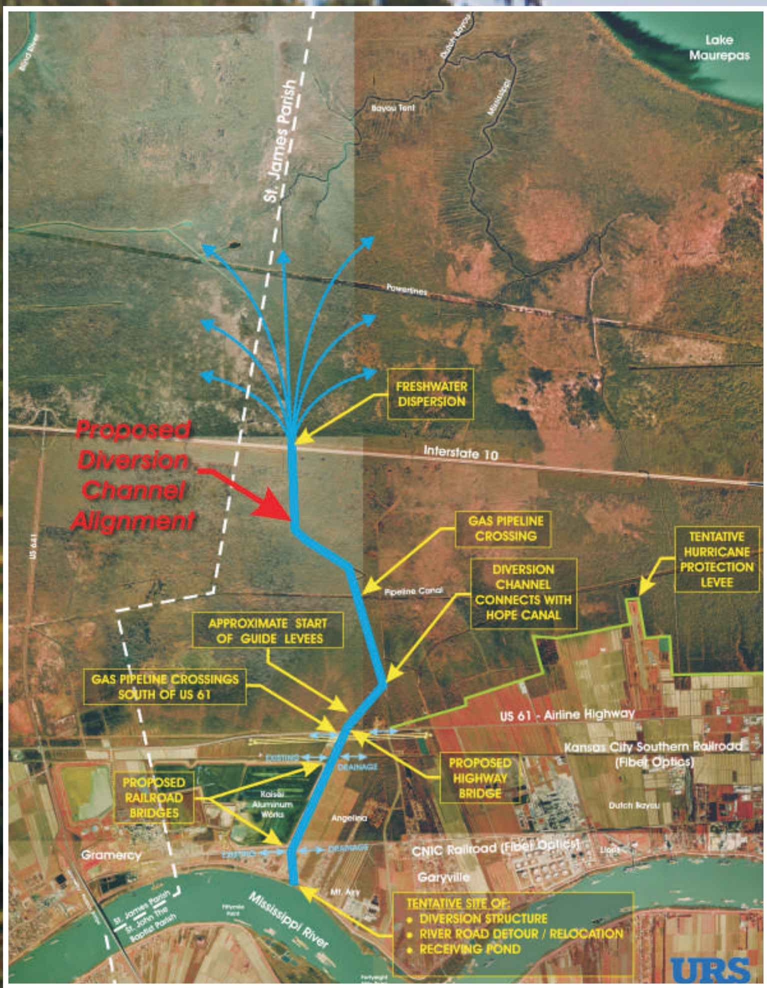
95% DESIGN REPORT

Mississippi River Diversion into Maurepas Swamp

(PO-29 CONTRACT NO. 2503-11-63)

Prepared for:
State of Louisiana
Coastal Protection & Research Authority

Project Federal Sponsor:
US Environmental Protection Agency



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ABBREVIATIONS KEY

ACI	American Concrete Institute
ADCIRC	**A**dvance Circulation Model
AREMA	American Railway Engineering & Maintenance-of-Way Association Manual
ASTM	American Society of Testing and Materials
ATS	Automatic Transfer Switch
CFS	Cubic Feet per Second
CN RR	Canadian National Railroad
CPT	Cone Penetration Test
CR	Contraction Rates
CWPPRA	Coastal Wetlands Planning Protection and Restoration Act
DO	Dissolved Oxygen
E-G	Evans-Graves
EPA	Environmental Protection Agency
ER	Expansion Rates
FPS	Feet per Second
FHWA	Federal Highway Administration
HEC-RAS	Hydrologic Engineering Center – River Analysis System
HI	Hydraulic Institute
HVAC	Heating Ventilating and Air Conditioning
I-10	Interstate 10
ISA	Instrument Society of America
KCS RR	Kansas City Southern Railroad
LA 44	Louisiana Highway 44, River Road
LA 54	Louisiana Highway 54
LDOTD	Louisiana Department of Transportation and Development
LCP	Lighting Control Panel

LDNR	Louisiana Department of Natural Resources
MRL	Mississippi River Levee
MSL	Mean Sea Level
NAVD	North American Vertical Datum
RCB	Reinforced Concrete Box
ROW	Right-of-Way
P&ID	Process and Instrumentation Diagram
POC	Point of Contact
PPC	Prestressed Precast Concrete
TSS	Total Suspended Sediment
US 61	United States Highway 61, also referred to as Airline Highway
USACE	United States Army Corps of Engineers
USGS	United States Geological Survey
WS PF 7	Water Surface Profile 7

EXECUTIVE SUMMARY

Introduction

The preface to ***Louisiana's Comprehensive Master Plan for a Sustainable Coast***, published by the State of Louisiana in May of 2012 states:

> *"Louisiana is in the midst of a land loss crisis that has claimed 1,880 square miles of land since the 1930s. Given the importance of so many of south Louisiana's assets – our waterways, natural resources, unique culture, and wetlands – this land loss crisis is nothing short of a national emergency.*
>
> *If we do not address this crisis, the problem intensifies. Our analysis confirmed that if we do nothing more than what has been done to date, we have the potential to lose up to an additional 1,750 square miles of land. This land loss will increase flooding risk, with disastrous effects. Put simply: the status quo cannot be maintained, and we must take bold action now to save our coast. At the same time, our analysis demonstrated that we do have the opportunity, if we continue to build upon current success, to avert an otherwise bleak future".*

The following quote, from the Executive Summary of the Coastal Wetland Forest Conservation and Use Science Working Group's 2005 Report to the Governor, ***Conservation, Protection and Utilization of Louisiana's Coastal Wetland Forests***, probably best describes the value of the Louisiana coastal forest resources, and the threats they face:

> *"Louisiana's coastal wetland forests are of tremendous economic, ecological, cultural, and recreational value to residents of Louisiana, the people of the United States, and the world. Although some two million acres of forested wetland occur throughout Louisiana, over half are in the coastal parishes. Large-scale and localized alterations of processes affecting coastal wetlands have caused the complete loss of some coastal wetland forests and reduced the productivity and vigor of remaining areas. This loss and degradation threatens ecosystem functions and the services they provide."*

As shown on Figure ES-1, the Maurepas Swamps are located in St. John the Baptist Parish, north of the Mississippi River and southeast of Lake Maurepas. Situated between the southern shore of Lake Maurepas and the developed uplands of the Mississippi River natural levee, the Maurepas Swamps represent a critical part of the coastal wetland forests described by the above passage. In addition, the Maurepas Swamps exemplify the *large-scale and localized alterations of processes* the Coastal Wetland Forest Conservation and Use Science Working Group (2005) discusses in their report. Another quote from the report precisely describes the fundamental problem the Maurepas Swamps face, as well as the general solution to that problem:

> *"…Louisiana's coastal wetland forests are sediment and nutrient deprived as a result of the Mississippi River levee system and are experiencing significant habitat loss. Under these conditions, the addition of nutrients and sediments is the only way for these ecosystems to maintain their surface elevation relative to sea-level rise."*

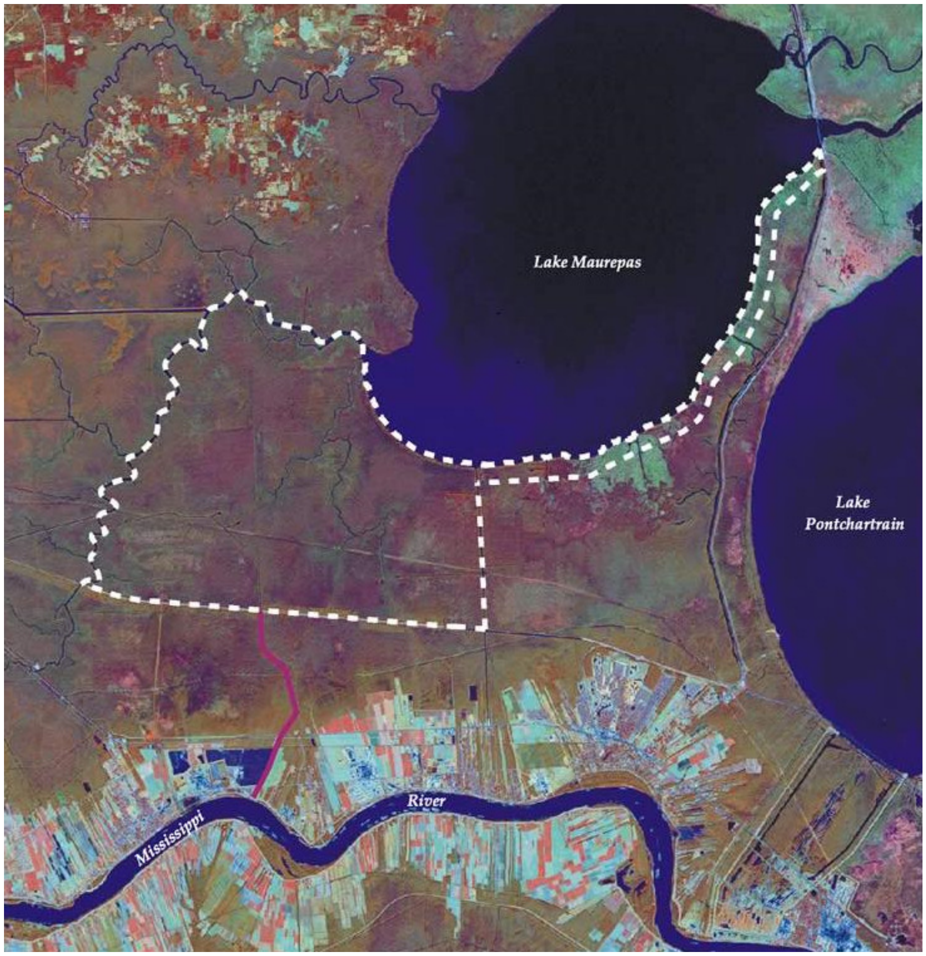


Figure: ES 1: Impacted Area of Maurepas Swamps

In addition to the problems caused by large-scale alterations of processes, the report cites the importance of localized alterations of processes. Several of these local-scale factors are major concerns in the Maurepas Swamps:

> *"The cumulative effects of small-scale or local factors can be of equal or greater importance in coastal wetland forest loss and degradation than large-scale alterations. These factors include increased depth and duration of flooding, saltwater intrusion, nutrient and sediment deprivation, herbivory, invasive species, and direct loss due to conversion. Causal agents include highways, railroads, channelization, navigation canals, oil and gas exploration canals, flood control structures, conversion of forests to urban and agricultural land, and non-sustainable forest practices."*

And finally, the Coastal Wetland Forest Conservation and Use Science Working Group (2005) points to the ramifications if appropriate restoration of Louisiana's coastal wetland forests is not undertaken. These apply to the Maurepas Swamps as well as all of Louisiana's coastal forests:

> *"Without appropriate human intervention to alleviate the factors causing degradation, most of coastal Louisiana will inevitably experience the loss of*

coastal wetland forest functions and ecosystem services through conversion to open water, marsh, or other land uses."

A great deal has been learned about the causes of coastal wetlands forest loss in Louisiana over the last several decades. One of the major factors was found to be the containment of the Mississippi River within the man-made levee system, which prevents freshwater, sediments, and nutrients from reaching these natural ecosystems. A secondary cause is the periodic introduction of brackish water into the swamps from the adjacent lakes. The goal of the subject project is to restore the health of the Maurepas Swamp by supplying freshwater, sediment and nutrients, simulating the periodic flooding of the Mississippi River which initially created the wetlands.

This report and the accompanying Plans and Specifications present the 95% level design of the proposed project to divert freshwater from the Mississippi River into the Maurepas Swamp near Garyville, Louisiana. This 95% Submittal is a deliverable from Task 3 of the subject project, *Mississippi River Reintroduction into Maurepas Swamp*, Project PO-29, Contract No. 2503-11-63. The work has been conducted under the auspices of the Coastal Protection and Restoration Authority (CPRA) and the U.S. Environmental Protection Agency (EPA). The initial project work was funded by the federal Coastal Wetlands Planning, Protection and Restoration Act (CWPPRA).

Prior Studies

Pre-"Phase 0"

Several key studies identified the need for restoration of the Maurepas Swamps, as well as the importance of Mississippi River reintroduction as the appropriate restoration approach, prior to the beginning of this project (Lee Wilson & Assoc. 2001):

- The Louisiana Coastal Restoration Plan (1993)
- The Louisiana Coast 2050 Report (Louisiana Coastal Wetlands Conservation and Restoration Task Force and the Wetlands Conservation and Restoration Authority 1999)
- The Mississippi River Sediment, Nutrient and Freshwater Diversion Study (U.S. Army Corps of Engineers 1999)

The Coast 2050 Report concluded that subsidence, permanent flooding, and sediment and nutrient starvation are significant factors contributing to the stress and predicted loss of the south Maurepas swamps (Lee Wilson & Assoc. 2001). The primary regional strategy recommended by Coast 2050 was consideration of relatively small (about 2000 cfs) Mississippi River diversions at Convent (into the Blind River) and Reserve Relief Canal (directly into the swamps).

In addition, while certainly not "Pre-Phase 0", another very important, and recent, Louisiana coastal wetland restoration planning effort- the ***Louisiana Coastal Area (LCA), Louisiana-Ecosystem Restoration Study*** (U.S. Army Corps of Engineers 2004) - identified the Maurepas Swamp as in need of restoration via reintroduction of Mississippi River water, in this case explicitly via a small reintroduction at Hope Canal, the same location proposed in this project.

CWPPRA Complex Projects / Phase 0

In 2000, the Coastal Wetlands Planning, Protection and Restoration (CWPPRA) Task Force decided that, in addition to the program's "normal" Priority Project List (PPL) development, the program would undertake a few "complex projects"- that is, projects that were deemed more complicated than the typical CWPPRA project, requiring additional funds and time for preliminary study. One of those projects was the *Diversion into the Maurepas Swamps*. The Task Force approved substantial funding for "Phase 0" preliminary study for this project, and EPA was selected to be the lead Federal agency on this work. EPA subsequently convened an advisory group, and contracted with Lee Wilson & Associates to assist with the study. Lee Wilson & Associates in turn, sub-contracted with several Louisiana researchers at LSU and SELU (Shaffer, Day, Kemp, Mashrique, Lane, etc.) knowledgeable about the Maurepas Swamps, and the related issues.

This study resulted in the report, ***Diversion into the Maurepas Swamps*** (Lee Wilson & Assoc. 2001), a key report that laid the foundation for the project. The work was a reconnaissance-level effort to develop a potentially viable project, including preliminary site reviews, hydrologic modeling, ecological field studies, and field surveys. The study evaluated basic alternatives of diversion location and size, estimated costs and environmental benefits, and conducted preliminary studies of swamp vegetation, soils, water quality, and hydrology. The advisory group nominated the diversion of Mississippi River water into the degraded swamp south of Lake Maurepas for Priority List 9 of the CWPPRA program. The project size was based on the need to fit the diversion into the existing channel beneath I-10, which restricted the discharge to between 1,500 and 2,000 cfs.

The diversion was predicted to greatly increase flow through the area, providing oxygen and nutrient-rich waters to the swamp. Benefits were expected to include accretion of sediment and increased vegetation growth. The study showed that the Maurepas swamps are almost certainly nutrient limited and thus the nutrients added from the diverted river water would be almost completely absorbed within the swamp prior to discharge into Lake Maurepas. Saltwater intrusion was shown to be further harming the already stressed vegetation within Maurepas Swamp. The proposed diversion was expected to directly reduce the salinity in the swamp as well as the lake. Rivers and bayous entering the lake would thus garner freshwater benefits from the proposed diversion as well.

Based on the results of the Phase 0 study, EPA proposed Phase I funding to the CWPPRA Task Force in 2001, which was granted. This initiated the work in "Phase 1 Engineering and Design".

CWPPRA Phase 1 Engineering and Design

LSU and SELU Studies

After receiving approval for Phase 1, EPA ensured that the momentum and project team expertise developed under the Phase 0 work was not lost before it could be transferred to the Phase 1 Engineering and Design Team. This was accomplished by funding two important studies:

- Ecosystem Health of the Maurepas Swamp: Feasibility and Projected Benefits of a Freshwater Diversion (Shaffer et al. 2003)
- Development Plan for a Diversion into the Maurepas Swamp (Day et al. 2004)

The former study, completed by SELU in 2003, continued basic swamp vegetation and soils work begun by Shaffer et al. (2001). It evaluated the condition of the Maurepas Swamp and assessed the potential benefits of a freshwater diversion. It addressed the rate of local wetland subsidence and the factors causing the decline of the native vegetation. The study showed that salt stress was killing the trees near Lake Maurepas, while standing water and nutrient deprivation were stressing the interior of the swamp.

The study found that fresh water throughput from the proposed diversion would decrease the detrimental effects of salinity throughout the swamp. The diversion was also predicted to increase the sediment load and nutrient supply to the wetlands. Hydrologic modeling showed that most of the diversion water is likely to sheet flow through the interior swamps. The result was an expected increase in the regeneration of several wetland forest species.

Nutrient augmentation was shown to enhance the growth of vegetation by up to 300%. Swamps as nutrient poor, stagnant, and impounded as the interior Maurepas Swamp were predicted to double their rates of production with an infusion of water from the Mississippi River. Such enhanced productivity was deemed essential to wetland restoration.

The latter study, completed by LSU in 2004, continued basic hydrologic and water quality monitoring and analysis. The study greatly expanded on previous hydrodynamic modeling begun by Day et al (2001) and Kemp et al. (2001), including the acquisition of hydrographic information for calibration of the hydrodynamic models constructed by LSU and URS. The hydrodynamic modeling predicted water levels to rise by less than 0.25-ft with diversions ranging from 500 to 2,500 cfs. Flow velocities in the swamp for all diversion flow rates were predicted to be less than 0.3 fps. The model showed that a 2,500 cfs diversion would reduce Lake Maurepas salinity by 30% after only one month, an important benefit to a swamp forest severely stressed by saltwater intrusion.

The study also established a representative baseline of pre-diversion water quality parameters, which were used to construct, calibrate, and validate water quality models. Monthly water samples, taken from April 2002 to May 2003 throughout the study area, were analyzed for suspended sediment, nitrogen, phosphorous, silicate, chlorophyll, and salinity. Nitrogen-to-phosphorous ratios from Maurepas were generally found to be very low, indicating that the Maurepas basin is nearly always nitrogen limited. Introduction of inorganic nitrogen to such a nitrogen-limited ecosystem was predicted to support increased plant production.

The work also introduced new ecological modeling of swamp vegetation along with techniques for predicting long-term ecological benefits and impacts of a Maurepas diversion on the swamp and lake. The ecological model predicted that between 2,000 and 4,000 hectares of the Maurepas Swamp could be restored within 50 years if diversions greater than 1,000 cfs were initiated.

In addition to helping to ensure project momentum and knowledge transfer during a critical transition, these studies contributed to EPA's knowledge base for an Environmental Impact Statement for the project.

NEPA Studies

Shortly after the beginning of the Phase 1 Engineering and Design Work, EPA began funding several studies explicitly to support the development of an Environmental Impact Statement:

- Phase 1 Assessment of Potential Water Quality and Ecological Risk and Benefits From a Proposed Reintroduction of Mississippi River Water into the Maurepas Swamp (Battelle 2005)
- EIS Risk Assessment-Identification of Potential Hazardous Waste Sites (TechLaw, Inc. 2006)
- Impacts of a Freshwater Diversion on Wildlife and Fishes in the Maurepas Swamp (Fox et al. 2007)
- Limited Phase II Assessment of Ecological Risks of Contaminants from a Proposed Reintroduction of Mississippi River Water into Maurepas Swamp (Battelle 2007)
- Cultural Resources Survey of the River Reintroduction Corridor, Maurepas Swamp (PO-29), St. John the Baptist Parish, Louisiana (Coastal Environments Inc. 2007)
- Evaluation of Potential Impacts of the Lake Maurepas Diversion Project to Gulf and Pallid Sturgeon (Kirk et al. 2008)

For the most part, these studies either supported general project design concepts, or lead to specific project design criteria (e.g. Kirk et al. 2008).

URS Engineering and Design

During the period August-December, 2002 LDNR and URS negotiated the scope of work, cost estimate, and schedule for URS's efforts on engineering and design of the Mississippi River Diversion into Maurepas Swamp (PO-29). The scope of work was organized around three tasks: 1) Task 1 - Hydraulic Feasibility Study; 2) Task 2 - Preliminary Engineering; and 3) Task 3 - Final Design.

Task 1 - Hydraulic Feasibility Study

Task 1 was an extensive data collection and analysis effort by URS to address the hydraulic feasibility of the Maurepas diversion. The work involved the collection of detailed topographic, bathymetric, and hydrologic survey data, which was used to create hydrodynamic models of the proposed diversion. The modeling showed that the reintroduction of the Mississippi River into the Maurepas Swamp via the Hope Canal was technically feasible.

The results indicated that flow distribution throughout the swamp could be improved by including outfall management features in combination with pulsing of the diversion flow. Flow pulsing was also demonstrated to extend the swamp retention time and reduce short-circuiting to the lake, thus enhancing sediment deposition. A pump station was shown to be capable of conveying the gravity flow from the Hope Canal watershed, thus mitigating the impacts of the diversion on the Garyville/Reserve drainage system. Diversion velocities at I-10 were found to be in a moderate range, which can be readily addressed to prevent scouring.

Eight specific recommendations, including refinements and additions to the Phase 0 Report, were issued:

1. Gapping the railroad embankment, flow control devices underneath I-10, and flow restrictions at the Bourgeois Canal are required to improve circulation and increase swamp retention time.
2. A diversion design flow of at least 2,000 cfs and controls to respond to Lake Maurepas water surface elevations are required.
3. Flow control features are needed for the culverts under I-10 between LA 641 and Mississippi Bayou.
4. Devices to provide occasional limited flow from the diversion channel into the swamp south of I-10 are required.
5. The gravity drainage system for the Hope Canal watershed must be replaced by a pump station of adequate capacity.
6. Increased drainage or pumping capacity for the eastern Garyville/Reserve drainage system will be needed.
7. Additional armoring of the diversion channel at the I-10 overpass above Hope Canal is needed where velocities exceed 2 fps.
8. The sand/silt settling basin must be designed to prevent sediment deposition that would adversely affect circulation.

Task 2 - Preliminary Engineering (30% Design)

The Project Team has implemented the majority of the recommendations from Task 1 into the Task 2 design:

1. The intake structure has been designed to flow at 2,000 cfs for approximately half of the year, with gates that enable flow control.
2. The design includes one-way check valves on all culverts underneath I-10 from LA 641 to the Mississippi Bayou overpass.
3. Culverts with control valves have been designed to divert 125 cfs to each side of the conveyance channel between US 61 and I-10.
4. A pump station has been designed to pump the local drainage flow from the Bourgeois and Hope Canals over the proposed guide levees.
5. A large sedimentation basin has been designed to remove the sand entrained in the diverted flow-stream.

<u>Project Description</u>

The Maurepas Diversion Project consists of the following major components, designed to divert freshwater from the Mississippi River into the Maurepas Swamp: 1) a gated river intake structure, 2) box culverts through the levee, 3) a sedimentation basin, 4) a conveyance channel, and 5) a drainage pump station. The LDNR and EPA have requested that a design flow of 2,000 cfs be obtained for as much of the year as possible. Based on Mississippi River stage data, the intake structure and conveyance channel were designed to convey this flow for approximately six months each year.

The project intake will be located on the river side of the Mississippi River levee near Garyville, LA in St. John the Baptist Parish. A 200-ft long by 60-ft wide inflow channel will be constructed in the batture area between the Mississippi River and the levee. The channel will connect to a gated intake structure about 100-ft from the levee crown. The intake structure will be comprised of three 10-ft x 10-ft sluice gates, which will be hydraulically actuated to control the flow of water into the diversion. The gates will connect to three 10-ft x 10-ft box culverts that travel through the levee and underneath LA 44.

Beyond the roadway, the culverts transition from a concrete u-channel into a large sedimentation basin. The basin is 265-ft long with a 60-ft wide flat bottom and sloped sides extending another 66-ft on each side. It is designed to remove sand from the flow-stream and prevent clogging of the conveyance channel. The basin has sufficient volume to store six months of sediment prior to cleaning without impacting the system's hydraulic performance. The sedimentation basin discharges over an outflow weir into the conveyance channel.

The proposed conveyance channel extends just under 5½ miles from the sediment basin at LA 44 to a discharge point in the Maurepas Swamp approximately 1,000-ft north of I-10. The channel will have a typical bottom width of 40-ft and will be bounded on both sides by guide levees. The levee side slopes will be 3:1 and 5:1 (horizontal:vertical) for the sections south and north of US 61, respectively. The channel will be constructed within a 300-ft right-of-way, to be acquired by LDNR for the project.

A 250 cfs drainage pump station will be constructed approximately 2,500 feet north of US 61 to transfer flow from the existing Hope Canal and Bourgeois Canal into the conveyance channel. The station is required because the guide levees of the proposed conveyance channel will block the existing drainage pathways of these canals.

Channel Alignment

The proposed conveyance channel alignment was selected to divert the river flow to the targeted discharge location within the Maurepas wetlands at a minimum cost. The proposed right-of-way covers a 300-ft wide, 5½-mile long swath from the Mississippi River to just north of Interstate 10. Route selection was governed by two key constraints: 1) the acquisition of right-of-way, and 2) the existing path of the Hope Canal. South of US 61, the alignment runs within the property boundaries of Pin Oak Holdings, LLC (formerly Safeland Storage, LLC), which minimizes the number of property owners affected. North of US 61, the alignment veers westward away from private residences on the east side. Beyond this area, the channel connects with the Hope Canal near the Bourgeois Canal intersection. Utilizing the existing canal minimizes the construction cost and conforms as closely as possible to the current drainage routing. From the Hope Canal interception point, the alignment follows the existing canal route underneath I-10 and terminates 1,000 feet north of the interstate.

Topographic, Bathymetric & Hydrographic Surveys

Under Task 1, the proposed outfall area was surveyed sufficiently to support the development of the one-dimensional hydrologic model. During Task 2, more detailed topographic, bathymetric and hydrographic surveys were conducted. The Secondary Control monuments used in Task 1 were also updated to reflect the more accurate elevations established by the USACE to correct for subsidence. A new Project Benchmark located in the levee near LA 44 was established at a corrected elevation of 15.73-ft NAVD88-LDNR.

Additional topographic surveying was also conducted in the levee batture to define the extent and depth of the former borrow area, or pond, on the northwest side of the proposed intake channel. This data was collected because the toe of the proposed cofferdam would intersect this area and the existing topographic information was needed to run the requisite geotechnical stability analyses to insure the USACE factors of safety were met.

Geotechnical Investigation

A comprehensive geotechnical investigation included the collection of samples and lab analyses, from which design recommendations were issued. The principal geotechnical considerations addressed include: slope stability, scour protection, foundation requirements, pile capacities, settlement, and pavement recommendations. Approvals were required from the USACE, LDNR-CMD, LDOTD, and the Pontchartrain Levee District prior to performing the field work.

The initial data collection effort included total of 17 soil borings and 17 Cone Penetration Tests (CPTs) performed along the proposed diversion route from the Mississippi River levee to I-10. In-situ CPTs were used in locations inaccessible to the soil boring rig; to insure their validity, conventional borings were taken adjacent to three of the CPT sites - the results were in close agreement. The tests at the levee ranged from 125 to 150-ft below grade while all other locations were tested to a depth of 50 to 80-ft.

Based upon the original geotechnical findings, the following side slope grading recommendations were issued to insure slope stability: Cofferdam – 4H:1V, Inflow channel – 3H:1V, Inlet structure – 3H:V1, Sediment basin – 3H:1V, Conveyance channel south of US 61 – 3H:1V, and Conveyance channel north of US 61 – 5H:1V. Mixing of the in-situ soil with cementitious materials was recommended for the sediment basin slopes for both strengthening and permeability reduction.

Project Design Constraints

The task of diverting 2,000 cfs of flow from the Mississippi River into the Maurepas Swamp in a controlled manner presents a number of technical design challenges. These include designing a gated river intake structure, a sand settling basin, and a 5½ mile long conveyance channel. In addition, the design must accommodate the existing man-made as well as natural hydrologic features of the subject area.

The inflow channel must be capable of withstanding the forces of the river while also supporting the levee. It must be anchored securely and armored to prevent erosion. The intake structure must also provide a means of controlling the volume of flow allowed into the diversion. The sediment basin must have the cross-sectional and surface areas needed to settle out sand particles. It must also have the volume to store a six month accumulation and provide access for excavation equipment to remove the accumulated sediment.

The diversion channel will require the removal of massive quantities of unacceptable material and the placement of large volumes of fill to create the guide levees. The side slopes must be as flat as 5H:1V for the levees north of US 61, which must fit within the available 300-ft right-of-way. The design must also maintain effective drainage throughout the project area, which will require the construction of a drainage pump station to transfer flow from the Hope and Bourgeois Canals into the proposed channel.

The design must minimize the impact on the wetlands during construction and also provide protection for the native fauna, including the local endangered species: the Pallid Sturgeon, the Bald Eagle, and the Manatee. The intake structure must have a high elevation and low velocity to prevent small riverine species from being inadvertently swept into the diversion. Construction must be avoided in areas with eagle nests during nesting season.

Infrastructure & Utilities

There are six key crossings along the project right-of-way: LA 44, US 61, KCS RR, CN RR, I-10, and a major pipeline corridor between US 61 and I-10. The Project Team contacted all companies with identified utilities or product lines to describe the project, establish a point of contact, and request their preferences on relocations. The companies provided offsets, depths of cover, and other pertinent requirements, along with drawings detailing the relocation criteria for their infrastructure components. The anticipated utility/pipeline dispositions described in this report are based on approximate depth of cover. The exact locations will be field-verified prior to final design and construction.

Railroad and Roadway Crossings

The CN RR has indicated that the reinforced concrete box (RCB) culvert must be a cast-in-place reinforced concrete structure. CN will relocate the switch gear and signal equipment near the crossing to accommodate installation of the RCB. The minimum distance required between tracks is 15-ft and from the base of rail to the top of the RCB it shall be 3-ft. A permit from the railroad will be required; the review process will take 4 - 6 months. LDNR is to provide flagmen at all times during construction.

The KCS RR crossing will be piled-supported and either cast-in-place or pre-cast box sections of 4,000 psi reinforced concrete. An earthen levee will be constructed within the upstream and downstream channel sections since the water surface elevations will be higher than the existing grade and track. A temporary false-work bridge will be used during the construction. Upon completion of the RCB, backfilling, and construction of the containment levees, the track shall be placed back in its original location on a new bed of compacted structural fill and rock ballast.

The US 61 crossing shall consist of a 375-ft, five-barrel, 9-ft x 9-ft, RCB constructed per LDOTD standards. The culvert may be either pre-cast or cast-in-place 4,000 psi reinforced concrete and shall be piled supported. An earthen levee will be required to contain the water within the upstream and downstream channel sections. The levee side slopes shall be 3H:1V on the south side and 5H:1V on the north side of US 61. The RCB and diversion channel shall be centered within the proposed 300-ft right-of-way.

Hydraulic Model

A computer model was developed to verify the hydraulic performance of the proposed conveyance channel design. The US Army Corps of Engineers (USACE) Hydraulic Engineering Center River Analysis System (HEC-RAS) program was chosen to model the diversion. The HEC-RAS model was used to simulate a steady-state, sub-critical flow condition corresponding to a conveyance of 2,000 cfs. The Rating Curve developed during Task 1 was used to correlate the rate of discharge from the channel to the tail-water elevation in the Maurepas Swamp. As a result, the design operating condition for the waterway was designated as 2,000 cfs of flow with 5- to 6-ft of head loss from the intake to the discharge. This must include the head losses incurred through the following five structures: 1) intake structure, including the crossing under LA 44, 2) culverts at the CN Railroad, 3) bridge at the KCS Railroad, 4) culverts at US 61, and 5) existing bridge piers at I-10.

The model results showed that a minimum river stage of +9.38 ft NAVD88-LDNR was necessary to sustain the design flow of 2,000 cfs. Historic river stage data indicates that this level is achieved approximately 190 days a year. The remainder of the year the intake structure will still flow, but at a lower capacity. A sensitivity analysis indicated that if the hydrodynamic coefficients used in the HEC-RAS model are in error by 10% in either direction, the resulting river stage required ranges from +8.63 ft to +10.24 ft NAVD88-LDNR. The corresponding periods during which the river would be capable of delivering the requisite flow would range from 136 to 158 days a year, a difference of about a month. Thus, even if all of the coefficients are in error by 10% in the same direction, an unlikely event, the proposed diversion will still be able to convey the design flow for about six months out of the year.

Intake Structure

The intake structure must convey the flow from the river into the diversion channel with a minimum of head loss. It must also function reliably over the life of the project under a wide range of conditions. The most cost-effective design to achieve these objectives was determined to be a multi-cell box culvert with vertical lift sluice gates. This configuration has been used successfully for similar diversion structures at Caernarvon (Eastbank of the Mississippi River near the St. Bernard/Plaquemines Parish line) and Davis Pond (Westbank of St. Charles Parish).

The proposed intake structure will be located approximately 100-ft south of the crown of the levee. Its platform will support a control house at elevation 31-ft NAVD88-LDNR to protect against high river stages. Placing the structures close to the levee provides a solid foundation and minimizes the required length of the culverts. The sluice gate and culvert elevations were set as high as possible to minimize excavation costs. The culverts will be installed flat, since they will operate under outlet control and slope is irrelevant to their hydraulic performance. The culverts must pass under the roadside drainage ditch along LA 44, which has an invert of +7 ft NAVD88-LDNR. Subtracting 1-ft for depth of cover yields a top-of-culvert elevation of +6 feet NAVD88-LDNR. The top wall of the culverts is expected to be up to 3-ft thick, resulting in a top-of-gate elevation of +3-ft NAVD88-LDNR.

The stage of the Mississippi River is the driving force for delivering the target flow to the conveyance channel. The water levels in the river and the channel are thus the starting points for designing the intake gates. To maximize the duration of peak flow conditions, the head-losses through the intake structure must be kept to a minimum. Increasing the size of the gate cross-section lowers the head-loss, but it also increases the cost. The hydraulic performance and construction costs of nine sluice gate configurations, ranging from a single 12-ft x 12-ft gate to three 8-ft x 8-ft gates, were compared. A group of three 10-ft x 10-ft gates was selected as the optimum configuration to balance the flow delivery capacity against the construction cost. A gate adjustment chart was developed for this configuration to deliver the design flow under various river stages.

Sedimentation Basin

There is a high concentration of sand, silt and clay entrained in the Mississippi River flow-stream. To re-nourish the Maurepas Swamp, the fine silt and clay particles must be carried throughout the diversion to its outfall. However, the sand particles must be removed upstream of the conveyance channel, lest they settle in the downstream reaches where they would have to be removed by dredging. A sedimentation basin was designed to remove the unwanted sand from the diversion flow-stream.

The LDNR indicated that the sedimentation basin should be designed to remove all sand particles ≥ 0.2-mm in diameter. They further stipulated that the basin must have adequate storage capacity to accumulate six month's of sediment without requiring cleaning. The settling velocity of a 0.2 mm particle of sand in water is approximately 4-ft per minute. Based on that value and the design flow rate, the surface area of the sediment basin was established. The cross-sectional area of the basin was then calculated to achieve a flow velocity of approximately 1 ft/s, which would prevent re-suspension of the settled solids due to turbulence.

The percent sand in the river water at Maurepas was derived by interpolating from data recorded at St. Francisville and Belle Chasse, which are upstream and downstream of the site, respectively. Data from the Caernarvon project provided a ratio of the percent sand in a diversion to that in the adjacent river water. Applying that ratio to the subject site yielded the percent sand expected in the influent to the Maurepas diversion. Based on that value, the mass and volumetric accumulation rate of sand expected in the sedimentation basin was calculated. This enabled determination of the additional basin volume required to contain a six month accumulation. The designed basin will have a central section 265-ft long by 66-ft wide, with 3:1 side slopes adding 60-ft of width on each side.

Pump Station

A 250 cfs pump station will be constructed approximately 2,500-ft north of US 61. The station will transfer the gravity flow from the Hope and Bourgeois Canals into the proposed conveyance channel. The station is required to restore the drainage pattern in the area, since the guide levees of the channel will cut-off the existing hydraulic route of the two canals. The required flow rate was estimated in Task 1 to represent the existing flow in the Garyville/Reserve area drainage system.

The proposed pump station will consist of three 125 cfs pumps. The pumps will alternate duty cycles to provide a peak flow of 250 cfs with two pumps in service; the third pump will serve as a back-up. The proposed pumps are of the vertical line shaft type, which are designed to move large volumes of flow against relatively low head. An approach basin will be constructed upstream of the pump intakes to impart a uniform velocity distribution to the inflowing water. The approach to the basin will be gradually sloped to the design elevation of the pump intakes. Both canals will be dredged and improved in the immediate vicinity to provide uninterrupted flow to the pump station. The pumps will discharge through three 48-in pipes over the eastern levee of the conveyance channel to an armored outlet structure.

The design criteria used were those set forth in the Engineering Manuals, Regulations, and Technical Letters for Civil Works Construction published by the USACE, as amended in the design guidelines developed by the New Orleans District. The LDNR recommended using the pump station at Davis Pond as a guide for the subject design, due to the success of that station and the similarity of the two applications. A formed suction intake (FSI) was selected because it requires much less submergence than a conventional rectangular intake. The shallower depth of the FSI design will significantly reduce pump station excavation, dewatering, and sub-structure costs. The pumps will be driven by motors connected to the impeller shaft by a direct coupling, the most energy efficient means of connection. Natural gas motors were selected because there is no adequate electrical power supply in the area that can be routed to the remote project site.

<u>Cost Estimate</u>

An opinion of probable construction costs was created for the "River Reintroduction into Maurepas Swamp" project based on the 30% design. The total cost of the items included is estimated to be $151,725,000.

The cost estimate is comprised of seven sections; only the major components of each section are highlighted: 1) Site Work: clearing & grubbing, excavation, and embankment; 2) Concrete: intake structure, u-channels, and control building; 3) Railroad and Road: track and road removal & replacement, concrete piles, reinforced concrete box culverts; 4) Utilities: Relocation of petroleum, gas, water, sewer, electrical transmission, and fiber-optic lines; 5) Engineered Equipment: sluice gates, hydraulic power unit, and control system. 6) Electrical: intake structure service connections, lighting and control panels; and 7) Instrumentation: flow and water quality monitoring equipment.

The cost estimate was based on the following assumptions:

- Field engineering and inspection are not included.
- Multiple contractors and tasks will be coordinated concurrently.
- A 10-15 acre site will be provided by LDNR for on-site storage of excavated and embankment material.
- All excavated material below 2-ft depth, between LA 44 and US 61, is suitable as embankment material.
- 20% of the suitable excavated material will be lost due to settlement.
- All excavated material north of US 61 is unsuitable as embankment material and must be hauled from the site.
- All costs include labor, installation, and materials.
- Concrete costs include reinforcing steel.
- The contractor will use a board road system.
- The following expenses were factored into the total cost:
    - 20% Overhead
    - 10% Contractor's Profit
    - 2% Bond

- o 30% Contingency
- o 4% Escalation
- o 7% Changes & Claims

Task 3 - Final Engineering (<u>95% Design – Current Work</u>)

<u>Project Description</u>

The Project Team has refined the Task 2 design into a completed design package, consisting of the same components:

1. An intake structure capable of providing 2,000 cfs with sluice gates for flow control.
2. Check valves on all culverts underneath I-10 from LA 641 to the Mississippi Bayou overpass to prevent backflow south of the roadway.
3. Lateral discharge valves capable of diverting 125 cfs to each side of the conveyance channel between US 61 and I-10.
4. A pump station to convey the local drainage collected in the Bourgeois and Hope Canals into the Conveyance Channel.
5. A sedimentation basin to remove the sand from the diverted flow-stream.

The basic project description remains the same as described in Task 2 above. Only the **refinements and changes** are discussed below.

<u>Intake Features</u>

The design of the intake features has retained its basic configuration as a concrete headworks facility with vertical lift sluice gates that convey water under the Mississippi River Levee (MRL) via three 10-ft by 10-ft box culverts. The intake U-Channels have been re-designed to include three sections instead of two to facilitate constructability. The river-side wingwalls have also been re-configured to be straight instead of curved to reduce the complexity of construction as well.

Significant additional Geotechnical investigation has been performed in the batture area to design the cofferdam required to serve as the mainline temporary flood protection during the construction period. This has included the collection of additional field samples, including soil borings and CPTs, as well as the associated laboratory analyses, on two separate occasions. Numerous computer analyses were conducted based on the data to establish an accurate strength line upon which the cofferdam design could be based. The data was submitted to the USACE and comment and review exchanges were conducted until an approved strength line was developed as the basis of design.

Additional topographic surveying was also conducted in the levee batture to define the extent and depth of the pond area, on the northwest side of the proposed intake channel. This data was collected because the toe of the proposed cofferdam would extend into this area and the existing topographic information was needed to run the requisite geotechnical stability analyses to insure the USACE factors of safety were met, and to be able to quantify fill for bid documents.

<u>Sedimentation Basin</u>

The fundamental design of the sedimentation basin has remained the same: it will capture all sand particles ≥ 0.2-mm in diameter, while enabling the fine silt and clay particles necessary for sustaining the marsh to be carried through the conveyance channel into the swamp. Refinements to the design have included the modifications to horizontal and vertical layout and the foundation of the Basin. The Basin Foundation had to be designed to sustain a long term maintenance and operation activity. The intended maintenance methodology has changed to include a ramp that will route from the levee crown to the base of Sedimentation Basin. A rubber tired backhoe front end loader will excavate the sediment and load trucks parked on the ramp. URS has evaluated the need for soil cement mixing vs. riprap base for the Basin and concluded that the riprap base would be a cost effective alternative.

<u>Conveyance Channel</u>

The 5½ mile conveyance channel alignment and the 300-ft right-of-way width remain the same. However, the conveyance channel has been widened to provide additional freeboard between the top of the guide levees and the water surface elevation. The side slopes have also been adjusted to minimize potential sloughing. South of the Kansas City Southern Railroad (KCS RR) crossing the channel will have an adjusted typical bottom width of 40-ft with a flattened side slope of 4H:1V (4-ft horizontal: 1-ft vertical) within the wetted portion of the channel and the same 3H:1V on the outsides. North of the KCS RR the bottom width has been widened to 60-ft and the water-side slope will remain 5H:1V while the land-side slope will be changed to 3H:1V.

<u>Railroad and Roadway Crossings</u>

- River Road (LA 44)

  The original design of the LA 44 crossing was based on the roadway being closed for the entire construction period with traffic detoured to an alternate route. Under this scenario, the roadway would have been open cut which would have provided adequate room to access the bottom of the excavation and provide staging areas for the contractor. Subsequently the LDOTD advised URS that River Road could only be closed for 45 days. While this change does not affect the design of the intake and headworks features, maintaining traffic on the roadway during construction significantly changes the approach to building the facilities.

  An detailed seven phase sequence of construction was developed to comply with the LDOTD's restriction on the road closure. Two very significant changes were: 1) the design of a 35 mph temporary by-pass roadway through the construction area made to maintain traffic per LDOTD requirements, and 2) the incorporation of multiple temporary retaining structures (TRS) in the design to provide stability and enable access to the bottom of the excavation. Geotechnical stability analyses were performed for each of the seven phases of the revised design to insure that the USACE's factors of safety are met for each stage of construction.

- Canadian National Railroad (CNRR)

  The CN RR dictated that the turn-out previously installed over the culvert crossing, which was to be temporarily removed and replaced, be permanently relocated to the east of its existing location. This will provide the CN RR with an additional 1260-ft of siding. This change required the design of the siding, including its horizontal and vertical geometry, along geotechnical analyses to develop the fill and ballast requirements to support the additional portion of track.

The change also placed two tracks instead of a single line over the reinforced box culvert crossing. Thus, the culverts would have to support twice the load as that for which they had been designed. The Structural redesign of the section of culverts under the railroad along with redesign of the supporting pile foundation was conducted.

- Kansas City Southern Railroad (KCS RR)

  At the original request of the KCS RR, the crossing was designed as a group of piled-supported, cast-in-place reinforced concrete box culverts. An earthen levee was to be constructed within the upstream and downstream channel sections since the water surface elevations would be higher than the existing grade and track. The construction was to be effected by the use of a temporary false-work bridge, for which a detailed set of design drawings, incorporating a multiple phased construction process, was developed.

  Upon review of the design, KCS RR revised their decision and advised that a bridge structure be used instead. Subsequently, the crossing design was changed to a 105-ft span railroad bridge. An additional soil boring was obtained at the location of the crossing to acquire the data needed to determine the strength of the deep subsurface strata for the development of pile curves for deeper piles that would be used to support the bridge. The Geotechnical, Structural, and Civil Engineering aspects of the bridge were re-designed and incorporated into the revised plans and specifications.

- Airline Highway (US 61)

  As indicated earlier of the changes of the channel dimensions, the culverts under the U.S. Hwy. 61 has also been updated with 6-9'x9' box culverts.

<u>Hydraulic Model</u>

The USACE advised that the hydraulic coefficients (Manning's "n" values) used in the model should be revised slightly upward. The Hydraulic Engineering Center River Analysis System (HEC-RAS, v.3.1.3) model was updated to reflect the USACE coefficients, the changed design at the KCS RR and US 61 crossings, and the revised geometric configuration of the conveyance channel. The results confirm that the modified design would convey the requisite 2,000 cfs from the Mississippi River to the Maurepas Swamp with a minimum freeboard of 2.5-ft throughout the system. The revised model also showed that 2,000 cfs of flow could still be sustained at a minimum river stage of +9.38 ft NAVD88-LDNR and that based on historic river stage data this, level would be achieved approximately 190 days a year.

<u>Pump Station</u>

The design of the 250 cfs pump station approximately 2,500-ft north of US 61 has remained essentially the same. It will consist of three 125 cfs vertical line shaft pumps each receiving their influent flow from a USACE Type 10 formed suction intake (FSI).

The pumps will discharge through three 48-in pipes over the eastern levee of the conveyance channel to an armored outlet structure. The approach basins, access roads, bridge and all structural components were designed in detail, as well as the pumps and engines. In addition, a new boat launch has been designed to replace the existing launch, which is currently located at the site of the new station.

The major changes in the design from the preliminary Design Report includes the soil cement mixing that is proposed under the foundation of the guide levee and the intake basin for the pump station. The soil improvement at this stretch were designed to reduce long term maintenance required due to levee settlement. The levee settlement in this stretch is anticipated to be more if the levee is built in wet in the current footprint of the Hope Canal without soil improvements.

Cost Estimate

The opinion of probable construction costs was revised based on the new 95% design. The total cost of the items included is estimated to be $133,960,000.

1.0 GENERAL

Introduction

The rate of wetlands loss in Louisiana has reached the state at which system collapse threatens the productivity of these bountiful ecosystems. The demise of the wetlands is accompanied by the loss of the various functions they serve, such as fish hatcheries, endangered species habitats, natural water filters, and buffers against hurricane storm surges, to name just a few. The commercial and recreational value of these losses is enormous, but the losses associated with our cultural heritage cannot even be measured. The value Louisiana's coastal wetlands, in particular, and the threats they face are described in the Coastal Wetland Forest Conservation and Use Science Working Group's 2005 Report to the Governor, ***Conservation, Protection and Utilization of Louisiana's Coastal Wetland Forests***:

> "*Louisiana's coastal wetland forests are of tremendous economic, ecological, cultural, and recreational value to residents of Louisiana, the people of the United States, and the world. Although some two million acres of forested wetland occur throughout Louisiana, over half are in the coastal parishes. Large-scale and localized alterations of processes affecting coastal wetlands have caused the complete loss of some coastal wetland forests and reduced the productivity and vigor of remaining areas. This loss and degradation threatens ecosystem functions and the services they provide*."

The Maurepas Swamp is located in St. John the Baptist Parish, north of the Mississippi River and southeast of Lake Maurepas. The swamp is a freshwater cypress-tupelo forested wetland on the upper tidal margin of the Lake Pontchartrain / Lake Maurepas estuary system. Situated between the southern shore of Lake Maurepas and the developed uplands of the Mississippi River natural levee, the Maurepas Swamps represent a significant part of the coastal wetland forests described by the above passage. It also exemplifies the large-scale and localized alterations of processes the Coastal Wetland Forest Conservation and Use Science Working Group (2005) discusses in their report. The fundamental problem the Maurepas Swamps face, as well as the general solution to that problem is precisely described in the report:

> "…*Louisiana's coastal wetland forests are sediment and nutrient deprived as a result of the Mississippi River levee system and are experiencing significant habitat loss. Under these conditions, the addition of nutrients and sediments is the only way for these ecosystems to maintain their surface elevation relative to sea-level rise.*"

Over the last several decades, a great deal has been learned about the natural processes of subsidence as well as the effect of man-made flood control measures that cause marshes to change to open water. A major cause of swamp deterioration throughout southeast Louisiana is the channelization of the Mississippi River within the levee system. This measure has eliminated the natural inputs of freshwater, nutrients, and sediment that historically built and maintained the wetlands. The Maurepas Swamp has experienced severe decline after having been cut-off for decades from the periodic inundation that the Mississippi River had provided over thousands of years, which had nourished its development. The swamp is threatened by episodic brackish water intrusion from Lake Maurepas, long-term subsidence, and the elimination of nutrient inputs.

In addition to the problems caused by large-scale alterations of processes, the Coastal Wetland Forest Conservation and Use Science Working Group (2005) report cites the importance of localized alterations of processes. Several of these local-scale factors are major concerns in the Maurepas Swamps:

> "*The cumulative effects of small-scale or local factors can be of equal or greater importance in coastal wetland forest loss and degradation than large-scale alterations. These factors include increased depth and duration of flooding, saltwater intrusion, nutrient and sediment deprivation, herbivory, invasive species, and direct loss due to conversion. Causal agents include highways, railroads, channelization, navigation canals, oil and gas exploration canals, flood control structures, conversion of forests to urban and agricultural land, and non-sustainable forest practices.*"

Finally, the Coastal Wetland Forest Conservation and Use Science Working Group (2005) points to the ramifications if appropriate restoration of Louisiana's coastal wetland forests is not undertaken. These apply to the Maurepas Swamps as well as all of Louisiana's coastal forests:

> "*Without appropriate human intervention to alleviate the factors causing degradation, most of coastal Louisiana will inevitably experience the loss of coastal wetland forest functions and ecosystem services through conversion to open water, marsh, or other land uses*."

Project Authority

The Federal Coastal Wetlands Planning, Protection and Restoration Act (CWPPRA), originally known as the Breaux Act, is the legislative authority under which the subject project is funded. CWPPRA has identified the area south of Lake Maurepas as a region where wetlands are in need of restoration. The subject project, *River Reintroduction into Maurepas Swamp*, was selected for Phase 1 (Engineering and Design) funding at the August 2001 Breaux Act Task Force meeting. The project is on Priority Project List 11.

The URS Team has been tasked with the design of the diversion project by the Louisiana Coastal Protection and Restoration Authority and the Environmental Protection Agency under PO-29, Contract No. 2503-11-63. This 95% Design Report and the accompanying plans are being submitted under TASK 3 of the contract and are at the 95% completion stage.

Project Description

The proposed Maurepas Freshwater Diversion Project consists of diverting flow from the Mississippi River deep into the heart of the Maurepas Swamp wetlands. As illustrated in the schematic on Figure 1-2, the starting point for the proposed diversion project is located on the Mississippi River between Garyville, LA and Gramercy, LA. The project features a gated intake structure at the river, a large sand settling basin, and a very long banked conveyance channel. At roughly the mid-point of the route, just north of US 61, the channel follows the existing Hope Canal alignment to ultimately distribute the diverted water into the wetlands on the north side of Interstate 10.

Project Purpose

The viability of the Maurepas wetlands is threatened by a synergistic combination of factors. The precipitate cause is the man-made channelization of the Mississippi River by the construction of levees for flood control and navigation. The levees disrupt the hydrologic connection between the river and the wetlands, thus preventing the periodic natural inundation of the area by the sediment laden, nutrient rich, fresh water of the river. Without the input of sand, silt and clay entrained in the river water, mineral accretion in the wetland areas becomes limited. The denial of dissolved nutrients, especially nitrogenous compounds, severely limits plant production, which leads to limited organic accretion.

As the existing materials in the wetlands undergo decomposition, compaction, and settlement, without a compensating accretion of mineral and organic soil components to rebuild the land surface, the area subsides, effectively lowering the elevation of the wetland. The loss of vegetation reduces the wetlands' ability to trap whatever sediment is available to promote vertical build-up. The local subsidence enables brackish water from Lake Maurepas to flow deeper into the wetlands on a more frequent basis, further stressing the plant life, causing additional vegetative die-off, which accelerates the cycle of decline. Water also pools in the lower elevations, causing wetlands loss as larger areas become permanently flooded and convert to open water.

Thus, the severing of the hydrologic connection between the Maurepas Swamp and the Mississippi River has interrupted the natural accretion processes and lead to a decline of the wetlands. To restore the wetlands, the major cause of swamp deterioration - flood control on the Mississippi River, which has eliminated the natural inputs of freshwater, nutrients, and sediment, must be counterbalanced by controlled diversions, enabling an infusion of river water into the swamp. River diversion into wetland areas is a recognized restoration strategy, with the purpose of reversing existing conditions of cypress-tupelo stress and loss by addressing the problems of subsidence, permanent flooding, and sediment/nutrient starvation.

The ultimate goal of this project is to restore the health and essential functions of the Maurepas Swamp wetlands. The principal task in meeting that goal is to provide the area with freshwater and sediment, thereby simulating the periodic flooding which occurred prior to the construction of the levees along the banks of the Mississippi River. This action is intended to achieve two primary objectives: 1) Significant reduction of the saltwater intrusion into the area that is currently occurring from Lake Maurepas, and 2) Provision of fine-grained sediments and nutrients needed to enhance the productivity of the native swamp vegetation.

2.0 PRIOR STUDIES

Introduction:

The Maurepas Swamp ecosystem and the proposed diversion channel have been studied in great detail prior to this 95% Design Report. The following is a list of the reports generated in earlier Phases. Included with each report listing is a summary of its major findings.

PHASE 0

Phase 0 of this study consisted of several studies to predict the effects of the proposed diversion project on the Maurepas Swamp wetland area. These studies were conducted by Louisiana State University and Southeastern Louisiana University, as well as by the private consulting firms of Lee Wilson & Associates, Inc., and Battelle.
These studies and their findings are summarized below; the reports can be obtained in their entirety from EPA's website:
http://www.epa.gov/region6/6wq/ecopro/em/cwppra/maurepas/index.htm

"Diversion into the Maurepas Swamps", prepared by: Lee Wilson & Associates, Inc. with: Drs. Gary Shaffer and Mark Hester of Southeastern Louisiana University and Dr. Paul Kemp, Hassan Mashriqui, Dr. John Day, and Robert Lane of Louisiana State University, dated June 2001.

Summary:

This study addressed the wetlands south of Lake Maurepas, a large water body located near and northwest of New Orleans, Louisiana. Federal and state restoration initiatives, especially the Coastal Wetlands Planning, Protection, and Restoration Act (CWPPRA), had identified the region south of Lake Maurepas as an area where wetlands vegetation (especially the cypress tupelo swamp) is stressed and dying, and in need of restoration.

A major cause of swamp deterioration posited was that flood control on the Mississippi River has eliminated the natural inputs of freshwater, nutrients, and sediment that historically built and maintained the wetlands. River diversions into the South Maurepas swamps were proposed as a recognized restoration strategy, with the purpose of reversing existing conditions of cypress-tupelo stress and loss by addressing the problems of subsidence, permanent flooding, and sediment and nutrient starvation.

The concept to divert Mississippi River water into the region of degraded swamp south of Lake Maurepas was nominated for consideration on Priority List 9 of the CWPPRA program, and was defined as a complex project. The Maurepas Phase 0 complex project study was a reconnaissance-level effort to develop and compare project alternatives, and select the most appropriate project to be recommended for further evaluation. Activities within the scope of this study included: 1) Preliminary site reviews 2) Hydrologic modeling of existing conditions and basic diversion scenarios, 3) Baseline ecological field studies, and 4) Surveying of elevations and cross-sections.

Diversion size was provisionally set based on the assumption that cost and logistical factors would make it important to fit a diversion project into the existing channel beneath I-10. The limiting discharge capacity through the I-10 bridge was found to be between 1,500 and 2,000 cfs; the more conservative value of 1,500 cfs was used for modeling the project to limit water velocities within the channel at the I-10 bridge.
To estimate potential costs and benefits of a diversion, a conceptual project in the Hope Canal area was defined to include the following features.

- A diversion at the Mississippi River, using box culverts. These would give the greatest flexibility in diversion operations, allow diversion of water throughout most of the year, enable the most control over volume discharged, and provide the greatest potential sediment benefits. Two 10-ft x 10-ft box culverts would be capable of achieving the targeted flow of 1,500 to 2,000 cfs. Inverts would be set to assure capability of essentially year-round diversion. A 100-ft x 100-ft (bottom dimension) receiving pond at the outfall of the box culverts would be used to slow water velocities and cause coarser sediments to drop out for ease of maintenance.

Update: Detailed hydraulic modeling has since shown that three 10-ft x 10-ft gated openings are required. The revised design also features a much larger settling basin. Both of these revisions are discussed in detail within the Preliminary Design Report.

- A new channel from the diversion structure to a point just north of US 61, where the constructed channel would intersect Hope Canal. The channel, to be located just east of the Kaiser Tailings ponds, would be used to convey water safely across agricultural/industrial lands and developed infrastructure. Relocations and structures needed to cross US 61, LA 44, the railroads, and the intervening pipelines were included in the project cost estimate.

- An improved channel along the existing Hope Canal from just north of US 61 to I-10. The improvements, including guide levees, would increase the carrying capacity of Hope Canal from the existing 100 - 150 cfs to 2,000 cfs. The conveyance channels were sized at 2,000 cfs to assure sufficient capacity for both the diverted plus existing flows. The guide levees would be required to contain the water within the channel until the release point north of I-10. Without guide levees, the flow entering Hope Canal north of US 61 would be dispersed into the swamps south of I-10, which were not the main target of the project. Guide levees along this portion of Hope Canal also would prevent impacts to water levels in the swamps that adjoin the developed areas south of US 61. Use of the large conveyance capacity of the improved channel could be coordinated with Parish drainage plans to provide substantial benefits to local drainage and flood control needs.

- The existing Hope Canal channel, with outfall management structures between I-10 and Lake Maurepas. The existing low capacity of the canal (100 - 150 cfs), numerous existing breaks in the canal banks, and outfall management were predicted to minimize the amount of diverted water that would remain channelized and thus flow directly into Lake Maurepas without first flowing through the swamp. Outfall management would include additional gaps in a remnant railroad bed that parallels the west side of Hope Canal, and channel constrictions to be constructed in the canal.

Long-lived species regenerate slowly, so high mortality rates can't be tolerated. To preserve swamps over the long-term, conditions must be re-established that allow survival of existing cypress and tupelo trees and also permit at least periodic reproduction and recruitment of seedlings. Non-stagnant water, accretion, and freshening are all needed to achieve these goals. From the perspective of sustainable ecosystem management, implementation of an appropriate size diversion into the swamps south of Lake Maurepas was believed to be the essential and singular approach that could restore the swamps to environmental sustainability.

Implementation of the proposed diversion was predicted to greatly increase flow through the project area, which would provide a constant renewal of oxygen and nutrient-rich waters to the swamps. Benefits would include measurable increases in productivity, which would help build swamp substrate and balance subsidence, as well as increases in tree growth, reduced mortality, and an increase in soil bulk density. As accretion improves, an increase in recruitment of new cypress and tupelo, required for long-term sustainability of the swamp, would also be expected.

Anticipated sediment benefits to the swamp included direct contributions to accretion, as well as to biological productivity through the introduction of sediment-associated nutrients, which would also contribute to the production of substrate. A conservative estimate of >1000 $g/m^2/yr$ sediment loading to the target swamps was predicted from a Maurepas diversion, or about twice the quantity needed to keep up with subsidence.

Results of the Phase 0 Study showed that the Maurepas swamps are almost certainly nutrient limited. Other studies provided the expectation that the addition of nutrients with diverted water would at least double growth rates of the dominant swamp trees. An important adjunct to this finding was that nutrients added with the diverted river water would be essentially completely taken up within the swamp (i.e., prior to discharge to Lake Maurepas). The addition of nutrients and the associated increase in production was predicted to contribute substantially to the buildup of swamp substrates (accretion) through organic contribution, which would help counterbalance subsidence. So, nutrient additions would directly improve the health of the trees and the condition of the swamp, and in the long run also help generate a more conducive environment to sprouting and recruitment of cypress and tupelo seedlings.

This study also showed the impacts of saltwater intrusion on the cypress-tupelo swamps, indicating significant mortalities of tupelo, red maple and ash, and suppression of tree productivity in the areas of highest salinity. Saltwater intrusion in the Maurepas swamps was shown to be impacting swamp vegetation already stressed by excessive flooding. The proposed diversion was expected to directly ameliorate increasing salinities in the swamps south of Lake Maurepas, as well as in the lake itself. This was expected to largely prevent the high mortalities previously observed in the project area. More persistently freshwater conditions were also expected to help increase tree and herbaceous productivity, which, along with the flow-through of oxygen, sediment, and nutrient rich waters, would contribute to stronger (i.e., higher bulk density) substrates and increased accretion.

Beyond direct benefits to the swamps, it was also expected that Lake Maurepas would experience significant freshening as a benefit, which could have a positive impact on fisheries as well as other ecosystem components. Rivers and bayous entering the lake, such as Blind River, have also been impacted by increasing salinities and stagnant water conditions, and would garner freshwater benefits from the proposed diversion as well.

The Gulf of Mexico continental shelf off Louisiana currently experiences widespread hypoxia (low dissolved oxygen conditions) during the summer, attributed to direct introduction of nutrient-rich water from the Mississippi River. It has been recommended that wetlands and shallow water bodies be used to process river water before it enters the Gulf, to reduce the magnitude of this hypoxic zone as well as help restore the wetlands. Since this study indicates that 94% to 99% of the nutrients introduced in diverted water will be processed and retained by the swamps, it can be assumed that the contribution of the diversion toward the amelioration of Gulf hypoxia would be proportional to the magnitude of flow diverted from the Mississippi River. Because the volume of the proposed Maurepas diversion is small compared to average flows in the river, by itself this diversion would not have a measurable impact on the size of the hypoxic zone. But the proposed diversion should be viewed as a functional component of a potentially larger system of diversion that together can ameliorate nutrient delivery to the Gulf.

"Ecosystem Health of the Maurepas Swamp: Feasibility and Projected Benefits of a Freshwater Diversion", prepared by: Southeastern Louisiana University, Wetland Restoration Laboratory, Department of Biology and Louisiana State University, Department of Oceanography and Coastal Science, dated June 2003.

Summary:

This was a feasibility study of re-introducing Mississippi River water into the Lake Maurepas Swamp, a highly degraded bald cypress-tupelo gum swamp system located in the northern Lake Pontchartrain Basin. The work was a continuation of similar efforts conducted by Dr. Shaffer as part of the Phase 0 project. The purpose of the study was to evaluate the current condition of these swamp forests and assess the potential benefits to the whole ecosystem from a freshwater diversion into the area. The subject wetlands were noted as part of the Blind and Amite River mapping units within Region 1 of the Louisiana coastal zone as defined in the Coast 2050 (1998) planning effort and restoration report, an area identified as stressed and dying, and in need of restoration.

The proposed freshwater diversion was sponsored by the Environmental Protection Agency (EPA) as the recommended strategy for restoring these wetlands under funding from the Coastal Wetland planning, Protection, and Restoration Act (CWPPRA, 1990).

This study consisted of an investigation of the potential effects of freshwater diversion on the rate of local wetland subsidence. It also addressed the specific abiotic conditions found at the study site to provide insight into which factors most affect the observed vegetative conditions of the swamp and how these factors may be affected by a diversion. The health and rates of primary production of the woody and herbaceous components of the vegetation at these sites were also evaluated.

The report cites several studies that indicated the Maurepas swamp appears to be converting to marsh and open water primarily due to the lack of riverine input. Salt stress is killing trees that are proximal to Lake Maurepas, whereas stagnant standing water and nutrient deprivation appear to be the largest stressors at interior swamp sites. Furthermore, as increasing periods of flooding have been found to decrease the allocation of carbon to the root system (Powell and Day 1991), sites with stagnant standing water such as interior swamp sites are expected to show a greater rate of subsidence than sites that are only seasonally flooded. On average, flood durations in the Maurepas swamps have doubled over the past half century.

Severe increases in salinity, like those experienced during the drought in 1999 - 2000, however, may be prevented or greatly ameliorated by the increased fresh water throughput that the proposed diversion would bring. It is likely that the influence of freshening would be felt in areas as remotely located as Jones Island and the Manchac land bridge, as the proposed diversion could replace all of the water in Lake Maurepas roughly twice each year, and Pass Manchac and North Pass are the only two direct conduits that will allow the additional fresh water to eventually reach Lake Pontchartrain. Besides decreasing the detrimental effects of salinity throughout the Maurepas swamp, the proposed diversion would also increase the sediment load and nutrient supply to these wetlands. Hydrologic modeling showed that due to the low water holding capacity of Hope Canal, most of the diversion water is likely to flow as sheet flow through the interior Maurepas swamps. The resulting, evenly distributed influx of sediments was expected to strengthen the highly organic soils of the Maurepas swamp and to increase elevation in certain areas sufficiently to make the natural regeneration of several wetland forest species possible. The potentially negative impacts of lake eutrophication due to the increase in nutrient loading to the swamp were seen as unlikely to occur, as nutrient models indicated high nutrient retention in the swamp with removal efficiencies of 94%-99% from the time diversion water enters the swamp until it reaches Lake Maurepas.

Experimental nutrient augmentation enhanced biomass production of the herbaceous vegetation by up to 300%. Furthermore, several studies conducted over the last decade have demonstrated that nutrient augmentation to bald cypress seedlings doubles growth rates in the Manchac/Maurepas area. Swamps as nutrient poor, stagnant, and impounded as the interior Maurepas swamps would be expected to at least double their rates of production if they received an infusion of freshwater and nutrients from the Mississippi

River. This enhanced productivity is essential for subsiding coastal wetlands to offset Relative Sea Level Rise (RSLR), as roots may contribute as much as 60% of the annual increment to soil organic matter. The exact duration and depth to cause the transition from swamp to marsh remains an unresolved mystery. The report concludes that without a diversion from the Mississippi River, the Maurepas swamp may resolve this issue all too clearly.

"Development Plan for a Diversion Into the Maurepas Swamp, Water Quality and Hydrologic Modeling Components", prepared by John W. Day, Jr., G. Paul Kemp, Hassan S. Mashriqui, Robert R. Lane, Dane Dartez, and Robert Cunningham of the Louisiana State University, School of the Coast & Environment, Natural Systems Modeling Group, dated September 2004.

<u>Summary:</u>

This EPA-funded study was a continuation of similar efforts conducted by Day et al. as part of the Phase 0 project. The work addressed the issue of whether diverting up to 2,500 cfs of Mississippi River water into an estuarine cypress-tupelo swamp would save the trees. The answer was shown to involve a complex mix of history, hydrology, chemistry and ecology.

Critical baseline hydrologic and water quality information was acquired through a two-year field study. Extensive use was also made of information acquired during the drought of 2000. These results were used to calibrate and validate linked hydrodynamic and water quality models. The calibrated models were set up to answer questions about nutrient uptake and the likely response of the forest community to diversions operated at maximum discharges of 500, 1,500 and 2,500 cfs. To improve the linkage to the ecology, results were reported for specific forest plot locations that had previously been studied.
Monthly water samples were acquired from April 2002 to May 2003 throughout the study area in a pattern established to support the forest ecology work. Samples were analyzed for the constituents of most importance to diversion design, namely suspended sediment, nitrogen, phosphorous, silicate, chlorophyll, and salinity. These provided a baseline for a year of normal rainfall.

Nitrate concentrations at sampling stations ranged up to 0.32 mg-N L^{-1} (ppm), with a mean of 0.09 ppm. The highest concentrations occurred from November 2002 to May 2003 in Lake Maurepas and the Amite River. These were generally higher than observed during the 2000 drought, but even the highest was relatively low compared to concentrations in the Mississippi River (0.75 to 2.0 ppm). More dissolved inorganic nitrogen in waters of the Maurepas was in the form of ammonium, NH_4-N, rather than as nitrate in 2002-2003. Ammonium concentrations ranged up to 1.2 ppm, and averaged 0.40 ppm, an order of magnitude higher than measured during the 2000 drought.

The highest ammonium concentrations were measured in the Blind River, Reserve Relief Canal, and at the I-55 canal, probably because of runoff from developed areas. Mean values in the Maurepas are higher than ammonium levels in the Mississippi River, which are generally below 0.1 ppm. During the drought, most nitrogen found in the Maurepas area was in complex organic forms, such as humic substances, tannins, and phytoplankton. During 2002-2003, however, only half of the nitrogen found in water samples was in the organic form, while ammonium was the predominant dissolved inorganic form. In the swamp interior, nitrogen concentrations are similar to those found in other wetlands along the Louisiana coastal zone that are not receiving river water. Nitrogen to phosphate ratios of 16:1 and greater were found in individual samples from Maurepas but were generally confined to the Amite and Blind Rivers. These streams receive runoff from developed areas to the west. Low nitrogen to phosphate ratios is evidence that the Maurepas basin is nearly always nitrogen limited. Introduction of inorganic nitrogen to such a nitrogen-limited ecosystem will support increased plant production, particularly for algae and floating vegetation, even if other nutrients are not increased.

Light Imaging Detection and Ranging (LIDAR) data acquired in 1999 was used to construct the geometry of a receiving swamp that ranges in elevation between 1.0 and 1.8 (NAVD88), and averages 1.15-ft. The mean tide elevation, in contrast, is 1.5-ft, meaning that the swamp is inundated more than half of the year. A Canopy Index (CI) created from LIDAR returns from different elevation slices was used to create a map of forest canopy integrity. Results showed the promise of this approach for generalizing from forest plot data to the landscape scale.

Finite-element hydrodynamic and water quality models produced predictions of the immediate effects of river water diversion on the swamp and adjacent water bodies. Water levels were predicted to rise by less than 0.25-ft under discharge scenarios ranging from 500 to 2,500 cfs and were fully developed in less than one month. This stage increase was less than estimated earlier. Flow velocities in the swamp for all diversion discharges were predicted to be less than 0.3 fps. The model showed that a 2500 cfs diversion would reduce Lake Maurepas salinity by 30% after only one month, an important benefit to a swamp forest that experienced salinities greater than 5 ppt in 2000. Mean nitrate concentration for river water reaching Blind River or the Lake was predicted to range from 0.05, 0.15 to 0.19 ppm, for 500, 1500 and 2500 cfs diversions, respectively. The value for a 1500 cfs diversion was higher than concentrations measured in 78% of the samples acquired during the baseline period, but was within one standard deviation of the mean observed in 2002 - 2003. The mean exit concentration predicted for a 2,500 cfs diversion was greater than values measured in 96% of all samples collected. This analysis supports the earlier finding that a 1500 cfs diversion would provide a significant nutrient infusion to about half of the swamp south of the Lake, while reducing transiting nitrate by 90%.

Seven diversion operation scenarios were simulated that resulted in mean annual discharges ranging from 500 to 2,500 cfs. These scenarios covered the range of possibilities for proposed diversion structure and conveyance channel designs. The hydrodynamic and water quality models were too computationally intensive to continuously simulate more than a few months in the prototype. Such models cannot directly drive an ecological model for a period of 50 to 200 years, the appropriate timeframe over which forest evolution should be evaluated.

An ecological model was developed (SWAMPSUSTAIN) to bridge the gap between the hydrodynamic model and a fully functional individual orientated model. It predicts that between 2,000 and 4,000 ha of the Maurepas swamp could be restored to sustainability within 50 years if mean diversion discharges greater than 1,000 cfs were initiated. This leaves a substantial portion of the project area that would benefit from salinity control and nutrient addition, but would not be restored to sustainability without additional restoration efforts.

Task 1

Task 1 of the project consisted of an extensive data collection and analysis effort to address the physical hydrodynamics of the Maurepas diversion. The hydraulic feasibility study is contained within seven volumes as listed below:

- Volume I, Executive Summary
- Volume II, Secondary Benchmark GPS Static Survey
- Volume III, Topographic and Bathymetric Survey
- Volume IV, Hydrologic Data
- Volume V, One Dimensional (SWMM) Model
- Volume VI, Two Dimensional Hydrodynamic Swamp Area Model, Development and Calibration
- Volume VII, Diversion Modeling
-

This report can be found in its entirety on the Louisiana Department of Natural Resources website:
http://sonris.com/direct.asp?path=/sundown/cart_prod/cart_bms_avail_documents_f
The seven volumes describe the methodologies, detailed findings, conclusions and recommendations of the hydraulic feasibility study. A brief summary follows:

"Mississippi River Reintroduction Into Maurepas Swamp Project (PO-29)", prepared by URS Corporation, with Evans-Graves Engineers, Inc., dated March 2007.

Summary:

The findings from the physical hydrodynamic modeling support the reintroduction of the Mississippi River into the Maurepas swamp via Hope Canal as technically feasible. All model findings must be considered in light of the model calibration/validation results, which showed that the model under-represents swamp resistance relative to the channels. This indicates that while swamp velocities are likely to be lower, diversion flow through channels (i.e., short-circuiting) is likely to be greater than found in the model results.

Thus, median swamp retention times may be shorter. Also, drainage impacts could be slightly higher than estimated, particularly during Lake Maurepas surge events.
With regard to the four objectives of the study:

1. Flow distribution throughout the North Swamp (between Blind River and Reserve Relief Canal) can be improved by including the identified outfall management features in combination with pulsing the diversion flow. Targeting sustained flow for prolonged periods above the mean water surface elevation of Lake Maurepas, and controlling minimum diversion velocities, will also aid in diversion distribution.

2. Pulsing and control of diversion flow in response to Lake Maurepas water surface elevation should aid in extending the median swamp retention time and reducing short-circuiting to the lake. Control of sediment deposition and aquatic vegetation is crucial to long-term circulation maintenance.

3. The planned diversion and associated outfall management features will not adversely impact the stormwater drainage systems for the Hope Canal watershed provided that a forced drainage system of adequate capacity replaces the gravity drainage system. The impact on the Garyville/Reserve gravity drainage system east of Hope Canal is minimal for a 24-hour/10-year return frequency rainfall event and can be easily mitigated.

4. Diversion velocities at I-10 are in a moderate range and can be readily addressed to prevent scouring. Isolated locations of minimal bank and gap scouring potential can also be addressed.

Recommendations: The simulation findings provide the basis for eight specific project design and operating requirements.

1. The major features included in the "Refined Outfall Management" simulation and those indicated by its results, are required to provide improved circulation and mean swamp retention time. Major features include gapping the abandoned railroad embankment, flow control devices underneath Interstate-10, and flow restrictions at the mouth of the Bourgeois Canal.

2. A maximum diversion design flow of at least 2,000 cfs is required, along with controls to manage flow circulation and retention time in response to forecasted Lake Maurepas water surface elevations.

3. Flow control features to regulate flow through the culverts under I-10 between LA 641 and Mississippi Bayou are needed.

4. Additional flow control features to provide limited introduction of water into the swamp south of I-10 from the diversion channel are required. Occasional

introduction of low rates of diversion water is needed to prevent stagnation and improve nourishment of the swamp south of the interstate.

5. Replacement of the Hope Canal watershed gravity drainage system by forced drainage, including a pump station of adequate capacity will be necessary.
6. Increased drainage or pumping capacity for the eastern Garyville and Reserve drainage systems will be needed to mitigate for minor impacts. This could include several options: a) increasing drainage capacity from Godchaux Canal to Reserve Relief Canal via the Cross-Over Canal; b) increased capacity of the Hope Canal pump station and drainage system, or c) increased capacity for the Reserve Airport and/or Reserve Relief Canal pump stations. The Reserve Airport and Reserve Relief pump station currently provide limited augmentation to the gravity drainage system.

7. Upgraded armoring of the diversion channel at the current I-10 overpass over Hope Canal and additional erosion controls are needed at locations where diversion velocities may exceed the scouring threshold of 2 fps.

8. Design and operating measures must be included to prevent sediment deposition and aquatic vegetation growth that would adversely affect circulation, including optimization of the sand/silt settling basin.

These requirements are refinements of and in some cases additions to, the Phase 0 Report conceptual diversion plan.

<u>Task 2</u>

The Project Team has implemented the majority of the recommendations from Task 1 into the Task 2, 30% Preliminary Design Report and the accompanying Preliminary Construction Plans. Below is a list of the recommendations made from Task 1 and the subsequent features incorporated into the design by the Project Team.

<u>Recommendation 1</u>: Improve water circulation and retention time in the swamp.

The design includes one-way check valves to control the flow underneath I-10. A pump station at the mouth of the Bourgeois Canal has been designed to aid in drainage operations for the local area. Gaps in the existing railroad embankment are also incorporated into the design.

<u>Recommendation 2</u>: Size diversion for 2,000 cfs and install controls.

The intake structure has been designed to achieve a peak design flow of 2,000 cfs for approximately half of the year. The structure will be comprised of three sluice gates, which will be able to control the flow at all times.

Recommendation 3: Install flow controls on culverts under I-10 between LA 641 and Mississippi Bayou.

The design includes the placement of one-way check valves on all culverts underneath I-10 from LA 641 to the Mississippi Bayou overpass.

Recommendation 4: Provide limited flow into the swamp south of I-10.

Culverts with control valves have been designed along the east and west banks of the conveyance channel from US 61 to I-10. These culverts will divert approximately 125 cfs (under peak design flow) to each side of the conveyance channel. An addendum to this report will describe these project features in more detail.

Recommendation 5: Replace the gravity drainage system for the Hope Canal watershed with a pump station.

A pump station has been designed for the intersection of the Bourgeois and Hope Canals to pump the local drainage flow over the proposed guide levees. An addendum to this report will further explain this portion of the project.

Recommendation 6: Increase the capacity of the eastern Garyville and Reserve drainage systems to mitigate for minor impacts.

Based upon the detailed hydrologic and hydraulic modeling conducted, there will not be any significant impacts in this area. The drainage capacity of the Hope and Bourgeois Canals will be significantly increased by the proposed pump station and the area is hydraulically distant from the project site.

Recommendation 7: Upgrade the armoring at the I-10 overpass where diversion velocities may exceed the scouring threshold.

Based upon a detailed investigation, upgraded armoring of the diversion channel at I-10 will not be necessary. The piles that support the interstate are extremely deep, as noted by as-builts obtained from LDOTD, and the side slopes leading up to the bridge deck are concrete. These measures are adequate protection for the existing structures, thus additional erosion protection will not be required.

Recommendation 8: Measures must be included to prevent sediment deposition and aquatic vegetation growth in the conveyance channel.

A large sedimentation basin has been designed to remove the majority of the large and/or heavy suspended solids entrained in the diverted flow-stream. The remaining smaller and/or lighter particles will stay in suspension until the flow discharges from the channel, at which point the velocity will be significantly reduced. Under the gentler flow regime as the water disperses throughout the swamp, the finer solids will gradually settle, providing the material needed for the accretion of new substrate.

3.0 LAND OWNERSHIP AND ALIGNMENT

Introduction

The primary objective in laying-out the proposed conveyance channel alignment was to insure that the diverted river flow was conveyed to the optimum discharge point within the targeted Maurepas wetlands. A second objective was to design a system that could reliably deliver the requisite flow at a minimum cost. The alignment design for the conveyance channel was governed by two key constraints: 1) the acquisition of right-of-way, and 2) the existing path of the Hope Canal.

To minimize the number of property owners affected, South of US 61 the primary concern was to stay within the property boundaries of Pin Oak Holdings, LLC. Pin Oak was cooperative in providing the needed right-of-way within their property limits, but they did request that the design be laid out so as to minimize their property loss. This meant keeping the alignment as far to the east as possible to prevent the creation of a large unusable outparcel of property on the east side.

North of US 61, the alignment veers away from the private residences located along the north side of the roadway. Once the conveyance channel passes this area, it connects with the Hope Canal near its intersection with Bourgeois Canal. By utilizing this existing drainage feature, the construction cost of the proposed conveyance channel is minimized. This path also conforms as closely as possible to the current drainage routing. From the Hope Canal interception point, the alignment proceeds north along the existing canal route until it passes underneath Interstate 10 and terminates 1,000 feet north of the interstate.

Land Ownership

The following parcels of land are impacted by the 300-ft right-of-way required for the diversion project to proceed. LDNR is in the process of acquiring land rights from these property owners. Plan Sheet AG-6.01 in Appendix A, illustrates the 300-ft permanent easement required for the conveyance channel. The affected tracts of land are highlighted in red and the proposed path of the conveyance channel is shown in blue. The impacted properties are listed below in order (left to right) starting at the Mississippi River bank and continuing to Interstate 10.

Pin Oak Holdings, LLC
- 233.06 acre parcel

Pin Oak Holdings, LLC
- 171.*32 acre parcel*

Ernest Amann
- 83.46 acre parcel

St. Amant, Erick and Judith
- *1.78 acre parcel*

Estate of Sidney Levet, Jr.
- 103.99 acre parcel

Blind River Properties, Inc.
- 31.01 acre parcel

St. John Shingle Company, et al
- 173.54 acre parcel

Blind River Properties, Inc.
- 168.56 acre parcel

Blind River Properties, Inc.
- 155.48 acre parcel

Estate of A D Bougere
- 177.71 acre parcel

Blind River Properties, Inc.
- 1226.60 acre parcel

Alignment Alteration

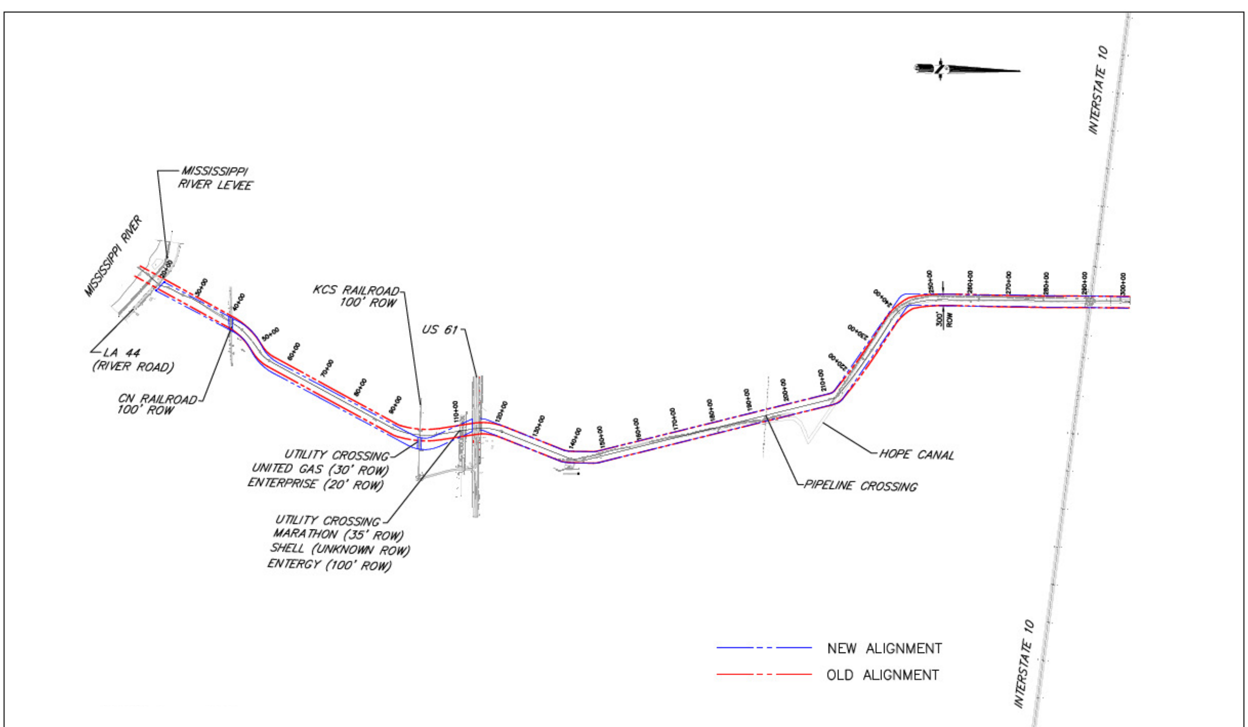
The original alignment of the conveyance channel was based upon the survey conducted by Wink Inc., which itself was based upon GIS information obtained from LDNR. This information was used to survey a 300-ft wide swath of land from the Mississippi River to just north of Interstate 10. With the exception of the area immediately adjacent to the roadway, the portion of the alignment north of US 61 is basically undeveloped swamp. The remote nature and significant tree cover of this area made surveying more of a challenge, and thus slightly less accurate. However, nominal shifts in the alignment through this area would have no practical effect on any landowners or on the functioning of the conveyance system.

In their review of the preliminary alignment plans, LDNR compared the proposed route to updated GIS maps and revised land ownership data within the area. During that review, slight discrepancies were found at the beginning of the project, near the private residences, in some areas south of US 61, and at the three major crossings (US 61, KCS Railroad, and CN Railroad). For most of the route, the discrepancies could be accommodated by shifting the alignment approximately 50-ft. At the Kansas City Southern Railroad and US 61 crossings, the alignment right-of-way required a shift of approximately 100-ft.

These discrepancies have no significant effect on the overall design of the project. However, they did require revision of the design plans. These changes not made for the initial 30% submittal, but have been incorporated into the subject 95% submittal. Please see Figure 3-1 for an illustration of the modifications from the original alignment to the revised alignment.

As a further point of note, the channel, centered within the revised alignment, was modeled in HEC-RAS and presented to the USACE Hydraulics Section. They requested that the centerline of the channel at the various crossings be made as smooth as possible. These transitions were accommodated within the revised right-of-way alignment and thus had no effect on it.

Figure 3-1: Modifications from the original alignment



4.0 TOPOGRAPHIC, BATHYMETRIC & HYDROGRAPHIC SURVEYS

Introduction

To address the physical hydrodynamics of the Mississippi River diversion into the Maurepas swamp, URS completed an extensive data collection and analysis effort. Under Task 1, the proposed outfall area was surveyed as required to support the development of the Hydrologic Model. The methodology and results of the initial control survey conducted in 2003 are included in *Volume II, Secondary Benchmark GPS Static Survey*, which was submitted to the LDNR and the USEPA in April 2005. During Task 2, topographic and bathymetric surveys were conducted in 2007 along with a supplemental comprehensive hydrologic data collection survey. Complete documentation, discussion and data tabulations for these efforts are presented in: *Volume III, Topographic and Bathymetric Survey,* and *Volume IV, Hydrologic Data*.

Based upon the survey data, an extensive effort was undertaken to develop a detailed Digital Terrain Model (DTM) of the subject area for implementation into the Advanced Circulation Model (ADCIRC). ADCIRC is a finite element hydrodynamic model for computing flow and transport in coastal oceans, shelves, beaches, estuaries, inlets, rivers and floodplains. The following sections summarize the main points of each of the surveying tasks.

Benchmark Verification

During Task 1, the Project Team established a network of seven Secondary Control monuments in the project area using static global positioning system (GPS) survey techniques in accordance with LDNR's *A Contractor's Guide to Minimum Standards.* Five of these monuments were installed specifically for the subject work, while two were already established. The locations were adjusted following the protocol of the *Geometric Geodetic Accuracy Standards and Specifications for using GPS Relative Positioning Techniques* published by the Federal Geodetic Control Committee (FGCC). The horizontal positional accuracy between all pairs of stations in the adjusted network was better than (1:100,000). The Secondary Control points established were based on the Primary Control published in 2003 by the National Geodetic Survey (NGS) at the continuously operating reference stations (CORS) HAMM in Hammond, Louisiana and NDBC in Stennis, Mississippi.

The CORS system enables positioning accuracies that approach a few centimeters relative to the National Spatial Reference System (NSRS), both horizontally and vertically. However, the NGS coordinate data published in 2003 was based on the 1999 Geoid, which was the most accurate representation of the earth's surface in accepted use at that time. While the local surveying community and the COE was aware of the subsidence issue in southeast Louisiana and were actively working to correct the monument data prior to Hurricane Katrina, the flooding caused by the hurricane greatly accelerated that effort. Subsequently, the entire regional control network was corrected to the more accurate Geoid termed the 2004.65 Epoch, with the vertical coordinates being corrected to the North American Vertical Datum of 1988 (NAVD88). All project elevations referenced in this report are tied to the corrected LDNR Primary Network and are therefore referenced as NAVD88-LDNR.

The Secondary Control point closest to the subject project was Levee Monument Station (LMS) 4370+42.1, which was reported in 2003 to be at elevation 31.35-ft. In conducting the topographic survey in 2007, the Project Team established a corrected elevation, based on the new datum, of 31.19-ft, indicating a deviation of 0.16-ft. NGS Monument U-379 (Project Designation PO29-SM-06), located in the levee near LA 44, was selected as the best location for the Project Benchmark. The elevation published by NGS in 2003 at this point was 15.82-ft. The Project Team conducted two calibration surveys to obtain an elevation for this benchmark, resulting in elevations of 15.86-ft and 15.92-ft, on the 2003 datum. Applying the 0.16-ft correction yielded an average corrected elevation of 15.73-ft NAVD88-LDNR, which has been established as the Project Benchmark.

The relative local vertical accuracy of the secondary network is thus considered to be within plus/minus 0.06-ft, which is more than sufficient for the subject project.

<u>Topographic and Bathymetric Surveys</u>

Task 2 consisted of topographic and bathymetric survey work, which was divided into nine activities:

- Review of Existing Data and Planning
- Channel Cross Sections South of Airline Highway
- Channel Cross Sections North of Airline Highway / South of I-10
- Channel Cross Sections North of I-10
- Crossing and Culvert Surveys
- Additional Channel Bathymetry
- Additional Embankment Topography
- Swamp Topography
- Data QC and Development of High Resolution Digital Terrain Model

All project topographic and bathymetric data, along with 1-ft resolution infrared imagery and a broad range of publicly available geospatial data, were incorporated into a Geographic Information System (GIS). Previous channel survey work for the Phase 0 study by Pyburn and Odom were also converted to digital format and included in the GIS. Various other historic topographic and bathymetric data were also collected, reviewed, and incorporated as deemed appropriate.

To establish accurate conveyance properties for the extensive network of small channels within the project area, detailed surveying was performed, encompassing:

- 28 drainage cross-sections of the populated area south of US 61;

- 62 swamp channel cross-sections of the area north of US 61, covering 6 primary channels (45 miles), 17 secondary channels (29 miles) , and 25 minor channels (35 miles);
- 47 spot invert elevations in the swamp channels and bank gaps north of US 61; and
- Topographic surveys of crossings and culverts underneath Interstate 10, US 61, LA 44, and the local railroads.

URS supplemented the initial field reconnaissance with additional field inspections and spot surveying of the channel banks to obtain high resolution data on both natural and artificial (spoil) banks and bank gaps. In addition, complete walking surveys and spot inspections were performed on: 1) the low levees north of US 61, 2) the railroad embankment formerly used for cypress lumbering that divides the swamp, and 3) the berm that rims the south shore of Lake Maurepas.

In general, swamp vegetation is very dense and the swamp floor is heavily littered with detritus throughout the project area. Except for the dragline scars created from cypress lumbering in the early 20th century, point elevations tend to fall within a narrow range. The swamp channel cross-sections and spot survey shots routinely indicated that the swamp elevations range from 0.0 to 1.0 ft., with an average of about +0.5 ft. Elevations around 0.0 ft are seen in sloughs while elevations near +1.0 are common on the natural banks. This narrow range of swamp elevations is consistent with the observations made in the area by the LSU researchers during their study. Slightly higher swamp elevations are present around the lower end of Blind River and Alligator Bayou due to active deposition of fines from the Amite River Diversion Channel (ARDC). Higher elevations are also seen toward the southern margins of the swamp in the central area of the project (between US 61 and I-10), as typified by the presence of oak and palmetto vegetation.

Light Detection and Ranging (LIDAR) data was obtained for the project area from the LSU Atlas website. This information confirmed the location of higher artificial banks within the swamp areas, but did not indicate the presence of any significant additional topographic features. Based on the concurrence between the LIDAR data, discussions with local landowners and hunting guides, and the field reconnaissance surveys, the topography of the area was judged to be reasonably well defined. Therefore, swamp profile surveys were not considered necessary.

Topographic and bathymetric data for the area north of US 61 were used to prepare a high resolution DTM. Development of the DTM enabled a detailed quality control (QC) review to be conducted on the topographic and bathymetric data. Establishing an accurate depiction of the swamp geometry was an essential first step to planning for the two-dimensional (2D) modeling of the project hydrodynamics. Based on the survey data, a DTM with over 2.5 million vertices was created in Land Development Desktop (LDD), a three-dimensional terrain modeling software component for use with AutoCAD. The high resolution DTM was subsequently used to generate a lower resolution triangular irregular network (TIN) with 175,000 vertices for the development of the finite element mesh of the 2D model.

Under Task 3, topographic and bathymetric work consisted of obtaining data relevant to a pond in the batture area west of the proposed intake structure.

The survey for the pond was necessary to determine the fill volume and the depth to which the required earthern cofferdam construction would be required. The survey was performed in 2013.

<u>Hydrologic Data Collection And Analysis</u>

The hydrologic data collection and analysis work tasks consisted of seven activities:

- Review of Existing Data
- Summary of Background Information
- Planning for Hydrologic Data Collection
- Installation and Operation of Continuous Hydrologic Data Instruments
- Additional Field Hydrologic Data Collection
- Data Compilation and QC
- Analysis of Hydrologic Data

Terrain data from the *Volume III, Topographic and Bathymetric Survey* were used to develop a detailed description of the hydrographic features in the project area, including those raised features that can significantly control surface water flow patterns. Existing regional and project area hydrologic data were also used to construct a water balance.

Continuous hydrologic data were collected primarily by researchers from LSU with support from URS. Stage data was continuously recorded at 13 selected channel locations for approximately one year (November 2003 through October 2004). Velocity data was obtained using an Acoustic Doppler Current Profiler. The continuous data was supplemented by an extensive number of discrete stage and velocity measurements obtained by URS during field topographic and bathymetric investigations.

Gage data were adjusted to the project datum (NAVD88-LDNR) and corrected using additional field leveling observations and hydrograph inspections. URS estimates that the final stage data are accurate to better than plus/minus 0.1 feet. Given that the observed range in stage over the period exceeded 4 feet, an estimate of the relative error is less than 2.5 percent of the range.

URS analyzed the project area hydrologic characteristics and trends for the following parameters:

- Precipitation
- Stage Ranges
- Velocity, Flow, and Water Balance

- Water Surface Slopes
- Tidal Propagation and Channel Over-Banking
- Low Frequency Signal Propagation and Channel-Swamp Exchange Resistance

Stage hydrographs by waterbody were prepared for three critical data periods using the final adjusted data. The periods selected included:

- December 26, 2003 to January 25, 2004. This was the period used by the LSU researchers to calibrate their RMA-2 model.

- April - June 2004. The lowest water surface elevation (WSE) observed in Lake Maurepas during the study occurred during this period. The low WSE was followed by a modest flood on the ARDC and Blind River. The period also included the 3-day 5-inch Garyville rain event.

- September - October 2004. Three tropical storms occurred during this period: 1) Hurricane Ivan (as it passed to the east of Louisiana en route to a northwest Florida landfall), 2) Tropical Storm Ivan (as the regenerated storm passed through the Gulf of Mexico to the south of Louisiana), and 3) Tropical Storm Matthew, which passed directly over the project area.

Surface water gradients throughout the project area are extremely mild, consistent with the very flat topography and bathymetry. Evaluation of gradient data indicates that the typical slope of the surface water within the swamp interior is probably very low – less that 1×10^{-6}. At these very low gradients, the flow is essentially stagnant, and critical thresholds for full turbulence are unlikely to be reached. As turbulence declines, the physical mechanisms controlling water velocities and solute mixing (e.g., nutrients and salinity) require special consideration.

Under normal low stage conditions, very small amplitude tidal signals can readily propagate up the project area channels. However, as the stages rise and channel flow exchanges with the adjacent swamps, the tidal signals are lost. Over-banking then occurs in two phases: 1) as stages reach the inverts of bank gaps, limited flow is exchanged via the small openings; and 2) with further stage increases, the channels overflow their banks. After the stages fall below the bank level, the tidal signal is once again observed in the channels. Thus, the characteristics of high frequency signal propagation in the channel hydrographs reflect the elevations of the banks and bank gaps, which control the stage-dependent exchange between the channels and swamp.

Tides also propagate into the northern swamp, but are much more dampened there than in the channels. Tides do not appear to propagate into the isolated central swamp. The velocity and stage hydrographs at cross-section S-9 on the Godchaux Canal in Laplace, between the Mississippi River and US 61, just east of LA 637 were compared. The comparison shows that the "high frequency" tidal velocity and stage signals are generally in phase. Thus, tidal prism – the change in water volume covering an area between a low tide and the subsequent high tide – is not a very meaningful calculation for the Maurepas project area. There are two reasons for this: 1) Lake-driven tidal signals affect only the footprint of the interior channels, which comprises less than 10 percent of the overall interior project area, and 2) The long lag time in the tidal propagation up the interior channels means that a simultaneous change in volume within the entire channel network does not occur.

The characteristic propagation of low frequency shifts (with periods of one to several days) in WSEs through the project area is also an important aspect of the project *Conceptual Hydrologic Model*. The low frequency signatures of the system – including both the incoming (filling or wetting) and outgoing (draining or drying) phases of the events – are important indicators of several "resistance" factors which control the extent and/or rate of channel-swamp exchange:

- Bottom friction, or shear stress, in the swamp and on the banks,
- Vegetation form drag in the swamp and on the banks,
- The width, bottom friction, and drag of the gaps (which determines their conveyance), and
- The "effective" exchangeable storage volume of the interior swamp areas.

As with tidal propagation, low frequency propagation characteristics are stage dependent, indicating that resistance factors vary with water depth, which is consistent with the physical nature of shear stress, drag, and swamp storage.

The various signatures of channel-swamp exchange are indicative of the system's response to hydrologic forcing. Understanding and modeling these observed events, including quantifying the resistance factors in the swamp, enable prediction of the system's response to a diversion. Taken together, these characteristics comprise a Maurepas Swamp Conceptual Hydrologic Model. This conceptual model has been used to develop a high resolution 2D hydrodynamic model of the swamp and to evaluate the swamp circulation, retention, and water depth associated with a freshwater diversion.

5.0 GEOTECHNICAL DESIGN

A separate Geotechnical Design Report is included in Appendix C.

6.0 PROJECT DESIGN CONSTRAINTS

Introduction

The task of diverting 2,000 cfs of flow from the Mississippi River into the Maurepas Swamp in a controlled manner presents a number of technical design challenges. These include designing a gated river intake structure, a sand settling basin, and a 5½ mile long conveyance channel crossing critical infrastructure in place. In addition, the design must accommodate the existing man-made as well as natural hydrologic features of the subject area. The following sections briefly summarize each of the technical constraints to be addressed during the design effort.

Technical Constraints

The intake facilities consist of an inflow channel in the batture area between the Mississippi River and the levee, an intake structure, and gated box culverts to convey the flow from the river underneath the levee and into the conveyance channel. The inflow channel must be a very sound structure, capable of withstanding both the forces of the river current while also supporting the levee on its back-slope. It must be anchored securely and armored to prevent erosion in the inlet area. The intake structure must also provide a means of controlling the volume of flow allowed into the diversion. Three sluice gates and corresponding box culverts have been selected as the best design option for achieving this objective. The gated box culverts function not only as the conveyance mechanism, but also will be required to function as the primary flood protection system in place of the Mississippi River Levee at the proposed diversion. Their design must address the structural, hydraulic, and operational issues associated with such facilities.

The sedimentation basin must be designed to settle out large sand particles and thus prevent clogging of the conveyance channel. By its nature, the basin will require periodic cleaning; to prevent undue maintenance, the basin must be designed to store several months of sediment. Thus, the basin must have the cross-sectional area, surface area, and volume needed to both capture the entrained particulates and store them for a reasonable period of time. It must also provide access for excavation equipment to regularly remove the accumulated sediment.

The diversion channel will require a massive excavation effort as well as large amounts of embankment materials for construction of the guide levees. The earthwork operations must both remove unacceptable materials and also place structurally sound materials into the guide levees, all while minimizing the impact to the surrounding environment. Geotechnical slope stability analyses have indicated that the proposed side slopes must be as flat as 5H:1V for the guide levees north of US 61 and 3H:1V for the guide levees south of US 61. The available right-of-way width in which the channel is to be built is limited to 300-ft; both the channel and its guide levees must fit within this footprint.

While the goal is to restore the wetlands, the design is still constrained to maintain effective drainage throughout the project area. Since the proposed guide levees of the conveyance channel will close off the area where the Bourgeois Canal discharges into the Hope Canal, the design must provide for the restoration of the drainage currently conveyed by these waterbodies. The proposed method to achieve this is to construct a drainage pump station to transfer flow from the canals into the proposed channel.

Environmental Constraints

As stated above, the design must minimize the impact on the wetlands during construction. It must also incorporate measures to insure that the energy of the discharging water is dissipated to prevent erosion. The discharge velocity must also be reduced sufficiently to insure that the sedimentation of the fines that is needed to replenish the wetlands occurs. Finally, provision must be made for alternately inundating the wetlands and then allowing them to drain to avoid creating open water, which would destroy the natural flora, thus defeating the purpose of the project.

The design must also provide protection for the native fauna. This is especially important for the local endangered species, which include the Pallid Sturgeon, the Bald Eagle, and the Manatee. The sturgeon are primarily bottom feeders, thus the intake structure design has a minimum elevation constraint to avoid disturbing them. The intake structure must also be designed so that the velocities are minimized in order to prevent small riverine species from being inadvertently swept into the diversion. Rip-rap and concrete bottom materials are also dictated for the front channel of the intake structure, since fish and other aquatic creatures tend to avoid non-natural materials. Construction operations must also be avoided around those areas which have eagle nests during their nesting seasons.

Transportation Infrastructure Constraints

The conveyance channel will cross several infrastructure systems, including three roadways and two railroads. Three of the crossings will require box culvert installations, one bridge, and for I-10, which is elevated and has sufficient area beneath it to fully accommodate the design flow. The design must allow for the maintenance of transportation access throughout the construction process as well as afterward. The design of detour plans for US 61 and LA 44 (River Road) will have to be submitted to the LDOTD for approval prior to construction. US 61 is a primary evacuation route during hurricane season (June through November) and thus all of its lanes must remain open to traffic during this period. The Canadian National Railroad has indicated that the design should incorporate the use of “shoo-fly” as the preferred detour alternative for their rail traffic. The Kansas City Southern Railroad indicates that they would require a bridge, but should be built with minimum outages.

Pipelines & Infrastructure Constraints

The proposed diversion channel intersects six major utility rights-of-way en route from the Mississippi River to the Maurepas Swamp: 1) LA 44, 2) Canadian National Railroad, 3) Kansas City Southern Railroad, 4) Entergy electrical transmission corridor, 5) US 61, and 6) A major petroleum and gas pipeline corridor. The utilities were identified from field reconnaissance, survey documents, and information provided by the utility companies. The design must address the relocation of each of the active utility lines which cross the proposed conveyance channel alignment.

Timely restoration of service will be a key component of a successful execution of the project. The various utilities cover a broad range, including: high energy electrical power transmission lines; complex and fragile fiber-optic telecommunications cables; high pressure liquid petroleum and natural gas pipelines; pressurized hydrogen, oxygen and nitrogen lines; and common water and sewer pipes. Each of these presents its own technical challenge to insure that the utility is relocated quickly, yet safely, and reconnected properly for return to service. The design must insure that each is placed in a location that provides the proper depth of cover and separation from other utilities. They must also be designed for installation in an almost fail-safe manner, since future maintenance opportunities underneath the conveyance channel will be severely limited.

Fiscal Constraints

This is a significant public works project involving very large expenditures for its construction, which must be justified based on the greater good achieved by the restoration of the Maurepas Swamp. The design has been constrained to using proven technologies that are fiscally conservative wherever possible. The costs can be broken down into five distinct components: 1) Site Work, 2) Concrete, 3) Railroad and Roadway, 4) Utility Relocation, 5) Equipment, Electrical, and Instrumentation. Each of these items are explained in detail in Section 10. The major design constraints associated with each are briefly summarized below.

The most costly tasks under the site work component include clearing the 5½ mile long by 300-ft wide conveyance right-of-way along with the earthwork operations involved in excavating the channel and building the embankment levees. The most efficient construction practice is to use the excavated material for the embankment, where possible. However, much of the existing soil is not suitable for that purpose and will have to be removed and replaced with borrow material from off-site. This substantially increases the project cost, but is an unavoidable physical condition that creates a fiscal constraint. Site work also include ground improvements for levee construction at the pump station. The challenge for levee construction at this location is long term maintenance issues due to anticipated settlement of levee, if built in wet. Ground improvements in this location will result in reduced settlement issues there by reducing maintenance needs.

A great deal of concrete will be poured on-site to construct the intake structure, its u-channels, and its foundations. Along with the cost of supplying the cement, sand and aggregate materials comes the cost of mixing and placing the concrete. This includes a large quantity of reinforcing steel, along with the requisite form-work, sheeting, shoring and bracing. The design must incorporate the use of local materials, where suitable, and insure that the structures not only function correctly, but are also designed with the idea of constructability in mind.

The work associated with re-routing two major roads and railways includes building temporary by-passes, removing the existing track-work and pavement, constructing the large reinforced concrete box culverts required, and then rebuilding the roadways and rail networks on top of them. When a major transportation route is out of service, time is of the essence, so costs can quickly escalate when change orders have to be issued and carried out in short order. Thus, the key design parameter to restrain the budgetary expenditures in this portion of the project is to obtain as much information as possible on all aspects of each crossing so that no major “surprises” occur during construction.

Relocating existing utilities is also a time critical task, which can cause costs to escalate quickly should unexpected difficulties arise. Again, knowledge of the site conditions is the most important design factor in keeping costs under control. Also, since these utilities include not just water, sewer, electrical transmission, and fiber optic lines, but also high pressure natural gas, liquid petroleum products, and even compressed hydrogen, safety must also be of paramount consideration in the design process.

The equipment, electrical and instrumentation associated with operating the sluice gate and the canal pump station are also very costly elements in the overall budget. The primary cost-cutting measure to be taken in the design phase for these items is to use proven technology. Much of the gate operational equipment can be optimized by applying the lessons learned from the Davis Pond project. The drainage pump station design can be based on innumerable successful examples. Similarly, the flow monitoring and water quality analysis equipment also has a historical database to draw upon for its cost efficient design.

7.0 UTILITY MODIFICATIONS

Introduction

The proposed alignment for the Maurepas freshwater diversion channel stretches 5½ miles from the Mississippi River to deep within the Maurepas Swamp. Due to the length of the proposed construction, the channel intersects numerous utility and industrial product pipeline rights-of-way. In order to construct the channel, these utilities and other infrastructure components must be relocated to positions that will not adversely affect the construction process.

Infrastructure & Utility Company Contacts

In Task 2, the Project Team issued requests to each of the utility and industrial companies with services in the area of the proposed diversion to provide the means by which they preferred their utilities/lines to be relocated. The options included: "By Owner" or "By Contractor" and "Prior To" or "Concurrent With" construction. The companies have been directed to coordinate with the LDNR in regards to reimbursement for the relocation expenses that they will incur.

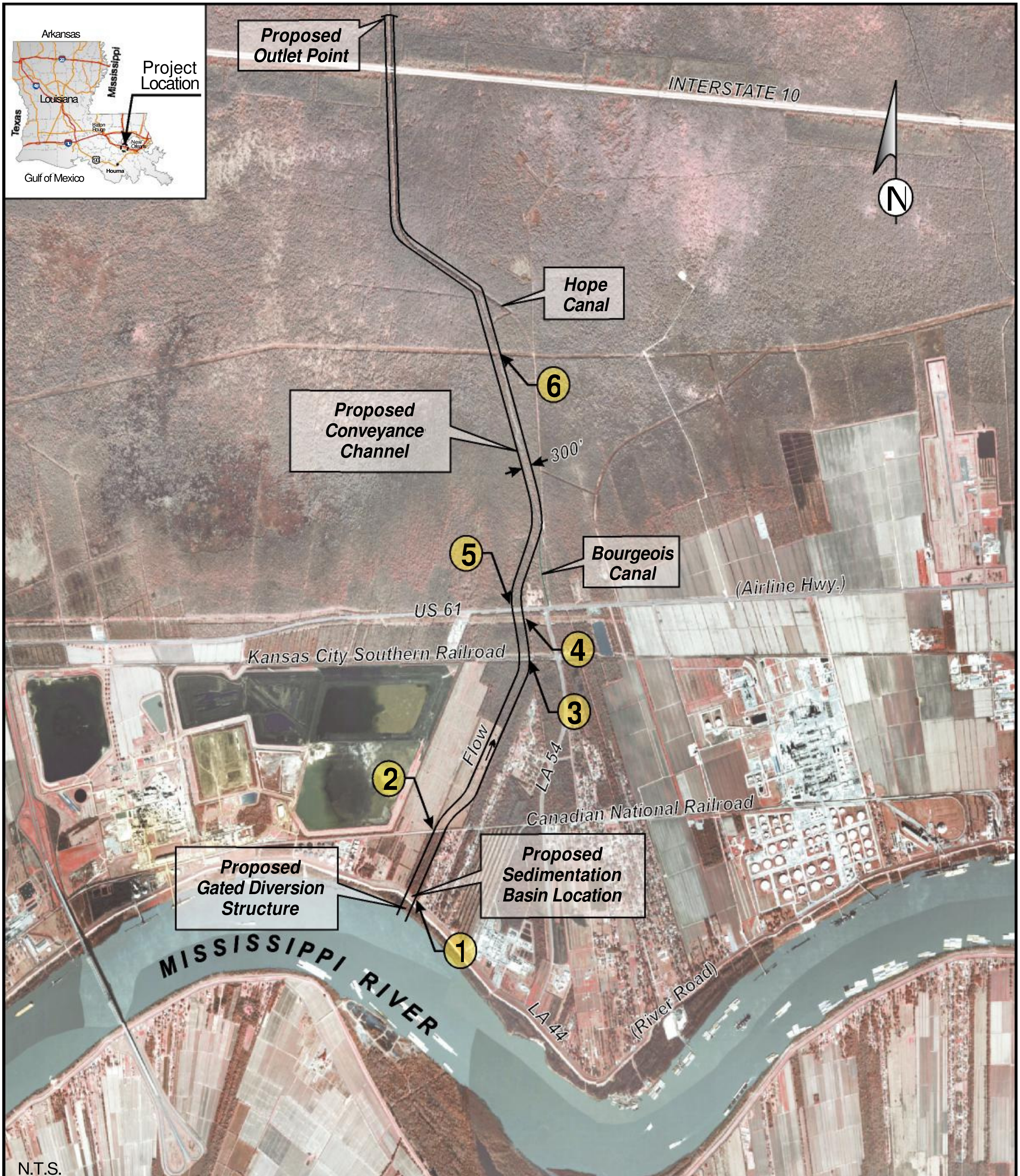
The utilities were identified from field reconnaissance, historical surveys, maps, and records, as wells as information provided directly from the utility and industrial companies. A list of the known companies which have services and product lines within the proposed right-of-way corridor follows this summary.

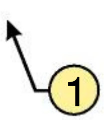
There are six key locations along the proposed conveyance channel right-of-way that have numerous utilities and/or product lines which intersect the proposed alignment. These locations include, but are not limited to, LA 44, US 61, Kansas City Southern Railroad, Canadian National Railroad, I-10, and a major pipeline corridor which runs between US 61 and I-10. Figure 7-1 displays the overall route of the proposed channel alignment, and highlights the potential utility and other infrastructure conflict areas.

The Project Team contacted all companies with identified utilities or product lines in the proposed alignment right-of-way via telephone on several occasions from November 2007 to September 2013. In those conversations, URS described the project and established a point of contact within each company for coordination of potential utility/pipeline relocations necessitated by the proposed channel alignment. In Task 2 preliminary design packages containing a letter describing the project, figures displaying the location of the proposed channel alignment, as well as plan, profile, and cross-section drawings were mailed to each potentially affected company between January 22, 2008 and January 28, 2008. Each company was requested to provide utility/pipeline offsets, depths of cover, and other pertinent requirements, along with any drawings detailing how their utilities and/or product lines are to be relocated.

During Task 3, URS contacted each of the utility companies to re-establish contacts and confirm the dispositions. For most utility companies the point of contacts have changed and required updated information. URS coordinated with the utility firms and developed the relocations as depicted in the 95% Design Drawings. It was observed that in most cases the utility firms were interested to know about the anticipated construction date.

All utility information provided by the survey data, field visits, or from the impacted companies is summarized in Table 7-1. Plans displaying the estimated utility/pipeline dispositions are based on approximate depths; the exact locations will have to be field-verified prior to final design and construction.





Utility Crossing area

Utility Conflict Location Map

Figure 7-1

Table 7-1 Utility Disposition and Modifications

Company	POC	Phone #	Address	Email	Area	Utility Description	Disposition	Relocation By	Minimum cover required
Comcast (Time Warner)	Larry Landry	985-637-2868	104 Lois Rd. Houma, LA 70363	Larry_Landry@cable.comcast.com	1	Above ground feeder line along river rd.	Entergy will relocate poles, Comcast will relocate feeder line	Comcast (after Entergy Poles have been relocated)	A Minimum of 16' above the height of the roadway is required by the state
Enterprise (Acadian)	Enterprise (Liquid): Justin Chauvin Acadian (Gas): Brian P. Giroir	Enterprise Office: 225-675-2510 Acadian Office: 985-493-4619 Acadian Cell: 985-414-2824	PO Box 337 Sorrento, LA 70778	Enterprise: jechauvin@eprod.com Acadian: bpgiroir@eprod.com	1	4" Nat Gas Pipeline along River Rd. (Acadian)	Relocate prior to construction	Enterprise (Acadian)	minimum 5' of cover
					3	6" active, high pressure liquid petroleum (butane)			
						6" active, high pressure liquid petroleum (propane) (Enterprise)			
						8" active, high pressure, liquid petroleum (Enterprise)			
					4	12" pipeline (12.75") (Enterprise)			
Entergy Distribution	Chris Gilliland	Office: 985-549-6903	2200 W. Church St. Hammond, LA 70401	cgillil@entergy.com	1	13.8 kV 3 phase Distribution Line w/ poles along River Rd.	Relocate Poles currently in ROW prior to construction	Entergy	Coordinate the required height of the line
						34.5 kV, 3 Phase parallel to US 61 (w/ walkway)			
Reserve Communication	Barry Firmin	Office: 985-536-1111 Cell: 985-966-3476	203 West 4th St. Reserve, LA 70084	b.firmin@rtconline.com	1	Overhead Line on Entergy Poles	Relocate to new Poles prior to construction	Reserve Communications (after Entergy Poles have been relocated)	22' over state Highway 18' over roadway
						U/G Line	Relocate from U/G to overhead on new Poles prior to construction		
					5	Overhead Line on Entergy Poles	Relocate to new Poles prior to construction		
Level 3 (Wiltel)	Adam Edwards	Office: 720-888-4518	1025 Eldorado Blvd Broomfield, CO 80023	Adam.Edwards@level3.com, level3.networkrelocations@level3.com	3	U/G fiber-optic line	Restore Line to Original condition, provide required bracing and temorary measures to remain during construction.	Contractor incoordination with Level 3	
					5	U/G fiber-optic line, parallel to US 61			
AT&T Fiber-Optic (Long Distance)	Ricky B. Howard	Office: 214-821-9846	3910 San Jacinto St. Rm. 4 Dallas, TX 75204	rh1854@att.com	2	U/G fiber-optic	Restore Line to Original condition, provide required bracing and temorary measures to remain during construction.	Contractor incoordination with AT&T	
Gulf South	Jeff Pendleton	Cell: 225.236.8260	520 Alliance St. Kenner, LA 70062	jeff.pendleton@bwpmlp.com	3	24" Natural Gas Line (active)(high pressure)	Relocate prior to Construction	Gulf South	6' below the bottom of the canal
						18" abandoned pipeline			
						16" abandoned pipeline			
MCI (Verizon Business)	Mike Williams	218-901-3324	2400 N. Glenville Richardson, TX 75082		3	U/G fiber-optic line, along KCS railroad and south of Airline Hwy	Restore Line to Original condition, provide required bracing and temorary measures to remain in service during construction.	Contractor incoordination with MCI	24" vertical & 60" horizontal clearances
Shell (Bengal & Colonial)	Jamie Honses (or Terri Howel)	Office: 504-728-4340	701 Poyrdas St., Suite 4146 New Orleans, LA 70139	jamie.honses@shell.com	4	24" High Pressure Line	Relocate prior to Construction	Shell	7 ft. cover, 2 ft. from other lines
						12" Nitrogen line (idle)			
					6	24" high pressure line			
						pipeline			
Marathon	Jeff Erwin, Land Agent	Cell: 270-925-1978	5555 San Trevino Houston, TX 77056	jaerwin@marathonoil.com	4	20" product line just S. of US 61	Relocate prior to Construction	Marathon	ABSOLUTE MINIMUM of 36" of cover. Marathon will review proposal for relocation of line
	Ricky Landry, ROW	Office: 225-654-8854, ext. 204	Box 922 Hwy 61 Jackson, LA 70748			30" Crude Line			
Entergy Transmission	Jimmy Sholar	Office: 504-390-9414	100 Harimaw Ct. West Metairie, LA 70001	jsholar@entergy.com	4	Just south of U.S 61 has 2 large metal support strucures (poles) w/in ROW, supports 2 lines (230 kV and 115kV)	Entergy may add a line on these poles. If so, Entergy will move poles outside of ROW when they do so.	Entergy	230 kV Lines must have an ABSOLUTE minimum height over top of levee of 28' 115 kV Lines must have an ABSOLUTE
						Line on South side of CN railroad (230 000 kV)	Do not disturb		
St. John The Baptist Parish	Virgil Rayneri	Main: 985-652-9569 Cell: 504-234-5136	1801 W. Airline Hwy LaPlace, LA 70068	v.rayneri@sjbparish.com	5	Water Line, North side of Airline Hwy	Relocate during Construction	Contractor	Per Parish approval
						Sewer Line, North side of Airline Hwy			
Air Products	Sidney Cavalier	Cell: 225-715-7423 Office: 225-677-5658	36637 Hwy 30 Geismar, LA 70734		6	12" Carbon stell pipe w/ casing. High Pressure Hydrogen Line, between I-10 and Us. 61 (API 5L x42, wall thickness = 0.219")	Relocate prior to Construction	Air Products	5'
Chevron (Texaco)	Larry Tabor Kieth Esserman	Office: 985-758-0231 Cell: 504-415-8386 Office: 985-758-0207 Cell: 504.415.0587	15849 Old Spanish Trail, Hwy 631 Paradis, LA 70080	LJ.Tabor@Chevron.com k.eserman@chevron.com	6	20" Natural Gas, 10 ft. Corridor between I-10 and US. 61	Relocate prior to Construction	Chevron	4' of cover 2' separation
						6" HVL (Propane, propylene, butane), 10 ft. Corridor between I-10 and US. 61			
						6" HVL (Propane, propylene, butane), 10 ft. Corridor between I-10 and US. 61			
Air Liquide	Robert Hracek	Office: 225-685-4282 Cell: 225-268-8743	57805 Evergreen Rd. Plaquemine, LA 70764	robert.hracek@airliquide.com	6	12 carbon steel line, between I-10 and U.S. 61	Relocate prior to Construction	Air Liquide	To Be Confirmed by Air Liquide
						13 carbon steel line, between I-10 and U.S. 61			
						3' depth Oxygen Line, between I-10 and U.S. 61			
						4' depth Nitrogen Line, between I-10 and U.S. 61			

8.0 HYDRAULIC MODEL

Introduction

At the request of LDNR and EPA, the Project Team designed the Maurepas diversion to operate at 2,000 cfs for as much of the year as possible. A hydraulic model was developed for the 29,000-ft conveyance channel from the Mississippi River intake to 1,000-ft north of I-10. The model was used to simulate a steady-state, sub-critical flow condition to verify the hydraulic performance of the proposed channel design. The Rating Curve developed during Task 1 was used to correlate the rate of discharge from the channel to the tail-water elevation in the Maurepas Swamp. As a result, the design operating condition for the waterway was designated as 2,000 cfs of flow with 5- to 6-ft of head loss from the intake to the discharge. This must include the head losses incurred through the following five structures: 1) intake structure, including the crossing under LA 44, 2) culverts at the CN Railroad, 3) bridge at the KCS Railroad, 4) culverts at US 61, and 5) existing bridge piers at I-10.

Task 1 Model

Under the Task 1 initiative, the Project Team conducted a Hydraulic Feasibility Study to model the physical hydrodynamics of the proposed diversion. The objectives of the study were to determine: 1) the impact on the Garyville/Reserve drainage system, 2) the distribution of flow throughout the swamp, 3) the hydraulic retention time of the diverted water, and 4) the velocity of the moving water.

To estimate the impact on the local drainage system, a model was constructed using the EPA Storm Water Management Model (SWMM) program. The area modeled was divided into nearly 2,000 individual links (channel segments) and 23 storage areas, to simulate the various flow regimes predicted. The geometry of the model was based on the available topographic and bathymetric survey data. Runoff from rural sub-catchments was modeled using the Soil Conservation Service method, while more urban sub-catchments were addressed using the SWMM model subroutine based on size, width, slope, and imperviousness. This model was used to estimate drainage flow rates and water surface elevations within the drainage network under various rainfall inputs and tail-water elevations. The study determined that a diversion of 2,000 cfs of river water through the proposed conveyance route is hydraulically feasible.

This SWMM model was subsequently coupled with a two-dimensional finite element hydrodynamic model called Advanced Circulation (ADCIRC), which computes flow and transport in coastal areas, estuaries, inlets, rivers and floodplains. The combined model represented the two-dimensional physical hydrodynamics in the project area, including channel flow, propagation of tidal signals, over-bank flow, flow through bank gaps, and swamp circulation during various conditions. The objective was to compare the performance of the drainage network for a 10-year rainfall event with and without the proposed diversion. The model performance was then evaluated through a series of calibration and validation simulations, which demonstrated the model to be appropriate for a feasibility-level analysis of diversion alternatives in meeting the design objectives.

The model corroborated that the reintroduction of the Mississippi River into the Maurepas Swamp via the Hope Canal was technically feasible. Based on the model's results, the Design Team issued several recommendations to be incorporated into the final design, as discussed within Section 2 - Prior Studies of this report.

One Dimensional Hydraulic Model

The US Army Corps of Engineers' (USACE) Hydraulic Engineering Center River Analysis System (HEC-RAS, v.3.1.3) was chosen for the development of a one-dimensional computer model of the diversion project. The HEC-RAS software allows for one-dimensional computer modeling of both steady- and unsteady-state conditions for sub-critical, super-critical or mixed flow conditions. The subject simulation would be for the steady-state sub-critical flow condition.

For these conditions, the simulation applies the energy equation for sub-critical flow assuming steady flow rates with respect to time. The calculations begin at the downstream end, using an initial water surface elevation determined from the discharge rating curve assuming a known flow. The velocity head is then calculated based on the flow rate and fluid cross-sectional area and the energy grade line elevation is determined at the discharge point. The program then works upstream, establishing the energy grade line for each successive section of channel. Frictional head losses are calculated using Manning's equation plus contraction/expansion losses proportional to changes in the velocity head between sections. The water surface elevations are determined by subtracting the respective velocity heads from the energy grade line at each point.

The program continues to work upstream in the above manner until a structure is reached. For each bridge, both the energy and momentum equations are computed; the highest head loss calculated by either of the two is then applied. For each culvert, an evaluation is made as to whether it is under inlet or outlet control. For the subject application, outlet control governs since all of the culverts are fully submerged. In outlet control conditions, exit losses are calculated based on the change in velocity head at the culvert discharge. Manning's equation is again used to calculate the frictional losses through the culvert, and the entrance losses are calculated based on the velocity head within the culvert. For each inline structure (e.g., gates and weirs), the losses are calculated based on the depth of flow. If the inline structure is overflowed, the losses associated with weir flow are calculated. If portions of the flow are going through the structure via a gate opening, the opening is evaluated for its degree of submergence. For this application, the gates at the intake structure operate under full submergence; therefore the gates simulate orifices and the losses through them are calculated using the orifice equation. This method of calculating losses is repeated for each consecutive channel section, crossing structure, and inline structure until the upstream end of the channel is reached.

River Stage & Channel Flows

The USACE, New Orleans District, maintains detailed records of the Mississippi River stages at various locations along the river. The closest recording point to the project site is the Reserve Gage (Gage ID 01260), which is located at river mile 138.7 in Reserve, LA. The Maurepas diversion channel is to be constructed near Garyville, LA, which is near river mile 144, or approximately 5.3 miles upstream from the Reserve Gage.

Historical data ranging from January 1, 1953 thru March 15, 2008 was analyzed for the Reserve Gage. The time period covers 20,162 days and the record contains 17,553 recordings, which is 87% complete. The typical river stage was extracted from the record for each of the days in the dataset and then the values were tabulated by month. Table 8-1 lists the low, average, and high river stage for each month based on the 55 years of recorded data. Low stage is defined as the average stage minus 1 standard deviation, which would comprise the 16th percentile under a normal distribution. High stage is defined as the average stage plus 1 standard deviation, which would typically correspond to the 84th percentile. By definition, the average stage would represent the 50th percentile in a normal distribution.

Month of Year	Low River		Average River		High River	
	Low Stage	Percentile	Average Stage	Percentile	High Stage	Percentile
January	3.81	26th	9.43	60th	15.04	80th
February	5.82	43rd	11.10	66th	16.37	85th
March	8.72	57th	13.44	74th	18.16	92nd
April	9.67	60th	14.60	79th	19.52	95th
May	8.15	55th	13.56	75th	18.97	94th
June	5.06	38th	10.68	64th	16.30	85th
July	3.01	15th	6.81	49th	10.61	64th
August	1.71	2nd	4.57	34th	7.43	52nd
September	2.22	5th	4.01	28th	5.80	42nd
October	1.97	3rd	4.49	33rd	7.00	50th
November	1.90	2nd	5.09	38th	8.28	55th
December	2.68	10th	8.24	55th	13.80	75th
Annual	**3.07**	**15th**	**8.89**	**57th**	**14.70**	**79th**

Table 8-1: Low, Average and High Stages for the Mississippi River at the Reserve Gage.

Figure 8-1 presents the recorded data from the Reserve gage dataset normalized to an annual basis, which illustrates the number of days that the Mississippi River has historically achieved a given stage at the recording station. Since the station is only five miles from the proposed Maurepas diversion intake, the graph is a reasonable approximation of the river stage at the project site over the last five decades.

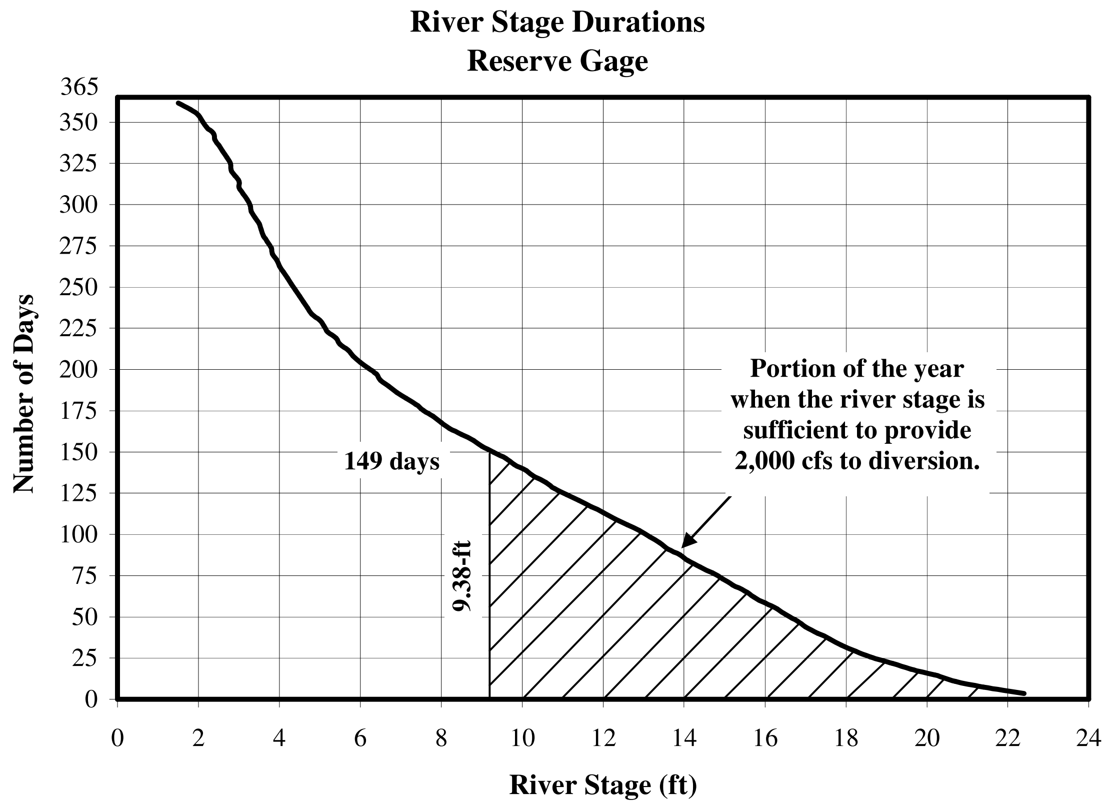


Figure 8-1: Expected number of days the river will meet or exceed a given river stage at the Reserve Gage.

Table 8-2 provides the required Mississippi River stage and the expected number of days the river will meet or exceed that stage in an average year, based on data from the Reserve Gage, for the very low flow, low flow, design flow, and high flow conditions.

Flow Condition	Flow (cfs)	Mississippi River Stage Required (ft)	Historical River Stage Percentile	Average No. of Days Exceeding Stage per Year
Very Low Flow	1,000	4.92	37th	231
Low Flow	1,500	6.69	48th	191
Design Flow	2,000	9.38	59th	149
High Flow	2,250	10.83	65th	128

Table 8-2: Stage requirements and the expected duration for various flows.

Model Results

The design flow of 2,000 cfs in the channel requires a river stage of 9.38 ft NAVD88-LDNR. Figure 8-1 illustrates that, if historical trends hold, the water surface elevation in the river at the proposed intake structure will be at or above 9.38-ft for an average of 149 days per year, or approximately 40% of the time.

Figure 8-2 depicts the water surface profile (WS PF8) for the diversion channel generated by HEC-RAS during the design flow condition (the 7 simply denotes that the 2,000 cfs was the 7th condition modeled). The profile also plots the channel invert, channel banks and levee elevations. Also shown are the locations of all of the major structures as well as the level of allowable sediment fill in the sediment basin.

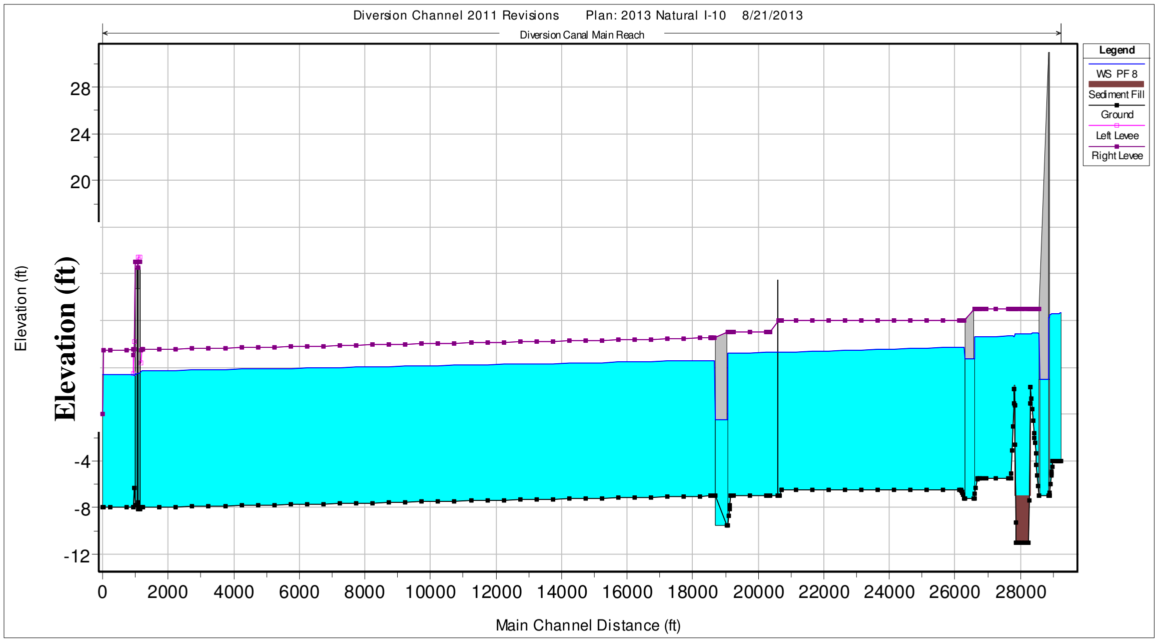


Figure 8-2: The diversion channel HEC-RAS model water surface profile.

Figure 8-3 provides the stage-discharge relationship for the diversion channel under the free flow condition; the free flow condition is where the gates are open fully.

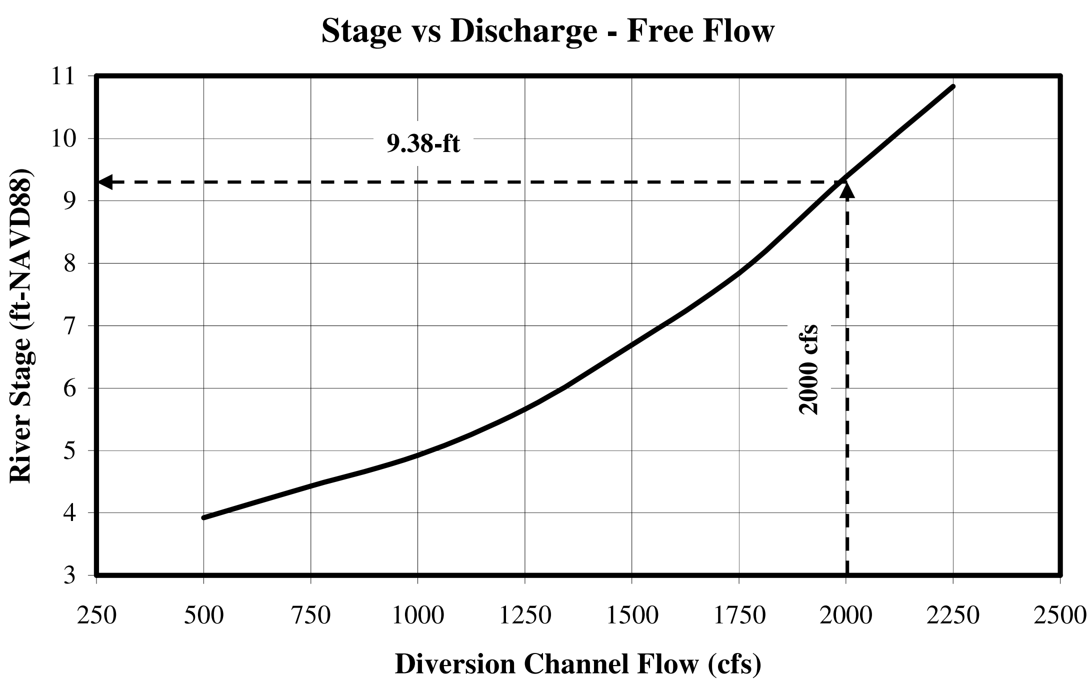


Figure 8-3: Stage-discharge relationship for the free flow condition.

Figure 8-4 shows the minimum number of days in an average year that various flows are expected in the proposed diversion channel.

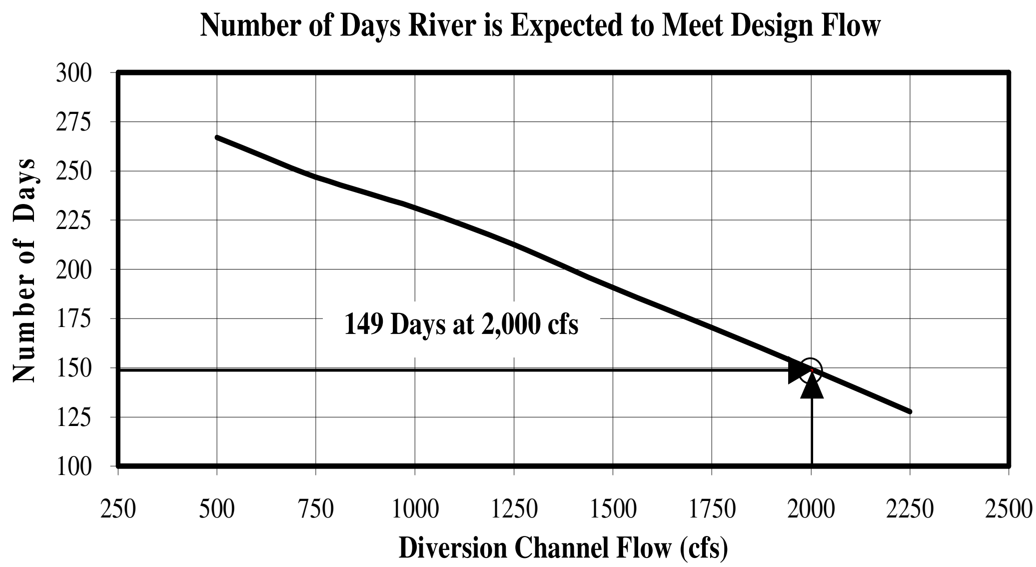


Figure 8-4: Days river is expected to meet or exceed the design flow of 2,000 cfs.

Figure 8-5 shows the low, average and high flows expected for each month based on the stage data provided in Table 8-2.

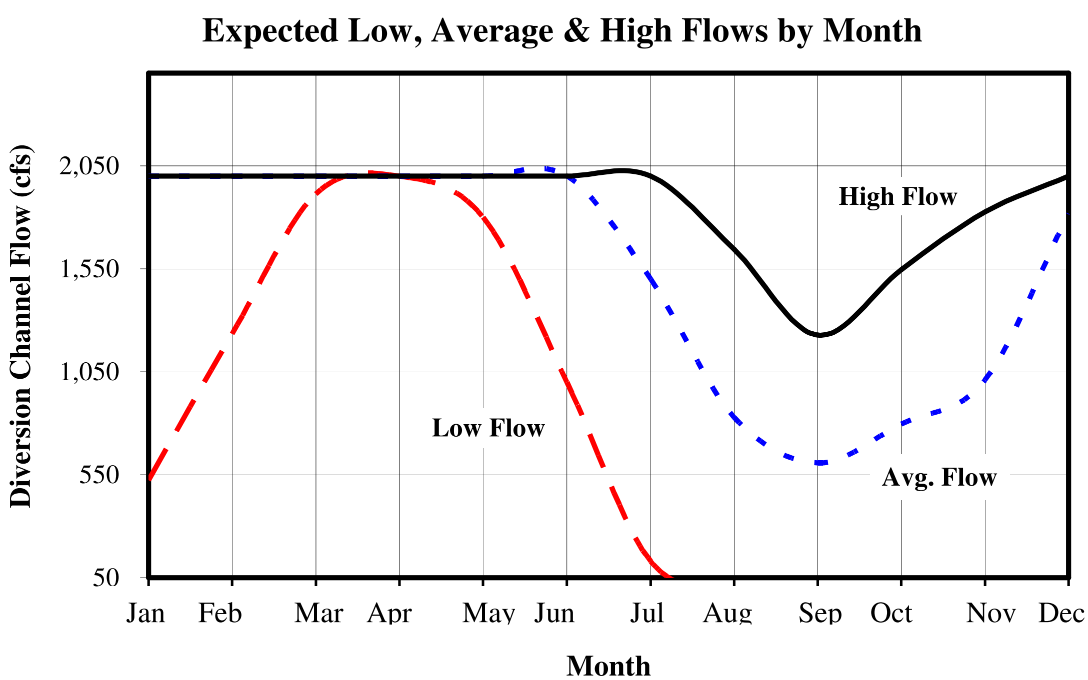


Figure 8-5: The projected low, average, and high flows for each month for an average year.

Table 8-3 provides the required river stage for gate openings in one foot increments and the expected number of days for each gate opening height.

Gate Opening Height (ft)	Mississippi River Stage Required (ft)	Historical River Stage Percentile	Average No. of Days Exceeding Stage per Year
10	9.38	59th	149
9	9.64	60th	145
8	9.99	61st	141
7	10.51	64th	132
6	11.31	67th	122
5	12.41	70th	108
4	13.59	75th	93
3	15.61	83rd	64

Table 8-3: Mississippi River stage requirements for various gate openings & the number of days the river will meet/exceed the stage in an average year.

Figure 8-6 provides the river stage and gate operation chart to maintain a flow of 2,000 in the diversion channel.

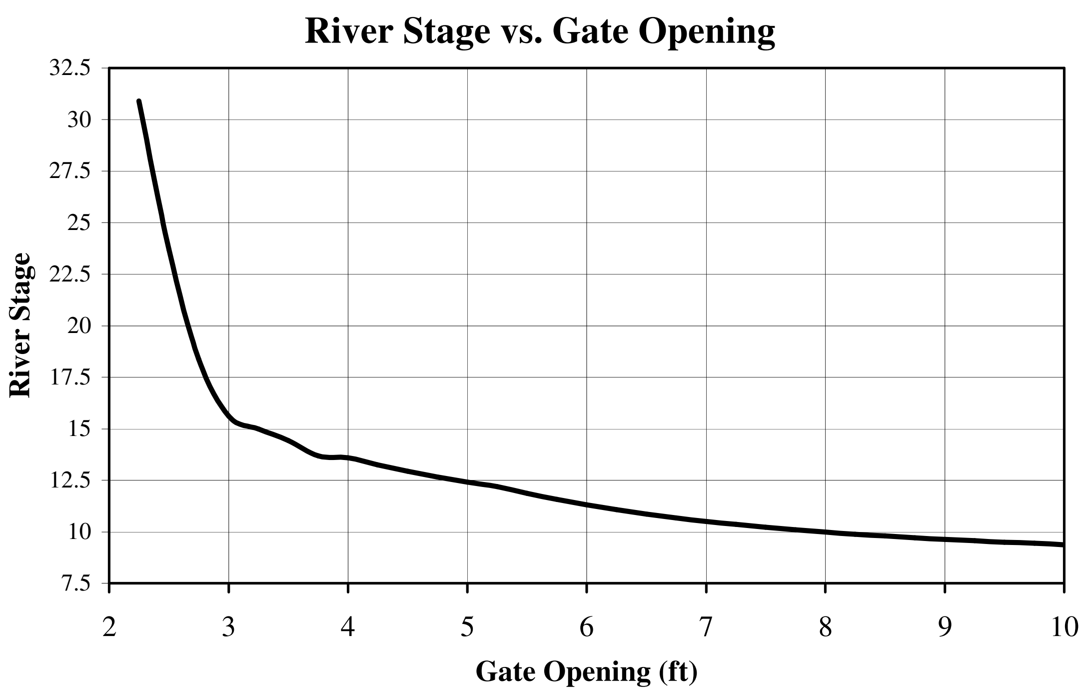


Figure 8-6: River stage versus the gate opening required to maintain a flow of 2,000 cfs in the diversion channel.

Figure 8-7 provides the expected levee freeboard for the design flow of 2,000 cfs and the high flow of 2,250 cfs. The levees were initially set to provide one foot of freeboard for the design flow and to contain the high flow.

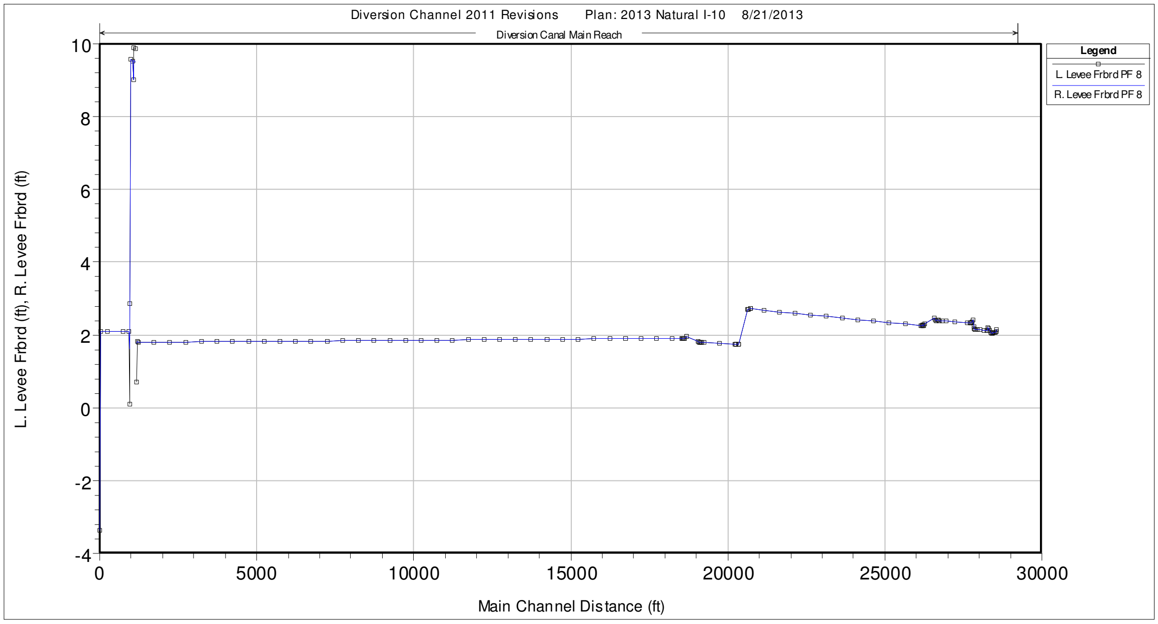


Figure 8-7: Diversion channel levee freeboard for the design flow (2,000 cfs)-vs. distance from downstream boundary (Maurepas Swamp)

Figure 8-8 shows the average channel velocities for the diversion channel flow rates. The target velocity for the design flow rate was 2.0 feet per second.

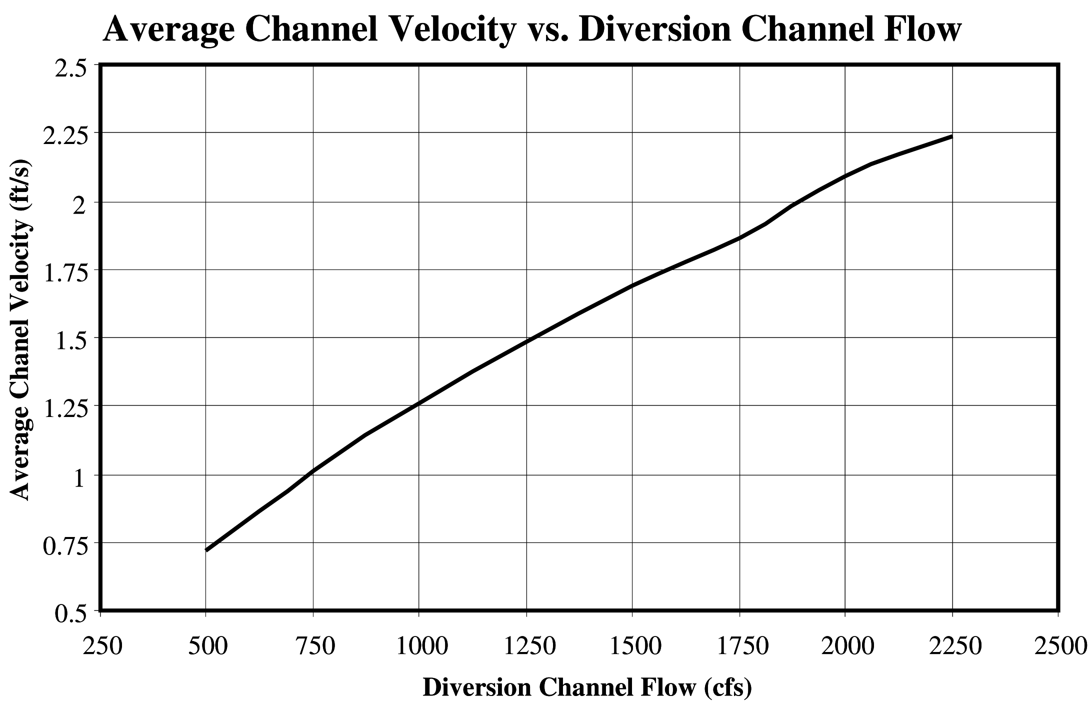


Figure 8-8: The velocity versus discharge relationship graph.

Model Assumptions

Since the channel will be of new construction, there are no existing conditions, so the hydraulic input parameters for the model cannot be calibrated to actual performance data. Thus, the parameters will have to be assumed. The one-dimensional HEC-RAS model requires hydraulic parameters and coefficients to be provided for each channel cross-section, inline structure, culvert and bridge. The parameters and coefficients applicable to this project include:

Open Channel Friction Coefficients
Open Channel Contraction & Expansion Coefficients
Channel Bank Stations
Channel Ineffective Flow Area Stations & Elevations
Channel Geometry
Channel Levee Stations & Elevations
Weir Elevations and Weir Coefficients
Gate Discharge & Orifice Coefficients
Culvert Friction Loss Coefficients
Culvert Entrance & Exit Coefficients
Bridge Pier Drag & Shape Factors

The above coefficients and cross-sectional data are used by the various HEC-RAS program algorithms as it processes each cross-section. The coefficients are entered directly into the head loss equations. The program provides several options for entering the cross-sectional data, which can be utilized to divide up or restrict the cross-section flow area(s) as needed to better simulate the conditions being modeled. These options allow the modeler to instruct the programming routines on how to handle each cross-section.

Manning's *n* Coefficients

The HEC-RAS program uses Manning's equation (Equation 8-1) for open channel flow to calculate the quantity of water conveyed given the input parameters of the conduit.

$$Q = \frac{1.486}{n} \times A \times R^{2/3} \times S^{1/2}$$ **Equation 8-1**

Where:
Q = Quantity of flow per time, (ft^3/s)
n = Manning's coefficient, (--)
A = Area of conveyance, (ft^2)
R = Hydraulic radius, ($R = A/P_w$)
and P_w = Wetted Perimeter, (ft)
S = Slope of the water surface, (ft/ft)

Using Manning's equation, the HEC-RAS program solves for S, the slope of the water surface (also known as the friction slope). This slope is then multiplied by the channel length (L) between two sections of a given reach to calculate the head loss between them (see Equation 8-2).

$$H_L = L \times \left[Q \times \frac{n}{1.486 \times A \times R^{2/3}} \right]^2$$ **Equation 8-2**

The coefficient, n, used in Manning's equation for open channel flow, represents the frictional resistance of the channel being modeled. It is one of the primary parameters that require the most interpretative judgment from the modeler to accurately represent the physical situation being modeled. Since n is inversely proportional to the flow rate, the smaller the value of n, the greater the flow; the selection of the proper value of n is essential to the construction of an accurate model. Manning's n values are empirically derived and are tabulated based on the characteristics of the conveyance channel.

For the main diversion channel, the n value of 0.03 was selected. For those lined with heavy gravel rip-rap both before and after structures, the typical value of 0.033 was chosen, which represents the significantly greater resistance to flow that these rough surfaces present. For sections where the rip-rap is only used on the bottom and the sides are dredged earth, 0.033 will be used for the bottom portion and 0.03 for the sides. Areas of cast-in-place reinforced concrete are considerably smoother; an n value of 0.017 was selected for these areas, assuming rough wood forms will be used for their construction (this is the worst-case scenario). Portions of the conveyance channel outside of the banks are rougher than the dredged earthen portions, but not as rough as the rip-rap areas; they were given an n value of 0.035, representing short grass with few weeds.

Expansion & Contraction Coefficients

HEC-RAS appends the head loss calculated from the Manning equation with an additional head loss term to account for the losses incurred by the contraction and expansion of the channel. These losses are a function of the change in velocity head between two adjacent cross-sections. The contraction/expansion head loss between cross-sections (where A is the current cross-section and B is the cross-section immediately downstream) is calculated as follows:

$$H_L = K \times \left[\frac{v_B^2}{2g} - \frac{v_A^2}{2g} \right]$$ **Equation 8-3**

Where:
H_L = Head loss between cross-section A and B, (ft)
K = Contraction/expansion head loss coefficient (--)
v_A = Velocity of flow at cross-section A, (ft/s)
v_B = Velocity of flow at cross-section B, (ft/s)

When the velocity head increases in the downstream direction, the contraction coefficient is used; when the velocity head decreases in the downstream direction, the expansion coefficient is used. Table 5 provides a listing of typical contraction and expansion coefficients. (A sensitivity analysis was conducted for the typical values and the results are presented at the end of this section of the report.)

Description of Transition Section	Contraction Coefficient	Expansion Coefficient
Gradual Transitions	0.10	0.30
Typical Bridge Transitions	0.30	0.50
Abrupt Transitions	0.60	0.80

Table 8-4: Typical contraction and expansion coefficients used in HEC-RAS.

Gradual transition values are used for the typical shift from one cross-section to the next in the conveyance channel. Higher values may be used in areas where channel geometry is less consistent and/or the water flow is expected to be more turbulent. Examples may be areas of the stream containing obstructions (e.g., at bridge piers), multiple or drastic section changes (e.g., at culvert transitions), channel alignment changes (e.g., at the discharge of the intake structure), or other irregularities not adequately captured by the cross-sections. The Typical Bridge Transition values were used where the channel crosses under KCS Bridge and I-10. Abrupt Transitions were used where the channel enters and exits the submerged structures. The transition channel between the intake structure and the sediment basin is a unique situation. The channel alignment at this point changes significantly and abruptly, while at the same time the channel velocity drops from 6.67 feet per second (fps) to around 1 fps within only 250 feet. This creates a more turbulent flow regime, but it is not as tumultuous as the abrupt transitions between the culverts and the channel. Thus, average K values, between those of the Gradual Transitions and the Abrupt Transitions were used for this location: contraction coefficient = 0.35, expansion coefficient = 0.55.

<u>Channel Bank Stations</u>

The cross-sectional geometry is input into the program with station numbers, which differentiates portions of the conveyance area within the main channel from those outside the banks. The flow areas beyond the banks are termed over bank flow. This allows the modeler to use different friction coefficients for the main channel versus the over bank. This Maurepas conveyance channel does contain any over bank flow areas, such as river batture, in the traditional sense. Therefore, this modeling feature was used to differentiate between the areas of the channel which have clean dredged earth and those with short grass. The water surface elevation under a very low flow regime of 1,000 cfs was assumed to define the boundary between the clean dredged sections and the short grassy slopes. The resulting water surface profile was used to determine the stations within the channel banks. For flows higher than 1,000 cfs, areas of the flow outside these bank stations were assigned a higher surface friction factor (i.e., *n*) to account for greater resistance of the short grass. When flow occurs in areas with different Manning's *n* values, HEC-RAS solves for the flow rate in each of the areas to obtain the same energy slope for that section. In general, over-bank areas with higher *n* values will have lower velocities than the main channel bank areas.

Guide Levee Elevations

Guide levees are designated in HEC-RAS to prevent the program from routing water outside the channel, as shown in Figure 8-9, which is not representative of reality. If water were routed outside the channel, the conveyance area would be falsely increased, reducing channel velocity and leading to the channel energy losses being under estimated.

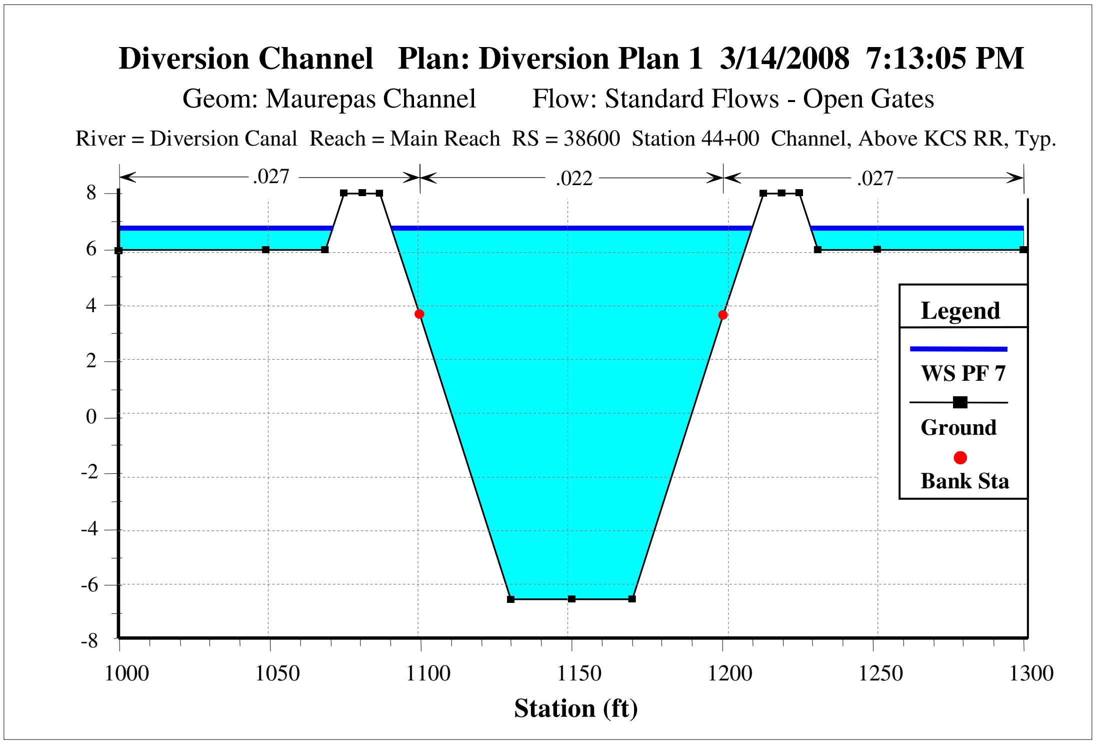


Figure 8-9: Without guide levee designation, HEC-RAS will convey water evenly across a cross-section, treating high spots as islands.

Guide levee designations, as shown in Figure 8-10, force the program to restrain the water within the levees until they are over topped. The diversion channel guide levee elevations were set based on three criteria: 1) To maintain a minimum freeboard of 18 inches for 2,000 cfs of river flow, plus 250 cfs pumped flow above US 61; 2) To contain 2,250 cfs of river flow, plus 250 cfs pumped flow above US 61; and 3) To contain a still water elevation of 5-ft.

Ineffective Areas

Ineffective flow areas designate portions of a cross-section that do not uniformly convey water along with the main channel. These areas typically occur around culverts, small bridges, swales, sharp bends and other obstructions. At such obstructions, water is forced to expand and/or contract around the impediment, as illustrated in Figures 8-11 and 8-12. Water outside the contraction and/or expansion zone may be either stagnant or turbulent, but it contributes very little to actual conveyance. These areas are removed from the effective conveyance cross-section by the use of ineffective flow markers.

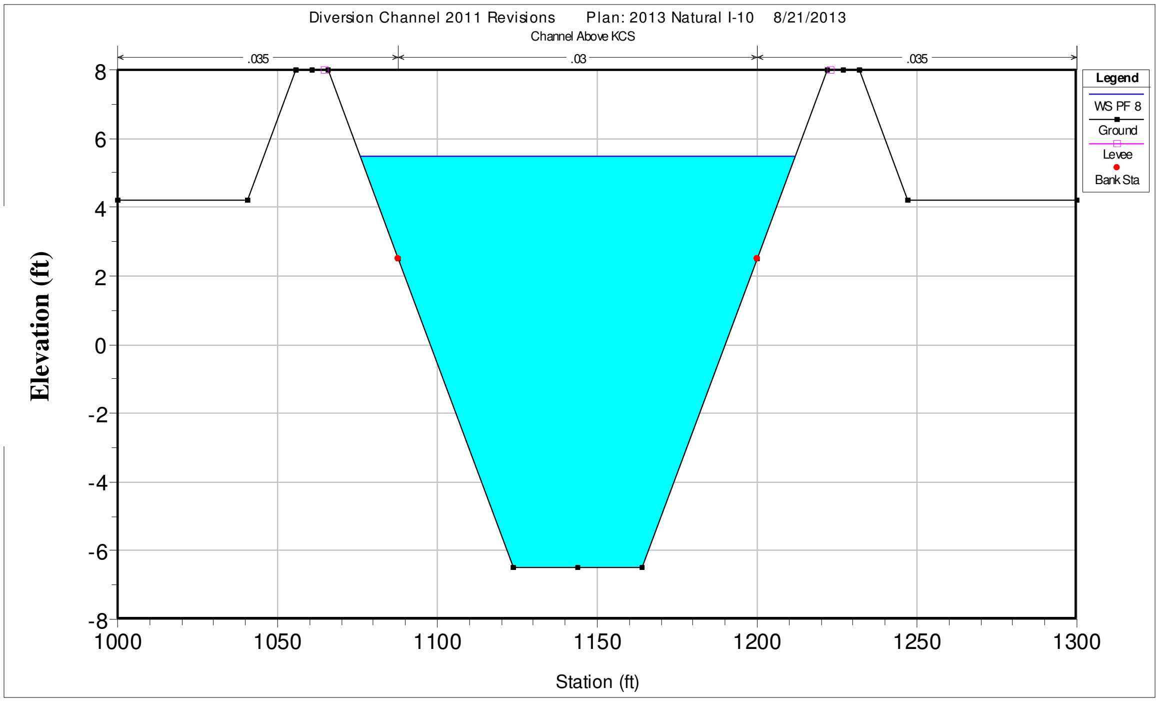


Figure 8-10: Levee designations restrict HEC-RAS to convey the water between the levee markers until the levees are overtopped.

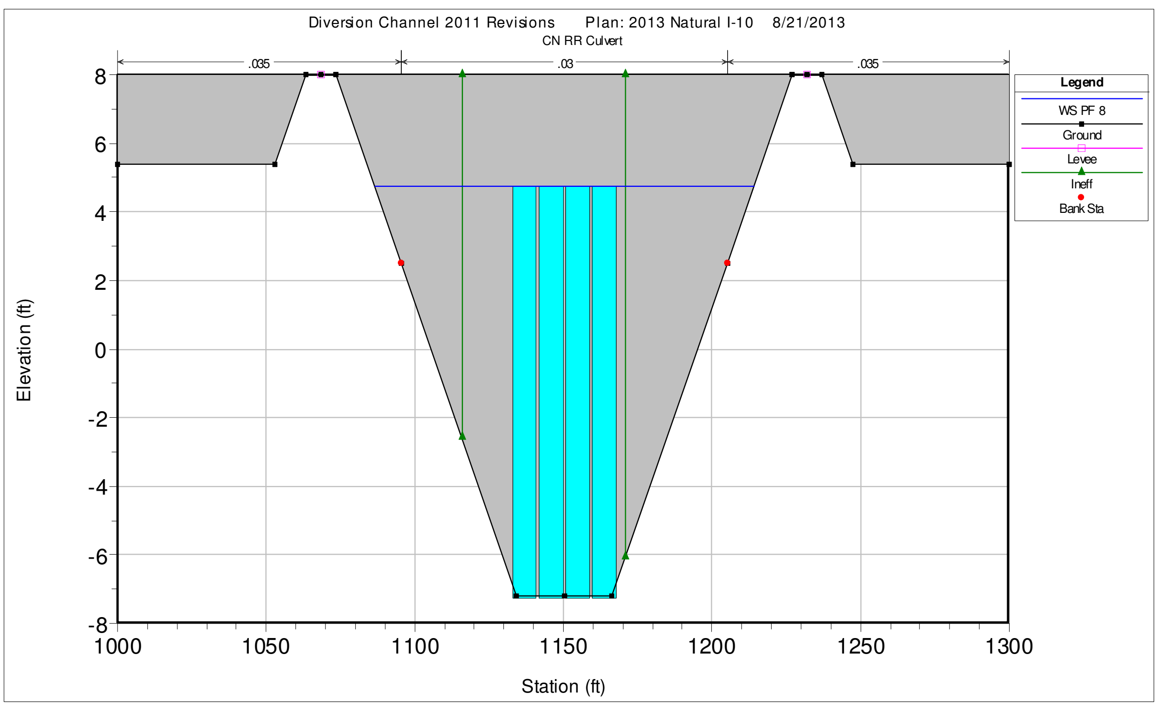


Figure 8-11: Ineffective Flow areas are typical around culverts.

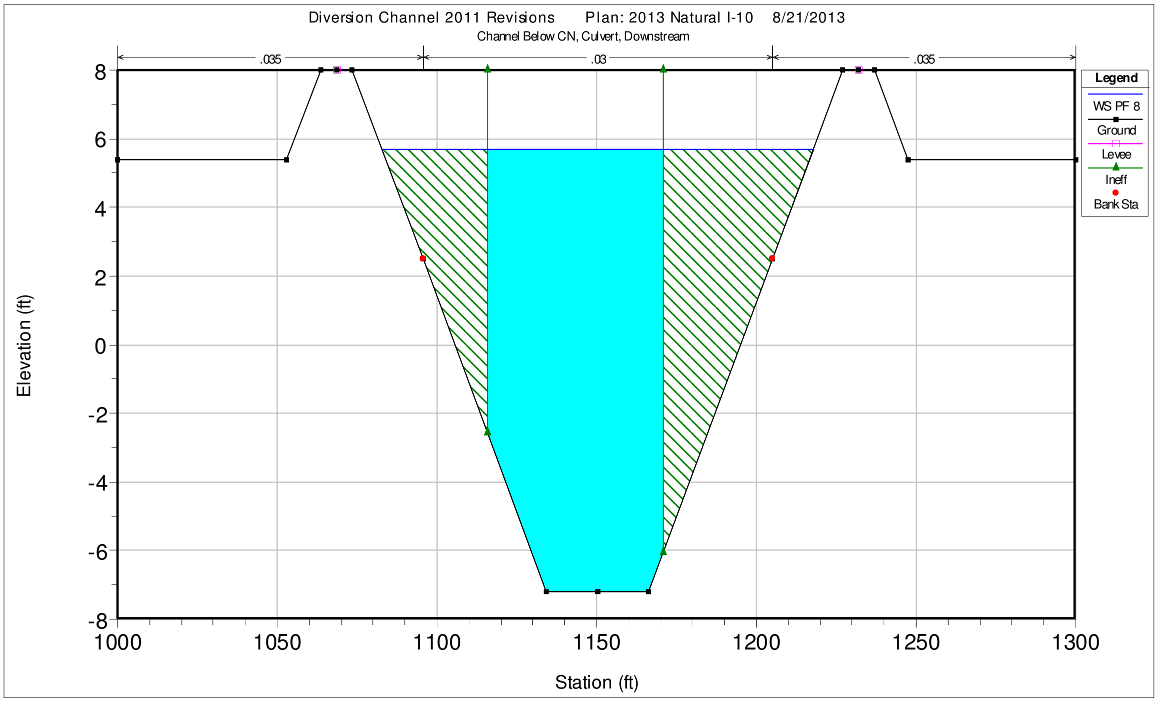


Figure 8-12: Ineffective Flow areas are used upstream and downstream from the same culvert as in Figure 8-4. The ineffective areas decrease further away from the culvert until the full cross-section is effective flow.

Flow obstacles have to be identified and the rates of contraction and expansion determined to properly assign ineffective flow areas. The ineffective flow area stations are calculated based on the rate of contraction (CR, typically upstream of a culvert) and expansion (ER, typically downstream of a culvert). These rates are expressed as ratios (CR:1 and ER:1). Ideally, in high flow situations these rates are determined in the field; however typical values can be used if field determination is not practical, such as for a new channel. Typical contraction rates are assumed to be 1:1. If a culvert or other impediment obstructs the channel cross-sectional area by a width of 50 feet on each side, the total length of contraction will be 50 feet. Typical expansion rates are dependent on the obstruction to channel width ratio (b/B), the over-bank to channel Manning's *n* ratio (n_{ob}/n_c), and the friction slope (S). An expansion rate of 2:1 is typical; values for specific combinations of the three independent parameters are tabulated in Table 8-5.

The value b is the channel width at the obstruction, while B is the width of the channel floodplain upstream and/or downstream of it. For a 160-ft wide channel from bank-to-bank (B) and a 60-ft wide culvert (b), the b/B ratio is 0.375. S is measured in feet of energy loss per length of channel in miles. The 28,785-ft (5.45 mile) Maurepas channel is relatively uniform in slope and the total change in head from intake to discharge is approximately 5.49 feet at the design flow. Thus, the friction slope (S) for the entire channel is 5.49 ft/5.45 mile, or approximately 1 ft/mile. The typical Manning's *n* of the over-bank, n_{ob}, is 0.035, while that for the channel, n_c, is 0.03. The n_{ob}/n_c value is thus 0.03/0.035, to which the closest whole number is 1. For the subject channel, b/B is either 0.25 or 0.50, while both S and n_{ob}/n_c equal 1, yielding ER values that range from 1.4 to 3.0. The smaller ER values are for lower velocities while the larger values are for higher velocities. Since 2.0 is a typical ER value and it falls within the calculated range, it was assumed for the model. Applying this value for an impediment that obstructs the channel cross-section area by a width of 50 ft on each side, the total length of expansion will be 100 ft.

Typical Expansion Rate Ranges	**Friction Slope S (ft/mile)**	n_{ob}/n_c **= 1***	n_{ob}/n_c **= 2**	n_{ob}/n_c **= 4**
b/B = 0.10	1	1.4 – 3.6	1.3 – 3.0	1.2 – 2.1
	5	1.0 – 2.5	0.8 – 2.0	0.8 – 2.0
	10	1.0 – 2.2	0.8 – 2.0	0.8 – 2.0
b/B = 0.25*	1	1.6 – 3.0*	1.4 – 2.5	1.2 – 2.0
	5	1.5 – 2.5	1.3 – 2.0	1.3 – 2.0
	10	1.5 – 2.0	1.3 – 2.0	1.3 – 2.0
b/B = 0.50*	1	1.4 – 2.6*	1.3 – 1.9	1.2 – 1.4
	5	1.3 – 2.1	1.2 – 1.6	1.0 – 1.4
	10	1.3 – 2.0	1.2 – 1.5	1.0 – 1.4

Table 8-5: Typical expansion rate ranges for determining the length of the channel conveyance expansion area downstream of an obstruction. Values indicated with an asterisk (*) are from the HEC-RAS Modeling Manual.

The elevations of the ineffective flow areas are determined by the vertical height of the upstream or downstream obstruction. For a culvert under a highway, the side walls determine the stationing of the ineffective flow areas while the top of the roadway determines the elevation. This designation tells the program that the ineffective area can be overflowed, as would be the case if the roadway were overtopped. Since the levees of the Maurepas conveyance channel will be tied around the channel upstream and downstream of the railroads and roadways, the ineffective flow elevations were set to match the levee elevations.

Structure Geometry

The HEC-RAS program simulates three types of structures: 1) culverts and bridges, 2) inline weirs, and 3) lateral weirs. Culvert and bridge structures were input for the intake structure culvert under LA 44, the culverts at the railroad crossing and at US 61, and for the bridges at KCS Railroad and I-10. An inline weir was used to approximate the control gate at the intake structure and for flow control into and out of the sediment basin. Lateral weirs were not used on this project. Each structure has its own set of coefficients that model the impact the structure has on the energy losses and thus the water surface elevations.

Gate Discharge and Orifice Coefficients

The Maurepas diversion intake structure was modeled using three sluice gates. The free flow sluice gate equation (Equation 8.4) is applicable for conditions where the gate is less than two-thirds submerged. Typical discharge coefficient values for sluice gates range from 0.5 to 0.7.

$$Q = CWB\sqrt{2gH} \qquad \textbf{Equation 8-4}$$

Where : Q = Quantity of flow, (ft^3/s)
C = Discharge coefficient, (--)
W = Width gate opening, (ft)
B = Height from spillway invert to gate bottom, (ft) and
H = Head from spillway invert at the gate to upstream energy grade line.

The gates and culverts on the intake structure may be either partially or fully submerged. Submergence is defined by the HEC-RAS program as the tail-water depth divided by the headwater energy grade line elevation. The free flow condition occurs when the submergence value is less than 0.67. For the Maurepas diversion, this requires very high river stages and partially closed gates, so Equation 8.4 is only applicable under these conditions.

When the river is moderately high, the gate will operate under partially submerged flow, which is defined as a submergence ratio between 0.68 and 0.79. During partially submerged flow, the free flow sluice gate equation is modified as shown in Equation 8.5.

$$Q = CWB\sqrt{2g3H} \qquad \textbf{Equation 8-5}$$

Where: Q, C, W & B are as defined in Equation 8-4
H = Head difference between upstream energy grade line and downstream water surface elevation.

During average or low river stages, the control gates will operate in fully submerged flow. In this case, the submergence value is greater than 0.79 and the orifice discharge equation (Equation 8.6) governs. The orifice discharge coefficient is typically 0.80.

$$Q = CA\sqrt{2gH} \qquad \textbf{Equation 8-6}$$

Where: Q & C are as defined in Equation 8.4
A = Area of the orifice (submerged gate) opening.
H = Head difference between upstream energy grade line and downstream water surface elevation.

In all three of the gate equations (Equations 8.4, 8.5, and 8.6), the HEC-RAS program solves for the head loss through the gate, H, which is then added to the downstream water surface elevation to yield the upstream elevation of the energy grade line.

Weirs and Weir Coefficients

HEC-RAS employs a weir routine when the water surface elevation is higher than the top of any structure. In addition to actual weirs, structures can include intakes as well as roadway and railroad embankments. Since the main intake structure passes through the Mississippi River levee and the box culverts go under LA 44, the program views the levee and roadway as valid weirs. Similarly, the two railroad crossings along with US 61 and I-10 are all considered weirs in the HEC-RAS model. In reality, because the diversion channel has a controlled input with levees on both sides that wrap around the intake and discharge areas, none of the embankments would overflow unless the levees did. Therefore, the weirs for these structures have been set at a water surface elevation equal to the levee height plus 0.1-ft, instead of the default of the embankment height plus 0.1-ft.

To direct a more uniform flow into the sediment basin, the Maurepas diversion structure will actually use two weirs. The crest elevations of these weirs are set at +2.5 ft NAVD88-LDNR, and will thus operate fully submerged at the design flow rate. As in the gate equations, submergence is defined as the tail-water depth divided by the depth of the headwater energy grade line. By default, HEC-RAS disregards any weir with submergence greater than 0.95 and processes it as a regular cross-section using the energy equation (Equation 8.1). For all weirs less than 95% submerged, the weir flow equation (Equation 8.7) is used.

$$Q = CLH^{3/2}$$ **Equation 8-7**

Where:
- Q = Quantity of flow, (ft^3/s)
- C = Weir coefficient, (--)
- L = Length of the weir, (ft)
- H = Head difference from the upstream energy grade line and the downstream water surface elevation (ft).

The default weir coefficient of 2.60 is applicable for low flows where the submergence ratio is less than 0.76. For flows where the weir submergence is between 76% and 95%, the weir coefficient is reduced by 75% as the routine phases the weir equation out and transitions to the energy equation.

Culvert Entrance & Exit Coefficients

For modeling culverts, the HEC-RAS program first determines whether inlet or outlet control conditions apply. All culverts throughout the Maurepas diversion channel are under outlet control. Outlet control routines use the energy equations for calculating entrance losses, frictional losses, and exit losses. For the concrete box culvert at the intake structure, which is constructed with the headwall as an extension of the culvert walls with square edges, the entrance loss coefficient has been set to the HEC-RAS default value of 0.7. For the concrete box culverts under the CN railroad and US 61, where the headwalls are parallel to the embankment and there are no wing walls, the entrance loss coefficients have been set to the default value of 0.5. The Manning's *n* value has been set to 0.017 for formed concrete, unfinished, using rough wood forms. The exit loss coefficient has been set to 1.0, the standard value.

The HEC-RAS outlet control culvert routine determines the energy losses through the culvert in several steps: 1) It evaluates the downstream energy grade elevation. 2) It adds the head loss through the culvert exit due to expansion using Equation 8.3. 3) It adds the head loss through the culvert pipe due to friction calculated by Equation 8.2. and 8.4) It then adds the head loss through the culvert entrance due to contraction applying Equation 8.3. Note that in the entrance loss computation, the culvert velocity head is used and the incoming stream velocity head is neglected. The combined losses in an outlet controlled culvert are shown in Equation 8.8.

$$H_L = K_{Entr} \times \left[\frac{v_A^2}{2g} \right] + L \times \left[Q \times \frac{n}{1.486 \times A \times R^{2/3}} \right]^2 + K_{Exit} \times \left[\frac{v_A^2}{2g} - \frac{v_B^2}{2g} \right] \qquad \textbf{Equation 8-8}$$

Where:
H_L = Total head loss, (ft)
K_{Entr} = Entrance loss coefficient, (--)
v_A = Velocity in the culvert, (ft/s)
L = Length of culvert, (ft)
Q = Quantity of flow, (ft^3/s)
n = Manning's coefficient, (--)
A = Culvert cross-sectional area, (ft^2)
R = Culvert hydraulic radius, (ft)
K_{Exit} = Exit loss coefficient, (--)
v_B = Velocity in channel downstream of culvert, (ft/s).

Bridge Pier Drag Coefficients

Similar to the culvert routines, when HEC-RAS encounters a bridge, the program first determines whether low or high flow methods will control and then determines the class of flow. Low Flow Class A (sub-critical) methods are applicable for the Maurepas diversion. The bridge routine then computes the head losses using the energy equations, momentum equations, and the Yarnell equations; the highest head loss calculated governs. The energy equations (Equations 8.2 and 8.3) calculate the contraction losses upon encountering the piers, the friction losses through the bridge section, and the expansion losses as the flow passes beyond the piers. The momentum equation selects a drag coefficient based on the pier shape and calculates the head loss required to conserve momentum through the bridge section. The HEC-RAS default value for the drag coefficient of 2.0 was used for square nose piers. The Yarnell equation uses an empirically derived relationship to calculate the head loss. A Yarnell K Coefficient of 1.25 was used for the square nose and tail pier shape.

Hydraulic Flows & Velocities

The stage in the Mississippi River establishes the head available to drive flow through the intake structure gates and the conveyance channel to the downstream tail waters of the Maurepas Swamp. The intent is to adjust the three intake gates so that the flow in the channel is maintained at 2,000 cfs or lower. In a HEC-RAS steady state simulation, the flow is specified, a downstream boundary condition is provided to determine the tail-water elevation, and then the head losses are calculated throughout the system modeled. The upstream water surface elevation (or river stage) required to force the specified flow through the channel to the downstream boundary is then calculated.

The channel geometry was designed to maintain a velocity near 2 fps throughout as much of the route as possible at the design flow rate of 2,000 cfs. This target was established to minimize head losses, maximize the duration of operation at the design flow, and prevent sedimentation in the channel. A sediment basin, geometrically designed to slow the velocity to less than 1 fps, is provided for the settling of sands and heavier silts. Note that all figures showing cross-sections are from Water Surface Profile #8 (WS PF 8).

To develop a rating curve for the channel, various flows were modeled. Also, to develop a gate opening schedule, various gate openings were modeled for the design flows. Table 8-6 lists the flows and gate openings modeled.

Profile No.	Flow From River (cfs)	With Pump Station Flow (cfs)	Gate Opening Height (ft)	Flow Description
1	250	250	10.0	Low Flow
2	500	500	10.0	Low Flow
3	750	750	10.0	Low Flow
4	1000	1000	10.0	Low Flow
5	1250	1250	10.0	Low Flow
6	1500	1500	10.0	Low Flow
7	1750	1750	10.0	Low Flow
8	2000	2250	10.0	Design Flow

Table 8-6: The Maurepas diversion channel operation was simulated using 19 profiles to cover low flow, design flow, high flow and high stage conditions, with all three gates opened to the same level.

Boundary Conditions

A downstream boundary condition is required because HEC-RAS sub-critical flow routines begin at the downstream end of the channel. Figure 8-13 presents the rating curve established for the downstream boundary condition, which was based on the results of the Task 1 study.

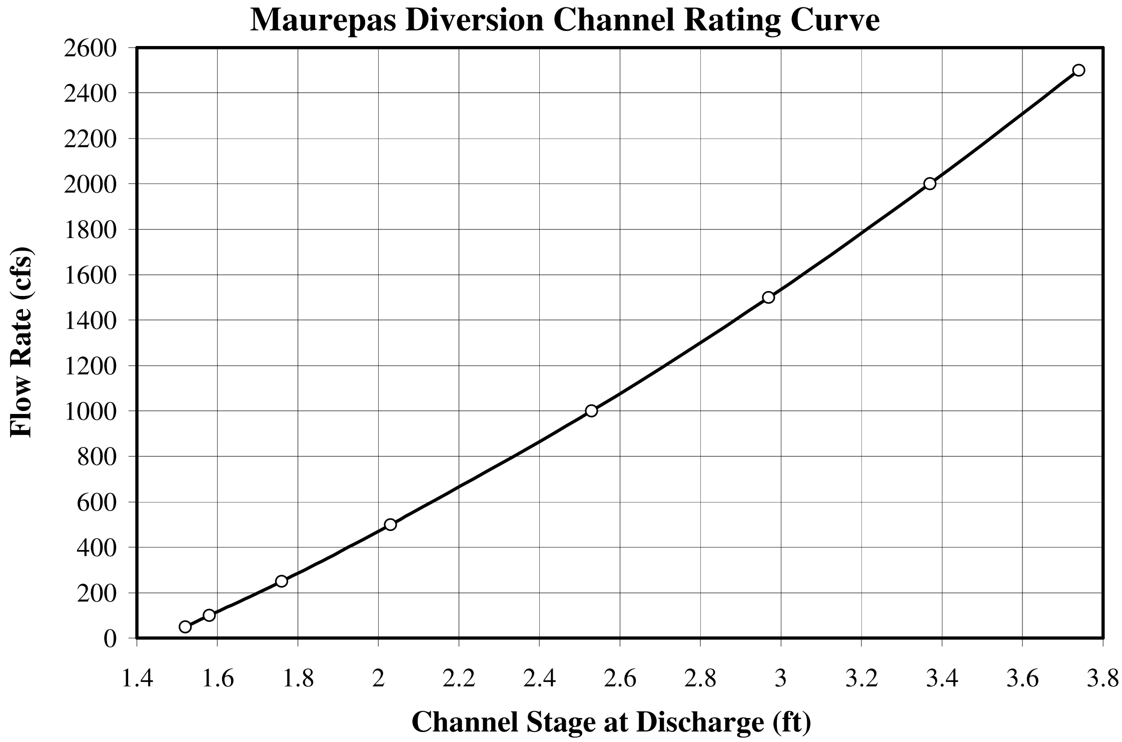


Figure 8-13: Maurepas diversion channel rating curve.

Channel & Structure Geometry

The Maurepas diversion channel model is comprised of 148 cross-sections and structures over three reaches: 1) the intake channel, 2) the channel from the river to US 61, and 3) the channel from US 61 to I-10. The overall conveyance channel model incorporates the intake structure and LA 44 crossing, a sediment basin, two railroad crossings, the US 61 crossing, and the bridge crossing underneath I-10. Table 8-7 tabulates the reaches, lengths and stations.

Reach	Length (ft)	Upstream Station	Structure Station	Downstream Station
Intake Channel	175	14+40	N/A	16+15
Intake Structure & Basin Transition	515	16+15	16+90 Gate 17+00 Culvert	21+30
Sediment Basin & Channel Transition	610	21+30	22+55 Upstream 26+15 Downstream	27+40
Channel Reach 1	1,109	27+40	N/A	38+49
CN RR	305	36+66	36+66	39+16
Channel Reach 1	5,426	39+16	N/A	97+23
KCS RR	436	97+23	97+49	98+59
Channel Reach 1	1,009	98+59	N/A	111+69
US 61	575	111+69	111+69	115+78
Channel Reach 2	17,235	115+78	N/A	290+00
I-10	310	290+00	290+85 Eastbound 291+80 Westbound	293+10
Discharge Channel	915	293+10	N/A	302+25
Total	**29,213**	**14+40**		**302+18**

Table 8-7: Maurepas diversion channel reaches and structures.

<u>Cross-Section Design, Intake Channel</u>

The channel between the Mississippi River and the intake control structure was modeled as a trapezoidal shape with a 40-ft wide bottom, an invert of -4.00 ft NAVD88-LDNR, and 3H:1V side slopes. Manning's *n* was set to 0.033 for the main channel because of the rip-rap protection. Contraction and expansion coefficients were set to 0.1 and 0.3, respectively. The channel banks were set to elevation +4.90 ft NAVD88-LDNR. The levees were set to the existing batture elevation of +18.00 ft NAVD88-LDNR. The cross-sections were extended up to elevation +31.00 ft NAVD88-LDNR for numerical stability of the model; the actual channel will not be constructed to this elevation. Figure 13 illustrates the typical cross section, with no structures within the channel reach.

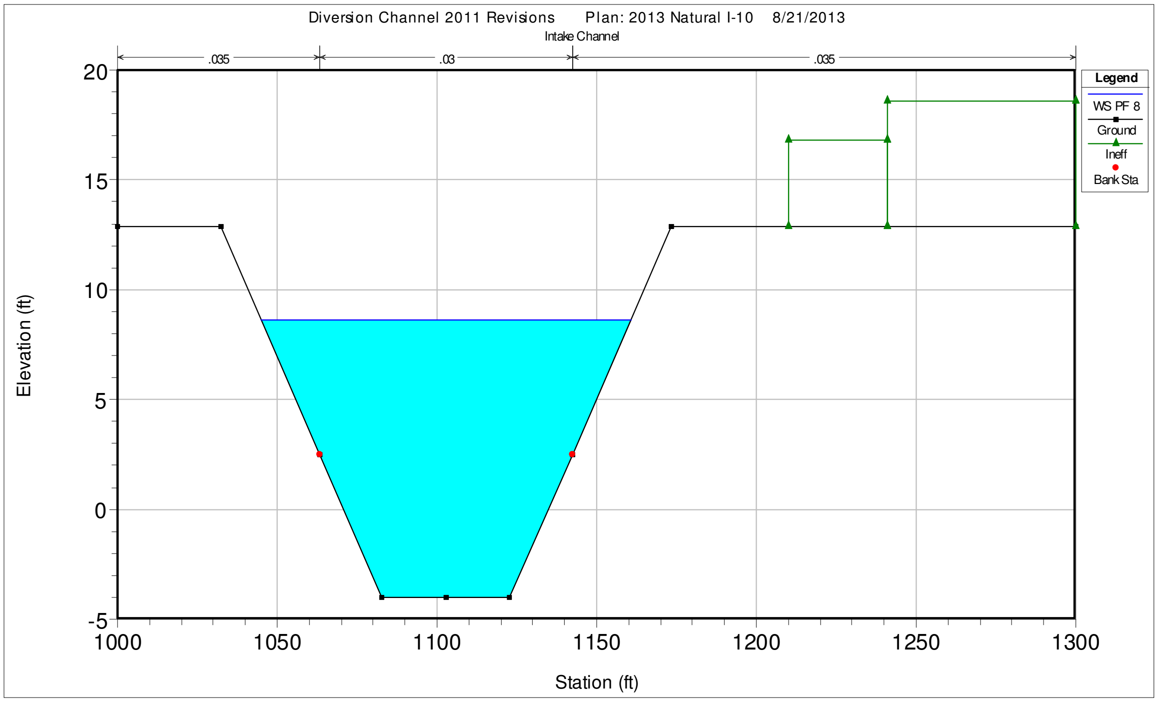


Figure 8-14: Typical cross-section for the intake channel.

The intake channel remains flat until 100 feet upstream of the intake structure at station 15+80, at which point it becomes sloped. The intake channel ties-in to the structure U-channels at elevation -2.30 ft NAVD88-LDNR at station 16+15.

This reach of channel begins at station 14+40, however, actual construction will begin at station 13+10. The section from station 13+10 to 14+40, where the batture slopes down to elevation -4.00 ft NAVD88-LDNR (plan sheet HC-1.00) was not modeled. Rip-rap protection will be applied from the river edge at station 13+10 to the channel bottom, on the side slopes, and along both sides on top of the batture to provide erosion protection.

The intake channel design incorporates certain criteria to minimize the danger to the endangered pallid sturgeon. First, the channel invert is set at elevation -4.00 so that it pulls water from much nearer the river's surface than the bottom-dwelling sturgeon normally range. Second, the design geometry accommodates a velocity of 2 fps at the design flow, so that the fish do not become trapped within the diversion waters. Velocities in the channel do not exceed 4 fps until within 25-ft of the gated structure, leaving a 150 ft stretch of the intake channel with low velocities, from which the sturgeon can escape. Third, the area of rip-rap stretches 150-ft upstream from the U-channels leading to the intake structure, presents an unnatural environment which the fish tend to avoid, thus further minimizing the risks of their entrapment.

<u>Intake Structure</u>

The modeled intake structure features three (3) gates, each sized 10-ft x 10-ft. These gates are connected to three concrete culverts spanning 290 feet starting at an invert elevation of -7.00 ft NAVD88-LDNR, (plan sheet C-19). The intake structure and concrete culverts are each joined by concrete U-channel transitions (see plan sheets HC-1.00). The HEC-RAS program is not designed to model a gate structure combined with a culvert, so a replicated section was inserted between them. This replicated section was provided with a lid to cap the cross-section so that it resembled the three 10-ft x 10-ft culvert bank in conveyance. The reach length for this section was set to 2.5 feet, (see Figure 8-15). Manning's n values were set to 0.017, reflecting that of the culvert. Contraction and expansion coefficients were set to 0 since these losses are calculated as part of the gate losses and the culvert exit losses.

An inline weir set to an elevation of 31.00 feet NAVD88-LDNR to match the Mississippi River levee was provided at station 16+90. Three 10-ft x 10-ft sluice gates set at an invert of -7.00 ft NAVD88-LDNR were provided for flow control. Discharge, orifice, and weir coefficients were required for the gate operations. The discharge coefficient was set to 0.60 and the orifice coefficient to 0.80, from Table 8-4. Since the gate is fitted to the culvert, there will be no expansion of the water conveyance sideways or downward, thus these coefficients are conservative. The weir coefficient was neglected because the water levels in the Mississippi will never be below the sill invert of the gates (-7.00 ft NAVD88-LDNR). Contraction and expansion coefficients for the cross-section at the upstream face of the gate were set to 0.6 and 0.8 for the abrupt transition.

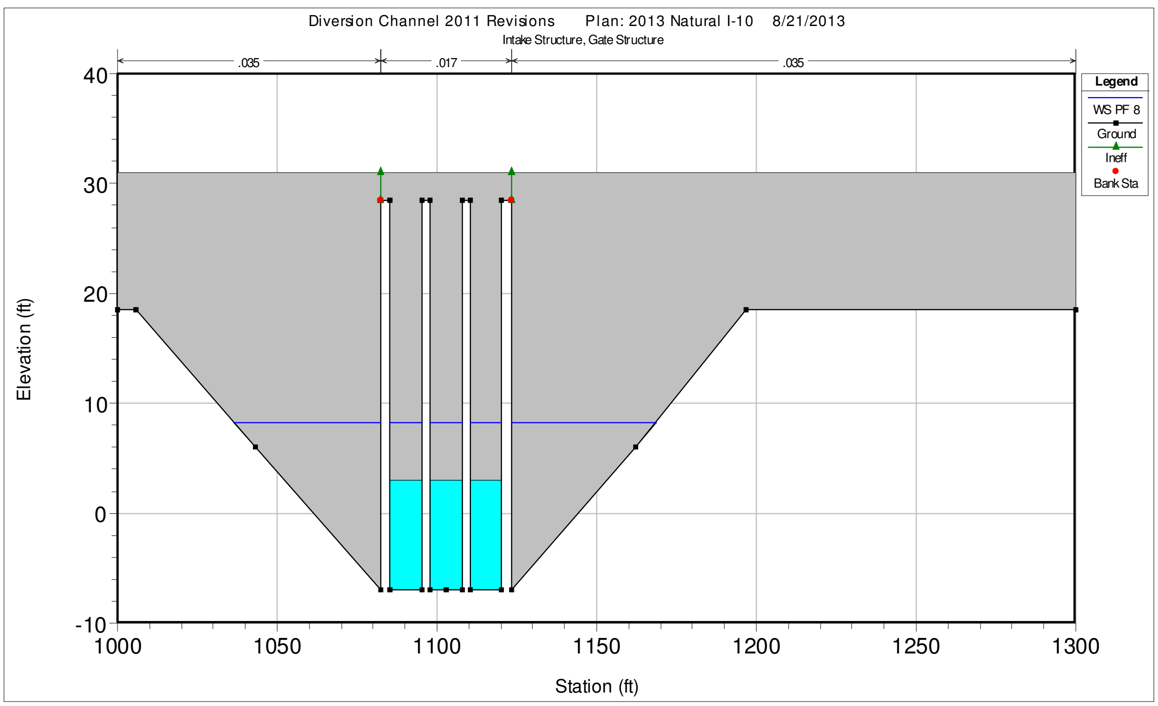


Figure 8-15: An inline gate structure was modeled to determine the energy losses due to gate operations at the intake.

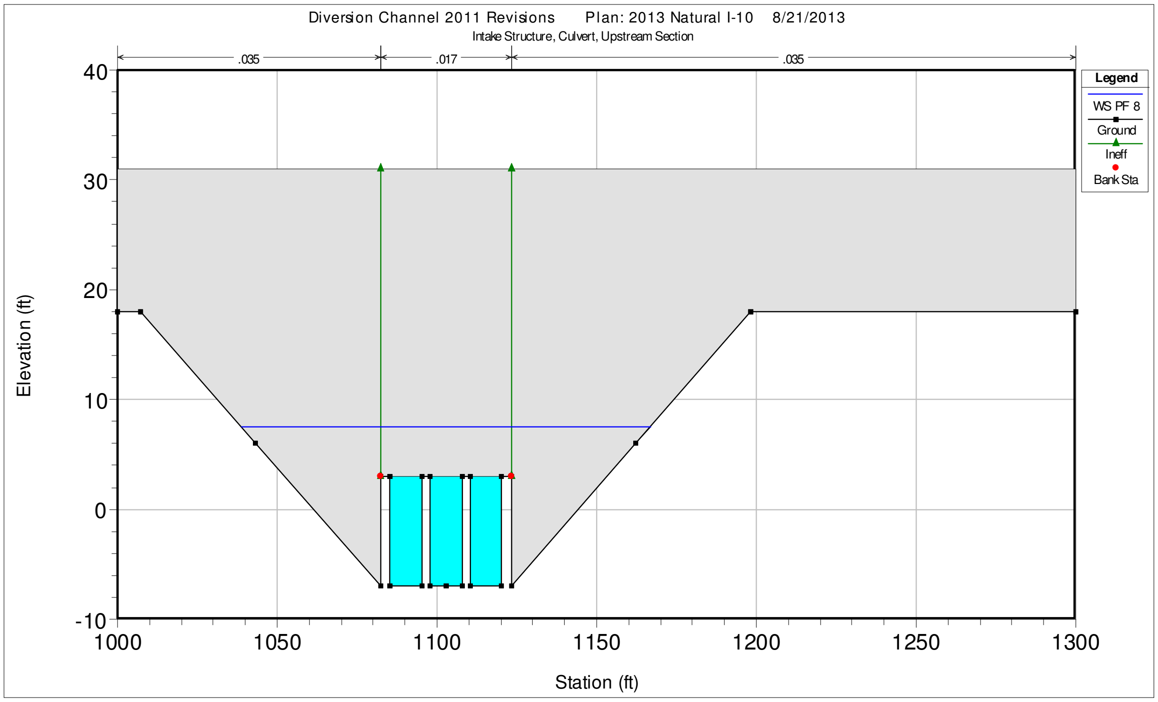


Figure 8-16: A replicated cross-section was created in the HEC-RAS model between the inline gate and culvert structures.

Manning's n for the culverts was set to 0.017 for formed concrete with rough wood forms; smooth wood or steel forms will yield in better hydraulic performance to a Manning's n value of 0.013. (It is expected that the Contractor will use smooth wood or steel forms, however for this model the worst case scenario will govern.) The entrance coefficient was set to 0.0 as these losses are calculated at the gate structure using the more conservative gate discharge and orifice coefficients. The exit coefficient was set to 1.0 as a standard practice. Contraction and expansion coefficients for the cross-section at the downstream face of the culvert were set to 0.6 and 0.8 for the abrupt transition. The culvert is illustrated in Figure 8-16.

The transitions between the intake channel and structure and between the intake structure and the sediment basin are modeled as U-channels. These channels have an invert elevation of -7.00 ft NAVD88-LDNR at the intake structure and are sloped to the proper tie-in elevations at the upstream and downstream ends. The channels are 36-ft wide at the intake structure and gradually widen to match the width of the channel at the tie-in point. The U-Channels feature vertical concrete walls on the side at elevation +9.00 ft NAVD88-LDNR and are 3-ft thick. On the backsides of the walls, earth fill is used on a 3H:1V slope to the proper levee elevations. Figures 8-17 and 8-18 show typical cross-sections of the U-Channels upstream and downstream of the intake structure.

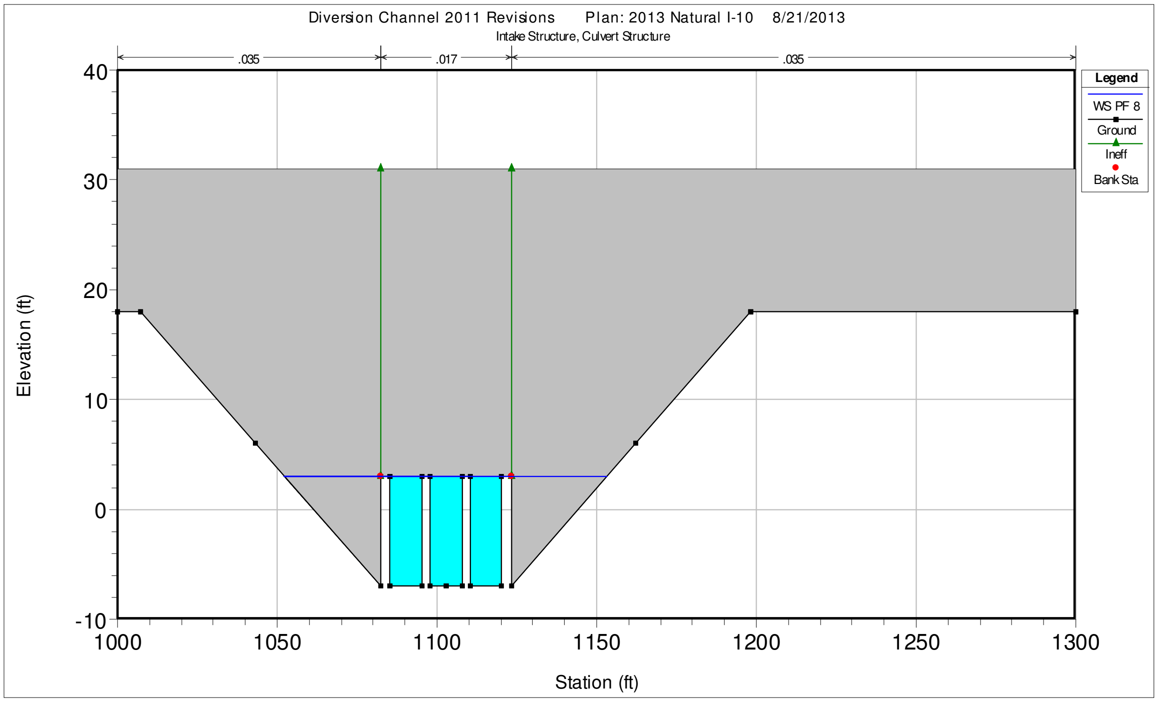


Figure 8-17: A 290' culvert was modeled in HEC-RAS as the final part of the intake structure simulation.

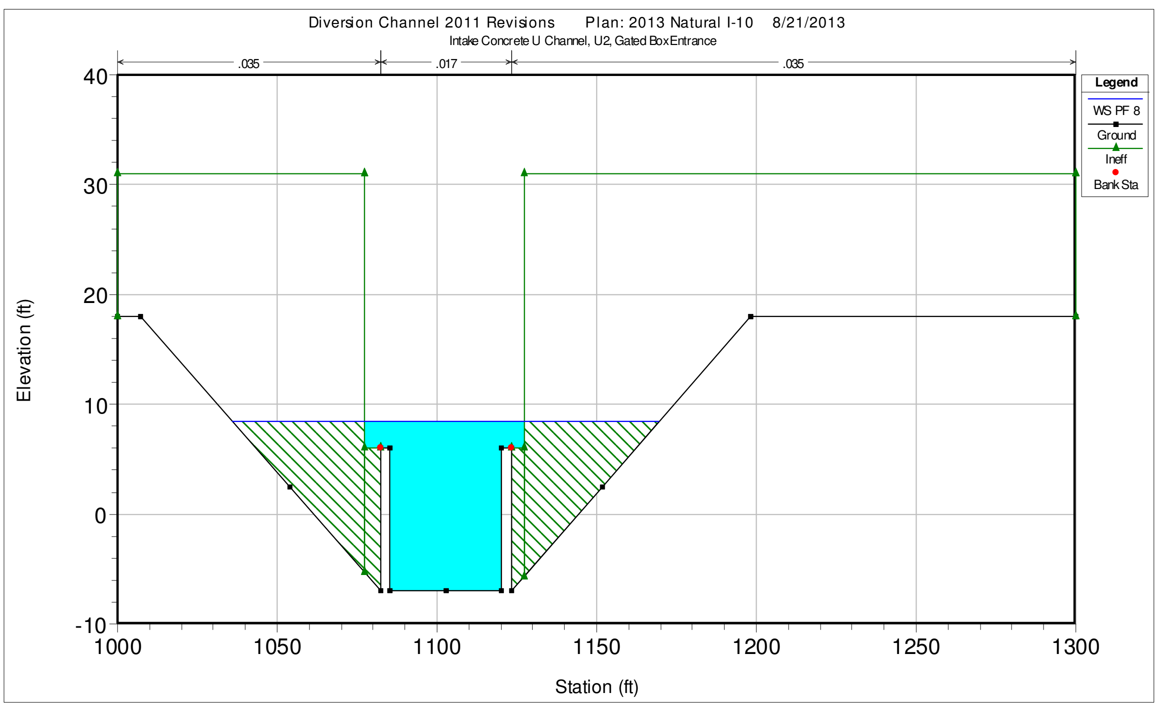


Figure 8-18: U-Channels are modeled to transition the intake channel to the intake structure.

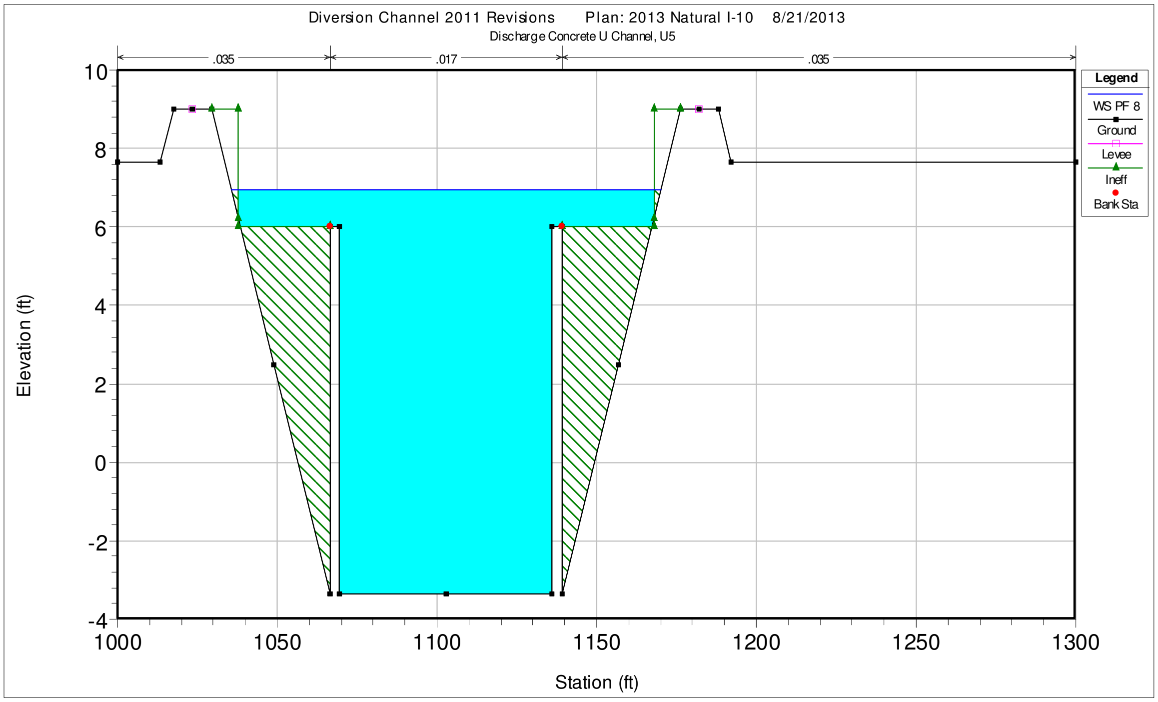


Figure 8-19: U-Channels are also modeled to transition the intake structure to the sediment basin transition channel.

Manning's *n* values of 0.017 and 0.035 were used for the concrete and grassy over-bank areas, respectively. The channel banks were set to the top of the U-Channel walls at elevation +9.00 ft NAVD88-LDNR. Levee widths and elevations were matched to the tie-in channel requirements: +18.00 ft NAVD88-LDNR on the intake and +9.00 ft NAVD88-LDNR on the sediment basin. Contraction and expansion coefficients were set to 0.1 and 0.3, respectively, for the upstream side reflecting the gradual transitions. They were set to 0.35 and 0.55, respectively, on the downstream side also reflecting the gradual change, with an expected increase in turbulent flow at the discharge structure.

Sediment Basin

The sediment basin features an earthen channel transition from the U-Channels over an inflow weir and into the basin. It discharges through an outflow weir and a transition section to the diversion channel. For the full length of this section, including both weirs, both transitions, and the sediment basin itself, the side slopes are 4H:1V and the levees are set to elevation +9.00 ft NAVD88-LDNR. The levees also include a 12-ft wide crown for light vehicle access. The channel bank elevations vary throughout the sediment basin to match the 1,000 cfs low flow water surface elevation.

The bottoms of the transition channels and weirs have rip-rap from the toe of the weir inside the sediment basin and along the earth side slopes and levees. A Manning's *n* values of 0.03 was selected for this section. Contraction and expansion coefficients for the 125-ft basin intake transition channel are 0.35 and 0.55, respectively due to the gradual change and the increase in turbulence caused by the alignment shift. For the 115 foot transition from the sediment basin to the main diversion channel, the coefficients are set to 0.1 and 0.3 to reflect the gradual change in geometry. For the 150-ft wide inflow and outflow weirs, default coefficient of 2.60 was used and the elevations were set to +2.5 ft NAVD88-LDNR.

The sediment basin is 265-ft long with a bottom width of 66-ft at elevation -11.00 ft NAVD88-LDNR. The basin was designed to provide up to 4-ft of sediment storage without impacting the hydraulics of the basin. Manning's *n* values inside the basin were set to 0.03 for the channel and 0.035 for the over-banks. Contraction and expansion coefficients were set to 0.1 and 0.3, respectively represent the gradual changes at the transitions.

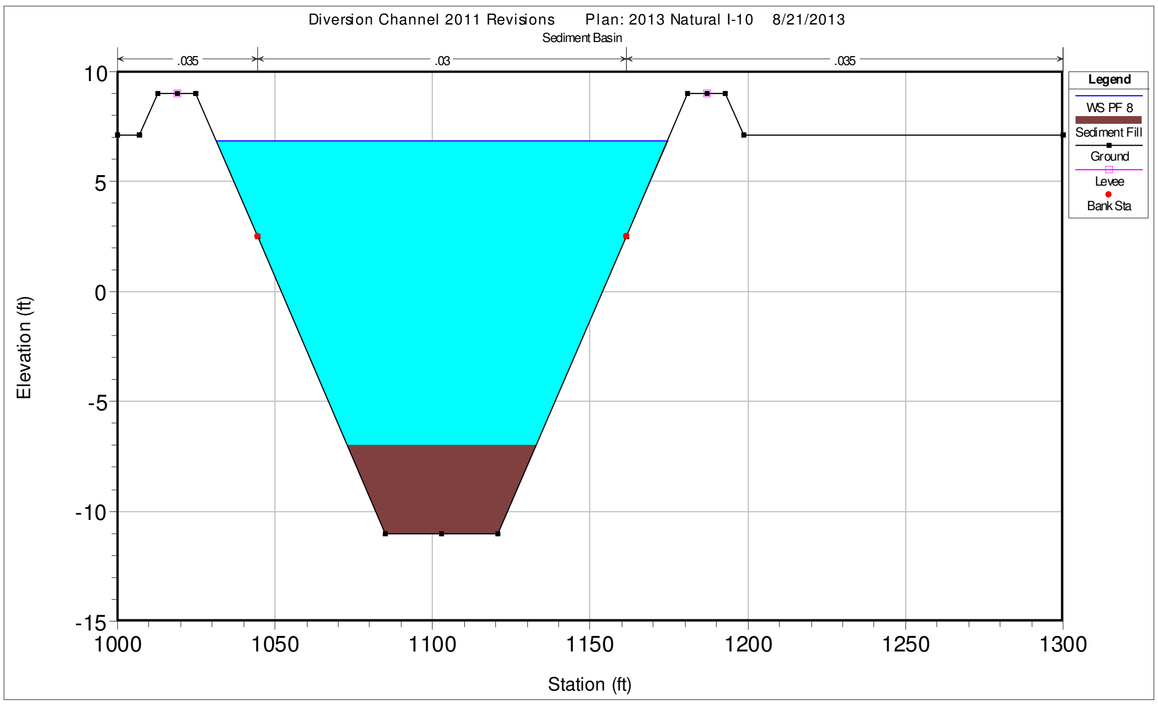


Figure 8-20: The sediment basin was modeled to have an invert of -11.00 ft NAVD88-LDNR and allow for up to 4-ft of fill.

<u>Cross-Section Design, LA 44 to US 61</u>

The channel between LA 44 and US 61 was modeled as a trapezoid with 4H:1V side slopes, a 40-ft wide bottom, and inverts of -5.50 ft and -7.00 ft NAVD88-LDNR, upstream and downstream, respectively. This geometry is shown in Figure 8-19. Manning's *n* was set to 0.03 for the main channel and 0.035 for the over-bank. Contraction and expansion coefficients were set to 0.1 and 0.3, respectively. The channel banks were set to elevation +3.90 ft NAVD88-LDNR at the sediment basin and slope to +3.20 ft NAVD88-LDNR at US 61. Levees elevations of +9.00 ft NAVD88-LDNR were established at the sediment basin and US 61 ends, respectively. The levees have a 12-ft wide crown to serve as an access road for light weight vehicles. There are two structures within this reach of channel, one at each of the railroad crossings. Figure 8-21 illustrates the cross-section within this portion of the channel.

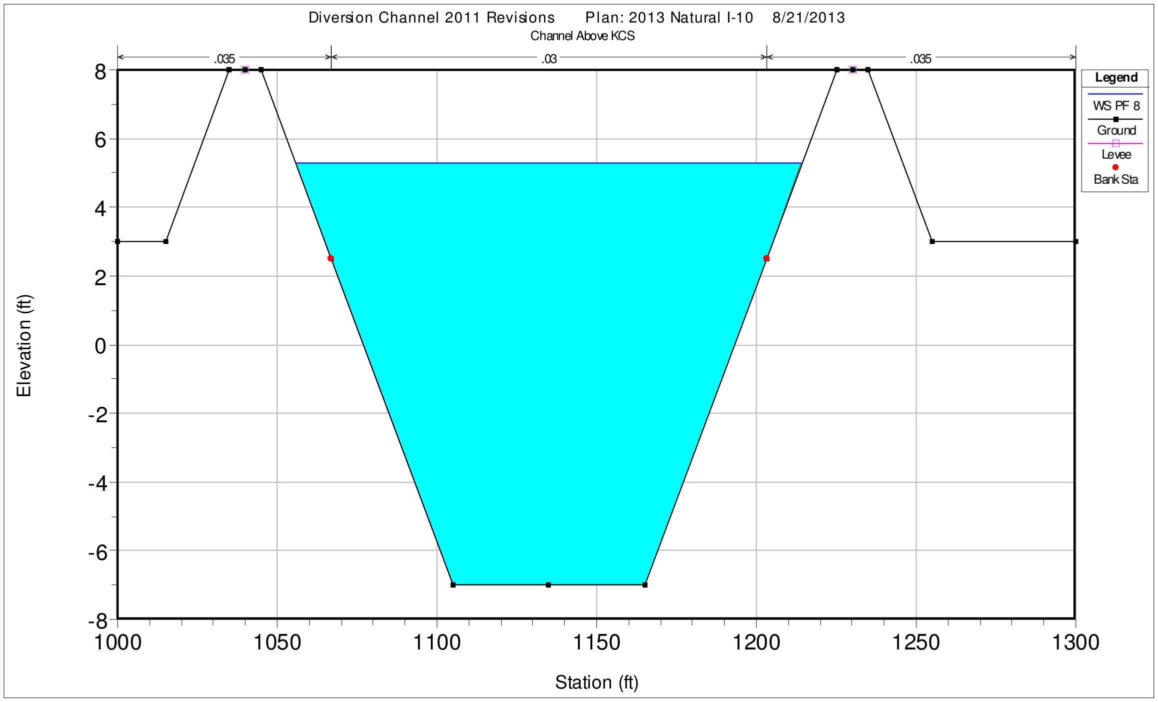


Figure 8-21: A typical cross-section of the diversion channel between the sediment basin and US 61.

<u>Canadian National (CN) Railroad Crossings</u>

The CN RR crossing at model station 39+49 consists of four 8-ft x 12-ft reinforced concrete box culverts, each at 271-ft in length. The levees on both the upstream and downstream ends of the culvert will wrap around the channel in a 'U' shape to prevent the flooding of the railroad during operation. The boxes will pass under the railroad and the channel levees. To avoid conflicts with local drainage, the boxes will be set at an invert of -7.25 ft NAVD88-LDNR. The box culvert configuration will require a 60-ft wide channel bottom. The upstream and downstream transitions from the culverts into the 40-ft wide main channel are 100-ft long, with inverts of -5.50 ft and -6.25 ft NAVD99-LDNR on the upstream and downstream ends, respectively. Upstream of the structure, the channel has 3H:1V side slopes with levees at elevation +9.00 ft NAVD88-LDNR. Downstream of the structure, the channel maintains its 3H:1V side slopes, while the levee elevation falls to +8.00 ft NAVD88-LDNR. The upstream face of the culvert is skewed to match the westward alignment of the channel, maximizing the buffer between the channel and the residential neighborhood. This geometry is shown in Figure 8-21.

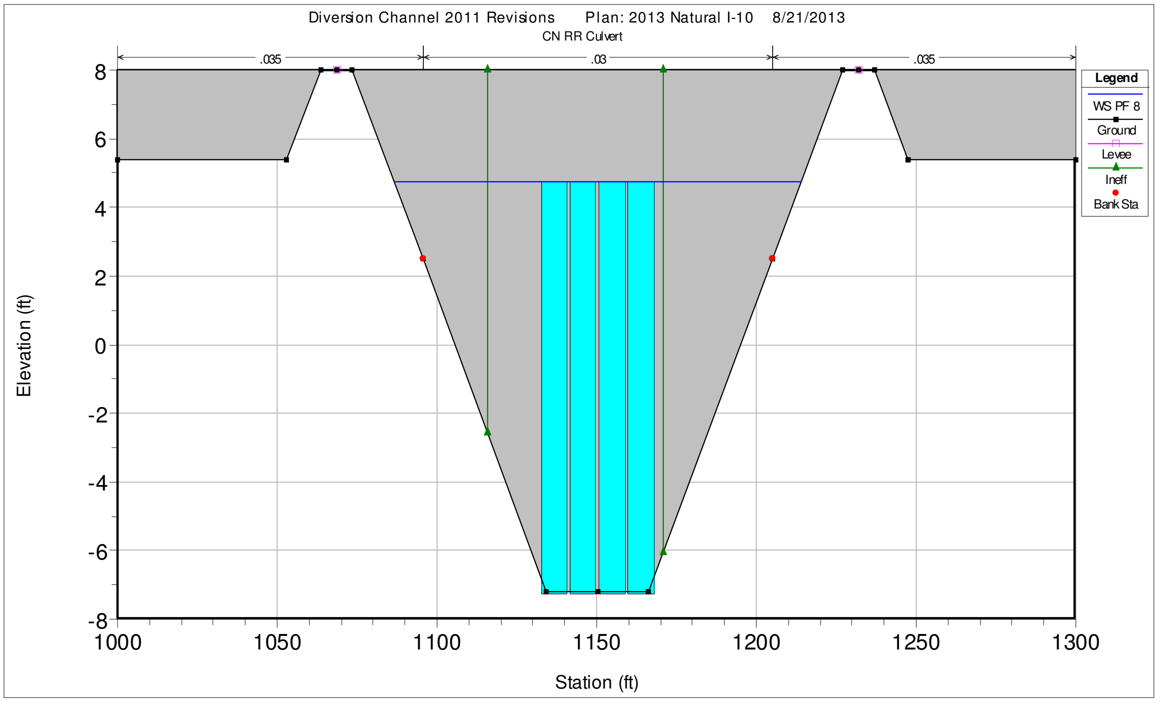


Figure 8-22: A four barrel 8-ft x 12-ft box culvert arrangement is used at the CN RR crossing.

Manning's *n* for the culverts was set to 0.017 for formed. The entrance coefficient was set 0.5 for a concrete box with headwalls parallel to the embankment and no wing walls. Rounding of the top, side and intermediate walls to a 10-in or greater radius will reduce the entrance coefficient to 0.2, also improving hydraulic performance. The exit coefficient was set to the standard 1.0 value. Contraction and expansion coefficients for the cross-sections at the face of the culvert were set to 0.6 and 0.8, respectively, to model the abrupt transitions.

<u>Kansas City Southern (KCS) Railroad Crossings</u>

During Task 3, KCS railroad directed URS to change the crossing from culverts to bridge. URS subsequently updated the model developed under Task 1 with the said change.

The KCS Railroad crossing at station 97+49 consists of a 100-ft long bridge with 19'10" between piers. The piers themselves are composed of three 24" diameter piles per bent with 3' pilecaps. The piers are aligned such that the bents are not parallel with the alignment of the channel, and thus the direction of the flow. Therefore, the projection of the overlap of the piles was used to represent the bents, and was thus 4.4'.

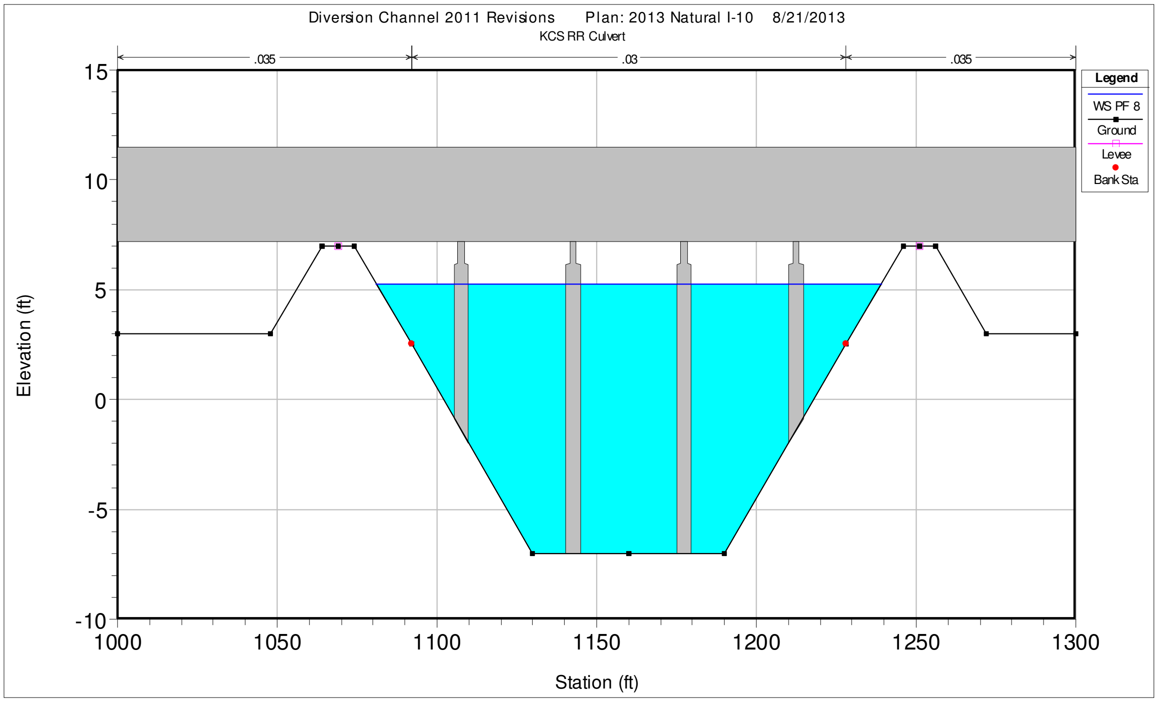


Figure 8-23: The bridge used at the KCS RR crossing.

<u>US 61 Crossing</u>

Under Task 3, with changes of KCS railroad, there had to be changes made downstream of KCS bridge to accommodate required freeboard under the bridge.

The US 61 crossing at station 111+69 had to be modified to culvert with six 375-ft long 9-ft x 9-ft reinforced concrete boxes. The levees on both ends of the culvert will wrap around the channel to prevent flooding the highway during operation. The boxes will pass under US 61 and the channel levees. To avoid conflicts with local drainage, the boxes will be set at an invert of -9.50 ft NAVD88-LDNR. The culvert configuration will require a 60-ft wide channel bottom. The transitions into the 60-ft bottom width of the main channel are modeled as 100-ft long with inverts set at a -7.00 ft NAVD88-LDNR on both sides. Upstream of the structure, the channel consists of 4H:1V side slopes and levees at elevation +7.00 ft NAVD88-LDNR. Downstream of the structure, the channel has 4H:1V side slopes with levees at elevation +6.50 ft NAVD88-LDNR. This geometry is shown in Figure 8-23.

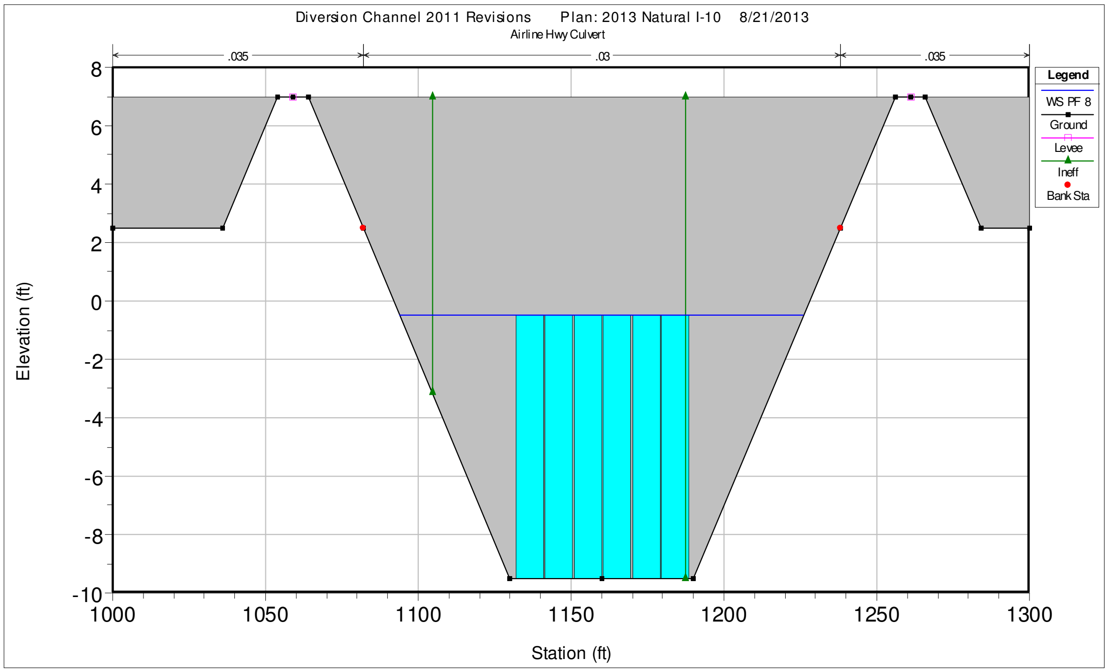


Figure 8-24: The six barrel 9-ft x 9-ft box culvert arrangement at the US 61 crossing.

As with the culverts at the two railroad crossings, the following values were used: Manning's *n* = 0.03, entrance coefficient = 0.5, and exit coefficient = 1.0. As in the previous cases, if actual construction creates a more hydrodynamic surface, the actual hydraulic performance will be improved, thus the parameters chosen are conservative. The contraction and expansion coefficients were set to 0.6 and 0.8, respectively and the *n* value for the 100-ft transitions was chosen as 0.03.

<u>Cross-Section Design, US 61 to I-10</u>

As mentioned earlier, changes and updates under Task 3 included changes to channel section downstream of the US 61 crossing.

The channel between US 61 and I-10 was modified and modeled as a trapezoid with a 60-ft wide bottom and 1V:5H side slopes. The starting invert was set at -7.00 ft NAVD88-LDNR at US 61 and the ending invert was set to -8.00 ft NAVD88-LDNR at I-10. Manning's *n* was set to 0.03 for the main channel and 0.035 for the over-bank. Contraction and expansion coefficients were set to 0.1 and 0.3, respectively. The channel banks were set to elevation +3.00 ft NAVD88-LDNR at US 61 and +2.65 ft NAVD88-LDNR at I-10. Levees were set to elevation +6.50 ft NAVD88-LDNR at US 61 and +5.50 ft NAVD88-LDNR at I-10 with a 12-ft wide crown to serve as an access road for light weight vehicles. This geometry is shown in Figure 8-24. There are no structures within this reach of channel.

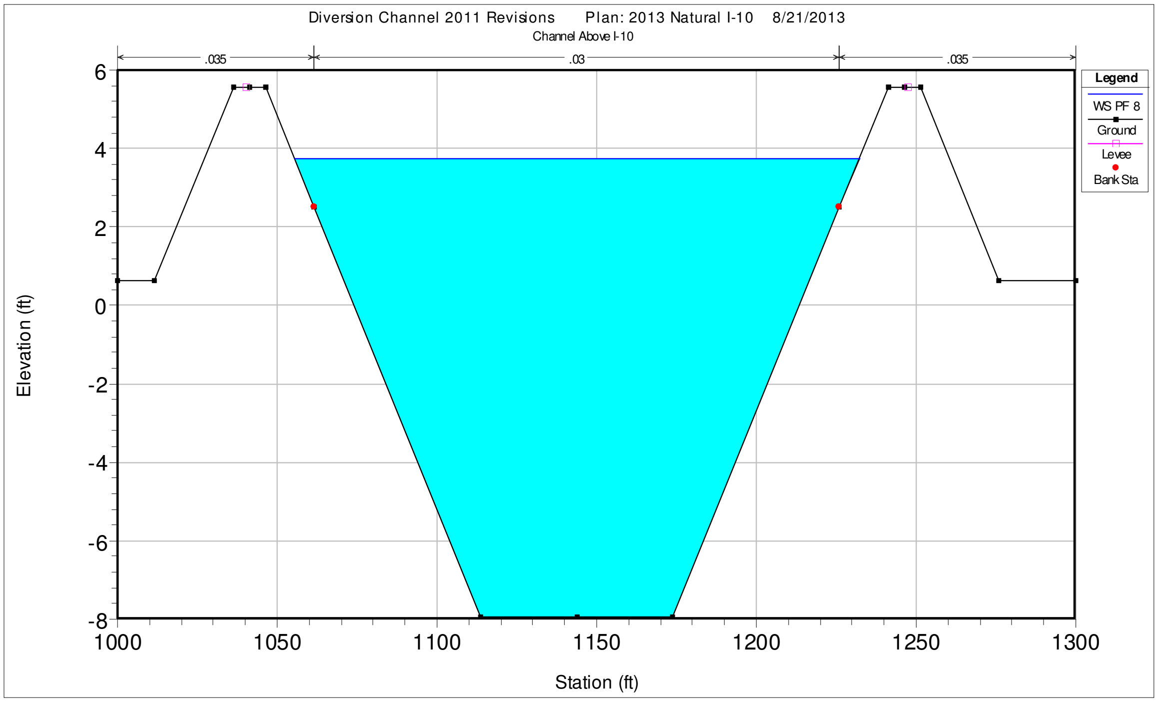


Figure 8-25: A typical cross-section of the channel between US 61 and I-10.

<u>I-10 Crossing</u>

The two I-10 bridges are existing structures at stations 290+85 and 291+80, which will not be modified. Information regarding these bridges was obtained from LDOTD plans. The bridges feature seven spans, five of which will be impacted by the conveyance channel: the two 20-ft wide outside spans and three 25-ft wide center spans. The existing 16-in square concrete piles are tipped at elevation -54.0 ft MSL, except for those supporting the wing walls, which are tipped at -29.0 ft MSL. The overburden material down to elevation -12.0 ft MSL is assumed to be muck. The bridge decks are at an approximate elevation of +15.00 ft NAVD88-LDNR and the low chords clear elevation +12.00 ft NAVD88-LDNR. Each bridge is 45-ft wide and they are spaced 49.5-ft apart.

The geometry of the existing canal was obtained from LDOTD "As-Built" plans (the drawings are included in Appendix H); it has side slopes of 3H:1V with revetment down to existing grade (approximate elevation 0.00 ft MSL) and an approximate invert at elevation -5.00 ft MSL. The channel will not be modified in this area. This geometry is shown in Figure 8-25.

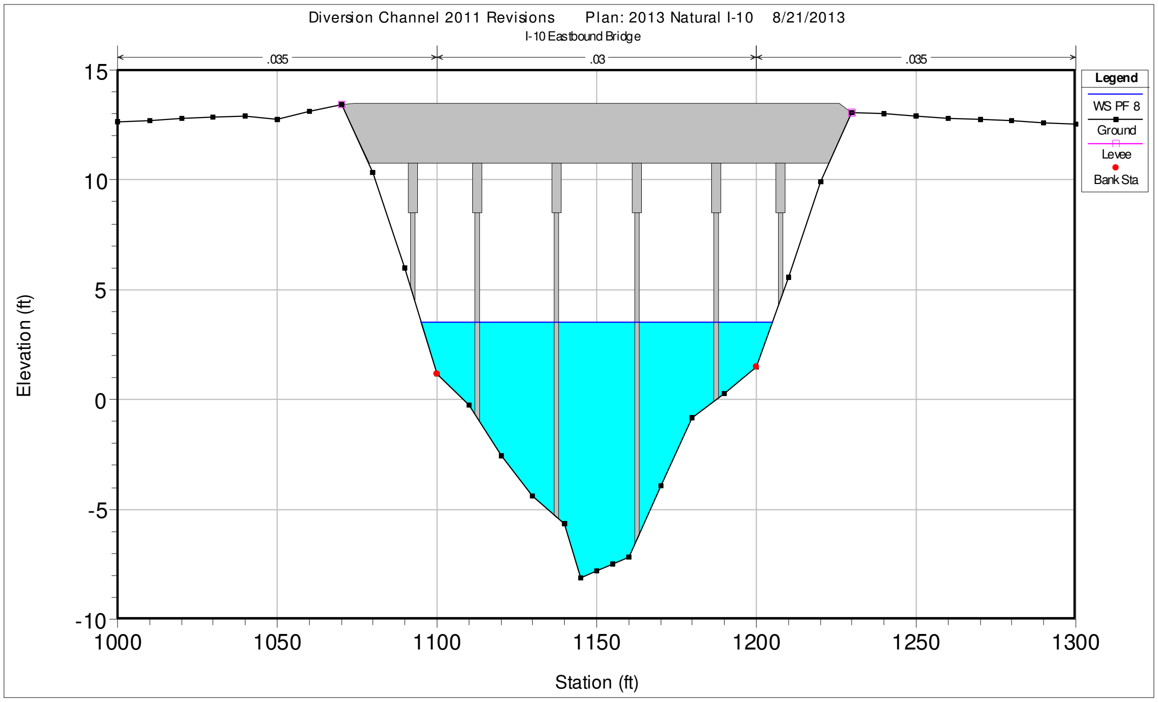


Figure 8-26: The diversion channel will will flow through an unmodified hope canal in the vicinity of the I-10 crossing as to have no impact on the existing bridges.

Manning's *n* values for the channel upstream and downstream of the bridge were set at 0.03 for the main channel and 0.035 for the over-bank, which are typical values for dredged channels. For the channel section through the structures, starting 85-ft upstream of the eastbound bridge to 85-ft downstream of the westbound bridge, an *n* values of 0.033 was selected to model the rip-rap and/or revetment lining. Contraction and expansion coefficients were chosen as follows: 0.1 and 0.3 for the main channel upstream and downstream of the bridges, respectively; 0.6 and 0.8 where the main channel transitions to the bridge channel 75-ft upstream of the eastbound bridge and 75-ft downstream of the westbound bridge, respectively; and 0.3 and 0.5 for the start and end of channel sections through the bridges, respectively. The channel banks are set at elevation +2.60 ft NAVD88-LDNR. The channel levees from the main channel shall be extended into the interstate embankment, thus neither ineffective areas nor transitions were necessary.

HEC-RAS models bridges by comparing the head losses calculated by the energy, momentum and Yarnell methods and selecting the highest value. The momentum equation has a pier shape drag coefficient of 2 for rectangular piles. The Yarnell equation has a pier shape factor of 1.25 for rectangular piles. For the subject channel, the momentum method governed for design flows and lower, while the energy method governed for flows greater than the design flow.

Channel Design, I-10 to Discharge

The channel north of I-10 was modeled as a trapezoidal shape with a 40-ft wide bottom at an invert of -8.00 ft NAVD and 1V:5H side slopes for a length of approximately 1,000-ft from the downstream face of the westbound I-10 structure. Manning's *n* was set to 0.03 for the main channel and 0.035 for the over-bank. Contraction and expansion coefficients were set to 0.1 and 0.3, respectively. The channel banks were set to elevation +2.55 ft NAVD88-LDNR. Levees were set to elevation +6.50 ft NAVD88-LDNR and have a 12-ft wide crown. This geometry is shown in Figure 8-26. There are no structures within this reach of channel.

This reach of channel terminates with a 50-ft transition from the design channel to the existing channel. The termination was not modeled but should be fully protected with rip-rap on both sides of the levees and on the side slopes and channel bottom.

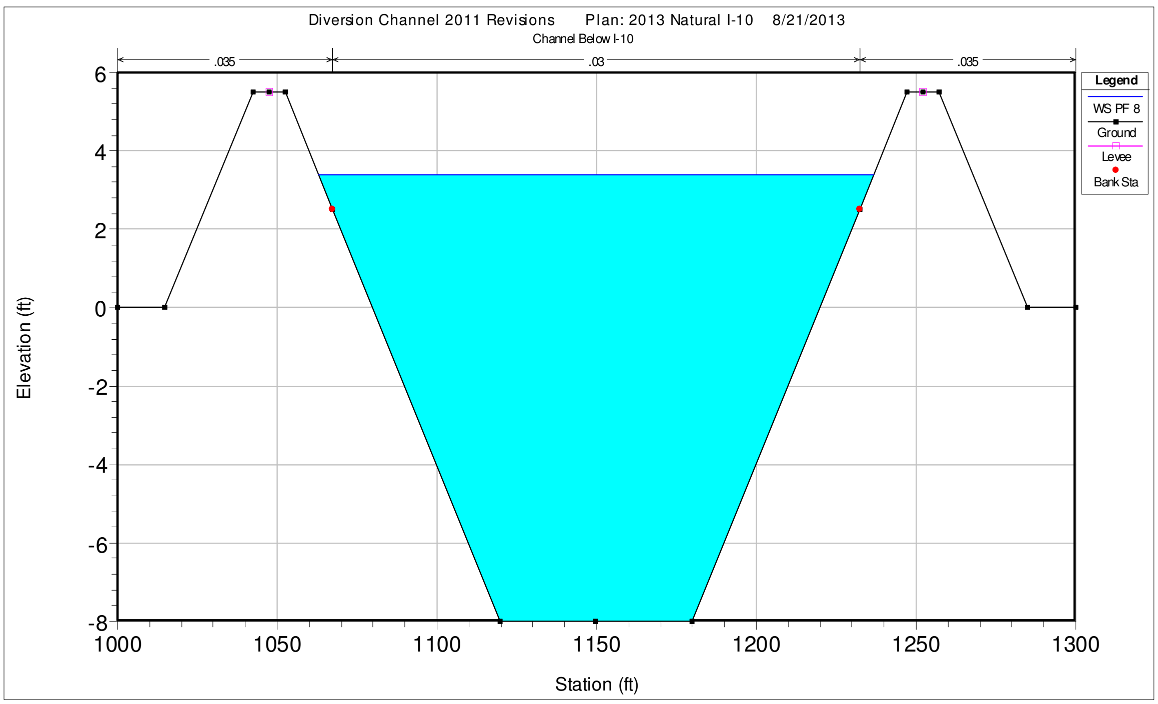


Figure 8-27: A typical cross-section of the diversion channel above I-10.

Channel Roughness

For the 35% design a sensitivity analysis was performed to select the appropriate manning's n values for the channel. Subsequent dialogues with USACE over concerns of maintenance result in the decision to model the diversion with a more conservative manning's n of 0.03 for the channel and 0.035 for the overbanks.

9.0 PROJECT DESIGN

Introduction

The intake structure must convey flow from the Mississippi River into the diversion channel through the levee and underneath LA 44. To maximize the opportunities for conveyance of the 2,000 cfs design flow, the structure was designed to minimize head loss. It must also function reliably over the life of the project and under a wide range of conditions. The most cost-effective design to achieve these objectives was determined to be a multi-cell box culvert with vertical lift gates, also known as sluice gates. This arrangement of box culverts and sluice gates are used for similar diversion structures along the Mississippi River at Caernarvon (Eastbank of the Mississippi River near the St. Bernard and Plaquemines Parish line) and Davis Pond (Westbank of St. Charles Parish). Since these diversions have been operating successfully for a number of years using the proposed configuration, they provide an excellent example to follow. Several other types of diversion structures, including, pumps, tainter gates, and siphons were evaluated for the previous projects and were deemed unsuitable because of cost and/or functionality constraints.

Intake Structure Location

The proposed intake structure will be located approximately 100-ft south of the crown of the existing Mississippi River levee. The proximity to the levee enables both the intake structure and support platform to be constructed on as solid a foundation as possible. The platform will support a control house, to be built at a top floor elevation of approximately 33.5-ft NAVD88-LDNR for protection against high river stages. Placing the structures close to the levee also minimizes the required length of the culverts and therefore the head losses through them.

The elevations of the intake structure, sluice gates, and culverts were geometrically constrained by several factors. First, the gate should be set as high as possible to minimize the excavation costs. Second, since the water surface elevation on either side of the culverts will be above their crowns, the diversion structure will operate under outlet control. Under this scenario, the slope of the culvert is irrelevant to its hydraulic performance. Therefore, the culverts will be installed flat, with no slope from the upstream to downstream ends. Third, the culverts will have to pass under LA 44, which has a roadside drainage ditch with an invert of +7 ft NAVD88-LDNR. So, the elevations were established initially by assuming that the culverts will require at least 1-ft of cover from the existing grade at the bottom of the ditch. Subtracting the 1-ft of cover from the +7 ft NAVD88-LDNR elevation of the ditch invert yields a top of culvert elevation of +6 feet NAVD88-LDNR. Finally, preliminary investigations indicated that the top wall of the culverts may have to be up to 3-ft thick. Subtracting an additional 3-ft for the culvert wall thickness results in a top-of-gate elevation of +3-ft NAVD88-LDNR.

Facility Sizing

The design flow rate and head-loss were the two primary factors used to design the intake structures. The objective was to minimize the head-loss while maximizing the time during which the design flow of 2,000 cfs could be achieved. Both of these factors ultimately depend upon the stage of the Mississippi River; without a river stage that exceeds the tail-water elevation by a sufficient amount, the diversion structure will be incapable of conveying the requisite flow.

The elevation difference between the head-water (water level in the Mississippi River) and tail-water (water level in the conveyance channel) is the basis for computing the required diversion structure gate openings. The tail-water was computed through a backwater analysis beginning at the Maurepas Swamp and ending at the intake structure. When the calculated tail-water conditions are compared with the seasonal Mississippi River Stages, it is feasible to predict the times throughout the year when the diversion will be able to deliver the desired 2,000 cfs design flow. (Appendix E contains the "Mississippi River Stage Hydrographs at the Reserve Gage".) To maximize the duration of peak flow conditions, the head differential must be kept to a minimum. At the 2,000 cfs design flow, the water surface elevation on the outfall (tail-water) side of the diversion structure will be at approximately +7.0 ft NAVD88-LDNR. This requires a minimum river stage of +7.0 ft NAVD88-LDNR plus the head losses through the intake structures at the design rate of flow.

Table 9-1 compares the geometric and hydraulic performance parameters of nine sluice gate configurations, ranging from a single 12-ft x 12-ft gate to three 8-ft x 8-ft gates. The table is based upon hand calculations* and operation at the design flow rate of 2000 cfs.

Number of Gates	Width (ft)	Height (ft)	Area per Gate (ft^2)	Total Area (ft^2)	Minimum Head Difference (ft)	Required River Stage (ft, NAVD)
3	12	12	144	432	0.6	+7.60
2	14	14	196	392	0.8	+7.80
3	11	11	121	363	0.9	+7.90
3	10	10	100	300	1.3	+8.30
3	9	9	81	243	1.9	+8.90
2	10	10	100	200	2.8	+9.80
3	8	8	64	192	3.1	+10.10
2	9	9	81	162	3.6	+10.60
1	12	12	144	144	5.4	+12.40

Table 9.1 – Sluice Gate Comparison Chart

*Note: The hand calculations shown in this section and the HEC-RAS model use very different approaches to calculate the river stage needed to generate the design flow. Each method requires the selection of appropriate head-loss coefficients, equations, and various other parameters. The fact that the results are within 1-ft of each other indicates a general agreement between the two methods.

The main factors used to evaluate the various configurations are cost and functionality. The cost is directly proportional to the size of the structures, which is reflected in the "Total Area" column of Table 9-1. Thus, three 12-ft x 12-ft gates, which encompass a cross-sectional area of 432 ft^2, will require a lot more concrete and steel, and thus cost significantly more, than three 8-ft x 8-ft gates, which only provide a 192 ft^2 cross-section. On the other hand, the larger cross-section of the 12-ft x 12-ft gates provides much less resistance to flow. That means the head differential required to overcome the losses through the bigger openings is also much less. That translates to a lower river stage required to deliver the design 2,000 cfs flow rate, which means that the diversion could provide the target flow for a greater portion of the year. In short, increased size costs more, but it also delivers an improvement in the hydraulic performance.

Based on historical river stage data and the goal of operating at full capacity for at least half of the year, a river stage of +8.3-ft was established as the point for which the intake structure should be designed. As Table 9-1 shows, three 10-ft x 10-ft gates was selected as the optimum configuration, since it will minimize construction costs while also meeting the flow delivery requirements.

The diversion has been designed to overcome the head losses created by both the structures encountered as well as those developed throughout the channel itself. The structural head-losses include those through the intake gates and all of the crossings, including those at the intake, at the railroad crossings, and at the roadway crossing. The channel head-losses are due to the friction from the canal walls as well as changes in the channel alignment and cross-section from one location to the next. The total head-loss is not only a function of the physical attributes of the channel and structures, but it is also dependent upon the quantity of flow being conveyed. Thus, the driving head, which is provided by the Mississippi River stage, must increase to push more flow through the diversion. The design must take into account this relationship of the stage required to overcome the losses associated with a given flow. Based upon the historical river stage data and the hydraulic modeling results, on average, the conveyance channel will be able to deliver the design flow of 2,000 cfs from December through May. From June to November the channel will still be able to divert flow, but only in the range of approximately 500 to 1,500 cfs.

Operations

The water surface elevations in the Mississippi River and conveyance channel will be measured by gages upstream and downstream of the diversion structure. Using the head differential data, the gate operator will adjust the gate openings to control the flow through the channel. The operation of the diversion structure will be conducted in a manner similar to that of the structures at Caernarvon and Davis Pond. The following calculations were used to determine the gate openings required to achieve a range of discharge values. A table of the "Head Differential" vs "Gate Opening" will be provided in the operation Manual for the facility.

Orifice equation: $Q = C\,A\sqrt{2\,g\,H}$ **Equation 9-1**

Continuity Equation: $Q = A\ v$ **Equation 9-2**

Where:
Q = Quantity of flow (cfs)
C = Discharge coefficient (--)
A = Cross-sectional area (ft^2)
g = Gravitational constant (32.2 ft/sec^2)
H = Head differential (head-water vs. tail-water) (ft)
v = Velocity (ft/s)

To model the hydraulic performance of the gate opening, the discharge coefficient (C) must first be determined. This coefficient has been calculated as follows:

$$C = \left(\frac{1}{\left(1.4 + f\left(L/D\right)\right)}\right)^{1/2}$$ **Equation 9-3**

Where:
C = Discharge coefficient (--)
f = Friction factor = .013 (--)
L = Length of culvert (ft)
D = Diameter of equivalent circular section (ft)

Since the culvert is a box in this case, the hydraulic radius must be equated to an equivalent diameter. By relating the diameter (D) of a circular section to its hydraulic radius (R), we obtain:

R = D/4
R = Area / Wetted perimeter = A/P_w
A = (No. culverts) · (Culvert height) · (Culvert width)
3 x 10 ft x 10 ft = 300 ft^2
P_w = (No. culverts) · (Wall length) · (No. walls)
3 x 10 ft x 4 = 120 ft
R = 300 ft^2 / 120 ft = 2.5 ft

The discharge coefficient can now be found by substituting into Equation 9-3:

$$C = \left(\frac{1}{\left(1.4 + .013\left(300/4(2.5)\right)\right)}\right)^{1/2} = 0.747$$

The area term, A, in Equation 9-1 is a function of the gate opening height:

$$A = 3\,Gates \times Width \times Opening\,Height = 3 \times 10 \times O_H = 30\,O_H$$

Substituting the above expression for A and the calculated value for C into Equation 9-1, and then re-arranging enables us to solve for the opening height, O_H. For a flow rate of

2,000 cfs, a 2-ft head differential between the head-water and tail-water across the intake structure, and three 10-ft wide gates, the required gate opening should be:

$$O_H = \frac{Q}{30\ C\ \sqrt{2\ g\ H}} = \frac{2{,}000\ cfs}{30\,ft\ (0.747)\ \sqrt{2 \times 32.2\,ft/s \times 2\,ft}} = 7.86\ ft$$

Using the calculated gate opening, the velocity through the gate can be calculated from Equation 9-2 as follows:

$$Q = A\ v \quad \rightarrow \quad v = \frac{Q}{A} = \frac{2{,}000\ cfs}{3\ gates \times 10\,ft \times 7.86\,ft} = 8.48\ \frac{ft}{s}$$

Note that this is strictly the velocity through gate; the velocity in the intake channel is significantly lower due to its much larger cross-sectional area.

<u>Outlets between US 61 & I-10</u>

Between I-10 and US 61 there will be four points at which pipes will traverse the levee and carry flow from the conveyance channel to the areas east and west of the channel. These points will be spread out evenly from the pump station north of US 61 to just south of the Interstate. The flow will be carried by means of 24-in reinforced concrete pipes approximately 80-ft long. There will be a total of eight pipes, four on each side, carrying a combined flow of 280 cfs. Each pipe is designed to carry slightly over 30 cfs. The estimated discharges through each of the conduits at the four stations along the conveyance channel are tabulated in Table 9-2. When the conveyance channel is operating at full design flow, the cumulative flow through the lateral pipes will be approximately 280 cfs.

The discharge pipes exiting the conveyance channel will also have controllable knife-gate valves that can be manually closed should a drainage problem arise. Knife-gate valves are a cross between a regular gate valve and a sliding gate valve. This type of valve is ideal when attempting to control a fixed flow under low pressure. Knife-gate valves also seat better due to their "knife-like" closing mechanism. The closing action pushes any sediment or solids out of the seating area and provides a tighter seal. Headwalls will be installed at the ends of the pipes; these will provide a location for the knife-gate valves to be installed.

The surface runoff between US 61 and I-10 drains north towards the Interstate. The 280 cfs of lateral discharge is designed to be evenly split, delivering approximately 140 cfs of freshwater to each side of the conveyance channel. This water will disperse throughout the area between the two roadways and follow the natural drainage gradient to the north. The topography of the area and the orientation of the existing culverts suggest that the additional water will not cause drainage problems. The portion of the discharged water that does not either infiltrate into the ground or collect in ponding areas will eventually travel to the culverts underneath I-10 and proceed northward. The existing culverts underneath I-10 between Mississippi Bayou and LA 641 will be equipped with one-way check valves to prevent the movement of any excess water southward across I-10. These lateral discharge conduits were originally recommended in the Task 1 hydraulic study.

Station	Headwater in Conveyance Channel (ft)	Tailwater, Eye of Pipe Datum (ft)	Diameter of Pipe (inches)	Area of Pipe (ft)	Anticipated Flow (cfs)
Sta. 185+00	5	2.00	24	3.14	31.88
	4	2.00	24	3.14	26.03
	3	2.00	24	3.14	18.40
	2	2.00	24	3.14	0.00
Sta. 210+00	5	1.00	24	3.14	36.81
	4	1.00	24	3.14	31.88
	3	1.00	24	3.14	26.03
	2	1.00	24	3.14	18.40
Sta. 235+00	5	1.50	24	3.14	34.43
	4	1.50	24	3.14	29.10
	3	1.50	24	3.14	22.54
	2	1.50	24	3.14	13.01
Sta. 260+00	5	1.00	24	3.14	36.81
	4	1.00	24	3.14	31.88
	3	1.00	24	3.14	26.03
	2	1.00	24	3.14	18.40

Table 9-2: Lateral Discharge Pipe Operation Data

I-10 Drainage Culverts

The outlet for the conveyance channel is approximately 1,000-ft north of I-10, along the existing centerline of Hope Canal. At this point the diverted water will begin to overflow the canal banks and dissipate into the area above I-10, south of Lake Maurepas. In Task 1, the hydraulic modeling showed that some of the diverted water could backflow into the area south of I-10, since cross-drains traverse underneath the interstate. Drainage within this area typically travels northward, so any backflow from the area north of I-10 could impede the normal drainage patterns. The installation of check valves on the culverts underneath the roadway is proposed to mitigate for this potential problem.

A check valve is an apparatus that allows flow to travel in only one direction. The valves proposed have specially manufactured rubber devices that are installed on the discharge side of a culvert. The rubber apparatus is flexible enough to open and allow discharge under even low head conditions, yet stiff enough to remain closed and thus prevent flow from entering the line from the outside. The pressure created on the exterior of the valve by reverse flow or submersion will actually seal the lips tightly together, preventing backflow into the culvert. These valves will be installed on the outflow side (north side of I-10) of each drainage culvert in the affected area.

Flow Monitoring and Water Quality Analysis

As discussed earlier, the proposed sluice gates will be operated to accommodate an estimated 2,000 cfs flow, during adequate river stages. For the gates to function at their highest efficiency, the operational charts must be calibrated with real-time data. URS suggests installing flow monitors within the culverts to insure that the design flow is adequately met under various operating conditions. The proposed flow monitors are most accurate within a defined closed area. Since there will be three culverts located at the intake structure, three monitors are needed to give a cumulative flow reading. The flow rate information from these monitors will be displayed within the control room. Based on these flow measurements, the sluice gate operational chart can be calibrated. Additional flow measurement devices will probably be installed downstream from the intake structure. These flow monitoring devices have typically been provided by USACE or the United States Geological Survey (USGS) and displayed on-line for public view.

At the EPA's request, a water quality monitoring device will also be incorporated into the intake structure design. This device will measure multiple water quality parameters, including the temperature, turbidity, and concentration of various chemical species within the diverted water. The measurements obtained can be recorded within the device itself or stored on a hard drive in the control room. .

<u>Cofferdam</u>

An earthen cofferdam will be used as temporary flood protection while the existing levee is removed for construction of the intake structure and the headworks. The cofferdam design conforms to USACE provided criteria as included in the Appendix C. The cofferdam and associated braced wall retaining walls are presented in plan sheets . The construction of the intake structure, and headworks will be accomplished in several phases as summarized below (Refer to Plan Sheets HC-3.01 thru HC-6.03):

Phase I – Construct Access Ramps and Partial Cofferdam

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

Phase II – Completion of Cofferdam Construction

Complete cofferdam construction to El 32.

Phase III – By-Pass Roadway and Initial Culvert Construction

Remove section of landside toe of MRL, construct by-pass roadway S of existing River Road, remove section of River Road, install sheet piling for excavation on N side of by-pass, excavate, construct temporary access road to bottom of excavation, install culvert sections C-4, C-5, C-6, U-4, U-5 & U-6, and remove sheet piling.

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

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

Phase V – Construction of Culvert on South End

Install sheet piling for excavation both N and S of the culvert, partially excavate, install mechanically stabilized earth on each side at N end of culvert, complete excavation, construct temporary access roads to bottom of excavation, install culvert sections C-1, C-2, & U3 as well as intake structure, remove sheet piling and backfill

Phase VI – MRL Construction and Cofferdam Removal

Reconstruct MRL to El 33.5, providing overbuild for anticipated settlement, tie E and W ends into original section, install slope paving except for small area adjacent to intake structure, degrade cofferdam to batture elevation El 18.

Phase VII – U-Channel Construction

LA 44 Crossing Traffic Impacts

During Task 2 based on communication with LDOTD it was considered that the installation of the culverts across LA 44 will be an open cut section. The Design Team had met with LDOTD in 2007 to discuss this issue and arrived at a consensus for traffic control.

In early 2013, Subsequent to the cofferdam submittal to USACE, URS contacted the LDOTD to update them on the project status and inquire as to what drawings would be needed for approval of the project crossings under their roadways. Specifically, this included the crossings at River Rd (LA 44), Airline Highway (US 61), and Interstate 10. During that discussion, URS was informed that the LDOTD would only allow River Rd to be closed to traffic for a 45-day period, instead of the year-plus duration that had been indicated in our past discussions with them. While this change does not affect the design of the intake and headwork features, maintaining traffic on the roadway during construction dramatically changes the entire approach to building the facilities. That made addressing the sequence of construction. To that end, URS has re-designed every stage of the construction process and analyzed each one to insure that it meets the USACE's factors of safety for geotechnical stability.

Under the changed conditions required by the LDOTD, i.e., having to maintain the roadway in service, the contractor would be restricted to working on only one side of it at a time. This would effectively cut his working area in half (depending on how far the road is relocated in either direction). To excavate to a depth of 30-ft below grade (the batture is at elevation +18 and the bottom of the culvert is at -12) without sufficient room to open-cut, requires some type of braced excavation. URS proposed the above mentioned phases under the cofferdam section as a workable design using temporary retaining structures, where required, to facilitate construction.

Velocity and Endangered Species Considerations

The EPA has identified a potential impact to the endangered pallid sturgeon due to increased velocity in the vicinity of the diversion structure. To prevent the sturgeon from being entrained into the diversion flow, the velocity must be kept below the threshold beyond which the fish cannot escape. The inflow channel is within the batture area between the toe of the levee and the bank of the river. The relatively wide cross-sectional area of the inflow channel will reduce the velocity below that commonly encountered in the river. Based on historical data obtained from the USACE website, the velocity in the Mississippi River ranges from 2.0 ft/s at low stage to 9.0 ft/s at high stage. The hydraulic model shows that the velocity within the inflow channel reaches a maximum of 2.5 ft/s. Therefore, the pallid sturgeon will not be adversely impacted by the inflow channel.

At the intake gates, with a fixed design flow of 2,000 cfs, the velocity is dependent upon the size of the gate opening. As the gate opening widens, the velocity near the gates will decrease, conversely it will increase as the gate opening becomes smaller. The gates were designed to minimize the amount of head-loss while achieving the design flow. Velocities near the intake structure can become high when the gates are only slightly open, such as when they start opening from a shut position or they are just about to close. However, these higher velocities will only occur for very brief periods during the opening and closing operations. It is important to bear in mind that these increased velocities will only affect fish in extremely close proximity to the gates. Any sturgeon within the inflow channel will have more than adequate opportunity to reverse direction. Both the Caernarvon and Davis Pond diversion structures operate similar to the proposed configuration, without reported instances of negative impacts on pallid sturgeon. Due to their higher design flows both of these structures have higher calculated velocities than those anticipated for the proposed Maurepas Diversion.

Gate Mechanical Design

The proposed gates will be operated by a hydraulic system. A fluid reservoir will hold excess hydraulic fluid to accommodate volume changes from cylinder extension and contraction, temperature shifts, and leaks. The reservoir is also designed to aid in the removal of air from the fluid and it functions as a heat accumulator to cover system losses when peak power is used. It is recommended that the hydraulic system's fluid reservoirs be constructed of stainless steel for corrosion resistance. Non-stainless steel reservoirs are typically painted, but the areas subject to corrosion are extremely hard to access and are therefore often not painted properly. Although stainless steel reservoirs are more expensive they require less long-term maintenance and are less likely to fail. Additionally, the hydraulic reservoir will be protected from the intrusion of water vapor and other contaminants by the use of a hydraulic reservoir isolator. The reservoir isolator provides a closed system into which the reservoir may breathe.

Sedimentation Basin Design

There is a large amount of both organic and inorganic sediment entrained in the Mississippi River flow-stream. Controlling the amount of sediment transported into the conveyance channel is a significant design variable. A balance must be struck between the amount of sediment transported to re-nourish the swamp and the amount to be settled out in the upstream sediment basin to avoid constant dredging operations within the conveyance channel.

A review of the technical literature indicates that the conveyance channel will receive a much lower concentration of heavy particles carried in suspension than is conveyed within the river as a whole. The Mississippi River is much deeper than the proposed conveyance channel; in a typical water column, the heavy sediment particles tend to travel close to the bottom. Since the channel will receive water from the upper portion of the river, there will be a lower concentration of heavier sand particles in the diverted flow-stream.

USGS water quality data from the Mississippi River collected at Belle Chasse during the years of 1977-1997 is shown graphically in Figure 9-1. The figure depicts the percentage of sand in the total suspended solids (TSS) of the river water column.

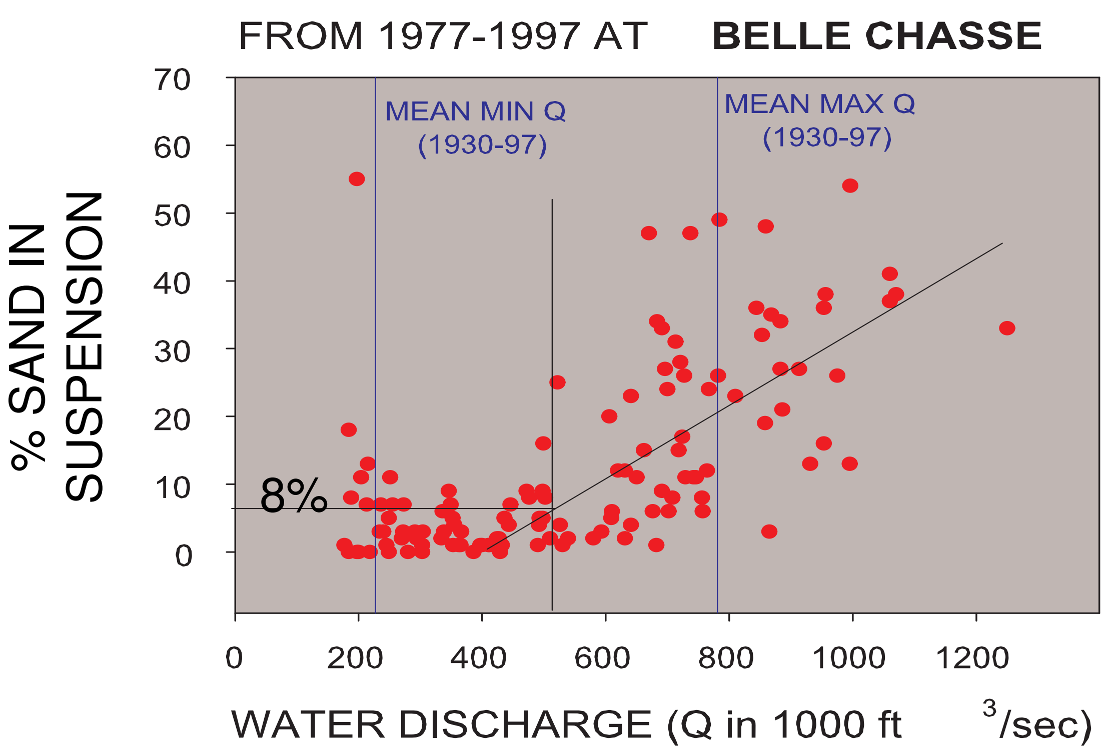


Figure 9-1: % Sand carried in suspension in the Mississippi River near Belle Chasse (from USGS Water Quality Data).

The mean minimum and maximum flows over the 67-year interval from 1930-1997 are superimposed onto Figure 9-1, as indicated by the two vertical blue lines. The black line in the middle of the two designates the average flow of the Mississippi River. The diagonal line is a linear interpolation of the scattered data. The intersection of the average flow line and the line fitted to the data corresponds to the average percentage of sand within the total mass of suspended sediment, which is 8%.

A suspended sediment grain size analysis run on water from within the Caernarvon diversion channel shows a particle size distribution of 63% silt, 36% clay, and only 1% sand. Thus, the percentage of sand is far greater within the river at Belle Chasse, which is in close proximity to the Caernarvon site, than that in the nearby diversion channel. A drop in the percentage of sand from 8% to 1% has a significant impact on the design of the sedimentation basin. Per LDNR's request, the basin must have adequate storage capacity to accumulate six month's of sediment without requiring cleaning.

The percentage of sand in the Mississippi River water column increases in the upstream direction. Figure 9-2 shows the USGS Mississippi River water quality data collected at St. Francisville, which is upstream from Belle Chasse, between 1978-2001. The data at St. Francisville indicate an average composition of 18% sand within the total suspended sediment load. Note that the subject Maurepas project location is situated between St. Francisville and Belle Chasse.

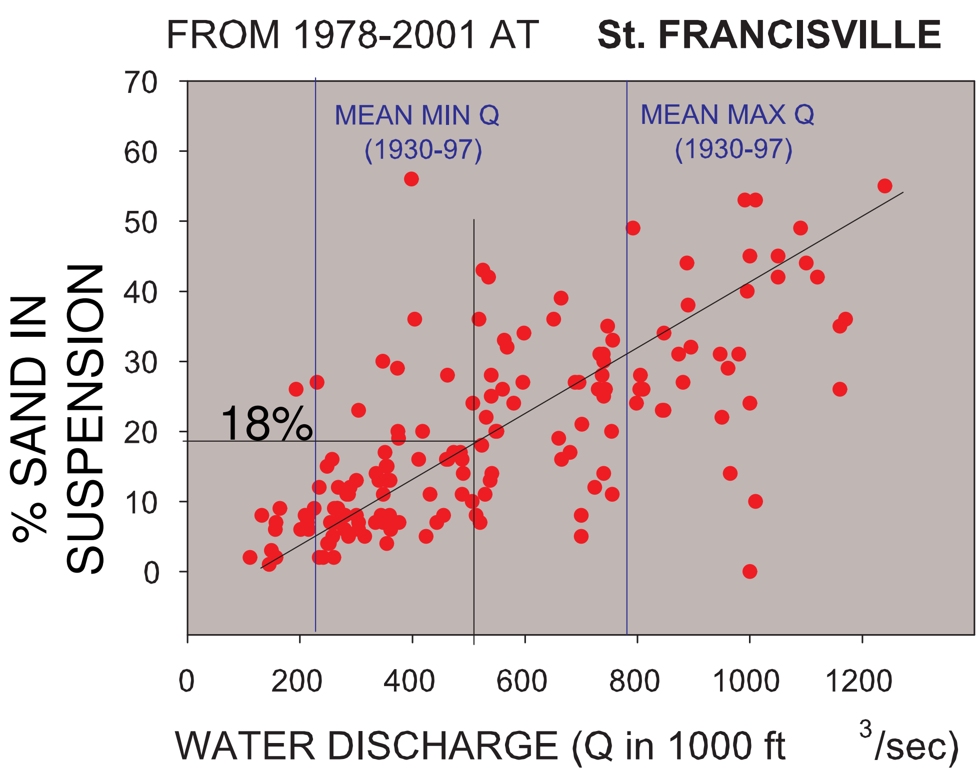


Figure 9-2: % Sand carried in suspension in the Mississippi River near St. Francisville (from USGS Water Quality Data).

The percent of sand expected within the Maurepas conveyance channel was derived by extrapolating from the recorded data. At Belle Chasse, the percent sand in the river is 8% and at St. Francisville it is 18%, while the value at Maurepas is unknown. However Maurepas is between the two sampled locations; in relative terms, it is approximately 2½ times farther from St. Francisville than it is from Belle Chasse. Assuming a linear interpolation of the data between the two end-points, yields about 11% sand in the river at Maurepas. At Caernarvon, the river carries 8% sand while the channel has only 1%, thus the channel receives one-eighth of the river's sand load.

Applying this ratio to the proposed Maurepas project yields a concentration of approximately 1.36% sand in the subject diversion's TSS load. However, since these calculations are being performed with data that varies widely, a large factor of safety was added to this value in order to insure a conservative design. Based upon the extrapolations of the documented sediment data, along with a generous safety factor, the proposed sediment basin will be designed for 5% sand in the influent water.

Determining the percentage of sand within the TSS conveyed into the channel is the first step in sizing the sedimentation basin. The next step is to establish the target particle size which is to be captured. Based upon experience in other diversion projects, the LDNR has indicated that the majority of the sand should be trapped before it reaches the swamp. They further defined the smallest particle which needs to settle out in the sedimentation basin as a 0.2 millimeter (mm) sand particle. The sedimentation basin has been designed to remove all sand particles this size and larger.

The sedimentation process is basically just the gravitational settling of particles through a liquid due to their greater density. A suspended particle is acted upon by three forces: 1) gravity pulls it downward, 2) buoyancy pushes it upward, and 3) frictional resistance impedes its movement; in the case of settling, this force also acts upward. The rate of settling is a function of the size and density of the particle and the viscosity and density of the fluid. The particle size and density determine its weight and therefore the gravitational force acting downward. The difference in densities between the particle and the fluid along with the particle's volume determine the upward buoyant force. The viscosity of the fluid creates the frictional resistance to movement.

For a 0.2 mm particle of sand settling through a column of water, all of the above variables are known. Table 9-3 lists the settling velocities for various sizes and types of particles (sand, sand & silt, silt, silt & clay, and clay) moving through water at 50 ºF (adapted from "Integrated Design of Water Treatment Facilities" by Kawamura, 1991).

Type of Particle	***Mesh Size***	***Particle Diameter (mm)***	***Particle Settling Rate***	
			(mm/s)	**(ft/min)**
Sand	18	1.0	100	19.7
Sand	20	0.85	73	14.3
Sand	30	0.6	62	12.2
Sand	40	0.4	42	8.2
Sand	70	0.2	21	4.1

*Type of Particle**	Mesh Size	Particle Diameter (mm)*	Particle Settling Rate	
			(mm/s)	(ft/min)
Sand & Silt	100	0.15	15	3.0
Silt	140	0.10	8	1.6
Silt & Clay	200	0.03	6	1.2
Silt & Clay	230	0.06	3.8	0.75
Silt & Clay	400	0.04	2.1	0.41
Clay	-	0.02	.062	0.12
Clay	-	0.01	.0154	0.03

Table 9-3: Settling Velocities of Particles, from "Integrated Design of Water Treatment Facilities" by Kawamura, 1991.
*Particles classified by size based on the Udden-Wentworth scale.

The data for just the sand particles has been extracted from Table 9-3 and it is plotted on Figure 9-3.

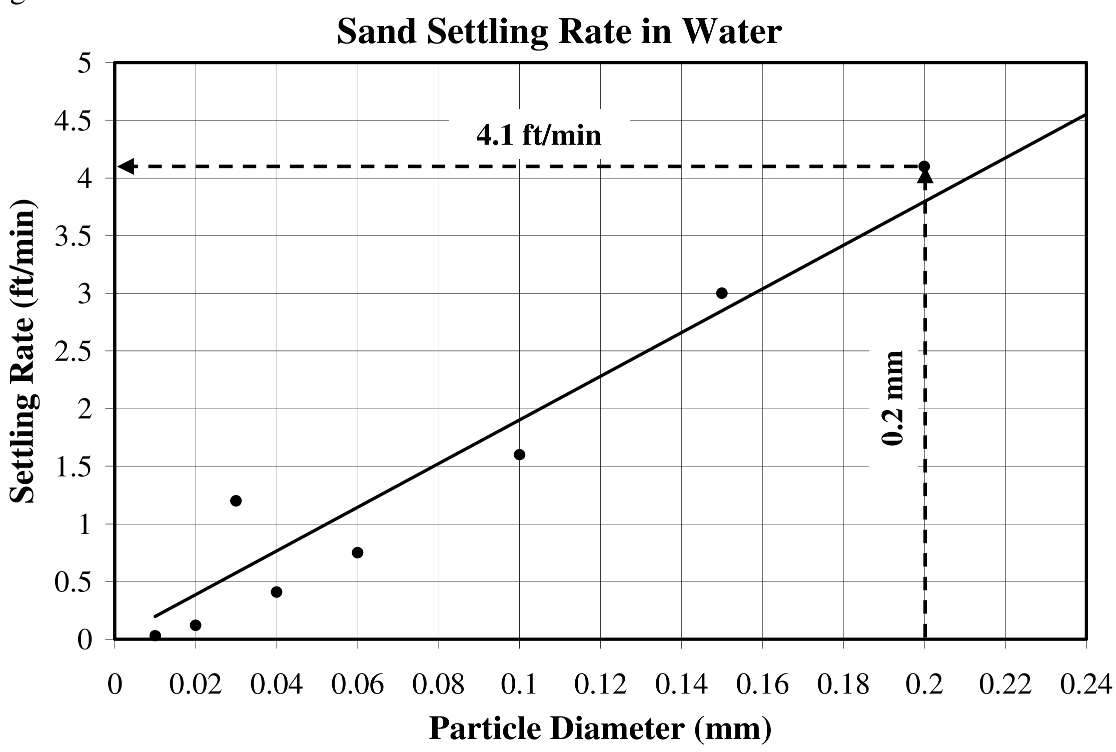


Figure 9-3: Sand settling rates in water. (Particles classified as sand based on size, using the Udden-Wentworth scale.)

Based on the sand settling rate data presented in Table 9-3 and Figure 9-3, a settling velocity of 4.1 feet per minute was selected as the design basis for sizing the sedimentation basin.

The time that a particle has to settle out in the sedimentation basin can be calculated by either dividing the vertical distance traveled by the vertical velocity (the settling velocity) or the horizontal distance traveled by the horizontal velocity (the fluid velocity). Equations 9-7 and 9-8 illustrate these two calculations.

$$t = \frac{D}{v_s}$$ **Equation 9-7**

$$t = \frac{L}{v_h}$$ **Equation 9-8**

Where:
t = Time, sec
D = Depth of sediment basin, ft
v_s = Vertical settling velocity of particle, ft/s
L = Length of sediment basin, ft
v_h = Horizontal velocity of fluid, ft/s

The horizontal velocity is simply the volumetric flow rate divided by the tank cross-sectional area, as shown in Equation 9-9:

$$v_h = \frac{Q}{W\ D}$$ **Equation 9-9**

Where:
v_h = Horizontal velocity of fluid, ft/s
W = Width of sediment basin, ft
D = Depth of sediment basin, ft

Substituting into Equation 9-8 yields:

$$t = \frac{L\ W\ D}{Q}$$ **Equation 9-10**

Since L x W x D equals the volume of the basin, the time equals the volume divided by the flow, or the hydraulic retention time:

$$t = \frac{V}{Q}$$

Setting Equations 9-7 and 9-10 equal yields: $$\frac{L\ W\ D}{Q} = \frac{D}{v_s}$$

Solving for v_s yields an expression that is independent of the basin depth: $$v_s = \frac{Q}{L\ W}$$

Recognizing L x W as the surface area (A_s, ft^2) of the basin, yields an expression for the volumetric discharge per unit surface area, or the surface settling rate:

$$v_s = \frac{Q}{A_S}$$

Converting v_s to the units of gal/day-ft^2, gives the Surface Overflow Rate:

$$SOR\ \frac{gal}{day\ ft^2} = v_s\ \frac{ft}{\min}\ 7.48\ \frac{gal}{ft^3}\ 1440\ \frac{\min}{day}$$

These are the units commonly used in water and wastewater treatment applications. For the subject sedimentation basin in the diversion channel, the units are converted to flow in ft^3/s per ft^2 of surface area, and then re-arranged to solve for the required surface area:

$$Surface\ Area\ ft^2 = \frac{Flow\ \dfrac{ft^3}{s}}{\left(v_s\ \dfrac{ft}{min} \times \dfrac{1\,min}{60\ s}\right)}$$ **Equation 9-11**

Solving Equation 9-11 with the given flow rate of 2,000 cfs and the settling velocity of 4.1 ft/min for the target 0.2 mm sand particle yields a required surface area of 29,268 ft^2 for the sedimentation basin. The central section of the basin is 265-ft long by 66-ft wide. The side slopes add an additional 60-ft of width on each side, making the total surface area 265-ft long by 186-ft wide = 49,290 ft^2. This is more than sufficient to achieve the desired theoretical removal efficiency. The actual removal efficiency of the basin will depend on the size and density distribution of the particles in the influent flow stream.

A second objective in the sedimentation basin design process was to provide sufficient cross-sectional area to insure that the fluid velocity is low enough to prevent re-suspension of the settled particles. For the prevention of sediment accumulation in typical drainage conveyance systems, a velocity criterion of 2 ft/s is used. In the case of the proposed Maurepas sedimentation basin, a lower velocity of 1 ft/s was selected to make sure that the accumulated sediment remains on the bottom of the basin.

With a fixed flow rate, particle size and settling velocity, the design variables remaining for reduction of the water velocity are the width and depth of the basin, which comprise its cross-sectional area. As the cross-sectional area increases, the fluid velocity and hence the horizontal velocity of the particles, decreases. When the velocity decreases the particles spend more time within the conveyance channel, which provides the opportunity for them to settle out. The width of the sedimentation basin and conveyance channel was limited to a 300 foot right-of-way corridor obtained by LDNR from private landowners.

Table 9-4 shows the relationship between the basin cross-sectional area and the water velocity, along with the settling rates and times for various particles to be captured within the basin. The 265-ft length of the basin divided by the water velocity yields the time in the basin. Particles that spend more time in the basin than required for them to settle are captured; those that transit through the basin faster are not retained.

Cross-Sect. Area (ft^2)	Type of Particle	Size (mm)	Settling Rate (fpm)	Water Velocity (ft/sec)	Time to Settle (min)	Time in Basin (min)	Does Particle Settle Out?
1000	Sand	0.40	8.2	2.00	1.71	2.21	Yes
1000	Sand	0.20	4.1	2.00	3.41	2.21	No
1000	Sand & Silt	0.15	3.0	2.00	4.67	2.21	No
1000	Silt	0.10	1.6	2.00	8.75	2.21	No
1000	Silt & Clay	0.03	1.2	2.00	11.67	2.21	No
1500	Sand	0.40	8.2	1.33	1.71	3.31	Yes
1500	Sand	0.20	4.1	1.33	3.41	3.31	No
1500	Sand & Silt	0.15	3.0	1.33	4.67	3.31	No
1500	Silt	0.10	1.6	1.33	8.75	3.31	No
1500	Silt & Clay	0.03	1.2	1.33	11.67	3.31	No

Table 9-4: Sedimentation basin cross-sectional area versus particle capture for a channel length of 265-ft.

Cross-Sect. Area (ft^2)	Type of Particle	Size (mm)	Settling Rate (fpm)	Water Velocity (ft/sec)	Time to Settle (min)	Time in Channel (min)	Does Particle Settle Out?
1700	Sand	0.40	8.2	1.18	1.71	3.75	Yes
1700	Sand	0.20	4.1	1.18	3.41	3.75	Yes
1700	Sand & Silt	0.15	3.0	1.18	4.67	3.75	No
1700	Silt	0.10	1.6	1.18	8.75	3.75	No
1700	Silt & Clay	0.03	1.2	1.18	11.67	3.75	No
1885	Sand	0.40	8.2	1.06	1.71	4.16	Yes
1885	Sand	0.20	4.1	1.06	3.41	4.16	Yes
1885	Sand & Silt	0.15	3.0	1.06	4.67	4.16	No
1885	Silt	0.10	1.6	1.06	8.75	4.16	No
1885	Silt & Clay	0.03	1.2	1.06	11.67	4.16	No
2150	Sand	0.40	8.2	0.93	1.71	4.75	Yes
2150	Sand	0.20	4.1	0.93	3.41	4.75	Yes
2150	Sand & Silt	0.15	3.0	0.93	4.67	4.75	Yes
2150	Silt	0.10	1.6	0.93	8.75	4.75	No
2150	Silt & Clay	0.03	1.2	0.93	11.67	4.75	No

Table 9-4: (Cont.) Sedimentation basin cross-sectional area versus particle capture for a channel length of 265-ft.

As Table 9-4 shows, the target 0.2 mm sand particle can be captured with a cross-sectional area of 1,700 ft^2; any smaller cross-section will not remove this size particle. Significantly larger cross-sections, such as 2,150 ft^2, will readily capture the 0.2 mm particle, but also remove particles down to 0.15 mm in diameter. This results in more particulates being removed than required, which has several drawbacks: 1) it costs more to construct a larger basin, 2) more sediment will be accumulated over a given time and thus the maintenance requirements will be greater, and 3) the smaller particles are needed to enable the accretion of land-mass, which is one of the key objectives of the project. As highlighted in yellow, the optimum cross-section is 1885 ft^2, which captures the 0.2 mm particle and yields a water velocity of approximately 1 ft/s.

Several factors contribute to the accumulation of sediment within the basin. Seasonal changes will influence the TSS concentration in the diverted water; data from the Caernarvon diversion illustrate fluctuations throughout the year. Estimates of the total sediment accumulation need to take into account these variances. The TSS levels at the Caernarvon diversion are in the range of 25 mg/l to 135 mg/l. Since the Maurepas diversion will be located upstream from Caernarvon, and the TSS concentrations increase in the upstream direction (as described above in comparing the data from Belle Chasse to that of St. Francisville), larger values are to be expected for the subject site. Therefore, the TSS concentration range selected as the design basis for the Maurepas diversion was from 80 mg/l to 200 mg/l.

Table 9-5 lists the monthly anticipated sediment accumulation, by weight and by volume, for the above range of TSS concentrations, assuming a commensurate change in sand percentage for the peak six months during the year.

TSS (mg/l)	% Sand	Flow Q (cfs)	Total Sediment (lbs/day)	Sand Accumulation Rate		
				Daily		Monthly
				Mass (lbs/day)	Volume (ft^3/day)	Volume (ft^3)
200	5	2,000	2,160,343	108,017	908	27,231
150	5	2,000	1,620,257	81,013	681	20,423
100	5	2,000	1,080,172	54,009	454	13,616
120	4	1,500	972,154	38,886	327	9,803
100	4	1,000	540,086	21,603	182	5,446
80	3	500	216,034	6,481	54	1,634
Total accumulation over six months:						**78,154**

Table 9-5: Anticipated sand accumulation rates.

The chart above illustrates that the total volume which the sedimentation basin could accumulate over a six-month period is approximately 78,000 cubic feet. Therefore, a provision must be made to accommodate this volume of sediment. As described above, the length and width of the central area of the basin are 265-ft and 66-ft, respectively. To provide the needed volume, the cross-section of the basin will be constructed approximately 4-ft deeper than that of the channel. Based upon the geotechnical side slope stability analysis, the sedimentation basin will require the same 3H:1V side slopes as the adjacent channel sections. The invert for the sedimentation basin will be established at -11 ft NAVD88-LDNR. Thus, the sediment storage area will extend up the side slopes to elevation -7 ft NAVD88-LDNR, across a horizontal distance of 12-ft. The resulting available storage volume will be 82,680 ft^3, which can be calculated as follows:

Volume = Length x (Central Area + Triangular Side Areas)

Volume = 265-ft x (66-ft x 4-ft + 2 sides x ½ x 12-ft x 4-ft) = 82,680 ft^2

In the preliminary design report submitted to CPRA in 2008 for the sedimentation basin included a deep soil improvements based approach. This approach incorporated the use of Deep Mixing Method along the entire stretch of the sedimentation basin. This approach was utilized due to the method of maintenance required for the sedimentation which incorporated the use of heavy equipment such as an excavator system similar to CL 385 Hydraulic excavator (95 ton operating weight, equipment similar to one Manufactured by Caterpillar). The soil improvements were necessary to meet the levee factors of safety provided by the USACE for the maintenance cases. This equipment would be located along the crown of the levee and have the ability to excavate out the settled sediment from that location. URS believed this form of maintenance created a situation that may unnecessarily drive up the cost of construction of the basin.

During Task 3, URS proposed and designed ways and means to reduce the construction costs of the sedimentation basin. By following the USACE criteria for levee factors of safety (FOS) for construction loading conditions, URS developed the alternative to remove sediments by creating a ramp down into sedimentation basin. This alternative proposes using lighter equipment like Backhoe loaders (15 tons operating weight, e.g. CAT 450F, information attached) to travel down into basin and conduct maintenance operations. This option eliminates use of soil mixing and incorporates grouted riprap surface resulting in an estimated $3 million cost saving in construction.

The levee elevation in the vicinity of the sedimentation basin shall be set at +9 ft NAVD88-LDNR. During the 2,000 cfs design flow, the highest water surface level anticipated in the basin will leave 1.5-ft of freeboard remaining.

The following figures illustrate the typical cross-sections of the conveyance channel on the north and south sides of US 61. The entire length of the channel will have a 12-ft wide access road along the crown of both guide levees.

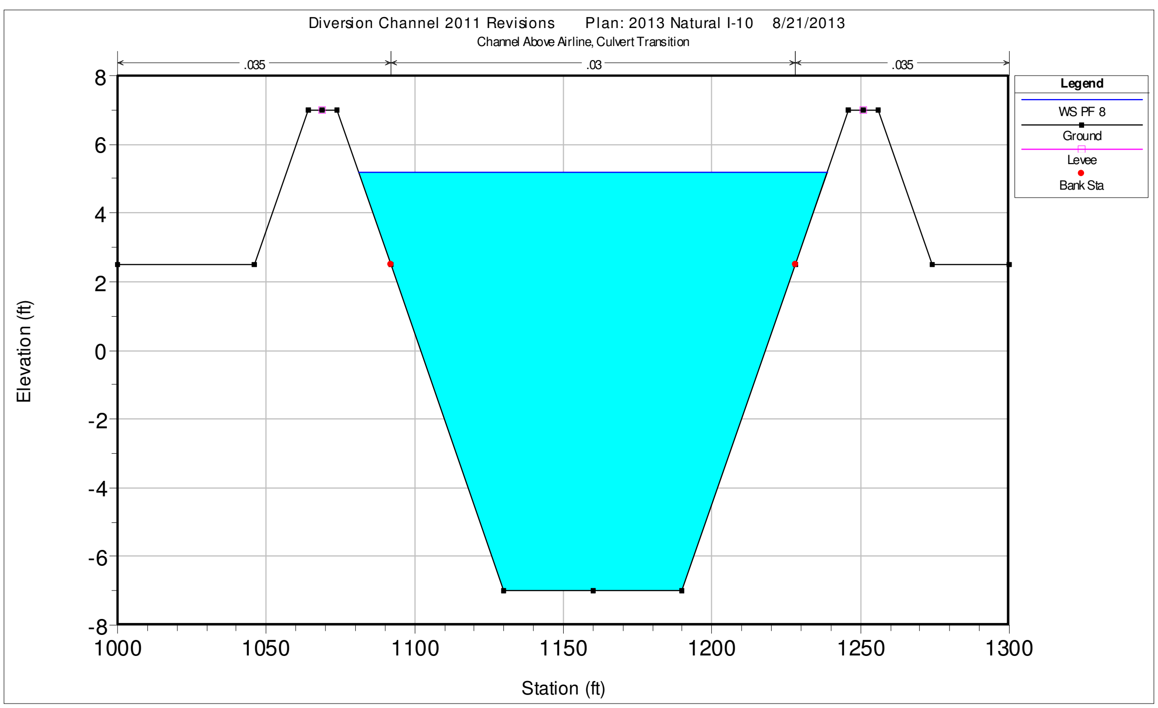


Figure 9-4: Typical conveyance channel south of US 61.

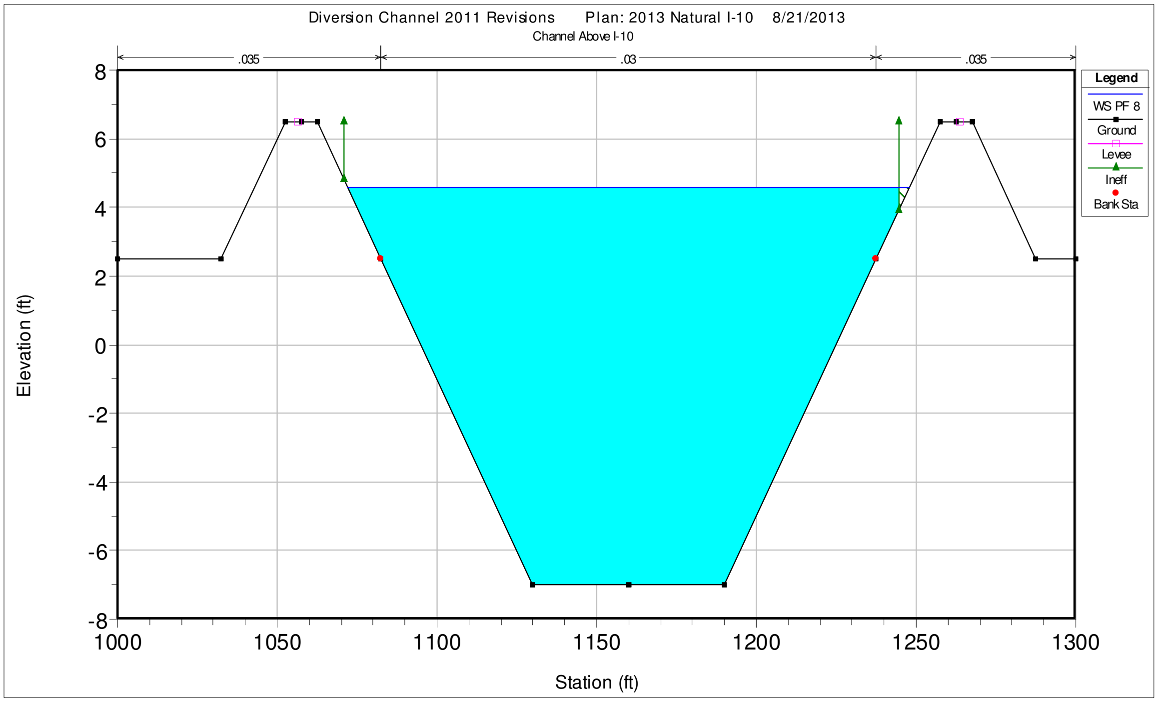


Figure 9-5: Typical conveyance channel north of US 61.

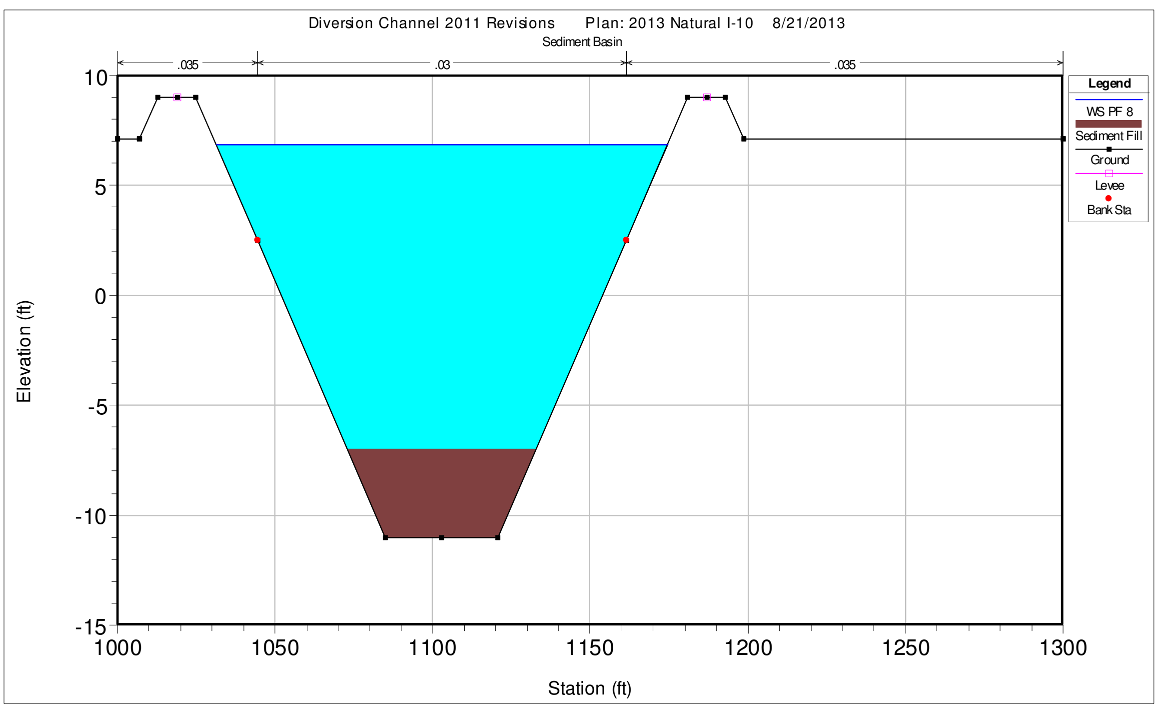


Figure 9-6: Typical sediment basin cross-section.

Sedimentation Basin Weirs

The sedimentation basin will utilize two weirs, one at each end; both will be set at an elevation of +2.5 ft NAVD88-LDNR. The purpose of the first weir is to establish more uniform flow conditions by reducing the turbulence created at the entrance structure. The weir presents a gradual rise in the bottom elevation of the conveyance channel that will equalize the differential velocities across the channel, creating a more laminar flow regime. This will reduce hydraulic short-circuiting through the sediment basin.

Sediment basins perform best when the flow pattern is uniform and the velocity is low (about 1 ft/s). These conditions allow particles to settle out of suspension and collect on the bottom of the basin. Any pockets of high velocity or turbulence will re-suspend the settled material back into the water column. This occurs when the water flows in a non-uniform pattern, such as that created by a short-circuit. In such a case, the bulk flow is concentrated through one area, while other areas have either stagnant or non-productive turbulent flow patterns.

The second weir serves an additional purpose. The sediment particles contained within the diverted water will not settle out uniformly. In fact, the sedimentation basin will trap progressively greater accumulations of sediment as the flow moves to the downstream end. The second weir will act as a barrier to retain trapped sediment within the storage area. In effect, it will add an extra 2.5-ft to the downstream side's storage capacity.

Sedimentation Build-Up in the Conveyance Channel

The amount of sediment traveling through the conveyance channel will vary over the course of a calendar year. Pulses of high flow will occur periodically depending upon rapid changes in the river stage. The Mississippi River stage at the Maurepas site can be estimated using well-documented seasonal information. During the late winter and early spring, with heavier rainfall and snow-melt contributing to the flow stream, the river stage is typically at its peak. During the late summer and early fall months, with drier upstream conditions and no snow remaining, the river is typically at its lowest stage.

Sediment is expected to build up in the inflow channel on the river side of the intake structure during low river stages. However, the high river stages and the resulting increase in flow through the diversion structure should cleanse the area and push the accumulated sediment towards the settling basin. New cross-section and profile surveys should be conducted periodically during the first few years of operation to inspect the balance between the scouring and depositional forces. The surveys should be taken under varying conditions, such as seasonal differences, water level variances, and flow changes, to assess the effects upon the system. During high flow events, the sediment build-up in the channel should be less, since the increase in flow and velocity will place more particles in suspension. When low flows are present, higher volumes of sediment may be observed.

Mechanical Design Analysis

The flow into the Maurepas diversion will be controlled by three sluice gates. The gates will be hydraulically actuated and controlled from a central control panel. The system operator will be able to manually adjust the positions of the gates by actuating them from the control room. This should not have to be done more often than once a day. The gates shall be designed to be operated individually or in unison.

Cast Iron Sluice Gates

The three gates will each be 10-ft high x 10-ft wide and will be designed to withstand an upstream high water elevation of 31-ft against downstream low water elevation of 0-ft NAVD88-LDNR. The centerline of the gates will be at -2 feet NAVD88-LDNR. This will result in a maximum seating head of 31-ft. The gates will be of the flat-back type, mounted on a wall thimble, which is integrally cast into the concrete structure. The hydraulic actuators will be mounted on structural steel members that are a part of the intake structure; therefore the gates will not require yokes as a part of the frame and guide rails. The gates will be of a flush-bottom design, eliminating any channel or sill at the base that might inhibit flow or catch debris. The gates will be sealed in the closed position by the action of a series of wedges at the perimeter of the gate frame. The downward force of the actuator and the weight of the gate will drive the gate slide into the wedges which will in turn force the slide into seat facings around the frame periphery.

The gates will be operating in Mississippi River water and will not be subject to high salinity, corrosive chemicals, or sewage. The gate, frame, and wall thimble will be constructed of cast iron. The stem block, guide bushings, wedges, and wedge blocks will be manganese bronze. The seating faces will be naval bronze. The flush bottom seal will be neoprene. The stem, stem couplings, and all fasteners will be type 304 stainless steel.

A sluice gate is usually constructed of cast iron and slides vertically to open. Sluice gates have relatively low overall maintenance with typical life-spans of 30 to 50 years. Periodic cleaning and lubrication of the stem and hoisting mechanism will be required. The gates will be designed to withstand seating head conditions in excess of 31-ft, as measured from the center of the gate to the stage of the Mississippi River.

The sluice gates will use a conventional gate system, as opposed to a "self-contained" gate system. Functionally, a conventional gate has the same general features and operations as a self-contained gate. The difference is that a self-contained gate absorbs the operating load created during opening and closing. Self-contained systems are normally used for much smaller gates and have not been deemed practical for this application. Additionally, conventional gates are less expensive than the self-contained units.

Hydraulic System

The gates will be raised and lowered by hydraulic cylinders operating on the gate stems through a 10-ft stroke. The cylinders will be driven by hydraulic fluid from an actuation package. The hydraulic actuation package will be powered by two pumps driven by an electric motor. The use of two pumps will insure that one is always available to close the gates in an emergency. Electric power for the pumps will come from the local grid, with a stand-by generator as back-up. In the case of a temporary electrical outage, the back-up generator will provide the operator with full control of the gates. The main reasons for selecting this configuration were its reliability, cost effectiveness, and the ability to place the gates in precise positions. Such accuracy in positioning the gates is essential for the success of this project. A stop log system will also be incorporated into the inlet structure as an additional back up closure option should extreme conditions dictate.

The hydraulic actuation system will only operate when the operator wishes to change the position of the gate(s). This is expected to occur no more often than daily, depending on changes in river conditions, etc. The operator will raise or lower the gates to achieve a desired flow rate, and will be guided by the flow-indicating instruments in the channel. Each gate-actuating cylinder will have a position-indicating sensor that will show the operator the position of the gate and enable him to observe the change in position as adjustments are made. The gates will be capable of moving at about 7-in/min in unison, or a single gate may be raised or lowered at a speed of 21-in/min.

When operating in unison, one of the gate cylinders will act as the master and the other two gates will follow as slaves. Pulse control of the solenoid operated valves controlling the flow of hydraulic fluid to the cylinders will allow a PLC. This will enable monitoring the position indication signals from the master and slave hydraulic cylinders, thus keeping the three gates in unison. All regulation of the gates will occur at the hydraulic control panel which will be located on the hydraulic actuation package. This package will be housed inside a small building on top of the intake structure. There will be no operating switches at the individual gates, and there will be no provision for remote operation of the gates.

To activate the system the operator will push an OPEN or CLOSE button. When the button is depressed, the hydraulic motor will start and the gate(s) will begin to move. The gate(s) will continue to move until the operator pushes the STOP button or the gate reaches the full open or full closed position. When the STOP button is pushed (or the gate is fully open or closed), the flow of hydraulic fluid to the gate mechanism will cease and the gate will stop. When all of the gates are stopped, the pump will automatically shut down. Hydraulic pressure retained in the system will hold the gates in their last position until the next time they are actuated. On the loss of power the gates will remain in their last position until power is restored.

The hydraulic actuation system will have two pumps operating at 2500 psi. Only one of the pumps will operate at a time. The pumps will be sized to operate a single gate at a speed of twenty inches per minute. When the gates are operating in unison, the operating speed will be about seven inches per minute. The two pumps will automatically alternate at each start; however, the operator will also have the option at the hydraulic control panel to choose which pump to run. Should the selected pump fail to provide the required flow or pressure, that pump will automatically stop and the other pump will begin to pump. Hydraulic fluid will be stored in a stainless steel reservoir integral to the hydraulic actuation system. The reservoir will have a visible sight glass and two level switches. The first switch will alarm on low level and the second (lower) switch will shut down and prevent the operation of the pumps. A hydraulic reservoir isolator will isolate the vapor space of the hydraulic reservoir from the atmosphere, preventing the entry of dust, dirt, and water. The hydraulic reservoir isolator will be carbon steel with a buna-nitrile bladder.

Construction

The sluice gates, hydraulic cylinders, and hydraulic actuation system will be provided by the same vendor. The hydraulic actuation package will be completely assembled on a painted, structural steel skid. All hydraulic components other than the hydraulic reservoir isolator will be mounted on the skid. The contractor will install the wall thimbles at the inlet to the intake structure culverts, install the gates on the thimbles, and erect the hydraulic cylinders. The contractor will install the hydraulic actuation package and reservoir isolator in the equipment room. The contractor will connect 480 volt, three-phase power to the hydraulic actuation package, and will pull instrument cables from the hydraulic cylinder position transducers to the hydraulic control panel. The contractor will install and support stainless steel, and socket-welded interconnecting hydraulic piping between the hydraulic package and the cylinders. The contractor will install valves and hydraulic hoses as necessary at the hydraulic cylinders and at the hydraulic package. The contractor will supply the hydraulic fluid, and will flush the interconnecting piping, and test the lines. The hydraulic system manufacturer will assist in the checkout and commissioning of the sluice gates and hydraulic operating system.

Electric Service

The electric service voltage required to supply energy to the Maurepas Intake Structure will be 480V, three-phase, 4-wire. This is a common service voltage for customers serving motor loads. The electric service provider will be Entergy Power Company. The electrical service will enter the premises via an overhead line and transition under ground and terminate to a service-entrance rated automatic transfer-switch (ATS) located in the control building.

Standby Power

A generator and hydraulic accumulators were evaluated as the two main sources of back-up power. There are several reasons why only a sole back-up generator was chosen over the hydraulic accumulators. The primary reason is cost-effectiveness. The hydraulic accumulators are far more expensive than a traditional back-up generator. The main benefit to using accumulators is that when they fail they can be set to fail in the closed position. However, for this to be of benefit both the primary power supply and the accumulators have to fail, which is a rare situation. If a hurricane or powerful storm were to occur in the area, the generator would be able to perform the operations, since the operator should have adequate time to close the gates prior to the power failing.

A generator and diesel-fueled engine set will be installed to provide standby power for critical loads in the event of failure of the utility company service. The generator set will be connected to the ATS. Upon the loss of utility service, automatic controls will start the generator and thereby provide power. Upon restoration of utility service, controls will automatically reverse the process. The generator will include a skid mounted, double wall fuel tank. The double wall fuel tank will provide a means for secondary spill containment. The fuel tank will be sized for a capacity that will provide a minimum of 48-hours of continuous full-load operation. It takes approximately 18 minutes to close the gates, so the back-up generator should be more than adequate. The generator will be located near the intake structure front gate.

Power Distribution

The electrical system will be sized to adequately supply electrical power to the hydraulic pumps for the diversion gates. Please see the attached load tabulation sheet depicting the anticipated load demand. A 480Y/277-volt lighting and branch circuit panel-board, with service will be provided for building and security lighting.

Lighting

Lighting in the control building will be open, turret-type fluorescent fixtures with energy saving electronic ballasts. The lighting level will be designed to 30-40 foot-candles. This lighting level gives adequate lighting for a building for this type of operation. A self-contained, emergency battery light will be located in the control building. The battery light will illuminate the control room for a means of egress.

The access walkway will have pole mounted light fixtures, which will illuminate the walkway for night time safety and security. Wall mounted light fixtures will also be located on the control building. All of the outdoor light fixtures will be controlled by a photo cell.

HVAC

Ventilation in the control building will include a package type HVAC system. The system will include provisions for heating and cooling. The system will be designed to maintain a maximum temperature of 80 ºF and a minimum temperature of 40 ºF.

Control and Data Collection

The sluice gate control panel will be mounted in the control room located above the intake structure. The control panel will include all of the required electrical components and devices for the operation of the hydraulic gates. The control panel will include front mounted push-button devices, pilot lights and gate position indicators for operator control. The operator will be able to raise and lower the sluice gates to control the water flow through the conveyance channel.

The instrumentation data signals will be gathered to a central location for indication and recording. This control panel will be located in the control building and tagged (LCP-

100). The operator will be capable of viewing and recording all of the following process operations at LCP-100:

- Intake structure water level
- Outflow structure water level
- Channel flow
- Analyze outflow water parameters.
- Sluice gate alarm parameters

The intake structure levels; LE/LT-4600A & LE/LT-4600B (inflow & outflow) will be measured by a hydrostatic type level transmitter. The instrument will be located in a stilling well pipe located on the inflow and outflow structure walls.

Channel flow; FE-4100, FE-4200 & FE-4300 (west wall, intermediate wall and east wall channel) will be measured by an open channel pulse-Doppler flow meter. The flow elements instruments will be located in the centers of the outflow channels.

The water quality analyzer transmitter will be comprised of a multi-channel instrument. The analyzing instrument will be able to monitor the following components in the water:

- Turbidity
- Nitrate ($NO3^-$)
- Ammonium ($NH4^+$)
- Chloride (Cl^-)
- DO (Dissolved Oxygen)
- pH

The instrument will be located at an accessible location on the outflow structure for periodic maintenance and calibration.

As described in the previous paragraphs, the Sluice Gates will be controlled from a vendor supplied control panel (FCP-SG). The control panel will be designed to have output alarm signals to be monitored by a system control/recorder (LCP-100). The alarm signals will be time and date stamped.

Pump Station

The following narrative presents the preliminary design and associated costs to construct a drainage pump station at the point where Bourgeois Canal discharges into Hope Canal. As shown on Plan Sheet PC-1.00, currently the Hope and Bourgeois Canals converge approximately 2,500-ft north of US 61 into a single channel (retaining the name Hope Canal) which conveys the combined flow. The guide levees of the proposed conveyance channel, which will generally follow the Hope Canal alignment north of the convergence point, will cut-off the existing drainage route of the these two canals. The purpose of the pump station is to maintain the existing drainage pattern in the area by transferring approximately 250 cfs from the two canals at their point of convergence into the conveyance channel. The 250 cfs is a conservative estimate of the volume of flow currently carried by the two canals based on the hydraulic modeling conducted.

Description

The proposed pump station will consist of three 125 cfs pumps that operate in cycles to provide a peak flow of 250 cfs when two pumps are operating in unison. The third pump will serve as a back-up in case of the failure of one of the active pumps. The proposed pumps are of the vertical line shaft type, which is ideal for this project since such pumps can move large volumes of flow against relatively low head conditions.

The intake pumps will receive waters from the existing Hope and Bourgeois Canals. An approach basin will be constructed upstream of the intake basin to impart a uniform velocity distribution to the inflowing water. The approach to the basin will be gradually sloped to the design elevation of the intake and guarded against erosion with rip-rap and geotextile fabric. Both canals approaching the intake structure will be dredged and improved to provide uninterrupted flow to the pump station. The dredging will be required to adjust the existing canal elevations to the design elevation of the pump station intake.

The pumps will discharge into three 48-in diameter pipes that run over the eastern-most levee of the conveyance channel and outfall onto an armored outlet structure. The outlet structure shall be covered with geotextile fabric and rip-rap, as illustrated on the design plans. The protective armor at the outlet is necessary to prevent localized erosion, due to the water discharge velocity.

Design Basis

The pump station is designed to pump 250 cfs from the existing Hope and Bourgeois Canals into the proposed conveyance channel. This flow rate was estimated by the SWMM model developed in Task 1 to represent the existing flow in the drainage system of the Garyville/Reserve area. The 250 cfs capacity will be achieved by the installation of three pumps, each with a capacity of 125 cfs. The pumps will rotate duty cycles with two pumps being utilized at a time, while the third will serve as a standby.

Hydraulic Design

The hydraulic design for the pump station was performed in accordance with standard engineering practice. The design criteria used were that set forth in the Engineering Manuals, Regulations, and Technical Letters for civil works construction published by the USACE Office, Chief of Engineers, as amended in the design guidelines developed by the New Orleans District. The pump station at Davis Pond was recommended by the LDNR to use as a design guide for the subject pump station, due to its similarity in application and hydraulic conditions. Both stations pump approximately 250 cfs at their peak flow, a relatively high volume of water, against a relatively low head.

Vertical line shaft axial pumps were selected because they are designed for applications that require high-volumes of flow and low head operating conditions. Line shaft pumps are defined as pumps that connect to the impeller by means of a long shaft. In this case, the pumps will be driven by natural gas motors connected to the impeller shaft by a direct coupling, the most energy efficient means of connection. The natural gas motors were selected since there is no adequate electrical power supply in the area that can be routed to the rather remote project site. Submersible pumps were ruled out for this application due to the difficulty in access for maintenance and the relatively short distance to the bottom of the intake sump.

URS investigated alternative intake designs for the proposed pump station. A formed suction intake (FSI) was selected because it requires much less submergence than a conventional rectangular intake. The shallower depth of the FSI design will significantly reduce pump station excavation, dewatering, and sub-structure costs.

Intake Diameter

To determine the intake dimensions for the FSI, the diameter of a normal bell shape intake typically used for a vertical line shaft pump was initially calculated. The following ANSI / HI (American National Standards Institute / Hydraulic Institute) equation was used, with a recommended velocity at the bell, v, of 5.5 ft/s and a flow rate, Q, of 56,000 gpm (250 cfs):

$$D = \left(0.409 \, \frac{Q}{v}\right)^{0.5} = 64.53 \text{ inches}$$

Since 64.53-in is not a practical dimension, for a typical pump inlet design, the suction bell would be upsized to a nominal 72-in diameter pipe size.

Then, the area required for the subject FSI design as defined by the ANSI / HI standard was equated to that of the bell:

$$(2.31\,d)\,(0.88\,d) = \frac{\pi \, D^2}{4}$$

Using D = 64.53-in from above and solving for d yields the required value of the outlet diameter for the FSI:

$$d = \sqrt{\frac{\pi D^2}{4 \cdot 2.31 \cdot 0.88}} = 44.75 \text{ inches}$$

Again, since 33.75 inches is not a practical dimension, d is rounded to a nominal pipe size, which yields a value of 48 inches.

Submergence

FSI designs were developed by the USACE's Hydraulics Laboratory (formerly the Waterways Experiment Station) in Vicksburg, MS. To minimize the required submergence width in comparison to a rectangular intake, the USACE experimented with a number of intakes and evaluated their performance based on the velocity distribution at the impeller. The geometry for the intake is presented in Figure 9-7 from their Engineering Manual EM 1110-2-3105, Type 10 FSI.

The USACE's research determined how much the height of the FSI could be reduced without adversely affecting its performance. The Engineering Manual 1110-2-3105 presents the results of the research.

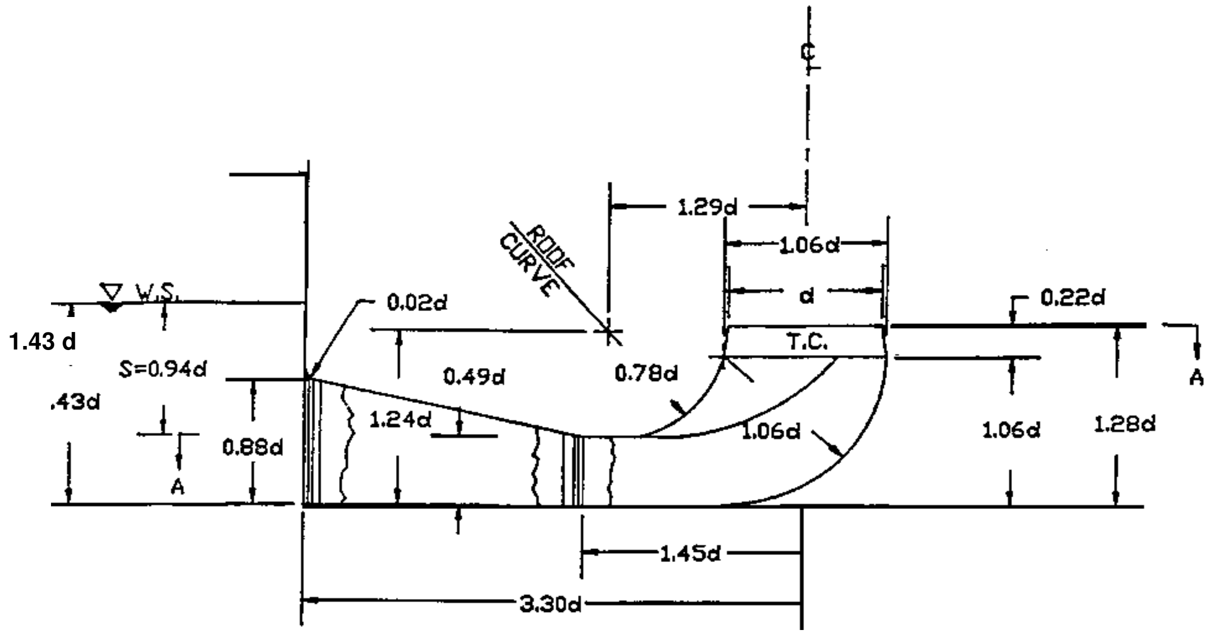


Figure 9-7: Formed Suction Intake (FSI) diagram - As per Type 10 design from Appendix I of EM 1110-2-3105.

Note the 1.43d on the left side of Figure 9-8 from the water surface (WS) to the bottom of the formed suction intake. Below is the USACE standard design equation for calculating Submergence used in the USACE design procedure.

$$\text{Submergence,}\quad S = 1.43\,\frac{d}{12}$$

Substituting d = 48-in → S = 5.72 feet

For confirmation, URS compared the USACE results with that from the application of the Hydraulic Institute standards. The Hydraulic Institute intake design procedure yielded a considerably deeper required depth of submergence.

$$\text{Submergence:}\quad S = \frac{\left(D + 0.574\left(\frac{Q}{D^{1.5}}\right)\right)}{12} \quad \rightarrow \quad S = 10.38\text{-ft}$$

However, this does not include the depth to the sill at low water, which must be added to the value calculated above to yield the total depth of submergence:

$$D_{submergence} = S + 0.44\,\frac{d}{12} \quad \rightarrow \quad D_{submergence} = 12.14\text{-ft}$$

Since the USACE standards had been used to design the Davis Pond pump station, and it is performing satisfactorily, a value between the conservative (Hydraulic Institute) and the more exacting (USACE) approach is appropriate for design.

A model test is commonly run on a scaled down replica prior to installation of a large pumping system. Model tests were performed for the Davis Pond Pump Station to establish the required intake sump and piping geometry. The model also certified the uniformity and symmetric nature of the approach flows as well as the performance of the pumps. Due to the success of the Davis Pond pumping station, the proposed design has been based on that configuration, with pumps in a vertical configuration and piping geometry of a similar nature. This is the best configuration for the installation due to its cost effectiveness and reliability. Additionally, since the head conditions calculated by the USACE approach were very close to the actual results, its' submergence calculations have credibility.

Hydraulic Stages

The hydraulic stages of the water surfaces on both the suction and discharge sides of the pump station are presented in Table 9-6. Water Stage information was derived from the Hydraulic Feasibility Study, conducted by URS as part of Task 1. That study evaluated the hydrologic impacts of the proposed freshwater diversion from the Mississippi River to the Maurepas Swamp, near Garyville, Louisiana. The data from that study, shown in Table 9-6, were used to calculate various parameters for the pump station design.

Table 9-6: Pump Station Water Levels

Water Stage	Intake Side	Discharge Side
High Water Conditions	EL. 3.0	EL. 5.0
Low Water Conditions	EL. 0.5	EL. 2.0

Net Positive Suction Head Available (NPSHa)

For a pump to operate properly it must have sufficient head above its' intake to prevent the liquid from boiling as it goes through the very low pressure region around the impeller. If the liquid vaporizes, then it forms a bubble, which subsequently collapses and causes the pump to cavitate, which extensively damages the pump.

The equations to calculate the $NPSH_a$ are as follows:

$$NPSH_a = h_{atm} + h_z - h_f - h_{vp}$$

$$h_{atm} = 33.9\text{-ft (atmospheric pressure)}$$

$$h_f = 0.5\text{-ft (friction losses)}$$

$$h_{vp} = 1\text{-ft (vapor pressure of water)}$$

$$h_z = S + 0.44\,\frac{d}{12} - \left(1.28\,\frac{d}{12} + 1\right)$$

Substituting S = 10.38-ft, d = 48-in $\rightarrow$ h_z = 6.02-ft (static water level above impeller eye)

$$\text{Thus,}\quad NPSH_a = 33.9\text{-ft} + 6.02\text{-ft} - 0.5\text{-ft} - 1\text{-ft} = 38.42\text{ ft}$$

With a known 'NPSH available', the 'NPSH required' must be computed. The required NPSH is a characteristic of the pump design. It is determined by the pump manufacturer during performance testing. The NPSHr is the fluid head required on the suction side of the pump to prevent the formation of water vapor. Required NPSH varies with pump design, pump size, impeller diameter, and operating conditions and is supplied by the pump manufacturer during performance testing.

<u>Discharge Design</u>

The diameter of the piping can be calculated by selecting a target velocity and then solving for it using the continuity equation with the design flow. The calculation process is as follows:

$$\text{Diameter} = \sqrt{4\,\frac{Q}{448 \cdot v \cdot \pi}}\ ;\quad \text{where Q is in gpm, and v is in ft/s}$$

Using a target velocity of 12 ft/s, and solving, yields a diameter of 3.64-ft. Round-up to 4-ft, to yield a practical pipe size of 48-in. Based on available standard pipe sizes, the discharge pipe will be ½ inch thick and 48 inches in diameter. Solving for the actual velocity: $v = \dfrac{4\ Q}{448\ \pi\ D^2}$ $\rightarrow$ v = 9.95 ft/s.

<u>Discharge Design</u>

The diameter of the piping can be calculated by selecting a target velocity and then solving for it using the continuity equation with the design flow. The calculation process is as follows:

$$\text{Diameter} = \sqrt{4\,\frac{Q}{448 \cdot v \cdot \pi}}\ ;\quad \text{where Q is in gpm, and v is in ft/s}$$

Using a target velocity of 12 ft/s, and solving, yields a diameter of 3.64-ft. Rounding-up to 4-ft, yields a practical pipe size of 48-in. Based on available standard pipe sizes, the discharge pipe will be ½ inch thick and 48 inches in diameter.

Solving for the actual velocity: $v = \dfrac{4\ Q}{448\ \ \pi\ \ D^2}$ $\rightarrow$ v = 9.95 ft/s.

Priming Static Head

The priming static head is the difference in elevation between the highest point of the discharge pipe and the lowest sump water level during the pump start-up.

Highest Pt Discharge Pipe = Levee Elev. + Pipe Diam. = 7.5 ft + 4 ft = 11.5 ft.

Priming Static Head = 11.5 ft – 0.5 ft = 11 ft.

With a siphon assist system, the siphon recovery must not be greater than 28-ft to prevent possible vapor lock and priming problems. An up-turned saxophone discharge pipe or a weir is typically used to limit the recovery to 28-ft and seal the end of the pipe. When one of these means is used, the low head is established as the saxophone or weir elevations. If, at the pumping mode, the lowest water level on the discharge side provides for a recovery less than 28-ft, then a saxophone discharge or weir is not required. The discharge end of the pipe should be submerged when a separate vacuum priming system is provided to prime the pump. (EM – 1110-2-3105)

Since the priming static head is 11-ft, it is well under the maximum limit of 28 ft, and thus a siphon assistance system is not required.

Total Dynamic Head (TDH)

The total design head is the sum of all the sources of head (resistance to flow) for the design. In this case there are minor losses, pipe losses, static head, etc. The equations listed below show the loss calculations expected for the proposed pump station.

Based on the Hazen-Williams equation the headloss through the pipe due to friction can be calculated as follows:

$$h_f = 10.44\ \frac{Q^{1.85}\ \ L}{C^{1.85}\ \ D^{4.87}}$$

where: Q = Flow (gpm)
L = Length (ft)
C = Hazen-Williams Coefficient (dimensionless)
D = Diameter (inches)

The headloss for 56,000 gpm through 250 linear feet of 48-in diameter ductile iron pipe with a C of 120 is:

$$h_f = 10.44\ \frac{56{,}000^{1.85}\ \ 250}{120^{1.85}\ \ 48^{4.87}} \rightarrow h_f = 1.47\text{-ft}$$

Minor losses for fittings can be calculated from the following equation:

$$h_{minor} = K \; D^{-c} \frac{v^2}{2\,g}$$

where, K = loss coefficient for a given type of fitting (--)
D = Diameter (inches)
v = Velocity (ft/s)
g = gravitational constant (32.17 ft/s^2)

For 2 – 90° bends: $h_{90} = 2\left(0.43\,\left(D^{-0.224}\right)\frac{v^2}{2\,g}\right) \rightarrow$ 0.56-ft

For 2 – 45° bends: $h_{45} = 2\left(0.22\,\left(D^{-0.298}\right)\frac{v^2}{2\,g}\right) \rightarrow$ 0.21-ft.

The velocity head, h_v is defined as: $h_v = \frac{v^2}{2\,g} \rightarrow$ 1.54-ft

The static head is defined as the difference in elevation from the intake water surface to the highest point on the discharge side.

The maximum static head is thus: $h_{s\text{-}max} = 5 - 0.5 = 4.5$ ft;

The minimum static head is thus: $h_{s\text{-}min} = 5$ ft – 2 ft = 3 ft

The total dynamic head is the sum of all the losses: TDH = $h_f + h_{90} + h_{45} + h_v + h_{static}$

TDH = 1.47 ft + 0.56 ft + 0.21 ft + 1.54 ft + 4.5 ft = 8.28 ft (Maximum)

TDH = 1.47 ft + 0.56 ft + 0.21 ft + 1.53 ft + 3.0 ft = 6.77 ft (Minimum)

TDH = 1.47 ft + 0.56 ft + 0.21 ft + 1.53 ft + 11.0 ft = 14.77 ft (Priming)

Table 9-7 and Figure 9-8 illustrate the relationship between head and flow rates. The figure represents the system curves for the proposed pump station. This information will be used in the next phase for pump selection.

Table 9-7: Data Input for System Curves

Q (gpm)	**Dia (in)**	**Vel (ft/s)**	**L (ft)**	**h_f (ft)**	**TDH_{MIN} (ft)**	**TDH_{MAX} (ft)**	**TDH_{Prime} (ft)**
20000	**48**	3.55	250	0.22	**5.5**	**7.0**	**13.5**
30000	**48**	5.32	250	0.46	**5.7**	**7.2**	**13.7**
40000	**48**	7.09	250	0.79	**6.1**	**7.6**	**14.1**
50000	**48**	8.86	250	1.19	**6.5**	**8.0**	**14.5**
60000	**48**	10.64	250	1.67	**6.9**	**8.4**	**14.9**
70000	**48**	12.41	250	2.22	**7.5**	**9.0**	**15.5**

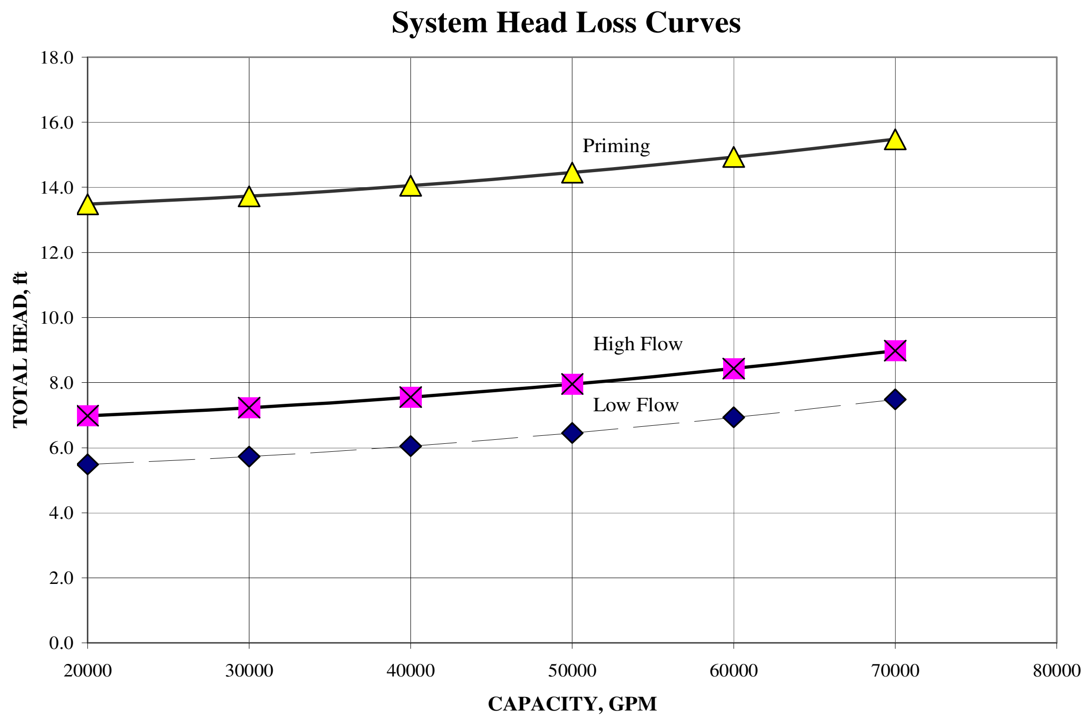


Figure 9.8 Pump Station System Curve

<u>Motor Sizing</u>

The power required to operate the pump at a given flow against a given head is defined by the following equation.

HP_{req} (Horse Power) = (TDH * Q_{gpm} * SG) / (Mechanical Efficiency x 3956)

Mechanical Efficiency = 80%

SG = Specific Gravity of fluid fro water = 1 (dimensionless)

HP = (14.77 ft * 56000 gpm * 1) / (0.80 * 3956) = 261.35 HP

The motor efficiency is approximated to be operating at 85% therefore:

HP / 0.85 = 307.47 HP

The motor should be sized at 307 HP

<u>Fuel System and Utility Considerations</u>

Three types of driver systems were considered for the pump station.

1. Electric Motors – This option would require that three phase power be run to the station. A high capacity generator would also have to be installed, to serve as a back-up should there be an interruption in the electrical power supply.
2. Diesel Engines – This option was considered as an option; however a large supply of diesel fuel would have to be stored at the pump station site. The fuel storage facility would require a separate structure adjacent to the pump station would significantly increase the project cost.

Diesel driven pumps would require additional mechanical support systems such as diesel day tanks and fuel transfer pumps with associated piping. There are also environmental concerns to consider when storing diesel fuel, especially in such an environmentally sensitive area.

3. Natural Gas Engines – A natural gas supply line exists in the general vicinity of the proposed pump station site. Under this option, the pipeline, located south of US 61, would be extended by Atmos Energy to the pump station site. Natural gas can also be used for the back-up generator to maintain other station operations should a power failure occur. This option is the most cost effective and will adequately provide the necessary power for the system to operate.

Option 3 – Natural gas engines were selected as the most viable option.

Water Service

Water service for the Maurepas Pump Station will be furnished by the St. John Parish Department of Public Works. The provision of water service to the pump station will require the extension of an existing water line located on the north side of the US 61, approximately 200 ft west of LA 54. The water supply shall provide for potable water consumption, wastewater disposal, and fire protection.

Telephone Service

Telephone service will be required at the pump station for operator communication. There are two companies with facilities in the area: AT&T and MCI; both have fiber optic lines along the railroad corridor south of US 61.

Electrical Service

Entergy electrical service will supply the power for the pump station. Power will have to be run from the nearest connection point or substation along US 61. It is anticipated that the line will be run overhead on poles from the connection point to the pump station. Routing for the power lines will be along the existing road that leads to the boat launch. The power requirements should be approximately equal to that of a small commercial building.

Pump Station Site Access

Access to the construction site will be available from US 61 via an unpaved road on the north side of US 61, at the intersection of LA 54 and US 61. The contractor may mobilize his equipment on the unpaved road but will have to consider improvements to the bridge over Bourgeois Canal before using it.

Required Relocations

The location of the proposed pump station is at an existing boat launch for the Hope Canal. This will require two major relocations: 1) the boat launch, and 2) the associated access road. Plan Sheet PC-6-06 of the pump station design plans illustrate how the revised site will be configured.

Soil Mixing

The levee section adjacent to the pump station would need be constructed in the existing wet of the Hope Canal. Implications of such construction would potentially be long term extensive settlements of the levee in that area. This condition would require not only continuous maintenance issues on levee but also potential significant down drag loading and potential differential settlement affects to the pumping station. In order to avoid such maintenance problems, the design includes incorporating soil mixing in the vicinity of the levee which will be located on existing Hope canal. Plan Sheets PC-3.00 show the location of the proposed levees and the extents of the proposed soil mixing.

10. COST ESTIMATE

Introduction

An opinion of probable costs was created for the 95% design submittal package for the "River Reintroduction into Maurepas Swamp" project. These costs were based on the project's design and assumptions made during the 95% design submittal.

The cost estimate is broken down into lump sum and unit price items. The following is a brief description of the major work items:

1) Mobilization and Demobilization: This work consists of the mobilization and demobilization of the contractor's forces and equipment necessary for performing the work required under the contract. The unit price for this item is estimated at 8% of the total cost of all work items.
2) Temporary Retaining Structures: This work consists of material and labor for the installation of sheetpile, walers, struts and combination wall for temporary retaining structures used for the construction at pump station, intake structure, headworks, and CN railroad.
3) Traffic Control and Coordination: This work consists of material and labor for the installation of signs and barricades for traffic control used for the construction at River Road and Airline Hwy.
4) Structural Concrete: This work consists of material and labor used for the construction at culverts, headworks, pump station and railroad crossings.
5) Metal Building system: This work consists of material and labor for the installation of pre-engineered metal building at the pump station.
6) Instrumentation: This work includes material and labor used for the installation of all items associated with the flow monitoring equipment at intake structure.
7) Vertical Pumps, Axial Flow: This work consists of material and labor for the installation of vertical pumps at the pump station.
8) Electrical work at pump station and head works: This work consists of material and labor for the installation of all items associated with electrical work at pump station and headworks.
9) Clearing and Grubbing: This work consists of clearing and grubbing for the construction of conveyance channel, pump station, headworks and intake structure.
10) Excavation: This work consists of material and labor for the excavation at intake structure, culverts, cofferdam, sedimentation basin, conveyance channel, I-10 crossing, pump station, bypass channel, access roads, and embankment cuts.
11) Dewatering: This work consists of material and labor for the installation of dewatering system including pumps, piping, engines, generators, etc. at various project locations.
12) Deep Mixed Columns: This work consists of material and labor for the installation of deep mixed columns.

13) Rip Rap: This work consists of material and labor for the installation of various sizes of rip rap at pump station, bypass channel, weirs, sedimentation basin, and intake structure.

14) Steel Sheet Piling: This work consists of material and labor for the installation of various types of steel sheet piling for the construction at cofferdam, headworks, and pump station.

15) Prestressed Concrete Piles: This work consists of material and labor for the installation of prestressed concrete piles.

16) Compression Pile Load Tests: This work consists of material and labor for the compression pile load tests to be performed at headworks, pump station, airline hwy. crossing and CN railroad crossing.

17) Steel H-Piles: This work consists of material and labor for the installation of various types of H-piles.

18) Steel Pipe Piles: This work consists of material and labor for the installation of steel pipe piles.

19) Asphaltic Pavement: This work consists of material and labor for the construction of asphaltic pavement at River road.

20) Crushed Stone Surfacing: This work consists of material and labor for the installation of crushed stone surfacing for access roads, ramps and embankment crown.

21) Turf Establishment and Maintenance: This work consists of material and labor for the turf establishment and maintenance work at various locations on the project.

22) Modifications to Existing Utilities: This work includes all items associated with relocating existing utilities. These utilities include gas, water, sewer, electrical transmission, and fiber optic lines.

23) Steel Discharge Pipe: This work consists of material and labor for the installation of steel discharge pipe at pump station.

24) New Water Main: This work consists of material and labor for the installation of new water main from Airline Hwy. to pump station.

25) Elastomeric Check Valves: This work consists of material and labor for the installation of elastomeric check valves for strom drain culverts under I-1O Hwy.

26) New Gas Main: This work consists of material and labor for the installation of gas main from Airline Hwy. to pump station.

27) New KCS & CN Railroad Tracks: This work includes all items associated with both railroad crossings (Kansas City Southern & Canadian National). This work includes but is not limited to existing track removal, excavation, embankment, precast concrete piles, cast-in-place reinforced concrete box culverts, precast concrete bridge, handrails, tie-in to existing tracks, installation and removal of false work, shoofly, temporary signs and barricades, etc.

28) Sluice Gates and Hydraulic Operated Valve Actuators: This work consists of material and labor for the installation of sluice gates, hydraulic power unit, piping, and valve actuators at the headworks.

29) Gate Valves: This work consists of material and labor for the installation gate valves for the discharge pipes located along conveyance channel.
30) Natural Gas Fueled Engine Pump Drive: This work consists of material and labor for the installation of natural gas fueled engine pump drive at the pump station.

Assumptions:

This cost estimate was developed based upon the 95% design plans and the following assumptions:

- Field engineering and inspection are not included
- Multiple contractors and tasks will be coordinated concurrently; several sub-contractors will be required to complete the work
- Costs were not included for land acquisition
- All item costs include labor, installation, and materials
- Concrete costs include reinforcing steel
- Overhead and profit are factored into the total cost

Definitions:

- Bond – The obligated amount of money forfeiture agreed upon if the project is not completed. The bond cost for this project is estimated at 2% of the total cost.
- Escalation – The anticipated amount of change in cost or price of specified materials, goods, or services for construction over a period of time, from the present till the expected date of construction. The escalation cost for this project is estimated at 3% of the total cost.
- Changes & Claims – This item refers to change orders, which are common during construction operations. Through this agreement the contractor and the client can make alterations to the original business contract. The changes and claims cost for this project is estimated at 5% of the total cost.

The estimated construction cost for this project is $133,960,000.

PO-29 RIVER REINTRODUCTION TO MAUREPAS SWAMPS

COASTAL PROTECTION AND RESTORATION AUTHORITY

Section 10 - 95% Cost Estimate
11/20/2013

Item No.	Item Description	Unit of Measure	Quantity	Unit Price	Total Price
1	**Mobilization and Demobilization`**	**LS**	**1**	**$8,566,558.272**	**$8,566,558.27**
2	**Temporary Retaining Structures**	**LS**	**1**	**$3,665,124.05**	**$3,665,124.05**
A)	**TRS at Pump station**				
	Sheet pile (PZ 27)	SF	9,778.50	$32.50	$317,801.25
	W14x30 Waler	LF	246.00	$20.47	$5,035.62
	W24x84 Waler	LF	246.00	$105.00	$25,830.00
	W10x45 Strut	LF	246.00	$50.29	$12,371.34
	W14x90 Strut	LF	246.00	$88.76	$21,833.73
	Temporary sheet pile (PZ19) at pump station by-pass channel (Sta. 138+00 to sta. 143+50), assume 20' high	SF	11,000.00	27.50	$302,500.00
B)	**TRS at Intake Structure & HW**				
a)	**TRS at Culverts C-4 &C-5 @ Intake structure**				
	Sheet pile (PZ 27)	SF	10,528.00	$32.50	$342,160.00
	W14x30 Waler	LF	227.60	$30.71	$6,988.46
	W24x84 Waler	LF	227.60	$105.00	$23,898.00
	W10x45 Strut	LF	1,503.00	$50.29	$75,585.87
	W14x90 Strut	LF	1,503.00	$88.76	$133,398.77
b)	**TRS at Culverts U-1 & U-2 @Intake structure**				
	Sheet pile (PZ 27)	SF	13,865.00	$32.50	$450,612.50
	W14x30 Waler	LF	288.70	$30.71	$8,864.53
	W24x84 Waler	LF	288.70	$105.00	$30,313.50
	W10x45 Strut	LF	2,606.30	$50.29	$131,070.83
	W14x90 Strut	LF	2,606.30	$88.76	$231,322.16
c)	**TRS at U-3 AT Intake structure**				
	sheetpile PZ-27	SF	1,710.00	$32.50	$55,575.00
C)	**CN Railroad**				
a)	TRS at C-3, C-4, & C-5				
	Combination wall (PPZ72/PZ27)	SF	45,845.00	$32.50	$1,489,962.50
				Sub-Total =	**$3,665,124.05**

PO-29 RIVER REINTRODUCTION TO MAUREPAS SWAMPS

COASTAL PROTECTION AND RESTORATION AUTHORITY

<u>Section 10 - 95% Cost Estimate</u>
<u>11/20/2013</u>

Item No.	Item Description	Unit of Measure	Quantity	Unit Price	Total Price
3	**Traffic Control and Coordination**	**LS**	**1**	**$472,431.14**	**$472,431.14**
	River Road Temporary Detour				
	Signs (Phase I & II)				
a)	End Road Work (G20-2, 48"X24")	EA	2	$33.00	$66.00
b)	Road Work Ahead (W20-1, 48"X48")	EA	2	$41.40	$82.80
c)	Trucks Entering Hwy (48"x48")	EA	4	$41.40	$165.60
	Signs (Phase III & IV)				
a)	End Road Work (G20-2, 48"X24")	EA	2	$33.00	$66.00
b)	Road Work Ahead (W20-1, 48"X48")	EA	5	$41.40	$207.00
c)	Trucks Entering Hwy (48"x48")	EA	4	$41.40	$165.60
d)	Speed Limit 35 MPH (W3-5, 48"X48")	EA	2	$41.40	$82.80
e)	Speed Limit 35 MPH (R2-1, 24"X30")	EA	2	$14.78	$29.56
f)	Detour Sign (W1-4R, 48"X48")	EA	2	$41.40	$82.80
g)	Speed Limit 35 MPH (R2-1, 24"X30")	EA	4	$14.78	$59.12
h)	Detour Sign (W1-4L, 48"X48")	EA	2	$41.40	$82.80
i)	Road Closed Barricade (R11-2, 48"X30")	EA	2	$36.00	$72.00
j)	Arrow Sign (W1-6R, 48"X18")	EA	2	$24.00	$48.00
	Signs (Phase V, VI& VIII)				
a)	End Road Work (G20-2, 48"X24") =	EA	5	$33.00	$165.00
b)	Road Work Ahead (W20-1, 48"X48")=	EA	2	$41.40	$82.80
c)	Trucks Entering Hwy (48"x48") =	EA	4	$41.40	$165.60
	Airline Hwy (US 61) (Phase I)				
a)	Temporary crash cushion	EA	2	$670.00	$1,340.00
b)	Temporary concrete median barrier	LF	1637	$67.00	$109,679.00
c)	Temporary Asphalt widening (Phase I)				
	2" asphalt concrete wearing course	TONS	227.00	$85.00	$19,295.26
	4" Superpave Asphaltic Concrete Base Course (Level 2)	TONS	454.01	$85.00	$38,590.52
	10" Class II Base Course (level 2)	TONS	597.38	$110.00	$65,711.42
d)	End Road Work (G20-2, 48"X24")	EA	4	$33.00	$132.00
e)	Road Work Ahead (W20-1, 48"X48")	EA	4	$41.40	$165.60
f)	Speed Limit 55 MPH (W3-5, 48"X48")	EA	4	$41.40	$165.60
g)	Speed Limit 55 MPH (R2-1, 36"X48")	EA	4	$14.78	$59.12
h)	Lane shift ahead (W20-1, 48"x48")	EA	4	$41.40	$165.60
i)	Lane shift arrow (W1-4bR, 48"x48")	EA	3	$41.40	$124.20
j)	Type III Road Closed Barricade (R11-2, 48"X30")	EA	4	$36.00	$144.00
i)	Lane shift Arrow Sign (W1-4bL, 48"X18")	EA	3	$41.40	$124.20
j)	4" Solid white striping	LF	3450	$0.65	$2,242.50
h)	4" solid yellow striping	LF	3450	$0.65	$2,242.50
	Airline Hwy (US 61) (Phase II)				
a)	Temporary crash cushion	EA	2	$670.00	$1,340.00
b)	Temporary concrete median barrier	LF	865	$67.00	$57,955.00
c)	Temporary Asphalt widening (Phase II)				
	2" asphalt concrete wearing course	TONS	143.84	$85.00	$12,226.15
	4" Superpave Asphaltic Concrete Base Course (Level 2)	TONS	287.67	$85.00	$24,452.30
	10" Class II Base Course (level 2)	TONS	719.19	$110.00	$79,110.37
d)	End Road Work (G20-2, 48"X24")	EA	4	$33.00	$132.00
e)	Road Work Ahead (W20-1, 48"X48")	EA	4	$41.40	$165.60
f)	Speed Limit 55 MPH (W3-5, 48"X48")	EA	4	$41.40	$165.60
g)	Speed Limit 55 MPH (R2-1, 36"X48")	EA	4	$14.78	$59.12
h)	Lane shift ahead (W20-1, 48"x48")	EA	4	$41.40	$165.60
i)	Lane shift arrow (W1-4bR, 48"x48")	EA	3	$41.40	$124.20
j)	Type III Road Closed Barricade (R11-2, 48"X30")	EA	3	$36.00	$108.00
i)	Lane shift Arrow Sign (W1-4bL, 48"X18")	EA	3	$41.40	$124.20
j)	4" Solid white striping	LF	3460	$0.65	$2,249.00
h)	4" solid yellow striping	LF	3460	$0.65	$2,249.00
i)	Miscellaneous Tarffic maintenance	LS	1	$50,000.00	$50,000.00
				Sub-total =	**$472,431.14**

PO-29 RIVER REINTRODUCTION TO MAUREPAS SWAMPS

COASTAL PROTECTION AND RESTORATION AUTHORITY

Section 10 - 95% Cost Estimate
11/20/2013

Item No.	Item Description	Unit of Measure	Quantity	Unit Price	Total Price
4	**Silt Fences**	**LF**	**83,838**	**$2.25**	**$188,635.50**
a)	Silt Fence at Pump Station	LF	5,400.00		
b)	Silt Fence at other locations (2 sides of ROW)	LF	68,438.00		
c)	Miscellaneous location (add 10, 000 LF)		10,000.00		
		Sub-Total =	**83,838**		
5	**Turbidity Curtain**	**LF**	**1,000**	**$75.00**	**$75,000.00**
a)	Assume qty. for Turbidity Curtain	LF	1,000		
6	**Truck Wash Down Racks**	**LS**	**1**	**$390,412.22**	**$390,412.22**
	(Use conversion factor for Asphalt = 2.025 TONS/CY)				
	(Use conversion factor for crushed stone = 1.90 TONS/CY)				
a)	3" Asphalt Pavement (Grade 1)	TONS	106.88	$138.00	$14,748.75
b)	Prime Coat (Type MC-30)	SY	633.33	$2.00	$1,266.67
c)	12" Crushed Stone Surfacing	TONS	612.22	$50.00	$30,611.11
d)	Geotextile Fabric	SY	966.67	$2.25	$2,175.00
	Estimate for 8 locations			**Sub-total =**	**$390,412.22**
7	**Demolition**	**LS**	**1**	**$406,136.11**	**$406,136.11**
a)	Remove Slope Paving at MS Levee (Phase I) =	SY	3,375	$25.00	$84,375.00
b)	Remove Slope Paving at MS Levee (Phase III) =	SY	1,500	$25.00	$37,500.00
c)	Remove existing Asphalt River Road (Phase III) =	SY	576	$25.00	$14,400.00
d)	Airline Hwy demolition	SY	9,794	$25.00	$244,861.11
e)	Miscellaneous Demolition	LS	1	$25,000.00	$25,000.00
				Sub-total =	**$406,136.11**
8	**Concrete for Minor Structures**	**LS**	**1**	**$50,000.00**	**$50,000.00**
a)	Concrete for Minor Structures (vaults, equipment pads, bollards, etc)	CY	200	$250.00	$50,000.00
9	**Structural Concrete**	**LS**	**1**	**$15,033,600.00**	**$15,033,600.00**
a)	Structural concrete for culverts, headworks, pump station, railroad crossings	CY	18,792	$800.00	$15,033,600.00
10	**Miscellaneous Metalwork**	**LS**	**1**	**$51,749.99**	**$51,749.99**
a)	Trash screens (4.375 Tons/Ea)	EA	3	$7,583.33	$22,749.99
b)	Grating at Headworks (8'-5"x11"-3")	EA	3	$3,000.00	$9,000.00
c)	Miscellaneous steel	LS	1	$20,000.00	$20,000.00
				Sub-total =	**$51,749.99**
11	**Metal Ladders**	**LS**	**1**	**$3,029.91**	**$3,029.91**
a)	Access ladder at pump station sump				
	(no. of ladders = 3, rung count per ladder = 17)	LF	51	$59.41	$3,029.91
				Sub-Total =	**$3,029.91**

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12	**Metal Railings**	**LS**	**1**	**$3,000.00**	**$3,000.00**
a)	Guardrail at HW equipment room	LF	60	$50.00	$3,000.00
b)	Guardrail at pump station	LF	121	$50.00	$6,050.00
				Sub-total =	**$9,050.00**
13	**Steel Doors and Frames**	**LS**	**1**	**$16,352.00**	**$16,352.00**
a)	**Doors, frames and windows at pump station**				
	3'x6'-8" insulated metal door w/ steel frame	EA	3	$834.00	$2,502.00
	2'-8"x 6'x8" wooden door w/ steel frame	EA	1	$500.00	$500.00
	6'x8' gravity exhaust louver w/ steel frame	EA	3	$1,000.00	$3,000.00
	6'x12' manual intake louver w/ steel frame	EA	3	$1,500.00	$4,500.00
	4'x4' double insulated fixed glass window w/metal frame	EA	1	$500.00	$500.00
	6'x4' double insulated fixed glass window w/metal frame	EA	1	$700.00	$700.00
	4'x4' double insulated fixed wire glass window w/metal frame	EA	1	$500.00	$500.00
b)	**Door at head works equipment room**				
	3'-4"x7' hollow metal door w/steel frame	EA	1	$950.00	$950.00
c)	**Installation & Miscellaneous hardware**	LS	1	$3,200.00	$3,200.00
				Sub-total =	**$16,352.00**
14	**Overhead Coiling Doors**	**LS**	**1**	**$6,370.43**	**$6,370.43**
a)	12'x16' Rollup Door w/ manual operator at Pump Station	EA	1	$4,370.43	$4,370.43
b)	Installation and miscellaneous hardware	LS	1	$2,000.00	$2,000.00
				Sub-Total =	**$6,370.43**
15	**Paints and Coatings**	**LS**	**1**	**$20,000.00**	**$20,000.00**
	For touch up painting				
16	**Painting: Coal Tar Epoxy System**	**LS**	**1**	**$704,528.32**	**$704,528.32**
a)	Coal Tar epoxy painting at pump station				
	Wing Walls:PAZ24/AZ19-700	SF	7,778.98	$28.00	$217,811.32
	PZ 27 at intake sill	SF	400.00	$28.00	$11,200.00
b)	Coal Tar epoxy painting at HW				
	HP-14	SF	12,780.00	$28.00	$357,840.00
	HP-16	SF	2,472.75	$28.00	$69,237.00
	PZ 27 (I wall)	SF	600.00	$28.00	$16,800.00
	PZ-22 Seepage Wall (Under Culvert C 1, Ref Sheet HS-2.02)	SF	1,130.00	$28.00	$31,640.00
		Sub-total =	**$25,161.73**	**Sub-total =**	**$704,528.32**

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17	**External Signage**	**LS**	**1**	**$5,000.00**	**$5,000.00**
a)	Project external signage	EA	4	$5,000.00	
18	**Metal Building System**	**LS**	**1**	**$69,000.00**	**$69,000.00**
a)	Metal Building System Material at pump station	LS	1	$39,000.00	$39,000.00
b)	Office area/rest room	LS	1	$10,839.00	
c)	Installation & Miscellaneous	LS	1	$30,000.00	$30,000.00
				Sub-Total =	**$69,000.00**
19	**Instrumentation**	**LS**	**1**	**$50,000.00**	**$50,000.00**
a)	**Instrumentation at Headworks**				
	Scour Indicators	EA	3	2,000	$6,000.00
	1" Scour indicator conduits	LF	360	5	$1,800.00
	Channel Master H-ADCP Acoustic Doppler Current Profiler	EA	2	18,200	$36,400.00
	Wireless signal w/ solar panel	EA	1	2,000	$2,000.00
	3/4" stainless steel pipe (3 pipes/unit, 3 units)	EA	9	200	$1,800.00
	Installation	LS	1	2,000	$2,000.00
				Sub-total =	**$50,000.00**
20	**Plumbing System at Pump Station**	**LS**	**1**	**$15,444.00**	**$15,444.00**
a)	Plumbing for pump station (52'x66' = 3,432 SF)	SF	3,432	$5	$15,444.00
21	**Vertical Pumps, Axial Flow**	**EA**	**3**	**$507,791.67**	**$1,523,375.00**
a)	Vertical Pumps, Axial Flow, Gear Reducers, Angle gear drive, SS propeeler and shaft, base plate, sole plate, spare parts, FSI	EA	3	$482,791.67	$1,448,375.00
b)	Installation	EA	3	$25,000.00	$75,000.00
				Unit price =	**$507,791.67**
22	**Natural Gas Fuel Piping System at Pump Station**	**LS**	**1**	**$12,012.00**	**$12,012.00**
	Natural gas fuel piping at pump station (52'x66' = 3,432 SF)	SF	3432	$3.50	$12,012.00

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23	**Electrical Work at Pump Station**	**LS**	**1**	**$260,800**	**$260,800.00**
a)	PP1 Panelboard 480/277V 225A 42 circuit MB 1-100A/3 , 1-50A/3, 2 - 40A/3, 3- 30A/3, 7-20A/3	EA	1	$9,600	$9,600.00
b)	PP2 Panelboard 480/277V 100A 24 circuit MLO 5-20A/3 , 3-20A/1	EA	1	$3,970	$3,970.00
c)	LP1 Panelboard 120/208V 100A 24 circuit 50A MB 17 - 20A/1, 1 - 20A/1 GFI	EA	1	$2,100	$2,100.00
d)	LIGHT FXTURE TYPE BA HOLOPHANE DESOTO M-60	EA	4	$700	$2,800.00
e)	LIGHT FXTURE TYPE FA HOLOPHANE HES SERIES	EA	4	$250	$1,000.00
f)	LIGHT FXTURE TYPE HA HOLOPHANE MODULE 600	EA	6	$600	$3,600.00
g)	LIGHT FXTURE TYPE HB HOLOPHANE WALLPACK IV W415AHP27SZ	EA	8	$400	$3,200.00
h)	65KW 75KVA 480/277V GENERATOR	EA	1	$30,000	$30,000.00
i)	225A 480V 3 PHASE AUTOMATIC TRANSFER SWITCH	EA	1	$5,500	$5,500.00
j)	15KVA 480V - 120/208V TRANSFORMER	EA	1	$2,500	$2,500.00
k)	GROUND RODS 10 FEET X 3/4" COPPER CLAD	EA	4	$75	$300.00
l)	4/0 BARE CABLE COPPER	EA	195	$3	$585.00
m)	EXOTHERMIC WELDS	EA	28	$60	$1,680.00
n)	PHOTOCELL AND LIGHTING CONTACTOR	EA	1	$65	$65.00
o)	BRIDGE CRANE DISCONNECT SWITCH 480V 3 PHASE 60A	EA	1	$500	$500.00
p)	GAS DETECTION SYSTEM	EA	1	$5,000	$5,000.00
q)	LEVEL TRANSMITTERS - ULTRASONIC	EA	6	$3,000	$18,000.00
r)	MISCELLANEOUS ELECTRICAL WORK (CONDIUT, WIRE, WIRING DEVICES, CONDUIT FITTINGS, TEMPERATURE SENSORS, PRESSURE SENSORS, SOLENOID VALVES, GAS REGULATORS, BATTERY CHARGER, ETC.)	LS	1	$40,000	$40,000.00
s)	Labor for Installation	LS	1	$130,400	$130,400.00
				Sub-Total =	**$260,800.00**

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Item No.	Item Description	Unit of Measure	Quantity	Unit Price	Total Price
24	**Electrical Work at Head Works**	**LS**	**1**	**$144,150**	**$144,150.00**
a)	PP1 Panelboard 480/277V 100A 24 circuit MLO 1 - 60A/3, 1- 30A/3, 1-20A/3, 4-20A/1	EA	1	$4,200	$4,200.00
b)	LP1 Panelboard 120/208V 100A 24 circuit 50A MB 17 - 20A/1, 1 - 20A/1 GFI	EA	1	$2,100	$2,100.00
c)	LIGHT FXTURE TYPE BA HOLOPHANE DESOTO M-60	EA	1	$700	$700.00
d)	LIGHT FXTURE TYPE FA HOLOPHANE HES SERIES	EA	3	$250	$750.00
e)	LIGHT FXTURE TYPE HA GE DECASHIELD SPMM17POA1AMC3DB	EA	3	$600	$1,800.00
f)	LIGHT FXTURE TYPE HB HOLOPHANE WALLPACK IV W415AHP27SZ	EA	2	$400	$800.00
g)	SQUARE STEEL	EA	3	$700	$2,100.00
h)	35KW 50KVA 480/277V GENERATOR	EA	1	$20,000	$20,000.00
i)	100A 480V 3 PHASE AUTOMATIC TRANSFER SWITCH	EA	1	$4,000	$4,000.00
j)	15KVA 480V - 120/208V TRANSFORMER	EA	1	$2,500	$2,500.00
k)	GATE HYDRAULIC CONTROL PANEL (FURNISHED WITH PUMPS)	EA	1	$0	$0.00
l)	GROUND RODS 10 FEET X 3/4" COPPER CLAD	EA	12	$75	$900.00
m)	4/0 BARE CABLE COPPER	EA	160	$3	$480.00
n)	EXOTHERMIC WELDS	EA	28	$60	$1,680.00
o)	PHOTOCELL AND LIGHTING CONTACTOR	EA	1	$65	$65.00
p)	MISCELLANEOUS ELECTRICAL WORK (CONDIUT, WIRE, WIRING DEVICES, CONDUIT FITTINGS, ETC.)	LS	1	$30,000.00	$30,000.00
q)	Labor for Installation	LS	1	$72,075	$72,075.00
				Sub-Total =	**$144,150.00**
25	**Separator Geotextile**	**SY**	**25,598**	**$2.25**	**$57,596.48**
a)	Geotextile fabric under riprap at intake structure	SY	7,342.22		
b)	Geotextile fabric at sediment basin	SY	14,571.11		
c)	Geotextile fabric (for release valves)	SY	77.38		
d)	Geotextile fabric (weirs)	SY	300.83		
e)	Geotextile (pump station access road)	SY	915.56		
F)	Geotextile at pump station intake and discharge side	SY	2,391.33		
		Sub-Total =	**25,598.44**		
26	**Geogrid Soil Reinforcement**	**SY**	**438**	**$4.00**	**$1,751.47**
a)	Geogrid at Access Road (West side -Sta. 16+49 to Sta. 18+12.26) and (East side - Sta. 15+99 to Sta. 18+12.26)	SY	437.87		
27	**Clearing and Grubbing**	**LS**	**1**	**$15,102,892.56**	**$15,102,892.56**
a)	Clearing and Grubbing (300' ROW, Sta. 10+00 to Sta. 302+19)	ACRE	201.23	$75,000.00	$15,092,458.68
b)	Clearing and Grubbing at pump station	ACRE	0.42	$25,000.00	$10,433.88
				Sub-Total =	**$15,102,892.56**

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Item No.	Item Description	Unit of Measure	Quantity	Unit Price	Total Price
28	**Excavation**	**CY**	**1,040,721**	**$15.00**	**$15,610,813.93**
a)	Intake Phase 1	CY	29,680		
b)	Intake Phase 2	CY	63,800		
c)	Excavation U1 & U2	CY	9,300		
d)	Degrade Cofferdam to EL 22	CY	20,000		
e)	Intake Excavation from River	CY	34,700		
f)	Sedimentation Basin	CY	42,670		
g)	Conveyance Channel	CY	813,905		
h)	I-10 crossing	CY	1,111		
i)	Pump station and Byapass channel, assume	CY	10,000		
j)	Excavation at pump station access road	CY	777		
k)	Embankment cuts	CY	4,777		
l)	Miscellaneous	CY	10,000		
		Sub-total =	1,040,721		
29	**Structural Excavation and Backfill**	**LS**	**1**		
	This item is included in excavation bid item - Not Used				
30	**Dewatering**	**LS**	**1**	**$250,000.00**	**$250,000.00**
31	**Embankment, Compacted Fill**	**CY**	**447,772**	**$30.00**	**$13,433,149.20**
a)	Cofferdam Pond Fill	CY	2,920		
b)	Remaining Pond Fill	CY	10,540		
c)	Mainline Levee Reconstruction	CY	102,000		
d)	Cofferdam	CY	32,220		
e)	Sedimentation Basin	CY	3,930		
f)	Conveyance Channel	CY	285,385		
g)	Fill for pump station access road	CY	777		
h)	Miscellaneous Fill	CY	10,000		
		Sub-total =	447,772		
32	**Deep Mixed Columns**	**LS**	**1**	**$804,000.00**	**$804,000.00**
a)	Pump Station soil mixing	CY	12,000	$67.00	$804,000.00

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Item No.	Item Description	Unit of Measure	Quantity	Unit Price	Total Price
33	**Riprap (Class 30 lb)**	**TONS**	**1,399**	**$95.00**	**$132,883.96**
	(Use conversion factor 1.5 TONS/CY)				
a)	Riprap (Class 30 lb) @Bayou secret and Bourgeois canal Weirs	TONS	370		
b)	Riprap for pump station bypass channel, 27"Thk.	TONS	1,010		
c)	Riprap at 24" steel dischrage pipe on conveyance channel side (Assume 5'x5'x20"thk.)	TONS	19		
		Sub-total =	**1,399**		
34	**Riprap (Class 55 lb)**	**TONS**	**10,917**	**$100.00**	**$1,091,666.67**
	(Use conversion factor 1.5 TONS/CY)				
a)	Riprap at sedimentation basin	TONS	10,917		
35	**Riprap (Class 130 lb)**	**TONS**	**347**	**$105.00**	**$36,458.33**
	(Use conversion factor 1.5 TONS/CY)				
a)	Rip-Rap on pump station discharge side	TONS	347.22		
36	**Riprap (Class 250 lb)**	**TONS**	**11,013**	**$125.00**	**$1,376,666.67**
	(Use conversion factor 1.5 TONS/CY)				
a)	Riprap at Intake structure	TONS	11,013		
37	**Grouted Riprap**	**CY**	**1,982**	**$230.00**	**$455,911.11**
a)	Grouted Riprap at sedimentaion basin weir	CY	1,982		
b)	Grout Mix	LS	1		
		Sub-total =	**1,982**		
38	**Steel Sheet Piling, Type PZ-22**	**SF**	**54,890**	**$60.00**	**$3,293,414.40**
a)	Seepage cutoff wall (Cofferdam)	SF	45,048		
b)	Sheet pile PZ-22 at HW	SF	9,842		
		Sub-total =	**54,890**		
39	**Steel Sheet Piling, Type PZ-27**	**SF**	**1,140**	**$75.00**	**$85,500.00**
	Sheet pile, Type PZ-27 at pump station	SF	1,140		
40	**Combined wall system PAZ 24/AZ 19-700**	**SF**	**6,270**	**$79.00**	**$495,330.00**
	Combined wall system PAZ 24/AZ 19-700 at pump station	SF	6,270		

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41	**Prestressed Concrete Piles, Type 14"x14"**	**LF**	**9,587**	**$65.00**	**$623,155.00**
42	**Prestressed Concrete Piles, Type 18"x18"**	**LF**	**59,345**	**$85.00**	**$5,044,325.00**
42	**Compression Pile Load Tests**	**EA**	**5**	**$75,000.00**	**$375,000.00**
	5 Compression Pile Load Tests will be required				
a)	2 at HW (HP 14X89 & HP 16X101)	EA	2		
b)	1 at pump station (14" PPC)	EA	1		
c)	1 AT Airline Hwy (18" PPC)	EA	1		
d)	1 at CN Crossing (18" PPC)	EA	1		
		Sub-total =	**5**		
43	**Steel H-Piles, HP 14 x 89**	**LF**	**28,553**	**$80.00**	**$2,284,240.00**
44	**Steel H-Piles, HP 16 x 101**	**LF**	**6,371**	**$100.00**	**$637,100.00**
45	**Steel Pipe Piles, 24" Diameter**	**LF**	**2,520**	**$80.00**	**$201,600.00**
46	**Timber Piles, 12" Diameter**	**LF**	**1000**	**$20.00**	**$20,000.00**
	Timber Piles, 12" Diameter at Boat Launch	LF	480		
47	**Asphaltic Pavement**	**TON**	**3,396**	**$78.01**	**$264,906.00**
	(use conversion factor for Asphalt = 2.025 TONS/CY)				
	River Road Bypass (Hwy 44)				
a)	2" superpave Asphaltic Concrete Wearing Course	TONS	307.80	$80.00	$24,624.00
b)	12" Superpave Asphaltic Concrete Base Course (Level 2)	TONS	1,846.80	$80.00	$147,744.00
c)	Mill and Overlay River Road, Hwy 44 (Pahse IV)	TONS	338	$80.00	$27,000.00
d)	2" Cold Planing & Superpave Asphaltic Concrete Wearing Course (Level 2F)	TONS	225	$50.00	$11,250.00
e)	2" superpave Asphaltic Concrete Leveling Course (Level 2)	TONS	225	$80.00	$18,000.00
	Reconstruct River Road (Hwy 44)				
a)	2" Superpave Asphaltic Concrete Wearing Course (Level 2F)	TONS	65	$80.00	$5,184.00
b)	12" Superpave Asphaltic Concrete Base Course (Level 2)	TONS	388.80	$80.00	$31,104.00
		Sub-total =	**3395.70**	**Unit price =**	**$78.01**

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48	**Crushed Stone Surfacing**	**TONS**	**12,610**	**$110.00**	**$1,387,118.86**
	(Use conversion factor 1.9 TONS/CY)				
a)	Temporary Access Ramp at Headworks (Phase I & II)	TONS	517.22		
b)	Permanent Access Ramp (HW, Phase I, East side)	TONS	487.31		
c)	Surfacing for Cofferdam Crown (HW)	TONS	384.40		
d)	Temporary Access ramp west side (Phase V, HW)	TONS	144.12		
e)	Temporary Access ramp east side (Phase V, HW)	TONS	153.13		
f)	Access Road west side (SedBasin, Sta. 16+49 to Sta. 18+12.26), add 3 CY for turns	TONS	98.74		
g)	Access Road east side (SedBasin, Sta. 15+99.44 to Sta. 18+12.26), add 3 CY for turns	TONS	127.80		
h)	Access Road at Pump Station	TONS	386.57		
i)	Crown Conveyance Channel, both sides (Sta. 26+78.99 to 299+00)	TONS	9,577.76		
j)	Levee crown at sedimentaion basin (18+12.26 to sta. 26+79.05	TONS	733.12		
		Sub-Total =	**12,610.17**		
49	**Pavement Markings**	**LS**	**1**	**$26,532.60**	**$26,532.60**
	River Road Bypass (Hwy 44)				
a)	Pavement Striping (2 sides)	LF	2,052	0.65	$1,333.80
b)	Pavement Striping and Reflectorized Markers	LF	1,026	5	$5,130.00
	River Road Bypass (Hwy 44) - Phase IV				$0.00
a)	Pavement Striping, 2 sides (Phase IV)	LF	1,500	0.65	$975.00
b)	Pavement Striping and Reflectorized Markers (Phase IV)	LF	750	5	$3,750.00
	Reconstruct River Road (Hwy 44)				$0.00
a)	Pavement Striping (2 sides)	LF	432	0.65	$280.80
b)	Pavement Striping and Reflectorized Markers	LF	216	5	$1,080.00
	Airline Hwy (US 61) (Phase I)				$0.00
a)	4" Solid white striping	LF	3450	$0.65	$2,242.50
b)	4" solid yellow striping	LF	3450	$0.65	$2,242.50
	Airline Hwy (US 61) (Phase II)				
a)	4" Solid white striping	LF	3460	$0.65	$2,249.00
b)	4" solid yellow striping	LF	3460	$0.65	$2,249.00
c)	Miscellaneous pavement markings	LS	1	$5,000.00	$5,000.00
				Sub-total =	**$26,532.60**
50	**Vehicular Precast Concrete Bridge and Platform**	**LS**	**1**	**$9,331.00**	**$9,331.00**
a)	Vehicular Precast Concrete Bridge and Platform at HW	CY	13.33	700	$9,331.00

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51	**Chain Link Fence and Gate**	**LF**	**1,598**	**$59.13**	**$94,487.48**
a)	Chainlink fence and gate @ HW				
	7' High chainlink fence	LF	128	$58.00	$7,424.00
	Double swing gate (18')	EA	1	$1,800.00	$1,800.00
b)	Fencing (visual buffer at residential areas adjacent to HW) Sta. 26+78.99 to Sta. 35+67.05	LF	888	$58.00	$51,507.48
c)	Chainlink fence at pump station	LF	582	$58.00	$33,756.00
				Unit price =	**$59.13**
52	**Turf Establishment and Maintenance**	**AC**	**107**	**$4,500.00**	**$479,385.43**
a)	Turf Establishment (Use Avg width 128' within ROW from MS river levee to end of conveyance chaneel, add 20 AC for additional areas around pump station, access road, etc)	AC	106.53	$4,500.00	$479,385.43

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53	**Modifications to Existing Utilities**	**LS**	**1**	**$6,300,840.00**	**$6,300,840.00**
a)	**Air Liquide**				
	1ea, 12" CS Pipe- Oxygen (36" Dp) between I-10 and US-61	LF	400	$ 720.00	$288,000.00
	1ea, 12" CS Pipe- Nitrogen (48-72" Dp) between I-10 and US-61	LF	400	$ 960.00	$384,000.00
	3' Depth oxygen pipeline, size unknown between Airline and US-61	LF	400	$ 960.00	$384,000.00
	4' depth nitrogen pipeline, size unknown between Airline and US-61	LF	400	$ 960.00	$384,000.00
b)	**Air Products**				
	1 ea, 12" CS Pipe - Hydrogen (36" Dp) between I-10 and US-61	LF	400	$ 720.00	$288,000.00
c)	**Gulf South**				
	1ea, 24" Natural Gas (HP)	LF	400	$ 900.00	$360,000.00
d)	**Marathon**				
	1ea, 20" pipe	LF	400	$ 840.00	$336,000.00
	1 ea, 30" crude line	LF	400	$ 1,080.00	$432,000.00
e)	**Shell (Bengal & Colonial)**				
	1ea, 24" CS Pipe - Nitrogen	LF	400	$ 720.00	$288,000.00
	1ea, 24" refined product	LF	400	$ 900.00	$360,000.00
	1ea, 24" pipe	LF	400	$ 900.00	$360,000.00
f)	**Enterprise (Acadian)**				
	2ea, 6" Liquid petroleum BUTANE & PROPANE(HP)	LF	800	$ 540.00	$432,000.00
	1ea, 8" Liquid petroleum (HP)	LF	400	$ 600.00	$240,000.00
	1ea, 4" Natural gas pipeline	LF	400	$ 900.00	$360,000.00
	1ea, 12" pipeline	LF	400	$ 600.00	$240,000.00
g)	**Chevron**				
	1ea, 20" natural gas	LF	400	$ 840.00	$336,000.00
	2ea, 6" HVL Propoane, propylene, butane	LF	800	$ 540.00	$432,000.00
h)	**Reserve Communication**			$ -	
	2ea, above ground lines ON Entergy poles	LS	1	$ 16,440.00	$16,440.00
i)	**Comcast**			$ -	
	Above ground feeder line along River Road	LS	1	$ 12,000.00	$12,000.00
j)	**St. John Parish**				
	1 ea, water line (size unkown) at Airline Hwy	LF	400	$ 240.00	$96,000.00
	1 ea,sewer line (size unkown) at Airline Hwy	LF	400	$ 240.00	$96,000.00
k)	**Entergy Transmission**				
	2 ea. lines, 5 ea. Poles at US 61	LS	1	$ 27,600.00	$27,600.00
l)	**Entergy Distribution**				
	2 ea lines, 5 ea. Poles ALONG River Rd and US 61	LS	1	$ 36,000.00	$36,000.00
m)	**ATMOS Energy**				
	Size unkown	LS	1	$ 24,000.00	$24,000.00
n)	**AT&T Fiberoptic**				
	1 line underground, size unkown	LS	1	$ 21,600.00	$21,600.00
o)	**Level 3 (Wittel) Fiberoptic**				
	1 line underground, size unkown at KCS RR	LS	1	$ 21,600.00	$21,600.00
p)	**MCI Fiberoptic**				
	1 line underground, size unkown at KCS RR	LS	1	$ 21,600.00	$21,600.00
q)	**Temporary relocation of gas line at River road (PhaseIII &IV)**	ls	1	$ 24,000.00	$24,000.00
				Sub-total =	**$6,300,840.00**

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54	**Steel Discharge Pipe, 48" Diameter**	**LF**	**404**	**$623.95**	**$252,075.00**
a)	48" Steel discharge pipe	LF	376.5	$550.00	$207,075.00
b)	48" Dresser style flexible couplings	EA	3	$6,000.00	$18,000.00
c)	48"x72" Steel eccentric diffusers	LF	27	$1,000.00	$27,000.00
				Unit price =	**$623.95**
55	**New Water Main**	**LF**	**2724**	**$44.54**	**$121,325.18**
a)	6" HDPE Pipe	LF	2,724	$25.00	$68,100.00
b)	Hot Tap and Sleeve (12" x 6")	EA	1	$6,000.00	$6,000.00
c)	Tapping Valve	EA	1	$2,500.00	$2,500.00
d)	Excavation	CY	807	$8.00	$6,456.89
e)	Select Backfill	CY	605	$15.00	$9,080.00
f)	Pipe Bedding (Compacted Crushed Lime Stone)	TON	246	$72.00	$17,688.29
g)	Fittings (90 Deg Bend)	EA	1	$500.00	$500.00
h)	Fire Hydrant	EA	1	$1,000.00	$1,000.00
i)	Water Meter	EA	1	$1,000.00	$1,000.00
j)	Gravel Surfacing	TONS	125	$72.00	$9,000.00
				unit price =	**$44.54**
56	**Storm Drain Pipes**	**LF**	**1,414**	**$64.12**	**$90,670.00**
a)	24" Storm RCP Drain Pipes (HW, Phase I), Temporary	LF	124	$85.00	$10,540.00
b)	24" Storm RCP Drain Pipes (HW, Phase I), at permanent access ramps	LF	256	$85.00	$21,760.00
c)	36" Storm RCP Drain Pipes (HW, Phase III), Temporary	LF	48	$95.00	$4,560.00
d)	15" RCP Drain Pipes (Phase I, Airline Hwy US 61)	LF	558	$55.00	$30,690.00
e)	15" RCP Drain Pipes (Phase I, Airline Hwy US 61)	LF	400	$55.00	$22,000.00
f)	12" CMP Drain Pipe, Pump station access road	LF	28	$40.00	$1,120.00
		Sub-total =	**1,414**	**Unit price =**	**$64.12**
57	**Elastomeric Check Valves, 42" Dia. w/44"x55" Elliptical Cuff**	**EA**	**16**	**$26,933.50**	**$430,936.00**
a)	Elastomeric Check Valves, 42" Dia. w/44"x55" Elliptical Cuff	EA	16	$24,933.50	$398,936.00
b)	Installation	LS	1	$32,000.00	$32,000.00
				Unit price =	**$26,933.50**
58	**Elastomeric Check Valves, 36" Diameter**	**EA**	**8**	**$13,234.00**	**$105,872.00**
a)	Elastomeric Check Valves, 42" Dia. w/44"x55" Elliptical Cuff	EA	8	$10,234.00	$81,872.00
b)	Installation	LS	1	$24,000.00	$24,000.00
				Unit price =	**$13,234.00**
59	**Speed Reducers for Storm Water Pumps**	**EA**	**3**		
	Included in New pumps bid item - Not Used				
60	**Fuel Service Piping at Pump Station**	**LS**	**1**		
	Covered under natural gas fuel piping system - Not Used				
61	**New Gas Main**	**LS**	**1**	**$55,000.00**	**$55,000.00**
62	**Railroad Falsework at KCS Railroad**	**LS**	**1**		
	Not Used				

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Item No.	Item Description	Unit of Measure	Quantity	Unit Price	Total Price
63	**Railroad Falsework at CN Railroad**	**LS**	**1**		
	Not Used				
64	**Ballast and Sub-Ballast at KCS Railroad**	**LS**	**1**		
	Included in KCS RR New Tracks bid item - Not Used				
65	**Ballast and Sub-Ballast at CN Railroad**	**LS**	**1**		
	Included in CN RR New Tracks bid item - Not Used				
66	**Railroad Flagman at KCS Railroad**	**LS**	**1**	**$187,200.00**	**$187,200.00**
a)	Railroad flagmen (assume 2 flagmen for 180 days)	HR	2,880	$65.00	$187,200.00
67	**Railroad Flagman at CN Railroad**	**LS**	**1**	**$187,200.00**	**$187,200.00**
a)	Railroad flagmen (assume 2 flagmen for 180 days)	HR	2,880	$65.00	$187,200.00
68	**Railroad Insurance for KCS Railroad**	**LS**	**1**	**$50,000.00**	**$50,000.00**
a)	Contractor's Public Liability and Property Damage Liability	$2 million per Occurrence $6 million aggregate			
b)	Contractor's Protective Public Liability and Property Damage Liability	$2 million per Occurrence $6 million aggregate			
c)	Railroad's Protective Public Liability and Property Damage Liability	$2 million per Occurrence $6 million aggregate			
69	**Railroad Insurance for CN Railroad**	**LS**	**1**	**$50,000.00**	**$50,000.00**
a)	Contractor's Public Liability and Property Damage Liability	$2 million per Occurrence $6 million aggregate			
b)	Contractor's Protective Public Liability and Property Damage Liability	$2 million per Occurrence $6 million aggregate			
c)	Railroad's Protective Public Liability and Property Damage Liability	$2 million per Occurrence $6 million aggregate			
70	**Demolition of Existing KCS Railroad Tracks**	**LS**	**1**	**$35,461.60**	**$35,461.60**
a)	Track work Removal (Begin Station 6+50.00 to End Station 24+23.08)	TFT	1773.08	$20.00	$35,461.60

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Item No.	Item Description	Unit of Measure	Quantity	Unit Price	Total Price
71	**Demolition of Existing CN Railroad Tracks**	**LS**	**1**	**$53,600.00**	**$53,600.00**
a)	Demolition of Track	TFT	2680	$20.00	$53,600.00
72	**New KCS Railroad Tracks**	**TFT**	**1,773**	**$800.35**	**$1,419,025.05**
a)	Bents				
	24" Diameter Stee Pipe Pile (No. of Bents =6; No. of Piles per Bent = 3; Pile length = 140')	LF	2,520.00	$140.00	$352,800.00
	Precast Pile Cap (count = 4, length = 15, height = 2.67, width = 3)	CY	17.80	$400.00	$7,120.00
b)	Precast Concrete Bridge				
	Concrete Volume (L= 100.17; H=1.67'; W=13.5')	CY	83.64	$800.00	$66,913.56
c)	Handrails				
	Length =(100.17+3)*2 =206.3'; No. of posts = 52	LF	412.60	$50.00	$20,630.00
d)	Track work				
	(Begin Station 6+50.00 to End Station 24+23.08)	TFT	1,773.08	$200.00	$354,616.00
e)	Excavation	CY	14,924.76	$15.00	$223,871.39
f)	Backfill	CY	12,451.80	$30.00	$373,554.10
g)	Masonry Closure Wall	SF	768.00	$15.00	$11,520.00
h)	Tie to existing tracks	EA	4.00	$2,000.00	$8,000.00
				Unit price =	**$800.35**
73	**New CN Railroad Tracks**	**TFT**	**1,375**	**$2,659.98**	**$3,657,477.53**
a)	Prestressed, Pre-cast Concrete Piles	LF	15,275.00	$83.28	$1,272,102.00
b)	Concrete for Culvert (includes reinforcing steel, formwork, and expansion joints)	CY	910.39	$800.00	$728,314.81
c)	Concrete for Culvert Slab (includes reinforcing steel, formwork, and expansion joints)	CY	753.83	$600.00	$452,296.30
d)	Concrete Stabilization Slab (includes reinforcing steel, formwork, and expansion joints)	CY	125.64	$500.00	$62,818.93
e)	Excavation	CY	14,924.76	$15.00	$223,871.39
f)	Backfill	CY	12,451.80	$30.00	$373,554.10
g)	Masonry Closure Wall	SF	768.00	$15.00	$11,520.00
h)	Track Work (Rails, Ties and Ballast)	TFT	1,375.00	$200.00	$275,000.00
i)	Removal of falsework	LS	1.00	$250,000.00	$250,000.00
j)	Tie to existing tracks	EA	4.00	$2,000.00	$8,000.00
				Unit price =	**$2,659.98**
74	**Shoofly at CN Railroad**	**LS**	**1**	**$3,251,775.00**	**$3,251,775.00**
a)	Rails, Ties, and Ballast	TF	5,630	$250.00	$1,407,500.00
b)	Turnout	EA	5	$200,000.00	$1,000,000.00
c)	Signalization	EA	1	$250,000.00	$250,000.00
d)	Infrastructure/Controls	EA	1	$275,000.00	$275,000.00
e)	Track Fill Material	LCY	21,285	$15.00	$319,275.00
				Sub-total =	**$3,251,775.00**

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Item No.	Item Description	Unit of Measure	Quantity	Unit Price	Total Price
75	**Sluice Gates and Hydraulic Operated Valve Actuators**	**EA**	**3**	**$672,368.33**	**$2,017,105.00**
a)	Sluice Gates	EA	3	$159,035.00	$477,105.00
b)	Hydraulic power unit	LS	1	$1,460,000.00	$1,460,000.00
c)	Hydraulic piping	LS	1	$30,000.00	$30,000.00
d)	Installation	LS	1	$50,000.00	$50,000.00
				Unit price=	**$672,368.33**
75 a	**Sluice Gates**	**EA**	**2**	**$164,035.00**	**$328,070.00**
a)	Sluice gates (10'x10')	EA	2	$159,035.00	$318,070.00
b)	Installation	LS	1	$10,000.00	$10,000.00
				Unit price =	**$164,035.00**
76	**Gate Valves**	**EA**	**8**	**$59,973.53**	**$479,788.24**
a)	24" Gate Valves	EA	8	$21,000.00	$168,000.00
b)	Valve Box	CY	23.80	$350.00	$8,330.30
c)	Crushed stone, compacted (for valve box)	TONS	31	$110.00	$3,414.77
d)	Excavation for valvebox	CY	228	$15.00	$3,423.17
e)	4'x4' Grating for valve box (Type W-19-4 (1x3/16) Galvanized)	EA	8	$800.00	$6,400.00
f)	1' thk. Headwall for pipe on conveyance channel side	EA	8	$1,500.00	$12,000.00
g)	Concrete slab (3'x3'x4")	CY	1	$250.00	$220.00
h)	24" steel discharge pipe	LF	560	$75.00	$42,000.00
i)	Utility penetration through sheet pile	EA	8	$2,500.00	$20,000.00
l)	Sheetpile (PZ-22)	SF	3,200	$65.00	$208,000.00
m)	Installation of valve and valve box	LS	1	$8,000.00	$8,000.00
				Unit price =	**$59,973.53**
77	**Stone Bedding**	**CY**	**904**	**$14.00**	**$12,657.56**
a)	6" Bedding material at pump station	CY	399	$14	$5,579.78
b)	12" bedding material at HW	CY	506	$14	$7,077.78
		Subtotal =	**904**		
78	**MSE Wall/Reinforced Soil Slope**	**CY**	**1,413**	**$32.81**	**$46,355.56**
a)	Fill for MSE Retaining Wall	CY	1,413	$30.00	$42,382.22
b)	Geotextile fabric	SY	1,324	$3.00	$3,973.33
				Unit price =	**$32.81**

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79	**Concrete Slope Pavement**	**SQ**	**544**	**$232.00**	**$126,282.24**
a)	Concrete Slope Pavement (Phase VI at HW)	SQ	544.32		
80	**Diesel-Generator Set Standby**	**EA**	**1**	**$32,000.00**	**$32,000.00**
a)	Diesel Generator set standby	EA	1		
81	**Overhead Electric Bridge Crane**	**EA**	**1**	**$34,000.00**	**$34,000.00**
a)	15-Ton Overhead Electric Bridge Crane	EA	1	$24,000.00	$24,000.00
b)	Installation & Miscellaneous	LS	1	$10,000.00	$10,000.00
				Sub-Total =	**$34,000.00**
82	**Natural Gas Fueled Engine Pump Drive**	**EA**	**3**	**$292,693.33**	**$878,080.00**
a)	Natural Gas Fueled Engine Pump Drive	EA	3	$229,360.00	$688,080.00
b)	Gearbox and Drive shaft	EA	3	$30,000.00	$90,000.00
c)	Installation & Miscellaneous costs	LS	1	$100,000.00	$100,000.00
				Sub-Total =	**$878,080.00**
83	**Flow Measuring Equipment System**	**LS**	**1**		
	Covered under instrumentation bid item - Not Used				
84	**Sewerage Aeration Treatment System**	**LS**	**1**	**$10,000.00**	**$10,000.00**
a)	Sewerage Aeration Treatment System	1	EA	$10,000.00	
85	**Boat Launch and Dock**	**LS**	**1**	**$27,815.63**	**$27,815.63**
a)	Concrete Ramp Planks	CY	8.2	$250.00	$2,055.56
b)	Crushed Stone Surfacing	TONS	56.3	$72.00	$4,053.33
c)	Floating Dock	EA	1	$10,000.00	$10,000.00
d)	Rip-Rap	TONS	103.18	$65.00	$6,706.74
e)	Marine grade wood retaining wall	LS	1.00	$5,000.00	$5,000.00
				Sub-Total =	**$27,815.63**

Total =		**$115,650,000.00**
Bond (2%) =	$	2,313,000.00
Contingency (5%) =	$	5,898,150.00
Escalation (3%) =	$	3,715,834.50
Changes & Claims (5%)=	$	6,378,849.23
Grand Total =		**$133,960,000.00**

11.0 REFERENCES

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