement, and other miscellaneous work items may be performed by either the Conveyance Channel contractor or the contractor chosen for WSLP KCS work. If a culvert is chosen, the Reach begins and ends where the Conveyance Channel transitions to riprap beyond the concrete culvert structure. Major construction items include:

- WSLP T-walls, Swing Gate Crossing, and sheet pile transitions.
- Deconstruction and Reconstruction of the rail lines
- Installation of new drainage ditches and their associated culverts
- (if required) Maurepas Concrete Box Culverts and the riprapped Conveyance Channel transition sections beyond or Rail Road Bridge.
- (if required) Shoofly for temporary relocation of KCS service

Portions of the work may be required to be done by KCS Railroad personnel. The contractor(s) involved in this phase will need to coordinate all work with KCS in addition to their CPRA, USACE, and private clients.

- North of KCS RR to South of Airline Hyw.
  - WSLP Levee and adjacent drainage ditch
  - Maurepas Conveyance channel

### 10.2.5 Airline Highway Crossing

Work in this Reach is specific to the Maurepas Diversion and WSLP crossing of Airline Highway. Regarding Maurepas, the Reach begins and ends where the Conveyance Channel transitions to riprap beyond the concrete culvert structure. The proposed WSLP Alternative is an embankment fill road raise; the edges of this Reach will occur at a reasonable point beyond this raise where the Conveyance Channel and WSLP Levee have returned to a typical cross-section. Major construction items include:

- Maurepas Concrete Box Culverts and the riprapped Conveyance Channel transition sections beyond
- Raising Airline Hwy. Significant amounts of embankment earthwork. Some MSE or similar retaining walls may be required.
- Drainage Culverts.
- Access and service roads to adjacent properties

### 10.2.6 Airline Highway to End of Project

Work in this Reach includes all features of both projects north of Airline Highway. Major construction items include:

- Maurepas Conveyance channel from Airline Highway to its terminus north of I-10; this include the crossing under I-10. Conveyance Channel work: dredging the existing channel to the proposed Conveyance Channel geometry, building guide levees with access roads, constructing drainage ditches, and placing small structures (e.g. check valves) or scour protection
- Construction of the WSLP Levee and adjacent drainage until it ties into the planned USACE Pump Station near Airline Highway
- Weir installation at Bayou Secret and Bourgeois Canal
- Embankment cuts within the Maurepas Swamp

## 10.3 Temporary Structures Planning

As part of this Task Order, AECOM has identified locations that will require Temporary Retaining Structures (TRS), dewatering systems, and construction platforms during construction.

These items have been designed to a minimum 15% feasibility level.

### 10.3.1 Maurepas Headworks

The most significant temporary structure at the Headworks is the Mississippi River Cofferdam. This item is described in detail in Section 6.3.

Construction of the Headworks structures is phased from north to south (inland to river) and includes three proposed temporary retaining structures. The first is a U-shaped length of sheet pile surrounding Headworks Culvert C-4. Next, a straight sheet pile wall will be driven south of Headworks monolith U-3. Lastly, as construction moves towards the river a sheet pile TRS box will be constructed around the proposed Headworks monoliths U-2 and U-1. All TRS structures have a top elevation of 14.0 NAVD88. The U-shape TRS around C-4 and box TRS around U-2 and U-1 have tip elevations of -35.0; tip elevation of the straight wall south of U-3 is EL -15.0. This site will require dewatering efforts throughout construction, as the bottom of the excavation ranges from EL -4.0 to EL -13.0. Dewatering will occur in sections that follow the Construction and TRS movement from north to south.

### 10.3.2 Maurepas Crossing of CN Railroad

A combination pipe wall (combi-wall) retaining system is proposed for the construction effort at CN RR. A shoofly will be installed to the north of the existing CN lines. In order to take full advantage of the work area created to the south, the combi-wall is placed in an L-shape north of proposed culvert C-4 and retains the temporary railroad embankment. The top of the wall is at EL 11.5 and the combi-wall components are driven to a tip elevation of -75.0 NAVD88. After construction a large portion of this steel wall will remain in place with top elevation cut down to EL 2.75.

### 10.3.3 Maurepas Airline Highway Culverts

Airline highway construction is organized into two phases split along the existing median centerline. A straight sheet pile wall will be driven along this boundary to allow for construction of the northern half of culverts. The sheet pile wall has a top elevation of 8.0 and a tip elevation of -65.0 NAVD88.

The Design Team anticipates that this item will need new analysis when the Airline Highway crossing embankment is finalized. The sheet pile wall will likely be replaced with a combi-wall similar to the type used at CN RR.

### 10.3.4 Temporary Staging Area / Construction Platform in MR

As discussed in Section 6.1.2.2, new Marathon docks and pipe bridges have blocked access to the Maurepas Headworks construction site via the Mississippi River. In order to avoid high projected costs of overland transport of heavy construction materials, a temporary staging and delivery platform is proposed for the batture area of the MRL just downstream of the Marathon structures. This item is highly dependent on how the contractor’s work plan and schedule (contractor may not use a temporary platform) and is therefore considered a Contractor Means and Methods item.

### 10.3.5 Considered Locations

The locations described above are the only anticipated TRS, dewatering, or construction platform locations. The following areas were also examined during this review:

– WSLP River Road Crossing: Open cut construction anticipated

- WSLP CN RR Crossing: Construction of floodwall monoliths will be phased in conjunction with the shoofly and Maurepas culverts. Open cut construction anticipated in both phases.
- Maurepas & WSLP KCS Crossings: The Maurepas KCS Bridge will need minimal excavation beyond shaping the channel and installing wingalls. This work and the construction of the KCS swing gate and floodwall monoliths anticipated as open cut.
- Relief Valves: 24” steel pipes will be installed through the guide levee. Pipes will terminate in a concrete discharge box below existing grade. Open cut construction is anticipated. Rough-Order-of-Magnitude Cost Estimate

# Delft 3D Hydraulic and Water Quality Modeling of Proposed River Reintroduction into Maurepas Swamp (PO-0029)

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

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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.

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# 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.

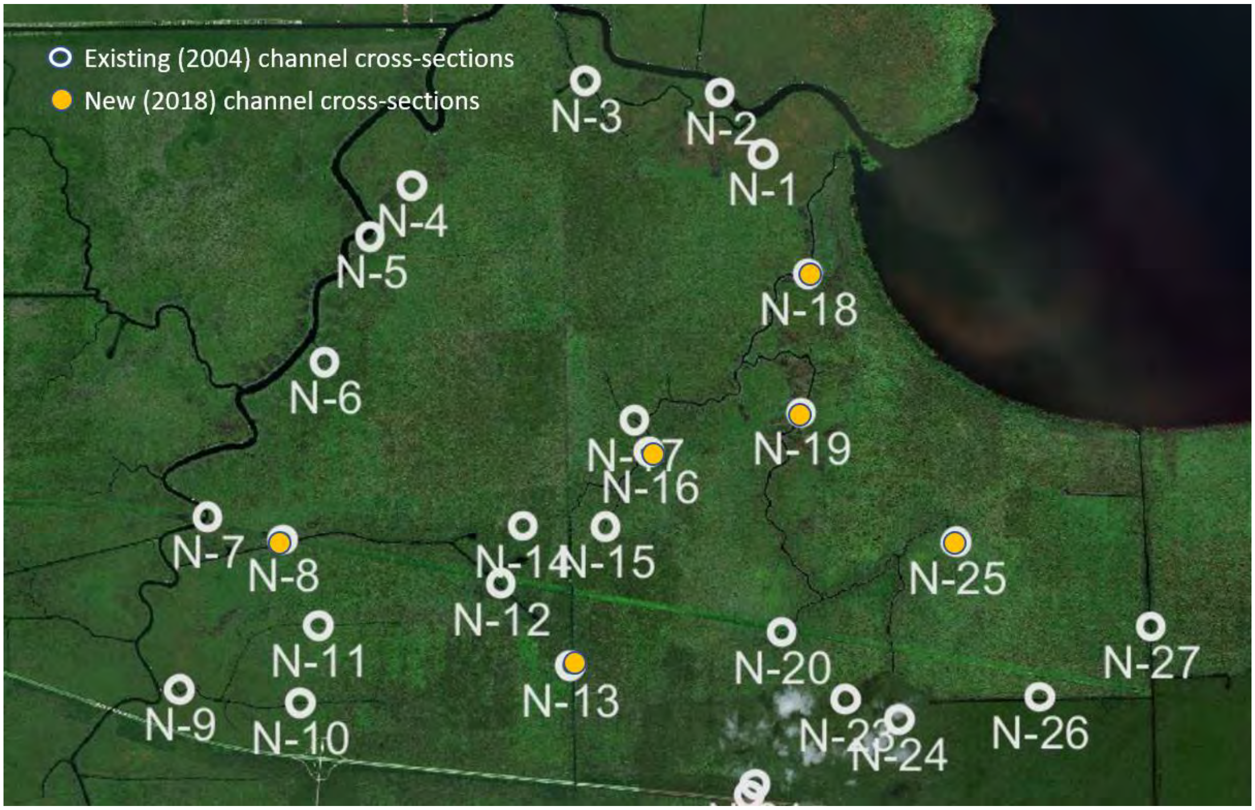
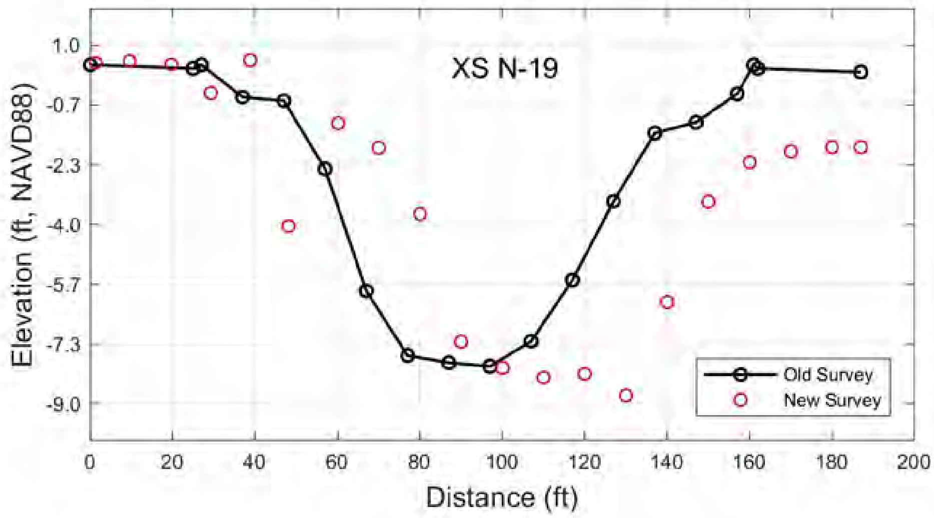



Figure 5.1. Locations of existing (2004) and new (2018) channel cross-section field surveys.



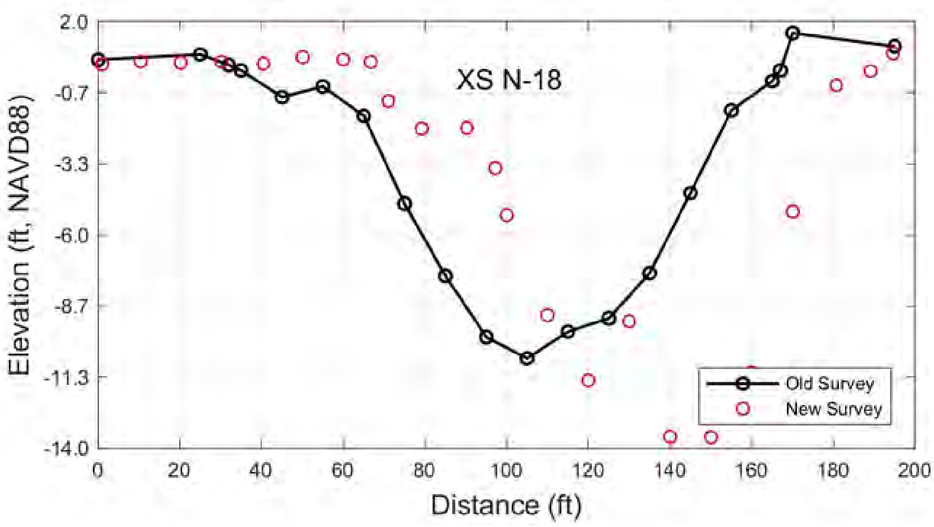




Figure 5.2. Comparison of old (2004) and new (2018) channel cross-sections at N-19 and N-18.

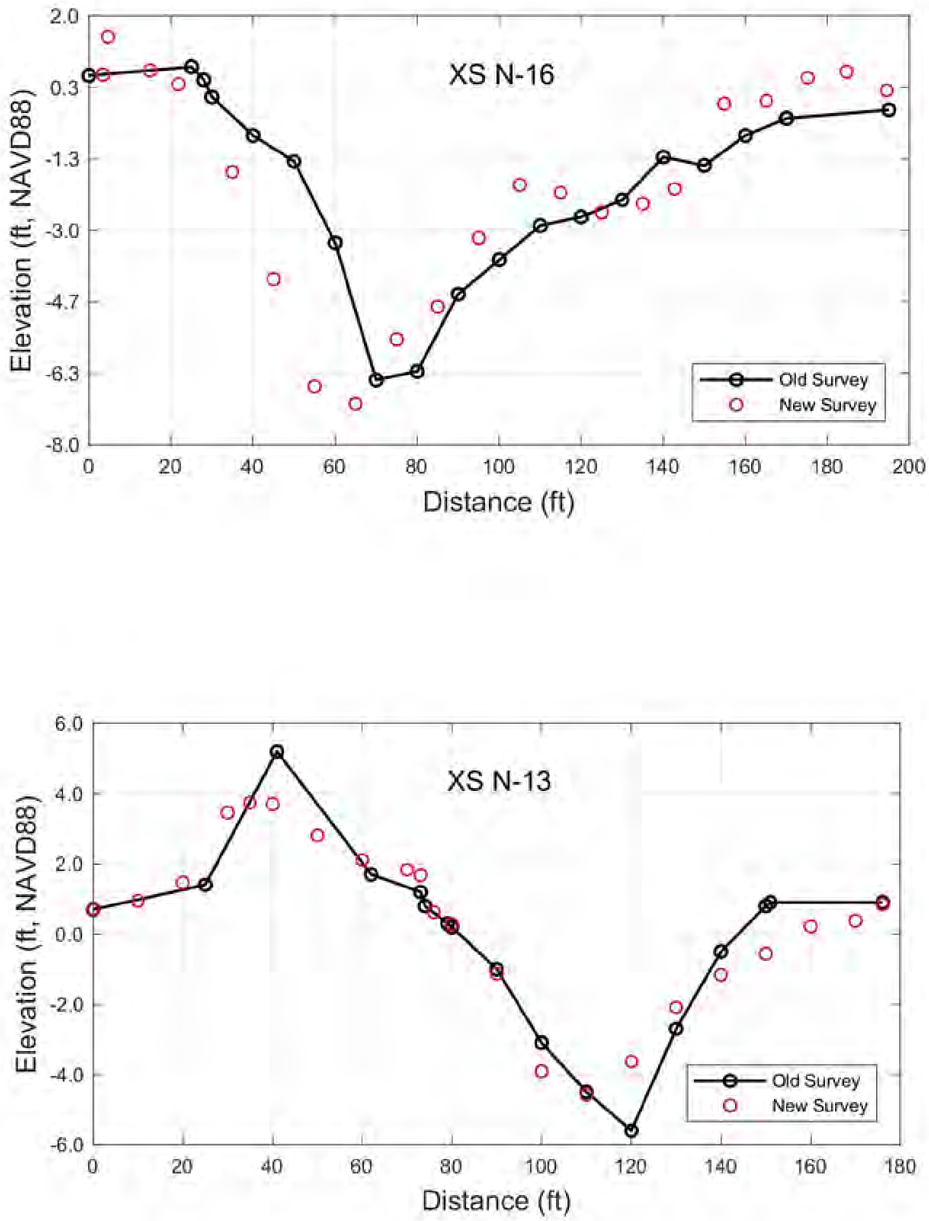
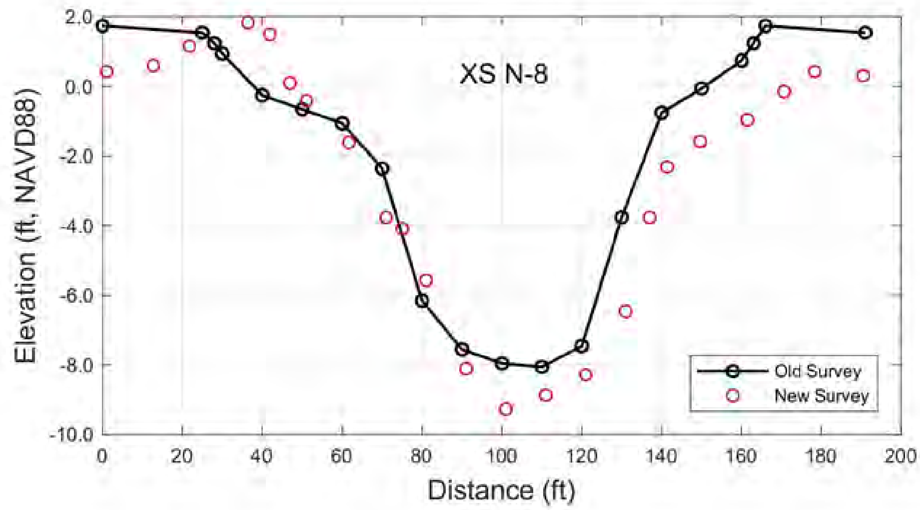



Figure 5.3. Comparison of old (2004) and new (2018) channel cross-sections at N-16 and N-13.



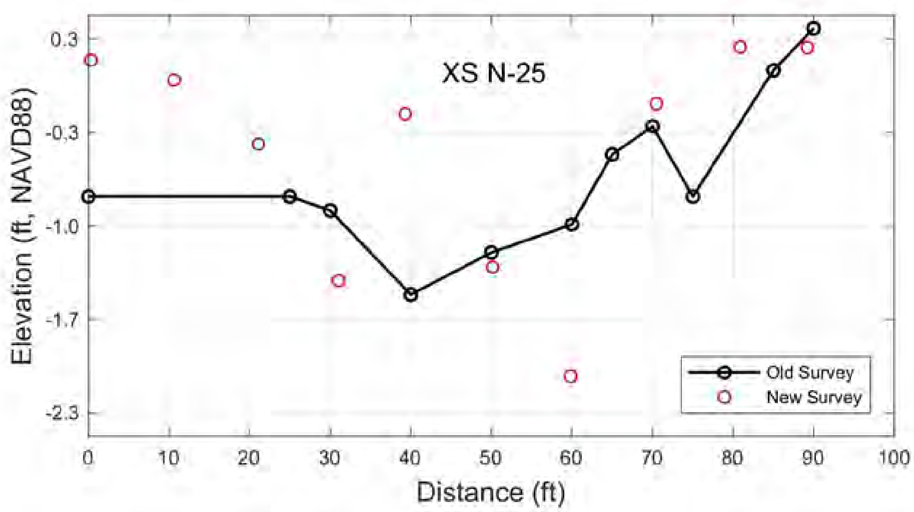




Figure 5.4. Comparison of old (2004) and new (2018) channel cross-sections at N-8 and N-25.

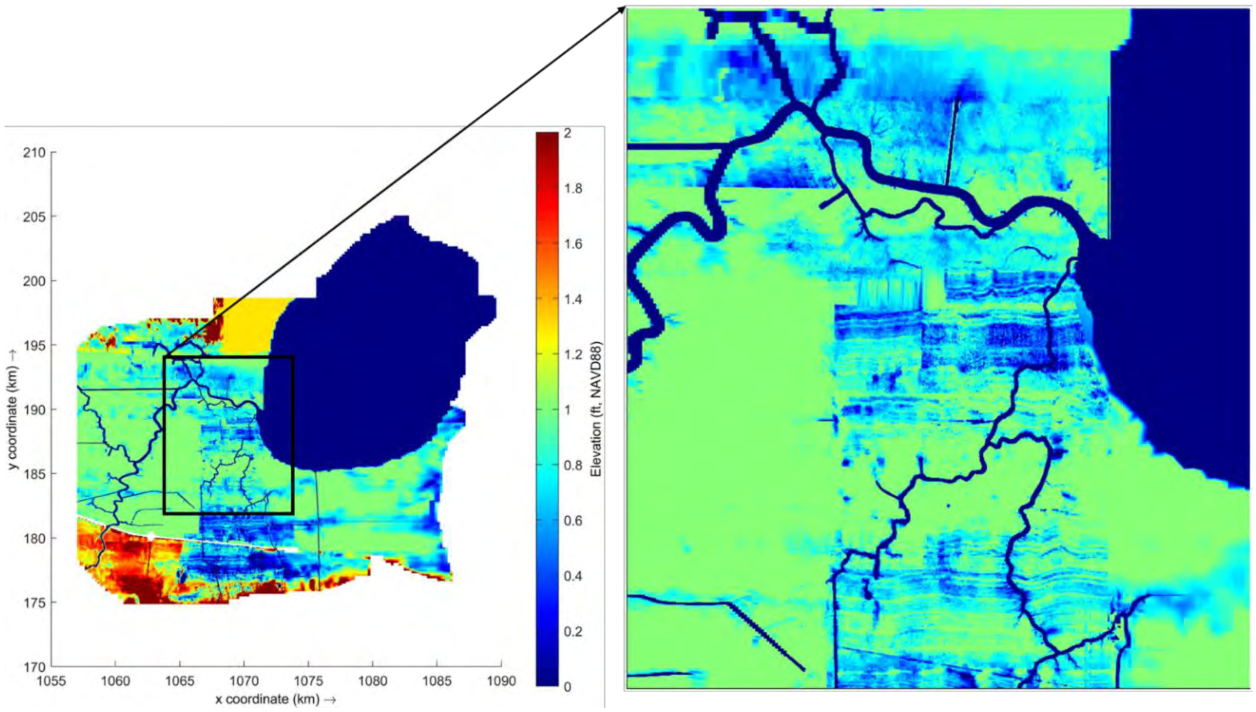



Figure 5.5. Delft3D model bathymetry using topographic contours from 2012 LIDAR data. Swamp floor elevation capped at 1.0 ft in the region shown by the inset.

## 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.

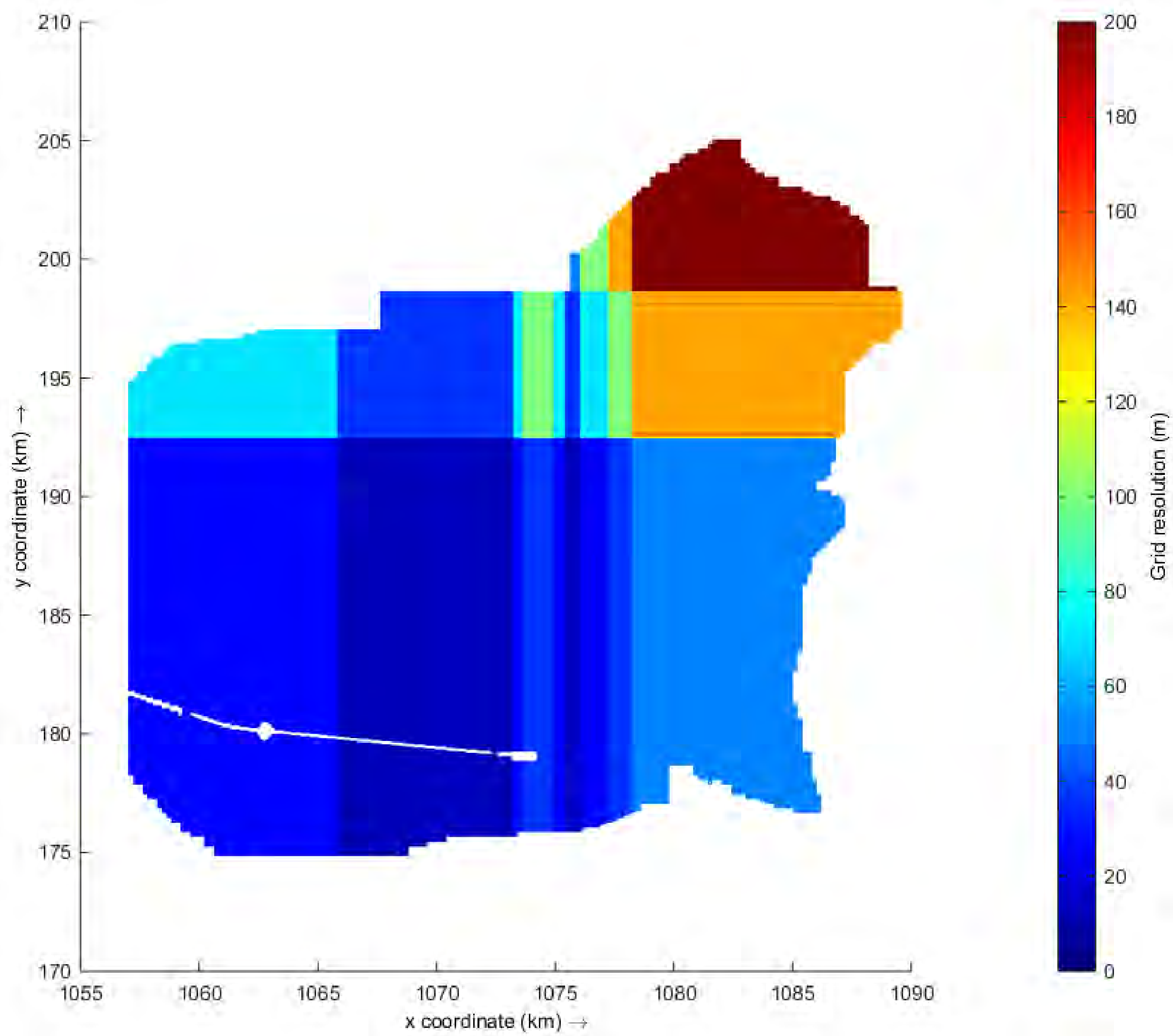



Figure 6.1. Maurepas swamp Delft3D model grid resolution.

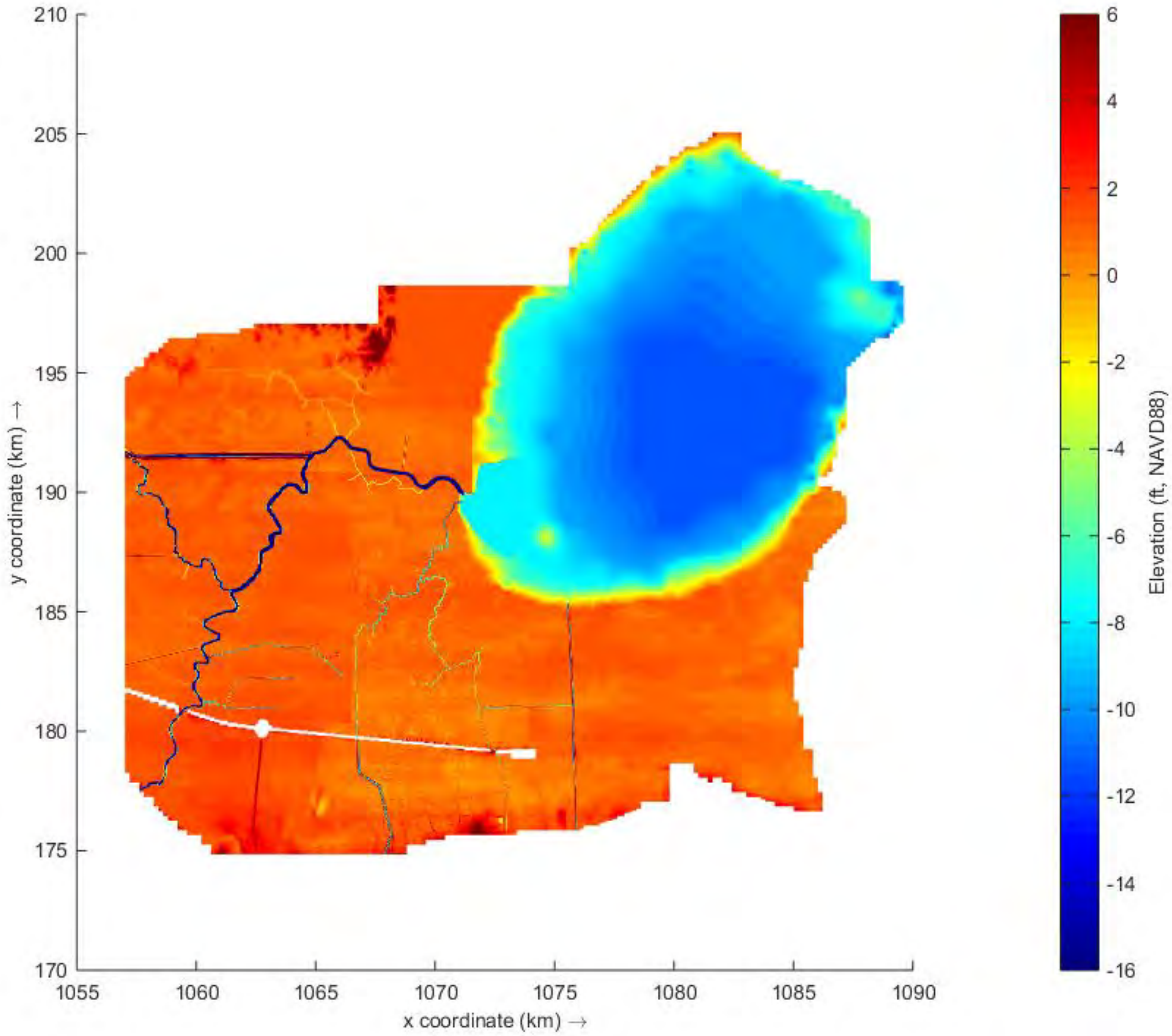



Figure 6.2. Maurepas swamp Delft3D model bathymetry.

# 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:

FWOP conditions: Year 50 *without* the diversion project

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.

FWP conditions: Year 50 *with* the diversion project

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.

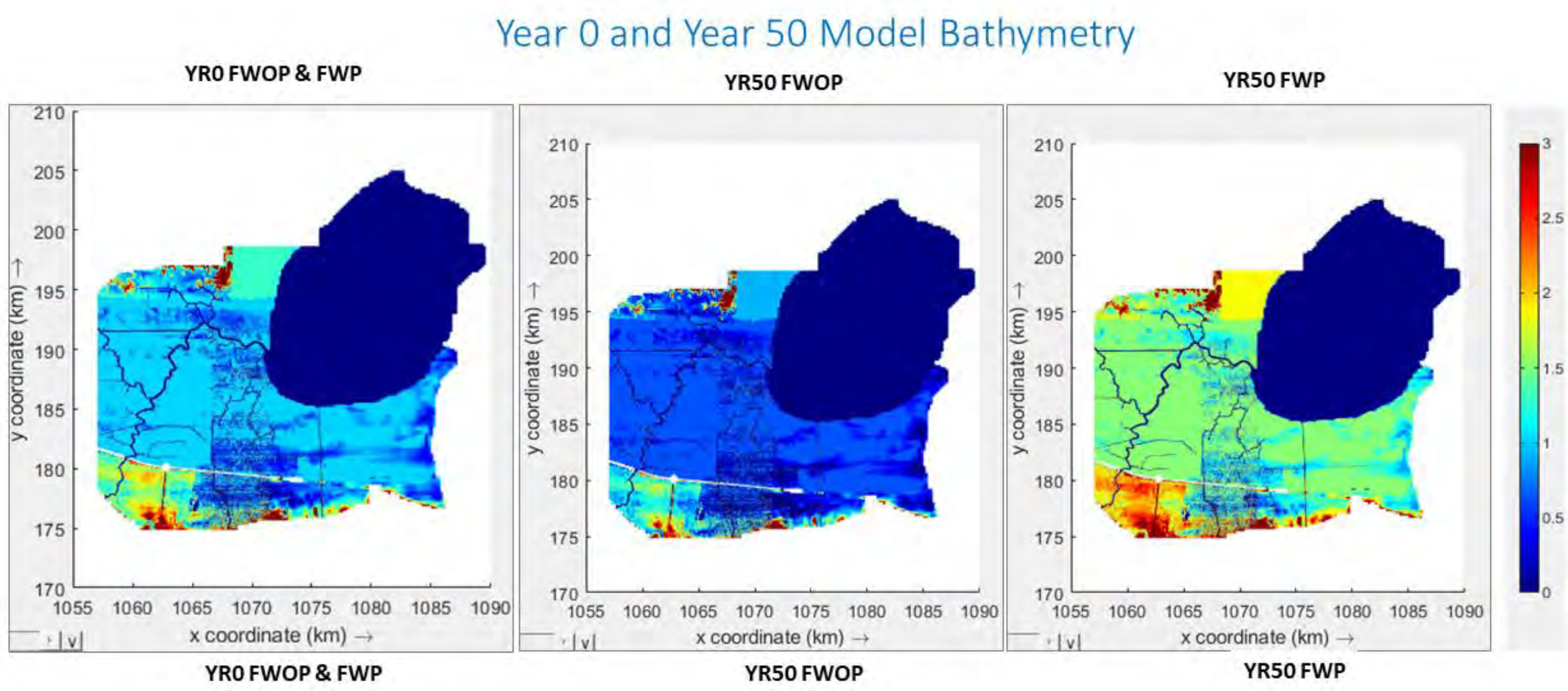



Figure 9.1 Delft3D model geometries used to the production runs.

## 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<br>0.25 mg/L TP | 1.61 mg/L TN<br>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<br>0.13 mg/L TP | 0.38 mg/L TN<br>0.03 mg/L TP | Median values for LDEQ station 1106 (Tickfaw River near Lake Maurepas) for \|2001 – 2015 |
| Amite River<br>(old channel) | 0.86 mg/L TN<br>0.12 mg/L TP | 0.26 mg/L TN<br>0.02 mg/L TP | Median values for LDEQ station 0228 (Amite River at mile 6.5, at Clio) for<br>2001 – 2018 |
| Amite River Diversion Canal | 0.86 mg/L TN<br>0.13 mg/L TP | 0.26 mg/L TN<br>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<br>0.24 mg/L TP | 0.73 mg/L TN<br>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<br>0.11 mg/L TP | 0.16 mg/L TN<br>0.01 mg/L TP | Median values for Station 5 (Mississippi Bayou) from<br>Rob Lane’s 2002 – 2003 data |
| Reserve Relief Canal | 0.79 mg/L TN<br>0.13 mg/L TP | 0.19 mg/L TN<br>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<br>0.10 mg/L TP | 0.30 mg/L TN<br>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.

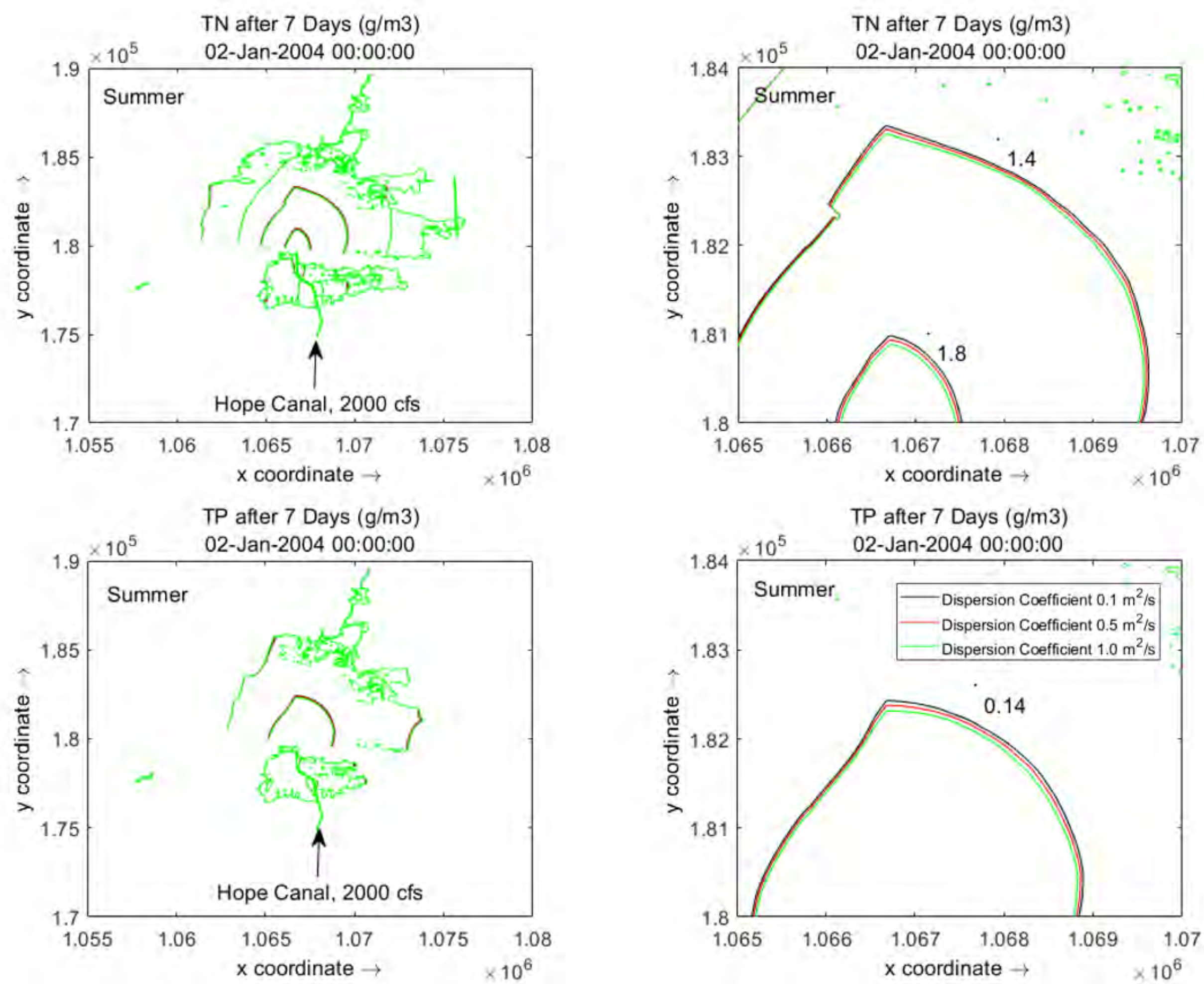



Figure 9.3. Contours of TN and TP from the diffusion coefficient sensitivity simulations.

# 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).

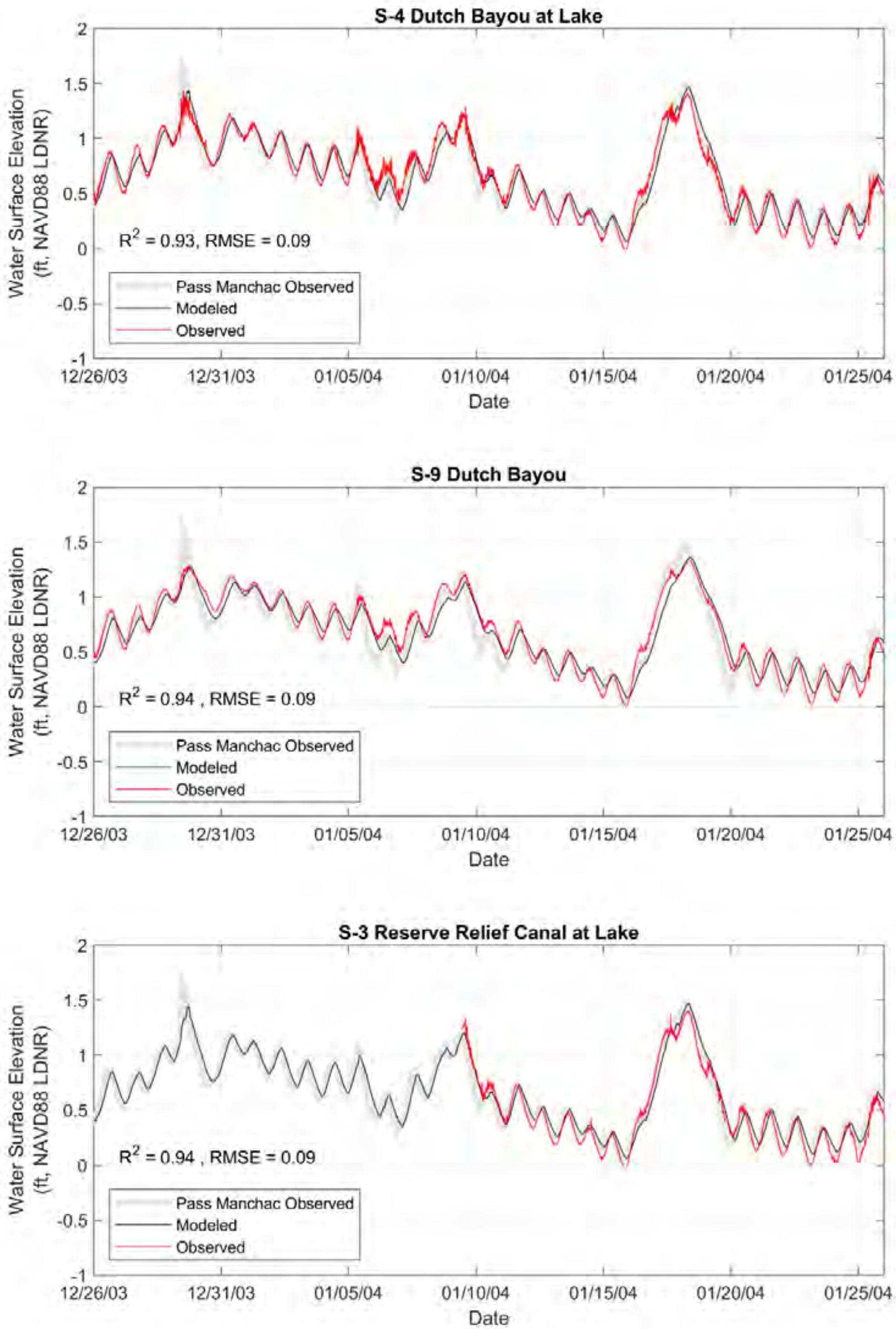



Figure A2. Observed and predicted water surface elevations at gages S-4, S-9 and S-3 under normal conditions.

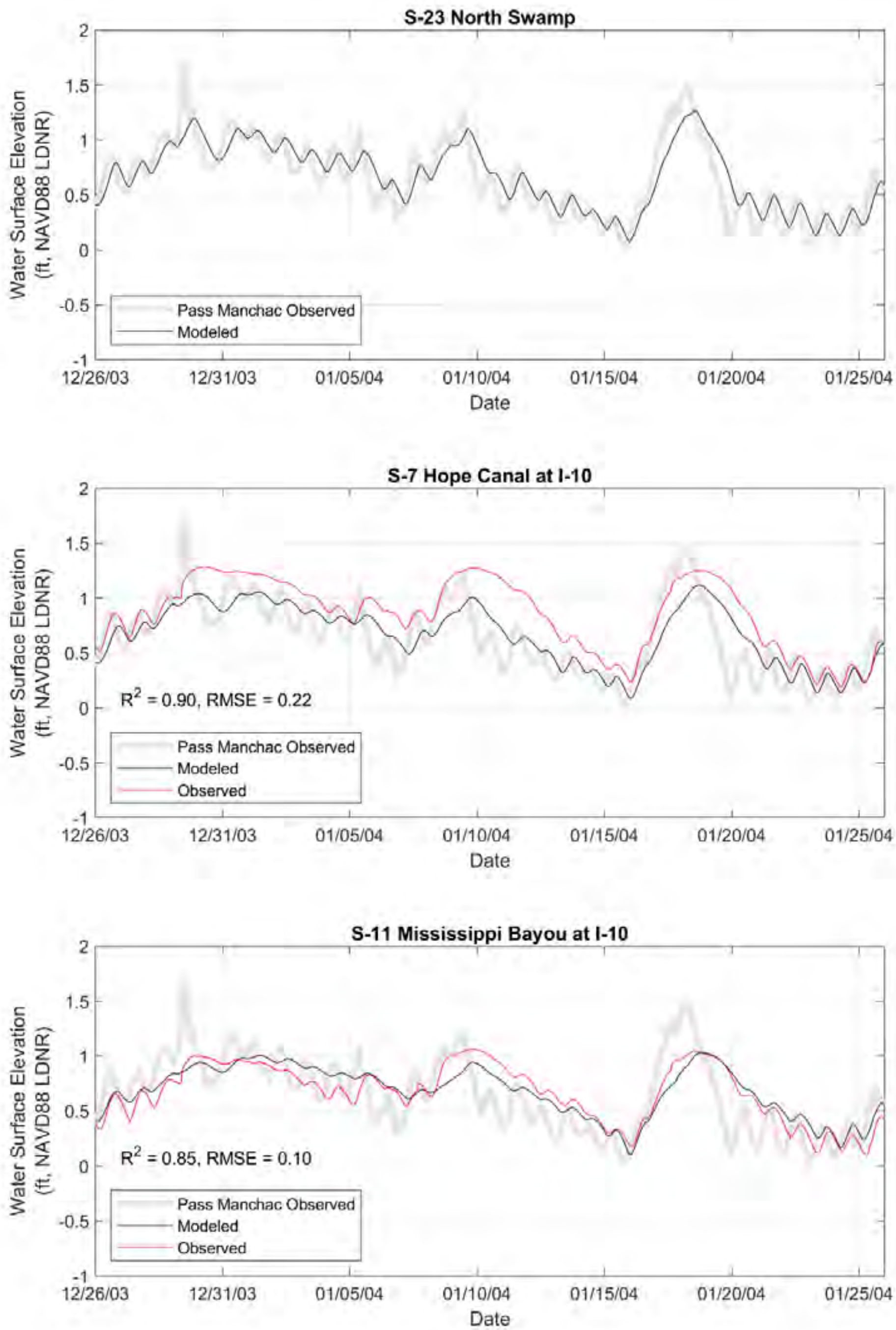



Figure A3. Observed and predicted water surface elevations at gages S-23, S-7 and S-11 under normal conditions.

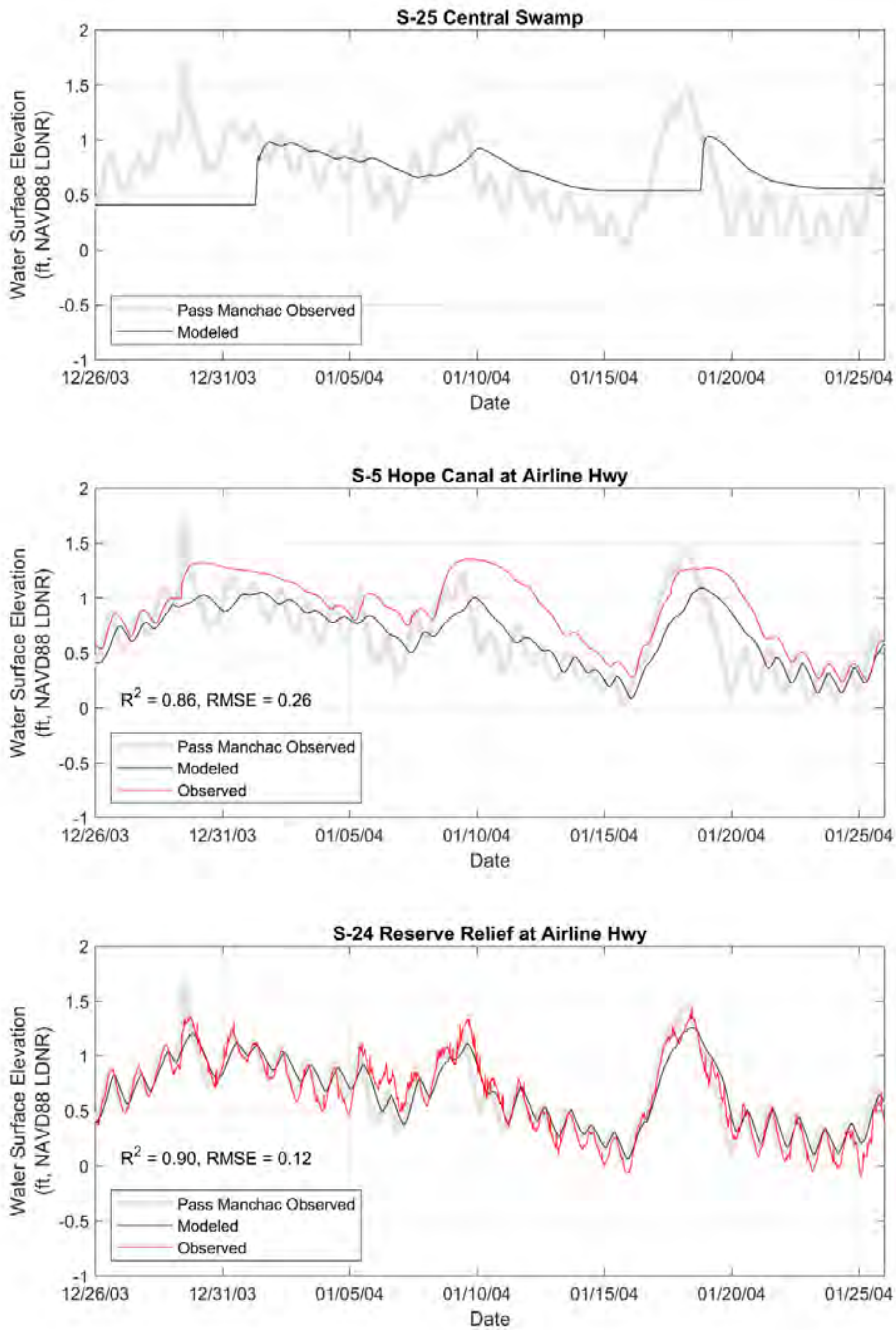



Figure A4. Observed and predicted water surface elevations at gages S-25, S-5 and S-24 under normal conditions.

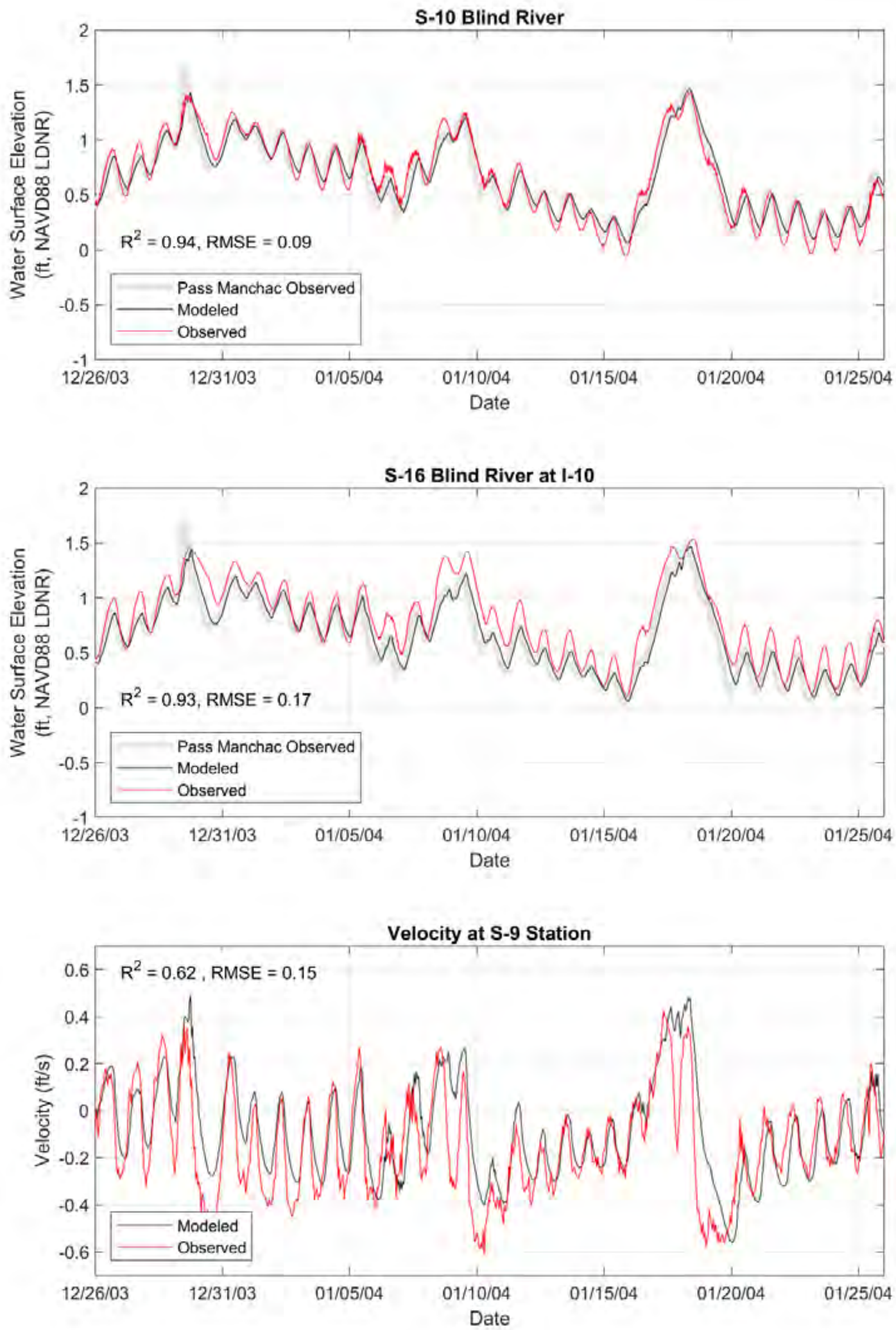



Figure A5. Observed and predicted water surface elevations at gages S-10, S-16 and velocity at S-9 under normal conditions.

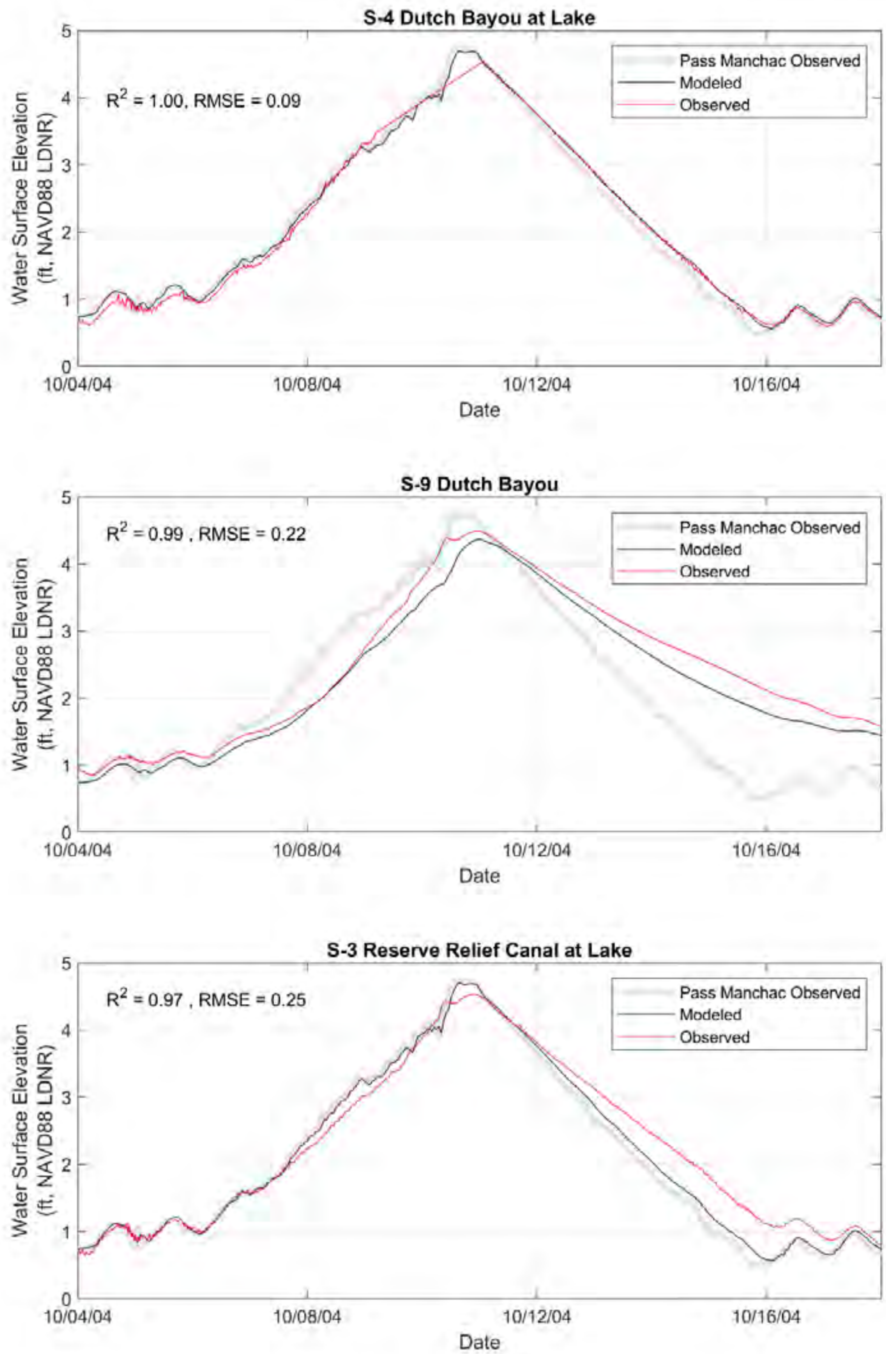



Figure A6. Observed and predicted water surface elevations at gages S-4, S-9 and S-3 under tropical storm conditions.

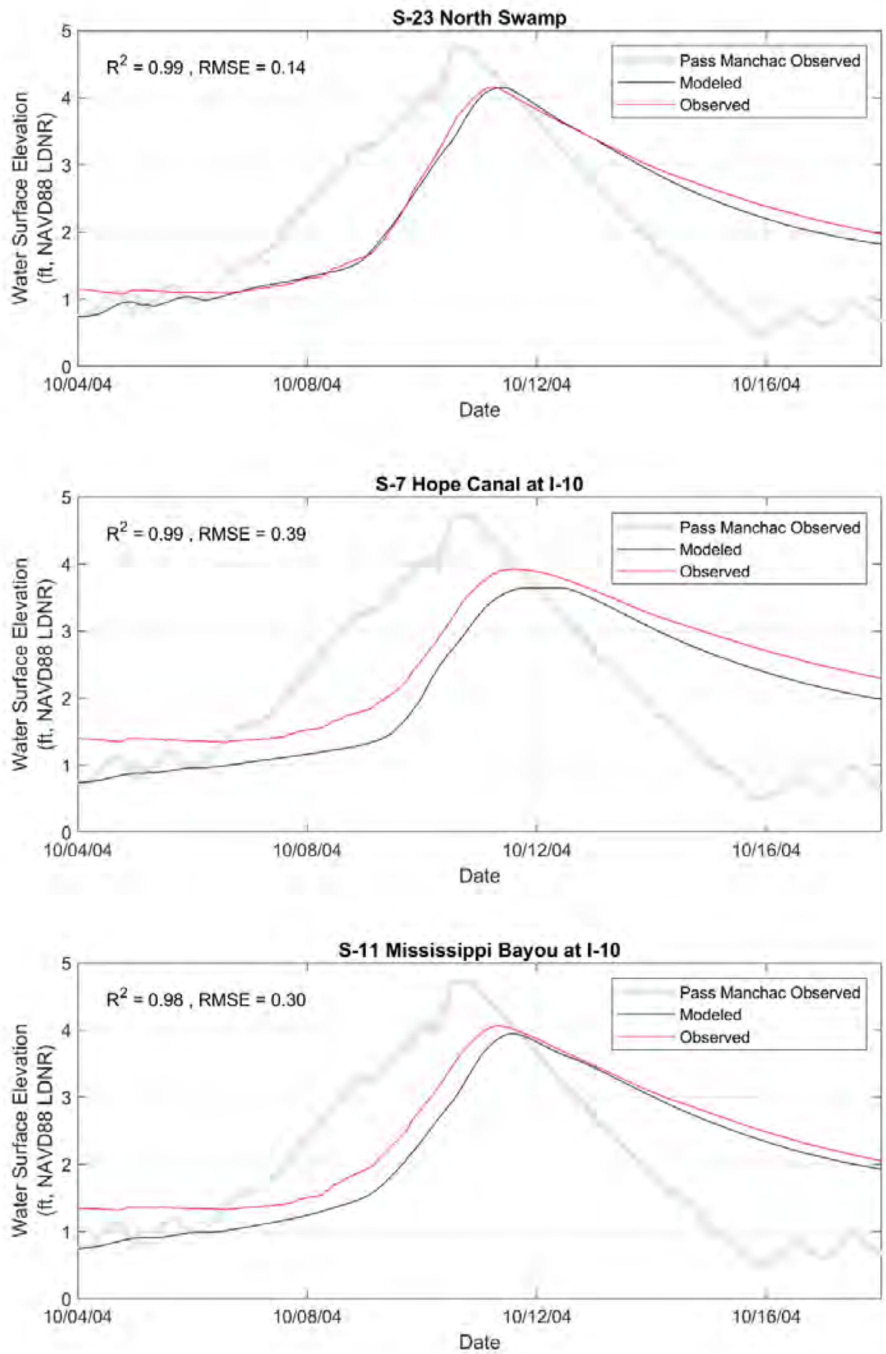



Figure A7. Observed and predicted water surface elevations at gages S-23, S-7 and S-11 under tropical storm conditions.

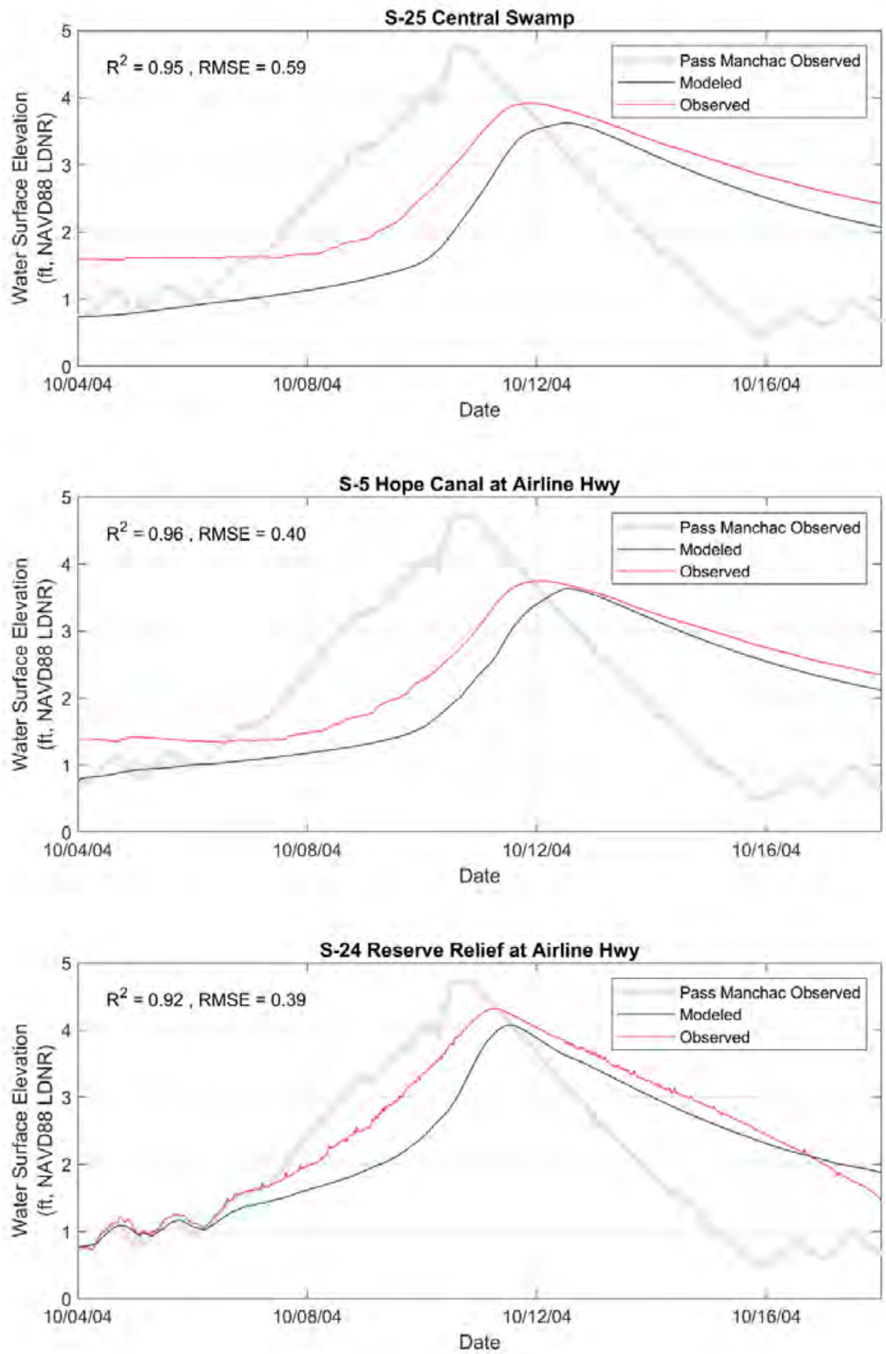



Figure A8. Observed and predicted water surface elevations at gages S-25, S-5 and S-24 under tropical storm conditions.

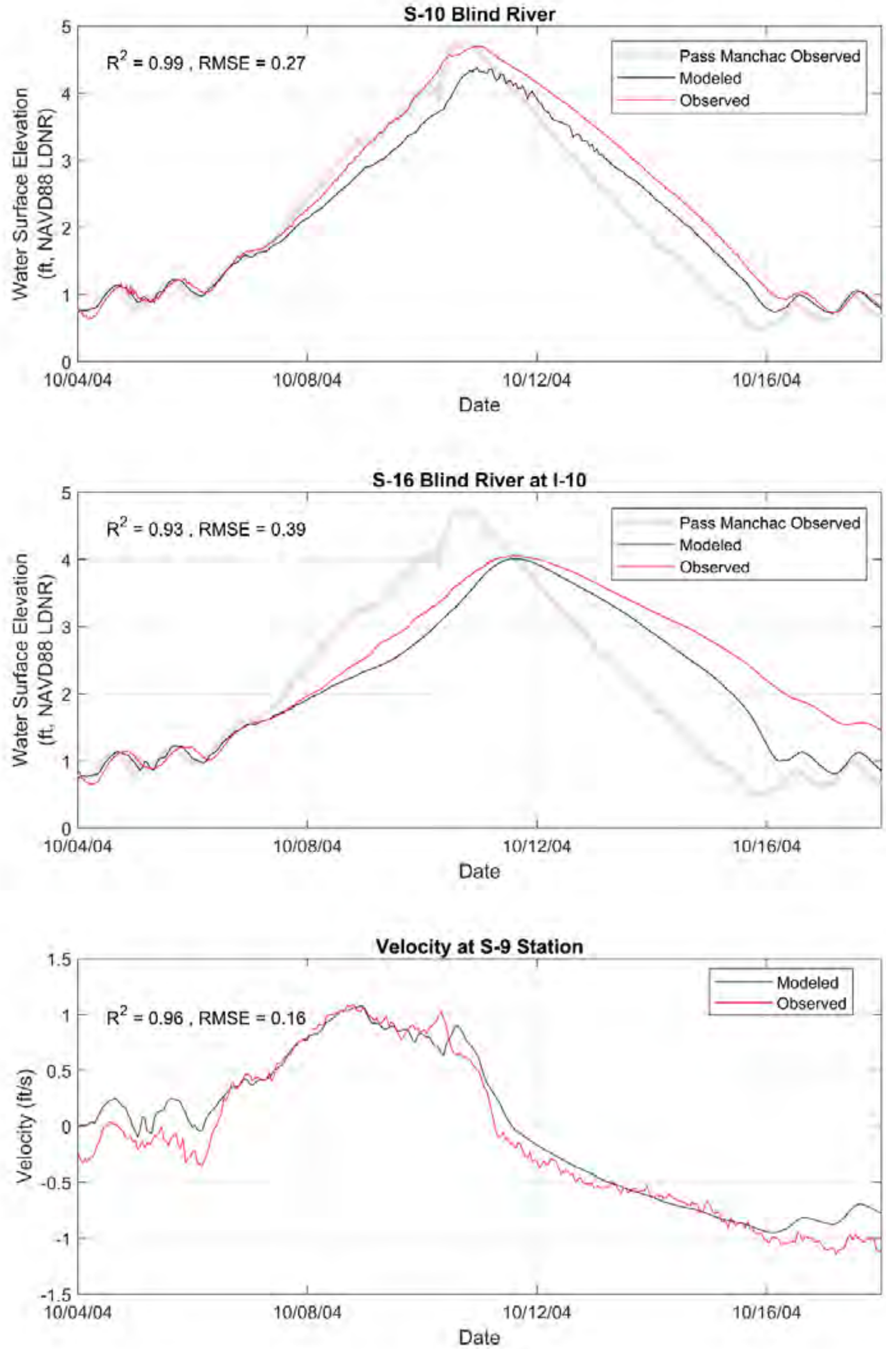



Figure A9. Observed and predicted water surface elevations at gages S-10, S-16 and velocity at S-9 under tropical storm conditions.

# APPENDIX B

## Model Validation Results

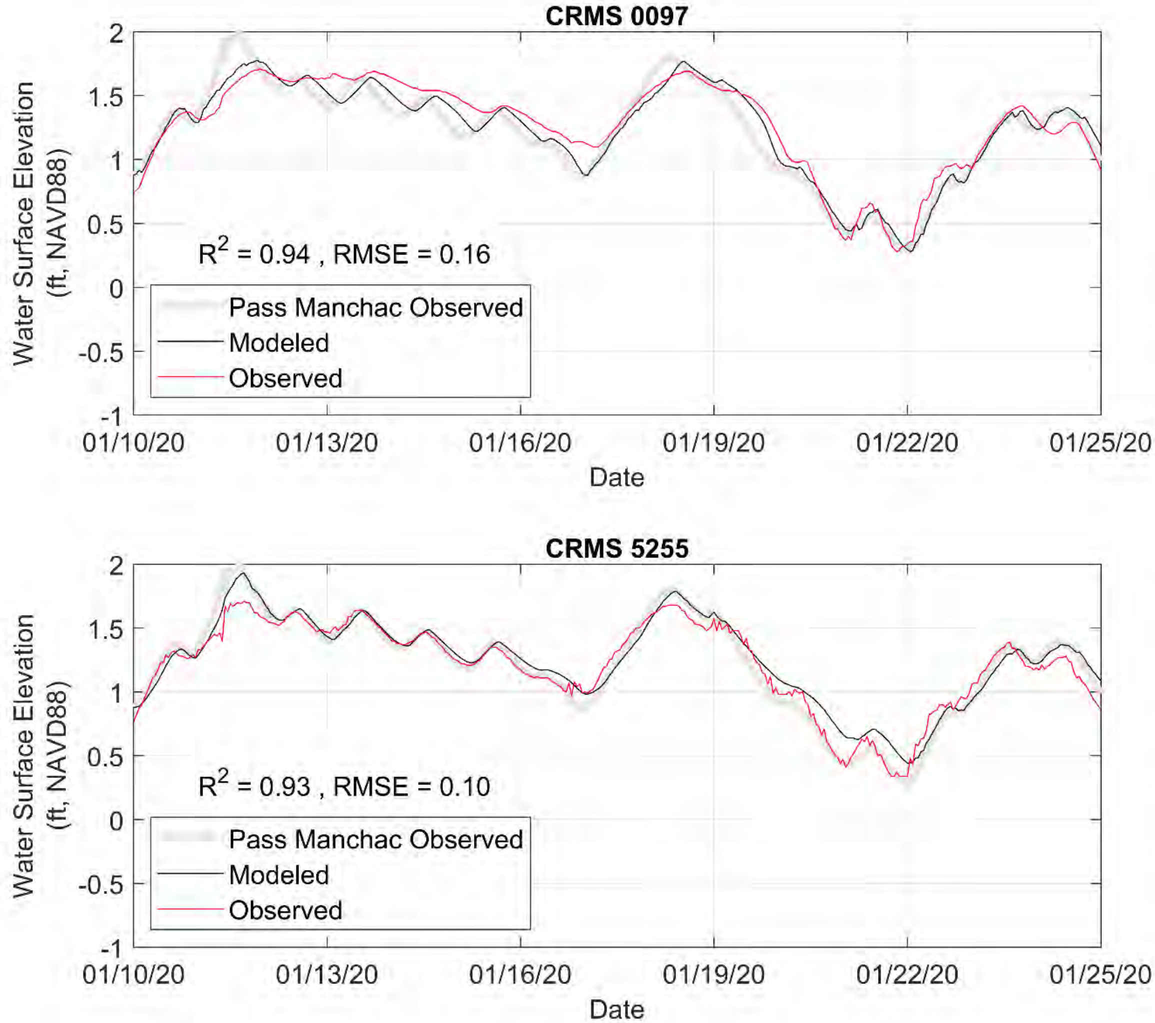



Figure B1. Observed and predicted water surface elevations at CRMS gages 0097 and 5255.

# 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

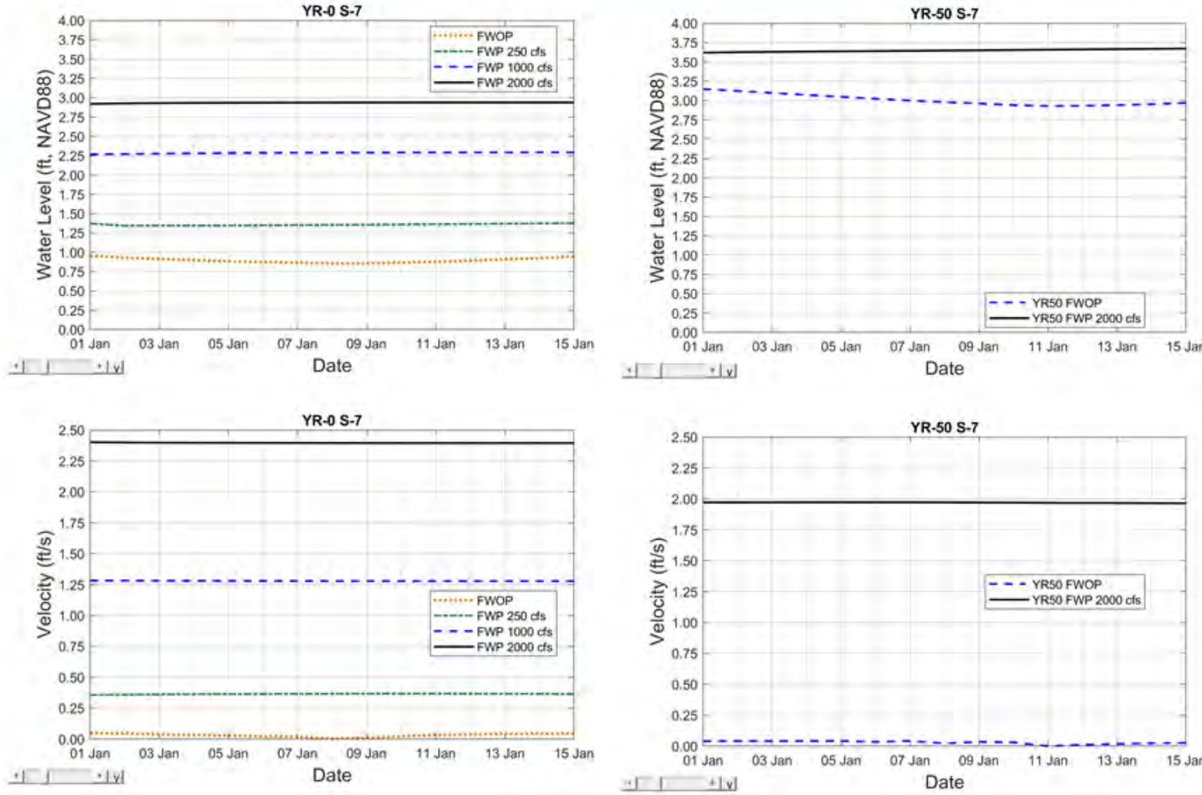



Figure C1. Predicted water surface elevation (upper panel) and velocity (lower panel) profiles over the model simulation period at location S-7 (Hope Canal north of I-10).

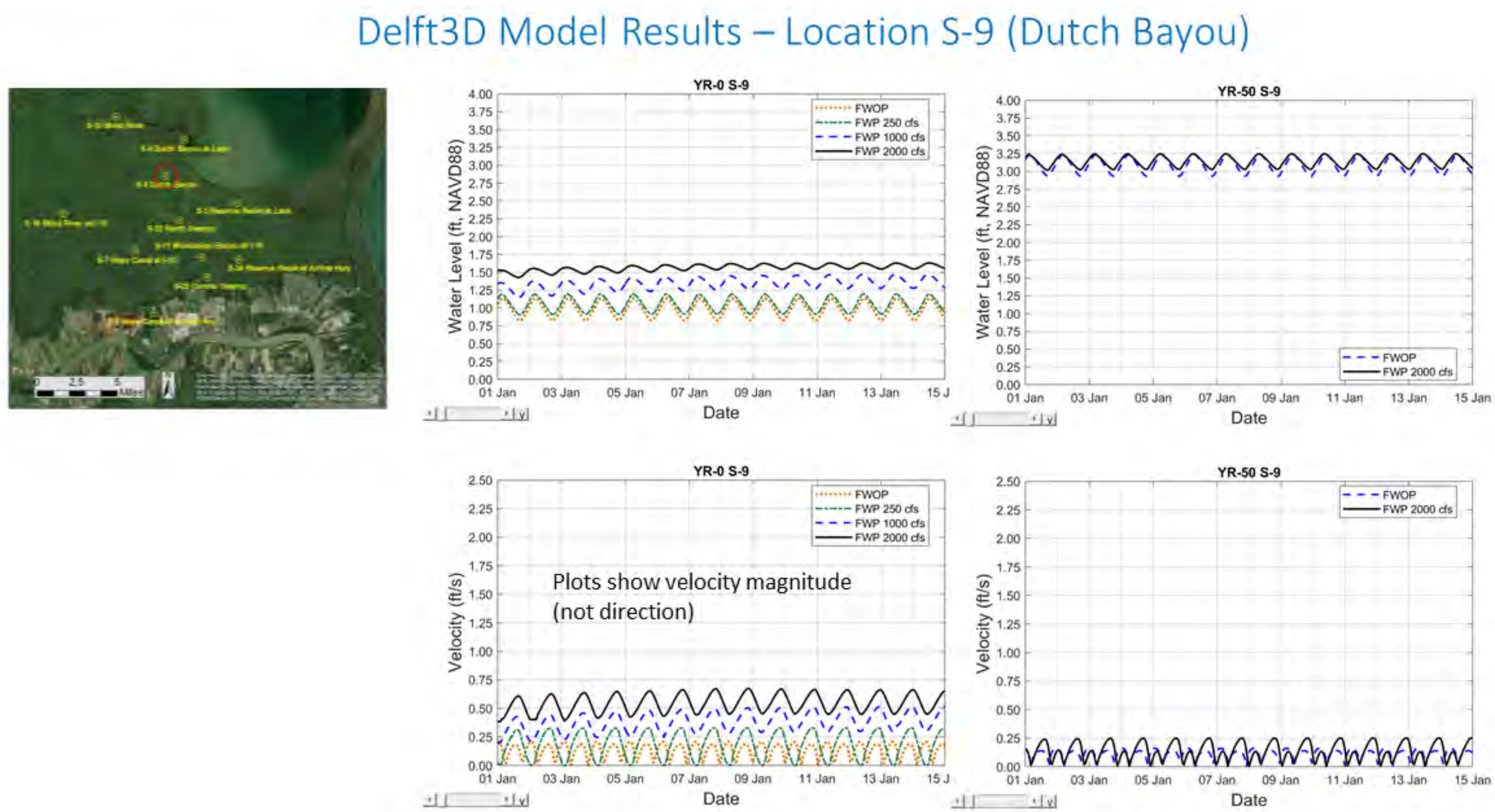



Figure C2. Predicted water surface elevation (upper panel) and velocity (lower panel) profiles over the model simulation period at location S-9 (Dutch Bayou).

## 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

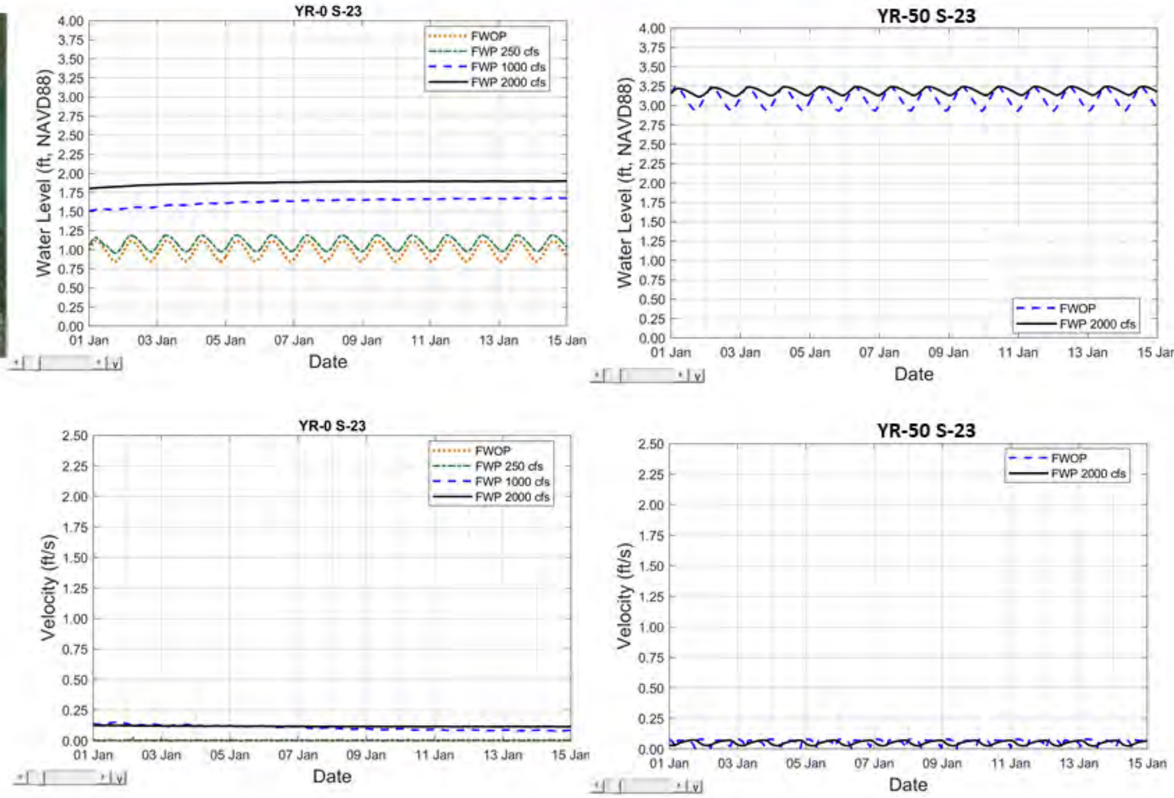



Figure C3. Predicted water surface elevation (upper panel) and velocity (lower panel) profiles over the model simulation period at location S-23 (North Swamp).

## Delft3D Model Results – Location Relief Canal at WSLP

Indication of effects near the WSLP drainage structures

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

<u>YR 50</u>
Normal is 3 ft, NAVD88
2000 cfs adds 0.1 ft

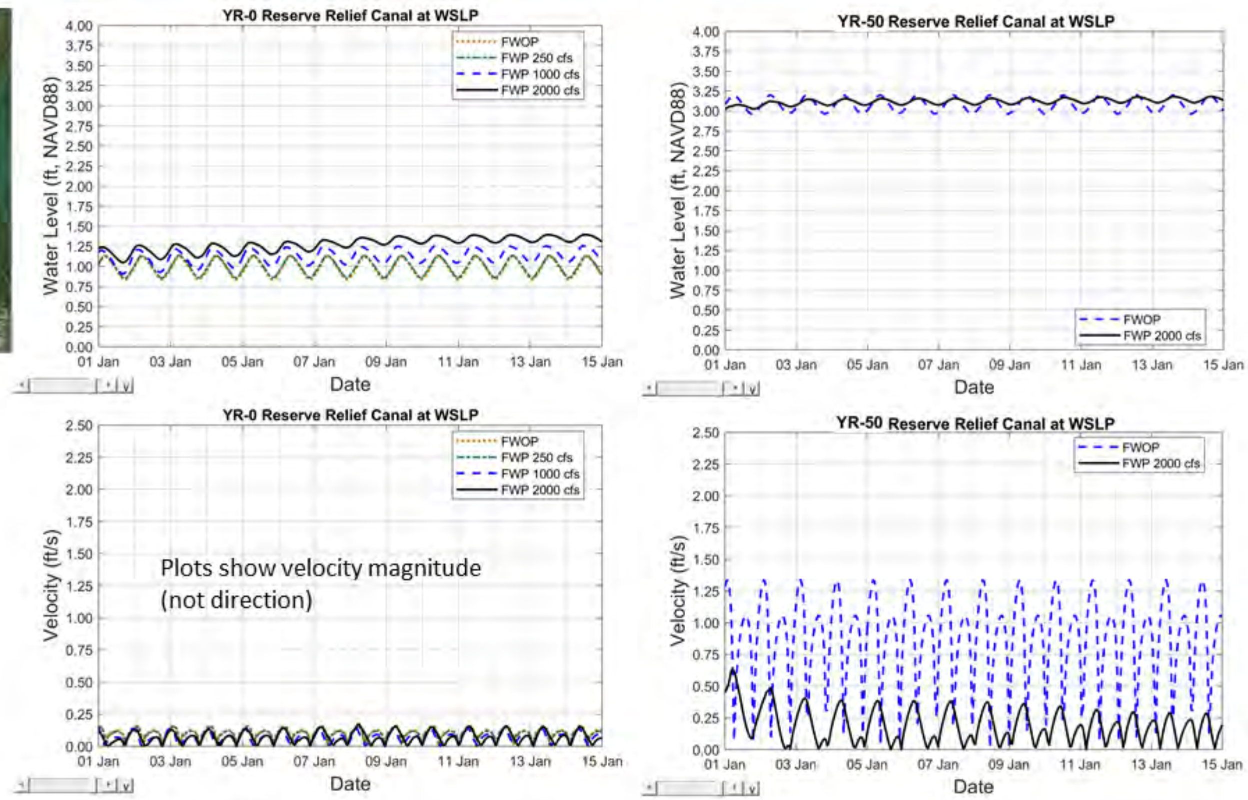



Figure C4. Predicted water surface elevation (upper panel) and velocity (lower panel) profiles over the model simulation period in Relief Canal near WSLP levee.

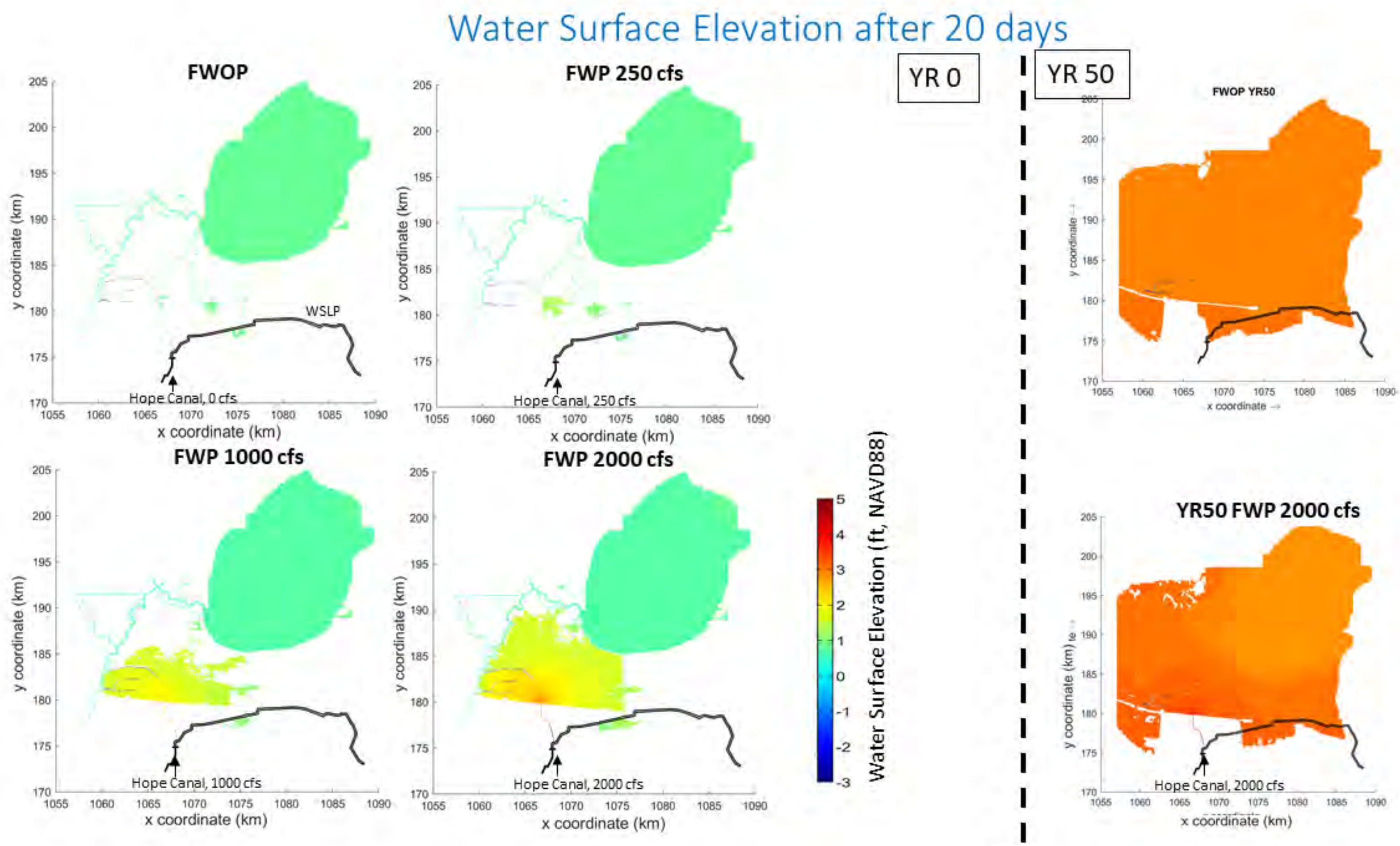



Figure C5. Predicted water surface elevation contours at the end 20 days.

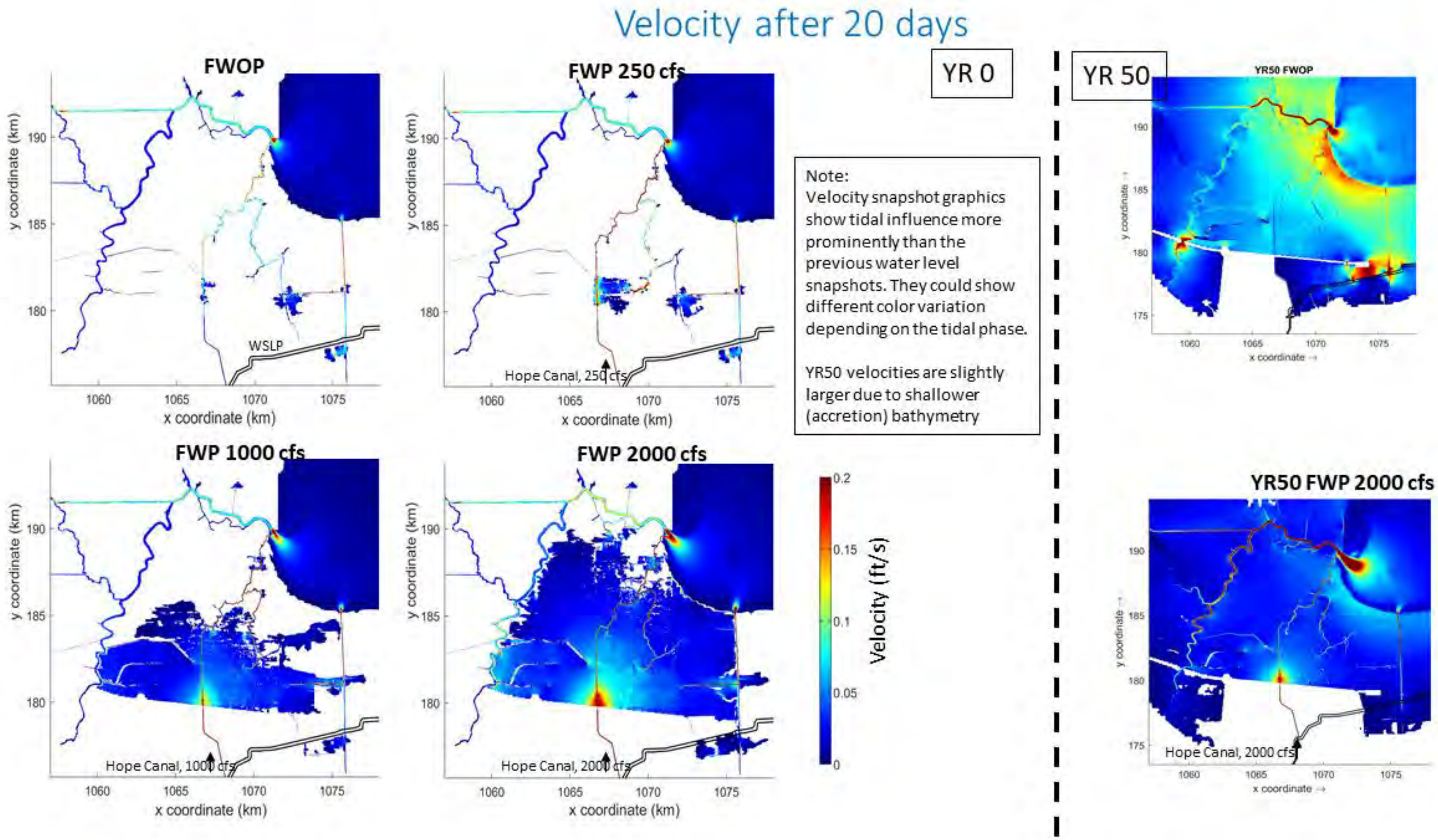
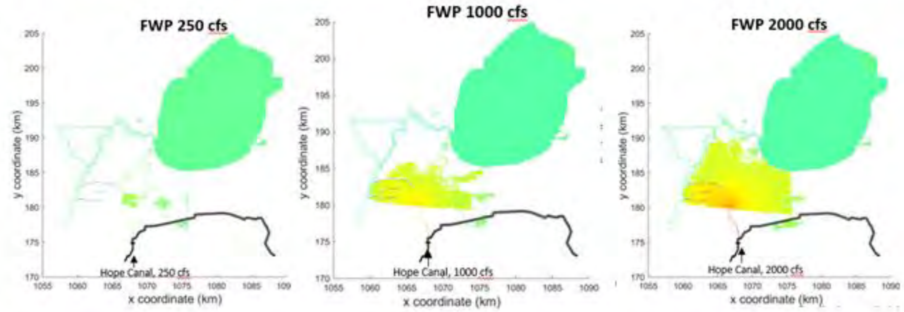



Figure C6. Predicted velocity contours at the end of 20 days.


## General Flow Distribution – YR 0 FWP

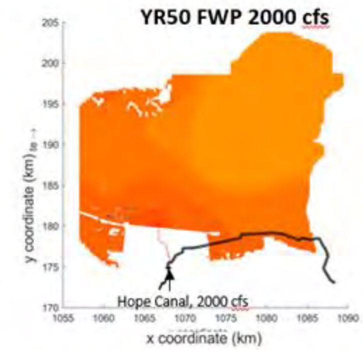



| Diversion Inflow (cfs) | Div. flow existing through Dutch Bayou (cfs) | Div. flow existing through the Blind River (cfs) | Div. flow exiting through Reserve Canal (cfs) |
|---|---|---|---|
| 250 | 210 (84%) | 0% | 29 (12%) |
| 1000 | 462 (46%) | 176 (18%) | 251 (25%) |
| 2000 | 648 (32%) | 570 (29%)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

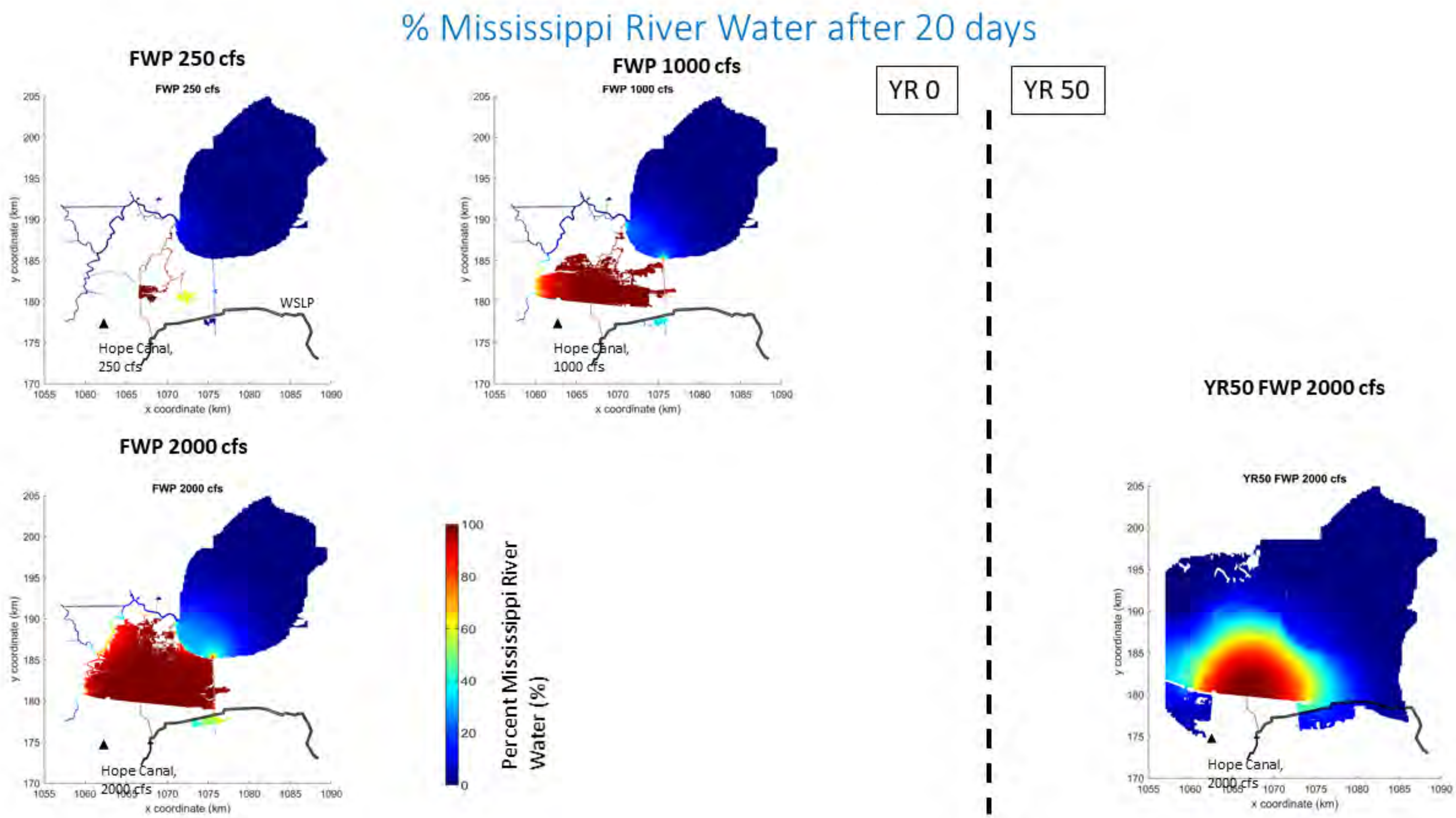



Figure C9. Predicted percent Mississippi River water contours at the end of 20 days.

## 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.

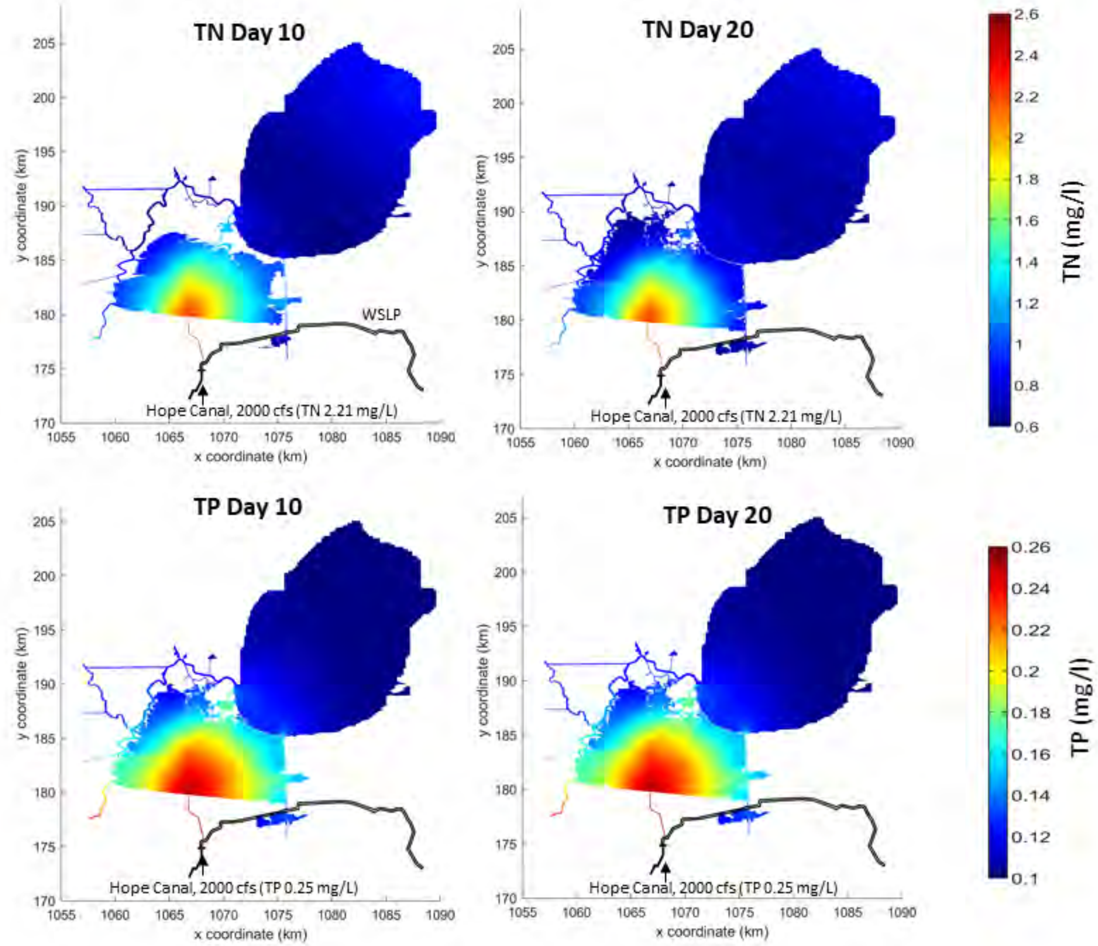



Figure C10. Predicted TN and TP concentrations at the end of 10 and 20 days (Year 0 conditions).

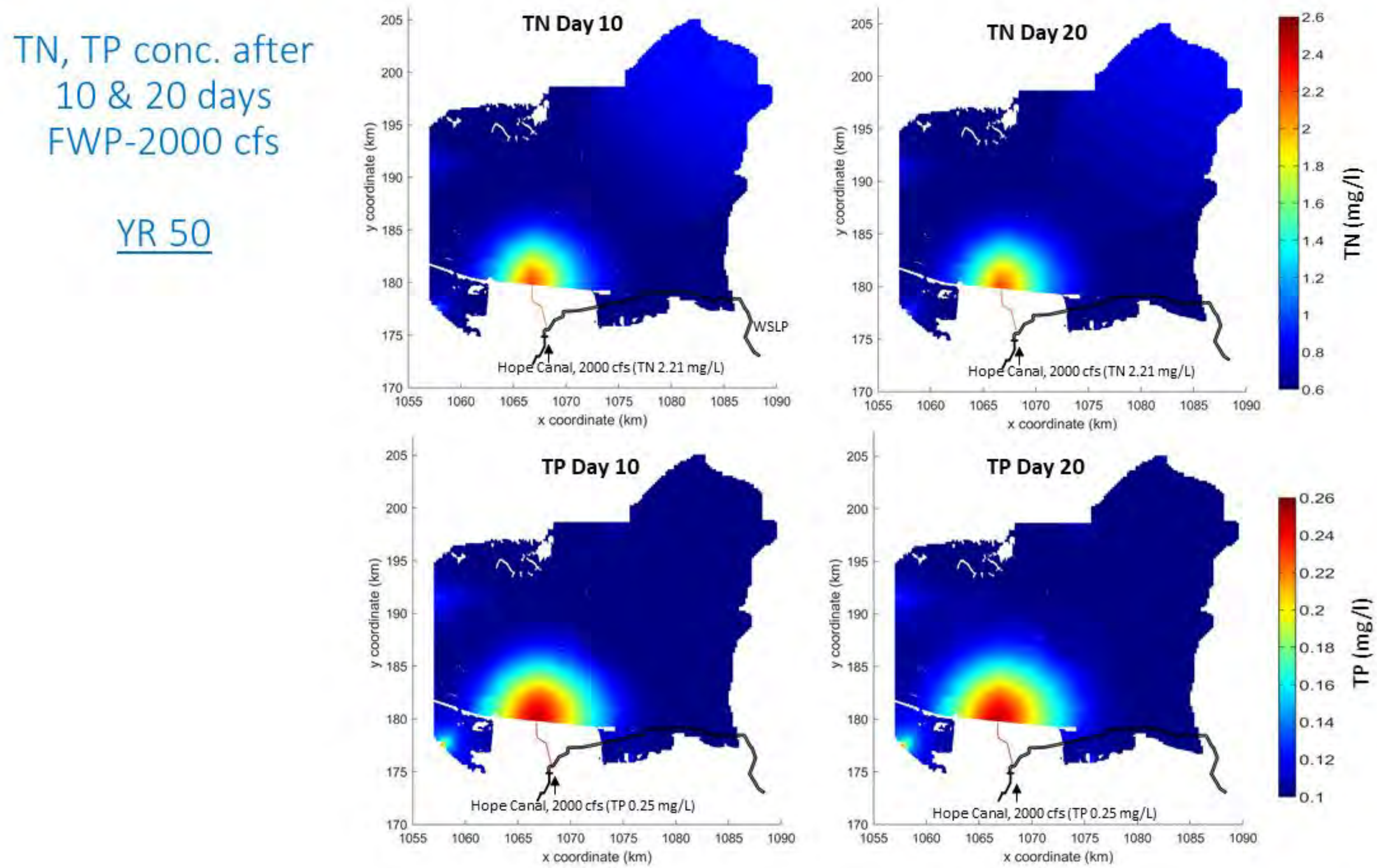



Figure C11. Predicted TN and TP concentrations at the end of 10 and 20 days (Year 50 conditions).

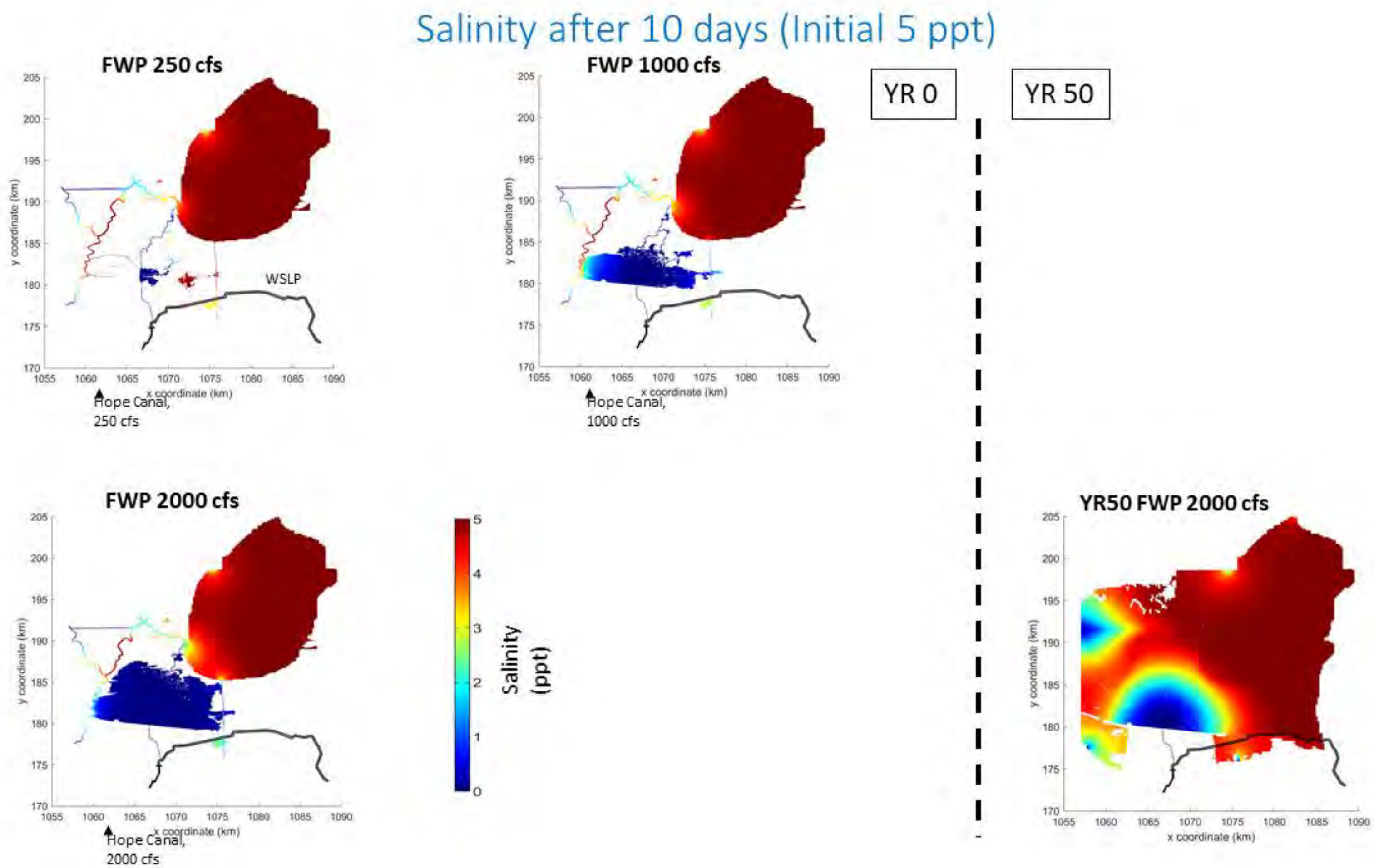



Figure C12. Predicted salinity concentrations at the end of 10 days.

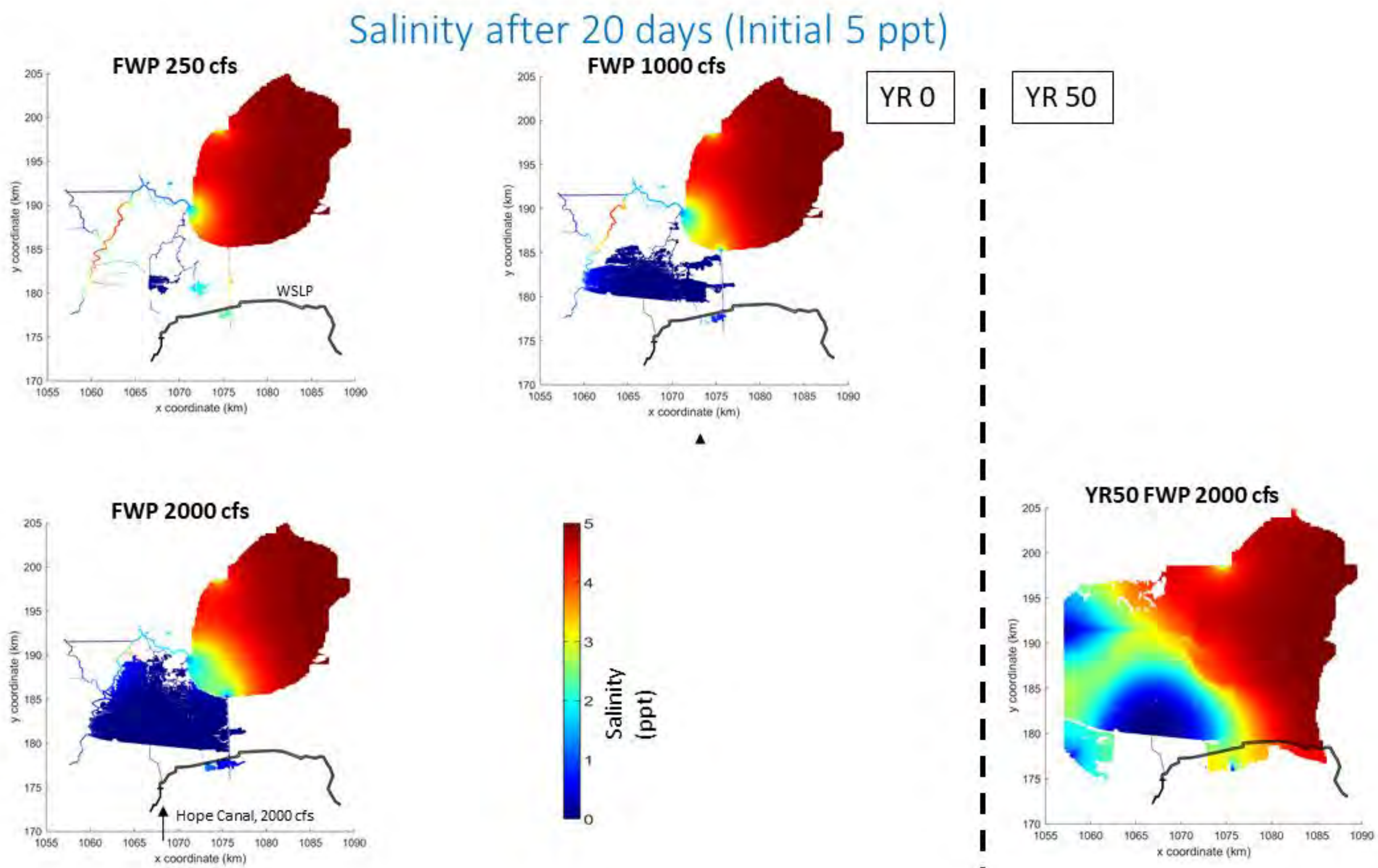



Figure C13. Predicted salinity concentrations at the end of 20 days.

# 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]<br>Mar: 38% [C]<br>Apr: 37% [C] | Feb: 5.3<br>Mar: 5.0<br>Apr: 18.7 | Feb: 0.103<br>Mar: 0.096<br>Apr: 0.025 | ~ 1 | Feb: 37.6<br>Mar: 34.9<br>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<br>PS2: 4.43<br>PS3: 4.43<br>FSS: 3.53 | PS1: 1.32<br>PS2: 1.14<br>PS3: 1.36<br>FSS: 1.44 | PS1: 73%<br>PS2: 74%<br>PS3: 69%<br>FSS: 59% | PS1: 9.2<br>PS2: 7.8<br>PS3: 11.2<br>FSS: 8.2 | PS1: 0.144<br>PS2: 0.174<br>PS3: 0.105<br>FSS: 0.110 | PS1: 0.29<br>PS2: 0.25<br>PS3: 0.28<br>FSS: 0.40 | PS1: 33.0<br>PS2: 55.4<br>PS3: 29.0<br>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%<br>EW4: 75%<br>EW5: 59% | EW3: 12<br>EW4: 95<br>EW5: 13 | EW3: 0.065<br>EW4: 0.015<br>EW5: 0.069 | 0.6 – 0.7 G | EW3: 14.6<br>EW4: 3.6<br>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]<br>Mar: 12% [C]<br>Apr: 58% [C] | Feb: 5.3<br>Mar: 5.0<br>Apr: 18.7 | Feb: 0<br>Mar: 0.025<br>Apr: 0.046 | ~ 1 | Feb: 0<br>Mar: 9.1<br>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<br>PS2: 0.719<br>PS3: 0.724<br>FSS: 0.888 | PS1: 0.457<br>PS2: 0.342<br>PS3: 0.347<br>FSS: 0.539 | PS1: 37%<br>PS2: 52%<br>PS3: 52%<br>FSS: 39% | PS1: 9.2<br>PS2: 7.8<br>PS3: 11.2<br>FSS: 8.2 | PS1: 0.050<br>PS2: 0.095<br>PS3: 0.066<br>FSS: 0.061 | PS1: 0.29<br>PS2: 0.25<br>PS3: 0.28<br>FSS: 0.40 | PS1: 6.2<br>PS2: 10.9<br>PS3: 5.7<br>FSS: 10.7 | Data were collected during Nov. 1993 – Jul. 2000 for pilot systems and Jun. 2003 – May 2008 for field scale system.<br>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**

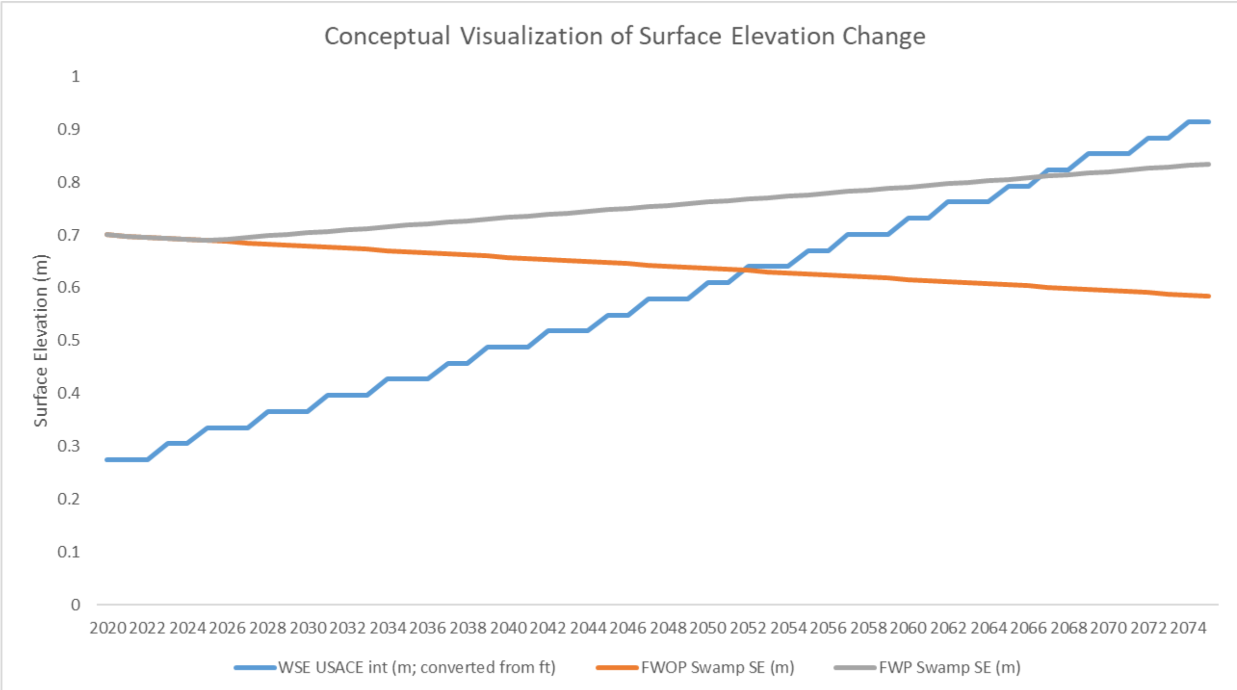



**Figure 1.** Graphical depiction of water and swamp surface elevations from 2020-2075 in Maurepas Swamp for FWOP and FWP assumptions using arbitrary starting swamp surface elevation of 0.5m for demonstrative purposes only.

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. <u>Transforming model depth-averaged velocities to the depth of the instrument</u>

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.

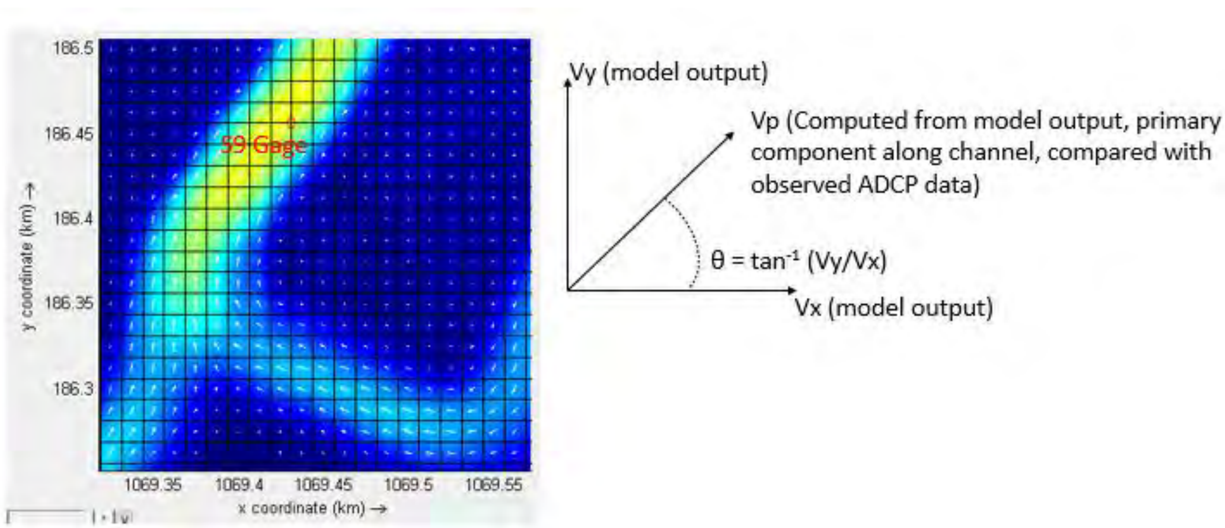



Figure F1. Left panel: Vector plot of velocities in the Dutch Bayou in vicinity of the S-9 Gage during a typical outgoing tide. Contours are colored by the velocity magnitude. The legend is not shown because the exact values are not relevant for this discussion. Right panel: Definition of resultant angle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

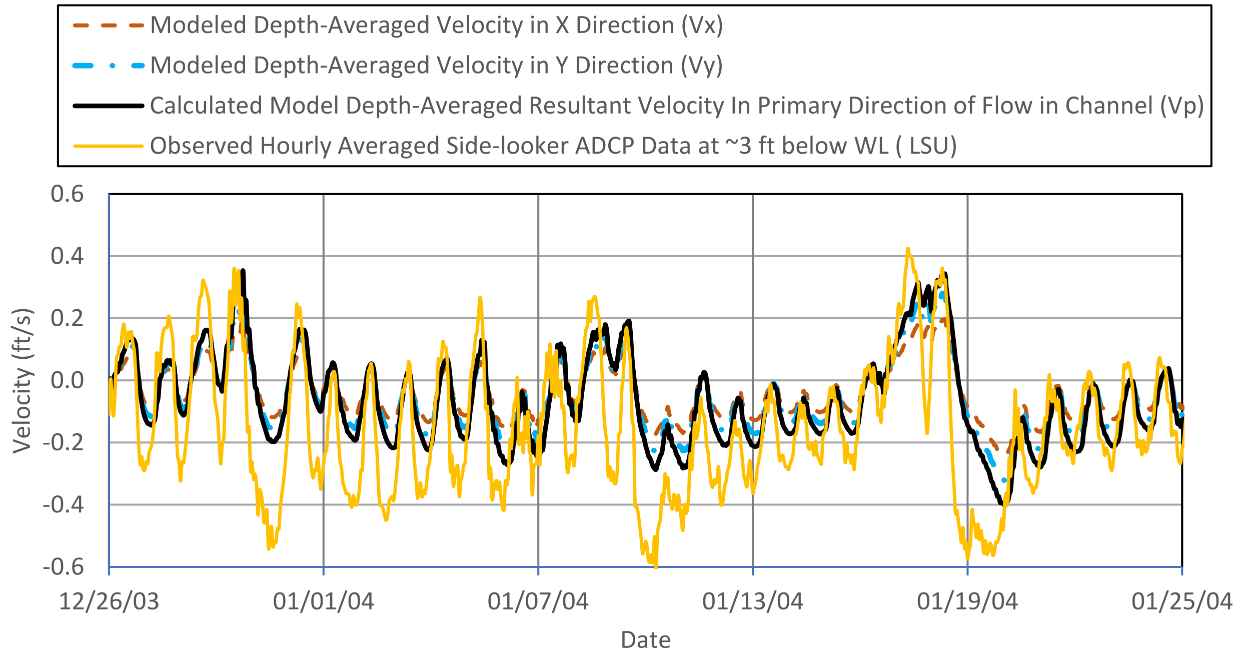



Figure F2. Normal Conditions Time Series of Velocities (S-9 Gage) Comparison. Previously in the report, only the X Direction (Vx) velocity from the model was compared against the observed data.

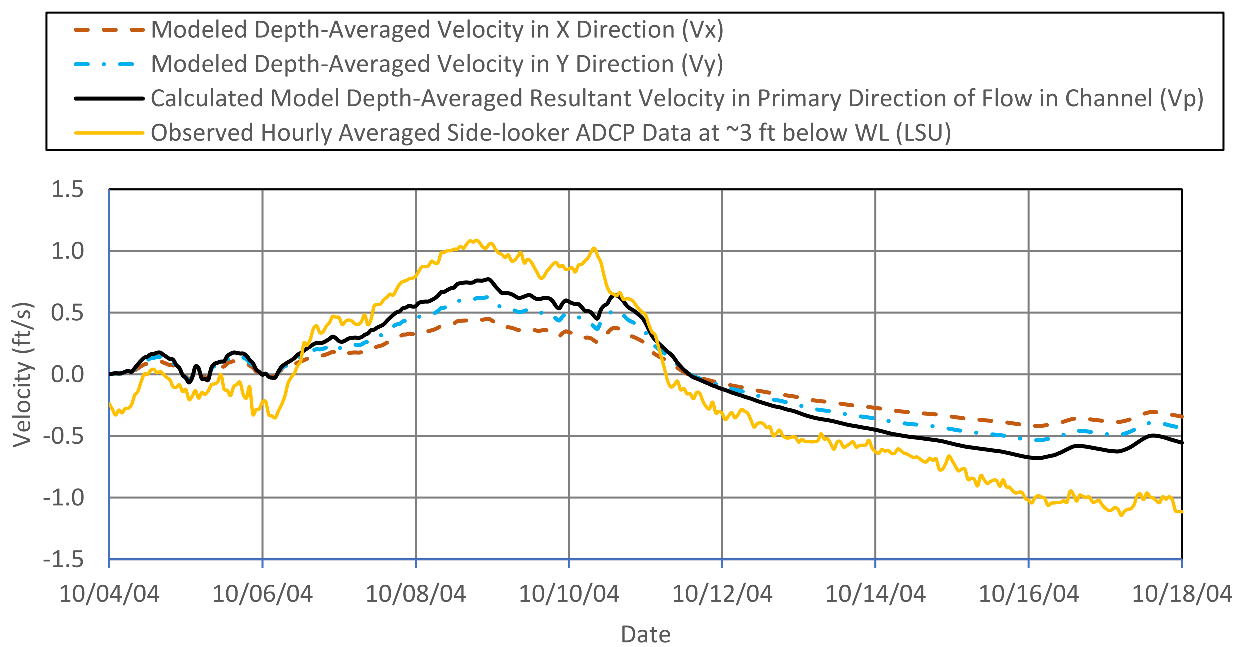



Figure F3. Tropical Storm Conditions Time Series of Velocities (S-9 Gage) Comparison. Previously in the report, only the X Direction (Vx) velocity from the model was compared against the observed data.

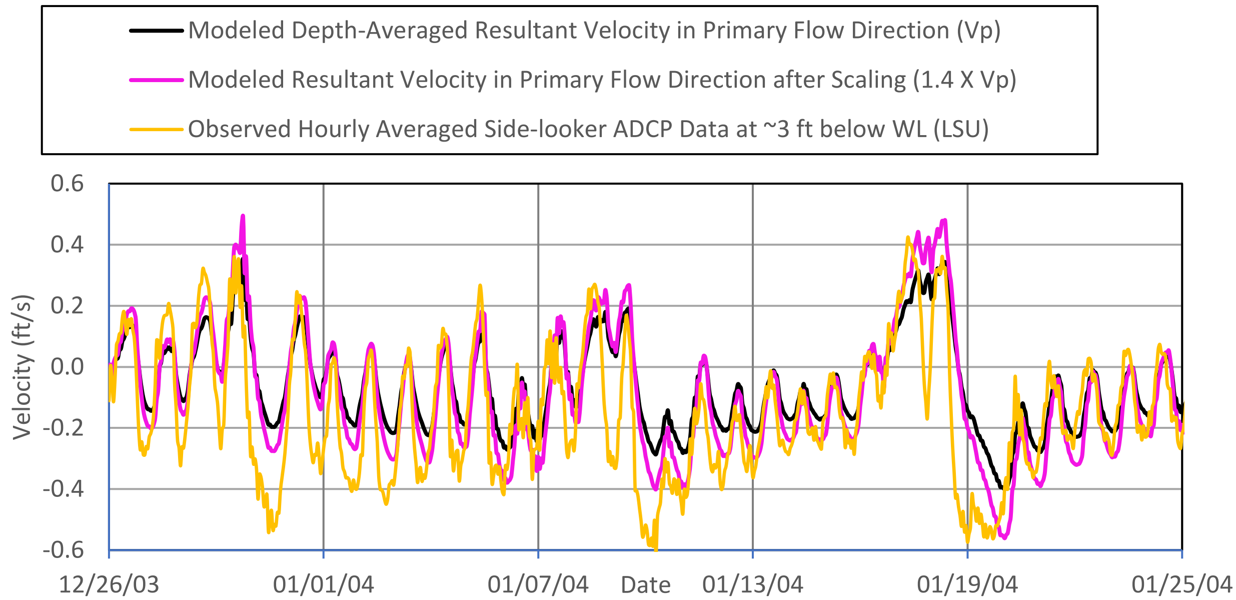



Figure F4. Normal Conditions Time Series of Velocities (S-9 Gage) Comparison. A scale factor of 1.4 is applied to the projected depth-averaged velocity to convert to velocity at a depth of 3 ft measured by the gage.

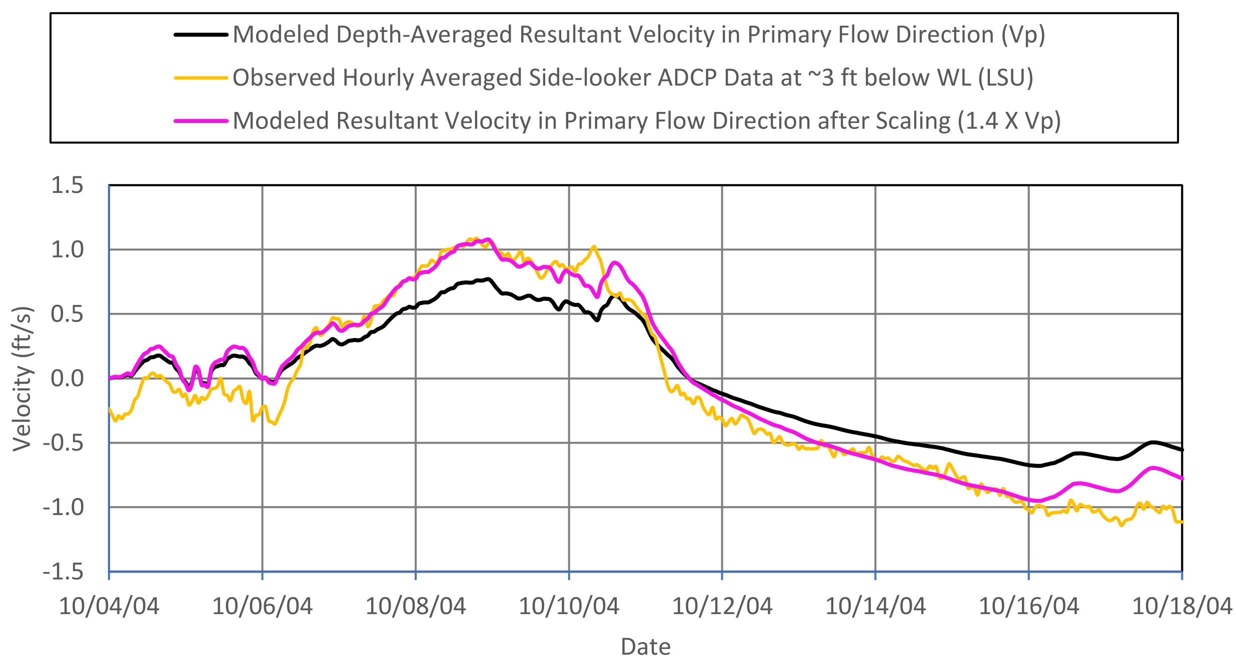



Figure F5. Tropical Storm Conditions Time Series of Velocities (S-9 Gage) Comparison. A scale factor of 1.4 is applied to the projected depth-averaged velocity to convert to velocity at a depth of 3 ft measured by the gage.

**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.

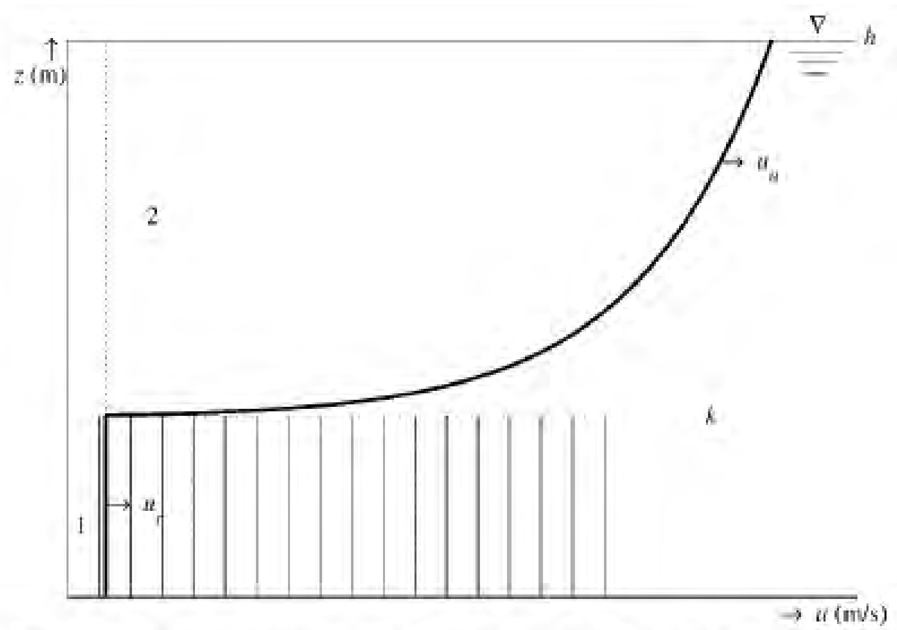



Figure 2 Representation of the vertical velocity profile in two zones for the method of effective water depth. $h$ = water depth (m), $k$ = vegetation height (m), $u_c$ = uniform flow velocity profile (m/s), $u_u$ = logarithmic flow velocity profile (m/s).

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:

$$u_{\mathrm{u}}(z) = \frac{u_*}{\kappa} \ln\left(\frac{z-k}{z_0}\right) + u_{\mathrm{c}} \qquad \text{.................................................. (1)}$$

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

$$\begin{aligned}\bar{u}_{\mathrm{u}} &= \frac{1}{h-k}\int_k^h u_{\mathrm{u}}(z)\mathrm{d}z \\ &= \frac{u_*}{\kappa}\ln\left(\frac{h-k}{z_0} - 1\right) + u_{\mathrm{c}} = \frac{u_*}{\kappa}\ln\left(\frac{h-k}{ez_0}\right) + u_{\mathrm{c}}\end{aligned} \qquad \text{.............................. (2)}$$

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

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

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,

$$c_{\mathrm{p}} = \frac{1}{20}\frac{h-k}{\ell}$$

$$L = \sqrt{\frac{c_{\mathrm{p}}\ell}{C_{\mathrm{D}} m D}}$$

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<br>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<br>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<br>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.

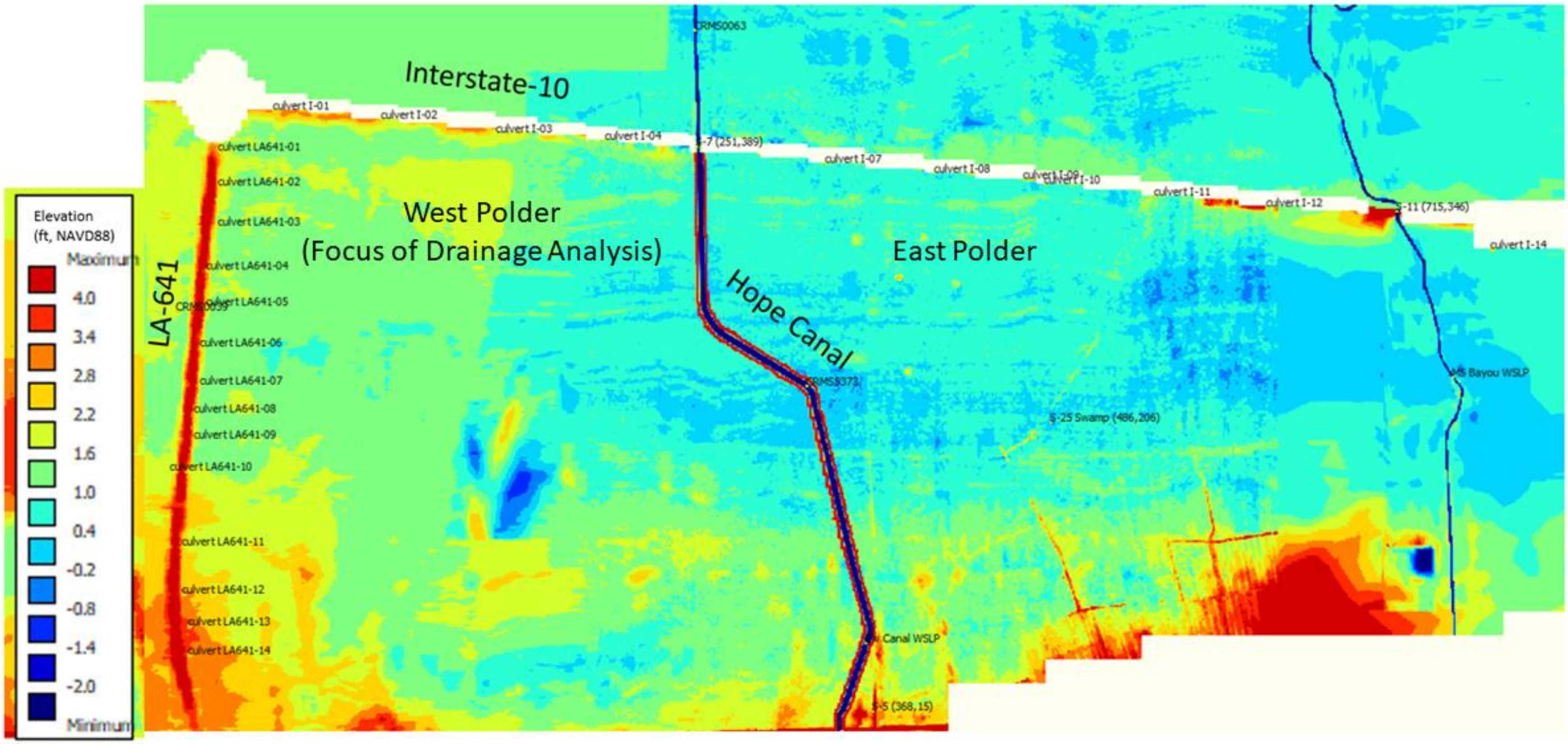
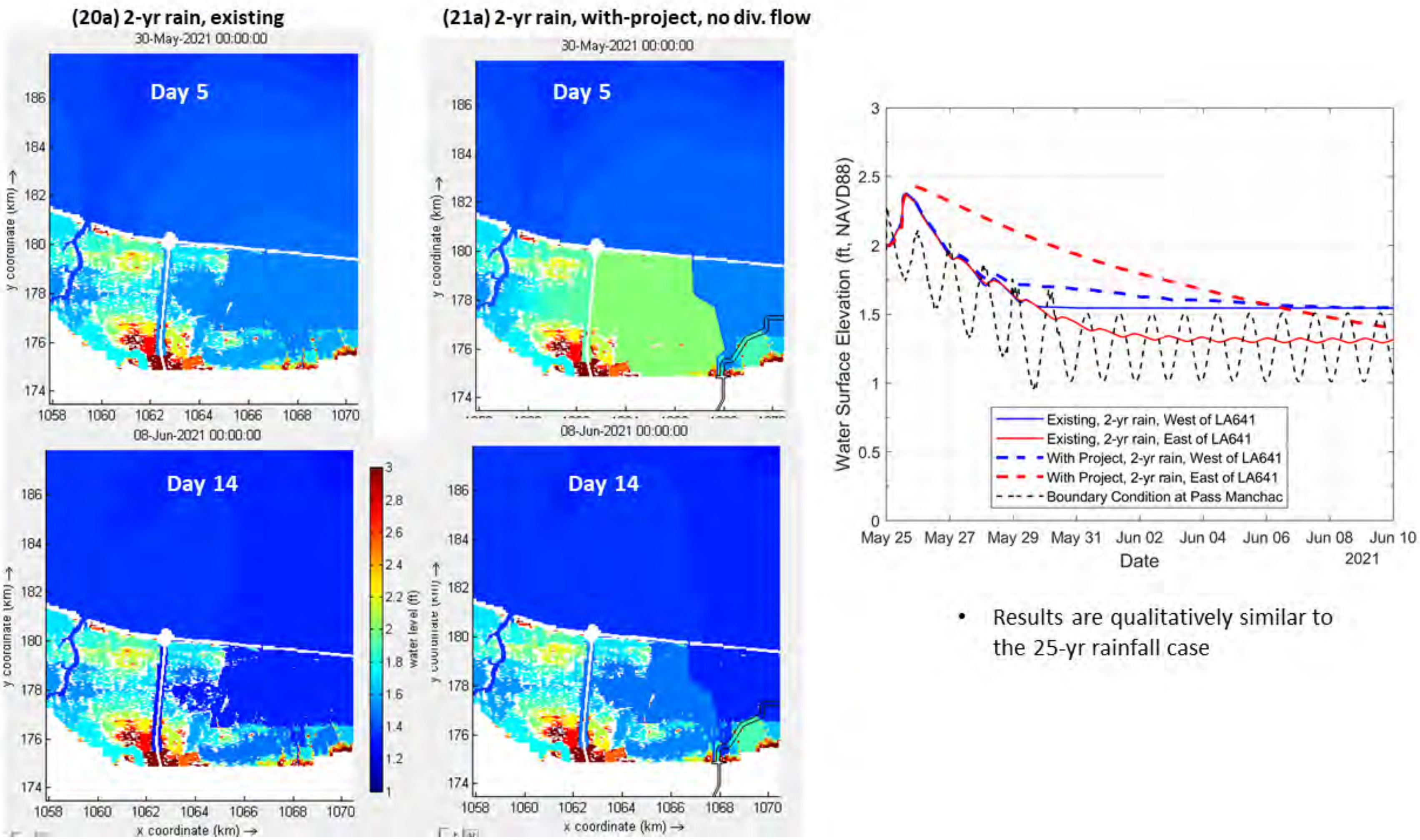



Figure G1. West and East Polders Created by the Proposed Hope Canal Alignment. Locations of highway culverts are shown with labels.




- 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).

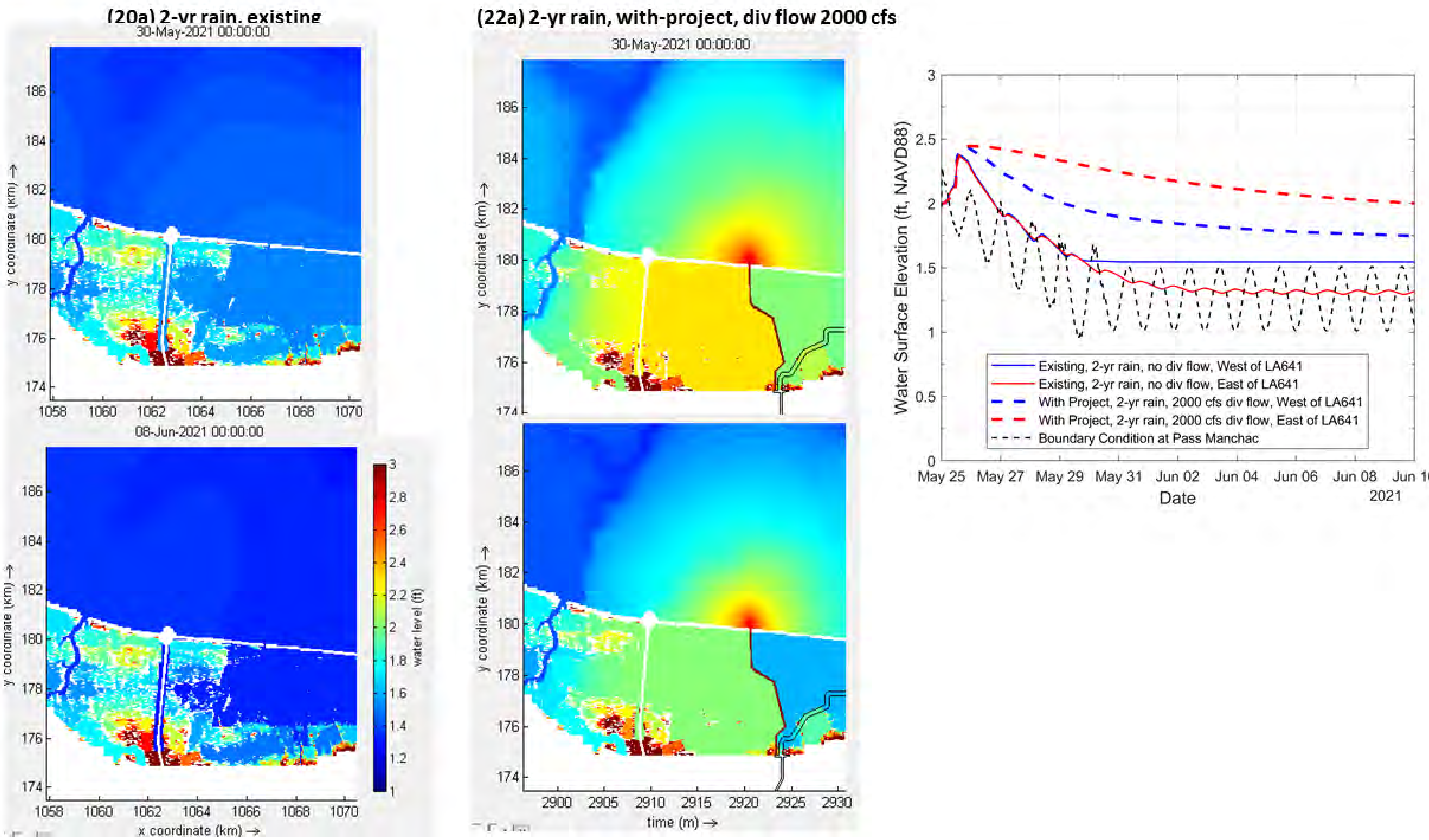



Figure G3. Comparison of existing and with-project, 2,000 cfs diversion flow conditions (2-year rainfall).

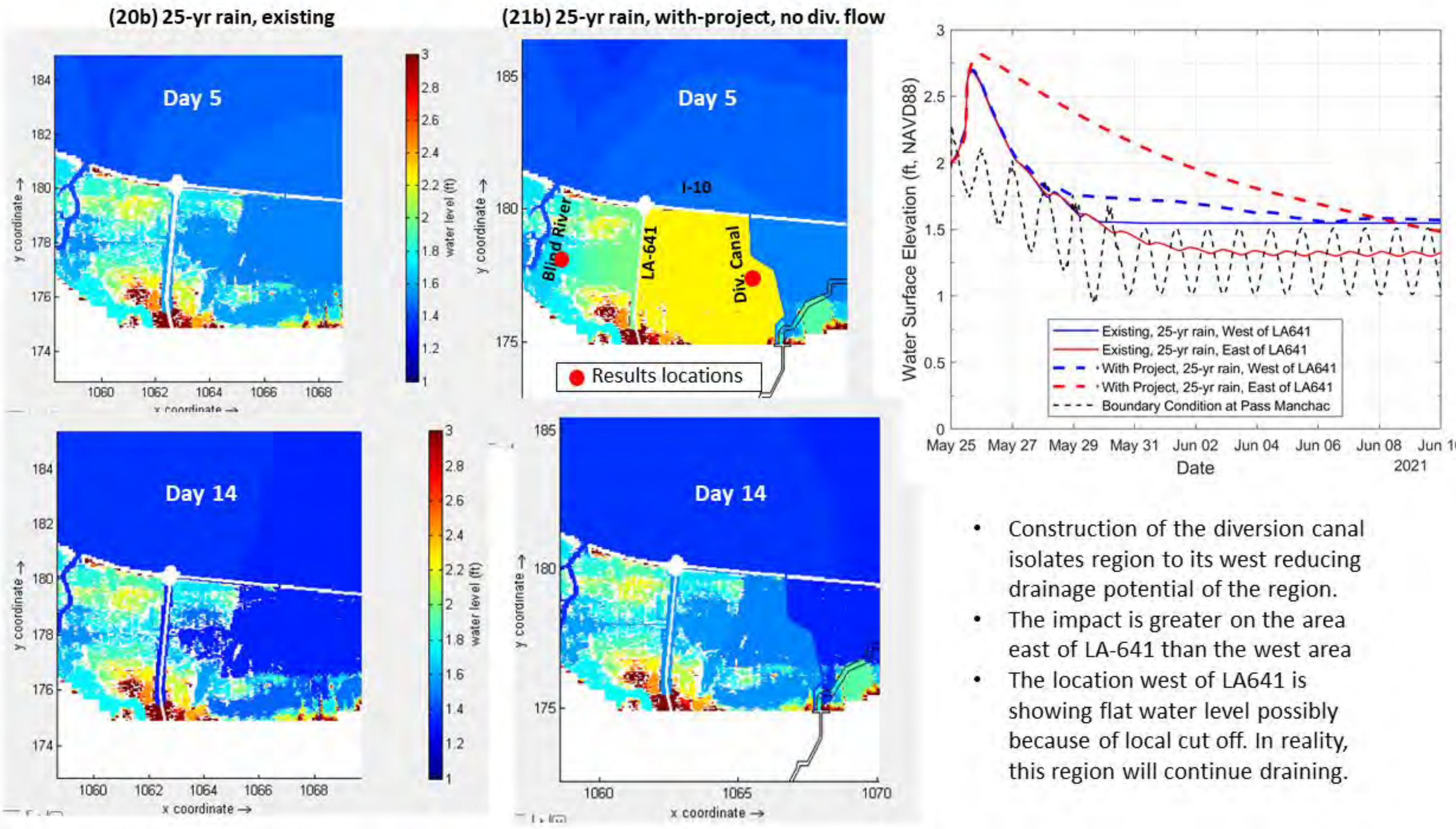



Figure G4. Comparison of existing and with-project, no diversion flow conditions (25-year rainfall).

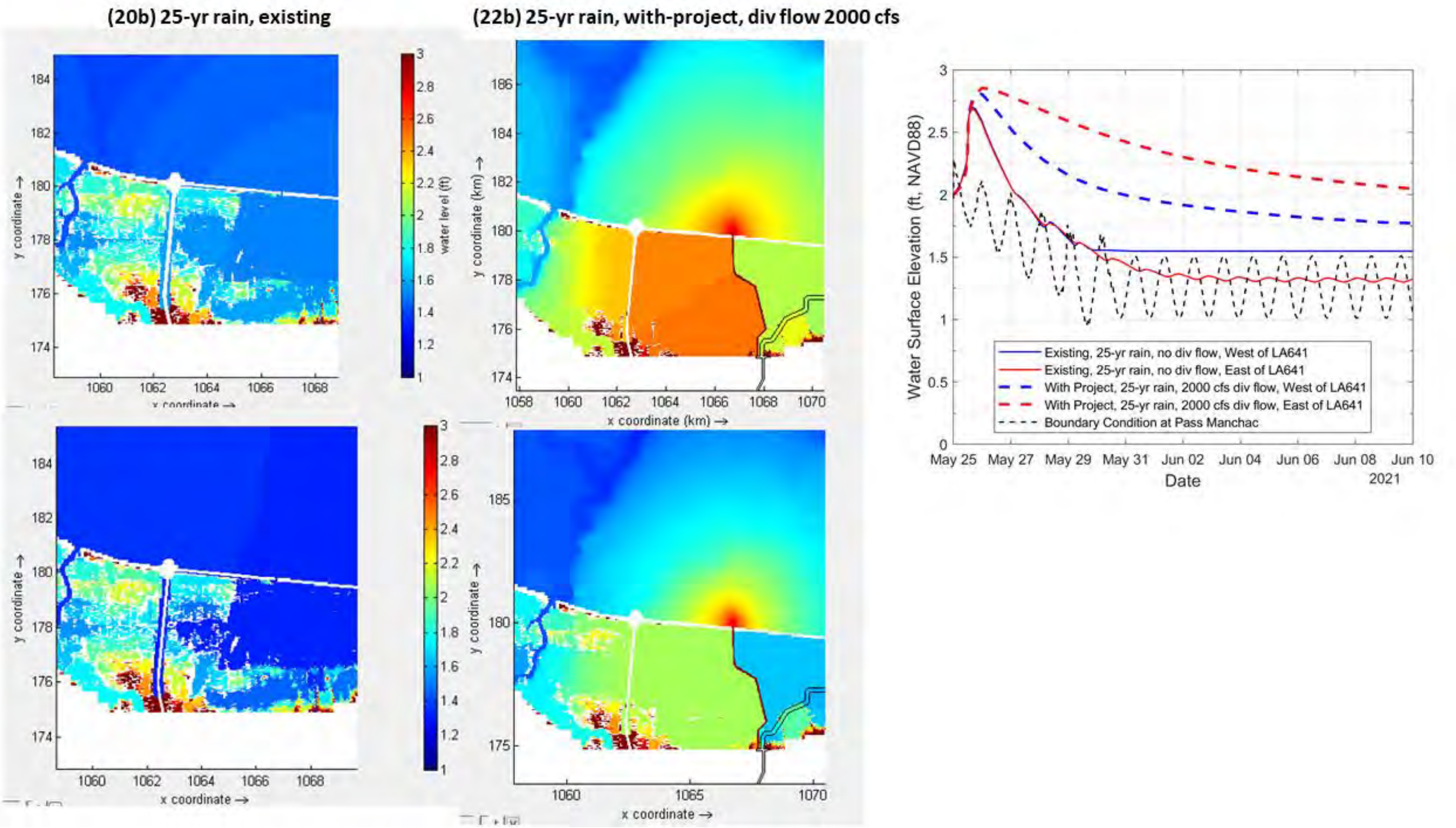
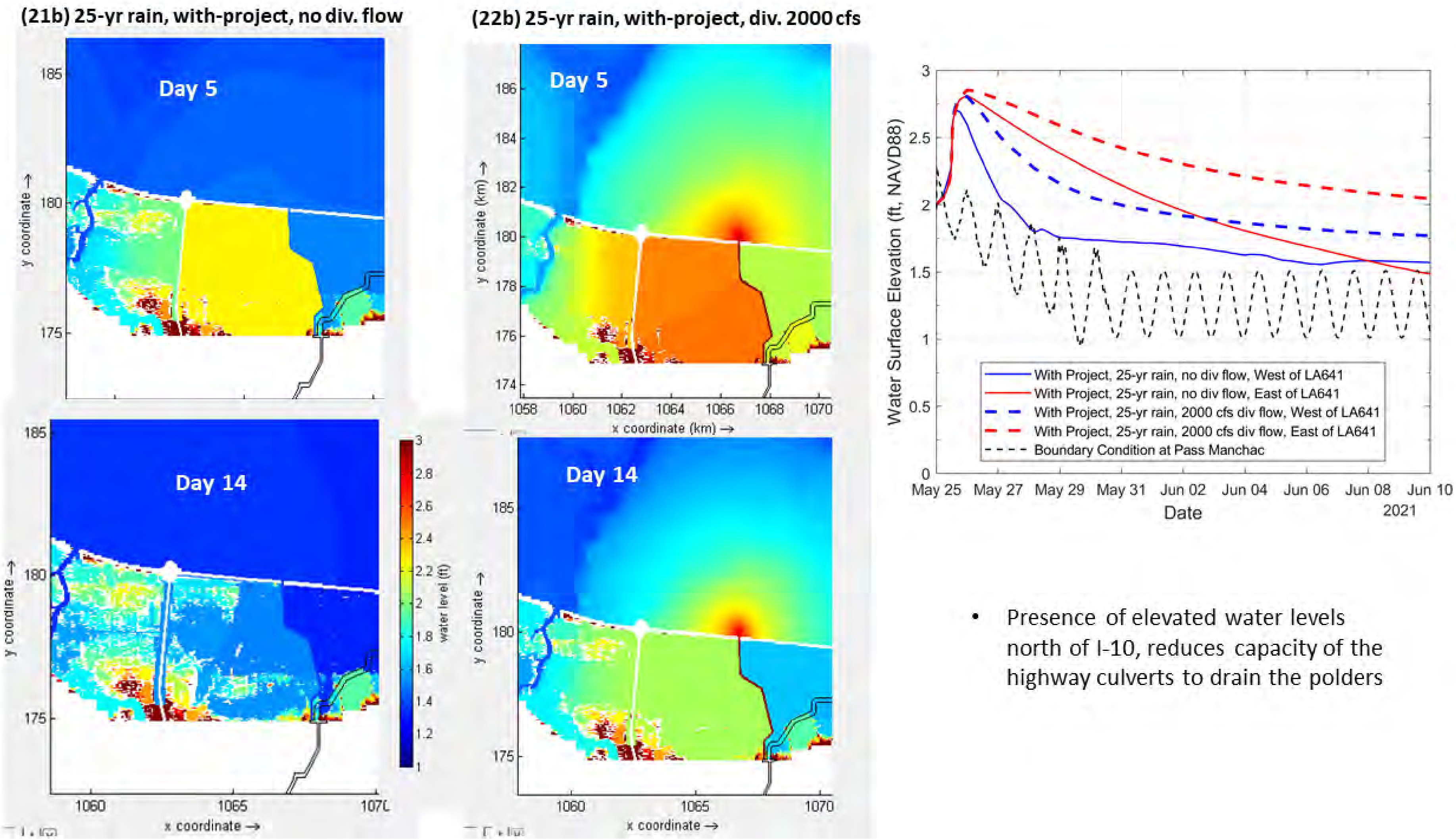
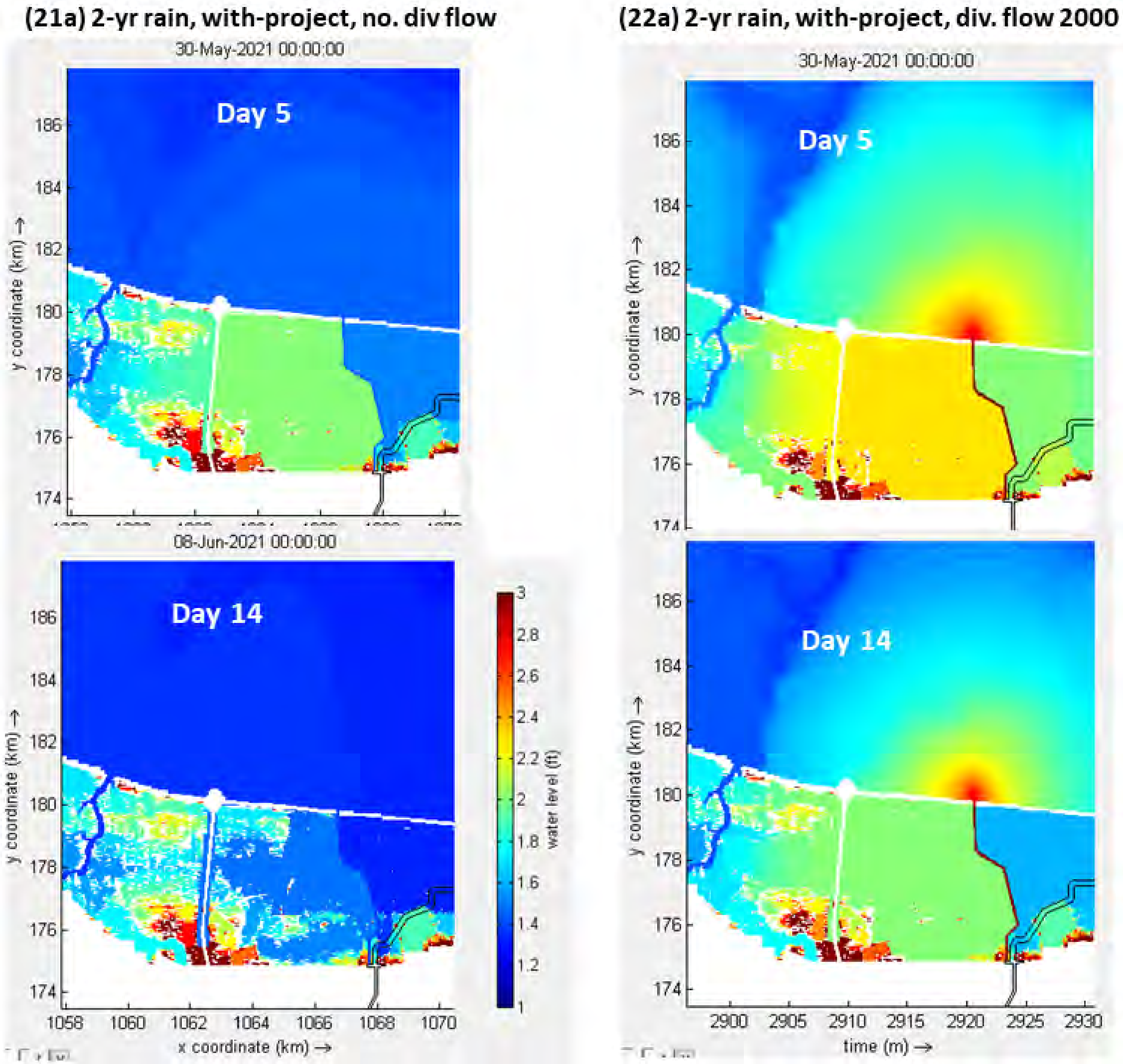


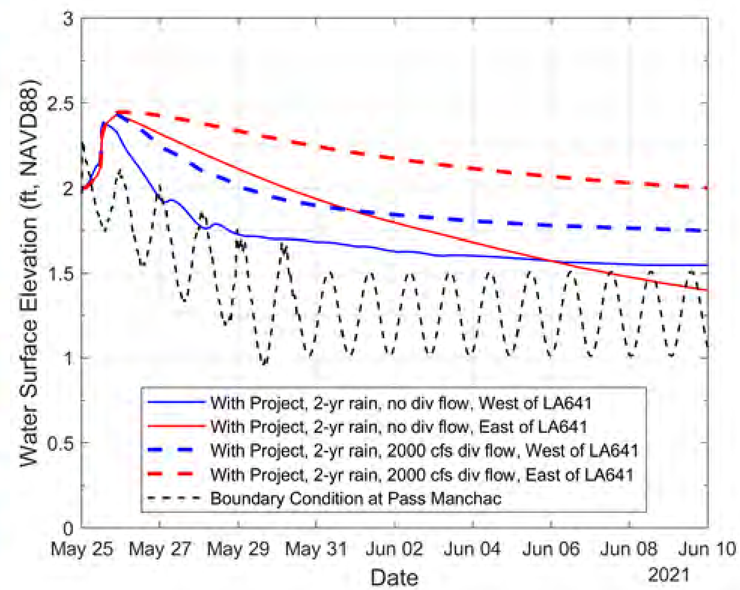

Figure G5. Comparison of existing and with-project, 2,000 cfs diversion flow conditions (25-year rainfall).




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

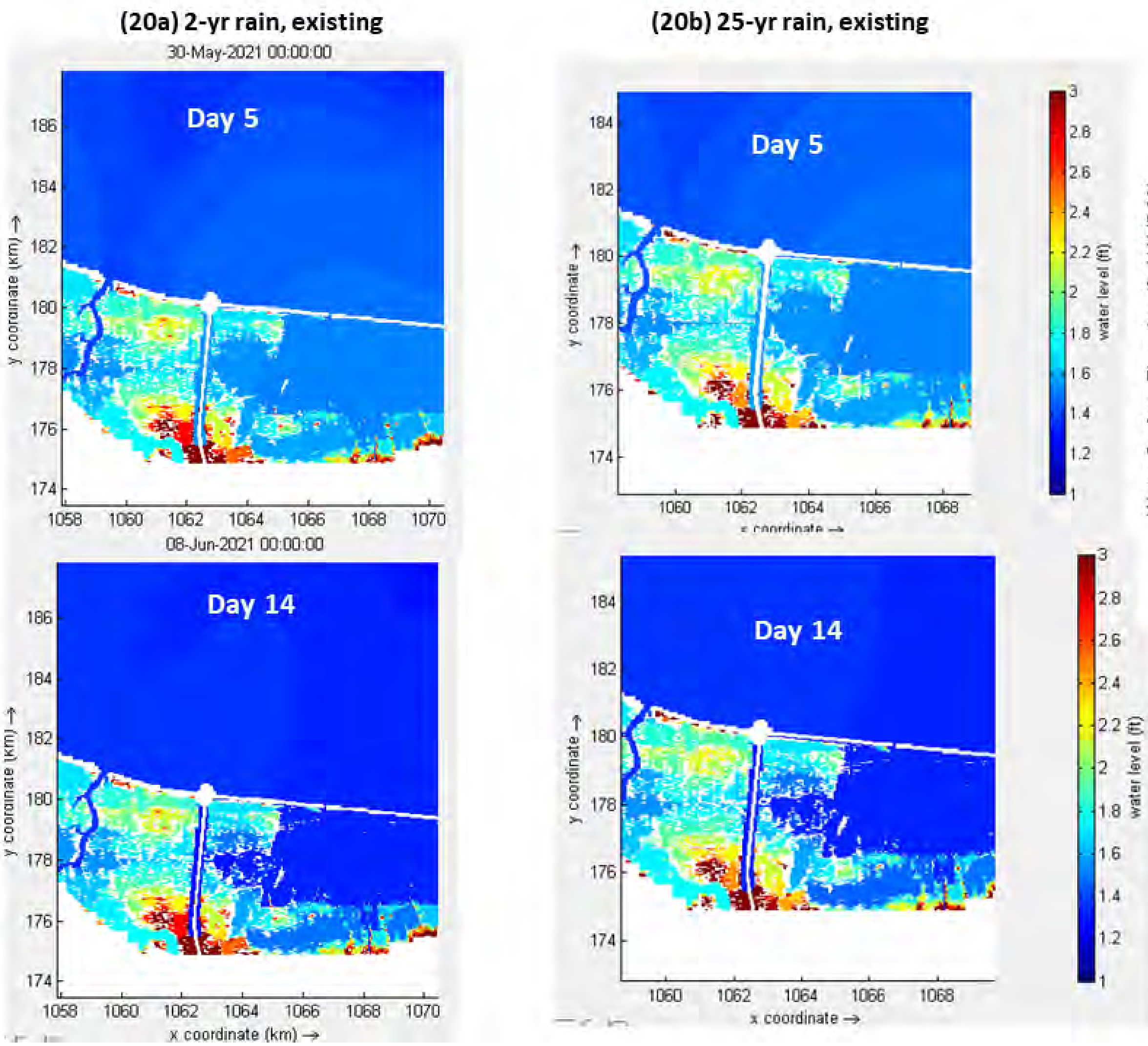
Figure G6. Comparison of with-project, no diversion flow and with-project, 2,000 cfs diversion flow conditions (25-year rainfall).

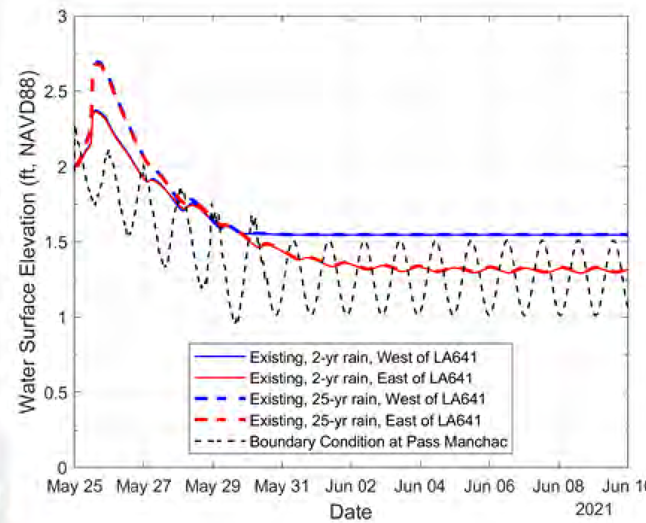



- Results are qualitatively similar to the 25-yr rainfall case

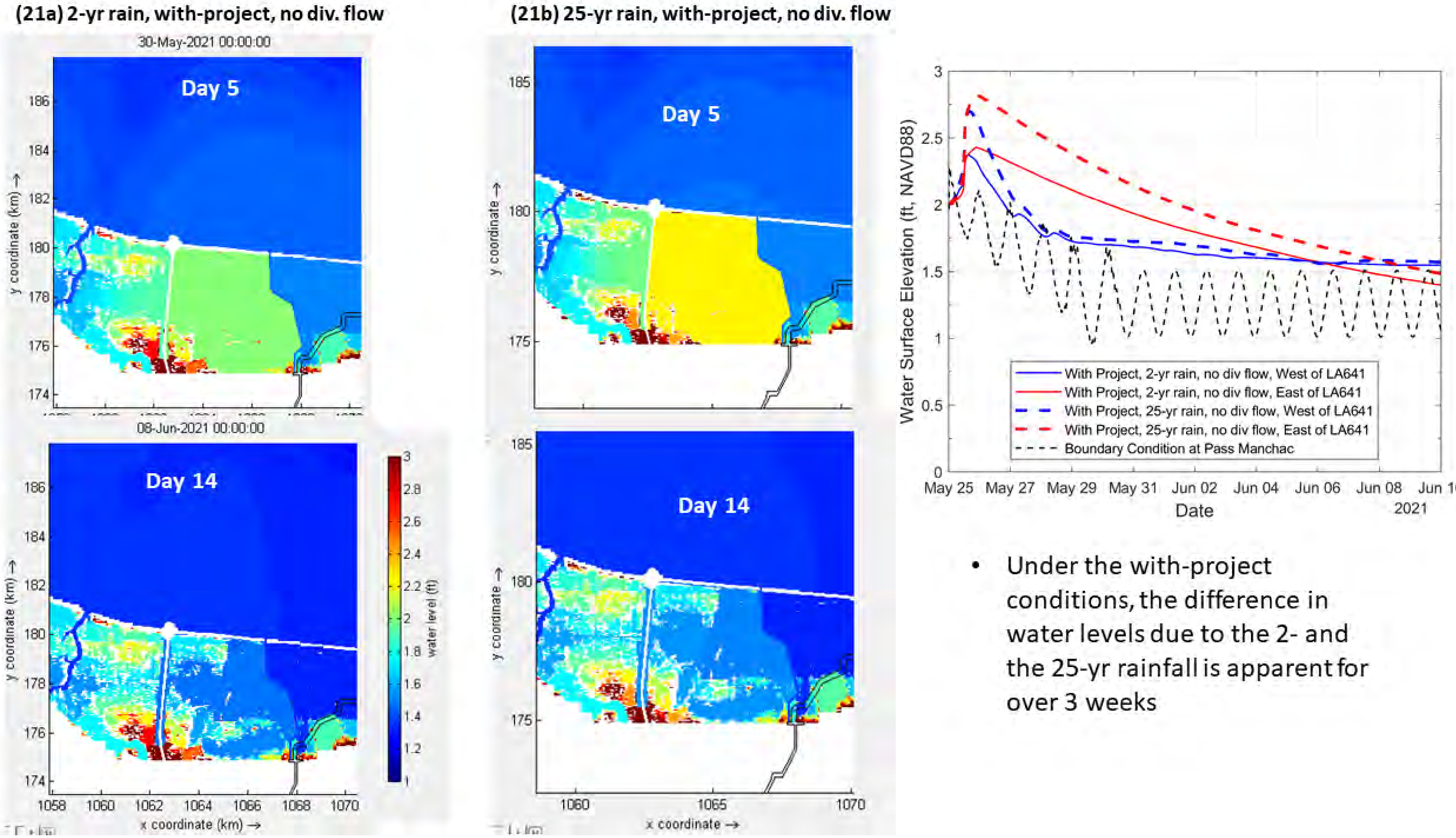
Figure G7. Comparison of with-project, no diversion flow and with-project, 2,000 cfs diversion flow conditions (2-year rainfall).




- 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.

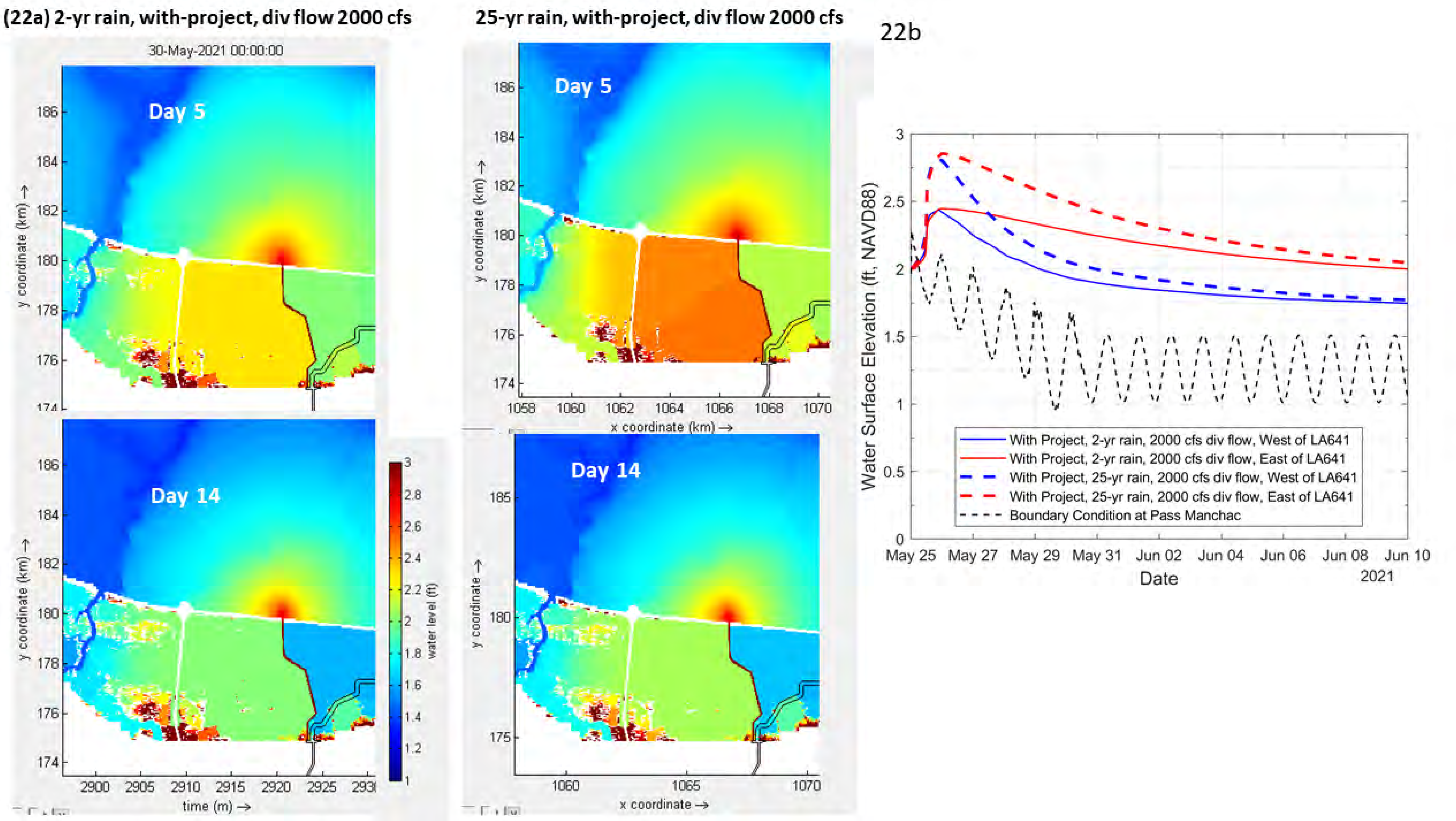



Figure G10. Comparison of with-project, 2,000 cfs diversion flow conditions under 2-year and 25-year rainfall.

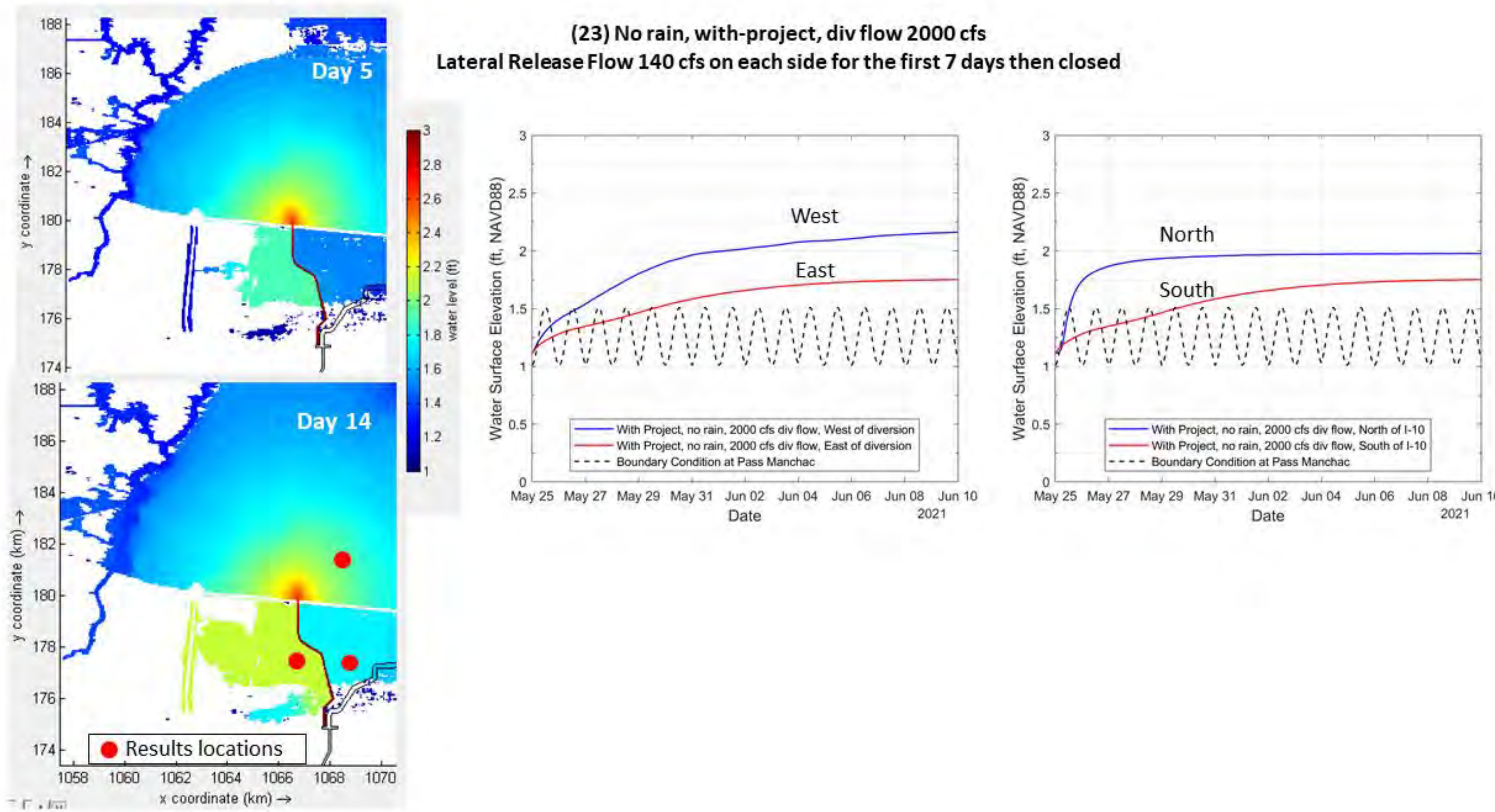
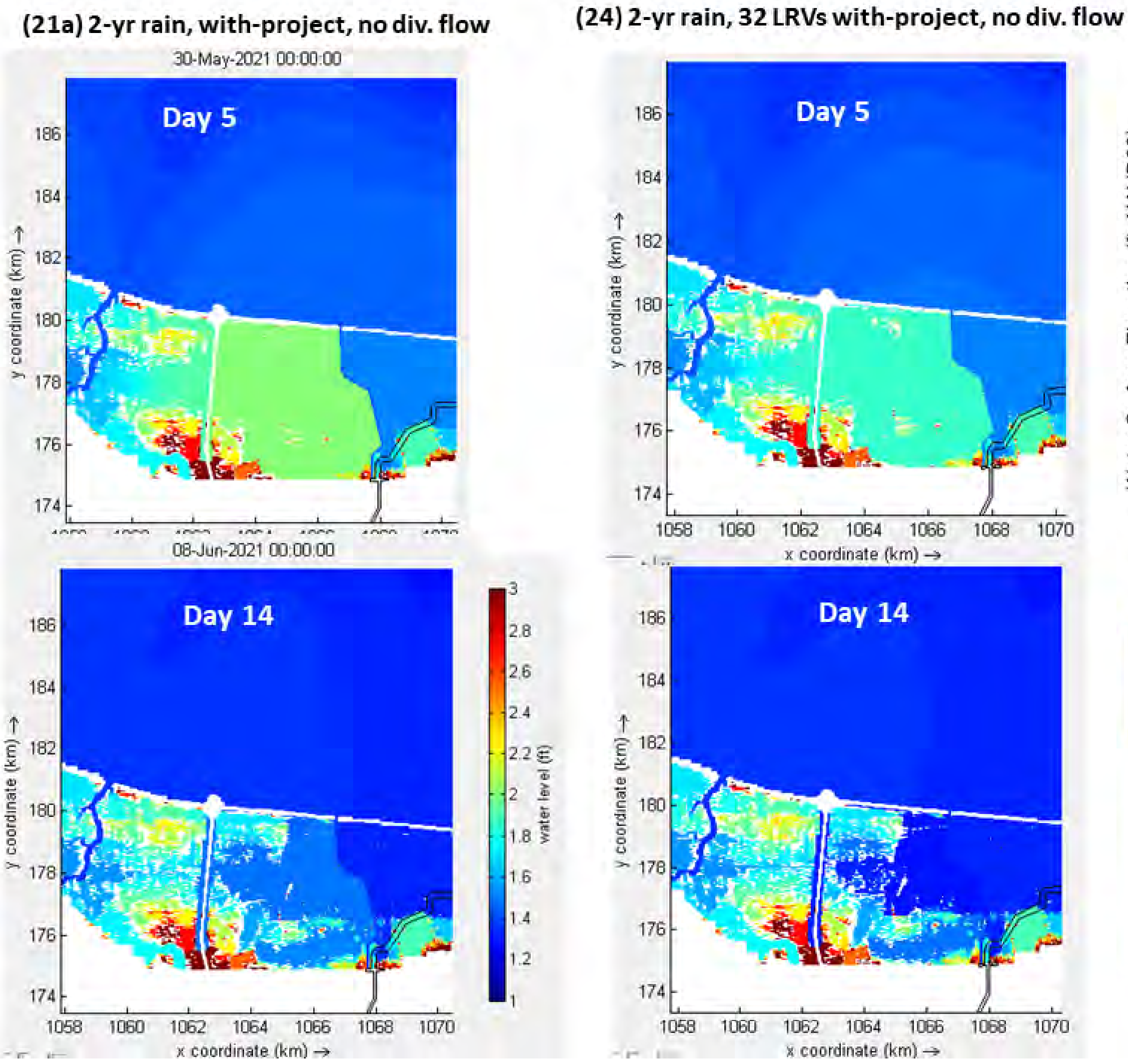
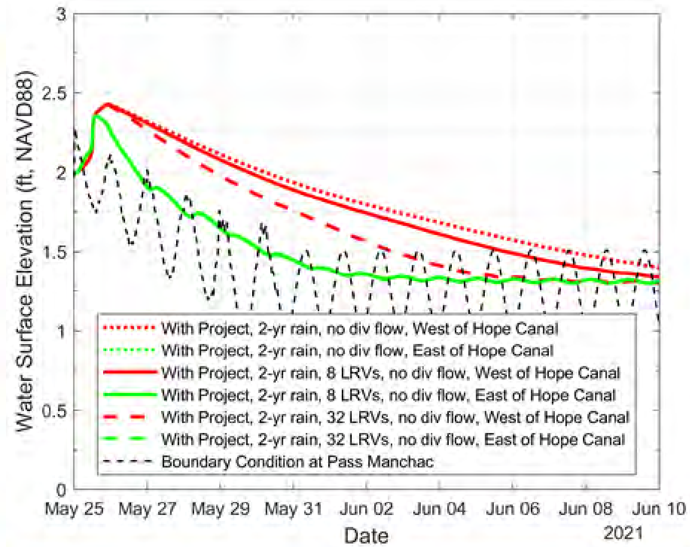



Figure G11. Polder water levels due to Lateral Release Flow 140 cfs on each side for the first 7 days (No rain, with-project, diversion flow of 2,000 cfs).




| 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).

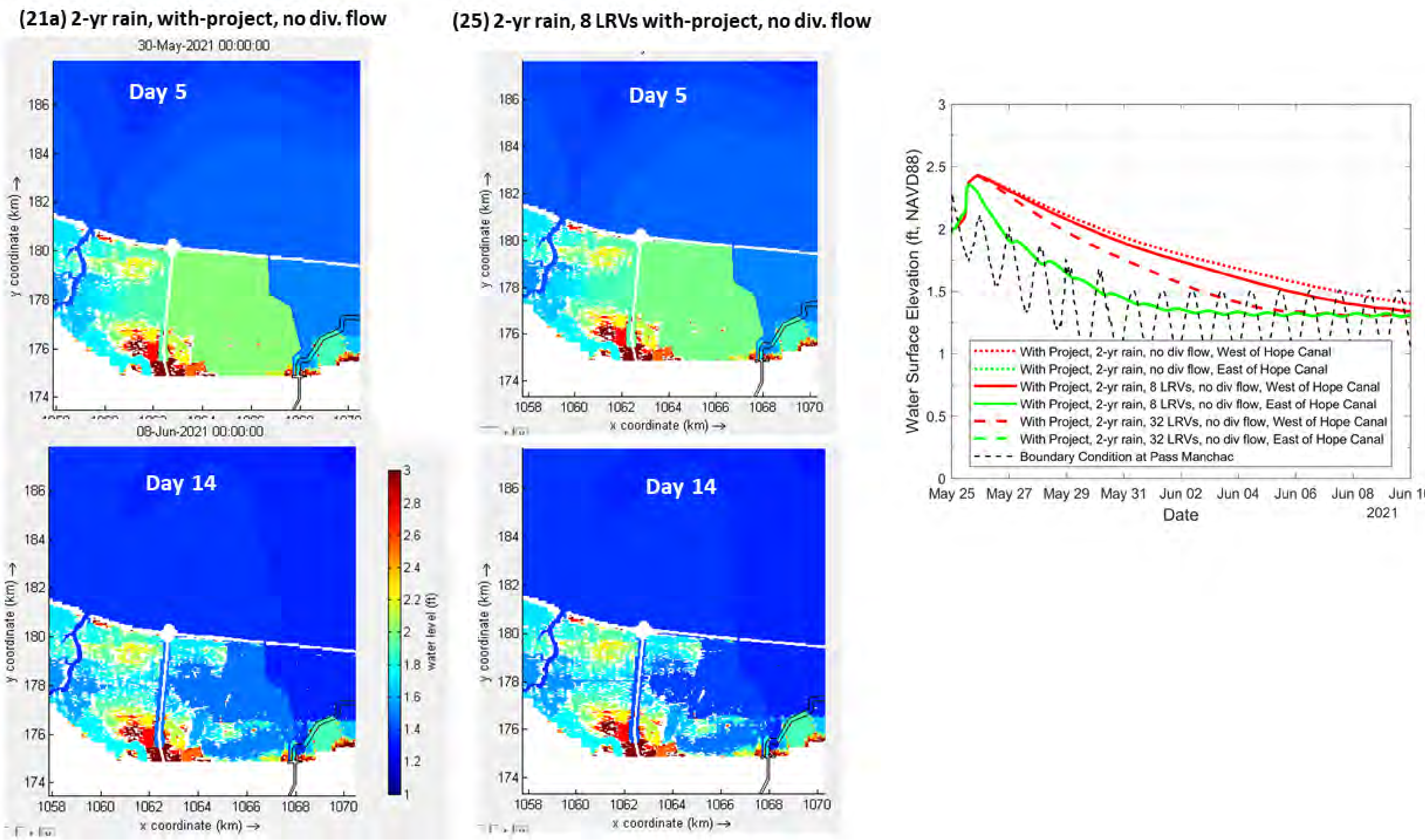
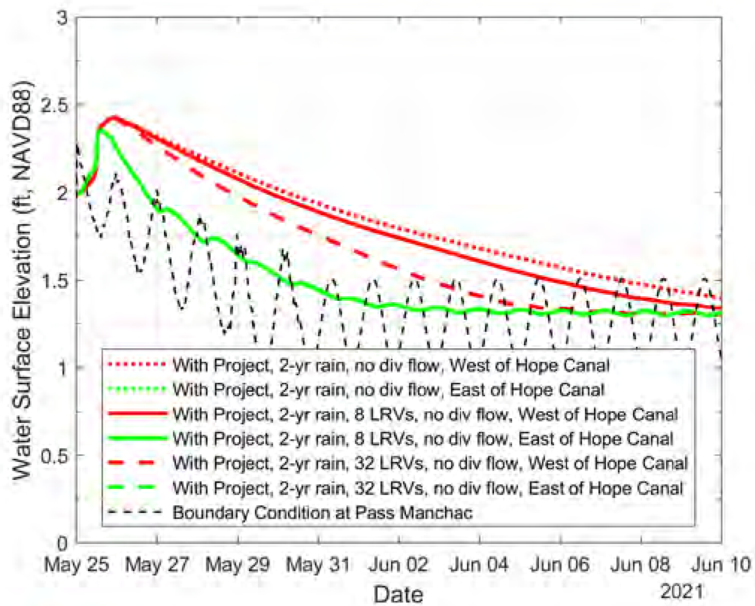
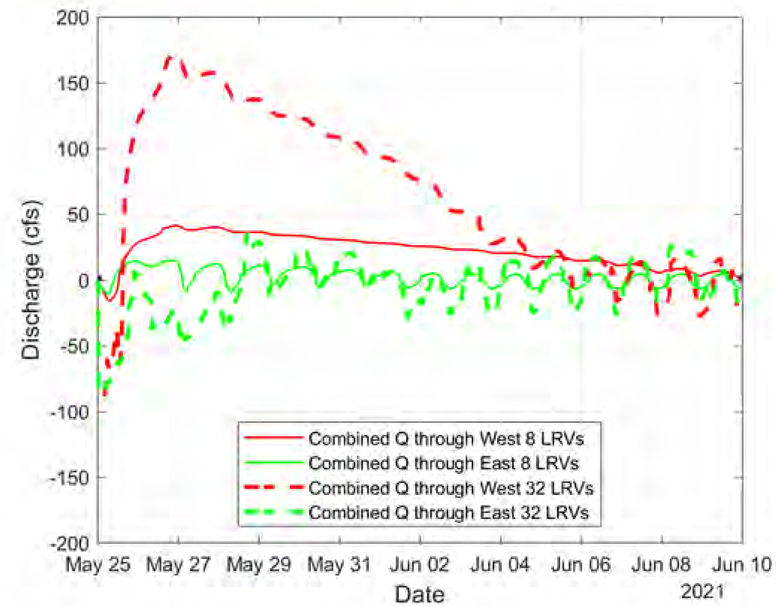



Figure G13. Effect of 8 lateral release valves on the polder water levels (25-year rainfall, with-project, no diversion flow).

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



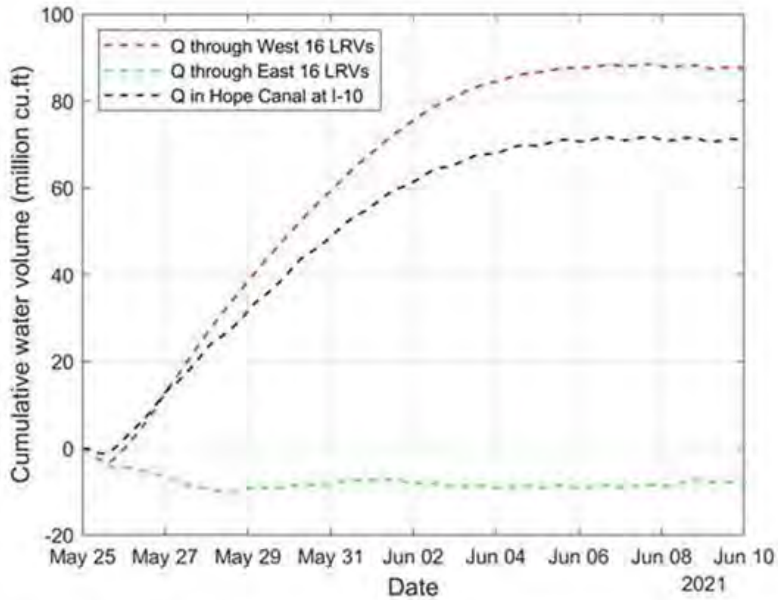




- Red lines are the water levels in the west polder
- Green line is close to the water levels in Hope Canal
- Difference between the two, shows head across the LRVs

- 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.

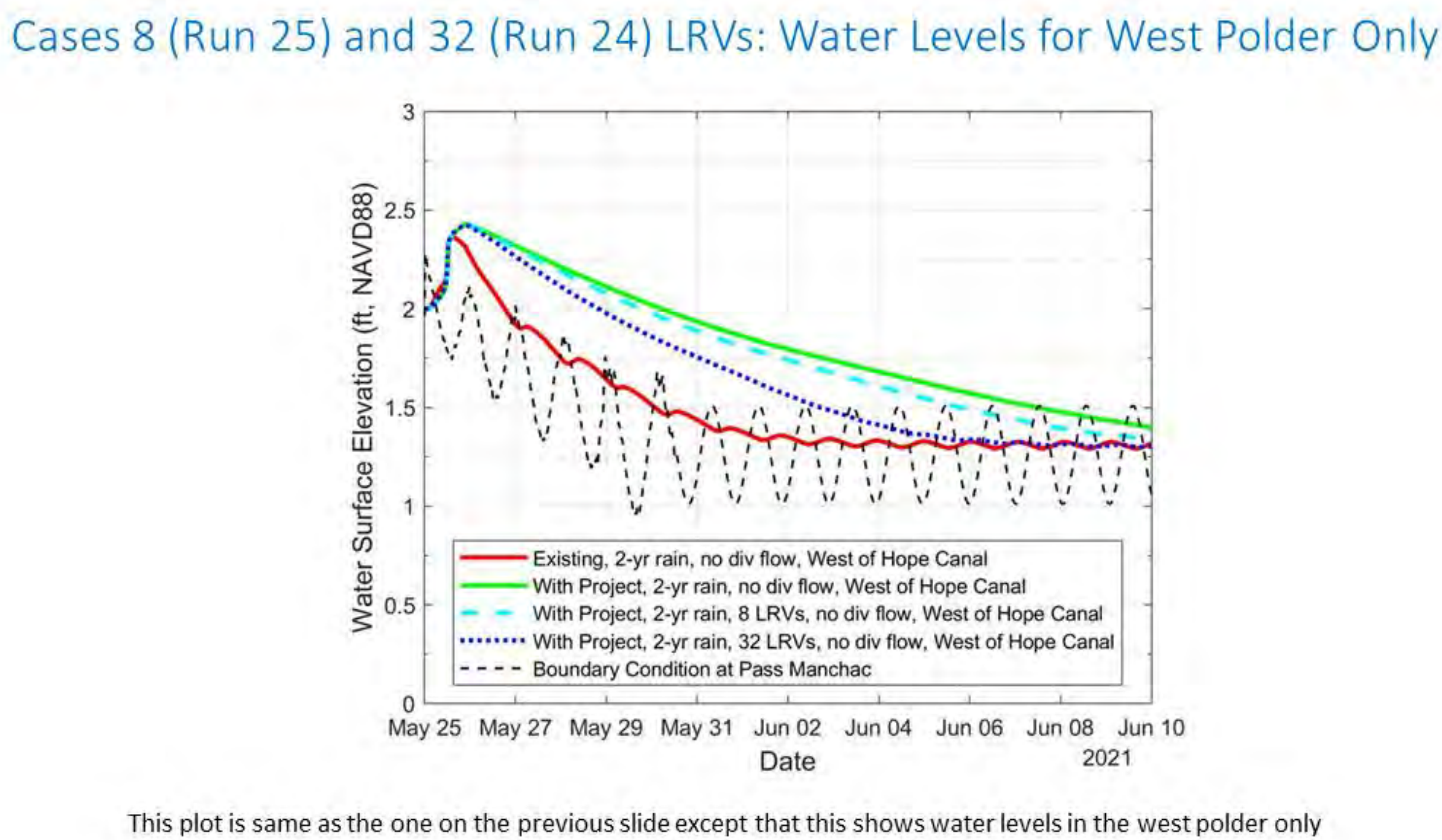
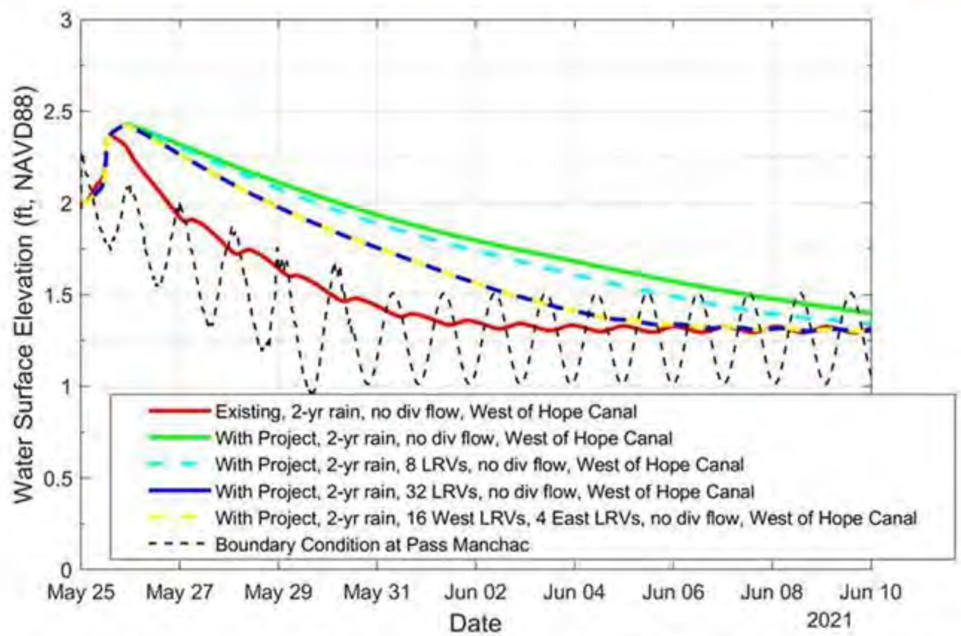



Figure G16. Comparison of water levels in west polder for the 8 and 32 later release valves scenarios (2-year rainfall).

## 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.

# Delft 3D Water Quality & Polder Drainage Model

# Mississippi River Reintroduction into Maurepas Swamp (PO-0029)

## Delft3D Water Quality & Polder Drainage Modeling

### Summary of Model Setup, Scenarios & Results

This slide deck is prepared by combining 3 submittals. It is divided into 3 sections based on the 3 deliveries to the CPRA team.

| | | |
|---|---|---|
| Section 1<br>Slides 1-22 | Presented<br>12/14/2020 | Water Quality Runs (Water Surface Elev, Nitrogen, Phosphorous)<br>Year 0 Runs: FWOP, FWP (250-, 1000-, 2000 cfs)<br>Year 50 Runs: FWOP, FWP (2000 cfs) |
| Section 2<br>Slides 23-39 | Presented<br>9/1/2021 | Preliminary Evaluation of Polder Drainage<br>Model Results – Water Surface Elevations |
| Section 3<br>Slides 40-49 | Delivered<br>12/2021 | Lateral Release Valves (LRVs) Runs |

FTN Associates

December 17, 2021

# Section 1

# Water Quality Modeling
# Model Results – Water Surface Elevation, Velocity, Nitrogen, Phosphorous

Year 0 Runs: FWOP, FWP (250-, 1000-, 2000 cfs)
Year 50 Runs: FWOP, FWP (2000 cfs)

# Delft3D Model Runs Plan

✓ **Year 0 Simulations - completed**

| Run ID | Project | Diversion Inflow | Model Output |
|---|---|---|---|
| 10 | FWOP | N/A | Water level, depth, velocity |
| 11 | FWP | 250 cfs | Water level, depth, velocity, salinity |
| 12 | FWP | 1,000 cfs | Water level, depth, velocity, salinity |
| 13 | FWP | 2,000 cfs | Water level, depth, velocity, salinity, TN, TP |

✓ **Year 50 Simulations – completed**

| Run ID | Project | Diversion Inflow | Model Output |
|---|---|---|---|
| 50 | FWOP | N/A | Water level, depth, velocity |
| 53 | FWP | 2,000 cfs | Water level, depth, velocity, salinity, TN, TP |

*All above scenarios were run for the same 20-day period with continuous inflow from the diversion, normal tides and no rainfall.*
*All scenarios included the proposed West Shore Lake Pontchartrain (WSLP) levee.*

# Development of Year 50 Model Bathymetry and Boundary Water Level

- Basis – Technical note by Leigh Anne Sharp (CPRA, 2020).

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

- Applied above values assuming that the effective accretion occurs above 0 ft, NAVD88. Below this elevation, apply only subsidence (Similar approach in 2017 Coastal Master Plan Attachment C3-22: Integrated Compartment Model, ICM, Development; Jung 2016)

- FWOP: (Yr 50 bed elevations)

Net subsidence = 7.1-5.0 = 2.1 mm/ yr or (2.1*55)=0.116 m (0.378 ft) in 55 yrs. Therefore, in the model geometry

Nodes with bed elevations above 0 ft, NAVD88 were lowered by 0.378 ft

Nodes with bed elevations below 0 ft, NAVD88 were lowered by 1.28 ft (= 7.1 x 55 = 0.391 m)

- FWP: (Yr 50 bed elevations)

Net subsidence = 7.1-10.0 = -2.9 mm/yr or -0.160 m (-0.525 ft) in 55 yrs. Therefore, in the model geometry

Nodes with bed elevations above 0 ft, NAVD88 were raised by 0.525 ft

Nodes with bed elevations below 0 ft, NAVD88 were lowered by 1.28 ft (= 7.1 x 55 = 0.391 m)

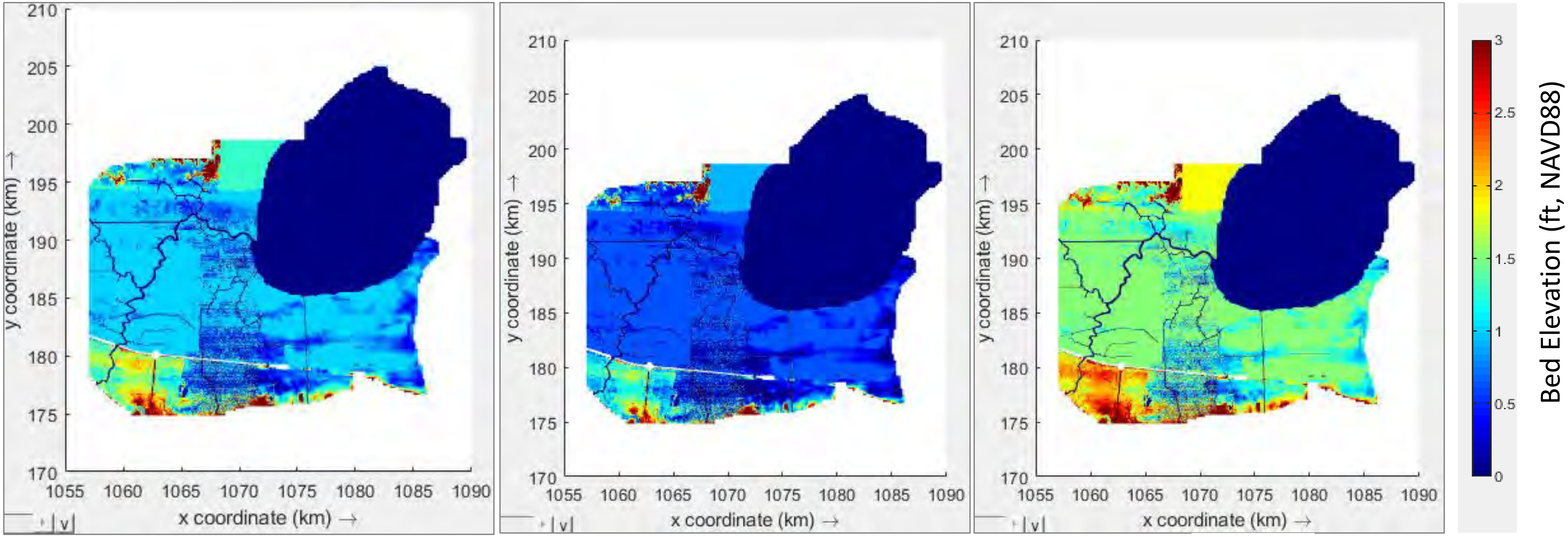
- Boundary water level adjusted for sea level rise

Added 2.1 ft (0.64 m) to the Yr 0 water level boundary at Pass Manchac.

| 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 |

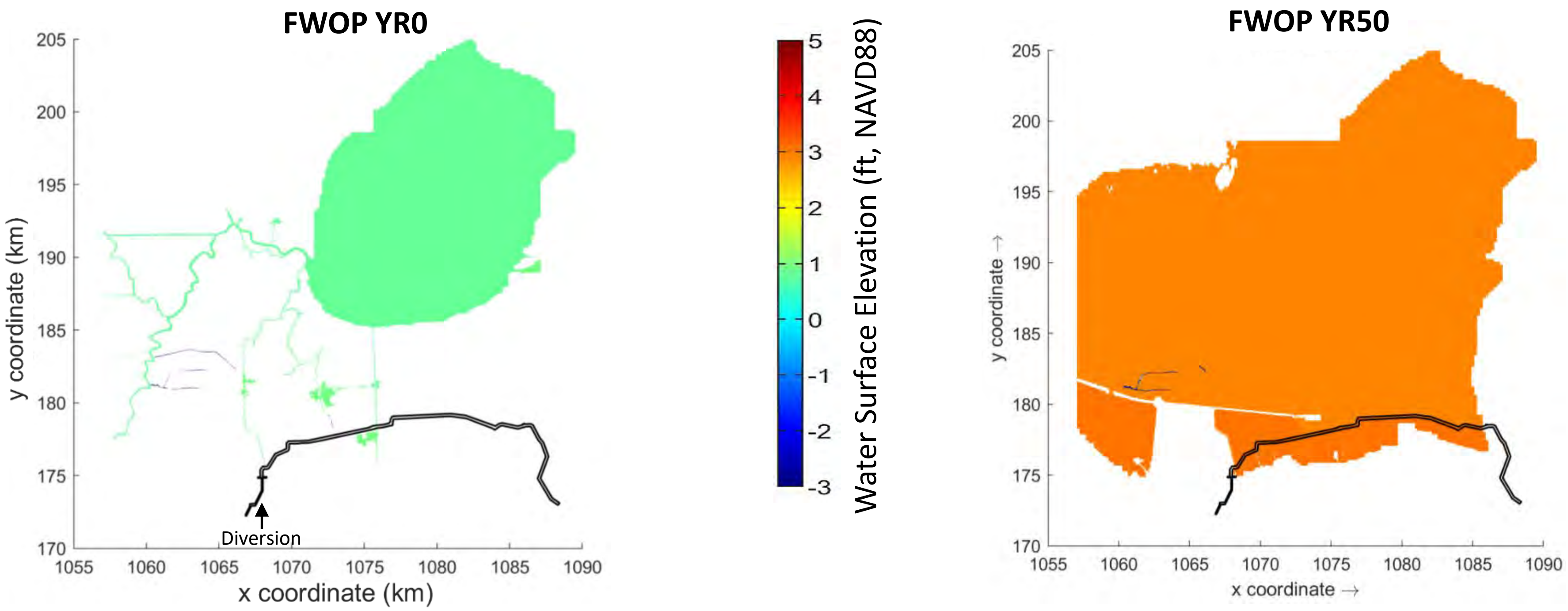
# Year 0 and Year 50 Model Bathymetry

**YR0 FWOP & FWP** **YR50 FWOP** **YR50 FWP**




**YR0 FWOP & FWP** **YR50 FWOP** **YR50 FWP**

# Delft3D Model Results – A Quick Look - General normal water levels FWOP




Note:

In the YR50 scenarios, water moves south of Interstate-10 through Blind River and the Mississippi Bayou.

In reality, such flow will not be allowed. So this inundation can be ignored.

This has no impact on the results north of Interstate-10.

# Notes on the Presented Model Results

Year 0 Runs

- The FWOP and FWP model bathymetry is the same. Therefore, the difference between the two scenarios is

  - the addition of the diversion flow

Year 50 Runs

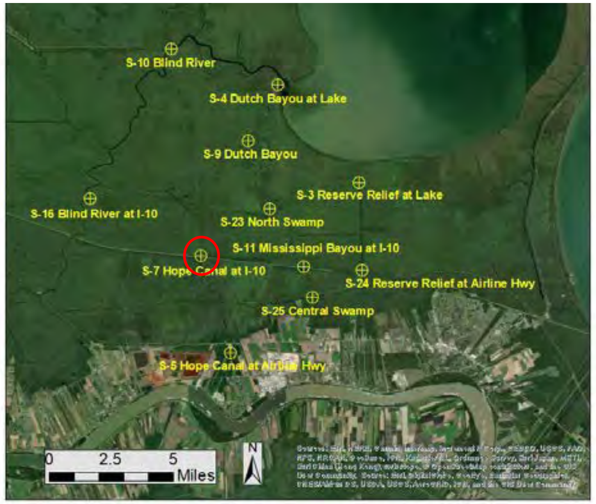
- The FWOP and FWP model bathymetry is NOT the same. Therefore, the difference between the two scenarios is

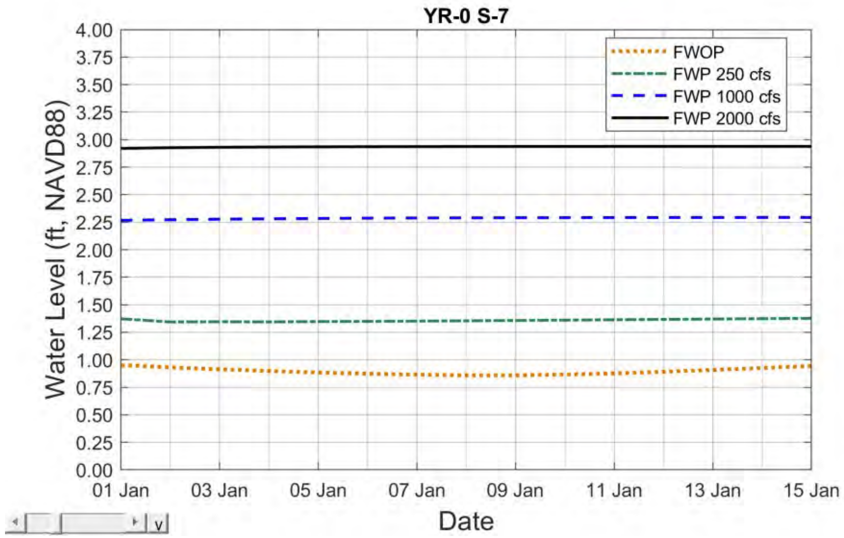
  - the addition of the diversion flow, AND

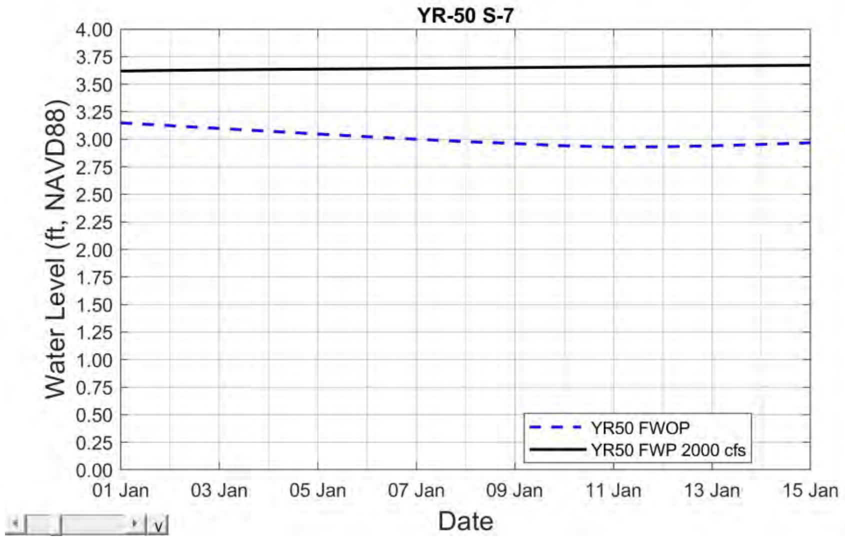
  - the bathymetry

- All runs are 20 day duration. For charts, the last 15 days are shown. Generally, the first couple of days are needed for model "spin-up" when the solutions stabilize.

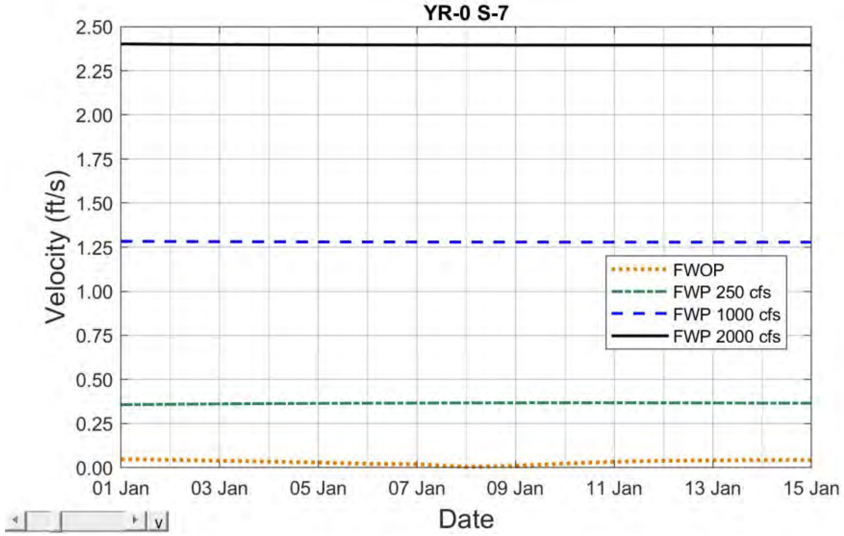
# Delft3D Model Results – Location S-7 (Outfall at I-10)

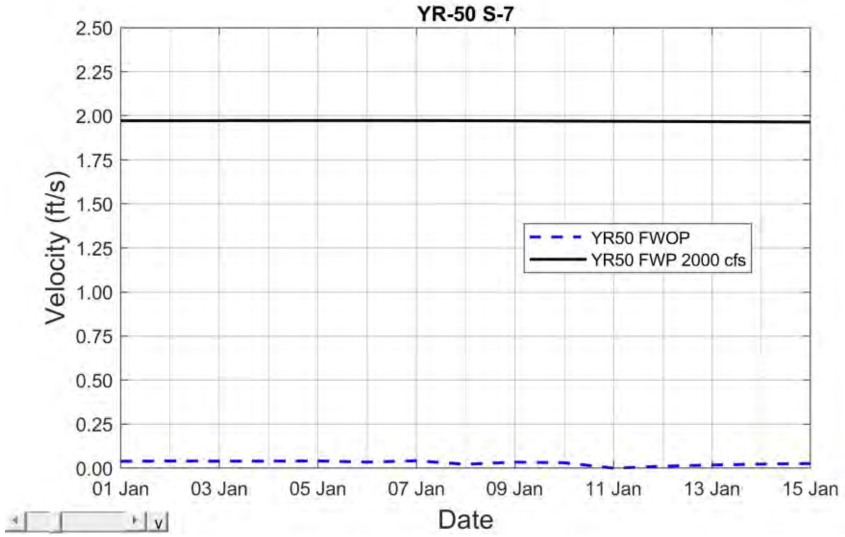


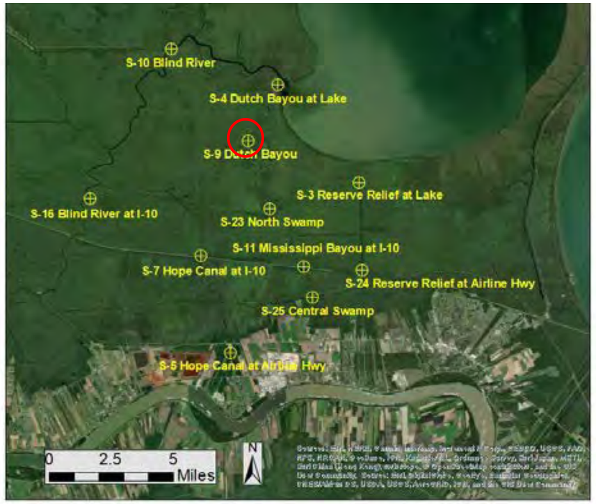

Highest water levels and velocity magnitude are in Hope Canal as it exits I-10 bridge

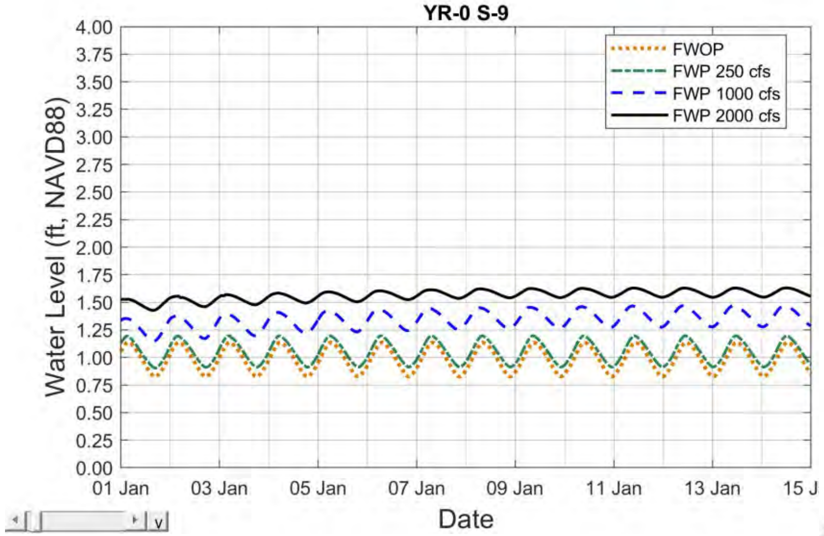


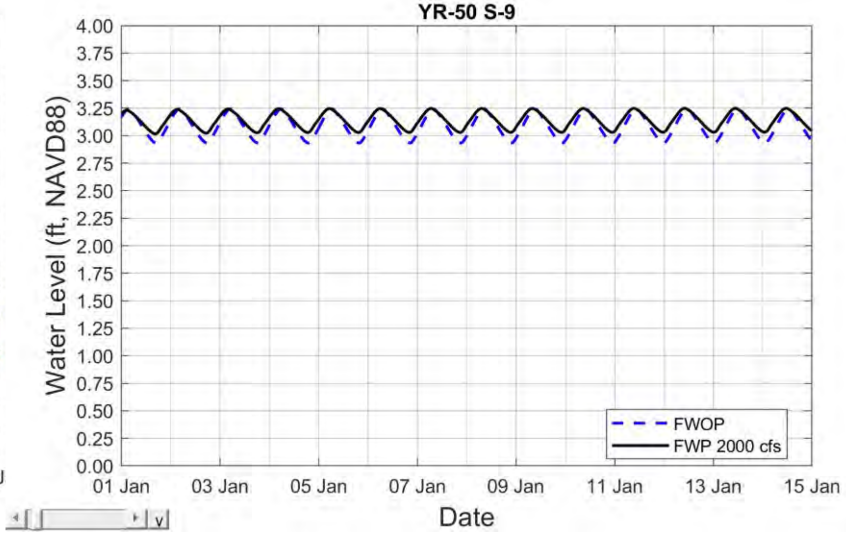



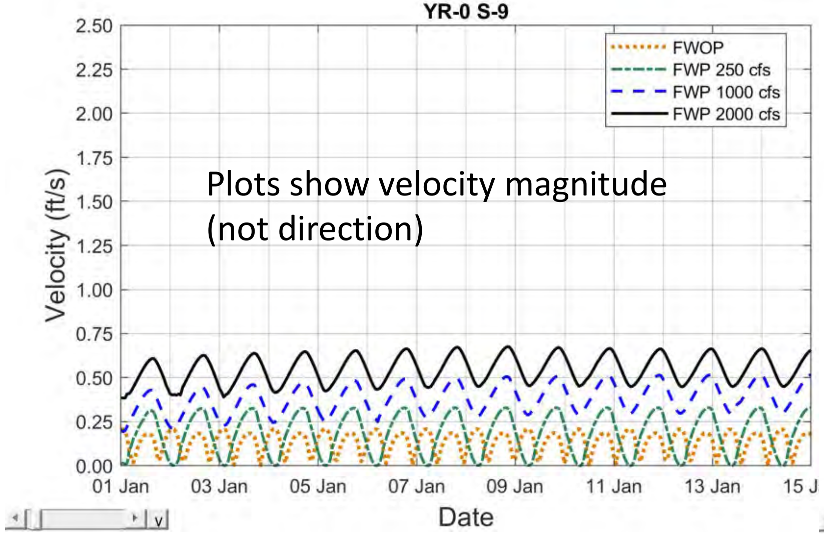

<u>YR 0</u>
Normal is 1 ft, NAVD88
250 cfs adds 0.3 ft
1000 cfs adds 1.3 ft
2000 cfs adds 1.9 ft
<u>YR 50</u>
Normal is 3 ft, NAVD88
2000 cfs adds 0.6 ft

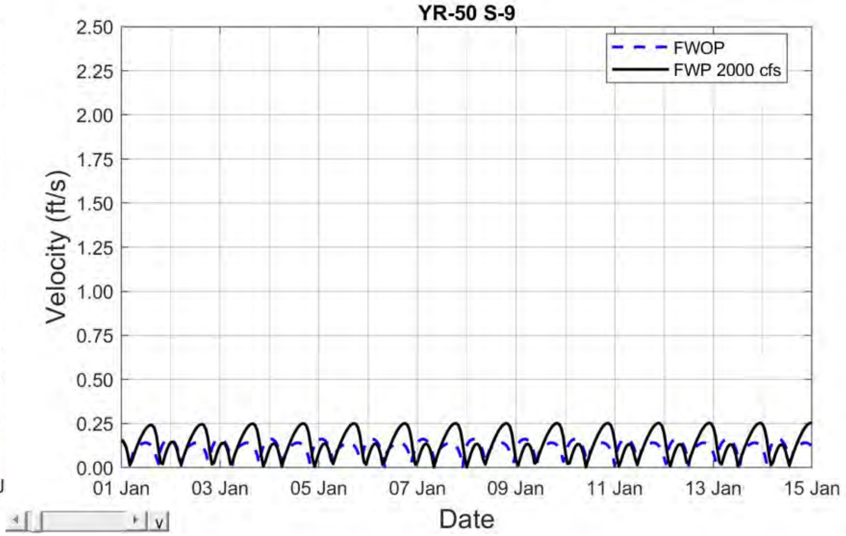
# Delft3D Model Results – Location S-9 (Dutch Bayou)



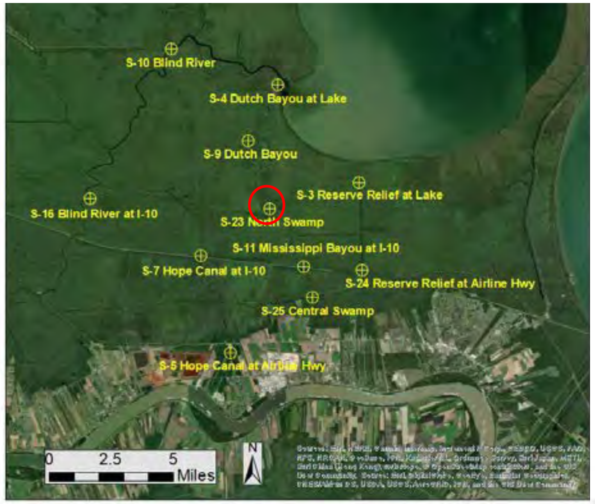



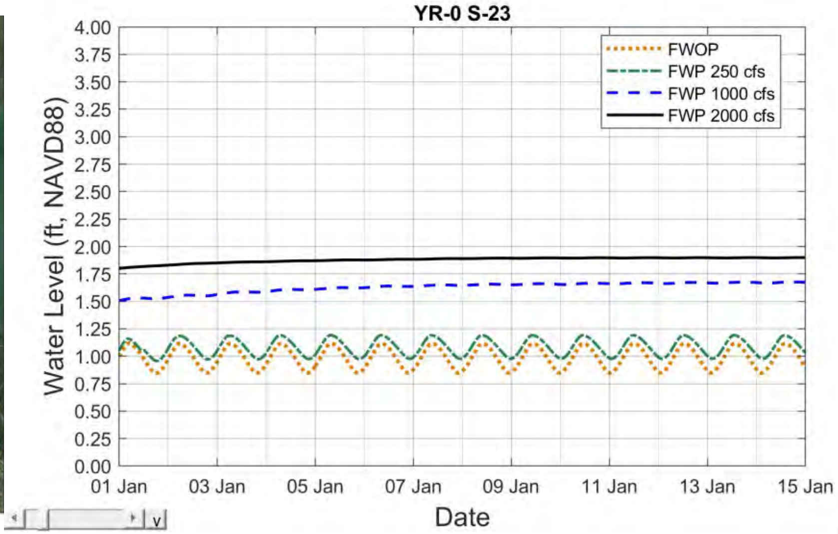



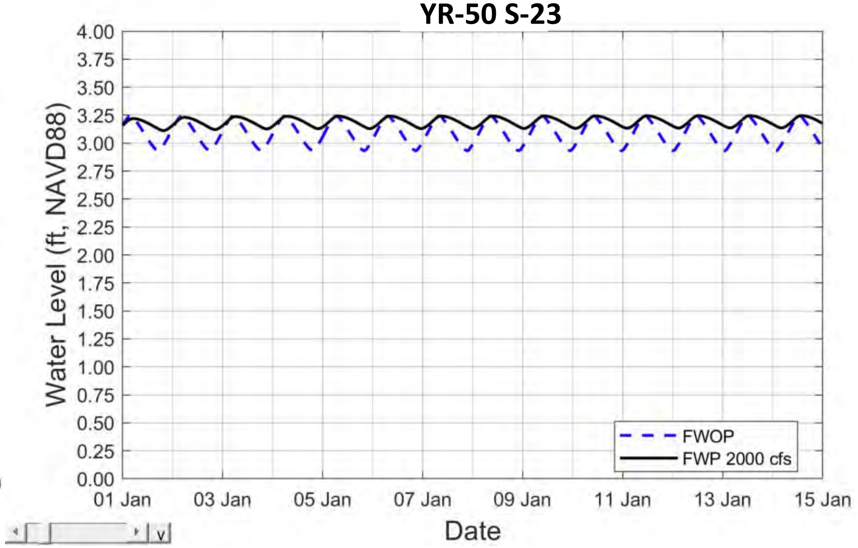



# Delft3D Model Results – Location S-23 (Mississippi Bayou)




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

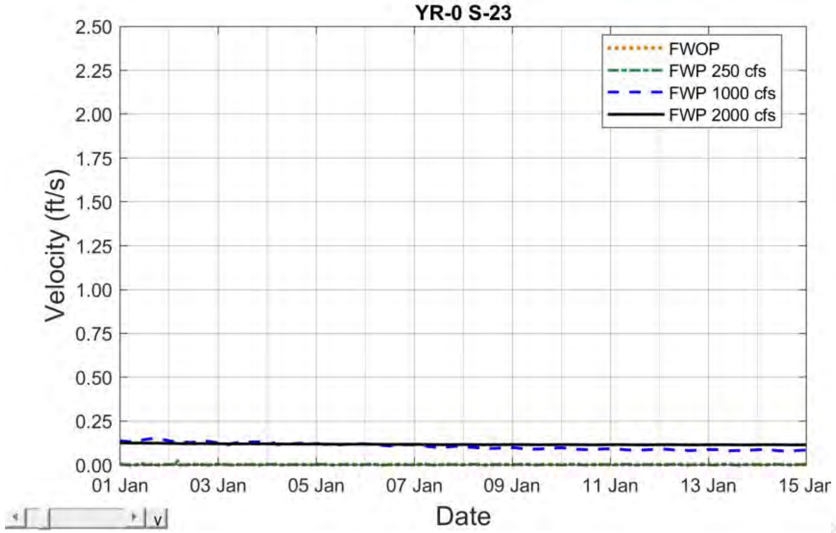
1000 cfs adds 0.7 ft

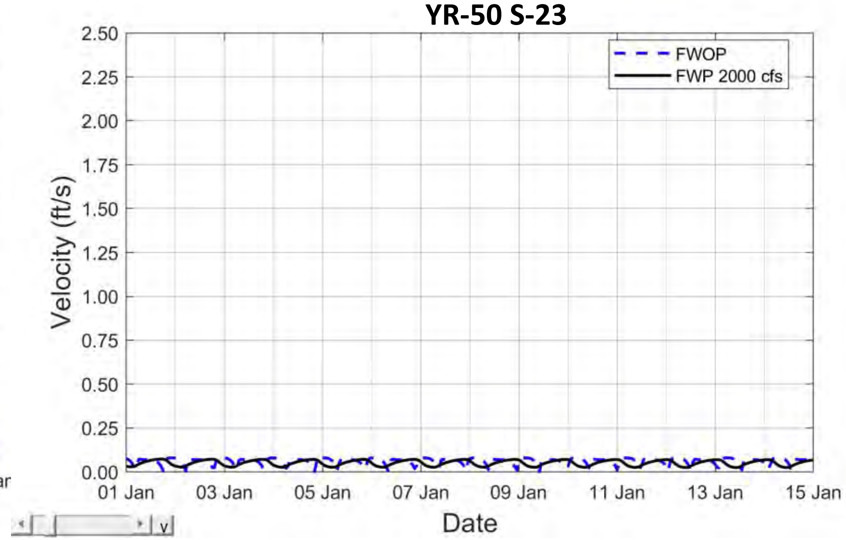
2000 cfs adds 0.9 ft

<u>YR 50</u>

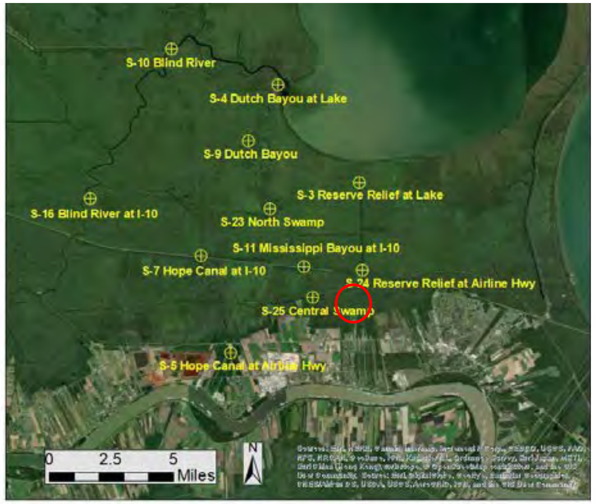
Normal is 3 ft, NAVD88

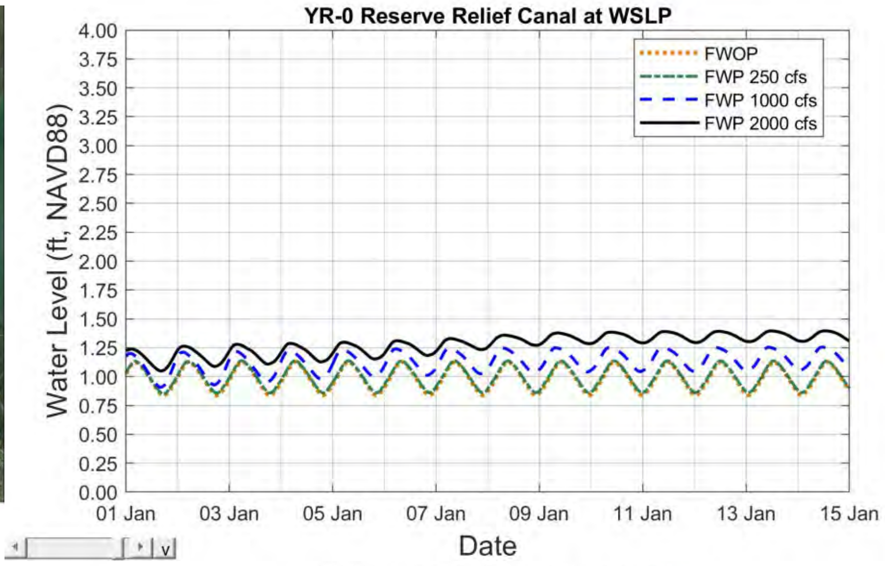
2000 cfs adds 0.2 ft

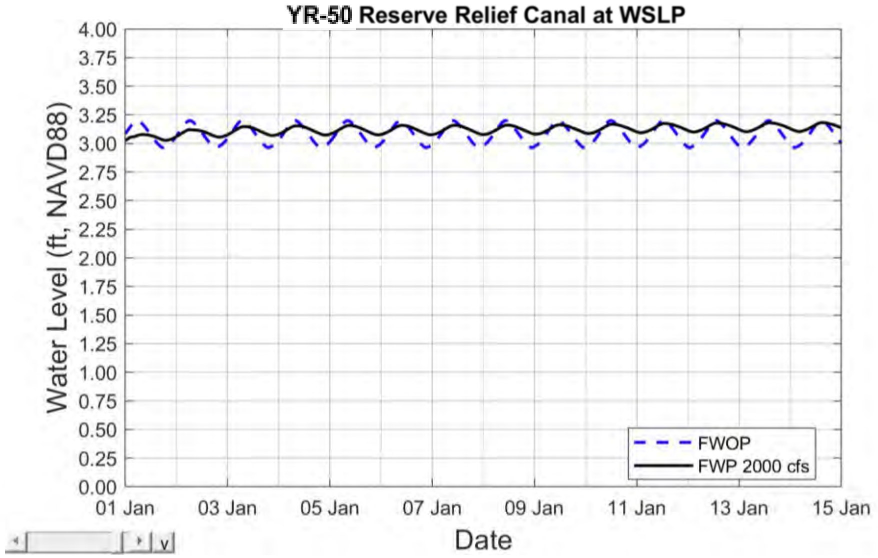


# Delft3D Model Results – Location Relief Canal at WSLP



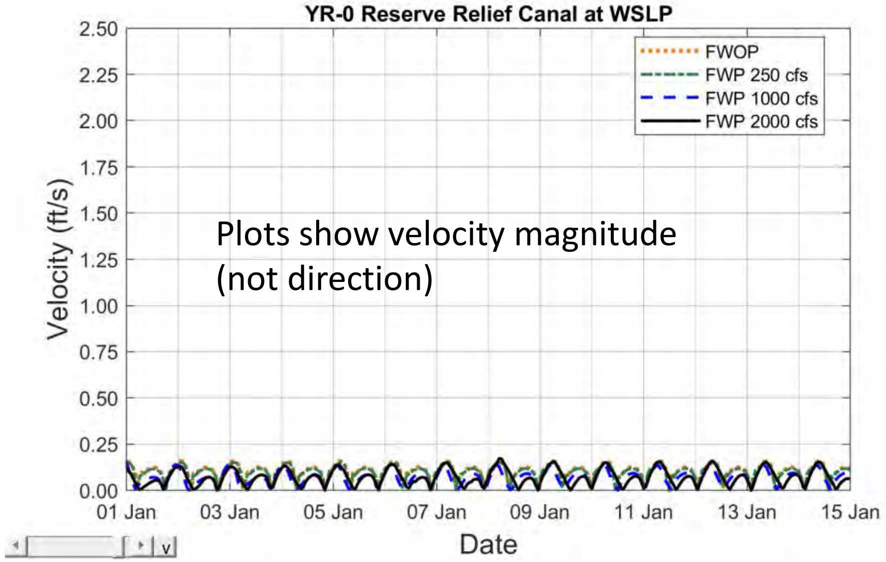



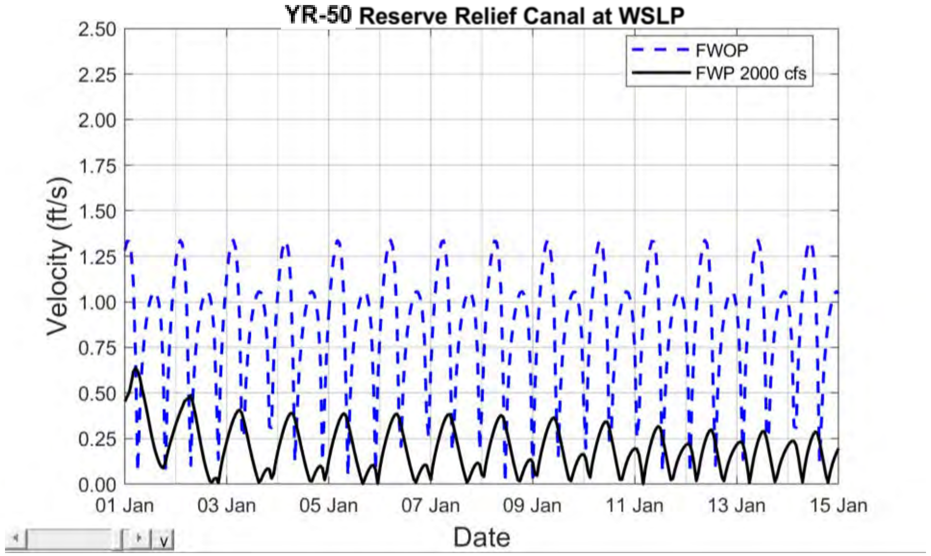




Indication of effects near the WSLP drainage structures

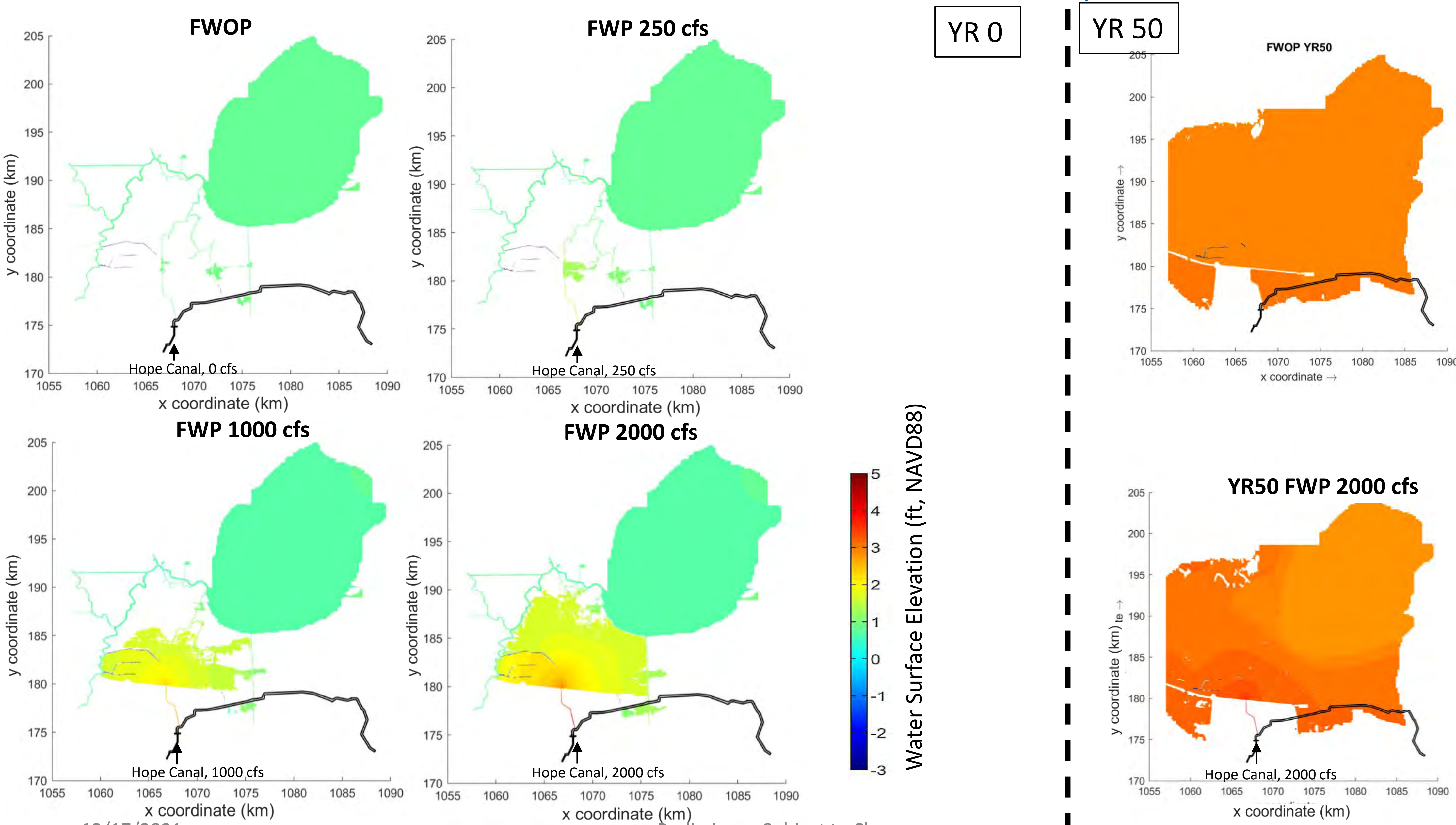
<u>YR 0</u>
Normal is 1 ft, NAVD88
250 cfs has no effect
1000 cfs adds 0.2 ft
2000 cfs adds 0.3 ft

<u>YR 50</u>
Normal is 3 ft, NAVD88
2000 cfs adds 0.1 ft



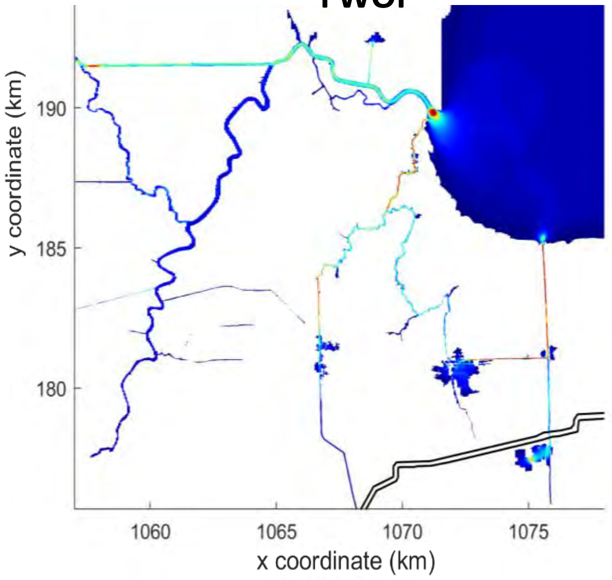
# Water Surface Elevation after 20 days



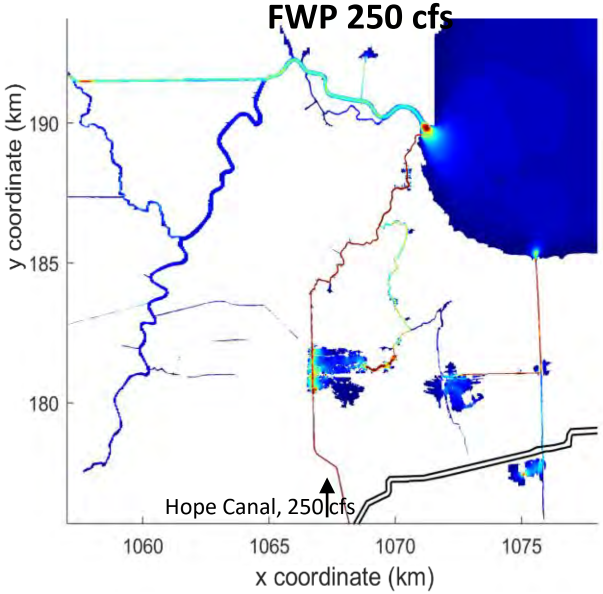
# Velocity after 20 days

YR 0

**FWOP**



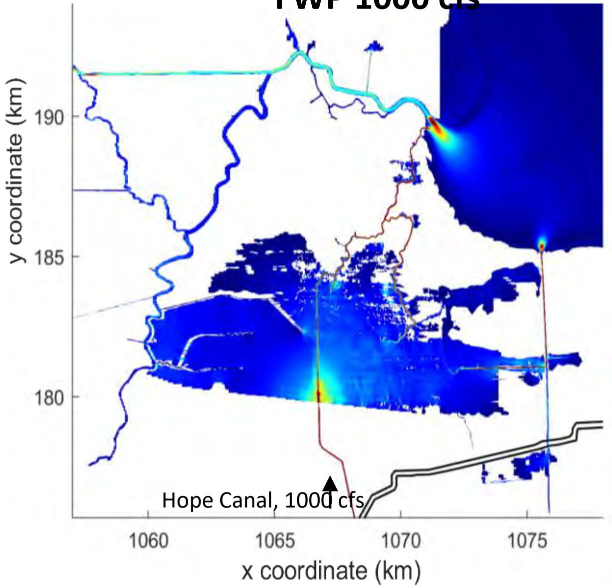

**FWP 250 cfs**



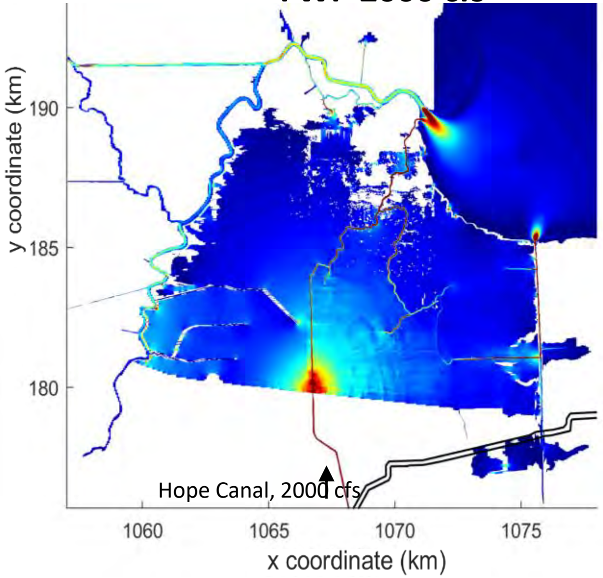

Note:
Velocity snapshot graphics show tidal influence more prominently than the previous water level snapshots. They could show different color variation depending on the tidal phase.

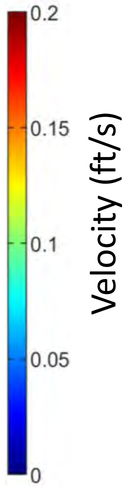
YR50 velocities are slightly larger due to shallower (accretion) bathymetry

**FWP 1000 cfs**

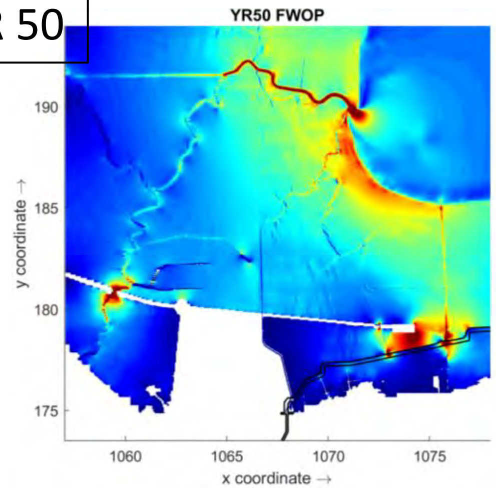



**FWP 2000 cfs**

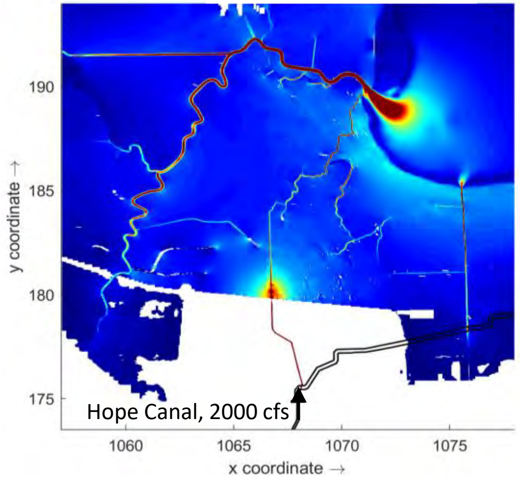



YR 50




**YR50 FWP 2000 cfs**

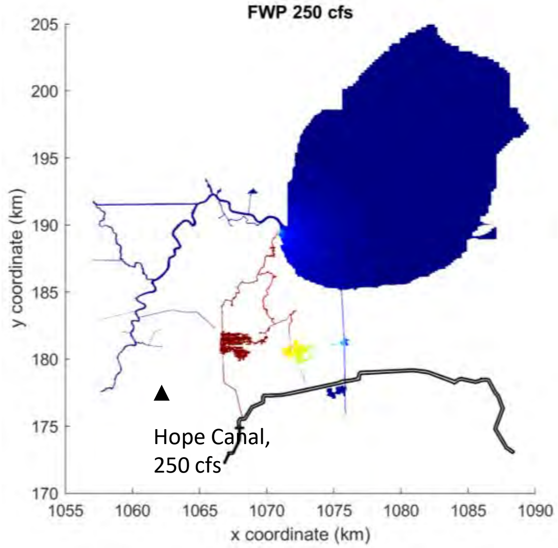


# % Mississippi River Water after 20 days

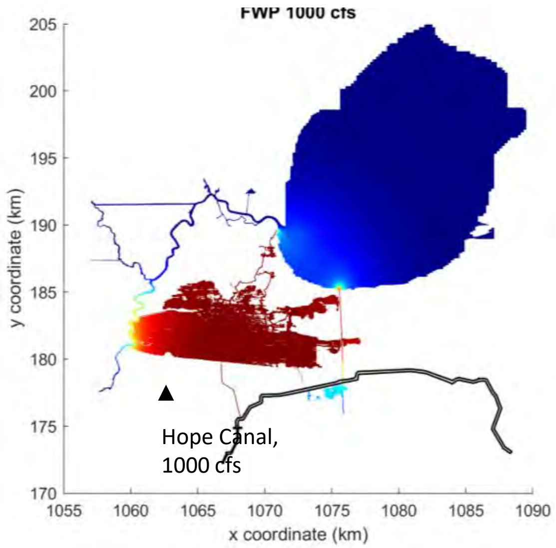
YR 0

YR 50

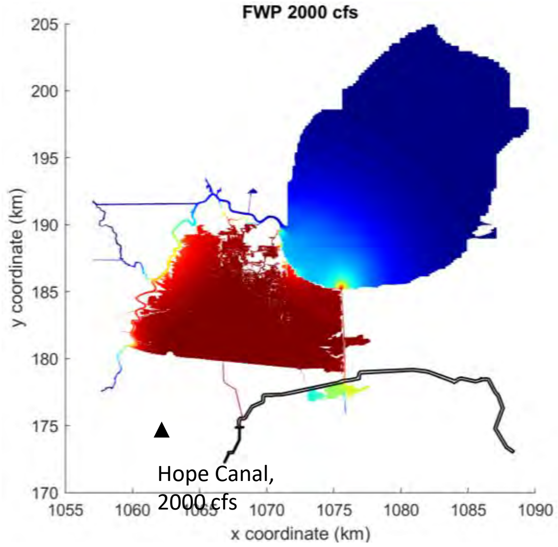
**FWP 250 cfs**

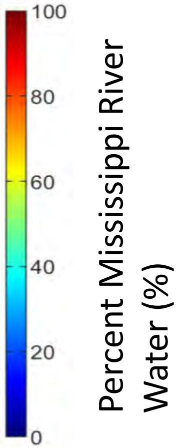



**FWP 1000 cfs**

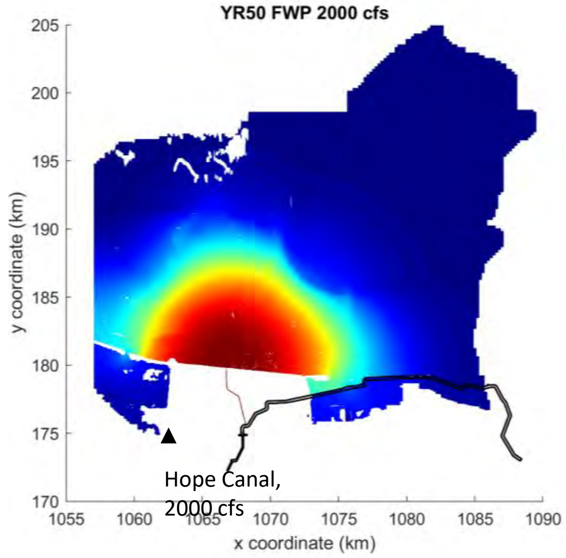



**FWP 2000 cfs**




**YR50 FWP 2000 cfs**





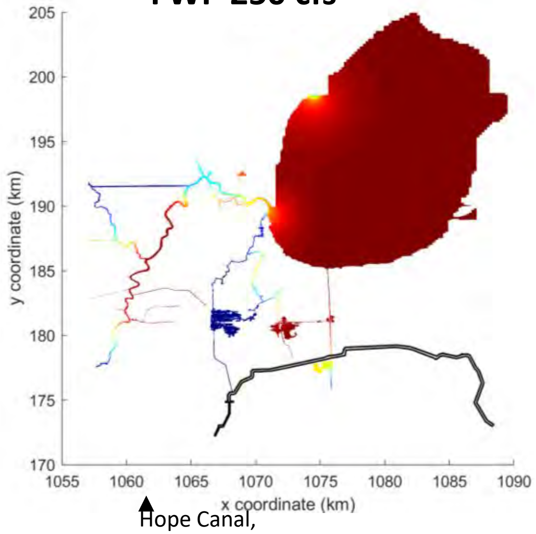

Preliminary. Subject to Change.

# Salinity after 10 days (Initial 5 ppt)

YR 0

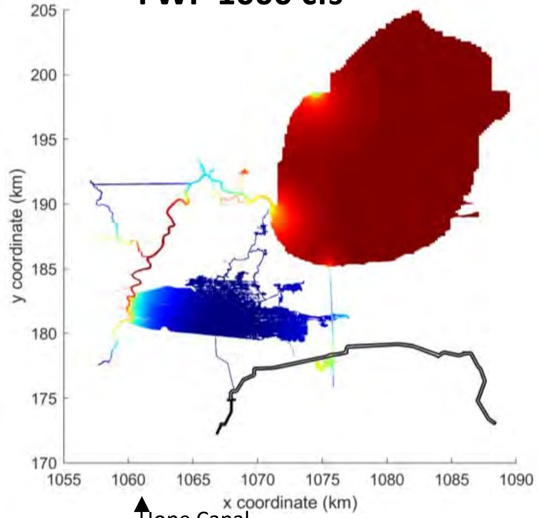
YR 50

**FWP 250 cfs**



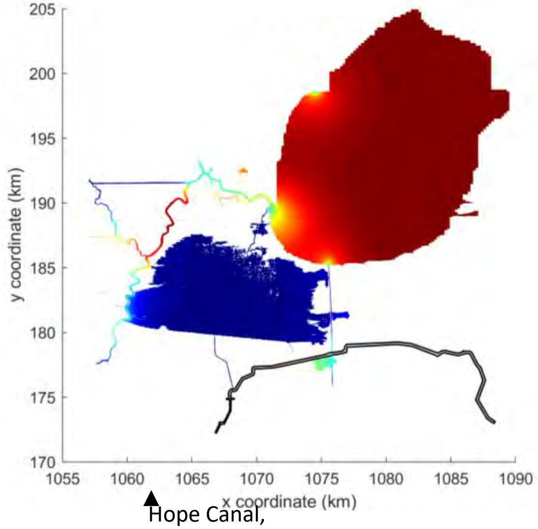

Hope Canal,
250 cfs

**FWP 1000 cfs**

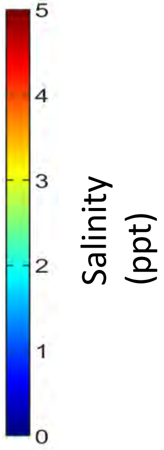



Hope Canal,
1000 cfs

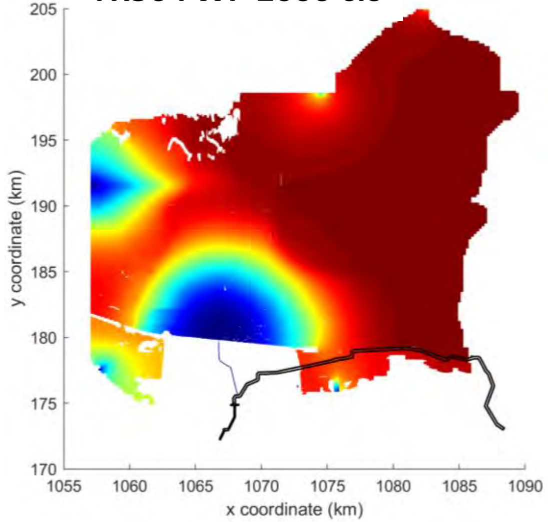
**FWP 2000 cfs**




Hope Canal,
2000 cfs



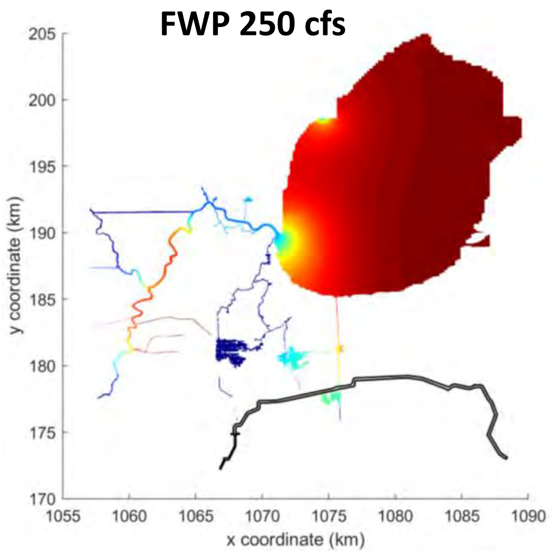

**YR50 FWP 2000 cfs**

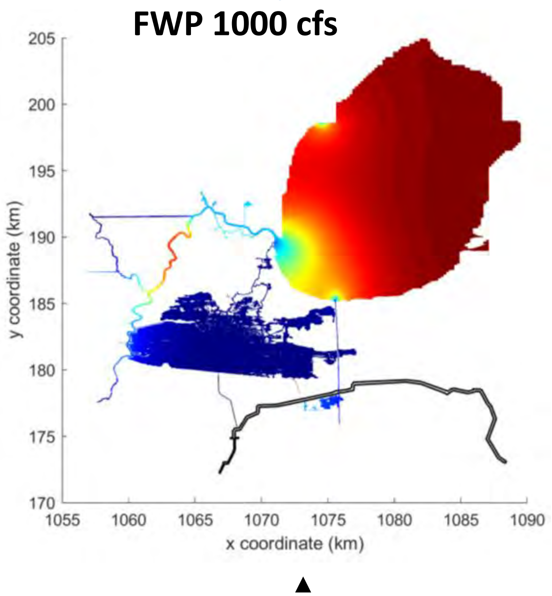


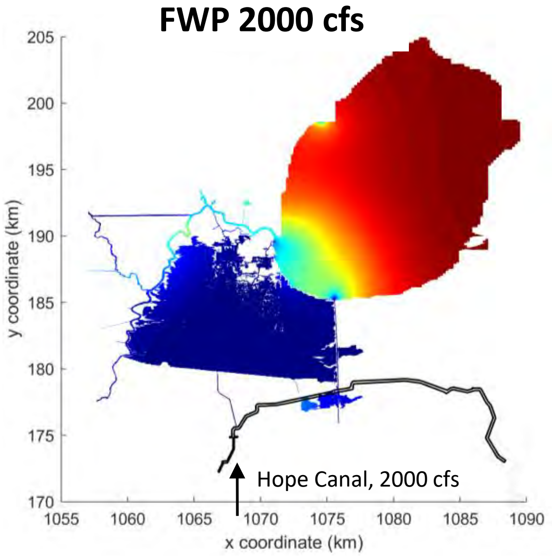
# Salinity after 20 days (Initial 5 ppt)

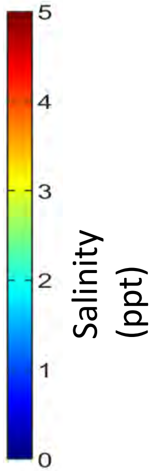
YR 0

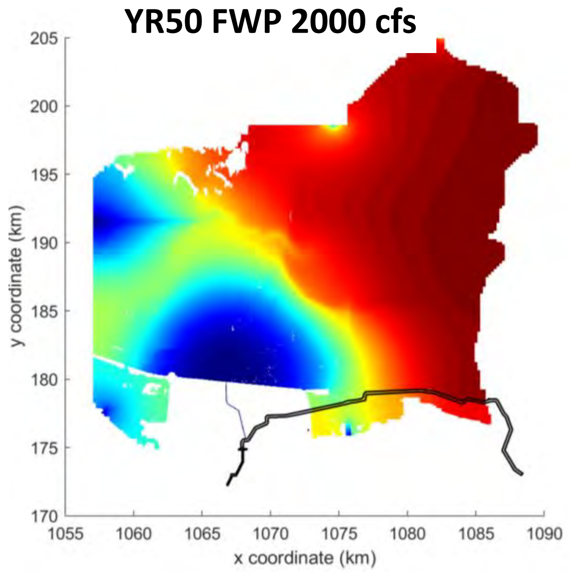
YR 50



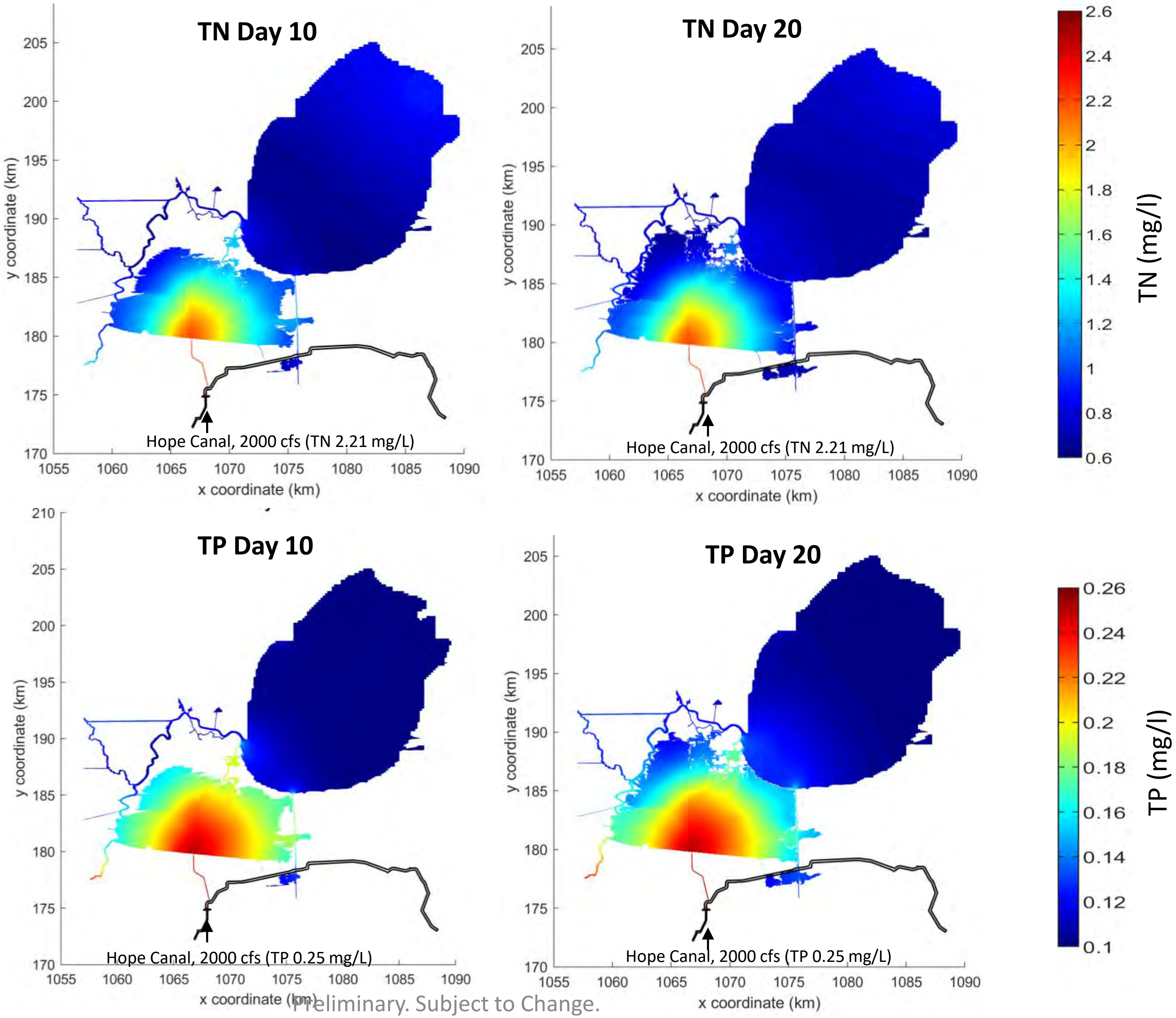
# TN, TP conc. after 10 & 20 days FWP-2000 cfs

## 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 |

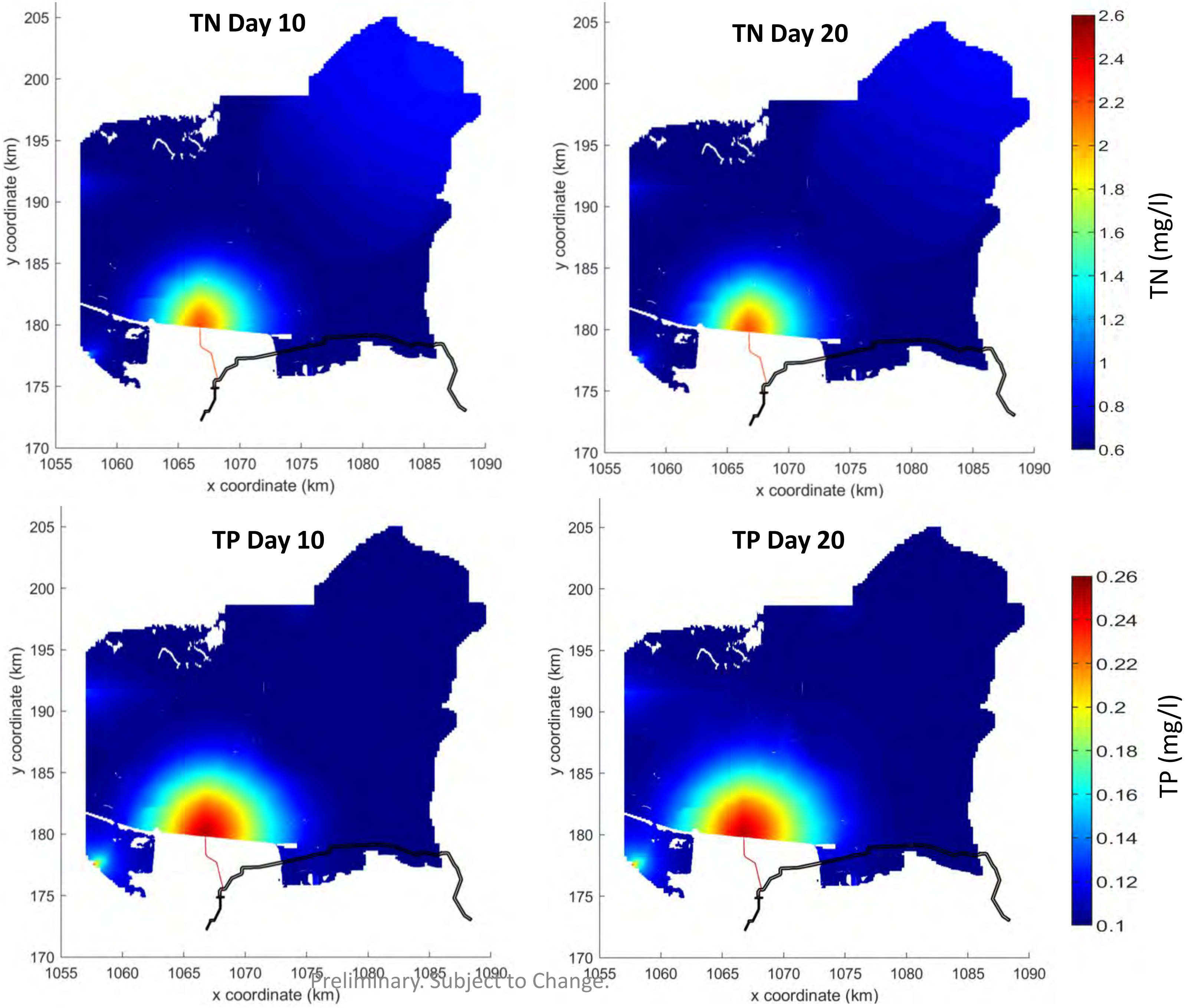
The average TN and TP concentrations for the Mississippi River for January 1 – August 31 are:

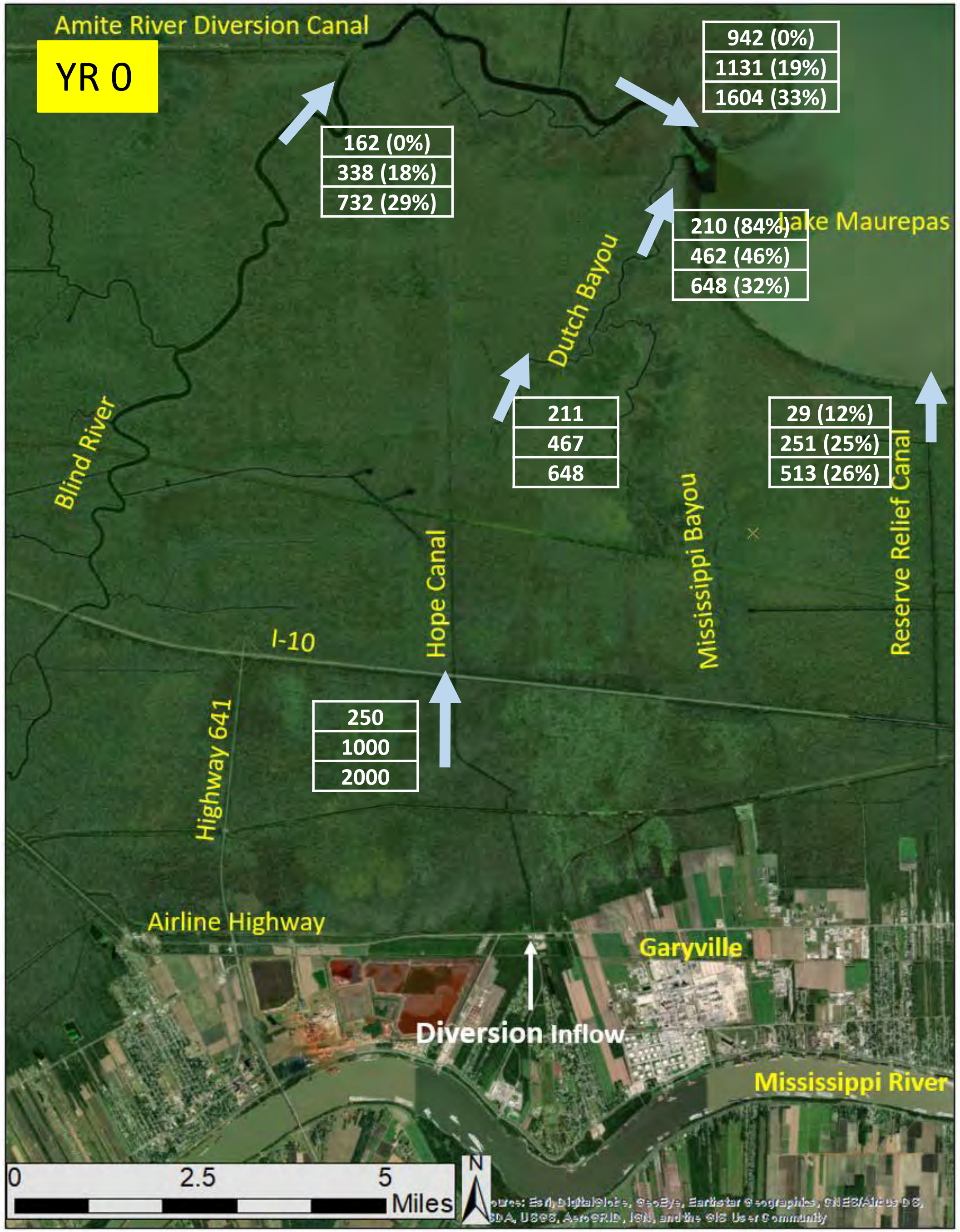
* Averages of USGS data from Baton Rouge and Belle Chasse during 2004 – 2018 (same period of record used previously).



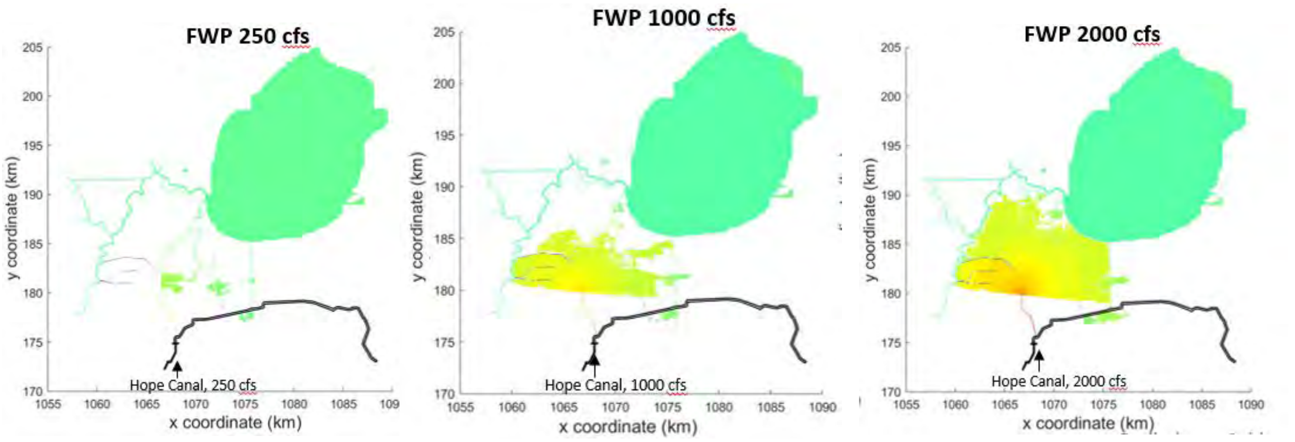
# TN, TP conc. after 10 & 20 days FWP-2000 cfs

YR 50



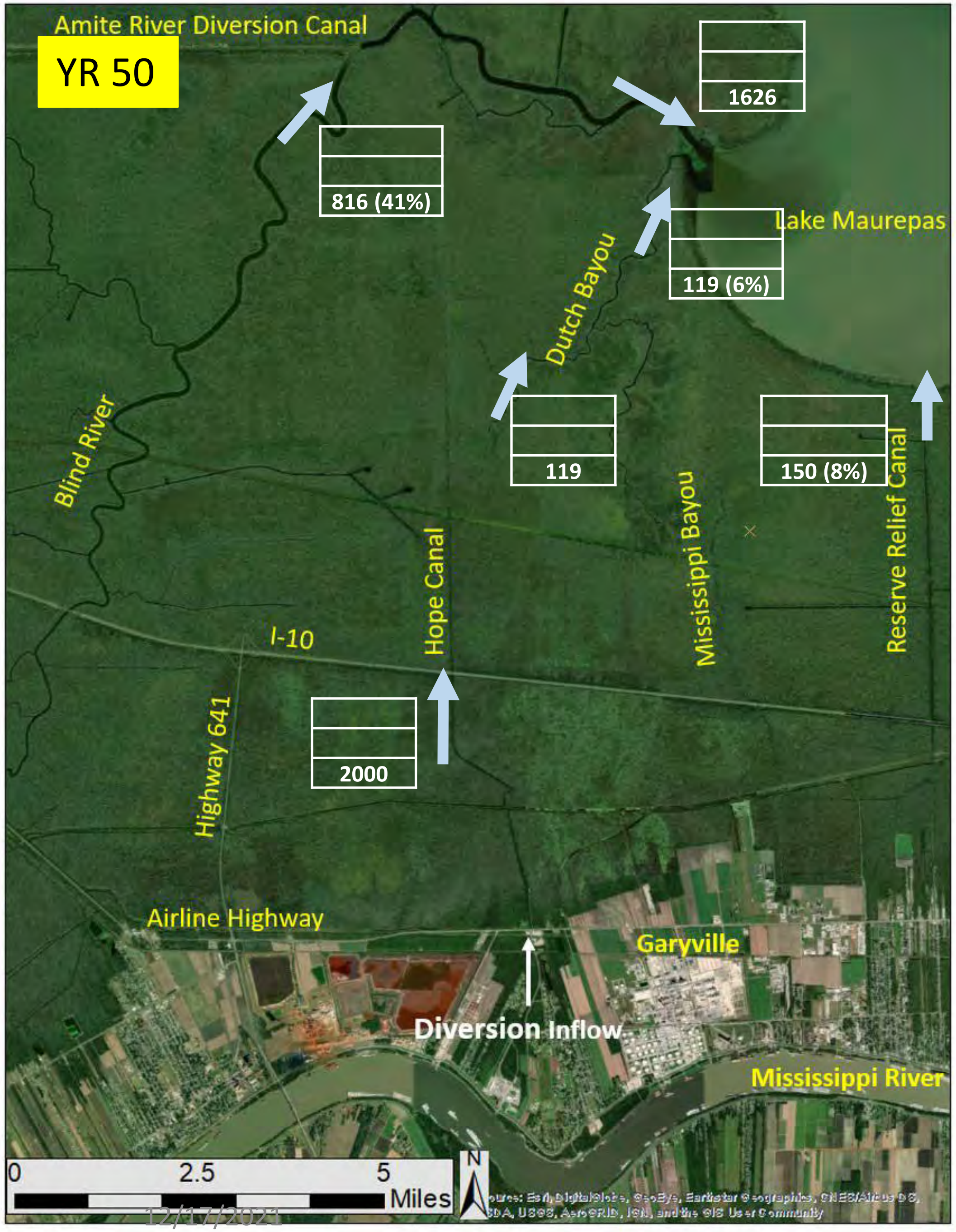



# General Flow Distribution – YR 0 FWP

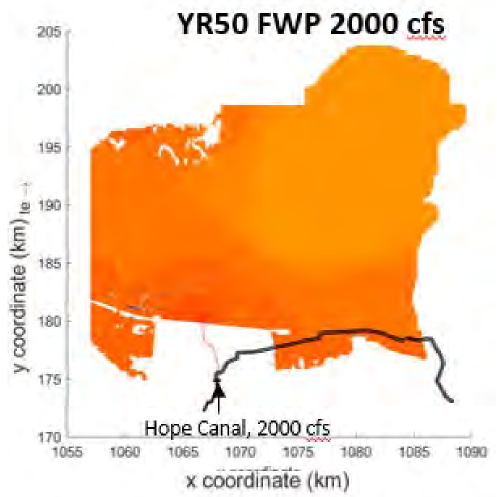



| Diversion Inflow (cfs) | Div. flow existing through Dutch Bayou (cfs) | Div. flow existing through the Blind River (cfs) | Div. flow exiting through Reserve Canal (cfs) |
|---|---|---|---|
| 250 | 210 (84%) | 0% | 29 (12%) |
| 1000 | 462 (46%) | 176 (18%) | 251 (25%) |
| 2000 | 648 (32%) | 570 (29%)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




# 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.

# Section 1: Summary

- The normal water level is about 1 ft and 3 ft, NAVD88 for the Year 0 and Year 50 FWOP conditions, respectively.

- The highest water levels are in Hope Canal as it exits I-10 bridge.
  - YR 0: Diversion flow of 250-, 1000- and 2000 cfs raises water level by 0.3, 1.3 and 1.9 ft, respectively
  - YR 50: Diversion flow of 2000 cfs raises water level by 0.6 ft.

- The average water levels in the swamp
  - YR 0: Diversion flow of 250-, 1000- and 2000 cfs raises water level by 0.1, 0.7 and 0.9 ft, respectively
  - YR 50: Diversion flow of 2000 cfs raises water level by 0.2 ft.

- Water levels near the WSLP drainage structures
  - YR 0: Diversion flow of 2000 cfs raises water by less than 0.3 ft.
  - YR 50: Diversion flow of 2000 cfs raises water level by 0.1 ft.

- For the YR 0 scenarios 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 (1000 cfs diversion)
  - 26% flows towards the Reserve Canal and 29% towards the Blind River (2000 cfs diversion)

- For the YR 50 scenario, due to significant inundation, only 6% is channelized through Dutch Bayou.

# Section 2

## Preliminary Evaluation of Polder Drainage Model Results – Water Surface Elevations

2-yr and 25-yr (24 hr) Rainfall Events

# Introduction

## Objective

- Evaluate drainage of the area between LA-641, I-10, Diversion Canal and Airline Highway (Central swamp) after 2- and 25-yr rainfall events under TY0 conditions

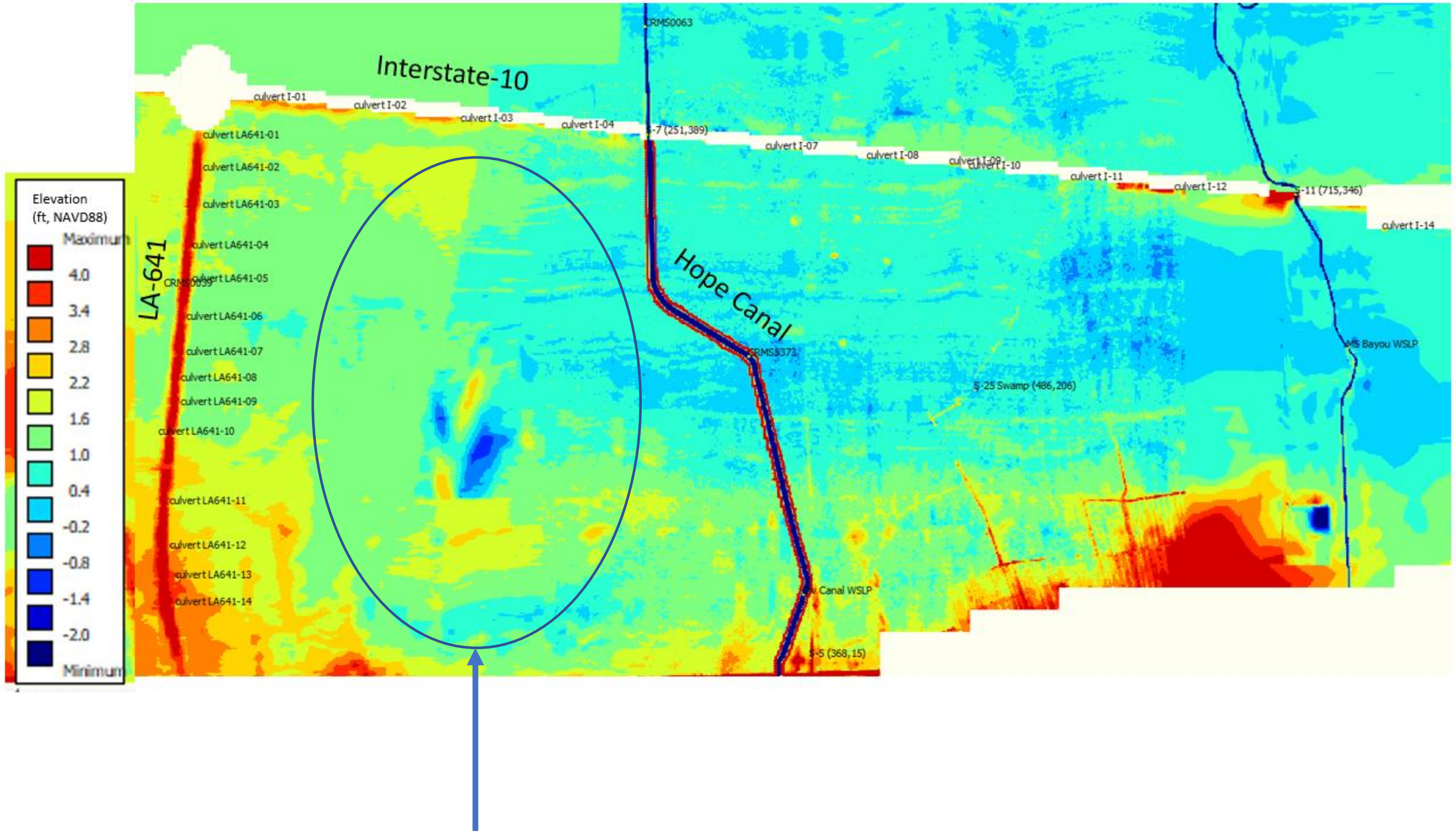
## Assumptions and Limitations

- The Delft3d model 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 has not been 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.

# Methodology

- 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.
- 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.

# Delft3D Model Bathymetry (based on LIDAR Data)




Primary polder of interest
Bounded by LA-641 and the Proposed Diversion Canal

- Existing general drainage is likely from west to east.
- Model "sees" the overall lay of the land but cannot capture narrow pathways leading to the culvert helping drain the swamp due to grid resolution limits
- Locations of highway culverts are shown with labels

# Delft3D Model Runs Plan for Polder Drainage Evaluation

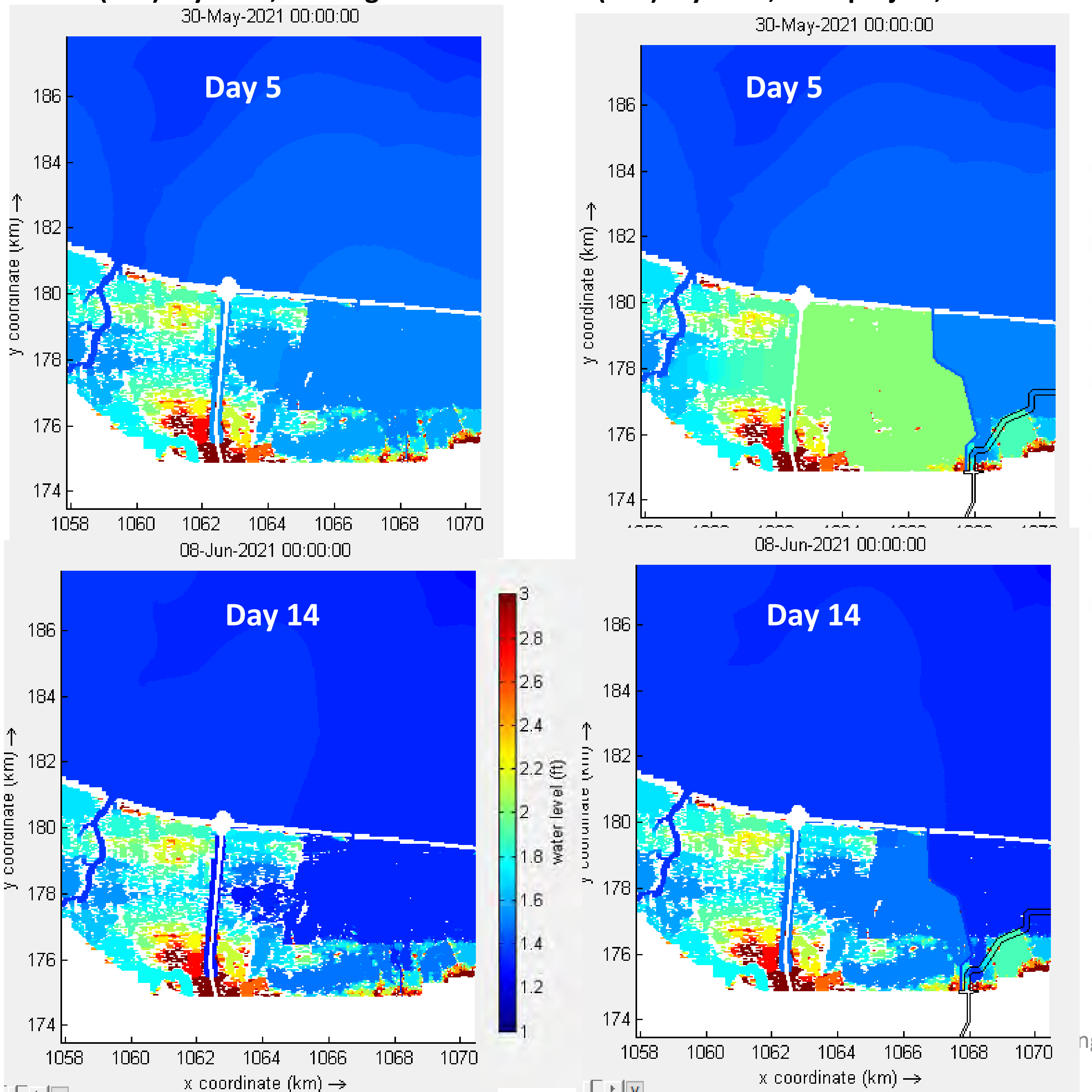
| Run ID | Diversion Flow (cfs) | Diversion Channel | Lateral Release Valves | Rainfall Frequency | Tidal Boundary at Lake Maurepas | Comments |
|---|---|---|---|---|---|---|
| 20a | 50 | Existing Hope Canal | N/A | 2-year | Rain-elevated then normal | FWOP No Div. 2-yr |
| 20b | 50 | Existing | N/A | 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 |

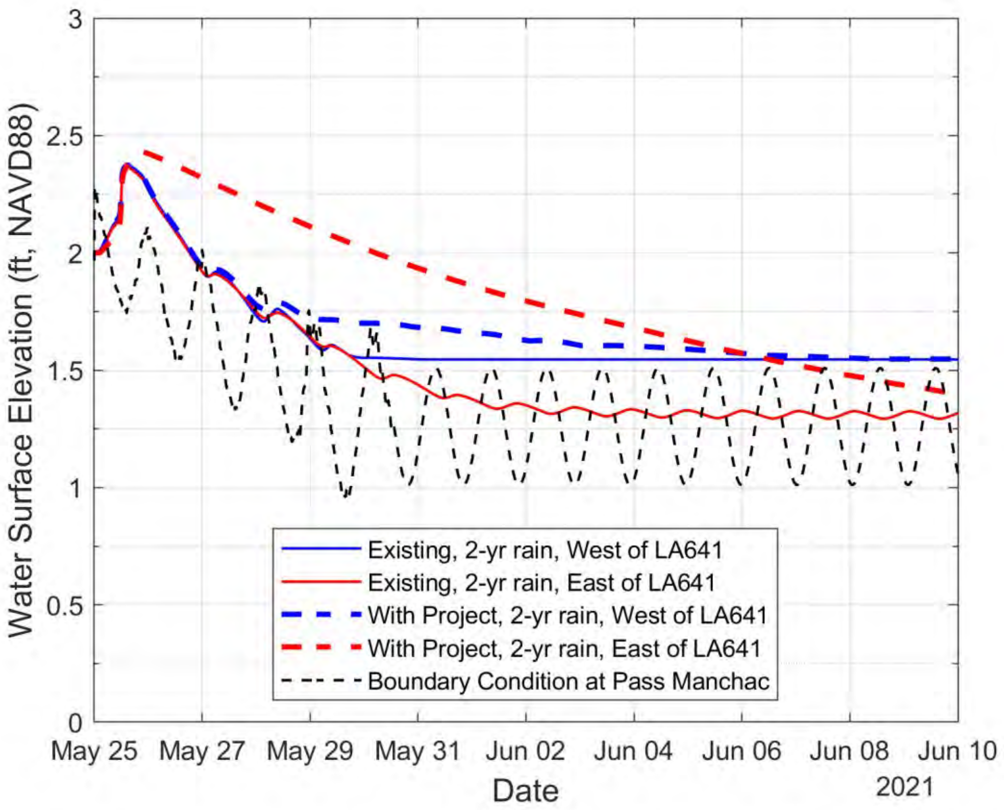
*All scenarios were run for the same 16-day period.*
*All scenarios included the proposed West Shore Lake Pontchartrain (WSLP) levee.*

# Delft3D Model Water Surface Elevation Results

- Model output contains water surface elevations and velocities at every model node for the 16-day simulation period.
- The total rainfall was applied on the first day using SCS distribution.
- Spatial contours of water surface elevations are presented at the end of 5- and 14 days.
- Time series charts of water surface elevations are shown 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.

**(20a) 2-yr rain, existing** **(21a) 2-yr rain, with-project, no div. flow**

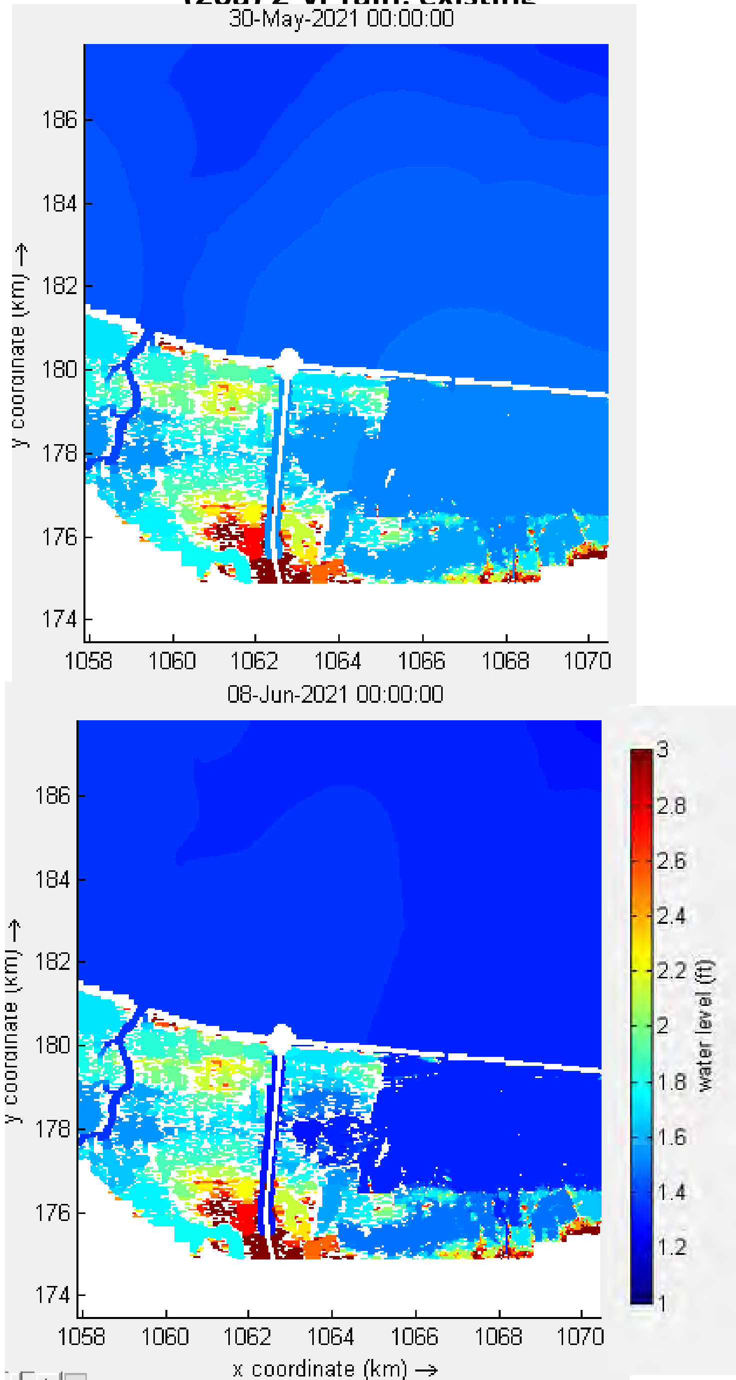


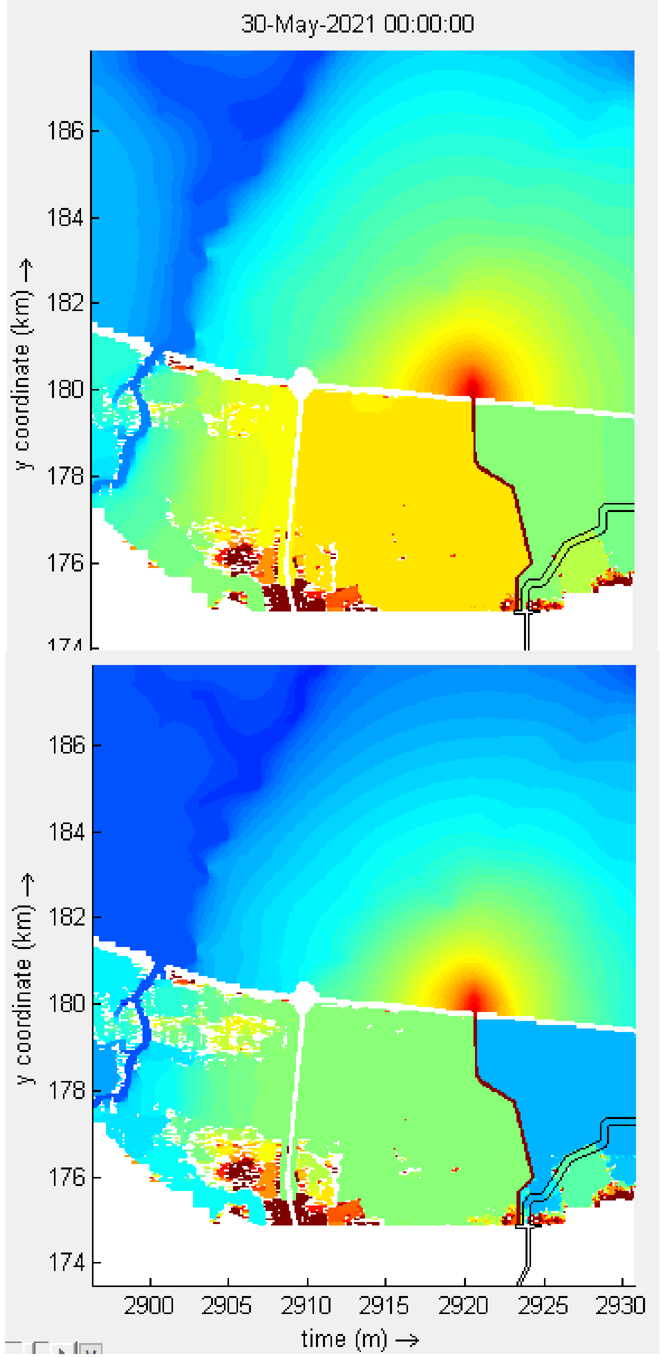

- Results are qualitatively similar to the 25-yr rainfall case

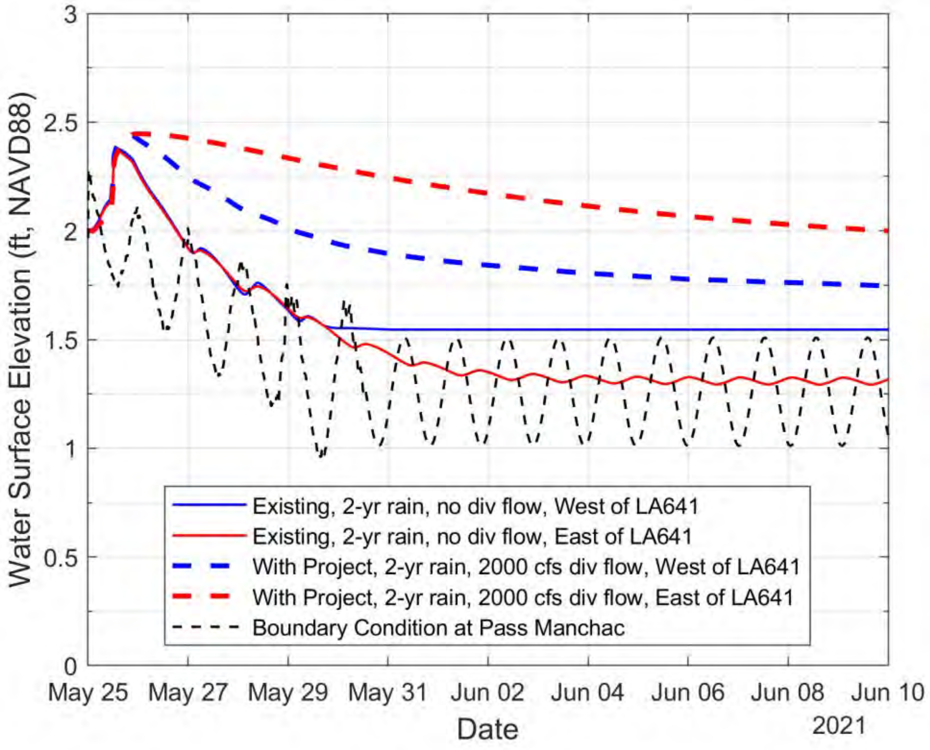
nge.

**(20a) 2-yr rain, existing**



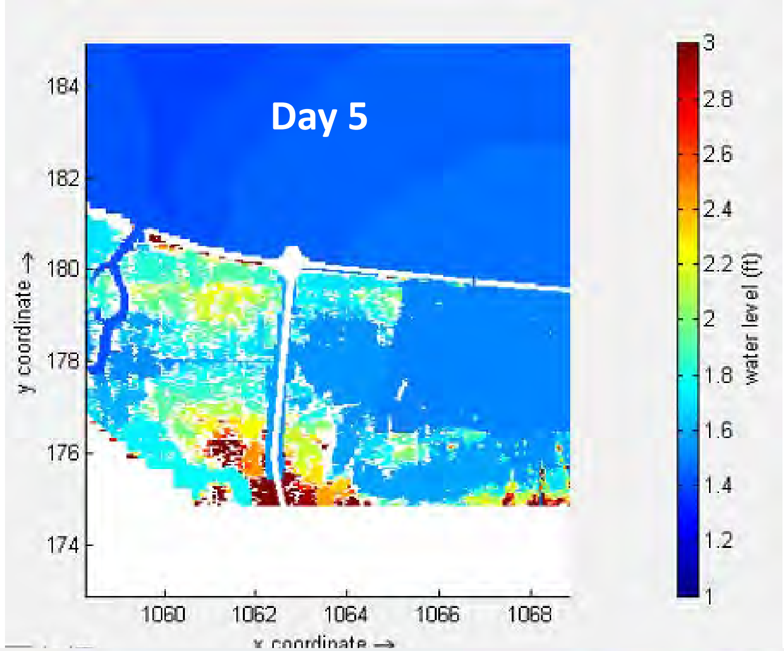

**(22a) 2-yr rain, with-project, div flow 2000 cfs**

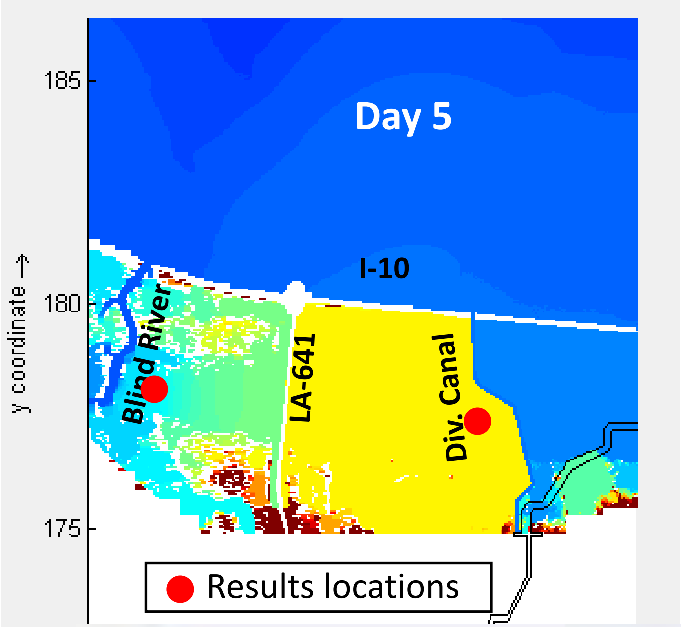


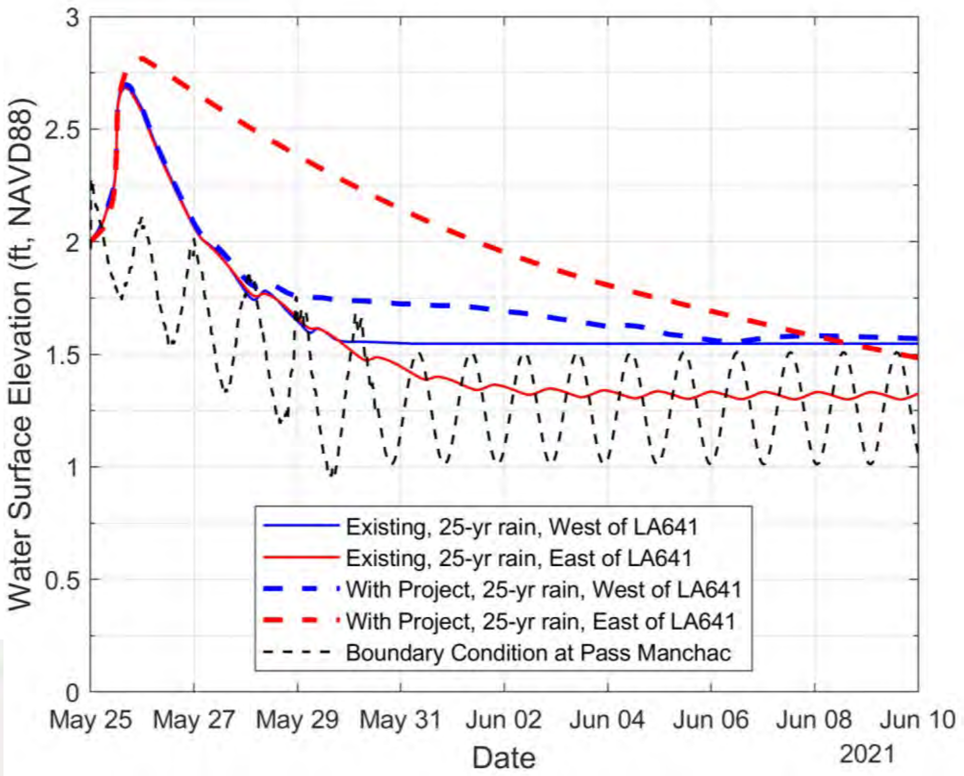



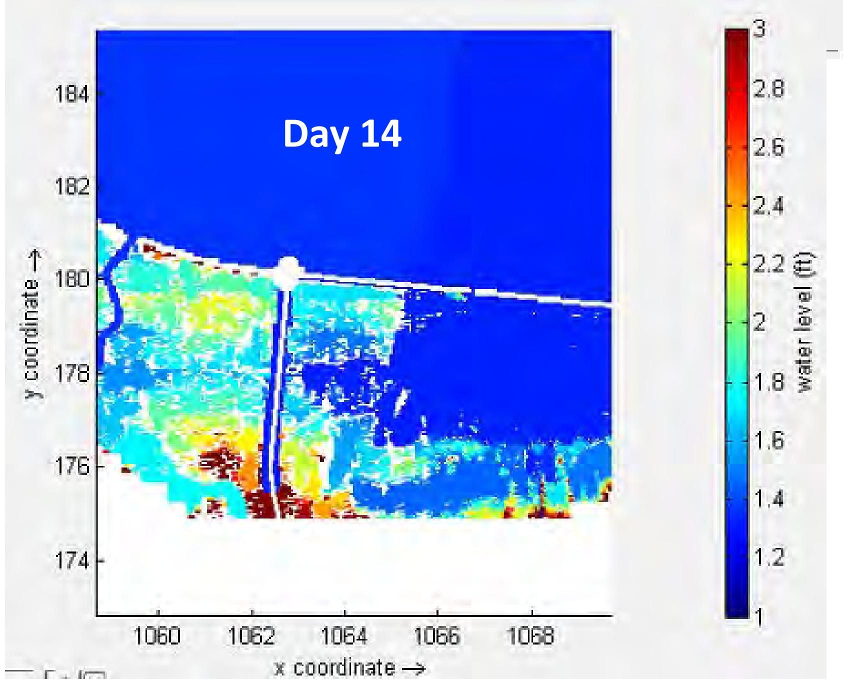
(20b) 25-yr rain, existing

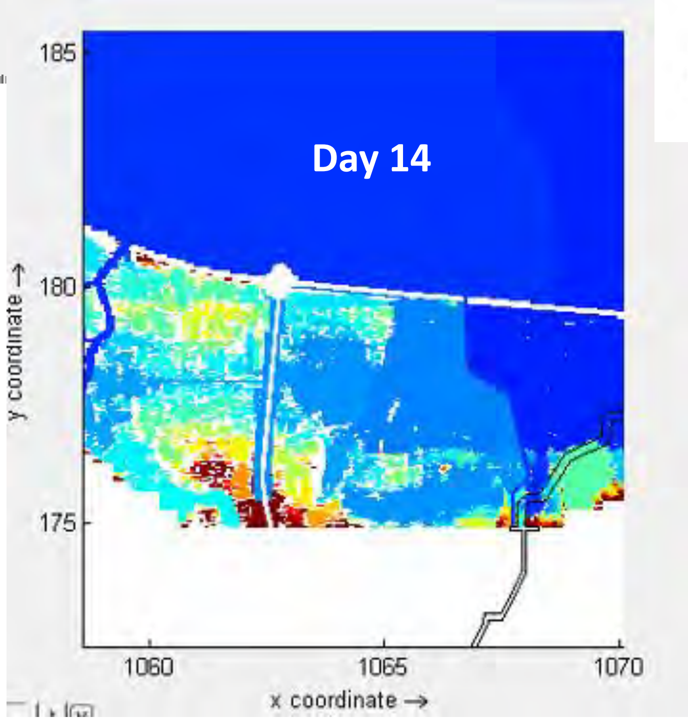
(21b) 25-yr rain, with-project, no div. flow



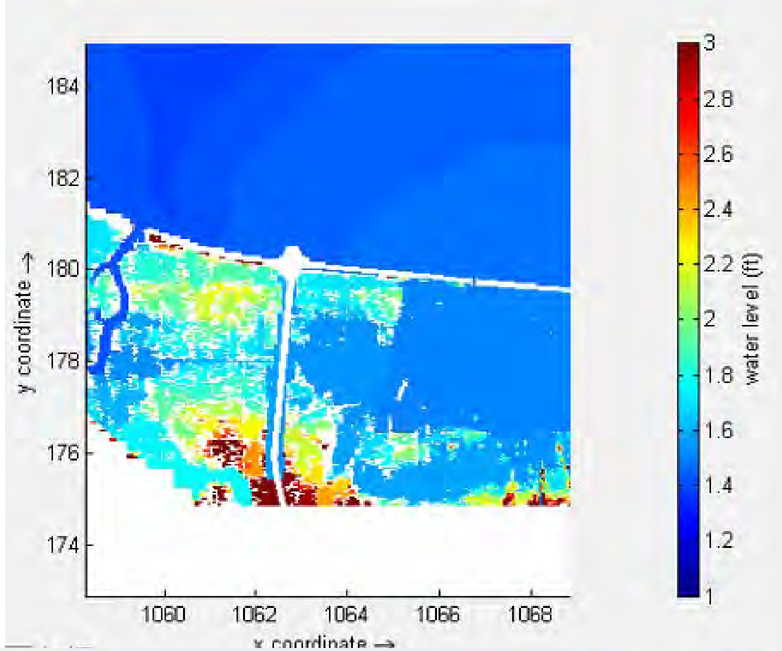



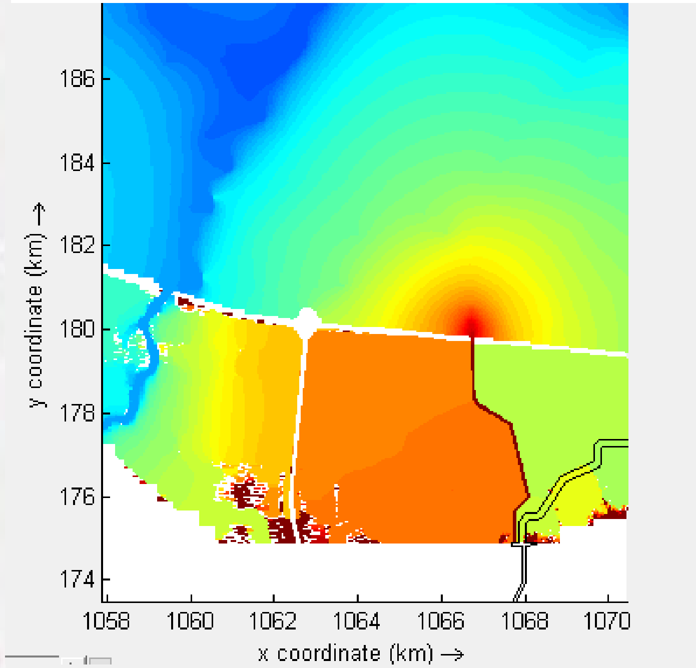



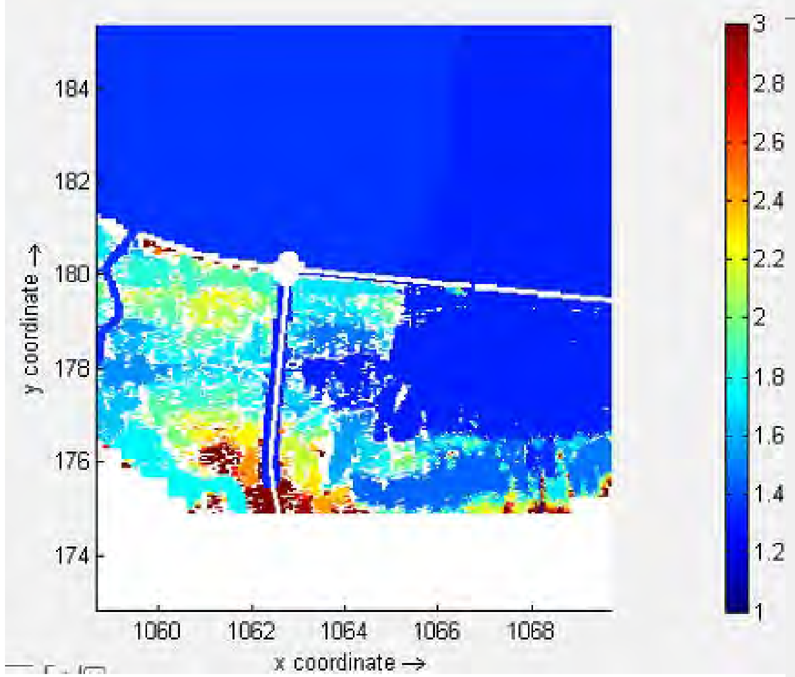

- Construction of the diversion canal isolates region to its west reducing drainage potential of the region.
- The impact is greater on the area east of LA-641 than the west area
- The location west of LA641 is showing flat water level possibly because of local cut off. In reality, this region will continue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

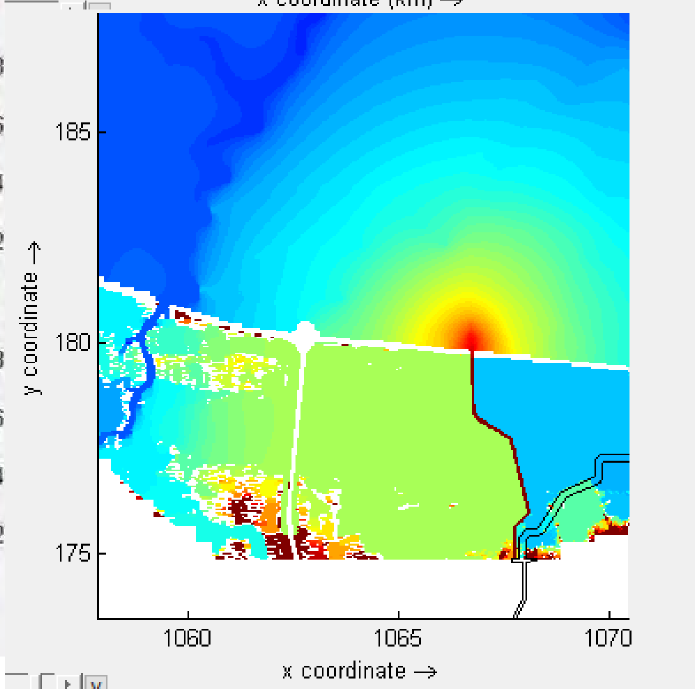
(20b) 25-yr rain, existing

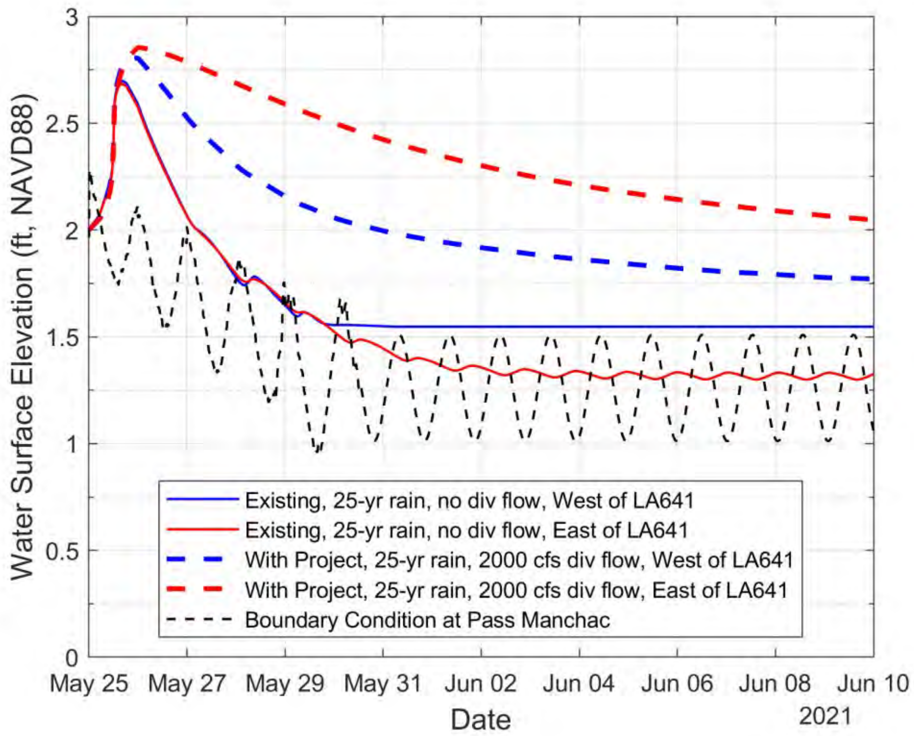
(22b) 25-yr rain, with-project, div flow 2000 cfs



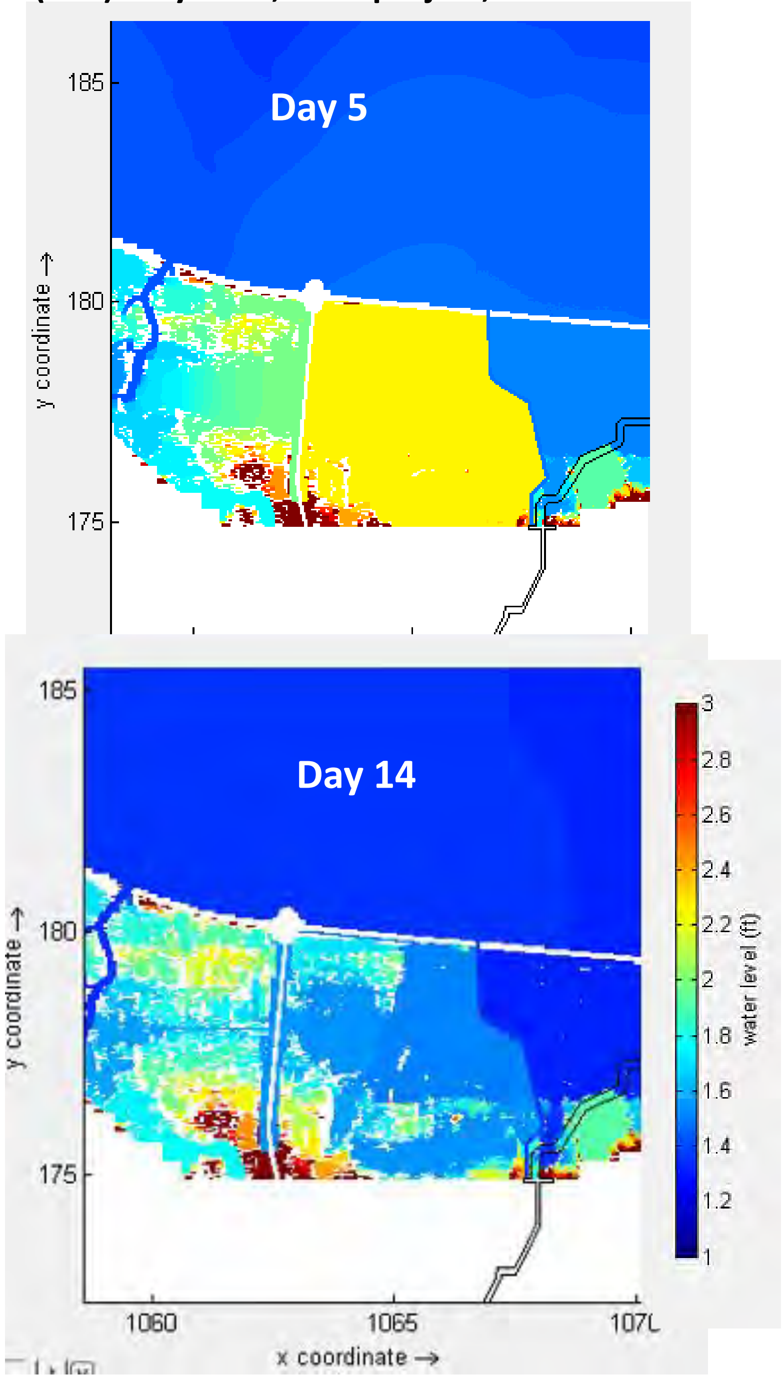



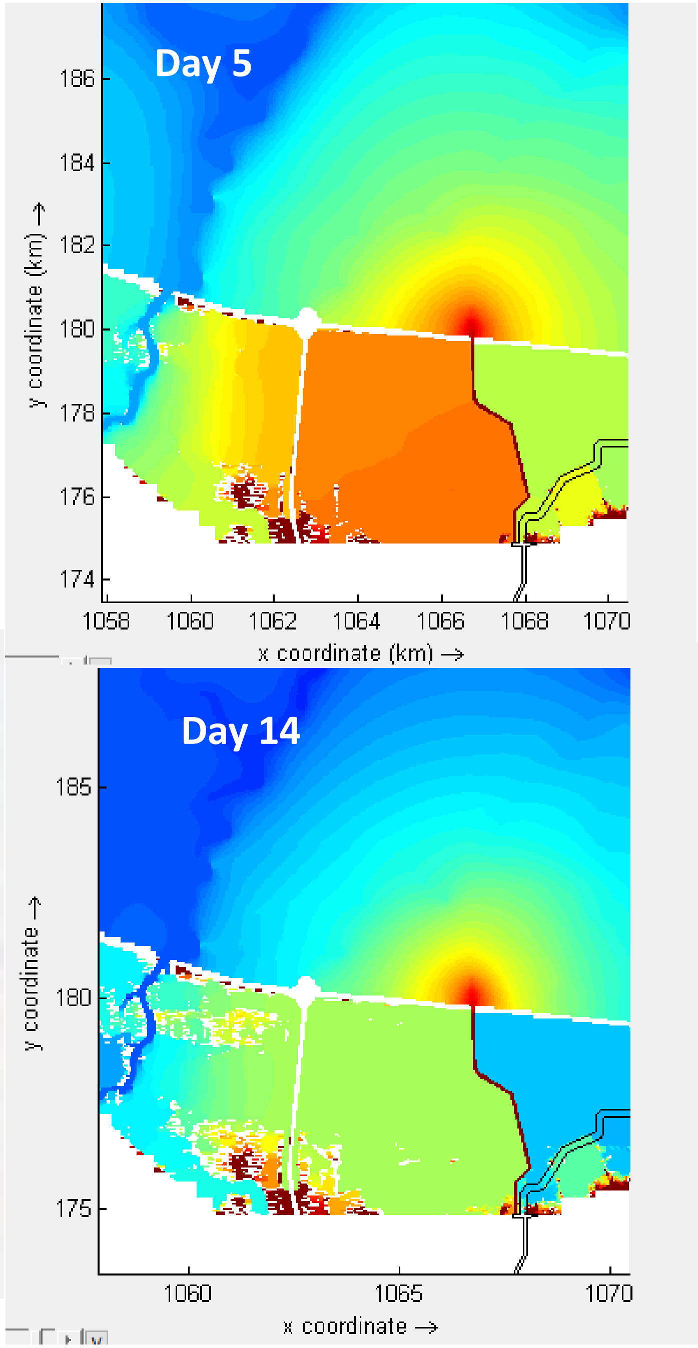



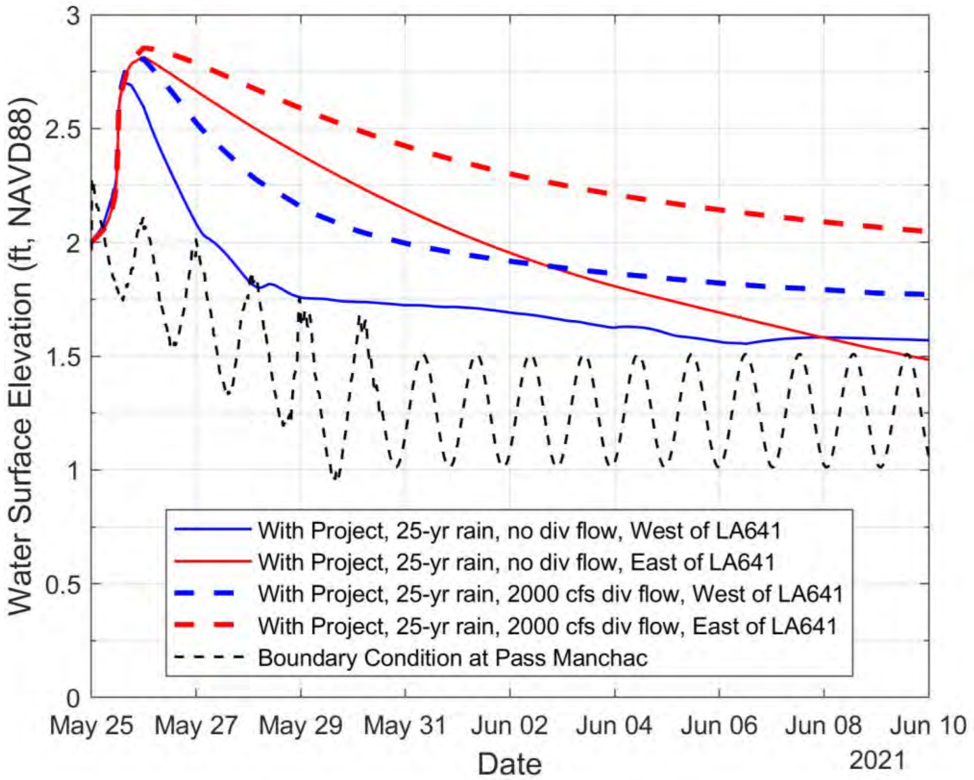



(21b) 25-yr rain, with-project, no div. flow



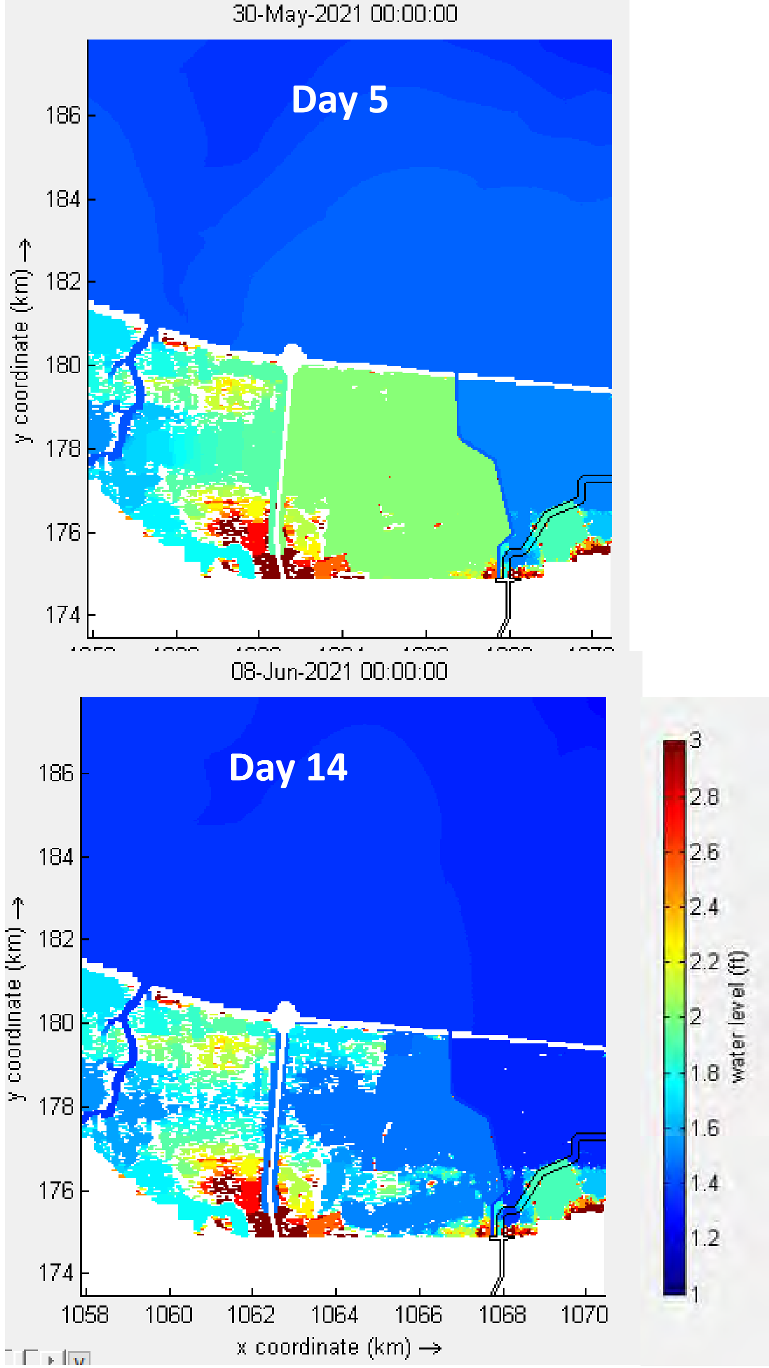

(22b) 25-yr rain, with-project, div. 2000 cfs



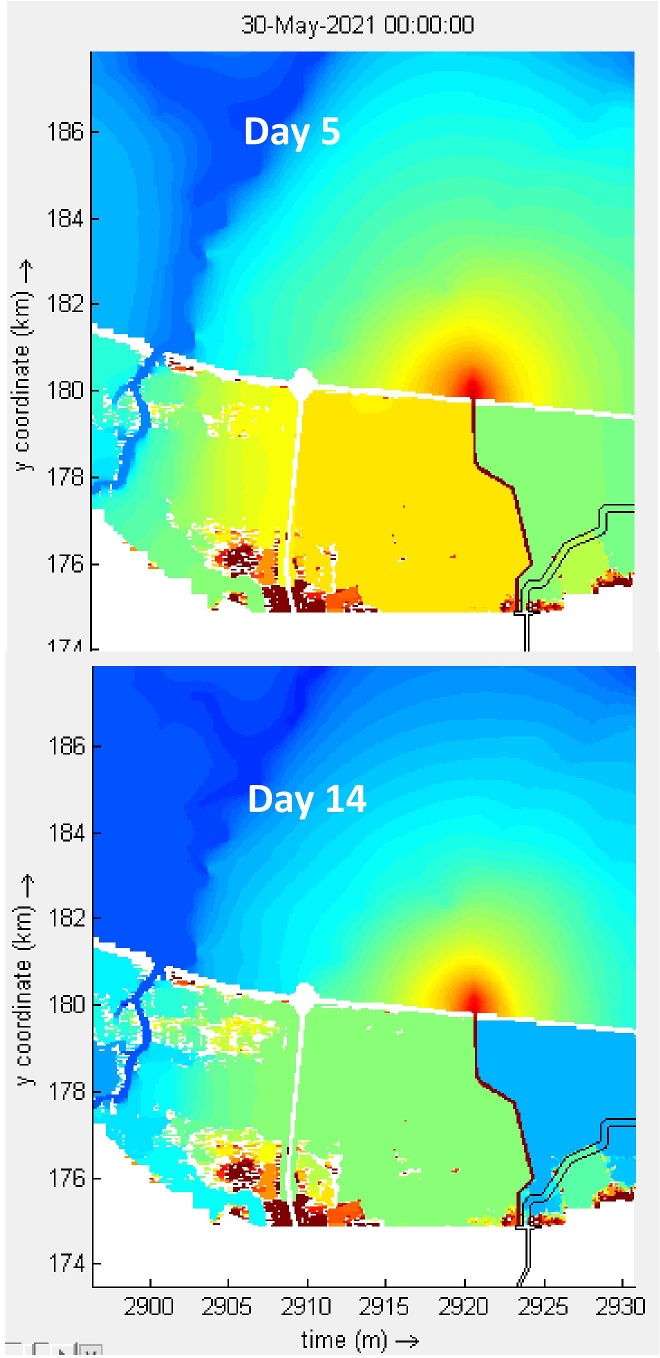



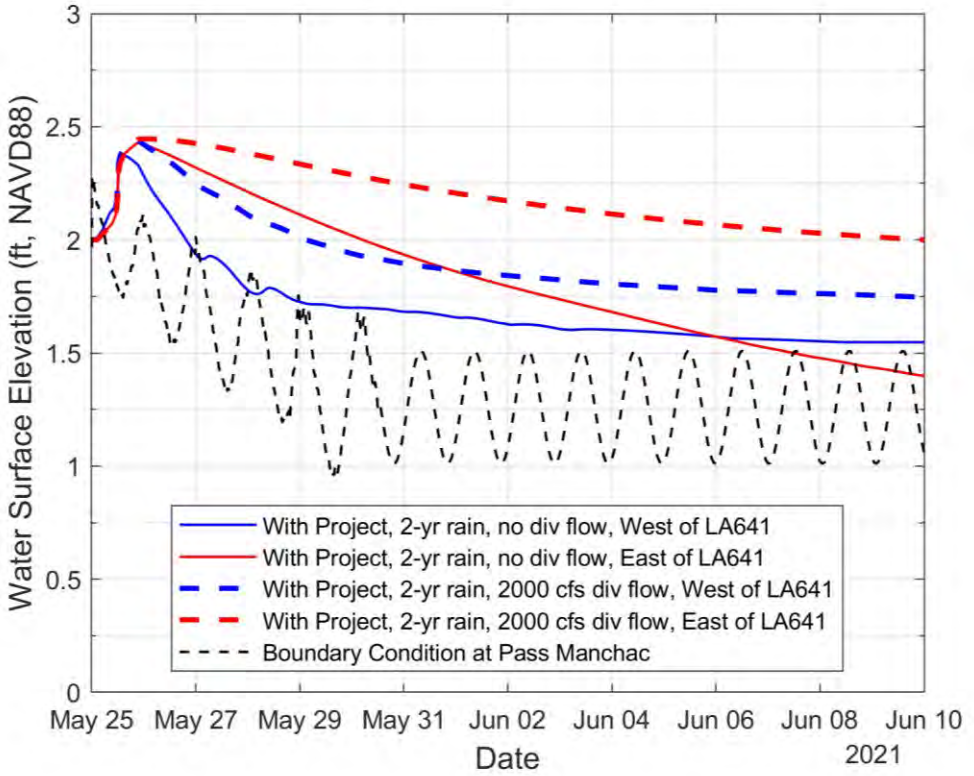

- Presence of elevated water levels north of I-10, reduces capacity of the highway culverts to drain the polders

(21a) 2-yr rain, with-project, no. div flow



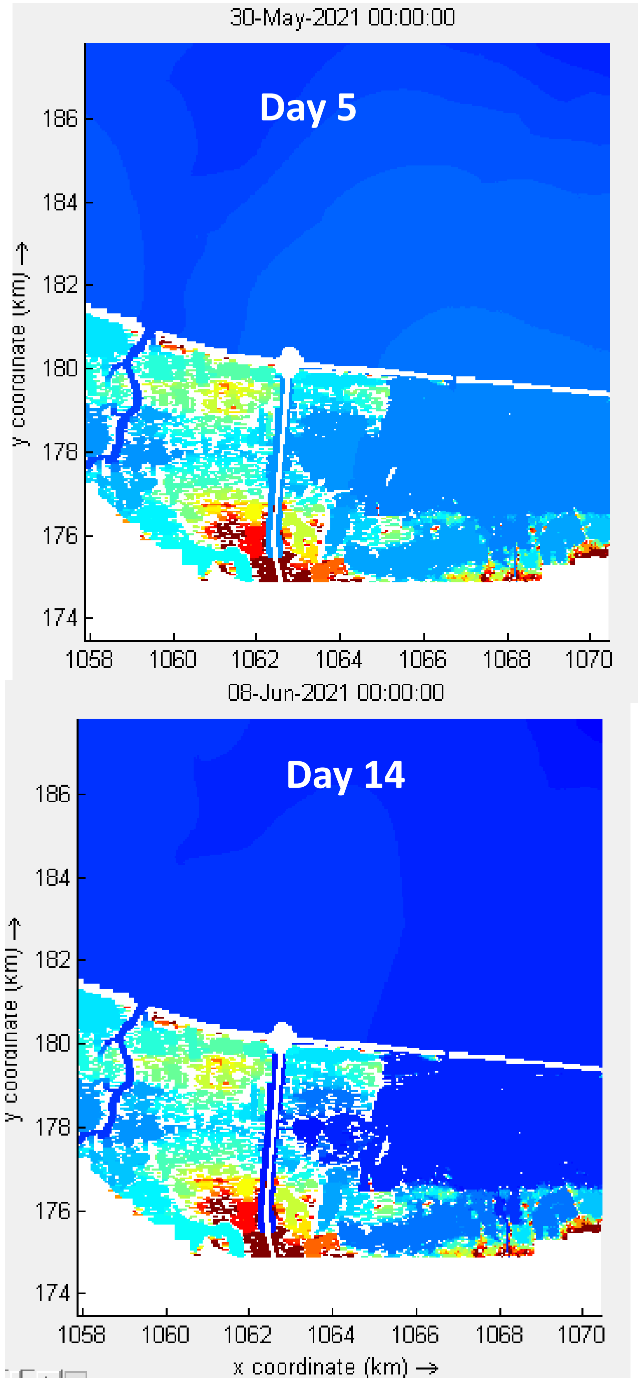

(22a) 2-yr rain, with-project, div. flow 2000 cfs



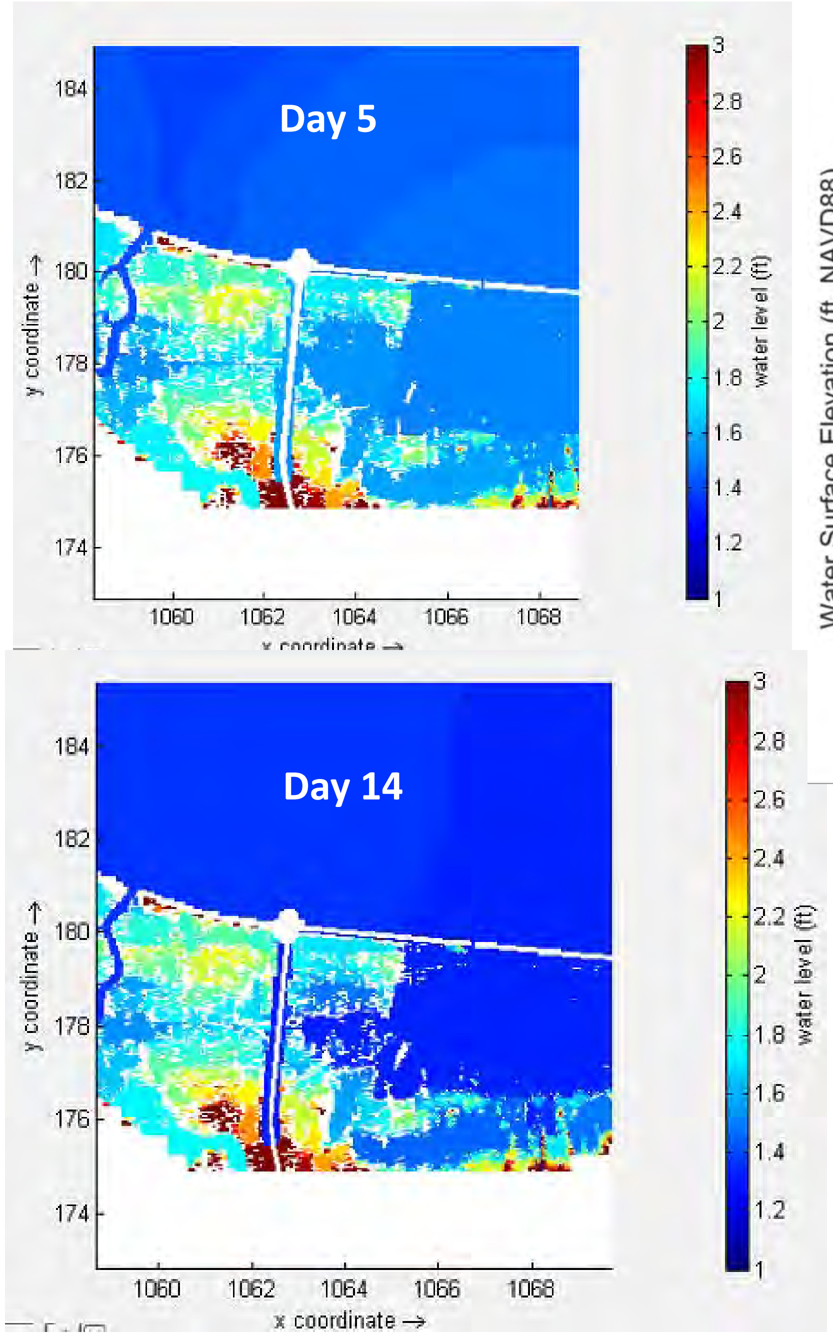



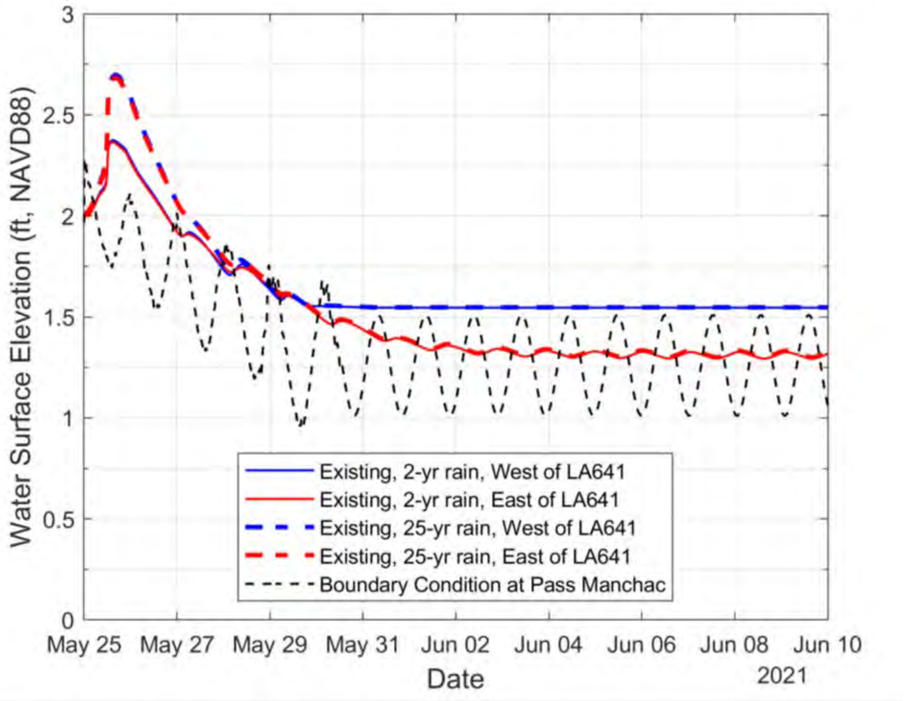

- Results are qualitatively similar to the 25-yr rainfall case

**(20a) 2-yr rain, existing**



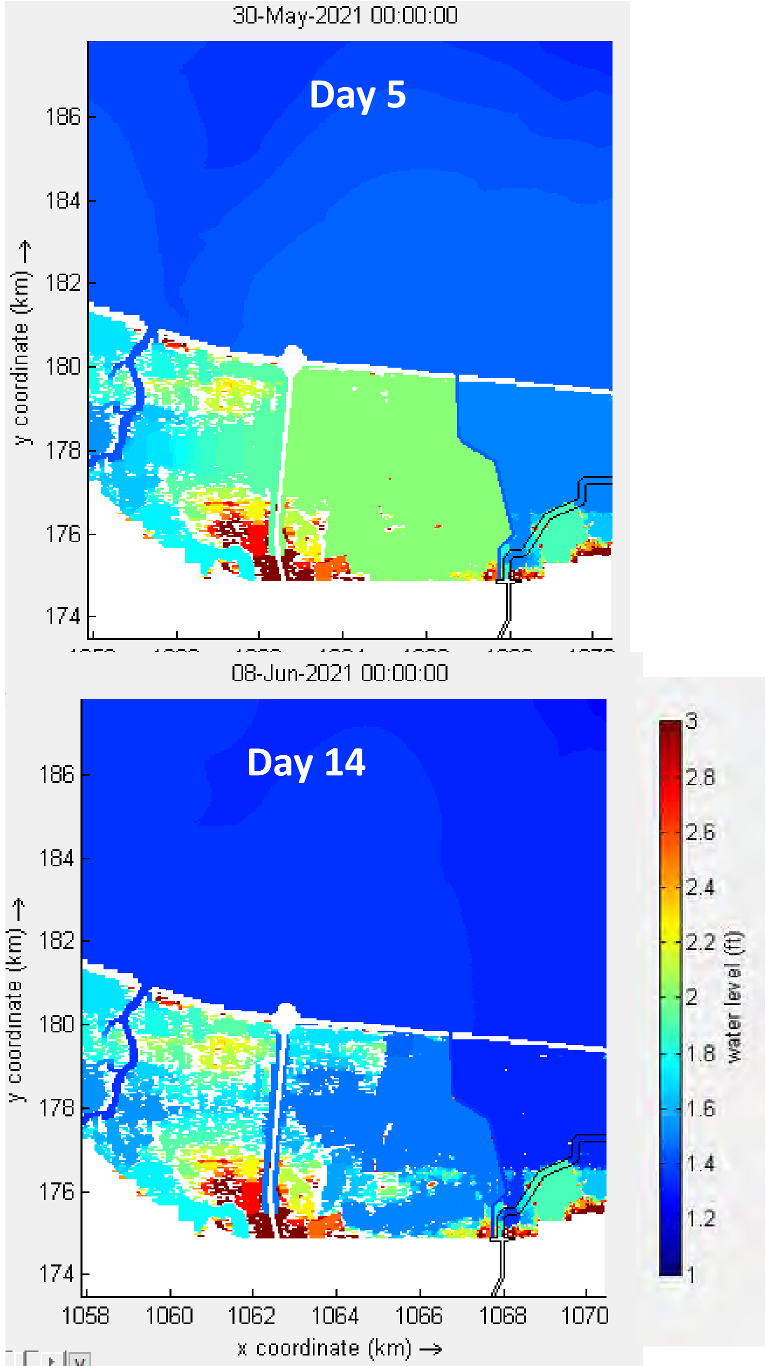

**(20b) 25-yr rain, existing**



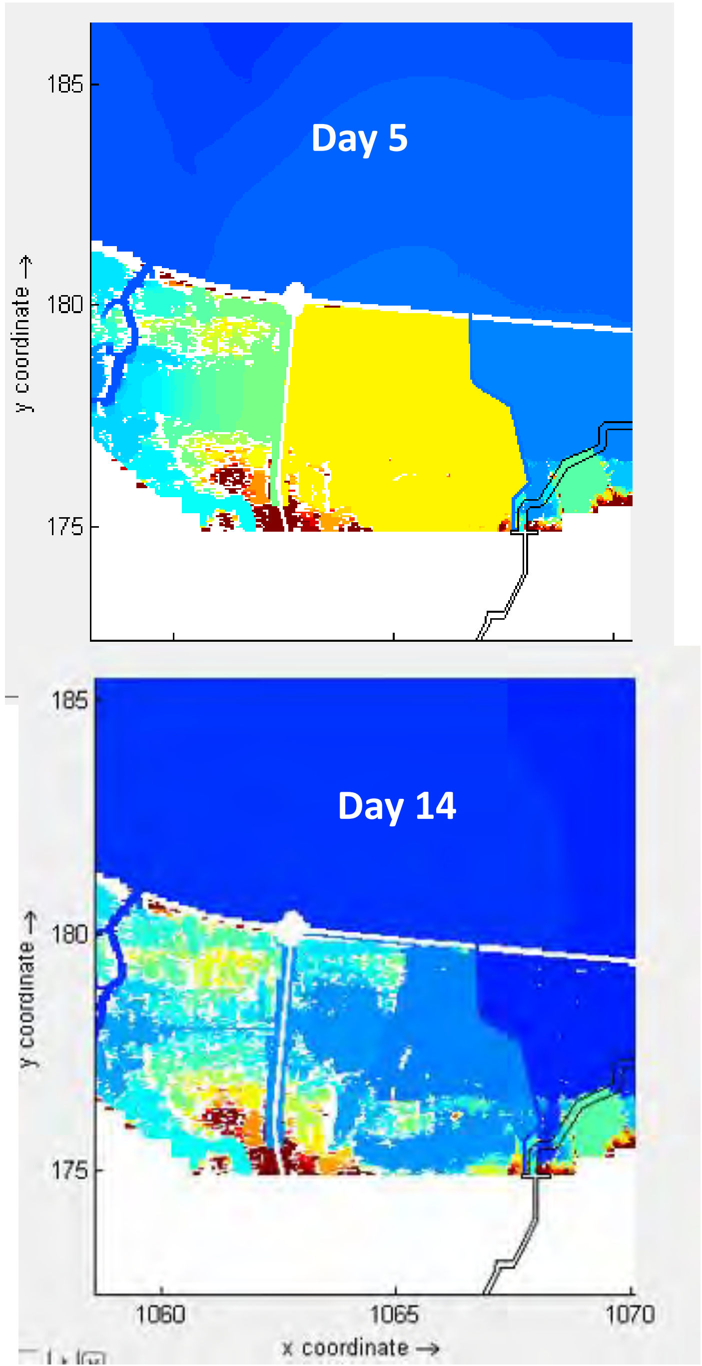



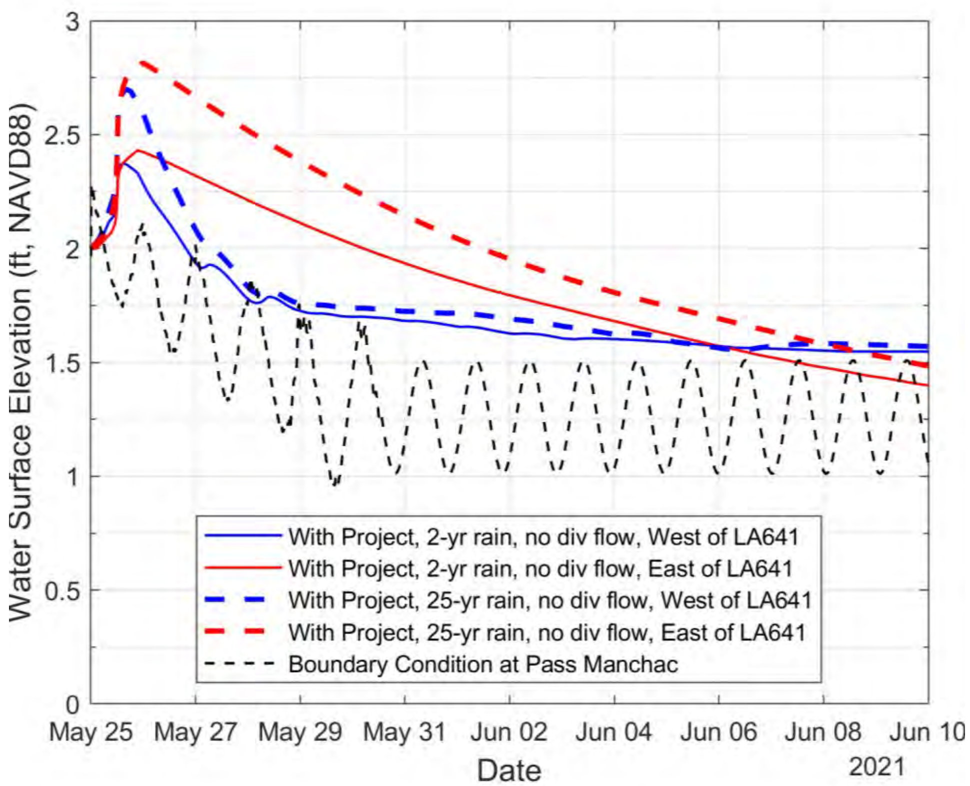

- Under the existing conditions, the difference in water levels due to the 2- and the 25-yr rainfall is apparent for about 4 days

**(21a) 2-yr rain, with-project, no div. flow**



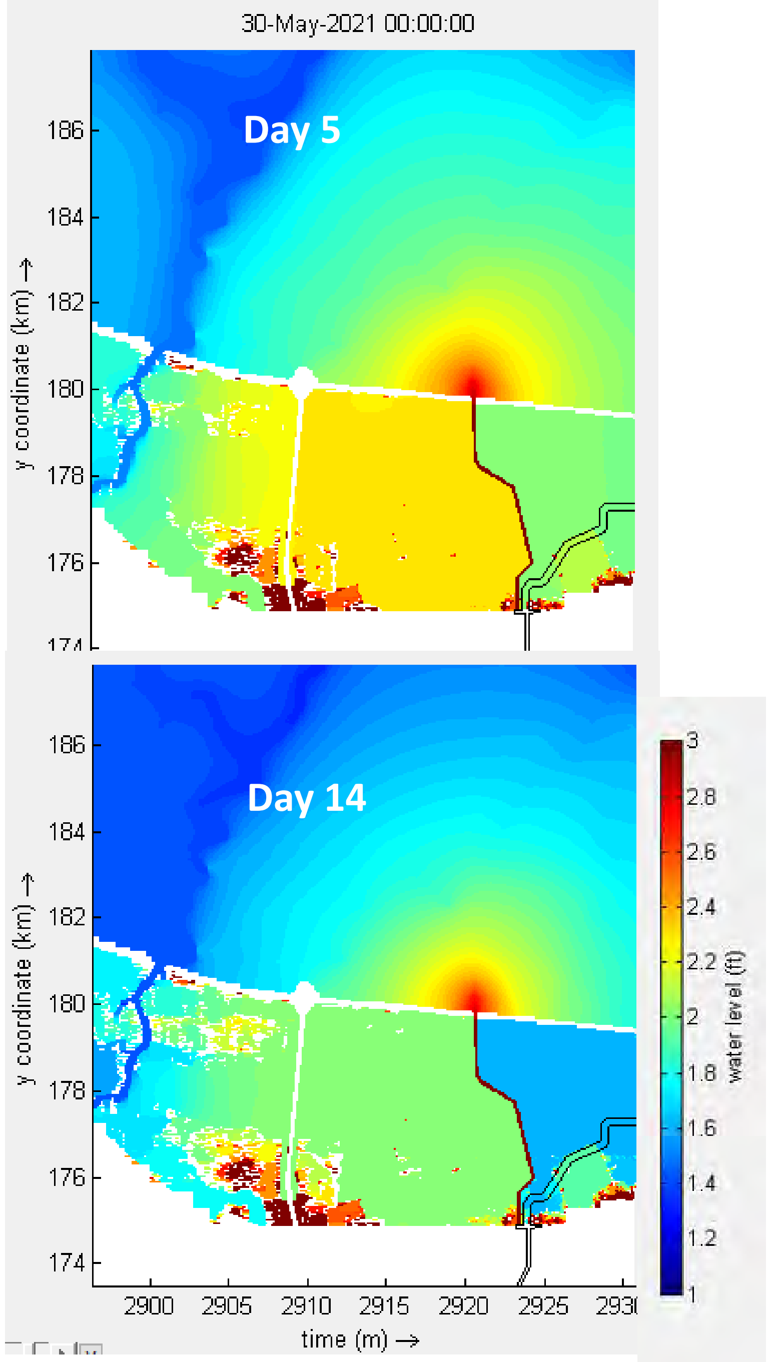

**(21b) 25-yr rain, with-project, no div. flow**



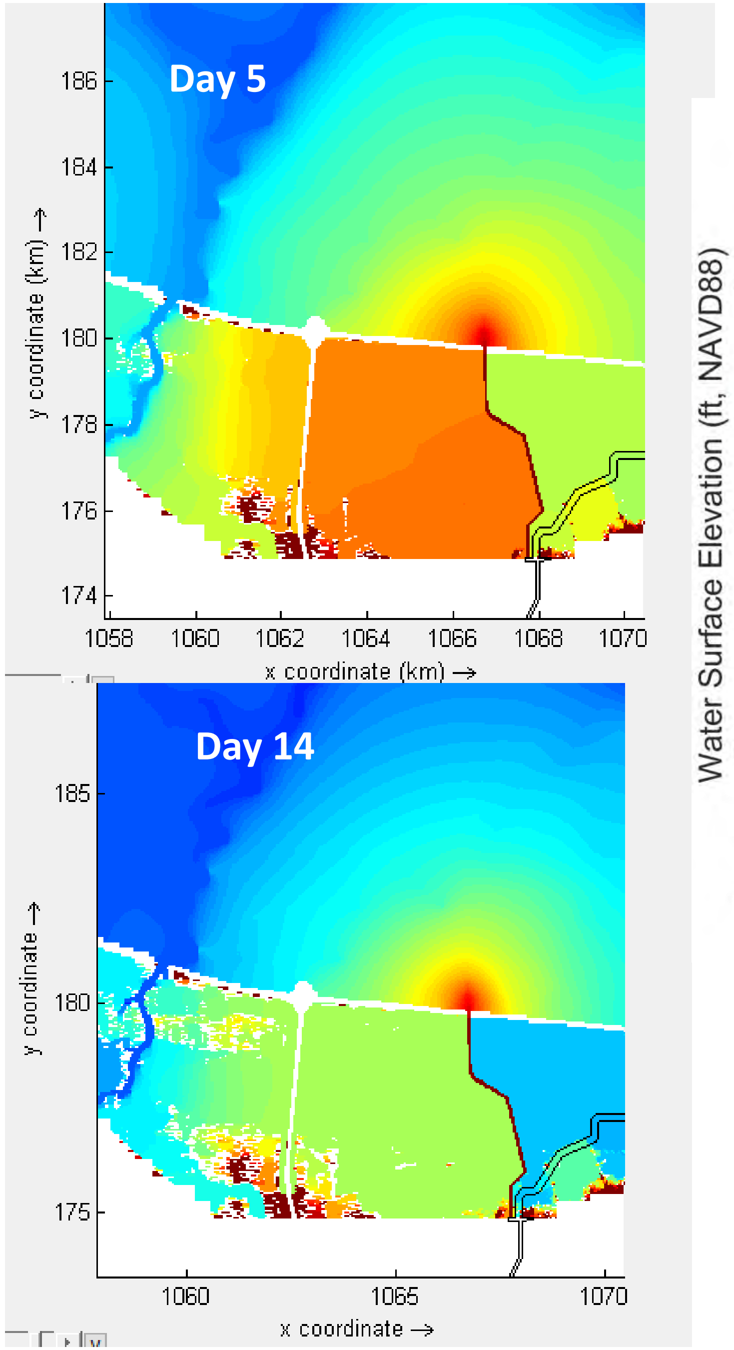




- Under the with-project conditions, the difference in water levels due to the 2- and the 25-yr rainfall is apparent for over 3 weeks

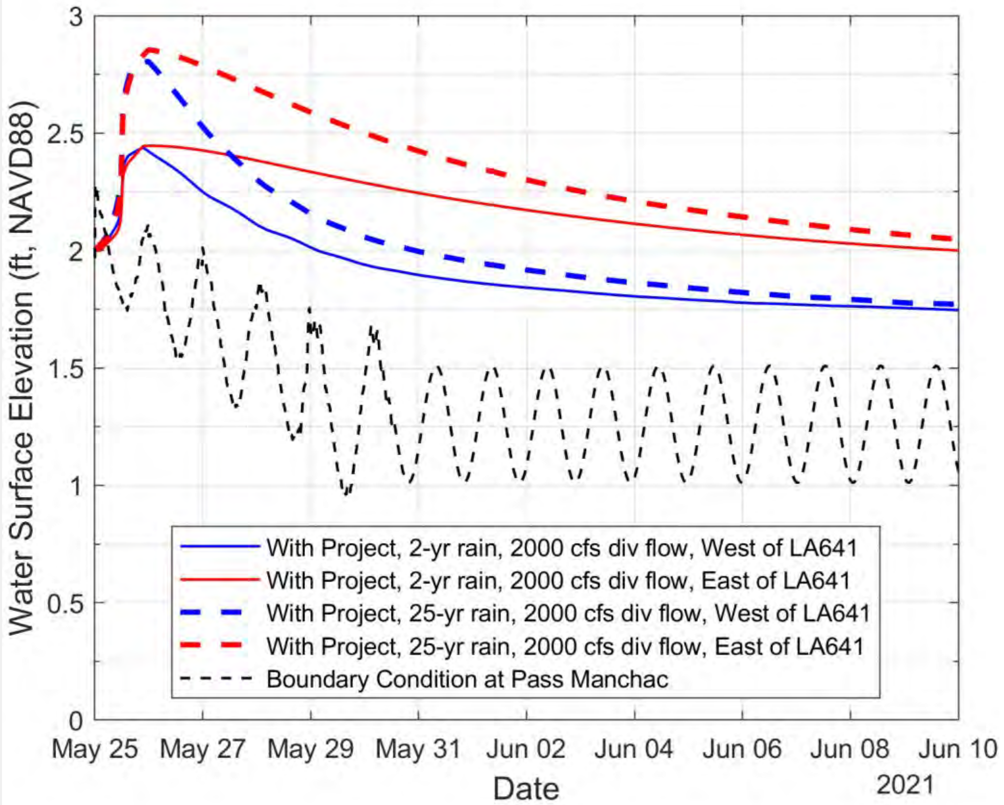
(22a) 2-yr rain, with-project, div flow 2000 cfs



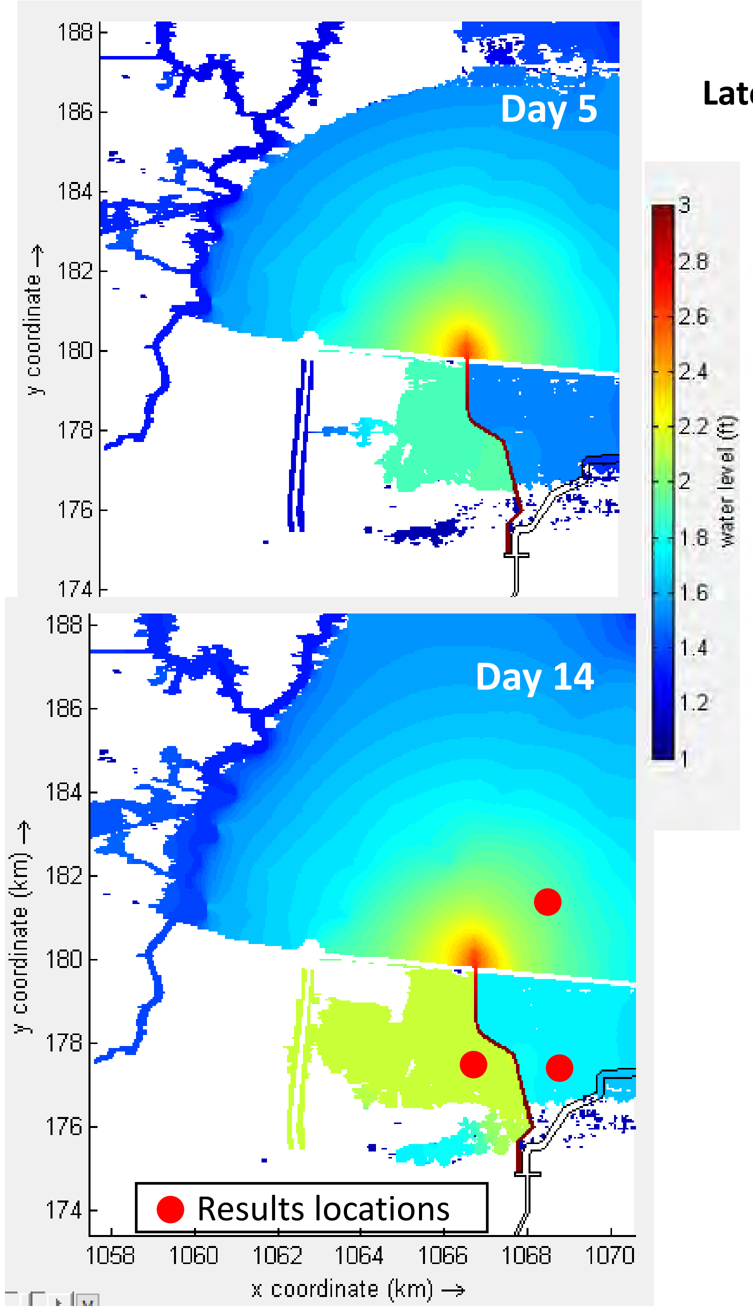

25-yr rain, with-project, div flow 2000 cfs

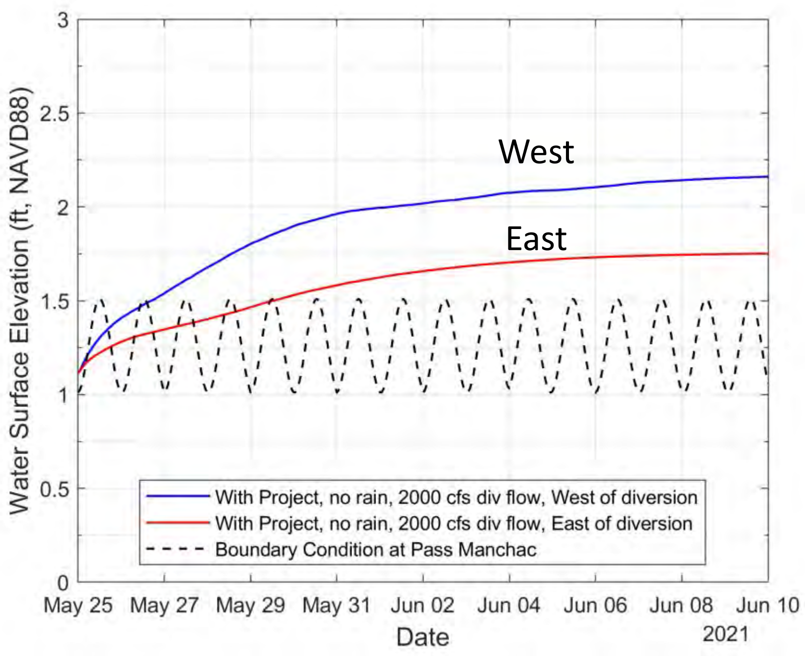


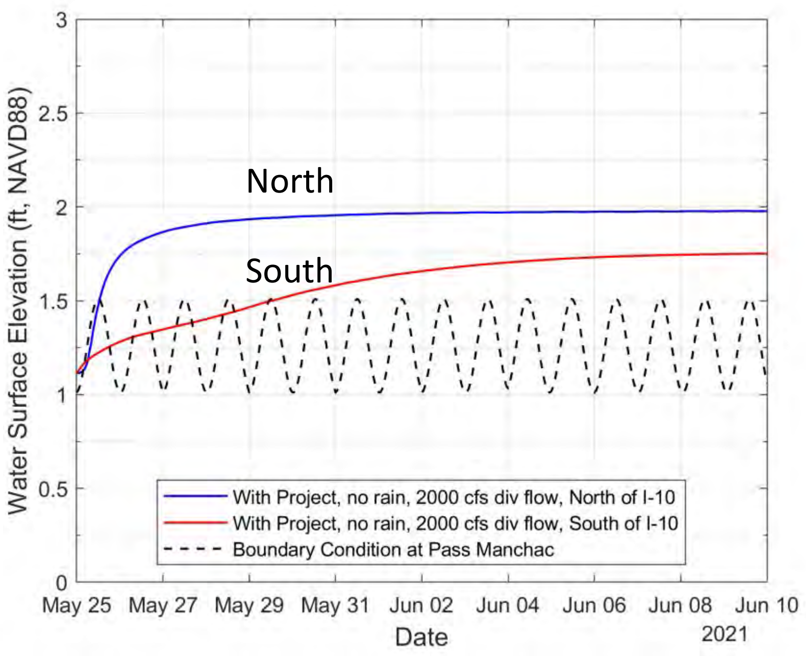

22b



**(23) No rain, with-project, div flow 2000 cfs**
**Lateral Release Flow 140 cfs on each side for the first 7 days then closed**

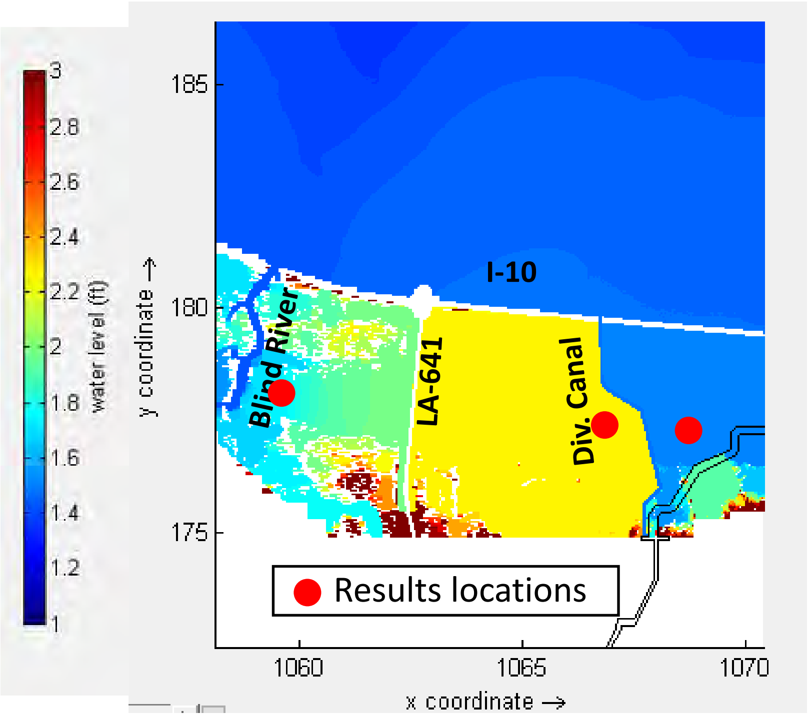


# Table of Predicted Water Surface Elevations

| 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 |



# Section 2: Findings

- 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.

- 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.

# Section 3

## Lateral Release Valves (LRVs) Runs 24, 25 & 26

Objective:

Evaluate the alternative of using the Lateral Release Valves (LRVs) to drain the area between Interstate-10 and Airline Highway (Central Swamp) following a 2-year rainfall event under Future-With-the Maurepas Project (FWP) for TY0 conditions.

The general goal is to provide initial hydraulic analysis of the potential of this option for further consideration.

# Delft3D Model Runs Plan for Polder Drainage, LRV Evaluation

| 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 |

LRV Runs

*Runs 20-23 were previously completed. All scenarios were run for the same 16-day period.*
*All FWP scenarios included the proposed West Shore Lake Pontchartrain (WSLP) levee.*

# LRV Evaluation Method and Configuration

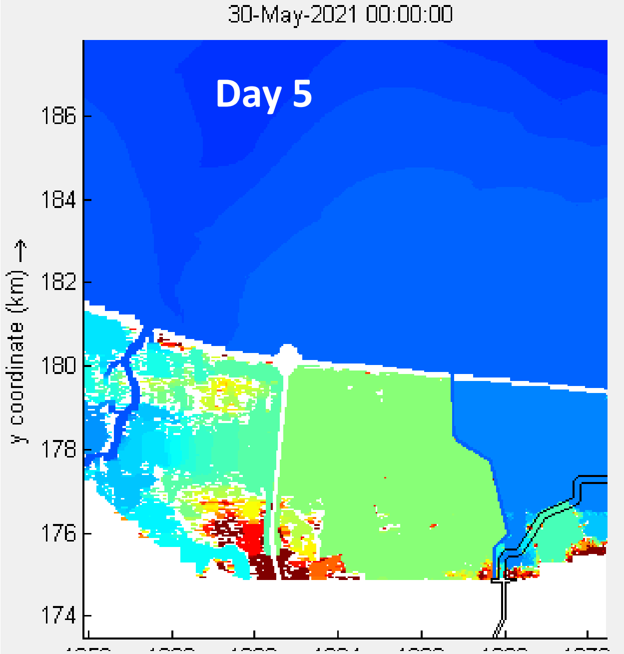
Methodology:

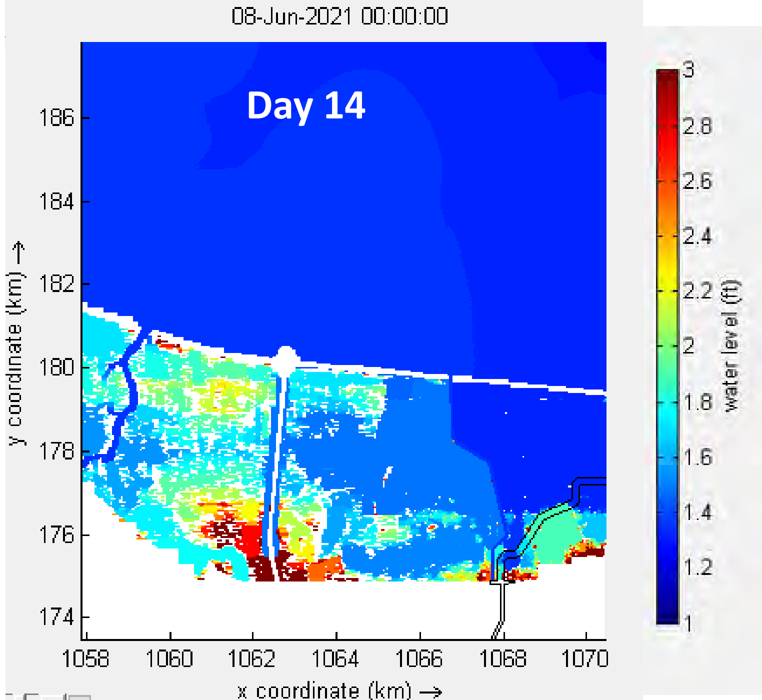
- Make the currently proposed LRVs bi-directional so that they can flow either from the swamp to the diversion channel or from the channel to the swamp depending on the head difference.
- Perform a 2-week simulation where rainfall event occurs on the first day like in the previous simulations

LRV Configurations:

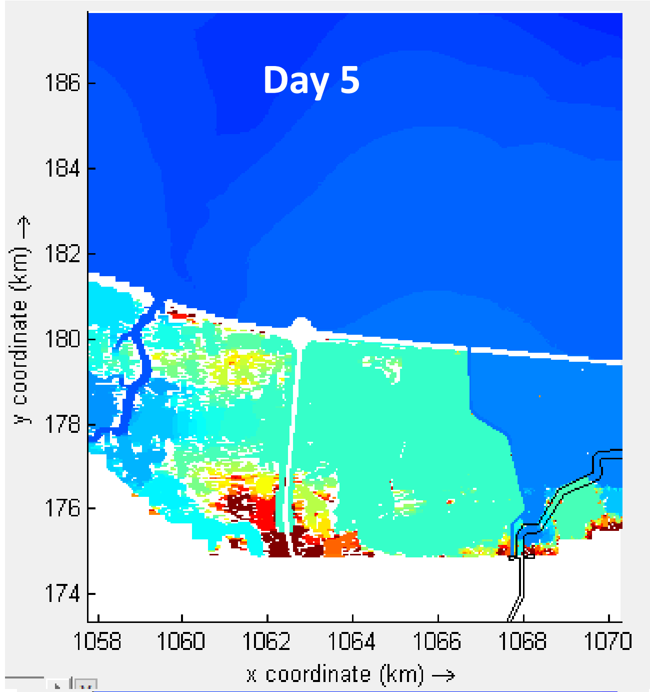
- <u>Configuration 1 (32 LRVs)</u>: Much greater capacity; 16-24” steel pipes on each side of the canal.
- <u>Configuration 2 (8 LRVs)</u>: As in the 95% design report; 4-24” steel pipes of unspecified invert elevation on each side of the diversion canal.
- <u>Configuration 3 (20 LRVs)</u>: 16-24” steel pipes on the west and 4 on the east side of the canal.
- All 3 configurations set pipe invert at 0.0 ft, NAVD88.

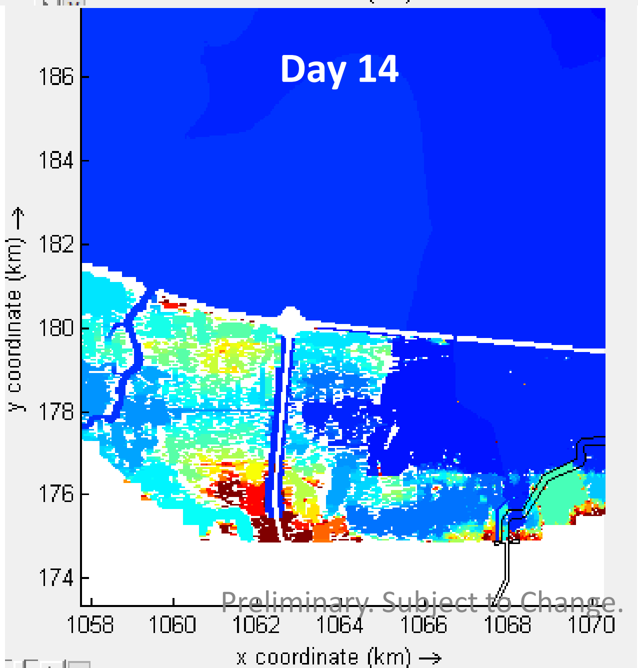
**(21a) 2-yr rain, with-project, no div. flow**

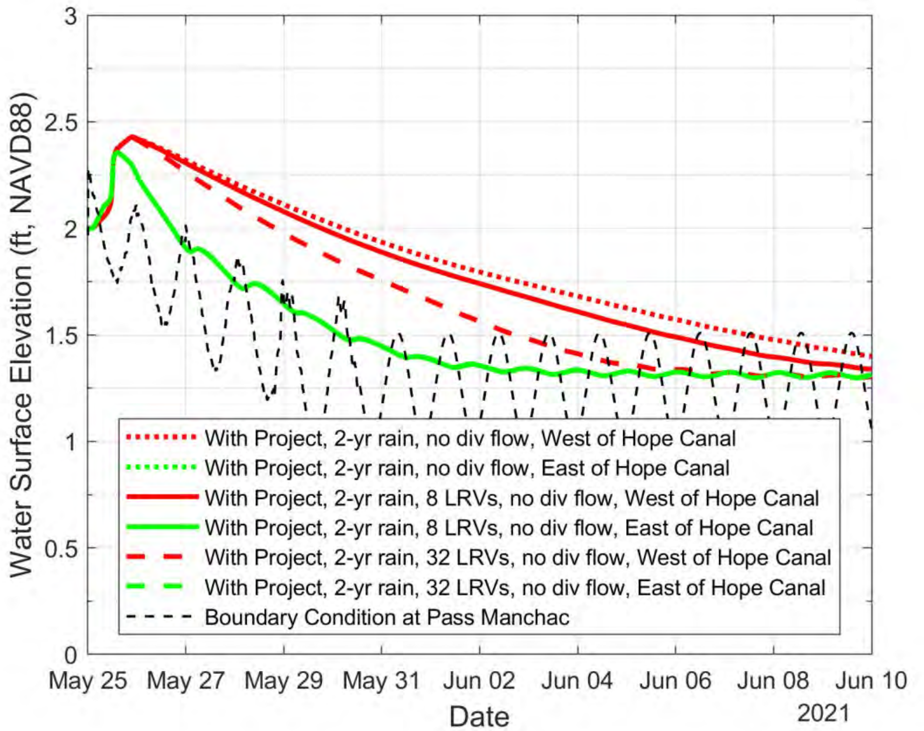



**(24) 2-yr rain, 32 LRVs with-project, no div. flow**

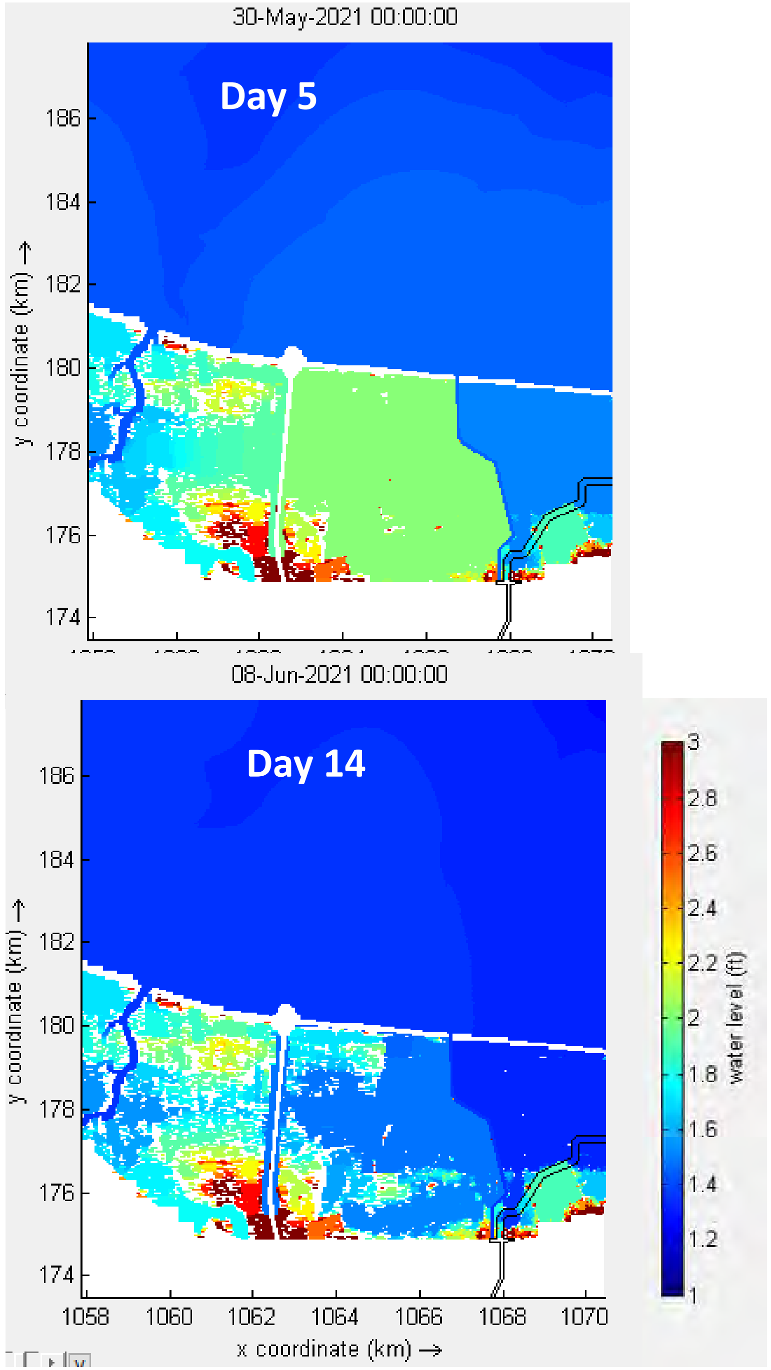


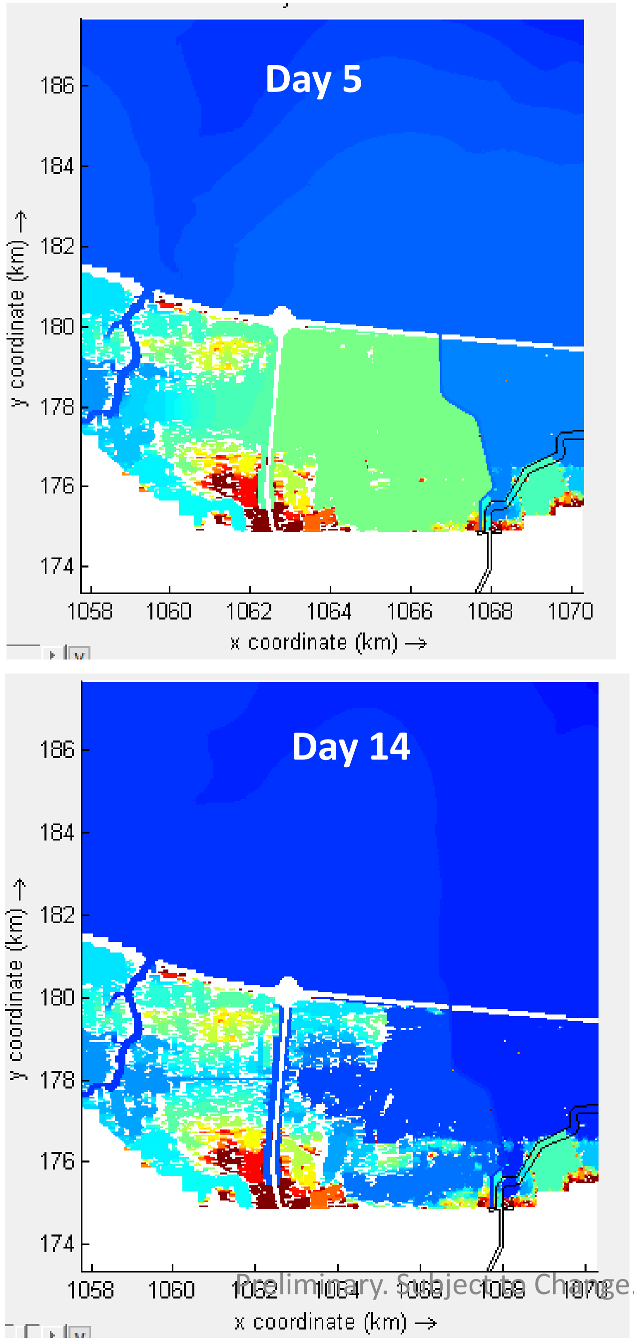



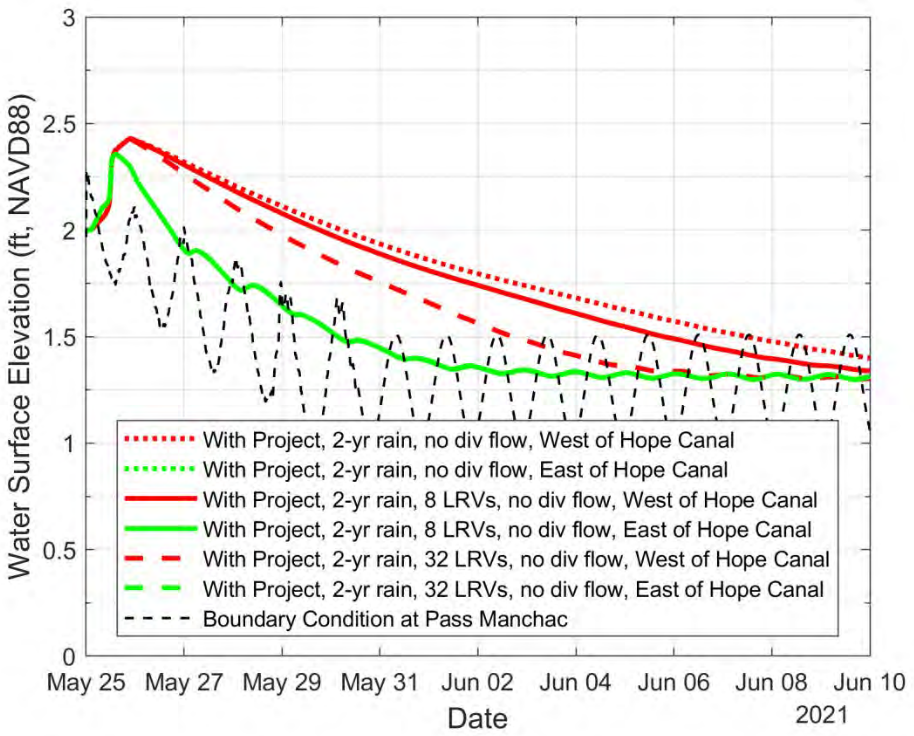

| WSE ft, NAVD88 | Days to reach a specific WSE | | |
|---|---|---|---|
| | Run 21a No LRVs | Run 25 8 LRVs | Run 24 32 LRVs |
| 2.25 | 2.6 | 2.4 | 2.0 |
| 2 | 5.1 | 4.7 | 3.8 |
| 1.75 | 8.6 | 7.7 | 6.0 |
| 1.5 | 13.3 | 11.6 | 8.6 |

**(21a) 2-yr rain, with-project, no div. flow**

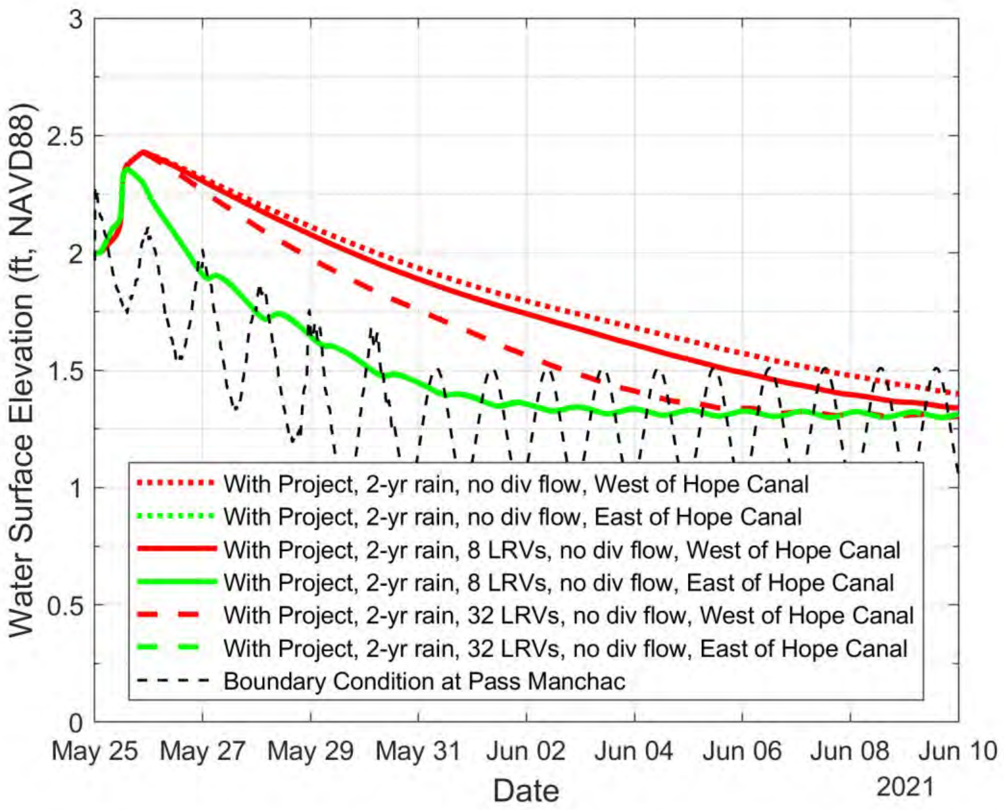


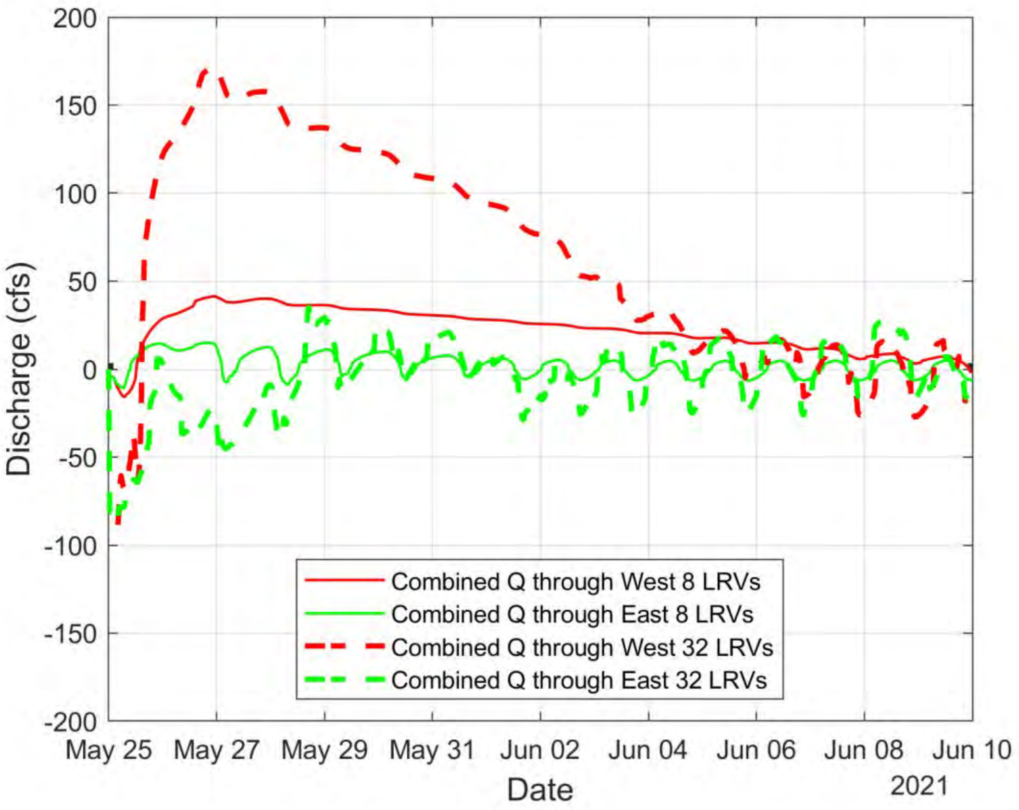

**(25) 2-yr rain, 8 LRVs with-project, no div. flow**



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

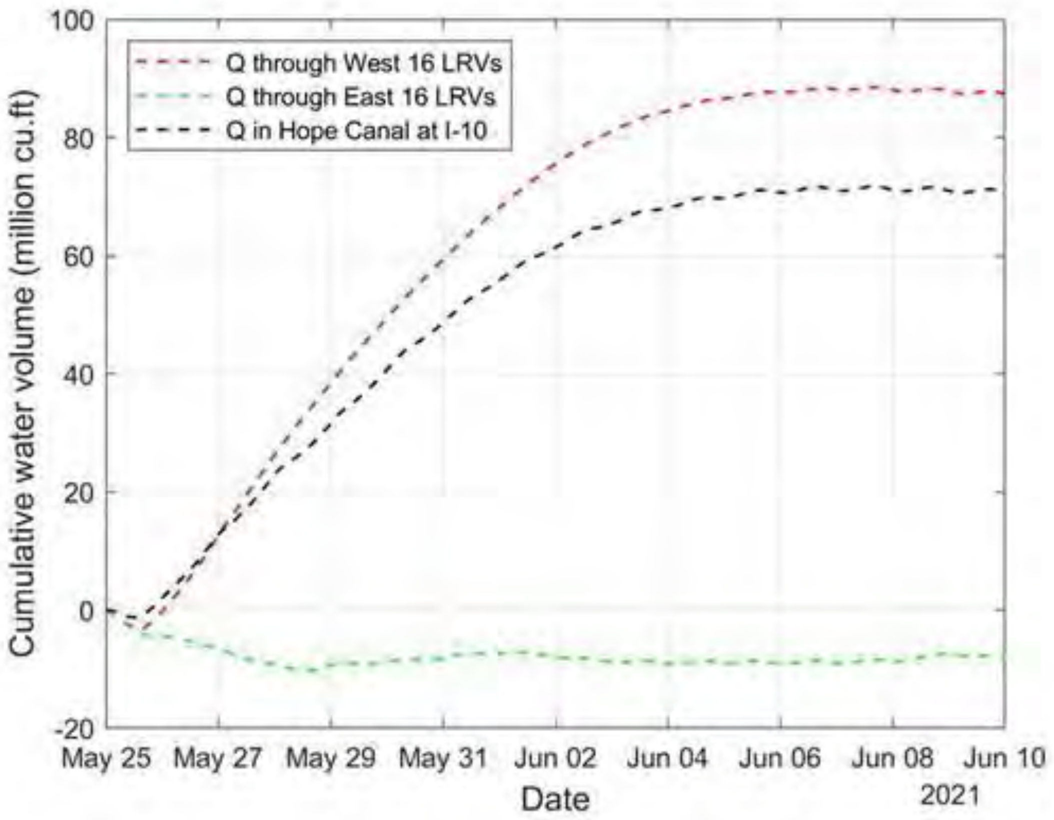



- Red lines are the water levels in the west polder
- Green line is close to the water levels in Hope Canal
- Difference between the two, shows head across the LRVs

- 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

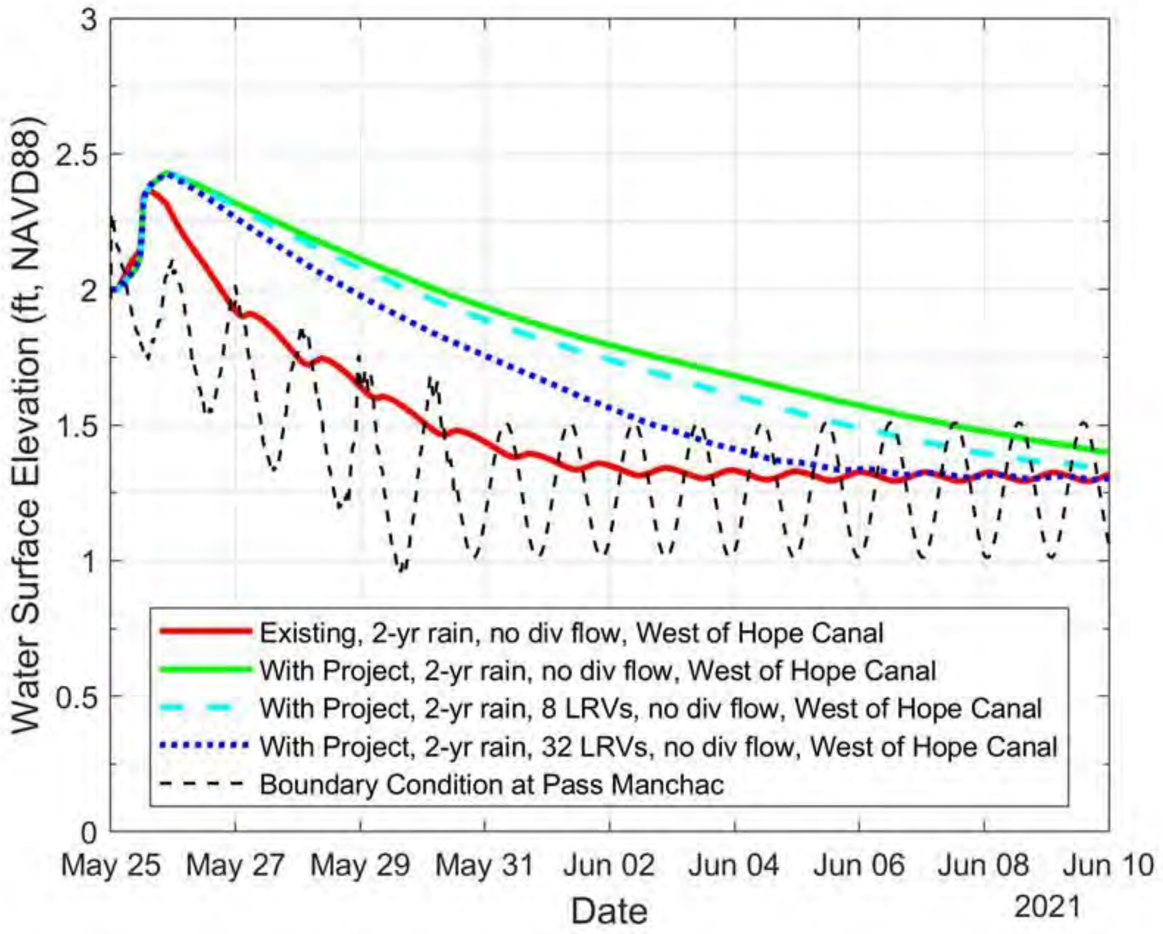
# Run 24: 32 LRVs- Cumulative volume of water draining through LRVs and Hope Canal




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.

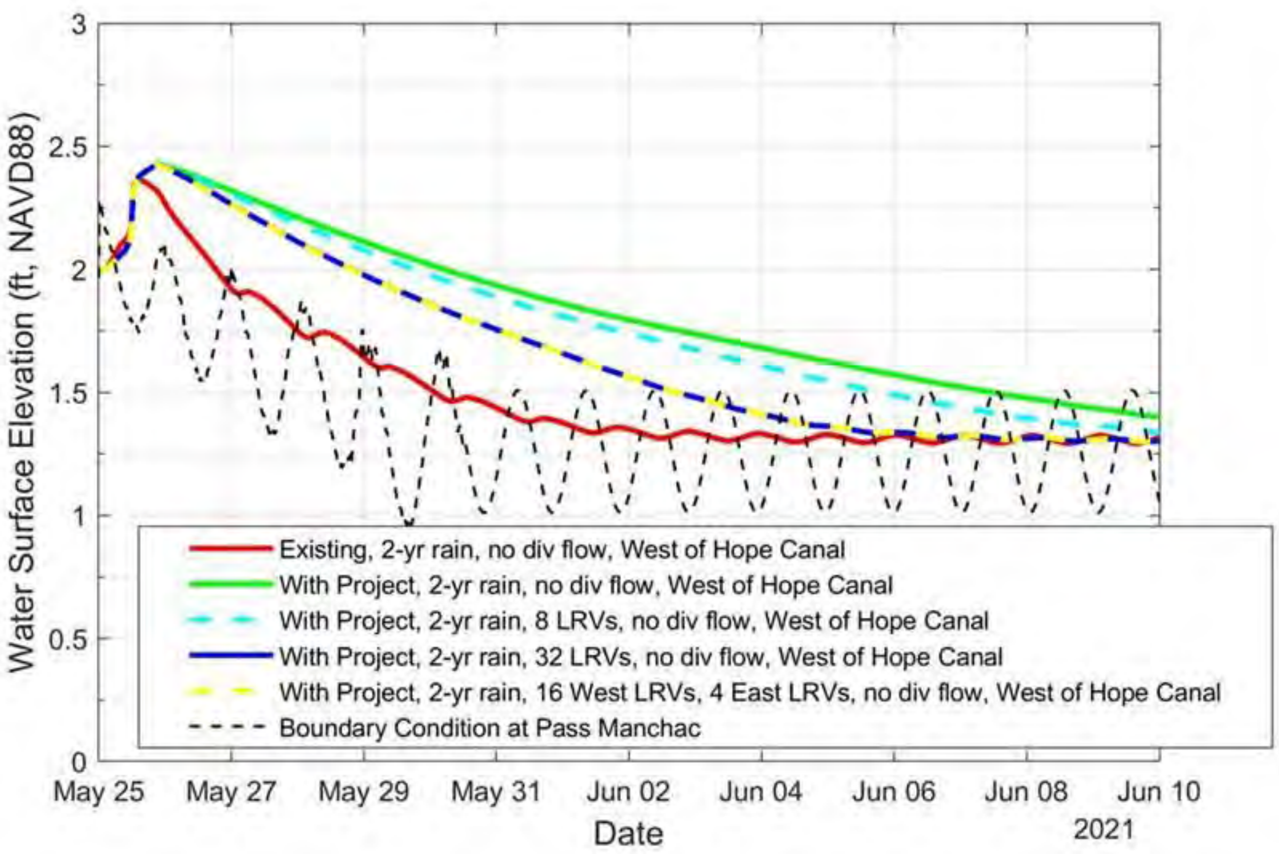
Note that the final volumes add up approximately to provide a mass balance. They will not add up exactly because the Hope Canal tracking is slight north of the exact outfall location and we loose some accounting due to this.

# Cases 8 (Run 25) and 32 (Run 24) LRVs: Water Levels for West Polder Only




This plot is same as the one on the previous slide except that this shows water levels in the west polder only

# 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.

# Section 3: Findings

- 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.
- 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. The east bank culverts are of no significant benefit to drain water out to east
- A model scenarios with 32 LRVs (16 west + 16 east) and 20 LRVs (16 west + 4 east) have similar drainage benefit to the west polder.

# Flow-3D Model Computational Fluid Dynamic Simulations

THE WATER INSTITUTE
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# Simulation of Flow near Proposed Dock Facility and Freshwater Diversion Reserve, Louisiana at River Mile 144.2


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John Richardson (ARCADIS)
Randy Lagumbay (ARCADIS)


July 15, 2015

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## SUGGESTED CITATION

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# Table of Contents

# List of Figures

# 1 Introduction

This report summarizes the results of Computational Fluid Dynamics (CFD) simulations used to study the impact of the construction of a proposed dock facility on the operation of a freshwater diversion at Mississippi River Mile (RM) 144.2 in Reserve, Louisiana. The dock facility and freshwater diversion proposed for construction at RM 144.2 are shown on Figure 1, and a location map is provided on Figure 2. Numerical flow simulations were carried out with the commercially available CFD program known as ***FLOW-3D*** (www.flow3d.com). This program is designed for the simulation of free surface and closed conduit flows, and was previously used by The Water Institute and ARCADIS U.S., Inc., to carry out hydrodynamic and sediment transport studies (Meselhe et al. 2012 and 2013).

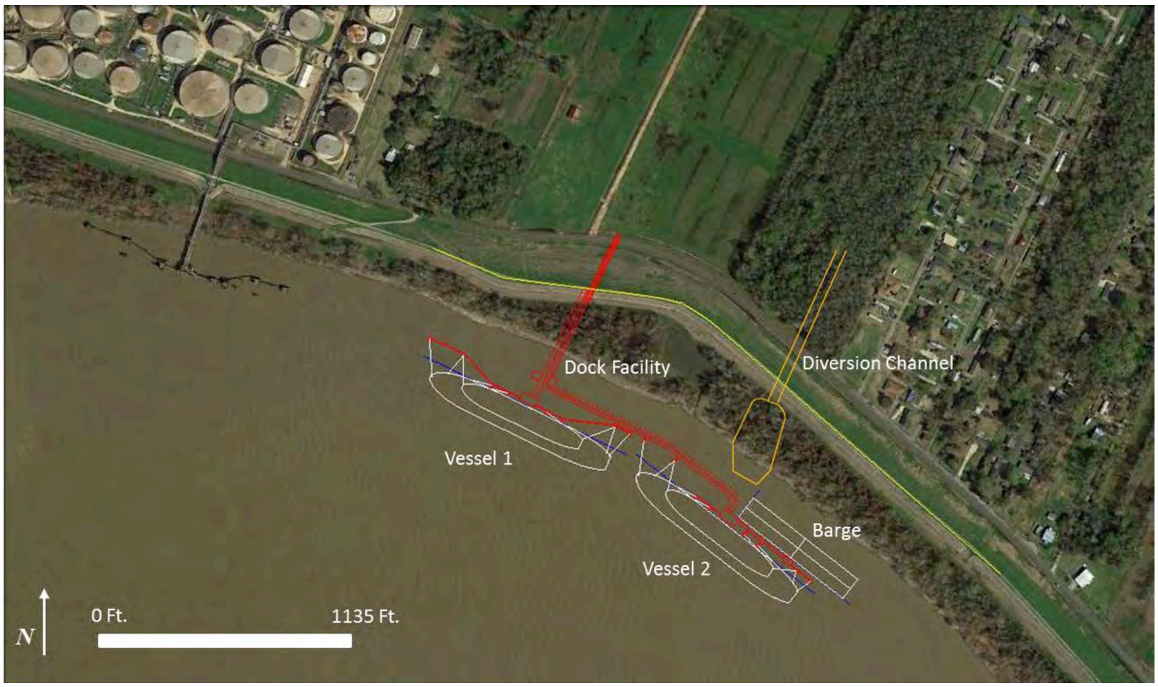



**Figure 1 – Proposed project – dock facility, freshwater diversion, vessels, and barge are shown.**

The primary objective of this analysis is to study the potential impact of the proposed dock facility on the freshwater diversion. Specific questions to be addressed are:

1. Does the proposed dock facility impact the flow of water into the diversion?
2. If a spill occurred at the proposed dock facility, would spilled material enter the freshwater diversion and make its way to the receiving marsh area?

The organization of this report is as follows. The approach used in the study is explained in Section 2, study scenarios are described in Section 3, model calibration and validation are discussed in Section 4, results of the model runs are provided in Section 5, and summary points and conclusions are made in Section 6. Additional graphics are provided in Appendix A.


**Figure 2 – Location map.**

# 2 Approach

A three-dimensional (3-D) CFD model of the proposed dock facility and freshwater diversion combined with a portion of the Mississippi River was used to simulate post-construction flow patterns at RM 144.2. The model was constructed within the framework of the commercially available CFD program known as ***FLOW-3D***[1] (Flow Science 2010). This program was selected for use because it is designed for the simulation of free surface and closed conduit flow (e.g., to simulate flow patterns in the Mississippi River) and it can be used to simulate scalar advection and dispersion (e.g., to simulate the movement of pollutants released into the river).

The procedure used to carry out the analysis was as follows:

1. Collect and review information for CFD model construction:
   a. Mississippi River bathymetry data;
   b. Drawings of the proposed diversion; and
   c. Drawings of the proposed dock facility and typical vessels likely to use the facility.

[1] ***FLOW-3D*** solves the Reynolds Averaged Navier-Stokes equations and uses various closure schemes to simulate the creation, transport, and dissipation of turbulent kinetic energy. The program also has the ability to calculate the movement of pollutant that is carried by the flow. The governing equations of fluid motion are formulated using non-linear transient, second-order differential equations. For more details, the interested reader is referred to the ***FLOW-3D*** Users' Manual (Flow Science 2010).

2. Specify model boundary conditions and construct a computational mesh for the CFD analysis. Construct Computer Aided Design (CAD) of:
    a. A portion of the Mississippi River (from approximately RM 150 to RM 140);
    b. The proposed diversion channel and intake structure; and
    c. The proposed dock facility including vessels.
3. Gather field data to calibrate and validate the computer model. Acoustic Doppler Current Profiler (ADCP) was used to collect velocity measurements at several cross sections in the study area.
4. Validate model results using existing and recently collected field data (i.e., without project).
5. Carry out simulations of flow patterns and pollutant transport for two river flow conditions.
6. Analyze the model results and draw conclusions regarding the effect of the proposed construction of the dock facility on the freshwater diversion and the movement of pollutants in the event of a spill to the proposed diversion.

## 2.1 FIELD DATA COLLECTION

A total of 12 ADCP transects in the Mississippi River reach (RM 148 to RM 140) in the vicinity of the project site were collected. These velocity transects were used to validate the ***FLOW-3D*** model (see Section 4). The transects were selected to ensure the model would capture the approach flow (velocity distribution) upstream of the project area as well as the velocity distribution downstream of the project area. The average discharge of all transects was approximately 625,000 cfs with a standard deviation of +/-9,294 cfs. At the time of collecting the ADCP data, the USGS stations at Baton Rouge (RM 229) and Belle Chase (RM 76) reported discharges of 690,000 and 656,000 cfs, respectively, while the water stage measured at Reserve (RM138.7) was 13.95 ft. A report detailing the data collection effort can be found as an appendix to this report.

## 2.2 3-D MODEL CONSTRUCTION

CAD drawings provided by the Coastal Protection and Restoration Authority (CPRA) were reviewed and used to construct the 3-D CAD model of the proposed dock facility and freshwater diversion combined with a portion of the Mississippi River bathymetry and topography. The 3-D model mesh was created using **Rhino** software (https://www.rhino3d.com/; Rhinoceros 2014). The overall model domain included a portion of the Mississippi River, proposed diversion channel, and proposed dock facility including vessels in some cases. Four separate models were developed to support different types of simulations:

- The first simulation was of flow in the Mississippi River without the project. The river bathymetry was created from a digital elevation model (DEM) provided by The Water Institute (Figure 3). The DEM is based on a multi-beam bathymetric survey performed by the USA Corps of Engineers – New Orleans District and was used as part of the LCA- Mississippi River Hydrodynamic and Delta Management Study. The model domain extended from RM 150 to RM 140. Figure 4 shows the completed bathymetry model ready for FLOW-3D simulation.
- The second model combined a portion of the Mississippi River, the proposed diversion structure, and diversion channel. The entrance to the diversion structure is 106 feet wide, it is lined with riprap, and it is set at an elevation of -4 feet North American Vertical Datum of 1988 (NAVD88).

The diversion channel, located at the end of the structure, is nominally 36 feet wide and extends for about 29,000 feet. Design drawings were provided by CPRA/URS.

- The third model was of a portion of the Mississippi River, proposed diversion structures, and proposed dock facility. The dock facility is located about 200 feet away from the diversion entrance and in this analysis two vessels and a barge were located at the facility (Figures 5 and 6). Design drawings were provided by CPRA/Lanier & Associates Consulting Engineers, Inc.
- The fourth model included a portion of the Mississippi River, proposed diversion channel, proposed dock facility, and vessels. The vessels had an overall length of 905 feet and were fully laden. Details regarding the setup of the spill simulations appear in Appendix A (Figures A-25 through A-27) and were provided by Lanier & Associates Consulting Engineers, Inc. Figures 5 and 6 show the model with diversion channel, dock facility, and vessels incorporated into the Mississippi River bathymetry.

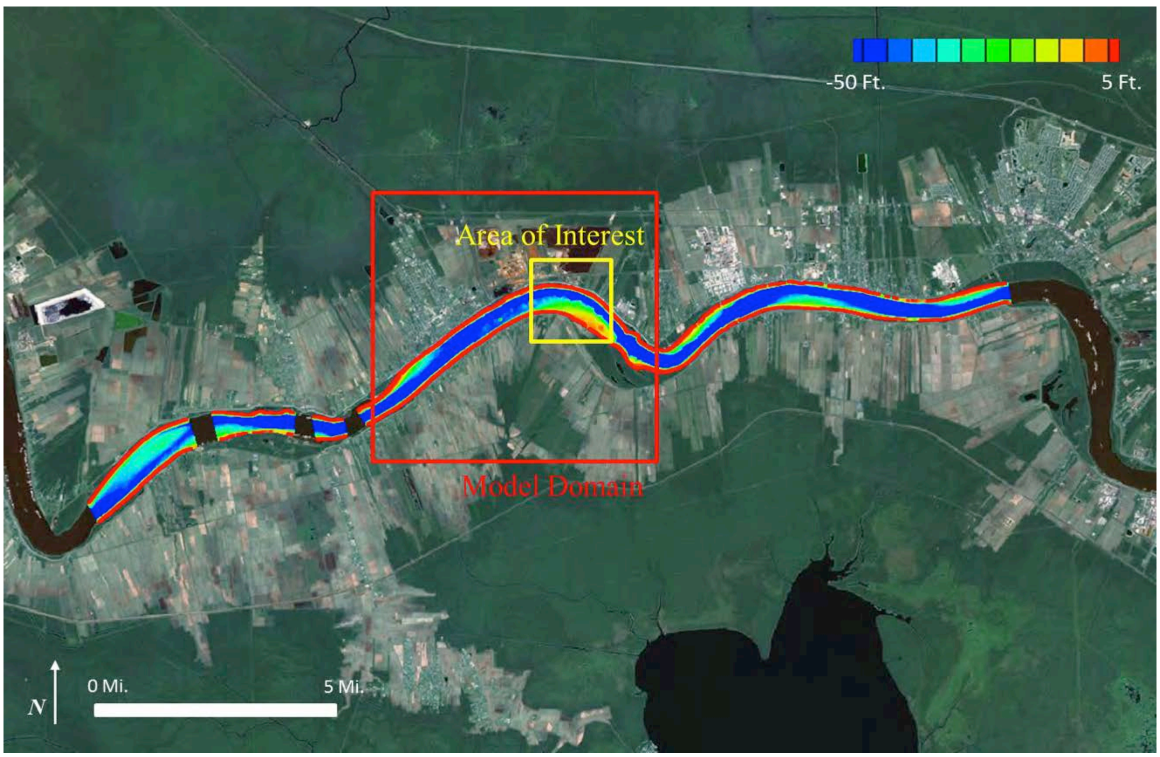



**Figure 3 – Mississippi River DEM.**

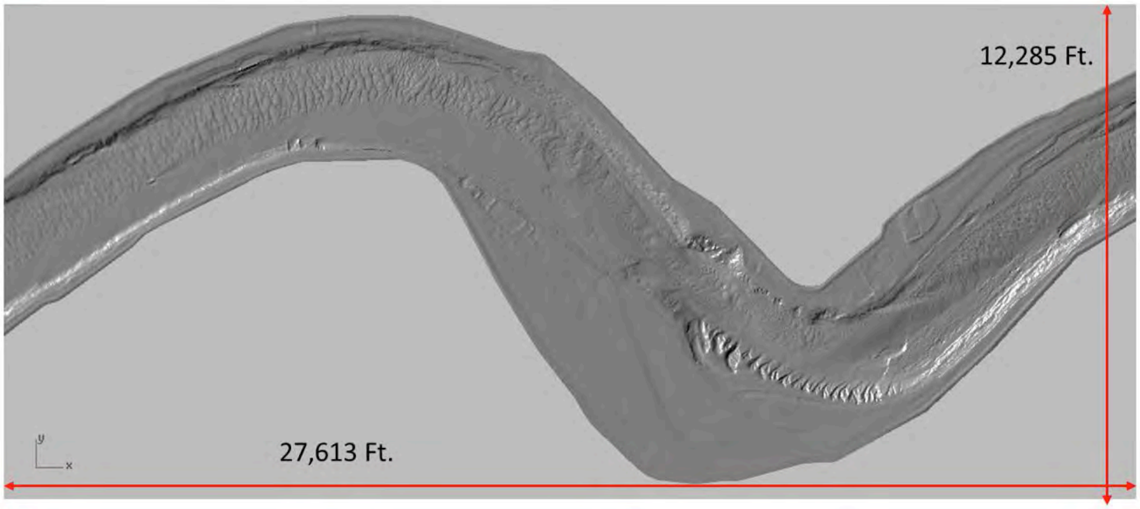



**Figure 4 – A portion of the Mississippi River ready for use as FLOW-3D input. Plan view.**

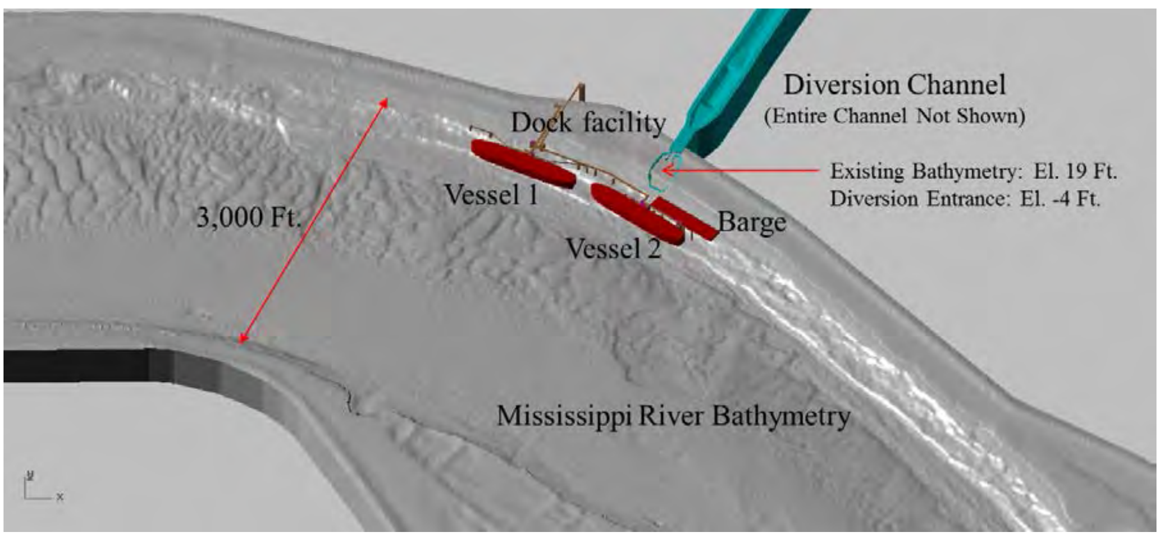



**Figure 5 – 3-D model of Mississippi River, diversion channel, dock facility, vessels, and barge. Oblique view.**

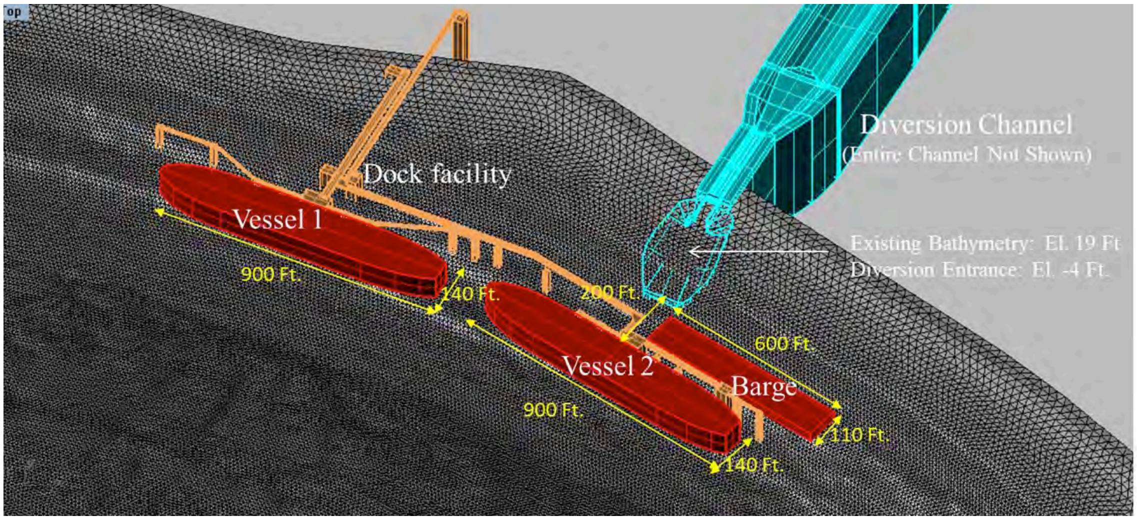



**Figure 6 – Mississippi River Model – Terminal and Diversion Structure. Oblique view.**

## 2.3 BOUNDARY CONDITIONS

Boundary conditions were specified as follows:

- Volumetric flow rate at the upstream end of the model (approximately 27,000 feet from the project area);
- Water surface elevation at the downstream end of the model (approximately 6,000 feet from the project area);
- Water surface elevation at the downstream end of the diversion channel (Note: the diversion channel was straightened – compare Figures 7 and 8 – to reduce the computational demands of the model);
- No-slip conditions were applied at solid boundaries; and
- Free-surface conditions were imposed at the air/water interface.

Model boundary conditions are identified on Figure 7, and the treatment of the diversion channel is illustrated on Figure 8.

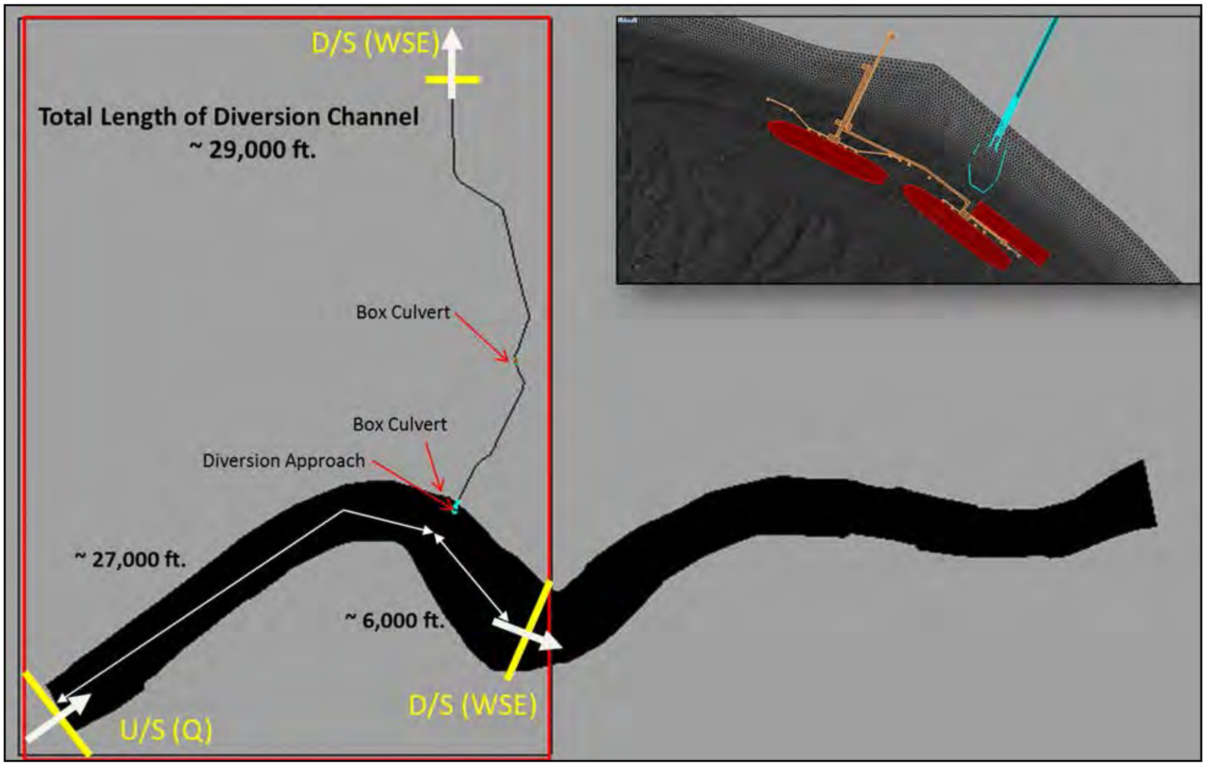



**Figure 7 – Model boundary conditions. Flow rate (Q) specified at the upstream (U/S) boundary; water surface elevations (WSE) specified at the downstream (D/S) boundaries.**

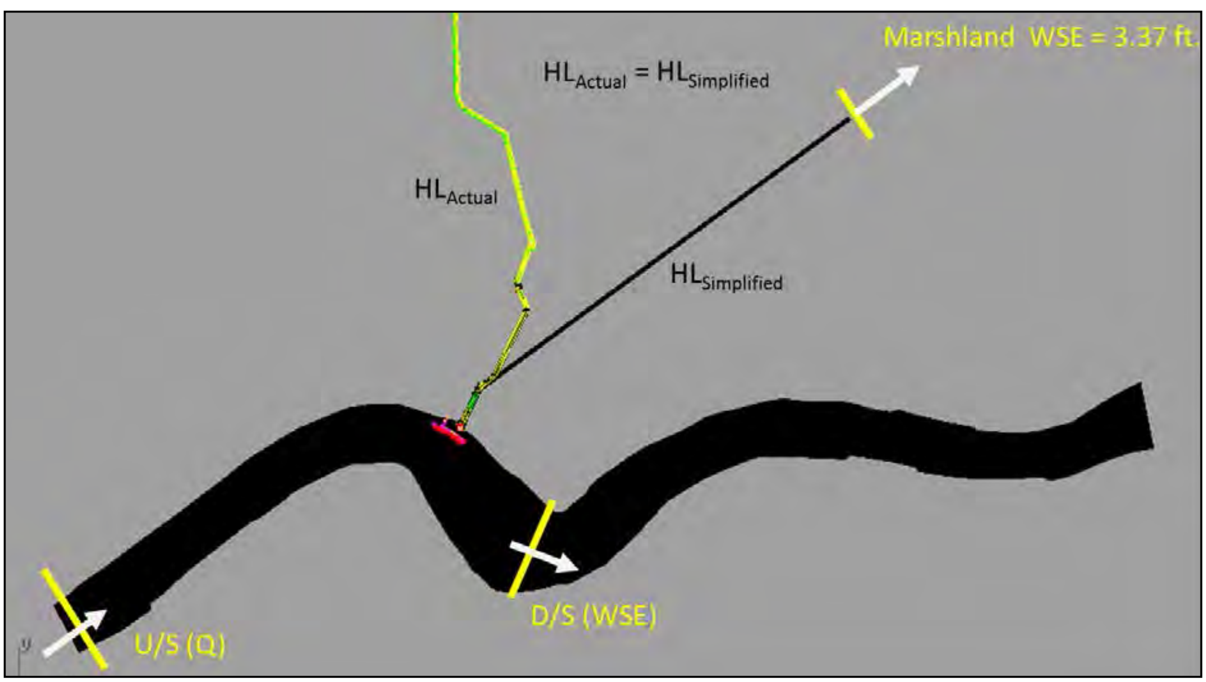



**Figure 8 – Model setup with diversion channel – actual and simplified. The headloss (HL) in the actual channel is the same as the HL in the simplified channel.**

## 2.4 COMPUTATIONAL MESH

In order to carry out the simulations, the model domain was subdivided into a number of small elements (cells) within which the governing equations of fluid flow were solved. Refinements to the mesh were made locally to improve the ability of the model to capture details of the flow near the dock facility and entrance to the diversion.

In this study, a multi-block mesh was used. Cell sizes in each block varied from 2 to 40 feet horizontally and 2 to 10 feet vertically. Cell sizes were the smallest near the dock facility and entrance to the diversion. Figure 9 shows the computational mesh used in the model, and Figure 10 shows the enhancement of the mesh near the dock facility and entrance to the diversion.

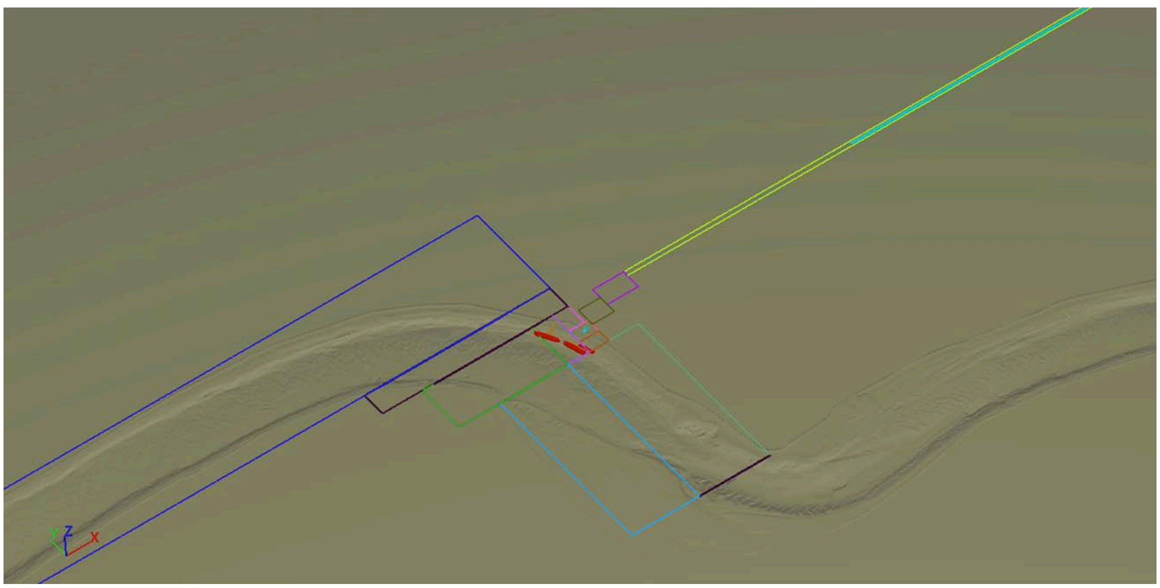


**Figure 9 – Computational model mesh block configuration.**

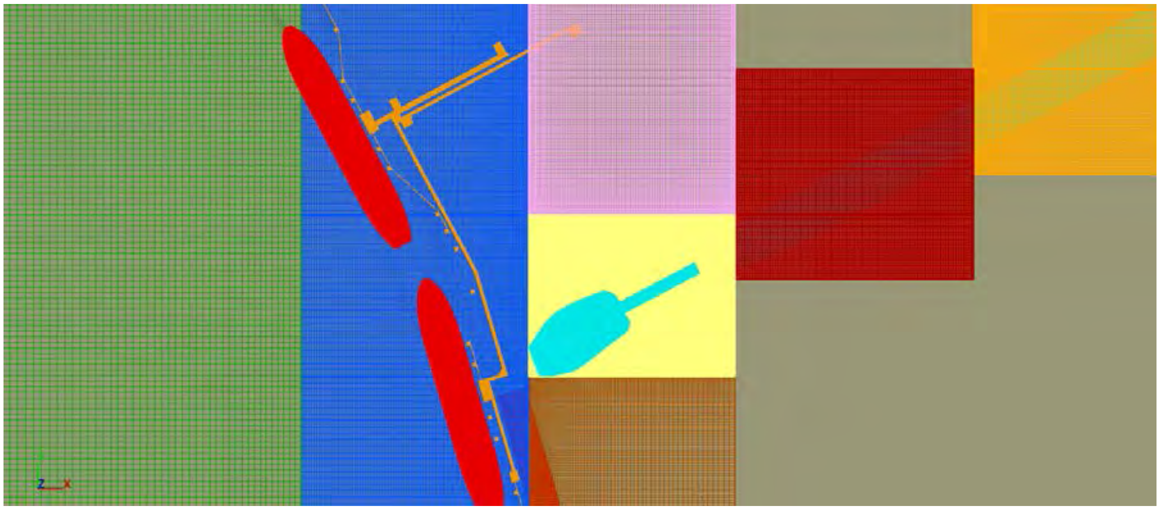



Figure 10 – Mesh in the vicinity of the facility and entrance to the diversion channel. Green – 20x20x6, Blue – 4x4x4, Brown – 4x4x4, Yellow – 2x2x3, Pink – 4x4x4, Maroon – 4x4x3, Orange – 4x4x3. Units are in feet.

# 3 Simulations

Several simulations were carried out in this study at two different Mississippi River flows – 656,000 cubic feet per second (cfs) corresponding to the condition when data were collected and 1,500,000 cfs corresponding to a higher river flow rate (together these are referred to as the low-flow and high-flow simulations). A description of these runs is provided below:

<u>Run #1 - River Flow Only (Low Flow, Validation)</u>
The purpose for this simulation was to provide a comparison with measured data for model validation. The discharge in the Mississippi River was set to 656,000 cfs (based on information from Belle Chasse, and confirmed by the ADCP measurements, and from the Baton Rouge station) at the upstream model boundary and water surface elevation was set equal to 13.95 feet NAVD88 (from Reserve, RM 138.6) at the downstream boundary.

<u>Run #2 - River Flow with Operating Freshwater Diversion (Low Flow)</u>
The purpose for this simulation was to predict changes in flow patterns resulting from the construction of the proposed freshwater diversion. Similar to Run #1, the discharge in the Mississippi River was set to 656,000 cfs at the upstream model boundary and water surface elevation was set equal to 13.95 feet NAVD88 at the downstream boundary. In addition to these specifications, the water surface elevation at the downstream end of the diversion channel was set to 3.37 feet NAVD88 (from URS Hydrologic Engineering Center's River Analysis System [HEC-RAS]). The magnitude of the diversion flow was, therefore, determined by a balance between the difference in water surface elevations (10.58 feet) and headloss in the diversion channel.

Run #3 - River Flow with Diversion and Dock Facility (Low Flow)
The purpose for this simulation was to predict changes in flow patterns resulting from the construction of the proposed freshwater diversion and the docking facility.  Similar to Runs #1 and #2, the discharge in the Mississippi River was set to 656,000 cfs, the downstream water surface elevation equal to 13.95 feet NAVD88, and the water surface elevation at the end of the diversion channel was set to 3.37 feet NAVD88.

Run #4 - River Flow with Diversion, Dock Facility, and Fully Laden Vessels (Low Flow)
The purpose for this simulation was to predict changes in flow patterns resulting from the construction of the proposed freshwater diversion and the dock facility in operation (i.e., with fully laden vessels tied up to the facility).  Model boundary conditions were similar to those used in Runs #2 and #3.

Run #5 - River Flow Only (High Flow)
The purpose for this simulation was to provide a comparison with low-flow model results (Run #1).  The model geometry was the same as that used in Run #1, but the discharge in the Mississippi River was increased to 1,500,000 cfs, and the water surface elevation at the downstream model boundary was increased to 23.16 feet NAVD88.

Run #6 - River Flow with Operating Freshwater Diversion (High Flow)
The purpose for this simulation was to predict changes in flow patterns resulting from construction of the freshwater diversion and to provide a comparison with the results of low-flow model simulation #2. Similar to Run #5, the discharge in the Mississippi River was increased to 1,500,000 cfs, and the water surface elevation at the downstream river boundary was increased to 23.16 feet NAVD88.  In addition to these boundary conditions, the water surface elevation at the end of the diversion channel was set at 3.37 feet NAVD88 (the same as the elevation used in Runs #2, 3, and 4), so the magnitude of flow in the diversion channel was determined by a balance between the difference in water surface elevations (20.78 feet) and headloss in the diversion channel.

Run #7 - River Flow with Diversion and Dock Facility (High Flow)
The purpose for this simulation was to predict changes in flow patterns resulting from construction of the freshwater diversion and dock facility and to compare with low-flow model results from Run #3.  The model boundary conditions were the same as those used in Run #6.

Run #8 - River Flow with Diversion, Facility, and Fully Laden Vessels (High Flow)
The purpose for this simulation was to predict changes in flow patterns resulting from construction of the proposed freshwater diversion and docking facility with vessels and to compare with low-flow model results from Run #4.  The model boundary conditions were the same as those used in Runs #6 and #7.

Run #9 – Pollutant Transport with Fully Laden Vessels (Low Flow)

The purpose for this simulation was to predict the movement of Bitumen (a.k.a. pollutant) away from the docking facility in the event of a spill.[2] Low-flow boundary conditions similar to those used in Runs #2, #3, and #4 were used in this simulation.

Input parameters[3] used to characterize a temporary Bitumen spill from a loading arm were as follows:

*Discharge Characterization:* *Volumetric Flow Rate = 46.79 $ft^3$/sec*
*Time Duration of Discharge = 35 seconds*
*Constant or Time Varying Discharge= Constant*

*Fluid Properties of Discharge:* *Density = 8.48 lb/gal*[4]
*Viscosity = 19770 cSt@104$^{o}$F*

*Discharge Location:* *Vessel 1 = 165,000 DWT (See Figure 11)*

*Size and Shape of Discharge:* *Discharge from two 12-inch Loading Arms*

[2] In this analysis, the pollutant (Bitumen) was modeled as a second fluid with a density and viscosity different from that of water; in the computations, local fluid properties were based on the amount of each constituent (Bitumen or water) in a given control volume. In this way, the tendency for Bitumen to sink could be accounted for in the simulations. Note: Changes in fluid properties associated with degradation, the loss of volatiles, etc., were not modeled.

[3] See Appendix A (Figures A-25 through A-27) for additional information used to determine input parameters for this simulation.

[4] The density of Bitumen is greater than that of water; therefore, without mixing, this pollutant would fall directly to the bottom.

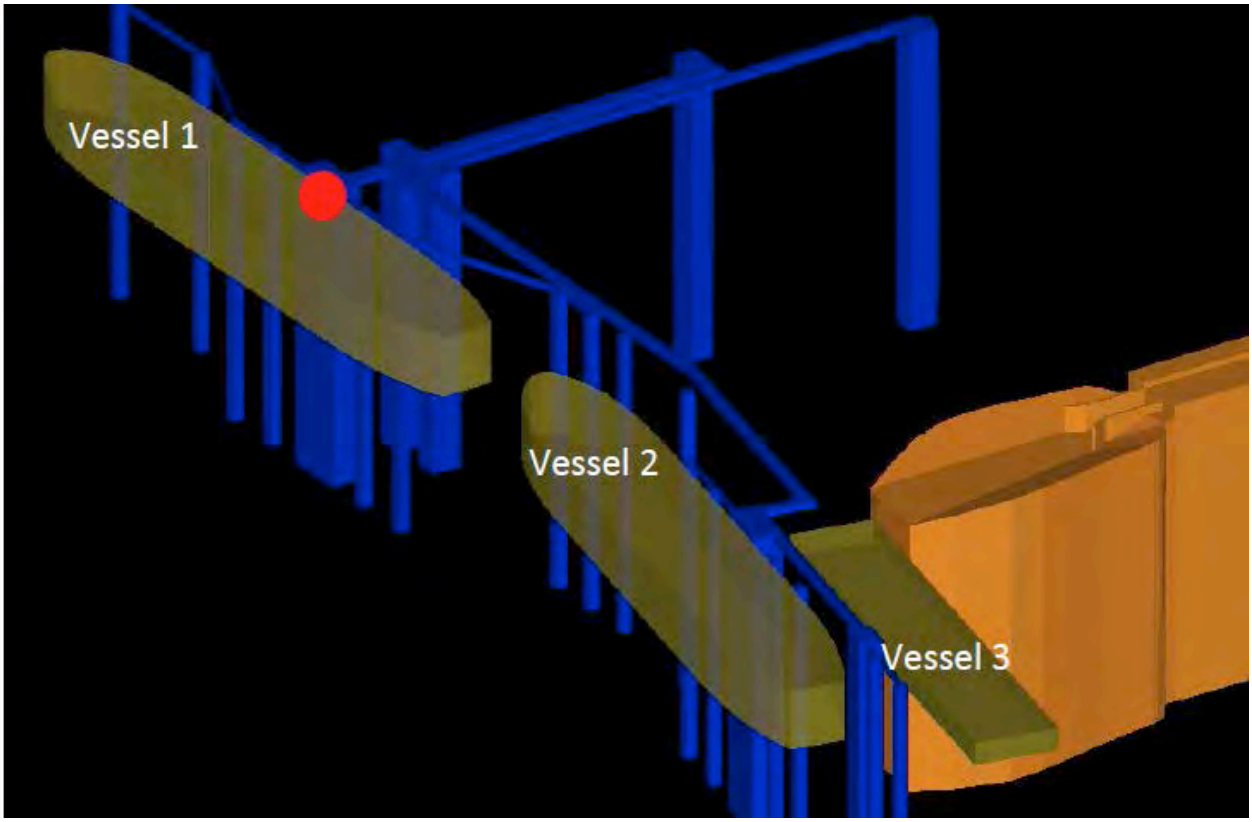



**Figure 11 – Discharge location (red dot) for Run #9.**

Run #10 – Pollutant Transport with Fully Laden Vessels (High Flow)

The purpose for this simulation was to predict the movement of pollutants away from the docking facility in the event of a vessel collision resulting in a Bitumen spill. High-flow boundary conditions similar to those used in Runs #6, #7, and #8 were used in this simulation.

The input parameters[5] used for simulation of pollutant transport are given below:

*Discharge Characterization:* *Volumetric Flow Rate = 13.23 $ft^3$/sec*
*Time Duration (Constant) = 3.87 Hours*
*Constant or Time Varying = Constant*

*Fluid Properties of Discharge:* *Density = 8.48 lb/gal*
*Viscosity = 19770 cSt@104$^{o}$F*

*Discharge Location:* *Vessel 1 = 165,000 DWT (See Figure 12)*

*Size and Shape of Discharge:* *Discharge through an 8-inch X 16-inch gash (Diamond Shape)*

[5] See Appendix A (Figures A-25 through A-27) for additional information used to determine input parameters for this simulation.

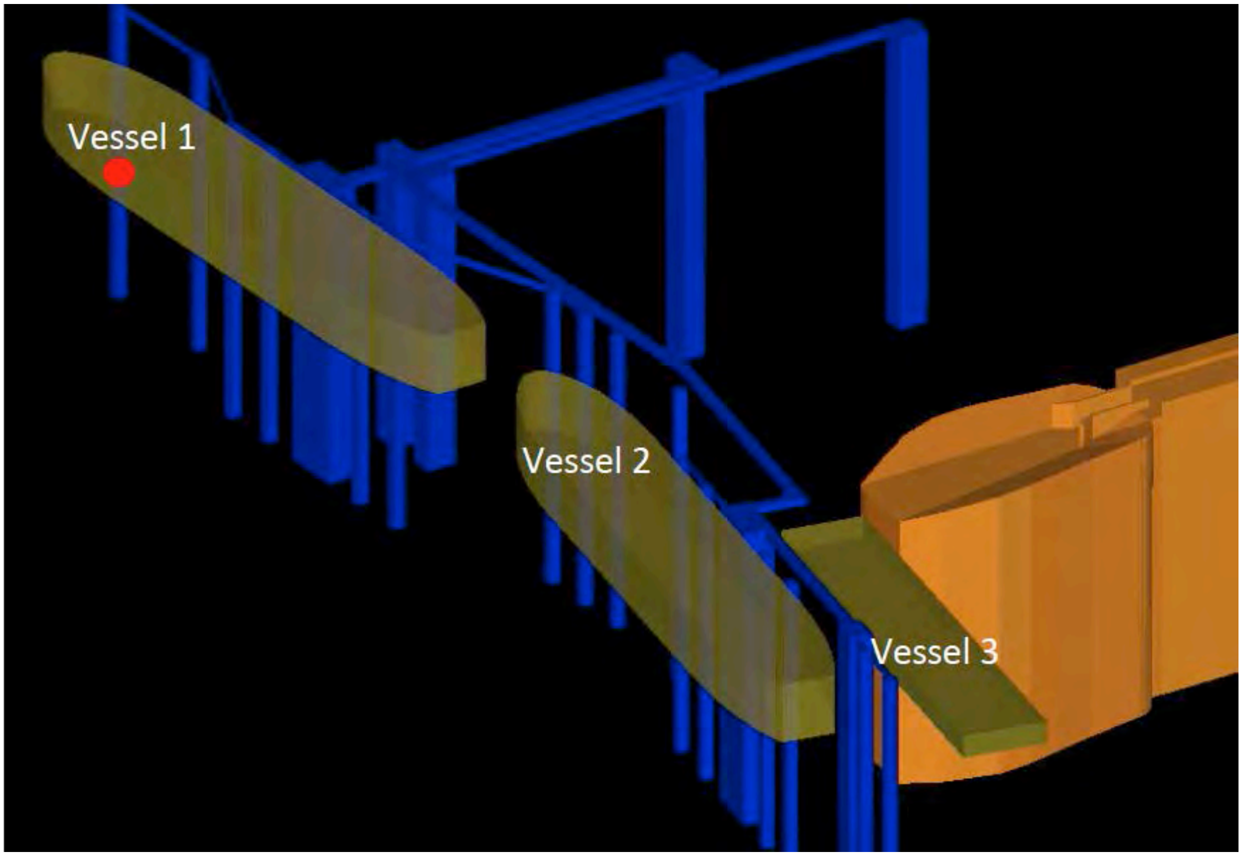



**Figure 12 – Discharge location (red dot) for Run #10.**

# 4 Model Validation

A comparison of calculated results and velocities measured at 12 different transects in the Mississippi River using an ADCP (Figure 13) was used to validate the model.

Model results from Run #1 described in Section 3 were compared to measured data. Run #1 was based on the river's existing layout (without project) and was used to simulate flow patterns in the Mississippi River similar to those observed in the field. Figure 14 shows a snapshot of the velocity contours in the Mississippi River for the existing flow condition. A comparison of velocity data at transects "006" and "007" is provided on Figure 15. As can be seen, model results agree well with the data. Additional graphics showing similar comparisons at other transects are provided in Appendix A (Figures A-1 through A-6).

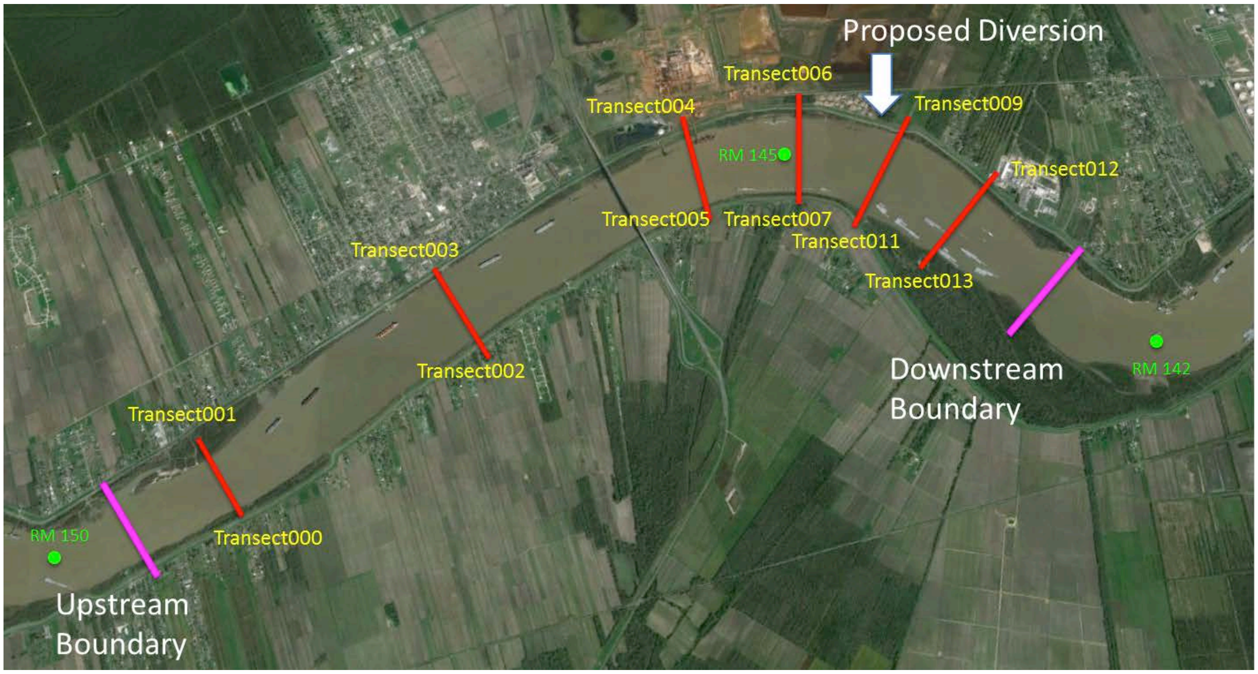



**Figure 13 – ADCP transect locations.**

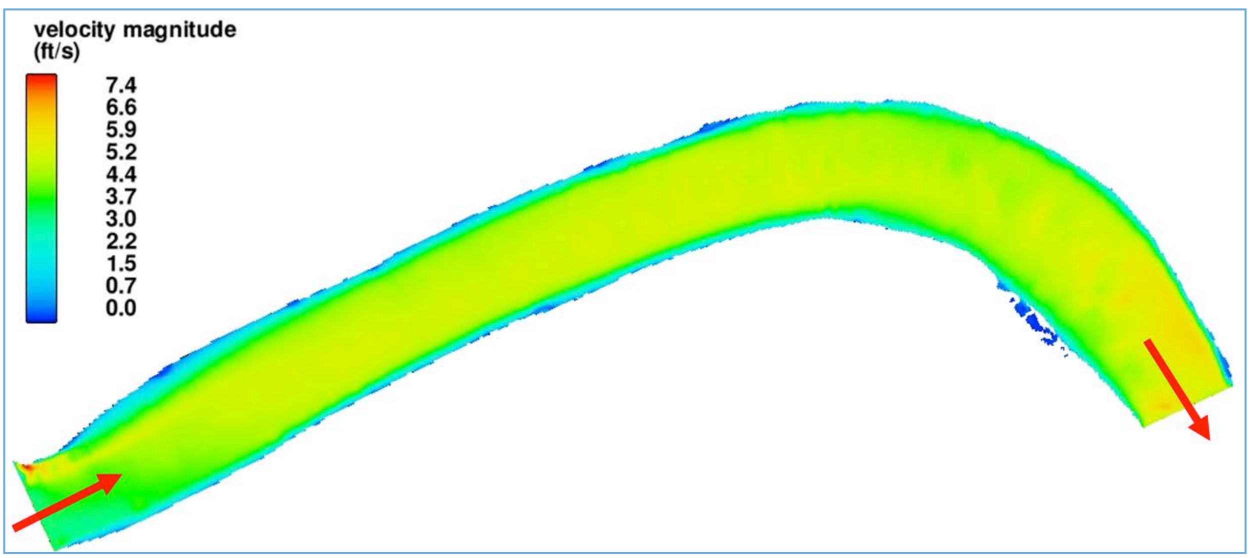
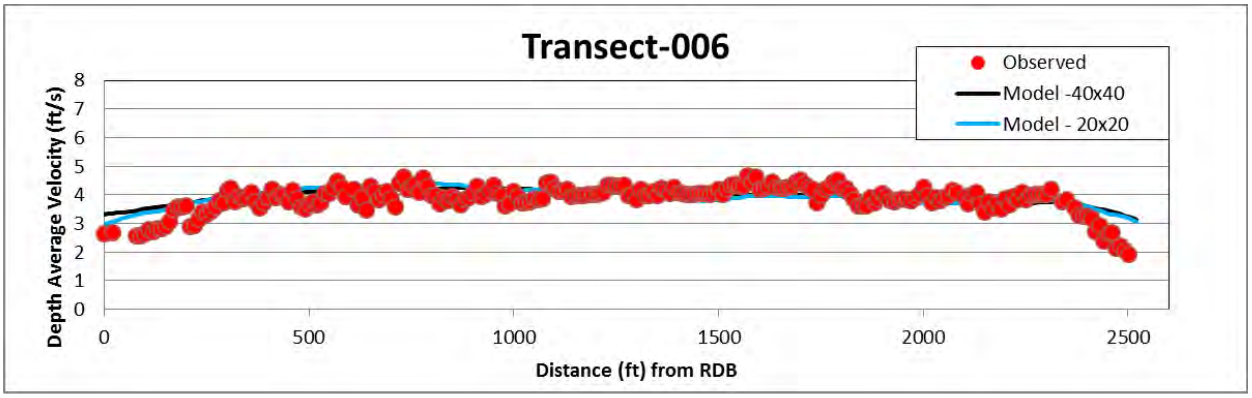



**Figure 14 – Surface velocity contours in the Mississippi River (low-flow condition – 656,000 cfs).**



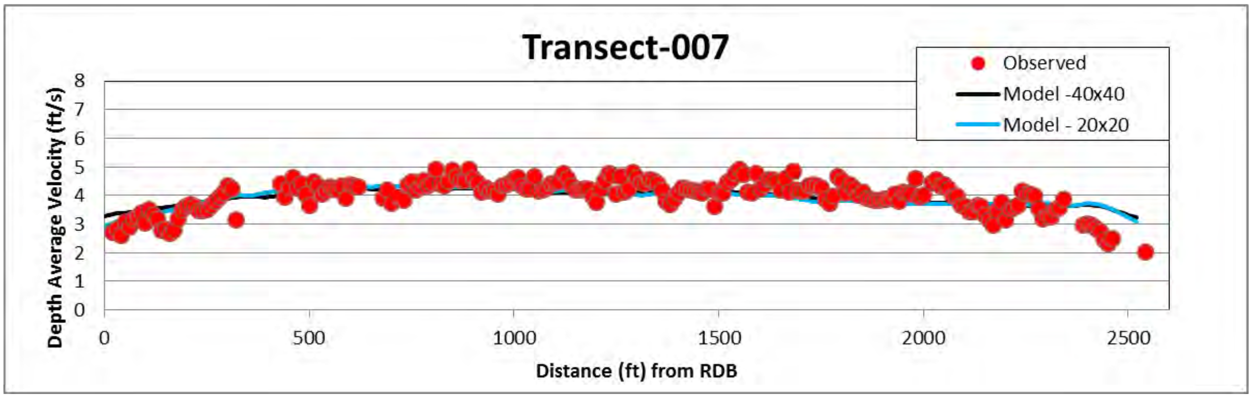




**Figure 15 – Comparison between CFD model and field data at transects #006 and #007. Red dots show the field data and model results are given for two different meshes – 40x40 feet in size and 20x20 feet in size. The results with either mesh are similar.**

# 5 Results

## 5.1 LOW RIVER FLOW

Simulation results, appearing on Figures 16 through 25, show velocity contours and the trajectory of streamlines in the vicinity of the docking facility and the entrance to the diversion channel for baseline conditions. Streamlines were back calculated from the diversion channel to show the source of water entering the diversion. Velocity contours were used to show approach flow patterns and flow separation behind the facility and vessels.

Figures 16 and 17 show velocity contours at elevation -3.0 feet NAVD88 and streamlines entering the diversion from different elevations in the water column for Run #2 (Low River Flow with Diversion).

Figures 18 and 19 show velocity contours at elevation -3.0 feet (NAVD88) and streamlines entering the diversion from different elevations in the water column for Run #3 (Low River Flow with Diversion and Facility).

Figures 20 and 21 show velocity contours at elevation -3.0 feet (NAVD88) and streamlines entering the diversion from different elevations in the water column for Run #4 (Low River Flow with Diversion, Facility, and Vessels).

Additional graphics showing modeled flow patterns for Runs #2, #3, and #4 are provided in Appendix A (Figures A-7 through A-12).

**Summary**

The streamlines shown on Figures 17, 19, and 21 approach the diversion along the shoreline and all look similar. Even though the proposed dock facility and diversion are only about 200 feet apart, according to these results, flow into the diversion is unaffected. For all of these simulations, the flow rate into the diversion was calculated to be about 3,200 cfs and was not affected by the presence of the dock facility or the vessels.

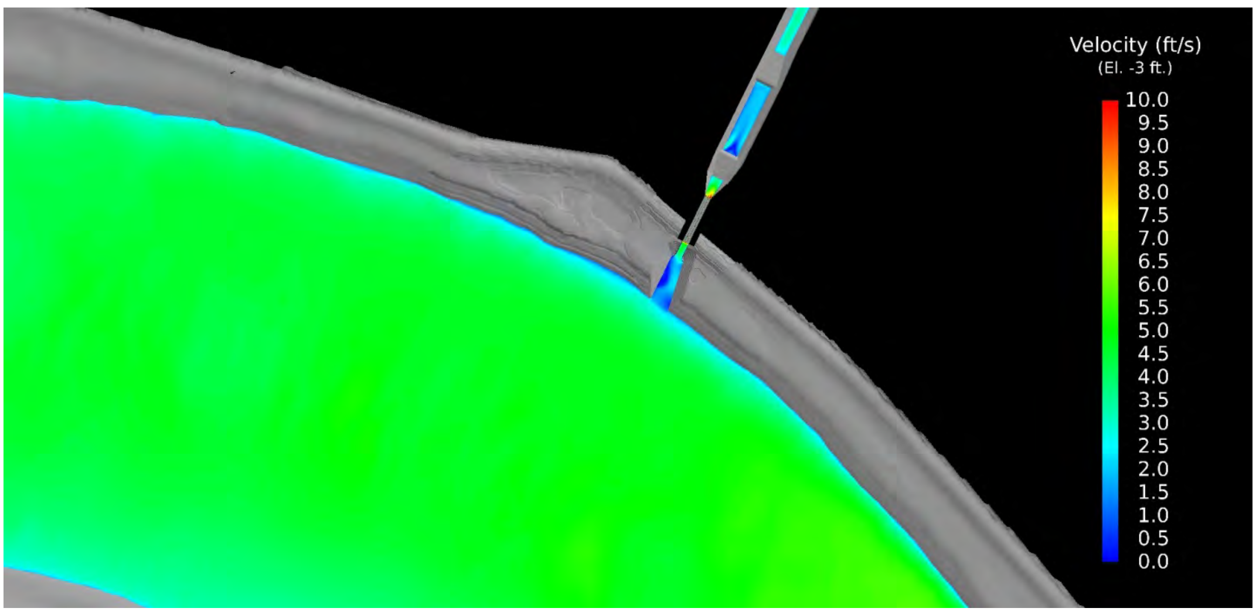



**Figure 16 – Run #2 model results. Velocity contours in the Mississippi River with the proposed diversion. Slice plane cut at elevation = -3.0 feet.**

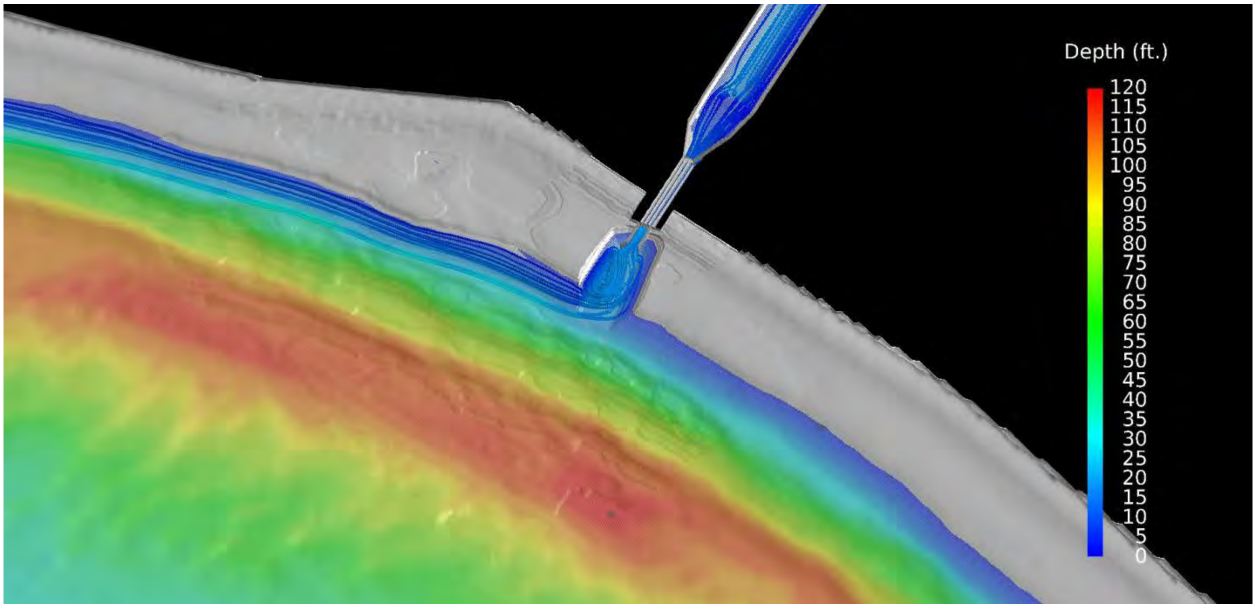



**Figure 17 – Run #2 model results. Streamlines colored by depth (back calculated from the diversion structure) with the proposed diversion. Background water surface (50% transparent) colored by depth. Diverted flow follows the shoreline near the water surface.**

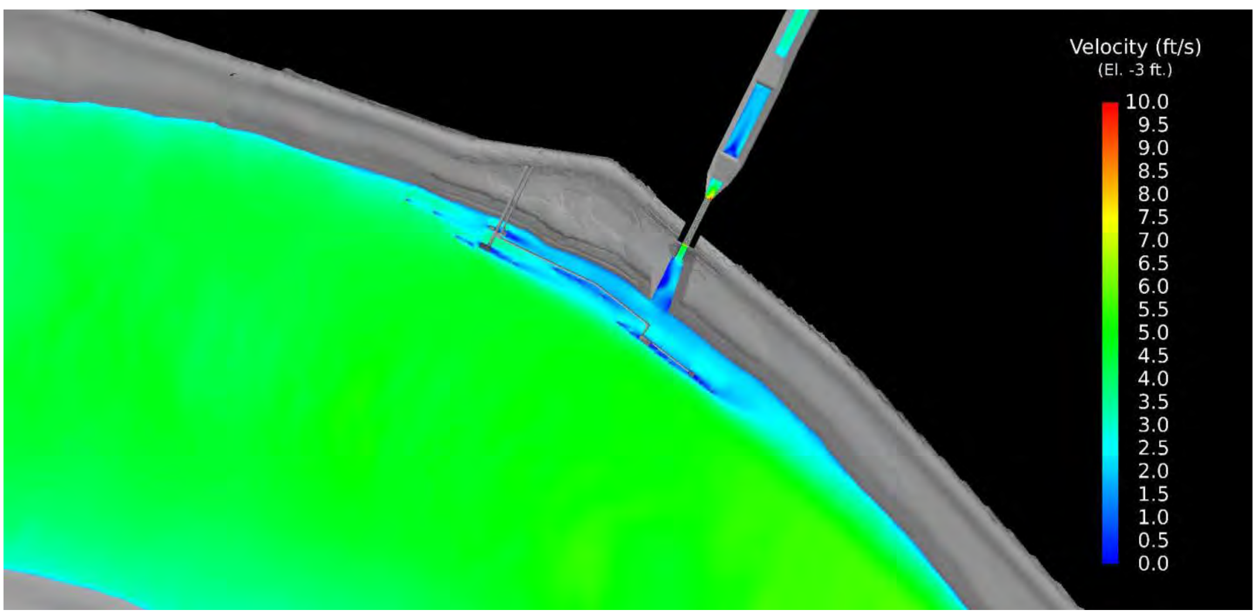



**Figure 18 – Run #3 model results. Velocity contours in the Mississippi River with the proposed diversion and dock facility. Slice plane cut at elevation = -3.0 feet.**

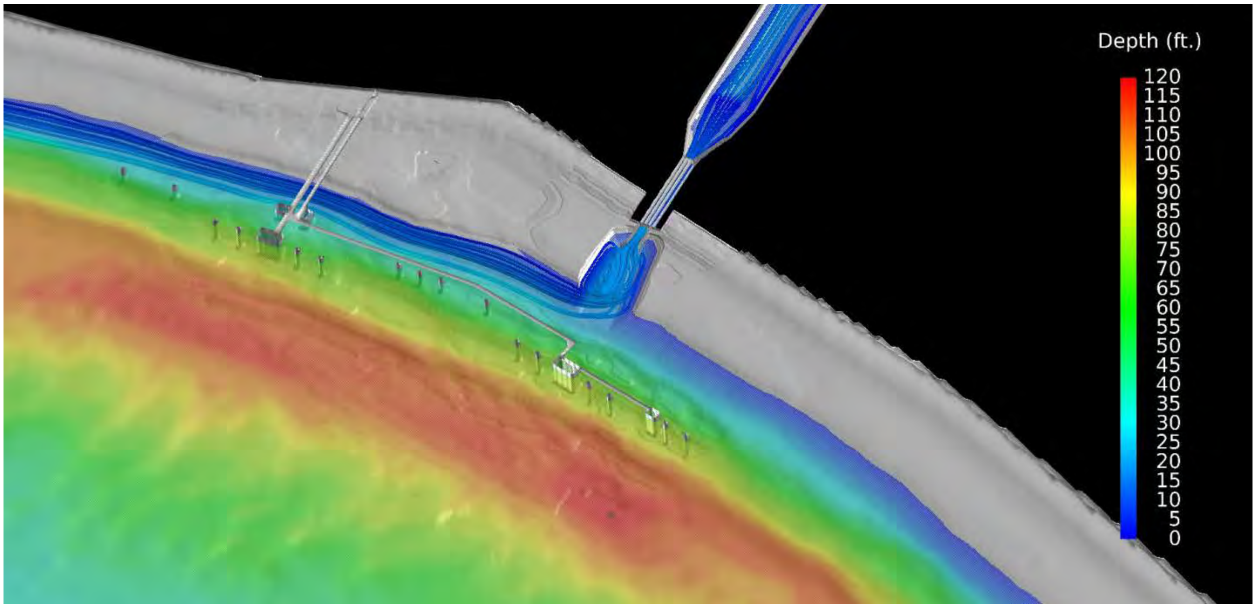



**Figure 19 – Run #3 model results. Streamlines colored by depth (back calculated from the diversion structure) with the proposed diversion and dock facility. Background water surface (50% transparent) colored by depth. Diverted flow follows the shoreline and near the water surface.**

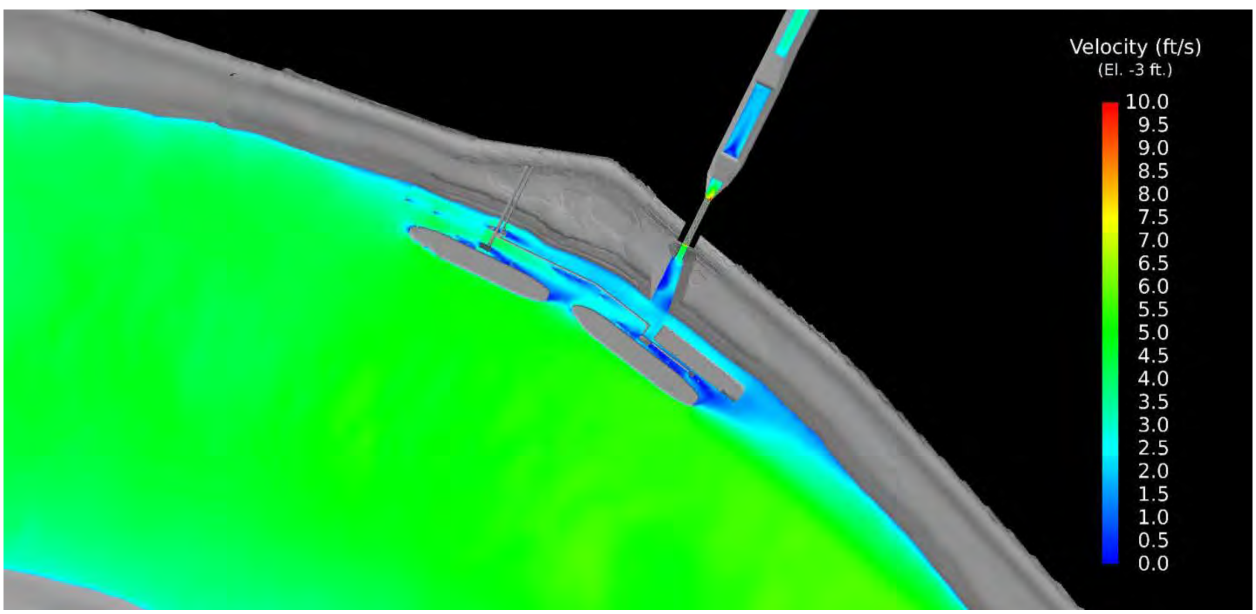



**Figure 20 – Run #4 model results. Velocity contours in the Mississippi River with the proposed diversion and dock facility including vessels. Slice plane cut at elevation = -3.0 feet.**

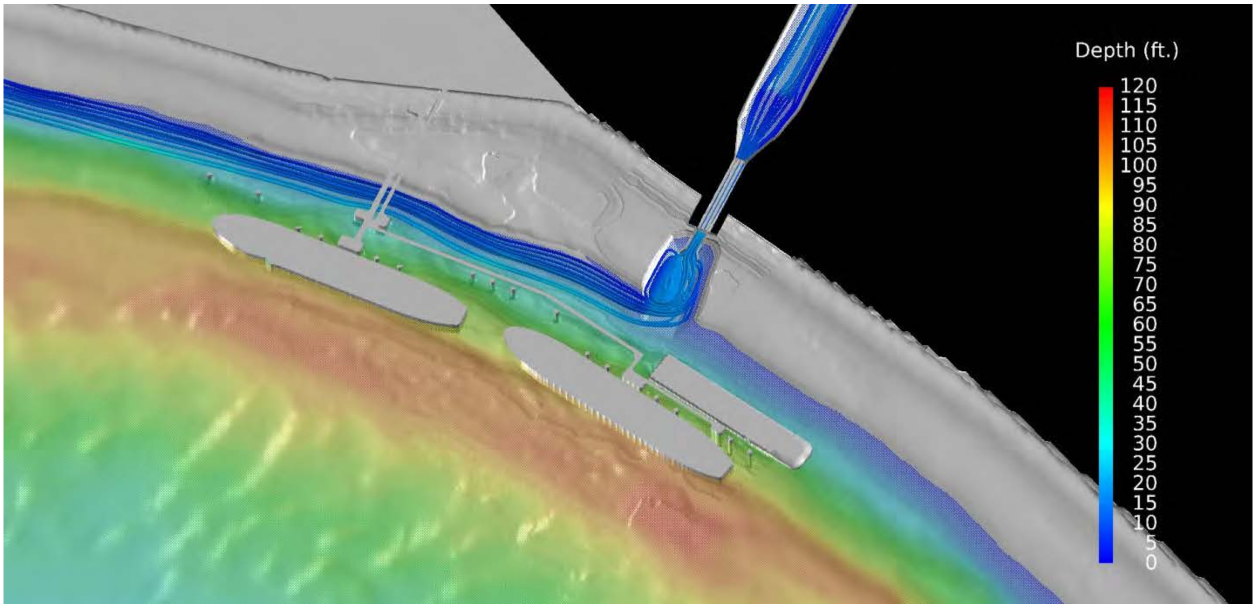



**Figure 21 – Run #4 model results. Streamlines colored by depth (back calculated from the diversion structure) with the proposed diversion and dock facility including vessels. Background water surface (50% transparent) colored by depth. Diverted flow follows the shoreline and near the water surface.**

## 5.2 HIGH RIVER FLOW

Figure 22 shows velocity contours at elevation -3.0 feet NAVD88 for Run #6 (High River Flow with Diversion).

Figure 23 shows velocity contours at elevation -3.0 feet NAVD88 for Run #7 (High River Flow with Diversion and Docking Facility).

Figures 24 and 25 show velocity contours and vectors at elevation -3.0 feet NAVD88 and streamlines entering the diversion from different elevations in the water column for Run #8 (High River Flow with Diversion, Docking Facility, and Vessels).

Additional graphics showing modeled flow patterns for Run #5 are provided in Appendix A (Figures A-13 and A-14).

**Summary**

Similar to the low-flow results, flow patterns approaching the intake of the diversion are the same with or without the dock facility and vessels in place. As before, flow entering the diversion follows the shoreline near the water surface.

Note: The flow rate into the diversion was calculated to be about 4,800 cfs for the conditions we studied. The conditions URS studied were based on a design flow rate of 2,000 cfs with a river stage of 9.38 feet NAVD88.

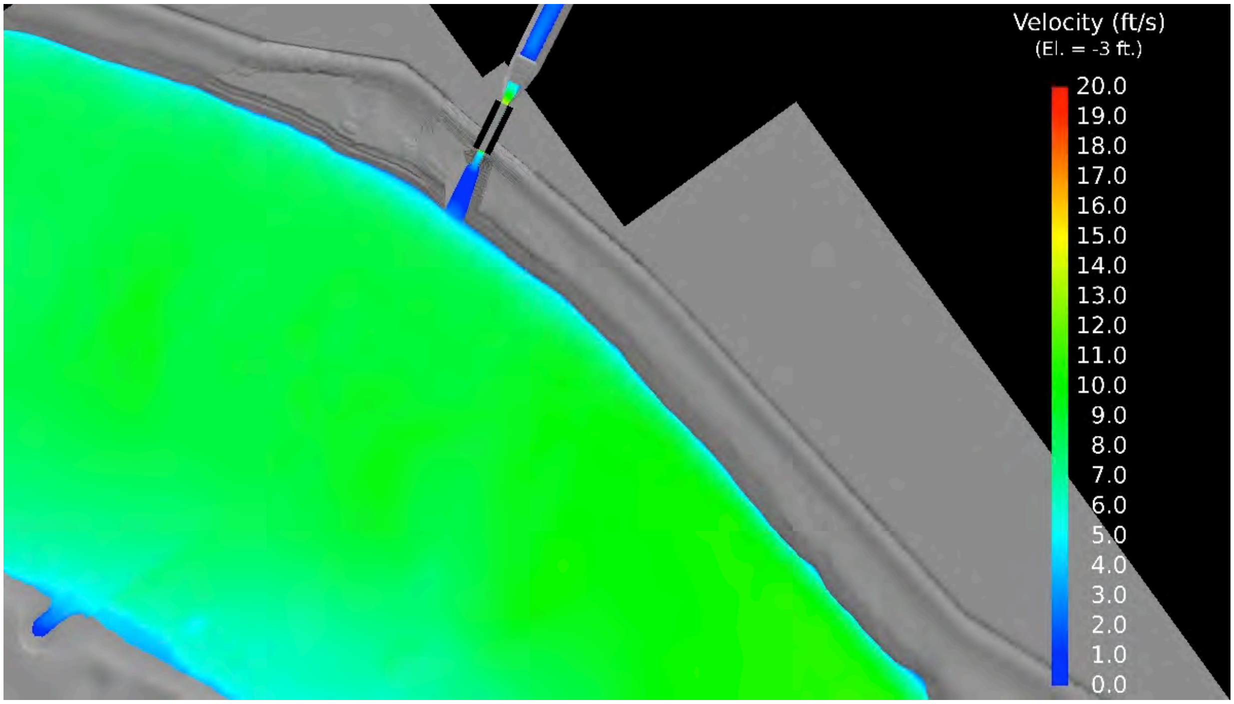



**Figure 22 – Run #6 model results. Velocity contours in the Mississippi River with the proposed diversion. Slice plane cut at elevation = -3.0 feet.**

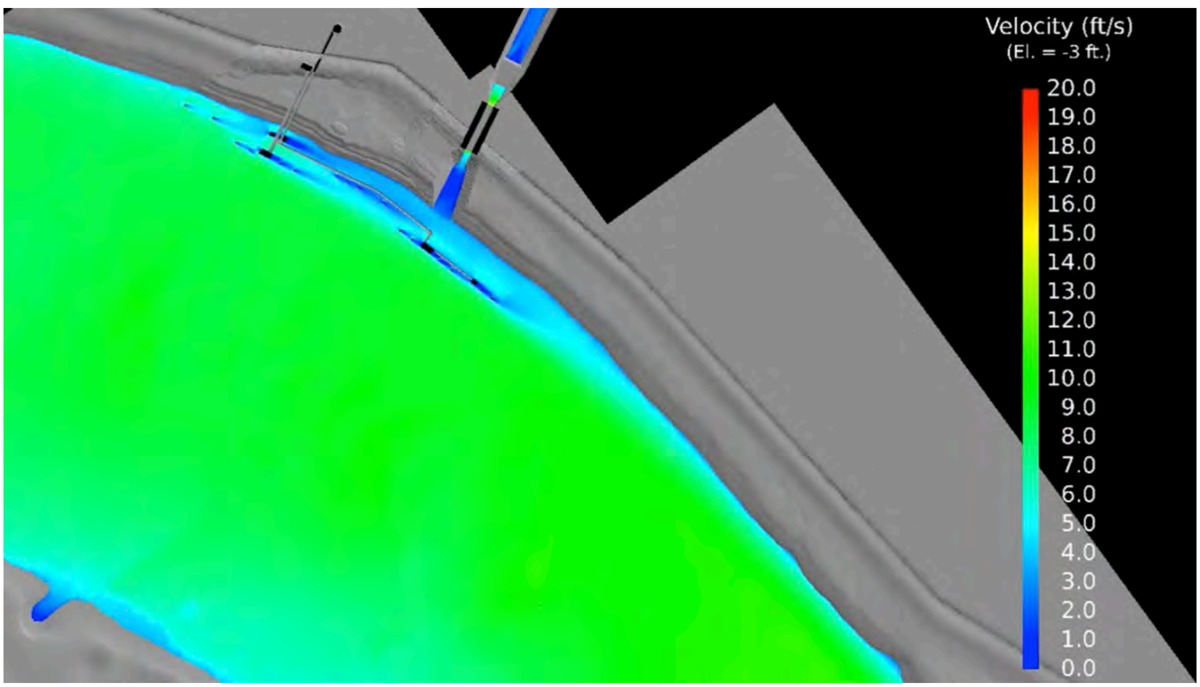



**Figure 23 – Run #7 model results. Velocity contours in the Mississippi River with the proposed diversion and dock facility. Slice plane cut at elevation = -3.0 feet.**

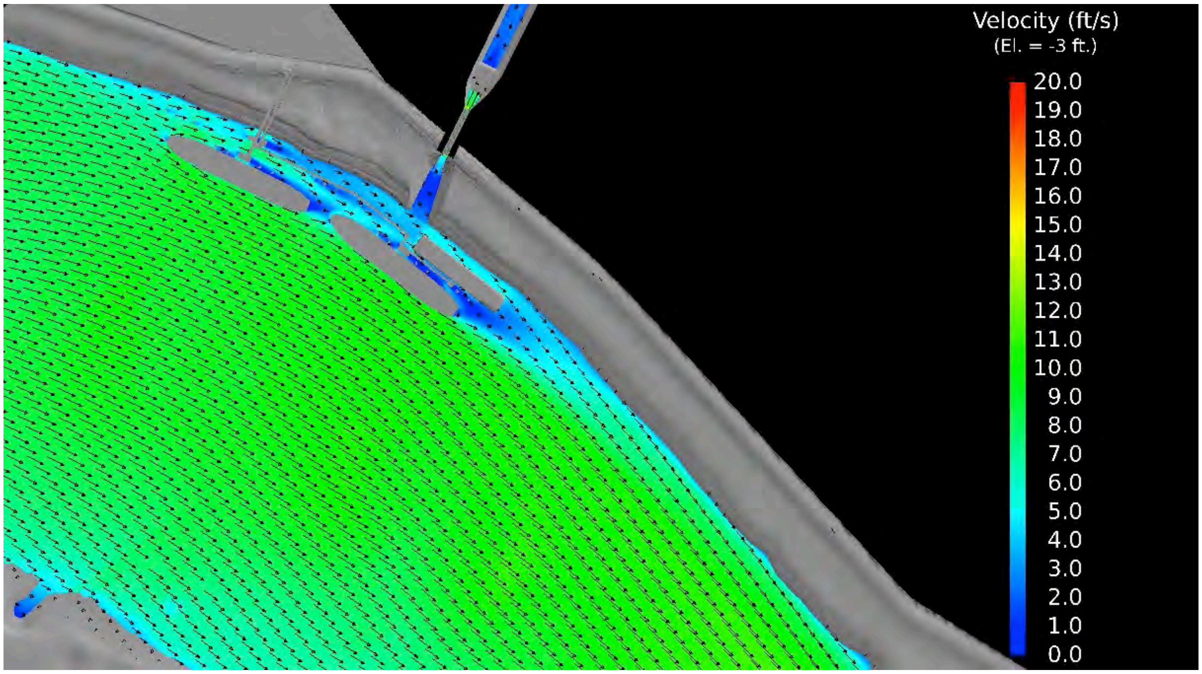



**Figure 24 – Run #8 model results. Velocity contours (not all vectors are shown) in the Mississippi River with the proposed diversion and dock facility including vessels. Slice plane cut at elevation = -3.0 feet.**

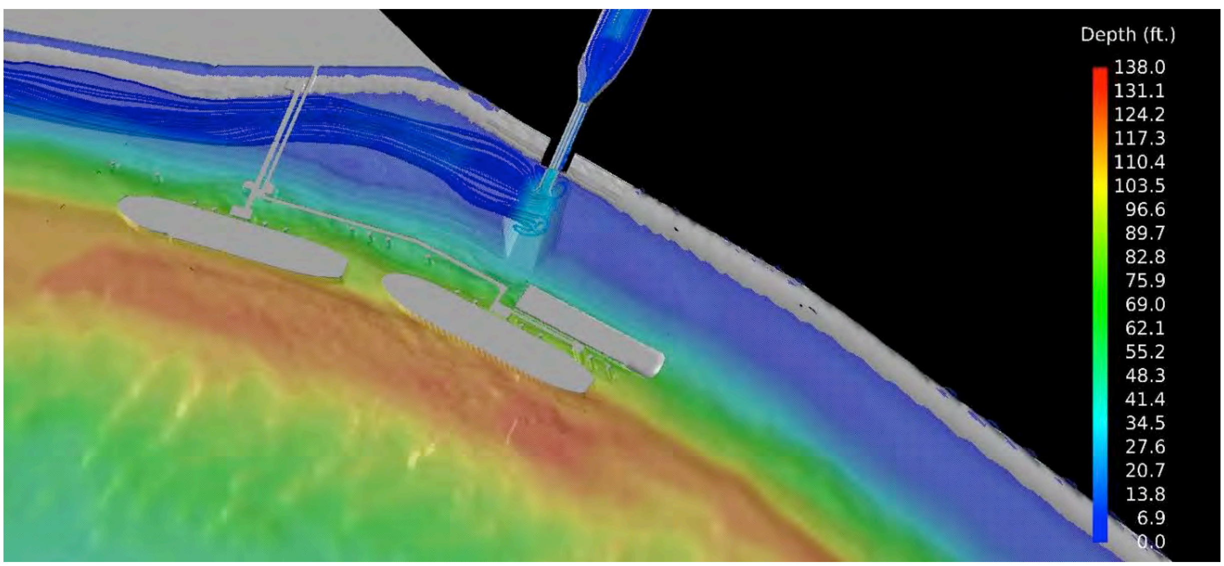



**Figure 25 – Run #8 model results. Streamlines colored by depth (back calculated from the diversion structure) with the proposed diversion and dock facility including vessels. Background water surface (50% transparent) colored by depth. Diverted flow follows the shoreline and near the water surface.**

## 5.3 POLLUTANT TRANSPORT

Results, appearing on Figures 26 through 29, show the movement of pollutant (Bitumen in this study) away from the dock facility in the event of a spill. Dilution Factor *(DF)* [6] was used to convey results of the simulations and was computed using the following formula:

$$DF = \frac{Mass\ of\ Bitumen + Mass\ of\ Water}{Mass\ of\ Bitumen}$$

$$DF = \frac{\rho_B C + \rho_W (1 - C)}{\rho_B C}$$

Where: $DF$ = Dilution Factor
$C$ = Bitumen Concentration
$\rho_B$ = Density of Bitumen
$\rho_W$ = Density of Water

Figures 26 and 27 show dilution factors at elevation 12.0 feet NAVD88 (near water surface) 10 minutes after and 30 minutes after a spill for Run #9 (Pollutant Transport during Low-Flow Condition). Even though the density of Bitumen is greater than that of water, the Bitumen is dispersed into the Mississippi River flow and as a result it sinks slowly and is transported downstream of the spill location. Figures A-15 through A-24 (Appendix A) show similar results at different elevations in the water column.

Figures 28 and 29 show dilution factors at elevation 22.0 feet NAVD88 (near the water surface) 1 hour after and 3.87 hours after a spill for Run #10 (Pollutant Transport during High-Flow Condition). As shown, the distribution of Bitumen near the diversion entrance remains the same after about 1 hour. The flow of Bitumen into the Mississippi River stops after 3.87 hours in this scenario.

According to these results and for all scenarios simulated herein, Bitumen does not enter the freshwater intake located at an elevation of -4.0 feet NAVD88. Consistent with computed flow patterns, the Bitumen is carried downstream from the facility, away from the diversion.

Additional graphics showing the concentration of Bitumen at different elevations in the water column for Run #9 and #10 appear in Appendix A (Figures A-15 through A-24).

[6] Dilution factor is a parameter that varies inversely with concentration and is often used to show compliance with mixing zone criteria (higher dilutions are associated with lower pollutant concentrations).

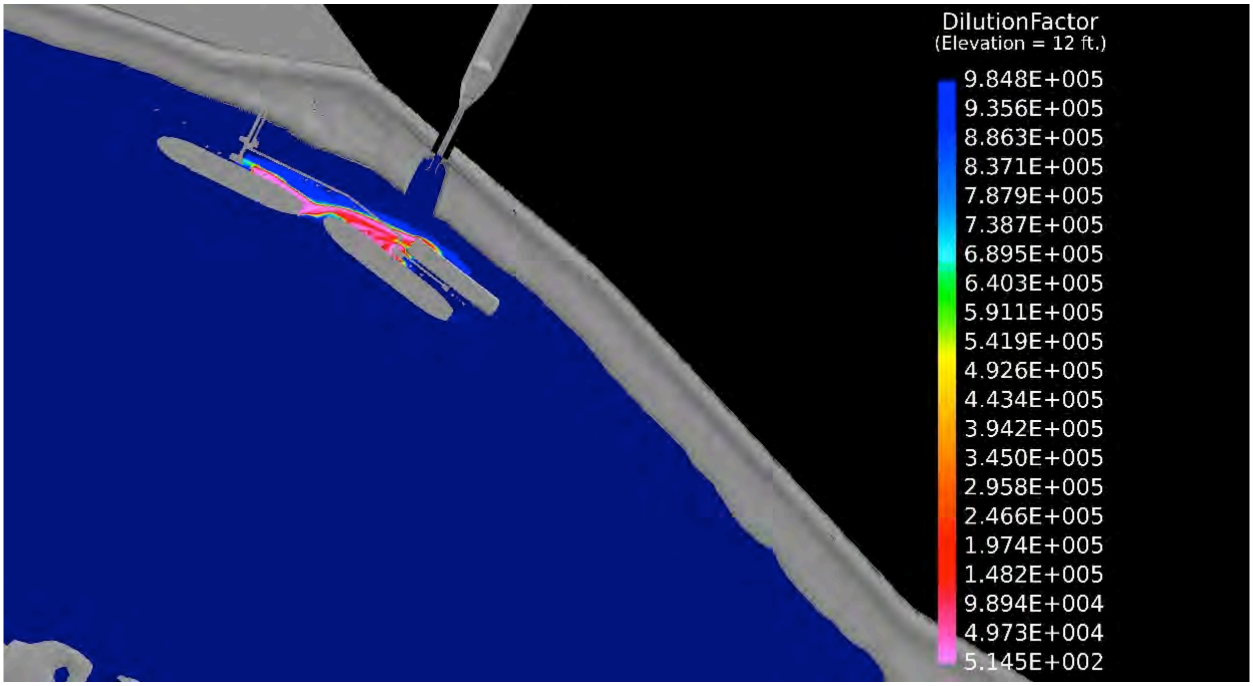



**Figure 26 – Run #9 model results. Dilution factors for Bitumen at elevation 12.0 feet (near water surface) 10 minutes after spill.**

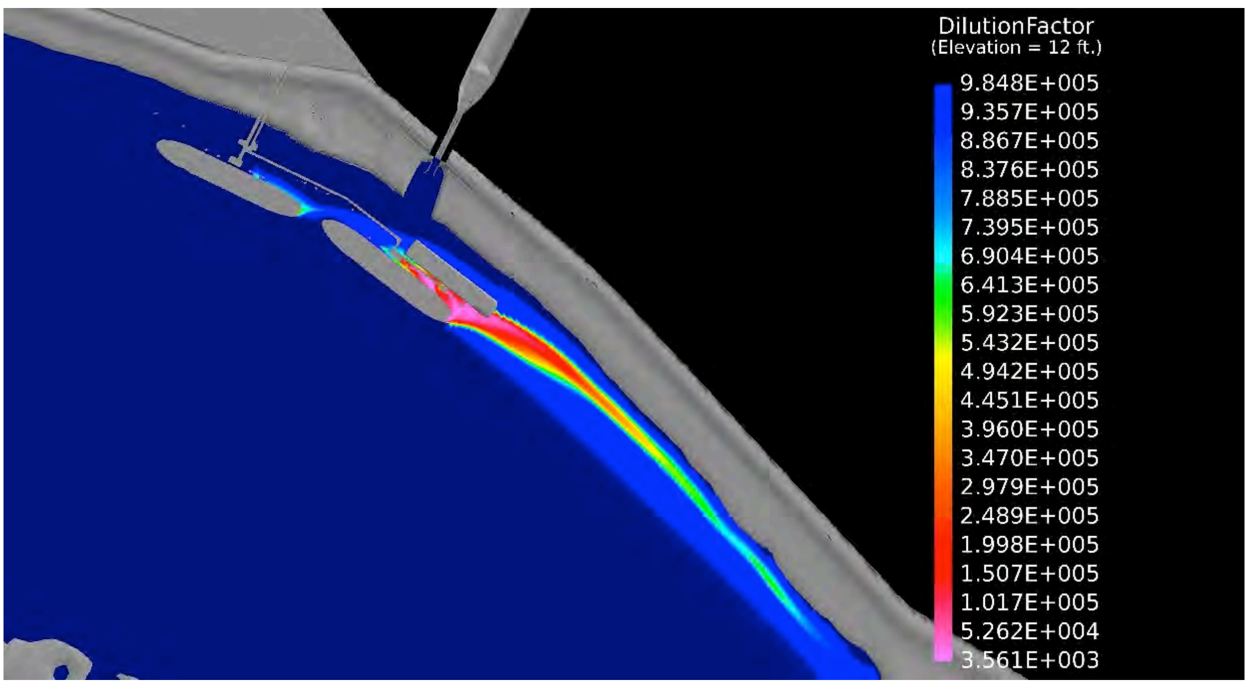



**Figure 27 – Run #9 model results. Dilution factors for Bitumen at elevation 12.0 feet (near water surface) 30 minutes after spill.**

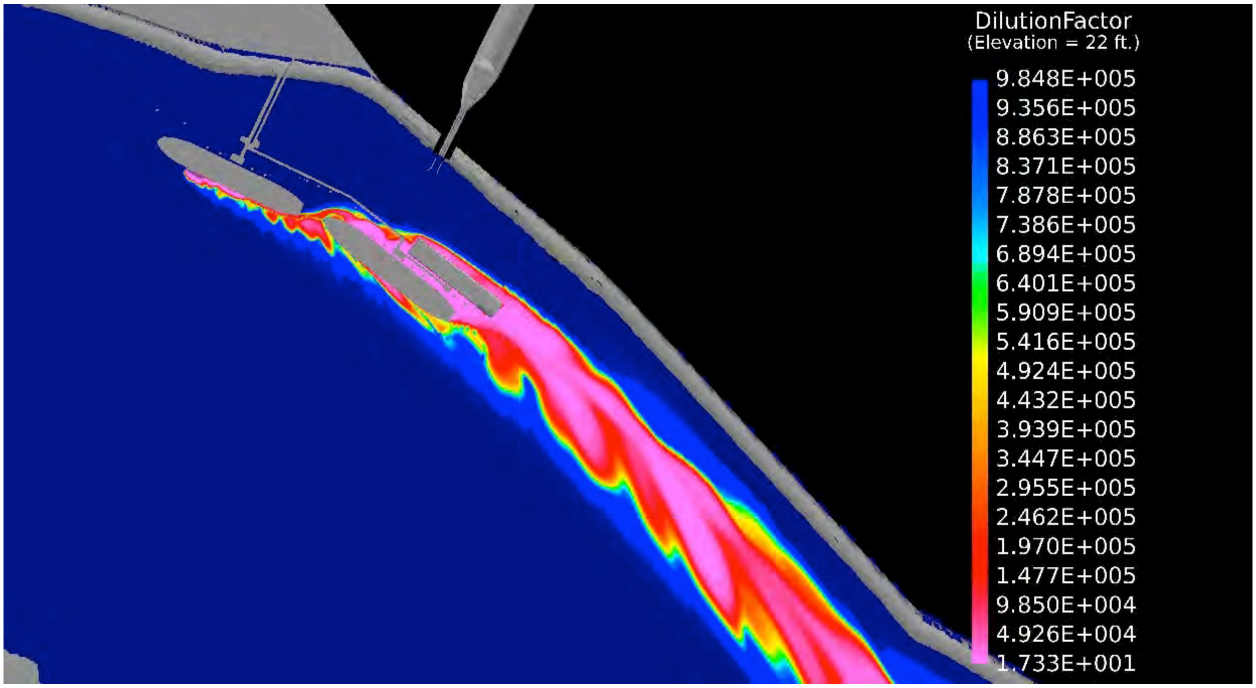



**Figure 28 – Run #10 model results. Dilution factors for Bitumen at elevation 22.0 feet (near water surface) 1 hour after spill.**

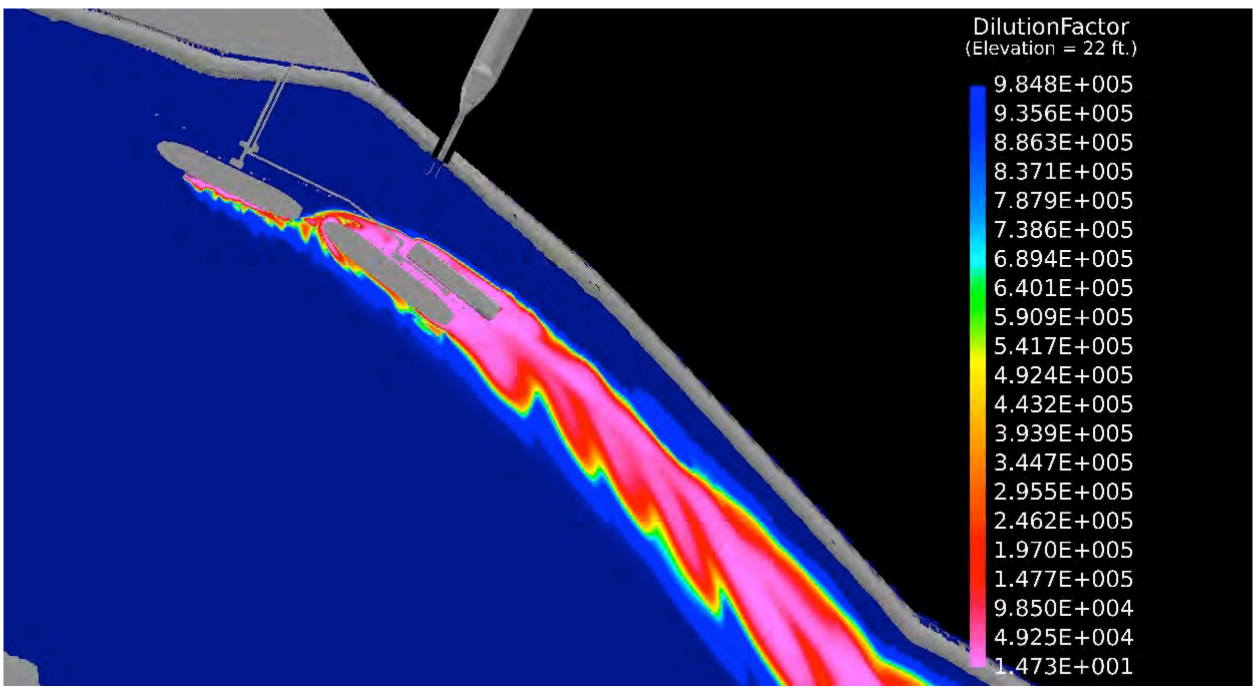



**Figure 29 – Run #10 model results. Dilution factors for Bitumen at elevation 22.0 feet (near water surface) at the end of the spill event (3.87 hours, constant discharge).**

# 6 Conclusions

This report provides a summary of modeling performed to assess the potential impact of a docking facility on flows in the vicinity of a proposed freshwater diversion at Mississippi RM 144.2.

As stated in the introduction, the analysis sought to answer two specific questions:

1. Does the proposed dock facility impact the flow of water into the diversion?
2. If a spill occurred at the proposed dock facility, would spilled material enter the freshwater diversion and make its way to the receiving marsh area?

With respect to Question No. 1, the proposed dock facility has little impact on the flow of water into the diversion. As shown for high-flow and low-flow conditions (with and without the dock facility and vessels), flow approaches the diversion entrance along the shoreline of the Mississippi River (Figures 17, 19, 21, and 25). The results of the analysis also indicate that the amount of flow captured by the diversion will not change as a result of the proposed project (i.e., for a given set of water levels in the Mississippi River and at the downstream end of the diversion channel – the amount of flow bypassed is calculated to be same with or without project).

With respect to Question No. 2, Bitumen released into the river as a result of a spill or accident does not enter the diversion. In all cases, the Bitumen mixes with river water, descends toward the bottom of the river, and is transported downstream of the diversion. This conclusion is supported by the fact that the dock facility and vessels are not calculated to have any effect on the trajectory of flow entering the diversion or the capacity of the structure to divert flow.

# References

Flow Science. 2010. ***FLOW-3D*** – Excellence in flow modeling software. User Manual 9.4 Volumes 1 and 2. Santa Fe, NM. http://www.flow3d.com/

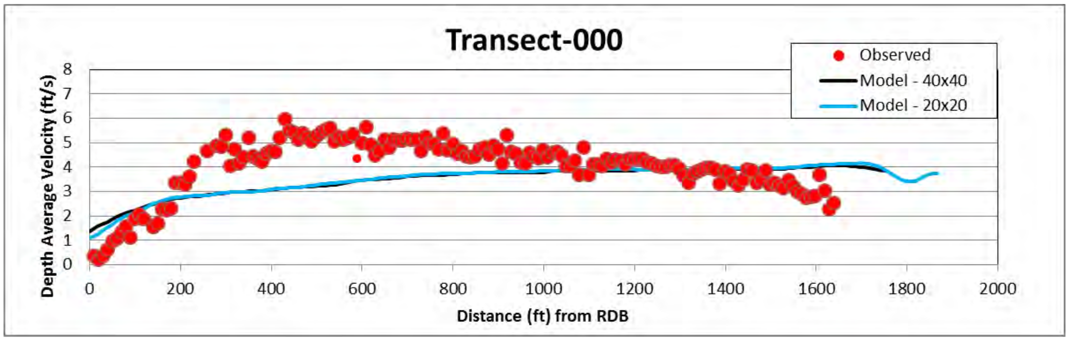
Meselhe, E.A., J. Richardson, H. Roberts, and R. Lagumbay. 2012. "RAM Terminal Facility CFD Modeling Report".

Meselhe, E.A., J. Richardson, H. Roberts, R. Lagumbay, and Y. Ma. 2013. "Hydrodynamic and Sediment Transport Modeling using FLOW-3D for Siting and Optimization of the LCA Medium Diversion at White Ditch Report".

Rhinoceros. 2014. Modeling Tools for Designers V4, Seattle, WA. https://www.rhino3d.com/

# Appendices

## APPENDIX A – ADDITIONAL GRAPHICS AND FIGURES



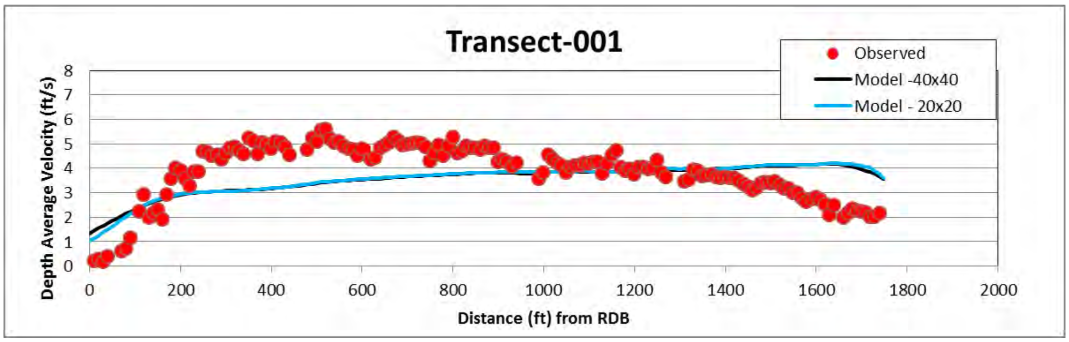




**Figure A-1. Comparison between CFD model and field data at transects #000 and #001. Red dots show the field data and model results are given for two different meshes – 40x40 feet in size and 20x20 feet in size. The results with either mesh are similar.**

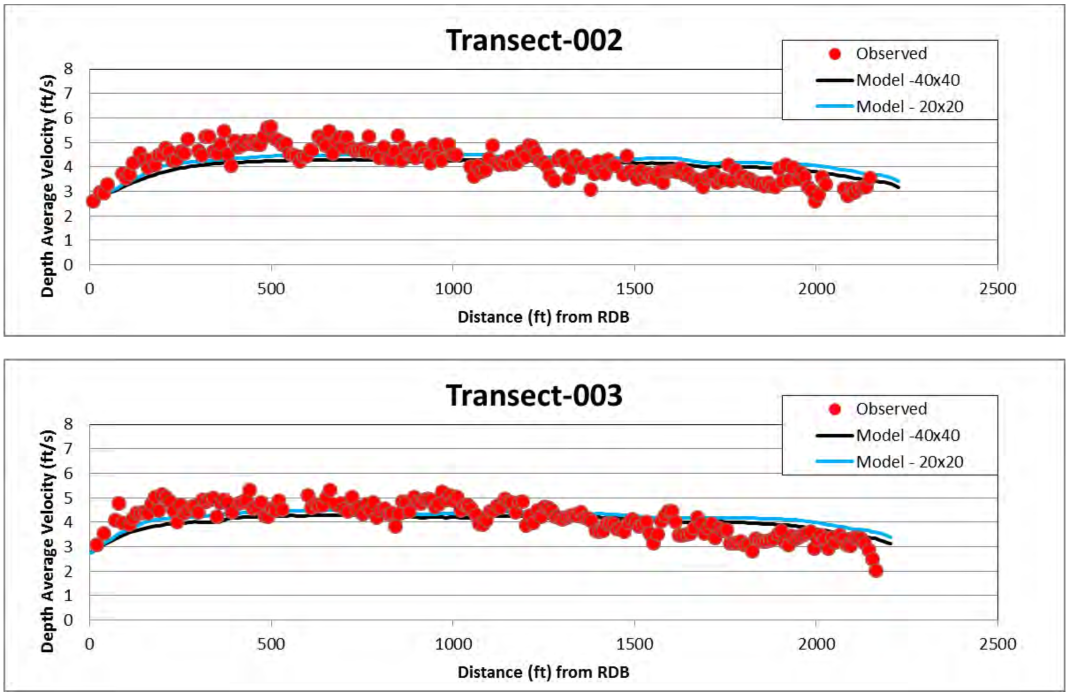



**Figure A-2. Comparison between CFD model and field data at transects #002 and #003. Red dots show the field data and model results are given for two different meshes – 40x40 feet in size and 20x20 feet in size. The results with either mesh are similar.**

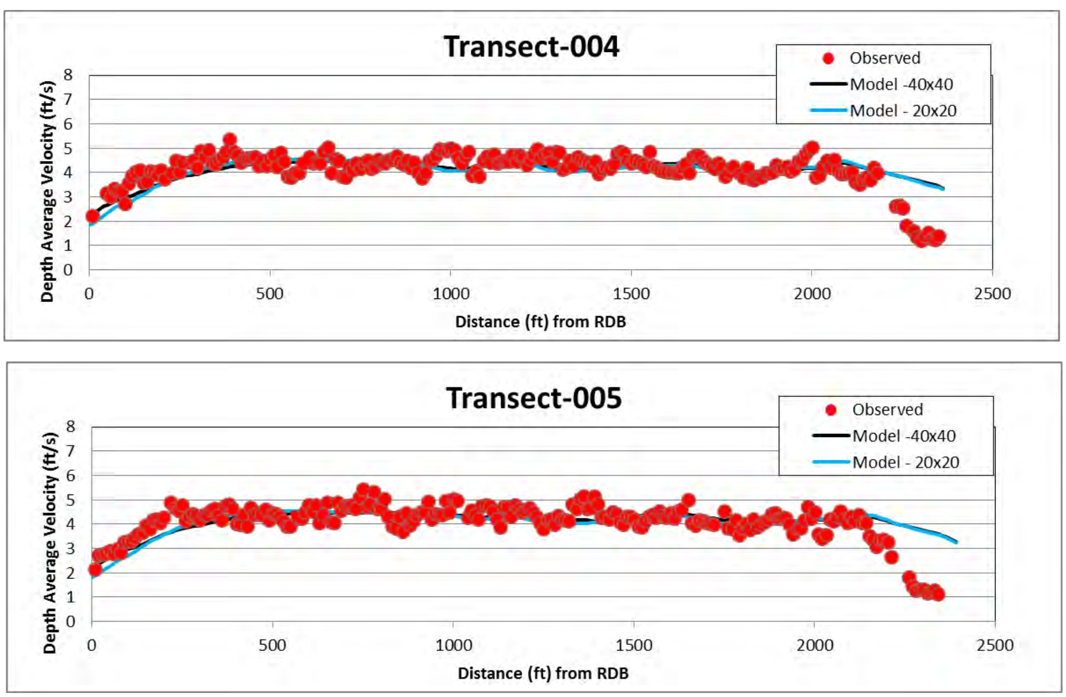
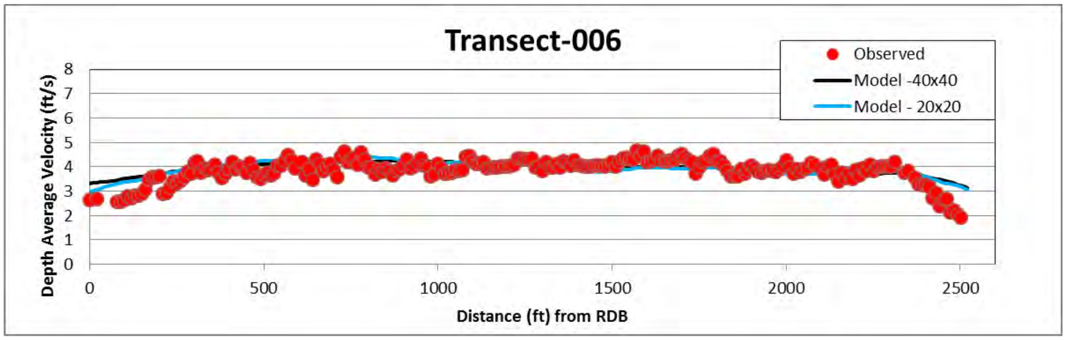



**Figure A-3. Comparison between CFD model and field data at transects #004 and #005. Red dots show the field data and model results are given for two different meshes – 40x40 feet in size and 20x20 feet in size. The results with either mesh are similar.**



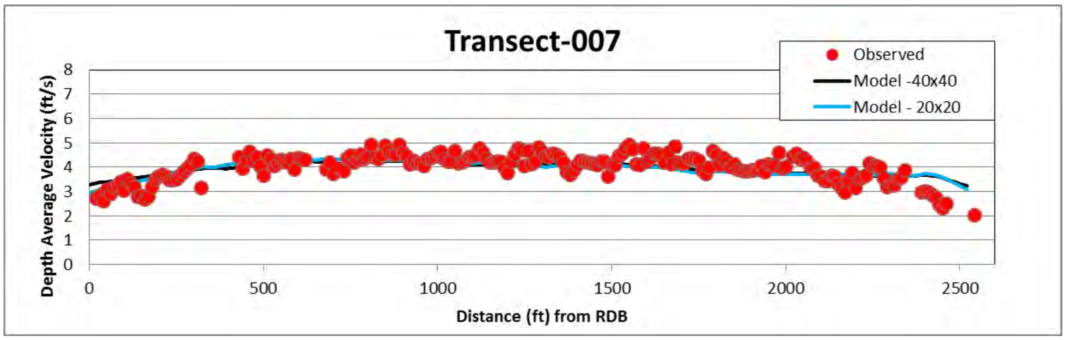
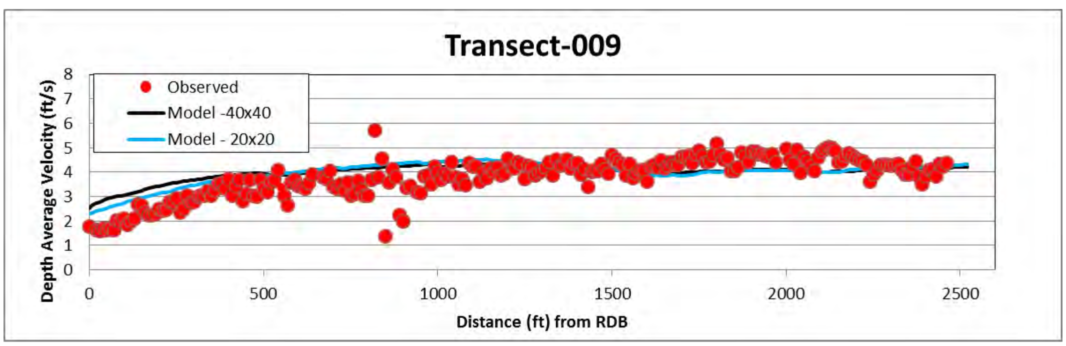
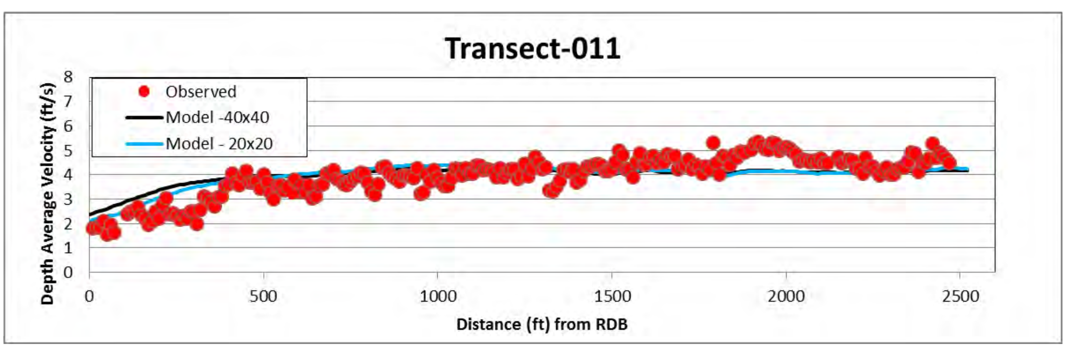
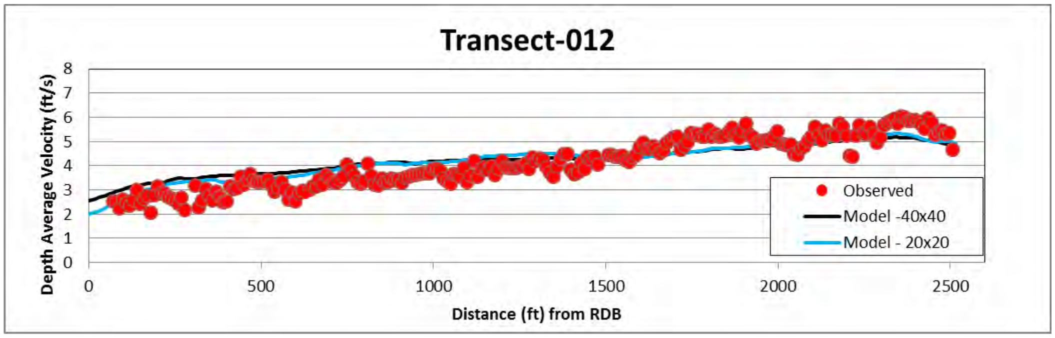




**Figure A-4. Comparison between CFD model and field data at transects #006 and #007. Red dots show the field data and model results are given for two different meshes – 40x40 feet in size and 20x20 feet in size. The results with either mesh are similar.**




**FigureA-5. Comparison between CFD model and field data at transects #009 and #0011. Red dots show the field data and model results are given for two different meshes – 40x40 feet in size and 20x20 feet in size. The results with either mesh are similar.**



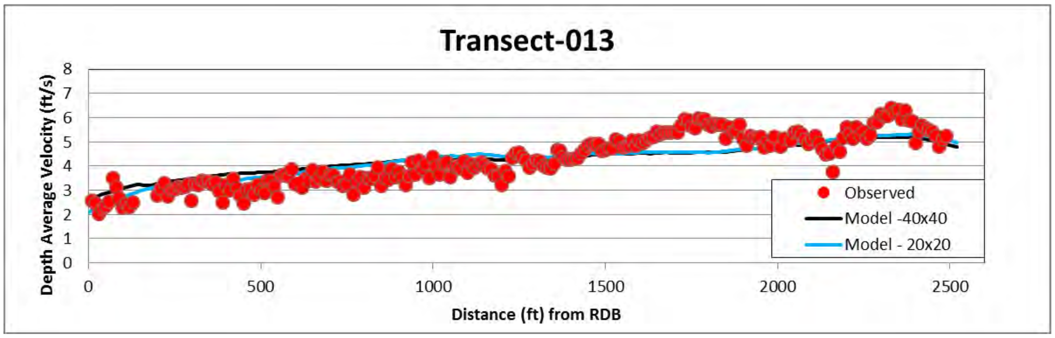




**Figure A-6. Comparison between CFD model and field data at transects #012 and #013. Red dots show the field data and model results are given for two different meshes – 40x40 feet in size and 20x20 feet in size. The results with either mesh are similar.**

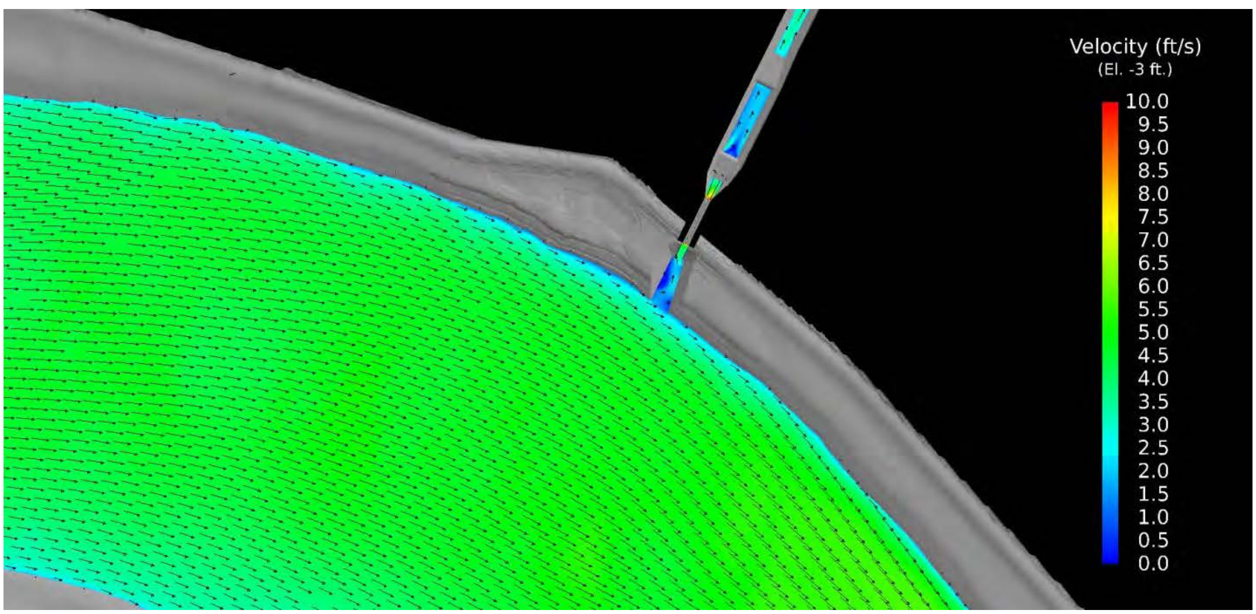



**Figure A-7. Run #2 model results. Velocity contours (not all vectors are shown) in the Mississippi River with the proposed diversion. Slice plane cut at elevation = -3.0 feet.**

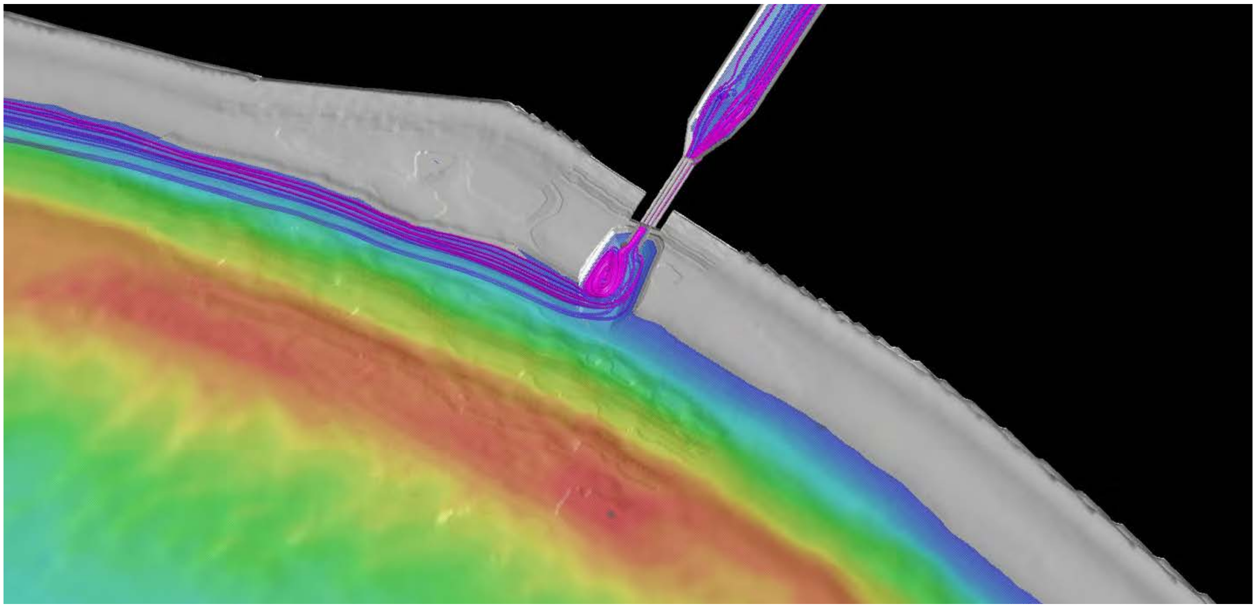


**Figure A-8. Run #2 model results. Streamlines (back calculated from the diversion structure) with the proposed diversion. Background water surface (50% transparent) colored by depth. Diverted flow follows the shoreline near the water surface.**

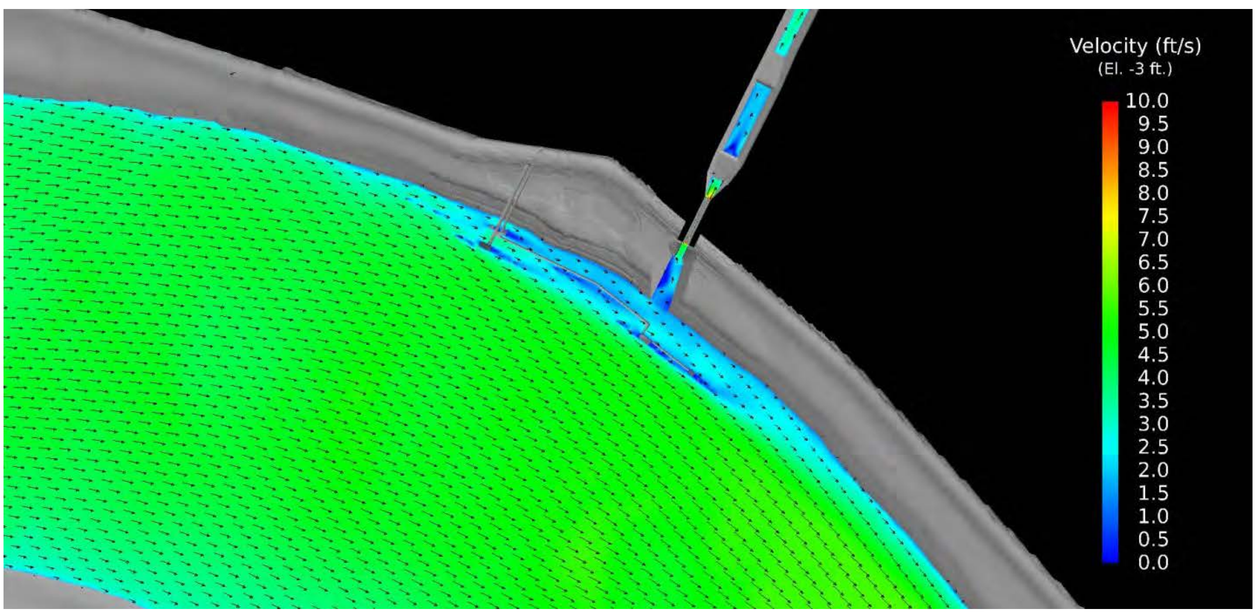



**Figure A-9. Run #3 model results. Velocity contours (not all vectors are shown) in the Mississippi River with the proposed diversion and dock facility. Slice plane cut at elevation = -3.0 feet.**

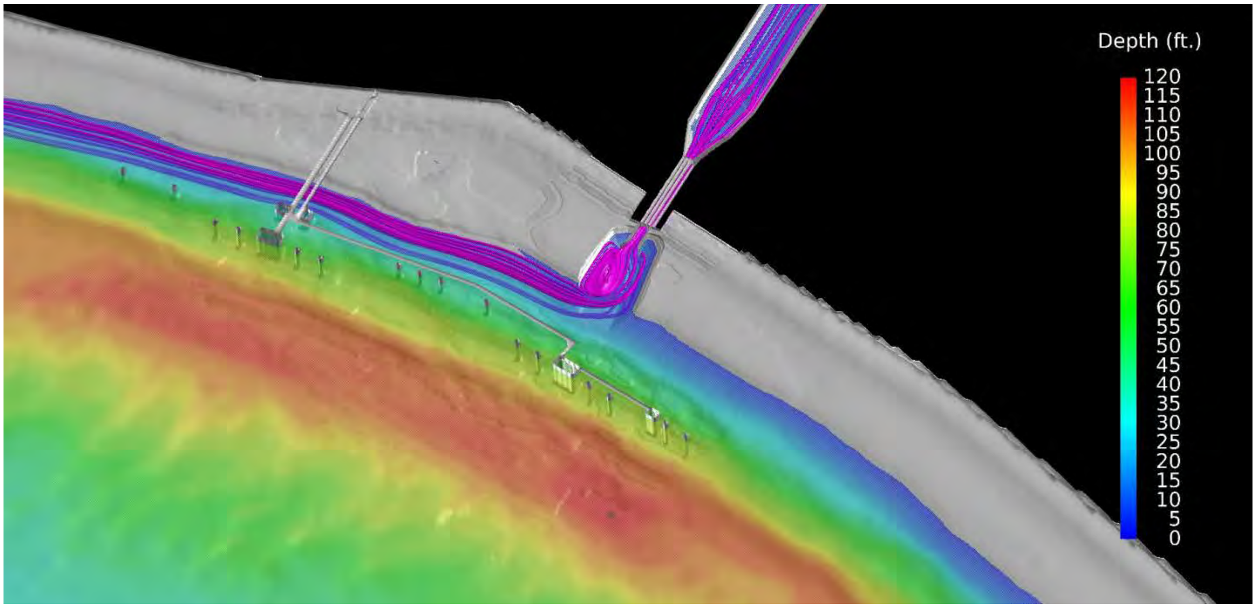



**Figure A-10. Run #3 model results. Streamlines (back calculated from the diversion structure) with the proposed diversion and dock facility. Background water surface (50% transparent) colored by depth. Diverted flow follows the shoreline near the water surface.**

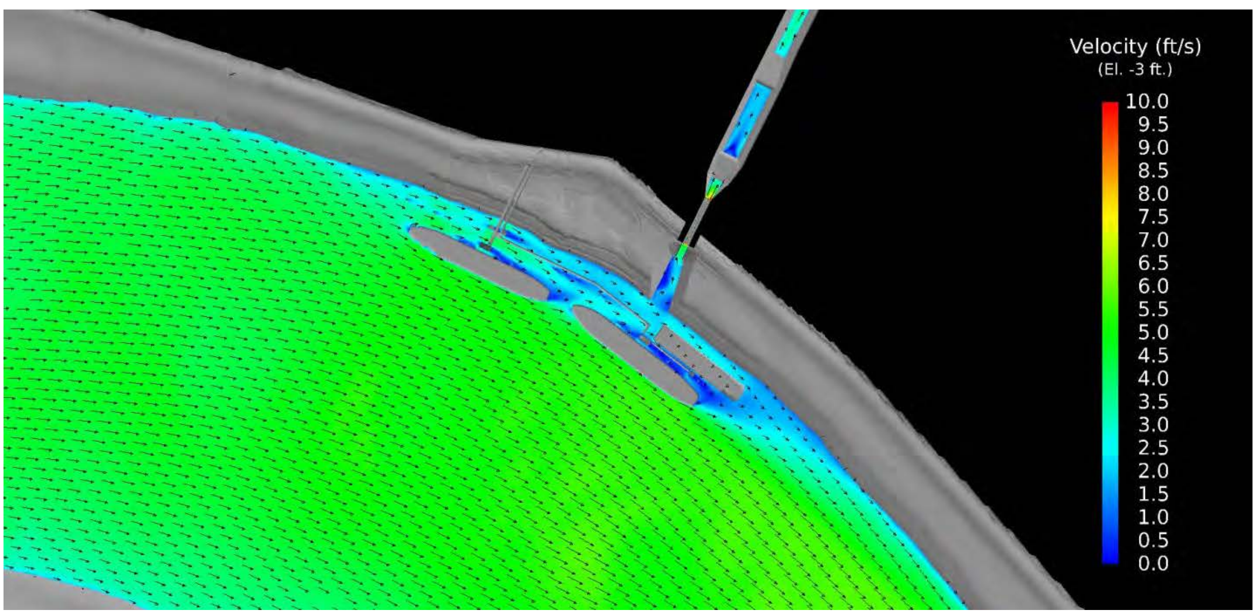



**Figure A-11. Run #4 model results. Velocity contours (not all vectors are shown) in the Mississippi River with the proposed diversion and dock facility. Slice plane cut at elevation = -3.0 feet.**

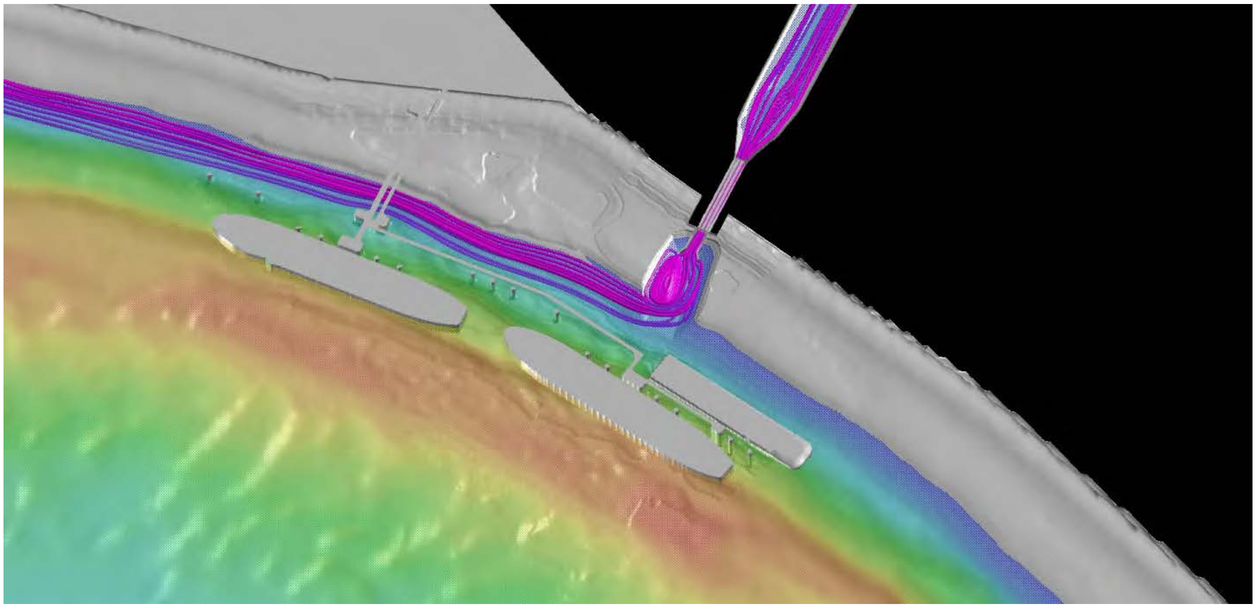


Figure A-12. Run #4 model results. Steamlines (back calculated from the diversion structure) with the proposed diversion and dock facility including vessels. Background water surface (50% transparent) colored by depth. Diverted flow follows the shoreline near the water surface.

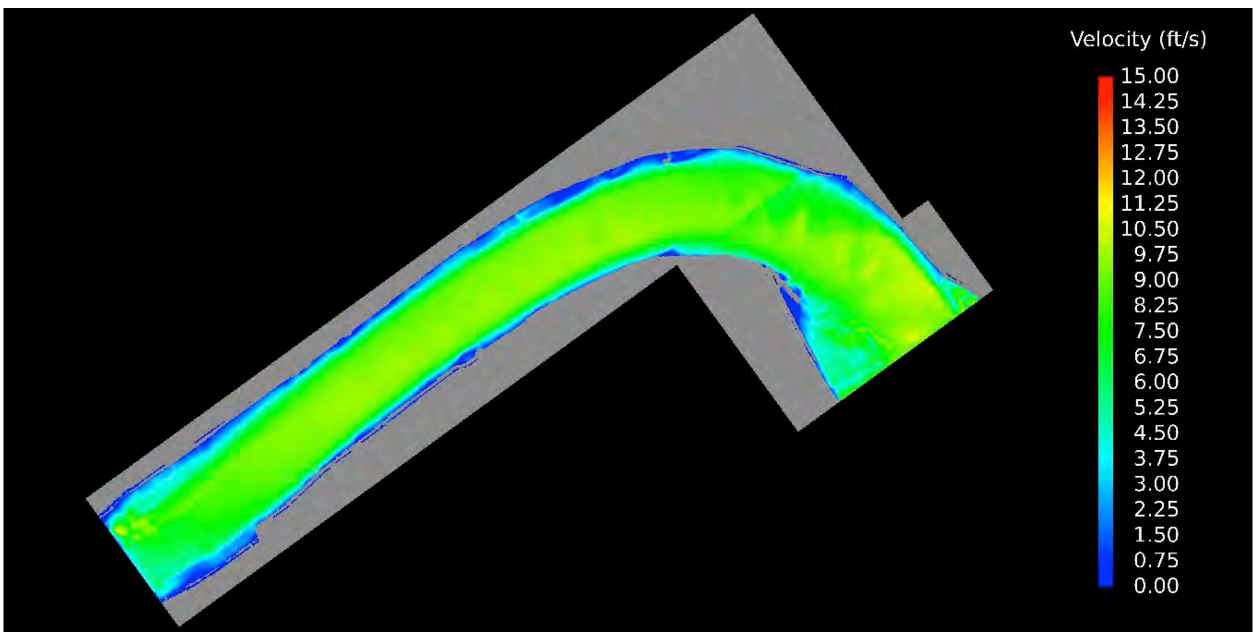



Figure A-13. Run #5 model results. Water surface colored by velocity in the Mississippi River during high-flow condition.

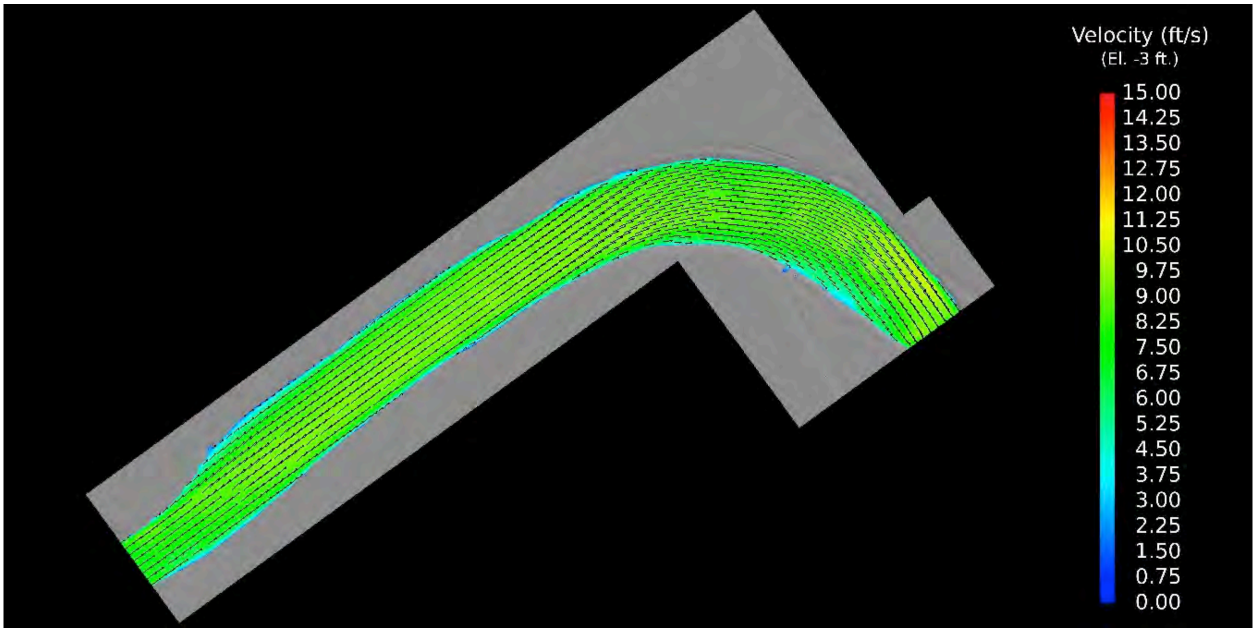



**Figure A-14. Run #5 model results. Velocity contours (not all vectors are shown) in the Mississippi River during high-flow condition. Slice plane cut at elevation = -3.0 feet.**

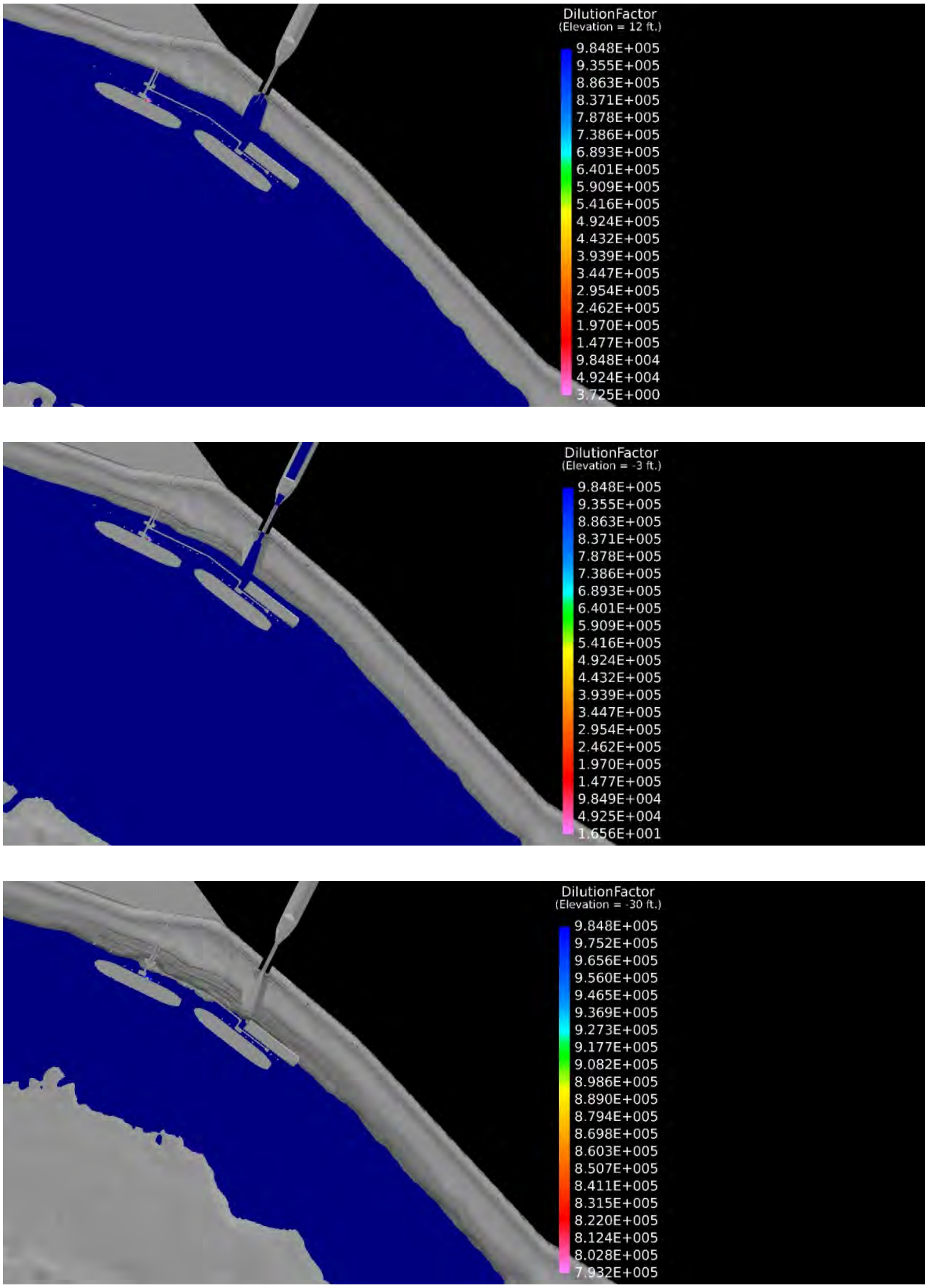



**Figure A-15. Run #9 model results. Dilution factors at elevations 12 feet (top), -3 feet (middle), and -30 feet (bottom) 35 seconds after Bitumen spill.**

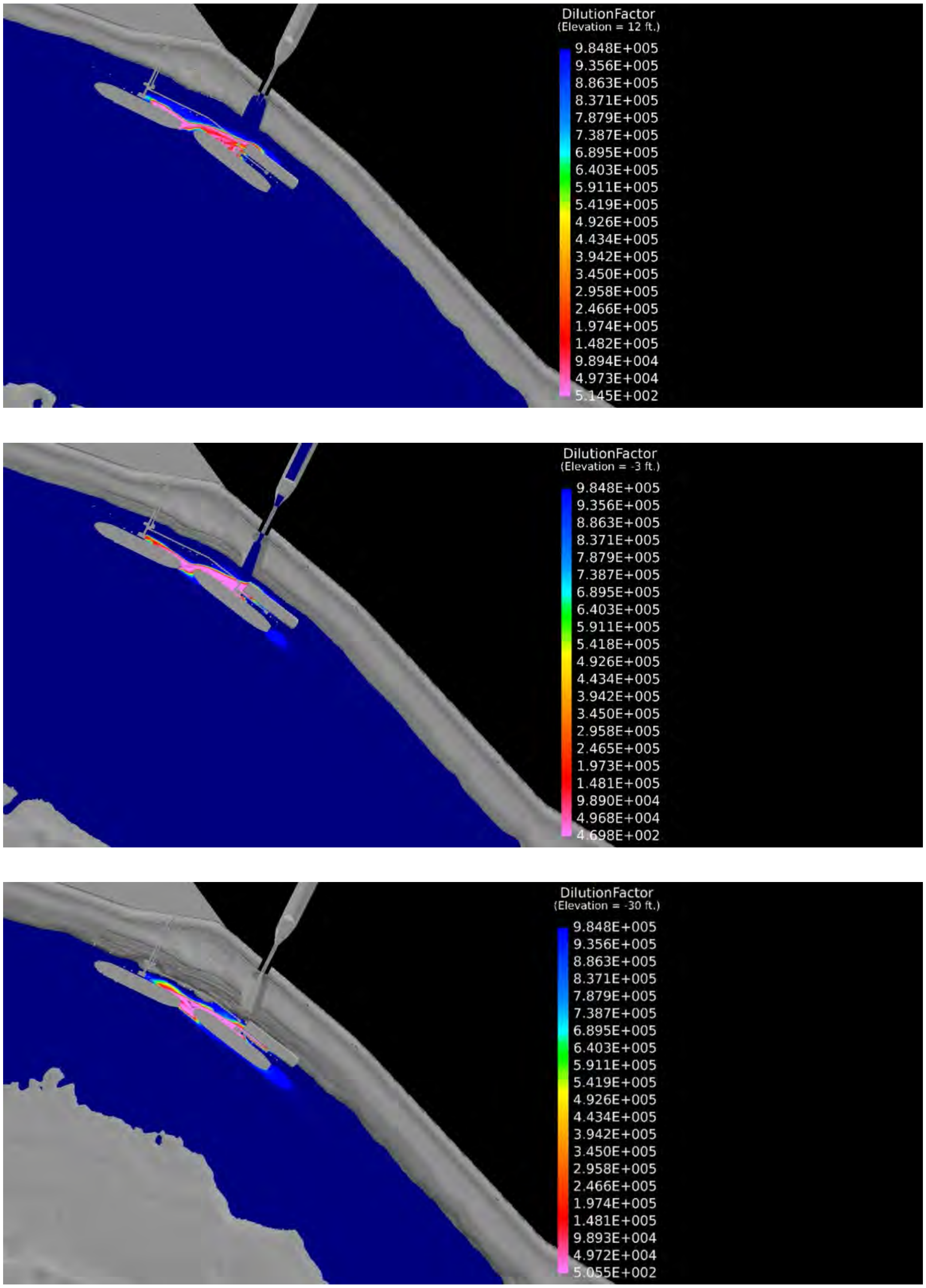



**Figure A-16. Run #9 model results. Dilution factors at elevations 12 feet (top), -3 feet (middle), and -30 feet (bottom) 10 minutes after Bitumen spill.**

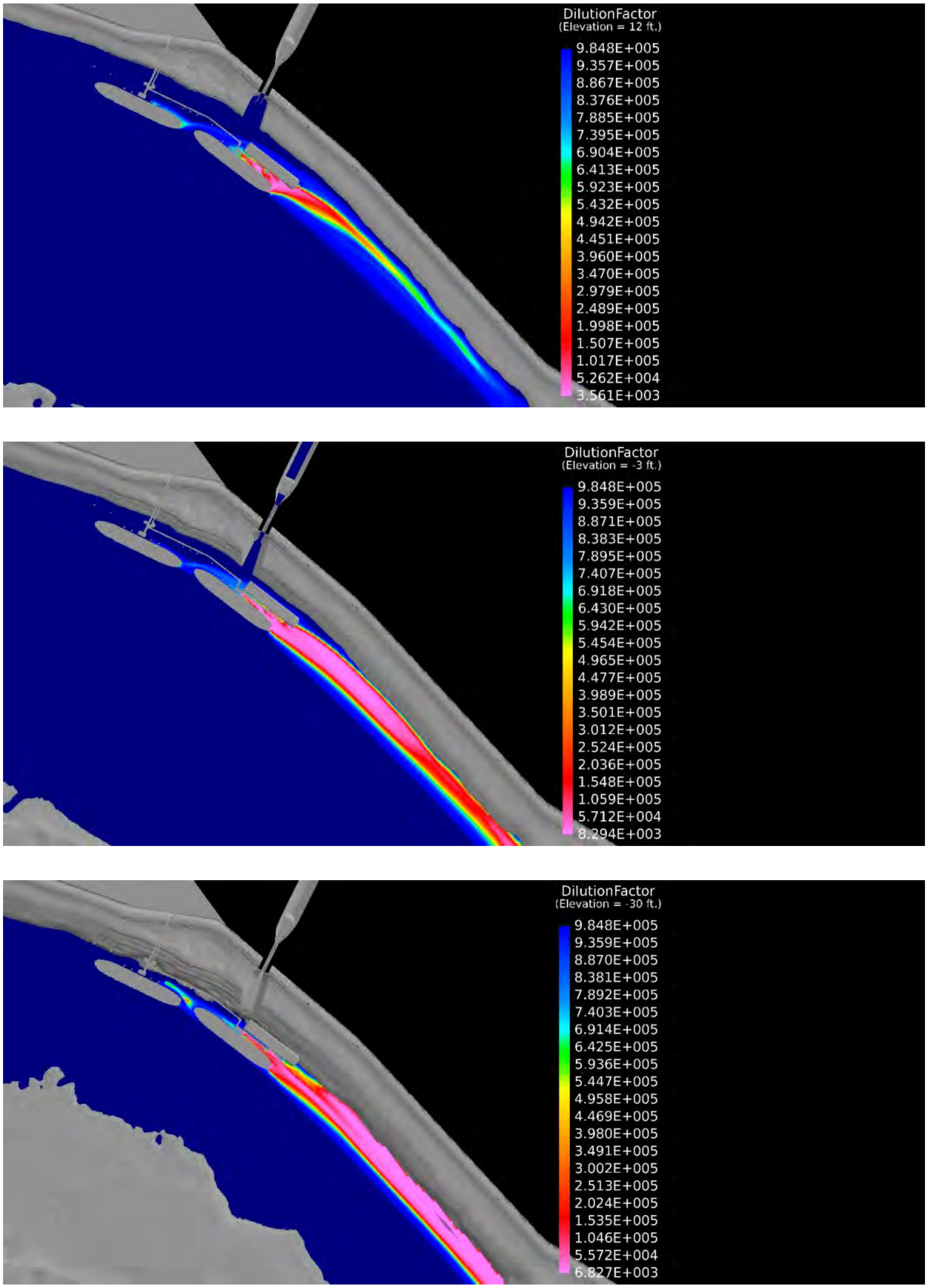



**Figure A-17. Run #9 model results. Dilution factors at elevations 12 feet (top), -3 feet (middle), and -30 feet (bottom) 30 minutes after Bitumen spill.**

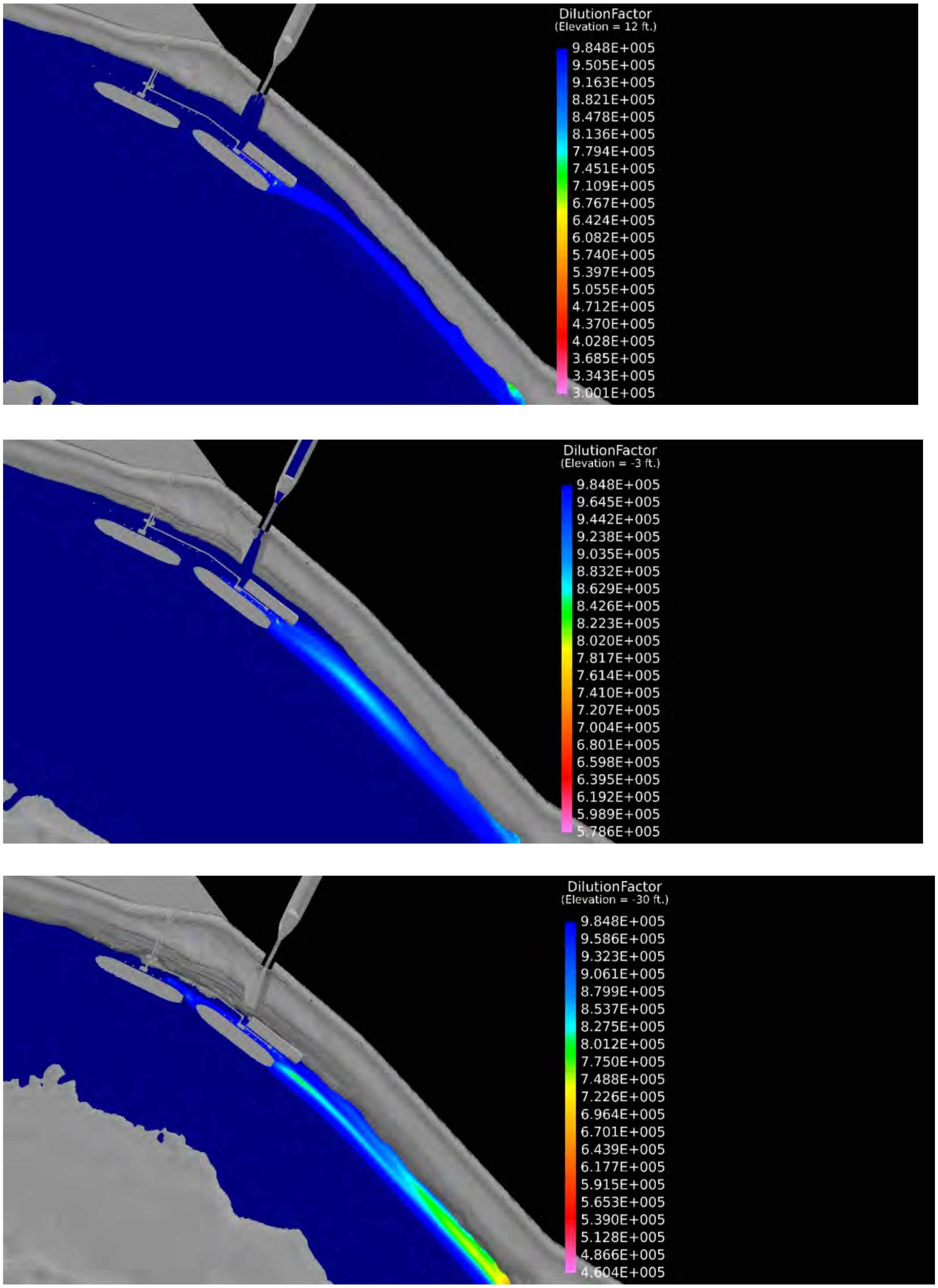



**Figure A-18. Run #9 model results. Dilution factors at elevations 12 feet (top), -3 feet (middle), and -30 feet (bottom) 1 hour after Bitumen spill.**

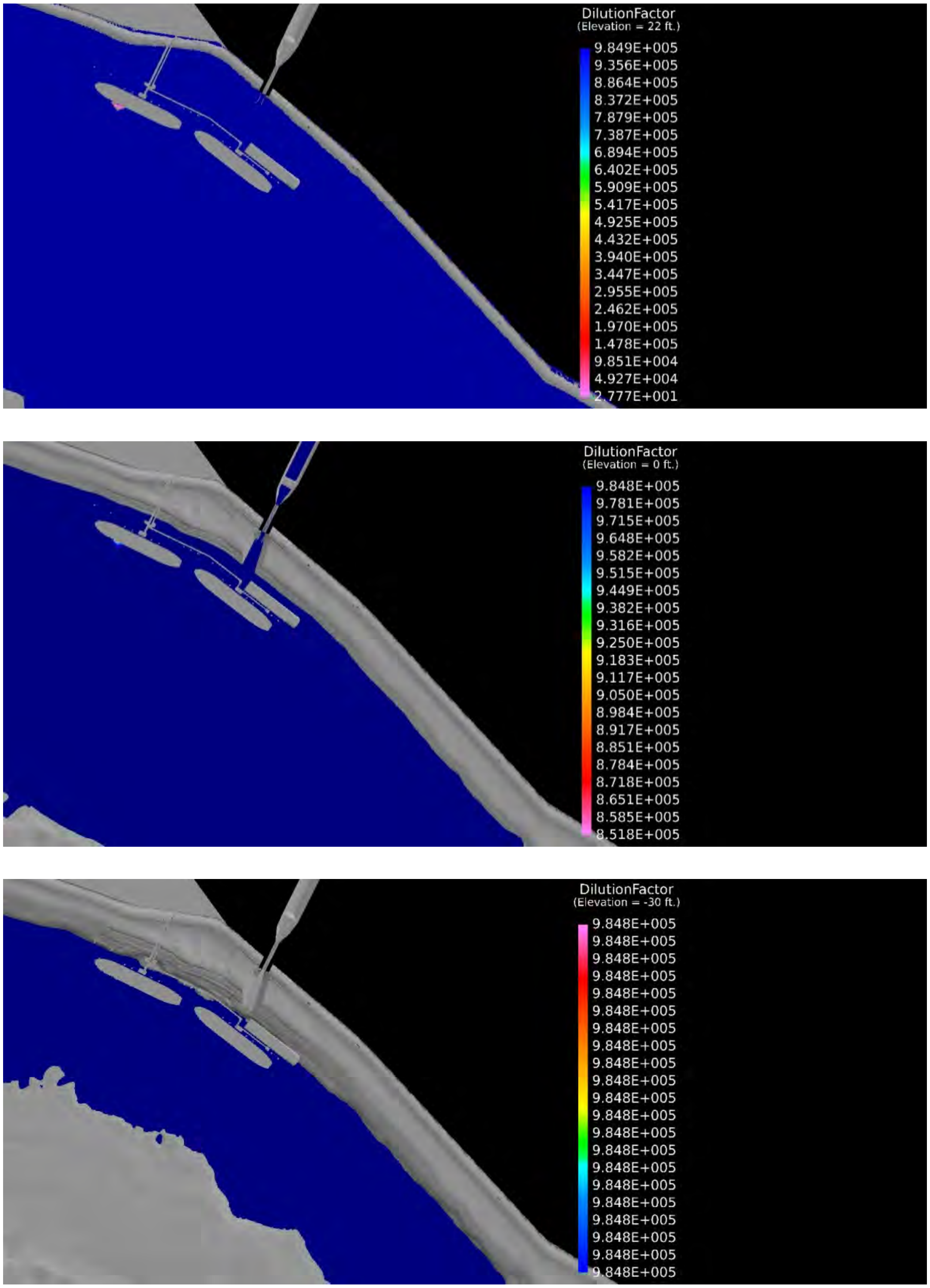



**Figure A-19. Run #10 model results. Dilution factors at elevations 22 feet (top), 0 feet (middle), and -30 feet (bottom) 10 seconds after Bitumen spill.**

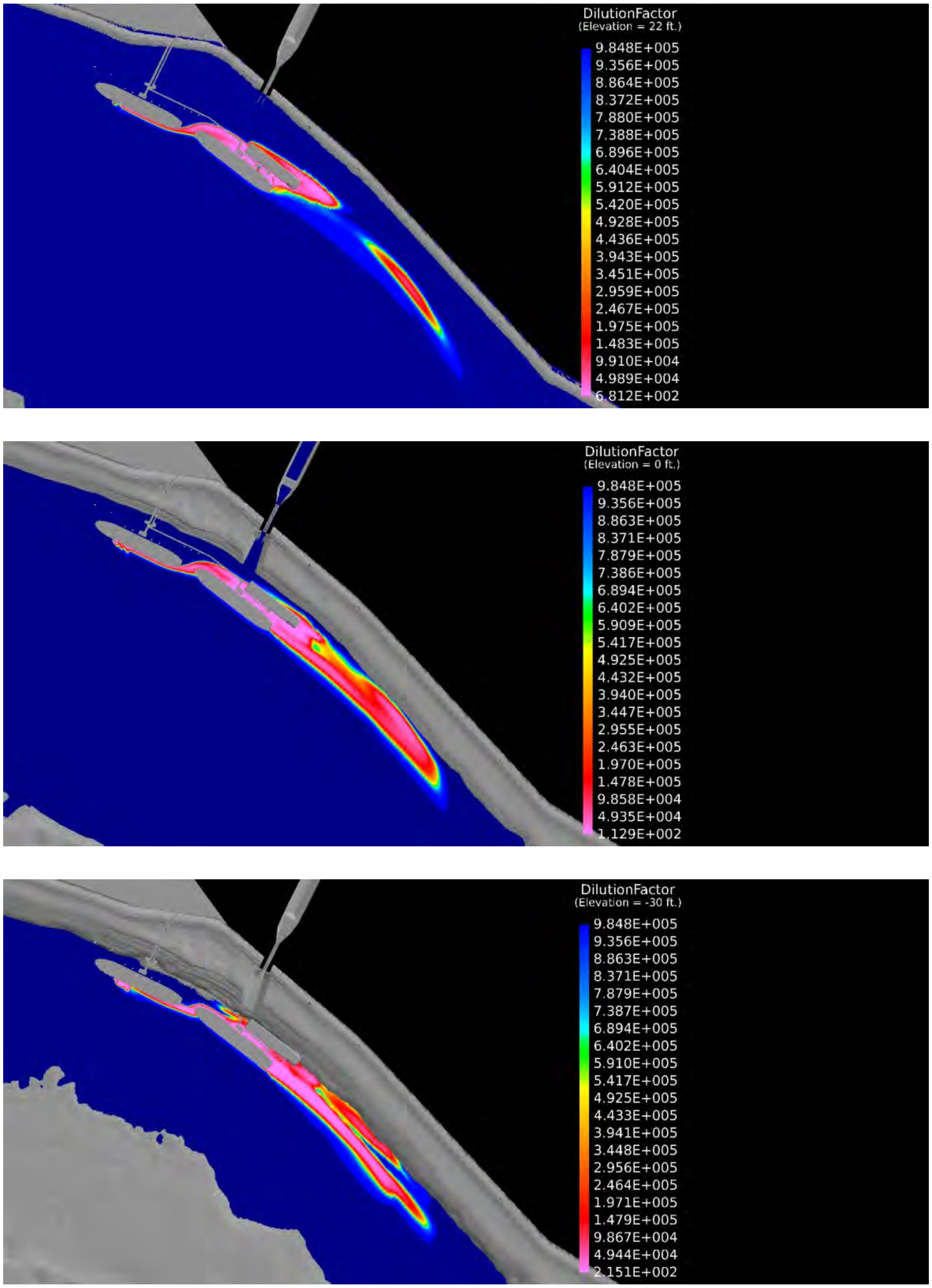



**Figure A-20. Run #10 model results. Dilution factors at elevations 22 feet (top), 0 feet (middle), and -30 feet (bottom) 0.12 hour after Bitumen spill.**

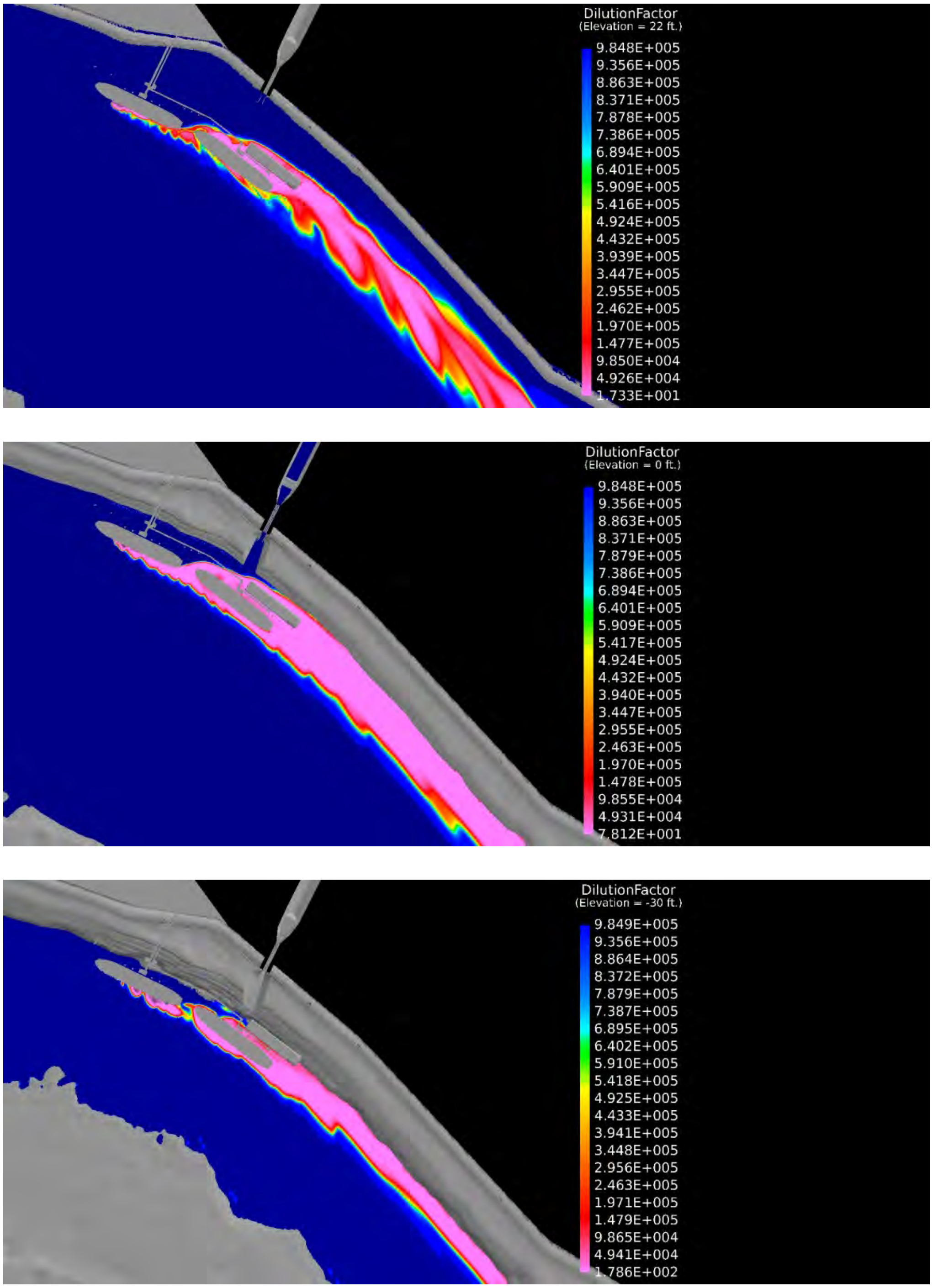



**Figure A-21. Run #10 model results. Dilution factors at elevations 22 feet (top), 0 feet (middle), and -30 feet (bottom) 1 hour after Bitumen spill.**

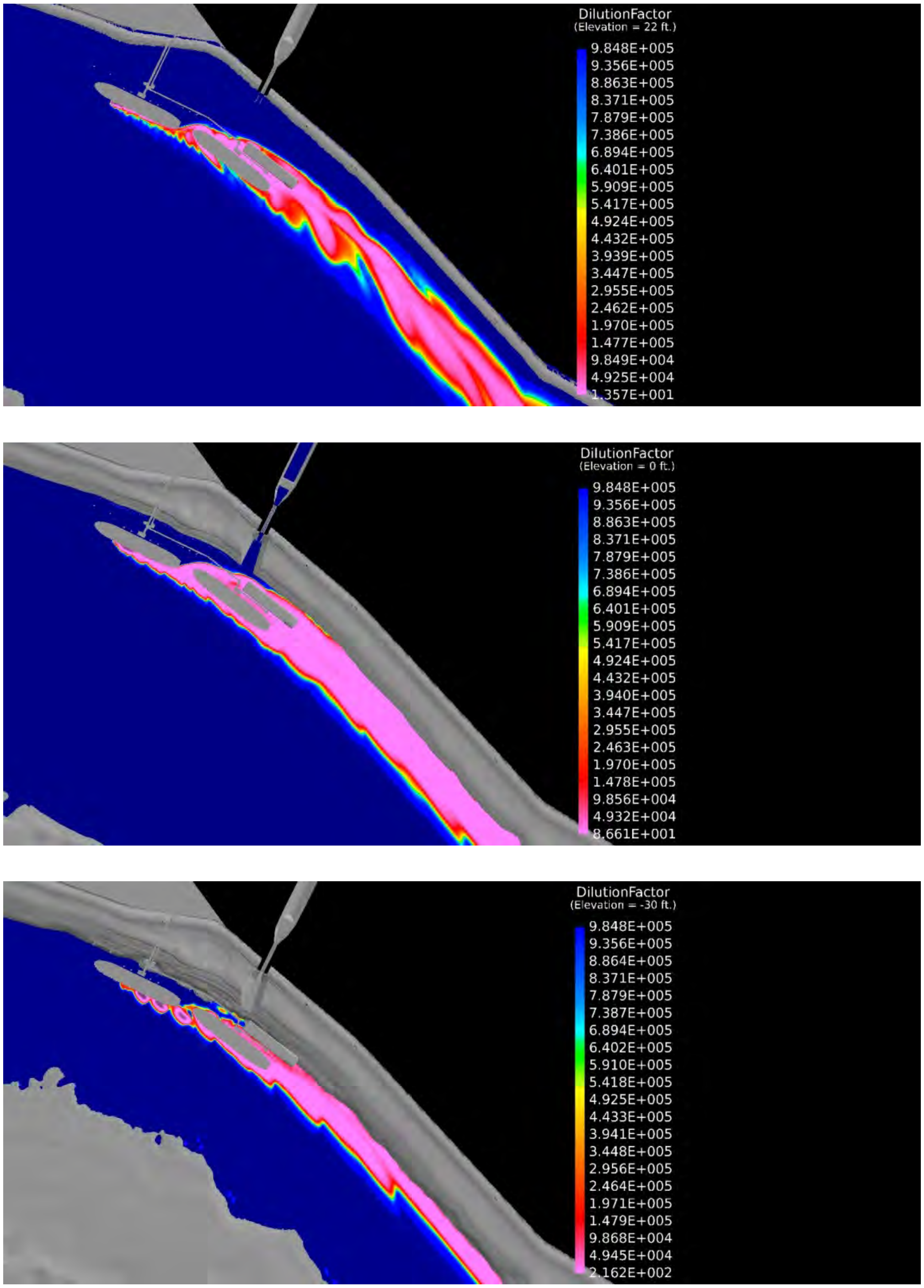



**Figure A-22 Run #10 model results. Dilution factors at elevations 22 feet (top), 0 feet (middle), and -30 feet (bottom) 2 hours after Bitumen spill.**

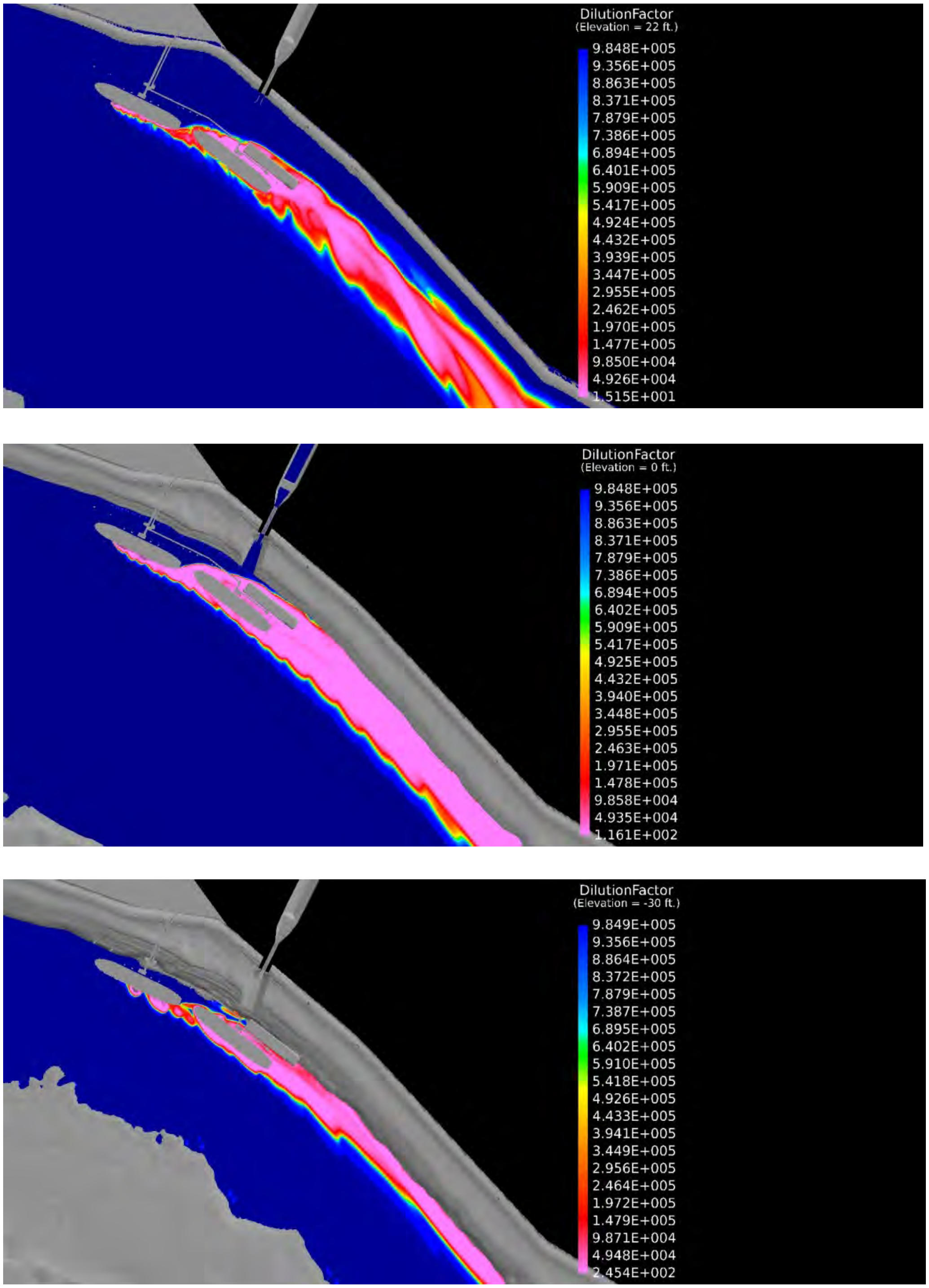



**Figure A-23. Run #10 model results. Dilution factors at elevations 22 feet (top), 0 feet (middle), and -30 feet (bottom) 3 hours after Bitumen spill.**

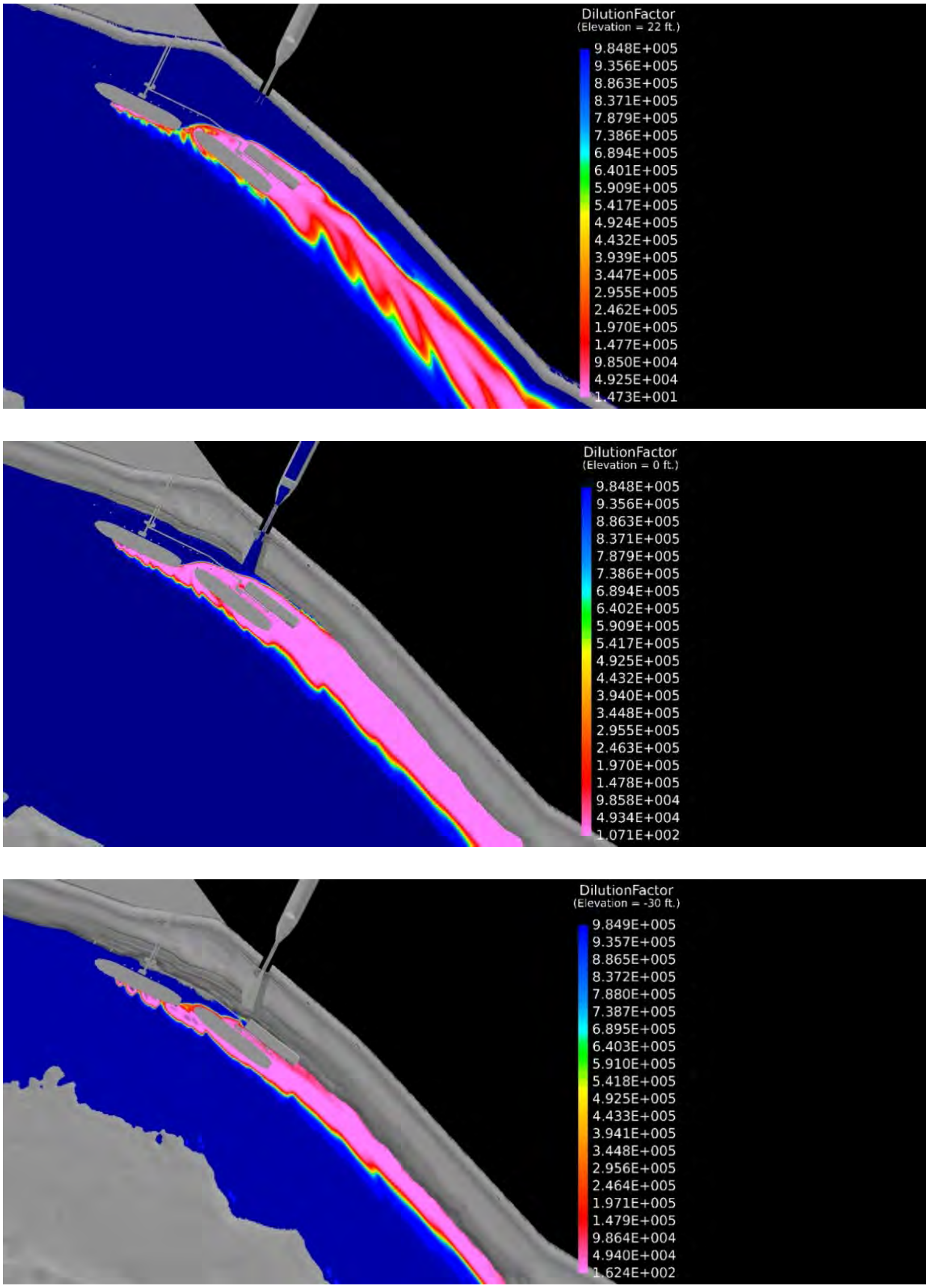



**Figure A-24. Run #10 model results. Dilution factors at elevations 22 feet (top), 0 feet (middle), and -30 feet (bottom) 3.87 hours after Bitumen spill.**

Pin Oak Holdings
New Marine Facility

October 16, 2014
Water Quality

The purpose of this task is to predict the movement of pollutants away from the terminal facility in the event of a spill during a period of high flow.

Below are a list of worst case scenario assumptions and criteria:

Vessel Data:

Vessel 1
165,000 DWT
905 LOA
6 Tanks with solid centerline bulk head
369,665.8 bbl tank capacity (Cargo Tank #2, largest tank)

Product Date:

| | |
|---|---|
| Bitumen | |
| API Gravity | 7.8 |
| Density (lb/gal) | 8.48 |
| Viscosity @ 104°F (cSt) | 19770 |

Assumptions for Worst Case Scenario:

Ballast tanks are between our shell of vessel and cargo tank shell. The colliding vessel would have to penetrate both tanks (See Figure 1)

Solid centerline bulk head put only half cargo capacity at risk due to vessel side collision

Vessel 1 is fully laden at time of impact.

Impact occurs just above water line on Vessel 1

35% of tank above water line and at risk of discharge (See Figure 1)

Total capacity above impact point 57,726.5 bbl

Ballast tank empty when fully loaded

Ballast tank capacity below impact point 24,976.5 bbl

Maximum at risk cargo **32,750 bbl**

Reduce flow rate 20% to account for losses into ballast tank

Size of cargo tank penetration is 8"x16" gash (Diamond Shaped)

Gash molded as re-entrant tube for flow calculations ($C_D$)

Historically only ballast tanks penetrated gash sized estimated to cargo tank as absolute worst case

Assumptions for Spill From Loading Arms at Dock:

Vessel and Product Data as stated above

Maximum vessel loading rate 30,000 bph

Two 12" loading arms transferring simultaneously

ESD system on dock to close butterfly valves at base of arm within 30 seconds in accordance with US Coast Guard

Total time from start of spill to full closure of valve is only 35 seconds

Loading arm size is 15' riser, 30' inboard arm, 35' outboard arm. 100% outboard at risk 50% of inboard arm is at risk.

Total amount spilled 298.65 bbl.

**Figure A-25. Input parameters used for pollutant transport study.**

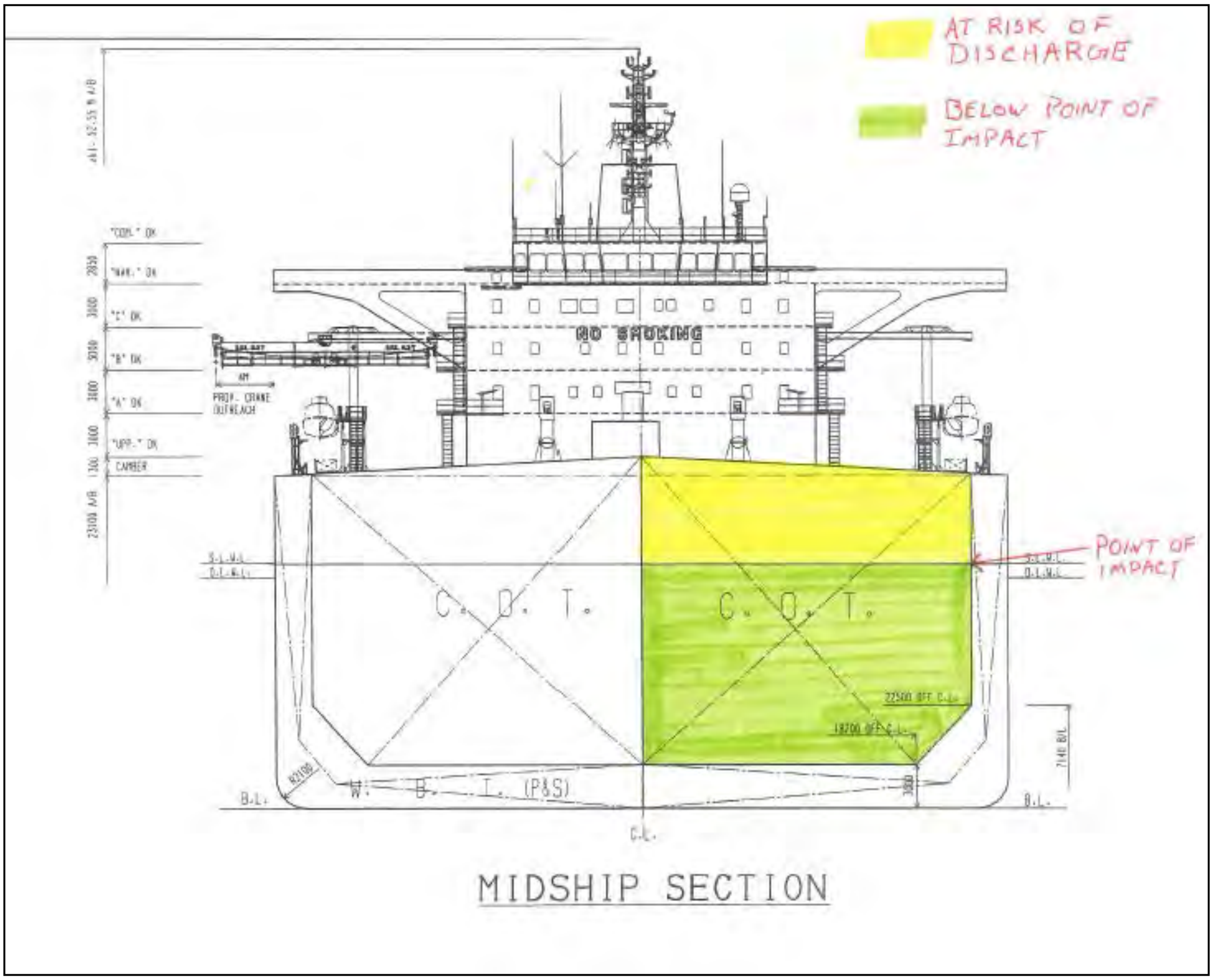



Figure A-26. Point of impact on the vessel (Run #10).

$V = C_D A (2gh)^{1/2}$


$V = (0.52)(0.8867)\left[2(32.2)(20)\right]^{1/2}$

$A = 0.8867\ ft^2$

$C_D = 0.52$

$V = 16.54\ ft^3/sec$

$g: 32.2\ ft/sec^2$

$V = 10,605.24$ bph

$h = 20\ ft$ from point of impact to top of tank

80% $V = 8,484.2$ bph

Amount of CARGO ABOVE point of impact 57,726.5 bbl

Capacity of empty ballast tank between Cargo tank and River. 24,976.5 bbl

At Risk cargo of discharging in the River 32,750 bbl

Time to empty 3 hours 52 minutes

A = Area of opening

$C_D$ = Coefficient of Discharge

g = gravity $ft/sec^2$

h = head due to product in tank

| | | |
|---|---|---|
| DESIGNED TMO — DATE 10/15/14 | CLIENT PIN OAK HOLDINGS | SHEET 1 OF 1 |
| CHECKED ECS — DATE 10/16/14 | PROJECT CPRA REQUEST | JOB NO. 9231 |
| SUBJECT PREDICTING MOVEMENT OF POLLUTANTS | | |

LANIER & ASSOCIATES CONSULTING ENGINEERS, INC. NEW ORLEANS, LA & BEAUMONT, TX

**Figure A-27. Calculation of discharge volume and duration (Run #10).**

## APPENDIX B: RIP-RAP SIZING

**<u>Background</u>**

Rip-rap placed in the entrance of the diversion must be able to prevent scour during periods of maximum discharge and withstand disturbances caused by the propellers of a vessel working near the proposed docking facility. Making use of additional information provided by the CFD analysis, rip-rap lining the entrance to the diversion was sized and the results were compared to current design specifications.

**<u>Methodology</u>**

To account for the effect of a vessel working near the entrance to the diversion, the speed of flow produced by its propellers was added to the maximum entrance flow speed determined from the results of the CFD analysis, and the results were used to size rip-rap based on the following procedure:

1. Flow speeds in the entrance of the diversion were determined from the results of the CFD analysis and the maximum value was used for rip-rap sizing.
2. The flow speed produced by a vessel operating in the vicinity of the diversion was estimated based worst case assumptions of propeller thrust and proximity (*i.e.,* prop wash).
3. Adding together the flow speeds determined in points (1) and (2) - riprap sizes were selected using a chart published by the Oregon DOT and the results were compared to current design specifications.[7]

**<u>Results</u>**

<u>Maximum Flow Speed in Diversion Entrance (No Vessels)</u>

As shown in Figure B-1, depth-averaged flow speeds at eight different locations in the diversion entrance were output from the CFD model for the two conditions studied (<u>Low Flow</u> – Mississippi River Flow Rate 656,000 cfs, Diversion Flow Rate 3,200 cfs, nominal water surface elevation 13.95 ft NAVD88; <u>High Flow</u> – Mississippi River Flow Rate 1,500,000 cfs, Diversion Flow Rate 4,800 cfs, nominal water surface elevation 23.16 ft NAVD88).[8] According to these data, the maximum flow speed in the diversion is equal to 6.2 ft/s (Table 1) and is located on the upstream embankment of the entrance.

---

[7] Information that appears on this chart was originally published in the American Society of Civil Engineers Proceedings (June, 1948). It includes the effects of bottom slope.

[8] Since the water level in the river changes significantly (Low Flow vs. High Flow) the results from both simulations were output so that a reliable estimate of maximum flow speed in the diversion could be identified.

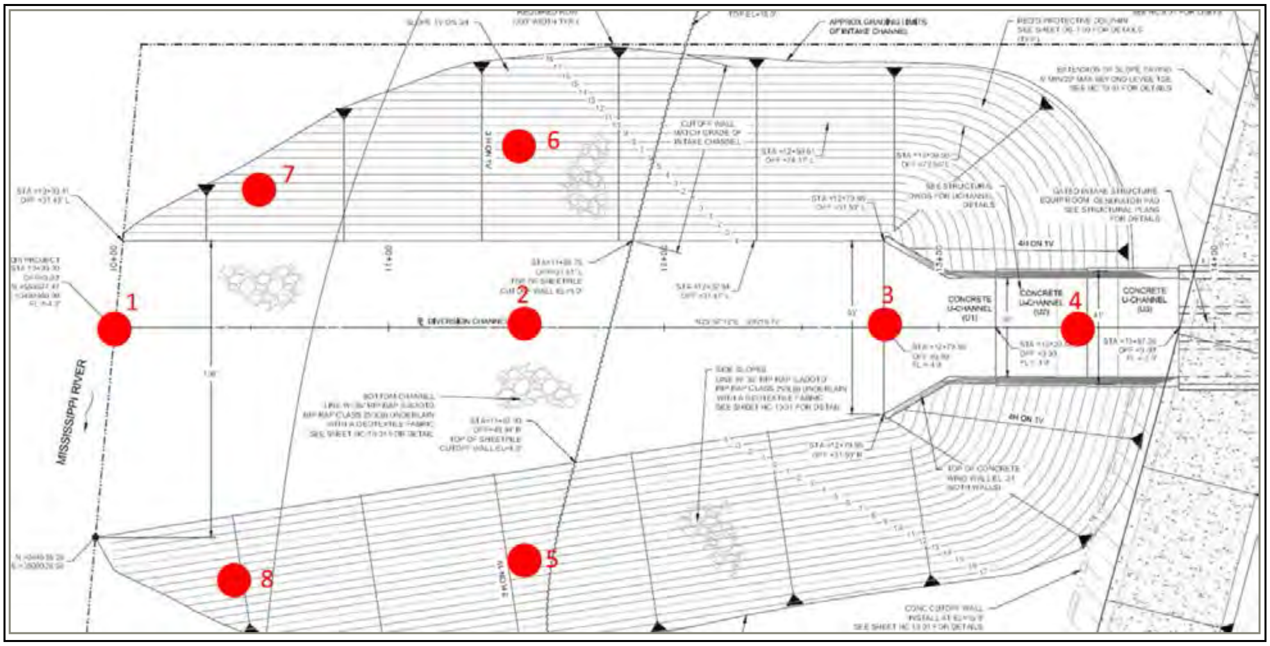



**Figure B-1. Sampling Locations.**

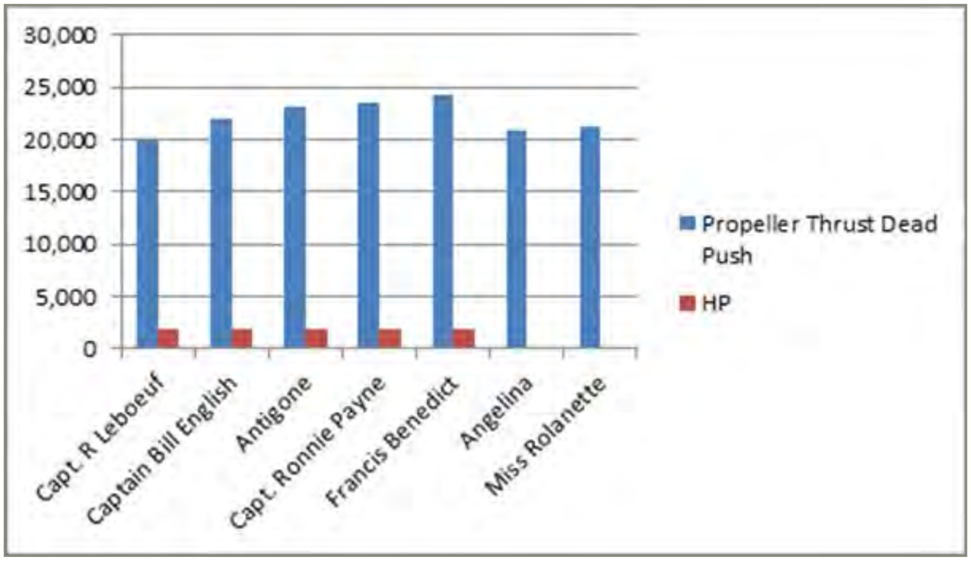
**Table B-1. Sampling Results[9].**

| Location | Depth Average Flow Speed (ft/s) | |
|---|---|---|
| | Low River Flow | High River Flow |
| 1 | 2.6 | 4.8 |
| 2 | 3.0 | 4.0 |
| 3 | 3.8 | 4.8 |
| 4 | 5.4 | 5.7 |
| 5 | 3.4 | 2.0 |
| 6 | 1.7 | 1.5 |
| 7 | 4.0 | **6.2** |
| 8 | 2.4 | 6.0 |

[9] ***FLOW-3D*** has the facility to report depth averaged flow speeds and the values shown in Table 1 were output directly from the CFD model. Maximum flow speeds were reported at Location 7.

Prop Wash

The flow speed (*i.e.,* prop wash) produced by a vessel operating in the vicinity of the diversion was based on information provided by Lanier & Associates Consulting Engineers (Figure B-2). These data provide maximum propeller thrust estimates for different vessels operating on the Mississippi River. For the purpose of rip-rap sizing, a thrust value of 35,000 pounds was used – this value being slightly greater than the maximum reported value for the tugboat Betty Brent.



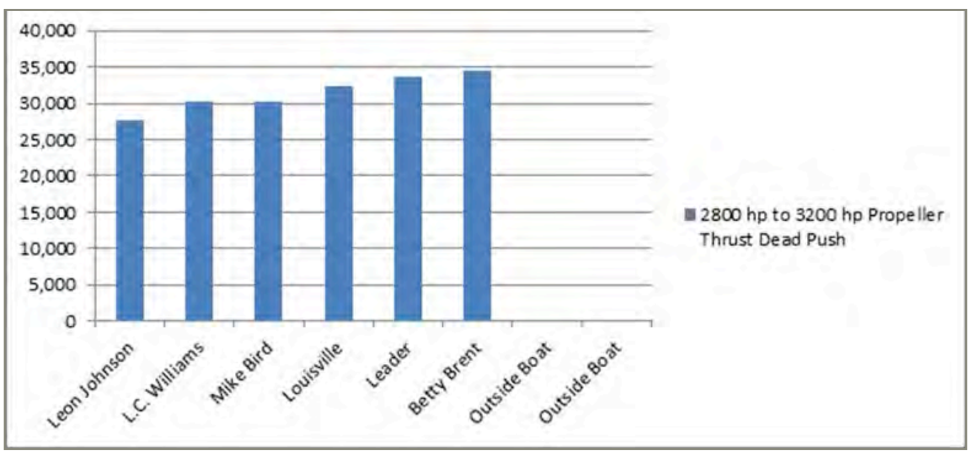




**Figure B-2. Propeller Thrust (units are pounds).**

Having selected an appropriate thrust value for analysis, a two–step calculation was used to estimate the speed of flow produced by the propeller at the entrance to the diversion. In step one, an equation for static thrust (eq. [1]) was used to calculate the velocity of accelerated water at the ship's propeller based on a propeller diameter of 9 feet (a typical value recommended by Lanier & Associates).[10]

$$T = \frac{\pi}{8} D^2 \rho (\Delta v)^2 \tag{1}$$

where,

$T$ = thrust (force)
$D$ = propeller diameter (length)
$\rho$ = density of water [mass/length$^3$]
$\Delta v$ = velocity of accelerated water (length/time)

Using a density for water equal to 1.94 slug/ft$^3$, the velocity of accelerated water at the propeller was calculated to be 24 ft/s.

In step two, the minimum distance between the ship's propeller and the intake entrance was determined and the velocity of accelerated water at the intake entrance was calculated based on jet theory. As shown in Figure B-3, the minimum distance between an operating vessel and the diversion entrance was taken to be 200 feet. The selection of this value was somewhat arbitrary - having no information about how and where a vessel would actually operate in the vicinity of the diversion. The analysis does, however, assume that the vessel would be operating under full power, which is unlikely given the layout of the facilities.

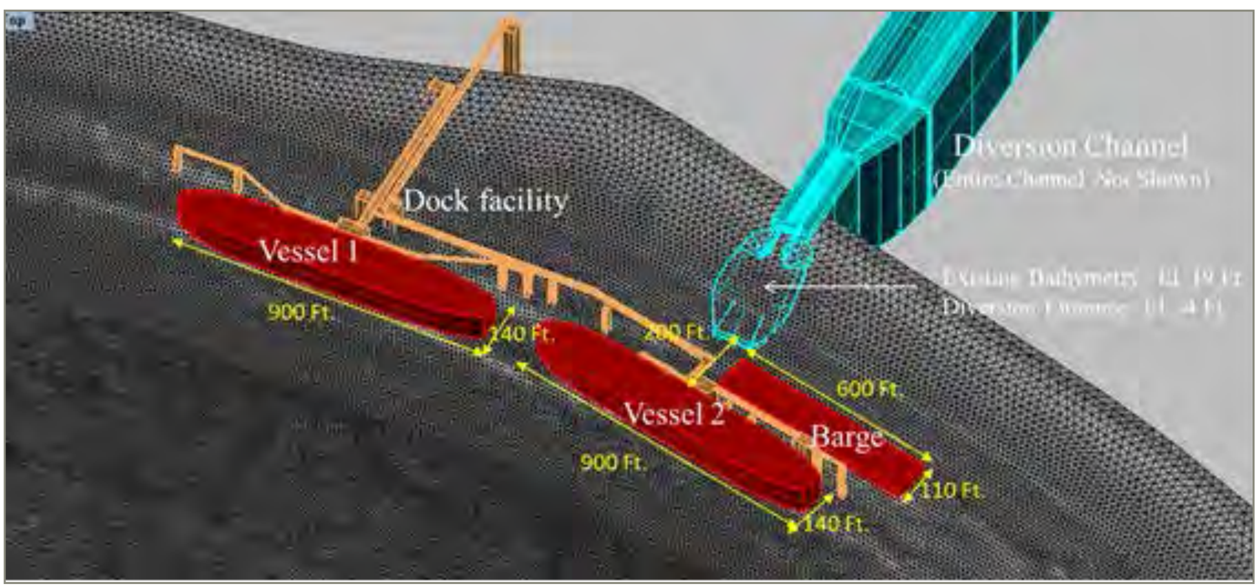



**Figure B-3. Proximities.**

[10] ref. https://quadcopterproject.wordpress.com/static-thrust-calculation/

The velocity of accelerated water at the intake was estimated from jet theory (eq. [2])[11] which assumes that the high speed flow generated by the propeller penetrates into a fluid body at rest and that the jet is directed straight towards the intake.

$$u_{max} = \frac{5d}{x} U \quad (2)$$

where,

$u_{max}$ = maximum velocity
$U$ = average exit velocity
$d$ = orifice/propeller diameter
$x$ = distance from the source

Given a propeller diameter of 9 feet and velocities of accelerated water equal to 24 ft/s, the maximum velocity along the jet's centerline as a function of distance is,

$$u_{max} = \frac{1080}{x} \quad (3)$$

and assuming that the jet originates 200 feet away from the diversion, the maximum velocity of accelerated water at the entrance would be 5.4 ft/s.

Rip-Rap Sizing

In a final step of the analysis, rip-rap was sized using information published by the Oregon DOT based on a flow velocity of 12 ft/s.[12]

Using a velocity based rip-rap design chart adapted from the Oregon DOT Hydraulics Manual (Figure B-4), the equivalent spherical diameter of median stone size ($D_{50}$) for rip-rap designed to withstand 12 ft/s flow speeds on a 3H:1V slope is about 1.125 ft. This compares favorably to the size of rip-rap currently specified for the intake – LADOTD rip-rap class 250 lb – ref. Figure B-5.

[11] Ref. B. Cushman-Roisin, Environmental Fluid Mechanics, Chapter 9, Thayer School of Engineering, Dartmouth College, Hanover, NH 03755, 2014.

[12] The maximum velocity of flow in the diversion during normal operation added to the maximum velocity of accelerated water produced by a propeller generating 35,000 lbs of thrust – rounded up from 11.6 ft/s.

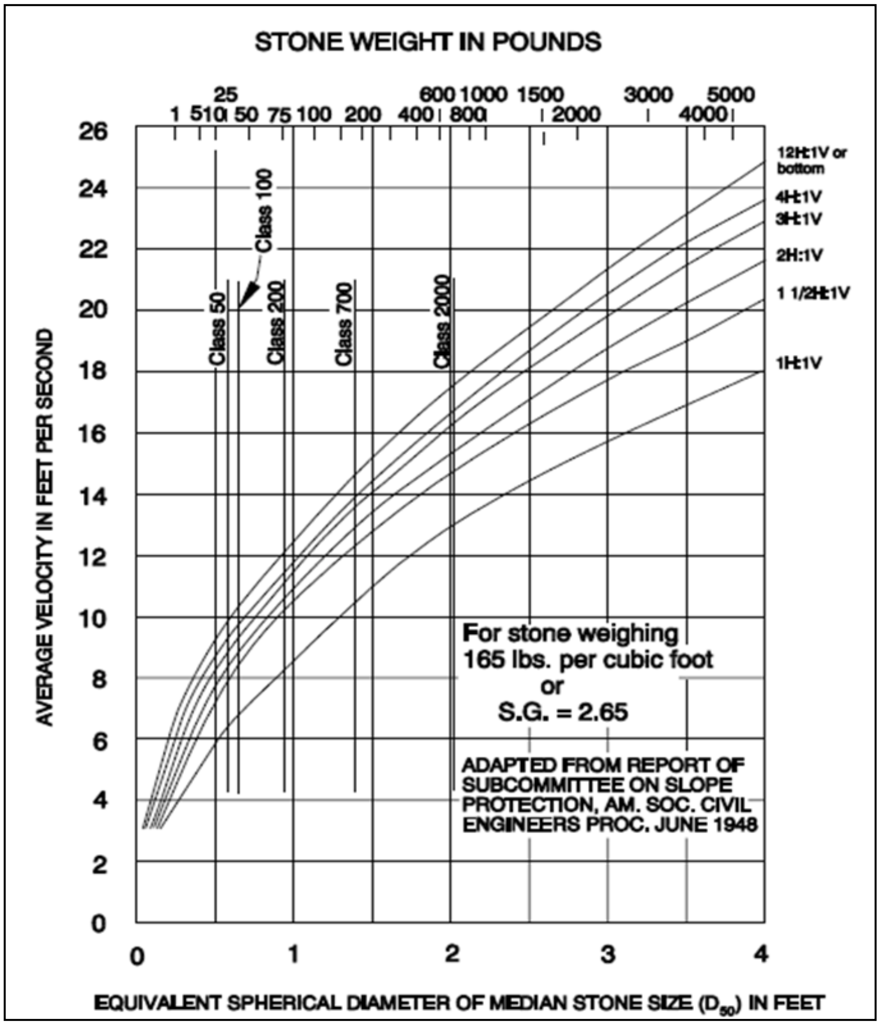


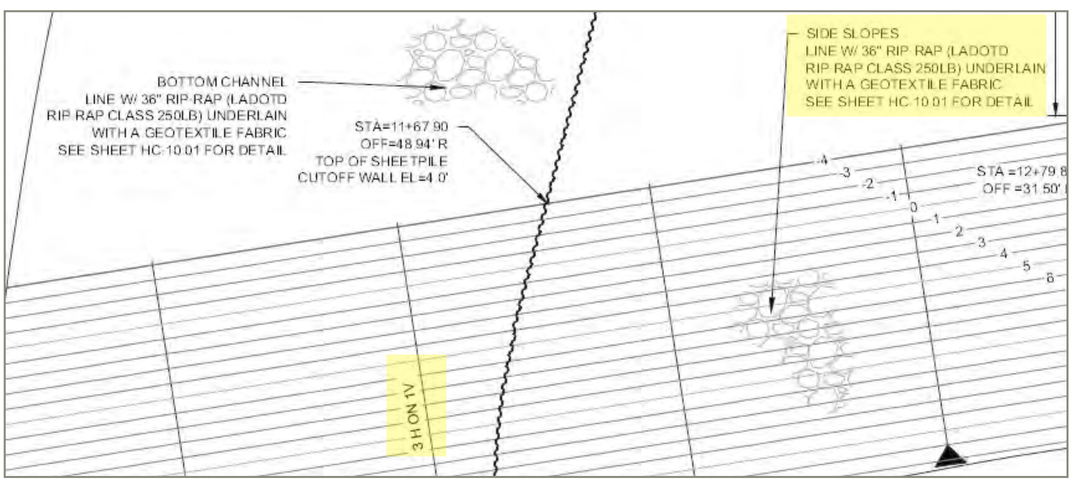

**Figure B-4. Velocity Based Rip-Rap Design Chart (adapted from Oregon DOT Hydraulics Manual).**

| Riprap Class[1] | Stone Size (lb) | Spherical Diameter[2] (ft) | Percent of Stone Smaller Than |
|---|---|---|---|
| 2 lb | 10 | 0.51 | 100 |
| | 4 | 0.38 | 45 - 100 |
| | 2 | 0.30 | 15 - 50 |
| | 0.75 | 0.22 | 0 - 15 |
| 10 lb | 50 | 0.88 | 100 |
| | 20 | 0.65 | 45 - 100 |
| | 10 | 0.51 | 15 - 50 |
| | 5 | 0.41 | 0 - 15 |
| 30 lb | 140 | 1.24 | 100 |
| | 60 | 0.94 | 45 - 100 |
| | 30 | 0.74 | 15 - 50 |
| | 10 | 0.51 | 0 - 15 |
| 55 lb | 275 | 1.50 | 100 |
| | 110 | 1.11 | 45 - 100 |
| | 55 | 0.88 | 15 - 50 |
| | 20 | 0.63 | 0 - 15 |
| 130 lb | 650 | 2.00 | 100 |
| | 260 | 1.46 | 45 - 100 |
| | 130 | 1.17 | 15 - 50 |
| | 40 | 0.79 | 0 - 15 |
| 250 lb | 1250 | 2.50 | 100 |
| | 500 | 1.83 | 45 - 100 |
| | 250 | 1.46 | 15 - 50 |
| | 80 | 1.00 | 0 - 15 |
| 440 lb | 2200 | 3.00 | 100 |
| | 900 | 2.23 | 45 - 100 |
| | 440 | 1.76 | 15 - 50 |
| | 130 | 1.17 | 0 - 15 |
| 1000 lb | 5000 | 4.00 | 100 |
| | 2000 | 2.91 | 45 - 100 |
| | 1000 | 2.31 | 15 - 50 |
| | 300 | 1.55 | 0 - 15 |

[1]The stone size used to define the Riprap Class is the minimum median stone size for the stone class. The minimum thickness of a riprap layer shall be no less than the spherical diameter of the maximum stone size in the Riprap Class.

[2]Spherical diameters of riprap classes up to 30 lb are based on a solid weight of 140 lb/cu ft. Spherical diameters of riprap classes above 30 lb are based on a soild weight of 155 lb/cu ft.

(a)




(b)

**Figure B-5. Rip-Rap Specification. (a) LADOTD Design Manual, (b) Excerpt from Design Drawing.**

## Conclusions

According to this analysis, the size of rip-rap currently specified for use in the entrance of the diversion structure should be able to prevent scour during periods of maximum discharge and withstand disturbances caused by the propellers of a vessel working near the proposed docking facility.

## Final Note

On June 8, 2015 the results of this analysis were provided by Brad Miller (CPRA) to Bruce Lelong (AECOM) for external peer review. On June 17, 2015 AECOM provided the following comments paraphrased below –

> *URS used the USACE HEC-11 method with a Factor of Safety of 2. Their calculations led to a $D_{50}$ of 1.4 ft and a weight of 250 lb stone with a recommended thickness of 2.3 ft. Therefore, URS used 36 inches of LADOTD rip rap class 250 (lb). AECOM concurs with the findings in the report provided by the CPRA and agrees that no change in rip rap size is required.*

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# Technical Memorandum

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From:
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8/14/2017

Date:
June 28, 2017

Project No.:

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Subject:
TO28.2 Simulation of Flow near Proposed Docking Facility and Freshwater Diversion

## 1. Introduction

This memorandum summarizes results of Computational Fluid Dynamics (CFD) simulations used to study the impact of a newly proposed docking facility on the operation of a freshwater diversion at Mississippi River Mile (RM) 144.2 in Reserve, Louisiana. This analysis is an extension of a previous study performed by The Water Institute and ARCADIS U.S., Inc. (Meselhe et al, 2015). Similar to the previous study, the commercially available CFD program known as ***FLOW-3D*** **(www.flow3d.com)** was used to carry out the flow simulations.

The primary objective of this study was to analyze flow patterns and sediment transport near the proposed docking facility and diversion intake. The results of the analyses described herein were used to evaluate the effect of the development on the operation of the fresh water diversion.

## 2. Approach

A three-dimensional (3-D) CFD model of the docking facility, freshwater diversion, and a portion of the Mississippi River was used to simulate flow patterns near the facility and diversion intake. The ***FLOW-***3D computer program was used because it is specially designed for the simulation of complex free surface flows (e.g., to simulate flow patterns in the vicinity of proposed construction area).

The procedure used to carry out the analysis was as follows:

1. review CAD drawings of the newly proposed docking facility, Figures 1 and 2;
2. Update the 3-D CFD model used in the previous study;
   a. replace the old docking facility with the new one,
   b. refine the computational mesh to resolve pilings that are part of the new facility,
   c. apply boundary conditions similar to those used in the previous study for consistency,
3. carry out simulations for a low river flow condition; and
4. analyze model results and draw conclusions regarding the effect of the new docking facility on the operation of the freshwater diversion.

## Model Geometry

CAD drawings provided by the Coastal Protection and Restoration Authority (CPRA) were reviewed and used to construct a 3-D CAD model of the proposed docking facility. The 3-D CAD model was created using Rhino software (https://www.rhino3d.com; Rhinoceros 2014). Utilizing the previous 3-D CAD model, the previous docking facility was replaced with the new one. Figure 3 shows the updated model with the new docking facility and freshwater diversion channel combined with a portion of the Mississippi River.

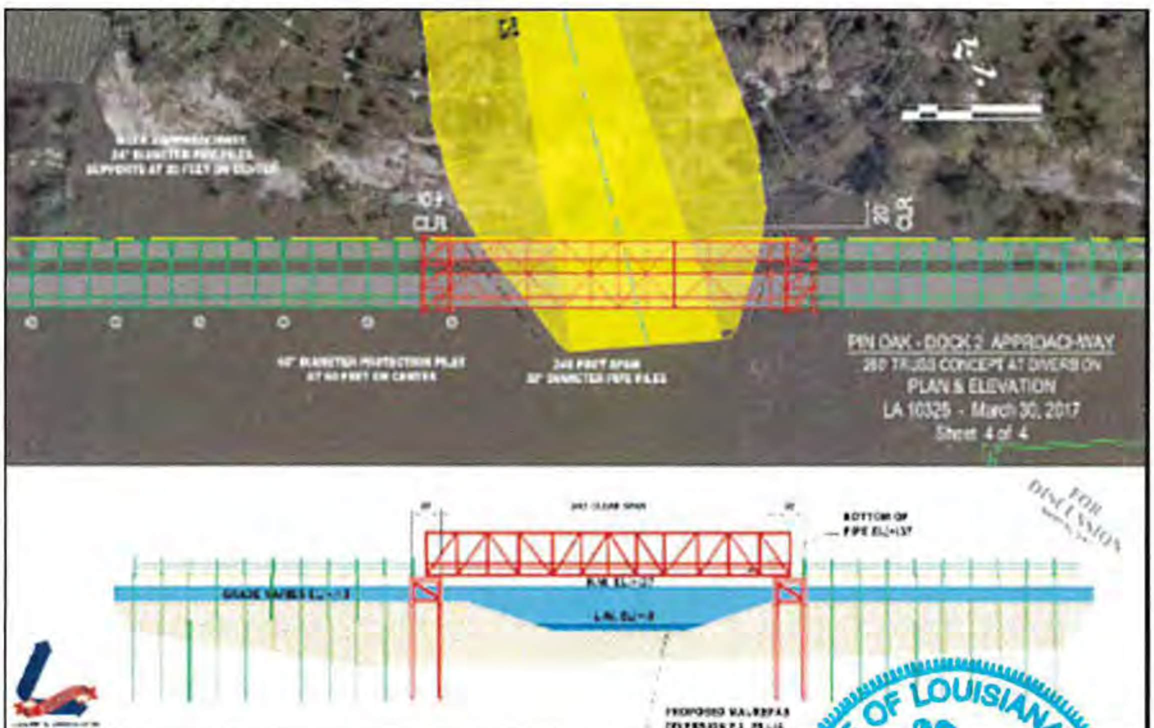


Figure 1 - New proposed docking facility – Alternative 1.

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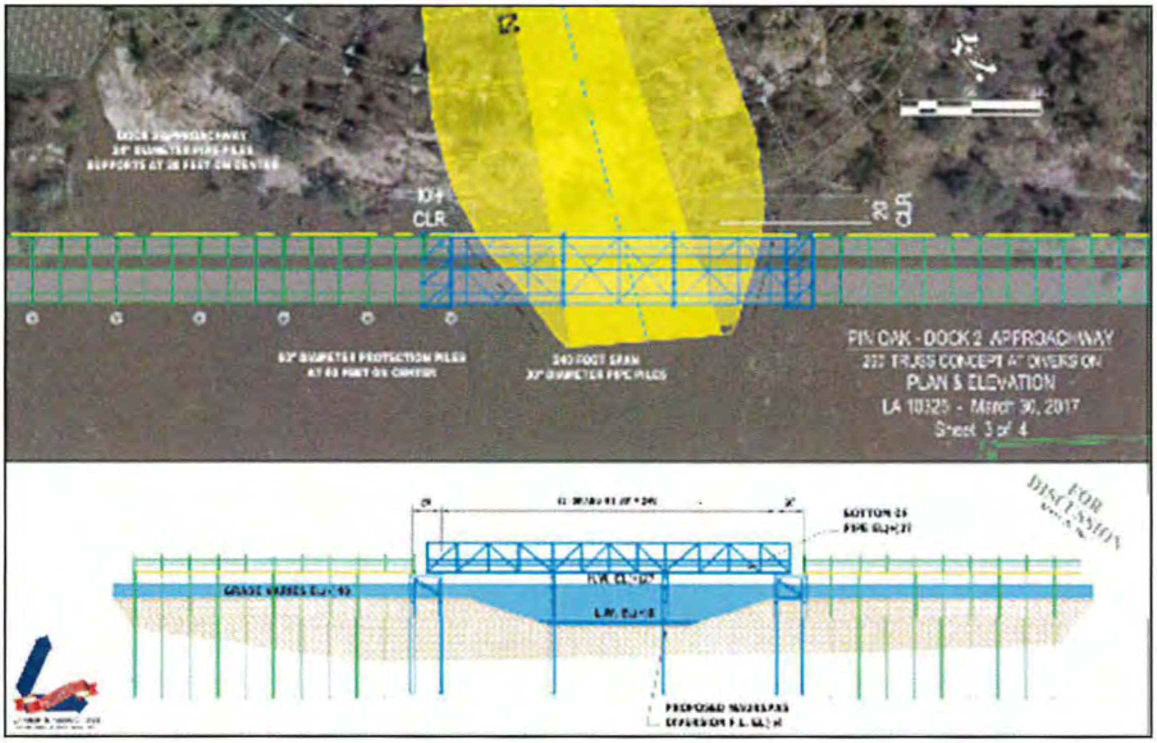


Figure 2 - New proposed docking facility – Alternative 2. Similar to Alternative 1, but with additional pilings at the entrance of the diversion.

Figure 3 - 3-D model of the Mississippi River, diversion channel, and proposed docking facility (Alternative 1). For Alternative 2, additional pilings are added at the entrance of the diversion.

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Boundary Conditions

For consistency, boundary conditions used in this analysis were of the same type as those used in the previous study (Meselhe et al, 2015). These were:

- volumetric flow rate at the upstream end of the model (approximately 27,000 feet from the project area);
- water surface elevation at the downstream end of the model (approximately 6,000 feet from the project area);
- water surface elevation at the downstream end of the diversion channel;
- no-slip conditions were applied at solid boundaries; and
- free-surface conditions were imposed at the air/water interface.

Computational Mesh

To perform the simulations, the model domain was subdivided into small elements (cells) within which the governing equations of fluid flow were solved. Refinements to the mesh were made locally to improve the ability of the model to capture details of the flow near the docking facility and the entrance to the diversion.

In this analysis, the computational mesh used in the simulation was the same as that used previously, except near the docking facility where additional mesh blocks were added and further refinements were made to resolve the pilings. A total of 16 mesh blocks were used with cell sizes varying from 2 to 40 feet horizontally and 2 to 10 feet vertically in each block. Figure 4 shows the computational mesh used in the model.

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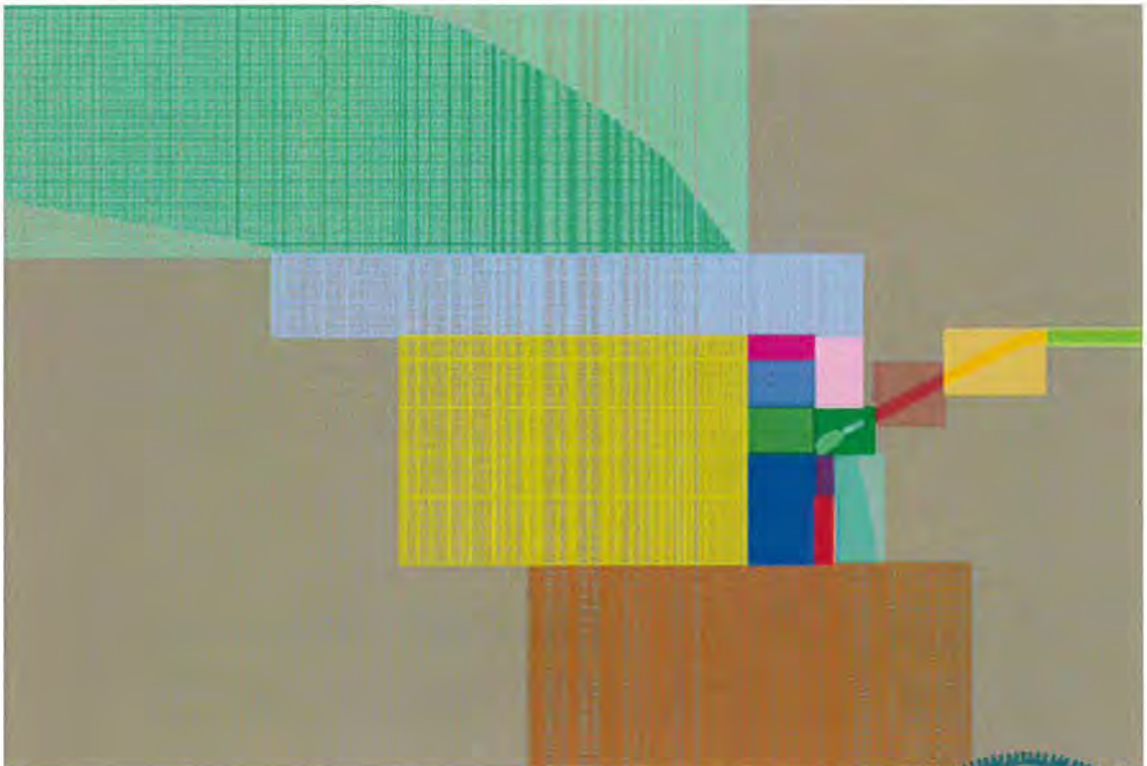

Figure 4 – Computational model mesh block configuration. Cell sizes were smallest near the docking facility and entrance of the diversion.

## 3. Simulations

Three simulations were carried out using a Mississippi River flow rate equal to 656,000 cubic feet per second (cfs). Descriptions of these runs are provided below.

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Run #1 – River Flow with Operating Freshwater Diversion and New Dock Facility (Alternative 1)

The purpose of this simulation was to predict changes in flow patterns resulting from construction of the new docking facility (Alternative 1) with an operating freshwater diversion. The discharge in the Mississippi River was set to 656,000 cfs at the upstream model boundary and water surface elevation was set equal to 13.95 feet NAVD88 at the downstream model boundary. In addition to these specifications, the water surface elevation at the downstream end of the diversion channel was set to 3.37 feet NAVD88.

Run #2 – River Flow with Closed Freshwater Diversion and New Dock Facility (Alternative 1)

The purpose of this simulation was to predict changes in flow patterns resulting from construction of the new docking facility (Alternative 1) with a non-operating (closed) freshwater diversion. Like Run #1, the discharge in the Mississippi River was set to 656,000 cfs at the upstream model boundary, the water surface elevation was set equal to 13.95 feet NAVD88 at the downstream model boundary, and the water surface elevation at the end of the diversion channel was set to 3.37 feet NAVD88.

Run #3 – River Flow with Operating Freshwater Diversion and New Dock Facility (Alternative 2)

The purpose of this simulation was to predict changes in flow patterns resulting from construction of the proposed docking facility (Alternative 2) with an operating freshwater diversion. Like Runs #1 and #2, the discharge in the Mississippi River was set to 656,000 cfs at the upstream model boundary, the water surface elevation was set equal to 13.95 feet NAVD88 at the downstream model boundary, and the water surface elevation at the end of the diversion channel was set to 3.37 feet NAVD88.

## 4. Results

Computed diversion flows and water surface elevations (near the entrance to the diversions) are provided in Table 1. These results show no significant differences between the new alternative designs and the one studied before.

Table 1. Computed flow diversion and water surface elevation.

| Docking Facility | Flow Diversion (cfs) | Water Surface Elevation (ft) |
|---|---|---|
| Previous Design | 3,200 | 14.26 |
| Alternative 1 | 3,246 | 14.29 |
| Alternative 2 | 3,238 | 14.27 |

Computed flow patterns near the docking facility and entrance of the diversion are shown in Figures 5 through 13. Velocity contours were used to show approach flow patterns near the facility. Streamlines were back calculated from the diversion channel to show where water entering the diversion came from.

Figure 5 shows velocity contours near the facility and entrance of the diversion with the previous docking facility in place. This figure was included so that comparisons could be made between the previous study results and these new ones.

Velocity contours with Alternative 1 in place are shown in Figure 6. Compared to the results shown in Figure 5, flow speeds in front of and upstream of the diversion are slower. The reduction in flow speeds is caused by the clusters of pilings, which create resistance to the approaching flow. Figure 7 shows velocity contours with streamlines superimposed to show where most of the water entering the diversion comes from. Figure 8 shows a comparison of streamlines between the previous results and Alternative 1 results.

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In Figure 8, streamlines extend further into the river with Alternative 1 because of the increase in flow resistance.

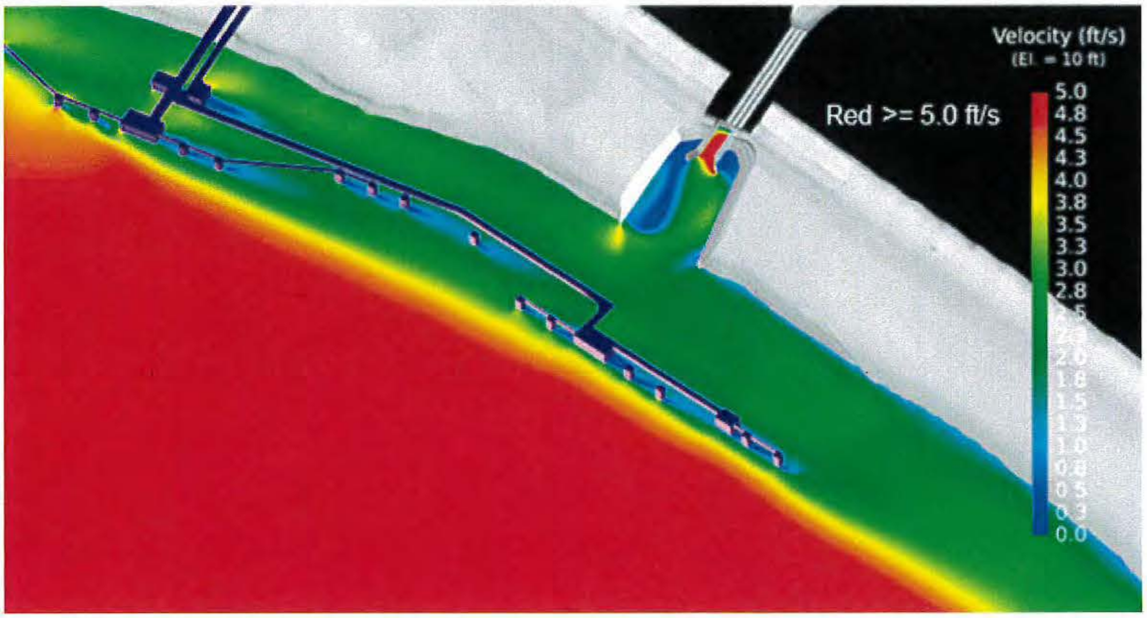


Figure 5 – Velocity contours with previous docking facility and operating diversion. Slice plane cut horizontally at an elevation equal to 10 ft.

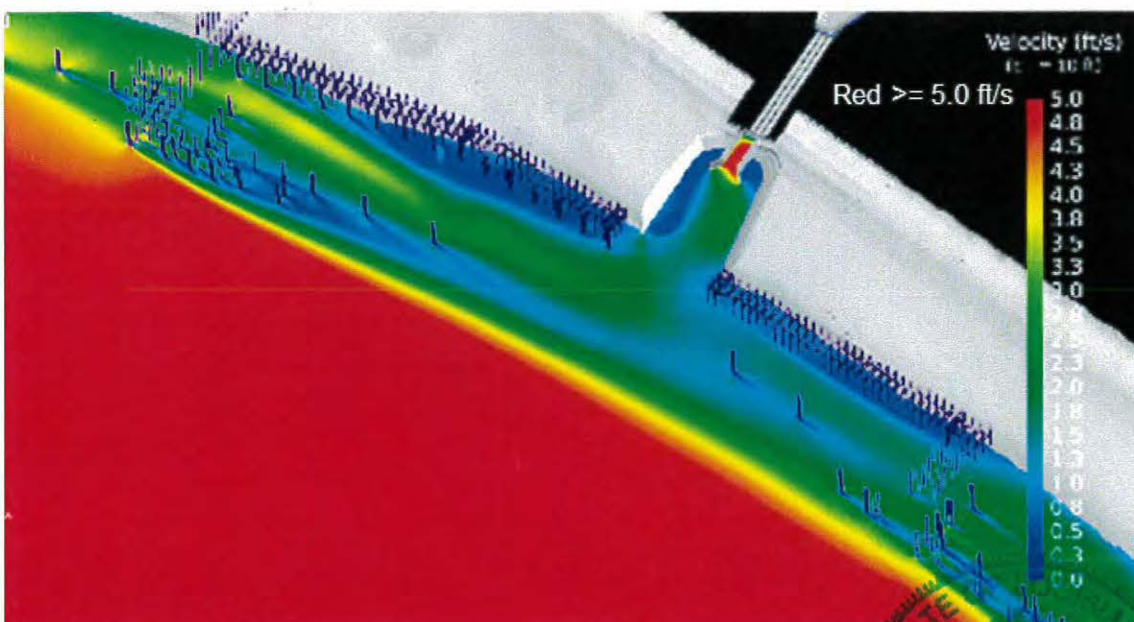


Figure 6 - Velocity contours with Alternative 1 docking facility and operating diversion. Slice plane cut horizontally at an elevation equal to 10 ft.

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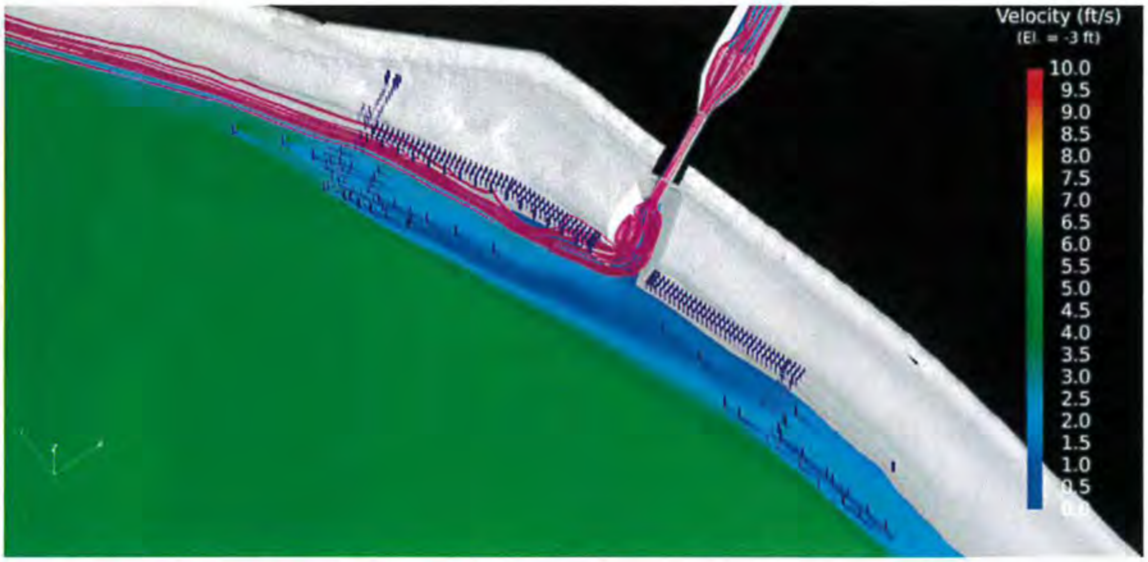


Figure 7 – Velocity contours with streamlines superimposed with Alternative 1 docking facility and operating diversion.


Figure 8 – Comparison of streamlines between the previous docking facility (cyan) and Alternative 1 (pink) docking facility.

The reduction in velocities near the intake associated with the Alternative 1 docking facility could result in siltation upstream of the diversion intake when it is closed. A simulation with the diversion closed was carried out to verify if velocities are reduced in area of interest to indicate whether sedimentation would occur during closure periods. Figure 9 shows the velocity contours near the facility and entrance of the

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diversion with diversion closed. As shown in the figure, low velocities are observed in the vicinity of the intake when the diversion is closed.

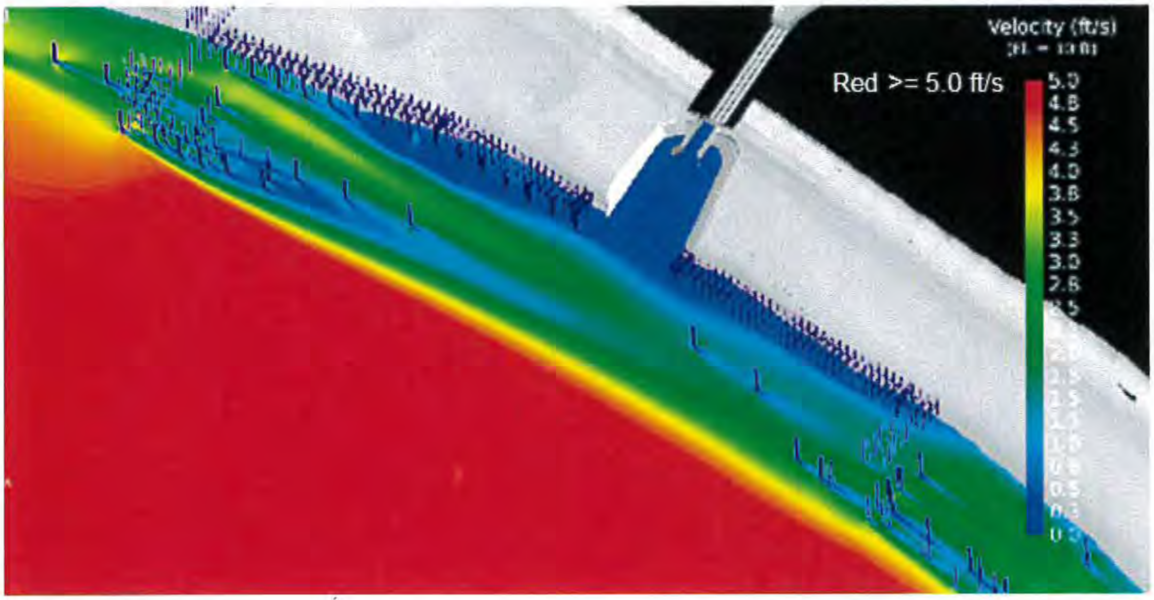


Figure 9 - Velocity contours with Alternative 1 docking facility and non-operating diversion. Slice plane cut horizontally at an elevation equal to 10 ft.

To assist in determining the potential for sediment deposition near the facility, the critical depth-averaged velocity for incipient motion was calculated and estimates of transport regime were obtained from the Rousean suspension profile shown in Table 2. According to this table, the condition $u_*/w_s = 0.2$ will initiate motion. Shear velocity, also called friction velocity, is a form by which a shear stress may be re-written in units of velocity, while settling velocity is the rate at which sediment deposit.

Table 2. Transport regime estimates.

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| Limits | Transport regime |
|---|---|
| $u_*/w_s < 0.2$ | Deposition Dominant |
| $u_*/w_s = 0.2$ | Incipient Motion |
| $0.2 < u_*/w_s < 0.4$ | Bedload Transport |
| $0.4 < u_*/w_s < 2.5$ | Transitional Zone, Mixed Load |
| $2.5 < u_*/w_s < 100$ | Suspended Load Dominant |
| $100 < u_*/w_s$ | Suspended Load with Negligible Settling |

In this analysis, the settling velocity, $w_s$, of particles was computed using the Van Rijn formulation (Van Rijn, 1993),

$$w_{s,0}^{(\ell)} = \begin{cases} \dfrac{(s^{(\ell)}-1)gD_s^{(\ell)2}}{18\nu}, & 65\ \mu\text{m} < D_s \leq 100\ \mu\text{m} \\ \dfrac{10\nu}{D_s}\left(\sqrt{1+\dfrac{0.01(s^{(\ell)}-1)gD_s^{(\ell)3}}{\nu^2}}-1\right), & 100\ \mu\text{m} < D_s \leq 1000\ \mu\text{m} \\ 1.1\sqrt{(s^{(\ell)}-1)gD_s^{(\ell)}}, & 1000\ \mu\text{m} < D_s \end{cases}$$

where:

| | |
|---|---|
| $s^{(\ell)}$ | relative density $\rho_s^{(\ell)}/\rho_w$ of sediment fraction$(\ell)$ |
| $D_s^{(\ell)}$ | representative diameter of sediment fraction $(\ell)$ |
| $\nu$ | kinematic viscosity coefficient of water [$m^2/s$] |

Knowing the transport regime and settling velocity of particles, the critical depth-average velocity, $\overline{U}$, for incipient motion is computed using the equation below,

$$\overline{U} = 5.75u_* \log\left(\frac{12.2h}{k_s}\right)$$

where; $h$ is the water depth, $k_s = 2.5d_{50}$, and $u_* = 0.2w_s$.

Table 3 shows the computed settling velocity and critical depth-average velocity for incipient motion of various particle sizes.

Table 3 – Computed settling velocity and incipient motion of various particle sizes.

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| Materials | Description | Particle Sizes | Settling Velocity | Settling Velocity | Incipient Motion of Particles |
|---|---|---|---|---|---|
| | | | mm/s | ft/s | ft/s |
| clay | <0.002 mm | | 0.001 | 0.0000033 | |
| consolidated clay | <0.002 mm | | 0.1 | 0.0003280 | |
| silt | 0.002 mm - 0.06 mm | **32 µm** | 0.1 | 0.0003280 | **0.002** |
| sand | 65 µm < D <= 100 µm | **63 µm** | 3.6 | 0.0117067 | **0.076** |
| sand | 65 µm < D <= 100 µm | **96 µm** | 8.3 | 0.0271830 | **0.170** |
| sand | 100 µm < D <= 1000 µm | **125 µm** | 11.8 | 0.0386338 | **0.237** |
| sand | 100 µm < D <= 1000 µm | **250 µm** | 35.1 | 0.1152724 | **0.667** |

Figure 10 is an iso-surface plot of flow speed colored by different ranges of speed to identify areas where sediment deposition may occur with the newly proposed docking facility (Alternative 1) and operating diversion.

Figure 11 is an iso-surface plot of flow speed colored by different ranges of speed to identify areas where sediment deposition may occur with the newly proposed docking facility (Alternative 1) and closed (non-operating) diversion.

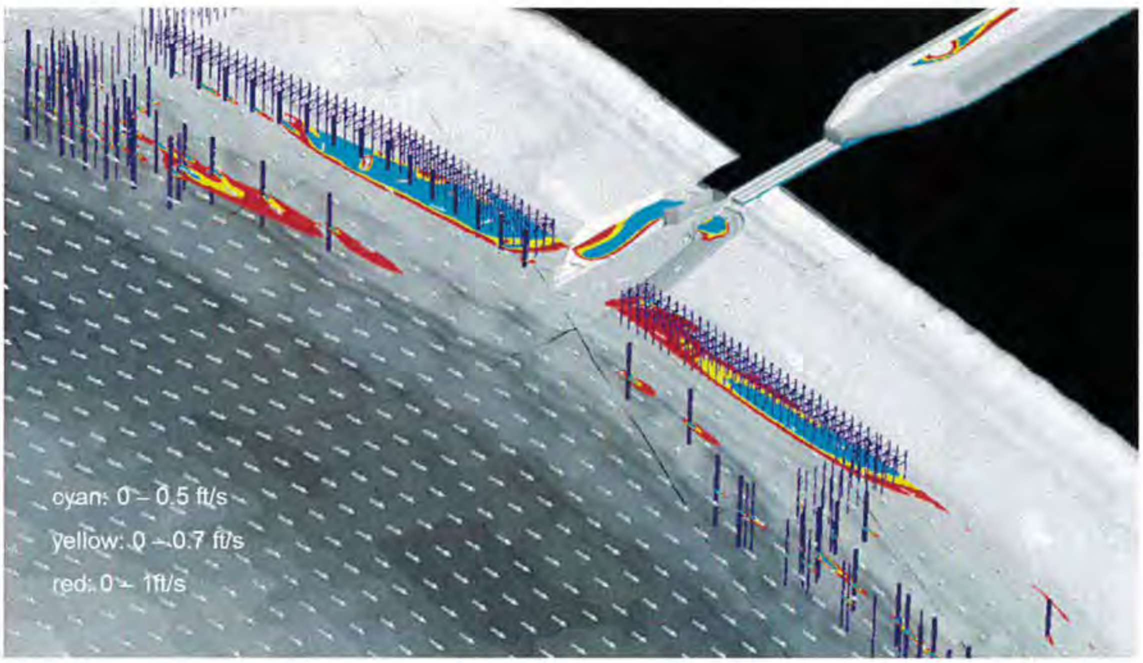


Figure 10 – Iso-surface plot of flow speed with Alternative 1 docking facility and operating diversion.

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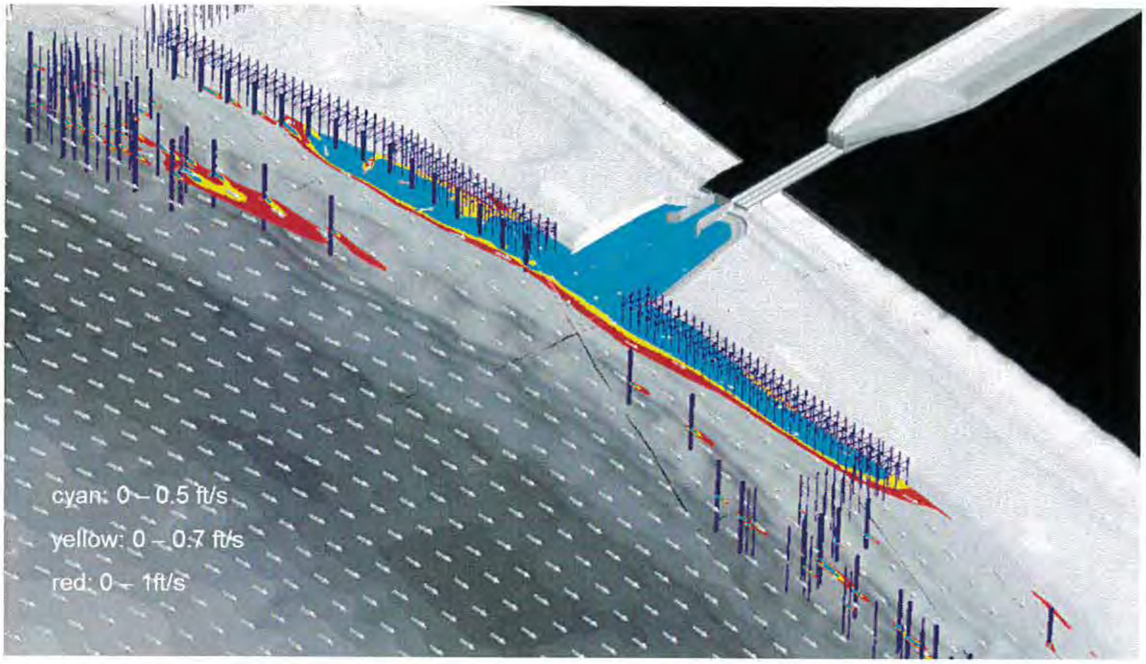


Figure 11 – Iso-surface plot of flow speed with Alternative 1 docking facility and non-operating diversion.

Velocity contours produced with Alternative 2 and an operating diversion are shown in Figure 12. Computed flow patterns are similar to those produced by Alternative 1 except at the entrance of the diversion where flow separation behind the added pilings is observed. Figure 13 is an iso-surface plot of flow speeds colored by different ranges to show areas where the deposition of sediment may occur with the Alternative 2 docking facility and operating diversion. The addition of pilings at the entrance of the diversion may result in localized deposition of sediment in front of the intake.

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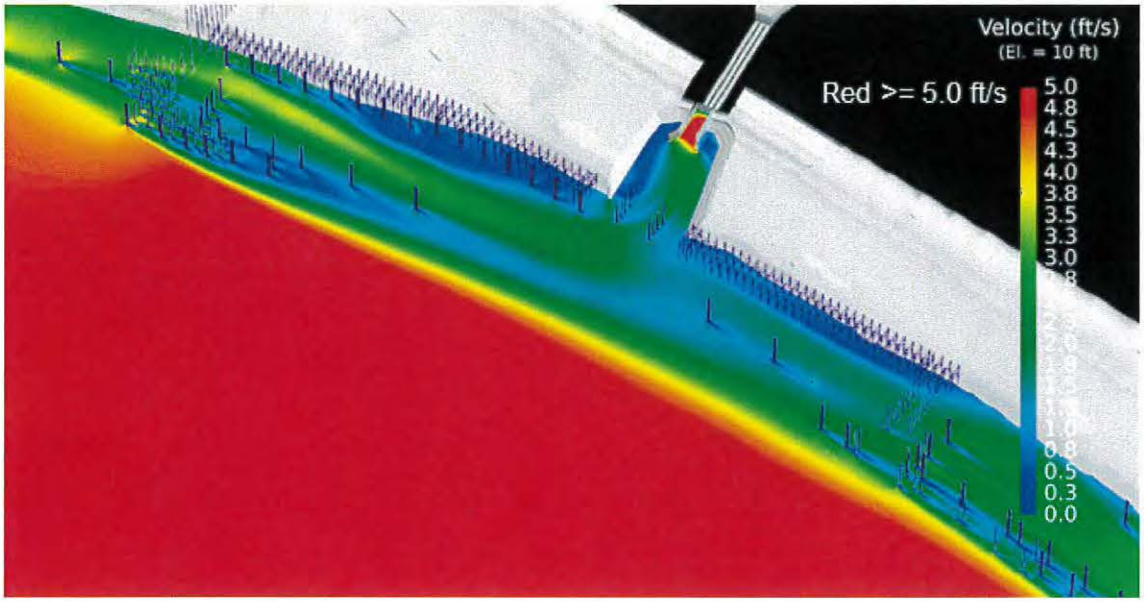


Figure 12 – Velocity contours with Alternative 2 docking facility and operating diversion. Slice plane cut horizontally at an elevation equal to 10 ft.

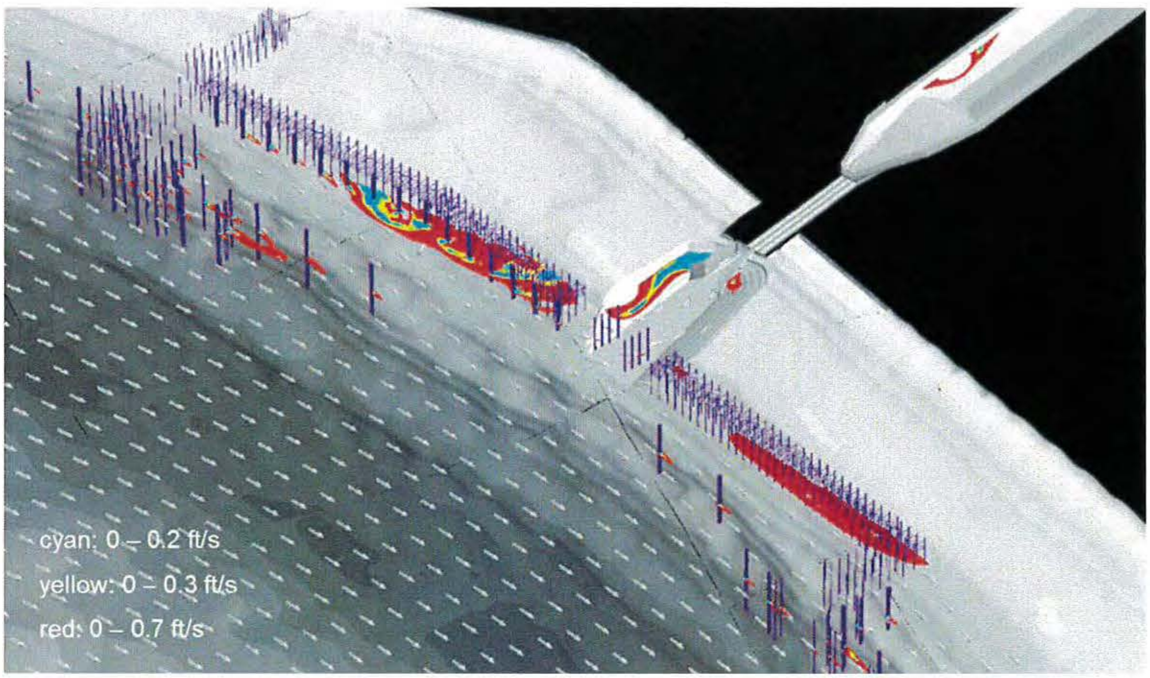


Figure 13 – Iso-surface plot of velocities with different ranges of velocities with Alternative 2 docking facility and operating diversion.

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Additional graphics showing modeled flow patterns for Runs #1, #2, and #3 are provided in Figures 14 through 31.

## 5. Conclusions

This technical memorandum provides a summary of additional modeling used to assess the potential impact of proposed docking facilities (referred to herein as Alternative 1 and Alternative 2) on the flow field in the vicinity of the proposed freshwater diversion intake at Mississippi River Mile 144.2.

For the modeled flow conditions, the proposed docking facility has only minor impacts on the flow of water into the diversion. The results of the analysis indicate that the discharge through the diversion and the water surface elevation in front of the diversion intake are only minimally affected by the proposed construction of the new docking facility. The results of the analysis also show slower velocities in front of and upstream of the diversion intake. This reduction in velocities may result in deposition in front of and upstream of the diversion intake.

## 6. References

Meselhe, E.A., Richardson, J.E., and Lagumbay, R.S., "Simulation of Flow near Proposed Dock Facility and Freshwater Diversion at RM 144.2 Report", May 15, 2015.

## 7. Figures

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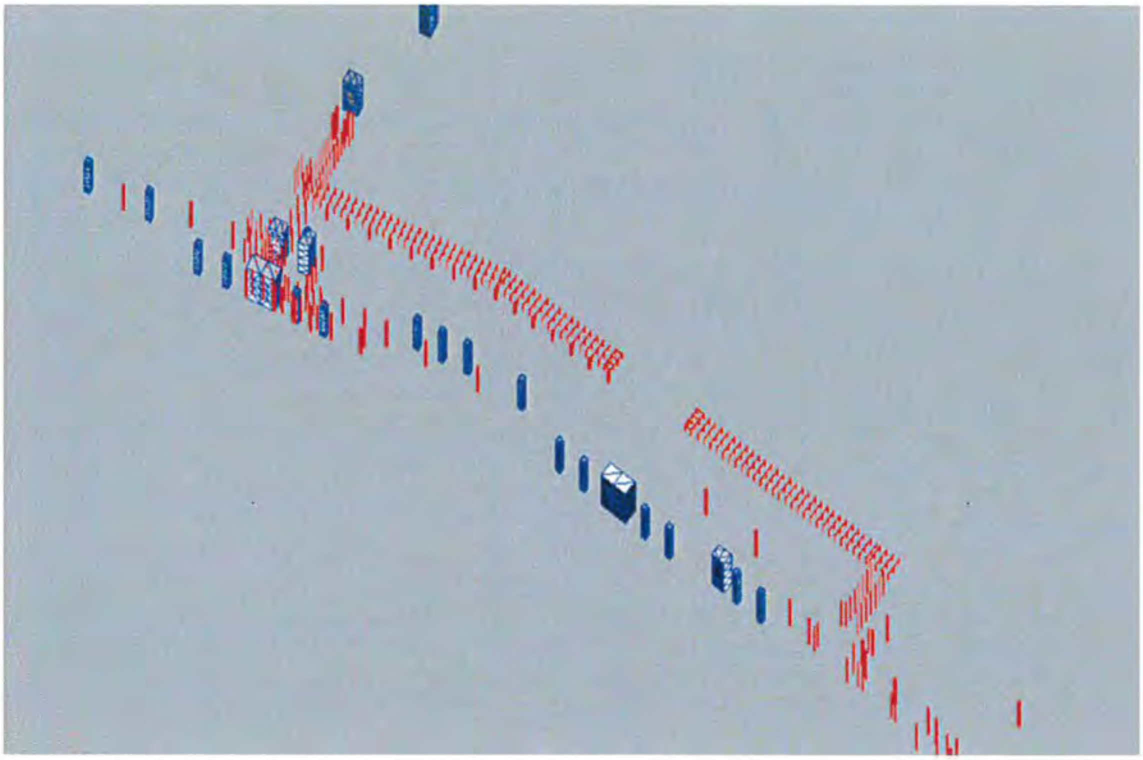

Figure 14 – Previous (blue) and Alternative 1 (red) docking facilities. Alternative 2 (not shown) docking facility is like Alternative 1, but with additional pilings added at the entrance of the diversion.

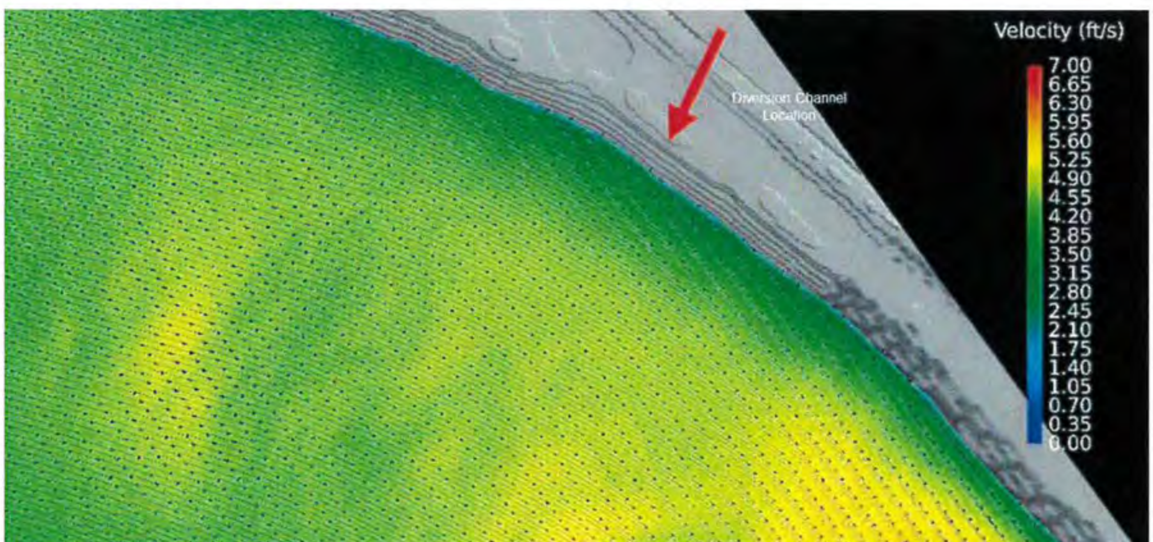


Figure 15 – Velocity contours without the facility and diversion. Red arrow shows the location of the project.

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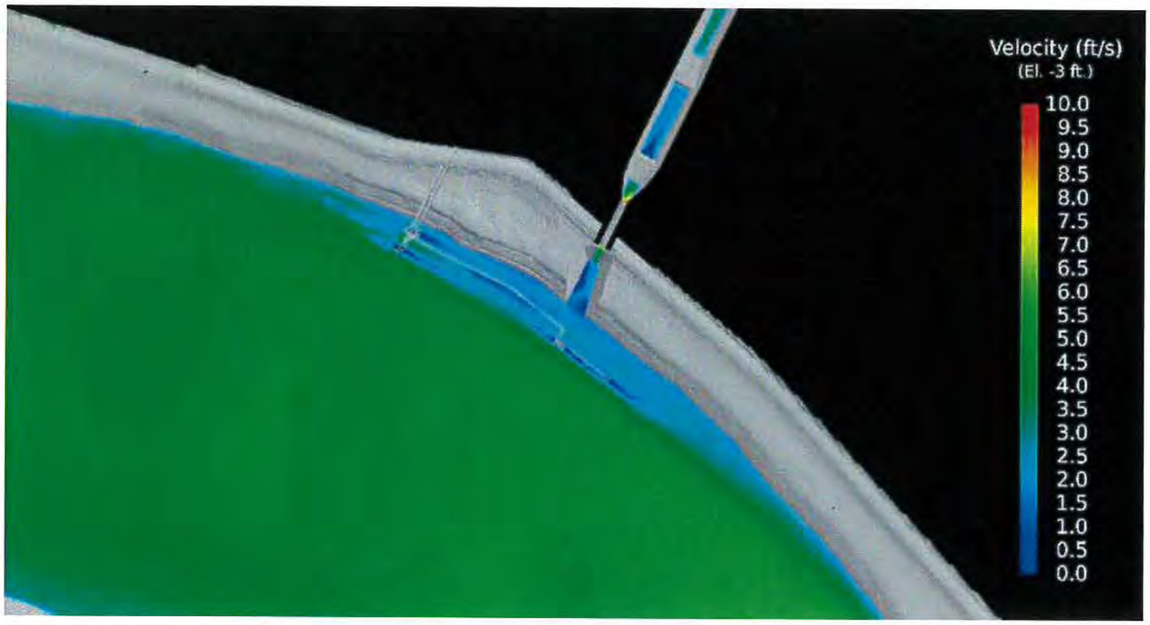


Figure 16 - Velocity contours with previous docking facility and operating diversion. Slice plane cut horizontally at an elevation equal to -3 ft.

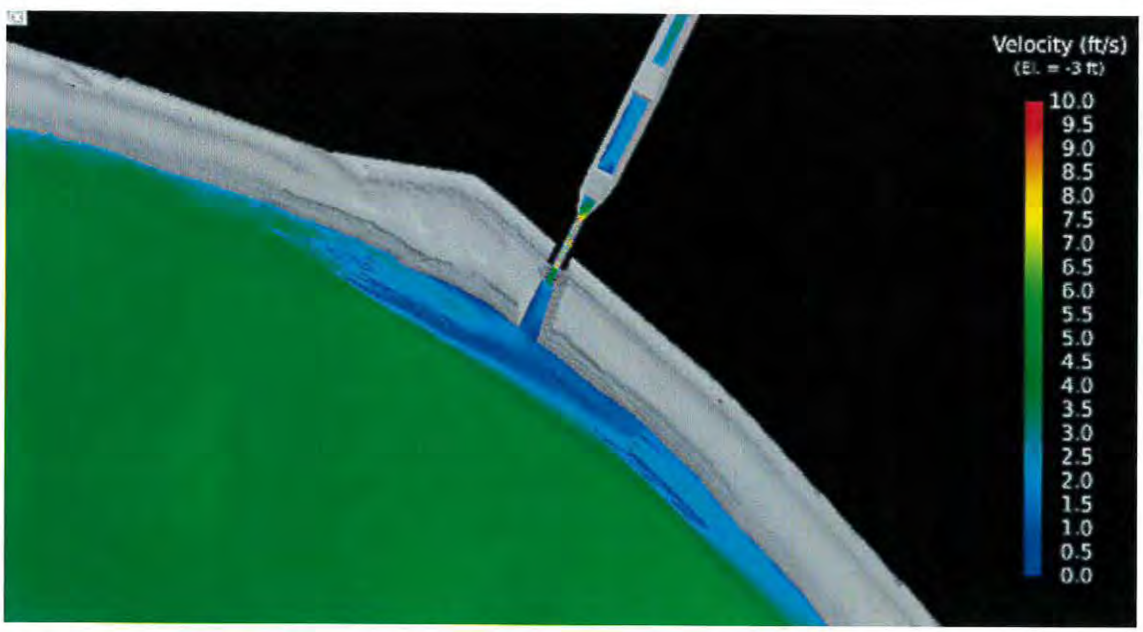


Figure 17 - Velocity contours with Alternative 1 docking facility and operating diversion. Slice plane cut horizontally at an elevation equal to -3 ft.

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Figure 18 – Streamlines colored by velocity with Alternative 1 docking facility and operating diversion.

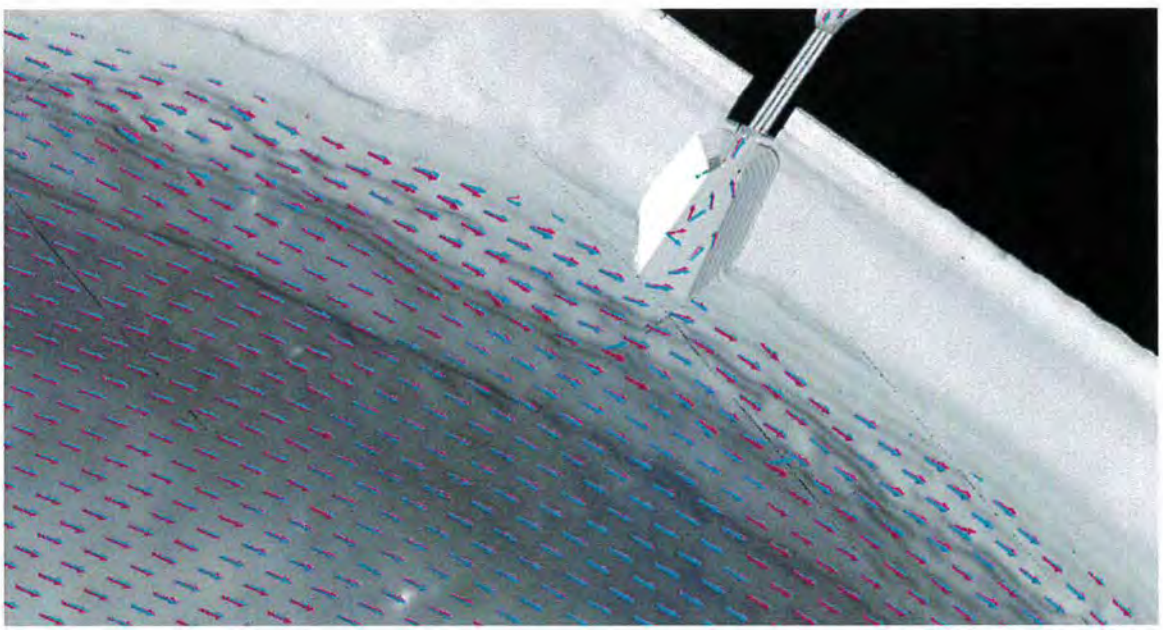

Figure 19 - Comparison of flow vectors between the previous docking facility (cyan) and Alternative 1 (pink) docking facility.

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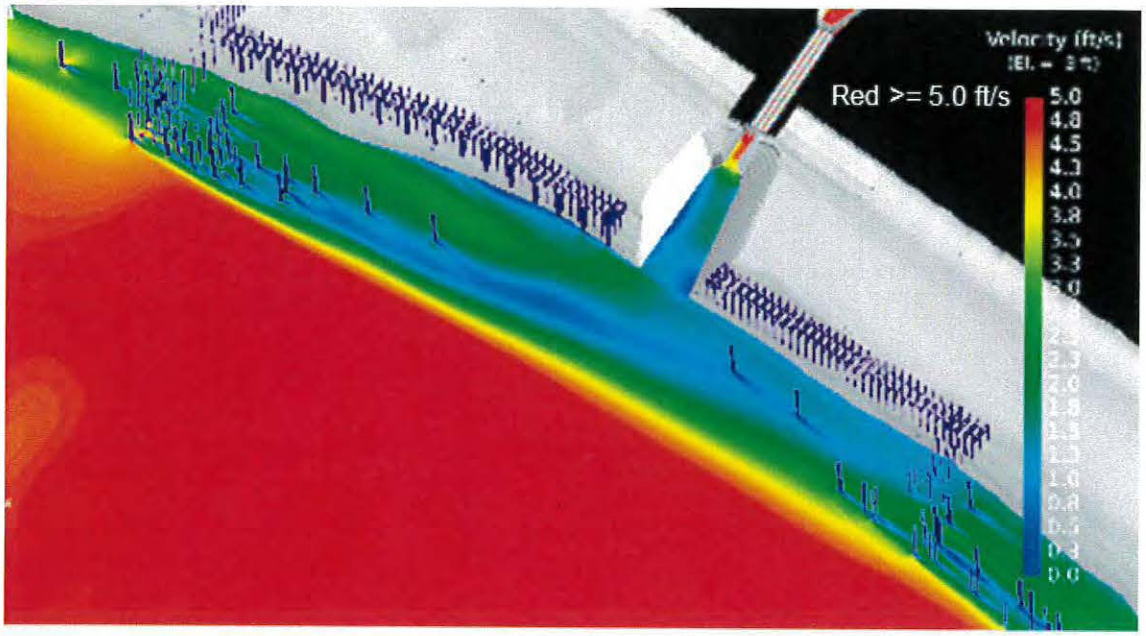


Figure 20 - Velocity contours with Alternative 1 docking facility and operating diversion. Slice plane cut horizontally at an elevation equal to -3 ft.

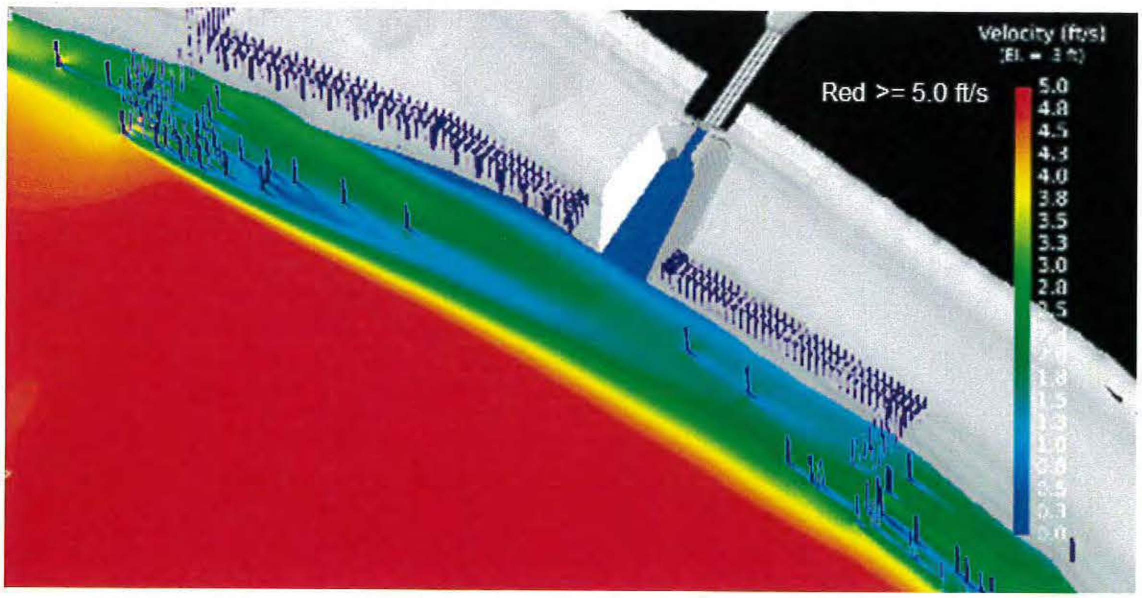


Figure 21 - Velocity contours with Alternative 1 docking facility and non-operating diversion. Slice plane cut horizontally at an elevation equal to -3 ft.

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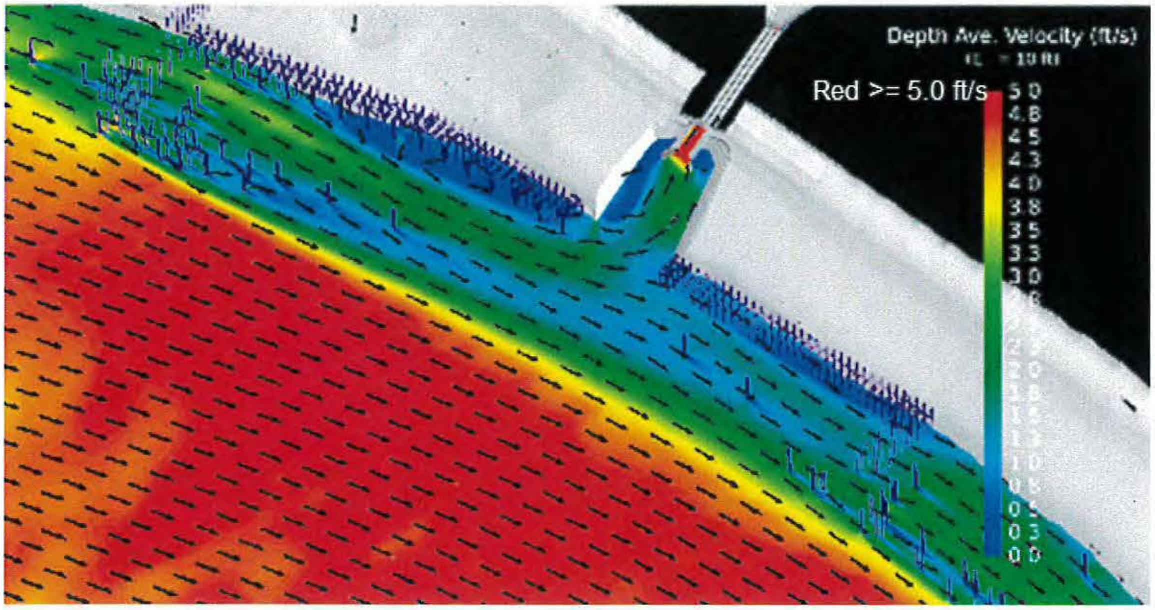


Figure 22 – Depth average velocity contours with vectors superimposed with Alternative 1 docking facility and operating diversion. Not all vectors are shown. Slice plane cut horizontally at an elevation equal to 10 ft.

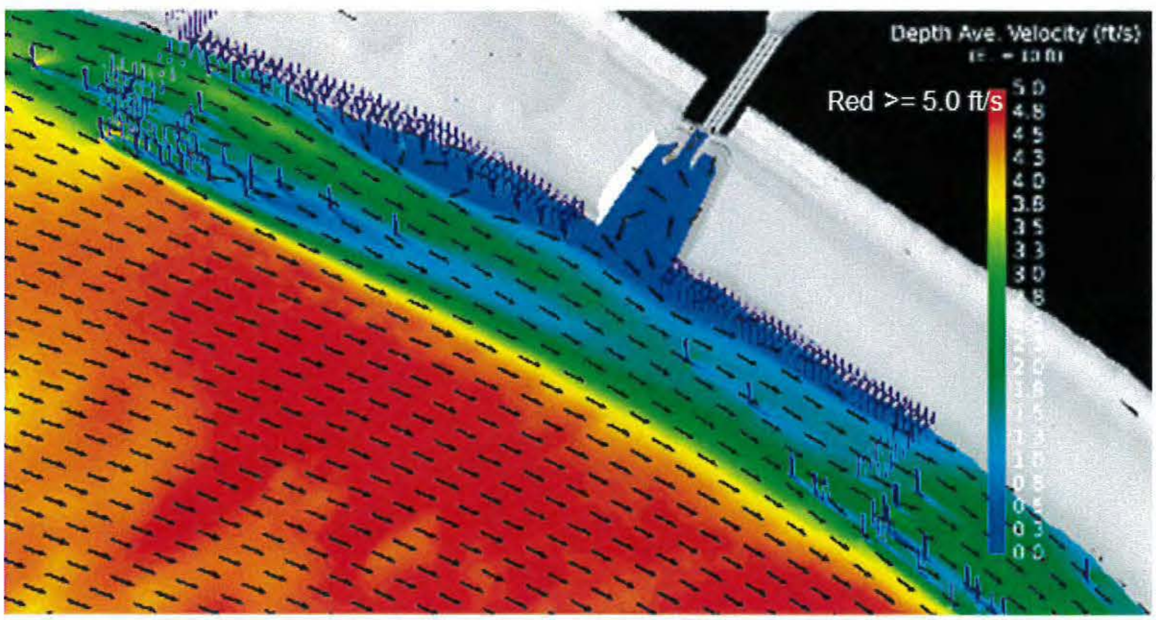


Figure 23 - Depth average velocity contours with vectors superimposed with Alternative 1 docking facility and non-operating diversion. Not all vectors are shown. Slice plane cut horizontally at an elevation equal to 10 ft.

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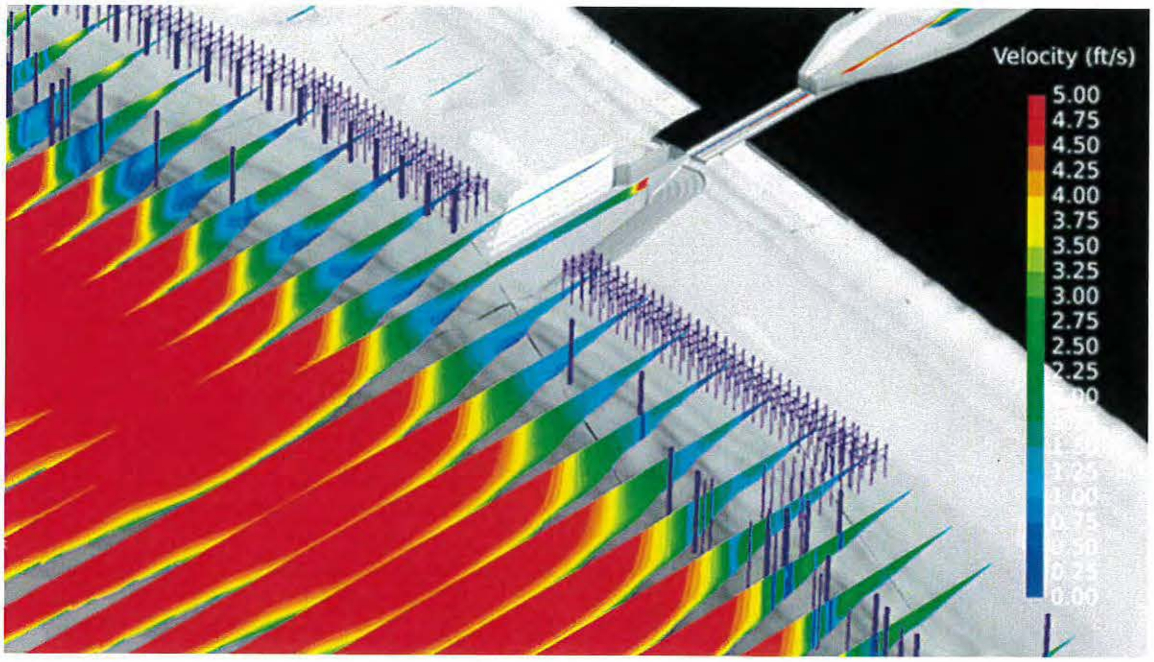


Figure 24 – Velocity contours with Alternative 1 docking facility and operating diversion. Slice plane cut vertically at different sections of the Mississippi river near the facility and entrance of the diversion.

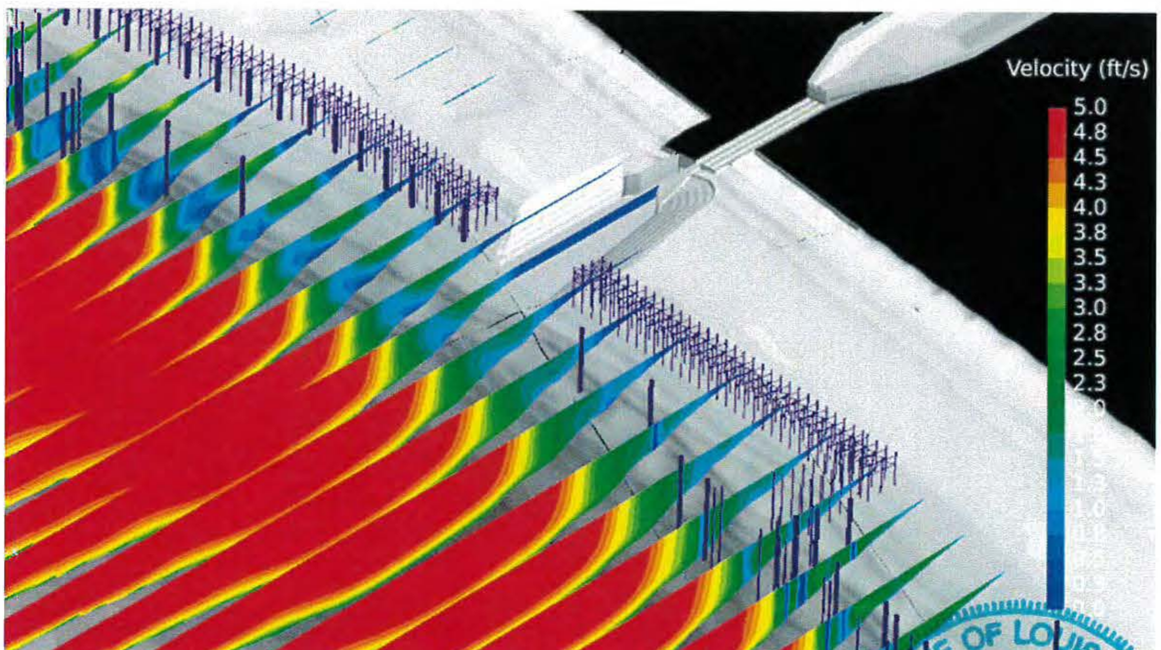


Figure 25 – Velocity contours with Alternative 1 docking facility and non-operating diversion. Slice plane cut vertically at different sections of the Mississippi river near the facility and entrance of the diversion.

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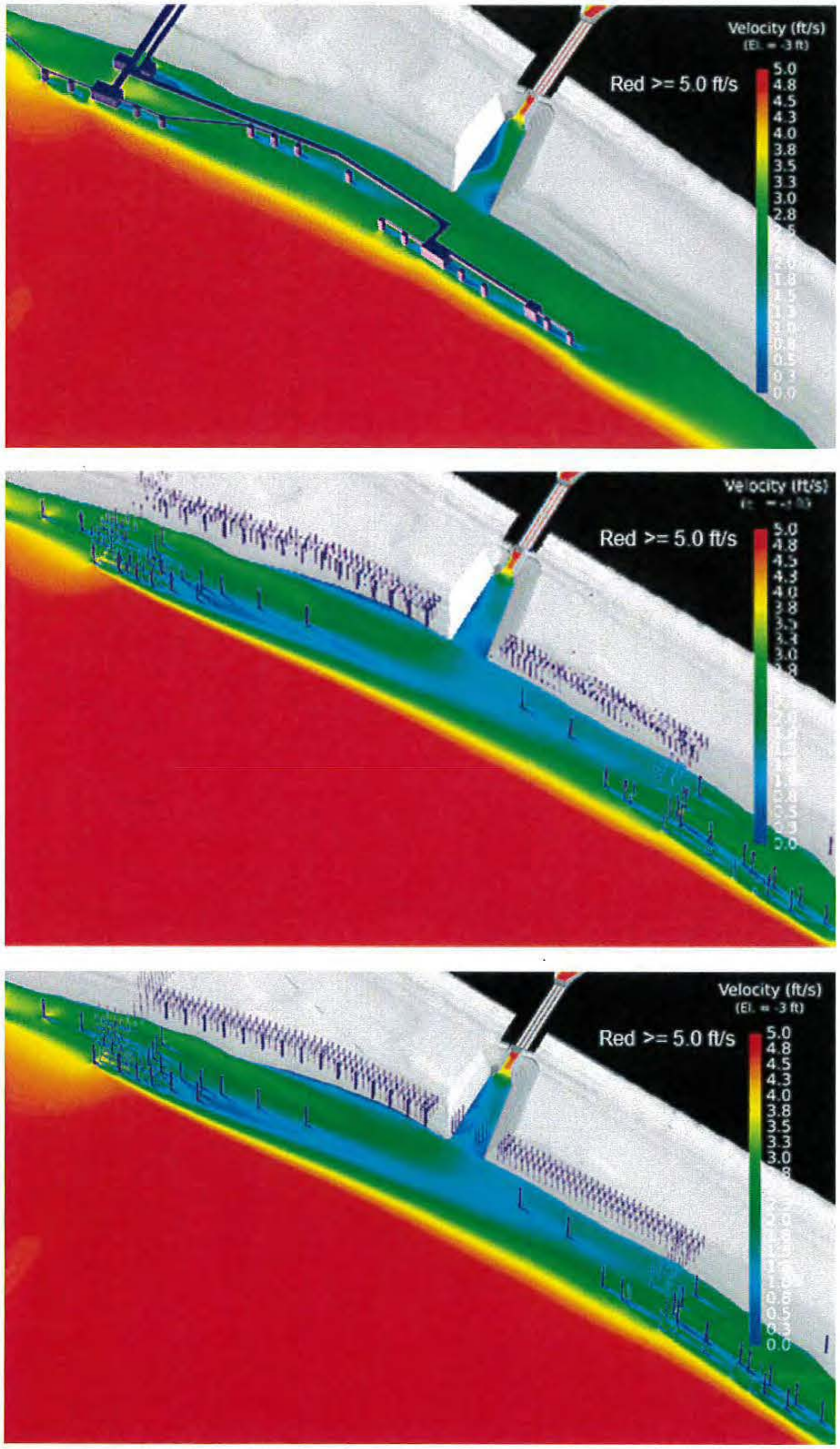


Figure 26 – Comparison of velocity contours. (Top) previous docking facility, (Middle) Alternative 1 docking facility, and (Bottom) Alternative 2 docking facility. Diversion is operating. Slice plane cut horizontally at an elevation equal to -3 ft.

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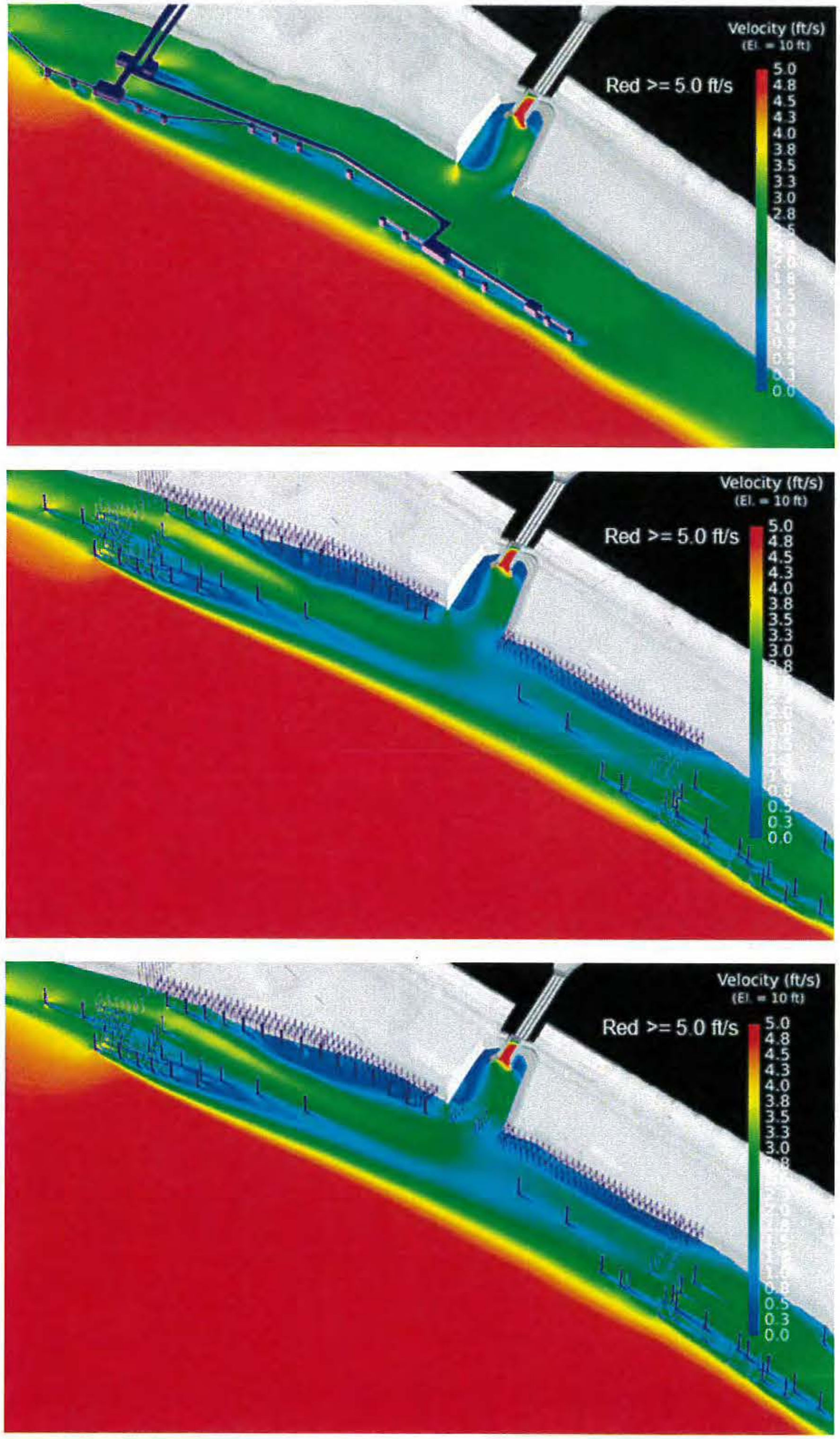


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Figure 27 – Comparison of velocity contours. (Top) previous docking facility, (Middle) Alternative 1 docking facility, and (Bottom) Alternative 2 docking facility. Diversion is operating. Slice plane cut horizontally at an elevation equal to 10 ft.

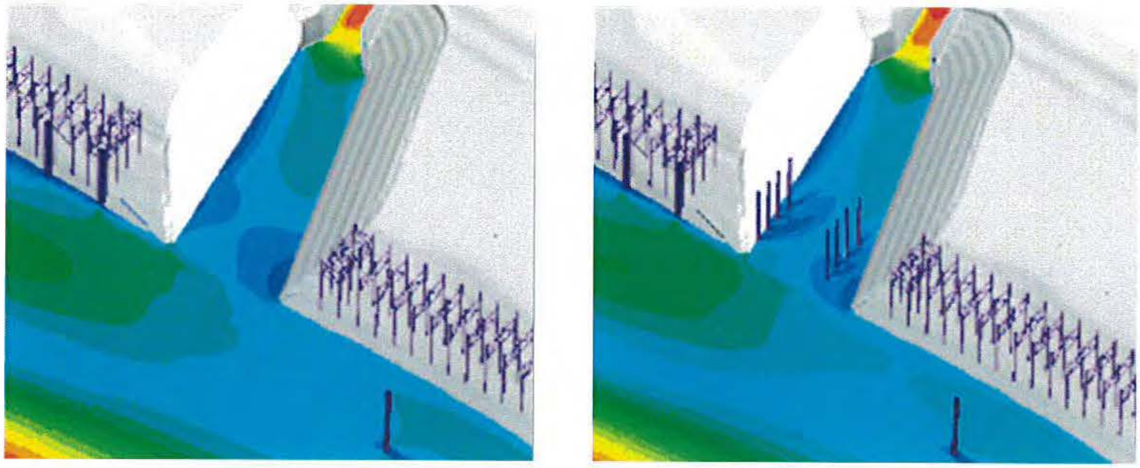

Figure 28 – Comparison of velocity contours at the entrance of the diversion channel between Alternative 1 (left) and Alternative 2 (right) docking facilities. Slice plane cut horizontally at an elevation equal to -3 ft.

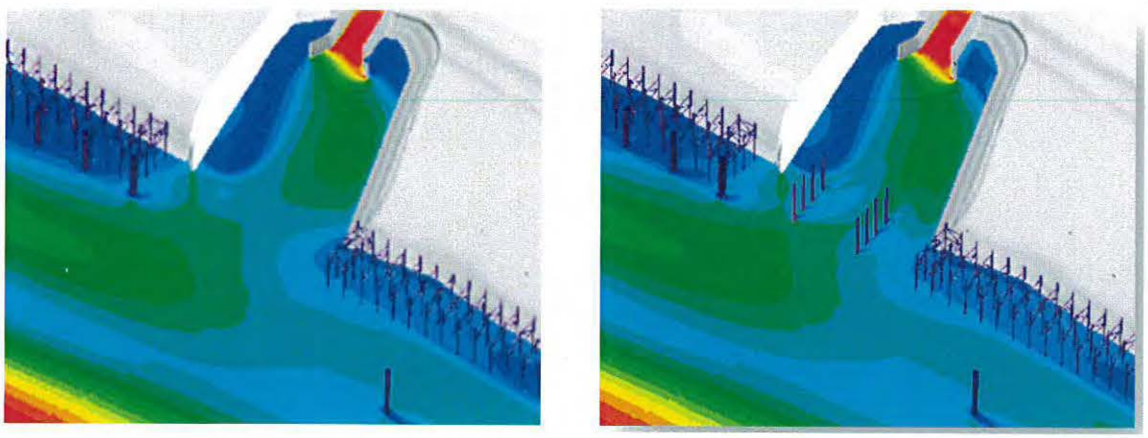

Figure 29 – Comparison of velocity contours at the entrance of the diversion channel between Alternative 1 (left) and Alternative 2 (right) docking facilities. Slice plane cut horizontally at an elevation equal to 10 ft.

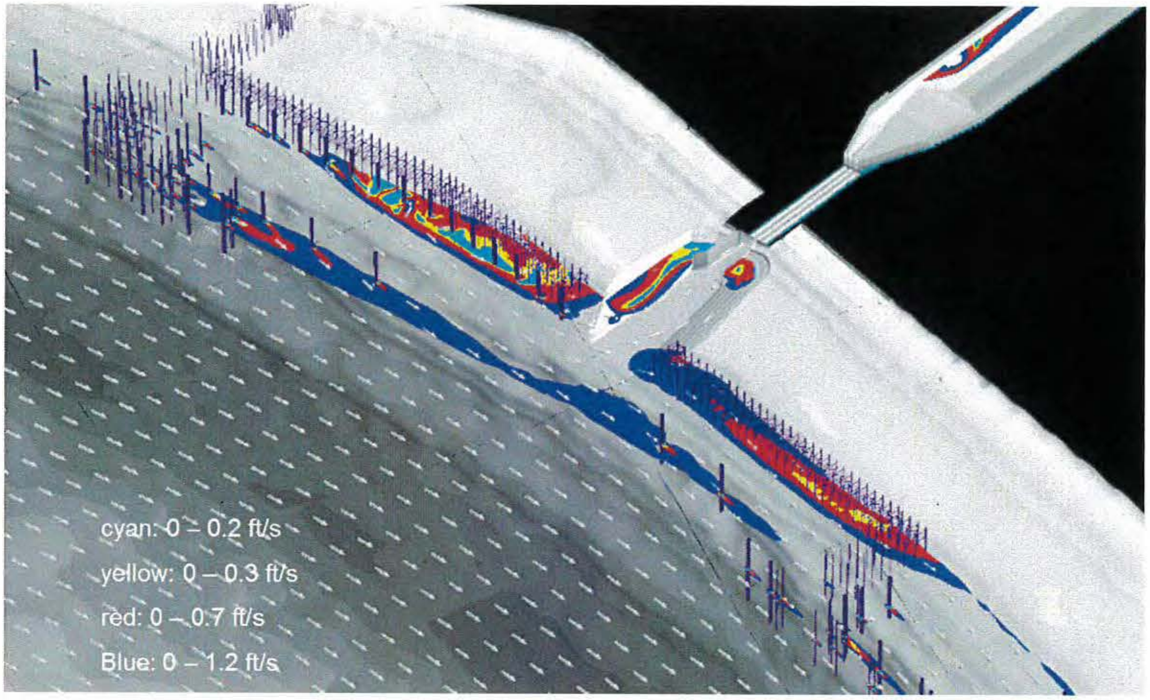


Figure 30 – Iso-surface plot of flow speeds within different ranges - Alternative 1 docking facility and operating diversion.

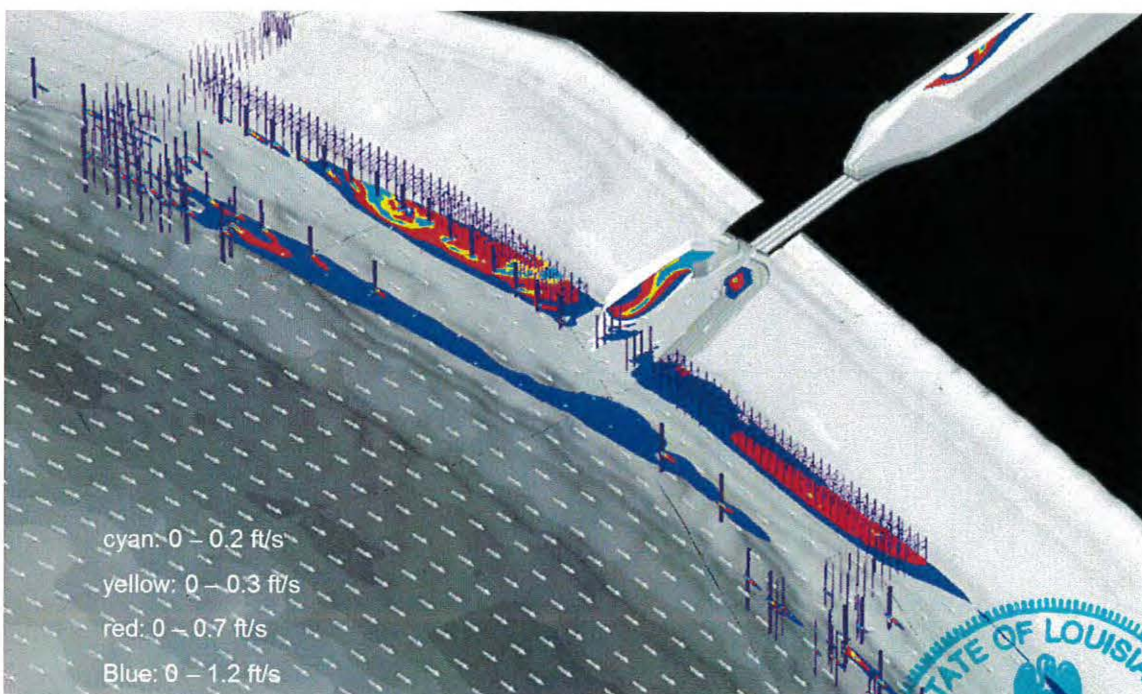


Figure 31 – Iso-surface plot of flow speeds within different ranges - Alternative 2 docking facility and operating diversion.

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# HEC-RAS Model of Conveyance Channel for Relative Sea Level Rise

# STATE OF LOUISIANA
# COASTAL PROTECTION AND RESTORATION AUTHORITY

## RIVER REINTRODUCTION INTO MAUREPAS SWAMP AND WEST SHORE LAKE PONTCHARTRAIN FLOOD RISK REDUCTION PROJECT PO-0029
## LAGOV NO. 4400019214

# HEC-RAS Model of Conveyance Channel - Incorporation of Relative Sea Level Rise

## For

**Prepared By:**
**AECOM Technical Services**
**1555 Poydras Street, Suite 1200**
**New Orleans, LA 70112**

## December 8, 2021

| Rev | Date | Description |
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# HEC-RAS MODEL OF THE MAUREPAS DIVERSION FOR THE RIVER REINTRODUCTION INTO MAUREPAS SWAMP AND WEST SHORE LAKE PONTCHARTRAIN FLOOD RISK REDUCTION PROJECT PO-0029 INCORPORATION OF RELATIVE SEA LEVEL RISE

## Introduction

At the direction of the Coastal Protection and Restoration Authority (CPRA), AECOM is designing the Maurepas diversion to operate at 2,000 cfs for as much of the year as possible. The diversion system includes: 1) a 5½ mile Conveyance Channel, 2) an Intake Structure, including culverts crossing under LA 44, 3) culverts at the CN Railroad, 4) a bridge at the KCS Railroad, 5) culverts at US 61, and 6) the existing section with bridge piers crossing beneath I-10.

A 1D model of the diversion was created in HEC-RAS 4.0 and submitted to CPRA in 2013. Subsequently, in advancing the development of the design, the geometry of the Conveyance Channel has changed due to the impact of the West Shore Lake Pontchartrain (WSLP) Flood Risk Reduction Project. Revisions to the channel alignment and cross-sections, as well as changes to the structures at the railroad and roadway crossings have been made to the current 15% Design. The rationale for these revisions is documented in the 15% BODR report.

To incorporate the changes in the updated design, a new hydraulic model was constructed in the current version of HEC-RAS 6.1. The model simulates a steady-state, sub-critical flow condition to verify the hydraulic performance of the proposed diversion design. Two phenomena have occurred over the recent past that will impact the performance of the Maurepas Diversion: 1) the stage of the Mississippi River (MR) has been higher for longer over recent years, and 2) the global sea level is rising. The potential effect of the higher MR stage due to more significant snowmelt along with the increased intensity of rainfall during storm events will be developed in a subsequent section of this report. The topic of this report is the effect of Sea Level Rise (SLR) on the Maurepas Diversion.

## Effect of Sea Level Rise

The earth's climate is changing, with significant warming occurring that is currently raising the sea level around the globe. The levels of SLR have been documented and extrapolated to predict the future water levels of the seas around the world. Both the upstream and downstream ends of the Maurepas Diversion are connected to the Gulf of Mexico (GOM), and thence to the Atlantic Ocean and the Caribbean Sea. The MR makes a circuitous trip from the diversion intake at River Mile (RM) 144, through New Orleans, and discharges primarily through various passes, as tabulated in Table 1, into the GOM.

**Table 1 - MR Discharge Division[1]**

| Location | % of Flow |
|---|---|
| Southwest Pass | 40% - 50% |
| South Pass | 10% |
| Cubits Gap | 5% - 20% |
| Pass A Loutre | 10% |
| Grand Pass | 8% - 10% |
| Baptiste Collette | 5% - 15% |

1. Adapted from: *Numerical Modeling of the Lower Mississippi River-Influence of Forcings on Flow Distribution and Impact of Sea Level Rise on the System*, Conference Paper, E. Karadogan, C. S. Willson, and C. R. Berger, November 2009

As shown in Table 2, the Intake of the Maurepas Diversion is 144 River Miles Above Head of Passes, significantly upstream of the river's outfall.

**Table 2 – River Mile from Maurepas Diversion to Southwest Pass[1]**

| River Mile | Location |
|---|---|
| 144 | Maurepas Diversion Intake |
| 102.8 | Carrollton - New Orleans, LA |
| 98.3 | Harvey Lock |
| 92.7 | Inner Harbor Navigation Canal Lock |
| 91 | Chalmette, LA |
| 88.3 | Algiers Lock |
| 76.6 | Near Braithwaite, LA |
| 62.5 | Alliance, LA |
| 48.7 | W. Pointe A La Hache, LA |
| 39.3 | Port Sulphur, LA |
| 29.5 | Empire, LA |
| 10.7 | Venice, LA |
| -0.6 | Head of Passes |
| -17.4 | Southwest Pass |

1. Adapted from: *Numerical Modeling of the Lower Mississippi River-Influence of Forcings on Flow Distribution and Impact of Sea Level Rise on the System*, Conference Paper, E. Karadogan, C. S. Willson, and C. R. Berger, November 2009

As shown in Figure 1, the Maurepas Swamp is directly connected to Lake Maurepas, which is connected to Lake Pontchartrain, thence to the Rigolets, which connects to Lake Borgne, which is connected to the GOM. As Figure 1 illustrates, the Maurepas Swamp to GOM route is much more direct than the MR route. The more direct connection results in the WSE in the Maurepas Swamp being tidally influenced, whereas the tides are not a significant factor on the river stage 144 miles upriver from the GOM. The MR is also flowing (typical range is 400,000 cfs to 1,000,000 cfs), unlike the relatively still water in the swamp.

In their paper cited above, Karadogan, Willson, and Berger note that the effect of the SLR is most significant in the lower 20 miles of the river and it loses its effect with increasing flow rates. Thus, at the Maurepas Diversion intake, which is 144 miles upriver, the effect of SLR is negligible. Therefore, in the following analyses, SLR is not accounted for as an impact on the MR stage; however, SLR has a significant effect on the WSEL in the Maurepas Swamp, which increases approximately 2.1-ft over 50 years. Thus, the river stage, or headwater elevation, is not expected to be affected by SLR, but the swamp WSE, or tailwater elevation, is expected to rise steadily.

Table 3 shows the USACE's Middle value of projected Sea Level Rise for each year up to 2075.

**Figure 1 – Map of Area from Maurepas Diversion to the Gulf of Mexico**

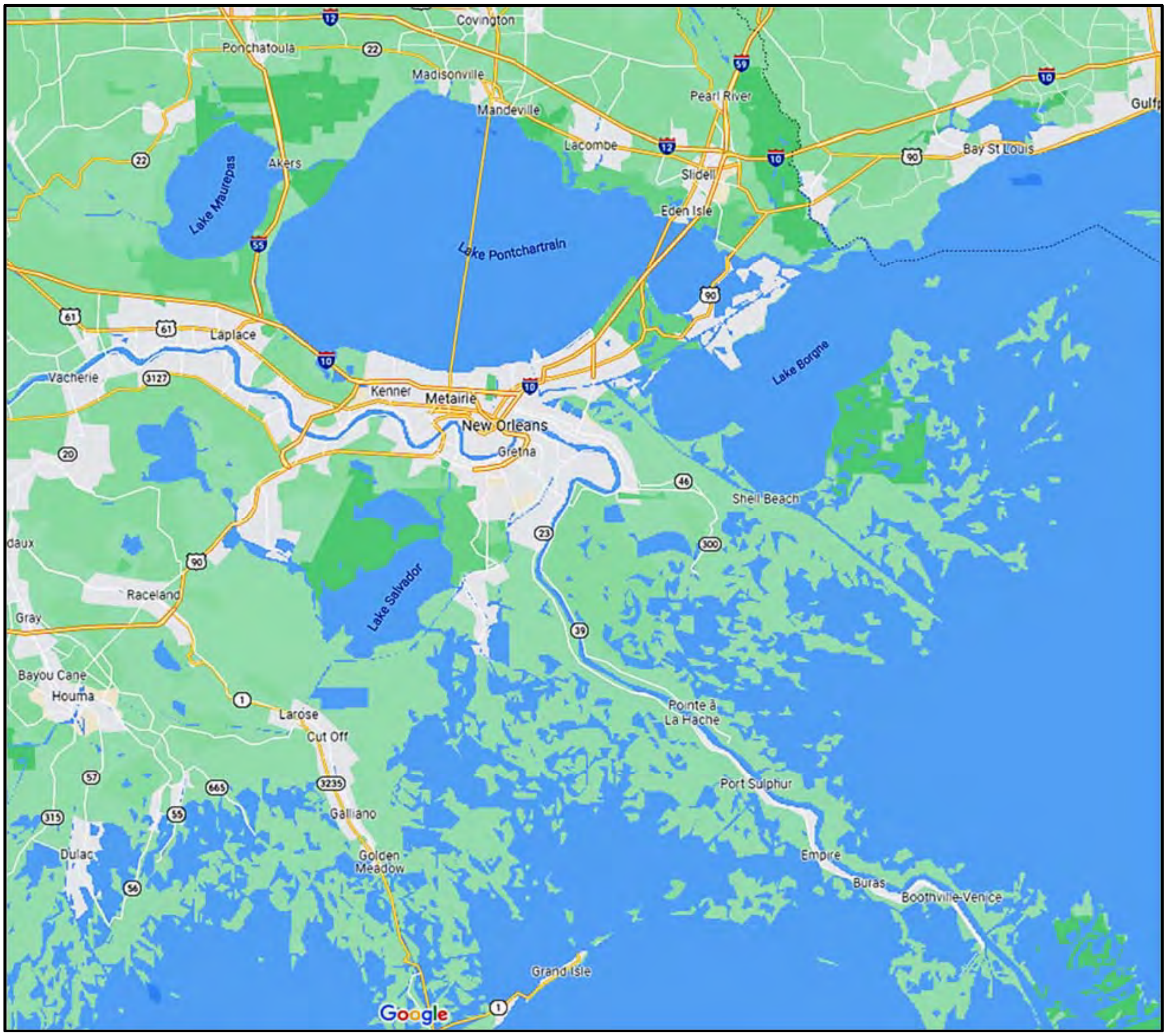


**Table 3 – Projected Sea Level Rise per Year[1]**

| Year | SLR (ft) |
|---|---|
| 2020 | 0 |
| 2025 | 0.2 |
| 2030 | 0.3 |
| 2035 | 0.5 |
| 2040 | 0.7 |
| 2045 | 0.9 |
| 2055 | 1.3 |
| 2060 | 1.5 |
| 2065 | 1.7 |
| 2070 | 1.9 |
| 2075 | 2.1 |

1. Data from USACE Middle Sea Level Rise Estimate

Table 4 indicates the projected WSEL in the Maurepas Swamp per year based on the HEC-RAS model, including the effect of the various SLR values for various Maurepas Diversion flows.

**Table 4 – WSEL in the Maurepas Swamp for Given Diversion Flows[1]**

| SLR (ft) | 0 | 0.2 | 0.3 | 0.5 | 0.7 | 0.9 | 1.1 | 1.3 | 1.5 | 1.7 | 1.9 | 2.1 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Flow (cfs)** | **2020** | **2025** | **2030** | **2035** | **2040** | **2045** | **2050** | **2055** | **2060** | **2065** | **2070** | **2075** |
| **50** | 1.52 | 1.72 | 1.82 | 2.02 | 2.22 | 2.42 | 2.62 | 2.82 | 3.02 | 3.22 | 3.42 | 3.62 |
| **100** | 1.58 | 1.78 | 1.88 | 2.08 | 2.28 | 2.48 | 2.68 | 2.88 | 3.08 | 3.28 | 3.48 | 3.68 |
| **250** | 1.76 | 1.96 | 2.06 | 2.26 | 2.46 | 2.66 | 2.86 | 3.06 | 3.26 | 3.46 | 3.66 | 3.86 |
| **500** | 2.03 | 2.23 | 2.33 | 2.53 | 2.73 | 2.93 | 3.13 | 3.33 | 3.53 | 3.73 | 3.93 | 4.13 |
| **1000** | 2.53 | 2.73 | 2.83 | 3.03 | 3.23 | 3.43 | 3.63 | 3.83 | 4.03 | 4.23 | 4.43 | 4.63 |
| **1500** | 2.97 | 3.17 | 3.27 | 3.47 | 3.67 | 3.87 | 4.07 | 4.27 | 4.47 | 4.67 | 4.87 | 5.07 |
| **2000** | 3.37 | 3.57 | 3.67 | 3.87 | 4.07 | 4.27 | 4.47 | 4.67 | 4.87 | 5.07 | 5.27 | 5.47 |

1. WSEL's from HEC-RAS model of the Maurepas Diversion, including Data from USACE Middle Sea Level Rise Estimate

Table 5 tabulates the Mississippi River stages required to achieve given Maurepas Flows, including the effect of the various SLR values.

**Table 5 – Mississippi River Stage Required to Achieve Given Diversion Flows[1]**

| Flow (cfs) | 2020 | 2025 | 2030 | 2035 | 2040 | 2045 | 2050 | 2055 | 2060 | 2065 | 2070 | 2075 |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **50** | 2.82 | 2.82 | 2.82 | 2.82 | 2.82 | 2.82 | 2.82 | 2.86 | 3.05 | 3.25 | 3.44 | 3.64 |
| **100** | 3.03 | 3.03 | 3.03 | 3.03 | 3.03 | 3.03 | 3.03 | 3.03 | 3.14 | 3.33 | 3.52 | 3.72 |
| **250** | 3.49 | 3.49 | 3.49 | 3.49 | 3.49 | 3.49 | 3.49 | 3.49 | 3.52 | 3.68 | 3.85 | 4.00 |
| **500** | 4.13 | 4.13 | 4.13 | 4.13 | 4.13 | 4.13 | 4.13 | 4.13 | 4.18 | 4.30 | 4.45 | 4.62 |
| **1000** | 5.30 | 5.30 | 5.30 | 5.31 | 5.30 | 5.32 | 5.40 | 5.50 | 5.62 | 5.75 | 5.89 | 6.02 |
| **1500** | 6.59 | 6.66 | 6.70 | 6.78 | 6.87 | 6.98 | 7.08 | 7.20 | 7.31 | 7.43 | 7.57 | 7.71 |
| **2000** | 8.53 | 8.62 | 8.66 | 8.75 | 8.85 | 8.94 | 9.05 | 9.17 | 9.29 | 9.42 | 9.56 | 9.68 |

1. WSEL's from HEC-RAS model of the Maurepas Diversion, including Data from USACE Middle Sea Level Rise Estimate

Figure 2 illustrates the projected number of days that the Maurepas Diversion can operate at its full capacity of 2,000 cfs, both with and without Sea Level Rise. At the current time, without SLR, the diversion would require a River Stage of 8.53-ft to be able to convey 2,000 cfs. It could operate for approximately 180 days per year at that capacity. Projecting into the future, considering the effect of SLR, in the year 2075 the diversion will require a River Stage of 9.68-ft. In that future year the diversion will only be able to convey the target 2,000 cfs for approximately 161 days per year.

**Figure 2 – Annual 2,000 cfs Flow Duration, with and without SLR**