# Volume IV

# APPENDIX L:

# Engineering

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## L1 SUMMARY

## L1.1 Project Scope and Objectives

The Blind River Fresh water diversion Project is to divert water from the Mississippi river for a distance of approximately 3 miles to the St. James Parish drainage system that will be modified to allow the diverted flows to provide freshwater, nutrients and sediment to the Maurepas swamp. The project area is approximately 35 square miles and extends from the St. James Parish drainage system which is north and parallel to LA 3125 to Interstate 10 on the north. The Blind river flows east and then north through the project area.

The engineering components to complete the project include:

- Intake structure with screens on the bank of the Mississippi river
- Culverts with motor operated sluice gates through the flood control levee and under LA 44
- Earthen diversion channel from north of LA 44 to the St. James Parish Drainage System.
- Relocation and reconstruction of the Canadian National Railroad and LA 1325 to cross the transmission channel
- Swamp modifications to include gapping existing berms to allow distribution of flow through the swamp
- St. James parish channel modifications to distribute flows and to restrict flow from short circuiting directly to the Blind River
- Control structures to allow distribution options within the St. James drainage system

The overall objective of the Small Diversion at Convent/Blind River Project was to reverse the trend of deterioration of southeast Maurepas Swamp and Blind River.

### Specific Project Objectives

- **Objective 1: Promote water distribution in the swamp**

  **Target for Objective 1**: Increase the area of freshwater inundation for low to average flood events by 10 to 25% from existing conditions to increase swamp productivity and wetland assimilation. Increase nutrient input to the swamp to increase swamp productivity as measured by a 5 to 10% annual increase in the diameter at breast height (dbh) of bald cypress and tupelo from existing conditions, and increase wetland assimilation as measured by a 10 to 25% decrease in the average TN and TP in Blind River and a 5 to 10% increase in the average dissolved oxygen in Blind River from existing conditions.

- **Objective 2: Facilitate swamp building** at a rate greater than swamp loss due to subsidence and sea level rise.

  **Target for Objective 2**: Increase swamp productivity, as described above and by increasing sediment input by up to 1,000 grams per square meter per year in order to decrease the annual subsidence rate 50 to 100% in the swamp.

- **Objective 3: Establish hydroperiod fluctuation in the swamp** to improve bald cypress and tupelo productivity and their seeding germination and survival.

  **Target for Objective 3**: Decrease flood duration in the swamp by 10 to 25% for high flood events, increasing the length of dry periods in the swamp (no standing water) by 10 to 25%, and by increasing the number of bald cypress and tupelo saplings per acre by 25 to 50% from existing conditions.

- **Objective 4: Improve fish and wildlife habitat in the swamp and in Blind River**

  **Target for Objective 4**: Increase the existing Wetland Value Assessment (WVA) Habitat Suitability Index (HSI) in the swamp by 10 to 25% five years after project implementation and by a 5 to 10% increase in the average dissolved oxygen in Blind River from existing conditions.

## L1.2 Alternatives Analyzed

There were 5 major alternatives analyzed after screening down from an initial array of 15 alternatives. The final array of alternatives is described as follows:

- No Action (required to establish baseline conditions and the need for a diversion)
- Alternative 2 – 3000 cfs Diversion at Romeville (Gated Culvert System)
- Alternative 4 – 3000 cfs Diversion at South Bridge (Gated Culvert System)
- Alternative 4B – 3000 cfs Diversion at South Bridge with split flows (Gated Culvert System)
- Alternative 6 – Two 1500 cfs Diversions at Romeville and South Bridge (Siphons)

## L1.3 Analysis Approach

The approach to the analysis was to focus on the project objectives and to develop a system to analyze the diversion flow quantity, quality and distribution to determine the effects on the swamp. The analysis involved analyzing existing conditions and then applying the alternatives at various flow rates to determine the effectiveness of each alternative at specific flow rates.

After the first round of analysis it was determined that the most cost effective flows for final analysis was approximately 3000 cfs. The Tentatively Selected Plan (TSP) is based on the 3000 cubic feet per second flow rate.

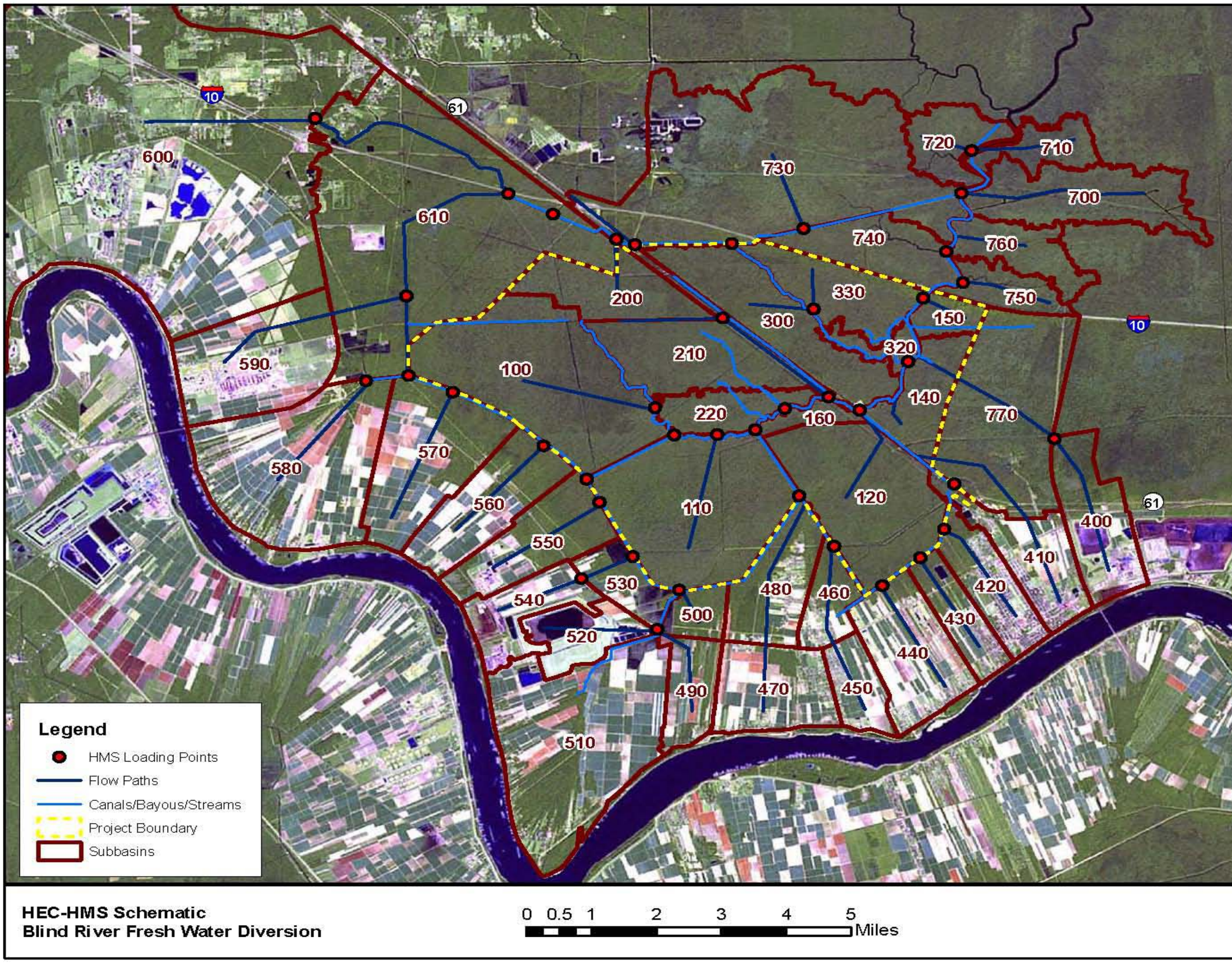
The alternatives in the final array were compared based on benefits, costs, and impacts. Alternative 2 is the least expensive with a first cost of about $102 million. Alternative 6 is the most expensive at over $155 million. Alternatives 4 and 4B are slightly less expensive than Alternative 6 at $152.2 million and $146.9 million, respectively.

Although Alternative 6 provides the greatest number of environmental benefits in terms of AAHUs estimated using the WVA process. Alternative 2 provides over 90% of the benefits for about 67% of the cost of Alternative 6. The cost per AAHU is much lower for Alternative 2 that for the other three alternatives and the incremental cost per habitat unit in going from Alternative 2 to Alternative 4B and/or Alternative 6 is quite high. Another factor to consider is that Alternative 2 impacts the smallest number of wetland acres. Accordingly, Alternative 2 is the alternative that reasonably maximizes ecosystem restoration benefits compared to costs and is designated as the National Ecosystem Restoration Plan and the Tentatively Selected Plan.

## L1.4 Analysis Results

The results of the analysis yielded flow volumes that are distributed through specific hydrologic areas of the swamp. The following sections contain the results by each of the hydrographic areas shown in **Figure L1.4-1.**

Figure L1.4.1 Hydrographic Units



## L2 HYDROLOGY, HYDRAULICS, AND WATER QUALITY

### L2.1 Introduction

The objective of the Convent/Blind River Small Diversion project is to introduce freshwater, sediment, and nutrients from the Mississippi River into the southeast portion of the Maurepas swamp to improve biological productivity that will facilitate organic deposition in the swamp, and prevent further deterioration. In order to determine the most feasible solution to meeting the above objective, the existing hydrologic, hydraulic, and water quality characteristics have been compared to proposed future conditions (alternatives evaluation). To complete this task, CDM has applied the following tools:

- *Hydrologic Engineering Center River Analysis System (HEC-RAS)* – A dynamic hydraulic model used to calculate flow routing through the drainage canals and swamp storage within the study area. The HEC models work in tandem with HEC-HMS providing flow inputs to HEC-RAS, which simulates dynamic flow and storage through the study area. Results from HEC-RAS were utilized in a variety of ways to support the project analyses, including evaluation of swamp hydroperiod, evaluation of potential project improvements, and confirmation that the project will not adversely impact flooding outside of the project boundary.
- *Hydrologic Engineering Center Hydrologic Modeling System (HEC-HMS)* – A model used to quantify surface water hydrology for the project area and tributary watershed by simulating and calculating the rainfall-runoff process. From a systems perspective, the watershed runoff process is a portion of the hydrologic cycle, and the primary focus for quantifying water that flows through elements of the study area such as drainage canals and stream channels.
- *Environmental Fluid Dynamics Code (EFDC)* – A two dimensional (2D) hydrodynamic and water quality model used to analyze the effects of the freshwater diversion to the Blind River and associated wetlands with consideration of nutrients.
- *Engineering Calculations* – A set of standard engineering equations for daily runoff estimation and water balance was developed for evaluating long-term conditions in a networked system. The engineering calculations were further used as a supplemental way of cross checking of the HEC models by reproducing similar trends for runoff and water level dynamics using independent techniques and reasonable parameterization.

These analyses were used to evaluate different levels of detail to support feasibility and design level decisions as well as allow for quality management checking and evaluation of different flow rate capacities, diversion locations, and operational scenarios. The following eleven subsections are included in support of the hydrologic, hydraulic, and water quality evaluations performed for existing conditions as well as future project alternative conditions:

- Model Study Methodologies (Section L2.2);

- Watershed Hydrology and Hydraulics Analysis (Section L2.3);
- Swamp Hydroperiod Analysis (Section L2.4);
- Hydrodynamic and Water Quality Analysis (Section L2.5);
- Hydraulic Analysis of Romeville Diversion and Transmission Components (Section L2.6);
- Hydraulic Analysis of South Bridge Diversion and Transmission Components (Section L2.7);
- Swamp Distribution System Analysis (Section L2.8);
- Swamp Flow Outlet Control Analysis (Section L2.9);
- Project Alternative Analysis (Section L2.10); and
- Hydrologic Uncertainties (Section L2.11).

## L2.2 Model Study Methodologies

The following subsections describe the methodologies used in developing the project-specific tools (HEC-HMS, HEC-RAS, EFDC, and Engineering Calculations) used to evaluate existing conditions and future project alternative conditions in support of the Convent/Blind River Small Diversion project.

### L2.2.1 HEC-HMS

Hydrologic analysis was completed for the project using the USACE Hydrologic Modeling System Version 3.3 (HEC-HMS). HEC-HMS quantifies surface water hydrology for the project area and tributary watershed by simulating and calculating the rainfall-runoff process resulting from user defined precipitation input. From a systems perspective, the watershed runoff process is a portion of the hydrologic cycle, and the primary focus for quantifying water that flows through elements of the study area such as drainage canals and stream channels.

The use of HEC-HMS to simulate surface water runoff was paired with the USACE River Analysis System (HEC-RAS) software, a one-dimensional hydraulic model. Results from the HEC-HMS model were used to generate the inflow to the HEC-RAS model, which was used to calculate flow routing through the drainage canals and swamp storage within the study area. The models represent approximately 165 square miles of tributary area and include the Blind River from its headwaters to approximately four miles downstream of I-10 near the confluence of the Blind River and Amite River. The models also represent a complex network of drainage canals and stream reaches that include the St James Parish canals (e.g., St. James Parish Canal, East St. James Parish Canal, Lateral 4 Canal, Latitude 3D Canal, Old New River, Canals parallel to US-61, Bayou Fusil, Romeville Canal, Bayou Des Acadie, and Pipeline Canals), and Ascension Parish canals (Conway Canal, and Canal Parallel to US-61). The HEC-HMS model schematic is presented on **Figure L2.2.1-1**.

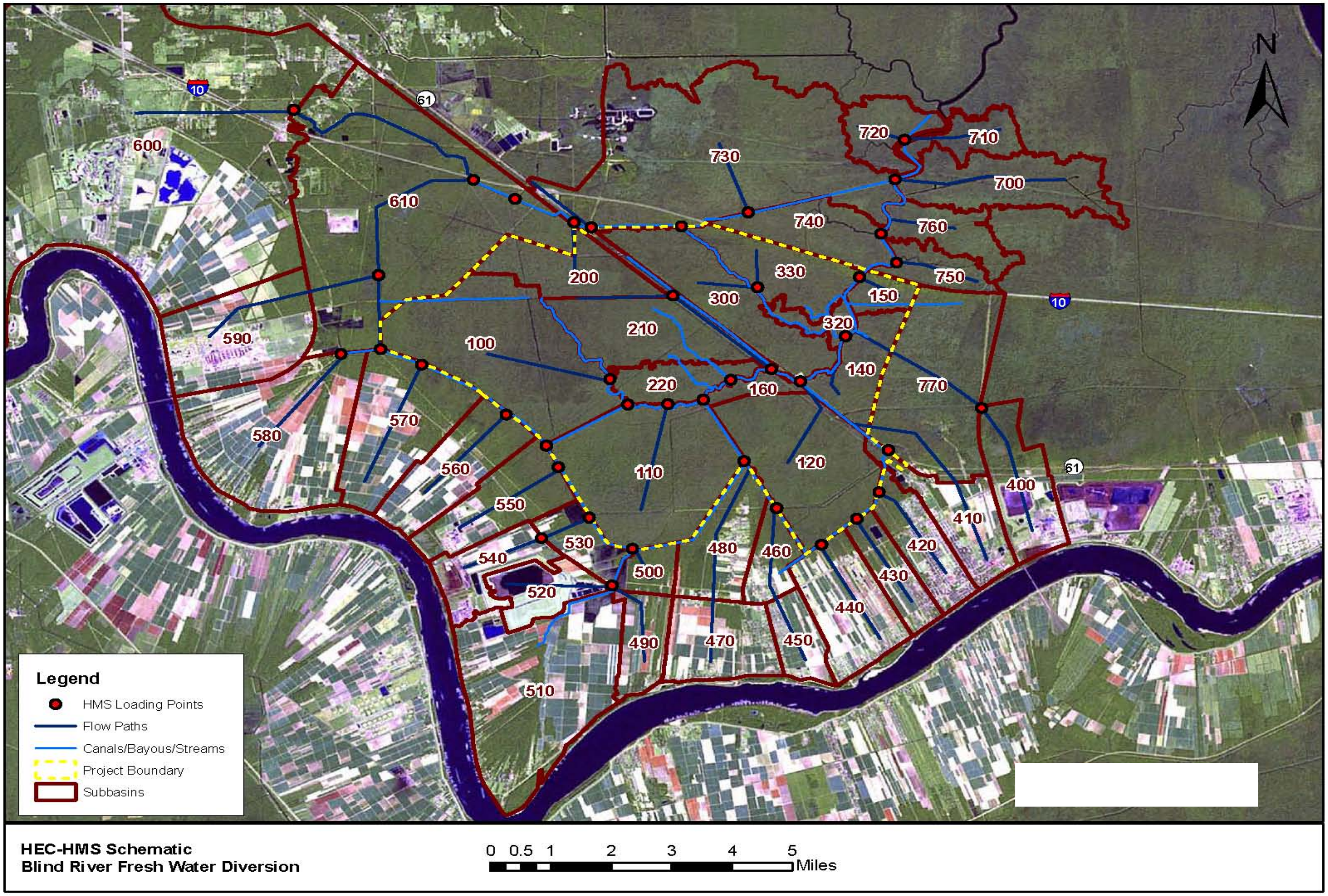



**Figure L2.2.1-1 HEC-HMS Model Schematic**

HEC-HMS was used to perform design storm simulations, and to perform continuous simulations throughout the representative year 2003 to analyze flows to the study area in dry as well as wet periods. Model parameters and supporting data sources are discussed in more detail in Section L2.3.4. Fundamental methodologies to develop the HEC-HMS model are summarized below in **Table L2.2.1-1**.

**Table L2.2.1-1 HEC-HMS Attribute Methodology**

| Hydrologic Attribute | Applied Method |
|---|---|
| Runoff Method | Composite Soil Conservation Service (SCS) Runoff Curve Numbers |
| Land Use | Anderson land use/land cover classification system with simplification into seven representative categories |
| Soils | Soil types were grouped into hydrologic soil groups per the Natural Resource Conservation Service (NRCS) |
| Runoff Characteristics | Curve Number values range from 79 to 96, which represent both impervious land cover and pervious cover with applicable soil characteristics |
| Time of Concentration | Time of Concentration and the corresponding Lag Time were calculated based on flow lengths and average slopes in each subbasin |
| Unit Hydrograph | SCS Unit Hydrograph |
| Flood Routing | Routing was not performed in HEC-HMS. Flood routing was accounted for in the HEC-RAS hydraulic model. |

### L2.2.2 HEC-RAS

The USACE HEC-RAS software version 4.0 was used to simulate hydraulic performance in the study area. HEC-RAS is a one-dimensional hydraulic model and was applied in unsteady flow mode, which is well suited to represent the open channel dynamics of the canal network and storage characteristics of the swamp (**Table L2.2.2-1**). Existing topographic and bathymetric data were used in combination with available engineering plans to define channel cross-sections, roadway culverts, and surface storage areas. Stage-storage relationships were defined for surface storage areas to account for storage volume and flow attenuation in the study area. Standard engineering references, field photos, and aerial photography were utilized to input Manning's roughness and loss coefficient values in the HEC-RAS model. The existing condition of the study area includes culverts under US 61 and I-10 as per field observations and available engineering survey. HEC-RAS was also used to represent the influence of potential project improvements, such as various berm gaps and control structures.

The unsteady flow HEC-RAS input file was developed from the HEC-HMS model output. The flow hydrographs information from HEC-HMS stored in HEC Data Storage System (DSS) file was loaded at the appropriate locations along the Blind River and interior drainage canals and bayous. **Figure L2.2.2-1** shows a schematic of the intricate network of streams in the study area, and respective model features in HEC-RAS.

**Table L2.2.2-1 HEC-RAS Attribute Methodology**

| Hydraulic Attribute | Feature Overview |
|---|---|
| Channel Cross-Sections | HEC-RAS supports irregular cross-sections and represents channel roughness using Manning's n coefficients |
| Culverts and Bridges | Federal Highway Administration (FHWA) methodology for computing losses through bridges and culverts |
| Loops and Flow Splits | Unsteady flow capability of HEC-RAS supports flow splits and network loops, both of which are present in the study area |
| Surface Storage | Storage-Area-Elevation (SAE) curves define storage volume in the portions of the system where water is stored but does not exhibit channel flow |
| Evaporation | Evaporation from the large surface storage areas in the swamp was represented as a flow time series computed and applied to each storage area based on available pan evaporation data |
| Canal to Surface Storage Flow Exchange | Lateral weirs defined based on available topographic data were used to allow for flow exchange between drainage canals and adjacent swamp areas |
| Surface Storage to Surface Storage Exchange | HEC-RAS storage area connections were included to facilitate flow exchange between adjacent surface storage areas |
| Downstream Boundary Condition | Water elevations observed at Lake Maurepas were used to define the downstream boundary condition of the model |

### L2.2.3 EFDC

Hydrodynamic and water quality analysis were completed for the project using the Environmental Fluid Dynamics Code (EFDC), which is a US EPA-sponsored public domain model. The EFDC is a general-purpose modeling package for simulating three-dimensional (3-D) flow, transport, and biogeochemical processes in surface water systems including rivers, lakes, estuaries, reservoirs, wetlands, and near-shore to shelf-scale coastal regions. In addition to hydrodynamic and salinity and temperature transport simulation capabilities, EFDC is also capable of simulating cohesive and non-cohesive sediment transport, near-field and far-field discharge dilution from multiple sources, the transport and fate of toxic contaminants in the water and sediment phases, and the dissolved oxygen, algae and nutrient process (i.e., eutrophication). Special enhancements to the hydrodynamics of the code, including vegetation resistance, drying and wetting, hydraulic structure representation, wave-current boundary layer interaction, and wave-induced currents, allow refined modeling of wetland and marsh systems, controlled-flow systems, and near-shore wave-induced currents and sediment transport. More information regarding the EFDC model can be found at the US EPA website.

The primary use of the EFDC model for the project was to quantify the water depth and elevation, hydraulic residence time (HRT), sedimentation and erosion, and water quality for various hydrologic response units (HRUs) with a relatively high resolution.

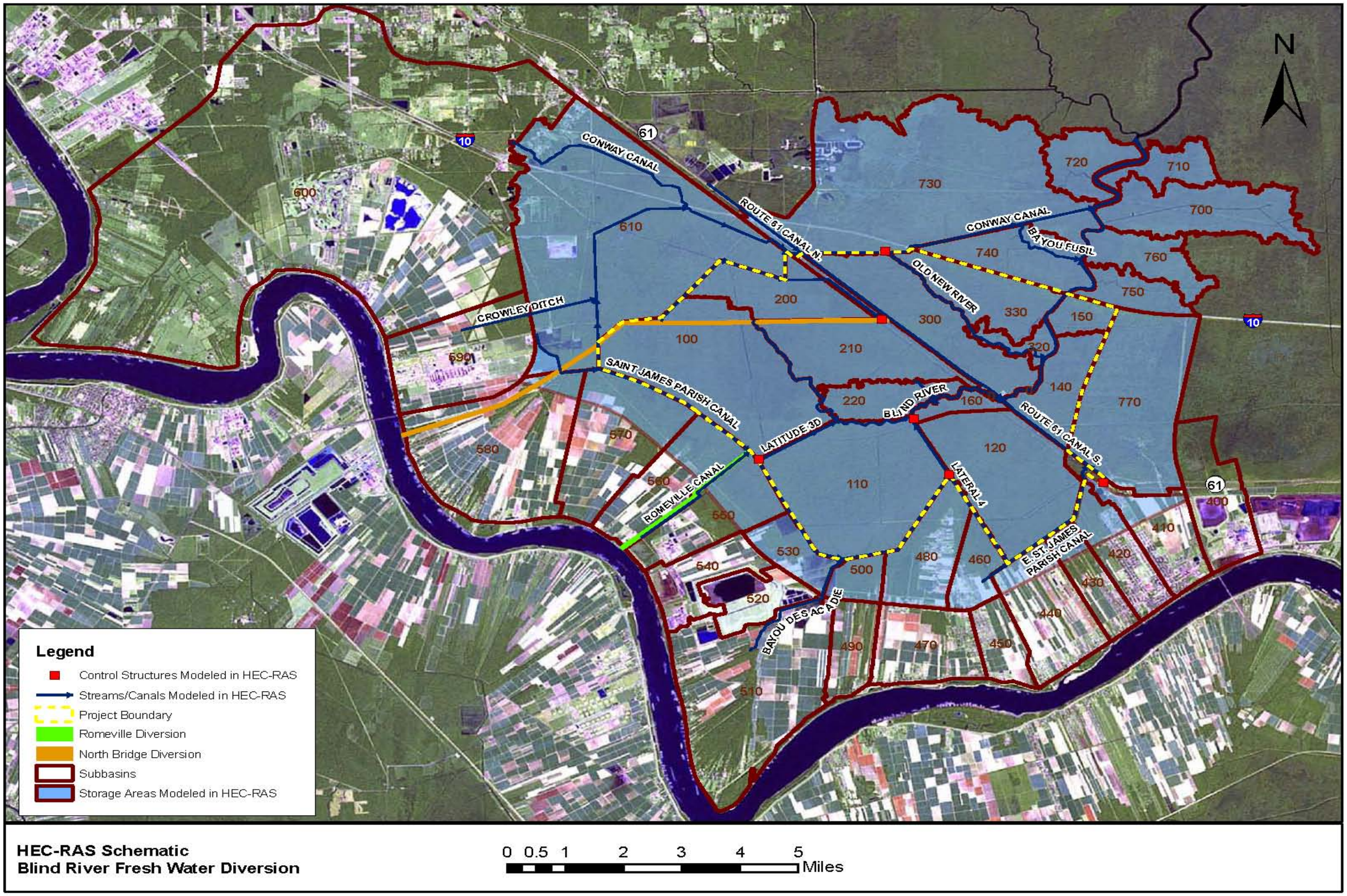



**Figure L2.2.2-1 HEC-RAS Model Schematic**

It is essential to design a model grid that has a proper spatial resolution to represent key hydrodynamic, sediment, and water quality processes for the feasibility study. At the same time, the model grid should be computationally efficient so that one-year simulation can be accomplished within a reasonable CPU time (i.e., <24 hour CPU time). This would allow various project alternatives to be analyzed in a timely manner. This issue impacted the selection of the grid resolution due to computational issues and modeling productivity.

The project area is located about 43 miles northwest of New Orleans, between west of I-10 and St. James, Louisiana. Historically, without the Mississippi River levees, the flood water propagated toward the east into the Lakes Maurepas and Pontchartrain via the Blind River and the low-lying flat topography in the area during the periods of high Mississippi river flow conditions.

To drain the Maurepas Swamp, various man-made channels/canals were constructed in the area during the past several decades. **Figure L2.2.3-1** shows the location of the project area and major canals, such as St. James Parish and Conway Canals along the project boundary.

One of the physical characteristics of the project area is the low-lying flat topography. **Figure L2.2.3-2** shows the topographical variations based on the previous LiDAR survey data in the area. In general, the land area is higher in the south and west and lower in north with a typical variation range of 1 ft.

Wetland flow is typically dominated by both the topography and vegetation resistance. To accurately represent the variations of the topography and vegetation resistance as much as possible without sacrificing model computational efficiency, a cell size of 39,701 square meters or 427,323 square feet was selected such that a total of 2,345 model cells are included in the 35 square miles project area.

To easily reconfigure the model to various alternative project conditions, a Cartesian coordinate system was used to develop the model grid. A constant square cell size, that is, 199.25 m x 199.25 m or 653.70 ft x 653.70 ft was used throughout the system. The model grid system that covers the study area including the Blind River is shown on **Figure L2.2.3-3**.

Because the existing canals and Blind River are deeper and narrower than the wetland cells, a subgrid channel approach, with channel grid cells embedded in the wetland cells, was initially considered. However, several early model test runs showed that with the subgrid channel, the model became very unstable and the computational time step had to be significantly reduced to less than one second.

The subgrid channel approach was not used since: 1) this is a feasibility study project; 2) several dozen model simulations with different model configurations for different purposes were needed through the project cycle; and (3) the project schedule meant model running efficiency was extremely critical.

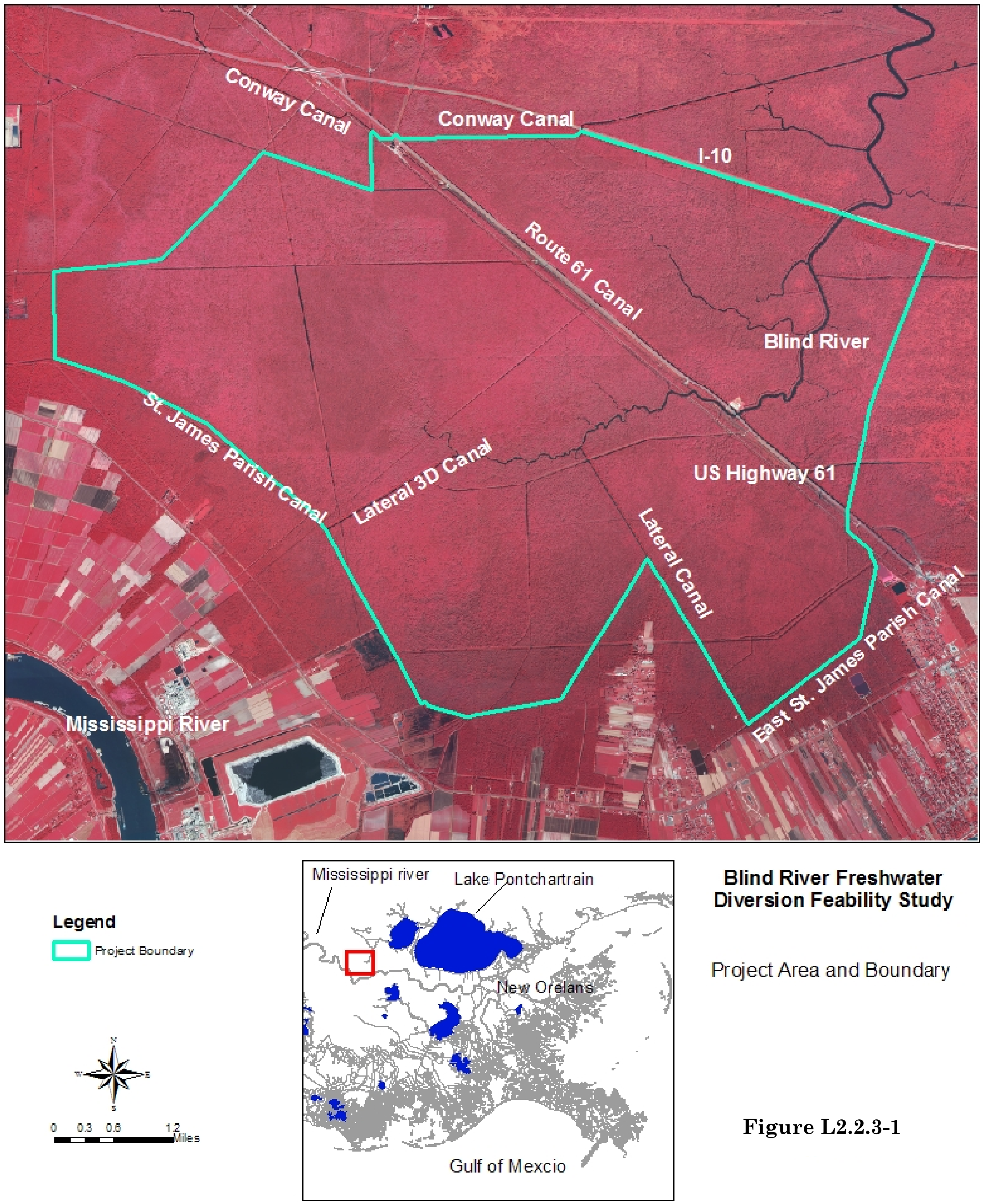
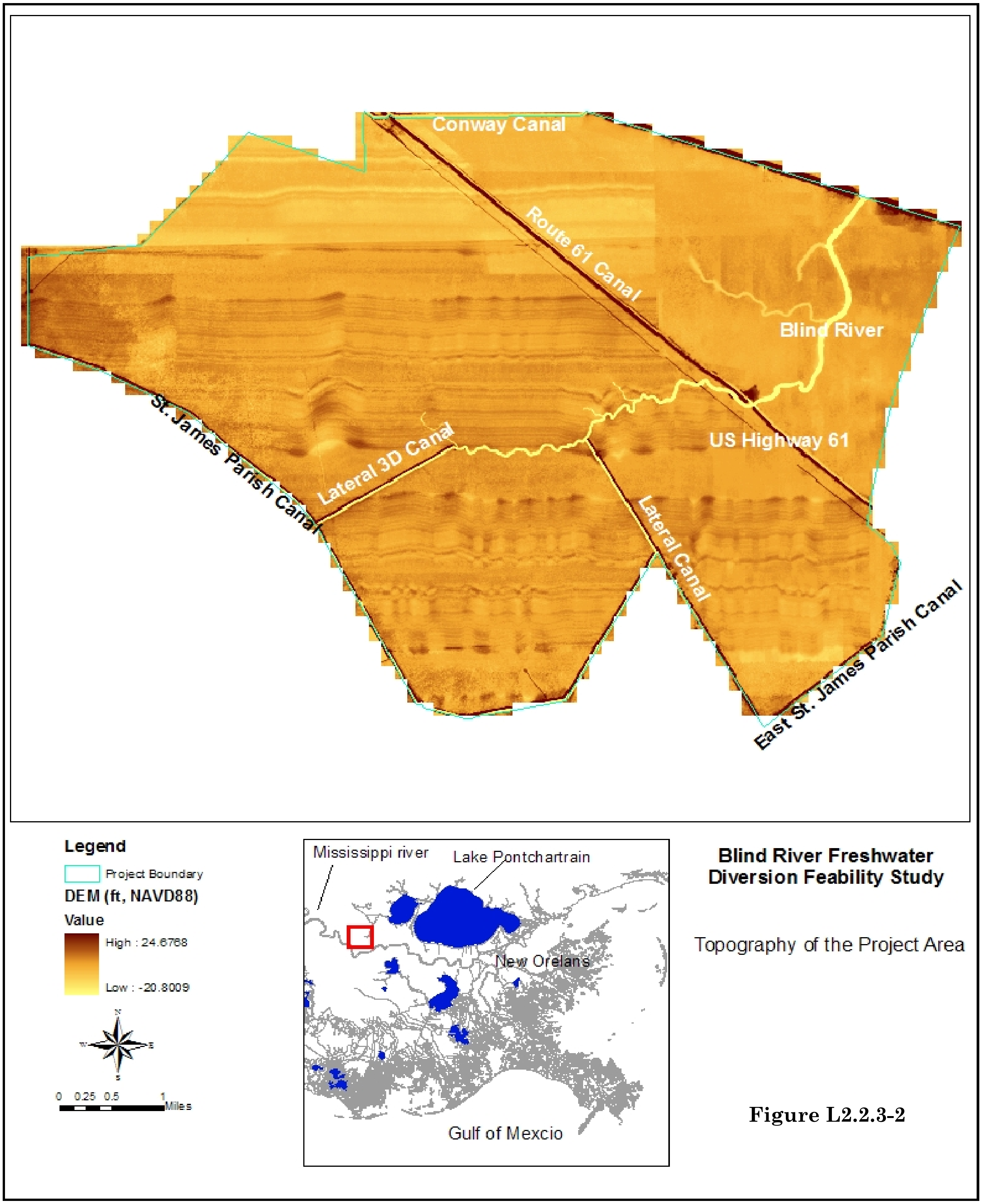



**Figure L2.2.3-1**




**Blind River Freshwater Diversion Feability Study**

Topography of the Project Area

**Figure L2.2.3-2**

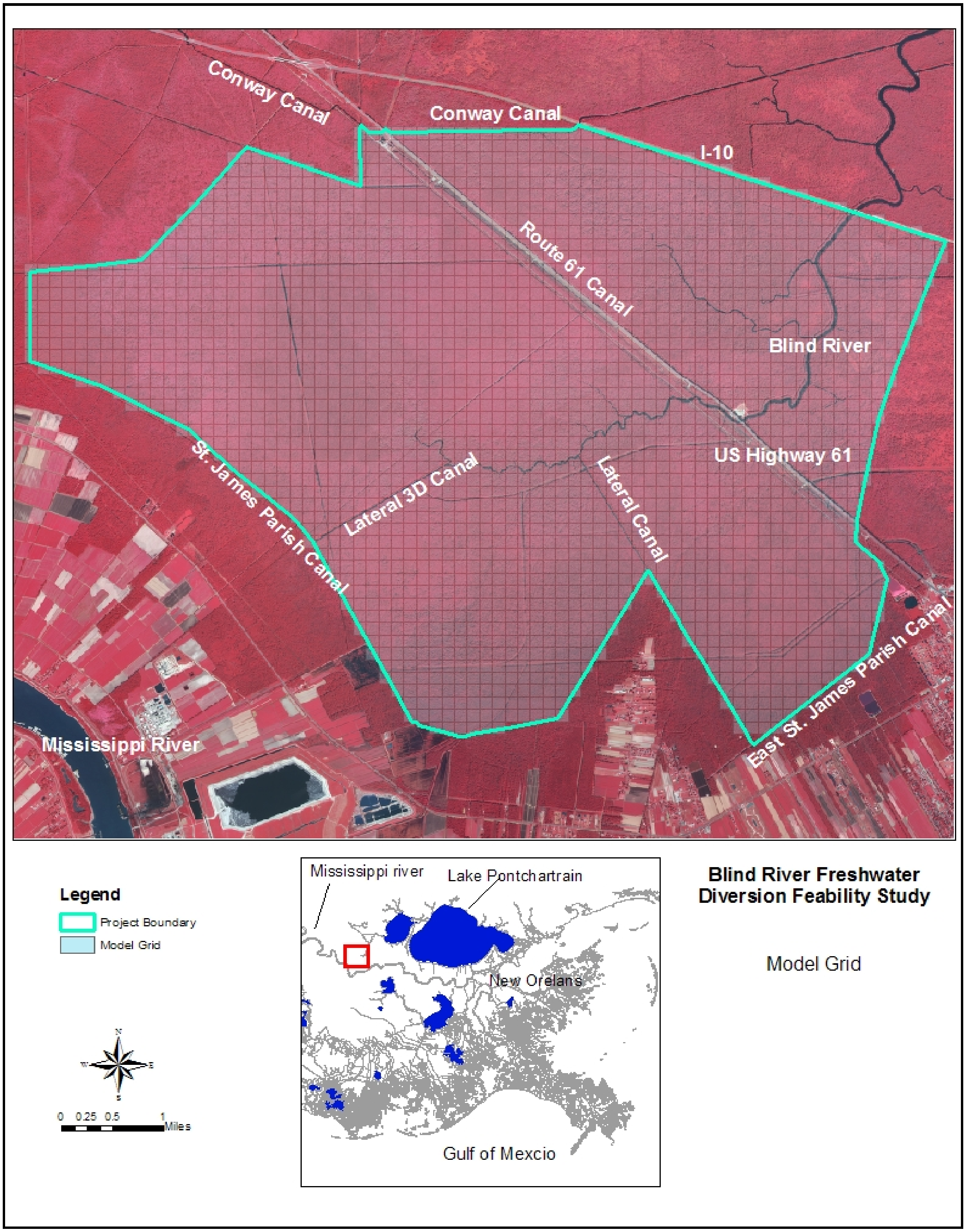



**Figure L2.2.3-3 EFDC Model Grid**

Instead, an equivalent channel approach was used to represent the existing canals and Blind River in the model.

The transformation of the existing canals and the Blind River into the equivalent channel is illustrated on **Figure L2.2.3-4**.

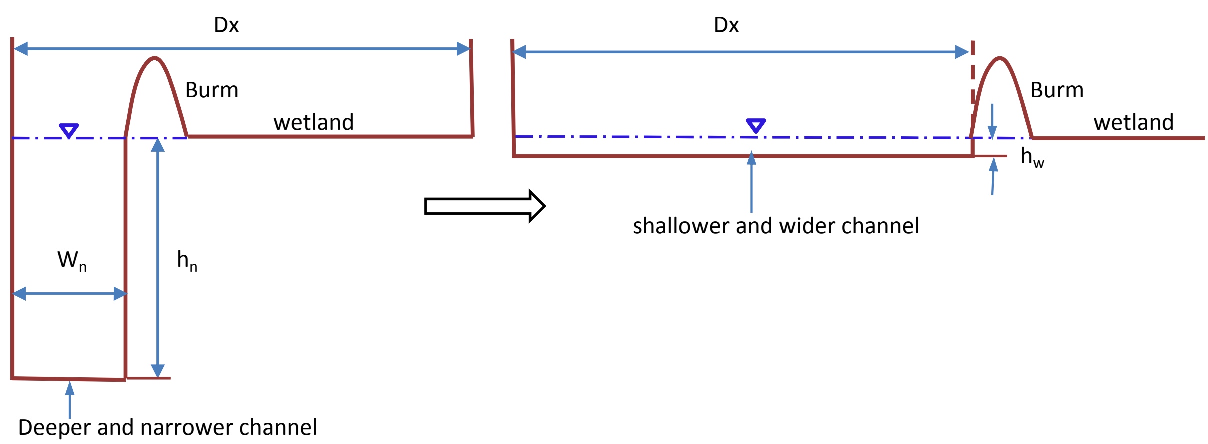



**Figure L2.2.3-4 Schematic of the Existing Canals/Blind River and Equivalent Channel**

For the equivalent channel, the flow conveyance capacity should match the capacity of the narrower but deeper channel when the channel water level is at the wetland elevation, that is,

$$Q_n=Q_w \quad (1)$$

where $Q_n$ = the flow in the narrower channel (cfs);

$Q_w$ = the flow in the wider channel (cfs); and

Subscript n stands for the narrower channel and w stands for the wider channel.

To calculate channel flow, the Manning Equation was used:

$$Q=VA= K/n * (A/P)^{2/3} * S^{1/2} * A \quad (2)$$

where k = 1.49;

A = flow cross-sectional area ($ft^2$);

P = wetted perimeter (ft);

S = bottom slope (ft/ft); and

n = Manning roughness coefficient.

Combining Equations (1) and (2) gives:

$$K/n_n*(A_n/P_n)^{2/3} *S_n^{1/2} *A_n = K/n_w * (A_w/P_w)^{2/3} * S_w^{1/2} * A_w \quad (3)$$

Let the equivalent flow cross-sectional area be the same as the existing channel, that is

$$W_n*h_n = Dx*h_w \quad (4)$$

where $W_n$ = canal or Blind River width (ft);

$h_n$ = depth relative to the adjacent wetland cell (ft);

$Dx$ = model cell width, that is 653.70 ft; and

$h_w$ = equivalent channel depth (ft).

Rearranging Equation (4) yields

$$h_w = W_n*h_n/Dx \quad (5)$$

Combining Equation (3) and (4) results in

$$n_w = n_n * (W_n+2h_n)^{2/3}/(Dx+2h_w)^{2/3} \quad (6)$$

Equations (5) and (6) were used to calculate the channel bottom elevation relative to the adjacent wetland cell and roughness, respectively, for the new wider and shallower channel.

Thus, the equivalent channel approach yields the same flow conveyance capacity and velocity in the equivalent channel wetland cells as in the existing canals/Blind River. However, one drawback of this approach is that simulated sediment deposition in the equivalent channel wetland cells will exceed expected sedimentation in the existing canals and Blind River because the equivalent settling depth is less than the actual channel or river depth.

Hydraulic flow barriers were used to represent the berms along the existing canals. Flow control structures represented with the head difference flow rating tables, which were derived from the HRC-RAS model simulation results, were used for various sizes of the existing berm gaps and proposed berm gaps in the following model simulations.

### L2.2.4 Engineering Calculations

#### L2.2.4.1 Purpose

A set of standard engineering equations for runoff estimation and water balance in a networked system was developed for three purposes:

- Because of a lack of hydrologic data throughout the swamp and Blind River system, the HEC-HMS and HEC-RAS models could not be truly calibrated to historical data. They were programmed with the best available information and parameterized with professional judgment and knowledge of the hydrologic flashiness of contributing watersheds and tendencies toward stagnation within the swamp areas. In lieu of direct calibration of the HEC models, the engineering equations were used as a supplemental way of cross checking of the HEC models by reproducing similar trends for runoff and water level dynamics using independent techniques and reasonable parameterization.

- Because the dynamic HEC-RAS model represents a complex, highly-linked network of canals and storage areas, each 1-year model run takes a very long time to execute. The HEC-RAS analysis focused on the year 2003, which represents average hydrologic conditions and for which daily lake level data for Lake Maurepas are available (an important boundary condition due to backwater effects throughout the swamp). Therefore, to enhance the breadth of conditions over which the alternatives were analyzed, the engineering calculations were used to evaluate daily water balance throughout the swamp for the period 1989 – 2004. This period represents the time for which all necessary input data were available, including precipitation and water levels in Lake Maurepas. The calculations were used to extend the period of record once the results for 2003 were shown to agree well with the dynamic trends for swamp filling and drawdown as represented in the HEC-RAS model.
- Fundamentally, the use of simple standard equations that replicate the general trends and patterns of more complex numerical models can help improve understanding of complex networks by focusing on key response patterns using familiar terms and expressions.

Most of the key hydrologic metrics, such as flow through the swamp and prevention of saline backflow from Lake Maurepas, were derived for the alternatives analysis directly from the HEC-RAS model. The engineering calculations were used to supplement that key hydrologic information with more general metrics such as:

- Average water elevation over the fuller time period;
- The potential for the swamp to experience periodic dry conditions that may be conducive to cypress germination and sapling survival; and
- Average annual sediment load into the swamp from the Mississippi River.

Lastly, the engineering calculations were used to develop time series of diversion flows for 2003 that were subsequently used by the HEC-RAS model. These were based on the simple equations and constraints presented below. By conditioning the diversions on the relativity between water levels in the swamp and water levels in the lake, the HEC-RAS analysis was able to avoid continuous diversions, and simulate a diversion schedule more likely to provide water only when it is most needed.

#### 2.2.4.2 System Representation

To help identify the inflows and outflows comprising the water balance throughout the study area, the basic network diagram shown on **Figure L2.2.4-1** was developed as a guide. Some of the benefit areas were consolidated to simplify the water balance estimates in areas where internal topography was uncertain. The diagram essentially illustrates the flow pathways into and out of each swamp area via their connectivity to adjoining canals. The circled letters at the boundaries represent the introduction of natural watershed runoff into the system. At the outlet of the system, Lake Maurepas acts as a stage boundary condition that limits the passage of water out of the swamp under conditions described in the following sections.

#### 2.2.4.3 Hydrologic Calculations

To provide an independent estimate of the hydrologic rainfall-runoff relationships in the HEC-HMS model, a simple set of equations for daily runoff and hydrograph recession was developed for each point of natural system inflow illustrated in the figure above. The equations represented each contributing subwatershed as a storage unit, and outflow from the watersheds as inflow to the study area (the watershed storage is simply a means to compute runoff into the swamp, in which storage areas are represented separately). Daily NOAA precipitation from station 2534 (Donaldsonville 4SW) was input to watershed storage, and a constant fraction of each daily rainfall amount was removed as the overall loss (representing watershed evaporation and seepage to groundwater). Hydrograph recession was accomplished by allowing water to accumulate in the watershed, and removing each day a constant fraction of the total accumulated storage. In this way, the effects of precipitation could be spread over multiple days, the slope of the recession curve could be tuned, and the long-term runoff volumes could be maintained.

Mathematically, the equations were expressed as follows, where *t* represents a daily index:

$$Loss_t = \mathbf{X}(Precip_t) \qquad \{0 \le X \le 1\text{: } X \text{ is a constant fraction of precipitation}\}$$

$$Storage_t = Storage_{t-1} + Precip_t - Loss_t - Streamflow_t$$

$$Streamflow_t = \mathbf{C}(Storage_{t-1}) \qquad \{0 \le C \le 1\text{: } C \text{ is a constant fraction of accumulated storage}\}$$

Watershed storage was also limited to an upper limit, which, if exceeded on any day, resulted in extra runoff equivalent to the precipitation (less the loss) for that particular day. However, this was fairly arbitrary, and did not appear to have a substantial impact on the calculations.

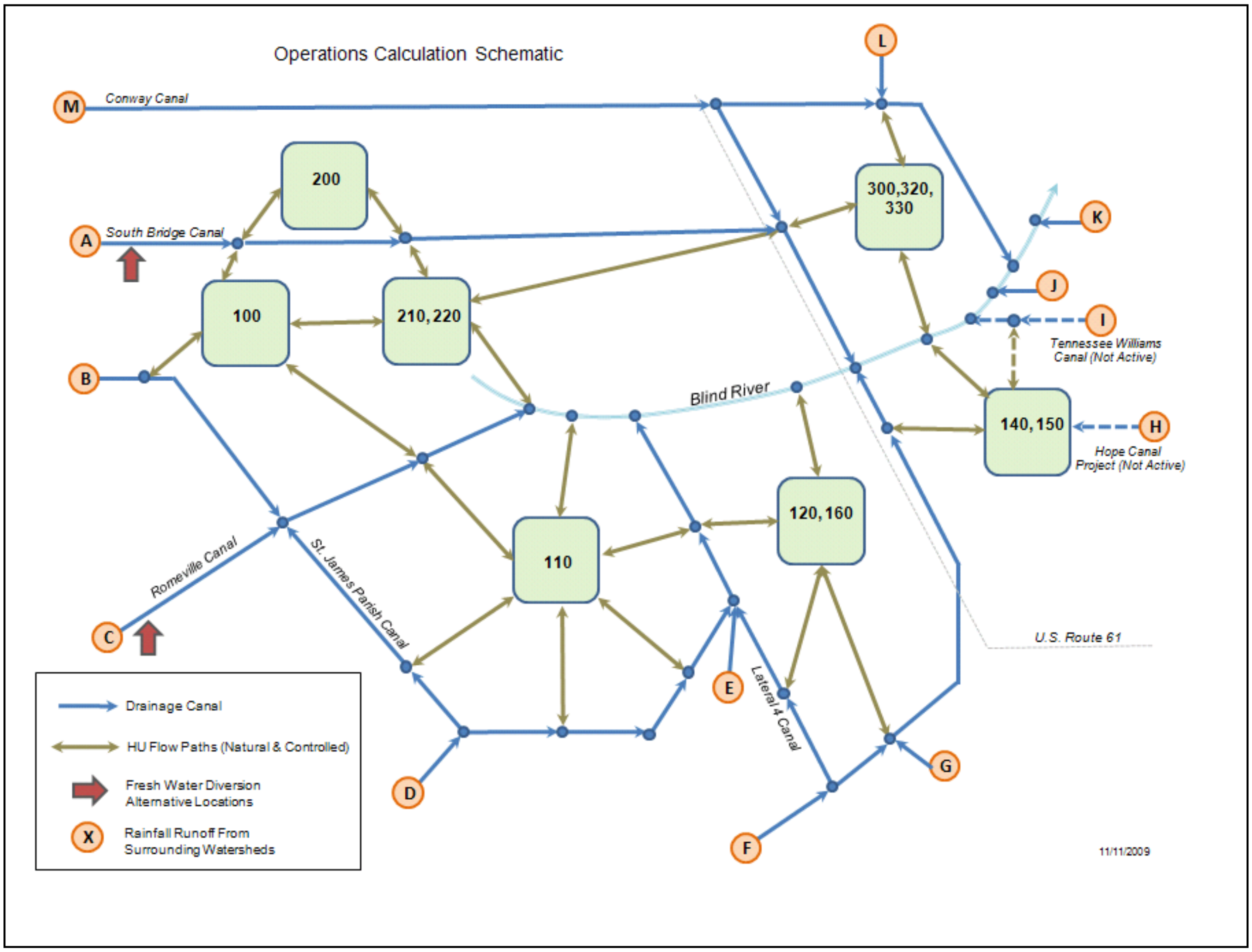



**Figure L2.2.4-1 Functional Diagram for Water Balance Calculations**

*Dashed lines illustrate flows that are boundary inflow from other project models

**Figure L2.2.4-2** illustrates the basic structure of the equations:

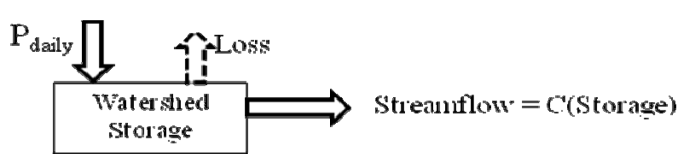



**Figure L2.2.4-2 Basic Rainfall-Runoff Representation**

Hence, three parameters were used for each watershed to simulate rainfall-runoff relationships and generate independent estimates which were compared with HEC-HMS. The parameters were tuned to match HEC-HMS results and then evaluated for the efficacy and hydrologic validity of final values, based on knowledge of the hydrologic flashiness of the canals and overall low permeability of the contributing subwatersheds. The value of X was tuned to match long-term runoff volumes from HEC-HMS, and the value of C was tuned to match the slope of the hydrograph recession.

#### 2.2.4.4 Operational Calculations

As noted above, the runoff was computed as an input to the study area (upstream contributing watersheds to the canals flowing into and through the study area). A separate set of equations was developed to compute water balance for each benefit area (consolidated as shown above) on a daily timestep. These equations were matched to the hydrographs produced by HEC-RAS to test the overall validity of the swamp representation, and to subsequently extend the hydraulic analysis over the longer period of record.

The water balance for each of the seven consolidated benefit areas was calculated with the standard storage equation (where *t* represents a daily time index):

$$Storage_t = Storage_{t-1} + \sum Inflow_t - \sum Outflow_t$$

$$Where:\ \sum Inflows = Direct\ Precip + Inflow\ from\ Canals$$

$$\sum Outflows = Evaporation + Outflow\ to\ Canals$$

Resulting storage was converted into water stage (averaged across each individual benefit area) and water surface area using available topographic data. Direct daily precipitation was computed from the same gage as the data used for the runoff calculations leading into the study area (NOAA station 2534). Daily evaporation data were collected from NOAA station 5620 (LSU Ben-Hur Farm). The data were multiplied by a pan coefficient of 0.77 per guidance in *Evaporation Maps for the United States, Technical Paper No. 37, US Weather Bureau, 1959.* Inflows and outflows from the swamp areas from/to the canals, as well as diversions from the Mississippi River into the canal system, were computed with the following Boolean logic relationships:

- Mississippi River Diversions
- Diversions occur when the average water stage in the swamp (averaged over the seven benefit areas) is less than the water level in Lake Maurepas. The rationale is to try to prevent backflow by removing or reducing the negative (reverse) head potential caused when the lake rises above the swamp. The comparison is done on a daily basis, and the findings suggest that diversions normally remain in effect for several weeks or months, followed by extended

periods of no diversions (the logic does not seem to cause a great deal of "on/off" operations). A buffer of 0.1 feet is added to the target swamp stage to avoid counting days in which the swamp is right at or just below the lake as problematic.

- Diversions cease if the swamp stage is greater than the lake stage (no further need to prevent backflow), and also when the lake stage drops below 0.5 feet, as these conditions create the potential for swamp dryout and seed germination.
- Diversion flow rates are governed by rating curves that are dependent on water levels in the Mississippi River. Below certain river stages, capacity in the gravity-fed system will diminish below design capacity along an exponential curve computed with hydraulic models. The maximum flow rate will be capped at the design capacity of the alternative, even though higher flows could be achieved hydraulically.
- Internal Water Exchanges
- Water flows into swamp areas from canals if canal carrying capacity is exceeded.
- Water flows into canals from swamp areas if:
- Local swamp water elevation is higher than Lake Maurepas elevation
- Water in swamp is higher than the elevation of the berm crests and/or gaps.
- Water flows between connected swamp areas directly based on head differential and the elevation of crests/gaps of the separating berms.
- Outflow to Blind River
- The network diagram above shows the connectivity between the canals, the benefit areas, and the Blind River. These equations allow water to flow into the Blind River only when the swamp stage is at or above the lake stage. When the lake is higher than the swamp, no outflow occurs.
- Backflow from Lake Maurepas
- These calculations do not explicitly account for potential backflow from Lake Maurepas into the study area (backflow is accounted for with dynamic backwater simulation in HEC-RAS). Rather, periods of time in which calculated swamp stage is less than the lake stage can be considered to create the potential for such backflow, and the logic for the diversions was developed to prevent backflow occurrences with reasonable and practical regularity.

#### 2.2.4.5 Checking

As stated, very little measured historical data are available to check the accuracy or predictive strength of the equations. In this case, the equations were developed and then tuned to match results from the HEC-HMS and HEC-RAS models to show that the results could be reproduced with independent techniques using standard parameters with reasonable values. Fundamentally, the equations are intended to increase overall credibility of the HEC models while also improving overall understanding of the fundamental dynamics of this system in simple hydrologic terms.

Three types of cross-checking were conducted between the standard equations and the HEC models:

- Runoff time series generated with the standard equations were compared with HEC-HMS results for each contributing subwatershed.
- Runoff time series in the Blind River as computed with these equations was compared to transposed river gage data from a nearby watershed – this effectively tested the overall efficacy of the calculations with respect to both runoff and the passage of water through the swamp.
- Time series of water stage within the swamp generated with the standard water balance equations were compared with results from HEC-RAS to test storm response (peak stage and drainage time) as well as longer-term patterns of filling and draining.

The results of these cross-checks between the HEC models, standard equations, and available data are presented in Section 2.3.6, along with the results of alternatives analysis.

## L2.3 Watershed Hydrology and Hydraulics

Appendix Section L2.2 presented an overview of multiple analysis methods that were used to analyze and evaluate hydrology and hydraulics within the study area. The multi-tiered approach to completing hydrologic and hydraulic analysis was formulated to support the project objectives of understanding and quantifying the movement of freshwater, sediment, and nutrients into the southeast portion of the Maurepas Swamp. For the benefit of the reader, conceptual objectives of the project that guided the hydrologic and hydraulic analysis are listed below:

- Enhance water quality in the Blind River by increasing the flow of freshwater to the Blind River;
- Promote water distribution in the swamp to increase the area of freshwater inundation from existing conditions to increase swamp productivity and wetland assimilation;

- Increase nutrient input to the swamp to increase swamp productivity from existing conditions, and increase wetland assimilation;
- Facilitate swamp building, at a rate greater than swamp loss due to subsidence and sea level rise, by increasing swamp productivity, as described above and by increasing sediment input; and
- Establish hydroperiod fluctuation in the swamp to improve bald cypress and tupelo productivity and their seedling, germination, and survival by decreasing flood duration in the swamp and increasing the length of dry periods in the swamp.

This section discusses each component of hydrologic and hydraulic analysis completed to evaluate existing conditions, including supporting data, model set-up, model testing and validation, and specific simulations and evaluations. This section also discusses preliminary investigations that were used to understand hydrologic and hydraulic constraints and opportunities, which guided the refinement of project alternatives that are presented in greater detail in Section L2.10.

### L2.3.1 Climatology and Physical Data

The climate of the study area is subtropical marine with long humid summers and short moderate winters, and regional atmospheric circulation is strongly influenced by many surrounding sounds, bays, lakes and the Gulf of Mexico. From a hydrologic perspective, the study area is also subject to periods of both drought and flood. During the spring and summer, the study area experiences warm, moist tropical air masses that are conducive to thunderstorm development. In addition, the study area is susceptible to tropical waves, tropical depressions, tropical storms and hurricanes. Historical data from 1899 to 2007 indicate that 30 hurricanes and 41 tropical storms have made landfall along the Louisiana coastline (NOAA, 2009).

The wide range of climate conditions expected within the study area provides the potential for hydrologic conditions ranging from extreme flooding to extended drought. The full range of conditions is essential to the fundamental ecosystem restoration project objectives. For the purpose of the hydrologic and hydraulic analysis, the trends of continuous periods such as frequency and duration of flood and dry conditions were considered. Flood conditions were also considered to understand existing conditions and evaluate potential project impacts. The following sections discuss specific climate and physical data that were utilized to complete the hydrologic and hydraulic analysis.

#### L2.3.1.1 Precipitation and Evaporation Data

##### Observed Precipitation

Specific sources of measured hydrologic data, such as precipitation data and stream flow measurements, are generally lacking within the study area. No rainfall gages are present in the study area, and based on review of available precipitation data near the study area presented on **Figure L2.3.1-1** the nearby

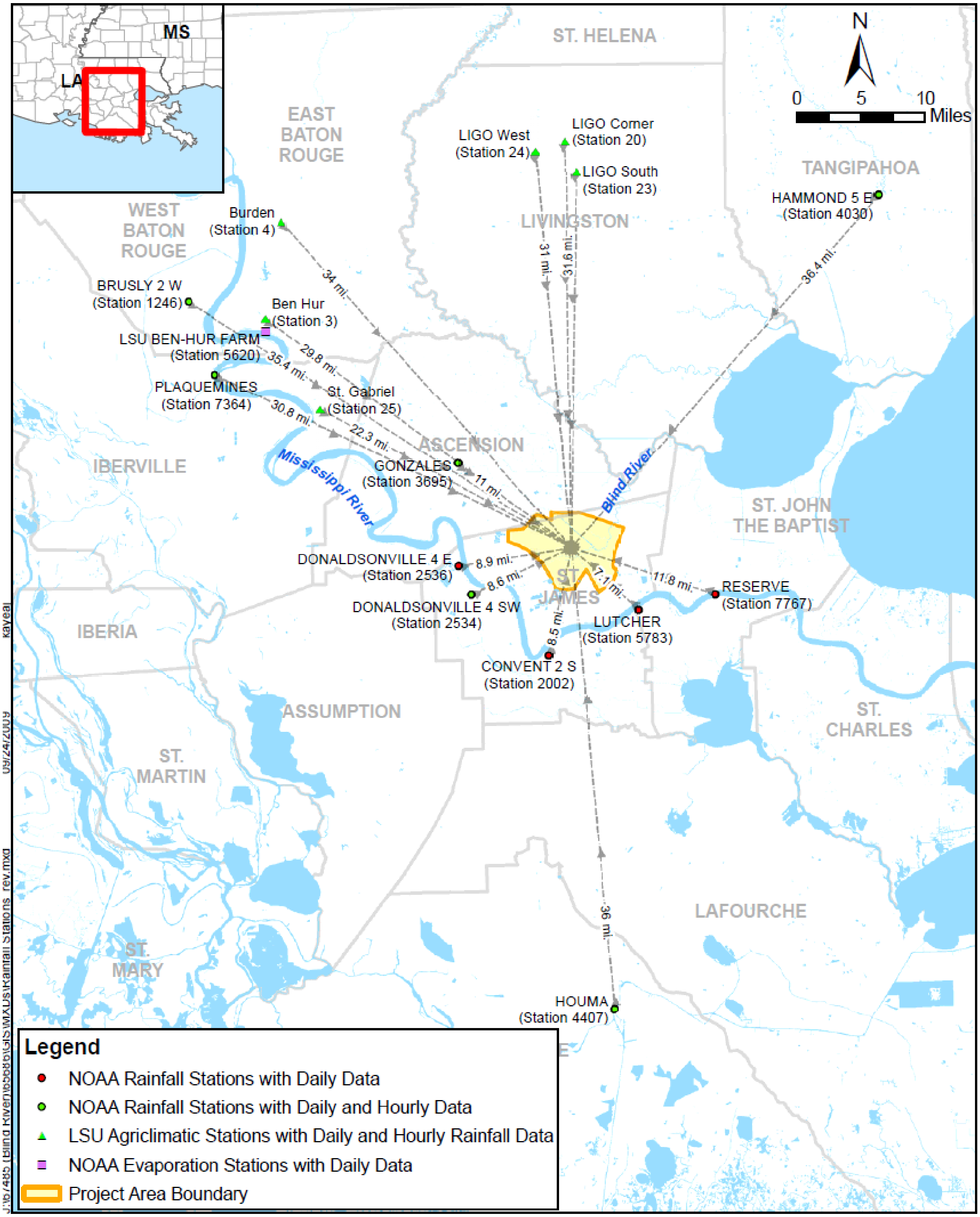



**Figure L2.3.1-1 Existing Precipitation and Evaporation Station Locations**

Donaldsonville 4 SW Station (NOAA Station 2534) was found to have the most complete continuous hourly rainfall record. A summary of precipitation data records that were reviewed is presented in **Table L2.3.1-1**. Rainfall data from the Donaldsonville 4SW Station were used to support both short duration and long duration hydrologic and hydraulic analyses.

**Table L2.3.1-1 Available Precipitation Data**

| Station Name | Station Number | Period of Record | | Distance from Study Area Centroid (Miles) | Data Source |
|---|---|---|---|---|---|
| | | Daily Data | Hourly Data | | |
| Donaldsonville 4 E | 2536 | 11/1996-12/2008 | NA | 8.9 | NOAA |
| Donaldsonville 4 SW | 2534 | 1/1930-7/2009 | 6/1988-12/2007 | 8.6 | NOAA |
| Convent 2 S | 2002 | 11/1996-12/2008 | NA | 8.5 | NOAA |
| Houma | 4407 | 1/1930-12/2006 | 11/1947-1/2007 | 36 | NOAA |
| Lutcher | 5783 | 2/1993-12/2008 | NA | 7.1 | NOAA |
| Reserve | 7767 | 1/1948-12/2008 | NA | 11.8 | NOAA |
| Hammond 5 E | 4030 | 1/1981-11/2007 | 12/1983-12/2007 | 36.4 | NOAA |
| Gonzales | 3695 | 3/1978-12/2008 | 10/1969-1/1982 | 11 | NOAA |
| Brusly 2 W | 1246 | 6/1987-11/2007 | 11/1989-12/2007 | 35.4 | NOAA |
| Plaquemines | 7364 | 1/1948-4/1962 | 10/1947-11/1964 | 30.8 | NOAA |
| LIGO Corner | 20 | 1/2006-12/2006 | 1/2006-12/2006 | 31.6 | LSU Ag |
| LIGO South | 23 | 1/2006-12/2006 | 1/2006-12/2006 | 29.3 | LSU Ag |
| LIGO West | 24 | 8/2001-7/2009 | 8/2001-7/2009 | 31 | LSU Ag |
| Burden | 4 | 8/2001-7/2009 | 8/2001-7/2009 | 34 | LSU Ag |
| Ben Hur | 3 | 8/2001-7/2009 | 8/2001-7/2009 | 29.8 | LSU Ag |
| St. Gabriel | 25 | 8/2001-7/2009 | 8/2001-7/2009 | 22.3 | LSU Ag |

NA indicates data not available.

**Design Storm Precipitation**

Surface water modeling was performed in the HEC-HMS and HEC-RAS models using rainfall estimates for the 2-, 5-, 10-, 25-, 50-, and 100-yr, 24-hour design storms defined for the study area using values provided in the National Weather Service Technical Paper 40. The Type III SCS rainfall distribution was applied in the HEC-HMS model in combination with rainfall depths corresponding to the depths of precipitation determined for each frequency, which are presented in **Table L2.3.1-2**.

**Table L2.3.1-2 Design Rainfall Depths**

| Recurrence Interval (Years) | Rainfall Depth (Inches) |
|---|---|
| 2-Year | 5.5 |
| 5-Year | 7.5 |
| 10-Year | 8.8 |
| 25-Year | 10.2 |
| 50-Year | 11.3 |
| 100-Year | 12.8 |

**Evaporation**

Daily evaporation data were collected from NOAA station 5620 (LSU Ben-Hur Farm). For application in the analysis, the measured daily data were multiplied by a pan coefficient of 0.77 per guidance in *Evaporation Maps for the United States, Technical Paper No. 37, US Weather Bureau, 1959*. Monthly evaporation values for 2003 and historical monthly averages (1989-2004) are presented in **Table L2.3.1-3**.

**Table L2.3.1-3 Monthly Evaporation Values**

| Month | Evaporation (inches) | |
|---|---|---|
| | 2003 Monthly Values | Historical Monthly Average Values |
| January | 2.98 | 2.84 |
| February | 2.58 | 3.18 |
| March | 3.49 | 4.58 |
| April | 5.98 | 6.13 |
| May | 7.72 | 7.75 |
| June | 6.52 | 7.43 |
| July | 6.69 | 7.18 |
| August | 7.10 | 6.95 |
| September | 6.07 | 5.90 |
| October | 4.70 | 4.78 |
| November | 3.77 | 3.31 |
| December | 3.94 | 2.75 |
| TOTAL | 61.54 | 62.78 |

#### L2.3.1.2 Topographic Data

A combination of available mapping data and new data collected during the project was used to support the hydrologic and hydraulic analysis. Topographic and field surveys conducted during the project are discussed in Appendix Section L3. Other available data sources were also used to support hydrologic and hydraulic modeling in the project area and in the tributary watershed area. Topographic data are pertinent to the hydrologic analysis in order to define hydrologic boundaries used to calculate runoff that occurs in response to rainfall. For the hydraulic model, topographic and bathymetric data were critical for determining overland flow slopes, channel cross-sectional geometry, critical elevations, and stage-area-storage relationships. In addition to the data discussed in Appendix Section L3, topographic data were available in the watershed from two other major sources:

- Digital Elevation Model (DEM) from the United States Geologic Survey (USGS); and
- Light Detection and Ranging (LiDAR) Data from the Louisiana State University (LSU) Atlas.

Two-foot interval LiDAR data collected by LSU for Ascension and St. James Parishes as part of their state-wide effort were used to develop a Digital Terrain Model (DTM) in the form of a Triangular Irregular Network (TIN) in ESRI Arc View GIS 9.2. The TIN together with USGS quadrangle maps and Google Earth aerial images were used to visualize the terrain and to identify and digitize the stream centerlines, swamps, and connected conveyance areas. Knowledge gained from field visits and surveys was used to interpret features visible in the available aerial photography, and differentiate between drainage canals that will convey flow through the study area and utility corridors that will not provide significant flow conveyance.

The Blind River Diversion Project used the 1988 North American Vertical Datum (NAVD). The bridge, culvert, and cross-section survey data were provided by the surveyors and LiDAR data in both NAVD and 1929 National Geodetic Vertical Datum (NGVD); however, the hydraulic models for the project used only the NAVD datum. Historical data such as road profiles, stage elevations, and structural controls were converted from NGVD to NAVD where necessary using a constant offset of -1.3 ft. The offset between the two datums varies little over the project area (less than 0.1 ft).

### L2.3.2 Blind River and Swamp System

The study area for this project is located within the Mississippi River Deltaic Plain within coastal southeast Louisiana in the Lake Pontchartrain Basin. The study area for this project is within the Upper Lake Pontchartrain Sub-basin, and consists of areas located within the Louisiana parishes of St. John the Baptist, St. James, and Ascension.

Since the construction of the Mississippi River flood control levees, the Maurepas Swamp and Blind River have been virtually cut off from periodic overflows from the Mississippi River, which included freshwater, sediment, and nutrient input. With minimal soil building and moderately high subsidence rates, there has been a net lowering of ground surface elevation, so that now the swamps are persistently inundated.

The limited ability to drain and persistent flooding are characteristics of existing hydrology in the study area, which conflict with the historical seasonal drying of the Swamp. The soils within the Swamp area are inundated or saturated by surface water or ground water on a nearly permanent basis throughout the year except during periods of extreme drought. Additional features within the study area that influence hydrology are associated with past construction of logging trails, drainage channels, pipe lines and other utilities, and roads through the Swamp. These facilities disrupt the natural flow and drainage patterns. Short circuiting of the natural drainage patterns has created ponding in some areas.

#### L2.3.3 Lake Maurepas

The Maurepas Swamp is one of the largest remaining tracts of coastal freshwater swamp in Louisiana. The Blind River flows from St. James Parish, through Ascension Parish and St. John the Baptist Parish, and then discharges into Lake Maurepas. The Maurepas Swamp serves as a buffer between the open water areas of Lakes Maurepas and Pontchartrain and developed areas along the I-10/Airline Highway corridor.

Because of past hydrologic alterations, water levels in Maurepas Swamp are primarily influenced by the stage level of Lake Maurepas, with strong winds also exerting significant effect (Lee Wilson & Associates 2001, Mashriqui et al. 2002, Lane et al. 2003, Day et al. 2004). Tidal pulses are introduced into the Lake Maurepas system through Pass Manchac. Fluctuations in water level are generally expected to be similar throughout Maurepas Swamp, acknowledging slight variability associated with landscape position and elevation. Within any given year, stage is characterized by a bimodal hydrograph. Water level rises in the spring, then falls to its lowest level during the summer, rises to its highest level in the fall, and again falls to low levels in the winter (Thomson et al. 2002, Keddy et al. 2007). The intensity of peaks and troughs is typically associated with meteorological events, such as droughts and hurricanes.

Based on the strong correlation between lake and swamp water levels, the observed doubling of flood durations from 1955 to present at Pass Manchac (Thomson et al. 2002) coupled with lower swamp than lake elevations (Shaffer et al., unpubl. data) suggests that the duration of inundation within the project area has drastically increased over the last 50 years. Increased wetland impoundment also has been driven by the construction of canals, berms, and other artificial structures that alter the existing hydrology by proportionally increasing channelized flow volumes while reducing overland flow volumes.

Relative to historical flooding events, freshwater inputs presently have a substantially reduced influence on the hydrology of Maurepas Swamp. Inflow of freshwater into the project area occurs through drainage of runoff regionally, through riverine systems, and more locally through man-made channels. A series of dredged canals to the southwest and southeast of the swamp transport local drainage into the project area from the residential, industrial, and agricultural lands associated with the Mississippi River levee. Affected by these channels, the western and southwestern portions of the project area constitute the headwaters of the Blind River. General flow direction is southeast, then east and northeast toward its confluence with the Amite River Diversion Canal (ARDC), after which the combined water discharges into Lake Maurepas. The ARDC, a flood-control structure authorized by Congress in 1955 and completed in 1967, is 10 miles in length, 300 feet in width, and 25 feet in depth and connects mile marker 25.3 of the Amite River to mile marker 4.8 of the Blind River. At present, approximately half of the Amite River's flow discharges into Lake Maurepas via the diversion, and the other half along the natural flowpath. Modification of the ARDC to restore the adjacent bald cypress-tupelo swamp impaired by its construction is a near-term critical feature of the LCA Plan.

The Blind River and the Amite River, in particular, receive input from the drainage of urban areas to the west, most notably Baton Rouge. Due to increased urbanization, runoff contribution to these streams has increased in recent decades (LCA 2004). The smaller Tickfaw River also flows into Lake Maurepas from the north. These rivers—flashy streams prone to brief, high-intensity flood events throughout the year—contribute the majority of freshwater and sediment that enters Lake Maurepas, with an average flow rate of 1,000 to 4,000 cfs (Day et al. 2004). The flashy nature of inflow into Lake Maurepas is largely dependent on meteorological conditions; for instance, the Amite River may discharge at 10,000 cfs during storm events, but averages only 1,000 cfs in drought conditions (Day et al. 2004).

### L2.3.4 HEC-HMS

Simulation of hydrologic response of the study area to precipitation and surface water flow was performed in HEC-HMS. HEC-HMS is a hydrologic model capable of performing continuous or event simulations of surface runoff and groundwater baseflow. The hydrologic system operates by applying precipitation across Hydrologic Units (HUs). Precipitation is converted to surface runoff or infiltrates into the subsurface, and the runoff of infiltrated water is conveyed to receiving water loading points. Runoff and baseflow hydrographs at these loading points provide input for hydraulic routing in downstream reaches. The hydrologic flow routing of HEC HMS uses a sub-basin-reach representation of the hydrologic modeling system to route flows. The hydrologic model parameters used for the model simulations are described below.

Establishment of hydrologic parameter values utilized available GIS tools to automate the generation of data needed for the model. The base hydrologic GIS data is a representation of the watershed terrain, also known as a digital elevation model (DEM). Hydrographic (swamps, canals/ bayous, and streams) and transportation information obtained in ESRI Arc View GIS shapefiles format from St. James and Ascension Parishes, aerial photography from LSU, land cover information from USGS Land Cover Data Set and soils survey information from NRCS SSURGO in Arc View GIS shapefiles format were used to develop the hydrologic parameters for use in the HEC-HMS model of the watershed. Spatial and 3D analyst extensions available in ESRI Arc View GIS 9.3 were used to develop hydrologic unit boundaries and the parameters needed for each hydrologic unit, which includes composite runoff curve number, flow path locations, lengths and slopes, and channel flow lengths and slopes.

The preliminary hydrologic unit boundaries developed using Arc View GIS tools were refined to locate appropriate flow junctions using knowledge gained from field visits and engineering judgment. In all, 42 hydrologic units were delineated to comprise approximately 165 square miles of study area modeled in HEC-HMS.

#### L2.3.4.1 HEC-HMS Model Set-up

The study area is divided into HUs. In the model, the HUs were delineated by topographic highs, roads, levees, streams, and canals within the project area. Due to the relatively flat nature of the topography, HU divides are often overtopped during high intensity events. The hydrologic parameters assigned to each HU include area, width, slope, impervious area, roughness, initial abstraction, infiltration, and groundwater parameters.

**Hydrologic Characteristics of Study Area Soils**

Soil classification for the study area was obtained from the SSURGO database. **Figure L2.3.4-1** presents the predominant soil classification in the study area. As presented on the figure, the predominant soil group in the study area is Group D, with a very high runoff potential. The predominant groups in the study area are Hydrologic Soil group (HSG) C and D. Group C soil has low infiltration rates when thoroughly wetted, moderately fine to fine texture, and a higher runoff potential.

**Hydrologic Characteristics of Study Area Land Cover**

The land cover for the study area was obtained using the most recent available aerial photographs and is supported by field reconnaissance and observations. The hydrologic land cover for the study area is classified based on the Anderson land use/land cover classification system. The land cover category was further simplified into seven primary types with one category being all wetlands grouped together and the other six as listed below. The remaining land cover types such as transportation and urban-built up land were grouped into category “other” since the percentage of land in this category, in the study area, is relatively small. **Figure L2.3.4-2** presents the land cover distribution for the study area.

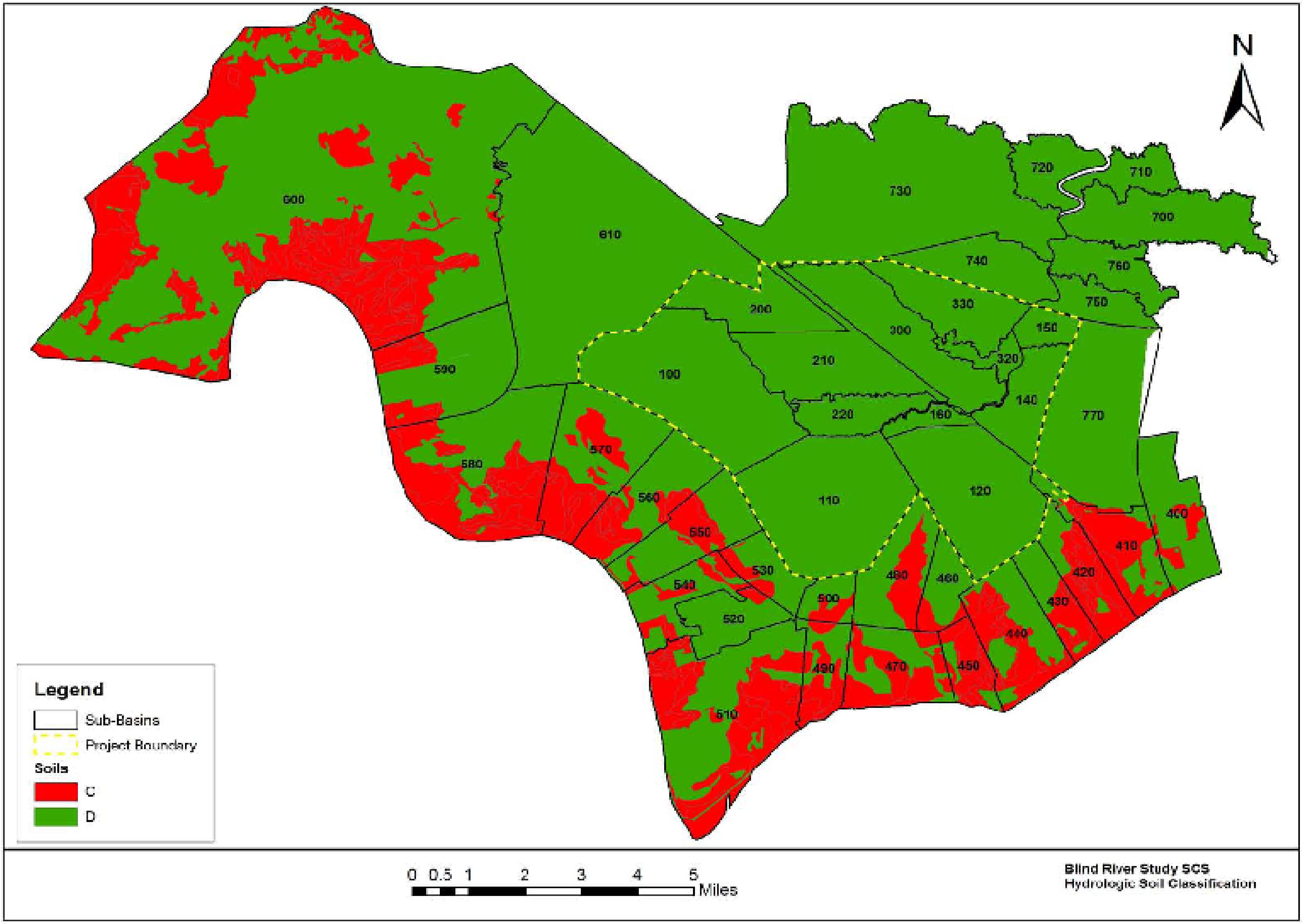



**Figure L2.3.4-1 Soil Classification**

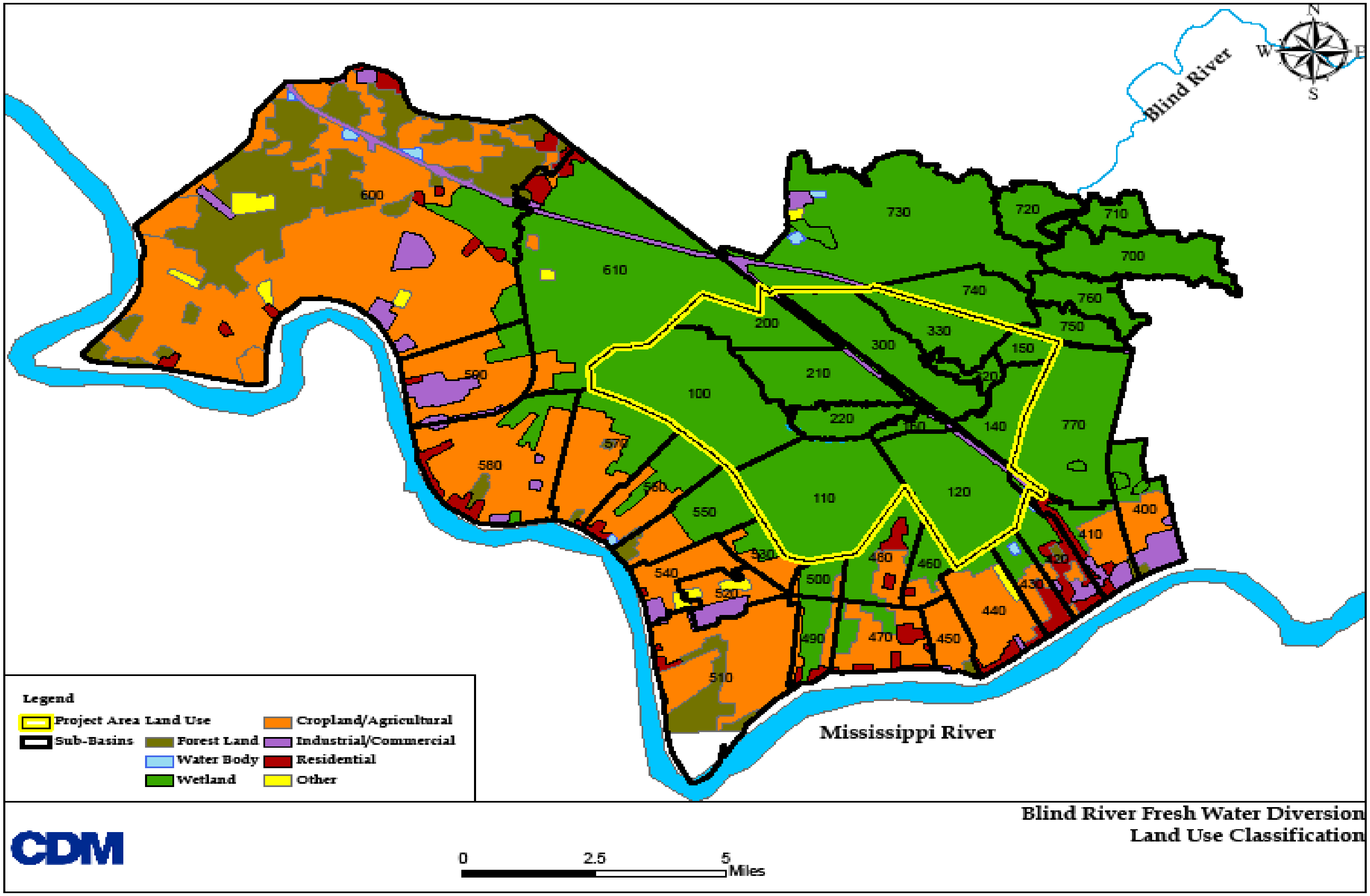



Figure L2.3.4-2 Land Use Classification

Based on the area percentages, the dominant land cover in the study area is wetland. The hydraulic and hydrologic model parameters were estimated based on these land cover categories in combination with soil classification. **Table L2.3.4-1** presents the percentage of land use type in the study area.

**Table L2.3.4-1 Hydrologic Land Cover Percentages**

| Land Cover Type | Percentage of Watershed Area (%) |
|---|---|
| Forest Land | 7.7 |
| Water body | 0.3 |
| Wetland | 53.4 |
| Cropland/Agriculture | 30.6 |
| Commercial/Industrial | 4.3 |
| Residential | 3.0 |
| Others | 0.7 |

**Runoff Curve Numbers**

The study area is divided into hydrologic units with each hydrologic unit assigned a unique hydrologic unit ID. Study area characteristics and hydrologic parameters were estimated for each hydrologic unit using SCS_methodology. Runoff curve numbers and time of concentration were extracted for each hydrologic unit and used as HEC-HMS model parameters.

The SCS runoff curve number method is calculated using the methodology described in NRCS TR-55 manual titled "Urban Hydrology for Small Watersheds". The runoff curve number is an empirical coefficient that relates runoff potential to land cover and hydrological soil classification. The NRCS hydrological soil groups include four groups designated as A, B, C and D (TR-55). The runoff curve number was computed for each hydrologic unit using the soil classification and land use classification together with the curve number tables in TR-55 manual. **Table L2.3.4-2** presents the list of hydrologic unit IDs with the composite curve number calculated for each hydrologic unit.

**Time of Concentration**

Time of concentration is defined as the time required for all the drainage area to contribute to the flow. The longest flow path in each hydrologic unit is estimated using the slope of the hydrologic unit, which was calculated from USGS topographical maps and contours. The time of concentration along the selected flow path was estimated using a segmental approach based on the type of flow such as shallow concentrated flow or channel flow. Hydrologic parameters were estimated using aerials and field observations. For comparing the results of segmental approach, SCS lag equations was used to compute the time of concentration. Lag time is computed as 60 percent of time of concentration. Lag time for each hydrologic unit is presented in Table L2.3.4-2.

**Table L2.3.4-2 HEC-HMS Model Parameters**

| Sub-basin Number | Drainage Area (Square Miles) | Composite Curve Number | Lag Time (Hours) | Percent Impervious Area (%) |
|---|---|---|---|---|
| 100 | 6.8 | 79.0 | 20.8 | 0 |
| 110 | 6.24 | 79.4 | 10.1 | 0 |
| 120 | 4.93 | 79.1 | 20.1 | 0 |
| 140 | 1.84 | 79.0 | 3.2 | 0 |
| 150 | 0.7 | 79.1 | 8.8 | 0 |
| 160 | 0.52 | 79.0 | 2.0 | 0 |
| 200 | 2.27 | 79.0 | 9.5 | 0 |
| 210 | 3.42 | 79.1 | 18.1 | 0 |
| 220 | 1.42 | 79.1 | 2.4 | 0 |
| 300 | 3.58 | 79.2 | 9.8 | 0 |
| 320 | 0.62 | 80.8 | 8.9 | 0 |
| 330 | 2.38 | 81.0 | 9.8 | 0 |
| 400 | 2.58 | 85.0 | 10.4 | 10 |
| 410 | 2.33 | 83.6 | 9.2 | 15 |
| 420 | 1.91 | 83.0 | 10.5 | 20 |
| 430 | 1.41 | 83.8 | 10.4 | 20 |
| 440 | 3.04 | 83.4 | 10.7 | 10 |
| 450 | 1.21 | 82.5 | 9.1 | 10 |
| 460 | 1.17 | 77.9 | 9.7 | 10 |
| 470 | 2.61 | 83.2 | 9.4 | 10 |
| 480 | 2.36 | 79.0 | 9.7 | 10 |
| 490 | 1.46 | 83.2 | 10.2 | 0 |
| 500 | 0.79 | 79.0 | 8.2 | 0 |
| 510 | 7.38 | 84.0 | 17.2 | 0 |
| 520 | 1.57 | 96.0 | 8.3 | 0 |
| 530 | 1.21 | 85.0 | 8.3 | 0 |
| 540 | 1.86 | 85.0 | 9.4 | 0 |
| 550 | 2.68 | 79.0 | 11.3 | 0 |
| 560 | 2.84 | 82.3 | 12.4 | 0 |
| 570 | 4.08 | 83.8 | 11.4 | 0 |
| 580 | 6.47 | 83.4 | 11.9 | 0 |
| 590 | 4.16 | 86.5 | 10.4 | 0 |
| 600 | 37.63 | 76.0 | 21.6 | 0 |
| 610 | 13.79 | 79.0 | 21.2 | 0 |
| 700 | 3.31 | 79.0 | 12.3 | 0 |
| 710 | 1.16 | 79.0 | 5.4 | 0 |
| 720 | 1.51 | 79.0 | 5.4 | 0 |
| 730 | 8.81 | 75.0 | 20.7 | 0 |
| 740 | 2.2 | 79.0 | 6.8 | 0 |
| 750 | 1.2 | 79.0 | 7.3 | 0 |
| 760 | 1.65 | 79.0 | 7.5 | 0 |
| 770 | 5.72 | 79.3 | 14.0 | 0 |

### L2.3.4.2 HEC-HMS Model Testing and Validation

Except for one stream gage in Ascension Parish that measures less than 25% of the watershed, there are no known stream gages available within the study area to calibrate the HEC-HMS model. Lacking available measured data, the storm runoff flows calculated in HEC-HMS were checked for reasonableness by comparing them to USGS regression equation estimate (USGS NSS version 4.0) and effective FEMA Flood Insurance Study (FIS) report of Ascension Parish. Neither the USGS stream flow estimates nor the FEMA FIS flows are directly applicable to the entire Blind River watershed due to the significant surface storage present in the swamp. However, the comparison presented in **Table L2.3.4-3** is appropriate, as it compares the HEC-HMS flows to the USGS flow estimates for drainage areas upstream of the swamp, such as the Conway Canal and other areas that are served by local drainage canals.

**Table L2.3.4-3 HEC-HMS Validation Summary**

| HEC-HMS Model Location | Tributary Area (Square Miles) | Peak Storm Runoff Flow (cfs per Square Mile) | | | |
|---|---|---|---|---|---|
| | | HEC-HMS (2-Year) | USGS NSS (2-Year) | HEC-HMS (100-Year) | USGS NSS (100-Year) |
| 400 | 2.58 | 385 | 394 | 1090 | 994 |
| 410 | 2.33 | 382 | 444 | 1078 | 1162 |
| 420 | 1.91 | 286 | 300 | 801 | 723 |
| 430 | 1.41 | 215 | 233 | 599 | 545 |
| 440 | 3.04 | 433 | 444 | 1238 | 1020 |
| 450 | 1.21 | 193 | 254 | 556 | 577 |
| 460 | 1.17 | 163 | 191 | 490 | 354 |
| 470 | 2.61 | 408 | 471 | 1176 | 1154 |
| 480 | 2.36 | 335 | 402 | 996 | 866 |
| 490 | 1.46 | 206 | 248 | 610 | 529 |
| 500 | 0.79 | 120 | 229 | 373 | 512 |
| 510 | 7.38 | 689 | 587 | 2019 | 1396 |
| 530 | 1.21 | 208 | 315 | 602 | 727 |
| 540 | 1.86 | 290 | 348 | 842 | 863 |
| 550 | 2.68 | 319 | 379 | 987 | 768 |
| 560 | 2.84 | 336 | 343 | 1002 | 711 |
| 570 | 4.08 | 534 | 518 | 1569 | 1362 |
| 580 | 6.47 | 809 | 731 | 2390 | 1951 |
| 590 | 4.16 | 619 | 596 | 1768 | 1632 |
| 600 | 37.63 | 974 | 1101 | 3107 | 2259 |

Multiple HEC-HMS model simulations, or plans, were created to generate runoff hydrographs for six design storm events ranging from 2-year to 100-year recurrence intervals using the runoff and routing parameters for hydrologic units and channels modeled. A separate plan was also created to simulate storm runoff flows over an entire year to evaluate hydroperiod within the swamp. A summary of the HEC-HMS plans is provided in **Table L2.3.4-4**. The flows generated by HEC-HMS are written to a HEC-DSS database and subsequently loaded to the companion hydraulic model developed in HEC-RAS. Results of the hydrologic and hydraulic models for existing conditions are presented in Appendix Section L2.3.5.3.

**Table L2.3.4-4 HEC-HMS Plan Summary**

| HEC-HMS Plan Name | Source Rainfall |
|---|---|
| 2-yr run | 2-year design storm. 24-hr rainfall depth, SCS Type-III |
| 5-yr run | 5-year design storm. 24-hr rainfall depth, SCS Type-III |
| 10-yr run | 10-year design storm. 24-hr rainfall depth, SCS Type-III |
| 25-yr run | 25-year design storm. 24-hr rainfall depth, SCS Type-III |
| 100-yr Simulation | 100-year design storm. 24-hr rainfall depth, SCS Type-III |
| Continuous Simulation | Continuous simulation. 2003-yr, 1-hr interval rainfall from Donaldsonville, LA gage |

### L2.3.5 HEC-RAS

HEC-RAS was used in combination with HEC-HMS to simulate the movement of water through the study area with particular focus on the drainage canals that drain through the project area and flow exchange between the drainage system and surface storage in the swamp. HEC-RAS is well suited for this task, especially with the capability to simulate unsteady flow conditions.

#### L2.3.5.1 HEC-RAS Model Set-up

To simulate the flow routing in the existing and proposed conditions through the study area, an unsteady state, one-dimensional hydraulic model was developed in version 4.0 of HEC-RAS. All topographic and planimetric data used in the model development are projected to State Plane 1983 Louisiana South (feet) coordinates and utilize elevations per the North American Vertical Datum of 1988 (NAVD 88). The geometric data input file in HEC-RAS model was prepared using HEC-GeoRAS version 4.1.1 and ESRI Arc View GIS 9.2. HEC-GeoRAS is an ArcView extension developed by HEC in cooperation with ESRI, specifically to process geospatial data for use with HEC-RAS. The unsteady flow HEC-RAS input file was developed from the HEC-HMS model output. The flow hydrographs information from HEC-HMS stored in HEC Data Storage System (DSS) file was loaded at the appropriate locations along the Blind River and interior drainage canals and bayous.

Cross-sectional geometry that describes the conveyance and storage capacity of open channels were entered into the model at regular intervals of approximately 500-600 feet. Information obtained from the project channel bathymetric surveys and design drawings of Conway Canal together with LiDAR data was used to create composite cross-sections for all canals and streams modeled in HEC-RAS. Flow regulating structures such as bridges/culverts, natural weirs (allows flow transfers between canals and swamps), and levees in the existing conditions, and weirs and control gates in proposed conditions were also modeled in HEC-RAS. Field survey data and design drawings were used for bridge and culvert representation. Standard engineering references, field photos, and aerial photography were utilized to input Manning's roughness and loss coefficient values in the HEC-RAS model. The existing condition of the study area includes culverts under US 61 and I-10 as per field observations and available engineering survey. Refinements were provided with various berm cuts and control structures.

**Downstream Boundary Condition**

To accurately assess the influence of Lake Maurepas, time varying head boundary conditions were applied at the downstream study boundary to control flows in and out of the intricate network of canals and swamps in the HEC-RAS model. The Lake Maurepas water surface elevation was approximated using observed stage time series data from the Pass Manchac gage that is maintained by USACE. Continuous simulations were performed on the existing and proposed conditions model for one full year (Dec 31$^{st}$, 2002-Dec 31$^{st}$, 2003) that was selected to represent the average meteorological conditions over the study area.

**Stage-Area Relationships**

Stage-storage area relationships were estimated for each HU using the topography DEM and ArcGIS 9.2 with 3D Analyst. Stage-storage area relationships are necessary in relatively flat models where flood waters may overflow the channel banks and fill low-lying areas. An accounting of the volume of these areas is needed for both accurate water surface elevation predictions as well as peak flow estimates.

**Evaporation Time Series**

The HEC-RAS model is refined to reflect the bathymetric data and to account for the evaporation losses over the 2003 simulation run. Hence, the evaporation losses were incorporated into the HEC-RAS model as a time series data for all storage areas. The HEC-RAS model does not provide a mechanism to include evaporation loss. The volume of water lost due to evaporation is then extracted from the storage areas in the model. Pre-processing of the data was required to estimate the time series with negative flows.

**Control Structures**

Various control structures such as bridges, lateral structures, and storage areas are entered into the HEC-RAS model. There are 30 storage areas, 35 storage area connections and 67 weir/lateral structures in the HEC-RAS model. The bridge data typically included one upstream section, one downstream section (includes bridge pier locations and low chord information). For the bridges, the most confining section was chosen to represent the structure in the model. Typically, this included reducing the cross-sectional area by removing the bridge pier area and by cutting off the cross-section at the low chord. The bridge data was obtained from State of Louisiana, Department of Highway drawings. The datum was verified with that in the drawings and necessary adjustments were made to match the data obtained from field observations and DEM elevation data.

Dimensions and geometry for many of the ditches, bridges, and culverts in the model were not field surveyed during completion of this project. However, hydraulic model data were developed using available engineering and as-built drawings to determine approximate length, size, and inverts of these elements. Some data were also estimated from aerial photos. These elements are not within the primary conveyance of the system, but are necessary to provide an approximation of the mass flows between HUs.

**System Inflows**

Boundary conditions are defined in the hydraulic model as flow hydrographs, uniform lateral flows, lateral inflow hydrographs, and stage hydrographs. The stage hydrograph is set only at the Blind River outfall reach. All the other hydrographs at various reach locations are read from the DSS file that contains output from the HEC-HMS model continuous simulation run.

### L2.3.5.2 HEC-RAS Model Testing

**Existing Drainage Canals**

The total watershed that drains to the Blind River contains numerous existing drainage canals. Areas of Ascension Parish located in the northern extent of the watershed generally drain to the Conway Canal, which conveys flows along the northern boundary of the project area and discharges to the Blind River downstream of I-10. Areas of St. James Parish drain to drainage canals that generally flow from high ground along the Mississippi River toward the Blind River. The existing drainage canals convey flows to the St. James Parish Canal that surrounds the swamp and in many places is coincident with the project area boundary. The Lateral 3D and Lateral 4 drainage canals convey flow from the St. James Parish Canal to the Blind River upstream of US 61. The extent of the watershed and the network of drainage canals downstream of US 61 is more limited, and primarily consists of natural channels and drainage canals that convey flows to the Blind River. Test simulations completed using both design rainfall of various magnitudes and observed rainfall from 2003 indicate that the majority of rainfall runoff volume to the Blind River flows through the drainage canals and bypasses the swamp.

**Influence of Lake Maurepas**

Lake Maurepas influences both flow and water elevations in the Blind River and the project area. Test simulations with HEC-RAS indicate that peak water surface elevations in the Blind River and the swamp result from Lake Maurepas water elevations, as opposed to stormwater runoff from the Blind River watershed. This effect is apparent on **Figure L2.3.5-1**, which compares the resulting stage hydrograph of two hydraulic simulations in HEC-RAS with two different boundary conditions. The result presented on Figure L2.3.5-1 is for Sub-basin 110, and is typical of the change in response observed in all the swamp hydrologic units. The first simulation shown by the light blue line on Figure L2.3.5-1 was calculated using a constant tailwater elevation at Lake Maurepas of 0.5 feet NAVD. The second simulation produced the stage hydrograph represented by the dark blue line on Figure L2.3.5-1 used the observed water surface elevation at Lake Maurepas. As shown, the dark blue line is much higher and more variable than the light blue line.

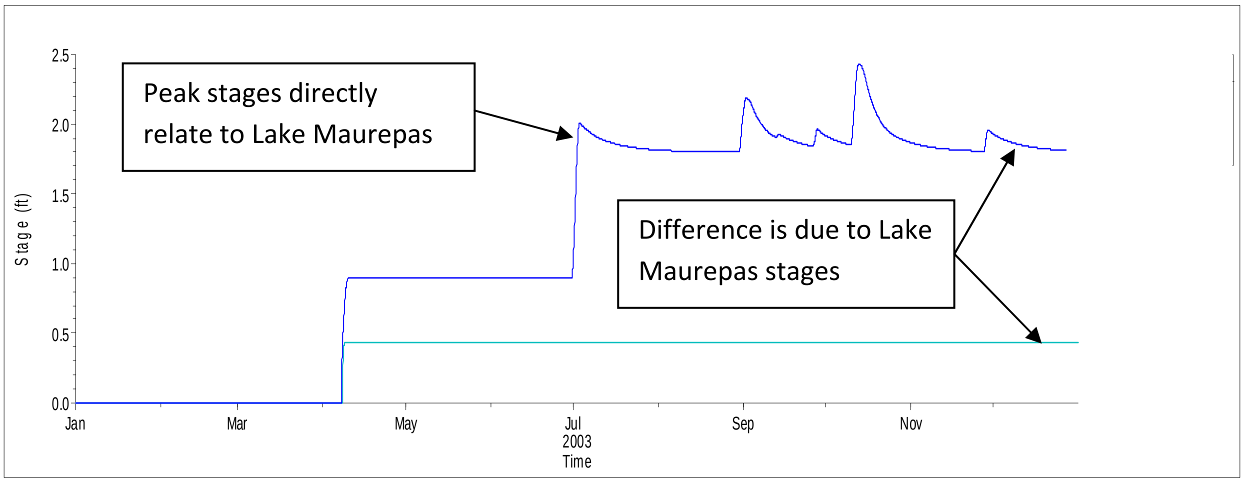



**Figure L2.3.5-1 Influence of Lake Maurepas on Blind River System Stages**

**Figure L2.3.5-2** presents flow simulated in the Blind River at I-10 in HEC-RAS with 2003 rainfall and observed water surface elevations at Lake Maurepas. As shown, flow in the Blind River at I-10, the downstream limit of the project area, is both positive and negative over the course of the year. This occurrence of flow in two directions further demonstrates the influence Lake Maurepas has on the Blind River and the swamp, as well as the importance of including this downstream boundary condition in the hydraulic analysis.

Review of the observed water surface elevations at Lake Maurepas during 2003 indicates that four peak water surface elevations occurred that were significantly higher than other peak elevations measured in 2003. Additional research of hydrologic conditions during 2003 revealed that three of the four highest peak water surface elevations at Lake Maurepas during 2003 were caused by Tropical Storms Bill, Isidore, and Lily that tracked through the region. The fourth peak water surface elevation occurred in response to a large regional precipitation event that caused flooding in portion of the Amite River watershed, a much larger tributary to Lake Maurepas than the Blind River. Additional information about the frequency of Lake Maurepas water surface elevations and associate effects on the study area is provided in Section L2.9.

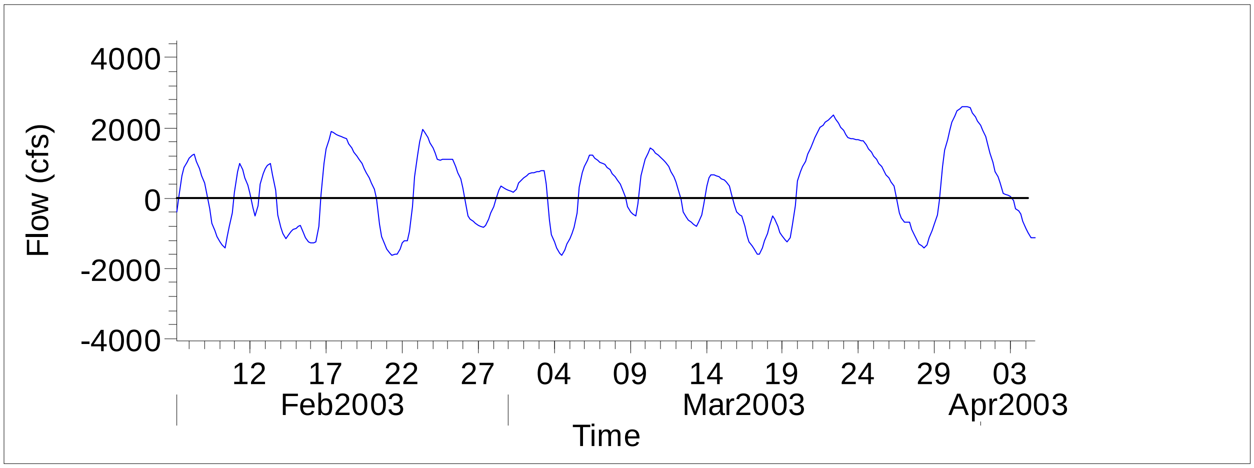



**Figure L2.3.5-2 Blind River Flow at I-10**

**Typical Flow Patterns**

HEC-RAS test simulations were also conducted to understand typical flow patterns in the existing drainage canals and the swamp. **Figure L2.3.5-3** shows typical flow patterns and flow magnitudes calculated by HEC-RAS in response to 2003 hydrologic conditions. Observations from the model results suggest that a number of factors influence flow patterns that occur in response to frequent rainfall events that occur in the Blind River watershed:

- Drainage canal cross-section dimensions;
- Locations and elevations of existing berms;
- Elevations of ground elevations adjacent to the existing drainage canals;
- Tributary area to each lateral drainage canal; and
- Blind River channel and overbank cross-sections.

**L2.3.5.3 HEC-RAS Model Results**

Two types of simulations were completed with HEC-HMS and HEC-RAS; a simulation of the watershed with design storm rainfall and a simulation using observed rainfall and Lake Maurepas water elevations during the year 2003, an average hydrologic year. Model results produced with design rainfall depths are intended to define peak water surface conditions in the existing study area for comparison with project alternatives to identify the potential adverse project impacts to flooding. Model results produced with the simulation of the year 2003 are intended to establish existing hydroperiod characteristics for comparison with the project alternatives.

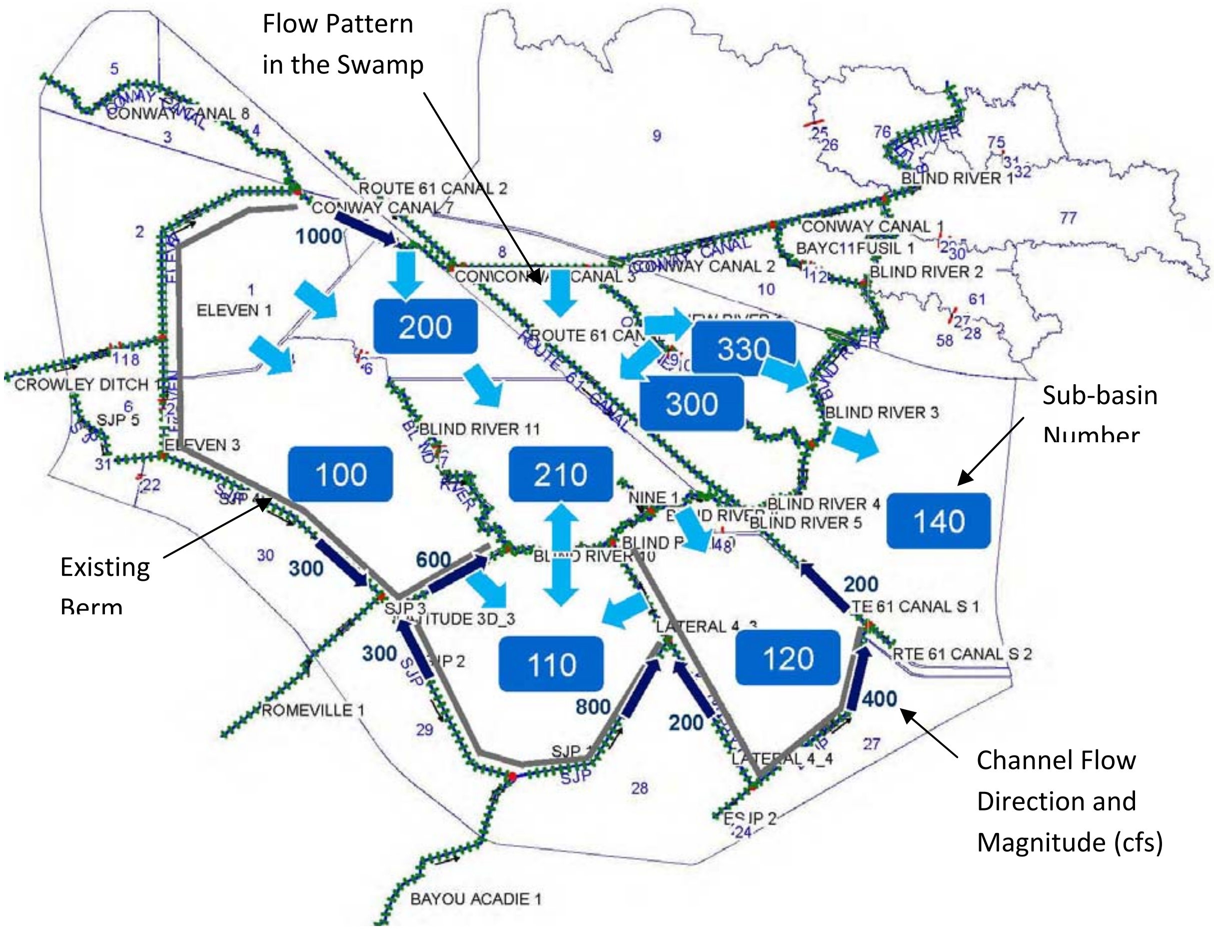



Figure L2.3.5-3 Typical Flow Patterns

The results for each type of simulation are summarized as follows:

- Peak Water Surface Elevations in feet-NAVD were produced by the design storm simulations at locations throughout the existing drainage canal network, and are reported for the 2-, 5-, 10-, 25-, 50- and 100-year rainfall events (**Table L2.3.5-1**).
- Average Water Surface Elevations in feet-NAVD were produced by the simulation of the year 2003 for specific hydrologic units in the swamp (**Table L2.3.5-2**).
- Net Freshwater Throughputs in acre-feet were produced by the simulation of the year 2003 for specific hydrologic units in the swamp (**Table L2.3.5-3**). The net freshwater throughput is calculated as the total inflow to a hydrologic unit minus the inflow volume attributed to backflow from Lake Maurepas.
- Backflow in acre-feet were produced by the simulation of the year 2003 for specific hydrologic units in the swamp (**Table L2.3.5-4**).

Average water surface elevations and volumes for throughput and backflow were calculated for existing sea level conditions and with projected increases in mean sea level that will result in increases to Lake Maurepas water levels. Similar results are calculated for the array of alternatives (Section L2.10) for consistency with USACE policy for considering the effects of sea level rise in civil works programs. The sea level rise scenarios presented correspond to projected sea level rise for 20-year, 30-year and 50-year increments that are utilized by the Wetland Value Assessment (WVA) to quantify project benefits.

**Table L2.3.5-1 HEC-RAS Peak Water Surface Elevations**

| Location | HEC-RAS Cross-Section Number | Water Surface Elevation (feet - NAVD) | | | | | |
|---|---|---|---|---|---|---|---|
| | | 2-Yr | 5-Yr | 10-Yr | 25-Yr | 50-Yr | 100-Yr |
| Conway Canal | 39851.31 | 3.35 | 4.13 | 4.68 | 5.26 | 5.63 | 5.92 |
| Conway Canal | 19048.58 | 1.17 | 1.33 | 1.42 | 1.51 | 1.57 | 1.65 |
| Crowley Ditch | 474.9322 | 3.31 | 3.79 | 4.07 | 4.40 | 4.60 | 4.90 |
| St. James Parish Canal | 45041.35 | 3.34 | 3.82 | 4.07 | 4.40 | 4.59 | 4.90 |
| St. James Parish Canal | 28520.59 | 2.80 | 3.32 | 3.67 | 4.05 | 4.34 | 4.69 |
| St. James Parish Canal | 14574.43 | 3.07 | 3.55 | 3.90 | 4.27 | 4.55 | 4.88 |
| East St. James Parish Canal | 13175.13 | 2.58 | 3.07 | 3.38 | 3.71 | 3.96 | 4.29 |
| Blind River | 42581.05 | 1.37 | 1.56 | 1.69 | 1.80 | 1.88 | 2.00 |
| Blind River | 27088.77 | 0.95 | 1.07 | 1.14 | 1.21 | 1.26 | 1.32 |

Note: Design storm simulations completed with downstream boundary condition water surface of 0.5 feet.

**Table L2.3.5-2 HEC-RAS Average Water Surface Elevations**

| Sea Level Condition | Water Surface Elevation (feet, NAVD) by Sub-basin Number and Hydrologic Unit | | | | | | |
|---|---|---|---|---|---|---|---|
| | 100 | 200 | 210, 220 | 110 | 120, 160 | 300, 320, 330 | 140, 150 |
| | HU 1 | HU 2 | HU 3 | HU 4 | HU 5 | HU 6 | HU 7 |
| Existing | 1.79 | 1.34 | 1.66 | 1.74 | 1.64 | 1.36 | 1.37 |
| 20-Year | 1.98 | 1.74 | 1.77 | 1.98 | 2.06 | 1.72 | 1.75 |
| 30-Year | 2.21 | 2.09 | 2.12 | 2.19 | 2.32 | 2.06 | 2.09 |
| 50-Year | 2.85 | 2.83 | 2.84 | 2.85 | 3.00 | 2.81 | 2.81 |

- Note: Average water surface elevation based on simulation of hydrologic conditions observed in 2003.

**Table L2.3.5-3 HEC-RAS Net Freshwater Throughput Volumes**

| Sea Level Condition | Net Freshwater Throughput (Acre-feet) by Sub-basin Number and Hydrologic Unit | | | | | | |
|---|---|---|---|---|---|---|---|
| | 100 | 200 | 210, 220 | 110 | 120, 160 | 300, 320, 330 | 140, 150 |
| | HU 1 | HU 2 | HU 3 | HU 4 | HU 5 | HU 6 | HU 7 |
| Existing | 20,700 | 47,400 | 55,900 | 9,400 | 3,900 | 172,100 | 98,300 |
| 20-Year | 62,300 | 63,500 | 121,300 | 34,800 | 13,300 | 226,200 | 69.900 |
| 30-Year | 87,300 | 75,800 | 196,300 | 52,000 | 17,900 | 346,300 | 121,000 |
| 50-Year | 127,900 | 91,200 | 359,800 | 97,600 | 25,500 | 554,800 | 248,800 |

Note: Average water surface elevation based on simulation of hydrologic conditions observed in 2003.

**Table L2.3.5-4 HEC-RAS Backflow Volumes**

| Sea Level Condition | Backflow Volume (acre-feet) by Sub-basin Number and Hydrologic Unit | | | | | | |
|---|---|---|---|---|---|---|---|
| | 100 | 200 | 210, 220 | 110 | 120, 160 | 300, 320, 330 | 140, 150 |
| | HU 1 | HU 2 | HU 3 | HU 4 | HU 5 | HU 6 | HU 7 |
| Existing | 7,900 | 11,500 | 7,000 | 5,400 | 5,200 | 42,400 | 24,000 |
| 20-Year | 42,300 | 45,500 | 64,200 | 25,100 | 15,700 | 139,000 | 83,700 |
| 30-Year | 64,700 | 50,200 | 94,600 | 44,800 | 21,800 | 175,700 | 111,000 |
| 50-Year | 88,000 | 53,900 | 151,200 | 72,200 | 28,200 | 233,500 | 147,500 |

Note: Average water surface elevation based on simulation of hydrologic conditions observed in 2003.

In addition to the numerical HEC-RAS results presented above, a summary of general observations and conclusions based on review of the HEC-RAS model results for existing conditions are provided below:

- Local drainage contributes storm runoff to the Blind River and surrounding swamp from multiple rainfall events each year. Most storm runoff is conveyed by existing drainage channels directly to the Blind River.
- Under existing conditions, the swamp has minimal circulation of water, and the only water movement occurs during an average of 5-7 rainfall events per year that are large enough to exceed drainage canal capacity and contribute flow to the swamp. Although not simulated by HEC-RAS, a reasonable inference is that contribution and circulation of nutrients and sediment will also be minimal and limited under existing conditions.

- Water levels in Lake Maurepas significantly influence the ability for the Blind River system to drain, and significant backflow from Lake Maurepas to the Blind River system occurs multiple times per year.
- Peak flows and stages for the Blind River range from 500 to 6,000 cfs and -0.5 to 2.5 feet (NAVD), respectively for the 2003 simulation year.
- Three of four peak stages in Lake Maurepas during 2003 were the result of tropical storms in the region (Bill, Isidore, Lily), and the fourth peak stage in Lake Maurepas resulted from a large regional system that caused significant flooding in the Amite River watershed.
- Local runoff to the Blind River can occur during periods when Lake Maurepas levels are high, but typically does not coincide with peak lake levels.
- Lake Maurepas stage is not closely related to runoff flows contributed to the Blind River. Lake Maurepas stage appears to be most closely related to runoff response from the larger Amite River watershed.
- Peak Lake Maurepas levels rise and recede on the order of two weeks while runoff in the Blind River watershed occurs in 2-3 days.
- The US 61 bridge has a maximum capacity of approximately 7,000 cfs while the I-10 bridge has a much larger capacity. Half of the US 61 bridge opening is below elevation 0.5 feet (NAVD). With a flow of 3,500 cfs the US 61 bridge will have 2 feet of freeboard from the low chord elevation of 4.5 feet with a downstream water elevation of 0.5 feet at Lake Maurepas.
- Future conditions that include both mean sea level rise and continued subsidence will increase the magnitude of land area in the swamp that is inundated during average hydrologic conditions, primarily because the average and peak water elevations in Lake Maurepas will increase relative to the ground elevation in the swamp. While this will deliver more water to the swamp, conditions will continue to be stagnant and the risks of potential impacts associated with inundation resulting from storm surge, such as salinity, will increase.

### L2.3.6 Engineering Calculations

The following section builds on the methodology discussion presented in Section L2.2.4, and explains how the engineering calculations were tested with comparisons to HEC-HMS results, HEC-RAS results, and another nearby stream gage. Once tested, the engineering calculations were used to extend the period of record for hydraulic analysis from 1989-2004. At the conclusion of this section, engineering calculations results are provided for existing conditions over the 1989-2004 period of record.

#### L2.3.6.1 Engineering Calculations Set-up

The study area is represented conceptually for the purpose of the engineering calculations on Figure L2.2.4-1. As shown, the existing swamp is comprised of seven hydrologic units in the engineering calculations, and includes many flow paths between each of the swamp areas, drainage canals and the Blind River. Figure 2.2.4-1 also includes the conceptual locations of alternatives for the Mississippi River diversion, shown in red arrows. Additional discussion of diversion alternatives screening is provided in Section L2.4, and discussion of the final alternatives is included in Section L2.10.

Existing conditions represent all the possible flow paths, and only allows water exchanges into or out of the seven swamp hydrologic units if the water level is higher than the estimated berm crest elevation between adjacent swamp areas or swamp areas and adjacent canals. Results for existing conditions will be compared with results for additional scenarios that include gaps in the berms to improve freshwater delivery to the swamp, from the Mississippi River, and drainage out of the swamp. For clarification, the engineering calculations analysis is not a rigorous hydraulic analysis, but rather a conceptual representation of the potential connectivity and water exchange potential that could occur under a range of hydrologic conditions over the period of record from 1989 to 2004. The explicit representation of hydraulics is included in the HEC-RAS and EFDC analyses.

#### L2.3.6.2 Engineering Calculations Testing

As stated, very little measured historical data are available to check the accuracy or predictive strength of the equations. In this case, the equations were developed and then tuned to match results from the HEC-HMS and HEC-RAS models to show that the results could be reproduced with independent techniques using standard parameters with reasonable values. Fundamentally, the equations are intended to increase overall credibility of the HEC models, extend the period of record that can be analyzed, and improve overall understanding of the dynamics of this system in simple hydrologic terms.

Three types of cross-checking were conducted between the standard equations and the HEC models. Results of each are presented below:

- Runoff time series generated with the standard equations were compared with HEC-HMS results for each contributing subwatershed.
- Time series of water stage within the swamp generated with the standard water balance equations were compared with results from HEC-RAS to test storm response (peak stage and drainage time) as well as longer-term patterns of filling and draining.
- Runoff time series in the Blind River as computed with these equations was compared to transposed river gage data from a nearby watershed – this effectively tested the overall efficacy of the calculations with respect to both runoff and the passage of water through the swamp.

**Watershed Runoff Time Series**

Per the methodology described in Section L2.2, the daily runoff from each of the contributing subwatersheds to the study area (not including the study area itself) was estimated using standard hydrologic equations that account for bulk loss of total precipitation, retention of water in the soil, and gradual discharge of soil moisture into the canals flowing into and through the study area. **Figure L2.3.6-1** shows the contributing watersheds (areas 400 – 770) in addition to the study area (areas 100-330).

**Figures L2.3.6-2** to **L2.3.6.2-13** illustrate the runoff performance as estimated with the standard equations, and compared with the HEC-HMS predictions for the year 2003 (which represents typical hydrologic conditions).

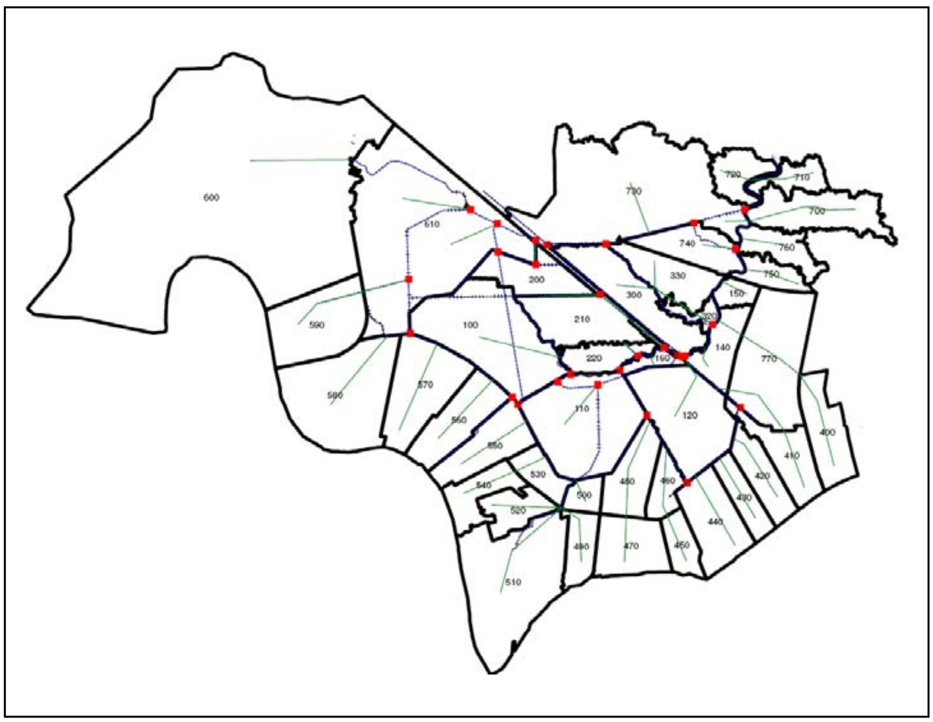



**Figure L2.3.6-1 Contributing Watershed Surrounding the Study Area**

The resulting parameters (listed in the figures for each watershed) were determined using the following criteria:

- Total annual runoff predicted by HEC-HMS would be preserved with the simplified calculations;
- The timing and magnitude of peak flows would be matched reasonably well (judging qualitatively); and
- The slope of the hydrograph recession curves would also be matched reasonably well.

When these criteria are applied, most of the contributing watersheds are hydrologically "flashy" – that is, runoff occurs very quickly after rainfall events, and very little precipitation is lost to evaporation or groundwater seepage (the contributing watershed areas do not include the study area itself). These findings were compatible with the basic hydrologic features of the contributing watersheds, which are geographically small and characterized by poorly drained soils (predominantly Type D soils per SCS).

Fundamentally, then, in the absence of data with which to confirm HEC-HMS hydrologic predictions, the basic rainfall-runoff dynamics of the system as predicted by HEC-HMS could be reproduced using standard hydrologic relationships with parameters that reasonably conform to the hydrographic characteristics of the contributing subwatersheds.

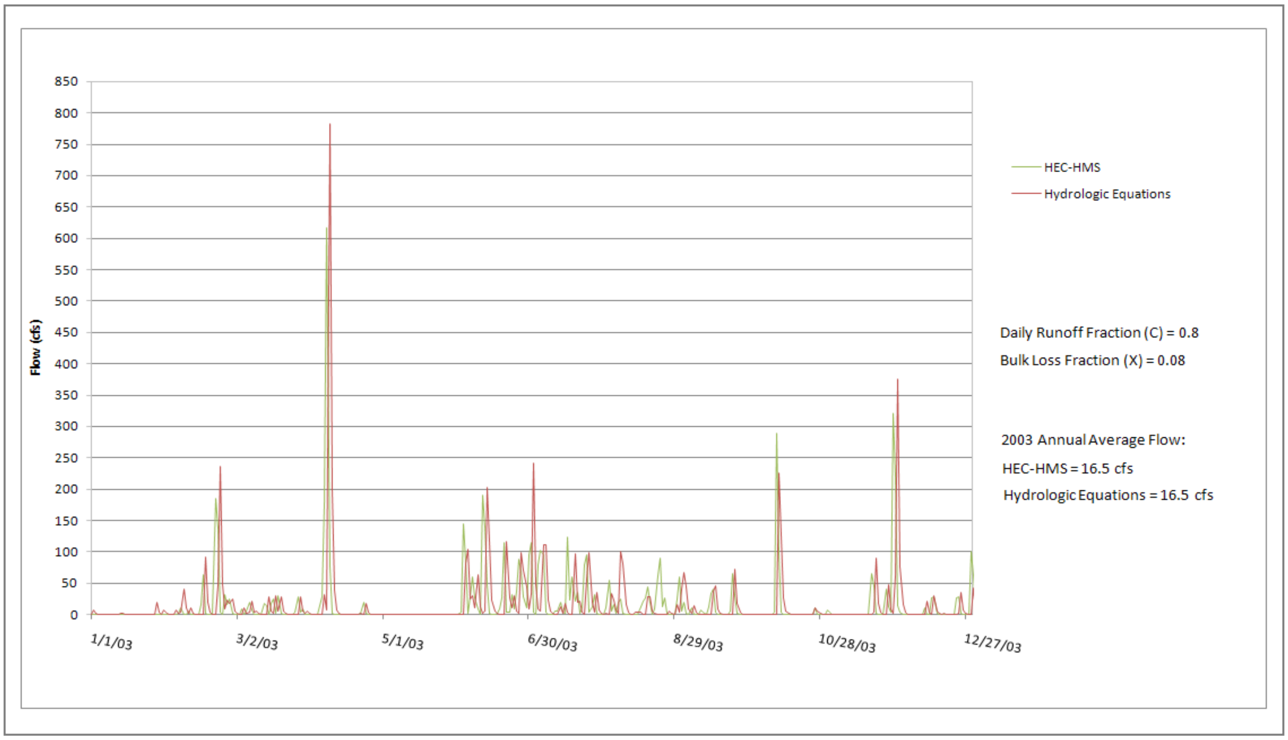



**Figure L2.3.6-2 South Bridge Canal Runoff (Sub-basin 590)**

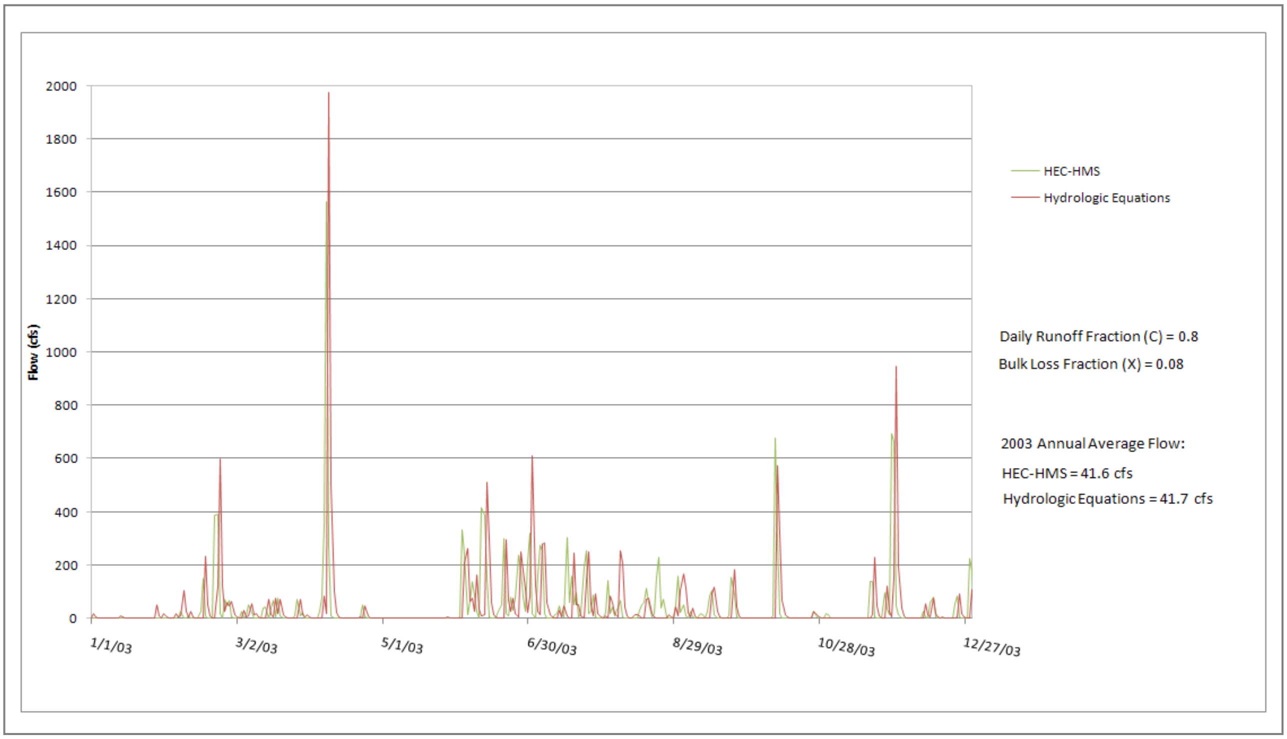



**Figure L2.3.6-3 St. James Parish Canal Runoff (Sub-basins 570, 580)**

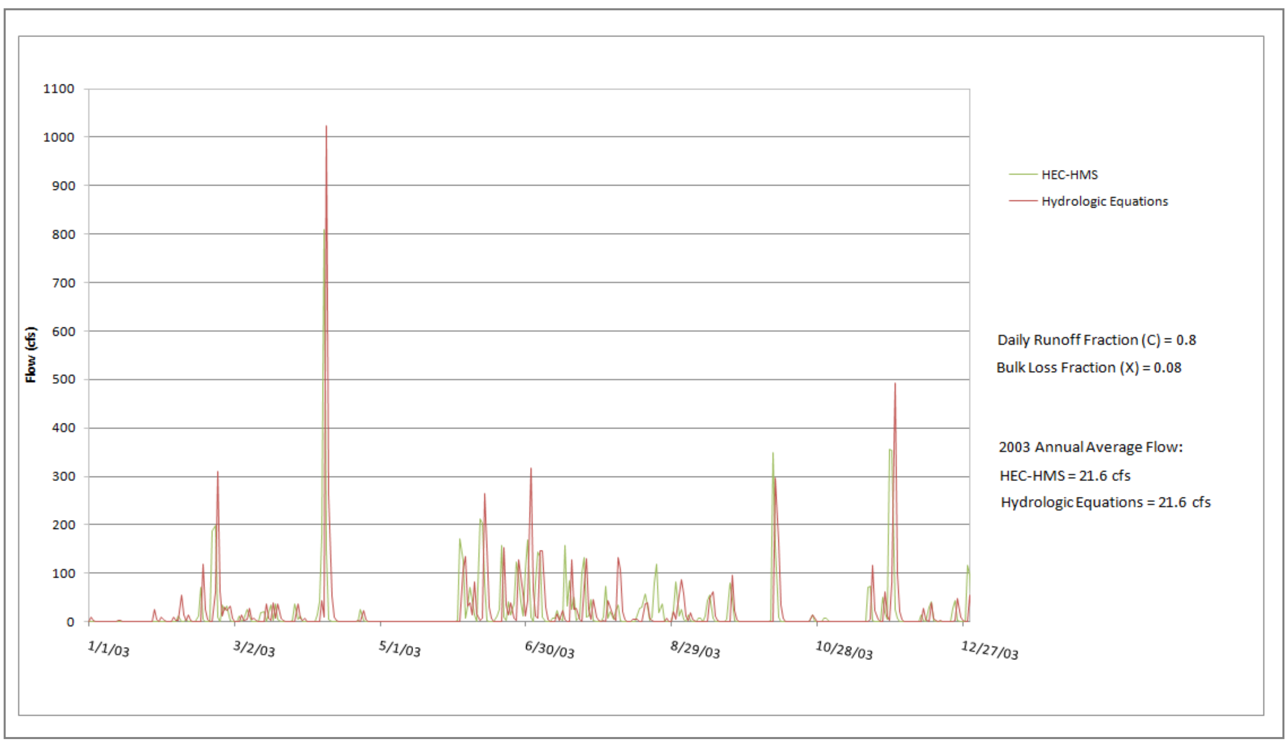



**Figure L2.3.6-4 Romeville Canal Runoff (Sub-basins 550, 560)**

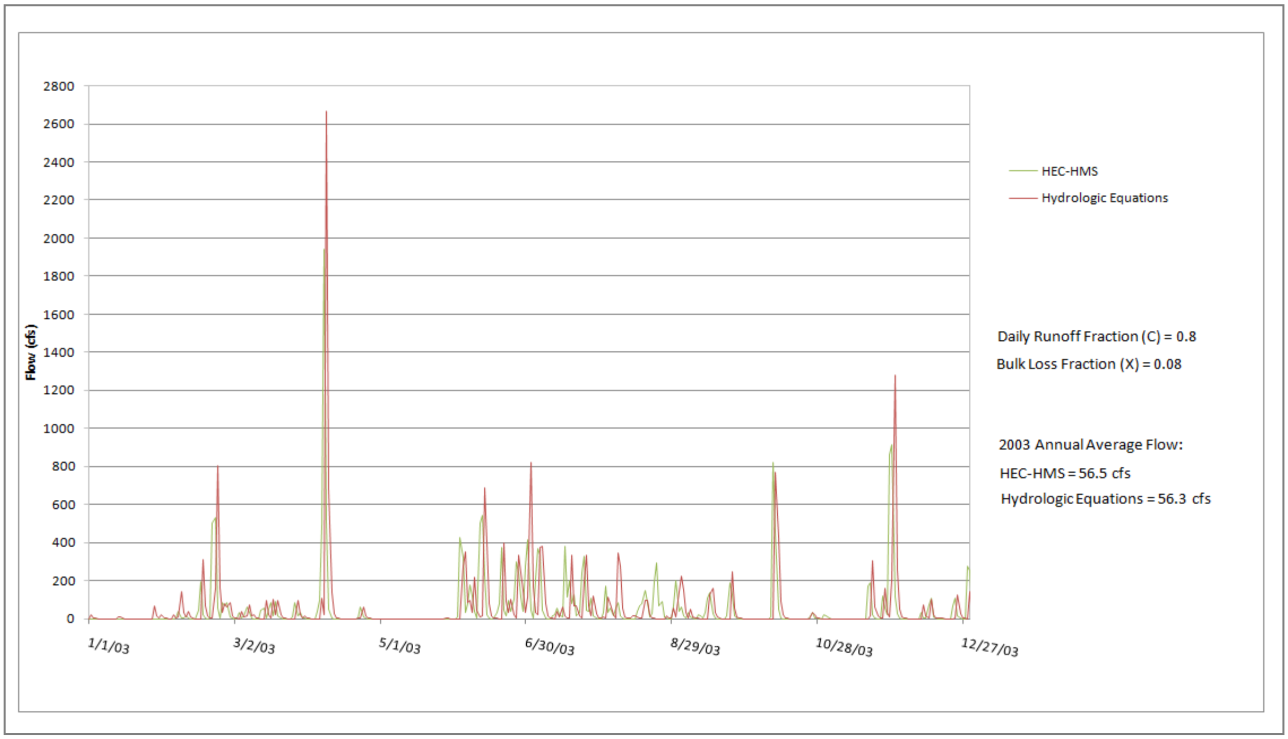



**Figure L2.3.6-5 External Drainage Canal 1 Runoff (Sub-basins 490, 500, 510, 520, 530, 540)**

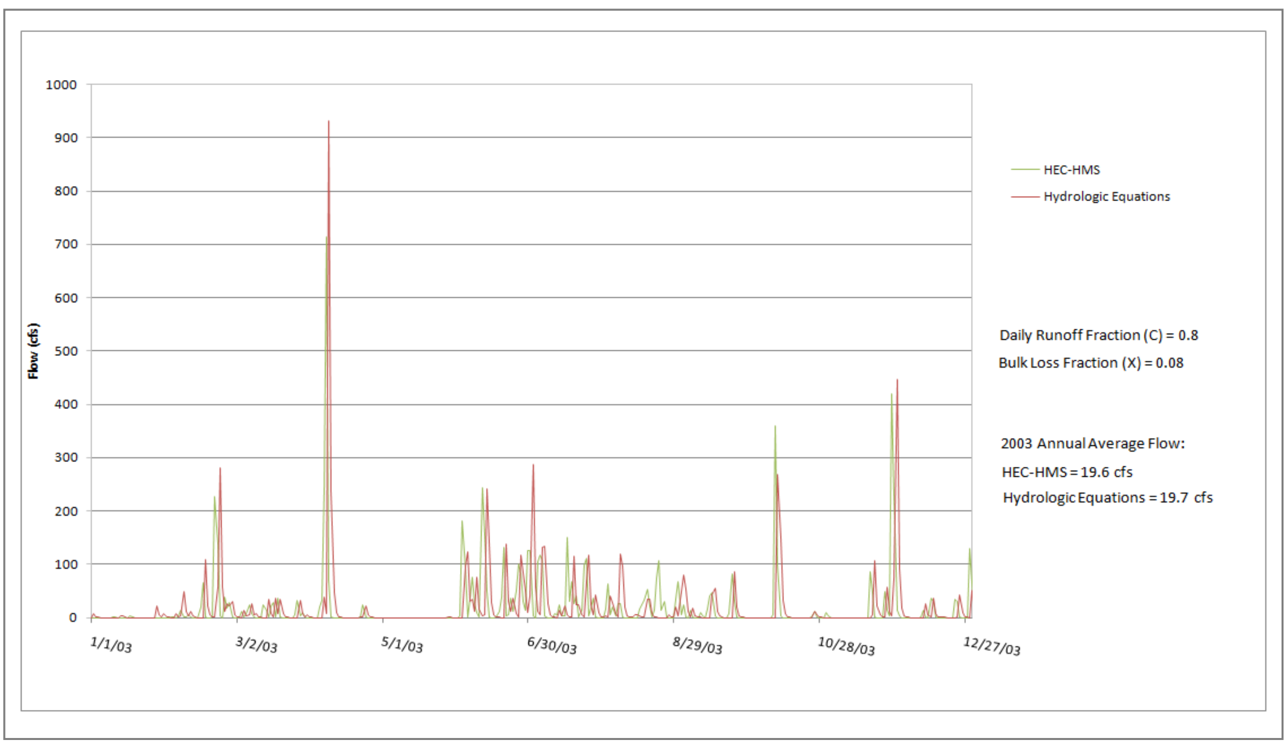



**Figure L2.3.6-6 External Drainage Canal 2 Runoff (Sub-basins 470, 480)**

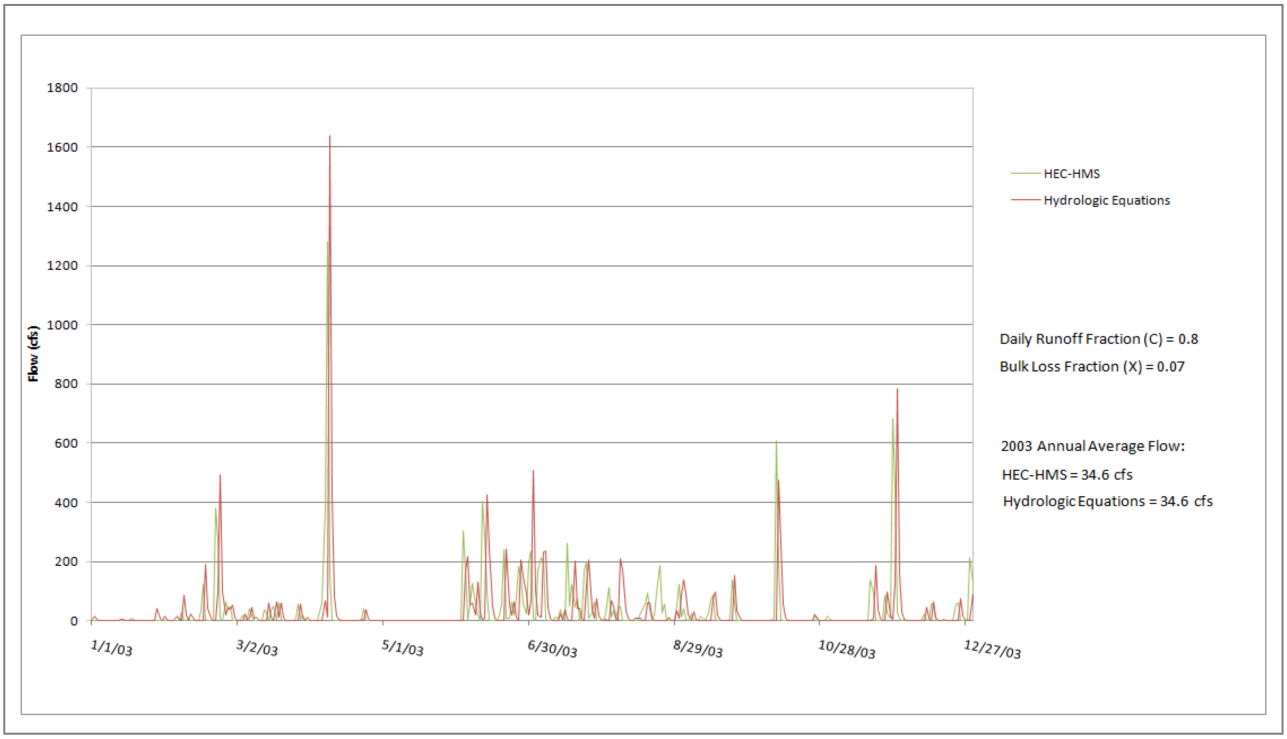



**Figure L2.3.6-7 External Drainage Canal 3 Runoff (Sub-basins 420, 430, 440, 450, 460)**

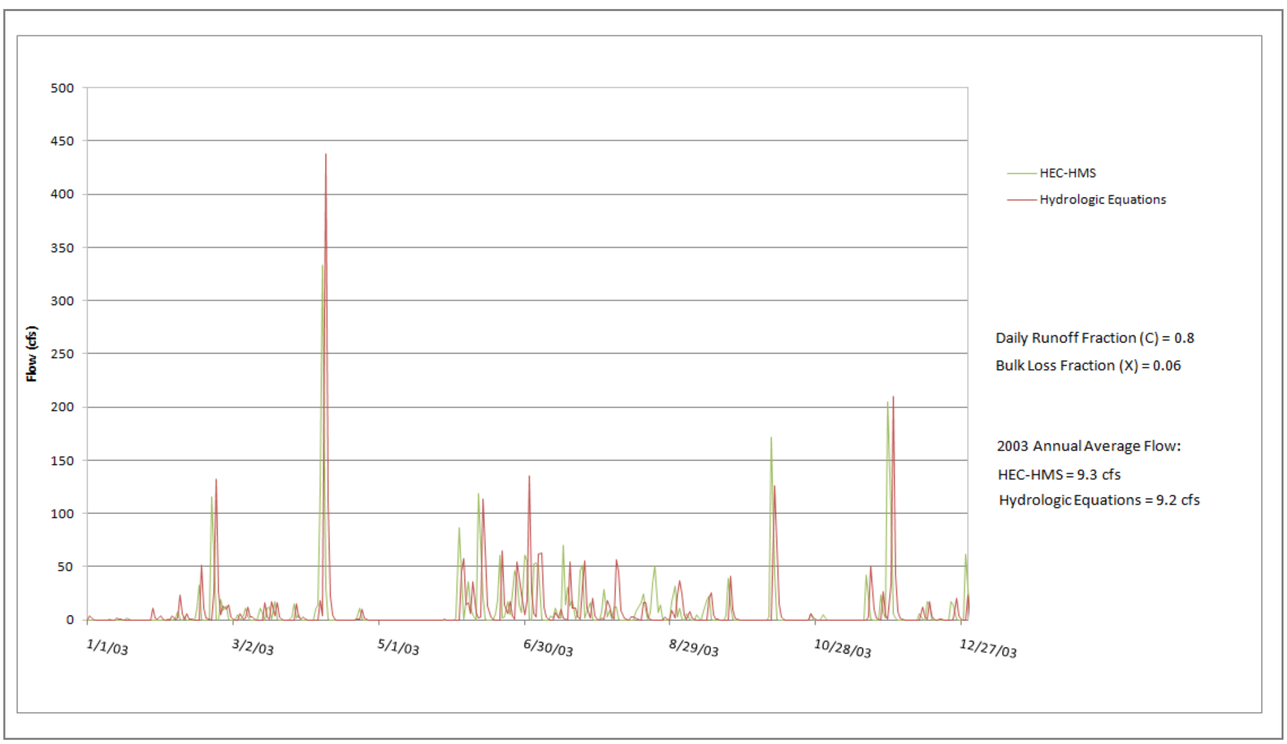



**Figure L2.3.6-8 Southeast US 61 Canal Runoff (Sub-basin 410)**

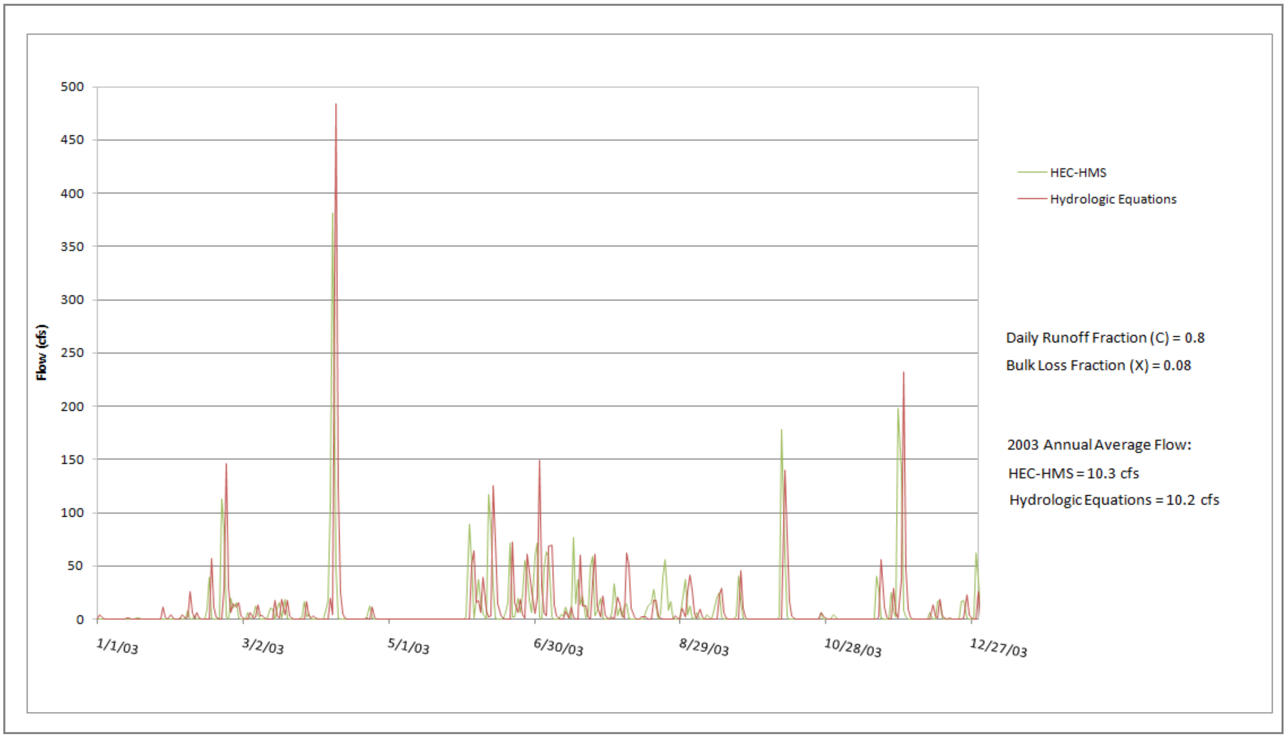



**Figure L2.3.6-9 Watershed Runoff (Sub-basin 400)**

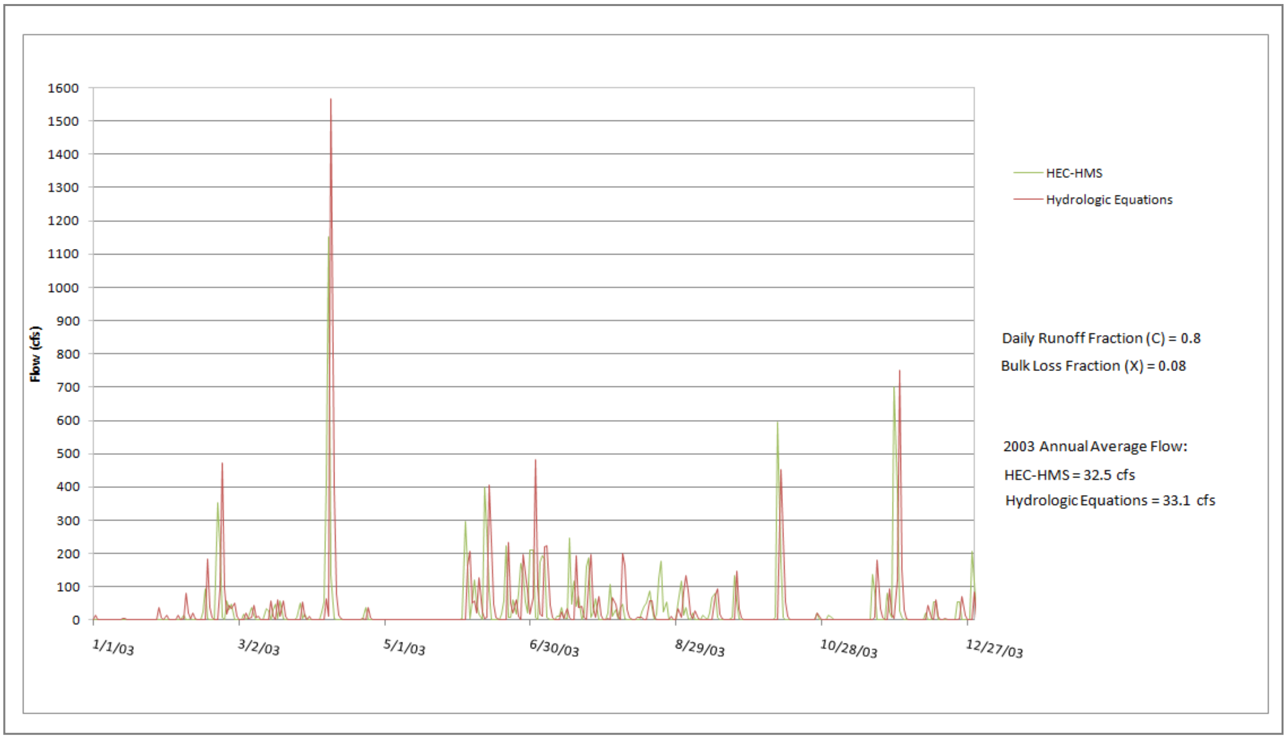



**Figure L2.3.6-10 Watershed Runoff (Sub-basins 700, 740, 750, 760)**

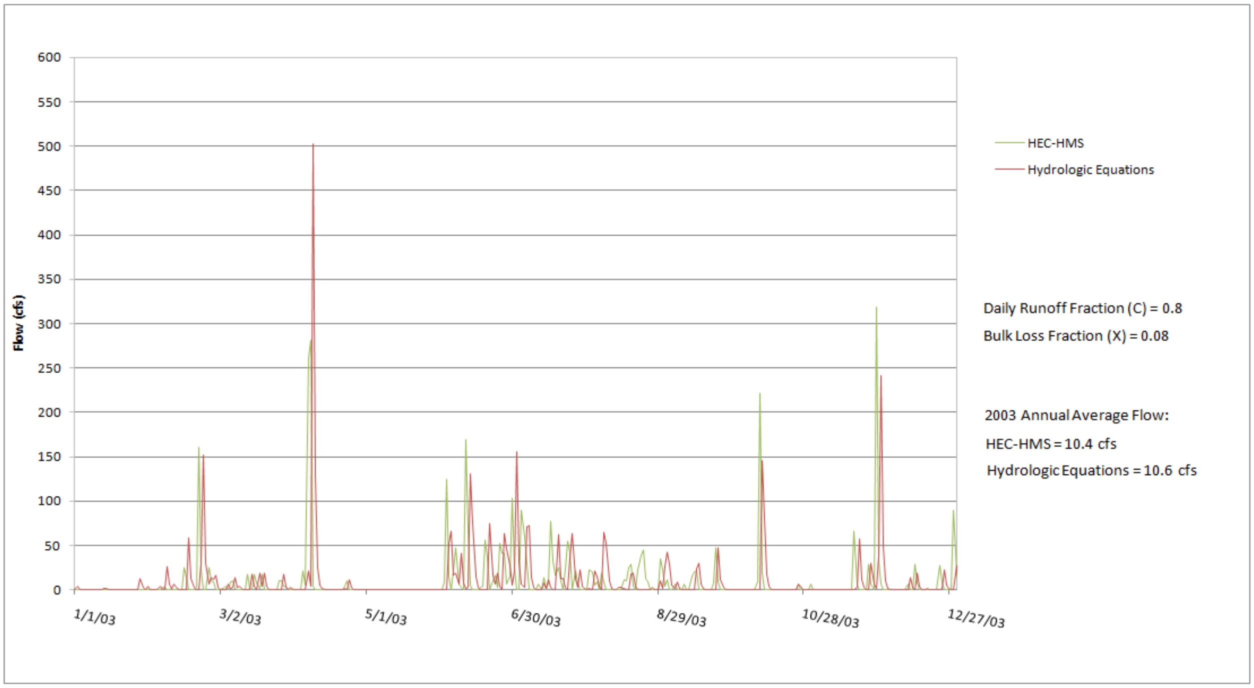



**Figure L2.3.6-11 Watershed Runoff (Sub-basins 710, 720)**

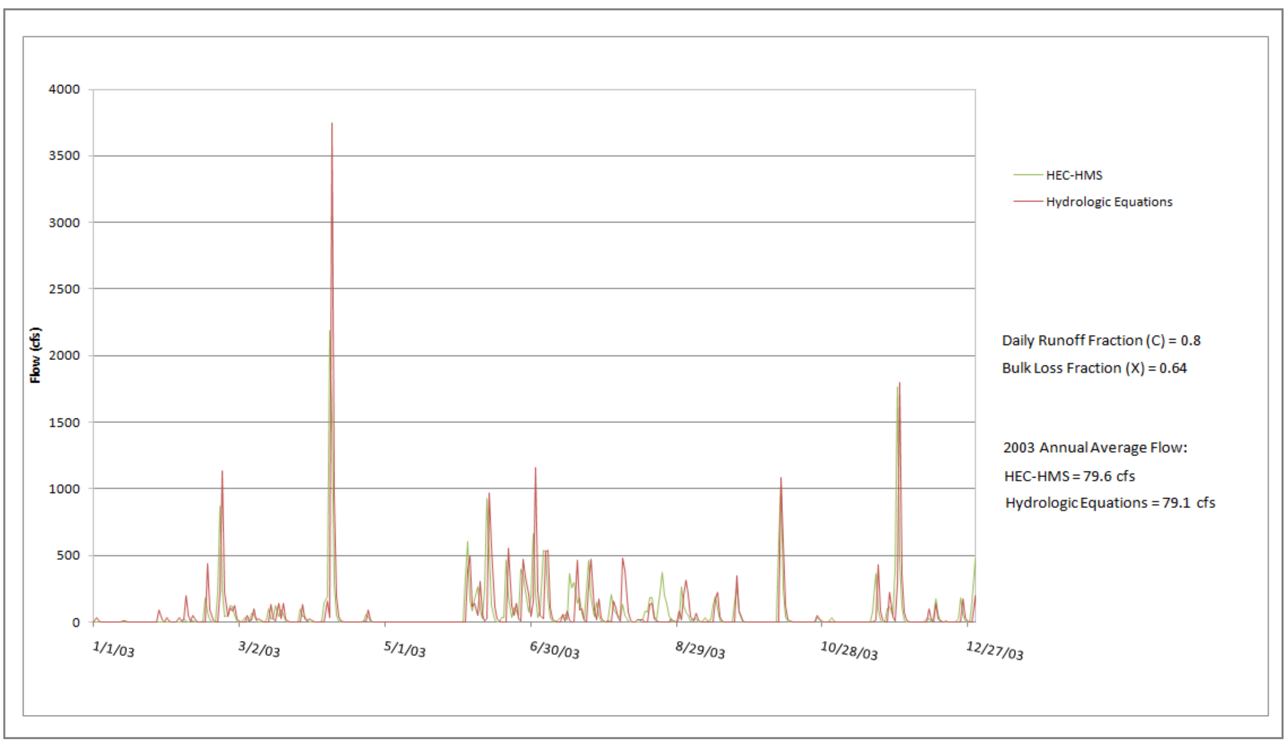



**Figure L2.3.6-12 Watershed Runoff (Sub-basins 600, 610)**

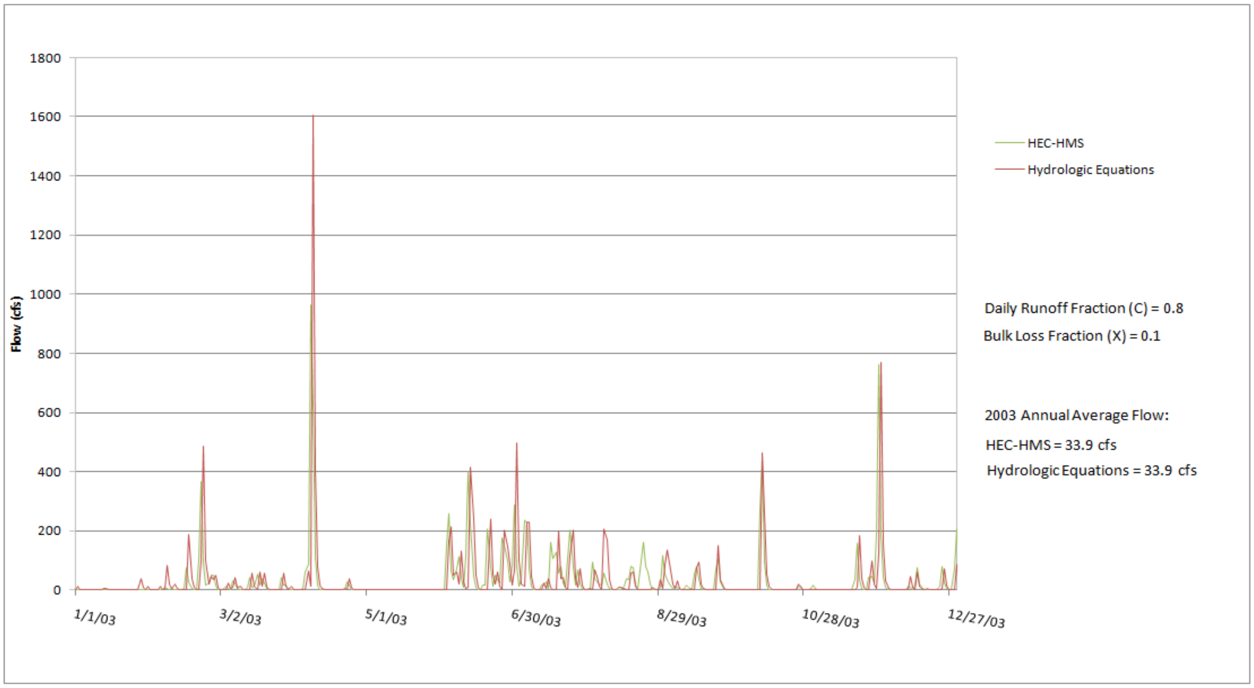


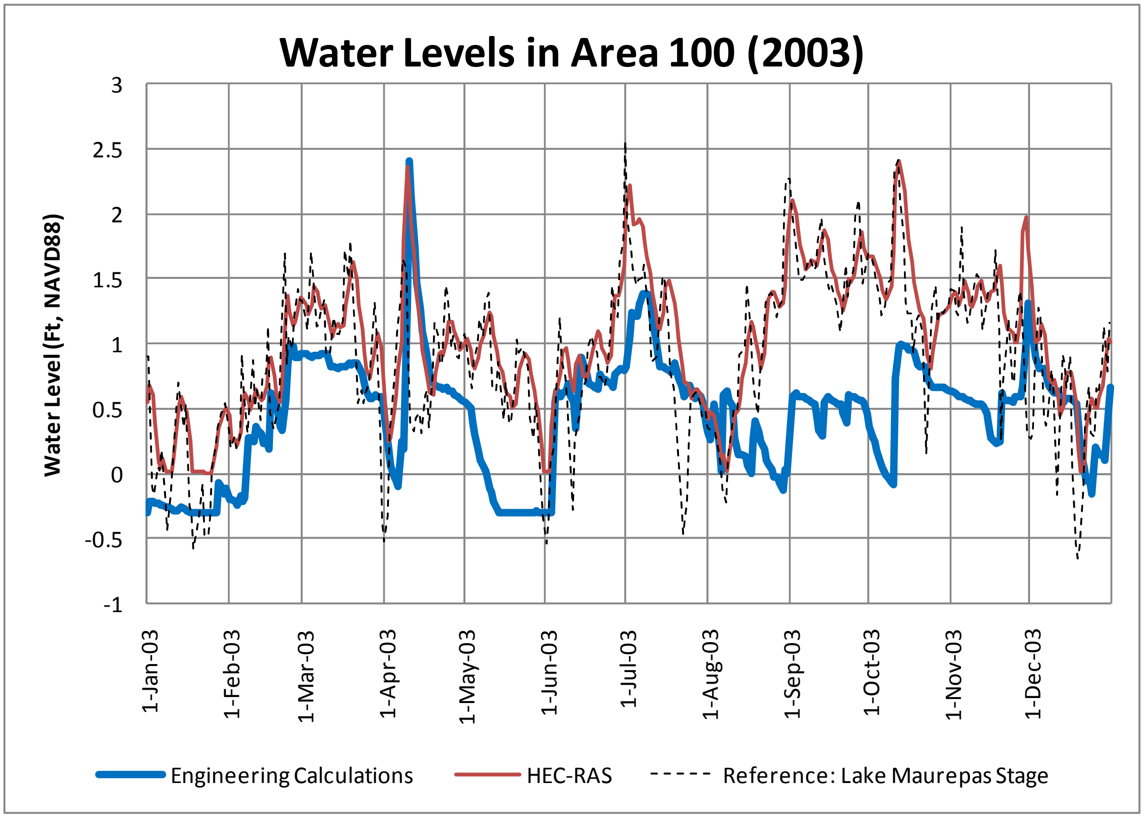

**Figure L2.3.6-13 Conway Canal 2 Runoff (Sub-basin 730)**

**Hydrologic Response Patterns within the Swamp**

The engineering equations also compute generalized routing trends as the runoff (and diversions) pass through the swamp, in accordance with the logic described in Section L2.2.4. The recession coefficient applied to the entire swamp area (governing the rate of discharge from overflowing swamp areas to a canal, the Blind River, or adjacent swamp areas) was adjusted to match the overall rate at which the HEC-RAS model predicted drawdown of the swamp area.

**Figure L2.3.6-14** illustrates the simulated response in swamp area 100 (as a typical example). Several important aspects of the swamp dynamics as represented in the engineering equations:

- The rate at which the swamp drains (draws down) following large storms matches the rate predicted by HEC-RAS reasonably well. Different recession coefficients were applied to represent differences in drawdown tendency for the swamp with and without cuts in the berms (these cuts also allowed drawdown to lower elevations). Without berm cuts, the recession coefficient that best matched HEC-RAS drawdown patterns was 0.4 throughout the swamp. That is, on any day, if there was surplus water in a swamp area above the berm elevation, 40% of that surplus would flow to downstream or adjacent water bodies. Over time, the pattern is exponential drawdown. Likewise, to represent berm gaps, the recession coefficients ranged between 0.2 and 0.3 for the various swamp areas – slightly slower overall drawdown, but the faster drawdown associated with no berm gaps only occurs at very high water surfaces (above berm crests), and is not expected to occur over the much broader range of elevations in which connectivity is afforded with gaps in the berms. (These values were tuned to match the recession rates observed in the HEC-RAS model, mostly to accommodate the extension of the period of record, and not necessarily to prove the validity of either approach – unlike standard rainfall-runoff dynamics, the swamp dynamics are less intuitive and not as easily correlated to physical landscape features).




- *Results are for existing conditions with berm cuts (no diversions)*
- *Results show similar fill times and drawdown times, both short-term (days) and longer-term (week/months).*
- *Differences in absolute water elevations are attributed to the inclusion of free-surface evaporation in the engineering calculations, and the inclusion of backwater influence in the HEC-RAS results. When the blue trace of the engineering calculations (fresh water only) is below the dashed trace of Lake Maurepas, the results are suggestive of backflow potential from the lake into the swamp (not enough fresh water input on its own to provide enough countering head). These differences become much smaller when diversion flow is added into the swamp.*
- *Only swamp area 100 is shown in the graph above. Results are representative of other swamp areas.*

**Figure L2.3.6-14 Comparison of Swamp Fill and Drain Dynamics in Sub-basin 100**

- Differences between the HEC-RAS trace for 2003 and the engineering equation results can be explained with two important distinctions:
- The HEC-RAS model includes backwater calculations from Lake Maurepas, and hence, the water level in the swamp never drops below that of the Lake. The engineering equations do not account for backflow, just runoff and direct precipitation (and diversions when alternatives are analyzed later). Hence, when the trace representing the engineering calculations drops below the lake level, this is representative of times when the lake would have the potential to flow back into the swamp.
- The engineering equations account more specifically for surface evaporation from the open water surfaces in the swamp than is possible in HEC-RAS.

When diversions were included in the water balance, the differences between the engineering calculations and the HEC-RAS results were substantially reduced. Therefore, it was determined that the general dynamics of filling time, draining time, and peak water levels were captured effectively with the engineering calculations, and that the calculations could be used to extend the period of record analysis from 1989-2004.

**Flow in the Blind River Downstream of Study Area**

Because there is no long-term continuous flow record for the Blind River, the computed outflow from the study area to the river (as computed with the engineering calculations) was compared to a synthesized flow record using data from a nearby watershed. The accumulated effects accounted for in the calculated data include both the hydrologic and hydraulic dynamics described above: watershed runoff and the retention/passage of that water through the canals and swamp areas under investigation (without any external diversions).

The goal of this comparison was not to achieve exact replication of the synthesized flow data, since the transposition itself is subject to uncertainty. Specifically, uncertainty with respect to hydrologic similarity, effects of impounded water within the swamp, precipitation patterns, etc. render an exact comparison impractical.

Rather, the comparison was made to determine if the total hydrologic output from the Blind River study area, as represented by the engineering equations, was reasonable. A number of USGS stream flow gages were considered as potential reference gages with which to compare computed Blind River flow. Ultimately, the gage that was selected for comparison was on the Natalbany River at Baptist (USGS Station ID #07376500, identified as Station #9 on **Figure L2.3.6-15**).

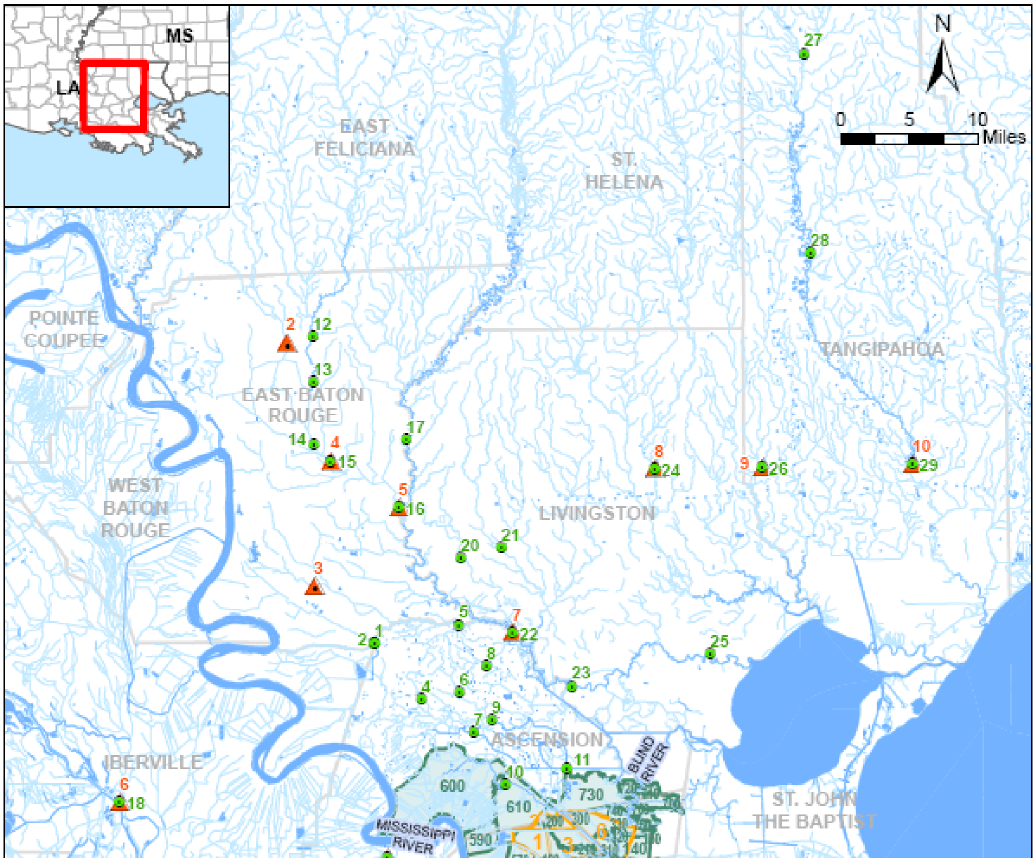



**Figure L2.3.6-15 Map of Comparative USGS Stream Flow Gages**

This station was selected for the following reasons:

- It is comparable in drainage area to the Blind River study area (79.5 square miles compared with 166 square miles for the Blind River). Many other drainage basins were either much smaller or much larger, and would therefore offer poor correlative value.
- It is comprised of similar land use types, though the fractions vary significantly. Both include substantial areas of woody wetlands (roughly a quarter of the drainage area of the Natalbany River, and more than half the drainage area of the Blind River).
- The available period of record includes the time period used for this analysis (1989-2004), which was determined based on the overlapping periods of necessary data from climate stations and Lake Maurepas.

Key differences between the reference basin and the Blind River Basin include:

- The soils in the Blind River Basin are more poorly drained, as indicated in Table L2.3.6-1. This suggests that unit runoff will be higher in the Blind River than in the Natalbany River.
- The precipitation near the Natalbany Basin (Measured at Hammond, LA – NOAA Station 4030) differs from that of the Blind River Basin (measured at Donaldsonville, LA – NOAA Station 2534), particularly later in the year during 2003. The comparison of 2003 monthly precipitation is shown in Figure L2.3.6-16. This suggests that more runoff would be observed in the Blind River during later months.

**Table L2.3.6-1 Soil Type Comparison**

| Hydrologic Soil Type | Percentage of Watershed Area | |
|---|---|---|
| | **Blind River Watershed*** | **Natalbany Watershed**** |
| A | -- | -- |
| B | -- | 11% |
| C | 23% | 58% |
| D | 77% | 30% |

***Based on data used in HEC-HMS*

***Based on analysis of NRCS soil data*

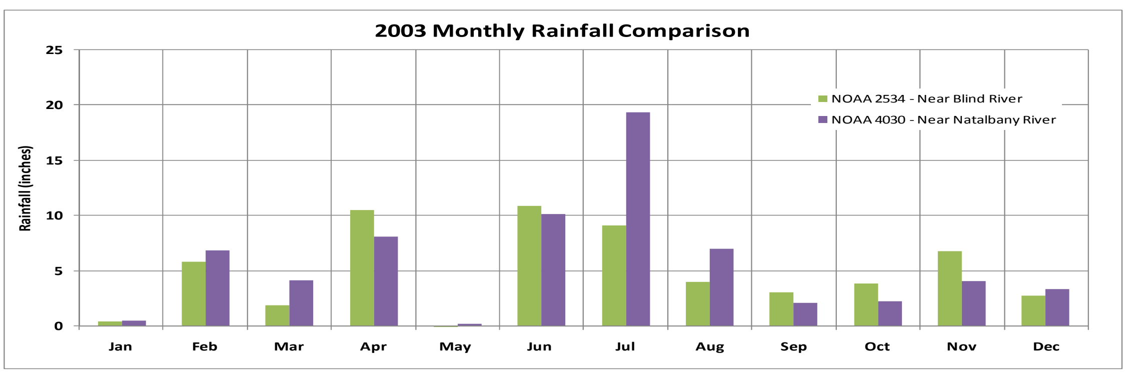
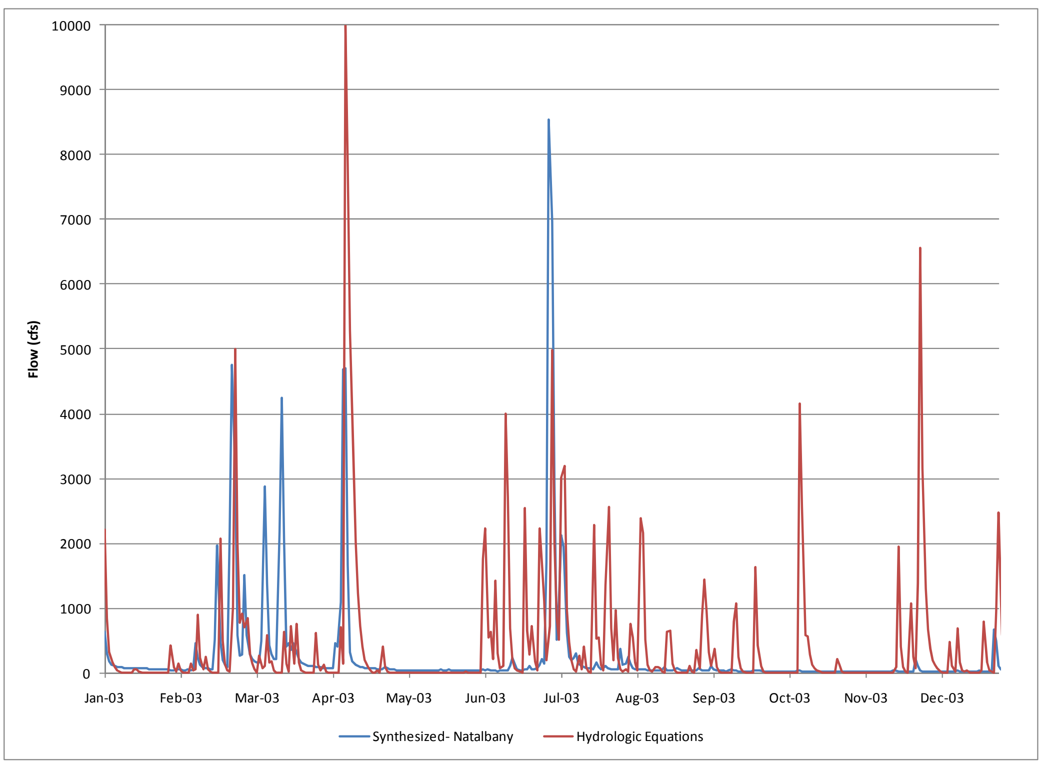



**Figure L2.3.6-16 Comparison of Precipitation near Blind River and Natalbany River**

The comparison of the engineering calculations (accounting for the combined effects of contributory runoff and simplified hydraulic routing through the swamp areas) with the measured Natalbany data is shown on **Figure L2.3.6-17**. The Natalbany flow data were scaled up by the drainage area ratio of 2.1:1 in order to approximate Blind River flows. As expected, the figure illustrates that the Blind River generally exhibits higher unit discharge, due in part to the poorer drainage ability of the soils.

Also, the figure shows more unit runoff in the Blind River in later months, due to the much higher precipitation in the Blind River vicinity during those months (and a corresponding higher unit runoff in the Natalbany River in July, when that basin received much more precipitation). Over the course of the year, the annual unit discharge for the Natalbany River was just 55% of the unit discharge for the Blind River, but this can be explained in part by the significant differences in soils and precipitation. There is, too, uncertainty inherent in the engineering calculations, which represent complex hydrologic and hydraulic phenomena with simplified relationships. Overall, the engineering equations yielded an average Blind River flow for 2003 (downstream of the study area) of 439 cfs, which accounts for approximately 36 inches of rainfall over the entire 166 square miles of upstream contributory area (or 60% of the total estimated rainfall of 59 inches). Fundamentally, the responses match expectations – higher unit discharge in the Blind River, much more discharge in the later months due to higher regional precipitation, overall high percentage of precipitation converted into runoff due to poorly drained soils, and an overall tendency to respond to large rainfall events with similar patterns of peak flow and hydrograph recession.




*"Hydrologic Equations" represents flow downstream of the study area, accounting for natural runoff and simplified hydraulic routing through the swamp, without diversions or berm cuts. "Synthesized Natalbany" represents measured flow in the Natalbany River scaled to the watershed size of the Blind River watershed.*

**Figure L2.3.6-17 Comparison of Scaled Flow at Natalbany with Calculated Blind River Flow**

#### L2.3.6.3 Engineering Calculations Results (Existing Conditions 1989-2004)

The engineering calculations provide reasonable approximations of hydrologic runoff into the study area and the hydraulic routing through the study area. Using reasonable hydrologic parameters, they help corroborate the HEC-HMS and HEC-RAS results in the absence of actual historical data, and can also be credibly used to extend the hydraulic analysis over the period from 1989-2004, and overcome the limitations of simulation periods that can be completed with HEC-RAS. Consideration of an extended period over multiple years is important, as analysis results demonstrate that boundary conditions are important in this system. The water level in the swamp is highly dependent on the water level in Lake Maurepas, which frequently back-flows into the swamp each year.

Two specific categories of results are provided from the engineering calculations to describe existing conditions within the swamp:

- Annual Average Water Depth; and
- Annual Average Dry-out Frequency.

The annual average water depths were calculated from the 16-year simulation of conditions based on rainfall conditions and Lake Maurepas water levels observed from 1989 to 2004. The engineering calculations produce a daily water level for each hydrologic unit, and the average water depth for each year of the period of record was calculated to determine the annual average. Similarly, results were calculated to characterize the frequency of dry-out conditions in the swamp. Dry-out conditions are defined as times when the water depth drops below 0.5 feet. This metric is indicative of conditions that provide the potential for seedling germination, which is a desirable element for ecosystem enhancement.

In addition, engineering calculations results were developed with projected increases in mean sea level that will result in increases to Lake Maurepas water levels. Similar results are calculated for the array of alternatives (see Section L2.10) for consistency with USACE policy for considering the effects of sea level rise in civil works programs. The sea level rise scenarios presented below correspond to projected sea level rise for 20-year, 30-year and 50-year increments that are utilized by the Wetland Value Assessment (WVA) to quantify project benefits. Average annual water depths are presented in **Table L2.3.6-2** and annual average dry-out frequency is presented in **Table L2.3.6-3**.

**Table L2.3.6-2 Average Annual Water Depths (Existing Conditions)**

| Sea Level Condition | Annual Average Water Depth (feet) by Sub-basin Number and Hydrologic Unit | | | | | | |
|---|---|---|---|---|---|---|---|
| | 100 | 200 | 210, 220 | 110 | 120, 160 | 300, 320, 330 | 140, 150 |
| | HU 1 | HU 2 | HU 3 | HU 4 | HU 5 | HU 6 | HU 7 |
| Existing | 1.91 | 1.86 | 1.85 | 2.09 | 1.34 | 1.53 | 1.61 |
| 20-Year | 1.93 | 1.90 | 1.87 | 2.11 | 1.37 | 1.57 | 1.65 |
| 30-Year | 1.97 | 1.97 | 1.90 | 2.14 | 1.42 | 1.61 | 1.71 |
| 50-Year | 2.19 | 2.24 | 2.10 | 2.33 | 1.64 | 1.83 | 2.02 |

**Table L2.3.6-3 Average Annual Dry-out Frequency (Existing Conditions)**

| Sea Level Condition | Annual Average Dry-out Frequency (%) by Sub-basin Number and Hydrologic Unit | | | | | | |
|---|---|---|---|---|---|---|---|
| | 100 | 200 | 210, 220 | 110 | 120, 160 | 300, 320, 330 | 140, 150 |
| | HU 1 | HU 2 | HU 3 | HU 4 | HU 5 | HU 6 | HU 7 |
| Existing | 1% | 1% | 2% | 1% | 3% | 2% | 0% |
| 20-Year | 1% | 1% | 2% | 1% | 3% | 2% | 0% |
| 30-Year | 1% | 1% | 2% | 1% | 3% | 2% | 0% |
| 50-Year | 1% | 1% | 1% | 1% | 3% | 2% | 0% |

Note: Dry-out conditions defined as water depth less than 0.5 feet

## L2.4 Swamp Hydroperiod Analysis and Alternatives Screening

The term hydroperiod is often used to describe the duration of time that a wetland is inundated by standing water. Applied in context to this project, the term hydroperiod is also used in reference to the cycle of wetting and drying of the swamp areas within the study area. This broader concept of hydroperiod as a cyclical and repeated occurrence is critical to defining conditions from a hydrologic perspective that will promote ecosystem restoration benefits. This section presents hydrologic and hydraulic analyses completed with specific focus on hydroperiod, including hydroperiod conditions that presently occur within the study area, desirable hydroperiod conditions, and potential modifications to the hydroperiod from the project.

### L2.4.1 Hydroperiod Characteristics

Since the construction of the Mississippi River flood control levees, Maurepas Swamp and Blind River have been virtually cut off from periodic overflows from the Mississippi River that brought freshwater, sediment, and nutrients to the swamp. With minimal soil building and moderately high subsidence rates, there has been a net lowering of ground surface elevation, so that now the swamps are persistently inundated. A limited ability to drain and persistent flooding characterize the existing hydrology in the swamp, which conflict with historic drying cycles in the swamp. Features within the study area such as drainage canals, roads, and other utilities disrupt natural flow and drainage patterns. Short circuiting of the natural drainage patterns has created ponding and stagnant waters in some areas. The contribution and circulation of nutrients and sediments is minimal and limited under existing conditions.

In contrast to existing conditions, historic hydroperiod characteristics prior to extensive human modification was dominated by overbank flow of the Mississippi River during spring floods and tidal inflow through Pass Manchac, into Lake Maurepas, and southwest to the study area. Overbank flows from the Mississippi river brought nutrients, sediment, and fresh water that promoted productivity and sustained the health of the swamp ecosystem. As floodwaters receded, surface flows traveled eastward as sheetflow into existing channels and subsequently Lake Maurepas.

### L2.4.2 Management Measures to Enhance Hydroperiod Characteristics

During the formulation of potential project alternatives, a number of management measures were conceptualized with the intent of promoting hydroperiod characteristics that would be beneficial to the existing ecosystem in the study area. Early in the planning process, it was recognized that from a hydrologic perspective, beneficial changes to the existing hydroperiod would encompass a balance of multiple elements:

- Increased delivery of freshwater that includes sediment and nutrients to promote productivity;
- Increased circulation of freshwater to increase dissolved oxygen levels;
- Reduced inundation depths within the swamp;
- Reduced durations of inundation within the swamp; and
- Attainment of extended dry periods to promote cypress and tupelo germination and sapling survival.

The project formulation process continued with the identification of potential management measures intended to achieve the desirable hydroperiod characteristics. The following specific management measures were identified and evaluated with respect to their influence on the hydroperiod:

- Water management enhancements in the swamp and redirection of local hydrology. The features of this measure are designed to manage the water that enters the system as rainfall or as drainage from outside of the study area to improve the distribution of water and hydroperiod across the swamp. This will increase the length of dry periods in the swamp and reduce the areal distribution and timing of standing/stagnant water to the extent possible, in order to increase productivity and assimilation while promoting cypress/tupelo germination and sapling growth.

- Gaps in Existing Embankments. There are more than 40 miles of existing embankments (levees and spoil banks throughout the swamp that would be potentially gapped (cut) at regular intervals (e.g., every 500 to 1,000 ft) to allow a more distributed flow pattern in the swamp and better drainage from the swamp.
- New or Improved Culverts at Highway 61. This management measure could also include new and/or improved culverts-bridges under Highway 61 to improve flow and reconnect the hydrology of the Swamp across man-made features.
- Control Structures in Existing Drainage Canals. This management measure consists of gates constructed at strategic locations in the existing drainage canals for lateral distribution of local rainfall-runoff into the swamp (and not bypass the swamp). The distribution weirs would be variable and could be raised during dry, normal, and/or small storm conditions (as determined in the operations plan) to facilitate the dual distribution-drainage system by using the existing drainage and pipeline channels to distribute water, sediments, and nutrients.
- Romeville Freshwater Diversion. This management measure consists of a diversion structure at the Mississippi River and transfer canal from the Mississippi River to the swamp. From a hydrologic and hydraulic perspective, the key aspects of this management measure are that diversion flows enter the St. James Parish Canal at the location of the existing Romeville Canal. The magnitude of the diversion flow and the potential influence area are characteristics analyzed with hydrologic and hydraulic analysis.
- North Freshwater Diversion. This management measure is similar to the Romeville division above, and consists of a diversion structure at the Mississippi River and transfer canal from the Mississippi River to the swamp. From a hydrologic and hydraulic perspective, this management measure provides an opportunity to influence different areas that are likely to be influenced by the Romeville diversion. The magnitude of the diversion flow and the potential influence area are characteristics analyzed with hydrologic and hydraulic analysis.

### L2.4.3 Water Management Enhancements

**Figure L2.4.3-1** illustrates the potential impacts of improved connectivity between swamp elements associated with gaps in the existing berms. Using swamp area 100 as an example, the figure shows that without cuts or gaps in the existing berms, water levels tend to remain high; generally at or near the berm crest elevation, even when Lake Maurepas is drawn down. This condition suggests that dryout of the swamp in many areas (necessary for seed germination and sapling survival) will be very difficult to achieve, as the internal berms will tend to retain water flowing into the swamp.

The figure also shows that when the berms are gapped to improve connectivity and flow potential throughout the swamp, the water level within the swamp has much more flexibility to follow the lake levels both up and down. This connectivity will allow water to pass out of the swamp more easily during periods when the lake is low, thereby allowing the necessary dry-out conditions for tree regrowth. However, the connectivity can also cause more backwater from the lake to enter the swamp when there is not enough inflowing freshwater from natural runoff. This can be visualized in the difference in the right-hand graph between the blue line for engineering calculations (which is due to fresh water runoff only, and no backflow) and the water level in Lake Maurepas. For these reasons, it was deemed necessary to include gaps in the internal berms as an integral element of every alternative under evaluation.

It was also determined that gaps on their own, without the benefit of diverted water from outside the study area, would be insufficient as a stand-alone alternative, since just as they would facilitate beneficial drainage when Lake Maurepas is low, so too would they allow backflow into the swamp when Lake Maurepas is high. Backflow from Lake Maurepas will potentially increase the movement of water in the swamp, but will provide a source of sediment and nutrients to the project area. More importantly, there is a potential that backflow from Lake Maurepas can at times introduce water with increased salinity levels, which would adversely impact the project area. Reducing the potential for backflow is a compelling reason for including a diversion flow to "push against" the backflow from the lake, and the berm gaps are needed to promote drainage of the swamp when conditions allow it.

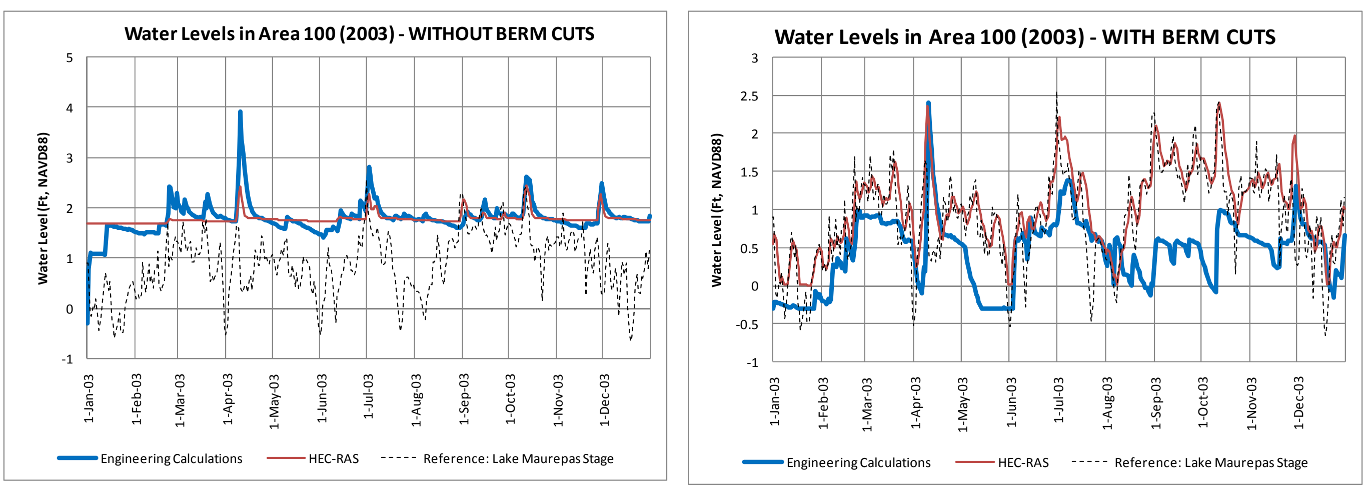



**Figure L2.4.3-1 Representative Hydroperiod Modifications from Berm Gaps**

### L2.4.4 Screening Analysis for Diversion Location and Capacity

A screening analysis was conducted using both HEC-RAS and engineering calculations to evaluate a range of potential freshwater diversion scenarios. A number of variables related to the magnitude, location, and frequency of freshwater diversion flow were initially identified. Fundamentally, the appropriate magnitude of a freshwater diversion to the project area is a balance between maximizing the delivery of sediment, nutrients, and water without exacerbating inundation and stagnation that typify existing conditions. Additional considerations are constraints on the ability to discharge from the project area as a result of Lake Maurepas water levels and the desire to reduce backflow from Lake Maurepas to the project area.

Initial analysis of potential diversion magnitude was completed using HEC-RAS. The existing conditions HEC-RAS model was modified to include a constant diversion inflow that enters the St. James Parish Canal from the Romeville Canal. Simulations completed with various diversion flow rates and resulting flows and water surface elevations in other drainage canals, the swamp, and the Blind River were reviewed. Of particular note was the relationship observed between diversion flow rate and response in the Blind River near the downstream boundary of the project area at I-10. As indicated on **Figure L2.4.4-1**, a diversion flow rate of 500 cfs appears to produce minimal change in the Blind River flow conditions downstream, while a flow rate of 4,000 cfs appears to prevent most occurrences of reverse flow in the Blind River.

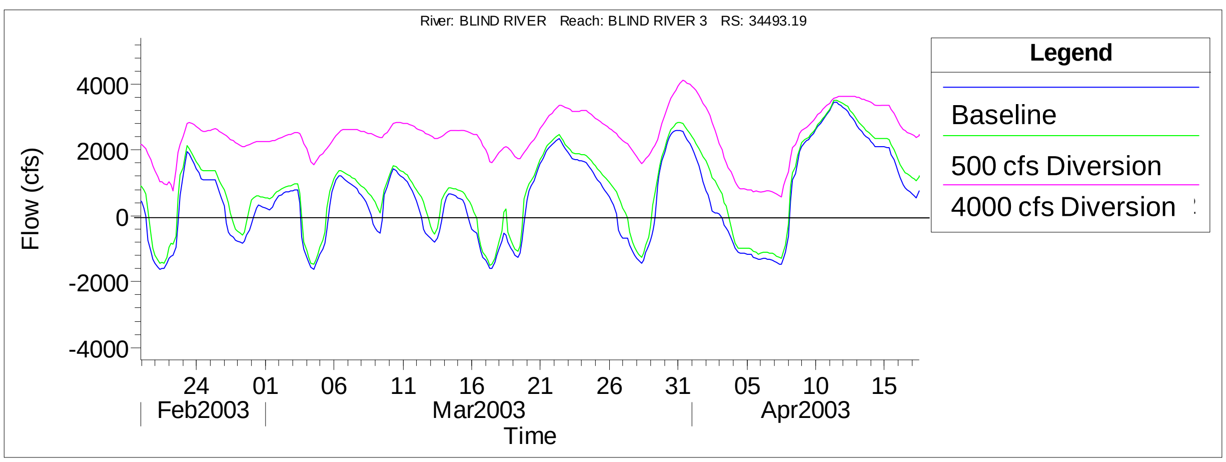



**Figure L2.4.4-1 HEC-RAS Simulations with Multiple Diversion Flow Rates**

The engineering calculations were then applied over the daily period of record for 1989-2004 in order to screen the broad range of potential diversion capacities, and to test the alternative effects of different diversion locations.

Three diversion locations were screened:

- Romeville;
- South Bridge (originally referred to as “North Bridge”); and

- Combination of Romeville and South Bridge.

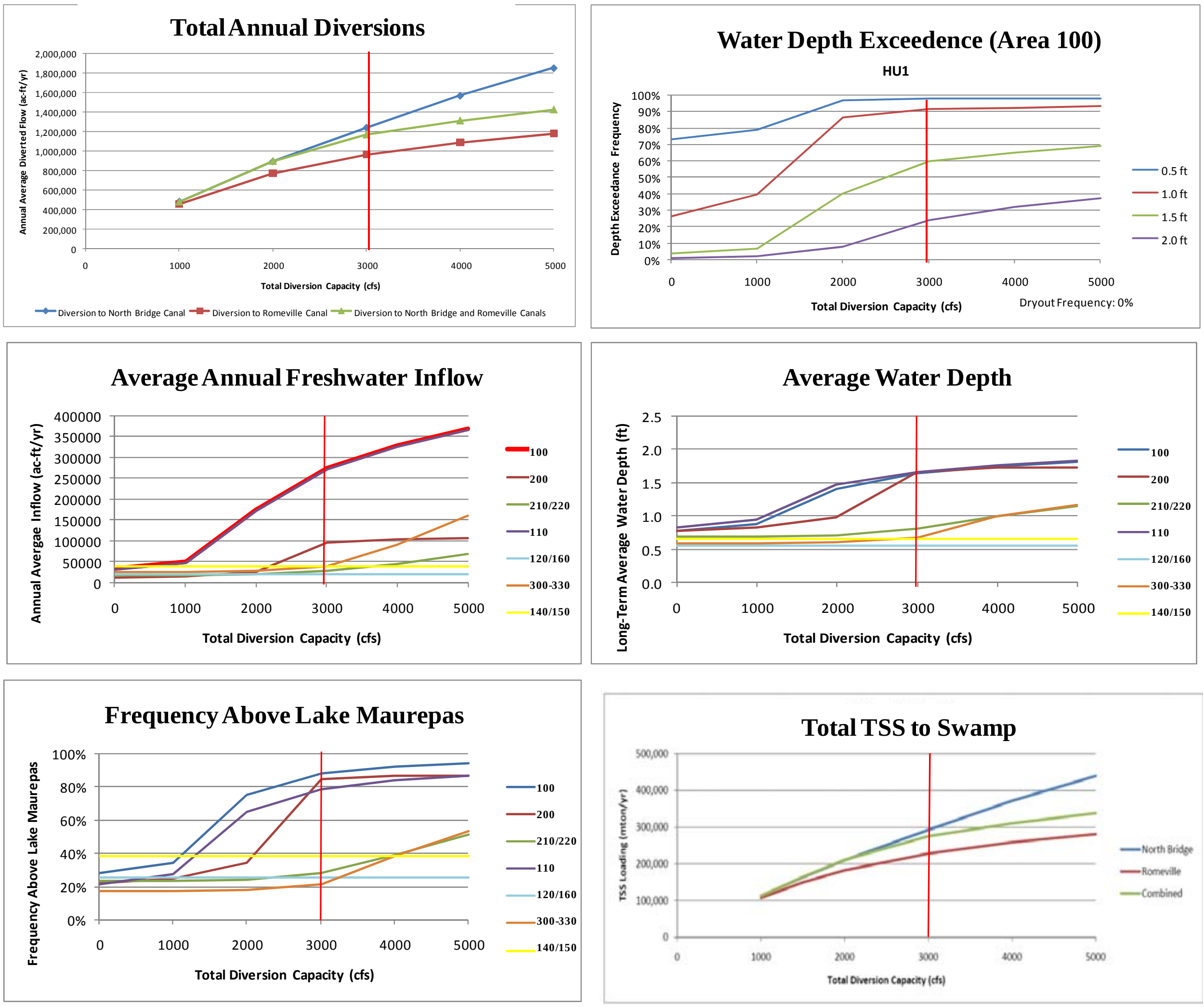
Diversion capacity was then incrementally increased in successive analyses, applying the logic outlined in Section L2.2. Water was diverted only when the average water level in the swamp was below the Lake level. Diversions were discontinued when the average water level in the swamp exceeded the Lake level, or when the lake dropped below 0.5 feet NAVD (to accommodate potential dry-out conditions).

In addition to tracking the total volume of diverted water, five other hydrologic metrics were tracked over the 16-year analysis period for comparative purposes:

- Average annual freshwater inflow (includes runoff and diversions);
- Frequency at which the swamp water level exceeds Lake Maurepas water level (to help prevent backflow);
- Frequency at or above certain water depths in the swamp;
- Long-term average depth of water in the swam; and
- Annual average Total Suspended Solids (TSS) into the swamp (using data from the USGS NWISWeb database, Station 07374000: Mississippi River at Baton Rouge).

**Figure L2.4.4-2** illustrates the results of the sensitivity analysis, as the diversion capacity was increased from 1,000 cfs to 5,000 cfs. The graphs illustrate two important findings. First, no substantial change in the response of the system to the introduction of diversions occurs until a capacity of at least 1,000 cfs is provided. At this "point of departure," many of the hydrologic metrics outlined above begin to respond dramatically to increased diversion capacity. Second, once diversion capacity exceeds 2,000 – 3,000 cfs, the hydrologic metrics are generally much less sensitive to increased diversion capacity. That is, above 2,000 – 3,000 cfs, there would be diminishing returns on further increases in capacity with respect to hydrologic sensitivity. This is due in part to the fact that additional capacity may not always be needed to help keep the swamp above the lake elevation. These findings were confirmed (and refined) with the HEC-RAS hydraulic model, which suggested that a minimum capacity of 1,500 cfs would be required to substantively reduce backflow potential, and that 3,000 – 4,000 cfs would be required to practically guard against it completely (based on 2003 conditions).

For these reasons, a minimum diversion capacity of 1,500 cfs was established for the alternatives, and a maximum capacity of 3,000 cfs was established.




*Results represent the introduction of water at both Romeville and South Bridge locations simultaneously, with the capacity divided equally between the two. Trends are similar with respect to individual locations for water introduction to the swamp.*

**Figure L2.4.4-2 Sensitivity of Hydrologic Metrics to Diversion Capacity**

The results of the sensitivity analysis were confirmed for each of the alternative locations, and the same trends were observed. That is, whether the diversion site was Romeville, South Bridge, or a division of the total capacity between the two, the range of sensitivity in the hydrologic parameters was very similar. Additionally, all three alternatives for the diversion location yielded substantive hydrologic effects. **For these reasons, none of the three alternatives for diversion location/division were screened out.**

Therefore, the following six alternatives in **Table L2.4.4-1** were identified as a result of this screening process, each of which included as a key element the gaps in berms discussed above to improve connectivity of flow paths throughout the study area:

**Table L2.4.4-1 Preliminary Alternatives Identified Through Screening**

| Alternative Number | Diversion at Romeville (cfs) | South Bridge Diversion (cfs) | In-Swamp Management Measures |
|---|---|---|---|
| 1 | 1,500 | - | Yes |
| 2 | 3,000 | - | Yes |
| 3 | - | 1,500 | Yes |
| 4 | - | 3,000 | Yes |
| 5 | 750 | 750 | Yes |
| 6 | 1,500 | 1,500 | Yes |

Note: In-swamp management measures include berm gaps, control structures and new culverts under US 61.

Further analysis of the potential diversion structure discussed below, indicated that water levels in the Mississippi River could diminish the diversion capacity during certain months (generally August – November) to varying degrees. This, in turn, would effectively reduce the total average capacity of each alternative. For the 1,500 cfs alternatives, this was a concern, since hydraulic modeling and the engineering equations suggested that it was at or near the lower end of prospective capacities capable of providing substantive hydrologic effects. Reduction in the 3,000 cfs capacity was less of a concern, since many of the sensitivity curves actually began to exhibit diminishing hydrologic effects at capacity levels below 3,000 cfs. **For this reason, the alternatives at 1,500 cfs capacity were removed from further consideration.**

The three remaining alternatives were analyzed in detail. Alternative 4 was subdivided into 2 alternatives:

- Alternative 4A: 3,000 cfs diversion at South Bridge, delivered entirely to the South Bridge Canal (passing through swamp areas 100, 200, 210, and 220).
- Alternative 4B: 3,000 cfs diversion at South Bridge: 1,500 cfs delivered to the South Bridge Canal, 1,500 cfs delivered to the St. James Parish Canal for introduction into the swamp in similar fashion to the Romeville Diversion.

Therefore, the final array of alternatives for detailed analysis of costs, environmental impacts, hydrology and hydraulics, and ecosystem restoration potential is identified in **Table L2.4.4-2**.

**Table L2.4.4-2 Final Array of Alternatives**

| Alternative Number | Diversion at Romeville (cfs) | South Bridge Diversion (cfs) | In-Swamp Management Measures |
|---|---|---|---|
| 2 | 3,000 | - | Yes |
| 4 | - | 3,000 | Yes |
| 4B | - | 3,000* | Yes |
| 6 | 1,500 | 1,500 | Yes |

*Note: Flow split for Alternative 4B accomplished with a single north diversion transfer canal and control structures to split the flow between the swamp transmission canal and the St. James Parish Canal.

### L2.4.5 Consideration of Sea Level Rise

The effects of potential sea level rise (discussed elsewhere in this report) are included in the discussion of detailed results of the four alternatives in the final array (Section L.2.10). Ultimately, the final four alternatives are not distinguished by differences in capacity; they all are characterized by diversions up to 3,000 cfs. As such, the impact of sea level rise will not further distinguish the alternatives based on the information available at this time. Detailed consideration of impacts on the Tentatively Selected Plan (TSP) will continue as the TSP advances through the design process, and as further information becomes available on the topography of the swamp and the potential ways that future accretion rates may partially offset sea level rise impacts.

### L2.4.6 Effects of Mississippi River Water Level on Diversion Capacity

Boundary conditions are important factors in the dynamics of the Blind River / Maurepas Swamp system. Just as Lake Maurepas water levels can govern water levels in the swamp, and backflow into the swamp, so too can water levels in the Mississippi River affect the ability of an engineered system to move fresh water through the system.

**Figure L2.4.6-1** illustrates the relativity of the two boundary conditions, upstream and downstream. While there is almost always positive driving head from the Mississippi River to Lake Maurepas, there are times when the differential is marginal, and also times when the head in the Mississippi River is low enough that full capacity of the diversion structures cannot be achieved. **Figure L2.4.6-2** shows the flow rating curves for conceptually designed diversion culverts at the Romeville diversion location. The design capacity can be achieved when the stage in the Mississippi River is at or above 10 feet NAVD. However, the capacity is diminished when the stage drops below 10 feet.

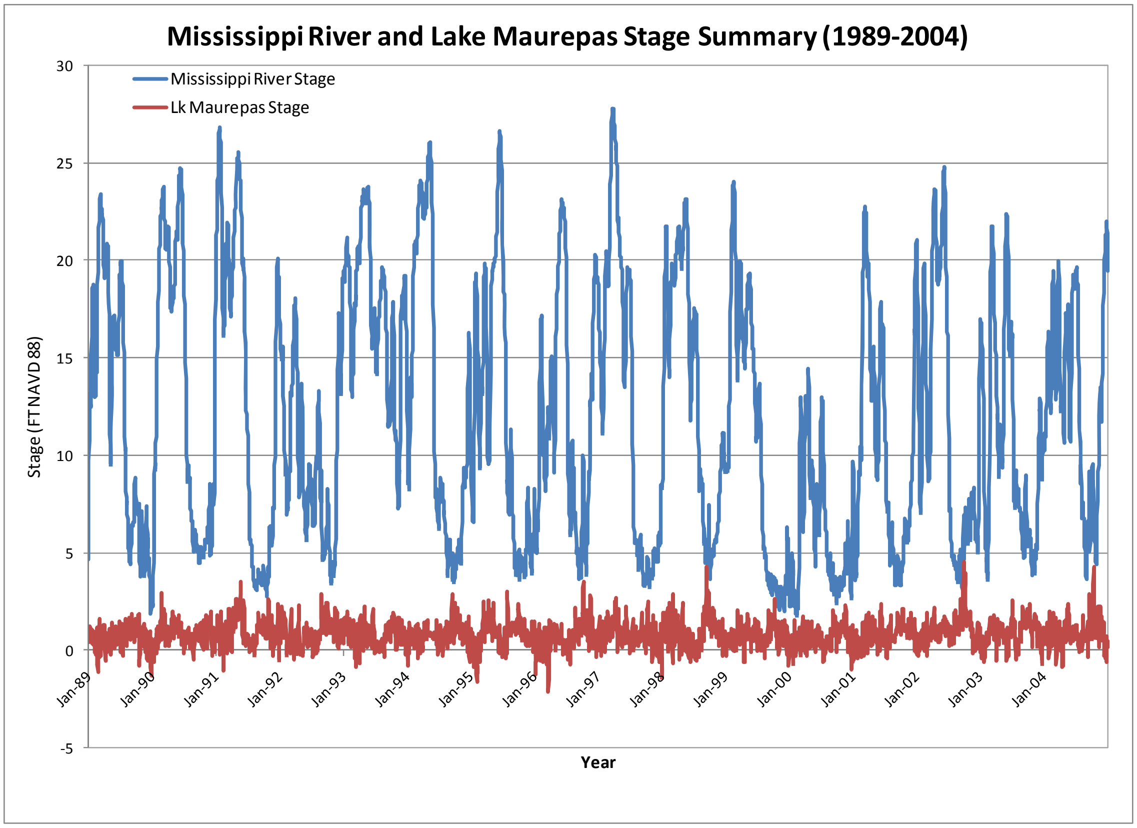



**Figure L2.4.6-1 Upstream and Downstream Boundary Condition Water Levels**

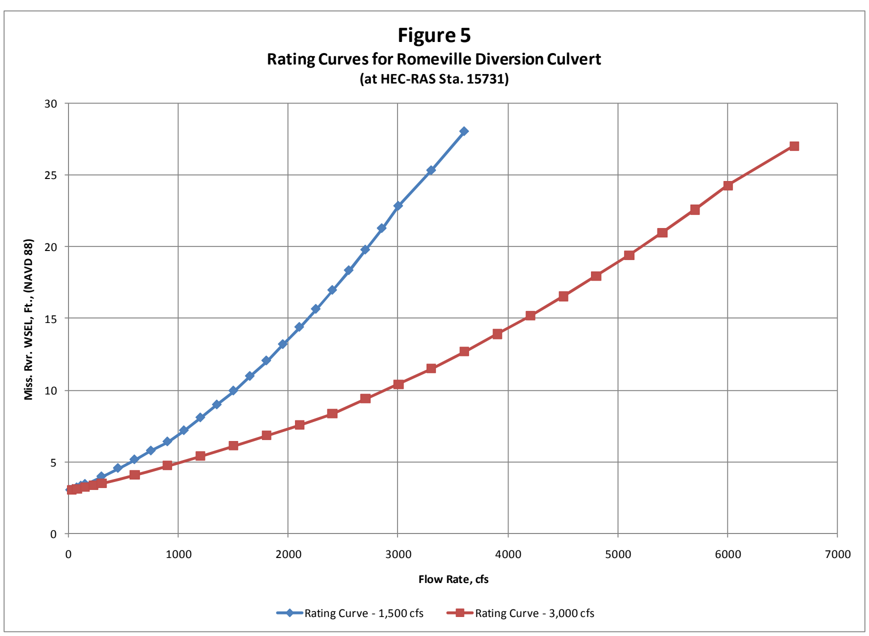
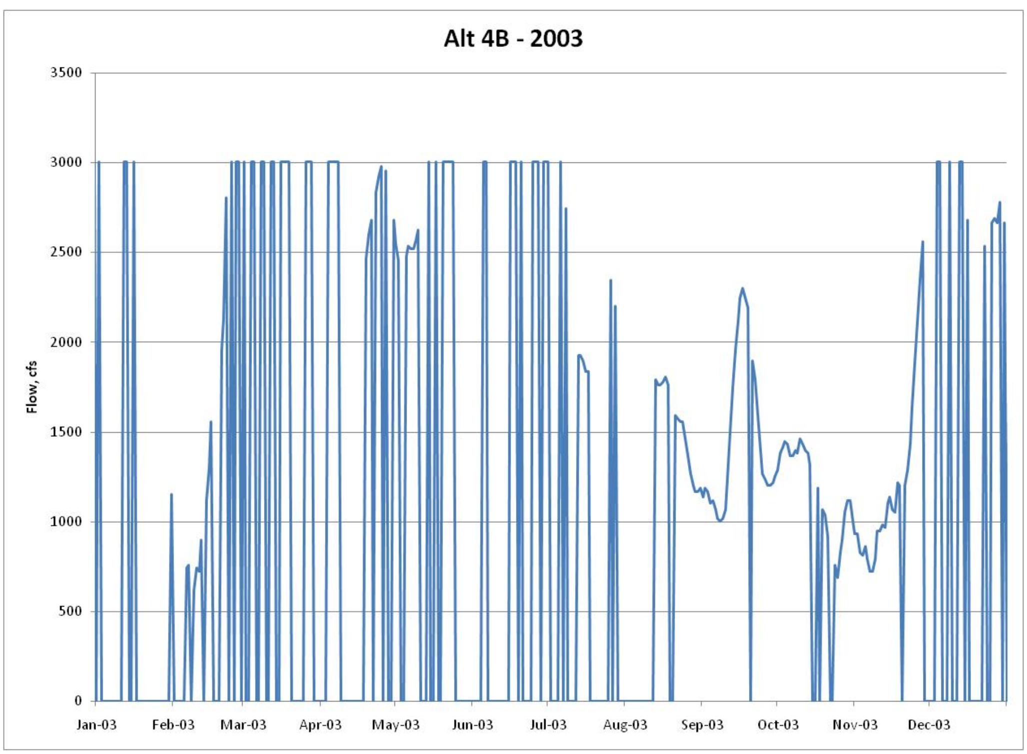



**Figure L2.4.6-2 Rating Curve for Romeville Diversion Culvert**

The engineering calculations were used to evaluate the potential effects of periodic reductions in diversion capacity. **Figure L2.4.6-3** illustrates the diversion time series for 2003 (the representative year of average hydrologic conditions). In most of the observed years, the reductions in capacity were observed less frequently – 2003 seems to have been a particularly low-flow year for the Mississippi River, even though relatively average for the Blind River system.

Clearly, there is an extended period of time toward the end of the year (generally August through November in most years) in which capacity is diminished. This is partially offset by more frequent and extended diversions. In general, the incorporation of the rating curves and their effects on diversion capacity reduced the overall throughput of freshwater for the alternatives from 15% - 25% when no adjustments to operating logic are applied. However, on average, approximately 5% to 8% of the total throughput can be recovered through alternative operations, such as continuous diversions in July and August, or even continuous diversions between July and November (as long as Lake Maurepas is above 0.5 feet).

With this refinement in the analysis, the basic trends in hydrologic effects did not change, even though the magnitude of effects was diminished somewhat by the upstream boundary conditions. The operational flexibility of the system, and its performance, will be further examined as the TSP is advanced to further planning and design.




*Alternative 4B is shown – others follow very similar trends.*

**Figure L2.4.6-3 Diversion Time Series for 2003 with Capacity Constraints**

### L2.4.7 Conclusions

This section presented the process by which a very broad spectrum of alternatives was screened down to the most practical and promising four alternatives using simple techniques and an understanding of the dynamics of the Blind River / Maurepas Swamp system. More detailed analysis was conducted on the final array of alternatives, and results are presented in Section L.2.10. The following conclusions can be drawn from the screening analysis discussed above:

- The engineering calculations are reasonable approximations of hydrologic runoff into the study area and the hydraulic routing through the study area. Using reasonable hydrologic parameters, they help corroborate the HEC-HMS and HEC-RAS results in the absence of actual historical data, and can also be credibly used to extend the hydraulic analysis over the period from 1989-2004 (whereas HEC-RAS is practically limited to one-year simulations due to the hydraulic complexity of the system).
- There are three principal areas of uncertainty that must be considered when evaluating these results:
- The topographic information used to define the bathymetry of swamp area is coarse, and should be refined as the TSP is advanced into more detailed planning and design phases.
- The analysis is based on historic climate and hydrologic conditions – future conditions cannot be forecast, although the four alternatives in the final array are evaluated under the influence of projected sea level rise in Section L.2.10.
- The swamp areas in this analysis were represented as storage areas with uniform access to available water. In reality, they will be characterized by overland flow. Three-dimensional hydrodynamic modeling is currently underway to better define flow pathways and the potential for equitable distribution of water throughout the targeted swamp areas.
- Boundary conditions are very important in this system. The water level in the swamp is highly dependent on the water level in Lake Maurepas, which has historically back-flowed into the swamp regularly (the diversion will be aimed at limiting this to the extent practical). Likewise, at the upstream end of the system, the Mississippi River stage can significantly reduce the diversion capacity during certain months, which can, in turn, permit unwanted backflow from Lake Maurepas more frequently.
- Gaps in the existing berms are an essential element in each of the alternatives. They will provide flow pathways to allow drainage when Lake Maurepas is at low levels, which in turn can promote periodic dry conditions in the swamp that are needed for seed germination and sapling survival. However, the gaps on their own without the benefit of diverted water from outside the study area would be insufficient as a stand-alone alternative. Just as they would facilitate beneficial drainage when Lake Maurepas is low, so too would they allow detrimental backflow into the swamp when Lake Maurepas is high. Therefore,

the gaps are included as an element to each alternative to promote drainage, but are not considered as an alternative on their own.

- A minimum of 1,500 cfs diversion capacity is required to provide substantive hydrologic effects in the swamp, including the prevention of backflow from Lake Maurepas. However, this lower limit is fairly marginal in its potential hydrologic effectiveness.
- Diversion capacities above 3,000 cfs begin to exhibit diminished hydrologic effects (sometimes even lower than 3,000 cfs), as measured by five hydrologic metrics: freshwater inflow, frequency of backflow prevention, frequency of water depths, long-term average water depth, and introduction of Total Suspended Solids. Therefore, 3,000 cfs was identified as the upper limit of diversion capacity for the alternatives analysis.
- Both diversion locations offer the potential for hydrologic effects, though the distribution of these effects is different. Therefore, the hydrologic screening analysis did not rule out either diversion location alternative (Romeville or South Bridge, or both). Hence, the preliminary array of screened alternatives included six (in addition to the No-Action alternative): 1,500 and 3,000 cfs diversions at Romeville, South Bridge, and divided equally between both.
- The reduction in driving head when the Mississippi River is at low stages (below 10 feet NAVD) can reduce the capacity of the diversion system. This, in turn, can reduce the average total freshwater throughput in the system, though the reduction can be partially offset by preemptive diversions in certain months (this will be further developed as the TSP advances through further planning and design, and as hydrodynamic modeling is advanced to provide better understanding of the effectiveness of water distribution throughout the study area). However, since the alternatives with 1,500 cfs capacity were deemed marginal even without this periodic reduction in capacity, the potential for reduced flow effectively ruled out the 1,500 cfs alternatives. The final array is comprised of four alternatives, all of which include 3,000 cfs diversions.
- The final array of alternatives includes 4 configurations, all of which include berm gaps to improve internal drainage:
- Alternative 2: Romeville at 3,000 cfs;
- Alternative 4A: South Bridge at 3,000 cfs, routed primarily to the North of the Blind River;
- Alternative 4B: South Bridge at 3,000 cfs, divided between North and South of the Blind River by diverting 50% of the flow southward through the St. James Parish Canal; and
- Alternative 6: 1,500 cfs diversions at both Romeville and South Bridge.

## L2.5 Hydrodynamic Analysis

Appendix Section L2.2 presented an overview of multiple analysis methods used within the study area and Appendix Section L2.3 discussed detailed watershed hydrology and hydraulics analysis with HEC-HMS and HEC-RAS. As discussed in Section L2.2.3, the EFDC model provides a more detailed spatial and temporal representation of hydrodynamic variation and characteristics within the project area.

This section discusses each component of the hydrodynamic analysis completed to evaluate existing conditions, including supporting data, model testing, and specific simulations. This section also presents supporting information relevant to understanding hydrodynamic constraints and opportunities, which were considered during evaluation of project alternatives, as presented in Section L2.10.

### L2.5.1 Flow and Stage Boundary Conditions

A local watershed model, HEC-HMS, and flow routing and water surface calculation model, HEC-RAS, were developed to provide channel/canal flows and stages as the boundary conditions for the EFDC model. Detailed information regarding these two models can be found in Sections L2.2 through L2.4. The locations of boundary conditions provided by 18 HEC-HMS simulated channel/canal flows and 2 HEC-RAS simulated canal/river stages (**Figure L2.5.1-1**) were incorporated in the EFDC model to define flow and stage boundaries from upstream and downstream locations that influence the project area. For this feasibility study, the year 2003 was selected as a basis for representing average hydrologic conditions during model setup, testing, and alternative analysis. HEC-HMS model simulated 2003 local watershed inflows to the project area are shown on **Figure L2.5.1-2a** and **Figure L2.5.1-2b**.

HEC-RAS simulated flows for the Conway Canal and Blind River stages at I-10 (Figure L2.5-1) were used as stage boundaries in the EFDC model. In general, the Blind River stages at I-10 significantly influence water elevation in the swamp although the water elevations in the swamp do not instantaneously respond to the river stages due to high vegetation resistance in the wetland. **Figure L2.5.1-3** shows the 2003 stage data at these two locations simulated by the HEC-RAS model for existing conditions. It is noted that the range and pattern of the simulated stages at the two locations are very similar.

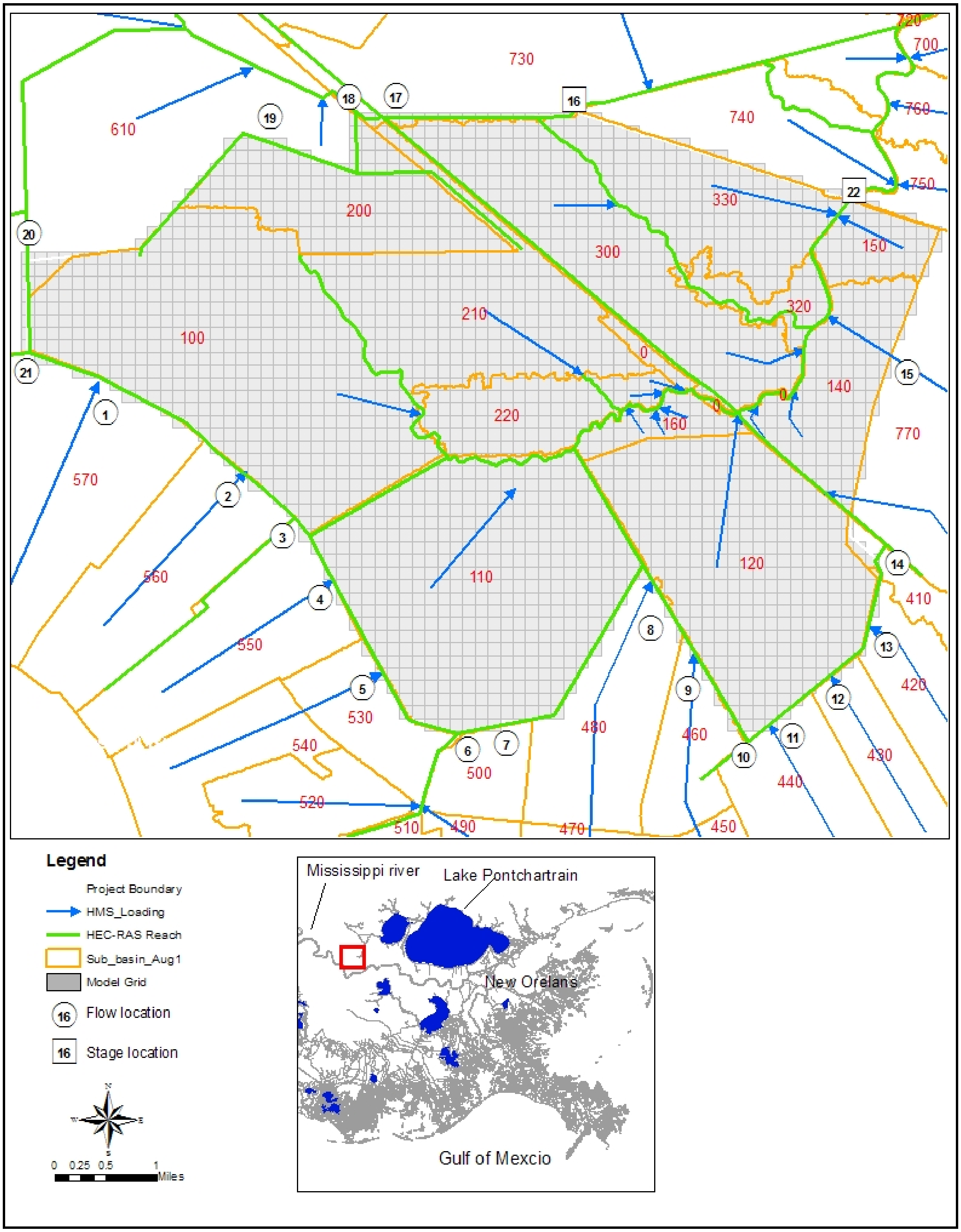



**Figure L2.5.1-1 Locations of EFDC Flow and Stage Boundaries**

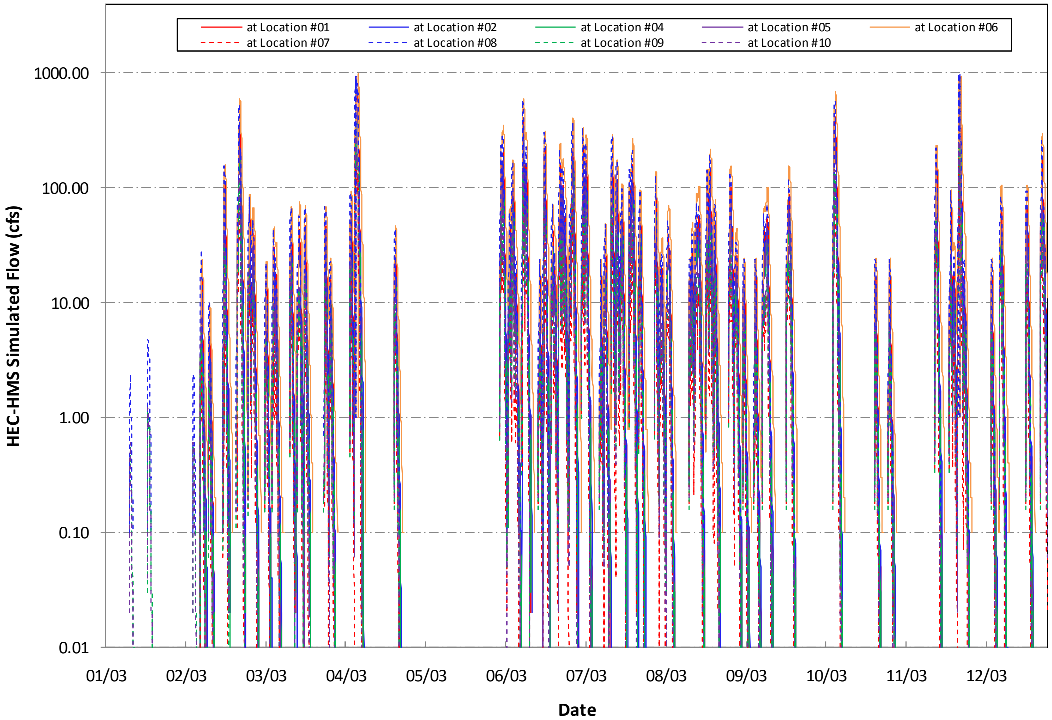



**Figure L2.5.1-2a Boundary Inflows (Locations 1 to 10)**

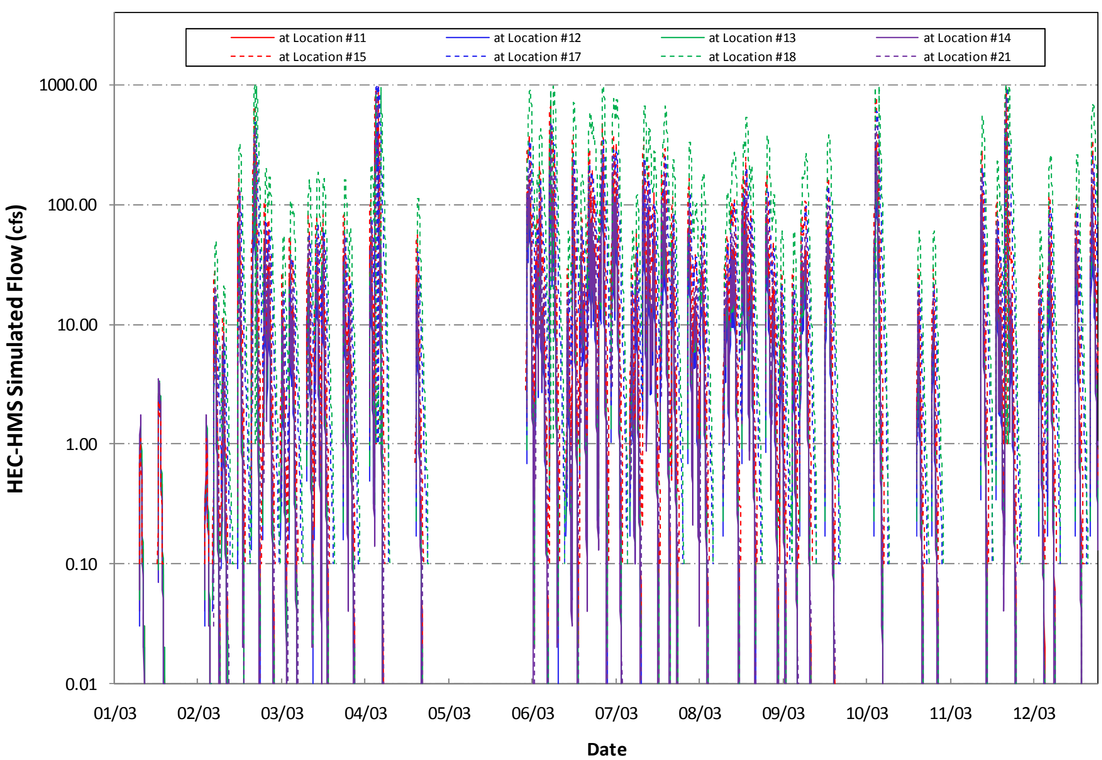



**Figure L2.5.1-2b Boundary Inflows (Locations 11 to 21)**

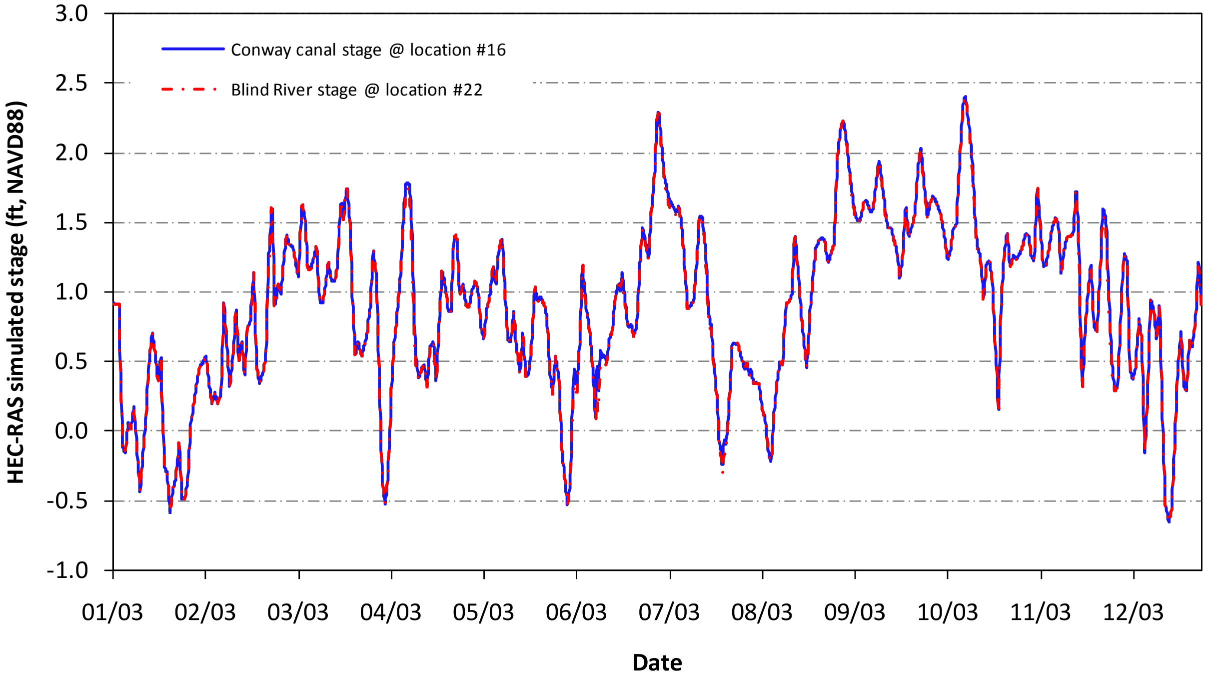



**Figure L2.5.1-3 Downstream Boundary Stage**

### L2.5.2 Temperature Boundaries

To simulate water temperature in the wetland, water temperature data along with the flows and stages at the boundaries are needed. However, no measured watershed runoff temperature data in the area were available and very limited water temperature data at the two most nearest gauges - Baton Rouge gauge (Gauge ID 7374000) and Belle Chasse gauge (Gauge ID 7374525) on the Mississippi River were measured during 2003. For modeling purposes, continuously monitored daily mean temperature data from the Baton Rouge gauge in 2006 was selected because it provided the most complete period of record.

A plot of the temperature data is shown on **Figure L2.5.2-1**. The plot demonstrates a seasonal pattern, where warmest water temperatures occur late June through August, while cooler temperatures occur in late December through early March. In general, although water temperature associated with the watershed runoff will be slightly different from that of a large river, such as the Mississippi River, the river water temperature shown on Figure L2.5.2-1 was used for all watershed inflow boundaries and stage boundaries.

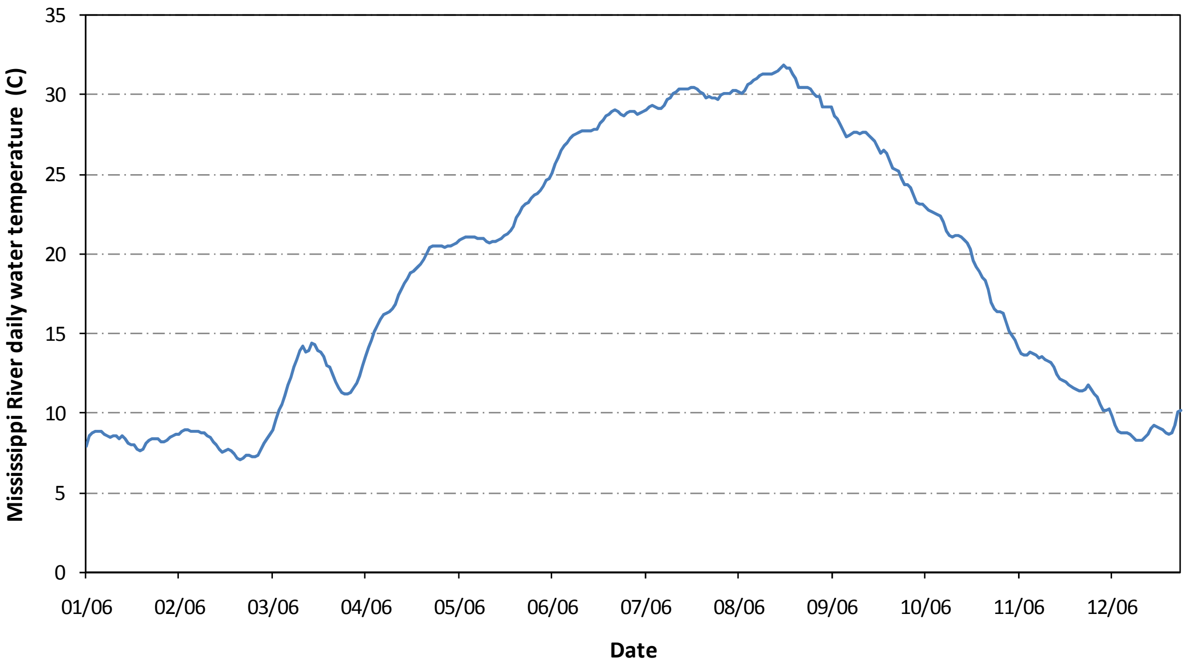



**Figure L2.5.2-1 Mississippi River Daily Water Temperature (2003)**

### L2.5.3 Sediment Boundaries

Sediment is an essential component for evaluating hydrologic benefits and impacts from the project. As discussed in Section L2.4, historical supply of sediment was interrupted when the Mississippi River levees were constructed. For existing conditions, local watershed runoff will contribute minimal amounts of sediment. For the purpose of representing sediment contributions from forests/wetlands, a mean sediment concentration of 8.5 mg/L was used based on Harper and Baker (2007). Sediment associated with the watershed runoff will be delivered into the project area at various locations as shown on Figure L2.5.1-1. Due to lack of sediment concentration data, the sediment concentrations associated with the Blind River stage boundary were considered to be a constant value of 10 mg/L.

The introduction of sediment from a constructed freshwater diversion has the potential to reduce the subsidence of the project area and bring more nutrients to revitalize the ecosystem of the swamp. The main source of sediment from potential freshwater diversion flows will be the Mississippi River. Limited sediment concentration data collected at the Baton Rouge gauge on the Mississippi River were used to developed monthly average sediment concentration for year 2003 as shown on Figure L2.5.3-1. Field data in other studies (Snedden et al., 2006) showed that in a normal river flow condition, about 99% of the suspended sediment in the river water column is fine cohesive sediment with particle size varying from ~63 um (coarse silt) to ~1 um (very fine clay). For purpose of sediment transport deposition and erosion model simulation, the median sediment particle size was considered to be 2.5 um (USGS, 1988). Based on Stokes Law, for a particle size of 2.5 um, the settling velocity is 8.2 ft/day. This settling velocity was used in the model although the actual settling velocity can be higher due to sediment flocculation.

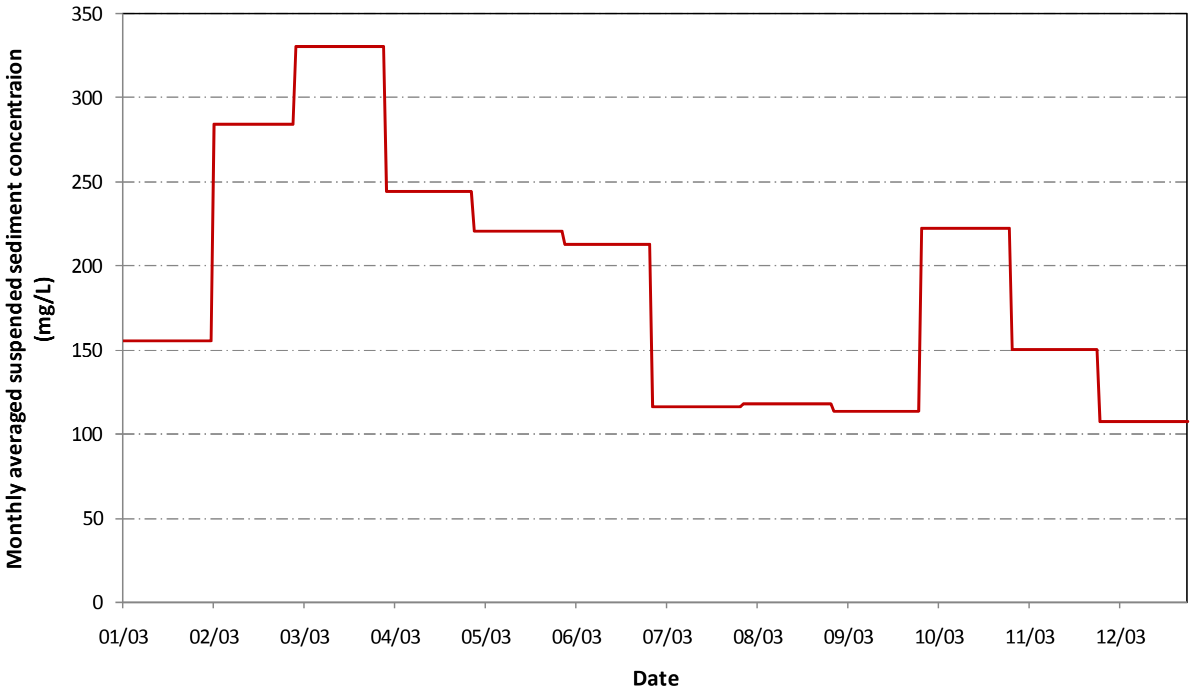



**Figure L2.5.3-1 Monthly Average Mississippi River Suspended Sediment Concentration (2003)**

### L2.5.4 Water Quality Boundaries

For water quality associated with stormwater runoff, hourly loading was calculated using flows from the HEC-HMS model at each flow location shown on Figure L2.5.1-1 multiplied by the typical concentrations reported in the literature and a conversion factor. **Table L2.5.4-1** presents a summary of the parameters that were included in the water quality for runoff. As shown in the table, the majority of the runoff values were obtained from Harper and Baker (2007). The data from Harper and Baker (2007) only provided total concentrations for nitrogen and phosphorus. Therefore, the runoff concentrations of various nitrogen and phosphorus species were estimated using averages of observed data from the Mississippi River at both Baton Rouge and Belle Chasse stations. The refractory and labile components were also calculated using the same assumptions presented in the diversion discussion above. A summary of model input loads from runoff at each flow location shown on Figure L2.5.1-1 are presented in **Table L2.5.4-2**.

In the event of a freshwater diversion from the Mississippi River, additional nutrients will be delivered to the project area. The Baton Rouge (Gauge ID 7374000) and Belle Chasse (Gauge ID 7374525) monitoring stations were used to supply water quality input information for the diversion flow. For both gauges, average concentrations for the period of record at each gauge were calculated and are shown in **Table L2.5.4-3**. As can be seen from the table, average concentrations of most parameters are quite similar between the two stations (less than 20%) with the only exceptions being BOD5, dissolved organic nitrogen, and dissolved organic carbon. Average BOD5 concentrations were found to be higher at Baton Rouge, while dissolved organic nitrogen and dissolved organic carbon were found to be higher at Belle Chasse.

The saturated dissolved oxygen concentration was first calculated using a standard empirical equation as follows (Weiss, 1970):

$$\ln(DO) = A_1+A_2*100/T+A_3*\ln(T/100)+A_4*T/100+S*[B_1+B_2*T/100+B_3*(T/100)^2] \quad (1)$$

where $A_1$ = -173.4292, $A_2$ = 249.6339, $A_3$ = 143.3483, $A_4$ = -21.8492,

$B_1$= -0.033096, $B_2$ = 0.014259, $B_3$ = -0.001700,

T = temperature in Kelvin, and S = salinity (g/kg).

The daily mean temperature data from the Baton Rouge gauge in 2006 as shown on Figure L2.5.2-1 were used in the DO calculation with the above Eqn. (1). Then, the estimated diversion flow DO was obtained by multiplying the calculated saturated DO with the average DO percent saturation (89.6%), which was calculated from limited DO percent saturation data collected at the Baton Rouge and Belle Chasse gauges.

**Table L2.5.4-1 Runoff Water Quality Concentrations**

| Constituent | Runoff value | Source |
|---|---|---|
| BOD5 | 1.40 | Harper & Baker, 2007 |
| Organic Carbon - Dissolved | 9.07 | Suarez et al., 2006[1] |
| Organic Carbon - Particulate | 7.83 | Suarez et al., 2006[1] |
| Dissolved Ammonia | 0.02 | Harper & Baker, 2007[2] |
| Organic Nitrogen - Dissolved | 0.17 | Harper & Baker, 2007[2] |
| Organic Nitrogen - Particulate | 0.13 | Harper & Baker, 2007[2] |
| Nitrate + Nitrate - Dissolved | 0.78 | Harper & Baker, 2007[2] |
| Total Phosphorus | 0.06 | Harper & Baker, 2007[2] |
| Organic Phosphorus - Dissolved | 0.00 | Harper & Baker, 2007[2] |
| Organic Phosphorus - Particulate | 0.03 | Harper & Baker, 2007[2] |
| Total Nitrogen | 1.15 | Harper & Baker, 2007 |

Notes: [1] Site SS-9 used to provide total organic carbon value; distributed based on average observed concentrations at Baton Rouge and Belle Chasse USGS stations; and [2] Total provided, distributed over

**Table L2.5.4-2 Runoff Load Summary**

| Location | Total Phosphorus (lb/year) | Total Nitrogen (lb/year) | Nitrate + Nitrite (lb/year) |
|---|---|---|---|
| #14 | 41,895 | 875,978 | 591,115 |
| #13 | 29,051 | 607,434 | 409,900 |
| #10 | 27,176 | 568,226 | 383,442 |
| #5 | 31,617 | 661,085 | 446,104 |
| #1 | 79,741 | 1,667,305 | 1,125,108 |
| #3 | 8,011 | 167,505 | 113,033 |
| #9 | 48,796 | 1,020,285 | 688,494 |
| #7 | 11,900 | 248,810 | 167,898 |
| #2 | 12,443 | 260,178 | 175,570 |
| #4 | 31,326 | 654,993 | 441,993 |
| #6 | 14,606 | 305,403 | 206,088 |
| #8 | 19,754 | 413,048 | 278,727 |
| #11 | 24,074 | 503,367 | 339,675 |
| #15 | 72,451 | 1,514,889 | 1,022,257 |
| #20 | 85,624 | 1,790,317 | 1,208,117 |
| #19 | 176,345 | 3,687,210 | 2,488,152 |
| #16 | 42,995 | 898,995 | 606,647 |
| Total | 774,364 | 16,191,253 | 10,925,958 |

**Table L2.5.4-3 Diversion Flow Water Quality Concentrations**

| Constituent | Belle Chasse Average (mg/l) | Baton Rouge Average (mg/l) | Average Value Used for Model Input (mg/l) |
|---|---|---|---|
| BOD5 | 3.60 | 1.97 | 2.79 |
| Organic Carbon - Dissolved | 3.89 | 3.68 | 3.79 |
| Organic Carbon - Particulate | 3.39 | 3.15 | 3.27 |
| Dissolved Ammonia | 0.04 | 0.02 | 0.03 |
| Organic Nitrogen - Dissolved | 0.35 | 0.35 | 0.35 |
| Organic Nitrogen - Particulate | 0.28 | 0.25 | 0.27 |
| Nitrate + Nitrate - Dissolved | 1.58 | 1.51 | 1.55 |
| Total Phosphorus | 0.27 | 0.23 | 0.25 |
| Organic Phosphorus - Dissolved | 0.01 | 0.01 | 0.01 |
| Organic Phosphorus - Particulate | 0.17 | 0.14 | 0.16 |
| Total Nitrogen | 2.35 | 2.23 | 2.29 |

Model input loads were calculated using the diversion flow multiplied by the various water quality concentrations and a conversion factor. Labile and particulate components were estimated using the assumption that the refractory component represented 80% of the organic species, while the labile component was comprised of the remaining 20%. A summary of the daily and yearly loads for total phosphorus, total nitrogen and nitrate+nitrite (NOx) from the diversion flow is shown in **Table L2.5.4-4** for a 3,000 cfs diversion flow. As shown in the table, the total nitrogen load for the diversion flow is 37,062 lb/day, while the total phosphorus load is 4,043 lb/day.

**Table L2.5.4-4 Nutrient Loads for 3,000 cfs Diversion Flow**

| Constituent | Load (lb/day) | Load (lb/year) |
|---|---|---|
| Total Phosphorus | 4,043 | 1,475,608 |
| Total Nitrogen | 37,062 | 13,527,628 |
| Nitrate+Nitrite | 25,010 | 9,128,527 |

### L2.5.5 Wetland Vegetation and Bottom Resistance

Flow resistance within the project area will relate directly to the type and density of existing vegetation and trees. Roughness coefficients were developed based on a wetland evaluation completed in support of the project. **Figure L2.5.5-1** illustrates how varying quality and characteristics of existing wetlands were applied to the model grid cells in order to represent 1) the Blind River; 2) Canals; 3) Areas with trees classified as 20-30 years to marsh; 4) Areas with trees classified 30-50 years to marsh; and 5) Areas with trees classified >50 years to marsh. For three different types of vegetation/tree zones, average tree diameters and tree densities were used in the EFDC model to better simulate vegetation resistance (**Table L2.5.5-1**).

**Table L2.5.5-1 Average Tree Diameters and Densities by Condition Class**

| Condition Class | Average tree diameter (ft) | Average tree density (tree/acre) |
|---|---|---|
| 20-30 years to marsh | 0.92 | 119 |
| 30-50 years to marsh | 1.21 | 184 |
| >50 years to marsh | 1.31 | 205 |

Bottom roughness values were used for different tree/vegetation zones in the project area. Based on field observation, for the areas with trees classified 20-30 years to marsh, the areas with trees classified 30-50 years to marsh, and the areas with trees classified > 50 years to marsh, the log law roughness height was assigned to a constant value of 15 cm, 10 cm, and 8 cm, respectively.

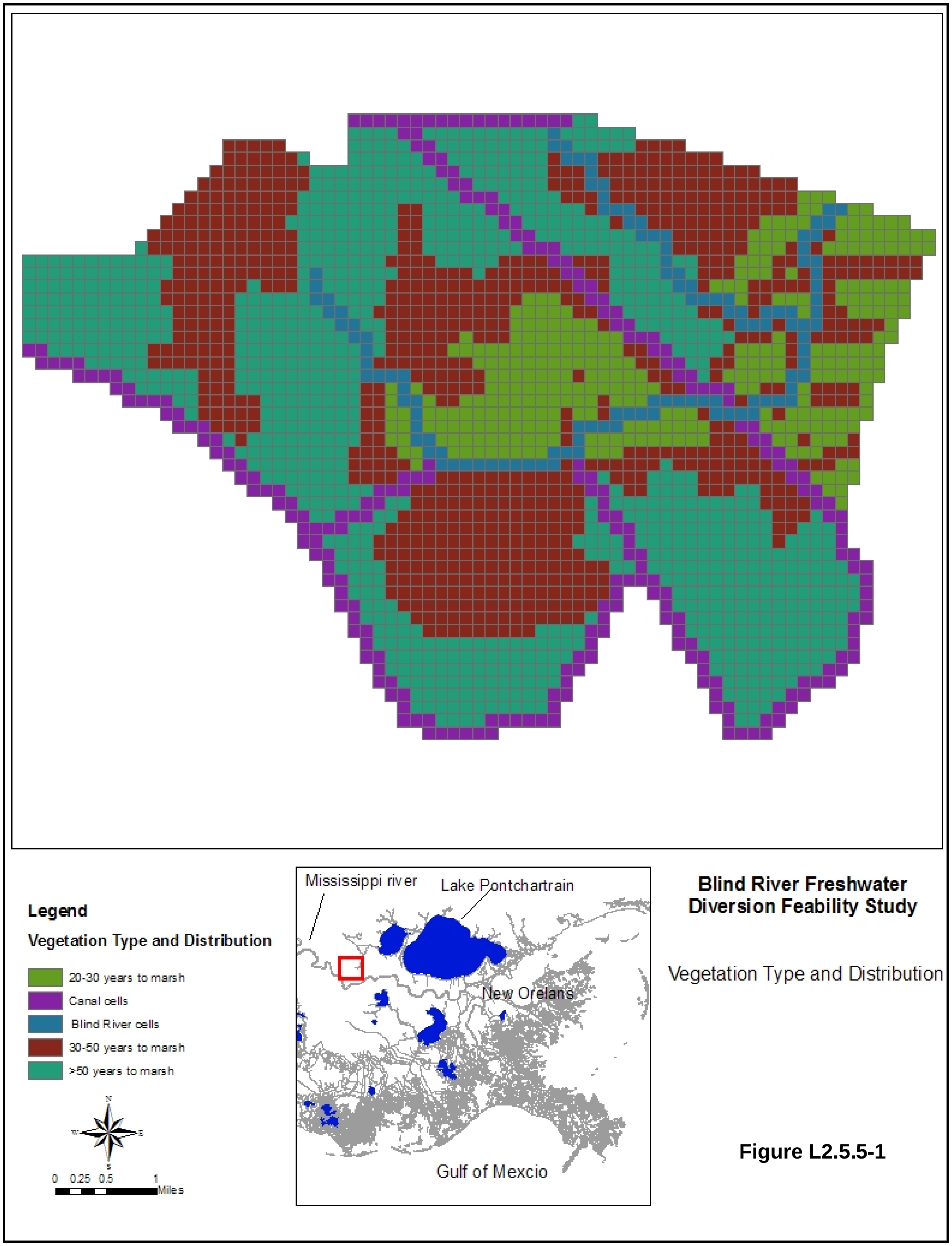



Figure L2.5.5-1

### L2.5.6 Meteorological Forcing

Meteorological forcing boundary data include atmospheric pressure, dry atmospheric temperature, wet bulb atmospheric temperature, rainfall, evaporation rate, solar short wave radiation, fractional cloud cover, and wind speed and direction. Because no meteorological stations are located near the project area, the Donaldsonville station (Station 2534), which is located about nine miles west from the center of the project area, and the LSU Ben-Hur Farm station (Station 5620), which is located about 30 miles northwest from the center of the project area, were used for rainfall and evaporation data collection, respectively. All other meteorological forcing parameters were collected from the Baton Rouge station. To more accurately represent the tree canopy shading effect that blocks part of the solar radiation, the solar radiation data were multiplied by a factor of 0.67 and 0.33 for the wetland and the canals / Blind River, respectively, in the model simulations.

### L2.5.7 EFDC Model Testing

Initial EFDC model setup was based on existing conditions. The year 2003 boundary data including meteorological forcing data, watershed inflows, canal and river stages, and water quality data were considered in the model setup. To test and verify the model setup, a dye test run was conducted to check mass balance, model connectivity, and water depth and velocity against best knowledge of the swamp. The test showed that the model was properly set up and all simulated results were reasonable.

It should be pointed out that no model calibration and validation were conducted due to lack of field data.

### L2.5.8 Existing Condition Simulation

In the existing condition, the existing canal/channel system conveys/drains the surrounding watershed flow and wetland flow into the Blind River when the Blind River stage is low. However, when the river stage is higher than the swamp bottom elevation due to the higher stage of Lake Maurepas, the river flows backward and floods the swamp.

In the model simulation for the existing condition, represented by simulation of the year 2003, the HEC-HMS simulated surrounding watershed runoffs were treated as point sources feeding into the project area at various locations and the HEC-RAS simulated stages at Conway Canal and I-10 on the Blind River were used as stage boundaries. The locations of the flow and stage boundaries are indicated on Figure L2.5.1-1.

Each simulation ran for the first 300 days of 2003. The simulated water depth and elevation, sediment, and water quality results of the swamp are summarized and discussed in the following sections.

#### L2.5.8.1 Water Depth and Elevation

**Table L2.5.8-1** summarizes the simulated average water depth for different hydrologic response units (HRUs) as shown on Figure L2.5.1-1 and the project area excluding the canals and Blind River for different scenarios. The largest and smallest water depth occurred in Subbasins 200 and 150 with a depth of 1.45 ft and 0.64 ft, respectively. The initial model condition for the Existing Conditions (2003) lasted for almost eight months due to the stagnant swamp flow with very small velocity; therefore, the average water depth and elevation and flow velocity shown in Table L2.5.8-1 were calculated based on the results of the last two months of the 10-month model run. The average water depth for the project area excluding the canals and Blind River was 0.90 ft. The water elevations for various HRUs in the project are also summarized in Table L2.5.8-1. Except for Subbasin 150, all other HRUs have a similar water elevation of 2.40 ft NAVD88.

The model simulated spatial distributions of water depths and elevations throughout the project area are presented on **Figure L2.5.8-1** and **Figure L2.5.8-2**, respectively.

**Table L2.5.8-1 Average EFDC Results by Hydrologic Response Unit**

| Hydrologic Response Unit/Subbasin | Water Depth (ft) | Velocity (ft/day) | Sediment Volume (cubic yards) | Hydraulic Residence Time (days) |
|---|---|---|---|---|
| Project Area | 0.90 | 256 | -2.78E+04 | - |
| 100 | 0.74 | 154 | -6.24E+00 | 42.0 |
| 110 | 0.78 | 158 | 0.00E+00 | 37.8 |
| 120 | 0.86 | 149 | -1.09E+01 | 37.4 |
| 140 | 1.02 | 416 | -1.71E+02 | 37.4 |
| 150 | 0.64 | 2,127 | -2.76E+04 | 8.1 |
| 160 | 0.83 | 258 | 0.00E+00 | 37.4 |
| 200 | 1.45 | 263 | -1.66E+01 | 38.3 |
| 210 | 0.81 | 200 | 0.00E+00 | 38.3 |
| 220 | 0.80 | 318 | 0.00E+00 | 37.8 |
| 300 | 1.25 | 294 | 0.00E+00 | 37.4 |
| 320 | 0.76 | 357 | 0.00E+00 | 37.2 |
| 330 | 1.08 | 383 | 0.00E+00 | 37.4 |

#### L2.5.8.2 Water Velocity

The model simulated existing conditions average velocity for different HRUs and the project area excluding the canals and Blind River are also summarized in Table L2.5-6. The largest and smallest velocity occurred in Subbasins 150 and 120 with a velocity of 2,127 ft/day and 149 ft/day, respectively.

Existing wetland flow velocity was extremely small and most of the water in the wetland was stagnant during dry periods. However, during or shortly after a storm event, wetland velocity could be relatively higher due to runoff from the surrounding watershed. **Figure L2.5.8-3** shows the spatial distribution of the flow velocity in the project area at day 300. On that specific day, the flow velocities were very small, except those near the river exit at I-10.

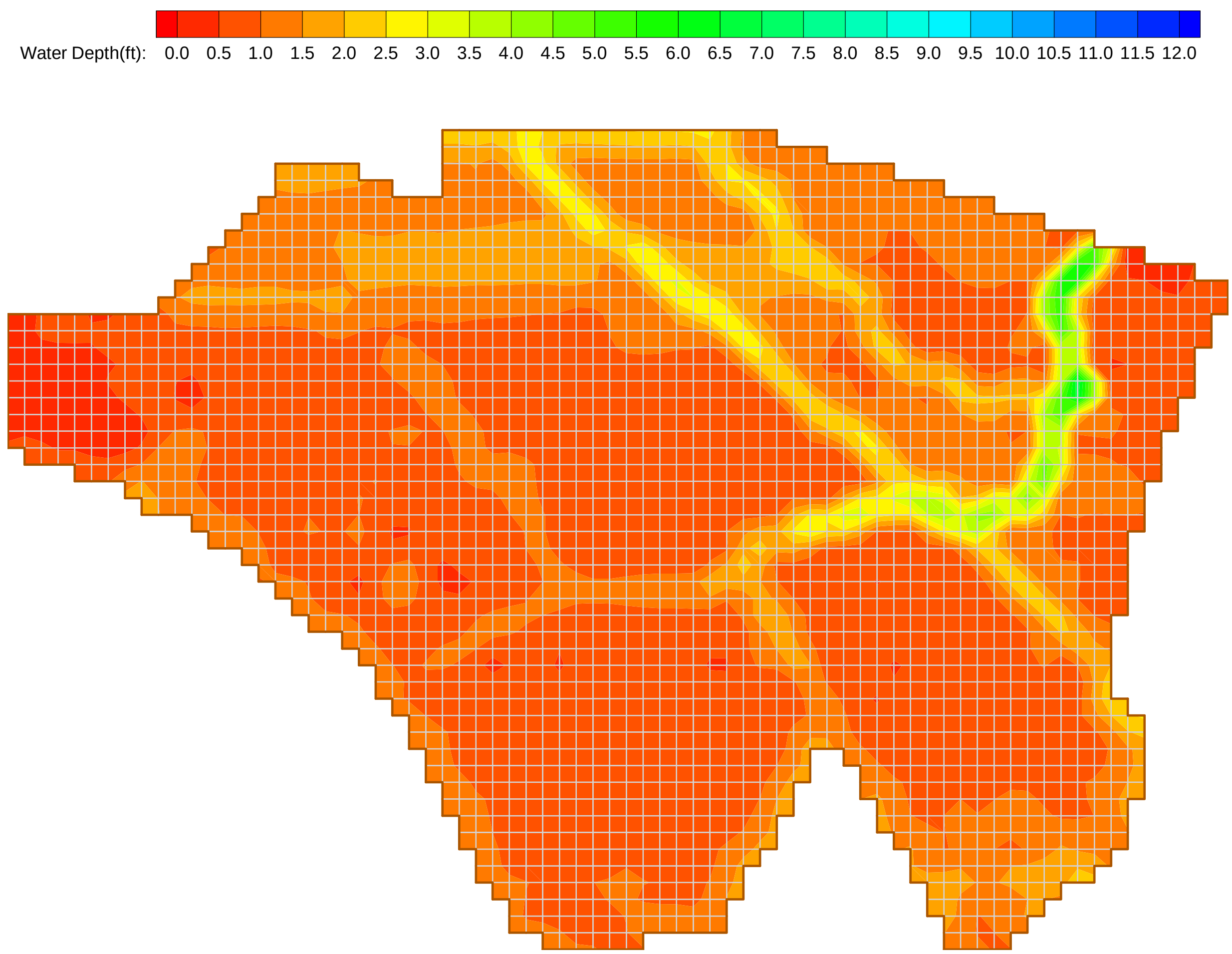



**Figure L2.5.8-1 Simulated Water Depth (Day 300)**

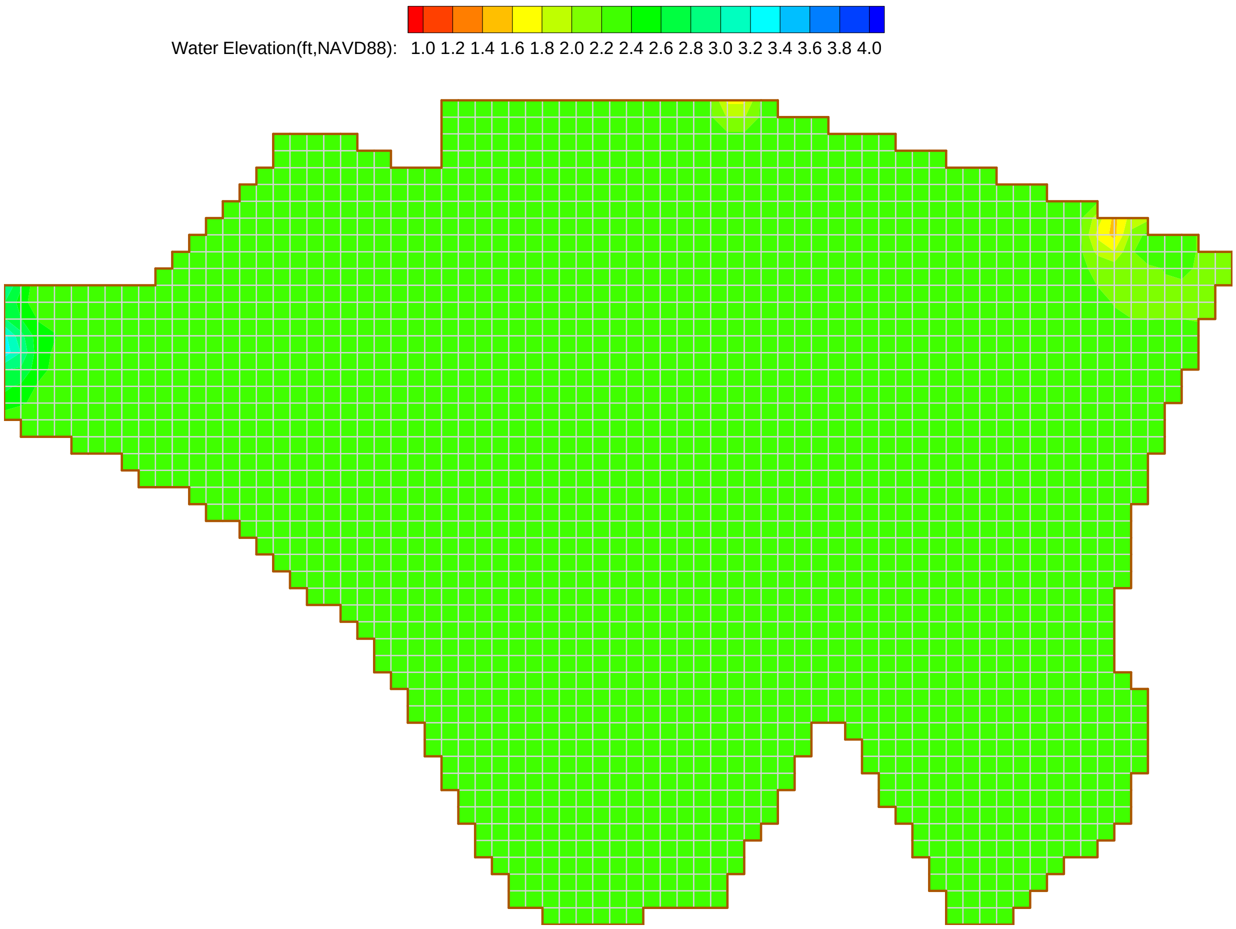



**Figure L2.5.8-2 Simulated Water Elevation (Day 300)**

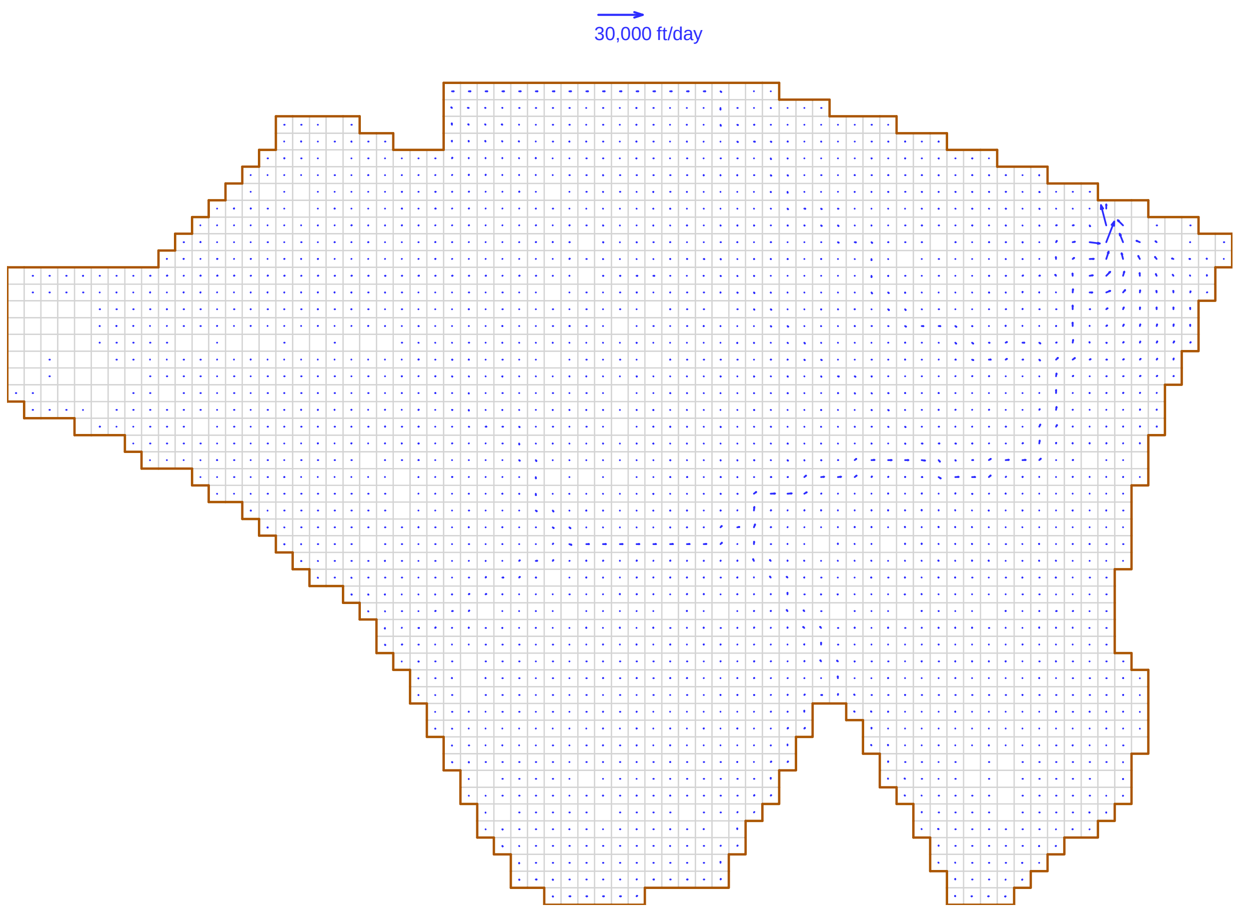



**Figure L2.5.8-3 Simulated Flow Velocity (Day 300)**

### L2.5.8.3 Hydraulic Residence Time

Hydraulic residence time (HRT) is another target parameter in wetland restoration. HRT is defined as the time a fluid parcel takes to travel from its initial location to one of the model domain exits. Therefore, HRT is a function of velocity and can vary spatially and temporally. In general, higher velocity results in lower HRT. However, if water flows circularly, the HRT can still be large.

For each HRU, an EFDC conservative dye model was set up to accurately simulate the HRT. In this approach, both flow advection and diffusion were evaluated in the EFDC model during calculation of the HRT for each HRU; therefore, it is more accurate than other methods, such as calculating HRT using particle tracking technique, which only accounts for advection.

For existing conditions, HRT depends on how long a storm event occurs after dye release. Therefore, it is expected that the HRT will be very big during the dry season and will be relatively small during the wet season. For consistency, the time when the peak dye concentration exited from the river at I-10 was used in calculating the HRT for each HRU. The simulated average HRTs for different HRUs are also summarized in Table L2.5.8-1. For Existing Conditions (2003), the HRTs are generally larger due to stagnant or very low velocity flow, with the largest HRT (42 days) at Subbasin 100 and the smallest HRT (8 days) at Subbasin 150.

**Figure L2.5.8-4** shows an example of the dye concentration plot at the I-10 Bridge on the Blind River for the Subbasin 100. It should be noted that none of the dye releases at any of the subbasins completely exit from the I-10 Bridge during one storm event, but rather after several storm events. **Figure L2.5.8-5** shows the spatial distribution of HRT for each model cell for the Existing Conditions (2003) at day 300. Although the color shaded HRT map only shows the HRT at the model cell scale, not the subbasin scale, it does indicate which part of the wetland has large or small HRTs, with pink-orange color for HRT around 2 hours and purple color for HRT around 168 hours (one week). The gray shaded area on Figure L2.5.8-5 indicates that the HRT is higher than one week. Based on the definition of HRT, the HRT for each subbasin is the integration of the HRT at each model cell along the path of fluid parcel traveling from each HRU to the river exit at I-10 Bridge. Therefore, the HRT for each subbasin is very large as shown in Table L2.5.8-1.

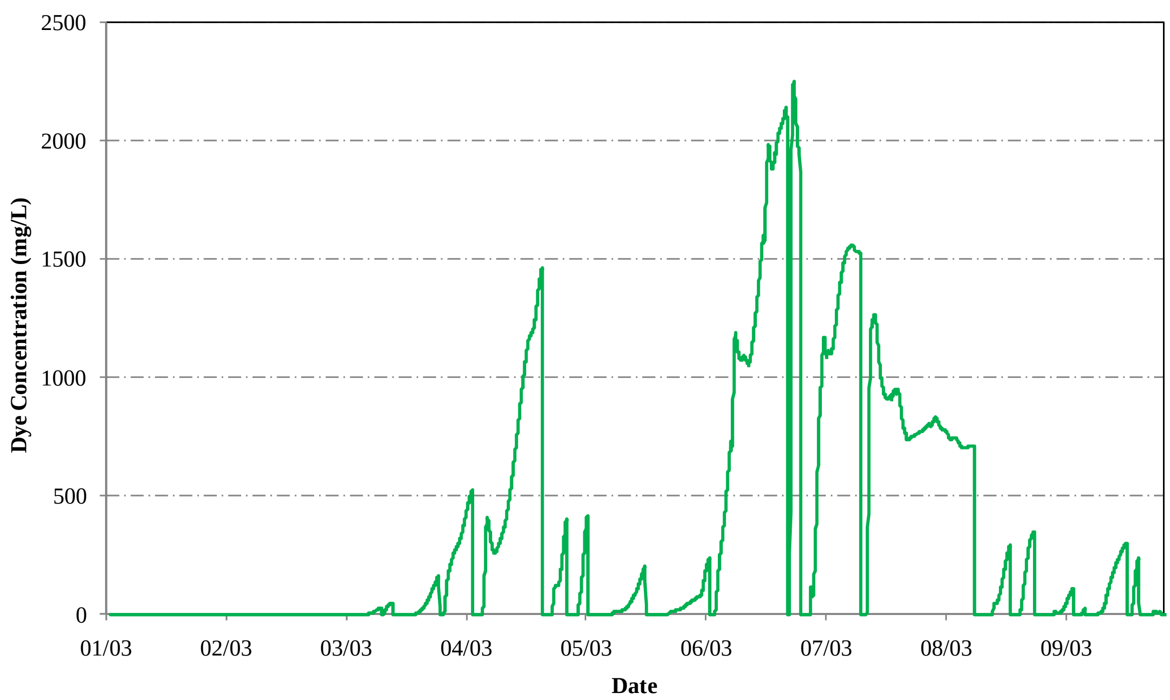



**Figure L2.5.8-4 Simulated Dye Concentration (Blind River at I-10)**

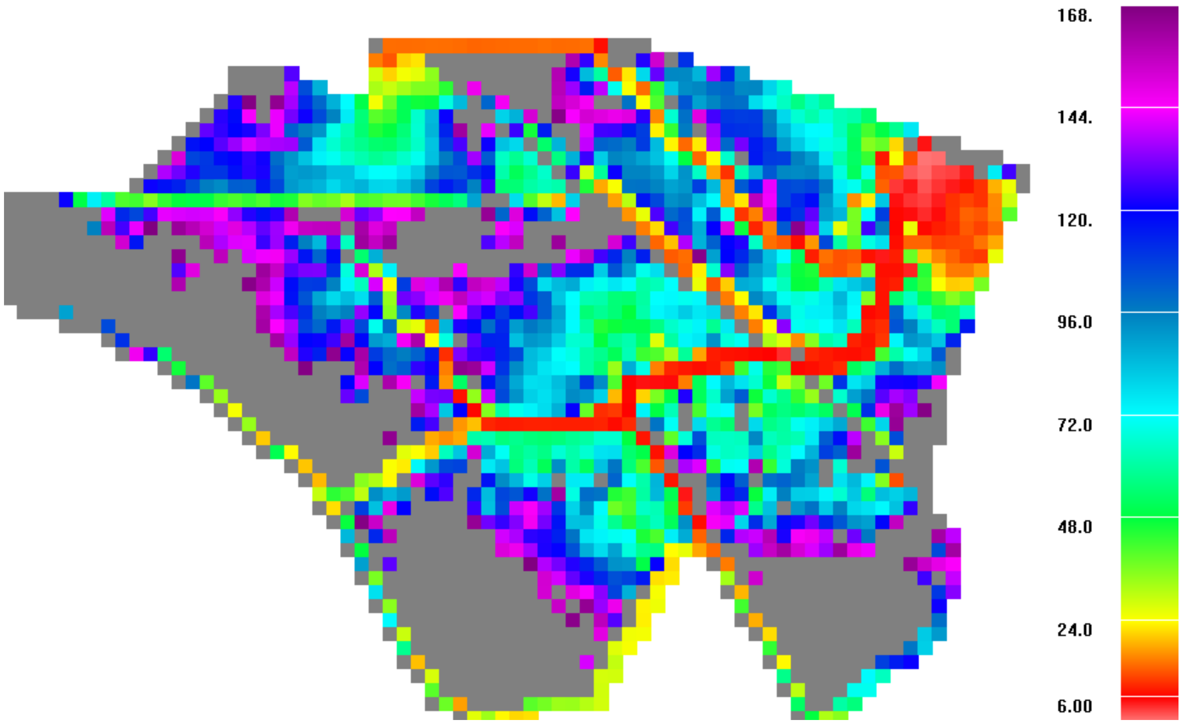



**Figure L2.5.8-5 Model Simulated Hydraulic Residence Time (Day 300)**

### L2.5.8.4 Sediment

For existing conditions, minimal sediment was brought into the project area from surrounding watershed runoff with very low sediment concentration. It is interesting to note, however, that some sediment erosion was simulated in Subbasins 140 and 150. The erosion occurred during the largest storm events during 2003 when relatively high flow velocity developed in the Blind River near the I-10 Bridge. Without any field data, however, it cannot be confirmed if the erosion truly occurred in these areas and if so, at what degree the erosion occurred. If no erosion actually occurred in these areas, then the shear stress for erosion used in the model should be increased to better reflect existing conditions. Spatial distribution of sediment cumulative erosion map for year 2003 is presented on **Figure L2.5.8-6**.

For the current sediment model, the five key parameter values used in the model are listed in **Table L2.5.8-2**.

**Table L2.5.8-2 Key Sediment Parameters and Assigned Model Values**

| **Parameter** | **Value** | **Unit** |
|---|---|---|
| Sediment bulk density | 1.66 | $g/cm^3$ |
| Settling velocity | 2.89E-05 | m/s |
| Boundary shear stress for deposition | 1.0E-05 | $m^2/s^2$ |
| Parameter | Value | Unit |
| Surface erosion rate | 5.0E-05 | $g/m^2/s$ |
| Boundary shear stress for erosion | 1.2E-05 | $m^2/s^2$ |

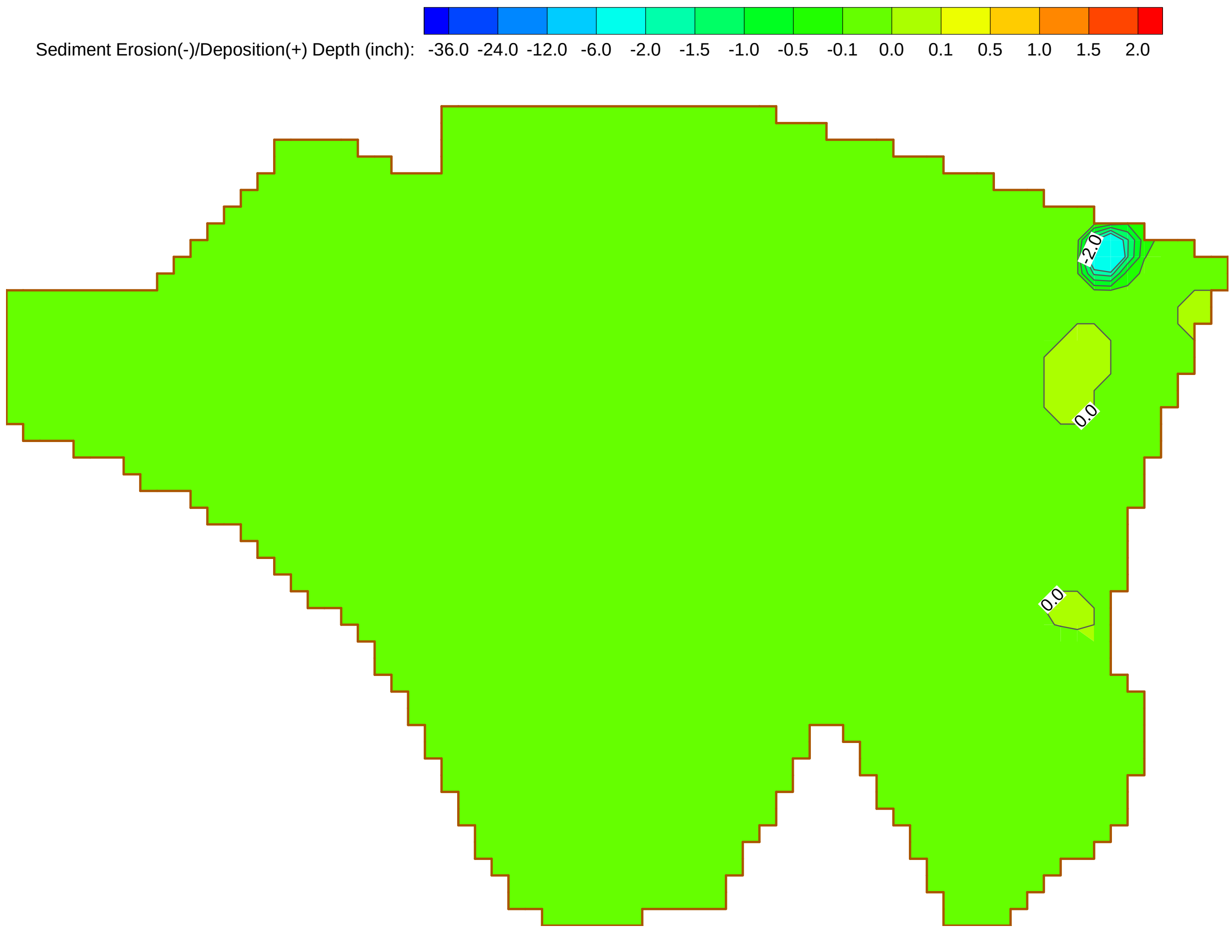



**Figure L2.5.8-6 Cumulative Sediment Deposition and Erosion**

**L2.5.8.5 Water Quality**

For existing conditions, no water quality data were collected. However, monthly-average total phosphorus (TP), total nitrogen (TN), and nitrate concentration data from the Maurepas swamp (Lane, et al., 2003) were available. The average concentrations of nitrate, TN, and TP were 0.008, 0.58, and 0.055 mg/L, respectively.

Furthermore, continuous water temperature and dissolved oxygen (DO) data were collected in the Blind River at Highway 61 Bridge during November and December of 2009. **Figures L2.5.8-7** and **L2.5.8-8** show the variations of the measured water temperature and DO, respectively. During this period, DO varied greatly from 0.6 mg/L up to 7.7 mg/L, which is primarily attributed to the local sediment oxygen demand (SOD) and low DO water exchange with the swamp by advection and diffusivity.

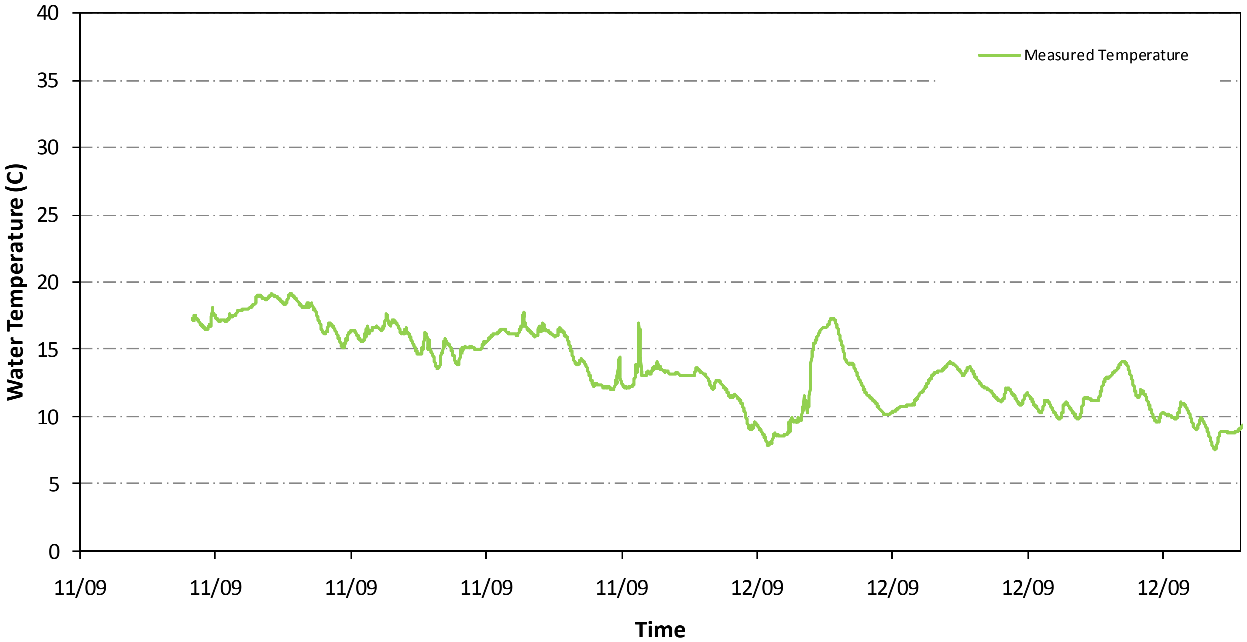



**Figure L2.5.8-7 Measured Blind River Water Temperature (November-December 2009)**

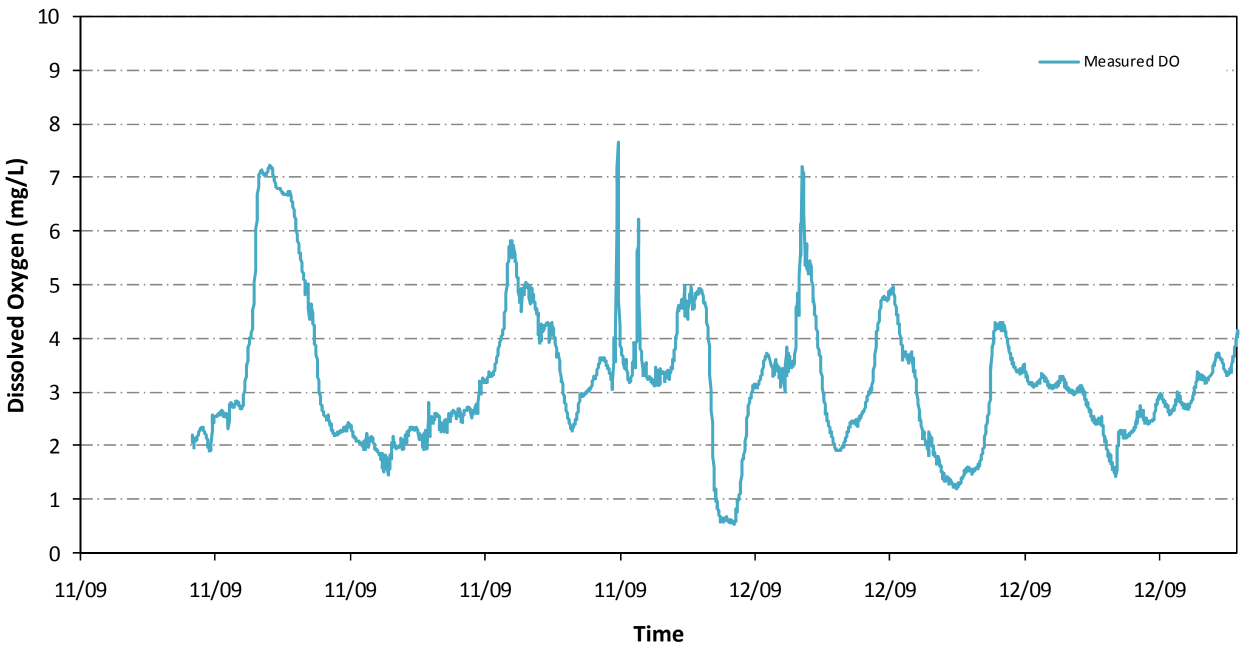



**Figure L2.5.8-8 Measured Blind River DO Concentrations (November-December 2009)**

## L2.6 Hydraulic Analysis of Romeville Diversion and Transmission Components

The proposed Romeville diversion point is located on the east bank of the Mississippi River near Mile 162.0 (2004 Hydrographic Survey), as shown on **Figure L2.6-1**.

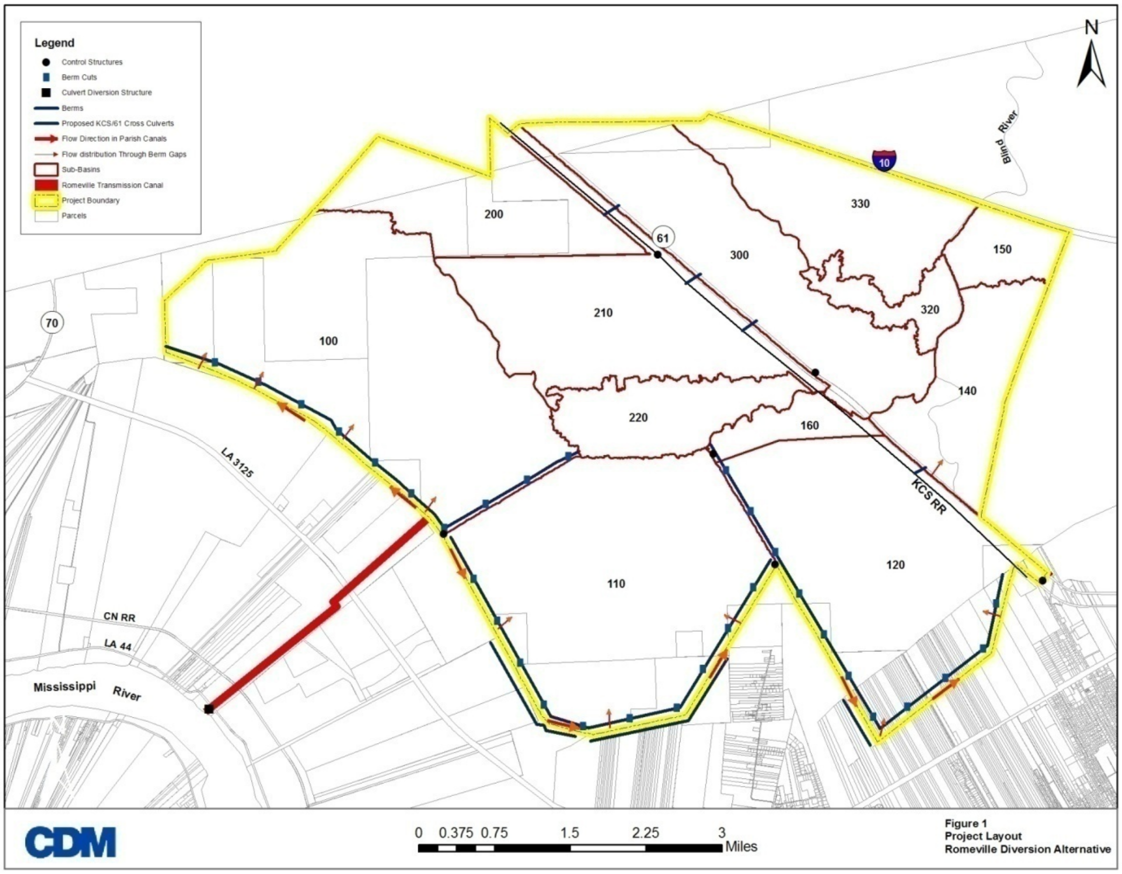



**Figure L2.6-1 Romeville Diversion Project Layout**

The diversion project requires several different types of management measures, or components, serving different functions, which will be combined to form the alternative plans. The major components are:

- Diversion facility;
- Transmission canal;
- Control structures;
- Berm gaps;
- Cross culverts at the Highway 61corridor; and
- Instrumentation.

This section presents the preliminary hydraulic analysis and design of the diversion facility and transmission canal components for the Romeville alignment. These two components were combined in this section as the design of both is based on the diversion flow rate, and both have a common hydraulic grade line.

The analysis addressed the full range of potential flow rates, management measures, and alternative plans considered as the project developed and the evaluation and screening occurred. The initial array of alternatives included diversion flow rates from 500 cfs to 20,000 cfs, and the preliminary hydraulic designs were prepared for this full range of flows. The specific flow rates being used in the initial alternative arrays are in 500 cfs increments from 500 cfs up to 5,000 cfs, then in 5,000 cfs increments to 20,000 cfs.

The proposed Blind River Diversion Project is on the NAVD 88 vertical datum. Other major topographic datasets being used on the project are also on the NAVD 88 vertical datum, including the 2001 LiDAR data and the 2004 Hydrographic Survey.

### L2.6.1 Mississippi River Stage Analysis at Romeville

The Mississippi River stage was analyzed to determine the statistical characteristics at the Romeville diversion point. The stage data will be used as input data to:

- Develop the overall diversion system hydraulic grade line (HGL);
- Hydraulically design the diversion structure;
- Hydraulically design the transmission canal;
- Determine probable diversion flow rates, total diversion volumes, and the likely operational characteristics throughout the calendar year; and
- Identify constraints to the diversion project, such as stage trends during each season, and limits to the diversion period and flow rates.

**Mississippi River Stage Data**

The proposed Romeville diversion point, near Mile 162.0 on the Mississippi River, is located between the College Point Landing Gage (Mile 156.9) and the Donaldsonville Gage (Mile 173.6). The river miles are from the Mississippi River Hydrographic Survey of 2004. Stage records were obtained for both gages from the USACE New Orleans Engineering Division website.

The lower Mississippi River flow management was changed by the USACE in 1977 when the Old River Control Structure (ORCS) was completed and put into service. This resulted in a revision to the balance of flow rates between the Atchafalaya River and the Mississippi River. Therefore, gage data was collected and analyzed for only the time period after the flow management change in 1977. The period of records analyzed covered 31 years from January 1, 1978 through December 31, 2008.

The stage records at the two gage sites are incomplete, with 258 and 96 data points missing at the College Point Landing and the Donaldsonville gages, respectively. Values were created for the missing data by linear interpolation and extrapolation from adjacent values and adjacent gages. Such values are high-lighted in the electronic files containing the data analysis. The approach to interpolate values was considered reasonable, as the Mississippi River stage varies slowly day to day.

The Mississippi River stage data is on the NGVD 29 vertical datum. The stage elevations were converted to NAVD 88, the project datum, using the vertical datum adjustment, as follows. The USACE provided a vertical adjustment value of -0.8 feet for the Donaldsonville gage. The USACE stated that a datum adjustment value is not available for the College Point Landing gage, but indicated that it is likely to be less than the adjustment at the Donaldsonville gage. Therefore a value of -0.7 feet was assumed for the College Point Landing gage, and will be used until an updated adjustment value is provided. The vertical datum conversion equations are as follows:

- College Point Landing Gage: NAVD 88 = NGVD 29 - 0.7 feet
- Donaldsonville Gage: NAVD 88 = NGVD 29 - 0.8 feet

The daily stage data, on both the original NGVD 29 vertical datum and adjusted to the NAVD 88 vertical datum is in the electronic files. The analysis and all stage data presented in this report is on the NAVD 88 vertical datum. **Table L2.6.1-1** provides a summary of the gage information.

**Table L2.6.1-1 Gage Data Summary**

| Item | Gage at College Point Landing | Gage at Donaldsonville |
|---|---|---|
| Gage ID | 01240 | 01220 |
| Vertical Datum (for data on website) | NGVD 29 | NGVD 29 |
| Gage 0 | Elev. 0 | Elev. 0 |
| River Mile (1962 Survey) | 157.4 | 175.4 |
| River Mile (2004 Survey) | 156.9 | 173.6 |
| Vertical Datum Adjustment (from NGVD 29 to NAVD 88) | | |
| Adjustment value | -0.7 | -0.8 |
| Adjustment equation | NAVD 88 = NGVD 29 - 0.7 | NAVD 88 = NGVD 29 - 0.8 |
| Start of Records | Dec. 18, 1879 | June 9, 1890 |
| Records used | 1/1/78 thru 12/31/08 | 1/1/78 thru 12/31/08 |

**Stage Data at Diversion Point**

Mississippi River stage data was developed for the proposed Romeville diversion point by linear interpolation between the two gage locations, based on river miles from the 2004 Hydrographic Survey. This resulted in new set of daily stage data at Romeville. The interpolated stage data at Romeville is in the electronic files and plotted on **Figure L2.6.1-1**. The plot of the Mississippi River stage data indicates two very distinct three-month periods, as follows:

- High Stage – Spring (March 1 through May 31)
- Low Stage – Summer-Fall (mid-August through mid-November)

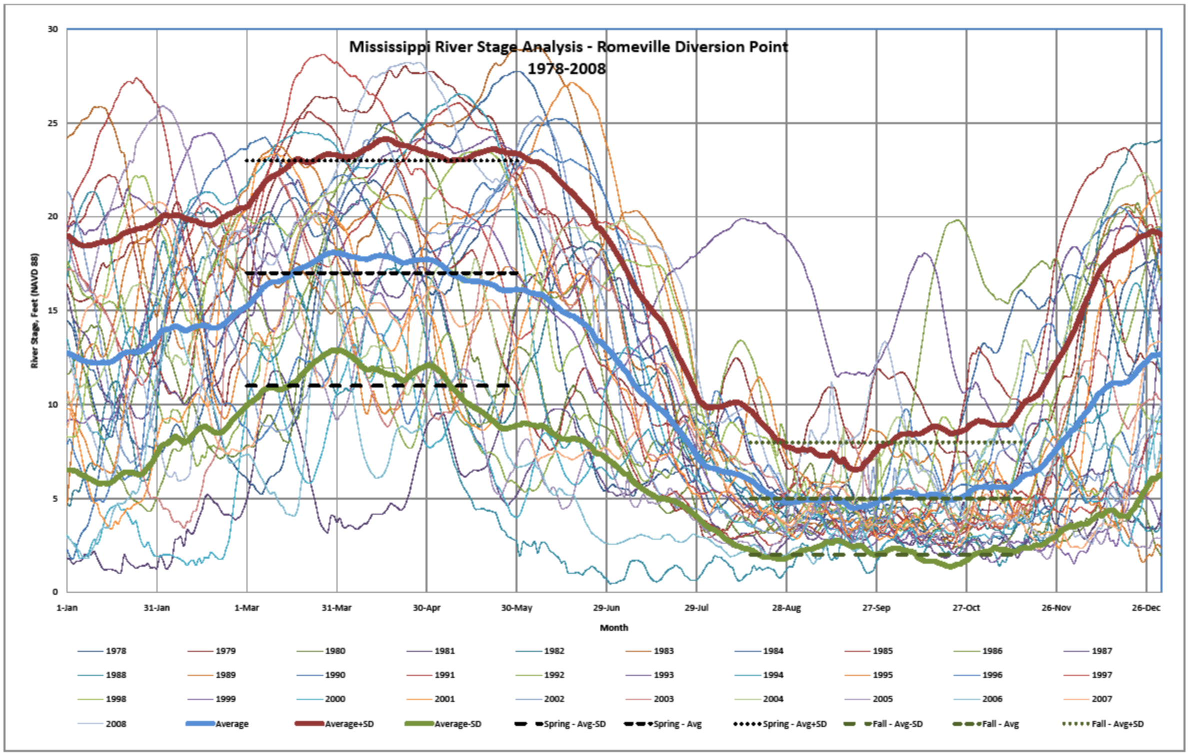



**Figure L2.6.1-1 Interpolated Stage Data (Romeville Diversion)**

**Statistical Analyses**

Three sets of statistical analyses were performed on the daily stage data at Romeville to determine trends and typical values for use in the analysis and design of the diversion system. The analyses included averages and standard deviations, percent exceedance, and histograms.

**Averages and Standard Deviations**

The average and standard deviation was calculated for each day of the year, using 31 values from the daily stage data. This analysis considered a normal distribution of the data. The daily average stage, the average stage minus one standard deviation, and the average stage plus one standard deviation are plotted on Figure L2.6.1-1. The averages and standard deviations were also calculated separately for the 3-month low-stage and high-stage periods, as shown in the following **Table L2.6.1-2** and plotted on Figure L2.6.1-1.

**Table L2.6.1-2 Standard Statistics for Stage**

| Period | Standard Deviation | Avg. - 1 SD (84.14% Exceedance) | Average (50% Exceedance) | Avg. + 1 SD (15.86% Exceedance) |
|---|---|---|---|---|
| Full Year | 7.03 | 4.29 | 11.32 | 18.35 |
| Spring (Mar. 1 – May 31) | 5.99 | 11.14 | 17.13 | 23.12 |
| Summer-Fall (Aug. 16 – Nov. 15) | 3.07 | 2.09 | 5.16 | 8.23 |

**Percent Chance Exceedance**

A percent chance exceedance curve (**Figure L2.6.1-2)** was developed for the Romeville diversion point using the Weibull formula. A Pearson Type III analysis was not used, as this stage and diversion analysis is not concerned with the extreme events. The stage values at selected statistical points are included in **Table L2.6.1-3**.

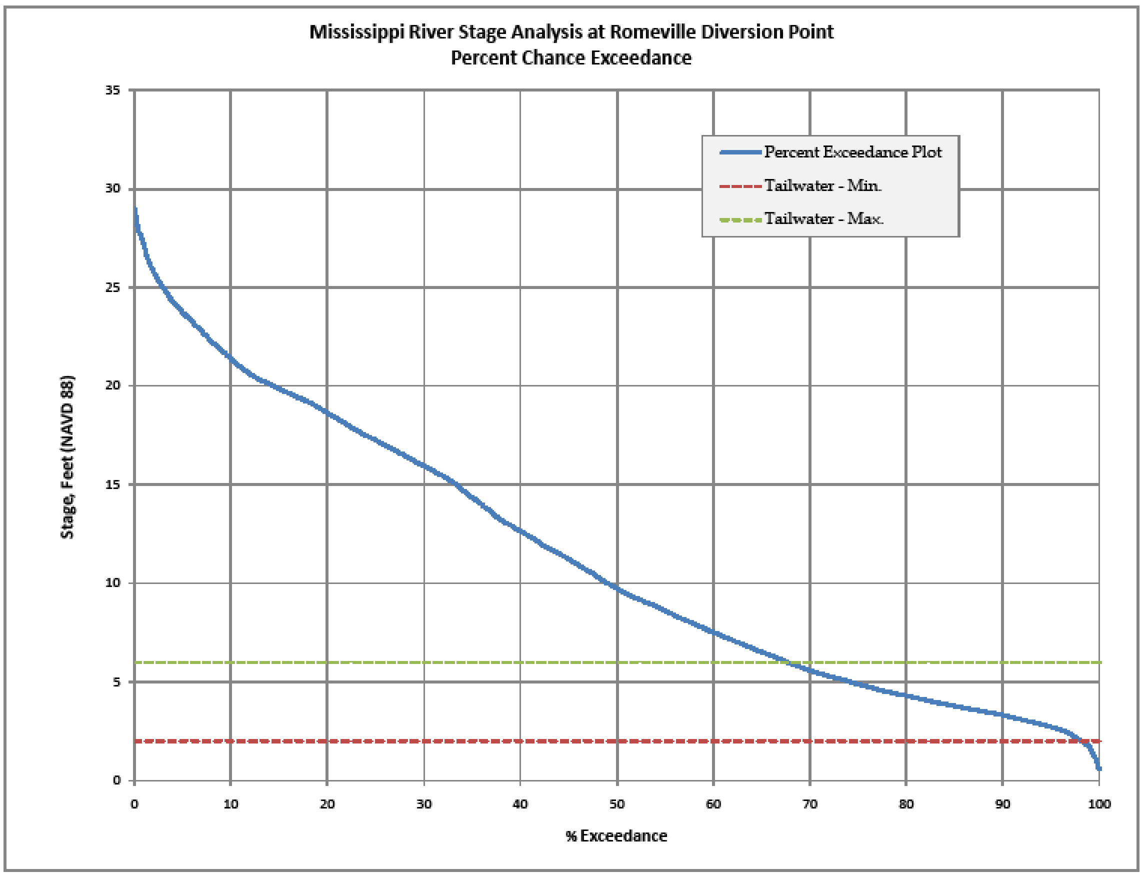



**Figure L2.6.1-2 Percent Chance Exceedance Curve (Romeville Diversion)**

**Table L2.6.1-3 Stage vs. Percent Exceedance**

| Period | Avg. - 1 SD (84.14% Exceedance) | Average (50% Exceedance) | Avg. + 1 SD (15.86% Exceedance) |
|---|---|---|---|
| Full Year | 3.86 | 9.73 | 19.66 |
| Spring (Mar. 1 – May 31) | 10.31 | 17.79 | 23.42 |
| Summer-Fall (Aug. 16 – Nov. 15) | 3.00 | 4.21 | 7.17 |

Of interest, is the relation of the diversion structure tail water conditions versus the Mississippi River stage. The Mississippi River stage will be at or above the following elevations:

- Elev. 2 – lower tail water limit – 98% Exceedance
- Elev. 6 – design tail water – 67% Exceedance

**Histograms**

Histograms were created by grouping the stage data into 1-foot increments and plotted to visually observe distribution trends in the stage data. **Figure L2.6.1-3** is a plot of all of the data, and indicates a distinct peak at Elev. 4 and a lesser peak at Elev. 20. **Figure L2.6.1-4** is a plot of the spring stage from March 1 through May 31, indicating a relatively wide range of values during the spring period. There is a peak at Elev. 20, with a generally even distribution from Elev. 11 to Elev. 24. **Figure L2.6.1-5** is a plot of the summer-fall stage data from August 16 through November 15, showing a distinct peak at Elev. 4, and demonstrates that most of the stage values are well below Elev. 8. In reviewing Figure L2.6.1-1, the late summer and early fall stages are consistently low, corresponding to the histogram on Figure L2.6.1-5, and indicates that diversions may be difficult during the late summer and early fall period.

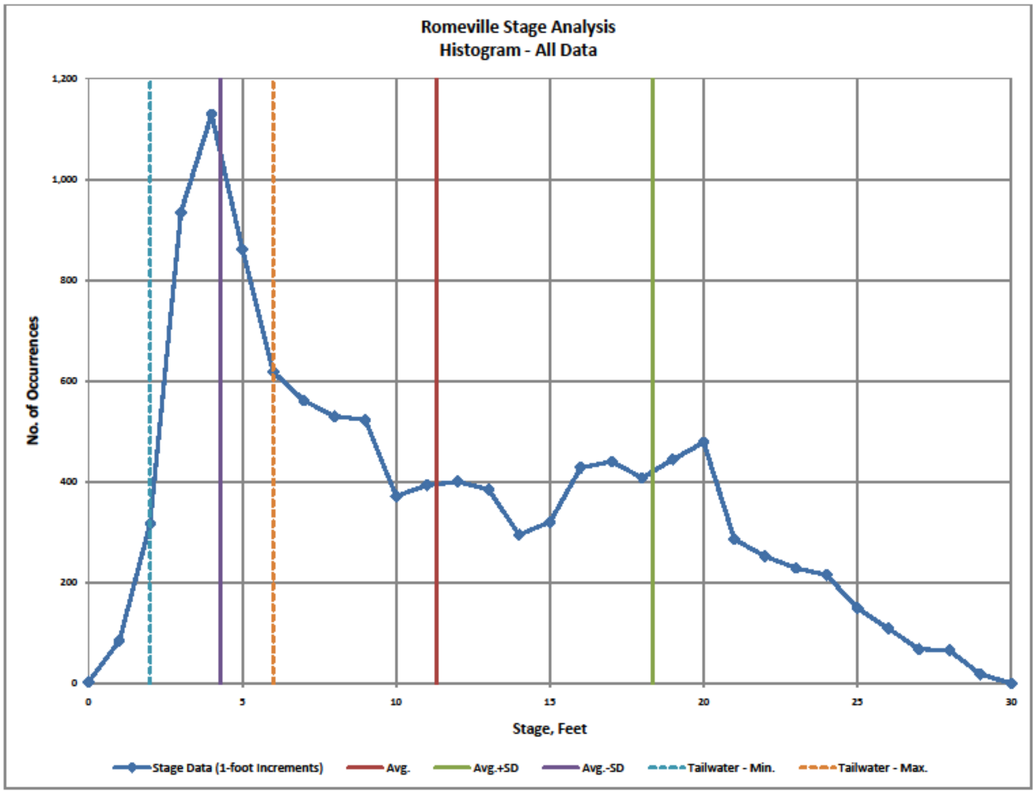



**Figure L2.6.1-3 Stage Histogram – All Data**

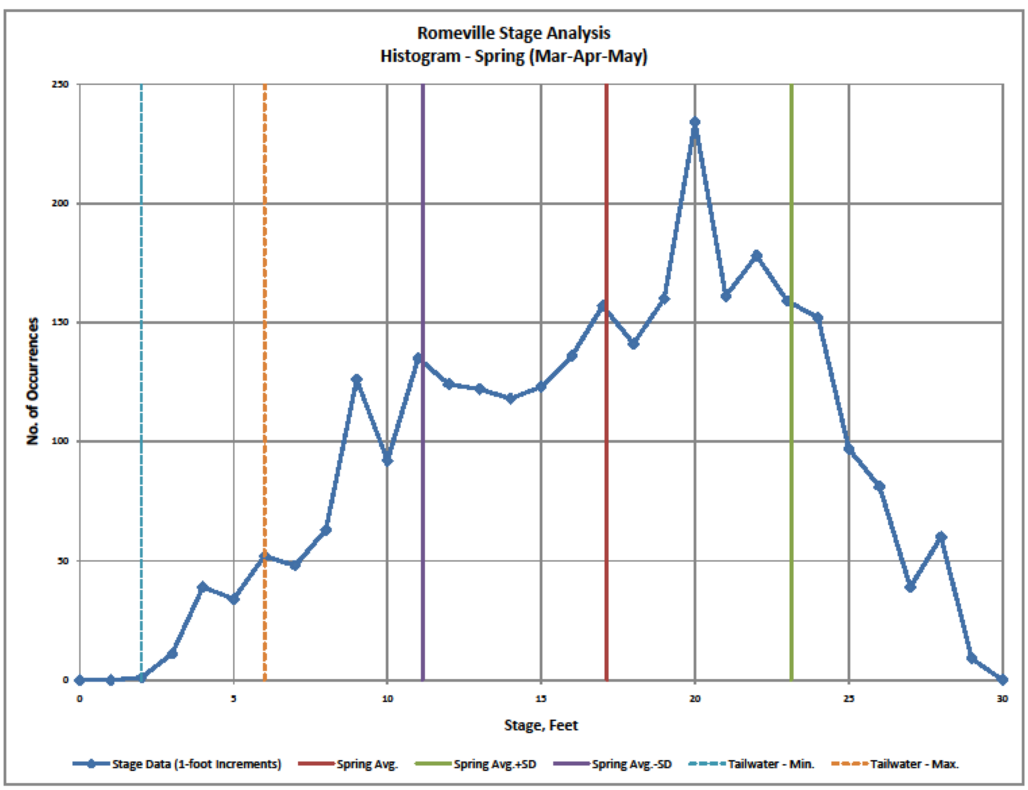



**Figure L2.6.1-4 Stage Histogram - Spring**

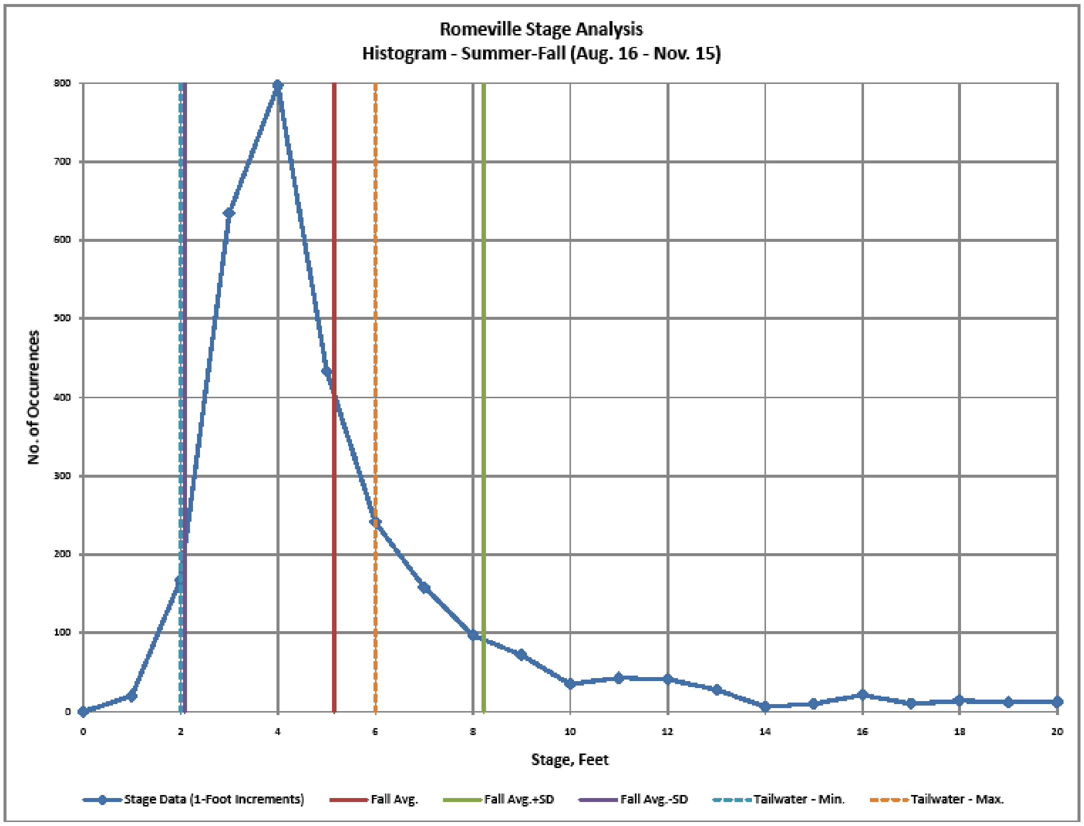



**Figure L2.6.1-5 Stage Histogram – Summer-Fall**

## Approximate Values for Design

Based on the statistical analyses, the following approximate values (**Table L2.6.1-4**) are recommended for design purposes.

**Table L2.6.1-4 Summary Stage Statistical Values**

| Period | Avg. - 1 SD (84.14% Exceedance) | Average (50% Exceedance) | Avg. + 1 SD (15.86% Exceedance) |
|---|---|---|---|
| Full Year | 4 | 10 | 19 |
| Spring (Mar. 1 – May 31) | 11 | 17 | 23 |
| Summer-Fall (Aug. 16 – Nov. 15) | 3 | 5 | 8 |

## L2.6.2 Romeville Hydraulic Grade Line

The hydraulic head available from the Mississippi River is the driving force for flow of the diverted water through the entire system. The principle hydraulic elements and segments of the overall system are:

- Mississippi River stage – the upstream boundary condition for the hydraulic grade line;

- Diversion Structure – to divert the flow through/under the east levee of the Mississippi River via a culvert or siphon;
- Transmission Canal – to transfer the flow from the diversion structure to the edge of the Swamp;
- Distribution System – to distribute flow into the Swamp;
- Overland Flow and Drainage System – to direct the diverted water through the Swamp, and then to drain it from the Swamp to the Blind River; and
- Blind River stage – the downstream boundary condition for the hydraulic grade line.

Based on the conceptual and preliminary design analyses, the following water surface elevations were used for the preliminary hydraulic design of the system components, thus establishing the system's hydraulic grade line (HGL). These values will be revised as the hydrodynamic modeling progresses and the designs for the various project components progress and are refined in the final design phase. **Figure L2.6.2-1** illustrates the HGL profile.

- Elev. 11 – The recommended design stage in the Mississippi River. The development of this recommendation is documented in the hydraulic design analyses for the culvert and siphon diversion structures.
- Elev. 7 – The proposed design tail water elevation at the diversion structure outlet. This allows a 1-foot drop in the HGL from the culvert or siphon outlet to the head of the transmission canal to route flows through a water quality treatment basin or a settling basin, if necessary.
- Elev. 6 – The approximate design water surface elevation at the head (upstream end) of the transmission canal, based on a 2-foot drop in the HGL for the canal. This hydraulic grade results in moderate velocities below 2 feet per second in the transmission canal.
- Elev. 4 – Water surface elevation at the downstream end of the transmission canal
- Elev. 4 – The proposed operating design water surface elevation in the existing drainage canals at the edge of the Swamp, providing a 2-foot driving head into and through the Swamp. This elevation will be set by the proposed control gates in the existing perimeter drainage channels.
- Elev. 1.5 to Elev. 2 – The approximate static water surface elevation throughout the Swamp, the drainage system, and Blind River between storm events.

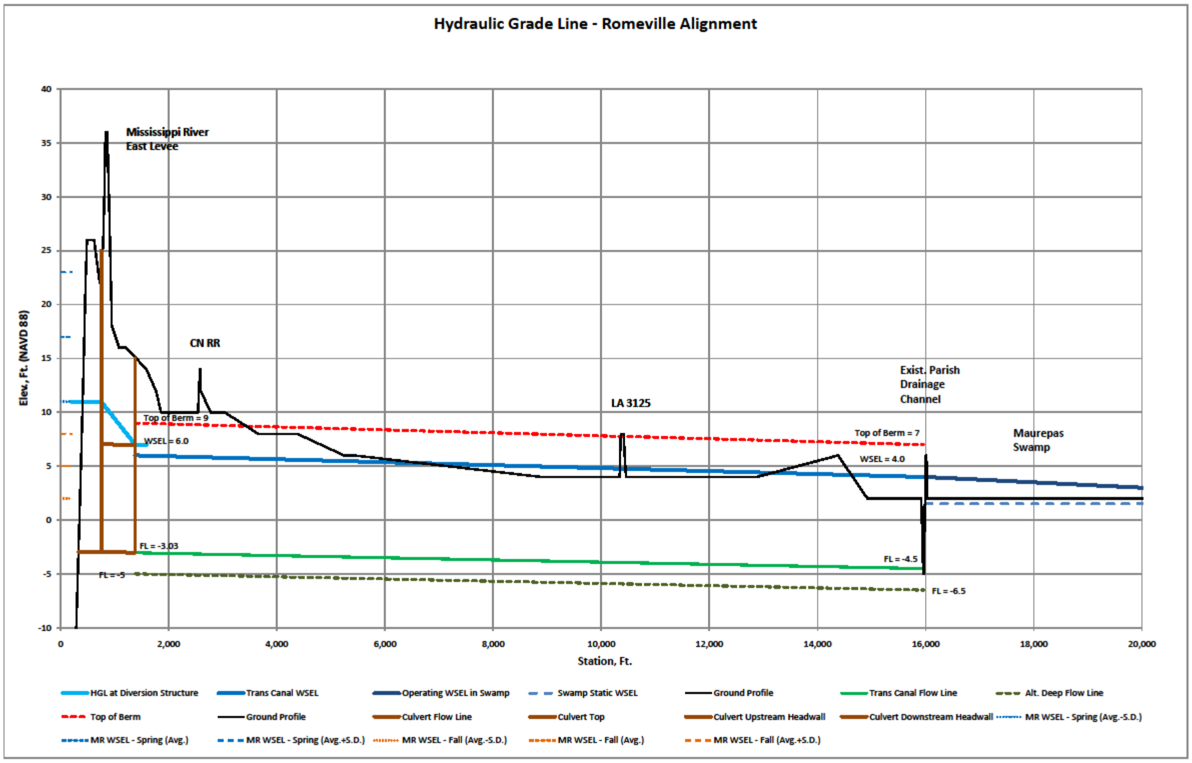



**Figure L2.6.2-1 Hydraulic Grade Line Profile (Romeville Diversion)**

The static HGL described above is to establish a single hydraulic design basis to size the diversion structure and the transmission canal. During actual operation, the HGL will vary significantly as the following items change: Mississippi River stage, Lake Maurepas stage, Blind River stage conditions, control gate settings, and diversion flow rates. In recognizing the variability in flow rate and HGL, the diversion culvert analysis and siphon analysis determined flow rates at other HGL conditions to indicate facility performance under the varying conditions.

**Minimum Stage Limits for Diversions**

The Mississippi River experiences saltwater intrusions from the Gulf of Mexico along the river bottom during extended periods of low flow in the river. The USACE installed an earthen/sand saltwater barrier or sill to approximately Elev. -55 to reduce the magnitude of the saltwater intrusions in 1988 and 1999. At the diversion point, the Mississippi River channel bottom is near Elev. -120 and the intake invert will be in the range of Elev. 0 to Elev. -10, and will not be extended to near the river bottom. For these two reasons, it is not anticipated that saltwater will be diverted into the Swamp.

For this analysis, it was considered that regulatory authorities may limit diversions at low stages in the Mississippi River. Other users may have prior rights, or more critical needs, such as for municipal water supplies or industrial uses. Therefore, the hydraulic analysis assumes there will be no diversion below a Mississippi River stage at Elev. 5. No attempts were made to make special provisions for hydraulic capacity below that elevation.

### L2.6.3 Diversion and Transmission System Flow Line Profile

The diversion facility and transmission canal flow line profile was established by existing physical conditions and the need to maximize depth at the Mississippi River to allow diversions during low stages in the river. The transmission canal will discharge into the existing St. James Parish drainage channels along the south and west perimeter of the Maurepas Swamp. At the Romeville diversion alignment, the existing drainage channel has a flow line of approximately Elev. -4.5, and this elevation was used as the limiting depth for the transmission canal. The diversion culvert flow line was set at Elev. -3.0 at the levee to have a minimum of an 8-foot depth to operate at Elev. 5 in the Mississippi River. Figure L2.6.2-1 illustrates the HGL profile.

**Alternate Flow Line Profile**

The 2009 bathymetric survey data for the existing drainage channel at the transmission canal discharge point has a flow line at Elev. -4.5. The bathymetric data indicates that the Blind River has a flow line deeper than Elev. -6.5 where the existing drainage channel discharges into it. The drainage channel plans from 1973 show a flat flow line at Elev. -10, indicating that there is several feet of silt in the existing drainage channels. If the existing drainage channels are lowered to Elev. -6.5, the transmission canal and diversion culvert could be lowered 2 feet, possibly reducing facility size and right-of-way requirements. During final design, a comparison of facility sizes should be done to determine cost and operational benefits of de-silting the existing outfall drainage channel, allowing the flow line for the diversion culvert and transmission canal to be lowered.

### L2.6.4 Romeville Diversion Culvert

The diversion culvert drawings for the Tentatively Selected Plan are included in **Annex L-5**. The diversion culvert will consist of a multi-cell box culvert. The culvert will cross under the east levee of the Mississippi River, and will be extended east under the local road, LA 44, which is located at the exterior base of the levee. The culvert will be extended an additional 100 feet east of the road right-of-way for a safety buffer, and to allow space for future potential widening of the road. The batture crossing, from the east Mississippi River bank to the inside base of the levee, can be either an extension of the culvert to the bank of the Mississippi River, or an inlet canal, as done with the Davis Pond diversion structure. Based on an initial cost comparison, an inlet canal was used for this preliminary design analysis. **Figure L2.6.4-1** illustrates the Romeville Diversion culvert profile.

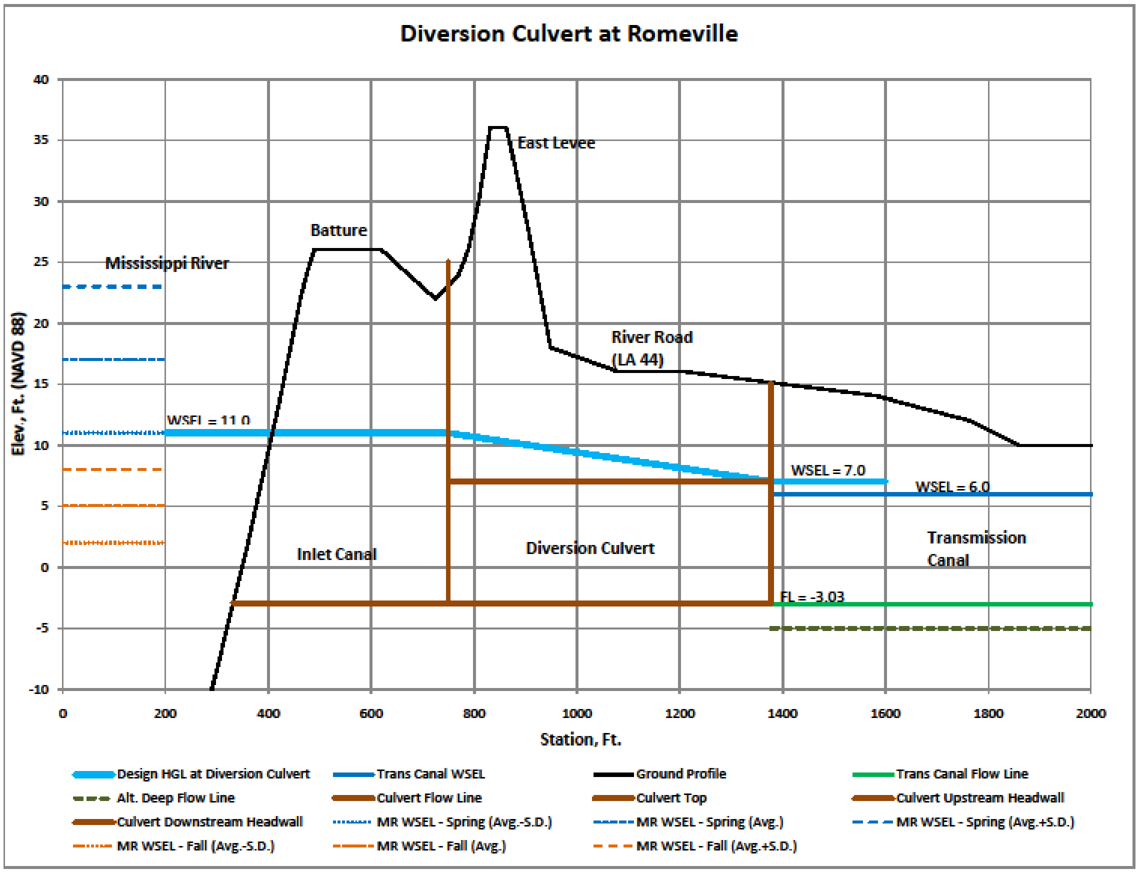



**Figure L2.6.4-1 Romeville Diversion Culvert Profile**

**Configuration**

The size and number of the culvert cells (or barrels) has a wide range of possibilities to accommodate the large range of potential design flow rates and Mississippi River stages. The initial diversion culvert design development used the following constraints:

- The lower design flow rates should have a minimum of two barrels to allow a degree of flow control by taking one or more barrels out of operation;
- The higher design flow rates should have a minimum of three barrels to allow a degree of flow control by taking one or more barrels out of operation;
- No limit on the maximum number of barrels;
- The culvert is expected to be a monolithic cast-in-place reinforced concrete structure; therefore shipping size limitations on pre-cast units are not applicable;
- Culvert sizes are limited to 14 feet, the largest sluice gates readily available;
- The culvert sections should be square, or nearly square, to maximize hydraulic efficiency;

- The top of the culverts are to be below the design WSEL in the Mississippi River to fully utilize the hydraulic capacity at design conditions; and
- No stand-by barrels or excess capacity are included, as the installation is non-critical for health and safety, such as would be the case for water supply, wastewater, or flood control pumps.

**Hydraulic Calculations**

A HEC-RAS model was developed for the diversion culvert and the transmission canal, extending from the Mississippi River, across the east levee, to the existing drainage channel at the perimeter of the Swamp. For the culvert analysis, this model was truncated at the downstream end of the levee culvert. Separate HEC-RAS geometry files, flow files, and plans were developed for each design flow rate to develop the culvert configurations (size and number of cells). The upstream channel (the inlet canal) and the downstream channel (transmission canal) widths were varied, corresponding to the widths determined for the preliminary transmission canal designs.

The headlosses through the culvert structure were calculated, as follows:

- Manning's n value – 0.035 for earthen channels and 0.015 for concrete channels and the concrete box culverts;
- Expansion and Contraction Losses – typical values were obtained from the HEC-RAS manual;
- Entrance and Exit Losses – typical entrance and exit loss coefficients were obtained from the HEC-RAS manual. Both the inlet and the outlet will be at a concrete headwall; and
- Trash Racks – trash racks will be included in the installation, but these were not included in the preliminary hydraulic calculations. Headlosses for trash racks should be included in the final design.

The diversion culvert will operate in a system with changing head and flow rates, as the Mississippi River stage varies through the year, and the tail water stage varies as the flow rate varies. For each design scenario, a set of flow rates were used in the hydraulic analysis to establish a rating curve for each culvert design option. At the downstream end of the culvert, the variable tail water conditions were incorporated into the HEC-RAS model. The starting water surface elevations for the truncated HEC-RAS model were linearly interpolated between the HGL parameters stated previously, using the following key values:

- Elev. 2.0 – no flow in the system;
- Elev. 7.0 – design flow rate; and
- Elev. 11.0 – maximum possible water surface elevation on the transmission canal, at the approximate adjacent ground elevations.

**Design and Analysis Process**

As shown on Figure L2.6.1-1, the Mississippi River stage varies through the year. However, in order to size the diversion culvert, a single Mississippi River stage needs to be selected. The culvert design can then be evaluated, to determine the availability of the design flow rate at other stages. For the preliminary analysis, two sets of diversion culvert designs were developed to provide the design flow rate at either Elev. 11 or Elev. 17 in the Mississippi River. For each design configuration, flow rates were then determined for three stages of interest in the Mississippi River:

- Elev. 5 – Average stage in late summer and early fall. This indicates the capabilities to provide a base flow rate during the summer and fall low stages in the Mississippi River;
- Elev. 11 – 84% exceedance stage (average stage minus one standard deviation) during the spring high water period; and
- Elev. 17 – 50% exceedance stage (average stage) during the spring high water period.

The flow rate characteristics were then reviewed to assist in selecting and recommending a single design stage in the Mississippi River. **Figure L2.6.4-2** illustrates a plot of the flow rate versus the culvert area for the three stages of interest in the Mississippi River. The plots indicate linear characteristics for flow rate versus culvert area. Based on the data, the culverts have relatively uniform unit flow rates for each design river stage (**Table L2.6.4-1**).

**Table L2.6.4-1 Mississippi River Design Stage vs. Unit Flow Rate**

| Mississippi River Design Stage (ft) | Unit Flow Rate cfs/SF (or fps) |
|---|---|
| Elev. 5 | 4.5 |
| Elev. 11 | 10.7 |
| Elev. 17 | 16.3 |

**Table L2.6.4-2** shows the percent of design flow rate diverted at each Mississippi River stage for the two design conditions.

**Table L2.6.4-2 Mississippi River Design Stage vs. Percent of Design Flow Rate**

| Mississippi River Design Stage (ft) | Percent of Design Flow at Each Mississippi River Stage (%) | | |
|---|---|---|---|
| | **Elev. 5** | **Elev. 11** | **Elev. 17** |
| Elev. 11 | 35 | 100 | 155 |
| Elev. 17 | 29 | 73 | 100 |

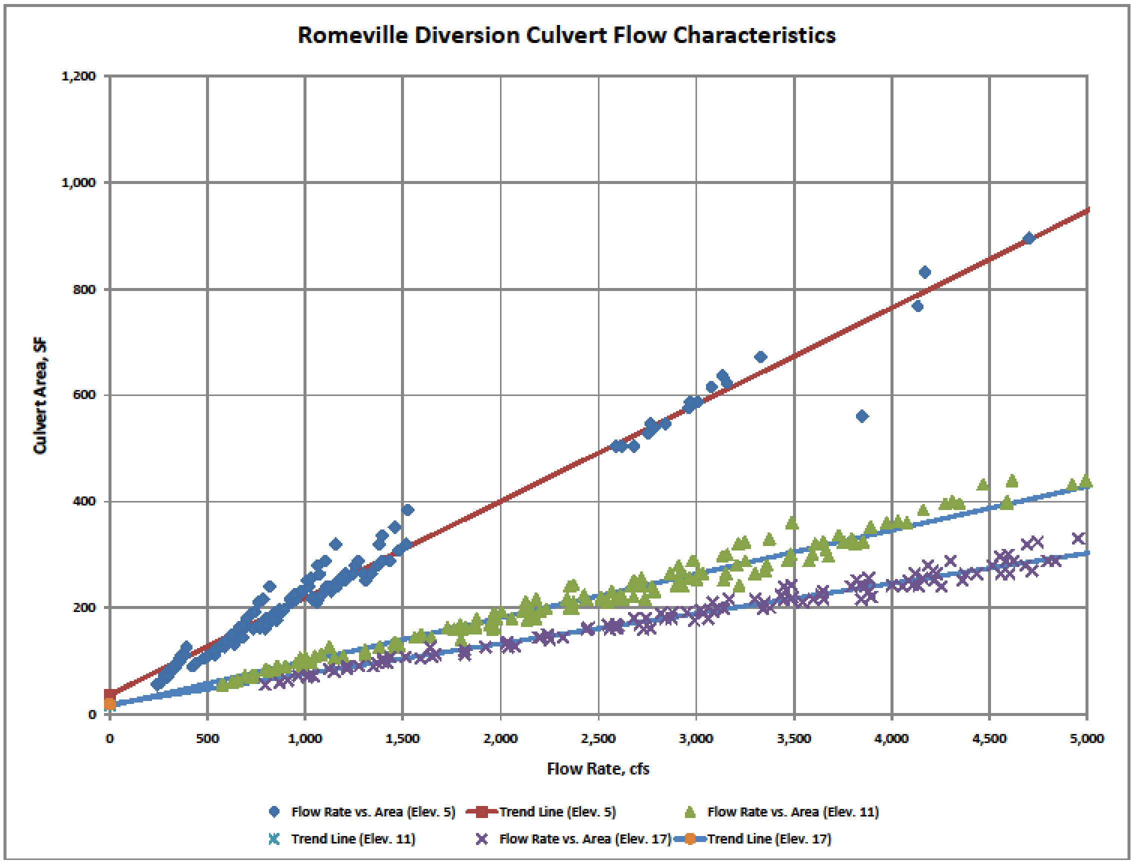



**Figure L2.6.4-2 Culvert Flow Characteristics (Romeville Diversion)**

Based on a comparison of the culvert sizes (area), changing the design basis from Mississippi River Elev. 17 to Elev. 11 results in a 52% increase in culvert size. Limited culvert designs were prepared for Elev. 5. This design level was considered impractical, as the box culvert flow area would have to be 2.4 times the size designed to Elev. 11.

It is recommended that the diversion be designed to deliver the design flow rate at Mississippi River stage Elev. 11, as this provides a higher flow rate potential during summer periods.

**Table L2.6.4-3** has the recommended culvert configurations for the potential design flow rates in the initial alternative arrays.

The primary design basis for the recommended configurations is as follows:

- Design water surface in the Mississippi River is Elev. 11.0;
- Design tail water is Elev. 7.0; and
- Culvert flow line is Elev. -3.0 at the levee.

Rating curves for the diversion culvert are plotted on **Figure L2.6.4-3** for the 1,500 cfs and 3,000 cfs designs for Elev. 11.

**Table L2.6.4-3 Recommended Culvert Configurations for Alternate Design Flow Rates**

| Design Flow Rate, cfs | Culvert Size, ft (height x width) |
|---|---|
| 500 | 2 - 6 x 6 |
| 1,000 | 2 - 8 x 7 |
| 1,500 | 2 - 9 x 9 |
| 2,000 | 3 - 9 x 8 |
| 2,500 | 3 - 9 x 9 |
| 3,000 | 3 - 10 x 10 |
| 3,500 | 3 - 11 x 10 |
| 4,000 | 3 - 12 x 11 |
| 4,500 | 4 - 11 x 10 |
| 5,000 | 4 - 11 x 11 |
| 10,000 | 7 - 12 x 12 |
| 15,000 | 10 - 12 x 12 |
| 20,000 | 13 - 12 x 12 |

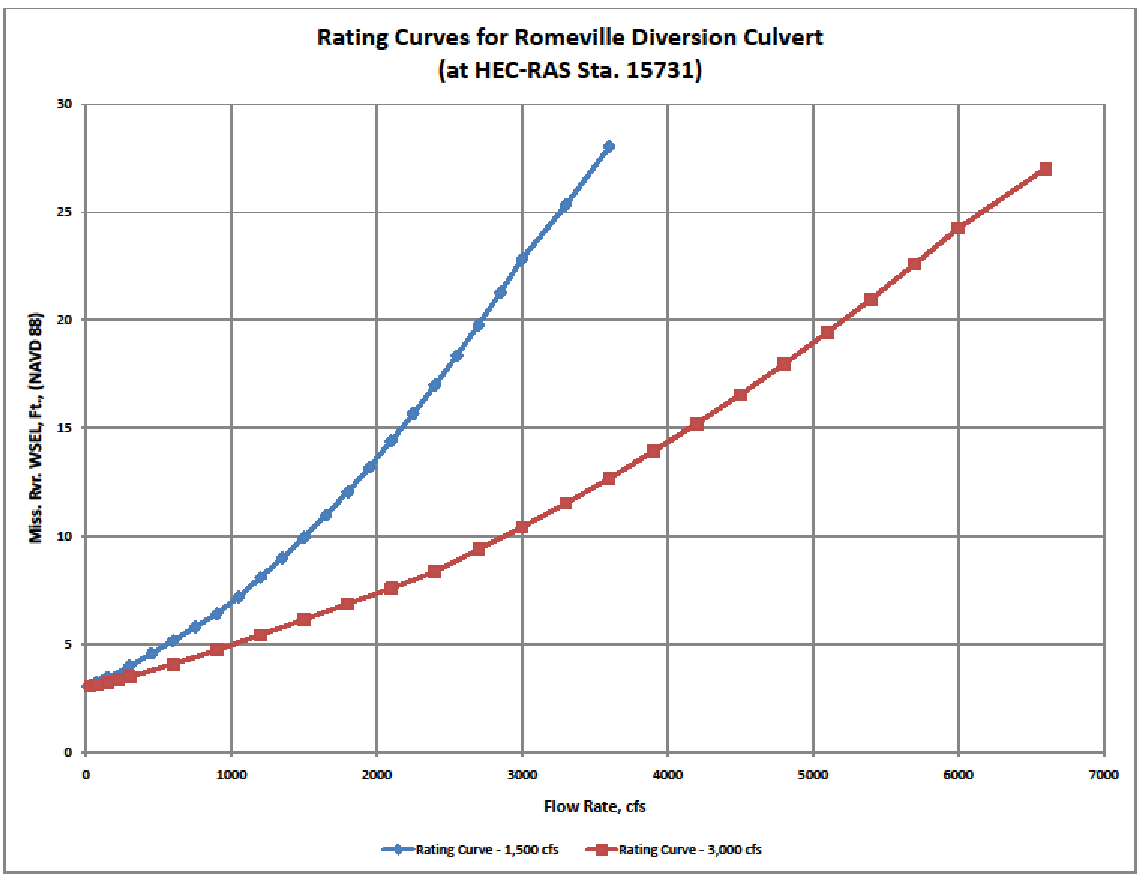



**Figure L2.6.4-3 Rating Curves for Romeville Diversion Culvert**

### L2.6.5 Romeville Diversion Siphon

A siphon was considered as a viable diversion structure. The siphon structure will consist of multiple independent siphon barrels operating in parallel. The siphon structure will require other facilities, which are addressed in Section L2.7.

**Design Basis**

The siphon pipe was extended across the batture and down the east bank of the Mississippi River for the preliminary hydraulic design. As an option, if the siphon is selected for the final design, an inlet canal could be considered from the Mississippi River to near the inside base of the levee. The siphon p;ipes were extended east across LA 44 to discharge into the transmission canal at a similar location as described for the diversion culvert.

The range of flow rates being considered for the Romeville diversion point is from 500 cfs to 20,000 cfs, or higher. The siphon design analysis covers a range of flows from 500 cfs to 5,000 cfs, as the siphon installation becomes too large and impractical at higher flow rates.

The HGL for the siphon is described in Section L2.6.2 and shown on Figure L2.6.2-1.

A siphon has a theoretical lift of 34 feet, and a practical maximum lift of 28 feet. To be conservative, 25 feet was used as the limiting value in this analysis, as the siphon will have large-diameter pipes.

**Siphon Profile**

Optional profiles were considered for routing the siphon at the Mississippi River levee, as shown on the following figures (a 72" diameter pipe was used for the example profiles):

- **Figure L2.6.5-1** – Route the siphon over the levee.
- **Figure L2.6.5-2** – Route the siphon over the River Road, to reduce the number of bends, and related headlosses. The River Road pavement is at approximately Elev. 16 and the bottom of the siphon pipe will be at Elev. 32 to provide the standard 16-foot clearance for trucks. The siphon pipe would be routed through the levee, to reduce the number of pipe bends.
- **Figure L2.6.5-3** – Route the siphon through the levee at an elevation allowing diversion down to minimal stages in the Mississippi River. If Elev. 5 is the minimum stage in the River for diversions, the siphon top-of-pipe would be at Elev. 30.

**Table L2.6.5-1** summarizes the siphon elevations and resultant lowest operating WSELs in the Mississippi River for the range of pipe sizes being considered.

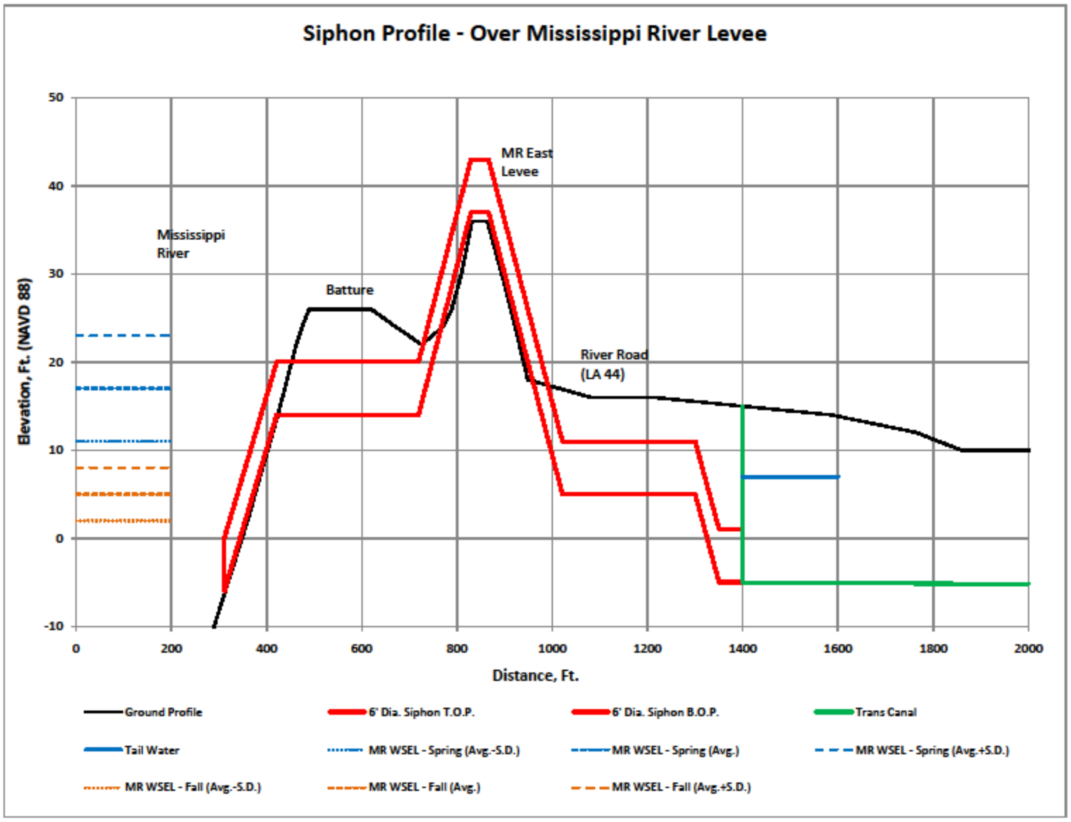



**Figure L2.6.5-1 Siphon Profile (Over Mississippi River Levee)**

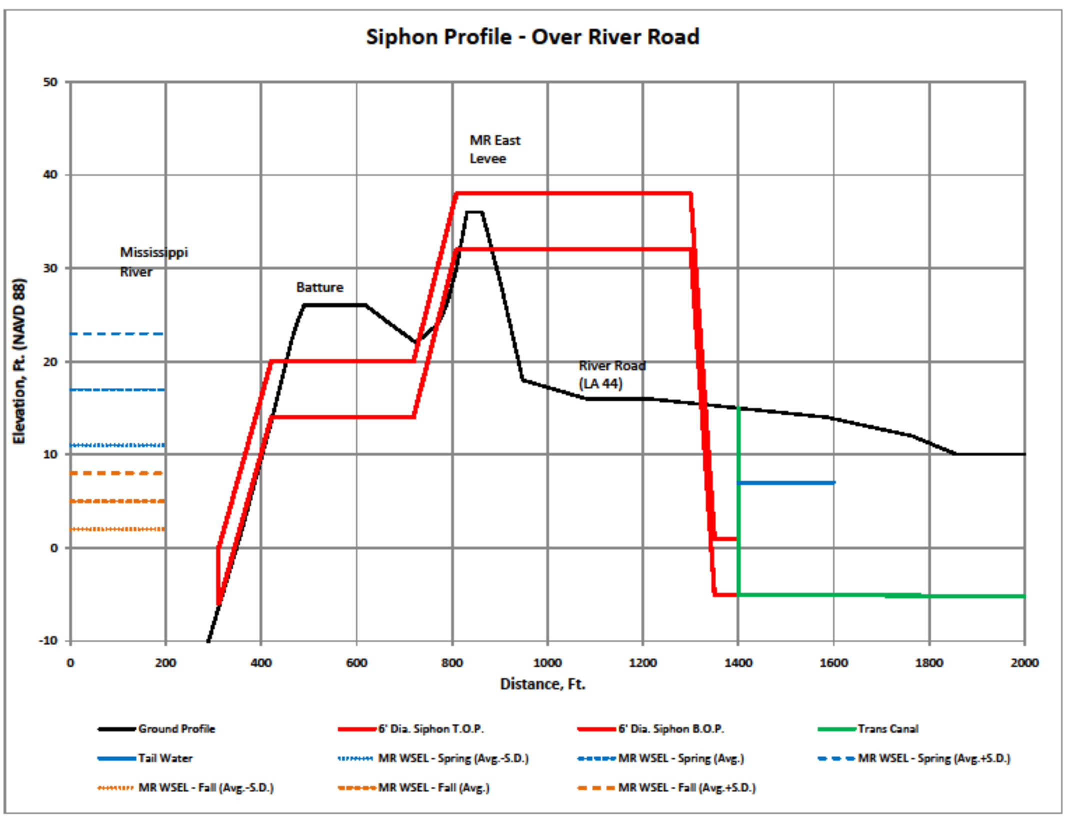



**Figure L2.6.5-2 Siphon Profile (Over River Road)**

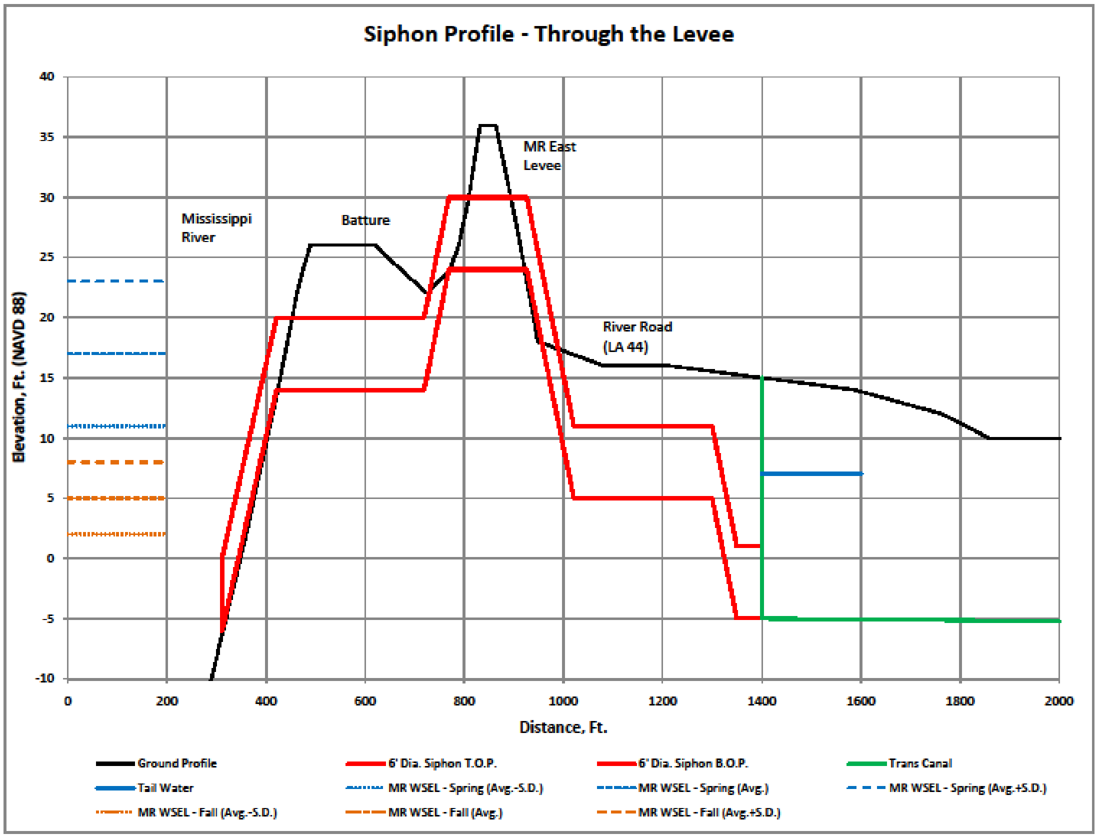



**Figure L2.6.5-3 Siphon Profile (Through the Mississippi River Levee)**

**Table L2.6.5-1 Siphon Profiles versus Minimum Operating WSEL in Mississippi River**

| Siphon Profile at Levee | Pipe Size, ft | B.O.P. at Levee | T.O.P. at Levee | Minimum Operating WSEL in Mississippi River |
|---|---|---|---|---|
| Siphon Over Levee | 5 | 37 | 42 | 17 |
| | 6 | 37 | 43 | 18 |
| | 7 | 37 | 44 | 19 |
| | 8 | 37 | 45 | 20 |
| Siphon over River Road | 5 | 32 | 37 | 12 |
| | 6 | 32 | 38 | 13 |
| | 7 | 32 | 39 | 14 |
| | 8 | 32 | 40 | 15 |
| Siphon Through Levee | 5 | 25 | 30 | 5 |
| | 6 | 24 | 30 | 5 |
| | 7 | 23 | 30 | 5 |
| | 8 | 22 | 30 | 5 |

Notes:
1. Top of levee is approximately Elev. 36.
2. Use 25 feet as the maximum practical siphon lift for large diameter pipe.
3. Calculate the siphon lift from the top of pipe (not centerline).
4. B.O.P = Bottom of Pipe.
5. T.O.P. = Top of Pipe.

Analysis of the alternate siphon profiles:

- Routing the siphon over the levee will require high stages in the Mississippi River for operation. This will severely limit the potential diversion periods, and practically eliminate the possibilities of base flow diversions during the summer and fall periods when the Mississippi River stage is low.
- The option over the levee will block the existing access road on top of the levee. Other accommodations may need to be incorporated into the site design to provide access to the top of the levee.
- The option over the River Road is also relatively high, and will limit the diversion period and ability to operate during low stages in the Mississippi River. This option also blocks access along the top of the levee.
- The options at lower elevations will penetrate the existing flood control levee, and may require a cut-off wall, filter diaphragms, and other seepage control measures, designed to USACE requirements, to protect the integrity of the levee.

The recommended siphon design is based on routing the siphon through the levee with the top-of-pipe at Elev. 30, as shown on Figure L2.6.5-3, in order to divert flows down to Elev. 5 in the Mississippi River. The following hydraulic calculations are based on this profile.

**Siphon Configuration**

The size and number of the siphon pipes ("barrels") has a wide range of possibilities to accommodate the large range of potential design flow rates. The State of Louisiana has two large siphon diversion installations in operation, Naomi and West Pointe a la Hache. Both of these installations have 8 – 72" diameter barrels, which are routed over the top of the levee. The initial Blind River siphon designs used the same concept of multiple barrels, with the following constraints:

- The installation should have a minimum of 3 barrels to allow a degree of flow control by taking one or more barrels out of operation.
- There is no limit to the number of barrels, although more than 12 to 15 barrels will result in large site.
- No stand-by barrels or excess capacity are included, as the installation is non-critical for health and safety, such as would be the case for water supply, wastewater, or flood control pumps.
- Pipe sizes of 60" through 96" were considered to cover the large range of potential design flows.

**Hydraulic Calculations**

An Excel spreadsheet was developed for the siphon hydraulic calculations, and is available electronically. The hydraulic design calculations are set up to readily change input data and assumptions, as the project design progresses.

The following items were considered to be included in the siphon installation:

- Grate/bar screen – located at the inlet end in the Mississippi River to block debris from entering the siphons. The initial concept is to provide a coarse bar screen to block large debris (tree limbs, etc.) At this point in the design development and in the environmental analysis process, fine screens (i.e. to block fish passage) are not required.
- Control valve – to control flow rates. See later discussion on potential needs for a throttling valve.
- Shut-off valve – to provide positive shut-off. The proposed siphon profile penetrates the Mississippi River levee and special measures may be required to maintain the levee flood protection integrity, as the pipe will be partially below maximum flood stages in the river. Therefore, it is considered that an isolation valve will be required by the regulatory authorities to provide redundancy in the event either the control valve or the shut-off valve gets damaged or blocked by debris.
- Pipe bends – consider mitered bends, typically 22.5 degrees or less.

The headlosses through the siphon structure were calculated, as follows:

- Pipe Friction Losses – The Hazen-Williams equation was used to calculate the headlosses in the siphon pipe. A friction factor of C = 110 was used to represent moderately aged, but not old steel pipe with an appropriate protective coating.
- Miscellaneous Losses – The velocity head method was used to calculate the headlosses for the valves, fittings, and appurtenances. The Excel design file includes a table of loss coefficients for each pipe size considered.
- Entrance and Exit Losses – Typical entrance and exit loss coefficients were obtained from hydraulic manuals. The inlet will be protruding into the Mississippi River. The outlet is considered to be flush with a concrete headwall at the discharge into the transmission canal.
- Bar Screens –The standard bar screen headloss equation, found in the USACE EM 1110-3-172 (May 11, 1984), page 7-6, was used for the headloss calculations. The initial design assumes 1” wide bars on a 12”x12” grid, and debris blocking 10% of the bar screen opening. A typical blockage would likely be higher on individual pipes; however, the analysis assumes that not all intakes would be blocked at the same time. The bar screen characteristics can be varied in the Excel design file, as design decisions are made on the bar width, grid spacing, and level of blockage.

The driving head for the siphon is the Mississippi River WSEL minus the tail water elevation in the transmission canal, as shown on Figure L2.6.2-1, the HGL profile. As the Mississippi River stage increases, the flow rate will increase, and the tail water elevation will increase. The design spreadsheet incorporates a tail water rating curve from the HEC-RAS model for the transmission canal, based on the design for a 1,000 cfs flow rate. The maximum possible tail water was set at Elev. 8

at the siphon outlet. For single barrel operation, the flow rates in the siphon and the canal are the same. For multi-barrel operation, the siphon losses are based on the flows in one barrel, but the transmission canal tail water WSEL is based on the total flow.

Due to the high potential head differential from the Mississippi River to the transmission canal, the siphon has high potential velocities. A control valve, or throttling valve, may be required to limit velocities when the Mississippi River stages are high.

**Siphon Design**

The siphon will operate in an environment with changing head and flow rates, as the Mississippi River stage varies through the year. As the River stage varies, more, or less siphon barrels will need to be in service to provide the design flow rate. The siphon design file contains a second set of spreadsheets to determine the number of siphon barrels for a given design flow rate, pipe size, and Mississippi River stage. These spreadsheets were then used to select the size and number of barrels required for each design flow rate at a given River stage.

**Design Stage in the Mississippi River**

An analysis was performed to evaluate and recommend a single design stage for the Mississippi River as the basis for the siphon hydraulic design. The operating characteristics were then reviewed for other elevations to assist in the evaluation. This analysis considered Elev. 11 and 17 as the optional design WSEL's. Additional analyses were then done to determine flow rate at lower elevations. Of interest are flow rates in the summer period at low River stages, and the capability to divert a base flow to the Blind River down to a stage of Elev. 5 in the Mississippi River.

- Elev. 11 Design WSEL
- **Table L2.6.5-2** has the number of siphon barrels required for 60", 72", 84", and 96" diameter pipes for the full range of design flows with a Mississippi River design stage at Elev. 11. This design will also provide the full design flow rate at Mississippi River stages above Elev. 11, with a reduced number of siphon barrels in operation. For a Mississippi River stage at Elev. 5, the design will provide approximately 40% of the design flow rate.
- Elev. 17 Design WSEL
- **Table L2.6.5-3** has the number of siphon barrels required for 60", 72", 84", and 96" diameter pipes for the full range of design flows with a Mississippi River design stage at Elev. 17. This design will also provide the full design flow rate at Mississippi River stages above Elev. 17, with a reduced number of siphon barrels in operation. For a Mississippi River stage at Elev. 11, the design will provide approximately 70% of the design flow. At a stage of Elev. 5, the design will provide approximately 30% of the design flow rate.

**Table L2.6.5-2 Siphon Barrels Required versus Design Flow Rate (Mississippi River WSEL = 11 Ft)**

| Design Flow Rate, cfs | No. of Siphon Barrels Required at Mississippi River WSEL = 11 ft | | | |
|---|---|---|---|---|
| | 60" | 72" | 84" | 96" |
| 500 | 4 | 3 | 2 | 1 |
| 1,000 | 9 | 6 | 4 | 3 |
| 1,500 | 13 | 9 | 6 | 5 |
| 2,000 | 17 | 11 | 8 | 6 |
| 2,500 | 22 | 14 | 10 | 8 |
| 3,000 | 26 | 17 | 12 | 9 |
| 3,500 | 30 | 20 | 14 | 11 |
| 4,000 | 35 | 23 | 16 | 12 |
| 4,500 | 39 | 26 | 18 | 14 |
| 5,000 | 43 | 28 | 20 | 15 |

**Table L2.6.5-3 Siphon Barrels Required versus Design Flow Rate (Mississippi River WSEL = 17 Ft)**

| Design Flow Rate, cfs | No. of Siphon Barrels Required at Mississippi River WSEL = 17 ft | | | |
|---|---|---|---|---|
| | 60" | 72" | 84" | 96" |
| 500 | 3 | 2 | 1 | 1 |
| 1,000 | 6 | 4 | 3 | 2 |
| 1,500 | 8 | 6 | 4 | 3 |
| 2,000 | 11 | 7 | 5 | 4 |
| 2,500 | 14 | 9 | 7 | 5 |
| 3,000 | 17 | 11 | 8 | 6 |
| 3,500 | 19 | 13 | 9 | 7 |
| 4,000 | 22 | 15 | 10 | 8 |
| 4,500 | 25 | 17 | 12 | 9 |
| 5,000 | 28 | 18 | 13 | 10 |

The siphon hydraulic performance is very similar to the diversion culverts. Based on a comparison of the hydraulic capacity of the siphons, changing the design basis from Mississippi River Elev. 17 to Elev. 11 results in a 50% increase in siphon size.

We recommend designing the diversion to deliver the design flow rate at Mississippi River stage Elev. 11, as this provides a higher flow rate potential during summer periods.

**Operation through Typical Annual Cycles**

The daily stage statistics from the Mississippi River Stage Analysis were used to demonstrate the operation of the siphon diversion structure through an annual cycle of varying stages. A 72" siphon designed for 1,000 cfs was used for the example. The three statistical daily stages shown on Figure L2.6.1-1 were used for the analysis: the average stage, the average minus the standard deviation, and the average plus the standard deviation. **Figure L2.6.5-4** shows the number of siphon barrels in operation, and **Figure L2.6.5-5** illustrates the daily flow rate (cfs).

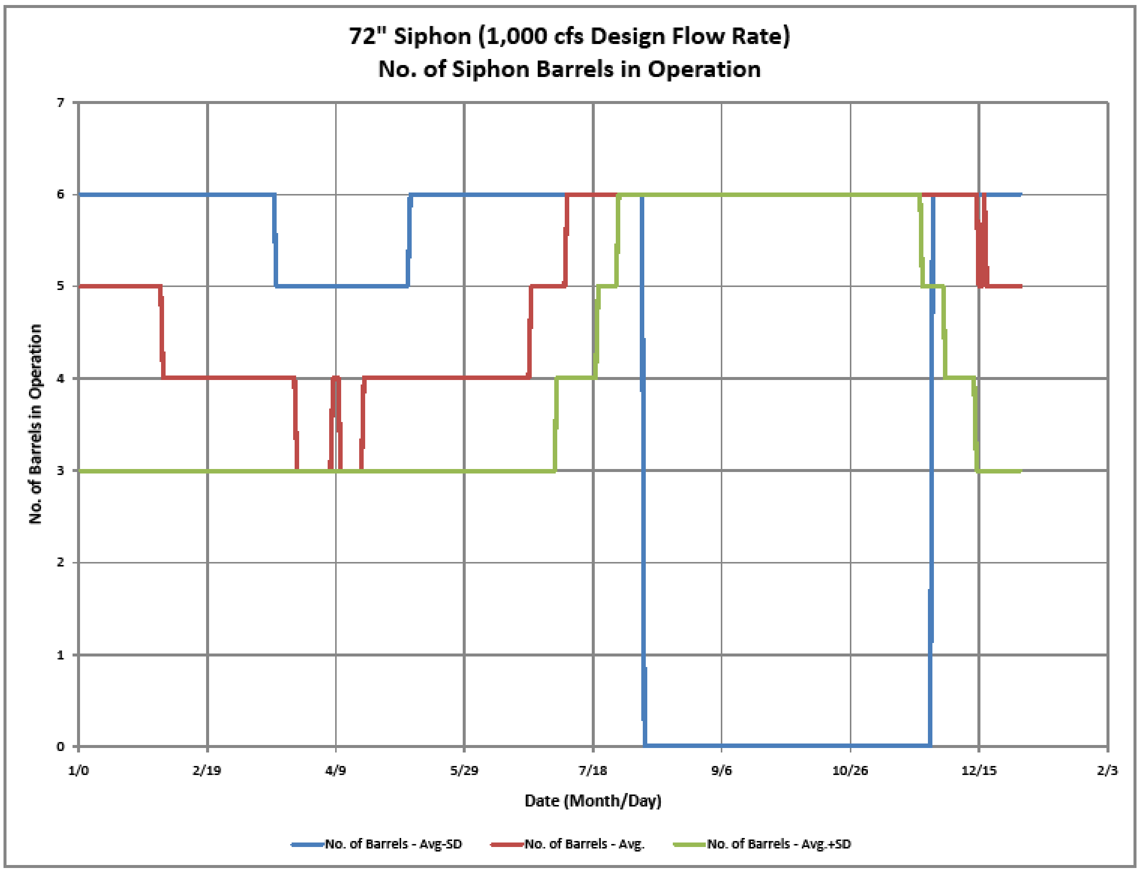



**Figure L2.6.5-4 Summary of 72" Siphon Barrels in Operation (1,000 cfs Design Flow Rate)**

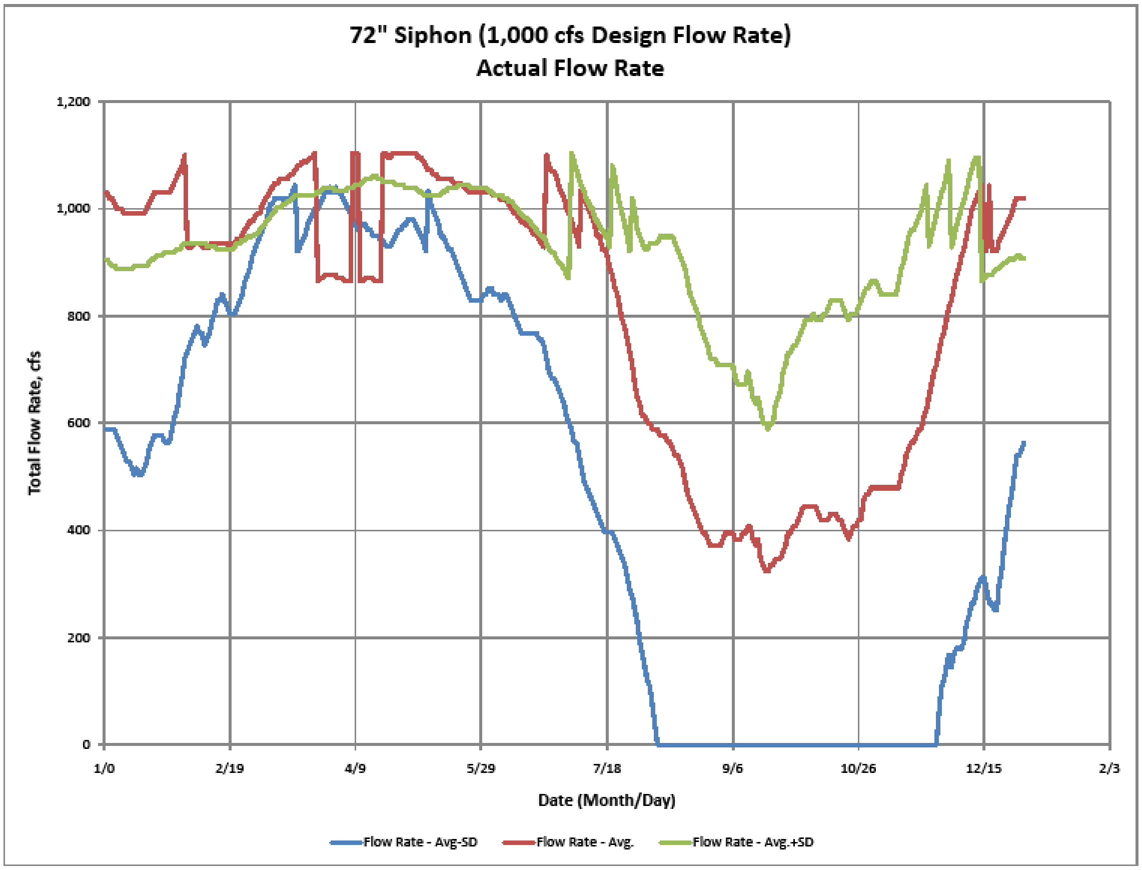



**Figure L2.6.5-5 Actual Flow Rate of 72" Siphon (1,000 cfs Design Flow Rate)**

**Summary and Recommendations**

**Table L2.6.5-4** has the recommended siphon size and number of barrels for each potential design flow rate.

**Table L2.6.5-4 Recommended Siphon Configurations for Alternate Design Flow Rates**

| Design Flow Rate, cfs | Siphon Size, in | No. of Barrels |
|---|---|---|
| 500 | 60 | 4 |
| 1,000 | 72 | 6 |
| 1,500 | 72 | 9 |
| 2,000 | 84 | 8 |
| 2,500 | 84 | 10 |
| 3,000 | 84 | 12 |
| 3,500 | 96 | 11 |
| 4,000 | 96 | 12 |
| 4,500 | 96 | 14 |
| 5,000 | 96 | 15 |

The primary design basis for the recommended configurations is as follows:

- Design water surface in the Mississippi River is Elev. 11; and
- Design tail water is Elev. 7.

### L2.6.6 Romeville Transmission Canal Analysis

The transmission canal layout for the Tentatively Selected Plan is included in **Annex L-5**. The transmission canal will transfer the diverted water from the diversion facility to the existing drainage channels along the south and west perimeter of the Swamp. The alignment is approximately 15,300 feet long, and will cross one road (LA 3125) and one railroad (Canadian National Railroad). The canal will have the following features:

- Stilling basin at head of the canal (hydraulic design addressed elsewhere);
- Earthen canal with a flat bottom and 4:1 side slopes (H:V);
- Earthen berms – the canal design HGL will be above ground for most of the alignment, requiring earthen berms on both sides;
- Railroad crossing – reinforced concrete box culverts; and
- LA 3125 crossing – reinforced concrete box culverts.

**Design Basis**

The canal design is based on the following:

- See Section L2.6-2 and Figure L2.6.2-1 for the hydraulic grade line.
- See Section L2.6-3 and Figure L2.6.2-1 for the flow line profile.
- At the design flow rate, the HGL will be Elev. 4.0 at the downstream end and Elev. 6.0 at the upstream end.
- The proposed flow line will be from Elev. -4.5 at the downstream end to Elev. -3.0 at the upstream end.
- Side slopes are 4:1 (H:V) to be conservative. The geotechnical investigation may allow steeper side slopes.
- Erosion protection – the design velocities are low, and erosion potential is minimal. Concrete channel lining and riprap will be used at the upstream and downstream sides of the culverts, and at the outfall into the existing Parish drainage channel.
- Manning's n value – 0.035 for a well-maintained vegetative lined or earthen canal.
- Design for steady-state flow.
- Freeboard – The conceptual civil design and construction estimate were based on a 3-foot freeboard at the design flow rate.

- Berms – 12-foot wide top width to allow maintenance vehicle access, 4:1 side slopes (interior), and 4:1 or 5:1 exterior side slopes for mowing safety.
- Right-of-way width –
- Without berms – minimum of 30 feet each side for access by large maintenance equipment and for drainage
- With berms – minimum of 10 feet beyond the outer toe of berm on each side, for a local drainage swale and mowing access
- ROW drainage – provide a small drainage swale at the ROW line and discharge to local drainage.

Options considered:

- Match flow line at existing outfall drainage channels (Elev. -4.5);
- Desilt the existing outfall drainage channel 2 feet and lower the transmission canal downstream end flow line to Elev. -6.5. This should be reviewed during final design; and
- Concrete-lined channel. Preliminary cost comparisons indicate that a concrete-lined canal will increase costs; however, this option should be reviewed during final design.

The range of flow rates being considered for the Romeville diversion point is from 500 cfs to 25,000 cfs, or higher. The preliminary design analysis for the transmission canal covers a range of flows from 500 cfs to 20,000 cfs.

The system HGL provides a 2-foot differential for the transmission canal from the upstream end to the downstream end.

**HEC-RAS Model for Canal Design**
A HEC-RAS model was developed to analyze the transmission canal hydraulics. The cross sections were cut from the 2001 LiDAR-based DEM obtained from the State of Louisiana. The DEM has gaps and cells with no elevation data. CDM adjusted the cross section data at such locations, based on best available data and engineering judgment.

The headlosses for the canal were calculated, as follows:

- Manning's n value – 0.035 for earthen channels and 0.015 for concrete channels and the concrete box culverts;
- Expansion and Contraction Losses – typical values were obtained from the HEC-RAS manual; and
- Entrance and Exit Losses – typical entrance and exit loss coefficients were obtained from the HEC-RAS manual. All culverts are considered to have concrete headwalls.

To develop a canal section for each design flow rate, the canal bottom width was adjusted to obtain a water surface profile meeting the HGL criteria noted above. The flow line profile and berm elevations remained the same for all design flow rates.

For each design, a series of flow rates was used in the design analysis to determine the operating characteristics of the canal through the expected flow rates. Starting water surface elevations were linearly interpolated, matching the HGL design basis stated above.

- Elev. 2.0 – With no flow in the system, the starting WSEL is the same as the static WSEL in the Swamp;
- Elev. 4.0 – Design flow rate; and
- Elev. 5.5 – Assumed maximum WSEL in the Swamp, under a high diversion flow rate.

Based on the HEC-RAS analysis, the recommended channel design sections are included in **Table L2.6.6-1**. The right-of-way widths vary throughout the length of the canal, as natural ground elevations vary. The table below uses the maximum width. Actual right-of-way acquisitions could be less in certain reaches.

**Table L2.6.6-1 Recommended Channel Design Sections**

| Diversion Design Flow Rate, cfs | Bottom Width, ft | Proposed ROW Width, ft |
|---|---|---|
| 500 | 12 | 170 |
| 1,000 | 40 | 195 |
| 1,500 | 70 | 230 |
| 2,000 | 100 | 260 |
| 2,500 | 125 | 290 |
| 3,000 | 155 | 315 |
| 3,500 | 185 | 345 |
| 4,000 | 215 | 375 |
| 4,500 | 240 | 405 |
| 5,000 | 270 | 430 |
| 10,000 | 555 | 710 |
| 15,000 | 840 | 1,000 |
| 20,000 | 1,125 | 1,285 |

**Freeboard and Excess Capacity**

The diversion structure will operate with a varying driving head, varying diversion structure capacity in service, and possible with variable control, all resulting in the likelihood that flow rates will not be finely controlled to the design flow rate. Therefore, the transmission canal needs excess capacity to avoid overtopping the berms. With the 3-foot freeboard design, the canal has the excess capacity as shown in **Table L2.6.6-2**. For the 3,000 cfs transmission canal design with berms set to provide a 3-foot freeboard, the channel could be overtopped at 5,100 cfs, or 1.7 times the diversion design flow rate.

**Table L2.6.6-2 Freeboard Summary**

| Flow Rate, cfs | Design Flow Rate Multiple | Freeboard Reduction, ft | Freeboard Remaining, ft | Comments |
|---|---|---|---|---|
| 3,000 | 1.0 | 0 | 3 | Design diversion flow rate |
| 3,900 | 1.3 | 1 | 2 | - |
| 4,500 | 1.5 | 2 | 1 | - |
| 5,100 | 1.7 | 3 | 0 | Berms overtopped |

The rating curves (flow rate vs. WSEL) for the 1,500 cfs and 3,000 cfs designs are on **Figure L2.6.6-1**.

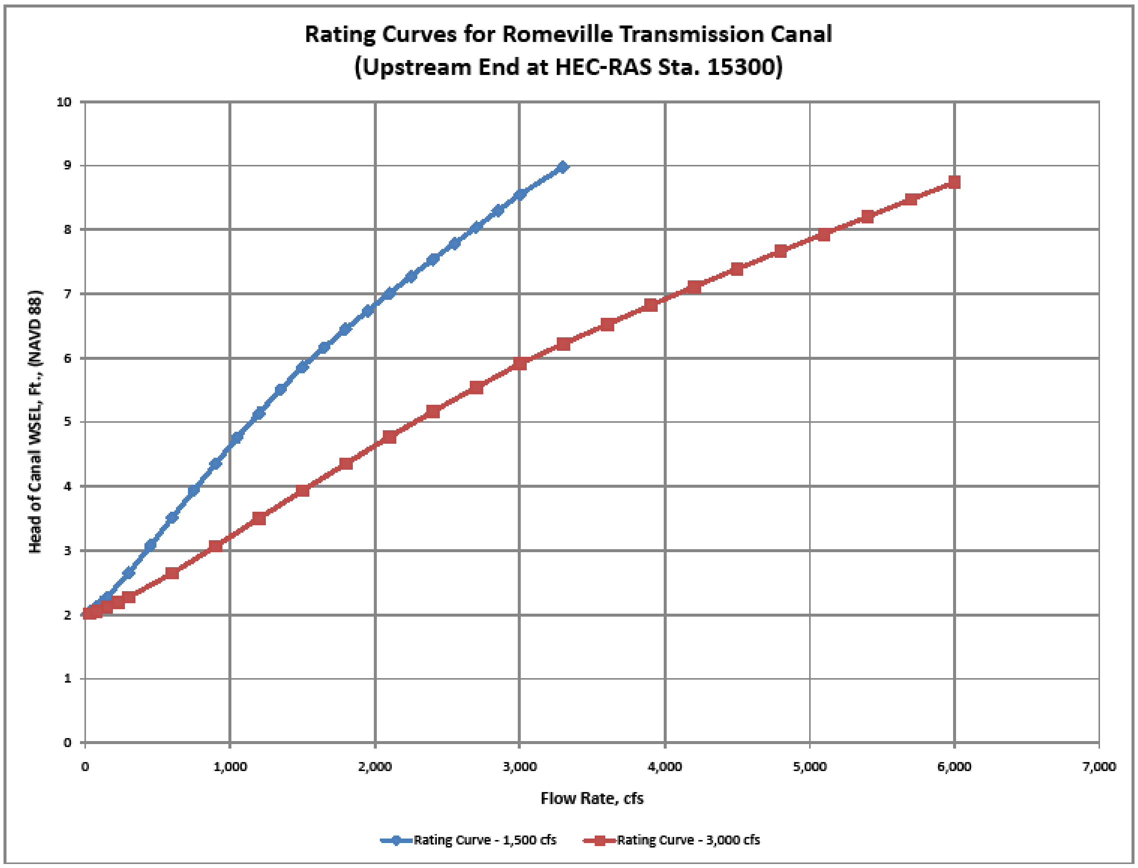



**Figure L2.6.6-1 Romeville Transmission Canal Rating Curves**

The velocities in the canal are relatively low, due to the restrictions on the HGL. These will need to be reviewed in the next design phase, as the Mississippi River sediment data becomes available. **Figure L2.6.6-2** has the velocity plots for the 1,500 cfs and 3,000 cfs designs.

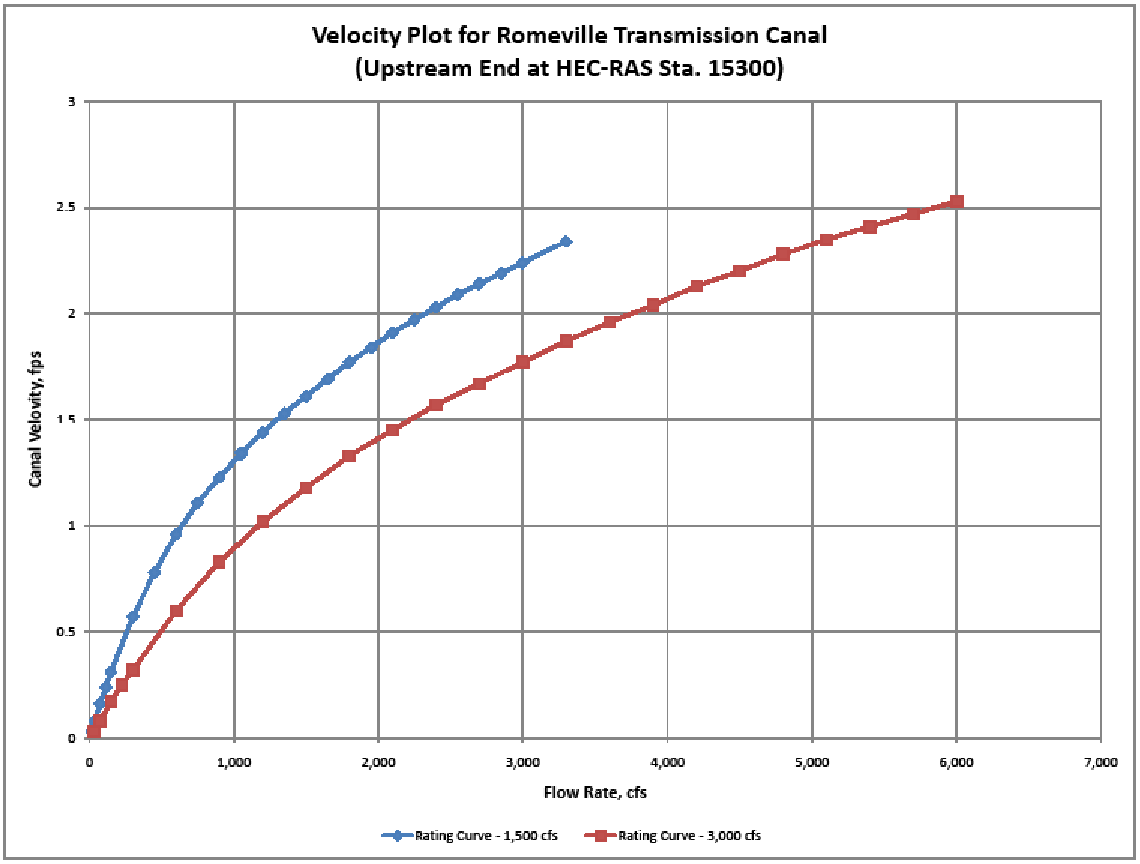



**Figure L2.6.6-2 Romeville Transmission Canal Velocity Plots**

**Road and Railroad Crossings**

For the initial transmission canal design, culverts were placed into the HEC-RAS model at the LA 3125 and CN RR crossings for only the 1,000 cfs design. That analysis showed that minimal headlosses in the order of 0.2 feet occur with velocities of approximately 4 fps through the culverts. Culvert designs were omitted from all remaining canal geometry design files, on the assumption that reasonable culvert designs are possible to have low head losses. Reinforced concrete box culverts will be used to cross the existing transportation facilities for the lower design flow rates. Bridges could be used for the higher design flow rates. For preliminary design and costs, culverts were sized for 4 fps. As noted in **Table L2.6.6-3**, the water depths are less than 9 feet; therefore, the box culverts will have a maximum height of 8 feet to fully utilize conveyance capacity at the design flow rate.

**Table L2.6.6-3 Culvert Data at Transportation Crossings**

| Item | CN RR Crossing | LA 3125 Crossing |
|---|---|---|
| HEC-RAS Station | 13760 | 5600 |
| Road/Track Elevation | 12 | 8 |
| Natural Ground in Area | 6 - 10 | 6 |
| Proposed Flow Line Elevation | -3.13 | -3.95 |
| Proposed Design WSEL | 5.8 | 4.73 |
| Depth of Water | 8.93 | 8.68 |

All elevations and dimensions in feet.

Based on these criteria, the initial culvert sizes for both the CN RR and the LA 3125 crossings are the same size, as summarized in **Table L2.6.6-4**.

**Table L2.6.6-4 Proposed Culvert Sizes at Transportation Crossings**

| Design Flow Rate, cfs | Required Area at 4 fps, SF | Recommended Size, ft (WxH) | Recommended Area, SF |
|---|---|---|---|
| 500 | 125 | 2 – 8 x 8 | 128 |
| 1,000 | 250 | 3 – 11 x 8 | 264 |
| 1,500 | 375 | 4 – 12 x 8 | 384 |
| 2,000 | 500 | 6 – 11 x 8 | 528 |
| 2,500 | 625 | 7 – 12 x 8 | 672 |
| 3,000 | 750 | 8 – 12 x 8 | 768 |
| 3,500 | 875 | 10 – 11 x 8 | 880 |
| 4,000 | 1,000 | 11 – 12 x 8 | 1,056 |
| 4,500 | 1,125 | 12 – 12 x 8 | 1,152 |
| 5,000 | 1,250 | 13 – 12 x 8 | 1,248 |
| 10,000 | 2,500 | 26 – 12 x 8 | 2,496 |
| 15,000 | 3,750 | 39 – 12 x 8 | 3,744 |
| 20,000 | 5,000 | 52 – 12 x 8 | 4,992 |

## L2.7 Hydraulic Analysis of South Bridge Diversion and Transmission Components

The Diversion project requires several different types of management measures, or components, serving different functions, which will be combined to form the alternative plans. This section presents the preliminary hydraulic analysis and design of the diversion and transmission components for the South Bridge alignment. The analysis of the South Bridge alignment in this section includes a distribution canal that is routed across the northern portion of the proposed target area in the Maurepas Swamp. The diversion and transmission components were combined in this section as the design of both is based on the diversion flow rate, and both have a common hydraulic grade line.

The analysis addressed the full range of potential flow rates, management measures, and alternative plans considered as the project developed and the evaluation screening occurred. The initial array of alternatives considered diversion flow rates from 500 cfs to 20,000 cfs, and the preliminary hydraulic designs were prepared for this full range of flows. The specific flow rates being used in the initial alternative arrays are in 500 cfs increments from 500 cfs up to 5,000 cfs, then in 5,000 cfs increments to 20,000 cfs.

The proposed South Bridge diversion point is located on the east bank of the Mississippi River near Mile 167.0 (2004 Hydrographic Survey), as shown on **Figure L2.7-1** and **Figure L2.7-2**.

The proposed Blind River Diversion Project is on the NAVD 88 vertical datum. Other major topographic datasets being used on the project are also on the NAVD 88 vertical datum, including the 2001 LiDAR data and the 2004 Hydrographic Survey.

### L2.7.1 Development of the South Bridge Diversion Alignment

As the Blind River diversion project developed, multiple diversion alignments were considered to divert the flow from the Mississippi River and transfer it to the Swamp. The Romeville diversion alignment will transfer water to the existing perimeter drainage channels along the south and west perimeter of the Swamp. The diverted water will then be forced into the Swamp by control structures in the existing channels, flow overland and discharge into the Blind River. Without transfer canals, pipe lines, inverted siphons, or other means such as pump stations, the diverted water cannot be moved across the Blind River and be applied to approximately half of the targeted service area on the north and east side on Blind River. Such facilities would be expensive and highly disruptive to the environmental conditions in the Swamp. Therefore, the Romeville alignment primarily serves the Series 100 HUs, as shown on Figure L2.6-1, the layout for Alternative 2.

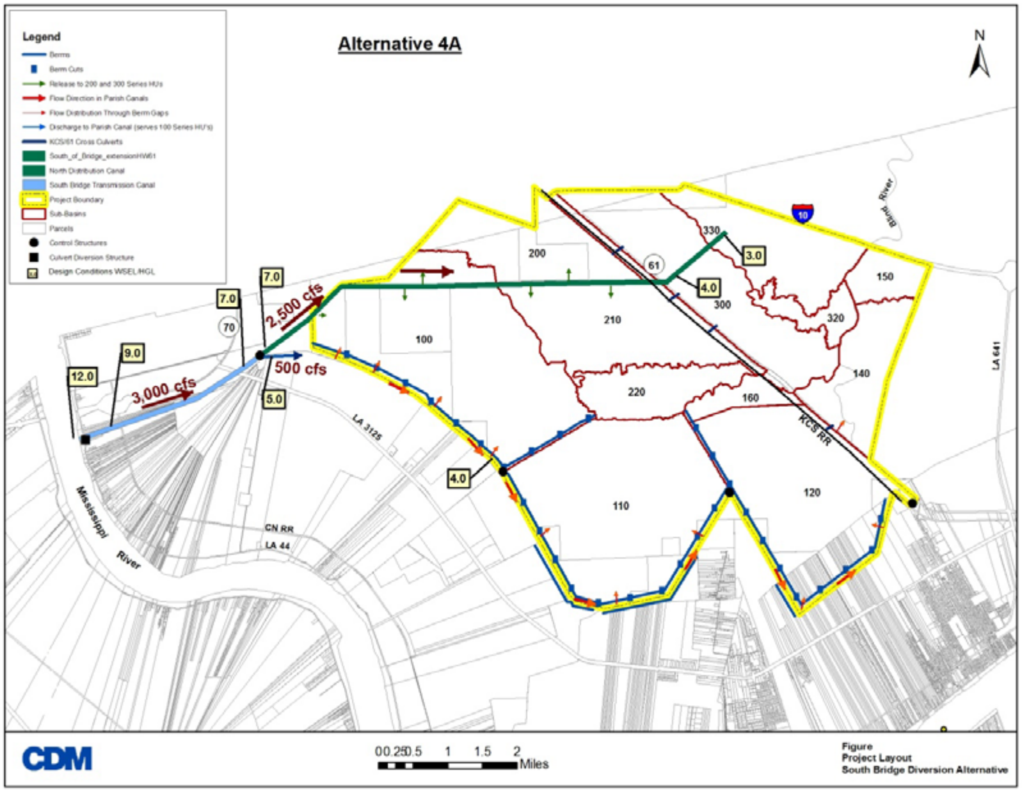



**Figure L2.7-1 South Bridge Diversion Project Layout (Alternative 4A)**

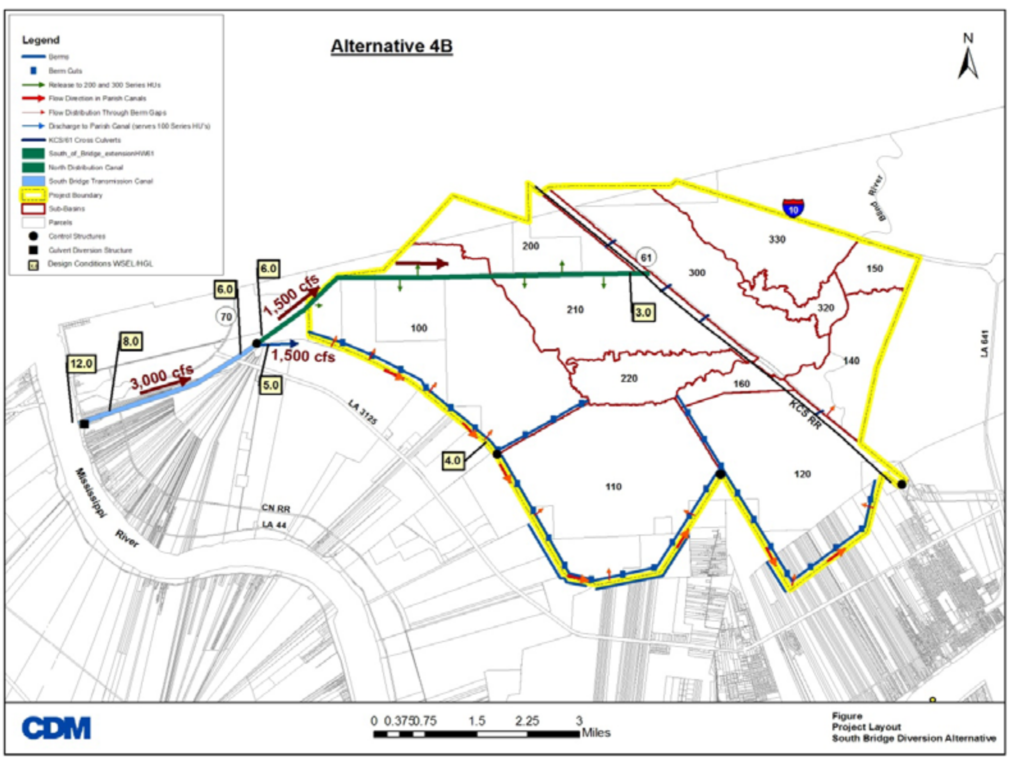



**Figure L2.7-2 South Bridge Diversion Project Layout (Alternative 4B)**

Several alignments were reviewed to divert water into the upstream area of the targeted Swamp to expand the potential area of influence to include the area north and east of the Blind River, the Series 200 and 300 Hydrologic Units (HUs). The intent was to divert the water into the northwest part of the targeted service area, and then transfer the water across the Blind River headwaters to the north and east side of the Blind River. Diversion alignments on both the north and south sides of Highway 70 at the Sunshine Bridge were initially considered. The South Bridge alignment, the most direct and shortest route, was selected for the more detailed analysis. Figure L2.7-1 and Figure L2.7-2 for Alternatives 4A and 4B illustrate this approach to expand the influence area.

The optional transmission canal alignments in the area of the Sunshine Bridge terminate near a common point, at an existing Parish drainage channel near the far west corner of the proposed Swamp service area. At that point, the flow will be split at the downstream end of the transmission canal, as follows:

- A portion of the flow will be discharged into the existing Parish drainage channel. This water will then be forced into the Swamp with control structures in the existing Parish drainage channels, with locations, designs, and functions very similar to the ones for the Romeville alignment. This portion of the flow will primarily go to the 100 Series HU's.
- The remainder of the flow will continue east in a canal tentatively identified as the North Distribution Canal. The flow would go to part of Hydrologic Unit 100 and to the 200 and 300 Series HUs. The canal will be designed for a decreasing flow rate from west to east, as water will be released into the Swamp along the entire alignment, as with an irrigation canal.

The flow split will be at a control structure with sluice gates controlling flow in both directions. The control structure will be located approximately 2,200 feet east of LA 3125 and 3,500 feet west of the far west corner of the project service area, as shown on the figures for Alternative 4A and Alternative 4B.

The South Bridge transmission canal could have the following optional service areas:

- 100 Series HUs - Discharge all flow into the existing Parish drainage channels
- 100 and 200 Series HUs – Split the flow, as described above. End the North Distribution Canal near the west side of the KCS RR.
- 100, 200, and 300 Series HUs – Split the flow, as described above. Extend the North Distribution Canal across the KCS RR and Highway 61 to serve the 300 Series HUs.

In the later phases of the feasibility study, the array of alternative plans considered four diversion alignment options. In the final array of alternative plans, these were identified as follows:

- Alternative 2 – 1,500 or 3,000 cfs diversion at the Romeville Alignment near Mississippi River Mile 162.0
- Alternative 4 – 1,500 or 3,000 cfs diversion to the South Bridge Alignment near Mississippi River Mile 167.0
    - Alternative 4A – 500 cfs to the existing Parish drainage channel and 2,500 cfs to the North Distribution Canal (Figure L2.7-1)
    - Alternative 4B – 1,500 cfs to the existing Parish drainage channel and 1,500 cfs to the North Distribution Canal (Figure L2.7-2)
- Alternative 6 – Dual Alignment, consisting of a total 3,000 cfs diversion, using a 50/50 flow split between the Romeville Alignment and the South Bridge Alignment

### L2.7.2 Mississippi River Stage Analysis at River Mile 167.0

A Mississippi River stage analysis was prepared for the South Bridge diversion point at River Mile 167.0. This analysis used the same source data and procedures described for the Romeville stage analysis in Section L2.6.2. The statistical results are very similar to the Romeville values, but are slightly higher, as shown in **Table L2.7.2-1** and **Table L2.7.2-2**.

**Table L2.7.2-1 Standard Statistics for Stage**

| Period | Standard Deviation | Avg. - 1 SD (84.14% Exceedance) | Average (50% Exceedance) | Avg. + 1 SD (15.86% Exceedance) |
|---|---|---|---|---|
| Full Year | 7.33 | 4.48 | 11.81 | 19.14 |
| Spring (Mar. 1 – May 31) | 6.24 | 11.63 | 17.87 | 24.11 |
| Summer-Fall (Aug. 16 – Nov. 15) | 3.23 | 2.11 | 5.34 | 8.57 |

**Table L2.7.2-2 Stage vs. Percent Exceedance**

| Period | Avg. - 1 SD (84.14% Exceedance) | Average (50% Exceedance) | Avg. + 1 SD (15.86% Exceedance) |
|---|---|---|---|
| Full Year | 4.01 | 10.25 | 20.42 |
| Spring (Mar. 1 – May 31) | 10.68 | 18.54 | 24.50 |
| Summer-Fall (Aug. 16 – Nov. 15) | 3.11 | 4.34 | 7.40 |

**Approximate Values for Design**

Based on the statistical analyses, we recommend using the following approximate values in **Table L2.7.2-3** for design purposes:

**Table L2.7.2-3 Summary Stage Statistical Values**

| Period | Avg. - 1 SD (84.14% Exceedance) | Average (50% Exceedance) | Avg. + 1 SD (15.86% Exceedance) |
|---|---|---|---|
| Full Year | 4 | 11 | 20 |
| Spring (Mar. 1 – May 31) | 11 | 18 | 24 |
| Summer-Fall (Aug. 16 – Nov. 15) | 3 | 5 | 8 |

### L2.7.3 South Bridge Hydraulic Grade Line

The diversion facility and the transmission canals are part of an overall hydraulic system being designed to divert water from the Mississippi River, transfer it to the Maurepas Swamp, distribute it within the Swamp, and drain it to the Blind River. The South Bridge alignment is different from the Romeville alignment in that the flow will be split and discharged to the existing St. James Parish drainage channel, and to an extension of the transmission canal named the North Distribution Canal, as shown on Figure L2.7-1 and Figure L2.7-2.

The hydraulic head available from the Mississippi River is the driving force for flow of the diverted water through the entire system. The principal hydraulic elements and segments of the overall system are:

- Mississippi River stage – the upstream boundary condition for the hydraulic grade line;
- Diversion Structure – to divert the flow over or through the east levee of the Mississippi River;
- Transmission Canal – to transfer the flow from the diversion structure to the edge of the Swamp;
- North Distribution Canal – to transfer flow across the Blind River headwaters and distribute the flow to the hydrological units east of the Blind River;
- Distribution System – to distribute flow into the Swamp;
- Overland Flow and Drainage System – to direct the diverted water through the Swamp, and then to drain it from the Swamp to the Blind River; and
- Blind River stage – the downstream boundary condition for the hydraulic grade line.

The plan to transfer the water long distances across the north portion of the project area and then release it into the Swamp results in a need for a higher HGL with a flatter slope than the Romeville diversion alignment. Based on conceptual and preliminary design analyses, the following water surface elevations are being used for the hydraulic design of the system components for the South Bridge alignment. These values may be revised as the hydro-dynamic modeling progresses and as the designs for the various components of the project progress.

**North Distribution Canal – Alternative 4A**
The canal service area for Alternative 4A is both west of the KCS RR and east of Highway 61.

- Elev. 1.5 to Elev. 2 – The approximate static WSEL throughout the Swamp, the drainage system, and Blind River between storm events.
- Elev. 3 – The minimum WSEL in the North Distribution Canal for release into the Swamp, providing a 1-foot driving head through the Swamp. This results in a minimum WSEL of Elev. 3 at the downstream (east) end, or terminal end, of the canal.
- Elev. 4 – The minimum WSEL in the North Distribution Canal immediately east of Highway 61, to allow for an extension of the North Distribution Canal further to the east to serve the 300 Series hydrologic units (HU's).
- Elev. 7 – The minimum WSEL at the upstream end of the North Distribution Canal, resulting in 3 feet of head driving the North Distribution Canal system from the end of the transmission canal to the east side of Highway 61.

**North Distribution Canal – Alternative 4B**
The canal service area for Alternative 4B is west of the KCS RR.

- Elev. 1.5 to Elev. 2 – The approximate static WSEL throughout the Swamp, the drainage system, and Blind River between storm events.
- Elev. 3 – The minimum water surface elevation (WSEL) in the North Distribution Canal for release into the Swamp, providing a 1-foot driving head through the Swamp. This results in a minimum WSEL of Elev. 3 at the downstream (east) end, or terminal end, of the canal near the west side of the KCS RR.
- Elev. 6 – The minimum WSEL at the upstream end of the North Distribution Canal, resulting in 3 feet of head driving the North Distribution Canal system from the transmission canal to the west side of the KCS RR.

**Existing Parish Drainage Channels**
- Elev. 1.5 to Elev. 2 – The approximate static WSEL throughout the Swamp, the drainage system, and Blind River between storm events.
- Elev. 4 – The proposed design WSEL in the existing drainage channels at the edge of the Swamp near the Romeville diversion alignment, providing a 2-foot driving head into and through the Swamp.
- Elev. 5 – The assumed WSEL in the existing drainage channel at the discharge point from the South Bridge transmission canal. This provides a 1-foot head differential to force flow south in the existing drainage channel.

**South Bridge Transmission Canal – Alternative 4A**
- Elev. 7 – The minimum WSEL at the downstream end of the transmission canal, based on the controlling WSEL in the North Distribution Canal for Alternative 4A.

- Elev. 9 – The approximate design WSEL at the head (upstream end) of the transmission canal, based on a 2-foot drop in the HGL for the canal.
- Elev. 9 – The proposed design tail water elevation at the diversion structure outlet. At this phase of the preliminary design analysis, a 1-foot drop in the HGL has not been provided from the diversion structure outlet to the head of the transmission canal to route flows through a potential water quality treatment basin or a settling basin.
- Elev. 12 – The recommended design stage in the Mississippi River. The development of this recommendation is documented in the hydraulic design analyses for the culvert and siphon diversion structures.

**South Bridge Transmission Canal – Alternative 4B**

- Elev. 6 – The minimum WSEL at the downstream end of the transmission canal, based on the controlling WSEL in the North Distribution Canal for Alternative 4B.
- Elev. 8 – The approximate design WSEL at the head (upstream end) of the transmission canal, based on a 2-foot drop in the HGL for the canal.
- Elev. 8 – The proposed design tail water elevation at the diversion structure outlet. At this phase of the design analysis, a 1-foot drop in the HGL has not been provided from the diversion structure outlet to the head of the transmission canal to route flows through a potential water quality treatment basin or a settling basin.
- Elev. 12 – The recommended design stage in the Mississippi River.

The controlling WSEL values listed above are shown on Figure L2.7-1 for Alternative 4A and on Figure L2.7-2 for Alternative 4B. The proposed HGL for both Alternative 4A and Alternative 4B values are also listed in **Table L2.7.3-1** and plotted on **Figure L2.7.3-1**.

The steady-state HGL described above is to establish a single hydraulic design basis to size the diversion structure and the transmission canal. During actual operations, the HGL will vary significantly as the following items change: Mississippi River stage, Lake Maurepas stage, Blind River stage conditions, control gate settings, and diversion flow rates. In recognizing the variability in flow rate and HGL, the diversion culvert analysis and siphon analysis determined flow rates at other HGL conditions to indicate facility performance under the varying conditions.

**Table L2.7.3-1 Hydraulic Grade Line Data for Alternatives 4A and 4B**

| HGL options for South Bridge Diversion Alignment | | | |
|---|---|---|---|
| | WSEL's for South Bridge Options | | |
| Description | (a) | Alt. 4B | Alt. 4A |
| | | | |
| **Diversion Structure and Transmission Canal** | | | |
| Design stage in Mississippi River | 12.0 | 12.0 | 12.0 |
| Tail water at downstream end of Diversion Structure | 7.0 | 8.0 | 9.0 |
| WSEL at upstream end of Transmission Canal | 7.0 | 8.0 | 9.0 |
| WSEL at downstream end of Transmission Canal | 5.0 | 6.0 | 7.0 |
| | | | |
| **North Distribution Canal** | | | |
| WSEL at upstream end on N. Distribution Canal | N/A | 6.0 | 7.0 |
| WSEL at west side of KCS RR | N/A | 3.0 | |
| WSEL at east side of Hwy 61 | N/A | N/A | 4.0 |
| WSEL at 300/330 | N/A | N/A | 3.0 |
| Static WSEL in Swamp | 2.0 | 2.0 | 2.0 |
| | | | |
| **Release to Existing Drainage Channel** | | | |
| Operating WSEL in channel at release point | 5.0 | 5.0 | 5.0 |
| Operating WSEL in Channel at control gates | 4.0 | 4.0 | 4.0 |
| Static WSEL in Swamp | 2.0 | 2.0 | 2.0 |

HGL Options:
(a) - Discharge to perimeter Parish drainage channel, only.
Alt. 4A - Serve Series 200 and 300 HU's, and discharge to perimeter Parish drainage channels
Alt. 4B - Serve Series 200 HU's, and discharge to perimeter Parish drainage channels

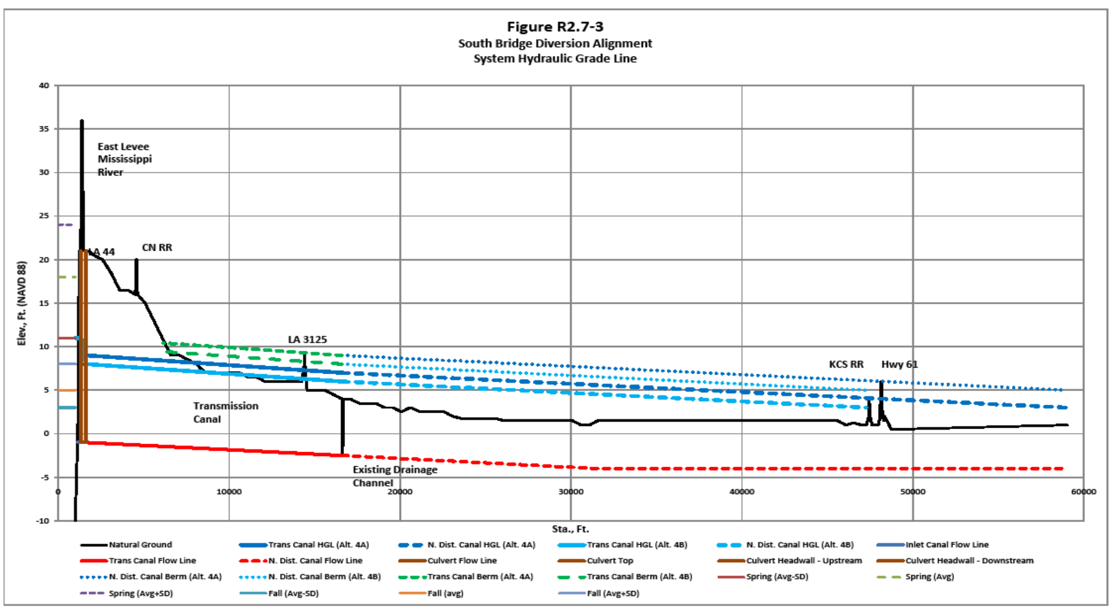



**Figure L2.7.3-1 System Hydraulic Grade Line (South Bridge Diversion)**

**Minimum Stage Limits for Diversions**
Section L2.6.2 previously provided a discussion of stage limits in the Mississippi River for diversion.

### L2.7.4 Diversion and Transmission System Flow Line Profile

The diversion facility and transmission canal flow line profile was established by existing physical conditions and the need to maximize depth at the Mississippi River to allow diversions during low stages in the river. The transmission canal will discharge into an existing St. James Parish drainage channel along the west perimeter of the Maurepas Swamp. Bathymetric flow line data was obtained for much of the existing perimeter drainage channels; however, elevations were not obtained at the South Bridge alignment. The nearest elevation is over one mile downstream. Based on that elevation, it was considered that the existing drainage channel flow line is Elev. -2.5 at the South Bridge alignment, and this elevation was used as the limiting depth for the transmission canal. The diversion culvert flow line was set at Elev. -1.0 at the levee to have a minimum of a 6-foot depth to operate at Elev. 5 in the Mississippi River. The flow line profile is included in the HGL profile (Figure L2.7.3-1).

**Alternate Flow Line Profile**
If the existing drainage channels are lowered to Elev. -6.5, the transmission canal and diversion culvert could be lowered 2 feet, possibly reducing facility size and right-of-way requirements. During final design, a comparison of facility sizes should be done to determine cost and operational benefits of de-silting the existing outfall drainage channels, allowing the flow line for the diversion culvert and transmission canal to be lowered.

### L2.7.5 South Bridge Diversion Culvert

The system HGL for the South Bridge diversion culvert is provided on **Figure L2.7.5-1.** The head loss through the culvert is similar to the Romeville Diversion Culvert discussed in Section L2.6.4.. The similarity was noted and enabled cost to be determined for each of the alternatives.

### L2.7.6 South Bridge Diversion Siphon

The South Bridge Diversion Siphon is similar to the Romeville Diversion Siphon discussed in section L2.6.5. The cost estimate for comparison of alternatives was based on similar configurations.

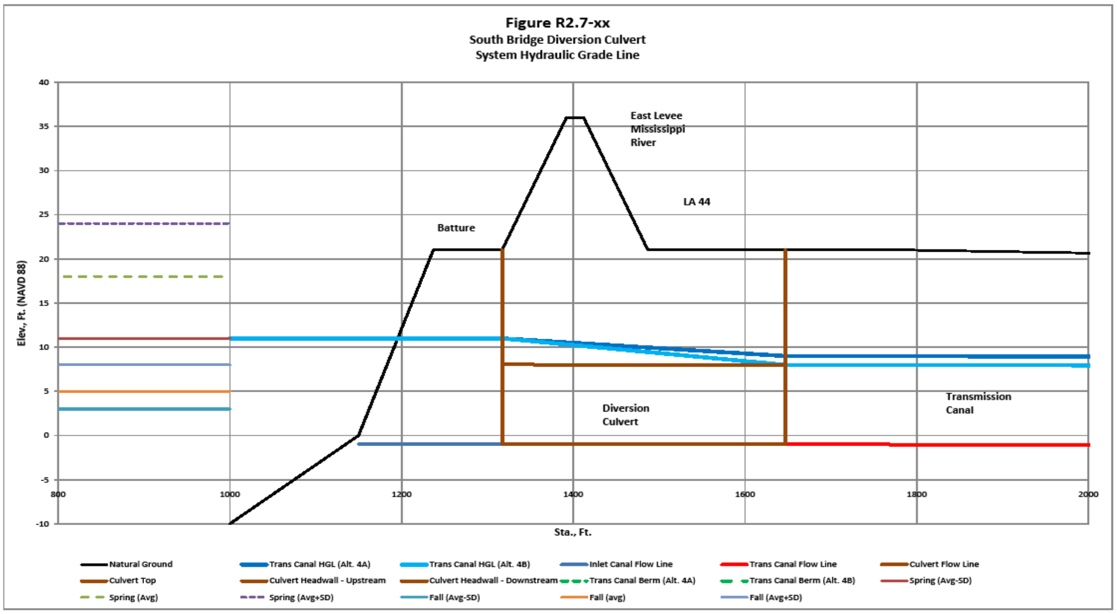



**Figure L2.7.5-1 System Hydraulic Grade Line (South Bridge Diversion Culvert)**

### L2.7.7 South Bridge Transmission Canal

The transmission canal will transfer the diverted water from the diversion facility to the existing drainage channel near the northwest corner of the target project area in the Swamp. The South Bridge transmission canal alignment is approximately 14,800 feet long, and will cross one road (LA 3125) and one railroad (Canadian National Railroad). The canal will have the following features:

- Stilling basin at head of the canal (hydraulic design addressed elsewhere);
- Earthen canal with a flat bottom and 4:1 side slopes (H:V);
- Earthen berms – the canal design HGL will be above ground for approximately 2/3 of the alignment, requiring earthen berms on both sides;
- Railroad crossing – reinforced concrete box culverts; and
- LA 3125 crossing – reinforced concrete box culverts.

**Design Basis**

The canal design is based on the following:

- See Section L2.7.3 and Figure L2.7.3-1 for the hydraulic grade line.
- See Section L2.7.4 and Figure L2.7.3-1 for the flow line profile.
- Side slopes are 4:1 (H:V) to be conservative. The geotechnical investigation may allow steeper side slopes.

- Erosion protection – the design velocities are low, and erosion potential is minimal.  Concrete channel lining and riprap will be used at the upstream and downstream sides of the culverts, and at the outfall into the existing Parish drainage channel.
- Manning's n value – 0.035 for a well-maintained vegetative lined or earthen canal.
- Design for steady-state flow.
- Freeboard – Provide 3 feet of freeboard.
- Berms – 12-foot wide top width to allow maintenance vehicle access, 4:1 side slopes (interior), and 4:1 or 5:1 exterior side slopes for mowing safety.
- Right-of-way width –
    - Without berms – minimum of 30 feet each side for access by large maintenance equipment and for drainage
    - With berms – minimum of 10 feet beyond the outer toe of berm on each side, for a local drainage swale and mowing access
- ROW drainage – provide a small drainage swale at the ROW line and discharge to local drainage.

Options to be considered for the transmission canal design:

- Match the proposed transmission canal flow line to the existing outfall drainage channel assumed flow line (Elev. -2.5).  This option is used for the current design recommendation.
- Design the proposed transmission canal flow line to be lower than the existing drainage systems.
- Desilt the existing outfall drainage channel and lower the transmission canal downstream end flow line.  This should be reviewed during final design.
- Concrete-lined channel.  Preliminary cost comparisons indicate that a concrete-lined canal will increase construction costs substantially; however, this option should be reviewed during final design.

The range of flow rates being considered for the South Bridge diversion point is from 500 cfs to 25,000 cfs, or higher.  The preliminary design analysis for the transmission canal covers a range of flows from 500 cfs to 20,000 cfs.

For this analysis, the transmission canal was designed for the Alternative 4B conditions.

**HEC-RAS Model for Canal Design**

A HEC-RAS model was developed to analyze the transmission canal hydraulics. The cross sections were cut from the 2001 LiDAR-based DEM obtained from the State of Louisiana.  The headlosses for the canal were calculated, as follows:

- Manning's n value – 0.035 for earthen channels and 0.015 for concrete channels and concrete box culverts.
- Expansion and Contraction Losses – typical coefficients were obtained from the HEC-RAS manual.
- Entrance and Exit Losses – typical coefficients were obtained from the HEC-RAS manual. All culverts are assumed to have concrete headwalls perpendicular to the flow.

For each design, a series of flow rates were used in the design analysis to determine the operating characteristics of the canal through the expected range of flow rates. Starting water surface elevations were linearly interpolated, matching the HGL design basis stated above.

- Elev. 2.0 – With no flow in the system, the starting WSEL is the same as the static WSEL in the Swamp
- Elev. 7.0 – WSEL at the design flow rate
- Elev. 8.5 – Assumed maximum WSEL in the North Distribution Canal, under a high diversion flow rate

To develop a canal section for each design flow rate, the canal bottom width was adjusted to obtain a water surface profile meeting the HGL criteria noted above. The flow line profile and berm elevations remained the same for all design flow rates. The HEC-RAS model was used to create canal designs for 500, 1000, 2000, 5000, and 10000 cfs flow rates. A plot of these bottom widths versus the design flow rates approximates a straight line; therefore, the bottom widths for other flow rates were interpolated. The recommended channel design sections are in the following table. The right-of-way widths vary throughout the length of the canal, as natural ground elevations vary. The table below uses the maximum right-of-way width. Actual right-of-way acquisitions could be less in certain reaches.

**Freeboard and Excess Capacity**

The diversion structure will operate with a varying driving head, varying diversion structure capacity in service, and possible with variable control, all resulting in the likelihood that flow rates will not be finely controlled to the design flow rate. Therefore, the transmission canal needs excess capacity to avoid overtopping the berms. With the 3-foot freeboard design, the canal has the excess capacity as shown in **Table L2.7.7-2**. For the 3,000 cfs transmission canal design with berms set to provide a 3-foot freeboard, the channel could be overtopped at 4,800 cfs, or 1.6 times the diversion design flow rate.

**Table L2.7.7-1 Recommended Channel Design Sections**

| Design Flow Rate, cfs | Bottom Width, ft | Proposed ROW Width, ft |
|---|---|---|
| 500 | 10 | 250 |
| 1,000 | 30 | 270 |
| 1,500 | 60 | 295 |
| 2,000 | 80 | 320 |
| 2,500 | 105 | 345 |
| 3,000 | 130 | 370 |
| 3,500 | 150 | 395 |
| 4,000 | 175 | 420 |
| 4,500 | 200 | 440 |
| 5,000 | 220 | 460 |
| 10,000 | 450 | 690 |
| 15,000 | 685 | 930 |
| 20,000 | 920 | 1,160 |

**Table L2.7.7-2 Freeboard Data**

| Flow Rate, cfs | Design Flow Rate Multiple | Freeboard Reduction, ft | Freeboard Remaining, ft | Comments |
|---|---|---|---|---|
| 3,000 | 1.0 | 0 | 3 | Design diversion flow rate |
| 3,600 | 1.2 | 1 | 2 | - |
| 4,200 | 1.4 | 2 | 1 | - |
| 4,800 | 1.6 | 3 | 0 | Berms overtopped |

The rating curves (flow rate vs. WSEL) at the upstream end of the transmission canal for the 1,500 cfs and 3,000 cfs designs are shown on **Figure L2.7.7-1**. The rating curves presented are the averages for the designs performed in HEC-RAS.

The velocities in the canal are relatively low, due to the restrictions on the HGL. These low velocities will need to be reviewed in the next design phase, as the Mississippi River sediment data becomes available. **Figure L2.7.7-2** has the velocity plots for the 1,500 cfs and 3,000 cfs designs at the upstream end of the transmission canal. The values are interpolated from the HEC-RAS designs for other flow rates.

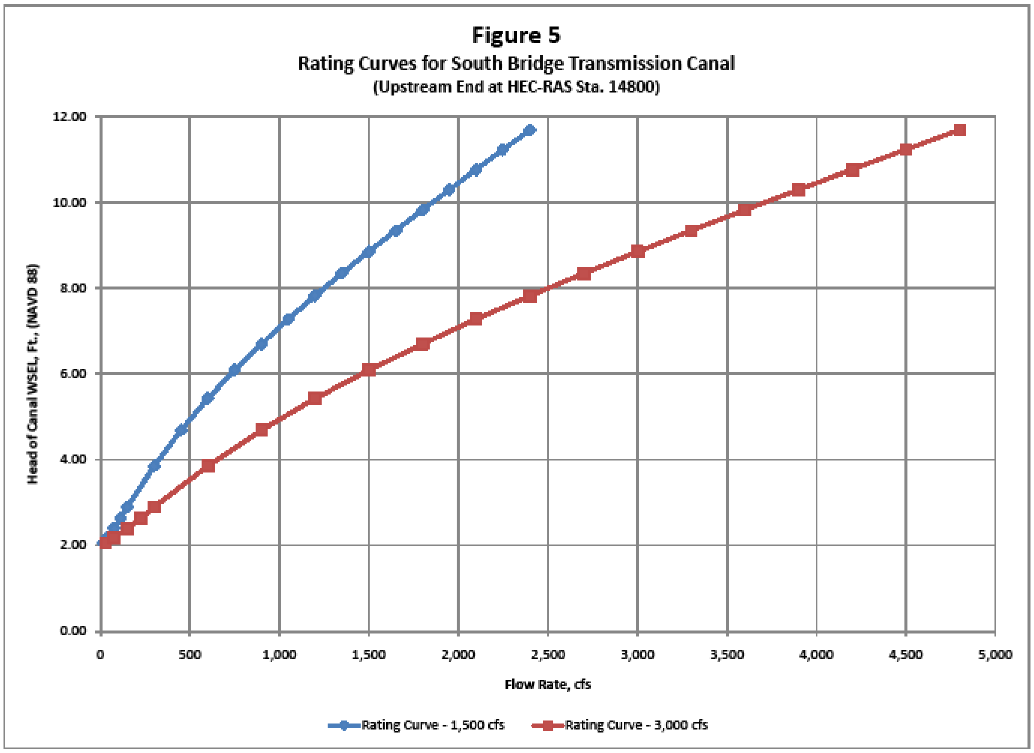
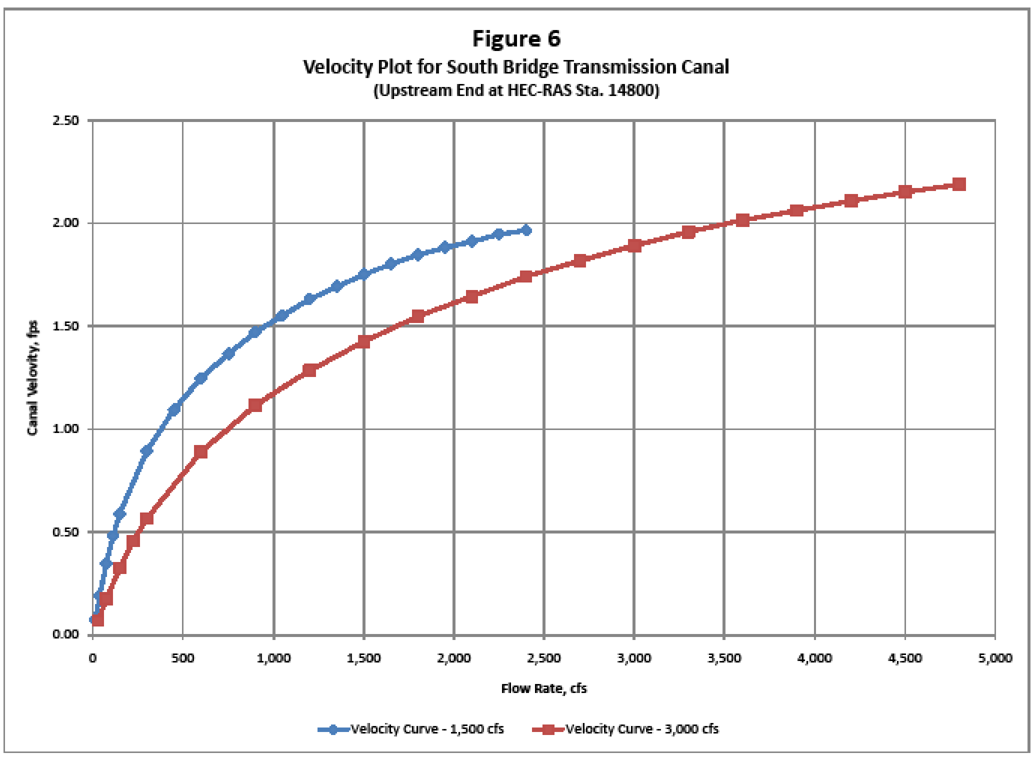



**Figure L2.7.7-1 Rating Curves for South Bridge Transmission Canal**




Blind River Romeville Diversion Culvert 1 of 1

**Figure L2.7.7-2 Velocity Plots for South Bridge Transmission Canal**

**Road and Railroad Crossings**

For the initial transmission canal designs, culverts were not placed into the HEC-RAS model at the LA 3125 and CN RR crossings, on the assumption that the culverts will be sized to have low head losses. Reinforced concrete box culverts will be used to cross the existing transportation facilities for the lower design flow rates and bridges could be used for the higher design flow rates. For preliminary design and costs, culverts were sized for 4 fps. As noted in **Table L2.7.7-3**, the water depths are less than 10 feet; therefore, the box culverts could have a maximum height of 9 feet. The CN RR crossing has sufficient clearance for 9-foot high boxes. The LA 3125 top of pavement is below the proposed HGL, and either shallower boxes or an inverted siphon will be required.

**Table L2.7.7-3 Transportation Crossing Data**

| Item | CN RR Crossing | LA 3125 Crossing |
|---|---|---|
| HEC-RAS Station | 12015 | 2177 |
| Road/Track Elevation | 16 + | 9 |
| Natural Ground in Area | 16 | 5 - 6 |
| Proposed Flow Line Elevation | -1.30 | -2.32 |
| Proposed Design WSEL | 8.6 | 7.3 |
| Depth of Water | 9.9 | 9.6 |

All elevations and dimensions in feet.

Using 9-foot high boxes for both the CN RR and the LA 3125 crossings, the initial box culvert sizes are summarized in **Table L2.7.7-4**.

**Table L2.7.7-4 Proposed Culvert Sizes**

| Diversion Design Flow Rate, cfs | Required Area at 4 fps, SF | Recommended Size, ft | Recommended Area, SF |
|---|---|---|---|
| 500 | 125 | 2 - 8 x 8 | 128 |
| 1,000 | 250 | 3 - 10 x 9 | 270 |
| 1,500 | 375 | 4 - 12 x 9 | 432 |
| 2,000 | 500 | 5 - 12 x 9 | 540 |
| 2,500 | 625 | 6 - 12 x 9 | 648 |
| 3,000 | 750 | 7 - 12 x 9 | 756 |
| 3,500 | 875 | 9 - 12 x 9 | 972 |
| 4,000 | 1,000 | 10 - 12 x 9 | 1,080 |
| 4,500 | 1,125 | 11 - 12 x 9 | 1,188 |
| 5,000 | 1,250 | 12 - 12 x 9 | 1,296 |
| 10,000 | 2,500 | 24 - 12 x 9 | 2,592 |
| 15,000 | 3,750 | 35 - 12 x 9 | 3,780 |
| 20,000 | 5,000 | 47 - 12 x 9 | 5,076 |

### L2.7.8 North Distribution Canal

The north distribution channel extends into the Maurepas Swamp from the west corner of the project area to the KCS RR and Hwy 61 corridor. For the segment west of the KCS RR, most if the alignment is immediately south of and parallel to an existing pipeline easement. Flow will be released from the canal into the Swamp at multiple locations, as with irrigation canals. Two options were considered for the north distribution canal capacity and service area, as follows:

- Alternative 4A – Extend the canal east across the KCS RR and Highway 61 to add hydrologic units east of Highway 61 to the service area. To serve the eastern 300 Series HUs, the canal would be approximately 39,000 feet long (Figure L2.7-1).
- Alternative 4B – End the canal west of the KCS RR and serve the areas west of the RR. The canal will be approximately 30,900 feet long (Figure L2.7-2).

The canal will have the following features:

- Earthen canal with a flat bottom and 3:1 side slopes (H:V).
- Earthen berms – the canal HGL will be above natural ground for the entire alignment, requiring earthen berms on both sides.
- An inverted siphon will be required at the existing Parish drainage channel, at the start of the north distribution canal. The initial design concept is to put the drainage channel through an inverted siphon under the canal (and not put the canal in an inverted siphon).
- Additional inverted siphons may be required at other drainage features in the service area.
- KCS RR crossing – inverted siphon (reinforced concrete box culverts).
- Hwy 61 crossing – inverted siphon (reinforced concrete box culverts).

**Design Basis**

The North Distribution Canal design is based on the following:

- The flow line of upstream end of the north distribution canal will match the downstream end of the transmission canal.
- The flow line of the north distribution canal will be flat, and the bottom width will be reduced as the flow rate is reduced with releases into the Swamp.
- Side slopes are 3:1 (H:V), as the channel is relatively shallow. The geotechnical investigation may recommend flatter side slopes.
- Erosion protection – the design velocities are low, and erosion potential is minimal. Concrete channel lining and riprap will be used at the upstream and downstream sides of the culverts, and at the outfall into the existing Parish drainage channel.

- Manning's n value – 0.035 for a well-maintained vegetative lined or earthen canal.
- Design for steady-state flow.
- Freeboard – Provide 2 feet of freeboard.
- Excess capacity – see discussion below.
- Berms – 12-foot wide top width to allow maintenance vehicle access, 3:1 side slopes (interior), and 4:1 or 5:1 side slopes (exterior). The berm height is generally low, and steeper exterior side slopes should be stable.
- Right-of-way width – A separate right-of-way may not be required on State land in the Wildlife Management Area (WMA). The following criterion is an approximation of area required for construction and maintenance of the canal system.
    - Without berms – minimum of 30 feet each side for large maintenance equipment and drainage
    - With berms – minimum of 10 feet beyond the outer toe of berm on each side, for a local drainage swale and mowing access
- ROW drainage – typically none, as the Swamp normally is saturated and has standing water. If necessary, provide a small drainage swale outside of the berm and discharge to local drainage.

**Excess Capacity**
The north distribution canal is in the Swamp, where overflows are non-critical to the area. Protect the canal with control sections in the berms, such as weir sections set at one foot above the design HGL. At the overflow area, line the berm with erosion protection, such as rip rap or concrete channel lining. This would protect the canal from uncontrolled overflows and berm washout, without oversizing the canal. A primary overflow control section could be placed at the upstream end of the north distribution canal and direct the overflow into the existing Parish drainage channel.

**Design Flow Rates**
The following flow rates were used for the preliminary canal design, as the alternative plans were developing:

- Alternative 4A – 900, 1,000, 1,500, 2,000, and 2,500 cfs;
- Alternative 4B – 500, 1,000, and 1,500 cfs;
- Option 1 – 500 cfs to serve the 200 Series HUs; and
- Option 2 – 900 cfs to serve the 200 and 300 series HUs.

The flow rates were uniformly reduced through the canal reach, to represent multiple releases to the Swamp service area, as with irrigation canals. For Alternative 4A, it was considered that 40% of the flow in the north distribution canal would be transferred across the KCS RR/Hwy 61 corridor to the Series 300

HUs. This flow proration will need to be adjusted as the hydro-dynamic modeling progresses.

**HEC-RAS Model for Canal Design**

A HEC-RAS model was developed to size and analyze the canal hydraulics. The cross sections were cut from the 2001 LiDAR-based DEM obtained from the State of Louisiana. The headlosses for the canal were calculated, as follows:

- Manning's n value – 0.035 for earthen channels, 0.015 for concrete channels, and 0.013 for concrete box culverts.
- Expansion and Contraction Losses – typical values were obtained from the HEC-RAS manual.
- Entrance and Exit Losses – Typical entrance and exit loss coefficients were obtained from the HEC-RAS manual. All culverts are considered to have concrete headwalls.

To develop a section for each design flow rate, the canal bottom width was adjusted to obtain a water surface profile meeting the HGL criteria noted above. The flow line profile and berm elevations remained the same for all design flow rates.

For each design, a series of flow rates were used in the design analysis to determine the operating characteristics of the canal through the expected range of flow rates. Starting water surface elevations are from the HGL design basis stated above.

The recommended channel design sections are summarized in **Table L2.7.8-1**. The right-of-way widths vary throughout the length of the canal, as natural ground elevations vary. The table below uses the maximum right-of-way width. Actual right-of-way acquisitions could be less in certain reaches.

**Table L2.7.8-1 Recommended Channel Design Sections**

| Design Flow Rate, cfs | Bottom Width, Ft. | Proposed ROW Width, Ft. |
|---|---|---|
| 500 | 10 | 250 |
| 1,000 | 30 | 270 |
| 1,500 | 60 | 295 |
| 2,000 | 80 | 320 |
| 2,500 | 105 | 345 |
| 3,000 | 130 | 370 |
| 3,500 | 150 | 395 |
| 4,000 | 175 | 420 |
| 4,500 | 200 | 440 |
| 5,000 | 220 | 460 |
| 10,000 | 450 | 690 |
| 15,000 | 685 | 930 |
| 20,000 | 920 | 1,160 |

The rating curves (flow rate vs. WSEL) for the 1,500 cfs and 3,000 cfs designs are illustrated on Figure L2.7.7-1. The values presented are the averages for the designs performed in HEC-RAS.

The velocities in the canal are relatively low, due to the restrictions on the HGL. These low velocities will need to be reviewed in the next design phase, as the Mississippi River sediment data becomes available. Figure L2.7.7-2 has the velocity plots for the 1,500 cfs and 3,000 cfs designs. The values are interpolated from the HEC-RAS designs for other flow rates.

**Road and Railroad Crossings**
For the initial Alternative 4A canal design, culverts were not placed into the HEC-RAS model at the KCS RR and Highway 61 crossings, on the assumption that the culverts will be sized to have low head losses. Reinforced concrete box culverts will be used to cross the existing transportation facilities for the lower design flow rates and bridges could be used for the higher design flow rates. For preliminary design and costs, culverts were sized for 4 fps. As noted in the table below, the water depths are less than 10 feet; therefore, the box culverts could have a maximum height of 9 feet. The CN RR crossing has sufficient clearance for 9-foot high boxes. The LA 3125 top of pavement is below the proposed HGL, and either shallower boxes or an inverted siphon will be required.

**Table L2.7.8-2 Transportation Crossing Data**

| Item | KCS RR Crossing | Hwy 61 Crossing |
|---|---|---|
| HEC-RAS Station | 1984 | 1278 |
| Road/Track Elevation | 4 | 6 |
| Natural Ground in Area | 1 to 2 | 1 to 2 |
| Proposed Flow Line Elevation | -4 | -4 |
| Max. Inside Top-of-Box Elevation | 0 | 2 |
| Proposed Design WSEL | 4 | 4 |
| Inside Height of Box, max. | 4 | 6 |
| Depth of Water in Canal | 8 | 8 |

Note: All elevations and dimensions in feet.

At the KCS RR crossing, the boxes would have to be 4' high, maximum. At the Highway 61 crossing, 6' high boxes could be used. At both locations, higher box culverts by dropping the profile and creating inverted siphons, if the overall width becomes excessive. For preliminary hydraulic design, consider 6-foot high box culverts.

**Table L2.7.8-3 Proposed Culvert Sizes**

| At head of canal - Design Flow | At KCS RR/Hwy 61 - | Required Area at 4 fps, | Recommended Size, Ft. | Recommended Area, SF |
|---|---|---|---|---|

| Rate, cfs | Design Flow Rate, cfs | SF | (WxH) | |
|---|---|---|---|---|
| 500 | 200 | 50 | 2 – 5’ x5’ | 50 |
| 900 | 400 | 100 | 3 – 6’ x 6’ | 108 |
| 1,000 | 400 | 100 | 3 – 6’ x 6’ | 108 |
| 1,500 | 600 | 150 | 3 – 9’ x 6’ | 162 |
| 2,000 | 800 | 200 | 4 – 9’ x 6’ | 216 |
| 2,500 | 1,000 | 250 | 5 – 9’ x 6’ | 270 |

### L2.7.9 Existing Drainage Channel Improvements

Sufficient topographic surveying data is not yet available to analyze the hydraulic capacity of the existing Parish drainage channel. See Section L7 for assumptions used to develop conceptual sizing, quantities, and costs for screening.

## L2.8 Swamp Distribution System Analysis

Various water management measures have been identified to apply freshwater to the swamp to beneficially allow transfer of freshwater, nutrients and sediments to the swamp. The flow rate will need to be controlled at both the inlets and outlets to the swamp in order to control the depth and detention time of the water directed into the swamp. A fluctuating hydroperiod (water depth and duration) with dry periods is critical to the germination and sapling survival of bald cypress and tupelo. A fluctuating hydroperiod will also enhance assimilation and improve the quality of water-released to the Blind River. The benefits of several types of water management facilities have been reviewed including berm gaps/cuts, control structures, and culverts. These facilities are described in further detail below.

### L2.8.1 Berm Gaps/Cuts

Following a review of the topographic data and field reconnaissance within the project boundary, it was determined that there are approximately 163 existing berm openings in the man-made berms along the southern border of the project area and the St. James Parish Canal Systems. These berms are spaced at approximately 610 feet and the average dimensions are 10 feet in length (measured parallel to the canal) and 20 feet in width (measured perpendicular to the canal). These natural weirs, presented on **Figure L2.8.1-1**, will allow for some exchange of flow between the canals and the adjacent swamps. In an effort to increase the capacity, the proposed berm cuts will utilize and expand on the existing berm cuts. The proposed berm cuts, however, will be expanded to a length of 500 feet with a width of 20 feet and extend to existing grade. The side slopes of the berm gaps will be protected with articulated concrete block mats or other erosion protection measures. The proposed berm cut locations are displayed on **Figure L2.8.1-2**. The existing spoil from the spoil piles will be excavated at the proposed berm cut locations and placed behind the existing spoil piles on either side of the gap. The spoil material will be used to expand existing berm width, creating more upland habitat.

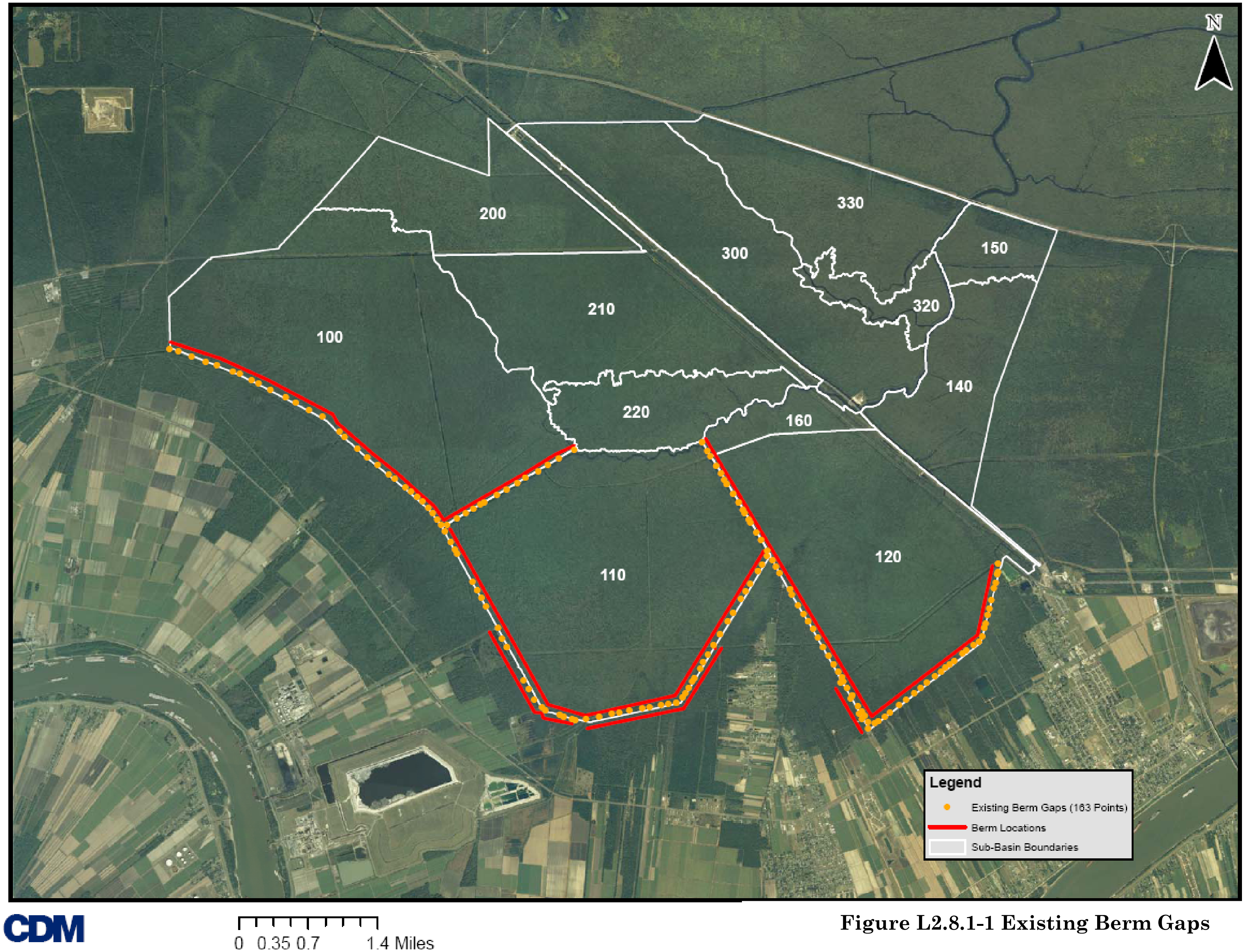



**Figure L2.8.1-1 Existing Berm Gaps**

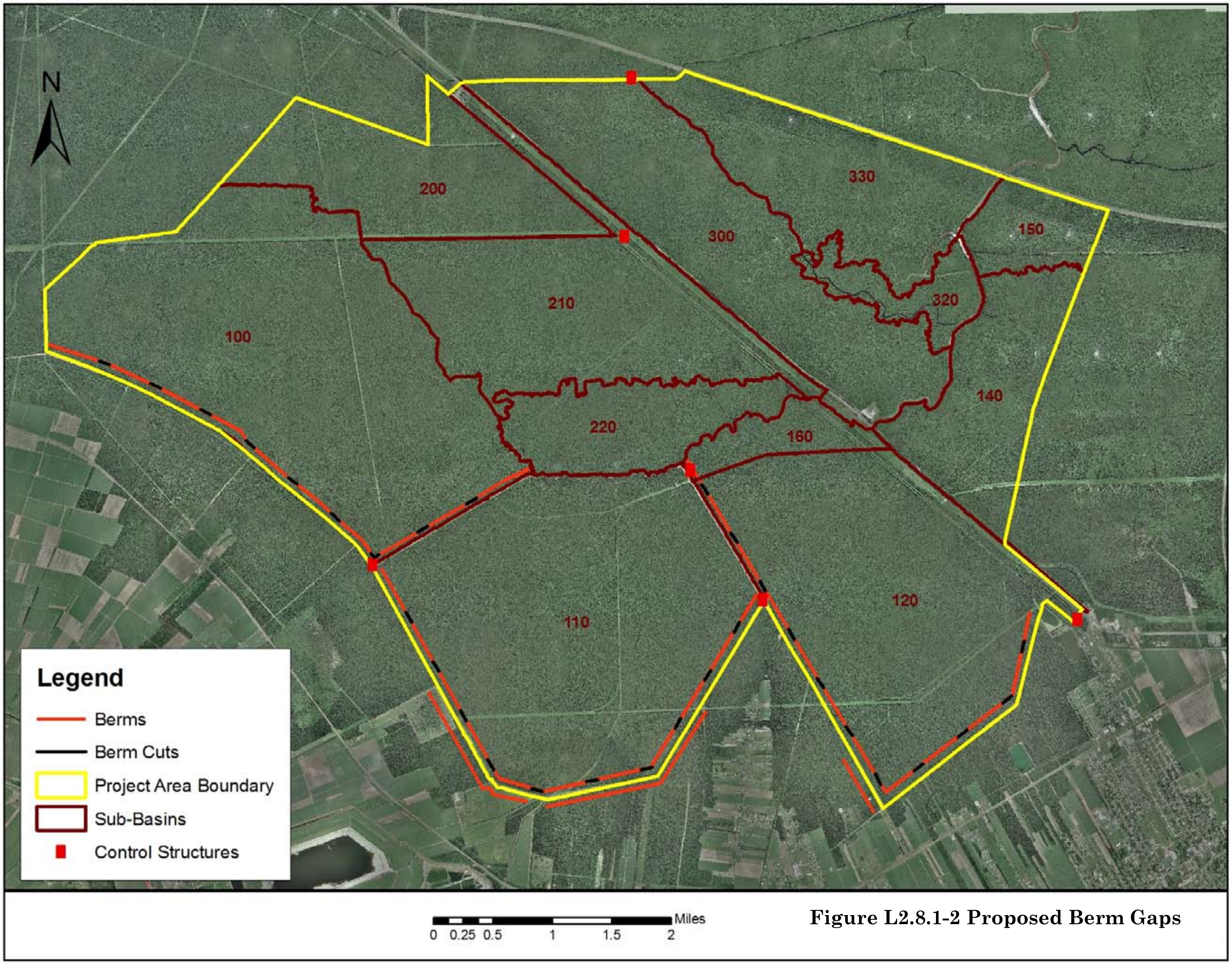



**Figure L2.8.1-2 Proposed Berm Gaps**

### L2.8.2 Control Structures

The purpose of adding control structures in the canals is to force water into the Swamp. This dynamic can be achieved by placing a barrier within the existing channel. The structure would allow an increase in the water surface level behind it, providing a hydraulic gradient to force the freshwater into the swamp. The canals currently serve drainage and flood control purposes and measures would be incorporated into the design to accommodate these needs. Active monitoring and management would be required for this management measure to avoid flood impacts to the developed areas adjacent to the swamp.

The type of gate that has been chosen for this application is a rotating gate. Both Rodney Hunt and Obermeyer Hydro, Inc manufacture rotating control gates that have a parabolic configuration rotating from 0 degrees (flush with the canal bottom) to 90 degrees (perpendicular to canal flow) and any angle in between. One of these types of control weirs will be chosen for the application of this project. Initially, the opportunity for control structures was provided at numerous locations. A detailed analysis was performed for all these potential control structure locations and is included in **Annex 6**. This analysis was used to determine which control structures will continue to be used in future alternatives. Structures were eliminated because they didn't control flows, or would require difficult construction and implementation. Five structures will remain in the project and are displayed on **Figure L2.8.2-1**.

Two of the five structures are located a "tee" in the canal system. The control structures at these two locations (1-6 and 1-8) have the capability of controlling flow in three directions. Bathymetric data and LiDAR data were used to determine the dimensions for the control structures and are summarized in **Table L2.8.2-1**. A control structure isometric view is displayed on **Figure L2.8.2-2**.

**Table L2.8.2-1 Structure Summary Table**

| Control Structure | Est. Channel Width (ft) | Est. Channel Depth (from TOB) (ft) | Location Description |
|---|---|---|---|
| 1-3 | 164 | 8.4 | St. James Parish Canal |
| 1-6 East | 147 | 6.7 | St. James Parish Canal, At Romeville transmission connection |
| 1-6 South | 66 | 4.9 | St. James Parish Canal, At Romeville transmission connection |
| 1-6 North | 66 | 4.7 | St. James Parish Canal, At Romeville transmission connection |
| 1-7 | 65 | 5.6 | St. James Parish Canal near Hwy 61 |
| 1-8 Southwest | 130 | 8.1 | St. James Parish Canal |
| 1-8 Southeast | 130 | 10.5 | St. James Parish Canal |
| 1-8 Northwest | 130 | 10.5 | St. James Parish Canal |
| 2-4 | 65 | 0 | Adjacent to Hwy. 61 |
| 3-1 | 213 | 12.3 | Conway Canal |

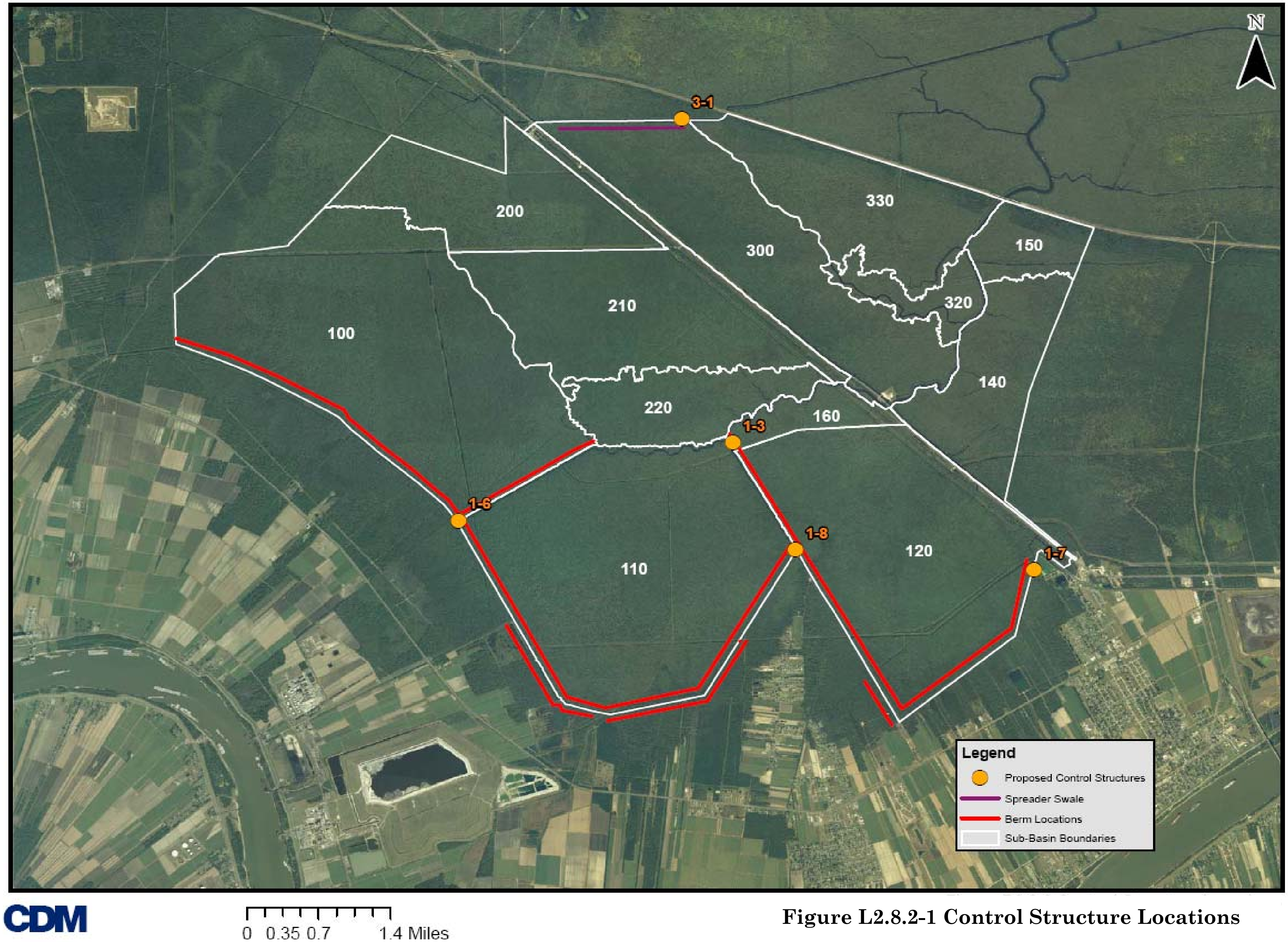
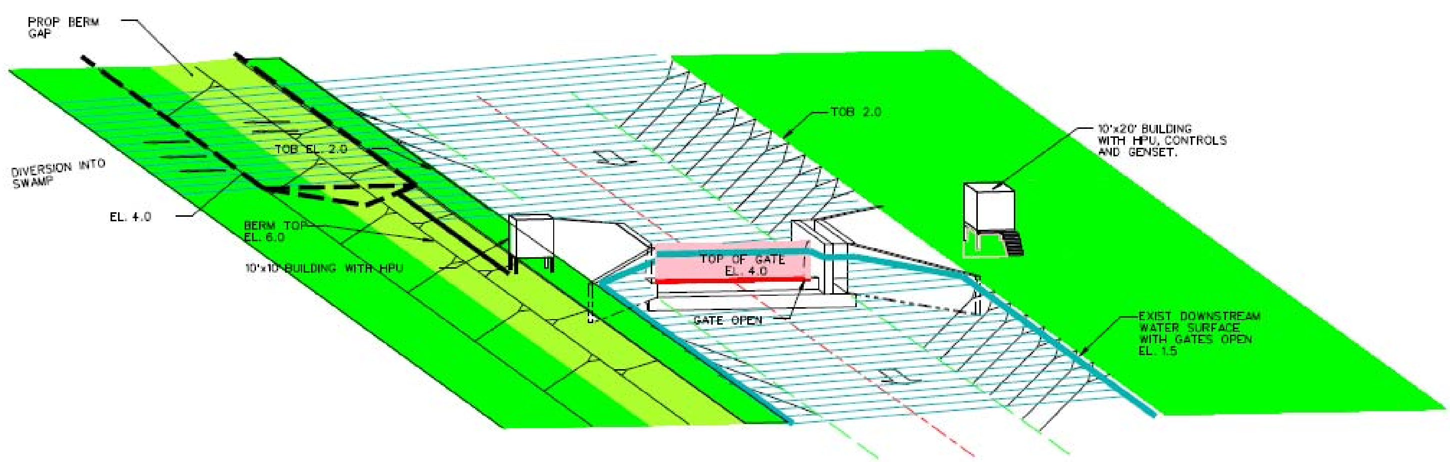



**Figure L2.8.2-1 Control Structure Locations**




CDM

**Figure L2.8.2-2 Control Structure Isometric View**

N.T.S.

### L2.8.3 Culverts and Bridges

A field survey was performed to verify all the existing culvert and bridge crossings along Highway 61 and Interstate 10. A summary of the findings are included in **Table L2.8.3-1**. In close proximity to the project site, there are five bridge crossings located through Highway 61 and two bridge crossings through Interstate 10. In addition, there are 10 culvert crossings through Interstate 10. These culverts range in size from a 36-inch-diameter reinforced concrete pipe (RCP) to a 6-foot by 4-foot reinforced concrete box culvert (RCBC).

Highway 61 and the KCS Railroad act as barriers between hydrologic units on the northeast and southwest of the project area. In an effort to distribute flow to both sides of the highway and railroad, consideration has been made to the addition of new culvert crossings at four locations along the road. These crossing locations will each consist of four 6-foot by 4-foot RCBCs under Highway 61 and under the KCS Railroad. The culverts will be connected via a 580 foot long channel. The channel will be excavated with a bottom width equal to 27 feet and 4:1 side slopes. The proposed culvert crossing locations are presented on **Figure L2.8.3-1**.

## L2.9 Swamp Flow Outlet Control Analysis

Maurepas Swamp and Blind River are part of a regional system that is influence by both rainfall-runoff characteristics of riverine watersheds and coastal processes, such as tidal cycles and storm surge. As noted in Appendix Section L2.4, portions of the existing swamps have subsided and are now persistently inundated. In addition, field observations have revealed that the existing swamp has a limited ability to drain following local and regional rainfall events. Based on this understanding, additional analyses were completed to expand consideration of downstream conditions and how they affect the study area. Downstream water levels were considered in two ways. First, available water elevation data for Lake Maurepas and Lake Pontchartrain were reviewed to determine how existing downstream conditions should be incorporated into the project analyses. Second, information related to projected sea level rise was compiled for incorporation into the project analyses.

### L2.9.1 Lake Maurepas

Analysis was completed to determine the statistical characteristics for the two lakes downstream of the Blind River project area in order to understand how downstream conditions affect the study area. Since freshwater diversion flows are primarily anticipated to function during average hydrologic conditions the focus of this analysis was on typical downstream water levels.

#### Stage Data

There are multiple long-term stage gage locations in the Lake Pontchartrain Basin. The two upstream gages in the basin, nearest to Blind River, were used for this analysis, as follows:

**Table L2.8.3-1 Field Survey of Bridges along Highway 61 and Culverts along Interstate 10**

| Location | Number of Culverts | Culvert Type | Culvert Size | Channel Width | Channel depth | Water Elevation | Additional Comments |
|---|---|---|---|---|---|---|---|
| 64 | 2 | RCBC | 6' x 4' | 25' | 4'-9" | 4'-9" | Clear |
| 65 | 4 | RCP | 3' | 28' | 3' | 2'-6" | Water surface elevation is at 75% of barrel diameter |
| 66 | 2 | CMP | 5' | 40' | 5' | 2'-8" | Heavily vegetated in outfall canal |
| 67 | 2 | CMP | 5' | N/A | N/A | 2'-6" | |
| 68 | N/A | Bridge | N/A | 140' | | 12'-6" | |
| 69 | Could not locate culvert | | | | | | |
| 70 | 2 | CMP | 5' | N/A | | 2' | |
| 71 | Could not locate culvert | | | 50' | 10' | 10' | Channel is relatively deep at this location |
| 72 | 2 | CMP | 5' | N/A | | 3' | Tree debris in inlet drain area |
| 73 | 3 | CMP | 5' | 30' | 5' | 3' | Tree debris in inlet drain area |
| 74 | Could not locate culvert | | | | | | |
| 75 | 2 | CMP | 5' | 30' | 5' | 3' | Vegetation and tree debris in inlet drain area |
| 76 | 3 | CMP | 5' | 20' | 5' | 2'-6" | Vegetation and tree debris in inlet drain area |
| 77 | N/A | Bridge | N/A | 114' | 5' | 2'-6" | Water stagnant with heavily vegetated outfall channel/channel is completely full of thick aquatic plants |
| 78 | N/A | Bridge | N/A | 90' | 12' | 8' | Clear |
| 79 | N/A | Bridge | N/A | 104' | 13' | 8' | Salvania is present throughout the water surface |

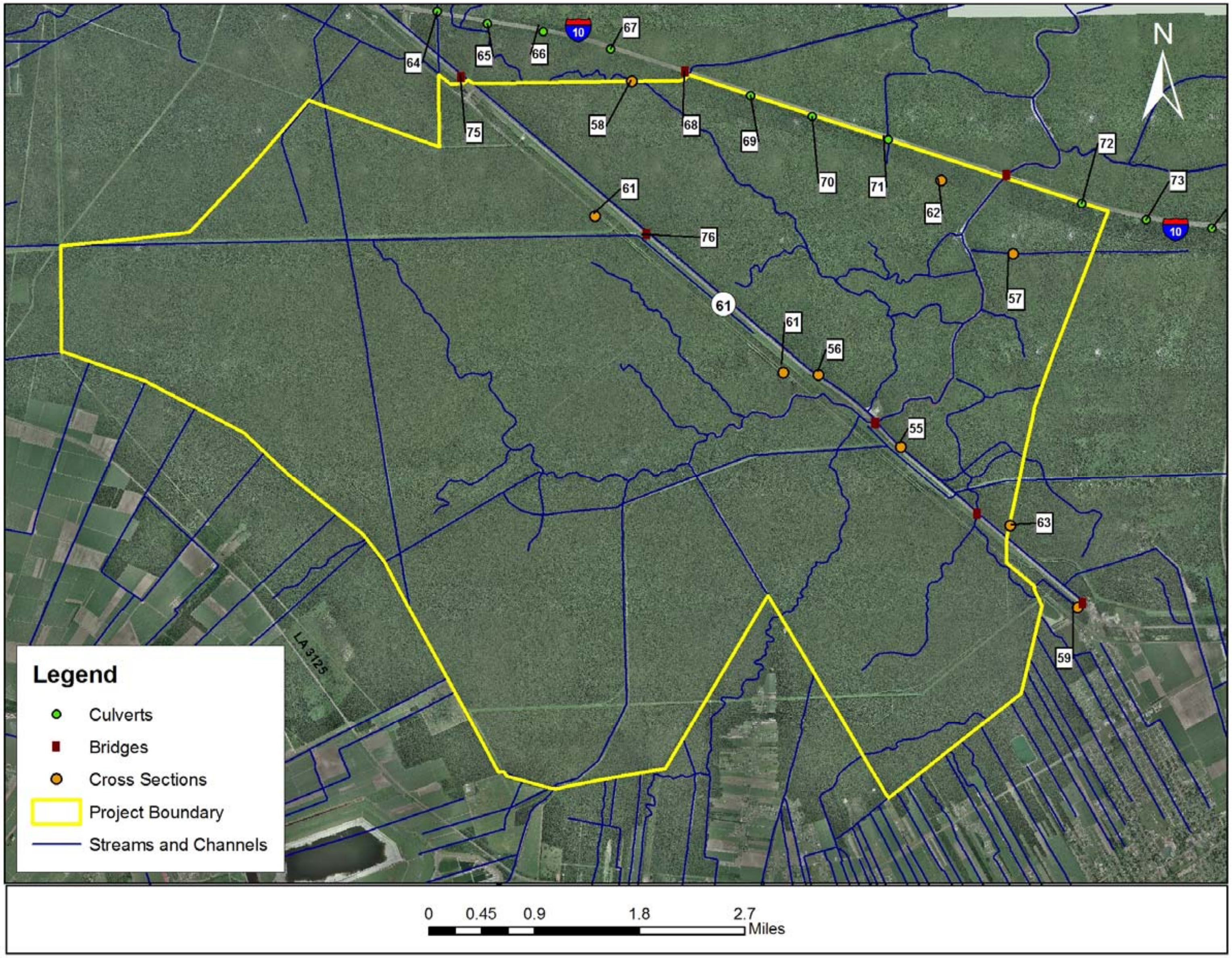



**Figure L2.8.3-1 Locations of Culverts, Cross-Sections, and Bridges**

- Pass Manchac near Pontchatoula – this gage is located on the upstream side of Pass Manchac and represents the stage in the east end of Lake Maurepas.
- Lake Pontchartrain near Frenier – this gage is located on the west end of Lake Pontchartrain.

Stage data were obtained for both gages from the USACE New Orleans Engineering District website. 30 years of data were collected for each gage. Both gages were apparently out of service part way through 2005 to 2009. Therefore, the data covers the period of January 1, 1975 through Dec. 31, 2004. The stage records at the two gage sites are incomplete, with multiple data points missing at both the Pass Manchac and the Lake Pontchartrain Frenier gages. As the stage varies significantly due to tides, no attempts were made to re-create or fill these missing values.

The available daily stage data at both gages are referenced to the NGVD (1929) vertical datum. For consistency with other project data, the stage data were converted to NAVD (1988). **Table L2.9.1-1** provides a summary of the data available at each gage and the datum adjustment between NGVD and NAVD. These datum conversion values were obtained through correspondence with staff from the USACE New Orleans District. The USACE provided a vertical adjustment value of -0.5 feet for the Pass Manchac gage, but stated that a datum adjustment value is not available for the Lake Pontchartrain Frenier gage. Therefore, for the purpose of this analysis the same value of -0.5 feet was assumed for the Pontchartrain Frenier gage. Note that in mid-2009, the gages were placed back into service, and the stage data is now being reported on the NAVD 88 vertical datum.

**Table L2.9.1-1 Downstream Gage Summary**

| Gage Data Feature | Pass Manchac Gage Near Pontchatoula | Lake Pontchartrain Gage at Frenier |
|---|---|---|
| Gage ID | 85420 | 85550 |
| Vertical Datum[1] | NGVD 1929 | NGVD 1929 |
| Gage 0 (feet) | Elevation 0 | Elevation 0 |
| Vertical Datum Adjustment[2] | | |
| Adjustment Value (ft) | -0.5 | |
| Adjustment Equation (ft) | NAVD = NGVD - 0.5 | NAVD = NGVD - 0.5 |
| Period of Record | July 1955 to Aug 2005 | Sep 1931 to May 2005 |
| Data Period Used | Jan 1975 to Dec 2004 | Jan 1975 to Dec 2004 |

Notes:

[1]Vertical datum refers to the datum applicable to data obtained from the source website

[2]Vertical datum adjustment from NGVD 1929 to NAVD 1988

**Statistical Analyses**
Statistical analyses were performed on the stage data to extract values and trends for use in the analysis and design of the diversion system and to gain insight into how downstream conditions influence the study area. Analyses completed included review of data averages, standard deviations, exceedance frequency and tides.

**Averages and Standard Deviations**
The averages and standard deviations were calculated for each day of the year from the daily stage data and summarized in **Table L2.9.1-2**. The daily average stage, the average stage minus one standard deviation, and the average stage plus one standard deviation are plotted on **Figure L2.9.1-1** for Pass Manchac and **Figure L2.9.1-2** for Lake Pontchartrain Frenier. The stage data for the two gages are daily values recorded at 8:00 AM. Since the locations are tidally influenced, the stage readings will be for different parts of the tide, ranging from the high to low tide. This will impact the statistical values noted in Table L2.9.1-2.

**Table L2.9.1-2 Observed Water Surface Elevation Statistics**

| Gage Summary | Observed Water Surface Elevation Statistics (feet - NAVD) | | | |
|---|---|---|---|---|
| | Standard Deviation | Average Minus One Standard Deviation | Average Water Elevation | Average Plus One Standard Deviation |
| **Pass Manchac Gage** | | | | |
| Annual | 0.6 | 0.1 | 0.7 | 1.3 |
| Spring (Mar 1 to May 31) | 0.7 | 0.1 | 0.8 | 1.5 |
| Summer (Jul 1 to Aug 31) | 0.5 | 0.0 | 0.5 | 1.1 |
| Fall (Sep 1 to Nov 30) | 0.5 | 0.4 | 0.9 | 1.4 |
| Winter (Dec 1 to Feb 28) | 0.6 | -0.1 | 0.5 | 1.1 |
| **Lake Pontchartrain Frenier Gage** | | | | |
| Annual | 0.6 | 0.0 | 0.6 | 1.2 |
| Spring (Mar 1 to May 31) | 0.6 | -0.1 | 0.6 | 1.2 |
| Summer (Jul 1 to Aug 31) | 0.5 | -0.1 | 0.4 | 0.9 |
| Fall (Sep 1 to Nov 30) | 0.6 | 0.5 | 1.1 | 1.7 |
| Winter (Dec 1 to Feb 28) | 0.6 | -0.1 | 0.5 | 1.1 |

The plot of the stage values on Figure L2.9.1-1 and Figure L2.9.1-2 indicate subtle trends corresponding to the seasons. The spring period (March through May) tends to have a higher stage compared to the summer and winter. Water levels observed in the fall (September to October) also appear to have higher than average stages, possibly indicating a statistical influence of tropical storm surges or increased seasonal precipitation.

**Percent Chance Exceedance**

A percent chance exceedance curve was developed for the Pass Manchac gage using the Weibull formula. A Pearson Type III analysis was not used, as the stage analysis is more concerned with long-term trends of typical values, and not with the extreme events. The percent chance exceedance plot for Pass Manchac is on **Figure L2.9.1-3**.

**Tide Height Analysis**

Limited hourly stage data is available for the Pass Manchac gage for part of 2009 (April 27, 2009 to the present) and plotted on **Figure L2.9.1-4**. Based on this very limited data, the tide height statistics indicated an average tide height of 0.4 feet and a standard deviation of 0.2 feet.

### L2.9.2 Sea Level Rise

Based on US Army Corps of Engineers guidance (EC 1165-2-211), potential relative sea-level change must be considered in every USACE coastal activity as far inland as the extent of estimated tidal influence. The guidance further states that planning, engineering, and designing for sea level change must consider how sensitive and adaptable 1) natural and managed ecosystems and 2) human systems are to climate change and other related global changes and that planning studies and engineering designs should consider alternatives that are developed and assessed for the entire range of possible future rates of sea-level change. Section L2.9.2.1 describes the projection for relative sea level rise developed for the project, and Section L2.9.2.2 discusses how the projected relative sea level rise values were applied.

#### L2.9.2.1 Projected Relative Sea Level Rise

Guidance was provided from staff at the USACE New Orleans District regarding projected sea level rise for the study area.

With respect to future sea level rise scenarios the guidance requires project performance to be assessed using three sea level change scenarios, a low estimate, an intermediate estimate, and a high estimate. The low estimate uses a projection of the historic rate for the study area. The intermediate estimate is based on the modified National Research Council (NRC) Curve I and the local historic subsidence rate, and the high estimate is based on the modified NRC Curve III and the local historic subsidence rate.

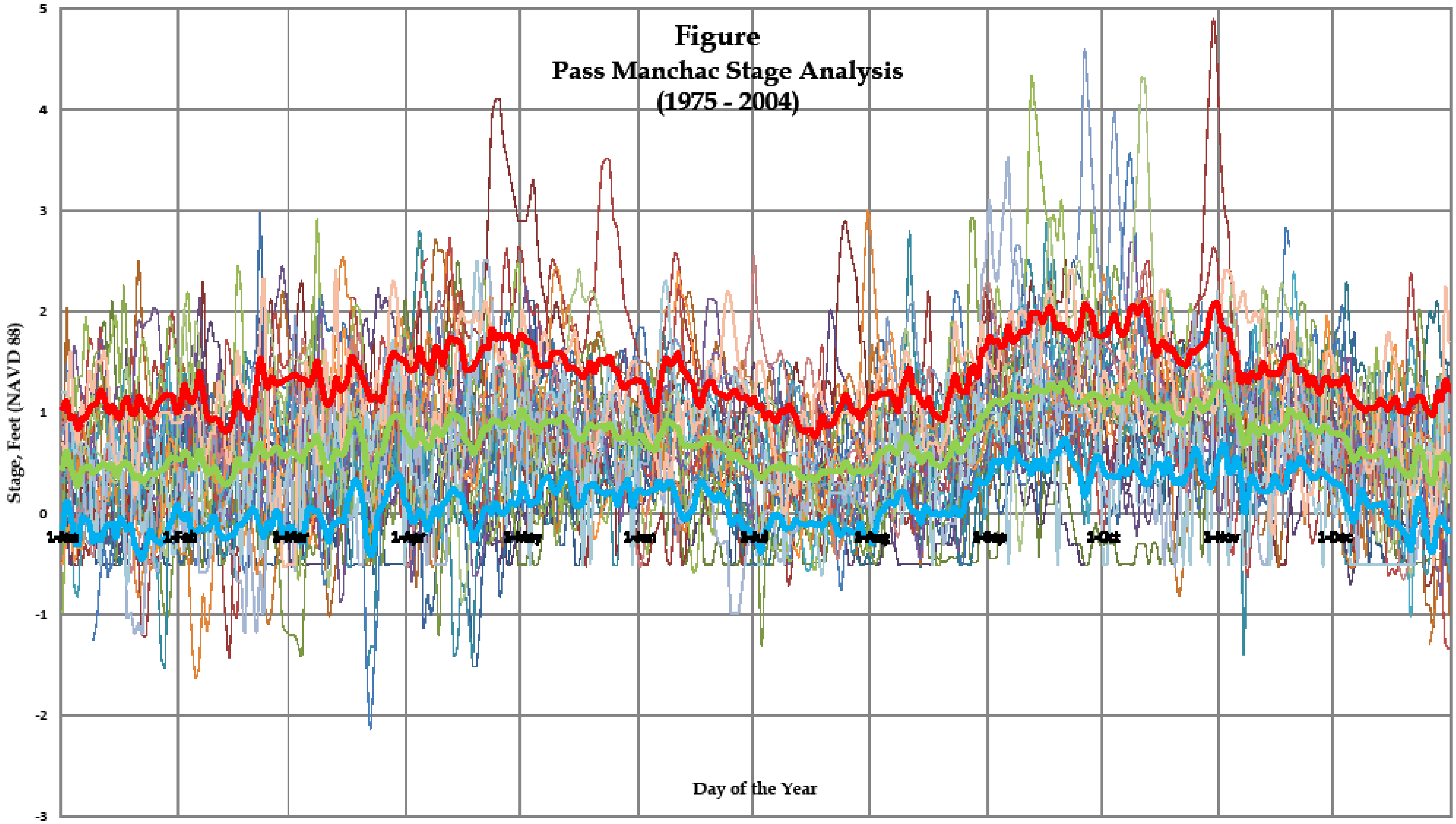



Figure L2.9.1-1 Pass Manchac Stage Analysis

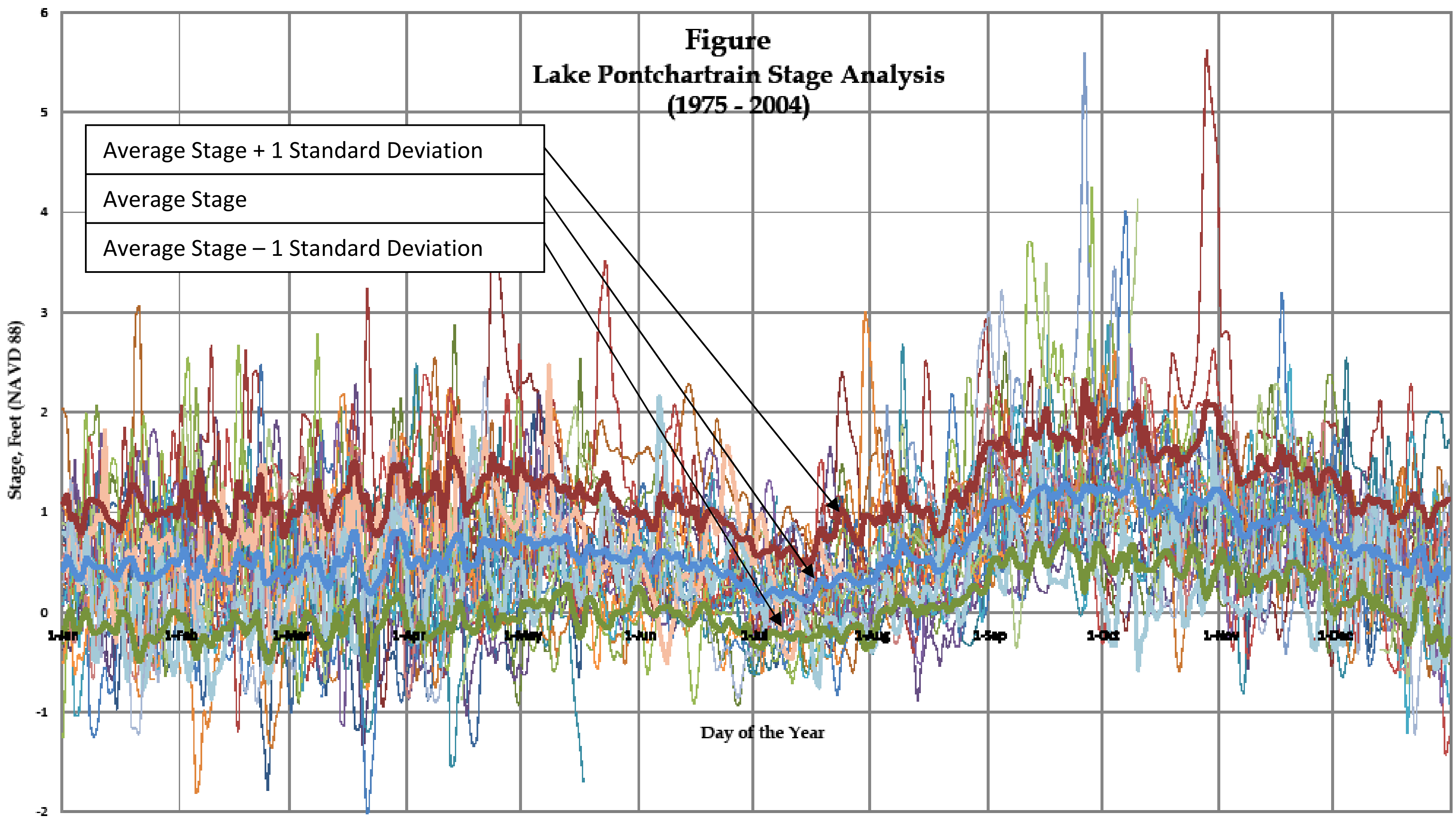



**Figure L2.9.1-2 Lake Pontchartrain Stage Analysis**

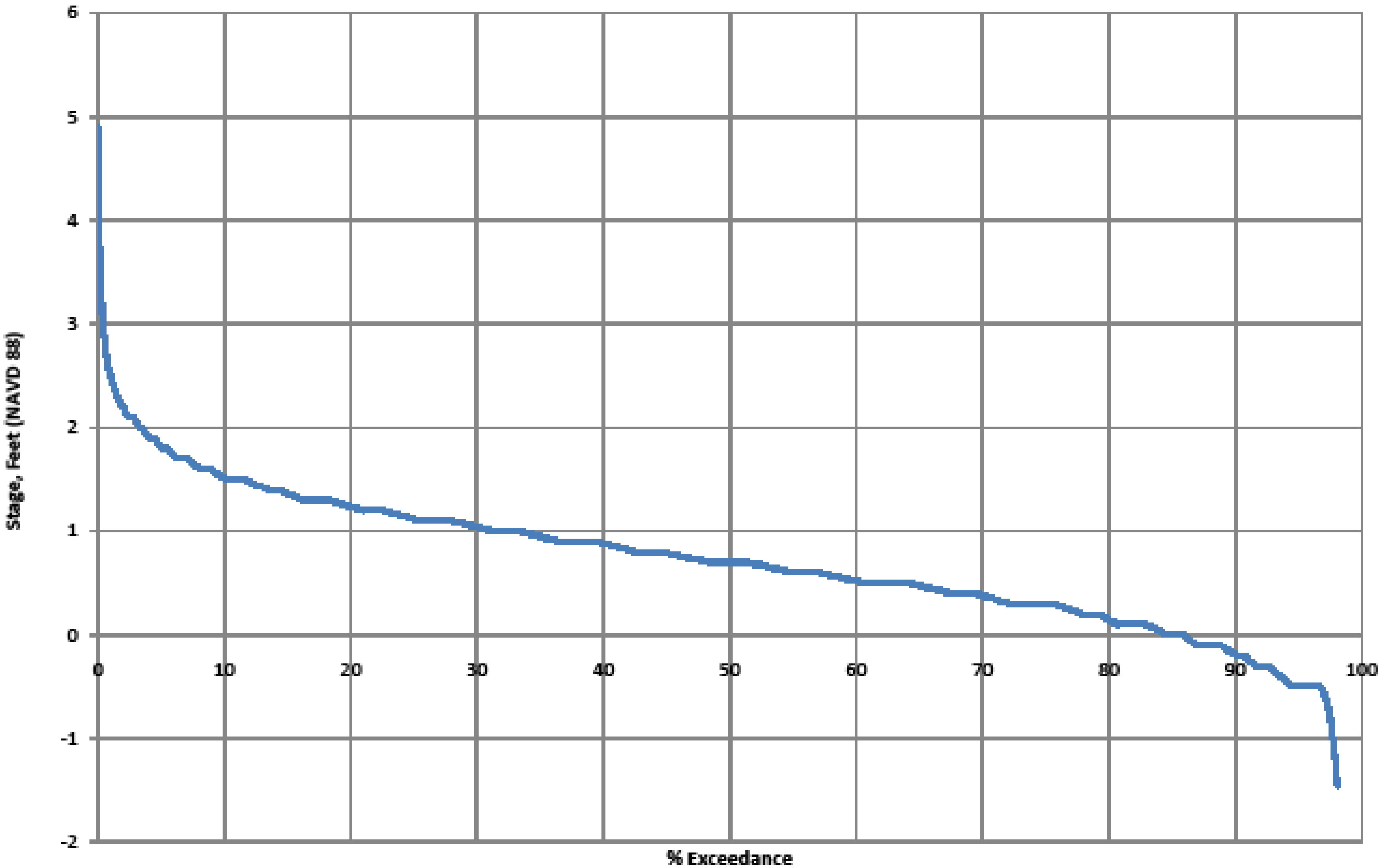



**Figure L2.9.1-3 Stage Percent Exceedance at Pass Manchac**

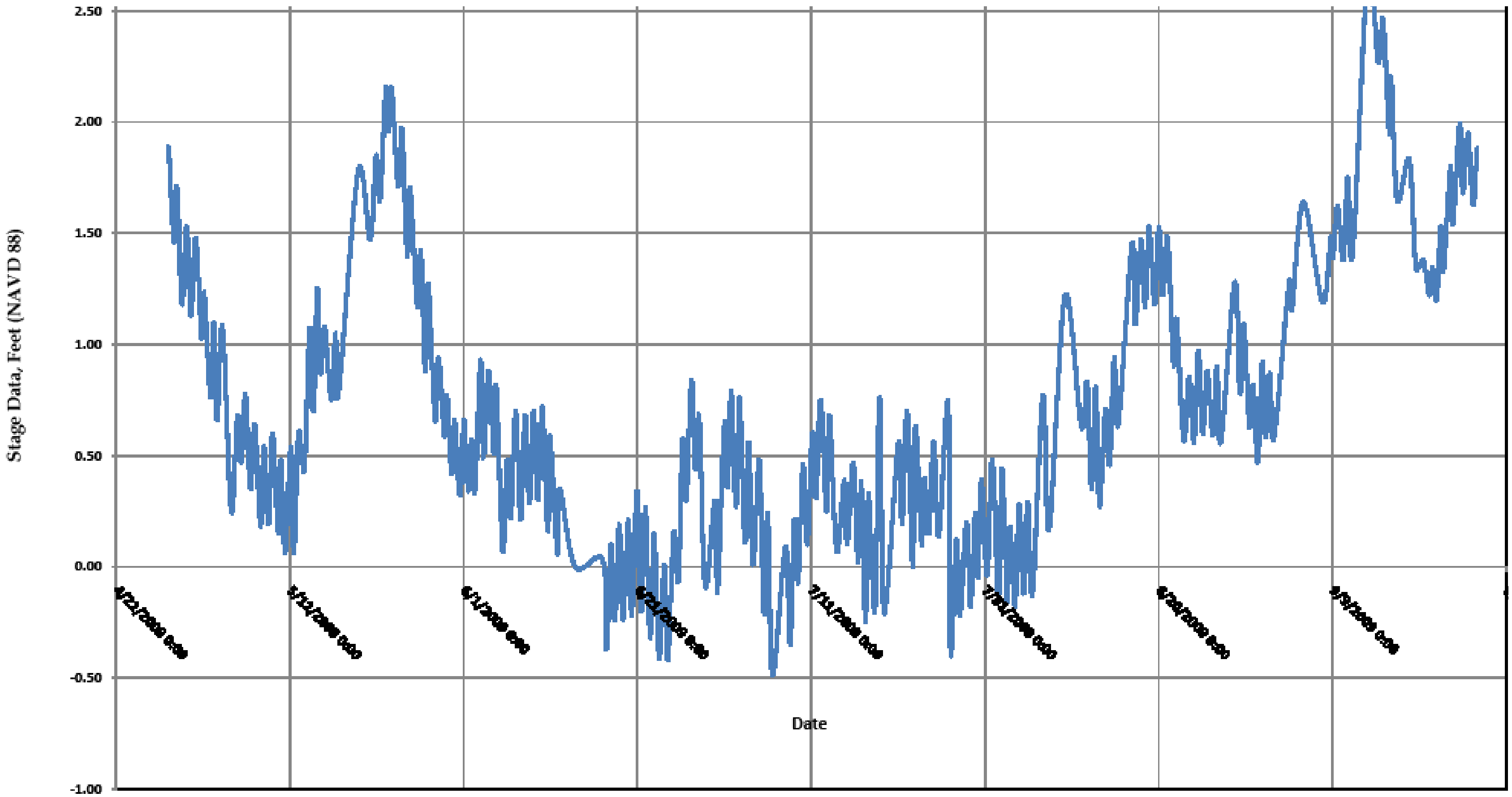



**Figure L2.9.1-4 Tidal Influence at Pass Manchac Gage**

A historic rate considered to be representative of the project area is calculated using the West End at Lake Pontchartrain gage (85625). Daily stage data over the period 1959 to 2009 indicate a rate of 9.20 mm/year (0.0302 feet/year). The standard error of the linear trend line is 0.65 feet. Using the rate of 9.20 mm/year, a starting year of 2011, and a 50-year project life, a sea-level rise of 1.5 feet is projected for the year 2061. The rate of 9.20 mm/year is considered to include both the eustatic and local subsidence contributions to the estimated total sea-level rise. In order to estimate the local subsidence rate for the project area, the global eustatic rate (1.7 mm/yr) is subtracted from the local sea level rate or:

Local subsidence rate = 9.20 mm/yr – 1.7 mm/yr = 7.50 mm/yr.

The estimate for the local subsidence rate is used in conjunction with estimates for the eustatic rates using NRC curves I and III to determine the intermediate and high projections of sea level rise for the project. The following formula is used to estimate the total rise in eustatic sea level for the project life for the intermediate and high rate scenarios of sea level rise:

$$E(t_2) - E(t_1) = 0.0017(t_2 - t_1) + b(t_2^2 - t_1^2)$$

Where:

$b$ is the acceleration factor related to NRC curves I and III or 2.36E-5 and 1.005E-4 respectively,

$t_1$ is the time in years between the project's construction date and 1986,

and

$t_2$ is the time between a future date at which one wants an estimate for sea-level rise and 1986.

These eustatic estimates are added to the local subsidence estimate to calculate the total relative sea-level rise for the intermediate and high rate scenarios.

**Table L2.9.2-1** provides a summary of the estimated total sea-level rise for each of the three scenarios through the project life of 50-years.

**Table L2.9.2-1 Summary of Estimated Total Sea Level Rise (Low to High Rate) for 50-Year Project Life**

| Project Year | Scenario 1 Low Rate (feet) | Scenario 2 Intermediate Rate (feet) | Scenario 3 High Rate (feet) |
|---|---|---|---|
| 2011 | 0.0 | 0.0 | 0.0 |
| 2016 | 0.2 | 0.2 | 0.2 |
| 2021 | 0.3 | 0.3 | 0.5 |
| 2026 | 0.5 | 0.5 | 0.8 |
| 2031 | 0.6 | 0.7 | 1.1 |
| 2036 | 0.8 | 0.9 | 1.4 |
| 2041 | 0.9 | 1.1 | 1.7 |
| 2046 | 1.1 | 1.3 | 2.0 |
| 2051 | 1.2 | 1.5 | 2.4 |
| 2056 | 1.4 | 1.7 | 2.8 |
| 2061 | 1.5 | 1.9 | 3.2 |

### L2.9.3 Application of Relative Sea Level Rise Projection

The final array of alternatives for the LCA Small Diversion at Convent-Blind River project consists of four alternatives. The Tentatively Selected Plan (TSP) will be selected based on wetland valuation assessments coupled with hydrologic, hydraulic, water quality, hydrodynamic, and operations analyses to evaluate the benefits and potential impacts from the alternatives. These benefits were included in an IWR-PLAN analysis that takes into account each alternative's incremental cost versus corresponding average annual habitat units. With the exception of no action, all proposed alternatives involve diversions (re-introduction) of freshwater from the Mississippi River into the swamp and improvements in the swamp to enhance the movement of water through the swamp. All of the proposed alternatives will reintroduce freshwater into the Maurepas Swamp and improve the hydraulic connection between the Blind River and the swamp, thereby allowing inflow to the swamp during periods of high stages and outflow from the swamp into the Blind River and Lake Maurepas during low stages.

From the table above, the projected relative sea-level rise for the 50-year life of the project is 1.5 feet (low), 1.9 feet (Intermediate), and 3.2 feet (High). According to Light Detection and Ranging ( LiDAR) elevations observed within the project area, the average natural ground elevations for the benefit areas range from approximately 0.6 to 1.0 feet referenced to North American Vertical Datum of 1988 (ft-NAVD). These elevations will be verified with ongoing field survey data. The changes in elevations due to the projected relative sea-level rise would affect the project area during low flow and low stage conditions by reducing the duration of dry periods in the swamp and in periods when the elevation of Lake Maurepas is higher than the ground elevation in the swamp by increasing the severity and length of backflow from Lake Maurepas into the swamp.

When considering the hydrology and hydraulics modeling requirements for relative sea-level rise predictions, the following considerations were made. First, all alternatives proposed with the final array include diversions of freshwater from the Mississippi River into the swamp and improvements in the swamp to enhance the movement of water through the swamp. Second, all swamp areas to benefit from the proposed alternatives have a similar elevation. It was also determined that all sea-level rise predictions would affect each alternative in the same manor by reducing the dry periods within the swamp and eventually resulting in a permanently flooded swamp. Finally, because the final two alternatives in the final array provide the same capability to reintroduce freshwater into the swamp (i.e. 3,000 cfs), it could be determined that the cost-effective, incremental cost findings from the IWR-PLAN analysis would be the same for an analysis of the year 2012 and year 2062. This is because the affects of relative sea-level rise would be similar for each alternative regardless of the time period analyzed. Therefore, there would be no value added to incorporating all three levels of relative sea-level rise estimates over the next 50 years to the final array of alternatives. The results of modeling with or without relative sea-level rise would result in the same tentatively selected plan.

To further demonstrate the implications of relative sea level rise on the plan selection process and the tentatively selected plan, the intermediate sea level rise forecast was built into the without- and with- project future conditions as the basis for plan formulation, evaluation, and selection with the low and high forecasts analyzed through sensitivity analysis. The intermediate sea level rise forecast has the additional advantage of incorporating a rate of eustatic sea level rise that accelerates over time, which may likely occur due to accelerated global warming (IPCC, 2007). Additionally, the low and high scenarios bracket the intermediate and capture the range of potential outcomes.

## L2.10 Project Alternatives Analysis

This section presents hydrologic and hydraulic analyses completed to evaluate the final array of project alternatives. Hydrologic and hydraulic analyses that supported the process to formulate the final array of alternatives were discussed in Appendix Section L2.4. Also relevant to evaluation of the project alternatives is the definition of existing conditions hydrology and hydraulics, which was presented in Appendix Sections L2.3 and L2.5. Sections L2.3 and L2.5 also describe anticipated hydrologic and hydraulic impacts to the existing swamp from projected mean sea level rise, which are indicative of future conditions if the Blind River project is not implemented.

The final array of alternatives for the Small Diversion at Blind River project includes four alternatives:

- Alternative 2 – Romeville Diversion at 3,000 cfs;

- Alternative 4 – South Bridge Diversion at 3,000 cfs;
- Alternative 4B – South Bridge Diversion at 3,000 cfs split between the South Bridge Canal and the St. James Parish Canal; and
- Alternative 6 – Romeville and South Bridge Diversions total of 3,000 cfs (1,500 cfs each).

Analyses were completed to understand and quantify the hydrologic benefits of each of the final alternatives. The hydrologic benefits were then incorporated in the Wetland Value Assessment (WVA) to determine the project benefits for each alternative. As discussed in Section L2.2, a multi-tiered approach was employed to complete hydrologic and hydraulic analyses. Analyses were initially completed with HEC-HMS and HEC-RAS for 2003, which represents average hydrologic conditions. The HEC-HMS and HEC-RAS results were used to calculate freshwater throughput and backwater in each project area hydrologic unit for average hydrologic conditions and provide a base of reference for subsequent analyses. Engineering calculations were used to extend the evaluation of each alternative over a 16-year period of record (1989-2004) and evaluate annual average water depth, backflow prevention, frequency of dry-out conditions, frequency of diversion, and loading of total suspended solids (TSS).

Multi-dimensional hydrodynamic analysis was then completed using EFDC to confirm the distribution of flow, nutrients, and sediment for each alternative. For consistency with USACE water resources planning guidelines for civil works programs, all alternatives and existing conditions were evaluated using a medium rate projection for 20-year, 30-year, and 50-year sea level rise conditions. The results and general observations of the analyses are summarized by alternative in the following subsections.

#### L2.10.1 Alternative 2

Alternative 2 includes a freshwater diversion of 3,000 cfs that follows the Romeville alignment. A plan view of Alternative 2 is presented on **Figure L2.10.1-1**. This alternative has six major components:

**Diversion facility at the Mississippi River.** The diversion culvert facility will divert fresh water from the Mississippi River, transfer it beyond the east levee, and discharge to the transmission canal.

**Transmission canal.** The transmission canal will transfer the diverted water approximately three miles from the diversion culvert facility to an existing drainage channel at the perimeter of the swamp. The project will use the existing drainage channels at the perimeter of the swamp to distribute the diverted flow throughout and into the swamp.

**Control Structures.** Approximately six control structures of various sizes with control gates will be installed at key locations in the existing channels to force water out of the drainage channels and into the swamp through the berm gaps.

The proposed control gate is a specialty gate that rotates on a shaft at the bottom of the channel and is operated by large hydraulic cylinders. (One of the gate options, the Obermeyer gates, uses an air-inflated bladder to operate the gates.) The gate will be rotated up to the vertical position to increase the water surface elevation during the flow diversion and promote flow distribution to the swamp. The gate will be rotated down to the channel bottom into the open position when there is no diversion, to allow for normal drainage.

**Berm Gaps.** When the existing drainage channels were excavated in the Swamp, the excavated material was cast to one side of the channel forming spoil banks. In addition, sediment deposition from past flood events has created high ground that blocks flow into and out of the swamp. These man-made and natural obstructions currently block flow circulation into and out of the swamp, resulting in stagnant areas and poor circulation of water through the hydrologic units. In addition to the existing smaller berm gaps, new 500-foot wide berm gaps will be constructed to improve flow circulation in the swamp.

**Highway 61 and KCS Culverts.** New culvert crossings will be added under the KCS RR and Hwy 61 at four locations to improve drainage and flow circulation to areas east of US 61. Each installation will consist of 4 – 6' x 4' reinforced concrete box culverts.

**Instrumentation.** Instrumentation will be required to monitor and control the diversion flow rate and the water surface elevations in the diversion, transmission, and distribution system in the Swamp. Typically, flow rates and water levels will be measured and the feedback data will be used to adjust gate positions to control the desired parameters at the diversion culvert.

#### L2.10.1.1 HEC-HMS and HEC-RAS

HEC-HMS and HEC-RAS were used to simulate hydrologic and hydraulic conditions associated with each project alternative. For the project alternatives, stormwater runoff within the Blind River watershed is unchanged from the existing conditions HEC-HMS model representation presented in Section L2.3. However, the improvements contained in Alternative 2 required the following modification to the system hydraulics in the HEC-RAS model:

- HEC-RAS lateral structures that allow flow exchange between the drainage canals and the swamp were modified to reflect the berm gaps included in this alternative. The geometry of the lateral structures were modified to lower invert elevations to elevation 0 feet NAVD and widened to represent increased flow capacity compared to existing conditions.
- HEC-RAS culverts were added across US 61 to provide more conveyance capacity between swamp storage areas on both sides of the existing highway embankment.

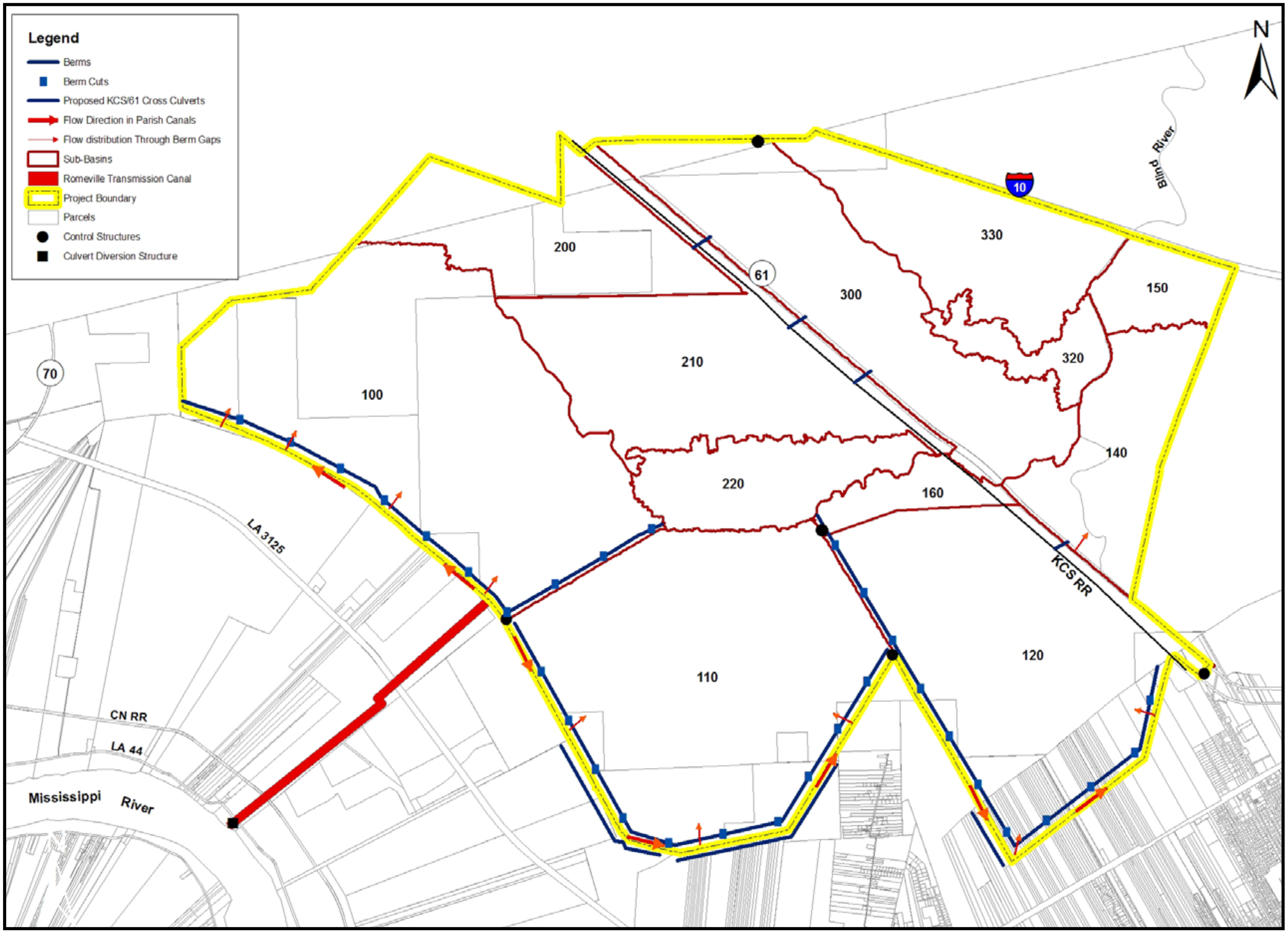



**Figure L2.10.1-1 Alternative 2 Layout**

- Fixed weirs were added in HEC-RAS at the locations of the control structures to simulate dry conditions when diversion flow will be distributed to the swamp. For simulations of wet weather conditions, the weirs in HEC-RAS were modified to represent the control gates in the lowered position.

Alternative 2 also includes a diversion of 3,000 cfs along the Romeville alignment. Approximately 500 feet of the transmission canal was included in the HEC-RAS model and the diversion flow was added in HEC-RAS as a boundary condition flow time series.

As presented in Section L2.3, two types of HEC-RAS simulations were conducted. Model results produced with the simulation of the year 2003 are intended to establish a swamp hydroperiod with greater fluctuation than the existing hydroperiod. Model results produced with design rainfall depths are intended to define peak water surface conditions in the study area to identify potential adverse drainage impacts from the project. For the purpose of understanding hydrologic benefits to the project area ecosystem, net freshwater throughput and backflow in acre-feet were calculated for each project hydrologic unit from the simulation of 2003, an average hydrologic year. A comparison of Alternative 2 results with existing conditions is presented in **Table L2.10.1-1** and **Table L2.10.1-2**. The net freshwater throughput is calculated as the total inflow to each hydrologic unit minus the inflow volume attributed to backflow from Lake Maurepas. Backflow to each hydrologic unit is the hydrologic unit inflow that coincides with reverse flow in the Blind River due to backflow from Lake Maurepas. **Figure L2.10.1-2** provides a comparison of the net freshwater throughput and backflow for all four alternatives in the final array and existing conditions. Throughput and backflow values are also included for existing and future sea level conditions.

Review of the HEC-HMS and HEC-RAS results indicates that Alternative 2 increases net freshwater throughput compared to existing conditions. Backflow is also reduced, especially with projected increases in sea level. The throughput and backflow values for Alternative 2 are comparable to the other alternatives in the final array. Total system throughput is slightly higher than for the other alternatives because more frequent diversions are required for Alternative to counter backflow. Alternative 2 provides the most throughput to volume to Subbasins 110, 100, and 120, which are all located south of the Blind River. Alternative 2 also provides improved throughput to Subbasin 140.

**Table L2.10.1-1 Alternative 2 HEC-RAS Net Freshwater Throughput Volumes**

| Sea Level Condition | Net Freshwater Throughput (Acre-feet) by Subbasin Number and Hydrologic Unit | | | | | | |
|---|---|---|---|---|---|---|---|
| | 100 | 200 | 210, 220 | 110 | 120, 160 | 300, 320, 330 | 140, 150 |
| | HU 1 | HU 2 | HU 3 | HU 4 | HU 5 | HU 6 | HU 7 |
| Existing | 20,700 | 47,400 | 55,900 | 9,400 | 3,900 | 172,100 | 98,300 |
| Berm Gaps (No Diversion) | 94,000 | 33,800 | 34,000 | 78,100 | 64,200 | 172.700 | 93,900 |
| Alternative 2 | 420,500 | 96,300 | 152,700 | 651,800 | 459,400 | 267,000 | 153,100 |

Note: values based on simulation of hydrologic conditions observed in 2003.

**Table L2.10.1-2 Alternative 2 HEC-RAS Backflow Volumes**

| Sea Level Condition | Backflow (Acre-feet) by Subbasin Number and Hydrologic Unit | | | | | | |
|---|---|---|---|---|---|---|---|
| | 100 | 200 | 210, 220 | 110 | 120, 160 | 300, 320, 330 | 140, 150 |
| | HU 1 | HU 2 | HU 3 | HU 4 | HU 5 | HU 6 | HU 7 |
| Existing | 7,900 | 11,500 | 7,000 | 5,400 | 5,200 | 42,400 | 24,000 |
| Berm Gaps (No Diversion) | 7,300 | 17,800 | 20,900 | 4,900 | 4,800 | 41,800 | 28,300 |
| Alternative 2 | 6,900 | 2,100 | 1,700 | 4,100 | 3,900 | 41,800 | 28,300 |

Note: values based on simulation of hydrologic conditions observed in 2003.

### L2.10.1.2 EFDC

Appendix Section L2.5 presented hydrodynamic analysis for existing conditions. This section presents analysis completed using EFDC to evaluate hydrodynamic performance for Alternative 2. As with the HEC-RAS model, the EFDC model for existing conditions was modified to represent the improvements and diversion flow included in the alternative. Refinements incorporated into the model include representation of berm gaps along existing berms adjacent to the drainage canals with evenly distributed spacing and a constant width of 500 ft. Because of the resolution of the model grid, the EFDC model cannot directly simulate flow through the berm cuts; therefore, HEC-RAS simulated stage and flow were used to develop a head difference and flow rating table for each berm cut, which was used in the EFDC model.

For the Alternative 2 EFDC simulations, the downstream stage boundaries at the Conway Canal and Blind River at I-10 Bridge were updated with the HEC-RAS simulated stages which are higher than those in the existing conditions because of additional diversion flow of 3,000 cfs. The Alternative 2 simulation ran the first 300 days of 2003, and results are summarized and discussed in the following sections.

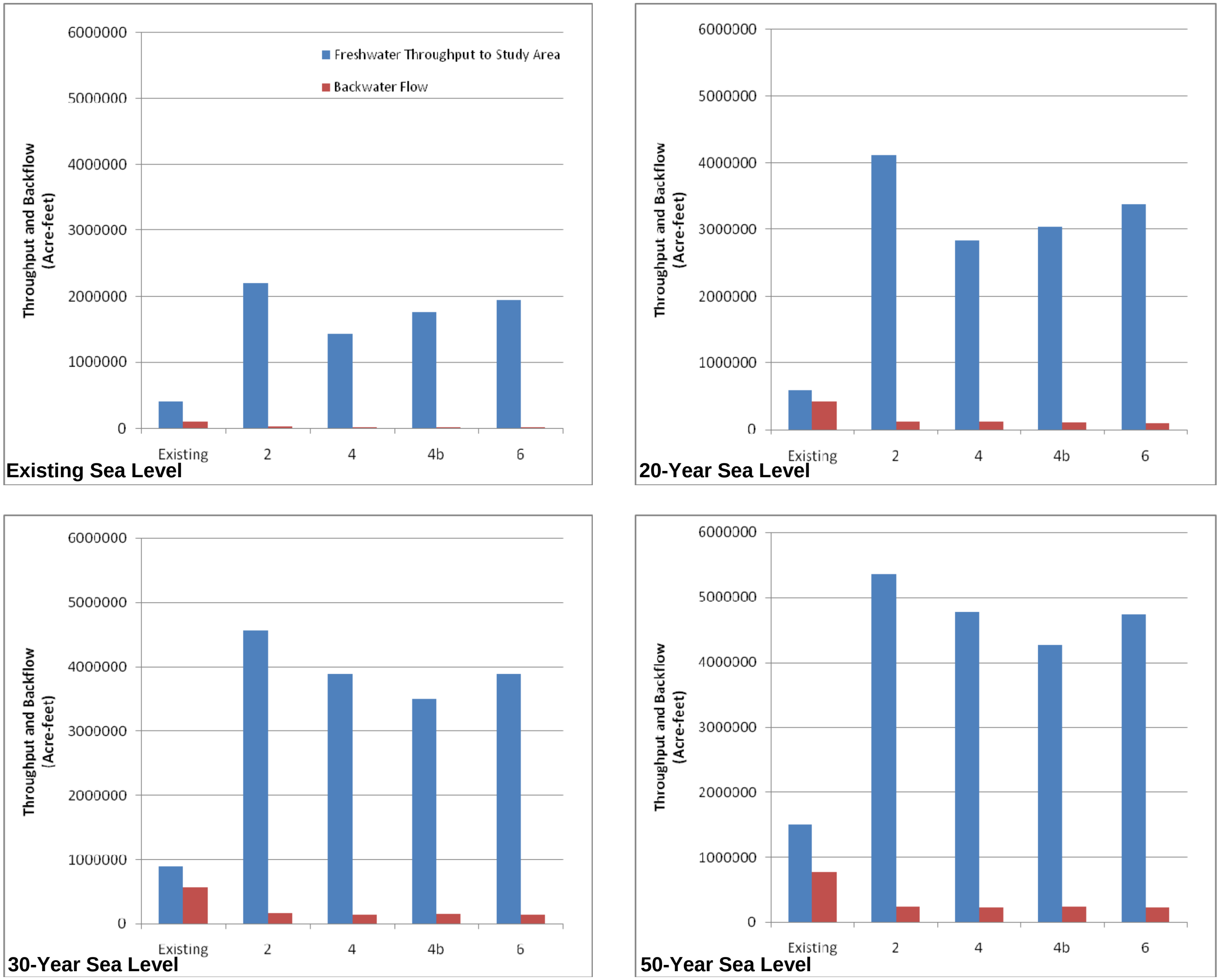



**Figure L2.10.1-2 Freshwater Throughput and Backflow for the Final Array of Alternatives**

**Flow Velocity**

Particular focus during the EFDC analysis was placed on evaluating flow velocity, hydraulic residence time and sediment distribution. **Table L2.10.1-3** summarizes the EFDC results for these parameters with Alternative 2. Because of additional 3,000 diversion flow, the average wetland flow velocity for each subbasin in Alternative 2 as shown in Table L2.10.1-3 is more than one order of magnitude larger than for existing conditions. As expected, the highest average flow velocity 13,948 ft/day occurred in the Subbasin 150, which is adjacent to the river exit near I-10 Bridge while the lowest average flow velocity 1,539 ft/day occurred in the Subbasin 120. Compared to existing conditions, the average velocity in Subbasin 110 increased more than 29 times, while the average velocity in the Subbasin 140 only increased about five times. The spatial distribution of flow velocity at day 300 (day 1= 1/1/2003) as shown on **Figure L2.10.1-3** indicated that the highest flow velocity occurred at the diversion flow entry point and near the river exit at I-10 Bridge.

**Table L2.10.1-3 Alternative 2 EFDC Summary Results**

| Subbasin | Velocity (ft/day) | Sediment Volume (cubic yards) | Hydraulic Residence Time (days) |
|---|---|---|---|
| All | 3,971 | -9.24E+04 | - |
| 100 | 2,948 | 5.54E+04 | 13.5 |
| 110 | 4,610 | 5.06E+04 | 7.5 |
| 120 | 1,539 | 3.01E+03 | 4.2 |
| 140 | 2,065 | -3.11E+00 | 2.3 |
| 150 | 13,948 | -2.12E+05 | 1.5 |
| 160 | 5,934 | 0.00E+00 | 2.8 |
| 200 | 4,336 | 7.58E+02 | 9.8 |
| 210 | 4,497 | 7.11E+03 | 6.6 |
| 220 | 5,557 | 2.46E+03 | 4.3 |
| 300 | 3,861 | 4.57E+02 | 6.6 |
| 320 | 5,432 | 0.00E+00 | 10.2 |
| 330 | 5,453 | 3.12E+01 | 2.4 |

**Hydraulic Residence Time**

Compared to existing conditions, the hydraulic residence time (HRT) for each subbasin in Alternative 2 reduced significantly due to the increased flow velocity as discussed in the above section. The largest HRT (13.5 days) resulted in Subbasin 100 and the smallest HRT (1.5 days) resulted in Subbasin 150, as shown in Table L2.10.1-3.

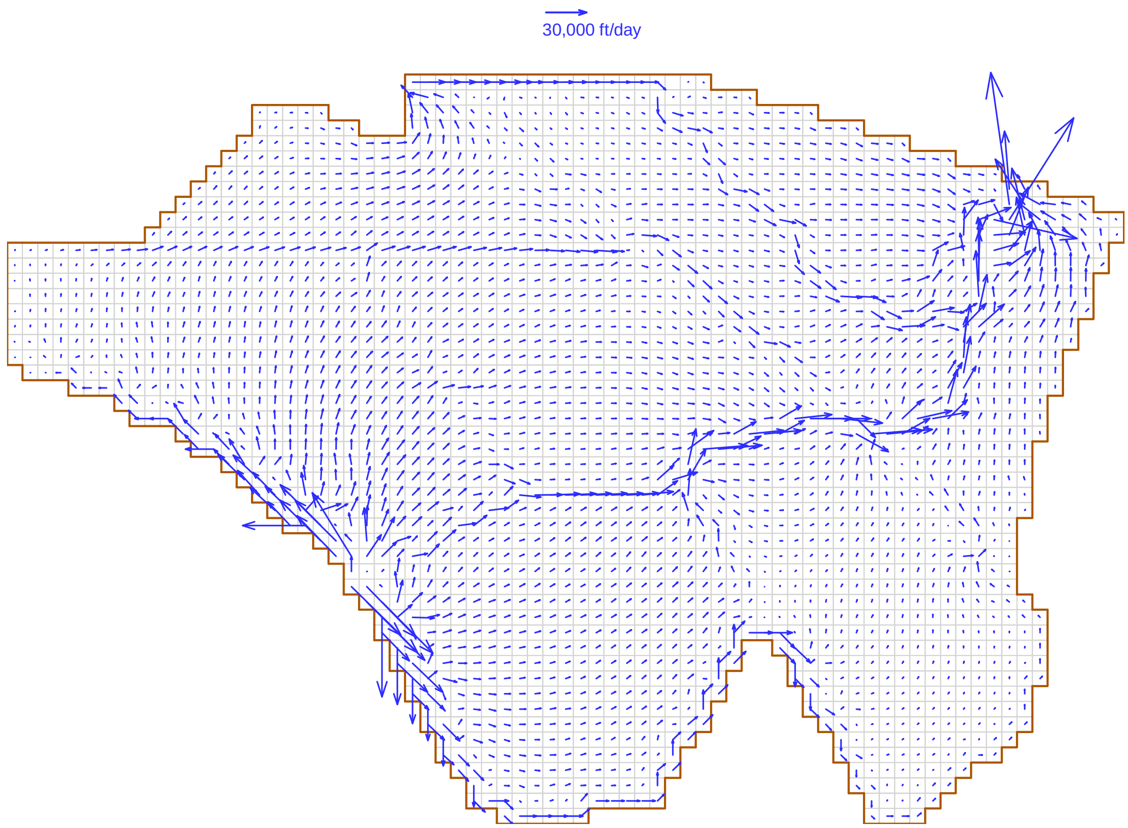



**Figure L2.10.1-3 Alternative 2 EFDC Flow Velocity**

Based on the simulated HRT for each HRU, it is suggested that the constant diversion flow of 3,000 cfs should be introduced into the project area periodically such that the HRT can be increased to achieve an optimal HRT for each HRU for purpose of wetland restoration,

**Figure L2.10.1-4** shows the dye concentration plots at I-10 Bridge on the Blind River for the 12 HRUs. Unlike in the Existing Conditions, the well-defined bell-shaped dye plume in Alternative 2 exited completely from the I-10 Bridge about one month after release. For each subbasin, the HRT was estimated as the time when the peak dye concentration passed the I-10 Bridge on the Blind River minus the time when the dye was released in each subbasin.

Compared to the existing conditions HRTs shown on Figure L2.5-11, the HRTs for Alternative 2 on **Figure L2.10.1-5** indicates that the HRTs at the model cell scale are significantly reduced. It should be pointed that the color shaded HRT scale bar in Figure L2.10.1-6 is different from that on Figure L2.5-11 and the gray shaded area on Figure L2.10.1-5 indicates that the HRT is higher than eight hours.

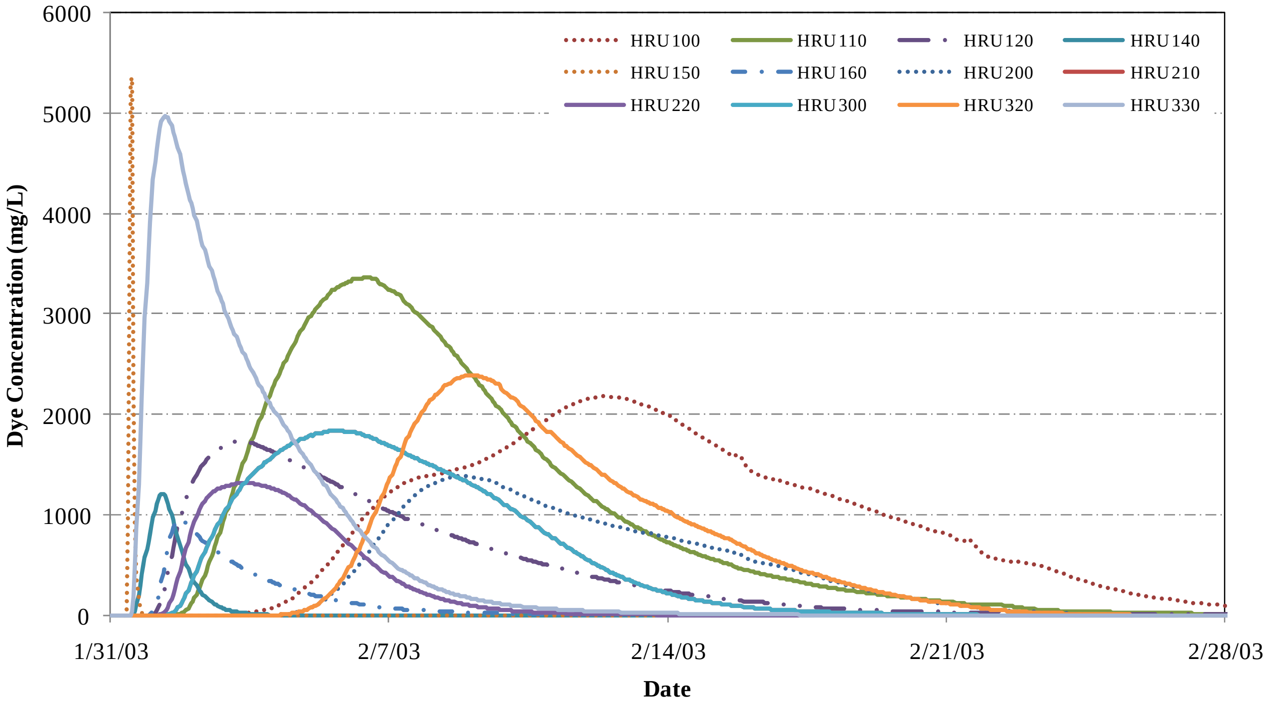



**Figure L2.10.1-4 Alternative 2 EFDC Simulated Dye Concentration for Blind River at I-10**

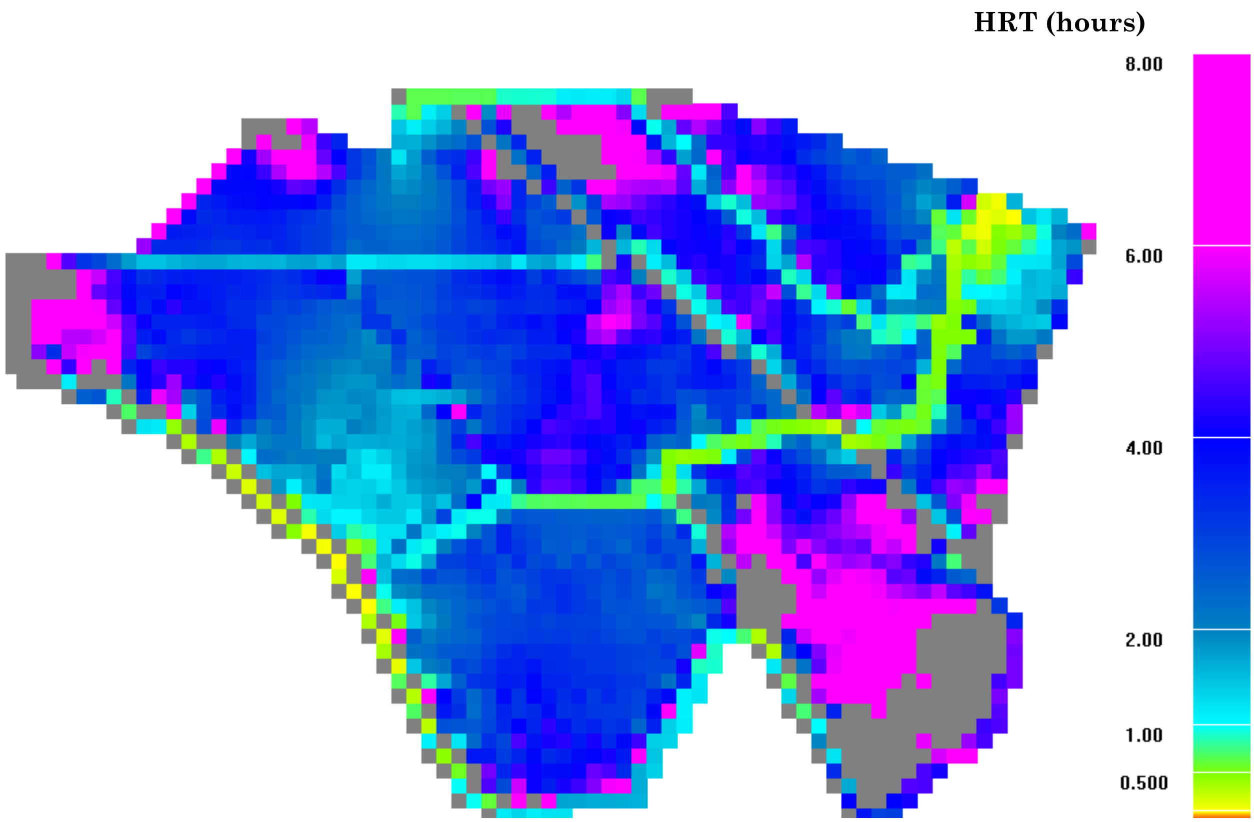



**Figure L2.10.1-5 Alternative 2 EFDC Hydraulic Residence Time**

**Sediment**

For Alternative 2, the diversion flow introduced an increase in sediment compared to existing conditions using the sediment concentration basis presented in Section L2.5. With the current sediment model using five key sediment parameter values shown in Table L2.5-7, the simulated sediment deposition as shown in Table L2.10.1-3, Alternative 2 produced sediment deposition in Subbasins 100, 110, 120, 210, and 220, and less sediment deposition in the HRUs 200, 300, and 330. Spatial distribution of sediment cumulative deposition and erosion for Alternative 2 is presented on **Figure L2.10.1-6**.

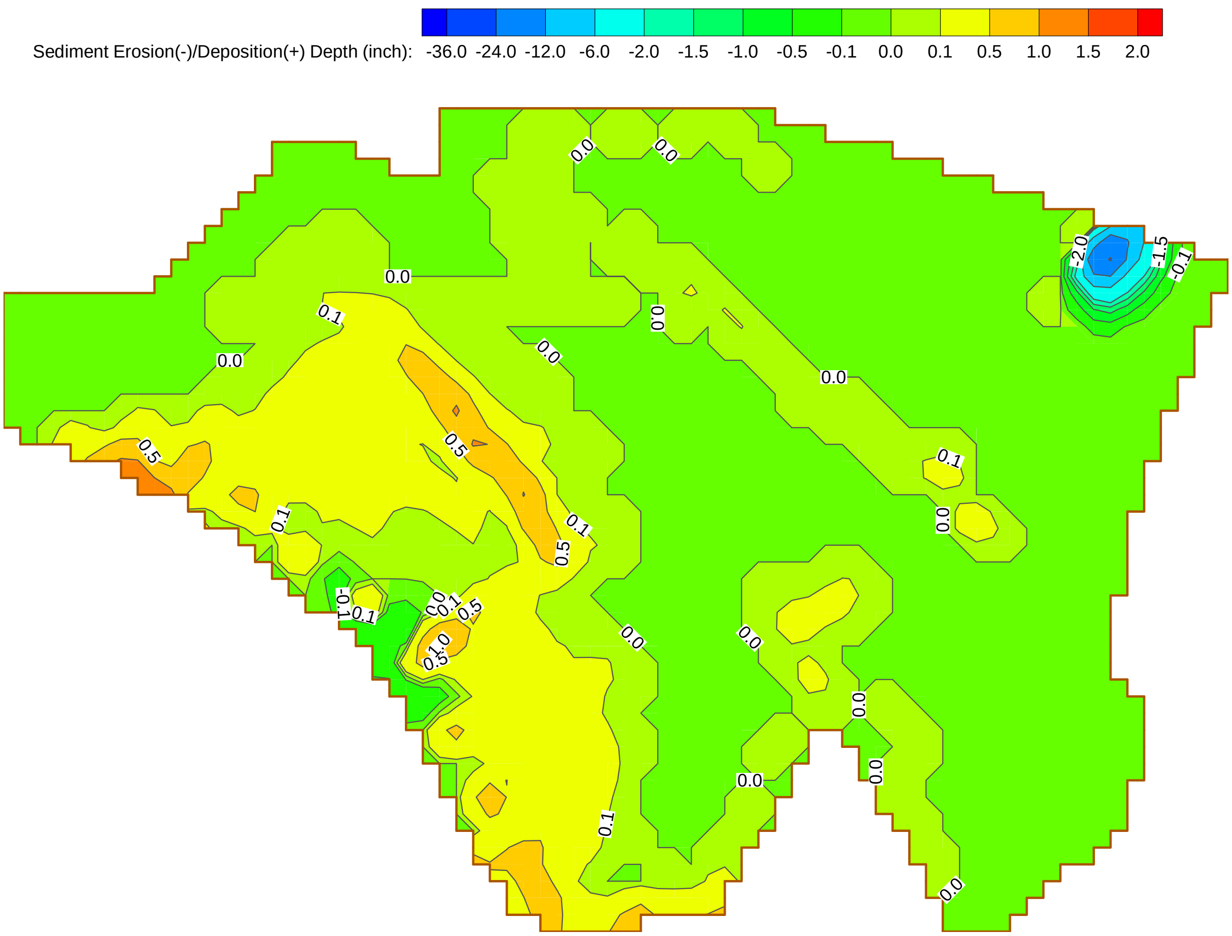



**Figure L2.10.1-6 Alternative 2 EFDC Sediment Deposition and Erosion**

**Water Quality**

The average or range of nitrate, ammonium, TN, and TP concentrations reported by Lane et al. (1999) for the Mississippi River compared well with the data collected at Belle Chasse on the Mississippi River. The average concentrations of nitrate, TN, and TP (1.73, 2.26, and 0.22 mg/L, respectively) were used for the diversion flow in the water quality analysis.

Due to the diversion flow over time, the water quality within the swamp and downstream of the swamp will inevitably change. There have been many studies of the relationship between the nutrient loading rate into wetlands and associated removal efficiency. The nutrient (nitrate, TN, and TP) removal efficiency in wetlands primarily depends on the nutrient loading rate and the HRT.

For nitrate, the average nitrogen removal efficiency ranges from 95 to 100 percent when the nitrate:ammonium ration is greater than 1 and loading rate is relatively low (e.g., less than 10 g-N/m$^2$/yr). The Mississippi River has an average molar nitrate:ammonium ratio of 18 (Lane et al., 1999). Therefore, the removal efficiency of nitrogen in the swamp is expected to be very high. For TN, the average removal efficiency ranges from 50 to 65 percent and for TP, the average removal efficiency ranges from 20 to 35 percent when the average HRT is about seven days.

For this project, the average removal efficiencies for nitrate, TN, and TP were estimated to be 95 percent, 65 percent, and 30 percent, respectively, and the average HRT to achieve the removal efficiencies was estimated to be seven days (Kadlec and Wallace, 2008). Therefore, the first-order decay rates were calculated to be 4.95E-06, 1.74E-06, and 5.89E-07 second$^{-1}$, respectively. To evaluate how the swamp benefits from the nutrient loadings associated with diversion flow, the first-order decay model was used to evaluate nutrient load reduction and removal efficiency of TP, TN, and nitrate. As discussed in Section L2.5.8.5, the average nitrate, TN, and TP concentration data (0.008, 0.58, and 0.055 mg/L, respectively) were used as initial background water quality concentrations.

For the first-order decay model, the average concentration in each HRU and overall removal efficiency are summarized in **Table L2.10.1-4.** The removal efficiency is defined as:

$$\text{Removal Efficiency (RE)} = (C_i - C_o)/C_i \times 100\%$$

Where $C_i$ is swamp inflow concentration (mg/L) and $C_o$ is swamp outflow concentration (mg/L.)

The simulation results indicate that very high removal efficiencies can be achieved for nitrate and TN (99.4 percent and 90.6 percent), while TP removal efficiency was estimated as high as 66.5 percent. Spatial distributions of nitrate, TN, and TP concentrations presented on **Figures L2.10.1-7**, **L2.10.1-8,** and **L2.10.1-9**, respectively, indicate that HRUs 330, 320, 300, 150, 140, and 120 will not benefit much from the nutrients brought by the diversion flow compared to other HRUs in the swamp.

**Table L2.10.1-4 Average Concentration in each HRU and Overall Removal Efficiency of TP, TN, and Nitrate**

| Hydrologic Response Unit | Average Concentration | | | Overall Removal Efficiency (%) | | |
|---|---|---|---|---|---|---|
| | $NO_3$ (mg/L) | TN (mg/L) | TP (mg/L) | $NO_3$ | TN | TP |
| All | 0.292 | 0.903 | 0.141 | 99.4 | 90.6 | 66.5 |
| 100 | 0.732 | 1.563 | 0.187 | - | - | - |
| 110 | 0.563 | 1.402 | 0.180 | - | - | - |
| 120 | 0.060 | 0.511 | 0.111 | - | - | - |
| 140 | 0.031 | 0.312 | 0.083 | - | - | - |
| 150 | 0.034 | 0.361 | 0.093 | - | - | - |
| 160 | 0.166 | 0.904 | 0.153 | - | - | - |
| 200 | 0.054 | 0.556 | 0.123 | - | - | - |
| 210 | 0.208 | 0.986 | 0.160 | - | - | - |
| 220 | 0.320 | 1.181 | 0.172 | - | - | - |
| 300 | 0.041 | 0.408 | 0.102 | - | - | - |
| 320 | 0.023 | 0.366 | 0.102 | - | - | - |
| 330 | 0.023 | 0.258 | 0.079 | - | - | - |

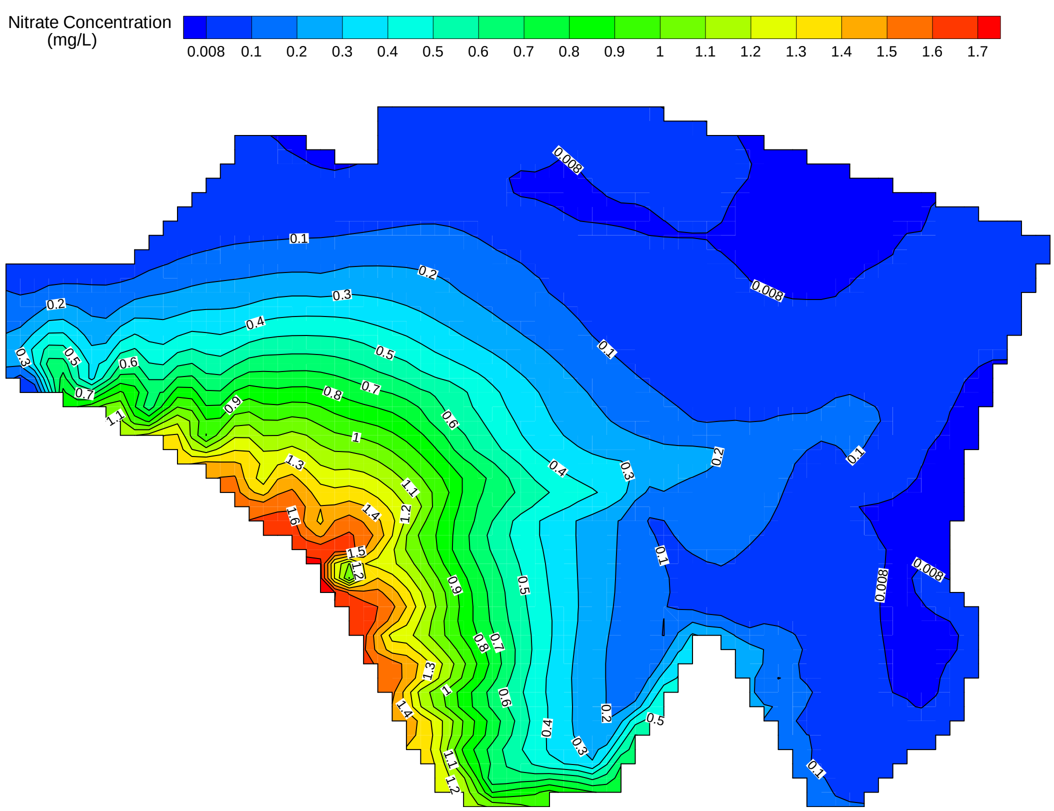



**Figure L2.10.1-7 Spatial distribution of NO3 Concentration at Day 300 for Alternative 2**

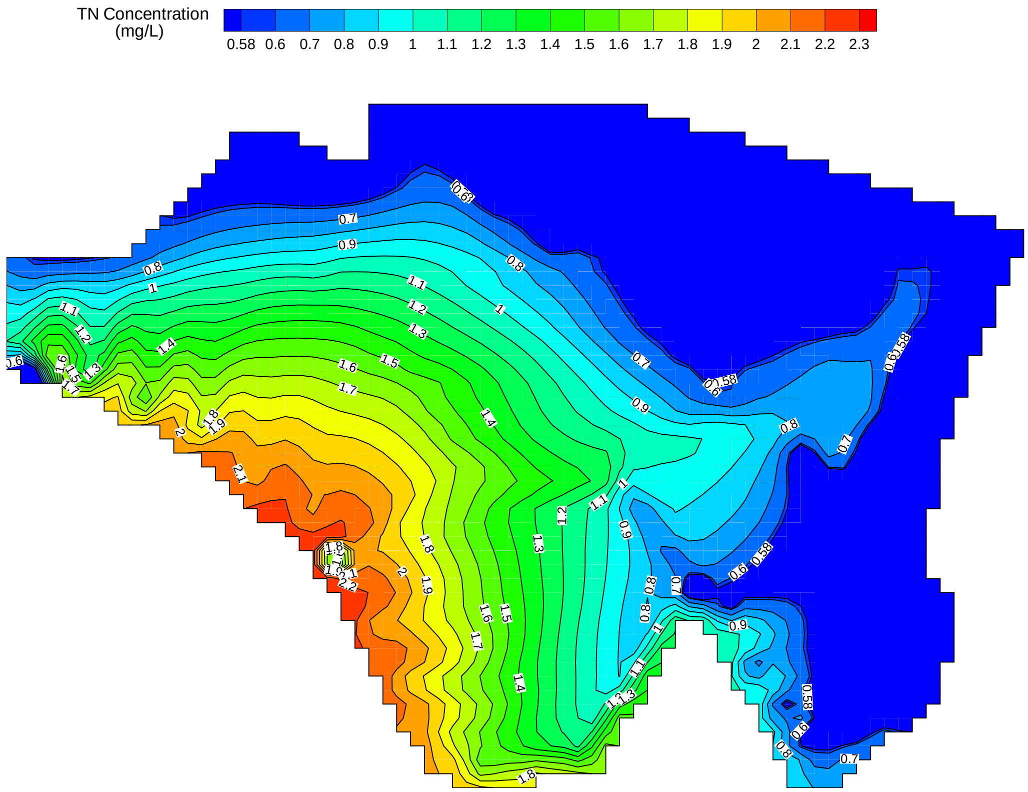



**Figure L2.10.1-8 Spatial Distribution of TN Concentration at Day 300 for Alternative 2**

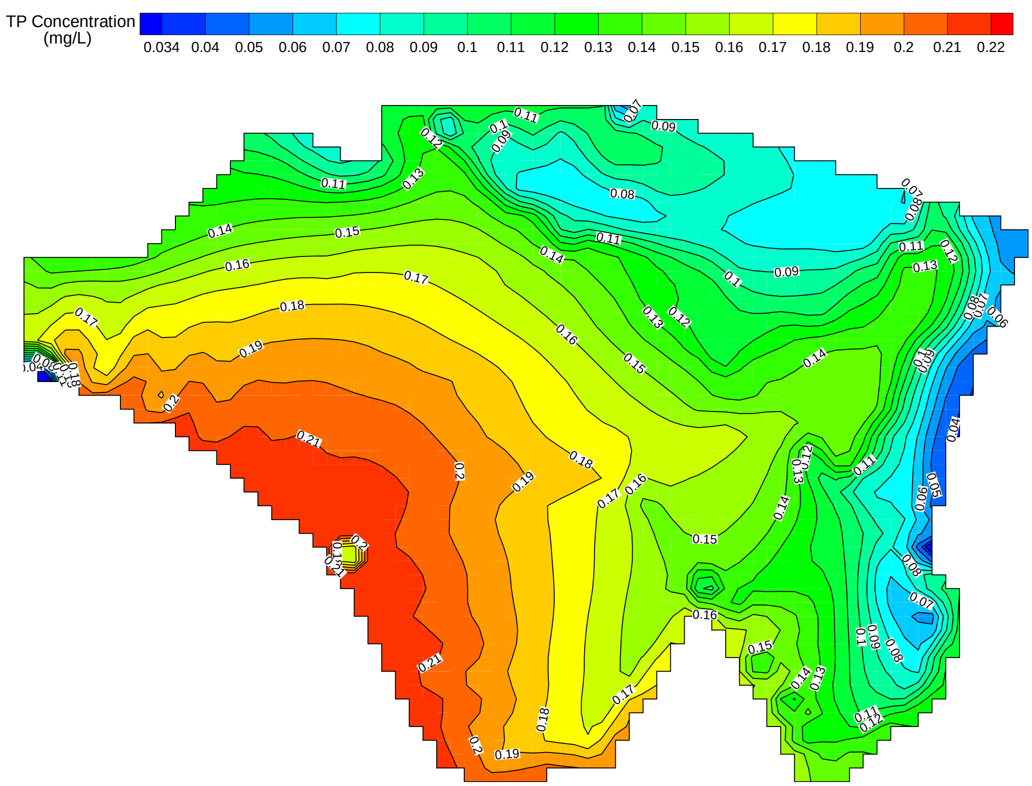



**Figure L2.10.1-9 Spatial Distribution of TP Concentration at Day 300 for Alternative 2**

**Sea Level Rise**
The average water depth and elevation increased as expected with increased sea level. Presented in **Table L2.10.1-5**, average velocity for each subbasin also decreased with sea level rise. Reduction of the average velocity in Alternative 2 resulted from a smaller head gradient due to the increased tail water from future sea levels. However, Alternative 2 did not experience the increases in reverse flow that were simulated for the future without project conditions as a result of higher tail water.

**Table L2.10.1-5 Alternative 2 EFDC Flow Velocity with Sea Level Rise**

| Hydrologic Response Unit | Alternative 2 | | | | Existing Conditions - 2003 | | | |
|---|---|---|---|---|---|---|---|---|
| | Current seal level | 20-year sea level rise | 30-year sea level rise | 50-year sea level rise | Current seal level | 20-year sea level rise | 30-year sea level rise | 50-year sea level rise |
| All | 3,971 | 3,779 | 3,635 | 3,276 | 256 | 255 | 369 | 572 |
| 100 | 2,948 | 2,729 | 2,582 | 2,254 | 154 | 154 | 175 | 234 |
| 110 | 4,610 | 4,247 | 3,990 | 3,389 | 158 | 157 | 231 | 327 |
| 120 | 1,539 | 1,534 | 1,521 | 1,458 | 149 | 149 | 174 | 263 |
| 140 | 2,065 | 2,428 | 2,575 | 2,710 | 416 | 414 | 651 | 1,029 |
| 150 | 13,948 | 13,861 | 13,648 | 12,884 | 2,127 | 2,129 | 2,869 | 4,281 |
| 160 | 5,934 | 6,083 | 6,171 | 6,329 | 258 | 256 | 606 | 1,310 |
| 200 | 4,336 | 3,923 | 3,652 | 3,066 | 263 | 262 | 356 | 479 |
| 210 | 4,497 | 4,285 | 4,137 | 3,788 | 200 | 198 | 360 | 608 |
| 220 | 5,557 | 5,390 | 5,269 | 4,959 | 318 | 316 | 584 | 982 |
| 300 | 3,861 | 3,525 | 3,282 | 2,714 | 294 | 293 | 391 | 572 |
| 320 | 5,432 | 5,364 | 5,274 | 4,967 | 357 | 354 | 632 | 1,163 |
| 330 | 5,453 | 5,177 | 4,977 | 4,499 | 383 | 381 | 602 | 1,061 |

With the rise of sea level, thus the tail water, the HRT for each subbasin changed and responded differently in Alternative 2 than for the future without project conditions. **Table L2.10.1-6** summarizes the HRT simulated for both existing conditions and Alternative 2 for each subbasin with existing sea level and 50-year sea level. **Figure L2.10.1-10** indicates that the HRTs at the model cell scale.

As with HRT, sediment transport within of the project area also responded differently for Alternative 2 and existing conditions, which is shown in **Table L2.10.1-7**. For existing conditions some sediment deposition occurred in subbasins 140, 150, 300, 320, and 330 with increase of the tail water. Due to the reverse flow, the sediment of the Blind River attributed to the deposition inside of the subbasins near to the river exit at I-10 Bridge. For Alternative 2, the extent of sediment deposition generally reduced compared to existing sea level due to the reduced flow velocity that resulted from the higher tail water with future sea level rise. Alternative 2 still shows increased deposition compared to existing conditions with sea level rise, and does not display the reverse flow characteristics simulated for existing conditions. **Figure L2.10.1-11** shows the extent of sediment deposition for Alternative 2 with 50-year sea level rise.

**Table L2.10.1-6 Alternative 2 EFDC Hydraulic Residence Time with Sea Level Rise**

| Hydrologic Response Unit | Alternative 2 | | Existing Conditions - 2003 | |
|---|---|---|---|---|
| | current sea level | 50-year seal level rise | current sea level | 50-year seal level rise |
| 100 | 13.5 | 13.0 | 42.0 | 83.6 |
| 110 | 7.5 | 6.7 | 37.8 | 65.6 |
| 120 | 4.2 | 5.2 | 37.4 | 44.7 |
| 140 | 2.3 | 2.2 | 37.4 | 6.3 |
| 150 | 1.5 | 1.5 | 8.1 | 2.4 |
| 160 | 2.8 | 2.8 | 37.4 | 32.6 |
| 200 | 9.8 | 9.1 | 38.3 | 81.9 |
| 210 | 6.6 | 6.4 | 38.3 | 65.6 |
| 220 | 4.3 | 3.9 | 37.8 | 65.5 |
| 300 | 6.6 | 3.4 | 37.4 | 32.6 |
| 320 | 10.2 | 1.9 | 37.2 | 6.5 |
| 330 | 2.4 | 2.3 | 37.4 | 32.6 |

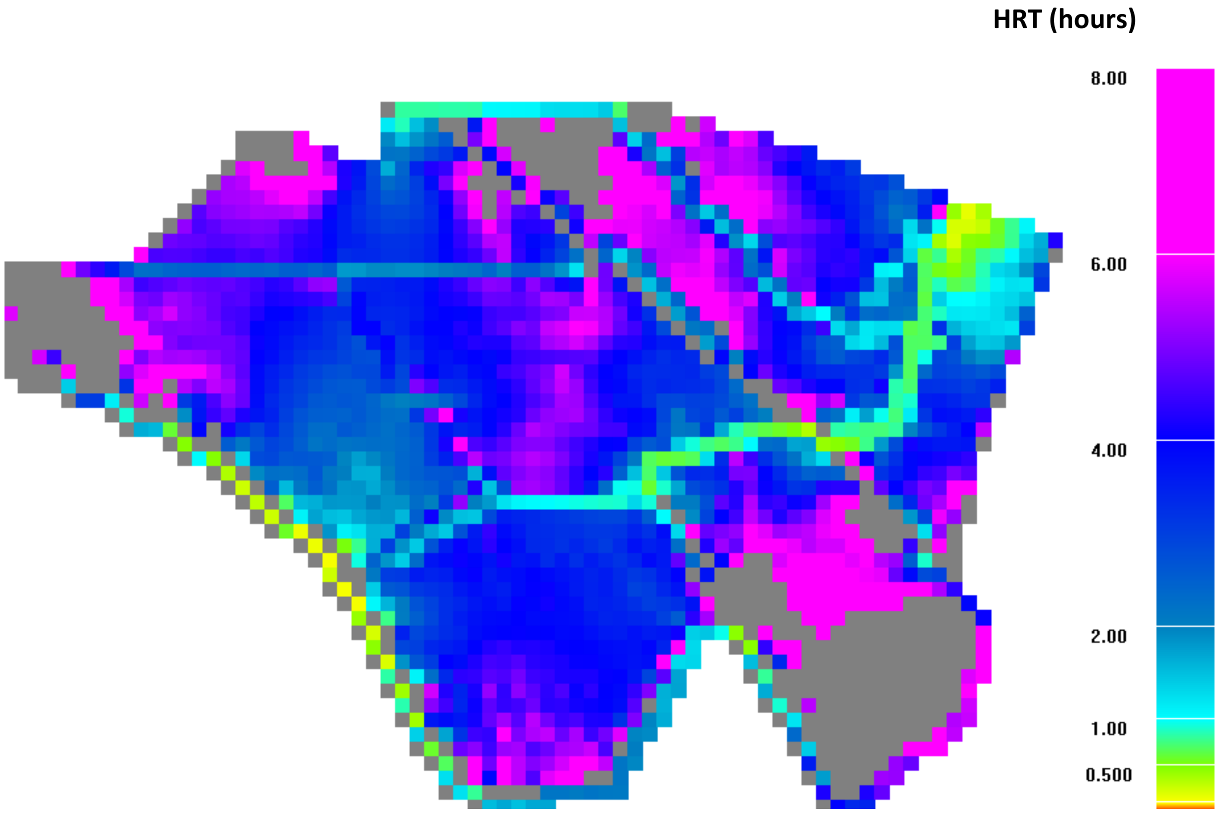



**Figure L2.10.1-10 Alternative 2 EFDC Hydraulic Residence Time with Sea Level Rise (50 years)**

**Table L2.10.1-7 Alternative 2 EFDC Sediment Transport Summary with Sea Level Rise**

| Hydrologic Response Unit | Alternative 2 | | | | Existing Conditions - 2003 | | | |
|---|---|---|---|---|---|---|---|---|
| | current seal level | 20-year sea level rise | 30-year sea level rise | 50-year sea level rise | current seal level | 20-year sea level rise | 30-year sea level rise | 50-year sea level rise |
| All | -9.24E+04 | -7.79E+04 | -5.55E+04 | 4.41E+03 | -2.78E+04 | -2.78E+04 | -2.06E+04 | -1.64E+04 |
| 100 | 5.54E+04 | 5.69E+04 | 5.88E+04 | 6.71E+04 | -6.24E+00 | -6.24E+00 | -5.72E+00 | 0.00E+00 |
| 110 | 5.06E+04 | 5.27E+04 | 5.69E+04 | 6.71E+04 | 0.00E+00 | 0.00E+00 | 0.00E+00 | 0.00E+00 |
| 120 | 3.01E+03 | 2.03E+03 | 1.32E+03 | 1.09E+02 | -1.09E+01 | -1.04E+01 | -1.45E+01 | -1.56E+00 |
| 140 | -3.11E+00 | -6.87E+03 | -8.20E+03 | -6.25E+03 | -1.71E+02 | -1.68E+02 | -7.94E+01 | 5.49E+02 |
| 150 | -2.12E+05 | -1.94E+05 | -1.76E+05 | -1.36E+05 | -2.76E+04 | -2.77E+04 | -2.04E+04 | -1.69E+04 |
| 160 | 0.00E+00 | 0.00E+00 | 0.00E+00 | 0.00E+00 | 0.00E+00 | 0.00E+00 | 0.00E+00 | 0.00E+00 |
| 200 | 7.58E+02 | 5.04E+02 | 7.27E+01 | 0.00E+00 | -1.66E+01 | -1.61E+01 | -3.01E+01 | -8.83E+00 |
| 210 | 7.11E+03 | 8.39E+03 | 9.01E+03 | 8.83E+03 | 0.00E+00 | 0.00E+00 | 0.00E+00 | 0.00E+00 |
| 220 | 2.46E+03 | 2.34E+03 | 2.44E+03 | 3.94E+03 | 0.00E+00 | 0.00E+00 | 0.00E+00 | 0.00E+00 |
| 300 | 4.57E+02 | 1.61E+02 | 4.67E+01 | 2.60E+01 | 0.00E+00 | 0.00E+00 | 0.00E+00 | 2.08E+01 |
| 320 | 0.00E+00 | 0.00E+00 | 0.00E+00 | 0.00E+00 | 0.00E+00 | 0.00E+00 | 5.19E+00 | 1.56E+01 |
| 330 | 3.12E+01 | 0.00E+00 | 0.00E+00 | 0.00E+00 | 0.00E+00 | 0.00E+00 | 5.19E+00 | 5.19E+00 |

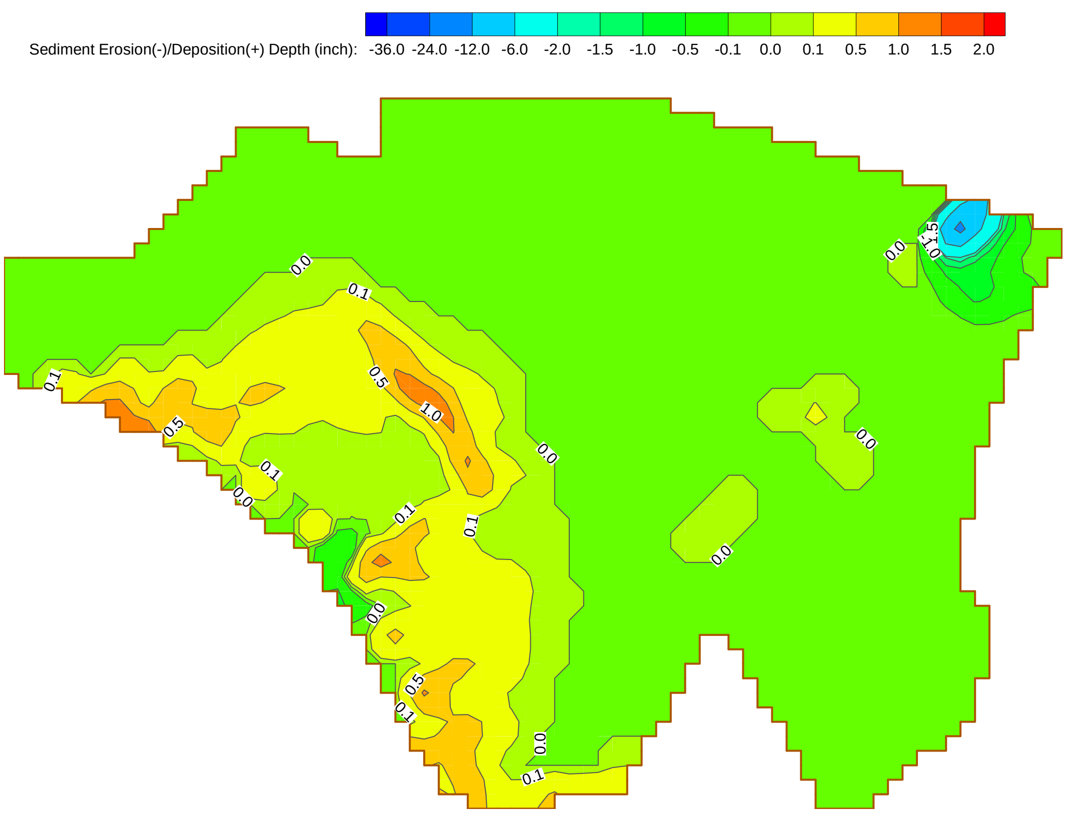



**Figure L2.10.1-11 Alternative 2 EFDC Sediment Deposition and Erosion with Sea Level Rise (50 years)**

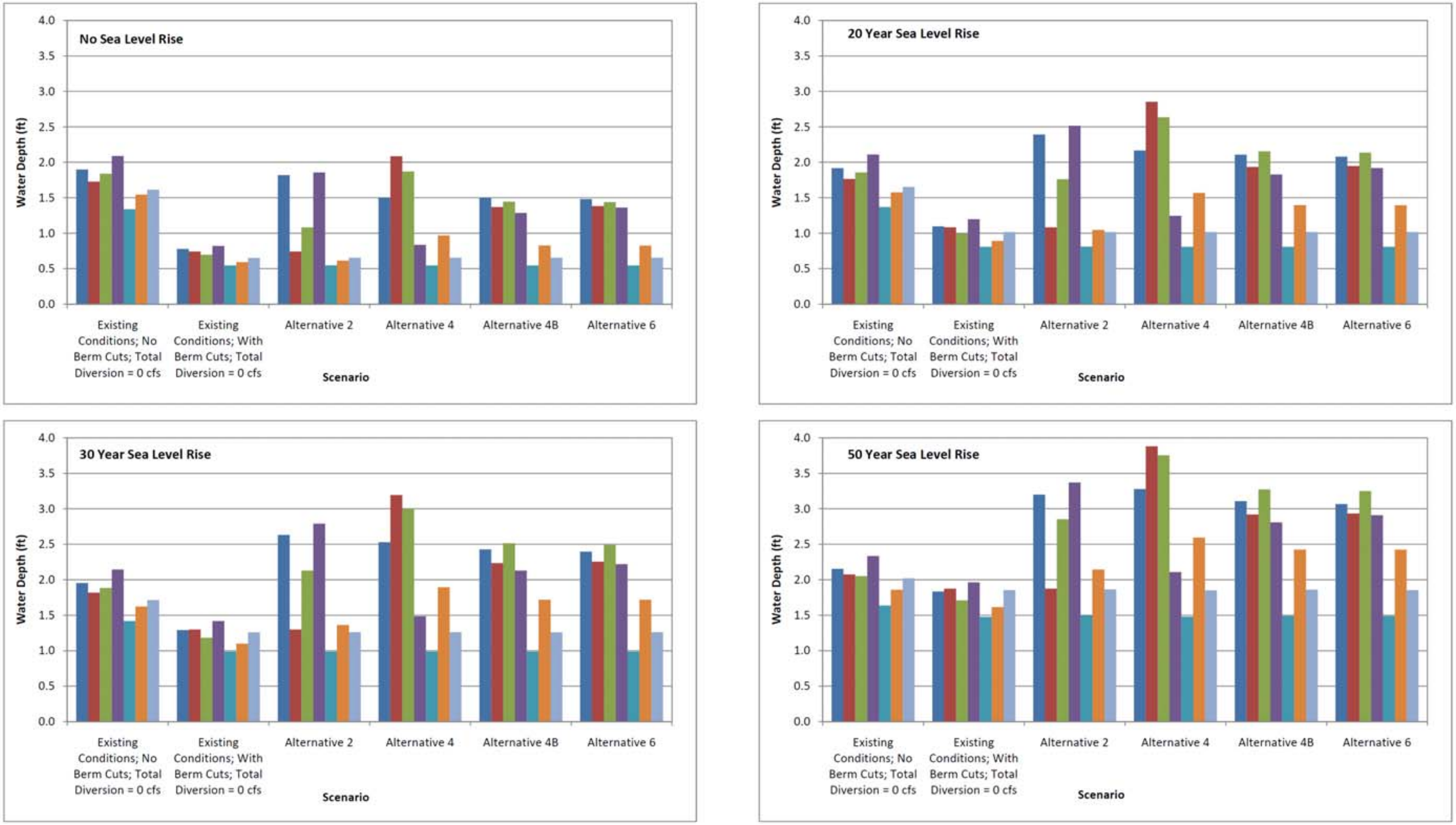
#### L2.10.1.3 Engineering Calculations

**Water Depth and Backflow Prevention**
Existing conditions (no sea level rise) without berm gaps indicate an average long-term water depth ranging from 1.34 to 2.09 feet (**Figure L2.10.1-12** and **Table L2.10.1-8**) and a backflow prevention ranging from 73 to 91 percent (**Figure L2.10.1-13** and Table L2.10.1-8). By incorporating improved berm gaps in the existing condition scenario, hydraulic routing is improved and average water levels (0.55 to 0.82 feet) are reduced. The frequency of time during the 16-year period analysis that backflow is prevented (17 to 38 percent) is also reduced since the average water levels in the swamp are lower than the Lake Maurepas stage more often and, thus, the hydraulic gradient for backflow is more favorable.

For Alternative 2 (Romeville Diversion at 3,000 cfs), the average water depth ranges from 0.55 to 1.86 feet and backflow prevention ranges from 19 to 85 percent under no sea level rise conditions. This increase in water depths and backflow prevention when compared to the existing conditions with berm gaps is attributed to the additional diversion flow. The subbasins benefiting the most hydrologically from this alternative are 100 and 110 due to their adjacent location to the Romeville diversion point. Figure L2.10.1-12 further illustrates that as sea level rises, water depths increase throughout the swamp since more diversion flow is initiated and more water is routed through the system to attenuate backflow conditions. As sea level rises, backflow prevention generally decreases throughout the swamp (Figure L2.10.1-13) since the system is less effective at preventing backflow due to the increase in stages at the downstream boundary condition (Lake Maurepas).

**Table L2.10.1-8 Summary of Average Water Depth and Backflow Prevention (No Sea Level Rise) for Existing Conditions and Alternative 2**

| Annual Average Water Depth (feet) | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Subbasin Number** | **100** | **200** | **210, 220** | **110** | **120, 160** | **300, 320, 330** | **140, 150** |
| **Hydrologic Unit Number** | **HU 1** | **HU 2** | **HU 3** | **HU 4** | **HU 5** | **HU 6** | **HU 7** |
| Existing | 1.91 | 1.86 | 1.85 | 2.09 | 1.34 | 1.53 | 1.61 |
| Alternative 2 | 1.82 | 0.74 | 1.08 | 1.86 | 0.55 | 0.61 | 0.65 |
| **Annual Average Backflow Prevention (%)** | | | | | | | |
| Existing | 88 | 78 | 85 | 88 | 73 | 74 | 91 |
| Alternative 2 | 85 | 20 | 48 | 79 | 25 | 19 | 38 |




**Notes:**

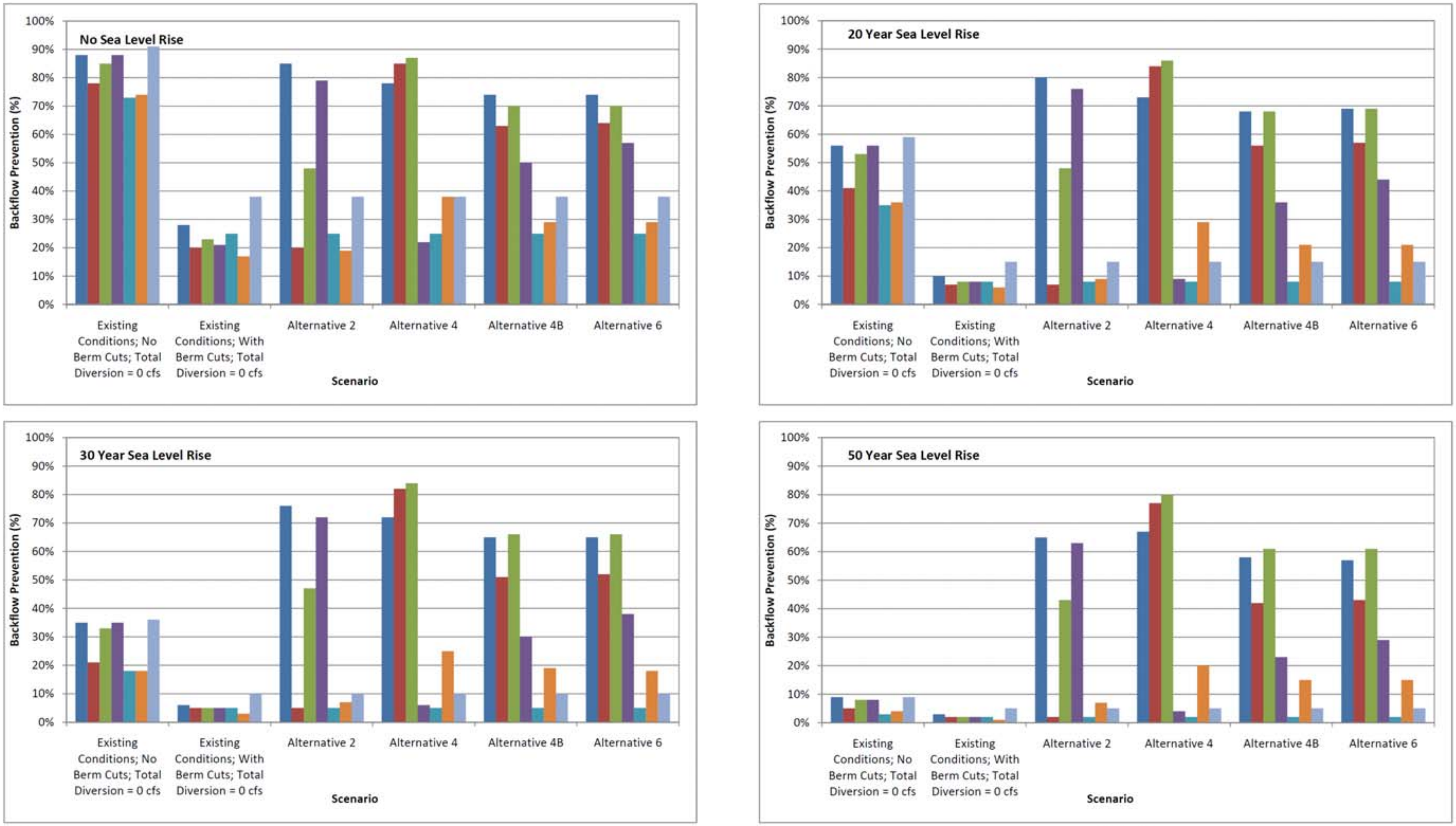
Alt 2 = Romeville Diversion (3,000 cfs)

Alt 4 = South Bridge Diversion (3,000 cfs)

Alt 4B = South Bridge Diversion (3,000 cfs Split)

Alt 6 = Total Diversion (3,000 cfs); Romeville Diversion (1,500 cfs)/South Bridge Diversion (1,500 cfs)

**Figure L2.10.1-12 Average Water Depth for Final Array of Alternatives**




**Notes:**

Alt 2 = Romeville Diversion (3,000 cfs)

Alt 4 = South Bridge Diversion (3,000 cfs)

Alt 4B = South Bridge Diversion (3,000 cfs Split)

Alt 6 = Total Diversion (3,000 cfs); Romeville Diversion (1,500 cfs)/South Bridge Diversion (1,500 cfs)

**Sub-basins**

100 | 200 | 210, 220 | 110 | 120, 160 | 300, 320, 330 | 140, 150

**Figure L2.10.1-13 Backflow Prevention for Final Array of Alternatives**

**Frequency of Dry-Out Conditions**
As shown on **Figure L2.10.1-14** and in **Table L2.10.1-9,** existing conditions (no sea level rise) without functional berm gaps indicate a long-term dry-out frequency (below 0.5 feet) from 0 to 3 percent. By incorporating additional wider berm gaps in the existing condition scenario, dry-out frequency (25 to 44 percent) increases since there is less opportunity for the water to remain stagnant in the swamp.

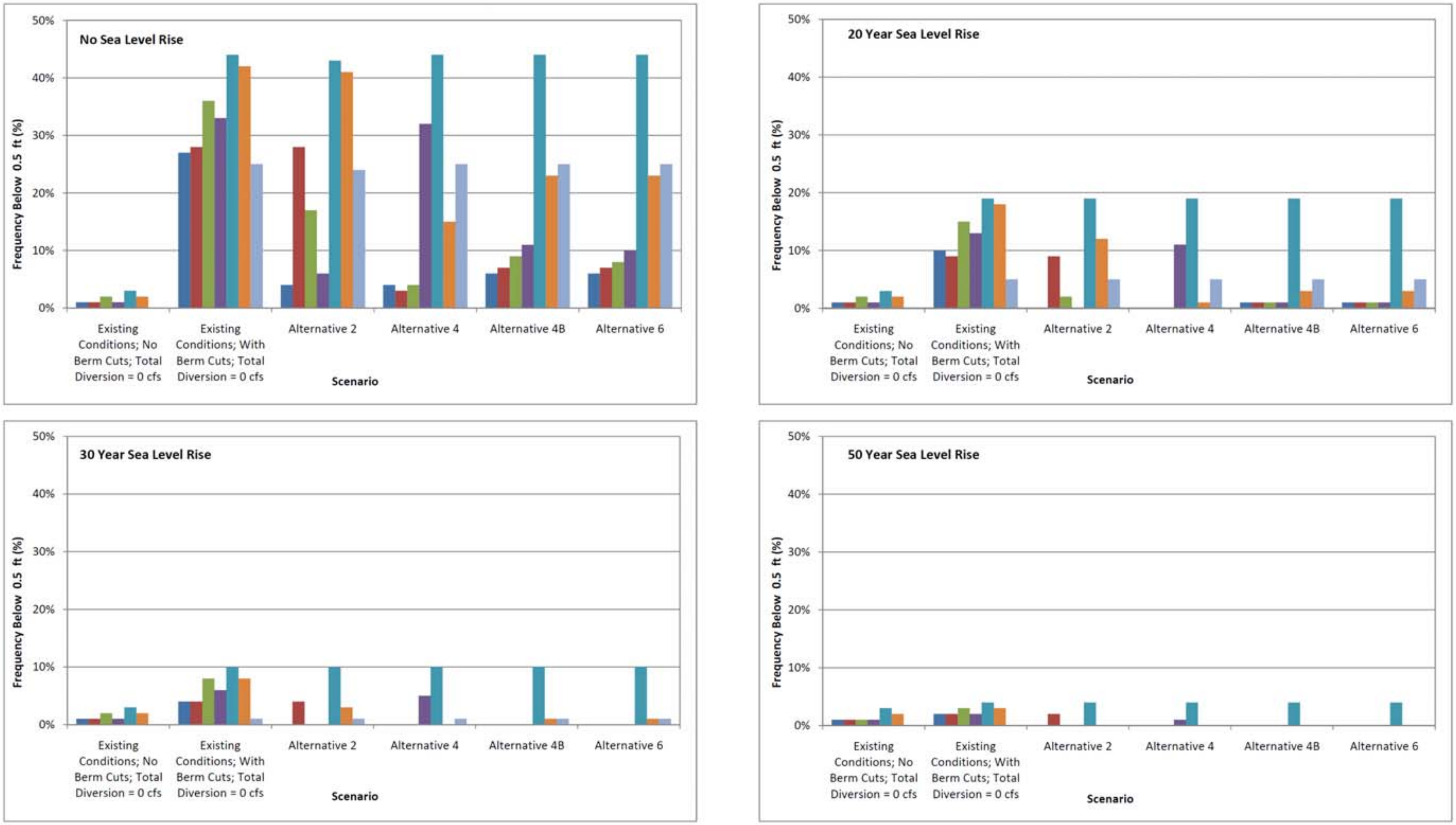
For Alternative 2, the average dry-out frequency ranges from 4 to 43 percent under no sea level rise conditions. There is a reduction in dry-out frequency when compared to the existing conditions with berm gaps mainly in the upstream subbasins (100, 110, 210, and 220), which are more directly impacted by the Romeville diversion due to their proximity. As sea level rises, dry-out frequency decreases throughout the swamp since more diversion flow is initiated to prevent backflow; therefore, reducing the occurrence of dry-out conditions.

**Table L2.10.1-9 Summary of Average Dry-Out Frequency (No Sea Level Rise) for Existing Conditions and Alternative 2**

| Annual Average Dry-Out Frequency (%) | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Subbasin Number** | **100** | **200** | **210, 220** | **110** | **120, 160** | **300, 320, 330** | **140, 150** |
| **Hydrologic Unit Number** | **HU 1** | **HU 2** | **HU 3** | **HU 4** | **HU 5** | **HU 6** | **HU 7** |
| Existing | 1 | 1 | 2 | 1 | 3 | 2 | 0 |
| Alternative 2 | 4 | 28 | 17 | 6 | 43 | 41 | 24 |

Note: Dry-out conditions defined as water depth less than 0.5 feet.

**Frequency of Diversions and TSS Loading**
As shown on **Figure L2.10.1-15**, the diversion is initiated under Alternative 2 (no sea level rise) an average 50 percent of the time. Also, approximately 1.5 mm/year of TSS loading from the Mississippi River (**Figure L2.10.1-16**) is introduced to the project area in this scenario. As sea level rises, additional diversion flow (up to 85 percent) is initiated and, hence, more TSS loading is introduced to the system (up to 2.7 mm/yr).




**Notes:**
Alt 2 = Romeville Diversion (3,000 cfs)
Alt 4 = South Bridge Diversion (3,000 cfs)
Alt 4B = South Bridge Diversion (3,000 cfs Split)
Alt 6 = Total Diversion (3,000 cfs); Romeville Diversion (1,500 cfs)/South Bridge Diversion (1,500 cfs)

**Sub-basins**
100
200
210, 220
110
120, 160
300, 320, 330
140, 150

**Figure L2.10.1-14 Average Dry-Out Frequency for Final Array of Alternatives**

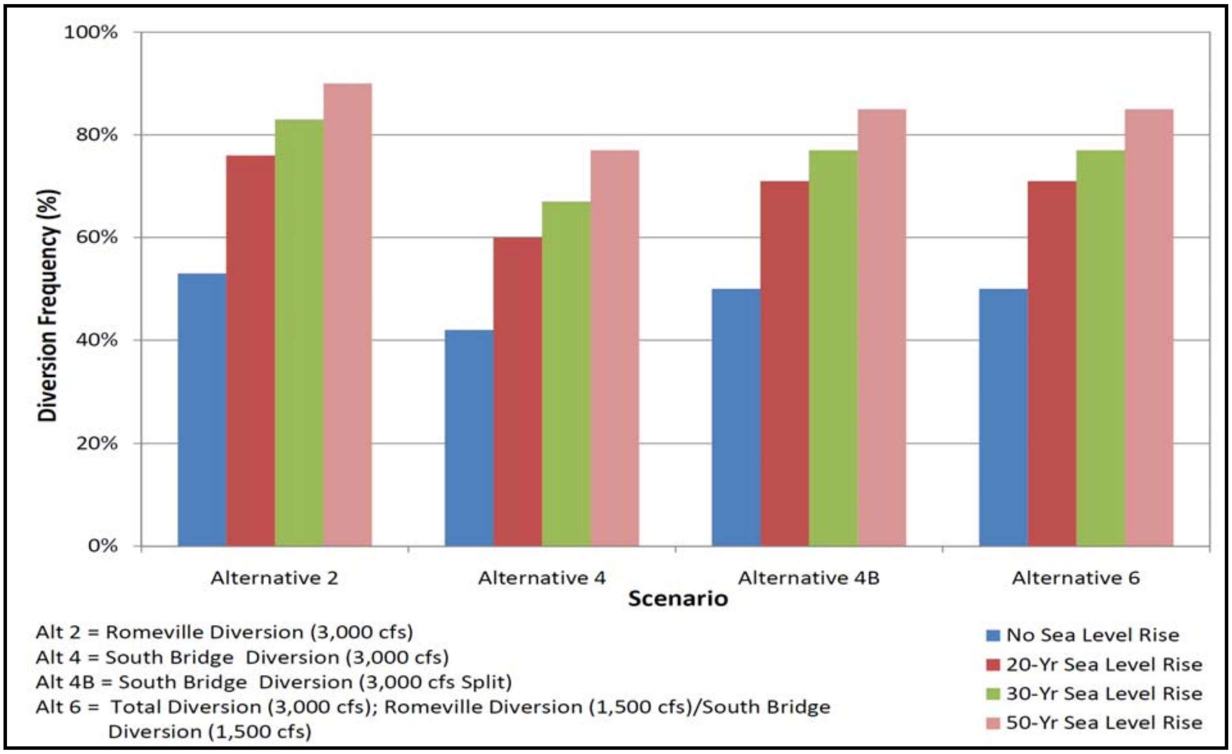



**Figure L2.10.1-15 Diversion Frequency for Final Array of Alternatives**

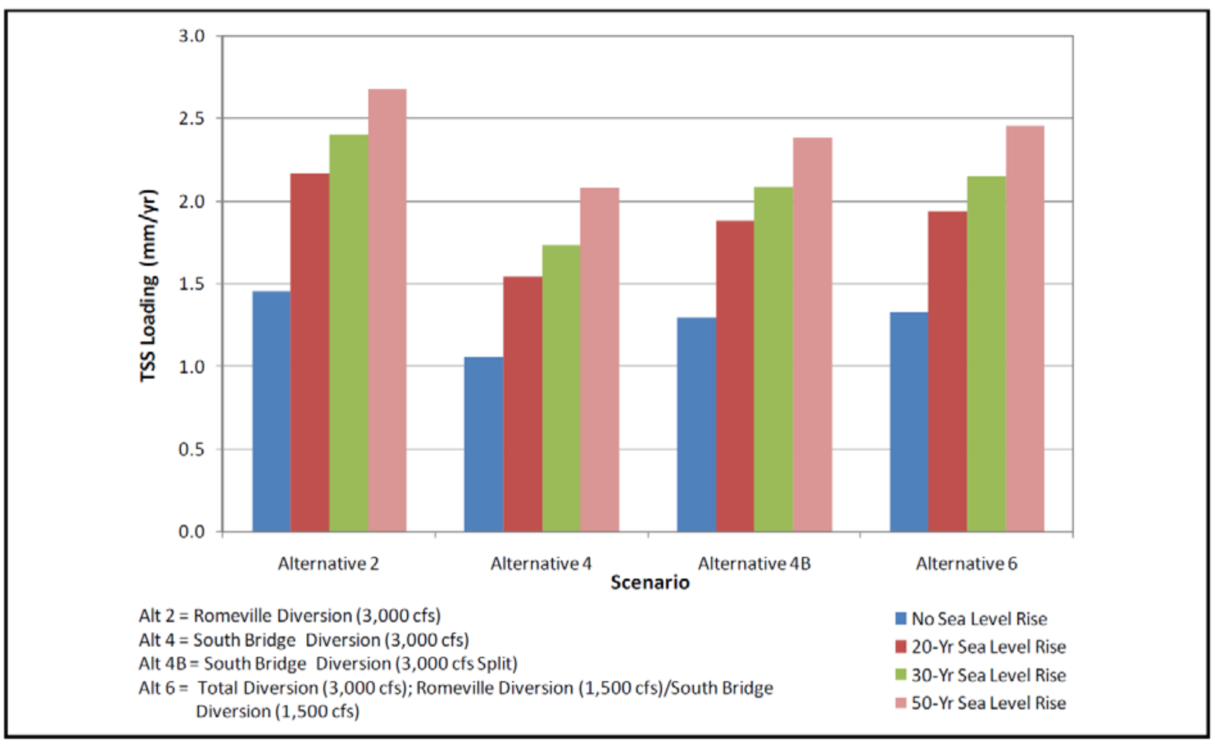



**Figure L2.10.1-16 TSS Loading for Final Array of Alternatives**

### L2.10.2 Alternative 4

Alternative 4 includes a freshwater diversion of up to 3,000 cfs that follows the South Bridge alignment. A plan view of Alternative 4 is presented on **Figure L2.10.2-1**. This alternative has seven major components:

**Diversion facility at the Mississippi River.** The diversion culvert facility will divert fresh water from the Mississippi River, transfer it beyond the east levee and discharge to the transmission canal.

**Transmission canal.** The transmission canal will transfer the diverted water approximately four miles from the diversion culvert facility to an existing drainage channel at the perimeter of the swamp. The project will use the existing drainage channels at the perimeter of the swamp to distribute the diverted flow throughout and into the swamp.

**Distribution Canal.** A distribution canal will be constructed across the swamp from the west edge of the project area to US 61. The distribution canal will be constructed with levees on each side to elevate the water surface above the swamp, similar to an irrigation canal, and allow for distribution of flow to adjacent areas of the swamp.

**Control Structures.** Approximately six control structures of various sizes with control gates will be installed at key locations in the existing channels to force water out of the drainage channels and into the swamp through the berm gaps. The proposed control gate is a specialty gate that rotates on a shaft at the bottom of the channel and is operated by large hydraulic cylinders. (One of the gate options, the Obermeyer gates, uses an air-inflated bladder to operate the gates.) The gate will be rotated up to the vertical position to increase the water surface elevation during the flow diversion and promote flow distribution to the swamp. The gate will be rotated down to the channel bottom into the open position when there is no diversion, to allow for normal drainage.

**Berm Gaps.** As identified in Alternative 2, new 500-foot wide berm gaps will be constructed to improve flow circulation in the swamp.

**Highway 61 and KCS RR Culverts.** As identified in Alternative 2, new culvert crossings will be added under the KCS RR and Hwy 61 at four locations.

**Instrumentation.** As identified in Alternative 2, instrumentation will be required to control the diversion flow rate and control structures.

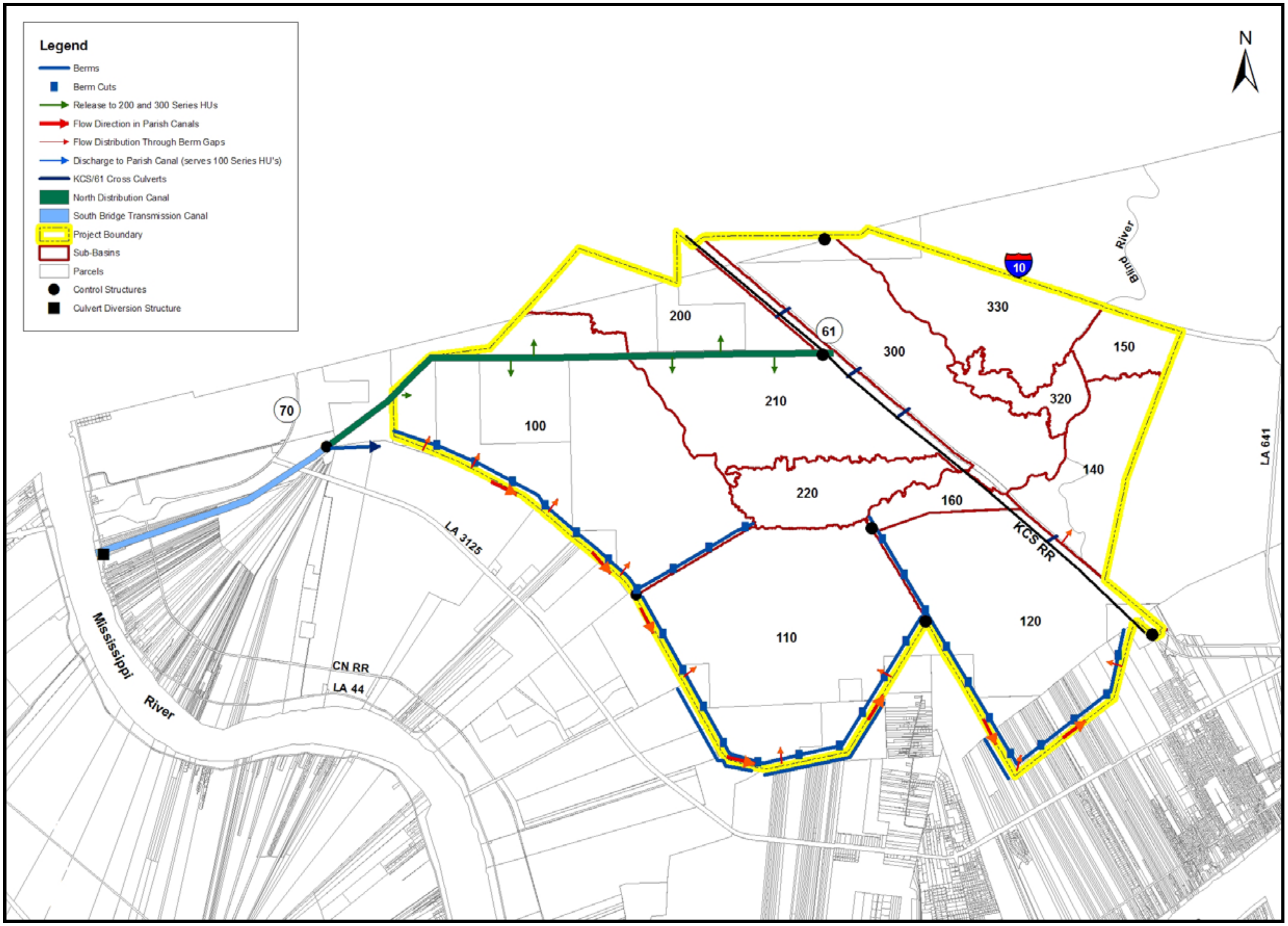



**Figure L2.10.2-1 Alternative 4 Layout**

#### L2.10.2.1 HEC-HMS and HEC-RAS

The following modifications to the existing conditions HEC-RAS model were incorporated to represent Alternative 4:

- HEC-RAS lateral structures that allow flow exchange between the drainage canals and the swamp were modified to reflect the berm gaps included in this alternative. The geometry of the lateral structures were modified to lower invert elevations to elevation 0 feet NAVD and widened to represent increased flow capacity compared to existing conditions.
- HEC-RAS culverts were added across US 61 to provide more conveyance capacity between swamp storage areas on both sides of the existing highway embankment.
- Fixed weirs were added in HEC-RAS at the locations of the control structures to simulate dry conditions when diversion flow will be distributed to the swamp. For simulations of wet weather conditions the weirs in HEC-RAS were modified to represent the control gates in the lowered position.

Alternative 4 also includes a diversion of 3,000 cfs along the South Bridge alignment and a distribution canal across the swamp to US 61. The distribution of diversion flows into the swamp for Alternative 4 was simulated in HEC-RAS with multiple boundary condition flow time series assigned to various swamp storage areas represented in the model. This conceptual modeling approach was used for the alternative analysis to simplify the detail that would have been needed to explicitly model the distribution canal and associated control structures in HEC-RAS. The flow split that was used consisted of 17% (500 cfs) assigned to the St. James Parish Canal, 17% (500 cfs) assigned to hydrologic unit 100, 33% (1,000 cfs) assigned to hydrologic unit 210, 17% (500 cfs) to hydrologic unit 220, and 17% (500 cfs) to hydrologic unit 300.

Review of the HEC-HMS and HEC-RAS results indicates that Alternative 4 increases net freshwater throughput compared to existing conditions. Backflow is also reduced, especially with projected increases in sea level. The throughput and backflow values for Alternative 4 are comparable to the other alternatives in the final array. Alternative 4 provides the most throughput volume to hydrologic units 100, 200, and 210, which are all located west and north of the Blind River. Alternative 4 also provides improved throughput to hydrologic units 300 and 220.

#### L2.10.2.2 Engineering Calculations

**Water Depth and Backflow Prevention**

For Alternative 4 (South Bridge Diversion at 3,000 cfs), the average water depth ranges from 0.54 to 2.09 feet (Figure L2.10.1-3 and **Table L2.10.2-1**) and backflow prevention ranges from 22 to 87 percent (Figure L2.10.1-4 and Table L2.10.2-1) under no sea level rise conditions. This increase in water depths and backflow prevention when compared to the existing conditions with berm gaps is attributed to the additional diversion flow. The subbasins benefiting the most hydrologically from this alternative are 100, 200, 210, and 220 due to their adjacent location to the South Bridge diversion point, and the 300 series as a result of proposed improvements to routing flow through the Route 61 control structure. Figure L2.10.1-3 further illustrates that as sea level rises, water depths for this alternative increase throughout the swamp since more diversion flow is initiated and more water is routed through the system to attenuate backflow conditions. As sea level rises, backflow prevention (Figure L2.10.1-4) generally decreases throughout the swamp since the system is less effective at preventing backflow due to the increase in stages at the downstream boundary condition (Lake Maurepas). The reduction in backflow prevention for the subbasins listed above is less pronounced since these locations are more directly influenced by the South Bridge diversion due to their proximity.

**Frequency of Dry-Out Conditions**

As shown on Figure L2.10.1-5 and in Table L2.10.2-1, the average dry-out frequency for Alternative 4 ranges from 3 to 44 percent (no sea level rise). There is a reduction in dry-out frequency when compared to the existing conditions with berm gaps mainly in the upstream subbasins (100, 200, 210, and 220), which are more directly impacted by the South Bridge diversion due to their proximity to the entry point, and throughout the 300 series due to improved routing benefits. As sea level rises, dryout frequency decreases throughout the swamp since more diversion flow is initiated to prevent backflow; therefore, reducing the occurrence of dry-out conditions.

**Frequency of Diversions and TSS Loading**

As shown on Figure L2.10.1-6, the diversion is initiated under Alternative 4 (no sea level rise) an average 42 percent of the time. Also, approximately 1.1 mm/year of TSS loading from the Mississippi River (Figure L2.10.1-7) is introduced to the project area in this scenario. As sea level rises, additional diversion flow (up to 77 percent) is initiated and, hence, more TSS loading is introduced to the system (up to 2.1 mm/yr).

**Table L2.10.2-1 Summary of Average Water Depth, Backflow Prevention, and Dry-Out Frequency (No Sea Level Rise) for Alternative 4**

| **Subbasin Number** | **100** | **200** | **210, 220** | **110** | **120, 160** | **300, 320, 330** | **140, 150** |
|---|---|---|---|---|---|---|---|
| **Hydrologic Unit Number** | **HU 1** | **HU 2** | **HU 3** | **HU 4** | **HU 5** | **HU 6** | **HU 7** |
| Annual Average Water Depth (ft) | 1.50 | 2.09 | 1.87 | 0.84 | 0.54 | 0.97 | 0.65 |
| Annual Average Backflow Prevention (%) | 78 | 85 | 87 | 22 | 25 | 38 | 38 |
| Annual Average Dry-Out Frequency (%) | 4 | 3 | 4 | 32 | 44 | 15 | 25 |

### L2.10.3 Alternative 4B

Alternative 4B includes a freshwater diversion of up to 3,000 cfs that follows the South Bridge alignment in combination with control structures and channel improvements to promote distribution of flow to more of the swamp than Alternative 2 or 4. A plan view of Alternative 4B is presented on **Figure L2.10.3-1**. This alternative has eight major components:

**Diversion facility at the Mississippi River.** The diversion culvert facility will divert fresh water from the Mississippi River, transfer it beyond the east levee and discharge to the transmission canal.

**Transmission canal.** The transmission canal will transfer the diverted water approximately four miles from the diversion culvert facility to an existing drainage channel at the perimeter of the swamp. The project will use the existing drainage channels at the perimeter of the swamp to distribute the diverted flow throughout and into the swamp.

**Distribution Canal.** A distribution canal will be constructed across the swamp from the west edge of the project area to US 61. The distribution canal will be constructed with levees on each side to elevate the water surface above the swamp, similar to an irrigation canal, and allow for distribution of flow to adjacent areas of the swamp.

**Channel Improvements.** Conveyance of 1,500 cfs in the St. James Parish Canal to areas south of the Blind River will require the existing drainage canal to be widened between the transmission canal and hydrologic unit 110, as shown in Figure L2.10.3-1.

**Control Structures.** Approximately six control structures of various sizes with control gates will be installed at key locations in the existing channels to force water out of the drainage channels and into the swamp through the berm gaps. The proposed control gate is a specialty gate that rotates on a shaft at the bottom of the channel and is operated by large hydraulic cylinders. (One of the gate options, the Obermeyer gates, uses an air-inflated bladder to operate the gates.) The gate will be rotated up to the vertical position to increase the water surface elevation during the flow diversion and promote flow distribution to the swamp. The gate will be rotated down to the channel bottom into the open position when there is no diversion, to allow for normal drainage.

**Berm Gaps.** As identified for the other alternatives, new 500-foot wide berm gaps will be constructed to improve flow circulation in the swamp.

**Highway 61 and KCS RR Culverts.** As identified for the other alternatives, new culvert crossings will be added under the KCS RR and Hwy 61 at four locations.

**Instrumentation.** As identified for the other alternatives, instrumentation will be required to control the diversion flow rate and control structures.

### L2.10.3.1 HEC-HMS and HEC-RAS

The following modifications to the existing conditions HEC-RAS model were incorporated to represent Alternative 4B:

- HEC-RAS lateral structures that allow flow exchange between the drainage canals and the swamp were modified to reflect the berm gaps included in this alternative. The geometry of the lateral structures were modified to lower invert elevations to elevation 0 feet NAVD and widened to represent increased flow capacity compared to existing conditions.
- HEC-RAS culverts were added across US 61 to provide more conveyance capacity between swamp storage areas on both sides of the existing highway embankment.
- Fixed weirs were added in HEC-RAS at the locations of the control structures to simulate dry conditions when diversion flow will be distributed to the swamp. For simulations of wet weather conditions the weirs in HEC-RAS were modified to represent the control gates in the lowered position.
- Alternative 4B also includes a diversion of 3,000 cfs along the South Bridge alignment and a distribution canal across the swamp to US 61. Similar to Alternative 4, the distribution of diversion flows into the swamp for Alternative 4b was simulated with multiple boundary condition flow time series assigned to various swamp storage areas represented in HEC-RAS. The flow split that was used consisted of 50% (1,500 cfs) assigned to the St. James Parish Canal, 8.3% (250 cfs) assigned to hydrologic unit 100, 17% (500 cfs) assigned to hydrologic unit 210, 8.3% (250 cfs) to hydrologic unit 220, and 8.3% (250 cfs) to hydrologic unit 300.

Review of the HEC-HMS and HEC-RAS results indicates that Alternative 4B increases net freshwater throughput compared to existing conditions. Backflow is also reduced, especially with projected increases in sea level. The throughput and backflow values for Alternative 4B are comparable to the other alternatives in the final array, and provide distribution to more areas of the project area than either Alternative 2 or 4. Alternative 4 provides significant throughput volume to hydrologic units located both north and south of the Blind River, including hydrologic units 100, 110, 200, and 210. Alternative 4 also provides improved throughput to hydrologic unit 300, 220 and 140.

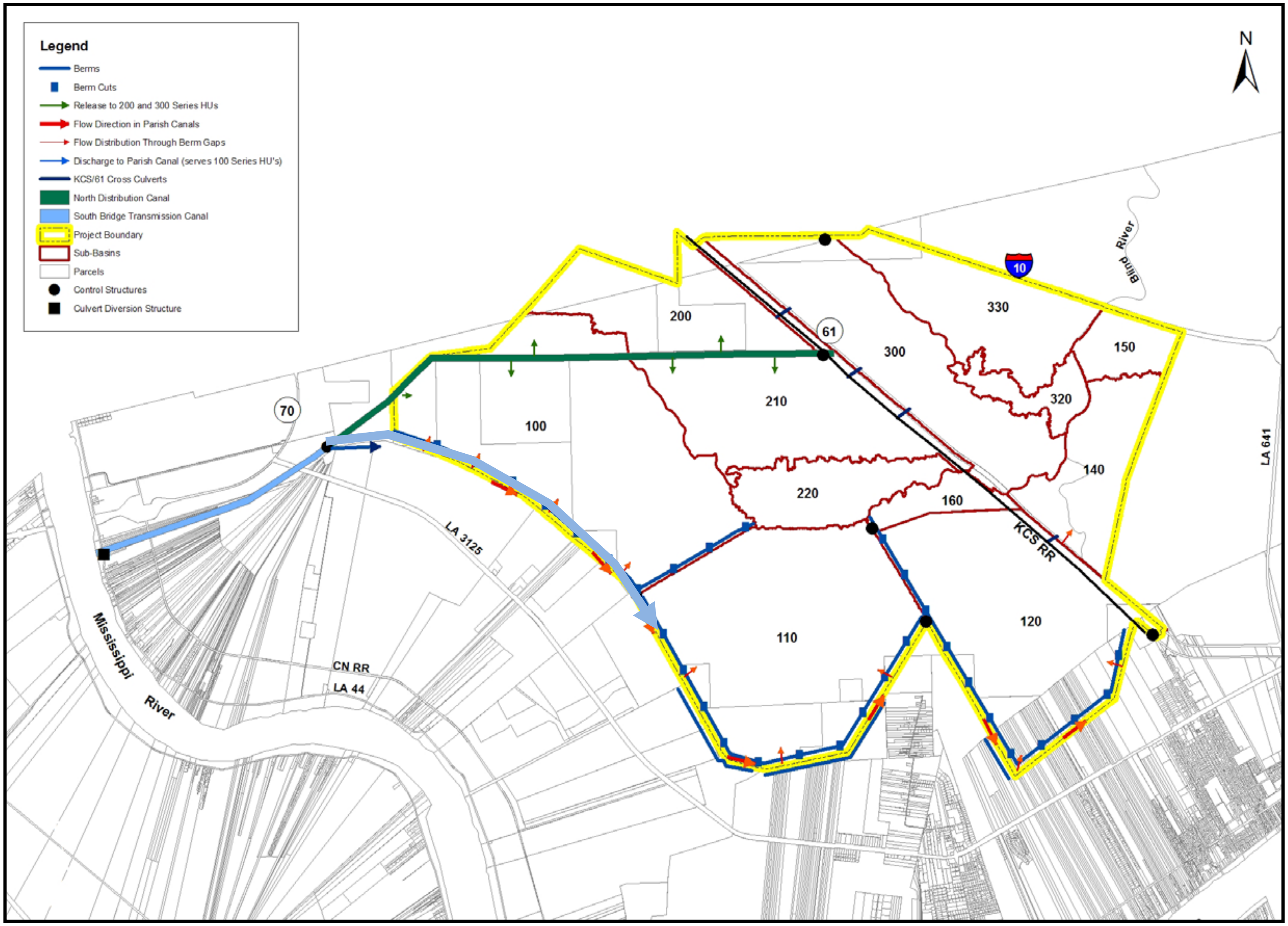



**Figure L2.10.3-1 Alternative 4B Layout**

### L2.10.3.2 Engineering Calculations

**Water Depth and Backflow Prevention**

For Alternative 4B (South Bridge Diversion split at 3,000 cfs), the average water depth ranges from 0.55 to 1.50 feet (Figure L2.10.1-3 and **Table L2.10.3-1**) and backflow prevention ranges from 25 to 74 percent (Figure L2.10.1-4 and Table L2.10.3-1) under no sea level rise conditions. This increase in water depths and backflow prevention when compared to the existing conditions with berm gaps is attributed to the additional diversion flow. The subbasins benefiting the most hydrologically from this alternative are 100, 110, 200, 210, and 220 due to their adjacent location to the South Bridge diversion point and St. James Parish Canal, and the 300 series as a result of proposed improvements to routing flow through the Route 61 control structure. Figure L2.10.1-3 further illustrates that as sea level rises, water depths for this alternative increase throughout the swamp since more diversion flow is initiated and more water is routed through the system to attenuate backflow conditions. As sea level rises, backflow prevention (Figure L2.10.1-4) generally decreases throughout the swamp since the system is less effective at preventing backflow due to the increase in stages at the downstream boundary condition (Lake Maurepas). The reduction in backflow prevention for the subbasins listed above is less pronounced since these locations are more directly influenced by the diversions due to their proximity to the South Bridge and St. James Parish Canals.

**Frequency of Dry-Out Conditions**

As shown on Figure L2.10.1-5 and in Table L2.10.3-1, the average dry-out frequency for Alternative 4B ranges from 6 to 44 percent (no sea level rise). There is a reduction in dry-out frequency when compared to the existing conditions with berm gaps mainly in the upstream subbasins (100, 110, 200, 210, and 220), which are more directly impacted by the South Bridge diversion due to their proximity to the entry point and to the St. James Parish Canal, and throughout the 300 series due to improved routing benefits. As sea level rises, dryout frequency decreases throughout the swamp since more diversion flow is initiated to prevent backflow; therefore, reducing the occurrence of dry-out conditions.

**Frequency of Diversions and TSS Loading**

As shown on Figure L2.10.1-6**,** the diversion is initiated under Alternative 4b (no sea level rise) an average 53 percent of the time. Also, approximately 1.3 mm/year of TSS loading from the Mississippi River (Figure L2.10.1-7) is introduced to the project area in this scenario. As sea level rises, additional diversion flow (up to 90 percent) is initiated and, hence, more TSS loading is introduced to the system (up to 2.4 mm/yr).

**Table L2.10.3-1 Summary of Average Water Depth, Backflow Prevention, and Dry-Out Frequency (No Sea Level Rise) for Alternative 4B**

| Subbasin Number | 100 | 200 | 210, 220 | 110 | 120, 160 | 300, 320, 330 | 140, 150 |
|---|---|---|---|---|---|---|---|
| **Hydrologic Unit Number** | **HU 1** | **HU 2** | **HU 3** | **HU 4** | **HU 5** | **HU 6** | **HU 7** |
| Annual Average Water Depth (ft) | 1.50 | 1.37 | 1.44 | 1.29 | 0.55 | 0.83 | 0.65 |
| Annual Average Backflow Prevention (%) | 74 | 63 | 70 | 50 | 25 | 29 | 38 |
| Annual Average Dry-Out Frequency (%) | 6 | 7 | 9 | 11 | 44 | 23 | 25 |

### L2.10.4 Alternative 6

Alternative 6 includes two freshwater diversions that each has a capacity of 1,500 cfs, for a total diversion capacity of 3,000 cfs. The alternative includes two transmission canals that follow the Romeville and South Bridge alignments in combination with control structures to maximize distribution of flow to the swamp. A plan view of Alternative 6 is presented on **Figure L2.10.4-1**. This alternative has nine major components:

**Diversion facility at the Mississippi River (Romeville).** The diversion culvert facility will divert fresh water from the Mississippi River, transfer it beyond the east levee and discharge to the Romeville transmission canal.

**Diversion facility at the Mississippi River (South Bridge).** The diversion culvert facility will divert fresh water from the Mississippi River, transfer it beyond the east levee and discharge to the South Bridge transmission canal.

**Romeville Transmission canal.** The transmission canal will transfer the diverted water approximately three miles from the diversion culvert facility to an existing drainage channel at the perimeter of the swamp.

**South Bridge Transmission canal.** The transmission canal will transfer the diverted water approximately four miles from the diversion culvert facility to an existing drainage channel at the perimeter of the swamp.

**Distribution Canal.** A distribution canal will be constructed across the swamp from the west edge of the project area to US 61. The distribution canal will be constructed with levees on each side to elevate the water surface above the swamp, similar to an irrigation canal, and allow for distribution of flow to adjacent areas of the swamp.

**Control Structures.** As identified for the other alternatives, approximately six control structures of various sizes with control gates will be installed at key locations in the existing channels to force water out of the drainage channels and into the swamp through the berm gaps.

**Berm Gaps.** As identified for the other alternatives, new 500-foot wide berm gaps will be constructed to improve flow circulation in the swamp.

**Highway 61 and KCS RR Culverts.** As identified for the other alternatives, new culvert crossings will be added under the KCS RR and Hwy 61 at four locations.

**Instrumentation.** As identified for the other alternatives, instrumentation will be required to control the diversion flow rate and control structures.

#### L2.10.4.1 HEC-HMS and HEC-RAS

The following modifications to the existing conditions HEC-RAS model were incorporated to represent Alternative 6:

- HEC-RAS lateral structures that allow flow exchange between the drainage canals and the swamp were modified to reflect the berm gaps included in this alternative. The geometry of the lateral structures were modified to lower invert elevations to elevation 0 feet NAVD and widened to represent increased flow capacity compared to existing conditions.
- HEC-RAS culverts were added across US 61 to provide more conveyance capacity between swamp storage areas on both sides of the existing highway embankment.
- Fixed weirs were added in HEC-RAS at the locations of the control structures to simulate dry conditions when diversion flow will be distributed to the swamp. For simulations of wet weather conditions the weirs in HEC-RAS were modified to represent the control gates in the lowered position.

Review of the HEC-HMS and HEC-RAS results indicates that Alternative 6 provides increases net freshwater throughput compared to existing conditions. Backflow is also reduced, especially with projected increases in sea level. The total throughput and backflow values for Alternative 6 are comparable to the other alternatives in the final array, and provide distribution to more areas of the project area than either Alternative 2 or 4. Alternative 6 provides significant throughput volume to hydrologic units located both north and south of the Blind River, including hydrologic units 100, 110, 200, and 210. Alternative 4 also provides improved throughput to hydrologic unit 300, 220, and 140.

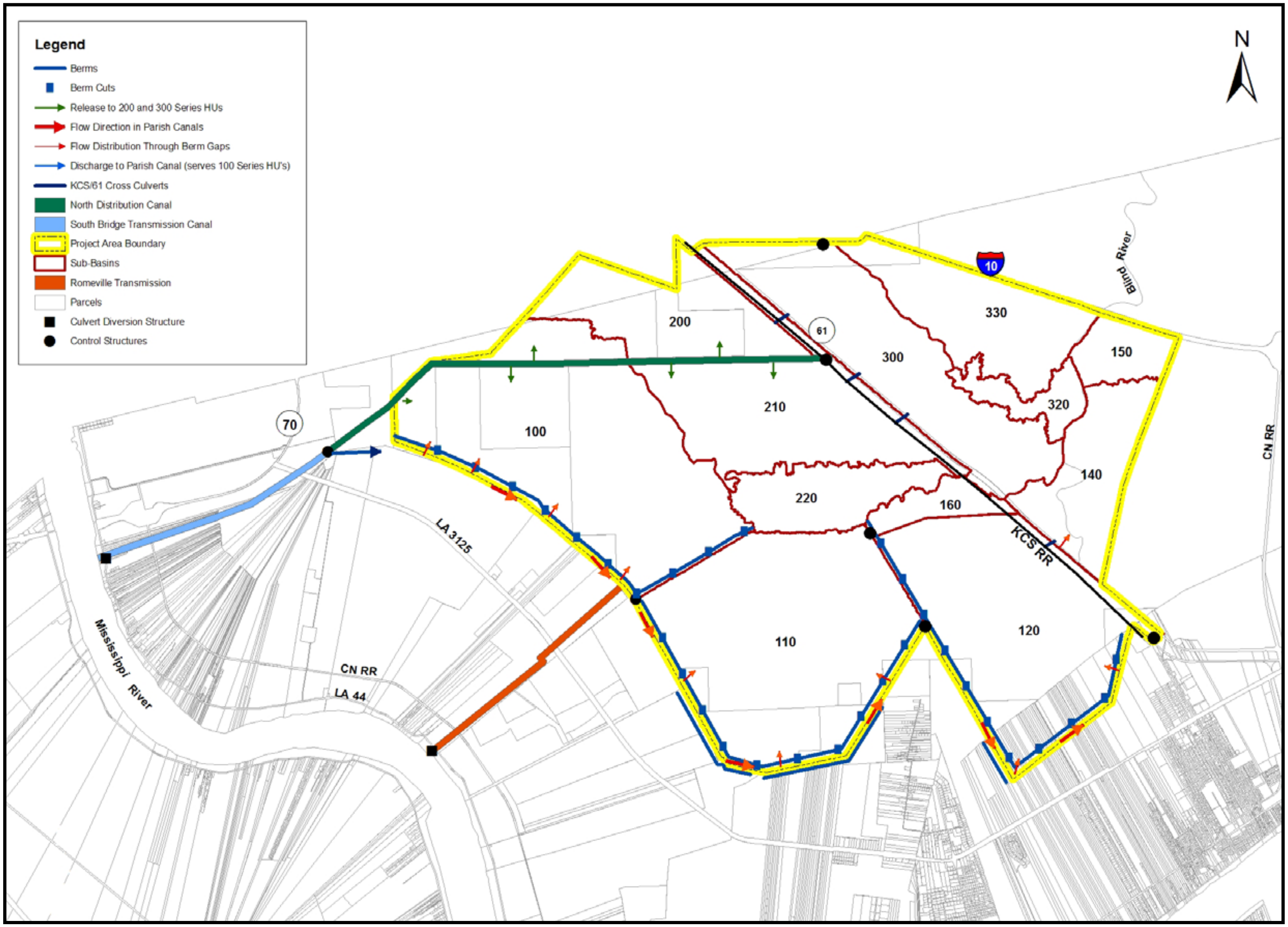



**Figure L2.10.4-1 Alternative 6 Project Layout**

### L2.10.4.2 Engineering Calculations

**Water Depth and Backflow Prevention**

For Alternative 6 (Romeville and South Bridge Diversions split at 1,500 cfs each), the average water depth ranges from 0.55 to 1.48 feet (Figure L2.10.1-3 and **Table L2.10.4-1**) and backflow prevention ranges from 25 to 74 percent (Figure L2.10.1-4 and Table L2.10.4-1) under no sea level rise conditions. This increase in water depths and backflow prevention when compared to the existing conditions with berm gaps is attributed to the additional diversion flow. The subbasins benefiting the most hydrologically from this alternative are 100, 110, 200, 210, and 220 due to their adjacent location to the Romeville and South Bridge diversion points, and the 300 series as a result of proposed improvements to routing flow through the Route 61 control structure. Figure L2.10.1-3 further illustrates that as sea level rises, water depths for this alternative increase throughout the swamp since more diversion flow is initiated and more water is routed through the system to attenuate backflow conditions. As sea level rises, backflow prevention generally decreases throughout the swamp (Figure L2.10.1-4) since the system is less effective at preventing backflow due to the increase in stages at the downstream boundary condition (Lake Maurepas). The reduction in backflow prevention for the subbasins listed above is less pronounced since these locations are more directly influenced by the diversions due to their proximity to the Romeville and South Bridge entry points.

**Frequency of Dry-Out Conditions**

As shown on Figure L2.10.1-5 and in Table L2.10.4-1, the average dry-out frequency for Alternative 6 ranges from 6 to 44 percent (no sea level rise). There is a reduction in dry-out frequency when compared to the existing conditions with berm gaps mainly in the upstream subbasins (100, 110, 200, 210, and 220), which are more directly impacted by the Romeville and South Bridge diversions due to their proximity to the entry points, and throughout the 300 series due to improved routing benefits. As sea level rises, dry-out frequency decreases throughout the swamp since more diversion flow is initiated to prevent backflow; therefore, reducing the occurrence of dry-out conditions.

**Frequency of Diversions and TSS Loading**

As shown on Figure L2.10.1-6, the diversion is initiated under Alternative 6 (no sea level rise) an average 50 percent of the time. Also, approximately 1.3 mm/year of TSS loading from the Mississippi River (Figure L2.10.1-7) is introduced to the project area in this scenario. As sea level rises, additional diversion flow (up to 85 percent) is initiated and, hence, more TSS loading is introduced to the system (up to 2.4 mm/yr).

**Table L2.10.4-1 Summary of Average Water Depth, Backflow Prevention, and Dry-Out Frequency (No Sea Level Rise) for Alternative 6**

| Subbasin Number | 100 | 200 | 210, 220 | 110 | 120, 160 | 300, 320, 330 | 140, 150 |
|---|---|---|---|---|---|---|---|
| **Hydrologic Unit Number** | **HU 1** | **HU 2** | **HU 3** | **HU 4** | **HU 5** | **HU 6** | **HU 7** |
| Annual Average Water Depth (ft) | 1.48 | 1.38 | 1.44 | 1.36 | 0.55 | 0.82 | 0.65 |
| Annual Average Backflow Prevention (%) | 74 | 64 | 70 | 57 | 25 | 29 | 38 |
| Annual Average Dry-Out Frequency (%) | 6 | 7 | 8 | 10 | 44 | 23 | 25 |

## L2.10.5 Summary and Recommendations

### Hydrologic Influence Areas

A primary output from the hydrologic and hydraulic analyses is the determination of the hydrologic influence on hydrologic units in the project area. All of the alternatives in the final array produced similar types of hydrologic influence that include improved throughput of freshwater, reduced backwater from Lake Maurepas, and improved frequency of dry-out. The differentiator between each alternative is the area of influence that receives hydrologic benefits. Results from each analysis method were used to characterize hydrologic influence into three categories:

- Areas that will benefit from the distribution of freshwater, nutrient, and sediment
- Areas that will benefit from the distribution of freshwater and nutrients
- Areas that will benefit from the distribution of freshwater

**Figures L2.10.5-1** through **L2.10.5-4** present the allocation of hydrologic benefits to the study area according to the three categories listed above. In general, the Romeville diversion primarily benefits areas south of the Blind River, while the South Bridge diversion (no split) primarily benefits areas north of the Blind River. Sea level rise significantly increases backflow, water levels, and reduces dry-out frequency.

### Consideration of Other Planned Projects

During completion of hydrologic and hydraulic analyses completed for the Blind River potential benefits from the Hope Canal project were also considered. Review of the Hope Canal report indicates that project diversions will be focused on areas north of I-10 (URS, 2007). Based on the provided hydraulic modeling results, the total recommended diversion flow can range from 1,500 cfs to 2,000 cfs. Approximately half of the diversion flow will be uniformly distributed along reaches of the Blind River and Amite River upstream of Lake Maurepas, and the remainder of the diversion will flow directly to Lake Maurepas. At times a small portion of flow, approximately 300 cfs, will be diverted to areas upstream of I-10 and will sheet flow to hydrologic units 140 and 150 in the Blind River project area.

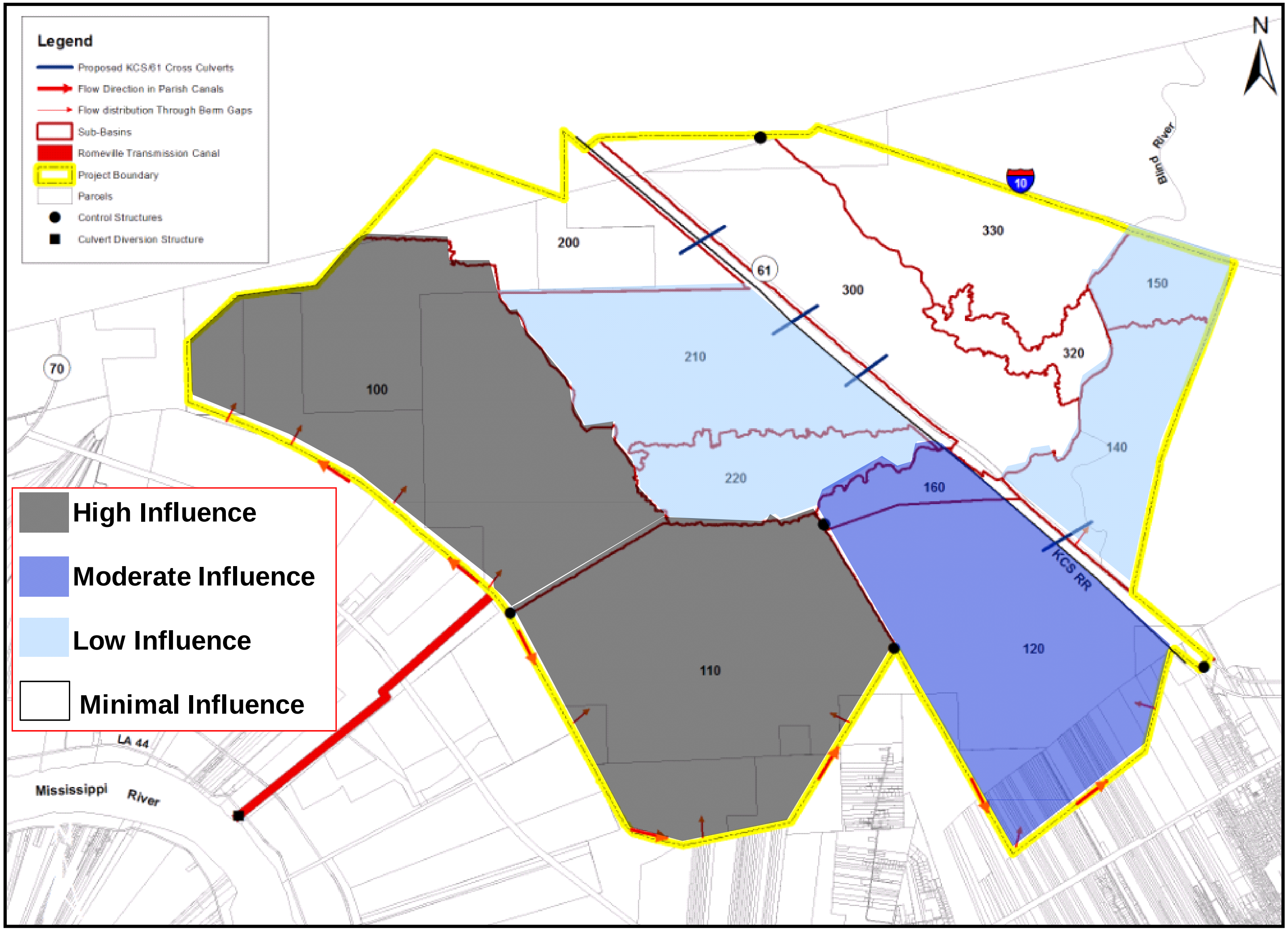



**Figure L2.10.5-1 Alternative 2 Hydrologic Influence Areas**

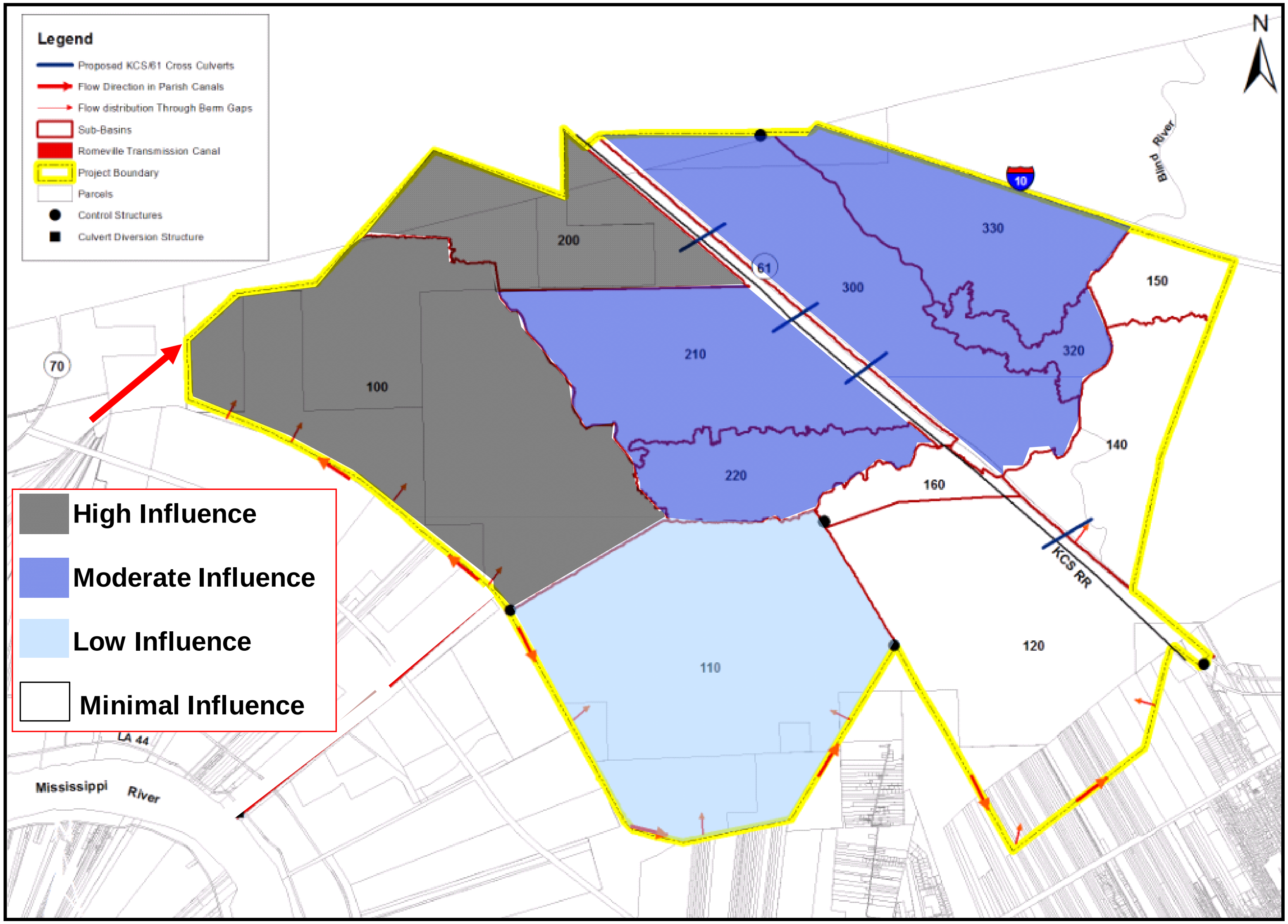



**Figure L2.10.5-2 Alternative 4 Hydrologic Influence Areas**

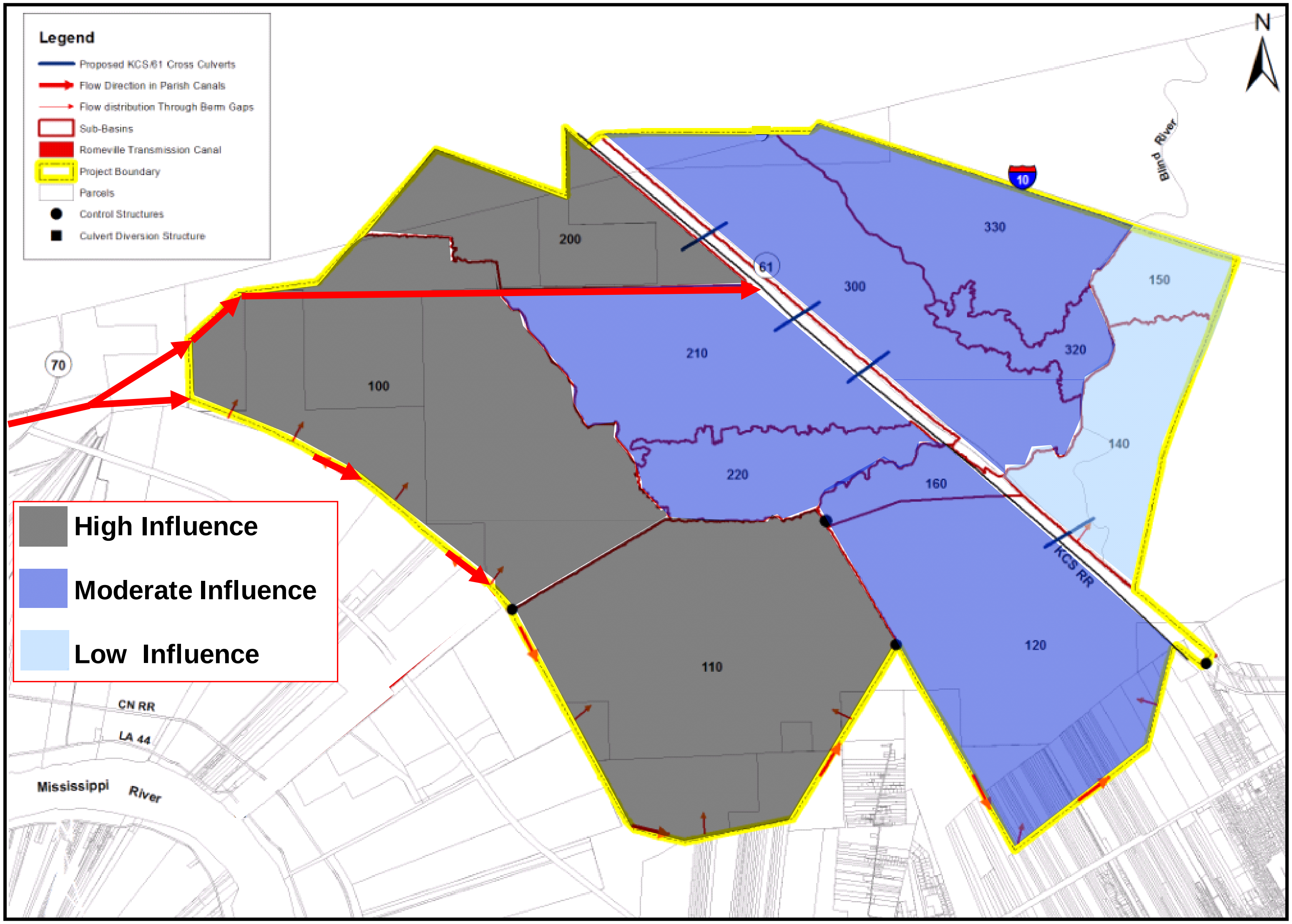



**Figure L2.10.5-3 Alternative 4B Hydrologic Influence Areas**

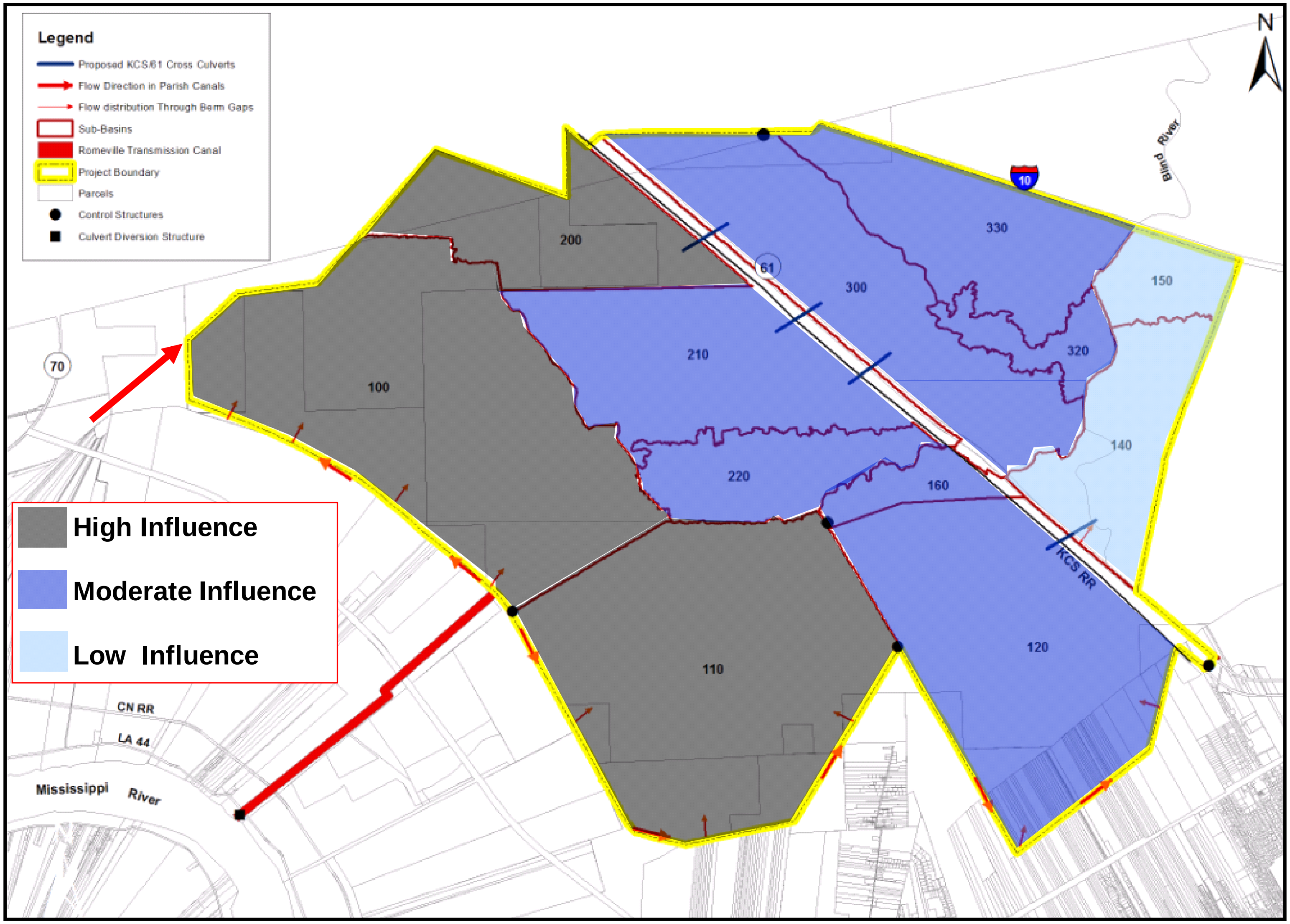



Figure L2.10.5-4 Alternative 6 Hydrologic Influence Areas

HEC-RAS simulations were completed with 1,000 cfs allocated uniformly to reaches of the Blind River downstream of I-10 to represent diversion flows from the Hope Canal project. Model results indicated minimal increases in stage of 0.1 to 0.2 feet on the Blind River with the additional flow, which is consistent with findings presented in the Hope Canal project report. Because diversion flow from the Hope Canal project enters the Blind River downstream of I-10, it does not reduce backflow from Lake Maurepas, and most likely will comprise a portion of the backflow when the swamp water levels are low.

The small portion of flow that will be released intermittently upstream of I-10 provides some potential for benefit to hydrologic units 140 and 150. However, because the diversion flow from the Hope Canal project will sheet flow over a significant distance through portions of the Maurepas swamp before reaching the Blind River, it is unlikely that nutrients or sediment will be contributed to those hydrologic units.

**Plan Selection**

The hydrologic and hydraulic analyses contribute significantly to identification of viable alternatives and guiding the selection of the Tentatively Selected Plan. However, the hydrologic and hydraulic analyses are only informative, as the net benefits of each alternative for this project are to be determined by the WVA. The completed hydrologic and hydraulic analyses produced the following findings:

- All four alternatives in the final array were found to be feasible from the hydrologic and hydraulic perspective and provide similar types of benefits. The primary differentiator among the alternatives was the portion of the project area that receives hydrologic influence.
- By minimizing headloss and provision of controllable operations to both the diversion flow and control structures in the project area, the use of existing drainage canals to distribute diversion flow to the project area is feasible without adversely impacting the existing drainage functionality of the system.
- In order to provide hydrologic influence to areas north of the Blind River it was found necessary to include the construction of a distribution canal across the swamp as opposed to exclusive use of existing drainage canals for the distribution of the diversion flow.
- Alternatives 4B and 6 provide hydrologic influence to the most hydrologic units in the project area compared to Alternatives 2 and 4. The dual diversion component of Alternative 6 also appears to perform more efficiently from a hydraulic perspective than the other alternatives. However, this additional hydrologic influence is only attainable with substantial increases in the scope of the required conveyance improvements.
- Future conditions that include both mean sea level rise and continued subsidence will increase the magnitude of land area in the swamp that is

inundated during average hydrologic conditions, primarily because the average and peak water elevations in Lake Maurepas will increase relative to the ground elevation in the swamp.

- While sea level rise will deliver more water to the swamp, the frequency and duration of stagnant conditions will increase. The risks of potential impacts associated with inundation resulting from storm surge, such as salinity, will also increase.

**Plan Refinement**

As the selected plan proceeds through subsequent design phases, continued evaluation of the following hydrologic and hydraulic aspects of the project are recommended:

- Refine the operational logic for controlling diversion flows to allow for preemptive diversions and continuous seasonal diversions;
- Further evaluate the sensitivities and effects of sea level rise with explicit incorporation of accretion in the project area, including corresponding adaptations to system operations;
- Evaluate Blind River diversion, Amite River diversion, and Hope Canal diversion and effects on <u>total</u> ecosystem, in addition to each incremental project;
- Utilize monitoring data collected at the new Blind River stream gage and piezometers installed within the swamp to calibrate the developed HEC-HMS, HEC-RAS, and EFDC models;
- Refine the HEC-HMS, HEC-RAS and EFDC models using additional topographic and field survey data to support design and implementation of critical hydraulic features of the selected plan, including in-swamp control structures, proposed culverts, and berm gaps.

## L2.11 Hydrologic Uncertainties

The results presented in the analysis to date have been developed with the best available information on historical hydrology, existing topography, and future conditions. However, each of these factors is subject to uncertainties, which could pose risks to the hydraulic and ecological functionality of the project. The uncertainties are discussed below:

- **Topography**: All modeling to date has been completed using best available topographic and bathymetric data, in combination with available engineering plans to define channel cross-sections, roadway culverts, and surface storage areas. The available topographic data coupled with field reconnaissance provided sound definition of major hydrologic and hydraulic features for use in the development models, but available data were not high resolution. LiDAR data collection was included in the scope of the study, in parallel with modeling activities, with the intent of providing greater resolution for topography of the system. However, wet conditions in the swamp resulted in the LiDAR acquisition being completed while much of the swamp was inundated with water, and prevented the desired data resolution for swamp bottom surface topography. Due to limitations in the available data, the model calculations that rely on topographic input such as estimates of water depths, residence times, and propensity of water to flow in assumed directions are similarly limited in their resolution.

- **Future hydrology:** The period of record used for extended analysis covered the period from 1989 through 2004. During this period, it appears that extended dry conditions that would support cypress germination and sapling survival occurred only every 5 to 6 years. The frequency at which conditions in the future may support growth cannot be accurately forecasted based only on this available data record. Future tree growth will be a function of climate patterns, management of the diversion and control structures, and the factors listed below, each of which includes inherent uncertainty. What can be inferred from the analysis is that careful flow management within the system can facilitate periodic hydrologic conditions that would support tree re-growth, but favorable ecological factors will also need to be present for this desired outcome.
- **Relative Sea Level Rise:** The basis for estimating relative sea level rise and associated impacts to the project are based on multiple components that all contain elements of uncertainty:
  - **Sea level rise:** USACE estimates for 50-year eustatic sea level rise (without the relative impacts of subsidence or accretion) range from 0.28 feet to 2.00 feet. This is a very broad range, as it coincides generally with the magnitude of normal water level fluctuations in the swamp. Future conditions for this project used the intermediate eustatic sea level rise estimate of 0.67 feet (coupled with subsidence for a relative rise of 1.90 feet).
  - **Subsidence:** Future subsidence rates used in this project, per USACE guidance, were 7.5 mm per year. This corresponds to 1.23 feet over a 50-year period. This is based on the measured local increase in sea level over 50 years (9.20 mm/yr) – the global eustatic rate of sea level rise (1.7 mm/yr). Coupled with the intermediate value of sea level rise, this yields a relative sea level rise of 1.90 feet over a 50-year period. However, the range of 50-year relative sea level rise estimates when subsidence is included is still very broad: 1.51 – 3.23 feet. Further uncertainty is introduced when considering the subsidence value alone. For example, the Amite River Project used a subsidence estimate of 8.5 mm/year, selected from an estimate range of 4 mm/yr to 20 mm/year based on projects and limited

research available for the region. This range alone translates to 2.62 feet of uncertainty with respect to future subsidence.

- **Accretion:** Estimates of future accretion rates are not included in the projections of future relative sea level rise. The Amite River Project identified a range of 5 mm/year to 25 mm/year of accretion, with an intermediate estimate of 12 mm/year. Over a 50-year period, this range translates into 3.28 feet of uncertainty with respect to accretion alone. The intermediate rate of 12 mm/year translates into 1.97 feet over 50 years, which would roughly offset the relative sea level rise of 1.90 feet (eustatic rise plus subsidence).
- **Combined Effects:** Using ranges applied to the Blind River project and also developed for the Amite River project, the cumulative 50-year effects of uncertainty with respect to eustatic sea level rise, subsidence, and accretion are as follows, using combinations of extreme values:
  - **Highest Estimated Relative Sea Level Rise:**

    Maximum Eustatic Rise + Maximum Subsidence – Minimum Accretion

    2.00 ft + 3.28 ft – 0.82 ft = **4.46 feet**
  - **Lowest Estimated Relative Sea Level Rise:**

    Minimum Eustatic Rise + Minimum Subsidence – Maximum Accretion

    0.28 ft + 0.66 ft – 4.1 ft = **-3.16 feet**

  The total range, then, of cumulative effects of land and sea changes is approximately 7.62 feet, which represents a large range of potential future conditions, especially considering that the range spans almost equally in opposing directions. Relative sea rise conditions that result in a relative sea level reduction will not pose risk to the project, while increases in relative sea level could impact project performance. The use of intermediate values for all factors produces an estimated relative sea level rise is -0.07 feet, representing a condition in which accretion effectively offsets the combined effects of subsidence and eustatic sea level rise.

As discussed elsewhere in this Feasibility Study and Engineering Appendix report, there is considerable uncertainty in the future relative elevations of land and sea in the study area due to a combination of effects, including eustatic sea level rise, land subsidence, and accretion through swamp biological productivity (growth) and sedimentation. The potential projected ranges of these interacting effects are broad enough to cause uncertainty not only in the magnitude of potential changes, but also the direction. That is, the combined effects of these phenomena could lead to future conditions characterized by sea levels that are either higher or lower than land elevations today.

As presented in Section L2.10, using intermediate values from available regional estimates of each contributing factor (eustatic sea level rise, subsidence, and accretion) suggest that relative sea level rise over 50 years will not produce the adverse hydrologic impacts to project performance that were analyzed. Analysis results developed for Alternative 2 are presented in this section utilized relative sea level rise for all three projections: low, medium and high.

The analyses in this Engineering Appendix have considered a portion of this range of combined effects, looking primarily at future estimates of relative sea level rise accounting for subsidence, but hydraulic modeling was not completed with explicit representation of accretion and sedimentation (in order to offer conservative "worst case" estimates). The relative rise has been applied in the modeling analysis at the downstream boundary condition, specifically the water level in Lake Maurepas, and the primary impact it has on model results is increased backflow of Lake water into the swamp, and a greater need for diverted water in future years to overcome the backflow.

However, it is conceivable that the water levels in the Mississippi River (upstream boundary condition and flow input for this project) could also be affected by combined effects of eustatic sea level rise and changes in sediment load. This is important because the flow rating curves developed for the gravity-based diversion structure are based on the differential head across the system, not just on the water level in the Mississippi River. If downstream water level rises in Lake Maurepas but Mississippi River water levels are largely unchanged, the physical ability to divert water could be diminished.

Specific forecasts of future water elevation trends in the Mississippi River near the study area are not readily available, so the analysis presented herein should be evaluated with the following considerations:

- If the Mississippi River water level does not change appreciably in the future, total diversion capacity could be diminished based on the assumptions guiding the application of sea level rise estimates to Lake Maurepas (less differential head across the system, and correspondingly lower diversion flows). As stated elsewhere, if intermediate projections for all contributing factors to relative sea level rise are applied together, the net effect could be almost negligible (counterbalancing effects). Hence, while there is the potential that rising relative sea level coupled with stationary river level could reduce diversion throughput, there is some uncertainty with these projections.
- If the Mississippi river water level rises in future decades, it should improve the ability to divert water to the Blind River system when compared to stationary water level in the river.

It is uncertain which of these scenarios is more likely to occur, and to what degree. Therefore, the project team has evaluated the effects of the different phenomena in sensitivity analyses. The worst case for diversion project performance would be higher levels in Lake Maurepas that do not appreciably affect the Mississippi River. This case would effectively reduce the gravity head gradient from the diversion to the Maurepas Swamp system and increase the need for more diverted flow to provide equal swamp restoration and flushing benefits.

The following two factors were used in deciding how to estimate the design level upstream boundary conditions in future decades:

- Intermediate (medium) projections of relative sea level rise, accounting for eustatic changes, subsidence, and accretion, suggest that the relative rise could be practically negligible.
- If relative sea level does change appreciably, it might be inferred that backwater elevations in the Mississippi River could also increase, if not in direct proportion, somewhat commensurately.

For these reasons, neither the historic water surface elevations in the Mississippi River (used in the hydrologic and hydraulic analysis) nor the flow rating curves for the diversion structure (in which the Mississippi River water level is the independent variable) were adjusted for the analysis of sea level rise in future decades.

Analysis results were developed for Alternative 2 with low, medium and high projections of sea level rise. The trends of the results for low and high relative sea level projections are consistent with the results for medium sea level rise presented in Section L2.10. **Figure L2.11-1** through **Figure L2.11-5** present analysis results developed for low, medium, and high sea level rise projections in combination with the approach and assumptions previously discussed.

- **Throughput and Backflow:** Figure L2.11-1 compares freshwater throughput experienced for existing conditions with increased throughput from the freshwater diversion. As shown, the increases in throughput as sea level rises results from increased diversion flows introduced to prevent backflow from Lake Maurepas. The analysis results indicated that the project can substantially increase throughput and prevent backflow over the range of potential relative sea rise conditions.
- **Average Water Depth:** Figure L2.11-2 illustrates that as sea level rises, water depths can be expected to increase accordingly throughout the swamp. The average water depth is a function of both the increased downstream water levels in Lake Maurepas as well as recommended increases in diversion flow that is initiated through the system to attenuate backflow conditions.

- **Backflow Prevention:** Figure L2.11-3 presents backflow prevention for all three sea level rise projections. The graphs indicate that subbasins receiving hydrologic influence from the freshwater diversion will be protected from frequent backflows originating from Lake Maurepas, and that backflow prevention will be marginally reduced with the high sea level rise projection compared to lower projections of sea level rise.
- **Dry-Out Frequency:** Presented on Figure L2.11-4 is the dry-out frequency that can be expected with and without the project. As presented, the project will increase the ability of subbasins within the project area to dry out and support the potential for bald cypress and tupelo germination and sapling survival. As sea level rises, this potential is expected to diminish over time and to different degrees within each subbasin.

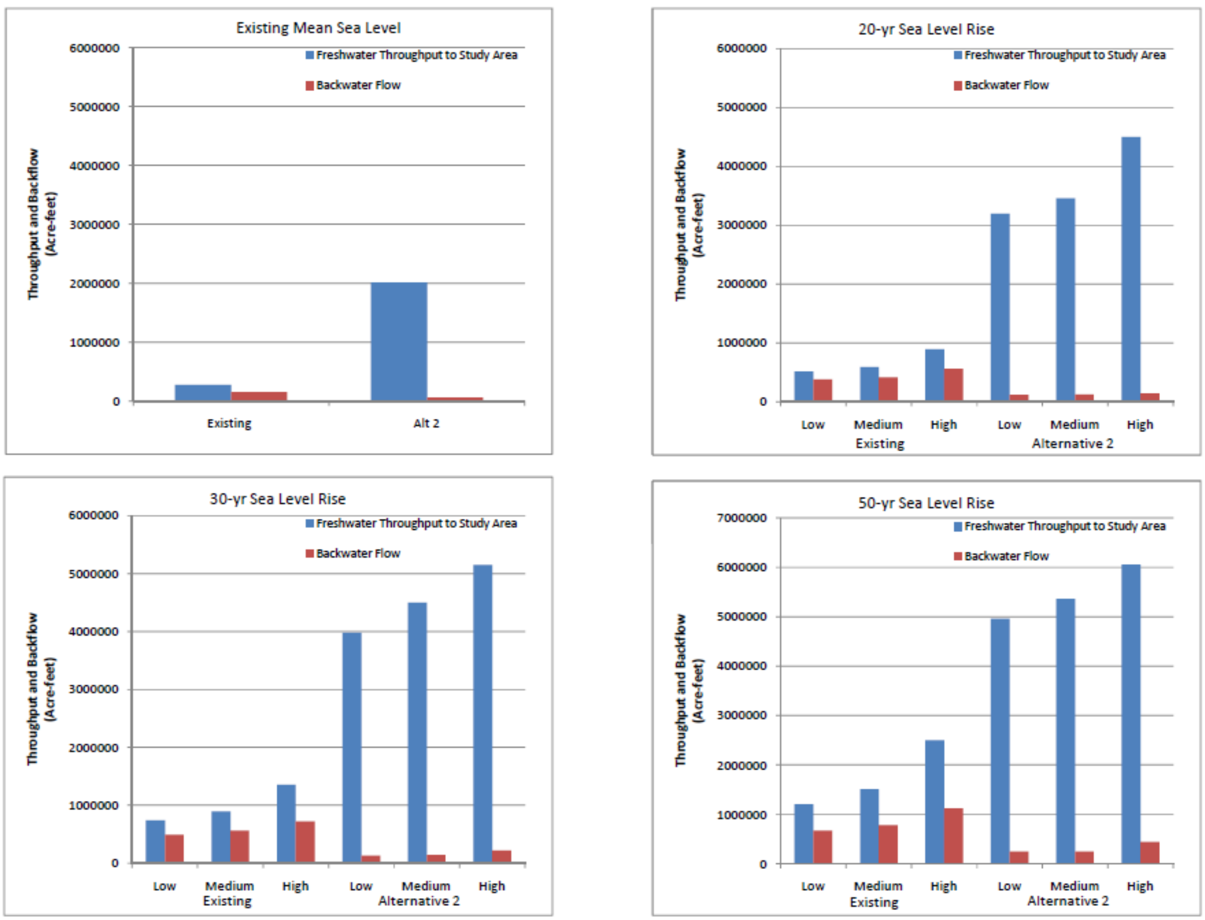
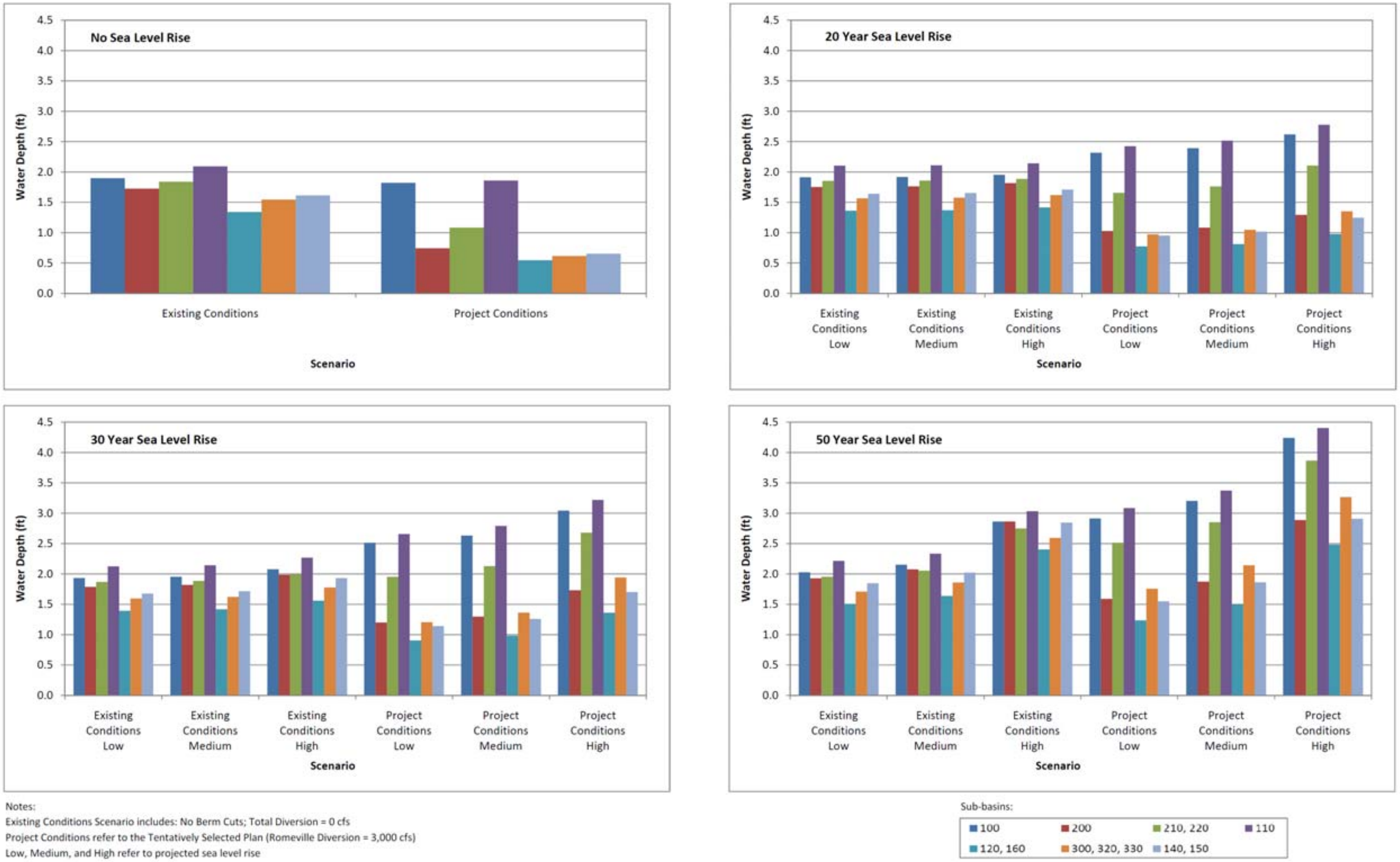
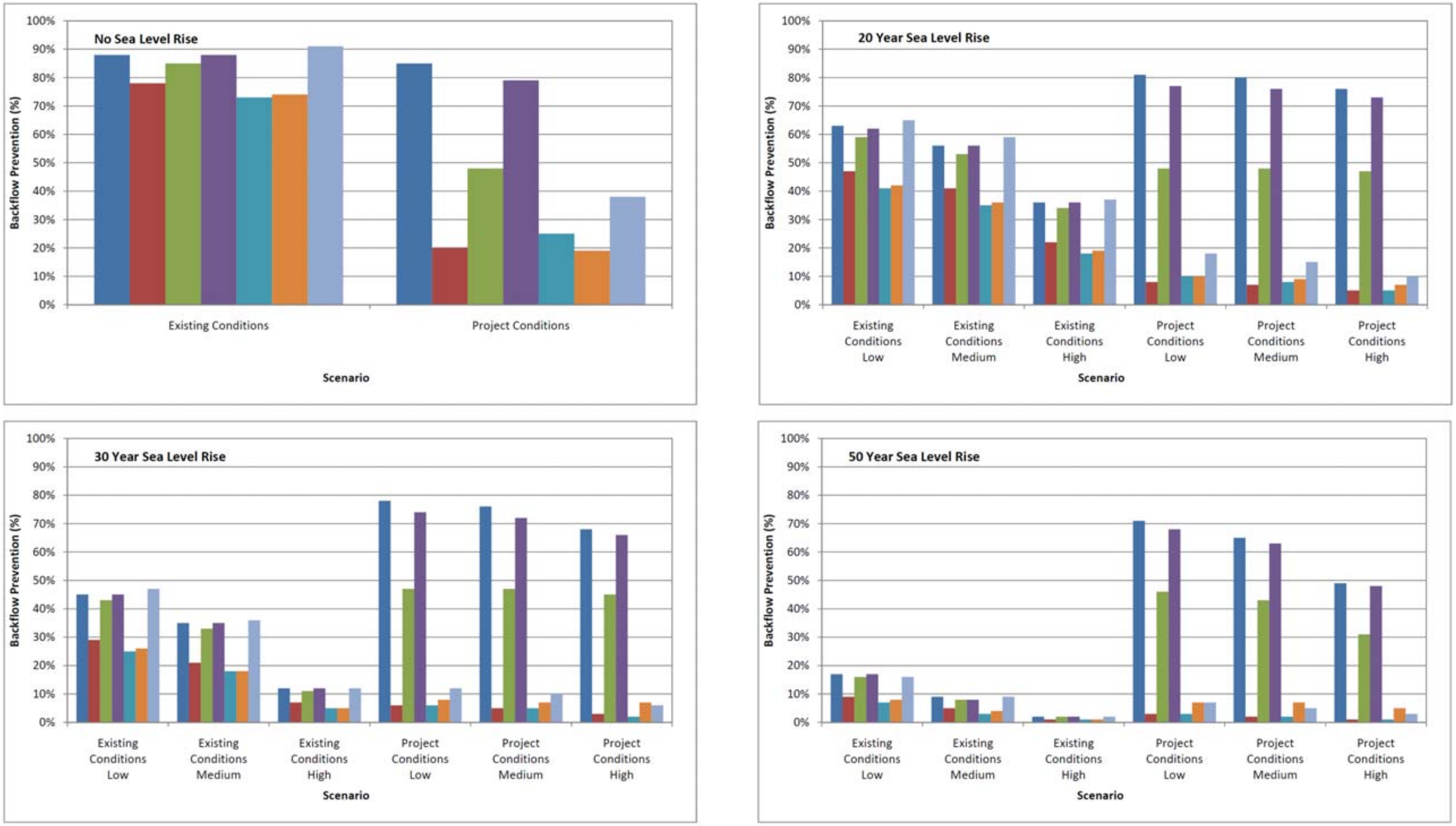



**Figure L2.11-1 Throughput and Backflow with Projected Relative Sea Level Rise**




Notes:
Existing Conditions Scenario includes: No Berm Cuts; Total Diversion = 0 cfs
Project Conditions refer to the Tentatively Selected Plan (Romeville Diversion = 3,000 cfs)
Low, Medium, and High refer to projected sea level rise

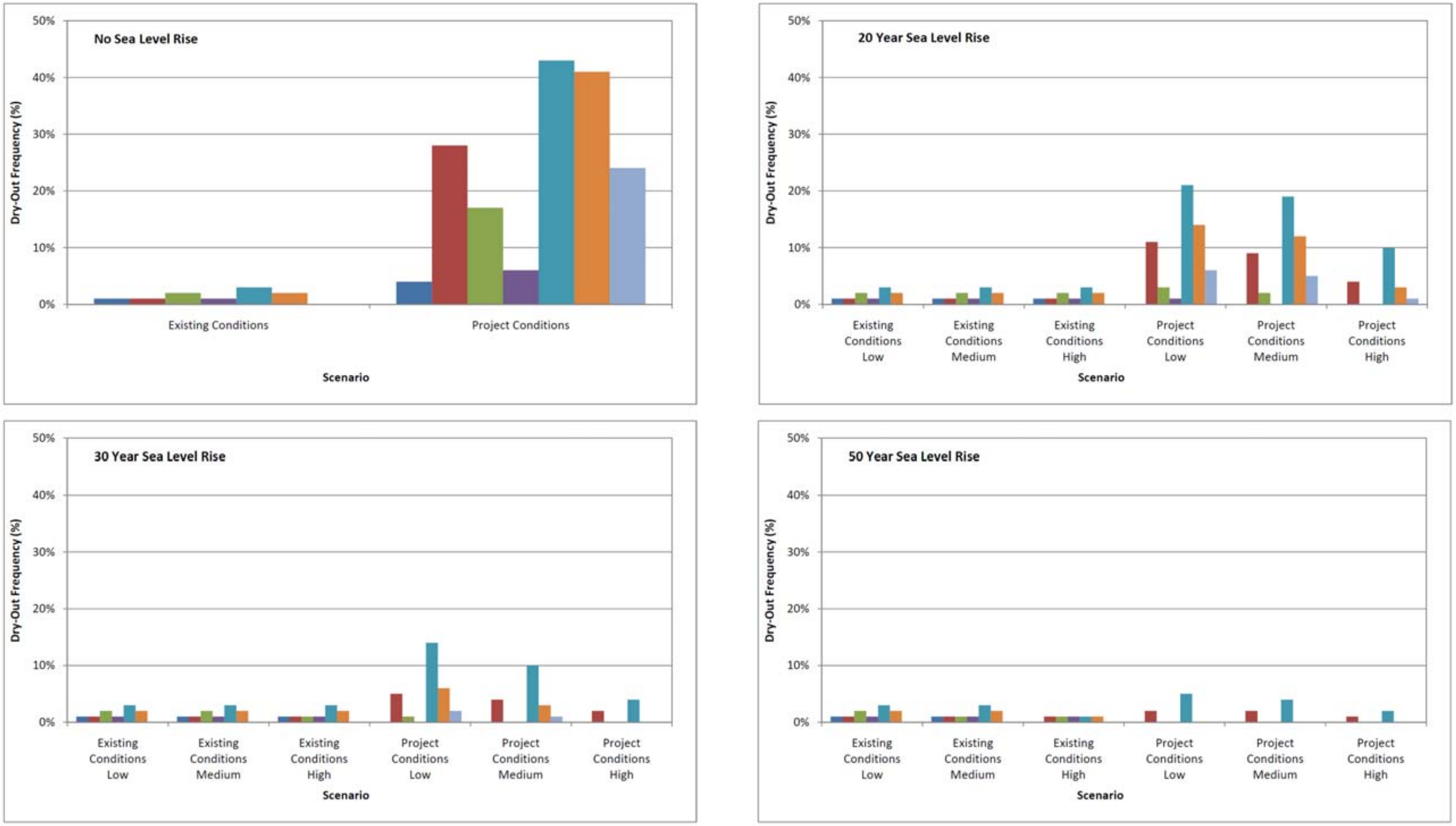
**Figure L2.11-2 Average Water Depth with Projected Relative Sea Level Rise**




Notes:
Existing Conditions Scenario includes: No Berm Cuts; Total Diversion = 0 cfs
Project Conditions refer to the Tentatively Selected Plan (Romeville Diversion = 3,000 cfs)
Low, Medium, and High refer to projected sea level rise

Sub-basins:
100 | 200 | 210, 220 | 110
120, 160 | 300, 320, 330 | 140, 150

**Figure L2.11-3 Backflow Prevention with Projected Relative Sea Level Rise**




Notes:
Existing Conditions Scenario includes: No Berm Cuts; Total Diversion = 0 cfs
Project Conditions refer to the Tentatively Selected Plan (Romeville Diversion = 3,000 cfs)
Low, Medium, and High refer to projected sea level rise

**Figure L2.11-4 Dry-Out Frequency with Projected Relative Sea Level Rise**

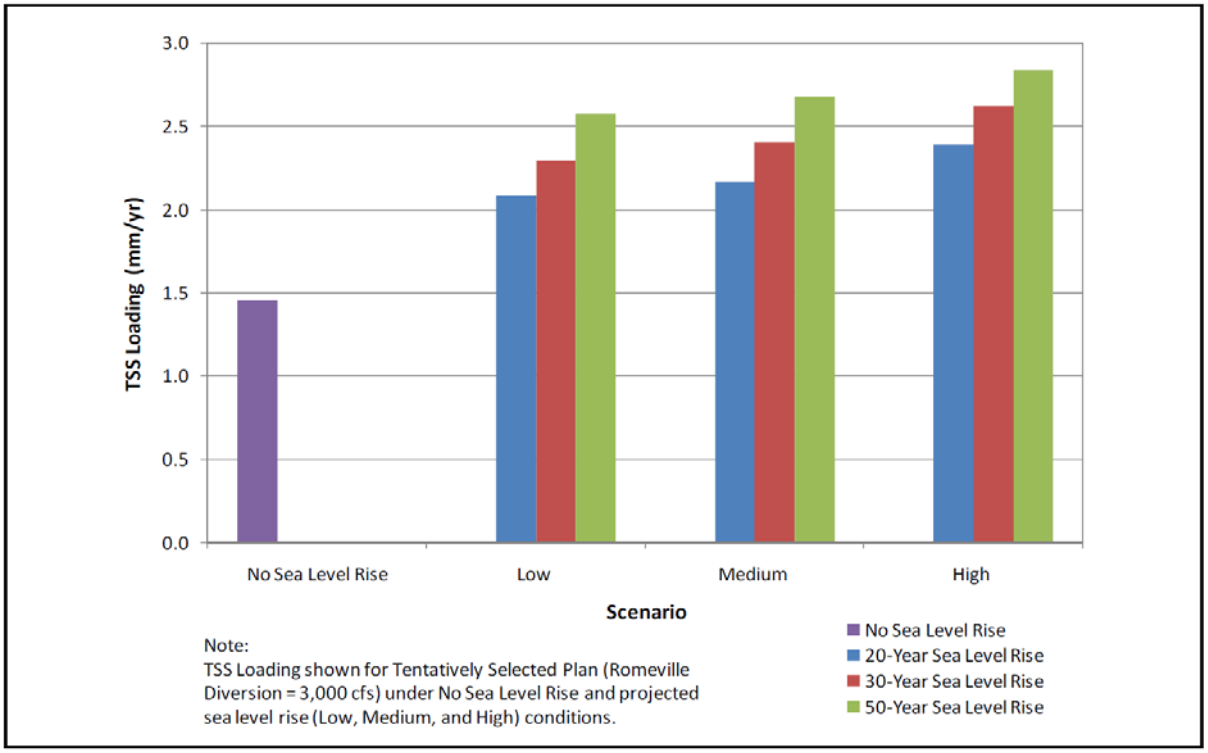



**Figure L2.11-5 TSS Loading with Projected Relative Sea Level Rise**

- **TSS Loading:** Figure L2.11-5 displays anticipated TSS loading to the project area for various sea level conditions. Since the delivery of TSS is closely correlated with the volume of freshwater diversion flow introduced to the project area, TSS loading is shown to increase as diversion flows are increased to maintain throughput and prevent backflow.

Significant uncertainty in each contributing factor provides the possibility for relative sea rise conditions that could affect the performance of the project. The sea level rise scenarios that were evaluated are considered to be conservative, since they account for eustatic rise and subsidence, but not for accretion. Uncertainty associated with relative sea level rise can be reduced with the collection and incorporation of additional information during subsequent project phases to better define local subsidence and probable accretion rates. In addition, adaptive management strategies should continue to be incorporated into the planned project in order to minimize potential impacts of relative sea and land elevations in the future. As additional information becomes available consideration of future conditions will continue to be refined during project design and to facilitate adaptive management after construction.

## L3 SURVEYING, MAPPING AND GEOSPATIAL DATA REQUIREMENTS

## L3.1 General

The feasibility study performed limited surveying and bathymetric surveys to identify key components of the project for the purpose of establishing correct costs for the various alternatives. the surveys were tied to existing benchmarks in or near the project area and adjusted to a NAVD 88 datum. The results of those surveys are in the following sections.

## L3.2 Ground Topographic Surveys

SJB Group was selected to be the ground surveyors for this project. Their tasks, as stated in an August 12th contract, included: a Centerline Profile Survey of the transmission canal, a Partial Topographic Survey of US 61 (Airline Highway), establish six temporary benchmarks, and a Partial Topographic Survey of I-10 crossing at Blind River.

### L3.2.1 Vertical and Horizontal Control Data

The Centerline Profile Survey was taken from the water's edge of the Mississippi River to the northeastern side of LA 3125 at a total distance of 10,150 feet. Data was collected at approximately every 500 feet or at sudden changes in elevation. Elevations were recorded at geographical features, drainage, and irrigation ditches encountered by the surveying team. Pipeline marker locations, culverts, and utility poles were identified by station number and distance from the channel centerline. Exact pipeline locations and cover were not included in the scope of work.

Horizontal and vertical control data were established for this project utilizing GPS Observations and made relative to NAD 83. Horizontal positions are expressed in Louisiana State Plane Coordinate System, Louisiana South Zone. Vertical datum is NAVD88 epoch 2006.81 GEIOD 03 updated.

### L3.2.2 Cross Section Locations

A cross-section was taken of the canal bottom at bridge crossings along US 61. The cross-sections show piers, abutments, and the lowest horizontal members of the bridge structure perpendicular to water flow. A cross-section of the canal bottom was taken adjacent to and west of the south bound lane and adjacent to and east of the north bound lane.

### L3.2.3 Profile Alignment and Orientation

The channel alignment survey was conducted from the riparian zone between the levee and Mississippi River to the northern side of LA 3125. The survey can be found as plan and profile sheets in Annexure 4.

## L3.3 Bathymetric Survey

Coastal Engineering Consultants, Inc was selected to perform bathymetric surveys at specific locations for this project. Their surveying tasks, as stated in a June 29th contract, included: a survey of Blind River, a survey within the Mississippi River at the location of the proposed siphon, and processing and reviewing of all compiled data.

### L3.3.1 Vertical and Horizontal Control Data

Tidal correction information was collected from CDM's survey control points to calibrate the collected bathymetric data to the project datum.

### L3.3.2 Cross Section Locations

Cross-sections through the waterways were taken at 2000-foot intervals. If a branch channel length was shorter than 2000 feet, cross-sections were conducted at its intersection with Blind River, midway, and at the end of the navigable portion of the branch channel.

### L3.3.3 Profile Alignment and Orientation

Bathymetric profiles were conducted in the Mississippi River at the location of the proposed siphon/culvert at the levee. Profiles began 500 feet from the river bank and proceeded landward to the shallowest possible water depth for the survey vessel and equipment. Survey transects were taken at 500-foot intervals beginning 2500 feet upstream and ending 2500 feet downstream of the proposed culvert/siphon location.

# L4 GEOLOGY

## L4.1 Geology of St James Parish

St James Parish lies on Alluvium and Natural Levees deposits. The Alluvium consists of gray to brownish to reddish brown or gray clay and silty clay with some sand and gravel locally. It includes all alluvial valley deposits except natural levees of major streams. Natural Levees are gray and brown or reddish brown silt, silty clay, with some very fine sand. The natural levees are near the Mississippi River, with point bars and backswamps further inland. In general, on the concave sides of the river are fine-grained natural levee deposits, undifferentiated deltaic plain swamp, and marsh materials. On the convex sides of the river bends are accretionary and point bar deposits. The alluvial deposits are fluvial sediments deposited by a rise in sea level in this region between 4000 and 6000 years ago.

Sediments underlying this region are of the Holocene Epoch, overlying Pleistocene formations. The Mississippi River valley had become deeply entrenched in the coastal plain sediments at the end of the Pleistocene Epoch when the sea level had been lowered 400 to 450 feet below its present level. About 3500 to 5000 years ago, as the sea approached its present levels, the entrenched valley gradually filled up with Holocene alluvial sediments, covering the exposed weathered and eroded

Pleistocene stratum. When the sea reached its present level, the Mississippi River migrated back and forth across the alluvial plain, building a series of delta complexes, while continually shifting the center of deposition to steeper areas. This shift displaced the Gulf waters with deposits of fine-grained material, and eventually formed the existing Mississippi River deltaic plain. The elevations of the top of the Pleistocene layer generally vary between -25 and -250 feet MSL. The Holocene sediments dip gently at about three feet per mile to the south to fill the Gulf of Mexico Basin. Local subsidence of the Holocene deltaic sediments due to compaction and consolidation contributes to loss of wetlands in the Mississippi River Delta plain.

The fine-grained natural levee and inland swamp deposits typically have lower moisture contents and higher shear strengths than similar fine-grained soils from shallow water settings. Late Pleistocene soils typically also have lower moisture contents and higher shear strengths than the younger Holocene soils.

The physical descriptions of the soils in the various geologic environments are as follows:

- Natural levee – Interfingering layers of fat and lean clays and layers of silt;
- Pointbar – Silts, silty sands, and sands with layers of clay;
- Backswamp – Homogeneous fat clays with wood, organic matter, and a few layers of silt;
- Undifferentiated deltaic plain – Fat and lean clays with lenses and layers of silt;
- Accretionary – Alternating layers of clay, silt, silty sands, and sands;
- Holocene – Fine-grained, usually clayey, and often organically rich soils;
- Pleistocene – Stiff to very stiff oxidized clays with lenses and layers of silt, silty sands, and sand.

### L5 GEOTECHNICAL INVESTIGATIONS AND DESIGN

The State of Louisiana, together with the Louisiana Coastal Authority (LCA) and the United States Army Corps of Engineers (USACE) New Orleans District, is conducting a feasibility study to restore part of the Maurepas Swamp in St. James Parish, Louisiana. CDM was retained to conduct the feasibility study for the proposed project.

## L5.1 Project Description

The Maurepas Swamp (Swamp) is one of the largest coastal fresh water swamps in the State of Louisiana, covering an area of approximately 233,000 acres. Since the construction of the Mississippi River flood control levees in the region, the swamp has been cut off from freshwater infusion, as well as sediments and nutrients hitherto provided by the Mississippi River. As a result, the swamp has undergone considerable degradation of its ecosystem, together with continual local subsidence.

The proposed project involves designing and constructing a small freshwater diversion canal from the Mississippi River to the Swamp. The proposed flow rate in the diversion canal would be less than 5000 cubic feet per second, discharging into the Blind River, which is located within the Swamp.

## L5.2 Purpose and Scope

This report presents geotechnical field investigations being undertaken at the project location.

The investigations consist of drilling and sampling 21 test borings, and installing seven (7) piezometers within the project area. **Figure L5.2-1** and **Figure 5.2-2** show the boring location plan.

Results of laboratory testing of the soil samples and water level readings from the piezometers will furnish information pertinent to the geotechnical design of the diversion canal.

## L5.3 Existing Site Conditions

**Terrain**

The project area is relatively flat, with elevations within the Swamp ranging from 1 to 3 feet, gradually increasing to about 10 feet near the Mississippi River levees south of the Swamp. The Swamp is wooded with cypress trees and other vegetation. The Blind River runs through the Swamp along with connected canals. The Interstate 10 corridor and Airline Highway also cross the Swamp.

Existing soil survey information from the United States Department of Agriculture (USDA) indicates that soils in the area are predominantly clay with occasional layers of silt; the top six inches is mostly peat. Soil information was only available to approximately 6.5 feet below ground surface.

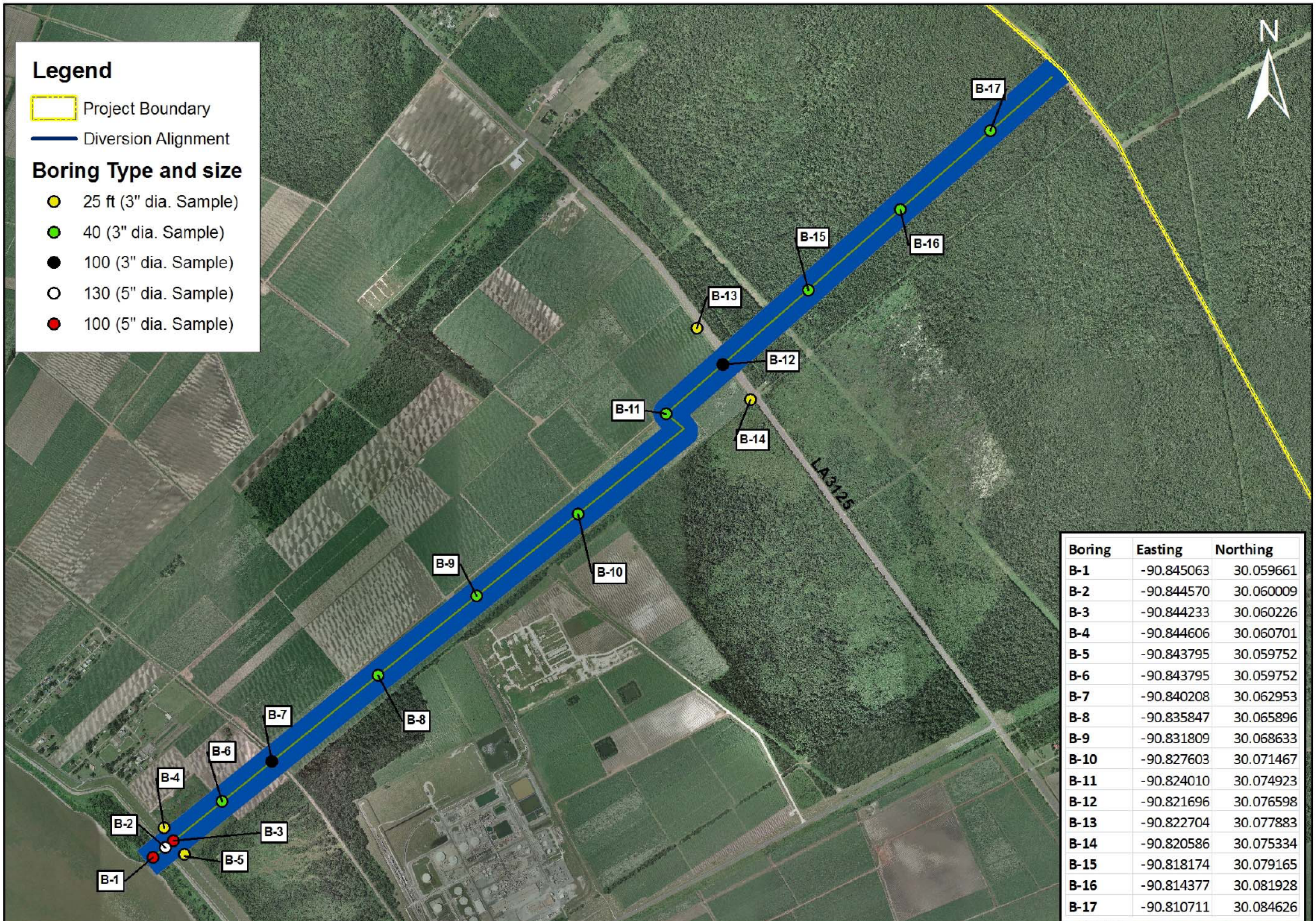
**Geology**

St James Parish lies on Alluvium and Natural Levees. Sediments underlying this region are of the Holocene Epoch, overlying Pleistocene formations. The Alluvium consists of gray to brownish gray clay and silty clay, reddish brown in the Red River Valley, with some sand and gravel. Natural Levees are gray and brown silt, and silty clay, with some very fine sand, reddish brown along the Red River. The natural levees lie near the Mississippi River, with point bars and backswamps further inland. In general, on the concave sides of the river are fine-grained natural levee deposits, undifferentiated deltaic plain swamp, and marsh materials. On the convex sides of the river bends are accretionary and point bar deposits. The alluvial deposits are fluvial sediments deposited by a rise in sea level in this region between 4000 and 6000 years ago.

## L5.4 Subsurface Investigations

### Field Exploration

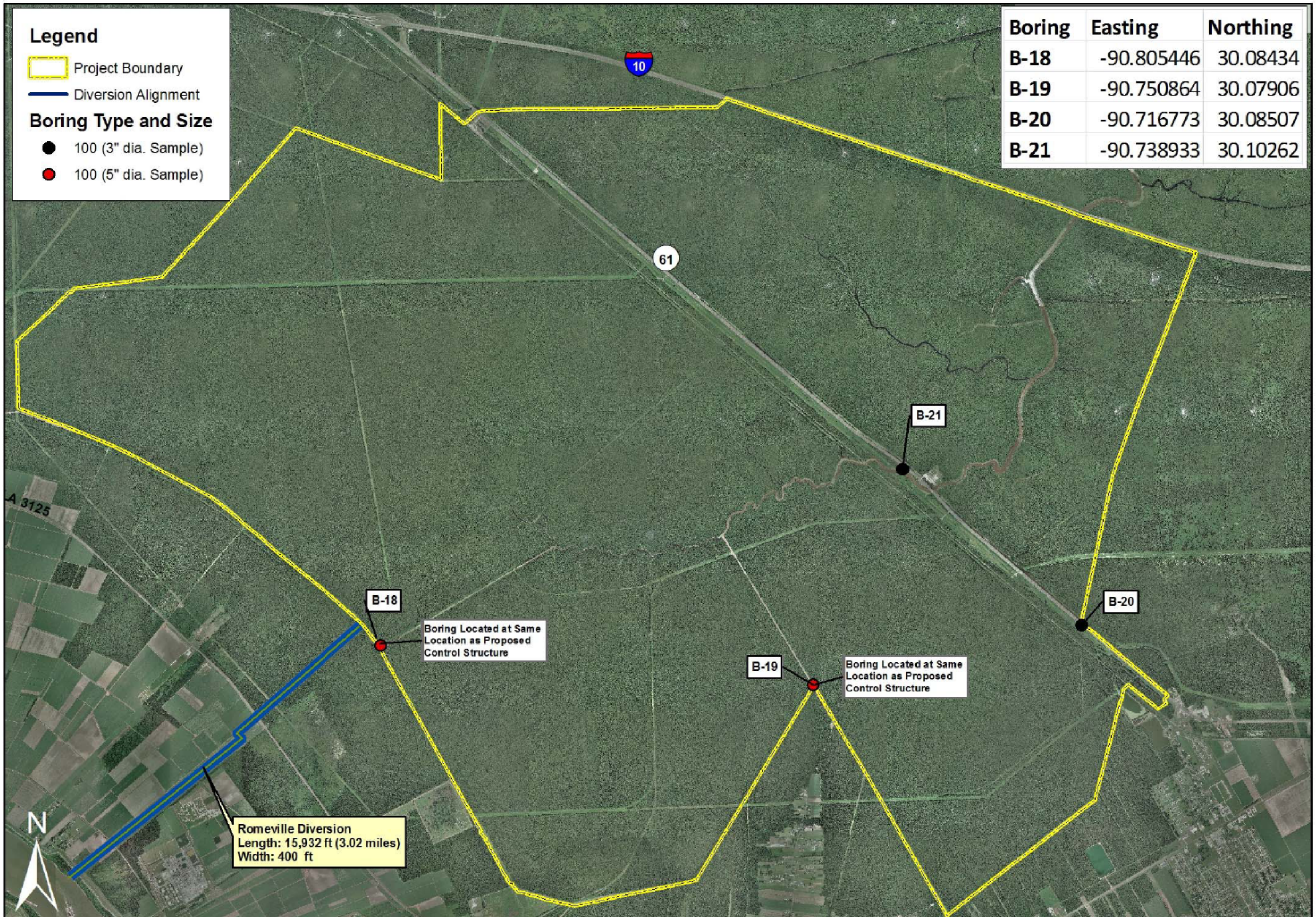
As mentioned earlier, the geotechnical field investigation consisted of drilling a total of 21 test borings and installing seven (7) piezometers. The test borings consisted of sixteen 3-inch diameter, and five 5-inch diameter borings. **Table 5.4-1** presents some information for the test borings.




| Boring | Easting | Northing |
|---|---|---|
| B-1 | -90.845063 | 30.059661 |
| B-2 | -90.844570 | 30.060009 |
| B-3 | -90.844233 | 30.060226 |
| B-4 | -90.844606 | 30.060701 |
| B-5 | -90.843795 | 30.059752 |
| B-6 | -90.843795 | 30.059752 |
| B-7 | -90.840208 | 30.062953 |
| B-8 | -90.835847 | 30.065896 |
| B-9 | -90.831809 | 30.068633 |
| B-10 | -90.827603 | 30.071467 |
| B-11 | -90.824010 | 30.074923 |
| B-12 | -90.821696 | 30.076598 |
| B-13 | -90.822704 | 30.077883 |
| B-14 | -90.820586 | 30.075334 |
| B-15 | -90.818174 | 30.079165 |
| B-16 | -90.814377 | 30.081928 |
| B-17 | -90.810711 | 30.084626 |

CDM

**Figure L.5.2-1 Boring Location Plan Romeville Diversion – Blind River Fresh Water Diversion**




| Boring | Easting | Northing |
|---|---|---|
| B-18 | -90.805446 | 30.08434 |
| B-19 | -90.750864 | 30.07906 |
| B-20 | -90.716773 | 30.08507 |
| B-21 | -90.738933 | 30.10262 |

CDM

**Figure 5.2-2 Boring Location Plan Project Boundary Area – Blind River Fresh Water Diversion**

Borings B-7 through B-14 and B-18 through B-21 have been completed, with the samples at the laboratory testing stage. Borings B-1 through B-6, which are close to the Mississippi River levee, will be drilled once the Pontchartrain Levee District approves the drilling permit application. Borings B-15 through B-17 will be drilled upon permit approval by the Louisiana Office of Coastal Restoration and Management. The completed borings were drilled and sampled between January 18 and March 5, 2010.

Before drilling, the borings were located and staked in the field using a handheld GPS device. The boring locations are shown on Figures L5.2-1 and L5.2-2.

**Table 5.4-1. Test Borings Information**

| Boring | Boring Depth (ft) | Boring Diameter (in.) | GPS Coordinates | | Groundwater Depth (ft) |
|---|---|---|---|---|---|
| | | | Easting | Northing | |
| B1 | 100 | 5 | -90.84506 | 30.05966 | |
| B2 | 130 | 5 | -90.84457 | 30.06000 | |
| B3 | 100 | 5 | -90.84423 | 30.06023 | |
| B4 | 25 | 3 | -90.84461 | 30.06070 | |
| B5 | 25 | 3 | -90.84380 | 30.05975 | |
| B6 | 40 | 3 | -90.84380 | 30.05975 | |
| B7 | 100 | 3 | -90.84021 | 30.06295 | 0.5 |
| B8 | 40 | 3 | -90.83585 | 30.06590 | 0.2 |
| B9 | 40 | 3 | -90.83181 | 30.06863 | 0.3 |
| B10 | 40 | 3 | -90.82760 | 30.07147 | 0.3 |
| B11 | 40 | 3 | -90.82401 | 30.07492 | 1.5 |
| B12 | 100 | 3 | -90.82170 | 30.07660 | Not Recorded |
| B13 | 25 | 3 | -90.82270 | 30.07788 | 1.0 |
| B14 | 25 | 3 | -90.82059 | 30.07533 | 3.0 |
| B15 | 40 | 3 | -90.81817 | 30.07917 | |
| B16 | 40 | 3 | -90.81438 | 30.08193 | |
| B17 | 40 | 3 | -90.81071 | 30.08463 | |
| B18* | 100 | 5 | -90.80545 | 30.08434 | 3** |
| B19* | 100 | 5 | -90.75086 | 30.07906 | 8** |
| B20 | 100 | 3 | -90.71677 | 30.08507 | |
| B21 | 100 | 3 | -90.73893 | 30.10262 | |

*Drilled in Blind River

**Depth to mudline

The borings were drilled using a track-mounted drilling rig, except borings B-18 and B-19 in the Blind River, which were drilled with a pontoon-mounted drilling rig. Each boring was sampled with the solid stem auger technique until groundwater was first encountered and recorded; the wet rotary sampling technique was used thereafter.

Split spoon samples, typically taken in cohesionless soils, and Shelby tube samples, typically taken in cohesive soils, were collected continuously to a depth of 10 feet below existing ground surface, and then at 5-foot intervals thereafter until boring termination. Shelby tube sampling was conducted in general accordance with ASTM D 1587, Standard Practice for Thin-Walled Tube Sampling of Soils for Geotechnical Purposes. The Shelby tubes were extruded on-site for visual classification and storage. Split-spoon sampling was conducted in general accordance with ASTM D 1586, Standard Test Method for Standard Penetration Test (SPT) and Split-Barrel Sampling of Soils. For the 24-inch split-spoon sampler used, the sampler was driven 18 inches into the ground at 6-inch increments. The number of blows required to drive the sampler each 6-inch increment was recorded, and the Standard Penetration Resistance (N-value) was determined as the sum of the blows over the 2nd and 3rd increments. Representative soil samples were taken from each split-spoon or Shelby tube sample, stored in moisture proof containers, and securely transported to the laboratory for later review and geotechnical laboratory testing. The borings were backfilled with cement-bentonite slurry after final groundwater readings were recorded. Borings drilled in the Blind River were backfilled immediately after drilling.

Field logs were prepared by a CDM geotechnical engineer, who also observed the test borings in the field. Final boring logs will be prepared upon receiving test results back from the laboratory. Drilling and laboratory testing are being performed by Professional Service Industries, Inc. (PSI). Completed boring logs are provided **Annex L-3**.

## L5.5 Laboratory Testing Program

The laboratory testing program for undisturbed and disturbed samples obtained from the borings consisted of the following:

- Moisture Content
- Atterberg Limits
- Unit Weight
- Sieve Analysis (percent passing #200)
- Unconfined Compression Test
- Triaxial Test ( UU test- 3 point)

The preceding laboratory tests, conducted according to ASTM standards, will provide the necessary geotechnical parameters for design and construction purposes. Available laboratory test results are shown on the completed boring logs in Annex L-3.

## L5.6 Subsurface Conditions

Final boring logs from completed sample testing indicate that subsurface soils are mostly brown and gray stiff clay with occasional loose silt and fine sand lenses and layers. The silt and sand layers were usually encountered between 30 and 50 feet below ground surface.

Some soft clay was encountered in some of the borings, usually between 0 and 25 feet below ground surface. In boring B-18, the soft clay extended to 65 feet, and in B-21 soft clay was encountered at 73 to 78 feet.

In most of the borings, soil color changed to red-brown between 25 and 50 feet.

## L5.7 Groundwater

Final groundwater levels were usually measured 24 hours after drilling. Groundwater generally varied between 0.2 and 3 feet below ground surface.

## L5.8 Variation in Subsurface Conditions

The interpretation of general soil conditions is based on soil and groundwater conditions observed at the test boring locations. However, subsurface conditions may vary at locations other than the subsurface exploration locations.

Groundwater levels are expected to fluctuate with season, temperature, river stage, and other factors.

## L5.9 Closure

This geotechnical field investigation report has been prepared for the proposed Blind River Freshwater Diversion canal in St. James Parish, Louisiana. This report presented geotechnical field investigations, including available results of laboratory testing on selected soil samples. The methods and procedures used in this report are in accordance with generally accepted engineering practices. No other warranty, expressed or implied, is made.

A final geotechnical report including recommendations for slope stability, foundation support for various diversion structures and other relevant design requirements for the proposed diversion project will be issued once the final alignment, depth, hydraulic modeling and other design features have been completed.

## L6 ENVIRONMENTAL ENGINEERING

### L6.1 Incorporation of Environmental Compliance Measures into Project Design

Environmental compliance measures are an integral component of all planning, construction and operation & maintenance activities associated with this project. These measures have been developed in full coordination with involved Federal and state agencies and made part of the public review process as required by regulations. The documentation and details of all environmental compliance is reported in the Integrated Feasibility Report. Environmental compliance measures are related solely to the timing and methods used for dredged material disposal during both project construction and project maintenance. The plan for dredged material disposal is contained in the project EIS. The EIS will be referred to during the Preconstruction Engineering and Design (PED) phases of this project. Additionally, a detailed description of the long-term dredge disposal plan will be included in the project OMRR&R Manual.

### L6.2 Incorporation of Environmental Sensitivity

Environmental sensitivity has been incorporated into all aspects of project design, construction, and operation & maintenance activities associated with this project. The beneficial use of dredged material incorporates the recommendations of Federal and state resource agencies to the maximum extent practicable. This also includes their recommendations on the avoidance and minimization of adverse impacts which may occur during construction and operations & maintenance activities. Construction methods that will enhance environmental features to the maximum extent practicable will be incorporated into the designs of the various features of this Project.

## L7 CIVIL DESIGN

This section presents the preliminary civil design for the components making up the Convent/Blind River Small Diversion project. Since the project is primarily civil works, this section also discusses the coordination needs for other design and engineering disciplines, as these typically have direct impacts on the civil design. These include hydraulics, transportation, geotechnical, mechanical, structural, electrical, and instrumentation.

The purpose of the Blind River diversion project is to divert fresh water into the Maurepas Swamp to freshen the Swamp, provide nutrients and sediment to enhance growth, and counter potential backflow of water from Lake Maurepas containing elevated levels of salinity. The hydraulic and the hydro-dynamic analyses identified means to divert the fresh water from the Mississippi River, deliver it to the Swamp, and distribute it within the Swamp to accomplish the environmental goals. The hydro-dynamic analysis also identified specific actions necessary to improve the distribution and circulation of the water into and within the Swamp. These included opening large gaps in the existing spoil banks along the existing drainage channels and adding cross culverts at the KCS RR and Hwy 61 corridor to improve drainage and circulation between the hydrologic units in that area. The environmental and hydro-dynamic analyses identified 3,000 cfs as the appropriate design flow rate for the project.

The system components for the Romeville diversion were designed to address the hydro-dynamic and environmental concerns identified in the analyses, and to meet the project's environmental goals. The Romeville diversion alignment has six major components: a diversion culvert facility, a transmission canal, approximately six control structures of various sizes, approximately 30 berm gaps, cross culverts at four locations along the Highway 61 corridor, and instrumentation. The major project components are primarily hydraulic conveyance and hydraulic control structures designed to divert fresh water from the Mississippi River, transfer it to the Maurepas Swamp, and distribute and direct the diverted water into and through the Swamp. The preliminary hydraulic design is documented in Section L2.

## L7.1 Diversion Culvert Facility

### L7.1.1 Description – Diversion Culvert

The diversion culvert facility will divert fresh water from the Mississippi River, transfer it under the east levee through a box culvert, and discharge it into the transmission canal. The primary hydraulic elements of the diversion culvert facility are as follows:

- 3 – 10' x 10' multi-cell cast-in-place reinforced concrete box culverts under the east levee and LA 44;
- 3 – 10'x10' Sluice gates with motor operators on the culvert inlets to control the diversion flow rate and, when required, to completely block the flow;
- Trash racks near the culvert inlet to keep large debris out of the diversion system; and
- Inlet canal across the batture from the Mississippi River to the culvert inlet.

LA 44 (River Road) is adjacent to the levee and the box culvert will be extended under the road and discharge into the transmission canal 100 feet east of the road. Erosion protection will be provided at locations with relatively high flow velocities and turbulence, such as at the Mississippi River bank, in the inlet canal entrance, at the box culvert entrance and exit.

Ancillary elements at the diversion culvert facility include a gate, a cut-off wall in the levee for seepage control, and two sets of stop logs. The diversion site will include an access driveway, a site road for access to the top of the levee, fence (6' chain link fence with 3-strand barbed wire), drainage, lighting, a security system, and a control building. The major temporary construction facilities will include a temporary earthen levee, a cofferdam on the east bank of the Mississippi River, and a temporary detour road for LA 44.

### L7.1.2 Civil Design – Diversion Culvert

The box culvert will be installed through/under the existing east levee of the Mississippi River by open-cut methods. The facility design is focused on engineering and design, construction techniques, temporary construction facilities, and sequencing to maintain the flood protection integrity of the east levee to current levels.

The elements of the diversion facility were designed on the following basis:

- Temporary earthen levee – A temporary earthen levee will be constructed on the batture, on the river side of the existing levee, to allow open-cut construction of the box culverts through the levee. The temporary levee will have riprap lining on the river side to protect the more exposed levee from erosion during floods stages in the river. The temporary levee will be designed to the standards of the existing levee to maintain the full flood protection integrity of the existing levee system. The temporary levee may be in place for approximately two years.
- Cofferdam – A cofferdam will be installed on the east bank of the Mississippi River to make the tie-in of the inlet canal into the Mississippi River, including placing riprap on the river bank. The initial concept extended the cofferdam down to Elev. -10 on the river bank. The cofferdam protects only access to the final tie-in to the river, and will not be part of the overall levee flood protection system. Therefore, the top of the cofferdam will not protect from the higher Mississippi River flood elevations, but will match the batture, near Elev. 24.
- Erosion protection – Erosion protection consisting of riprap and concrete channel lining will be installed at areas with potential for destructive erosion, such as high velocities or turbulence. The upstream end of the inlet canal and the entrance and exit at the box culvert will also have erosion protection.
- Romeville revetment – the inlet canal will penetrate the Romeville revetment. Large riprap will be extended into the inlet canal to maintain the existing revetment erosion protection system.
- Inlet canal – The intake for diversion facility will need to cross the 200 to 300-foot-wide batture from the Mississippi River bank to the levee. Based on a cost comparison, an inlet canal will be less expensive than extending the box culverts across the batture. See the transmission canal for discussion of inlet canal design.
- Cut-off wall – A steel sheet pile cut-off wall will be incorporated into the box culvert and a gate tower wall to maintain levee stability and to reduce the

potential for seepage and piping (loss of fines) through the reconstructed levee and along the box culvert backfill.

- Box culverts – The box culverts will be large reinforced concrete structures designed to support the levee overburden. The culverts are designed as cast-in-place monolithic structures supported on a pre-cast concrete pile foundation.
- Sluice gates –Sluice gates with motor operators will be installed on the upstream end of the box. The sluice gate position is designed to have a positive seating head during Mississippi River flood stages.
- Stop logs – Stop logs will be provided on both sides of the sluice gates to fully isolate the sluice gates for maintenance. The upstream stop logs will be placed at the gate tower and will block the Mississippi River. In the event a sluice gate is out of service, the upstream stop logs will also provide redundant capabilities to fully block flow through the culverts during flood stages in the Mississippi River. The downstream stop logs will block backflow from the transmission canal. Both sets of stop logs will be positioned to have a positive seating heads.
- Trash racks – Trash racks with a coarse grid size will be place near the culvert inlet to reduce the potential for large debris from passing through the installation. At this point in the design, fine screens are not considered necessary to reduce the potential for fish passage.
- Gate tower – The sluice gates and one set of stop logs are on the inside, or river side, of the levee. A gate tower will be constructed over the box culverts to elevate the motor operators and provide access to the top of the levee, well above the Mississippi River flood stage
- Site facilities – The site facilities will be designed during the final design phase.

#### L7.1.3 Geotechnical Coordination

The following items will require geotechnical input for the design and layouts:

- Temporary earthen levee – Foundation preparation, material specifications, slope, placement and compaction, settlement potential, seepage control measures, tie-in to the existing levee, use of material from the transmission canal excavation.
- Cofferdam – Horizontal loadings, sheet pile length, sheet pile embedment, pile section.
- Riprap – Bedding or geotextile requirements.
- Dewatering – Existing groundwater conditions (batture, levee, and LA 44), dewatering methods.

- Cut-off wall at the levee – Location in relation to levee, width along levee, depth, seepage potential, cut-off wall material (i.e. – steel sheet piling), loading, pile section, length; other potential seepage control measures.
- Box culvert foundation – Foundation requirements, such as concrete piles (loading, size, length, loading/bearing capacity)
- Box culvert design – Vertical loading on box culvert at the levee and at LA 44, horizontal loading, bedding and backfill.
- Temporary shoring – Temporary shoring design for open-cut excavations.
- Headwall and retaining wall designs – Loads, backfill material specification
- Permanent levee reconstruction – Material specifications, verify removed material is suitable, placement and compaction, settlement potential, seepage control measures, tie-in to existing levee.
- LA 44 road crossing – Subgrade, base, and pavement recommendations for the detour, and for the reconstructed road.
- Control building – Foundation recommendations.
- Inlet canal – See the transmission canal paragraph.

## L7.2 Transmission Canal

### L7.2.1 Description – Transmission Canal

The transmission canal will transfer the diverted water approximately three miles from the diversion culvert facility to an existing drainage channel at the perimeter of the Swamp. The primary hydraulic elements of the transmission canal are as follows:

- Earthen trapezoidal channel Canadian National Railroad (CN RR) crossing; and
- LA 3125 crossing.

Other site improvements for the transmission canal include fences at each right-of-way line, access driveways, access roads on the berms, drainage, and vegetation cover. Site lighting and security cameras are not planned for the transmission canal. The only major temporary construction facilities are cofferdams.

### L7.2.2 Civil Design – Transmission Canal

The civil design basis for the transmission canal elements is as follows:

- Earthen canal – The canal will be an earthen trapezoidal channel section, with a 155-foot-wide bottom, 4:1 (H:V) side slopes, and a depth of approximately 12 feet, including a 2-foot freeboard. The top width will be approximately 250 feet.
- Canal Side Slopes – The transmission canal is currently designed conservatively for 4:1 (H:V) side slopes. Steeper side slopes would be desirable

to increase the conveyance effectiveness of the section, and to reduce the canal width. The geotechnical report will provide the final value.

- Embankments/Berms – Embankments or berms will be constructed on both sides of the canal. The berms have a top width of 12 feet, for use as access for maintenance and operations. The exterior side slope will have a 4:1 or 5:1 (H:V) side slope to allow safe access by mowing equipment. These slopes will be determined during future geotechnical analyses.
- Freeboard – The diversion flow rate through the diversion structure is expected to vary due to changing stages in the Mississippi River, and changing sluice gate settings. Therefore, the transmission canal was designed for a 25% higher flow rate. Provide a freeboard of 2 feet above the design water surface elevation at the canal design flow rate.
- Right-of-Way Width – A minimum right-of-way width was estimated to allow space for the channel, the berms, drainage along the edge of the right-of-way, and fences. The space from the toe of the berm to the fence line is recommended to be 10 feet to provide for drainage and allow mower access. Without berms, provide a minimum of 30 feet each side for large maintenance equipment and drainage ditches. The actual right-of-way is anticipated to be wider than the minimum required, as the available tract is 400 feet wide.
- LA 3125 Road Crossing – Reinforced concrete box culverts will be used for the LA 3125 road crossing. The culvert will extend across the full right-of-way width. During final design, it will be investigated if a bridge is more cost effective.
- CN RR Crossing – Reinforced concrete box culverts will be used for the CN RR crossing. The culvert will extend across the full right-of-way width. During final design, it will be investigated if a bridge is more cost effective.
- Erosion Protection – Riprap and concrete channel lining will be installed at both sides of the culvert crossings. Riprap will be installed at the outfall into the existing drainage channel.
- Cofferdam – A cofferdam will be installed in the existing Parish drainage channel at the downstream (east) end of the transmission canal. This will allow excavation of the final segment of the canal at the drainage channel, which is nearly full with standing water.

### L7.2.3 Geotechnical Coordination

The geotechnical investigation will address the following transmission canal design items:

- Channel side slopes – The preliminary hydraulic design is based on 4:1 side slopes. The geotechnical investigation will provide a slope recommendation based on long-term stability and on rapid drawdown conditions.

- Canal berms/embankments – Material, placement, and compaction requirements for the fill sections to create the berms.
- Liner - The HGL will be above natural ground. Determine if there will be a seepage concern through the berm and natural ground, and if a liner, such as clay, would be required to control seepage losses or if seepage-control drains are needed for stability.
- Excavated material suitability – Evaluate the properties of material to be excavated for the canal, and determine if it can be used for the berms and for the temporary earthen levee at the Mississippi River. The analysis will also indicate other beneficial uses for disposal of excess material on-site or off-site.
- Groundwater conditions and dewatering – Define existing groundwater conditions and expected conditions after construction. Also define dewatering methods for construction.
- Erosion protection – The canal will have relatively low velocities; however, the geotechnical investigation will need to identify soil layers that may be vulnerable to erosion, and if erosion protection is required.
- Culverts at railroad – Foundation, bedding, and backfill, temporary shoring, and loading recommendations for the box culverts and associated retaining walls and wingwalls.
- Culverts at road - foundation, bedding, and backfill, temporary shoring, and loading recommendations for the box culverts and associated retaining walls and wingwalls.
- Cofferdam – cofferdam design parameters for a cofferdam at the downstream of the channel, including loadings, sheet pile section, length, embedded length

## L7.3 Control Structures

### L7.3.1 Description – Control Structures

The project will use the existing drainage channels at the perimeter of the Swamp to distribute the diverted flow throughout and into the Swamp. The hydraulic grade line, or water surface elevation, will need to be raised above the existing levels and controlled to force the diverted water out of the drainage channels into the Swamp. Control structures with control gates will be installed at key locations in the existing channels to perform this function.

The proposed control gate is a specialty gate that rotates on a shaft at the bottom of the channel and is operated by large hydraulic cylinders. The gate will be rotated up to the vertical position to increase the water surface elevation during the flow diversion. The gate will be rotated down to the channel bottom into the open position when there is no diversion, to allow for normal drainage, and to allow the passage of boats and barges. The control gates will be installed in large concrete structures constructed in the existing drainage channel.

Instrumentation, controls, a hydraulic power unit, and a generator will be located in a precast concrete building at each control structure site.

### L7.3.2 Civil Design – Control Structures

The control structure designs are based on establishing and maintaining a set water surface elevation to force flow through the proposed berm gaps into the Swamp and to allow flow over the control gate to downstream segments of the project. The control gates require power for the hydraulic power unit to operate the large gates.

The control structures are located in the Swamp, where the existing natural ground is approximately Elev. 2 and the static water surface is Elev. 1.5 to 2.0. The proposed operating water surface is approximately Elev. 4.0. The civil design items are:

- Control Gate – The control gates are designed to fully block flow in the existing channel and raise the upstream water surface to Elev. 4.0. The gates are not designed based on flow rate. However, the gate position will be based on allowing water flow over the gate.
- Control Structure - The size of the control structures will vary at each location, and are based on the width and depth of the existing drainage channels. The top of the side walls will be at Elev. 5.0, 1 foot above the operating water surface elevation.
- Berms/embankments – The existing berms typically have a top at Elev. 4 to 6. A berm will be extended from the control structure to the existing berm to prevent the diverted water from by-passing the control structure.
- Control Building – The HPU, controls, and generator will be housed in a pre-cast concrete building, which will be elevated above flood levels on concrete columns or piles. The existing 100-year flood level is approximately Elev. 5.0, and the finished floor will be placed at Elev. 7.0 so that the bottom is a minimum of 1 foot above the 100-year elevation.
- Power – The installations are in remote areas, and a generator will be required for power.
- Hydraulic Power Unit – A hydraulic power unit will be required to operate the large control gate hydraulic cylinders. Large gate installations may require cylinders on each side. If routing hydraulic lines through the concrete structure is not feasible, it may be necessary to have HPU's on each side of the channel, and an additional protective pre-cast concrete building.
- Site improvements – 6' chain link fence with 3-strand barbed wire, video camera. As the site is isolated in the Swamp, there are no drainage or access provisions.

### L7.3.3 Geotechnical Coordination

Geotechnical recommendations will be for:

- Cofferdam – horizontal loadings, sheet pile length, sheet pile embedment, pile section.
- Foundation preparation – require removal of silt and unsuitable material in channel.
- Foundation requirements – Concrete piles – loads, size, length, loading/bearing capacity.
- Groundwater conditions and dewatering recommendations.
- Retaining wall designs – Loads, backfill material specification
- Foundation for pre-cast building – pre-cast concrete piles vs. drilled shaft piers, load, size, length, loading/bearing capacity.

## L7.4 Berm Gaps

### L7.4.1 Description – Berm Gaps

When the existing drainage channels were excavated in the Swamp, the excavated material was cast to one side of the channel forming spoil banks. The size of the spoil banks vary, with the top elevations ranging from Elev. 4 to Elev. 12. From field observations and the hydro-dynamic modeling, it has been determined that the spoil banks currently block flow circulation into and out of the swamp, resulting in stagnant areas and poor circulation of water through the hydrologic units. In the current configuration, the spoil banks would continue to prevent the diverted water from easily entering and flowing through the Swamp. Therefore, new 500-foot-wide berm gaps will be excavated in the spoil banks at an approximate spacing of 2,500 feet on center.

### L7.4.2 Civil Design – Berm Gaps

The gaps will be excavated to the elevation of the adjacent Swamp natural ground elevations and the spoil will be disposed behind the existing spoil banks. The spoil will be piled up to Elev. 6 to provide additional refuge areas for wildlife during flood events in the Swamp.

The proposed berm gaps were sized to have low velocities through the opening, and to reduce the chance of complete blockage by debris. The hydro-dynamic modeling confirmed that the berm gap size and spacing provided adequate capacity for distribution from the channels into the Swamp.

The gap elevations, design WSEL in the channel, and downstream HGL conditions will vary, resulting in different flow rates through each gap. However, assuming uniform conditions, the gaps will have the flow characteristics presented below for a 3,000 cfs diversion. Note that not all diverted flow will go through the berm gaps. Also, the berm gaps are excavated to the elevation of the Swamp, and a weir is not created.

| | |
|---|---|
| No. of gaps: | 30 |
| Length of gap: | 500 feet |
| Floor of gap: | Elev. 2.0 |
| Operating WSEL: | Elev. 4.0 |
| Flow rate each gap: | 100 cfs |
| Approx. area: | 1,000 SF |
| Velocity: | 0.1 fps |

Construction will consist of moving approximately 3,000 CY of material at each gap. The fill will be placed in an unconsolidated, uncontrolled compaction disposal area.

### L7.4.3 Geotechnical Coordination

The following input will be from the geotechnical investigation:

- Reinforce the soil to support construction equipment – (i.e., geotextiles)

## L7.5 Cross Culverts at the Highway 61 Corridor

### L7.5.1 Description – Cross Culverts

The hydrodynamic modeling of the Swamp project area indicated that the KCS RR and the Highway 61 embankments disrupted the natural flow and circulation of water through the Swamp. This resulted in hydrologic units east and west of the KCS RR/Hwy 61 corridor having stagnant water, poor drainage, and lack of sources of fresh water input. New culvert crossings will be added under the KCS RR and Hwy 61 at four locations to re-establish circulation. Each installation will consist of 3 – 3' x 4' reinforced concrete box culverts. Note that there may be sufficient cross drainage openings at the KCS RR and additional culverts may not be required. Earthen channels (large ditches) will be excavated across the 500-foot space between the KCS RR and Hwy 61 to interconnect the drainage capacity at the railroad with the new culverts at Hwy 61.

### L7.5.2 Civil Design – Cross Culverts

Civil design for the cross culverts include:

- Culvert cross sectional area - The combined cross sectional area of the culverts is 144 SF. At 0.5 fps, a velocity indicated by the hydro-dynamic modeling, the flow rate is 72 cfs, or a daily volume of 150 ac-ft. If the total Swamp has an average depth of 6", the total volume is 11,200 ac-ft. The cross culverts will allow approximately 1.5% of the Swamp volume to circulate per day, or ½ of the total Swamp volume each month.

- Cross culverts – Reinforced concrete box culverts. The culverts at Highway 61 will likely have minimal cover under the pavement and will be ASTM C-850 precast units. Cast-in-place construction will also be allowed.
- Interconnecting drainage channel – A small drainage channel will be extended across the 500-foot wide corridor between the railroad and the highway. The flow line profile will be set to match existing drainage flow lines at the upstream and downstream ends. The channel section will have a 6-foot-wide flat bottom and 3:1 side slopes (to be confirmed by the geotechnical investigation).
- Erosion protection – The channel will have low velocities, and no erosion protection will be required. However, the channel will have 10-foot segments of riprap-lined channel at the culvert entrances and exits. The channels will not have embankments/berms, as the HGL will be at or near natural ground.
- Access roads – There will be 12-foot wide access roads on each side of the interconnecting drainage channel for maintenance access, and for access to the crossings at the railroad.

#### L7.5.3 Geotechnical Coordination

Geotechnical input will be required for the following items.

- Interconnecting channel – see transmission canal
- Box culvert design – Vertical loading on box culvert, horizontal loading, bedding and backfill.
- Temporary shoring for open-cut excavations
- Highway 61 road crossing – Subgrade, base, and pavement recommendations for the detour and for the reconstructed road.
- Groundwater conditions and dewatering recommendations

## L7.6 Instrumentation

### L7.6.1 Description - Instrumentation

Instrumentation will be required to monitor and control the diversion flow rate and the water surface elevations in the diversion, transmission, and distribution system in the Swamp. Design of the instrumentation has been coordinated with the civil design, the hydraulic design, and the operation scheme.

Typically, flow rates and water levels will be measured and the feedback data will be used to adjust gate positions to control the desired parameters at the diversion culvert and the control structures. The monitoring and control data will be collected, analyzed, and transmitted to and from a control building on the diversion culvert site. Following are the main instrumentation for data collection and control at each component:

- Diversion Culvert – The flow control at the diversion culvert will establish the flow rate for the project. The diversion flow rate will likely be set manually by an operator, with adjustments as necessary. The diversion culvert will have instrumentation for water levels at the culvert entrance and exit, for flow measurement, and for sluice gate positions. The control system at the diversion structure will be designed to automatically adjust the sluice gate openings as the Mississippi River stage varies to maintain a constant flow rate.
- Control Structures – The control gates at the control structures will require water level measurement on both sides of the gates, and gate position measurement, to control gate position, water levels, and flow rates over the gates. The control gates will likely have manually set positions, with occasional adjustments based on feedback from system monitoring.
- Transmission Canal – There will be no instrumentation in the transmission canal to control flow rates or water surface elevations. However, the transmission canal will have level monitors at several locations to ensure that the berms are not overtopped.
- Water level monitors will be required in the Blind River at Highway 61, at I-10, and possibly additional locations on Blind River and on the existing drainage channel network within the Swamp. These monitors will provide feedback for the flow rate control and control gate settings.
- The environmental monitoring and hydrological monitoring and data collection within the Swamp have not yet been defined in detail.
- Communications to remote sites will be via satellite, with dishes installed on top of the control buildings at the control structure sites.

There will be no flow or water level control at the following components:

- Berm Gaps – there will be no flow measurement, level measurement, or controls at the individual berm gaps. All water level control will be at the control structures.
- Cross Culverts at the Hwy 61 Corridor – there will be no flow measurement, level measurement, or controls at the four cross culvert locations.

The data collected from the project will be used as input for adaptive management.

Real-time data are required from the system components to allow the operator to control and adjust the system flow rates.

Initial designs considered communications via radio towers at each control structure and in the Highway 61 corridor to communicate to the central control building via a radio tower at the diversion facility. The towers would be 150 to 200 feet tall to have clear line-of-sight communications above the mature Bald Cypress trees. The satellite system was selected as a more cost effective alternative, and to reduce the visual impacts to the natural setting in the Swamp.

#### L7.6.2 Civil Design – Instrumentation

None.

#### L7.6.3 Geotechnical Coordination

None.

## L8 STRUCTURAL DESIGN

## 8.1 Codes and Standards

### 8.1.1 Governing Code.

The governing code for the project will be as follows. In no case will the strength, serviceability, or quality standards for materials and procedures be less than that required by the governing code.

- 2006 International Building Code

Where the provisions contained or referenced herein differ from those in the governing code, design will be performed in accordance with the most stringent.

### 8.1.2 Supplemental Codes and Standards.

The following codes and standards will be used for design when and as specifically referenced herein.

- USACE Engineering Manual 1110-2-1913 Design and Construction of Levees.
- National Flood Insurance Program - NFIP Regulations.
- American Concrete Institute - ACI 318

- American Concrete Institute - ACI 350R with recommendations
- American Association of State Highway and Transportation Officials - AASHTO Bridge Specification
- American Welding Society - AWS D1.4

## 8.2 Design Loads and Serviceability

### 8.2.1 Scope

All applicable loads and load combinations will be determined as required by the governing code, occupancy, site and environmental effects, equipment and processes. Appropriate load combinations will be established and as well as appropriate allowable stresses, load factors and safety factors (as applicable). These criteria will be established at the beginning of preliminary design and confirmed at the beginning of final design.

### 8.2.2 Dead Loads

Dead loads are those resulting from the weight of all fixed construction such as walls, partitions, floors, roofs, cladding, equipment bases and all permanent, nonremovable, stationary furnishings.

Numerical values for the dead load of well-defined components of a structure will be used as documented in the following publications:

- ASCE 7
- AISC Manual
- CRSI Handbook
- Manufacturer's catalogs for fabricated components

### 8.2.3 Live Loads

Live loads will consist of all loads due to occupancy, furnishings and equipment. Live load reduction will not be employed for members of large influence area in the design of environmental and industrial facilities, due to the relatively high probability of simultaneous loads on all areas. Live load reduction may be employed in general buildings that are not part of environmental or industrial facilities, with approval.

#### 8.2.3.1 Uniform Live Loads

Uniform live loads will be established in accordance with the governing code. Values are listed below for purposes of preliminary design. Actual usage and equipment will be considered during final design and higher loadings used when appropriate.

**Process Structures**

- Roadways and slabs designated — AASHTO load or for vehicle traffic design vehicle
- Slabs on grade — 300 psf

**8.2.3.2 Equipment Loads**

Loads from equipment will be considered live loads. The maximum loads and support details for each major piece of equipment will be provided by the discipline designing or specifying it. Final weights of process-mechanical equipment and gates will be established during preliminary design.

In addition to the mechanism's static dead load, design will be performed for other effects, such as those due to operation, maintenance and malfunction. Examples include, but are not limited to, the following.

- Sluice gates, non-self-contained: Design will be performed for a load equal to the breaking strength of the operating stem, or the stalling torque of the motorized operator, in the event the gate is frozen.
- All equipment: Design will be performed for required maintenance procedures, such as the removal of a large component and the placing of it temporarily on the adjacent structure.

**8.2.3.3 Impact Loads**

Static loads will be increased for the effects of impact by the following percentages.

- Vehicular loads: In accordance with the AASHTO Specification

**8.2.3.4 Construction Live Loads**

The contractor is normally responsible for maintaining loads on partially or fully complete structures at or below the design live loads noted on the drawings, or for providing supplemental support. However, in certain cases, if the service load on a structural element is negligible, a design load will be used which would accommodate reasonable construction activities.

When it is necessary to provide particular restrictions on construction sequencing, special loads conditions may result. This is particularly applicable to work involving the modification of existing structures. These cases will be evaluated and appropriate criteria established during final design. Such restrictions will be indicated in the drawings or specifications.

### 8.2.4 Environmental Loads

#### 8.2.4.1 Wind Loads

Wind loads will be developed from the following criteria in accordance with the governing codes. Appropriate shape modification factors, uneven distributions, and orthogonal effects will be considered for each structure. Increased allowable stresses or reduced load factors will be used, as appropriate. Concrete mass foundations and/or guy-wire anchors will be used to resist wind loads on tower foundations.

#### 8.2.4.2 Seismic Loads

Seismic loads will be developed from the following criteria in accordance with the governing codes. Increased allowable stresses or reduced load factors will be used, as appropriate.

### 8.2.5 Process Liquid Loads

Design will be performed for liquid loads assuming liquid surface at the maximum working level using normal allowable stresses, or the load factor for a live load, as appropriate. In addition, design will be performed assuming the liquid surface at the maximum possible level under surcharge conditions using an increase in allowable stresses, or the load factor for a dead load, as appropriate.

### 8.2.6 External Earth and Groundwater Loads and Freeboard

Earth and water loads will be developed in accordance with the project geotechnical report when complete and the governing codes. A minimum freeboard as required in coastal levees as required by NFIP Regulations but not less than 3.0 feet above the Base Flood Elevation (BFE) all along length, and an additional 0.5 foot at the upstream end of a levee.

Design will be performed for pressures generated by groundwater acting laterally, downward and upward, as appropriate. Loads factors appropriate for live loads will be used. Design will be performed for groundwater at the normal elevation for normal allowable stresses or load factors, as appropriate. Design will be performed for groundwater at the following flood elevation for increased allowable stresses or reduced load factors, as appropriate.

**Lateral Soil and Groundwater Pressures**

**Hydrostatic**

The following equivalent fluid pressures will be used in preliminary design for well-graded, granular, mineral soils with an estimated moist unit weight of 120 pcf. Soil pressures for final design will be developed in accordance with the geotechnical report. Design for cantilevered walls of environmental engineering structures will be performed for at-rest soil pressures.

**Surcharge**
Walls and structures to which vehicles can reasonably be expected to approach within a distance equal to half the wall height will be designed for a uniform surcharge equal to 2 feet of soil.

### 8.2.7 Combination of Loads

#### 8.2.7.1 General

Design will be performed for combinations of loads, along with appropriate load factors or allowable stresses, in accordance with the governing code. In the absence of specific direction by the code, the most severe distribution, concentration and combination of design loads and forces will be used. These combinations may be limited by practical considerations, such as the following:

- Combination of certain loads will not be considered when the probability of their simultaneous occurrence is negligible. Such loads include wind and seismic on superstructures; and seismic, live load surcharge, and flood on substructures.
- An increase in allowable stress of 33 percent, or a reduced load factor of 0.75, will be applied to the entire load combination where such is permitted for any of the loads considered in the combination.
- The effects of any load type (other than dead load) will not be used to reduce the effects of another load type. A maximum of 90 percent of the dead load will be used in any combination where it reduces the effects of another load type.

#### 8.2.7.2 Below Grade Structures including Box Culverts

Design will be performed for structures that contain liquids, extend below grade, or both, for the following load combinations.

- Liquid-containing compartments full, no backfill for liquid containing compartments. No reduction will be made for any counteracting soil pressure on the face remote from a contained liquid unless approved.
- Backfill and groundwater with liquid-containing compartments empty and full.
- Liquid containing compartments empty or full in any combination.

### 8.2.8 Serviceability

Additional requirements for serviceability will be considered as provided in subsequent sections and referenced standards for specific materials.

## 8.3 Foundation Design

### 8.3.1 Scope

Criteria will be established for the design of structure foundations in coordination with the geotechnical recommendations. Permanent structure foundation elements will be designed to distribute loads to the supporting soil, rock, or piling in accordance with their allowable loads, and to accommodate predicted deformations of the structure caused by settlement or movement of the supporting elements. Piling elements (piles and caissons) will be designed as structural elements to the accommodate stresses generated by the design loads. The design of the transmission of loads from the pilings to the supporting soil or rock will be performed by the geotechnical discipline. Structure foundation elements will be designed to resist effects of groundwater, including buoyancy.

### 8.3.2 Shallow Foundation Support

Design of shallow foundation elements (footings and mats), including excavation and backfill limits and details, will be performed in accordance with the recommendations of the geotechnical report.

To the extent possible, buried piping and ductbanks will be maintained outside the influence zone of the foundation elements. Limits of this zone will be established based on bearing materials' characteristics as documented in the geotechnical report. At a minimum, this zone will be defined by a line extended outward and downward from the bottom corners of a foundation element at a 1 vertical to 1 horizontal slope. A reinforced concrete encasement or other appropriate protection will be provided for any utilities extending into this zone.

### 8.3.3 Deep Foundation Support

Piling will be design in accordance with the recommendations of the final geotechnical report. Where a transition is required from pile supported to soil supported elements of a structure, design will be performed to accommodate the predicted deformation from such a transition.

Lateral loads to the structures will be resisted by the piling elements, the surrounding elements, or both. Where appropriate, the strain compatibility of the elements will be considered to determine the distribution of the lateral reactions.

### 8.3.4 Buoyancy

#### 8.3.4.1 General Criteria

Buoyancy is defined as the condition of instability resulting when uplift forces due to groundwater exceed resisting forces due to dead load and anchorage systems. Design will be performed in accordance with the following.

**Complete Structures**

For groundwater at the design level, structures will be designed to resist buoyancy considering only the structure dead load, soil directly above the structure and footing extensions. The effects of live loads, liquid contents, vertical soil friction and soil cohesion will be neglected. When anchorage systems are used, they will be designed to resist the net uplift force transmitted to the components of the anchorage.

**Partially Complete Structures**

Since the contractor will normally be required to maintain a dewatered excavation, it will be considered that groundwater will be maintained, at any given time, at or below the surface of the backfill currently in place. If the completed portion of the structure has insufficient resistance against pressures generated in this condition, the groundwater elevations at which the structure is stable will be provided in the contract documents.

**Anchorage Systems**

Where appropriate geotechnical conditions exist, rock anchors or tension piles may be used to resist buoyancy. Design these elements will be performed considering recommendations from the geotechnical engineer.

## L9 ELECTRICAL POWER SYSTEM DISTRIBUTION

### L9.1 Codes, Standards and References

The Blind River Fresh Water Diversion Project electrical design concept will comply with all federal, state and local laws or ordinances, as well as all applicable codes, standards, regulations and/or regulatory agency requirements including the partial listing below:

| | |
|---|---|
| ANSI | American National Standards Institute |
| IEEE | Institute of Electrical and Electronics Engineers |
| IES | Illuminating Engineering Society Lighting Handbook |
| IPCEA | Insulated Power Cable Engineers Association |
| NFPA | Life Safety Code |
| NFPA 70 | National Electrical Code |
| NFPA 70E | Electrical Safety in the Workplace |
| NEMA | National Electrical Manufacturers Association |
| UL | Underwriters Laboratories |

## L9.2 General Power Distribution

To the greatest extent possible and practicable, power for the new control structures, sluice gates, diversion structures and monitoring systems will be provided with permanent power utility service drops from Entergy. In remote locations where utility service drops are not available, standalone power will be needed.

Based on preliminary load information single phase power will be adequate for most control equipment locations for the project; however, preliminary load calculations indicate that power for the six control structures and Romeville diversion structure will require 3 phase electrical services. Typical three phase service voltages are rated 120/208 , 3 phase or 277/480 volt, 3 phase depending on the maximum power demand and control structure equipment motor sizes. For this project, new three phase power utility service drops will be provided at the control structures where reasonable access to permanent local power utility service can be obtained. At critical control structure and diversion structure locations where a normal utility service drop is provided, standby back-up emergency power will also be provided to provide power in the event there is loss of the normal utility feed.

For control structures located in remote locations where local power utility service is not available, standalone power systems will be provided. Depending on the location and requirements for power at the remote sites, stationary or portable generators will be utilized.

## L9.3 Utility Power Services

New electrical services from Entergy will be provided at Romeville diversion structure and control structures No. 3-2 and 1-7. Based on preliminary load information the Romeville Diversion structure will be served at 120/208 volts, 3 phase, 4 wire from new Entergy Louisiana transformers. Service at control structure No. 3-2 and 1-7 will be obtained at 277/480 volts, 3 phase from Entergy. The electrical power system will employ single ended simple radial type distribution at all location. Based on preliminary load information, a 100 amp main service circuit breaker and transfer switches will be provided. Switchboards utilizing group mounted bolt-on circuit breakers will distribute the incoming power to the diversion and control structure equipment including the sluice gates, sump pumps, control gates and control systems. The power distribution equipment will be located an equipment building adjacent to the structure.

A diesel engine driven generator will be provided for standby emergency back-up power. The generators will be sized for all diversion and control structure operation and life safety loads including, lighting, ventilation, controls and required safety equipment. Generator controls will be provided to allow automatic transfer to generator power when normal power is interrupted. Based on preliminary load data, the generator for the diversion structures will be sized at 25 kW. Back-up generators for the control structures will be sized at 60kW.

The generator will operate at 1800 rpm and provide back-up system power at 480 or 208 volts. Open transition will be utilized and the generator will not synchronize with the utility. A weather protected enclosure with sub-base mounted fuel tank will be provided.

## L9.4 Standalone Power Services

At control structure No. 1-6A, 1-6B, 1-8A and 1-8B, standalone diesel (or LP gas) engine driven generators are proposed for gate hydraulic motor power, lighting, security systems, monitoring instruments and controls. In addition, solar photo voltaic (PV) panels would be installed at these remote sites to provide power during times when control structure gates are not being operated and generator power is not needed. Power from the PV panels would be stored in battery racks to power security systems, monitoring instruments and SCADA system equipment at these locations when power from the generator is not available. Generator controls will be provided to allow manual or automatic starting of the generator when needed to control the position of the control structure gate system. For control structure 1-6A and 1-6B, a single generator and power distribution system sized to power both control structures will likely be used. The standalone power generator and control building housing for the power distribution equipment would be located on the north side of the canals to simplify cable routing and access to the control structure equipment. A similar concept would be employed for control structure 1-8A and 1-8B. The proposed location for the standalone power generator and control building at 1-8A and 1-8B is housing location for 1-building at Based on preliminary load data, the generator would be sized at 100kW.

Operation at remote locations would be as follows:

When called to run by the SCADA system or control structure PLC, the generator would start and operate at 1800 rpm and provide system power at 277/480 or 120/208 volts. Generator system power would be stepped down as needed to 120 volt levels for powering lights, instruments, battery chargers and other miscellaneous equipment at the control structure location. When operating, the generator would feed power through a transfer switch that would transfer power to the control structure electrical distribution system when the generator has reached proper operating speed. All power requirements at the control structure would be met with the use of the generator system including charging systems for the PV panels. During times of inclement weather or cloudy days when available solar power is not sufficient to power the SCADA and instrument monitoring systems, the generator can be operated as a back-up to charge batteries and power the system.

The generator would be located within a weather protected enclosure at the control structure location. A self contained base mounted diesel fuel tank (or separate LP gas tank) sized to run the generator continuously for 12 hours will be provided.

### L9.5 Exterior Lighting

Exterior lighting needs at the Blind River Diversion and Control Structures will be evaluated to limit light pollution falling on neighboring property. Fixtures with sharp cutoff type III lenses or back reflectors to control the direction of light will be provided. Individual fixture control will also be provided to switch off light if distribution control measures are not effective enough. At remote locations where normal utility power is not available, solar powered fixture will be considered.

### L9.6 Transient Voltage Surge Suppressors (TVSS)

Transient Voltage Surge Suppression will be provided for the electrical distribution system equipment to reduce the destructive effects of electrical transients and temporary excess voltage and/or current in the electrical circuits. The TVSS devices will be incorporated to limit short duration events, typically lasting from a few thousandths of a second (milliseconds) to billionths of a second (nanoseconds).

The electrical system equipment will be protected by transient voltage surge suppressors (TVSS) on the 480-volt line entering the Switchboards, lighting and power panelboards and control panels. UL1449 standard will be specified.

### L9.7 Wire and Conduit

All wire and cables will be run in galvanized rigid steel (GRS) conduit or PVC coated GRS conduit. Wires and cables shall be of annealed, 98 percent conductivity, soft drawn copper. All conductors shall be stranded, except that lighting and receptacle wiring may be solid. Except for control, signal and instrumentation circuits, wire smaller than No. 12 AWG shall not be used. Minimum conduit size will be ¾". Boxes for terminations etc. will be 316 stainless steel or FRP to resist the corrosive environment depending on availability. Conduit in hazardous locations will be galvanized rigid steel.

## L10 OPERATION, CONTROL, AND INSTRUMENTATION

This section presents the current planning on the operation, control, and instrumentation needs for the diversion project.

## L10.1 Diversion System Operation

The Blind River diversion project is a hydraulic system that will divert fresh water from the Mississippi River, transfer it to the fringes of the Maurepas Swamp, distribute it into the Swamp, and then drain the water into the Blind River. The principle hydraulic elements and functions of the overall system are:

- Mississippi River stage – the upstream boundary condition for the hydraulic grade line and the hydraulic driving force for the entire system.
- Diversion Structure – diverts the flow through/under the east levee of the Mississippi River via a culvert.
- Transmission Canal – transfers the flow from the diversion structure to the edge of the Swamp.
- Distribution System – distributes the flow into the Swamp.
- The existing drainage channels in the Swamp will serve as the distribution system.
- Control structures will be installed in the existing channels to control the water surface elevations (WSELs) and the hydraulic grade line to force the water into the Swamp.
- The proposed berm gaps will open up the existing spoil banks, allowing the higher water surface to force flow from the existing drainage channels through the gaps into the Swamp.
- Overland Flow and Drainage System – after flowing through the proposed berm gaps and other routes, the diverted water will flow through the Swamp via overland flow, and then to drain from the Swamp and discharge into the Blind River.
- The proposed cross culverts and interconnecting channels at Highway 61 will improve overland flow and circulation within the Swamp
- Blind River stage – the downstream boundary condition for the hydraulic grade line. The stage is influenced by runoff from rainfall in the watershed and back-flow from Lake Maurepas due to:
- Tidal effects
- Tropical storm surges
- High water levels due to flood events in other drainage systems discharging into Lake Maurepas

The diversion system will typically operate long-term at steady-state flow conditions. The primary flow control at the diversion structure will likely be manually set to a given flow rate, then the sluice gate openings will automatically vary based on feedback from the flow meters, to maintain the set flow rate. The flow rate may be changed manually from time to time. The Mississippi River stage varies widely, from Elev. 0.5 to Elev. 29, although the day-to-day variation is very low. As the Mississippi River stage varies, it is anticipated that the automatic flow rate feedback will result in gradual automatic adjustments to the sluice gate positions to maintain the set flow rate.

During diversion periods, regional hydrologic events may dictate changes in the flow rate, or a decision to completely stop the diversion flow. These include:

- More intense local rainfall events which may cause local flooding and require a decision to discontinue the diversion to avoid increasing the flood levels.
- Tropical storm surges through Lake Maurepas and up the Blind River may cause flooding. A decision may be required to discontinue the diversion flow to avoid increasing the flood levels.

These events should be monitored by the operating staff, which would then make the decisions to manually shut down the system. A backup/fail-safe system needs to be in place to reduce flood risk in case the staff fails to manually shut down the system. The Blind River water levels will be monitored at Highway 61 and at IH-10 to indicate rising water and probable back-flow from Lake Maurepas. Flow and velocity meters could also be included to better define that back-flow is occurring. The WSEL's at the control structures should also be monitored for high water conditions. At certain high WSEL's, the diversion system would then be automatically shut down.

## L10.2 Diversion System Control

The diversion system operation and control is centered on hydraulic elements. Other parameters will be monitored. Categories of operation/control/monitoring data include:

- Primary hydraulic system control – control the diversion flow rate and key WSEL's in the Swamp
- Secondary hydraulic system monitoring and controls – WSEL and limited flow
- Others – security, motors, mechanical, electrical systems, etc.

Other data will be collected and monitored, but will not be used directly to control diversion flow rate or water levels. The additional data can be used to refine control parameters and settings, monitoring system performance, and provide a basis to for adaptive management changes. These items are not yet defined at the conceptual stage. These include the following:

- Hydrologic data collection – additional WSEL data can be collected inside the Swamp. At this point in the design, these data sets are not expected to be used for real-time monitoring and control of WSEL's and diversion flow rates.
- Environmental data – water quality, etc.

### L10.2.1 Primary Hydraulic System Control

The three main hydraulic controls on the overall system are:

- Control of the diversion flow rate at the diversion culvert – The diversion flow rate will be controlled by a sluice gate on the inlet to each box culvert cell (barrel). The sluice gate will have a motor operator and the flow rate will be controlled by varying the sluice gate position. Flow meters in each culvert cell will provide real-time feedback to reposition the sluice gates, thereby adjusting the flow rate.
- Control of water surface elevations at each control structure – In the Swamp, the control structures with the control gates will control the WSEL in the existing drainage channels on the upstream side of the control gate to force the diverted water through the proposed berm gaps into the Swamp. The WSEL will be measured on the upstream side of the gate, and the gate height will be adjusted to maintain a set WSEL.
- Control of the flow rate at each control structure – Part of the diverted water will be required to pass over the control gate to continue downstream in the existing drainage channel. At each control structure, the same WSEL measurement on the upstream side of the gate will be used to set the gate height to serve as a weir, allowing a set flow rate to pass over the control gate.

### L10.2.2 Secondary Controls and Monitoring

A secondary set of instrumentation will monitor hydraulic parameters at multiple locations for the purpose of protecting the system, safety, avoiding overflows, and avoiding flood conditions due to too high of water surface elevations. This monitoring effort will mainly consist of measuring the WSEL at various locations throughout the system, and collecting limited flow rate data at several locations. These monitoring points will trigger alarms and some will trigger system shut-down. Some of the monitoring involves comparative WSEL across structures to indicate blockages, or other problems.

**Diversion Culvert**

The WSEL will be monitored at three locations at the diversion culvert structure:

- Inlet canal WSEL – this indicates the WSEL in the Mississippi River and provides the upstream WSEL for the system
- Sluice Gates WSEL – the WSEL on the upstream side of each sluice gate

- Culvert Outlet WSEL – the WSEL at the diversion culvert outlet, in the upstream end of the transmission canal

The data will be used as follows:

- The inlet WSEL will be used to monitor the Mississippi River WSEL. At low WSEL's in the river, the diversion may have to be discontinued to avoid conflicts with other water users.
- WSEL differential at the trash racks – a high differential in WSEL from the inlet canal to the sluice gates indicates clogging by debris. This will trigger an alarm. The individual sluice gate will be shut down at an extreme differential to avoid damage to the trash rack.
- The WSEL differential between the sluice gate and culvert outlet could be used as an input to verify flow meter calibration.

**Transmission canal**
The WSEL in the transmission canal will be monitored to avoid the canal berm being overtopped, resulting in damage to the canal and potential local flooding. The design freeboard is 2 feet. The system will provide an alarm when the freeboard is reduced to 1 foot due to a rising water surface. Above that WSEL, the feedback to the sluice gates could gradually close the gates if the WSEL continues to rise, and shut down the diversion flow at approximately 0.5 feet of freeboard. The WSEL monitoring will be at:

- CN RR – upstream side of the culverts at the railroad
- LA 3125 – upstream side of the culverts at the road

**Control Structures**
The WSEL will be monitored on the upstream and downstream sides of the control gates. Alarms will be tripped by excessive WSELs. Extreme WSELs would indicate flooding conditions, and should initial system shut-down.

**Blind River**
Monitor the Blind River stage at IH-10 and Highway 61. High stages will trip alarms, and then shut down of the diversion system. Monitor the velocity and direction of flow, to indicate back-flow into the project area.

**Berm Gaps**
There will be no flow control at the individual berm gaps. All control will be by the control gates in the control structures.

**Cross Culverts at Highway 61**
There will be no level or flow control capabilities at the Highway 61 crossings.

### L10.2.3 Hydrologic Data Collection

Hydrologic – rainfall and additional WSELs in downstream system

### L10.2.4 Environmental

Environmental and water quality monitoring – not yet defined, and not addressed any further in this section.

### L10.2.5 Others - Security, Motors, Mechanical, Electrical Systems, etc.

## L10.3 Instrumentation and Control

This section provides the conceptual instrumentation and control design for the project. Each area to be controlled will be equipped with an independent Programmable Logic Controller (PLC) that will monitor process values and statuses and perform control of the local equipment. These PLCs will be linked by a satellite communication system that will allow data to be shared from site to site in order for the system to operate as a whole. The system will be able to be monitored from a single location by an operator. See **Annex L-5** for instrumentation drawings and schematics.

The current design layout provides a control building on the site of the diversion culvert. Provisions will be made to monitor the system from other locations via the internet or other means. All required real-time control and monitoring data will be supplied to the operator to be used for operational decision making. All sites will be monitored by video security with local digital video recorders (DVR) to record video. The sites will also be equipped with intrusion switches on the gates and doors to monitor entry to the sites and notify the on-call operator.

## L10.4 Instrumentation and Control Requirements for Power Supply Systems

For this system, the primary source of power for the diversion facility and the transmission system will be grid power. The grid power will be backed up with a standby generator to keep the system operational during a grid power outage. The control system will monitor the availability of grid power, the generator status, and whether the systems are operating on grid power or generator power.

The control structures are located at remote sites and will have one generator at each site as the primary power supply to operate the control gates. Solar power and a battery pack will be used as back-up power for all needs except control gate operations. The control system will monitor the generator status.

## L10.5 Instrumentation and Control Design for Romeville Diversion Facility

The proposed Instrumentation and Control design for the Romeville diversion facility is shown on the engineering panels in **Annex L-5**. The levels in each of the primary areas will be monitored and logged for trending purposes. The three diversion culvert gates will be controlled by the local PLC and any problems will be alarmed at the central control site. The flows in the three culverts will be monitored and logged for trending and reporting purposes.

## L10.6 Instrumentation and Control Design for Romeville Transmission System

The water level will be monitored at the upstream side of the CN RR culvert and the upstream side of the LA 3125 culvert. High water levels within 1 foot of the top of berm will trigger an alarm and higher levels will initiate sluice gate closures at the diversion culvert.

## L10.7 Instrumentation and Control Design for Swamp Control Structures

The proposed Instrumentation and Control design for the control structures in the Swamp is shown on the engineering panels in **Annex L-5**. At each structure, the water level on each side of the gate will be monitored and recorded. The gate will be controlled by the local PLC and any problems will be alarmed at the central control site. The number of times that this information is passed each day will be reviewed in order to keep the total amount of data passed to a minimum while ensuring that sufficient data is communicated for monitoring.

## L11 CONSTRUCTION PROCEDURES

This section describes construction procedures, requirements, and special considerations necessary to construct the proposed facilities. The discussions include existing site conditions, access, construction techniques, temporary construction facilities, detours for transportation facilities, construction sequences, dewatering, and special items. The permanent project components are described in detail in Section L7.

The discussions in this section describe conditions impacting the construction effort, as these influence design, opinion of estimated costs, scheduling, permitting, and other project factors. The construction contractor will be fully responsible for selecting and implementing the construction means and methods.

### Material Availability

The project is near industrial facilities along the Mississippi River. Construction materials, such as steel sheet piling, ready-mix concrete, stone and gravel, drainage pipe, and road materials should be readily available. Highway, rail, and river transportation facilities are in the immediate area, for delivery of items that need to be shipped to the area. Imported fill material may be in short supply, or expensive, due to high demands on reconstruction projects after Hurricane Katrina.

### Upland Construction Sites

The diversion culvert facility and the transmission canal will be constructed outside of the Swamp in upland areas. Conventional land-based heavy civil construction equipment and procedures will typically be used for these two components. Probable equipment to be used include bulldozers, track-hoes, earthmovers, cranes, pile drivers, concrete pumps, dewatering equipment (well points), and other items.

### Swamp Construction Sites

The control structures, berm gaps, cross culverts, and radio towers will be constructed in the Maurepas Swamp, in potential conditions of saturated soils, weak/soft soils, standing water, and flooded sites. Special equipment and procedures will be required for these heavy civil construction project components. Equipment will likely require wider tracks for work on soft ground. Specialized equipment for soft/marshy/swampy areas, such as Rolligons, might also be used in the Swamp.

Natural ground in the Swamp is typically near Elev. 1.5 to 2.0, the static water surface between storm events is approximately Elev. 1.5 to 2.0, and flood levels can approach Elev. 5. The ground conditions in the Swamp are typically described as saturated and soft, consisting of organic material, silts, and clays. The water table will vary from slightly below ground surface to standing water on the sites, to flooded conditions. The sites typically have heavy brush and trees, especially at the fringes of the Swamp along the existing drainage channels.

There are no existing roads into the Swamp to the project component sites. No access roads are included in the current plans, and temporary access roads will likely not be allowed. The conceptual designs and costs estimates consider that access is to be via barges and crew boats. These will be used to transport all construction equipment, materials, and crews to the control structure sites. An existing access point with a boat ramp is north of Grand Point. Construction of the transmission canal to the existing Parish drainage channel will provide an additional access point.

The control structures use control gates that rotate down to the channel bottom in the open position, thereby allowing boat and barge access past the control structure. Construction of a control structure, with the control gate in place, will not hinder construction access to the other construction sites.

**SWPPP**
Storm water pollution prevention plan (SWPPP) will be prepared for each construction site to meet the requirements of the EPA and State NPDES. The SWPPP will include provisions to control erosion and minimize sedimentation of streams during construction, using such measures as filter fabric fences, rock filter dams, stabilized construction exits, inspection and good housekeeping practices, and vegetation. The SWPPP is not included in the current preliminary plans, but will be developed during final design. (Erosion protection for the permanent facilities is discussed in Section L7, civil design section).

## L11.1 Diversion Culvert Facility

Construction of the diversion culvert will require specific construction techniques, sequencing, and temporary facilities for the following three major portions of the facility:

- Connection of the inlet canal to the Mississippi River
- Construction of the culverts through the Mississippi River East Levee
- Construction of the culverts under LA 44

**Existing Site Conditions**
The diversion facility site is located adjacent to the Mississippi River and includes the batture inside the levee (river side of levee), the levee, and the area outside of the levee (land side of levee) to 100 feet east of LA 44. The site has no existing development, such as residential, commercial, or industrial facilities. The batture appears to be in original natural conditions, and is covered with trees and brush, and is considered to be poorly drained. The area adjacent to LA 44 is clear, being agriculture land, or fallow. LA 44 and the fields in the area have existing drainage infrastructure. The LA 44 right-of-way has utility lines, including overhead electric distribution, and underground communication lines, which will have to be adjusted for construction of the diversion culvert. There are no known water or sanitary sewer lines in the road right-of-way.

**Access**
Access to the diversion facility will be from the local paved road, LA 44 (River Road). Access is also available from the Mississippi River for delivery of large items or large volume items to the site via barges, such as the sluice gates, steel sheet piling, and riprap.

**Construction Techniques**
Installation of the diversion facility involves heavy civil construction and anticipated equipment includes large track-hoes, earthmovers, cranes, pile drivers, concrete pumps, and bulldozers. The site is outside of the Swamp, and normal land-based construction techniques apply. The diversion culvert is a large hydraulic structure, and will have to be constructed by open-cut techniques across the batture, through the Mississippi River levee, and under LA 44. The culvert is designed as a cast-in-place reinforced concrete structure and will likely be on a prestressed concrete piling foundation.

**Temporary Construction Facilities**
The current level of flood protection provided by the East Levee of the Mississippi River must be maintained at all times during the construction phase. A temporary earthen levee will be placed on the batture, inside of the existing levee, to allow open-cut construction of the culvert through the existing levee. The temporary levee will be designed and constructed to standards matching the permanent levee. The inside face of the levee will be lined with riprap for erosion protection, as the temporary levee could be exposed to flowing water during flood stages. During final design, an alternate steel sheet pile cofferdam will be considered at the levee to avoid a need for the temporary levee.

A steel sheet pile cofferdam will be installed on the east bank of the Mississippi River to allow excavation of the inlet canal and placement of the riprap in dry conditions. The work should be done during the summer months, when the Mississippi River normally has the lowest stages. The cofferdam protects only the inlet canal earthwork, and is not part of the overall levee flood protection system, and as such, the enclosure could be considered a non-critical enclosure. Therefore, the top of the cofferdam was initially designed to Elev. 24, the approximate elevation of the batture. It is considered that the cofferdam on the Mississippi River bank will be installed and removed from a barge in the River.

**Detours**
The diversion culvert will be constructed across LA 44 in an open-cut excavation, requiring the existing LA 44 pavement to be removed for construction. Re-routing traffic from LA 44 to LA 3125 will result in an unacceptably long detour route. Therefore, local traffic will be maintained through the construction area at all times by providing a temporary detour road. The detour road will be an asphalt road approximately 1,000 feet long with 2 – 12' lanes (one each direction).

**Construction Sequences**

The construction will be phased and sequenced at the levee to ensure that the east levee flood protection integrity is maintained at all times, in the following sequence:

- Construct temporary levee – the temporary levee must be in place on the batture before any excavation or disturbances to the existing levee. The temporary levee is to remain in place until the diversion culvert is constructed through the levee and the sluice gates are installed and operable, ensuring full blockage of flow through the culverts.
- Remove the existing concrete liner on the inside face of the levee
- Excavate the existing levee
- Install the cut-off wall in the levee
- Construct the box culverts through/under the levee, including the trash racks, gate tower, and installation of the sluice gates
- Reconstruct the main levee
- Reconstruct the concrete liner on the inside levee face
- Remove the temporary levee

The inlet canal should be constructed after the removal of the temporary levee, as part of the inlet canal is under the footprint of the temporary levee, using the following sequence:

- Install the cofferdam on the east bank of the Mississippi River
- Excavate the inlet canal and place the riprap
- Remove the cofferdam

The construction sequence for the culvert east of the levee under LA 44 is noncritical for levee integrity, and can be done at any time, either before, during, or after construction through the levee. The following sequence is required to maintain local mobility and traffic:

- Construct the LA 44 temporary detour
- Remove the existing pavement at the culvert crossing
- Construct the culvert
- Reconstruct LA 44 in the existing location
- Remove the temporary detour

The culvert under LA 44 should be completed before the downstream segment of the transmission canal is completely excavated and connected to the existing drainage channel in the Swamp to avoid flooding the culvert construction site. If this isn't possible, an earthen plug should be left in the transmission canal to protect the culvert work site from flooding by backflow from the Swamp. The existing water surface at in the parish perimeter drainage channels is Elev. 1.5 and the proposed culvert flow line is Elev. -3.0 at LA 44.

**Dewatering and Surface Water Management**

Ground water conditions are currently unknown; however, it is considered that water tables are high and that dewatering, such as well points, will be required. Outside of the levee, surface water can be directed to the local drainage infrastructure. The batture may be poorly drained, but drainage could be redirected away from work sites to the Mississippi River.

**Net Earthwork Quantities**

No attempts were made to balance cut and fill quantities on the diversion culvert site. It is anticipated that excess spoil from the transmission canal excavation will be used for the temporary levee. The material from the removal of the existing levee will be stockpiled and reused for the reconstructed levee. Excess material from removal of the temporary levee, excess material from the original levee, material from the box culvert excavation, and material from the inlet canal will be removed from the site.

**Example Diversion Culvert**

The construction requirements for the Romeville diversion culvert are similar to those used on the Davis Pond diversion culvert. See the following photograph of Davis Pond (**Figure L11.1-1**), in which the following construction items are visible and applicable to the Romeville diversion location:

- Temporary earthen levee on the batture
- Riprap on temporary levee
- Open-cut excavation through the existing levee
- Pipes for a dewatering system in the excavation
- Concrete piling foundation
- Barge access from the Mississippi River
- Detour for local road
- Shoofly to temporarily relocate the railroad (see railroad discussions under the transmission canal paragraph)

**Figure L11.1-1 Davis Pond Diversion Photo**

## L11.2 Transmission Canal

The proposed transmission canal is primarily an earthwork project with culvert crossings at LA 3125 and the CN RR.

**Existing Site Conditions**

Most of the transmission canal alignment is located on agricultural or fallow land at elevations above the Maurepas Swamp. Routes considered are typically clear of residential, commercial, and industrial development. The area has existing drainage infrastructure, and surface water from rainfall is expected to drain off of the site in reasonable times. The downstream (east) end of the transmission canal is in the Swamp and likely has nearly permanent standing water and saturated conditions.

**Access**

The entire transmission canal alignment is readily accessible from the two roads it crosses – LA 44 and LA 3125. During construction, the right-of-way can be graded, and the berms can be constructed, providing good construction access along the entire alignment to each work site.

**Construction Techniques**

Installation of the transmission canal involves heavy civil construction. The majority of the transmission canal is in the upland areas and can be constructed using conventional equipment, such as track-hoes, bulldozers, and earth movers.

At the downstream end of the canal, the earthwork will be in wet conditions at the fringes of the Swamp. The berms will be extended along both sides of the canal to the existing drainage channel, providing access to the area. A temporary berm could be constructed across the work site at the downstream end, fully isolating the excavation area from the Swamp. This would allow conventional earthwork equipment to be used. Special construction approaches will be required to remove the final plug of earth to daylight the canal into the existing drainage channel. The soil could be excavated in the wet, a practice that results in sedimentation in the downstream channel. The tie-in area should be isolated with a cofferdam, allowing excavation and placement of the riprap in protected and dry conditions.

The culverts at the road and railroad are large hydraulic structures, and will have to be constructed by open-cut techniques. The reinforced concrete box culverts can be either pre-cast or cast-in-place, if allowed by permits from the owners. It is anticipated that cranes and concrete pumps will be required, in addition to the equipment listed above. The conceptual estimate is based on cast-in-place.

During final design, alternate construction techniques should be investigated for the railroad crossing, to determine if the shoofly can be avoided.

**Temporary Construction Facilities**

The only major temporary facility required will be a steel sheet pile cofferdam at the downstream end of the transmission canal to allow excavation of most of the east end in dry conditions.

**Detours**

The CN RR is an active rail line serving the mid-west and the south Louisiana port area. Based on informal discussions with railroad representatives, rail service must be maintained at all times, and speed reductions will likely not be allowed. The design assumes a 2,000-foot long shoofly (temporary track) to allow open-cut construction of the box culverts across the existing CN RR tracks. The shoofly will include turnouts at each end to expedite final switch-over to the shoofly. The railroad will likely discourage a temporary rail crossing at the work site. Such a crossing is not necessary, as access is available to both sides of the railroad from the existing public roads.

The proposed construction sequence at the CN RR is as follows:

- Construct the shoofly and re-route rail traffic
- Remove the existing tracks at the culvert crossing

- Construct the culvert
- Reconstruct the railroad tracks in the existing location and place rail traffic back on the original alignment
- Remove the shoofly track

A nearby, temporary detour will be required for construction of the LA 3125 crossing, due to the unacceptable length of potential detour routes if the road were closed. The detour was not designed in the conceptual phase, but was considered to be 1,000 feet long, consisting of a 2-lane asphalt road (1 – 12' lane each direction) with shoulders. LA 3125 has a 300-foot wide right-of-way, and the work should be done in one detour phase. The following sequence will be required:

- Construct the LA 3125 temporary detour road
- Remove the existing pavement at the culvert crossing
- Construct the culvert
- Reconstruct LA 3125 in the existing location
- Remove the temporary detour road

LA 3125 and the CN RR are to remain open to traffic at all times, except for brief periods to switch over traffic, as allowed by the permits.

**Construction Sequences**

Construction of the transmission canal does not impact the integrity of major flood protection facilities. However, specific sequences are necessary to avoid negative impacts to local drainage and transportation facilities. Following is the recommended construction sequence:

- Local drainage – the project will disrupt the local drainage patterns and revised drainage facilities are included in the design. These new drainage facilities need to be constructed first.
- Pipeline and utility adjustments need to be completed prior to the canal excavation.
- The road detour and railroad shoofly are to be constructed prior the canal excavation in the area of these crossings
- The isolation plug and temporary berm at the downstream end of the transmission canal are to be the last construction items

After the above items are done, sequencing of the overall transmission canal is non-critical.

**Dewatering and Surface Water Management**

Ground water conditions are currently unknown, although it is considered the groundwater levels are high. Dewatering, such as well points, is considered to be required for the entire transmission canal length. The groundwater is likely at or near ground surface downstream (east) of LA 3125 and additional dewatering capacity may be required.

The design includes drainage ditches along both right-of-way lines to direct surface drainage to the local drainage infrastructure. These ditches should be excavated early in the construction process to direct surface drainage away from the work areas.

The segment of the transmission canal in the Swamp will require additional water management tasks. It is considered that the work area can be isolated from the Swamp surface with earthen berms, and that more extensive dewatering will be required.

**Net Earthwork Quantities**

The initial design for the transmission canal does not have the cut and fill quantities balanced. It is anticipated that there is a market for the excess spoil, and that the contractor will be able to readily sell or dispose of the spoils. If the material is to be disposed on site, the berms along the canal could be widened and raised to absorb the excess earthwork volumes. Or adjacent lands could be acquired as part of the project, and the excess spoil could be disposed on them.

The current design assumes that the excavated soil is suitable for the embankments creating the berms.

Based on the current ground profile and transmission canal design, there will be no need for imported fill.

## L11.3 Control Structures

The control structures are large concrete structures with control gates to control the water surface elevations, as described in Section L7.

**Existing Site Conditions**

The control structures will be located in existing drainage channels with permanent standing water within the Swamp. Most of the control structure sites have an existing spoil bank on at least one side of the channel. Existing conditions in the Swamp are described in the introduction to this section.

**Access**

Access to the sites will be via barges, as described in the introduction to this section.

**Construction Techniques**

Installation of the control structures involves heavy civil construction and at a minimum, cranes, track-hoes, pile driving equipment, and bulldozers will be required at the sites. The control structures will require structural excavation in the existing drainage channels, fill behind the retaining walls, and re-grading the berms on the site to control the flow of the diverted water, requiring land-based construction operations. Some construction operations can be performed from barges. However, some construction contractors may prefer to have a land-based operation. Work areas can be built up above the standing water levels and stabilized to allow the soil to support the construction equipment. This could be accomplished by using a soil reinforcement such as a geotextile and placing fill to the desired elevation. Excavated material from structural excavation could be used for the fill.

Concrete will have to be delivered to the site by barge.

**Major Temporary Construction Facilities**
The control gate structure will be constructed in the existing drainage channels, which have permanent standing water. Temporary steel sheet pile cofferdams will be installed in the channels to isolate the work area and allow them to be dewatered. The control structures will have to be phased and sequenced to maintain the drainage and flood control capacity of the existing drainage channels. For the preliminary design, it was considered that half of the channel can be blocked at each location. The final design will address specific allowable extents of channel blockage.

As an alternate, a by-pass channel could be excavated around the control structure site, allowing the entire control structure to be installed in one phase. This approach will be investigated during final design.

**Construction Sequences**
The work sites in the Swamp for the berm gaps and the control structures are independent and can be constructed in any sequence. The approximate construction sequence far an individual control structure will be as follows:

- Clear the site
- Establish work pads (build up the ground elevation) for land-based construction operations
- Phase 1
- Install a cofferdam on half of the channel
- Excavate half of the channel
- Construct half of the concrete structure
- Install half of the channel transition sections and erosion protection
- Backfill at the walls on one side

- Remove the end sections of the cofferdam and flood Phase 1, opening it to flow in the channel
- Phase 2
- Install a cofferdam on the second half of the channel
- Excavate half of the channel
- Construct half of the concrete structure
- Install half of the channel transition sections and erosion protection
- Backfill at the walls on one side
- Isolate the entire gate area
- Install the control gate
- Complete area grading
- Remove the cofferdam
- Install the control building and all remaining appurtenances

**Dewatering and Surface Water Management**

The site is in the Swamp, and will have a high ground water table, possible standing water, and potentially flooded conditions. The construction contractor will have to use construction methods to deal with these conditions. Deep sheet piling may be required to cut off deeper sand layers to control water from flowing into the cofferdam enclosures. Well points and deep wells may also be required. Surface water control can include building earthen berms around the work site and building up earthen work pads.

**Net Earthwork Quantities**

Earthwork quantities will be balanced on each site. Due to the isolated locations, no soil will be imported into, or exported from the work sites. Excess spoil will be disposed near the existing berms, as discussed under the Berm Gaps. If additional soil is required, the existing channels will be widened.

## L11.4 Berm Gaps

Large gaps will be excavated in the existing spoil banks along the drainage channels in the Swamp. These new gaps will provide open and free flow between the existing perimeter drainage channels and the Swamp.

**Existing Site Conditions**

See the introduction to this section for a description of existing conditions in the Swamp.

The spoil banks were created or added to in the 1970's during the last known construction effort to deepen and widen the existing drainage channels. The size of the spoil banks vary widely, with the top ranging from Elev. 4 to Elev. 12. The spoil banks are covered by moderate to dense brush and trees approximately 30 years old. The general area in the Swamp also has moderate to dense trees and brush.

**Access**
See the introduction to this section for comments on access to the work sites.

**Construction Techniques**
Track-hoes, bulldozers and earth moving equipment will be moved to the site via barge. The existing spoil banks are considered to be stable and capable of supporting construction equipment.

The spoil will be disposed on site. The soil will be moved from the proposed gaps to areas behind the existing spoil banks that are to be left in place. Disposal areas could be built up above the 100-year flood level to provide additional wildlife refuge areas during floods. Alternatively, the spoil could be spread out in a 1 to 2 foot layer, to allow Bald Cypress seedlings a better chance of survival.

The construction contractor will have to use equipment and techniques adapted for swamp/marsh/soft ground conditions. These can include Rolligons and wide tracks on the track-hoes and bulldozers.

**Major Temporary Construction Facilities**
None.

**Construction Sequences**
None required.

**Dewatering and Surface water Control**
The contractor will need to have techniques to manage the high water levels in the swamp. These could include constructing low berms (1' – 2' high) around work sites, and pumping out the interior.

**Net Earthwork Quantities**
All excavated soil will be disposed on site. There will be no imported fill or exported spoil.

## L11.5 Cross Culverts

Cross culverts will be constructed under Hwy 61 at four locations to improve water circulation. The cross drainage capacity at the Kansas City Southern railroad appears to be adequate, however, additional analysis to verify the quantities of flow will be completed during the PER phase of the project.

**Existing Site Conditions**
The proposed culvert locations are within the Maurepas Swamp where Highway 61 crosses the Swamp. Existing conditions in the Swamp are described previously.

The Hwy 61 crossing has two lanes each direction on an embankment at Elev. 6. The road has a large barrow ditch parallel on the east side, with standing water. The road and KCS railroad are approximately 500 feet apart. Pipelines and overhead power transmission lines are in the corridor between the road and railroad and the area appears to be highly disturbed by construction and maintenance activities for these facilities.

**Access**
The four culvert crossings are readily accessible from Hwy 61, including the work sites on both sides of Hwy 61, the area between the road and railroad, and the east side of the railroad. There are no known or apparent access points to the west side of the railroad. The corridor is largely disturbed by previous construction and maintenance activities. Access roads, either temporary, but preferably permanent, along the cross connect routes should be acceptable.

**Construction Techniques**
The work sites are within the Swamp, and specialized construction equipment will likely be required. If the access roads are allowed, stabilized work areas can be constructed consisting of geotextile and fill. This will allow use of more conventional equipment and techniques.

The cross culverts at the road are large hydraulic structures, based on the current designs, and will be constructed by open-cut techniques. The box culverts are reinforced concrete structures, and can be either cast-in-place or precast units, if allowed by the permits. As access is readily available, the ready-mix concrete, or pre-cast units, can be delivered to the site. It is anticipated that cranes and concrete pumps will be required, in addition to the equipment listed above.

If smaller culverts are used, the culverts could be installed by micro-tunneling or augering and jacking under the road and railroad.

**Major Temporary Construction Facilities**
Small cofferdams may be required to control local drainage ditches and surface runoff, especially in the barrow ditch on the east side of Hwy 61.

**Detours**
A detour will be required for construction of the Hwy 61 crossing, due to the unacceptable length of potential detours if the road were closed. Also, Highway 61 is an evacuation route, and full traffic capacity must be maintained at all times. The detour was not designed in the conceptual phase, but was considered to be 1,000 feet long, consisting of a 4-lane asphalt road (2 – 12’ lanes each direction)

with shoulders. The following sequence will be required:

- Phase 1
- Construct the Hwy 61 temporary detour road in the east side of the right-of-way
- Remove the existing pavement at the culvert crossing
- Construct half of the culvert
- Reconstruct Hwy 61 in the existing location
- Remove the temporary detour road
- Phase 2
- Construct the Hwy 61 temporary detour road in the west side of the right-of-way
- Remove the existing pavement at the culvert crossing
- Construct the second  half of the culvert
- Reconstruct Hwy 61 in the existing location
- Remove the temporary detour road

Hwy 61 is to remain open to traffic at all times, except for brief periods to switch over traffic, as allowed by the permits.

**Construction Sequences**
Specific sequences are necessary to avoid negative impacts to local drainage and transportation facilities. Following is the recommended construction sequence:

- Local drainage – the project will disrupt the local drainage patterns and revised drainage facilities are included in the design. These facilities need to be constructed first.

Construction sequences are not critical for the other elements of the cross culverts.

**Dewatering and Surface Water Control**
The sites are in the Swamp, and will have a high ground water table, possible standing water, and potentially flooded conditions. The construction contractor will have to use construction methods to deal with these conditions. Deep sheet piling may be required to cut off deeper sand layers to control water from flowing into the cofferdam enclosures. Well points and deep wells may also be required. Surface water control can include building earthen berms around the work site and building up earthen work pads.

**Net Earthwork Quantities**
The cut and fill will be balanced at each site, and no soil will be imported into, or

exported out of the individual sites. If there is excess spoil, it will be disposed in the corridor between the road and railroad.

## L11.6 Instrumentation and Communication

Local instrumentation, controls, and control panels will be included in the designs of each project component. This paragraph addresses communication facilities, which tentatively consisted of radio towers for the conceptual design and construction cost estimating. The towers will be 150 to 200 feet high to be above mature Bald Cypress trees, and to provide clear line-of-sights for radio signal transmission. Radio towers will be located at the diversion culvert facility, at each control structure, and in the Hwy 61 corridor.

See the above discussions of project components for existing site conditions, access, major temporary construction facilities, and dewatering and surface water control.

**Construction Techniques**

The radio tower subcontractor will determine the appropriate construction method.

**Construction Sequences**

Construction sequences are not critical for the radio towers, in relation to other facilities.

# L12 OPERATION AND MAINTENANCE

## L12.1 Maintenance Dredging

The following text describes operation and maintenance activities, including maintenance dredging, associated with operation of the 3000 cfs Small Diversion at Convent/Blind River. The selected alternative is located at Romeville.

Operation and maintenance activities will also include (but are not limited to) starting and stopping the diversion(s), routine equipment and instrument maintenance, corrective equipment and instrument maintenance, and berm gap and culvert cleaning.

### L12.1.1 Regular Maintenance Dredging

Maintenance dredging or de-silting is anticipated to remove sediments deposited in the Transmission Canal and drainage channels during operation of the diversion system. The Mississippi River carries a significant suspended solids load and it is expected that the flow diverted into the diversion operation will have the same characteristics. The velocities in the Transmission Canal and the drainage channels are relatively low, and sediment accumulation is anticipated in the system. The opinion of estimated cost assumes that this accumulated sediment volume is removed annually.

The Mississippi River sediment data is based on the two following sources, both of which provide similar coarse suspended solids characteristics:

- **Mississippi River Sediment, Nutrients, and Freshwater Redistribution Study, dated July 2000 (2000 MRSNFRS)** – The suspended solids load in the spring (high flow periods) is approximately 300 PPM and the fall (low flow periods) load is 200 PPM. The sand portion varies from 20 to 30% of the total suspended solids load. This results in loads of 60 to 90 PPM in the spring.
- **Sediment Flux and Fate in the Mississippi River Diversion at West Bay: Observation Study (a thesis at LSU)** – The coarse suspended sediment (0.0625 mm diameter and greater) has a typical value of 70 PPM in the spring and 20 PPM in the fall.

### L12.1.2 Costs for Regular Maintenance Dredging

To be conservative, we made the following assumptions regarding maintenance dredging to establish sediment volumes and cost for sediment removal:

- The coarse suspended solids load will average 75 PPM during the entire diversion season
- 100% of the coarse sediments will be deposited in the transmission canal and the existing drainage channels
- The deposits will consolidate to 80 PCF
- The annual diversion period is eight to nine months

With these assumptions, 150,000 CY of settled sediment is expected to accumulate in the system annually.

The transmission canal and the drainage channels are wide channels and have permanent standing water; therefore, the estimate assumes hydraulic dredging. The spoil will be reused beneficially and discharged into the Swamp in a controlled manner that will supplement the land-building processes. At 2004 hydraulic dredging costs, which include reuse of the sediment at $12.00 per CY, we estimate that a 3,000 cfs diversion operation producing 150,000 CY will cost $1,800,000 in 2004 dollars, and with 20 percent added for contingencies will cost approximately $2,200,000.

### L12.1.3 Single 3000 cfs Culvert Diversion O&M at Romeville or South Bridge

During operation of the single 3000 cfs culvert diversion, the following O&M activities (costs are provided later in this subsection) can be expected during diversion operation:

- Startup and shutdown
- Pre-startup, routine, and post shutdown O&M
- Preventive maintenance in accordance with manufacturer's recommendations
- Instrument maintenance and preventive maintenance

- Alarm response
- Vehicle operation and maintenance
- Electrical costs
- Mowing
- Gap clearing
- Materials and supplies
- Spare parts
- Miscellaneous activities (contingencies)

## L12.2 Facilities Operations Plan

### L12.2.1 Operating Assumptions

Once the diversion system(s) is operating, operating staff will enact regular operational changes in response to the hydrologic situation in the target area(s). The following text summarizes expected operation for the diversion.

We can expect to follow the following basic operating strategy:

- Diversion operations will begin whenever the average swamp level is less than the level in Lake Maurepas based on datum from strategically located level gauges and sensors.
- Diversion operation will be stopped whenever Lake Maurepas is at 0.5 NAVD or lower.
- Target is 3000 cfs.

Modeling indicates the diversions will operate for several months, followed by extended periods of no or reduced diversion.

Based on river level data, diversion could start in late winter and continue through late spring (May) and on into summer as river levels allow. Late summer and early fall river levels are typically low, and will yield lower rate diversions which will allow for a draining of the swamp during fall and winter when seed germination can occur.

### L12.2.2 Anticipated Operational Sequencing

The operation of the diversion will be driven by a number of factors, including:

- Diversion Flow Rate
- Blind River System Discharge Flow Rate into Lake Maurepas
- Blind River System Swamp Water Level
- Amount of Freeboard in the Transmission Channel

- Lake Maurepas Level

These factors are discussed below:

**L12.2.2.1 Diversion Flow Rate**

The diversion flow rate will be dependent on the level in the Transmission Channel and the respective flow rate through the Transmission Channel. This flow will be controlled by manually initiated set points at the remotely located Operations Building from the Operator Control Panel (OCP). Operating staff will adjust the diversion flow rate weekly during normal operation by adjusting a flow set point. This flow set point will allow a range of operation between 0 - 4000 cfs through a real-time feedback loop controlling the Diversion Structure culvert gates. The flow set points will be remotely adjustable through input at the Operations Control Panel (OCP). Level and flow instrumentation will measure the flow rate and channel level, and transmit signals to the OCP to allow these parameters to be monitored.

**L12.2.2.2 Blind River System Discharge Flow Rate into Lake Maurepas**

The outflow rate from Blind River (with the exception of naturally induced outflow from precipitation events) will be controlled by the diversion flow rate. Strategically positioned level and flow instrumentation will transmit signals to the OCP to allow operators to monitor the Blind River flow rate and level. If the flow rate is too high or too low, the operating staff will then adjust the Diversion Structure culvert gates accordingly.

**L12.2.2.3 Blind River System Swamp Water Level**

The swamp water level in the Blind River System will be controlled from the diversion gates and the gates at the control structures. Staff will set the desired flow into the Transmission Channel using the gates at the Diversion Structure and then perform more finite adjustment of the control structure gates to keep the swamp water level elevation between 0 - 2 feet. These adjustments will be manually input by adjusting the set points through the OCP.

**L12.2.2.4 Amount of Freeboard in the Transmission Channel**

Transmission channel freeboard will be dependent on the operation of the gates at the Diversion Structure and the control structures. Operators will adjust the operating set points for the diversion flow rate and the flow rates through the control structures to maintain a freeboard at approximately 3 feet in the Transmission Channel.

**L12.2.2.5 Lake Maurepas Level**

While the level in Lake Maurepas is not actually controlled by operation of this diversion, it will influence operation of the diversion. When the lake level is higher than 0.5 NAVD (the lake is higher than the swamp), this will allow operation of the diversion system to flood the swamp. When the lake level gets below 0.5 NAVD, the diversion operation will be stopped or reduced to allow the swamp to drain, providing an artificial dry period.

### L12.2.3 Expected Operating Setpoints

The following table (Table L12.2.3-1) will provide the expected operating setpoints and range of measurement provided for each of the control parameters.

**Table L12.2.3-1 - Expected Blind River Diversion Operating Setpoints**

| Operating Parameter | Set Point | Range of Measurement |
|---|---|---|
| Transmission Channel Level | 1.5 to 3.0 foot freeboard | (-3.0) - (0) feet |
| Transmission Channel Flow Rate | 0 - 4000 cfs | 0 - 4000 cfs |
| Blind River System Outflow Rate | Monitoring only | (- 2000) - (6000) cfs |
| Blind River System Swamp Elevation | Varies | 0 – 2.5 feet |
| Lake Maurepas Level | Monitoring only | (-2.0) - (10) feet |

## L12.3 Long-Term Monitoring Plan

Long term monitoring by O&M personnel includes:

- Maintenance of operation and maintenance logs for the structures and equipment.
- Monitoring of any of the alternatives daily by a technician
- Monitoring/inspecting all gates and structures (including testing generator operation) once a month in the distribution system
- Inspection and calibration of instrumentation weekly at six sites (control structures)
- Recording and documenting levels, flow and level daily for the Mississippi River, diversion structure(s) and distribution flow control structure
- Recording and documenting readings from water level gauges and indicators throughout the system.
- Using depth profiles to determine the degree of siltation in the sedimentation trap areas, transfer canal(s), distribution canal(s), and berm gaps annually to determine the amount of sedimentation that has occurred
- Regular monitoring of the culverts under roads and railroads and berm gaps to make sure they are clean and flow freely.
- Assessment of vegetation types at inception of project with follow-up assessments annually to determine the impact the project is having on the wetland system. (Not included in O&M costs)

## L12.4 Operation and Maintenance Costs

Excluding annual labor, cyclical costs, equipment costs, electrical costs, and mowing costs, the estimated O&M cost per annum for the Blind River Diversion Project alternatives are as follows:

### L12.4.1 3000 cfs Culvert Diversion O&M Costs

Expected annual culvert diversion O&M costs for the diversion structure are:

- Labor - $40,000
- Equipment - $15,000
- Annual Costs including electrical, mowing, etc. - $8,300

Total Diversion Structure O&M costs with 20 percent contingency allowance - $76,000

### L12.4.2 3000 cfs Transfer Canal O&M Costs

Expected annual Transfer Canal O&M costs are:

- Labor - $42,000
- Equipment - $12,000
- Annual Costs including electrical, mowing, etc. - $12,000

Total Transfer Canal O&M costs with 20 percent contingency allowance - $79,000

### L12.4.3 Distribution Canal O&M Costs

Expected annual Distribution Canal O&M costs are:

- Labor - $42,000
- Equipment - $10,000
- Annual Costs including electrical, mowing, etc. - $11,000

Total Transfer Canal O&M costs with 20 percent contingency allowance - $76,000

### L12.4.4 Control Structure O&M Costs

Expected annual Distribution Canal O&M costs are:

- Labor - $134,000
- Equipment - $23,000
- Annual Costs including electrical, mowing, etc. - $12,000

Total control structure O&M costs with 20 percent contingency allowance - $203,000

#### L12.4.5 Gap and Culvert O&M Costs

Expected annual Gap and Culvert O&M costs are:

- Labor - $15,000
- Equipment - $7,000
- Annual Costs including, etc. - $2,000

Total Gap and Culvert O&M costs with 20 percent contingency allowance - $29,000

#### L12.4.6 Total O&M Cost for 3000 cfs Diversion

The total operation and maintenance cost (excluding dredging) for the 3000 cfs Romeville Diversion is expected to be $463,000 including contingencies.

L12.5 Repair, Rehabilitation and Replacement

Excluding annual labor, equipment costs electrical costs, and mowing costs, the estimated cyclical repair, rehabilitation, and replacement costs for the Blind River Diversion Project alternatives are as follows.

#### L12.5.1 3000 cfs Culvert Diversion RR&R Costs

Expected annual culvert diversion RR&R costs for the diversion structure are:

- Labor -- $12,000
- Materials -- $10,000

Total Diversion Structure RR&R costs with 20 percent contingency allowance - $27,000

#### L12.5.2 3000 cfs Transfer Canal RR&R Costs

Expected annual Transfer Canal RR&R costs are:

- Labor -- $5,000
- Materials -- $4,000

Total Transfer Canal RR&R costs with 20 percent contingency allowance - $11,000

#### L12.5.3 Distribution Canal RR&R Costs

Expected annual Distribution Canal RR&R costs are:

Total Distribution Canal RR&R costs with 20 percent contingency allowance - $0

### L12.5.4 Control Structure RR&R Costs

Expected annual Control Structure RR&R costs are:

- Labor -- $20,000
- Materials -- $18,000

Total control structure RR&R costs with 20 percent contingency allowance - $46,000

### L12.5.5 Gap and Culvert RR&R Costs

Expected annual Gap and Culvert RR&R costs are:

- Labor -- $5,000
- Materials -- $3,000

Total Gap and Culvert RR&R costs with 20 percent contingency allowance - $9,600

### L12.5.6 Total RR&R Cost for 3000 cfs Diversion

The total repair, rehabilitation, and replacement costs (excluding dredging) for the 3000 cfs Romeville Diversion are expected to be $93,600 including contingencies.

## L13 PRELIMINARY OPINION OF ESTIMATED COSTS

Preliminary opinion of estimated costs were used to assist in the evaluation and screening of the array of management measures (components) and the alternative plans considered for the diversion project. This section presents these preliminary opinion of estimated costs and the procedures used to develop them. The estimates are presented in the standard cost categories used by the USACE, as follows:

- Construction costs
- Engineering and design (E&D) costs
- Supervision and administration (S&A) costs
- Costs for lands, easements, rights-of-way, relocations, and disposal areas (LERRD)
- Contingencies
- Total first costs (capital costs)
- Operation and maintenance (O&M) costs
- Total project costs

**Conceptual-level Design**

The preliminary estimates described in this report are based on conceptual-level design analyses, topographic data, and schematics. These estimates are intended as comparative estimates to assist in the screening steps, and to indicate relative total project costs. These estimates should not be used to establish final budgets or funding for the project.

**Opinion of Probable Project Costs**
The opinion of estimated costs developed for the project are the engineer's opinion of probable project costs, and should not be considered as construction quotes, estimates, or bids. See Para. L13.8 for a discussion of risks and uncertainties that could impact the construction costs.

## L13.1 Background Information

**Detailed Estimate for the Tentatively Selected Plan**
A more detailed opinion of estimated cost will be prepared separately for the tentatively selected plan (TSP), as described in **Annex L-7**. The TSP opinion of estimated cost will include a construction opinion of estimated cost using the USACE MCACES-II cost estimating program. The TSP estimate will be prepared as the preliminary design progresses on the TSP.

**Design Flow Rates**
The initial array of alternatives considered diversion flow rates from 500 cfs to 20,000 cfs, and the preliminary opinion of estimated costs covered this full range of flows. The specific flow rates being used in the initial alternative arrays are in 500 cfs increments up to 5,000 cfs, then in 5,000 cfs increments to 20,000 cfs.

**System Components**
The diversion project requires several different types of management measures, or components, serving different functions, which will be combined to form the alternative plans. Separate opinion of estimated costs were prepared for each component and for each design flow rate. These estimates were then combined to develop the overall opinion of estimated cost for each alternative plan for each design flow rate. The components are:

- Diversion facility
- Transmission canal
- Control structures
- Berm gaps
- Cross culverts at the Highway 61corridor
- Instrumentation

**Diversion Alignments**
The array of alternative plans considered three diversion alignment options. In the final array of alternative plans, these were identified as follows:

- Alternative 2 – Romeville Alignment near Mississippi River Mile 162.0
- Alternative 4 – South Bridge Alignment near Mississippi River Mile 167.0
- Alt. 4A – 500 cfs to the existing Parish drainage channel and 2,500 cfs to the North Distribution Canal
- Alt. 4B – 1,500 cfs to the existing Parish drainage channel and 1,500 cfs to the North Distribution Canal
- Alternative 6 – Dual Alignment, using a 50/50 flow split between the Romeville Alignment and the South Bridge Alignment

**Topographic Data**

Most of the construction quantities and estimates were based on the 2001 LiDAR-based topographic data obtained from the Louisiana State University. Both a digital elevation model (DEM) and a set of 2-foot contours were available and used. This LiDAR dataset is the best available topographic data, but may not accurately represent the topographic features and trends in the Swamp, and must be used with the following cautions:

- The LiDAR pulses have difficulty in penetrating dense vegetative cover, as is typical in the Swamp
- The LiDAR pulses are absorbed by water, and topography at water bodies may not be well-represented
- The DEM did not appear to be thoroughly processed and cleaned up, as the data coverage has gaps and apparent errors

The LiDAR-based contours appear to reasonably represent the topographic trends outside of the Swamp, and were used with more confidence for the diversion structure and the transmission canal. Also, the contours appear to accurately represent the embankments for Highway 61 and Kansas City Southern Railroad (KCS RR) where they cross the Swamp.

A bathymetric survey was obtained in 2009 for the Blind River upstream of IH-10 and for much of the existing perimeter drainage channels. These elevations were used to establish the downstream flow line elevations for the transmission canal options, and for the designs and sizes of the control structures.

Limited on-the-ground surveys became available late in the screening process; however, these surveys were not used for the preliminary construction opinion of estimated costs. Also, a new LiDAR-based topographic dataset was obtained in the summer of 2009, but was not processed in time for use in the preliminary construction opinion of estimated costs.

**Schematics, Exhibits, and Drawings**

The preliminary opinion of estimated costs for the various components were developed from conceptual exhibits, schematics, sketches, and in limited cases, from drawings for the components. Many of the exhibits were GIS-based, and should be considered less accurate than construction plans would be.

## L13.2 Preliminary Construction Opinion of estimated costs

The estimating basis for each component included the permanent and temporary facilities and considered existing conditions that would have significant impacts on construction pricing. These items included:

- Permanent constructed facility
- Ancillary site improvements, such as driveway, drainage, fence, security, lighting, etc.
- Existing site conditions
- Access
- Construction techniques
- Major temporary construction facilities, such as coffer dams and temporary levees
- Temporary road relocations and detours
- Temporary railroad relocations
- Pipeline adjustments
- Utility adjustments
- Construction sequences
- Storm water pollution prevention plan (SWPPP)
- Dewatering and management of surface water
- Net earthwork quantities – need for importing fill or exporting excess excavated material

**Cost Data Sources**

Multiple sources of construction cost and pricing information were used to develop the construction costs estimates for the management measures.

- The primary source of construction pricing was from a nationwide database developed and maintained by CDM Constructors, Inc. (CCI) using the Timberline cost estimating software. The data includes material and equipment costs, labor costs, and construction equipment costs (i.e. hourly costs).
- The costs of various equipment and specialty items were based on quotes from vendors and manufacturers. Examples include sluice gates, control gates, valves, vacuum pumps, and rip rap. The quotes provided costs for the equipment or material delivered to the site. The installation costs (labor and construction equipment) were developed in the Timberline program.

- Published construction cost information from R.S. Means Company, Inc. was used to supplement the cost data from the CCI database and the vendor quotes.
- CDM engineers and CCI construction professionals provided input on construction costs, techniques, and expected production rates.
- Several construction contractors and construction equipment vendors were contacted for input on construction techniques, construction equipment, and productivity, especially for work within the Maurepas Swamp.

**Construction Opinion of estimated costs**

The construction opinion of estimated costs were prepared in an Excel spreadsheets. The estimates developed an expected direct construction cost, and then a 30% markup was added to account for subcontractor markups, insurance, overhead, profit, and other such costs. As the design is still at the conceptual level, a 15% contingency was also added to the construction opinion of estimated cost. A 10% contingency was used for the canals, which consisted primarily of large quantities of earthwork. These construction cost contingencies are separate, and in addition to, the contingency factor applied to the overall project costs, as described in Paragraph L13.4.

A broad range of flow rates were being evaluated for the diversion and it was impractical to prepare opinion of estimated costs for each reasonable flow rate with the limited schedule and budget available for the feasibility study. Therefore, a limited number of estimates were used to identify cost trends, and these trends were then used to project costs for other flow rates. **Table L13.2-1** and **Table L13.2-2** show which estimates were prepared for each component and design flow rate. The detailed construction opinion of estimated costs are in **Annex L-1**.

**Table L13.2-1 Estimates for Components Sized for the Diversion Flow Rate**

| | **Diversion Flow Rate, x1,000 cfs** | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **Item** | **0.5** | **1.0** | **1.5** | **2.0** | **2.5** | **3.0** | **3.5** | **4.0** | **4.5** | **5.0** | **10.0** | **15.0** | **20.0** |
| | | | | | | | | | | | | | |
| **Romeville - Base Design** | | | | | | | | | | | | | |
| Diversion Culvert (Elev. 11) | X | X | | X | | X | | X | | X | X | X | X |
| Transmission Canal - earthen | X | X | | X | | X | | X | | X | X | X | X |
| | | | | | | | | | | | | | |
| **Romeville - Alternative Designs** | | | | | | | | | | | | | |
| Diversion Siphon | X | X | | X | | X | | X | | X | | | |
| Batture Crossing - Siphon Pipe | X | X | | X | | X | | X | | X | | | |
| Batture Crossing - Inlet Canal | X | X | | X | | X | | X | | X | | | |
| Diversion Culvert (Elev. 5) | X | X | X | X | X | X | X | X | X | X | X | | |
| Trans. Canal - earthen, deep | X | X | | X | | X | | X | | X | X | X | X |
| Trans. Canal - concrete-lined | X | X | | X | | X | | X | | X | X | X | X |
| | | | | | | | | | | | | | |
| **South Bridge** | | | | | | | | | | | | | |
| Diversion Culvert | X | X | | X | | X | | X | | X | X | X | X |
| Diversion Siphon | | | | | | | | | | | | | |
| Trans. Canal - earthen | X | X | | X | | X | | X | | X | X | X | X |
| North Distribution Canal | | | X | | X | | | | | | | | |
| Parish Ditch Widening | | | X | | | | | | | | | | |

| Item | Diversion Flow Rate, x1,000 cfs | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | 0.5 | 1.0 | 1.5 | 2.0 | 2.5 | 3.0 | 3.5 | 4.0 | 4.5 | 5.0 | 10.0 | 15.0 | 20.0 |
| | | | | | | | | | | | | | |

**Table L13.2-2 Estimates for Water Distribution and Water Management Elements**

| Item | Description |
|---|---|
| Control Gates | Based on existing drainage channel dimensions |
| Berm Gaps | Gap widths - 20', 100', 250', and 500' |
| Circulation Improvements KCS RR and Hwy 61 Corridor | |
| Small capacity culvert | 1 - 4' x 4' box culvert |
| Large capacity culvert | 4 - 5' x 5' box culverts |
| Bridge | 50' span |
| Improve Existing Drainage Channels | 2' deep and 20' wide |
| Diversions from Conway Canal | |
| Berm Gaps | |
| Diversion Ditch to HU 200's | |
| Instrumentation | Radio towers, gage stations, monitoring stations, control room |
| Romeville Alignment | |
| South Bridge Alignment | |
| Dual Alignment | |
| | |

**Projected Construction Costs**

The detailed construction opinion of estimated costs were used to project the costs for the various flow rates. The detailed estimates were plotted, demonstrating that all elements have a near linear trend of cost versus flow rate. The unit costs versus flow rate were also plotted to verify uniform trends. As shown on the plots, the costs are not completely linear, due to approximations in quantities and design features made at the conceptual design level. The linear trends were calculated, and the projected costs were then used for the construction opinion of estimated costs for the management measures and alternative plans. The plots and projected cost calculations are in **Annex L-1**.

Annex L-1 has the following construction cost estimating items:

- Summary tables of the detailed estimates
- Summary table of the project estimates
- Tables with calculations to project the costs
- Figures of the construction costs versus flow rates

- Figures of the unit construction costs versus the flow rate
- Detailed construction opinion of estimated cost for each element

**Construction Opinion of estimated cost Summary**
**Table L13.2-3** has a summary of the construction opinion of estimated costs for the final array of alternative plans, using the projected construction costs. Paragraph L13.7 provides a comparison and evaluation of the various components and elements, and selection of specific elements. In **Table L13.2-3**, the diversion structure cost is for a diversion culvert, as the siphons are more expensive at the designated flow rates.

**Table L13.2-3 Construction Costs for Final Array of Alternative Plans**

| Item | Alt.2 Romeville 3,000 cfs | Alt. 4A South Bridge 3,000 cfs | Alt. 4B South Bridge 3,000 cfs | Alt. 6 Dual/Split 3,000 cfs |
|---|---|---|---|---|
| | | | | |
| Diversion Structure | | | | |
| Romeville | 12,600,000 | | | 10,800,000 |
| South Bridge | | 13,900,000 | 13,900,000 | 11,900,000 |
| Transmission Canal | | | | |
| Romeville | 23,600,000 | | | 15,300,000 |
| South Bridge | | 29,800,000 | 29,800,000 | 19,800,000 |
| North Distribution Canal | | 29,300,000 | 15,500,000 | 15,500,000 |
| Parish Drainage Channel | | | 9,900,000 | |
| Control Structures (6) | 24,000,000 | 24,000,000 | 24,000,000 | 24,000,000 |
| Berm Gaps (30) | 4,000,000 | 4,000,000 | 4,000,000 | 4,000,000 |
| Hwy 61 Crossings (4) | 8,400,000 | 8,400,000 | 8,400,000 | 8,400,000 |
| Instrumentation | 900,000 | 1,300,000 | 1,300,000 | 1,500,000 |
| Totals | 73,500,000 | 110,700,000 | 106,800,000 | 111,200,000 |

## L13.3 Lands, Easements, Rights-of-Way, Relocations, and Disposal Areas

Costs for lands, easements, rights-of-way, relocations, and disposal areas (LERRD) were developed based on current real estate costs and utility relocation costs. CDM obtained the services of a real estate services firm, with local experience, to evaluate and recommend current acquisition costs for the various tracts of land. Specific real estate acquisition needs are described in Section L15.

The project will cross existing pipelines and utility lines, which will have to be adjusted or relocated to accommodate the diversion project. Preliminary costs were based on budgetary estimates because specific pipeline and utility conflicts are still under evaluation. Final opinion of estimated costs will be based on quotes from the owners to adjust their facilities.

The project will also cross several roads and railroads. The costs for culverts under the transportation facility are included in the preliminary construction opinion of estimated costs. Likewise, temporary detour roads and temporary railroad relocations (shoofly tracks) are included in the preliminary construction opinion of estimated costs.

## L13.4 First Costs (Capital Costs)

The first costs, or capital costs, of the alternative plan is the sum of the construction costs, engineering and design, supervision and administration, LERRD, and a contingency. These cost categories were estimated as follows:

- Construction Costs – see Para. L13.1 and L13.2
- E&D Costs – 5% of the construction costs
- S&A Costs – 3% of the construction costs
- LERRD Costs – see Para. L13.3
- Contingencies – 25% of the subtotal of construction, E&D, S&A, and LERRD costs

The first costs for the initial array of alternatives are on **Table L13.4-1**.

**Table L13.4-1 Project First Costs**

| Item | Alt.2 Romeville 3,000 cfs | Alt. 4A South Bridge 3,000 cfs | Alt. 4B South Bridge 3,000 cfs | Alt. 6 Dual/Split 3,000 cfs |
|---|---|---|---|---|
| | | | | |
| Construction | 73,500,000 | 110,700,000 | 106,800,000 | 111,200,000 |
| Real Estate | 2,200,000 | 2,200,000 | 2,200,000 | 4,400,000 |
| Engineering & Design (E&D) | 3,700,000 | 5,500,000 | 5,300,000 | 5,600,000 |
| Supervision & Administration (S&A) | 2,200,000 | 3,300,000 | 3,200,000 | 3,300,000 |
| Contingencies (25%) | 20,400,000 | 30,400,000 | 29,400,000 | 31,100,000 |
| **Total First Costs** | **102,000,000** | **152,100,000** | **146,900,000** | **155,600,000** |

[1] All costs are in October 2009 prices

## L13.5 O&M Opinion of estimated costs

Operation and maintenance opinion of estimated costs were based on optional diversion arrangements as outlined in the alternatives to help in determining approximate cost of the single and dual diversion alternatives. These options had differing operation and maintenance requirements affecting operating cost. Operation and maintenance costs were based on estimations of the amount of labor to operate and maintain the equipment and structures with each option, materials, spare parts and supplies, electrical power, and other related costs. This cost estimation provided a basis for selection of the final alternatives based on operating cost added to the other costs on an annualized basis.

A final O&M opinion of estimated cost needs to be prepared after all engineering and electrical/instrumentation components have been selected.

Conceptual O&M costs were prepared for both single and dual siphon diversion structures and culvert diversion structures. Both single and dual diversions can result in meeting project goals, but operating costs are higher for the dual diversion alternative. (See discussions elsewhere on considerations of other performance and operational factors). Single or dual siphons will require higher O&M costs, which when combined with construction costs, make the single diversion culvert system more attractive. The following table (**Table L13.5-1**) provides the O&M opinion of estimated cost for each alternative.

**Table L13.5-1 Diversion Alternative O&M Opinion of estimated costs**

| Diversion Alternative | Estimated O&M Costs Annually |
|---|---|
| 1500 cfs Siphon System | $564,000 |
| 1500 cfs Culvert System | $525,000 |
| 3000 cfs Siphon System | $640,000 |
| 3000 cfs Culvert System | $590,000 |
| Dual 750 cfs Siphon System | $689,000 |
| Dual 1500 cfs Siphon System | $740,000 |

## L13.6 Total Project Costs

The total project costs are shown on Table L13.6-1, and consist of the first costs on an annualized basis and the O&M costs. The annual costs for the capital costs were developed using an interest rate of 4-3/8% over a 50-year period, the USACE rate for 2011.

**Table L13.6-1 Total Project Costs**

| Item | Alt.2 Romeville 3,000 cfs | Alt. 4A South Bridge 3,000 cfs | Alt. 4B South Bridge 3,000 cfs | Alt. 6 Dual/Split 3,000 cfs |
|---|---|---|---|---|
| First Costs | 5,060,000 | 7,540,000 | 7,280,000 | 7,710,000 |
| Operation & Maintenance Costs | 590,000 | 590,000 | 670,000 | 740,000 |
| **Total Project Costs** | **5,650,000** | **8,130,000** | **7,950,000** | **8,450,000** |

[1] All costs are in October 2009 prices

[2] First costs were annualized using a discount rate of 4-3/8% over a 50-year period

## L13.7 Cost Comparisons and Evaluations

Opinion of estimated costs were prepared for optional components and for optional elements of the components to help in refining the design and selecting components. These options typically had similar hydraulic or other performance characteristics. Therefore, the selection of elements could be based primarily on cost considerations. These comparisons were a refinement to the designs and opinion of estimated costs. Similar cost comparisons need to be prepared as the preliminary and final design progresses to develop a more cost effective project.

**Siphon vs. Culvert**

Hydraulic designs and layouts were prepared for both a siphon diversion structure and for a culvert diversion structure. Theoretically, both methods can provide similar hydraulic performance. (See discussions elsewhere on considerations of other performance and operational factors). As shown on the **Figure L13.7-1**, siphons are the more cost effective low flow rates and culverts for higher flow rates. The cross-over is near 1,000 cfs. Siphons will require higher O&M costs; however, including the O&M costs will adjust the cross-over point only slightly.

**Siphon Pipe vs. Inlet Canal**

At the diversion points, the east levee of the Mississippi River is typically 200 to 300 feet from the river bank and the diversion structure needs to be extended across the batture to the river bank. With the siphon pipe in the river, the pipes need to be protected with bollards, as on the West Pointe a la Hache and Naomi siphons. Also, providing a trash rack on the inlet end of each pipe could introduce a maintenance problem. With an inlet canal, there would be no facilities projecting into the river. The trash racks could be installed in the inlet canal, providing better access for cleaning and maintenance. The cost comparison is in **Annex L-1**.

Based on this comparison, and the trash rack considerations, the recommended design is to use an inlet canal.

**Diversion Culvert vs. Inlet Canal**

As with the siphon installation, the diversion culverts could be extended across the batture, or an inlet canal could be extended from the River to near the base of the levee.

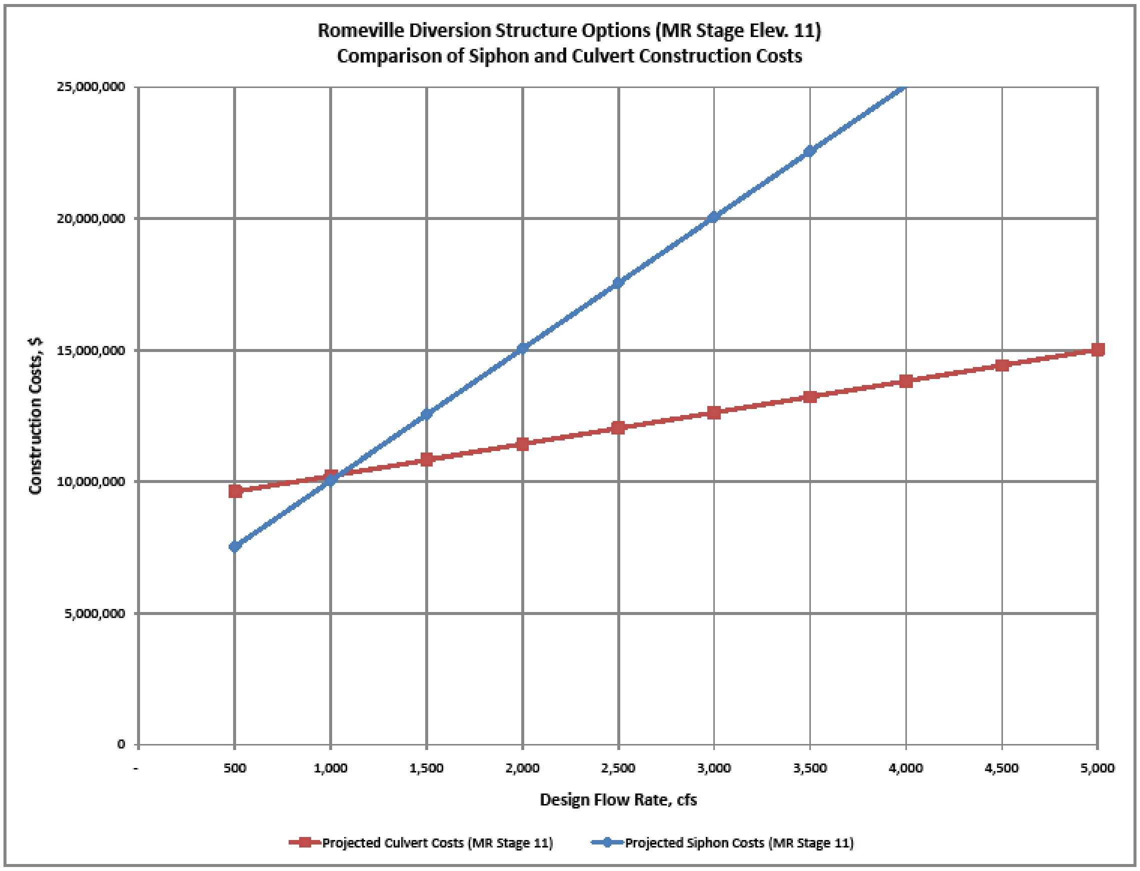



**Figure L13.7-1 Romeville Diversion Structure Options Comparison of Siphon and Culvert Construction Costs**

**Earthen Canal vs. Concrete-lined Canal**

Earthen canals are typically used for the transport of water and for drainage in open areas. Concrete-lined channels can be used to increase the hydraulic conveyance capacity. Concrete-lined channels are also used in areas with right-of-way constrictions, or areas with very high real estate acquisition costs. For this project, alternate concrete-lined channel designs and opinion of estimated costs were prepared for the Romeville alignment. As shown in the following **Figure L13.7-2**, the earthen channels are less expensive. Therefore, earthen channels are recommended for this project. The figure also shows the cost trend for a lower earthen canal, which would also require de-silting the outfall drainage channels, as discussed in Section L2.6. This item should be re-evaluated during the final designs.

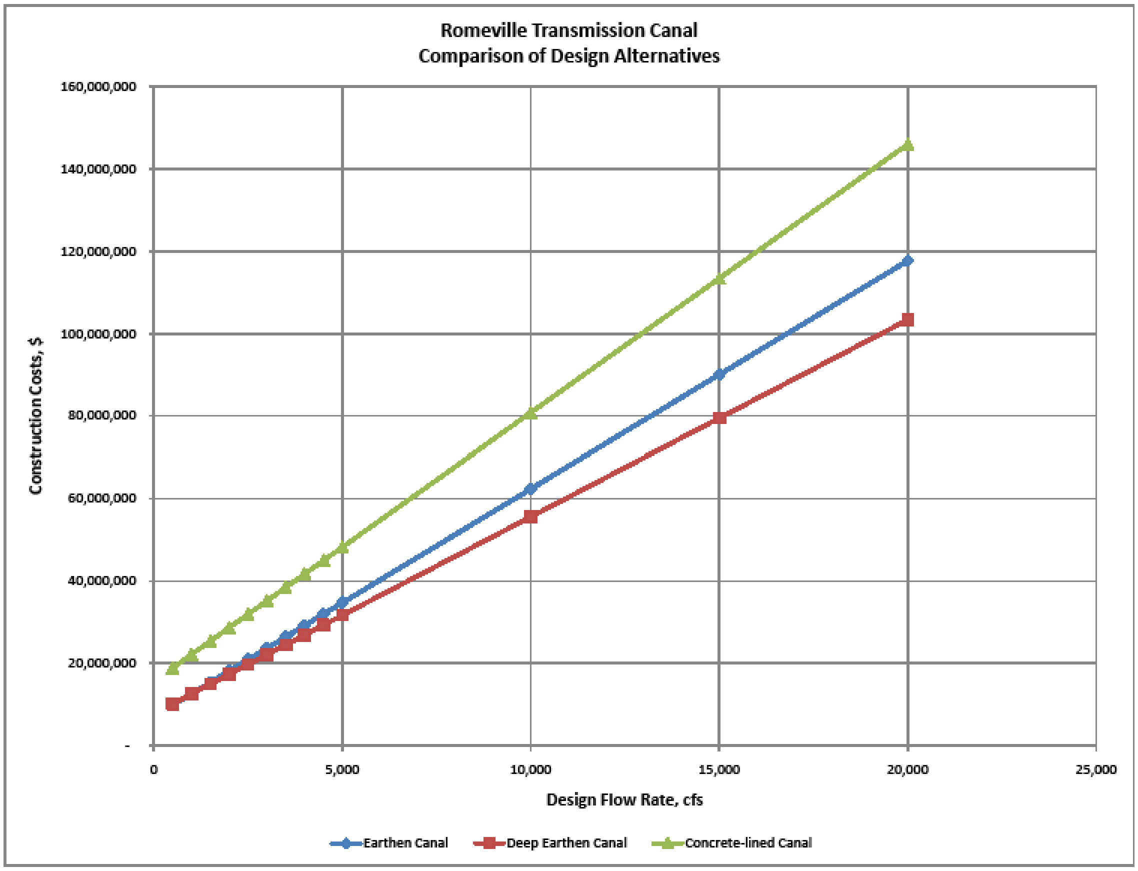



**Figure 13.7-2 Romeville Transmission Canal – Comparison of Construction Costs for Design Alternatives**

**Diversion Culvert Design Basis**

As discussed in the hydraulic design of the diversion culvert, three Mississippi River stages were analyzed as the design basis. Construction opinion of estimated costs were developed for the Romeville diversion culvert designed for Mississippi River stage Elev. 5 and for Elev. 11. The following figure demonstrates the major increases in construction costs to be able to divert the full design flow rate at the lower stages in the Mississippi River.

**Railroad Culverts vs. Railroad Bridge**

(To be analyzed during final design)

**Road Culverts vs. Road Bridge**

(To be analyzed during final design)

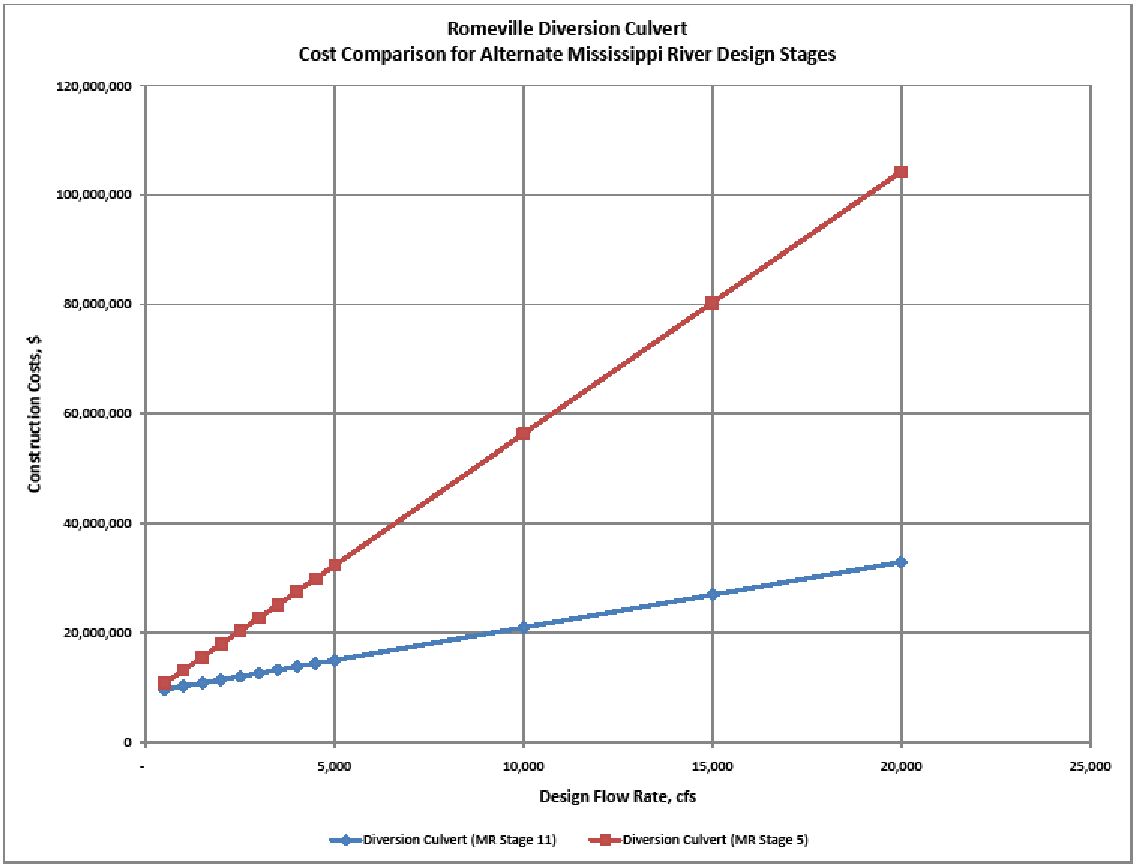



**Figure L13.7-3 Romeville Diversion Culvert – Design for MR Stage 5 vs. 11 Construction Costs**

Additional cost comparisons and evaluations need to be provided in the design phase to select alternate elements, including the following:

## L13.8 Cost Risk Analysis

A detailed risk analysis will be prepared as part of developing the MCACES-II construction opinion of estimated cost.

This section addresses the risks and uncertainties of cost escalation related to the conceptual construction opinion of estimated costs in three major categories: costs/pricing, excess excavated material, and construction conditions.

### L13.8.1 Cost Risks

The construction opinion of estimated costs have the following risk factors:

- Quantities – The earthwork quantities were developed from conceptual design drawings and used the old LiDAR topographic data. The quantities can be expected to vary substantially compared to future quantities developed from topographic surveys.

- Pricing – The unit pricing was based on a nationwide database of prices, vendor data, and contractor input, all considered as budgetary level, and not firm quotes.
- Geotechnical – The geotechnical investigation was not available, and assumptions had to be made on soil and groundwater conditions.
- Structural designs – No structural designs have been prepared, and certain dimensions and quantities (i.e. – structural concrete) were based on professional experience on other large civil works projects.
- Inflation – future inflation trends are unknown.
- Price trends – there are currently lower prices due to the recession, but prices could increase as the economy improves.
- Higher prices in Louisiana – the area experienced high labor and material prices, causing significant construction price increases due to the extensive reconstruction activity after Hurricane Katrina.
- Scope changes – scope changes will impact pricing.

**Upland Areas**
The diversion culvert and the transmission canal involve heavy civil construction, with construction cost/pricing information readily available. The quantities could vary significantly, as discussed above. The prices used should establish adequate budgets.

**Swamp**
Price risk is high for the control structures and the berm gaps in the Swamp. There is little database information on costs for work in such conditions. Access will be difficult and could cause a major price escalation.

**Transition Areas**
The Highway 61 cross culverts and drainage channels will be constructed in the Swamp conditions, but with excellent access from Highway 61. The work involves common construction practices with readily available construction pricing information. The quantities still need to be better defined with on-the-ground topographic surveys. The current price risk level is moderate, and the price risk at the end of final design will be much lower, with more confidence in quantities.

**Summary of Construction Cost Estimate Risks**
Diversion structure and transmission canal – moderate risk

Control structures and berm gaps – high risk

Highway 61 Cross Culverts – moderate risk

### L13.8.2 Beneficial Use of Excess Excavated Material

This section addresses the risks and uncertainties of finding a beneficial use for the excess excavated material, or disposing it with minimal impacts, and the related risks to project cost escalations.

**Upland Areas**

Construction of the transmission canal will result in large volumes of excess excavated material, with a much smaller excess volume from the diversion culvert. The opinion of estimated costs are based on the assumption that there is a market for fill material, and that the construction contractor will be able to sell the material, or dispose of it off of the Blind River project site for use on other unrelated projects. This assumption is based on the excess material being suitable for fill, and on the current or recent shortage of fill material in South Louisiana. If the material can't be sold or disposed of off-site, a disposal area will have to be acquired for the project. The price risk should be considered moderate, as the quality of the spoil material is not yet defined.

A temporary levee will be constructed on the river side of the east Mississippi River levee to allow open-cut construction of the diversion culverts through the levee. The current design concept and cost estimating is based on the use of excess material from the transmission canal for the temporary levee. If the material is not suitable for levee construction, appropriate soil will have to be imported to the site, at a significant cost increase.

After the geotechnical investigation identifies the soil properties, the final approach to spoil disposal can be developed, possibly using the following approaches. If the approach is clearly defined, the construction contractor can then include the necessary costs in the bid, reducing the risks of changed costs during contract performance.

- Suitable for fill – Require the contractor to remove the material from the site, becoming the owner of the spoil. Allow it to be sold or disposed, as determined by market conditions.
- Unsuitable material – Acquire a spoil disposal site for placement of the material.

**Swamp**

Small volumes of excess excavated material will result from excavation of the berm gaps and construction of the control structures in the Swamp. The material will not be removed from the Swamp, but will be disposed of locally. The current concept is to place the excess excavated material on and adjacent to the existing spoil banks along the existing drainage channels. This will increase the size and height of the existing berms (spoil banks), thus developing a beneficial use for the spoil material by creating additional refuge areas for wildlife to escape flood conditions in the Swamp.

**Transition Areas**
The excess excavated material at the Highway cross culverts will be disposed of in the Swamp, creating wildlife refuge areas, creating a beneficial use.

**Summary of Risks from Excess Excavated Materials**
In summary, the cost escalation risks due to excess excavated material disposal are:

- Diversion structure and transmission canal – moderate risk, as the material quality is still unknown
- Control structures and berm gaps – low risk, as the excess material will be beneficially disposed of locally
- Highway 61 cross culverts – low risk, as the excess material will be beneficially disposed of locally

**L13.8.3 Construction Conditions**
The project will be constructed under two very different site conditions: upland areas and the Swamp. This section discusses construction risks primarily impacting costs and completion schedule.

**Upland Areas**
The diversion culvert and most of the transmission canal will be constructed in upland areas using conventional construction techniques. The downstream end of the transmission canal is in the Swamp and subject to the risks described in the next section. The sites are in an area of industrial, commercial, and residential development, and site conditions and required construction approaches are known from experience with such developments. The two components have direct access from two paved public roads. Also, large equipment and high volume items could be delivered to the diversion culvert site via barges on the Mississippi River. Some of the major construction risks and how these were considered in design and cost estimating are:

- Mississippi River flood conditions – A temporary levee will be designed and constructed to permanent levee standards, reducing risks from Mississippi River flooding.
- Unknown underground conditions – These two components are in largely undeveloped agricultural areas. The geotechnical investigations will define soil conditions. Therefore, risks from unknown conditions will be reduced.
- Groundwater – High groundwater conditions, high volumes of groundwater, and special water control techniques were anticipated in the construction cost estimates, reducing risks of additional costs.
- Extended wet weather – Extended wet weather could delay the project schedule and increase costs incurred during delays. These two components are

in upland areas, with positive drainage towards the Swamp, reducing the impact of wet weather.

- Conventional construction techniques – The components will be constructed with conventional and well-established construction practices and with convention equipment, resulting in reduced risks.
- Floods – Minimal risk of losing equipment, materials, and partially constructed facilities to flood damage, as the sites are above flood elevations or protected by the temporary levee.

The opinion of estimated costs developed to date considered these conditions; therefore, risks of cost escalations due to them are reduced.

**Swamp Conditions**
The control structures and the berm gaps will be constructed at remote sites deep within the Swamp. Special construction equipment and construction techniques will be necessary to work in a setting with soft and saturated soils, standing water, and extended periods of high water. Access will be via barges and work boats in the existing drainage channels. Risks include:

- Use of inappropriate construction equipment
- Construction techniques not suited to the Swamp
- Difficulty in moving large construction equipment, materials (large control gates and high volumes of concrete), and labor to the work sites

The risks to the contractor would be in the form of equipment mired in muck, low productivity, higher costs for access, extended delays to the schedule due to wet conditions, and higher direct costs to perform the work. The bidders can be expected to increase bid prices to cover such risks, and the contractor can be expected to more aggressively pursue change orders and claims to recover cost over-runs.

The construction opinion of estimated costs to date considered these risks, using higher unit prices for the work, and estimating additional costs to gain access. However, these risks need to be re-evaluated as the designs progress to determine if there is sufficient cost coverage. Another consideration that can be reviewed during the design phase is to allow a construction access road to one or more of the control structures, thereby reducing access costs.

**Transition Areas**
The Highway 61 crossings and channels between the road and the KCS RR will have excellent direct access from Highway 61. The work site conditions will be similar to the other sites deeper in the Swamp; however, spoil from the proposed drainage channels can be placed to establish raised access roads and work pads above the Swamp conditions. The direct access from Highway 61 to the work sites greatly reduces the access costs.

**Summary of Construction Risks**

In summary, the construction risks and uncertainties, primarily related to cost and schedule, are:

- Diversion structure and transmission canal – low risk
- Control structures and berm gaps – high risk
- Highway 61 Cross Culverts – low risk

### L14. SCHEDULE FOR DESIGN AND CONSTRUCTION

## L14.1 Design Schedule

The design services for Blind River are in two distinct parts: Preliminary and Final. The preliminary design services will include additional field investigations to refine the hydraulic modeling. The hydraulic modeling will refine the expected water surface elevations for the project and define flow patterns through the swamp based on the diversion flows and downstream water level conditions. Additional geotechnical and surveying services will be required to provide data for property acquisition and hydraulic modeling of water surfaces.

Schedule for design services is as follows:

| | |
|---|---|
| Preliminary design field services | 6 months |
| Preliminary design supplemental hydraulic modeling | 3 months |
| Preliminary design supplemental water quality modeling | 2 months |
| Preliminary design calculations | 4 months |
| Preliminary engineering drawings | 3 months |
| Preliminary engineering specifications | 2 months |
| Preliminary design opinion of estimated cost | 1 month |
| Total Preliminary Engineering Duration | 12 months |
| (Note: activities are in some cases concurrent) | |
| Final design calculations | 2 months |
| Final design drawings | 6 months |
| Final design specifications | 3 months |
| Final design opinion of estimated cost | 1 month |
| Final design bid documents | 1 month |
| Total Final Engineering Duration | 10 months |
| (Note: activities are in some cases concurrent) | |

## L14.2 Construction Schedule

The construction for Blind River will involve several pipeline, utility and transportation relocations that will impact the construction schedule. It is not known whether the temporary and permanent relocations will be performed under the project contract(s) or will be contracted independently by the pipeline or utility owner. The railroad in many cases provides their own relocation design and construction. These relocations will need to be carefully coordinated with the primary construction contract(s).

Each element of the project can be constructed concurrently under a single contract or with multiple contracts since several specialized services are required. The diversion structure will involve temporarily relocating the flood protection levee and electrical and mechanical skilled workers. The transmission channel will require large earth moving equipment and compaction equipment. The berm gaps and control structures will require barge operations for installation. It may be more cost effective to procure each of these under separate contracts to optimize the use of specialty services. The proposed schedule will consider separate relocation and construction contracts.

| | |
|---|---|
| Utility Temporary Relocations | 4 months |
| Pipeline Permanent Relocations | 8 months |
| Railroad Temporary Relocations | 6 months |
| Utility Permanent Relocations | 4 months |
| Railroad Permanent Relocations | 12 months |
| Diversion Structure Levee Temporary Relocation | 6 months |
| Diversion Structure Construction | 15 months |
| Transmission Channel Construction | 8 months |
| Berm Gap and Control Structure Construction | 10 months |
| Total Construction Duration | 24 months |
| (Note: activities are in some cases concurrent) | |
| Start up and training activities | 4 months |

# L15 REAL ESTATE AND RIGHT-OF-WAY REQUIREMENTS

## L15.1 Scope of Requirements

This section presents the basis for real estate necessary to accommodate the construction, operation, and maintenance of the proposed Blind River project. The project will require acquisition of permanent rights-of-way, temporary construction easements, and permits to construct and operate the proposed facilities. The following paragraphs provide the basis for determining actual real estate acquisition dimensions and locations.

The detailed real estate analysis and acquisition process is described in Appendix K of the main report.

Current land ownership in the project area is identified, in general terms, on Figure L15-1. Following is a summary of ownership, as it impacts the proposed project:

**Diversion Facility**

- The general area is in private ownership, with private ownership extending across the levee, across the batture, to the edge of the Mississippi River.
- The Pontchartrain Levee District has a levee easement for the levee.
- LA 44 is in an 80-foot wide road right-of-way owned by State of Louisiana.

**Transmission Canal**

- The general area is in private ownership.
- The Canadian National Railroad is in a 100-foot wide right-of-way.
- LA 3125 is in a 300-foot wide road right-of-way owned by State of Louisiana.
- The east end of the transmission canal alignment is in the Maurepas Wildlife Management Area, owned by the State of Louisiana, Department of Wildlife and Fisheries
- Several utilities and pipelines cross the alignment – these facilities are apparently in easements.

**Control Structures**

- All of the control structures are in the Maurepas Wildlife Management Area, owned by the State of Louisiana, Department of Wildlife and Fisheries

**Cross Culverts and Interconnecting Channels at Highway 61 Corridor**

- Highway 61 is in a 300-foot wide road right-of-way owned by State of Louisiana.
- Ownership of the corridor between the KCS RR and Highway 61 is unclear.
- Pipelines in the corridor are apparently in easements.

**Road and Railroad Crossings**

It is considered that rights-of-way or easements will not be acquired across the roads and railroads to accommodate the components. Instead, the road and railroad owners will allow construction, operation, and maintenance under a permit approval.

## L15.2 Permanent Real Estate Acquisition Requirements

### L15.2.1 Diversion Facility

The diversion site size will be determined by final design and layout of the diversion culvert and inlet canal, and the ancillary facilities planned at the site. Current planning of a 400-foot wide easement needs to be verified as the design advances.

### L15.2.2 Transmission Canal

Most of the transmission canal alignment crosses undeveloped land in private ownership. The downstream end (east end) is within the wildlife management area owned by the State. The right-of-way width has been developed to accommodate the canal, berms on both sides of the canal, and additional strips on both sides for drainage and mowing access. The typical canal section and right-of-way width is further defined as follows:

- Berms – 12-foot wide minimum top width to allow maintenance vehicle access, 4:1 side slopes (interior), and 4:1 or 5:1 exterior side slopes for mowing safety.
- Right-of-way width – 500 feet
- Without berms – minimum of 30 feet each side for large maintenance equipment and drainage
- With berms – minimum of 10 feet beyond the outer toe of berm on each side, for a local drainage swale and mowing access
- ROW drainage – provide a small drainage swale at the ROW line and discharge to local drainage.

The calculated right-of-way width varies as existing ground elevations vary along the alignment. The current design for the 3,000 cfs transmission canal requires a maximum width of approximately 315 feet. The current acquisition plan has identified a 500-foot wide strip, a width that will be adequate for the current design. The final plan may require wider berms than the required minimum to allow a balance of cut and fill quantities to optimize construction costs.

The transmission canal will cross several existing local drainage channels. Some or all of these may need to be relocated. Additional permanent right-of-way may be required to accommodate these drainage relocations.

The box culverts at LA 44, CN RR, and LA 3125 will be installed under permits.

### L15.2.3 Control Structures

The proposed control structures are located in the Swamp, on land owned by the State. Therefore, no real estate acquisition will be required.

Alternate access may be considered, consisting of a permanent access road to each control structure. Such access routes are likely completely within lands owned by the State, and no additional real estate acquisition will be required.

### L15.2.4 Cross Culverts and Interconnecting Channels at KCS RR/Highway 61 Corridor

The proposed box culverts will be installed under Highway 61 by permit.

The land ownership of the corridor between the KCS RR and Highway 61 is unclear, although it is considered to be private. Rights-of-way may need to be acquired to promote drainage under the railroad crossing and the HWY 61 culverts. The right-of-way width should be established, as discussed for the transmission canal to provide sufficient space for the channel, berms, and drainage.

## L15.3 Temporary Construction Easements

This section defines temporary construction easements and temporary access easements required to construct the proposed project components. Dimensions of the actual temporary easements will be defined in the final design phase, as the components are designed, construction phasing and sequence are determined, and temporary easements needs are finalized.

### L15.3.1 Diversion Facility

The land ownership on the batture and at the levee is unclear. The existing levee will be temporarily removed at the box culvert crossing. The soil will have to be stockpiled near the site in a temporary construction easement. This stockpile site should be between the levee and the temporary LA 44 relocation. If the soil is stockpiled on the batture, it would be subject to flooding from the Mississippi River.

The temporary levee and the limits for the existing levee removal extend past the right-of-way requirements for the permanently installed facilities. Temporary construction easements will be required for the levee work.

### L15.3.2 Transmission Canal

The transmission canal has good access from LA 44 and from LA 3125 and the proposed permanent right-of-way is adequate for the construction activities. Therefore, no temporary construction or access easements are required for the transmission canal. However, see the following items for construction at the transportation facilities crossings.

### L15.3.3 LA 44 Temporary Relocation

The proposed diversion box culvert will be installed under LA 44 by open-cut construction. LA 44 will be relocated temporarily to the east, and the existing pavement removed to accommodate the box culvert construction. The road relocation will require a temporary construction easement approximately 1,000 feet long and 80 feet wide. The actual temporary easement dimensions will be determined in final design, as the temporary pavement relocation design is completed.

### L15.3.4 CN RR Temporary Relocation

The proposed transmission canal will have a multi-cell concrete box culvert installed under the CN railroad tracks by open-cut construction. The railroad will have to be temporarily relocated (shoofly) to accommodate the construction. The final design requirements have not yet been obtained from the railroad. It is likely that the shoofly will have to be designed to maintain full operating speeds, resulting in a long shoofly layout. The existing railroad right-of-way is 100 feet wide, and the shoofly might be located totally in the existing right-of-way. For planning purposes, it is considered that the shoofly will have to be at least partially outside of the right-of-way. Consider that the construction easement will be 2,000 feet long and 50 feet wide.

### L15.3.5 LA 3125 Temporary Relocation

The proposed diversion box culvert will be installed under LA 3125 by open-cut construction. LA 3125 will be relocated temporarily to one side, and the existing pavement removed to accommodate the box culvert construction. The existing road right-of-way is 300 feet wide, and this should be adequate space to contain the temporary relocation totally within the existing right-of-way. The temporary relocation can be done in phases, if necessary, to avoid acquiring temporary construction easements.

### L15.3.6 Control Structures

The proposed control structures are located in the Swamp, on land owned by the State. Access will be via the existing drainage channels. No temporary construction or access easements are necessary.

### L15.3.7 Cross Culverts at Highway 61

Concrete box culverts will be constructed under Highway 61 at four locations by open-cut construction. The pavement will be temporarily relocated to accommodate the construction. Highway 61 has a 300-foot wide right-of-way, and the temporary relocations can be done within that space. Several phases may be necessary.

### L15.3.8 Interconnecting Channel at KCS RR/Highway 61 Corridor

Interconnecting channels may be excavated across the corridor between the KCS RR and Highway 61. The ownership of the corridor is unclear. All access will be from Highway 61 and no temporary access easements will be needed. If an adequate permanent right-of-way width is acquired, temporary construction easements will not be necessary.

## L16 INFRASTRUCTURE RELOCATIONS

### L16.1 Purpose and Scope

The purpose of this section is to identify public and private infrastructure that may impact the design, construction, operation, and maintenance of the Blind River diversion project. The categories of facilities include public utilities, private utilities, pipelines, roads, and railroads. Central facilities, such as water treatment, water storage, water pump stations, water wells, wastewater pump stations, and wastewater treatment plants are also included in the public infrastructure category. The investigation included both underground and overhead utilities.

The investigations included the site of the various project components and adjacent areas to identify constraints on project alignments, location of components, and to identify concerns for safety and potential disruptions to utility service and pipeline operations. Relocation costs will be developed for those utilities and pipelines conflicting with the project, and which need to be relocated to accommodate the project.

Drainage facilities will require redesign and relocations to accommodate the project. These are addressed in Section L7 Civil Design and were not included in the relocation discussions.

Through-out the remainder of this section, the term “utility” is typically used to apply to all categories of infrastructure.

### L16.2 Investigation Process

The conceptual phase investigation and analysis effort was performed in three steps:

- Identify utilities and pipelines in and adjacent to the project area, and obtain contact information.
- Obtain facility data and record drawings from the owners or operators and determine if conflicts may exist.
- Request budgetary relocation estimates from the owners, as the project is in the conceptual and feasibility phases.
- In the next phases of the project, preliminary and final design, the following steps will be taken to better define the relocation needs and costs:
- Request that the utility/pipeline owner probe their underground facility, if appropriate, to determine accurate plan and profile locations
- Pot-hole critical locations, if necessary, to obtain more accurate location data
- Have the surveyor tie in the probe stakes and pot-holes

- Analyze component location and design, and refine relocation requirements
- Prepare exhibits of relocations and coordinate requirements with the utility/pipeline owner
- Obtain relocation opinion of estimated costs from the utility/pipeline owner

**Relocation Design and Reimbursement**

In most cases, the private utility owners will be fully responsible for the design and relocation of their facility. The State will enter into a contract with the private utility owners to have the relocations completed prior to start of project construction in the area.

Public utilities adjacent to public roadways are common civil construction items, and could be included in the construction contract.

The railroad relocations and reconstruction will likely be done by the owner through a reimbursement agreement with the State. Careful coordination would be required during construction, as the temporary railroad relocation and reconstruction and the project construction will be done in planned sequential phases.

**Data Collection**

Existing private utilities and pipelines within the vicinity of the restoration area, the diversion facility, and the transmission canal were identified through LA OneCall, record drawings, and limited field investigations. Using coordinates provided by CDM, LA OneCall alerted utility owners and operators within the project area. Owners of known utilities, including power, pipelines, and telecommunication were individually contacted in order to obtain specific details about their facilities. Each owner received a letter of transmittal from CDM, a USACE "Existing Utilities" questionnaire, and an aerial photograph of the project's boundaries. As-built drawings and alignment sheets were requested from each company along with the completed questionnaire.

To date, CDM has not received responses from all utility and pipeline owners, and the investigation efforts are still in progress. The utility research effort will continue through the next design phases and it can be anticipated that additional utilities will be found. This is due to, among other things, incomplete records, non-responsive owners, and facilities being sold multiple times.

As the utility data was collected, the information was tabulated and plotted in GIS. The figures primarily show major utility conflicts. Minor utilities in the roadway right of way are not shown, but are tabulated at the end of this section. **Figure L16.1-1** shows initial plotting of utility data for the project area. **Figure L16.1-2** shows a detail at the proposed control structure near the east end of the transmission canal. **Figure L16.1-3** is a detail of a potential culvert crossing and interconnecting channel at the KCS RR/Highway 61 corridor. At the end of this section, **Table L16.1-1** and **Table L16.1-2** lists the utilities identified to date.

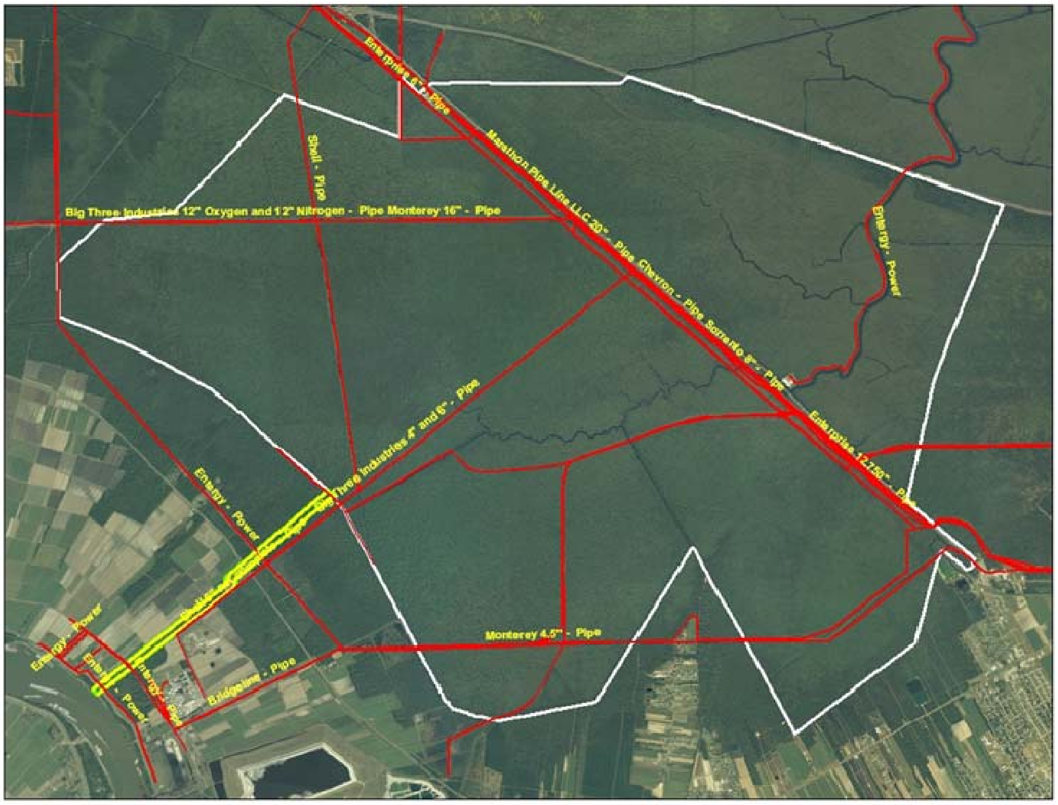



Figure L16.1-1Utilities and Pipelines in Project Area

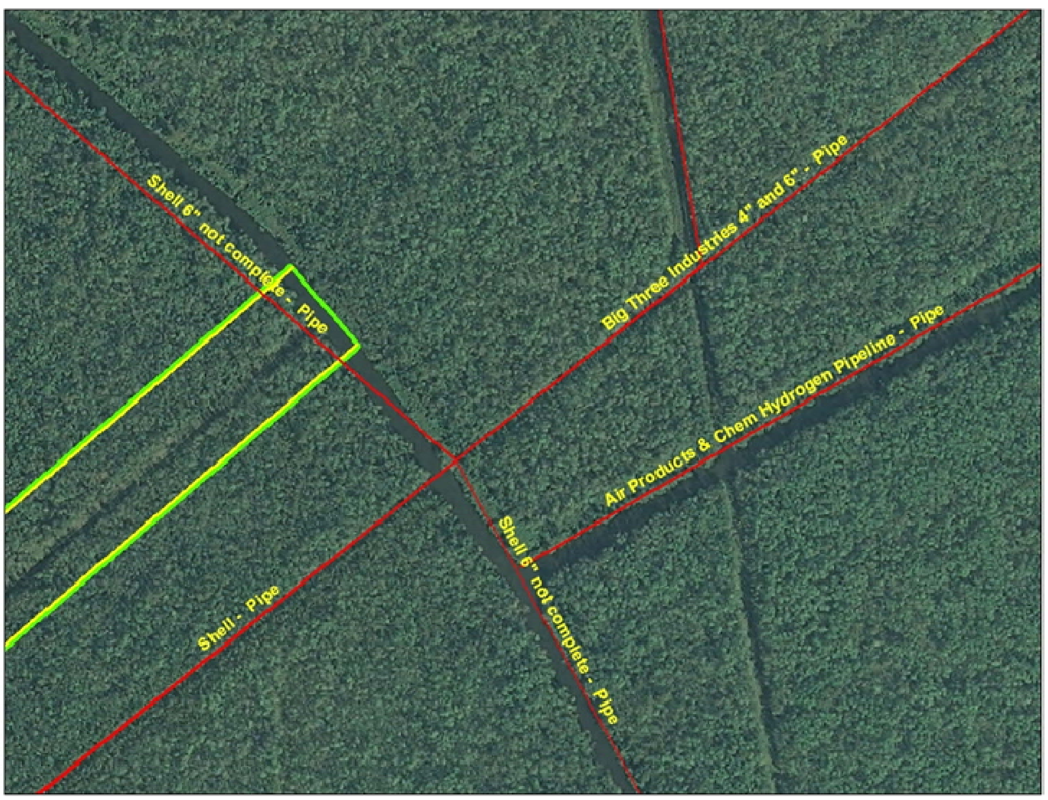



Figure L16.1-2 Pipelines at East End of Transmission Canal

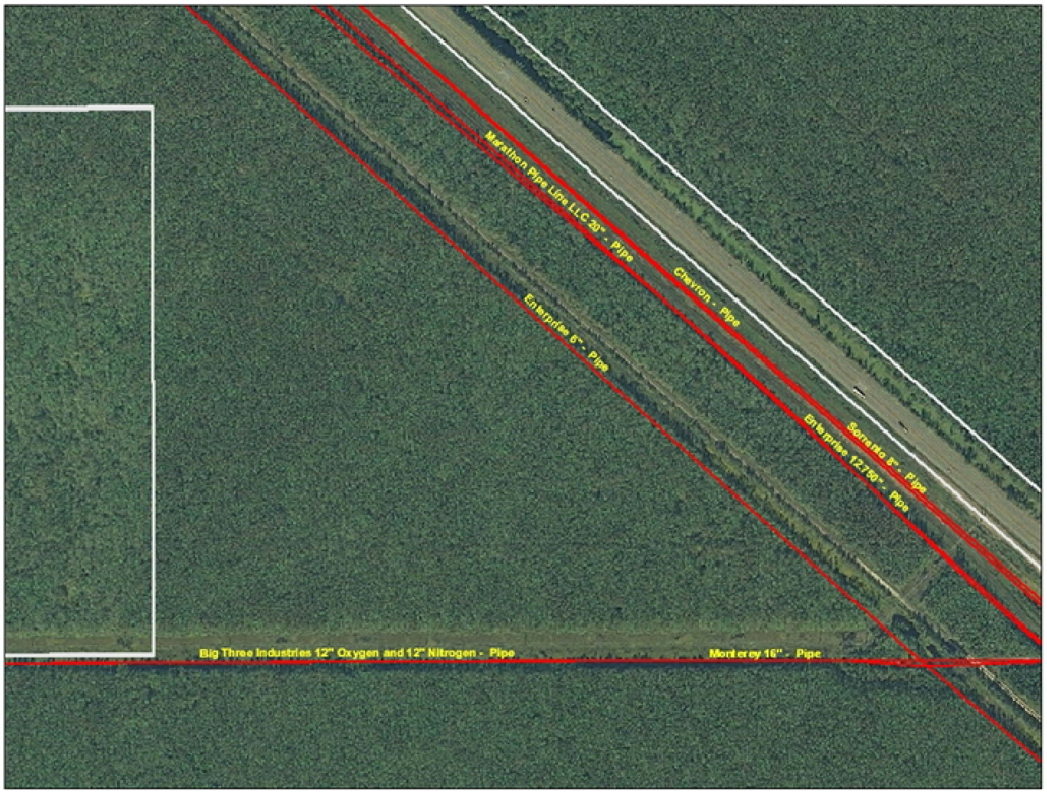



**Figure L16.1-3 Utilities and Pipelines at the KCS RR/Hwy 61 Corridor**

Note that there is conflicting information on owner names, line sizes, and product transported. These are expected to be resolved as additional data is collected and verified directly with the owners. No central facilities were found in or adjacent to the project area which would impact the project. The table uses shading to indicate utilities located, but not requiring adjustment or temporary relocation. The utility information will be plotted on the design drawings in the PED design phase.

## L16.3 Underground Utilities and Pipelines

This section describes the known underground utilities, addresses the clearance criteria, presents a conflict analysis, and defines proposed relocations for underground utilities (public and private) and pipelines. **Table L16.1-1** and **Table L16.1-2** also summarize the conflict and relocation recommendations.

### L16.3.1 Descriptions

**AT&T Distribution**

AT&T has two routes of telecommunication lines, both of which run across the proposed Romeville diversion alignment. Buried lines run parallel to LA 44 on the eastern side and the other line may run parallel to LA 3125. The lines are considered active; however, any further information concerning the type of line, clearance, size, and date installed has been withheld as proprietary.

**Air Liquide America**

Air Liquide operates two carbon steel pipelines that run parallel to the southeast side of the transmission canal from the Occidental Chemical (OxyChem) plant to the existing St. James Parish drainage channel at the Swamp. The 4-inch line carries pressurized nitrogen and the 6-inch line carries pressurized oxygen. The lines were installed in 1977 at a depth of approximately 5 feet. Design life is unknown. Both lines continue past the St. James Parish drainage channel in a northeasterly direction, intersecting Air Liquide's two 12-inch pipelines at US 61.

**Gulf South Pipeline Company, LP**

Gulf South Pipeline Company, LP operates a 24-inch, carbon steel pipeline that carries natural gas. The line was built in 1990 and has a design life of 100 years. Correspondence with the Gulf South Pipeline's Engineering Operations division confirms interference with the pipeline and the transfer canal between stations 63+00 and 64+00. The pipeline runs parallel to LA 3125. It was also noted that Gulf South owns an abandoned 18-inch pipeline that runs adjacent to the 24-inch line, approximately 20 – 30 feet towards the Mississippi River.

**Petrologistics Olefins, L.L.C.**

Petrologistics Olefins, LLC operates a 6-inch, steel pipeline that carries ethylene to and from the OxyChem plant. The line was built in 1980 with a design life 50 years, assuming adequate corrosion protection. The depth varies from 6.5 to 7.2 feet of cover. The pipeline will cross the transfer canal at three locations. The pipeline splits on the east side of the OxyChem plant. The line traveling away from the plant parallels the Canadian National Railroad and subsequently crosses the transfer canal. The pipeline exiting the OxyChem plant runs parallel to the diversion channel on the eastern side for approximately .5 miles before it encroaches 30 to 40 yards onto the proposed route of the transfer canal (90°49'21"W 30°4'27"N). It continues to travel along the route of the proposed channel, crossing LA 3125. The pipeline turns parallel to LA 3125 and travels WNW. It again intersects the proposed diversion channel at N. Dornier Road. Alignment sheets have been provided by Petrologistics' pipeline supervisor.

**Shell Pipeline Company, LP**

Shell Pipeline Company, LP owns and operates a 6-inch, steel pipeline that carries ethylene. The line was built in 1967 with a design life of 100 years. The depth varies between 5 to 6 feet of cover. Upon exiting OxyChem, the line runs parallel to the proposed location of the transfer canal. The line crosses perpendicularly at the St. James Parish canal and immediately turns northeast, where it follows another canal within the project boundaries. Shell Pipeline's Land and Permitting Department confirmed that the pipeline runs parallel to the diversion channel and does not cross.

**Texas Brine Company, L.L.C.**

Texas Brine Company, LLC operates a 14-inch carbon steel pipeline that carries saturated salt brine. The line was installed in 1980 and has a design life of 75 years. The average depth is 5 feet. An intersection point occurs approximately 700 feet southwest of the St. James Parish canal. Pipeline markers for Texas Brine were identified by the surveying team between stations 73+00 and 74+00, 190 feet towards the southern ROW boundary. Another marker was located at station 83+00. Exact direction of the line at this station is unclear.

**Williams Gas Pipeline (Transcontinental Pipeline Company)**

Williams Gas Pipeline operates a 12-inch, steel pipeline that carries natural gas. The line was installed in 1971 and has an indefinite design life. Depth of the pipeline varies between 10 to 15 feet. Field survey crews have located this pipeline between stations 63+00 and 64+00. It runs parallel to LA 3125 and approximately 50 feet south of the Gulf South Pipeline.

### L16.3.2 Conflict Analysis and Proposed Relocations

All underground utilities that conflict with the project will be permanently adjusted vertically under the project component, or re-routed to eliminate the conflict. For vertical adjustments, the USACE requires 10 feet vertical clearance below the constructed facility to the utility. All underground utilities crossing the project alignment for the diversion culvert and the transmission canal are less than 10 feet deep. The project facilities are deeper than 10 feet; therefore, all such crossing underground utilities will need to be adjusted vertically. The current plan is to do a vertical adjustment, and not relocate the utilities around the project.

**Diversion Culvert**

- Pipelines – none
- An underground AT&T telecommunications cable may be in the LA 44 right-of-way, and will need to be adjusted vertically under the culvert. As an alternate, the utility could be relocated to be over top of the box culvert, as there is sufficient space above the top of the box culvert.
- Parish water line may be in the LA 44 right-of-way and will need to be adjusted vertically under the culvert. As an alternate, the utility could be relocated to be over top of the box culvert, as there is sufficient space above the top of the box culvert.

**Transmission Canal**

- There are seven pipelines crossing the proposed transmission canal alignment, six of which will need to be adjusted vertically. The seventh line is apparently abandoned and can be removed from the proposed right-of-way.

- A pipeline is mapped in GIS as crossing the east end of the transmission canal on the west side of the existing perimeter drainage channel. It is likely that the pipeline is actually on the east side of the existing drainage channel. Therefore, there is no conflict.
- West of LA 3125, a Petrologistics pipeline encroaches into the southeast side of the proposed transmission canal right-of-way for a distance of 2,000 feet. The pipeline will have to be probed or pot-holed to define the actual extent of the encroachment. The general area is undeveloped and the transmission canal alignment could be adjusted instead of relocating the pipeline.
- Several pipelines are located on the southeast side of the proposed right-of-way, but outside of the proposed taking. Therefore, there are no conflicts.
- An underground fiber optics cable is in the Canadian National (CN) RR right-of-way, which conflicts with construction.
- An underground AT&T telecommunications cable may be in the LA 3125 right-of-way, and will need to be adjusted vertically under the culvert. As an alternate, the utility could be relocated to be over top of the box culvert, as there is sufficient space above the top of the box culvert.
- Parish water line may be in the LA 3125 right-of-way and will need to be adjusted vertically under the culvert. As an alternate, the utility could be relocated to be over top of the box culvert, as there is sufficient space above the top of the box culvert.

**Control Structure Nos. 1-6A and 1-6B (near the transmission canal outfall)**

- Up to five pipelines may be in the area of the control structures, as shown on Figure **L16.1-2**. Of these, two are likely far enough from the sites to not conflict with the project. These need to be identified for construction. The other pipelines are in easements adjacent to the existing drainage channels, and likely close to the proposed control structures. The proposed structure can probably be designed to avoid conflicts, thereby avoiding relocations.
- There are no other underground utilities at the area

**Control Structure Nos. 1-8A and 1-8B2 (near Grand Point)**

There are no known underground utilities or pipelines near the control structure site, as shown on **Figure L16.1-1**.

**Control Structure No. 1-7 (near Highway 61)**

Utility investigations are incomplete in the area. There are underground utilities and pipelines in the general area, but the control structure site has not yet been analyzed. The location is flexible and will be sited to avoid utilities.

**Cross Culverts along the Highway 61 Corridor**

There are four locations proposed for cross culverts. See Figure L16.1-3 for one location, typical of all locations.

- There may be six pipelines in the corridor between the KCS RR and Highway 61. All of these are outside of the highway right of way and are not expected to need adjustment.
- No underground telecommunication lines have been identified, but may be found after site investigations.

## L16.4 Overhead Utilities

This section addresses the clearance criteria, conflict analysis, and proposed relocations for underground utilities (public and private) and pipelines.

### L16.4.1 O/H Utility Descriptions

**AT&T Distribution**

AT&T has two routes of telecommunication lines. Both lines run across the proposed Romeville diversion channel. A pair of aerial and buried lines runs parallel to LA-44 on the eastern side. The other line is aerial and runs parallel to LA-3125. The lines are considered active; however, any further information concerning the type of line, clearance, size, and date installed has been withheld as proprietary. A cost analysis for the removal and relocation of the buried telecommunication line will be performed upon receiving more information.

**Entergy Louisiana, LLC**

Entergy Louisiana, LLC owns and operates aerial power transmission and distribution lines that cross the proposed diversion channel. One distribution line runs parallel to LA 44; the other runs parallel to the Canadian National Railroad. Aside from drawings marking the route of the distribution lines, no other information was received by CDM. Entergy also owns and operates a 230 kilo-Volt transmission line, which provides service to various substations and interconnects various generating plants. The line runs parallel to LA 3125, on the northeastern side. It was installed between 1966 and 1967 and was designed to last sixty years. The distribution lines will have to be relocated during construction in order to provide a safe working space for the machinery, equipment, and personnel needed to construct culverts at each location. Steel towers spaced 400 – 500 feet apart support the transmission line. Construction of the diversion channel should not interfere with the towers. The only precaution taken will be a survey of the vertical clearance at the midpoint between the two towers. Heavy loading during summer and winter months may be of concern during excavation. Entergy will advise the maximum available vertical clearance of the power lines before construction of this phase. Entergy is currently performing cost estimations for the temporary relocation of the two distribution lines.

### L16.4.2 Conflict Analysis and Proposed Relocations

**Diversion Culvert**

At LA 44, the poles for the overhead power and telecommunication lines conflict with the project, and will have to be relocated.

**Transmission Canal**

- At the CN RR, there may be an overhead power line.
- At LA 3125, overhead lines include power, telecommunications, and CATV. The poles conflict with the project and will need to be relocated.
- Approximately 800 feet east of LA 3125, an electric transmission lines crosses the proposed alignment. The towers are spaced at approximately 500 feet and one tower conflicts with the transmission canal. It is most likely that the single tower will be replaced with two towers that will span across the canal.

**Control Structures**

- There are no known overhead utilities at the two control structures near the transmission canal and near Grand Point.
- The area has not been investigated for the control structure near Highway 61.

**Cross Culverts at the Highway 61 Corridor**

- There is an electric transmission line in the corridor. The culverts will be located to avoid conflicts with the poles/towers.

## L16.5 Transportation Facilities

The culverts at the roads and railroad are large hydraulic structures, and will have to be constructed by open-cut techniques. Three existing transportation routes will be temporarily relocated during the construction phase of this project - LA 44, LA 3125, and the Canadian National Railroad.

### L16.5.1 Transportation Descriptions

LA 44 is a two-lane asphalt road (one lane each direction) in an 80-foot wide right-of-way. Drainage is by open ditch.

LA 3125 is a two-lane asphalt road (one lane each direction) in an 300-foot wide right-of-way. Drainage is by open ditch.

The CN RR has a single track in a 100-foot wide right-of-way.

### L16.5.2 Conflicts and Proposed Temporary Relocations

The temporary relocations will be designed to the standards of the UMTCD.

**LA 44**

The diversion culvert will be constructed across LA 44 in an open-cut excavation.

This requires the existing northbound and southbound lanes of LA 44 to be removed for construction. Re-routing traffic from LA 44 to LA 3125 is an unacceptable detour route. Therefore, local traffic will be maintained through the construction area at all times by providing a temporary detour road. The detour road will be an asphalt road approximately 1,000 feet long with 2 – 12' lanes (one each direction).

**LA 3125**

A detour will be required for construction of the LA 3125 crossing, due to the unacceptable length of potential detour routes if the road were closed. The detour was not designed in the conceptual phase, but was considered to be 1,000 feet long, consisting of a 2-lane asphalt road (1 – 12' lane each direction) with shoulders. LA 3125 has a wide right-of-way, and the work should be done in one detour phase.

The following sequence will be required for each highway detour:

- Construct the temporary detour road
- Remove the existing pavement at the culvert crossing
- Construct the culvert
- Reconstruct highway in the existing location
- Remove the temporary detour road

**Canadian National Railroad (CN RR)**

The CN RR is an active rail line serving the mid-west and the south Louisiana port area. Based on informal discussions with railroad representatives, rail service must be maintained at all times, and speed reductions will likely not be allowed. The design assumes a 2,000-foot long shoofly (temporary track) to allow open-cut construction of the box culverts across the existing CN RR tracks. The shoofly will include turnouts at each end to expedite final switch-over to the shoofly. The railroad will likely discourage a temporary rail crossing at the work site. Such a crossing is not necessary, as access is available to both sides of the railroad from the existing public roads.

The proposed construction sequence at the CN RR is as follows:

- Construct the shoofly and re-route rail traffic
- Remove the existing tracks at the culvert crossing
- Construct the culvert
- Reconstruct the railroad tracks in the existing location and place rail traffic back on the original alignment
- Remove the shoofly track

LA 44, LA 3125, and the CN RR are to remain open to traffic at all times, except for brief periods to switch over traffic, as allowed by the permits.

## L16.6 Construction Coordination

### Underground Utilities

Each underground utility needs to be relocated prior to start of construction, to avoid scheduling conflicts and delays with the project's construction contractor. Each utility owner should be notified by the construction contractor prior to start of construction in the vicinity of the utility.

### Overhead Utilities

Temporary relocations will occur prior to construction and relocate after construction is complete.  No extraordinary conditions exist for temporary relocations.  The one transmission tower will need to be removed necessitating the addition of two transmission towers on the line north of LA3125.

### Transportation Facilities

Detours will be provided ahead of construction and permanent roads and railroads replaced upon completion.  The detours and relocations are all considered not to be extraordinary.

## L16.7 Opinion of estimated costs

Opinion of estimated costs for utility relocations are included in the MCACES cost estimate for the project.  During preliminary design when construction drawings become available additional cost estimates will be obtained from the utility owners.

## L16.8 Summary Tables

The following tables summarize the utilities located and contacted.  The items which are shaded do not conflict with the project and will not be relocated.  The remaining are part of the relocation opinion of estimated cost.

## Table L16.1-1
## Utility and Pipeline Relocations

*Shaded utilities will not be relocated.*

| Item No. | Owner | Utility/Pipeline Description | Location | Relocation Description |
|---|---|---|---|---|
| **Diversion Alignment - Diversion Culvert and Transmission Canal** | | | | |
| ***Utilities*** | | | | |
| 1 | Entergy Louisiana, LLC | Electric Distribution - O/H | LA 44 ROW - Sta. 5+56 | Temporary relocation |
| 2 | AT&T | Telecommunication Cable - O/H | LA 44 ROW - Sta. 5+56 | Temporary relocation |
| 3 | ??? | CATV - O/H | LA 44 ROW - Sta. 5+56 | Temporary relocation |
| 4 | AT&T - (verify exists??) | Telecommunication Cable - U/G | LA 44 ROW | Permanent - adjust below canal |
| 5 | St. James Parish | Water Line | LA 44 ROW | Permanent - adjust below canal |
| 6 | Entergy Louisiana, LLC | Electric Distribution - O/H | CN RR - Sta. 20+75 | Temporary relocation |
| 7 | AT&T | Fiber Optics - U/G | CN RR - Sta. 21+40 | Permanent - adjust below canal |
| 8 | Entergy Louisiana, LLC | Electric Distribution - Overhead | LA 3125 ROW - Sta. 99+75 | Temporary relocation |
| 9 | AT&T | Telecommunication cable - overhead | LA 3125 ROW - Sta. 99+75 | Temporary relocation |
| 10 | ??? | CATV | LA 3125 ROW - Sta. 99+75 | Temporary relocation |
| 11 | AT&T - (verify exists??) | Telecommunication Cable - U/G | LA 3125 ROW | Permanent - adjust below canal |
| 12 | St. James Parish | Water Line | LA 3125 ROW | Permanent - adjust below canal |
| | | | | |
| ***Pipelines*** | | | | |
| 13 | Petroligistics Olefins, LLC | Pipeline - 6" Steel - Ethylene | Near CN RR ROW | Permanent - adjust below canal |
| 14 | Petroligistics Olefins, LLC | Pipeline - 6" Steel - Ethylene | Sta. 62+00 to 82+00 | Permanent - remove laterally from ROW |
| 15 | Williams Gas Pipeline | Pipeline - 12" Steel - Natural Gas | Sta. 63+25 | Permanent - adjust below canal |
| 16 | Gulf South Pipeline Co., LP | Pipeline - 18" Steel (Abandoned) | Sta. 63+50 | Remove |
| 17 | Gulf South Pipeline Co., LP | Pipeline - 24" Steel - Natural Gas | Sta. 63+75 | Permanent - adjust below canal |
| 18 | Texas Brine Co., LLP | Pipeline - 14" Steel - Brine | Sta. 73+00 | Permanent - adjust below canal |
| 19 | Texas Brine Co., LLP | Pipeline - 14" (??) Steel - Brine | Sta. 83+00 | Permanent - adjust below canal |
| 20 | Petroligistics Olefins, LLC | Pipeline - 6" Steel - Ethylene | 200' east of LA 3125 | Permanent - adjust below canal |
| | | | | |
| **Control Structure 1-6A** | | | | |
| Locate structure to avoid conflicts | | | | |

**Table L16.1-2**
**Utility and Pipeline Relocations**

| Item No. | Owner | Utility/Pipeline Description | Location | Relocation Description |
|---|---|---|---|---|
| | | | | |
| **Control Structure 1-6B** | | | | |
| Locate structure to avoid conflicts | | | | |
| | | | | |
| **Control Structure 1-7** | | | | |
| Locate structure to avoid conflicts | | | | |
| | | | | |
| **Control Structure 1-8A** | | | | |
| No utilities or pipelines known to be in immediate area | | | | |
| | | | | |
| **Control Structure 1-8B** | | | | |
| No utilities or pipelines known to be in immediate area | | | | |
| | | | | |
| **Control Structure 3-2** | | | | |
| No utilities or pipelines known to be in immediate area | | | | |
| | | | | |
| **Cross Culverts at Hwy 61** | | | | |
| | **(4 locations - each the same)** | | | |
| 21 | Marathon Pipeline, LLC | Pipeline - 20" - unknown product | KCS RR and Hwy 61 | No adjustment anticipated |
| 22 | Air Liquide America | Pipeline - 12" Steel - Nitrogen | KCS RR and Hwy 61 | No adjustment anticipated |
| 23 | Air Liquide America | Pipeline - 12" Steel - Oxygen | KCS RR and Hwy 61 | No adjustment anticipated |
| 24 | Sorrento | Pipeline - 8" - Unknown product | KCS RR and Hwy 61 | No adjustment anticipated |
| 25 | Montery | Pipeline - 16" - Unknown product | KCS RR and Hwy 61 | No adjustment anticipated |
| 26 | Chevron | Pipeline - Unknown size and product | KCS RR and Hwy 61 | No adjustment anticipated |

**Notes:**
1. Stationing is from the Romeville Transmission Canal survey.
2. O/H = Overhead
3. U/G = Underground
4. CN RR = Canadian National Railroad
5. KCS RR = Kansas City Southern Railroad